Long-Term Field Evaluation of a Geosynthetic-Stabilized Roadway Founded on Expansive Clays

Gholam H. Roodi, M.ASCE1; and Jorge G. Zomberg, F.ASCE2

Abstract: This paper presents an evaluation of the long-term field performance of Farm-to-Market Road 2, the construction of which involved geosynthetic stabilization to address concerns related to the presence of expansive clays and associated environmental loads. A comprehensive study was conducted to quantify the benefits of geosynthetic stabilization in the performance of the roadway after having been subjected to 9 years of wet and dry season cycles. Control sections and seven design schemes (including various combinations of geosynthetic-stabilized base and lime-stabilized subbase courses) were incorporated in 32 test sections. Evaluation of the development of environmental longitudinal cracks over the 9-year period showed that the use of geosynthetics to stabilize the roadway base led to a significantly improved performance, as quantified based on the extent and length of environmental load–induced longitudinal cracks. The improvement, observed for all the geosynthetics considered in this study, was found to be more significant during dry seasons, which is when environmental longitudinal cracks develop. In addition, results from the field performance monitoring program revealed that lime stabilization of the subbase not only did not help but generally compromised the performance of road sections subjected to environmental loads. DOI: 10.1061/(ASCE)GT.1943-5606.0002206. © 2020 American Society of Civil Engineers.

Author keywords: Geosynthetics; Base stabilization; Expansive soil; Environmental load; Lime stabilization.

Introduction

The presence of expansive soils has been the source of significant problems for a wide range of structures, including foundations, pipes, buildings, roads, airports, and retaining walls. Damage associated with volumetric changes in soils is particularly significant in geotechnical projects, such as roadways, where overburden pressures are comparatively low. Jones and Holtz (1973) and Jones and Jones (1987) reported that the damage caused by expansive soils to structures in the United States exceeds $9 billion annually, with over half of those annual costs being associated with the presence of expansive soils in roadway projects.

The impact of expansive clay subgrades on roadways is critical in states such as Texas, where the majority of the roadway transportation network involves low-volume roads whose design is characterized by a comparatively low structural capacity (Dessouky et al. 2012). The Texas Department of Transportation (TxDOT) has identified longitudinal cracks along roadways as the primary type of damage associated with expansive clay subgrades. Common strategies adopted by TxDOT to mitigate problems associated with the presence of expansive clay subgrades have included the use of chemical (e.g., lime) stabilization of the subgrade or subbase courses, as well as geosynthetic stabilization of the base course.

Significant research has been conducted to understand the impact of traffic loads on the performance of roadways when using geosynthetic inclusions within pavement layers (e.g., Perkins et al. 2010; Zomberg 2017a, b). However, performance quantification is not available to characterize the impact of environmental loads (i.e., those resulting from subgrade volumetric changes) on roadway performance. The case study presented in this paper involves the evaluation of unique long-term data on the field performance of geosynthetic-stabilized roadway sections as well as comparing it against the performance of control sections under actual environmental conditions. Specifically, a total of 32 full-scale test sections, with an average length of 137.5 m each, were constructed using 8 different design schemes, which involved combinations of geosynthetic stabilization of the base course (including different geosynthetic types and control sections without geosynthetics) and lime stabilization of the subbase course (including control sections without lime). Accordingly, one of the design schemes included control sections that did not incorporate geosynthetic or lime stabilization. The long-term performance of the test sections was evaluated following the collection of performance data over a period of 9 years using a comprehensive protocol, established as part of this study, tailored to investigate the field performance of roadways subjected to environmental loads.

Project Description

Farm-to-Market Road 2 (FM2) is a low-volume road located approximately 16 km south of the city of Navasota in Grimes County, Texas. Owing to excessive maintenance costs associated with distresses induced by expansive clay subgrades, full reconstruction was conducted in 2006, involving both a geosynthetic-stabilized base and a lime-stabilized subbase. However, to gain insight that will benefit future projects, the rehabilitation of FM2 included a comprehensive program involving the construction of test sections aimed at evaluating the long-term field performance of a number of rehabilitation alternatives.

FM2 was characterized by an average daily traffic (ADT) of approximately 800 vehicles, as quantified in 2002, with a predicted increase to 1,300 vehicles in 2022. The total number of equivalent 18-kip...
single-axle load (ESAL) was estimated to exceed 91,000 in each direction over a 20-year period (from 2002 to 2022). The road extends 10.3 km from Courtney in the West to FM362 in the East (Fig. 1). Significant problems involving poor ride quality and various types of distresses, particularly in the form of longitudinal cracks, had been reported on a 6.75-km-long portion of FM2 that extended from Highway 6 to FM362. Evaluation of cores collected along the road revealed that the original pavement included a 25-mm-thick asphalt layer underlain by a 380-mm-thick base course constructed with iron ore. Preliminary subgrade soil investigation indicated the presence of expansive clays along significant portions of the road length to be rehabilitated.

A rehabilitation design often adopted by TxDOT for roadways prone to damage induced by expansive clay subgrades involved a combination of chemical stabilization of the subbase and geosynthetic stabilization of the base course. Also, a common practice was to construct the subbase of the new road using materials recycled from the original roadway stabilized with lime or cement (Chen 2007). In this particular project, road design involved placing the geosynthetic over a chemically stabilized subbase and subsequently constructing a comparatively thin flexible base layer and a thin asphalt layer.

Field evidence was available indicating that the adopted road rehabilitation design leads to good performance, based on the experience of several TxDOT districts where expansive clay subgrades are prevalent. However, the adequacy of such practice to mitigate the development of environmental longitudinal cracks had not been quantified. As will be discussed next, and through a collaborative effort with the University of Texas at Austin, a comprehensive field monitoring program involving a series of test sections was implemented as part of the FM2 reconstruction to evaluate the merits of alternative designs aimed at mitigating the detrimental effect of expansive clay subgrades on roadway performance.

**Design of Field Monitoring Program**

An important objective of the field monitoring program was to compare the long-term performance of test sections constructed using different mitigation schemes and subjected to actual environmental loads. Preliminary evaluations of the FM2 subgrade soils indicated a prevalence of highly expansive soils in two portions of the road, which had also been reported to correspond to poorly performing road portions. The experimental test sections were constructed in these two poorly performing road portions (Fig. 1). Specifically, three geosynthetic products were adopted with the objective of comparing their performances when placed over subbase courses to be constructed both with and without lime stabilization.

**Layout of Test Sections**

The rehabilitation of FM2 involved scarification, remixing and compaction of the original base course to form a 250-mm-thick subbase, and subsequent construction of a 180-mm-thick new base layer and a thin asphaltic layer. The original design was modified at the locations of the experimental test sections using three stabilization schemes (with multiple repeats for each scheme), as follows:

1. Geosynthetic stabilization: In this scheme, a geosynthetic layer was placed at the subbase–base interface. Four geosynthetic stabilization configurations were adopted, including the use of two geogrids and one geotextile, as well as the control (i.e., no geosynthetic) sections;

2. Lime stabilization: In this scheme, the subbase layer was stabilized using approximately 5% lime, leading to two different lime-stabilization configurations (i.e., with and without lime stabilization); and

3. Combined geosynthetic and lime stabilization: In this scheme, both a geosynthetic-stabilized base and a lime-stabilized subbase were adopted.

The control sections were used as reference for comparative evaluation of the performance of the test sections designed using the different stabilization schemes. Because three geosynthetic products were incorporated in the various test sections, a total of eight different design profiles were ultimately constructed (Fig. 2). This includes three geosynthetic-stabilized section profiles using the three geosynthetics, referred to herein as Test Section Profiles GS1, GS2, and GS3 [Fig. 2(a)]; three combined geosynthetic and lime stabilization section profiles using the three geosynthetics, referred to herein as Test Section Profiles GS1+LM, GS2+LM, and GS3+LM [Fig. 2(b)]; a lime-stabilized section profile, referred to herein as Test Section Profile LM.
To account for variability due to differences in subgrade soil properties, environmental conditions, construction techniques, and other influencing factors, an average of four repeats were constructed for each section profile. That is, the overall program involved a total of 32 test sections.

As previously mentioned, the 32 test sections were constructed in the 2 portions of FM2 reported to have historically exhibited poor performance. The poorly performing portions of the road originally included test sections with the same length of 137.5 m (16 sections in each direction), using side-by-side arrangement of the 8 various design schemes. Eventually, and partly due to the need to accommodate the road construction sequence, the constructed lengths ranged from 45 to 182.5 m (Fig. 3).

**Performance Evaluation Protocol**

Since traffic and environmental loads result in different types of pavement distresses, it was essential to differentiate the pavement distresses resulting from each load type. Geosynthetics used for base stabilization have been reported to improve road performance under both traffic and environmental loads. However, the mechanisms leading to such improvement in a geosynthetic-stabilized base subjected to traffic loads may differ from those leading to improvement under environmental loads. The extent and severity of longitudinal cracks reveals the level of damage induced by environmental loads, whereas vertical deflection in the wheel path (i.e., rutting depth) indicates the distress induced by traffic loads. Consequently, different criteria should be considered to evaluate the performance of roads under these two load conditions.

Data collected to evaluate roadway performance (e.g., pavement profile, deformations, ride quality, skid resistance, structural adequacy) has often been automated and collected using a number of well-established techniques (e.g., wave propagation, ground-penetration radar, nondestructive pavement deflection tests, laser scanning). However, identification and characterization of surface distresses induced specifically by environmental loads has remained largely visual. In particular, assessment of surface distress relies mainly on observations and measurements collected as part of field campaigns involving condition surveys. Attempts have been made to automate the characterization of pavement surface distress resulting from expansive clays using road surface scanning, but these techniques do not necessarily provide the information needed to accurately describe pavement conditions (e.g., extent of longitudinal cracks). Thus, visual survey of the pavement surface remained the primary method of identifying the various types of distress and to assess their severity.

A specific monitoring program was developed for the FM2 project to evaluate the field performance of test sections subjected to environmental loads. The performance of the FM2 experimental test sections was routinely monitored for over 9 years, from 2006 to 2015. Specifically, the monitoring program included field evaluations conducted both before and after reconstruction. The focus of the evaluations presented in this paper is on the distresses

![Fig. 2. FM2 pavement cross sections: (a) geosynthetic-stabilized sections (GS1, GS2, and GS3); (b) combined geosynthetic- and lime-stabilized sections (GS1+LM, GS2+LM, and GS3+LM); (c) lime-stabilized sections; and (d) control sections.](image-url)
quantified in the experimental test sections that can be attributed to the expansive nature of the subgrade. Specifically, the evaluation presented herein involves three key components, as follows:

1. Characterization of subgrade soils: Since volumetric changes in subgrade soils is the source of vertical differential movements (both settlement and heave), a key considered aspect involved assessment of the variability of subgrade soils along the road. 

2. Evaluation of environmental conditions at site: Monitoring of environmental conditions at the site was also a key component, as recurring swelling and shrinkage of the expansive clay subgrade would not be triggered unless changes in moisture content occurred within the subgrade soils. 

3. Monitoring development and extent of environmental longitudinal cracks: The development of environmental longitudinal cracks was the most relevant distress resulting from the expansive nature of the subgrade. Consequently, a series of detailed condition surveys was conducted to characterize the development, extent, and opening width of the environmental longitudinal cracks in the test sections.

Material Characterization

Field and laboratory investigations were conducted from 2002 to 2012 to characterize the FM2 subsurface soils and geosynthetic materials. Findings from these investigations are discussed next. Additional details regarding the characteristics of the different materials relevant to the reconstruction of FM2 are provided by Roodi (2016).

Subgrade Soil Investigation

A total of 18 borings were drilled to a depth of approximately 2.7 m along the portions of FM2 where the test sections were constructed. Fig. 3 illustrates the location of the borings and the layout of the experimental test sections. The boring labels correspond to the numbers of the test sections where they were located. The maximum distance between consecutive borings was 275 m. Inspection of the soil samples from the various borings revealed a comparatively high variability in the subgrade soils. Atterberg limits and particle size distribution tests were conducted to characterize the subgrade soils collected from each boring. Tables 1 and 2 summarize the information obtained as part of the subgrade soil investigation. Specifically, these tables include the depth of the soil samples collected from 18 borings drilled from 2002 to 2012, along with their Atterberg limits and soil classification (according to the Unified Soil Classification System (USCS)). Inspection of the plasticity index (PI) values summarized in these tables shows a relevant spatial variability in the subgrade soils.

Considering the variability in the subgrade soils, two approaches were implemented to evaluate the swell potential of the subgrade soil at the location of the different test sections: (1) evaluation of the PI values of the subgrade soil, and (2) determination and evaluation of the potential vertical rise (PVR), calculated along profiles from the surface to the bottom of the borings (TxDOT 2014; Zornberg et al. 2017a). While further characterization of the expansive soils was beyond the scope of this study, the two approaches established index values that were deemed suitable for comparative assessment of the performance of the different test sections.

The approach involved classification of the expansive clay subgrades based on their Atterberg limits, with an emphasis on the results obtained for soil samples collected in the top meter of each soil boring. The volumetric changes at a given location depend highly on the characteristics of the surficial subgrade soils because they are the most susceptible to seasonal moisture changes. In addition, volumetric changes are highly influenced by confining pressure, with the low overburden pressure of surficial soils contributing to a comparatively high impact on the total vertical movements. Overall, since shallow soil layers contribute the most to subgrade volumetric changes, the soils in the top 1 m were selected as reference, particularly considering the results of an evaluation of changes in soil moisture content quantified from the soil samples collected from borings excavated in the FM2 subgrade (Zornberg et al. 2008). Following this approach, the subgrade soils at the

Fig. 3. Layout of FM2 test sections, also showing location of subsurface investigation borings and classification of subgrade soil in the test sections based on their expansiveness: (a) Portion 1; and (b) Portion 2.
Table 1. FM2 subgrade soil characterization and predicted potential vertical rise at locations of borings in Portion 1 of test sections

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>B1 (PVR = 117)</th>
<th>B2 (PVR = 36)</th>
<th>B3 (PVR = 78)</th>
<th>B4 (PVR = 68)</th>
<th>B5 (PVR = 78)</th>
<th>B6 (PVR = 78)</th>
<th>B7 (PVR = 60)</th>
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<tr>
<td></td>
<td>Soil class</td>
<td>LL</td>
<td>PI</td>
<td>Soil class</td>
<td>LL</td>
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<td>53</td>
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</table>

Note: PVR = potential vertical rise (values are in millimeters); CH = fat clay; CL = lean clay; SC = clayey sand; and SM = silty sand.
locations of borings were grouped into three classes based on the classification recommended by the Waterways Experiment Station (USACE 1983). Specifically, subgrade soils with a PI below 25 were classified as having low swell potential (low expansive), soils with a PI above 35 were classified as having high swell potential (highly expansive), and those with a PI ranging from 25 to 35 were classified as having marginal swell potential (medium expansive). The liquid limit (LL) values of the surficial soils at locations classified as highly expansive were found to exceed 60, and at locations where the subgrade soils were classified as low expansive, the LL values were below 40.

The second approach involved quantification of the PVR value based on the properties of the soils collected in each boring. The PVR was calculated according to TxDOT Procedure Tex-124-E (TxDOT 2014). Accordingly, the subsurface soil profile was divided into 300-mm sublayers, characterized by an average wet density of 19.65 kN/m³. Consistent with this procedure, the volume changes are predicted considering the changes in moisture from the soil’s “dry” to its “wet” conditions (based on Tex-124-E). Also consistent with Tex-124-E, results from sieve analyses were used to adjust the PVR values considering the percentage of soil binder. The total PVR values were then defined by adding the contribution to the PVR by individual sublayers along each boring depth. The total PVR values predicted for each boring were also presented in Tables 1 and 2.

An evaluation was conducted to assess the consistency of swell potentials within the different test sections, predicted using the two considered approaches (i.e., based on the PI and LL of surficial soils and on predicted PVR values). In general, the predicted PVR values were found to be consistent with the expansiveness class of subgrade soils established based on PI and LL values. The predicted PVR values ranged from 4 to 117 mm, with the PVR value at the low expansive locations ranging from 4 to 43 mm. On the other hand, except for the case of one of the borings, the PVR value at the medium expansive and highly expansive locations, ranged from 28 to 78 mm and from 78 to 117 mm, respectively.

To establish a basis for comparative evaluation of the field performance among the test sections, the findings from the subgrade soil investigation were used to classify each test section into one of three expansiveness classes (low, medium, and high). It should be noted, however, that information regarding the subgrade soil properties was limited to that obtained at the location of the borings and that each test section may have extended through areas with different expansive soil characteristics. Table 3 summarizes the criteria used to classify the test sections. Fig. 3 also illustrates the distribution of each class among the experimental test sections using a color-coded layout. According to the layout presented in this figure, highly expansive clay subsoils were particularly abundant in the middle of the experimental area (Sections 5–10 in the westbound direction and Sections 21–26 in the eastbound direction).

### Geosynthetic Materials

Three geosynthetics were used in the test sections of the FM2 project including a biaxial polypropylene geogrid, a biaxial polyester geogrid, and a polypropylene woven geotextile. According to the manufacturers’ specifications, the ultimate tensile strength values for the three products in the cross-machine direction were 19, 29, and 70 kN/m, respectively, while the ultimate tensile strength values in the machine direction were 12, 29, and 70 kN/m, respectively. The woven geotextile was reported to have a comparatively higher tensile modulus at 2% strain (700 and 965 kN/m in the machine and cross-machine directions, respectively) than the polypropylene geogrid (205 and 330 kN/m in the machine and cross-machine directions) and the polyester geogrid (365 kN/m in both machine and cross-machine directions). Specimens were collected from the geosynthetic rolls used for the construction of the field test sections and were independently characterized, with laboratory results generally confirming those reported by the manufacturers (Zornberg et al. 2008). The procedure used for the selection of the geosynthetics and potential correlation between geosynthetic properties and expected field performance of the test sections constructed with them are discussed in subsequent sections of this paper.

### Environmental Conditions

Environmental conditions were monitored at the FM2 site over the duration of this study because it was important to correlate this information with roadway distress as well as with expected patterns of moisture changes within the subgrade soils. In locations characterized by arid or semiarid climates with well-defined wet and dry periods, the soil moisture content may change significantly within the soil active zone. In these locations, low-precipitation summer

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**Table 2. FM2 subgrade soil characterization and predicted PVR at locations of borings in Portion 2 of test sections**

<table>
<thead>
<tr>
<th>Depth (m)</th>
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<th>PI</th>
<th>Soil class</th>
<th>LL</th>
<th>PI</th>
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<th>Soil class</th>
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<td>33</td>
<td>18</td>
<td>SC</td>
<td>43</td>
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<td>—</td>
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<td>57</td>
<td>42</td>
<td>SC</td>
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<td>CL</td>
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<td>CH</td>
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Note: PVR = potential vertical rise (values are in millimeters); CH = fat clay; CL = lean clay; and SC = clayey sand.

**Table 3. Classification of FM2 test sections based on swell potential of subgrade soil**

<table>
<thead>
<tr>
<th>Swell potential</th>
<th>PI</th>
<th>LL</th>
<th>PVR (mm)</th>
<th>Test sections in each class</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>&lt;25</td>
<td>&lt;40</td>
<td>4–43</td>
<td>1, 2, 11, 12, 13, 17, 18, 27, 28, 29</td>
</tr>
<tr>
<td>Medium</td>
<td>25–35</td>
<td>40–60</td>
<td>28–78</td>
<td>3, 4, 6, 14, 15, 16, 19, 20, 30, 31, 32</td>
</tr>
<tr>
<td>High</td>
<td>&gt;35</td>
<td>&gt;60</td>
<td>78–117</td>
<td>5, 7, 8, 9, 10, 21, 22, 23, 24, 25, 26</td>
</tr>
</tbody>
</table>
months are often accompanied by high temperatures, resulting in increased evaporation and transpiration. These conditions have resulted in significant soil shrinkage, which in turn triggers the development of longitudinal cracks. On the other hand, high precipitation during wet months results in swelling of the expansive clay subgrade, which may even partially close previously developed longitudinal cracks. Therefore, cycles of wet and dry periods as well as temperature fluctuations are particularly relevant to assess the expected performance of roadways constructed on expansive clay subgrades.

Environmental data were collected at the FM2 site in the form of precipitation and temperature records. Precipitation data were collected from two nearby stations, referred to as Stations 313608 and 313609 in Fig. 1. Fig. 4 illustrates the monthly precipitation at Station 313609 from January 2006 (the beginning of the project) to March 2015. The horizontal axis of this graph shows the months of each year, with the vertical bars corresponding to the monthly rainfall. This figure also shows the dates when condition surveys were conducted at FM2 to assess the roadway distresses. As discussed in the next section, this information was helpful to correlate the data from the condition surveys with the environmental conditions.

Evaluation of the FM2 precipitation data presented in Fig. 4 indicates several periods characterized by particularly wet or dry conditions. For example, the first 4 months of the project are a period characterized by a cumulative rainfall of 217 mm, which is equivalent to an average monthly rainfall of 54 mm. Yet the next 14 months (Month 6, 2006 to Month 7, 2007) define a period when the cumulative rainfall increased to 1,834 mm, which is equivalent to an average monthly rainfall of 131 mm. A dry period of over 12 months occurred from Month 8, 2007 to Month 7, 2008 at the site. A few comparatively short wet and dry periods can be then identified from Month 8, 2008 to Month 9, 2009, followed by a comparatively wet period starting in Month 10, 2009 and continuing for 1 year, reaching a cumulative rainfall of 1,368 mm (equivalent to an average monthly rainfall of 114 mm). This wet period was followed by a record-setting dry period, which went from Month 10, 2010 to the end of 2011 (Nielsen-Gammon 2011). The cumulative rainfall over this 15-month-long period was only 615 mm, equivalent to an average monthly rainfall of 41 mm. Precipitation data show that after this extended dry period, the road site was subject to relatively short cycles of wet and dry periods, with the longest dry period being less than 5 months. As will be discussed in subsequent sections of this paper, the successive wet and dry periods resulted in swelling and shrinkage cycles in the expansive clay subgrade, ultimately resulting in the development of longitudinal cracks in the pavement.

Temperature records at the FM2 site were collected from a weather station at the city of College Station, located 55 km north of FM2, as well as from three local weather stations located closer to the project site. The cyclic temperature variations over the years were also evaluated as part of this study. The lowest temperatures were reported toward the end of each year, with the highest temperature values corresponding to June or July of each year. The average weekly temperature never fell below the freezing point, so evaluation of road damages due to frost heave was not relevant for this study.

Performance Evaluation

The long-term performance data collected at FM2 are evaluated in this section. Focus is on the performance data collected from visual condition surveys 1 (opening of the road to traffic), 14, 16, 18 (before, during, and after the 2010–2011 period of extreme drought, respectively), and 21 (the latest condition survey).

Visual Condition Surveys and Characterization of Longitudinal Cracks

The extent and severity level of the different types of pavement distresses were characterized according to the guidelines outlined in the FHWA Distress Identification Manual and TxDOT Pavement Management Information System Rater’s Manual (FHWA 2003; TxDOT 2015). While the various distress types were documented as part of the condition surveys, the main focus was on gathering data on the longitudinal cracks that developed in the 32 experimental test sections. This is because an important benefit expected from the use of geosynthetics is the mitigation of environmental longitudinal cracks. Surveying wheels with an accuracy of 0.3 m were used to record the location of the beginning and end of each crack, and rulers were utilized to measure crack widths. The identified distress features were also documented with photographs. Field performance data were collected over multiple field campaigns conducted at intervals of a few months. The performance of the experimental test sections was evaluated based primarily on the extent of longitudinal cracks that were wider than 3 mm. The extent of longitudinal cracks in a given road section was expressed by the longitudinal crack index (LCI), which corresponds to the ratio between the total length of longitudinal cracks in a road section and the length of the section. It should be noted that LCI may exceed 100% if the cumulative length of multiple cracks in a test section exceeds the total length of the section. A total of 21 condition surveys were conducted from 2006 to 2015.
Extent of Environmental Longitudinal Cracks

Fig. 5 summarizes the performance data collected from the experimental test sections in the latest condition survey (Survey 21) in terms of LCI. The horizontal axis of this figure indicates the number corresponding to each test section and the vertical axis shows LCI. The test sections are grouped in the figure according to the eight different design schemes used in this project (Fig. 2). The last bar in each group, in white, corresponds to the LCI weighted average calculated using the individual LCI values from the several test sections of a given design scheme.

Evaluation of the performance data presented in Fig. 5(a) reveals that, on average, the geosynthetic-stabilized sections performed significantly better than the control sections. The average LCI was found to reach 86% in the control sections, whereas this index was below 29% in the geosynthetic-stabilized sections. The three geosynthetic-stabilized groups of sections were found to exhibit similar levels of performance. The average LCI for geosynthetic-stabilized sections was 28%, 24%, and 29% for the GS1, GS2, and GS3 sections, respectively.

The results also indicate that lime stabilization to the subbase layer did not result in performance improvement for the test sections in terms of longitudinal cracks. Indeed, as shown in Fig. 5(b), the lime-stabilized sections were found to have a performance similar to that of the control sections, with an average LCI of 76%. The combination of lime-stabilized subbase and geosynthetic-stabilized base techniques was also found to lead to comparatively poor performance in several test sections. Results from the GS1+LM test sections indicate that lime treatment of the subbase did not improve the average LCI in relation to that of the untreated (GS1) test sections. In fact, results collected from the other combined geosynthetic- and lime-stabilized sections (i.e., GS2+LM and GS3+LM) indicate that lime treatment of the subgrade led to average LCI values that significantly exceeded those obtained in the corresponding untreated geosynthetic-stabilized sections (i.e., GS2 and GS3). The different mechanisms involved in the two stabilization techniques were found to adversely affect the overall performance of the roadway under environmental loads. While the enhanced support provided by the chemically stabilized layer may result in an improved performance under traffic loads (e.g., reduced rut depth), the chemically stabilized layer negatively impacted the flexibility required to redistribute differential movements induced by environmental loads.

Of the 23 sections stabilized using geosynthetics, 5 (Sections 2, 5, 22, 23, and 24) were found to show a comparatively lower performance than the remaining 18 sections. Subsurface soil data were evaluated to identify potential causes for this performance. The closest borings to Sections 2, 5, 22, 23, and 24 are Borings B2, B6, B7, and B8, respectively. Evaluation of the soil characterization results revealed the presence of sandy soil layers in B2, B6, and B7. In B2, the sand was found to extend more than 1.2 m deep. The PI values obtained for these layers ranged from 25 to as low as 14, which classify as low to nonexpansive soils. Similar sand layers were also found in the surficial layers of B6 and B7. The comparatively lower performance in these test sections is attributed to the presence of the sandy soils. The interbedded sandy layers within predominantly clay soils may have provided comparatively easier access to water in zones under the paved road that would have shown only minor or no changes in moisture content. The nonuniform presence of sand within expansive clay subgrades in these sections may have ultimately facilitated the development of comparatively higher and more frequent differential settlements.

Evaluation of Performance Considering Subgrade Classes

The performance of the test sections was also evaluated considering the different classes of subgrade expansiveness. Since the swell potential of soils was found to vary along the roadway project, the FM2 test sections were grouped into the low, medium, and high expansiveness classes, based on the characteristics of the subgrade soils (Fig. 3 and Table 3). Fig. 6 shows the performance data results grouped into the identified expansiveness classes.

Fig. 6(a) presents the average LCI for the test sections in each design scheme, without taking into account the impact of differences in soil expansiveness. As previously noted, the extent of environmental longitudinal cracks was found to be significantly lower in the geosynthetic-stabilized sections than in the control and in the lime-stabilized sections. All three geosynthetics were found to be equally effective in mitigating the development of longitudinal cracks. Implementation of lime stabilization in sections that also included geosynthetic stabilization never led to improved performance, showing a comparatively high range of LCI values.

Figs. 6(b–d) present the average LCI, as reevaluated considering sections in the high, medium, and low expansiveness classes,
respectively. Evaluation of the data presented in these plots confirms the main findings elaborated previously without accounting for subgrade classes. Specifically, for all expansiveness classes, the geosynthetic-stabilized sections were found to perform better than the control and lime-stabilized sections. The control sections exhibited approximately the same level of cracking in all categories, with average LCI values ranging from 80% to 90%. While geosynthetic stabilization of the base course proved to be effective in mitigating the longitudinal cracks in all classes, lime stabilization of the subbase (without geosynthetic stabilization) was found to result in a performance ranging from detrimental to somewhat effective depending on the subgrade class [Figs. 6(b–d)]. The combination of lime-stabilized subbase and geosynthetic stabilization of the base course did not show consistent performance among various design schemes in test sections in the high expansiveness class [Fig. 6(b)]. In all cases, adding lime stabilization to geosynthetic stabilization resulted in a lower performance than using only geosynthetic stabilization. When comparing the performance of combined sections to that of the control sections, the development of longitudinal cracks was mitigated in the low and medium expansiveness classes [Figs. 6(c and d)], while it led to decreased performance in the high expansiveness class [Fig. 6(b)], at least for Test Sections GS2+LM and GS3+LM.

**Correlation of Performance with Environmental Conditions**

As previously described, the detrimental effects of expansive clay subgrades on pavements have been reported to occur in the form of longitudinal cracks. These cracks tend to open toward the end of dry periods and partially close during wet periods. Consequently, the potential benefits derived from geosynthetics are expected to be more pronounced toward the end of particularly dry seasons.

An additional evaluation of the test sections’ performance over time was conducted by assessing the impact of the environmental conditions at the site in order to further understand the benefits derived from using geosynthetics during periods with a variety of environmental conditions. Based on the previously discussed evaluation of precipitation data, a historic record-setting dry season was identified from October 2010 to the end of 2011, lasting over 15 months. The performance of the test sections over time was evaluated at times representative of conditions before, during, and after the record-setting drought.

**Performance of Test Sections during Stage 1: Before the Drought**

The performance of the test sections during this stage was evaluated by analyzing the field data collected in Survey 1, conducted on Day 0 (opening of the road to traffic), and those collected in Survey 14, conducted on Day 1,550 (onset of the drought). Although cycles of wet and dry periods were identified during this stage (Fig. 4), the length and severity of the dry periods were comparatively small. Therefore, the extent of longitudinal cracks was minor, as shown by the data collected in field condition surveys.

The LCI values in all test sections obtained in Survey 14 is presented in Fig. 7. These results indicate that, by the time of this
Performance of Test Sections during Stage 2: Drought Period

The performance of the test sections during the record-setting drought was evaluated using field data collected on Days 1,550, 1,700, 1,875, and 2,000, which correspond to Surveys 14, 16, 17, and 18, respectively. The average LCI values for the test sections in each design scheme are also summarized in Fig. 8 (shown as Stage 2 in the figure).

The data presented in Fig. 8(a) indicate that from the onset of the drought, the control sections showed a higher development of longitudinal cracks than the geosynthetic-stabilized sections. The LCI in the control sections was found to rapidly increase during the initial months of this period, whereas all three geosynthetic-stabilized sections exhibited a comparatively low rate of crack development. The high rate of crack development in the control sections continued until the time of Survey 16, when the most severe period of the drought had ended (Fig. 4). During Stage 2, the difference between LCI values in the control and those in the geosynthetic-stabilized sections increased from approximately 7% (Survey 14) to approximately 42% (Survey 16). The precipitation rate increased slightly from the time of Survey 16, which is also shown in Fig. 8(b). Consequently, the implementation of lime stabilization not only did not improve the performance of sections involving geosynthetic-stabilized bases, but it generally led to additional crack development. The comparatively poor performance of sections involving lime-stabilized subgrades can be attributed to the fact that lime stabilization not only did not improve the performance of sections involving geosynthetic-stabilized bases.
treatment led to a rigid yet brittle subbase layer within the pavement system. The brittle response of the lime-stabilized layer when subject to subgrade volumetric changes is expected to have resulted in the development of longitudinal cracks that would not have been triggered in comparatively more ductile, non-lime-stabilized sections.

**Performance of Test Sections during Stage 3: After the drought**

Environmental data collected after the drought indicate that the FM2 site was subject to comparatively wet conditions interspersed with comparatively short dry periods. The data presented in Fig. 8 actually show a reduction in the rate of longitudinal crack development during this period. Evaluation of the longitudinal crack data from Day 1,950 (Survey 18) to Day 3,150 (Survey 21) indicates that the rate of longitudinal crack development decreased in all sections. However, as expected, the rate of crack development during this period was still higher than that in Stage 1, which corresponds to a comparatively intact road structure. The higher rate of crack development in Stage 3, as compared to that in Stage 1, can then be attributed to the accelerated deterioration of the pavement structure expected once longitudinal cracks had already developed. The rate of longitudinal crack development in the control sections was found to be approximately 6.6% per year, whereas this rate was below 2.9% per year in the geosynthetic-stabilized sections. During this period, the performance of lime-stabilized sections was found to be similar to that of the control sections, while the test sections involving a combination of lime-stabilized subbase and geosynthetic-stabilized base continued to show a comparatively poor performance.

**Mechanisms and Design Implications**

**Development of Environmental Longitudinal Cracks**

The mechanism triggering environmental longitudinal cracks in roadways over expansive clay subgrades has been reported to involve an environmentally induced flexion of the pavement due to the differential vertical displacements that develop between the road edge and its centerline (Zornberg et al. 2012). Specifically, the moisture content of the subgrade soils near the pavement shoulders decreases rapidly during dry seasons, leading to settlement of the road edges in relation to its centerline [Fig. 9(a)]. Instead, the moisture content of the subgrade soils near the pavement shoulders increases also comparatively quickly during wet seasons, leading to heave in this area [Fig. 9(b)]. On the other hand, the subgrade soils
located toward the central section of roadways are subject to relatively small moisture changes, as the paved surface minimizes water infiltration into the subgrade. The main path for moisture migration into subgrade soils located under the road’s central portion is from the shoulder area, with moisture fronts advancing and retreating under the pavement during wet and dry periods, respectively. Therefore, vertical movements in the center of the road are comparatively smaller than the recurring heave and settlement occurring at the road edges.

The recurring differential movements of the road edges in relation to its centerline result in the development of environmental longitudinal cracks. These cracks are expected to develop approximately where the moisture front advances the most from the shoulders during a wet period before retreating with the advent of a dry period. This region would be subject to comparatively high flexural strains, being then more prone to trigger the development of tension cracks.

Observations from this case study, along with similar field evidence in other roadways, were evaluated to assess the consistency of the field observations with the aforementioned mechanism, for which little quantitative field evidence has been hitherto presented. The presence of geosynthetics in the base course of roadways founded on expansive clay subgrades constitute inclusions that proved capable of distributing environmentally induced vertical displacements over comparatively wide regions, hence minimizing the stress concentration needed to initiate the development of environmental longitudinal cracks.

**Location and Depth of Environmental Longitudinal Cracks**

Consistent with the previously described mechanism, environmental longitudinal cracks in FM2 test sections developed at high rates during dry periods [Fig. 8]. In addition, these cracks were found to develop with higher frequency toward the edge of the road, as illustrated in Fig. 10. The presence of geosynthetics was found to shift the location where environmental longitudinal cracks would develop from within the traffic lane [Fig. 10(a)] to the locations consistent with the width of the geosynthetic roll, beyond the traffic lane [Fig. 10(b)].

The forensic evaluation of longitudinal cracks that had developed at a different roadway (SH21), founded on an expansive clay subgrade with characteristics similar to those of the FM2 subgrade, provided additional evidence on the consistency of the aforementioned mechanism to explain the development of environmental longitudinal cracks. Specifically, and as part of a forensic effort,
the pavement was excavated at the location of an environmental longitudinal crack to assess its depth. Fig. 11(a) shows a view of the environmental longitudinal crack before excavation, while Fig. 11(b) shows the same location after a forensic excavation. As shown in Fig. 11(b), the environmental longitudinal crack was found to extend through the surficial hot mix asphalt layer and into the top portion of the base, while deeper sections of the pavement structure had remained intact. This observation indicates that the crack had developed from the pavement surface toward the base, which is consistent with the previously described mechanism. That is, the environmental longitudinal crack developed due to concentration of tensile strains toward the roadway surface after the pavement structure had flexed owing to differential settlements. This observation is important because environmental longitudinal cracks had previously been thought to develop because of the initial development of longitudinal cracks within the expansive clay subgrade, and finally reflected to the roadway surface. Ultimately, the mechanism leading to the development of environmental longitudinal cracks involves differential settlement-induced flexion of the pavement and not the reflection of a tension-induced crack in the subgrade.

Pattern of Moisture Fluctuations within Expansive Clay Subgrade

A complementary investigation was conducted as part of the FM2 field monitoring program to evaluate fluctuations in the subgrade soil moisture content. Specifically, a horizontal array of moisture sensors was installed within the expansive clay subgrade under a traffic lane during the 2006 reconstruction of FM2. Three moisture sensors were installed under the traffic lane, with the fourth sensor being placed under the edge of the road. The fluctuations of subgrade soil moisture content, measured using these sensors from January 2006 to January 2007, are presented in Fig. 12. The horizontal axis corresponds to the time since installation of the sensors, while the vertical axis shows the change in the moisture content in relation to the initial moisture content values for each sensor. Evaluation of the data presented in this figure reveals that the amplitude of the moisture fluctuations within the subgrade underneath the traffic lane (i.e., underneath the paved area of the roadway) was significantly smaller than that of the moisture fluctuations at the edge of the roadway. Such different changes in moisture content values over time result in significantly different soil volumetric changes. Specifically, high swelling and shrinkage of the expansive clay subgrade would occur at the edge of the roadway, while possibly negligible vertical movements would occur toward the center of the road. Although data on the vertical movement of the road surface were not collected as part of the case study presented in this paper, measurements of vertical movements in roadways with characteristics similar to those of this case study were collected in subsequent studies, confirming the fluctuations in vertical movements induced by corresponding fluctuations in moisture content within the subgrade soils (Roodi et al. 2016, 2018).

Further insight was gained by correlating the fluctuation in moisture content within the subgrade to environmental conditions. Fig. 13 presents average daily temperature and cumulative precipitation at the project site over the same period when moisture content data were monitored in the subgrade. The weather data at the project site were compared to historical weather patterns in Navasota, Texas, located near the project site, in order to identify periods representative of warm and cold seasons [Figs. 12 and 13]. The time history of weather data correlates well with the time history of the subgrade soil moisture content at the road edge. Specifically, the three periods during which the subgrade soil moisture content at the road edge was particularly low were found to correspond to environmental conditions characterized by high temperatures and low precipitation (see Dry Seasons 1–3 in Figs. 12 and 13). It should be noted that during the entire time history shown in Figs. 12 and 13, the subgrade soil moisture content did not change significantly at locations under the pavement. Similarly, during rain periods (e.g., between Dry Seasons 1 and 2, between Dry Seasons 2 and 3, after Dry Season 3) the subgrade soil moisture content at the road edge increased while the subgrade soil moisture content at the center of the road remained essentially unchanged. These moisture trends are consistent with the previously described mechanism regarding development of environmental longitudinal cracks. That is, moisture content fluctuations within the subgrade soil at the road edges during wet and dry periods would lead to heave and settlement of the road edges in relation to the reasonably stationary road centerline. The most detrimental environmental condition, as observed in FM2 during the record-long 2011 drought, corresponds to comparatively long periods characterized by high temperature and low precipitation.
Correlation of Performance with Geosynthetic Properties

In spite of the clear evidence presented in this paper that geosynthetic stabilization of the base course can improve pavement performance, identification and quantification of the geosynthetic properties that contribute to such improvement have remained, at best, inconclusive. Several research initiatives have aimed at establishing correlations between geosynthetic index properties and field performance. Such index properties have included the rib strength, junction strength, flexural rigidity, and aperture size in the case of geogrids, as well as wide-width tensile strength, tensile modulus, and tensile strength in the case of both geogrids and geotextiles (e.g., Perkins et al. 2004; Christopher et al. 2008; Cuelho and Perkins 2009; Mahmood et al. 2012; Chen and Abu-Farsakh 2012). However, most of the properties identified in these studies have focused on quantifying the behavior of geosynthetics in isolation rather than characterizing the soil–geosynthetic interaction under representative confinement.

In the absence of specifications that include proper characterization of the interaction between soil and geosynthetic, the specifications used for the selection of the geosynthetics used in the FM2 project were established largely based on TxDOT’s previous experiences. Specifically, TxDOT specifications for the selection of geogrids for base/embankment reinforcement and environmental cracking involved Departmental Material Specifications (DMS).
Table 4. Characteristics of geosynthetics used in FM2 test sections

<table>
<thead>
<tr>
<th>Property</th>
<th>Geogrid 1</th>
<th>Geogrid 2</th>
<th>Geogrid 3</th>
</tr>
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<tbody>
<tr>
<td>Geogrid type</td>
<td>Polypropylene</td>
<td>Polypropylene</td>
<td>Polyester</td>
</tr>
<tr>
<td>Geogrid type</td>
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<td>Woven</td>
<td>Woven</td>
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<tr>
<td>Ultimate tensile strength (kN/m)</td>
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<td>&gt;250,000</td>
<td>&gt;250,000</td>
</tr>
<tr>
<td>Tensile modulus at 10% strain (kN/m)</td>
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<td>&gt;200,000</td>
<td>&gt;200,000</td>
</tr>
<tr>
<td>Tensile modulus at 40% strain (kN/m)</td>
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<td>&gt;100,000</td>
<td>&gt;100,000</td>
</tr>
<tr>
<td>Elongation at break, %</td>
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<td>&gt;100</td>
<td>&gt;100</td>
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<tr>
<td>Junction strength (kN/m)</td>
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<td>&gt;250</td>
<td>&gt;250</td>
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<tr>
<td>Aperture stability (deg)</td>
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<tr>
<td>Percentage open area (%)</td>
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<td>&gt;70</td>
<td>&gt;70</td>
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<tr>
<td>Thickness (mm)</td>
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<tr>
<td>Note: MD = machine direction; CMD = cross machine direction</td>
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</table>

Summary and Conclusions

A comprehensive study was conducted on the long-term field performance under environmental loads of a roadway, FM2, in Grimes County, Texas, constructed on expansive clay subgrades. The focus of the study was on road stabilization using various techniques, 6240 for the selection of Geogrid for Base/Embankment Reinforcement and 6270 for Biaxial Geogrid for Environmental Cracking (TXDOT 2010a, b). Table 4 presents the requirements for the selection of geogrids consistent with the specifications available at the time of the FM2 reconstruction project. Evaluation of the information presented in this table reveals that TXDOT requirements for geogrids in base stabilization under traffic loads were similar to the requirements for geogrids used to mitigate environmental cracking. More importantly, requirements for both applications were based only on geogrid geometric characteristics (e.g., aperture size, percentage open area, thicknesses) and geogrid mechanical properties in isolation (i.e., without the confinement of soil) such as tensile modulus and junction efficiency. However, as concluded in subsequent studies, the actual performance of geogrid-stabilized systems should be determined considering the interaction between the geosynthetic and the surrounding soil (e.g., Archer and Wayne 2012; Roodi et al. 2018).

While index and in-isolation geosynthetic properties have been used in roadway projects, research has been conducted aimed at identifying soil–geosynthetic interaction parameters relevant to geosynthetic base stabilization. Most of the previously proposed experiments attempted to replicate the dynamic nature of traffic loads by applying cyclic loads to a stabilized soil layer using cyclic plate load tests (e.g., Perkins 1999), cyclic pullout tests (e.g., Cuelho and Perkins 2005), bending stiffness tests (e.g., Sprague et al. 2004), or cyclic triaxial tests (e.g., Perkins et al. 2004). However, comparatively less research has been undertaken to investigate parameters relevant to geosynthetic-stabilized base roadways subject to environmental loads. More recently, a new soil–geosynthetic interaction model has been developed focusing on the stiffness of geosynthetic-stabilized systems under small displacements that is relevant to stabilized roadways. This model, referred to as the soil–geosynthetic composite (SGC) model, introduced a new parameter (the stiffness of the soil–geosynthetic composite, or $K_{SGC}$), which captures both the tensile characteristics of the geosynthetic and the shear behavior of the soil–geosynthetic interface (Zornberg et al. 2017b; Roodi and Zornberg 2017). A comprehensive study was recently concluded to establish correlations between the $K_{SGC}$ value for a wide range of geosynthetics and field performance under environmental loads (Roodi et al. 2018). A good correlation was found between the $K_{SGC}$ values and the improved field performance associated with the use of geosynthetics to stabilize the base of roadways founded on expansive clays.

While a full discussion on the characteristics and determination of the $K_{SGC}$ values is beyond the scope of this paper, it is worth noting that $K_{SGC}$ was obtained for the geosynthetic products used in the FM2 case study (Roodi et al. 2018). In particular, it is revealing that the $K_{SGC}$ value obtained for the three geosynthetic products used in the FM2 investigation were very similar. Having similar $K_{SGC}$ values for these geosynthetics is consistent with the similar performance of the three different geosynthetic products revealed by the FM2 field monitoring data. This finding suggests that the stiffness of the soil–geosynthetic composite ($K_{SGC}$) is a relevant property to predict the expected level of improvement in the performance of geosynthetic-stabilized roadways under environmental loads.
particularly geosynthetic stabilization of the base course, to mitigate damage induced by environmental loads. A total of 32 experimental test sections with an average length of 137.5 m were constructed in 2006 using 8 different design schemes. The design schemes involved various combinations of geosynthetic-stabilized base courses and lime-stabilized subbase courses. Control sections, which did not involve geosynthetic or lime stabilization, were also constructed. This study developed and implemented protocols to investigate the field performance of roadways subject to environmental loads. The main components of such protocols include identification of the mechanisms resulting in roadway damage from environmental loads and the associated performance measurements, characterization of the subgrade soil, evaluation of environmental conditions, performance evaluation based on damage induced by environmental loads, and correlation between performance and environmental conditions.

Implementation of this approach confirmed that the main type of distress affecting roadways founded on expansive clay subgrades is the development of longitudinal cracks. Subgrade soil and environmental conditions were extensively investigated at the FM2 site, and a rigorous monitoring program was conducted to characterize the long-term performance of the test sections in terms of the development and extent of environmental longitudinal cracks. The performance of the different test sections was subsequently correlated with the expansive properties of the subgrade as well as with environmental conditions. The main findings resulting from this investigation are as follows:

1. Geosynthetic-stabilized sections in FM2, a road founded on expansive clays, were found to perform significantly better than the control sections. The average LCI reached 86% in the control sections, whereas this index remained below 30% in the geosynthetic-stabilized sections.

2. Base-stabilized sections using the three different geosynthetic types selected for FM2 were found to exhibit approximately the same level of performance. The average LCI values in these sections were 28%, 24%, and 29% for the GS1, GS2, and GS3 test sections, respectively.

3. Lime treatment of the subbase did not result in improved road performance, at least regarding the development of environmental longitudinal cracks. In fact, the lime-stabilized sections on FM2 performed similarly to the control sections, with an average LCI of 76%.

4. The incorporation of subbase lime stabilization to roadway sections already including geosynthetic stabilization not only did not improve the roadway performance but generally led to a decrease in roadway performance. Specifically, the average LCI values in the GS2+LM and GS3+LM test sections were significantly higher than those in the GS2 and GS3 test sections.

5. The variability of subgrade soil properties, which is often unavoidable along roadways, made it particularly challenging to categorize the test sections on the basis of their subgrade expansiveness. However, the use of geosynthetics led to mitigation of the development and extent of environmental longitudinal cracks irrespective of the level of expansiveness of the subgrade soils.

6. The long-term performance of the test sections was found to correlate well with the environmental data collected, with the use of geosynthetics showing improvement both over comparatively mild as well as significant droughts. The improved performance resulting from the use of geosynthetics was found to be particularly significant during the 2010–2011 record-setting drought in Texas. The rate of longitudinal crack development during this drought was 51% per year in the control sections, whereas this rate was below 19% per year in the geosynthetically-stabilized sections.

7. A mechanism for the development of environmental longitudinal cracks in roadways constructed on expansive clay subgrades was identified. Accordingly, the development of environmental longitudinal cracks can be attributed to the differential vertical movements between the road edges and its center during cycles of wet and dry periods. Observations of the extent, location, and depth of the environmental longitudinal cracks along with the data collected for fluctuations in the subgrade soil moisture content provided added field evidence for the consistency of the identified mechanism.

8. Comparison of the field performance of the geosynthetically-stabilized test sections with the experimental data reported from soil–geosynthetic interaction tests suggests that the stiffness of the soil–geosynthetic composite \(K_{SCG}\) can be a suitable property for predicting the level of performance improvement expected from the implementation of geosynthetic stabilization in roadways founded on expansive clays. Overall, significant benefits were observed to result from geosynthetic stabilization of base courses in roadways founded on expansive clay subgrades and subject to environmental loads. The benefits were realized by mitigating the development of longitudinal cracks. Observations from this study suggest that the combination of geosynthetic base stabilization and chemical subbase stabilization, as previously adopted by TxDOT and other transportation agencies, should be discontinued.

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