Joshi, R.V., and Zornberg, J.G. (2011). "Use of Falling Weight Deflectometer Data to Quantify the Relative Performance of Reinforced Pavement Sections," Proceedings of the *GeoFrontiers 2011 Conference*, Geo-Institute of ASCE, Dallas, Texas, March 13-16, pp. 4713-4722.

Use of Falling Weight Deflectometer Data to Quantify the Relative Performance of Reinforced Pavement Sections

Joshi, R.V.[1], *S.M. ASCE and Zornberg J.G.*[2], *PhD., P.E, A.M. ASCE*

[1]Research Assistant, Department of Civil, Architectural and Environmental Engineering, The University of Texas at Austin, 1 University Station, C 1792, Austin, Texas 78712; PH (512) 963-3965; email: rutuparnacivil@gmail.com
[2]Associate Professor, Department of Civil, Architectural and Environmental Engineering, The University of Texas at Austin, 1 University Station, C 1792, Austin, Texas 78712; PH (512) 232-3595; email: Zornberg@mail.utexas.edu

ABSTRACT

Falling Weight Deflectometer testing was conducted in a Farm-to-Market Road (FM 2), located in Grimes County, Texas. The pavement involves eight different experimental cross sections including control section, base reinforced with 3 different geosynthetic products, lime stabilized subbase and combinations of the latter two. The subgrade is high plasticity clay, which, due to moisture fluctuations subjects the flexible pavement to environmental loading. The objective of this paper is to characterize the possible benefits of the 3 types of geosynthetic reinforcements as well as of lime stabilization and quantify the relative benefits of each stabilization technique. Deflection measurements from 9 field trips conducted from February 2006 to August 2009 are evaluated. Modified deflection basin parameters (DBPs) are defined to identify layer properties and are used to assess the relative damage to the base, subbase and subgrade for different sections. The variations in the DBPs over three periods of wetting and drying are presented along with analysis of the observed trends. In addition, condition survey was performed, during 3 years, to visually identify distresses in various sections. Thus, the deflection data analyses complemented by visual observations reveal the relative field performance of different geosynthetics and throws light on the relative merits of base reinforcement against lime stabilization.

INTRODUCTION

For the past three decades, geosynthetics have shown to significantly improve the performance of pavements on weak subgrade. The benefits induced by the use of geosynthetics have been attributed to an increased initial stiffness, decreased creep, increased tensile strength, inhibited crack initiation and propagation, improved cyclic fatigue behavior, and lower overall life-cycle cost (Al-Qadi et al. 2003; Berg et al. 2000). Researchers have focused on quantifying the life cycle cost and increasing the benefit-cost ratio, although significant attention is still needed to describe the interactions that govern the complex behavior of the reinforced pavement system. Despite a consensus that geogrids and geotextiles are beneficial, the mechanism of pavement reinforcement has been complicated by the non-linear behavior of pavement system and by the dynamic nature of the applied vehicular loading (Haas et al. 1988; Al-Qadi et al. 2008). Full understanding of the short-term and long-term field performance of reinforced pavements under continued traffic and cyclic environmental loading has not been accomplished so far.

Quantification of the effect of geosynthetics in pavements can be achieved by monitoring instrumented full-scale pavement sections (Al-Qadi et al. 2008). This approach is an important tool for measurement of pavement response to traffic and environmental loading. Accordingly, many researchers have resorted to full-scale experimental and accelerated pavement testing. The quantification of the relative benefits of different types of reinforcement is needed for practical applications. Further, evaluation of the benefits and comparison of chemical stabilization (e.g. lime treatment) with mechanical stabilization (e.g. geosynthetic reinforcement) for pavements on soft soils has not been reported in the literature. Specifically, for low-volume pavements no design methodology incorporating geosynthetic basal reinforcement is available.

Consequently, the present research aims at gaining insight into the field performance of geosynthetic-reinforced pavements. It involves full-scale testing of an experimental pavement section. Field performance was quantified using Falling Weight Deflectometer (FWD) data, drilled cores, moisture sensors, and visual observation during condition surveys. In addition, an index of pavement performance was developed. This paper discusses the deflection basin parameter (DBP) approach applied to the FWD data obtained for FM 2.

PROJECT DESCRIPTION

AASHTO defines very low-volume roads as those with average daily traffic (ADT) < 400. Functionally, low-volume roads provide access to residences, farms and abutting properties. FM 2 can be classified as a low-volume road with ADT of 800 in 2002 and expectancy of 1300 vehicles in 2022 with only 6.6 % trucks and speed limit 55 mph. It is located in Grimes County, Texas. In January 2006, TxDOT supervised the reconstruction of FM 2. An experimental program was implemented with the objective of evaluating the performance of control sections against geosynthetic reinforcement and lime stabilization. A total of 8 sections with different reinforcement/lime stabilization schemes were constructed. Control sections, lime-only sections, sections reinforced (base) with 2 types of geogrids (GG1 and GG 2) and with a geotextile (GT), as well as combinations of lime treated subbase (LT) and 3 reinforcement types were adopted. To account for the variation in the field such as construction quality differences, defects in compaction, inhomogeneous lime mixing and site topography, 4 repeats of each test section were constructed. Therefore, a total of 32 test sections (4 reinforcement types x 2 stabilization approaches x 4 repeats) were constructed in FM 2. With regards to the available length for the experimental sections, each section was chosen to be 137 m (450 ft) long.

Weather data for FM 2 was obtained from the nearest weather station at College Station. The average high temperature was around 35.6 °C (96 °F) in August and average low temperature was 11.7 °C (53 °F) in January. The average annual precipitation was 1013 mm, with high rainfalls in May, June, September and October. The site showed two dry seasons and two rainy seasons in a year. Borings in FM 2 indicated the presence of high plasticity red clays (PI = 35) in some locations and very high plasticity black clay (PI = 50) in other locations. Moisture sensors were installed at 7 locations and consisted of horizontal and vertical moisture sensor profiles. A horizontal array of sensors was installed to assess the lateral migration of water under the pavement, while a vertical array was installed to assess moisture fluctuations in the soil profile without the influence of the pavement.

FWD TESTING

As a part of the post construction monitoring and assessment, 9 FWD testing surveys were performed over 3.5 years, thus encompassing all the seasons, in all the sections in FM 2. The tests were conducted in February 2006, August 2006, November 2006, February 2007, April 2007, June 2007, May 2008, February 2009 and August 2009. The typical distance between two consecutive FWD test stations was 15.24 m (50 ft). With a total length of 137 m (450 ft), each experimental section included nine FWD test stations. At each test station, 4 levels of dynamic loading were applied. Deflection measurements for 9000 lbs load were considered for the present study.

Initially, the traditional modulus determination approach was undertaken. It was noted that the Modulus 6.0 program does not account for the inclusion of geosynthetic in the pavement. Extensive moduli determination was carried out for all the sections in FM 2. It was consistently observed that the moduli for layers of the pavement in the same section varied significantly. Also, the control sections physically exhibiting distresses showed higher layer moduli than the reinforced sections. As noted in the technical literature (Mehta et al. 2003; Roque et al. 1997), the moduli back calculation process does not yield unique solutions and is heavily user dependent. Several combinations of the moduli values match the measured deflection basin to the calculated one, leading to unexpected and misleading results. Consequently, the quantification of modulus was deemed unsuitable to identify the relative benefits of reinforcement and lime stabilization.

Deflection Basin Parameter Approach

The DBP approach involves analyzing the deflection basins to assess the condition of and the distresses in different pavement layers (Xu et al. 2002; Gopalkrishnan 2004). The relationships between the FWD deflections and pavement layer condition indicators have been recognized by Kim et al. 2000. As indicated by Horak and Emery (2006), DBPs are commonly used in South Africa for individual layer strengths determination and to assist in 'pinpointing rehabilitation needs'. Typically, DBPs are classified into three categories viz. slope, curvature and area parameters. Correlations of the DBPs with layer properties are described in detail by Kim et al. 2000 and Xu et al. 2002.

The DBP approach was applied to deflections measured in FM 2. It was recognized that the configuration of the sensors for FWD equipment used in this project involved 7 sensors each 30.48 cm (12") apart (D_0 to D_6). The geometry of the pavement structure at FM 2 included a 2.54 cm (1") thick HMA layer underlain by a silty gravel base 17.8 cm (7") thick, underlain by 25.4 cm (10") thick subbase followed by high PI subgrade. Consequently, conventional DBPs were modified for the specific configuration of the pavement section in FM 2. D_0, Surface Curvature Index (SCI), modified Base Damage Index (BDI), modified Base Curvature Index (BCI) and Area Index 4 (AI_4) were used as layer indicators for the entire pavement, HMA layer, base layer, subbase layer and subgrade respectively. BDI and BCI were defined to identify the strength of respective layers as depicted in Figure 1. Likewise, $AI_4= (D_5+D_6)/2*D_0$ was used to assess the strength of the subgrade. In general, the higher the deflection values and the DBPs, the weaker the pavement layers.

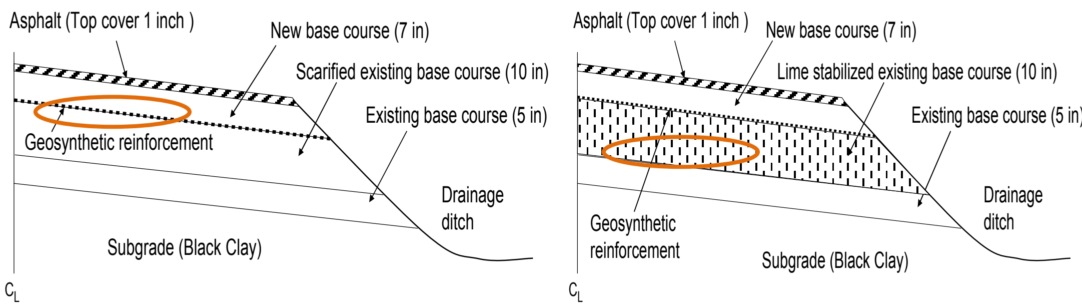

Figure 1(a) Modified BDI=D_1-D_2 used as base strength indicator (b) Modified BCI=D_4-D_5 used as subbase strength indicator

The deflection data of all the seven sensors at nine equidistant locations per section for 32 sections were evaluated. The DBPs for 7 locations in each section were averaged to get the averaged DBPs for that section for a given field trip (FT). Following a similar trend, the DBPs for all the 32 sections for each (FT 1 to 9) were determined. It should be noted that only the central 7 (out of 9) deflection measurements were considered to minimize possible edge effects.

DEFLECTION DATA ANALYSIS

D_0, the centerline deflection for 16 sections on the eastbound lane for the first and last FWD tests conducted at the site is shown in Figure 2. The data shows that the pavement structure has variable properties as indicated by the variability of D_0 along the various sections in the first reading (Feb 2006). Increase in the measured D_0, for 12 out of 16 sections, in the last reading (Aug 2009) indicates that deterioration has occurred in the pavement. 4 sections (3600 to 5400 ft viz. station 185 to 203) which exhibit a decrease in D_0 (and hence an increase in the stiffness of the pavement structure) were both lime treated and reinforced sections. A similar plot for the DBP-AI_4 indicates that the subgrade for the same 4 sections was severely deteriorated. It should be noted that boring at station 185 and 194 confirmed presence of high PI black clay.

Thus, the variation in D_0 and AI_4 indicates that the entire pavement structure showed a slight improvement, even though the subgrade showed deterioration. This clearly suggested that the reinforcement as well as the lime stabilization contributed to an enhanced pavement performance. The variation in the averaged BDI for the first (Feb 2006) and last (Aug 2009) surveys indicated a maximum damage of 280% in the control section (Figure 3). Subsequent in the estimated damage were the non-lime treated (NL) section reinforced with geotextile, NL geogrid1 (GG1), and NL GG2.

The data reported in Figure 3 is indicative of the pavement condition at two points in time. The pavement was subjected to 3 cycles of wetting and drying during this period. To better understand the response of the pavement and the variation in the layer properties, between these two points in time, the variation of DBPs for all the 9 field trips was evaluated. To account for the initial variability in the measurements of

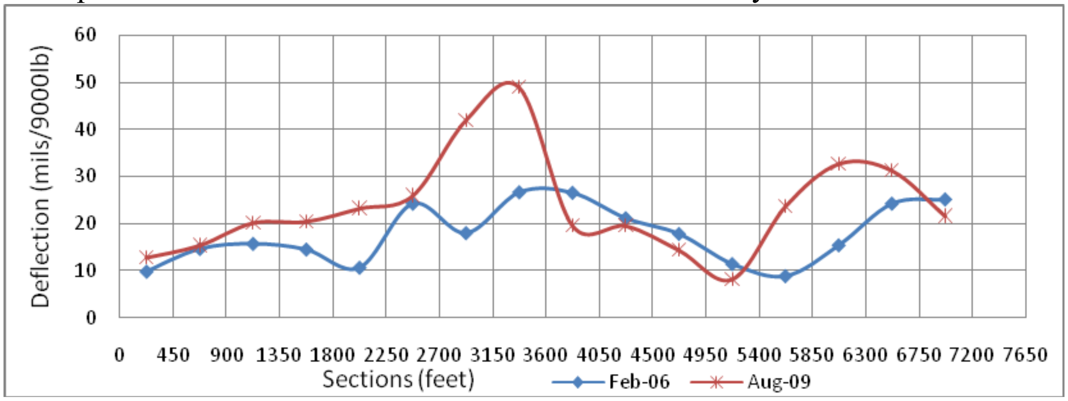

Figure 2. Comparison of average D_0 for the first and latest FWD testing surveys

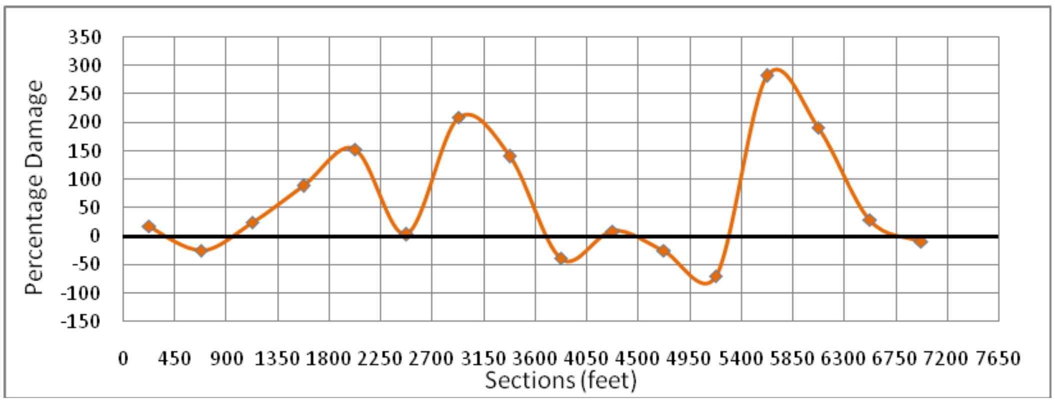

Figure 3. Percentage Damage in the base for 16 sections

DBPs, all the DBPs were normalized with respect to their values as obtained in the initial field trip (Figures 4 through 8). It should be noted that, normalizing the DBPs implies that the values of DBPs greater than one indicate degradation and lesser than one implies improvement as compared to the threshold behavior.

D_0 represents the strength of the entire pavement section. Figure 4 shows the variation of the normalized D_0 for various sections. Evidently, the control section 1Ea showed significantly deterioration, as the value of normalized D_0 was consistently above 1, reaching 260% damage in Aug 2009. The other 3 control sections (not shown in plot) also showed degradation and similar variation in D_0. Further, it was observed that the lime control and GG1 reinforced section depicted about 20% damage/improvement relative to their threshold value in Feb 2006. Sections with both lime treatment and reinforcement (LT GG1 and LT GG2 in Figure 4) were consistently better than their threshold value in Feb 2006.

Variation of BDI indicates the trend in the damage to the base. Accordingly, Figure 5 provides a comparison of the relative improvement in the reinforced bases of the experimental sections. The base layer of the control section was severely degraded, as also the base of GT reinforced section showed higher BDI values. The BDI values for both the GG reinforced sections followed almost similar trend with nearly no degradation for either of the bases. Similar observations were noted for another set of repeats of sections. The base layers were essentially unaffected by the cycles of wetting and drying. This indicates that the geogrid-reinforced sections showed consistently better performance than the control sections and than the geotextile-reinforced sections. Further, GG1 depicted a lesser BDI than GG2.

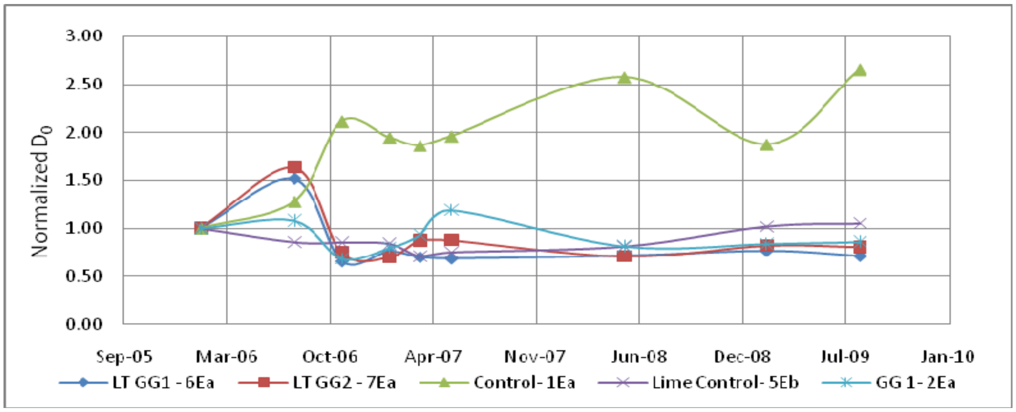

Figure 4. Variation of Normalized D_0 for various sections for 9 FWD field tests

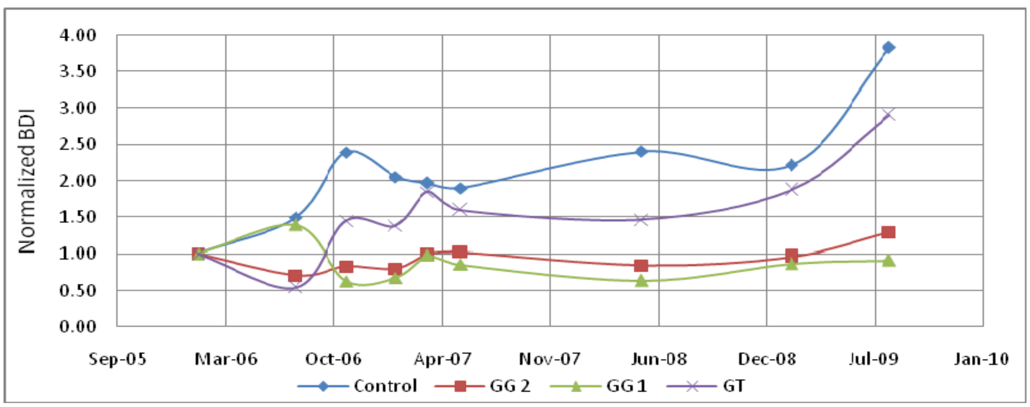

Figure 5.Variation of Normalized BDI for sections with only reinforcement

Figure 6 shows an interesting behavior of the LT reinforced sections on black clay subgrade. This is because there was continuous improvement in the base properties for all the 4 sections. The initial jump in BDI from Feb 2006 to Aug 2006, and in general the irregularities in the DBPs for the first year of measurement may be attributed to the stabilization of the pavement structure and conditioning required for the lime treatment and reinforcement to influence the pavement performance.

Although the magnitude of the BDI showed significant variation, all the sections showed a similar trend, with GG1 and GT depicting the lowest and highest BDI values, respectively. Periodic ups and downs (wave pattern) were observed in the normalized BDI from June 2007 to Aug 2009 for all the sections. When confirmed by the precipitation data during these two years, it was revealed that the trough of the wave corresponded to drying periods while the crest corresponded to higher average precipitation. This indicated that wetting led to infiltration of moisture in the subgrade and pavement layers, thus resulting in softer pavement layers and higher deflections of FWD. On the other hand, drying led to evaporation of moisture from subgrade and decreased moisture content under the pavement, thus resulting in stiffer pavement layers, reinforcement mobilization and lesser deflections. Also, inspection of the variation in BDI and BCI for lime stabilized sections indicated that lime treatment worked better in drier conditions.

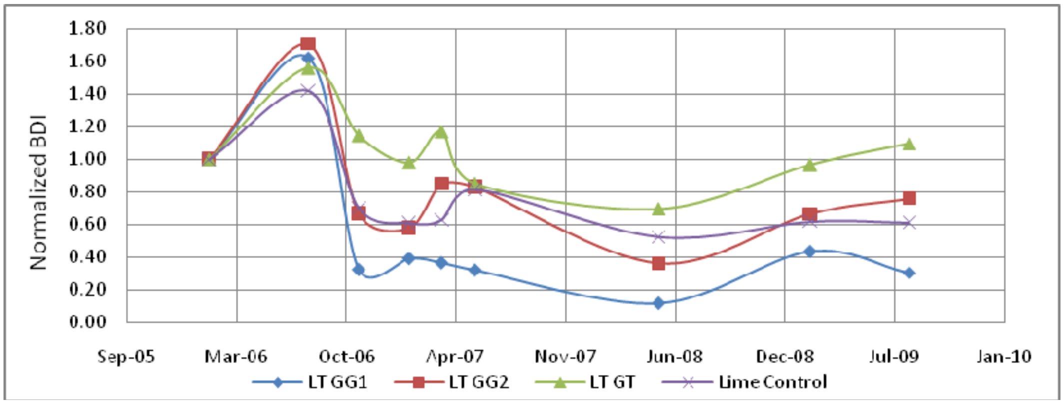

Figure 6. Variation of Normalized BDI for sections with only reinforcement

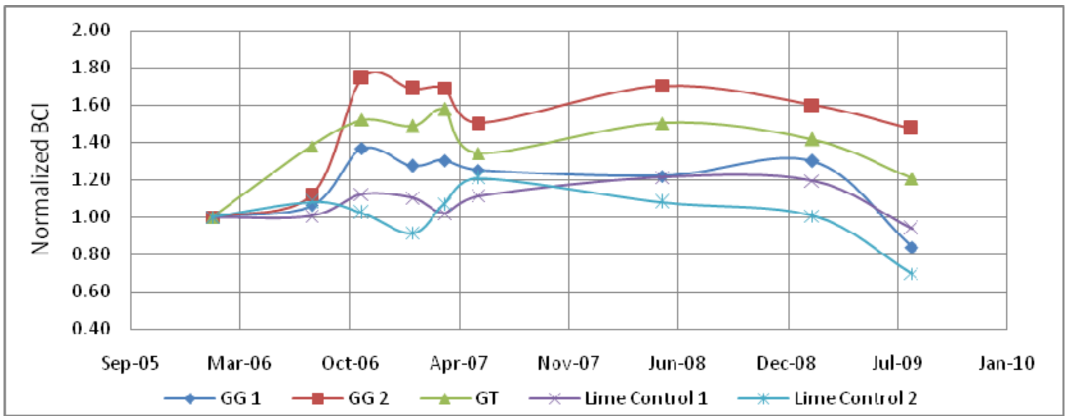

Figure 7. Variation of Normalized BCI for reinforced sections against lime control

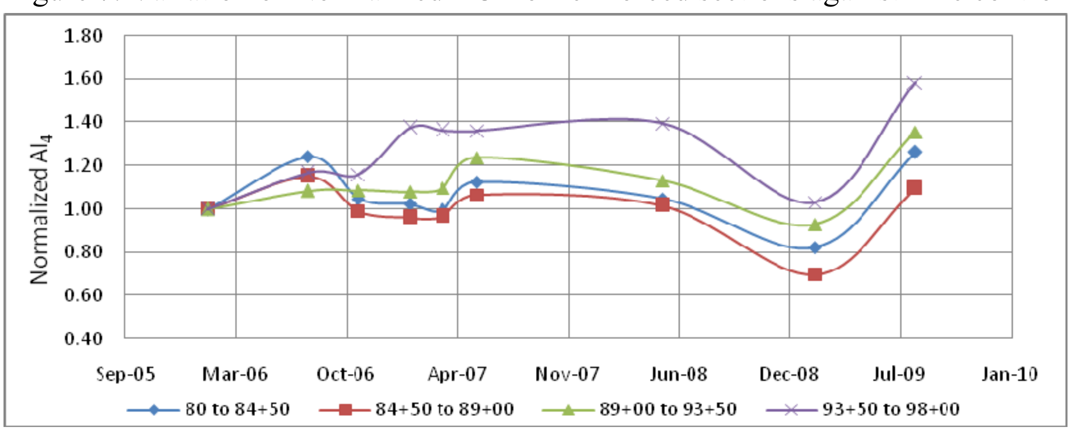

Figure 8. Variation of Normalized AI_4 for four consecutive sections

BCI was defined to capture the effect of lime stabilized subbase. Figure 7 clearly shows that the variation in BCI for both the lime control sections, though physically apart by thousands of meters, is similar and that the values are smaller than those in the non-lime stabilized sections. Also, the consistent decrease in the BCI from May 2007 to Aug 2009 in these sections indicates that the lime stabilization led to a stiffer subbase. Sections with reinforced bases exhibited lower BCI values than the control but higher than the lime treated ones. The BCI for GG1 was found to be comparable to one of the lime control sections. It should be noted that FM 2 is a low-volume road, primarily subjected to environmental loads. In case of higher traffic loads, it is expected that the geosynthetic reinforcement would play a significant role in reducing the distresses acting on the subbase. The geosynthetic is envisioned to act as a lateral restraint to the base as also help to spread the traffic load in a uniform pattern, thus reducing the stresses in the subbase.

Ideally, if the subgrade were the same in all of the sections, there must be a uniform value of AI_4. Figure 8 indicates slight variation for all the sections. Nonetheless, all the sections followed a similar trend. The same observation cannot be made for the other DBPs, which indicates that AI_4 is a good measure of the subgrade properties. When the trend was correlated to the precipitation data it showed that the stable AI_4 from April 07 to May 08 corresponded to an average to low precipitation during that year. Decrease in the AI_4 value for all the sections in February 2009 was attributed to the continuing dry season and an average precipitation of 0.05 inches for the past 13 months. Dry season which in turn implied less precipitation and higher temperatures led to evaporation of water from the subgrade in the shoulder. As a result, medium longitudinal cracking was developed in the shoulders, which was observed during the condition surveys.

CONDITION SURVEYS

A total of 12 condition surveys were conducted since the reconstruction of FM2 until March 2010. The performance of each section was quantified by taking into account distresses like edge cracking, longitudinal cracking, rutting, potholes, weathering and raveling, shoulder drop-off and bleeding. A major purpose of installing the geosynthetics was to mitigate longitudinal cracking occurring due to moisture fluctuations. Hence, it was considered to be the most influential distress and was given a higher weight than, say, bleeding which is a surface defect or potholes which can be easily repaired. Quantification was done by identifying distress severity levels (1 through 9), assigning weighing points (1 through 4) for distresses and calculating the developed index of pavement performance (IPP) for each section in FM 2. IPP was defined as $\sum D_i * W_i$ where D_i is the distress level for i^{th} distress and W_i is the weighing factor for that distress. Lower IPP implies better performance. Averaged IPP score for all the sections in FM 2 is indicated in Table 1.

Condition surveys showed significant distresses, rapid and continued deterioration as also poor riding quality for the control sections. The lime control sections depicted fewer distresses till August 2008. By March 2010, two lime control sections showed severe distress and the other two exhibited significant increase in interconnected cracks. GG1 reinforced sections consistently outperformed the other sections; devoid of any type of degradation in the pavement over three cycles of wetting and drying. In case of GG2 reinforced sections, minor distress was observed in latter surveys. In both GG1 and GG2, small edge cracking running parallel to the pavement centerline was commonly observed. This confirmed the envisioned mechanism of geogrids inhibiting crack propagation from the shoulders into the pavement. Since, the GT rolls were 0.9 m (3 ft) wider than the width of the pavement, GT reinforced sections did not show edge cracking.

Table 1. Averaged IPP for sections in FM 2

Description	Averaged IPP	Description	Averaged IPP
Lime treated GG 1	62.0	Lime treated GG 2	94.5
GG 1	67.8	Lime control	99.1
Lime treated GT	85.0	Control	99.3
GG 2	87.0	GT	119.3

GT sections were more distressed than the GG ones, in that, for hundreds of feet, rutting in GT sections amounted to as high as 3 cm. Figure 9 shows different types of distress observed for various sections in FM 2.

Figure 9. Overview of different distresses observed for various sections in FM 2

SUMMARY AND CONCLUSIONS

The following conclusions can be drawn from the field investigation: (1) Geosynthetic reinforcement led to the improved performance of a low-volume pavement system, reducing the rate of deterioration with time. (2) The DBP approach was found to be effective to identify and quantify the benefit of reinforcement and lime stabilization. On the other hand, modulus approach was found to be unsuitable. (3) The benefit derived from GG and GT reinforcement in the base and lime stabilization of the subbase can be quantified by using the proposed modified BDI and modified BCI, respectively. (4) Sudden jumps in the DBPs, from one point in time to other, can be attributed to high intensity rainfall and the decreases can be attributed to extreme hot and dry weather conditions. (5) Lime treatment indeed exhibits improved subbase properties. Field monitoring indicates delay in onset of distresses. But, relatively increased deterioration as indicated by the high values of IPP, observation of localized distress zones and wearing out of the lime treatment with time raises a question regarding long term serviceability of lime stabilized pavements. (6) Results from the visual assessments, determination of IPP and variations of the DBPs follow a similar trend showing that sections reinforced with GG1 consistently perform better than those reinforced with GG2 or GT. Further, GG2 reinforcement typically exhibited stiffer sections than GT reinforced ones. Continued monitoring is needed to confirm these trends in longer periods of time. (7) GG1 reinforced base section outperformed lime control section.

ACKNOWLEDGEMENT
The authors would like to thank TxDOT for their support in this research.

REFERENCES

Al-Qadi, I.L., Dessouky, S.H., Kwon J. and Tutumluer, E. (2008). “Geogrids in Flexible Pavements: Validated Mechanisms”, Transportation Research Record, *Journal of the Transportation Research Board*, No. 2045, Washington, D.C., pp. 102-109

Donovan, P., Tutumluer, E. (2009). “Falling Weight Deflectometer testing to determine relative damage in asphalt pavement unbound aggregate layers”, Transportation Research Record, *Journal of the Transportation Research Board*, No. 2104, Washington, D.C., pp. 12-23.

Gopalakrishnan, K. (2004). “Performance Analysis of Airport Flexible Pavement Subjected to New Generation Aircraft”, *PhD report*, University of Illinois at Urbana–Champaign, USA.

Horak, E., and Emery, S. (2006). “Falling Weight Deflectometer Bowl Parameters as analysis tool for pavement structural evaluations” *22nd Australian Road Research Board International Conference*, Brisbane, Australia

Joshi, R.V. (2010). “Field Performance of Geogrid Reinforced Low-volume Pavements”, *Masters Thesis*, The University of Texas at Austin, USA, 160p

Kim, R.Y., Ranjithan R.S., Troxler D. J., and Xu, B. (2000). “*NCHRP Research Results Digest 254: Assessing Pavement Layer Condition Using Deflection Data*”. TRB, National Research Council, Washington, D.C.

Mehta, Y., Roque, R. (2003). “Evaluation of FWD data for determination of layer moduli of pavements”, *Journal of Materials in Civil Engineering*, Vol. 15, No. 1, pp 25-31.

Xu, B., Ranjithan, R.S., Kim, R.Y. (2002). “New relationships between falling weight deflectometer deflections and asphalt pavement layer condition indicators”, Transportation Research Record, *Journal of the Transportation Research Board*, No. 1806, Washington, D.C., pp. 48-56

Zornberg, J.G., Gupta, R., Prozzi, J.A., and Goehl, D. (2008). “Case histories on geogrid reinforced pavements to mitigate problems associated with expansive subgrade soils”, *Geoamericas 2008*, Cancun, Mexico