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16. Abstract

The Texas Department of Transportation uses stabilized subgrades and bases extensively. In fact subgrade stabilization is almost routine in many districts and especially in those with clay subgrades. A pressing need exists to determine the effectiveness of stabilization of subgrades and base courses, to evaluate the current mixtures and thickness design approaches and to suggest realistic structural properties associated with these stabilized pavement layers.

Report 1287-3F considers both base course and subgrade stabilization. Stabilized bases are divided into three categories: heavily stabilized, moderately stabilized and lightly stabilized depending on the amount of stabilizer used. Heavily stabilized bases perform as rigid structural layers. This report suggests modifications to currently used TxDOT mixture design and thickness design approaches to minimize structural damage within the stabilized base layer due to both non-load associated cracking and load associated fatigue cracking. Moderate and light levels of base stabilization significantly improve the structural contribution of the layer without, in most cases, producing a rigid structural layer. This type of stabilization is advantageous in many applications. Report 1287-3F suggests appropriate mixture design approaches and thickness design approaches employing current TxDOT testing and analytical tools for moderately and lightly stabilized bases.

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GUIDELINES FOR MIXTURE DESIGN AND THICKNESS DESIGN FOR STABILIZED BASES AND SUBGRADES

by

Dallas N. Little
Research Engineer
Texas Transportation Institute

Tom Scullion
Associate Research Engineer
Texas Transportation Institute

Prakash B.V.S. Kota
Graduate Research Assistant
Texas Transportation Institute

and

Jasim Bhuiyan
Graduate Research Assistant
Texas Transportation Institute

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TEXAS TRANSPORTATION INSTITUTE
The Texas A&M University System
College Station, Texas 77843-3135

IMPLEMENTATION STATEMENT

The research in the Houston District with heavily stabilized bases demonstrates the importance of considering the effect of non-load associated cracking, such as shrinkage cracking, on the fatigue life of the bases. The research suggests using a simple stress ratio fatigue approach adjusted with a load transfer factor across shrinkage cracks of the type monitored in the Houston District. The rate of fatigue cracking is certainly accelerated due to diminished load transfer across shrinkage and reflection cracks. The computer program developed in project 1287 and presented in reports 1287-2F and 1287-3F is a reasonable approach in evaluating the life of heavily stabilized bases. This program should be evaluated in cooperation with the Houston District for implementation. Researchers on project 1287 should work with the Houston District on future design of heavily stabilized sections.

The research demonstrates that moderately and lightly stabilized bases provide good structural benefits for moderately and low trafficked pavements based on evaluations in the Atlanta, Bryan, Corpus Christi, Houston and Yoakum Districts. The researchers recommend treating these bases as flexible bases with enhanced structural properties due to stabilization. The relatively low level of stabilization does not result in the development of a rigid matrix. The suggested mixture design and pavement design approach using existing testing (Texas Triaxial) and analytical (FPS-19) procedures should be evaluated for implementation with the help and cooperation of the Atlanta, Lufkin and Bryan Districts.

The researches document that well-designed lime-stabilized subgrades have a significant structural benefit due to improved support of the flexible aggregate base course and the hot mix surface and due to the increase in insitu resilient modulus of the stabilized subgrade. The researchers in report 1287-3F recommend resilient moduli of lime-stabilized layers for design and analysis considerations. These recommendations should be considered for implementation. The researchers should work with the Bryan and Ft. Worth Districts in this implementation.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the Texas Department of Transportation (TxDOT), or the Federal Highway Administration (FHWA). This report does not constitute a standard, specification, or regulation nor is it intended for construction, bidding, or permit purposes. The engineer in charge of the project is Dallas N. Little, P.E. #40392.

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SUMMARY

In Texas stabilized bases can effectively fall into three categories: heavily, moderately and lightly stabilized. Heavily stabilized bases normally require six percent or more stabilizer, usually portland cement. These bases perform as rigid layers with very high stiffness based on laboratory and field calculations. The percentage of stabilizer required for these bases is based on a minimum level of unconfined compressive strength. These layers function very well as long as shrinkage cracking and fatigue cracking are held in check. Study 1287-2F suggests that bases can be too rigid, resulting in very low levels of load transfer across shrinkage cracks. This results in accelerated load-associated fatigue cracking and pavement failure. It is important to use no more stabilizer than that required to achieve the minimum required compressive strength. Furthermore, researchers found the severity of non-load associated cracking to be directly related to the compressive strength and the stiffness of the layers evaluated. Researchers developed a computer program which predicts the rate of load associated fatigue damage for heavily stabilized bases. The rate of fatigue is associated with the level of load transfer across shrinkage cracks and reflection cracks within the stabilized layers. Extensive Falling Weight Deflectometer (FWD) testing in the Houston District determined these load transfer factors. Six pavement sections with heavily stabilized bases were monitored, and the effects of seasonal variations were considered.

Researchers monitored moderately and lightly stabilized bases in the Atlanta, Bryan, Corpus Christi and Yoakum Districts. Moderately stabilized bases contain two to four percent stabilizer, and lightly stabilized bases contain less than three percent stabilizer. Moderately and lightly stabilized bases usually offer considerable structural improvement including 70 to 500 percent increases in strength and similar increases in resilient modulus. Moderately and lightly stabilized bases are attractive alternatives for moderate and low traffic areas. However, they may not be suitable for very heavily trafficked pavements. Report 1287-2F suggests mixture and thickness design approaches for moderately and lightly stabilized bases which employ current TxDOT testing procedures and analytical techniques. Moderately and lightly stabilized bases are not treated as rigid bases but as flexible bases with enhanced strength and stiffness.

The researchers evaluated lime-stabilized subgrades in the Houston, Bryan, Lufkin, Atlanta,

Austin, Fort Worth, Corpus Christi and Yoakum Districts. Researchers evaluated subgrades in situ using the Falling Weight Deflectometer (FWD) and the Dynamic Cone Penetrometer (DCP). Both DCP and FWD testing demonstrated significant structural improvement in the majority of the subgrades evaluated. Certain cases were found where little or no structural improvement resulted. The lack of structural improvement is likely due to insufficient lime used in the construction process to assure pozzolanic reaction. The report suggests an improved mixture design approach using the Eades and Grim pH test, following with strength testing to help insure the use of adequate lime for pozzolanic reaction.

Backcalculated resilient moduli of lime-stabilized subgrades demonstrated a significant structural improvement with stiffness increases typically in the order of 5 to 10 over that of the natural, untreated subgrade. These values were verified in situ using the DCP.

This report presents guidelines for mixture design and thickness design for stabilized bases and subgrades.

1. INTRODUCTION

This report concisely presents recommendations for mixture design and thickness design for stabilized bases and subgrades which have been established as a result of research in project 1287. Report 1287-2 documents this research in detail.

This report divides stabilization into four categories: (1) base stabilization where greater than 4 percent stabilizer is used, termed heavily stabilized bases (where the stabilizer is usually portland cement or a combination of fly ash and either lime or cement); (2) bases stabilized with from two 2 to 4 percent stabilizer, termed moderately stabilized bases (where the stabilizer is typically lime - fly ash, portland cement or lime); (3) base stabilization with less than 2 percent stabilizer (usually lime, lime - fly ash or portland cement), termed lightly stabilized bases; and (4) stabilized subgrades.

For each category of stabilization, the report presents a discussion of the following topics: (1) purpose of stabilization technique, (2) mechanisms of failure associated with stabilization technique, (3) steps to reduce risk of failure, (4) mix design considerations, (5) thickness design considerations, and (6) a summary of recommended changes to Texas Department of Transportation procedures and specifications.

2. HEAVILY STABILIZED BASES

2.1 Purpose

Bases are often stabilized in Texas to provide serviceable pavements under heavy traffic. Portland cement is most often selected as the stabilizer as it provides a very substantial improvement in shear strength and a stiffness or modulus increase of approximately 20 to 30 fold over that of the unstabilized material. This strength and stiffness increase considerably enhances the ability of the pavement to support heavy traffic, both in terms of magnitude of wheel load and number of applications of the loads.

2.2 Mechanisms of Failure

Heavily stabilized bases can fail in fatigue due to high tensile stresses induced by traffic if they are too thin. However, it is easy to design against such failure. Most often distress in heavily stabilized bases occurs due to shrinkage cracking in the stabilized bases, thermal movement of the slab or a combination of shrinkage cracking, thermal contraction and load-induced stresses.

Distress may be viewed as a three-step process: (1) development of transverse cracking due to shrinkage, (2) widening of the cracks due to thermal contraction, and (3) extended damage of the cracked, stabilized bases due to load-induced fatigue.

Upon curing, cementitious mixtures shrink due to water loss. The subbase offers resistance to the induced horizontal movement. The greater the resistance to horizontal movement of the base due to subbase restraint, the greater is the magnitude of tensile stresses induced within the stabilized base. Equations can calculate the tensile stresses and the spacing between tensile shrinkage cracks by relating these distresses to selected parameters of the pavement and the stabilized layer. However, these equations demonstrate that crack spacing depends primarily on tensile strength of the stabilized layer and the frictional resistance between the base and subbase.

Once these tensile cracks occur, their width can also be calculated. The opening of the cracks is primarily related to the tensile stiffness of the stabilized mix.

Cracking distress within the cement stabilized base worsens when thermally-induced contraction causes the stabilized base to move. This movement is restrained by the subbase and the surface layer . The stresses induced by this movement can also be mathematically modeled. Such a model demonstrates that the degree of damage caused by temperature change is also related to the stiffness or modulus of the cement stabilized layer.

Loading induces tensile stresses within the stabilized base. When a wheel load is applied to a pavement containing a stabilized base, the wheel load induces tensile flexural stresses at the bottom of the base which may cause flexural fatigue cracking if the pavement is not adequately designed. Research in project 1287 has shown that transverse cracks induced by shrinkage and exacerbated by thermal contraction can increase the intensity of load-induced flexural stresses by a factor of as much as 2.0. The width of the transverse crack and the corresponding load transfer across the crack strongly affects this factor.

2.3 Steps to Reduce Risk of Failure

Mixture Design

1. Where possible, designate more stringent limits on plasticity of fines and fines content (less than 10%) of aggregate stabilized with portland cement. As the 200 fraction increases, the shrinkage potential of the mix increases, particularly if clay minerals are contained in the mixture's minus 200 fraction.

2. Use as little portland cement or other stabilizer as possible to achieve the required strength. Smaller quantities of portland cement result in less hydration products and, hence, less shrinkage. Mixtures should be designed with unconfined compressive strengths (UCCS's) which meet but do not greatly exceed TxDOT Item 276 strength criteria. When strength significantly exceeds this criteria, more hydration products than necessary for strength development may result in excessive shrinkage, and the higher stiffness (modulus) results in wider crack openings and less load transfer efficiency.

3. The compaction moisture content should not exceed the value that produces maximum dry density. Evaporation of excess water leads to excessive shrinkage, and lower resistance to fracture, and, in some instances, rapid failure of the upper part of the created base layer.

4. If a more precise mixture design is accomplished, the shrinkage strain limit, ϵ_t, can be calculated according to the following equation:

$$\epsilon_t = 1/2 \left[\frac{\sigma^t}{\epsilon_t} + \frac{\delta_t \, \tau_s/h + \mu\gamma}{\sigma_t} \right]$$

In order to calculate ϵ_t according to the above equation, one must know the tensile strength, σ_t; tensile modulus, ϵ_t; allowable crack opening, δ_t; adhesion between the stabilized base and the subbase, τ_s; Poisson's ratio of the stabilized base, μ; unit weight of the stabilized base, γ; and tensile stress at the center of the slab between cracks, σ_t. Fortunately, calculations using realistic values and realistic limits yield a critical value for ϵ_t of 250 micrometers per meter. Report 2919-1 gives further details of this test.

A linear shrinkage test should be conducted using the optimal mixture design to ensure that shrinkage strains will not exceed $\epsilon_t = 250$ micrometers per meter.

Thickness Design

1. Keeping the stress ratio (SR - ratio of stress induced under load to flexural strength of the stabilized layer) below 50% can control fatigue crack initiation induced by load applications. However, the load induced stress has been shown in this research to increase by as much as 100% when the load is applied at the transverse crack. This worst case condition should be assumed, and design should be based on the worst case assumption.

2. Once a tensile crack is initiated in the stabilized base, the crack will propagate as defined by the laws of fracture mechanics which dictate that the rate of crack propagation is controlled by the stress intensity factors (K_I and K_{II}).

   As K_I and K_{II} increase, the rate of crack growth increases. K_I and K_{II} can be minimized by

   a. Reducing the stiffness of the stabilized base and

   b. Increasing the modulus of the supporting subgrade through proper stabilization. This reduces the modulus ratio of the stabilized base (E_{Base}) to the subgrade ($E_{Subgrade}$): $E_{Base}/E_{Subgrade}$.

2.4 Mix Design Considerations

The current basis for mix design of portland cement stabilized bases is the Minimum Design Compressive Strength as described in Item 276 of the TxDOT, 1993 Construction Specifications. Table 1 reproduces the strength requirement.

Table 1. TxDOT Strength Specifications for Item 276.

Classification	Minimum Design Compressive Strength, KPA	Allowable Cement Content Percent
L	5,250	4 - 9
M	3,500	3 - 9
N	Shown on plans	-
O	None	Shown on plans

Prior to these specifications, earlier approaches (1982 TxDOT Construction Specifications) were recipe-type where the recommended levels of stabilizer were given for each aggregate type. No matter which specification is used, the percentage stabilizer is frequently in the range of 5 - 6 percent by weight.

The Houston District has made extensive use of these specifications and has constructed many miles of highways with heavily stabilized cement treated bases. In this study, researchers monitored six sections with portland cement stabilized bases. Each of the sections provided good riding quality. The major difference in performance was the amount of non-load associated surface cracking, which ranged from none to extensive. The severity of cracking was directly related to the strength of the stabilized layer and was strongly dependent upon the type of aggregates used.

Evaluation of the Houston District experience revealed that performance of these bases could be improved considerably if more frequent, fine cracks occurred rather than wide cracks which deteriorate rapidly over time. For new construction, consideration should be given to the following:

1. Controlling the plasticity index (PI) and linear shrinkage of the fine portion of the base material. (Caltabiano recommended maximum values of 4 percent and 2.5 percent respectively). A max PI of 4 may not be practical with Houston materials. In this case the PI should be reduced to B or below.

2. Introducing a Shrinkage Test in which shrinkage within a control beam of cement treated base (CTB) should not exceed 250 micrometers per meter after 20 days (Caltabiano, 1992, Van Berk 1995).

3. Limiting the stabilizer content in order to reduce non-load associated cracking yet develop adequate compressive strength to maintain a significant structural contribution and maintain durability. The long-term field strengths obtained with the current specifications are exceedingly high.

Backcalculation of resilient moduli from Falling Weight Deflectometer (FWD) data in the Houston District reveal that very high moduli of cement-treated bases are clearly associated with wide shrinkage cracks and, ultimately, considerable non-load damage. For this reason, a realistic mixture design consideration is to add enough stabilizer to achieve the minimum strength requirement listed in Table 1, yet set an upper limit on the maximum allowable cement content. The Houston District study indicates that the minimum strength requirements set forth in Table 1 may be higher than required for optimal performance.

At this time, mix design should be based on a target 7-day pressive strength of the value stipulated in Table 1 ± 10 percent. Consideration should be given to eliminating the L classification. This tends to lead to overstabilization.

2.5 Thickness Design Considerations

General

The principal thickness design procedure for flexible pavements in use within TxDOT is Flexible Pavement Strength (FPS) 11. This procedure was developed primarily for "flexible" bases with either asphalt stabilized or unstabilized granular materials. The strength parameter used within FPS 11 is the stiffness coefficient parameter which was backcalculated from Dynaflect deflection data using procedures developed by Scrivner (1968) in Study 32. For typical pavements the following stiffness coefficients are generally used:

Asphalt Surfacings	0.95 - 1.0,
Asphalt Stabilized Base	0.80 - 0.90, and
Granular Materials	0.55 - 0.65.

Work in the Lufkin District based on observed field performance with cement-treated bases indicated that a representative stiffness coefficient for CTB is 0.70. The use of this value in FPS 11 resulted in design thicknesses that the District thought were reasonable. In all cases, researchers recommend a minimum layer thickness of 200 mm for these semi-rigid layers.

The Pavement Design Section of TxDOT recognizes the limitations of this procedure. The FPS system is based on a criteria of limiting deflections Surface Curvative Index (SCI) and the relationship between SCI and loss in Serviceability Index. While reasonable for "true flexible" pavements, this criteria is not suitable for heavily stabilized bases as the failure mechanisms for these two pavement types are very different. The approach that is typically used for heavily stabilized bases involved limiting the tensile stress at the underside of these layers to a specified stress ratio (SR = actual induced flexural stress/ ultimate flexural strength). Typically, the thickness of the layer is increased until the stress ratio falls below 0.50.

The problems with this approach are

1. Determining what value to use for the ultimate strength of the stabilized base since the semi-rigid materials gain strength with time, and

2. Calculating the actual stress under the design load, and determining what moduli value should be used for the stabilized layer (given the fact that the layer contains shrinkage cracks).

Thompson (1994) recommended a rational approach to both of these issues. He proposed using, as the design ultimate strength, the strength achieved when the highway is first opened to traffic. He also stated that shrinkage/thermal cracks are inevitable in these materials and that they must be accounted for in the design process. The CTB should be thick enough to prevent significant secondary load associated cracking that initiates at the transverse shrinkage cracks. The presence of these transverse cracks causes an increase in tensile stress at the underside of the slab, and this is where fatigue cracking will initiate.

In the testing of the Houston pavements, researchers collected FWD data on cracked and uncracked sections in both summer and winter. The load transfer efficiencies across these cracks varied considerably. In evaluating the impact of these variations in load transfer on induced stresses, the ILLI-SLAB finite element program developed by University of Illinois was used. The aggregate interlock factor of the joints within the program were modified until a deflection bowl similar to that measured under the FWD was obtained. The increase in stress at the bottom of the slab was calculated and the ratio of stress at the crack to the stress in the uncracked section was calculated. In order to use this information within the pavement design process, a simple microcomputer-based program was written to compute fatigue life. The user inputs the anticipated uncracked layer moduli and the average of the 10 heaviest wheel loads to be experienced by the pavement. The algorithms used in this program are described in the following sections.

Design Algorithm For Heavily Stabilized Bases

Traditional layered elastic theory computer programs (such as Chevron, BISAR and WESLEA etc.) predict the pavement response by assuming axi-symmetric loading which is equivalent to the interior loading. The critical stress to consider for stabilized base thickness design is the maximum flexural stress at the bottom of the stabilized base course. This approach is valid as long as the pavement is uncracked. But cementitious base materials typically shrink, forming transverse shrinkage cracks. Once a transverse crack forms, a different situation exists. Pretorious and Monismith (1972) described the critical stress condition for post-cracked stabilized bases. Increased stabilized base course tensile stresses must be anticipated due to the loss of continuity, and the critical loading is no longer interior loading. Depending on the width and the load transfer efficiency (LTE) across the crack, a critical loading condition equivalent to edge loading may result. The ILLI-SLAB program was used to predict the response of the cracked pavement since this program can directly model the cracks of different load transfer efficiencies by specifying different aggregate interlock factors. The maximum tensile stress occurs when the load is adjacent to the crack. This critical stress is at the bottom of the stabilized material layer and acts parallel to the crack. Researchers in this study recommend a multiplier of 2.0 to account for wide shrinkage cracks and their influence on the critical flexural tensile stress for interior loading. This multiplication factor can be reduced to as low as 1.0 depending on the LTE across the crack. Tight, hairline cracks have high LTE and in turn allow a low stress multiplication factor. Moderately stabilized bases in the Atlanta district are examples of pavements with good transfer efficiency which justify using a low value of stress multiplication factor (1.3 to 1.4). In the stabilized base thickness design program developed in this study, researchers used a conservative stress multiplication factor of 2.0 to insure that the heavily stabilized pavement sections, like those in the Houston District, are safe against fatigue-induced cracking.

Stabilized Base Thickness Design Program

In developing the microcomputer based thickness design procedure for CTB, the critical strains and stresses are calculated for specific traffic and pavement configurations. The following performance models are used to estimate the number of load repetitions to failure:

1. Fatigue in the asphalt concrete surface based on tensile strain at the bottom of the asphalt layer is evaluated using the ARE fatigue equation ,

2. Rutting and roughness due to deep layer distress based on subgrade compressive strain are evaluated using the model developed,

3. Load-induced fatigue in the stabilized layer is evaluated using the American Coal Ash Association approach.

The approach uses the WESLEA program to calculate strains due to 80 KN Equivalent Single Axle Loads.

The program incorporates a pre-processing stage in which all the material properties and layer thicknesses are input, a processing stage in which induced stresses and strains are calculated together with the critical stresses and strains, and a post-processing stage. The post-processing stage compares load-induced stresses and strains with critical strains calculated from the performance equations to evaluate the fatigue cracking and rutting potential in the asphalt, base and subgrade layers, respectively. The base layer thickness is incrementally increased until all criteria are met. A description of the input is given below.

Stage I

INPUT: In this stage the user inputs the following information:

1. Elastic moduli and Poisson's ratio for all layers,
2. Thickness for all layers except stabilized layer,

3. Range of acceptable thicknesses for the stabilized base,
4. Initial Serviceability Index,
5. Terminal Serviceability Index, and
6. No. of 80 KN Equivalent Single Axle Loads.

Stage II

Processing Stage: The processing stage is in two phases.

Phase I:

Critical strains for resistance to fatigue of asphalt and stabilized base layers and subgrade rutting are calculated using the following performance models:

Asphalt Layer Fatigue:

ARE fatigue equation is

$W_{18} = 9.73 * 10^{-15} (1/\epsilon_t)^{5.16}$

where

W_{18} = Weighted 80 KN applications before class-2 cracking

ϵ_t = Critical tensile strain at the bottom of asphalt layer

Subgrade Rutting:

$\mathrm{Log}_{10}^{Nx} = 2.15122 - 597.662 (\epsilon_{SG}) - 1.32967 (\log_{10} \epsilon_{SG}) + \log_{10} [(PSI - TSI) / (4.2 - 1.5)]^{1/2}$

where

$\mathrm{Log}_{10}^{Nx} = \mathrm{Log}_{10}$ of allowable applications of axle load x (In the general case axle load = 80 KN)

ϵ_{SG} = Subgrade Compressive strain due to axle load 'x'

PSI_x = Initial PSI of the pavement

TSI = Terminal Serviceability Index

Stabilized Base Fatigue:

The program checks for this criteria only when $E_{base} > 7,000$ MPa

The American Coal Ash Association equation is as follows:

Stress Ratio = $0.972 - 0.0825 \log_{10}^{N}$

where

$\log_{10}^{N} = \log_{10}$ of allowable application of 80 KN axle loads

Phase II:

Load- induced stresses and strains at designated location within the pavement structure are calculated.

Stage III

The WESLEA program calculates, for a pre-assumed thickness of stabilized base, strains and stresses at the top of the subgrade, bottom of bituminous surface layer and bottom of stabilized base. The tensile strain at the bottom of stabilized base is multiplied by a factor of 2.0 to account for poor load transfer across wide shrinkage cracks.

The WESLEA calculated induced stresses and strains are compared with the corresponding critical stresses and strains. The assumed thickness is considered sufficient when none of the calculated WESLEA stresses and strains exceed the corresponding critical stresses or strains from the applicable performance models. Otherwise, the thickness of the stabilized base is increased for the next trial. The process is repeated (within the thickness range) until a suitable thickness is found which satisfies all requirements.

Users Guide to Stabilized Base Thickness Design Program

Researchers developed a microcomputer program of the design procedure described earlier. The program is supplied on diskette. To run the program type "DISPLAY" and type "STBC" to load the input screen. It is assumed that the stabilized layer is Layer 2 of the structure. The user has to supply the following information:

1. Design volume of traffic in 80 KN Equivalent Single Axle Loads (in millions),
2. Initial Serviceability Index of the Pavement (PSI),
3. Terminal Serviceability Index of the Pavement (TSI),
4. Thickness, modulus and Poisson's ratio of the asphalt surface layer (in ins, ksi),
5. Range of thickness for stabilized base, modulus and Poisson's ratio of the stabilized base (in ins, ksi),
6. Thickness, modulus and Poisson's ratio of the subbase layer, if present (in ins, ksi), and
7. Depth to bedrock (if not known, leave for a default value by the program), modulus and Poisson's ratio of the subgrade (in ins, ksi).

Note: Press "F1" to edit the first three fields.
Press "F2" to edit the thickness, modulus and Poisson's ratio fields.

The program handles up to four layers (surface, stabilized base, subbase, subgrade) above the bedrock. If no subbase is present, enter 0.0 for the subbase thickness. The thickness of layer four is the depth to a stiff layer as calculated by MODULUS 5.0 program or assumed by the designer.

The program uses the WESLEA computer program as a subroutine to calculate induced tensile strain at the bottom of asphalt layer, the tensile strain at the bottom of stabilized base and the vertical strain at the top of the subgrade. The program performs this calculation for the following three positions: (a) under the center of tire, (b) at the edge of the tire, and (c) between the tires. The results are then passed to the main thickness design program to compare with the critical strains calculated using performance models.

The user has to supply a practical possible range for the thickness (say for example 150 mm to 500 mm) of the stabilized base. The program tries to find a solution within the prescribed range. A message is displayed identifying whether or not the program succeeded. If a solution is not found, the program displays the message "Failed." In order to save computational time, it is advantageous to identify a narrow range of potential base thicknesses for design.

The program displays a final output showing the final thicknesses and modulus values for various layers of the pavement. If the program fails to find the solution within the user input range, then the "failed" message will be displayed under all those layers where the criteria cannot be met within the range of thicknesses. As an example, if the failed message is displayed under both asphalt and subgrade layers, then the user input range for stabilized base thickness is not sufficient to meet both asphalt layer fatigue and subgrade rutting criteria. In order to allow for an acceptable solution, it is advisable to increase the upper limit for the thickness of the stabilized base.

An extensive sensitivity analysis was performed using the design algorithm. This sensitivity analysis evaluated the following matrix:

1. Hot Mix Asphalt Concrete (HMAC) surface: 37.5 and 75 mm;
2. HMAC modulus: 3,500 MPa;
3. Stabilized base modulus: 700; 1,400, 3,500, 7,000 and 14,000 MPa;
4. Subbase thickness: 0, 100 and 200 mm;
5. Subbase modulus: 140 MPa and 280 MPa; and
6. Subgrade modulus: 3,500 and 7,000 KPA.

The thickness values of the stabilized bases calculated were reasonable.

2.6 Summary of Recommended Changes in TxDOT Procedures and Specifications

1. Use criteria in Item 276 as a target. Base optimum design on percent stabilizer that achieves the target strength value within a 10 percent tolerance. Use fabrication and strength testing protocol described in test method TEX-120-E.

2. Use the PC computer program described in this section to insure that the thickness of the stabilized base is sufficient to resist load-induced fatigue damage.

3. MODERATELY AND LIGHTLY STABILIZED BASES

3.1 Purpose

Bases stabilized with 2 to 4 percent stabilizer (moderately stabilized bases) have functioned well in some districts. These bases typically perform with substantially higher resilient moduli than do unstabilized flexible bases but do not show the degree and severity of shrinkage cracking that many rigidly stabilized bases show. On the other hand, moderately stabilized bases may not be appropriate to carry very high volumes of traffic such as that experienced by urban interstates. Experience will dictate whether or not lower degrees of stabilization are appropriate.

Lightly stabilized bases (less than about 2 percent stabilizer) can also provide a significant increase in modulus. However, these bases, like the moderately stabilized bases, behave like higher stiffness flexible bases and, generally, not like rigidly stabilized bases. Thus, although the potential for shrinkage cracking may be reduced, the high strength and stiffness of heavily stabilized bases is not achieved. Certain heavily-trafficked pavements may require this high strength and stiffness.

3.2 Mechanisms of Failure

Since these bases are not rigid slabs, neither shrinkage, thermal cracking nor fatigue cracking are typically considered as failure mechanisms. Instead, the layer is designed as a high modulus flexible base. Such a base must provide adequate support for the surface layer and adequate load-spreading capability to protect the subgrade from being over-stressed. The layer must be adequately thick and of substantial strength so that it can resist shear stresses induced by traffic loading.

Another concern with these moderately stabilized bases is the non-permanency of the stabilization. In this case, the base reverts to a flexible base. Every effort must be made to supply adequate stabilizer content to promote permanent stabilization.

3.3 Steps to Reduce Risk of Failure

1. Utilize mixture design procedures which promote permanent and durable stabilization.

2. Utilize thickness design protocols which provide a base with adequate support for the surface layer, adequate resistance to shear stresses within the layer and adequate protection for the underlying subgrade.

3.4 Mix Design Considerations

Moderately Stabilized Bases

This classification was originally selected for stabilized bases in the Atlanta District where between 2 and 4 percent of total stabilizer was used. These correspond with Item 262 in the 1993 Texas Construction Specifications. Currently no recommendations on the level of stabilizer to use are given in the Specification, other than "as shown on plans."

The work in the Atlanta District has concentrated on upgrading existing base materials. The procedure used in the Atlanta District is the unconfined Texas Triaxial Test procedure (Tex-117-E) with 7 KPa confining pressure. The raw materials are tested and compared with those treated with different levels of stabilizer. Curing of the stabilized material involves 7 days moist curing at 25°C, 2-3 hours in an oven at 60°C followed by 10 days of capillary rise. This approach is similar to the Tex-121-E method. The samples are tested after the 10 days of capillary rise. The criteria is to identify the level of stabilizer that produces a 2- to 3-fold strength increase over that of the unstabilized aggregate.

The sections in the Atlanta District, which received the level of stabilization recommended by this procedure (2 - 4 percent), all performed very well. Some bases had been in service over 15 years with light to moderate traffic. They were performing as flexible pavements and had not developed the crack patterns associated with heavily stabilized materials. The backcalculated moduli for these materials were between 2 and 4 times higher than would have been anticipated for an untreated flexible base.

The researchers recommend the approach used in the Atlanta District, however, it is based on observations of the performance of the specific aggregates used in the Atlanta District, which are largely iron-ore gravels. A more thorough evaluation should be made of aggregates from other areas of the state. It may be appropriate to supplement the strength test with a test to evaluate durability. The rolling wheel durability test described in TTI Report 2919-1 is a protocol which can be effectively used to access durability in bases. However, further experience with this test is required in order to develop criteria by which to adequately access durability. The linear shrinkage test should not be necessary for moderately or lightly stabilized bases as matter of routine as these bases are essentially "stiffened flexible bases." However, the linear stress should not exceed 250 millimeter per meter.

Lightly Stabilized Bases

Calcareous bases in several districts have been stabilized with very low percentages of lime (1 to 4 percent, but more typically 1 to 2 percent). Chapter 4 of the 1287-2 report discusses this type of stabilization in detail. It is a unique type of stabilization when calcareous bases with very little or no clay fraction are being treated. This is because the cement matrix is chiefly carbonate in lieu of pozzolanic cement or hydrated calcium silicates or calcium aluminates.

Mixture design should be based on the Texas Triaxial test and should follow the general procedure outlined in Tex-121-E except that moist curing time should be extended to at least 14 days.

Data in Chapter 4 of Report 1287-2 suggest that 1 to 2 percent lime in either caliche or limestone bases typically will increase compressive strength by 50 percent or more and resilient moduli by a similar amount. This level of strength and modulus increase represents a very significant structural upgrade.

3.5 Thickness Design Considerations

In this study lightly stabilized bases are defined as those bases containing between 1 and 2 percent stabilizer. Moderately stabilized bases contain between 2 and 4 percent stabilizer. In both cases the pavements behave primarily as flexible pavements in that the major forms of structural distress are wheel path rutting and cracking. They do not exhibit the longitudinal and transverse cracking associated with heavily stabilized materials. Consequently, researchers proposed designing the thickness of these layers using the FPS 19 program. This program requires that an elastic modulus and Poisson's ratio be input for each layer.

Table 2 records the modulus increase due to moderate or light levels of stabilization. The Base Improvement Factor (BIF), ratio of modulus of the stabilized layer to modulus of the unstabilized layer, is the criterion upon which to evaluate the effect of stabilization.

Table 2 shows that the modulus improvement for lightly stabilized sections is only 10 percent on average in the Atlanta District where the aggregate is iron ore gravel (BIF = 1.1). This is insignificant. For these types of materials, no significant increase in modulus. However, significant improvements in layer moduli were observed with the high stabilizer contents can be assigned and the thickness should be designed as an unstabilized base. In the Yoakum District, the addition of 1 to 2 percent lime to calcareous bases results in a considerably high level of BIF. However, the base still retains a flexible nature and should be treated as a flexible base in thickness design.

Table 2. Influence of Stabilizer Content of Base Modulus. BIF is Ratio of Backcalculated Modulus of the Moderately or Lightly Stabilized Layer to the Backcalculated Modulus for an Unstabilized Layer.

Percent Stabilizer	Number of Sections	Base Improvement Factors	
		Range	Average
1 - 2	4 (Atlanta)	0.8 - 2.7	1.1
2 - 3	4 (Atlanta)	1.7 - 3.1	2.4
3 - 4	2 (Atlanta)	3.1 - 4.1	3.6
1 - 2	10 (Yoakum)	1.0 - 5.1	3.0

For stabilizer contents greater than 2 percent, the increase in modulus for the base layer was both substantial and permanent. For 2 - 3 percent stabilizer, the range of improvement was from 70 percent to 210 percent, with an average improvement of 140 percent. Therefore, for an unstabilized granular base which typically has a modulus of 200 MPa, the average modulus for the layer stabilized with 2 - 3 percent stabilizer would be 480 MPa.

Several runs of FPS-19 were made to evaluate the consequences of incorporating these base improvement factors. As a sensitivity evaluation, a low BIF (1.7) and an average BIF (2.4) were evaluated. In all cases, the design thickness for the stabilized layer was compared with that obtained if untreated flexible base was used. Table 3 shows the results. Three subgrade strengths ranging from very poor to very good and two design traffic loadings, 1 and 2 million 80 KN axle equivalents, were considered for the Houston and Atlanta Districts. In all cases, the pavement was designed to have a 37 mm thick hot mix surfacing. The stabilized layers were assigned higher moduli values and were predicted to require between 50 and 100 mm less base material for the same design life.

The researchers propose using the conservative BIF of 1.7 for design until more field performance data are collected. When running FPS 19, the user should select option 1 for untreated base design. Based on the subgrade soils in that county the program will recommend both a subgrade and flexible base moduli value. If the base is to be moderately stabilized, as described earlier, the unstabilized base modulus should be multiplied by 1.7 for thickness design purposes.

Table 3. Impact of Base Stabilization on Thicknesses Predicted Using FPS19.

· Flex = Untreated Flexible Base
·E_B*1.7 = Modulus Improvement Caused by Stabilization
·Design parameters
·HMAC Thickness 37mm Time to First Overlay = 12 years
Reliability Level = C
ADT = 2500
= Section hit minimum thickness. (200mm)

District	Sub-grade		Design 80 KN applications, X10^6	Base Thickness (mm)		
	Description	E_S, MPa		Flex (E_B)	Stab. (E_B*1.7)	Stab. (E_B*2.4)
Houston	V. Poor	27.6	2	575	425	400
	Int.	55.2	2	450	375	325
	V. Good	138	2	350	250	200
Atlanta	V. Poor	27.6	2	625	450	425
	Int.	55.2	2	500	400	350
	V. Good	138	2	375	300	225
Houston	V. Poor	27.6	1	275	375	350
	Int.	55.2	1	375	325	275
	V. Good	138	1	250	200+	200+
Atlanta	V. Poor	27.6	1	500	400	400
	Int.	55.2	1	400	325	300
	V. Good	138	1	300	200	150

3.6 Summary of Recommended Changes in TxDOT Procedures and Specifications

1. Use test methods TEX-121-E with a moist curing period of 7-days at approximately 25°C followed by 2-3 hours at 60°C and 10 days of capillary soak to determine the unconfined compressive strength of the mix at various stabilizer contents. Identify a stabilizer level that will produce a 2-to 3-fold increase in strength over that of a typical unstabilized base, approximately 300KPa when tested in triaxial compression at a confining pressure of 7 kPa for moderately stabilized bases. For lightly stabilized bases, the compressive testing must show a strength increase of at least 50 percent to warrant consideration in structural thickness design.

2. Establish an acceptable stabilized base thickness using the FPS-19 protocol and a BIF of 1.7 for moderately stabilized bases and 1.1 for lightly stabilized bases.

4. STABILIZED SUBGRADES

4.1 Purpose

Subgrades are commonly stabilized in Texas to improve workability and constructability, reduce swell and shrinkage potential, improve support of flexible bases and surface layers and improve the structural capacity of the pavement. Several stabilizers have been successfully used such as, lime, lime - fly ash, fly ash, portland cement and asphalt. Lime and portland cement are the most commonly used.

4.2 Mechanisms of Failure

The mechanisms of failure are synergistically related to mix design and thickness design. Improper or inadequate stabilizer content due to poor mix design may lead to reversal in stabilization or a lack of durability or inadequate strength development to serve the design function. Improper thickness design may lead to insufficient thickness to resist flexural stresses induced by traffic loading leading to flexural fatigue damage. This can result in a progressive loss of strength in the stabilized layer, usually from the bottom of the layer (where the tensile zone develops) upward. Researchers documented this loss of strength in the lower extremities of the stabilized subgrades in a number of cases in this study using the Dynamic Cone Penetrometer (DCP).

4.3 Steps to Reduce Risk of Failure

1. Rely on a good mixture design approach to produce a durable, permanently stabilized mix with optimal strength.

2. Use thickness design guidelines to provide stabilized layers with adequate thickness to resist flexural fatigue and to provide adequate structural contribution to the pavement.

4.4 Mix Design Considerations

Current Approach

Test method Tex-121-E presents the approach used by TxDOT to design lime-soil and lime-aggregate mixtures. In this method the recommended percentage of lime for stabilization is based on the percent binder (minus 40 sieve size fraction) and plasticity index (PI). A plot of the locus of these two index properties on a lime content design chart defines the trial lime content to be used in a lime-stabilized soil or base.

Tex-121-E requires samples to be fabricated in accordance with Tex-113-E at a compactive effort of 1.09 Joules per cubic centimeter (13.26 ft-lb per cu. in.). Samples are next moist cured at room temperature for 7-days, dry cured at a temperature not to exceed 60 °C for

6-hours or until one-third to one-half of the molding moisture is removed. Finally, the sample is subjected to capillary rise for 10-days prior to triaxial compressive strength testing.

Unconfined triaxial compressive strength testing is performed in accordance with Tex-117-E. Tex-121-E recommends that a strength of 700 KPa is satisfactory for final course of base construction, and at least a 350 KPa unconfined compressive strength is required for subbase soils.

Recommended Changes

Figure 1 presents the recommended approach for mixture design. The first step in this approach is to perform the pH test in accordance with ASTM C 977. This test defines the percentage of lime required to satisfy initial soil-lime reactions and still provide enough residual lime to drive the pozzolanic reaction. Verification of the pH test defined lime content is based on Tex-121-E strength testing.

The authors recommend modification of Tex-121-E to accommodate longer curing of lime-soil and/or lime-aggregate mixtures. The recommended approach is 14-day moist curing instead of 7-day moist curing. All other curing procedures remain the same as defined in Tex-121-E. The longer period of moist curing is necessary because pozzolanic reactions in lime-soil mixtures occur more slowly than cementitious hydration reactions. Furthermore, some lime-soil mixtures do not respond predictably to accelerated curing.

Unconfined Compressive Strength (Note 1)
a. Natural Soil
b. Soil + Lime; Curing: 48 hours at 49^0C

Determine Strength Increase

<350^0 KPa
Nonreactive Soil
(Mixture not suitable for structural layer application)

> 350^0 KPa
Reactive Soil
(Mixture suitable for structural layer application)

Unconfirmed Compressive Strength Testing
(Variable Lime %, Note 2)

Analyze Strength Test Results

Determine Design Lime % above which further increases do not produce significant additional strength

Check Strength of Design Mixture with Criteria

Design Field Lime % Add 1/2 to 1% to design lime % to account for construction losses, uneven distribution, etc.

Notes

1. All specimens compacted at optimum water content to maximum dry density or TEX-121-E Lime treatment level for b may be 5% or as determined by the "ph procedure" (ASTM C977)

2. Specimens compacted at optimum moisture content and maximum dry density. Use the "ph procedure" to estimate the optimum lime content. Specimens should be prepared at optimum, optimum +2, and optimum -2 lime percentages. Additional and/or different lime percentages. May be required for some soils. Four specimens should be prepared for each treatment lime percentage. Recommended curing conditions are 48 hours @ 49^0 C or 28 days @ 22^0 C.

Figure 1. Recommended Approach for Mixture Design for Lime-Stabilized Subgrades.

4.50 Thickness Design Considerations

General

In order to be able to assign structural significance to a stabilized subgrade the designer must be reasonably confident that the stabilization is permanent and that the structural contribution is significant. Although permanency or durability cannot be absolutely assured, it is possible to provide a high level of reliability by following the mixture design procedures established in the preceding section.

Procedure

The process for assigning structural significance to lime-stabilized subgrades in Texas is a two-phase process. The first phase is to assign a realistic approximate resilient modulus to the stabilized layer. This approximation is based on laboratory testing and field FWD evaluations.

Assigning a realistic resilient modulus for design and analysis involves the following steps:

1. Estimate the average annual roadbed resilient modulus from FWD backcalculations based on the MODULUS 5.0 program. Calculate a reasonable weighted average annual modulus using the approach described in the 1986 AASHTO Pavement Design Guide.

2. Determine the unconfined compressive strength of the lime-stabilized mixture in accordance with Tex-121-E following a curing period of 14-days and cured at a temperature of 25 °C.

3. Determine a representative design modulus for the stabilized subgrade layer based on an average annual roadbed modulus and an average stabilized subgrade modulus to natural subgrade modulus ratio.

The second phase involves evaluation of the structural compatibility and capacity of the lime-stabilized subgrade with the pavement system. This phase involves the same three steps as listed above plus evaluation of the flexural fatigue damage potential within the stabilized subgrades.

Estimation of Stabilized Subgrade Modulus

A realistic and conservative estimate of the resilient modulus of a lime-stabilized subgrade can be determined based on the 14-day unconfined compressive strength determined in accordance with Tex-121-E at a test temperature of 25 °C and an estimate of the average annual subgrade modulus based on FWD data and MODULUS backcalculations.

A review of work by Suddath and Thompson (1975) and Thompson and Figueroa (1989) supplemented by testing in this study reveals a relationship between the unconfined compressive strength of the lime-soil mixture and the resilient modulus of the mixture.

Figure 2 presents a relationship between unconfined compressive strength and flexural modulus (based on data from Thompson and Figueroa (1989)), unconfined compressive strength and backcalculated field moduli (determined from FWD data from the 1287 study) and unconfined compressive strength and compressive moduli (based on data from Thompson and Figueroa (1989)). From this figure, it can be seen that the relationship between unconfined compressive strength and flexural modulus and between unconfined compressive strength and field (FWD backcalculated from study 1287) modulus are in reasonable agreement. The compressive modulus approximated from unconfined compressive strength data appears to be a conservative approximation of the modulus of the lime stabilized layer. Based on the findings summarized in Figure 2, a realistic and conservative approximate modulus for the lime-stabilized layer that can be used in design approximations is presented by the dashed line in Figure 2. For clarity, this relationship is replotted in Figure 3.

The researchers believe feel that it is reasonable that the resilient modulus of the stabilized subgrade should also be affected by the level of support provided by the natural subgrade. Figure 4 is a plot of subgrade resilient modulus versus the ratio of modulus of the lime-stabilized subgrade (from FWD backcalculations) to modulus of the natural subgrade (from FWD backcalculations). These data indicate that for natural subgrade moduli below about 50 MPa, the modulus ratio is typically 10 or above. For subgrade moduli between 50 MPa and 200 MPa, the modulus ratio is between 5 and 10, and for subgrade moduli exceeding 200 MPa, the modulus ratio is less than about 5.

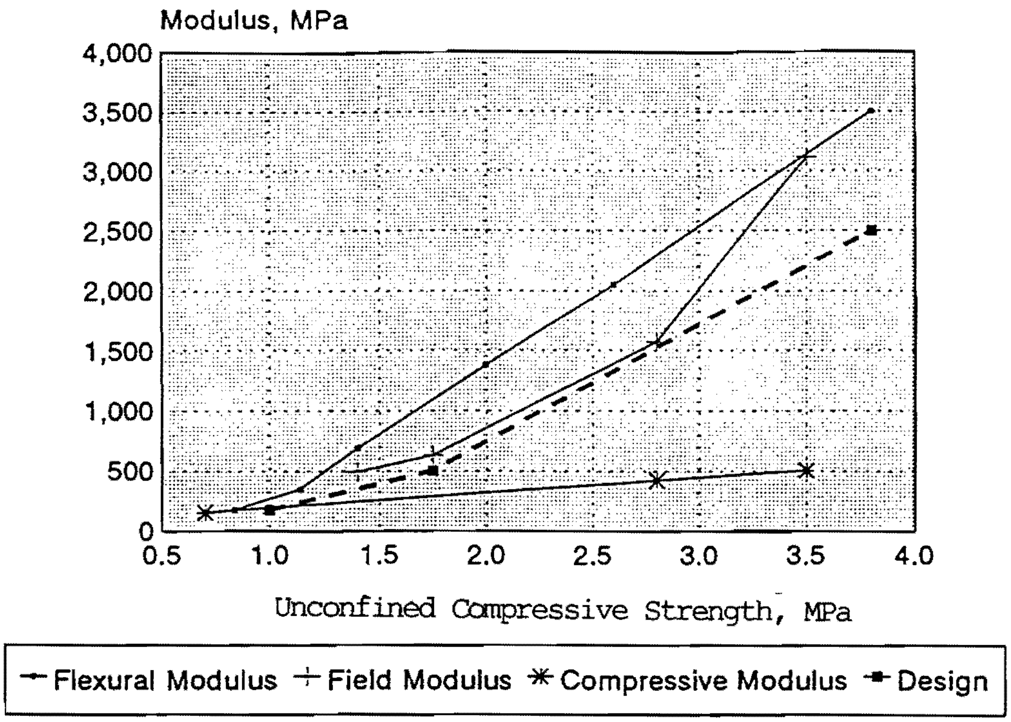


Figure 2. Relationships Between Unconfined Compressive Strength and Moduli of Lime-Stabilized Soils.

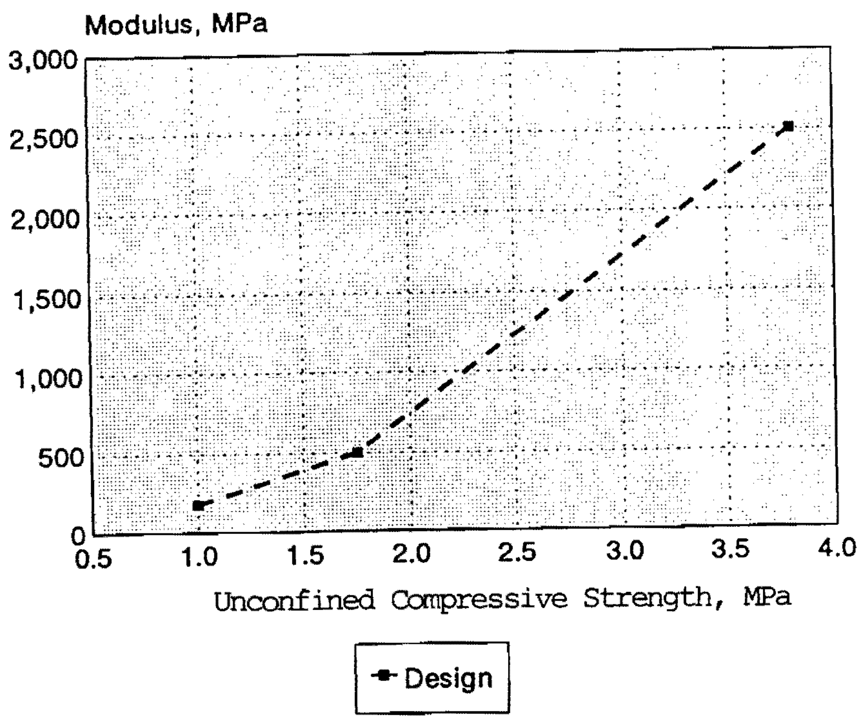


Figure 3. Selected Design Relationship Between Unconfined Compressive Strength and Resilient Modulus for Lime-Stabilized Subgrade Pavement Layers.

Resistance of Lime-Stabilized Layers to Flexural Fatigue

Once the lime-stabilized soil mixture has been determined to be reactive, e.g., unconfined compressive strength of 1,000 KPa or greater and an increase in unconfined compressive strength of at least 350 KPa over that of the unstabilized soil, and the average annual roadbed modulus and stabilized layer moduli have been determined, evaluate the ability of the pavement structure to resist flexural fatigue.

Perform this evaluation using any layered elastic computer model. This evaluation is easily incorporated into computer models such as FPS-19 or the heavy stabilized CTB model. In the absence of a computer model, assess the ability of the stabilized layer to resist fatigue damage by

1. Determining the critical radial tensile stress developed under load within the lime-stabilized layer and

2. Comparing the flexural strength of the stabilized layer with the critical flexural tensile stress developed within the stabilized layer.

As shown in Figure 5, the stress ratio, ratio of induced tensile flexural stress to flexural strength, should be less than 0.50 to insure a long (10^7 axle applications or greater) life or a fatigue resistant layer. Since the flexural strength is approximately 0.25 times the unconfined compressive strength and since the ratio of tensile strength induced within the stabilized layer should be less than 0.50, the critical flexural stress within the stabilized layer should not exceed 12 percent of the compressive strength.

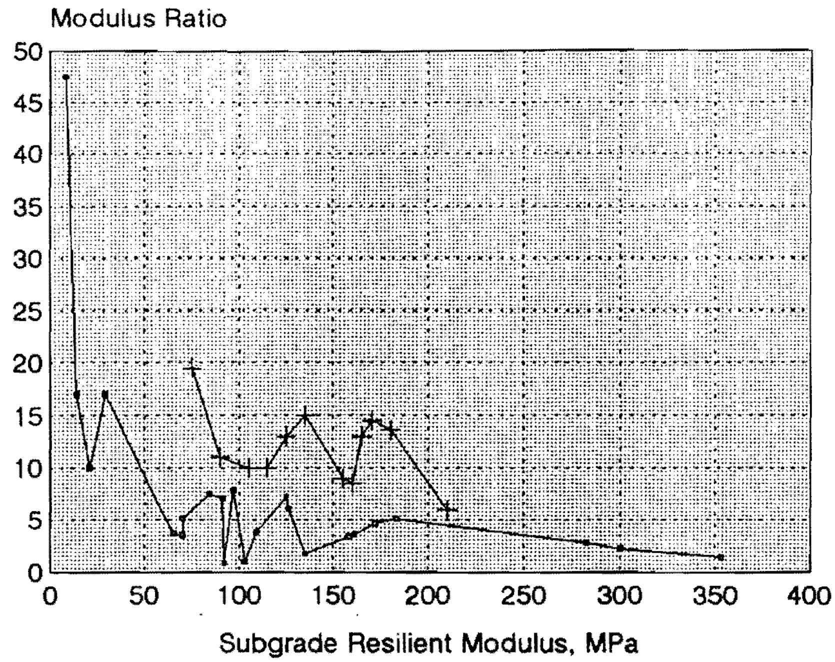
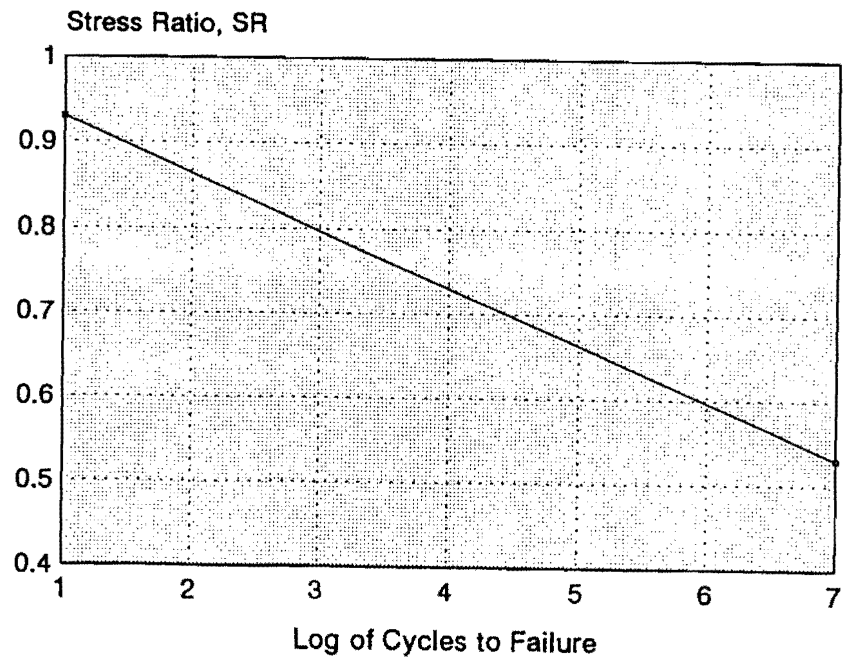


Figure 4. Relationship Between In Situ Modulus of the Natural Subgrade Soil as Determined by FWD Measurements and the Moduli Ratio (Lime-Stabilized Layer to Natural Subgrade Layer) as Determined by FWD Measurements.



$S = 0.923 - 0.058 \log N$

Figure 5. Stress Ratio Versus Cycles to Failure Fatigue Relationship (After Thompson and Figueroa (1989)).

Thompson and Figueroa (1989) calculated radial stresses in lime-stabilized subgrades of various moduli as a function of the layer thickness of the stabilized layer under an 80 KN axle load for soft, medium and stiff subgrades, Figures 6 through 9. In these figures, it is assumed that the surface layer is merely a surface treatment and does not contribute to the structural integrity of the pavement.

From the data in these figures, Thompson and Figueroa (1989) developed a regression model by which to calculate flexural tensile stress as a function of the thickness of the stabilized layer and the resilient modulus of the subgrade.

Although developed for a two-layer system, the model can be used in a multilayered structure by using Odemark's transformation to approximate the effect of the HMA and the unbound base layers. Applying the Odemark transformation and assuming realistic and conservative average annual moduli for the HMA and unbound base layers in Texas, calculate the effective thickness of the pavement as follows:

$$T_{eff} = A\ T_{HMA} + B\ T_{Flex.} + T_{LSS}$$

In this relationship A is calculated as the cube root of a representative, the quotient of HMA modulus (2,590 MPa) and the lime-stabilized modulus (E_{LSS}):

$$A = (2{,}590\ \text{MPa}/E_{LSS}\ \text{MPa})^{0.33}$$

B is the cube root of the quotient of a representative unbound base modulus and the modulus of the lime-stabilized subgrade:

$$B = (245\ \text{MPa}/E_{LSS}\ \text{MPa})^{0.33}$$

and T_{HMA} is the actual thickness of HMA, $T_{Flex.}$ is the actual thickness of the flexible base, and T_{LSS} is the actual thickness of the lime-stabilized layer.

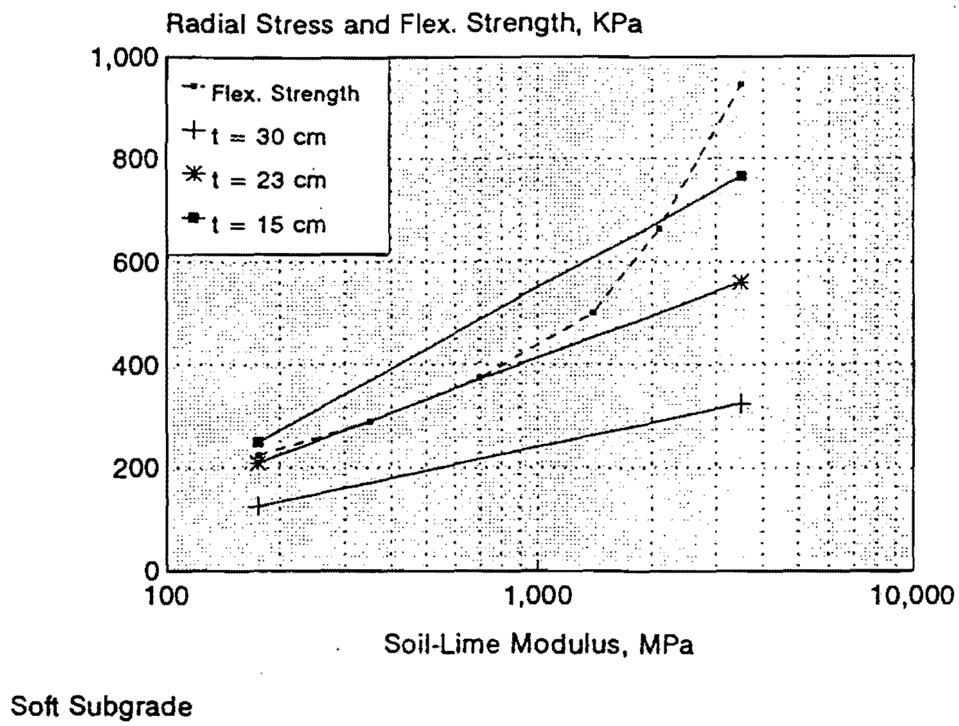


Figure 6. Relationship Between Modulus of Lime-Soil Mixture and Radial Stress Induced in the Lime-Stabilized Layer and Flexural Strength for Soft Natural Subgrades (After Thompson and Figueroa (1989)).

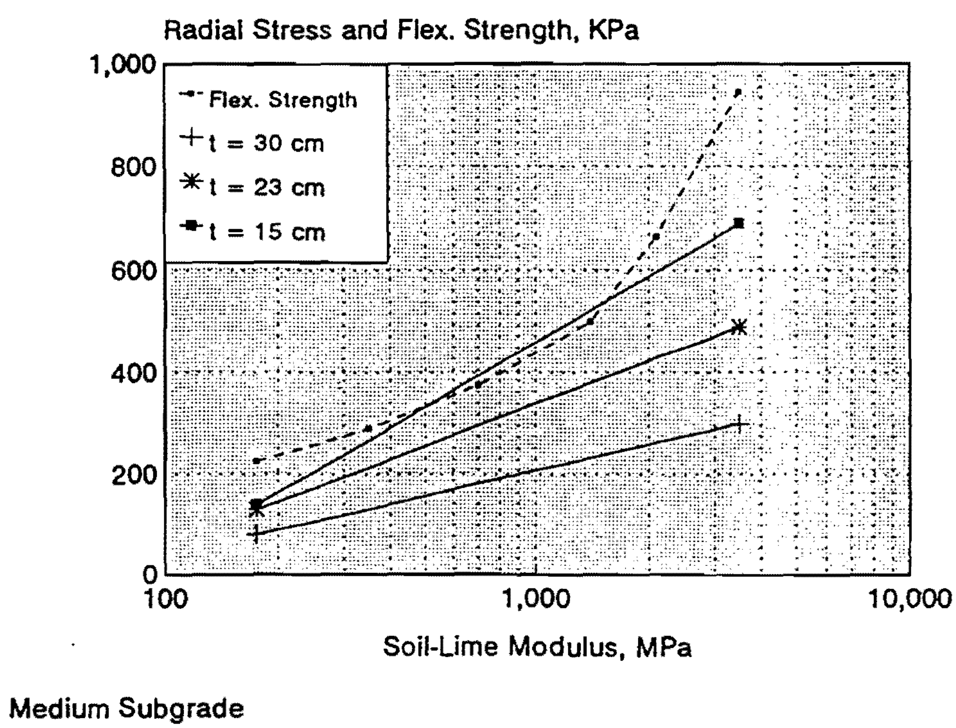
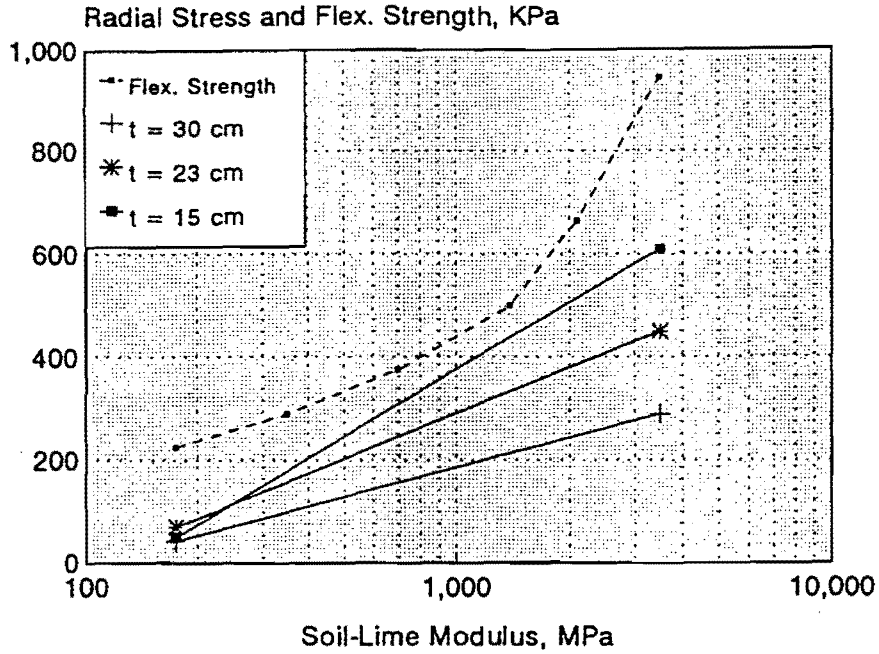


Figure 7. Relationship Between Modulus of Lime-Soil Mixtures and Radial Stress Induced in the Lime-Stabilized Layer and Flexural Strength for Medium Stiffness Natural Subgrades (After Thompson and Figueroa (1989)).



Stiff Subgrade

Figure 8. Relationship Between Modulus of Lime-Soil Mixtures and Radial Stress Induced in the Lime-Stabilized Layer and Flexural Strength Stiff Natural Subgrades (After Thompson and Figueroa (1989)).

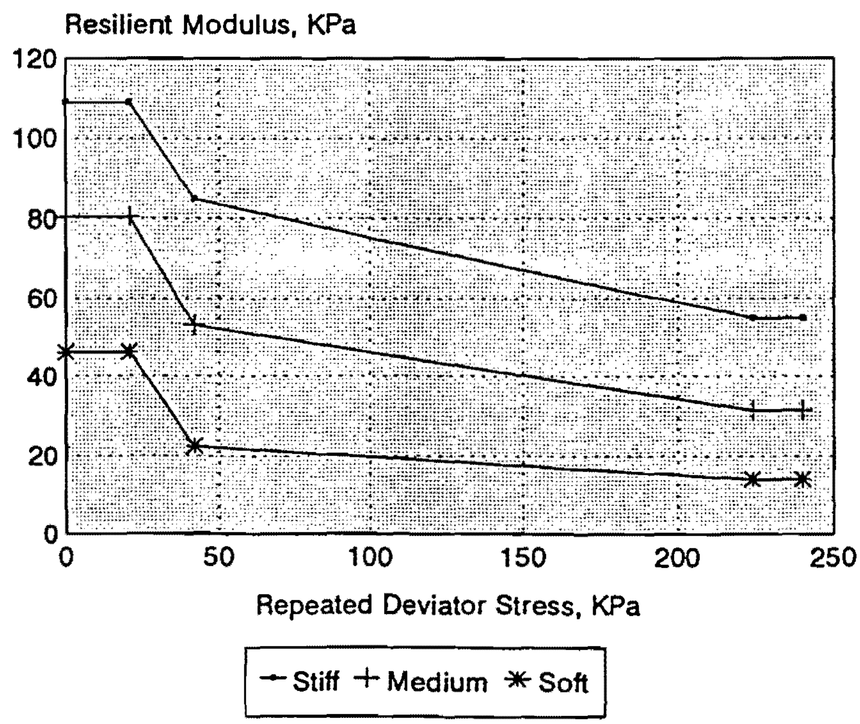


Figure 9. Typical Resilient Modulus Versus Deviatoric Stress Relationships for Soft, Medium and Stiff Subgrades.

Example Calculation

Assume a lime-stabilized layer, where the mix was designed in accordance with the procedure set forth, has a compressive strength of 2,000 KPa. From Figure 3, the approximate resilient modulus is 800 MPa. The natural subgrade average annual modulus is 133 MPa which is within the range of acceptable modulus ratio criteria, Figure 5.

The pavement structure is to consist of 75 mm of HMA, 305 mm of flexible base (crushed limestone) and 150-mm of Load Stabilized Subgrade (LSS). The effective thickness in terms of the LSS is:

$$T_{eff} = (2{,}590/800)^{0.33} \times 75 + (245/800)^{0.33} \times 305 + 150 = 466 \text{ mm}$$

The flexural radial stress in the LSS is calculated from the Thompson and Figueroa (1989) regression equation:

$$\sigma_r = 23.22 - 4.66(T_{eff}) + 42.66 \log E_{subg.} - 29.11 \log E_{LSS}$$

where T_{eff}, $E_{subg.}$ and E_{LSS} are in inches, psi and psi, respectively.

From this calculation, σ_r is -187.5 KPa, and the stress ratio, SR = -187.5/0.5(2,000) = 0.187, which is far less than 50 percent and is safe against fatigue.

Evaluation of flexural fatigue using the aforementioned approach should be made when either a thin HMA surface (less than 75 mm) or a surface treatment is placed directly over the lime-stabilized subgrade or over a thin aggregate base (less than 150 mm) and lime-stabilized subgrade. Otherwise, under typical highway wheel loads, significant flexural fatigue damage in the lime-stabilized layer is not an significant problem.

If fatigue damage is a potentially significant problem in flexible pavements due to heavy wheel loads, a layered elastic stress evaluation should be made using the subgrade and lime-treated subgrade moduli calculated as discussed in the preceding sections. The stress ratio fatigue evaluation explained in the preceding section should be used. The Thompson and Figueroa (1989) algorithm is only for an 80 KN axle load.

When flexural fatigue in the stabilized layer is not a consideration, the approximate modulus value of the lime stabilized layer may be appropriate for use in design algorithm. Such moduli values can be derived as previously discussed.

4.6 Summary of Recommended Changes to TxDOT Procedures and Specifications

1. Use the Eades and Grim pH test as described in ASTM C-977 to determine the starting lime content for mixture design. Using this lime content, perform test method TEX-121-E (using a 14-day moist cure in lieu of a 7-day moist cure) at the optimum lime content (according to ASTM C-977), at the optimum minus 1 percent, at the optimum plus 1 percent and at the optimum plus 2 percent. The strength results will

then be plotted versus lime content. The optimum lime content is that which produces optimum compressive strength.

The 1287 study revealed that TEX-121-E often underpredicts the required stabilizer content to produce a permanent reaction. Stabilizer contents of 4 percent were found to be inadequate occasionally in the Bryan District. Stabilizer contents of 6 percent used in the Ft. Worth District were documented to be permanent. These findings are substantiated by the work of McCallister and Petry (1990). Their study for the U. S. Army Corps of Engineers revealed permanency of stabilization when 5 to 7 percent lime was added to selected Texas soils but potential look stabilization benefits when 3 or 4 percent lime was added to the same soils.

2. Estimate the modulus of the lime stabilized subgrade based on the TEX-121-E compressive strength, Figures 4 and 5. The modulus of the lime stabilized layer should not exceed 15 for subgrades with an average annual resilient modulus of less than 50 MPa, should not exceed 10 for subgrades with an average annual resilient modulus of between 50 MPa and 200 MPa and should not exceed 5 for subgrades with an average annual modulus of above 200 MPa.

3. Use the approach in the 1986 AASHTO Pavement Design Guide to compute the average annual resilient modulus of the subgrade soil.

4. Evaluate the ability of the lime stabilized subgrade to resist flexural fatigue damage using the protocol outlined under ***Thickness Design Considerations*** in this section.

5. Consider the benefits of a weak stabilized subgrade in terms of the performance of the flexible base. Typically the flexible base modulus is approximately 2.0 to 2.5 times the subgrade modulus. Falling Weight Deflectometer (FWD) data in the Bryan District demonstrates a substantially higher response modulus of a flexible base placed over a stabilized subgrade.

REFERENCES

AASHTO *Pavement Design Guide*, 1986.

American Society Testing Materials, 1989, C977-87 Specification for Quicklime and Hudrated Lime for Soil Stabilization 04.01.

Catabiano, M.A. and Rawlings, R.E., 1992, "Treatment of Reflection Cracks in Queensland," Proceedings of 7th International Conference on Asphalt Pavements, pp. 1-21.

Flexible Pavement Manual, 1991. American Coal Ash Association, Washington, D.C., 1991.

Little, D.N., Scullion, T., Kota, P. And Bhuiyan, J., 1995. "Identification of the Structural Benefits of Base and Subgrade Stabilization," Research Report 1287-2, Texas Transportation Institute, Texas A & M University, College Station, Texas.

McCallister, L.D., and Petry, T.M., 1991. "Property Changes in Lime Treated Expansive Clays Under Continuous Leaching," Technical Report GL-90-17, U.S. Army Corp of Engineers.

Pretorious, P.C., and Monismith, C.L., 1972. "Fatigue Crack Formation and Propagation in Pavements Containing Soil Cement Bases," ***Highway Research Record 407***, Transportation Research Board, Washington, D.C.

Scrivner, E.H., Moore, W.M., McFarland, F., and Carey, G.R.. "A Systems Approach to the Flexible Pavement Design Problem," Research Report Number 32-11, Oct. 1968

Suddath, L.P., and Thompson, M.R., 1975. "Load Deflection Behavior of Lime Stabilization Layers," Technical Report M-11B, Construction Engineering Research Laboratory, Champaign, Illinois.

Thompson, M.R. and Figureoa, J.L., 1989. "Mechanistic Thickness Design Procedure for Soil-Lime Layers," ***Transportation Research Record*** 754, Transportation Research Board, Washington, D.C.

Thompson, M.R., 1970. "Soil Stabilization for Pavement Systems - State of the Art," Technical Report, Construction Engineering Research Laboratory, Champaign, Illinois.

Thompson, M.R., 1994. "Design of Heavily Stabilized Bases," Paper presented in 1994 meeting of the Transportation Research Board.

Van Blerk, G. and Scullion, T. "Evaluating Stabilized Durability with the Modified South African Wheel Tracking Device," TTI Report 2919-1, Sept. 1995.