EVALUATING UNPAVED ROAD RESEARCH

APRIL, 2017
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Executive Summary

HOW TO ASSESS UNPAVED ROAD DESIGN AND PRODUCT RESEARCH
The mechanisms governing unpaved road performance are complex. If poorly designed – for instance, utilizing the wrong type of ground stabilization system - unpaved roads can become expensive, dangerous and environmentally damaging liabilities. Diligent civil engineers and contractors lower these risks by relying on quantitative data from laboratory and field tests to select the right designs and stabilization products.

Unfortunately, it is increasingly difficult to distinguish between research that is valid, reliable and unbiased from that which is faulty or confusing. Research that compares the performance of various road designs and related products easily falls into this category – where testing conditions and hypotheses may favor certain conditions or beliefs, and results can often be misunderstood, misleading, and even erroneous or biased. Useful research, therefore, depends heavily on the level of researcher independence and the rigor applied to test planning and implementation to ensure the validity of test results. Third party review of such research, by trusted experts, is a critical step that identifies flaws, process issues or additional factors that might impact a professional’s understanding of test results and engineering decisions derived from the data.

This summary and supporting set of foundational documents includes one such third party research review as well as articles by two geosynthetic experts who share their experience with conducting and evaluating road design research. Taken as a whole, this collection provides guidance for properly planning, executing and interpreting unpaved road design field tests for quality assurance, comparative study, and unpaved road design decision-making.

TOP 10 QUESTIONS TO ASK ABOUT UNPAVED ROAD RESEARCH
1. Do the study results vary significantly from other studies with similar objectives or from nationally recognized design procedures that have gone through significant peer review, calibration and validation?
2. Did the testing conditions unfairly advantage one type of product over another?
3. Were the designs and products being compared appropriately matched to the application being tested?
4. Were the trial sections randomly selected and appropriately designed based on conditions provided?
5. Was the aggregate fill, or other ground materials used in the test, DOT approved and applied uniformly?
6. Were the geosynthetic products applied in a consistent manner that is recommended by the manufacturers?
7. Was the aggregate appropriately placed and compacted to allow for proper interlock to develop with products being tested?
8. Were climatic conditions (i.e., moisture, temperature…) appropriately accounted for during testing?
9. Were subgrade test conditions held constant during the trial?
10. Was third party review of the testing done by trusted experts to identify flaws, process issues or additional needs?

The importance of these questions is highlighted in the study conducted by the Western Transportation Institute (WTI) in 2014, which will be referred to as the Montana Study. This research provides a prime example of the challenges that exist in deciphering results from full-scale comparative field experiments. The Montana Study was intended to:

- Look at the performance of geosynthetics for subgrade stabilization
- Determine which material properties were relevant to performance
For the Montana Study, sections using materials and construction procedures outlined by the researchers were constructed and trafficked. The Montana testing yielded results that were significantly different from those found in related, previously executed studies. The curious differences warranted detailed review prior to releasing such information. Regardless, results were published and have been heavily promoted by a few manufacturers, distributors with the author of the study acting as a spokesman for one of the sponsoring manufacturers. The significant variance in the results of the Montana Study from those of other studies completed with nationally recognized design procedures and which have gone through significant peer review, calibration and validation, should be of concern.

In order to address this concern, a study was undertaken by Dr. Tim Stark (University of Illinois) and Rob Swan (Drexel University), evaluating the Montana Study to another similar field study conducted by SGI Testing Services in 2011. Both studies had similar conditions, but dramatically different results. The study provides greater insight into why the results from the Montana Study were significantly different than those of other field trials. The full report titled “EVALUATION OF GEOSYNTHETIC SUBGRADE STABILIZATION FIELD TESTS” is available.

Specifically, here are several items found effecting results of the Montana Study:

- There was a significant amount of rain (1.9 inches) that occurred during the “Montana Study” testing. This may have contributed to significant weakening of the subgrade soils in the lined trench, and allowed for the early onset of rutting. This is outlined in detail in section 5.3 of the report by Dr. Stark and Swan.

- The aggregate used, and the testing conditions, were advantageous for geotextiles that provided a filtration function. The soil and aggregate conditions allowed for contamination of the aggregate in the geogrid sections, especially with the high rainfall. Following standard filtration design procedure, the Average Particle Size Ratio for these materials was much higher than what is acceptable (ratio of 150 versus the desired value of 25). This significantly effects performance. In conditions similar to the “Montana Study”, a designer would normally require a non-woven geotextile be placed beneath the geogrid with this combination of base and subgrade materials.

- Some materials placed in the Montana Study were not installed across the entire width of the test lanes. It is well-known standard practice for geogrids and geotextiles to be installed across the entire width of the roadways they are incorporated into to avoid edge effects or other issues. Experimental conditions in research should reflect standard practice, but this was overlooked in the Montana Study.

- Aggregate was placed using a screed. This method of placement is not performed in the field and does not allow for proper interlock to develop with a geogrid. It also results in aggregate compaction different from what would be experienced in the field and effects performance characteristics of a geogrid stabilized section.
EVALUATION OF GEOSYNTHETIC SUBGRADE STABILIZATION FIELD TESTS

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EXECUTIVE SUMMARY

This report compares two field trafficking studies that investigated the performance of geosynthetic stabilized subgrades in unpaved road applications. The first study reviewed was conducted by the Western Transportation Institute (WTI) in the College of Engineering at Montana State University in Bozeman, Montana in 2014 and is referred herein as the WTI Study. The second study reviewed was conducted by SGI Testing Services, LLC in Norcross, Georgia in 2011 and referred herein as the SGI Study. This comparison was undertaken because these two seemingly similar field studies yielded significantly different results with regard to the effectiveness of specific geosynthetic materials, e.g., Tensar geogrids, in the performance of unpaved roads.

This report discusses the 2014 WTI results and factors that contributed to several geosynthetic materials not exhibiting performance characteristics that have been demonstrated in prior laboratory and field testing programs as well as performance in actual field installations. The four most likely reasons several geosynthetic materials, e.g., geogrids, did not perform as well as expected in the 2014 WTI Study are:

1. The inclusion of a plastic trench liner to encapsulate the compacted imported clay subgrade together with about 1.9 inches of rain occurring during the period of trafficking resulted in a subgrade strength that was lower than intended and therefore lower than assumed in the design of the trial sections. This ultimately contributed to the early onset of rutting and premature failure of all the sections.
2. The aggregate selected for the base course material was a rounded fractured faced or poor quality aggregate which has poor mechanical properties.
3. Another reason for the poor performance is incomplete coverage of the complete width of the compacted clay subgrade with some of the geosynthetics included in the WTI study compared to complete coverage across the subgrade in the SGI Study including 12 inch overlaps between rolls of geosynthetic to provide continuous coverage.
4. The last major reason for the poor performance of geogrids in the WTI Study is the non-representative placement of the aggregate layer above the clay subgrade soil. This placement technique resulted in the geogrids, not developing good interlock with the aggregate layer. For example, the aggregate layer was not compacted to the same level experienced in most field applications, which resulted in limited aggregate interlock with the geogrid. Additionally, for the WTI study the Piping Ratio (D_{15} (aggregate) to D_{85} of (subgrade)) was found to be less than 5 and the Average Size Ratio (D_{50} (aggregate) to D_{50} (subgrade)) was found to be 150, significantly greater than the desired value of less than 25. This high Average Size Ratio is advantageous for geotextile reinforcements but not for geogrid materials where one would require
the use of a nonwoven geotextile being placed above the subgrade and below the geogrid reinforcement in order to insure proper separation of the aggregate and subgrade which was not done in the WTI study.

In summary, the review of the 2014 WTI and 2011 SGI studies shows these two seemingly similar field studies yielded significantly different results with regard to the performance of unpaved roads. Based on this review, the 2011 SGI Study used construction techniques that are more typical of field applications and thus produced results that are more indicative of the performance of geosynthetics in actual field installations for unpaved roads than the 2014 WTI Study.
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1. INTRODUCTION

1.1 Terms of Reference

Stark Consultants, Inc. (SCI) has prepared this report, which compares and evaluates two field trafficking studies used to investigate the performance of geosynthetic stabilized subgrades. The first study was conducted by the Western Transportation Institute (WTI) in the College of Engineering at Montana State University in Bozeman, Montana. This study was published in May 2014 and is referred to herein as the WTI Study. The second study was conducted by SGI Testing Services, LLC in Norcross, Georgia and was published in August 2011 and referred herein as the SGI Study.

This report has been prepared at the request of Dr. Mark H. Wayne, P.E., Director of Application Technology at Tensar International Corporation (Tensar) to Dr. Timothy D. Stark, P.E. of Stark Consultants, Inc. (SCI) and the University of Illinois at Urbana-Champaign, Mr. Robert H. Swan, Jr of Drexel University, Dr. Zehong Yuan of SGI Testing Services, LLC both on behalf of SCI. The report has been reviewed by Dr. Zehong Yuan, P.E. of SGI in accordance with the peer review policy of SCI.

1.2 Purpose

The purpose of this study is to evaluate these two field trafficking studies to establish the preferred method for conducting such studies, constructing trafficking test sections, and assess the performance of Tensar geogrids relative to unpaved road design and other geosynthetics. This study was undertaken because these two seemingly similar field studies yielded significantly different results with regard to several geosynthetic materials, e.g., Tensar geogrids, used in the studies. The main goals of the study are to:

- determine the main factors (conditions) that affect the performance of the geosynthetics used in unpaved roads over weak subgrades in each study,
- establish the proper test section construction, methodology, and conditions for conducting field trafficking studies for unpaved road design, and
- re-evaluate the performance of geosynthetic stabilized subgrades in these two studies and field applications based on the two previous goals and evaluation of the test results.
1.3 **Report Organization**

The remainder of the report is organized in the following sections:

- Section 2 presents a summary of the 2014 WTI study outlining all of the field and test conditions, measurements, and test results.
- Section 3 presents a summary of the 2011 SGI study outlining all of the field and test conditions, measurements, and test results.
- Section 4 compares and contrasts the merits and shortcomings of each study.
- Section 5 presents a summary of the findings from the analysis of the two studies.
- Section 6 presents the conclusions from the analysis of the two studies.

2. **WTI STUDY**

2.1 **Overview**

The main objective of the WTI study was to determine the geosynthetic material properties that are most reflective of their in-field performance for subgrade stabilization in unpaved roads. By determining these properties, a designer can objectively specify geosynthetics for subgrade stabilization based on material properties, construction techniques, and cost for a specific application while still allowing competition from different geosynthetic manufacturers. In an attempt to accomplish this, field test sections were constructed at the WTI to investigate the relative benefit of various geosynthetics to unpaved road performance. The field test sections were designed to allow controlled application of traffic loads to the test sections.

The various geosynthetic field test sections were constructed on a three (3) foot thick artificial clay soil subgrade to provide uniform conditions for each test section. A gravel base course was uniformly spread and compacted above the subgrade with the geosynthetic material to be installed in each test section. This was done to allow direct comparison of the different geosynthetics due to the imposed trafficking loads from the test vehicle under similar subgrade and base course conditions. Transverse and longitudinal rut measurements were the primary indictors of performance benefits of each geosynthetic due to the imposed trafficking loads. Additionally, post-traffic examination provided information regarding the performance and installation survivability of the geosynthetics.
2.2 Field and Loading Conditions

This section describes the field and loading conditions used in the 2014 WTI study so they can be compared with the field and loading conditions used in the 2011 SGI study. This is important because Section 4 of this report compares and contrasts the important test details from these two studies to help explain the significant differences in the measured trafficking and geosynthetic performance under similar field testing conditions.

2.2.1 Design and Layout

The design and layout of the WTI test area focused on creating a uniform roadway to study the effects of geosynthetic stabilization, subgrade strength, and the depth of base course gravel over the prepared clay soil subgrade. The test sections for the WTI study were constructed at the TRANSCEND test facility located in Lewistown, Montana on an existing taxiway. The test sections were designed and constructed to minimize differences in site characteristics along the length of the test sections. The test sections were planned to study differences in the performance of various geosynthetic materials under similar loading, subgrade conditions and base course thickness. Additional test sections were prepared to study geosynthetic performance under variable subgrade strength and base course thickness.

In addition, three control sections with no geosynthetics were constructed with varying thicknesses of base course for comparison with three test sections constructed with Tensar BX Type-2 geogrids. Each of the three control and the three Tensar geogrid sections had different clay subgrade strengths. Figure 1 shows the test section layout and includes the target subgrade strength, base course thickness, and geosynthetic constructed in each section.
Figure 1: General layout of WTI test sections with target construction conditions (from Cuelho et. al, 2014).
2.2.2 Construction

The test section consists of a single-lane gravel road constructed on a 3 ft thick soft clay soil imported subgrade overlain by a poor quality aggregate base material consisting of fractured faced round to sub-rounded stone. To construct this test section, a trench was excavated into an existing airport taxiway such that the final grade of the imported subgrade would be at the same elevation as the taxiway grade. The trench excavation was 16 ft wide and 3 ft deep to construct the 3 ft thick clay soil subgrade and attempt to minimize boundary effects from the trench walls and the foundation. However, it should be noted that some of the geosynthetics that were used in the study had roll widths greater than 16 ft. These geosynthetics when placed exceeded the limits of the trench therefore their performance may have been influenced by the boundary conditions beyond the limits of the trench. The excavation was 860 ft long to accommodate the 14 different test sections shown in The trench was lengthened at each end to accommodate tapered sections to facilitate movement of construction equipment in and out of the trench.

The trench was lined with a 6-mil thick plastic liner to help maintain the constructed moisture content of the clay subgrade throughout the study. The effects of this plastic liner are discussed further in Section 5 of this report. After placement of the 6-mil thick plastic liner, the soft clay subgrade was prepared and compacted into the lined trench. Therefore, the compaction moisture was retained in the trench as well as precipitation that occurred before and during testing. This is significant because WTI reports over 1.4 inches of rainfall occurred during the trafficking portion of the study as discussed below.

The geosynthetics were then placed upon the clay soil subgrade and the poor quality aggregate then spread and compacted over and around the geosynthetic above the clay soil subgrade. A cross-sectional view of a typical test section is shown in Figure 2 with the test vehicle used for the trafficking study. Figure 2 shows the clay soil subgrade placed such that it is level with the surrounding grade and then various thicknesses of aggregate placed above the geosynthetics and compacted clay soil subgrade. Additionally, Figure 2 shows the typical installation width for the Tensar geogrids and Figure 3 shows the typical installation width for the geotextiles in the study.
Figure 2: Cross-section of trafficking test section with test vehicle to approximate scale and typical Tensar Geogrid Installation (after Cuelho et. al, 2014).

Figure 3: Cross-section of trafficking test section with test vehicle to approximate scale and typical geotextile Installation (after Cuelho et. al, 2014).
Looking at the overall design of the test section based on the selected clay soil subgrade and the poor quality aggregate and the target values of Piping Ratio ($D_{15}$ (aggregate) to $D_{85}$ of (subgrade)) being less than 5 and the Average Size Ratio ($D_{50}$ (aggregate) to $D_{50}$ (subgrade)) being less than 25 when considering the design of an unconfined aggregate layer, one can see some issues with regard to the selection of the materials. The Piping Ratio was found to be less than 5 however the Average Size Ratio was found to be 150, significantly greater than the desired value of less than 25. (For reference the actual soil material properties are presented in Sections 2.2.3 and 2.2.5.) This high Average Size Ratio could work well for geotextile reinforcements but with these types of materials one would require the use of a nonwoven geotextile being placed above the subgrade and below the geogrid reinforcement which was not done in the WTI study.

2.2.3 Artificial Subgrade

The three (3) ft of clay subgrade soil classified as A-6 according to the AASHTO classification system (AASHTO M-145) or CL (sandy lean clay) according to the USCS classification system (ASTM D 2487). Other relevant properties of the clay soil subgrade soil are presented in Table 1 and Figure 4.

Table 1: General properties of clay soil subgrade (Cuelho et. al, 2014)

<table>
<thead>
<tr>
<th>Soil Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid Limit</td>
<td>34</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>17</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>17</td>
</tr>
<tr>
<td>Fines Content (% passing #200 sieve)</td>
<td>55%</td>
</tr>
<tr>
<td>Standard Proctor Max Dry Unit Weight (ASTM D698)</td>
<td>116 pcf</td>
</tr>
<tr>
<td>Standard Proctor Optimum Moisture Content</td>
<td>16%</td>
</tr>
</tbody>
</table>
Figure 4: Grain-size distribution of clay soil subgrade (from Cuelho et. al, 2014).

The clay soil subgrade was constructed in six lifts that were approximately 6 inches thick for a final thickness of about three (3) feet. The soil was stockpiled near the trench and was moisture conditioned with the use of a water truck until the soil reached the desired compaction moisture content for each test section as shown in Figure 5. The clay subgrade soil was then placed in the plastic lined trench using an excavator (see Figure 6) and tracked in place with a track-mounted skid-steer dozer to initially level and compact the subgrade as shown in Figure 7. The lining of the trench with the plastic liner prevented water from draining from the clay subgrade soil before and during the trafficking. This resulted in the clay subgrade being softer than planned and softer than assumed in the design of the trial sections. This ultimately contributed to the early onset of rutting and failure of all the sections. The precipitation that occurred before and during testing was also retained in the trench. Water was also found to have collected in the rutted subgrade profile. As such, areas with greater rut depths retained a disproportionate amount of water. Subgrade testing within these areas was not performed after each rain event. This is important because the geosynthetics, e.g., geogrids, were placed directly on the top of the clay soil subgrade that was encapsulated by this plastic liner. This resulted in the clay soil subgrade being softer than expected under the geosynthetics and providing less support than in the SGI Study. This would have required the placement of an additional amount of aggregate over the sections with the lower subgrade strength. This is confirmed in the WTI Study which states the aggregate layer moved into the clay subgrade by about 0.7 inches causing the geosynthetics to behave like tensioned membranes which is not beneficial to geogrids. Aggregate also moved sideways as rutting and
heave occurred, thinning the aggregate layer. Without adequate distance being maintained the subgrade begins to fail and geosynthetics go into tension. The Giroud-Han design equations (Giroud and Han (2011)) are predicated on maintaining the required aggregate thickness between wheel and subgrade/geosynthetic. This is reinforced by the following summary statement in the WTI Study on Page 103 that:

“As the subgrade and base were continually shoved away from the center of the rutted area under continued traffic loading, the primary mechanism of support from the geosynthetics transitioned from lateral confinement to tensioned membrane (refer to Figure 1).”

A smooth single-drum vibratory roller (5.5 ft wide and 15,500 lbs) was used to compact the subgrade by making two passes in three longitudinal paths. The top surface of the subgrade was kept moist with the use of the water truck and covering the subgrade with plastic until the next layer of subgrade could be placed. The top surface was smoothed and screeded to the height of the existing surface by tilling the top surface and pushing a large metal trench box across the surface removing any excess soil material and then rerolled using a smaller single-drum vibratory roller as shown in Figure 8. Field measurements of shear strength using a hand held vane shear device were made to characterize the subgrade as it was placed in the trench. Additionally, dynamic cone penetrometer (DCP) and lightweight deflectometer (LWD) tests were performed as a comparison to monitor the subgrade material properties which will be further discussed in Section 2.3 (Measurements).

Figure 5: Watering and mixing clay subgrade soil before placement and compaction in the excavated trench (from Cuelho et. al, 2014).
Figure 6: Placing clay subgrade soil in trench lined with 6-mil thick plastic liner before compaction (from Cuelho et. al, 2014).

Figure 7: Grading of clay subgrade soil in lined trench using a track-mounted skid-steer dozer (from Cuelho et. al, 2014).

Figure 8: Screeding the final clay Subgrade soil layer to complete filling of excavated trench (from Cuelho et. al, 2014).
2.2.4 Geosynthetics Used in Test Sections

Twelve geosynthetic products were used in the WTI study to evaluate their relative performance under the various conditions used in the test sections. A summary of these products is presented in Table 2. Table 2 also includes some basic characteristics of each product with regard to manufacturer, structure, polymer type, roll width, mass per unit area, and aperture size. The twelve geosynthetic products were further characterized by the following five laboratory tests:

- Wide-width tensile strength by ASTM D4595 (geotextiles) or ASTM D6637 (geogrids),
- Cyclic tensile modulus by ASTM D 7756,
- Resilient interface shear stiffness by ASTM D 7499,
- Junction Strength by ASTM D 7737, and
- Aperture stability modulus (Kinney, 2000).

Table 2: Summary of geosynthetic materials used in WTI Study (from Cuelho et al., 2014).

<table>
<thead>
<tr>
<th>Geosynthetic Test Section</th>
<th>Product Manufacturer - Name</th>
<th>Structure</th>
<th>Polymer³</th>
<th>Roll Width (in)</th>
<th>Mass per unit area (oz/yd²)</th>
<th>Aperture Size (in) MD x XMD</th>
</tr>
</thead>
<tbody>
<tr>
<td>1, 2 and 3</td>
<td>Tensar - BX Type2</td>
<td>integrally-formed, biaxial geogrid</td>
<td>PP</td>
<td>160</td>
<td>8.9</td>
<td>1.0 x 1.3</td>
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<td>4</td>
<td>NAUE - Secagrid 30/30 Q1</td>
<td>vibratory-welded, biaxial geogrid</td>
<td>PP</td>
<td>186</td>
<td>5.9</td>
<td>1.3 x 1.3</td>
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<tr>
<td>5</td>
<td>Colbond - Enkagrid MAX 30</td>
<td>biaxial, welded geogrid</td>
<td>PP</td>
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<td>6.0</td>
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<td>6</td>
<td>Synteen - SF 11</td>
<td>PVC-coated, woven, biaxial geogrid</td>
<td>PMY</td>
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<td>9.5</td>
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<td>7</td>
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<td>12.3</td>
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<td>TenCate - Mirafl BXG11</td>
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<td>PMY</td>
<td>158</td>
<td>9.1</td>
<td>1.0 x 1.0</td>
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<td>9</td>
<td>Haasker - Formit 30</td>
<td>polymer-coated, knitted, biaxial geogrid</td>
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<td>206</td>
<td>6.5</td>
<td>0.6 x 0.6</td>
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<td>SynTec - Tenax MS 330</td>
<td>extruded, triple-layer, biaxial geogrid</td>
<td>PP</td>
<td>156</td>
<td>9.7</td>
<td>1.7 x 2.0⁶</td>
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<td>11</td>
<td>Tensar - TX140</td>
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<td>5.3</td>
<td>1.6 x 1.6⁶</td>
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<td>Tensar - TX180</td>
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<td>PP</td>
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<td>6.4</td>
<td>1.6 x 1.6⁶</td>
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<td>13</td>
<td>TenCate - Mirafl RS580i</td>
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<td>PPF</td>
<td>204</td>
<td>12.3</td>
<td>80⁵</td>
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<td>14</td>
<td>Propex - Geotex 801</td>
<td>non-woven, needle-punched geotextile</td>
<td>PP</td>
<td>186</td>
<td>8.0</td>
<td>80⁵</td>
</tr>
</tbody>
</table>

³ PP = polypropylene, PMY = polyester multifilament yarn, PPF = polypropylene fiber
⁶ MD = machine direction, XMD = cross-machine direction
⁶ for a single layer; apparent opening size is reduced when these layers are stacked on top of one another
⁶ reported as “rib pitch” in manufacturer’s specification sheet
⁶ Apparent Opening Size (AOS) in U.S. Standard sieve size, ASTM D 4751
The results of the wide-width testing and cyclic tensile modulus testing are presented in the WTI Study Report for both the machine direction (MD) and cross machine direction (XMD) for each product. The results are not repeated here to shorten this report.

The WTI Study Report also presents the results of the resilient interface shear stiffness tests for each geosynthetic product. The results of the junction strength and aperture stability modulus testing are also presented for each product in the WTI Report. The junction strength and aperture stability testing were conducted by SGI testing Services, LLC, Norcross, Georgia for the WTI Study. The authors of the WTI study performed the wide width tensile testing, the cyclic tensile modulus testing, and the resilient interface shear stiffness testing.

The geosynthetics were delivered to the test site and stored until they were installed in the test sections. The geosynthetics were installed on the surface of the clay soil subgrade in each test section by carefully rolling them out in the direction of trafficking as shown in Figure 8. Any wrinkles were removed by gently pulling on the end of the geosynthetic material. The edges of each geosynthetic were not tensioned or staked in place. Because the widths of the geosynthetics varied from product to product as shown in Table 2, each product was centered on the subgrade from side to side so the test vehicle would be centered on the material during the trafficking. Figure 9 shows the installation of a geotextile which had a roll width of 17-ft which was greater than the 16-ft width of the trench excavation allowing the geotextile to develop additionally anchorage in a tensioned membrane condition compared to the installation of the Tensar geogrids which had roll widths of 13.1-ft which were less that the width of the excavated trench. Additionally, some strain gage instrumentation was attached to each geosynthetic in two locations prior to the installation of the geosynthetics on the subgrade.

![Figure 9: Installation of geotextile on top of compacted clay soil subgrade in test section (from Cuelho et. al, 2014).]
2.2.5 Base Course Aggregate

The aggregate that was used in the WTI Study is Montana 5A aggregate. The gradation of this aggregate is similar to the average range of gradations found in specifications of the various states involved in the pooled funds for the study, as shown in Table 3: The states involved in the pooled funds are Idaho, Montana, New York, Ohio, Oklahoma, Oregon, South Dakota, Texas, and Wyoming. The Montana 5A aggregate specification and the average range of gradations are presented in Figure 10, respectively.

Table 3: Comparison of Montana 5A aggregate with average gradation of states involved (from Cuelho et. al, 2014).

<table>
<thead>
<tr>
<th>Sieve (US)</th>
<th>Sieve (in)</th>
<th>Montana 5A (% pass.)</th>
<th>Average range (% pass.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-inch</td>
<td>2</td>
<td>100</td>
<td>98-100</td>
</tr>
<tr>
<td>1 1/2-inch</td>
<td>1.5</td>
<td>94-100</td>
<td>89-100</td>
</tr>
<tr>
<td>3/4-inch</td>
<td>0.75</td>
<td>70-88</td>
<td>62-90</td>
</tr>
<tr>
<td>3/8-inch</td>
<td>0.375</td>
<td>50-70</td>
<td>46-74</td>
</tr>
<tr>
<td>#4</td>
<td>0.187</td>
<td>34-58</td>
<td>33-62</td>
</tr>
<tr>
<td>#40</td>
<td>0.0167</td>
<td>6-30</td>
<td>9-33</td>
</tr>
<tr>
<td>#200</td>
<td>0.00295</td>
<td>0-8</td>
<td>2-11</td>
</tr>
</tbody>
</table>
Figure 10: Range of base course aggregate gradations for the various states involved (from Cuelho et. al, 2014).

The Montana 5A aggregate classifies as A-2-4 according to the AASHTO classification system (AASHTO M-145) or GP-GC (poorly graded gravel with clay with sand) according to the USCS classification system (ASTM D 2487) is considered a base course material by Montana under FHWA/MT-07-011/8117-30 report, However, unlike many of pooled fund state agency specifications, where this aggregate is classified as a poorly graded gravel with clay and sand having an AASHTO designation of A-2-4 which is more commonly used as a subbase material than a base course material. Other relevant properties of the poorly graded gravel are presented in Table 4 and Figure 11 presents the grain size curve for the aggregate used in the WTI Study.
Table 4: General properties of poorly graded gravel (Cuelho et. al, 2014).

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid Limit of fines</td>
<td>23</td>
</tr>
<tr>
<td>Plastic Limit of fines</td>
<td>15</td>
</tr>
<tr>
<td>Plasticity Index of fines</td>
<td>8</td>
</tr>
<tr>
<td>% passing #200 sieve</td>
<td>10%</td>
</tr>
<tr>
<td>Max. dry unit weight(^\d)</td>
<td>139 lb/ft(^3)</td>
</tr>
<tr>
<td>Optimum moisture content(^\d)</td>
<td>6.0%</td>
</tr>
<tr>
<td>% fractured faces</td>
<td>55%</td>
</tr>
<tr>
<td>CBR(^\d) (at (p_{dry} = 140) lb/ft(^3))</td>
<td>&gt;100</td>
</tr>
</tbody>
</table>

\(^\d\) using modified Proctor procedure (ASTM D1557)

Figure 11: Grain-size distribution of Montana 5A aggregate (Cuelho et. al, 2014).

The Montana 5A gravel was prepared by adding water and mixing it with an end loader until it reached the desired optimum moisture content. During placement of the gravel, the material was dumped from the side onto the geosynthetics and then pushed or screeded over the geosynthetics,
as shown in Figure 12. The gravel was never tracked in place due to concerns over unequal construction traffic over the various test sections. The lack of tracking in of the gravel would most likely cause a reduction in the performance of the section due to a reduced interlock. The gravel was constructed in two layers for all test sections, except for the Control 3 test section. For the test sections, which included geosynthetics, and for the Control 1 test section, the final thickness of the first layer of gravel was about 8 inches when compacted and the second was about 3 inches thick for a total of about 11 inches of compacted aggregate. For the two other control test sections (Control 2 and 3) the gravel contained thicker sections. For Control 2 test section the final thickness of the first layer of gravel was about 8 inches when compacted and the second was about 8 inches thick for a total of about 16 inches compacted of aggregate. For Control 3 test section the gravel was constructed in three layers, the final thickness of each layer of gravel was about 8 inches when compacted for a total of about 25 inches of compacted aggregate. The compaction was achieved using a smooth single drum vibratory roller which was 54 inches wide and weighed 12,000 lb. Eight passes of the roller were made per layer of gravel at three transverse positions. The properties of the gravel were measured using DCP, LWD, California Bearing Ratio (CBR) and nuclear densometer testing which will be further discussed in Section 2.3 (Field Measurements).

Figure 12: Screeded gravel surface over geosynthetics (Cuelho et. al, 2014).
2.3 Field Measurements

2.3.1 Overview

During the construction of each test section, the placement of the clay soil subgrade and poorly graded gravel were monitored to ensure the materials were placed in a consistent and uniform manner. Each 50-foot long test section was delineated into 14 subsections so there would be seven subsections within each wheel path, which were labeled A through G as shown in Figure 13. These subsections are the locations where the monitoring of the materials was performed. There was also a 1.6 ft buffer zone placed at the end of each test section to allow a transition between test sections where there was an overlap of the geosynthetics.

2.3.2 Clay Soil Subgrade

Monitoring of the clay soil subgrade during placement was done by taking four measurements using the hand held vane shear device in each of the subsections. Therefore, fifty-six (56) vane shear tests were performed per layer in each test section. Specifically, half of the measurements were made in the east wheel path and the other half were made in the west wheel path. The vane shear tests had been correlated to laboratory performed CBR tests on the subgrade material as shown in Figure 14. This correlation was used to establish the targeted CBR value for each test section. Additionally, six LWD measurements were made immediately after each layer of subgrade was compacted. Once the final lift of subgrade was completed eight (8) DCP tests, one in-field CBR test, and two (2) nuclear densometer tests were conducted within each test section. A summary of the measurements that were made on the subgrade during construction are presented in Table 5 with the location of measurement keyed to the letter system in Figure 13.
Figure 13: Measurement areas for field soil tests within a single test section (from Cuelho et. al, 2014).

Figure 14: Relationship between California Bearing Ratio (CBR) and the hand held vane shear strength (from Cuelho et. al, 2014).
Table 5: Summary of soil measurements of the subgrade during construction. (from Cuelho et. al, 2014).

<table>
<thead>
<tr>
<th>Measurement Device</th>
<th>Layers</th>
<th>Measurements per Layer</th>
<th>Location of Measurement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vane Shear</td>
<td>all</td>
<td>56</td>
<td>A,B,C,D,E,F,G</td>
</tr>
<tr>
<td>Light-Weight Deflectometer</td>
<td>all</td>
<td>6</td>
<td>B,D,F</td>
</tr>
<tr>
<td>Dynamic Cone Penetrometer</td>
<td>final</td>
<td>6</td>
<td>A,D,G</td>
</tr>
<tr>
<td>In-Field CBR</td>
<td>final</td>
<td>2</td>
<td>D</td>
</tr>
<tr>
<td>Nuclear Density Gage</td>
<td>final</td>
<td>2</td>
<td>D</td>
</tr>
</tbody>
</table>

2.3.3 Base Course Aggregate

Testing of the poorly graded gravel during placement was done using six LWD measurements after the first and final passes of the compactor on the first lift. After the last pass of the compactor on the final lift an additional six LWD, six DCP, one in-field CBR and two nuclear densometer tests were conducted. A summary of the measurements that were made on poorly graded gravel during construction are presented in Table 6.

Table 6: Summary of soil measurements of the poorly graded gravel during construction. (from Cuelho et. al, 2014).

<table>
<thead>
<tr>
<th>Base Course Aggregate</th>
<th>Layers</th>
<th>Measurements per Layer</th>
<th>Location of Measurement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light-Weight Deflectometer</td>
<td>first</td>
<td>12</td>
<td>B,D,F</td>
</tr>
<tr>
<td>Light-Weight Deflectometer</td>
<td>final</td>
<td>6</td>
<td>B,D,F</td>
</tr>
<tr>
<td>Dynamic Cone Penetrometer</td>
<td>final</td>
<td>6</td>
<td>A,D,G</td>
</tr>
<tr>
<td>In-Field CBR</td>
<td>final</td>
<td>2</td>
<td>D</td>
</tr>
<tr>
<td>Nuclear Density Gage</td>
<td>final</td>
<td>2</td>
<td>D</td>
</tr>
</tbody>
</table>
Additional instrumentation was installed in an attempt to measure displacement and strain in the cross machine direction of the various geosynthetics products and to measure the development of the pore-water pressure within the subgrade. Linear variable differential transformers (LVDTs) were used to make three measurements of displacement along the geosynthetics at two different locations, approximately 15 feet apart, within each test section. Resistance strain gauges were bonded to the surface of each geosynthetic in the transverse direction. A strain gauge was mounted to the top and bottom surface of the geosynthetic at two different locations, approximately 15 feet apart, within each test section. Three pore-water pressure gauges were installed within the subgrade. Two pore-water pressure gauges were installed at a depth of 6 inches and the third was installed at a depth of 10 inches within each test section. Figure 15 provides an illustration of the instrumentation within each test section and Figure 16 provides a cross-section view of the instrumentation layout. During the trafficking phase of the study, air bubbles had developed in several of the pore-water pressure gauges indicating the sensors were not fully saturated. Therefore, the pore-pressure measurements may not be reliable and may not represent the actual conditions within the subgrade. Additionally, the gauges were removed in mid to late October 2012 to prevent damage to the sensors due to below-freezing weather conditions.

Figure 15: Illustration of the instrumentation within each test section (Cuelho et al., 2014).
2.4 Field Test Results

2.4.1 Clay Soil Subgrade

Some of the results of the various measurements on the compacted clay soil subgrade are presented Figures 17 and 18. In particular, Figure 17 presents the results of in-field CBR tests and composite CBR from vane shear and Figure 18 presents the dry unit weight and moisture content of the constructed subgrade. Figure 17 show considerable variability in the CBR and moisture content of the compacted clay soil subgrade.

Figure 17: Comparison of constructed subgrade strength from in-field CBR tests and composite CBR from vane shear in region D west rut path (from Cuelho et. al, 2014).
2.4.2 Base Course Aggregate

Some of the results of the various measurements on the Montana 5A gravel are presented in Figure 19 which presents a comparison of in-field CBR strength and CBR strength as measured using DCP in region D. Based on statements within the WTI study report it should be noted that the DCP penetration data from the upper and lower 2 inches of the Montana 5A gravel were not used in the calculation of average CBR for each region. The report stated that due to the granular materials near the surface of the gravel being easily disturbed since they were unbound and the lower 2 inches of granular materials were considered too close to the weak subgrade soil which would cause an influence on the DCP measurements. The data presented shows considerable variability in the CBR of the Montana 5A gravel. Figure 20 presents the in-place dry unit weight and moisture content of the poorly graded gravel.
2.4.3 Trafficking Study

The trafficking of the test sections began on 13 September 2012 and continued until 7 November 2012. A three-axel dump truck was used to conduct the all of the trafficking, which weighed 45,420 lb and had 90-psi tire pressure (see Figure 21). The specific dimensions and weights for the individual axels are shown in Figure 22. The trafficking was conducted in one direction from north to south through the test sections at a speed of approximately 5 mph to minimize dynamic loads on the test sections due to unevenness in the poorly graded gravel surface.
There were longitudinal lines painted on the gravel surface to guide the truck during the trafficking that were also used to mark rut measurements made throughout the study. Trafficking continued across the test sections until rut levels reached 3 inches which was considered failure in the study. Once 3 inches of rut was attained repairs were made by placing additional poorly graded gravel in the rutted areas using a skid-steer loader and leveling the surface as shown in Figure 23. Repairs within the test sections were made incrementally so that any non-failed portions of the test sections could be continued to be trafficked until they reached failure. Once failure occurred no additional rut measurements were made in the repaired areas.
Throughout the trafficking study there were occasional rainstorms having accumulations greater than 0.1 inch over a 24-hour period. When such storms occurred, the trafficking was interrupted. Trafficking would resume once the gravel surface had dried significantly. A total of 1.4 inches of rainfall was reported during the entire trafficking study. The history of the precipitation during the study is presented in Figure 24. These rain events are discussed further in Section 5 of this report but it is anticipated this rainfall resulted in water being retained in the clay soil subgrade because the trench was lined with a 6-mil thick plastic liner.
Figure 24: Precipitation events that occurred during the trafficking study (from Cuelho et. al, 2014).

Rut measurements were made at 40-inch intervals along the longitudinal lines that corresponded to the outside rear wheels of the dump truck with the use of a robotic total station. There were 28 longitudinal rut measurements made in each test section at various trafficking levels, 14 measurements in the east rut and 14 measurements in the west rut. Additionally, transverse rut measurements were also made at the same time in two locations in each test section. Therefore, 16 individual measurements were taken to develop a single transverse surface contour. An example of a transverse surface contour is presented in Figure 25.

Figure 25: Transverse rut profiles for Test Section 10 (north) with Tenax MS 330 geogrid (from Cuelho et. al, 2014).
Figure 25 shows that bearing capacity failures were occurring along the road surface parallel to the wheel paths by evidence of the heaving of the road surface. For this study heave was defined as the difference in the apparent rut depth and the elevation rut depth as shown in Figure 26. These measurements were used to determine the depth of the rut as a function of the difference in elevation of the measurement points over time. The total rut was determined by comparing the current measurement to a base line measurement before trafficking began. This rut was defined as the “elevation rut” as shown in Figure 26. Heave occurred at different levels of trafficking in each test section but most of the heave began around 100 to 300 truck passes. The initiation of heave was defined to occur at a rut depth of 0.5 inches for each test section. The occurrence of heave will be further discussed in Section 5 of this report. Figure 27 presents a comparison of heave, the longitudinal rut and change in displacement for all of the test sections.

Figure 26: Illustration of rut measurement (Cuelho et. al, 2014).

Figure 27: Heave, longitudinal rut and change in displacement comparison for all test sections (Cuelho et. al, 2014).
A review of the LVDT displacements measured during the WTI trafficking study indicate the geosynthetics transitioned from lateral confinement of the poorly graded gravel aggregate into tension membrane support of the geosynthetic as the rut depth increased and the bearing capacity failures (soil heave) were occurring. This transition was observed to typically occur between 100 and 300 truck passes. The occurrence of this transition will be further discussed in Section 5 of this report.

Two separate post trafficking evaluations were made; one immediately after the trafficking was complete during the week of 19 November 2012 and the second one the following summer during the week of 8 July 2013. The first evaluation was limited in scope due to the cold weather and focused on areas in each test section that experienced different levels of longitudinal rutting. The second evaluation was more extensive but occurred after the test sections were exposed to winter and spring seasonal conditions but had remained idle for 6 to 8 months. During the second evaluation, large samples of the geosynthetics were removed for further evaluation. Since the aggregate was not well compacted directly above the geosynthetics, the samples of geosynthetics were easily recovered with the use of compressed air to remove the aggregate from the geosynthetic surfaces. Table 7: presents the subgrade properties that were assessed during the first evaluation in November 2012.

Table 8: presents the subgrade properties that were assessed during the second evaluation in July 2013. Table 9: presents a comparison of the poorly graded gravel thickness before and after trafficking that shows that the thickness reduced in most test sections. Figure 28 presents the percentage of fines that have contaminated the poorly graded gravel causing the reduction in the thickness. All of the geogrid test sections, post trafficking, had increased fines/contamination of the base when compared to the geotextile sections. As discussed towards the end of Section 2.2.2 (Construction), proper design would have included a nonwoven filter geotextile beneath the geogrid in these test sections.
Table 7: Subgrade properties from the first evaluation, November 2012 (from Cuelho et. al, 2014).

<table>
<thead>
<tr>
<th>Geosynthetic Test Section</th>
<th>Excavation Location&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Rut Depth at 300 Truck Passes (in.)</th>
<th>CBR from DCP (%)</th>
<th>Dynamic Deflection Modulus (ksf)</th>
<th>Subgrade Moisture Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensar BX Type 2 (Section 2)</td>
<td>7 – East</td>
<td>1.64</td>
<td>2.56</td>
<td>112</td>
<td>21.2</td>
</tr>
<tr>
<td></td>
<td>4 – East</td>
<td>3.22</td>
<td>1.90</td>
<td>85</td>
<td>22.2</td>
</tr>
<tr>
<td>NAUE Secugrid 30/30 Q1 (Section 4)</td>
<td>12 – West</td>
<td>2.85</td>
<td>1.91</td>
<td>118</td>
<td>21.3</td>
</tr>
<tr>
<td></td>
<td>12 – East</td>
<td>1.67</td>
<td>1.43</td>
<td>126</td>
<td>21.7</td>
</tr>
<tr>
<td>Colbond Enkagrid Max 30 (Section 5)</td>
<td>9 – West</td>
<td>1.82</td>
<td>2.64</td>
<td>182</td>
<td>19.3</td>
</tr>
<tr>
<td></td>
<td>2 – West&lt;sup&gt;b&lt;/sup&gt;</td>
<td>4.00</td>
<td>2.31</td>
<td>97</td>
<td>21.8</td>
</tr>
<tr>
<td>Synteen SF 11 (Section 6)</td>
<td>13 – East</td>
<td>4.02</td>
<td>2.58</td>
<td>106</td>
<td>21.7</td>
</tr>
<tr>
<td></td>
<td>5 – East</td>
<td>1.65</td>
<td>2.56</td>
<td>153</td>
<td>22.1</td>
</tr>
<tr>
<td>Synteen SF 12 (Section 7)</td>
<td>6 – East&lt;sup&gt;b&lt;/sup&gt;</td>
<td>2.69</td>
<td>2.34</td>
<td>115</td>
<td>21.5</td>
</tr>
<tr>
<td></td>
<td>11 – East</td>
<td>5.36</td>
<td>2.51</td>
<td>78</td>
<td>22.7</td>
</tr>
<tr>
<td>Tencate Mirafi BXG 11 (Section 8)</td>
<td>11 – East</td>
<td>3.17</td>
<td>2.35</td>
<td>84</td>
<td>22.3</td>
</tr>
<tr>
<td></td>
<td>8 – East</td>
<td>1.48</td>
<td>2.39</td>
<td>80</td>
<td>22.7</td>
</tr>
<tr>
<td>Huesker Fornit 30 (Section 9)</td>
<td>10 – West</td>
<td>3.58</td>
<td>2.00</td>
<td>81</td>
<td>22.9</td>
</tr>
<tr>
<td></td>
<td>2 – East</td>
<td>2.99</td>
<td>2.43</td>
<td>110</td>
<td>23.6</td>
</tr>
<tr>
<td>Tensar TX 140 (Section 11)</td>
<td>12 – East</td>
<td>1.75</td>
<td>2.59</td>
<td>140</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>7 – West&lt;sup&gt;c&lt;/sup&gt;</td>
<td>7.38</td>
<td>2.52</td>
<td>83</td>
<td>22.0</td>
</tr>
<tr>
<td>Tensar TX 160 (Section 12)</td>
<td>6 – East&lt;sup&gt;c&lt;/sup&gt;</td>
<td>1.55</td>
<td>2.57</td>
<td>102</td>
<td>22.7</td>
</tr>
<tr>
<td></td>
<td>6 – West&lt;sup&gt;c&lt;/sup&gt;</td>
<td>7.67</td>
<td>2.00</td>
<td>82</td>
<td>21.3</td>
</tr>
<tr>
<td>Tencate Mirafi RS580i (Section 13)</td>
<td>11 – West</td>
<td>4.18</td>
<td>2.13</td>
<td>83</td>
<td>22.5</td>
</tr>
<tr>
<td></td>
<td>4 – East&lt;sup&gt;b&lt;/sup&gt;</td>
<td>1.13</td>
<td>2.37</td>
<td>121</td>
<td>22.8</td>
</tr>
<tr>
<td>Propex Geotex 801 (Section 14)</td>
<td>7 – East&lt;sup&gt;b&lt;/sup&gt;</td>
<td>1.57</td>
<td>2.54</td>
<td>122</td>
<td>23.0</td>
</tr>
<tr>
<td></td>
<td>11 – East</td>
<td>2.17</td>
<td>2.24</td>
<td>96</td>
<td>21.2</td>
</tr>
</tbody>
</table>

<sup>a</sup> number = longitudinal measurement point along test section – direction = east or west wheel path
<sup>b</sup> small amounts of water accumulation at the interface between subgrade and base course
<sup>c</sup> material fully ruptured at this location resulting in higher rut levels at the point of excavation
--- missing data
Table 8: Subgrade properties from the second evaluation, July 2013 (from Cuelho et. al, 2014).

<table>
<thead>
<tr>
<th>Geosynthetic Test Section</th>
<th>Excavation Location&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Rut Depth at 300 Truck Passes (in.)</th>
<th>CBR from DCP (%)</th>
<th>Dynamic Deflection Modulus (ksf)</th>
<th>Subgrade Moisture Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensar BX Type 2 (Section 1)</td>
<td>10 - West</td>
<td>2.54</td>
<td>3.29</td>
<td>108</td>
<td>15.4</td>
</tr>
<tr>
<td>Tensar BX Type 2 (Section 2)</td>
<td>10 - East</td>
<td>2.91</td>
<td>3.73</td>
<td>111</td>
<td>15.9</td>
</tr>
<tr>
<td>Tensar BX Type 2 (Section 3)</td>
<td>9 - West</td>
<td>2.03</td>
<td>2.67</td>
<td>105</td>
<td>16.1</td>
</tr>
<tr>
<td>Tensar BX Type 2 (Section 4)</td>
<td>9 - East</td>
<td>1.78</td>
<td>2.87</td>
<td>105</td>
<td>15.9</td>
</tr>
<tr>
<td>NAUE Securgrid 30/30 Q1 (Section 5)</td>
<td>9 - West</td>
<td>2.33</td>
<td>3.04</td>
<td>93</td>
<td>16.5</td>
</tr>
<tr>
<td>Colbond Ekaagrid Max 30 (Section 6)</td>
<td>9 - East</td>
<td>1.74</td>
<td>3.13</td>
<td>86</td>
<td>16.0</td>
</tr>
<tr>
<td>Synteen SF 11 (Section 7)</td>
<td>9 - East</td>
<td>2.02</td>
<td>3.07</td>
<td>98</td>
<td>16.6</td>
</tr>
<tr>
<td>Synteen SF 12 (Section 8)</td>
<td>9 - West</td>
<td>2.21</td>
<td>3.23</td>
<td>148</td>
<td>16.2</td>
</tr>
<tr>
<td>TenCate Minif BXG 11 (Section 9)</td>
<td>9 - East</td>
<td>2.91</td>
<td>2.75</td>
<td>88</td>
<td>16.7</td>
</tr>
<tr>
<td>Huesker Formit 30 (Section 10)</td>
<td>9 - West</td>
<td>1.92</td>
<td>3.05</td>
<td>91</td>
<td>16.0</td>
</tr>
<tr>
<td>Syntec – Tenax MS 330 (Section 11)</td>
<td>9 - East</td>
<td>3.11</td>
<td>3.33</td>
<td>220</td>
<td>17.1</td>
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<tr>
<td>TenCate Minif BXG 110 (Section 12)</td>
<td>9 - East</td>
<td>3.52</td>
<td>3.12</td>
<td>79</td>
<td>17.2</td>
</tr>
<tr>
<td>Huesker Formit 30 (Section 13)</td>
<td>9 - East</td>
<td>3.63</td>
<td>3.08</td>
<td>85</td>
<td>16.8</td>
</tr>
<tr>
<td>Syntec – Tenax MS 330 (Section 14)</td>
<td>9 - West</td>
<td>2.89</td>
<td>2.73</td>
<td>94</td>
<td>16.1</td>
</tr>
<tr>
<td>TenCate Minif RS580i (Section 15)</td>
<td>9 - East</td>
<td>3.94</td>
<td>3.08</td>
<td>94</td>
<td>16.4</td>
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<tr>
<td>Propex Geotex 801 (Section 16)</td>
<td>9 - East</td>
<td>2.85</td>
<td>2.99</td>
<td>102</td>
<td>16.3</td>
</tr>
<tr>
<td>TenCate Minif BXG 110 (Section 17)</td>
<td>9 - West</td>
<td>2.25</td>
<td>2.86</td>
<td>107</td>
<td>16.0</td>
</tr>
<tr>
<td>Control 1 (Section 18)</td>
<td>9 - East</td>
<td>2.90</td>
<td>2.92</td>
<td>119</td>
<td>16.7</td>
</tr>
<tr>
<td>Control 2 (Section 19)</td>
<td>9 - East</td>
<td>2.56</td>
<td>3.03</td>
<td>89</td>
<td>16.6</td>
</tr>
<tr>
<td>Control 3 (Section 20)</td>
<td>9 - West</td>
<td>2.01</td>
<td>3.01</td>
<td>90</td>
<td>16.0</td>
</tr>
<tr>
<td>Control 4 (Section 21)</td>
<td>9 - East</td>
<td>1.92</td>
<td>2.93</td>
<td>217</td>
<td>16.4</td>
</tr>
<tr>
<td>Control 5 (Section 22)</td>
<td>9 - East</td>
<td>2.67&lt;sup&gt;b&lt;/sup&gt;</td>
<td>3.26</td>
<td>87</td>
<td>16.5</td>
</tr>
<tr>
<td>Control 6 (Section 23)</td>
<td>9 - East</td>
<td>5.15&lt;sup&gt;b&lt;/sup&gt;</td>
<td>2.84</td>
<td>199</td>
<td>15.9</td>
</tr>
<tr>
<td>Control 7 (Section 24)</td>
<td>9 - East</td>
<td>2.04</td>
<td>2.79</td>
<td>99</td>
<td>17.2</td>
</tr>
<tr>
<td>Control 8 (Section 25)</td>
<td>9 - East</td>
<td>1.26</td>
<td>2.69</td>
<td>108</td>
<td>16.9</td>
</tr>
<tr>
<td>Control 9 (Section 26)</td>
<td>9 - East</td>
<td>0.90</td>
<td>2.49</td>
<td>74</td>
<td>17.6</td>
</tr>
<tr>
<td>Control 10 (Section 27)</td>
<td>9 - East</td>
<td>1.26</td>
<td>3.12</td>
<td>99</td>
<td>17.0</td>
</tr>
</tbody>
</table>

<sup>a</sup> number = longitudinal measurement point along test section – direction = east or west rut wheel path
<sup>b</sup> rut depth at 102 truck passes when ruts were filled in
Table 9: Poorly graded gravel thickness before and after trafficking (from Cuelho et. al, 2014).

<table>
<thead>
<tr>
<th>Geosynthetic Test Section</th>
<th>Location</th>
<th>Original Thickness (in.)</th>
<th>Post-Trafficking Thickness (in.)</th>
<th>Thickness Difference (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>West</td>
<td>Rut</td>
<td>East</td>
</tr>
<tr>
<td>Tensor BX Type 2 (Section 1)</td>
<td>9</td>
<td>11.6</td>
<td>10.8</td>
<td>10.7</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>11.4</td>
<td>11.0</td>
<td>11.2</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>11.8</td>
<td>10.3</td>
<td>10.4</td>
</tr>
<tr>
<td>Tensor BX Type 2 (Section 3)</td>
<td>12</td>
<td>10.7</td>
<td>10.2</td>
<td>9.9</td>
</tr>
<tr>
<td></td>
<td>13</td>
<td>10.0</td>
<td>10.0</td>
<td>10.2</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>10.0</td>
<td>10.4</td>
<td>9.7</td>
</tr>
<tr>
<td>NAUE Standard 10/30 Q1 (Section 4)</td>
<td>15</td>
<td>10.8</td>
<td>9.7</td>
<td>9.4</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>11.2</td>
<td>9.9</td>
<td>10.0</td>
</tr>
<tr>
<td>TexCot Material RS508i (Section 13)</td>
<td>17</td>
<td>12.0</td>
<td>12.3</td>
<td>11.8</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>15.3</td>
<td>15.0</td>
<td>14.3</td>
</tr>
<tr>
<td>Control 2</td>
<td>19</td>
<td>10.0</td>
<td>14.9</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>15.3</td>
<td>15.7</td>
<td>16.6</td>
</tr>
<tr>
<td>Control 3</td>
<td>21</td>
<td>25.0</td>
<td>25.4</td>
<td>26.1</td>
</tr>
<tr>
<td></td>
<td>22</td>
<td>26.4</td>
<td>24.8</td>
<td>25.7</td>
</tr>
<tr>
<td></td>
<td>23</td>
<td>25.2</td>
<td>24.8</td>
<td>24.0</td>
</tr>
</tbody>
</table>

*Longitudinal measurement point along test section
--- missing data

Figure 28: Percent fines in base aggregate samples above geosynthetics (Cuelho et. al, 2014).
Most of the geosynthetics that were recovered from the test sections during the second evaluation in July 2013 were distorted in the rutted area due to the drive wheels of the dump truck during trafficking. In addition, several of the test sections showed evidence of tensioned membrane effect. The occurrence of this tensioned membrane effect is discussed further in Section 5 of this report. The final results of the WTI trafficking study are in the form of ‘corrected’ rut depth verses truck passes for each geosynthetic and are shown in Figure 29 for all of the test sections. The vertical axis would more accurately be described as ‘adjusted mean rut depth’.

Figure 29: Corrected rut response for all test sections (from Cuelho et. al, 2014).

3. SGI STUDY

3.1 Overview

The objective of the SGI study was to determine the rutting depths developed due to an 18-kip axle load after repeated passes above two types of Tensar geogrids. Therefore, this study differs from the WTI Study in that only Tensar geogrids were tested instead of a range of geosynthetics. The SGI Test Sections were constructed to investigate the relative benefit of using Tensar BX1100 and TriAx® TX 130S geogrids in an unpaved road.

A compacted clay soil subgrade was constructed to provide uniform conditions for each test section. A gravel base course was uniformly spread and compacted above the geosynthetic in each test section to allow direct comparison of the performance of the BX and TX Tensar geogrids subjected to the same trafficking loads. Transverse and longitudinal rut measurements are the
primary indicators of the performance of an unpaved road due to the imposed trafficking loads as in the WTI Study. Additionally, post-traffic examination of the geogrids provided information regarding the performance and installation survivability of the geosynthetics.

3.2 Field and Test Conditions

This section describes the field and test conditions for the SGI study so they can be compared with the field and test conditions used in the WTI study. This is important because Section 4 compares and contrasts the important test details, which help explain the differences in the measured trafficking and geosynthetic performance between the two studies.

3.2.1 Design and Layout

The design and layout of the SGI test area focused on creating a uniform roadway to study the effects of geosynthetic stabilization, subgrade strength, and the depth of base course gravel. The test sections for the SGI Study were constructed in Alpharetta, Georgia and designed to minimize the differences along the length of the test sections and study the performance of two types of Tensar geogrids under similar loading conditions, subgrade strength, and base course thickness.

The SGI study was setup to have two lanes of trafficking, a left hand lane and a right hand lane. Each of the test sections had similar subgrade strengths. Figure 30 shows the general test section layout. The target subgrade strength is a CBR of 2% and base course thickness was 15 inches in the left hand lane and 10 inches in the right hand lane.
3.2.2 Construction

The test section consists of a two-lane gravel road constructed on a 2 ft thick soft clay imported subgrade overlain by an aggregate base course. To construct the test sections, the clay subgrade was prepared and compacted directly on the existing grade of the site. The geosynthetics were then installed and the base course then spread and compacted over and around the geosynthetic/subgrade. As a result, the compacted clay subgrade was not confined by a trench as in the WTI Study.

Looking at the overall design of the test section based on the selected clay soil subgrade and the aggregate base course and the target values of Piping Ratio being less than 5 and the Average Size Ratio being less than 25 when considering the design of an unconfined aggregate layer, one can see very little issues with regard to the selection of the materials. The Piping Ratio was found to be less than 5 and the Average Size Ratio was found to be 44, significantly closer to the desired value of less than 25 when compared to the value of 150 in the WTI study. (For reference the actual soil material properties are presented in Sections 3.2.3 and 3.2.5.) With this value of Average Size Ratio there is no concern of needing a nonwoven geotextile placed above the subgrade and below the geogrid reinforcement which should have been considered in the WTI study.

Figure 30: General layout of test sections for SGI study (SGI, 2011).
3.2.3 Compacted Clay Subgrade

The compacted clay subgrade soil classifies as A-7-5 according to the AASHTO classification system (AASHTO M-145) or SM (silty sand) according to the USCS classification system (ASTM D 2487). Other relevant properties of the clayey subgrade soil are presented in Figure 31. The clayey subgrade had a maximum dry unit weight of 98 pcf and an optimum moisture content of 20% based on standard Proctor compaction conditions (ASTM D 698).

The clayey subgrade was placed and compacted in approximately 8-inch thick lifts for a total thickness of about 2 feet. The soil was stockpiled at the test site and was moisture conditioned with the use of a water truck until the soil reached the target moisture content of approximately 29% or nine percent points wet of optimum. Figure 32. shows the subgrade soil being placed on the existing grade at the test site. The top surface of the subgrade was keep moist with the use of a water truck. Field measurements of subgrade strength and stiffness were made using a dynamic cone penetrometer (DCP), which is discussed further in Section 3.3 (Measurements). After the completion of DCP testing, the top surface of subgrade soil layer was then leveled and rolled with the lightweight rubber-belt dozer before placement of the geogrids.
Figure 31: Soil properties of the compacted clayey subgrade soil (from SGI, 2011).
3.2.4 Geosynthetics

There were two geosynthetic products used in the SGI study to evaluate their relative performance under the conditions of the various test sections. The two products installed are Tensar BX1100 geogrid from Lot Number 311843 and TriAx® TX130S geogrid from lot number 114233. The materials were provided by Tensar directly from their manufacturing plant in Morrow, Georgia.

The geogrids were delivered to the test site and stored until they were installed in the test sections. The geosynthetics were installed on the surface of the subgrade in each test section by carefully rolling them out in the direction of trafficking. Starting from the north end of the subgrade soil layer, two 50 ft long by full roll width (13.1-ft) of TX130S geogrid samples and then two 50 ft long by full roll width (13.1-ft) of BX1100 geogrid samples were placed on top of the subgrade soil layer. The overlap between two TX130S or two BX1100 was approximately 12 inch as shown in Figure 33 and Figure 34, respectively. Any wrinkles were removed by gently pulling on the end of the material. The edges of each geosynthetic were not tensioned or staked in place similar to the WTI Study and a 6-mil thick plastic liner was not installed along the edges of the test section to trap water in the clayey subgrade soil.
3.2.5 Base Course Aggregate

The base course aggregate used in the study was a GA DOT Graded Aggregate Base (GAB) material that classifies as A-1-a according to the AASHTO classification system (AASHTO M-145) or as a GW-GM according to the USCS classification system (ASTM D 2487). The GAB
material was manufactured by crushing granite rock by the Vulcan Materials Company. Other relevant properties of the GAB material are presented in Figure 35. The GAB material had a corrected maximum dry unit weight of 137 pcf and an optimum moisture content of 5.7% under standard Proctor compaction conditions (ASTM D 698).

Figure 35: Properties of Granular Base Aggregate Material placed of the geogrids (from SGI, 2011) (from SGI, 2011).
The base course was constructed in approximately 8-inch thick lifts for a total thickness of about 10 to 15 inches depending on the test section. The GAB material was stockpiled at the test site and was moisture conditioned with the use of a water truck until the GAB material reached the target moisture content. GAB material was placed on top of geogrids using a lightweight rubber-belt dozer in approximately 8-inch lift as shown in Figure 36. The loosely placed GAB material layer was first leveled and rolled by the lightweight rubber-belt dozer, and then compacted by using an Ingersoll Rand SD45F vibratory compactor for 10 passes as shown in Figure 37 to achieve approximately 95% of maximum standard Proctor density.

Figure 36: Placement of GAB material in 8-in lifts over geogrids (from SGI, 2011) (from SGI, 2011).

Figure 37: Compaction of GAB material with smooth drum roller (from SGI, 2011) (from SGI, 2011).
The completed test section was a two-lane unpaved road reinforced by two types of geogrids. The two lanes and their initial properties are described below:

- **Left Lane**: 100 ft long by 12 ft wide by 15-inch thick GAB layer. Starting from the north end, the first 50 ft section was supported by the subgrade soil (CBR = 2) and stabilized by TX130S geogrid. The second 50’ section was supported by the same subgrade soil (CBR = 2) and stabilized by BX1100 geogrid; and

- **Right Lane**: 100 ft long x 12 ft wide x 10-inch thick GAB layer. Starting from the north end, the first 50 ft section was supported by the subgrade soil (CBR = 2) and stabilized by TX130S geogrid. The second 50 ft section was supported by the same subgrade soil (CBR = 2) and stabilized by BX1100 geogrid.

### 3.2.6 Trafficking Equipment

The rear axle of a panel truck was used in conducting this trafficking study. The truck had a dual-wheel rear axle are shown in Figure 38. To verify the truck was delivering the desired American-British standard axle load of 18,000 lbs (18 kips), a 20,000 lb load cell was placed underneath a set of dual wheels and concrete blocks were then loaded into the truck to achieve the rear axle load of 18,000 lbs or the dual-wheel load of 9,000 lbs as shown in Figure 38. Dual wheel loads were measured using a data acquisition and the actual dual-wheel load was measured to be 9,041 lbs.

*Figure 38: Loading concrete blocks into truck to achieve the 18,000 lbs axle load (from SGI, 2011) (from SGI, 2011).*
3.3 Measurements

3.3.1 Overview

During the construction of each test section, the placement of the clay subgrade and base course aggregate were monitored to ensure the materials were placed in a consistent and uniform manner. Elevations of top surface of the subgrade soil layer or base course were surveyed along four sections at 26 points along each section as shown in Figure 39.

![Figure 39: Four selected sections for survey measurements (from SGI, 2011).]

3.4 Test Results

3.4.1 Compacted Clay Subgrade Properties

A summary of the as-placed moisture content and dry unit weight for the compacted clayey subgrade is provided in Figure 40. An example of the DCP results presented as CBR vs. depth of the compacted clayey subgrade is presented in Figure 41 for Location #3.
Figure 40: Summary of as-placed conditions of the constructed subgrade (from SGI, 2011).

TENSAR INTERNATIONAL CORPORATION - FULL-SCALE RUTTING TEST
WATER CONTENTS AND DENSITIES OF SUBGRADE SOIL LAYER

SGI Sample ID # S15970

<table>
<thead>
<tr>
<th>Sample #</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
<th>12</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Depth (in.)</td>
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<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>6.0</td>
<td>12.0</td>
<td>24.0</td>
<td>6.00</td>
<td>6.0</td>
<td>12.0</td>
<td>12.0</td>
<td>24.0</td>
</tr>
<tr>
<td>Water Content (%)</td>
<td>30.2</td>
<td>32.6</td>
<td>32.5</td>
<td>29.2</td>
<td>29.9</td>
<td>28.1</td>
<td>26.9</td>
<td>30.1</td>
<td>29.9</td>
<td>28.8</td>
<td>28.4</td>
<td>27.3</td>
</tr>
<tr>
<td>Dry Unit Weight (pcf) by Driving Cylinder</td>
<td>87.5</td>
<td></td>
<td></td>
<td></td>
<td>88.7</td>
<td></td>
<td></td>
<td></td>
<td>87.9</td>
<td></td>
<td>89.0</td>
<td></td>
</tr>
</tbody>
</table>

Measured
Target (29.0%)
Average of 13 Measured Moisture Content (29.3%)

DATE REPORTED: 6/27/2011
FIGURE NO. C-1
PROJECT NO. SGI11016
DOCUMENT NO. FILE NO.
Figure 41: DCP measurements for Location #3 of compacted clayey subgrade (from SGI, 2011).
3.4.2 Base Course Aggregate

An example of the DCP results presented as CBR vs. depth after final compaction of the base course aggregate is presented in Figure 42 for Location #3.

Figure 42: DCP measurements for Location #3 of compacted clayey subgrade after compaction of base course aggregate (from SGI, 2011).
3.4.3 Trafficking Study

The trafficking of the test sections began on 30 June 2011 and continued until 8 July 2011 for a total of 8 days. A two-axle panel truck with a dual tire rear axle was used to conduct the trafficking. The truck applied a rear axle load of 18,000 lb with 80-psi tire pressure. The trafficking was conducted in two directions through the test sections at a speed of approximately 8 mph. The truck was steered to move along the same path for each pass. Each set of dual wheels moved approximately along the same channel (rut) each time. The rut width created by each set of the dual wheels was approximately 20 to 24 inch wide.

For the left lane with the 15-inch thick base course layer, a total 1,700 18-kip axle passes were performed. For the right lane with a 10-inch thick base course layer, a total 1,200 18-kip axle passes were performed. At the completion of each 100 or 200 passes, the four sections were surveyed. There was no observed heaving observed at any time during the trafficking study. The average accumulated rut depths for both TX130S and BX1100 reinforced unpaved test sections were determined and are presented in Figure 43.
Figure 43: Accumulated rut depth verses the number of 18-kip axle passes for the two geogrids (from SGI, 2011)
After completion of trafficking study, large samples of the geogrids were recovered from the test sections for further evaluation. Two trenches were excavated at two of the survey locations to observe how the geogrids interacted with the soils. Figure 44 shows the TX130S geogrid was embedded into the subgrade and Figure 45 shows the BX1100 geogrid was still on top of the subgrade. Both of these photos show no evidence of the tension membrane effect that was observed after trafficking in the WTI Study. It should also be noted that there was no observed heaving in any of the test sections within the SGI study.

Figure 44: TX130S geogrid ribs partially or fully “penetrated” into subgrade soil after the completion of trafficking study (from SGI, 2011).

Figure 45: BX1100 geogrid ribs partially or fully “penetrated” into subgrade soil after the completion of trafficking study (from SGI, 2011).
4. COMPARISON BETWEEN 2014 WTI AND 2011 SGI STUDIES

4.1 Overview

To assess the similarities and differences between the 2014 WTI and 2011 SGI studies a comparison of the test methods, materials, and test conditions was made to determine the main differences that affected the performance of the geosynthetics used in the two studies. This section presents this comparison and is the basis for recommending the test methodology and conditions that should be used for conducting future field trafficking studies.

4.2 Comparison of Studies

Table 10 compares and contrasts the merits and shortcomings of each study by comparing the various test conditions to evaluate the impact of the differences on the measured results. The conditions compared include:

- Size of test section
- Subgrade properties, classification, thickness, CBR strength, and compaction
- Method of construction (lined or unlined trench) and geosynthetic coverage
- Types of geosynthetics evaluated
- Base course aggregate properties, thickness, strength, and compaction
- Methods of trafficking (trafficking loads, tire pressure, speed, time period, number of passes
- Rut measurements (criteria and depth)
Table 10: Comparison of test details for 2014 WTI and 2011 SGI Trafficking Studies.

<table>
<thead>
<tr>
<th>Condition</th>
<th>2014 WTI Study</th>
<th>2011 SGI Study</th>
<th>Difference</th>
<th>Potential Impact on Geogrid Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size of Test section</td>
<td>16 ft by 825 ft</td>
<td>28 ft by 100 ft</td>
<td>WTI study is larger due to the number of materials evaluated</td>
<td>None</td>
</tr>
<tr>
<td>Imported Subgrade thickness</td>
<td>36 in</td>
<td>24 in.</td>
<td>WTI had a 12 in thicker subgrade</td>
<td>Little effect</td>
</tr>
<tr>
<td>Initial Subgrade CBR</td>
<td>1.4 to 2.0 (1.7)</td>
<td>1.5 to 2.0</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>Subgrade Material</td>
<td>A-6 or CL (sandy lean clay)</td>
<td>A-7-5 or SM (Silty Sand)</td>
<td>WTI study used a fine grain soil and SGI Study used a coarse grain soil</td>
<td>Little effect because the fines content dominates the soil behavior. WTI’s fines classify as “CL” with a PI = 17 and SGI’s fines classify as “ML” with a PI = 11</td>
</tr>
<tr>
<td>Subgrade Compaction</td>
<td>94% ASTM D698 at 6 to 7% points wet of OMC</td>
<td>90% ASTM D698 at 9% points wet of OMC</td>
<td>WTI study soil was placed at a higher relative compaction and lower % points of OMC then SGI study</td>
<td>One would expect better performance (less rutting) in the WTI study due to a drier and denser condition of the subgrade then in the SGI study</td>
</tr>
<tr>
<td>Subgrade Testing</td>
<td>Vane Shear (VS), Light Weight Deflectometer (LWD), Dynamic Cone Penetrometer (DCP), Field California Bearing Ratio (CBR), Nuclear Densometer (In-place Density and Moisture Content), In-Situ Pore Pressure Measurements</td>
<td>In-place Density and Moisture Content, Dynamic Cone Penetrometer</td>
<td>WTI study used more field measurements to confirm the subgrade conditions than SGI. However, both studies verified compaction unit weight, moisture content, and strength (CBR) of the subgrade</td>
<td>Little because both studies verified the subgrade conditions</td>
</tr>
</tbody>
</table>


<table>
<thead>
<tr>
<th>Lining of Subgrade Trench</th>
<th>Yes, a 6 mm thick plastic Liner was used along vertical sides and bottom of subgrade</th>
<th>No</th>
<th>WTI study attempted to control and maintain compaction moisture content of the subgrade soil with installation of a 6-mm thick plastic liner along the bottom and sides of the trench</th>
<th>Large impact because the 6 mm plastic liner could have created a “bath tub” situation that collected rainfall before and during the trafficking study causing an increase in subgrade moisture content and reduction in subgrade CBR and shear strength.</th>
</tr>
</thead>
</table>
| Geogrid and Geotextile Reinforcement Types | Tensar BX Type 2  
Tensar TX140  
Tensar TX160  
Enkagrid Max 30  
Fornit 30  
Mirafi BXG 11  
Secugrid 30/30 Q1  
Tenax MS 330  
Geotex 801  
Mirafi RS589i  
Synteen SF 11  
Synteen SF 12 | Tensar BX1100  
Tensar TX130S | WTI study evaluated more materials than the SGI study, but similar Tensar geogrids were used in both studies | None because the Tensar BX Type 2 biaxial geogrid is comparable to Tensar BX1100 biaxial geogrid and Tensar TX140 and TX160 triaxial geogrids are comparable to Tensar TX130S triaxial geogrid |
| Geogrid and Geotextile Instrumentation | Strain gage and LVDT Measurements | None | WTI study attempted to collect additional data from reinforcing materials | None because the measured rutting depth, not LVDT data, was used to evaluate performance |
| Subgrade Coverage by Geogrid/Geotextile | Incomplete coverage across subgrade | Complete coverage of subgrade with 12 inch overlaps | WTI study used only the provided roll width of each material and centered the material within the test section while SGI overlapped rolls to ensure coverage of entire test section width. | Potentially large impact on rutting depth due to limited geogrid coverage. Several of the narrowest products, e.g., Tensar TX geogrid, only covered 80% of the test lane and were 25% narrower than the best performing geosynthetic that covered the entire test area in the WTI study.  

This is significant when geosynthetics are forced to act as a tensioned membrane and |
anchorage length beyond the wheel path becomes a critical factor.

<table>
<thead>
<tr>
<th>Base Material</th>
<th>Montana’s 5A Base Course A-2-4 or GP-GC</th>
<th>Georgia GAB A-1-a or GW-GM</th>
<th>Little difference because both aggregates have similar grain size distributions</th>
<th>Significate effect due to the Average Size Ratio (ASR) of each material 150 for the WTI material and 44 for the SGI material. The ASR of 150 for the WTI study is significantly higher than the desired ASR of 25 indicating an improper design where a nonwoven geotextile should have been placed below all of geogrids in the test sections. As seen in Figure 50, post trafficking there were significant fines contamination in the base materials in each of the geogrid test sections. It appears that both materials have similar percentage of fractured faces.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base Material Thickness</td>
<td>15 test sections with 12-inch thick base (also one test section of 16 in and 25 in)</td>
<td>10- and 15-inch thick base course layers</td>
<td>Both studies used similar base course thicknesses.</td>
<td>SGI study may have been more severe with a 10-inch thick base course but 10 and 15 inches encompasses the 12 inches used by WTI.</td>
</tr>
<tr>
<td>Base Material Placement</td>
<td>Montana 5A gravel was prepared by adding water and mixing it with an end loader until it reached the desired optimum moisture content. During placement of the gravel, the material was dumped from the side onto the geosynthetics and Moisture conditioned GAB material was placed on top of geogrids using a lightweight rubber-belt dozer. Each loosely placed GAB material layer was first leveled and rolled by the lightweight rubber-belt dozer, and then compacted by using</td>
<td>Both studies used very different placement techniques for base material placement.</td>
<td>Significate effect due the non-representative placement of the aggregate layer above the clay subgrade soil in the WTI study. This placement technique of screening the base and then rolling resulted in the fact some of the geosynthetics, e.g., geogrids, not developing good interlock with the aggregate layer. For example, the aggregate layer was not compacted to the same level experienced in most field applications through the use of dozers, compactors and trafficking by dump trucks, which resulted</td>
<td></td>
</tr>
</tbody>
</table>
then pushed or screeded over the geosynthetics. The gravel was never tracked in place due to concerns over unequal construction traffic over the various test sections. An Ingersoll Rand SD45F vibratory compactor for 10 passes to achieve approximately 95% of maximum standard Proctor density. In limited aggregate interlock with the geogrids in the various test sections.

<table>
<thead>
<tr>
<th>Base Material Testing</th>
<th>Light Weight Deflectometer (LWD), Dynamic Cone Penetrometer (DCP), Field California Bearing Ratio (CBR), Nuclear Densometer (In-place Density and Moisture Content)</th>
<th>In-place Density and Moisture Content, Dynamic Cone Penetrometer</th>
<th>WTI used more field measurements to confirm conditions of the aggregate base than SGI but both studies verified compaction unit weight, moisture content, and strength (CBR) of the base course aggregate</th>
<th>Little impact because both studies verified initial base course conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Truck Loading</td>
<td>3 axial dump truck Dual tire rear axels Total load 16,920 lb per rear axial</td>
<td>2 axial truck Dual tire rear axial Total load 18,000 lb at rear axial</td>
<td>Different trucks were used in each study but the single axial loads are similar</td>
<td>Little impact because axial loadings are similar</td>
</tr>
<tr>
<td>Truck Tire Pressure</td>
<td>90 psi</td>
<td>80 psi</td>
<td>WTI used a slightly higher tire pressure than SGI</td>
<td>Little impact because tire pressures are similar</td>
</tr>
<tr>
<td>Truck Speed</td>
<td>5 mph</td>
<td>8 mph</td>
<td>SGI truck used a faster speed than WTI</td>
<td>Little impact because both studies minimized dynamic wheel loading</td>
</tr>
<tr>
<td>Trafficking Method</td>
<td>Channelized traffic in one direction and stayed within wheel path of previous pass.</td>
<td>Channelized traffic in two directions and stayed within wheel path of previous pass.</td>
<td>WTI study used uni-directional trafficking compared to two-way trafficking by SGI.</td>
<td>Little impact because both studies used channelized trafficking and minimized wheel wander.</td>
</tr>
<tr>
<td>Trafficking Period</td>
<td>2 months</td>
<td>8 days</td>
<td>WTI study required longer duration to accomplish a lower number of passes compared to SGI study</td>
<td>Extended duration to complete the WTI study could have significant impact because there is greater likelihood of precipitation being trapped in clayey subgrade trench due to presence of 6-mm thick plastic liner.</td>
</tr>
<tr>
<td>--------------------</td>
<td>----------</td>
<td>--------</td>
<td>-------------------------------------------------------------------------------------------------</td>
<td>-------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Number of Passes</td>
<td>740</td>
<td>1,200 and 1,700 for 10 and 15 inches of GAB</td>
<td>SGI study applied a greater level of trafficking than WTI study</td>
<td>Some effect for SGI Study because increasing passes usually results in greater rutting but geogrids performed better in SGI Study even with a greater number of passes.</td>
</tr>
<tr>
<td>Rut Failure Criteria</td>
<td>3 inches</td>
<td>Number of passes fixed (1,200 for 10&quot; thick base and 1,700 for 15&quot; base). Testing was not continued to &quot;failure&quot; (i.e., 3&quot; rutting depth).</td>
<td>Different failure criterion was used in the studies.</td>
<td>In WTI study, all test sections reached failure, i.e., a rut depth of 3 inches, and some sections exceeded 3 inches under only 740 passes. In SGI study, neither geogrid test section reached failure, i.e., a rut depth of 3 inches, even though 1,200 and 1,700 passes were applied. The greater rutting and reduced number of passes in the WTI study implies the geosynthetics transitioned from providing lateral confinement to tensioned member behavior making the results of limited value.</td>
</tr>
<tr>
<td>Rut Depth Measurement Frequency</td>
<td>Variable ranging from 20 to 200 passes</td>
<td>Every 100 to 200 passes</td>
<td>None</td>
<td>None because rut depth measurements were taken frequently in both studies</td>
</tr>
</tbody>
</table>
5. SUMMARY OF FINDINGS

5.1 Overview

This section summarizes the findings from the comparison of the 2014 WTI and 2011 SGI studies (see Table 10) and presents the impact of these findings on the measured results. The impact of these findings were then used to determine the proper test methodology and conditions for conducting field trafficking studies involving geosynthetics.

5.2 Discussion of Findings

There are a number of important differences and findings from the comparison of these to similar unpaved road studies (see Table 10) that are believed to have significant impact on the outcome and the differing results of the two studies. These key findings are presented below for the three main factors, i.e., compacted clay subgrade, geosynthetics, and selection of base course aggregate:

Effect on Compacted Clay Subgrade

- Extended period of trafficking of 60 days in the WTI study compared to a short trafficking period of 8 days in the SGI study.
- During the extended period of trafficking in the WTI study there were a number of rainstorms that produced accumulations greater than 0.1 inches in 24 hours for a total of 1.4 inches during the trafficking phase of the study compared to no reported rainfall during the SGI study.
- The use of a 6-mil thick plastic liner encapsulating the subgrade soil and creating a “bath tub” situation that collected and entrapped the rainfall compared to the SGI study where the clay subgrade was not compacted in a trench or encapsulated in a plastic liner.
- Excessive amount of rutting that occurred in the WTI study (in excess of 3 inches) compared to the SGI study (with a maximum of 1.6 inches) even though much fewer passes were used in the WTI study.
- Significantly fewer trafficking passes in the WTI study (maximum of 740 passes) compared to the SGI study (minimum of 1,200 and maximum of 1,700 passes).
- Evidence of bearing capacity failures (heaving) along the wheel paths during trafficking in the WTI study compared to no observed heaving in the SGI study.
Effect on Geosynthetics

- Incomplete coverage of the geosynthetics across the wide of the subgrade in the WTI study compared to complete coverage across the wide of the subgrade with 12 inch overlaps between one layer of geosynthetic with another layer of the same geosynthetic to provide continuous coverage in the SGI study.
- Observation of the geosynthetics transitioning from providing lateral confinement to acting as tensioned membrane support in the WTI study compared to the observation that the geosynthetics were providing lateral confinement in the SGI study.

Effect on Base Course Aggregate

- Lack of compaction of the poorly graded gravel base course aggregate in the WTI study compared to the SGI study. In the WTI study the aggregate was removed using compressed air and the DCP data 2” adjacent to the geosynthetics and at the surface were ignored when determining in-situ strength values.

5.3 Impact on Measured Performance of Tensar Geogrids

One of the main goals of this report was to determine the main factors (conditions) that affected the performance of Tensar geogrids in both trafficking studies. The 2014 WTI rutting results indicate rutting of three (3) inches or more especially for the Tensar TX140 and TX160 geogrid test sections with less than 700 truck passes. This differs greatly from the maximum rut depth of only 1.6 inches for the Tensar BX1100 geogrid in the SGI study after about 1,200 similar truck passes. Based on the comparison above, this difference in performance is due primarily to the following four factors: (1) lack of subgrade support causing heaving and bearing capacity failures, (2) development of tensioned membrane support instead of lateral confinement by the geogrids in the WTI Study, (3) lack of complete coverage of the width of the test section in the WTI Study, and (4) the potential of improper compaction of the base coarse aggregate above the geogrid in the WTI study. All four of these factors are interrelated and probably had a significant impact on the measured results of the WTI study.

The authors of this report believe the excessive rutting observed during the WTI study can be directly attributed to the weakening of the subgrade and consequent under design of the sections that resulted in the geosynthetics providing tension membrane support instead of lateral confinement as anticipated. Additionally, the method of construction would likely result in poor compaction of the base course aggregate reducing interlock within the geogrid products to effectively stabilize the base course aggregate. However, to experience the amount of rutting that occurred during the WTI study, the primary factor has to be inadequate design due to weakening of the subgrade via increased moisture content and reduced shear strength. By not taking into
account the lower subgrade strength the test sections in the WTI study were under designed and inadequate for the intent of the study.

The weakening of the subgrade can be traced to the WTI investigators deciding to encapsulate the compacted clay subgrade soil in a 6-mil thick plastic liner that was placed along the bottom and all sides of the excavated trench (see Figure 6 and Figure 7) prior to placement of the subgrade material. The benefit of using the plastic liner was believed to be for isolation and moisture content control of the compacted subgrade soil and reduce any desiccation prior to or during the WTI trafficking. However, installation of this plastic liner created a “bath tub” like condition that not only retained the subgrade compaction moisture content conditions but also the rainfall that occurred before and during the WTI Study. As discussed in Section 2.4.3 (Trafficking Study), there were a number of rain events with accumulations greater than 0.1 inches over a 24-hour period producing a total accumulation of 1.4 inches during the 60-day study (see Figure 24). Review of the precipitation data in Figure 24, indicates the total rainfall accumulation is closer to 2.5 inches during the 60-day study and not 1.4 inches as reported in the WTI study. The effect of these rain events becomes apparent when their occurrence is compared with the cumulative truck passes, the timing during the study, observed heave (bearing capacity failures), LVDT measured displacements along the geosynthetics, strains within the geosynthetics and the cumulative rut depths within each test section during the WTI study, which are discussed in the following paragraphs.

Looking closer at the rain events highlighted in the red box in Figure 46, the majority of the rain events occurred between the 2nd and 18th of October 2012. This time period corresponds to approximately 1.9 inches of accumulated rainfall in only 16 days or approximately 75% of the total accumulated rainfall during the entire study. This period of time also corresponds to the time period when approximately 225 to 400 truck passes were applied to the test section (green box in Figure 46). This time period and the related number of truck passes had a significant effect on the measured test results in the WTI study as discussed below.

The corrected rut response for all of the test sections in the WTI study is presented in Figure 47 and it shows that significant rutting occurred between approximately 225 and 400 truck passes, which is the same period when the majority of the rain events occurred. This period of rutting is highlighted in the red box in Figure 47. During this sequence of truck passes there was an increase in rut depth from 1.25 inches to over 3 inches for most of the test sections. In particular, for Test Section 11 (Tensar TX140 geogrid) and Test Section 12 (Tensar TX160 geogrid), there was an increase of 2.10 and 1.80 inches, respectively, of rut depth or 60 and 55% of the total accumulated rut depth, respectively, occurred during the entire WTI study. For Test Section 3 (Tensar BX Type 2 geogrid) there was an increase of 0.6 inches of rut depth or approximately 25 % of the total accumulated rut depth respectively during this time period. An explanation for this significant increase in rut depth over this specific period of 175 truck passes can be attributed to the observed
bearing capacity failures of the subgrade along the road surface parallel to the wheel paths by evidence of heaving of the road surface as discussed in Section 2.4.3 (Trafficking Study) and presented in Figure 25. The bearing capacity failures can be attributed to the loss of shear strength in the compacted clay subgrade due to increased moisture content due to the 1.9 inches of rainfall that occurred over this 16-day period. This precipitation increased the moisture content of the clay subgrade because some of it infiltrated the subgrade and was then trapped in the trench by the 6-mil thick plastic liner that encapsulated the subgrade material.

Figure 46: Highlighted precipitation events from 2 October to 18 October 2012 and the associated number of truck passes (from Cuelho et. al, 2014)

Figure 47: Highlighted rut response for all test sections (from Cuelho et. al, 2014)

Focusing on the performance of the Tensar geogrids for this study, Figure 48 shows the transverse rut profiles for the Tensar TX140 geogrid test section, which confirms development of
road surface heave (bearing capacity failure) between 225 and 300 truck passes. **Figure 49** and **Figure 50** shows the transverse rut profiles for the Tensar TX160 geogrid test section. In particular, **Figure 49** shows Test Section 12 (north) and **Figure 50** shows Test Section 12 (South) transverse rut profiles for the Tensar TX160 geogrid test section. Both of the graphs confirm development of road surface heave (bearing capacity failure) between 225 and 300 truck passes. These transverse rut profiles for the Tensar TX140 and TX160 geogrid test sections are also consistent with the overall heave experienced for all of the test sections between 225 and 300 truck passes, as shown in comparison highlighted in the red box in **Figure 51**.

![Figure 48: Transverse rut profiles for Test Section 11 (north) with Tensar TX140 geogrid (from Cuelho et. al, 2014)](image)

![Figure 49: Transverse rut profiles for Test Section 12 (north) with Tensar TX160 geogrid (from Cuelho et. al, 2014)](image)
To further support the main factor for poor performance of the Tensar geogrids being the weakening of subgrade support (strength) and consequent under design of the section, the dynamic LVDT displacements and the strains measured with the bonded strain gauges are reviewed below showing the measured displacements increased during the observed 16-day rutting period. Figure 52 presents the dynamic LVDT displacements for the Tensar TX140 geogrid in Test Section 11.
(North) and shows that significant displacements (0.2 to 0.5 inches), i.e., the majority of the measured displacements, were measured between 225 and 400 truck passes as shown by the red box in Figure 52. Figure 53 presents the dynamic LVDT displacements for the Tensar TX160 geogrid in Test Section 12 (South) and shows even greater displacements (0.1 to 0.8 inches) than Test Section 11 between 225 and 400 passes as shown by the red box in Figure 53. In particular, Figure 53 shows that more than half of the measured LVDT displacements for the Tensar TX160 geogrid occurred between 225 and 400 truck passes, which is the same time period in which about 1.9 inches of rainfall occurred.

**Figure 52:** Dynamic LVDT displacement results for Test Section 11 (North) with Tensar TX140 geogrid (from Cuelho et. al, 2014).

**Figure 53:** Dynamic LVDT displacement results for Test Section 12 (South) with Tensar TX160 geogrid (from Cuelho et. al, 2014).
The dynamic LVDT strains calculated from the LVDT displacement data in Figure 52 and Figure 53 provide additional support for the concept that there was a significant weakening of the subgrade during the 16-day period (2 October to 18 October 2012) of about 1.9 inches of rainfall that resulted in the poor performance of some of the geosynthetics in the WTI Study. Figure 54 presents the dynamic LVDT strains for the Tensar TX140 geogrid in Test Section 11 (North) and shows that significant strains (0 to 2.5%) occurred between 225 and 400 truck passes as shown by the red box in Figure 54. Figure 55 presents the dynamic LVDT strains for the Tensar TX160 geogrid in Test Section 12 (North) and shows even greater strains (1.0 to 3.5%) than Test Section 11 (North) between 225 and 400 passes as shown by the red box in Figure 55. In particular, Figure 55 shows the majority of the measured LVDT strains for the Tensar TX160 geogrid occurred between 225 and 400 truck passes, which is the same time period in which about 1.9 inches of rainfall occurred between 2 October to 18 October 2012.

Figure 54: Dynamic LVDT strain results for Test Section 11 (North) with Tensar TX140 geogrid (from Cuelho et. al, 2014).
With the above observations from the WTI trafficking study there is clear evidence that the geosynthetics, e.g., Tensar geogrids, transitioned from providing lateral confinement of the poorly graded gravel to tension membrane support of the geosynthetic as the rut depth increased deeper and bearing capacity failures (soil heave) were occurring. This transition was reported by WTI to occur between 100 and 300 truck passes or at or before 2.0 inches of rut depth in most of the test sections. This is highlighted by the red box in Figure 56, which compares the heave, longitudinal rut, and change in displacement for all of the test sections. The red box in Figure 56 corresponds to 225 and 400 truck passes, which is the time period in which about 1.9 inches of rainfall occurred between 2 October to 18 October 2012. Figure 56 shows the majority of the heave and rutting occurred between 225 and 400 truck passes.
Finally, there are some direct quotes in the WTI Study Report that support the occurrence of subgrade strength loss and transition from lateral confinement to tension membrane support. The following are some of these quotes, which are presented with the page number in the original WTI Report:

“Mechanical properties of geosynthetics were compared to truck passes at the transition from lateral confinement to membrane support…” (page 3)

“…test sections also showed evidence of tension (sic) membrane effect. After the base course was removed, the geosynthetic in the wheel path of several of the test sections was taut and, due to the tension in the material, became elevated from the permanent rut contour in the surface of the subgrade…” (page 81)

“…strength and stiffness of the junction correlated better with the performance as rut increased…” (page 96)

“…tensile strength in the material is also a good indicator of performance.” (page 97)

“…tensile strength in both material directions relate (sic) to performance at higher levels of rut…” (page 99)

“…woven and nonwoven geotextiles performed well…” (page 100)

“As the rut depth increased under increased truck passes, however, distortion of the rut bowl caused the gravel to gradually lose its ability to spread laterally which in turn caused the stresses in the base coarse layer to become more vertical…” (page 103)

“As the subgrade and the base were continually shoved away from the center of the rutted area under continued traffic loading, the primary mechanism of support from the geosynthetics transitioned from lateral confinement to tensioned membrane (refer to Figure 1).” (page 103). This Figure 1 is presented below as Figure 57 to emphasize “diagram (c) - membrane tension support”, which includes rutting and distortion of the geosynthetic and subgrade observed during the WTI study.

The rutting and distortion depicted in Figure 57(c) is directly related to the loss of subgrade support (strength) that allows excessive rutting to occur, which causes the geosynthetic to become loaded in tension. Thus resulting in the geosynthetic becoming strained and providing the vertical
support observed in the WTI study. This mechanism never occurred in the SGI study because the subgrade strength assumed in the design of the section was maintained in the field. The sections therefore performed as they were designed. The geogrids in the SGI performed a stabilization function, providing lateral confinement (restraint) as shown in Figure 57(a) and increased bearing capacity as shown in Figure 57(b).

![Figure 57: Possible reinforcement functions provided by geosynthetics in subgrade stabilization applications (from Haliburton et al., 1981).](image)

6. CONCLUSIONS

The following conclusions can be drawn from the evaluation of these two subgrade stabilization (trafficking) studies that help explain the significant difference in performance of several geosynthetics including Tensar geogrids in the two studies:

- First, there was a significant loss in subgrade strength during the WTI study, which resulted in the under design of the sections leading to heave and bearing capacity
failures in many of the test sections. As heaving developed, there was a transition of geosynthetic performance from lateral confinement to tensioned membrane support. This continued through the majority of the study and caused all but one of the test sections to fail, i.e., exhibit greater than three (3) inches of rut depth, within 400 truck passes. This loss in subgrade support or shear strength was primarily caused by the inclusion of a 6-mil thick plastic liner to encapsulate the compacted clay subgrade and about 1.9 inches of rain occurring between 2 October and 18 October 2012, i.e., most of the trafficking. The WTI test section was constructed using a trench excavation that is 16 ft wide and 3 ft deep trench to construct the 3 ft thick compacted clay subgrade on which the geosynthetics and overlying aggregate layer were placed. Before the clay soil subgrade was placed in lifts and compacted in the trench, the trench was lined with a 6-mil thick plastic liner believed to help maintain the initial compaction moisture content of the clay subgrade throughout the study. Unfortunately, this plastic liner resulted in the clay subgrade being softer than expected because not only the compaction moisture was retained in the trench but the precipitation that occurred before and during testing was also retained in the trench. The additional water caused an increase in moisture content reducing the shear strength of the compacted clay subgrade, which was initially partially saturated and able to absorb the moisture due to the suction pressures initially present. During truck passes 225 to 400, i.e., between 2 October and 18 October 2012, excessive pore-water pressures may have developed within the compacted clay subgrade contributing to the observed heave and bearing capacity failures. However, the presence of air bubbles in the pore-water pressure sensors prevented confirmation of the dissipation of the initial suction pressures and development of excess pore water pressures within the compacted clay subgrade during trafficking. Even if the sensors were working properly, WTI decided to stop recording all of the pore-water pressure sensors after 251 truck passes due to the onset of cold weather.

- Another reason for the poor performance of some geosynthetics in the WTI Study is the non-representative placement of the aggregate layer above the clay subgrade soil. This placement technique of screeding the base and then rolling resulted in the fact some of the geosynthetics, e.g., geogrids, not developing good interlock with the aggregate layer. For example, the aggregate layer was not compacted to the same level experienced in most field applications through the use of dozers, compactors and trafficking by dump trucks, which resulted in limited aggregate interlock with the geogrid. Additionally, the WTI study was not properly designed based on the actual Average Size Ratios of the aggregate layer and clay subgrade materials.
In summary, the measured behavior of geosynthetics in the 2014 WTI study is different than measured during the 2011 SGI study and observed in other field installations and laboratory testing. The maximum rut depth that was achieved in the 2011 SGI study is approximately 1.6 inches after 1,200 truck passes, which differs from the greater than three (3) inches in only 400 truck passes. The fundamental differences between the 2011 SGI and 2014 WTI studies are: (1) the compacted clay subgrade was not encapsulated in a plastic liner in the SGI study, (2) the SGI study occurred over a period of 8 days compared to 60 days for the WTI study. Additionally, the SGI study was properly designed taking into account the Average Size Ratios and standard construction methods which allowed for proper stabilization and interlock with the geogrid, (3) there were no rain events reported during the SGI study compared to about 1.9 inches of rainfall during the WTI study, and (4) the construction methods used in the SGI study represented those typically used in the field and enabled effective interlock to develop. The construction method, lack of rain events, the short period of trafficking, and no encapsulation of the compacted clay subgrade in the SGI study allowed the stabilized road to exhibit similar behavior as observed in other field installations and laboratory testing over the past thirty (30) years. Unlike the WTI study, the SGI study produced results that are in agreement with current state-of-the-practice design procedures, e.g., Giroud and Han (2011), and observations of field installations. Thus, the 2011 SGI Study used construction techniques that are more typical of field applications and produced results that are more indicative of the performance of geosynthetics in actual unpaved road applications than the 2014 WTI Study.

7. REFERENCES


Work platforms built with geotextile tubes at the Lach Huyen Bridge
Mechanically stabilized roadways over peat soils at the Braes of Doune Wind Farm in Scotland. Photo: Tensar International
Mechanisms governing the performance of unpaved roads incorporating geosynthetics

By J.P. Giroud and Jie Han

Introduction

Unpaved road performance mechanisms are complex

The mechanisms that govern the performance of unpaved roads are complex for two reasons. First, the road structure is highly heterogeneous: the two materials (granular in the base and cohesive in the subgrade) behave differently, which makes the mechanisms complex. Second, the road structure is progressively modified by load repetition due to traffic. If the road structure were not progressively modified by the traffic, the road performance would be simple: either the road would fail after one vehicle pass or it would never fail. In contrast, it is complex to analyze the performance of a structure that evolves with vehicle passes.

When a geosynthetic is added, the road structure becomes even more heterogeneous and the mechanisms more complex. Furthermore, the wheel load is vertical and the geosynthetic is horizontal. This situation is more complex than, for example, the situation of a geosynthetic incorporated in a sloping soil layer (i.e., the veneer stability situation) where the driving force and the geosynthetic are in the same direction.

As a result of these complexities, there are sometimes misunderstandings regarding the mechanisms that govern the performance of unpaved roads, especially unpaved roads incorporating geosynthetics. It is important to identify and clarify these mechanisms.

Overview of unpaved road performance mechanisms

Roads (paved and unpaved) are subjected to traffic by vehicles on wheels. Traffic is a type of repeated loading characterized by axle load and number of axle passes. The function of the road is to support the load for a certain minimum number of axle passes. Clearly, there are two aspects in the function of a road: load support and service life. Accordingly, a geosynthetic can improve a road in two ways: by contributing to load support and by increasing the service life.

The geosynthetic contribution to the performance of an unpaved road is achieved through several mechanisms that take place in the road structure, which consists of a base (made of granular material) resting on subgrade soil (typically a cohesive soil), with a geosynthetic included between the base and the subgrade. Mechanisms
related to the base will be presented first, followed by mechanisms related to the subgrade; finally the mechanism of separation between base and subgrade will be presented.

The focus of this article is on mechanisms that govern the performance of unpaved road structures under repeated traffic loads. This article does not address mechanisms that are not directly related to traffic, such as subgrade swelling (due, for example, to frost or presence of expansive soil).

Road improvement mechanisms related to the base
Thanks to the presence of an adequate geosynthetic, the performance of the base of an unpaved road is improved through two closely-related mechanisms.

Improvement of wheel load distribution
The role of the granular base is to distribute the wheel loads so that, if the base is sufficiently thick and stiff, the maximum vertical stress applied to the subgrade is below the stress level that causes excessive deformation of the subgrade soil. The total load on the subgrade is the same as the total load applied by the vehicle (plus the weight of the granular base), but, thanks to load distribution by the base, the load is applied over a much wider area than the contact area between the wheels and the road surface, which reduces the maximum vertical stress on the subgrade. The mechanism through which the granular base distributes the load is explained below.

From a mechanical standpoint, the granular base and the subgrade soil form a two-layer system. It is known from the theory of elasticity that, in a two-layer system, the stress distribution on the lower layer depends on the relative moduli of the upper layer and the lower layer. Thus, a high-modulus granular base (i.e., a stiff base) drastically reduces the maximum vertical stress on the subgrade compared to the maximum vertical stress at the same depth in a uniform soil. Therefore, the stress distribution effectiveness of the upper layer of a two-layer system can be increased by increasing the modulus of the upper layer, which is achieved by adding tensile stiffness to the upper layer, hence the use of a geosynthetic at the bottom of the granular base. A granular material is strain-dependent and stiffness is increased by reducing strain. Therefore, increasing tensile stiffness is linked to the lateral restraint mechanism discussed in the next section.

While “modulus” is the property generally mentioned in two-layer systems, the modulus involved in roads is the resilient modulus (i.e., the modulus value based on recoverable strain).

Just adding a geosynthetic at the bottom of the base is not sufficient (especially with repeated loading). It is essential that the geosynthetic and the granular material closely interact to form a composite system characterized by high tensile stiffness. This interaction is achieved by interlocking in the case of geogrids and confinement in the case of geocells. In the case of geotextiles, the interaction is based on friction, which is less effective than interlocking or confinement.

Lateral restraint of base course material
The theory of elasticity, which explains load distribution (as indicated above), shows that there are tensile stresses at the bottom of the upper layer of a two-layer system. In a road, tensile stresses are repeated at each axle pass. In the absence of an appropriate geosynthetic, these tensile stresses progressively cause displacement of particles of the granular base material, mostly in the horizontal direction, which is referred to as lateral spreading of the base material.
Lateral spreading of granular particles because of repeated traffic loads is one of the three dominant mechanisms of road base deterioration, along with the base-subgrade intermixing mechanism (discussed in a subsequent section) and particle breakage (especially in the case of angular aggregate). The lateral spreading of granular particles makes the base thinner and the base material less stiff. The decrease in base thickness contributes to the rutting observed at the road surface and the decrease in base thickness and stiffness progressively decreases the ability of the base to distribute the load transferred from the wheels to the subgrade soil. This increases the maximum vertical stress on the subgrade and, consequently, the deformation of the subgrade soil. Clearly, lateral spreading of granular particles of the base has significant detrimental consequences on both the base and the subgrade.

Lateral spreading of granular particles of the base is reduced and slowed thanks to lateral restraint provided by a geocell filled with granular material or a geogrid located within the base or at the bottom of the base. Evidence of lateral restraint has been provided by an increase of horizontal stress measured in a base incorporating a geogrid (Wayne et al. 2013). Indeed, a granular material is strain-dependent, as mentioned above; therefore, it is stress-dependent and lateral restraint is associated with an increase in horizontal stress.

The geogrid provides lateral restraint by interlocking with granular particles. Geogrid-granular particle interlocking depends on several factors including:

- geogrid aperture size relative to granular particle size and grading,
- geogrid aperture shape,
- shape and stiffness of the geogrid ribs,
- stiffness (more than strength) and integrity of junctions between ribs.

A detailed discussion of interlocking is in a paper by Giroud (2009).

While both geogrids (through interlocking) and geocells (through confinement) prevent lateral spreading of the base material, geogrids are easier to install and less expensive than geocells; as a result, geogrids are most commonly used in unpaved roads. As indicated above, geotextile/base material interaction relies on friction, which is less effective than interlocking or confinement.

Geogrids and geocells are also effective in preventing shear failure of the base.

**Conclusion on mechanisms related to the base**

The two mechanisms related to the base (improvement of load distribution and lateral restraint of base material) are closely linked. Both rely on geosynthetic/granular material interaction, which prevents lateral spreading of the base material and imparts tensile stiffness to the base. Furthermore, it will be seen in the next section that load distribution is also beneficial to the mechanical behavior of the subgrade.

**Road improvement mechanisms related to the subgrade**

Thanks to the presence of an adequate geosynthetic, the performance of the subgrade of an unpaved road is improved through three mechanisms.

**Impact of load distribution on subgrade soil resilient modulus**

If the maximum vertical stress on the subgrade is reduced because of the presence of a geosynthetic at the bottom of the base, the vertical stress on the subgrade is more uniformly distributed than in the absence of a geosynthetic. Consequently, the deviator stress (the difference between vertical and horizontal stress) in the subgrade material is decreased compared to the case without geosynthetic, as shown by Wayne et al. (2013). In the case of very small subgrade rutting—rutting on top of...
the subgrade (e.g., <13mm [0.5in.])—and firm subgrade (California Bearing Ratio [CBR] greater than about 3%), i.e., in situations where base and subgrade are stable, the reduction of deviator stress results in an increase of the resilient modulus of the subgrade, as shown by Elliott and Thomson (1985). The increase in resilient modulus of the subgrade reduces the deformation of the subgrade under load compared to the case where there is no geosynthetic.

**Subgrade soil vertical restraint**

In the case of unpaved roads without geosynthetic, stress distribution on the subgrade is not uniform. The subgrade soil, being more loaded under the wheels, tends to move upward in zones located between and outside the wheel load areas. In these zones, in particular between the wheels, the geosynthetic, after some deformation under traffic load, has a convex shape and applies a vertical stress on the subgrade soil. The weight of the base, which is uniformly distributed, and the fact that the wheel load is almost uniformly distributed (thanks to the geosynthetic in the base) contribute to applying a quasi-uniform vertical stress on the subgrade soil between and outside the wheel load areas.

The presence of a relatively uniform vertical stress on each side of the wheel load areas at the surface of the subgrade is similar to the lateral surcharge that is known to increase bearing capacity in foundation design. Also, the joint action of geosynthetic tension and geosynthetic-improved load distribution results in vertical restraint of the subgrade. As a result of lateral surcharge combined with vertical restraint, the subgrade soil may be loaded near its ultimate bearing capacity without excessive deformation, as demonstrated by Giroud and Han (2004). In contrast, without vertical restraint of the subgrade, a wheel load causing a vertical stress equal to the ultimate bearing capacity of the subgrade soil would cause excessive subgrade deformation and immediate failure.

Based on the above discussion, unpaved roads without geosynthetic must be designed to avoid loading the subgrade to its ultimate bearing capacity. They must be designed for the maximum vertical stress on the subgrade to be equal to the elastic limit of the subgrade soil, which is $3.14 \, c_u$ ($c_u$ being the undrained cohesion of the subgrade soil). Accordingly, as shown by Giroud and Noiray (1981) with further refinements by Giroud and Han (2004), the allowable stress on the subgrade soil is $3.14 \, c_u$ without geosynthetic, $5.14 \, c_u$ with a geotextile, and $5.71 \, c_u$ with a geogrid. The difference between the geotextile case and the geogrid case is due to the difference in stress orientation at the base/subgrade interface, which results from the difference between geotextile/granular material interface friction and geogrid/ granular material interlocking (i.e., the classical difference between smooth and rough base in foundation design).

**Load transfer by the tensioned membrane effect**

Under specific conditions, a geosynthetic located between the base and the subgrade can contribute to load support through a mechanism called “tensioned membrane effect.” This effect has been extensively discussed in the literature because, in early attempts at explaining the performance of unpaved roads, it was thought that the tensioned membrane effect was the main mechanism governing the performance of geosynthetics in unpaved roads. It is known today that this is not the case.

The tensioned membrane effect decreases the vertical load induced in the subgrade soil under the wheels by transferring part of the vertical load laterally (i.e., away from the wheels).

The mechanism is the following:
• If the subgrade soil undergoes large deformation because of traffic, the geosynthetic follows the shape of the subgrade and exhibits a concave shape under the wheels.
• The geosynthetic thus deformed is subjected to tension.
• The resultant of the geosynthetic tension on the two sides of the concave shape is an upward vertical force that contributes to wheel support.
• The tension of the geosynthetic on each side of the concave shape laterally transfers the portion of the wheel load supported through the tensioned membrane effect. As a result, smaller vertical stresses are applied to the subgrade beneath the wheels and greater vertical stresses are applied to the subgrade away from the wheels compared to the case without geosynthetic. Thus, thanks to the tensioned membrane effect, the vertical stress distribution on the subgrade is more uniform.

From this analysis, it is clear that the tensioned membrane effect requires a high-strength geosynthetic and deep rutting. Calculations show that, for typical rut depths (less than 100mm), the tensioned membrane effect is generally negligible. Also, the tensioned membrane effect works only with channelized traffic (traffic that keeps deepening the same ruts), which may not exist in the case of wide unpaved roads where traffic may wander. Another limitation of the tensioned membrane effect is the need for sufficient anchor length for the geosynthetic on each side of the axle.

Thus, the tensioned membrane effect is not the main mechanism governing the performance of unpaved roads. In usual service conditions, the tensioned membrane effect is almost always negligible in unpaved roads incorporating geogrids and geocells because such unpaved roads do not generally exhibit large deformations and the tensioned membrane effect carries a fraction of the wheel load when geogrids and geotextiles with high strength and high tensile modulus are used in unpaved roads that exhibit very deep ruts (Giroud and Noiray 1981, Giroud et al. 1984).

While the tensioned membrane effect does improve the performance of unpaved roads (such as some construction site access roads and lumber extraction roads) where deep ruts are acceptable, it is not really a soil improvement mechanism. The beneficial effect of this mechanism is applied directly to the load, not to the soil. In other words, the tensioned membrane effect is beneficial to the subgrade because it decreases the maximum vertical stress on the subgrade, but it does not directly improve the subgrade. However, it may be considered that the tensioned membrane effect results in long-term subgrade improvement because the repeated maximum vertical stress on the subgrade, which causes progressive deterioration of the subgrade, is reduced compared with the case without geosynthetic. The magnitude of the tensioned membrane effect tends to increase as the geosynthetic deflection increases at each vehicle pass, which further reduces the repeated maximum vertical stress on the subgrade, provided that: (i) the deterioration of the base due to the large deformation associated with the tensioned membrane effect is compensated by placing additional base material in the ruts (a standard practice); and (ii) the geosynthetic is able to resist the resulting additional tension.

Conclusion on mechanisms related to the subgrade

Three improvement mechanisms are related to the subgrade: increase of subgrade soil resilient modulus, subgrade soil vertical restraint, and tensioned membrane effect. Contrary to the case of the two mechanisms related to the base, the three mechanisms related to the subgrade are not related. In the case of very small subgrade rutting (e.g., <13mm [0.5in.]) and a rather firm subgrade (CBR greater than about 3%), increase in resilient modulus of the subgrade is the dominant mechanism of subgrade improvement. If rutting increases, the bearing capacity increase that results from the vertical restraint mechanism becomes progressively effective; it becomes fully effective if rutting is of the order of 50 to 75mm (2 to 3in.). At deep rutting (more than 100mm [4in.]), the tensioned membrane effect becomes effective without eliminating the two preceding mechanisms.

Road improvement mechanism related to base-subgrade interaction

Intermixing of base and subgrade material

Intermixing of subgrade soil and granular particles from the base results from repeated loading. It manifests itself in two ways: downward movement of granular particles (loss of granular particles into the subgrade) and upward movement of fine particles from the subgrade soil (infiltration of fine subgrade soil particles into the base).

The loss of granular particles into the subgrade decreases the thickness of the base, which decreases its ability to distribute the traffic loads. The intrusion of fine subgrade soil particles into the base alters the mechanical properties of the base material, which makes the base more likely to deform and less able to distribute the traffic loads. Only a small amount of fine soil particles is sufficient to significantly alter the base mechanical properties.

Use of geosynthetics for separation of base and subgrade

Intermixing of two materials squeezed together by applied loads is prevented or delayed by a geosynthetic that performs the function of separation. This is an important function because intermixing is a major cause of distress of paved and unpaved roads. The need for separation depends on several parameters (e.g., subgrade properties, amount of moisture,
base material gradation, stress level at base/subgrade interface, construction method). Generally, but not always, separation is needed with soft subgrade.

Geotextiles are typically used to perform the separation function. Indeed, a geotextile with adequate puncture and tear strength prevents the loss of granular material into the subgrade and, with adequate opening size, prevents intrusion of fine particles from the subgrade soil into the base.

However, a geogrid can also provide some degree of separation through individual action of each aperture and global action resulting from its continuity:

- A geogrid with adequate aperture size prevents the loss of individual granular particles into the subgrade.
- A geogrid that keeps the base material together reduces the opportunities for intrusion of the base by fine particles from the subgrade soil (in particular if the base material has a proper gradation relative to the size of subgrade particles).

However, if fine particles from the subgrade soil intrude into the base, the effectiveness of the interlocking between geogrid and base material is likely to be reduced, which can be very detrimental to the performance of the road structure because interlocking is the main mechanism of improvement of a road structure by a geogrid, as discussed earlier in this article. Even though geogrids can provide some degree of separation, geogrids used in road structures are essentially expected to perform a mechanical function. In fact, in some cases of very soft subgrade and/or open graded base material, a geotextile and a geogrid are used together: the geotextile to provide separation and the geogrid (overlying the geotextile) to perform a mechanical function through the other mechanisms described in this article. Similarly, a geotextile is generally used between a geocell and the subgrade soil.

**Discussion of mechanisms**

The mechanisms described in this article are complex and some discussion is useful.

**Reinforcement and stabilization**

Should roads incorporating geosynthetics be called “geosynthetic-reinforced roads” or “geosynthetic-stabilized roads”? Good terminology can result only from analysis. Accordingly, the words reinforcement and stabilization were not used so far in this article. At this point, it is possible to have a rational approach to terminology based on the foregoing discussions.

The term reinforcement implies “adding force.” This function is obviously performed by a geosynthetic involved in the tensioned membrane effect. This mechanism is effective if the forces are large, which requires a high-strength geosynthetic, typically a high-strength woven geotextile. The large forces are typically associated with relatively large strains in the geotextile (e.g., 5% or more) as shown by Giroud and Noiray (1981). Such large strains are associated with deep ruts and generally happen with a soft subgrade soil (e.g., a CBR less than 1%), high wheel load, and/or a large number of vehicle passes.

In contrast, the geosynthetic strains associated with the mechanism of base lateral restraint are low, typically less than 1% (Giroud and Han 2006) or even 0.5%. Rather than using the term reinforcement for the function performed by the geosynthetic in this mechanism, the term stabilization is increasingly accepted. This terminology is appropriate because, according to dictionaries, “to stabilize” means “to keep unchanged,” (i.e., keeping in its initial stage). Indeed, lateral restraint is a mechanism aimed at keeping the road base as close as possible to its initial stage for as long as possible, which is consistent with low strains, hence small road deformation and limited rutting. Rather than stabilization, it is preferable to use mechanical stabilization to differentiate
from chemical stabilization (which refers to addition of chemical products to soil) and physical stabilization, which characterizes the separation function of geosynthetics.

Herein, the terminology mechanical stabilization (used above for base lateral restraint) is extended to the case of vertical restraint of subgrade because in this case the role of the geosynthetic also consists in restraining displacement. In the case of vertical restraint of subgrade, the geosynthetic strain may not be as low as in the case of lateral restraint of base. A geosynthetic strain of perhaps 1% or less for geogrids and up to 2% for geotextiles may be considered for vertical restraint, but no experimental data are available to support this estimate.

The mechanisms of unpaved road improvement, along with the associated subgrade conditions, rut depths, and geosynthetic strains are summarized in Figure 1. Inspection of Figure 1 leads to the following comments:

- In contrast with the stabilization mechanisms, the tensioned membrane effect is not a mechanism of base or subgrade improvement. It simply consists in adding a force that reduces the load. Therefore, it may be called "load reduction due to reinforcement".
- Subgrade improvement is used to encompass the improvement of the subgrade that results from base mechanical stabilization, subgrade mechanical stabilization, and even base physical stabilization (i.e., separation between base and subgrade).
- The end results are subgrade loading improvement and subgrade properties improvement. These are the two aspects of load support: decreasing or redistributing the load on the subgrade and increasing the ability of the subgrade to bear the load.
- The mechanisms that contribute to increasing the service life of the unpaved road (in green boxes in Figure 1) have a beneficial impact on both load reduction and subgrade improvement. Therefore, the mechanisms described in this article (and summarized in Figure 1) address the two aspects of the function of an unpaved road defined in the introduction: load support and service life.

- Subgrade stabilization, which is often used to encompass the action of geosynthetics in unpaved roads, appears to be restrictive because it does not include base mechanical and physical stabilization. Subgrade improvement appears to better encompass the beneficial effects of geosynthetics in unpaved roads.

As mentioned in the above discussions, the amount of road deformation required to activate a mechanism varies depending on the mechanism. This is discussed in more detail in the following section.

**Road deformation and mechanisms of road improvement**

There is no mechanical action, such as mechanical stabilization, without deformation. Therefore, there is always some deformation of the road structure associated with the performance improvement resulting from the use of geosynthetics.

However, except in rare cases where deep ruts are acceptable and the tensioned membrane effect is effective, it is beneficial to keep the road structure deformations as small as possible:

- Small deformations mean less rutting and, therefore, better trafficability.
- If a granular layer that is temporarily used as an unpaved road is eventually incorporated in the structure of a paved road, it is important to minimize deformations during the service life of the unpaved road to preserve the integrity of the base and the subgrade to ensure long-term performance of the paved road.

The requirement for small deformation is achieved by using geogrids or geocells because the displacement required to mobilize interlocking between a geogrid and granular material or confinement of base material in a geocell is small and less than the relative displacement required to mobilize interface friction between geotextile and granular material. Therefore, for the same loading, unpaved roads incorporating geogrids or geocells can be expected to deform less than unpaved roads incorporating geotextiles.

Relationships between mechanisms of unpaved road improvement and road structure deformation can be summarized as follows, based on foregoing discussions:

- Lateral restraint of granular material provided by geogrid (i.e., reduction of granular material lateral movement) is a mechanism that is effective as soon as the road structure exhibits little deformation (e.g., rutting much less than 25mm [1in.]), and consequently very small geosynthetic strain (e.g., less than 1% or even 0.5%). Lateral restraint of granular material and the resulting load distribution are still effective, but to a lesser degree, if rutting increases.
- Increase of the resilient modulus of the subgrade soil that results from load distribution is essentially effective if the road deformation is small (e.g., rutting less than 13mm [0.5in.]). If rutting increases, the benefit regarding resilient modulus of subgrade becomes less marked and the main beneficial effect on subgrade becomes the vertical restraint.
- Vertical restraint of subgrade (which increases subgrade bearing capacity) is a mechanism that requires some deformation of the subgrade (e.g., rutting of 25 to 75mm [1 to 3in.]), and consequently some geosynthetic strain (e.g., 1 to 2%).
- The tensioned membrane effect requires large deformation of the subgrade to allow the geosynthetic to take a deep concave shape. Therefore, deep rutting is required for the tensioned membrane effect to be effective (e.g., rutting greater than 100mm [4in.]).
UNPAVED ROADS

Unpaved roads incorporating geosynthetics

MECHANISMS OF UNPAVED ROAD IMPROVEMENT

Type of mechanism

Load reduction due to reinforcement

Tension in geosynthetic

Base physical stabilization

Separation provided by geosynthetic

Base mechanical stabilization

Interaction provided by geosynthetic/base material

Subgrade mechanical stabilization

Tension in geosynthetic

Deep rutting (>100mm= 4in.) generally due to very soft subgrade (CBR<1%)

Concave shape of deformed geosynthetic under wheel

Large required geosynthetic strain e.g. >5%

Upward resultant carrying part of the load

Tensioned membrane effect

Generally, but not always, soft subgrade

No required geosynthetic strain

Base/subgrade intermixing prevention

Reduces maximum stress on subgrade

Subgrade loading improvement

Effective with any subgrade and any rutting

Lateral restraint of base material

Base lateral spreading prevention

Long-term modulus retention

Long-term load distribution

Reduces deviator stress in subgrade soil

Increases resilient modulus of subgrade soil

Firm subgrade (CBR>3%) and very small subgrade rutting (<13mm= 0.5in.)

Wider load distribution

Vertical stress on subgrade away from the wheels

Base modulus enhancement

Subgrade properties improvement

Small required geosynthetic strain e.g. 1% to 2%

Soft subgrade (CBR<3%) and medium rutting (25 to 75mm= 1 to 3in.)

Downward resultant on subgrade between the wheels

Convex shape of deformed geosynthetic between wheels

Downward surcharge of subgrade away from and between the wheels, combined with vertical restraint of the subgrade

Increases subgrade bearing capacity

Increases resilient modulus of subgrade soil

Increases subgrade bearing capacity

Subgrade loading improvement

Reduces deviator stress in subgrade soil

Very small required geosynthetic strain e.g. <0.5%

Firm subgrade (CBR>3%) and very small subgrade rutting (<13mm= 0.5in.)

Upward resultant carrying part of the load

Tensioned membrane effect

Generally, but not always, soft subgrade

No required geosynthetic strain

Base/subgrade intermixing prevention

Reduces maximum stress on subgrade

Subgrade loading improvement

Effective with any subgrade and any rutting

Lateral restraint of base material

Base lateral spreading prevention

Long-term modulus retention

Long-term load distribution

Reduces deviator stress in subgrade soil

Increases resilient modulus of subgrade soil

Firm subgrade (CBR>3%) and very small subgrade rutting (<13mm= 0.5in.)

Wider load distribution

Vertical stress on subgrade away from the wheels

Base modulus enhancement

Subgrade properties improvement

Small required geosynthetic strain e.g. 1% to 2%

Soft subgrade (CBR<3%) and medium rutting (25 to 75mm= 1 to 3in.)

Downward resultant on subgrade between the wheels

Convex shape of deformed geosynthetic between wheels

Downward surcharge of subgrade away from and between the wheels, combined with vertical restraint of the subgrade

Increases subgrade bearing capacity

Increases resilient modulus of subgrade soil

Increases subgrade bearing capacity

Subgrade loading improvement

Reduces deviator stress in subgrade soil

Very small required geosynthetic strain e.g. <0.5%
It can be concluded that:
- In most unpaved roads, the tensioned membrane effect is not effective. The only exception is where subgrade strength is low (e.g., CBR < 1%) and deep rutting can be tolerated.
- In unpaved roads over firm subgrade (and in paved roads), where deformations are limited, only lateral restraint, improved load distribution, and increase in subgrade resilient modulus are effective.

**Geosynthetic strain level**
With the exception of the rare cases where the tensioned membrane effect is effective, the geosynthetic strain is very small compared to the strain that causes the rupture of the geosynthetic in a tensile test. Therefore, in most unpaved roads, the ultimate tensile strength of the geosynthetic is not a relevant property. The discussions presented in this article show that the relevant properties of the geosynthetic are the ability to interact with the base granular material and the load/strain response at low strain values.

**Conclusions**

**Conclusion on mechanisms**
As shown in this article, geosynthetics improve unpaved roads through several mechanisms that can be summarized as follows:
- The inclusion of an adequate geosynthetic provides lateral restraint to the base granular material, which minimizes lateral spreading of the base while the association of the geosynthetic and the granular material of the base creates a high-stiffness composite material, resulting in load distribution improvement. This mechanism takes place at low strain, hence low rutting.
- Geosynthetic-enhanced load distribution has two beneficial effects on the subgrade soil: (i) it increases the subgrade resilient modulus compared to the case without geosynthetic; and (ii) along with the downward stress applied by geosynthetic tension, it increases the bearing capacity of the subgrade by applying vertical restraint to the subgrade.
- In rare cases where deep ruts are acceptable, the geosynthetic under tension supports part of the wheel load because of the tensioned membrane effect.

In addition to the above mechanisms, which are purely mechanical, the mechanism of prevention of intermixing of base and subgrade provided by a geosynthetic performing the function of separation is a mechanism of physical improvement, which benefits the long-term performance of the base and the subgrade.

It was also shown in this article that the mechanisms described address the two aspects of the function of an unpaved road: load support and service life. Therefore, the use of adequate geosynthetics in unpaved roads results in comprehensive improvement.

**Conclusion on terminology**
As indicated above, the analyses presented in this article show that it is appropriate to designate unpaved roads incorporating geosynthetics as *mechanically stabilized unpaved roads* because both the base and the subgrade are mechanically stabilized. Strictly speaking, *mechanically stabilized* does not include unpaved roads where the tensioned membrane effect plays a role. However, this is generally not a terminology problem because this mechanism is rarely effective because it requires deep rutting, which is rarely acceptable.

Furthermore, it was shown in this article that *subgrade improvement* is an appropriate terminology while *subgrade stabilization* does not encompass all the benefits that result from using a geosynthetic in unpaved roads. But *base stabilization* is essentially the base that benefits from the stabilization of paved roads.

**Conclusion on performance evaluation**
The mechanisms that govern road performance are complex. It is legitimate to try to get quantitative data from laboratory and field tests, but test interpretation can be correct only if there is a good understanding of the mechanisms (the purpose of Part 1 of this series) and rigorous planning of the tests (which is the purpose of Part 2 in the April/May issue of *Geosynthetics*).

**References**


Mechanically stabilized turbine access road, St Fergus Moss peat fuel extraction area, Scotland. Photo: Tensar International
Bridge building in Maine

First GRS bridge in a marine setting
Four Mile Road subgrade stabilization project, Clearfield County, Pa. 
Photo: Tensar International Corp.
Part 2

Field evaluation of the performance of unpaved roads incorporating geosynthetics—Planning

By Jie Han and J.P. Giroud

Introduction
Scope and terminology

The mechanisms that govern unpaved road performance are complex. Therefore, it is legitimate to try to get quantitative data from laboratory and field tests, but test interpretation can be correct only if there is a good understanding of the mechanisms (which was the purpose of the Part 1 article), rigorous planning of the tests (which is the purpose of this Part 2 article), and appropriate implementation of the tests (which will be the purpose of the Part 3 article).

A field test may include one or more test sections. In accordance with the terminology used in the Part 1 article (Giroud and Han, 2016), unpaved road test sections incorporating geosynthetics can be referred to as *mechanically-stabilized test sections* while unpaved road test sections without geosynthetic can be referred to as *non-stabilized test sections*. More specifically, a mechanically-stabilized test section may be referred to as *geosynthetic-stabilized test section* to indicate that stabilization results from the use of a geosynthetic; furthermore, the type of geosynthetic may be indicated, as in *geogrid-stabilized test section*. Non-stabilized test sections are often used as control sections for benefit evaluation of mechanically-stabilized test sections.

Unpaved road performance

Unpaved roads include haul roads, working platforms, and aggregate-surfaced roads, on which fewer vehicles travel at slower speed than those on paved roads. AASHTO (1993) allows aggregate-surfaced roads to be designed for up to 100,000 equivalent single axle loads (ESALs). The most relevant deformation related to unpaved road performance is rutting. Rutting is permanent deformation that accumulates as the number of axle loads increases. Large rutting may cause discomfort to drivers, damage to vehicles, and instability of the vehicles; therefore, excessive rutting should be avoided.

In the literature, rut depth is defined in two ways: apparent rut depth and elevation rut depth (Cuelho et al., 2014). The apparent rut depth is defined as the maximum vertical...
distance between the peak and the valley of a wheel path cross section. The elevation rut depth is defined as the maximum vertical distance between the original elevation of the road surface and the valley of the wheel path cross section. When the subgrade soil is saturated or nearly saturated, it is incompressible or nearly incompressible; in this case, the subgrade soil moves down under the wheel and moves up around the wheel. As a result, the apparent rut depth is larger than the elevation rut depth. In field tests reported by Cuelho et al. (2014), the apparent rut depth was about 1.5 to 2.0 times the elevation rut depth. Most agencies or projects adopt the apparent rut depth as a way to quantify road deformation and they allow its magnitude to be up to 50-100mm (2-4in.) for unpaved roads. Giroud and Han (2004) used an apparent rut depth of 75mm (3in.) as the serviceability limit for their unpaved road design method. AASHTO (1993) limits apparent rut depths to a typical value of 25-50mm (1-2in.) for aggregate surfaced roads. Since the rut depth generally used is the apparent rut depth, it is recommended to use this parameter for the evaluation of trafficking tests.

**Mechanisms that improve unpaved road performance**

Giroud and Han (2016) indicate that the mechanisms through which geosynthetics improve the performance of unpaved roads include separation between base and subgrade, lateral restraint of the base material, vertical restraint of the subgrade soil, and tensioned membrane effect. Among the mechanisms other than separation, the dominant mechanisms improving unpaved road performance within tolerable deformations (i.e., apparent rut depth smaller than 100mm [4in.]) are lateral restraint of the base material and vertical restraint of the subgrade soil. The tensioned membrane effect becomes important only when large deformations occur in soft subgrade (i.e., subgrade deformation that results in apparent rut depth larger than 100mm [4in.]). Measured data reported by Cuelho et al. (2014) show that: (i) the geosynthetic at the edge of the wheel first moved outward due to lateral spreading of the base course and then inward due to the accumulated rutting; and (ii) this displacement transition happened at the elevation rut depth of 50mm (2in.). Cuelho et al. (2014) attributed this transition to the start of the tensioned membrane effect. Indeed, an elevation rut depth of 50mm (2in.) may be equivalent to an apparent rut depth of 75 to 100mm (3 to 4in.), at which the tensioned membrane effect starts to become important, as shown by Giroud and Noiray (1981). This example illustrates that using appropriate parameters (such as apparent rut depth) is necessary to make correct interpretation of the mechanism involved.

**Objectives of field evaluation**

Field evaluation of unpaved road performance may be conducted with different objectives: (1) quality assurance, (2) benefit evaluation, and (3) comparative study.

Field evaluation is often performed as part of road construction quality assurance. Such field evaluation, being done for actual projects, is often performed using non-destructive methods in a fast manner.

The evaluation of the benefit provided by mechanical stabilization of unpaved roads is often done by constructing mechanically-stabilized test sections and comparing their performance to that of a control section, which consists of a non-stabilized test section. For easy evaluation, mechanically-stabilized test sections and control sections should be constructed on the same subgrade soil at the same moisture content and state of compaction, with a base layer of the same thickness, grading and moisture content, and using the same construction method.

A comparative study may be used to evaluate the relative performance of geosynthetic-stabilized unpaved roads with different base thicknesses or different geo-
synthetic products. For a fair comparison, the parameter being evaluated should vary from one section to another, while other parameters should be kept constant.

Test sections for benefit evaluation and comparative study may be evaluated by non-destructive methods and/or destructive methods.

**Test methods**

**Test methods used to evaluate soil properties**

In addition to density tests (e.g., nuclear gauge test, sand cone test, etc.), vane shear and dynamic cone penetrometer tests can be used to evaluate soil properties. The vane shear test consists in applying a torque to a metal vane inserted in soil to generate shear failure of the soil. As a result, the undrained shear strength of the soil is estimated. The dynamic cone penetrometer (DCP) test uses a falling weight to apply an impact load that forces a steel rod with a cone tip to penetrate into the soil. The amount of penetration under each blow can be used to estimate strength and modulus of the soil. The vane shear test is mainly used to evaluate the subgrade soil, while the DCP test can be used to evaluate subgrade soil and base course material. The vane shear test and the DCP test evaluate soil properties at specific depth. They have been mostly used for site investigation, quality control before construction, and quality assurance after construction, but they cannot be used to evaluate the performance of a test section.

**Test methods used to evaluate road performance**

Deflectometers (falling weight deflectometer [FWD] or light weight deflectometer [LWD]) generate a load pulse by dropping a weight on a circular plate, which induces a deflection basin at the road surface. Based on the load pulse and the road surface deflection with known layer thicknesses, moduli of layers under the plate can be back-calculated. There are several types of FWD devices, which have falling weights ranging from 445 to 6675 N (100 to 1,500 lbs). The LWD is a portable falling weight deflectometer that has a typical falling weight of 100 N (22 lbs). Since the LWD test has a light falling weight, it is mainly used to evaluate subgrade and base; since the FWD test has a heavy falling weight, it can be used to evaluate subgrade, base, and asphalt layer. Both FWD and LWD tests are considered non-destructive tests because they induce small deformations. Research showed that the FWD and LWD tests are not effective in detecting the improved performance immediately after the construction of test sections incorporating geosynthetics because their induced deformations are too small to mobilize the contribution of geosynthetics. However, after test sections are trafficked by wheels, geosynthetics can minimize the deterioration of granular bases so that the modulus of the base is retained for a longer performance period. Then, the FWD test can detect the higher retained composite modulus of the test section with geosynthetic compared to the composite modulus of the test section without geosynthetic, as demonstrated by Jersey et al. (2012).

The plate loading test consists in applying a load on a loading plate seated on a road surface. The road surface deforms with an increase of load magnitude, with time under a constant load, and/or with the number of load repetitions. Static and repetitive plate loading tests are used, as discussed below.

In the case of the static plate loading test, the load is maintained after each load increment and the deformation increase with time is measured. The initial deformation within the elastic limit is used to calculate the composite elastic modulus of the test section while the additional deformation close to failure is used to estimate its ultimate bearing capacity. Static plate loading
tests can be used to evaluate the benefits of geosynthetics in stabilizing base courses over soft subgrade, which include increased section composite modulus and bearing capacity.

In the case of the repetitive plate loading test, the load on the loading plate is repeatedly increased and reduced, and the total deformation and the rebound (or "recovery deformation") are measured for each loading cycle. The difference between the total deformation and the total rebound is the permanent deformation, which is often related to the rut depth of a road. During this loading process, the load intensity may be increased. The cyclic plate loading test is a special repetitive plate loading test, where a cyclic load is automatically and continuously applied at a fixed frequency by an actuator or air cylinder. White (2015) conducted cyclic plate loading tests including a sensor kit to measure ground deflections at selected radial distances from the plate center. Repetitive plate loading tests can induce elastic rebound and permanent deformations; therefore, these tests are effective in evaluating the benefit of geosynthetics in stabilizing base courses over subgrade under repeated loading, which includes increased section composite resilient modulus (related to the rebound). These tests can be conducted to large deformations, even up to failure of test sections.

The trafficking test consists in repeatedly applying axle loads on a road surface via moving wheel(s) and measuring rut depths as a function of the number of vehicle or axle passes. This is typically achieved by driving a loaded truck on the road. To reduce the time needed for evaluation, an accelerated pavement test (APT) facility can be used to run loaded wheels on the road surface in the field or laboratory in an accelerated manner. The APT is more commonly done in a laboratory than in the field. The main advantage of an APT done in the laboratory is to have better control of moisture, temperature, and wind. APT sections in the laboratory closely simulate road sections in the field; therefore, they can be considered equivalent to field evaluation. The trafficking can be conducted with reciprocating wheel action or single direction wheel travel. This detail should be documented. The APT method can generate small to large deformations and even failure of a road.

The FWD and LWD tests are the fastest and least expensive among all the tests discussed above, while the trafficking test is the slowest and most expensive, and the plate loading test is in the middle between FWD/LWD and trafficking tests. All of these test methods have been successfully used to evaluate the performance of unpaved roads without geosynthetics. The effectiveness of these test methods to evaluate the performance of geosynthetic-stabilized unpaved roads will be discussed in a later section.

Selection of test methods for field evaluation

Depending on the objective of field evaluation, different test methods and their procedures may be adopted.

Quality assurance

For quality assurance, field tests are performed to evaluate whether a geosynthetic-stabilized unpaved road meets the design requirements. FWD, LWD, and static and repetitive plate loading tests may be performed. Static plate loading tests can assess the composite elastic modulus increase of a test section by geosynthetic while repetitive plate loading tests can evaluate the composite resilient modulus increase of a test section by geosynthetic.

Benefit evaluation

To verify the benefit of geosynthetic stabilization, a control section should
be constructed on the same subgrade soil with the same granular layer of the same thickness using the same construction method as for the geosynthetic-stabilized section. FWD and LWD tests are not able to evaluate the benefit of geosynthetic immediately after the construction of the road because, then, the geosynthetics are not mobilized; but FWD and LWD tests can detect the improved performance after the road has been trafficked for a certain time period because, then, geosynthetics are mobilized as a result of accumulated deformation of the road structure. In particular, FWD and LWD tests may be used to assess the benefit of geosynthetic on the retained composite modulus of the test section over time as shown by Jersey et al. (2012). Static and cyclic plate loading tests can be performed to evaluate the benefit of geosynthetic in increasing the composite elastic modulus and composite resilient modulus of the test section, respectively. The benefit of geosynthetic in increasing the road life can be evaluated by trafficking tests. If test sections allow for tests to be run to failure, the increased bearing capacity can be evaluated by static plate loading tests or the prolonged road life can be evaluated by trafficking tests to large rut depths.

Comparative study
Comparing the performance of two or more test sections with different geosynthetics or different base thicknesses in a comparative study is difficult. Factors related to geosynthetics include, for example, type of geosynthetic, type of polymer, type of manufacturing process, and geometry and mechanical properties of the geosynthetics. Even if a comparative study is limited to a certain type of geosynthetic, the number of parameters can be large. For example, for geogrids, the relevant properties include, but are not limited to, aperture stability modulus, junction strength, aperture shape, aperture size and aspect ratio, rib thickness and profile, and tensile stiffness. It is hard to identify which parameters have the most important effect on the performance of unpaved road sections stabilized with different geosynthetics. The field study conducted by Cuelho et al. (2014) confirmed such difficulties. A comparative study is more feasible and reliable if the number of variables is limited, such as a study conducted for different geosynthetic products made with the same polymer, the same manufacturing process, and even the same manufacturer. In this case, the different geosynthetics of the same group are described using the term grade, which depends on basic parameters such as thickness and stiffness. The geosynthetic with a large thickness and high stiffness is considered as a high-grade product. For example, Qian et al. (2013) used three punched-drawn triangular aperture polypropylene geogrids of different grades and equivalent aperture size in cyclic plate loading tests in a large box and clearly demonstrated the effect of the geogrid grade on the performance of geogrid-stabilized bases over soft subgrade.

When different types of geosynthetics are used for a comparative study, an effort should be made to investigate and quantify the mechanisms that govern performance to ensure better interpretation and possible generalization. To that end,
The typical number of vehicle passes used in field trafficking tests of unpaved roads is 1,000.

Instrumentation of the field test sections should be undertaken. For example, White et al. (2010) installed earth pressure cells vertically in the base and the subgrade to measure horizontal stresses and evaluate the lateral restraint mechanism for woven geotextile, biaxial geogrid, and triangular aperture geogrid. They found that the triangular aperture geogrid was most effective in increasing the horizontal stress in the base as well as reducing the horizontal stress in the subgrade. To evaluate the benefit of geosynthetics for the tensioned membrane mechanism, a large rut depth (e.g., apparent rut depth greater than 100mm [4in.]) must be allowed to develop. As a general rule, a comparative study should be conducted by varying one influence factor of interest and fixing other influence factors. For example, to investigate the effect of base thickness on performance of geosynthetic-stabilized unpaved roads, the base thickness should be varied for a specific subgrade condition with a specific geosynthetic product.

When a thick granular base without any geosynthetic is compared with a thin granular base with a geosynthetic in a comparative study for equivalent performance of two unpaved test sections, it involves two variables: base thickness and geosynthetic. The equivalent performance is contributed by the combined effect of these two variables. Different base thicknesses of two test sections with equivalent performance may be designed using available design methods and evaluated by plate loading tests and/or trafficking tests. To ensure true equivalency in the mechanisms, mechanical action on the subgrade should be the same in the two test sections. Instrumentation is then needed to check that the surface deformation and the vertical stress on top of the subgrade under the same loading condition are equivalent.

**Design of test sections**

**Selection of test sections**

The design of test sections depends on the objective of field evaluation:

- When testing for quality assurance purposes, test sections should be randomly selected along the road.
- To evaluate the benefit of a geosynthetic, at least two test sections should be designed with same base material and thickness on the same subgrade, which include one control section without any geosynthetic and another section with a geosynthetic.
- For a comparative study, the number of test sections to be designed depends on the number of geosynthetics, subgrade conditions, and/or base thicknesses to be evaluated (at least two geosynthetic products or two base thicknesses should be used).

The size and the number of test sections are an important consideration. They will be addressed in the Part 3 article.

**Design methods**

Design methods available in the literature may be used for the design of unpaved test sections, for example, those included in the FHWA "Geosynthetic Design and Construction Guidelines" (Holtz et al., 2008). The outcome of the design of test sections is the thickness of base course.

To achieve meaningful results, field tests should be designed in accordance with the mechanisms that govern the performance of unpaved roads (see the Part 1 article, Giroud and Han, 2016). Since the performance of unpaved roads with or without a geosynthetic depends on several influence factors, these influence factors should be considered during the design of test sections. These factors, which include performance criteria, loading, and parameters related to the materials used in the tested unpaved roads, are discussed below.
Performance criteria
Performance criteria include rut depth and number of vehicle passes. An apparent rut depth of 75mm (3in.) has been commonly used as a serviceability limit for design of unpaved roads, which may also be used for the design of test sections. For benefit evaluation, the number of vehicle passes should be limited by a tolerable apparent rut depth (typically smaller than 25mm [1in.]) if test sections will be used as a service road or be paved later, or limited by time and/or budget. For a comparative study, the number of vehicle passes should also be limited due to time and cost considerations. The typical number of vehicle passes used in field trafficking tests of unpaved roads is 1,000. If an accelerated pavement testing facility is used, a large number of axle passes may be adopted, typically 5,000 passes or more.

Loading and tire pressure
For most unpaved road applications, tire pressure ranges from 400 to 700 kPa (approximately 60 to 100 psi) and wheel load ranges from 20 to 90 kN (5 to 20 kips) for a single axle or 35 to 180 kN (8 to 40 kips) for a tandem axle. The most commonly used tire pressure and wheel load for trucks in the United States are 550 kPa and 40 kN (80 psi and 9 kips), respectively. High tire pressure necessitates a high-quality granular material for the base in an unpaved road but does not necessarily require a thick base.

Subgrade strength
Subgrade strength is a key parameter for the design of unpaved roads with or without geosynthetic. Subgrade shear strength is often quantified using undrained shear strength, which can be measured by the vane shear test in the field or unconfined compression test in the laboratory; also, it can be estimated using available correlation with the California Bearing Ratio (CBR). However, for a specific subgrade soil, it is preferable to develop a site-specific correlation. There is also a common correlation between CBR and DCP penetration index (e.g., Webster et al., 1994). If rainfall is expected during field tests, the soaked subgrade shear strength should be used for design.

A sensitive subgrade, the strength of which decreases after disturbance by trafficking, should be avoided because it will introduce complexities in interpretation of test results. When a sensitive subgrade cannot be avoided in test sections, a remolded subgrade strength should be used for design.

Variability in subgrade strength exists in the field. Examples of variability characterized by the coefficient of variation (COV) are as follows:

- White et al. (2005) reported that the COV values for DCP penetration indices of base and natural subgrade ranged from 14.3% to 47.0%.
- Phoon (2007) indicated that: (i) COVs for geotechnical properties ranging from 10% to 30% are considered low; and (ii) typical COVs for undrained shear strengths of clays obtained from unconfined undrained (UU) tests and vane shear tests are 10% to 30% and 10% to 40%, respectively.

Avoiding variability in subgrade strength is key to achieving meaningful results. The variability in subgrade strength should be checked across individual sections as well as across all sections. The number of tests thus required depends on the variability in subgrade strength and will be further discussed in the section “Representativeness of test sections” in the Part 3 article.

Since almost all the design methods for unpaved roads have been developed based on 50% reliability (i.e., average performance), it is appropriate to use average subgrade strength for the design of test sections. However, if there is variability, using averaging over the whole test area...
may not be representative and could lead to premature failure of some test sections. In this case, averaging over each individual test section should be used rather than averaging over the whole test area.

Base material properties
The base in an unpaved road is in direct contact with wheels; therefore, the base material should have sufficient strength, modulus, and abrasion resistance to withstand trafficking effects for the service life of the road. Granular material is generally used as base material. For the selection of the base granular material, the following can be considered: (i) rounded or subrounded particles are not suitable for a granular layer used as a base because granular layers constructed with such particles have low strength and modulus; and (ii) single-sized angular particles are difficult to compact and tend to break under wheel loading. As a result, the most suitable granular material is well-graded crushed aggregate.

The strength and modulus of well-graded aggregate are often quantified by CBR tests and/or DCP tests. It should be pointed out that the CBR value of a granular layer in the field is often lower than that determined by standard CBR tests in the laboratory because the granular layer in the field is less confined and more difficult to compact than in the laboratory, especially when the subgrade is soft.

Geosynthetics
Geosynthetics, commonly used to improve the performance of unpaved roads, are nonwoven geotextile, woven geotextile, geogrid, and geocell.

Nonwoven and woven geotextiles can serve a function of separation between granular base and subgrade soil and the key geotextile parameter is then the apparent opening size, which should be selected based on the gradation of the subgrade soil. Geosynthetics with high tensile strength and low interlock capacities (such as some woven geotextiles and some geogrids with apertures too small to interlock with aggregate) may serve as a tensioned membrane providing additional force to support wheel loads if a large rut depth (> 100mm [4in.]) is allowed.

A stiff geogrid, able to restrict lateral displacement of aggregate by interlock, can contribute to separation between well-graded aggregate and fine-grained subgrade by maintaining the integrity of the aggregate layer. However, if the aggregate is open graded, a nonwoven geotextile may be placed under the geogrid. Geogrid properties considered to be important for lateral restraint of the granular material are rib shape, rib thickness, aperture size, initial tensile modulus, in-plane flexural stiffness of the ribs, and junction efficiency (Webster, 1992; Giroud, 2009). In addition, aperture shape plays an important role in geogrid-particle interlocking. To ensure effective interlocking between geogrid and granular material, the particle size and gradation should be controlled and a geogrid with compatible aperture size should be selected. Holtz et al. (2008) suggested that the geogrid aperture size should be larger than the mean particle size and smaller than twice the particle size corresponding to 85% finer. Giroud and Han (2015) concluded that the optimum aperture size for geogrid interlocking with granular material is approximately twice the mean particle size.

Geocells can provide closed confinement to granular material and their effectiveness depends on geocell height, pocket diameter, welding strength, and degree of compaction of the granular material.

Recommendations and conclusion
Recommendations
The following recommendations can be made from the above discussions.

The objective of field evaluation should be clearly defined. The methods for evalua-
tection may be different for different objectives (i.e., quality assurance, benefit evaluation, comparative study).

Test sections for benefit evaluation and comparative study should be planned in a way that ensures they will be performed under well-controlled conditions. In particular, uniformity of subgrade is essential and should be required.

Appropriate design of the base and appropriate test methods are key to a successful field evaluation.

**Conclusion**

Field evaluation of unpaved roads incorporating geosynthetics can have different objectives: quality assurance, benefit evaluation, and comparative study. Design of test sections and selection of test methods depend on the objective of field evaluation.

Representative test sections should be carefully designed. Falling weight deflectometer (FWD), lightweight deflectometer (LWD), static, and repetitive plate loading tests may be considered for quality assurance and benefit evaluation. Trafficking tests can be planned for benefit evaluation and comparative study.

In conclusion, this article provides guidance for properly planning field tests for quality assurance, benefit evaluation, and comparative study. Proper planning of field tests requires a good understanding of the mechanisms that govern the performance of unpaved roads (which was the purpose of the Part 1 article published in the February/March issue of *Geosynthetics*), while adequate implementation of the field tests is essential (which will be addressed in the Part 3 article to be published in the June/July issue of *Geosynthetics*).

**REFERENCES**


Mechanically stabilized main access road over peat soils at Whitelee Wind Farm near Glasgow, Scotland. Photo: Tensar International
Unpaved roads incorporating geosynthetics

Part 3—Implementation

By Jie Han and J.P. Giroud
Construction of mechanically stabilized access roads built between the turbines at the Renaico Wind Farm in Chile. Photo: Tensar International Corp.
Field evaluation of the performance of unpaved roads incorporating geosynthetics—Implementation

By Jie Han and J.P. Giroud

Introduction
The mechanisms that govern unpaved road performance are complex, and accurate field evaluation of the performance of unpaved roads incorporating geosynthetics is not easy because it involves many influence factors. Interpretation of test data can be correct only if there is a good understanding of the mechanisms (which was the purpose of the Part 1 article), rigorous planning of the field tests (which was the purpose of the Part 2 article), and appropriate implementation of the field tests (which is the purpose of this Part 3 article).

This article will focus on construction of test sections, representativeness of test sections, implementation of field evaluation, and interpretation of test results.

Construction of test sections
A construction plan should be prepared and discussed with the geosynthetic supplier. For both subgrade and base preparation, accurate measurement of ground levels is necessary for calculating base thickness and ensuring consistency. Specific considerations are presented below.

Subgrade preparation
The construction of a test section should start with the removal of topsoil, which often contains vegetation and organic matters. The subgrade should be graded or compacted to a level surface without any apparent voids. As a general rule, preparation and subsequent trafficking of all test sections should be consistent. The following constraints apply to construction equipment:

- In the case of natural subgrade soil, efforts should be made to minimize construction equipment traffic directly on the subgrade. If extra space is available, excavation

AUTHORS’ NOTE
The use of geosynthetics in unpaved roads involves several mechanisms that govern the performance of these roads. The Part 1 and Part 2 articles published in the preceding issues provide a concise description of these mechanisms and guidance for planning field evaluation of the performance of unpaved roads incorporating geosynthetics, respectively. To achieve meaningful results, field tests should be not only designed but also implemented and interpreted in accordance with the mechanisms that govern the performance of unpaved roads. Improper field tests and data interpretation may result in inconclusive and misleading outcomes, which should be avoided.

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and grading should be conducted from the sides of the test section. Otherwise, only the minimum required construction traffic to prepare the subgrade should be allowed. No further construction traffic is then allowed.

- In the case of compacted subgrade soil, construction equipment is allowed directly on the subgrade during construction but is not allowed after subgrade preparation.

Even though natural subgrade presents the drawback of being heterogeneous, it is preferred to compacted subgrade, especially when large test sections are needed. This is because constructing a uniformly compacted subgrade in a large area is very difficult. However, when a subgrade soil of a specific strength or CBR is required for test sections, a compacted subgrade soil may be used, with the following precautions:

- The soil to be compacted should be well mixed with water (if needed), which requires sufficient time for moisture content to even out across the section. Then, the soil must be placed and compacted in a consistent way.
- If soil is placed inside a trench to prepare a compacted subgrade, measures may be taken to maintain moisture of the compacted subgrade if needed. If the test site is expected to be subjected to rainfall during the test, a properly designed drainage system with equally spaced drain outlets should be installed in each test section; otherwise, the compacted subgrade soil may be ponded with water, which reduces subgrade strength and modulus.

Geosynthetic placement

Generally, the recommendations of the relevant geosynthetic manufacturers should be followed. The geosynthetics should be placed directly and flatly on top of the subgrade. If the width of the test section is wider than the roll or panel width of geosynthetic, additional geosynthetic should be installed and connection between the two pieces should be done by overlap or jointing. The overlap or jointing recommendations of the relevant manufacturers should be sought and followed where possible. In addition, the following recommendations may be considered, depending on the type of geosynthetic:

If geotextiles are used, Holtz et al. (2008) suggest a minimum unsewn overlap of 0.3–0.45m (12–18in.) for subgrade CBR > 2%, 0.6–0.9m (24–36in.) for subgrade CBR = 1–2%, and 0.9m (36in.) for subgrade CBR = 0.5–1%. Alternatively, the geotextile can be sewn for subgrade CBR < 1%. Geotextiles that are expected to act as tensioned membrane should be sewn. All geotextile roll ends should be overlapped for 0.9m (36in.) or sewn.

- If geogrids are used, an overlap of 0.3–0.45m (12–18in.) is often used.
- If geocells are used, connection should be done using staples.

Base course construction

Construction of a base course should follow the following typical procedure:

- If multiple trucks are required to deliver granular material to a test site, it is important that all material is dumped and mixed together on site to prevent segregation and reduce variability between loads of material in terms of grading and moisture content.
- Granular material should be end-dumped and cascaded over the geosynthetic using construction equipment. The starting granular material thickness is a function of the subgrade strength. The preferred thickness of each lift after compaction is 0.15m (6in.) to 0.20m (8in.). When a base course is too thick (e.g., greater than 0.3m or 12in.) to be placed in a single lift, it should be divided into two or more lifts. When two or more lifts of granular material are placed, they should be placed in equal or similar lift thickness to avoid non-uniform base densities in depth.
A well-graded granular material with fine particles should be compacted at its optimum water content within ±2% tolerance.

- It is important to ensure that the construction traffic is equal over all sections. All fill should not be brought in from one end, resulting in greater construction traffic over the initial sections.
- Depending on the type and quality of granular material used, pneumatic compaction equipment may be needed in lieu of a steel drum. Low ground contact pressure equipment can be used to perform placement of aggregate. This equipment should make complete passes over all sections. All passes should be recorded to be reviewed in the event that there are unexplained differences in test results. Video records can be used to substantiate this information. It is preferable that local compaction practice be followed.
- After placement and compaction of all lifts, the final compacted base thickness should be equal to the design base thickness.

**Representativeness of test sections**

To evaluate the performance of unpaved roads incorporating geosynthetics, representative test sections should be designed and prepared. To prepare representative test sections, the following factors should be considered.

**Required number of test data**

For test results to be reliable, a required number of test data is needed statistically. The basic formula to estimate the minimum number of test data assuming a normal distribution is:

\[ N = \frac{z_R^2 \times COV \times e}{\epsilon^2} \]

where \( N \) = required number of test data; \( z_R \) = standard normal deviate, which depends on a confidence level; \( COV \) = coefficient of variation (%); and \( e \) = allowable margin of error (%). The standard normal deviates corresponding to 90% and 95% confidence levels are 1.645 and 1.960, respectively.

At different COV (from 10% to 30%), allowable margin of error (5% or 10%), and confidence level (90% or 95%), the required number of test data can be determined as provided in Table 1.

If a test site can be constructed with a subgrade strength COV lower than 20% as an example, 11 to 16 subgrade strength tests are needed if the allowable margin of error is 10%, as shown in Table 1.

**Uniformity of subgrade and base**

The representativeness of test sections is significantly affected by variability of subgrade and base. Indeed, Han and Giroud (2012): (i) demonstrated that variation of subgrade soil at a low CBR value (e.g., CBR < 1%) has more effect on the performance of unpaved roads than that at a high CBR value (e.g., CBR > 1%); and (ii) calculated in a specific case that a 10% base thickness increase resulted in 100% to 330% increase of the service life of the considered unpaved road.

To ensure the validity of test sections, subgrade and base should be uniform. To ensure uniformity of subgrade and base, density tests should be performed during and after construction of test sections and typical density requirements should be followed. In addition, the COV values for subgrade strength and base modulus for all test sections should be lower than 20% and the allowable margin of error for the average value of each parameter with respect to its target value should be less than 10%. The COV values for individual test sections should be lower than 10%. The deviation of the compacted base thickness from the target thickness should be smaller than 13mm (0.5in.). If the COV values for the test sections are higher than 20% or the COV values for individual test sections are higher than 10%, closer examination is needed and the number of tests at least needs to be doubled. To minimize moisture variation...
due to rainfall, a drainage system should be properly designed and installed in test sections.

**Size of test section**
Test sections should be large enough to be representative. There is a general rule in geotechnical research that the size of the test section should be at least five times the loading plate size to avoid a boundary effect. The influence depth of a circular or square loading plate is two times the plate size. In the case of a loading plate size of 0.3m (1ft), these rules lead to the following:
- The size of the test section should be at least 1.5m × 1.5m (5ft × 5ft).
- For a circular or square loading plate, the depth of the test section should be at least two times the plate size (i.e., 0.6m [2 ft]).

Two- or three-axle dump trucks have been mostly used for trafficking tests in the field. Rear axles carry most of the truck load. An accelerated pavement testing (APT) facility has also been used for the same purpose, which may employ a full axle or a half axle. Commonly used wheel configurations include single wheel, dual wheels, and tandem wheels. To minimize possible boundary effects, a test section for the trafficking test by a full-size truck in the field or a full axle in an APT facility should be at least 6.0m (20ft) long, 4.5m (15ft) wide, and 1.2m (4ft) deep (including the base and the subgrade). A test section loaded by a half axle in an APT facility should be at least half of the plan dimension (i.e., 3m [10ft] long and 2.3m [7.5ft] wide) as that tested by a full-size truck or a full axle.

**Implementation of field evaluation**

**Instrumentation**

To investigate load transfer mechanisms and stress distribution, earth pressure cells and strain gauges may be used. Also, when a test section consists of sensitive soil, piezometers may be used to monitor accumulation of excess pore water pressure in the soil. All these sensors should be prepared and calibrated in the laboratory before they are installed in the field. The following comments are related to the use of earth pressure cells:
- Earth pressure cells placed horizontally measure vertical stresses. They are often used on top of the subgrade, i.e., at the interface between base course and subgrade if there is no geosynthetic or beneath the geosynthetic if any.
- Earth pressure cells placed vertically measure horizontal stresses. They are located at distances from the center of a loading area (in a radial direction if a circular plate is used or in the transverse direction with respect to traffic). They can be used in the base course (to evaluate the lateral restraint effect) and/or in the subgrade.
- An earth pressure cell placed close to crushed aggregate should be protected by sand. A large-size earth pressure cell may hamper the interaction between geogrid-stabilized base and subgrade. Therefore, a cell size of 50 to 75mm (2 to 3in.) is preferred.
- When a static plate loading test is performed on top of the base course, at least one earth pressure cell should be placed horizontally within 50mm (2in.) of the top of the subgrade under the center of the loading plate to measure vertical stress.
- During a trafficking test, earth pressure cells for the measurement of vertical stress should be placed horizontally within 50mm (2in.) of the top of the subgrade along wheel paths.
- Under static loading, vibrating wire type or resistance type earth pressure cell can be used while, under cyclic loading, only resistance type of earth pressure cell should be used because vibrating wire type of earth pressure cell cannot measure a dynamic stress.

Strain gauges may be placed on a geosynthetic to monitor the development of tension

A simplistic interpretation of field tests consisting in comparing only the overall performance of test sections and using only one criterion, e.g., the total number of vehicle passes, may be misleading.
in the geosynthetic. A strain gauge on a geosynthetic measures only the strain at its specific location and direction of installation; this strain is often referred to as a local strain. For geogrids, the strains at different locations along the rib under tension are not necessarily consistent due to variation in rib section along the rib. Therefore, the local strain on one location of the geogrid may not represent the overall strain, which is often referred to as the global strain. A relationship between local strain and global strain should be established through laboratory tensile tests of a geogrid, on which strain gauge measurement on the rib and external displacement measurement across an aperture are taken. As loading is not transferred to the geogrid unidirectionally, consideration must be given to the geometry of the geogrid and the rib orientation when measuring strains.

Trafficing implementation
Since each full axle of a truck has wheels at both ends, it is important to maintain equal wheel load on both ends of the axle. Uneven distribution of wheel loads will result in uneven development of rutting, which may cause a truck to tilt toward one side. Tilting of a truck switches more load to one side: this accelerates the development of rutting on one side of the test section, which may result in premature failure on this side. Therefore, uneven load distribution on an axle should be avoided in trafficking tests. Visual observation is an important component to recognize the development of uneven rutting. Any localized area with an excessive elevation rut should be marked prior to continuing further trafficking. If the same problem continues to occur at the same location, this location should be carefully examined in the forensic investigation.

To minimize the test variables, an accelerated pavement test (APT) approach is preferable to a driven truck. A moving wheel or a truck should be driven at a constant speed (typically 8km/h [5mph]) on all test sections. Turning and wandering should be avoided unless they are considered in design and should be reported. Climatic effects are minimized if the test area is indoor or covered (i.e., sheltered from rain, sun, wind, etc.). For outdoor testing, trafficking tests should be carefully planned and completed within a limited time frame or under a favorable climate condition to avoid any possible effect by rainfall, freeze-thaw, drying, or strong wind. If the test site experiences heavy rainfall, trafficking test should be paused and the test sections should be re-evaluated by dynamic cone penetration tests and/or lightweight deflectometer tests before trafficking test can be resumed.

Rut depth measurement
Both apparent and elevation rut depths should be measured. It is essential that the position of measurements remain consistent throughout the test. Permanent markers should be installed adjacent to the test sections at each measurement location to aid this measurement. For apparent rut depths, a straight edge or a laser rut measurement device may be used. For elevation rut depths, surveying technology may be used. The number of rut depth measurements depends on wheel configuration and variability of rut depths. The common wheel configurations are single, dual, or tandem wheels. Each wheel induces one wheel path during trafficking. Rut depths should be measured on each wheel path. Table 1 can be used to estimate the minimum number of rut depth measurements in each test section.

Repair of test section
Due to the variability of loading and the variability of mechanical properties of subgrade and base, rut depths on different wheel paths in a single section are likely to increase at different rates. As a result, one wheel path will have a deeper rut than the other. When a rut gets too deep, it may cause unbalanced loads, instability of a test truck, and disturbance of ruts by the base of the truck. The affected test section has, then, to be repaired if other sections are to be subjected to further trafficking. The rut depth set for repair of a wheel path should be larger than the pre-defined failure criterion so that the second wheel path or the average rut depth from two wheel paths is likely larger than the pre-defined failure criterion. It is recommended that the maximum rut depth in each test section set for repair be 1.5 times larger than the failure criterion. For example, if the failure criterion for rut depth is 75mm (3in.), it is reasonable to test one wheel path up to a maximum rut depth of 115 mm (4.5in.). After the repair of one or both paths of a test section, the rate of rutting will be changed because the thickness of the base course in the repaired path has been increased. Because of this fact, the measured rut depths in the repaired section should not be used to calculate an average rut depth or compared with those in other sections without any repair.

Forensic investigation
After a trafficking test, all test sections should be carefully excavated by trenching to examine changes of dimensions and properties of the materials (including subgrade, base course, and geosynthetic) and possible failures occurring in these materials. At least two transverse trenches per section are recommended and additional trenches will be needed if surface deformations after a test are not uniform. Dynamic cone penetration testing should be performed on the base and subgrade and vane shear testing should be performed on the subgrade across these trench areas, in particular where there is excessive rutting. The test data from all these locations should be included in the report.

Base thickness reduction may be an indication of compression, shear failure, or lateral spreading. The profiles revealed by the excavation should be measured and reported. Photographic record of the exposed vertical sides of trenches may
be used to show rutting profiles at each level. It may be necessary to mark the geosynthetic position with colored pins. The change of top elevations of subgrade may be an indication of subgrade deformation (such as heave) or shear failure. Intermixing between base course and subgrade in non-stabilized and mechanically-stabilized test sections should be reported.

Great care is needed to exhume geosynthetics for installation damage reporting, particularly if some physical or mechanical testing will be conducted on the exhumed geosynthetics. Examination of geosynthetic damage may include junction and rib breakage of geogrid, rupture of geotextile, and breakage of geocell internal welding. Investigations should examine any evidence of geosynthetic pullout from edges of the test section and distortion along the wheel path (especially in the case of geotextiles). The horizontal distortion of a geosynthetic may be caused by low friction between geosynthetic and subgrade and/or by low in-plane stiffness of the geosynthetic.

Interpretation of test results

Data reduction and analysis
It is a good practice to reduce test data immediately after measurements are taken and plot them with previously obtained data (e.g., rut depth vs. time or number of vehicle passes) so possible errors or improper test procedures can be detected and corrected. Sudden increase or decrease of measured values should be carefully examined and verified. This is often an indication of a recording error or malfunction of a sensor. Data analysis depends on the type of measurements used in test sections, such as surface deformation or rut depth, strain in geosynthetic, vertical stress on top of the subgrade, and horizontal earth pressures in base and subgrade. These measured values may be used to calculate the moduli of base course and subgrade, coefficient of lateral earth pressure, and stress distribution angle. Because almost all design methods for unpaved roads have been developed based on 50% reliability (i.e., average performance), it is appropriate to use average values for assessment of test sections (see Han and Giroud, 2016). These measured or calculated values are often plotted against the number of axle passes or applied pressure on a loading plate. From these plots, the benefits of geosynthetics may be identified, such as the increased base modulus, the reduced rut depth, and the prolonged service life. For example, White (2015) conducted cyclic plate loading tests on field test sections with different geosynthetics, from which resilient moduli of the subgrade and the base, with or without a geosynthetic, were calculated.

The Traffic Benefit Ratio (TBR), defined as the ratio of the number of axle passes of the test section with a geosynthetic to that of a test section without any geosynthetic of the same base thickness and at the same rut depth, has been commonly used to quantify the beneficial effect of geosynthetic on performance. The TBR depends on rut depth. However, when the TBR is used to compare the performance of different geosynthetic products, it may be misleading because the performance of each test section with a geosynthetic depends on test conditions. The TBR obtained will be valid only for the exact conditions and layer thicknesses used for the test. For a fair comparative study of different geosynthetic products, it is recommended to investigate and quantify through instrumentation the mechanisms that govern performance.

Regression analysis
Linear regression analysis has commonly been used to establish a relationship (i.e., statistical model) between two parameters. The coefficient of determination (often denoted as R²) indicates how well data fit a statistical model mathematically. This type of analysis has been performed by researchers to examine the effect of
a certain factor on the performance of a road. Such an analysis is meaningful only if the considered factor is related to a mechanism that contributes to road performance. A statistical model that is not related to any mechanism does not have any physical meaning and may lead to a false conclusion. For example, at small rut depths of unpaved roads, ultimate tensile strengths of geosynthetics are far from being mobilized; therefore, ultimate tensile strengths of geosynthetics are not related to small rut depths. However, a statistical model between two unrelated data set (e.g., small rut depths and ultimate tensile strengths of geosynthetic) may still be established with a high R² value as long as their values have similar trends among all the geosynthetic products. Clearly, this statistical model does not have any physical meaning; therefore, a conclusion drawn from this relationship is false. Multiple regression analysis may be more appropriate to establish the relationship between the road performance and multiple influence factors including properties of geosynthetic, subgrade, and base course. Unfortunately, multiple regression analysis requires extensive test data with a large number of variables, which makes this type of analysis impractical in most cases. In addition, different mechanisms contribute to the overall performance. The feasible and correct approach is to establish a relationship between data sets when they are mechanically related.

**Recommendations and conclusion**

**Recommendations**

The following recommendations can be made from the above discussions:

1. Uniformity of subgrade, appropriate design and construction of base, and appropriate test methods are key to a successful field evaluation.
2. The sizes and number of test sections should be properly designed to achieve the goal of field evaluation and avoid boundary effects.
3. Representative test sections should be carefully designed and constructed to limit the coefficient of variation of the subgrade strength and the base modulus to less than 20% for all test sections or 10% for individual test sections, with a margin of error less than 10%. Also, the deviation of the compacted base thickness from the target thickness should be less than 13mm (0.5in.); and minimum test section dimension (length, width, and depth) requirements should be met.
4. Field cyclic plate loading testing is a promising test method, which can be performed to evaluate the section composite modulus increase associated with the inclusion of the geosynthetic.
5. Trafficking tests can be performed to evaluate to which extent the service life of a test section is prolonged by the geosynthetic. Accelerated pavement test approach and indoor trafficking test are the preferred methods.
6. Instrumentation should be used to investigate the mechanisms of load transfer and stress distribution that govern the performance of unpaved roads incorporating geosynthetics.

**Conclusion**

A comparative study involving different geosynthetics requires careful interpretation because the mechanisms through which geosynthetics improve the performance of unpaved roads are complex, as discussed in detail in the Part 1 article (Giroud and Han, 2016). As a result of this complexity, the performance of unpaved roads incorporating geosynthetics depends on multiple factors. Therefore, a simplistic interpretation of field tests consisting in comparing only the overall performance of test sections and using only one criterion, e.g., the total number of vehicle passes, may be misleading. In other words, overall performance can be used to compare a test section with another test.
section, but overall performance alone is not sufficient to compare the effectiveness of two different geosynthetics in actual unpaved roads. An objective comparison of the contributions of two different geosynthetics to unpaved road improvement can result only from a comprehensive interpretation involving overall performance evaluation as well as detailed instrumentation aimed at evaluating the mechanisms through which the geosynthetic incorporated in the road structure improves the performance. This necessary condition makes it possible to generalize from field tests to actual situations because a meaningful regression analysis can be performed only for data sets that are mechanically related.

In conclusion, this article provides guidance for properly conducting field tests for quality assurance, benefit evaluation, and comparative studies. In comparative studies, the test sections should be instrumented to evaluate the mechanisms governing performance improvement by the geosynthetics used in the field tests. This is necessary to ensure that the results of field tests can be generalized.

Closure

This article concludes a series of three articles that provide guidance on understanding the mechanisms that govern the performance of geosynthetic-stabilized unpaved roads, planning field tests to evaluate the performance of these roads, and implementing the field tests.

REFERENCES


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