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PULCHOWK CAMPUS

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Appropriate Asphalt Mix Design for Nepal

by

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(071/MST/251)

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ABSTRACT

Pavement design can be done by analytical and empirical methods. Empirical methods are based on recipe approach whereas analytical method is used to obtain site specific design after proper analysis. Empirical methods are useful where prior experience has been gained otherwise analytical method becomes important. In analytical approach it is necessary to study the failure modes and to design the pavement to resist the cracking and rutting modes of failure up to the design life which requires design of proper mix which in turn requires proper selection of materials and analysis of structure so that critical levels of strain do not exceed in the design life.

In Nepal, mix is designed in general by Marshall Mix Design Method in confirmation to Standard Specification for Road and Bridge Works published by Department of Road, for general requirement and far from being site specific.

Asphalt Concrete pavement is going to be constructed in large volume in recent future as per the increasing trend of traffic volume.

This research is intended to provide recommendations on average compositions of typical mixes applicable to Nepalese condition. The research will be beneficial to the departments, organizations, practicing Engineers and designers as they will be able to select appropriate mix design within a range for site specific conditions.

Key words

Pavement design, Failure modes, Asphalt Mix Design, Average Compositions, Nepalese Condition.

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Aashutosh Karna
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ABBREVIATION

AC…………….Asphalt Concrete

BOQ…………..Bill of Quantities

DOR…………..Department of Roads

S_{me}…………….Elastic Stiffness of Bitumen

Ei……………...Equation i

HMA………….Hot mix Asphalt

PI……………...Penetration Index

SP……………..Softening Point

SSRB…………Standard Specification for Road and Bridge Works

S_b……………..Stiffness of Bitumen

S_{mv}...................Viscous Stiffness of Bitumen

VMA…………Voids in Mixed/Mineral Aggregate

V_V…………….Volume of air Voids as %

V_B…………….Volume of Binder as %

CHAPTER 1: BACKGROUND

1.1. General

Asphaltic concrete (AC) is a dense, continuously graded mix which relies for its strength on both the interlock between aggregate particles and, to a lesser extent, on the properties of the bitumen and filler. The mix is designed to have low air voids and low permeability to provide good durability and good fatigue behavior but this makes the material particularly sensitive to errors in proportioning, and mix tolerances are therefore very narrow (Jackson and Brien (1962), Asphalt Institute (1983), (1989) and (1991)) (1).

An asphalt concrete surface will generally be constructed for high-volume primary highways having an average annual daily traffic load greater than 1200 vehicles per day (2). Highways, Ring Road, Runways and Parking spaces in Nepal need Asphalt Concrete for better performance in terms of economy, safety, comfort and durability.

Design Methods

1. Hubbard Field Method
2. Hveem Method
3. Marshall Mix Design Method
4. Superpave Method of Mix Design

1.2. Problem Statement

In Nepal, Marshall Mix Design Method is generally used for mix design of Asphalt Concrete.

SSRBW specifies the materials, method of construction and requirements for the construction of Asphalt Concrete and also specifies its meaning as a thoroughly controlled, hot-mixed, hot-laid, plant mixture of well graded dried aggregate and penetration grade bitumen, which when compacted, forms a dense material.

SSRBW specifies requirements in general and also instructs the Engineer to follow the special specification or BOQ if required. It is necessary to design the mix for specific site conditions in different sections even in a single project. However, in practice, mix design conforming to general requirements is only used.

1.3. Objectives of the study

The purpose of this research is to provide recommendations on average compositions of typical mixes applicable to Nepalese condition. The departments, organizations, practicing Engineers and designers will be benefited as they will be able to select appropriate mix design within a range for specific site conditions.

1.4. Scope and Limitations of Study

Department of Roads has constructed Asphalt Concrete in low volumes in comparison to other bituminous pavements. AC for Department of Roads is a recent technology along with absence of data about performance of constructed AC pavements due to lack of monitoring. The traffic however is ever increasing and it is time that Department of Roads considers AC pavements more often which also has been the case recently. But construction of AC pavements needs proper design of pavement thickness and proper mix design. This research focuses on providing recommendations on average compositions of mixes through proper analysis of available data on design work and bitumen and aggregate tests.

As discussed earlier, AC pavements have not been constructed in large volumes and similarly not much design have been done which makes available data on design parameters scarce. So, data on design parameters has been taken from Central Road Laboratory, Lalitpur and some design documents available at Division Road Offices. Also, test data on bitumen has been taken from Bitumen Barrel Udhyog, Amlekhgunj, Bara. The analysis has been done using this minimum available data. The time frame of a semester did not allow the extensive lab tests that could have been done to obtain more specific results. Moreover, the data on actual loading conditions were also not available so categorization of loading time has been done as a representative of loading conditions. The outcome of the thesis may require revision in the light of future development in the field in terms of availability of data and development of knowledge base.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction to Pavement Design (3)

All pavements have essentially three components:

1. The pavement foundation which consists of underlying subgrade soil, capping material (if used) and sub-base. The foundation must be able to carry construction traffic and withstand the loading applied by pavers and rollers used to place the layers above. Good drainage is an essential requirement for foundations.
2. The road base is the main structural element of the pavement. It is required to spread the wheel load so that underlying materials are not overstressed. The road base in flexible construction is usually of dense bituminous material but, in semi rigid construction, lean concrete may be used. For less heavily trafficked situations an unbound granular base may be adequate.
3. The surfacing is principally to provide adequate skid resistance and has little structural significance. It also provides abrasive resistance, smooth riding surface, resists some pressure exerted by the wheel and resist surface water infiltration. It may vary between a surface dressing of “spray and chip” to a 40 or 50 mm dense bituminous mixture which will make some contribution to structural integrity.

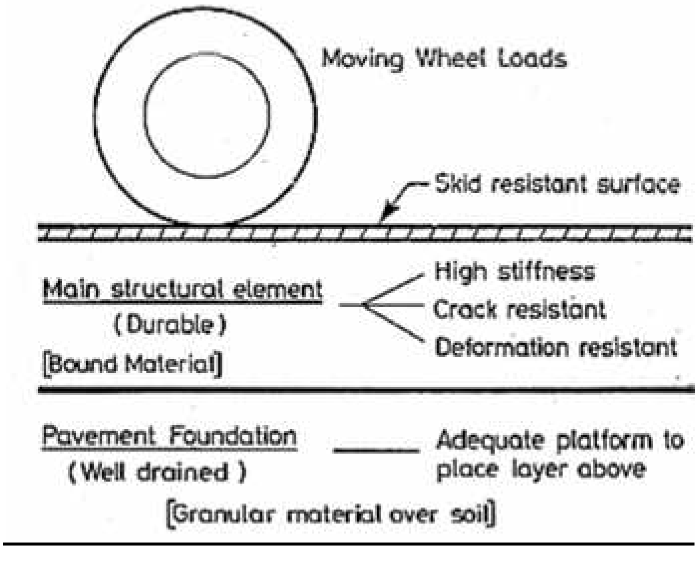


Figure1 The 'Ideal' Pavement

2.2 Pavement Failure (3) (4)

In any engineering design problem, it is essential to understand how the structure would fail so that the design process ensures that this does not occur. Pavements do not fail suddenly but gradually deteriorate in serviceability to a terminal level which may be defined as failure. For example in the U.K., failure is taken to be a 20 mm rut in the nearside wheel track or extensive cracking. The rut causes ponding of water, which is an obvious hazard, while cracking is clearly a failure mechanism which leads to break-up of the surface, formation of pot holes and ingress of water, which accelerates the failure process.

Two essential failure mechanisms in asphalt pavements (flexible pavements with bituminous bases) are rutting and cracking. In order to design for their prevention, the mechanics of pavement response to load need to be considered as shown in Figure 2.

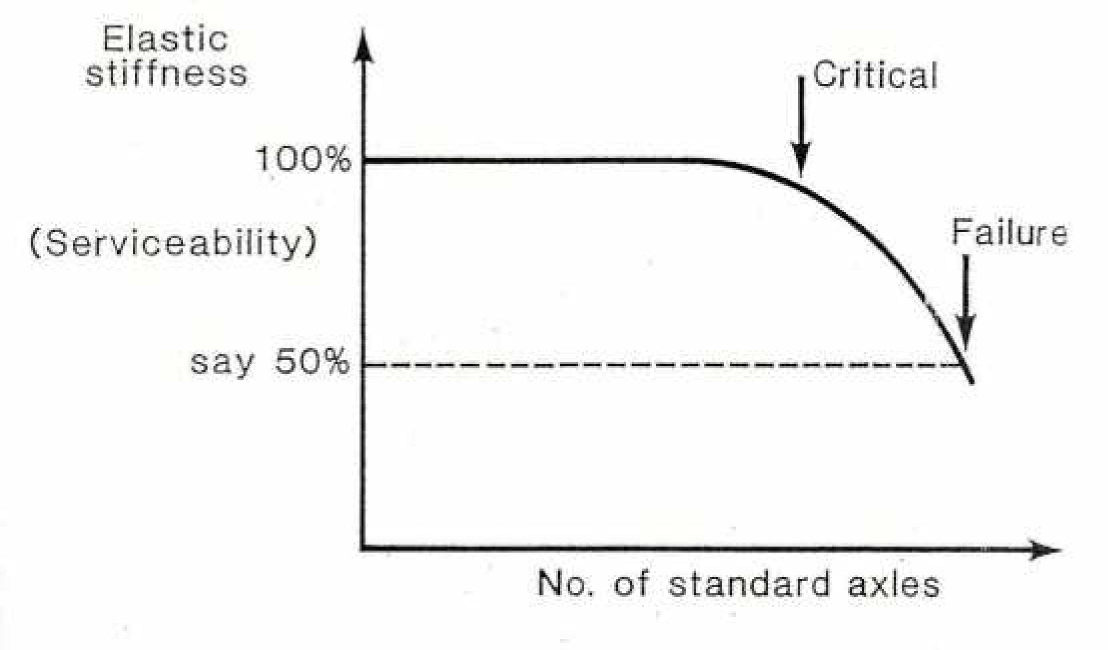


Figure2 Pavement Serviceability

Pavement failure is essentially a fatigue phenomenon, in the sense that the deterioration, which is caused by the stresses and strains in the structure, results from both the magnitude and the number of load applications the pavement experiences and which induce these stresses and strains as shown in Figure 3.

Cracking of the asphalt layer arises from repeated tensile strain, the maximum value of which occurs at the bottom of the layer. The crack, once initiated, propagates upwards causing gradual weakening of the structure as shown in Figure 4 (a). The development of a rut arises from the accumulation of permanent strain throughout the structure as shown in Figure4 (b).

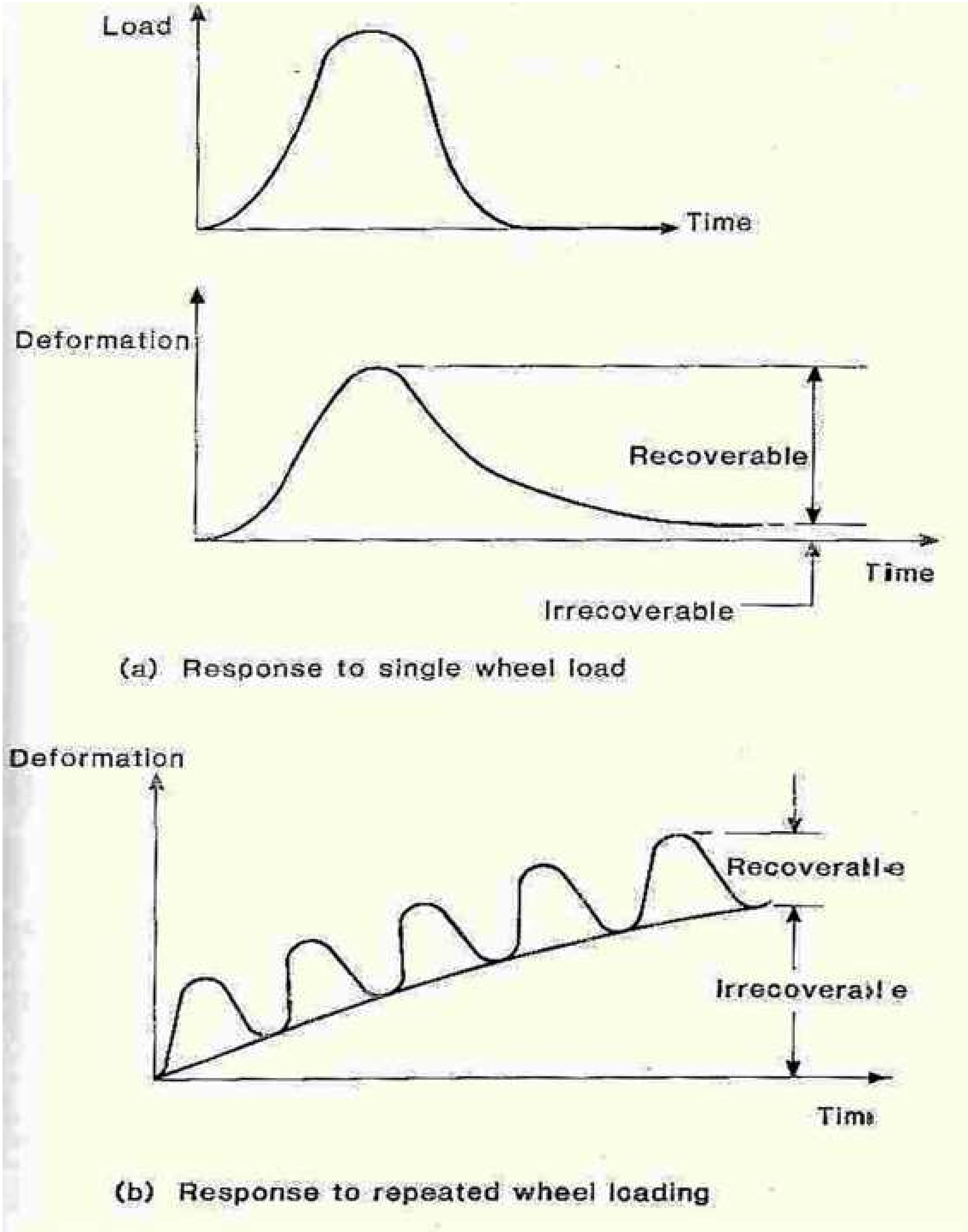
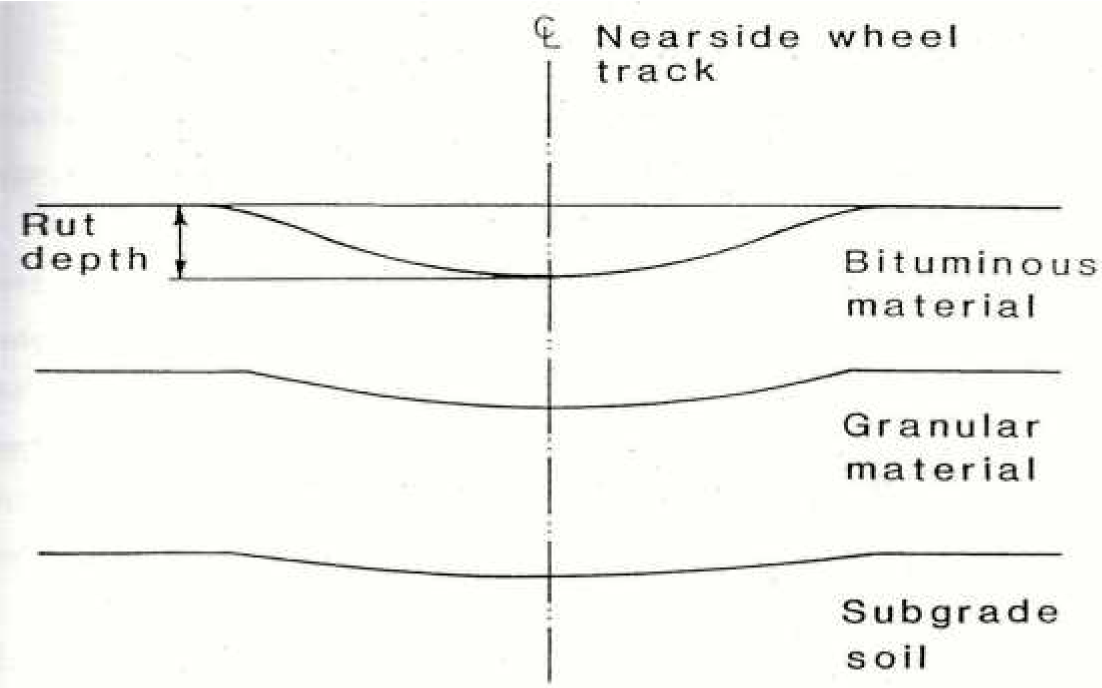
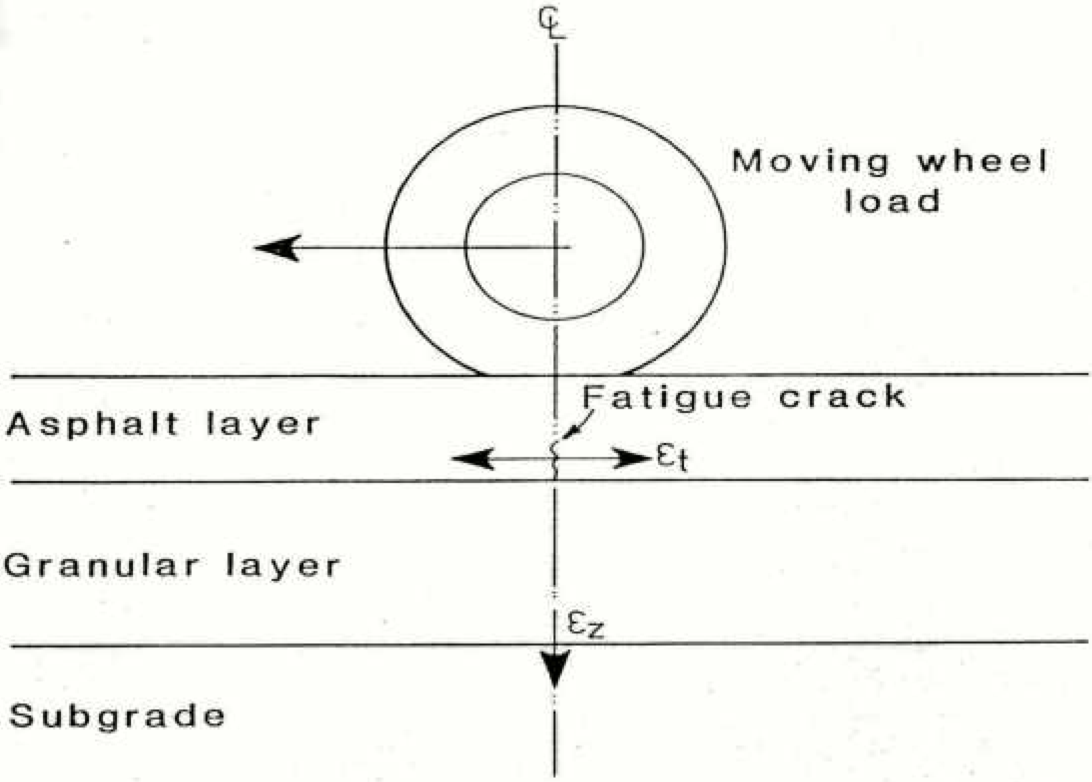


Figure3 Pavement Material Response to Loading



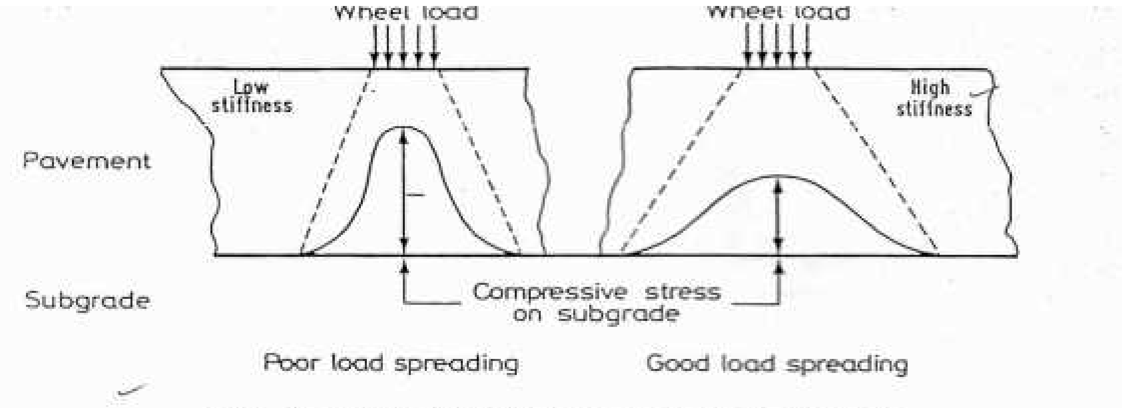
(a) Permanent deformation



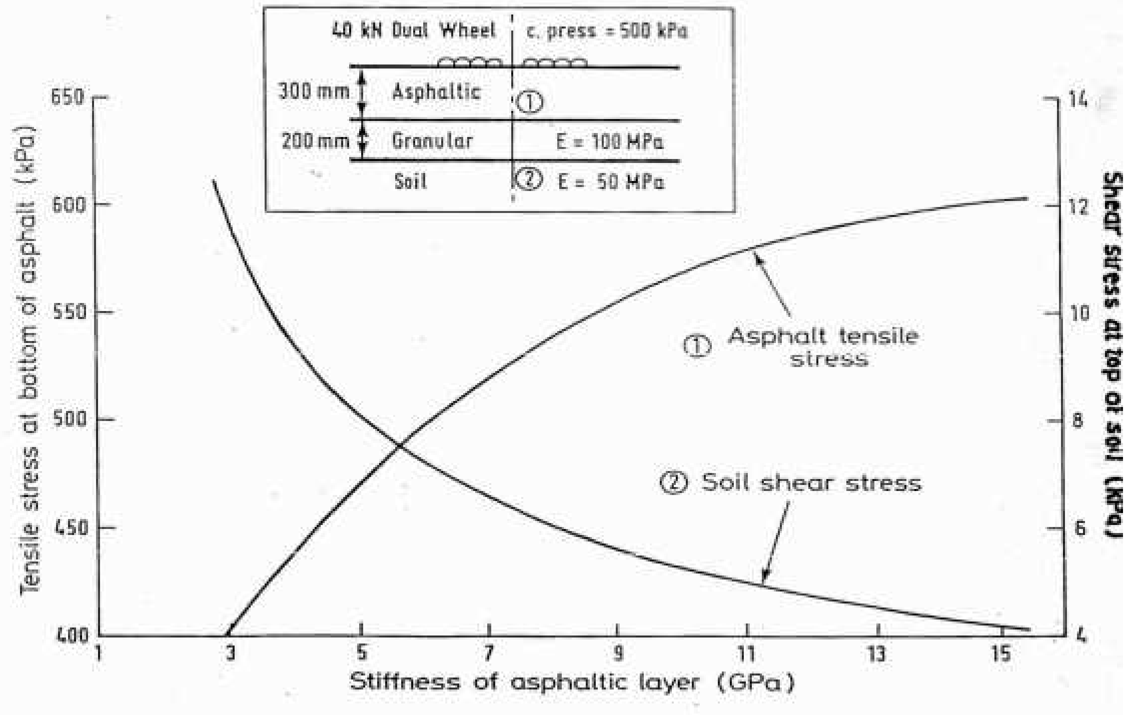
(b) Fatigue cracking and critical strains

Figure4 Failure Modes and Critical Strains in Asphalt Pavement

Increasing the elastic stiffness of a pavement improves its load spreading ability, thus reducing the peak stress transmitted to the subgrade as shown in Figure 5 (a). However, as the stiffness increases, in addition to this reduction of shear stress on the subgrade, there is an associated increase in tensile stress at the bottom of the bituminous layer. This creates the possibility for cracking and indicates the essential balance required in pavement design as shown in Figure 5 (b).



(a) Concept of load spreading to protect subgrade.



(b) Influence of asphaltic layer stiffness on critical stresses

Figure5 Importance of Elastic Stiffness

In case of rutting, so far as the asphalt layer is concerned, this can be minimized by a suitable mix design procedure based on the use of a performance test and by good compaction of all layers. If the vertical strain in the subgrade, is kept below a certain level, experience has shown that excessive rutting will not occur, unless poor mix design or inadequate compaction are involved.

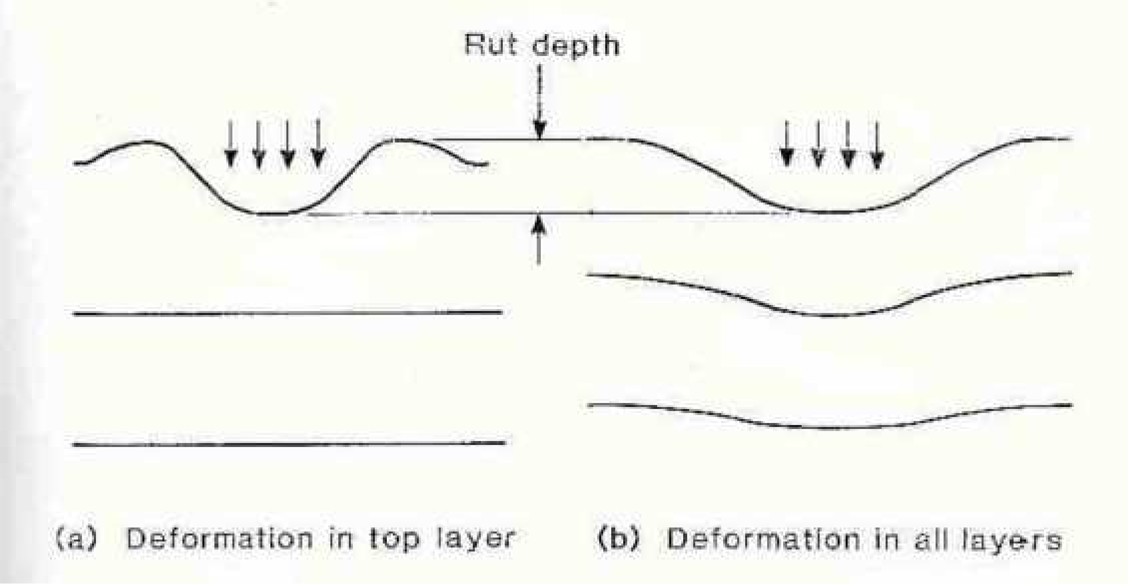


Figure6 Rutting Mechanisms

The design problem, then, is to so proportion the pavement structure that, for the chosen material, the critical levels of strain will not be exceeded in the design life. A simplified structure is used for design procedure and hence the variables in the design problem may be reduced to three:

a. The elastic stiffness of the asphalt layer.
b. Thickness of the asphalt layer.
c. The elastic stiffness of the subgrade, which may be related to the CBR.

2.3 Pavement Loading (3)

Loads are applied to pavements through the contact between tires and the wearing course. It is in the nature of vehicle loading that it is variable both in terms of magnitude and speed. The influence of speed or rate of loading will be discussed later. It is appropriate here to consider load magnitude and its relation to pavement design. Since loading is variable, it is necessary for design purposes to simplify the real situation by converting to an “equivalent” loading system. This idea originated with the AASHO Road Test in the USA and was adopted in U.K. practice with the publication of the third edition of Road Note No. 29 in 1970.

The mixed axle load spectrum is converted to an equivalent number of standard axles, the standard adopted in the U.K. and Nepal (5) being 80 kN. This equivalence is based on equivalent damage to the pavement. Although the relative damage done by wheel loads of different magnitudes depends on variety of factors, for normal design calculations use is made of a fourth power law.

2.4 Temperature Variations (3)

The literature on bitumen and bituminous mixtures under material properties demonstrates that the mechanical properties of such materials are strongly dependent on temperature. Since this parameter is varying almost continually in the field, a simplified approach is needed to deal with this variable in design.

2.5 Moisture Conditions (3)

The mechanical properties of all pavement materials and soils are to a greater or lesser extent adversely affected by water. It is particularly important that the subgrade and the granular layers should be well drained.

2.6 Pavement Design Philosophy (3)

There are basically two approaches to pavement design; empirical and analytical. Most design methods in current practice around the world are empirical, being based on experience accumulated in practice and from specially constructed test sections either on public roads or test tracks. It tends to be remote from engineering principles and cannot cope with circumstances beyond those included in the experience on which the method is based.

The analytical approach, by contrast, uses theoretical analysis, mechanical properties of materials and is capable, in principle, of dealing with any design situation.

2.7 Pavement Design Principles

Against the background of the foregoing literature, it is possible to identify the following basic principles for pavement design:

1. Ensure adequate drainage of the foundation.

2. Ensure adequate load spreading from the road base.
3. Ensure that the road base will not crack.
4. Ensure that the pavement deformations which accumulate in each layer are not excessive.

2.8 Material and Mix Properties

2.8.1 Asphalt

The link between material properties and pavement design is vital and forms a central theme of the analytical approach to design. Bitumen is obtained after separation of the lubricating oils. They are semisolid hydrocarbons with certain physiochemical characteristics that make them good binding agents. They are also very viscous and used as a binder for aggregates in pavement construction.

Bitumen are used mainly in the manufacture of hot-mix, hot-laid asphalt concrete, which is described later in this chapter. Bituminous concrete can be used in a variety of ways, including the construction of highways and airport pavement surfaces and bases, parking areas, and industrial floors. The specific use of a given sample depends on its grade.

1. Manufacture

Bitumen meeting the requirements of engineers can be refined from a wide variety of crude oils from any sources throughout the world.

In simple terms the manufacture of bitumen involves distillation, blowing and blending. Atmospheric distillation is used to separate gas, gasoline, kerosene and gas oil. The residue produced is then redistilled under vacuum to separate further distillates without subjecting the products to high temperatures. The vacuum residues obtained from some crude oils are bitumen satisfactory for use without further treatment. In many cases, however, vacuum residues are processed by air ratification (blowing) to produce harder penetration grade bitumen which can then be blended to produce intermediate grades.

2. Specification

The consistency, quality and certain other properties of bituminous binders must be carefully controlled if successful pavements are to be constructed with them;

specifications and tests furnish this control (6). BS 3690, Part 1, provides the comprehensive requirements for penetration bitumen. Standard specifications also have been published by the Asphalt Institute for the types of asphalts used in pavement construction. Procedures for selecting representative samples of asphalt for testing have been standardized and are given in MS-18 by the Asphalt Institute and in D140 by the ASTM.

In most engineering applications of bitumen their inherent temperature susceptibility is utilized to ensure they are sufficiently fluid during application and sufficiently stiff when in use. To increase their fluidity for some applications penetration grade bitumen may be mixed with volatile solvents such as kerosene to produce ‘cutbacks’ or used as ‘emulsions’. With cutbacks the volatile solvents evaporate during application leaving stiff residual bitumen. Emulsions are dispersions of bitumen in water and are designed to ‘break’ leaving a continuous film of stiff residual bitumen after application. BS 434 covers emulsions suitable for various applications.

3. Mechanical Properties and Performance

For a complete description of the mechanical properties of bitumen it is necessary to know the tensile strength and the ‘stiffness’ as a function of temperature and rate of loading. For most normal bitumen the tensile strength at low temperatures is approximately a constant, 4 Mpa. Only the stiffness requires a closer consideration for us to appreciate the mechanical properties.

Performance involves consideration of such things as stability, hardening during use, and weatherability. Perhaps the most important quality required to consider in bitumen performance is an appreciation of temperature – viscosity relationships.

a. Temperature-Viscosity Considerations

During application bitumen must be:

- Fluid enough to coat the aggregate;
- Viscous enough not to run off the aggregate during mixing or transport;
- Fluid enough for the mix to remain workable during compaction;
- Viscous enough to carry traffic

To achieve this, the operating temperature must be selected to ensure that the appropriate viscosity is attained.

Figure 7 shows two typical relationships between viscosity and temperature. Using the scales which are shown, these can be represented as straight lines. The viscosity scale is terminated at value of 10^5Ns/m^2 since above this the material is relatively solid and viscosity is an inappropriate characteristic.

The scale of penetration is also included in Figure 7 and it can be seen that the results from this test and the softening point test provide two points to define the line for a particular binder. The softening point viscosity corresponds to a penetration of 800 for practical purposes.

The slope of the characteristic line for a binder in Figure 7 indicates its temperature susceptibility and this is related to the term Penetration Index developed by Pfeiffer and Van Doormal (7).Using this system the PI of bitumen varies between about -2.5 and +8.0. Bitumens of lower and negative PI soften more readily than those with higher PI.

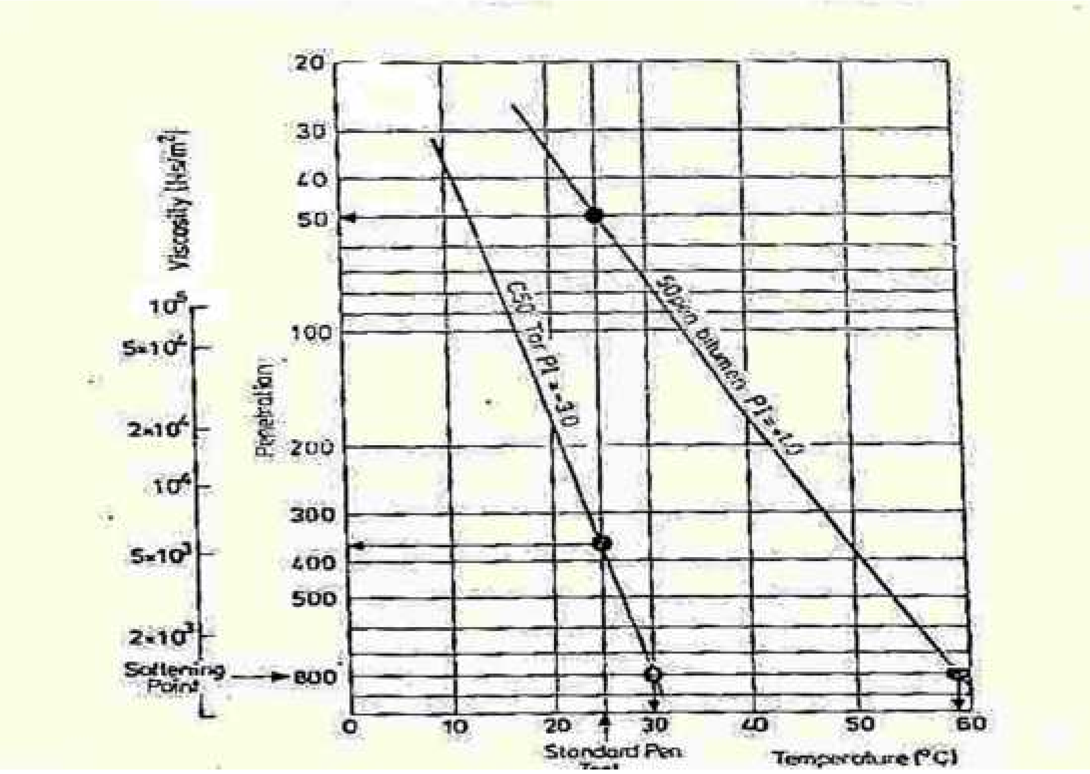


Figure7Relationship between Viscosity or Penetration and Temperature

b. Bitumen Stiffness

One of the most important concepts necessary to appreciate the behavior of bitumen, and bituminous materials, is the relationship between stress and strain. Unfortunately, the behavior of bitumen is very complex since their stress-strain characteristics are time dependent as well as temperature dependent as shown in Figure 8.

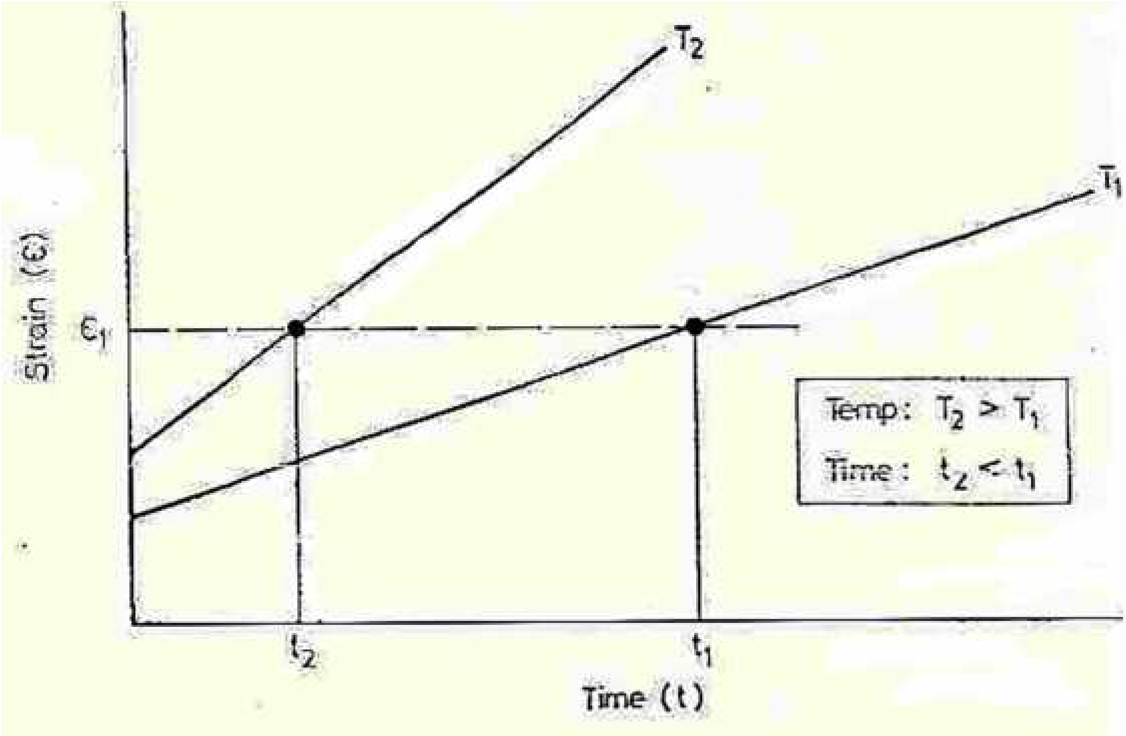


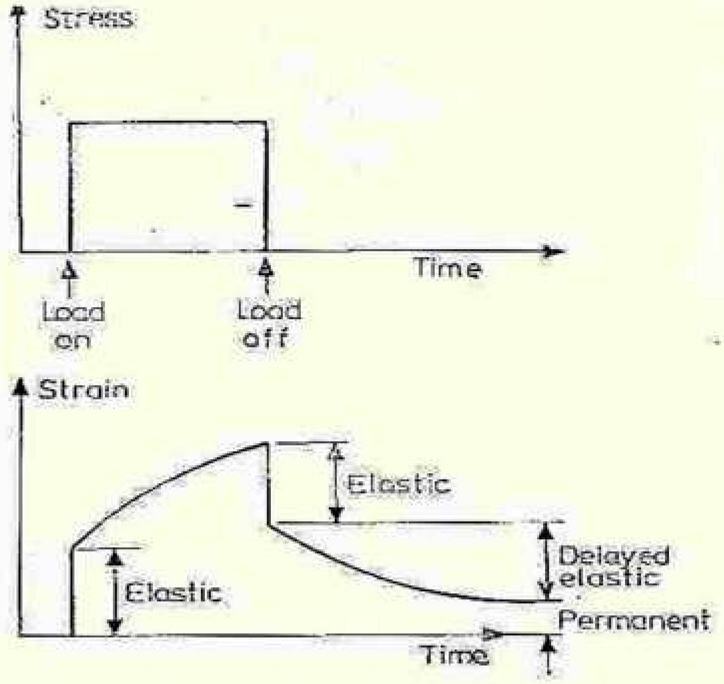
Figure8 Strain variations with Time under constant Stress

Loaded quickly they will respond elastically and recover most of the induced strain. Loaded for a long time they behave viscously and a major proportion of the induced strain will not be recovered this is visco-elastic response of bituminous material as shown in Figure 9. This irrecoverable strain is permanent strain and directly caused by viscous behavior.

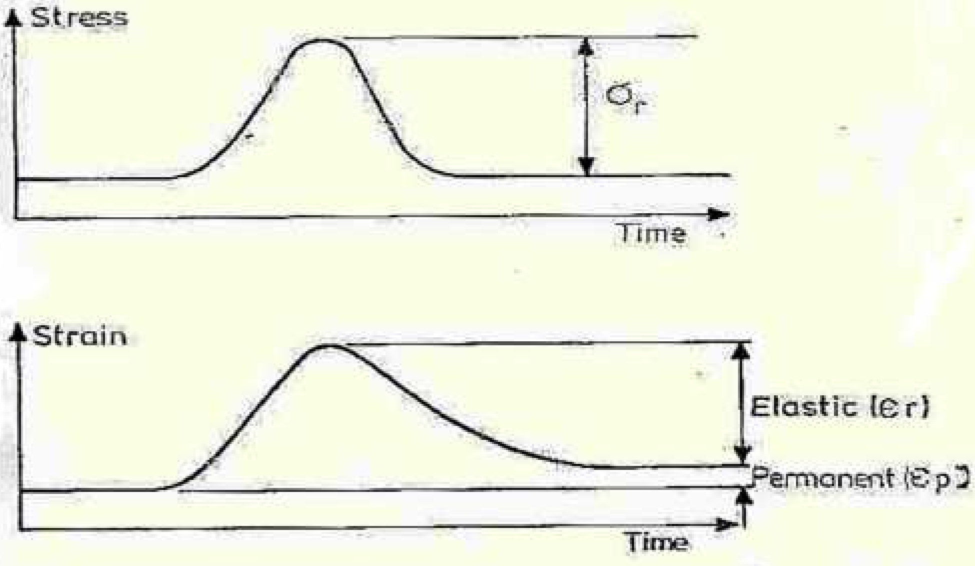
In addition, because they are thermoplastic their strain response will also depend upon temperature at which the stress is applied.

Hence, the simple concept of a Young’s modulus having a single value for a particular material, does not apply. Instead the term ‘stiffness’ S_b, the ratio of stress to strain for bitumen at a particular temperature and loading time is used: Stiffness (S_b) = stress/strain (E1).

An indication of loading time in some practical cases is given below and may be estimated by the following empirical relationship: Loading Time (sec) = 1/Traffic Speed (Km/hr) (E2)



(a) Simple creep test



(b) Pulse from moving wheel load

Figure 9 Visco-Elastic Response of Bituminous Material

The following loading times are representative of typical situations:

Fast road traffic	0.01 – 0.10 seconds
Braking and accelerating traffic	0.10 – 1.00 seconds
Parked Vehicles	1.0 minutes – 10 hours

The temperature experienced in a road pavement cover a wide range, (-10°C to +60°C), depending upon location and climate. In the U.K. typical average annual air temperatures are between 8.5 and 10°C, with pavement temperatures approximately 3°C above average air temperatures. For Nepal, the average annual air temperature and pavement temperature has to be found.

Prediction of Bitumen Stiffness: Measurement of stiffness is only possible with specialized equipment. However, the results of very many such measurements have been used to prepare a nomograph shown in Figure 10 to enable the prediction of bitumen stiffness for a range of loading times and temperatures. This nomograph was developed by Van der Poel. The nomograph permits the estimation of bitumen stiffness within an accuracy of a factor of two which is satisfactory when one considers the large variation of stiffness with time and temperature. To use the nomograph the following information is required:

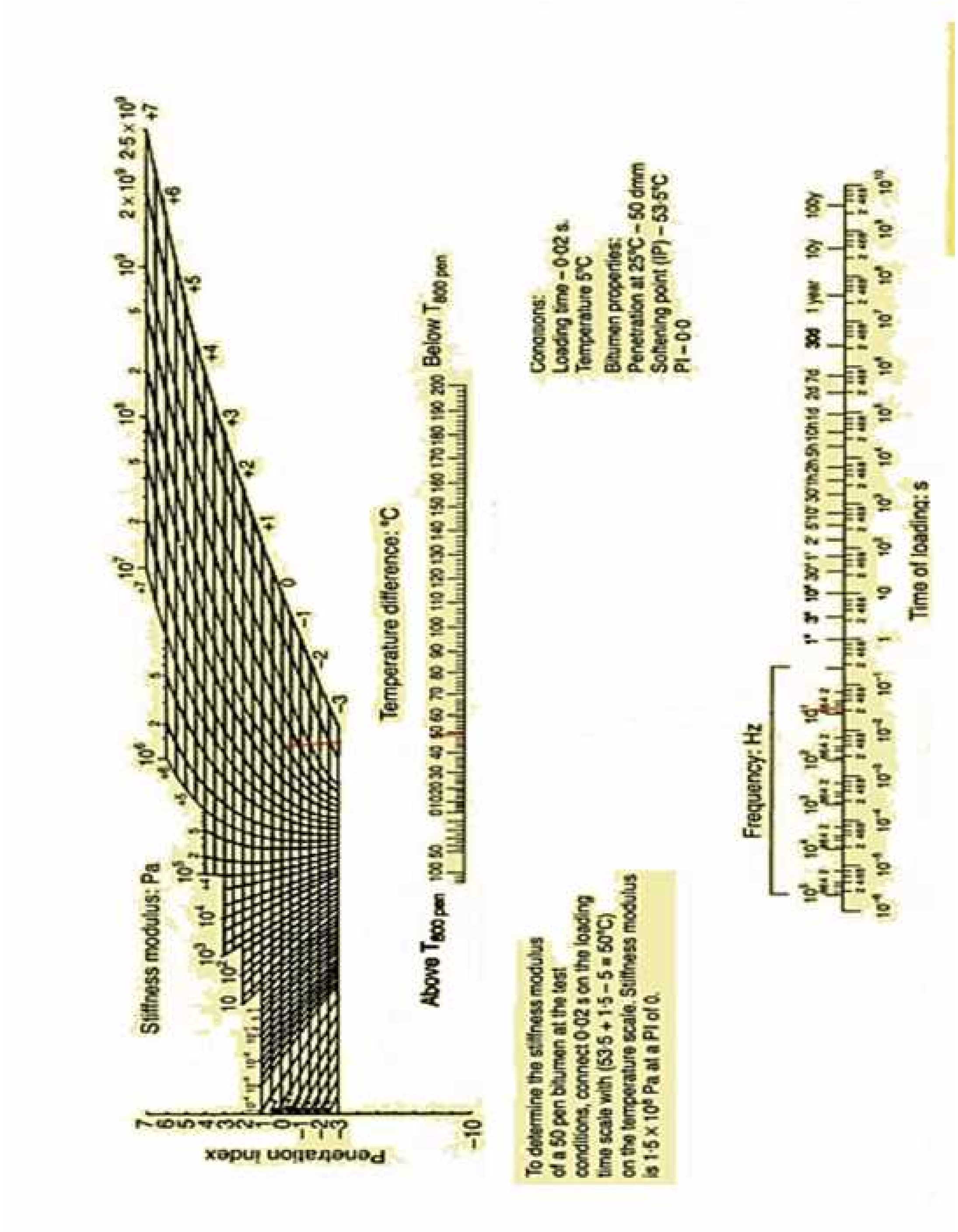
- Temperature, T, (°C)
- Softening Point (ASTM) of the bitumen, SP_r (°C), which in the nomograph is designated as the temperature at which the bitumen has a penetration of 800, T_{800} pen.
- Loading Time
- Penetration Index of the bitumen, PI_r

c. Hardening

The most significant hardening occurs during mixing and application when the bitumen is subject to high temperatures in thin films. British Standards allow for this hardening in their selection of particular grades for certain mixes. It is important however, not to exceed recommended mixing temperatures and to keep mixing times to minimum.

d. In Service Behavior (14)

It must be remembered that it is ‘in service behavior’ of the bitumen that is important. In the case of road pavement the bitumen properties must refer to the condition in the road following hardening that takes place during mixing and laying. Often the term



Conditions:
Loading time – 0·02 s.
Temperature 5°C
Bitumen properties:
Penetration at 25°C – 50 dmm
Softening point (IP) – 53·5°C
PI – 0·0

To determine the stiffness modulus of a 50 pen bitumen at the test conditions, connect 0·02 s on the loading time scale with (53·5 + 1·5 – 5 = 50°C) on the temperature scale. Stiffness modulus is 1·5 × 10⁶ Pa at a PI of 0.

Figure 10 Nomograph for determining the stiffness modulus of bitumens

'Recovered Properties' is used, denoted by the subscript 'r' for penetration (P_r), softening point (SP_r) and penetration index (PI_r).

If recovered properties of bitumen are not known they may be estimated from the equations below which have been established from studies of U.K. mixes used for pavement construction. Values prior to mixing are denoted by the subscript ‘i’ for initial.

1. Penetration

$P_r = 0.65\ P_i$

2. Softening Point

$SP_r = 98.4 – 26.4 \log P_r$

3. Penetration Index

$PI_r = (1951 – 500 \log P_r - 20\ SP_r)/(50 \log P_r - SP_r – 120)$

4. Service Conditions

During service (Construction and Under Traffic) it is necessary for bitumen to satisfy a variety of conditions such as mixing, placing, deformation and cracking.

2.8.2 Aggregate

The coarse aggregates used for making premix should be produced by crushing sound, unweathered rock or natural gravel. The specifications for the aggregates are similar to those for granular road bases. The aggregate must be clean and free of clay and organic material. To obtain good mechanical interlock and good compaction the particles should be angular and not flaky. Rough-textured material is preferable. Gravel should be crushed to produce at least two fractured faces on each particle. The aggregate must be strong enough to resist crushing during mixing and laying as well as in service. Aggregates which are exposed to traffic must also be resistant to abrasion and polishing. Highly absorptive aggregates are wasteful of bitumen and give rise to problems in mix design. They should be avoided where possible but if there is no choice, the absorption of bitumen must be taken into account in the mix design procedure. Hydrophilic aggregates which have a poor affinity for bitumen in the presence of water should also be avoided. They may be acceptable only where protection from water can be guaranteed.

The fine aggregate can be crushed rock and should also be clean and free from organic impurities. The filler (material passing the 0.075 mm sieve) can be crushed rock fines, Portland cement or hydrated lime. Fresh hydrated lime can help reduce the rate of hardening of bitumen in surface dressings and may have a similar effect in premixes.

1. Strength Properties

Angular and rough-textured aggregates are desirable within HMA to resist permanent deformation and fatigue cracking. Very angular and rough-textured aggregates provide better interlock between the aggregate particles which helps prevent plastic deformation (rutting) within HMA layers. Angular and rough-textured aggregates also help improve the strength of HMA mixtures, which can help prevent fatigue cracking. Angular aggregates with good surface texture also improve the frictional properties of pavement layers, an important safety consideration in the design of HMA for pavements.

The presence of flat or elongated particles within HMA is undesirable because these particles tend to break down during production and construction. Aggregates that break during production and construction will reduce the durability of the HMA layer, leading to raveling, pop-outs, and potholes.

Another aggregate characteristic related to performance is cleanliness and the presence of deleterious materials. Cleanliness is a term used to characterize the coatings on some aggregate particles. These coatings are often very fine clay-like materials and can affect the adhesion between the asphalt binder and aggregate particles leading to an increased potential for moisture damage. Deleterious materials are particles in an aggregate stockpile that are weak, prone to freeze-thaw damage or damage through repeated wetting and drying, or that otherwise can cause a pavement to deteriorate. Some examples of deleterious materials are clay lumps, friable particles, shale, coal, free mica, and vegetation. These types of materials are not as strong as mineral aggregates and break down during the life of a pavement layer. When this happens, pop-outs and potholes can occur.

Aggregate toughness and abrasion resistance have also been shown to be related to pavement performance. Aggregate particles that are tough and resistant to abrasion will not break down during the construction process, which helps ensure that an HMA mix can be properly constructed, placed, and compacted. Tough, abrasion-resistant aggregates also tend to produce a mix that is resistant to pop-outs and raveling. Because aggregate pop-outs and broken aggregate particles near the pavement surface make it easier for water to flow into a pavement, tough and abrasion- resistant aggregates help improve the moisture resistance of HMA pavements. Aggregates with poor abrasion resistance can also polish under the action of traffic. This can cause the pavement surface to lose skid resistance, especially when wet.

Another aggregate property that is closely related to toughness and abrasion resistance is durability and soundness. Freeze-thaw cycles and alternate periods of wetting and drying in a pavement can weaken poor-quality aggregates, causing pop-outs and raveling. Aggregates that possess good durability and soundness will resist the actions of wet-dry and freeze-thaw cycles during the life of the pavement.

2. Aggregate Gradation

Bituminous materials used in highway engineering are simply binary combinations of particles of mineral aggregates, and a binding agent which is usually a penetration grade bitumen. The majority of work associated with the design of bituminous mixtures has addressed the problem of identifying on optimum binder content, with much less effort concentrated on the influence of the aggregate gradation.

The structural strength of asphaltic concretes and coated macadam relies primarily on the friction, and mechanical interlock between aggregate particles. The addition of a binding agent to the gradation, such as bitumen, provides a lubricant which enables the material to become workable, allowing ease of compaction, and contributes to the final mixture properties.

The quantity of binder introduced to the gradation is critical, as too much, or too little will adversely affect the mixture properties, but it must be realized that similar consequences may result from variations in the aggregate grading.

The development of aggregate grading for use in road base materials has been empirical throughout practice in the United Kingdom, resulting in the envelopes currently specified in BS 4987 (9). These grading forms very dense matrices of stone particles, and are analogous to the type of gradations which have been developed for asphaltic concretes in the United States of America. The grading used in asphaltic concretes was developed through a philosophy which aimed to maximize the density of the mineral aggregate, and is based upon a gradation curve suggested by Fuller and Thomson in 1907 (10).

THE FULLER CURVE

Historically, the best known system which describes continuously graded material is the 'Fuller' curve, which was established from the results of laboratory experimentation. A mathematical relationship was formulated relating adjacent particle sizes, and led to the development of a series of curves represented by the equation:

$$P = (\frac{d}{D})^{n}$$

Where, P = percentage of material passing a sieve of size d mm

D = maximum particle size (mm)

n = an exponent between zero and 1

By selecting a value of the exponent between 0.4 and 0.5, the aggregate gradation will achieve maximum densification(13),and it is this format of the Fuller curve which has become the standard grading for asphaltic concretes. The n = 0.45 curve forms the target gradation for use in asphaltic concretes, irrespective of aggregate type, and is generally considered to be the optimum curve for a continuously graded system of rock particles. However, it has been proposed that an individual gradation cannot represent all aggregate types, because of the different properties exhibited by different rock types (11, 12). Therefore, it should be necessary to design a gradation according to the source material.

2.8.3 Asphalt Mix

Typical Compositions (Coated Stone, Continuously graded, Gap graded, and Mastic) has been reviewed. Load carrying mechanisms (Stone to Stone and Mortar Mechanism), mix requirements, engineering properties viz. elastic stiffness, viscous stiffness and permanent deformation and fatigue cracking along with other mix properties as durability (Adhesion and Bitumen Hardening) and workability (Spreading and Compaction) has been reviewed. Also asphalt mix design methods and recent development, principles of design, traditional design (Recipe Approach), laboratory test methods and test philosophy has been reviewed. Laboratory tests on aggregate, tests on asphalt and tests on asphalt mix have also been reviewed.

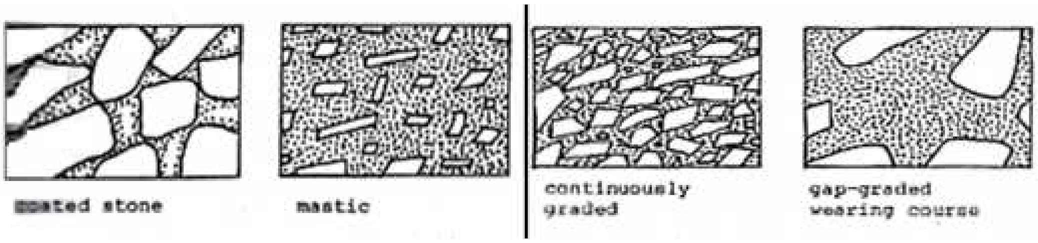


Figure 11 Typical Mixes

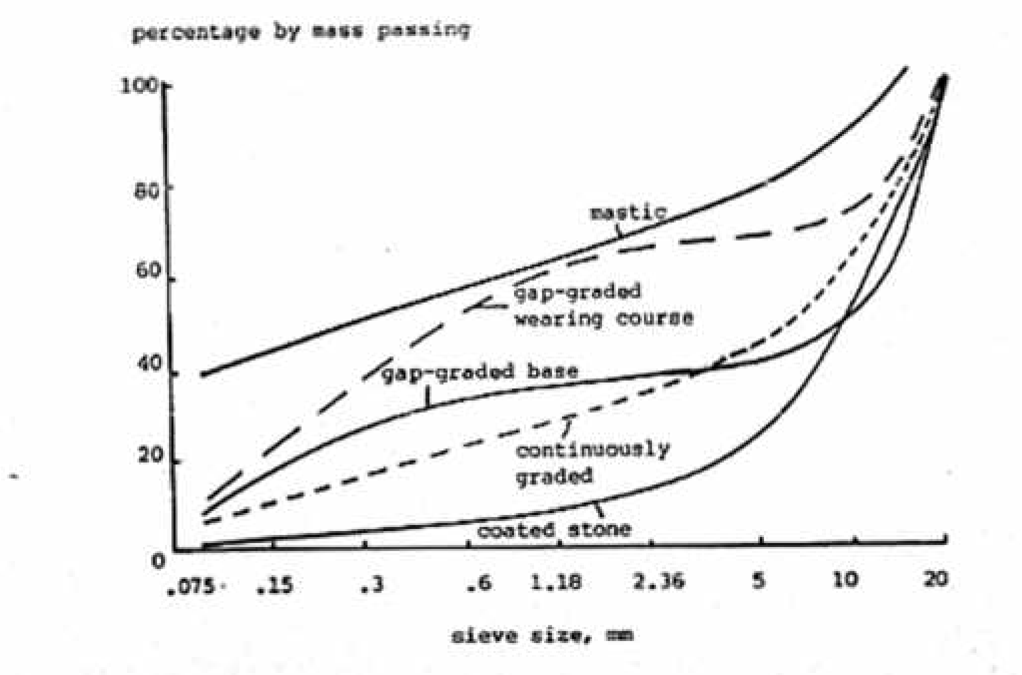


Figure 12 Aggregate Grading of Typical Mixes

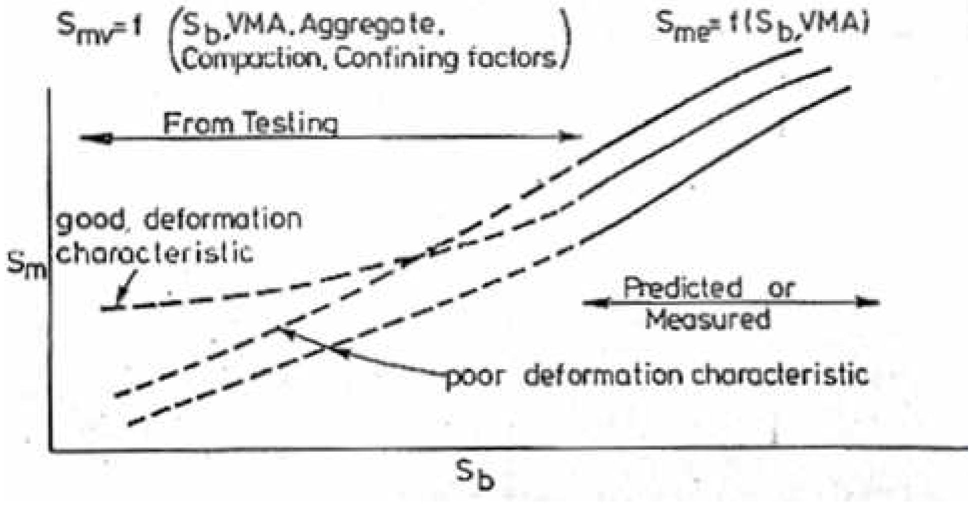


Figure 13 Complete Mix Stiffness and Bitumen Stiffness Relationship

2.8.4 Fatigue Strength Characteristics

One of the primary structural distress modes is fatigue cracking and it is necessary to discuss the fatigue strength characteristics of bituminous mixes and to determine the values of the fatigue performance necessary for design.

Fatigue has been defined as: the phenomenon of fracture under repeated or fluctuating stress having a maximum value generally less than the tensile strength of the material.

Under traffic loading the layers of a flexible pavement structure are subjected to continuous flexing. The magnitude of the tensile strains for a standard wheel load of the order of 30-200*10^-6 has been confirmed by measurements and under these conditions the possibility of fatigue cracking exists, and consequently fatigue is one of the failure criteria considered in pavement design (18).

Figure 14 shows some typical results of stress v cycles of load to failure using log-log scales where individual lives are plotted for tests at +10°C and it can be seen that a straight line passes through the mean of the logarithms of the lives at each stress level. It can be seen that a small change in stress level can result in a considerable change in life.

Figure 14 also shows fatigue lives for the same material at different temperatures from which it can be seen that the lines are approximately parallel with longer lives at lower temperatures. A similar effect is found if tests are carried out at different speeds, with longer lives at higher speeds.

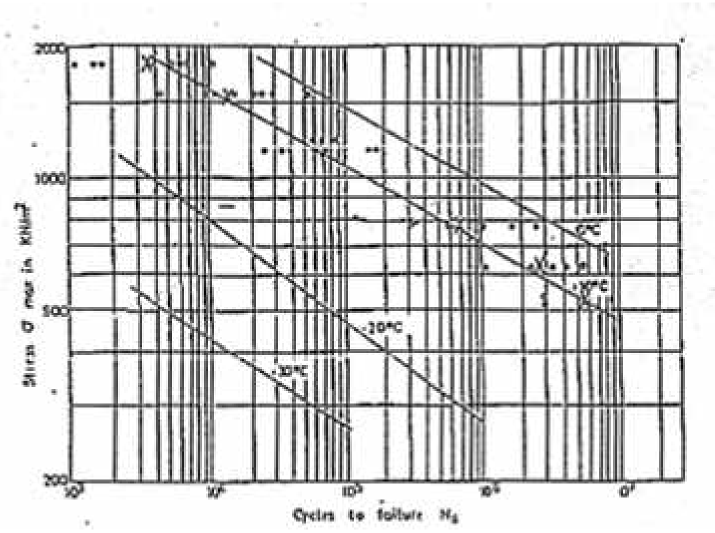


Figure 14 Typical Fatigue Lines (Stress v Cycles to Failure)

If the results of the fatigue tests are replotted in terms of strain as shown in Figure 15, then it is found that the results from different stiffness coincide, indicating that strain is criterion of failure, and that the effects of temperature and speed of loading can be accounted for by their effect on stiffness. This is known as “strain criterion” (19).

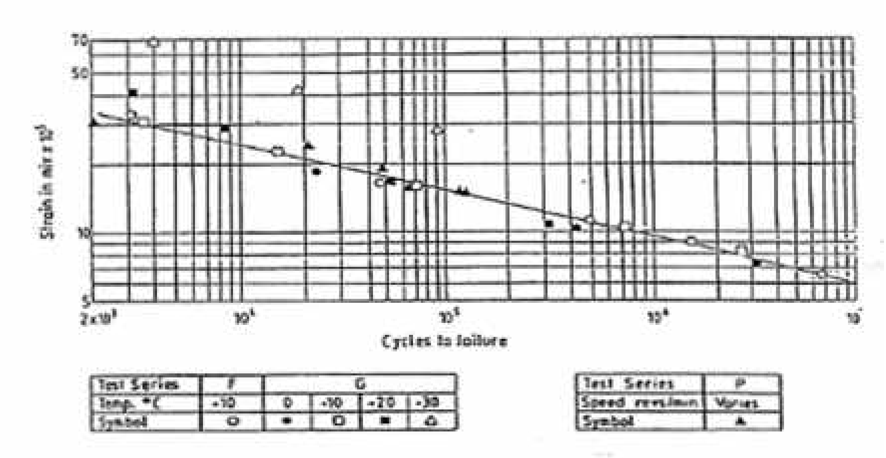


Figure 15 Fatigue Results - Strain Criterion

Strictly, this criterion of fatigue failure applies to crack initiation. The general relationship defining the fatigue life based on crack initiation is as follows:

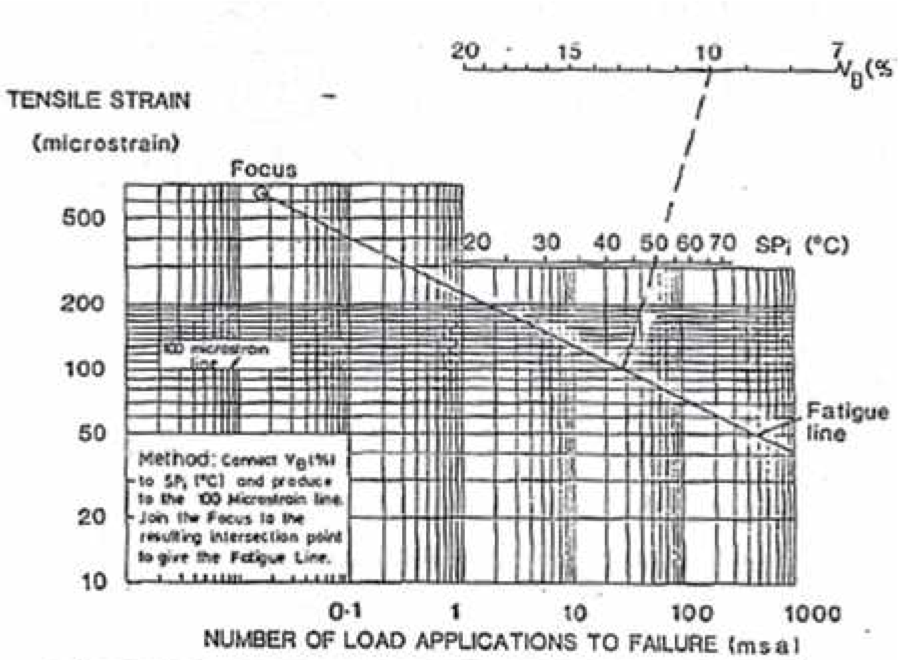
$N_f = c * (1/\varepsilon_t)^m$

Where, N_f = number of applications of load to initiate a fatigue crack,

ε_t =maximum value of applied tensile strain,

c and m = factors depending on the composition and properties of the mix.

Hence, the fatigue performance, based on crack initiation, for different mixes, can be predicted simply from the initial softening point temperature of the binder and the percentage volume of binder in the compacted mix as shown in nomograph, Figure 16 below.



Method: Connect V_B(%) to SP_i (°C) and produce to the 100 Microstrain line. Join the Focus to the resulting intersection point to give the Fatigue Line.

$$\log \varepsilon_t = \frac{14.39 \log V_B + 24.2 \log SP_i - 46.06 - \log N}{6.13 \log V_B + 8.63 \log SP_i - 15.8}$$

$$\log N = 15.8 \log \varepsilon_t - 46.06 - (6.13 \log \varepsilon_t - 14.39) \log V_B - (8.63 \log \varepsilon_t - 24.2) \log SP_i$$

Figure 16 Nomograph for the Determination of Fatigue Strength (Derived from Cooper and Pell)

2.8.5 Permanent Deformation Characteristics

The resistance of a bituminous mix to permanent deformation involves consideration of the low stiffness response at high temperatures or long loading times, expressed as the viscous stiffness (S_{mv}). Under these conditions when, $S_b < 5$ Mpa, the behavior of the mix is more complex than in the elastic zone. Its stiffness, in addition to depending on S_b and VMA is also affected by such factors as the grading, shape and texture of the aggregate, the confining conditions and the method and state of compaction. The simplest test to study the permanent deformation response of bituminous mixes is the static uniaxial unconfined creep test.

In view of the many factors affecting the resistance of a mix to permanent deformation discussed above, it is not possible to predict permanent deformation without laboratory testing. This situation is unlike that for elastic stiffness and fatigue performance, which can be predicted, from easily obtainable information on the composition of the mix, to an accuracy acceptable for pavement design. This means that a different, indirect approach is necessary in the design process to ensure adequate performance against the accumulation of permanent deformation compared with the direct approach possible for fatigue cracking performance.

The subgrade strain criterion is less fundamentally based than that for fatigue cracking. It is used to ensure that excessive rutting does not occur in the pavement design life and is based on the back-analysis of various pavements of known performance. It is hence an indication of rutting rather than a direct measure of it.

CHAPTER 3: STUDY DATA AND METHODOLOGY

3.1 General

The purpose of this research is to provide recommendations on average compositions of typical mixes applicable to Nepalese condition. Literature Review has been done which has helped to develop a methodology for undertaken research.

The present outcome of literature review has been presented as:

1. Stiffness of Bitumen, S_b depends on loading time, softening point, penetration index and temperature, i.e. $S_b = f$ (LT, SP, PI, T)
2. Voids in Mix/Mineral Aggregate, VMA = $V_V + V_B$
3. Volume of air voids depends on theoretical maximum density and mix density
4. Stiffness of mix when greater than 5 MPa depends on stiffness of bitumen and voids in mixed aggregate. $S_{me}= f$ (S_b, VMA) as shown in Figure 13.

Stiffness of mix when less than 5 MPa shows visco-elastic to viscous behavior as shown in Figure 17.

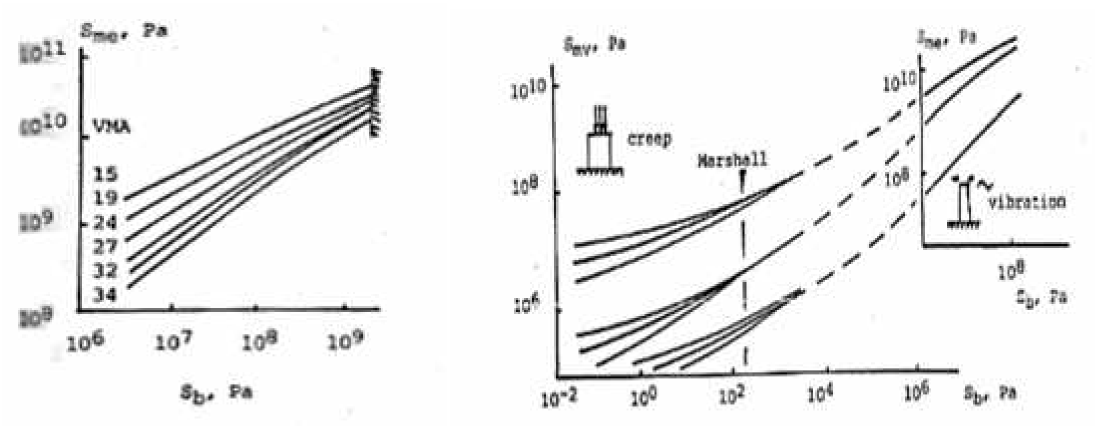


Figure 17 Elastic Stiffness of Mixes and General picture of Stiffness range

3.2 Data collection and Methodology

For Bitumen Stiffness

- Loading time has been categorized as

Fast road traffic	0.01 – 0.10 seconds
Braking and accelerating traffic	0.10 – 1.00 seconds
Parked Vehicles	1.0 minutes – 10 hours

- Temperature

Average annual air temperature has been found out from Department of Meteorology and for average pavement temperature readings has been taken at different locations.

- Softening point of different samples (60/70 and 80/100) of bitumen has been assesses from available secondary data and accordingly penetration index has been determined from empirical relation/nomograph.

Bitumen Stiffness in a range has been obtained after calculation from above collected data. Calculation has been done on the basis of empirical relation/nomograph.

For VMA

- Mix density of samples designed in laboratory, theoretical maximum density and specific gravity for different aggregates used in Nepal has been obtained from available secondary data.
- VMA in range has been obtained from calculations of V_V and V_B.

With range of bitumen stiffness and VMA so obtained a range of mix stiffness has been obtained.

CHAPTER 4: RESULTS AND ANALYSIS

1. For Bitumen Stiffness

As Stiffness of Bitumen, S_b depends on loading time, softening point, penetration index and temperature, i.e. $S_b = f$ (LT, SP, PI, T) so,

a. Loading time has been categorized as

Fast road traffic	0.01 – 0.10 s (such as NH)
Braking and accelerating traffic	0.10 – 1.00 s (at grades and urban area)
Parked Vehicles	1.0 minutes – 10 hours (Airports and others)

b. Temperature

Air temperature has been taken from reports of Department of Hydrology and Meteorology as -15°C to 47°C(15) (16) and corresponding pavement temperature has been established by readings at different locations taking note of temperature difference between air and pavement and plotting a best fit line using the obtained data as follows:

		1	Date	073-03-12		
Time	10:55 AM	Location	TRP	169+200	Pathlaiya Site Camp	
Ambient Temperature	35.5	35.3	35.6	35.47	Difference	
Pavement Temperature	45.5	45.2	45.7	45.47	10.00	
		2	Date	073-03-12		
Time	11:06 AM	Location	MRM	366+000	Pathlaiya East	
Ambient Temperature	36	37.4	37	36.80	Difference	
Pavement Temperature	48	50	49.2	49.07	12.27	
		3	Date	073-03-12		
Time	12:55 AM	Location	TRP	169+200	Pathlaiya Site Camp	
Ambient Temperature	36.7	37	37.2	36.97	Difference	
Pavement Temperature	53.3	52.2	52.3	52.60	15.63	
		4	Date	073-03-12		
Time	1:05 AM	Location	MRM	366+000	Pathlaiya East	
Ambient Temperature	38	38	38.2	38.07	Difference	
Pavement Temperature	55.5	55	55.7	55.40	17.33	
		5	Date	073-04-02		
Time	2:35 AM	Location	MRM		DRO, Hetauda	
Ambient Temperature	29.8	30	30.5	30.10	Difference	
Pavement Temperature	38.1	38.3	37.2	37.87	7.77	
		6	Date	073-04-04		
Time	6:40 AM	Location	Maitighar		Mandala	
Ambient Temperature	22.2	22.2	22.3	22.23	Difference	
Pavement Temperature	25.6	25.1	25.3	25.33	3.10	
		7	Date	073-04-04		
Time	7:10 AM	Location	Patan		DOR	
Ambient Temperature	22.6	22.5	22.6	22.57	Difference	
Pavement Temperature	26.6	26.6	26.7	26.63	4.07	

Figure 18 Recording Pavement Temperature

		8	Date	073-04-04		
Time	8:00 AM	Location	Harihar Bhawan		Pulchowk	
Ambient Temperature	24.2	24.1	24.2	24.17	Difference	
Pavement Temperature	28.2	28.3	28.3	28.27	4.10	
		9	Date	073-04-04		
Time	3:25 PM	Location	Patan		DOR	
Ambient Temperature	31	33	32	32.00	Difference	
Pavement Temperature	42	44	42.8	42.93	10.93	
		10	Date	073-04-25		
Time	8:30 PM	Location	Hetauda		DRO, Hetauda	
Ambient Temperature	33.5	33.5	33.5	33.50	Difference	
Pavement Temperature	39	38.6	39.2	38.93	5.43	
		11	Date	073-04-25		
Time	11:15 AM	Location	Simara		1 Km towards Birgunj	
Ambient Temperature	37	37	37	37.00	Difference	
Pavement Temperature	49	48.5	48.6	48.70	11.70	
		12	Date	073-04-25		
Time	12:00 PM	Location	Birgunj		1 Km from Ghantaghar towards Pathlaiya	
Ambient Temperature	37	37	37	37.00	Difference	
Pavement Temperature	47.7	47.2	47.1	47.33	10.33	
		13	Date	073-04-25		
Time	1:15 PM	Location	Birgunj		1 Km from Ghantaghar towards Pathlaiya	
Ambient Temperature	32	32	32	32.00	Difference	Cloudy
Pavement Temperature	44	43.8	43.9	43.90	11.90	
		14	Date	073-04-25		
Time	3:00 PM	Location	Birgunj		1 Km from Ghantaghar towards Pathlaiya	After Rain
Ambient Temperature	34	34	34	34.00	Difference	
Pavement Temperature	47.2	47.7	46.9	47.27	13.27	
		15	Date	073-05-01		
Time	10:45 AM	Location	Aghor Bazar, TRP		Chainage	
Ambient Temperature	25	25	25	25.00	Difference	
Pavement Temperature	35.8	35.6	34.9	35.43	10.43	
		16	Date	073-05-01		
Time	11:25 AM	Location	Simbhanjyang		Chainage (53 Km from Hetauda)	
Ambient Temperature	23	23	23	23.00	Difference	
Pavement Temperature	31.9	31.3	31.7	31.63	8.63	

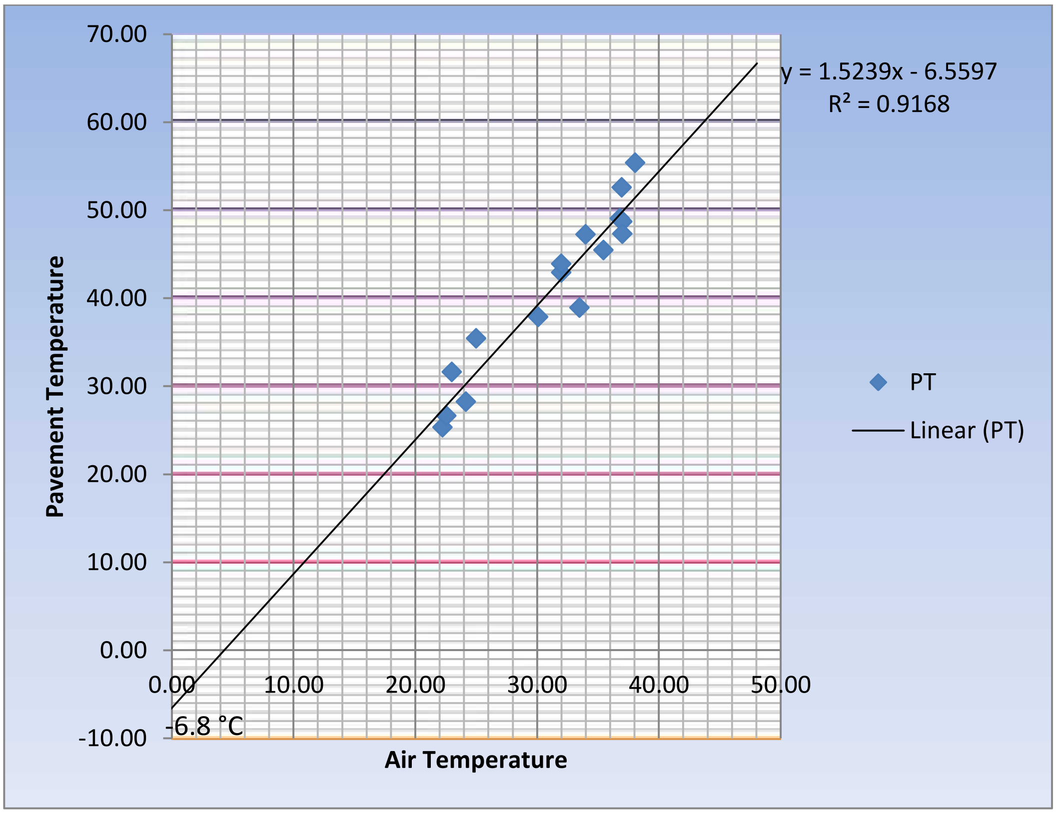


Figure 19 Pavement Temperature vs Air Temperature

The pavement temperature corresponding to highest temperature has been taken as 65°C and for lowest is taken as -7°C as there is no significant fall in pavement temperature beyond that as can be seen from Figure 15.

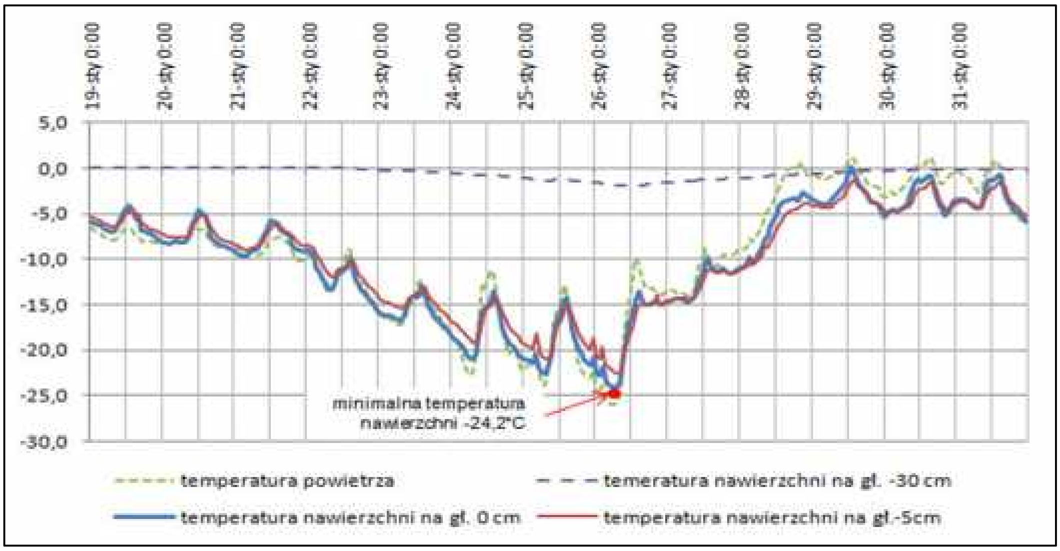


Figure 20The example of temperature changes recorded at the DK8 Podborze meteorological station in the Mazowieckie province, Poland. (17)

c. Softening point of different samples (60/70 and 80/100) of bitumen from test data (2062 BS- 2073 BS, random sampling) at Bitumen Barrel Udhyog, Amlekhgunj, Bara comes out to be in a range between 44°C-48.5°C with an average value of 46.74 for bitumen of penetration grade 80/100. The specification SSRBW specifies the range for 80/100 as 41°C-51°C and for 60/70 as 44°C-54°C so the range here has been taken as 41°C-54°C. For in service behavior then softening point comes out to be 50.54°C-56.4°C for penetration range of 60-100 from following relations.

   1. $P_r = 0.65\ P_i$, For 60-100, $P_r = 39\text{-}65$
   2. Softening Point

      $SP_r = 98.4 – 26.4 \log P_r$, For 41°C-54°C, $SP_r = 50.54°C\text{-}56.4°C$
   3. Penetration Index

      $PI_r = (1951 – 500 \log P_r - 20\ SP_r)/(50 \log P_r - SP_r – 120)$

   The penetration index (PI) has been determined from above empirical relation and comes out to be -0.28 to -0.42, so an average of -0.3 has been taken for use. Also nomograph has been shown in Figure 17 below.

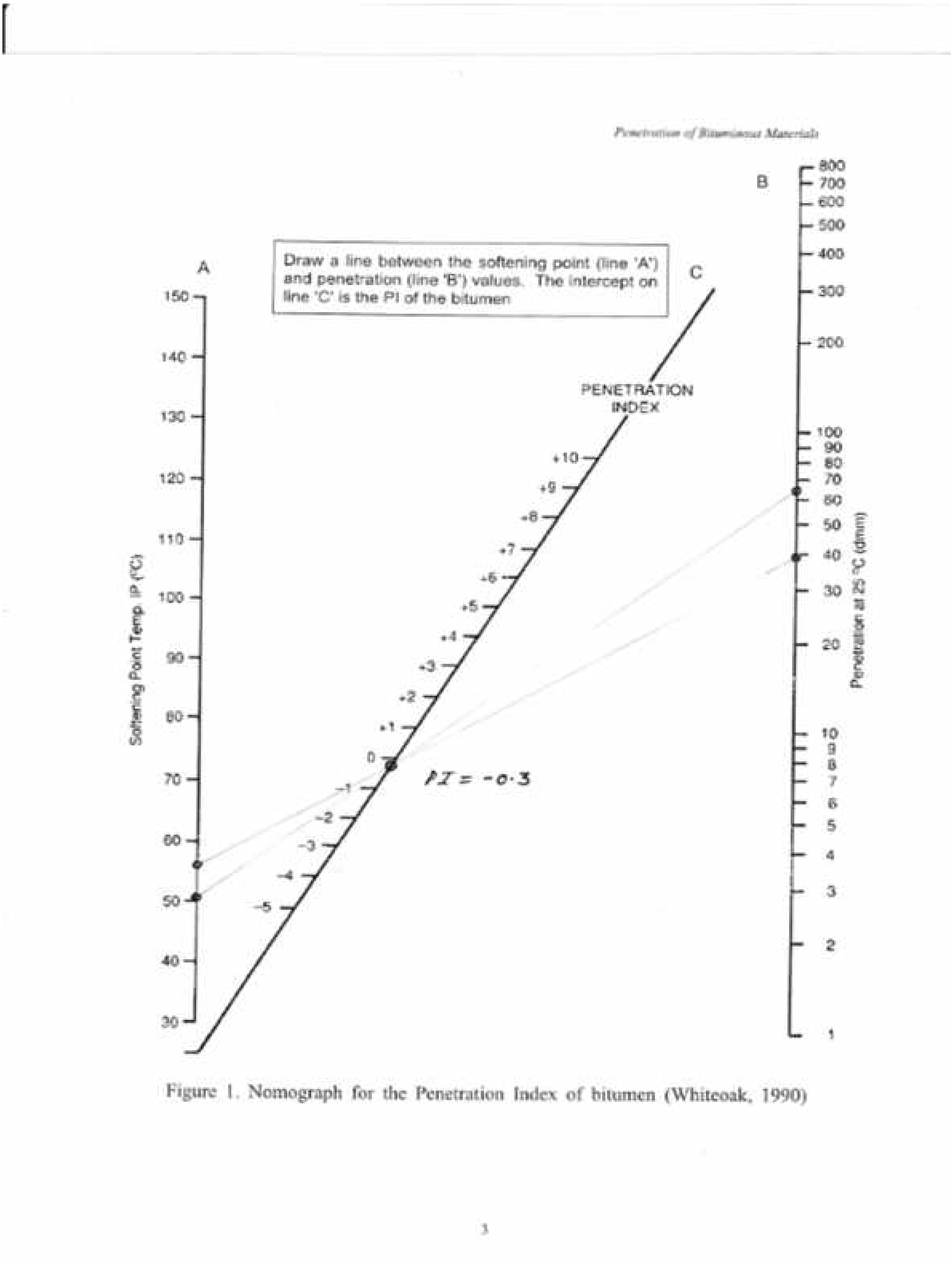
   The stiffness of Bitumen has been determined from use of above data using the nomograph for determining the stiffness modulus for bitumen and comes out to be in range of 2*10^-2 to 800 Mpa for loading time of 0.01s to 0.1s, 2*10^-3 to 500 Mpa for loading time of 0.1s to 1s, and 2*10^-7 to 75 Mpa for loading time of 1 min to 10 hrs as shown in Figure 18, Figure 19 and Figure 20 respectively.

2. For VMA

   For VMA Marshall Mix Design data was collected from Central Road Lab, DOR, Lalitpur and various mix designs available at Division Level of DOR, from these data after analysis of extremes the range for VMA was assessed as 12-24 for Nepalese Condition to make it compatible/adjustable to diverse conditions such as terrain, traffic, temperature and loading.

3. After determination ofrange of bitumen stiffness (2*10^-7 to 800 Mpa) and VMA, Mix (12-24) the mix stiffness in elastic region, S_{me} (Bitumen Stiffness > 5 Mpa) has been determined from empirical relationto be 370 Mpa to greater than 50000 Mpa.

   Mix Stiffness in viscous region, S_{mv} when bitumen stiffness is less than 5 Mpa depends upon bitumen stiffness, VMA, aggregate type, grading, shape, texture, interlock, confining conditions, compaction, voids and method as shown in Figure 13.



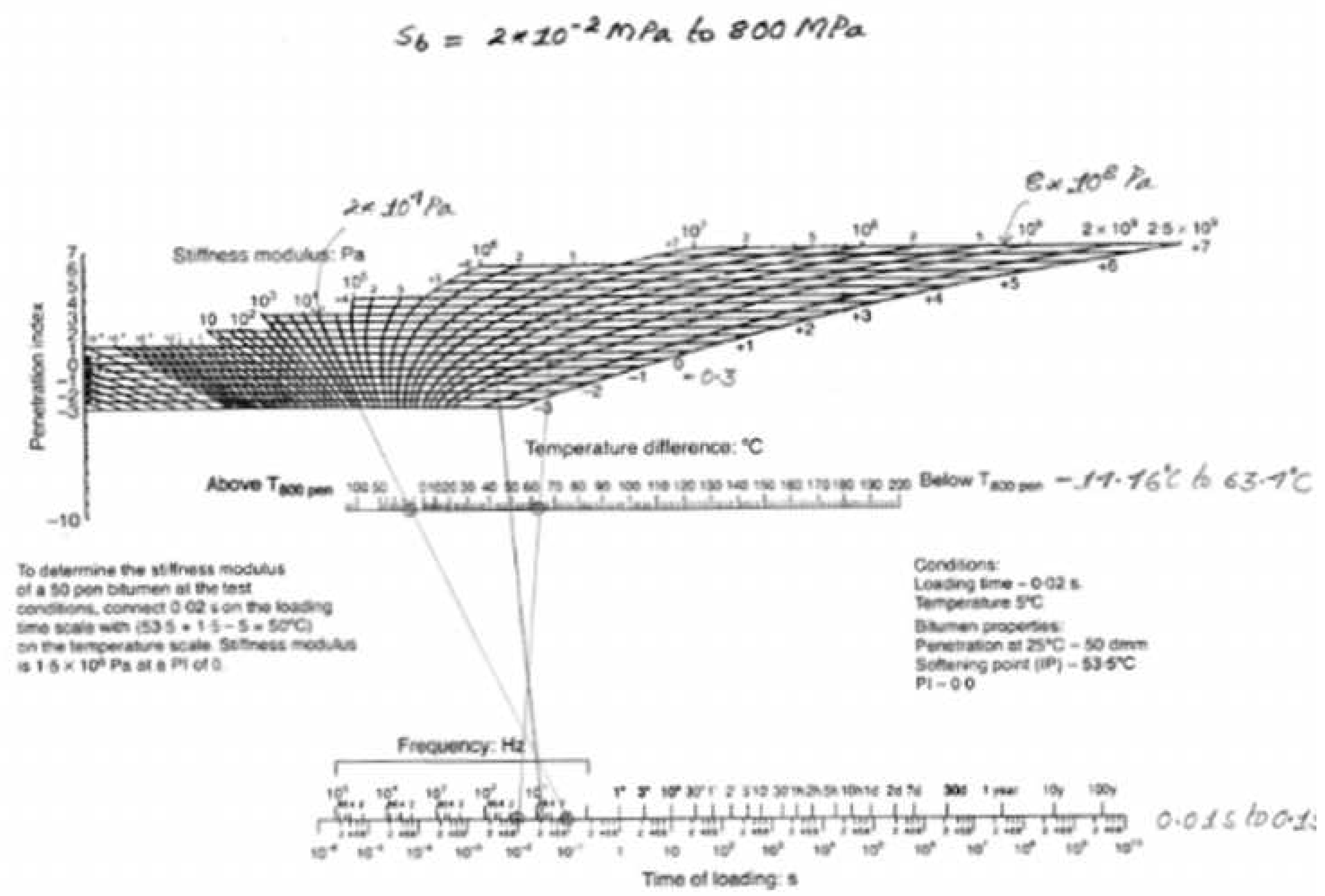
Draw a line between the softening point (line 'A') and penetration (line 'B') values. The intercept on line 'C' is the PI of the bitumen

PI = -0·3

Figure 1. Nomograph for the Penetration Index of bitumen (Whiteoak, 1990)

3

Figure 21 Penetration Index

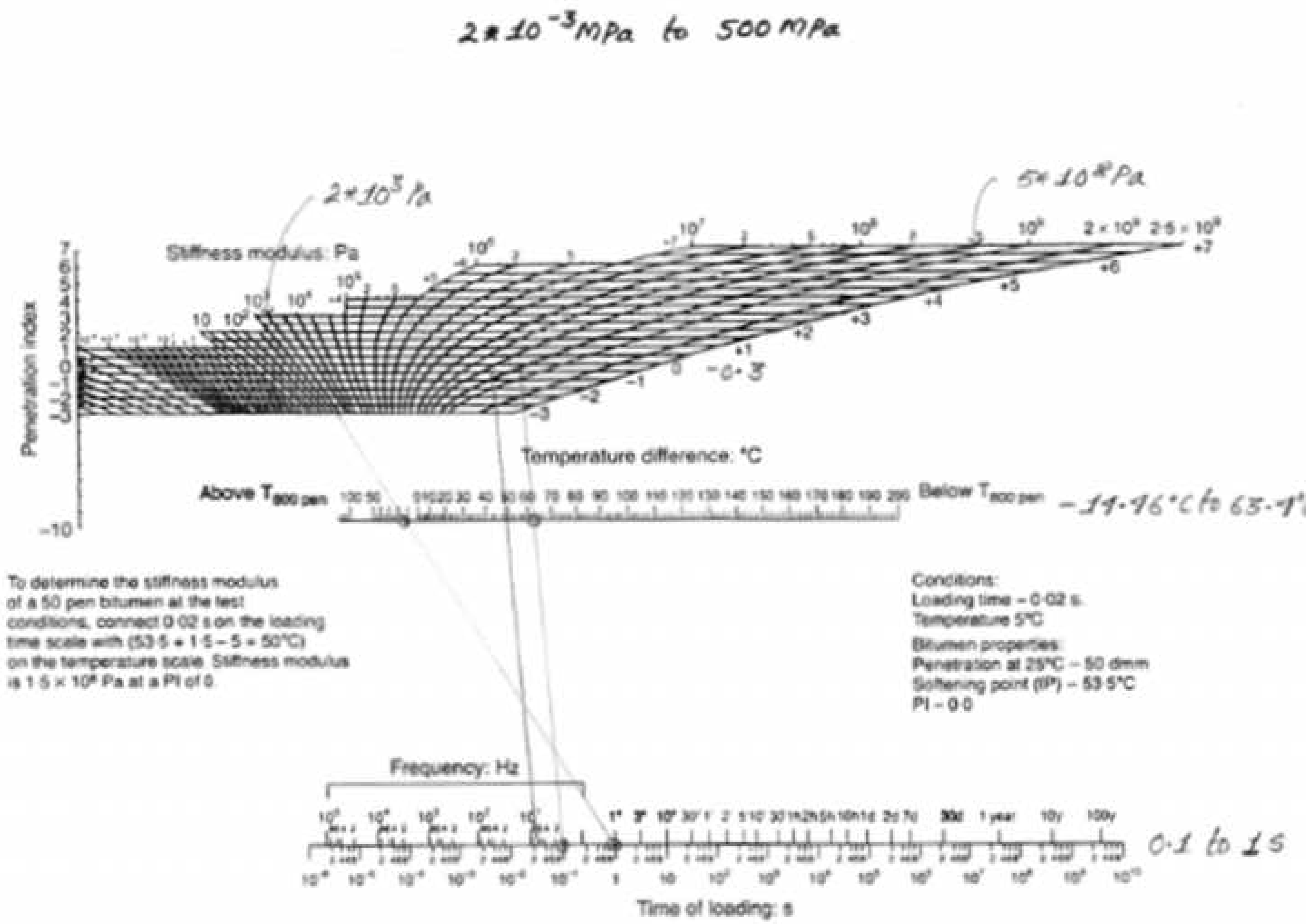


To determine the stiffness modulus of a 50 pen bitumen at the test conditions, connect 0·02 s on the loading time scale with (53·5 + 1·5 − 5 = 50°C) on the temperature scale. Stiffness modulus is 1·5 × 10⁸ Pa at a PI of 0.

Conditions:
Loading time – 0·02 s.
Temperature 5°C

Bitumen properties:
Penetration at 25°C – 50 dmm
Softening point (IP) – 53·5°C
PI – 0·0

Figure 22 Stiffness of Bitumen for Loading Time 0.01s to 0.1s

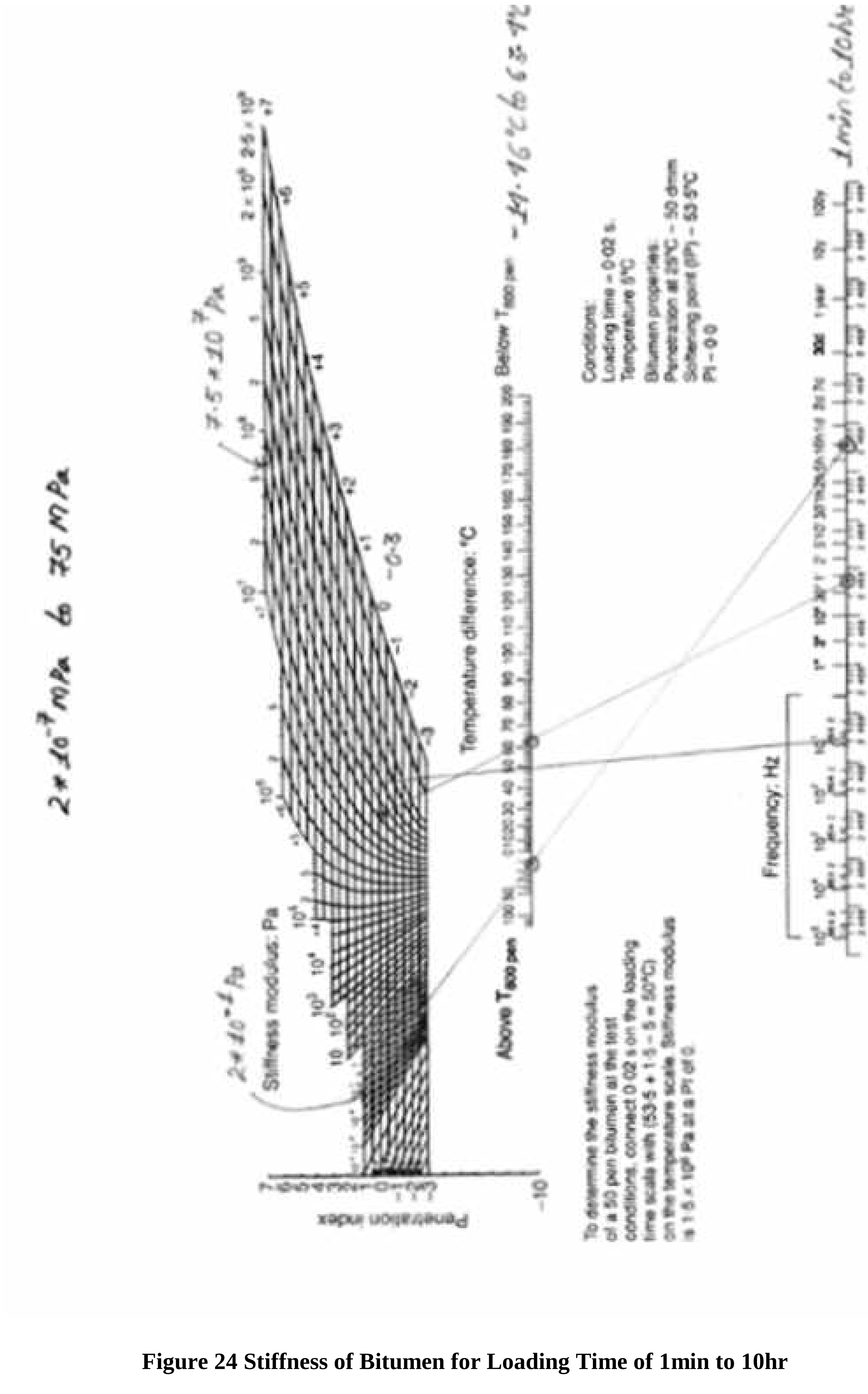


To determine the stiffness modulus of a 50 pen bitumen at the test conditions, connect 0·02 s on the loading time scale with (53·5 + 1·5 − 5 = 50°C) on the temperature scale. Stiffness modulus is 1·5 × 10⁸ Pa at a PI of 0.

Conditions:
Loading time – 0·02 s.
Temperature 5°C

Bitumen properties:
Penetration at 25°C – 50 dmm
Softening point (IP) – 53·5°C
PI – 0·0

Figure 23 Stiffness of Bitumen for Loading Time of 0.1s to 1s



To determine the stiffness modulus of a 50 pen bitumen at the test conditions, connect 0·02 s on the loading time scale with (53·5 + 1·5 − 5 = 50°C) on the temperature scale. Stiffness modulus is 1·5 × 10^8 Pa at a PI of 0.

Conditions:
Loading time – 0·02 s.
Temperature 5°C
Bitumen properties:
Penetration at 25°C – 50 dmm
Softening point (SP) – 53·5°C
PI – 0·0

Figure 24 Stiffness of Bitumen for Loading Time of 1min to 10hr

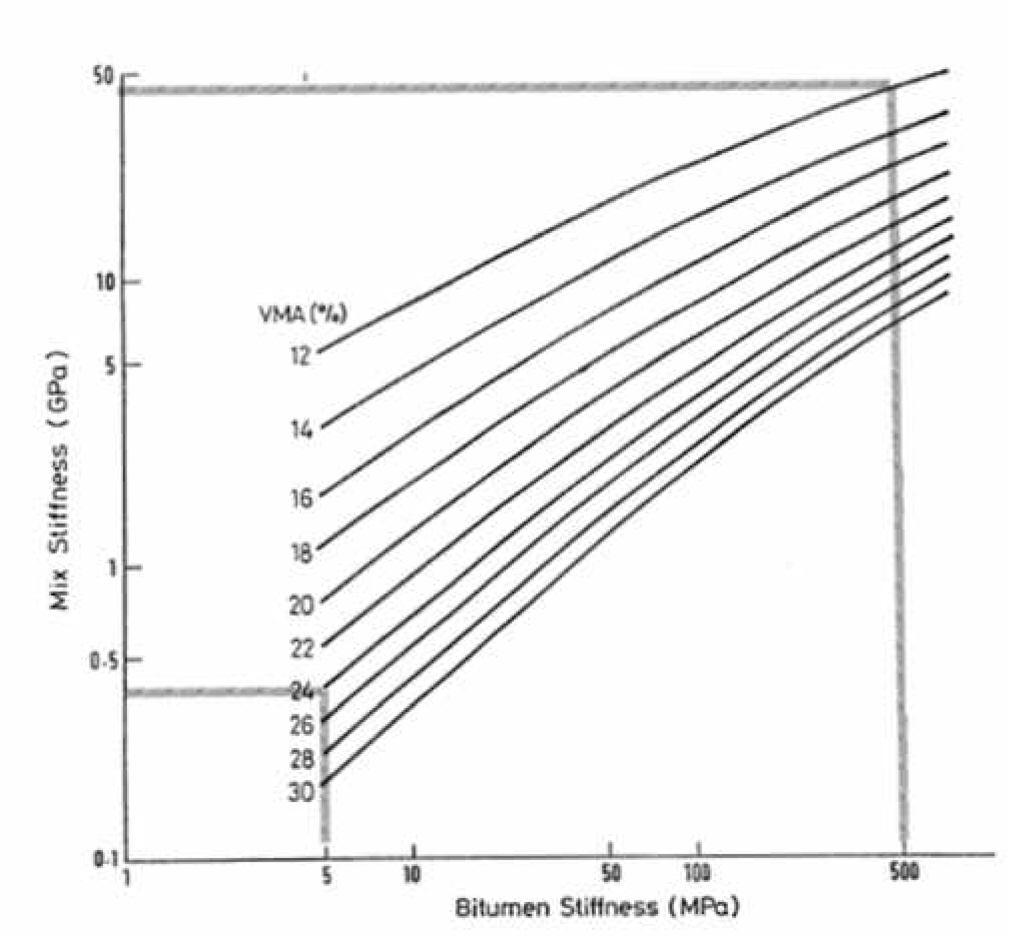


Figure 25The relationship between mix stiffness, binder stiffness and VMA in the elastic region

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusions

The pavement temperature was recorded in Terai/Plains to Hills and extrapolated for cold regions with pavement temperatures approximately 10°C above average air temperatures for average annual air temperature of 11.4°C and 22.4°C (16) in Nepal in comparison to pavement temperatures approximately 3°C above average air temperatures for average annual air temperature between 8.5°C and 10°C in the UK, indicating temperatures to be considered for site specific conditions for mix design. The softening point was assessed from available secondary data from mentioned supplier. With softening point, temperature, loading time (categorized) and penetration index (empirical relation) the stiffness of bitumen was assessed. The penetration index was found to be -0.3 which shows that relatively soft bitumen is being used.

Range for VMA was chosen as per Marshall Specification validating it from lab data on aggregates and bitumen used in Nepalese context. Finally, with Bitumen Stiffness and VMA the Mix Stiffness was determined which was found to be 0.37GPa at high temperature and creep speeds to about 50 GPa at low temperatures and high speeds in comparison to 1 GPa to 15GPa respectively under those conditions in general practice (20)

Thus, the process for mix design was identified in context of Nepal.

5.2. Recommendations

The mix design should incorporate the local pavement temperature and overall loading conditions (gradient and loading time) to make it site specific and practical (16) while the bitumen should be chosen depending upon the requirement criteria (penetration, softening point and viscosity) at the site. For example in Terai/Plains 60/70 and lower penetration grade bitumen is recommended with reasonable softening point.

In addition, the aggregate gradation should be chosen for maximum compaction using Fuller’s curve. However, mix design should be optimized regarding economy and performance.

Also, the design stiffness of pavement layer should be assessed according to loading conditions and mix should be designed conforming to requirements.

Finally, the mix design should also be analyzed for strain criterion assessing the fatigue life of pavement with due consideration of construction period and construction traffic load.

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