EXPERIMENTAL STUDY ON RESILIENT MODULUS OF PROBASE TX-85 STABILIZED SUBGRADE SOIL

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CHAPTER 1

INTRODUCTION

1.1 Background

Unpaved roads are usually used for low volume traffic and serve as access roads. Being basically an agricultural country low volume roads play a very important role in the rural economy and resource industries. When unpaved roads are built on soft foundation soils, large deformations can occur, which increase maintenance cost and lead to interruption of traffic service. The use of soil stabilizer products as an additive in flexible pavements for reinforcement has been demonstrated to be a viable technology through studies conducted over the last three decades which results in increased service life of the pavement or reduced base thickness to carry the same number of load repetitions. Benefits of reducing base course thickness are realized if the cost of the geosynthetic is less than the cost of the reduced base course material. In developing countries like India cost and availability of geosynthetics are the major constraining factors for the construction of reinforced soil structures (Nazarian and Feliberti, 1993).

Geiman 2005 reported subgrade quality has a dramatic impact on both the initial cost of pavements and on the subsequent maintenance costs. Options for dealing with soft pavement subgrades include attempting to dry and compact the subgrade, reinforcing the subgrade with a geosynthetic material, applying a chemical stabilizer such as lime, cement, polymer, and designing a very thick and expensive pavement section. Traditional lime and cement treatment can be very effective, but many contractors are hesitant to use lime and cement due to issues with dust control and other handling problems

Hydrated lime and pelletized lime offer alternatives that help reduce the handling issues, but they do not completely eliminate them. Many other non-traditional amendments, including resins and polymers, are marketed, but their performance record is mixed and solid engineering data is lacking, preventing reliable design (Geiman, 2005). The progressive downward movement of soil into the subgrade and the associated upward squeezing or pumping of subgrade soil into the subgrade base results in intermixing as shown in Figure 1.1.

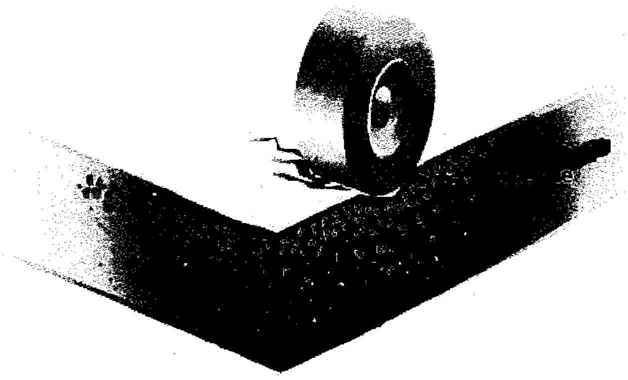


Figure 1.1: Soil contaminate resulting failure of the road without stabilizer

Characterizing subgrade soils in terms of resilient modulus (M_R) is essential for pavement design of both flexible and rigid pavements. M_R attribute has been recognized widely for characterizing materials in pavement design and evaluation. Resilient modulus is a measure or estimate of the elastic modulus of the material at a given stress or temperature. Mathematically it is expressed as the ratio of applied deviator stress to recoverable strain (George, 2004).

$$M_R = \sigma_d / \varepsilon_r \tag{1.1}$$

where, σ_d = Applied deviator stress

ε_r = Resilient strain.

M_R is generally estimated directly in the laboratory using repeated load triaxial testing, indirectly through correlation with other standard tests, or by back calculating from deflection tests results. For a new design, M_R is generally obtained by conducting repeated load triaxial tests on reconstituted/undisturbed samples.

The 1993 AASHTO Guide for Design of Pavement Structures: Appendix L (3), lists four different approaches to determine a design resilient modulus value. The first approach is laboratory testing, another approach is by Non-Destructive Testing (NDT) backcalculation, the third approach consists of estimating resilient modulus from correlations with other properties, and the last is from original design and construction data. According to George 2004, most of the agencies do not routinely measure the M_R in the laboratory, but estimate from experience or from other material or soil properties; i.e., CBR, R-value or physical properties.

Previous research has concentrated on refining the resilient modulus test and reducing its variability, or was directed toward modeling the resilient behavior of pavement materials, especially unbound materials (Nazarian and Feliberti, 1993)

The relative difficulty with which the resilient modulus test is conducted for unbound pavement materials is one of the reasons behind this research. Another reason was the variable nature of these materials whether it is the base, subbase or subgrade materials. By modeling the resilient modulus of unbound pavement materials, equations can be obtained that relate some or all of the resilient parameters to other properties of the material that can easily be tested and quantified (Ping, et al.,1998).

This research addresses these deficiencies by performing laboratory tests on subgrade soils using several different amendments at varying dose rates and curing times. The effectiveness of the stabilizers is then compared with the effectiveness of other form type of stabilizers, whose reactions are better understood and documented.

1.2 Problem Statement

Clays generally exhibit undesirable engineering properties compared with those of granular soils. They tend to have lower shear strength and to lose shear strength further upon wetting or other physical disturbances. They can be plastic and compressible and they expand when wetted and shrink when dried. The problem such as high compressibility and low shear strength usually occur and it is common in the construction of embankments. In construction of unpaved roads, the stress is always acting in the direction of vertical and horizontal, and effective stress is depend on stability condition during and after construction. Common behavior of foundation soil under the unpaved road is like settlement, lateral movement, pore water pressure and total stress. All of these behaviors are related to each other.

Unpaved roads and other constructions on deposits of natural soft soil is still a challenge in geotechnical engineering. Construction on soft soils becomes even more important as urban areas all over the world become more and more congested, and thus research on this soil is being conducted using Probase TX-85 soil stabilizer. The using of soil stabilizer such as Probase TX 85 reduced the plasticity index and improved bearing capacity. It also prevents overall failure of the embankment and soft foundation soil. So, by laboratory investigation conducted on stabilized subgrade soil, new finding will be developed inparticular, the resilient modulus of unpaved roads.

1.3 Aims and Objective

The purpose of this study is to determine the resilient of Probase TX-85 stabilized subgrade soil. Hence, the following objectives were persued in this study :

1. To investigate the engineering properties of subgrade soil used in this study

2. To determine the optimum TX-85 content as a soil stabilizer based on unconfined compressive strength.

3. To analyze and compare the subgrade soil strength and Resilient Modulus of unpaved roads with and without stabilizer

1.4 Scope and Limitation

The primary purpose of this research focused on the TX-85 effect of the subgrade soil that have caused problems during construction or resulted in poor performance in service. The selected stabilizers are Probase TX-85 (powder), Probase TX-85 (liquid) and Probase TX-85 combination (powder and liquid).The scope of this research includes:

- Characterizing the soils by performing the following tests: specific gravity, particle size distribution, Atterberg limits and moisture-density relationship using standard and modified Proctor effort.

- Reviewing literature pertaining to standardized laboratory procedures for preparing mixtures using Probase TX-85 stabilizers, as well as other procedures for mixtures involving nontraditional stabilizers that have been studied previously by other researchers.

- Developing a laboratory mixture preparation and testing procedure that can be used to evaluate and compare subgrade soil with and without stabilizers.

- Identifying the existence and significance of trends among base soil characteristics, amendment type, amendment dose rate, and strength characteristics using the laboratory procedure developed.

Specimens prepared near the optimum water content give an indication of how well the amendments can strengthen and stiffen a subgrade in order to help reduce the required thickness of the pavement section. Specimens prepared substantially above the optimum water content give an indication of whether the workability of the soil can be improved such that the soils can be compacted to an adequate strength and stiffness without extensive drying and/or processing.

Laboratory works has been carried out at the Research Centre of Soft Soil Engineering (RECESS), Universiti Tun Hussein Onn Malaysia (UTHM), Batu Pahat Johor under Short Term Grant phase 3/2011: Vote 0913.This research is lead by Mohammad Nasir bin Mohamad Taher and funding by ORICC UTHM. This research also collaborates between UTHM and Probase Manufacturing Sdn Bhd.

1.5 Significants of study

This research provides insight into which stabilizers are most effective for stabilizing subgrade soils commonly encountered in Parit Raja. This report is not meant to replace laboratory testing on specific projects; however, it can be used as a guide to help select an appropriate stabilizer type and amount based on soil properties and desired strength. In addition, the laboratory procedure developed for this research can be used to help evaluate specific soils for specific projects.

This study was undertaken to provide an understanding of the options for soil supply in road construction particular subgrade layer. This research fulfils the need of the person who involves on road design and construction. It also hopes to give an alternative idea to highway engineer and contractors in respect of stabilization on road subgrade. This research will provide the alternative material in unpaved road structure which will then eventually increase bearing capacity and also decrease the plasticity index.

There are many types of correlation equations have been developed in predicting resilient modulus from soil physical properties. Since several equations are available from past studies, there is a need to substantiate the predictability of these equations. Those equations, if proved to be valid, could serve a vital role in proposing a preliminary pavement design for budgeting purposes. Final design can await completion of the grading contract, followed by additional in-situ tests.

CHAPTER 2

LITERATURE RIVEW

2.1 Introduction

The objectives of this literature review are to identity the research of previously published on the unpaved road system on soft subgrade, type of soil stabilization and behavior of soft soil. The primary focuses are on determination of the resilient modulus from soil index properties and followed by review of previous research on similar work.

2.2 Soft Soil

2.2.1 Soil Classification

A soil consists of collection of separate particles of various shapes and sizes. The particle size analysis is to group these particles into separates ranges of sizes and so, determine the relative proportions, by dry mass of each size range. Soils may be separated into three very broad categories: cohesionless, cohesive and organic soils. In the case of cohesionless soils, the soil particles do not tend to stick together (Liu and Evett, 2004). On the other hand, cohesive soil particles do tend to stick together and it is categorized by very small particle size where the main element is due to effects of surface chemical. Organic soils are typically spongy, crumbly and compressible. They are undesirable for use in supporting structures. Based on simple definition, soils can be divided into component with particle size is usually given in terms of the equivalent particle diameter (Head, 1992):

i. Gravel – particles from 60 mm to 2 mm

ii. Sand - particles from 2 mm to 0.06 mm

iii. Silt - particles from 0.06 mm to 0.002 mm

iv. Clay – particles (clay mineral) smaller than 0.002 mm

v. Fines are particles which pass a 63 μm sieve

vi.Clay Fraction is the percentage of particles smaller than 2μm, as determined by a standard sedimentation procedure

2.2.2 Characteristic of Clay Soil

Particles forming clay consist of complex minerals which are mostly flat and plate-like or elongated, and of a size less than 0.002 mm. The most significant properties of clays are its plasticity and cohesion. Clay soils able to take and retain a new shape when compressed or moulded (Whitlow, 1995). The size and nature of the clay mineral particles, together with the nature of the adsorbed layer, controls this property. Where the average specific surface is high, this plasticity may be extremely high and the soil extremely compressible.

Cohesive soils generally exhibit undesirable engineering properties compared with those of granular soils. They tend to have lower shear strength further upon wetting or other physical disturbances. They can be plastic and compressible and they expand when wetted and shrink when dried. Clay soils can creep (deform plastically) over time under constant load, especially when the shear stress is approaching it shear strength, making them prone to landslides.

Being impervious, however, they make better core materials for earthen dams and dikes. With low permeability, cohesive soils compress much more slowly because of the expulsion of water from the small soil pores is so slow (Whitlow, 1995). Hence, the ultimate volume decrease of the cohesive soil and associated settlement of a structure built on this soil may not occur until sometime after the structure is loaded.

2.2.3 Problem of Clay Soil

Saturated cohesive soil can be susceptible to a large amount of settlement from structural loads. It is usually the direct weight of the structure that causes settlement of the cohesive soil. However, secondary influences such as the lowering of the groundwater table can also lead to settlement of cohesive soils. The soil parameters normally employed and characterized in soft soil problems are:

i. Classification and Index Properties, and Natural Moisture Content
ii.Undrained Shear Strength (S_u)
iii. Pre-Consolidation Pressure (P_c)
Iv.Compression Index (C_c) and the Coefficient of Volume Change (m_v)
v. Coefficient of Consolidation (c_v)

The parameters are very important in analyzing the behavior of this soil so that it can carry extra loads subjected to the soils. These nature creatures are widely found in Malaysia along the coastal plains area and with the increasing economic development over the soil; studies were carried out to determine the typical values of the soils that can contribute to the failure of the soil structure.

2.3 Stabilized Subgrade Road System

2.3.1 Introduction

The main objective of stabilization is to improve the performance of a material by increasing its strength, stiffness and durability. The performance should be at least equal to, if not better than that of a good quality natural material. The system is specially recommended for developing countries where their roads, be it rural or urban are mostly unpaved. The term 'stabilization' is the process whereby the natural strength and durability of a soil or granular material is increased by the addition of a stabilizing agent.

In addition, it may provide a greater resistance to the ingress of water. There are many types of stabilizer that can be used, each with their own advantages and disadvantages. The strength of a stabilised material will often continue to increase for a period ofseveral years from the time it is constructed, as shown in Figure 2.1 (Croney, 1998).

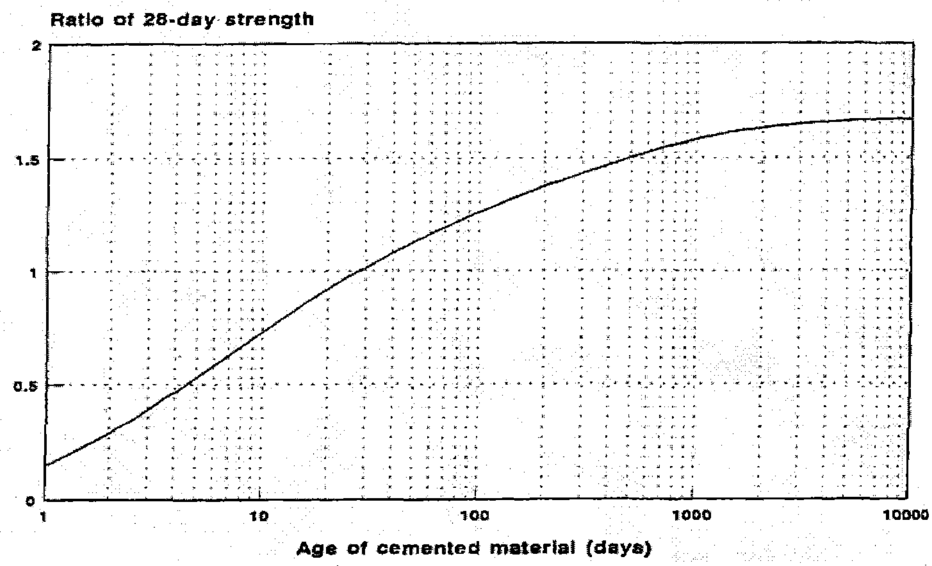


Figure 2.1: Rate of increase of strength with age for cemented material (Croney,1998)

The type and quantity of stabilizer added depends mainly on the strength and performance that needs to be achieved. The addition of even small amounts of stabilizer, for example up to 2 per cent cement, can modify the properties of a material. Larger amounts of stabilizer will cause a large change in the properties of that material, for example 5 to 10 per cent of cement added to clean gravel will cause it to behave more like a concrete. The strength of a stabilized material will depend on many factors. These include:

- the chemical composition of the material to be stabilised;
- the stabiliser content;
- the degree of compaction achieved;
- the moisture content;
- the success of mixing the material with the stabiliser;
- subsequent external environmental effects.

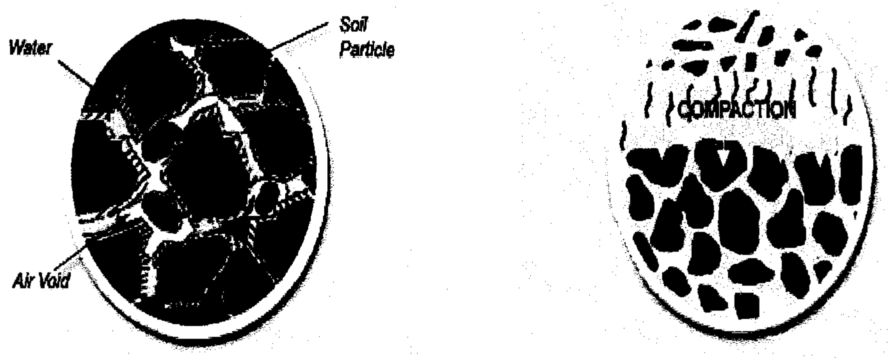
When small quantities of stabiliser are added, the material is often described as 'modified' rather than 'bound'. There are no fixed criteria for these definitions, but a limit of 80kPa (indirect tension) or 800kPa (Unconfined Compressive Strength after 7days moist curing) for a reasonably graded material is suggested by NAASRA (1986).

2.4 Types of Stabilisation

Stabilization is the process of mixing a stabilizer, for example cement, with a soil or imported aggregate to produce a material whose strength is greater than that of the original unbound material. The use of stabilization to improve the properties of a material is becoming more widespread due to the increased strength and load spreading ability that these materials can offer. Stabilization technology is extremely relevant for heavily trafficked pavements where its benefits are beginning to be appreciated (Little, 1995). This study describes the basic types of stabilization, indicates when it should be used, and discusses the main advantages and disadvantages of its use

.

Figure 2.2 shows normal soil densities are loose with air void content that easily allows water penetration. Soil particle with high Plastic Index (PI) will absorbe water. Water act as lubricant cause the soil road to be slippery. When vehicle passing through, soil particle easily move around with the present of water, eventually created pot hole and depression damage especially during rainy season.



Normal Soil Soil treatment Process

Figure 2.2: Comparison normal soil with the treated soil (Little, 1995)

2.4.1 Mechanical Stabilisation

The most basic form of mechanical stabilisation is compaction, which increases the performance of a natural material. Mechanical stabilisation of a material is usually achieved by adding a different material in order to improve the grading or decrease the plasticity of the original material. The physical properties of the original material will be changed, but no chemical reaction is involved. For example, a material rich in fines could be added to a material deficient in fines in order to produce a material nearer to an ideal particle size distribution curve (Metcalf, 1977).

This will allow the level of density achieved by compaction to be increased and hence improve the stability of the material under traffic. The proportion of material added is usually from 10 to 50 per cent. Providing suitable materials are found in the vicinity, mechanical stabilisation is usually the most cost-effective process for improving poorly-graded materials. This process is usually used to increase the strength of a poorly-graded granular material up to that of a well-graded granular material. The stiffness and strength will generally be lower than that achieved by chemical stabilisation and would often be insufficient for heavily trafficked pavements (Metcalf, 1977). It may also be necessary to add a stabilising agent to improve the final properties of the mixed material.

2.4.2 Cement Stabilisation

The addition of cement to a material, in the presence of moisture, produces hydrated calcium aluminate and silicate gels, which crystallise and bond the material particles together. Most of the strength of a cement-stabilised material comes from the physical strength of the matrix of hydrated cement. A chemical reaction also takes place between the material and lime, which is released as the cement hydrates, leading to afurther increase in strength. Cement stabilised materials can be mixed in-situ or mixed at a plant and transported to site. To achieve stronger cement bound materials, i.e. greater than about 10 MPa cube strength at 7 days, the materials should generally be plant mixed (Detr, 1998).

One of the main problems with stabilising a material is mixing in the cement. The particle size of ordinary Portland cement is quite well defined with a range of 0.5-100 microns and a mean of 20 microns (Ingles & Metcalf, 1972). The larger particles of cement never completely hydrate, and it has been found that the same amount of a more finely ground cement will produce higher strengths. Finely ground cements are, however, expensive to produce and it has been suggested (Ingles & Metcalf, 1972) that the larger particles of cement could be replaced with smaller particles of inert filler.The greater bulk would aid the distribution process so that the same amount of active cement would be available throughout the material. Thus producing an equally effective binder, this could be cheaper than ordinary cement.

The use of cement as a stabiliser is more widespread than lime. This is due to many reasons, but the main factors are likely to be the cost and the higher strengths that are attainable using cement. Other factors include availability, past experience and the more hazardous nature of lime. The price of cement is often similar to that of quicklime or hydrated lime, however cement can be used on a wider range of materials and the strengthening effect of cement is much more than that of an equal amount of lime. Hence either higher strengths are possible using an equal amount of cement instead of lime or the same specified strength can be achieved using a lower quantity of cement than lime (Sherwood, 1993).

2.4.3 Lime Stabilisation

The stabilisation of subgrade materials is not new; with examples of lime stabilisation being recorded in the construction of early Roman roads. However, the invention of Portland cement in the 19th Century resulted in cement replacing lime as the main type of stabiliser.

Lime stabilisation will only be effective with materials which contain enough clay for apositive reaction to take place.Attempts to use lime as a general binder in the same way as cement will not be successful (Watson, 1994).Lime is produced from chalk or limestone by heating and combining with water. The term 'lime' is broad and covers the following three main types:

i) quicklime -calcium oxide (CaO),

ii) slaked or hydrated lime - calcium hydroxide ($Ca(OH)_2$) and

iii) carbonate of lime - calcium carbonate ($CaCO_3$).

Only quicklime and hydrated lime are used as stabilisers in road construction. They are usually added in solid form but can also be mixed with water and applied as a slurry. It must be noted that there is a violent reaction between quicklime and water and consequently operatives exposed to quicklime can experience severe external and internal burns, as well as blinding.

Hydrated lime is used extensively for the stabilisation of soil, especially soil with a high clay content where its main advantage is in raising the plastic limit of the clayey soil. Very rapid stabilisation of water-logged sites has been achieved with the use of quicklime. Small quantities (typically 1-3 %) are used to reduce the plasticity of the clay. It is reported that such small quantities usually result in a small increase in CBR strength although no significant increase in compressive or tensile strength should be expected (Paige-Green,1998). Paige-Green reports that typically, a minimum of 3 to 5 per cent stabiliser is necessary to gain a significant increase in the compressive and tensile strength.

2.4.4 Asphalts

The use of cut-back liquid asphalts to surface-treat gravel roads was once popular for dust control. However, because of the great amount of fuel oil or kerosene in these products, they have been banned in many places. Some emulsified asphalts may work for this purpose, but their use is very limited. The product must be applied with special asphalt application equipment. Bitumen and tar are too viscous to use at ambient temperatures and must be made into either a cut-back bitumen (a solution of bitumen in kerosene or diesel), or a bitumen emulsion (bitumen particles suspended in water). When the solvent evaporates or the emulsion 'breaks', the bitumen is deposited on the material.

According to O'Flaherty, 1985, the bitumen merely acts as a glue to stick the material particles together and prevent the ingress of water. In many cases, the bituminous material acts as an impervious layer in the pavement, preventing the rise of capillary moisture.In a country where bitumen is relatively expensive compared to cement and where most expertise is in cement construction, it appears more reasonable to use a cement stabiliser rather than a bitumen/tar based product .

2.4.5 Soybean Oil

This product is known technically as Acidulated Soybean Oil Soap stock. It is a by-product of the caustic refining process of soybean oil. It is a biodegradable material that has many of the characteristics of light petroleum based oil. It will penetrate a gravel surface and provide a light bonding of the gravel that effectively reduces dust when it is used properly (Paige-Green,1998).

2.5 TX-85 Double Strength Liquid Soil Stabilizer

In this study, TX-85 double strength liquid and powder soil stabilizer have been used. It is 100% organic and derived from combined organic sulphur and buffered acids that are combined as bi-sulphates.It also water-soluble soil stabilizer chemical used in construction of unpaved roads.

Figure 2.3 shows that TX-85 is non toxic and poses no threat to groundwater supplies or flora and fauna. It also reduces the plastic index of the soil and improves its CBR ratings. These stabilizer is an economical construction methods especially for rural and estate roads and does not consume in its function but continues and perpetuates its action in the presence of water

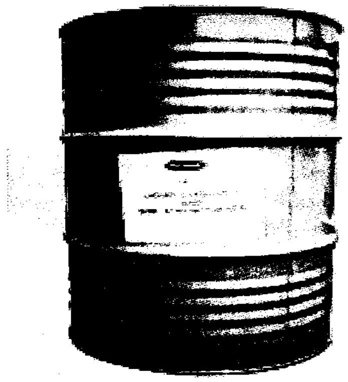


Figure 2.3: Probase TX85 Soil Stabilizer

2.6 TX-33 Liquid Sealer and Dust Control

TX-33 provides extra strength and protection on the treated road. This surface can also act as wearing surface for the road. It also prevents water penetration, erosion and provided a dust free road. Figure 2.4 shows the cross section for TX-85 treated soil consist a layer of quarry dust, TX-33 protective layer and TX-85 treated soil base.

2.7 Benefits of Stabilization

Once a road is stabilized there are several benefits. On high volume roads, these benefits can make stabilization very cost effective. It may be hard to justify the use of any of these products for dust control alone. However, when the products are working well, the added benefit of a stabilized surface that controls the loss of fines through dusting is a great economic benefit. When the fines are lost from a gravel surface, the stone and sand-sized particles that remain will tend to remain loose on the surface, leading to some distresses like washboarding and reduced skid resistance (Mathew, 2003).

A road surface that remains tightly bound and stable will require much less blade maintenance. The manufacturers of some dust control products highly recommend that the surface should not be bladed at all after their products are applied. While extra blading, shaping and mixing is needed to prepare a road for dust control, the overall need for blade maintenance should be greatly reduced.

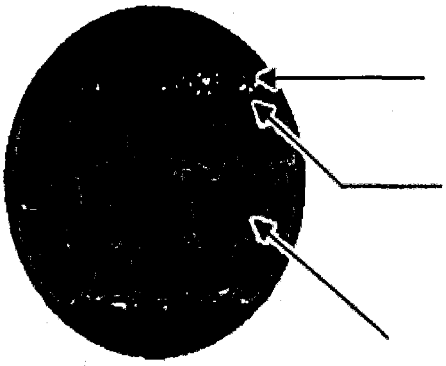


Figure 2.4: Cross section for TX-85 treated soil

2.8 Resilient Modulus (M_R)

Since the pavement materials are subjected to a series of distinct load pulses, a laboratory test duplicating this condition is desirable. The repeated load type test has been used for many years to simulate vehicle loading. In this test, cylindrical specimens of soil are subjected to a series of load pulses applied with a distinct rest period, simulating the stresses caused by multiple wheels moving over the pavement. A constant all-around confining pressure applied on the specimen simulates the lateral stresses caused by the overburden pressure and applied wheel load. The total resilient (recoverable) axial deformation response of the specimen to the stress pulses measured is used to calculate the resilient modulus of the material. Cited below are two reasons favoring the use of repeated load triaxial test for determination of M_R .

In order to incorporate the M_R into mechanistic pavement design method, some empirical correlations based on the California Bearing Ratio (CBR) are still used over the world. These approaches are limited because they describe only a "linear elastic" behavior non-stress dependent. However, laboratory and field evidence show that M_R is stress dependent. Thus, the use of a "non-linear elastic" hypothesis could be more accurate to describe the variation of the M_R with the applied stress state. Some model have been developed describing this type of behavior to be used in computational pavement design method and they are based on M_R result obtain in the laboratory from the repeated triaxial test (Angelone and Martinez, 2000).

Axial, radial, and volumetric strains can all be measured in the triaxial test. For about the last 35 years the repeated load triaxial compression test has been the basic test procedure to evaluate resilient modulus of cohesive and granular materials for pavement design applications.

Figure 2.5 shows the straining of a specimen under a repeated load test. At the initial stage of load applications, there are considerable permanent deformations, as indicated by the plastic strain in the figure. As the number of repetition increase, the plastic strain due to each load repetition decreases. After 100 to 200 repetition, the strain is practically all recoverable as indicated by $\square_r$ in the figure (Huang.Y.H,2004).

The procedure for determining the resilient modulus of aggregate materials is specified by AASHTO Designation TP 46-94. Meanwhile, Figure 2.6 shows the applying stresses to a triaxial specimen and relationship between resilient modulus and bulk stress for stabilized fine grained material as shown in Figure 2.7.

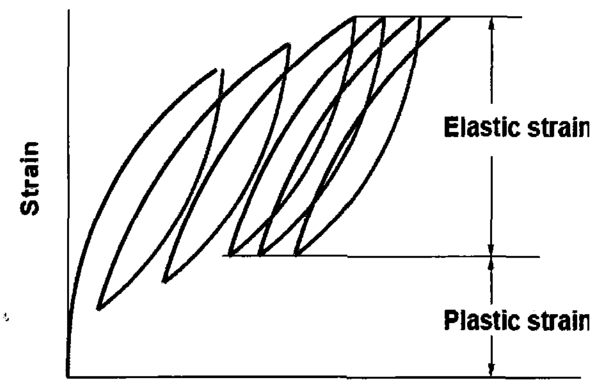


Figure 2.5: Recoverable strains under repeated loads (Mathew, 2003)

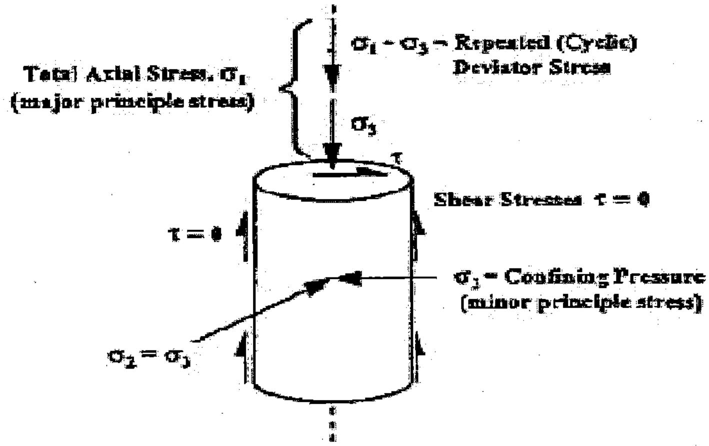


Figure 2.6: Stresses Applied to a Triaxial Specimen (Mathew, 2003)

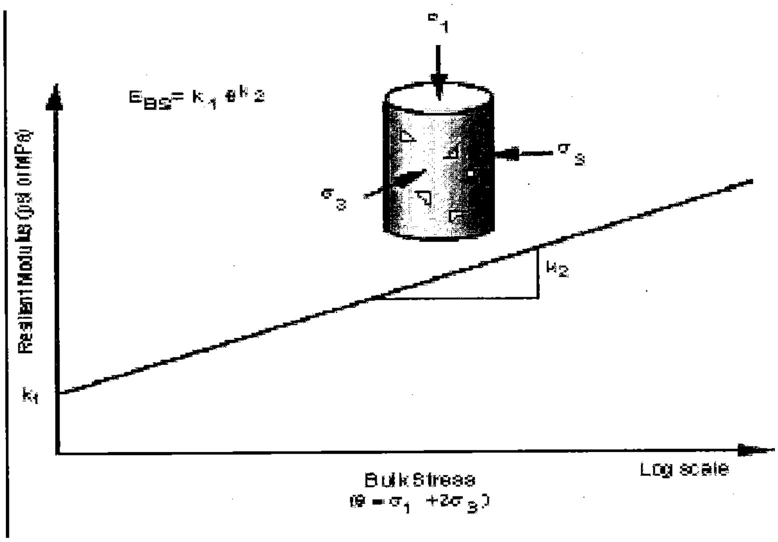


Figure 2.7: Resilient Modulus versus Bulk Stress for Stabilized Fine Grained Materials (Mathew, 2003).

2.9 Resilient Modulus Models

The resilient modulus M_R is a dynamic test response defined as the ratio of the repeated axial deviator stress d to the recoverable axial strain □a.

$$M_R = \frac{\sigma_d}{\square_a}$$

It is known that the resilient modulus is a stress dependent parameter. Research has been directed toward developing a reliable M_R testing system, modeling M_R as a function of stresses and/or strains and development of relationships between M_R or M_R model constants and soil properties. Several models relating M_R to stress state have been suggested (Pezo, 1993; Thompson, 1989; Mohammad et al., 1994; and Moassazadeh and Witczak, 1981) for cohesive and cohesionless materials. A relationship that has gained wide acceptance for fine-grained soil is of the form (Moassazadeh and Witczak, 1981):

$$M_R = K_1 (\sigma_d)^{K_2} \tag{2.1}$$

Where: M_R = resilient modulus

σ_d = sum of the principal stresses

K_1 and K_2 = experimental coefficient.

This model reflects the dependency of M_R on deviator stress. In another model, reported by Yoder and Witczak (1975), M_R values also depends on deviator stress, however, M_R decreases as deviator stress increases up to a point and then starts to increase. The model is of the following form:

$$M_R = K_2 + K_3\ (k1 - \sigma_d) \qquad \text{for } K_1 > \sigma d \qquad (2.2)$$

and

$$M_R = K_2 + K_4\ (\sigma_d - K_1) \qquad \text{for } K_1 < \sigma d \qquad (2.3)$$

Where: . K_2 = the value of MR at the point $\sigma d = K_1$ and K_3

K_4 = slopes of the portions of the curve representing M_R.

σ_d = relationship when M_R is less than and more than K_1 sum of the principal stresses

For granular materials, the most significant parameter that influences M_R is the confining stress. M_R is usually modeled in terms of either the confining stress ($\sigma 3$) or the bulk stress (□) (Yoder and Witczak, 1975) as follows:

$$M_R = K_1\ (\sigma_3)^{K_2} \qquad (2.4)$$

or

$$M_R = K1\ (\square)^{K_2} \qquad (2.5)$$

Where: $\square = \sigma_1 + \sigma_2 + \sigma_3 = \sigma_1 + 2\sigma_3 = \sigma_d + 3\sigma_3$

Historically, due to difficulties associated with resilient modulus testing, various approximate empirical formulae have been suggested to estimate MR from CBR by many researchers (Powell et al., 1984; Witczak et al., 1995). However, these types of models have not been universally accepted as on one hand, they were developed for certain localities and cannot be generalized, and on the other hand they do not recognize the stress dependence of MR (Rada and Witczak, 1981; Drumm et al., 1990).

Pezo (1993) proposed a stress dependent model that incorporates both deviatoric and confining stresses. The model can be used for both cohesive and cohesionless soils. The soil type is reflected by the weight given to either deviator or confining stresses in the model which is developed by regression analysis. The model is of the following form:

$$M_R = K_1(\sigma_d)^{K_2}\ (\sigma_3)^{K_3} \qquad (2.6)$$

Where : K1, K2 and K3 are regression constants.

σ_d= Standard deviator stress

σ_3= Confining Stress

This type of model was found to fit M_R data very well as demonstrated by an extensive study that included almost all types of soils (Al-Suhaibani et al., 1997).

2.10 Factors Affecting Resilient Modulus Test

The resilient modulus of fine-grain soils is not a constant stiffness property but depends upon various factors like load state or stress state, which includes the deviator and confining stress, soil type and its structure, which primarily depends on compaction method and compaction effort of a new subgrade. Previous studies show that deviator stress is more significant than confining stress for fine-grain soils. Resilient modulus is found to increase with a decrease in moisture content and an increase in density, and decrease with an increase in deviator stress.

For coarse-grain soils, M_R is primarily influenced by the stress state, degree of saturation and compactive effort (density). Lekarp, 2000 found that M_R increases with increasing confining stress. Studies have also indicated that there is a critical degree of saturation near 80-85 percent, above which granular material becomes unstable and undergoes degradation rapidly under repeated loading. Lekarp et al. noted, and other researchers concur that M_R of granular materials increases with increasing confining stress and sum of principal stresses, otherwise known as bulk stress (θ), and slightly increases with deviator stress.

For the road pavement design, it is important to consider how the resilient behavior varies with changes in different influencing factor. The resilient behavior of unbound granular material is affected by several factors such as

i. effect of stress
ii. effect of density
iii. effect of grading
iv. effect of moisture content
v. effect of number of load application

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