Investigation of Performance of Heavily Stabilized Bases in Houston, Texas, District

PRAKASH B.V. S. KOTA, TOM SCULLION, AND DALLAS N. LITTLE

In situ strengths and performance of heavily stabilized bases in Houston, Texas, are evaluated. The falling weight deflectometer was used to evaluate in situ moduli. All sections had similar stabilizer contents (5 to 6 percent) and similar thicknesses (approximately 300 mm). The aim of the study was to (a) evaluate performance in flexible pavement systems and (b) if necessary, alter mix designs. Although similar designs were used in the pavements evaluated, the performance of the sections was dictated by the amount of shrinkage cracking that occurred. In fact, it appeared that the performance was inversely related to layer strength and stiffness. It was found that the cracking was largely influenced by the aggregate type used. In terms of structural strength, all sections were adequate. Recommendations include (a) limit the amount of stabilizer based on shrinkage criteria and (b) use a stress multiplication factor of 2 to account for cracking to predict the tensile stress under load.

The purpose of this study was to collect the performance and deflection data from inservice pavements in the Houston district of the Texas Department of Transportation (TxDOT) containing heavily stabilized bases and to develop appropriate guidelines for the design and use of such layers. Pavement performance is defined as the history of the pavement condition over time or with increasing axleload applications. This study was conducted in an effort to (a) measure the in situ stiffnesses of the stabilized layers, (b) evaluate their performance in terms of cracking, and(c) make general recommendations. Typical pavement sections in the Houston district consist of 75- to 100-mm thick hot mix asphalt concrete (HMAC), 225- to 400-mm thick cement-treated base (CTB) or lime-treated base (LTB), and 150-mm lime-treated subgrade (LTS) over a fine sandy loam to clayey soil. Data were collected from seven pavements with either cement- or lime-stabilized bases. These pavements represent a wide range of ages and traffic volumes. Table 1 presents detailed information about the pavement sections, including a description of each layer, layer thicknesses, type of aggregate in the stabilized base, type and amount of stabilizer, and the age of the pavement.

CURRENT TxDOT DESIGN PROCEDURES

Mixture Design

The cement-stabilized bases were designed on the basis of specifications for gradations in accordance with Item 274 of TxDOT and the percentage of stabilizer based on a minimum strength requirement of 4.48 MPa after 7 days of moisture curing at room temperature. Additional specifications about material passing the No. 40 sieve (known as "Soil Binder") are that the plasticity index should not exceed 10 and the liquid limit should not exceed 35. An example of TxDOT gradation specification ranges for cement-stabilized base material is provided in Table 2. The specifications allow for a tolerance of ±5 percent from the values given in Table 2.

Thickness Design

Currently, the thickness of the stabilized base is designed using TxDOT FPS11 software. The engineer supplies a stiffness coefficient for each layer with a minimum acceptable serviceability index. This current study was initiated partly because of concerns regarding this procedure. TxDOT hopes to move toward a mechanistic design procedure once this study is complete.

MECHANISTIC DESIGN CONCEPTS

Procedure

An acceptable mechanistic design procedure should be able to predict the performance of the pavement consistent with observed field experience. The mechanistic design criterion for stabilized bases in previous studies was based on the fatigue consumption of the stabilized layer resulting in the formation of longitudinal cracks. Research studies by Mayhew and Potter (*1*), Wang and Kilareski (*2*), Pretorious and Monismith (*3*) observed that the formation of longitudinal cracks that intersect transverse shrinkage cracks leads to the structural failure of the stabilized base.

Fatigue

The fatigue concept relates to crack initiation by developing small micro-cracks at the bottom of the stabilized material. But additional load repetitions are required for the crack to grow and propagate to the surface of the layer. Raad (*4*) suggested using shift factors (the ratio of the number of load repetitions for crack propagation to the surface to the number of load repetitions for crack initiation) to estimate the additional repetitions of load needed to propagate cracks in the field. Bofinger (*5*) reported that a higher cement content and an increase in the initial dry density increases fatigue life. Traditional layered elastic theory computer programs (e.g., Chevron five-layer program and BISAR) predict the pavement response by assuming axi-symmetric loading, which is equivalent to the interior loading case. The critical stress to consider for thickness design in this case is the maximum flexural stress at the bottom of the stabilized base course. This approach is valid as long as the pavement is uncracked.

T. Scullion and D.N. Little, Texas Transportation Institute, Texas A&M University, College Station, Tex. 77843. P.B.V.S. Kota, Texas Department of Transportation, 125 E. 11th Street, Austin Tex. 78701-2483.

TABLE 1 Details of Pavement Sections Investigated

Pavement Section	Description	Aggregate type	Percent stabilizer	Age (years)
1	75 mm HMAC 350 mm CTB 150 mm LTS	LS	C = 6	7
2	75 mm HMAC 225 mm LTB 175 mm LTS	REB	L < 4	7
3	75 mm HMAC 350 mm CTB 150 mm LTS	LS	C = 6	3
4	100 mm HMAC 275 mm CTB 150 mm LTS	REB	C = 6	3
5	75 mm HMAC 300 mm CTB 150 mm LTS	RG	C = 6	7
6	75 mm HMAC 300 mm CTB 150 mm LTS	LS	C = 6	4
7	75 mm HMAC 350 mm CTB 150 mm LTS	OS	C = 6	5

C = Cement
L = Lime
LS = Limestone
REB = Recycled Existing Base
RG = River Gravel
OS = Oyster shell

Cementitious base materials typically shrink, forming transverse shrinkage cracks. Research performed by Pretorious and Monismith (*3*) described the critical stress condition for the postcracked situation in stabilized bases as transitioning from the interior toward the edge loading. This, of course, results in increased tensile stresses in the stabilized base course. Depending on the width and the load transfer efficiency (LTE) across the crack, a critical loading condition equivalent to the edge loading may result. The ILLI–SLAB computer program was used to predict the response of the cracked pavement because this program can indirectly model the cracks of different load transfer efficiencies by specifying different aggregate interlock factors. The maximum flexural tensile stress occurs when the load is adjacent to the crack (*6*). This critical stress is at the bottom of the stabilized material layer and acts parallel to the crack. Thompson et al. (*7*) suggest increasing the stress calculated for the interior loading case by a maximum of 50 percent to account for the edge loading conditions caused by cracking. To limit the early life fatigue consumption, it was recommended (*8*) providing adequate thickness or strength, or both, to limit the stress ratio to less than 0.6 to 0.65 before traffic loading.

Stabilized bases continue to gain strength with time. It can be conservatively estimated that portland cement stabilized bases will realize at least a 50 percent strength gain beyond the 28-day strength. Therefore, stress ratios for thickness design based on flexural fatigue will continue to decrease with age and additional strength gain.

One way to account for the reduction in stress ratio due to continued strength gain throughout the life of the stabilized base is to reduce the value of the traffic growth factor to account for the structural benefits of strength gain.

TABLE 2 TxDOT Specification for Gradation Requirement for Type-C Base Material

Sieve Size	Percent Retained	Percent Passing
45 mm	0 - 10	90 - 100
#4	45 - 75	25 - 55
#40	55 - 80	20 - 45

PROCEDURES TO ESTIMATE MODULI

Mechanistic design procedures use layer moduli or strength, or both, which may be based on laboratory and field tests. In the past decade, several researchers have raised concerns over the comparisons of laboratory-measured moduli with field-measured moduli, and it has been reported that they show little or no correlation. Houston et al. (*9*) explained in detail the advantages and disadvantages of laboratory and field testing procedures and recommended field testing. Nondestructive testing using the falling weight deflectometer (FWD) was selected for this study because of its very low operational costs and test efficiency, the nondestructive nature of the testing, and that it represents in situ conditions.

Field Testing

The following protocol of field testing was followed at all seven pavement sections investigated during this study:

1. A representative 150-m-long section was selected, and the pavement temperature was measured at a depth of 25 mm below the asphalt surface.
2. A visual survey of cracks and the condition of the pavement was conducted, and the width of the crack openings and their location on the asphalt concrete surface were recorded.
3. Samples of base, subbase, and subgrade were collected. Only the cores of the base could be obtained in an undisturbed state.
4. FWD and Dynamic Cone Penetrometer (DCP) tests were conducted.
5. The pavements were visited twice during the study period to investigate and record the seasonal effects on pavement strength.

FWD Testing

The FWD drops a weight on the pavement surface generating an impulse load, and the deflections are measured with geophones. One geophone is located directly under the load, and six others are spaced outward from the point of loading at 0.30-m increments (Figure 1). The weight and distance of drop were adjusted to simulate the movement of the 80.1-kN axle.

The FWD load-deflection measurements were made at approximately 15-m intervals on a representative 150-m-long section of the road. Deflection basin measurements were obtained in the outer wheelpath with the load and sensors located between transverse shrinkage cracks, which is shown as configuration A in Figure 2. The LTE of transverse shrinkage cracks was also measured by placing the FWD load plate and associated sensor or geophone across the crack from the other sensors (geophones), which is shown as configuration B in Figure 2. This configuration was used by Uzan (*10*). In this configuration the sensor positioned on the loaded side (the sensor at the center of the plate) is about 150 mm from the crack. The other sensors are positioned on the load-free side of the crack, and one of these sensors was positioned approximately 150 mm from the crack.

DCP Testing

The DCP, shown in Figure 3, was used to measure the in situ strength conditions of the stabilized layers with depth and also to help verify the presence of these layers. The test consists of driving a penetration cone through the pavement layers using a known weight dropped through a fixed (constant) height and thereby maintaining constant energy for each blow (drop of the weight). The basic philosophy is that stiffer (stronger) layers offer more resistance to

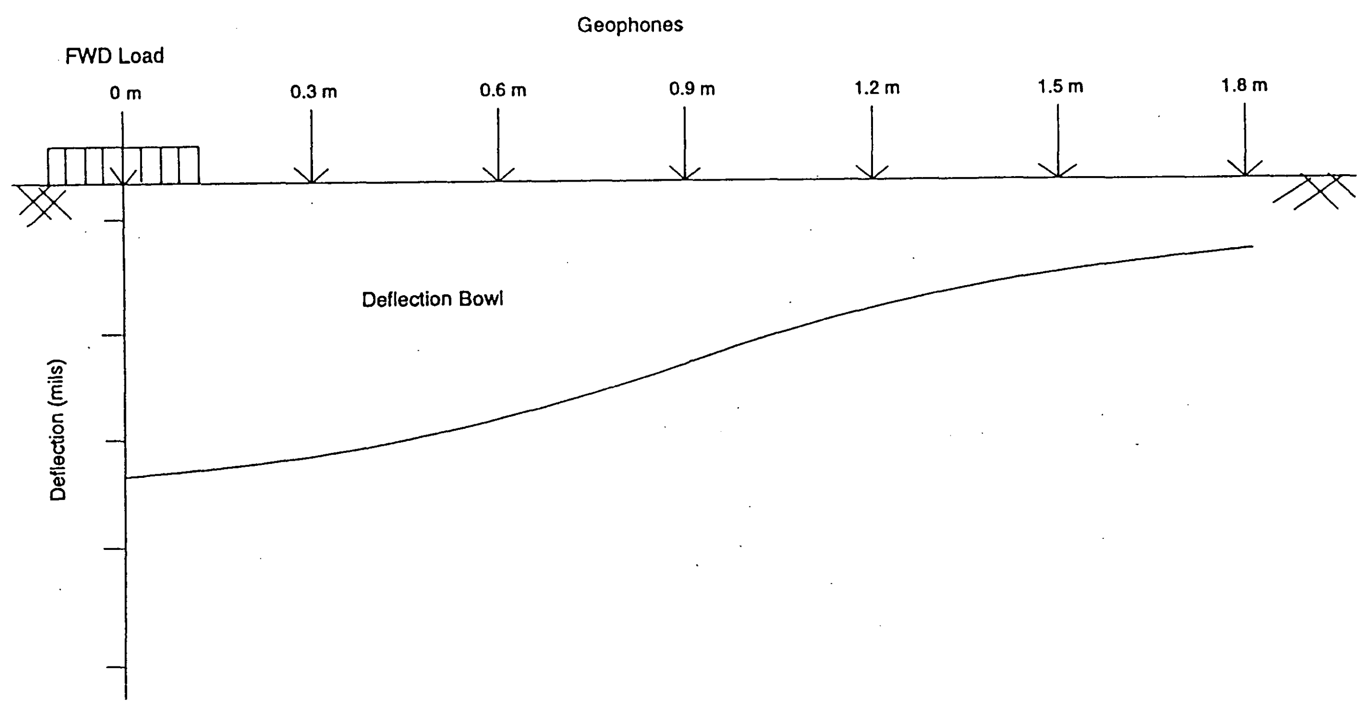

FIGURE 1 Geophone arrangement in FWD.

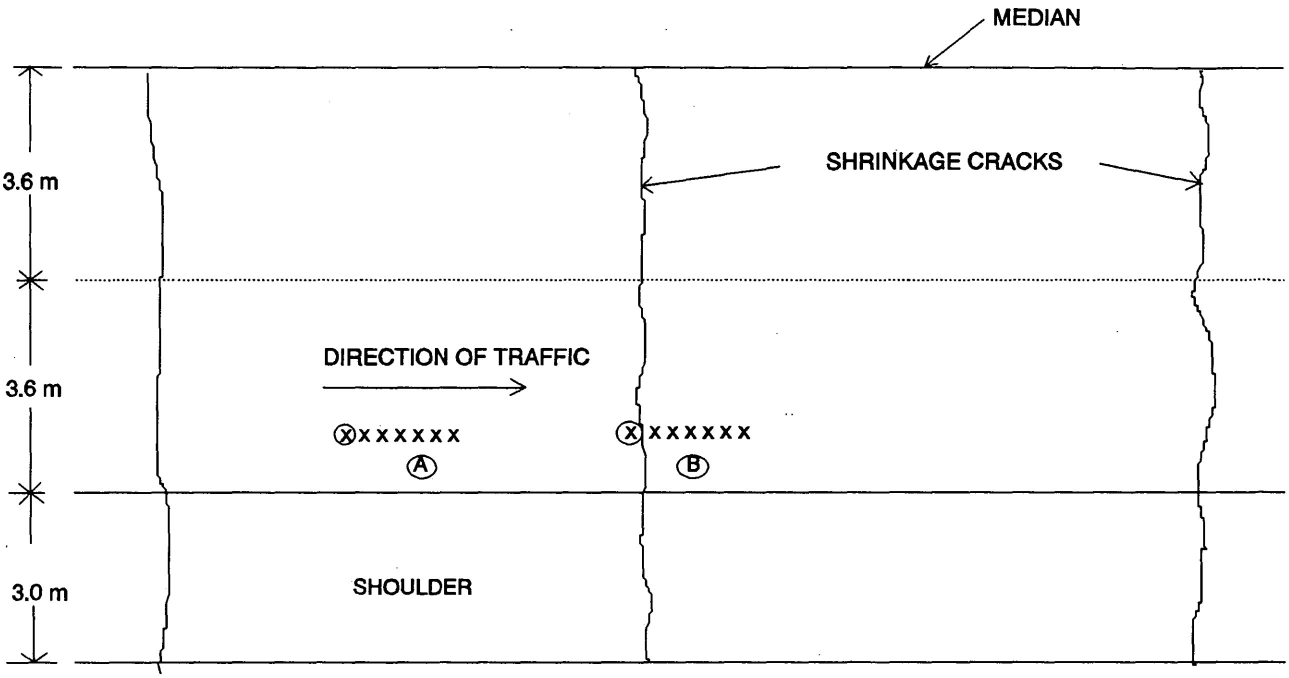

FIGURE 2 Load and geophone positioning for (*a*) interior loading case, (*b*) load-transfer case.

cone penetration, so the average depth of penetration will be lower than that for the softer layers. The rate of penetration of the cone has been correlated with the California Bearing Ratio (CBR). The DCP is an excellent tool for routine pavement evaluation and is the only test available that measures both layer thickness and relative layer strength. The DCP also is excellent at complementing FWD data collected from a test section. In instances where the engineer conducting the backcalculation does not know the actual layer thicknesses or whether a stabilized subbase is present, the DCP data can be used to supplement this information. A schematic of the number of blows versus depth of penetration is shown in Figure 4. The slope of the line is used to estimate the layer CBR, and the intercept of the upper and lower layer slopes is a measure of the layer thickness.

Laboratory Testing

The undisturbed cores of stabilized base were subjected to resilient modulus testing in accordance with Standard Test AASHTO–T274–82 (Standard 1986). In this procedure, a cylindrical test specimen is subjected to a pulsating axial load, and the recoverable axial strain was measured after a specified number of load repetitions. Because the testing is for a bound material, no confining pressure is applied. A preconditioning loading was applied to eliminate the effects of sample disturbance, which causes plastic strains to develop initially and then diminish. The cores were finally loaded to failure to determine the unconfined compressive strength.

Estimation of Layer Moduli-FWD Analysis

Asphalt Concrete

Because the modulus of asphalt concrete changes with temperature, measured pavement temperatures were used to estimate the stiffness of HMAC based on a model developed by Scullion (*11*). The asphalt layer modulus determined in this manner was corrected in the backcalculation process.

Stabilized Base

The layer moduli of the stabilized base, subbase, and subgrade were backcalculated using the MODULUS 4.2 backcalculation computer program developed by Uzan et al. (*12*). The results of the backcalculated layer moduli and laboratory testing are tabulated in Table 3.

Estimation of Load Transfer Efficiency

The deflection bowls measured across the cracks were used to calculate the LTE, which is defined as the ratio of second sensor deflection to the first sensor deflection. This ratio can never be one, even if there is perfect load transfer (LTE = 100 percent) because the sensors are at different distances (0 and 0.3 m) from the center of load drop. To account for this, the LTE ratio is divided by a similar ratio obtained for an interior loading case (without a crack) to determine the actual LTE.

Analysis of Pavement Sections

Two computer programs—BISAR and ILLI–SLAB—were used to analyze the pavement sections and predict the structural response. BISAR is a general-purpose layered elastic program for computing stresses, strains, and displacements in elastic multilayered systems subjected to one or more uniform loads (*13*). ILLI–SLAB, is a two-

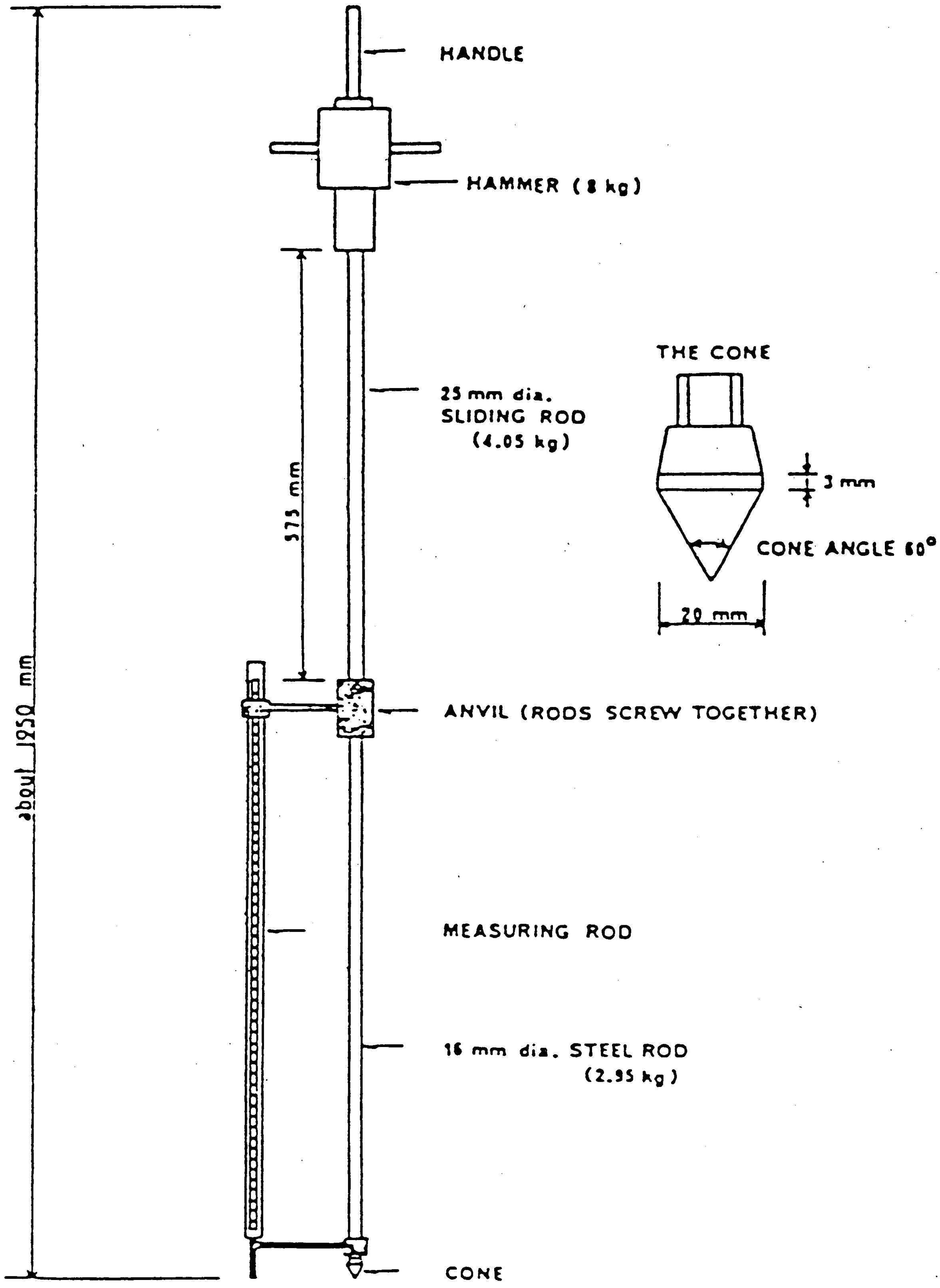

FIGURE 3 Dynamic cone penetrometer.

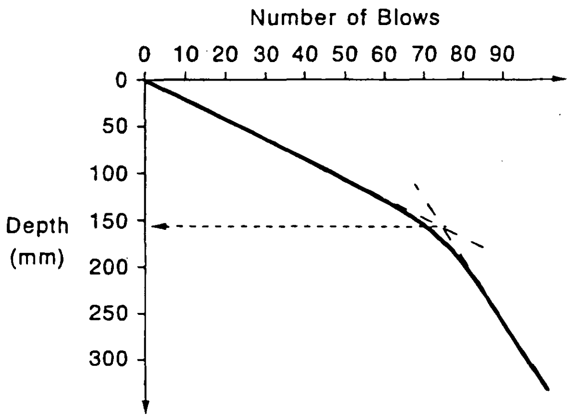

FIGURE 4 Diagram of number of blows versus depth of penetration.

dimensional, finite element, multislab rigid pavement model that considers joints and cracks (*14*).

BISAR cannot model the postcracked phase because it was developed for the interior loading case only. The elastic modulus in the vertical direction is unaffected by the cracks, but the elastic modulus in the horizontal direction is affected and the pavement is no longer isotropic. ILLI–SLAB, was used to investigate the effect of the presence of cracks on pavement response.

The pavement sections were analyzed to determine their fatigue lives, closely following the conceptual methodology of Thompson et al. (*8*). The maximum flexural stress for a given base thickness was computed for an interior loading condition. But it was found, based on ILLI–SLAB analysis for different LTEs, that the maximum stresses corresponding to the postcracked situation were as much as 2.0 times the stresses obtained for the internal (uncracked) loading condition, depending on the effectiveness of load transfer at the crack. The fatigue life was predicted based on the relationship between the stress ratio (defined as the ratio of maximum postcracked stress to flexural strength of the material) and the fatigue life. The compressive strengths of the cores tested in the laboratory were used to calculate the stress ratio. The analysis showed that, using the interior loading analysis, all the pavement sections were fatigue resistant and should not exhibit load-associated damage unless a significant LTE reduction occurs as the result of shrinkage cracking.

OBSERVED PERFORMANCE

The pavements examined in this study have been in service from 3 to more than 7 years as of summer 1994. Transverse shrinkage cracks of different widths were observed in all the pavement sections, except Section 7, which did not show any visible distress. None of the pavements showed any visible fatigue failure. Typical asphalt layer summer temperatures were from 95°F to 115°F, and winter temperatures ranged from 25°F to 50°F. An FWD evaluation was performed along with the crack survey at each site to determine changes in LTE and modulus values with seasons.

The FWD deflection basins demonstrated that the structural capacity (determined based on maximum deflection and area within the basin) of all the pavements did not change from season to season. The variation in moduli of the bases relative to season were obtained from backcalculations from FWD response as tabulated in Table 3. Crack surveys included the location, length, and width of cracks. The crack widths and lengths are measurements that are obtained only on top of the HMAC layer, and they may not be the same as the crack as it exists in the base.

Cracks were classified into three main groups, hairline crack (difficult to measure width), moderate crack (width less than 2.5 mm), and severe crack (width greater than 2.5 mm). Cracks with widths greater than 6 mm were also noticed in Section 6. Crack spacing varied from 4.50 to 24.5 m, with the wider cracks corresponding to increased spacing. Table 4 summarizes data from the more severe cracks and the associated LTE across these cracks. A typical transverse crack is shown in Figure 5. Limited longitudinal cracking appeared in both inner and outer wheelpaths.

The LTE values decreased in winter as expected because of increased crack widths at lower pavement temperatures. Load transfer values varied from 35 to 97 percent, with an average of 70 percent in winter. A typical LTE value for cracks wider than 2.5 mm

TABLE 3 Summary of FWD and Laboratory Test Results

Section #	Modulus of Base (GPa)			UCS of Cores (MPa)
	Back-Calculated from FWD		From Lab Testing	
	Summer	Winter		
1	34.45	33.08	14.48	16.06
2	1.57	3.42	2.54	9.67
3	18.65	14.65	8.86	23.33
4	31.34	20.91	8.78	8.33
5	23.74	26.54	21.90	17.43
6	32.60	28.13	26.14	13.75
7	5.21	-	2.82	11.69

TABLE 4 Summary of LTE and Crack Length (Severe Type)

Section #	LTE (percent)						Crack length[@] (meter)	
	Summer			Winter				
	Min	Max	Avg	Min	Max	Avg	Summer	Winter
1	47.9	94.1	73.5	35.5	70.2	56.8	16.5	22.9
2	-	-	-	81.1	81.1	81.1	-	1.5
3	67.5	95.9	80.3	53.0	88.6	76.9	15	18.9
4	73.7	91.5	87.0	66.4	88.3	75.4	5.8	15.5
5	68.7	97.6	89.6	48.8	77.5	64.2	76.3	77.0
6	63.2	91.6	83.3	36.9	90.7	60.1	30.5	30.5
7	-	-	-	-	-	-	-	-

[@] Length of severe type cracks (crack width > 2.5 mm) in 150 meter long section

was approximately 55 percent. The maximum pavement temperature difference between the winter and summer visits was only 22°F. The LTE values presented in Table 4 could be significantly lower under more extreme winter conditions and for colder climates. It is interesting to note that based on the last visit, new cracks were still developing in all the pavements, except in Sections 6 and 7. Some of the transverse cracks that were present for only a portion of the width of the pavement propagated to the full width of the pavement. Section 5, which had river gravel in the stabilized base, showed significantly more cracking. One of the best performing sections was the lightly stabilized lime-treated base. Although the in-place strengths and stiffnesses were relatively low (compared with the CTB), the section showed no significant cracking and the highest LTE.

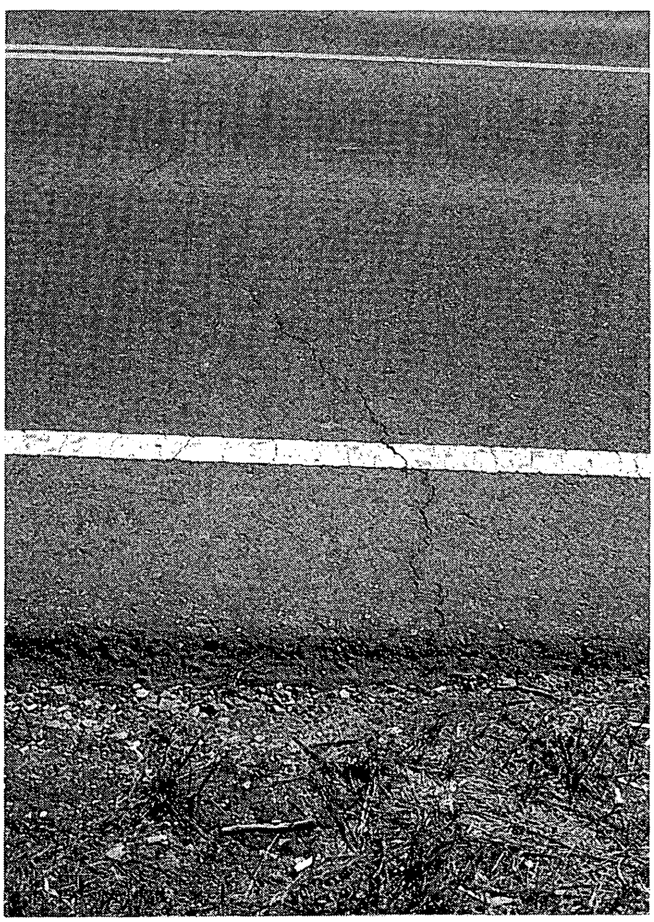

FIGURE 5 Typical transverse crack.

After 3 years in service, Section 6 showed significant distress with pumping observed at many transverse shrinkage cracks. The distress took the form of transverse depressions in the wheelpath, approximately 300 mm wide initially, but eventually covering the entire travel lane. The depressions were centered on existing reflection cracks. The riding quality was noticeably reduced, and the section rides like a faulted concrete pavement. Coring indicated that the CTB layer was disintegrating in the problem areas. The primary cause was water entering the cracks and getting trapped under the base. Under the action of traffic, the hydraulic forces of pressurized water moving under the base caused the erosion and loss of fines through pumping. The failure was so serious that a separate investigation is under way to identify the causes of the premature distress and to make recommendations on how to avoid such failures in the future.

It is clear that the formation of wide shrinkage cracks was the origin for the observed distress. Cores collected in the distressed areas show clean coarse aggregate with no fines. On the other hand, cores collected in adjacent areas were intact, and unbroken with high average resilient moduli (26.14 GPa) and unconfined compressive strength (17.43 MPa) values. Figure 6 shows a core hole where solid cores were obtained. Figure 7 shows the core hole in the distressed area. Disintegration of the CTB can be clearly seen from this figure. It is interesting to note that even though the strength results obtained from solid cores suggest that the pavement is safe from fatigue, the pavement failed in a few years. The reason for this is that the mechanistic design procedures satisfied the interior load-based fatigue failure criteria and not the critical edge-loading condition. The findings here are consistent with the concern that poor performance of stabilized layers is caused by increased pavement deflection, decreased load transfer at cracks, and increased potential for subgrade erosion and pumping at transverse cracks (*8*).

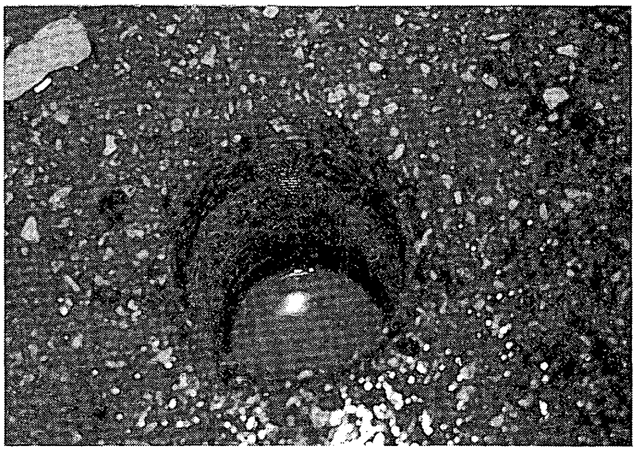

FIGURE 6 Core hole in solid area (intact cores were obtained).

DISCUSSION OF RESULTS

FWD test results and unconfined compressive strength test results from cores suggest that the stabilized base in pavement Section 6 has high strength but is still performing poorly because of wide shrinkage cracks. The performance of these pavements could have been better had the width of the shrinkage cracks been moderate to small.

Shrinkage crack widths can be reduced primarily by reducing the amount of the fine material and the amount of stabilizer used. An improved mixture design with more stringent limits on the amount of fine material or cement, or both, should help reduce transverse cracking and improve performance. Reducing the amount of stabilizer will lead to a reduction in the strength development, but the observed strengths of the pavements are so high that a reduction in strength requirement can still be permitted without failing to meet the fatigue criteria. Hence modifications to and possible reduction of the current strength requirement of 4.48 MPa based on 7 days of moist curing and incorporating shrinkage testing of the stabilized base material is being further investigated through field trials.

As explained before, the reduction in the stress ratio due to long-term strength gain of the stabilized material can be incorporated indirectly into the design by applying a reduction to the traffic growth factor used to calculate design equivalent single axle loads.

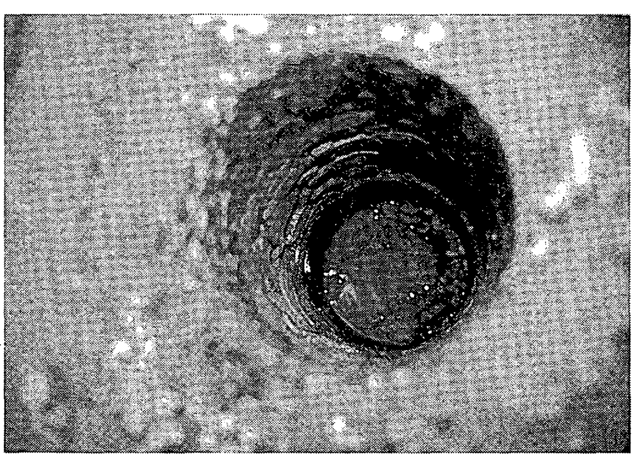

FIGURE 7 Core hole in distressed area.

CONCLUSIONS

- Transverse shrinkage cracks with widths greater than 2.5 mm significantly affect pavement performance. Minimum and typical load transfer efficiencies of 35 and 55 percent were noted for these wide cracks. These LTEs may even be less in lower-temperature periods of winter.
- Formation of wide shrinkage cracks in the pavement increases the critical flexural tensile stress for design by as much as two times the flexural (tensile) stress calculated for interior loading condition. A correction factor of two is recommended for design.
- New cracks were still developing in all the pavements even 7 years after construction, and cracks present for a portion of the width of the pavement continue to propagate to the full width of the pavement.
- The unconfined compressive strength of stabilized base cores show that all the pavement sections tested had strengths in excess of the existing TxDOT minimum strength requirement of 4.48 MPa.
- Excessive transverse shrinkage cracking and pumping were observed as the primary mode of distress in the pavements evaluated.
- Performance of pavement Section 6 shows that the design of stabilized bases based only on fatigue criteria may result in permitting development of very high strengths in the stabilized bases, which may lead to premature failures. The strongest sections may not be the best performing sections.
- It is important to understand the limitation of the current mechanistic design procedures for stabilized bases because they may not always result in best performing designs. Performance of the stabilized pavements can be improved by additional considerations that lead to the reduction of the formation of wide shrinkage cracks.
- Bases with lower levels of stabilizer or that are less rigidly stabilized may perform better than those with higher stabilizer content. This point is illustrated by pavement Section 2 (Table 1). This pavement includes a recycled base stabilized with 4 percent lime. This layer provided adequate strength for durability as is evidenced by the level of performance and adequate stiffness for load distributing purposes. However, because the lime does not produce a rigid stabilized layer, shrinkage cracks are minimal.

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