REPORT OF REVIEW TENSAR MESA MECHANICALLY STABILIZED EARTH (MSE) WITH CONCRETE MODULAR BLOCKS PAIRED WITH EXTENSIBLE SOIL REINFORCEMENTS

October 2026

HIGHWAY INNOVATIONS, DEVELOPMENTS, ENHANCEMENTS AND ADVANCEMENTS (IDEA)

The Tensar MESA MSE Retaining Wall System has been evaluated in accordance with the IDEA protocol. Key information regarding this system is presented in this section of this final report of review. Important details of the system's components, design, construction and quality control measures are presented in the attached final submittal. Design parameters defined within the submittal are summarized in the table located at the end of this IDEA report.

Applicant Information

Tensar, A Division of CMC
Attn: Willie Liew, P.E.
2500 Northwinds Pkwy, Suite 500
Alpharetta, GA 30009
Ph: 404.219.0431
www.tensarcorp.com

Review Summary

Following its initial review of the Tensar MESA Retaining Wall System submittal, the review team provided the applicant with a series of comments, and requests for clarification and additional analysis. The applicant has been thorough in its responses and the review team finds that there are no outstanding issues that should be brought to the attention of the transportation agencies. Rather, the agencies are encouraged to rely upon the Tensar MESA Retaining Wall System submittal for projects where the Tensar MESA System is proposed.

Submittal Checklist

The checklist used from the IDEA protocol for this evaluation is C1 – Initial Technical Evaluation Checklist for Concrete Modular Block Unit Paired with Extensible Reinforcement. This is the second evaluation of the Tensar MESA System the first was performed by the predecessor to the IDEA program, the Highway Innovative Technology Evaluation Center (HITEC) in 2000.

Confidential Information

The applicant has the option to omit information from the version of its submittal that is attached to the final report if it believes that such information is confidential. In such instances, the applicant will notify the review team. However, for the Tensar MESA System no information has been designated by the applicant as confidential.

System Description

Components

The major components of the MESA Retaining Wall System are concrete modular blocks, extensible soil reinforcements (geogrids), and select granular backfill. Other standard components include a concrete leveling pad, MESA connectors and a geotextile filter (If applicable) between modular blocks and reinforced fill. The MESA modular block is a dry-cast unit that is 18 inches wide, 11 inches deep (nominal), 8 inches high and weighs 75 pounds. Wall batter for the MESA Standard units can be set at 4.5 degrees or near vertical (0.45 degrees). The soil reinforcing consists of Tensar uniaxial geogrid (i.e., UX1100MSE, UX1400MSE, UX1500MSE, UX1600MSE, UX1700MSE) connected to the facing. The HDPE-fiberglass inclusion polymeric MESA® connectors are inserted through the apertures of

the geogrid, into prefabricated openings in the top of the facing units, connecting the Tensar geogrid to the MESA modular blocks.

System History

In 1994 the Mesa Retaining Wall was conceived as a system which would be designed to comply with the requirements for MSE walls as set forth in the AASHTO guidelines, Task Force 27 being the original AASHTO document. The unique structure of Tensar's geogrid permitted Tensar to address one of the key elements in the AASHTO guidelines, connection strength, in particular, mechanical connection strength. Testing of the connection was then performed to determine the allowable connection strength for each one of Tensar's geogrid types. Test walls were constructed in the Atlanta area to prove the viability of the connection and develop an installation procedure for the system.

The Highway Innovative Technology Evaluation Center, (HITEC), of the Civil Engineering Research Foundation (CERF), published an Evaluation of the Tensar MESA® Retaining Wall System, dated April 2000 with their CERF Report #40358, refer to Appendix – 0.3. The current MESA® system submitted herein differs from the system evaluated by HITEC in 2000 in the way that the facing elements are mechanically connected to the soil reinforcing geogrids. In the HITEC evaluated system, as is the case with present refinements made to the MESA® system, the geogrid layers were connected to the modular block facing units through the use of polymeric mechanical connectors.

System Properties

The following properties are reported by the applicant for the ARES Reinforcement Retaining Wall System.

Soil Reinforcement Ultimate (Nominal) Tensile Strength and Reduction Factors

The soil reinforcing used in the MESA® Retaining Wall System are uniaxial extensible highdensity polyethylene (HDPE) geogrids manufactured by Tensar. The geogrid used in the MESA® system are: UX1100MSE, UX1400MSE, UX1500MSE, UX1600MSE, AND UX1700MSE.

Specifications for the Tensar geogrid reinforcement are as provided in Submittal Appendix Tab 1.2.3. Laboratory test results for the Tensar geogrids are provided in the National Transportation Product Evaluation Program (NTPEP) report REGEO-2012-01-001- [Final Product Qualification Report for Tensar UX-MSE/UX-HS Geogrid Product Line], May 2014 is provided in Submittal Appendix 1.2.4.

Creep Reduction Factors: The creep reduction factor (RF_{CR}) for a 75-year design life varies from 2.54 (UX1400, UX1500MSE, UX1600MSE, AND UX1700MSE) to 2.68 (UX1100MSE), for a 100-year design life the creep reduction factor varies from 2.57 (UX1500MSE, UX1600MSE, AND UX1700MSE) to 2.73 (UX1100MSE, UX1400MSE), as defined in the NTPEP (2014) report (see Submittal 1.2.4 and Appendix Tab 1.2.4).

Installation Damage Reduction Factors: The full-scale installation damage test results, from the 2014 NTPEP report, are presented in Submittal Table 1-3. Refer to the NTPEP (2014) report (Appendix Tab 1.2.4) for the installation damage testing protocol and specific gradations of the backfill described in Table 1-3. The fill thickness over the geogrid reinforcement in the installation damage testing was 6 inches and the compaction was performed with a 25,000 lb single drum vibratory roller.

Durability Reduction Factors: As tested and reported in NTPEP (2014) all grades of the Tensar product line met the minimum UV requirement and Thermo-Oxidative resistance requirements. In general, default reduction factor for durability, RF_D, per AASHTO, Load Resistance Factor Design (LRFD) Bridge Design Specifications value of 1.3 for RF_D is typically selected for polyolefins. For HDPE, based on very long-term observations, default reduction factors as low as 1.1 have been used successfully by state departments of transportation. The very high percent strength retained in both the UV tests (98%) and the oven aging tests (98%) summarized in Table 6-2 of the NTPEP (2014) report indicates very good resistance to oxidation, the key issue for durability of HDPE geosynthetic reinforcement. Therefore, use of a default reduction factor of 1.1 is justified.

Soil Reinforcement-Modular Block Connection Capacity

The current MESA System connection utilizes the MESA connector to mechanically connect the Tensar geogrid to the MESA modular block facing units. The connection strength was evaluated by performing long-term connection tests at both 20° and 40° C.

Soil Reinforcement Design Tensile Strength

The nominal tensile strengths (T_{al}) for the Tensar UX geogrid soil reinforcements are computed as the ultimate strength ($T_{ult\text{-}MARV}$) divided by reduction factors for creep (RF_{CR}), degradation (RF_D), and installation damage (RF_{ID}). The equation for this calculation is:

$$T_{al} = \frac{T_{ult-MARV}}{RF_{CR} x RF_D x RF_{ID}} \qquad \text{Eq. 1}$$

The AASHTO NTPEP independently measured creep reduction values (NTPEP, 2014) vary from 1.54 to 1.73 depending on reinforcement grade and design life. The durability reduction factor is a function of wall fill specifications, particularly pH limits. A durability reduction value of 1.1 or 1.3 is typical. The installation damage reduction factor is a function of the wall fill properties (gradation, D_{50}, angularity, etc.) and placement techniques. Recommended values or value ranges are presented for two wall fills with maximum gradation sizes 3/8- and 3/4-inch. Ranges and average values of installation damage factor for larger fills, 2- and 1-inch maximum sizes, from the NTPEP testing (NTPEP, 2014) are presented for information. A maximum fill particle size of ¾-inch is recommended unless construction damage assessment tests are performed with the specific granular fill or have been performed with similarly graded large-size granular fill; and similar lift heights and compaction techniques/equipment.

Pullout Design Parameters

Independent pullout tests were performed by SGI Testing Services, LLC, in Norcross, GA, with <10mm Coarse Sand and Uniform graded crushed rock (No. 57 stone). The test report is included in Appendix – 1.2.7.

As shown in the test results, coefficient of pullout interaction factor C_i varies with overburden pressure and in general decrease with increase of overburden pressure. The reinforcement pullout is most critical at the top of wall typically within the 6ft below the top of wall. Because design calculations/software utilize a constant C_i, Tensar has selected a single value for each geogrid and backfill combination. Based on the test results, Tensar recommends C_i factor of 0.8 for crushed stone and coarse sand meeting AASHTO backfill specifications.

Retaining Wall Design

The design methodology of MESA® MSE wall considers both the external and internal stability of the structure. Internal stability is determined using the Simplified Method (SM). The SM is provided in "Design and Construction of Mechanically Stabilized Earth (MSE) Walls" Report No. FHWA-HIF-24-002, Federal Highway Administration.

The design examples in this submittal were performed using MSEW+.

Agencies may check designs with MSEW+ with the design properties listed in the table attached to this review report.

Reviewer Comments

Following its initial review of the MESA MSE with Concrete Modular Block Facing Units and Extensible Reinforcement Retaining Wall System submittal, the review team provided the applicant with 25 comments and requests for clarification. The applicant has been thorough in its responses, and the review team finds that there are no outstanding issues that should be brought to the attention of the transportation agencies. Rather, the agencies are encouraged to rely upon the final MESA® MSE with Concrete Modular Block Facing Units and Extensible Reinforcement Retaining Wall System for projects where the MESA System is proposed.

Closing

An update technical evaluation should be performed for the MESA MSE with Concrete Modular Block Facing Units and Extensible Reinforcement Retaining Wall System in five years (i.e., October 2031) or upon notice that a significant modification of the system has been made. For details regarding update technical evaluations and other guidance for the use of technical evaluations by transportation agencies, go to https://www.geoinstitute.org/special-projects/idea.

Summary Table MSEW Program Input Parameters for Tensar MESA Retaining Wall System

Geogrid Soil Reinforcement - Data/Geogrid		UX1100	UX1400	UX15000	UX16000	UX17000
$T_{ULT\text{-}MARV}$ (lb/ft)		3970	4,800	7810	9870	11990
Durability reduction factor, RF_D		1.1	1.1	1.1	1.1	1.1
Installation damage reduction factor, RF_{ID}[a]	100% < 3/4-in.; D_{50} = 0.21-in.	1.31	1.35	1.34	1.35	1.39
	100% < 3/8-in.; D_{50} < #16	1.24	1.15	1.14	1.15	1.18
Creep reduction factor, RF_{CR}	75-year	2.68	2.68	2.54	2.54	2.54
	100-year	2.73	2.73	2.57	2.57	2.57
Coverage Ratio		1.0	1.0	1.0	1.0	1.0
Friction coefficient along geogrid-soil interface, □	Fine uniform sands[b]	0.8	0.8	0.8	0.8	0.8
	Well graded sands[b]; sand and gravel[b]	0.8	0.8	0.8	0.8	0.8
Pullout resistance factor, F*	Sands[b]	0.8	0.8	0.8	0.8	0.8
	Gravels[b]	0.8	0.8	0.8	0.8	0.8
Connection Strength:		$CR_u = T_{ult\text{-}connection} / T_{ult\text{-}geogrid}$				
	$CR_u = T_{ult\text{-}connection} / T_{ult\text{-}geogrid}$	1.0	1.0	1.0	1.0	1.0
	$RF_{CR\ Connection}$ 75 yrs	2.96	2.96	2.96	3.33	3.33
	$RF_{CR\ Connection}$ 100 yrs	2.98	2.98	2.98	3.35	3.35
Connection strength durability reduction factor, $RF_{d\text{-}connection}$		1.1	1.1	1.1	1.1	1.1

Notes: [a] Installation damage factor based on 6" lift thickness and 25,000 lbs vibratory. Project-specific (or representative) installation damage testing and RF_{ID} quantification should be performed for MSE fills with a maximum aggregate size greater than ¾ in. (19 mm); [b] Predominant material.

Tensar MESA® System

Initial Technical Evaluation Checklist for Concrete Modular Block Unit Paired with Extensible Reinforcement

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Introduction

- Provide a **succinct description of the system** (i.e., facing, reinforcement, and connection type) that is being submitted for review.

Tensar Response:

The MESA® Retaining Wall (MESA) is a trademarked system developed and supplied by Tensar International Corporation (TIC). The MESA system is a mechanically stabilized earth (MSE) retaining wall system that features segmental concrete facing elements mechanically connected to high density polyethylene (HDPE) Tensar uniaxial soil reinforcement geogrids. The standard concrete segmental block facing elements are typically dimensioned at 8" tall x 18" wide x 12" nominal depth (front to back). Tensar polymeric soil reinforcing elements are manufactured of high-density polyethylene (HDPE) punched and drawn geogrids specifically developed for this type of vertical grade separation engineering application. The geogrids are placed in horizontal layers within the compacted backfill where they provide tensile reinforcement that mechanically stabilizes the earthen backfill, at prescribed levels within the segmental block facing units. The geogrids are mechanically connected to the facing units at predetermined (engineered) block courses using HDPE-fiberglass inclusion polymeric MESA® connectors that are inserted through the apertures of the geogrid, into fabricated openings in the top of the facing units. This integration ensures the creation of a highly stable and resilient MSE retaining wall, capable of withstanding significant loads and environmental stresses.

In 1994 the Mesa Retaining Wall was conceived as a system which would be designed to comply with the requirements for MSE walls as set forth in the AASHTO guidelines, Task Force 27 being the original AASHTO document. The unique structure of Tensar's geogrid permitted us to address one of the key elements in the AASHTO guidelines, connection strength, in particular, mechanical connection strength. Testing of the connection was then performed to determine the allowable connection strength for each one of our geogrid types. Test walls were constructed in the Atlanta area to prove the viability of the connection and develop an installation procedure for the system. Since the inception of the Mesa® System Tensar has engineered and supplied materials for the construction of several hundred structures on DOT projects throughout the United States, including many critical structures.

The Highway Innovative Technology Evaluation Center, (HITEC), of the Civil Engineering Research Foundation (CERF), published an *Evaluation of the Tensar MESA® Retaining Wall System,* dated April, 2000 with their CERF Report #40358, refer to Appendix – 0.3. The current MESA® system submitted herein differs from the system evaluated by HITEC in 2000 in the way that the facing elements are mechanically connected to the soil reinforcing geogrids. In the HITEC evaluated system, as is the case with present refinements made to the MESA® system, the geogrid layers were connected to the modular block facing units through the use of polymeric mechanical connectors. Now, through the use of an improved MESA® "DOT" connector, fabricated using an HDPE-fiberglass inclusion high strength material, Tensar has increased the number of connection points engaging the geogrid soil reinforcing elements, thereby increasing the efficiency of the system and allowing MESA® to meet the newest standards of the LRFD design methodology including long-term

connection strength. The use of the “DOT” connector permits the use of a wider, more efficient range of Tensar geogrid reinforcements.

The current series of geogrids UX****MSE are similar, but more efficient materials that differ from former versions identified as UXMESA* and UX****HS series Tensar uniaxial HDPE geogrids, in the increased length of the rib elements and apertures. These differences result in minor changes to design properties that increase the design strength and therefore the efficiency of the Tensar uniaxial geogrids. The Tensar UX***MSE series geogrid have also been evaluated by the National Transportation Product Evaluation Program (NTPEP).

Refer to Appendix – 0.2, wherein the full wall system specifications is presented. In addition, refer to Appendix – 0.1 for more information about TIC’s grade separation solutions.

1.1 Facing Unit

- 1.1.1 Innovation**:** Does the wall system contain what you consider to be an innovation that is related to the facing unit? If yes, please **describe the innovation briefly**. As items below apply to the innovation, please **describe the innovation in further detail**.

  Tensar Response: The wall system does not contain what is considered an innovation that is related to the facing unit.

- 1.1.2 Types of Facing Units**: List each type of facing unit**.

  Tensar Response: The MESA system facing utilizes a dry-cast concrete modular (segmental) block facing unit. The MESA Standard units are 18 inches wide, 11 inches deep (nominal), 8 inches high and weigh 75 lbs. Wall batter for the MESA Standard units can be set to 4.5-degree maximum or near vertical (0.45°) by orientation of the MESA connector. Refer to Appendix – 1.1.2.1 sheet 2 for detailed drawings of the commonly used MESA facing units.

- 1.1.3 Specifications**: Provide specifications for each facing component**.

  Tensar Response: Refer to Appendix – 0.2 for a broad description of the MESA system specifications including concrete requirements of the facing elements. For detailed description of each element refer to the following:

  a. MESA Dry-Cast Concrete Modular Facing Units: Appendix – 1.1.3.1a – Quality Control and Specifications sheet for manufacturing Blocks and Appendix – 1.1.3.1b – Full Quality Control Manual for manufacturing Blocks.

  b. MESA Connectors:

     Appendix – 1.1.3.2 – DOT Connector Specifications sheet.

     Appendix – 1.1.3.3 – Standard Connector Specifications sheet.

  c. Non-Woven Geotextile (whenever required): Appendix – 1.1.3.4 – Geotextile MESA Specifications.

- 1.1.4 Connection Details: **Provide description of Connection Details**.

  Tensar Response: The MESA connector is placed in a slot cast into each MESA unit. There are two types of connectors for the MESA system. Two standard connectors are required to be installed on non-geogrid courses as an alignment tool and provide the shear resistance during installation. One DOT connector per block is placed on geogrid courses by mechanically connecting the geogrid to the blocks while providing shear resistance and act as alignment tool. Refer to Standard connector fabrication detail in Appendix – 1.1.4.1 and DOT connector fabrication detail in Appendix – 1.1.4.2.

  The connector is injection molded using a compound of polypropylene (PP) or high-density polyethylene (HDPE) with fiberglass inclusions. All of the connection systems require

placing the geogrid over the slot on top of the Mesa block and then inserting the appropriate connector into the Mesa block slot and/or core, then tightening (pull-taut) the geogrid to the connector. Refer to connector drawings in Appendix – 1.1.2.1 sheet 3 for connector details.

- 1.1.5 Dimensions and Tolerances: **Provide standard dimensions and tolerances for each type of unit (e.g. standard, cap, corner, base, etc.) in plan and section drawings.**

  Tensar Response: the dimensions and tolerances of the facing units are presented in Section 1.1.3. In addition to that, typical details are presented in Appendix – 1.1.2.1.

- 1.1.6 Fabrication Process: **Describe wet- or dry-cast fabrication process.**

  Tensar Response: The facing unit for the Mesa System is done by the dry-cast fabrication process. Refer to Appendix – 1.1.3.1b for additional information.

- 1.1.7 Compressive Strength: **Provide the target 28-day minimum compressive strength.**

  Tensar Response: The target 28-day minimum compressive strength is 4,000 psi in 28 days. Refer to Appendix – 0.2.

- 1.1.8 Concrete Density and Water Absorption (Dry-Cast): **For dry-cast units, provide the target concrete density and maximum water absorption.**

  Tensar Response: The dry-cast units have a target maximum cold water absorption of 5% by weight per ASTM C140-91. For additional information, refer to Quality Control for Manufacturing Blocks in Appendix – 1.1.3.1a.

- 1.1.9 Percent Air Range (Wet-Cast): **For wet-cast units, provide the target percent air range.**

  Tensar Response: Not applicable for dry-cast MESA block units.

- 1.1.10 Inter-Unit Shear: **Provide inter-unit shear test results and design shear capacity envelopes.**

  Tensar Response: For the inter-unit shear test results refer to Appendix – 1.1.10.1.

- 1.1.11 Shear, Alignment, or Bearing Devices: **Describe with text any unit shear, alignment or bearing devices. Provide specifications and detail drawings.**

  Tensar Response: MESA is a dry stacked segmental block faced wall system without any bearing pads. MESA system uses connector so set the block alignment. Refer to Section 1.1.4 for details.

- 1.1.12 Filter: **Describe with text any filter which is used to prevent migration of fill soil through ERS face. Provide specifications.**

Tensar Response: The use of geotextile is required to prevent migration of fill soil through the MESA block face. However, no geotextile filter is required if the structural fill has less than 5% passing the No. 200 sieve. The geotextile filter is required to be installed behind the MESA facing units with no gravel unit fill unless specifically required by the project specifications. Refer to MESA standard/typical details in Appendix – 1.1.2.1 on sheets 10 and 11.

- 1.1.13 Aesthetic Facing Options: **Describe with text the aesthetic facing options that are available. Provide photos, drawings and brochures as appropriate.**

  Tensar Response: MESA offers a variation of texture, relief, color, and graffiti treatment. Refer to MESA system brochure in Appendix – 1.1.13.1. Also, refer to Appendix – 5.1.1.2 for aesthetic pictures.

- 1.1.14 Limits on Facing Units: **Describe any limits on the facing units that are created by curved ERS sections and corners.**

  Tensar Response: Tensar MESA System is versatile and can accommodate complex geometries including curved and corners. As shown in Appendix – 1.1.2.1 on sheet 13, the minimum radius that can be formed with MESA system is 55". Smaller radius will involve significant cutting or trimming of the blocks especially with 4.5-deg batter. A 90-deg. corner can be formed using a corner unit or fabricated on on-site as shown in Appendix – 1.1.2.1 on sheets 12 and 13. Additionally, acute corners can also be fabricated on site as shown in Appendix – 1.1.2.1 on sheets 14 and 15.

1.2 Extensible Reinforcement

- 1.2.1 Innovation**:** Does the ERS contain what you consider to be an innovation that is related to the reinforcement? If yes, please **describe the innovation briefly**. As items below apply to the innovation, please **describe the innovation in further detail**.

  Tensar Response: The ERS does not contain what is considered to be an innovation that is related to the reinforcement.

- 1.2.2 Reinforcement Styles/Types**: List each style or type** that is to be used with the facing system.

  Tensar Response:

  a. UX1100MSE Geogrid
  b. UX1400MSE Geogrid
  c. UX1500MSE Geogrid
  d. UX1600MSE Geogrid
  e. UX1700MSE Geogrid

- 1.2.3 Specifications**: Provide specifications for each style or type** that is to be used with the facing system.

  Tensar Response: Refer to Appendix – 1.2.3.a and Appendix – 1.2.3.b for Tensar's UX**MSE geogrids Strength Design Properties for 75-year and 100-year design life.

- 1.2.4 NTPEP Report**: Provide the current NTPEP report** (if a NTPEP report is not available, then a custom checklist is required).

  Tensar Response: See Appendix – 1.2.4 for NTPEP report and note the following regarding the installation damage factors in the NTPEP report.

  The reduction factor for installation damage for UX1100MSE through UX1700MSE geogrids have been determined through NTPEP evaluation – refer to Appendix – 1.2.4. The installation damage reduction factor range between 1.18 and 1.24 in Silty Sand between 1.31 and 1.39 in Sandy Gravel and between 1.78 and 2.57 in Coarse Gravel.

  Note that the installation damage testing through NTPEP evaluation is performed under a very aggressive damage test program with 6" compacted thickness and heavy 25,000 lbs vibratory compactor. Angular poorly-graded coarse gravel with 0% fines was utilized to determine the installation damage factor for typical crushed stone backfill. This test program resulted in higher than typical reduction factors measured from typical installation specifications with 8" to 10" compacted thickness and 15,000 to 20,000 lbs compactor.

  Based on Tensar's years of experience with site-specific installation damage testing, the typical installation damage factor for well-graded coarse gravel (as opposed to poorly-graded) is between 1.2 to 1.4. The recommended installation damage reduction factor for various aggregate type are listed in Table 1.

Table 1: Recommended Reduction Factor for installation damage, RF_{ID}

Fill material	Geogrid type				
	UX1100MSE	UX1400MSE	UX1500MSE	UX1600MSE	UX1400MSE
Sand, Silt & Clay	1.05*	1.05*	1.05*	1.05*	1.05*
< 3/4" angular aggregate	1.08*	1.08*	1.08*	1.08*	1.08*
< 1.5" angular aggregate	1.10	1.10	1.10	1.10	1.10
< 4" angular aggregate	1.25 - 1.4	1.25 - 1.4	1.25 - 1.4	1.25 - 1.4	1.25 - 1.4

* RF_{ID} of 1.1 as minimum per AAHSTO LRFD specifications.

- 1.2.5 Facing Unit-Reinforcement Connection**: Describe the facing unit-reinforcement connection with text and drawings and provide specifications for any connection devices**.

  Tensar Response: For detailed description refer Section 1.1.4.

- 1.2.6 Connection Strength Tests**: List short- and long-term facing unit reinforcement connection strength tests performed, provide test results and strength envelopes the Applicant recommends for design**.

  Tensar Response: Long-term connection tests were performed for the MESA connection system. For detailed description refer to Appendix – 1.2.6.1 and Appendix – 1.2.6.2

- 1.2.7 Reinforcement Pullout Tests**: List reinforcement pullout (ASTM D6706) tests performed and provide results**. *Provide test soil properties, corresponding pullout friction factors (F) and scale effect correction factors (a) Applicant recommends for design**. **Discuss how test results support these recommendations based on Appendix B at FHWA-NHI-10-025**. If no tests have been performed, **list the default values that should be used based on FHWA-NHI-10-024/025**.

  Tensar Response: The Pullout Coefficient of Interaction, C_i, is the ratio of the average shear stress mobilized on the upper and lower interfaces between the backfill and geogrid to the shear strength of the backfill. The pullout shear strength is developed by a combination of the frictional resistance between backfill and polymer on the surfaces of the geogrid ribs and transverse bars, and the passive resistance from the soil in the geogrid apertures that bears against the transverse bars. Both components of pullout shear are dependent upon the overburden pressure. The passive component of pullout shear is dependent upon the thickness of the transverse bars and the ribs.

  Various pullout tests were conducted on Tensar UXMSE geogrids by Bathurst, Clarabut Geotechnical Testing, Inc., (BCGT) and SGI Testing Services, LLC (SGI). The initial pullout tests were conducted in BCGT with < 65 mm Crushed Stone (GW), in < 10 mm Coarse Sand (SW), and in < 2 mm fine Silty Sand (SM). The Test Reports are included in Appendix – 1.2.7.a, Appendix – 1.2.7.b, Appendix – 1.2.7.c, Appendix – 1.2.7.d. The most recent pullout tests were performed at SGI with <10mm Coarse Sand and Uniform graded crushed rock (No. 57 stone). The test reports are included in Appendix – 1.2.7.

  As shown in the test results, coefficient of pullout interaction factor Ci varies with overburden pressure and in general decrease with increase of overburden pressure. The reinforcement pullout is most critical at the top of wall typically within the 6ft below the top of wall. Because design calculations utilize a constant C_i, it is necessary to select a single value for each geogrid and backfill combination. Based on the test results, Tensar recommends C_i factor of 0.8 for crushed stone and coarse sand meeting AASHTO backfill specifications. Even though the Ci factor may be lower at higher overburden pressure, the pullout is not controlling the design due high overburden pressure.

- 1.2.8 Soil-Geosynthetic Interface Shear Tests**: List soil-geosynthetic interface shear (ASTM D5321) tests performed and provide results**. **List interface friction angle (r) Applicant recommends for design**. **Discuss how test results support these recommendations**. If no tests have been performed, **list the default values that should be used based on FHWA-NHI-10-024/025**.

Tensar Response: The Direct shear interaction coefficient is the ratio of the shear resistance developed when the geogrid and backfill above the layer of reinforcement slide over the backfill below the geogrid to the shear strength of the backfill. The shear resistance in direct sliding is therefore a combination of the shear strength developed between the polymer geogrid and backfill on the bottom surface of the geogrid and the shear strength of the soil within the apertured of the geogrid. Direct Shear tests was performed by SGI Testing Services, LLC per ASTM 5321 using UX1400MSE geogrid and fine uniform sand backfill from Florida having peak friction angle of 32 degrees. The tests represent the worst-case scenario as the typical backfill requirements for MSE wall is crushed angular aggregate with peak friction angle greater than 34 degrees. The test results are attached in Appendix – 1.2.8.

The measured coefficient of sliding C_{DS} values are between 0.71 to 0.85. It varies due to the effect of overburden pressure with the lowest value at low overburden pressure (top of wall). Since sliding between geogrid and the reinforced backfill is critical at the bottom of the wall in design, Tensar recommends using C_{DS} of 0.8 for design with standard AAHSTO angular backfill with 34 degrees friction angle.

1.3 Other Components

- **1.3.1 Innovation:** Does the wall system contain what you consider to be an innovation that is related to a wall component? If yes, please **describe the innovation briefly**. As items below apply to the innovation, please **describe the innovation in further detail**.

  Tensar Response: The wall system does not contain what is considered to be an innovation that is related to the wall component.

- 1.3.2 Reinforced Soil**: Provide the standard Atterberg Limits range, grain-sized distribution range, minimum effective internal angle of friction and limiting electrochemical properties**. **Are these soil parameters consistent with current AASHTO requirements?**.

  Tensar Response: The reinforced soil properties shall be consistent with the current AASHTO LRFD Bridge Construction Specifications as described in Section 7.4.3. However, some state DOTs have lower requirements for structural backfill for MSE wall. The MESA system is adaptable to the state specific requirements for structural backfill due to the inert nature of the HDPE geogrid reinforcement. Nonetheless, the drainage characteristics of the structural backfill outside the AASHTO requirements shall be reviewed by the owner's engineer. Extra care and frequent panel alignment check shall be performed during installation for lower quality backfill outside the AASHTO requirements.

  The reinforced fill gradation requirements shown in Table 2 are provided in the current AASHTO LRFD Bridge Construction Specifications Section 7.4.3.

Table 2: Backfill Gradation Requirements	
Sieve Size	**Percent Passing (%)**
4-inch	100
No. 40	0-60
No. 200	0-15

In accordance with AASHTO T99 and AASHTO T27, the reinforced wall fill material shall exhibit an effective angle of internal friction of 34 degrees or better.

For geosynthetic soil reinforcement, the electrochemical requirement for HDPE is limited to pH >3 (AASHTO LRFD 11.10.6.4.2b), determined by AASHTO T289.

- 1.3.3 Drainage**: Describe with text any internal and external drainage measures that are inherent in the wall system**. That is, they are not optional measures such as blanket and chimney drains or drainage swales, but are built-into wall components.

  Tensar Response: There are no drainage measurements that are inherent in the MESA system.

- 1.3.4 Coping**: Describe with text standard coping that may be used with the wall system, not including the previously described cap units**. **Provide typical dimensions, and plan and section view drawings**.

  Tensar Response: Coping on top of a retaining wall serves as a protective cap or cover, providing both functional and aesthetic benefits. This can be done as a cast-in-place or a pre-cast component. CIP coping can be tailored to fit unique architectural designs and site specific conditions. Precast coping units come in standard sizes but can be customized to fit specific project needs. Refer to Appendix – 1.1.2.1 on sheet 9 for standard drawings.

- 1.3.5 Traffic Barriers**: Describe with text traffic barriers** (i.e. moment slab, post and beam or other) that may be used with the system and any **limitations that may apply**. **Provide typical plan and section view drawings**.

  Tensar Response: Traffic barrier supported on a moment slab can be placed directly on top of the MESA units. Refer to Appendix – 1.1.2.1 on sheet 8 for an example of traffic barrier on top of MESA units. Guardrail post can be installed or driven into the geogrid but should be placed 3ft away from the wall face. Refer to Appendix – 1.1.2.1 on sheet 5 for guardrail post on top of MESA system examples.

- 1.3.6 Slip Joints**: Describe with text how slip joints are made to accommodate potential differential settlement**. **Provide typical plan and elevation view drawings**.

  Tensar Response: Refer to Appendix – 1.1.2.1 on sheet 6 for standard slip joint details in plan and elevation view. If needed, slip joints within Tensar MESA MSE walls will allow for controlled movement between different sections of the wall. This helps accommodate changes in temperature, soil pressure, soil settlement, and other environmental factors that can cause expansion and contraction. Slip joints can also be used in the case that a new

MESA MSE wall must be tied to an existing wall to accommodate the potential differential movement between the existing and new wall.

Section 2: ERS Design

2.1 Design Methodology

- 2.1.1 Innovation**:** Does the wall system contain what you consider to be an innovation that is related to the design methodology? If yes, please **describe the innovation briefly**. As items below apply to the innovation, please **describe the innovation in further detail**.

  Tensar Response: The wall system is not considered to be an innovation that is related to design methodology.

- 2.1.2 Design Methodology Description**: Describe the design methodology thoroughly, and provide references to supporting literature as appropriate**.

  Tensar Response: The Simplified Method design methodology set by AASHTO (American Association of State Highway and Transportation Officials) Load and Resistance Factor Design (LRFD) and Federal Highway Administration (FHWA).

- 2.1.3 Obstructions**: Describe how and provide typical plan and section detail drawings of the facing and reinforcement to handle vertical and horizontal obstructions in the reinforced zone**.

  Tensar Response: In the case that obstructions are present in the reinforced zone, there are various methods in maintaining reinforcement coverage and embedment length requirements. This can be done by skewing, penetrating reinforcement through obstruction, reducing geogrid strength to account for the penetration and directing the geogrid underneath the obstruction. Refer to Appendix – 1.1.2.1 on sheets 5 to 7 for details.

2.2 Design Example

- **2.2.1 Design Calculations: Provide complete calculations for both problems using MSEW**. If the design is performed with software that is not commercially available or is proprietary, please **provide sample calculations with references to support the analysis**.

  Tensar Response: Refer to Appendix – 2.2.1.1a and Appendix – 2.2.1.1b for problem 1 and Appendix – 2.2.1.2a and Appendix – 2.2.1.2b for problem 2 wherein the completed calculations are presented by using of MSEW+ based on the reduction factors provided by NTPEP – refer to Appendix – 1.2.4 and summarized in the table below.

Tensar Structural Geogrid Strength Properties
75-year Design Life

Ultimate Tensile Strength, Creep Reduced Strength and Installation Damage Factors:

	UX1100		UX1400		UX1500		UX1600		UX1700	
	lb/ft	kN/m	lb/ft	kN/m	lb/ft	kN/m	lb/ft	kN/m	lb/ft	kN/m
Ultimate Stength (T_{ult}), ASTM D6637[2]	3970	58.0	4800	70.0	7810	114.0	9870	144.0	11990	175.0
RF_{creep} (Stress Rupture)	2.68		2.68		2.54		2.54		2.54	
Creep Reduced Strength	1481	21.6	1791	26.1	3075	44.9	3886	56.7	4720	68.9
$RF_{durability}$	1.10		1.10		1.10		1.10		1.10	
$RF_{installation\ damage}$ (Silty Sand)	1.24		1.15		1.14		1.15		1.18	
$RF_{installation\ damage}$ (Sandy Gravel)	1.31		1.35		1.34		1.35		1.39	

Long Term Design Strength (LTDS)[3]

	UX1100		UX1400		UX1500		UX1600		UX1700	
	lb/ft	kN/m	lb/ft	kN/m	lb/ft	kN/m	lb/ft	kN/m	lb/ft	kN/m
In Silty Sand	1086	15.9	1416	20.6	2452	35.8	3072	44.8	3637	53.1
In Sandy Gravel	1028	15.0	1206	17.6	2086	30.4	2617	38.2	3087	45.1

Section 3: Construction

3.1 Construction Procedures

- 3.1.1 Innovation**:** Does the wall system contain what you consider to be an innovation that is related to the construction procedures? If yes, please **describe the innovation briefly**. As items below apply to the innovation, please **describe the innovation in further detail**.

  Tensar Response: The MESA wall system does not contain what is considered to be an innovation that is related to the construction procedures.

- 3.1.2 Construction Manual**: Provide the construction manual for the wall system** and at a minimum they should include the following items.

  Tensar Response: Refer to Appendix – 3.1.2.1 for the Installation and special consideration manual of the MESA system.

- 3.1.3 Facing Unit Installation**: Describe any limitations of facing unit installation** at inside and outside curved sections of the wall and at corners as well as any modifications that are required to be made to the facing unit.

  Tensar Response: Refer to Section 1.1.14 for limitations description based on Tensar expertise.

- 3.1.4 Earth Reinforcement Installation**: Describe procedures to install earth reinforcement** at curved sections of the wall and at corners. **Specifically address any measures that are to be taken at intersection or overlapping panels of reinforcement**.

  Tensar Response: Refer to the Installation and special consideration manual of the MESA system in Appendix – 3.1.2.1. In addition to that, refer to MESA standard/typical details in Appendix – 1.1.2.1 on sheet 13.

- 3.1.5 Alignment**: Describe measures that are required to maintain the design vertical and horizontal alignment of the ERS face**.

  Tensar Response: Refer to the Installation and special consideration manual of the MESA system in Appendix – 3.1.2.1. In addition to that, refer to the MESA standard/typical details in Appendix – 1.1.2.1 on sheet 4 for the orientation of the MESA connectors for near vertical and 4.5-degree batter.

- 3.1.6 Soil Installation**: Describe the procedures to install soil in the reinforced soil zone**.

  Tensar Response: Refer to the Installation and special consideration manual of the MESA system in Appendix – 3.1.2.1.

- 3.1.7 Erosion Control**: Describe measures that are required to prevent erosion behind and in front of the wall during construction**.

  Tensar Response: Measures required to prevent erosion behind and in front of the wall during construction can be found in Appendix – 3.1.2.1.

  The contractor shall ensure that the reinforced backfill is graded so that water will drain away from the face of the wall.

- 3.1.8 Contractor Qualifications**: Describe experience or other special qualifications that are required of the wall construction contractor**.

  Tensar Response: Installer with experience on Tensar MESA system is preferred but not required. Tensar will have site representative at the beginning of project to assist and provide guidance on the installation.

- 3.1.9 Soil Installation (Repeated)**: Describe the procedures to install soil in the reinforced soil zone**.

  Tensar Response: Refer to Section 3.1.6 for soil installation procedures.

Section 4: Quality Control

4.1 Manufacturing

- 4.1.1 Facing Units**: Describe the quality control measures that are required for the manufacturing of facing units**. You may do this by providing a manufacturing QC manual.

  Tensar Response: Refer to Appendix 1.1.3.1b for the required quality control measures for the manufactured facing units.

- 4.1.2 Earth Reinforcement**: Describe the quality control measures that are required for the manufacturing of earth reinforcement components**. You may do this by providing a manufacturing QC manual.

  Tensar Response: Refer to Appendix – 4.1.2 for Tensar geogrid quality control and quality assurance manual.

- 4.1.3 Other Devices**: Describe the quality control measures that are required for the manufacturing of any shear, alignment, bearing or connection devices**. You may do this by providing a manufacturing QC manual.

  Tensar Response: Refer to Section 1.1.3 for connections manufacturing specifications and to Appendix – 4.1.3 for example of certification of compliance of MESA connector.

4.2 Construction

- 4.2.1 Construction Quality Control**: Describe the quality control measures that are required during construction of the wall system**. If these measures are described in the system's construction manual then state that they are so included and refer the reviewer to the appropriate section of the submittal.

  Tensar Response: Refer to Appendix – 4.2.1 for the Installation and Construction quality control measures for the MESA wall system. Also, refer to Appendix – 3.1.2.1 for the Installation and special consideration manual of the MESA system pages 6 to 14.

Section 5: Performance

5.1 Performance History

- 5.1.1 Development and Usage History**: Provide a description of the system's development and usage history**. Then describe the following:

  Tensar Response: Refer to Introduction in Section 1. Also, refer to Appendix – 5.1.1.1 for a list of the key MESA wall projects used in abutment applications. Some photos are presented in Appendix – 5.1.1.2.

- 5.1.2 Oldest Structures: **The oldest three structures**.
  Tensar Response:

  - **Pearl Street Overpass, Braintree, MA:** This project was constructed in 1995. See Appendix – 5.1.2.1a for a photo of the project and Appendix – 5.1.2.1b for a detailed report of the project.

  - **Brattleboro, Vermont:** This wall was built in 2003 as the northwest wing wall of the railroad bridge over RT 9. Refer to Appendix – 5.1.2.2 for a detailed report of the project.

- **Founders/Meadows Bridge, CO:** This project was constructed in 1998. This is true abutment MSE wall (GRS). Refer to Appendix – 5.1.2.3 for a detailed report of the project.

- 5.1.3 Tallest Structures: The **tallest three structures**.
  Tensar Response:
  - **I-95/I-695 Interchange, Maryland:** This project features a 45 foot tall wall that was part of a major highway interchange improvement.
  - **SH 65 CDOT project, Mesa County, CO:** This project features a 50 foot tall wall that was constructed in 2007/2008. See Appendix – 5.1.3.2 for additional projects.
  - **Briarcliff Apartments, Kansas City, Missouri:** This project features a 40 feet tall wall. Refer to Appendix – 5.1.3.3 for additional information about the project.

- 5.1.4 User Approvals and Contact Information: **Provide a list of private- and public sector users who have approved the use of the system**. Also **provide the contact information for a person at the user agency who may be contacted regarding the wall system's performance**.

  Tensar Response: Tensar's MESA Retaining Wall Systems have been widely approved and used by various public organizations. Refer to Appendix – 5.1.4. Below are some notable users and contact information for further inquiries:

1. **Baltimore DOT, MD**
   a. Contact: Geotechnical Engineering dept.
   b. Phone: (410) 396-3834
2. **Colorado DOT, CO**
   a. Contact: Dr. Hsing-Cheng Liu
   b. Phone: (303) 757-9011
3. **Washington Department of Transportation, WA**
   a. Contact: Tony Allen
   b. Phone: (306) 709-5450
4. **Utah Department of Transportation, Salt Lake City, UT**
   a. Contact: Darren Sjoblom
   b. Phone: (801) 964-4474
5. **NYSDOT D260830 and Adirondack Northway, NY**
   a. Contact: Mike McCarthy
   b. Phone: (518) 457-2124

6. **Oregon Department of Transportation, Oregon**
    a. Contact: Mike Snyder
    b. Phone: (401) 962-4772

Section 6: Other

6.0 Other Information

- 6.1 Additional Information**:** In this section, please **include anything you think will better help a reviewer understand your ERS that has not been adequately addressed in the previous questions**.

  Tensar Response: No additional information is required. Please refer to the above content for all details needed to assess the MESA MSE wall system, as provided by TIC.

Appendix 0.1

Tensar Grade Separation Solution Brochure

MESA® MSE Retaining Wall System

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ARES®
RETAINING WALL SYSTEMS

MESA®
RETAINING WALL SYSTEMS

SIERRASCAPE®
RETAINING WALL SYSTEM

SIERRA®
SLOPE RETENTION SYSTEM

TENSAR® GRADE SEPARATION SOLUTIONS
SYSTEMS OVERVIEW



Tensar®

We provide innovatively engineered, economical solutions for grade change challenges in the transportation, industrial, commercial and residential markets.



TENSAR® GEOGRIDS

Tensar's Grade Separation Solutions owe their long-term performance and durability to high-strength Tensar® Uniaxial (UX) Geogrids. With their stiff interlocking capabilities, Tensar Geogrids have stood the test of time, performing better than other commercially available geosynthetics.

Proven Solutions and Technologies

Tensar International Corporation (Tensar) is a full-service provider of specialty products and engineering services, offering economical solutions to common infrastructure and site development needs. We are the leading developer and manufacturer of high-performance geosynthetic products and engineered solutions for grade separation applications, as well as the world's leading developer and producer of high-density polyethylene (HDPE) soil reinforcement systems.

We combine our products with innovative application technologies and specialized technical services to offer engineered systems serving the transportation, industrial, commercial and residential markets. Our systems offer cost-effective alternatives to traditional earthwork materials and practices.

Since 1982, Tensar has provided economical solutions to the most challenging grade change requirements. Today, our expertise in grade separation systems includes:

- ARES® Modular Panel Walls
- ARES® Full-Height Panel Walls
- Mesa® Segmental Retaining Walls
- SierraScape® Stone Face Wire-Formed Walls
- SierraScape® Vegetated Face Wire-Formed Walls
- Sierra® Steepened Slopes
- Tensar® Temporary Walls



Eastgate Road – Henderson, Nevada

Tensar® Geogrids are inert to chemical and electrical exposure. ARES panel walls are well suited for fills that do not meet electrochemical backfill criteria.



Inverness Heights – Hoover, Alabama

The random placement of four different colors of Mesa Systems blocks provides a unique and attractive complement to the stores' facades.



Quality You Can Rely On and Have Come to Trust

Tensar offers engineered systems that combine exceptional products with technology, engineering and design. Unique in their quality, consistency and durability, Tensar® Geogrids are manufactured from polymers resistant to physical deterioration and loss of strength caused by aggressive chemical environments.

TENSAR® UNIAXIAL (UX) GEOGRIDS

Tensar UX Geogrids are manufactured from select grades of HDPE resins that are highly oriented and resist deformation when subjected to high tensile loads for long periods of time. These geogrids carry large tensile loads applied in one direction. Their open aperture structure interlocks with natural fill materials, making them ideal for Mechanically Stabilized Earth (MSE) and Reinforced Soil Slope (RSS) applications.

THE ENGINEERED ADVANTAGE™

Tensar is the industry leader in soil reinforcement solutions. We've developed products and technologies at the forefront of the geotechnical industry for more than three decades.

Our products are backed by the most thorough quality assurance practices in the industry. And we can provide full engineering and construction services including detailing, design, site assistance and final stamped drawings for each of our grade separation solutions.



Interstate H-3 – Oahu, Hawaii

ASCE recognized this Sierra® System project with the 1998 Outstanding Civil Engineering Achievement Award.



Blackstone Valley – Millbury, Massachusetts

The SierraScape® System was chosen because it met the aesthetic demands of the owner as well as the engineer's space limitations, all while saving time and money with the use of on-site soils.

When long-term performance and speed of construction are critical, ARES® Retaining Wall Systems offer unmatched advantages.



ARES®
RETAINING WALL SYSTEMS

DOTs, contractors and engineers have long appreciated the many advantages of MSE panel walls. Their wide range of designs and finishes, combined with their simplicity and speed of construction, make them more appealing when compared to other types of wall systems.

Although panel-faced MSE Walls are commonly tied back with steel reinforcing elements, the questionable durability of these elements and their connection to the concrete panels threatens to restrict their use in the future. As an alternative, ARES® Walls, utilizing highly durable, non-corrosive geosynthetic reinforcing elements, provide a cost-effective, functional and aesthetically versatile option.

ARES Walls are proven MSE retaining wall solutions. They have been assessed by the Highway Innovative Technology Evaluation Center (HITEC), and millions of square feet have been installed on a variety of transportation and site development projects. They are currently approved by 46 state DOTs.

NO METAL – NO CORROSION

With soil reinforcement that is 100% polymeric, ARES Walls eliminate corrosion concerns related to either the reinforcing element, or the connection of that element to the panel. These systems offer the cost advantages of an MSE retaining wall without the long-term consequences of exposure to chlorides, sulfates, low-resistivity soils or stray electrical current potential. This makes ARES Walls the logical choice for aggressive backfill soils, transformer platform areas and electrified rail projects.

THE ARES SYSTEMS ADVANTAGE

The use of non-metallic earth reinforcement makes ARES Walls inert to chemical and electrical corrosion. The inert properties of Tensar Geogrids permit the use of a wide range of backfills, including recycled materials, allowing greater economy and a sustainable design.

ARES® Systems Components	
Component	**Function**
Tensar UX Geogrids	HDPE structural geogrids internally reinforce fill materials. Inert to chemical degradation, they can be used with different backfill materials, including recycled concrete.
Precast Panel Facing	Available in standard 5 ft x 5 ft (1.5 m x 1.5 m), 5 ft x 9 ft (1.5 m x 2.75 m), 5 ft x 10 ft (1.5 m x 3 m) or customized for full-height construction.
Bodkin Connector	HDPE connector for high connection efficiency without concern for corrosion.
Full Engineering and Construction Services	Detailing, design, site assistance and stamped drawings for each ARES Wall project upon request.





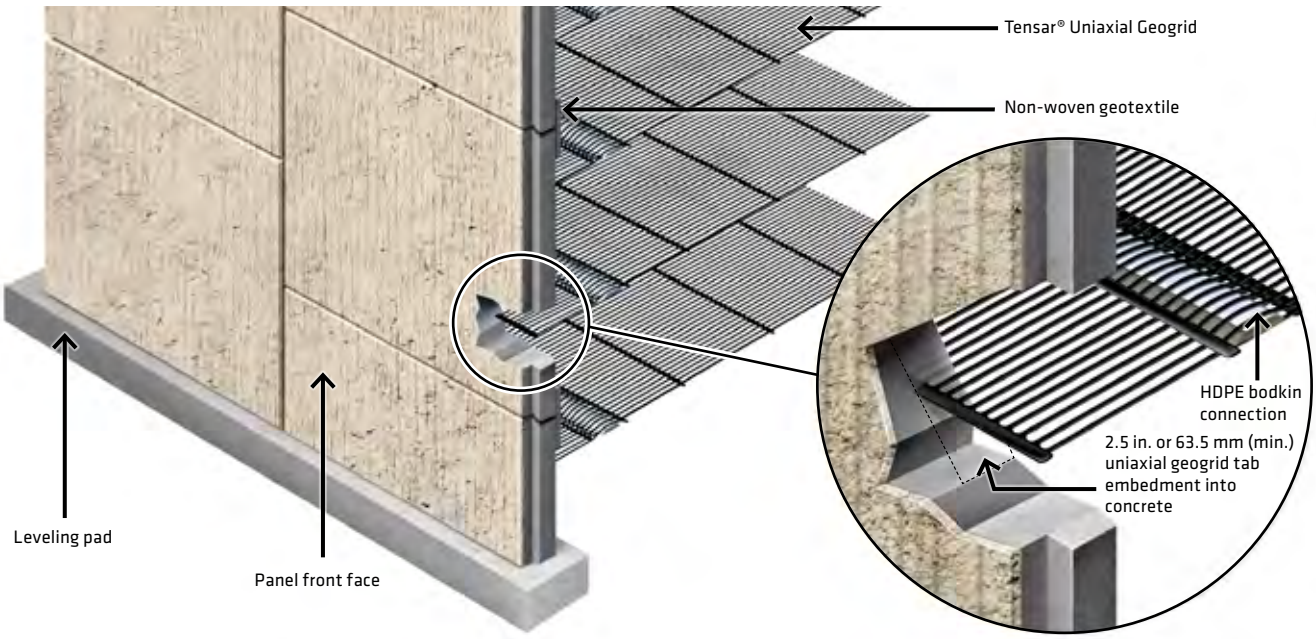
Tanque Verde Interchange – Tucson, Arizona

Constructed in 1984-85, this was one of the first Tensar walls built, demonstrating the long-term performance of the ARES® Full-Height Panel Retaining Wall System.

Proven in the Field

As testimony to the durability of the ARES® Systems, one of the first Tensar® Geogrid-reinforced panel walls was built as a seawall on the Gaspé Peninsula in Canada. After decades of North Atlantic storms and continuous exposure to salt water, there are no signs of corrosion or deterioration of the soil reinforcement. In fact, some of the first ARES Wall installations were instrumented and carefully observed to verify their effectiveness and long-term performance. As part of an FHWA study at the Tanque Verde project in Arizona (see photo, above left), the Tensar Geogrid behind sections of one of the walls was excavated to assess its durability. Installed in 1984-85, the walls continue to perform as designed, with no maintenance issues having occurred.

ARES Retaining Wall Systems – Modular Panel Walls



ARES® Systems provide a structurally reliable solution for a variety of grade change requirements, saving time and money during installation.



Structural Stability Combined with Aesthetic Appeal

AN ECONOMICAL ADVANTAGE

ARES® Systems provide an attractive and reliable solution for a variety of grade change requirements, saving time and money during installation. Modular and full-height panel walls offer designers a choice of textures and patterns to meet the architectural, structural and budgetary needs of retaining wall projects. Panels are cast with HDPE geogrid tabs embedded in the rear face for connection to Tensar® UX Geogrids. The 100% polymer connection assures load transfer to the reinforcement, with no loss in design strength over the project's service life.

In addition to the various facing options available, contractors benefit from the time and labor costs saved when installing ARES Walls. The systems' panels have significant facing area, while the reinforcement is lightweight and easy to connect – no nuts or bolts to tighten.



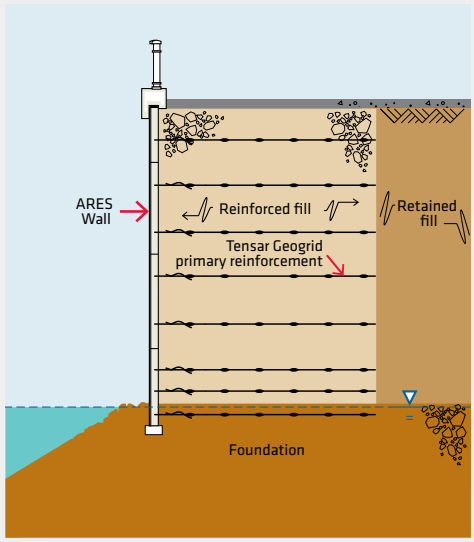
Memorial Causeway – Clearwater, Florida

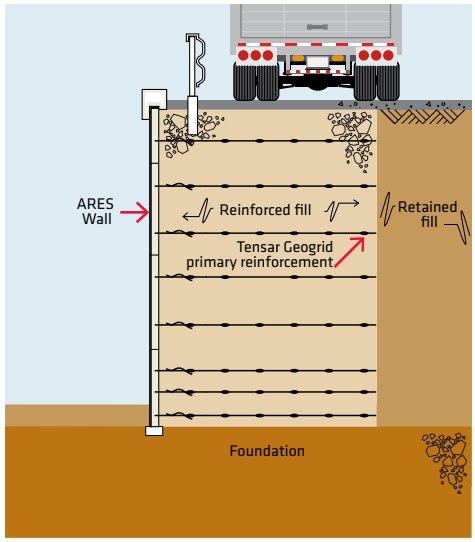
The non-corrosive properties of Tensar Geogrids allow the ARES System to be used in salt water applications.



King Kamehameha – Oahu, Hawaii

ARES Full-Height Panel Walls permit the use of a wide range of both architectural finishes and backfill materials.





NOTE: *All details referenced in this brochure represent typical cross sections.*



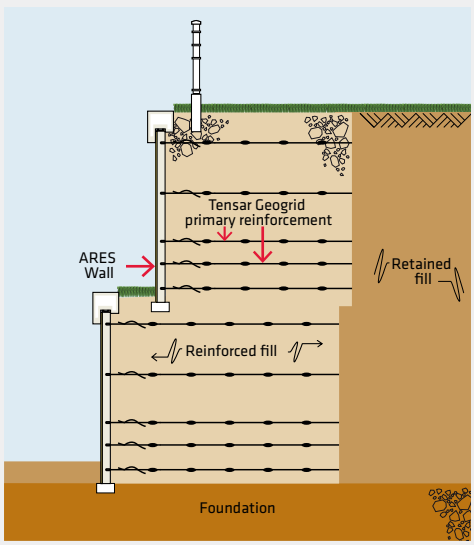






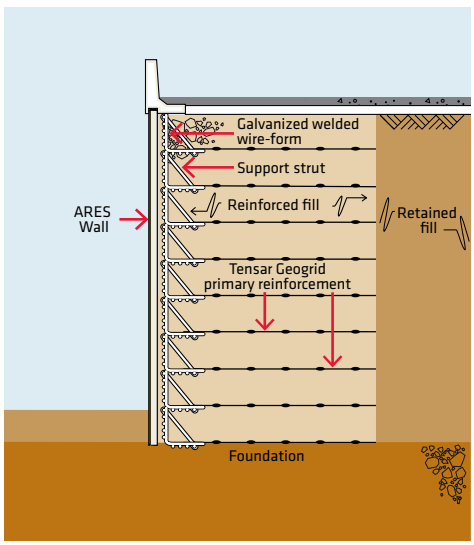
West Reynolds Road – Lexington, Kentucky

Using ARES® Full-Height Panel Walls, the contractor and owner saved a significant amount of time and expense on installation.



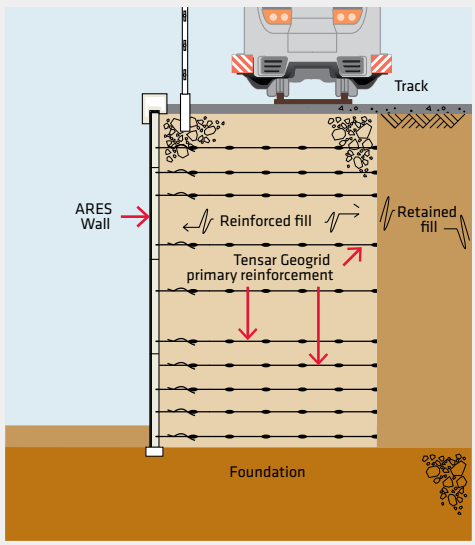
Third Street – Auburn, Washington

Faced with constructing an embankment composed of very soft soils, contractors installed an ARES 2-Stage Wall to allow for anticipated post-construction settlement.



San Diego Light Rail – San Diego, California

Project specifications required that metal not be in contact with the earthen backfill. The Tensar® Geogrids used with the ARES System provided the ideal solution.



Mesa® Systems provide the dependability engineers require, the installation efficiency contractors expect, and the aesthetics owners and architects demand.



MESA®
RETAINING WALL SYSTEMS

A SINGLE-SOURCE SOLUTION

Since 1999, the Mesa® Retaining Wall Systems have been the retaining wall solution of choice for many architects, engineers and DOTs. The first segmental retaining wall (SRW) system to incorporate a positive mechanical connection between the geogrid and the wall face, Mesa Systems offer superior and cost-effective solutions for both structural and landscape retaining wall needs in the transportation, industrial, commercial and residential markets.

With a network of licensed independent block manufacturers throughout the United States, Canada and Latin America, Mesa Systems have become the standard in SRW technology. A truly integrated solution, they are the only SRW system where block, geogrid and connector have been developed by one company; unlike other SRWs, the components of the Mesa Systems have been specifically designed to work together for optimum efficiency and performance. High-strength, low-absorption concrete units, high-shear strength connectors and Tensar® Geogrids work together to form an MSE system that meets or exceeds the industry standard. With the Mesa Systems' high connection strength and reliability, core fill* is rarely needed. Using less core fill reduces imported stone and labor requirements, resulting in greater project savings.

For long-term durability, increased structural integrity and simplified construction, all at costs lower than most conventional alternatives, specify the Mesa Retaining Wall Systems for your next project.

**Core fill is defined as the aggregate fill within the open void space of a SRW block. Core fill is recommended for wall segments that form convex curves with a radius less than 25 ft (7.6 m) and 90° outside corner units.*

Mesa® Systems Components	
Component	**Function**
Tensar UX Geogrids	HDPE structural geogrids internally reinforce fill materials. Inert to chemical degradation, they can be used with non-select fill or even recycled concrete.
Mesa Segmental Units	High-strength concrete block with a compressive strength that exceeds the National Concrete Masonry Association (NCMA) standards (>4,000 psi) (27.6 MPa).
Mesa Connectors	Unique locking connectors designed to mechanically connect Tensar Geogrids to the Mesa Units. Connectors provide a low-strain, end-bearing connection that is not dependent on friction for structural integrity and allows walls to be built near-vertical or with a ⅝" (1.6 cm) setback.
Full Engineering and Construction Services	Detailing, design, site assistance and stamped drawings for each Mesa Wall project upon request.





The Connection You Can Count On™

TESTED TRANSPORTATION SOLUTIONS

The long-term performance of any retaining wall system is challenged most rigorously in the public transportation market. Differential settlement, traffic barriers and seismic loads can test the capabilities of any retaining wall, so connection strength must be especially reliable.

Mesa® Connectors were specifically designed to meet these long-term performance demands, providing a mechanical connection where it's needed most – at the wall face. The system and connection components are largely unaffected by abrasion, tearing or hydrolysis – factors that can undermine the performance of a frictional system using a woven polyester reinforcement. When specifying a Mesa System, you can be assured that its structural components are both reliable and designed for the long term.

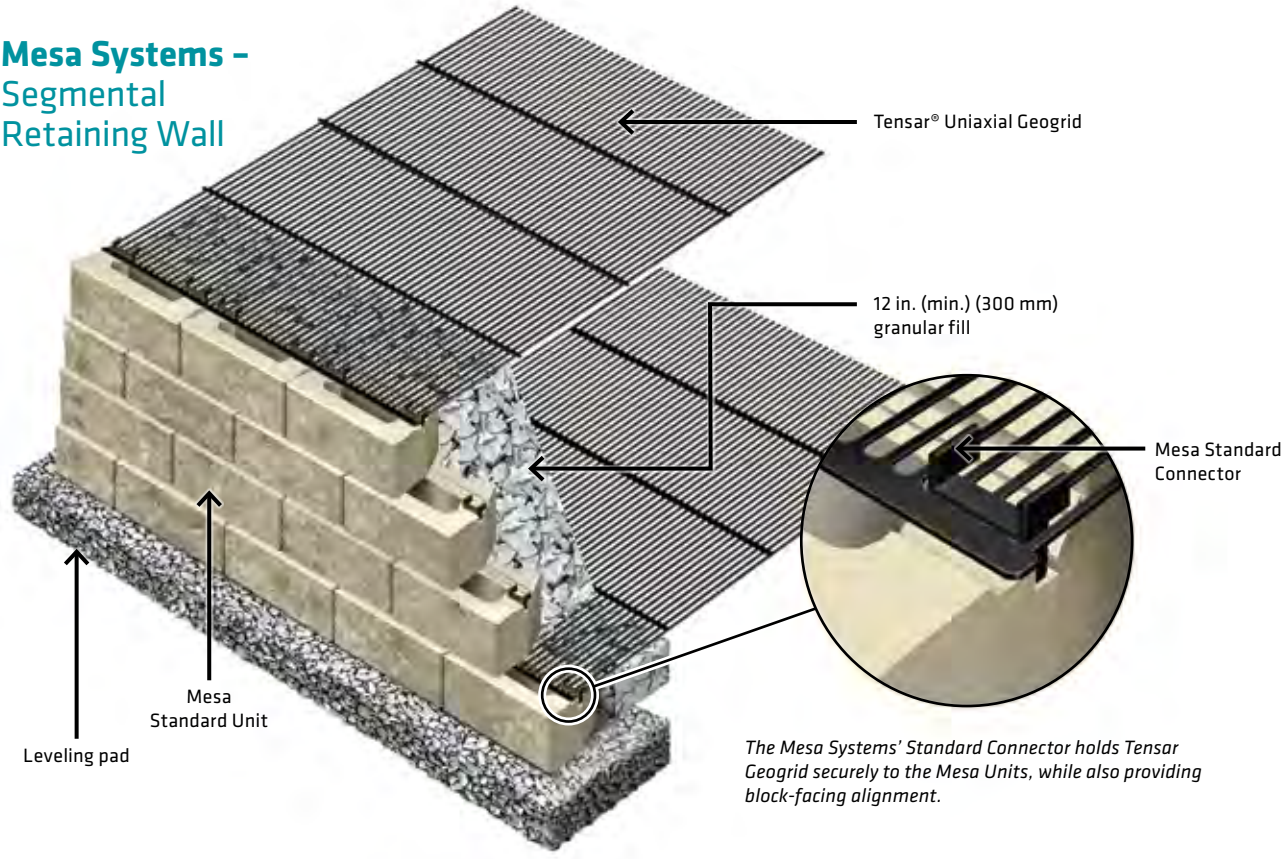
CREATIVE COMMERCIAL APPLICATIONS

When designing for commercial applications, the need to combine creativity, practicality and cost efficiency becomes a necessity. That's where the Mesa Systems can help. Mesa Walls offer both aesthetic appeal and structural stability, providing the ideal solution for almost any commercial project.

RELIABLE RESIDENTIAL PROJECTS

Whether you're increasing usable land or improving residential property value, the Mesa Systems can solve your most challenging grade change requirements with a full line of segmental retaining wall products for both structural and landscaping needs.

Mesa Systems –
Segmental
Retaining Wall



The Mesa Systems' Standard Connector holds Tensar Geogrid securely to the Mesa Units, while also providing block-facing alignment.



Exceptional Performance and Integrity

UNLIMITED DESIGN AND AESTHETIC OPTIONS

If your site requires a retaining wall, why not make it as attractive and useful as possible? In today's competitive and ever-changing wall market, architects and owners are continually looking for new ways to build walls that look as good as they perform.

Mesa® Systems are at the forefront of the industry with a wide array of design options. From blending and variegating colors and textures to planting greenery on tiered walls and top slopes, segmental retaining walls become more than a structural solution – they become an appealing aesthetic feature of your site.

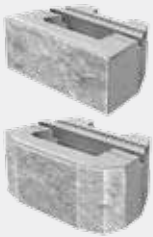
Mesa Standard and DOT Connectors



The Mesa Standard Connectors positively connect Tensar® Geogrid to the Mesa Facing Unit, often eliminating the need for core fill.



The Mesa DOT Connector engages the geogrid across the entire block width.



Standard Units
The most popular Mesa Unit in our full line of products. The Standard Unit can be used for most retaining wall needs.

8" h x 18" w x 11" d nom./75 lbs
(20 cm x 46 cm x 28 cm/34 kg)



Ashford™ Units
Create randomly patterned walls using the Mesa® Ashford™ System. Units are available in three sizes for a variety of facing options (available only with a straight split).

Standard Unit: 8" h x 18" w x 11" d nom./75 lbs (20 cm x 46 cm x 28 cm/34 kg)

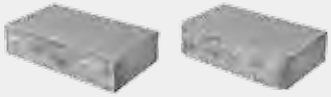
Medium Unit: 8" h x 12" w x 11" d nom./60 lbs (20 cm x 30 cm x 28 cm/27 kg)

Tall Unit: 16" h x 6" w x 11" d nom./60 lbs (40 cm x 15 cm x 28 cm/27 kg)



Corner Units
Used to create walls with clean and precise 90° corners.

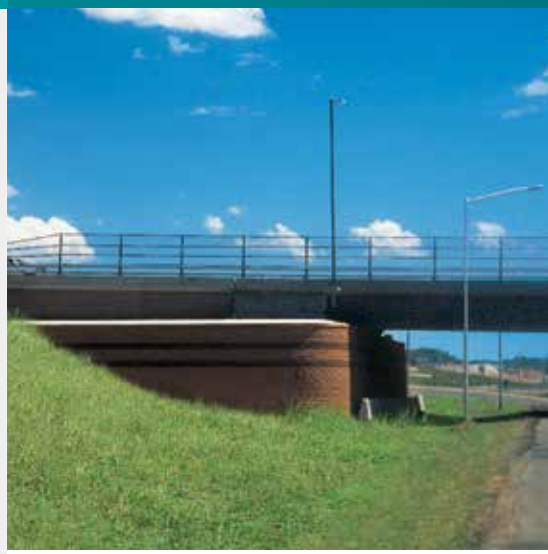
8" h x 18" w x 9" d nom./60 lbs (20 cm x 46 cm x 23 cm/27 kg)



Cap Unit
Used at the top of the wall for a finished look.

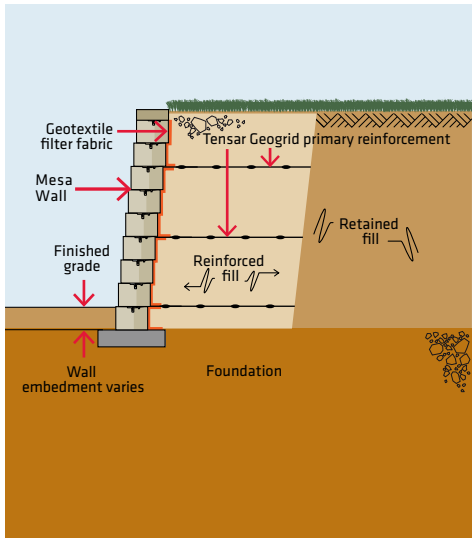
4" h x 18" w x 12" d nom./40 lbs (10 cm x 46 cm x 28 cm/18 kg)

NOTE: *Actual dimensions may vary slightly depending on location.*



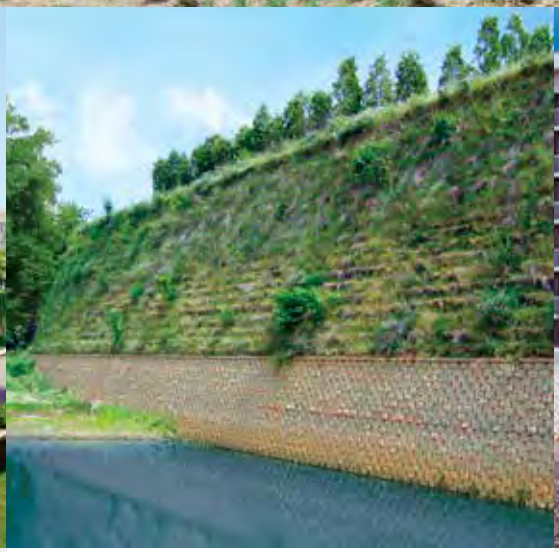
I-25 Founders Meadow – Castle Rock, Colorado

CDOT I-25 was the first major bridge in the United States to be built on footings supported by geogrid-reinforced abutments.











Avalon Bay Communities – Fort Lee, New Jersey

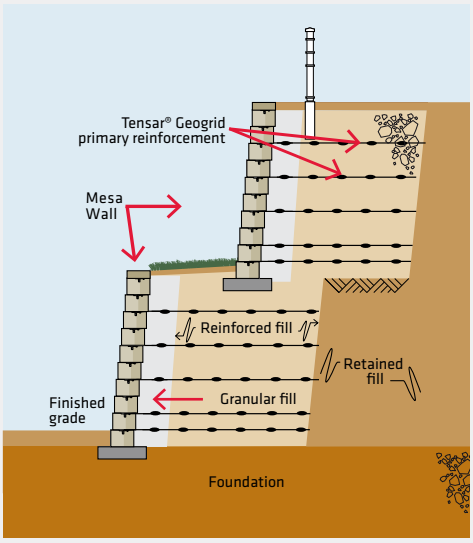
Varying elevations and tight property constraints posed a challenge for the residential site. A design-build solution provided over 48,000 sq ft (4,459 m²) of Mesa® Walls with heights exceeding 30 ft (9 m).

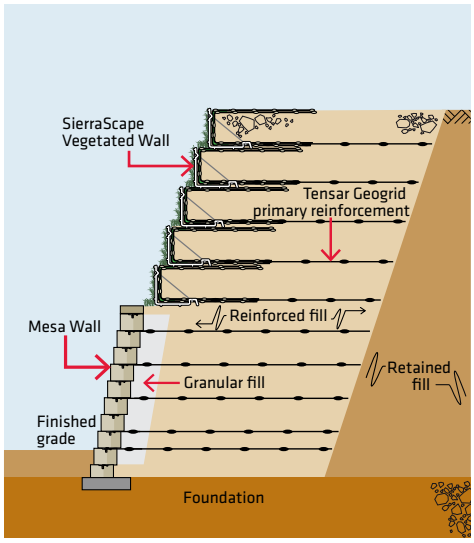
Brookworth Plaza – West Chester, Pennsylvania

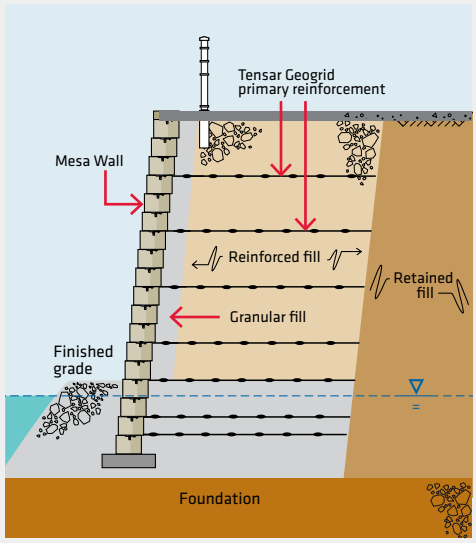
A combined installation of Mesa Walls and SierraScape® Walls provided a solution to the owner's request for green sightlines to the detention pond.

Westgate Villas – Orlando, Florida

Originally proposed 4:1 grass slopes would have resulted in an inefficient use of the property. Instead, Mesa Walls were installed to maximize land use, contain the water runoff and provide structural reliability.







The SierraScape® Retaining Wall System is a more affordable alternative to concrete for various grade change challenges.



SIERRASCAPE®
RETAINING WALL SYSTEM

Engineers and architects are under constant pressure to find cost-effective alternatives to traditional wall systems. For a range of applications, they are finding that the SierraScape® Wire-Formed Retaining Wall System is the ideal solution for appearance, performance and overall value. Backed by decades of engineering experience and with millions of square feet installed worldwide, the SierraScape System is a complete retaining wall solution, featuring materials, design, specifications and technical assistance as needed.

THE POSITIVE CONNECTION™

The SierraScape System combines Tensar® Geogrids with a positive mechanical connection between the geogrid and the wire-form for a dependable, cost-effective solution to the most challenging grade change projects. This connection better withstands differential settlement, offers exceptional performance in areas where seismic activity or heavy external loads are a concern and eliminates surficial stability problems often associated with other structures. It also provides a visual construction quality control check during installation.

INSIDE AND OUT, NO OTHER WALL SYSTEM COMPARES

When it comes to structural stability, no other wire-formed retaining wall compares. The SierraScape System adapts to a variety of project conditions, design requirements and aesthetic options.

Compared to concrete, the system is typically a more cost-effective solution as well. Its unique wire forms ease installation and help minimize construction time. The SierraScape System's low maintenance, design versatility and resistance to environmental degradation help make it the right choice for a number of retaining wall applications.

SierraScape® System Components	
Component	**Function**
Tensar UX Geogrids	HDPE structural geogrids internally reinforce fill materials. Inert to chemical degradation, they can be used with non-select fill or even recycled concrete.
SierraScape Wire-Forms	Galvanized wire-form baskets that provide permanent facial stability during placement and compaction of fill material, and simplify facing alignment.
Locking Tail Strut	Locking struts secure geogrid to the SierraScape System's basket tail and help stiffen the facing element to maintain alignment.
Geotextiles	Separation filter fabric provides a barrier between the backfill material and the stone fill at the face.
Turf Reinforcement Mats (TRMs)	Permanent erosion control products that aid in vegetation establishment and provide long-term turf reinforcement. Used only in vegetated face applications.
Full Engineering and Construction Services	Detailing, design, site assistance and stamped drawings for each SierraScape Wall upon request.





A Complete Retaining Wall Solution

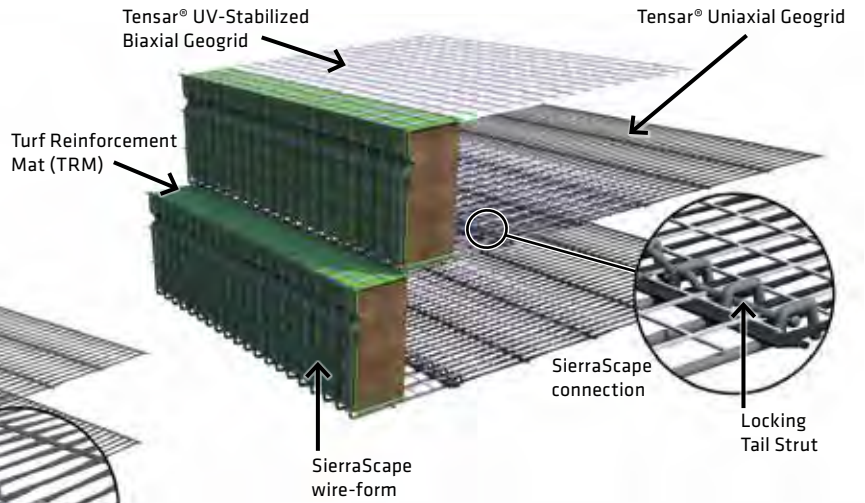
Unlike the stark appearance of most concrete wall systems, the SierraScape® System offers various facing options to meet your design needs. Options include:

- Stone – Components stack quickly and evenly to create a more uniform wall face; their flexibility and connection capabilities help resist differential settlement. Wire-form is filled with native or imported stone that can be color and size-specified for a desired look and feel.
- Vegetated – Provides a versatile soil retaining wall structure where different native vegetated covers can blend naturally with the surrounding environment.
- Architectural – Veneer treated with stacked stone or a shotcrete-sculpted face to provide a unique but traditional finish.

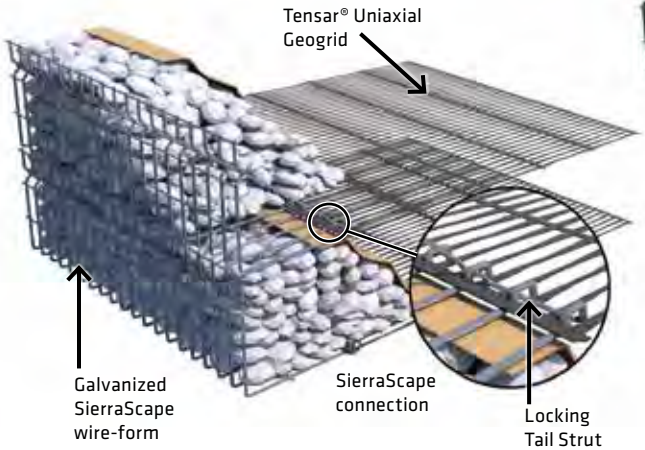
EASE OF INSTALLATION KEEPS COSTS DOWN

Integrating SierraScape System components creates durable, yet simple-to-build structures. With a small number of components, installation moves quickly, without the need for specialized equipment or labor. Welded wire-forms stack easily to create a uniform wall face, and unlike geotextile wrapped walls, Tensar® Geogrids and the unique SierraScape Locking Tail Strut help maintain facing alignment. Since the system can be backfilled with general embankment fills or on-site soils, both fill material and disposal costs can be reduced as well.

SierraScape System – Vegetated Face Option



SierraScape System – Stone Face Option





The Positive Connection™ with an Economical Advantage

THE SOLUTION OF CHOICE

The SierraScape® System has increasingly become the system of choice for grade separation projects in the transportation, industrial, commercial and residential markets. By combining beauty and elegance with efficiency and performance, SierraScape Walls are built to stand the test of time.

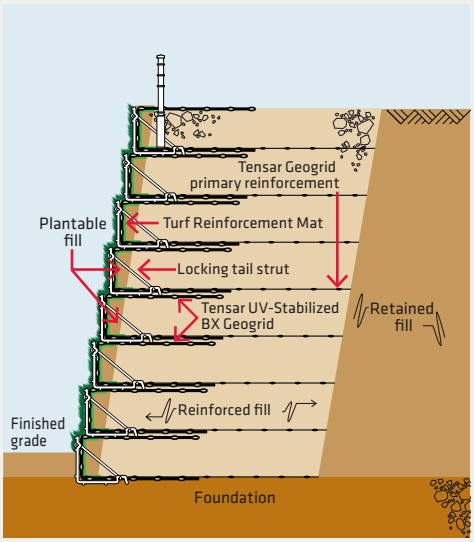
PRESSURE RELIEF APPLICATIONS

In addition to grade separation projects, the SierraScape System can be used for pressure relief to support soil and building loads below grade, providing a faster and more affordable solution. Because the system effectively supports surrounding fill, lateral soil loads against the below-grade structure are eliminated. The result: lower cost foundation walls that are structurally equivalent to conventional installations, including cast-in-place. SierraScape Pressure Relief Walls can be specified for building foundations and existing bridges, flood walls and other new construction or rehabilitation projects.



Post River Apartments – Atlanta, Georgia

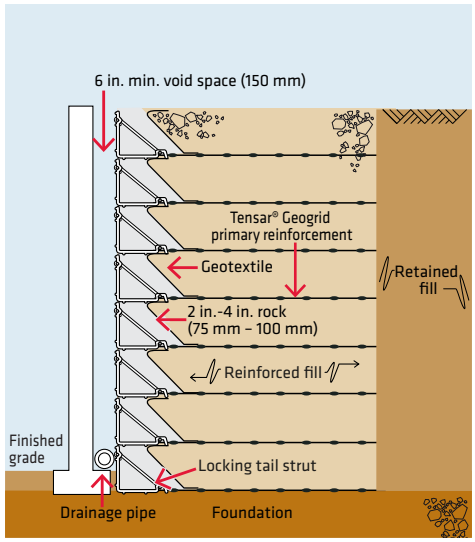
Environmental impact, budget and aesthetics were concerns when specifying a retaining wall for this apartment complex. The natural choice was the SierraScape System with a vegetated face.





Brentwood Towne Square – Pittsburgh, Pennsylvania

To eliminate lateral soil loads, a SierraScape® System Pressure Relief Wall was selected as a cost-effective alternative to traditional below-grade construction.





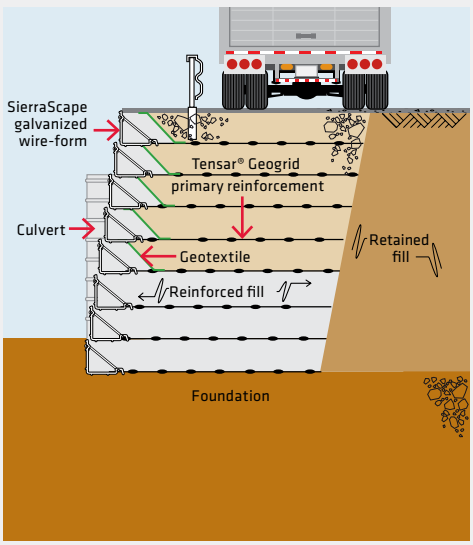






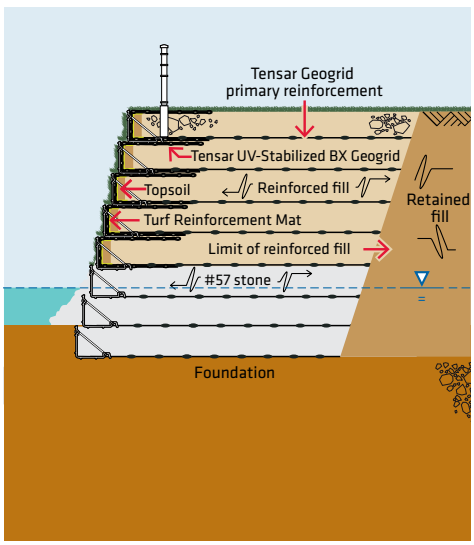
Vulcan Materials Plant – Columbia, South Carolina

The SierraScape System stone-filled wall and arch culvert structure allow heavy trucks easy and secure passage to and from the quarry.



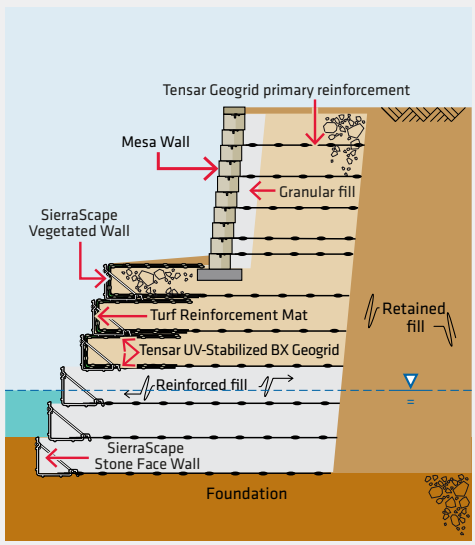
Quail Ridge – Kelowna, British Columbia

The combination of stone face walls for drainage and vegetated walls for a "green" and natural look provided the ideal solution at this upscale golf community.



Webb Gin – Gwinnett County, Georgia

A two-system design combined a Mesa® Wall with SierraScape vegetated and stone face walls for a unique and aesthetic solution.



The Sierra® Slope Retention System provides an economical and aesthetic alternative to conventional retaining walls.



SIERRA®
SLOPE RETENTION SYSTEM

The Sierra® Slope Retention System was introduced by Tensar in 1982 and is a complete and fully integrated MSE solution. Each component has been designed to work together for optimum efficiency in a variety of challenging site and loading conditions.

The Sierra System's cost effectiveness, coupled with its natural aesthetic appeal, provides a reliable solution routinely specified by government agencies, developers, engineers and architects for a variety of applications, including:

- Transportation infrastructure
- Commercial
- Industrial
- Retail
- Recreation facilities
- Municipal water and storm water management
- Single- and multi-family residential housing

The Sierra System significantly reduces material and installation costs by eliminating many limitations imposed by soil conditions, minimizing fill requirements and allowing the use of on-site fills. The primary cost benefit introduced by the Sierra Slope System is the elimination of the relatively costly concrete facing elements, in favor, instead, of less costly, more aesthetically pleasing green elements. The economic benefits of Sierra Slopes have led cost-conscious transportation agencies around the world to specify them for landslide repairs, overpasses and roadway widenings.

These same cost advantages have encouraged site developers to specify the Sierra System to enhance property values and preserve the natural beauty of their sites. Sierra Slopes can be graded so that the appearance of these reinforced earth retention structures is nearly indistinguishable from native terrain.

Sierra® System Components	
Component	**Function**
Tensar® UX Geogrids	Primary reinforcement that internally reinforces soil structure and fill materials.
Tensar® UV-Stabilized BX Geogrids	Secondary reinforcement that provides surficial stability to the slope structure.
Site-Specific Facing System	Provides aesthetic appeal with multiple facing options, including bioengineering.
Turf Reinforcement Mats (TRMs)	TRMs provide long-term erosion protection and help support vegetation establishment.
Full Engineering and Construction Service	Detailing, design, site assistance and stamped drawings for each Sierra Slope project upon request.



A Complete and Proven System

ECONOMICAL

- Creates usable land in undeveloped areas
- Saves up to 60% over conventional concrete retaining walls
- Allows for lower quality fills, so on-site soils are usually acceptable
- Installs quickly without specialized equipment or labor

FLEXIBLE

- Creates slopes from 26° to 70° to fit site development conditions
- Uses a smaller footprint to maximize land for development
- Creates curved slopes and varying face angles for a more natural look
- Works with a variety of facing options, from erosion blankets to professionally landscaped vegetation

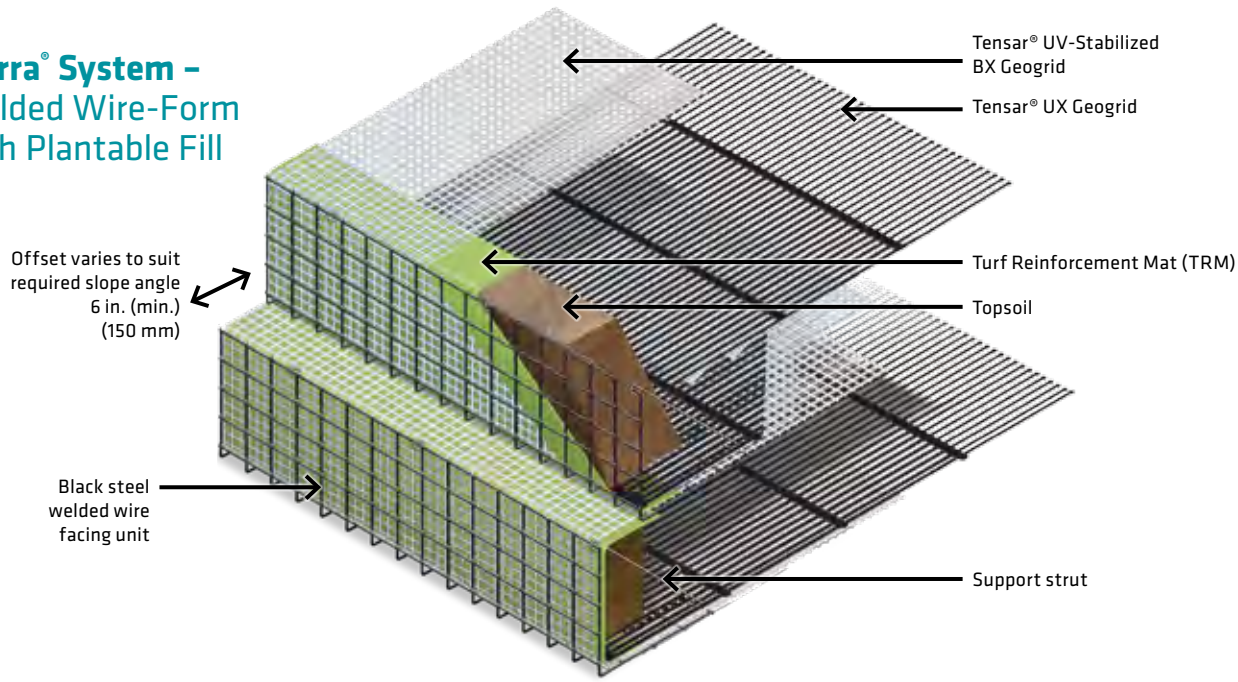
AESTHETICALLY PLEASING

- Blends naturally with surrounding environment
- Not subject to facial distortion and cracking, unlike concrete walls
- Resists effects of differential settlement and seismic activity

PROVEN

- Over 40 million square feet installed
- Stable under the most critical loading structures including railways, bridge abutments and building foundations
- Adaptable to extreme thermal conditions and withstood multiple seismic events exceeding 0.4 g
- Constructed with high-performance Tensar® Geogrids

Sierra® System – Welded Wire-Form with Plantable Fill



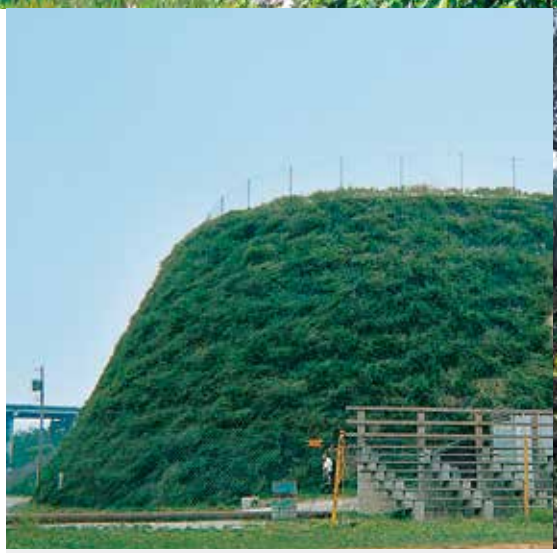


Optimal Designs Based on Site Constraints

SITE-SPECIFIC SOLUTIONS

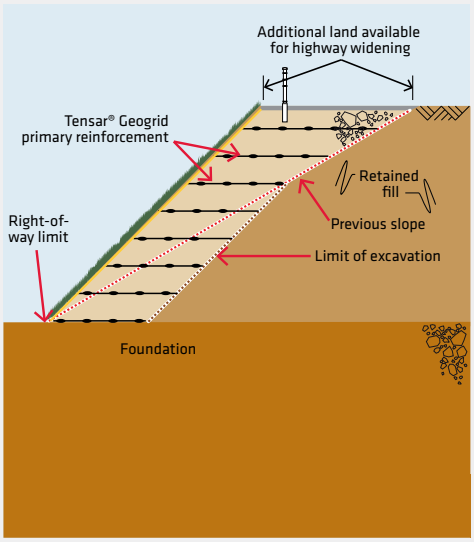
The Sierra® System was developed to provide designers and specifiers countless options for layout and slope angles. The flexibility of the system makes it an ideal choice for solving grade separation challenges.

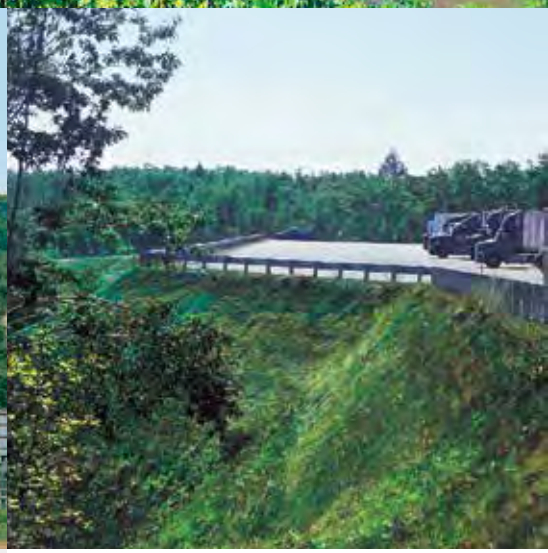
Designers and owners are continually faced with unique property and site constraints that require innovative solutions. Some projects may require a very steep grade change in which a green, vegetated face is the right solution. Other projects may incorporate complementary grade separation systems, such as a Mesa® Retaining Wall, to create an aesthetic and cost-effective alternative.



Bridge Street – Irwin, Pennsylvania

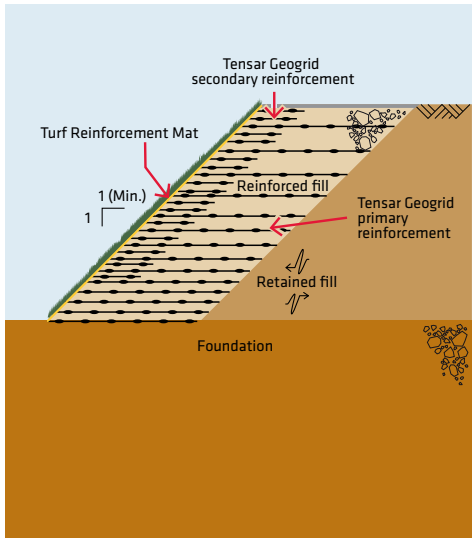
This project demonstrates how DOTs can use the Sierra System to stay within a restricted right-of-way while maximizing land use.





Poland Spring Plant Expansion – Poland Spring, Maine

Flower or grass seed installs quickly and is a low-cost alternative for many projects, including support of roadways and overpasses.





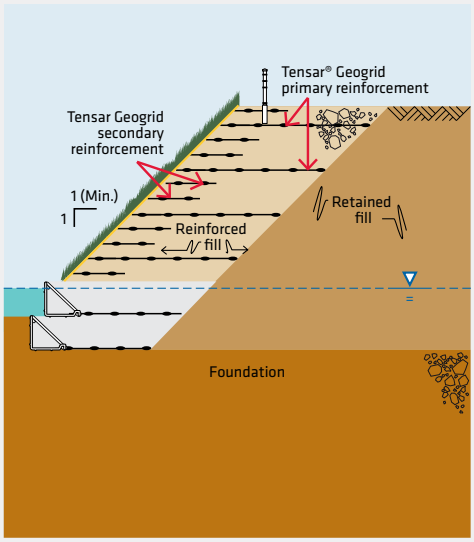






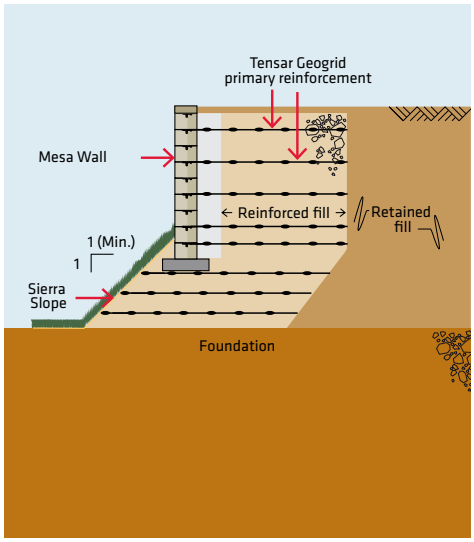
Marriott Riverchase – Hoover, Alabama

The Sierra® System easily adapts to a wide range of slope and slope/wall combinations, but the 1H:1V (above) remains the most commonly specified slope installation.



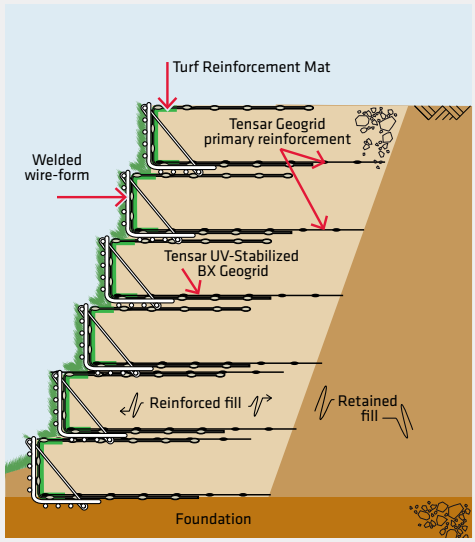
Pearl Street – Braintree, Massachusetts

Sierra Slopes and Mesa® Walls can be combined on projects like this where vertical retention, along with reduced material and labor costs, is essential.



Banks County Landfill – Banks County, Georgia

The Sierra System was used to create 1H:1V inboard slopes and 1H:6V outboard slopes to increase disposal capacity without expanding the landfill's footprint.



With lower cost materials, reduced labor and the use of lightweight installation equipment, a Tensar® Temporary Wall is the economical alternative.

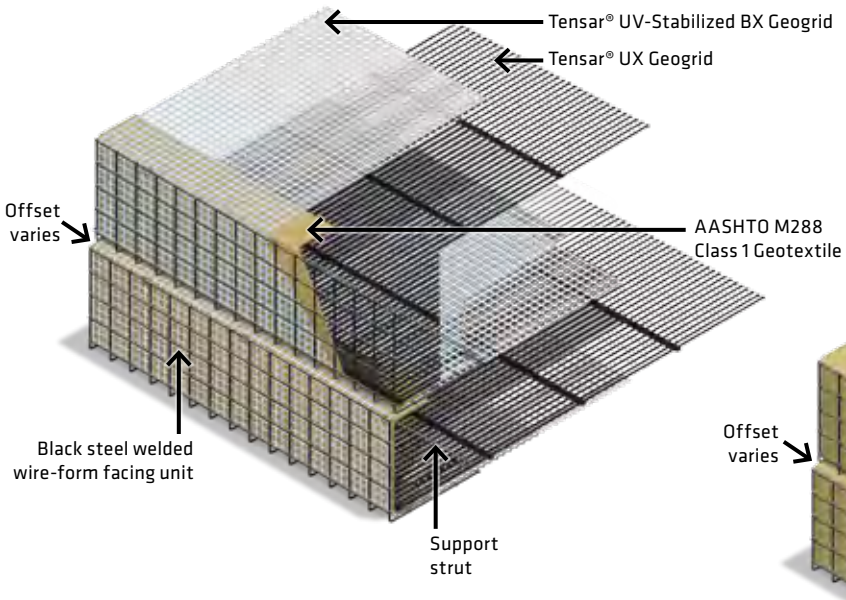


TENSAR® TEMPORARY WALL

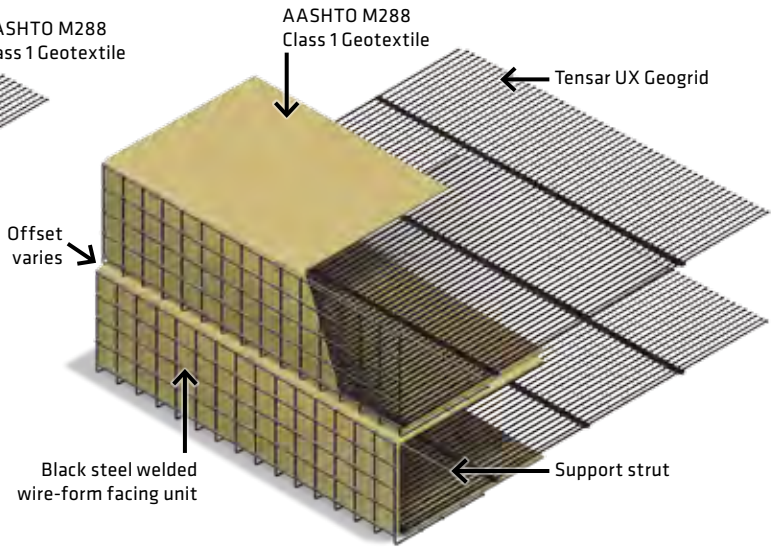
Temporary walls are a necessity for many types of staged construction, but the conventional means for installing them are expensive, requiring heavy lifting and pile-driving equipment. Structures such as soldier piles and lagging walls or sheet piling typically require toe penetration equal to or greater than the wall height, and they may also need secondary bracing or deadmen to retain the fill safely. These walls demand the use of expensive equipment and labor, resulting in significantly increased project costs.

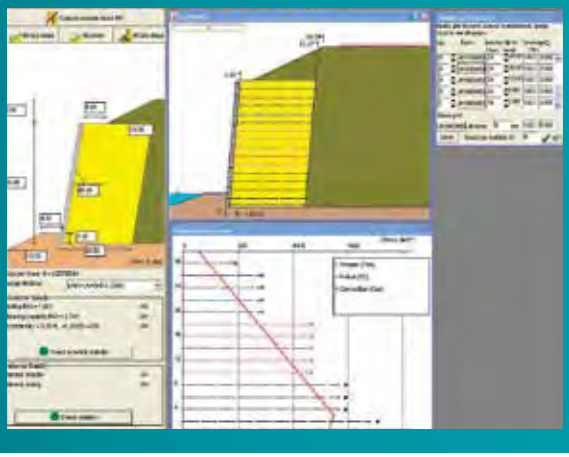
Fortunately, there is a proven technology that allows you to build temporary walls without the challenges and expense of traditional techniques. The Tensar Temporary Retaining Wall System uses an inexpensive wire-form facing system along with Tensar® Geogrids to reinforce the fill. As a result, Tensar Temporary Walls can simplify planning and allow for quicker construction for bridge improvement, road widening, surcharge load cell, phased or staged projects and more. And Tensar Temporary Walls are both durable and flexible; they may be left in place or easily removed as needed.

Temporary Welded Wire-Form – with UV-Stabilized BX Geogrid and Fabric



Temporary Welded Wire-Form – with Fabric





Several windows can be simultaneously displayed on a full screen. Windows can be redimensioned or repositioned for graphic emphasis, presentation purposes and more.

TENSARSOIL-PRO™

GRADE SEPARATION SOLUTION SOFTWARE

Tensar is a global site development solutions provider. And the more we grow, the more we're able to offer unique tools that make the design of grade separation systems simpler, faster and more cost-efficient.

That's why we offer TensarSoil-PRO™ Software to engineers and designers of retaining walls and reinforced soil slopes. First used in Europe, Asia and other regions, TensarSoil-PRO Software has been enhanced to evaluate the feasibility, potential performance and cost benefits of all of our grade separation systems. And it may be used with large precast block systems that incorporate Tensar® Uniaxial Geogrids into their design.

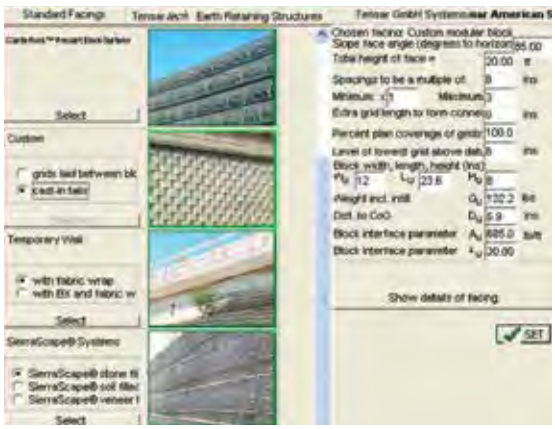
TENSARSOIL-PRO SOFTWARE IS COMPATIBLE WITH ALL MAJOR DESIGN METHODOLOGIES

TensarSoil-PRO Software offers versatile technology that enables you to design grade separation solutions in accordance with all major industry-standard protocols including:

- National Concrete Masonry Association – (NCMA) 1997
- Federal Highway Administration – Demonstration Project 82 (1997)
- Federal Highway Administration – National Highway Institute (FHWA NHI 043) 2001 (AASHTO ASD) 2002
- American Association of State Highway and Transportation Officials – Load and Resistance Factor Design (AASHTO LRFD) 2010

TensarSoil-PRO Software includes information on all Tensar wall and slope systems. Fully interactive, it allows you to input and easily alter wall/slope geometry, geogrid grade or layout, surcharge load and/or soil characteristics – all on a single screen image – to determine stability data and material costs instantaneously. With each change, results are updated in real time. For Tensar walls, that means that any under-designed area is immediately indicated, with no need to back-trace through previous screens for recalculation. Once internal and external stability parameters have been determined, design data can be exported to TensarSlope™ Software, our slope stability application, for comprehensive, compound and global stability analyses.

Also available via a hands-on demonstration is TensarSoil Software, featuring two design methodologies for the analysis of Mesa Retaining Wall Systems, SierraScape Retaining Wall Systems and Tensar Temporary Wall Systems.



TensarSoil-PRO Software allows you to design prototypes and compare costs of different grade separation solutions.



To eliminate costly concrete foundation walls with drilled shaft foundations, the John Paul Jones Arena at the University of Virginia combined the SierraScape® System Pressure Relief Wall with the Tensar Geopier System for foundation improvement.

Tensar Corporation Provides Complete Site Solutions

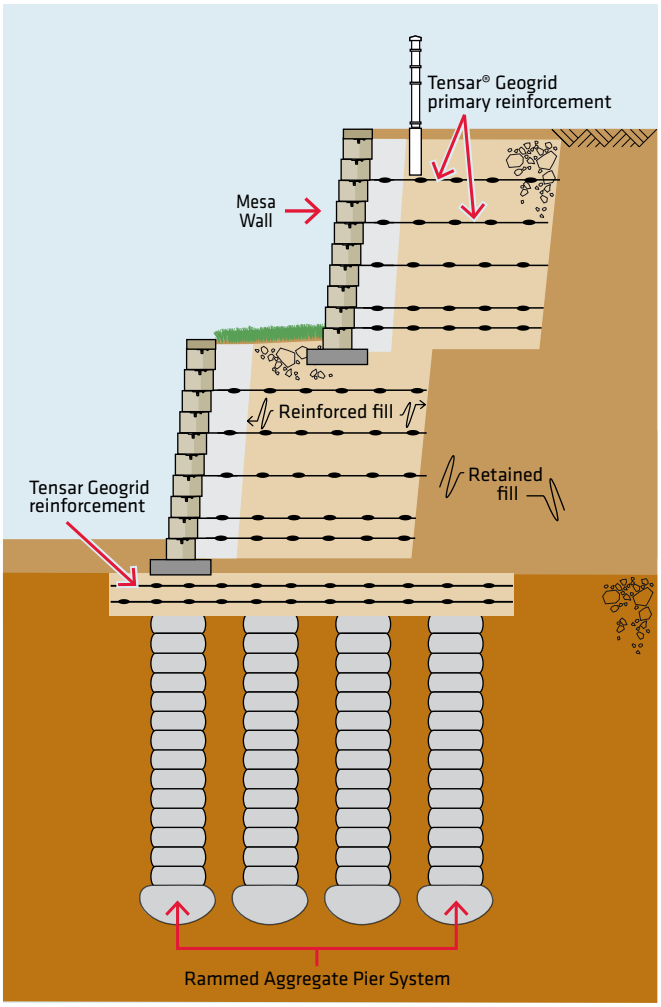
Tensar Corporation is a premier provider of technology-driven site solutions – solutions based on advanced soil reinforcement technologies incorporating high-performance patented products. The benefits of combining two or more of our company's products, services, applications and/or systems result in faster, stronger and more economical solutions that save time and money compared to conventional alternatives. Through Tensar's subsidiary units – Tensar International Corporation, Geopier® Foundation Company and Tensar® North American Green® – we're able to offer customized, engineering-based solutions that address a wide range of site development challenges.

An example of these benefits is found at the Southlands of Orchard project in Denver, Colorado. Existing conditions required the use of a Geopier Rammed Aggregate Pier® (RAP) System to reinforce soft soils found on the site. This solution provided total and differential settlement control as it increased bearing support for the installed Mesa® Retaining Wall Systems. By bringing together two systems from the same company, Tensar is able to provide a complete site solution, all from a single source.



Southlands Orchard Road – Denver, Colorado

Mesa Walls were combined with a Geopier RAP System providing a complete site solution.



Tensar®

Tensar International Corporation
2500 Northwinds Parkway, Suite 500
Alpharetta, Georgia 30009

800-TENSAR-1
tensarcorp.com

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Westminster Levee – Jefferson Parish, Louisiana

By using the Prism System, the Louisiana DOTD and the U.S. Army Corps of Engineers were able to build a 10 ft high (3 m) hurricane protection levee over weak, marshy soils. The project was completed in 1987.

Additional Systems and Services

APPLICATION BREADTH

As a complement to our traditional grade separation solutions, Tensar offers a more specialized application to accommodate embankment construction over soft soils called the Prism® Foundation Improvement System. The Prism System meets the needs of site and infrastructure developers by creating reliable foundations over poor soils.

The Prism System is similar in concept to a concrete raft foundation, but without the time and expense associated with the alternative. By distributing loads more efficiently over underlying soils, the system often eliminates the need for undercutting and backfilling. Furthermore, this system can be used alone or in combination with other soil modification techniques such as deep foundations, surcharging and chemical stabilization. When confronted with soft soils or wetlands, the Prism System is optimal for the construction of:

- Stable embankments
- Bridge approach fills
- Causeways
- Levees
- Dikes

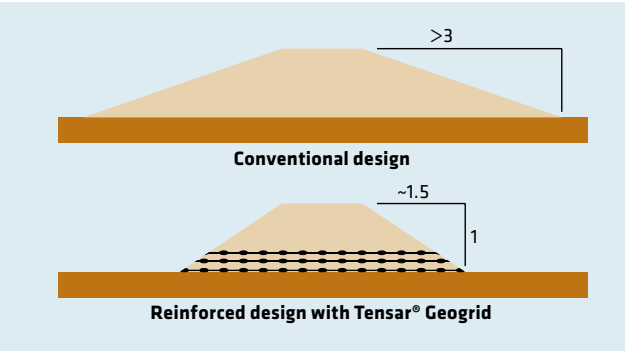
These projects are expensive and time-consuming when built using traditional means. However, with its composite soil and geogrid structure, the Prism System creates a reliable and cost-effective embankment foundation, either independently or in conjunction with other methods.

By creating an internally reinforced core, the Prism System can minimize the embankment footprint, differential settlement and fill requirements. The result is improved structural integrity with reduced construction schedules and lower project costs.

EXPERIENCE YOU CAN RELY ON

The leader in geosynthetic soil reinforcement, Tensar offers a variety of solutions for retaining wall and steepened slope projects. Our products and technologies, backed by the most thorough quality assurance practices, are at the forefront of the industry. Our support services include site evaluation, design consulting and site construction assistance.

For innovative solutions to your engineering challenges, rely on the experience, resources and expertise that have set the industry standard for decades. For more information on any of our grade separation systems, call **800-TENSAR-1**, visit **www.tensarcorp.com** or e-mail **info@tensarcorp.com**. We can provide you with system specifications, design details, conceptual designs, preliminary cost estimates and much more.



The Prism System minimizes the "footprint" of the embankment, as well as its fill requirement and settlement potential.

Appendix 0.2

MESA DOT System Specifications

MESA® MSE Retaining Wall System

Back to main document

SECTION 02834

MESA® MSE DOT RETAINING WALL SYSTEM SPECIFICATION

PART 1 GENERAL

1.1 SECTION INCLUDES

A. MESA Mechanically stabilized earth (MSE) retaining wall system with high-density polyethylene or polypropylene geogrids positively connected to modular concrete facing units.

B. Drainage Fill and Reinforced Backfill.

C. Geotextile, Turf Reinforcement Mat, Drainage Composite and Adhesive.

1.2 RELATED SECTIONS

A. Document 00300 - Information Available to Bidders: Geotechnical Report; Bore hole locations and findings of subsurface materials.

B. Section 01400 - Testing and Inspection Services.

C. Section 02200 - Site Preparation.

D. Section 02300 - Earthwork; Excavation and subgrade preparation.

E. Section 02310 - Grading.

F. Section 02315 - Excavation.

G. Section 02316 - Fill and Backfill.

H. Section 02920 - Lawns and Grasses; Ground cover at finished grade.

I. Section 03300 - Cast-In-Place Concrete: Concrete for leveling pad.

1.3 REFERENCES

A. AASHTO M288 - Standard Specification for Geotextiles.

B. AASHTO T289 - Determining pH of Soil for Use in Corrosion Testing.

C. AASHTO Standard Specification for Highway Bridges.

D. ASTM C 33 - Standard Specification for Concrete Aggregates.

E. ASTM C 90 - Standard Specification for Load-Bearing Concrete Masonry Units.

F. ASTM C 1372 - Standard Specification for Segmental Retaining Wall Units.

G. ASTM D 698 - Standard Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort.

H. ASTM D1556 - Standard Test Method for Density of Soil in Place by the Sand-Cone Method.

I. ASTM D2167 - Standard Test Method for Density and Unit Weight of Soil in Place by the Rubber Balloon Method.

J. ASTM D2922 - Standard Test Method for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth).

K. ASTM D3017 - Standard Test Method for Water Content of Soil and Rock in Place by Nuclear Methods (Shallow Depth).

L. ASTM D 4716 - Standard Test Method for Constant Head Hydraulic Transmissivity (In-Plane Flow) of Geotextiles and Geotextile Related Products.

M. ASTM D 6637 – Determining Tensile Properties of Geogrids by the Single or Multi-Rib Test Method.

N. ASTM F 904 - Standard Test Method for Comparison of Bond Strength or Ply Adhesion of Similar Laminates Made from Flexible Materials.

O. GRI-GG2 - Standard Test Method for Geogrid Junction Strength.

P. NCMA TEK 2-4B - Specification for Segmental Retaining Wall Units.

Q. NCMA – Design Manual for Segmental Retaining Walls.

R. NCMA – Segmental Retaining Walls – Seismic Design Manual.

S. TTN:MESA-DG - Tensar International Corporation “Design Guidelines for Tensar Geogrid Reinforced Soil Walls with Mesa Segmental Concrete Facing Units”.

T. Tensar International Corporation – MESA Retaining Wall Systems – Construction and Quality Control Manual.

U. ASTM C 1262 – Standard Test Method for Evaluating the Freeze-Thaw Durability of Manufactured Concrete Masonry Units.

1.4 SUBMITTALS

A. Submit under provisions of Section 01300.

B. Product Data: Manufacturer's data sheets on each product to be used, including:
   1. Preparation instructions and recommendations.
   2. Storage and handling requirements and recommendations.
   3. Installation methods.

C. Shop Drawings: Engineering drawings, elevations, and large-scale details of elevations, typical sections, details and connections.
   1. Include design calculations sealed by a Registered Professional Engineer licensed in the State where the project is located.
   2. Manufacturer's certifications that the ultimate tensile strength and junction strength of the geogrid are equal to or greater than those specified.

D. Samples: Two samples of each component including:
   1. Geogrid: 4 inch by 18 inch (100 mm by 450 mm) of each type required.
   2. Modular concrete facing unit: Full size facing unit of each type required showing selected color and texture.
   3. Connector: Each type connector.
   4. Geotextile, Turf Reinforcement Mat and Drainage Composite: 4 inch by 18 inch (100 mm by 450 mm) pieces.

E. Manufacturer's Certificate: Certify products meet or exceed specified requirements.

1.5 QUALITY ASSURANCE

A. Design Requirements: Design retaining wall system in accordance with the local codes and regulations and the design guidelines of AASHTO, NCMA or

Tensar Earth Technologies Design shall be by a professional engineer registered in the state where the project is located and who is employed by a firm that has designed at least five mechanically stabilized earth wall projects of similar construction and scope.

B. Manufacturer Qualifications: MSE wall system components manufactured by Tensar Corporation and its Licensees and by companies approved and authorized by Tensar International Corporation.

C. Installer Qualifications: Firm with documented experience of at least five projects of similar construction and scope. Include brief description of each project and name and phone number of owner's representative knowledgeable in each listed project.

D. Pre-Construction Meeting: Prior to erection of retaining walls, conduct a meeting at the site with the retaining wall materials supplier, the retaining wall installer, and the Contractor to review the retaining wall requirements. Notify the Owner and the Architect at least 3 days in advance of the time of the meeting.

1.6 DELIVERY, STORAGE, AND HANDLING

A. Store products in manufacturer's unopened packaging until ready for installation.

B. Prevent excessive mud, fluid concrete, epoxy, or other deleterious materials from coming in contact with retaining wall materials.

C. Polymeric Materials: Store at temperatures above minus 20 degrees F (minus 29 degrees C); rolled materials may be laid flat or stood on end.

D. Store and dispose of solvent-based materials, and materials used with solvent-based materials, in accordance with requirements of local authorities having jurisdiction.

1.7 PROJECT CONDITIONS

A. Do not install leveling pad when subgrade is wet or frozen.

B. Do not place or compact backfill during wet or freezing weather that prevents achievement of specified compaction requirements.

PART 2 PRODUCTS

2.1 MANUFACTURERS

A. Acceptable Manufacturer: Tensar Structural Geogrids: Tensar Corporation, 1210 Citizens, Parkway, Morrow, GA 30260. ASD. Tel: (770) 968-3255 (Intl.), Toll Free: (888) 828-5128. Fax: 404-250-0461. Web Site: www.tensarcorp.com/A. E-mail: info@tensarcorp.com.

B. Acceptable Manufacturer: Modular Concrete Facing Units: Approved Mesa Licensee or an authorized manufacturer of the Mesa Retaining Wall System.

C. Acceptable Manufacturers: MESA Connectors: Companies approved and authorized by Tensar International Corporation

D. Substitutions: Not permitted.

E. Requests for substitutions will be considered in accordance with provisions of Section 01600.

2.2 MATERIALS

A. System Description: Modular Concrete Retaining Walls consist of a mechanically stabilized wall system of engineered backfill reinforced with high-density polyethylene or polypropylene geogrids that are positively connected to modular concrete facing units.

B. Modular Concrete Facing Units: Units certified by an independent laboratory to meet ASTM C 1372 and specified absorption, compressive strength and dimensional tolerances.
   1. Load-bearing masonry units, ASTM C90, normal weight, Type II, minimum compressive strength of 4,000 psi (27,580 kPa), and produced by an approved Mesa System Licensee; conforming to NCMA TEK 2-4B, Section 3.1.
   2. Maximum water absorption rate of 5 percent by weight.
   3. Standard Unit: Mesa.
      a. Size: 8 inch by 18 inch by 11 inch (203 mm by 457 mm by 279 mm).
      b. Weight: 80 to 90 pounds (36 to 41 kg), nominal.
      c. Color: _____.
   4. Cap Unit: Mesa.
      a. Size: 4 inch by 18 inch by 11 inch (101 mm by 457 mm by 279 mm) minimum.
      b. Weight: 40 pounds (18 kg), nominal.
      c. Color: _____.
   5. Corner Unit: Mesa.
      a. Size: 8 inch by 18 inch by 9 inch (203 mm by 457 mm by 229 mm).
      b. Weight: 75 pounds (34 kg), nominal.
      c. Color: _____.

C. Structural Geogrid: Tensar UX1100MSE. Polymeric grid formed by regular network of integrally connected tensile elements with apertures of sufficient

size to allow interlocking with surrounding soil, rock or earth and function primarily as reinforcement.

1. Ultimate Tensile Strength: 3970 pounds per linear foot (58 kN/m), minimum average roll value, when tested in accordance with ASTM D 6637.
2. Junction Strength: 3690 pounds per linear foot (54 kN/m), minimum average roll value, when tested in accordance with GRI-GG2.

D. Structural Geogrid: Tensar UX1400MSE. Polymeric grid formed by regular network of integrally connected tensile elements with apertures of sufficient size to allow interlocking with surrounding soil, rock or earth and function primarily as reinforcement.

1. Ultimate Tensile Strength: 4800 pounds per linear foot (70kN/m), minimum average roll value, when tested in accordance with ASTM D 6637.
2. Junction Strength: 4520 pounds per linear foot (66kN/m), minimum average roll value, when tested in accordance with GRI-GG2.

E. Structural Geogrid: Tensar UX1500MSE. Polymeric grid formed by regular network of integrally connected tensile elements with apertures of sufficient size to allow interlocking with surrounding soil, rock or earth and function primarily as reinforcement.

1. Ultimate Tensile Strength: 7810 pounds per linear foot (114kN/m), minimum average roll value, when tested in accordance with ASTM D 6637.
2. Junction Strength: 7200 pounds per linear foot (105 kN/m), minimum average roll value, when tested in accordance with GRI-GG2.

F. Structural Geogrid: Tensar UX1600MSE. Polymeric grid formed by regular network of integrally connected tensile elements with apertures of sufficient size to allow interlocking with surrounding soil, rock or earth and function primarily as reinforcement.

1. Ultimate Tensile Strength: 9870 pounds per linear foot (135kN/m), minimum average roll value, when tested in accordance with ASTM D 6637.
2. Junction Strength: 9250 pounds per linear foot (135 kN/m), minimum average roll value, when tested in accordance with GRI-GG2.

G. Structural Geogrid: Tensar UX1700MSE. Polymeric grid formed by regular network of integrally connected tensile elements with apertures of sufficient size to allow interlocking with surrounding soil, rock or earth and function primarily as reinforcement.
   1. Ultimate Tensile Strength: 11,990 pounds per linear foot (175 kN/m), minimum average roll value, when tested in accordance with ASTM D 6637.
   2. Junction Strength: 10,970 pounds per linear foot (154 kN/m), minimum average roll value, when tested in accordance with GRI-GG2.

H. Connectors: High-density polyethylene or polypropylene with fiberglass inclusions.
   1. Use MESA Standard Connectors (4 teeth and 2 flags) to serve as alignment and shear connectors between facing courses that do not require geogrid reinforcement. Use two Standard connectors per facing unit with the four teeth driven into the two connector slots.
   2. Use MESA DOT connectors (19 teeth and 8 flags) to connect the geogrid to Standard facing units. One DOT connector shall be used per facing unit with the teeth at each end of the connector driven through the geogrid apertures into the connection slots of the facing unit and the four flags at each end extending into the core of the overlying facing unit.

I. Drainage Fill: Free-draining, coarse-grained soil.
   1. 100 to 75 percent passing a 1-inch (25 mm) sieve.
   2. 50 to 75 percent passing a 3/4-inch (19 mm) sieve.
   3. 0 to 60 percent passing a No. 4 sieve (4.75 mm).
   4. 0 to 50 percent passing a No. 40 sieve (0.425 mm).
   5. 0 to 5 percent passing a No. 200 sieve (0.075 mm).

J. Reinforced Backfill: Granular fill with a pH range of 3 to 11, when tested in accordance with AASHTO T 289 and graded as follows:
   1. 100 percent passing a 4-inch (100 mm) sieve.
   2. 0 to 60 percent passing a No. 40 sieve (0.425 mm).
   3. 0 to 15 percent passing a No. 200 sieve (0.075 mm).
   4. PI $\leq$ 6.

K. Geotextile: Non-woven geotextile, AASHTO M288, Class 2.

L. Turf Reinforcement Mat: North American Green P300 permanent turf reinforcement mat. Mat shall be a consistent thickness of evenly distributed synthetic fibers covered on top by a heavyweight UV stabilized polypropylene netting with an approximate mesh size of 0.5 inch by 0.5 inch (13 mm by 13 mm) and covered on the bottom with a similar net material with an approximate mesh size of 0.625 inch by 0.625 inch (16 mm by 16 mm). Turf Reinforcement Mat: North American Green C350 permanent turf reinforcement mat. Mat shall consist of evenly distributed 100 percent coconut fiber matrix weighing 0.50 lbs per SY (0.27 kg/sq m) encapsulated in a 3-D matting structure consisting of two, top and bottom, heavyweight UV stabilized polypropylene nets, with a nominal weight of 8 lbs/1000 SF (0.04 kg/sq m) and a corrugated high strength center net with an nominal weight of 24 lbs/1000 SF (0.12 kg/sq m). The three nets shall be stitched together on 1.50 inch (38 mm) centers with UV stabilized polypropylene thread to form a permanent three-dimensional turf reinforcement mat with a minimum thickness of 0.5 inches (13 mm).

M. Drainage composite: Non-woven geotextile, AASHTO M288, Class 2, bonded to both sides of a polyethylene net structure.
   1. Minimum Allowable Transmissivity: Not less than 1.5 gallon per minute per foot of width (3×10^{-4} square meters per second) when tested in accordance with ASTM D 4716 at a confirming pressure of 14.5 pounds per square inch (100 kPa).

2. Minimum Allowable Peel Strength of Geotextile from Polyethylene Net: Not less than 1 pound per inch of width (175 N/m) when tested in accordance with ASTM F 904.

N. Adhesive: Exterior, waterproof concrete bonding adhesive.

PART 3 EXECUTION

3.1 PREPARATION

A. Do not begin installation until excavation, foundation preparation and leveling pad have been completed and properly prepared.

B. If subgrade preparation is the responsibility of another installer, notify Architect of unsatisfactory preparation. Do not begin work until unsatisfactory conditions have been rectified.

C. Excavation:
1. Excavate the subgrade vertically to the plan elevation and horizontally to the extent of the geogrid lengths.
2. Excavate trench for leveling pad to the dimensions indicated on the approved shop drawings.
3. Remove soils not meeting required strength and replace with approved materials by the Owner’s Geotechnical Engineer.
4. Protect excavated materials to be used for backfilling the reinforcement zone from the weather.

D. Foundation Preparation:
1. Over-excavated areas of the subgrade and leveling pad trench shall be filled in maximum loose lifts of 10 inches (250 mm) and shall be compacted to a minimum of 95 percent Standard Proctor Dry Density in accordance with ASTM D 698.

2. Owner's Geotechnical Engineer will inspect the subgrade soil for the reinforced zone and leveling pad to ensure proper bearing strength in accordance with the Field Quality Control provisions specified.

E. Leveling Pad:
   1. Material: Unreinforced concrete.
   2. Dimensions: 6 inch (150mm) minimum thickness, 24 inch (600mm) minimum width.
   3. Surface of Leveling pad shall be smooth and horizontal both side-to-side and front-to-back to ensure the first course of units and subsequent courses are level.
   4. Vertical steps in the leveling pad shall be equal to the height of the MESA units or multiples of that height to provide uniform support to overlying units.

3.2 CONSTRUCTION

A. Construct modular concrete retaining walls in accordance with the approved shop drawings and Tensar International Corporation Construction and Quality Control Manual.

B. Facing Unit Installation:
   1. Place first course of modular concrete facing units on the leveling pad.
   2. Verify the first row of units is level from side-to-side and from front-to-back.
   3. Use a string line to align a straight wall; use flexible pipes to establish a smooth convex or concave curved wall.
   4. Use rear edges of the facing units for alignment and measurement.
   5. Sweep tops of modular concrete facing units clean of all debris before installing the next course of units or placing geogrid materials.
   6. Pull a string line after each course has been set to ensure maintenance of the wall's geometry. Reference the string line from the connector slot, the edges of the interior void, or the rear edges of the facing unit.

C. Geogrid Connection and Connector Installation:
   1. Unroll geogrid and cut to length indicated, minus distance between front face of the unit and the front of the connector. Cut geogrid ribs immediately in front of the transverse bar.
   2. Place the geogrid on the facing unit. Insert the connector teeth through the apertures of the geogrid into the slot in the underlying unit. Pull the grid snug against the teeth. Hammer the connector into the slot.
   3. For the DOT System, one DOT connector shall be used per facing unit to connect the geogrid reinforcement to the facing unit and two Standard connectors shall be used per facing unit where no geogrid reinforcement is required:
      a. Position the geogrid laterally on the facing units such that the four teeth at each end of the connector are driven into each of the two connector and the 11 intermediate teeth extend into the core of the facing unit. The DOT connector does not extend over the space between adjacent facing units. The DOT connector may be broken into three pieces if required for better alignment of its teeth with geogrid apertures
      b. Position the flags of the connectors forward for vertical walls and rearward for battered walls.
      c. In the next course, center each facing unit over the two underlying units such that the 8 flags of the connectors extend up into the core of the overlying facing units.
      d. Between facing courses where no geogrid reinforcement is required, insert two Standard connectors into connector slots in the top of each facing to align and provide resistance to sliding of the overlying unit. Orient Standard connector flags in same direction as above.

D. Drainage fill:
   1. Place drainage fill between the units and to a distance of 12 inches (300 mm) behind the rear edges of the units.
   2. Place drainage fill behind the wall before placing the geogrid materials.

3. Cover the drainage fill with geotextile to separate it from the reinforced fill as indicated on the Drawings.
4. Provide drainage fill to within 8 inches (200 mm) of the final grade. Seal the top of the drainage fill column with 8 inches (200 mm) of impervious soil or other material as indicated on the Drawings.

E. Reinforced backfill:
1. Place the reinforced backfill material in maximum loose lifts of 10 inches (250 mm) and compact to a minimum of 95 percent Standard Proctor Dry Density, per ASTM D 698.
2. Use only hand-operated compaction equipment within 3 feet (1 meter) of the rear edges of the facing units. Use a minimum of 3 passes to compact this zone.
3. Verify that backfill compaction requirements is achieved within 3 feet (1 meter) of the facing units.
4. Smooth and level or slope the backfill as indicated so that the geogrid lays flat.

F. Geogrid placement:
1. Unroll the geogrid on the compacted backfill and cut to the length indicated.
2. Pull the geogrid taut to remove slack in the geogrid and at the connectors.
3. Pre -tension the geogrid near the end to maintain alignment and tension during filling.
4. Place a minimum of 3 inches (75 mm) of fill between overlapping layers of geogrid where overlapping occurs behind curves and corners of a wall.
5. Rubber tired vehicles may travel on the geogrid at low speeds, less than 5 miles per hour. Turning of vehicles should be avoided to prevent dislocation or damage to the geogrid and the connected wall facing units.
6. Tracked vehicles shall not be operated directly on the geogrid. A minimum of 8 inches (200 mm) of fill cover over the geogrid is required for operation of tracked construction vehicles in the reinforced zone.

7. Place geogrid shims on the front flange of all facing units connected to geogrid as indicated or as shown in the Tensar Earth Technologies Construction and Quality Control Manual.

G. Toe Fill:
   1. Area in front of leveling footing and lower facing courses shall be filled and compacted before the wall is constructed above 5 feet (1.5 mm) high.
   2. Toe fill shall be placed in loose lifts of 10 inches (250 mm) and shall be compacted to a minimum of 95 percent Standard Proctor Dry Density in accordance with ASTM D 698.

H. Cap Installation:
   1. Install the cap units by bonding them to the units below using an approved exterior concrete adhesive.
   2. Place cap units to achieve a nominal 1-inch (25 mm) overhang.
   3. Clear cap units and modular concrete facing units of all debris and standing water before applying the approved adhesive.
   4. Use string line or flexible pipes to align cap units.

I. Tolerances:
   1. Variation from overall wall batter measured between top and bottom of the wall: Plus or minus 1/8 inch per foot (10 mm per meter), maximum.
   2. Horizontal and vertical alignment: 3/4 inch per 10 feet (6 mm per meter) excluding variations due to facing unit shape or split face irregularities.

3.3 FIELD QUALITY CONTROL

A. Testing and Inspection will be provided by the Owners Testing Agency as specified in Section 01400 Testing and Inspection Services. Notify the Architect 72 hours in advance of testing.

B. Testing and Inspection shall be provided by an independent laboratory provided by the Contractor and acceptable to the Architect.

C. Perform laboratory material tests in accordance with ASTM D 698.

D. Perform in place compaction tests in accordance with the following:
   1. Density Tests: ASTM D 1556, ASTM D 2167, or ASTM D 2922 as appropriate for material tested.
   2. Moisture Tests: ASTM D 3017.

E. Frequency of Tests:
   1. Leveling Pad Trench: A minimum of one test per 100 feet (30 m) of trench.
   2. Subgrade Soil: A minimum of one test per 50 feet (15 m) length of wall.
   3. Reinforced Backfill: Provide one test for every 50 cubic yards (40 cubic meters) of fill placed.

3.4 PROTECTION

A. Protect installed products until completion of project.

B. Repair or replace damaged products before Substantial Completion.

END OF SECTION

Appendix 0.3

MESA HITEC Report

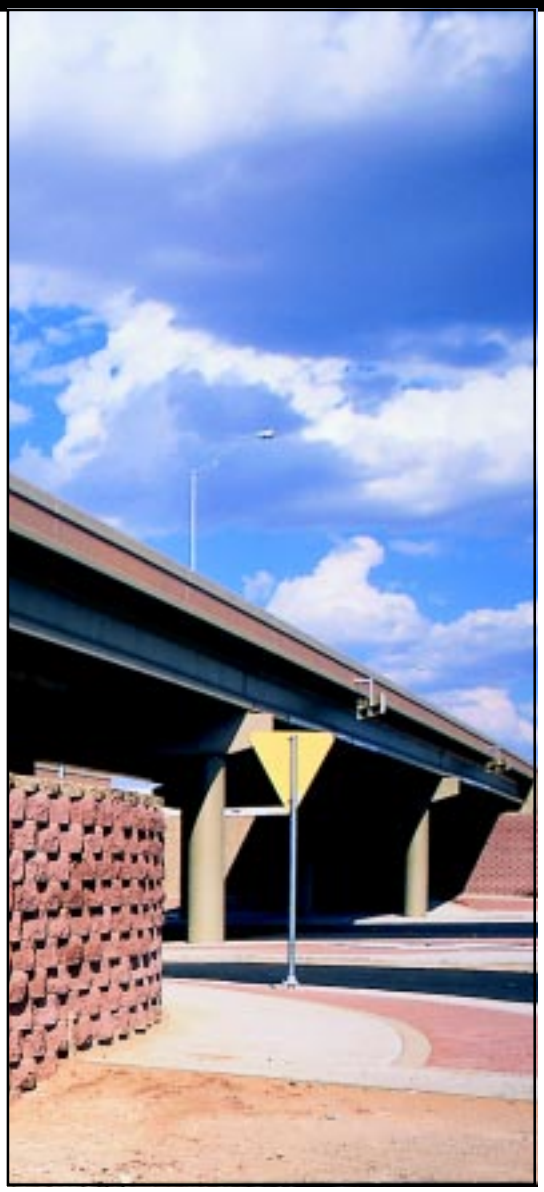
MESA® MSE Retaining Wall System

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TECHNICAL EVALUATION REPORT

EVALUATION OF THE TENSAR Mesa® RETAINING WALL SYSTEM







Prepared by the
Highway Innovative Technology Evaluation Center (HITEC), a service center of the Civil Engineering Research Foundation (CERF)

CERF REPORT: #40358
April 2000

HITEC

Abstract

The Highway Innovative Technology Evaluation Center (HITEC), a service center of the Civil Engineering Research Foundation (CERF), serves as a clearinghouse for implementing highway innovation by conducting nationally-focused, collaborative evaluations of new products and technologies. This report, *Evaluation of the Tensar Mesa® Retaining Wall System*, was prepared as part of the HITEC evaluation for earth retaining systems (ERS). This evaluation was performed on the Mesa® Retaining Wall System (Mesa System), a mechanically stabilized earth (MSE) structure developed, designed, and supplied by Tensar Earth Technologies, Inc.

This report describes a HITEC evaluation designed to determine the basic capabilities and limitations of the Mesa System for use as a technically-viable dry cast modular block MSE retaining wall system when designed with coverage ratios of 71 and/or 100 percent. The evaluation was conducted based on material, design, construction, performance, and quality assurance information outlined in the HITEC Protocol. The Mesa System features a modular block facing (MBW) of various configurations and high density polyethylene (HDPE) geogrid soil reinforcement, which are connected to the facing blocks with glass-reinforced polyethylene structural connectors.

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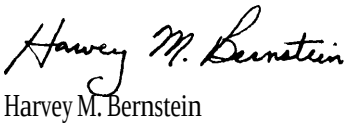
The Highway Innovative Technology Evaluation Center (HITEC), a service center of the Civil Engineering Research Foundation (CERF), prepared this report and wishes to acknowledge the contributions of individuals whose efforts and suggestions have significantly influenced the content of this report. Notably, this report is based on work by members of a technical evaluation panel (Panel) who volunteered to develop the evaluation plan for this project and carry out its objectives. The HITEC Panel is composed of Chairman Terry Shike, David Evans & Associates, Inc.; Tony Allen, Washington State Department of Transportation; Randy Cannon, South Carolina Department of Transportation; Todd Dickson, New York State Department of Transportation; Jerry DiMaggio, Federal Highway Administration; Chris Dumas, Federal Highway Administration; David Dundas, Ontario Ministry of Transportation; Dov Leshchinsky, University of Delaware; Mark McClelland, Texas Department of Transportation; and James Moore, New Hampshire Department of Transportation. Additionally, D'Appolonia served as the consultant to the Panel and was instrumental in producing this report.

CERF also wishes to thank the employees of Tensar Earth Technologies, Inc. for their cooperation during the evaluation process.

Among the staff who worked on this project, I wish to acknowledge the efforts of CERF Senior Vice President and HITEC's Director J. Peter Kissinger, HITEC Senior Program Manager, Scott Edwards, and CERF Communications Manager, Nicole Testa.

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Disclaimer

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Any opinions, findings, conclusions, or recommendations expressed in this publication are those of the Highway Innovative Technology Evaluation Center and do not necessarily reflect the view of the Federal Highway Administration.

This report is the result of an impartial, consensus-based approach to evaluating innovative highway technology in accordance with the HITEC Technical Protocol. The data presented are believed accurate and the analyses credible. The statements made and conclusions drawn regarding the product evaluated do not, however, amount to an endorsement or approval of the product in general or for any particular application.

Preface

When a manufacturer is introducing a new or innovative technology to the highway community, it is often necessary to demonstrate the product to many, if not all, state highway agencies to prove that it performs as claimed. This practice is inefficient, time consuming, and often costly. To overcome these barriers, the Highway Innovative Technology Evaluation Center (HITEC) was established in 1994 in cooperation with the Federal Highway Administration (FHWA), the American Association of State Highway and Transportation Officials (AASHTO), and the Transportation Research Board (TRB). HITEC's mission is to accelerate the process of introducing technological advances to the highway community.

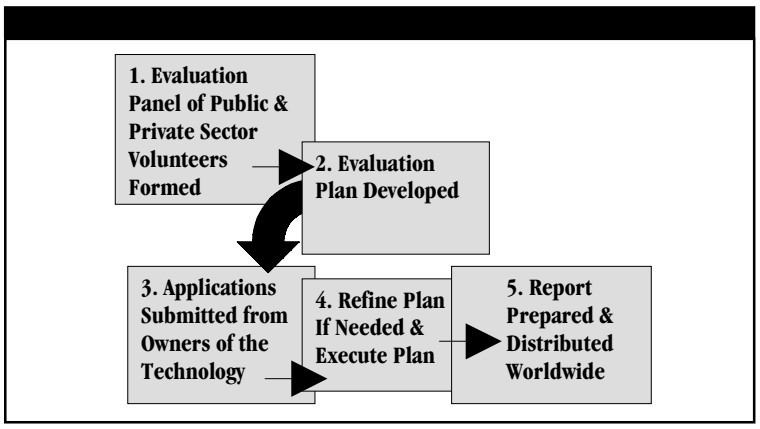
HITEC facilitates the conduct of consensus-based, nationally accepted performance evaluations of new or innovative technologies for the highway community. HITEC is available to evaluate both "high tech" and "low tech" products, systems, services, materials, equipment, or other technologies that the owners believe can be used beneficially on the nation's highways.

The HITEC earth retaining system (ERS) program was initiated at the request of federal and state highway officials and was established through a collaborative relationship with FHWA. It is an ongoing program to evaluate the performance of proprietary ERS technologies against a common evaluation plan. It is believed that the development of up-to-date evaluation criteria and performance information will help reduce the time and efforts required of suppliers and user agencies, and eliminate the inefficiency associated with the current agency-by-agency approval process. The figure below illustrates the step-by-step group evaluation process pioneered by HITEC and used for this ERS program.

The key feature of the HITEC process is the formation of the Technical Evaluation Panel (Panel), a group of key representatives from the user community, academia, and the private sector. The Panel, with the cooperation and assistance of the ERS technology suppliers, identified the specific performance issues and concerns requiring resolution for these products to be adopted by the highway community. The Panel oversaw the development and execution of the evaluation plan, and ultimately, reviewed the evaluation findings.

As a result of their participation in this ERS program, many system suppliers have taken advantage of the process to modify and improve their retaining wall systems so they conform to HITEC Protocol and AASHTO design methods. Consequently, it is important to verify that the retaining wall system currently provided by a supplier is the same as that evaluated in this program.

HITEC is accepting applications for this ERS program on an ongoing basis and will publish the results of each evaluation. Evaluation reports will be developed to provide an analysis of each of the ERS technologies participating in this program. Currently, there are several reports completed and/or scheduled for publication. Additionally, HITEC created the *Guidelines for Evaluating Earth Retaining Systems* report, which fully describes the scope and details of this evaluation program. These reports are available from ASCE at (800) 548-2723 or pubsful@asce.org.



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Executive Summary

This evaluation was performed on the Mesa® Retaining Wall System (Mesa System), a mechanically stabilized earth (MSE) structure, based on data submitted by the developer, designer, and supplier, Tensar Earth Technologies, Inc. (Tensar), Atlanta, Georgia.

The evaluation was conducted on design, construction, and quality assurance information provided by Tensar. The information was evaluated for conformance with the latest state-of-practice criteria as outlined in the HITEC Protocol. As of January 2000, 18 structures using this system have been completed or are under construction for various departments of transportation.

As shown in Figures 1 and 2, the Mesa System features a modular block facing (MBW) of various facial configurations and extruded high-density polyethylene (HDPE) geogrid soil reinforcement, which are connected to the facing blocks with glass-reinforced polyethylene structural connectors. The design of this type of structure is therefore governed by Article 5.8 of AASHTO Standard Specifications (AASHTO, 1999).

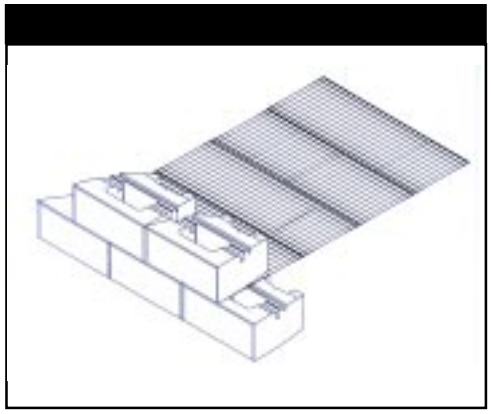


Figure 1. Isometric View of Mesa® – Front Face

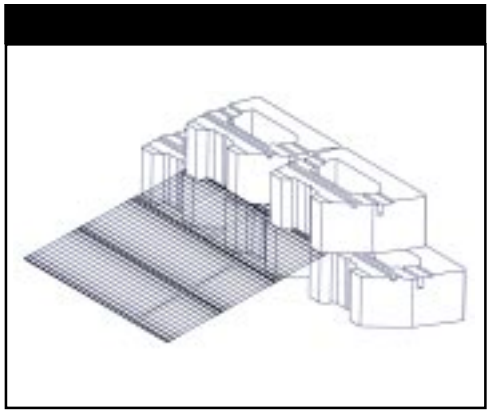


Figure 2. Isometric View of Mesa® – Rear Face

The design methods submitted for external and internal stability are in accordance with the requirements in AASHTO (1999a) except for any variations noted in this evaluation.

With respect to the submitted system-specific design parameters, the following are noted:

- Sufficient connection capacity testing has been performed to qualify all block geogrid combinations. The recommended connection capacity was developed by application of the AASHTO criteria. Interpretation of the connector testing mode of failure was made both visually and by examination of the test stress-strain relationships.
- No evaluation of face bulging potential was made for layers without geogrid by considering intershear block data.
- The proposed design procedure using coverage ratios of 71 and 100 percent have been adequately demonstrated.
- This evaluation for geogrid allowable strength is based on recently completed stress-rupture testing evaluated in full conformance with the HITEC Protocol. The recommended values of RF_{CR} supercedes previous recommendations for UXMESA and UXHS uniaxially drawn HDPE geogrids.

The construction materials and methods specifications and QA/QC programs submitted are in substantial agreement with current practice and AASHTO (1999a).

The present maximum height of the Mesa System using the High Performance (HP) Connectors is limited by the design strength of the geogrid or connection and the height of the modular block, which controls the minimum vertical spacing. This maximum height for static loading is on the order of 55 ft. (17 m). For the Mesa DOT[3] connection system unit under static loading, the maximum height is also on the order of 55 ft. (17 m). The tallest structure completed to date is on the order of 29 ft. (8.8 m).

The Mesa System is a technically viable MSE retaining wall system when designed with coverage ratios of 71 or 100 percent as required by the calculations.

Technical Evaluation Panel Key Contacts

Product: **Mesa® Earth Retaining System**

Chair:
Terry Shike
Senior Bridge Engineer
David Evans & Associates, Inc.

Panelists:

Tony Allen
State Geotechnical Engineer
Washington State Department of Transportation

Randy Cannon
Bridge Design Engineer
South Carolina Department of Transportation

Todd Dickson
Civil Engineer II
New York State Department of Transportation

Jerry DiMaggio
Senior Geotechnical Engineer
Federal Highway Administration

Chris Dumas
Geotechnical Engineer
Federal Highway Administration
Eastern Resource Center

David Dundas
Senior Foundation Engineer
Ministry of Transportation, Ontario

Dov Leshchinsky
Professor of Civil Engineering
University of Delaware

Mark McClelland
Technical Branch Manager
Texas Department of Transportation

James Moore
Administrator, Bureau of Bridge Design
New Hampshire Department of Transportation

HITEC Project Manager: **Scott Edwards**

Consultant:
D'Appolonia
Barry Christopher
Victor Elias
Jim Withiam

Client:
Tensar Earth Technologies, Inc.
5883 Glenridge Drive, Suite 200
Atlanta, GA 30328-5363
Phone: 404-250-1290
Fax: 404-250-9185
Web: www.tensarcorp.com

CHAPTER 1

Introduction



Neptune Terminal Underpass Created to Divert Traffic Flow Below Rail Lines

1.1 Purpose, Scope and Basis for Evaluation

This evaluation was conducted for the Mesa® Retaining Wall System (Mesa System) developed, designed, and supplied by Tensar Earth Technologies, Inc. (Tensar), Atlanta, Georgia. The essential elements of this mechanically stabilized earth (MSE) wall system are a concrete modular block facing (MBW), polymeric soil reinforcing elements, mechanical polymeric connectors between the block facing and the soil reinforcement, and a select granular backfill. Figures 1 and 2 in the Executive Summary show front and rear isometric views of the Mesa System.

The evaluation was conducted using material, design, construction, performance, and quality assurance information provided by Tensar, and was evaluated for conformance to the latest state-of-the-practice criteria outlined in the HITEC Protocol. This Protocol substantially incorporates the AASHTO Standard Specifications for Highway Bridges (AASHTO 1999a) and Demonstration Project 82 Guidelines, FHWA-SA-96-071 (Elias and Christopher, 1996), referred to as Demo 82. Where no applicable criteria in the referenced documents exists, evaluations were based on state-of-the-practice as indicated in the technical literature or documentation submitted by Tensar.

This evaluation is intended for readers who have a working knowledge of the design and construction specification requirements in AASHTO (1999a), Article 5.8 for MSE Walls and Demo 82. Understanding the test methods and interpreting procedures in the Appendices of Demo 82 is essential to understanding the test data submitted by Tensar in support of product-specific design parameters.

The submittal by Tensar for the Mesa System was evaluated relative to the Protocol developed by the HITEC Panel and the Consultant. The Protocol (see Appendix A) was further reviewed and commented on by industry in a public forum prior to being finalized

The results of this evaluation do not constitute an approval or a rejection of the system and/or its components. Further, any recommendations for modifications and/or conformance to specific evaluation criteria should not be construed as mandatory. The potential effects are noted, and each approval agency must determine its own requirements for implementation. It is suggested that manufacturers note any deviation from their HITEC submittal when submitting their system for acceptance by an approving agency.

1.2 Documents Reviewed

The documents that support this report were initially submitted in May 1997. An initial review of these documents indicated the need for additional information to complete the submittal. This information was received in October 1997. During the course of the evaluations, additional information, test data, and clarification was requested and was subsequently submitted for the record in January 1998, April 1998, and September 1998. The Mesa DOT[3] connection system was submitted for evaluation in lieu of previous connection systems in May 1999. Final testing results for both the connections and the geogrid were submitted in October 1999.

CHAPTER 2

History and System Concept



Interstate 40 Rockslide Repair

The Mesa System is an MSE retaining wall system comprised of dry-cast modular blocks for a facing and polymeric reinforcement. The design of this type of structure is therefore fully governed by applicable design criteria in AASHTO (1999).

The Mesa System features proprietary mechanical polymeric connectors, high-density polyethylene (HDPE) uniaxially drawn geogrids used for soil reinforcement and proprietary dry-cast modular block facing elements. The modular block units identified as Standard, High Performance, Cap and Corner, XL, and Landscape are applicable to a Mesa structure. The choice of facing block type is dependent on the specific application, design and performance requirements, and aesthetic requirements. The typical unit height is 8 in. (203 mm) except for Landscape and Cap units, which are 4 in. (102 mm). Widths across the front face range from 16 to 18 in. (406 to 457 mm) and depths into the wall range from 11 to 22 in. (279 to 559 mm). The connector is placed in a slot cast into each block.

The Landscape units are primarily used where accenting and banding as an architectural function is warranted. XL units are used on walls that are of heights that can be used without reinforcement or where project specifications dictate the use of deeper units.

The walls may be constructed vertically or with a batter of 0.675 in. (17 mm) per course.

Mechanical connectors identified as High Performance (HP) Connectors are 14.4 in. (366 mm) in length and have 17 teeth evenly distributed along its length with 14 flags. The DOT[3] connection system consists of two Standard Connectors and one six-tooth Flagless Connector. The Flagless Connector is also identified as a Center Connector. The Standard Connector is 3 in. (75 mm) long and has 4 teeth and 2 flags. The Flagless Connector is 4.5 in. (115 mm) long and has 6 teeth evenly distributed along its length. The connectors are illustrated in Appendix C.

Tensar markets the system through licensees in North and South America, and provides design, technical construction assistance, Mesa facing blocks, geogrid reinforcement materials, and other miscellaneous construction materials.

The first wall using this technology was constructed in 1995 for the Pearl Street Grade Separation project in Massachusetts. As of January 2000, numerous Mesa projects have been completed or are under construction. A detailed listing of 151 such projects totaling 1,735,300 ft.^{2} (161,200 m^2), including key contacts, are provided in Appendix E.

CHAPTER 3

Design Method Evaluations



Tren Urbano Expressway Relocation

3.1 Performance Criteria

The methodology submitted, and supported by typical computations, indicates a design practice that conforms to AASHTO criteria with respect to Factors of Safety (FS) for external and internal stability, foundation embedment, bearing pressure computations, minimum reinforcement length, and maximum vertical reinforcement spacing (Article 5.8, AASHTO, 1999a).

With respect to overall vertical tolerances during erection, project data was reviewed from only one project to indicate the acceptable tolerances in the construction specifications submitted can be met.

Regarding the facing unit(s) tolerance to differential settlement, no estimate or data was presented to indicate a maximum level that would not affect performance or require maintenance. One project experienced differential settlements on the order of 1/30, which caused significant distortions, spalling, and joint dislocation. Review of the submitted project descriptions suggests that differential settlements limited to 1/100 should be considered as guidance to minimally affect maintenance or performance.

3.2 External Stability

The submitted methodology for external stability computations under static and transient loading (dead and live load) conforms to AASHTO (1999a) criteria.

The project owner is responsible for providing strength parameters for the retained fill as well as allowable foundation bearing pressures, anticipated foundation settlement, and global stability determinations for each structure.

Global Stability

Tensar has developed a computer program (MesaPro) for the analysis of complex-tiered structures. The use of this or other global stability programs in general use is consistent with current practice; however, the accuracy and assumptions used in the program have not been reviewed. The program should be considered applicable for analyses of global stability (i.e., critical surfaces beyond the extent of the reinforcements).

3.3 Internal Stability

The submitted methodology for internal stability computations under static and seismic loading conforms with AASHTO (1999a) and Demo 82 criteria with respect to:

- Assumed failure surface for internal stability calculations and calculations for effective length, L_e.
- Horizontal stress computations using K_a.
- Distribution of surcharge and concentrated loads.
- Development of seismic loads and calculations to preclude pullout or rupture.

The maximum vertical spacing of reinforcement must be maintained at less than 800 mm (31.5 in.) to be consistent with AASHTO requirements.

With respect to design parameters needed to determine spacing and sizing of the reinforcement to preclude pullout or rupture, the submitted data for interaction coefficients and allowable strength are described in the following sections.

Interaction Coefficient (C_i)

Tensar methodology uses an interaction coefficient, C_i, which when multiplied by tan ϕ is equivalent to the normalized Friction Factor, F*, used in AASHTO (1999a) and Demo 82. This coefficient was determined in general accordance with the methods outlined in Appendix A of Demo 82. Review of the submitted pullout test data for the UXMESA3 through UXMESA6 geogrids indicates that the interaction coefficients are somewhat affected by normal load (height of fill above) and grain size of the reinforced backfill. For design (using UXMESA3 through UXMESA6 geogrids) in the absence of site-specific interaction coefficient development by test data, the following values of C_i are indicated:

1. For predominantly sand backfills
   Height of fill $\leq$ 1 m: $C_i = 0.80\ \alpha$
   Height of fill $>$ 1 m: $C_i = 0.75\ \alpha$

2. For predominantly gravel backfills
   Height of fill $\leq$ 1 m: $C_i = 0.90\ \alpha$
   Height of fill $>$ 1 m: $C_i = 0.80\ \alpha$

The correction factor for scale effects (α) has been developed by testing as required by Appendix A of Demo 82 and found to be 1.0. These coefficients are consistent with backfills that fall within the grain size range specified in AASHTO, Division II (1999a) and Demo 82. The test data in support of these coefficients are presented in Appendix B.

It should be noted that interaction coefficients are based on tests conducted with the nodes in place at the back of the geogrid. Length measurements in the field should be made to the node at the back of the grid.

Allowable Strength, HDPE Geogrids

The Mesa System features soil reinforcement of uniaxially drawn HDPE geogrids (Tensar UXMESA Series) of various configurations and strengths, which are identical to the UX-HS or UX-SB Series and are differentiated for application purposes.

In accordance with AASHTO (1999a) criteria, the allowable geosynthetic material strength (T_{al}) is defined as the ultimate strength from wide-width tests based on minimum average roll values (MARV) for each product, divided by reduction factors to account for creep (RF_{CR}), durability (RF_D), and installation damage (RF_{ID}) that can be anticipated over a design life of either 100 or 75 years.

The ultimate strength has been obtained from a single rib tensile test (GRI, 1988), which is permitted (AASHTO 5.8.6.1) in lieu of wide-width tests. Reduction factors are determined by laboratory or field tests in accordance with methods outlined in the Demo 82 Manual Appendices. The allowable design load (T_{al}) is the material strength divided by FS = 1.5 for retaining wall design.

The following MARV (T_{ult}) values were submitted for the geogrids. Note that MARV is defined as the mean production strength minus two standard deviations.

The MARV values submitted are greater than or equal to the minimum ultimate acceptable production tensile strength values per lot specified in the Quality Control/Testing and Inspection Manual. These values reflect an apparent industry practice of establishing production minimum strength acceptance criteria at the mean minus three standard deviations.

The 1997-1998 QC results and MARV determinations are presented in Appendix D.

Table 2-1

Product	MARV lb/ft (kN/m)
UXMESA3	4,395 (64.1)
UXMESA4	6,895 (100.6)
UXMESA5	8,995 (131.2)
UXMESA6	10,775 (157.2)

The reduction factors submitted for the UXMESA series Tensar HDPE geogrids are discussed in the following section. The factors are supported by laboratory and/or field test data developed in conformance to AASHTO (1999a) criteria except as noted.

Installation Damage (RF_{ID}) Reduction Factors, HDPE Geogrids

Significant field test data has been developed in support, based on testing in a variety of backfills. Recent tests on the UXMESA series and the older SB/HS series have been reviewed. The recommended installation damage reduction factors are summarized below in Table 2-2.

The reduction factors reflect a uniform average for each product based on the submitted data.

Durability (RF_D) Reduction Factor, HDPE Geogrids

The durability reduction factor is primarily focused on potential strength losses due to oxidation of the polymer during in-ground use over the design life. Currently, no widely accepted laboratory or field-testing protocol is available to measure potential oxidative strength losses.

Based on a review of the polymer literature, results from UV resistance field laboratory test data, and results from one retrieval site, the data strongly suggests that this polymer (HDPE), when used in conjunction with the antioxidant package, offers significant resistance to oxidation over 100 years. In further support, it is noted that HDPE pipe has been successfully performing in ground over the past 50 years with little or no evidence of strength loss under more severe service conditions. Preliminary testing developed by FHWA on the durability of geosynthetics (Elias, et al., 1999) generally confirms this conclusion with the possible exception of in-ground regimes below the water table characterized by very high levels of transition metals. These regimes are extremely rare and specific test data in support must be developed when encountered. Therefore, the submitted $RF_D = 1.1$ is in accordance with the minimum requirement specified in AASHTO (1999a) and is supported by the data.

Creep (RF_{CR}) Reduction Factor, HDPE Geogrids

Laboratory data and interpretations were submitted in support of the determination of a creep limiting long-term strength reduction factor in compliance with the HITEC Protocol.

Table 2-2

Product	Backfill Type	
	Sand D_{max} = 20 mm; D_{50} < 0.7 mm	Gravel D_{max} = 102 mm; D_{50} < 30 mm
UXMESA3	1.15	1.25
UXMESA4	1.10	1.25
UXMESA5	1.10	1.25
UXMESA6	1.10	1.25

Stress rupture testing for the UXMESA series was submitted. The test data are presented in Appendix B. Extrapolation of the data to 75 years indicates a RF_{CR} based on creep-rupture of 2.65. This reduction factor is valid for in-ground use where the temperature is no greater than 20 degrees C.

The previous RF_{CR} of 3.1 for the UXMESA and UX HS series of geogrids outlined in CERF Report # 40301 (HITEC, 1997) is therefore superceded.

Long-Term Allowable Design Strength (T_a), HDPE Geogrids

The long-term allowable design strength, T_a, used for Tensar UXMESA geogrid design depends on:

- Design life desired
- Specific geogrid used
- Predominant backfill size
- FS = 1.5

T_a is expressed by:

$$T_a = \frac{MARV}{RF \times FS} = \frac{MARV}{(2.65)(RF_{ID})(1.1)(1.50)}$$

The total reduction factor, RF, will vary from 3.21 to 3.64, depending on backfill used and be based on supporting data submitted in accordance with AASHTO (1999a).

3.4 Polymer Connectors

To connect the geogrids to the Mesa block facing, glass reinforced HDPE connectors, identified as Mesa DOT[3] and High Performance (HP) connection systems are used. The DOT[3] connection system consists of two Standard Connectors and one Flagless (Center) Connector at each geogrid level providing 14 teeth per standard unit. With the High Performance block units, a continuous HP Connector is used to connect at the geogrid level providing 17 teeth per unit. Two Standard Connectors per unit are used to connect at all other levels.

The polymer connectors are molded from pallets containing approximately 30 percent grade-E fiberglass conforming to ASTM D 578 (1999).

With regard to the creep-limiting strength of the HP Connector, data were submitted in support of a product-specific RF_{CR}. The data are based on a prototype test developed by Tensar, as no industry standard for such tests is presently available. The details are presented in Appendix B.

Note that in developing a method of assessing the connector RF_{CR}, both a short-term tensile/shear strength test and a creep test protocol required development and implementation as no industry standard exists. The short-term strength/shear test developed and used to normalize creep-rupture data, is subject to variability and is dependent on test loading methods and testing time. Therefore, the submitted connector RF_{CR} of 1.2 is currently subject to some uncertainty, but appears to be in the range of 1.2 to 1.4, which is significantly lower than the RF_{CR} for the HDPE geogrids.

No specific oxidation durability data were submitted in support of a design life of 75 to 100 years. However, the oxidative resistance of the connector should be consistent with the resistance of the HDPE matrix and anti-oxidant package as the fiberglass is essentially inert. The supplied connector polymer data indicates that the anti-oxidant package used is almost identical in composition and quantity to that of the HDPE geogrids. Therefore, a $RF_D = 1.1$ for the connector is indicated.

Details for connector placement are included in Appendix C.

Connection Capacity

The connection detail for the Mesa System requires placing the geogrid over the slot and then inserting the appropriate connector into the Mesa block slot and then tightening the geogrid to the connector. The slot and connectors have been designed for use with all UXMESA geogrids.

Connection capacity has been tested for all combinations of blocks and geogrids in general accordance with National Concrete Masonry Association (NCMA) Test Method SRWU-1, as required by AASHTO (1999a).

With respect to the design connection capacity of the UXMESA geogrids and the connectors, application of the AASHTO (1999a) connection design criteria requires that the capacity be the least of the:

- maximum allowable design load of the geogrid; or
- reduced capacity obtained by dividing the failure load, as obtained by testing, by the appropriate creep and aging

reduction factors and a FS = 1.5, if the failure occurs by breakage of the reinforcement; or

- reduced capacity obtained by dividing the failure load, as obtained by connection testing, by 1.5, provided the failure occurs by a pullout of the reinforcement from the connection.

The test data submitted indicate that a breakage failure of either the geogrid or the connector occurred for all connection tests. Therefore, the connection capacity can be calculated using the creep reduction and durability factors used for either the connector or the geogrid. On this basis and the test data submitted, the connection capacities for static loading are provided in the tables below.

Under seismic loading, the allowable connection capacity is the lesser of the geogrid seismic allowable load or the product obtained by dividing the average test load at failure by 1.25, which represents the reduced FS times RF_D.

Because of the apparent low RF_{CR} for the glass-reinforced HDPE connector, the capacity of both the DOT[3] and HP systems is typically governed by the allowable tensile capacity of the geogrids, even though the connection capacity by test of the DOT[3] system is significantly lower than that of the HP system. Use of the HP system should be considered where an additional margin of safety is warranted.

Block Shear Capacity

Block shear capacity was measured in accordance with NCMA Test Method SRWU-2 "Determination of Shear Strength Between Segmental Concrete Units" using both the HP and Standard Connectors (equal to DOT[3]) connecting the HP and/or Standard facing blocks. It should be noted that the shear capacity measured is the sum of friction and/or shear developed between the blocks, and the shear strength of the polymeric connector. The latter portion of capacity as tested in SRWU-2 represents a short-term strength that must be reduced for creep and durability in assessing a long-term relationship between connection shear capacity and normal load. Consequently, the submitted bulging calculations using NCMA procedures and the unfactored (i.e., short term) test values are inconsistent with the intent of the AASHTO Specifications and Demo 82, and no assessment of this mode of failure can be made at this time.

Table 2-3. DOT[3] System (Two Standard, one Center Connector per unit)

Product	Average Test Load @ Failure lb/ft (kN/m)	Allowable Capacity lb/ft (kN/m)
UXMESA3	2779 (40.54)	T_a
UXMESA4	3212 (46.86)	T_a
UXMESA5	3892 (56.78)	T_a
UXMESA6	4224 (61.62)	2000 (29.18)

Table 2-4. HP System (Full length Connector per unit)

Product	Average Test Load @ Failure lb/ft (kN/m)	Allowable Capacity lb/ft (kN/m)
UXMESA3	3480 (20.18)	800 (11.67)
UXMESA4	5000 (32.76)	T_a
UXMESA5	7220 (105.33)	T_a
UXMESA6	8050 (117.45)	T_a

3.5 Backfill in Reinforced Zone

Construction of the Mesa System requires the use of select granular fill in accordance with the grain size and soundness requirements in AASHTO Division II (1999b). No deviation is recommended.

The electrochemical requirements for minimum resistivity, maximum chloride, and sulfate content in AASHTO (1999b) should be waived because they are applicable to metallic reinforcement only. Tensar submitted a deviation from the permissible pH range of the backfill to a lower limit of 2, based on short-term (i.e., 120 day) immersion testing at room temperature. Longer-term immersion data of this type would be required to justify this deviation. In the absence of such data, a pH range of greater than 3 should be considered, which is consistent with current Demo 82 criteria for polyolefin products. The upper limit of 11 appears reasonable in view of the resistance of HDPE to an alkaline environment.

It should be noted that the effect of this lower and higher pH in the backfill on the durability of the facing concrete modular blocks has not been demonstrated. MSE structures utilizing metallic reinforcements and concrete panels limit the backfill pH to a range between 5 and 10 in accordance with AASHTO (1999b) criteria.

3.6 Design Computations

Design computations for a typical vertical wall with horizontal and sloping backfill have been checked for compliance to AASHTO (1999a) criteria. External stability calculations for static and seismic design are in compliance with respect to methodology for the typical cases submitted. Internal stability methodology is in compliance with respect to methodology.

Reinforcement length should be measured from the back of the facing units as per the submitted calculations and AASHTO.

Specific project computations should include the type of facing utilized and reflect the recommended design parameters from this evaluation.

Typical computations for the horizontal backfill case are enclosed in Appendix B.

Coverage Ratios

According to the submittal, coverage ratios can be designed at 71 and/or 100 percent only. Where a coverage ratio of 71 percent is used, a permanent, full-length rebar has often been inserted in the slot provided for the full length of the adjacent block course. The rebar is not required for stress transfer, but it is useful in ensuring proper alignment.

The analytical procedure for designing with coverage ratios of 71 percent or greater has been demonstrated in this submittal. It should be further noted that construction with coverage ratios of less than 100 percent requires careful monitoring in the field to ensure that the proper offset stagger at reinforcement layers is maintained. This would be essential for successful installations. Typical proposed spacing arrangements for a 71 percent coverage ratio are shown in Appendix C with the installation procedure.

3.7 Limitations

Current limitations in AASHTO (1999a) with respect to applications should be observed.

The Mesa System was not evaluated for bridge abutment applications in accordance with Tensar submittals.

Site specific testing for geogrid durability are indicated for regimes below the water table characterized by very high levels of transition metals.

This evaluation has identified a height limitation of approximately 55 ft. (17 m) for a vertical wall with horizontal backfill. This limit is based on the geogrid or connector capacity, a coverage ratio of 100 percent, the use of the HP facing unit, and a vertical spacing of 8 in. (200 mm) in the lower portion of the structure.

3.8 Design Details

Facing Units (MBW)

No structural design is required for the facing blocks. Minimum compressive strength and water absorption should be specified to ensure durable units. Additional freeze-thaw requirements should be required where site conditions warrant them. Where required, freeze-thaw testing should be performed in accordance with ASTM C 666 (1999) and acceptance based on the calculated durability factors

normally accepted by the owner for precast concrete. Alternatively, a surface seal coating may be applied to the facing in lieu of ASTM C 666 (1999) testing. Acceptance based on ASTM D 1262 (1999), as proposed by Tensar, is not considered sufficient, as the new AASHTO Specifications (2000 Interims to be published) have additional requirements which result in more stringent criteria than contained in ASTM D 1262. Acceptance based on AASHTO (2000 Interims to be published) should be satisfactory.

The annulus in the facing block may be filled where required with the specified unit fill. Connection capacities have been developed with a void in the annulus.

Architectural finishes are available. Mesa units can be produced in a variety of colors and appearances. Normally they are produced with a split face, but can be produced in straight or radius face configurations. Color and anti-graffiti treatments can be implemented.

The minimum constructible radius on curves is reported as 55 in. (1400 mm) for the Mesa facing units.

Available blocks, joint details, corner elements, etc. are enclosed in Appendix C.

Leveling Pad

A cast-in-place concrete pad is indicated in Demo 82, rather than the alternative gravel pad permitted by Tensar. Granular leveling pads may not provide the horizontal control required to preclude uneven horizontal joints.

Wall Drainage

Drainage at the face may be provided by a 1 ft. (350 mm) wide drainage column placed behind the facing unit for the entire height. The required gradation of the drainage column is detailed in the specifications.

A geotextile may be specified as required to prevent clogging. Typically, a non-woven geotextile meeting the requirements of AASHTO M 288 is specified.

Details have been developed for chimney drains and subsurface drainage layers as required and for surface water collections.

An alternate has been proposed by Tensar to substitute the drainage column with a geotextile along the back face of the blocks. This would leave a triangular area between the facing blocks unfilled and spanned by the geotextile. This alternate detail could provide satisfactory performance if the geotextile is placed in such a manner as to essentially follow the contours of the facing blocks rather than spanning between blocks. Alternatively, the unfilled areas could be filled with the specified drainage column fill. The long-term implications of adopting the detail as presently recommended by Tensar are unknown as long-term performance data are not available.

Typical details for the Tensar prepared alternative are shown in Appendix C.

Barriers and Copings

Cap blocks have been developed and are detailed in Appendix C. Tensar recommends that cap blocks be secured to the upper facing block using a synthetic resin and rubber adhesive. However, the longevity of this application has not been demonstrated. Therefore, it is preferable to mortar this connection. Cast-in-place concrete copings may be used in lieu of cap blocks. Where used, cast-in-place concrete copings must be constructed with closely spaced joints ($\pm$ 10 ft. [3 m]) to ensure facing flexibility.

Pedestrian and vehicular barrier and fence details have been developed and are shown in Appendix C.

A "Jersey" type highway traffic barrier detail using an overturning leg for stability has been proposed and is shown in Appendix C. Although feasible in principle, its performance under the AASHTO impact traffic loads has not been demonstrated.

Obstruction Avoidance Details

The submittals demonstrate an understanding of AASHTO requirements. Major obstructions to the normal placement of the geogrid reinforcement will be individually designed with the reinforcement capacity downgraded as required and shown in the typical detail in Appendix C.

Slip Joints

A slip-joint detail has been developed for use where a Mesa wall terminates and abuts another structure. Although used on a number of projects, the detail shown in Appendix C does not represent best practice. A special slip joint that allows independent movement on either side is preferable.

Core Fill in Facing Block Annulus

Filling of the facing block annulus is not required for structural or connection capacity reasons as connection capacity was developed with a void in the annulus. Erection with void annuli requires greater care by the erection contractor as the facing blocks are more easily displaced horizontally during construction.

CHAPTER 4

Construction Specifications



Metrolink Bridge Abutment

The submittal states that the Mesa System materials and construction specifications are intended to be in general conformance with the applicable provisions of the specifications for MSE Walls, Sections 8.8, 8.9, and 8.10 from Demo 82.

Significant editorial and technical revisions to the base specifications (Section 8.8) would be necessary to produce an appropr iate specification as described below.

4.1 Description

Editorial changes to reflect the use of geosynthetic grid reinforcement and modular block facings.

4.2 Materials

- (MBW) Facing and Drainage Fill: Delete "Reinforced Concrete Facing Panels" and substitute Section 8.9, "Concrete Modular Block."
- Delete Subsections 1 through 5 under "Soil Reinforcing and Attachment Devices," and substitute the following:

(1) The geogrids shall consist of Tensar structural geogrids formed by uniaxially drawing a continuous sheet of HDPE material and shall have aperture geometry, rib, and cross-section sufficient to allow significant interlock to permit mechanical interlock with the backfill soil.

The geogrid shall be stored in conditions above -20 °F (-29 °C) and not greater than 122 °F (50 °C) and in such a manner to prevent contamination from mud, wet cement, epoxy, and like materials. Geogrid rolls may be laid flat or stood on end for storage.

(2) Subsection 3.0 from Section 8.10. Table 1 should contain the applicable information as outlined in this evaluation for UXMESA3 through UXMESA6.

(3) Subsection 4.0 from Section 8.10.

(4) Subsections 5.0 and 6.0 from Section 8.10.

(5) Connectors - Tensar HP or DOT[3] connection systems shall be manufactured from HDPE with fiberglass inclusions to the dimensions shown on the plans. The contractor shall submit a manufacturer's certification verifying the connector supplied meets the index criteria set when approved by the agency.

- Delete Joint Materials.
- Select Granular Material.

(1) Use 20 mm as maximum size, unless design has considered the higher RF_{ID} factors.
(2) Eliminate electrochemical requirements except for pH, where the limits are greater than 3 and less than 11.

4.3 Construction

- Facing units shall be placed in successive horizontal lifts in the sequence shown on the plans as backfill proceeds. The connectors shall be installed and where required, the voids in and around the units filled with tamped unit fill. All excess material shall be swept from the top of the units prior to installing the next course. Facing vertical tolerances and horizontal alignment tolerances shall not exceed 20 mm when measured with a 3 m straight edge. The completed wall shall have overall tolerances (top to bottom) not exceeding 13 mm per 3 m from the plan dimensions.

Reinforcing geogrids shall be placed with the highest strength axis normal to the wall face and connected to the facing units. Prior to the placement of any backfill, the reinforcing geogrids shall be pulled taut perpendicular to the orientation of the geogrid with enough force to remove slack. Construction equipment shall not operate directly on the geogrid.

CHAPTER 5

Quality Control/ Quality Assurance Systems



Pearl Street Freight Train Travels Within 10 Feet of Wall Face

QA/QC programs have been developed for the manufacture of all supplied materials, erector training and assistance, and design. Each was reviewed and is discussed separately below.

5.1 Geosynthetic Manufacturing

A Quality Control/Testing and Inspection Procedures for Tensar Geogrids program manual dated March 3, 1997 was submitted and identifies all Tensar required testing and acceptance limits.

It is noted that the minimum acceptable density of all Tensar geogrids is 0.941 grams per cubic centimeter (g/cm^3), which is within the range for HDPE specified in ASTM D 1248 (1999). The melt flow range of 0.06 to 0.150 g/10 min. is within the range specified for Category 5 of ASTM D 1248 (1999).

Manufacturer certification of product delivered is based on meeting all of the requirements and within the tolerances specified in this document and summarized in the production specification and process control data sheets enclosed in Appendix D.

5.2 Connector Manufacturing

A Quality Control/Testing and Inspection Procedure for resin manufacture has been submitted by Aclo Compounders, Inc., an ISO 9002 certified producer, indicating acceptance limits and procedures. Resin is a HDPE to which approximately 30 percent glass fiber, carbon black, polypropylene, and anti-oxidants are added. Data sheets indicating polymer composition and properties are included in Appendix D.

The connectors are fabricated by Weber Plastics Company, Ltd., Ontario, Canada. The furnished quality control program for the finished product indicates that only dimensional and weight check testing are performed for acceptance. Strength index tests for acceptance are made by Tensar in Atlanta, Georgia for each new batch of resin used in manufacture. The submitted Quality Control and Inspection Procedures manuals by Weber Plastics have been prepared in accordance with ISO standards.

5.3 Modular Block Units (MBW)

A Quality Assurance Manual for the Mesa Segmental Wall System dated May 1999 has been developed and submitted. The QA program places responsibility on the producer to adhere to the applicable provisions and to certify conformance by marking each pallet of product "Approved." Tensar, in conjunction with the Mesa block manufacturer, is responsible for submission of the QC data to the owner.

5.4 Construction and Quality Control Manual (Erection Manual)

An erection manual was submitted and reviewed. It details the erection procedures for the system, materials supplied by Tensar, and materials supplied by the erection contractor. This manual is generally in accordance with industry practice, although it lacks documentation requirements. It is suggested that the manual be clarified to indicate that each course of facing blocks must be backfilled prior to erecting the next course.

The manual is enclosed in Appendix D.

5.5 Design QA/QC

System design for internal stability only (size, length, and spacing of geogrids) is the responsibility of Tensar. Quality control is the responsibility of the Manager of the Tensar Engineering Group, who is a registered Professional Engineer. All calculations are checked by an engineer other than the designer and verified by initialing, as are plan sheets. A registered Professional Engineer seals all drawings and calculations.

5.6 Warranties and Insurance

Tensar assumes responsibility for the internal stability of the system. The owner is responsible for external stability defined as sufficient bearing capacity to support the system and global stability of the construction.

Tensar submitted the following evidence of current insurance in force:

(i) Professional Liability Errors and Omissions

Amount:	$3,000,000. Aggregate Policy
Deductible:	$100,000.
Basis:	Claims Made
Insurer:	Admiral Insurance Company
Effective Dates:	November 1999 - November 2000
Renewal:	Renews annually

(ii) Product Liability

Amount:	$2,000,000. Aggregate Policy within General Liability plus $15,000,000. Aggregate "Umbrella" Policy within Excess Liability Policy and $10,000,000 Excess Liability Policy
Basis:	Occurence
Insurer:	Fireman's Fund Insurance Co., Union National Fire Insurance & Federal Insurance Co.
Effective Dates:	November 1999 - November 2000
Renewal:	Renews annually

CHAPTER 6

Performance Review



Founders/Meadows Built on Footings Supported by Geogrid Reinforced Abutments

Due to its recent introduction, no long-term performance data is available for the total system. Limited longer-term performance data for the HDPE geogrids used for soil reinforcement is available and was reviewed. The Tanque Verde project in Arizona was extensively studied; instrumentation data and geogrid retrieval studies indicated excellent performance after 10 years.

One project reportedly experienced longitudinal differential settlements on the order of 1/25 with ensuing spalling, joint separation, and openings.

Vertical offset data from one project with walls up to 20 ft. (6 m) in height was submitted to demonstrate the ability to construct the system to within project overall vertical tolerances.

6.1 Costs

Limited cost data for 5 bid projects are enclosed in Appendix E. It indicates in-place costs varying from $13/ft^2 to $18/ft^2 ($141 m^2 to $198 m^2) installed, excluding backfill.

Project information and contact personnel for the bid projects are also enclosed in Appendix E.

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Appendix 1.1.2.1

MESA Standard/Typical Details

MESA® MSE Retaining Wall System

Back to main document

Tensar®
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MESA®
RETAINING WALL SYSTEM

MSE STANDARD DETAILS
PREPARED FOR

MESA RETAINING WALL SYSTEM

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A Division of CMC
Tensar International Corporation
2500 Northwinds Parkway | Suite 500
Alpharetta, Georgia 30009 | 770-344-2090

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MESA®
RETAINING WALL SYSTEM

PROJECT NAME AND LOCATION

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OWNER

OWNER PROJECT No.

CLIENT

IDEA

TIC PROJECT No. ----

DRAWN BY: J. PERALTA/ O. MARTINEZ

DESIGNED BY: ----

CHECKED BY : V. JUAREZ/ W. LIEW

ENGINEER OF RECORD (MSE STRUCTURE ONLY):

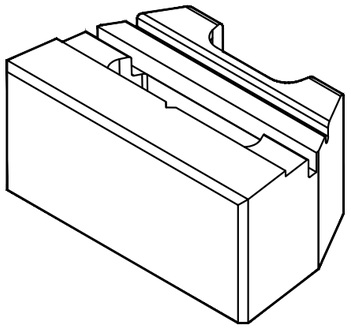
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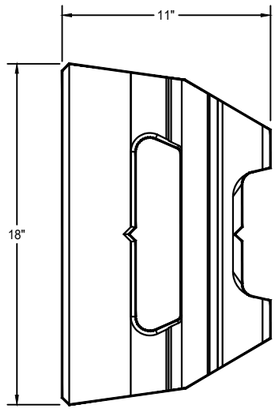
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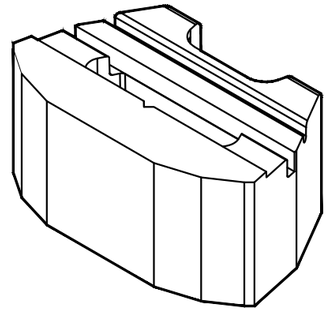
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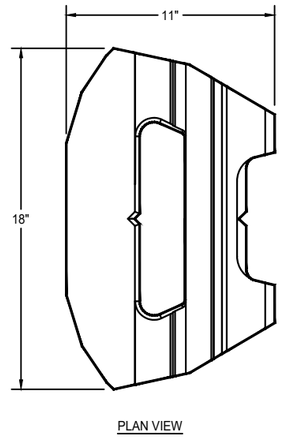
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SIDE VIEW

MESA STANDARD UNIT (STRAIGHT SPLIT FACE)
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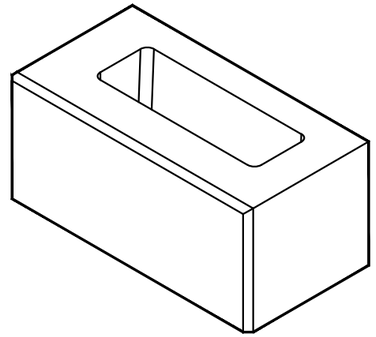
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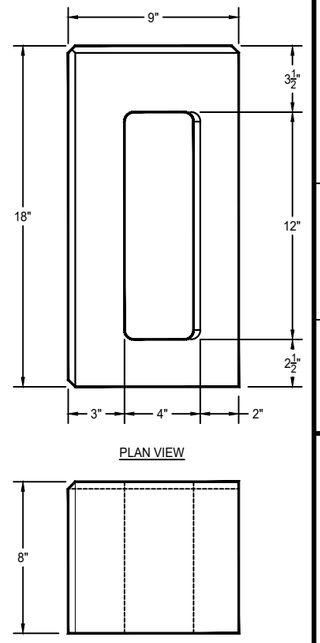
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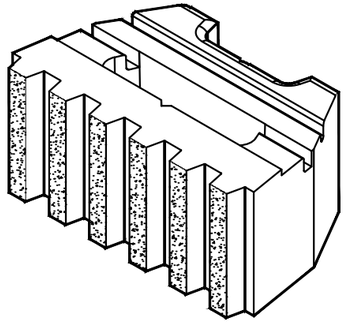
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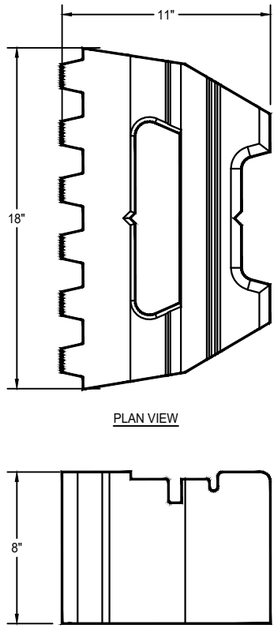
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SIDE VIEW

CORNER UNIT (STRAIGHT SPLIT FACE)
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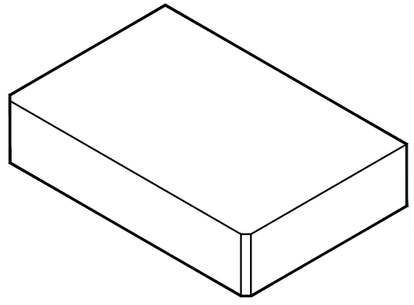
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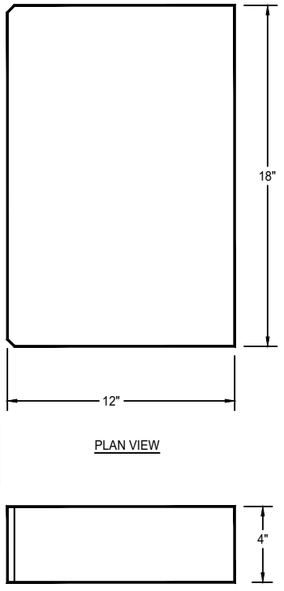
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MESA FLUTED UNIT
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ISOMETRIC VIEW

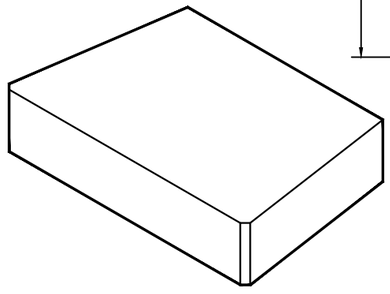


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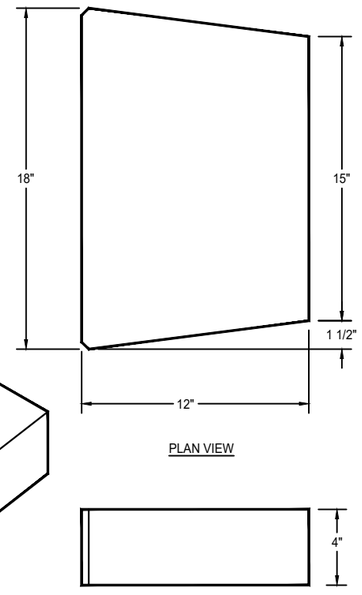
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NOTE: DIMENSIONS VARY BASED ON PRODUCT AVAILABILITY

CAP UNIT
NOT TO SCALE



ISOMETRIC VIEW



PLAN VIEW

SIDE VIEW

NOTE: DIMENSIONS VARY BASED ON PRODUCT AVAILABILITY

CAP UNIT
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MESA® RETAINING WALL SYSTEM

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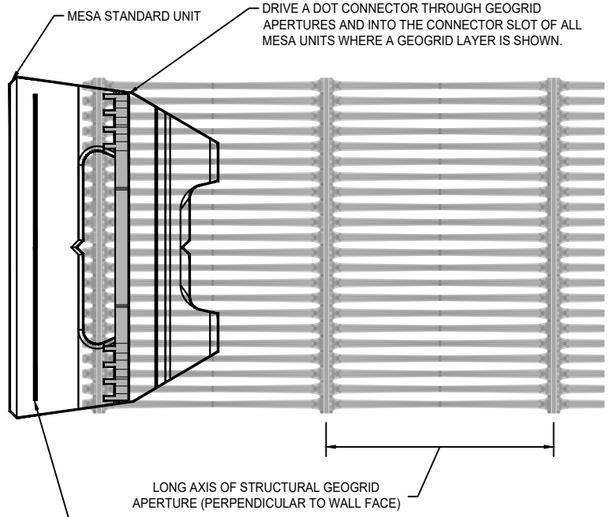
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DESIGNED BY: ----
CHECKED BY : V. JUAREZ/ W. LIEW
ENGINEER OF RECORD (MSE STRUCTURE ONLY):
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REVISION / ISSUE

SHEET TITLE

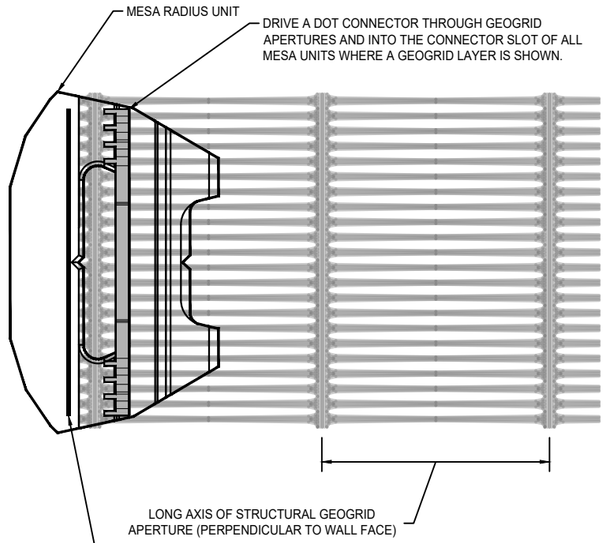
STANDARD DETAIL (1 OF 14)



PLAN VIEW

GEOGRID ORIENTATION (DOT CONNECTOR)

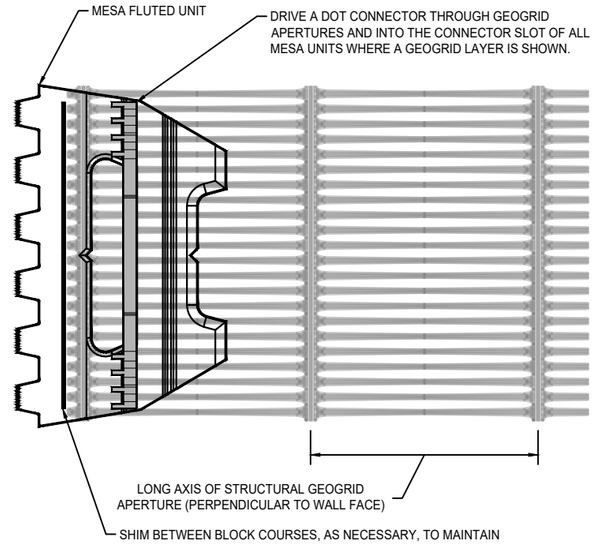
NOT TO SCALE



PLAN VIEW

GEOGRID ORIENTATION (DOT CONNECTOR)

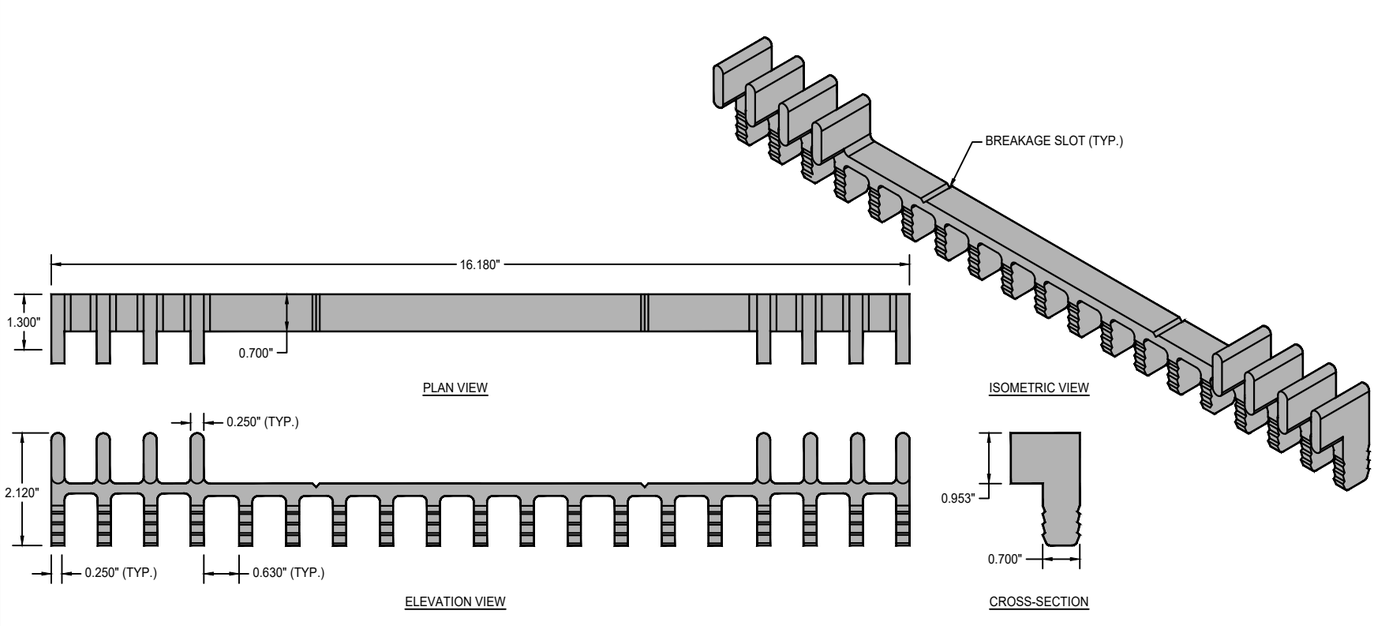
NOT TO SCALE



PLAN VIEW

GEOGRID ORIENTATION

NOT TO SCALE



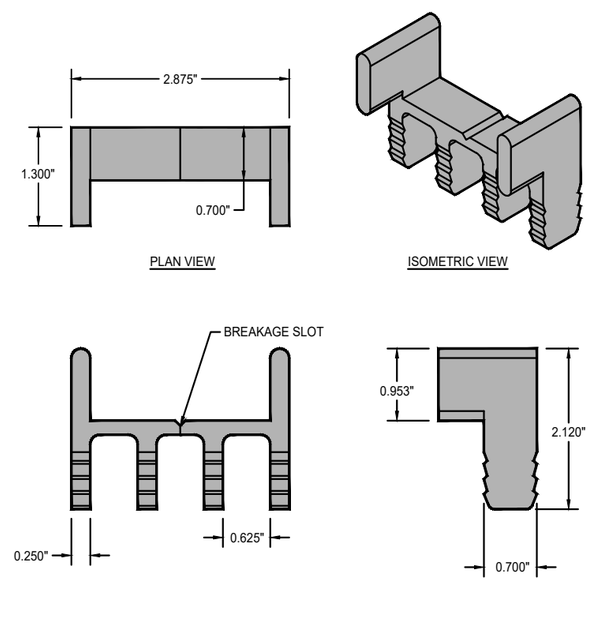
PLAN VIEW | ISOMETRIC VIEW | ELEVATION VIEW | CROSS-SECTION

NOTE:

DOT CONNECTOR CAN BE BROKEN AT BREAKAGE SLOTS TO FACILITATE INSTALLATION.

MESA DOT CONNECTOR

NOT TO SCALE



PLAN VIEW | ISOMETRIC VIEW | ELEVATION VIEW | SIDE VIEW

STANDARD CONNECTOR

NOT TO SCALE

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Tensar International Corporation
2500 Northwinds Parkway | Suite 500
Alpharetta, Georgia 30009 | 770-344-2090

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MESA® RETAINING WALL SYSTEM

PROJECT NAME AND LOCATION

----, ----

OWNER

OWNER PROJECT No.

CLIENT

IDEA

TIC PROJECT No. ----

DRAWN BY: J. PERALTA/ O. MARTINEZ

DESIGNED BY: ----

CHECKED BY : V. JUAREZ/ W. LIEW

ENGINEER OF RECORD (MSE STRUCTURE ONLY):

---- P.E. NO.

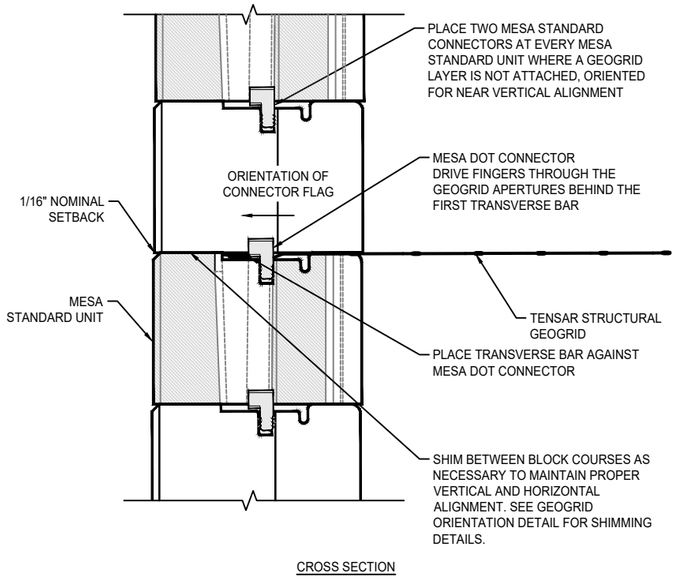
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0	09/19/2025	ISSUED FOR REVIEW	JP

REVISION / ISSUE

SHEET TITLE

STANDARD DETAIL (2 OF 14)

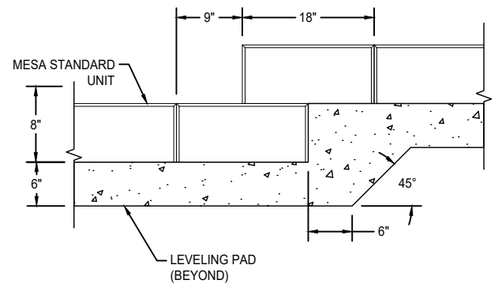
Plotted on: September 30, 2025
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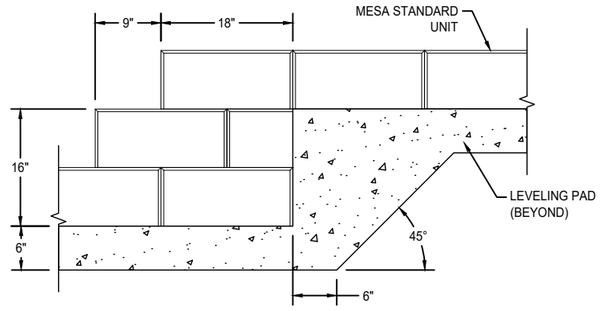
CROSS SECTION

GEOGRID DOT CONNECTION DETAIL (NEAR-VERTICAL 0.5°)

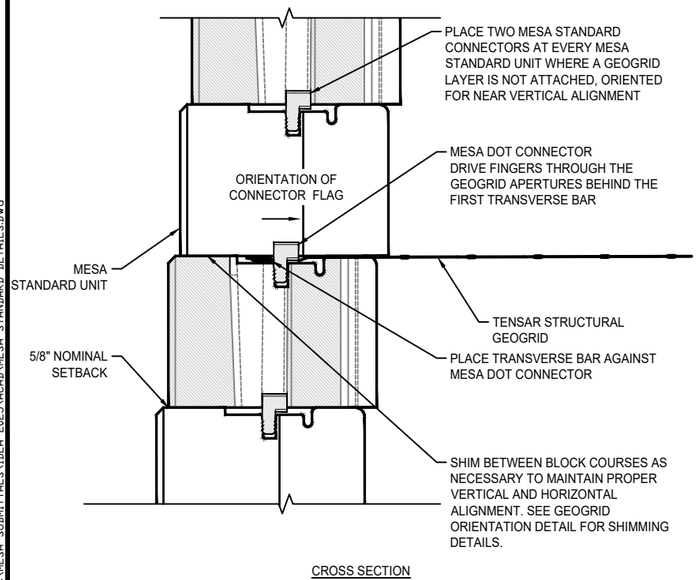
NOT TO SCALE



CASE 1: ONE BLOCK STEP (ELEVATION)



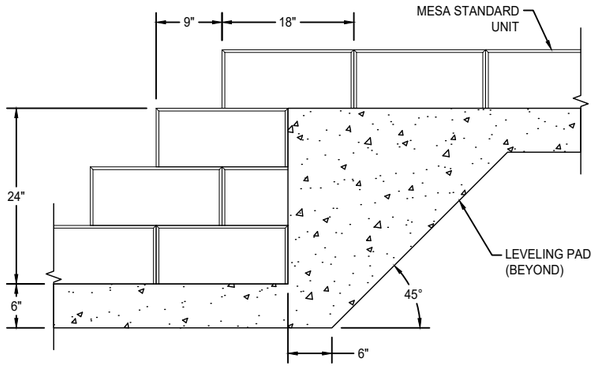
CASE 2: TWO BLOCK STEP (ELEVATION)



CROSS SECTION

GEOGRID DOT CONNECTION DETAIL (BATTERED 4.5°)

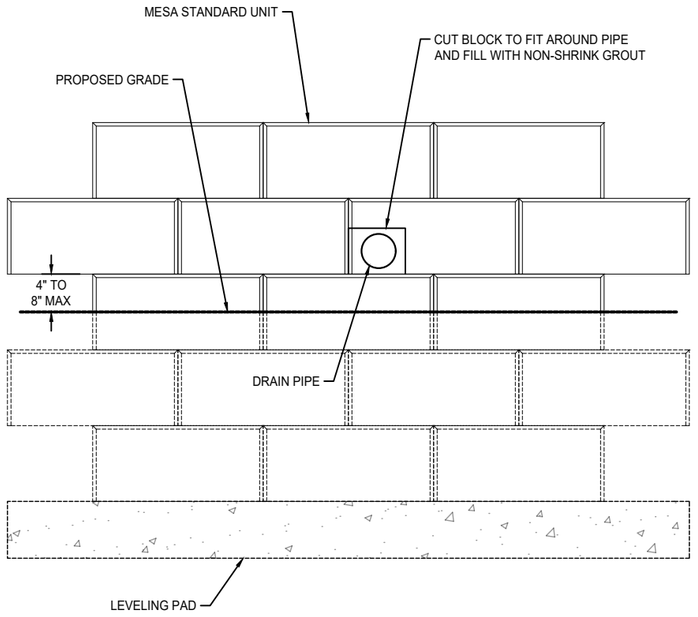
NOT TO SCALE



CASE 3: THREE BLOCK STEP (ELEVATION)

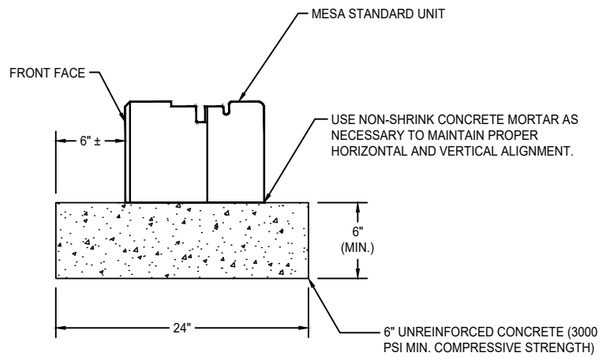
1, 2 & 3 BLOCK STEP DETAIL

NOT TO SCALE



DRAIN PIPE OUTLET DETAIL

NOT TO SCALE



CONCRETE LEVELING PAD DETAIL

NOT TO SCALE

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Tensar International Corporation
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MESA® RETAINING WALL SYSTEM

PROJECT NAME AND LOCATION

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OWNER

OWNER PROJECT No.

CLIENT

IDEA

TIC PROJECT No. ----

DRAWN BY: J. PERALTA/ O. MARTINEZ

DESIGNED BY: ----

CHECKED BY : V. JUAREZ/ W. LIEW

ENGINEER OF RECORD (MSE STRUCTURE ONLY):

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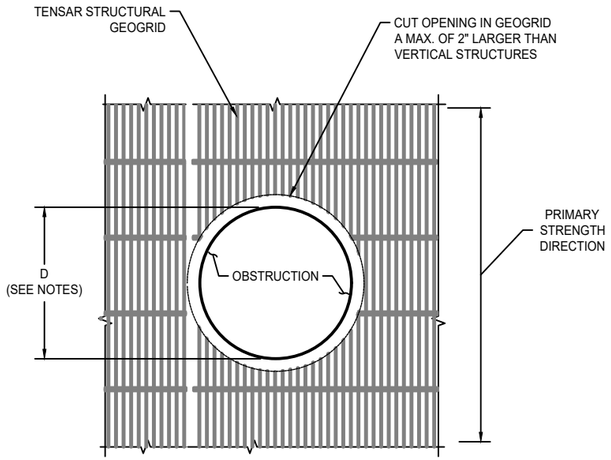
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SHEET TITLE

STANDARD DETAIL (3 OF 14)

SHEET 4 OF 15

Plotted on: September 30, 2025
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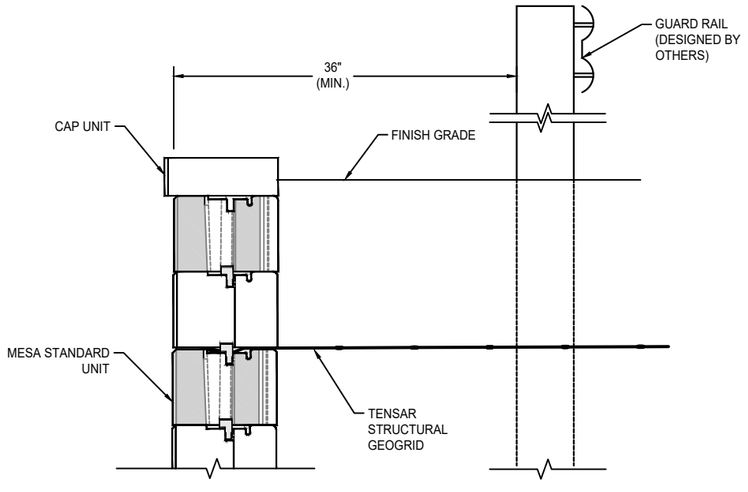
PLAN VIEW

NOTES:

1. ADDITIONAL PROVISIONS SHALL BE MADE WHEN DIAMETER EXCEEDS 24".
2. ADDITIONAL PROVISIONS SHALL BE MADE IF REGULARLY SPACED OBSTRUCTIONS ARE PLACED CLOSER THAN (10 * D)

GEOGRID PENETRATION DETAIL-PLAN VIEW

NOT TO SCALE



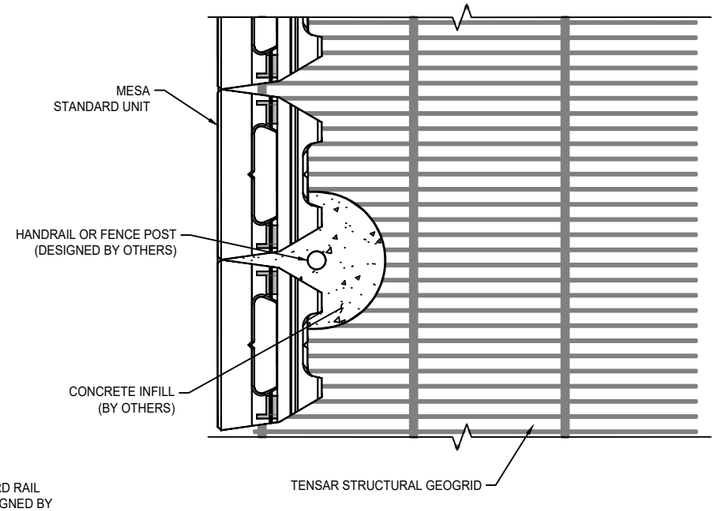
CROSS SECTION

NOTES:

1. PLACE AND COMPACT BACKFILL TO FINISH GRADE.
2. AUGER OR DRIVE POST THROUGH TENSAR GEOGRID, AS REQUIRED, TO SPECIFIED DEPTH.
3. INSTALL POST AND FILL HOLE WITH 2000 PSI (MIN) CONCRETE, OR IN ACCORDANCE WITH PROJECT SPECIFICATIONS, WHICHEVER IS MORE STRINGENT.

TOP OF WALL SECTION & GUARD RAIL DETAIL

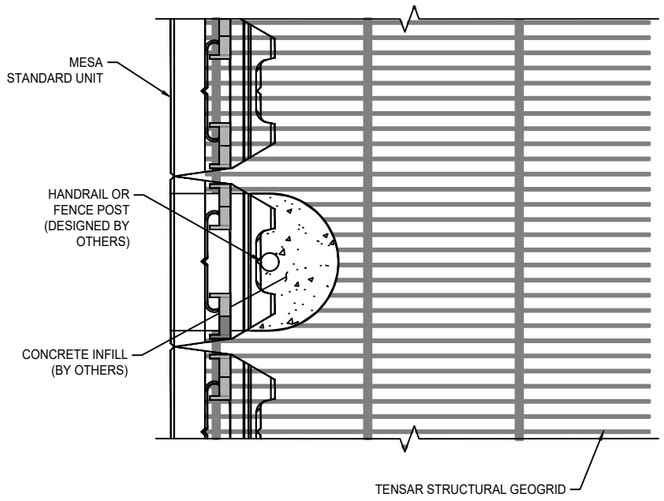
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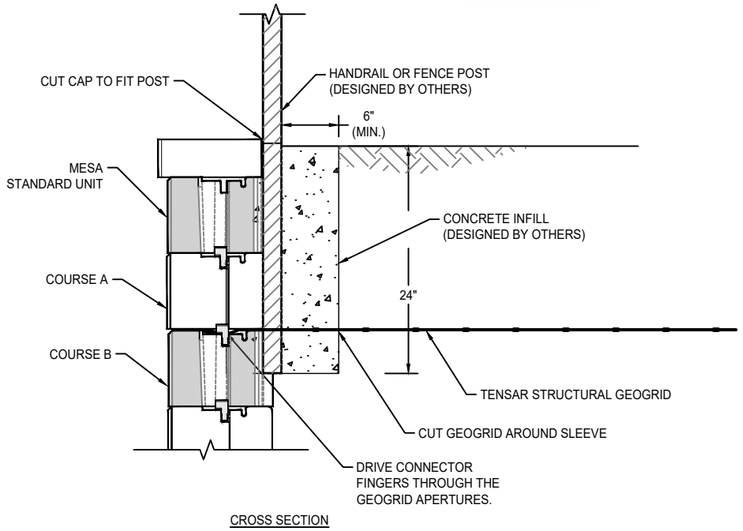
PLAN VIEW - COURSE A

NOTES:

1. PLACE AND COMPACT FILL SURROUNDING THE SLEEVE PER PROJECT SPECIFICATIONS.
2. PLACE TOP LAYER OF TENSAR STRUCTURAL GEOGRID AND REMAINING MESA STANDARD UNITS ABOVE IT.
3. CUT TENSAR STRUCTURAL GEOGRID AND THEN SET HANDRAIL OR FENCE POST.
4. FORM AND POUR CONCRETE INFILL AT TAIL OF MESA STANDARD UNITS.



PLAN VIEW - COURSE B



CROSS SECTION

HANDRAIL OR FENCE POST ON TOP OF WALL

NOT TO SCALE

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MESA® RETAINING WALL SYSTEM

PROJECT NAME AND LOCATION

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OWNER

OWNER PROJECT No.

CLIENT

IDEA

TIC PROJECT No. ----

DRAWN BY: J. PERALTA/ O. MARTINEZ

DESIGNED BY: ----

CHECKED BY : V. JUAREZ/ W. LIEW

ENGINEER OF RECORD (MSE STRUCTURE ONLY):

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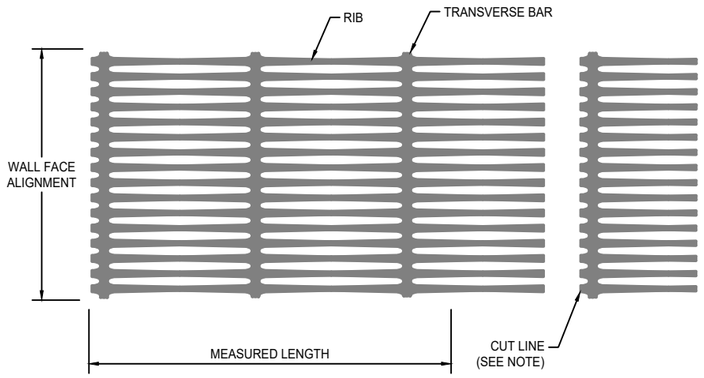
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REVISION / ISSUE

SHEET TITLE

STANDARD DETAIL (4 OF 14)

Plotted on: September 30, 2025
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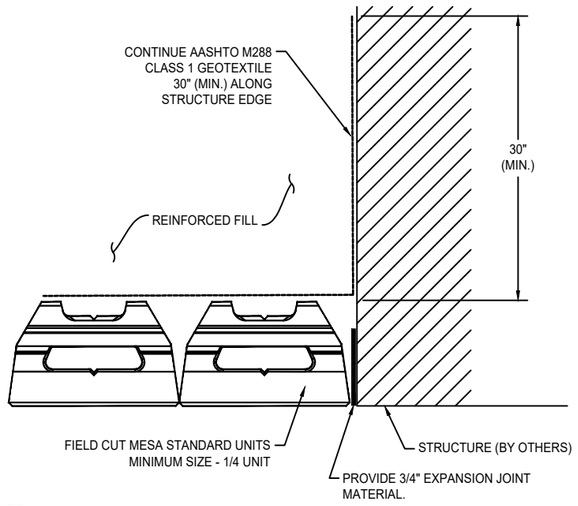
PLAN VIEW

NOTE:

CUT TENSAR GEOGRID AT NEAREST TRANSVERSE BAR BEYOND THE MEASURED LENGTH.

MEASURED LENGTH CUT DETAIL

NOT TO SCALE

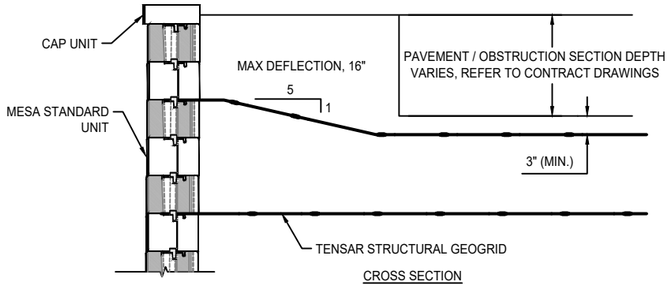


NOTES:

1. GEOGRID AND CONNECTORS NOT SHOWN FOR CLARITY.
2. FIELD CUT MESA STANDARD UNIT (MIN. 1/4 UNIT) FOR RUNNING BOND.
3. REFER TO TYPICAL CROSS-SECTION FOR FILL AND DRAINAGE REQUIREMENT AT BACK OF WALL.
4. ATTACH GEOTEXTILE TO STRUCTURE WITH CONSTRUCTION ADHESIVE.

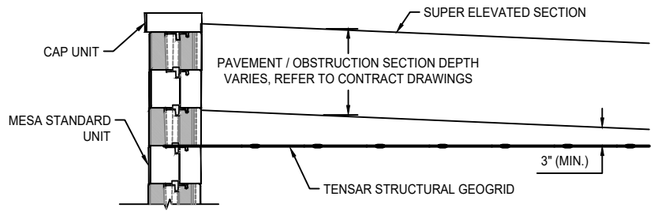
MESA WALL TRANSITION AT STRUCTURE

NOT TO SCALE



CROSS SECTION

HORIZONTAL PAVEMENT STRUCTURE



CROSS SECTION

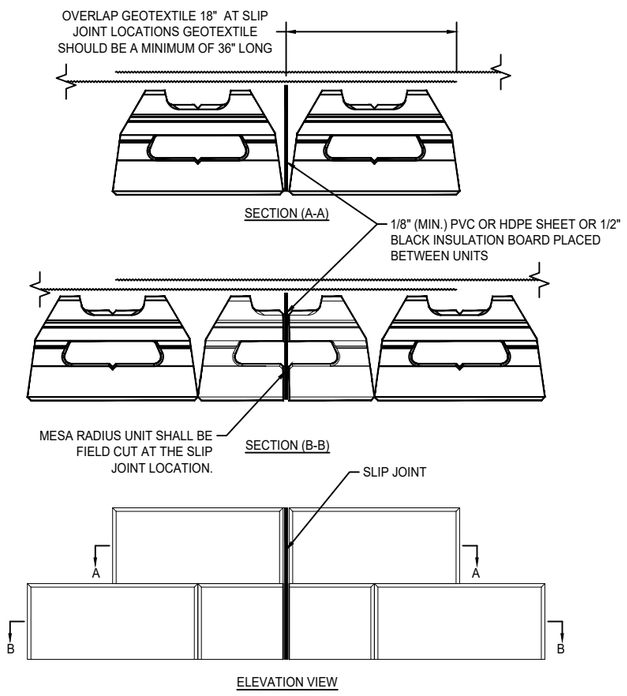
SUPER ELEVATED PAVEMENT / OBSTRUCTION

NOTE:

CONTRACTOR IS RESPONSIBLE TO COORDINATE THE PLACEMENT OF THE GEOGRID TO AVOID CONFLICT WITH THE CONTRACT PAVEMENT / OBSTRUCTION SECTION. GEOGRID MUST BE SEPARATED FROM THE PAVEMENT / OBSTRUCTION SECTION BY A MINIMUM OF 3".

GEOGRID PLACEMENT AT PAVEMENT / OBSTRUCTION

NOT TO SCALE



ELEVATION VIEW

SLIP JOINT DETAIL

NOT TO SCALE

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Tensar International Corporation
2500 Northwinds Parkway | Suite 500
Alpharetta, Georgia 30009 | 770-344-2090

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MESA® RETAINING WALL SYSTEM

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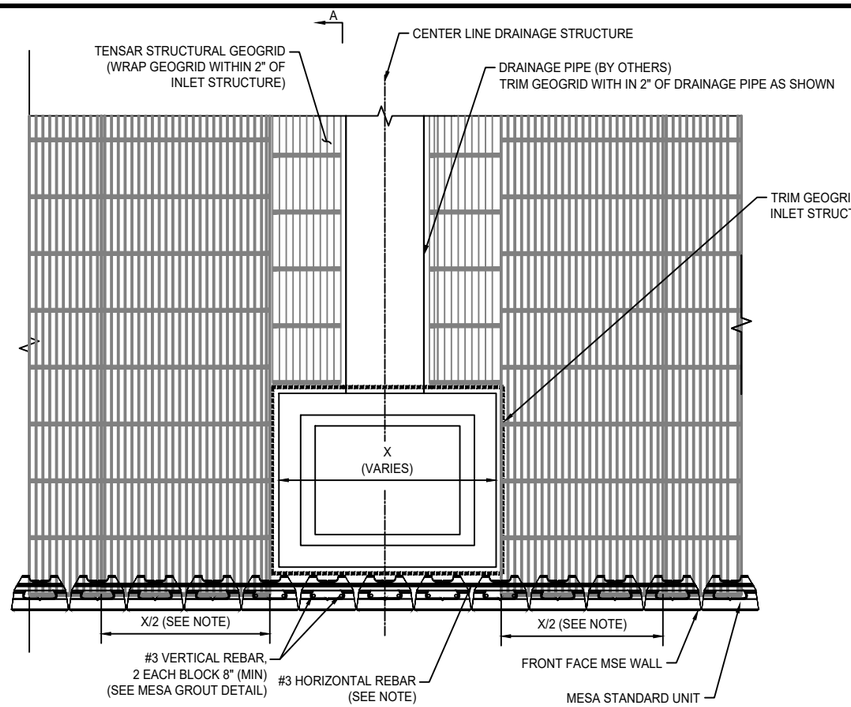
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REVISION / ISSUE

SHEET TITLE

STANDARD DETAIL (5 OF 14)

Plotted on: September 30, 2025
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NOTE:

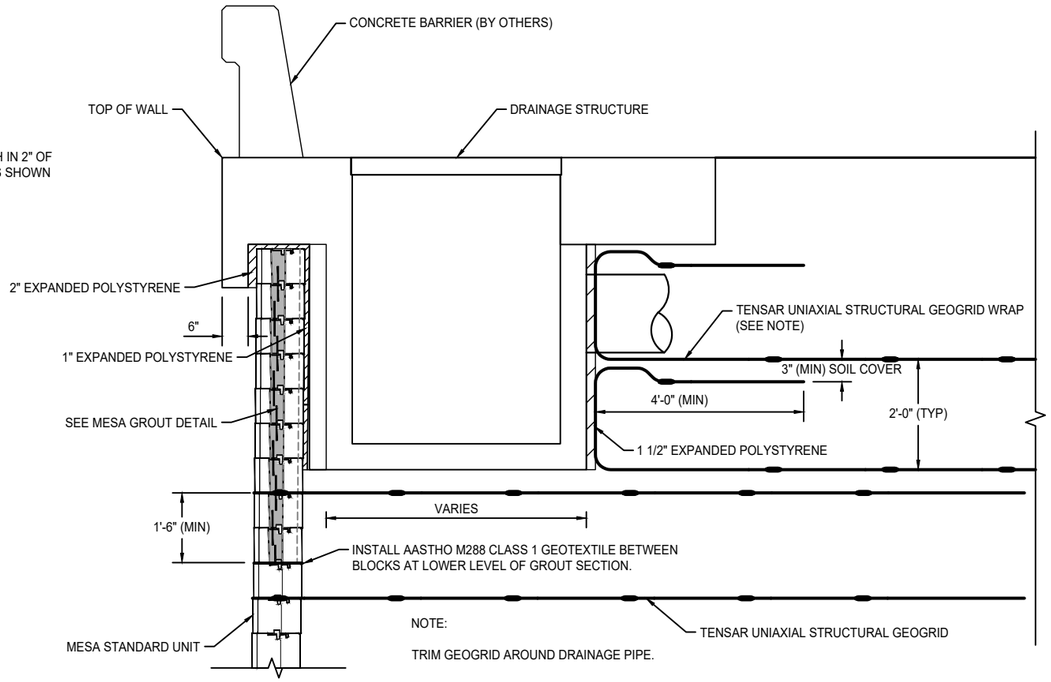
GROUT AND EXTEND ONE GEOGRID ROLL WIDTH OR AT A DISTANCE OF X/2, WHICHEVER IS GREATER ON EITHER SIDE OF CATCH BASIN (IN ALL COURSES OF MESA UNITS WITH CATCH BASIN DIRECTLY BEHIND WALL FACE).

A

PLAN VIEW

MSE WALL AT DRAINAGE INLET

NOT TO SCALE



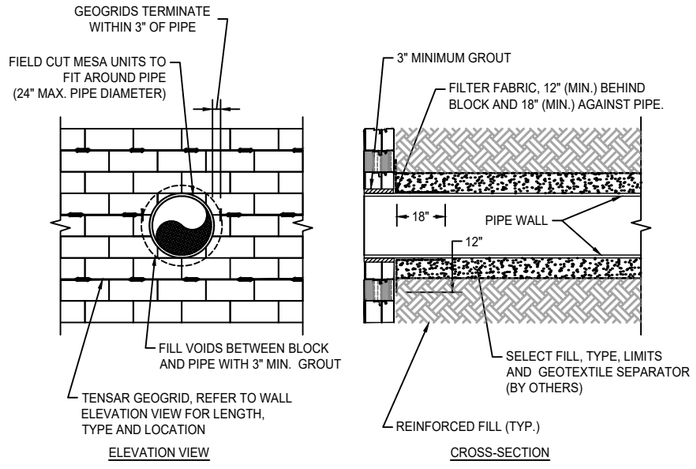
NOTE:

TRIM GEOGRID AROUND DRAINAGE PIPE.

SECTION A-A

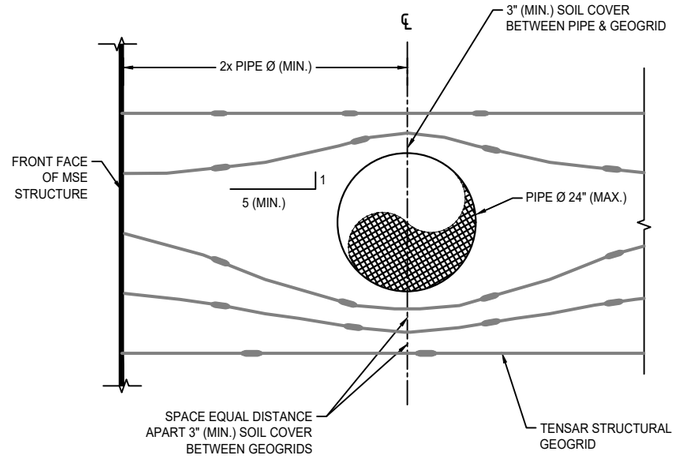
MSE WALL AT DRAINAGE INLET

NOT TO SCALE



PIPE PENETRATION DETAIL

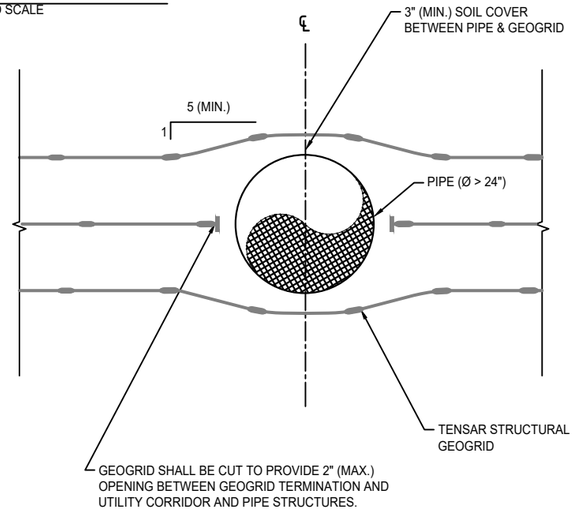
NOT TO SCALE



CROSS-SECTION

GEOGRID PLACEMENT AT PIPE PARALLEL TO MSE STRUCTURE FACE

NOT TO SCALE



ELEVATION VIEW

GEOGRID PLACEMENT AT PIPE PERPENDICULAR TO MSE STRUCTURE FACE

NOT TO SCALE

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Alpharetta, Georgia 30009 | 770-344-2090

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DESIGNED BY: ---

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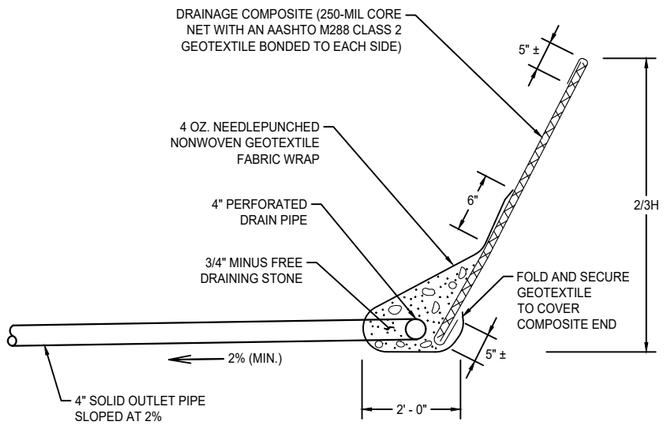
REVISION / ISSUE

SHEET TITLE

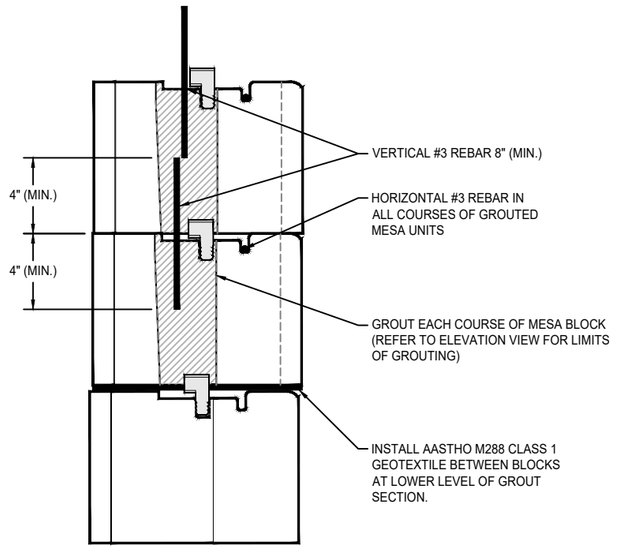
STANDARD DETAIL (6 OF 14)

SHEET 7 OF 15

Plotted on: September 30, 2025
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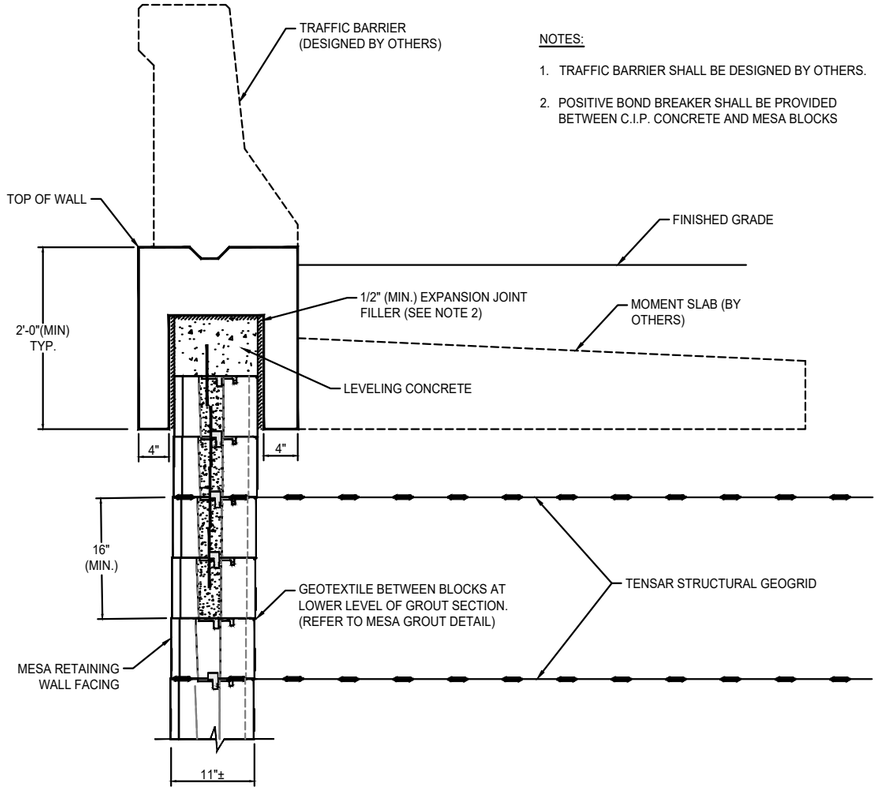


SUBDRAN DETAIL
NOT TO SCALE



CROSS SECTION

MESA GROUT DETAIL
NOT TO SCALE



TYPICAL TRAFFIC BARRIER ON TOP DETAIL
NOT TO SCALE

NOTES:

1. TRAFFIC BARRIER SHALL BE DESIGNED BY OTHERS.
2. POSITIVE BOND BREAKER SHALL BE PROVIDED BETWEEN C.I.P. CONCRETE AND MESA BLOCKS

Tensar.
A Division of CMC

Tensar International Corporation
2500 Northwinds Parkway | Suite 500
Alpharetta, Georgia 30009 | 770-344-2090

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MESA® RETAINING WALL SYSTEM

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OWNER

OWNER PROJECT No.

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DRAWN BY: J. PERALTA/ O. MARTINEZ
DESIGNED BY: ----
CHECKED BY : V. JUAREZ/ W. LIEW
ENGINEER OF RECORD (MSE STRUCTURE ONLY):
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REVISION / ISSUE

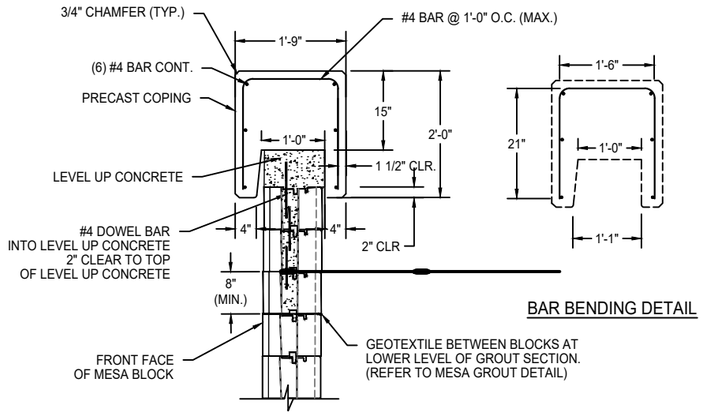
SHEET TITLE

STANDARD DETAIL (7 OF 14)

Plotted on: September 30, 2025
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NOTES:

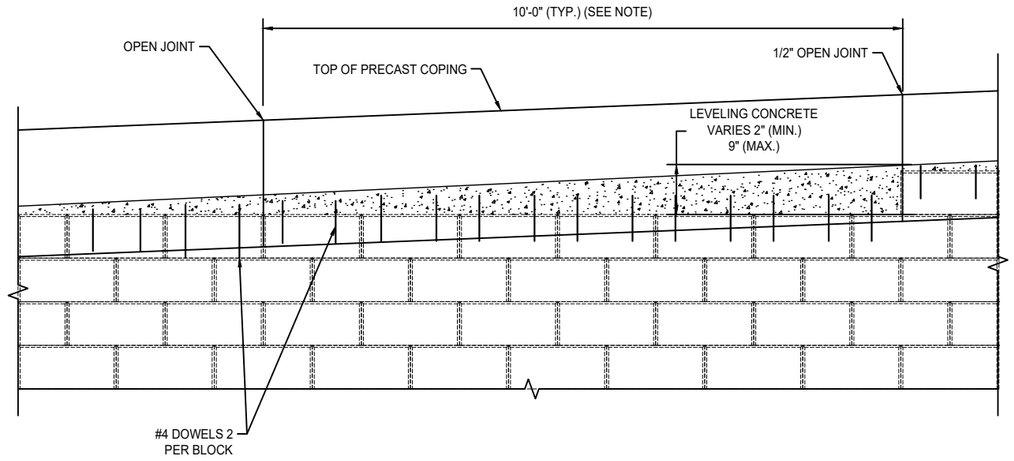
1. CONCRETE SHALL HAVE MINIMUM 28 DAY COMPRESSIVE STRENGTH OF 3000PSI.
2. OPEN JOINTS IN COPING SHALL BE AT NO MORE THAN 10'-0" INTERVALS. JOINTS SHALL BE PERPENDICULAR TO SLOPE LINE.
3. PROVIDE ALL MATERIALS AND WORKMANSHIP IN ACCORDANCE WITH STATE DOT REQUIREMENTS.
4. FIELD CUT DOWELS TO MAINTAIN COVER.



BAR BENDING DETAIL

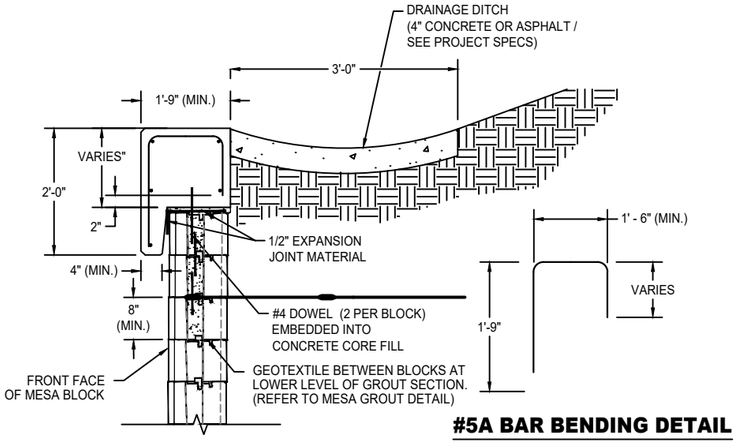
PRECAST COPING DETAIL

NOT TO SCALE



PRECAST COPING PARTIAL ELEVATION VIEW

NOT TO SCALE



#5A BAR BENDING DETAIL

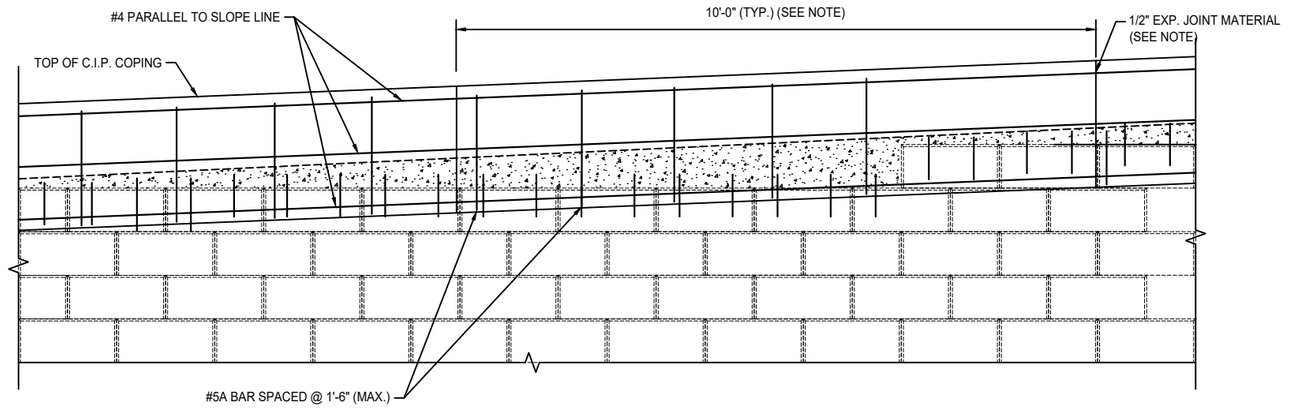
REINFORCING STEEL NOTES:

1. ALL BAR DIMENSIONS ARE OUT TO OUT.
2. LAP SPLICES FOR #4 BARS SHALL BE 20' MINIMUM.
3. ALL REINFORCING BARS SHALL BE PER ASTM-614, GRADE 60.

C.I.P COPING WITH SWALE

NOT TO SCALE

NOTE:
EXPANSION JOINTS IN C.I.P COPING SHALL BE AT NO MORE THAN 10'-0" INTERVALS. JOINTS SHALL BE PERPENDICULAR TO SLOPE LINE. REINFORCING STEEL SHALL BE STOPPED 2" SHORT OF EITHER SIDE OF THE EXPANSION JOINT.



C.I.P. COPING PARTIAL ELEVATION VIEW

NOT TO SCALE

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Tensar International Corporation
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Alpharetta, Georgia 30009 | 770-344-2090

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MESA® RETAINING WALL SYSTEM

PROJECT NAME AND LOCATION

----, ----

OWNER

OWNER PROJECT No.

CLIENT

IDEA

TIC PROJECT No. ----

DRAWN BY: J. PERALTA/ O. MARTINEZ

DESIGNED BY: ----

CHECKED BY : V. JUAREZ/ W. LIEW

ENGINEER OF RECORD (MSE STRUCTURE ONLY):

--- P.E. NO.

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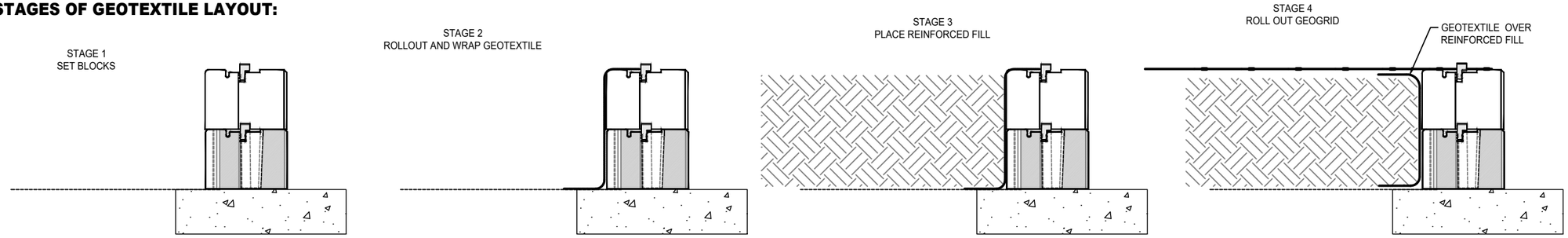
REVISION / ISSUE

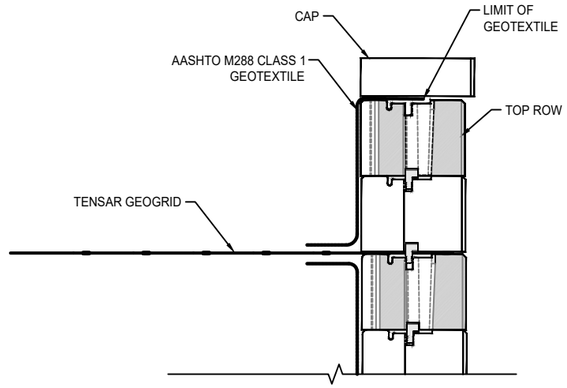
SHEET TITLE

STANDARD DETAIL (8 OF 14)

Plotted on: September 30, 2025
K:\00 SUBMITTALS\MESA SUBMITTALS\IDEA 2025\ACAD\MESA STANDARD DETAILS.DWG

STAGES OF GEOTEXTILE LAYOUT:





TYPICAL TOP SECTION
NOT TO SCALE

FABRIC AND GRAVEL FILL SPECIFICATION

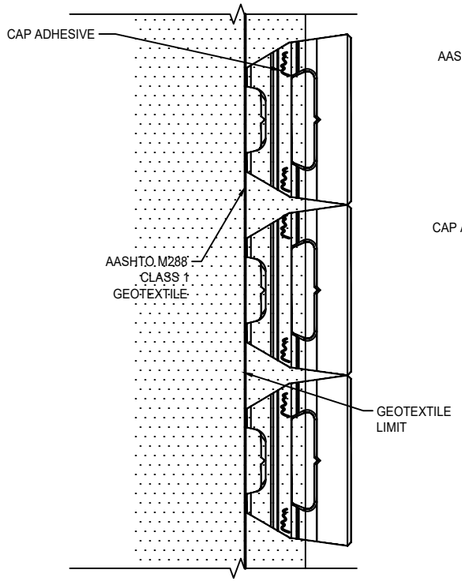
When the select fill used in the reinforced zone for the Project is considered free-draining (less than 5% passing the No. 200 sieve), no geotextile or gravel fill is required behind the Mesa facing units, unless prescribed by the Project Engineer. When the select fill contains fines between 5% and 15%, geotextile fabric is required behind the Mesa facing units; however, no gravel fill is needed, unless prescribed by the Project Engineer. Select fill with fines in excess of 15% requires the use of both geotextile fabric and gravel fill behind the Mesa facing units.

If required per the note above, the geotextile separator shall be an eight (8) oz. per square yard nonwoven needle-punched AASHTO Class 1 fabric. Unless otherwise approved by Tensar, the geotextile shall be delivered to the project site in rolls that have been factory cut to the specified widths for the installation.

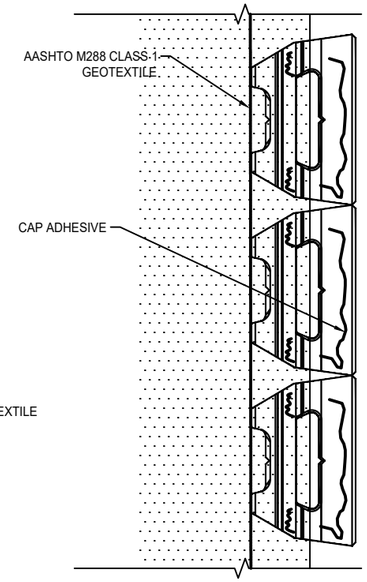
INSTALLATION PROCEDURE

1. Install each course of the Mesa facing units between the top of the last geogrid placed and the bottom the next layer of geogrid above as shown on the approved shop drawings. The facing units shall be aligned and leveled in accordance with installation guide.
2. Prior to placing the select backfill and if required, gravel fill and geotextile fabric shall be installed. The geotextile shall be placed behind the units such that a minimum of six (6) inches of material is turned into the fill at the top and the bottom. The geotextile shall then be adjusted to present a relatively smooth surface.
3. The select backfill shall then be placed and compacted in accordance with the approved shop drawings and project specifications.
4. After the select backfill has been compacted and properly graded for the installation of the next layer of geogrid reinforcement, the geotextile on the top units shall be pulled back onto the backfill.
5. Install the geogrid reinforcement and repeat the process commencing with step 1 above.
6. After the last elevation layer of primary geogrid reinforcement has been placed, install the remaining courses of Mesa facing units in accordance with the details on the approved shop drawings.
7. Prior to installation of cap unit, place a line of adhesive in the depressed area between the connector slot and the face of the unit per Detail S1.
8. Place a line of adhesive along the top of the Mesa facing units just behind the face per Detail S2.
9. Butt sides of cap units as shown in Detail S3.

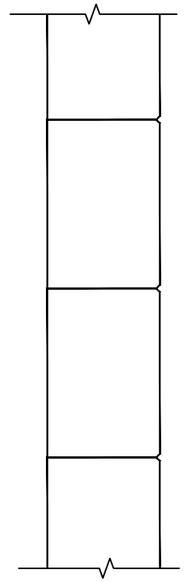
Geotextile widths required for the detail: AASHTO M288 Class 1: 36 inch



DETAIL S1: GEOTEXTILE ON TOP OF MESA UNITS
NOT TO SCALE



DETAIL S2: CAP ADHESIVE
NOT TO SCALE



DETAIL S3: CAP UNITS
NOT TO SCALE

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Tensar International Corporation
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Alpharetta, Georgia 30009 | 770-344-2090

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DRAWN BY: J. PERALTA/ O. MARTINEZ
DESIGNED BY: ---
CHECKED BY : V. JUAREZ/ W. LIEW
ENGINEER OF RECORD (MSE STRUCTURE ONLY):

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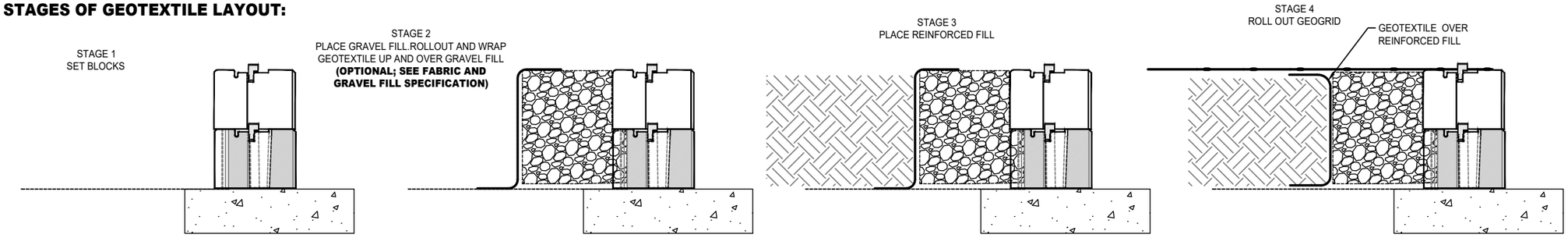
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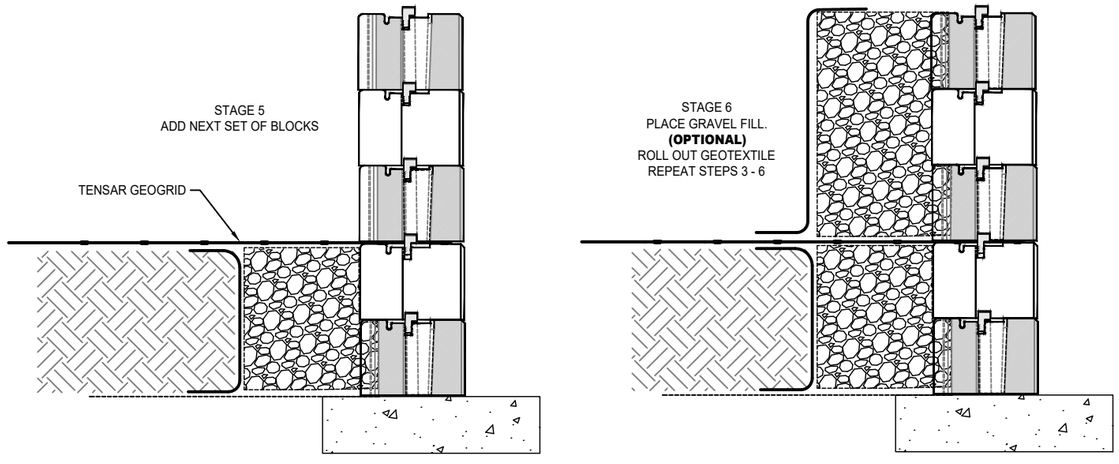
SHEET TITLE

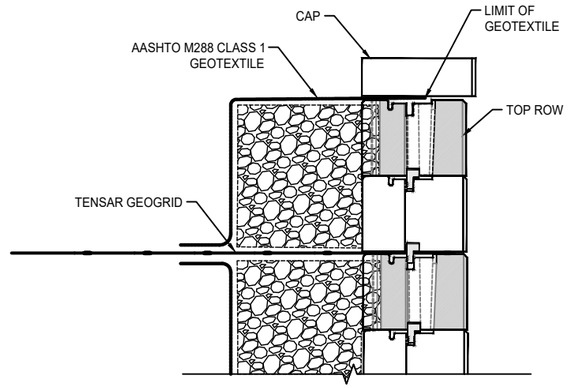
STANDARD DETAIL (9 OF 14)

Plotted on: September 30, 2025
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STAGES OF GEOTEXTILE LAYOUT:







TYPICAL TOP SECTION
NOT TO SCALE

FABRIC AND GRAVEL FILL SPECIFICATION

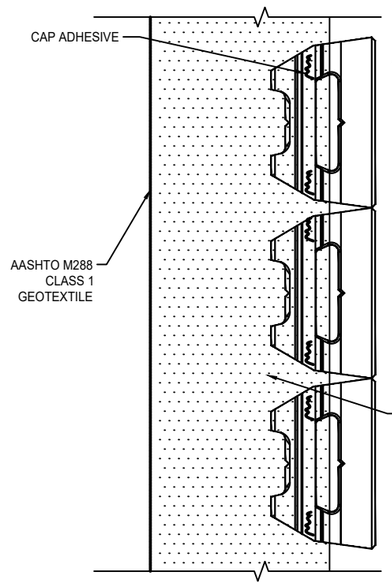
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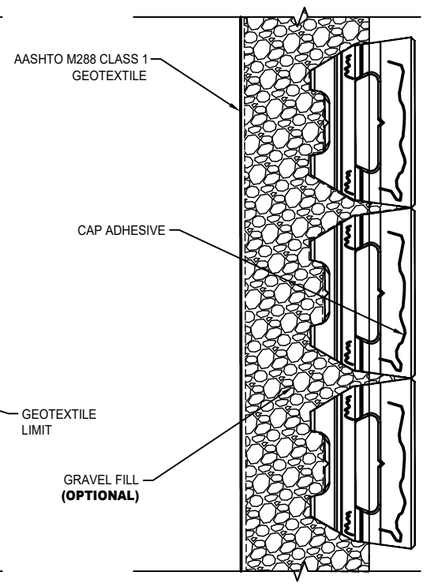
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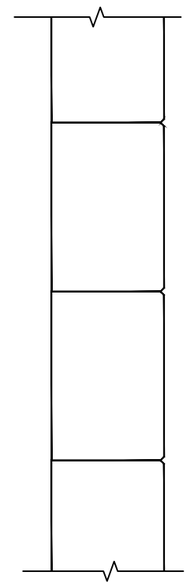
Geotextile widths required for the detail: AASHTO M288 Class 1: 36 inch



DETAIL S1: GEOTEXTILE ON TOP OF MESA UNITS
NOT TO SCALE



DETAIL S2: CAP ADHESIVE
NOT TO SCALE



DETAIL S3: CAP UNITS
NOT TO SCALE

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CLIENT

IDEA

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DESIGNED BY: ----
CHECKED BY : V. JUAREZ/ W. LIEW
ENGINEER OF RECORD (MSE STRUCTURE ONLY):
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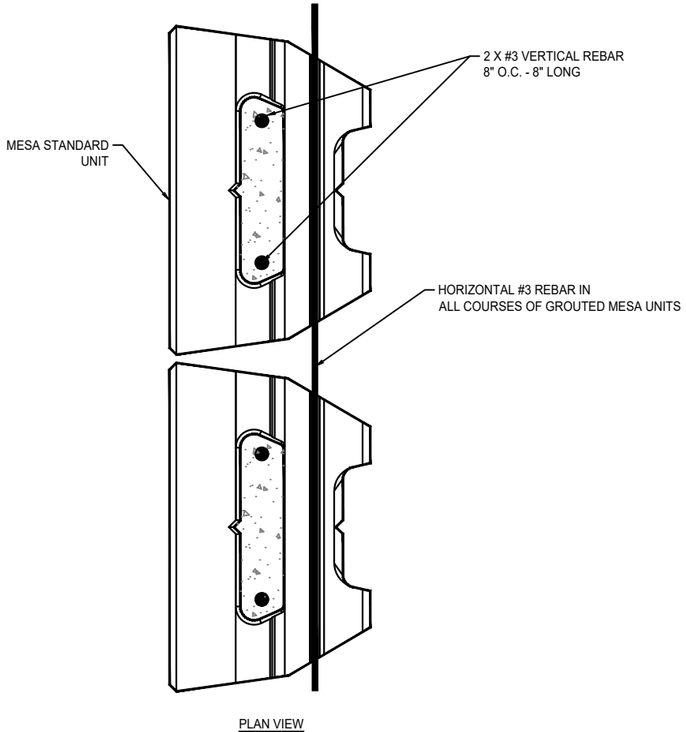
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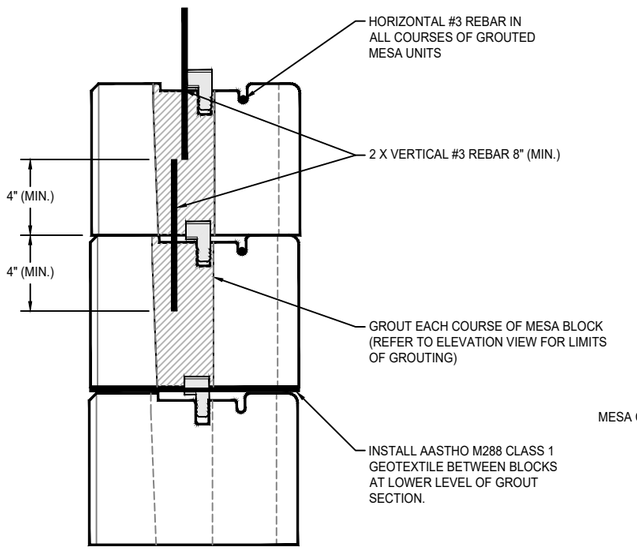
STANDARD DETAIL (10 OF 14)

SHEET 11 OF 15

Plotted on: September 30, 2025
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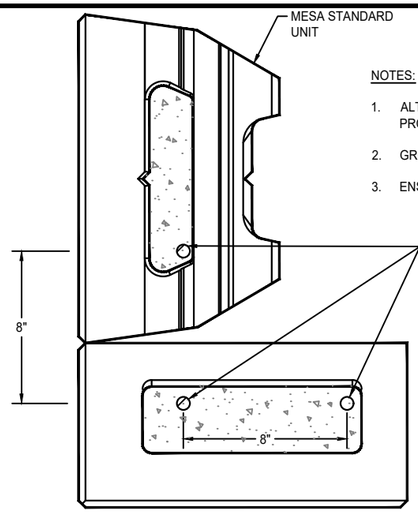
PLAN VIEW



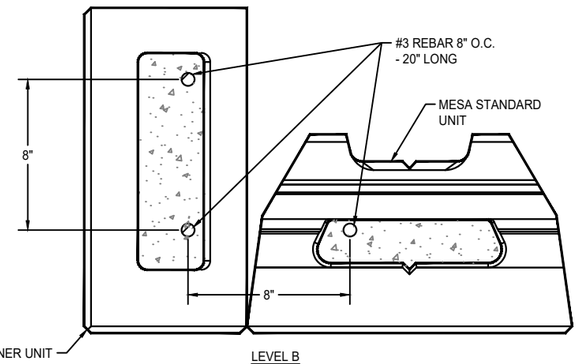
CROSS SECTION

MESA GROUT DETAIL

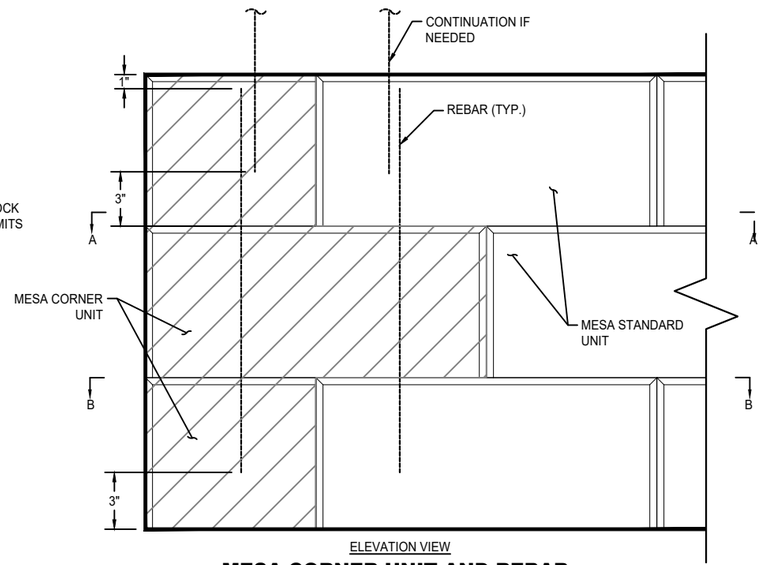
NOT TO SCALE



LEVEL A



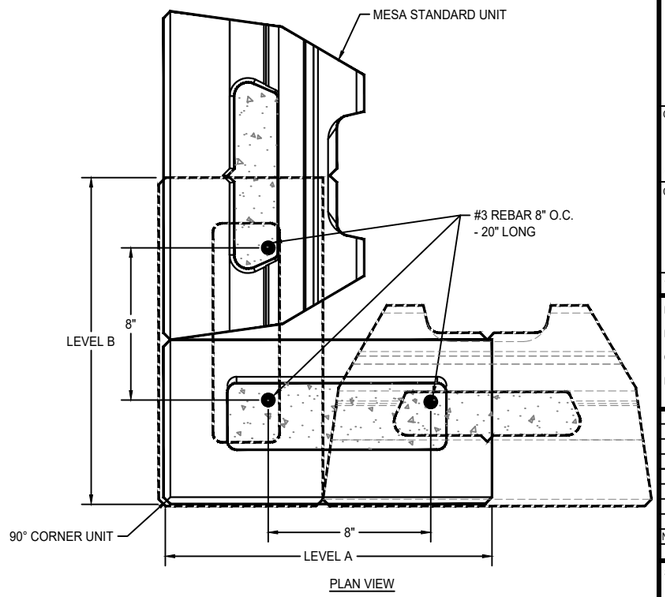
LEVEL B



ELEVATION VIEW

MESA CORNER UNIT AND REBAR

NOT TO SCALE



PLAN VIEW

MESA CORNER UNITS AND REBAR PLAN VIEW

NOT TO SCALE

NOTES:

1. ALTERNATE CORNER UNIT DIRECTION FOR PROPER RUNNING BOND.
2. GROUT CAVITY OF CORNER BLOCK.
3. ENSURE THAT SPLIT FACES ARE EXPOSED.

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MESA® RETAINING WALL SYSTEM

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DRAWN BY: J. PERALTA/ O. MARTINEZ

DESIGNED BY: ----

CHECKED BY : V. JUAREZ/ W. LIEW

ENGINEER OF RECORD (MSE STRUCTURE ONLY):

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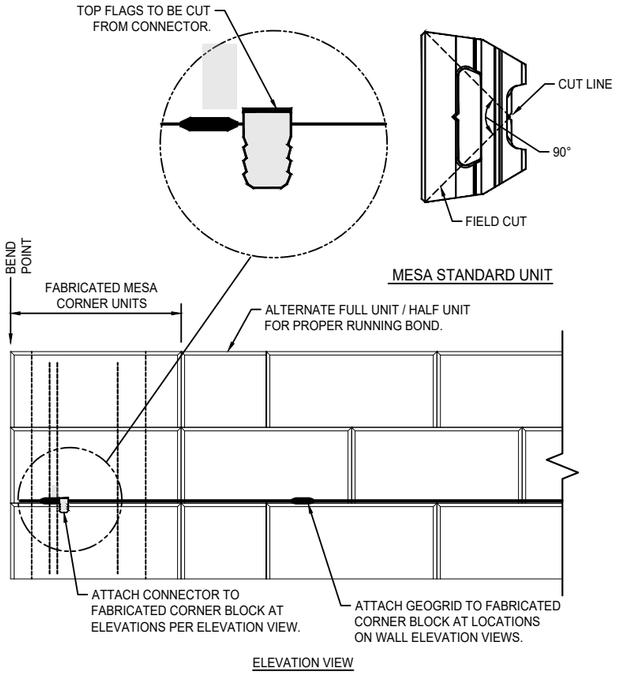
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SHEET TITLE

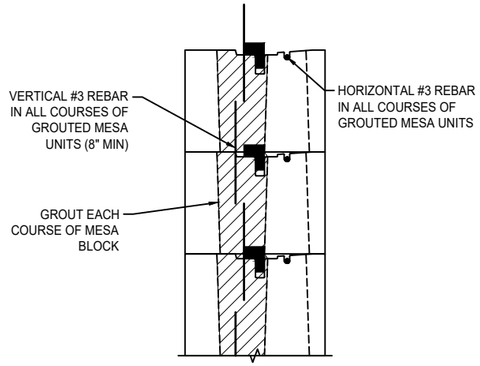
STANDARD DETAIL (11 OF 14)

Plotted on: September 30, 2025
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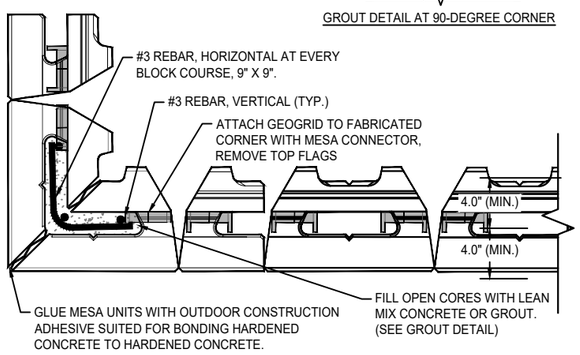


MESA STANDARD UNIT

ELEVATION VIEW



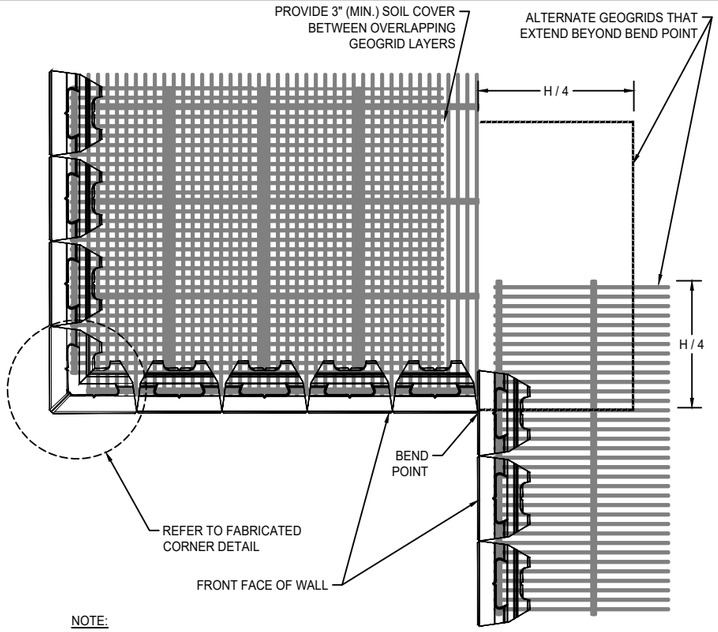
GROUT DETAIL AT 90-DEGREE CORNER



PLAN VIEW

FABRICATED MESA CORNER DETAIL (90°)

NOT TO SCALE



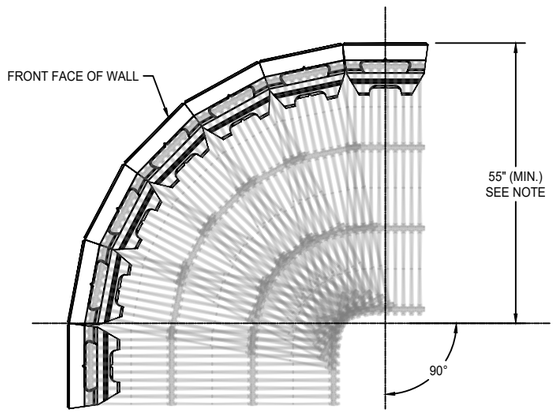
NOTE:

"H" IS DEFINED AS THE WALL HEIGHT

PLAN VIEW

MESA WALL ALONG CORNERS DETAIL

NOT TO SCALE

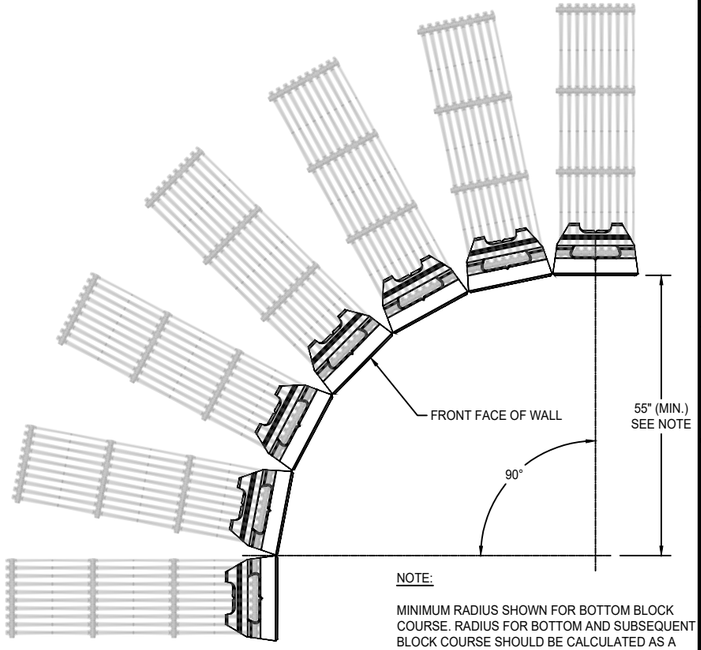


NOTE:

MINIMUM RADIUS SHOWN FOR TOP BLOCK COURSE. RADIUS FOR BOTTOM AND SUBSEQUENT BLOCK COURSE SHOULD BE CALCULATED AS A FUNCTION OF WALL HEIGHT AND BATTER.

90° OUTSIDE CURVE DETAIL

NOT TO SCALE

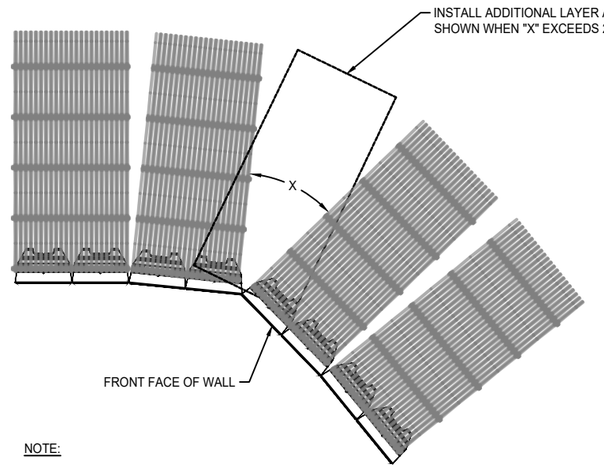


NOTE:

MINIMUM RADIUS SHOWN FOR BOTTOM BLOCK COURSE. RADIUS FOR BOTTOM AND SUBSEQUENT BLOCK COURSE SHOULD BE CALCULATED AS A FUNCTION OF WALL HEIGHT AND BATTER.

90° INSIDE CURVE DETAIL

NOT TO SCALE



NOTE:

MULTIPLE BENDS IN WALL ALIGNMENT MAY BE NESSESARY TO ACHIEVE LARGE RADIUS CURVES.

GEOGRID AT WALL BEND

NOT TO SCALE

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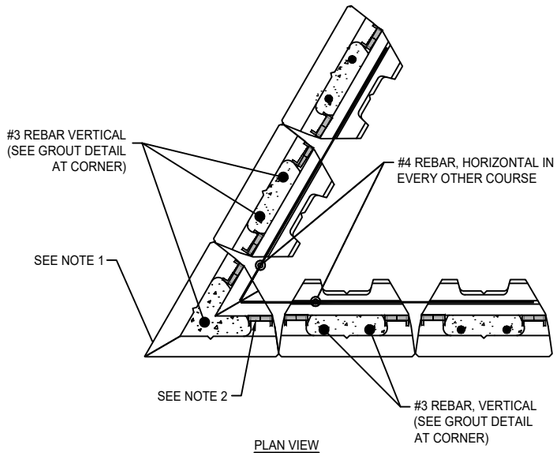
REVISION / ISSUE

SHEET TITLE

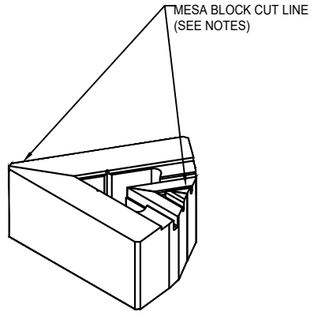
STANDARD DETAIL (12 OF 14)

SHEET 13 OF 15

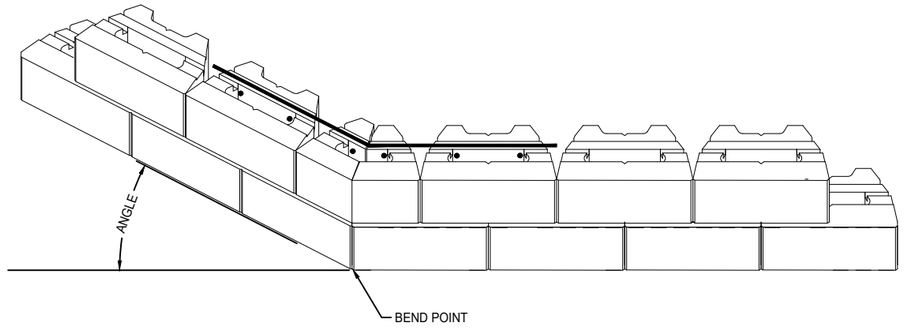
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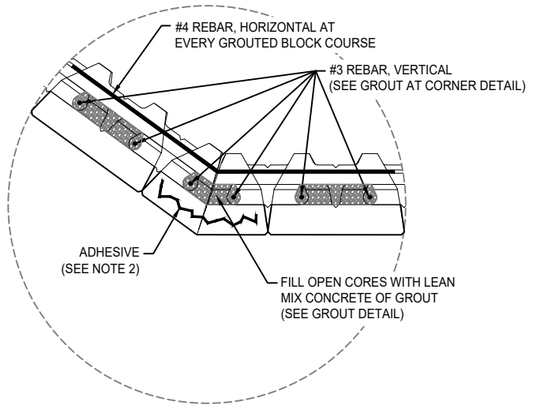
PLAN VIEW



ISOMETRIC VIEW



ISOMETRIC VIEW

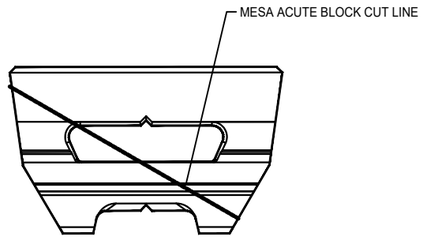


PLAN VIEW

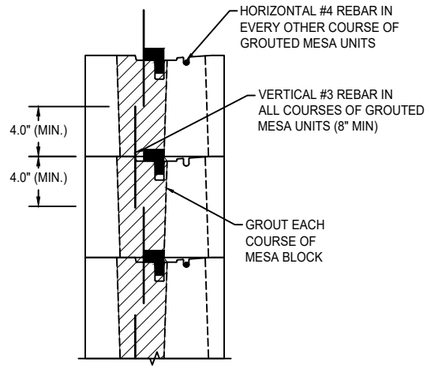
NOTES:

1. GLUE MESA UNITS WITH OUTDOOR CONSTRUCTION ADHESIVE (LEPAGE BULLDOG GRIP PL9000 OR APPROVED EQUIVALENT)

2. PLACE STANDARD CONNECTORS ON EVERY BLOCK COURSE IN THE ACUTE CORNER ZONE FOR ALIGNMENT PURPOSE.



BLOCK CUT PLAN VIEW



NOTES:

1. CUT UNITS TO SIZE FOR EACH COURSE TO MAINTAIN PROPER BATTER, ALIGNMENT AND ARCHITECTURAL PATTERN.

2. SECURE MESA UNITS WITH CONCRETE ADHESIVE OR OTHER BONDING AGENT, OR BY DRILLING REBAR HOLES IN EACH ADJOINING UNIT.

FABRICATED ACUTE CORNER DETAIL

NOT TO SCALE

GROUT AT CORNER DETAIL

NOT TO SCALE

FABRICATED OBTUSE CORNER DETAIL

NOT TO SCALE

Tensar.
A Division of CMC

Tensar International Corporation

2500 Northwinds Parkway | Suite 500
Alpharetta, Georgia 30009 | 770-344-2090

C.O.A. NO.

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MESA® RETAINING WALL SYSTEM

PROJECT NAME AND LOCATION

----, ----

OWNER

OWNER PROJECT No.

CLIENT

IDEA

TIC PROJECT No. ----

DRAWN BY: J. PERALTA/ O. MARTINEZ

DESIGNED BY: ----

CHECKED BY : V. JUAREZ/ W. LIEW

ENGINEER OF RECORD (MSE STRUCTURE ONLY):

---- P.E. NO.

NO.	DATE	DESCRIPTION	BY
0	09/19/2025	ISSUED FOR REVIEW	JP

REVISION / ISSUE

SHEET TITLE

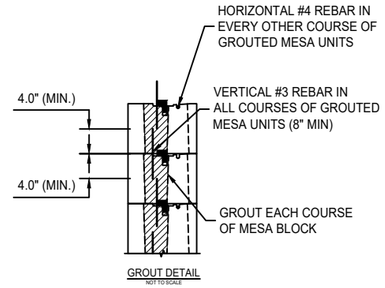
STANDARD DETAIL (13 OF 14)

SHEET 14 OF 15

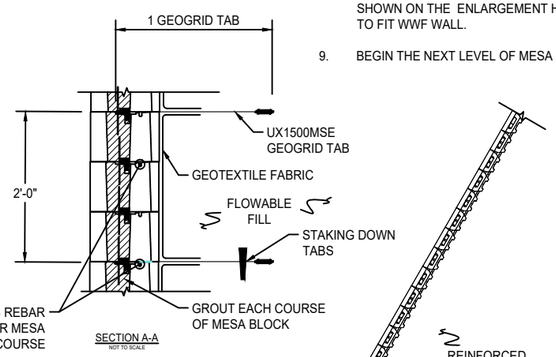
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GUIDELINES FOR STRUCTURAL BACKFILL (FLOW-FILL) PLACEMENT

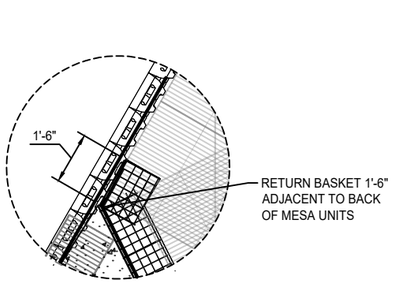
1. LAYOUT THE MESA FACING UNITS FROM THE ACUTE CORNER TO THE FIRST GEOGRID OR TAB LAYER.
2. GROUT MESA BLOCK COURSES AS SHOWN IN THE GROUT DETAIL HEREIN.
3. LAYOUT THE WWF WALL (TEMP. SUPPORT FORMWORK) TO MATCH AT / OR ABOVE THE TOP OF BLOCK ELEVATION IN FRONT OF THE WWF.
4. BACKFILL THE REINFORCED ZONE UP TO THE FACE OF WWF WALL.
5. PLACE FLOW-FILL LIFTS IN SUCH A MANNER AS TO MATCH GEOGRID AND GEOGRID TABS ELEVATIONS. ADDITIONAL LAYERS SHALL NOT BE PLACED UNTIL STRUCTURE BACKFILL (FLOW FILL) HAS LOST SUFFICIENT MOISTURE TO BE WALKED ON WITHOUT INDENTING MORE THAN TWO INCHES.
6. FOR EACH SUCCESSIVE LAYER, THE "REINFORCED ZONE" SHALL BE BACKFILLED FIRST BEFORE THE "ACUTE CORNER ZONE" CAN BE BACKFILLED.
7. ADDITIONAL BRACING MAY BE REQUIRED TO HOLD WALL FACING IN PLACE DURING PLACEMENT OF FLOW FILL.
8. THE FRONT FACE OF THE WELDED WIRE FORM AT TRANSITION TO MESA BLOCK WALL SHALL BE LAID OUT AS SHOWN ON THE ENLARGEMENT HEREIN AND TO FIT FIELD CONDITIONS. FIELD MODIFICATIONS MAY BE NEEDED TO FIT WWF WALL.
9. BEGIN THE NEXT LEVEL OF MESA BLOCK PLACEMENT.



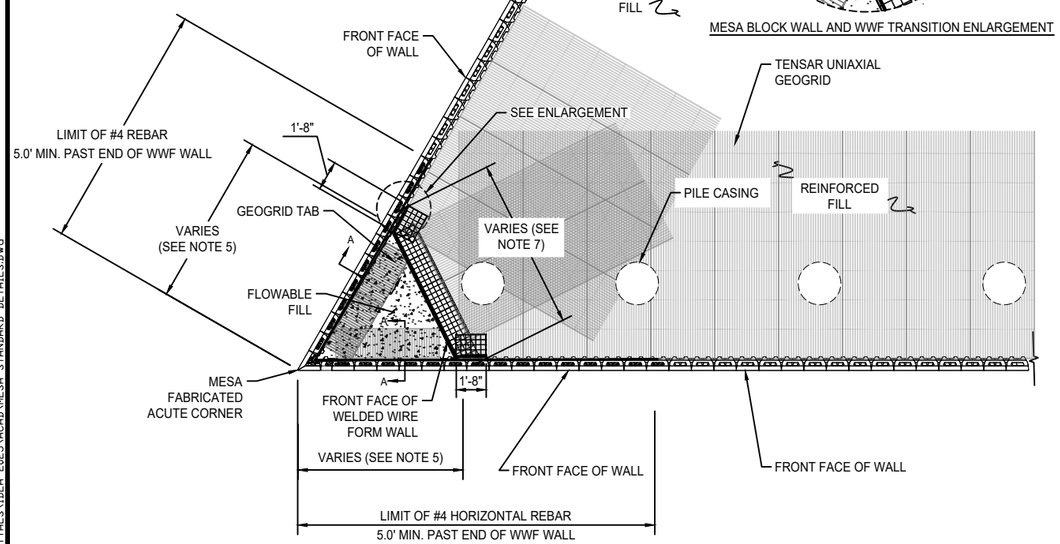
GROUT DETAIL
NOT TO SCALE



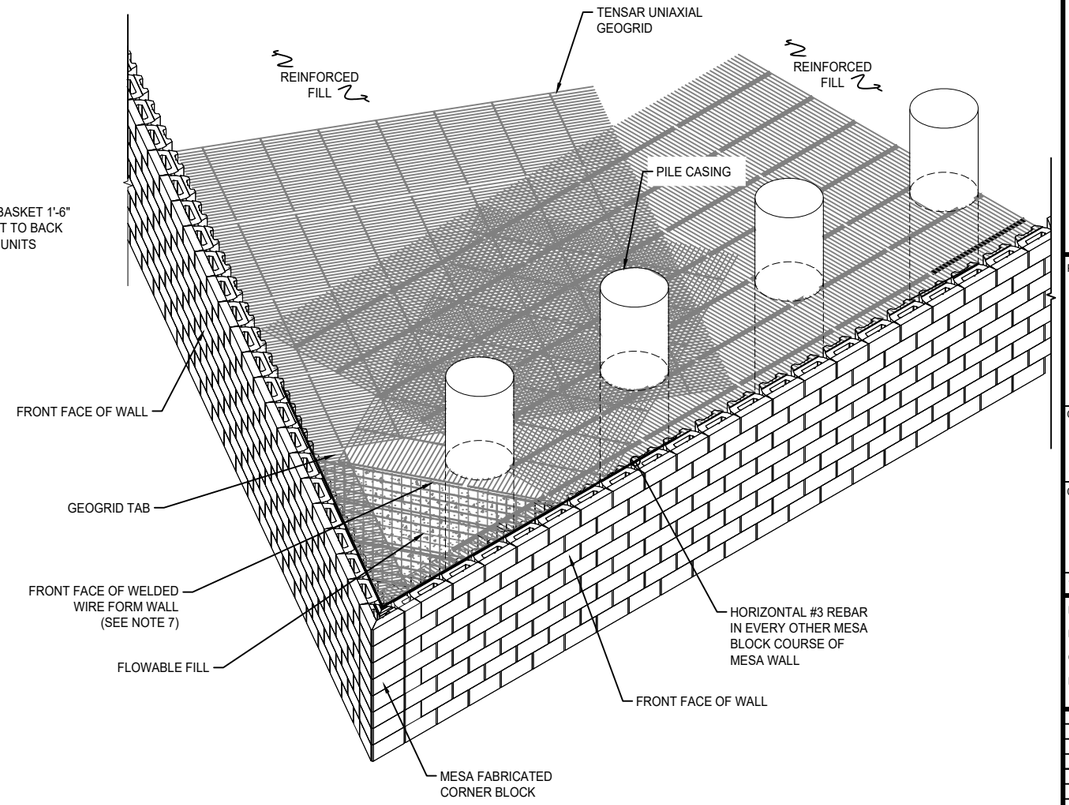
SECTION A-A
NOT TO SCALE



MESA BLOCK WALL AND WWF TRANSITION ENLARGEMENT



ACUTE CORNER - PLAN VIEW
NOT TO SCALE



ACUTE CORNER - ISOMETRIC VIEW
NOT TO SCALE

Plotted on: September 30, 2025
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Tensar.
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REVISION / ISSUE

SHEET TITLE
STANDARD DETAIL (14 OF 14)

Appendix 1.1.3.1a

Quality Control and Specifications sheet for Manufacturing Blocks

MESA® MSE Retaining Wall System

Back to main document

QUALITY CONTROL & TESTING REQUIREMENTS OF MESA® RETAINING WALL BLOCK UNITS

Blocks shall be manufactured in accordance to ASTM C90 from concrete which has a minimum 28 day compressive strength of 4000 psi or as required by project specifications, whichever is higher, as determined by ASTM C140-91.

Licensee shall have two (2) blocks per every (i) daily production run or as specifications require for public bid work or (ii) quarterly for other work tested at Licensee's own expense at an independent lab approved by the Licensor. Once a year, Licensee shall submit to the Licensor, annual test results.

Height tolerances:

front to rear	+/- 1/16" (0.0625")
side to side	+/- 1/16" (0.0625")

Wear part tolerances:

connector slot bar	+/- 0.005"
core puller knife	+/- 0.010"

Check mold box every 50,000 units:

center to back, each leg	+/- 1/16" (0.0625")
side to side at front and top slot	+/- 1/16" (0.0625")

Tolerances shall be checked, verified and recorded every 15,000 Units.

Special requirements for cold weather areas:

For climates which exhibit daily low temperatures of 32 deg. F (0 deg. C.) for thirty (30) days or more maximum, water absorption by weight shall be 5%, as determined by ASTM C140-91.

Freeze-thaw testing shall be conducted on an annual basis in accordance with ASTM C-1262. Tensar International Corporation reserves the right to request that additional freeze thaw testing be performed on a project specific basis. At the current time there is no true industry consensus on which test should be conducted on the units. ASTM C-666 may be required for public (DOT) projects, with the exception of MN DOT which requires C-1262 and 3% saline, and ASTM C-1262 is often required for private projects. In lieu of ASTM C-666, the blocks may be sealed with a water resistant coating.

These Requirements may be changed by Licensor from time-to-time in its sole discretion.

Appendix 1.1.3.1b

Full Quality Control Manual

MESA® MSE Retaining Wall System

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QUALITY MANUAL

FOR

THE MANUFACTURE OF MESA® RETAINING WALL BLOCK UNITS

October 18, 2007

Tensar International, Inc.
5883 Glenridge Drive, Suite 200
Atlanta, Georgia 30328

Introduction

The Tensar Segmental Retaining Wall Quality Assurance Program has been developed as a production process control designed specifically to assist the producers reduce manufacturing variables which, left uncontrolled, may create substandard units.

The recommendations and guidelines promoted within the Tensar Segmental Retaining Wall Quality Assurance Program are a combination of generally accepted industry standards of operation, recognized specifications along with best management practices.

The Tensar Segmental Retaining Wall Quality Assurance Program recognizes that the retaining wall systems will be produced on many different types of manufacturing equipment using a wide variety of locally available raw materials. In spite of these different production conditions, the vast majority of the recommendations and guidelines in this program will apply to all the manufacturing facilities that produce Tensar retaining wall systems.

The Tensar Segmental Retaining Wall Quality Assurance Program is not to be considered as a substitute for the original equipment manufacturer's operation manual or instruction handbook, but as a supplemental source of information designed specifically to promote continuous production of the highest quality retaining wall units.

It is most important that Tensar retaining wall manufacturers strictly adhere to all production safety procedures as noted by the equipment manufacturer and those required by national, state and local mandate. Tensar strongly recommends that each manufacturer of the Tensar retaining wall system maintain company specific safety program.

Tensar International Corporation Reserves the right to update this Quality Assurance program at any time. Updates will be notated by *Issue Date*. Producers will be expected to utilize and implement the provisions of the latest version in their possession. IT IS THE RESPONSIBILITY OF THE PRODUCER TO ADHERE TO THE PROVISIONS CONTAINED HEREIN AS AFFIRMED BELOW:

Licensee Information:

Company Name: _____
Address: _____
_____
_____

Authorized Rep: _____
Title: _____
Phone: _____

Affirmation:

I _____ authorized representative of _____
Licensed producer of Tensar International Corporation MESA Retaining Wall Block Units affirm that the procedures discussed in this manual are strictly followed during production of MESA block. Date of affirmation: _____ .

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Chapter 1 Required Statements

Tensar International corporation. Is dedicated to ensure the manufacture of high quality MESA® block units. Tensar International Corporation's intent is to meet the expectations of our customer's needs in product quality and field performance in the geotechnical and civil construction applications.

Use of ICBO ES Description

The ICBO ES name, logo and ER Number will only be used on products that are in compliance with the published evaluation report and the approved quality control documentation.

Performance Complaints

Tensar International Corporation will promptly investigate and respond to ICBO ES or a building official when appraised by ICBO ES or a Building Official of field complaints concerning product performance.

Inspection

Tensar International Corporation's block producing Licensees agree to permit ICBO ES to examine at distribution points and the manufacturing plant any product labeled as being in conformance with ICBO ES ER-5435. Prior to such inspection, the ICBO ES Representative will be required to sign a Confidentiality Agreement regarding proprietary manufacturing technology.

Notification of Change

Tensar International Corporation will notify ICBO ES in writing if the product has changed from what was originally recognized.

Chapter 2 Raw material suitability

Forward: The materials used in the production of Tensar retaining wall units shall conform to the adapted ASTM standards which apply to each individual material shown below. Further details on each material can be found in the most current ASTM Book of Standards. The actual ASTM specification designation number is shown for each material.

Portland Cement
Definition: Primary cementitious material
ASTM specification designation number: C 150, 595M, 1157M or 989.
Additional comments:
Material suitability certification requirements: Cement supplier shall provide quarterly chemical and physical property reports.

Portland cement substitution
Definition: Secondary cementitious material
ASTM specification designation number(s): C 618 with C 90-97 4.3
Additional comments:
Material suitability certification requirements: Material supplier shall provide quarterly chemical and physical property reports.

Fine aggregate
Definition: Typically sand or other fine aggregate
ASTM specification designation number: C 33
Additional comments: C 33 grading requirements shall not necessarily apply.
Material suitability certification requirements: Material supplier shall provide mechanical (grading) report monthly & complete chemical and physical property reports yearly.

Course aggregate
Definition: Typically rock, stone or other course aggregate
ASTM specification designation number: C 33
Additional comments: C 33 grading requirements shall not necessarily apply.
Material suitability certification requirements: Material supplier shall provide mechanical (grading) report monthly & complete chemical and physical property reports yearly.

Water
Definition: Batch mix water
ASTM specification designation number: N/A; water shall be potable
Additional comments:
Material suitability certification requirements: Water shall be tested annually

Admixture
Definition: Liquid and dry additives used to enhance unit performance
ASTM specification designation number: Refer to C90; 4.3
Additional comments:
Material suitability certification requirements: Material suppliers shall, on an annual basis, provide suitability certification of conformation to any applicable ASTM standards.

Pigments

Definition: Liquid and dry pigment additives used as a tinting agent

ASTM specification designation number: Refer to C90; 4.3

Additional comments: Synthetic oxide pigments only. Natural oxides not allowed.

Material suitability certification requirements: Material suppliers shall, on an annual basis, provide suitability certification of conformation to any applicable ASTM standards.

Chapter 3 Mix Design

Mix design objective:
The selection of raw materials and the proportions of these materials used to manufacture high quality Tensar retaining wall products is extremely important. Unfortunately, there is no standard, or universal mix design which can be used in all manufacturing facilities to produce Tensar retaining wall units that meet or exceed all performance requirements.

Therefore, it is the responsibility of each individual manufacturer of Tensar retaining wall products to design the best mixture of raw materials in order to meet unit specifications. Listed below are some generally accepted mix design philosophies which may serve as a good reference for Tensar retaining wall manufacturers.

A. Use the highest quality Portland cement available in the market. Cements that yield the highest 28 day cube strength may be the best selection.

B. Use the highest quality fine and course aggregate locally available in the market. Fractured aggregates that perform well in hardness and abrasion resistance testing may be the best selection.

C. The fine and course aggregates shall be combined into a blend that results in the fewest voids. In other words, the mechanical analysis of the blended fine and course aggregates shall not indicate a 'gap graded' condition on any individual gradation screen.

D. The aggregate : cement ratio should not be less than 9.5 : 1. In many cases, a higher cement content will be required with aggregate : cement ratios approaching 8.0 : 1.

E. The use of plasticizing (water reducing) admixtures is generally recommended for manufacturing high compressive strength / low absorption retaining wall units.

Chapter 4 Mix proportioning

Storage of raw materials

Cementitious Materials: This material should be kept inside of a fully enclosed, weather tight storage container. This container should be vented to the outside atmosphere to prevent pressure build up. Containers should be designed to provide free flow of product to the discharge point of the container. Containers should be inspected regularly to insure that interior walls are free from build up of hardened material. Divided containers with multiple interior compartments used for the storage of dissimilar materials should be inspected regularly to insure that common divider walls are not allowing contamination to occur. Pneumatic conveyance equipment used to fill these containers should be designed to provide a moisture free air supply. Mechanical conveyance equipment should be free from residual material prior to filling the container.

Fine and Course Aggregate: These materials should be stored in bins or bunkers which provide separate storage for each different type of material. Bin and bunker divider walls should be inspected regularly to prevent contamination from foreign materials. Bins and bunkers should be designed to provide free flow of material to the discharge point. Bins and bunkers shall be kept clean and free of residual material creating restrictions to material flow. Material distributor 'chutes' (if used) must be inspected regularly to prevent accidental contamination of material. Common truck, loader or rail car dump hoppers shall be completely free of dissimilar residual material prior to filling. It is strongly recommended that fine and course aggregate bins and bunkers be designed to allow excess surface moisture to drain from material and away from batching / weighing / mixing equipment. Furthermore, it is recommended that aggregate storage bins and bunkers have some type of roof structure to prevent the accumulation of additional moisture.

Water: In most cases, water is introduced to the mix from the source or supply that is typically a municipal water line or private well on site. However, some production facilities require water storage tanks that should be designed to not allow contamination of the water.

Liquid admixtures: Shall be kept in weather tight storage containers equipped with a pressure vent system. These containers shall be kept clean and free from residual material and debris. Liquid admixtures shall not be exposed to freezing environments.

Dry admixtures and pigments: These products should be kept in their shipping package until use. Store this material in a dry area.

Weighing / metering of raw materials

Cementitious Materials: Slide & clamshell gates, rotary feeders, augers, should provide a smooth and consistent flow of material throughout the entire weighing process. These devices should be designed to operate in a 'normally closed' control position to prevent accidental discharge due to loss of power, air pressure etc.. Chutes or 'boots' should be fitted to the gate, feeder or auger to prevent loss of material between the point of discharge to entry into the weigh batch container.

Production facilities that operate beam type or electronic load cell scales shall have these weighing systems inspected and certified to an accuracy of ±1% by a source approved by the state. This approval shall be done on an annual basis. Scale dials or CRT screens shall display approval sticker. Plant operators shall maintain a record of all scale inspection reports regardless if scale adjustment was required or not. Batch container shall be large enough to hold the entire amount of cement required for one complete batch. No double batching cementitious material allowed without approval by Tensar.

Production facilities using volumetric, or 'sphinx' type weigh batching, shall validate

the accuracy of this equipment by checking 'set points' with the manual addition of known amounts of material. This procedure should be performed semi annually.

Fine and Course aggregate: Slide & clamshell gates, rotary feeders, augers, should provide a smooth and consistent flow of material throughout the entire weighing process. These devices should be designed to operate in a 'normally closed' position to prevent accidental discharge due to loss of power, air pressure etc..

Production facilities that operate beam type or electronic load cell scales shall have these weighing systems inspected and certified to an accuracy of ±2% by a source approved by the state. This approval shall be done on an annual basis. Scale dial or CRT screens shall display an approval sticker. Plant operators shall maintain a record of all scale inspection reports regardless if scale adjustment was required or not.

Production facilities using volumetric, or 'sphinx' type weight batching shall validate the accuracy of this equipment by checking the 'set points' with the manual addition of known amounts of material. This procedure should be done semi annually. Multiple or 'double batching' is only allowed if the actual cumulative amount of material is within ±2% of the desired batch weight.

Water: Production facilities that operate with water flow meters shall have these metering devices inspected and certified to an accuracy of ±5% by a source approved by the state. This approval shall be done on an annual basis. The meter body shall display approval sticker. The flow valve, located in front of the meter, shall be designed to operate in a normally closed position to prevent accidental discharge due to loss of power, air pressure etc..

Liquid Admixtures: Production facilities that operate with flow meters shall follow the same guidelines as water.

Dry Admixtures and Pigments: If these products are proportioned by hand dosage with 'spring' type or electronic platform scales, accurate to ±1%, shall be used to determine dosage weight. Plant managers shall check 'spring' type scales monthly by placing a known amount of weight on the scale and adjusting the indicator needle to reflect true weight. Electronic scales should have true 'zero' established each time the scale is used.

Chapter 5 Mixing Raw Materials

Recommended mixing procedures for production plants manufacturing Tensar retaining wall units with heavy weight fine & course aggregates.

#1. Charge mixer with all fine & course aggregates, all dry admixtures and all liquid & dry pigments completely.

#2. Charge mixer with all cement & cement type replacement material.

#3. Dry mix the combination of above materials for 1 minute.

#4. Add all mix[1] water and liquid admixtures.

#5. Mix for a minimum of 3 minutes.

#6. If needed to achieve desired mix consistency, add trim[2]water and mix 1 additional minute. After the addition of trim water, no additional re-temper water is allowed.

[1]mix water is considered to be the maximum amount of water that you can safely introduce into the mixer being confident that you are at, or below total water requirements for the mix.
[2]trim water is considered to be the final addition of water required to adjust the mix to proper consistency

Chapter 6 Unit Molds

In an effort to produce the highest quality Tensar retaining wall units, plant operators shall inspect the condition of all molds prior to installing molds in the production machine. The following inspection steps shall be followed;

#1. Remove top assembly (stripper shoe / plunger / plunger top plate) from the mold cavity. Remove excess concrete build up on the entire mold.

#2. Inspect wearing surfaces of mold cavity for excessive wear[3] or damage.

#3. Inspect outside wear surface of cores (if equipped) for excessive wear or damage.

#4. Inspect core mounts. Check to make certain that mounts are fastened securely to mold box.

#5. Inspect mold side bars (Besser) or mold side mounts (Columbia) for cracks.

#6. Check wear liner fasteners to make certain that liners are secured to the mold box.

#7. Check stripper shoes for excessive wear.

#8. Measure shoe depressions (Mesa Standard, 'Plateau, Ashford, Adobe, Gardener11,HP & XL') for excessive wear. Refer to unit conformation drawings for measurements.

#9. Inspect electrical connectors and cords (on molds with heated stripper shoes).

#10. Test run heaters (on molds equipped with heated stripper shoes).

#11. Insert top assembly in mold cavity. Check for proper amount of clearance. Typically, new molds are designed with .040 to .060 inches of 'side to side' clearance between shoes and liners with the shoes centered in the mold. In most cases, you can run a mold to .100 'side to side' clearance measured with the shoes centered in the mold.

[3] excessive wear is typically considered as a wear pattern that has developed measured to a depth of .100 inch.

Chapter 7 Installing mold

Installation of the Tensar mold:
After positioning and securing mold cavity and top assembly in production machine, make certain that the complete mold is level and centered in the machine. If needed, on 18 inch product (Tensar Mesa Standard, 'HP' & 'XL'), make all necessary machine adjustments to accommodate the additional length of these units.

Machine operator should carefully 'dry-cycle' the machine / mold and check mold liner to stripper shoe ('side to side') clearance which should be a minimum of .040 to a maximum of .100 inches.

Install strike off bar and check that the lower portion of the blade is in the proper position to remove excess material from the top of the mold cavity. Also make certain the top of the blade wiper or brush is contacting the bottom surface of the stripper shoe.

Connect stripper shoe electrical cord to heater control box. Turn on power to heat stripper shoes. Check that the controller and shoe heater elements are functioning.

Chapter 8 Manufacturing / pre-production

Manufacturing equipment

Block Machine
Block manufacturing machine shall be in good repair. Machine should be structurally sound. All moving and reciprocating assemblies (bushings, bearings, shafts etc.) shall be within the original equipment manufacturers allowable tolerances for wear or excessive movement. All vertical slide shafts or guide posts, which limit side to side & front to rear movement of the stripper head, shall be maintained to the machine manufacturers' specification as delivered new.
Failure to keep block machine in the condition as stated above, will most likely result in cracked or broken units when stripped from the mold!

Feed box / feed drawer
Shall be adjusted to deposit the correct amount of material in the mold cavity. Agitator grids & material baffle plates may be required to add extra material to the area(s) of the mold cavity where raised features will be created on the top of the unit.
Failure to keep feed box / feed drawer in proper adjustment will most certainly result in poor unit height control, potentially low unit compressive strength and poor absorption characteristics. In addition, critical raised features on the top of the unit may be poorly formed which will compromise the engineering value of the system!

Strike off blade / rake off bar
Shall be fastened securely to the feed box / feed drawer assembly and positioned to remove excess material from the mold cavity. Strike off blade / rake off bar shall be adjustable to regulate material removal.
Potential problems are the same as those listed above!

Pallet placement assemblies
Shall be adjusted to position the steel production pallet in the center of the mold as measured front to rear. Pallet guides shall be used to control excessive right to left movement.
Failure to keep steel production pallets in correct position will create a condition where a portion of the concrete unit will overhang pallet and brake away!

Pallets
Steel production pallets shall be flat ±1/32 inch. Pallets shall be clean and free from hardened concrete build up in excess of 1/32 inch. Pallets shall be uniform in size and thickness.
Failure to maintain clean, flat and uniform steel production pallets will result in poor unit height control!

Machine material hopper
Shall be large enough to permit continuous production. Stationary hopper located above the production machine and / or feed belt systems shall provide constant material head pressure to feed drawer / feed box receiver area. Material hoppers shall be designed to provide uniform flow of material to feed drawer / feed box receiver. Hoppers shall be free from hardened material.
Failure to provide constant uniform and unrestricted flow of material to feed drawer / feed box receiver will result in poorly formed units with low compressive strengths and high absorption characteristics. In addition, dislodged hardened material may be fed into the mold creating a visual scar on the face of the unit!

Mold vibrators: 'outboard mounted' belt driven (Besser standard style)
Vibrator drive belts and pulleys shall be in used in matched sets and in good condition. Vibrators shall be synchronized (within 1%) in starting points, rpm, ramp rates and braking points. 'Clamp on' style weights shall be matched relative to total weight and position on the vibrator shaft. Selection of weight shall be the decision of the operator with consideration given to manufacturing the highest compressive strength and lowest absorption unit without excessive vibration that may cause material segregation in the unit.
Failure to synchronize vibrators, or the use of 'mismatched' vibrator components will create poorly consolidated and / or cracked units. In addition, poor unit height control and visual material segregation may occur!

Mold vibrators: 'outboard mounted' fluid driven (Besser *Smart Pac™* style):
Vibrators shall be matched and synchronized relative to start, rpm rates, amplitude, frequency, time and stop points to within 1% from side to side.

Failure to synchronize and match vibration characteristics will result in unit problems similar to those as stated above!

Mold vibrators: 'rigid shaker' (Columbia style):
Vibrator system shall be in good condition and provide uniform mold box movement, right to left and front to rear. Vibrator start / stop points & vibration dwell time shall be determined by the operator.
Failure to maintain uniform mold vibration will result in unit problems similar to those stated above!

'Stacker' / curing rack handling equipment:
This equipment shall provide adequate clearance to eliminate potential interference between concrete unit(s) and handling equipment.
Failure to provide adequate clearance will result in damaged concrete units due to contact between the units and the handling equipment!

Chapter 9 Manufacturing / Trial Batch

'Dry cycle' test run:
It is important to 'dry cycle' block machine prior to mixing the first test batch of concrete mix. Check mold liner / stripper shoe clearance to eliminate possible interference of these two assemblies. Check feed box and strike off blade / rake off bar position. Check vibration start / stop cycle. Check pallet delivery and placement under mold cavity.

Single test batch trial run:
Mix a single test batch of concrete material and produce test units. Produce a 'short run' (typically 5 to 10 machine cycles). Visually check units for proper height, adequate feed, fill and compaction. Pay particular attention to the top surface quality and the raised / depressed features of the unit. If units have core voids, check depth of core bar marks. These marks should leave a slight impression in the top of the unit roughly 1/16th inch in depth. Do not produce units that leave residual material on the top of the block that is a result of the core bar being too high in the mold. Stripping of the unit from the mold cavity shall be a smooth and fluid machinery movement. Rough motion of this stripping cycle could create cracks or fractures in the unit. Do not begin continuous batching until the machine operator confirms that the units meet or exceed the above visual quality control criteria. Any major machine adjustment should be accomplished during this single test batch trial run.
Failure to adapt single test batch procedure and proceeding directly to continuous batching may result in unit quality problems if machine adjustments are required and the concrete mixture begins the hydration process.

Chapter 10 Manufacturing / continuous production

Unit height control check:
The machine operator shall check unit height every 50 machine cycles to confirm that the unit height is not in excess of ±1/16th inch from unit standards. This unit height control check process shall not interrupt continuous machine cycling except in the event that machine adjustments are required. Plants with block production machines that are equipped with positive height stops with indicator lights shall pay close attention to insure that the right and left side lights illuminate at the same time.
Failure to check unit height at the 50 cycle interval as stated above could result in large production runs of product that will not meet quality standards!

Overall unit quality control check:
The machine operator shall perform an overall unit quality control check at least every 15 minutes of continuous production. This overall unit quality control check will interrupt the continuous manufacturing of product. Machine operator shall check the following:
A. unit height : 8 ± 1/16th inch
B. core bar impression depth
C. raised and depressed features on the top of the unit
D. visual check for cracks or fractures which may appear along any of the four sides of the unit.
E. visual check of the mix moisture content to confirm that it appears to be consistent with plant standards.
F. visual check of the edge surface around the top of the unit. Edge should appear sharp and not rounded.
Failure to perform 15 minute quality check may result in large production runs of product that will not meet quality standards.

Continuous production of Tensar units:
The machine operator shall make every attempt to operate the production machine on a continuous production run, limiting interruptions to quality control checks and machine adjustments. Unnecessary stopping of production shall be avoided whenever possible. Raw material storage, batching, mixing, rack handling, splitting and palletizing should each have the capacity to allow the actual production machine to run without interruption due to system limitations.

Assessing mold condition during continuous production runs:
Production machine operators shall assess the quality of the mold at not more than every 2,500 machine cycles. This assessment shall consist of stopping the machine with the mold top assembly raised (the feed position), securing the machine in this position for safety, and inspecting all mold wearing surfaces for excessive component wear.
Failure to assess the mold components every 2,500 cycles may result in large production runs of product that will not meet quality standards!

Chapter 11 Unit curing

Curing systems:
There are many different acceptable methods of curing Tensar retaining wall units. These curing methods range from low temperature (ambient) to high temperature (350 degrees F). In addition, some curing methods do not add any moisture to the air while other curing methods introduce 100% steam moisture at 150 pounds per square inch into curing chambers. Despite the great variations of curing systems, all systems that are capable of producing standard concrete masonry units that meet current ASTM C-90 specifications should be capable of producing high quality Tensar units.

With this in mind, it is important that all producers of Tensar retaining wall units understand the following curing management practices and procedures.

A. Producers shall take whatever steps necessary to adopt a 'first in / first out' product handling procedure to the best of their ability. The goal is to subject all units to the same amount of time in the same curing environment.
Minimizing or eliminating curing variables will result in greater product uniformity.

B. Individual curing chambers (commonly referred to as kilns) should be designed to provide the same temperature and moisture conditions within the individual kiln, top to bottom, front to rear, right to left.

C. Understand that when equilibrium is attained, the continued introduction of heat, steam and moisture should stop. In curing, equilibrium is defined as 'that point where the moisture in the air is equal to the moisture in the concrete unit'. When curing equilibrium is attained, additional moisture is not required and in some systems, could be detrimental to the quality of the unit.

D. A drying period prior to removing the block from the kiln is strongly recommended. If there is water or excessive moisture on the units as they are depalleted, split, palletized and inventoried, quality problems may occur. These problems include, but are not limited to; staining due to primary or secondary efflorescence, poor split face surface texture, excessive unit chipping etc.

Chapter 12 Unit splitting

Splitting equipment:
Equipment used for splitting of Tensar retaining wall units shall be kept in good condition. Prior to actual splitting of the units, splitter blade segments shall be inspected and replaced if necessary. Individual splitter blade segments shall be uniform in height and shall contact the unit evenly across the top and bottom of the unit. Top and bottom blade assemblies shall be in absolute alignment with each other. Splitter blade condition assessment shall be made every eight hours of continuous operation.
Failure to operate and maintain block splitter / splitter blade segments could produce poor quality rock face texture on the unit.

Chapter 13 Unit palletizing

Arrangement of product on shipping pallet:
Arrange product on shipping pallet in a manner that creates the least amount of potential damage to the units. 'Nesting' the units (keeping split pairs together) whenever possible, will create the tightest pattern arrangement. Since the dimensions of shipping pallets may vary, a standard unit pattern arrangement is not available. Therefore, it shall be the responsibility of each individual plant operator to develop the best unit pattern arrangement. Block(s) shall not overhang the shipping pallet. It is strongly recommended that the units are secured to the shipping pallet by means of plastic stretch wrap, shrink wrap or metal banding.
Failure to palletize product in a tight pattern will likely result in damage to the product during movement to inventory and shipment to the job site. Additionally, unsecured product could create further unit damage.

Chapter 14 Labeling & Unit identification code

Purpose:
Product labels allow for quick and easy identification of the product type and system. Product identification code on each pallet of Tensar retaining wall product will allow units stored in inventory and at installation sites, to be easily identified in regards to when the product was manufactured. This product identification code will provide the following benefits:

A. When producers are responsible for *LOT*[4] or *GROUP* testing of product, this coding system will allow the party responsible for testing to easily identify a continuous run of product. The testing party will have greater confidence that the sample(s) obtained can be linked directly to large inventory groups of product.

B. The code system will allow the producer to ship continuous run product
to installation sites. This practice will decrease product variation and increase product uniformity.

Unit labels and identification code:

Labels shall include the following information:

- Product Style
- The Tensar Logo
- Tensar International Corporation & Address
- Unit Identification code (as described below)
- ICBO ES ER-5435

Producers shall stamp or mark each pallet of product. Stamps or markings shall be permanent and include a code system which contains the following information; month, day, year and curing kiln number*.

An example of such a code system is shown below:

062987

06= 6th month	29= 29th day	8= 1998 year	7= kiln #7

The month, day, year & kiln number shall be selected to correspond to the time the units are palletized. An identification code stamp shall appear on the smooth side of one individual Tensar unit located on the bottom course of each shipping pallet of product. The identification code stamp shall be visible at all times, even when the product is stacked in storage.

*Note: Kiln identification (shown in example above as 7) can be changed to reflect the hour or other means of identifying a continuous production run not longer than 2 1/2 hours in duration.

[4]lot is defined as a continuous production run, not interrupted by the product mold or mix design change

Chapter 15 Unit inspection, sampling and testing.

Unit inspection:

All Tensar units shall be inspected the greater of every 4 hours or twice per shift. This visual inspection shall occur between the time the unit is split and palletized. This visual inspection shall consist of examining each unit for obvious quality flaws, which include, but is not limited to; cracks, excessive chipping, poor quality splits, poor compaction, material surface segregation, moisture damage from curing system etc.. Units that fail to meet visual quality standards shall be removed from the production line prior to palletization. In most cases, this visual unit inspection should not interrupt the production process.

Tensar molds shall be inspected prior to the start of each production run. At this time the following Critical Physical Measurements must be performed: 1.) Height: 8 ± 1/16th inch from side to side, 2.) Connector Slot: Shoes shall measure 0.69 - 0.71 inches, and 3.) The distance from the front edge of the block face to the back edge and bottom of the core shall be 6.25 inches. A mold that fails to meet these dimensions shall be removed from production. During the inspection process replace any and all parts that may adversely affect the quality of the finished unit.

Unit sampling:

From each lot, remove one representative sample (six units) from each 10,000 units or fraction thereof. The representative sample should be removed from the mid-point of each 10,000 units produced. As an example, if a lot (continuous production run of identical product) of 30,000 units is manufactured, a sample should be taken from a group which represents approximately the 5,000th unit, 15,000th unit and the 25,000th unit.

Individual samples shall be removed from the top layer of the palletized product. The individual responsible for the collection of the samples shall identify each sample with the product code number as shown on the unit located on the bottom layer of the palletized product from which the sample was removed. Once removed, the sample shall be replaced with a similar unit manufactured at the end of the lot. Pallet of product from which the sample was removed shall be stamped with the word SAMPLED using permanent ink. This stamp shall appear next to the product code number located on each pallet of product from which the sample was removed.

Each sample shall be subjected to the following destructive and non-destructive tests.
A. Unit weight as received.
B. Unit moisture content as received.
C. Unit dimension, height, depth and length.
D. Unit surface area, gross and net calculations.
E. Unit compressive strength, gross and net calculations.
F. Unit inspection, sampling and testing
G. Cold water absorption testing

Additional Testing:

Freeze-thaw testing shall be conducted on an annual basis in accordance with ASTM C-1262.

In addition, Tensar International Corporation reserves the right to request that additional freeze thaw testing be performed on a project specific basis. At the current time there is no true industry consensus on which test should be conducted on the units. ASTM C-666 may be required for public (DOT) projects, with the exception of MN DOT which requires C-1262 and 3% saline, and ASTM C-1262 is often required for private projects. In lieu of ASTM C-666, the blocks may be sealed with a water resistant coating. Samples taken for this testing shall be labeled with the same product identification number appearing on the unit located on the bottom layer of the palletized product from which the samples were removed.

Identifying unit(s) passing quality test:

Tensar retaining wall units that pass the quality testing program shall be identified as accepted product.

Producer shall indicate this product acceptance by stamping or marking each pallet of each approved lot with the word APPROVED. This approval identification stamp shall be placed adjacent to the unit identification code number already appearing on each individual pallet of product.

Identifying unit(s) failing quality test:

Tensar retaining wall units which fail to pass quality testing shall be identified as substandard product.

Producer shall indicate the product fails to meet quality standards by stamping or marking each pallet of each substandard lot with a symbol, letter or number that provides absolute identification of the substandard product. This identification stamp shall be placed adjacent to the unit identification code number already appearing on each individual pallet of product.

In addition, it is strongly recommended that producers remove substandard units from inventories of similar approved product to avoid shipments accidentally containing a combination of approved and substandard product.

Record Keeping:

Records of each inspection mentioned shall be approved by the licensee QC officer prior to final approval of the block produced. In addition, all inspection, QC and other production records shall be maintained on site for at least two years beyond the production date.

The OC officer will also maintain, on site, a complete list of all equipment necessary to perform all cited QC checks and controls. This list will include the date of last calibration. All equipment will be calibrated regularly in accordance with the manufactures (or test method) recommendation.

Chapter 16 Unit specification

Compressive strength:
Unit compressive strength;

Minimum compressive strength: 4,000 psi (27.6 MPa) net minimum or as specified
(Average of 3 coupons)

Minimum compressive strength: 3550 psi (24.5 Mpa) net minimum or as specified
(Individual coupon)

Absorption:
Unit cold water absorption; 5% maximum

Unit dimension tolerances:

Unit height; 1/16th inch (1.6 mm) over / under specification
Unit length; 1/8th inch (3.2 mm) over / under specification
Unit depth; 1/8th inch (3.2 mm) over / under specification (splitter blade contact point, net of rock face relief)
Dimension of the front face shall be within ± 1 inch (± 25 mm) of the theoretical dimension of the unit as depicted in the relevant Production Drawings.
Unit grid connector slot; minus 0 to +1/16th inch (0 to + 1.6 mm) over specification for slot depth
Unit grid connector slot; shall measure 0.69 - 0.71 inches for slot width

Production Drawings:
Tensar provides production drawings for each type of MESA unit on an annual basis. It is the responsibility of the Licensee to maintain a copy of all relevant drawings and specifications (bearing a date less than one year old) at the casting machine during block production.

Appendix 1.1.3.2

DOT Connector Specifications

MESA® MSE Retaining Wall System

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Tensar International Corporation
2500 Northwinds Parkway, Suite 500
Alpharetta, Georgia 30009-2247
Phone: 800-TENSAR-1
www.tensar-international.com

Product Specification – DOT MESA Connector

Tensar International Corporation reserves the right to change its product specifications at any time. It is the responsibility of the specifier and purchaser to ensure that product specifications used for design and procurement purposes are current and consistent with the products used in each instance.

DOT Mesa Connectors are injection molded pieces made with a compound of select polypropylene resin with 25% fiberglass filler. The DOT Connectors consist of 19 uniformly spaced "teeth" and eight "flags" connected by a horizontal bar. The teeth are positioned under the horizontal bar and are the same width as the horizontal bar. Four flags are positioned on top of the horizontal bar above the four teeth at each end of the connector. The connector teeth mechanically connect reinforcement to the facing elements of the mechanically stabilized earth retaining wall structure. The four teeth at each end of the DOT Connectors are driven through the apertures of Tensar geogrids into the two connector slots in Mesa Standard concrete facing blocks and the 11 interior teeth penetrate through the intermediate apertures of the geogrid into the core of the facing block. The connector flags extend up into the cores of overlying facing units and as act as alignment aids and shear keys. DOT connectors are fully stabilized to resist ultraviolet degradation.

Product Properties

Index Properties	Test Method	Units	Values
▪ Tensile Strength @ Yield	ASTM D638	MPa	50 to 70
▪ Melt Flow Rate	ASTM D1238	g/10 min	6.0 to 15.0
▪ Specific Gravity	ASTM D792		1.04 to 1.12
Material			
▪ Polypropylene	ASTM D4101	%	72 ± 2
	Group 1/Class 1/Grade 2		
▪ Carbon Black[1]	ASTM D4218	%	2.0 (min)
▪ Glass Fiber	ASTM D2584	%	25 ± 3

Notes:
1. Carbon black added as a UV stabilizer.

Tensar International Corporation warrants that at the time of delivery the bodkin bar or rod furnished hereunder shall conform to the specification stated herein. Any other warranty including merchantability and fitness for a particular purpose, are hereby excluded. If the bodkin bar or rod does not meet the specifications on this page and Tensar is notified prior to installation, Tensar will replace the geogrid at no cost to the customer.

This product specification supersedes all prior specifications for the product described above and is not applicable to any products shipped prior to January 1, 2012.

Appendix 1.1.3.3

Standard Connector Specifications

MESA® MSE Retaining Wall System

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Tensar International Corporation
2500 Northwinds Parkway, Suite 500
Alpharetta, Georgia 30009-2247
Phone: 800-TENSAR-1
www.tensar-international.com

Product Specification – MESA Standard Connector (OP474888L)

Tensar International Corporation reserves the right to change its product specifications at any time. It is the responsibility of the specifier and purchaser to ensure that product specifications used for design and procurement purposes are current and consistent with the products used in each instance.

Standard Mesa Connectors are injection molded pieces made with a compound of select high density polyethylene resin with 30% fiberglass filler. The Standard Connectors consist of four uniformly spaced "teeth" and two "flags" connected by a horizontal bar. The teeth are positioned under the horizontal bar and are the same width as the horizontal bar. The flags are positioned on top of and at the ends of the horizontal bar. The flags are approximately twice as wide as the horizontal bar and are offset so as to be flush with one side and centered over the other side of the bar. The Standard Connector teeth are driven into connector slots in Mesa Standard concrete facing blocks to mechanically connect elements of the mechanically stabilized earth retaining wall structure. The connector flags extend up into the cores of overlying facing units and act as alignment aids and shear keys. The connectors are fully stabilized to resist ultraviolet degradation.

Product Properties

Index Properties	Test Method	Units	Values
▪ Tensile Strength @ Yield	ASTM D638	MPa	55 to 65
▪ Melt Flow Rate	ASTM D1238	g/10 min	3 ± 2
▪ Specific Gravity	ASTM D792		1.10 to 1.22
Material			
▪ High Density Polyethylene	ASTM D1248 Type III/Class A/Grade 5	%	68 ± 3
▪ Carbon Black[1]	ASTM D4218	%	2.0 (min)
▪ Glass Fiber	ASTM D2584	%	30 ± 3

Notes:
1. Carbon black added as a UV stabilizer.

Tensar International Corporation warrants that at the time of delivery the bodkin bar or rod furnished hereunder shall conform to the specification stated herein. Any other warranty including merchantability and fitness for a particular purpose, are hereby excluded. If the bodkin bar or rod does not meet the specifications on this page and Tensar is notified prior to installation, Tensar will replace the geogrid at no cost to the customer.

This product specification supersedes all prior specifications for the product described above and is not applicable to any products shipped prior to January 1, 2012.

Appendix 1.1.3.4

MESA Geotextile Specifications

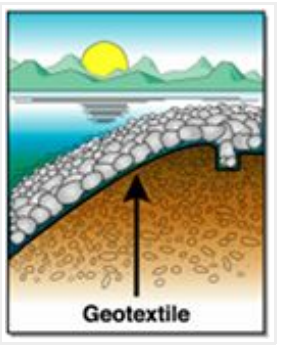
MESA® MSE Retaining Wall System

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US 205NW / TG 700

Nonwoven Geotextile

US 205NW/ TG 700 is a nonwoven needlepunched geotextile made of 100% polypropylene staple filaments. This product can be used in drainage applications or can be used for separation under roads, driveways or parking areas. US 205NW/ TG 700 will satisfy the requirements as outlined in AASHTO M-288-06 for Class 1 applications and meets the following M.A.R.V. values except where noted:





PROPERTY	TEST METHOD	ENGLISH	METRIC
Weight - Typical	ASTM D-5261	8.0 oz/sy	271 g/sm
Tensile Strength	ASTM D-4632	205 lbs	912 N
Elongation @ Break	ASTM D-4632	50%	50%
Mullen Burst	ASTM D-3786	350 psi	2,413 kPa
Puncture Strength	ASTM D-4833	130 lbs	579 N
CBR Puncture	ASTM D-6241	535 lbs	2,381 N
Trapezoidal Tear	ASTM D-4533	85 lbs	378 N
Apparent Opening Size	ASTM D-4751	80 US Sieve	0.180 mm
Permittivity	ASTM D-4491	1.35 Sec-1	1.35 Sec-1
Water Flow Rate	ASTM D-4491	90 g/min/sf	3,657 l/min/sm
UV Resistance @ 500 Hours	ASTM D-4355	70%	70%

ROLL SIZE	AREA	WEIGHT
12.5' x 360'	500 sys	250 lbs
15' x 300'	500 sys	250 lbs

The above information is to the best of our knowledge accurate, but it is not intended to be considered a guarantee. Any implied warranty for a particular use or purpose is excluded. If the product does not meet the above properties, and notice is given to US Fabrics, Inc., the product will be replaced or refunded (1/2011).

US Fabrics, Inc. | 3904 Virginia Avenue | Cincinnati, OH 45227 (USA) | Phone: 800-518-2290 | Fax: (513) 271-4420

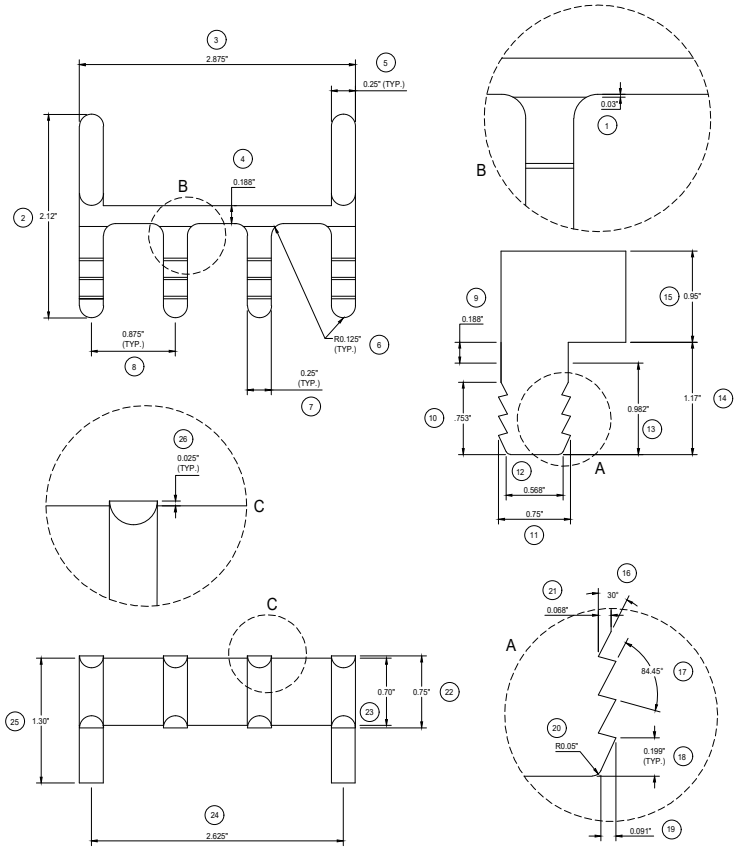
Appendix 1.1.4.1

MESA Standard Connector Fabrication Detail

MESA® MSE Retaining Wall System

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SPECIFIED MATERIAL: HDPE RESIN WITH 30% GLASS FILLED (ACLO HD0235G30CD1).



STANDARD CONNECTOR

NOT TO SCALE

MESA Standard Connector				
Dimesion Number	Dimension	Nominal Dimension (inches)	Minimum Dimension (inches)	Maximum Dimension (inches)
2	Total Height	2.120	2.105	2.145
3	Total Width at Bridge Bar	2.875	2.865	2.895
5	Thickness of Flags	0.250	0.245	0.260
7	Thickness of Teeth	0.250	0.245	0.260
11	Width of Teeth (maximum at serrations)	0.750	0.730	0.750
23	Width of Bridge Bar & Teeth above Serrations	0.700	0.680	0.720
25	Width of Flags	1.300	1.295	1.305

MESA STD CONNECTOR-FABRICATION ONLY-AUGUST 2025.DWG

Tensar®
A Division of CMC

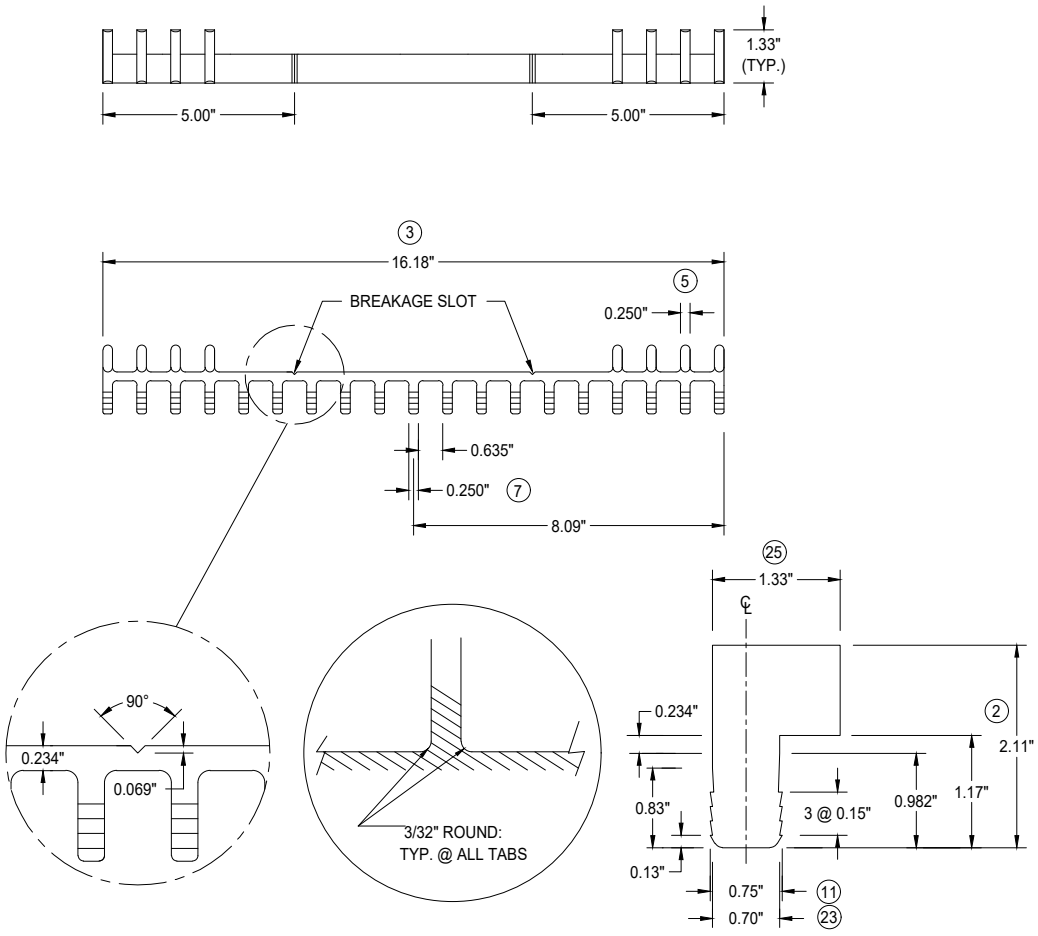
Tensar International Corporation | 2500 Northwinds Parkway, Suite 500 | Alpharetta, Georgia 30009
Toll Free: 1-888-828-5126 | Phone: 770-344-2090 | Fax: 770 344-2089 | www.tensarcorp.com

Appendix 1.1.4.2

MESA DOT Connector Fabrication Detail

MESA® MSE Retaining Wall System

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NOTE:

DOT CONNECTOR CAN BE BROKEN AT BREAKAGE SLOTS TO FACILITATE INSTALLATION.

DOT CONNECTOR

NOT TO SCALE

MESA DOT Connector				
Dimesion Number	Dimension	Nominal Dimension (inches)	Minimum Dimension (inches)	Maximum Dimension (inches)
2	Total Height	2.110	2.095	2.125
3	Total Width at Bridge Bar	16.180	16.165	16.350
5	Thickness of Flags	0.250	0.245	0.260
7	Thickness of Teeth	0.250	0.245	0.260
11	Width of Teeth (maximum at serrations)	0.750	0.720	0.750
23	Width of Bridge Bar & Teeth above Serrations	0.700	0.685	0.720
25	Width of Flags	1.330	1.315	1.345

DOT CONNECTOR-FABRICATION ONLY-AUGUST 2025.DWG

Tensar®
A Division of CMC

Tensar International Corporation | 2500 Northwinds Parkway, Suite 500 | Alpharetta, Georgia 30009
Toll Free: 1-888-828-5126 | Phone: 770-344-2090 | Fax: 770 344-2089 | www.tensarcorp.com

Appendix 1.1.10.1

MESA Block Shear Testing

MESA® MSE Retaining Wall System

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SGI TESTING SERVICES, LLC

20 December 2001

Mr. Andy Anderson, P.E.
Tensar Earth Technologies, Inc.
5883 Glenridge Drive, Suite 200
Atlanta, Georgia 30328

Subject: Laboratory Test Results Transmittal
Block Shear Testing
Mesa Standard Blocks with
Mesa Standard Connectors and UX1400HS Geogrid

Dear Mr. Anderson,

SGI Testing Services, LLC (SGI®) is pleased to present the attached results of block shear testing of Mesa standard block against Mesa standard blocks with Mesa standard connectors and UX1400HS geogrid. The note section below addresses sample preparation, sample disposal and a disclosure statement.

SGI® appreciates the opportunity to provide laboratory testing services to Tensar Earth Technologies, Inc. Should you have any questions regarding the attached document(s), or if you require additional information, please do not hesitate to contact the undersigned.

Sincerely,

Zehong Yuan, Ph.D., P.E
Chief Technical Officer

Attachment

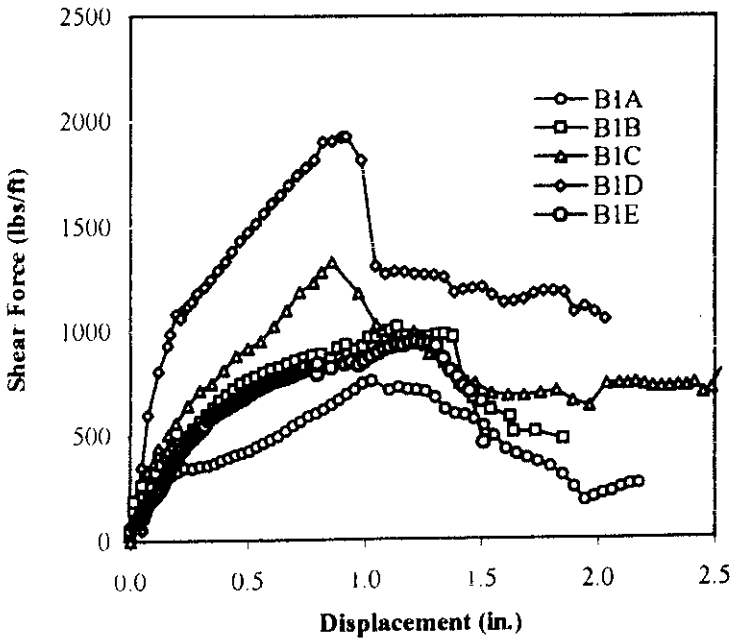
Notes
(1) Unless otherwise noted in the test results the sample(s)/specimen(s) were prepared in accordance with the applicable test standards or generally accepted sampling procedures
(2) Contaminated/chemical samples and all related laboratory generated waste (i.e., test liquids, PPE, absorbents, etc.) will be returned to the client or designated representative(s), at the client's cost, within 60 days following the completion of the testing program, unless special arrangements for proper disposal are made with SGI®
(3) Materials that are not contaminated will be discarded after test specimens and archived specimens are obtained. Archived specimens will be discarded 60 days after the samples are received at the laboratory, unless long-term storage arrangements are specifically made with the laboratory
(4) The reported results apply only to the materials and test conditions used in the laboratory testing program. The results do not necessarily apply to other materials or test conditions. The test results should not be used in engineering analysis unless the test conditions model the anticipated field conditions. The testing was performed in accordance with general engineering testing standards and requirements. The reported results are submitted for the exclusive use of the client to whom they are addressed.

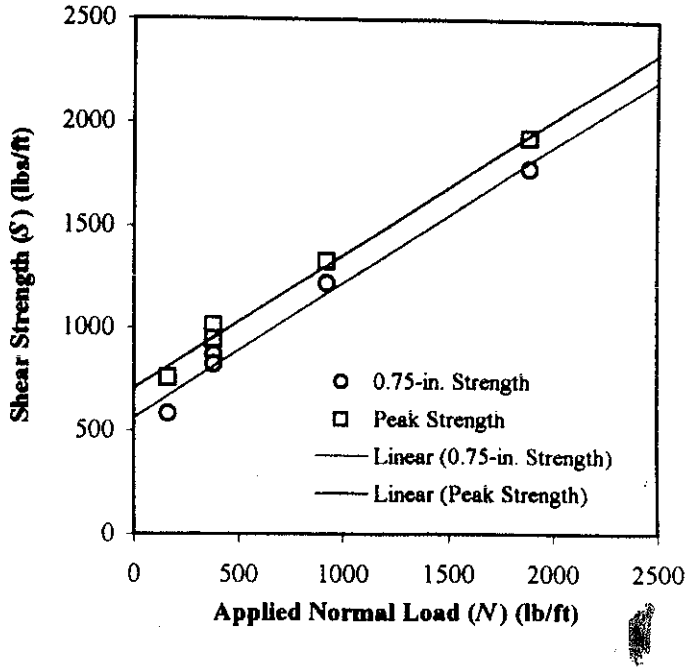
SGI1007/SGI01143

SGI® TESTING SERVICES, LLC
5775 PEACHTREE DUNWOODY ROAD
SUITE 11D
ATLANTA, GEORGIA 30342
PHONE: 404.256.9939 FAX: 404.705.9300
WEB SITE: WWW.INTERACTIONSPECIALISTS.COM

TENSAR EARTH TECHNOLOGIES, INC.
BLOCK SHEAR TESTING (NCMA TEST METHOD SRWU-2)

TEST SERIES NO. B1: Mesa Standard Block Unit against Mesa Standard Block Units with 2 Mesa Standard Connectors and UX1400HS Geogrid and No Gravel





Test No.	Date of Test	Test Specimen Width (in.)	Test Normal Stress (psi)	Equivalent Normal Load (lb/ft)	Approx. No. of Blocks	Approx. Wall Height (ft)	0.75-in. Strength (lb/ft)	Peak Strength (lb/ft)	Shear Strength Equations (S)
B1A	4/2/2001	18.0	1.2	160	3	2	584	758	$S_{0.75\text{-in.}} = 565 + (N) \tan(33°)$
B1B	4/3/2001	18.0	2.9	378	7	5	870	1013	$S_{peak} = 705 + (N) \tan(33°)$
B1C	4/23/2001	18.0	7.0	924	18	12	1223	1329	
B1D	4/24/2001	18.0	14.3	1890	37	24	1780	1927	
B1E	7/28/2001	18.0	2.9	378	7	5	825	943	

NOTES:

For each block shear test, rotation and shearing of Mesa standard connectors were observed at the completion of testing.

Tests B1A through B1D used connectors from 8-cavity mold and Test B1E used connectors from modified 12 cavity mold.

DATE TESTED: 4/2 to 7/18/01

SGI TESTING SERVICES, LLC

FIGURE NO.	C-1
PROJECT NO.	SGI1007
DOCUMENT NO.	SGI01143
FILE NO.	S1007 blockshear

Appendix 1.1.13.1

MESA System Brochure

MESA® MSE Retaining Wall System

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MESA®
RETAINING WALL SYSTEMS

MESA®
RETAINING WALL SYSTEMS

SYSTEM OVERVIEW



Tensar®

Mesa® Retaining Wall Systems provide the dependability engineers require, the efficient installation contractors expect, and the aesthetics owners and architects demand.



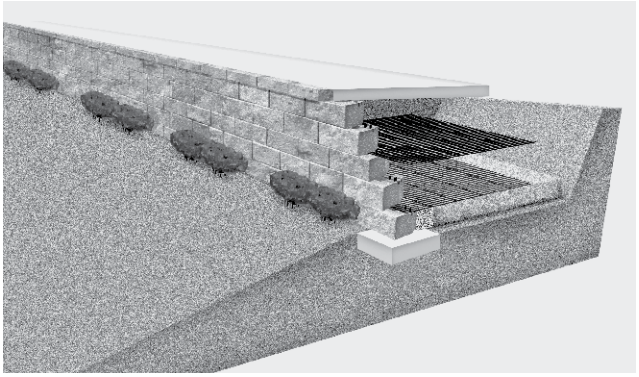
Tensar® Geogrids

Tensar International Corporation's Grade Separation Solutions owe their long-term performance and durability to high-strength Tensar® Uniaxial (UX) Geogrids. With their stiff interlocking capabilities, these geogrids stand the test of time, performing better than other commercially available geosynthetics.

The Solution You Can Count On™

A SINGLE-SOURCE SOLUTION

Since 1999, Mesa® Retaining Wall Systems have been the retaining wall solution of choice for many architects, engineers and DOTs. The first segmental retaining wall (SRW) system with a positive mechanical connection between the geogrid and the wall face, Mesa Systems offer superior and cost-effective solutions for both structural and landscape retaining wall needs in the transportation, industrial, commercial and residential markets.



With a network of licensed independent block manufacturers throughout the United States, Canada and Latin America, Mesa Systems have become the standard in SRW technology. A truly integrated solution, they are the only SRW system where block, geogrid and connector have been developed by one company; unlike other SRWs, the components of the Mesa Systems have been specifically designed to work together for optimum efficiency and performance. High-strength, low-absorption concrete units, high-shear strength connectors and Tensar® Geogrids work together to form a Mechanically Stabilized Earth (MSE) system that meets or exceeds the industry standard. With the Mesa Systems' high connection strength and reliability, core fill* is rarely needed. Using less core fill reduces imported stone and labor requirements, resulting in greater project savings.

For long-term durability, increased structural integrity and simplified construction, all at costs lower than most conventional alternatives, specify the Mesa Retaining Wall Systems for your next project.

** Core fill is defined as the aggregate fill within the open void space of a SRW block. Core fill is recommended for wall segments that form convex curves with a radius less than 25 ft (7.6 m) and 90° outside corner units.*

Mesa Systems' Standard Components	
Component	**Function**
Tensar Geogrids	High-density polyethylene (HDPE) structural geogrids internally reinforce fill materials. Inert to chemical degradation, Tensar Geogrids can be used with non-select fill or even recycled concrete.
Mesa Segmental Units	High-strength concrete block with a compressive strength that exceeds the National Concrete Masonry Association (NCMA) standards (>4,000 psi) (27.6 MPa).
Mesa Connectors	Unique locking connectors designed to mechanically connect Tensar Geogrids to the Mesa Units. Connectors provide a low-strain, end-bearing connection that is not dependent on friction for structural integrity and allows walls to be built near-vertical or with a ⅝ in. (1.6 cm) setback.
Full Engineering and Construction Services	Detailing, design, site assistance and stamped drawings for each Mesa Wall project upon request.



Full Line of Products

STRUCTURAL AND LANDSCAPE SOLUTIONS

Whether you're increasing usable land or improving property value, the Mesa® Systems can solve your most challenging grade change requirements with a full line of products. From building large structural walls to small, tiered garden installations, Mesa Units blend effortlessly with the natural surroundings of any site.

AESTHETICALLY VERSATILE

Whether you're creating stairs, 90° corners, or convex or concave curves, Mesa Walls can easily accommodate a variety of design considerations. Based on your aesthetic requirements, Mesa Units can be varied by color and texture, and walls can even be built mixing different facing options. Designs are nearly limitless – architects and designers are bound only by their imaginations.

In addition to the units shown here, we offer Landscape and XL Units as well as products for the do-it-yourself market. Consult your local licensed block manufacturer for the availability of each type of Mesa Unit.



Mesa Components

Mesa Standard and DOT Connectors



The Mesa Standard Connectors positively connect Tensar® Geogrid to the Mesa Facing Unit, often eliminating the need for core fill.



The Mesa DOT Connector engages the geogrid across the entire block width.



Standard Units
The most popular Mesa Unit in our full line of products. The Standard Unit can be used for most retaining wall needs.

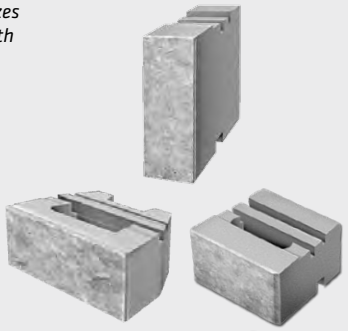
8" h x 18" w x 11" d nom./75 lbs
(20 cm x 46 cm x 28 cm/34 kg)

Ashford™ Units
Create randomly patterned walls using the Mesa® Ashford™ System. Units are available in three sizes for a variety of facing options (available only with a straight split).

Standard Unit: 8" h x 18" w x 11" d nom./75 lbs
(20 cm x 46 cm x 28 cm/34 kg)

Medium Unit: 8" h x 12" w x 11" d nom./60 lbs
(20 cm x 30 cm x 28 cm/27 kg)

Tall Unit: 16" h x 6" w x 11" d nom./60 lbs
(40 cm x 15 cm x 28 cm/27 kg)





Corner Units
Used to create walls with clean and precise 90° corners.

8" h x 18" w x 9" d nom./60 lbs
(20 cm x 46 cm x 23 cm/27 kg)



Cap Units
Used at the top of the wall for a finished look.

4" h x 18" w x 12" d nom./40 lbs
(10 cm x 46 cm x 28 cm/18 kg)

NOTE: *Actual dimensions may vary depending on location.*

› With the Mesa® Systems' structural mechanical connection, you're assured of connection integrity where it's needed most – at the wall face.



Tried and True Transportation Solutions

The long-term performance of any retaining wall system is challenged most rigorously in the public transportation market. Differential settlement, traffic barriers and seismic loads can test the capabilities of any retaining wall, so connection strength must be especially reliable. To ensure that Mesa® Systems meet and exceed the stringent standards of the transportation market, Tensar developed the Department of Transportation (DOT) Connector. With the Mesa Systems' DOT Connector, you're assured of a mechanical connection where it's needed most – at the wall face.

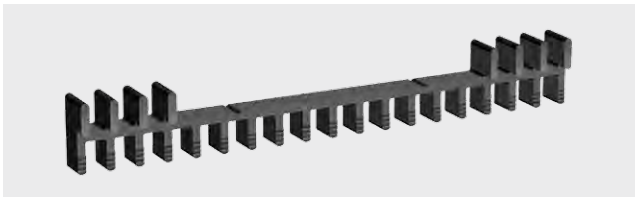
EXCEPTIONAL PERFORMANCE AND INTEGRITY

The Mesa Systems' DOT Connector was designed to meet the long-term performance requirements under current American Association of State Highway and Transportation Officials (AASHTO), Load and Resistance Factor Design (LRFD) and Allowable Stress Design (ASD) criteria. This connector structurally attaches geogrid to the Mesa Systems' concrete facing unit. The system and connection components are largely unaffected by abrasion, tearing or hydrolysis – factors that can undermine the performance of a frictional system using a woven polyester reinforcement. When specifying a Mesa System, you can be assured that its structural components are both reliable and designed for the long term.

RECOGNIZED FOR ITS HIGH STANDARDS

In 2000, the Highway Innovative Technology Evaluation Center (HITEC) completed its assessment of various Mesa Retaining Wall Systems' configurations. (Unbiased performance reviews conducted by HITEC provide information that helps Departments of Transportation make informed purchasing decisions.) The evaluation of the Mesa Systems as a geosynthetic-reinforced retaining wall solution is often a determining factor for their inclusion in transportation project specifications. Since the 2000 evaluation, Mesa Systems have undergone additional installation and design improvements. Many of the improvements made have been accepted and are being utilized by leading DOTs.

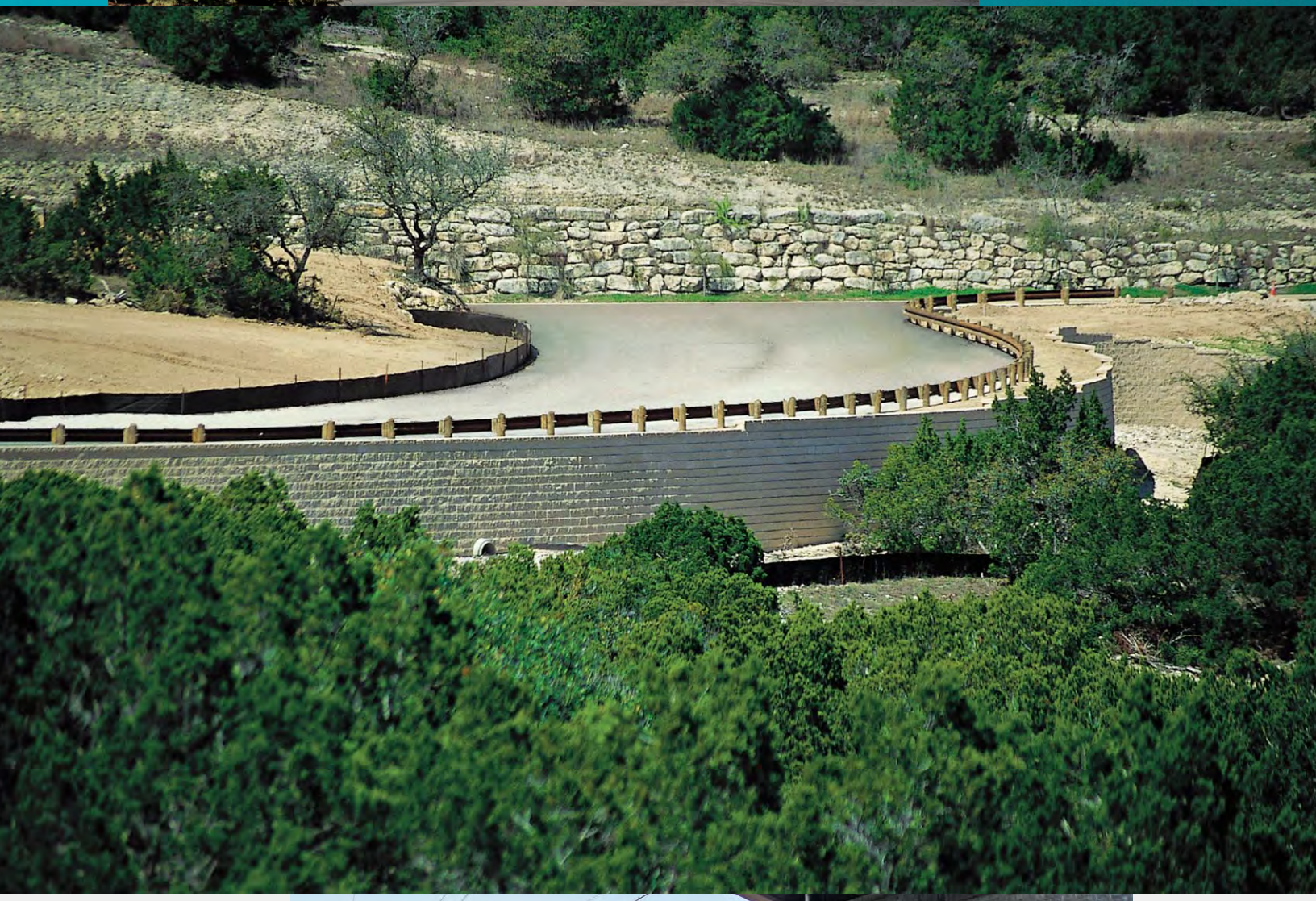
To obtain a copy of the Mesa Systems' HITEC evaluation, please call **800-TENSAR-1**.



The Mesa Systems' DOT Connector engages the geogrid across the entire block width.









Adaptable to a variety of environments and industry demands, a Mesa® Wall combines knowledge with creativity and functionality, providing the ideal solution for any setting.



Creative Commercial Projects

When designing for commercial applications, the need to combine design performance, practicality and cost efficiency becomes a necessity. That's where the Mesa® Systems can help. After all, a Mesa Wall offers both aesthetic appeal and structural stability, providing the ideal solution for almost any commercial project.

ENDLESS APPLICATIONS, COMMERCIAL AND MORE

- **Commercial Developments:** Retail malls, big box stores and office complexes
- **Municipal Projects** – Schools, hospitals, government buildings, public parks and libraries
- **Stormwater Management** – Detention ponds, dry ponds and extended storage ponds, including brackish and salt water environments
- **Recreational Jobs** – Golf courses, amusement parks, amphitheaters and sports facilities

BUILDING ON CODE

Local building codes are applicable to almost all retaining walls in the commercial, industrial and even residential markets. Most municipalities in the U.S. regulate, enforce and inspect the design standards for SRWs that exceed four feet in height. Conforming to these codes is imperative. If codes are not met, the chance of wall failure and/or fines is increased. We're proud that various configurations of the Mesa Systems have been rigorously evaluated by local and state governments across North America.

***NOTE:** Please consult with a local geotechnical engineer to determine the suitability of a Mesa Systems' installation for your particular needs.*










PGE PARK
11 AT BAT
CUBS
BEAVERS
BUD LIGHT


T.J.maxx
A.C. MOORE
NOW OPEN
PETSMART
Now Open!



Durable Tensar HDPE Geogrids are unaffected by aggressive soil conditions and resist chemical, biological and environmental degradation.



Up to the Task at Industrial Sites

Retaining walls are vital to the operational efficiency of industrial sites worldwide. They must be designed to be durable, with the ability to carry heavy loads and repeated traffic for many years. Mesa® Retaining Wall Systems featuring Tensar® UX Geogrid address grade separation and earth retention needs on and adjacent to a variety of industrial sites to keep operations running safely and smoothly.

When specifying a Mesa Wall, you can be assured that its structural components are reliable and designed for the long term. And with soil reinforcement that is 100% polymeric, Mesa Walls also allow structures to be built using a wide range of backfills, including recycled materials, for greater economy.

Tensar UX Geogrids are manufactured using select grades of HDPE resins that can be highly oriented to resist elongation when subjected to heavy loads for long periods of time. These geogrids carry large tensile loads applied in one direction, while their open aperture structure interlocks with natural fill materials, making them ideal for MSE walls required at industrial sites.

BENEFITS OF MESA® RETAINING WALL SYSTEMS

- Durable geogrids that are unaffected by aggressive soil conditions and resist chemical, biological and environmental degradation
- The ability to use a variety of backfill soils, including those with chlorides, sulfates, and high and low pH levels
- Rapid and economical construction, without the need for specialized equipment or labor
- Reliable structures that require little or no maintenance
- Resistance to impact loading and seismic activity
- Reduced need for expensive foundation treatments due to low bearing pressure

In addition, Tensar's team of professionals can provide a full range of services including initial planning, engineered drawings and on-site assistance. We offer solutions that combine technology, engineering, design and materials to meet the unique requirements of industrial operations and applications.









Reliable Residential Applications

Mesa® Walls provide a variety of solutions for single-family and multi-family home developments. Whether you need a structural or landscape wall, Mesa Units blend with the natural surroundings to combine creativity with functionality and provide a solution for almost any residential application. By building a Mesa Wall, you're adding property value to your land – it's that simple.

POWER OF THE POSITIVE

There's a lot to be said for beauty and versatility in the residential market. But you want to be sure that your wall withstands the test of time.

With structural walls, it can all come down to connection. If you compromise connection strength, you risk wall failure. Most other segmental retaining wall systems rely on aggregate interlock or friction between the components to hold the geogrid in place, calling it a "connection." By contrast, the patented, mechanical Mesa Connector provides a block-to-geogrid connection that does not rely on friction or the weight of the Mesa Unit to achieve connection strength.

MORE THAN JUST LANDSCAPE WALLS

Beyond structural walls, your property can be enhanced using simple landscape wall designs. Mesa Systems offer aesthetic options that complement any architectural style and are easier to maintain than conventional wall alternatives. Unlike treated lumber and railroad ties, Mesa Units will not rot or warp. Their high-compression strength concrete resists insect infestation, assuring a virtually maintenance-free and environmentally friendly structure that is sure to please any property owner.



The Mesa Systems' Standard Connector holds Tensar® Geogrid securely to the Mesa Block.



TensarSoil-PRO™ Software helps you make sound technical decisions based on specific site requirements while continuing to optimize the overall design of Mesa® Retaining Wall Systems.



Designing with TensarSoil-PRO™ Software

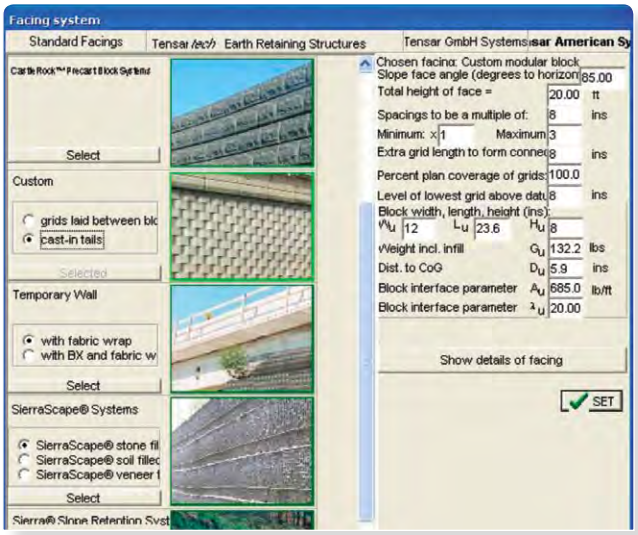
We're proud to offer TensarSoil-PRO™ Software to engineers and architects interested in state-of-the-art technology for designing Mesa® Retaining Wall Systems. This unique application has the flexibility to adjust design input variables to meet project specifications while helping you make sound technical decisions based on your specific site requirements.

TensarSoil-PRO Software offers versatile technology that enables you to design Mesa Walls in accordance with all major industry-standard protocols including:

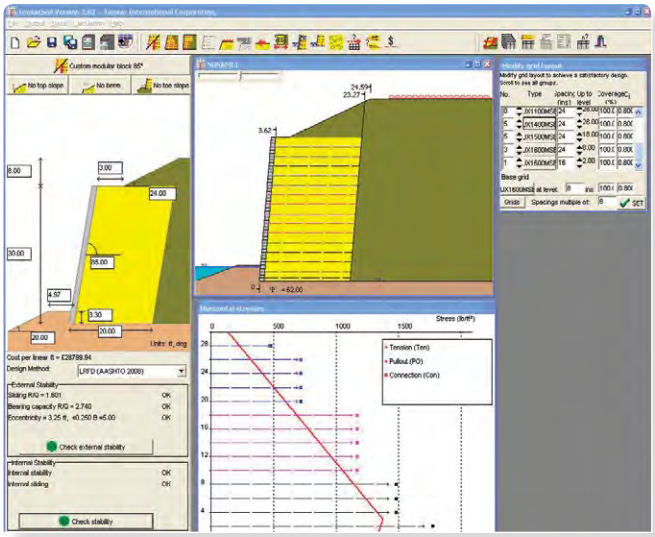
- National Concrete Masonry Association – (NCMA) 1997
- Federal Highway Administration – Demonstration Project 82 (1997)
- Federal Highway Administration – National Highway Institute (FHWA NHI 043) 2001 (AASHTO ASD) 2002
- American Association of State Highway and Transportation Officials – Load and Resistance Factor Design (AASHTO LRFD) 2010

This innovative software includes data on all of our retaining wall systems. Fully interactive, it allows you to input and easily alter wall geometry, geogrid grade or layout, surcharge load and/or soil characteristics – all on a single screen image – to determine stability data and material costs instantaneously. With each change, results are updated in real time. For Mesa Walls, that means that any under-designed area is immediately indicated, with no need to back-trace through previous screens for recalculation. Once internal and external stability parameters have been determined, design data can be exported to TensarSlope™ Software, our slope stability application, for comprehensive, compound and global stability analyses.

No other application offers the capabilities you'll find with TensarSoil-PRO Software. Training workshops are available through your local sales representative. In addition, a hands-on demonstration is available for TensarSoil® Software, which features two design methodologies for the analysis of Mesa Retaining Wall Systems, the SierraScape® Retaining Wall System and the Tensar® Temporary Retaining Wall System.



TensarSoil-PRO™ Software allows you to design prototypes and compare costs of different retaining wall solutions.



Several windows can be simultaneously displayed on a full screen. Windows can be re-dimensioned or repositioned for graphic emphasis, presentation purposes and more.

Standard Mesa® Systems Design

PLEASE NOTE: *The following information is provided for general illustration purposes only and does not constitute engineering advice. Final designs should be executed only by a qualified professional engineer providing sealed drawings, calculations and detailed installation requirements.*

USING THE CHARTS

The generalized design charts on pages 12-13 address six different design scenarios with wall elevations ranging from 4 ft (1.2 m) to 14 ft (4.2 m) and increasing in height incrementally by 2 ft (600 mm). The design scenarios alter the wall batter, backfill soil type and loading conditions. Understanding these different scenarios is important for selecting the most appropriate solution for your specific design.

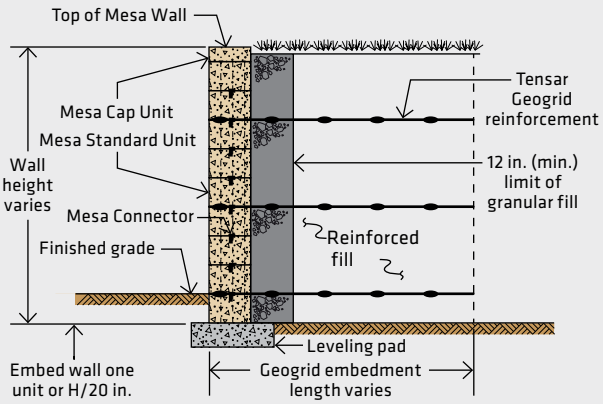
- **Wall Batter** – Mesa® Walls can be constructed at either near-vertical or 4.5° batter
- **Soil Types** – The two backfill soil types are a sand material (32°) and a silty sand or clayey sand (28°) that meet a minimum gradation and plasticity recommendation provided by NCMA
- **Loading Conditions** – The three loading conditions are:
  1. A horizontal surface at the top of the wall with no surcharge load
  2. A horizontal surface at the top of the wall with a uniform surcharge of 180 psf
  3. A 3H:1V slope on top of the wall

Once the most appropriate design has been selected, the charts will provide the suggested geogrid type, embedment length and geogrid spacing. All lengths listed are measured from the wall face to the last transverse bar* on the Tensar® Geogrid and are uniform throughout the given elevation of the wall.

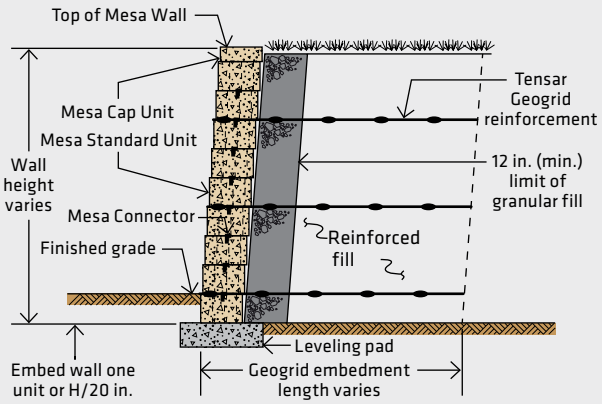
The design charts assume that the walls are constructed in accordance with the Mesa® Systems' standard specification and installation guidelines. Other requirements and limitations based on actual site conditions may also apply.

**The transverse bar is the solid section of the Tensar® UX Geogrid, approximately 3/4 in. wide, located parallel to the face of the retaining wall and in a repeat pattern at a 6 in. to 20 in. spacing (depending upon the type of UX Geogrid being evaluated).*

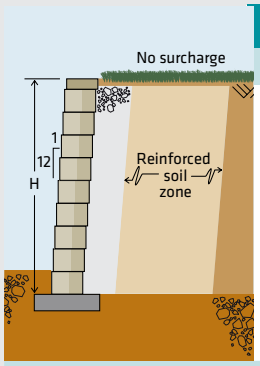
Near-Vertical Typical Cross-Section



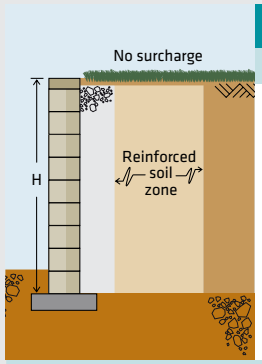
5/8 in. Setback Typical Cross Section



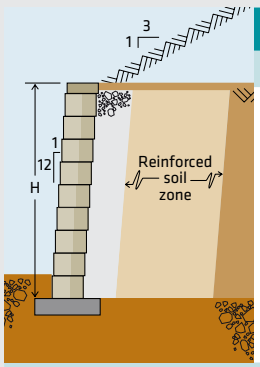
Mesa® Design Charts



REINFORCED BACKFILL SOIL TYPE												
	Sand: Ø=32°; γ=120 pcf; c=0						Silty Sand/Clayey Sand: Ø=28°; γ=120 pcf; c=0					
Wall Height (H)	H=4 ft	H=6 ft	H=8 ft	H=10 ft	H=12 ft	H=14 ft	H=4 ft	H=6 ft	H=8 ft	H=10 ft	H=12 ft	H=14 ft
Geogrid Length (L)	L=4 ft	L=4 ft	L=6 ft	L=6.5 ft	L=7.5 ft	L=8 ft	L=4 ft	L=4 ft	L=6 ft	L=6.5 ft	L=8 ft	L=9.5 ft
Geogrid Type	LH800	LH800	LH800	UX1100	UX1100	UX1100	LH800	LH800	UX1100	UX1100	UX1100	UX1400

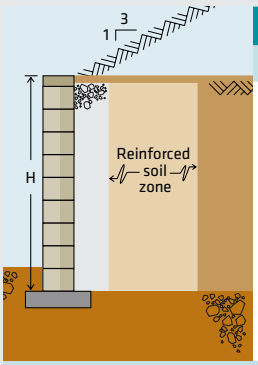


REINFORCED BACKFILL SOIL TYPE												
	Sand: Ø=32°; γ=120 pcf; c=0						Silty Sand/Clayey Sand: Ø=28°; γ=120 pcf; c=0					
Wall Height (H)	H=4 ft	H=6 ft	H=8 ft	H=10 ft	H=12 ft	H=14 ft	H=4 ft	H=6 ft	H=8 ft	H=10 ft	H=12 ft	H=14 ft
Geogrid Length (L)	L=4 ft	L=4 ft	L=6 ft	L=6.5 ft	L=7.5 ft	L=8.5 ft	L=4 ft	L=6 ft	L=6 ft	L=6.5 ft	L=8 ft	L=9.5 ft
Geogrid Type	LH800	LH800	LH800	UX1100	UX1100	UX1400	LH800	LH800	LH800	UX1100	UX1100	UX1400

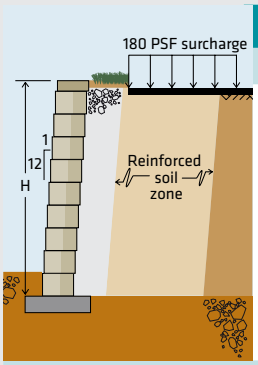


REINFORCED BACKFILL SOIL TYPE												
	Sand: Ø=32°; γ=120 pcf; c=0						Silty Sand/Clayey Sand: Ø=28°; γ=120 pcf; c=0					
Wall Height (H)	H=4 ft	H=6 ft	H=8 ft	H=10 ft	H=12 ft	H=14 ft	H=4 ft	H=6 ft	H=8 ft	H=10 ft	H=12 ft	H=14 ft
Geogrid Length (L)	L=4 ft	L=6 ft	L=6 ft	L=7.5 ft	L=9 ft	L=10.5 ft	L=4 ft	L=6 ft	L=8 ft	L=9.5 ft	L=11 ft	L=13 ft
Geogrid Type	LH800	LH800	LH800	UX1100	UX1100	UX1400	LH800	LH800	UX1100	UX1100	UX1400	UX1400

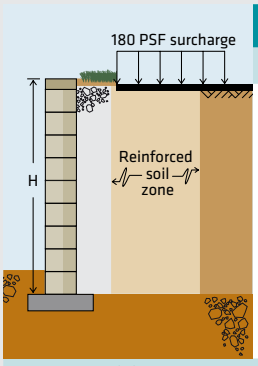
NOTE: *All Tensar Uniaxial Geogrids shown are UXMSE Geogrids.*



REINFORCED BACKFILL SOIL TYPE												
	Sand: Ø=32°; γ=120 pcf; c=0						Silty Sand/Clayey Sand: Ø=28°; γ=120 pcf; c=0					
Wall Height (H)	H=4 ft	H=6 ft	H=8 ft	H=10 ft	H=12 ft	H=14 ft	H=4 ft	H=6 ft	H=8 ft	H=10 ft	H=12 ft	H=14 ft
Geogrid Length (L)	L=4 ft	L=6 ft	L=6 ft	L=7 ft	L=8.5 ft	L=10 ft	L=4 ft	L=6 ft	L=7.5 ft	L=9 ft	L=11 ft	L=12.5 ft
Geogrid Type	LH800	LH800	LH800	UX1100	UX1100	UX1100	LH800	LH800	UX1100	UX1100	UX1100	UX1400



REINFORCED BACKFILL SOIL TYPE												
	Sand: Ø=32°; γ=120 pcf; c=0						Silty Sand/Clayey Sand: Ø=28°; γ=120 pcf; c=0					
Wall Height (H)	H=4 ft	H=6 ft	H=8 ft	H=10 ft	H=12 ft	H=14 ft	H=4 ft	H=6 ft	H=8 ft	H=10 ft	H=12 ft	H=14 ft
Geogrid Length (L)	L=4 ft	L=5 ft	L=6 ft	L=7.5 ft	L=8.5 ft	L=10 ft	L=6 ft	L=6 ft	L=8 ft	L=9.5 ft	L=11 ft	L=12 ft
Geogrid Type	LH800	LH800	LH800	UX1100	UX1100	UX1400	LH800	LH800	UX1100	UX1100	UX1400	UX1400



REINFORCED BACKFILL SOIL TYPE												
	Sand: Ø=32°; γ=120 pcf; c=0						Silty Sand/Clayey Sand: Ø=28°; γ=120 pcf; c=0					
Wall Height (H)	H=4 ft	H=6 ft	H=8 ft	H=10 ft	H=12 ft	H=14 ft	H=4 ft	H=6 ft	H=8 ft	H=10 ft	H=12 ft	H=14 ft
Geogrid Length (L)	L=4 ft	L=6 ft	L=6 ft	L=7.5 ft	L=8.5 ft	L=10 ft	L=6 ft	L=6 ft	L=8 ft	L=9.5 ft	L=11 ft	L=12 ft
Geogrid Type	LH800	LH800	LH800	UX1100	UX1100	UX1400	LH800	LH800	UX1100	UX1100	UX1400	UX1400

For further information, please refer to the TensarSoil-PRO™ Software.

Mesa® Walls are more than a site solution; they create a visually appealing landscape feature on your site.



The Many Faces of Mesa® Retaining Wall Systems

UNLIMITED DESIGN AND AESTHETIC OPTIONS

If your site requires a retaining wall, why not make it as attractive and useful as possible? In today's competitive and ever-changing wall market, architects and owners are continually looking for new ways to build walls that look as good as they perform.

Mesa® Systems are at the forefront of the industry with a wide array of design options. From blending and variegating colors and textures to planting greenery on tiered walls and top slopes, segmental retaining walls become more than a structural element – they become an appealing aesthetic feature of your site.

MESA® ASHFORD™ SYSTEM

Recent design and architectural trends are calling for solutions with a more natural and aesthetic appeal. Segmental retaining walls are no exception.

The Mesa® Ashford™ System is meeting the demand with an ashlar-inspired, mosaic-like pattern facing. Available in the same colors, blends and textures as other Mesa Units, Ashford Units can be configured in a number of ways. Walls can be built to a near-vertical facing angle equivalent to a 4.5° batter. They can also be installed to create 90° corners, stairs and dramatic serpentine curves.

The Mesa Ashford System's unique top and bottom connector slots allow the flexibility to design almost any look while maintaining the Mesa Systems' unique positive mechanical connection, which assures a stronger facing connection to the geogrid reinforcement.

For additional information on suggested Mesa Ashford wall patterns and how to calculate estimated block quantities, please contact your local sales representative.







DESIGNING FOR SPECIAL CONSIDERATIONS

Serpentine walls blend with naturally curving contours, while inside and outside corners complement the traditional angular look of existing structures. With the Mesa® Systems, your design options have no bounds.

- **Curves and Corners** – Designing for inside, outside and serpentine curves as well as 90° inside and outside corners is easy since the unique locking connector makes alignment and installation quick and simple.
- **Steps** – Whether you require steps inside or in front of your wall, the Mesa Systems are up to the challenge. By offering different size and style units to aid in your project requirements, virtually any design can be achieved.
- **Tiered and Terraced Walls** – A less obtrusive alternative than conventional wall solutions, tiered and terraced walls can be built in areas with sufficient land space. For each wall to be considered an independent structure, typical design guidelines require enough "green space" between the walls equal to or greater than two times the height of the lower wall.
- **Facing Combinations** – The blend of hardscape and softscape (vegetated face) is often preferred on projects that require an aesthetically green look.

For more information on design and construction of these or other special considerations, please refer to the *Mesa Systems Installation and Special Considerations Manual*. Other requirements and limitations based on actual site conditions may also apply.







Standard Installation Procedures

The following steps provide a generalized guideline for installing a Mesa® Retaining Wall System with the Mesa Standard Unit. These steps will take you through a typical installation from start to finish.

Additional information is contained within the *Mesa Systems Installation and Special Considerations Manual* available at **www.tensarcorp.com**.

NOTE: *Specific requirements and installation steps for your project are governed by the final drawings sealed by a registered professional engineer and the project specifications. Another useful document which should be consulted prior to final design and construction is the* NCMA Segmental Retaining Wall Drainage Manual (2002).







Step 1:
PRECONSTRUCTION PREPARATION

It's important to be familiar with the components of the Mesa® Systems prior to the start of construction. Below is a list of these components and the tools needed to aid you in the construction of a standard Mesa Wall. Mesa Corner Units, drainage composite, piping and geotextile materials may also be required.

MESA COMPONENTS:

- Mesa Block
- Mesa Connector
- Tensar® Geogrid
- Mesa Cap Unit (if applicable)
- Concrete adhesive (if applicable)

SUGGESTED TOOLS FOR INSTALLATION:

- Dead blow hammer
- 2- to 4-ft level
- Utility saw and/or grinder
- Masonry string and chalk line
- Pitchfork (used to help remove slack from geogrid)
- Shovels
- Compaction equipment

Step 2:
PREPARE THE LEVELING PAD

Prepare the subgrade by excavating or filling vertically to plan elevation and horizontally to design geogrid lengths. If the excavated material can meet the reinforced fill requirement, then it can be stockpiled with surface vegetation and debris removed prior to backfill. Start the leveling pad at the lowest elevation of the wall. Level the prepared base with 6 in. of unreinforced concrete or well-compacted granular fill (gravel, road base or ¾ in. minus crushed stone). The leveling pad is typically 12 in. wider than the Mesa Unit, extending 6 in. in front of and 6 in. behind the unit. Compact the well-graded stone in accordance with project plans and specifications. Aggregate leveling pads are generally overbuilt and should be carefully trimmed down to meet the proper elevation.

Steps in the leveling pad are required to change elevation. It is important that the height of the step is equal to the height of the number of unit courses. If a concrete leveling pad is used, it is important to have the step-up heights match the Mesa Unit's height exactly. If not, grinding and/or shimming may be required.

NOTE: *The leveling pad requirements and the suitability of the foundation soil for your project should be determined by a qualified professional engineer engaged by the installer on the project.*

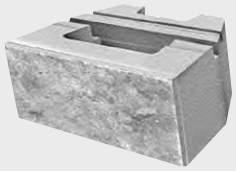
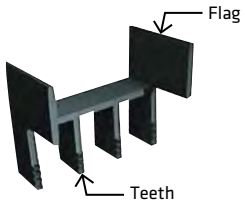
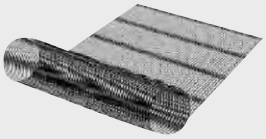
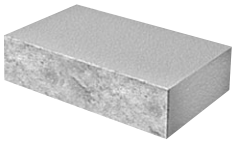
Step 3:
INSTALL THE BASE COURSE

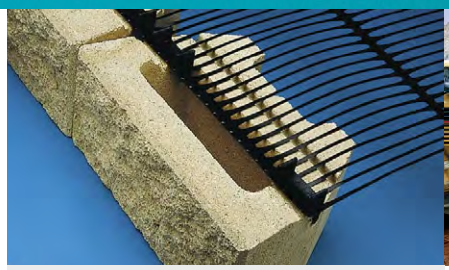
Once the pad is in place, begin by making a wall line where the units will be positioned. Chalk works well for concrete, while string works well for aggregate. Place the first course of Mesa Units tightly together, with the sides touching and the textured face outward. The first course must be accurately placed to align with the string line, carefully spaced and leveled to facilitate construction and enhance the appearance of the wall. The tail of the unit should always be used to align the wall face. Occasionally, a unit will have a slight difference in height. If this occurs, the rib or fingers of the geogrid may be used to shim the face or tail back to level.

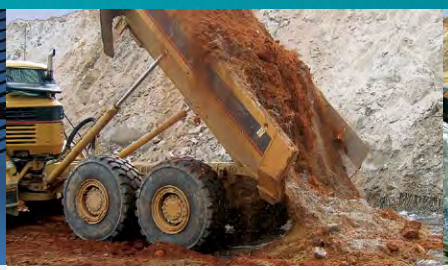
NOTE: *Surface or subsurface water should never be allowed to saturate the backfilled reinforcement zone. Adequate drainage measures, the testing of site soils and backfill compaction, and overall construction quality control are responsibilities of the owner or owner representatives (not Tensar).*

NOTE: *A qualified professional geotechnical engineer should evaluate the site, surface and subsurface conditions, other environmental factors and the intended use and location of the wall in advance of final wall design and installation.*

Mesa® Systems' Standard Components:

Mesa® Standard Unit	Mesa Connector	Tensar® UX Geogrid	Mesa Cap Unit
			
FIGURE 1	FIGURE 2	FIGURE 3	FIGURE 4







Step 4:
PLACE GEOGRID AND CONNECTORS

Prior to the placement of additional courses, and depending on the block being used, insert two Mesa® Connectors (Figure 2) into each preceding Mesa Unit (Figure 1). The orientation of the exposed portion of the connector, or flag, will create the batter of the wall face. If the design dictates the need for geogrid at a particular elevation, install Tensar® UX Geogrid (Figure 3) with the Mesa Connectors penetrating through the geogrid apertures. Snug the geogrid against the connector teeth, then drive the connector the rest of the way with a rubber mallet.

***NOTE:** The geogrid's transverse bar must be pulled taut against the connectors prior to final seating of the connector into the block. The geogrid may be held in place by anchoring it with stakes or rebar at the tail end.*

Once the connectors and geogrid (as required) are in place, the Mesa Units must be swept clean prior to placing the next course. Failure to do so can result in problems with seating and leveling of subsequent courses. Stone or core fill is not required to be placed inside the Mesa Units for wall segments that are straight, concave or have 90° inside corners.

Step 5:
PLACE AND COMPACT BACKFILL

Install granular fill, typically ¾ in. well-drained stone, behind the wall face as specified in the design drawings (12 in. min.). Pea gravel should not be used for granular fill. Behind the granular fill, use backfill material that meets project specifications. When placing backfill over the geogrid layer, fill should be placed to minimize any slack in the geogrid; placing the fill in a direction away from or parallel to the face of the wall will minimize this slack. In addition, a pitchfork or rake can be used to help remove slack during fill placement.

Loose lifts of the reinforced fill shall not exceed 6 in. where hand compaction equipment is used or 10 in. where heavier compaction equipment is used. These thicknesses may vary depending on the approved project-specific soil types specified. Compact fill to 95% Standard Proctor per ASTM D-698 or as required by the design and contract documents.

***NOTE:** Only hand-operated compaction equipment shall be used within 3 ft of the tail of the Mesa Units; heavy equipment in this area can dislocate the geogrids and face units.*

Step 6:
INSTALL ADDITIONAL COURSES

Place the next block course over the Mesa Connectors onto the previous course, fitting the flags inside the open cavity of the block. Push the unit forward so that it makes contact with the connectors. The vertical joint alignment should be checked frequently as the connectors allow the units to slide from side-to-side. As you build up, maintain level on each course by continually checking for level front-to-back and side-to-side. If necessary, shim back to level with a rib or fingers of the geogrid. Once the course is level, continue to repeat steps 4 through 6 until final elevation is reached.

Step 7:
PLACE CAP UNITS (WHEN REQUIRED)

Mesa® Cap Units (Figure 4) may be placed flush with the face of the wall or with a nominal 1 in. overhang. A concrete adhesive suitable for bonding concrete-to-concrete should be used to secure the units to the course below. The adhesive should be suitable for use in an outdoor environment and stable under the temperature extremes expected for the area. Apply the adhesive in accordance with the manufacturer's instructions.

› By combining beauty and elegance with efficiency and performance, Mesa® Walls are built to stand the test of time.



The Solution of Choice

The Mesa® Systems have increasingly become the solution of choice for transportation, industrial, commercial and residential applications. By combining beauty and elegance with efficiency and performance, Mesa Walls are built to stand the test of time.

Our distribution team, located throughout the United States, Canada and Latin America, is dedicated to providing you with the highest quality products, service and support. With a technically trained field sales staff and an in-house engineering department, Tensar succeeds in keeping its systems at the forefront of today's design technology and market trends.

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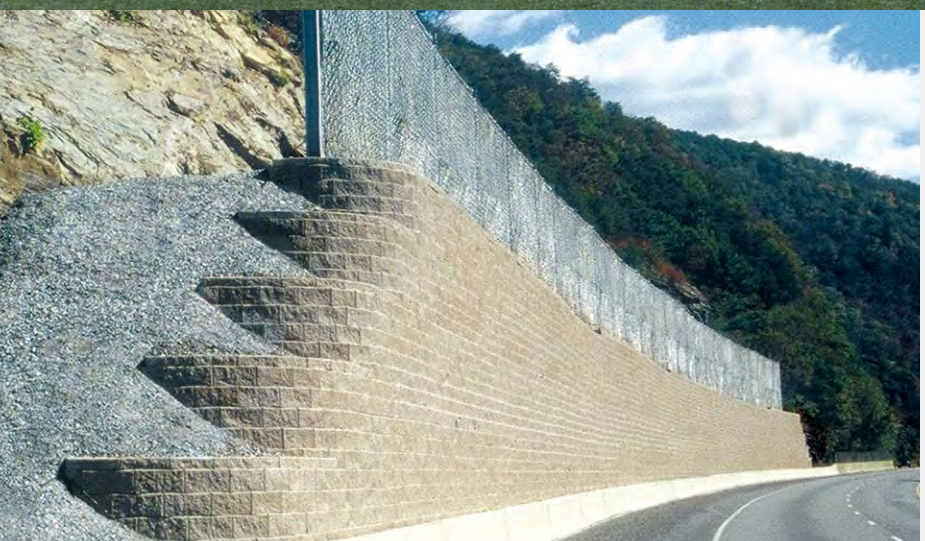












Tensar®

Tensar International Corporation
2500 Northwinds Parkway, Suite 500
Alpharetta, Georgia 30009

800-TENSAR-1
tensarcorp.com

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MESA_BRO_5.14

Appendix 1.2.3.a

75yr-Tensar UX Geogrid

Strength Design Properties

MESA® MSE Retaining Wall System

Back to main document

Tensar®

Tensar Structural Geogrid Strength Properties

75-year Design Life

Ultimate Tensile Strength, Creep Reduced Strength and Installation Damage Factors:

	UX1100		UX1400		UX1500		UX1600		UX1700	
	lb/ft	kN/m	lb/ft	kN/m	lb/ft	kN/m	lb/ft	kN/m	lb/ft	kN/m
Ultimate Stength (T_{ult}), ASTM D6637[2]	3970	58.0	4800	70.0	7810	114.0	9870	144.0	11990	175.0
RF_{creep} (Stress Rupture)	2.68		2.68		2.54		2.54		2.54	
Creep Reduced Strength	1481	21.6	1791	26.1	3075	44.9	3886	56.7	4720	68.9
$RF_{durability}$	1.10		1.10		1.10		1.10		1.10	
$RF_{installation\ damage}$ (Silty Sand)	1.24		1.15		1.14		1.15		1.18	
$RF_{installation\ damage}$ (Sandy Gravel)	1.31		1.35		1.34		1.35		1.39	

Long Term Design Strength (LTDS)[3]

	UX1100		UX1400		UX1500		UX1600		UX1700	
	lb/ft	kN/m	lb/ft	kN/m	lb/ft	kN/m	lb/ft	kN/m	lb/ft	kN/m
In Silty Sand	1086	15.9	1416	20.6	2452	35.8	3072	44.8	3637	53.1
In Sandy Gravel	1028	15.0	1206	17.6	2086	30.4	2617	38.2	3087	45.1

Updated: January 2016

Appendix 1.2.3.b

100yr-Tensar UX Geogrid

Strength Design Properties

MESA® MSE Retaining Wall System

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Tensar®

Tensar Structural Geogrid Strength Properties

100-year Design Life

Ultimate Tensile Strength, Creep Reduced Strength and Installation Damage Factors:

	UX1100		UX1400		UX1500		UX1600		UX1700	
	lb/ft	kN/m	lb/ft	kN/m	lb/ft	kN/m	lb/ft	kN/m	lb/ft	kN/m
Ultimate Stength (T_{ult}), ASTM D6637[2]	3970	58.0	4800	70.0	7810	114.0	9870	144.0	11990	175.0
RF_{creep} (Stress Rupture)	2.73		2.73		2.57		2.57		2.57	
Creep Reduced Strength	1454	21.2	1758	25.6	3039	44.4	3840	56.0	4665	68.1
$RF_{durability}$	1.10		1.10		1.10		1.10		1.10	
$RF_{installation\ damage}$ (Silty Sand)	1.24		1.15		1.14		1.15		1.18	
$RF_{installation\ damage}$ (Sandy Gravel)	1.31		1.35		1.34		1.35		1.39	

Long Term Design Strength (LTDS)[3]

	UX1100		UX1400		UX1500		UX1600		UX1700	
	lb/ft	kN/m	lb/ft	kN/m	lb/ft	kN/m	lb/ft	kN/m	lb/ft	kN/m
In Silty Sand	1066	15.6	1390	20.3	2423	35.4	3036	44.3	3594	52.5
In Sandy Gravel	1009	14.7	1184	17.3	2062	30.1	2586	37.7	3051	44.5

Updated: January 2016

Appendix 1.2.4

National Transportation Product Evaluation Program (NTPEP)

Latest report: May 2014

MESA® MSE Retaining Wall System

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2014 NTPEP Report Series

NTPEP Report REGEO-2012-01-001

LABORATORY EVALUATION OF GEOSYNTHETIC REINFORCEMENT

FINAL PRODUCT QUALIFICATION REPORT FOR TENSAR UX-MSE / UX-HS GEOGRID PRODUCT LINE

Report Issued: May 2014
Report Expiration Date: May 2023
Next Quality Assurance Update Report: 2020

American Association of State Highway and Transportation Officials (AASHTO)

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National Transportation Product Evaluation Program (NTPEP)

NTPEP Report REGEO-2012-01-001

LABORATORY EVALUATION OF GEOSYNTHETIC REINFORCEMENT

2012 PRODUCT SUBMISSIONS
SAMPLED APRIL 2012

Laboratory Evaluation by:

TRI/Environmental, Inc.
9063 Bee Caves Road
Austin, TX 78733-6201

Product Line Manufactured by:

Tensar International Corporation
5883 Glenridge Drive, Suite 200
Atlanta, GA 30328



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PROLOGUE

General Facts about NTPEP Reports:

- NTPEP Reports contain data collected according to laboratory testing and field evaluation protocols developed through consensus-based decision by the AASHTO's NTPEP Oversight Committee. These test and evaluation protocols are described in the *Project Work Plan* (see NTPEP website).
- Products are voluntarily submitted by manufacturers for testing by NTPEP. Testing fees are assessed from manufacturers to reimburse AASHTO member departments for conducting testing and to report results. AASHTO member departments provide a voluntary yearly contribution to support the administrative functions of NTPEP.
- AASHTO/NTPEP does not endorse any manufacturer's product over another. Use of certain proprietary products as "primary products" does not constitute endorsement of those products.
- AASHTO/NTPEP does not issue product approval or disapproval; rather, test data are furnished for the User to make judgment for product prequalification or approval for their transportation agency.

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- The User of this Report must be sufficiently familiar with the product performance requirements and/or (standard) specification of their agency in order to determine which test data are relevant to meeting those qualifying factors.
- NTPEP test data is intended to be predictive of actual product performance. Where a transportation agency has successful historical experience with a given product, it is suggested to factor that precedence in granting or withholding product approval or prequalification.

NTPEP Report Special Advisory for Geosynthetic Reinforcement (REGEO):

- This report contains product data that are intended to be applied to a product line, based on the test results obtained for specific products that are used to represent the product line for the purposes of NTPEP testing. It is expected that the User will estimate the properties of specific products in the line not specifically tested through interpolation or a lower or upper bound approach.
- It is intended that this data be used by the User to add products to their Qualified Products or Approved Products List, and/or to develop geosynthetic reinforcement strength design parameters in accordance with AASHTO, FHWA, or other widely accepted design specifications/guidelines. It is also intended that the User will conduct further, but limited, evaluation and testing of the products identified in this report for product acceptance purposes to verify product quality.
- Products included in this report must be resubmitted to NTPEP every three (3) years for a quality assurance evaluation and every six (6) years for a full qualification evaluation in accordance with the work plan. Hence, all product test results included in this Report supersede data provided in previous Editions of this report.
- The User is guided to read the document entitled "Use and Application of NTPEP Geosynthetic Reinforcement Test Results" (see NTPEP website) for instructions and background on how to apply the results of the data contained in this report.

Tony Allen **(Washington State DOT)**
Chairman, Geosynthetics
Technical Committee

Jim Curtis **(New York State DOT)**
Vice Chairman, Geosynthetics
Technical Committee

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Executive Summary

This test report provides data that can be used to characterize the short-term and long-term tensile strength of the Tensar UX-MSE / UX-HS integrally formed structural high density polyethylene geogrid reinforcement product line using testing conducted on representative products within the product line. The purpose of this report is to provide data for product qualification purposes.

The test results contained herein were obtained in accordance with AASHTO PP 66-10 and the NTPEP work plan (see www.NTPEP.org) and can be used to determine the long-term strength of the geosynthetic reinforcement, including the long-term strength reduction factors RF_{ID}, RF_{CR}, and RF_D, and also used to determine low strain creep stiffness values. All testing reported herein was performed on the materials tested in the direction of manufacture, i.e., the machine direction.

Product Line Description: The product line evaluated includes the following specific integrally formed structural high density polyethylene geogrid reinforcement products:

Tensar UX1100MSE, UX1400MSE, UX1500MSE, UX1600MSE and UX1700MSE

and

Tensar UX1100HS, UX1400HS, UX1500HS, UX1600HS and UX1700HS.

This product line was represented through testing of UX1100MSE, UX1400MSE, UX1500MSE, UX1600MSE and UX1700MSE. UX-MSE and UX-HS geogrids are two styles of geogrids which differ only in dimensional tolerances specific to the cross bar location and orientation. Geogrid products meeting more stringent tolerance criteria are removed from the HS series production and segregated into the MSE inventory. Still, the production process itself remains the same for both types and it is this aspect of same production parameters that enables both styles to be considered one type of product, described as high strength punched drawn high density polyethylene geogrids. Samples of these five products were taken by an independent sampler on behalf of NTPEP in April 2012 at the Tensar manufacturing plant located in Morrow, GA.

An on-site audit to verify the consistency of the Tensar UX-MSE / UX-HS product line was conducted at the Tensar manufacturing plant on April 27, 2012, in accordance with the REGEO work plan. The audit verified that the materials and processing used to manufacture each product in the line are consistent and meet the definition of a product line in the NTPEP work plan and AASHTO PP66-10. The audit report is available separately upon request to those who are authorized to have access to the audit report (i.e., member state departments of transportation, NTPEP staff, and the manufacturer of the product line).

Statistical Validation of Product Line: The creep rupture test results obtained were statistically evaluated in accordance with PP66-10 to assess the validity of treating the products submitted as a single product line. The following was verified:

i. Based on the creep data for all the products tested, with respect to creep rupture UX1100MSE and UX1400MSE statistically qualify as one product line and UX1500MSE, UX1600MSE and UX1700MSE statistically qualify as another product line (see Figures F-44, F-47 and F-48 in Appendix F for details). Recommendations on application of the representative product data to the rest of the product line for installation damage, durability, and creep stiffness are provided in their respective report sections, and summarized below in this executive summary.

Test Results for T_{ult}: All wide width test results (ASTM D6637) obtained for this product line through the NTPEP testing were greater than the minimum average roll values (MARV's) provided by the manufacturer (see Table 3-1).

Test Results for RF_{ID}: Installation damage testing on this product line resulted in values of RF_{ID} that ranged as follows:

$$RF_{ID} = 1.14 \text{ to } 2.57$$

The highest values of RF_{ID} occurred when the coarse gravel gradation was used.

The values of RF_{ID} for all of the products tested in the coarse gravel gradation demonstrated a trend of decreasing RF_{ID} as product unit weight/tensile strength. Therefore, interpolation of these test results (i.e. in coarse gravel) to products in the line not tested is feasible. This trend was not as clear for the other, less coarse, gradations. In general, as the test material gradation becomes more coarse, the value of RF_{ID} for a given product style increased. Therefore, interpolation of this data for a given product style to intermediate gradations appears to be feasible. See Table 4-3 and Figures 4-5 through 4-8 for details. Laboratory installation damage test data in accordance with ISO/EN 10722 are also provided for future use in comparison to quality assurance testing (see Table 4-4).

It should be noted that the installation damage testing conducted represents an increase in compaction and spreading equipment size (i.e., a 15,000 lb wheeled front end loader – Caterpillar 416E, and a 25,000 lb single drum vibratory roller- Caterpillar CS56) and a reduced aggregate lift thickness over the geogrid of 6 inches relative to the installation damage testing reported in previous NTPEP test reports. Therefore, the decrease in strength retained values relative to previous NTPEP test reports for this product line should not be assumed to represent a change in the products, but rather is more likely the result of the more severe installation damage conditions which represent a likely upper bound installation condition for geosynthetic reinforced soil structures. Actual RF_{ID} values could be lower if installation conditions are less severe (e.g., greater initial lift thickness over the geogrid, use of lighter weight equipment, etc.). Actual RF_{ID} values could be higher if the spreading or compacting equipment tires or tracks are allowed to be in direct contact with the geosynthetic before or during fill placement and compaction, if the thickness of the fill material between the equipment tires or tracks is inadequate (especially for high tire pressure equipment such as dump trucks), or if excessive rutting of the first lift of soil over the geosynthetic (e.g., due to soft subgrade soil) is allowed to occur.

Test Results for RF_{CR}: As explained above, relative to creep rupture, statistical evaluations differentiates UX1100MSE and UX1400MSE as having common creep characteristics and UX1500MSE, UX1600MSE and UX1700MSE as having their own common creep characteristics.

The creep rupture testing conducted indicates that the following value of RF_{CR} may be used for UX1100MSE and UX1400MSE:

$$RF_{CR} = 2.68$$

The creep rupture testing conducted indicates that the following value of RF_{CR} may be used for UX1500MSE, UX1600MSE and UX1700MSE:

$$RF_{CR} = 2.54$$

These values of RF_{CR} is applicable to a 75 year life at 68° F (20° C), and may be used to characterize the full product line as defined herein. See Figures 5-1 and 5-2 for detailed creep rupture envelope or to obtain values for other design lives using the regression curve equation.

Test Results for RF_D: The chemical durability index testing results meet the requirements in AASHTO PP66-10 to allow use of a default reduction factor for RF_D. See Table 6-2 for specific test results, and see AASHTO PP66-10 or the document entitled "Use and Application of NTPEP Geosynthetic Reinforcement Test Results" (see www.NTPEP.org) for recommended default reduction factors for RF_D. The UV test results (ASTM D4355) for this product line, as represented by the lightest weight product in the line (i.e., UX1100MSE), indicate a strength retained at 500 hours in the weatherometer of 98%. This value of UV strength retained should be considered to be a lower bound value for the product line, for UX1100MSE or heavier products in the line. Regarding transverse flexibility (WSDOT T926), all products in the product line were tested and all passed the test.

Test Results for Creep Stiffness: The 1000 hr, 2% strain secant stiffness ($J_{2\%,1000hr}$) test results ranged from 26,600 lb/ft for the lowest strength style to 73,400 lb/ft for the highest strength style. There exists a strong polynomial relationship between creep stiffness and the short-term tensile strength (T_{lot}), therefore the 1000 hr, 2% strain secant stiffness can be reasonably expressed for any product in the product line as:

$$J_{2\%,1000\ hr} = 0.00043(T_{lot})^2 - 1.14556(T_{lot}) + 23{,}279$$

Where, T_{lot} is the roll/lot specific single rib tensile strength per ASTM D6637. See Table 7-2 and Figure 7-1 for details. Note that once the stiffness is determined from this equation, an equivalent MARV for this property may be determined by multiplying the stiffness by the ratio of T_{MARV}/T_{lot}.

1.0 Product Line Description and Testing Strategy

1.1 Product Description

The **UX-MSE / UX-HS** family of geogrids are integrally formed punched, drawn uniaxial high density polyethylene geogrids. The product line evaluated consists of the products as manufactured by Tensar International Corporation listed in Table 1-1.

Table 1-1. Product designations included in product line.

UX-MSE and UX-HS Reinforcement Product Designations (i.e., Styles)	
UX1100MSE/HS	UX1600MSE/HS
UX1400MSE/HS	UX1700MSE/HS
UX1500MSE/HS	

Note: UX-MSE / UX-HS geogrids are two styles of geogrids which differ only in dimensional tolerances specific to the cross bar location and orientation. Geogrid products meeting more stringent tolerance criteria are removed from the HS series production and segregated into the MSE inventory. Still, the production process itself remains the same for both types and it is this aspect of same production parameters that enables both styles to be considered as one type of product, described as high strength punched drawn high density polyethylene geogrids.

Since all the products are uniaxial geogrids, the scope of the evaluation is limited to the strength in the machine direction (MD). The cross-machine direction (XD) was not specifically evaluated.

An on-site audit to verify the consistency of the Tensar UX-MSE / UX-HS product line was conducted at the Tensar manufacturing plant on April 27, 2012, in accordance with the REGEO work plan. The audit verified that the materials and processing used to manufacture each product in the line are consistent and meet the definition of a product line in the NTPEP work plan and AASHTO PP66-10. The audit report is available separately upon request to those who are authorized to have access to the audit report (i.e., members state departments of transportation, NTPEP staff, and the manufacturer of the product line).

1.2 Product Line Testing Approach

This product line was evaluated through detailed testing of UX1100MSE, UX1400MSE, UX1500MSE, UX1600MSE and UX1700MSE. For creep testing, UX1100MSE and UX1500MSE were used as the primary products for product line characterization purposes (i.e., the baseline to which the other products were compared). Samples of all tested products were taken by an independent sampler on behalf of NTPEP on April 27, 2012 at the Tensar manufacturing plant located in Morrow, GA.

Photographs of all the products tested are provided in figures 1-1 through 1-5

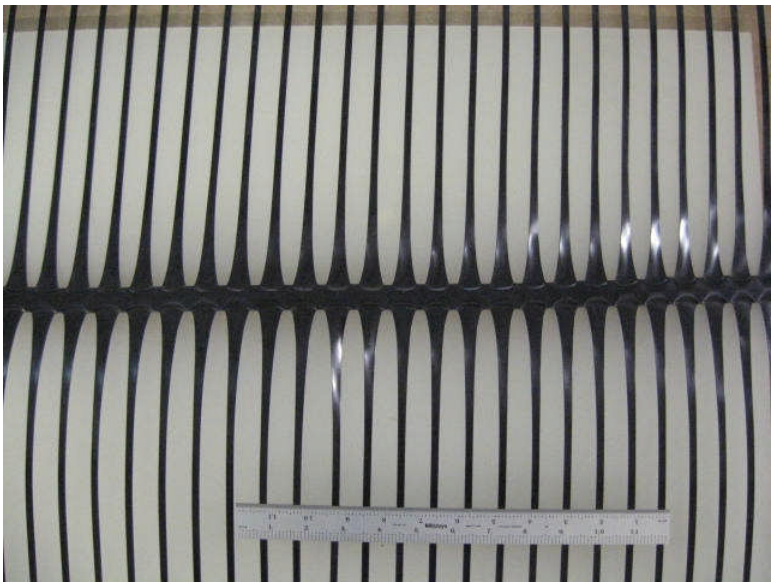


Figure 1-1. Photo of UX1100MSE (machine direction is perpendicular to ruler shown).

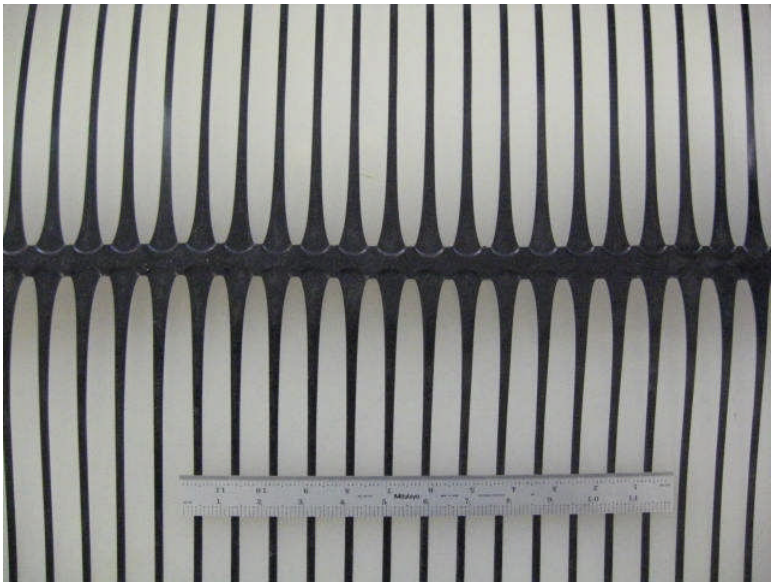


Figure 1-2. Photo of UX1400MSE (machine direction is perpendicular to ruler shown).

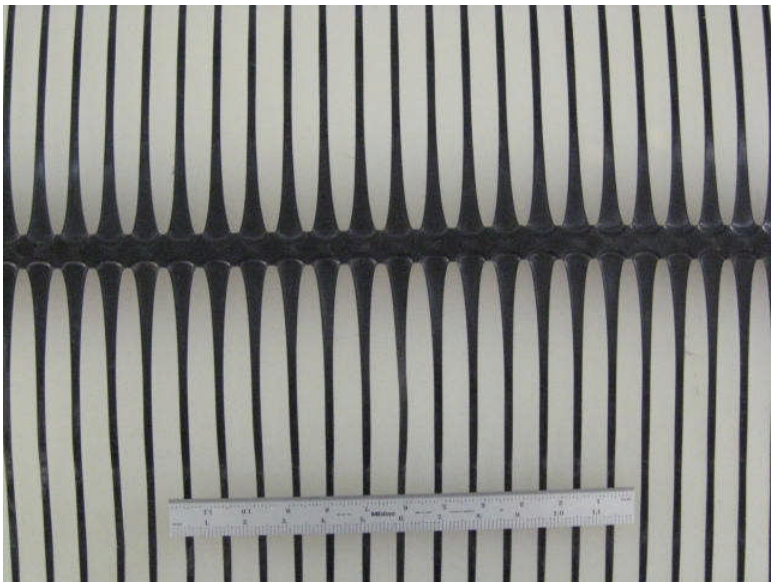


Figure 1-3. Photo of UX1500MSE (machine direction is perpendicular to ruler shown).

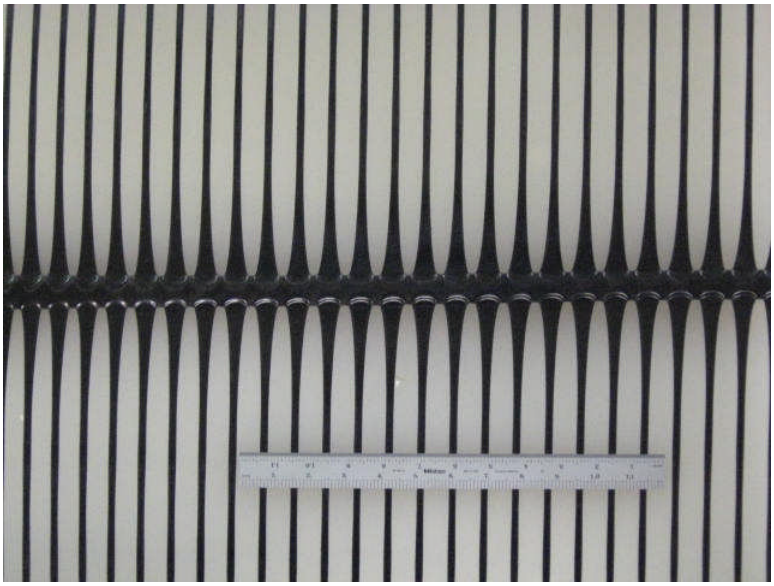


Figure 1-4. Photo of UX1600MSE (machine direction is perpendicular to ruler shown).

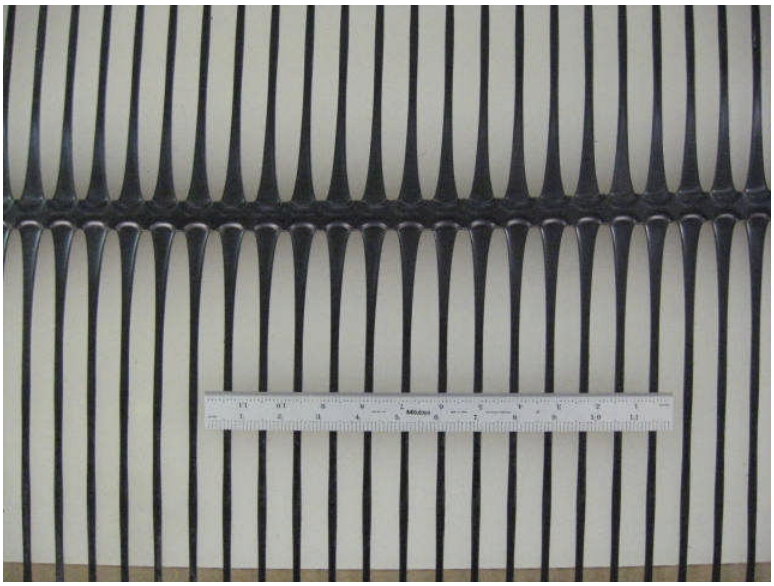


Figure 1-5. Photo of UX1700MSE (machine direction is perpendicular to ruler shown).

2.0 Product Polymer, Geometry, and Manufacturing Information

2.1 Product/Polymer Descriptors

Polymer used in all **UX-MSE / UX-HS** geogrids is 100% high density polyethylene. The source of resin is proprietary. **UX-MSE / UX-HS** geogrids are not coated.

For the HDPE resin, key descriptors include primary resin/class/grade/category per ASTM D 1248 / D41010:

- Resin is Type III / Class A / Grade 5
- % of regrind used in product: A small but unknown percentage of regrind is used.
- % of post-consumer recycled material by weight: 0%

2.2 Geometric Properties of Geogrids

Rib width, spacing, thickness, and product weight/unit area vary depending on geogrid style. While such data are generally not used for design, it can be useful for identification purposes, and to be able to detect any changes in the product. Measurements of geogrid rib spacing are also used to convert tensile test results (i.e., load at peak strength, T_{ult}, and load at a specified strain to obtain stiffness, J) to a load per unit width value (i.e., lbs/ft or kN/m). Detailed measurement results, as well as the typical values supplied by the manufacturer for each product, are provided in Appendix B, Section B.1.

2.3 Product Production Data and Manufacturing Quality Control

Geogrid roll sizes and weights, lot sizes, and a summary of the manufacturer's quality control program are provided in Appendix B, Sections B.2 and B.3. Such information can be useful in working with the manufacturer if product quality issues occur.

3.0 Wide Width Tensile Strength Data

Minimum average roll values supplied by the manufacturer and test results obtained on all the products in the product line for this NTPEP testing program are provided in Table 3-1. Wide width tensile tests were conducted in accordance with ASTM D6637. The measured geogrid dimensions discussed in Section 2 and provided in Appendix B, Section B.1, were used to convert test loads to load per unit width values. Note that the independently measured T_{ult} values only indicate that the sampled products have a tensile strength that exceeds the Manufacturer's minimum average roll values (MARV's). As such, these independently measured T_{ult} values should not be used directly for design purposes. However, these independently measured T_{ult} test results have been used as roll specific tensile strengths for establishing installation damage and creep reduction factors. Detailed test results are provided in Appendix C.

Table 3-1. Wide width tensile strength, T_{ult}, for the UX-MSE / UX-HS product line.

Product Style/Type	Test Method	MARV for T_{ult}, in MD (lb/ft)	T_{ult}, Independently Measured in MD (lb/ft)*
UX110MSE	ASTM D 6637	3,970	4,364
UX1400MSE	ASTM D 6637	4,800	4,928
UX1500MSE	ASTM D 6637	7,810	7,825
UX1600MSE	ASTM D 6637	9,870	9,927
UX1700MSE	ASTM D 6637	11,990	12,142

(Conversion: 1 lb/ft = 0.0146 kN/m)
MD = machine direction
*Average of 5 specimens from NTPEP sample.

4.0 Installation Damage Data (RF_{ID})

4.1 Installation Damage Test Program

Installation damage testing and interpretation was conducted in accordance with AASHTO PP66-10, Appendix A, except as noted herein. Samples were exposed to three "standard" soils: a coarse gravel, a sandy gravel, and a sand. Additional laboratory installation damage testing in accordance with ISO/EN 10722 was also conducted. The specific installation damage test program is summarized in Table 4-1.

Table 4-1. Independent installation damage testing required for NTPEP qualification.

Manufacturer: Tensar International **PRODUCT Line:** UX1100MSE/HS to UX1700MSE/HS			
Tests Conducted	**Qualification (every 6 yrs) / QA (every 3 yrs)**		
	Products Tested		**# of Tests (see Note 1)**
	Qualification	**QA**	
Index tensile tests on undamaged material (ASTM D 6637)	UX1100, UX1500, UX1700	NA	3
Three field exposures, including soil characterization and compaction measurements (ASTM D5818)	UX1100, UX1500, UX1700 in Types 1, 2, and 3 soils	NA	9
Tensile tests on damaged specimens (ASTM D 6637)	UX1100, UX1500, UX1700 in Types 1, 2, and 3 soils	NA	9
Laboratory installation damage testing –as basis for future QA and to help interpolate full scale field results to products not full scale field tested (ISO/EN 10722)	UX1100, UX1400, UX1500, UX1600 and UX1700	NA	5
Note 1 Each test is performed using the number of specimens required by the test standard. For example, for index tensile testing, a test is defined 5 to 6 specimens. See the specific test procedures for details on this.			

4.2 Installation Damage Full Scale Field Exposure Procedures and Materials Used

Three "standard" soils were used for the field exposure of the geogrid samples to installation damage. Soil gradation curves for each soil are provided in Figure 4-1. Photographs of each soil illustrating particle angularity are provided in Figures 4-2 through 4-4. LA Abrasion tests conducted to characterize the backfill materials indicted a maximum loss of 20%, which is well within the requirements stated in PP66-10. Note that the photograph of the Type 2 soil only shows the coarser particles since the percentage of sand in that soil is relatively small, and the sand particles have slipped into the voids in this poorly graded gravel just below the stockpile surface at the time this photograph was taken.

The approach specifically used for applying installation damage to the geosynthetic samples that allows for exhumation of the test samples while avoiding unintended damage was initially developed by Watts and Brady[1] of the Transport Research Laboratory (TRL) in the United Kingdom. The procedure generally conforms to PP66-10 and ASTM D 5818 requirements.

Since compaction typically occurs parallel to the face of retaining walls and the contour lines of slopes, the machine direction was placed perpendicular to the running direction of the compaction equipment. To initiate the exposure procedure, four steel plates each measuring 42-inches x 52-inches (1.07 m x 1.32 m), equipped with lifting chains, were placed on a flat clean surface of hardened limestone rock. The longer side of the plates is parallel to the running direction of the compaction equipment. A layer of soil/aggregate was then placed over the adjacent plates to an approximate compacted thickness of 6 inches (0.15 m). Next, each of four coupons of the tested geosynthetic sample was placed on the compacted soil over an area corresponding to an underlying steel plate. To complete the installation, the second layer of soil was placed over the coupons using spreading equipment and compacted to a thickness of 6 inches (0.15 m) using a vibratory compactor. The spreading equipment used included a wheeled front end loader and a 25,000 lb single drum vibratory roller with pneumatic rear wheels. The front end loader was allowed to spread the aggregate by driving over the geosynthetic with a 6 inch aggregate lift between the wheels and the geosynthetic.

The following construction quality control measures were followed during exposure:

- Proctor and sieve analyses were performed on each soil/aggregate, when possible. (Proctors could not be performed on Gradations 1 and 2.)
- Lift thickness measurements were made after soil/aggregate compaction.
- When possible, moisture and density measurements were made on each lift using a nuclear density gage to confirm that densities >90% of modified Proctor (per ASTM D 1557) were being achieved.

To exhume the geosynthetic, railroad ties were removed and one end of each plate was raised with lifting chains. After raising the plate to about 45°, soil located near the bottom of the leaning plate was removed and, if necessary, the plate was struck with a sledgehammer to loosen

[1] G.R.A. Watts and K.C. Brady (1990), *Site Damage trials on geotextiles*, Geotextiles, Geomembranes and Related Produts, Balkema Rotterdam.

the fill. The covering soil/aggregate was then carefully removed from the surface while "rolling" the geosynthetic away from the underlying soil/aggregate. This procedure assured a minimum of exhumation stress. Photographs of the installation damage field exposures are provided in Appendix D. A detailed tabulation of each soil gradation is provided in Appendix D, Table D-10.

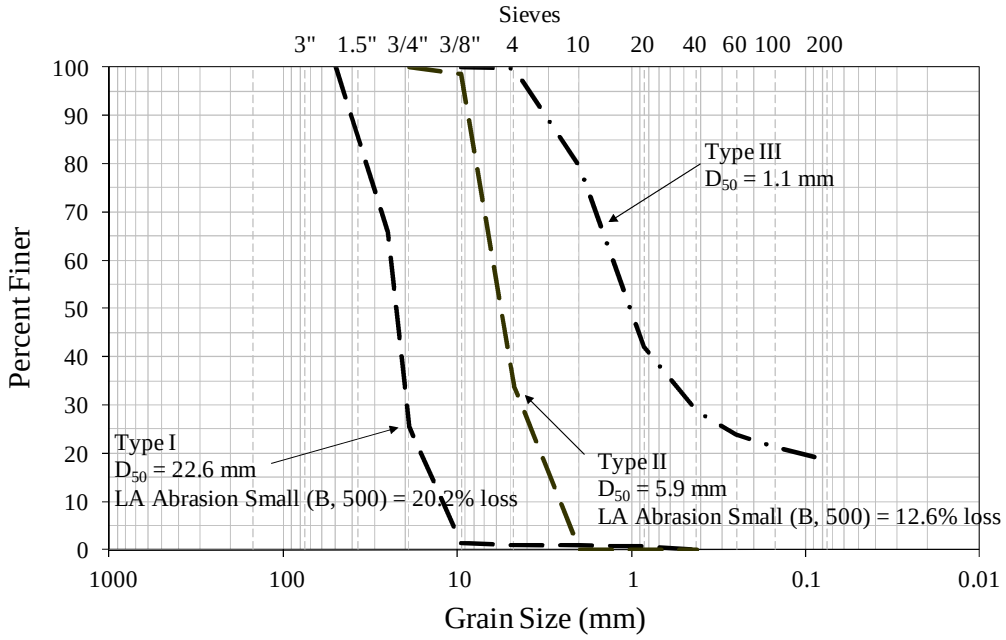


Figure 4-1. Test soil grain size distribution.

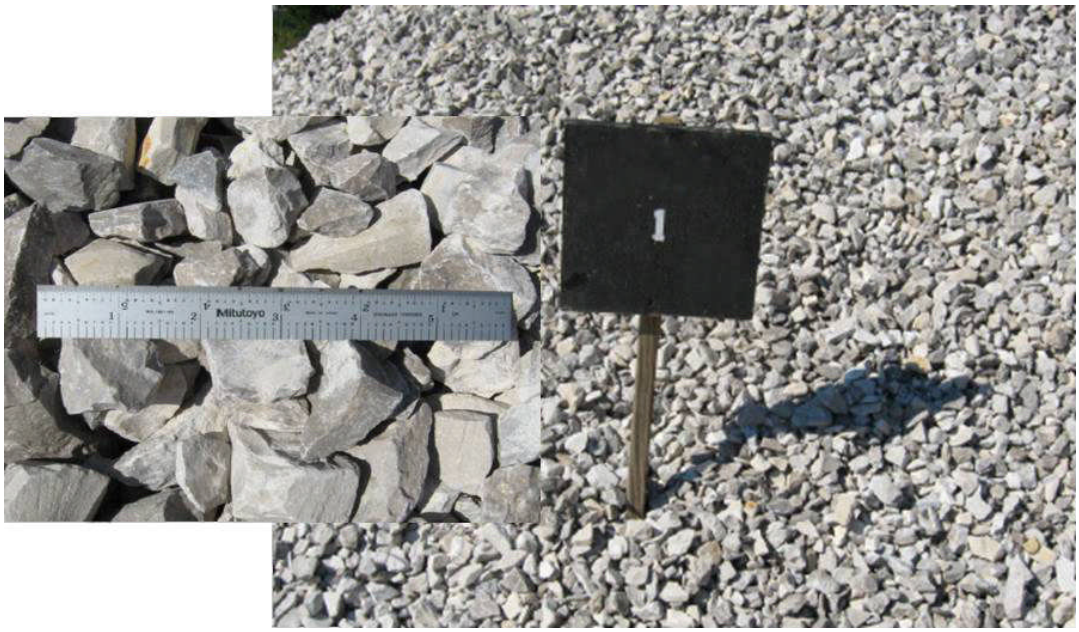


Figure 4-2. Installation damage Type 1 test aggregate.



Figure 4-3. Installation damage Type 2 test aggregate.



Figure 4-4. Installation damage Type 3 test aggregate.

4.3 Summary of Installation Damage Full Scale Field Exposure Test Results

The roll specific ultimate tensile strength (ASTM D6637) test results for the baseline, T_{lot} (i.e., undamaged tensile strength tested prior to sample installation in the ground) and the ultimate tensile strength of the installation damaged geogrid samples, T_{dam}, are provided in Table 4-2. RF_{ID}, calculated using the results shown in Table 4-2, are summarized in Table 4-3. Strength retained is calculated as the ratio of the average exhumed strength T_{dam} divided by the average baseline strength T_{lot} for the product sample. RF_{ID} is the inverse of the retained strength (i.e. 1 / 0.779 = 1.28). Detailed test results for each specimen tested are provided in Appendix D, Tables D-1 through D-9.

Table 4-2. Summary of installation damage tensile test results.

Backfill Type	Style	Baseline		Exhumed	
		[1]T_{lot} (lb/ft)	COV (%)	[2]T_{dam} (lb/ft)	COV (%)
Type 1 Coarse Gravel (GP)	UX1100	4,364	1.5	1,697	12.6
	UX1500	7,825	0.7	3,812	8.8
	UX1700	12,142	1.0	6,838	6.3
Type 2 Sandy Gravel (GP)	UX1100	4,364	1.5	3,322	4.3
	UX1500	7,825	0.7	5,833	5.8
	UX1700	12,142	1.0	8,744	4.0
Type 3 Silty Sand (SM)	UX1100	4,364	1.5	3,509	3.7
	UX1500	7,825	0.7	6,867	5.0
	UX1700	12,142	1.0	10,291	5.0

[1]Average of 5 specimens.
[2]Average of 10 specimens.
(Conversion: 1 lb/ft = 0.0146 kN/m)

Table 4-3. Measured RF_{ID}.

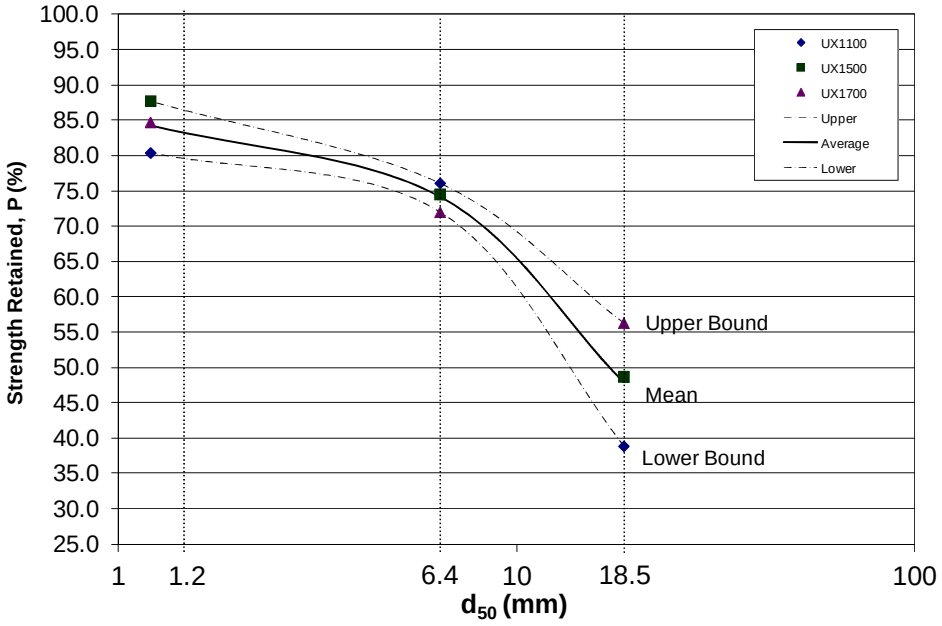
Style	Mass / Area ($oz./yd^2$)	Type 1 Coarse Gravel		Type 2 Sandy Gravel		Type 3 Silty Sand	
		% Retained	RF_{ID}	% Retained	RF_{ID}	% Retained	RF_{ID}
UX1100	8.55	38.9	2.57	76.1	1.31	80.4	1.24
UX1500	14.11	48.7	2.05	74.5	1.34	87.8	1.14
UX1700	23.72	56.3	1.78	72.0	1.39	84.8	1.18

4.4 Estimating RF_{ID} for Specific Soils or for Products not Tested

In general, as the test material gradation becomes more coarse, the value of strength retained decreased (i.e., RF_{ID} increased). Trend lines plotted in Figure 4-5 for the mean, upper bound and lower bound for all the installation damage data obtained for the product line illustrate the general trend of the installation damage data with regard to soil d_{50} size. Interpolation of this data to intermediate gradations appears to be feasible based on these test results, though the scatter in that trend should be recognized when estimating values of RF_{ID} for specific soils.

The UX-MSE / UX-HS product line generally exhibited moderately strong relationships between the weight or the tensile strength of the product and the strength retained after installation damage for the coarsest gradation (gradation 1) but showed no consistent relationship with product weight or tensile strength for the finer gradations. See Figures 4-6 through 4-8 for illustrations of those relationships. Therefore, interpolation of these test results to products in the line not tested based on product weight or strength may only be feasible for the coarsest soil tested, though caution should be exercised and appropriate judgment applied to insure a safe estimate of RF_{ID} each product.

Since only representative products from the product line were field tested for installation damage, all of the products in the product line were tested using ISO/EN 10722 (see Section 4.5 of this report) to investigate relative resistance to installation damage for all the products. As shown in Section 4.5, installation damage resistance of all of the products in that test was reasonably consistent. Therefore, for products in the product line not tested in the full scale installation damage tests, for the two finer soil gradations, use of a lower bound value of strength retained for the products not tested in the full scale installation damage tests (i.e., (P_{dmin} in figures 4-7 and 4-8) appears to be appropriate for design.



Note: $RF_{ID} = 1/P$; d_{50} = sieve size at which 50% of soil passes by weight

Figure 4-5. UX-MSE / UX-HS product line installation damage as a function of soil d_{50} size.

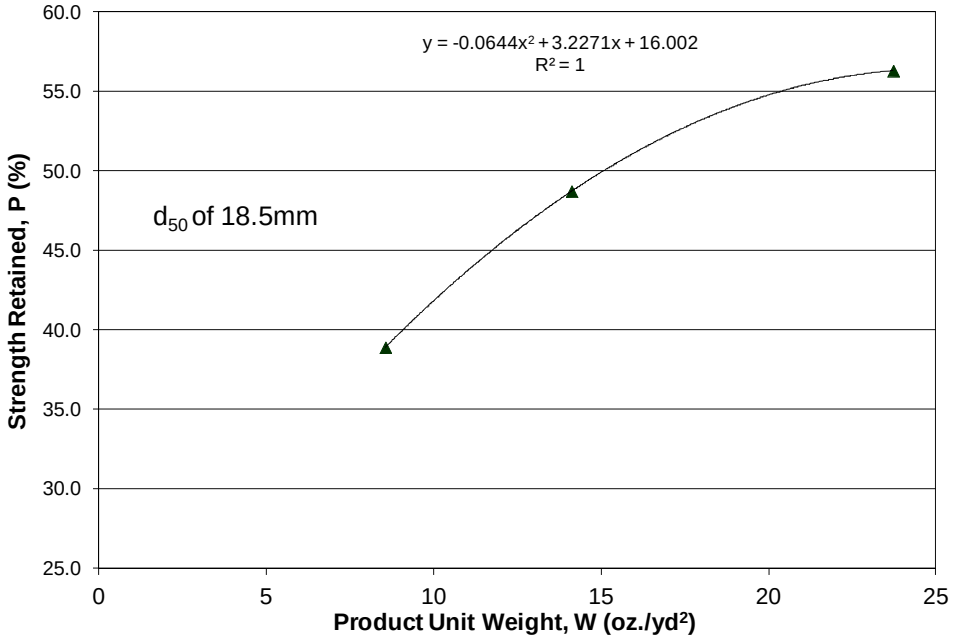


Figure 4-6. UX-MSE / UX-HS product line installation damage as a function of product unit weight for type 1 soil (coarse gravel - GP).

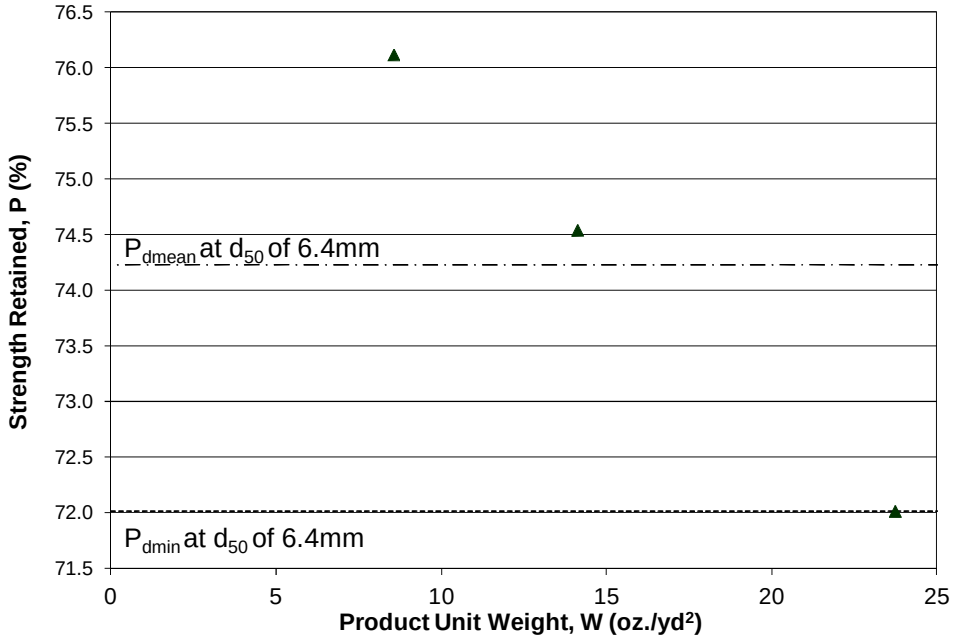


Figure 4-7. UX-MSE / UX-HS product line installation damage as a function of product unit weight for type 2 soil (sandy gravel - GP).

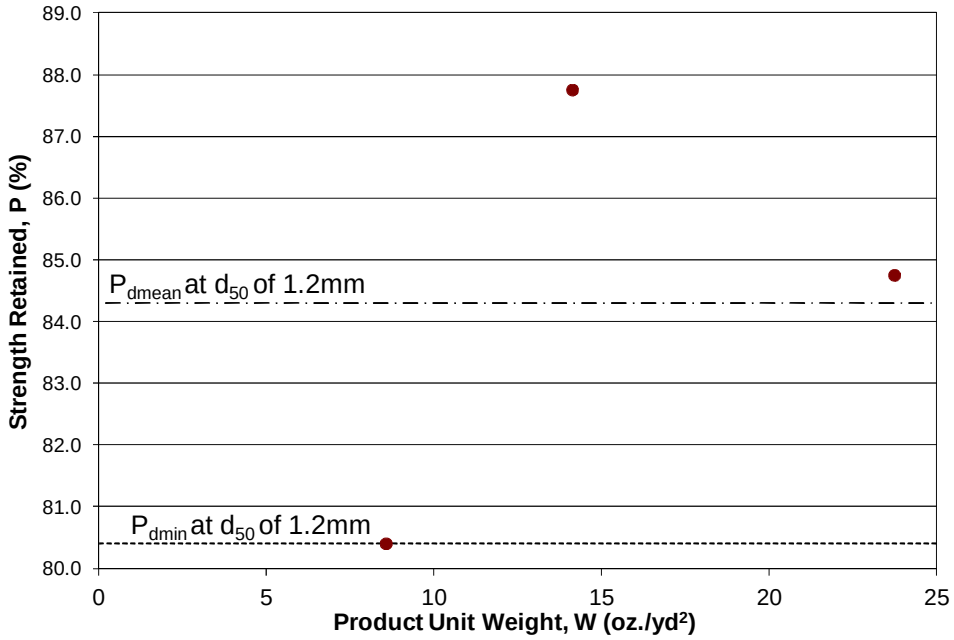


Figure 4-8. UX-MSE / UX-HS product line installation damage as a function of product unit weight for type 3 soil (silty sand – SM).

It should be noted that the installation damage testing conducted represents an increase in compaction and spreading equipment size (i.e., a 15,000 lb wheeled front end loader – Caterpillar 416E, and a 25,000 lb single drum vibratory roller- Caterpillar CS56) and a reduced aggregate lift thickness over the geogrid of 6 inches relative to the installation damage testing reported in previous NTPEP test reports. Therefore, the decrease in strength retained values relative to previous NTPEP test reports for this product line should not be assumed to represent a change in the products, but rather is more likely the result of the more severe installation damage conditions which represent a likely upper bound installation condition for geosynthetic reinforced soil structures. Actual RF_{ID} values could be lower if installation conditions are less severe (e.g., greater initial lift thickness over the geogrid, use of lighter weight equipment, etc.). Actual RF_{ID} values could be higher if the spreading or compacting equipment tires or tracks are allowed to be in direct contact with the geosynthetic before or during fill placement and compaction, if the thickness of the fill material between the equipment tires or tracks is inadequate (especially for high tire pressure equipment such as dump trucks), or if excessive rutting of the first lift of soil over the geosynthetic (e.g., due to soft subgrade soil) is allowed to occur.

4.5 Laboratory Installation Damage Test Results per ISO/EN 10722

Laboratory Installation damage testing and interpretation was conducted in accordance with ISO/EN 10722. In this procedure, geosynthetic specimens are exposed to simulated installation stresses and abrasion using a standard "backfill" material in a bench scale device. Once exposed, they are tested for tensile strength to determine the retained strength after damage. Five baseline and five exposed specimens from each product were tested. The test results are summarized in Table 4-4. Detailed test results are provided in Appendix E, as well as a photograph of the test set-up and a close up of the standard backfill material used.

This procedure is intended to be a reproducible index test to assess relative susceptibility of the geosynthetic to damage. In this NTPEP testing program, the results from this test are primarily intended to be used for future quality assurance to assess the consistency in the product's susceptibility to installation damage. It is not intended to be used directly in the determination of RF_{ID} for a given soil backfill gradation.

Table 4-4. Summary of laboratory (ISO procedure) installation damage test results.

UX-MSE UX-HS Style	Mean Baseline Tensile Strength (lb/ft)	Coefficient of Variation (%)	Mean Exposed Tensile Strength (lb/ft)	Coefficient of Variation (%)	Strength Retained (%)
UX1100	4,680	1.2	3,449	3.3	74
UX1400	5,198	2.1	4,038	6.2	78
UX1500	8,475	1.7	7,448	13.3	88
UX1600	10,224	4.1	7,109	11.3	70
UX1700	13,074	0.4	10,238	5.6	78

(Conversion: 1 lb/ft = 0.0146 kN/m)

5.0 Creep Rupture Data (RF_{CR})

5.1 Creep Rupture Test Program

NTPEP creep testing and interpretation has been conducted in accordance with AASHTO PP66-10, Appendix B. A baseline (i.e., reference) temperature of 68° F (20° C) was used. UX110MSE and UX1400MSE did not meet the qualification requirements for "family consistency" in creep behavior as outlined in PP66-10 and the NTPEP work plan. Therefore, UX110MSE and UX1400MSE must be considered as a separate product family with respect to creep rupture. Both "conventional" (ASTM D5262) and "accelerated conventional" (ASTM D5262) creep tests were conducted. The creep rupture testing program is summarized in Tables 5-1 and 5-2.

Table 5-1. Independent creep rupture testing required for NTPEP qualification.

Manufacturer: Tensar International **PRODUCT Line:** UX1100MSE/HS to UX1400MSE/HS			
Tests Conducted	**Qualification (every 6 yrs) / QA (every 3 yrs)**		
	Products Tested		**# of Tests (see Note 1)**
	Qualification	**QA**	
Index single rib tensile tests on lot specific material (ASTM D 6637)	NA	NA	0
Index wide width tensile tests on lot specific material (ASTM D 6637)	UX1100 & UX1400	NA	2
PRIMARY PRODUCT 6 Rupture Points – Conventional Creep testing up to 1000 hrs (ASTM D5262)	UX1100 @ 6 load levels	NA	6
PRIMARY PRODUCT 6 Rupture Points – Accelerated Conventional Creep rupture testing. (ASTM D5262)	UX1100 @ 6 load levels	NA	6
SECONDARY PRODUCT(S) Conventional Creep Testing (ASTM D5262)	UX1400 @ 1 load level	NA	1
SECONDARY PRODUCT(S) Accelerated Conventional Creep rupture testing. (ASTM D5262)	UX1400 @ 3 load levels	NA	3
Note 1: Each test is performed using the number of specimens required by the test standard. For example, for index tensile testing, a test is defined 5 to 6 specimens. See the specific test procedures for details on this.			

Table 5-2. Independent creep rupture testing required for NTPEP qualification.

Manufacturer: Tensar International **PRODUCT Line:** UX1500MSE/HS to UX1700MSE/HS			
Tests Conducted	**Qualification (every 6 yrs) / QA (every 3 yrs)**		
	Products Tested		**# of Tests (see Note 1)**
	Qualification	**QA**	
Index single rib tensile tests on lot specific material (ASTM D 6637)	NA	NA	0
Index wide width tensile tests on lot specific material (ASTM D 6637)	UX1500, UX1600 & UX1700	NA	2
PRIMARY PRODUCT 6 Rupture Points – Conventional Creep testing up to 1000 hrs (ASTM D5262)	UX1500 @ 6 load levels	NA	6
PRIMARY PRODUCT 6 Rupture Points – Accelerated Conventional Creep rupture testing. (ASTM D5262)	UX1500 @ 8 load levels	NA	8
SECONDARY PRODUCT(S) Conventional Creep Testing (ASTM D5262)	UX1600 & UX1700 @ 1 load level	NA	2
SECONDARY PRODUCT(S) Accelerated Conventional Creep rupture testing. (ASTM D5262)	UX1600 & UX1700 @ 3 load levels	NA	6
Note 1: Each test is performed using the number of specimens required by the test standard. For example, for index tensile testing, a test is defined 5 to 6 specimens. See the specific test procedures for details on this.			

5.2 Baseline Tensile Strength Test Results

Ultimate tensile strengths and associated streains of the tested products are given in Table 5-3. All creep testing for the conventional and accelerated conventional (ASTM D5262) creep tests were performed on multi-rib specimens, therefore it is not necessary to compare single-rib to multi-rib data. Sample specific geogrid dimensions were used to convert tensile test loads to load per unit width values. The tensile test specimens tested were taken from the same rolls of material that were used for the creep testing. The measured geogrid dimensions discussed in Section 2 and provided in Appendix B, Section B.1, were used to convert tensile test loads to load per unit width values.

Table 5-3. Ultimate tensile strength (UTS) and associated strain.

Product	Wide Width UTS per ASTM D6637, T_{lot} (lb/ft @ % Strain)
UX1100MSE	4,364 @ 10.5%
UX1400MSE	4,928 @ 11.4%
UX1500MSE	7,825 @ 11.6%
UX1600MSE	9,927 @ 11.9%
UX1700MSE	12,142 @ 10.4%

(Conversion: 1 lb/ft = 0.0146 kN/m)

5.3 Creep Rupture Test Results

A total of 23 conventional creep test and 20 accelerated conventional creep tests were run to fulfill the qualification requirements. Table 5-4 summarize the tests performed and their outcomes. Detailed test results, including creep curves for each specimen tested, are provided in Appendix F, Figures F-1 through F-43.

Table 5-4. Creep rupture test results for all tests conducted.

Style	Creep Load (% of T_{lot})	Actual Time to Rupture (log hrs)	Shifted Time to Rupture (log hrs)	Shift Factor		Style	Creep Load (% of T_{lot})	Actual Time to Rupture (log hrs)	Shifted Time to Rupture (log hrs)	Shift Factor
UX1100	64	1.4300	1.4300	1		UX1500	60	0.3655	0.3655	1
UX1100	60	1.7064	1.7064	1		UX1500	55	1.6875	1.6875	1
UX1100	58	2.1446	2.1446	1		UX1500	53	2.3762	2.3762	1
UX1100	56	2.8615	2.8615	1		UX1500	52	3.5298	3.5298	1
UX1100	53	3.5165	3.5165	1		UX1500	50	3.1278	3.1278	1
UX1100	50	3.8386	3.8386	1		UX1500	48.5	2.7758	2.7758	1
UX1100	47.5	3.9447	3.9447	1		UX1500	47	2.9689	2.9689	1
UX1100*	58	1.4477	2.447	10		UX1500	48.5	3.7678	3.7678	1
UX1100*	48	3.2433	4.2433	10		UX1500	45.5	4.000***	4.000***	1
UX1100*	44	3.8797	4.8797	10		UX1500*	47.5	2.9820	3.9820	10
UX1100**	52	0.984	2.984	100		UX1500*	45	3.4560	4.4560	10
UX1100**	49.4	1.6668	3.6668	100		UX1500*	52	0.9226	1.9226	10
UX1100**	41	3.159	5.159	100		UX1500*	42.5	4.0177	5.0177	10
						UX1500**	46	2.3600	4.3600	100
UX1400	59	2.0924	2.0924	1		UX1500**	43	2.8696	4.8696	100
UX1400	54	2.9834	2.9834	1		UX1500**	41.5	2.8129	4.8129	100
UX1400*	49	2.4130	3.4130	10		UX1500**	40	3.8294	5.8294	100
UX1400**	44	2.7373	4.7373	100		UX1500**	38.0	4.000***	6.000***	100
UX1600	54	2.0416	2.0416	1		UX1700	56	1.6390	1.6390	1
UX1600	50	2.9930	2.9930	1		UX1700	51	2.2196	2.2196	1
UX1600*	46	3.4283	4.4283	10		UX1700	48	3.0009	3.0009	1
UX1600**	42	3.3104	5.3104	100		UX1700*	44.5	3.8840	4.8840	10

*test run at 30C; ** test run at 40C, ***finished without rupture

5.3.1 Statistical Validation to Allow the Use of Composite Rupture Envelope for Product Line

Details of the confidence limits evaluation for the product line conducted in accordance with PP66-10 are contained in Appendix F. Figures F-44, F-47 and F-48 provide plots of the creep rupture envelope with the confidence limits and the rupture envelopes for the primary products and the other tested products (i.e.,UX1400MSE, UX1600MSE and UX1700MSE), illustrating this statistical test. Detailed calculation results for this statistical analysis are provided in Tables F-1, F-4 and F-5, and summarized in Tables F-2 and F-6. The results indicate that the rupture envelopes for the UX1600MSE and UX1700MSE products are within the specified 90% confidence limits of the primary product (i.e., UX1500MSE) creep rupture data, meeting PP66-10 requirements. However, UX1100MSE and UX1400MSE were not within the specified 90% confidence limits of the primary product (i.e., UX1500MSE) creep rupture data and must be considered as a separate product family with respect to creep rupture. Thus, two different composite creep rupture envelopes were constructed to represent the entire product line, the first containing UX1100MSE/HS and UX1400MSE/HS and the second containing UX1500MSE/HS, UX1600MSE/HS and UX1700MSE/HS. The calculation results for the statistical analysis and regression to create each full composite creep curve are provided in Tables F-3 and F-7. The complete composite creep rupture envelopes are provided in Figures 5-1 and 5-2.

5.3.2 Validation of Shift Factors used for Accelerated Creep Rupture Tests

Details of the shift factor determination for the product line conducted in accordance with PP66-10 are contained in Appendix F. Figures F-45, F-46, F-49 and F-50 provide plots of the creep rupture envelope before and after time shifting for each product line. The first product line includes UX1100MSE and UX1400MSE and the other product line includes UX1500MSE, UX1600MSE, and UX1700MSE. All the products within each product line are used for the time shift factors to be used to relate the elevated temperature rupture points to the baseline temperature of 20^oC (68^oF).

Evaluations were made using the "visual/graphical approach" as described in AASTO PP66-10, Appendix B to determine the best fit shift factors of 'x10' for 30^oC and 'x100' for 40^oC for UX1100MSE and UX1400MSE relative to the baseline temperature of 20^oC. See Figures F-45 and F-46 for the before and after time shift plots for the UX1100MSE and UX1400 MSE product line. Although a slightly different shift factor might produce a better best fit for some of the products in the product line, the difference in the RF_{CR} at 75 years would be minimal. Therefore the use of the best fit shift factors of 'x10' for 30^oC and 'x100' for 40^oC for all the products in the UX1100MSE to UX1400MSE product line is acceptable.

Similarly, for the UX1500MSE, UX1600MSE, and UX1700MSE "product line", the "visual/graphical approach" as described in AASTO PP66-10, Appendix B to determine the best fit shift factors of 'x10' for 30^oC and 'x100' for 40^oC relative to the baseline temperature of 20^oC. See figures F-49 and F-50 for the before and after time shift plots for the UX1500MSE, UX1600MSE, and UX1700MSE product line. Although a slightly different shift factor might produce a better best fit for some of the products in the product line, the

difference in the RF_{CR} at 75 years would be minimal. Therefore the use of the best fit shift factors of 'x10' for 30°C and 'x100' for 40°C for all the products in the UX1500MSE to UX1700MSE product line is acceptable.

5.4 Creep Rupture Envelope Development and Determination of RF_{CR}

In consideration of the statistical validation described in Section 5.3 of this report, two composite creep rupture envelopes for all products tested using log-linear regression, were constructed as shown in Figures 5-1 and 5-2. The mix of conventional and accelerated conventional creep rupture data points meets PP66-10 requirements. Based on these plots of all data, the regressions of the data shows that the r^2 values are 0.96 and 0.91(see Tables F-3 and F-7 in Appendix F for details). Per PP66-10, this degree of scatter in the data is acceptable for a composite rupture envelope.

The creep rupture envelope in Figure 5-1 should be considered valid for UX1100MSE/HS and UX1400MSE/HS only. The creep rupture envelope for UX1500MSE/HS, UX1600MSE/HS and UX1700MSE/HS is found in Figure 5-2. Since the temperature accelerated creep results produced through the accelerated conventional creep testing allowed time shifting of the creep rupture data points to over 1,000,000 hours (i.e., 114 years), no extrapolation uncertainty factor in accordance with PP66-10 need be applied. Table 5-4 provides the estimated value of RF_{CR} for MSE/HS series geogrids based on the reported testing for a period of long-term loading of up to 75 years. This rupture envelope can be used to determine RF_{CR} for times other than 75 years, if desired, using the regression curve equation.

Table 5-5. RF_{CR} value for MSE/HS series geogrids for a 75 yr period of loading/use.

Period of Use (in years)	RF_{CR} for Rupture –UX1100 & UX1400	RF_{CR} for Rupture –UX1500, UX1600 & UX1700
75	2.68	2.54

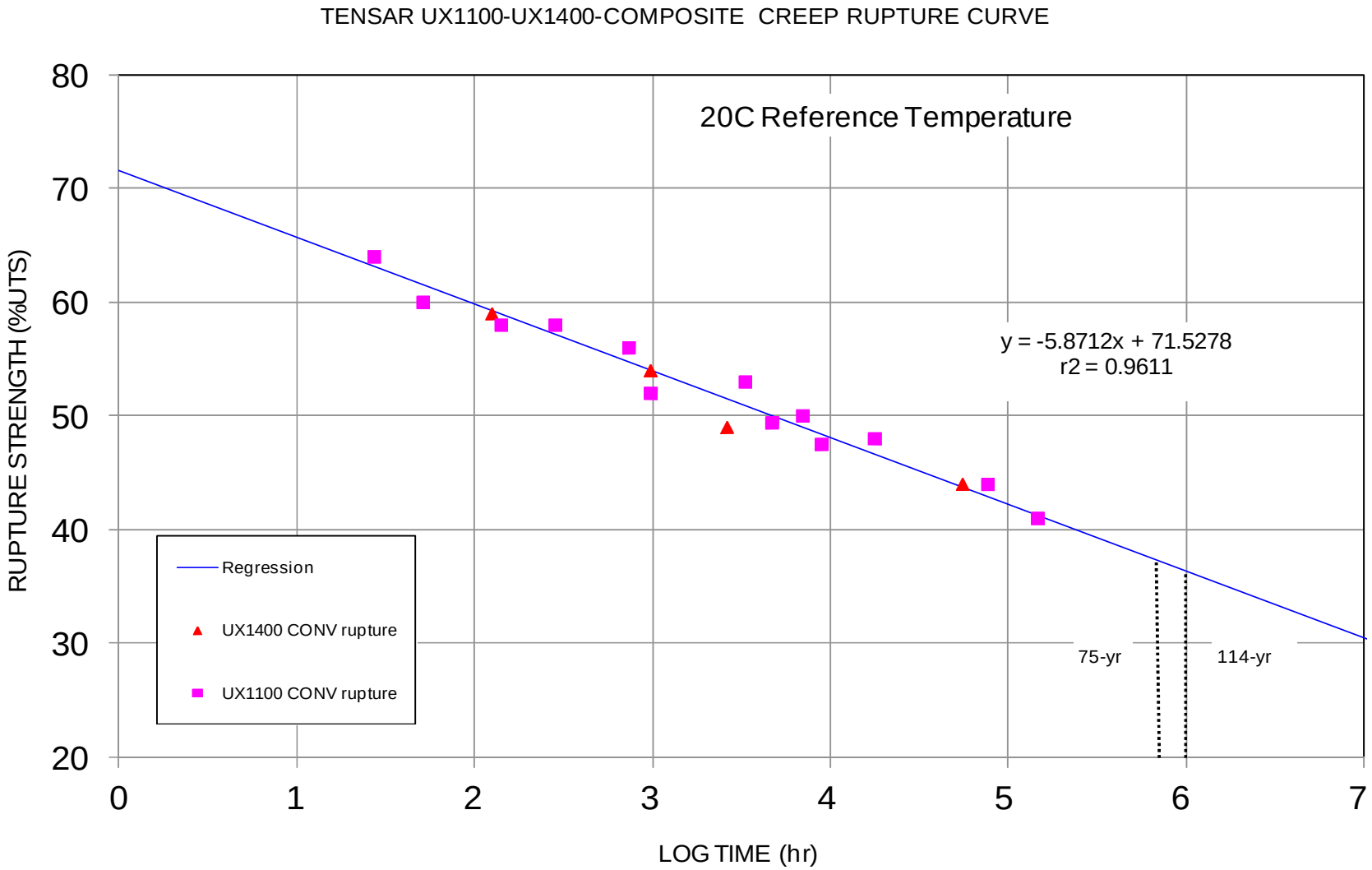


Figure 5-1. Composite creep rupture data/envelope for the UX1100MSE/HS and X1400MSE/HS geogrid products.

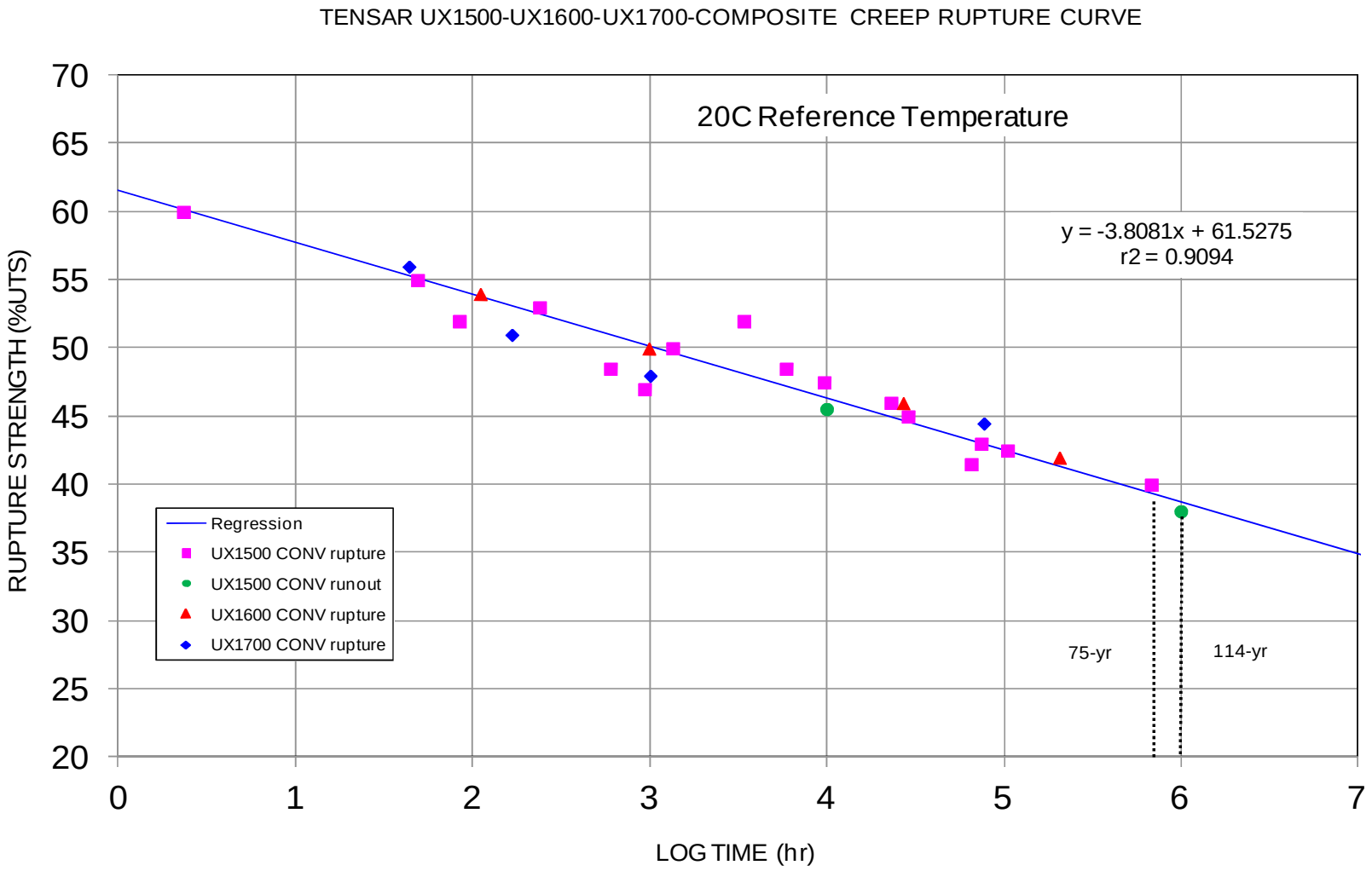


Figure 5-2. Composite creep rupture data/envelope for the UX1500MSE/HS, UX1600MSE/HS and UX1700MSE/HS geogrid products.

6.0 Long-Term Durability Data (RF_D)

6.1 Durability Test Program

Basic molecular properties relating to durability were evaluated, allowing a "default" RF_D to be used in accordance with AASHTO PP66-10, provided that the long-term environment in which the geosynthetic is to be used is considered to be non-aggressive in accordance with the AASHTO LRFD Bridge Design Specifications and PP66-10. A non-aggressive long-term environment is described in these documents as follows:

- A soil ph of 4.5 to 9.0,
- A maximum particle size of 0.75 inches or less unless installation damage effects are specifically evaluated using full scale installation damage testing in accordance with ASTM D 5818,
- A soil organic content of 1% or less, and
- An effective design temperature at the site of 86^{o}F (30^{o}C) or less.

Other specific soil/environmental conditions that could be of concern to consider the site environment to be aggressive are discussed in Elias, et al. 2009[2].

The index properties/test results obtained can be related to long-term performance of the polymer through correlation to longer-term laboratory durability performance tests and long-term experience. Note that long-term durability performance testing in accordance with PP66-10 and the NTPEP work plan to allow direct calculation of RF_D was not available from the manufacturer, nor evaluated as part of the testing program for this product line.

For polyethylene (PE) geosynthetics, key durability issues to address include ultraviolet (UV) degradation, oxidative degradation and brittleness. To assess the potential for these types of degradation, index property tests to assess ultraviolet (UV) degradation, oxidative degradation and brittleness are conducted. Criteria for test results obtained each of these tests are provided in PP66-10 as well as the AASHTO LRFD Bridge Design Specifications.

The UV and oxidation degradation tests were conducted on the lightest weight product in the product line (UX1100MSE/HS) as recommended in PP66-10. Since UV and oxidation degradation attacks from the surface of the geosynthetic, the heavier the product, the more resistant it will be to UV and oxidation degradation. Therefore, UV and oxidation resistance testing the lightest weight product should produce the most conservative result.

[2] Elias, V., Fishman, K.L., Christopher, B.R., and Berg, R.R. 2009, *Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, No.* FHWA-NHI-09-087, Federal Highway Administration, 142pp.

All products in the product line were evaluated for brittleness per WSDOT Test Method T926, Transverse Flexibility Test. The purpose of this test is to detect product polymer formulation or processing problems. The problems this test is intended to uncover, if they are present, can affect the ability of the product to resist installation stresses and abrupt bending over or around objects.

Table 6-1. Independent durability testing required for NTPEP qualification.

Manufacturer: Tensar International **PRODUCT Line:** UX1100MSE/HS to UX1700MSE/HS			
Tests Conducted	**Qualification (every 6 yrs) / QA (every 3 yrs)**		
	Products Tested		**# of Tests (see Note 1)**
	Qualification	**QA**	
All polymers, resistance to weathering @ 500 hrs (ASTM D4355), including before/after tensile strength	UX1100MSE	NA	1
For polyesters, molecular weight determination (ASTM D4603 and GRI-GG7) – **on yarn/strip**	NA	NA	0
For polyesters, carboxyl end group content determination (GRI-GG8) – **on yarn/strip**	NA	NA	0
CEG-MW Testing Coating Removal, if necessary	NA	NA	0
Transverse Flexibility (WSDOT T926)	UX1100MSE, UX1400MSE, UX1500MSE, UX1600MSE, & UX1700MSE	NA	5
For polyolefins, long-term evaluation via Oxidative degradation (ISO/EN 13438:1999)	UX1100MSE	NA	1
For polyesters, long-term evaluation via Hydrolytic degradation (AASHTO PP66-10)	NA	NA	0
For polyolefins, long-term evaluation via Oxidative degradation (AASHTO PP66-10)	None	None	0
Note 1: Each test is performed using the number of specimens required by the test standard. For example, for index tensile testing, a test is defined 5 to 6 specimens. See the specific test procedures for details on this.			

6.2 Durability Test Results

A summary of the test results is provided in Table 6-2. This table also includes the criteria to allow the use of a default reduction factor for RF_D provided in PP66-10 and the AASHTO LRFD Bridge Design Specifications. Detailed durability test results are provided in Appendix G.

Table 6-2. NTPEP durability test results for the UX-MSE / UX-HS geogrid product line and criteria to allow use of a default value for RF_D.

Polymer Type	Property	Test Method	Criteria to Allow Use of Default RF*	Test Result Obtained as Part of NTPEP Program
PP and HDPE	UV Oxidation Resistance	ASTM D4355	Min. 70% strength retained after 500 hrs in weatherometer	98% strength retained
PET	UV Oxidation Resistance	ASTM D4355	Min. 50% strength retained after 500 hrs in weatherometer if geosynthetic will be buried within one week, 70% if left exposed for more than one week.	NA
PP and HDPE	Thermo-Oxidation Resistance	ENV ISO 13438:1999, Method A (PP) or B (HDPE)	Min. 50% strength retained after 28 days (PP) or 56 days (HDPE)	98% strength retained
PP and HDPE	Transverse Flexibility	WSDOT T926	Pass	Pass
PET	Hydrolysis Resistance	Inherent Viscosity Method (ASTM D4603 and GRI Test Method GG8)	Min. Number Average Molecular Weight of 25,000	NA

Note: PP = polypropylene, HDPE = high density polyethylene, PET = polyester

Based on these test results, all products in the product line (UX1100MSE/HS and above) meet the minimum UV and Thermo-Oxidative resistance requirements shown in Table 6-2. In general, a default value of 1.3 for RF_D is typically selected for polyolefins. For HDPE, based on very long-term observations, default reduction factors as low as 1.1 have been used successfully by state departments of transportation. The very high percent strength retained in both the UV tests (98%) and the oven aging tests (98%) summarized in Table 6-2 indicate very good resistance to oxidation, the key issue for durability of HDPE geosynthetic reinforcement. Therefore, use of a default reduction factor of less than 1.3 may be justified. See AASHTO PP66-10, or the document entitled "Use and Application of NTPEP Geosynthetic Reinforcement Test Results" (see www.NTPEP.org), for guidance on the selection of a default value for RF_D.

Regarding transverse flexibility, cracking during bending was not observed for any of the specimens tested. Therefore, these test results indicate that the products in the MSE/HS product line are adequately flexible for typical geosynthetic reinforcement applications.

7.0 Low Strain Creep Stiffness Data

7.1 Low Strain Creep Stiffness Test Program

Creep stiffness testing was conducted in accordance with AASHTO PP66-10 and the NTPEP work plan. The creep stiffness determination was targeted to 2% strain at 1,000 hours.

Products selected to represent the UX-MSE / UX-HS product line (i.e., UX1100MSE, UX1500MSE, and UX1700MSE) were tested for creep stiffness. Roll specific wide width short-term rapid loading tensile strength tests (T_{lot}) were conducted for each product for correlation purposes and to calculate load levels. A total of nine Ramp and Hold (R&H), 1,000 second creep tests, were conducted on each product. Three specimens were R&H tested at each of the following stresses: 5, 10 and 20% of the ultimate tensile strength (UTS). A linear regression based on %UTS and % strain at 0.1 hour was used to normalize strain curves to reduce the variability of the elastic portion of the strain curve. The % UTS required to obtain 2% strain at 1,000 hours was then determined. Three R&H tests and two 1,000 hour conventional creep tests (ASTM D5262, but as modified for low strain in PP66-10 and using a multi-rib specimen) were conducted at this load. All tests were conducted at 68^{o} F (20^{o} C).

7.2 Ultimate Tensile Test Results for Creep Stiffness Test Program

The values provided in Table 7-1 represent the baseline, roll specific, ultimate tensile strength used to normalize the load level for the creep stiffness testing. Sample specific geogrid dimensions were used to convert tensile test loads to load per unit width values.

Table 7-1. Ultimate tensile strength (UTS) & associated strain.

Product	Wide Width UTS per ASTM D6637, T_{lot} (lb/ft @ % Strain)
UX1100MSE	4,364 @ 10.5%
UX1400MSE	4,928 @ 11.4%
UX1500MSE	7,825 @ 11.6%
UX1600MSE	9,927 @ 11.9%
UX1700MSE	12,142 @ 10.4%

(Conversion: 1 lb/ft = 0.0146 kN/m)

7.3 Creep Stiffness Test Results

Detailed test results are provided in Appendix H. Table 7-2 provides a summary of the creep stiffness values obtained. Note that the creep stiffness values at 1,000 hours and 5%UTS, 10%UTS and 20%UTS represent stiffness values at strains other than 2% strain. See Appendix H for details. Figure 7-1 shows the relationship between the measured tensile strength and the creep stiffness. Considering the strong polynomial relationship between the creep stiffness and

the product tensile strength, interpolation to other products in the product line not tested to determine creep stiffness values for those products is acceptable.

Table 7-2. Summary of creep stiffness test results.

UX-MSE UX-HS Series Style	Average Creep Stiffness @ 1000 hours for 5% UTS Ramp & Hold (lb/ft)	Average Creep Stiffness @ 1000 hours for 10% UTS Ramp & Hold (lb/ft)	Average Creep Stiffness @ 1000 hours for 20% UTS Ramp & Hold (lb/ft)	Average Creep Stiffness for 2% strain @ 1000 hrs (lb/ft)
UX1100MSE	39,425	32,094	19,897	**25,552**
UX1500MSE	57,112	43,823	29,322	**40,911**
UX1700MSE	106,252	82,073	57,120	**73,406**

(Conversion: 1 lb/ft = 0.0146 kN/m)

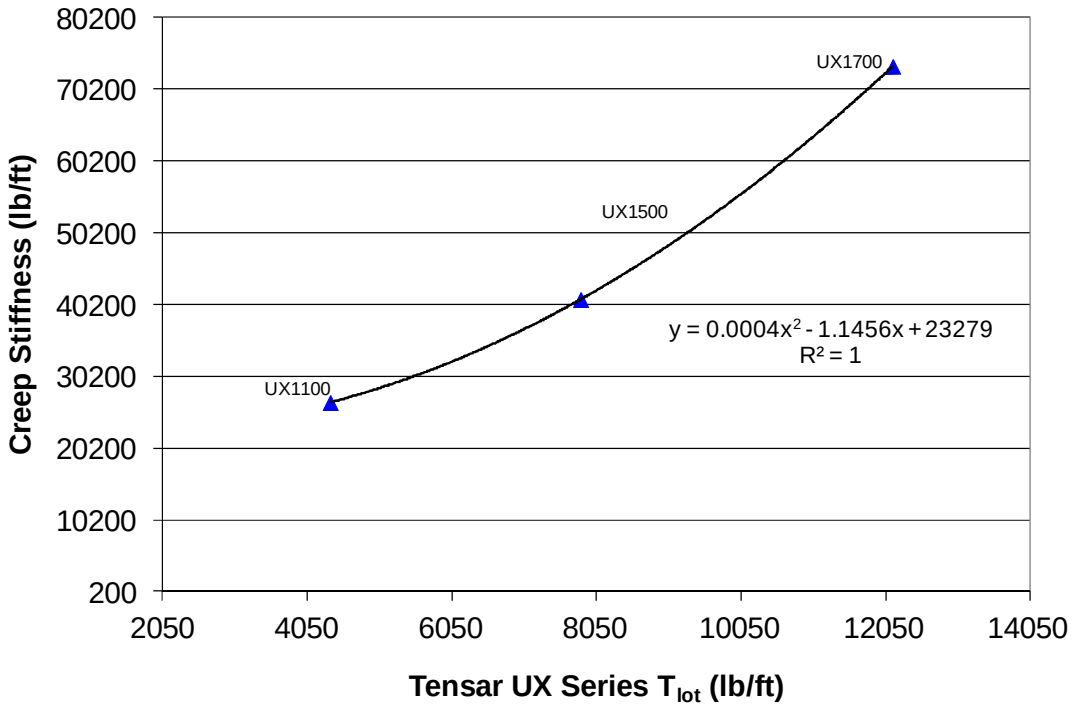


Figure 7-1. UX-MSE / UX-HS creep stiffness for 2 % strain @ 1000 hours.

To obtain the minimum likely stiffness value for each product in consideration of the MARV tensile strength, multiply the stiffness value from the plot by the ratio of T_{MARV}/T_{lot}. T_{MARV} is the minimum tensile strength, as provided by the manufacturer, for each product in the product line. T_{lot} is the actual roll specific tensile strength for the sample used in the creep stiffness testing.

APPENDICES

Appendix A: NTPEP Oversight Committee

Name	Email Address	Agency Name	Designation	Member Type
Hidden, Scott	shidden@ncdot.gov	North Carolina Department of Transportation	Chair	Voting
Brown, Sophie	sophie.brown@odot.state.or.us	Oregon Department of Transportation	Vice Chair	Voting
Glick, Vince	vglick@aashto.org	American Association of State Highway and Transportation Officials	Liaison	None
Golden, Shannon	goldens@dot.state.al.us	Alabama Department of Transportation	Member	Voting
Herrera, Rodrigo	rodrigo.herrera@dot.state.fl.us	Florida Department of Transportation	Member	Voting
Tobias, Daniel	daniel.tobias@illinois.gov	Illinois Department of Transportation	Member	Voting
Sommers, Scott	scott.sommers@iowadot.us	Iowa Department of Transportation	Member	Voting
Davis, Jason	jason.davis@la.gov	Louisiana Department of Transportation and Development	Member	Voting
Sajedi, Dan	dsajedi@sha.state.md.us	Maryland Department of Transportation	Member	Voting
Lee, Derek	derek.lee@state.ma.us	Massachusetts Department of Transportation	Member	Voting
Blakely, John	jblakely@mdot.ms.gov	Mississippi Department of Transportation	Member	Voting
Mclain, Kevin	kevin.mclain@modot.mo.gov	Missouri Department of Transportation	Member	Voting
Lindemann, Mark	mark.lindemann@nebraska.gov	Nebraska Department of Transportation	Member	Voting
Blair, Joseph	Joseph.Blair@dot.nh.gov	New Hampshire Department of Transportation	Member	Voting
Burnett, Thomas	Tom.Burnett@dot.ny.gov	New York State Department of Transportation	Member	Voting
Merklin, Christopher	chris.merklin@dot.ohio.gov	Ohio Department of Transportation	Member	Voting
Miller, Beverly	bevemiller@pa.gov	Pennsylvania Department of Transportation	Member	Voting
Coward, Clifton	clifton.coward@txdot.gov	Texas Department of Transportation	Member	Voting
Shi, Bin	bshi@utah.gov	Utah Department of Transportation	Member	Voting
Ewald, Callie	Callie.Ewald@vermont.gov	Vermont Agency of Transportation	Member	Voting
Kim, Wan Soo	wansoo.kim@vdot.virginia.gov	Virginia Department of Transportation	Member	Voting
Allen, Tony	allent@wsdot.wa.gov	Washington State Department of Transportation	Member	Voting
Baryluk, Greg	greg.baryluk@ads-pipe.com	ADS, Inc.	Other	None
Sarcinella, Robert	sarc.engineering@yahoo.com	Generic Agency	Other	None
Joss, Craig	cjoss@appliedtesting.com	Generic Agency	Other	None
Harris, Keith	keith.harris@hanescompanies.com	Hanes Geo Components	Other	None
Chuck, Fred	fchuck@huesker.com	Huesker, Inc.	Other	None
Curry, Jonathan	jicurry@ifai.com	Industrial Fabrics Association International	Other	None
Massingill, Curtis Randolph	randy@landmsupplyco.com	L&M Supply Company Inc.	Other	None
Mandavkar, Sachin	sachin@maccaferri-usa.com	Maccaferri Inc.	Other	None
Andrews, David	david.andrews@propexglobal.com	Propex Operating Company, LLC	Other	None
Bernardi, Michael	mbernardi@geogrid.com	Strata Systems, Inc.	Other	None
Steen, Carson Glenn	glenn@synteen.com	Synteen Technical Fabrics	Other	None
Lostumbo, John	j.lostumbo@tencategeo.com	TenCate Geosynthetics	Other	None
Gee, Bryan	bgee@tensarcorp.com	Tensar North American Green	Other	None
Sprague, C Joel	jsprague@tri-env.com	TRI/Environmental, Inc.	Other	None
Allen, Sam	sallen@tri-env.com	TRI/Environmental, Inc.	Other	None
Nelson, Jarrett	jnelson@tri-env.com	TRI/Environmental, Inc.	Other	None
Price, Michelle	michelle@usfabrics.com	US Fabrics Inc	Other	None

Appendix B: Product Geometric and Production Details

B.1 Product Geometric Information

Table B-1. Typical and measured MD geogrid geometry for the UX-MSE product line.

Machine Direction (MD) Ribs								
Style	**Width (in)**		**Spacing (in)**		**Aperture Size (in)**		**Rib Thickness (in)**	
	Typical Values	**As Measured***	**Typical Values**	**As Measured***	**Typical Values**	**As Measured ***	**Typical Values**	**As Measured***
UX1100 MSE	0.360	0.214	0.70	0.94	16.7	16.4	0.040	0.036
UX1400 MSE	0.338	0.207	0.67	0.86	17.5	16.9	0.040	0.036
UX1500 MSE	0.362	0.213	0.70	0.88	17.8	17.6	0.060	0.055
UX1600 MSE	0.368	0.224	0.67	0.90	17.7	17.2	0.080	0.077
UX1700 MSE	0.375	0.221	0.63	0.94	17.9	18.2	0.100	0.091

(Conversions: 1 in = 25.4 mm)

*Average of 5 readings obtained during NTPEP testing. Full test results in tables B-5 through B-9.

Table B-2. Typical and measured XD geogrid geometry for the UX-MSE product line.

Cross-Machine Direction (XD) Ribs								
Style	**Width (in)**		**Spacing (in)**		**Aperture Size (in)**		**Rib Thickness (in)**	
	Typical Values	**As Measured***	**Typical Values**	**As Measured***	**Typical Values**	**As Measured***	**Typical Values**	**As Measured***
UX1100 MSE	0.740	0.806	16.7	17.3	0.67	0.72	0.094	0.104
UX1400 MSE	0.762	0.859	17.5	17.8	0.69	0.65	0.096	0.113
UX1500 MSE	0.870	0.893	17.8	18.5	0.64	0.67	0.167	0.173
UX1600 MSE	0.830	0.891	17.7	18.1	0.64	0.68	0.211	0.231
UX1700 MSE	0.790	0.922	17.9	19.1	0.63	0.72	0.282	0.268

(Conversions: 1 in = 25.4 mm)

*Average of 5 readings obtained during NTPEP testing. Full test results in tables B-5 through B-9.

Table B-3. Typical and measured geogrid junction thickness for the UX-MSE product line.

Style	Junction Thickness (in)	
	Typical Values	As Measured*
UX1100MSE	0.094	0.100
UX1400MSE	0.096	0.104
UX1500MSE	0.167	0.168
UX1600MSE	0.211	0.228
UX1700MSE	0.282	0.265

(Conversions: 1 in = 25.4 mm)

*Average of 5 readings obtained during NTPEP testing. Full test results in tables B-5 through B-9.

Table B-4. Typical and measured geogrid unit weight for the UX-MSE product line.

Geogrid Style/Type	Typical Weight (oz/yd^2)	Measured Weight*, per ASTM D5261 (oz/yd^2)
UX1100MSE	9.12	8.55
UX1400MSE	9.58	9.50
UX1500MSE	15.69	14.11
UX1600MSE	20.14	19.65
UX1700MSE	25.76	23.72

(Conversion: 1 oz/ yd^2 = 33.9 g/m^2)

*Average of 5 readings obtained during NTPEP testing. Full test results in tables B-5 through B-9.

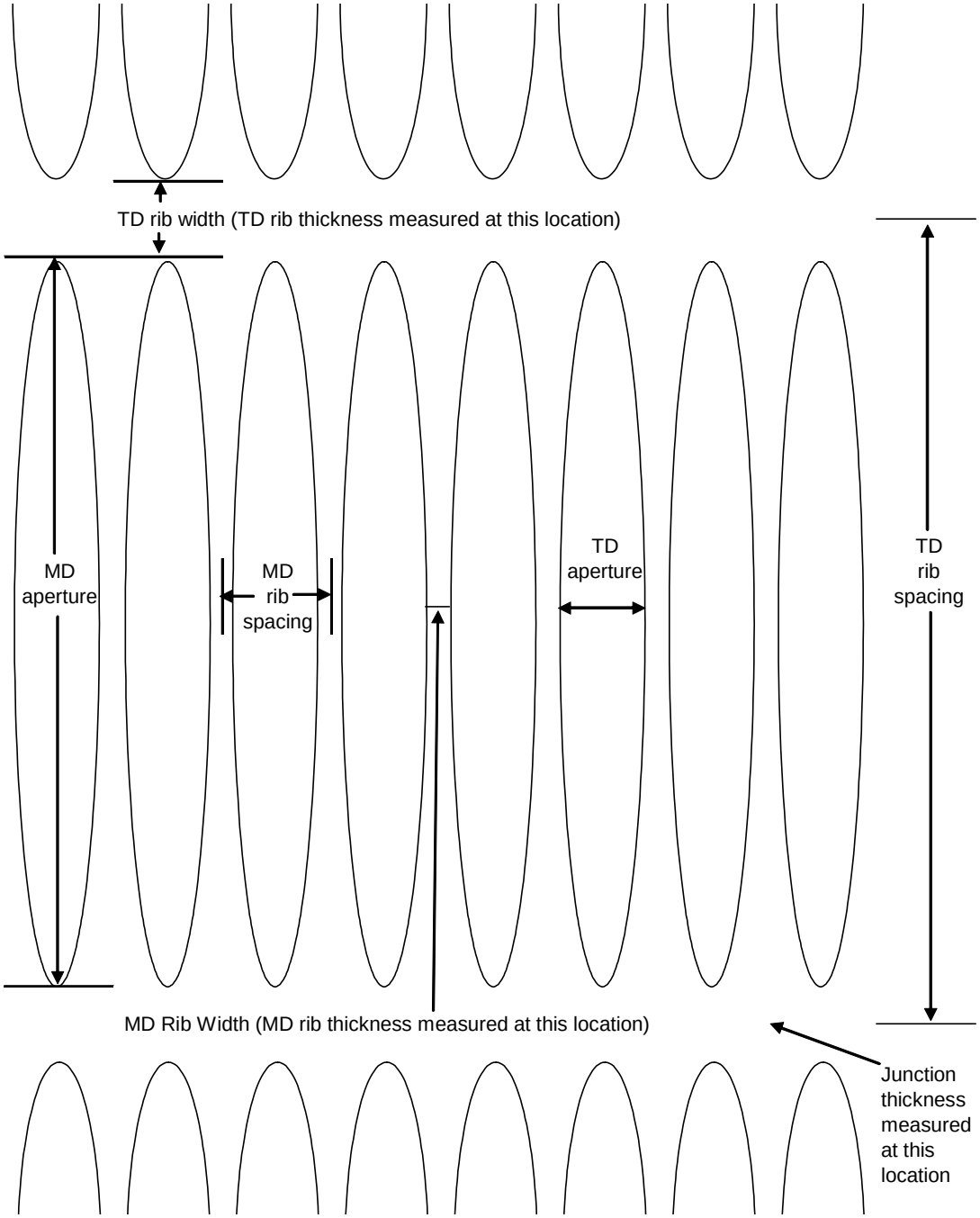


Figure B-1. Diagram of the geometric properties measurement locations.

Table B-5: Geogrid geometric measurements for Tensar UX1100

TRI Log #: E2280-68-03

PARAMETER	TEST REPLICATE NUMBER 1	2	3	4	5	MEAN	STD. DEV.	Typical Value
Mass/Unit Area (ASTM D 5261)								
Specimen Width (in) 8.32								
Specimen Length (in) 34.5								
Mass(g)	53.93	53.75	53.02	54.20	53.48			
Mass/unit area (oz/sq.yd)	8.59	8.56	8.44	8.63	8.51	**8.55**	0.07	
Mass/unit area (g/sq.meter)	291	290	286	293	289	**290**	2	NP
Aperature Size (Calipers)								
MD - Aperature Size (in)	16.5	16.5	16.4	16.4	16.5	**16.4**	0.0	
MD - Aperature Size (mm)	418.6	418.4	416.3	417.6	418.2	**417.8**	0.9	NP
TD - Aperature Size (in)	0.750	0.773	0.728	0.738	0.632	**0.724**	0.054	
TD - Aperature Size (mm)	19.1	19.6	18.5	18.7	16.0	**18.4**	1.4	NP
Rib Width (Calipers)								
MD - Width (in)	0.212	0.218	0.214	0.213	0.215	**0.214**	0.002	
MD - Width (mm)	5.37	5.52	5.44	5.41	5.45	**5.44**	0.06	NP
TD - Width (in)	0.796	0.794	0.802	0.786	0.851	**0.806**	0.026	
TD - Width (mm)	20.22	20.17	20.37	19.96	21.60	**20.46**	0.65	NP
Rib Thickness (Calipers)								
MD - Thickness (in)	0.032	0.038	0.036	0.037	0.038	**0.036**	0.002	
MD - Thickness (mm)	0.81	0.97	0.91	0.93	0.97	**0.92**	0.06	NP
TD - Thickness (in)	0.103	0.1035	0.105	0.1045	0.106	**0.104**	0.001	
TD - Thickness (mm)	2.62	2.63	2.67	2.65	2.69	**2.65**	0.03	NP
Node/Junction Thickness (Calipers)								
Thickness (mils)	0.100	0.099	0.100	0.100	0.100	**0.100**	0.000	
Thickness (mm)	2.54	2.51	2.54	2.54	2.53	**2.53**	0.01	NP

MD - Machine Direction TD - Transverse/Cross Machine Direction NP - Not Provided

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table B-6: Geogrid geometric measurements for Tensar UX1400

TRI Log #: E2280-68-03

PARAMETER	TEST REPLICATE NUMBER 1	2	3	4	5	MEAN	STD. DEV.	Typical Value
Mass/Unit Area (ASTM D 5261)								
Specimen Width (in) 8.02								
Specimen Length (in) 35.3								
Mass(g)	58.99	59.04	58.38	59.33	58.97			
Mass/unit area (oz/sq.yd)	9.51	9.51	9.41	9.56	9.50	**9.50**	0.06	
Mass/unit area (g/sq.meter)	322	323	319	324	322	**322**	2	NP
Aperature Size (Calipers)								
MD - Aperature Size (in)	16.9	16.9	16.9	16.9	16.9	**16.91**	0.03	
MD - Aperature Size (mm)	430.1	429.3	430.0	429.9	428.1	**429.5**	0.8	NP
TD - Aperature Size (in)	0.697	0.684	0.557	0.624	0.687	**0.650**	0.060	
TD - Aperature Size (mm)	17.7	17.4	14.1	15.8	17.4	**16.5**	1.5	NP
Rib Width (Calipers)								
MD - Thickness (in)	0.205	0.204	0.207	0.209	0.213	**0.207**	0.004	
MD - Thickness (mm)	5.19	5.17	5.25	5.31	5.40	**5.26**	0.09	NP
TD - Width (in)	0.853	0.843	0.837	0.844	0.921	**0.859**	0.035	
TD - Width (mm)	21.67	21.40	21.26	21.42	23.39	**21.83**	0.89	NP
Rib Thickness (Calipers)								
MD - Thickness (mils)	0.036	0.035	0.035	0.035	0.038	**0.036**	0.001	
MD - Width (mm)	0.91	0.89	0.89	0.88	0.95	**0.90**	0.03	NP
TD - Thickness (in)	0.1115	0.111	0.114	0.112	0.1145	**0.113**	0.002	
TD - Thickness (mm)	2.83	2.82	2.90	2.84	2.91	**2.86**	0.04	NP
Node/Junction Thickness (Calipers)								
Thickness (mils)	0.105	0.104	0.104	0.105	0.104	**0.104**	0.001	
Thickness (mm)	2.67	2.63	2.63	2.65	2.64	**2.64**	0.02	NP

MD - Machine Direction TD - Transverse/Cross Machine Direction NP - Not Provided

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table B-7: Geogrid geometric measurements for Tensar UX1500

TRI Log #: E2280-68-03

PARAMETER	TEST REPLICATE NUMBER 1	2	3	4	5	MEAN	STD. DEV.	Typical Value
Mass/Unit Area (ASTM D 5261)								
Specimen Width (in) 8.08								
Specimen Length (in) 37.1								
Mass(g)	92.43	92.27	92.68	92.67	92.69			
Mass/unit area (oz/sq.yd)	14.09	14.07	14.13	14.13	14.13	**14.11**	0.03	
Mass/unit area (g/sq.meter)	478	477	479	479	479	**478**	1	NP
Aperature Size (Calipers)								
MD - Aperature Size (in)	17.4	17.6	17.7	17.7	17.6	**17.6**	0.1	
MD - Aperature Size (mm)	441.9	446.5	448.4	450.2	447.4	**446.9**	3.1	NP
TD - Aperature Size (in)	0.636	0.451	0.723	0.704	0.829	**0.668**	0.140	
TD - Aperature Size (mm)	16.2	11.5	18.4	17.9	21.1	**17.0**	3.6	NP
Rib Width (Calipers)								
MD - Width (in)	0.213	0.215	0.214	0.226	0.196	**0.213**	0.011	
MD - Width (mm)	5.40	5.46	5.44	5.73	4.98	**5.40**	0.27	NP
TD - Width (in)	0.912	0.945	0.881	0.874	0.856	**0.893**	0.035	
TD - Width (mm)	23.15	24.00	22.38	22.20	21.73	**22.69**	0.89	NP
Rib Thickness (Calipers)								
MD - Thickness (in)	0.061	0.053	0.053	0.054	0.055	**0.055**	0.003	
MD - Thickness (mm)	1.54	1.33	1.35	1.36	1.40	**1.39**	0.08	NP
TD - Thickness (in)	0.173	0.169	0.174	0.176	0.1745	**0.173**	0.003	
TD - Thickness (mm)	4.39	4.29	4.42	4.47	4.43	**4.40**	0.07	NP
Node/Junction Thickness (Calipers)								
Thickness (mils)	0.171	0.165	0.168	0.169	0.170	**0.168**	0.002	
Thickness (mm)	4.33	4.18	4.25	4.28	4.31	**4.27**	0.06	NP

MD - Machine Direction TD - Transverse/Cross Machine Direction NP - Not Provided

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table B-8: Geogrid geometric measurements for Tensar UX1600

TRI Log #: E2280-68-03

PARAMETER	TEST REPLICATE NUMBER 1	2	3	4	5	MEAN	STD. DEV.	Typical Value
Mass/Unit Area (ASTM D 5261)								
Specimen Width (in) 8.06								
Specimen Length (in) 36.4								
Mass(g)	126.27	126.41	126.08	125.92	126.10			
Mass/unit area (oz/sq.yd)	19.66	19.68	19.63	19.61	19.64	**19.65**	0.03	
Mass/unit area (g/sq.meter)	667	667	666	665	666	**666**	1	NP
Aperature Size (Calipers)								
MD - Aperature Size (in)	17.2	17.2	17.2	17.2	17.2	**17.20**	0.02	
MD - Aperature Size (mm)	437.7	436.3	436.4	437.2	436.9	**436.9**	0.6	NP
TD - Aperature Size (in)	0.708	0.658	0.674	0.660	0.688	**0.677**	0.021	
TD - Aperature Size (mm)	18.0	16.7	17.1	16.8	17.5	**17.2**	0.5	NP
Rib Width (Calipers)								
MD - Width (in)	0.227	0.234	0.220	0.237	0.206	**0.224**	0.012	
MD - Width (mm)	5.77	5.93	5.58	6.01	5.22	**5.70**	0.32	NP
TD - Width (in)	0.911	0.896	0.902	0.884	0.860	**0.891**	0.020	
TD - Width (mm)	23.14	22.76	22.91	22.45	21.84	**22.62**	0.50	NP
Rib Thickness (Calipers)								
MD - Thickness (in)	0.081	0.082	0.078	0.071	0.076	**0.077**	0.004	
MD - Thickness (mm)	2.04	2.07	1.98	1.80	1.93	**1.97**	0.11	NP
TD - Thickness (in)	0.232	0.2265	0.2315	0.2305	0.236	**0.231**	0.003	
TD - Thickness (mm)	5.89	5.75	5.88	5.85	5.99	**5.88**	0.09	NP
Node/Junction Thickness (Calipers)								
Thickness (mils)	0.226	0.231	0.231	0.224	0.228	**0.228**	0.003	
Thickness (mm)	5.73	5.87	5.85	5.68	5.79	**5.78**	0.08	NP

MD - Machine Direction TD - Transverse/Cross Machine Direction NP - Not Provided

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table B-9: Geogrid geometric measurements for Tensar UX1700

TRI Log #: E2280-68-03

PARAMETER	TEST REPLICATE NUMBER 1	2	3	4	5	MEAN	STD. DEV.	Typical Value
Mass/Unit Area (ASTM D 5261)								
Specimen Width (in) 7.86								
Specimen Length (in) 38.4								
Mass(g)	156.05	156.66	156.50	157.20	157.67			
Mass/unit area (oz/sq.yd)	23.60	23.69	23.67	23.78	23.85	**23.72**	0.10	
Mass/unit area (g/sq.meter)	800	803	802	806	808	**804**	3	NP
Aperature Size (Calipers)								
MD - Aperature Size (in)	18.1	18.1	18.3	18.2	18.2	**18.19**	0.07	
MD - Aperature Size (mm)	459.9	460.5	464.4	462.5	462.6	**462.0**	1.8	NP
TD - Aperature Size (in)	0.705	0.756	0.710	0.733	0.710	**0.722**	0.022	
TD - Aperature Size (mm)	17.9	19.2	18.0	18.6	18.0	**18.3**	0.5	NP
Rib Width (Calipers)								
MD - Width (in)	0.219	0.222	0.228	0.229	0.210	**0.221**	0.008	
MD - Width (mm)	5.55	5.63	5.79	5.80	5.32	**5.62**	0.20	NP
TD - Width (in)	0.924	0.932	0.908	0.912	0.935	**0.922**	0.012	
TD - Width (mm)	23.47	23.66	23.06	23.16	23.75	**23.42**	0.30	NP
Rib Thickness (Calipers)								
MD - Thickness (in)	0.090	0.091	0.085	0.097	0.092	**0.091**	0.004	
MD - Thickness (mm)	2.29	2.31	2.15	2.46	2.32	**2.31**	0.11	NP
TD - Thickness (in)	0.266	0.262	0.27	0.266	0.274	**0.268**	0.005	
TD - Thickness (mm)	6.76	6.65	6.86	6.76	6.96	**6.80**	0.12	NP
Node/Junction Thickness (Calipers)								
Thickness (mils)	0.261	0.260	0.265	0.268	0.269	**0.265**	0.004	
Thickness (mm)	6.63	6.59	6.73	6.81	6.83	**6.72**	0.11	NP

MD - Machine Direction TD - Transverse/Cross Machine Direction NP - Not Provided

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

B.2 Product Production Information

Table B-10. Typical geogrid roll dimensions for the UX-MSE / UX-HS product line.

Style/Type	Width (ft)	Length (ft)	Area (yd^2)	Roll Diameter (ft)	Gross weight (lbs)
UX1100MSE	4.36	250	121.1	14	69
UX1400MSE	4.36	250	121.1	14	72.5
UX1500MSE	4.36	200	96.9	16	95
UX1600MSE	4.36	200	96.9	18	122
UX1700MSE	4.36	200	96.9	20	156

(Conversions: 1 ft = 0.3048 m; 1 yd^2 = 0.836 m^2)

B.3 Product Manufacturing Quality Control Program

Testing/sampling is done per the Tensar's Manufacturing Quality Control Program. A summary of the program is provided in Table B-11.

Table B-11. Typical summary of quality control testing conducted by the manufacturer for the UX-MSE and UX-HS product line.

Test Method	Property	Testing Frequency (greater of)				
		UX1100 MSE/HS	UX1400 MSE/HS	UX1500 MSE/HS	UX1600 MSE/HS	UX1700 MSE/HS
ASTM D 5261	Mass / Unit Area	Each Lot	Each Lot	Each Lot	Each Lot	Each Lot
ASTM D 6637	Single Rib Tensile	Each Lot	Each Lot	Each Lot	Each Lot	Each Lot
ASTM D 6637	Multi-Rib Tensile	Index Not Routine	Index Not Routine	Index Not Routine	Index Not Routine	Index Not Routine
GRI-GG2	Junction Strength	Each Lot	Each Lot	Each Lot	Each Lot	Each Lot
Hand measure	Aperture Size	Each Lot	Each Lot	Each Lot	Each Lot	Each Lot

Table B-16. Typical production lot size for the UX-MSE / UX-HS product line.

Style/Type	Lot Size (yd^2)	# of rolls per Lot
UX1100MSE	121.1 - 22,248	1 – approx 225
UX1400MSE	121.1 - 25,431	1 – approx 210
UX1500MSE	96.9 - 12,113	1 – approx 125
UX1600MSE	96.9 – 11,628	1 – approx 120
UX1700MSE	96.9 – 9,690	1 – approx 100

Appendix C: Tensile Strength Detailed Test Results

Table C-1: Geogrid wide width tensile test results for Tensar UX1100

TRI Log #: E2280-68-03

PARAMETER	TEST REPLICATE NUMBER 1	2	3	4	5	MEAN	STD. DEV.	MARV
Wide Width Tensile Properties (ASTM D 6637, Method B)								
MD Number of Ribs per Specimen:	7							
MD Number of Ribs per foot:	13.52							
MD Ultimate Strength (lbs)	2239	2226	2251	2310	2271	**2259**	33	
MD Ultimate Strength (lbs/ft)	4324	4299	4348	4462	4386	**4364**	63	3970 Min
MD Ultimate Strength (kN/m)	63.1	62.8	63.5	65.1	64.0	**63.7**	0.9	
MD Strength @ 2% Strain (lbs)	626	629	633	657	600	**629**	20	
MD Strength @ 2% Strain (lbs/ft)	1209	1215	1223	1269	1159	**1215**	39	
MD Strength @ 2% Strain (kN/m)	17.7	17.7	17.8	18.5	16.9	**17.7**	0.6	
MD Strength @ 5% Strain (lbs)	1157	1166	1176	1101	1112	**1142**	34	
MD Strength @ 5% Strain (lbs/ft)	2235	2252	2271	2127	2148	**2206**	65	
MD Strength @ 5% Strain (kN/m)	32.6	32.9	33.2	31.0	31.4	**32.2**	0.9	
MD Strength @ 10% Strain (lbs)	2238		2215	2212	2179	**2211**	24	
MD Strength @ 10% Strain (lbs/ft)	4323		4278	4272	4209	**4270**	47	
MD Strength @ 10% Strain (kN/m)	63.1		62.5	62.4	61.4	**62.3**	0.7	
MD Break Elongation (%)	10.0	9.9	10.8	10.8	10.8	**10.5**	0.5	

MD - Machine Direction TD - Transverse/Cross Machine Direction NP - Not Provided

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table C-2: Geogrid wide width tensile test results for Tensar UX1400

TRI Log #: E2280-68-03

PARAMETER	TEST REPLICATE NUMBER 1	2	3	4	5	MEAN	STD. DEV.	MARV
Wide Width Tensile Properties (ASTM D 6637, Method B)								
MD Number of Ribs per Specimen:	7							
MD Number of Ribs per foot:	14.04							
MD Ultimate Strength (lbs)	2520	2472	2345	2418	2531	**2457**	77	
MD Ultimate Strength (lbs/ft)	5054	4958	4703	4850	5076	**4928**	155	4800 Min
MD Ultimate Strength (kN/m)	73.8	72.4	68.7	70.8	74.1	**72.0**	2.3	
MD Strength @ 2% Strain (lbs)	662	644	634	611	664	**643**	22	
MD Strength @ 2% Strain (lbs/ft)	1328	1292	1272	1225	1332	**1290**	44	
MD Strength @ 2% Strain (kN/m)	19.4	18.9	18.6	17.9	19.4	**18.8**	0.6	
MD Strength @ 5% Strain (lbs)	1226	1193	1166	1148	1232	**1193**	37	
MD Strength @ 5% Strain (lbs/ft)	2459	2393	2339	2303	2471	**2393**	73	
MD Strength @ 5% Strain (kN/m)	35.9	34.9	34.1	33.6	36.1	**34.9**	1.1	
MD Strength @ 10% Strain (lbs)	2382	2315	2225	2207	2378	**2301**	83	
MD Strength @ 10% Strain (lbs/ft)	4778	4643	4463	4427	4770	**4616**	166	
MD Strength @ 10% Strain (kN/m)	69.8	67.8	65.2	64.6	69.6	**67.4**	2.4	
MD Break Elongation (%)	11.0	11.1	10.9	12.3	11.6	**11.4**	0.6	

MD - Machine Direction TD - Transverse/Cross Machine Direction NP - Not Provided

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table C-3: Geogrid wide width tensile test results for Tensar UX1500

TRI Log #: E2280-68-03

PARAMETER	TEST REPLICATE NUMBER 1	2	3	4	5	MEAN	STD. DEV.	MARV
Wide Width Tensile Properties (ASTM D 6637, Method B) - Retest								
MD Number of Ribs per Specimen:	7							
MD Number of Ribs per foot:	14.04							
MD Ultimate Strength (lbs)	3899	3946	3872	3882	3908	**3901**	29	
MD Ultimate Strength (lbs/ft)	7820	7915	7766	7786	7838	**7825**	57	7810 Min
MD Ultimate Strength (kN/m)	114	116	113	114	114	**114**	1	
MD Strength @ 2% Strain (lbs)	1033	1038	1039	1038	1019	**1033**	8	
MD Strength @ 2% Strain (lbs/ft)	2072	2082	2084	2082	2044	**2073**	17	
MD Strength @ 2% Strain (kN/m)	30.2	30.4	30.4	30.4	29.8	**30.3**	0.2	
MD Strength @ 5% Strain (lbs)	1880	1888	1886	1888	1853	**1879**	15	
MD Strength @ 5% Strain (lbs/ft)	3771	3787	3783	3787	3717	**3769**	30	
MD Strength @ 5% Strain (kN/m)	55.1	55.3	55.2	55.3	54.3	**55.0**	0.4	
MD Strength @ 10% Strain (lbs)	3621	3652	3639	3625	3603	**3628**	19	
MD Strength @ 10% Strain (lbs/ft)	7263	7325	7299	7271	7227	**7277**	37	
MD Strength @ 10% Strain (kN/m)	106	107	107	106	106	**106**	1	
MD Break Elongation (%)	11.4	11.4	12.6	11.4	11.3	**11.6**	0.5	

MD - Machine Direction TD - Transverse/Cross Machine Direction NP - Not Provided

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table C-4: Geogrid wide width tensile test results for Tensar UX1600

TRI Log #: E2280-68-03

PARAMETER	TEST REPLICATE NUMBER 1	2	3	4	5	MEAN	STD. DEV.	MARV
Wide Width Tensile Properties (ASTM D 6637, Method B)								
MD Number of Ribs per Specimen:	7							
MD Number of Ribs per foot:	14.04							
MD Ultimate Strength (lbs)	4958	4951	4975	4896	4966	**4949**	31	
MD Ultimate Strength (lbs/ft)	9944	9930	9978	9820	9960	**9927**	62	9870 Min
MD Ultimate Strength (kN/m)	145	145	146	143	145	**145**	1	
MD Strength @ 2% Strain (lbs)	1318	1249	1300	1302	1321	**1298**	29	
MD Strength @ 2% Strain (lbs/ft)	2644	2505	2607	2611	2650	**2603**	58	
MD Strength @ 2% Strain (kN/m)	38.6	36.6	38.1	38.1	38.7	**38.0**	0.8	
MD Strength @ 5% Strain (lbs)	2443	2369	2437	2436	2452	**2427**	33	
MD Strength @ 5% Strain (lbs/ft)	4900	4752	4888	4886	4918	**4869**	67	
MD Strength @ 5% Strain (kN/m)	71.5	69.4	71.4	71.3	71.8	**71.1**	1.0	
MD Strength @ 10% Strain (lbs)	4574	4465	4585	4580	4609	**4563**	56	
MD Strength @ 10% Strain (lbs/ft)	9174	8956	9196	9186	9244	**9151**	113	
MD Strength @ 10% Strain (kN/m)	134	131	134	134	135	**134**	2	
MD Break Elongation (%)	11.7	12.3	11.8	11.5	12.2	**11.9**	0.3	

MD - Machine Direction TD - Transverse/Cross Machine Direction NP - Not Provided

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table C-5: Geogrid wide width tensile test results for Tensar UX1700

TRI Log #: E2280-68-03

PARAMETER	TEST REPLICATE NUMBER 1	2	3	4	5	MEAN	STD. DEV.	MARV
Wide Width Tensile Properties (ASTM D 6637, Method B)								
MD Number of Ribs per Specimen:	7							
MD Number of Ribs per foot:	14.04							
MD Ultimate Strength (lbs)	6010	6137	5990	6056	6076	**6054**	58	
MD Ultimate Strength (lbs/ft)	12054	12309	12014	12147	12187	**12142**	116	11990 Min
MD Ultimate Strength (kN/m)	176	180	175	177	178	**177**	2	
MD Strength @ 2% Strain (lbs)	1673	1418	1730	1757	1759	**1667**	144	
MD Strength @ 2% Strain (lbs/ft)	3356	2844	3470	3524	3528	**3344**	288	
MD Strength @ 2% Strain (kN/m)	49.0	41.5	50.7	51.5	51.5	**48.8**	4.2	
MD Strength @ 5% Strain (lbs)	3303	3095	3377	3416	3419	**3322**	135	
MD Strength @ 5% Strain (lbs/ft)	6625	6208	6773	6852	6858	**6663**	271	
MD Strength @ 5% Strain (kN/m)	96.7	90.6	98.9	100.0	100.1	**97.3**	4.0	
MD Strength @ 10% Strain (lbs)	6005	5930	5967	6039	6060	**6000**	53	
MD Strength @ 10% Strain (lbs/ft)	12044	11894	11968	12113	12155	**12035**	106	
MD Strength @ 10% Strain (kN/m)	176	174	175	177	177	**176**	2	
MD Break Elongation (%)	10.1	11.3	10.2	10.1	10.1	**10.4**	0.5	

MD - Machine Direction TD - Transverse/Cross Machine Direction NP - Not Provided

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

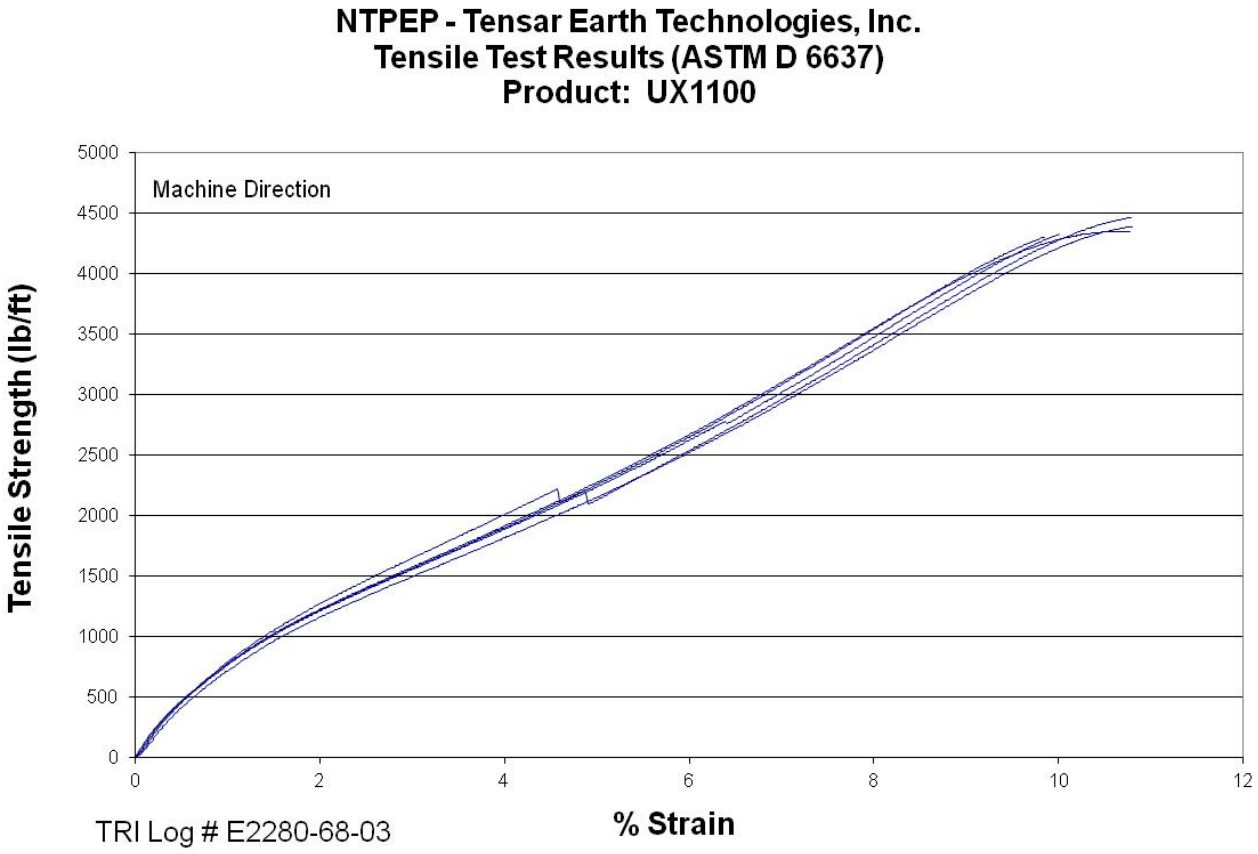


Figure C-1: Geogrid wide width tensile test load-strain curve for Tensar UX1100

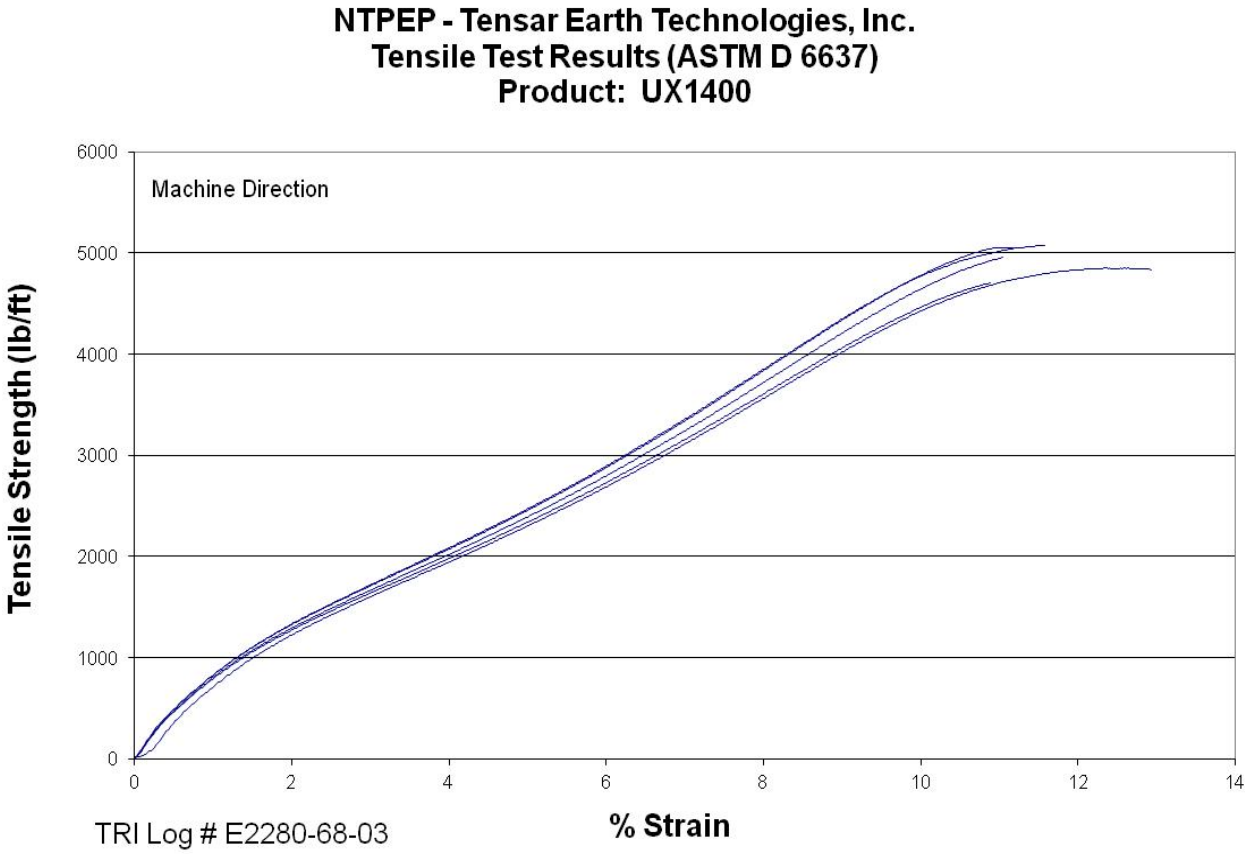


Figure C-2: Geogrid wide width tensile test load-strain curve for Tensar UX1400

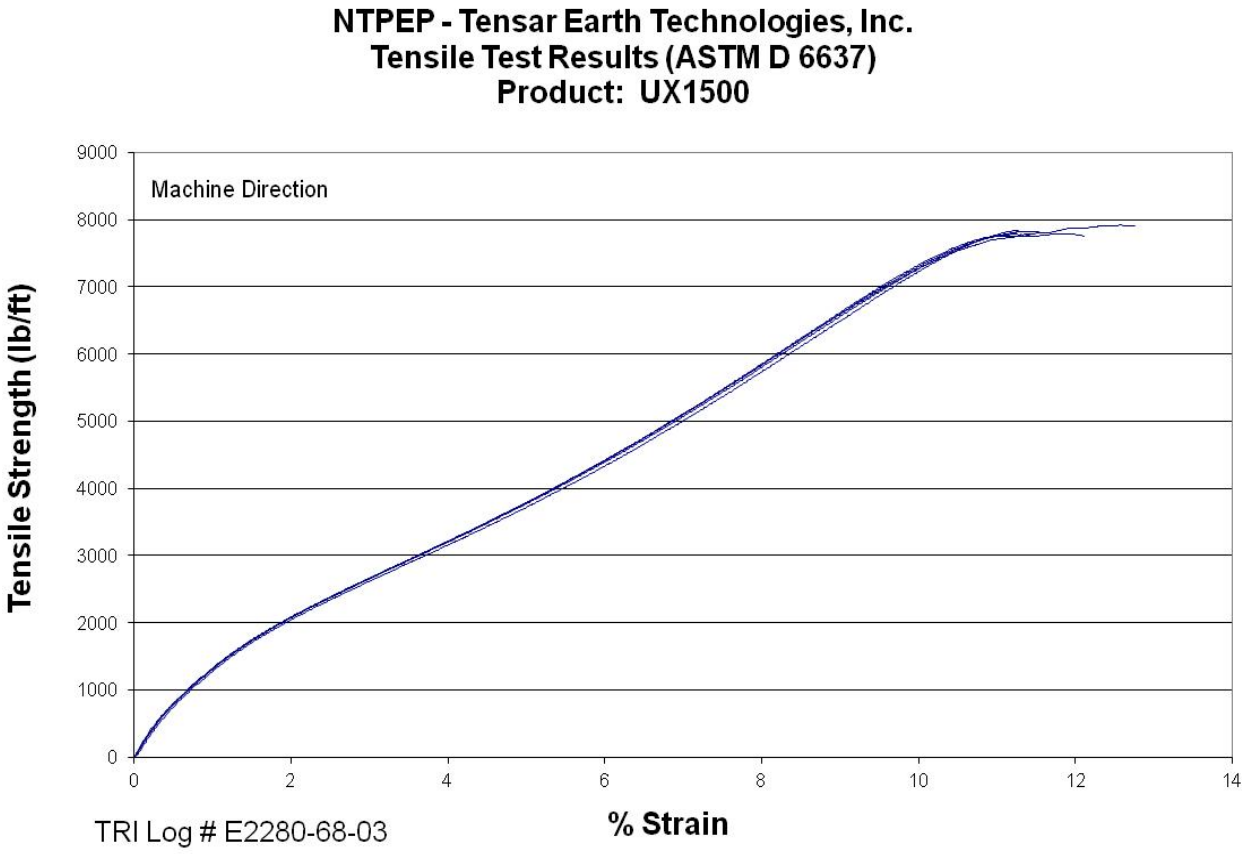


Figure C-3: Geogrid wide width tensile test load-strain curve for Tensar UX1500

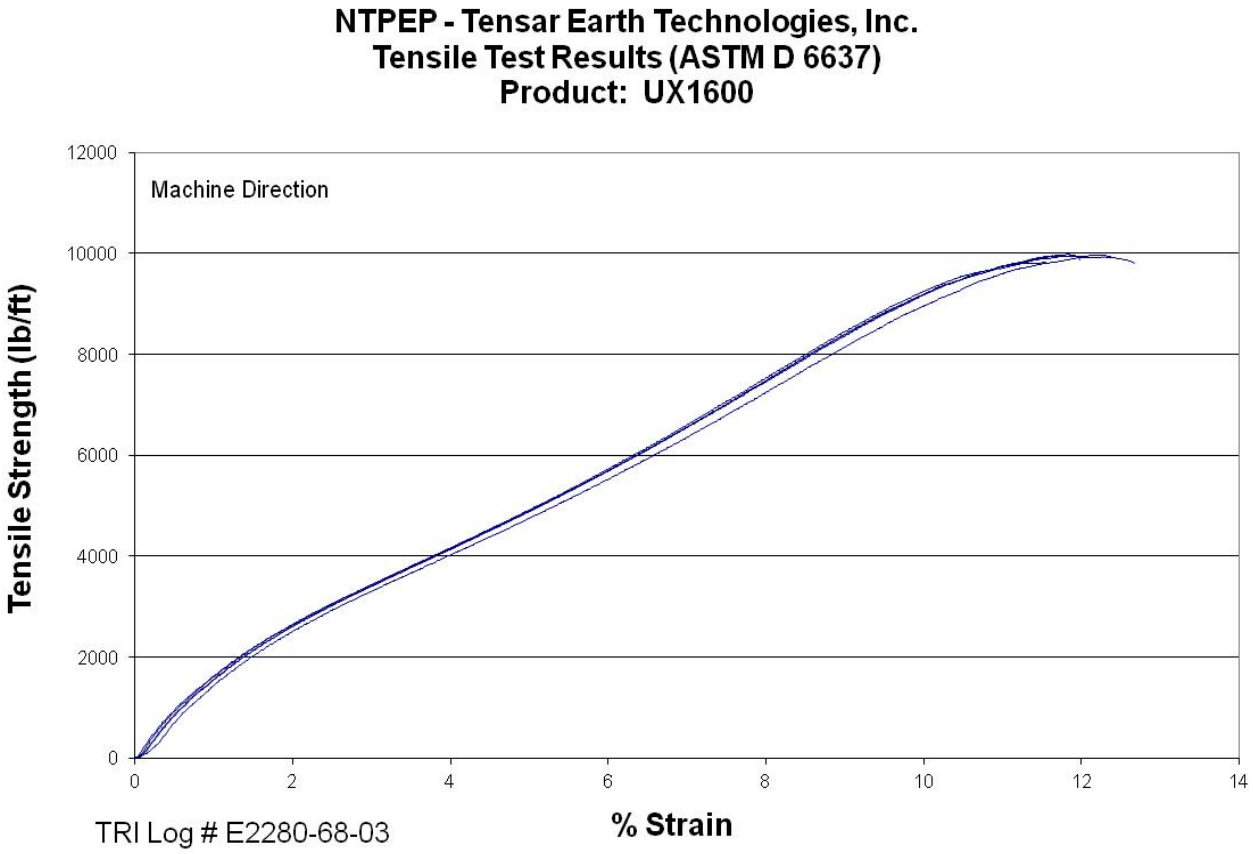


Figure C-4: Geogrid wide width tensile test load-strain curve for Tensar UX1600

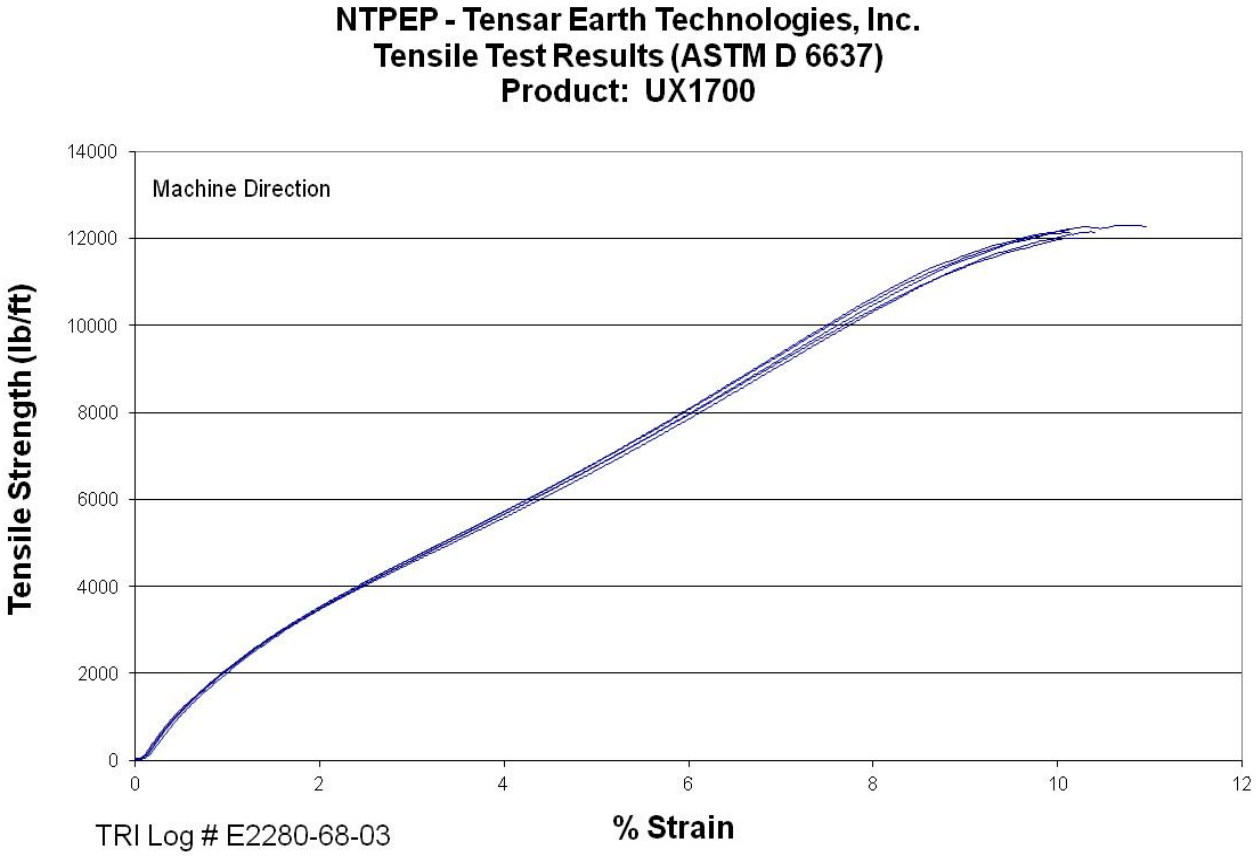


Figure C-5: Geogrid wide width tensile test load-strain curve for Tensar UX1700

Appendix D: Installation Damage Detailed Test Results

Table D-1. Installation damage wide width tensile test results for UX1100 geogrid, soil gradation 1.
Installation damage testing (ASTM D 5818, as modified in AASHTO PP66-10).
Wide wide tensile testing (ASTM D 6637, Method B).

Machine Direction

Sample Identification	Specimen Number	Ribs per Foot Width	Number of Ribs Tested	Maximum Load (lbs)	Maximum Load (lbs/ft)	Maximum Load (kN/m)	Elongation @ Break (%)	Load @ 2% lbs	Load @ 2% (lbs/ft)	Load @ 2% (kN/m)	Load @ 5% lbs	Load @ 5% (lbs/ft)	Load @ 5% (kN/m)	Load @ 10% lbs	Load @ 10% (lbs/ft)	Load @ 10% (kN/m)
UX1100 Baseline	1	13.52	7	2239	4324	63.1	10.0	626	1209	17.65	1157	2234	32.6	2238	4322	63.1
	2	13.52	7	2226	4300	62.8	9.85	629	1216	17.75	1166	2253	32.9			
	3	13.52	7	2251	4347	63.5	10.8	633	1223	17.86	1176	2271	33.2	2215	4278	62.5
	4	13.52	7	2310	4461	65.1	10.8	657	1269	18.52	1101	2127	31.1	2212	4273	62.4
	5	13.52	7	2271	4386	64.0	10.8	600	1158	16.91	1112	2149	31.4	2179	4208	61.4
Average				2259	4364	63.7	10.4	629	1215	17.74	1143	2207	32.2	2211	4270	62.3
Standard Deviation				32.7	63.1	0.92	0.47	20.4	39.3	0.57	33.5	64.7	0.95	24.4	47.1	0.69
% COV				1.45	1.45	1.45	4.50	3.24	3.24	3.24	2.93	2.93	2.93	1.10	1.10	1.10

Machine Direction

Sample Identification	Specimen Number	Ribs per Foot Width	Number of Ribs Tested	Maximum Load (lbs)	Maximum Load (lbs/ft)	Maximum Load (kN/m)	Elongation @ Break (%)	Load @ 2% lbs	Load @ 2% (lbs/ft)	Load @ 2% (kN/m)	Load @ 5% lbs	Load @ 5% (lbs/ft)	Load @ 5% (kN/m)	Load @ 10% lbs	Load @ 10% (lbs/ft)	Load @ 10% (kN/m)
UX1100 installed in Gradation 1 (Coarse Gravel)	1	13.52	7	812	1569	22.9	4.39	509	983	14.4						
	2	13.52	7	822	1588	23.2	4.79	526	1017	14.8						
	3	13.52	7	804	1554	22.7	4.01	522	1009	14.7						
	4	13.52	7	925	1786	26.1	5.70	543	1049	15.3	823	1589	23.2			
	5	13.52	7	726	1401	20.5	5.27	504	974	14.2	709	1369	20.0			
	6	13.52	7	889	1717	25.1	4.37	511	987	14.4						
	7	13.52	7	856	1654	24.1	4.60	546	1054	15.4						
	8	13.52	7	1026	1981	28.9	5.14	534	1031	15.1	1003	1936	28.3			
	9	13.52	7	1096	2117	30.9	5.54	532	1028	15.0	1025	1980	28.9			
	10	13.52	7	829	1601	23.4	4.28	495	957	14.0						
Average				879	1697	24.8	4.81	522	1009	14.7	890	1718	25.1			
Standard Deviation				111	214	3.13	0.58	17.0	32.8	0.48	151	292	4.26			
% COV				12.63	12.63	12.63	11.97	3.25	3.25	3.25	17.0	17.0	17.0			

Percent Retained				38.9	38.9	38.9	46.1	83.0	83.0	83.0	77.9	77.9	77.9			
RFid				**2.57**	**2.57**	**2.57**										

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table D-2. Installation damage wide width tensile test results for UX1100 geogrid, soil gradation 2.
Installation damage testing (ASTM D 5818, as modified in AASHTO PP66-10).
Wide wide tensile testing (ASTM D 6637, Method B).

Machine Direction

Sample Identification	Specimen Number	Ribs per Foot Width	Number of Ribs Tested	Maximum Load (lbs)	Maximum Load (lbs/ft)	Maximum Load (kN/m)	Elongation @ Break (%)	Load @ 2% lbs	Load @ 2% (lbs/ft)	Load @ 2% (kN/m)	Load @ 5% lbs	Load @ 5% (lbs/ft)	Load @ 5% (kN/m)	Load @ 10% lbs	Load @ 10% (lbs/ft)	Load @ 10% (kN/m)
UX1100 Baseline	1	13.52	7	2239	4324	63.1	10.0	626	1209	17.7	1157	2234	32.6	2238	4322	63.1
	2	13.52	7	2226	4300	62.8	9.85	629	1216	17.7	1166	2253	32.9			
	3	13.52	7	2251	4347	63.5	10.8	633	1223	17.9	1176	2271	33.2	2215	4278	62.5
	4	13.52	7	2310	4461	65.1	10.8	657	1269	18.5	1101	2127	31.1	2212	4273	62.4
	5	13.52	7	2271	4386	64.0	10.8	600	1158	16.9	1112	2149	31.4	2179	4208	61.4
Average				2259	4364	63.7	10.4	629	1215	17.7	1143	2207	32.2	2211	4270	62.3
Standard Deviation				32.7	63.1	0.92	0.47	20.4	39.3	0.57	33.5	64.7	0.95	24.4	47.1	0.69
% COV				1.45	1.45	1.45	4.50	3.24	3.24	3.24	2.93	2.93	2.93	1.10	1.10	1.10

Machine Direction

Sample Identification	Specimen Number	Ribs per Foot Width	Number of Ribs Tested	Maximum Load (lbs)	Maximum Load (lbs/ft)	Maximum Load (kN/m)	Elongation @ Break (%)	Load @ 2% lbs	Load @ 2% (lbs/ft)	Load @ 2% (kN/m)	Load @ 5% lbs	Load @ 5% (lbs/ft)	Load @ 5% (kN/m)	Load @ 10% lbs	Load @ 10% (lbs/ft)	Load @ 10% (kN/m)
UX1100 installed in Gradation 2 (Sandy Gravel)	1	13.52	7	1757	3394	49.6	8.06	595	1149	16.8	1116	2156	31.5			
	2	13.52	7	1777	3432	50.1	8.30	572	1104	16.1	1090	2106	30.7			
	3	13.52	7	1741	3363	49.1	8.12	597	1153	16.8	1102	2129	31.1			
	4	13.52	7	1721	3324	48.5	7.91	605	1169	17.1	1121	2165	31.6			
	5	13.52	7	1678	3242	47.3	7.70	601	1162	17.0	1120	2162	31.6			
	6	13.52	7	1754	3388	49.5	7.88	619	1196	17.5	1151	2222	32.4			
	7	13.52	7	1809	3495	51.0	8.26	611	1179	17.2	1129	2181	31.8			
	8	13.52	7	1692	3269	47.7	7.92	601	1160	16.9	1111	2146	31.3			
	9	13.52	7	1730	3341	48.8	8.13	587	1134	16.6	1094	2113	30.9			
	10	13.52	7	1541	2976	43.5	7.17	601	1161	16.9	1111	2146	31.3			
Average				1720	3322	48.5	7.95	599	1157	16.9	1114	2153	31.4			
Standard Deviation				73.7	142	2.08	0.33	12.9	25.0	0.36	17.5	33.8	0.49			
% COV				4.28	4.28	4.28	4.13	2.16	2.16	2.16	1.57	1.57	1.57			

Percent Retained				76.1	76.1	76.1	76.1	95.2	95.2	95.2	97.5	97.5	97.5			
RFid				**1.31**	**1.31**	**1.31**										

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table D-3. Installation damage wide width tensile test results for UX1100 geogrid, soil gradation 3.
Installation damage testing (ASTM D 5818, as modified in AASHTO PP66-10).
Wide wide tensile testing (ASTM D 6637, Method B).

Machine Direction

Sample Identification	Specimen Number	Ribs per Foot Width	Number of Ribs Tested	Maximum Load (lbs)	Maximum Load (lbs/ft)	Maximum Load (kN/m)	Elongation @ Break (%)	Load @ 2% lbs	Load @ 2% (lbs/ft)	Load @ 2% (kN/m)	Load @ 5% lbs	Load @ 5% (lbs/ft)	Load @ 5% (kN/m)	Load @ 10% lbs	Load @ 10% (lbs/ft)	Load @ 10% (kN/m)
UX1100 Baseline	1	13.52	7	2239	4324	63.1	10.0	626	1209	17.7	1157	2234	32.6	2238	4322	63.1
	2	13.52	7	2226	4300	62.8	9.85	629	1216	17.7	1166	2253	32.9			
	3	13.52	7	2251	4347	63.5	10.8	633	1223	17.9	1176	2271	33.2	2215	4278	62.5
	4	13.52	7	2310	4461	65.1	10.8	657	1269	18.5	1101	2127	31.1	2212	4273	62.4
	5	13.52	7	2271	4386	64.0	10.8	600	1158	16.9	1112	2149	31.4	2179	4208	61.4
Average				2259	4364	63.7	10.4	629	1215	17.7	1143	2207	32.2	2211	4270	62.3
Standard Deviation				32.7	63.1	0.92	0.47	20.4	39.3	0.57	33.5	64.7	0.95	24.4	47.1	0.69
% COV				1.45	1.45	1.45	4.50	3.24	3.24	3.24	2.93	2.93	2.93	1.10	1.10	1.10

Machine Direction

Sample Identification	Specimen Number	Ribs per Foot Width	Number of Ribs Tested	Maximum Load (lbs)	Maximum Load (lbs/ft)	Maximum Load (kN/m)	Elongation @ Break (%)	Load @ 2% lbs	Load @ 2% (lbs/ft)	Load @ 2% (kN/m)	Load @ 5% lbs	Load @ 5% (lbs/ft)	Load @ 5% (kN/m)	Load @ 10% lbs	Load @ 10% (lbs/ft)	Load @ 10% (kN/m)
UX1100 installed in Gradation 3 (Sand)	1	13.52	7	1703	3290	48.0	7.94	584	1128	16.5	1094	2112	30.8			
	2	13.52	7	1831	3536	51.6	8.31	599	1157	16.9	1117	2157	31.5			
	3	13.52	7	1829	3532	51.6	8.35	605	1169	17.1	1111	2145	31.3			
	4	13.52	7	1922	3713	54.2	8.87	599	1157	16.9	1107	2137	31.2			
	5	13.52	7	1780	3438	50.2	8.13	588	1136	16.6	1113	2150	31.4			
	6	13.52	7	1903	3676	53.7	8.62	601	1160	16.9	1120	2163	31.6			
	7	13.52	7	1777	3432	50.1	8.02	604	1167	17.0	1128	2179	31.8			
	8	13.52	7	1834	3543	51.7	8.36	592	1143	16.7	1114	2152	31.4			
	9	13.52	7	1844	3561	52.0	8.41	593	1145	16.7	1116	2155	31.5			
	10	13.52	7	1745	3370	49.2	8.14	537	1038	15.1	1090	2105	30.7			
Average				1817	3509	51.2	8.32	590	1140	16.6	1111	2145	31.3			
Standard Deviation				67.5	130	1.90	0.28	19.8	38.3	0.56	11.6	22.3	0.33			
% COV				3.71	3.71	3.71	3.37	3.36	3.36	3.36	1.04	1.04	1.04			

Percent Retained				80.4	80.4	80.4	79.6	93.8	93.8	93.8	97.2	97.2	97.2			
RFid				**1.24**	**1.24**	**1.24**										

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table D-4. Installation damage wide width tensile test results for UX1500 geogrid, soil gradation 1.
Installation damage testing (ASTM D 5818, as modified in AASHTO PP66-10).
Wide wide tensile testing (ASTM D 6637, Method B).

Machine Direction

Sample Identification	Specimen Number	Ribs per Foot Width	Number of Ribs Tested	Maximum Load (lbs)	Maximum Load (lbs/ft)	Maximum Load (kN/m)	Elongation @ Break (%)	Load @ 2% lbs	Load @ 2% (lbs/ft)	Load @ 2% (kN/m)	Load @ 5% lbs	Load @ 5% (lbs/ft)	Load @ 5% (kN/m)	Load @ 10% lbs	Load @ 10% (lbs/ft)	Load @ 10% (kN/m)
UX1500 Baseline	1	14.04	7	3899	7821	114	11.4	1033	2072	30.3	1880	3770	55.0	3621	7263	106
	2	14.04	7	3946	7914	116	12.6	1038	2083	30.4	1888	3787	55.3	3652	7325	107
	3	14.04	7	3872	7766	113	11.4	1039	2084	30.4	1886	3784	55.2	3639	7299	107
	4	14.04	7	3882	7787	114	11.7	1038	2082	30.4	1888	3787	55.3	3625	7271	106
	5	14.04	7	3908	7838	114	11.3	1019	2044	29.8	1853	3717	54.3	3603	7226	106
Average				3901	7825	114	11.7	1033	2073	30.3	1879	3769	55.0	3628	7277	106
Standard Deviation				28.5	57.1	0.83	0.52	8.47	17.0	0.25	14.9	30.0	0.44	18.6	37.3	0.54
% COV				0.73	0.73	0.73	4.48	0.82	0.82	0.82	0.80	0.80	0.80	0.51	0.51	0.51

Machine Direction

Sample Identification	Specimen Number	Ribs per Foot Width	Number of Ribs Tested	Maximum Load (lbs)	Maximum Load (lbs/ft)	Maximum Load (kN/m)	Elongation @ Break (%)	Load @ 2% lbs	Load @ 2% (lbs/ft)	Load @ 2% (kN/m)	Load @ 5% lbs	Load @ 5% (lbs/ft)	Load @ 5% (kN/m)	Load @ 10% lbs	Load @ 10% (lbs/ft)	Load @ 10% (kN/m)
UX1500 installed in Gradation 1 (Coarse Gravel)	1	14.04	7	1797	3604	52.6	5.95	815	1634	23.9	1553	3115	45.5			
	2	14.04	7	1962	3935	57.4	6.05	876	1758	25.7	1670	3350	48.9			
	3	14.04	7	1560	3129	45.7	4.91	781	1566	22.9	1406	2819	41.2			
	4	14.04	7	2033	4078	59.5	6.26	884	1773	25.9	1671	3351	48.9			
	5	14.04	7	1918	3848	56.2	5.95	889	1784	26.0	1682	3373	49.3			
	6	14.04	7	2007	4025	58.8	6.13	877	1759	25.7	1686	3382	49.4			
	7	14.04	7	1870	3752	54.8	5.77	878	1761	25.7	1660	3329	48.6			
	8	14.04	7	1768	3545	51.8	6.28	868	1740	25.4	1641	3292	48.1			
	9	14.04	7	1914	3838	56.0	5.81	895	1796	26.2	1699	3407	49.7			
	10	14.04	7	2176	4365	63.7	6.52	927	1859	27.1	1741	3491	51.0			
Average				1900	3812	55.7	5.96	869	1743	25.4	1641	3291	48.0			
Standard Deviation				168	337	4.92	0.44	41.6	83.5	1.22	95.6	192	2.80			
% COV				8.84	8.84	8.84	7.30	4.79	4.79	4.79	5.82	5.82	5.82			

Percent Retained				48.7	48.7	48.7	51.0	84.1	84.1	84.1	87.3	87.3	87.3	0.0	0.0	0.0
RFid				**2.05**	**2.05**	**2.05**										

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table D-5. Installation damage wide width tensile test results for UX1500 geogrid, soil gradation 2.
Installation damage testing (ASTM D 5818, as modified in AASHTO PP66-10).
Wide wide tensile testing (ASTM D 6637, Method B).

Machine Direction

Sample Identification	Specimen Number	Ribs per Foot Width	Number of Ribs Tested	Maximum Load (lbs)	Maximum Load (lbs/ft)	Maximum Load (kN/m)	Elongation @ Break (%)	Load @ 2% lbs	Load @ 2% (lbs/ft)	Load @ 2% (kN/m)	Load @ 5% lbs	Load @ 5% (lbs/ft)	Load @ 5% (kN/m)	Load @ 10% lbs	Load @ 10% (lbs/ft)	Load @ 10% (kN/m)
UX1500 Baseline	1	14.04	7	3899	7821	114.2	11.4	1033	2072	30.3	1880	3770	55.0	3621	7263	106
	2	14.04	7	3946	7914	115.5	12.6	1038	2083	30.4	1888	3787	55.3	3652	7325	107
	3	14.04	7	3872	7766	113.4	11.4	1039	2084	30.4	1886	3784	55.2	3639	7299	107
	4	14.04	7	3882	7787	113.7	11.7	1038	2082	30.4	1888	3787	55.3	3625	7271	106
	5	14.04	7	3908	7838	114.4	11.3	1019	2044	29.8	1853	3717	54.3	3603	7226	106
Average				3901	7825	114	11.7	1033	2073	30.3	1879	3769	55.0	3628	7277	106
Standard Deviation				28.5	57.1	0.83	0.52	8.47	17.0	0.25	14.9	30.0	0.44	18.6	37.3	0.54
% COV				0.73	0.73	0.73	4.48	0.82	0.82	0.82	0.80	0.80	0.80	0.51	0.51	0.51

Machine Direction

Sample Identification	Specimen Number	Ribs per Foot Width	Number of Ribs Tested	Maximum Load (lbs)	Maximum Load (lbs/ft)	Maximum Load (kN/m)	Elongation @ Break (%)	Load @ 2% lbs	Load @ 2% (lbs/ft)	Load @ 2% (kN/m)	Load @ 5% lbs	Load @ 5% (lbs/ft)	Load @ 5% (kN/m)	Load @ 10% lbs	Load @ 10% (lbs/ft)	Load @ 10% (kN/m)
UX1500 installed in Gradation 2 (Sandy Gravel)	1	14.04	7	2664	5343	78.0	7.85	936	1877	27.4	1747	3504	51.2			
	2	14.04	7	2768	5552	81.1	8.14	947	1900	27.7	1750	3510	51.2			
	3	14.04	7	2646	5306	77.5	7.73	975	1955	28.6	1788	3585	52.3			
	4	14.04	7	2895	5807	84.8	8.48	985	1975	28.8	1788	3585	52.3			
	5	14.04	7	2980	5976	87.3	8.61	980	1966	28.7	1792	3594	52.5			
	6	14.04	7	2977	5970	87.2	8.70	958	1921	28.0	1762	3535	51.6			
	7	14.04	7	2924	5865	85.6	8.51	961	1927	28.1	1768	3547	51.8			
	8	14.04	7	2988	5993	87.5	8.76	977	1959	28.6	1775	3560	52.0			
	9	14.04	7	3117	6252	91.3	9.20	975	1956	28.6	1776	3562	52.0			
	10	14.04	7	3125	6268	91.5	9.41	955	1916	28.0	1753	3515	51.3			
Average				2908	5833	85.2	8.54	965	1935	28.3	1770	3550	51.8			
Standard Deviation				168	338	4.93	0.53	15.9	32.0	0.47	16.5	33.1	0.48			
% COV				5.79	5.79	5.79	6.26	1.65	1.65	1.65	0.93	0.93	0.93			

Percent Retained				74.5	74.5	74.5	73.0	93.4	93.4	93.4	94.2	94.2	94.2			
RFid				**1.34**	**1.34**	**1.34**										

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table D-6. Installation damage wide width tensile test results for UX1500 geogrid, soil gradation 3.
Installation damage testing (ASTM D 5818, as modified in AASHTO PP66-10).
Wide wide tensile testing (ASTM D 6637, Method B).

Machine Direction

Sample Identification	Specimen Number	Ribs per Foot Width	Number of Ribs Tested	Maximum Load (lbs)	Maximum Load (lbs/ft)	Maximum Load (kN/m)	Elongation @ Break (%)	Load @ 2% lbs	Load @ 2% (lbs/ft)	Load @ 2% (kN/m)	Load @ 5% lbs	Load @ 5% (lbs/ft)	Load @ 5% (kN/m)	Load @ 10% lbs	Load @ 10% (lbs/ft)	Load @ 10% (kN/m)
UX1500 Baseline	1	14.04	7	3899	7821	114.2	11.4	1033	2072	30.3	1880	3770	55.0	3621	7263	106
	2	14.04	7	3946	7914	115.5	12.6	1038	2083	30.4	1888	3787	55.3	3652	7325	107
	3	14.04	7	3872	7766	113.4	11.4	1039	2084	30.4	1886	3784	55.2	3639	7299	107
	4	14.04	7	3882	7787	113.7	11.7	1038	2082	30.4	1888	3787	55.3	3625	7271	106
	5	14.04	7	3908	7838	114.4	11.3	1019	2044	29.8	1853	3717	54.3	3603	7226	106
Average				3901	7825	114.2	11.7	1033	2073	30.26	1879	3769	55.0	3628	7277	106.2
Standard Deviation				28.5	57.1	0.83	0.52	8.5	17.0	0.25	14.9	30.0	0.44	18.6	37	0.54
% COV				0.73	0.73	0.73	4.48	0.82	0.82	0.82	0.80	0.80	0.80	0.51	0.51	0.51

Machine Direction

Sample Identification	Specimen Number	Ribs per Foot Width	Number of Ribs Tested	Maximum Load (lbs)	Maximum Load (lbs/ft)	Maximum Load (kN/m)	Elongation @ Break (%)	Load @ 2% lbs	Load @ 2% (lbs/ft)	Load @ 2% (kN/m)	Load @ 5% lbs	Load @ 5% (lbs/ft)	Load @ 5% (kN/m)	Load @ 10% lbs	Load @ 10% (lbs/ft)	Load @ 10% (kN/m)
UX1500 installed in Gradation 3 (Sand)	1	14.04	7	3625	7270	106	10.5	976	1958	28.6	1796	3603	52.6	3490	7000	102
	2	14.04	7	3492	7005	102	9.85	1005	2015	29.4	1832	3675	53.7			
	3	14.04	7	3145	6307	92.1	8.88	1004	2015	29.4	1819	3648	53.3			
	4	14.04	7	3266	6551	95.6	9.17	998	2002	29.2	1822	3654	53.3			
	5	14.04	7	3467	6953	102	9.86	980	1965	28.7	1801	3613	52.7			
	6	14.04	7	3162	6343	92.6	10.2	999	2004	29.3	1828	3667	53.5	3144	6305	92.1
	7	14.04	7	3543	7106	104	10.1	983	1972	28.8	1824	3658	53.4	3508	7035	103
	8	14.04	7	3577	7174	105	10.2	1018	2041	29.8	1851	3713	54.2	3559	7138	104
	9	14.04	7	3463	6945	101	10.1	882	1769	25.8	1720	3449	50.4	3429	6877	100
	10	14.04	7	3496	7012	102	10.0	976	1957	28.6	1792	3595	52.5	3493	7006	102
Average				3424	6867	100	9.89	982	1970	28.8	1809	3627	53.0	3437	6894	101
Standard Deviation				171	343	5.00	0.49	37.8	75.8	1.11	36.0	72.2	1.05	150	300	4.38
% COV				4.99	4.99	4.99	5.00	3.85	3.85	3.85	1.99	1.99	1.99	4.35	4.35	4.35

Percent Retained				87.8	87.8	87.8	84.6	95.0	95.0	95.0	96.2	96.2	96.2	94.7	94.7	94.7
RFid				**1.14**	**1.14**	**1.14**										

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table D-7. Installation damage wide width tensile test results for UX1700 geogrid, soil gradation 1.
Installation damage testing (ASTM D 5818, as modified in AASHTO PP66-10).
Wide wide tensile testing (ASTM D 6637, Method B).

Machine Direction

Sample Identification	Specimen Number	Ribs per Foot Width	Number of Ribs Tested	Maximum Load (lbs)	Maximum Load (lbs/ft)	Maximum Load (kN/m)	Elongation @ Break (%)	Load @ 2% lbs	Load @ 2% (lbs/ft)	Load @ 2% (kN/m)	Load @ 5% lbs	Load @ 5% (lbs/ft)	Load @ 5% (kN/m)	Load @ 10% lbs	Load @ 10% (lbs/ft)	Load @ 10% (kN/m)
UX1700 Baseline	1	14.04	7	6010	12055	176	10.1	1673	3357	49.0	3303	6625	96.7	6005	12045	176
	2	14.04	7	6137	12309	180	11.3	1418	2845	41.5	3095	6207	90.6	5930	11894	174
	3	14.04	7	5990	12013	175	10.2	1730	3470	50.7	3377	6773	98.9	5967	11968	175
	4	14.04	7	6056	12146	177	10.1	1757	3525	51.5	3416	6852	100	6039	12112	177
	5	14.04	7	6076	12187	178	10.1	1759	3527	51.5	3419	6857	100	6060	12154	177
Average				6054	12142	177	10.4	1668	3345	48.8	3322	6663	97.3	6000	12035	176
Standard Deviation				58.0	116	1.70	0.49	144	288	4.20	135	271	3.96	52.5	105	1.54
% COV				0.96	0.96	0.96	4.76	8.61	8.61	8.61	4.07	4.07	4.07	0.87	0.87	0.87

Machine Direction

Sample Identification	Specimen Number	Ribs per Foot Width	Number of Ribs Tested	Maximum Load (lbs)	Maximum Load (lbs/ft)	Maximum Load (kN/m)	Elongation @ Break (%)	Load @ 2% lbs	Load @ 2% (lbs/ft)	Load @ 2% (kN/m)	Load @ 5% lbs	Load @ 5% (lbs/ft)	Load @ 5% (kN/m)	Load @ 10% lbs	Load @ 10% (lbs/ft)	Load @ 10% (kN/m)
UX1700 installed in Gradation 1 (Coarse Gravel)	1	14.04	7	3381	6780	99.0	5.80	1352	2712	39.6	2986	5989	87.4			
	2	14.04	7	3529	7077	103	5.82	1481	2970	43.4	3074	6166	90.0			
	3	14.04	7	3191	6400	93.4	5.22	1493	2995	43.7	3075	6167	90.0			
	4	14.04	7	3654	7330	107	5.97	1501	3011	44.0	3105	6228	90.9			
	5	14.04	7	3835	7691	112	6.30	1495	2999	43.8	3108	6233	91.0			
	6	14.04	7	3461	6941	101	5.83	1485	2978	43.5	3086	6190	90.4			
	7	14.04	7	3321	6661	97.2	5.53	1458	2924	42.7	3040	6096	89.0			
	8	14.04	7	3188	6394	93.3	5.88	1396	2801	40.9	2756	5527	80.7			
	9	14.04	7	3192	6401	93.5	5.15	1503	3015	44.0	3107	6232	91.0			
	10	14.04	7	3345	6708	97.9	5.48	1486	2981	43.5	3100	6218	90.8			
Average				3409	6838	99.8	5.70	1465	2939	42.9	3044	6105	89.1			
Standard Deviation				214.7	431	6.29	0.35	50.65	101.59	1.48	108.1	216.7	3.16			
% COV				6.30	6.30	6.30	6.18	3.46	3.46	3.46	3.55	3.55	3.55			

Percent Retained				56.3	56.3	56.3	54.9	87.9	87.9	87.9	91.6	91.6	91.6			
RFid				**1.78**	**1.78**	**1.78**										

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table D-8. Installation damage wide width tensile test results for UX1700 geogrid, soil gradation 2.
Installation damage testing (ASTM D 5818, as modified in AASHTO PP66-10).
Wide wide tensile testing (ASTM D 6637, Method B).

Machine Direction

Sample Identification	Specimen Number	Ribs per Foot Width	Number of Ribs Tested	Maximum Load (lbs)	Maximum Load (lbs/ft)	Maximum Load (kN/m)	Elongation @ Break (%)	Load @ 2% lbs	Load @ 2% (lbs/ft)	Load @ 2% (kN/m)	Load @ 5% lbs	Load @ 5% (lbs/ft)	Load @ 5% (kN/m)	Load @ 10% lbs	Load @ 10% (lbs/ft)	Load @ 10% (kN/m)
UX1700 Baseline	1	14.04	7	6010	12055	176	10.1	1673	3357	49.0	3303	6625	96.7	6005	12045	176
	2	14.04	7	6137	12309	180	11.3	1418	2845	41.5	3095	6207	90.6	5930	11894	174
	3	14.04	7	5990	12013	175	10.2	1730	3470	50.7	3377	6773	98.9	5967	11968	175
	4	14.04	7	6056	12146	177	10.1	1757	3525	51.5	3416	6852	100	6039	12112	177
	5	14.04	7	6076	12187	178	10.1	1759	3527	51.5	3419	6857	100	6060	12154	177
Average				6054	12142	177	10.4	1668	3345	48.83	3322	6663	97.3	6000	12035	175.7
Standard Deviation				58.0	116.3	1.70	0.49	143.5	287.9	4.20	135.4	271.5	3.96	52.5	105	1.54
% COV				0.96	0.96	0.96	4.76	8.61	8.61	8.61	4.07	4.07	4.07	0.87	0.87	0.87

Machine Direction

Sample Identification	Specimen Number	Ribs per Foot Width	Number of Ribs Tested	Maximum Load (lbs)	Maximum Load (lbs/ft)	Maximum Load (kN/m)	Elongation @ Break (%)	Load @ 2% lbs	Load @ 2% (lbs/ft)	Load @ 2% (kN/m)	Load @ 5% lbs	Load @ 5% (lbs/ft)	Load @ 5% (kN/m)	Load @ 10% lbs	Load @ 10% (lbs/ft)	Load @ 10% (kN/m)
UX1700 installed in Gradation 2 (Sandy Gravel)	1	14.04	7	4267	8559	125	6.60	1659	3328	48.6	3318	6655	97.2			
	2	14.04	7	4320	8665	127	6.80	1633	3274	47.8	3276	6571	95.9			
	3	14.04	7	4219	8463	124	6.49	1709	3427	50.0	3334	6688	97.6			
	4	14.04	7	4232	8489	124	6.47	1720	3450	50.4	3344	6707	97.9			
	5	14.04	7	4484	8993	131	7.07	1629	3267	47.7	3254	6527	95.3			
	6	14.04	7	4238	8500	124	6.79	1562	3133	45.7	3192	6401	93.5			
	7	14.04	7	4175	8375	122	6.64	1540	3089	45.1	3207	6433	93.9			
	8	14.04	7	4415	8856	129	7.62	1233	2472	36.1	2905	5826	85.1			
	9	14.04	7	4509	9043	132	7.02	1673	3356	49.0	3303	6625	96.7			
	10	14.04	7	4734	9495	139	7.45	1675	3360	49.1	3299	6617	96.6			
Average				4359	8744	128	6.90	1603	3216	46.9	3243	6505	95.0			
Standard Deviation				175	351	5.12	0.39	142	286	4.17	129.3	259.3	3.79			
% COV				4.01	4.01	4.01	5.71	8.88	8.88	8.88	3.99	3.99	3.99			

Percent Retained				72.0	72.0	72.0	66.5	96.1	96.1	96.1	97.6	97.6	97.6			
RFid				**1.39**	**1.39**	**1.39**										

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table D-9. Installation damage wide width tensile test results for UX1700 geogrid, soil gradation 3.
Installation damage testing (ASTM D 5818, as modified in AASHTO PP66-10).
Wide wide tensile testing (ASTM D 6637, Method B).

Machine Direction

Sample Identification	Specimen Number	Ribs per Foot Width	Number of Ribs Tested	Maximum Load (lbs)	Maximum Load (lbs/ft)	Maximum Load (kN/m)	Elongation @ Break (%)	Load @ 2% lbs	Load @ 2% (lbs/ft)	Load @ 2% (kN/m)	Load @ 5% lbs	Load @ 5% (lbs/ft)	Load @ 5% (kN/m)	Load @ 10% lbs	Load @ 10% (lbs/ft)	Load @ 10% (kN/m)
UX1700 Baseline	1	14.04	7	6010	12055	176	10.1	1673	3357	49.0	3303	6625	96.7	6005	12045	176
	2	14.04	7	6137	12309	180	11.3	1418	2845	41.5	3095	6207	90.6	5930	11894	174
	3	14.04	7	5990	12013	175	10.2	1730	3470	50.7	3377	6773	98.9	5967	11968	175
	4	14.04	7	6056	12146	177	10.1	1757	3525	51.5	3416	6852	100	6039	12112	177
	5	14.04	7	6076	12187	178	10.1	1759	3527	51.5	3419	6857	100	6060	12154	177
Average				6054	12142	177	10.4	1668	3345	48.83	3322	6663	97.3	6000	12035	175.7
Standard Deviation				58.0	116	1.70	0.49	144	288	4.20	135	271	3.96	52.5	105	1.54
% COV				0.96	0.96	0.96	4.76	8.61	8.61	8.61	4.07	4.07	4.07	0.87	0.87	0.87

Machine Direction

Sample Identification	Specimen Number	Ribs per Foot Width	Number of Ribs Tested	Maximum Load (lbs)	Maximum Load (lbs/ft)	Maximum Load (kN/m)	Elongation @ Break (%)	Load @ 2% lbs	Load @ 2% (lbs/ft)	Load @ 2% (kN/m)	Load @ 5% lbs	Load @ 5% (lbs/ft)	Load @ 5% (kN/m)	Load @ 10% lbs	Load @ 10% (lbs/ft)	Load @ 10% (kN/m)
UX1700 installed in Gradation 3 (Sand)	1	14.04	7	5332	10695	156	8.50	1621	3252	47.5	3243	6505	95.0			
	2	14.04	7	4904	9835	144	7.74	1600	3208	46.8	3241	6501	94.9			
	3	14.04	7	5401	10832	158	8.50	1705	3420	49.9	3315	6648	97.1			
	4	14.04	7	5178	10385	152	7.88	1752	3514	51.3	3390	6799	99.3			
	5	14.04	7	5181	10391	152	8.13	1667	3344	48.8	3296	6610	96.5			
	6	14.04	7	5298	10626	155	8.19	1727	3465	50.6	3350	6719	98.1			
	7	14.04	7	4534	9093	133	7.08	1688	3386	49.4	3301	6620	96.7			
	8	14.04	7	5279	10589	155	8.23	1738	3487	50.9	3360	6739	98.4			
	9	14.04	7	5024	10077	147	7.76	1722	3453	50.4	3333	6685	97.6			
	10	14.04	7	5178	10386	152	8.00	1739	3488	50.9	3362	6742	98.4			
Average				5131	10291	150	8.00	1696	3402	49.7	3319	6657	97.2			
Standard Deviation				256	513	7.50	0.42	51.9	104	1.52	49.8	99.9	1.46			
% COV				4.99	4.99	4.99	5.26	3.06	3.06	3.06	1.50	1.50	1.50			

Percent Retained				84.8	84.8	84.8	77.2	101.7	101.7	101.7	99.9	99.9	99.9	0.0	0.0	0.0
RFid				**1.18**	**1.18**	**1.18**										

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

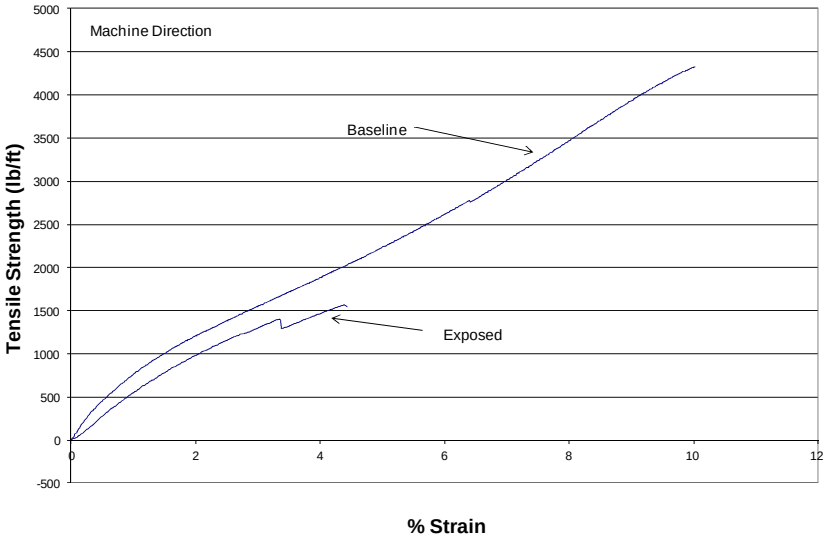


Figure D-1. Example baseline and exposed wide width tensile test load-strain curves for UX1100MSE geogrid installed in soil gradation 1.

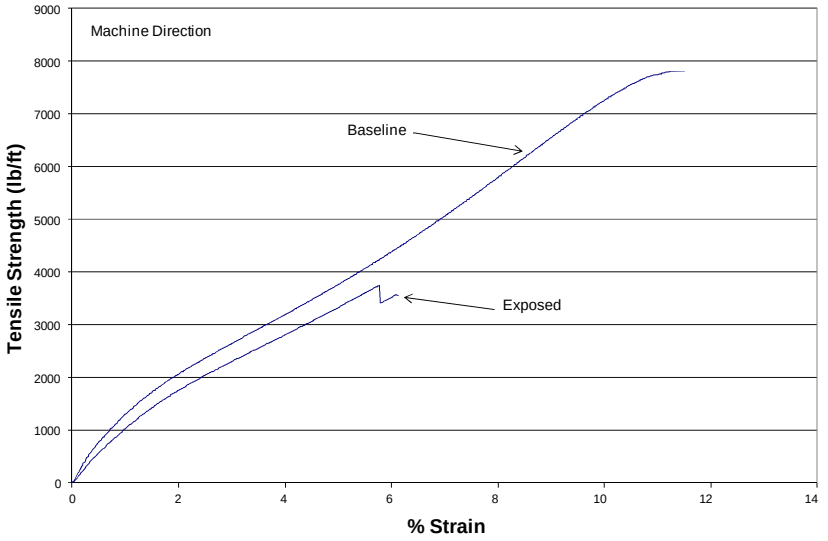


Figure D-2. Example baseline and exposed wide width tensile test load-strain curves for UX1500MSE geogrid installed in soil gradation 1.

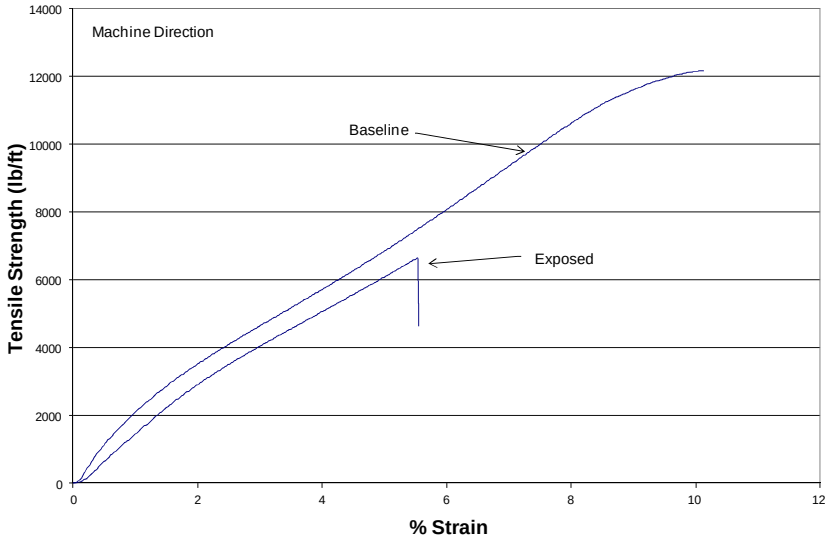


Figure D-3. Example baseline and exposed wide width tensile test load-strain curves for UX1700MSE geogrid installed in soil gradation 1.



Figure D-4. Photos of typical visual damage of UX1100MSE installed in soil gradation 1.



Figure D-5. Photos of typical visual damage of UX1500MSE installed in soil gradation 1.

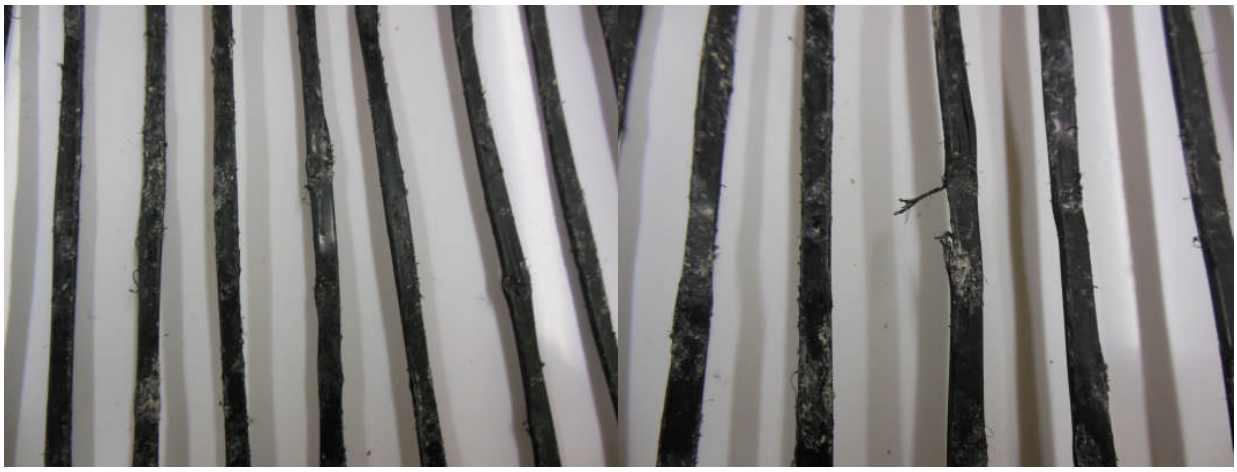


Figure D-6. Photos of typical visual damage of UX1700MSE installed in soil gradation 1.

Table D-10. Standard test soil gradations (% passing).

Standard Installation Damage Soils Used for Field Exposures				
US Sieve No.	**Sieve Size (mm)**	**Percent Passing by Weight**		
		Type 1 (Coarse Gravel)	**Type2 (Sandy Gravel)**	**Type 3 (Silty Sand)**
6 - in	150	100.0	100.0	100.0
3 - in.	75	100.0	100.0	100.0
2 - in.	50	100.0	100.0	100.0
1.5 - in.	38		100.0	100.0
1 - in.	25	65.8	100.0	100.0
3/4 - in.	19	25.4	100.0	100.0
1/2 - in.	12.5			100.0
3/8 - in.	9.5	1.3	98.7	100.0
No. 4	4.75	1.0	33.8	99.8
No. 10	1.7	0.9	0.0	79.5
No. 20	0.85	0.8	0.0	42.1
No. 40	0.425	0.0	0.0	28.7
No. 60	0.25	0.0	0.0	23.8
No. 100	0.15	0.0	0.0	21.2
No. 200	0.075	0.0	0.0	18.4
D50, mm		22.6	5.9	1.1
LA Abrasion Small Drum Method B 500 Cycles		20.2% loss	12.6% loss	
Liquid Limit, %		-	-	-
Plasticity Index, %		-	-	-
Angularity (ASTM D 2488)		Angular to Subangular	Angular	Angular to Subangular
Soil Classification		GP	GP	SM
		Poorly Graded Gravel	Poorly Graded Gravel with Sand	Well Graded Silty Sand



Figure D-7. Lifting Plates positioned between ties and covered with first lift of compacted soil/aggregate.



Figure D-8. Grid positioned over compacted base and covered. Cover soil/aggregate is uniformly spread and compacted using field-scale equipment and procedures.



Figure D-9. The density of the compacted soil is measured with a nuclear density gauge.



Figure D-10. The steel plates are tilted to facilitate exhumation.

Appendix E: ISO/EN Laboratory Installation Damage Detailed Test Results

E.1 ISO/EN Laboratory Installation Damage Test Program

Testing is done per the EN/ISO 10722. Five wide width tensile specimens are exposed to 200 cycles producing between 209 lb/ft^2 (10 kPa) minimum and 10,443 lb/ft^2 (500 kPa) maximum stress at a frequency of 1 Hz. The aggregate used is a sintered aluminum oxide with a grain size such that 100% shall pass a 10 mm sieve and 0% shall pass a 5 mm sieve. The exposed specimens and five baseline specimens are tested according to ISO/EN 10319.

Representative photos of test apparatus and aggregate are provided in Figures E-1 and E-2. Detailed test results are provided in Tables E-1 through E-5.



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Figure E-1. ISO/EN 10722, laboratory installation damage test apparatus.



Figure E-2. ISO/EN 10722, laboratory installation damage aggregate.

Table E-1. Laboratory installation damage (ISO/EN 10722) tensile test results for Tensar UX1100.

TRI Log #: E2280-68-03

PARAMETER	TEST REPLICATE NUMBER 1	2	3	4	5	MEAN	STD. DEV.	COEF. VARI.	PERCENT RETAINED
Laboratory Installation Damage (ISO/EN 10722)									
Strength Retained measured via wide width tensile (ISO/EN 10319)									
MD Number of Ribs per Specimen:	7								
MD Number of Ribs per foot:	13.52								
MD - Tensile Strength (lbs) - B	2398	2407	2447	2404	2459	**2423**	28	1.2	
MD Tensile Strength (lbs/ft) - B	4632	4649	4726	4643	4749	**4680**	54	1.2	
MD Tensile Strength (kN/m) - B	67.6	67.9	69.0	67.8	69.3	**68.3**	0.8	1.2	
MD - Tensile Strength (lbs) - E	1761	1782	1829	1855	1702	**1786**	60	3.3	
MD Tensile Strength (lbs/ft) - E	3401	3442	3533	3583	3287	**3449**	116	3.3	**74**
MD Tensile Strength (kN/m) - E	49.7	50.3	51.6	52.3	48.0	**50.4**	1.7	3.3	
MD - Elong. @ Max. Load (%) - B	10.8	10.3	10.3	10.0	10.7	**10.4**	0.3	3.1	
MD - Elong. @ Max. Load (%) - E	7.55	7.52	7.67	7.74	7.24	**7.54**	0.19	2.54	**72**

B - Baseline Unexposed
E - Exposed

MD - Machine Direction TD - Transverse/Cross Machine Direction

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table E-2. Laboratory installation damage (ISO/EN 10722) tensile test results for Tensar UX1400.

TRI Log #: E2280-68-03

PARAMETER	TEST REPLICATE NUMBER 1	2	3	4	5	MEAN	STD. DEV.	COEF. VARI.	PERCENT RETAINED
Laboratory Installation Damage (ISO/EN 10722)									
Strength Retained measured via wide width tensile (ISO/EN 10319)									
MD Number of Ribs per Specimen:	7								
MD Number of Ribs per foot:	14.04								
MD - Tensile Strength (lbs) - B	2595	2571	2513	2635	2645	**2592**	53	2.1	
MD Tensile Strength (lbs/ft) - B	5205	5157	5040	5285	5305	**5198**	107	2.1	
MD Tensile Strength (kN/m) - B	76.0	75.3	73.6	77.2	77.5	**75.9**	1.6	2.1	
MD - Tensile Strength (lbs) - E	1875	1926	2014	2196	2056	**2013**	124	6.2	
MD Tensile Strength (lbs/ft) - E	3761	3863	4040	4405	4124	**4038**	250	6.2	**78**
MD Tensile Strength (kN/m) - E	54.9	56.4	59.0	64.3	60.2	**59.0**	3.6	6.2	
MD - Elong. @ Max. Load (%) - B	11.7	10.6	10.1	10.9	10.8	**10.8**	0.6	5.4	
MD - Elong. @ Max. Load (%) - E	7.58	7.66	8.29	8.51	8.00	**8.01**	0.40	4.98	**74**

B - Baseline Unexposed
E - Exposed

MD - Machine Direction TD - Transverse/Cross Machine Direction

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table E-3. Laboratory installation damage (ISO/EN 10722) tensile test results for Tensar UX1500.

TRI Log #: E2280-68-03

PARAMETER	TEST REPLICATE NUMBER 1	2	3	4	5	MEAN	STD. DEV.	COEF. VARI.	PERCENT RETAINED
Laboratory Installation Damage (ISO/EN 10722)									
Strength Retained measured via wide width tensile (ISO/EN 10319)									
MD Number of Ribs per Specimen:	7								
MD Number of Ribs per foot:	14.04								
MD - Tensile Strength (lbs) - B	4283	4323	4190	4163	4167	**4225**	73	1.7	
MD Tensile Strength (lbs/ft) - B	8590	8671	8404	8350	8358	**8475**	147	1.7	
MD Tensile Strength (kN/m) - B	125	127	123	122	122	**124**	2	1.7	
MD - Tensile Strength (lbs) - E	3257	3770	4505	3339	3696	**3713**	495	13.3	
MD Tensile Strength (lbs/ft) - E	6533	7562	9036	6697	7413	**7448**	992	13.3	**88**
MD Tensile Strength (kN/m) - E	95.4	110	132	97.8	108	**109**	14	13.3	
MD - Elong. @ Max. Load (%) - B	12.0	12.1	11.5	11.2	11.1	**11.6**	0.5	3.9	
MD - Elong. @ Max. Load (%) - E	8.34	9.72	8.80	8.74	9.93	**9.11**	0.68	7.51	**79**

B - Baseline Unexposed
E - Exposed

MD - Machine Direction TD - Transverse/Cross Machine Direction

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table E-4. Laboratory installation damage (ISO/EN 10722) tensile test results for Tensar UX1600.

TRI Log #: E2280-68-03

PARAMETER	TEST REPLICATE NUMBER 1	2	3	4	5	MEAN	STD. DEV.	COEF. VARI.	PERCENT RETAINED
Laboratory Installation Damage (ISO/EN 10722)									
Strength Retained measured via wide width tensile (ISO/EN 10319)									
MD Number of Ribs per Specimen:	7								
MD Number of Ribs per foot:	14.04								
MD - Tensile Strength (lbs) - B	5297	5130	5294	4909	4856	**5097**	208	4.1	
MD Tensile Strength (lbs/ft) - B	10624	10289	10618	9846	9740	**10224**	418	4.1	
MD Tensile Strength (kN/m) - B	155	150	155	144	142	**149**	6	4.1	
MD - Tensile Strength (lbs) - E	3973	3854	2982	3326	3587	**3544**	402	11.3	
MD Tensile Strength (lbs/ft) - E	7969	7730	5981	6671	7194	**7109**	806	11.3	**70**
MD Tensile Strength (kN/m) - E	116	113	87.3	97.4	105	**104**	12	11.3	
MD - Elong. @ Max. Load (%) - B	11.1	10.5	11.3	10.0	10.4	**10.7**	0.5	5.0	
MD - Elong. @ Max. Load (%) - E	8.13	7.82	8.88	6.63	7.16	**7.72**	0.87	11.26	**72**

B - Baseline Unexposed
E - Exposed

MD - Machine Direction TD - Transverse/Cross Machine Direction

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table E-5. Laboratory installation damage (ISO/EN 10722) tensile test results for Tensar UX1700.

TRI Log #: E2280-68-03

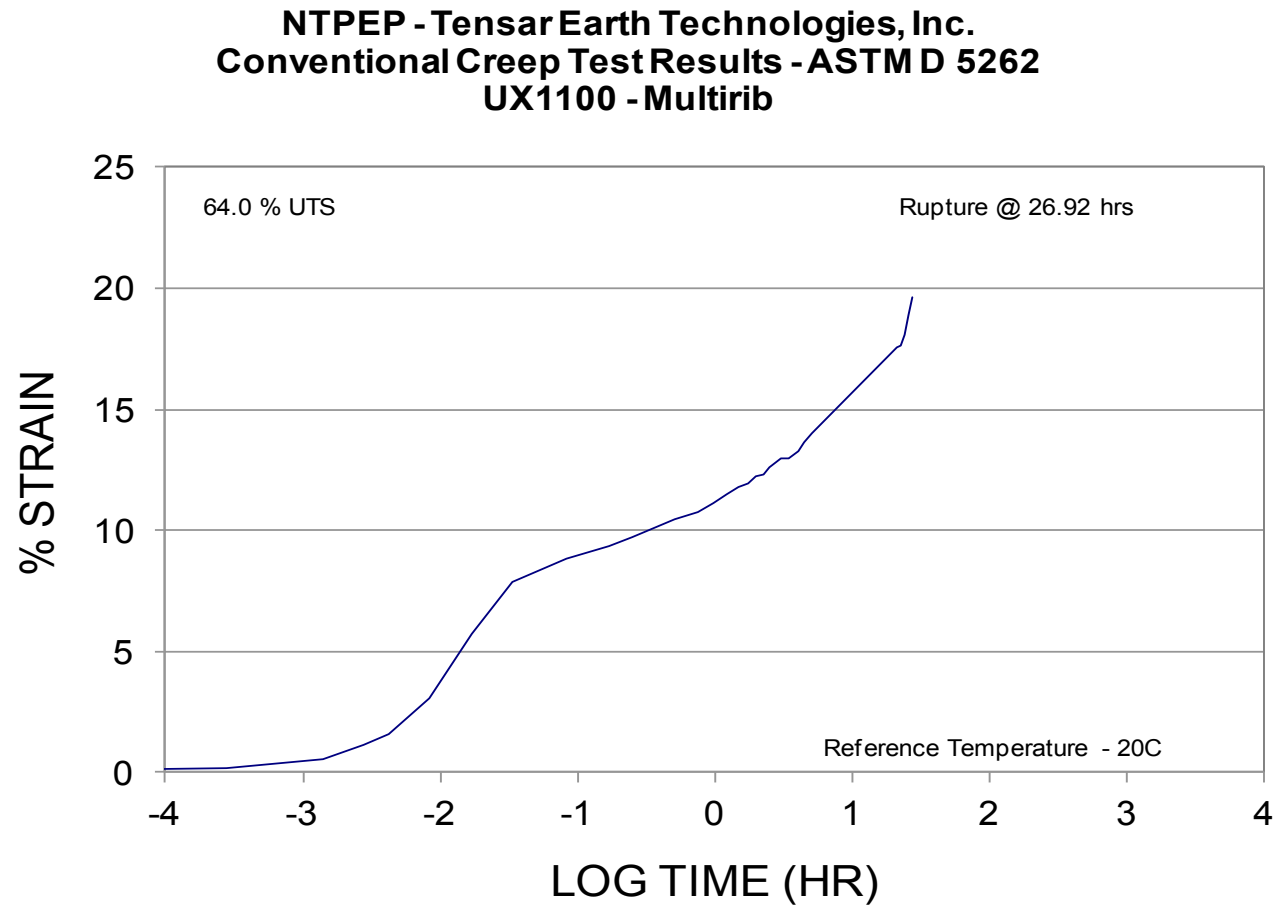
PARAMETER	TEST REPLICATE NUMBER 1	2	3	4	5	MEAN	STD. DEV.	COEF. VARI.	PERCENT RETAINED
Laboratory Installation Damage (ISO/EN 10722)									
Strength Retained measured via wide width tensile (ISO/EN 10319)									
MD Number of Ribs per Specimen:	7								
MD Number of Ribs per foot:	14.04								
MD - Tensile Strength (lbs) - B	6518	6521	6533	6543	6476	**6518**	26	0.4	
MD Tensile Strength (lbs/ft) - B	13073	13079	13103	13123	12989	**13074**	51	0.4	
MD Tensile Strength (kN/m) - B	191	191	191	192	190	**191**	1	0.4	
MD - Tensile Strength (lbs) - E	4727	5152	5139	4996	5509	**5105**	283	5.6	
MD Tensile Strength (lbs/ft) - E	9481	10333	10307	10021	11049	**10238**	568	5.6	**78**
MD Tensile Strength (kN/m) - E	138	151	150	146	161	**149**	8	5.6	
MD - Elong. @ Max. Load (%) - B	10.2	10.1	9.80	9.97	10.1	**10.0**	0.2	1.5	
MD - Elong. @ Max. Load (%) - E	7.26	7.70	7.27	7.09	7.77	**7.42**	0.30	4.03	**74**

B - Baseline Unexposed
E - Exposed

MD - Machine Direction TD - Transverse/Cross Machine Direction

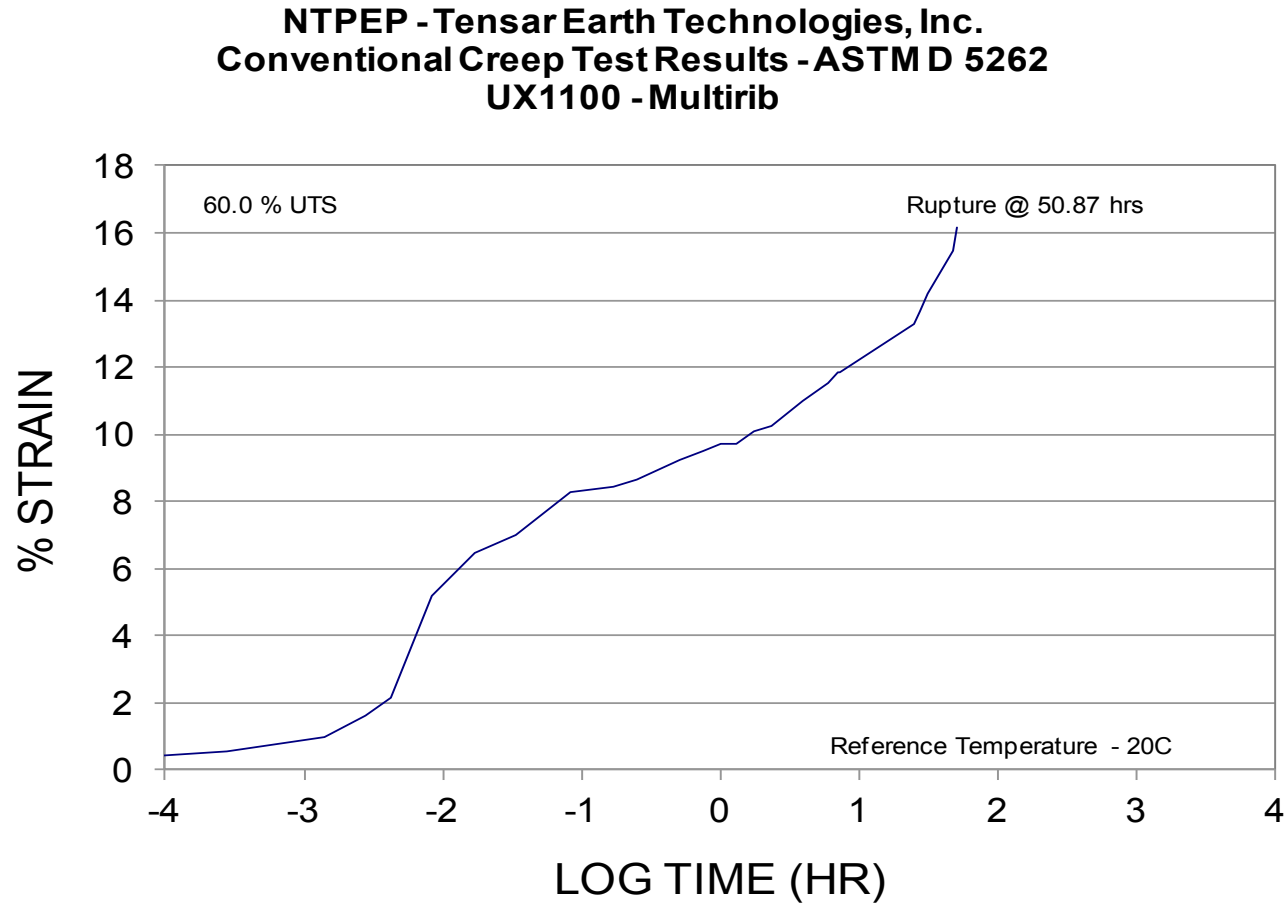
The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Appendix F: Creep Rupture Detailed Test Results



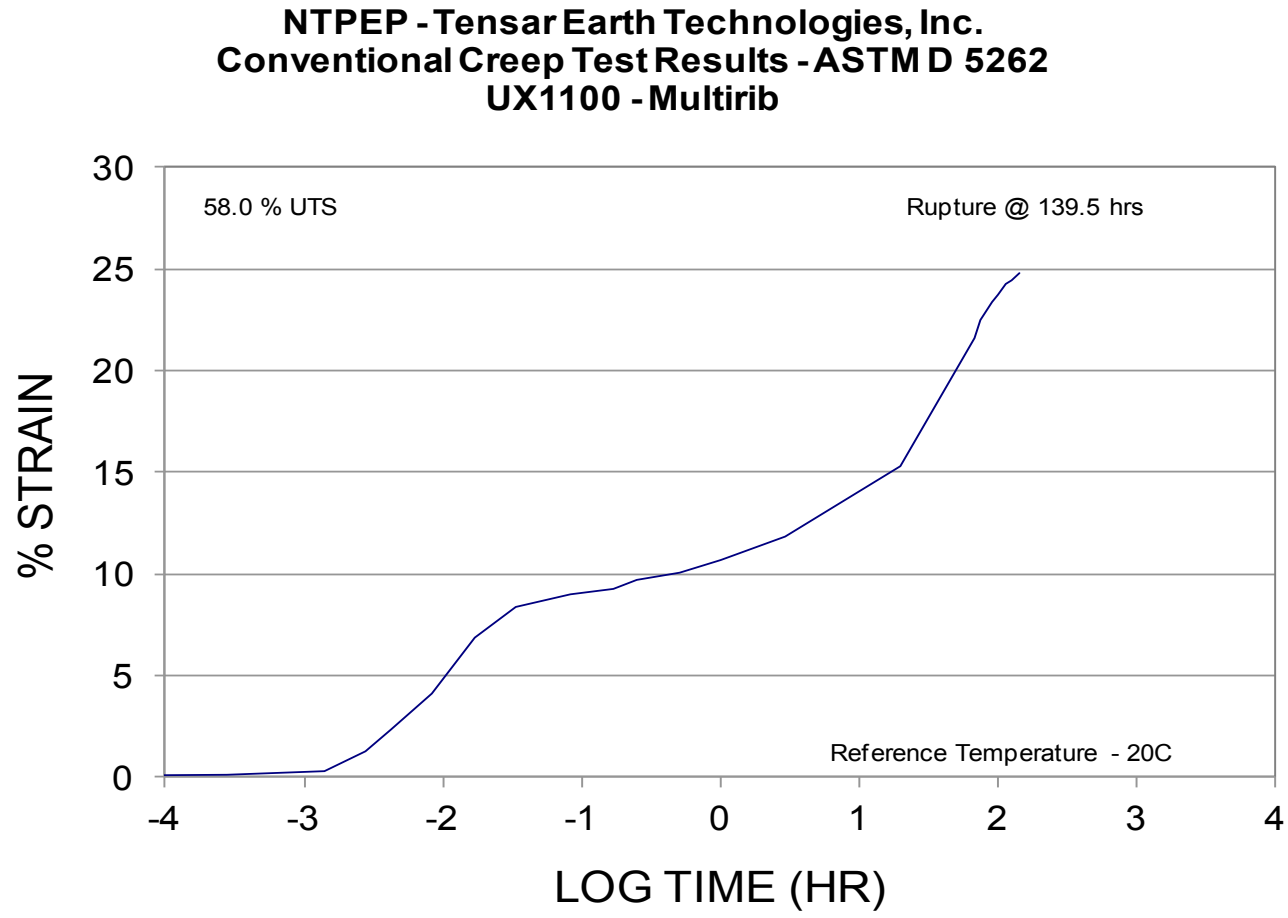
TRI Log # E2280-68-03

Figure F-1. Creep data/curve per ASTM D5262, multirib, for Tensar UX1100 at a load level of 64.0% UTS and 68°F(20°C)



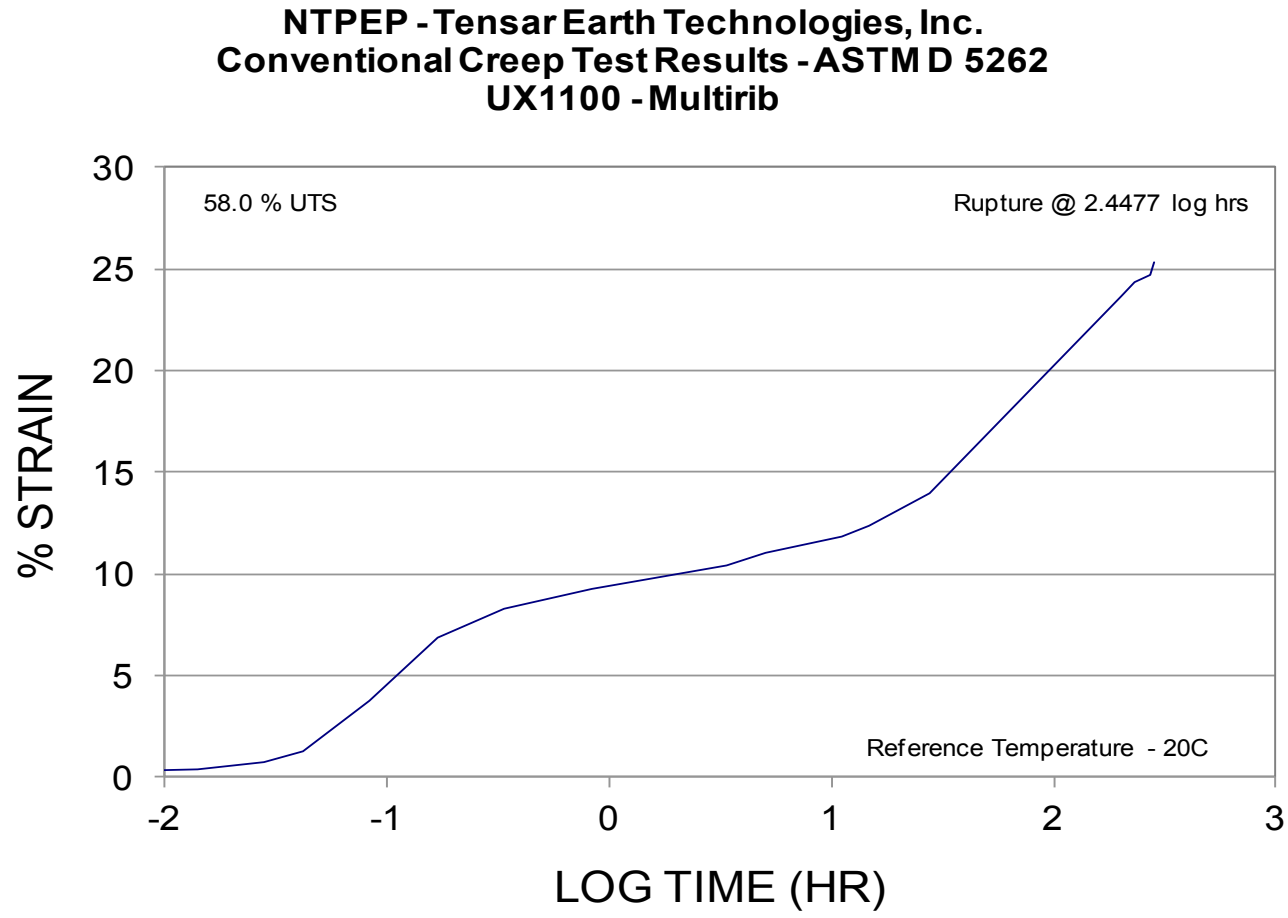
TRI Log # E2280-68-03

Figure F-2. Creep data/curve per ASTM D5262, multirib, for Tensar UX1100 at a load level of 60.0% UTS and 68°F(20°C)



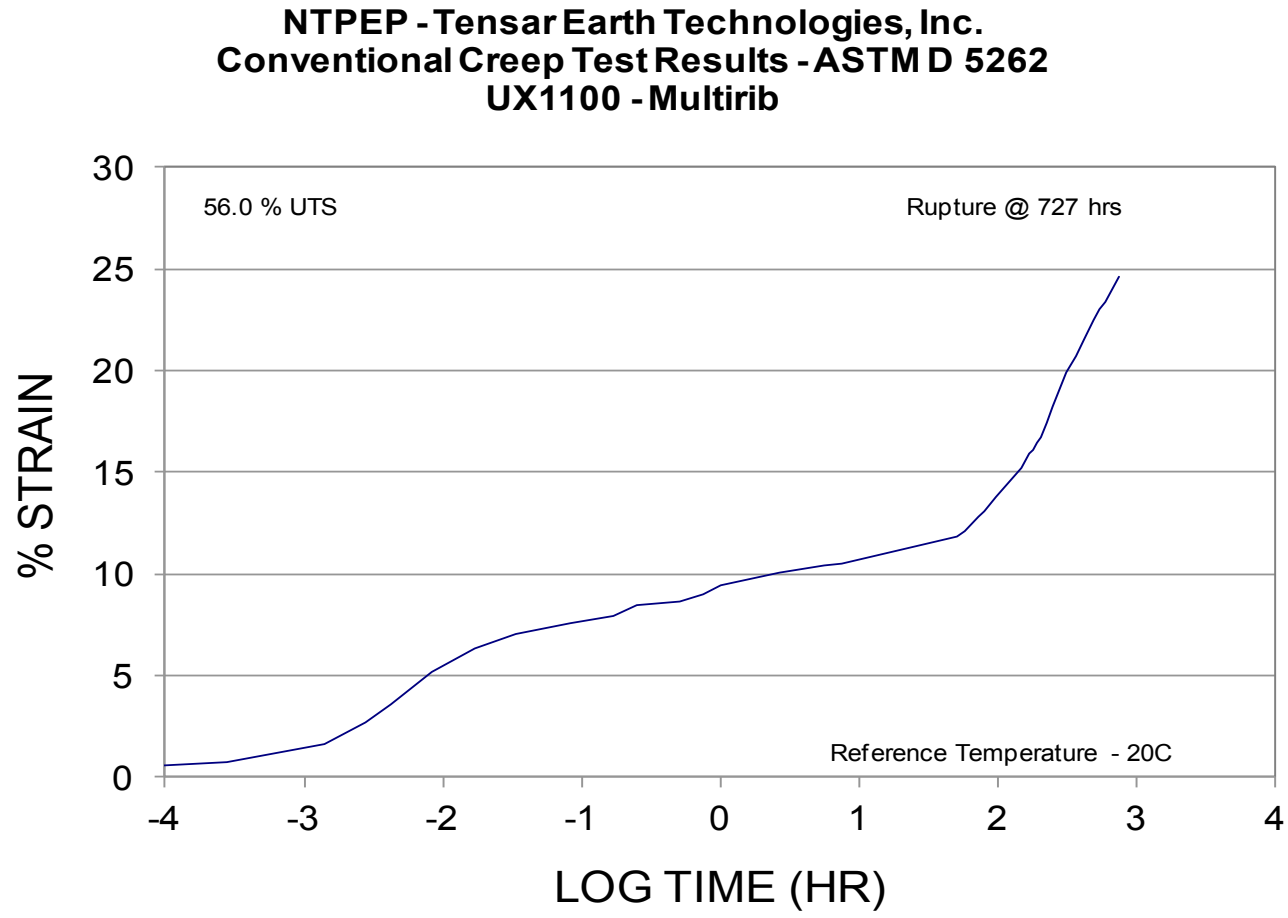
TRI Log # E2280-68-03

Figure F-3. Creep data/curve per ASTM D5262, multirib, for Tensar UX1100 at a load level of 58.0% UTS and 68°F(20°C)



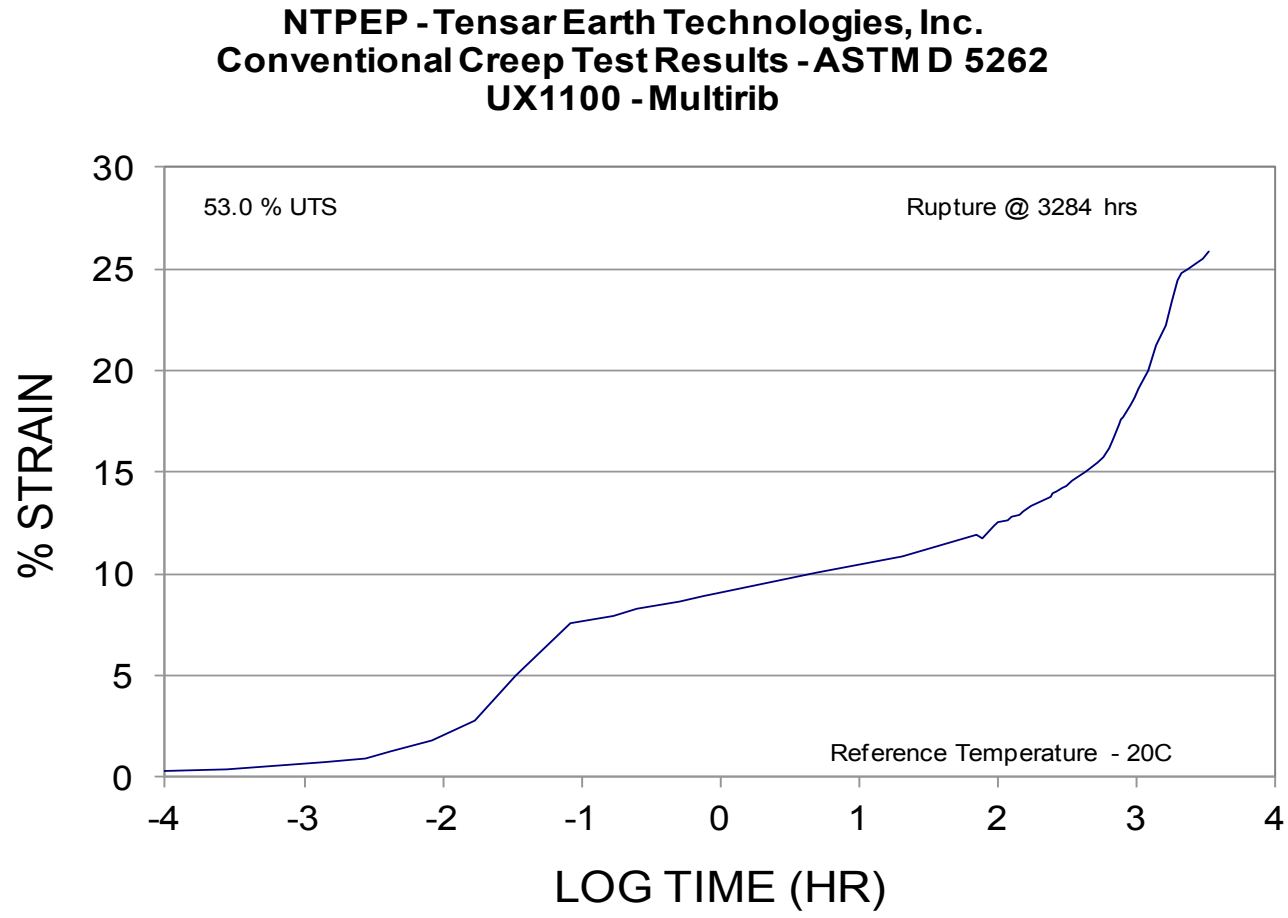
TRI Log # E2280-68-03

Figure F-4. Creep data/curve per ASTM D5262, multirib, for Tensar UX1100 at a load level of 58.0% UTS and 86°F(30°C) shifted to 68°F(20°C)



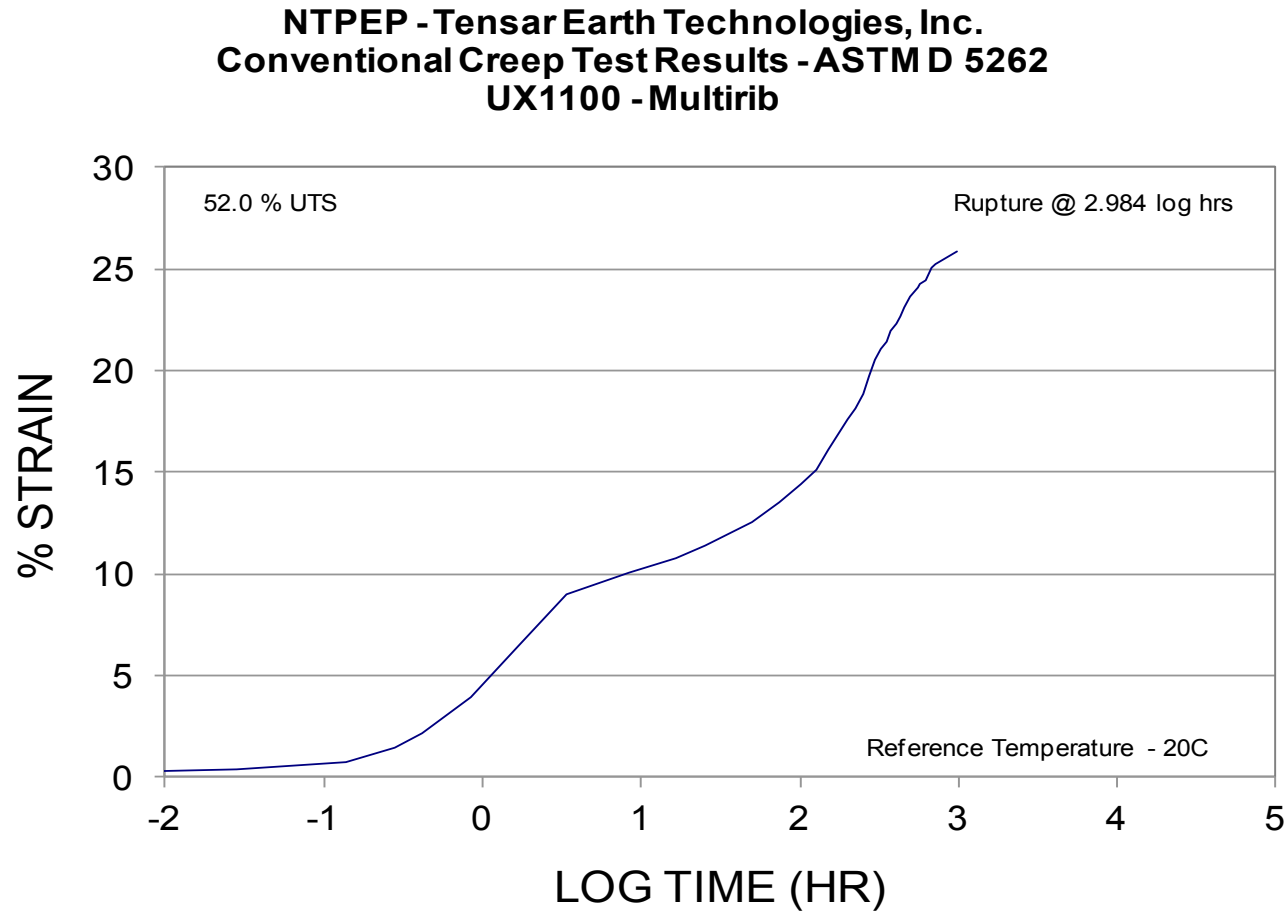
TRI Log # E2280-68-03

Figure F-5. Creep data/curve per ASTM D5262, multirib, for Tensar UX1100 at a load level of 56.0% UTS and 68°F(20°C)



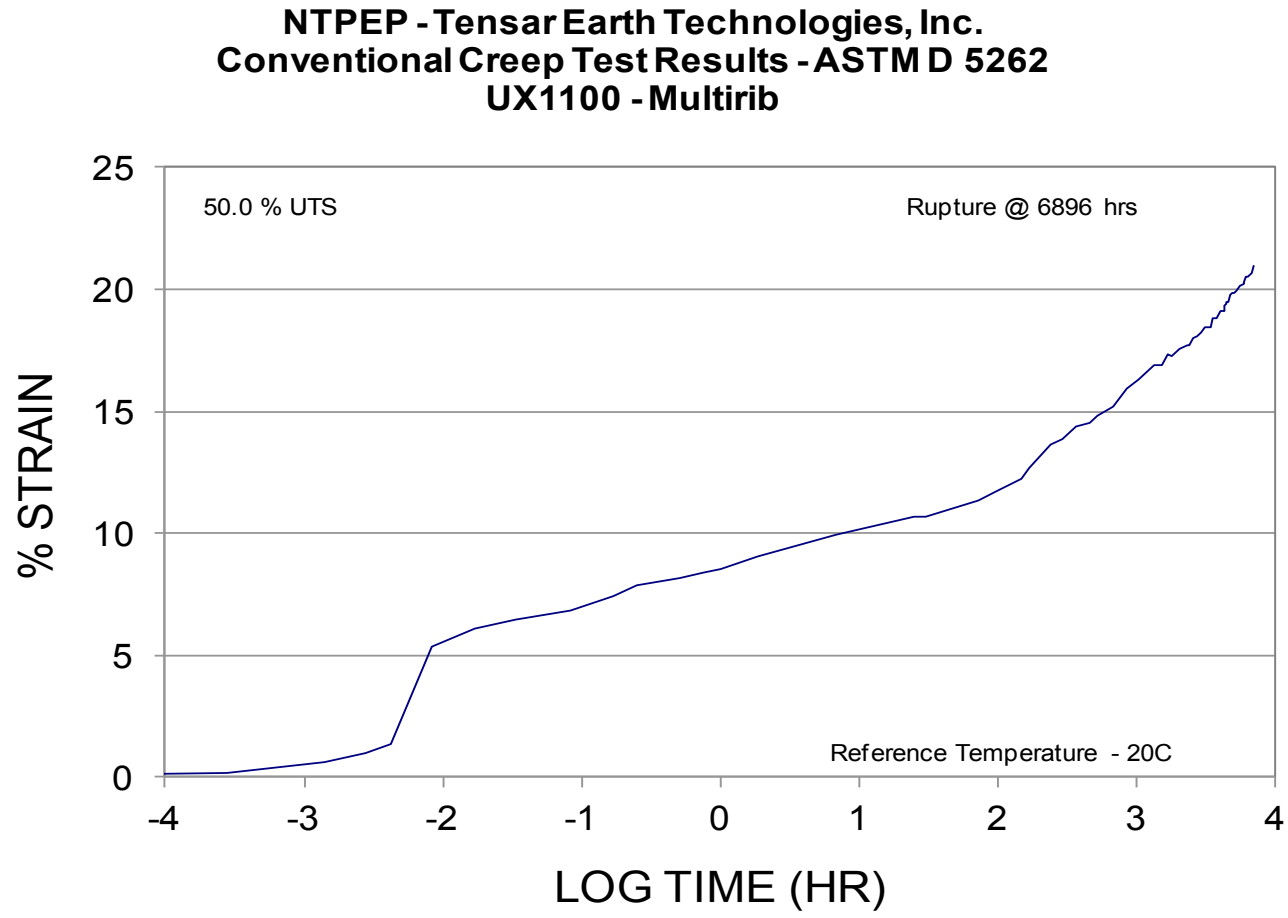
TRI Log # E2280-68-03

Figure F-6. Creep data/curve per ASTM D5262, multirib, for Tensar UX1100 at a load level of 53.0% UTS and 68°F(20°C)



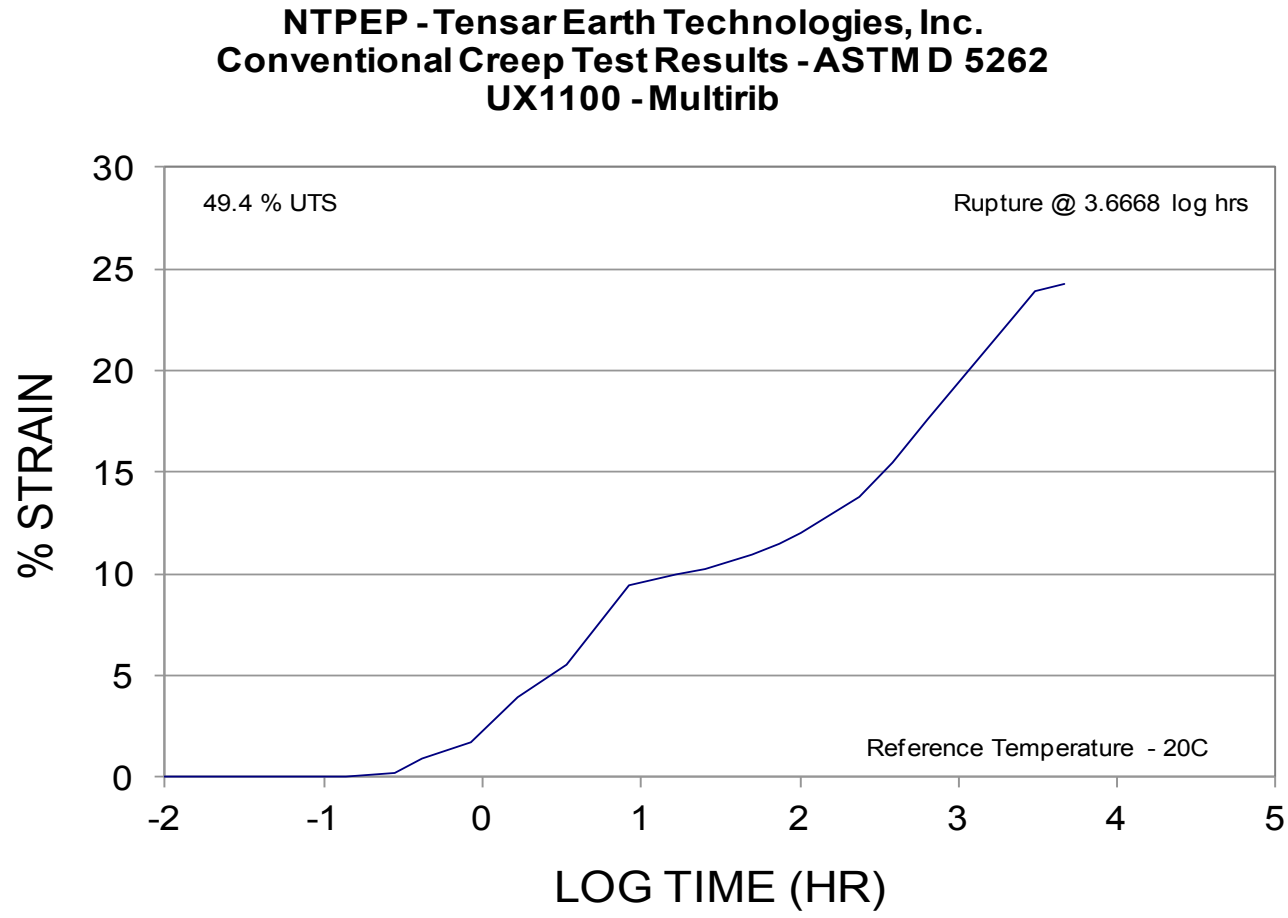
TRI Log # E2280-68-03

Figure F-7. Creep data/curve per ASTM D5262, multirib, for Tensar UX1100 at a load level of 52.0% UTS and 104°F(40°C) shifted to 68°F(20°C)



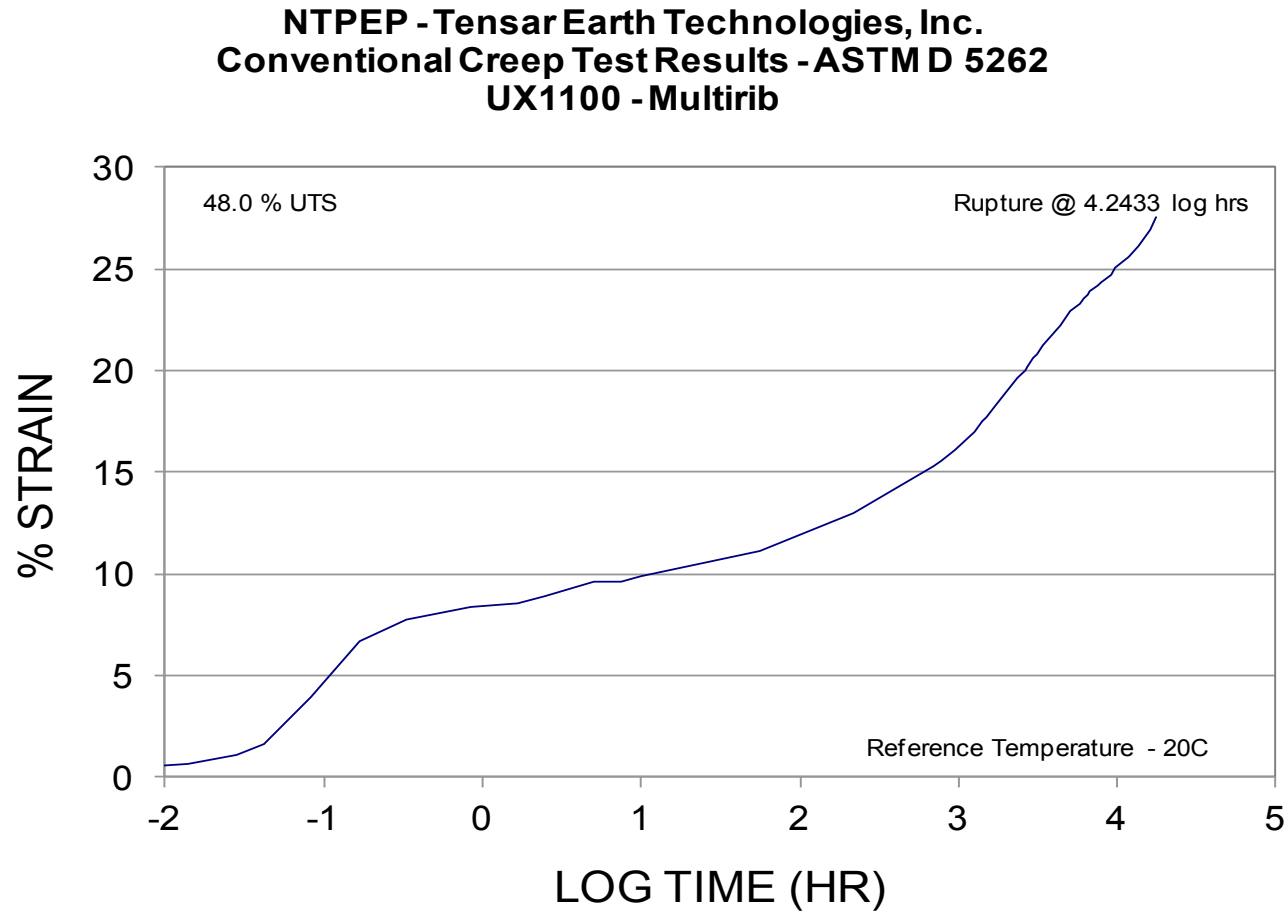
TRI Log # E2280-68-03

Figure F-8. Creep data/curve per ASTM D5262, multirib, for Tensar UX1100 at a load level of 50.0% UTS and 68°F(20°C)



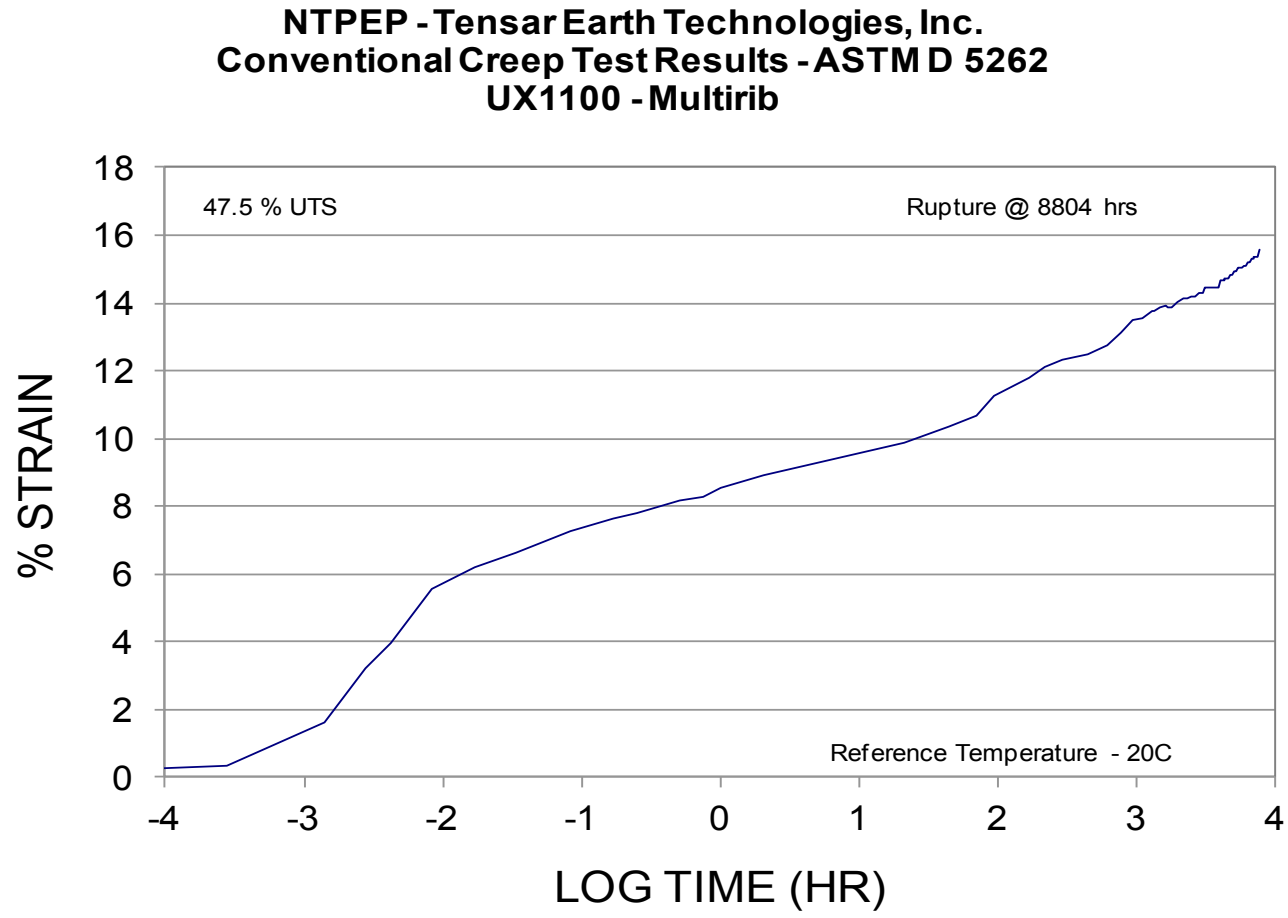
TRI Log # E2280-68-03

Figure F-9. Creep data/curve per ASTM D5262, multirib, for Tensar UX1100 at a load level of 49.4% UTS and 104°F(40°C) shifted to 68°F(20°C)



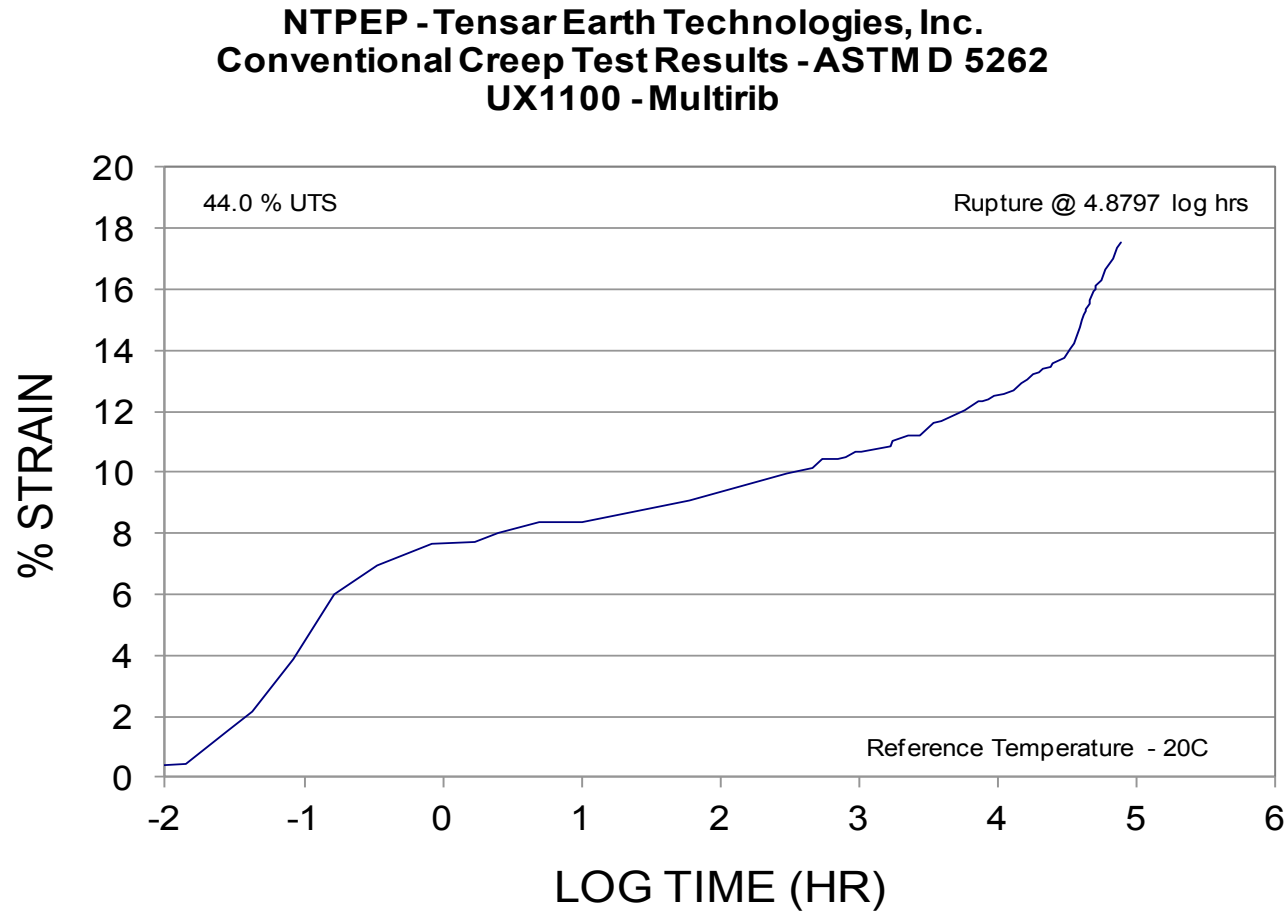
TRI Log # E2280-68-03

Figure F-10. Creep data/curve per ASTM D5262, multirib, for Tensar UX1100 at a load level of 48.0% UTS and 86°F(30°C) shifted to 68°F(20°C)



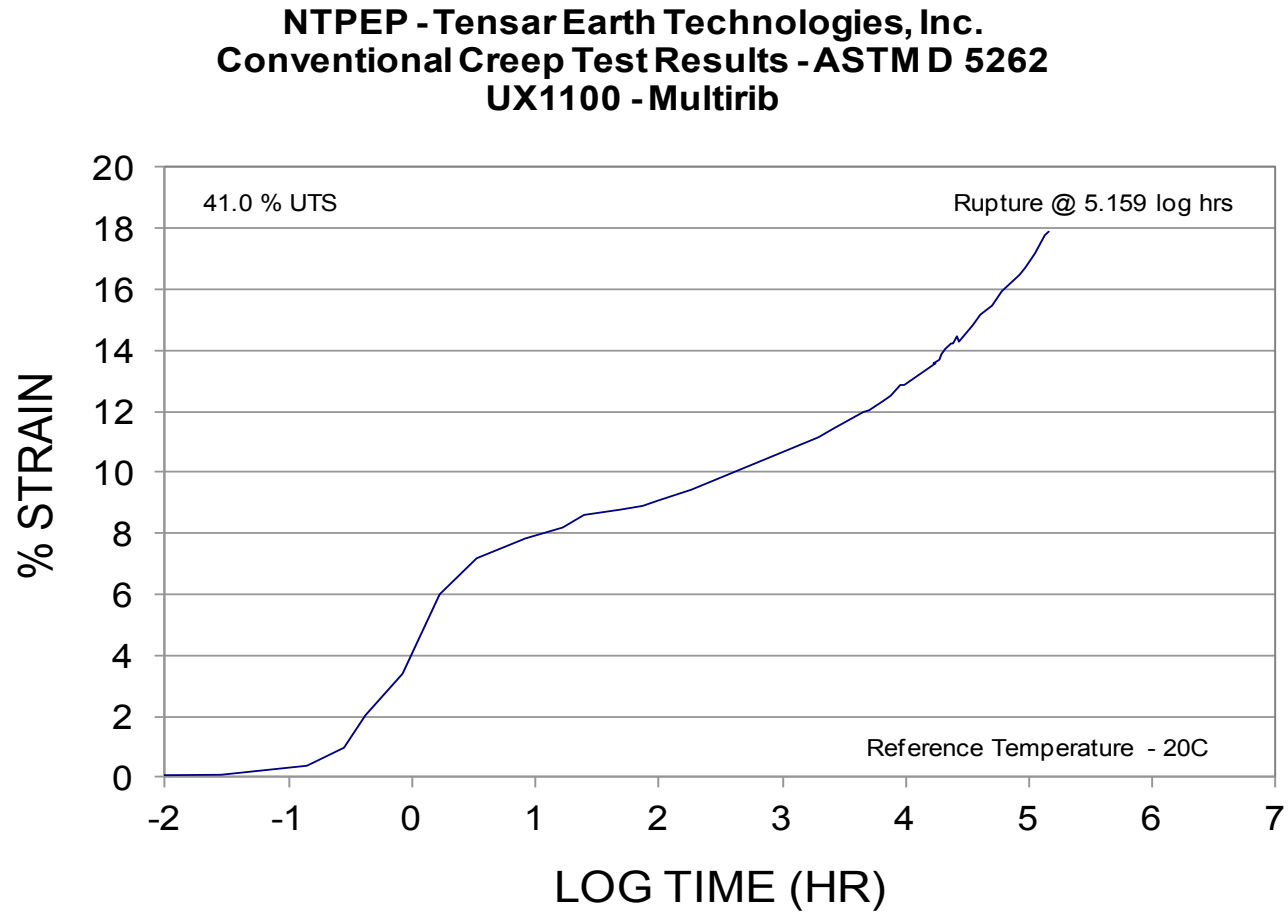
TRI Log # E2280-68-03

Figure F-11. Creep data/curve per ASTM D5262, multirib, for Tensar UX1100 at a load level of 47.5% UTS and 68°F(20°C)



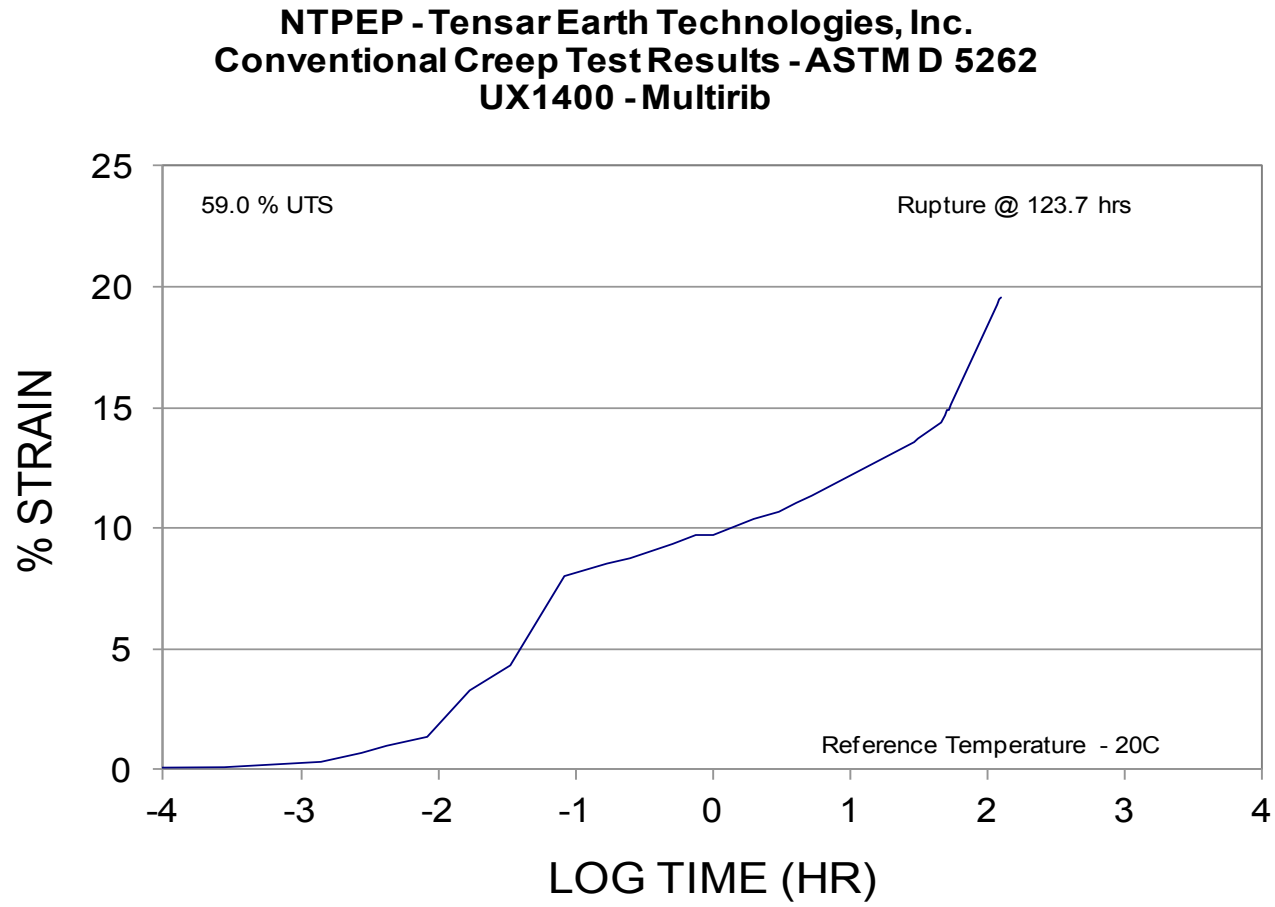
TRI Log # E2280-68-03

Figure F-12. Creep data/curve per ASTM D5262, multirib, for Tensar UX1100 at a load level of 44.0% UTS and 86°F(30°C) shifted to 68°F(20°C)



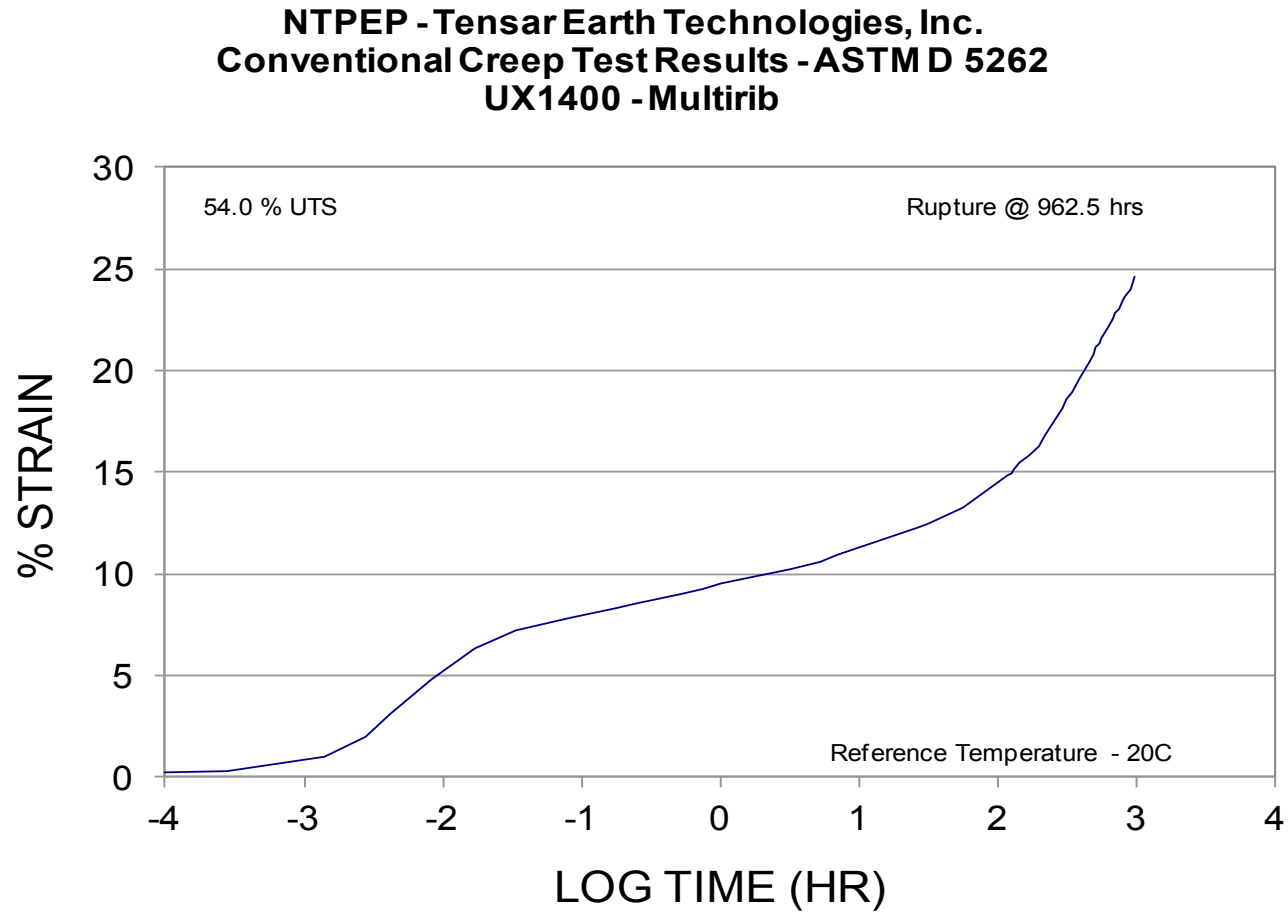
TRI Log # E2280-68-03

Figure F-13. Creep data/curve per ASTM D5262, multirib, for Tensar UX1100 at a load level of 41.0% UTS and 104°F(40°C) shifted to 68°F(20°C)



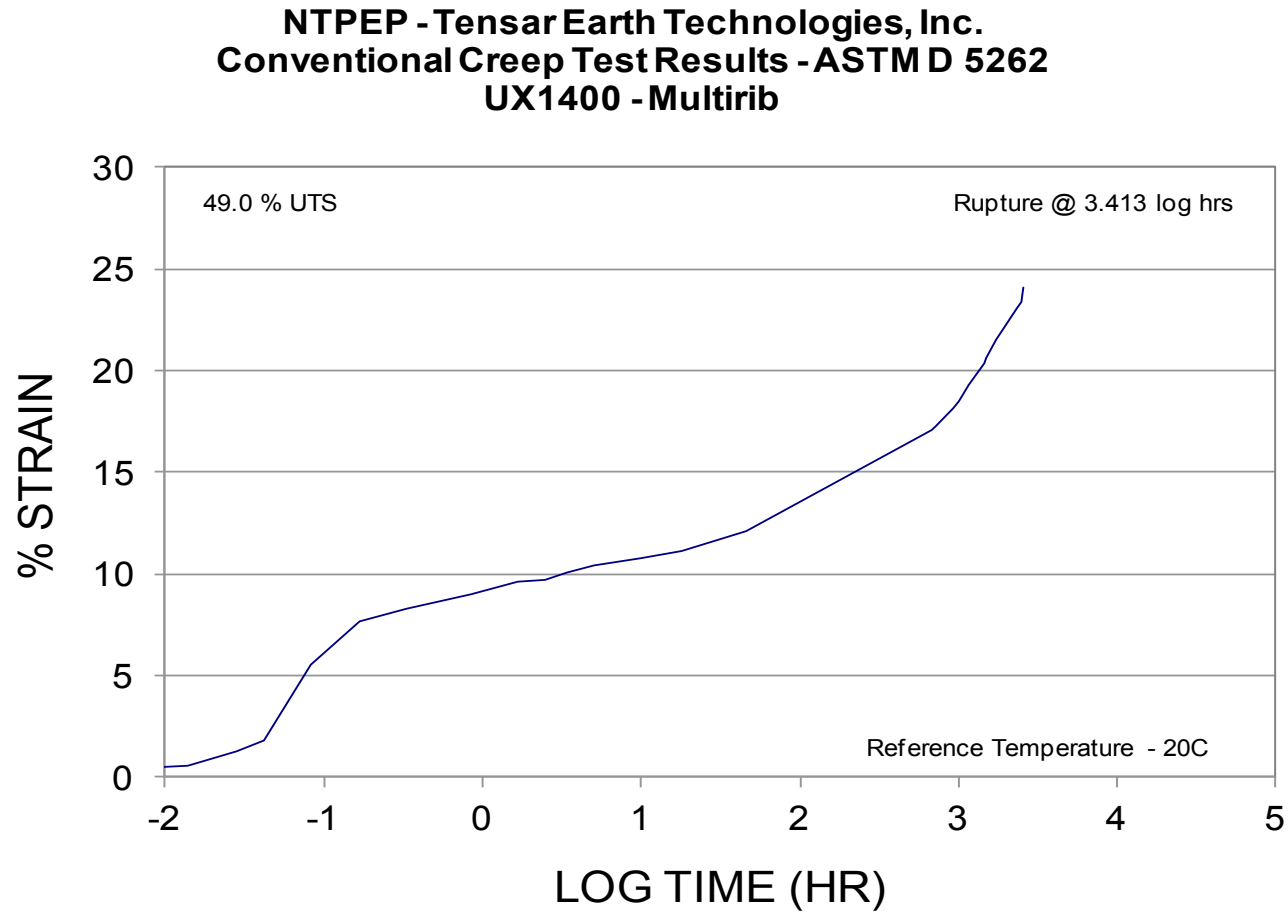
TRI Log # E2280-68-03

Figure F-14. Creep data/curve per ASTM D5262, multirib, for Tensar UX1400 at a load level of 59.0% UTS and 68°F(20°C)



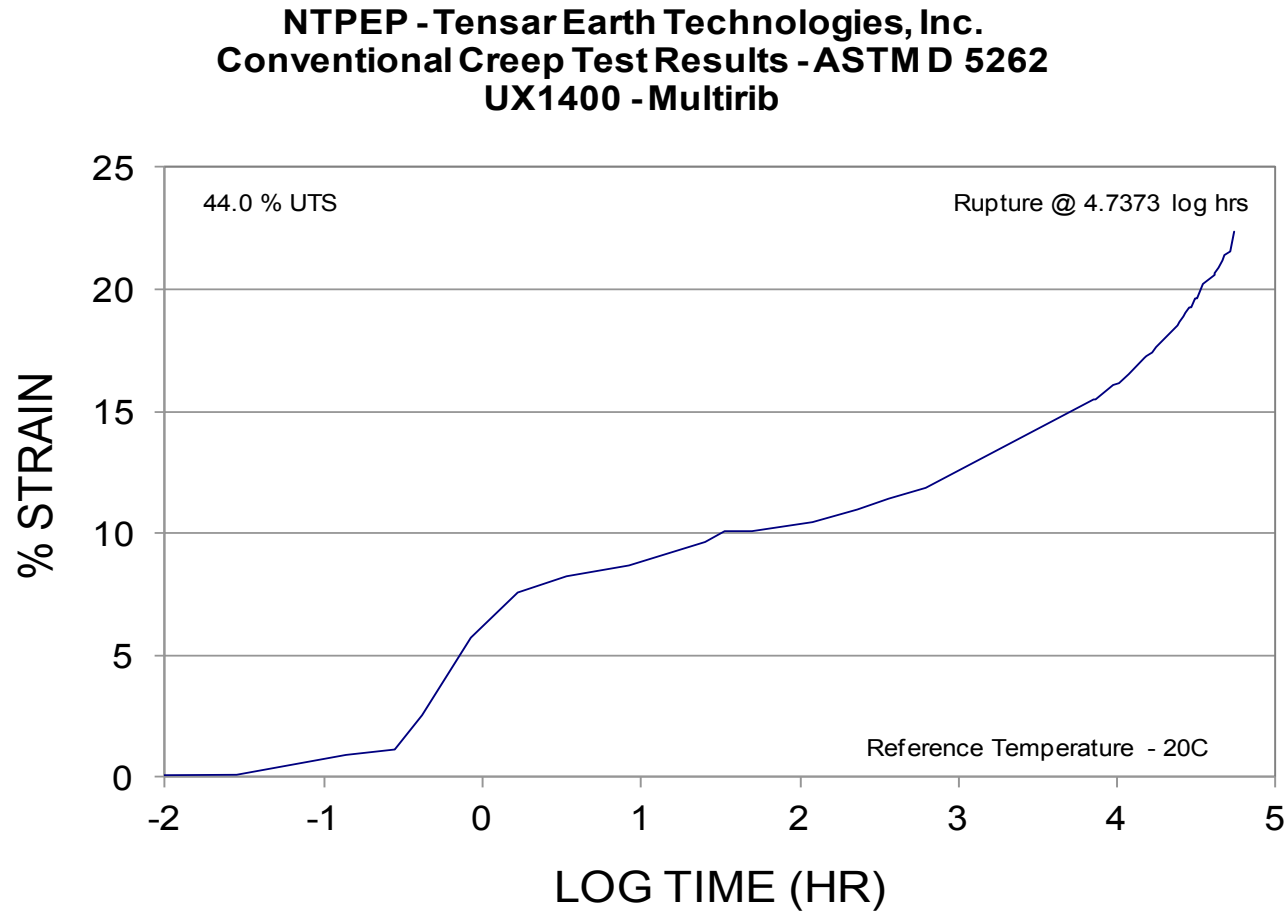
TRI Log # E2280-68-03

Figure F-15. Creep data/curve per ASTM D5262, multirib, for Tensar UX1400 at a load level of 54.0% UTS and 68°F(20°C)



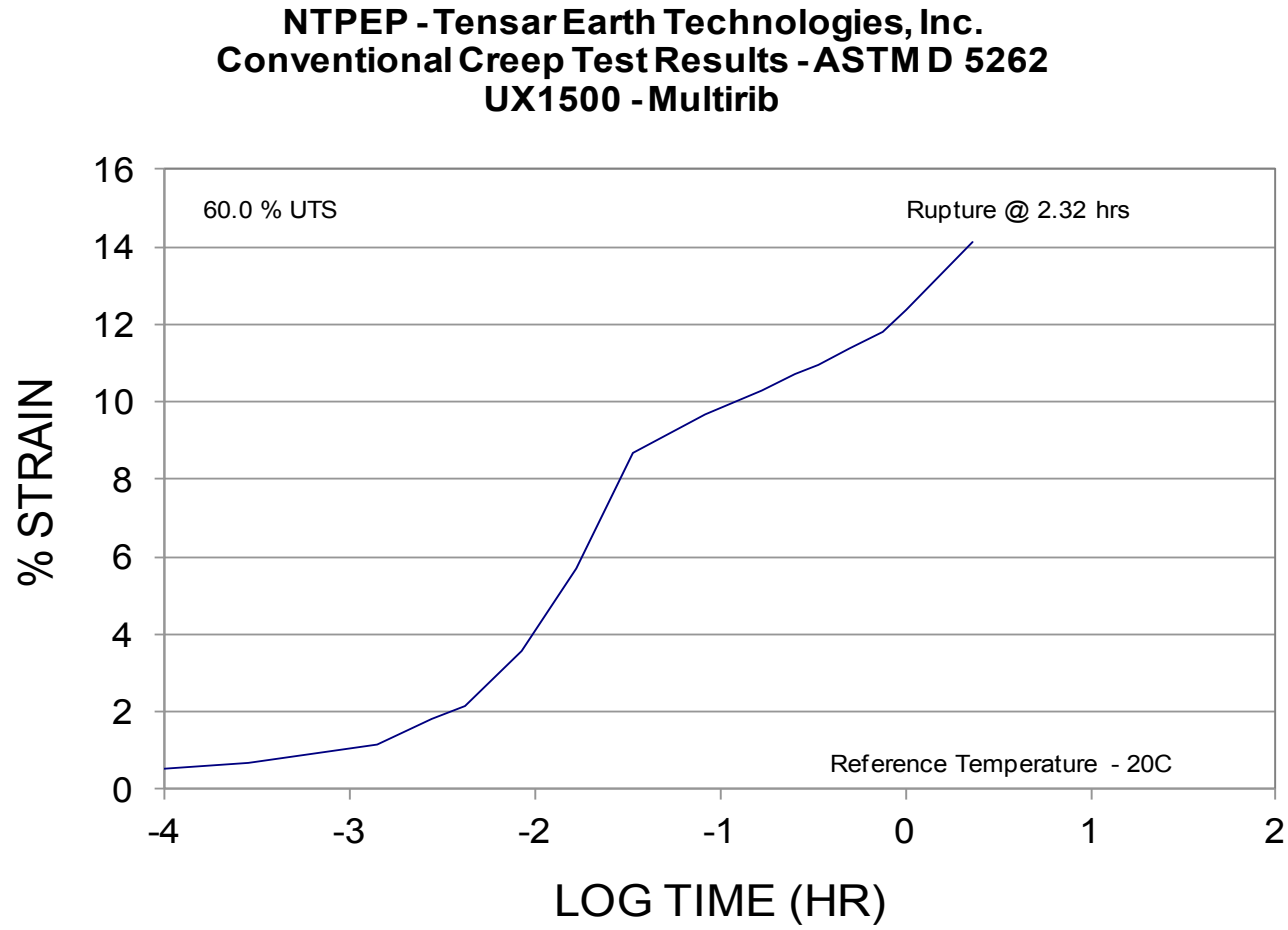
TRI Log # E2280-68-03

Figure F-16. Creep data/curve per ASTM D5262, multirib, for Tensar UX1400 at a load level of 49.0% UTS and 86°F(30°C) shifted to 68°F(20°C)



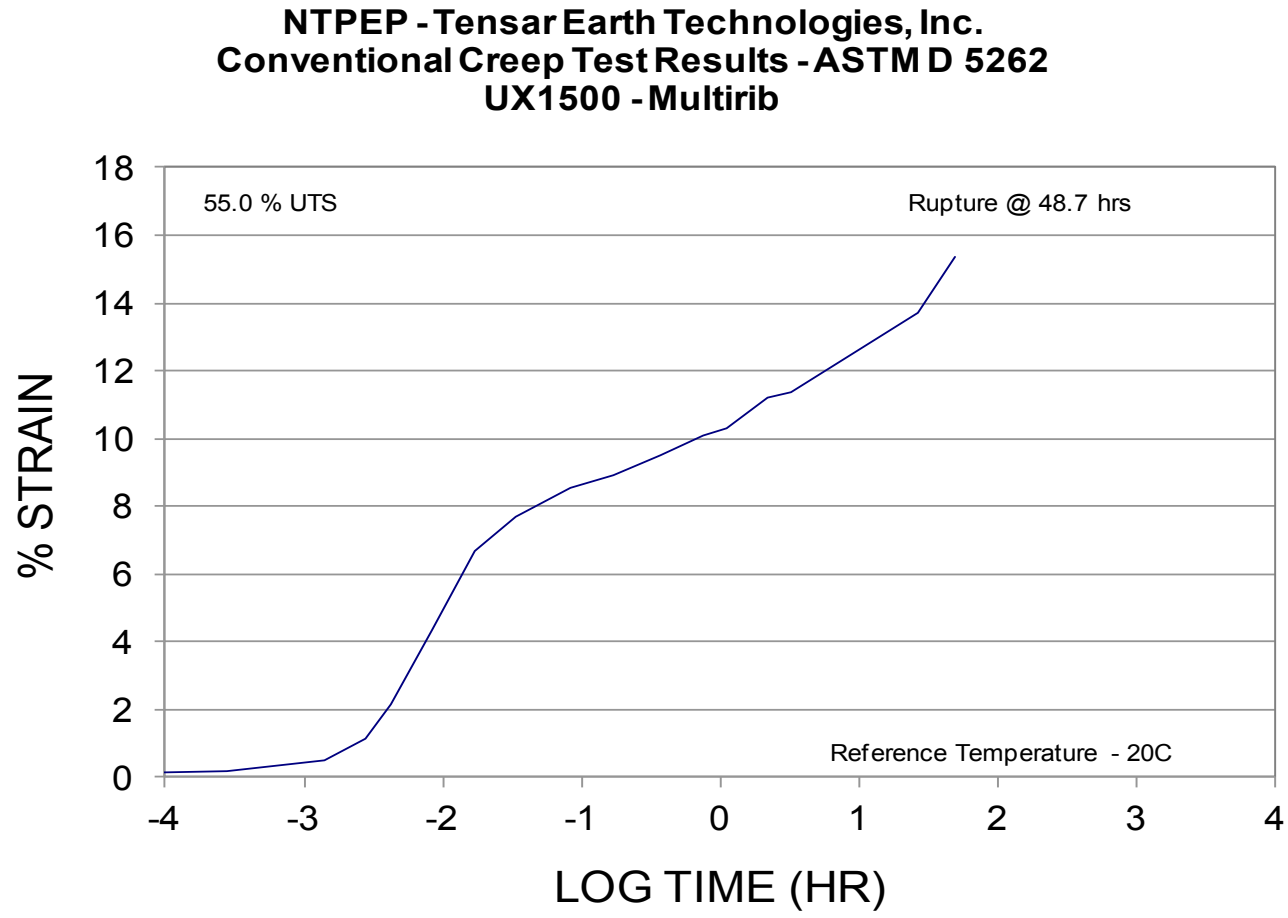
TRI Log # E2280-68-03

Figure F-17. Creep data/curve per ASTM D5262, multirib, for Tensar UX1400 at a load level of 44.0% UTS and 104°F(40°C) shifted to 68°F(20°C)



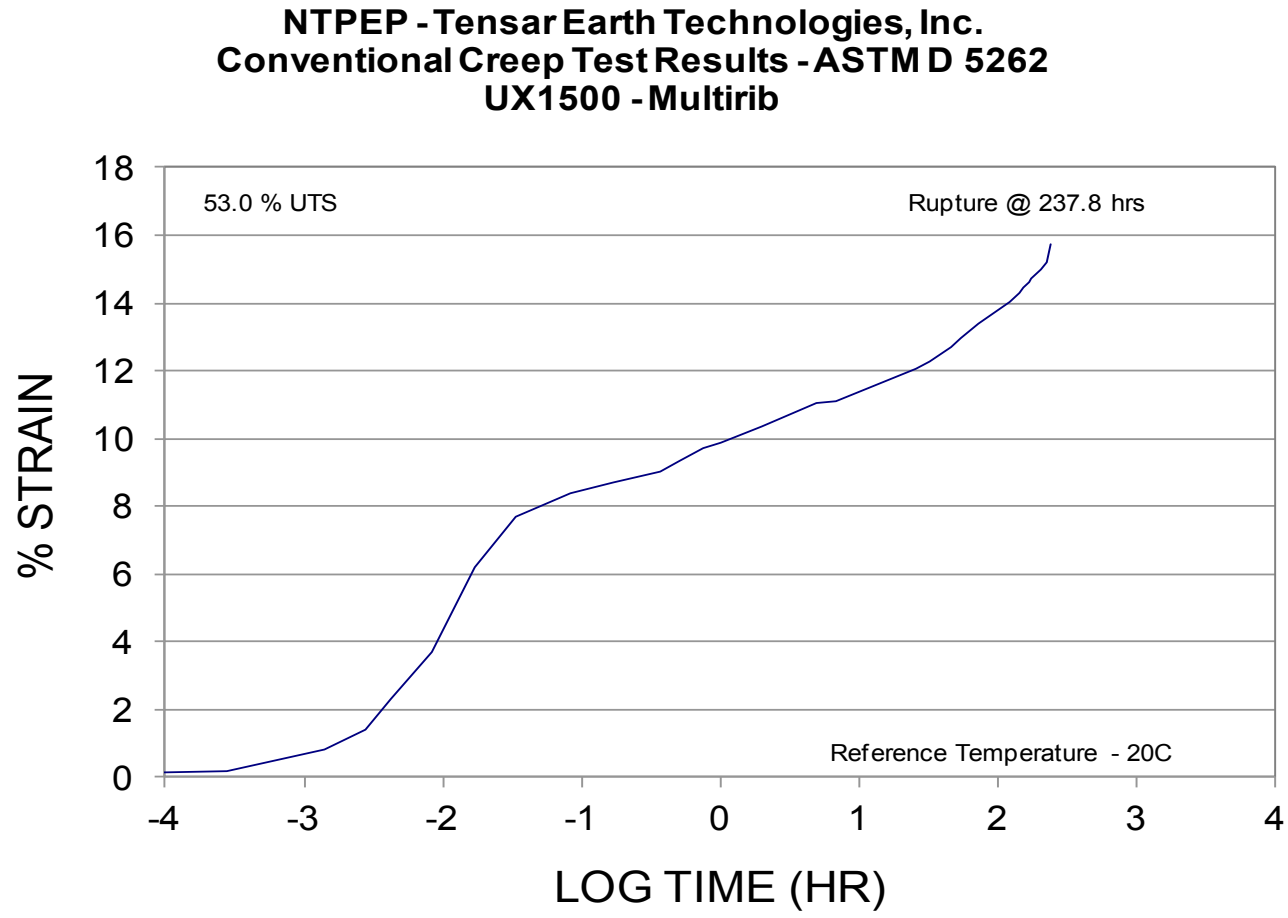
TRI Log # E2280-68-03

Figure F-18. Creep data/curve per ASTM D5262, multirib, for Tensar UX1500 at a load level of 60.0% UTS and 68°F(20°C)



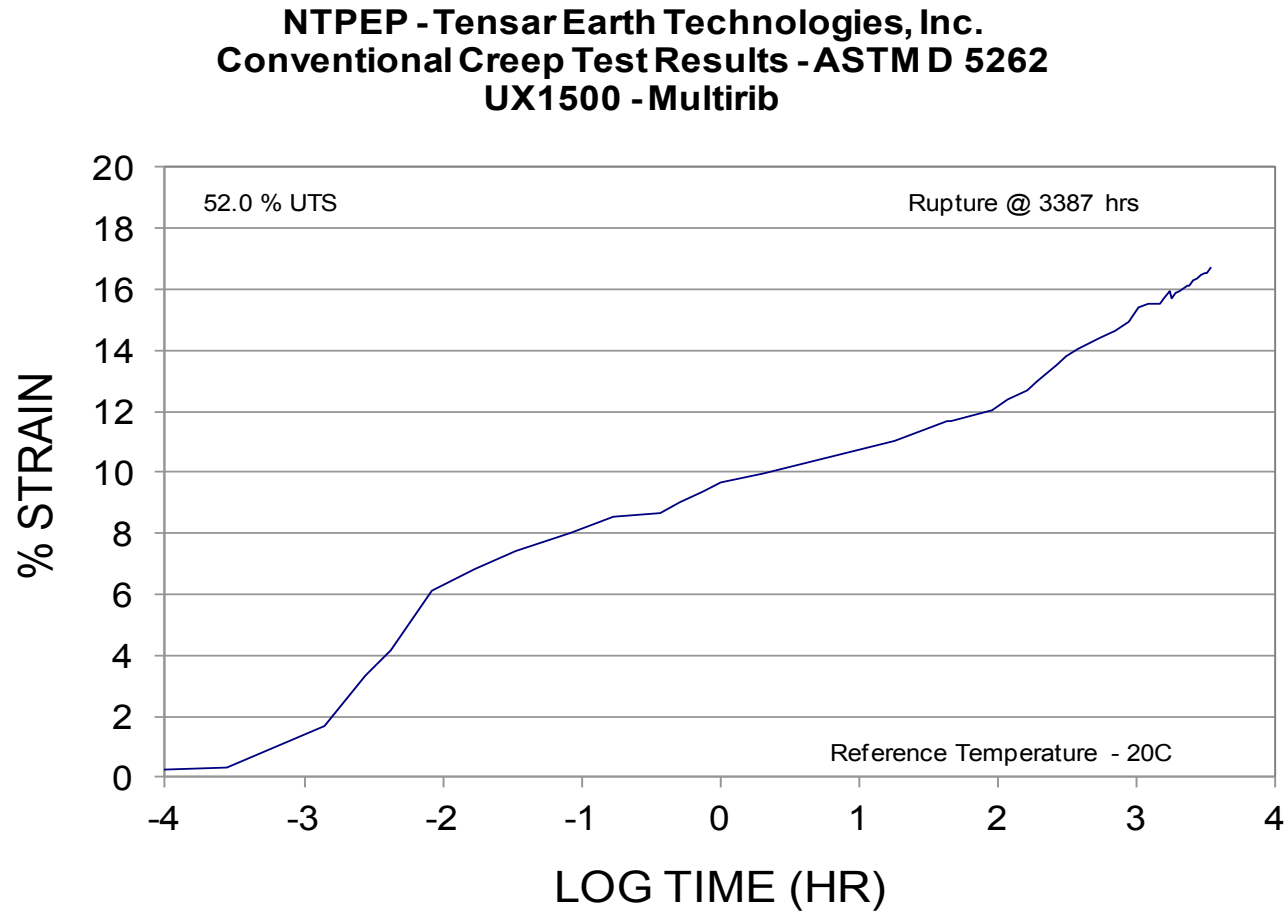
TRI Log # E2280-68-03

Figure F-19. Creep data/curve per ASTM D5262, multirib, for Tensar UX1500 at a load level of 55.0% UTS and 68°F(20°C)



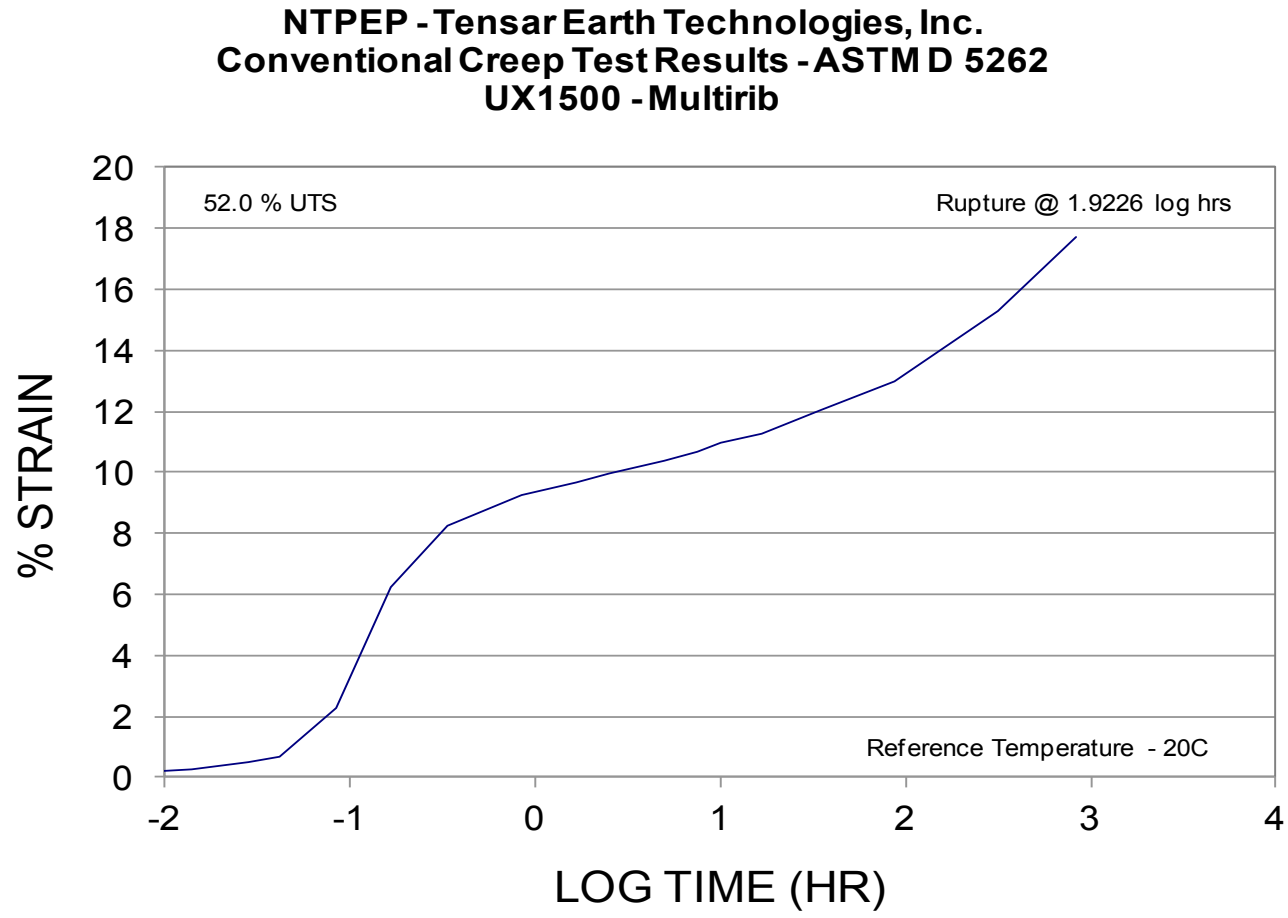
TRI Log # E2280-68-03

Figure F-20. Creep data/curve per ASTM D5262, multirib, for Tensar UX1500 at a load level of 53.0% UTS and 68°F(20°C)



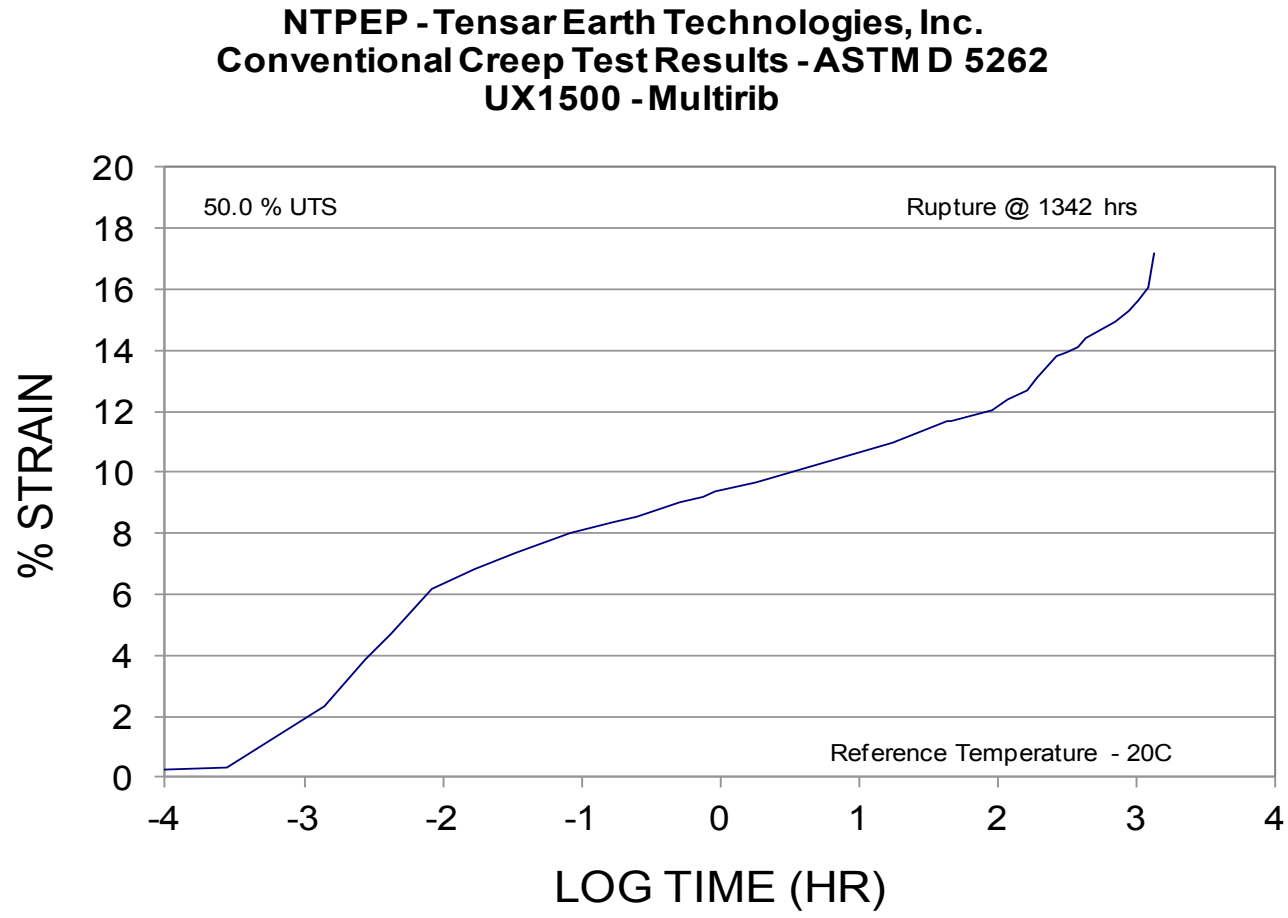
TRI Log # E2280-68-03

Figure F-21. Creep data/curve per ASTM D5262, multirib, for Tensar UX1500 at a load level of 52.0% UTS and 68°F(20°C)



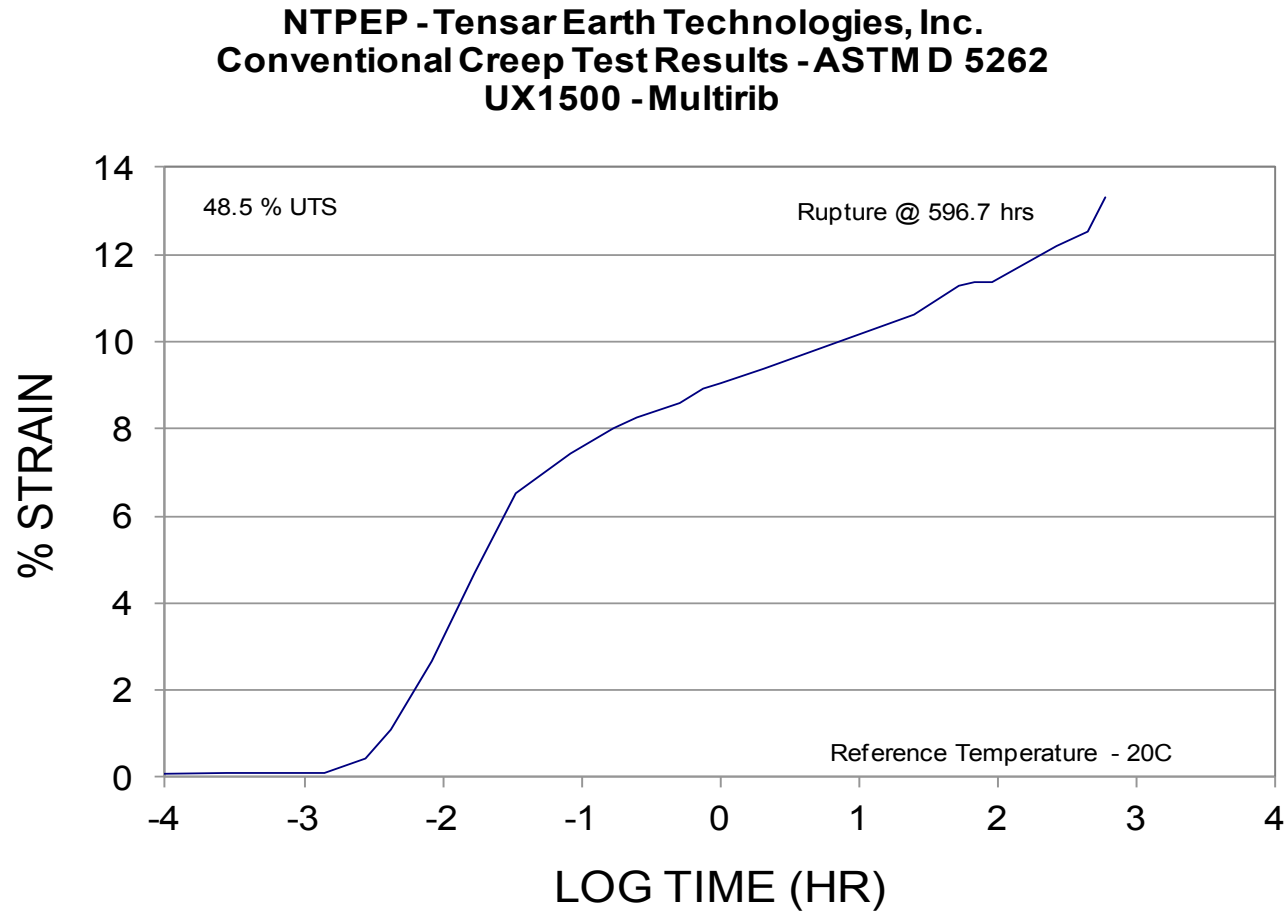
TRI Log # E2280-68-03

Figure F-22. Creep data/curve per ASTM D5262, multirib, for Tensar UX1500 at a load level of 52.0% UTS and 86°F(30°C) shifted to 68°F(20°C)



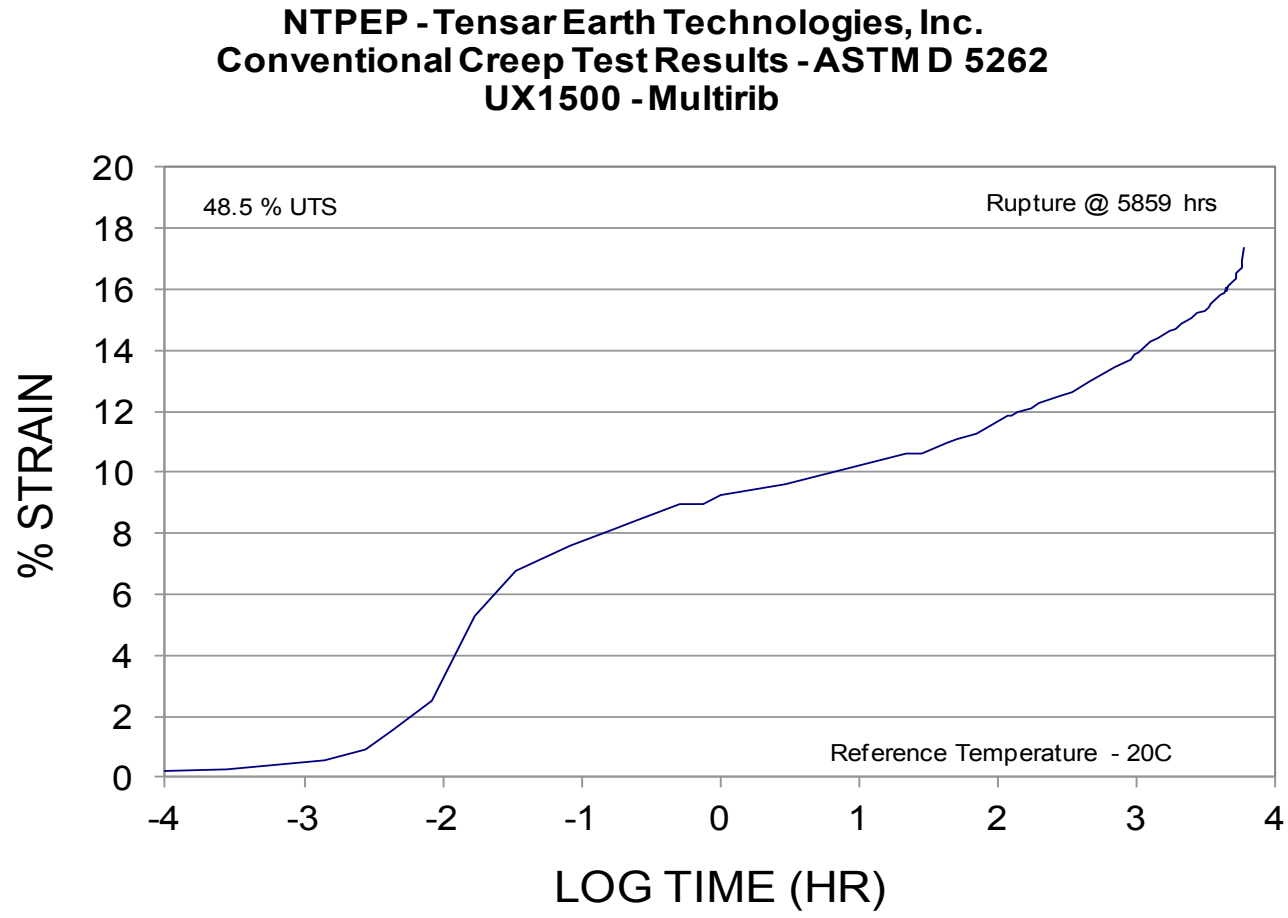
TRI Log # E2280-68-03

Figure F-23. Creep data/curve per ASTM D5262, multirib, for Tensar UX1500 at a load level of 50.0% UTS and 68°F(20°C)



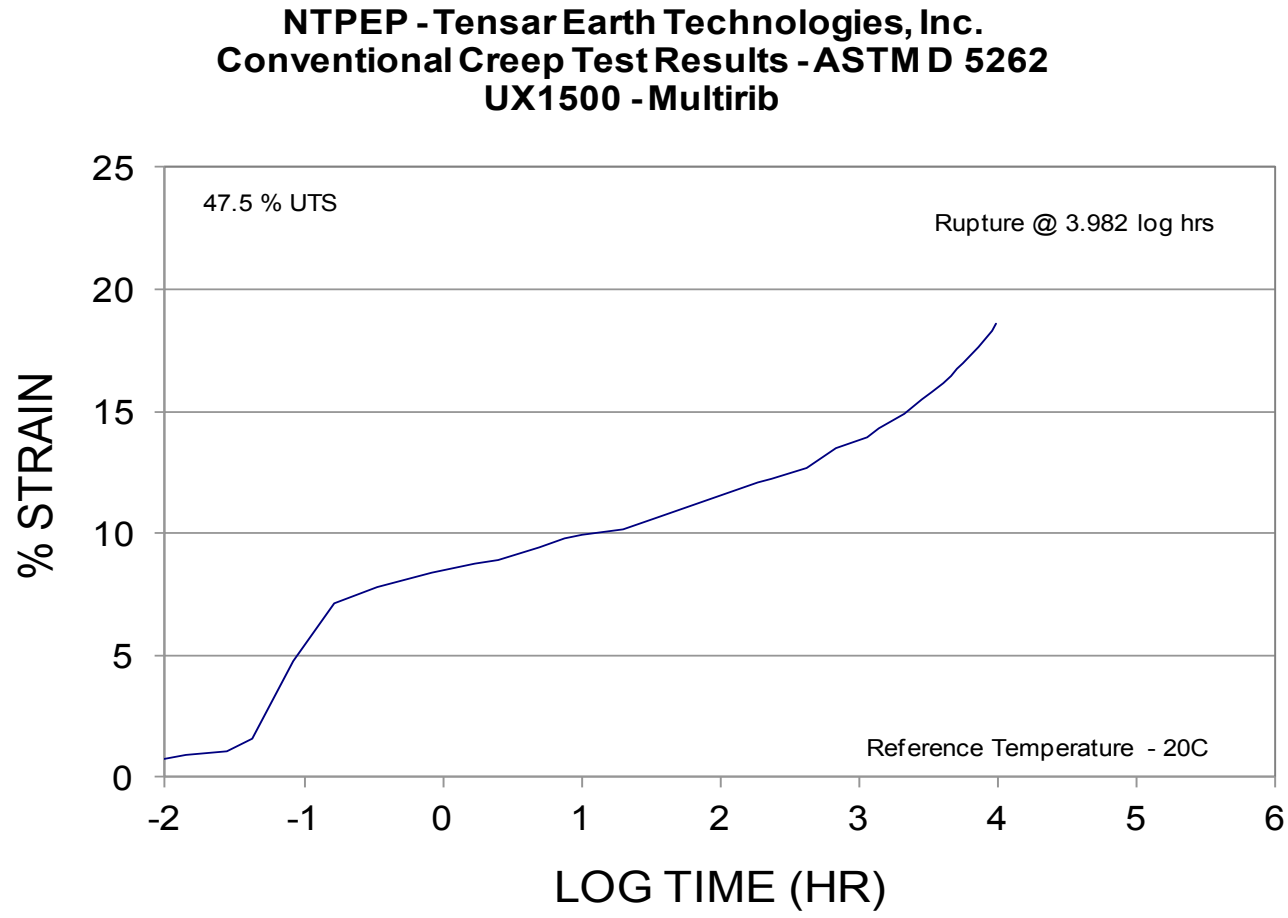
TRI Log # E2280-68-03

Figure F-24. Creep data/curve per ASTM D5262, multirib, for Tensar UX1500 at a load level of 48.5% UTS and 68°F(20°C)



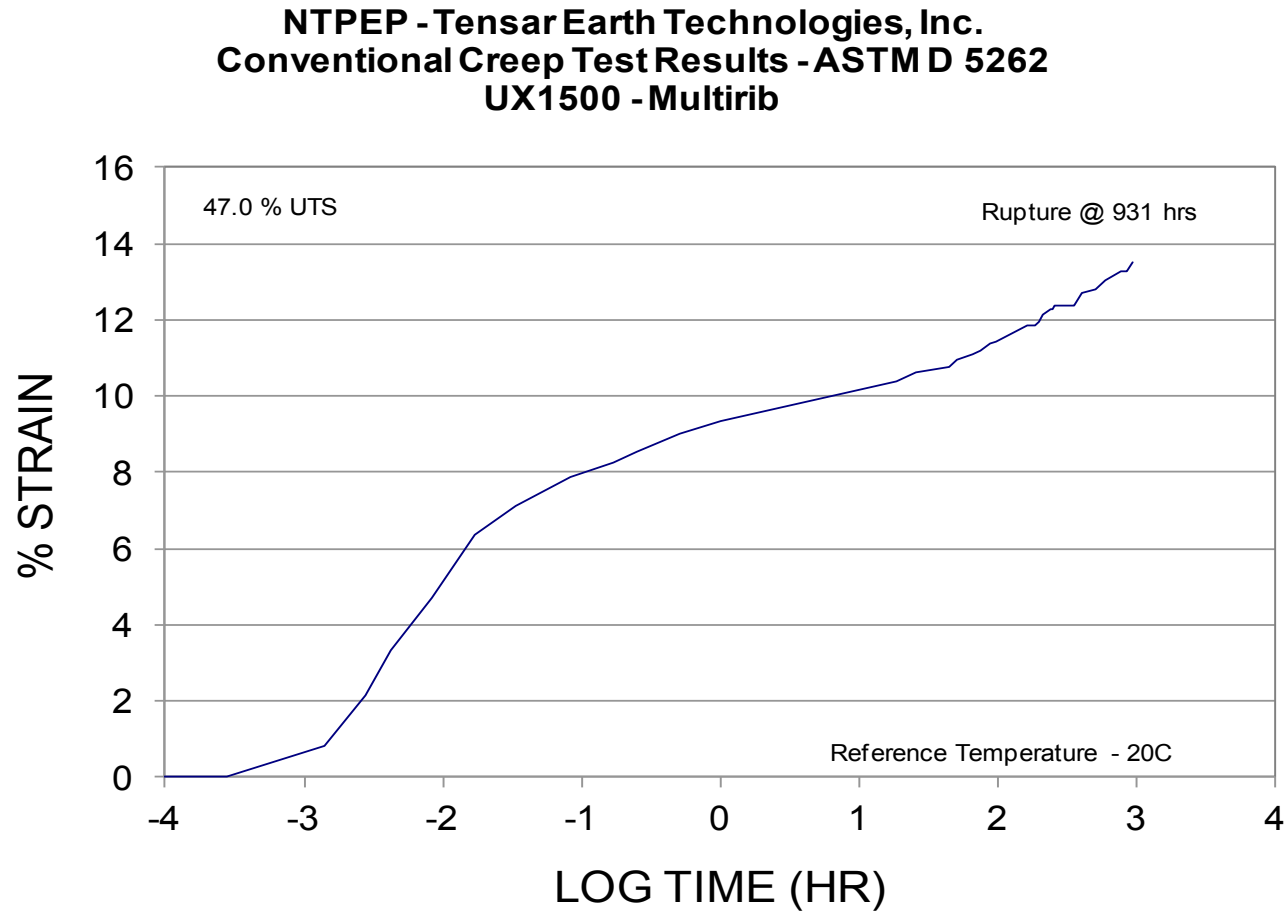
TRI Log # E2280-68-03

Figure F-25. Creep data/curve per ASTM D5262, multirib, for Tensar UX1500 at a load level of 48.5% UTS and 68°F(20°C)



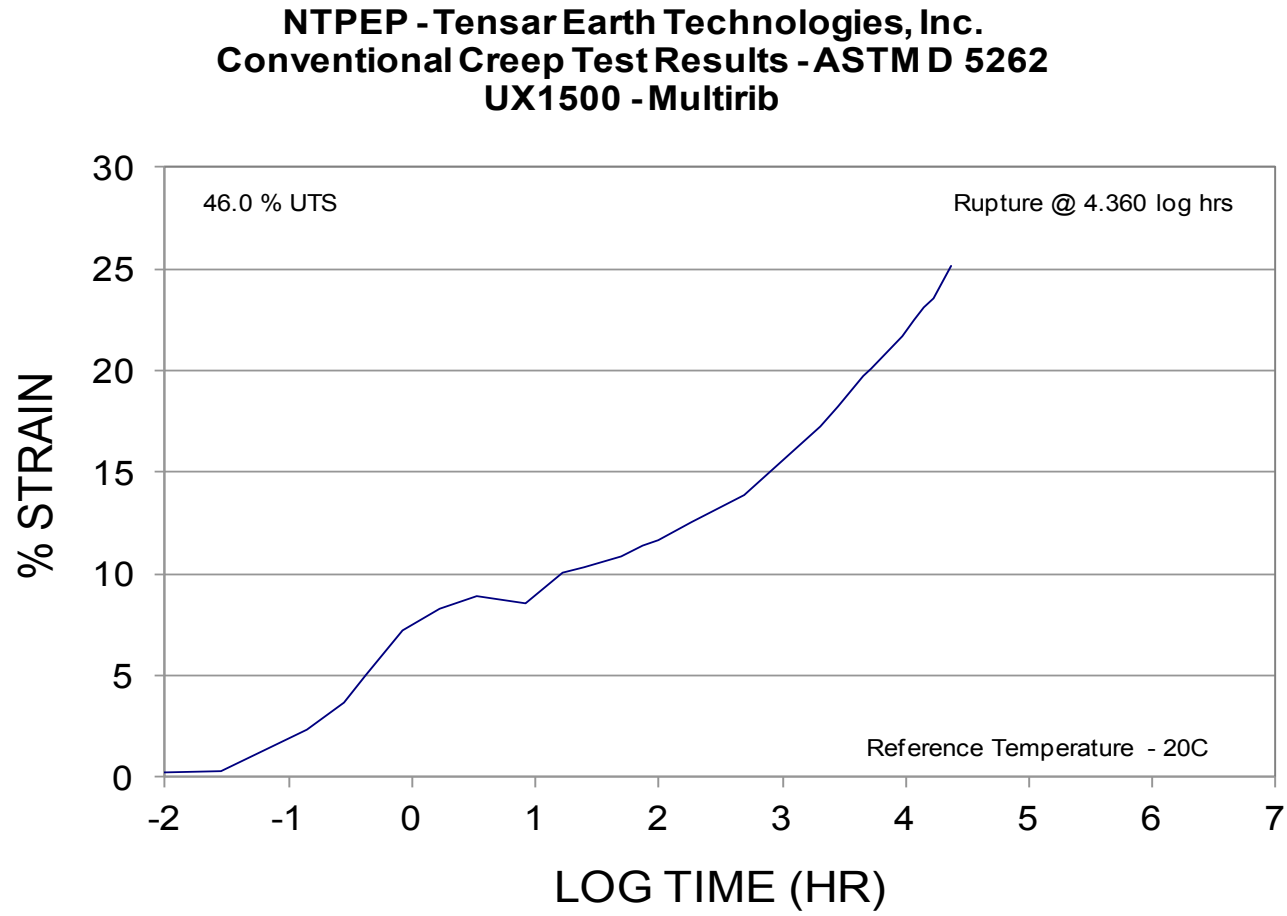
TRI Log # E2280-68-03

Figure F-26. Creep data/curve per ASTM D5262, multirib, for Tensar UX1500 at a load level of 47.5% UTS and 86°F(30°C) shifted to 68°F(20°C)



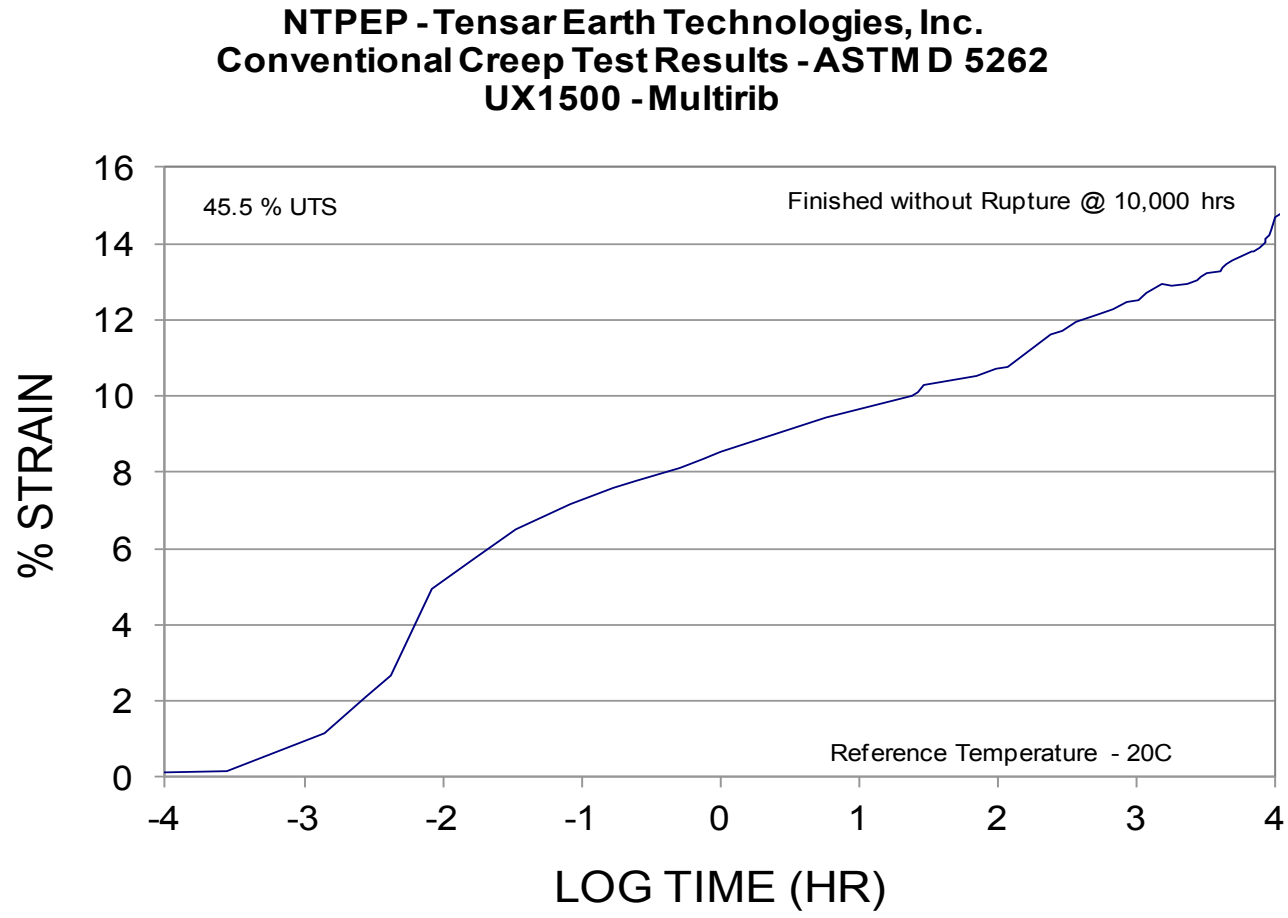
TRI Log # E2280-68-03

Figure F-27. Creep data/curve per ASTM D5262, multirib, for Tensar UX1500 at a load level of 47.0% UTS and 68°F(20°C)



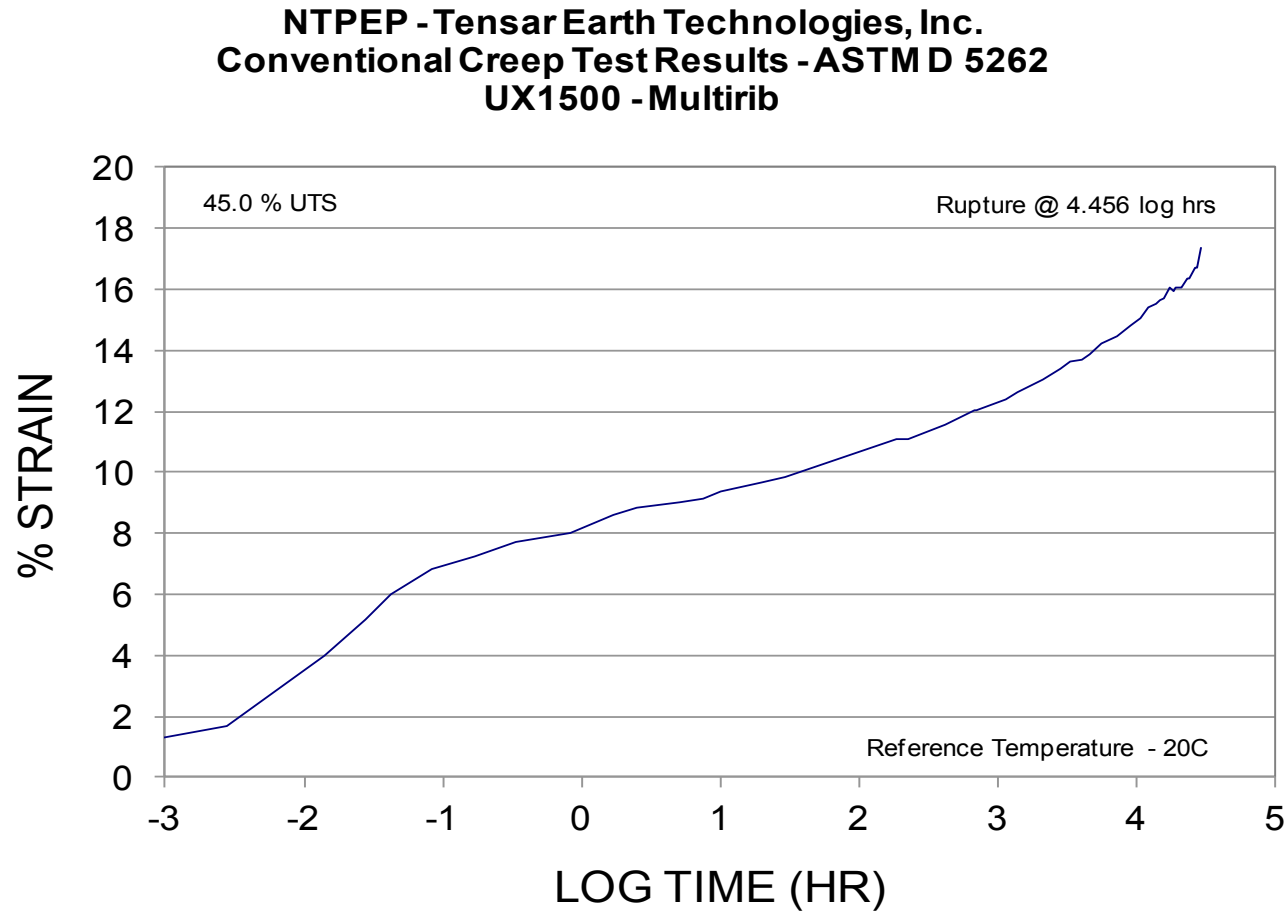
TRI Log # E2280-68-03

Figure F-28. Creep data/curve per ASTM D5262, multirib, for Tensar UX1500 at a load level of 46.0% UTS and 104°F(40°C) shifted to 68°F(20°C)



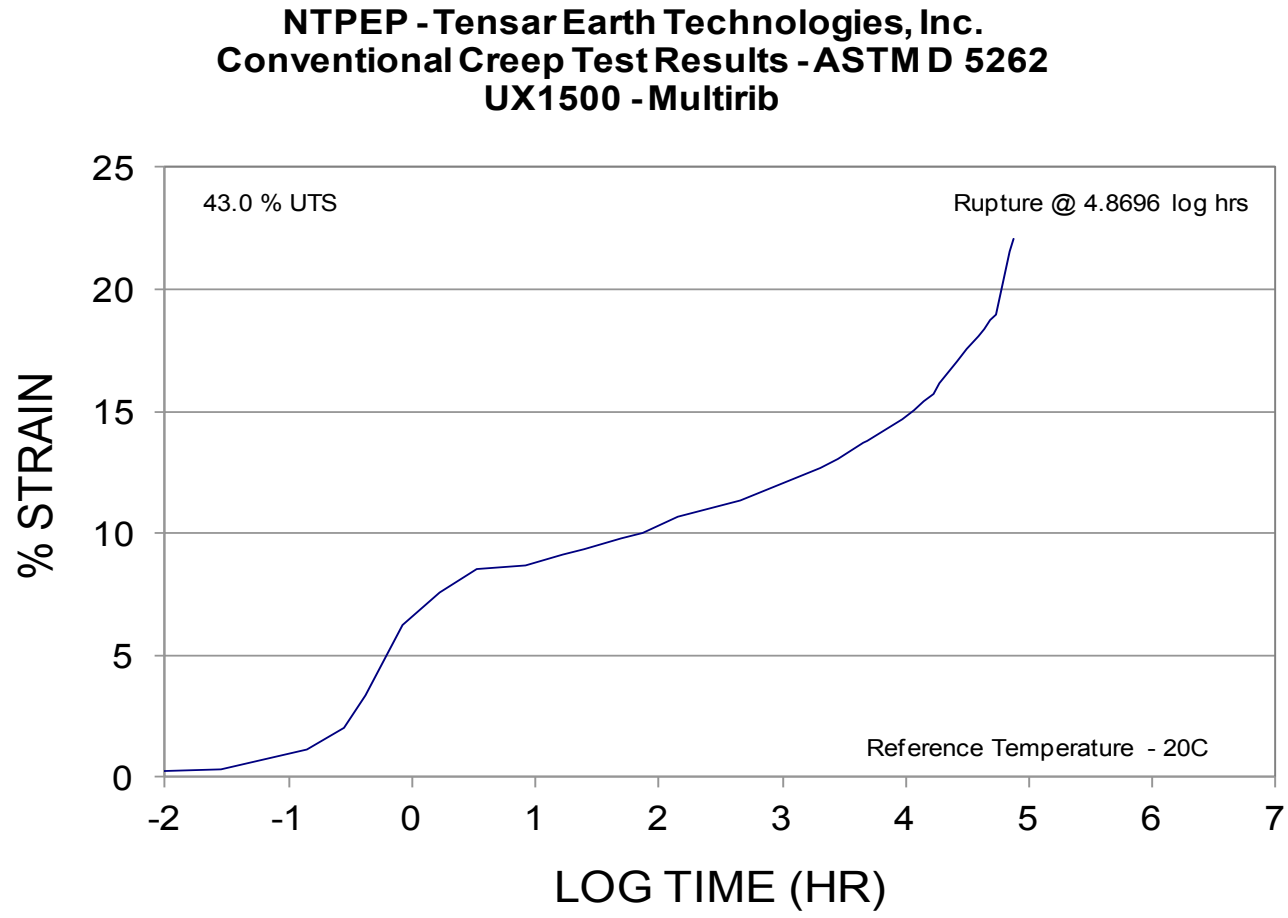
TRI Log # E2280-68-03

Figure F-29. Creep data/curve per ASTM D5262, multirib, for Tensar UX1500 at a load level of 45.5% UTS and 68°F(20°C)



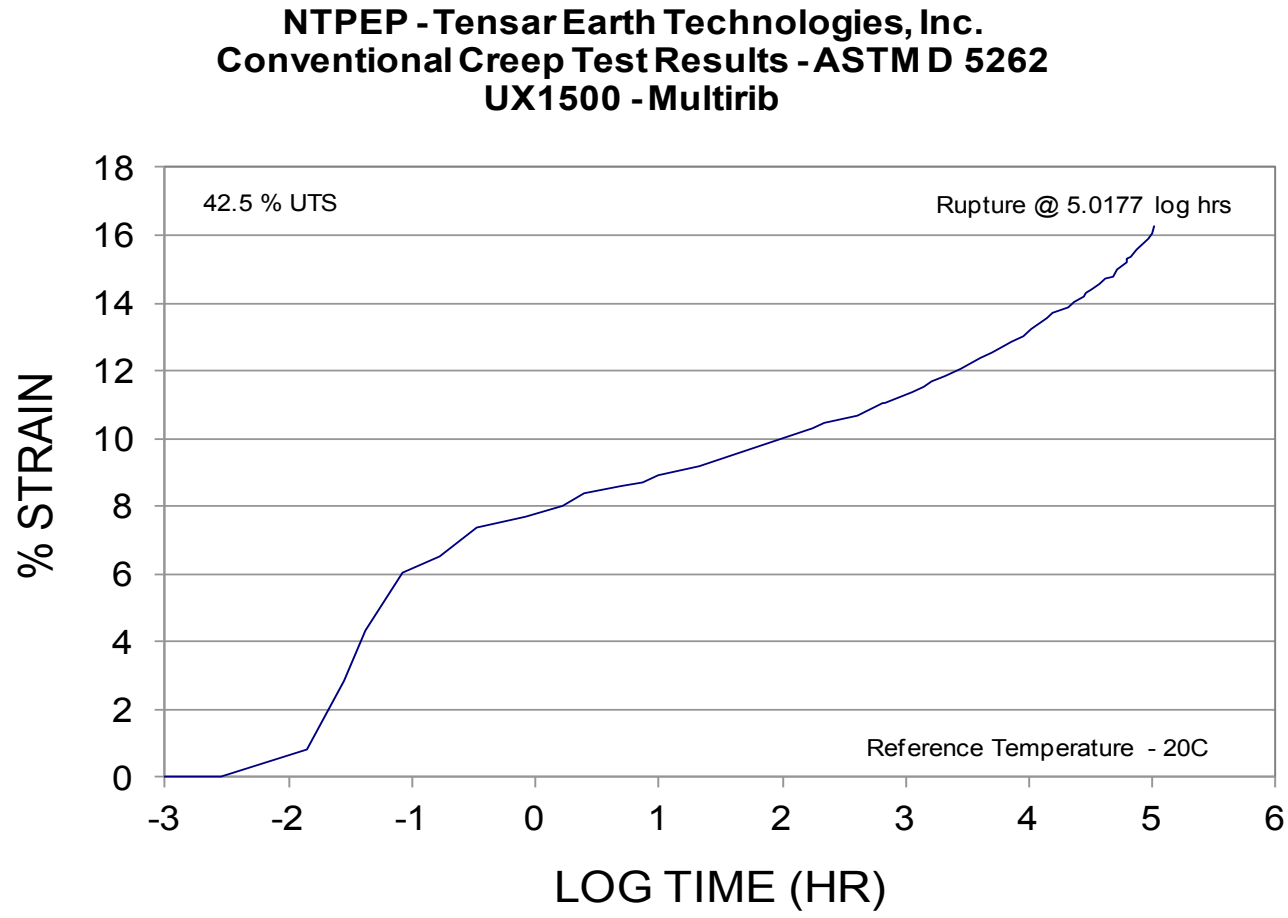
TRI Log # E2280-68-03

Figure F-30. Creep data/curve per ASTM D5262, multirib, for Tensar UX1500 at a load level of 45.0% UTS and 86°F(30°C) shifted to 68°F(20°C)



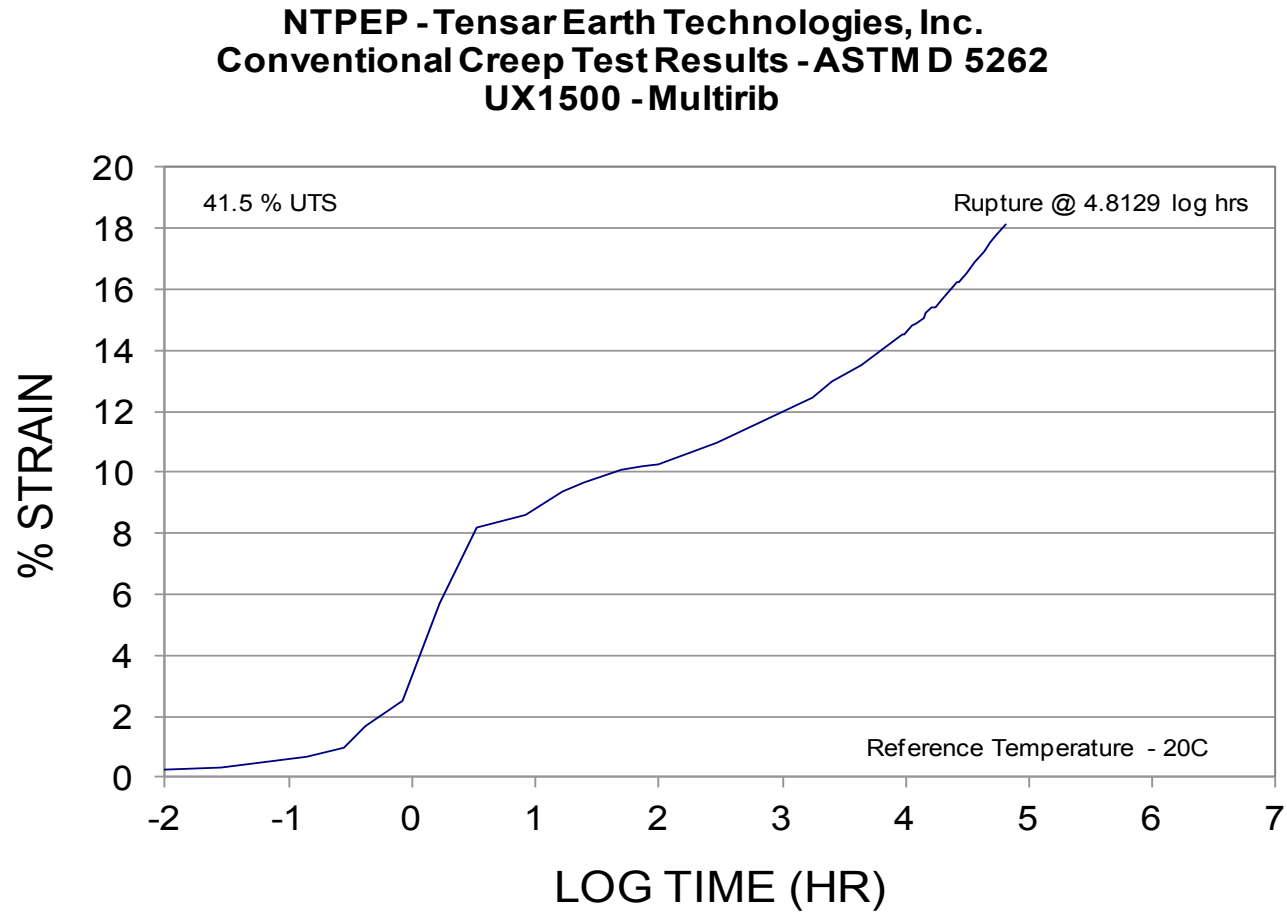
TRI Log # E2280-68-03

Figure F-31. Creep data/curve per ASTM D5262, multirib, for Tensar UX1500 at a load level of 43.0% UTS and 104°F(40°C) shifted to 68°F(20°C)



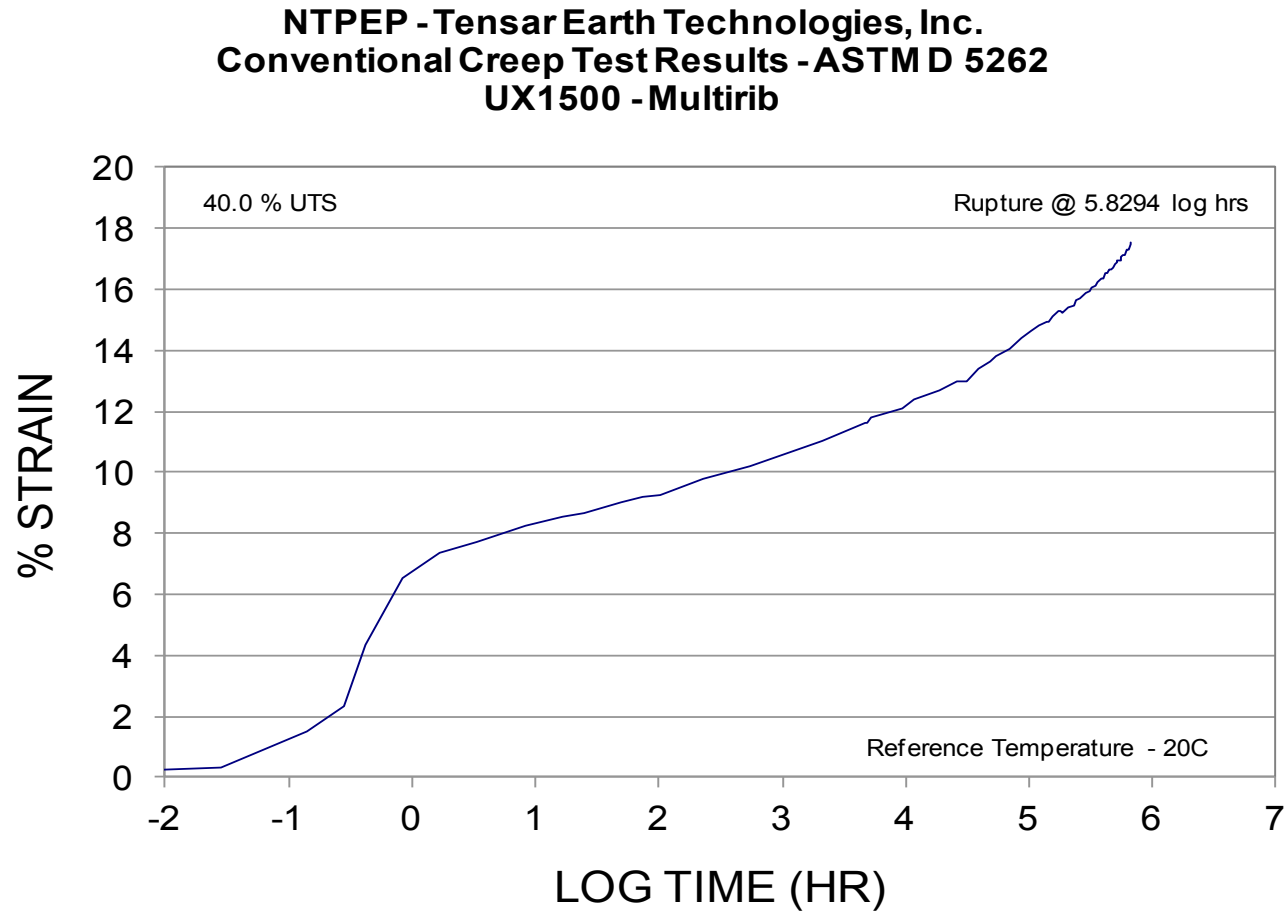
TRI Log # E2280-68-03

Figure F-32. Creep data/curve per ASTM D5262, multirib, for Tensar UX1500 at a load level of 42.5% UTS and 86°F(30°C) shifted to 68°F(20°C)



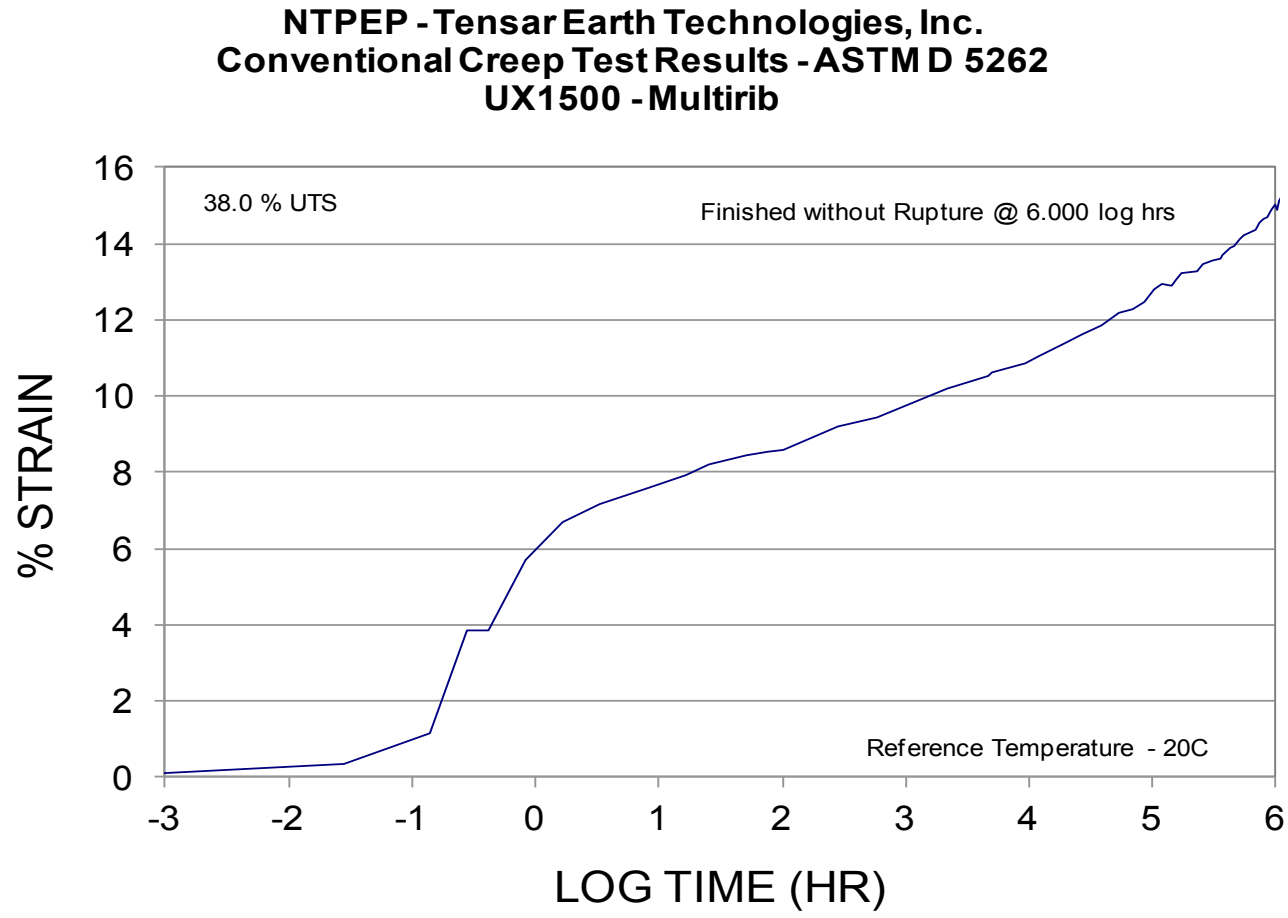
TRI Log # E2280-68-03

Figure F-33. Creep data/curve per ASTM D5262, multirib, for Tensar UX1500 at a load level of 41.5% UTS and 104°F(40°C) shifted to 68°F(20°C)



TRI Log # E2280-68-03

Figure F-34. Creep data/curve per ASTM D5262, multirib, for Tensar UX1500 at a load level of 40.0% UTS and 104°F(40°C) shifted to 68°F(20°C)



TRI Log # E2280-68-03

Figure F-35. Creep data/curve per ASTM D5262, multirib, for Tensar UX1500 at a load level of 38.0% UTS and 104°F(40°C) shifted to 68°F(20°C)

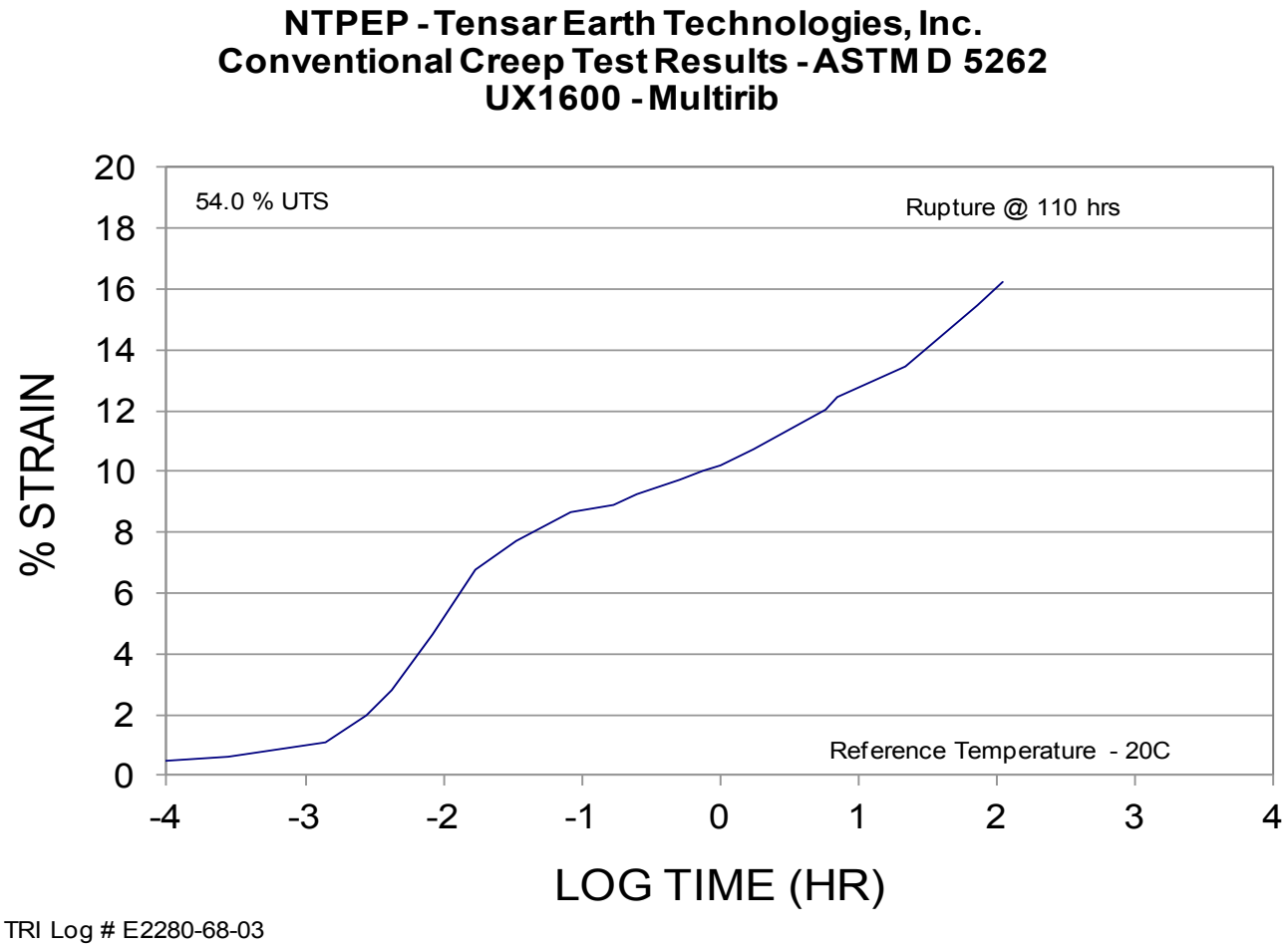
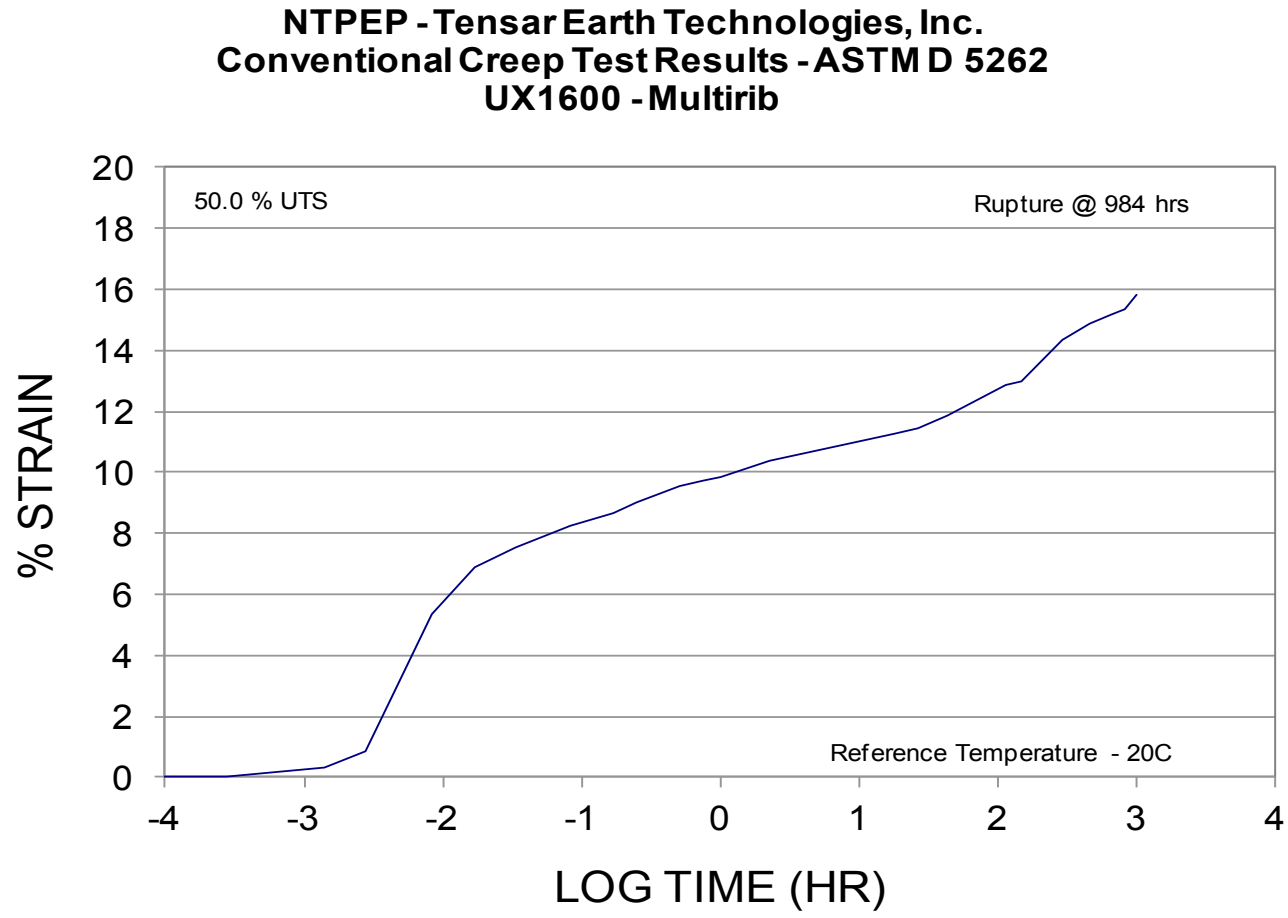
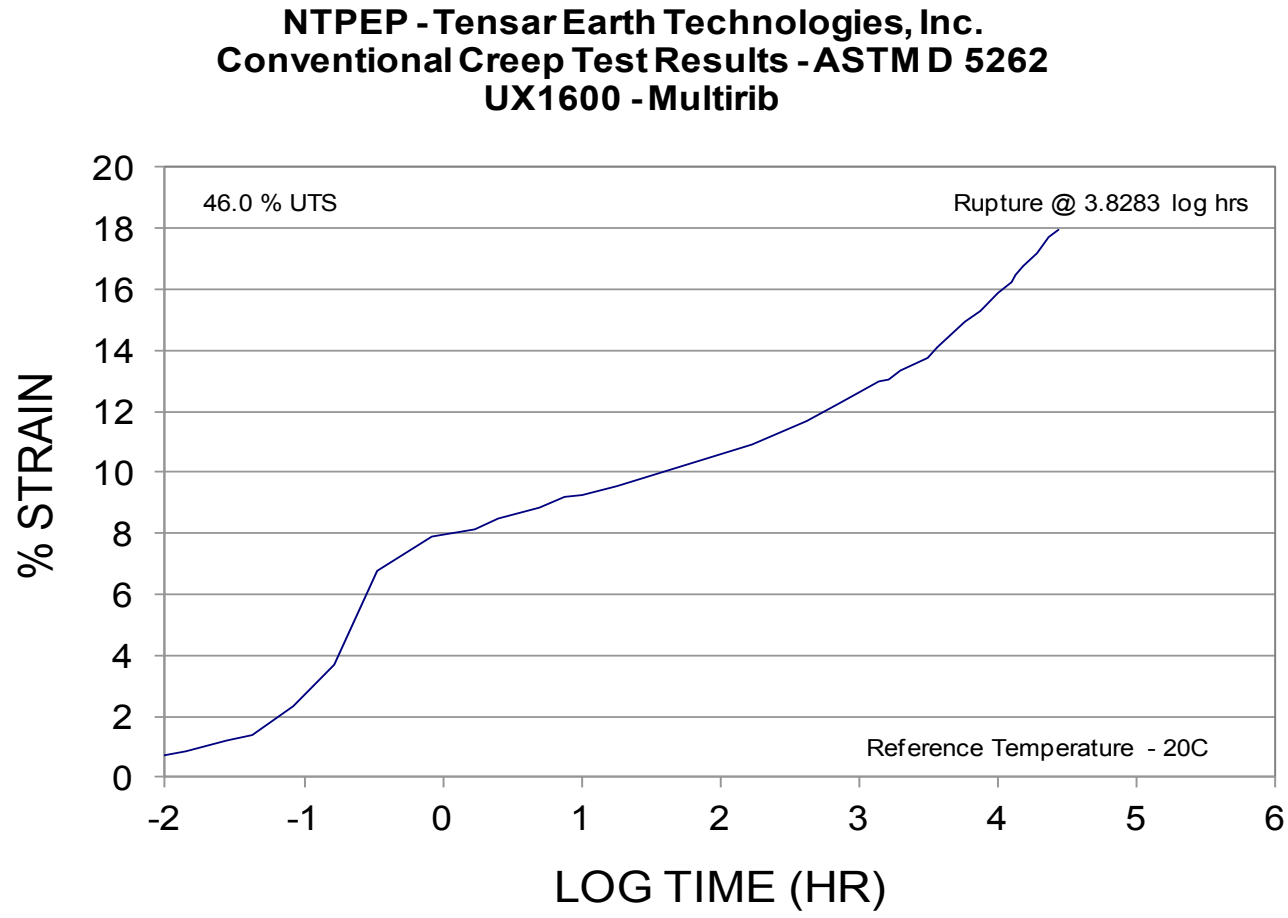


Figure F-36. Creep data/curve per ASTM D5262, multirib, for Tensar UX1600 at a load level of 54.0% UTS and 68°F(20°C)



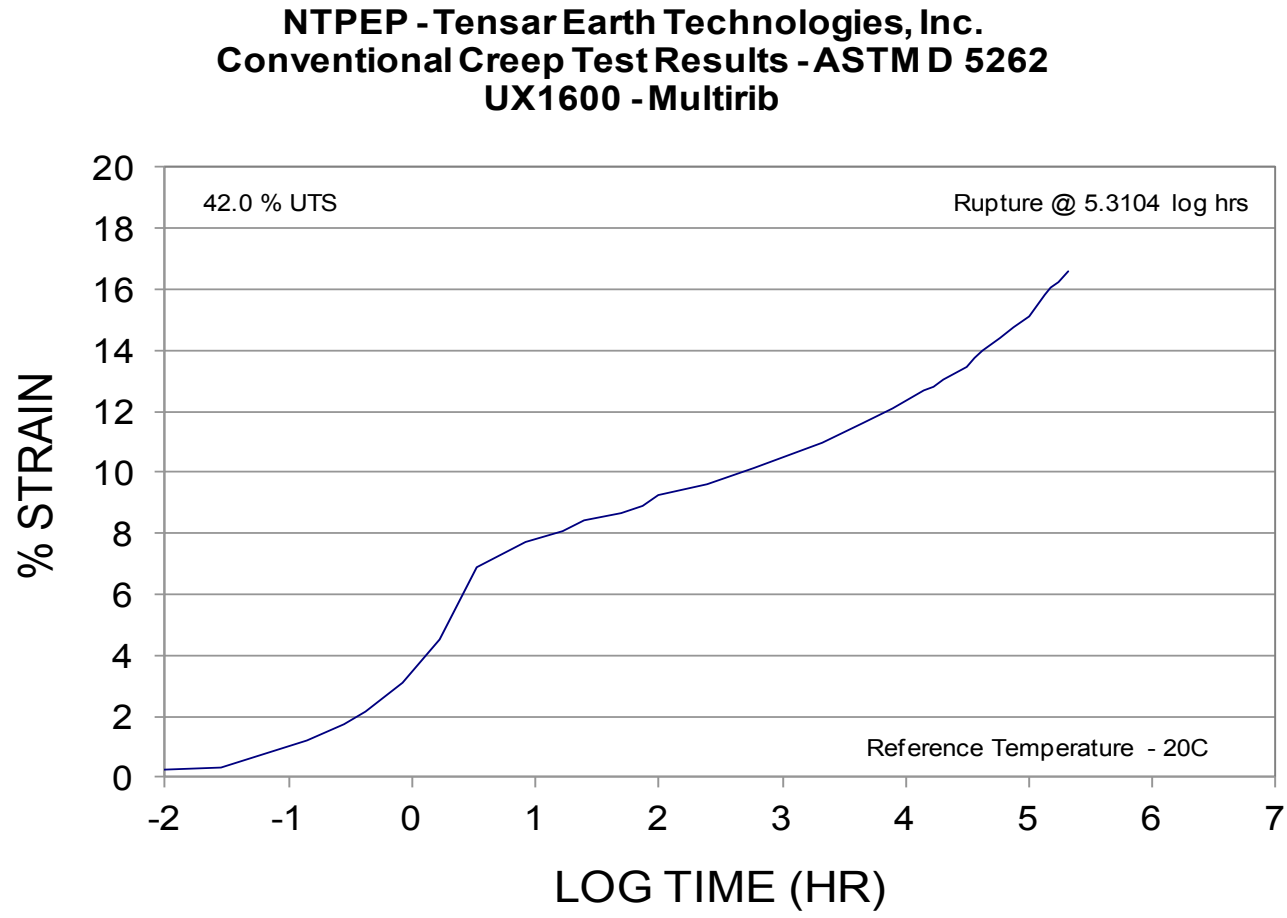
TRI Log # E2280-68-03

Figure F-37. Creep data/curve per ASTM D5262, multirib, for Tensar UX1600 at a load level of 50.0% UTS and 68°F(20°C)



TRI Log # E2280-68-03

Figure F-38. Creep data/curve per ASTM D5262, multirib, for Tensar UX1600 at a load level of 46.0% UTS and 86°F(30°C) shifted to 68°F(20°C)



TRI Log # E2280-68-03

Figure F-39. Creep data/curve per ASTM D5262, multirib, for Tensar UX1600 at a load level of 42.0% UTS and 104°F(40°C) shifted to 68°F(20°C)

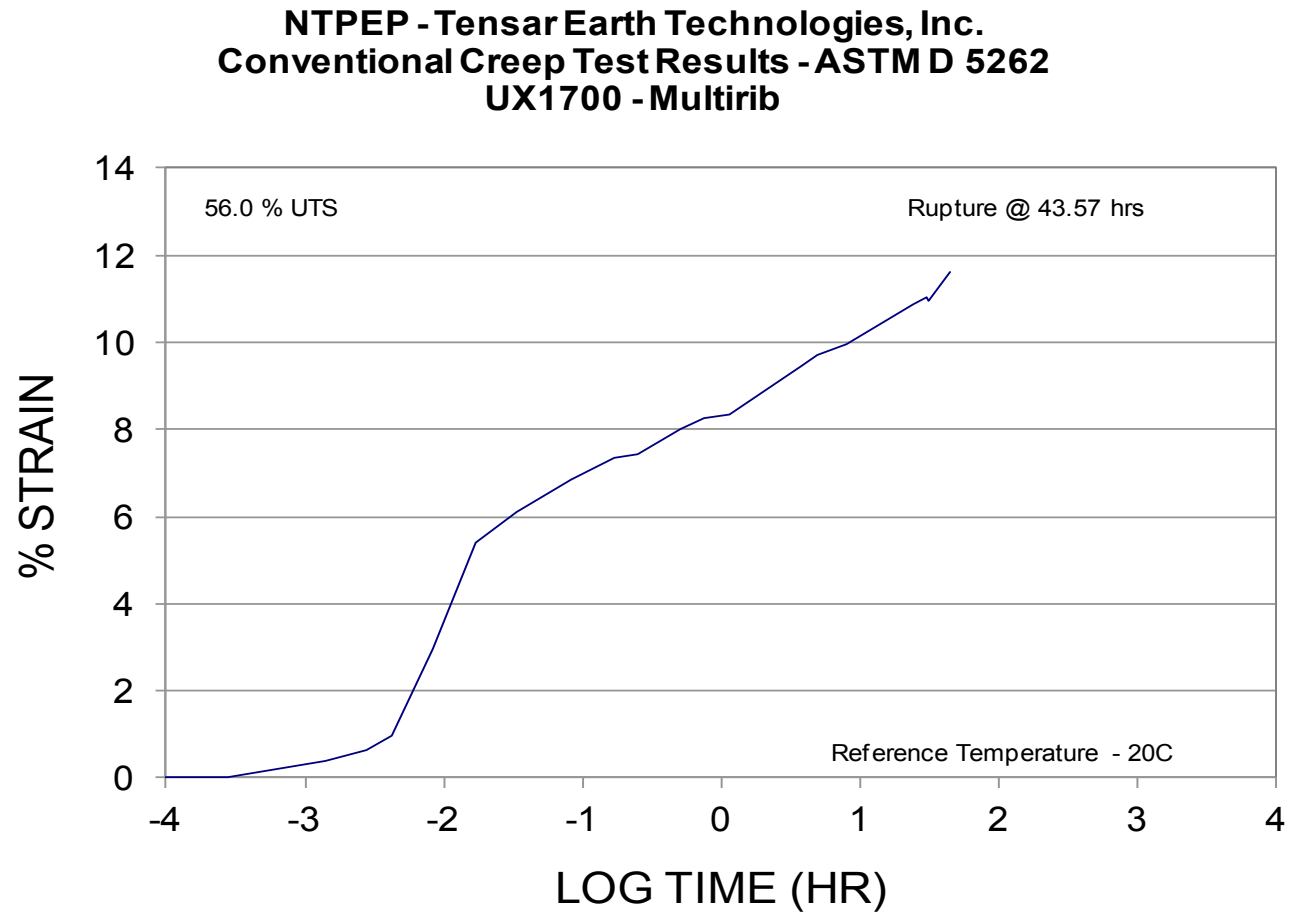
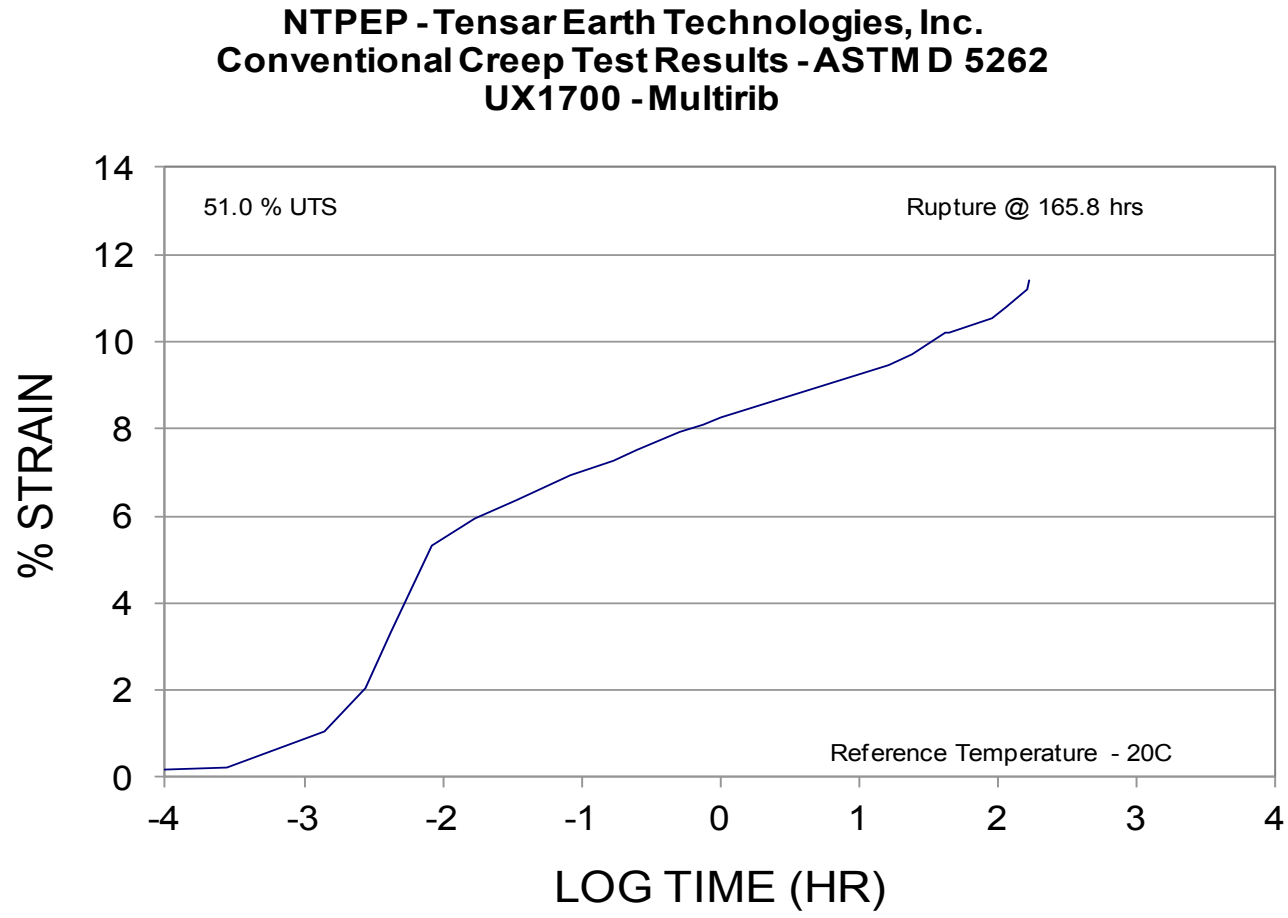
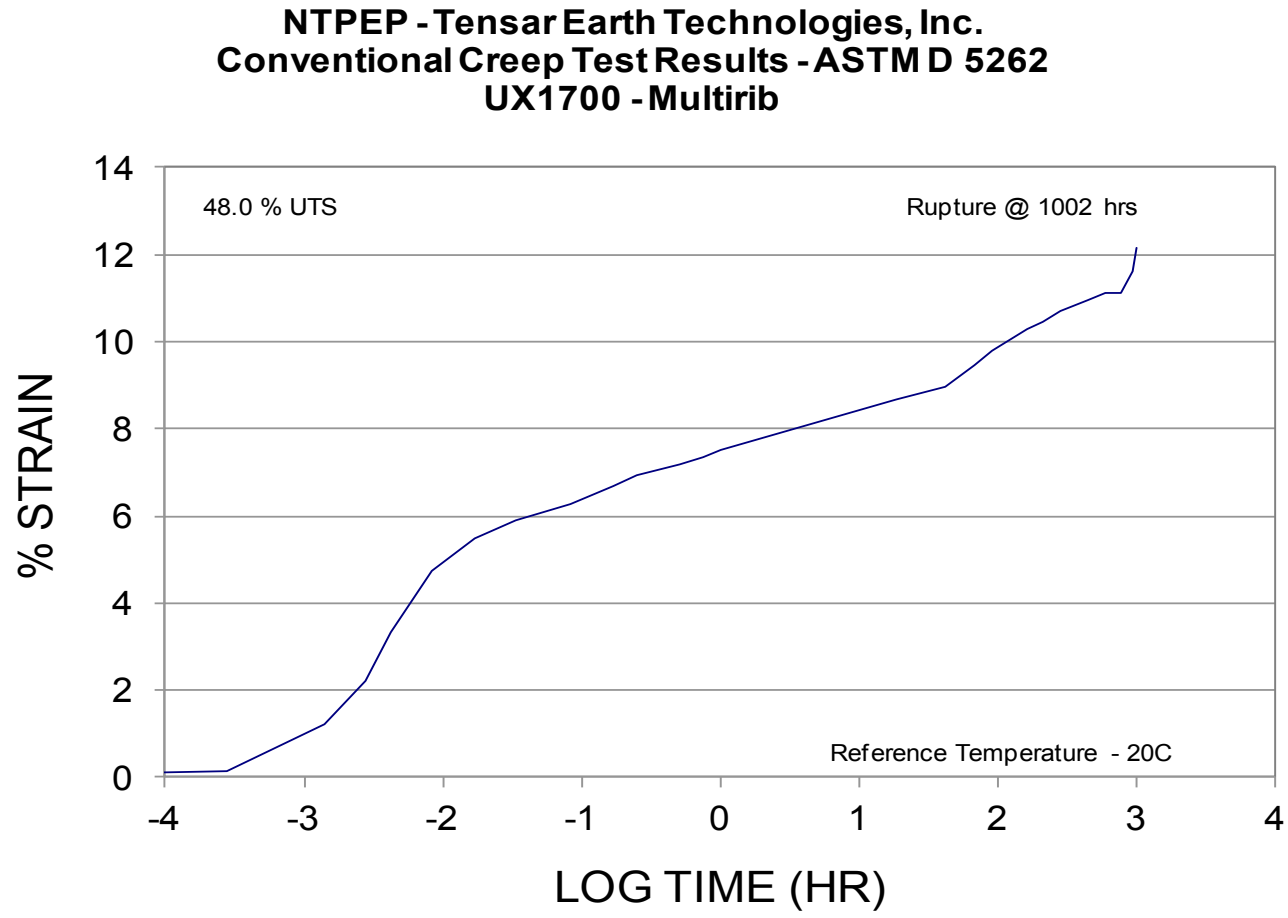


Figure F-40. Creep data/curve per ASTM D5262, multirib, for Tensar UX1700 at a load level of 56.0% UTS and 68°F(20°C)



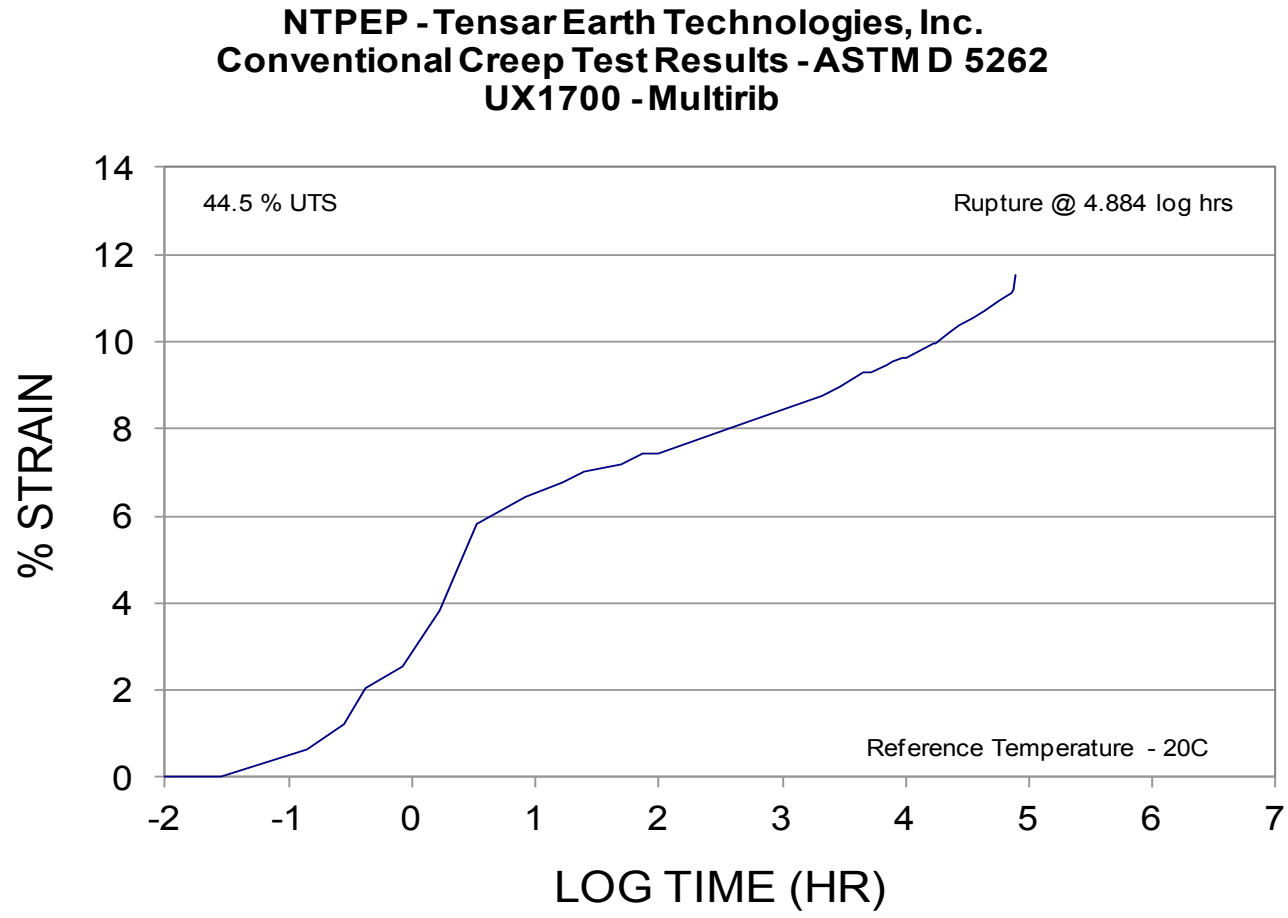
TRI Log # E2280-68-03

Figure F-41. Creep data/curve per ASTM D5262, multirib, for Tensar UX1700 at a load level of 51.0% UTS and 68°F(20°C)



TRI Log # E2280-68-03

Figure F-42. Creep data/curve per ASTM D5262, multirib, for Tensar UX1700 at a load level of 48.0% UTS and 68°F(20°C)



TRI Log # E2280-68-03

Figure F-43. Creep data/curve per ASTM D5262, multirib, for Tensar UX1700 at a load level of 44.5% UTS and 86°F(30°C) shifted to 68°F(20°C)

Tensar UX1100 - UX1400 - Creep Rupture

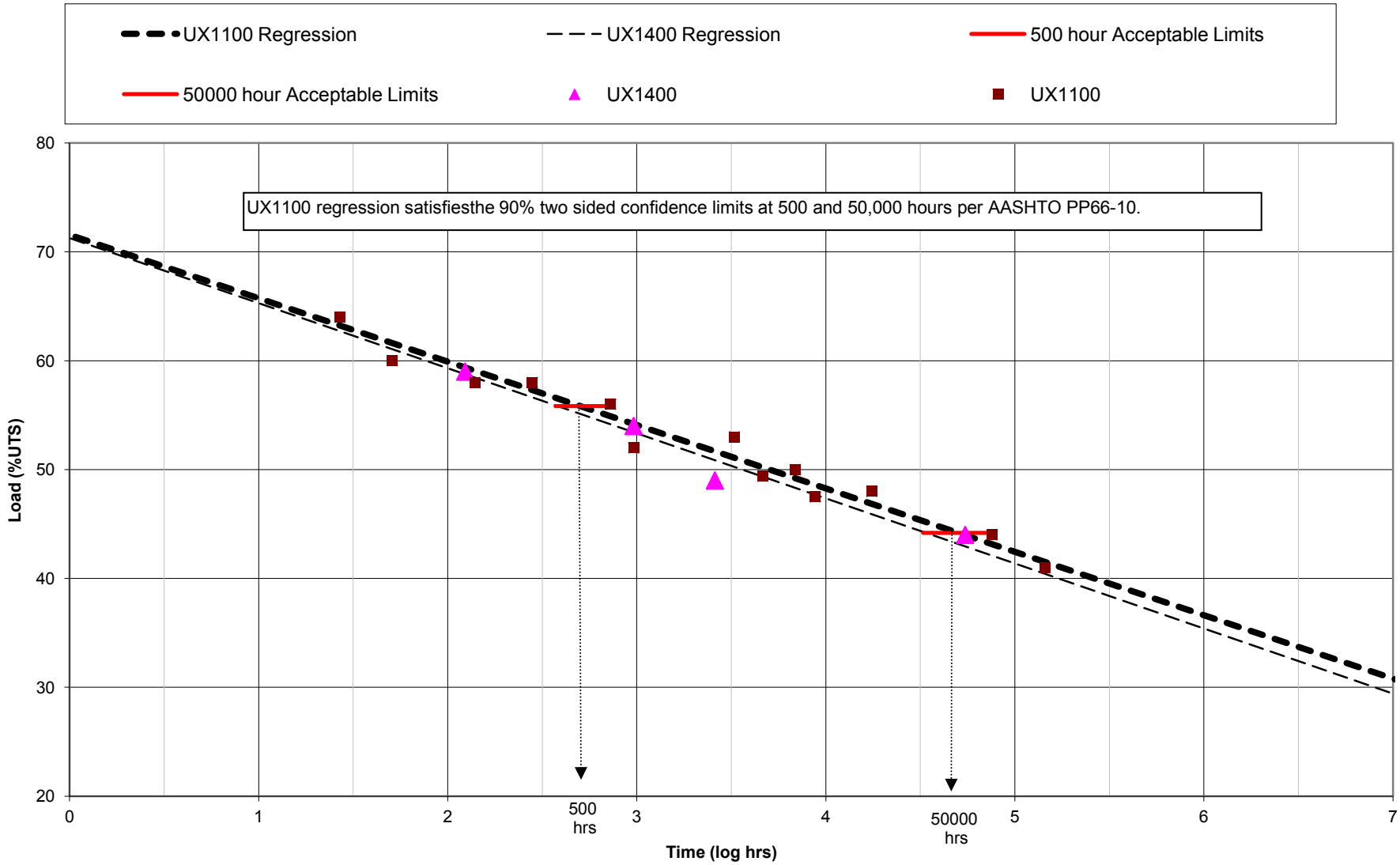


Figure F-44. Statistical evaluation results for determining validity of creating composite creep rupture envelope for the UX-MSE / UX-HS geogrids (UX1100MSE and UX1400MSE comparision) .

Table F-1. Computation table to determine statistical validity of creating composite creep rupture envelope for the UX-MSE / UX-HS geogrid product line - UX1100MSE and UX1400MSE comparision.

Tensar Creep Data Evaluation

Tests on UX1400

Test Report	Product	Expected time, log hrs	%UTS	Rupture	
	UX1400	2.0924	59	59	20C
	UX1400	2.9834	54	54	20C
	UX1400	3.4130	49	49	30C
	UX1400	4.7373	44	44	40C

Conventional Tests on UX1100

Test Report	Product	time, log hrs	%UTS	Rupture	Runout	logti-logtbar	(logti-logtbar)2	Pi - Pbar	(Pi-Pbar)2	K * L	
	UX1100	1.4300	64	64		-1.86	3.4747	11.62	135.0959	-21.6661	20C
	UX1100	1.7064	60	60		-1.59	2.5207	7.62	58.1113	-12.1029	20C
	UX1100	2.1446	58	58		-1.15	1.3213	5.62	31.61899	-6.46351	20C
	UX1100	2.8615	56	56		-0.43	0.1871	3.62	13.12669	-1.5672	20C
	UX1100	3.5165	53	53		0.22	0.0495	0.62	0.388225	0.138596	20C
	UX1100	3.8386	50	50		0.54	0.2965	-2.38	5.649763	-1.29433	20C
	UX1100	3.9447	47.5	47.5		0.65	0.4233	-4.88	23.78438	-3.17311	20C
	UX1100										
	UX1100	2.4477	58	58		-0.85	0.7163	5.62	31.61899	-4.75916	30C
	UX1100	4.2433	48	48		0.95	0.9011	-4.38	19.15746	-4.15474	30C
	UX1100	4.8797	44	44		1.59	2.5142	-8.38	70.17284	-13.2828	30C
	UX1100										
	UX1100	2.984	52	52		-0.31	0.0961	-0.38	0.142071	0.116869	40C
	UX1100	3.6668	49.4	49.4		0.37	0.1389	-2.98	8.862071	-1.10961	40C
	UX1100	5.159	41	41		1.86	3.4780	-11.38	129.4344	-21.2173	40C
Sum		42.8228	680.90		Sum	0.00	16.1178	0.00	527.1631	-90.5352	
Mean		3.2941	52.38								

student's t = 1.796 (90% 2-sided prediction limit)
n-UX1400 = 4
n-UX1100 = 13 d-o-f 11
treg = 2.6990
treg = 4.6990
P500 = 55.8418
P50000 = 44.1965
sigma squared = 0.0517
sigma = 0.2275

90% 2-sided conf. limit	
df	student's t
2	2.92
3	2.353
4	2.132
5	2.015
6	1.943
7	1.895
8	1.86
9	1.833
10	1.812
11	1.796
12	1.782
13	1.771
14	1.761

* runout plotting below the regression line is not included in the regression

UX1100 - 500 hrs (log 2.699)			**UX1100** - 50000 hrs (log 4.699)		
log tL - lower =	2.57	55.8418	log tL - lower =	4.51	44.1965
log tL - upper =	2.83	55.8418	log tL - upper =	4.88	44.1965

UX1400 - logtL @ Load = 2.58 **OK**

UX1400 - logtL @ Load = 4.53 **OK**

UX1400 - All Points
time is dependent variable:

	if time were the y axis	but time is the x axis	
slope	-0.16729	-5.97779	
intercept	11.92175	71.2657	
R squared	0.961812	0.961812	
	-2	83.22127	
	10	11.48784	
2.6990	55.13165	= 500 hr intercept	
4.6990	43.17626	= 50000 hr intercept	

UX1100 - All Points
time is dependent variable:

	if time were the y axis	but time is the x axis	
slope	-0.17174	-5.82274	
intercept	12.2893	71.55739	
R squared	0.964683	0.964683	
	-2	83.20286	
	10	13.32999	
2.6990	55.84181	= 500 hr intercept	
4.6990	44.19651	= 50000 hr intercept	

All Creep Data UX1100 & UX1400
time is dependent variable:

	if time were the y axis	but time is the x axis	
slope	-0.17032	-5.87118	
intercept	12.18288	71.52782	
R squared	0.961142	0.961142	
	-2	83.27017	
	10	12.81606	
5.8176	37.37167	= 75-yr intercept	
5.9425	36.63836	= 100-yr intercept	

The regression for the all creep tests on the primary product (UX1100MSE) produced log 2.6990 hr (500 hrs) and log 4.6990 hr (50,000 hrs) intercepts at 55.84% and 44.20% UTS, respectively. The regression for the creep tests on UX1400MSE produced log time intercepts for the same %UTS within the 90% confidence limits of log 2.57 to log 2.83 and log 4.51 to log 4.88 associated with those %UTS. This evaluation is summarized in Table F-2. Thus, the primary, UX1100MSE, and secondary product, UX1400MSE, data may be used together to construct the characteristic creep rupture curve of the family of products. Confidence limits satisfied per AASHTO PP66-10.

Table F-2. Summary of statistical comparison between rupture envelopes for UX1100MSE and UX1400MSE geogrid products, to test validity of composite creep rupture envelope for product line.

Product	Intercept at log 2.6690 & 4.6990 hrs, %UTS	Intercept at same % UTS, log hrs	90% Confidence Limits @ Higher %UTS, log hrs	90% Confidence Limits @ Lower %UTS, log hrs
UX1100MSE	55.84 & 44.20	2.6690 & 4.6990	-	-
UX1400MSE	-	2.58 & 4.53	2.57 to 2.83	4.51 to 4.88

TENSAR UX1100 - UX1400 - COMPOSITE CREEP RUPTURE CURVE

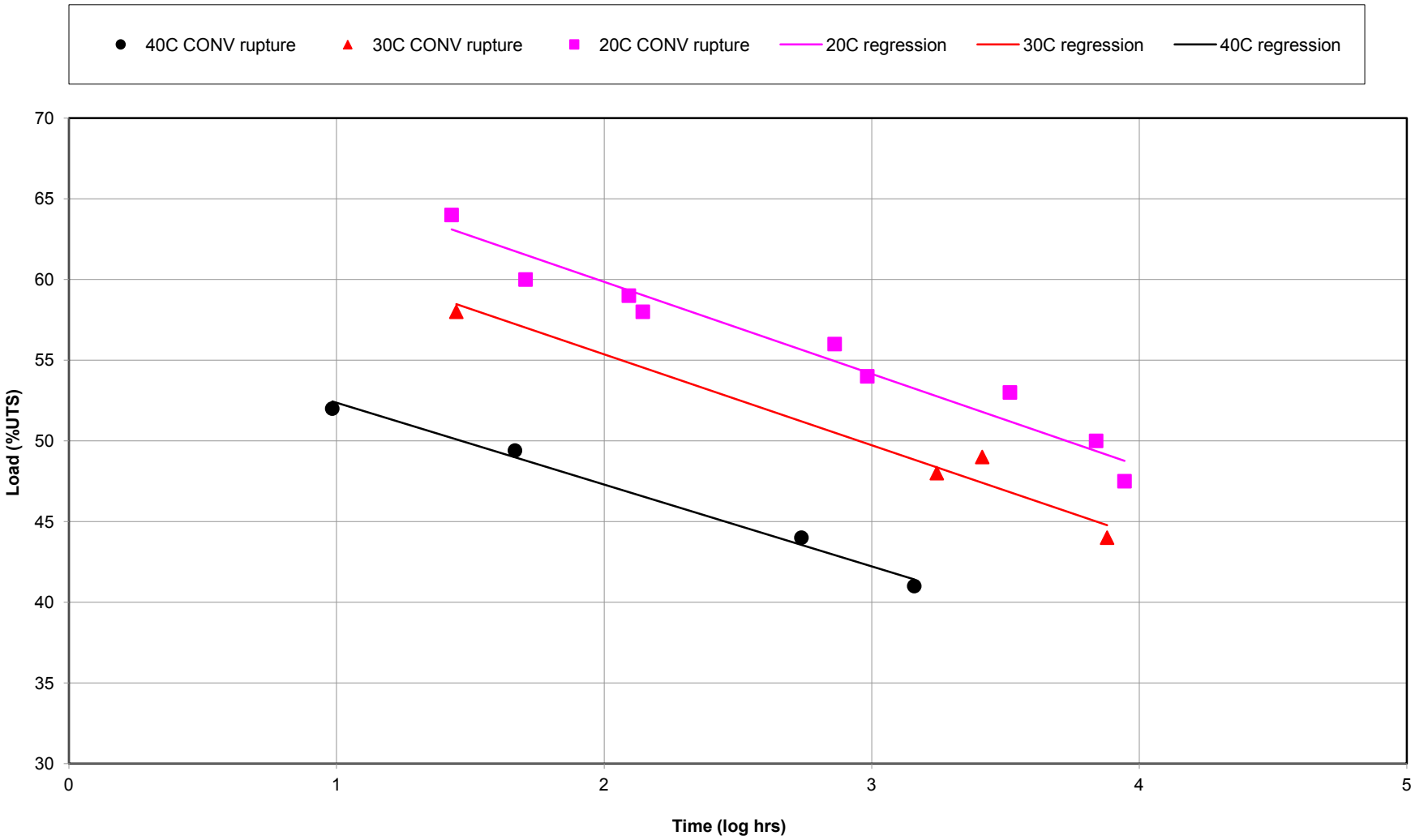


Figure F-45. Actual time to rupture data and regressions for verification of the best fit shift factors of 'x10' for 30°C and 'x100' for 40°C for UX1100 & UX1400.

TENSAR UX1100 - UX1400 - COMPOSITE CREEP RUPTURE CURVE

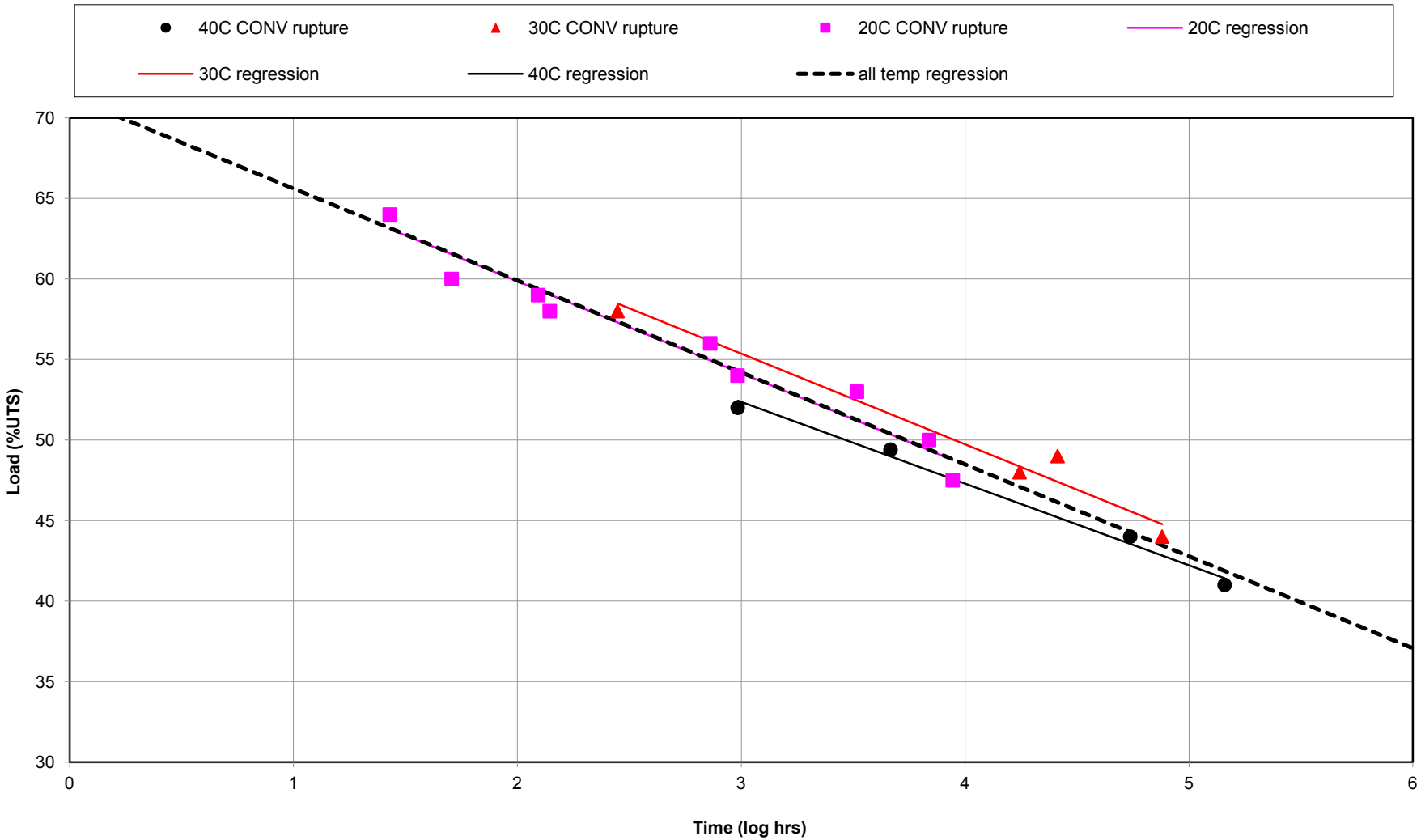


Figure F-46. Time shifted data and regressions for verification of the best fit shift factors of 'x10' for 30°C and 'x100' for 40°C for UX1100 & UX1400.

Table F-3. Computation table for composite creep rupture envelope for the Tensar UX1100 - UX1400 geogrid product line (in support of Figure 5-1).

Stress, % of UTS

	product:	data for regression calculation loghrs	all	UX1400		UX1100	UX	conv'l rupture	conv'l runout*	
	UX1400	2.0924	59	59				59		20C
	UX1400	2.9834	54	54				54		20C
	UX1400	3.4130	49	49				49		30C
	UX1400	4.7373	44	44				44		40C
	UX1100	1.4300	64			64		64		20C
	UX1100	1.7064	60			60		60		20C
CONV	UX1100	2.1446	58			58		58		20C
DATA:	UX1100	2.8615	56			56		56		20C
	UX1100	3.5165	53			53		53		20C
	UX1100	3.8386	50			50		50		20C
	UX1100	3.9447	47.5			47.5		47.5		20C
	UX1100	2.4477	58			58		58		30C
	UX1100	4.2433	48			48		48		30C
	UX1100	4.8797	44			44		44		30C
	UX1100	2.984	52			52		52		40C
	UX1100	3.6668	49.4			49.4		49.4		40C
	UX1100	5.159	41			41		41		40C
	UX1100									

NOTE: Don't include runouts in the regression calculation unless the points lie above the line

Conventional - UX1400
time is dependent variable:

	if time were the y axis	but time is the x axis	
slope	-0.16729	-5.978	
intercept	11.92175	71.27	
R squared	0.961812	0.962	
	-2	83.22	
	10	11.49	
6	35.39898	= 114 Year intercept	
5.817863	36.48776	= 75 Year intercept	

Conventional - UX1100
time is dependent variable:

	if time were the y axis	but time is the x axis	
slope	-0.17174	-5.823	
intercept	12.2893	71.56	
R squared	0.964683	0.965	
	-2	83.2	
	10	13.33	
5.999706	36.62266	= 114 Year intercept	
5.817863	37.68149	= 75 Year intercept	

Conventional - All
time is dependent variable:

	if time were the y axis	but time is the x axis	
slope	-0.1703236	-5.8711764	
intercept	12.182877	71.527822	
R squared	0.9611416	0.9611416	
	-2	83.270175	
	10	12.816059	
5.9997063	36.302488	= 114 Year intercept	
5.8178627	37.370124	= 75 Year intercept	

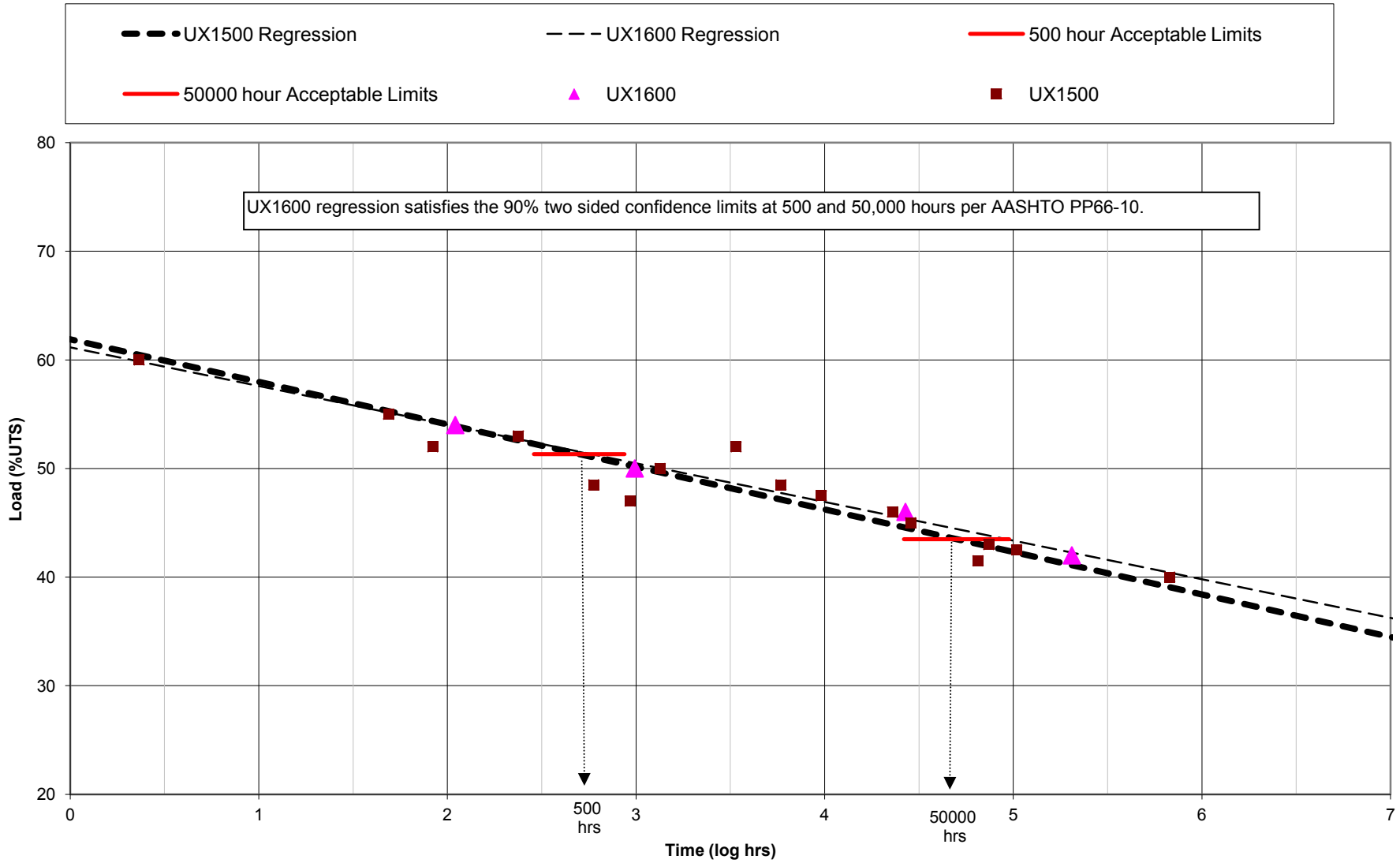


Figure F-47. Statistical evaluation results for determining validity of creating composite creep rupture envelope for the UX-MSE / UX-HS geogrids (UX1500MSE and UX1600MSE comparision) .

Table F-4. Computation table to determine statistical validity of creating composite creep rupture envelope for the UX-MSE / UX-HS geogrid product line - UX1500MSE and UX1600MSE comparision.

Tensar Creep Data Evaluation

Tests on UX1600

Test Report	Product	Expected time, log hrs	%UTS	Rupture	
	UX1600	2.0416	54	54	20C
	UX1600	2.9930	50	50	20C
	UX1600	4.4283	46	46	30C
	UX1600	5.3104	42	42	40C

Conventional Tests on UX1500

Test Report	Product	time, log hrs	%UTS	Rupture	Runout	logti-logtbar	(logti-logtbar)2	Pi - Pbar	(Pi-Pbar)2	K * L	
	UX1500	0.3655	60	60		-3.13	9.7662	11.78	138.7979	-36.8175	20C
	UX1500	1.6875	55	55		-1.80	3.2511	6.78	45.98535	-12.2272	20C
	UX1500	2.3762	53	53		-1.11	1.2419	4.78	22.86035	-5.3282	20C
	UX1500	3.5298	52	52		0.04	0.0015	3.78	14.29785	0.148249	20C
	UX1500	3.1278	50	50		-0.36	0.1316	1.78	3.172852	-0.64623	20C
	UX1500	2.7758	48.5	48.5		-0.71	0.5109	0.28	0.079102	-0.20104	20C
	UX1500	2.9689	47	47		-0.52	0.2722	-1.22	1.485352	0.635814	20C
	UX1500	3.7678	48.5	48.5		0.28	0.0768	0.28	0.079102	0.077964	20C
	UX1500	3.9820	47.5	47.5		0.49	0.2415	-0.72	0.516602	-0.3532	30C
	UX1500	4.4560	45	45		0.97	0.9320	-3.22	10.36035	-3.1074	30C
	UX1500	1.9226	52	52		-1.57	2.4586	3.78	14.29785	-5.92898	30C
	UX1500	5.0177	42.5	42.5		1.53	2.3321	-5.72	32.7041	-8.73314	30C
	UX1500	4.3600	46	46		0.87	0.7559	-2.22	4.922852	-1.929	40C
	UX1500	4.8696	43	43		1.38	1.9017	-5.22	27.23535	-7.19669	40C
	UX1500	4.8129	41.5	41.5		1.32	1.7485	-6.72	45.1416	-8.88425	40C
	UX1500	5.8294	40.0	40.0		2.34	5.4700	-8.22	67.54785	-19.2221	40C
	UX1500	4.0000			45.5						20C
	UX1500	6.0000			38.0						40C
	Sum	55.8495	771.50		Sum	0.00	31.0925	0.00	429.4844	-109.713	
	Mean	3.4906	48.22								

student's t = 1.761 (90% 2-sided prediction limit)
n-UX1700 = 5
n-UX1500 = 16 d-o-f 14
treg = 2.6690
treg = 4.6990
P500 = 51.3175
P50000 = 43.4884
sigma squared = 0.2190
sigma = 0.4680

90% 2-sided conf. limit	
df	student's t
2	2.92
3	2.353
4	2.132
5	2.015
6	1.943
7	1.895
8	1.86
9	1.833
10	1.812
11	1.796
12	1.782
13	1.771
14	1.761

* runout plotting below the regression line is not included in the regression

UX1500 - 500 hrs (log 2.699)			**UX1500** - 50000 hrs (log 4.699)		
log tL - lower =	2.46	51.3175	log tL - lower =	4.42	43.4884
log tL - upper =	2.94	51.3175	log tL - upper =	4.98	43.4884

UX1600 - logtL @ Load = 2.76 **OK** **UX1600 - logtL @ Load =** 4.96 **OK**

UX1600 - All Points
time is dependent variable:

	if time were the y axis	but time is the x axis	
slope	-0.28104	-3.55818	
intercept	17.18337	61.14152	
R squared	0.991374	0.991374	
	-2	68.25788	
	10	25.55971	
2.6990	51.53799	= 500 hr intercept	
4.6990	44.42173	= 50000 hr intercept	

UX1500 - All Points
time is dependent variable:

	if time were the y axis	but time is the x axis	
slope	-0.25545	-3.91462	
intercept	15.8082	61.8831	
R squared	0.901389	0.901389	
	-2	69.71235	
	10	22.73689	
2.6990	51.31754	= 500 hr intercept	
4.6990	43.48841	= 50000 hr intercept	

All Creep Data UX1500 & UX1600
time is dependent variable:

	if time were the y axis	but time is the x axis	
slope	-0.25967	-3.85102	
intercept	16.0408	61.7735	
R squared	0.914001	0.914001	
	-2	69.47555	
	10	23.26327	
5.8176	39.36979	= 75-yr intercept	
5.9425	38.8888	= 100-yr intercept	

Tensar UX1500 - UX1700 - Creep Rupture

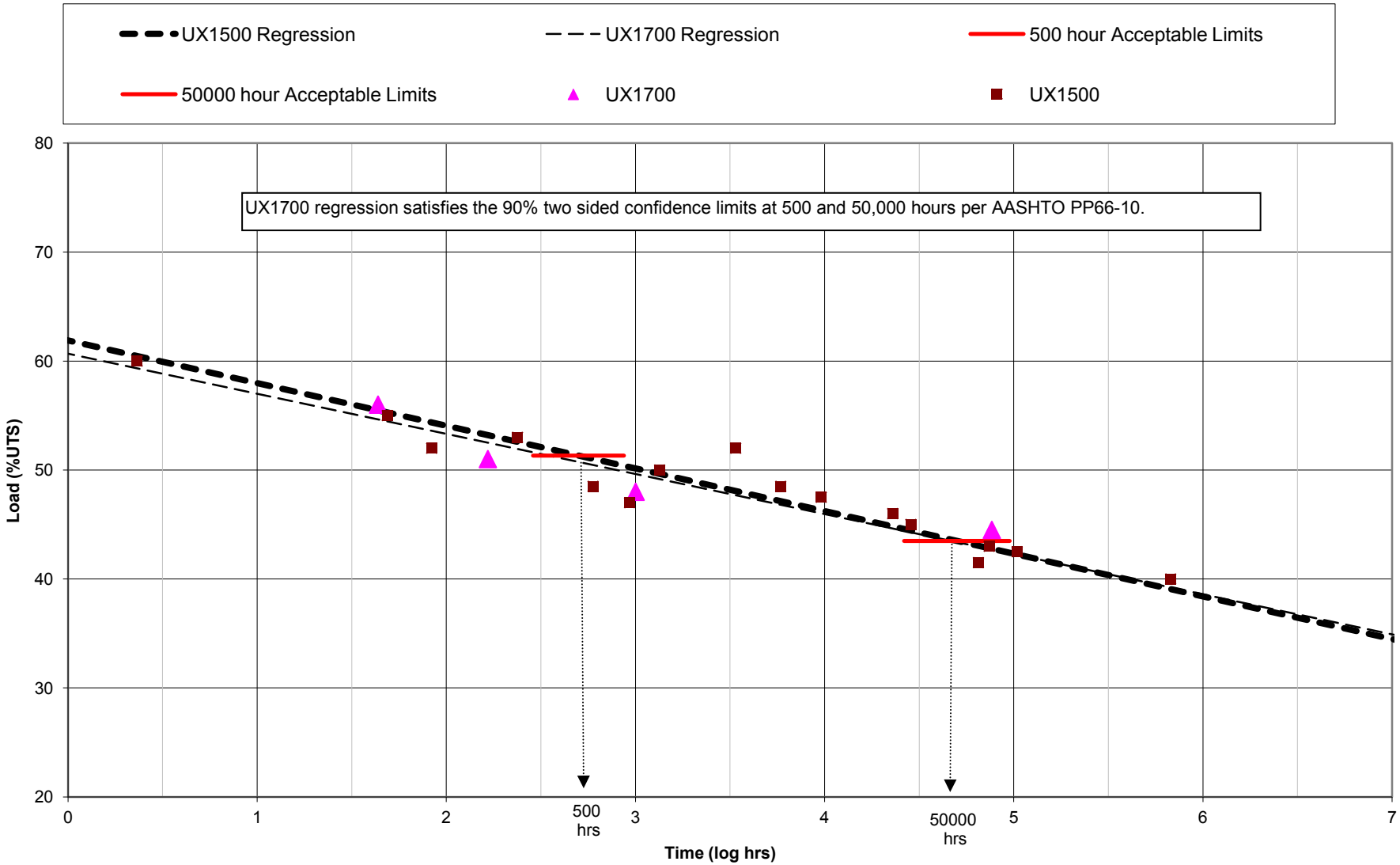


Figure F-48. Statistical evaluation results for determining validity of creating composite creep rupture envelope for the UX-MSE / UX-HS geogrids (UX1500MSE and UX1700MSE comparision) .

Table F-5. Computation table to determine statistical validity of creating composite creep rupture envelope for the UX-MSE / UX-HS geogrid product line - UX1500MSE and UX1700MSE comparision.

Tensar Creep Data Evaluation

Tests on UX1700

Test Report	Product	Expected time, log hrs	%UTS	Rupture	
	UX1700	1.6390	56	56	20C
	UX1700	2.2196	51	51	20C
	UX1700	3.0009	48	48	20C
	UX1700	4.8840	44.5	44.5	30C

Conventional Tests on UX1500

Test Report	Product	time, log hrs	%UTS	Rupture	Runout	logti-logtbar	(logti-logtbar)2	Pi - Pbar	(Pi-Pbar)2	K * L	
	UX1500	0.3655	60	60		-3.13	9.7662	11.78	138.7979	-36.8175	20C
	UX1500	1.6875	55	55		-1.80	3.2511	6.78	45.98535	-12.2272	20C
	UX1500	2.3762	53	53		-1.11	1.2419	4.78	22.86035	-5.3282	20C
	UX1500	3.5298	52	52		0.04	0.0015	3.78	14.29785	0.148249	20C
	UX1500	3.1278	50	50		-0.36	0.1316	1.78	3.172852	-0.64623	20C
	UX1500	2.7758	48.5	48.5		-0.71	0.5109	0.28	0.079102	-0.20104	20C
	UX1500	2.9689	47	47		-0.52	0.2722	-1.22	1.485352	0.635814	20C
	UX1500	3.7678	48.5	48.5		0.28	0.0768	0.28	0.079102	0.077964	20C
	UX1500	3.9820	47.5	47.5		0.49	0.2415	-0.72	0.516602	-0.3532	30C
	UX1500	4.4560	45	45		0.97	0.9320	-3.22	10.36035	-3.1074	30C
	UX1500	1.9226	52	52		-1.57	2.4586	3.78	14.29785	-5.92898	30C
	UX1500	5.0177	42.5	42.5		1.53	2.3321	-5.72	32.7041	-8.73314	30C
	UX1500	4.3600	46	46		0.87	0.7559	-2.22	4.922852	-1.929	40C
	UX1500	4.8696	43	43		1.38	1.9017	-5.22	27.23535	-7.19669	40C
	UX1500	4.8129	41.5	41.5		1.32	1.7485	-6.72	45.1416	-8.88425	40C
	UX1500	5.8294	40.0	40.0		2.34	5.4700	-8.22	67.54785	-19.2221	40C
	UX1500	4.0000			45.5						20C
	UX1500	6.0000			38.0						40C
	Sum	55.8495	771.50		Sum	0.00	31.0925	0.00	429.4844	-109.713	
	Mean	3.4906	48.22								

student's t = 1.761 (90% 2-sided prediction limit)
n-UX1700 = 5
n-UX1500 = 16 d-o-f 14
treg = 2.6690
treg = 4.6990
P500 = 51.3175
P50000 = 43.4884
sigma squared = 0.2190
sigma = 0.4680

90% 2-sided conf. limit	
df	student's t
2	2.92
3	2.353
4	2.132
5	2.015
6	1.943
7	1.895
8	1.86
9	1.833
10	1.812
11	1.796
12	1.782
13	1.771
14	1.761

* runout plotting below the regression line is not included in the regression

UX1500 - 500 hrs (log 2.699)			**UX1500** - 50000 hrs (log 4.699)		
log tL - lower =	2.46	51.3175	log tL - lower =	4.42	43.4884
log tL - upper =	2.94	51.3175	log tL - upper =	4.98	43.4884

UX1700 - logtL @ Load = 2.54 **OK**

UX1700 - logtL @ Load = 4.67 **OK**

UX1700 - All Points
time is dependent variable:

	if time were the y axis	but time is the x axis	
slope	-0.27171	-3.68041	
intercept	16.48735	60.68023	
R squared	0.876735	0.876735	
	-2	68.04105	
	10	23.87611	
2.6990	50.7468	= 500 hr intercept	
4.6990	43.38608	= 50000 hr intercept	

UX1500 - All Points
time is dependent variable:

	if time were the y axis	but time is the x axis	
slope	-0.25545	-3.91462	
intercept	15.8082	61.8831	
R squared	0.901389	0.901389	
	-2	69.71235	
	10	22.73689	
2.6990	51.31754	= 500 hr intercept	
4.6990	43.48841	= 50000 hr intercept	

All Creep Data UX1500 & UX1700
time is dependent variable:

	if time were the y axis	but time is the x axis	
slope	-0.25909	-3.85961	
intercept	15.95864	61.59414	
R squared	0.898285	0.898285	
	-2	69.31336	
	10	22.99802	
5.8176	39.14046	= 75-yr intercept	
5.9425	38.65839	= 100-yr intercept	

The regression for the all creep tests on the primary product (UX1500MSE) produced log 2.6990 hr (500 hrs) and log 4.6990 hr (50,000 hrs) intercepts at 51.32% and 43.49% UTS, respectively. The regression for the creep tests on UX1600MSE and UX1700MSE produced log time intercepts for the same %UTS within the 90% confidence limits of log 2.46 to log 2.94 and log 4.42 to log 4.98 associated with those %UTS. This evaluation is summarized in Table F-6. Thus, the primary, UX1500MSE, and secondary products, UX1600MSE and UX1700MSE, data may be used together to construct the characteristic creep rupture curve of the family of products. Confidence limits satisfied per AASHTO PP66-10.

Table F-6. Summary of statistical comparison between rupture envelopes for UX1500MSE, UX1600MSE and UX1700MSE geogrid products, to test validity of composite creep rupture envelope for product line.

Product	Intercept at log 2.6690 & 4.6990 hrs, %UTS	Intercept at same % UTS, log hrs	90% Confidence Limits @ Higher %UTS, log hrs	90% Confidence Limits @ Lower %UTS, log hrs
UX1500MSE	51.32 & 43.49	2.6690 & 4.6990	-	-
UX1600MSE	-	2.76 & 4.96	2.46 to 2.94	4.42 to 4.98
UX1700MSE	-	2.54 & 4.67	2.46 to 2.94	4.42 to 4.98

TENSAR UX 1500 - UX1600 - UX1700 - COMPOSITE CREEP RUPTURE CURVE

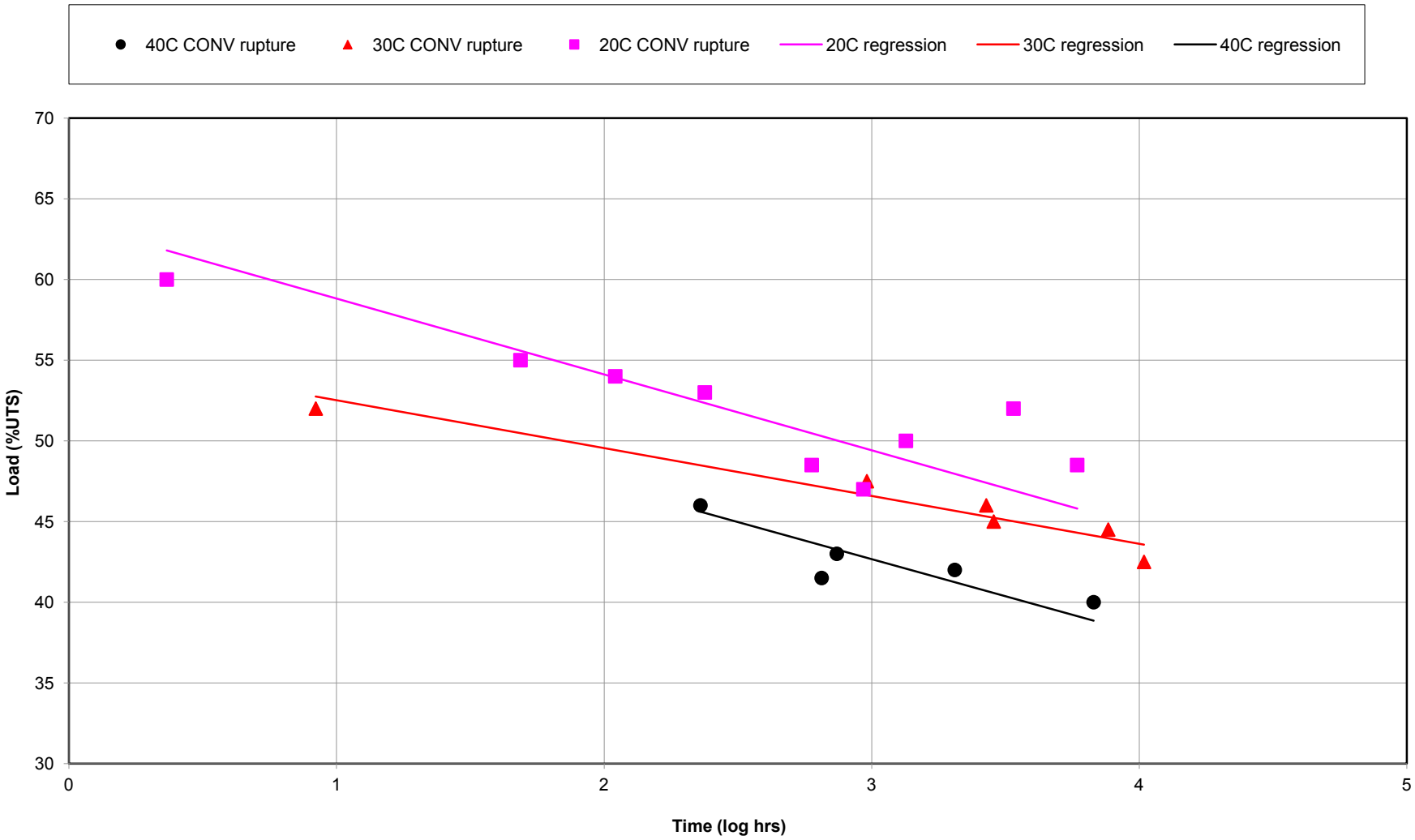


Figure F-49. Actual time to rupture data and regressions for verification of the best fit shift factors of 'x10' for 30°C and 'x100' for 40°C for UX1500, UX1600 & UX1700.

TENSAR UX 1500 - UX1600 - UX1700 - COMPOSITE CREEP RUPTURE CURVE

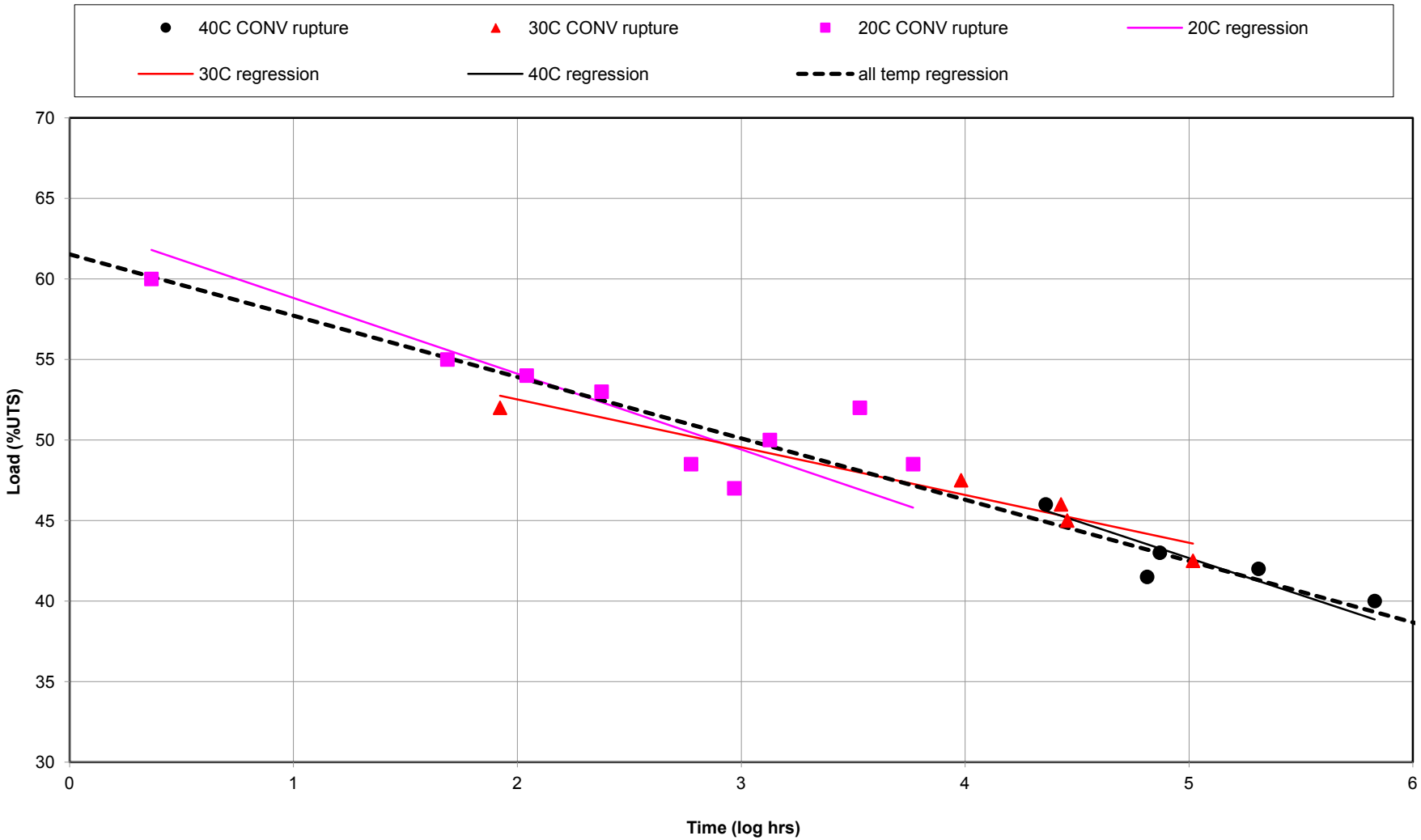


Figure F-50. Time shifted data and regressions for verification of the best fit shift factors of 'x10' for 30°C and 'x100' for 40°C for UX1500, UX1600 & UX1700.

Table F-7. Computation table for composite creep rupture envelope for the Tensar UX1500 - UX1700 geogrid product line (in support of Figure 5-2).

		Stress, % of UTS								
		data for regression calculation						conv"l	conv'l	
product:		loghrs	all	UX1600		UX1500	UX1700	rupture	runout*	
	UX1600	2.0416	54	54				54		20C
	UX1600	2.9930	50	50				50		20C
	UX1600	4.4283	46	46				46		30C
	UX1600	5.3104	42	42				42		40C
	UX1700	1.6390	56				56	56		20C
	UX1700	2.2196	51				51	51		20C
	UX1700	3.0009	48				48	48		20C
	UX1700	4.8840	44.5				44.5	44.5		30C
	UX1700									
	UX1700									
CONV DATA:	UX1500	0.3655	60			60		60		20C
	UX1500	1.6875	55			55		55		20C
	UX1500	2.3762	53			53		53		20C
	UX1500	3.5298	52			52		52		20C
	UX1500	3.1278	50			50		50		20C
	UX1500	2.7758	48.5			48.5		48.5		20C
	UX1500	2.9689	47			47		47		20C
	UX1500	3.7678	48.5			48.5		48.5		20C
	UX1500	4.0000							45.5	20C
	UX1500	3.9820	47.5			47.5		47.5		30C
	UX1500	4.4560	45			45		45		30C
	UX1500	1.9226	52			52		52		30C
	UX1500	5.0177	42.5			42.5		42.5		30C
	UX1500	4.3600	46			46		46		40C
	UX1500	4.8696	43			43		43		40C
	UX1500	4.8129	41.5			41.5		41.5		40C
	UX1500	5.8294	40.0			40.0		40.0		40C
	UX1500	6.0000							38.0	40C

NOTE: Don't include runouts in the regression calculation unless the points lie above the line

Conventional - UX1600
time is dependent variable:

	if time were the y axis	but time is the x axis
slope	-0.28104	-3.558
intercept	17.18337	61.14
R squared	0.991374	0.991
	-2	68.26
	10	25.56
6	39.79243	= 114 Year intercept
5.817863	40.44051	= 75 Year intercept

Conventional - UX1700
time is dependent variable:

	if time were the y axis	but time is the x axis
slope	-0.27171	-3.68
intercept	16.48735	60.68
R squared	0.876735	0.877
	-2	68.04
	10	23.88
5.999706	38.59884	= 114 Year intercept
5.817863	39.2681	= 75 Year intercept

Conventional - UX1500
time is dependent variable:

	if time were the y axis	but time is the x axis
slope	-0.25545	-3.915
intercept	15.8082	61.88
R squared	0.901389	0.901
	-2	69.71
	10	22.74
5.999706	38.39652	= 114 Year intercept
5.817863	39.10837	= 75 Year intercept

Conventional - All
time is dependent variable:

	if time were the y axis	but time is the x axis
slope	-0.2625988	-3.8080912
intercept	16.157027	61.527433
R squared	0.9094014	0.9094014
	-2	69.143615
	10	23.44652
5.9997063	38.680004	= 114 Year intercept
5.8178627	39.372481	= 75 Year intercept

Appendix G: Durability Detailed Test Results

Table G-1. Oxidation resistance test results of UX1100 geogrid.

TRI Log #: E2280-68-03

PARAMETER	TEST REPLICATE NUMBER 1	2	3	4	5	MEAN	STD. DEV.	PERCENT RETAINED	AASHTO R69-15 SPEC
Oxidation Resistance (EN ISO 13438)									
Strength Retained measured via wide width tensile (ISO/EN 10319, mod.)									
MD Number of Ribs per Specimen:	1								
MD Number of Ribs per foot:	13.52								
MD - Tensile Strength (lbs) - B	336	329	338	340	335	**336**	4.3		
MD Tensile Strength (lbs/ft) - B	4545	4444	4571	4600	4535	**4539**	58.7		
MD Tensile Strength (kN/m) - B	66.4	64.9	66.7	67.2	66.2	**66.3**	0.9		
MD - Tensile Strength (lbs) - E	334	320	326	336	321	**327**	7.2		
MD Tensile Strength (lbs/ft) - E	4510	4330	4406	4543	4339	**4426**	97.3	**98**	50 min
MD Tensile Strength (kN/m) - E	65.8	63.2	64.3	66.3	63.3	**64.6**	1.4		
MD - Elong. @ Max. Load (%) - B	11.2	10.8	10.8	11.1	10.4	**10.9**	0.3		
MD - Elong. @ Max. Load (%) - E	15.6	15.1	15.3	16.3	15.0	**15.5**	0.5	**142**	
B - Baseline Exposed to 100C for 6 hours									
E - Exposed to 100C for 28 days									

MD - Machine Direction TD - Transverse/Cross Machine Direction

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table G-2. UV resistance test results of Tensar UX1100 geogrid.

TRI Log #: E2280-68-03

PARAMETER	TEST REPLICATE NUMBER 1	2	3	4	5	MEAN	STD. DEV.	PERCENT RETAINED	AASHTO R69-15 SPEC
UV Resistance (ASTM D 4355)									
Strength Retained measured via single strip tensile (ASTM D 6637, Method A)									
MD - Number of Ribs per foot:	13.52								
MD - Tensile Strength (lbs) - B	333	334	338	331	331	**333**	2.8		
MD Tensile Strength (lbs/ft) - B	4504	4513	4568	4481	4472	**4508**	37.8		
MD Tensile Strength (kN/m) - B	65.8	65.9	66.7	65.4	65.3	**65.8**	0.6		
MD - Tensile Strength (lbs) - E	331	332	331	309	332	**327**	10.3		
MD Tensile Strength (lbs/ft) - B	4475	4489	4481	4174	4490	**4422**	138.7	**98**	70 min
MD Tensile Strength (kN/m) - B	65.3	65.5	65.4	60.9	65.6	**64.6**	2.0		
MD - Elong. @ Max. Load (%) - B	10.6	10.7	11.1	10.5	10.4	**10.7**	0.3		
MD - Elong. @ Max. Load (%) - E	11.9	11.1	11.7	11.0	11.5	**11.4**	0.4	**107**	

B - Baseline Unexposed
E - Exposed for 500 hours of ASTM D 4355 Cycle

MD - Machine Direction TD - Transverse/Cross Machine Direction

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested.

Table G-3. Summary of UV resistance test results for UX-MSE / US-HS geogrid.

UX-MSE Series Style	Mean Baseline Tensile Strength (lb/ft)	Standard Deviation (lb/ft)	Mean Exposed Tensile Strength (lb/ft)	Standard Deviation (lb/ft)	% Strength Retained
UX1100	4,508	37.8	4,422	138.7	98

(Conversion: 1 lb/ft = 0.0146 kN/m)

Table G-4. Summary of UV resistance test results for UX-MSE / US-HS geogrid.

UX-MSE Series Style	Mean Baseline Tensile Strength (lb/ft)	Standard Deviation (lb/ft)	Mean Exposed Tensile Strength (lb/ft)	Standard Deviation (lb/ft)	% Strength Retained
UX1100	4,539	58.7	4,426	97.3	98

(Conversion: 1 lb/ft = 0.0146 kN/m)

Appendix H: Creep Stiffness Detailed Test Results

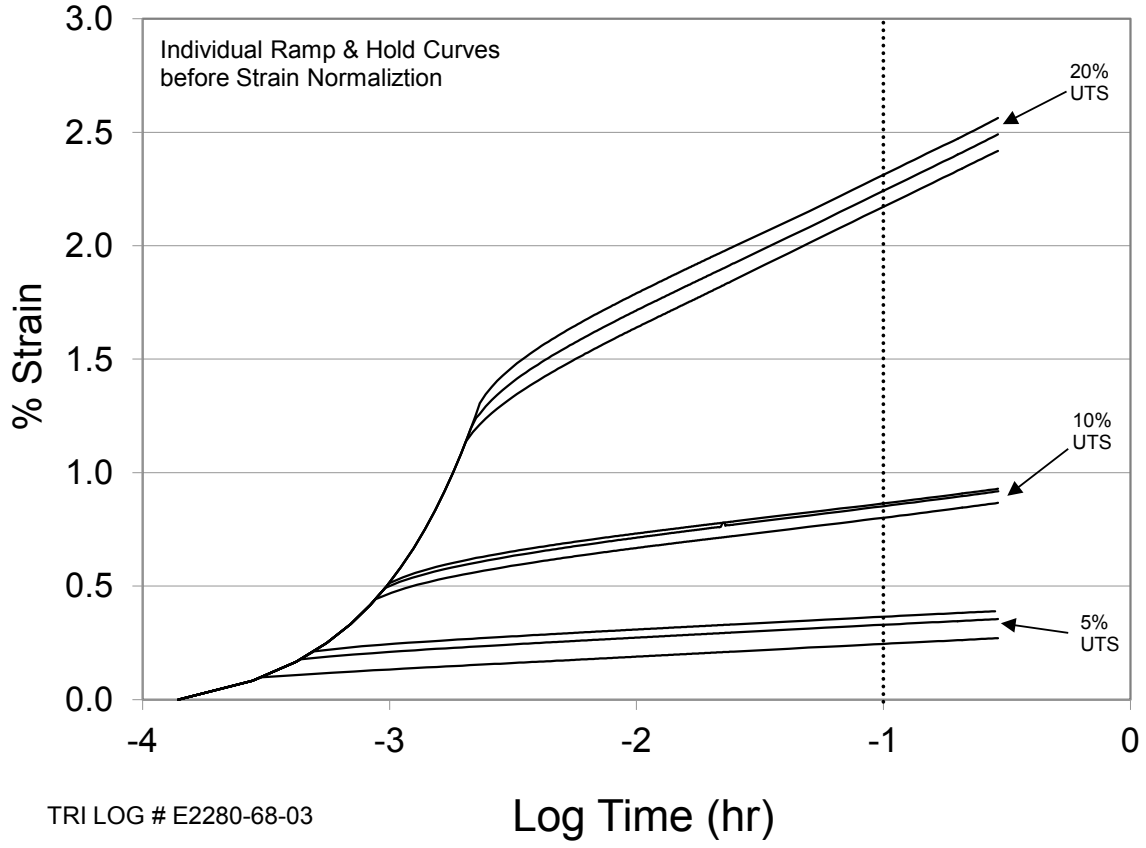


Figure H-1. Low strain ramp and hold tests for Tensar UX1100 geogrid, before strain normalization.

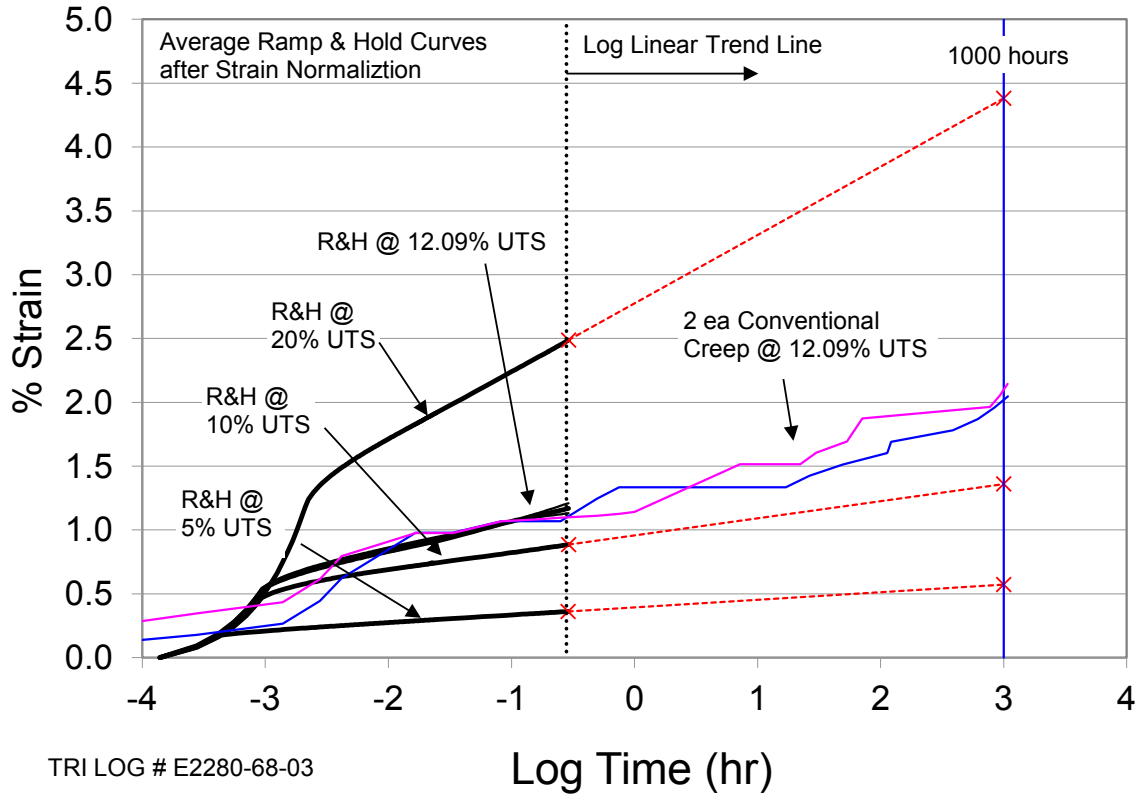


Figure H-2. Low strain ramp and hold tests for Tensar UX1100 geogrid, after strain normalization, with 1000 hour low strain creep tests.

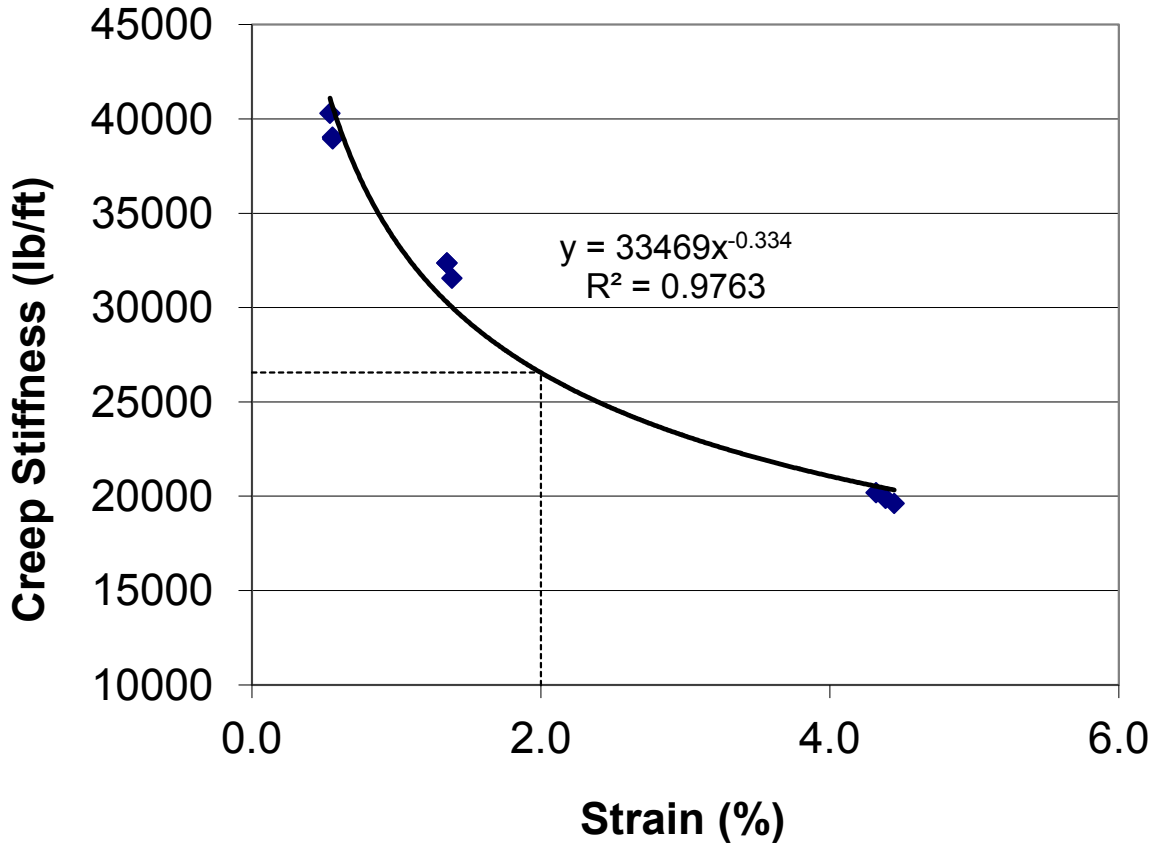
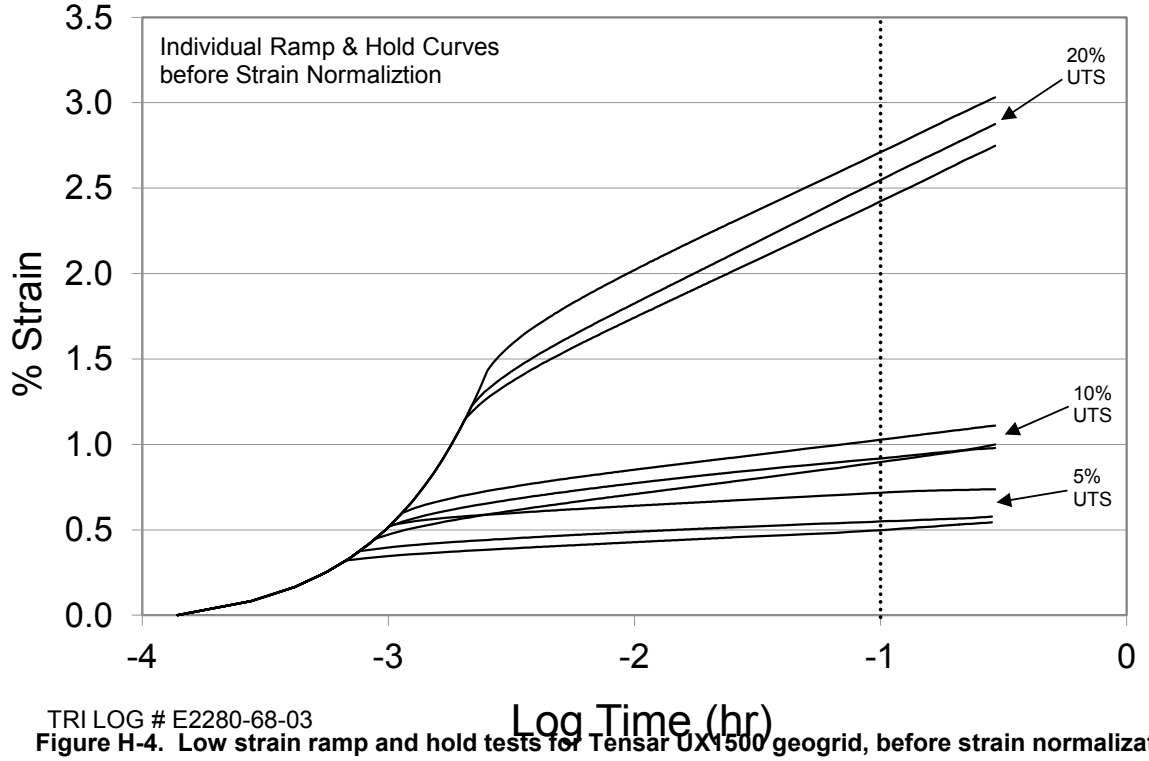


Figure H-3. Creep stiffness versus strain at 1,000 hours for Tensar UX1100 geogrid.



TRI LOG # E2280-68-03

Figure H-4. Low strain ramp and hold tests for Tensar UX1500 geogrid, before strain normalization.

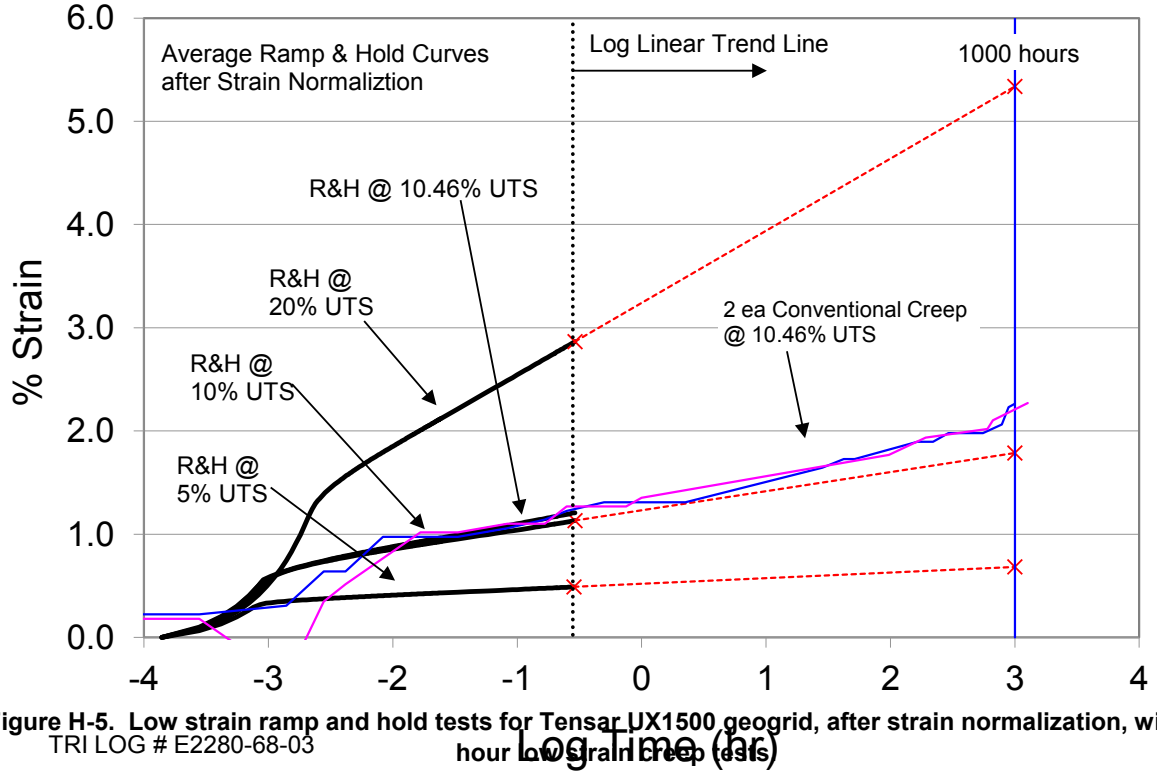


Figure H-5. Low strain ramp and hold tests for Tensar UX1500 geogrid, after strain normalization, with 1000 hour Low Strain Creep tests
TRI LOG # E2280-68-03

Log Time (hr)

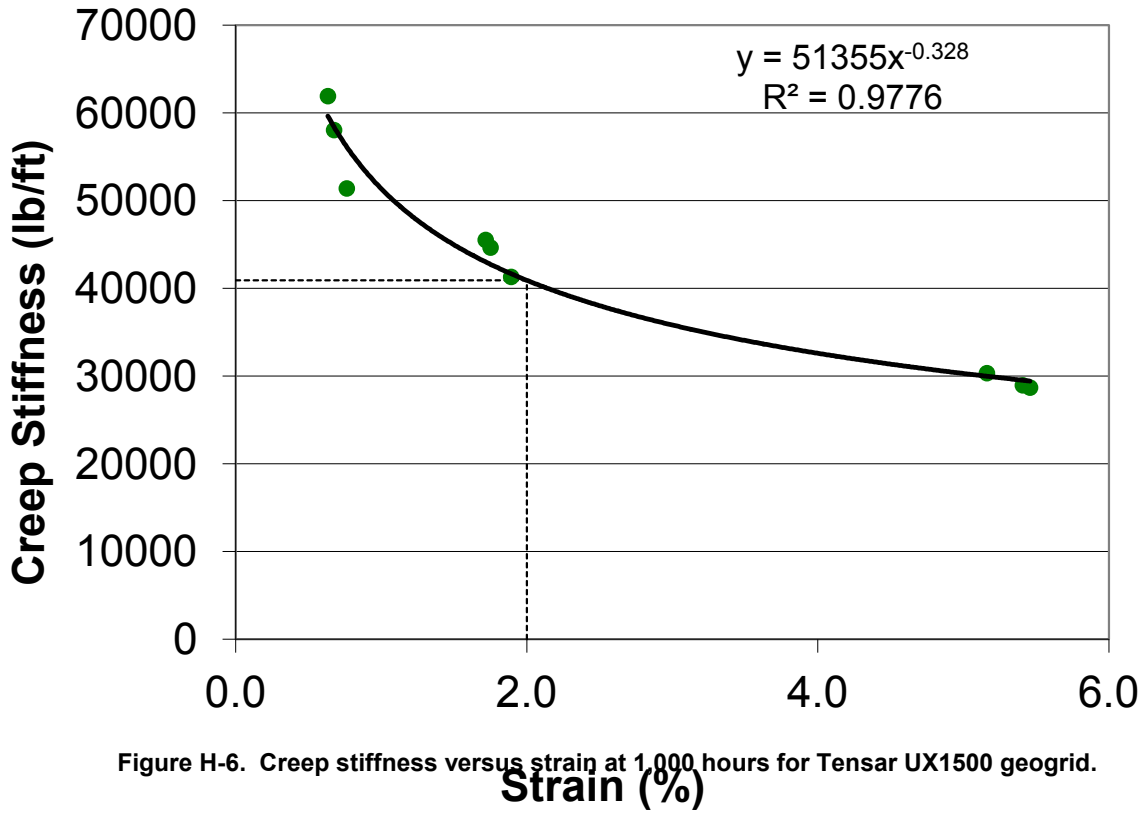


Figure H-6. Creep stiffness versus strain at 1,000 hours for Tensar UX1500 geogrid.

Strain (%)

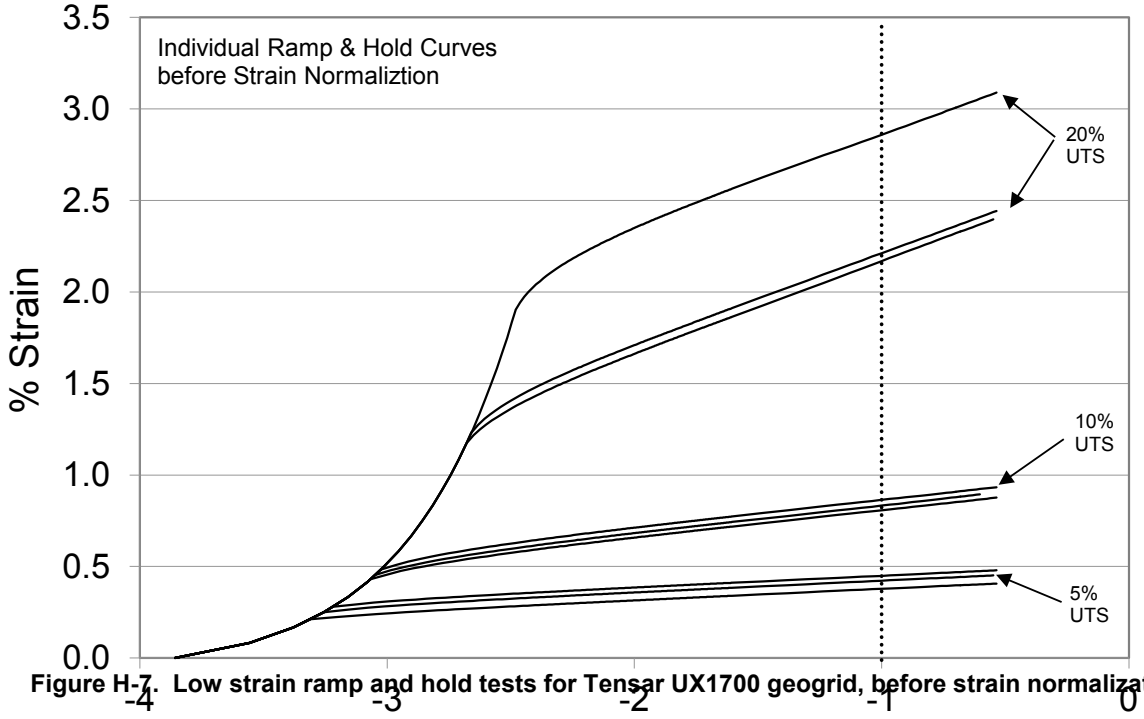


Figure H-7. Low strain ramp and hold tests for Tensar UX1700 geogrid, before strain normalization.

-4 -3 -2 -1 0

TRI LOG # E2280-68-03

Log Time (hr)

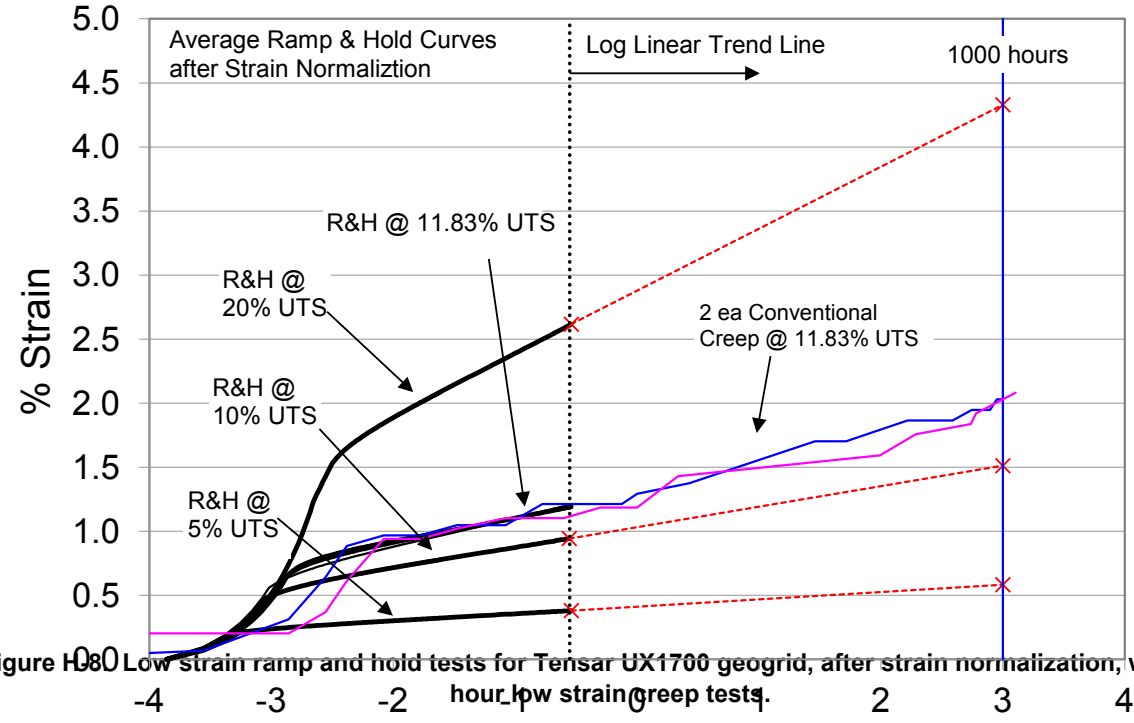


Figure H-8 Low strain ramp and hold tests for Tensar UX1700 geogrid, after strain normalization, with 1000 hour low strain creep tests.

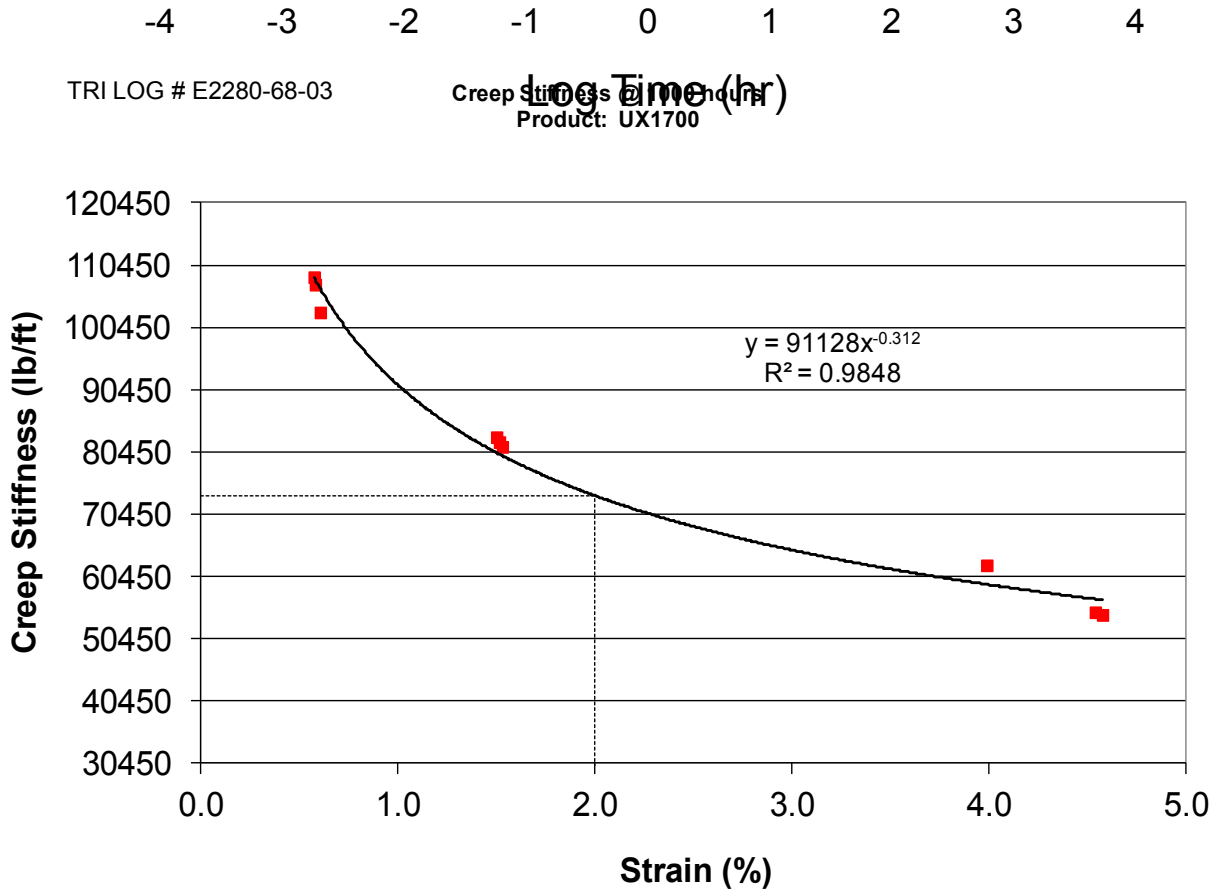


Figure H-9. Creep stiffness versus strain at 1,000 hours for Tensar UX1700 geogrid.

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-- Rick Smutzer (IN), former NTPEP Chairman

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Appendix 1.2.6.1

MESA System Long-Term Connection Test (A)

MESA® MSE Retaining Wall System

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SUPPLEMENTAL TEST RESULTS FOR MESA® LONG-TERM CONNECTION TEST

by Willie Liew, P.E. May 31st, 2011 (update 5/13/13)

The initial report "Long-term MESA DOT Connection Tests" from Tensar International Corporation dated September 26, 2007 contained the long-term connection test set up, testing protocols, test data and results. Extensive testing was performed and test data was collected for UX1400MSE and UX1500MSE geogrid. At the time, the connection tests with UX1600MSE geogrid were inconclusive based on a limited number of tests while the testing with UX1700MSE was pending.

Numerous long-term connection tests have been performed with UX1600MSE and UX1700MSE geogrids since the initial report. All test results including the results for UX1400MSE and UX1500MSE geogrid are shown in Table 1 and the composite long-term connection curve (%T_{LOT} versus log Time to Rupture) are presented in Figure 1.

TABLE 1 Full Scale Long Term Connection Tests

Geogrid	Ave. +95% Load (plf)	Load % of T_{LOT}	Time to Geogrid Rupture (hrs)	Time to Rupture 40C Shifted x 100
UX1400MSE @ 40C	1848	36.4	244	24445
	1699	33.5	7682	768200
UX1500MSE @ 20°C	3503	43.6	65.6	65.6
	3122	38.9	2841	2841
	3328	41.2	360	360
UX1500MSE @ 40°C	3129	38.9	27	2700
	3056	38.05	207	20700
	3032	37.8	220	22000
	3102	38.6	71.67	7167
	3113	38.8	84	8400
	3028	37.7	216.5	21650
	2978	37.1	198	19800
	2917	36.3	521	52100
UX1600MSE @ 20°C	3787	37.1	6956	6956
	3491	35.1	409	409
	3178	32	1027	1027
UX1600MSE @ 40°C	3435	34.6	14	2170
	3163	31.8	23	3565
UX1700MSE @ 20°C	4038	32.8	21996	21996
	4648	37.8	4030	4030
	4556	36.6	67	67
	4257	34.2	283	283
	4048	32.5	285.5	285.5

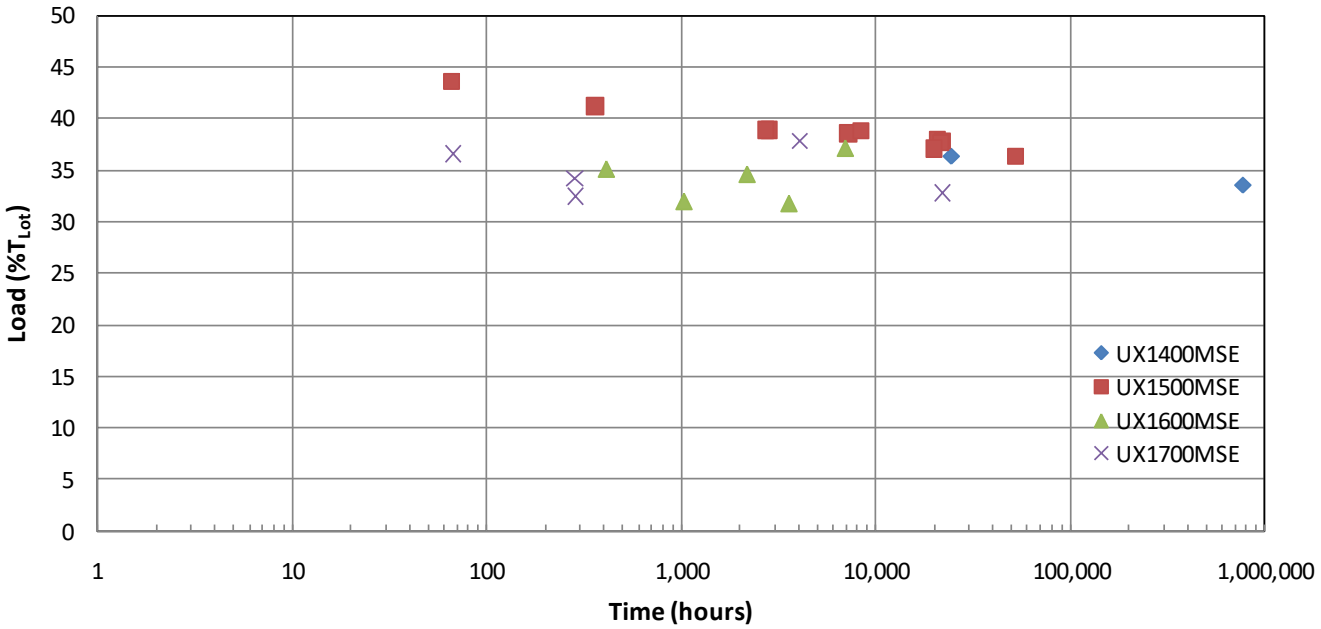


Figure 1 – Full-Scale MESA Long-term Connection Test Curve (%T_{LOT} versus log Time to Rupture)

The time to rupture for UX1400MSE and UX1500MSE remained the same as shown in the initial report except that the "shift factor" is adjusted from the x155 to x100. The adjustment is made to be consistent with the current shift factor used for extrapolating UX****MSE geogrid series.

As shown in Figure 1, the data points for UX1600MSE and UX1700MSE are scattered except couple of rupture points that fall on the same rupture envelope of the UX1400MSE and UX1500MSE. A different behavior is observed from the connection tests with UX1600MSE and UX1700MSE geogrids. Instead of geogrid rupture as observed with the lighter geogrid, the failure modes are a combination of geogrid rupture, shearing of transverse bar through the connector and connector failure.

The stronger and thicker geogrid creates greater separation between blocks, and produces inconsistent failure mechanism from rotation of block, connection and geogrid. The pinching of the thicker geogrid between blocks could also explain the inconsistent results. Nonetheless, according to AASHTO LRFD, long-term connection tests are performed and long-term connection strength is to be calculated regardless of the failure mode.

Two different long-term connection curves can be produced from Figure 1: One for UX1400MSE and UX1500MSE geogrid and one for UX1600MSE and UX1700MSE geogrid. The long-term connection curve for UX1400MSE and UX1500MSE is shown in Figure 2. Based on the best fit curve, the connection strength for UX1400MSE and UX1500MSE geogrid with DOT connector is 33.8% of T_{ULT} for a 75-year design life.

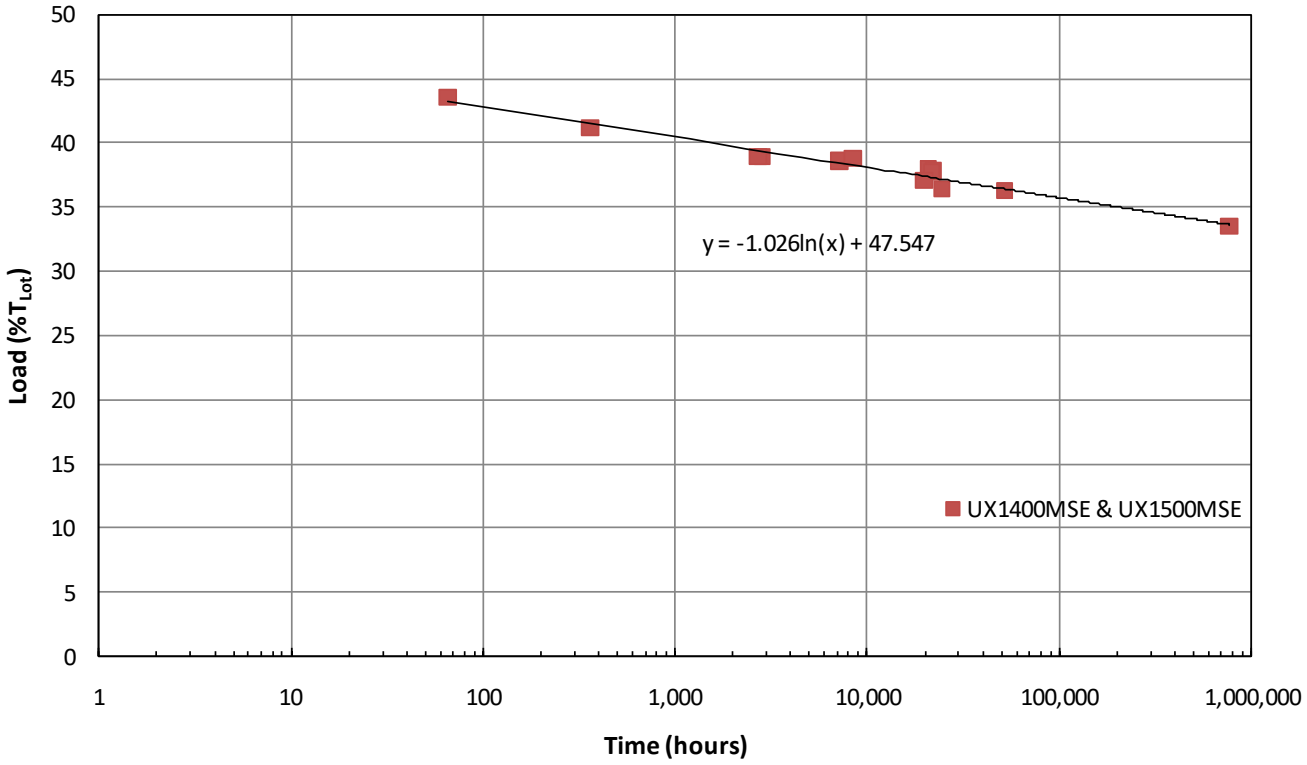


Figure 2 – Full-Scale MESA Long-term Connection Test Curve (UX1400MSE and UX1500MSE)

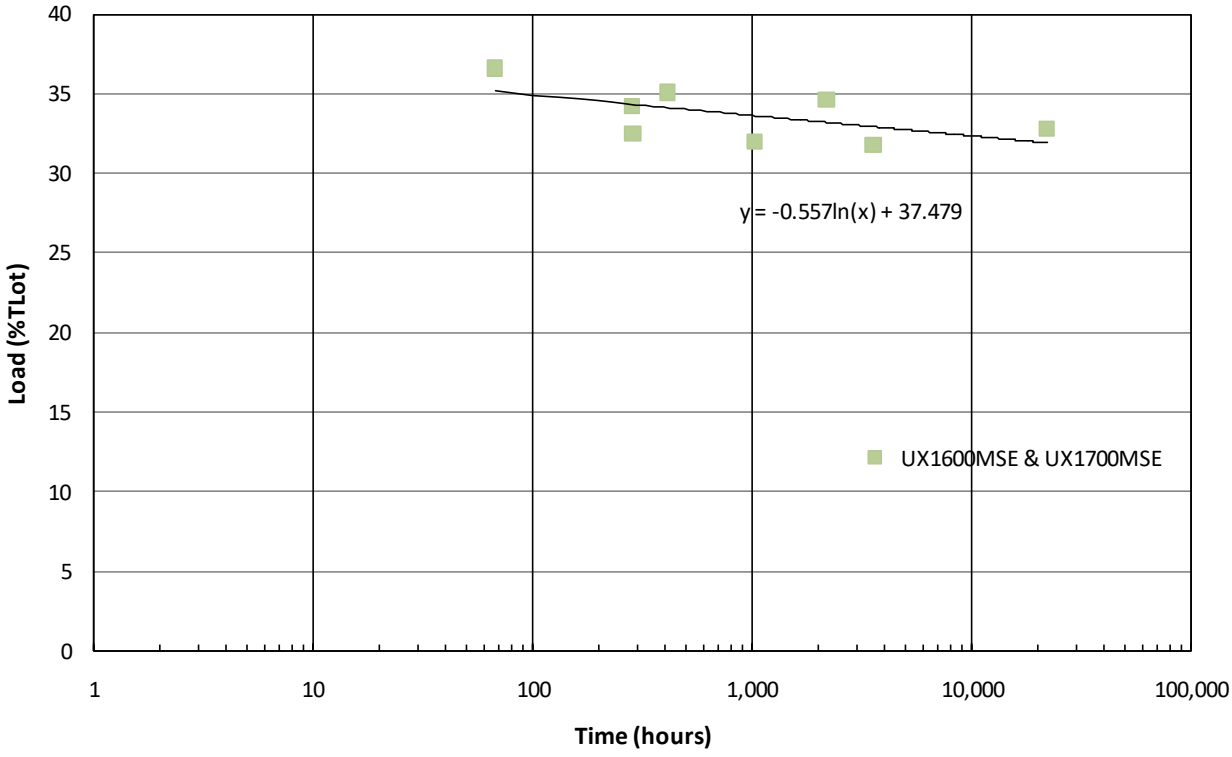


Figure 3 – Full-Scale MESA Long-term Connection Test Curve (UX1600MSE and UX1700MSE)

The Long-term connection curve for UX1600MSE and UX1700MSE geogrid are plotted as shown in Figure 3. Note that the two rupture points (one for UX1600MSE and one for UX1700MSE) that fall on the same rupture envelope as UX1400MSE and UX1500MSE geogrid are removed from the plot to conservatively calculate the connection strength. Based on the best fit curve, the connection strength for UX1600MSE and UX1700MSE geogrid with DOT connector is 30% of T_{ULT} for a 75-year design life.

From Figures 2 and 3, the long-term connection parameters and strength for UX1400MSE through UX1700MSE geogrid with DOT connector is summarized in Table 2 below.

Table 2 – Connection Strength Design Value for MESA Retaining Wall System

Geogrid Type	T_{ult} (MARV) for Geogrid Reinforcement in WSDOT QPL (lbs/ft)	CR_{cr} (75-yr design life)	Connection Strength T_{ac} (lbs/ft)
UX1400MSE	4,800	0.338	1,475
UX1500MSE	7,810	0.338	2,400
UX1600MSE	9,870	0.30	2,692
UX1700MSE	11,990	0.30	3,270

T_{ac}, the long-term connection strength is calculated as follows (AASHTO LRFD Equation 11.10.6.4.4b-1)

$$T_{ac} = \frac{T_{MARV} * CR_{cr}}{RF_d}$$

where, RF_d = 1.1 for all Tensar HDPE geogrids

Appendix 1.2.6.2

MESA System Long-Term Connection Test (B)

MESA® MSE Retaining Wall System

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LONG TERM MESA DOT CONNECTION TESTS

TENSAR UX****MSE GEOGRIDS

By Robert B. Anderson, PE
For
Tensar International, Inc.
14 October 2005,
Last Revised 26 September 2007 by VECC

LONG TERM MESA DOT CONNECTION TESTS
With TENSAR UX****MSE GEOGRIDS
By Andy Anderson, PE, Tensar International, Inc.
14 October 2005, Last Revised 26 September 2007

The Mesa "DOT" facing-to-geogrid connector has been developed as a replacement for the High Performance, (HP), connector. It is a mechanical connector that structurally joins Tensar geogrids to Mesa Standard facing blocks. The connectors are injection molded using a compound of Polypropylene with 25% fiber glass inclusions.

This long term connection strength test program has been underway since late in 2003 at The Tensar Corporation's Quality Control Laboratory in Morrow, Georgia. The purpose of the program is to directly measure long term connection strength. It provides connection strength information that can be used directly in the design of Mesa modular block faced Mechanically Stabilized Earth, (MSE), retaining walls in the same way that long term creep limited geogrid strength has been routinely used for geogrid soil reinforcement design in MSE retaining walls. In the past, long term connection strength has had to be estimated from short term connection tests.

Two test rigs were used. They apply a horizontal tensile load to the geogrid and its connection to Mesa Standard facing blocks and a vertical confining load to the top of the facing blocks. The tensile load is applied by dead load acting through a 6.4:1 mechanical advantage gear drive. The confining load is applied by a weight placed on a levered system. The test rigs are located in two temperature controlled rooms, one set at the standard laboratory temperature of 20^{o}C (68^{o}F), the other at the elevated temperature of 40^{o}C (104^{o}F). The elevated temperature has been shown to predictably accelerate creep rates in high density polyethylene, (HDPE), geogrids. This approach enables extrapolation of test data to the typical 75- to 120-year design lives of MSE structures using test times that range from days to months instead of years to decades.

TEST APPARTUS, SETUP AND PROCEDURES

One of the test rigs is shown in the photos in Figure 1. Two courses of Mesa block facing units are positioned on the platform shown in the upper photo on the right end of the rig. The lower course consists of an 18-inch wide full block centered between two half blocks and the upper course consists of two full blocks. The Tensar uniaxial geogrid samples, seen on the right side of lower photo, are about 17 inches wide and 21 inches long and consist of 20 longitudinal ribs and two transverse bars. The samples are connected to the lower centered Mesa block with the 19-tooth, 8-flag "DOT" connector which is shown in the photographs in Figure 2. The teeth are inserted through the 19 grid apertures with the four teeth at each end driven into the connector slots in the webs of the Mesa block. The intermediate 11 teeth are positioned in the core of the lower block. The four flags at each end of the connector extend up into the cores of the overlying blocks. The intermediate teeth and the flags at each end bear against the rear faces of the cores of the lower and upper blocks respectively. The transverse bar of the geogrid bears against each of the 19 teeth. The DOT connector may be split into three pieces to fit the aperture spacings.



Figure 1 Long Term Connection Test Apparatus

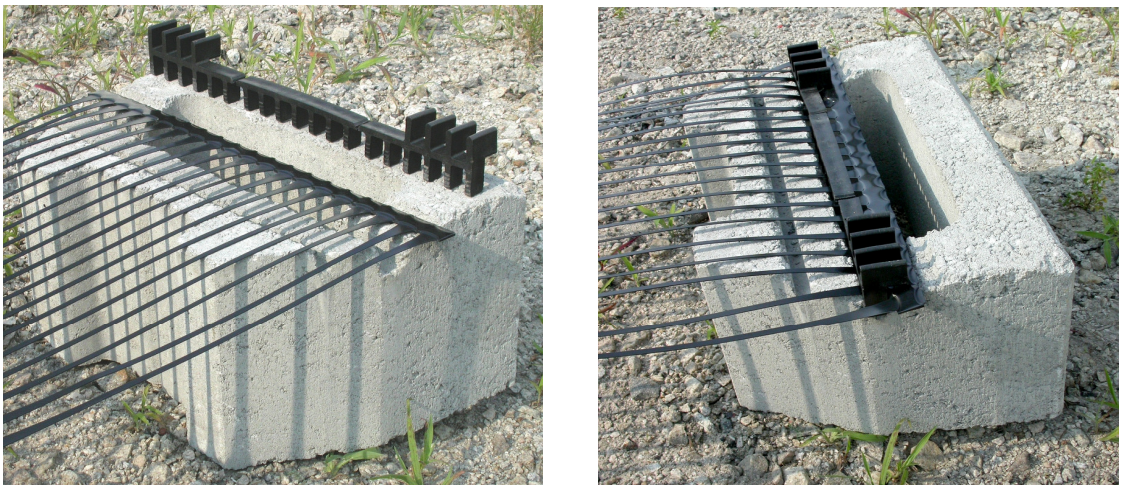


Figure 2 Mesa DOT Connector and Standard Facing Block

The opposite transverse bar of the geogrid is clamped and connected, through a load cell, to a tensioning chain. The chain engages a 15-tooth sprocket at the center of an axle that is driven by twin 96-tooth sprockets at each ends. The large drive sprockets are loaded by two chains connected to a dead load support cradle. The opposite ends of the two chains are connected to the red hydraulic cylinder mounted horizontally at the base of the unit. The hydraulic cylinder serves to raise the weights, to permit the gradual application of the load to the grid, and to absorb the shock when the geogrid ruptures and the weights drop at the end of the tests.

The confining load is applied by a cushioned steel plate placed on the two upper Mesa blocks. A lever system with a sliding weight shown upper center of the second photo is used to apply the confining pressure to the plate. A load cell was initially used to calibrate the position of the weight to the applied confining pressure. Pressures of 6.5 and 10 psi were used for the various tests.

The load on the geogrid and connection is applied by slowly releasing the pressure in the hydraulic cylinder which initially supports the weights. By slowly applying the load, over a period of a few hours, stresses are more evenly distributed over the connection due to creep strains that occur in the geogrid. The tensile load is monitored by a load cell and the deflection of the clamp, relative to the facing blocks, is monitored by a LVDT seen above the geogrid and clamp in the second photo. Data is collected and processed by a computer program by Laboratory Technical Corp. Initially, the program records loads and deflections at increasing time intervals ranging from a few seconds to several minutes. After 24 hours, it records data hourly. The load and deflection monitoring equipment, programs and procedures were initially supplied, set up and established by SGI Testing Services, LLC.

The time-to-failure of the geogrid connection is the principle property determined in these tests. Connection load versus time-to-failure for a range of loads can be used to predict the connection strength at the end of the design life of a structure, 75 or 120 years

for typical MSE retaining wall structures. The use of elevated test temperatures has been shown to accelerate stress-rupture in the HDPE geogrids in a uniform and predictable manner. At 40°C, the time it takes for Tensar HDPE geogrids to rupture has been found to be less than one percent of the time required at a normal test temperature of 20°C for any given load. Thus, a test that ruptures, two months at 40°C would take 15 to 20 years to rupture if tested at 20°C. Therefore, combined data from standard tests at 20°C and accelerated tests at 40°C can be extrapolated, using well established time-temperature superposition techniques, to estimate the connection strength at the required design life. Such techniques are concurrently being used to estimate the long term design strength of Tensar UX****MSE HDPE geogrids

TEST RESULTS

Example Load and Deflection versus log Time curves for a UX1600MSE at 20°C are presented in Figure 3. The curves show how the tensile load was transferred from the hydraulic cylinder to the geogrid and connector over the first 6 to 8 hours of the test. The tensile load is seen to vary slightly over time (and displacement of the geogrid clamp). These variations are caused by the slight changes in the applied dead load moment due to small changes in leverage radii as the chain links are picked up by the teeth of the rotating sprockets. For analysis, the average load applied between the maximum load and 95% of the maximum load was used. For the test in Figure 3, this average load between test times of 5 and 6358 hours was calculated to be 5681 pounds. This test was one of the longer in duration of the tests performed to date. Many tests ran for tens to several hundreds of hours. During this 20°C test, a power failure resulted in the room temperature rising to about 25°C for about two days in July. The effect of the elevated temperature is shown in the increased strain rate observed at about 3900 hours. The elevated temperature effect was estimated, using curve fitting procedures, to have reduced the time-to-failure by about 600 hours for this test., Therefore 600 hours were added to the observed time to give a corrected test time-to-failure of 6958 hours.

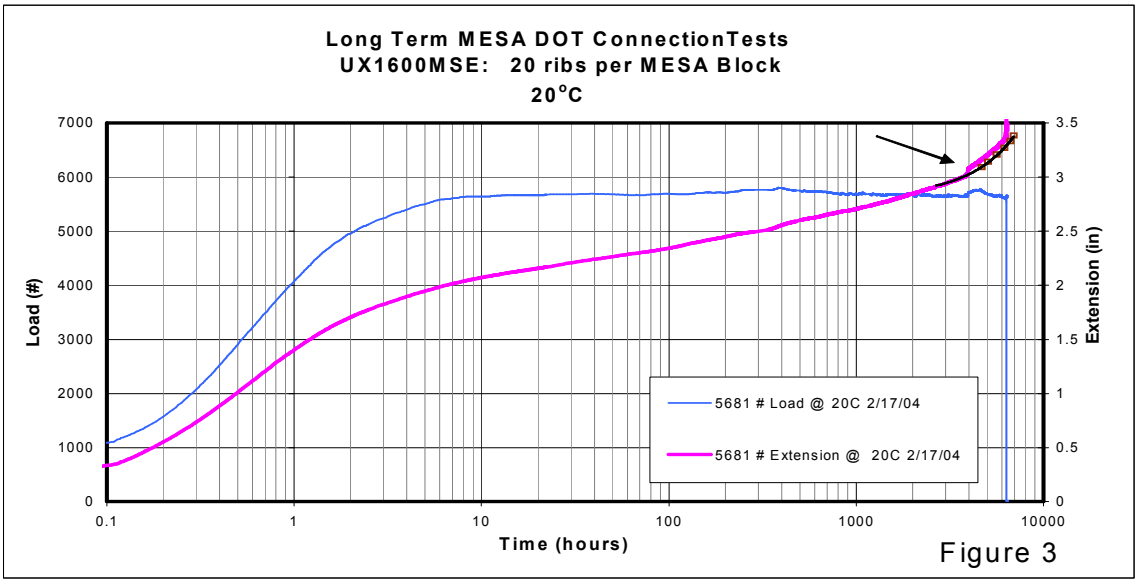


Figure 3: Connection Tests UX1600MSE

Mode of Failure

The mode of failure differed between tests and depended on the type of grid, test temperature and confining pressure. In the UX1400MSE and UX1500MSE tests at both temperatures and the UX1600MSE at 20°C, the predominant modes of failure were rupture of the geogrid ribs and shearing of the transverse bars at the one or both of the two outside apertures. These modes are shown in Figure 4.

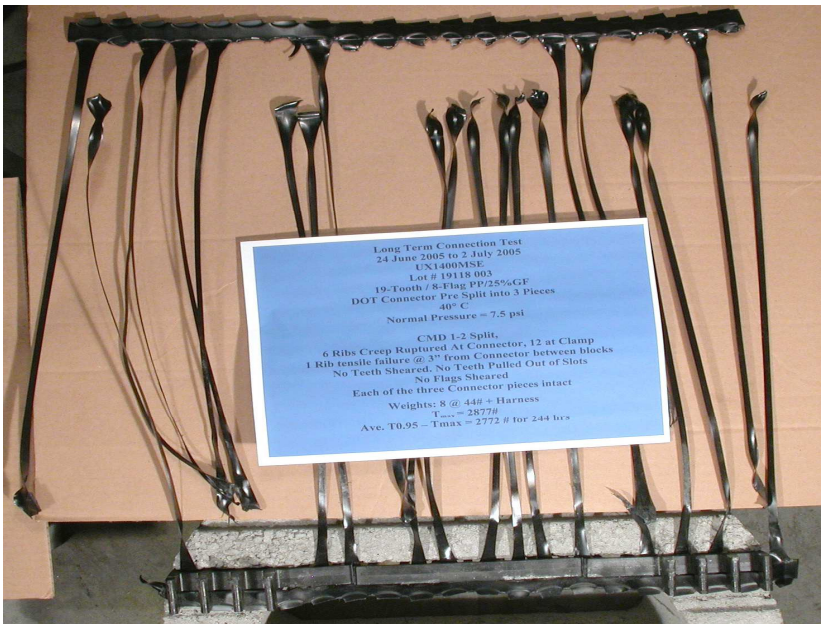


Figure 4: Geogrid Rupture Mode

In some of the UX1600MSE tests, the failure modes were a combination of 1), geogrid failure, that is rupture of the some ribs and shearing of some transverse bars, and of 2), connector failure, shearing of some connector teeth flags.

The following are general observations concerning geogrid rupture failure mode:

The transverse bars between ribs 1 and 2 and between 19 and 20 sometimes shear before general failure. This shearing appears to be due to the fact that the entire load in the outer most rib is carried by shear stress in the transverse bar on only one side of the rib, whereas inner rib loads are distributed to two transverse bars.

Ribs 2 to 4 and 17 to 19 usually fail by stress-rupture of the ribs. These ribs are completely supported by connector teeth embedded in the connector slots and the connector flags that bear against the face of the overlying block core.

Ribs 5 through 16 rupture in some tests and partially rotate and pullout the center section of the connector in others. The connector bridge over teeth 5 through 16 is confined vertically under the webs of the overlying blocks (except for the center tooth). The bridge tends to lift the overlying blocks as the moment induced by

geogrid tension tends to rotate the center section of the connector. When the ribs in the two flagged sections of the connector rupture, the tension is transferred to the center geogrid ribs which increases the lifting force exerted by the connector. The combination of high lifting force and low vertical confining pressure can result in sufficient movement of the connector to preclude rupture of some of the center ribs of the sample.

The problem of rotation of the center section of the DOT connector was most evident in the tests using UX1600MSE. The test rigs could not apply the relatively high vertical pressures that would confine UX1600MSE in higher retaining walls. Consequently, in some tests with UX1600MSE the overlying blocks lifted sufficiently to allow an increase and concentration of the shear and tensile stresses in the teeth and to thereby cause the teeth to shear. The results of tests at low confining pressures in which connector rupture occurred have not been considered in the following analyses of DOT connection strength for UX1400MSE, UX1500MSE and UX1600MSE.

It is important to note that in short term, full scale DOT connection tests with UX1600MSE, in which 20 psi and higher confining pressures were used, the only failure mode observed was geogrid rupture. In similar full-scale short term tests using UX1700MSE, both the connector and the concrete block ruptured, but the geogrid did not. The ultimate loads in the short term tests were much higher than design loads. Long term testing at loads closer to the normal design load for UX1700MSE is scheduled, pending test rig modifications to allow higher confining pressures, to determine whether the UX1700MSE geogrid or the DOT connector strength "controls". If it is found that connector shear strength controls, additional testing will be conducted to establish a 40^{o}C time-shift factor for connector shear strength.

Full-scale Long Term DOT Connection Test Results

A summary of the full-scale long term DOT connection tests conducted and their results is presented in Table 1. The average load applied to the geogrid and connection over the period of time between when the load first reached and when it later fell below 95% of the maximum load is listed as both the total load in pounds (per 18 inch wide facing block, per 20 geogrid ribs and per 19 connector teeth) and as the percentage of the short term, wide width or single rib, ultimate tensile strength of the geogrid production lot, "T_{LOT}". Use of the percentage of T_{LOT} permits comparison of normalized connection strengths between production lots of the same type of geogrid and between different types of geogrids. (The percentage of the geogrid T_{LOT} is applied to the geogrid $T_{ULTMARV}$ short term ultimate strength for each geogrid type to arrive at the Long Term Connection Strength for MSE retaining wall design.)

TABLE 1 Full Scale Long Term Connection Tests

Geogrid	Ave. +95% Load (plf)	Load % of T_{LOT}	Time to Geogrid Rupture (hrs)	Time to Rupture 40C Shifted x 155
UX1400MSE @ 40C	1848	36.4%	244	37890
	1699	33.5%	7682	1190710
UX1500MSE @ 20°C	3503	43.6%	65.6	65.6
	3122	38.9%	2841	2841
	3328	41.2%	360	360
UX1500MSE @ 40°C	3129	39.0%	27	4185
	3056	38.1%	207	32085
	3032	37.8%	220	34100
	3102	38.6%	72	11109
	3113	38.8%	84	13020
	3028	37.7%	217	33558
	2978	37.1%	198	30690
	2917	36.3%	521	80755
UX1600MSE @ 20°C	3787	37.1%	6956	6956

Laboratory Geogrid Stress-Rupture Test Results

The geogrid stress-rupture mode of failure observed early in the full-scale connection testing program using UX1500MSE tests suggested that standard creep tests on the geogrid alone could supplement the full-scale long term connection tests. This was confirmed by comparing the connection test results with standard laboratory-scale creep tests being conducted during the same period.

Standard creep tests use samples that are three ribs wide and three apertures long. The samples are hung with various dead loads in the same 20°C and 40°C rooms as the connection test rigs and also in a 30°C room. The samples are normally suspended using steel clamps that grip the top and bottom transverse bars. The clamps are similar to those used to grip the free end of the geogrid in the full-scale connection tests. To establish whether or not the Mesa connector changes stress distributions and affects creep test results, several creep tests were conducted using the connector teeth instead of the steel clamp to suspend the samples. The creep strains and failure mode results for the two support systems were found to be similar. Therefore laboratory-scale creep tests, with the samples supported by either standard steel clamps or by Mesa Connector teeth, could be used to determine long term connection strength when used in conjunction with the full-scale long term connection tests. This approach accelerated the connection testing

program since several stress-rupture tests could be conducted concurrently on each type of geogrid and in all three of the temperature controlled rooms.

These standard and connector supported laboratory-scale creep test data were used to determine that the elevated temperature of 40°C accelerates creep by a factor of about 155 times for UX1600MSE geogrids. The actual times-to-rupture of the 40°C tests are multiplied by this time-temperature shift factor to estimate the time it would take for the geogrid to rupture if tested with the same load at a temperature of 20°C.

Analysis of Full-Scale Long Term DOT Connection Test Results

The Load (in terms of % Tult lot of UX1500MSE) versus log of Times-to-Rupture for the full-scale long term connection tests on UX1500MSE in the 20°C and 40°C rooms are plotted in Figure 5 as solid and open symbols respectively.

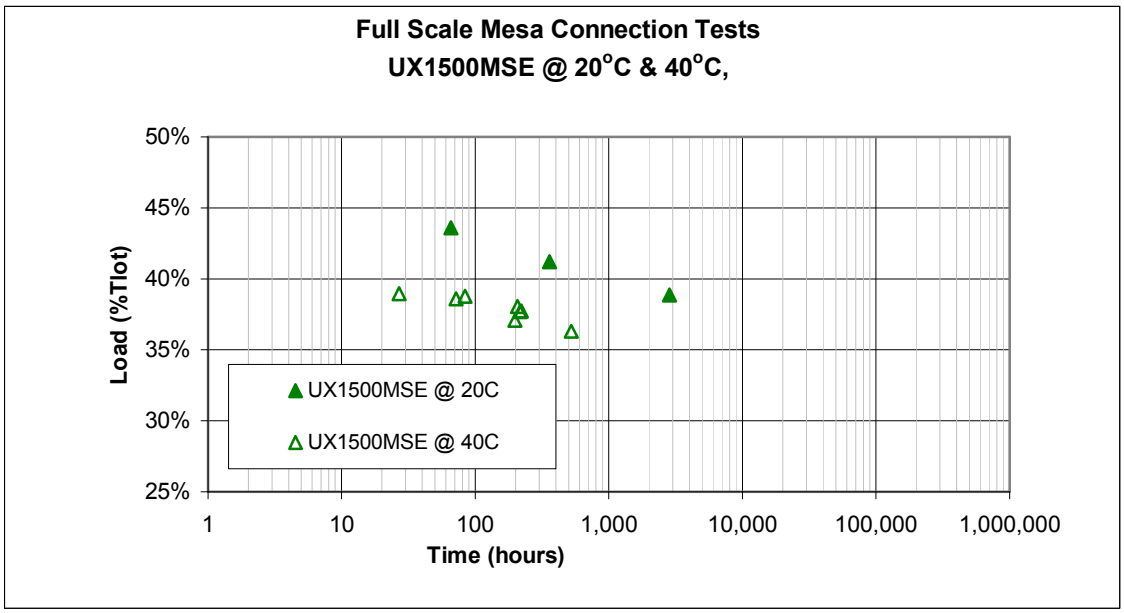


Figure 5: Connection Tests UX1500MSE

The time-temperature shift factor of 155 was applied to the UX1500MSE 40°C time-to-rupture data to estimate the time-to-rupture for similar loads if tested at 20°C. The shifted times are listed in Table 1 and are shown in Figure 6. The good correlation between 20°C and time-shifted 40°C data confirms that there is good agreement between full-scale connection tests and standard stress-rupture tests.

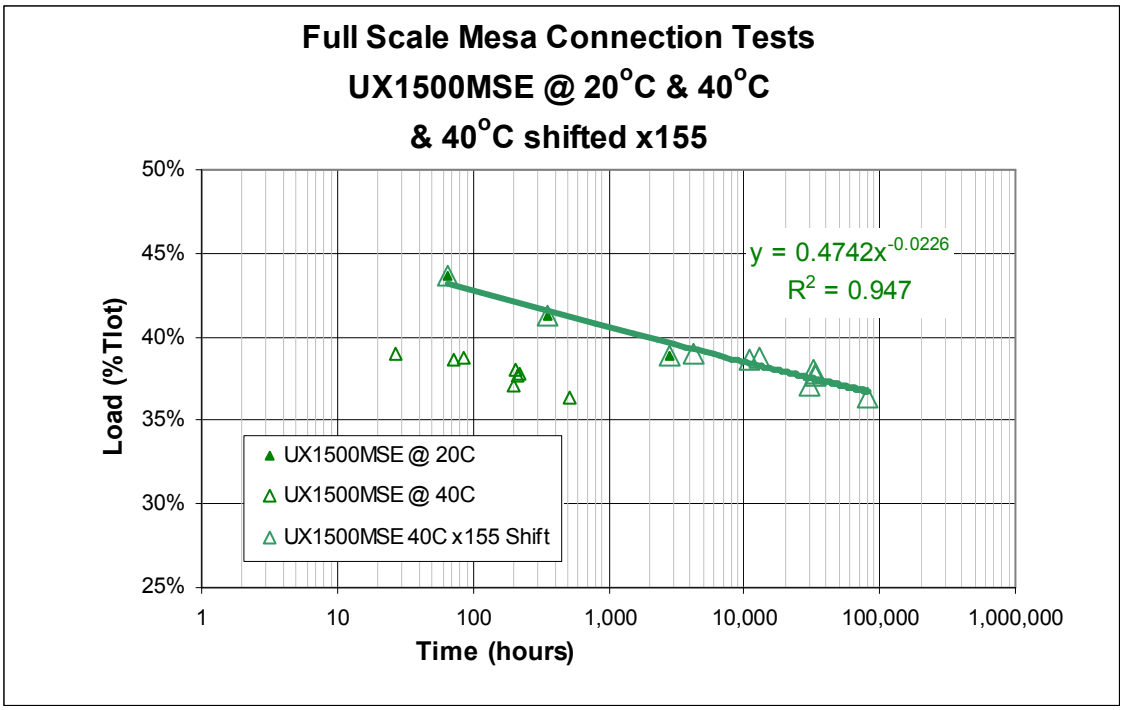


Figure 6: Regression Curve UX1500MSE

The shifted data extends the 20°C data and results in the "best fit" power equation for UX1500MSE shown in the graph and below:

$$T_{CONN} = 0.4742\ t^{-0.0226} \times 100\% \qquad (\%T_{ULT\ LOT})$$

This equation can be used to estimate the long term connection strength as a percentage of the short term, ultimate geogrid lot strength. For a 75-Year design life, **t** = 657,450 hours, T_{CONN} = 35% $T_{ULT\ LOT}$. Therefore, for design the Long Term Strength of the *Connection* is $T_{CONN\ CLS}$ = 35% $T_{ULT\ MARV}$ of the UX1500MSE geogrid.

Based on the standard creep tests on UX****MSE referenced above, the Long Term Strength of the *Geogrid* for a 75-Year design life is about 39.02% of the geogrid $T_{ULT\ LOT}$. Therefore, the long term connection strength of UX1500MSE with the DOT Connector is about 89.7%, (= 35% / 39.02%), of the long term geogrid strength. This is about equal to the percentage of geogrid ribs that are connected to the Mesa facing units. There are 18 to 19 ribs, (18 plus two half ribs), connected to each block versus about 21 ribs per 18 inch block width. Therefore, the percent of connected ribs is between about 85 and 90% of the total number of ribs, about 21, reinforcing the backfill behind an 18" wide Mesa Standard block.

In Figure 7, connection strength data points have been added for UX1600MSE at 20°C, (T = 3787 plf @ t = 6956 hours), and for UX1400MSE at 40°C, (T = 1848 plf @ t = 244 hours and T = 1699 plf @ t = 7682 hours). Assuming the best fit power curves for both geogrids have similar slopes and the same 155 time-temperature shift factor as the UX1500MSE, Creep Reduction Factors, (CRF, or "CR_{CR}" in AASHTO), are calculated to be about 33.5% for UX1600MSE, and 34.2% for UX1400MSE. These are 85.8% and 87.6% of the geogrid $T_{CLS} = 39.02\%T_{ULT\ LOT}$ for these two geogrids, respectively. Both of these percentages are within the same 85 to 90% range of the percentage of connected ribs as for UX1500MSE.

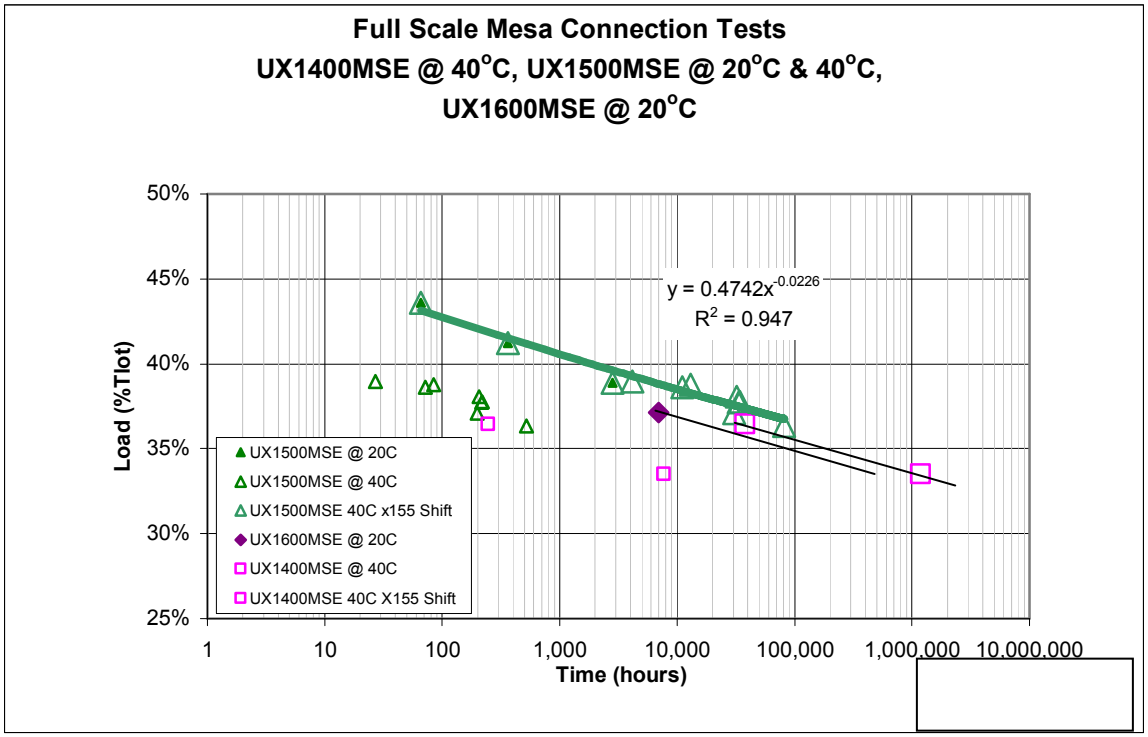


Figure 7: Connection tests UX1400MSE, UX1500MSE & UX1600MSE

CONNECTION STRENTH FOR MSE RETAINING WALL DESIGN

AASHTO long term *geogrid* design strength is calculated using the following formula:

$$T_{AL} = T_{ULT\ MARV} / [RF_{CR(Geogrid)} * RF_{ID} * RF_{D}]$$

where:

T_{AL} = Long Term Allowable Design Strength for Geogrid,
$RF_{CR\ (Geogrid)}$ = Reduction Factor for Creep of the Geogrid,
RF_{ID} = Reduction Factor for Installation Damage of the Geogrid,
RF_{D} = Reduction Factor for Durability of the Geogrid,

The Reduction Factor for Creep is equal to the inverse of the Creep Reduction Factor, CRF. Stress-rupture testing of UX****MSE geogrids determined that $RF_{CR} = 2.56$ for a geogrid 75-year design life. (CRF = 1 / RF_{CR} = 39%). (Note: for MSE retaining wall design, T_{AL} is factored for uncertainties by dividing it by FS_{UN})

The long term *connection* design strength can be calculated with a similar formula using the data from the full-scale connection tests to calculate $RF_{CR\ (Conn)}$, the connection strength RF_{CR}. Where sufficient full-scale long term connection test data is not yet available (e.g. UX1100MSE), RF_{CR} can be estimated using laboratory stress-rupture data (from tests in which the failure mode was stress rupture of the geogrid) and the percentage of geogrid ribs connected to the wall facing units. For DOT connectors and Mesa Standard blocks, the percentage of connected ribs is between 85% to 90%, so $CR_{CR(Conn)}$ can be conservatively taken as (1 / 85%) of $RF_{CR(Geogrid)}$. The long term DOT connection design strength can then be calculated with the following formula:

$$\mathbf{T_{AL\ Conn} = [T_{ULT\ MARV\ (Geogrid)}] / [RF_{CR\ (Conn)} * RF_{D}]}$$

where:

$T_{AL\ Conn}$ = Long Term Allowable Design Strength for DOT Connection
$T_{ULT\ MARV\ (Geogrid)}$ = Long Term Allowable Design Strength for Geogrid
$RF_{CR(Conn)}$ = Reduction Factor for Creep for Connection
RF_{D} = Reduction Factor for Durability for Connection and Geogrid
(*For connection strength, $RF_{ID} = 1.0$ since the connection is not subject to installation damage.)

Alternatively, the long term connection design strength can be calculated using the following formula from the AASHTO LRFD Bridge Design specifications:

$$\mathbf{T_{ac} = (T_{ult}\ x\ CR_{CR}) / RF_{D}}$$

where:

T_{ac} = Long Term Allowable Design Strength for DOT Connection, ($T_{AL\ Conn}$)
T_{ult} = Long Term Allowable Design Strength for Geogrid ($T_{ULT\ MARV\ (Geogrid)}$)
CR_{CR} = Long term strength reduction factor for reduced ultimate strength resulting from the connection, (CRF), [$CR_{CR} = 1 / RF_{CR\ (Conn)}$]
RF_{D} = Reduction Factor for Durability
(Note: $T_{AL\ Conn}$ and T_{ac} are also factored for uncertainties for MSE wall design)

For 75-year design life, the above formula give the Long Term Allowable Connection Strengths (which have not been factored for Uncertainties) shown in the table below. For $T_{AL\ Conn}$, $T_{ULT\ MARV}$ is divided by $RF_{CR\ (Conn)}$ in the third column, alternatively for T_{ac} , $T_{ULT\ MARV}$ is multiplied by $CR_{CR\ (Conn)}$ in the forth column.

Geogrid	Geogrid Ultimate Tensile Strength ASTM D6637 $T_{ULT\ MARV}$ (plf)	Reduction Factor for Creep for Connection (75-Year) $RF_{CR(Conn}$	*Long Term **C**reep **R**eduction **F**actor for Connection* CR_{CR}	Reduction Factor for Durability for Connection RF_D	UX MSE – DOT Connection Long Term Design Strength (75-Year) (before factoring for uncertainties) $T_{AL\ Conn}$ or T_{ac} (plf)
UX1100MSE	3970	3.01 (=2.56/0.85)	*0.33*	1.10	1199
UX1400MSE	4800	3.01 (=2.56/0.85)	*0.33*	1.10	1450
UX1500MSE	7810	3.01 (=2.56/0.85)	*0.33*	1.10	2359
UX1600MSE	9870	3.01 (=2.56/0.85)	*0.33*	1.10	2981
*UX1700MSE	11990	*3.2 (=2.56/0.80)	**0.31*	1.10	*3406

*NOTE: Pending completion of LT connection tests with UX1700MSE, conservatively reduce the percent of connected ribs from 85% to 80% .

Stress-rupture testing of UX****MSE geogrids determined that RF_{CR} = 2.6 for a geogrid 120-year design life. (CRF = 1 / RF_{CR} = 38.5%). For 120-year design life, using the above formula give the Long Term Allowable Connection Strengths (which have not been factored for Uncertainties) shown in the table below.

Geogrid	Geogrid Ultimate Tensile Strength ASTM D6637 $T_{ULT\ MARV}$ (plf)	Reduction Factor for Creep for Connection (120-Year) $RF_{CR(Conn}$	*Long Term **C**reep **R**eduction **F**actor for Connection* CR_{CR}	Reduction Factor for Durability for Connection RF_D	UX MSE – DOT Connection Long Term Design Strength (120-Year) (before factoring for uncertainties) $T_{AL\ Conn}$ or T_{ac} (plf)
UX1100MSE	3970	3.06 (=2.6/0.85)	*0.327*	1.10	1179
UX1400MSE	4800	3.06 (=2.6/0.85)	*0.327*	1.10	1426
UX1500MSE	7810	3.06(=2.6/0.85)	*0.327*	1.10	2320
UX1600MSE	9870	3.06 (=2.6/0.85)	*0.327*	1.10	2932
*UX1700MSE	11990	3.25 (=2.6/0.80)*	*0.307**	1.10	*3354

*NOTE: Pending completion of LT connection tests with UX1700MSE, conservatively reduce the percent of connected ribs from85% to 80%.

CONCLUSIONS

Full-scale long term connection tests using Tensar UX1400MSE, UX1500MSE and UX1600MSE connected to Mesa Standard facing blocks with glass filled polypropylene Mesa "DOT" Connectors show the following:

- The mode of failure at normal laboratory temperatures is stress-rupture of the geogrid ribs.

- For Mesa Standard Block and DOT connectors, the long term connection strength is between 85% and 90% of the long term strength of the geogrid. These measured long term strengths of the DOT connection, taken as a percentage of the long term strength of the geogrid, are essentially equal to the percentage of geogrid ribs connected to the facing block...

- For MSE Retaining Wall Design Methods, $RF_{CR\ Conn}$ = 3.01 to 3.25, or alternatively CR_{CR} = 0.33 to 0.31, for Mesa DOT connectors with Mesa Standard facing blocks.

Laboratory-scale stress-rupture testing on UX1600MSE shows the following:

- The referenced separate study shows laboratory-scale stress-rupture test samples supported by DOT connector teeth perform similarly to samples supported by standard clamps.

- Comparisons of data from laboratory and full scale tests also show that the stress-rupture characteristics of load, time-to-failure and elevated temperature-time shift factor determined in the laboratory-scale stress-rupture tests are similar to those same characteristics determined in tests on geogrids connected to Mesa Standard facing units with Mesa DOT Connectors.

- Given the comparable results from full-scale connection tests and laboratory stress-rupture tests, standard laboratory stress-rupture tests can be used to estimate long term DOT Connection strengths for all Tensar uniaxial HDPE geogrids that have been shown to experience a geogrid stress-rupture failure mode in full-scale tests.

Appendix 1.2.7

Pullout Tests in Soil Report

Tensar UX****MSE Geogrids

MESA® MSE Retaining Wall System

Back to main document

Prepared for:

Tensar International Corporation
2500 Northwinds Pkwy, Suite 500
Alpharetta, GA 30009

FINAL REPORT

PULLOUT TESTING TENSAR UNIAXIAL GEOGRIDS IN GRANULAR BACKFILL SOILS

Prepared by:



SGI TESTING SERVICES, LLC

4405 International Blvd., Suite B-117
Norcross, GA 30093

Project Number SGI22016

25 July 2022

CAVEAT

The reported results apply only to the materials and test conditions used in the laboratory testing program. The results do not necessarily apply to other materials or test conditions. The test results should not be used in engineering analysis unless the test conditions model the anticipated field conditions. The testing was performed in accordance with general engineering testing standards and requirements. This testing report is submitted for the exclusive use of the client to whom it is addressed.

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1. INTRODUCTION

SGI Testing Services, LLC (SGI) conducted a laboratory testing program to evaluate the pullout resistance of Tensar uniaxial geogrids in granular backfill soils . The sample preparation procedures and testing conditions used in the testing program were specified by Mr. Willie Liew, P.E. of Tensar International Corporation. All of the pullout tests were conducted at SGI located in Norcross, Georgia, United States.

2. TEST MATERIALS

Two types of materials were used in this testing program. Descriptions of these materials are given below:

- *Reinforcement Material:* Tensar UX1400MSE, UX1500MSE, and UX1700MSE geogrids.
- *Soil Material*: concrete sand and AASHTO #57 stone. Particle size analysis, compaction, and direct shear testing were conducted on the two soils, and the test results are presented in Appendix A.

Tensar UX1400MSE, UX1500MSE, and UX1700MSE geogrids were provided by Tensar International Corporation. Bulk samples of the two granular backfill soils used in pullout testing were provided by SGI.

3. PULLOUT TEST EQUIPMENT

The pullout testing device used in this testing program had plan dimensions of 2 ft by 5 ft and an overall depth of 12 inch. Normal (vertical) stresses were applied to the testing specimen by using dead weight for low normal stress tests or six hydraulic cylinders for high normal stress tests, and pullout (horizontal) loads were applied to the test specimen through two hydraulic cylinders. The schematic diagrams of the pullout testing device are shown in Figures 1 and 2.

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4. TEST METHOD AND PROCEDURES

The pullout tests were performed in accordance with the ASTM D 6706, "*Measuring Geogrid Pullout Resistance in Soil*". For each pullout test, the test specimen was set up in accordance with the following procedures and tested under the specific conditions as described below:

- For Test Series #1, 2, and 3, concrete sand was placed in the lower half of the pullout box and compacted by hand tamping to form a 6-inch (150-mm) thick layer. The concrete sand was compacted to approximately 95% of the maximum dry unit weight at optimum moisture content (OMC). For Test Series #4, 5, and 6, AASHTO #57 stone was placed in the lower half of the pullout box and tightly compacted by hand tamping to form a 6-inch (150-mm) thick layer under dry conditions;

- For each pullout test, a geogrid specimen was placed on top of the compacted soil as shown in Figures 3 and 4. The front end of the geogrid specimen was connected to the pullout loading harness;

- Three "tell-tale" wire gages were connected at selected locations as shown in Figures 3 and 4. Displacements of the geogrid specimen at these two tell-tale locations were measured by using LVDTs;

- For Test Series #1, 2, and 3, concrete sand was placed in the upper half of the pullout box and compacted by hand tamping to form a 6-inch (150-mm) thick layer. The concrete sand was compacted to approximately 95% of the maximum dry unit weight at optimum moisture content (OMC). For Test Series #4, 5, and 6, AASHTO #57 stone was placed in the upper half of the pullout box and tightly compacted by hand tamping to form a 6-inch (150-mm) thick layer under dry conditions;

- A load cell was then attached to the pullout loading harness to measure the pullout load of the test specimen at the specimen clamp. An LVDT was fixed to

the specimen clamp to measure the total pullout displacement at the specimen clamp.

- A specific normal stress was then applied to the test specimen through dead weight (steel plates) for low normal stress testing or hydraulic cylinders for high normal stress testing; and

- After application of the normal stress, the geogrid specimen was subjected to pullout loading by displacing it at a constant displacement rate of 0.04 inch/min (1 mm/min) as measured at the specimen clamp. The geogrid specimen was pulled until the pullout or tensile failure occurred.

A completed pullout test setup is shown in Figure 5 (normal stress applied by using dead weight) and Figure 6 (normal stress applied by using hydraulic cylinders).

5. PULLOUT TEST RESULTS

Six series of pullout tests were performed in this testing program. For each pullout test series, the test conditions and test results are presented on a summary page in Appendix C. The summary page includes:

- Pullout force versus displacement figure;
- Pullout resistance versus normal stress figure; and
- A table that summarizes test conditions, maximum pullout resistance, coefficient of interaction (C_i), and failure mode.

The maximum pullout resistance is defined as maximum pullout load divided by the initial width of a geogrid test specimen as follows:

$$P_{max} = \frac{F_{max}}{W} \quad (1)$$

where:

P_{max}	=	maximum pullout resistance (lbs/ft);
F_{max}	=	maximum pullout load (lbs); and
W	=	initial width of the geogrid specimen (ft).

The coefficient of interaction (C_i) was calculated using the equation as follows:

$$C_i = \frac{F_{\max}}{2(L_e W)(\sigma_n \tan\phi + c)} \quad (2)$$

where:

F_{max}	=	maximum pullout load;
L_e	=	initial embedded length of the geogrid specimen;
W	=	initial width of the geogrid specimen;
σ_n	=	total normal stress applied to the geogrid specimen;
ϕ	=	residual total-stress friction angle of soil; and
c	=	residual total-stress cohesion of soil.

The residual shear strength parameters (ϕ and c) of the two granular backfill soils were determined from direct shear testing and presented in Appendix A.

For each pullout test, the geogrid specimen was pulled until the pullout or tensile failure occurred. The geogrid specimen was considered to be in the pullout failure mode when the tell-tale gage attached to the rear end of the geogrid specimen displaced at least 0.5 inch at the completion of the test and pullout load decreased or remained constant with increasing pullout displacement.

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6. THE SCALE EFFECT CORRECTION FACTOR

At the request of Tensar, the scale effect correction factor (α) was calculated in accordance with the procedures given in Appendix A (*Determination of Pullout Resistance Factors) of U.S. Department of Transportation Publication No. FHWA-NHI-00-043 "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes"*.

For Test Series #1, 2, and 3 (UX geogrids in concrete sand), the scale effect correction factor (α) was calculated for each test in which the geogrid was pulled out (no rupture). The calculated α factor value is in the range of 0.96 to 1.00. Details of calculation and results are presented in Appendix C. For Test #1C at 800 psf, the UX1400MSE geogrid failed in rupture, and the scale effect correction factor was not calculated.

For Test Series #4, 5, and 6 (UX geogrids in AASHTO # 57 stone), the scale effect correction factor (α) was calculated for each test in which the geogrid was pulled out (no rupture). The calculated α factor value is in the range of 0.95 to 1.00. Details of calculation and results are presented in Appendix C. For Test #4C at 400 psf and 5C at 800 psf, the UX1400MSE and UX1500MSE geogrids failed in rupture, and the scale effect correction factors were not calculated.

It should be noted that the in-soil displacement measurement is influenced by the soil confining the geogrid test specimen regardless how accurate a measurement device itself is. Caution should be exercised in using the reported in-soil reinforcement for engineering analyses.

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7. CLOSURE

SGI appreciates the opportunity to provide laboratory testing services to Tensar International Corporation. Should you have any questions regarding this report, or if you require additional information, please do not hesitate to contact the undersigned.

Sincerely,



Zehong Yuan, Ph.D., P.E.
Laboratory Manager

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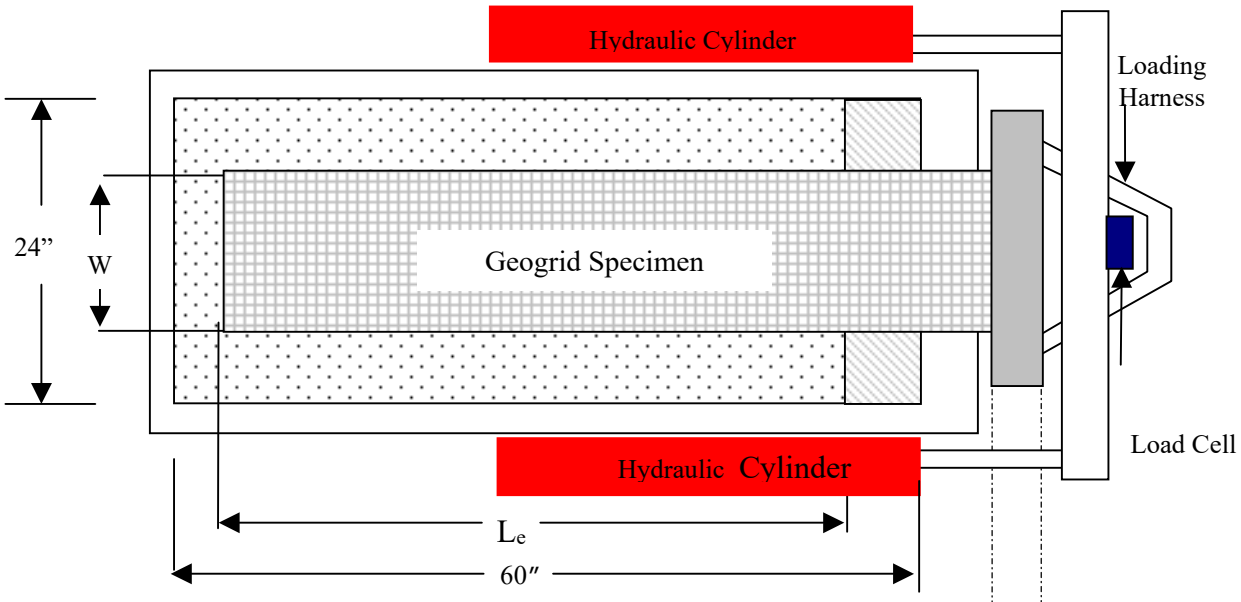


Figure 1. Plan view of pullout test setup (Note: not to scale).

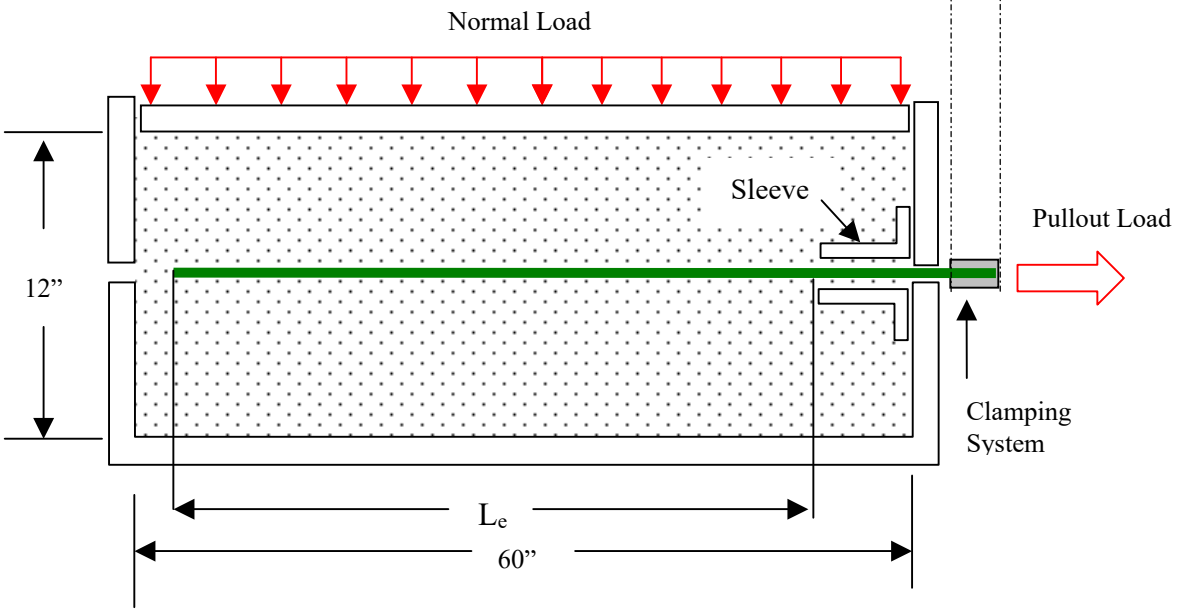


Figure 2. Cross-section of pullout test setup (Note: not to scale).

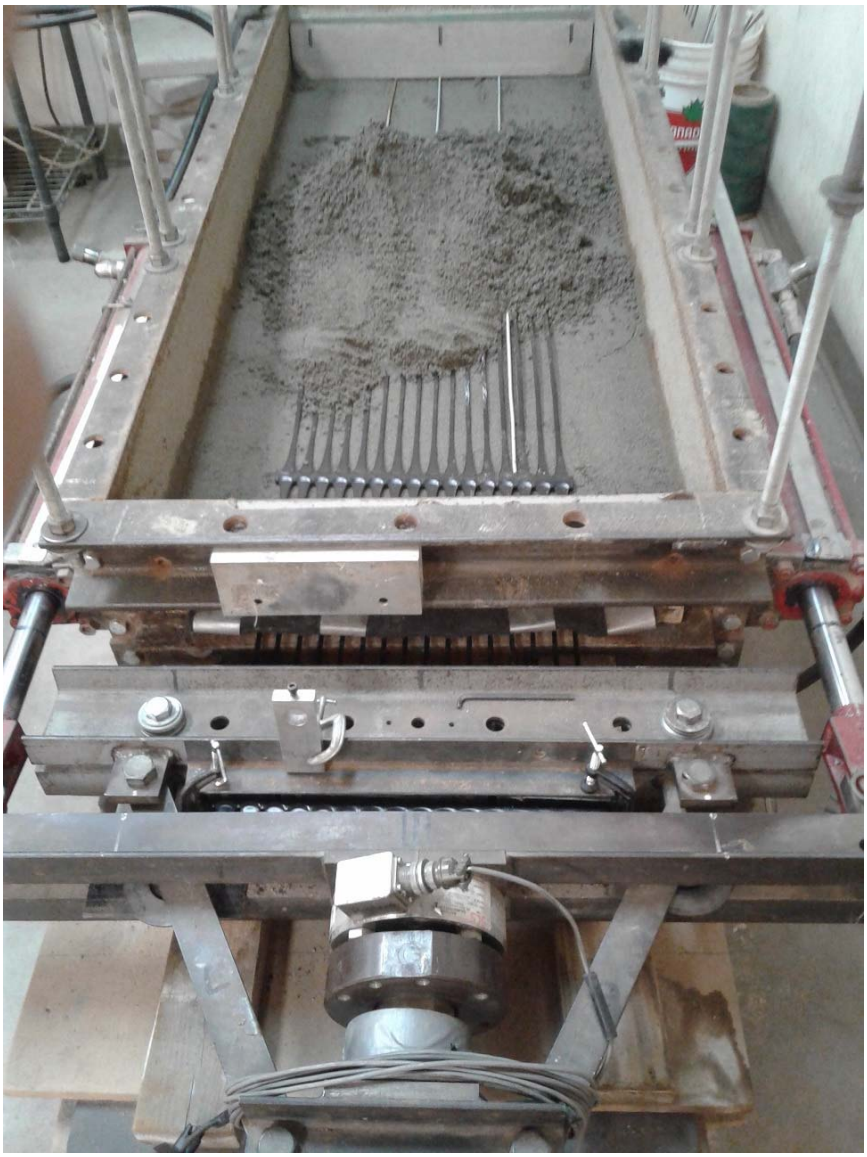


Figure 3. UX1700HS geogrid placed on compacted concrete sand.

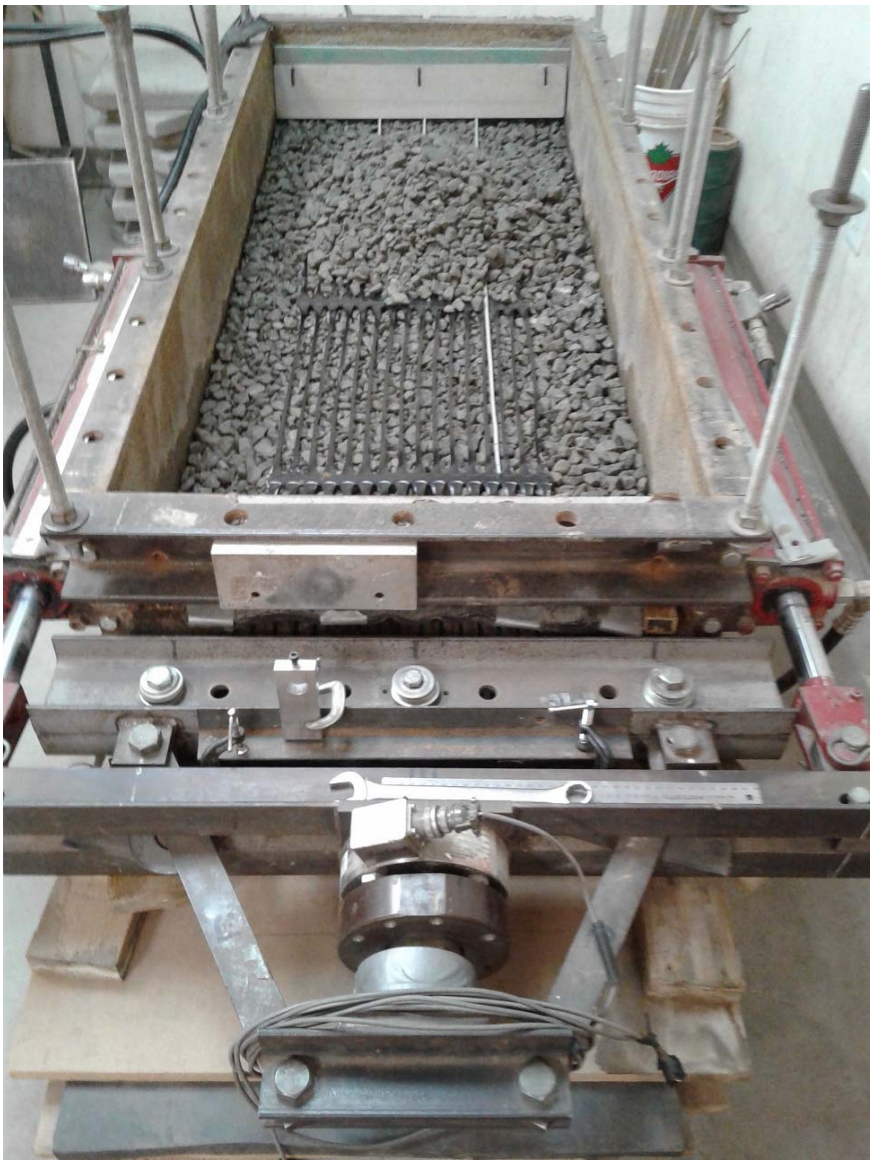


Figure 4. UX1700HS geogrid placed on compacted AASHTO #57 stone.



Figure 5. A completed setup of pullout test at a normal stress of 200 psf by dead weight.

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Figure 6. A completed setup of pullout test at high normal stress by six hydraulic cylinders.

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Figure 7. Three "tell-tale" wire gages (three LVDTs connected to wires) to measure in soil-displacements at three selected locations along the geogrid test specimen.

APPENDIX A

SUMMARY OF
SOIL PROPERTY TEST RESULTS

SGI Testing Services, LLC

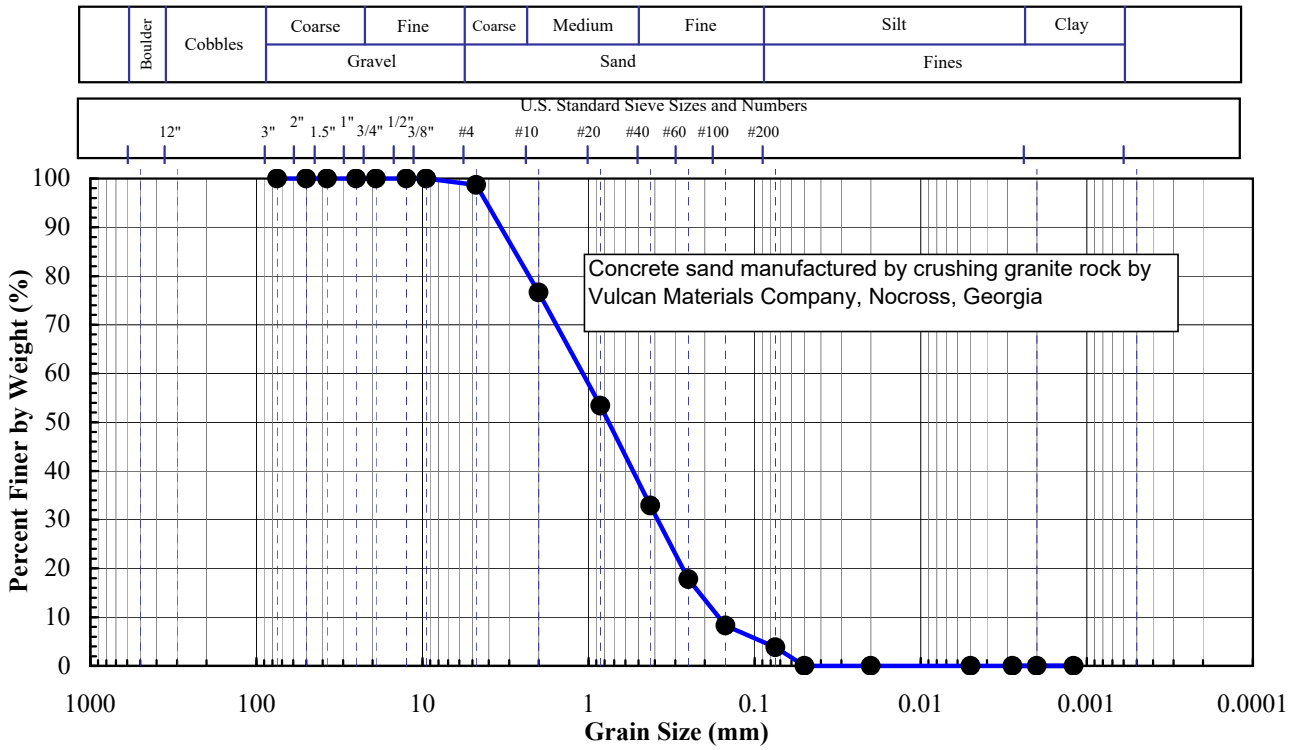
4405 International Blvd., Suite B-117, Norcross, GA 30093
Ph: (770) 931 8222 Fax: (770) 931 8240

Project Name:	Pullout
Project No:	SGI22016
Client Sample ID:	Concrete Sand
Lab Sample No:	SGISP

ASTM D 421, D 422, D 4318

SOIL INDEX PROPERTIES

Moisture Content, Grain Size, Atterberg Limits, Classification

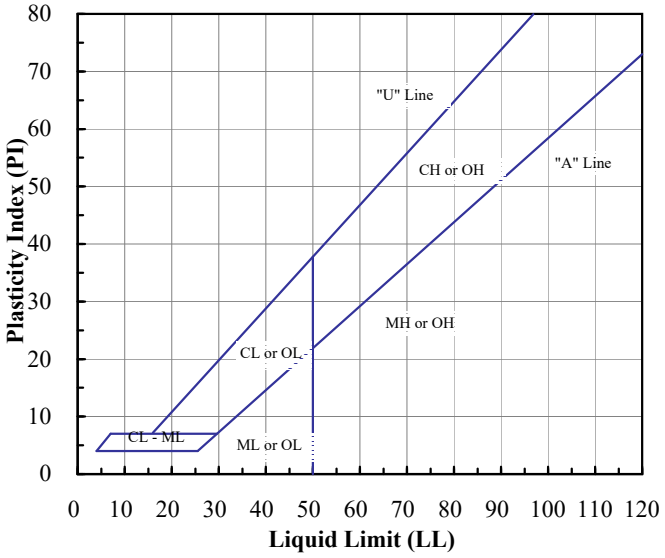


Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
1/2"	12.5	100.0
3/8"	9.5	100.0
#4	4.75	98.7
#10	2.00	76.6
#20	0.850	53.4
#40	0.425	32.9
#60	0.250	17.8
#100	0.150	8.3
#200	0.075	3.8

Hydrometer Particle Diameter (mm)	% Finer
0.0500	
0.0200	
0.0050	
0.0020	
0.0012	

Gravel (%):	1.3
Sand (%):	94.9
Fines (%):	3.8
Silt (%):	
Clay (%):	

Coeff. Unif. (Cu):	6.6
Coeff. Curv. (Cc):	0.83



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits LL (%)	Atterberg Limits PL (%)	Atterberg Limits PI (-)	Engineering Classification
Concrete Sand	SGISP	-	3.8				SP (Poorly-Graded Sand)

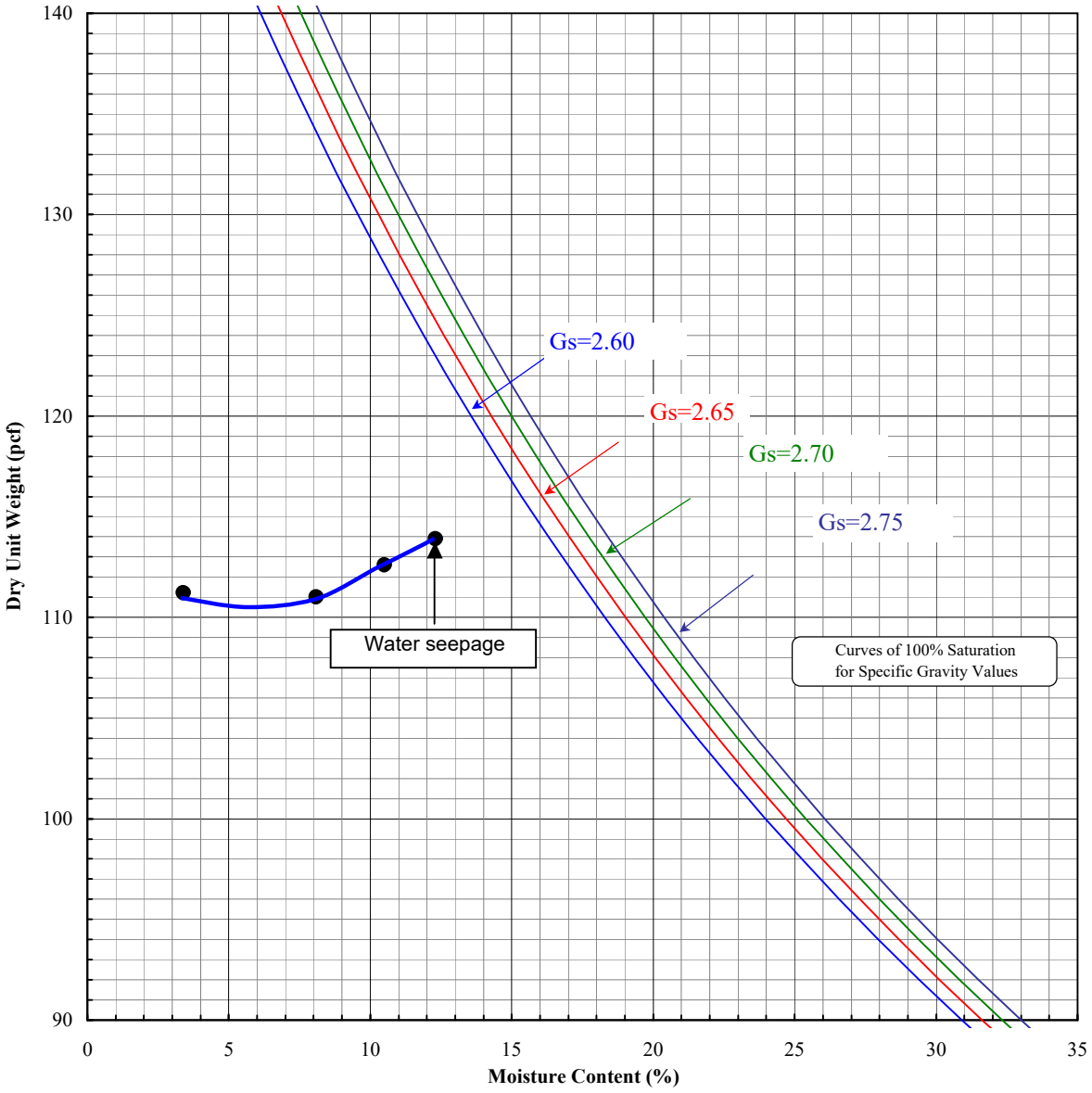
Note(s):

SGI Testing Services, LLC

4405 International Blvd., Suite B-117, Norcross, GA 30093
Ph: (770) 931 8222 Fax: (770) 931 8240

Project Name:	Pullout
Project No:	SGI22016
Client Sample ID:	Concrete Sand
Lab Sample No:	SGISP

COMPACTION MOISTURE-DENSITY RELATIONSHIP (ASTM D 698 - Method A)

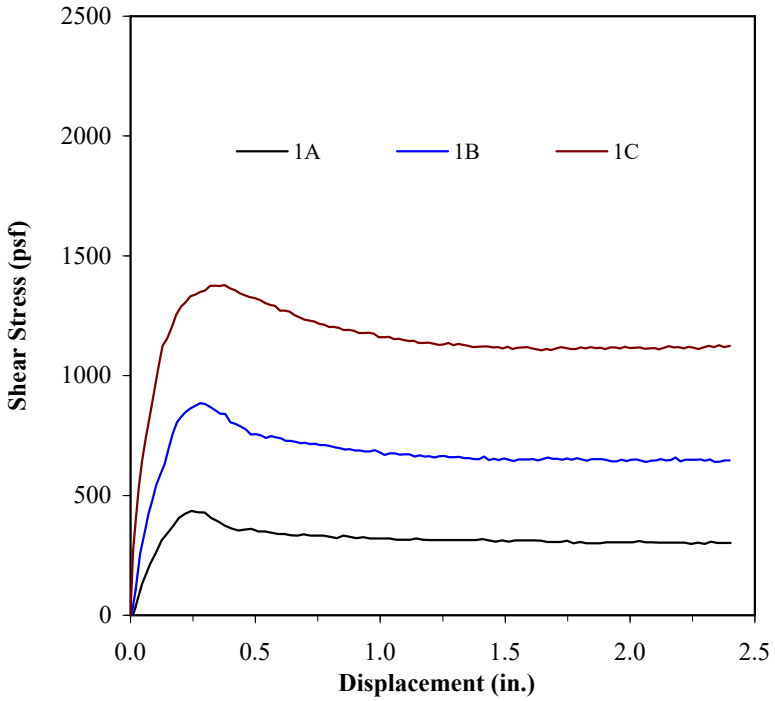


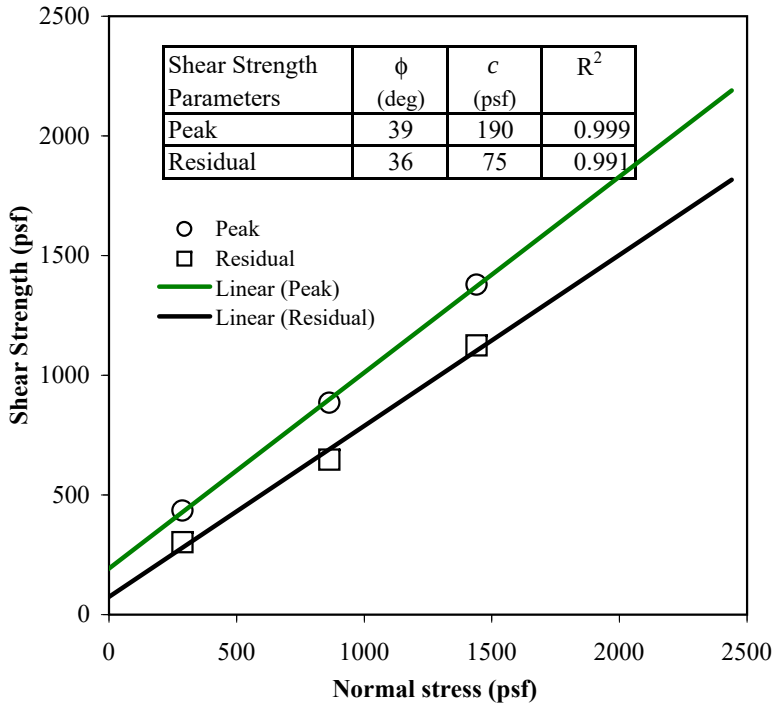
Client Sample ID.	SGI Sample ID NO.	Compaction Data		Maximum Dry Unit Weight and Optimum Moisture Content			
				Measured		Corrected	
		ω_i (%)	γ_d (pcf)	ω_{om} (%)	γ_{dmax} (pcf)	ω_{om} (%)	γ_{dmax} (pcf)
Concrete Sand	SGISP	3.4	111.2	12.0	114.0		
		8.1	111.0				
		10.5	112.6				
		12.3	113.9				

% Retained on #4 Sieve: 4.1
% Retained on #3/8" Sieve:
% Retained on 3/4" Sieve:
% Retained on 2.0" Sieve:

TENSAR INTERNATIONAL CORPORATION
DIRECT SHEAR TESTING (ASTM D 3080)

Internal shear strength of concrete sand compacted to approximately 95% of maximum standard Proctor dry density at OMC
(γ_{dmax} = 114 pcf, OMC = 12%)





Test No.	Shear Box Size (in. x in.)	Normal Stress (psf)	Shear Rate (in./min)	Soaking		Consolidation		Initial			After			Shear Strength		Failure Mode
				Stress (psf)	Time (hour)	Stress (psf)	Time (hour)	γ_d (pcf)	ω_i (%)	ω_f (%)	γ_d (pcf)	ω_i (%)	ω_f (%)	τ_P (psf)	τ_R (psf)	
1A	12 x 12	288	0.04	-	-	288	1.0	108.5	11.8					435	302	(1)
1B	12 x 12	864	0.04	-	-	864	1.0	109.2	11.1					885	647	(1)
1C	12 x 12	1440	0.04	-	-	1440	1.0	108.8	11.5					1378	1125	(1)

NOTES:
(1) Sliding (i.e., shear failure) was forced to occur along the shear plane between the upper and lower shear box.
(2) The reported total-stress parameters of friction angle and cohesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The residual shear strength was calculated using the shear force measured at the end of the test.

DATE OF REPORT: 4/13/2022

SGI TESTING SERVICES, LLC

FIGURE NO.	A-3
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

SGI Testing Services, LLC

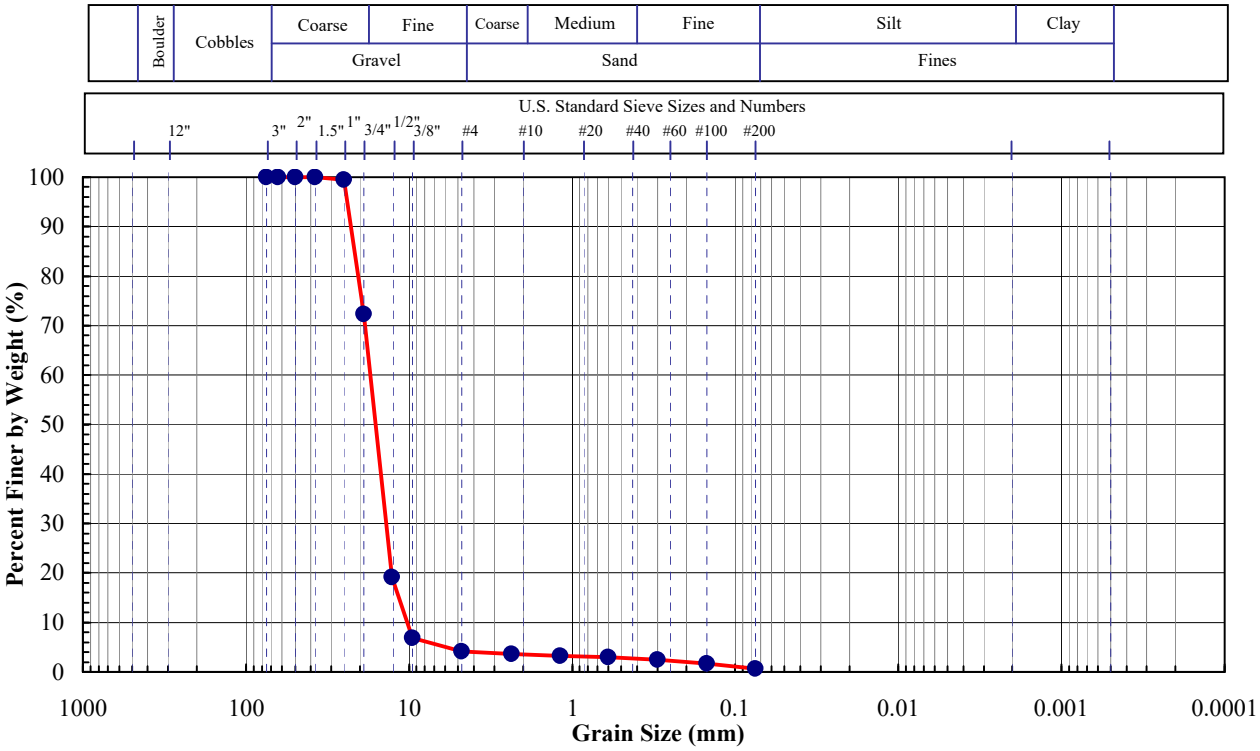
4405 International Blvd., Suite B-117, Norcross, GA 30093
Ph: (770) 931-8222 Fax: (770) 931-8240

Project Name:	Pullout
Project No:	SGI22016
Client Sample ID:	AASHTO #57 Stone
Lab Sample No:	SGIGP

ASTM D 2216, D 1140, D 422, C 136, D 4318, D 2487

SOIL INDEX PROPERTIES

Moisture Content, Grain Size, Atterberg Limits, Classification

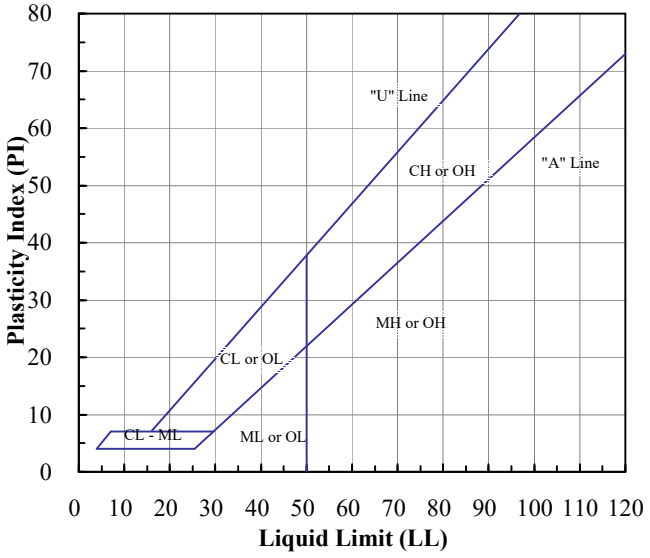


Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	99.5
3/4"	19	72.3
1/2"	12.5	19.2
3/8"	9.5	6.8
#4	4.75	4.1
#8	2.36	3.6
#16	1.180	3.2
#30	0.600	3.0
#50	0.300	2.5
#100	0.150	1.7
#200	0.075	0.7

Hydrometer Particle Diameter (mm)	% Finer
0.0340	
0.0220	
0.0064	
0.0032	
0.0013	

Gravel (%):	95.9
Sand (%):	3.4
Fines (%):	0.7
Silt (%):	
Clay (%):	

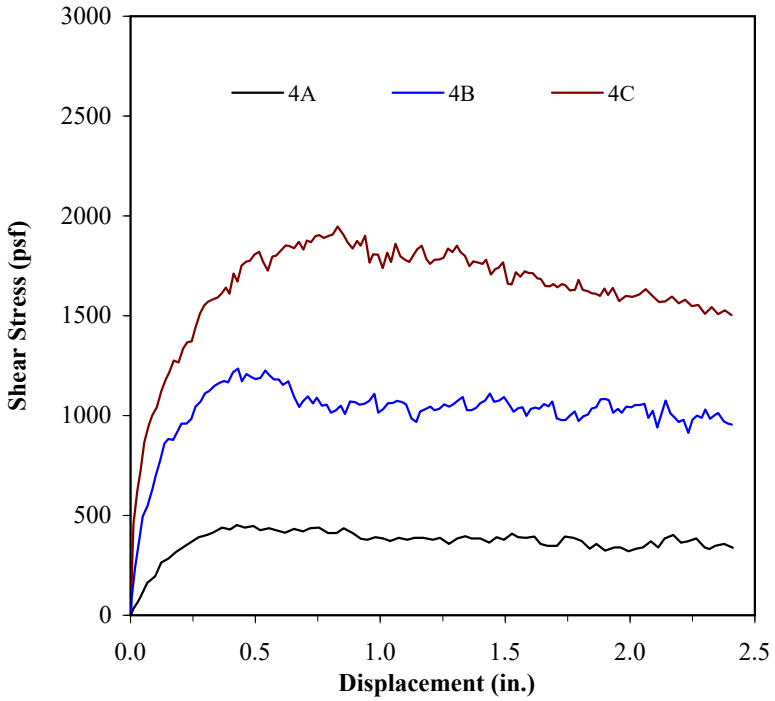
Coeff. Unif. (Cu):	1.7
Coeff. Curv. (Cc):	1.1

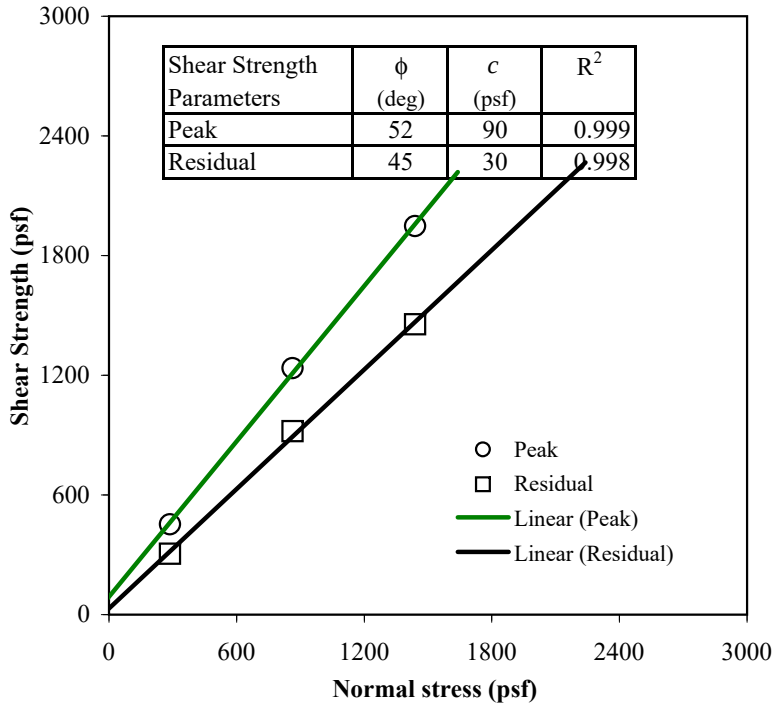


Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits LL (%)	Atterberg Limits PL (%)	Atterberg Limits PI (-)	Engineering Classification
AASHTO #57 Stone	SGIGP		0.7				GP

TENSAR INTERNATIONAL CORPORATION
DIRECT SHEAR TESTING (ASTM D 3080)

Internal shear strength of AASHTO #57 stone tightly compacted





Test No.	Shear Box Size (in. x in.)	Normal Stress (psf)	Shear Rate (in./min)	Soaking		Consolidation		Initial			After			Shear Strength		Failure Mode
				Stress (psf)	Time (hour)	Stress (psf)	Time (hour)	γ_d (pcf)	ω_i (%)	ω_f (%)	γ_d (pcf)	ω_i (%)	ω_f (%)	τ_P (psf)	τ_R (psf)	
4A	12 x 12	288	0.04	-	-	288	1.0							452	305	(1)
4B	12 x 12	864	0.04	-	-	864	1.0							1235	920	(1)
4C	12 x 12	1440	0.04	-	-	1440	1.0							1947	1455	(1)

NOTES:

(1) Sliding (i.e., shear failure) was forced to occur along the shear plane between the upper and lower shear box.

(2) The reported total-stress parameters of friction angle and cohesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The residual shear strength was calculated using the shear force measured at the end of the test.

DATE OF REPORT: 4/13/2022

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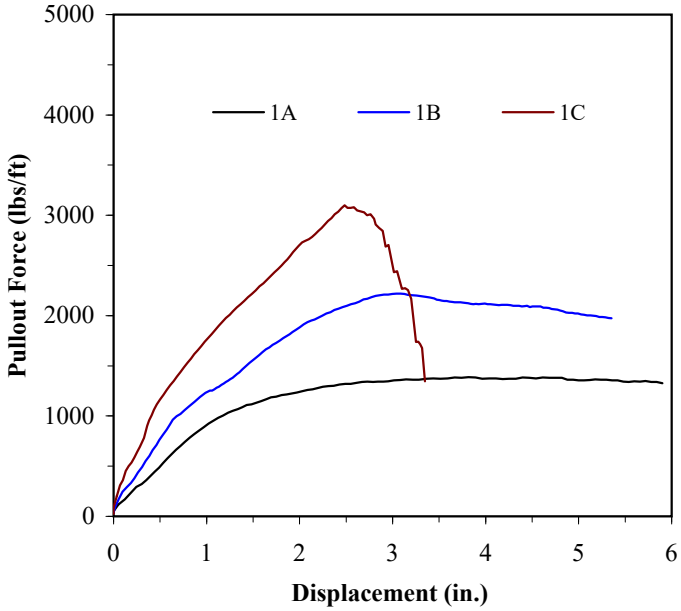
FIGURE NO.	A-5
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

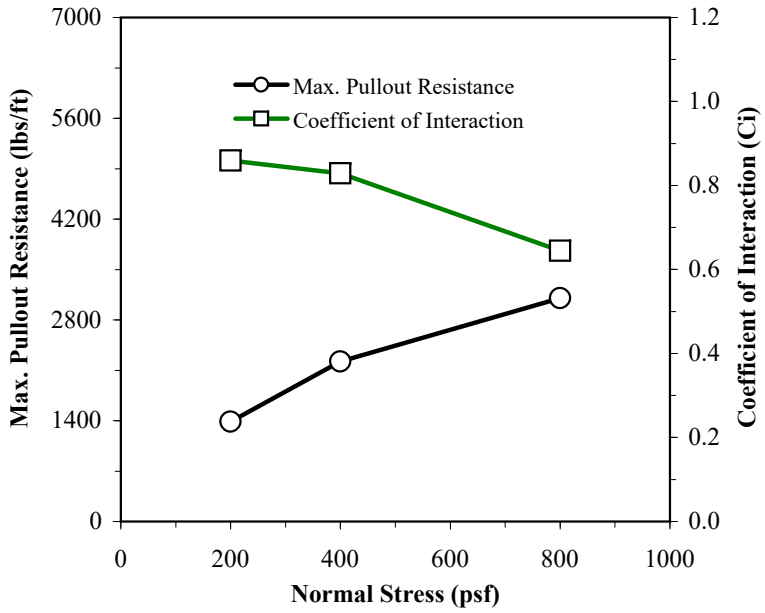
APPENDIX B

SUMMARY OF
PULLOUT TEST RESULTS

TENSAR INTERNATIONAL CORPORATION
GEOSYNTHETIC PULLOUT TESTING (ASTM D 6706)

TEST SERIES NO. 1: Tensar UX1400MSE geogrid (Lot #215080) in MD within concrete sand compacted to approximately 95% of max standard Proctor density at optimum moisture content (OMC) +/- 2%





Test No.	Test Specimen Width W (in.)	Embedment Length L (in.)	Normal Stress σ_n (psf)	Pullout Rate (in./min)	Soil Compaction: Initial Dry Unit Weight (pcf)	Soil Compaction: Initial Moisture Content (%)	Residual Soil Shear Strength Parameters: ϕ (degree)	Residual Soil Shear Strength Parameters: c (psf)	Max. Pullout Resistance P_{max} (lbs/ft)	Coefficient of Interaction $C_i = \frac{P_{max}}{2L(\sigma_n \tan\phi + c)}$ (-)	Failure Mode[1]
1A	15.0	44.0	200	0.04	108.8	11.5	36	75	1388	0.86	Pullout
1B	15.0	44.0	400	0.04	108.1	12.2	36	75	2223	0.83	Pullout
1C	15.0	44.0	800	0.04	108.4	11.9	36	75	3101	0.64	Rupture

NOTE:

(1) Failure Mode: pullout failure indicated by a minimum displacement of 0.5" (13 mm) measured at the rear end of the geogrid specimen at the completion of the test.

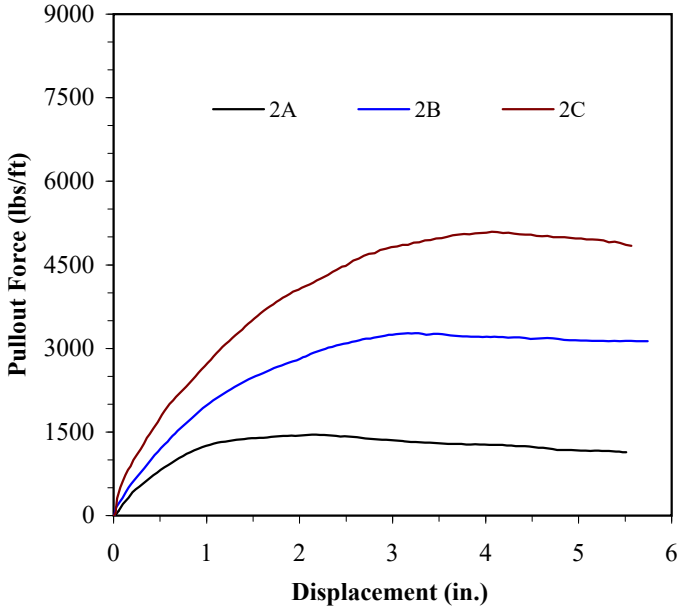
DATE REPORTED: 6/21/2022

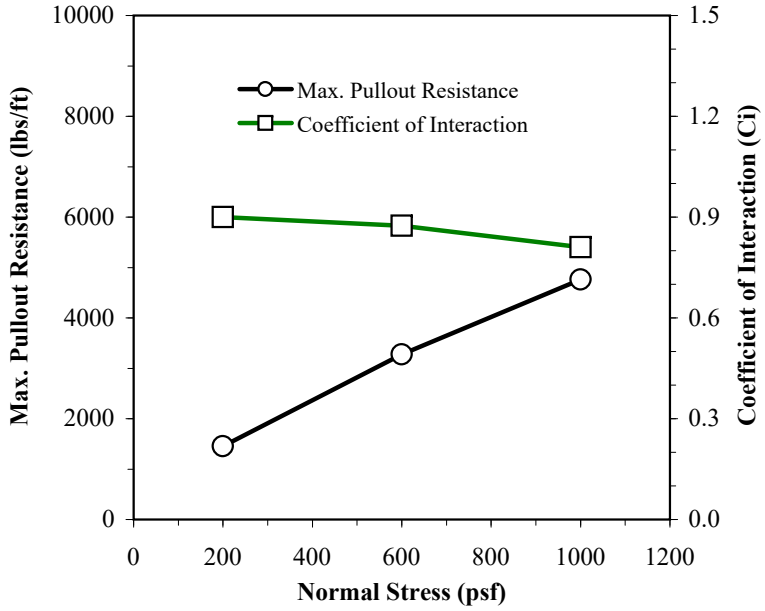
SGI TESTING SERVICES, LLC

FIGURE NO.	1
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

TENSAR INTERNATIONAL CORPORATION
GEOSYNTHETIC PULLOUT TESTING (ASTM D 6706)

TEST SERIES NO. 2: Tensar UX1500MSE geogrid (Lot #215133) in MD within concrete sand compacted to approximately 95% of max standard Proctor density at optimum moisture content (OMC) +/- 2%





Test No.	Test Specimen Width W (in.)	Embedment Length L (in.)	Normal Stress σ_n (psf)	Pullout Rate (in./min)	Soil Compaction: Initial Dry Unit Weight (pcf)	Soil Compaction: Initial Moisture Content (%)	Residual Soil Shear Strength Parameters: ϕ (degree)	Residual Soil Shear Strength Parameters: c (psf)	Max. Pullout Resistance P_{max} (lbs/ft)	Coefficient of Interaction $C_i = \frac{P_{max}}{2L(\sigma_n \tan\phi + c)}$ (-)	Failure Mode[1]
2A	15.0	44.0	200	0.04	108.7	11.6	36	75	1454	0.90	Pullout
2B	15.0	44.0	600	0.04	109.2	11.1	36	75	3275	0.87	Pullout
2C	15.0	44.0	1000	0.04	107.7	12.6	36	75	4764	0.81	Pullout

NOTE:

(1) Failure Mode: pullout failure indicated by a minimum displacement of 0.5" (13 mm) measured at the rear end of the geogrid specimen at the completion of the test.

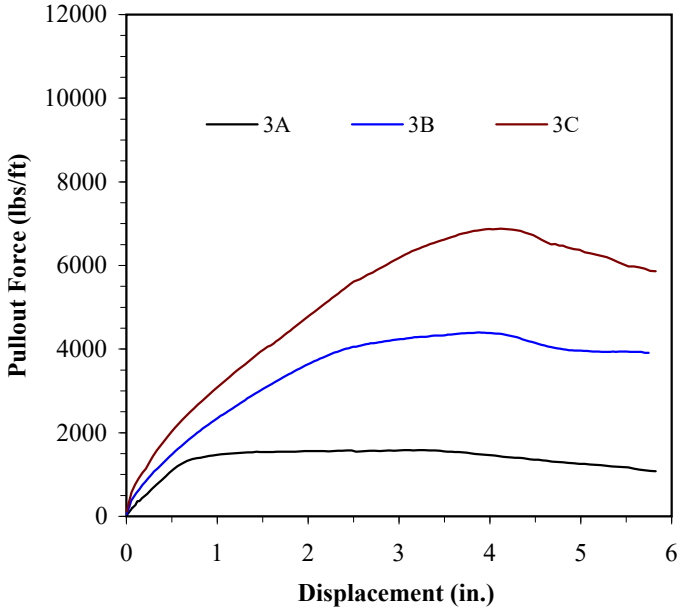
DATE REPORTED: 6/21/2022

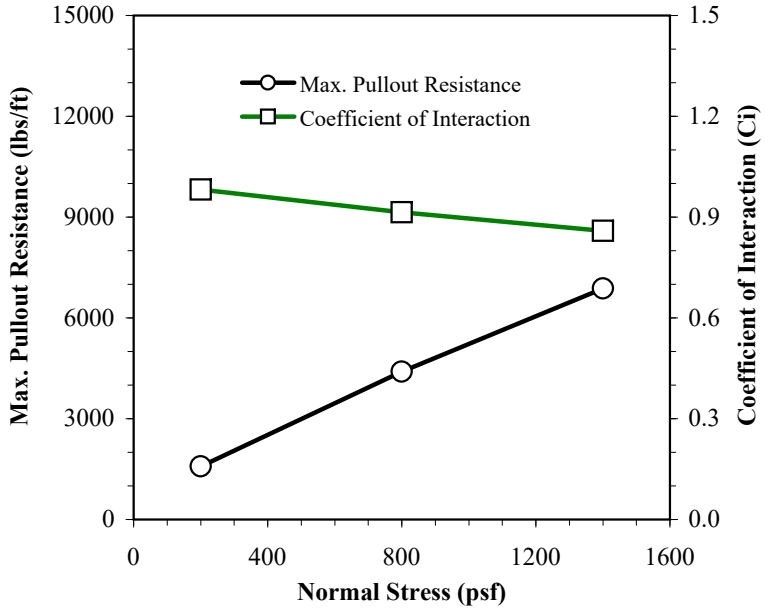
SGI TESTING SERVICES, LLC

FIGURE NO.	2
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

TENSAR INTERNATIONAL CORPORATION
GEOSYNTHETIC PULLOUT TESTING (ASTM D 6706)

TEST SERIES NO. 3: Tensar UX1700MSE geogrid (Lot #317965) in MD within concrete sand compacted to approximately 95% of max standard Proctor density at optimum moisture content (OMC) +/- 2%





Test No.	Test Specimen Width W (in.)	Embedment Length L (in.)	Normal Stress σ_n (psf)	Pullout Rate (in./min)	Soil Compaction: Initial Dry Unit Weight (pcf)	Soil Compaction: Initial Moisture Content (%)	Residual Soil Shear Strength Parameters: ϕ (degree)	Residual Soil Shear Strength Parameters: c (psf)	Max. Pullout Resistance P_{max} (lbs/ft)	Coefficient of Interaction[1] $C_i = \frac{P_{max}}{2L(\sigma_n \tan\phi + c)}$ (-)	Failure Mode
3A	15.0	44.0	200	0.04	104.3	11.2	36	75	1586	0.98	Pullout
3B	15.0	44.0	800	0.04	102.9	12.7	36	75	4400	0.91	Pullout
3C	15.0	44.0	1400	0.04	104.1	11.4	36	75	6880	0.86	Pullout

NOTE:

(1) Failure Mode: pullout failure indicated by a minimum displacement of 0.5" (13 mm) measured at the rear end of the geogrid specimen at the completion of the test.

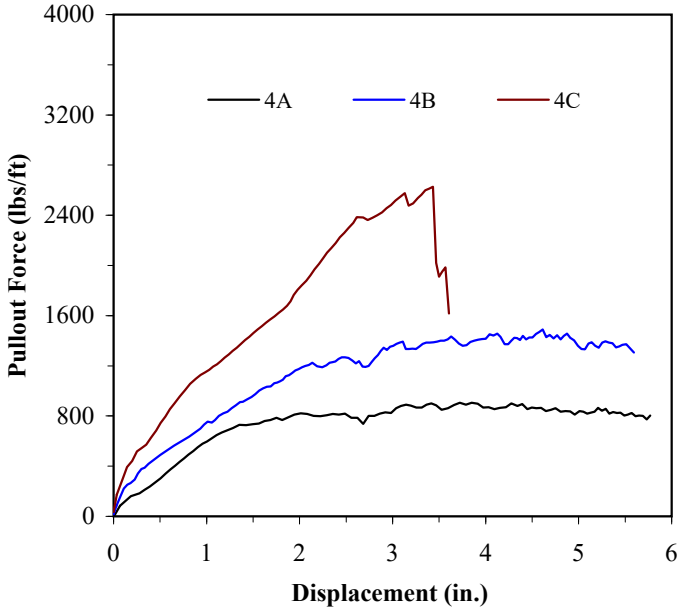
DATE REPORTED: 6/21/2022

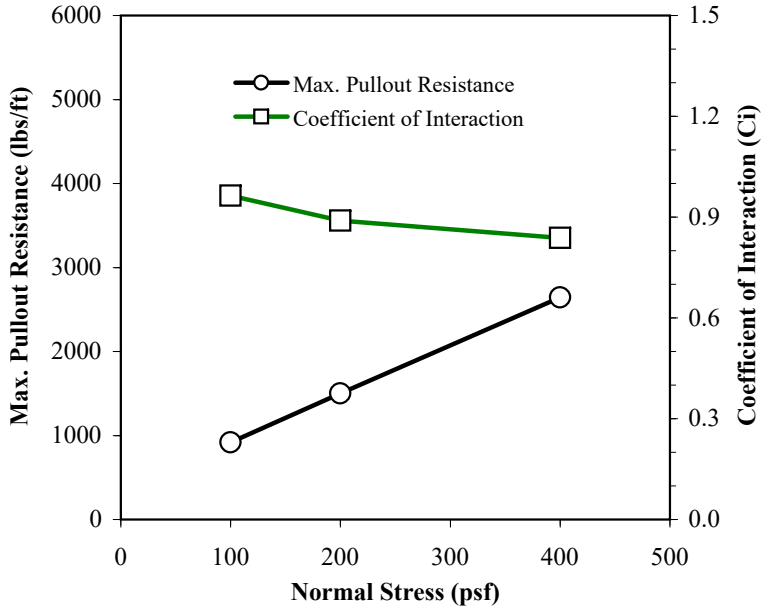
SGI TESTING SERVICES, LLC

FIGURE NO.	3
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

TENSAR INTERNATIONAL CORPORATION
GEOSYNTHETIC PULLOUT TESTING (ASTM D 6706)

TEST SERIES NO. 4: Tensar UX1400MSE geogrid (Lot #215080) in MD within AASHTO #57 stone tightly compacted





Test No.	Test Specimen Width W (in.)	Embedment Length L (in.)	Normal Stress σ_n (psf)	Pullout Rate (in./min)	Soil Compaction: Initial Dry Unit Weight (pcf)	Soil Compaction: Initial Moisture Content (%)	Residual Soil Shear Strength Parameters: ϕ (degree)	Residual Soil Shear Strength Parameters: c (psf)	Max. Pullout Resistance P_{max} (lbs/ft)	Coefficient of Interaction $C_i = \frac{P_{max}}{2L(\sigma_n \tan\phi + c)}$ (-)	Failure Mode[1]
4A	15.0	44.0	100	0.04			45	30	918	0.96	Pullout
4B	15.0	44.0	200	0.04			45	30	1500	0.89	Pullout
4C	15.0	44.0	400	0.04			45	30	2644	0.84	Rupture

NOTE:

(1) Failure Mode: pullout failure indicated by a minimum displacement of 0.5" (13 mm) measured at the rear end of the geogrid specimen at the completion of the test.

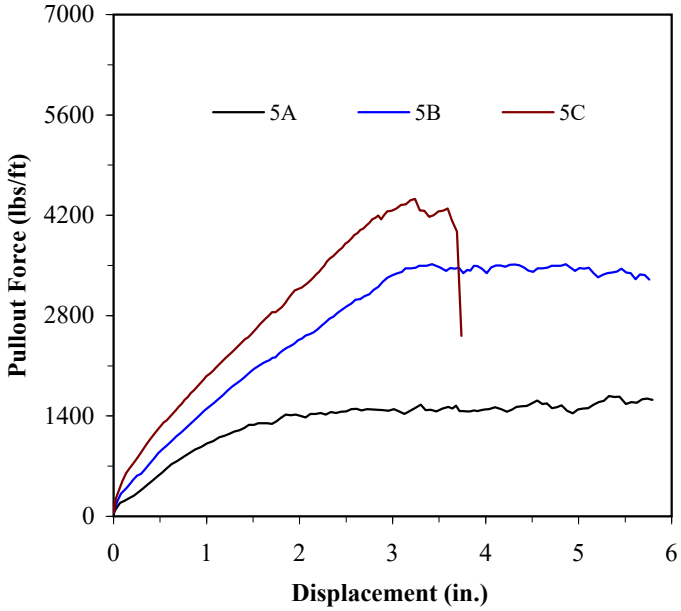
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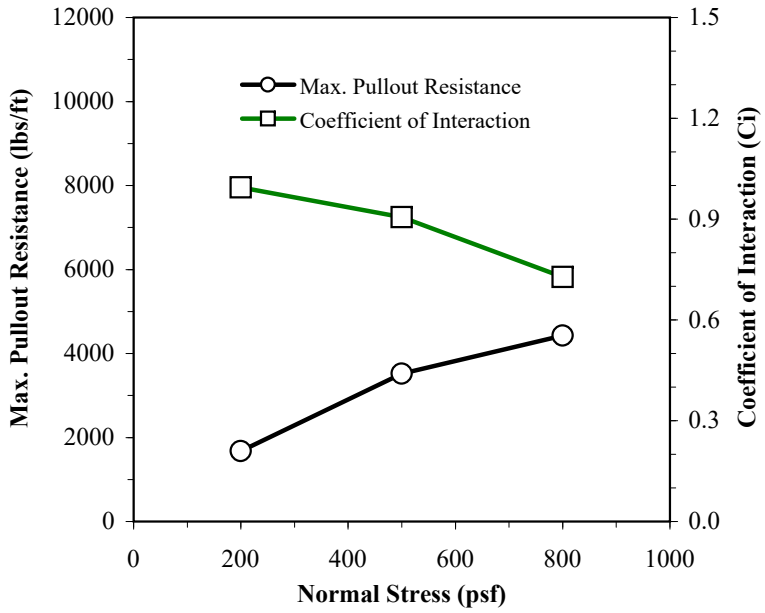
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FIGURE NO.	4
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

TENSAR INTERNATIONAL CORPORATION
GEOSYNTHETIC PULLOUT TESTING (ASTM D 6706)

TEST SERIES NO. 5: Tensar UX1500MSE geogrid (Lot #215133) in MD within AASHTO #57 stone tightly compacted





Test No.	Test Specimen Width W (in.)	Embedment Length L (in.)	Normal Stress σ_n (psf)	Pullout Rate (in./min)	Soil Compaction: Initial Dry Unit Weight (pcf)	Soil Compaction: Initial Moisture Content (%)	Residual Soil Shear Strength Parameters: ϕ (degree)	Residual Soil Shear Strength Parameters: c (psf)	Max. Pullout Resistance P_{max} (lbs/ft)	Coefficient of Interaction $C_i = \frac{P_{max}}{2L(\sigma_n \tan\phi + c)}$ (-)	Failure Mode[1]
5A	15.0	44.0	200	0.04			45	30	1677	0.99	Pullout
5B	15.0	44.0	500	0.04			45	30	3520	0.91	Pullout
5C	15.0	44.0	800	0.04			45	30	4429	0.73	Rupture

NOTE:
(1) Failure Mode: pullout failure indicated by a minimum displacement of 0.5" (13 mm) measured at the rear end of the geogrid specimen at the completion of the test.

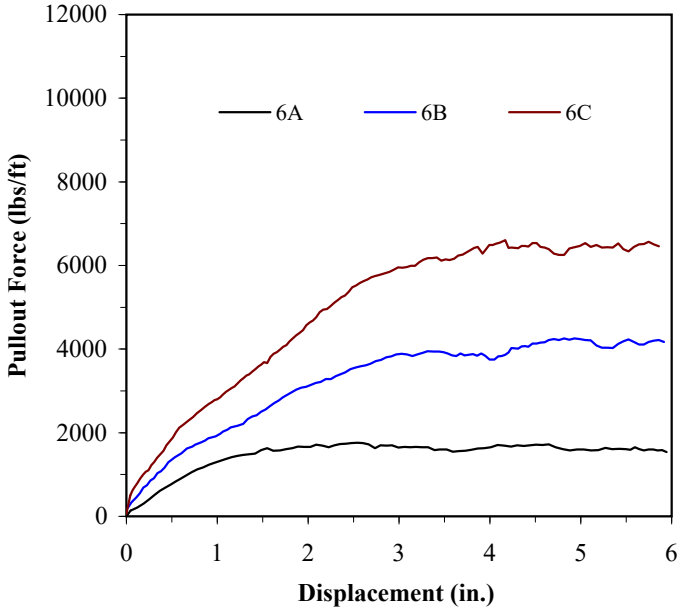
DATE REPORTED: 6/21/2022

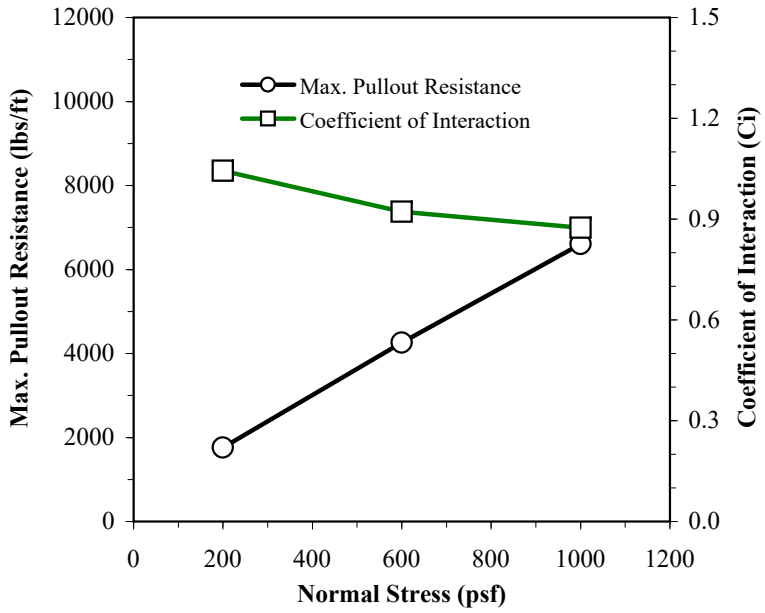
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FIGURE NO.	5
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

TENSAR INTERNATIONAL CORPORATION
GEOSYNTHETIC PULLOUT TESTING (ASTM D 6706)

TEST SERIES NO. 6: Tensar UX1700MSE geogrid (Lot #317965) in MD within AASHTO #57 stone tightly compacted





Test No.	Test Specimen Width W (in.)	Embedment Length L (in.)	Normal Stress σ_n (psf)	Pullout Rate (in./min)	Soil Compaction: Initial Dry Unit Weight (pcf)	Soil Compaction: Initial Moisture Content (%)	Residual Soil Shear Strength Parameters: ϕ (degree)	Residual Soil Shear Strength Parameters: c (psf)	Max. Pullout Resistance P_{max} (lbs/ft)	Coefficient of Interaction $C_i = \frac{P_{max}}{2L(\sigma_n \tan\phi + c)}$ (-)	Failure Mode[1]
6A	15.0	44.0	200	0.04			45	30	1760	1.04	Pullout
6B	15.0	44.0	600	0.04			45	30	4257	0.92	Pullout
6C	15.0	44.0	1000	0.04			45	30	6604	0.87	Pullout

NOTE:

(1) Failure Mode: pullout failure indicated by a minimum displacement of 0.5" (13 mm) measured at the rear end of the geogrid specimen at the completion of the test.

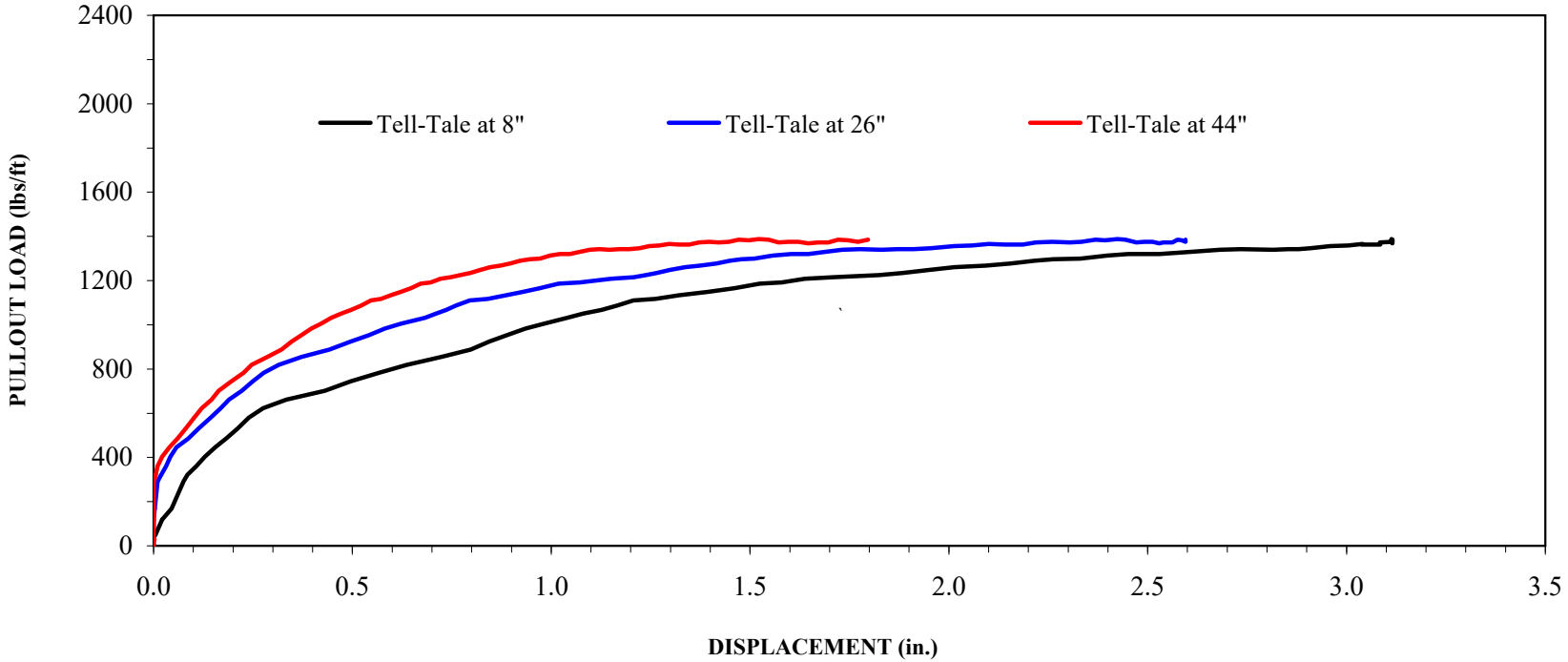
DATE REPORTED:	6/21/2022
FIGURE NO.	6
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

APPENDIX C

CALCULATION OF SCALE EFFECT CORRECTION FACTORS BY USING TELL-TALE GAGE DATA

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 1A at σ_n = 200 psf



Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
1A	14.0	44.0	200	8 26 44	22 72 122	50 168 292	4 14 24

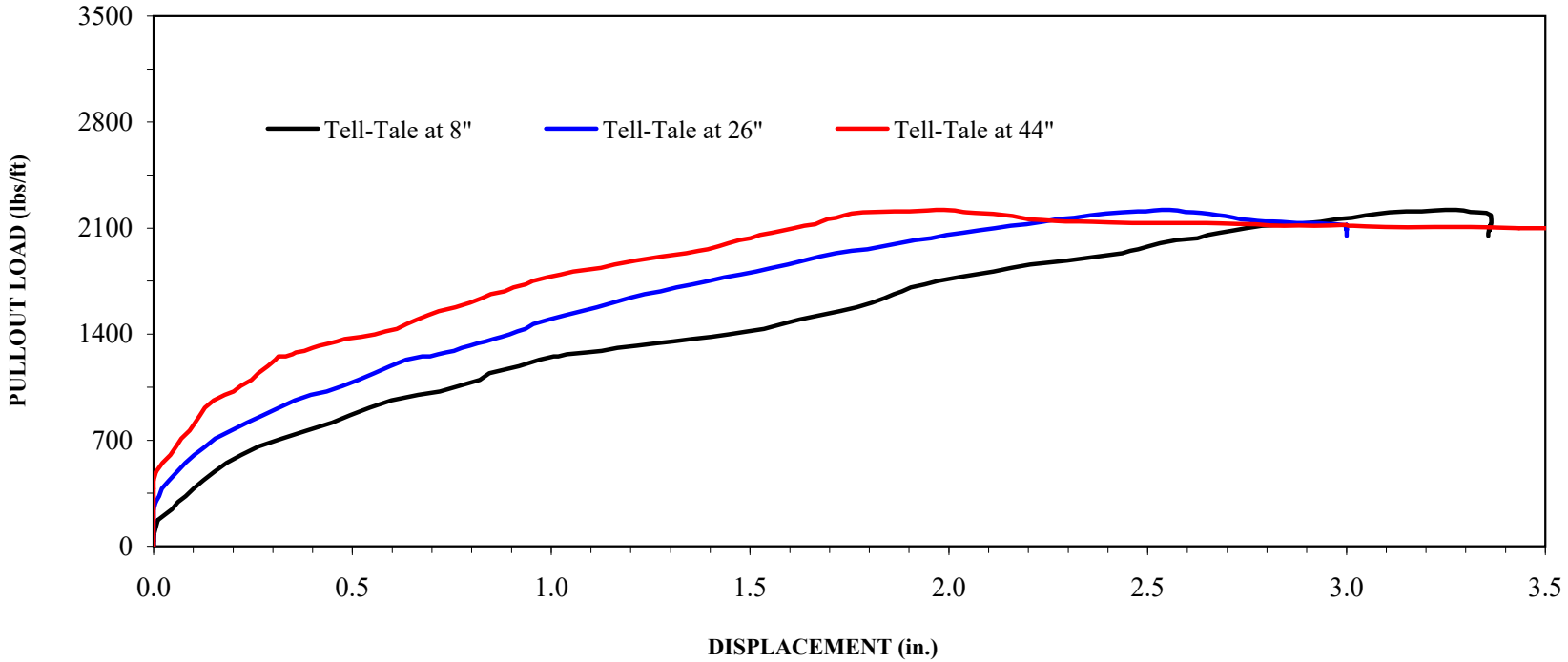
NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout force at the front of the specimen caused the very 1st movement of the tell-tale gage attached to the rear end of section #1, #2, or #3.

DATE REPORTED: 7/25/2022

SGI TESTING SERVICES, LLC

FIGURE NO.	1
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 1B at σ_n = 400 psf



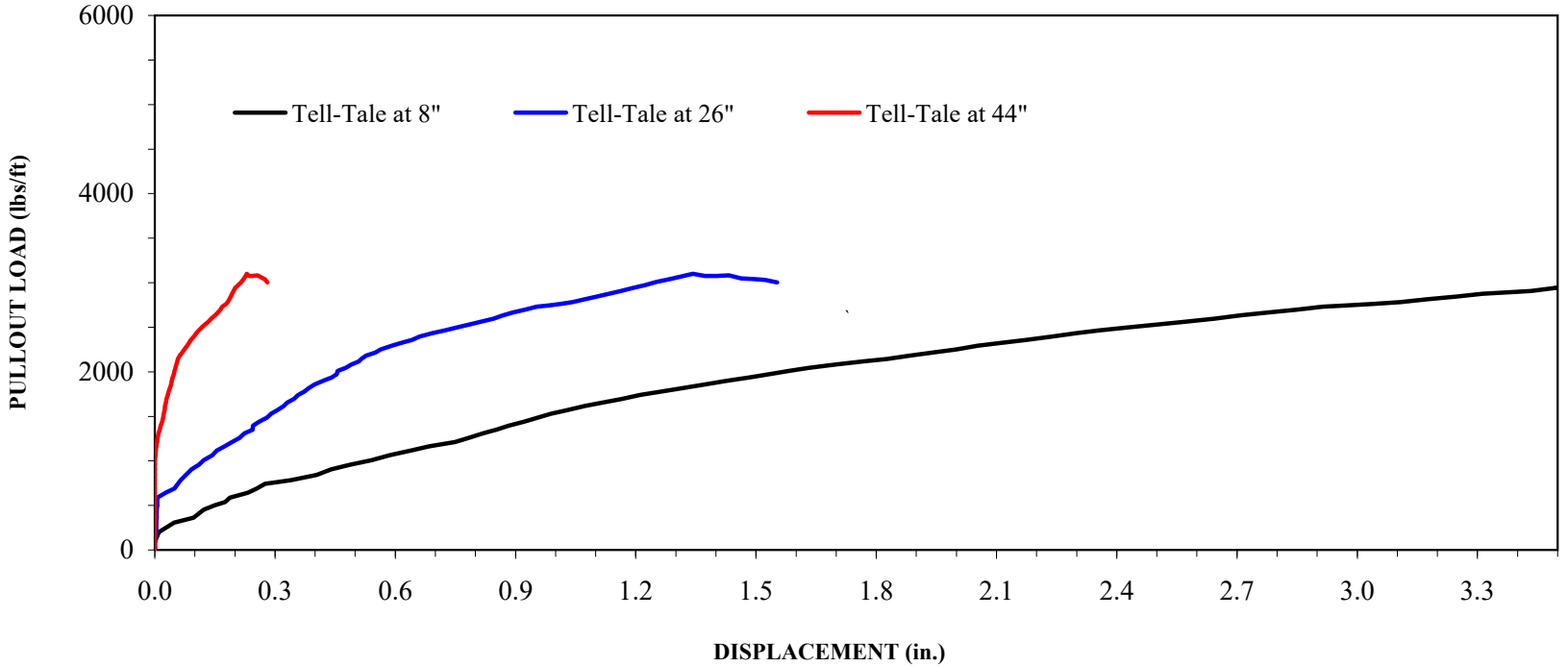
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
1B	14.0	44.0	400	8 26 44	44 144 244	86 292 492	7 24 41

NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout force at the front of the specimen caused the very 1st movement of the tell-tale gage attached to the rear end of section #1, #2, or #3.

DATE REPORTED:	7/25/2022
FIGURE NO.	2
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 1C at σ_n = 800 psf



Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
1C	14.0	44.0	800	8	89	200	17
				26	289	642	54
				44	489	1061	88

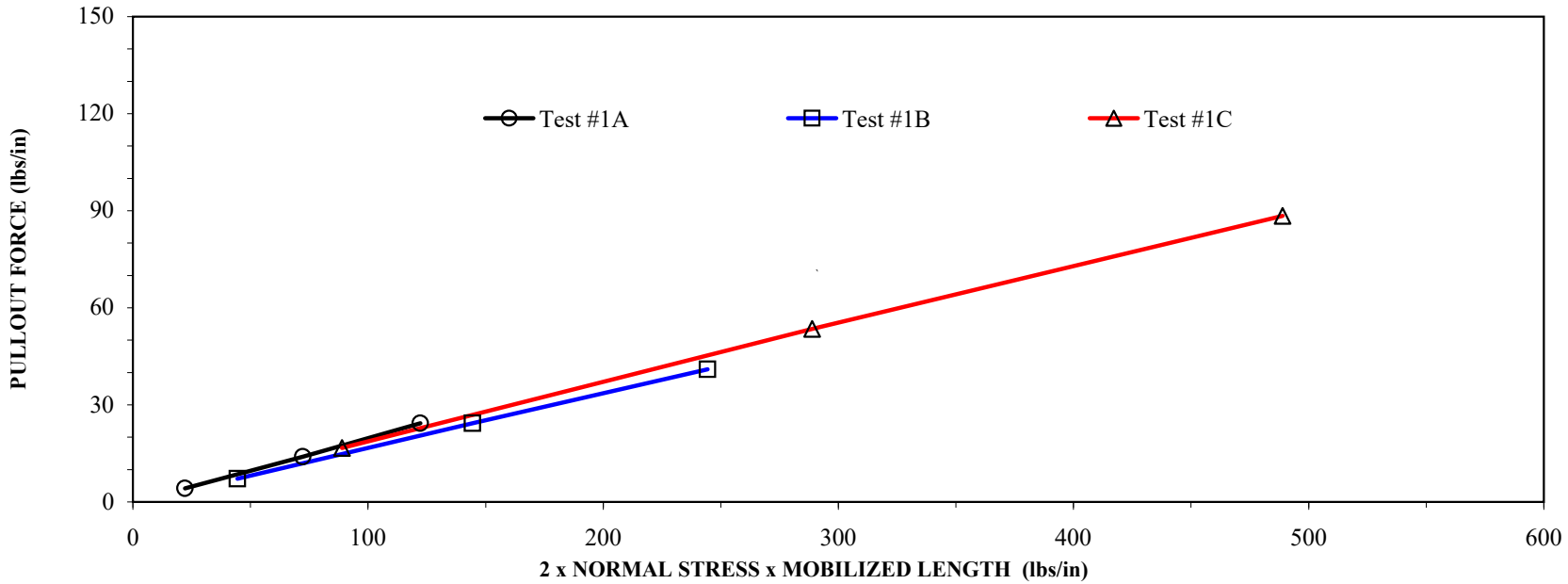
NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout force at the front of the specimen caused the very 1st movement of the tell-tale gage attached to the rear end of section #1, #2, or #3.

DATE REPORTED:	7/25/2022
FIGURE NO.	3
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

TENSAR INTERNATIONAL CORPORATION

SUMMARY OF SCALE EFFECT CORRECTION FACTORS (α) BASED ON TELL-TALE WIRE GAGE DATA



Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length L_p (inch)	Total Normal Force $2N = 2\sigma_n L_p$ (lbs/in)	Pullout Force P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)	Pullout Resistance Factor $F^* = P_r/(2N)$ (-)	Scale Effect Correction Factor $\alpha = F^*_{res}/F^*_{peak}$ Calculated $\alpha^{(1)}$ (-)	Scale Effect Correction Factor $\alpha = F^*_{res}/F^*_{peak}$ Reported $\alpha^{(1)}$
1A	15.0	44.0	200	8	22	50	4	0.19		
				26	72	168	14	0.19		
				44	122	292	24	0.20		
									1.06	**1.00**
1B	15.0	44.0	400	8	44	86	7	0.16		
				26	144	292	24	0.17		
				44	244	492	41	0.17		
									1.04	**1.00**
1C	15.0	44.0	800	8	89	200	17	0.19		
				26	289	642	54	0.19		
				44	489	1061	88	0.18		

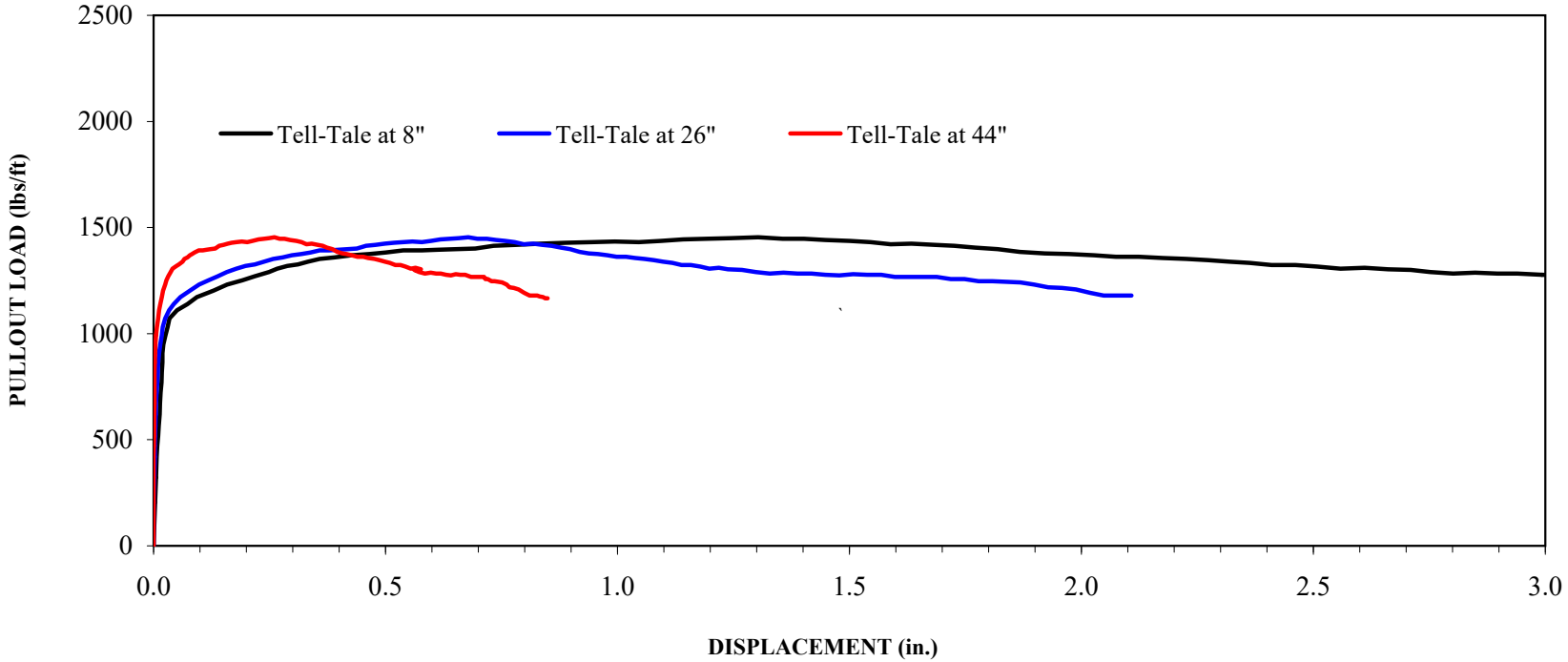
NOTE: (1) If the calculated α factor > 1 by the FHWA method (Appendix A FHWA-NHI-00-043), the reported α factor value is corrected to 1.0.

DATE REPORTED: 7/25/2022

SGI TESTING SERVICES, LLC

FIGURE NO.	4
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 2A at σ_n = 200 psf



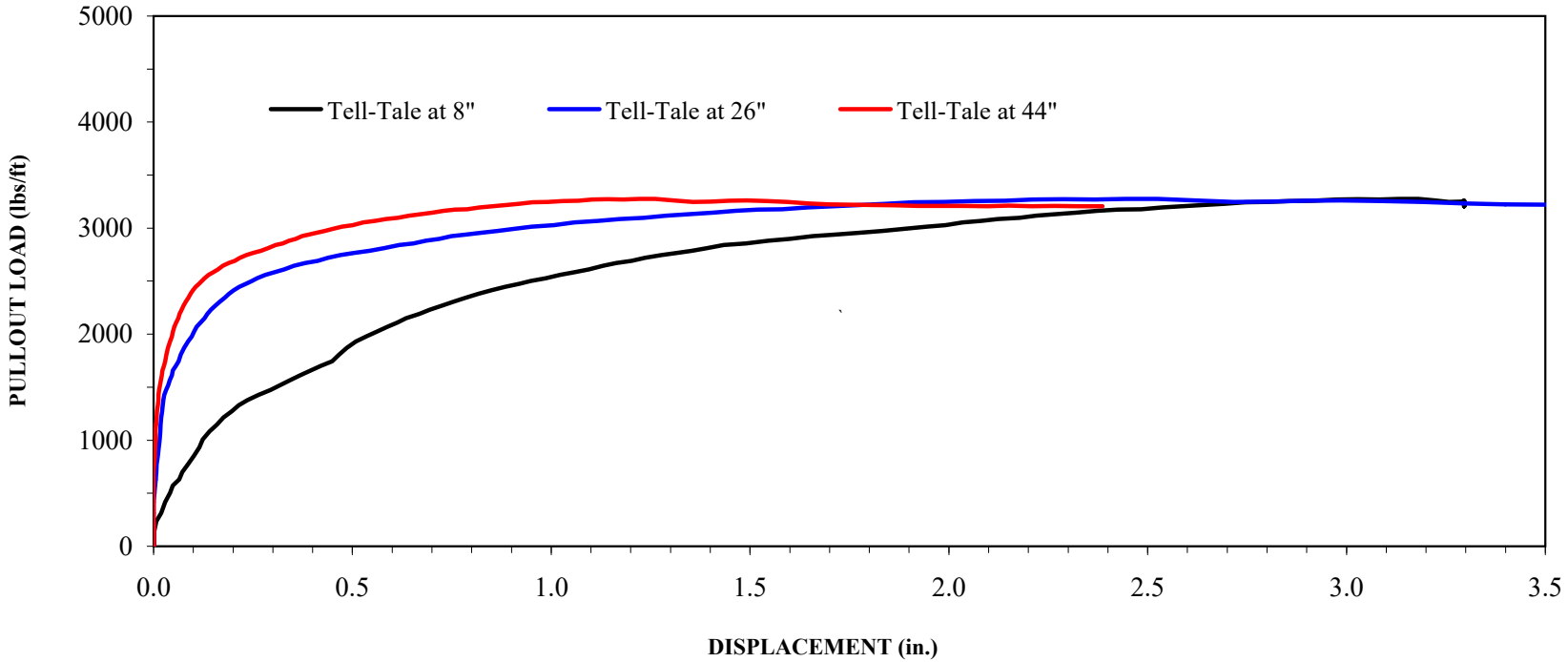
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
2A	15.0	44.0	200	8 26 44	22 72 122	86 322 472	7 27 39

NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout force at the front of the specimen caused the very 1st movement of the tell-tale gage attached to the rear end of section #1, #2, or #3.

DATE REPORTED:	7/25/2022
FIGURE NO.	5
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 2B at σ_n = 600 psf



Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
2B	15.0	44.0	600	8	67	138	12
				26	217	504	42
				44	367	774	65

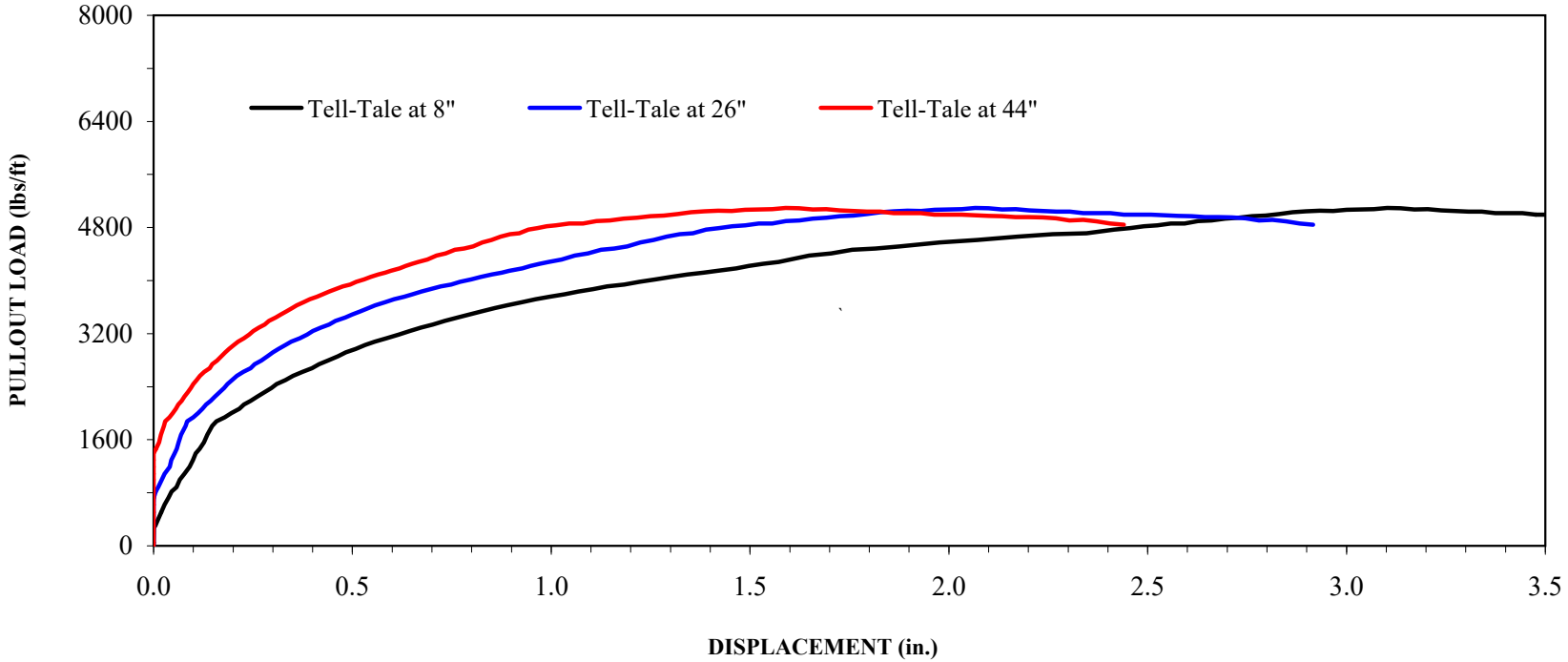
NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout force at the front of the specimen caused the very 1st movement of the tell-tale gage attached to the rear end of section #1, #2, or #3.

DATE REPORTED:	7/25/2022
FIGURE NO.	6
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 2C at σ_n = 1000 psf

C



Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
2C	15.0	44.0	1000	8	111	275	23
				26	361	820	68
				44	611	1463	122

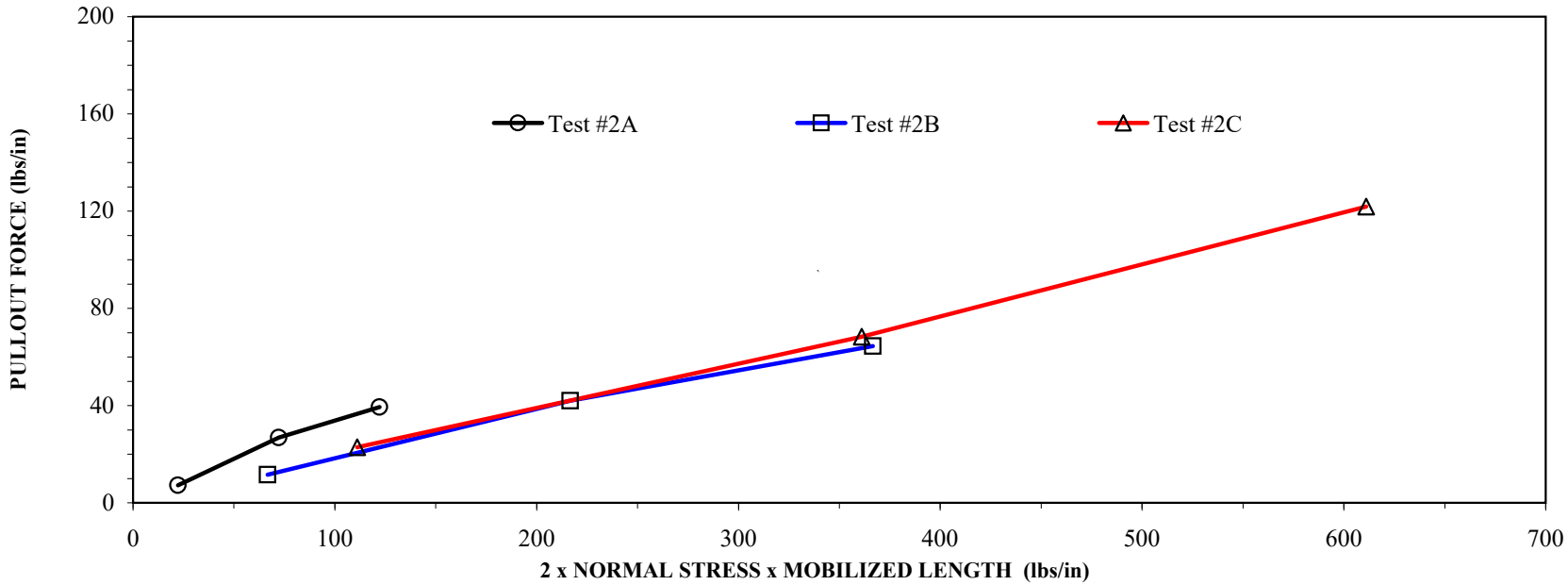
NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout force at the front of the specimen caused the very 1st movement of the tell-tale gage attached to the rear end of section #1, #2, or #3.

DATE REPORTED:	7/25/2022
FIGURE NO.	7
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

TENSAR INTERNATIONAL CORPORATION

SUMMARY OF SCALE EFFECT CORRECTION FACTORS (α) BASED ON TELL-TALE WIRE GAGE DATA



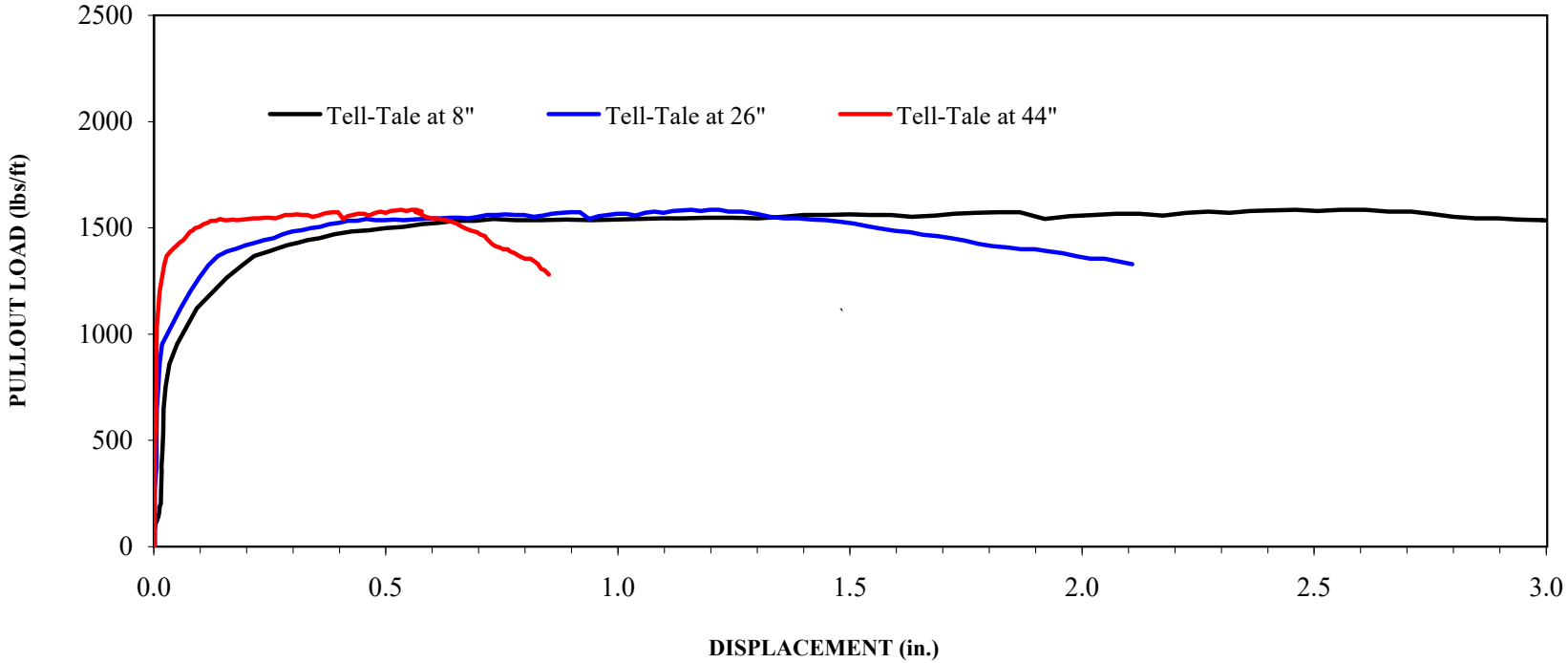
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length L_p (inch)	Total Normal Force $2N = 2\sigma_n L_p$ (lbs/in)	Pullout Force P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)	Pullout Resistance Factor $F^* = P_r/(2N)$ (-)	Scale Effect Correction Factor $\alpha = F^*_{res}/F^*_{peak}$ Calculated $\alpha^{(1)}$ (-)	Scale Effect Correction Factor $\alpha = F^*_{res}/F^*_{peak}$ Reported $\alpha^{(1)}$
2A	15.0	44.0	200	8	22	86	7	0.32		
				26	72	322	27	0.37		
				44	122	472	39	0.32		
									1.00	**1.00**
2B	15.0	44.0	600	8	67	138	12	0.17		
				26	217	504	42	0.19		
				44	367	774	65	0.18		
									1.02	**1.00**
2C	15.0	44.0	1000	8	111	275	23	0.21		
				26	361	820	68	0.19		
				44	611	1463	122	0.20		
									0.97	**0.97**

NOTE: (1) If the calculated α factor > 1 by the FHWA method (Appendix A FHWA-NHI-00-043), the reported α factor value is corrected to 1.0.

DATE REPORTED:	7/25/2022
FIGURE NO.	8
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 3A at σ_n = 200 psf



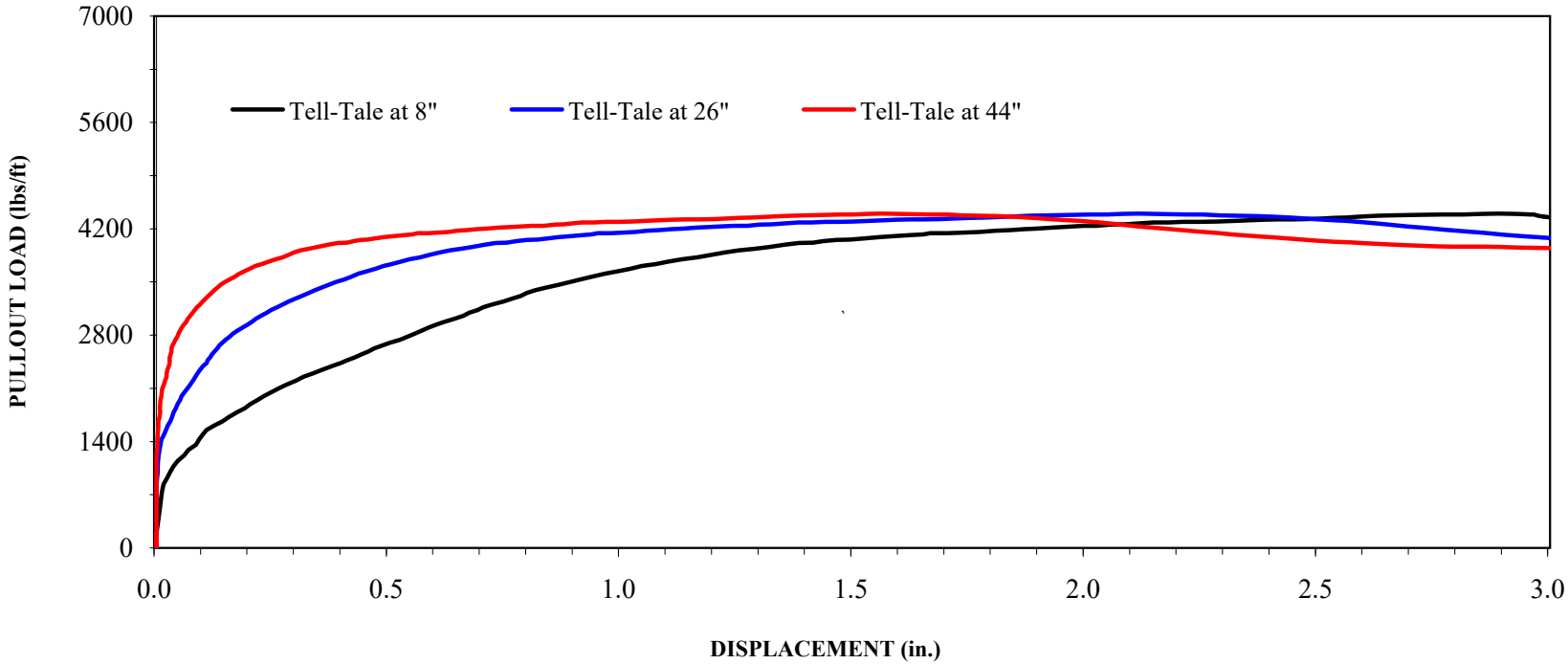
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
3A	15.0	44.0	200	8 26 44	22 72 122	101 266 538	8 22 45

NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout force at the front of the specimen caused the very 1st movement of the tell-tale gage attached to the rear end of section #1, #2, or #3.

DATE REPORTED:	7/25/2022
FIGURE NO.	9
PROJECT NO.	SGI16004
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 3B at σ_n = 800 psf



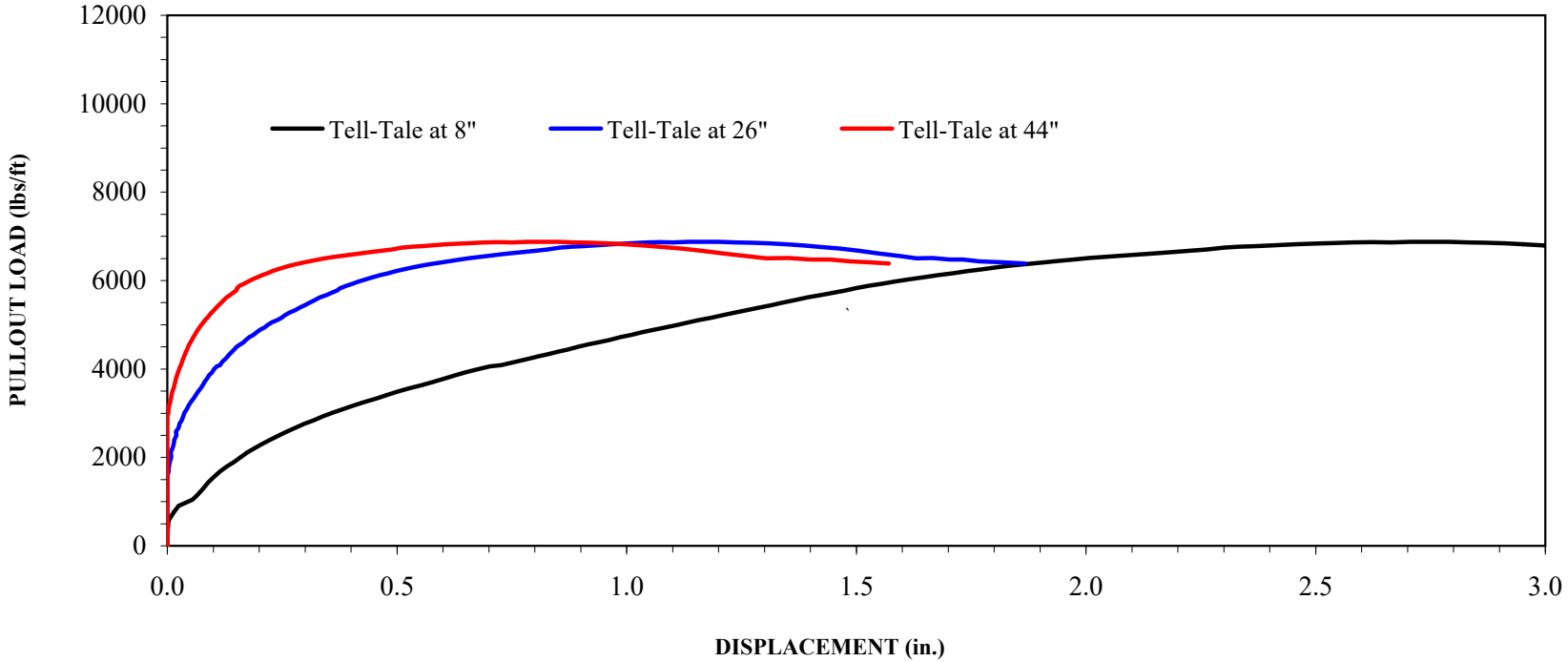
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2N = 2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
3B	15.0	44.0	800	8	89	226	19
				26	289	921	77
				44	489	1291	108

NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout force at the front of the specimen caused the very 1st movement of the tell-tale gage attached to the rear end of section #1, #2, or #3.

DATE REPORTED:	7/25/2022
FIGURE NO.	10
PROJECT NO.	SGI16004
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 3C at σ_n = 1400 psf



Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
3C	15.0	44.0	1400	8 26 44	156 506 856	570 1686 3008	48 141 251

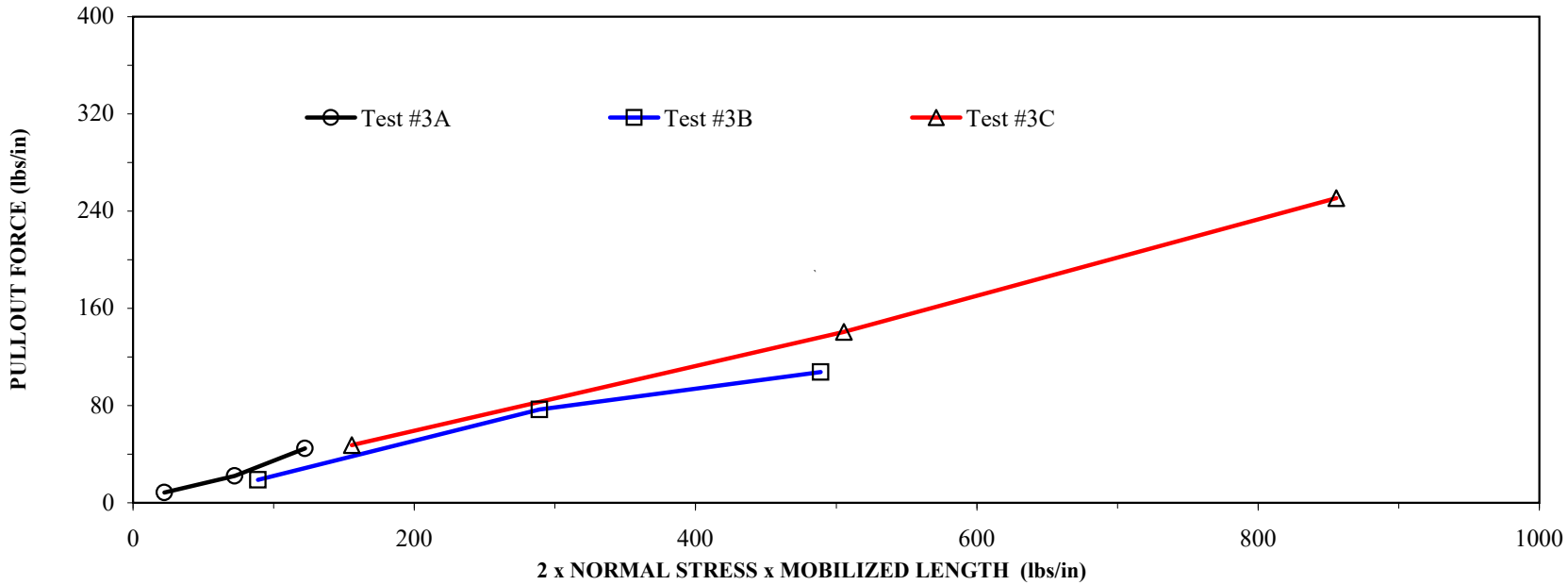
NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout force at the front of the specimen caused the very 1st movement of the tell-tale gage attached to the rear end of section #1, #2, or #3.

DATE REPORTED:	7/25/2022
FIGURE NO.	11
PROJECT NO.	SGI16004
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

TENSAR INTERNATIONAL CORPORATION

SUMMARY OF SCALE EFFECT CORRECTION FACTORS (α) BASED ON TELL-TALE WIRE GAGE DATA



Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length L_p (inch)	Total Normal Force $2N = 2\sigma_n L_p$ (lbs/in)	Pullout Force P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)	Pullout Resistance Factor $F^* = P_r/(2N)$ (-)	Scale Effect Correction Factor $\alpha = F^*_{res}/F^*_{peak}$ Calculated $\alpha^{(1)}$ (-)	Scale Effect Correction Factor $\alpha = F^*_{res}/F^*_{peak}$ Reported $\alpha^{(1)}$
3A	15.0	44.0	200	8 26 44	22 72 122	101 266 538	8 22 45	0.38 0.31 0.37	**0.97**	**0.97**
3B	15.0	44.0	800	8 26 44	89 289 489	226 921 1291	19 77 108	0.21 0.27 0.22	**1.04**	**1.00**
3C	15.0	44.0	1400	8 26 44	156 506 856	570 1686 3008	48 141 251	0.31 0.28 0.29	**0.96**	**0.96**

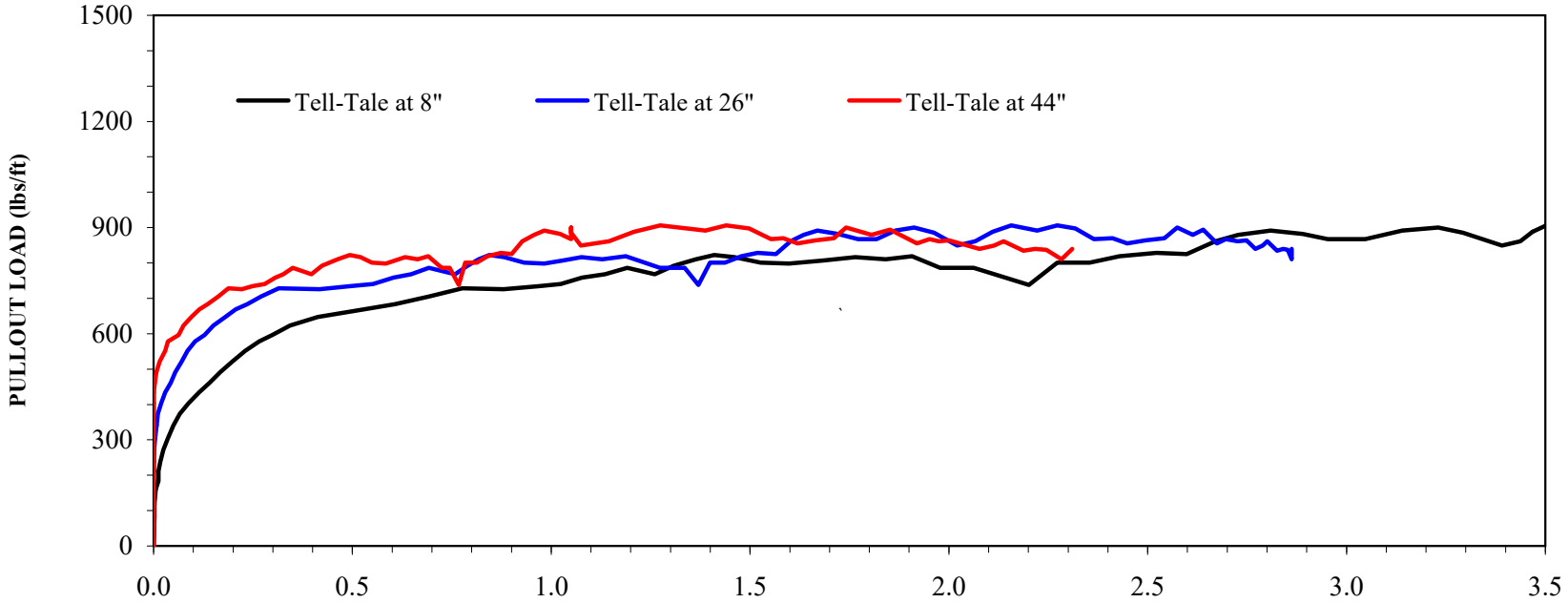
NOTE: (1) If the calculated α factor > 1 by the FHWA method (Appendix A FHWA-NHI-00-043), the reported α factor value is corrected to 1.0.

DATE REPORTED: 7/25/2022

SGI TESTING SERVICES, LLC

FIGURE NO.	12
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 4A at σ_n = 100 psf



Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
4A	15.0	44.0	100	8	11	84	7
				26	36	271	23
				44	61	461	38

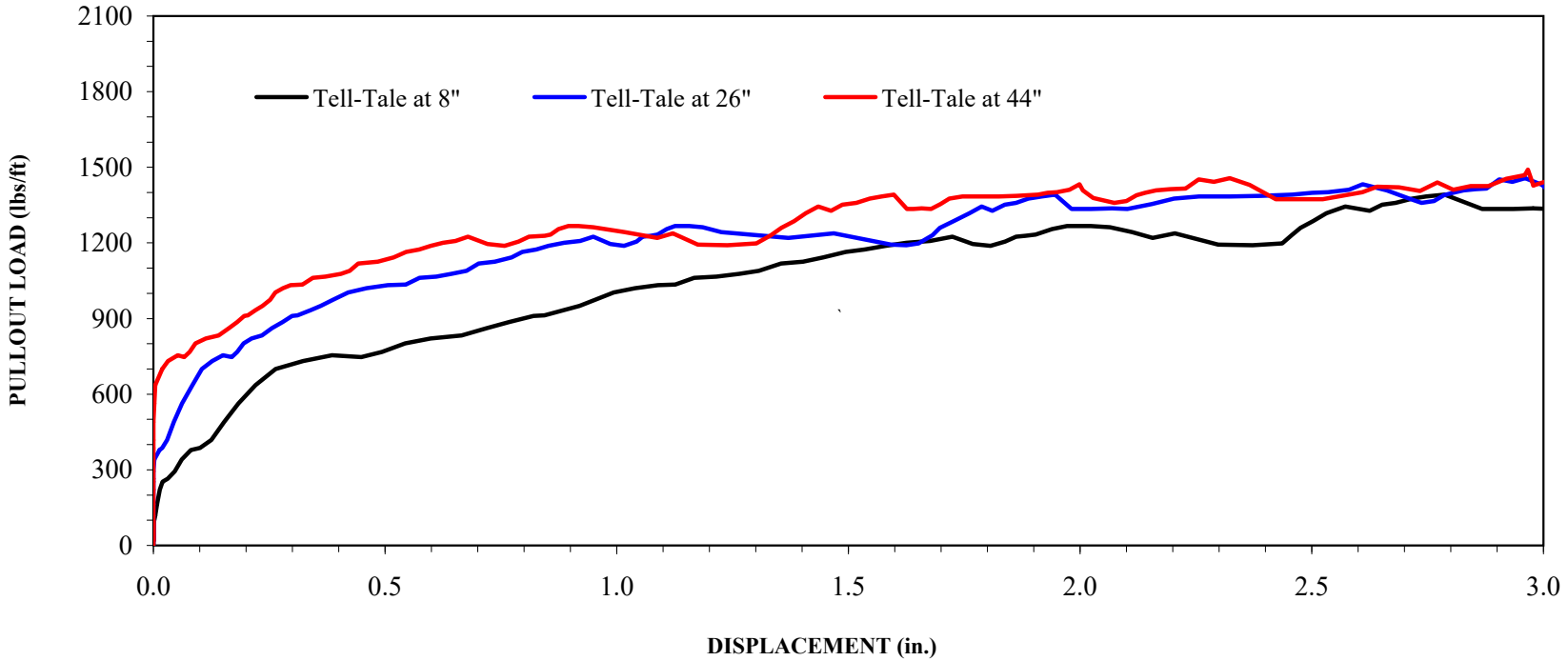
NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout force at the front of the specimen caused the very 1st movement of the tell-tale gage attached to the rear end of section #1, #2, or #3.

DATE REPORTED: 7/25/2022

SGI TESTING SERVICES, LLC

FIGURE NO.	13
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 4B at σ_n = 200 psf



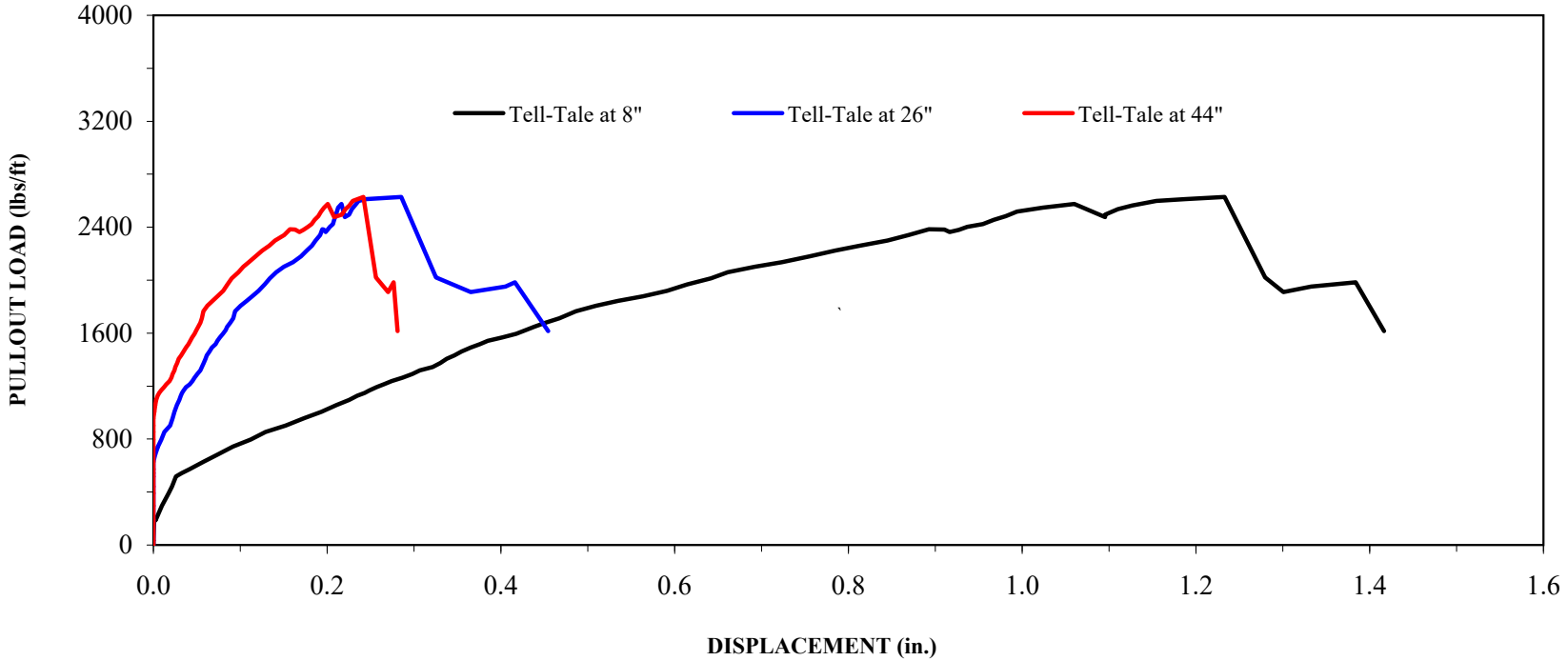
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
4B	15.0	44.0	200	8 26 44	22 72 122	118 378 636	10 32 53

NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout force at the front of the specimen caused the very 1st movement of the tell-tale gage attached to the rear end of section #1, #2, or #3.

DATE REPORTED:	7/25/2022
FIGURE NO.	14
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 4C at σ_n = 400 psf



Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
4C	15.0	44.0	400	8	44	190	16
				26	144	682	57
				44	244	1005	84

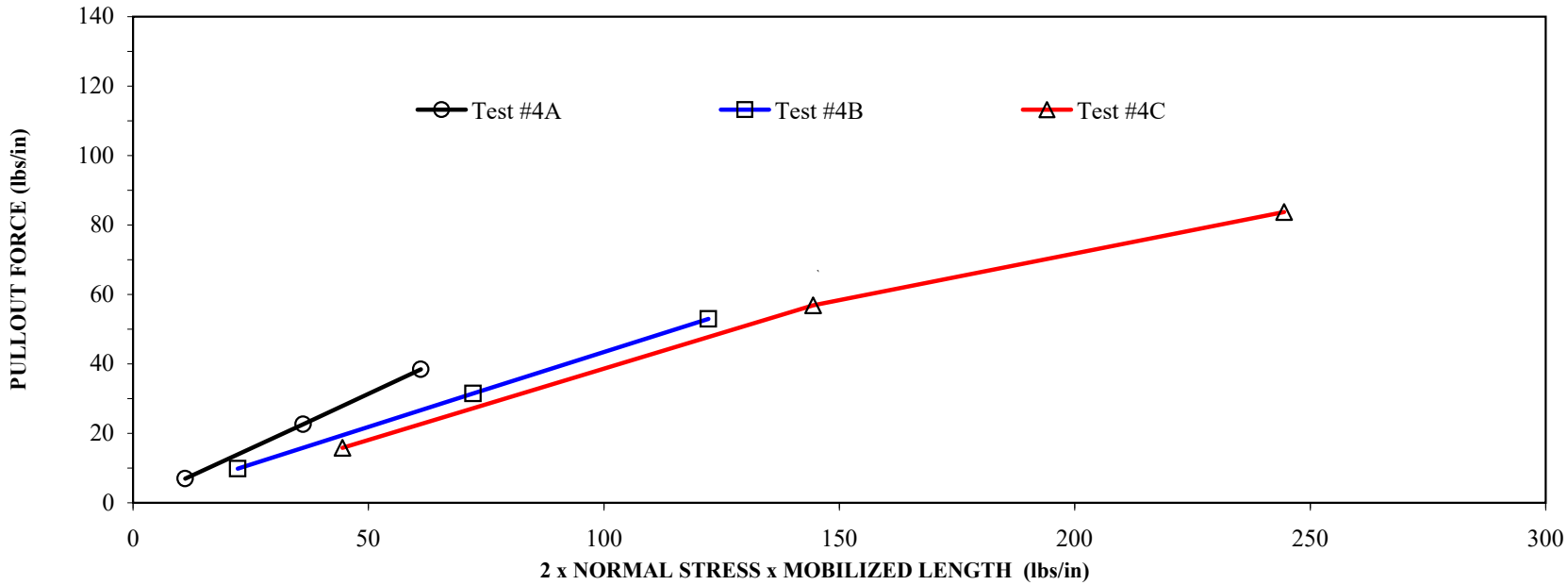
NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout force at the front of the specimen caused the very 1st movement of the tell-tale gage attached to the rear end of section #1, #2, or #3.

DATE REPORTED: 7/25/2022

SGI TESTING SERVICES, LLC

FIGURE NO.	15
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

TENSAR INTERNATIONAL CORPORATION
SUMMARY OF SCALE EFFECT CORRECTION FACTORS (α) BASED ON TELL-TALE WIRE GAGE DATA



Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length L_p (inch)	Total Normal Force $2N=2\sigma_n L_p$ (lbs/in)	Pullout Force P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)	Pullout Resistance Factor $F^* = P_r/(2N)$ (-)	Scale Effect Correction Factor $\alpha = F^*_{res}/F^*_{peak}$ Calculated $\alpha^{(1)}$ (-)	Scale Effect Correction Factor $\alpha = F^*_{res}/F^*_{peak}$ Reported $\alpha^{(1)}$
4A	15.0	44.0	100	8	11	84	7	0.63		
				26	36	271	23	0.63		
				44	61	461	38	0.63		
									1.00	**1.00**
4B	15.0	44.0	200	8	22	118	10	0.44		
				26	72	378	32	0.44		
				44	122	636	53	0.43		
									0.98	**0.98**
4C	15.0	44.0	400	8	44	190	16	0.36		
				26	144	682	57	0.39		
				44	244	1005	84	0.34		

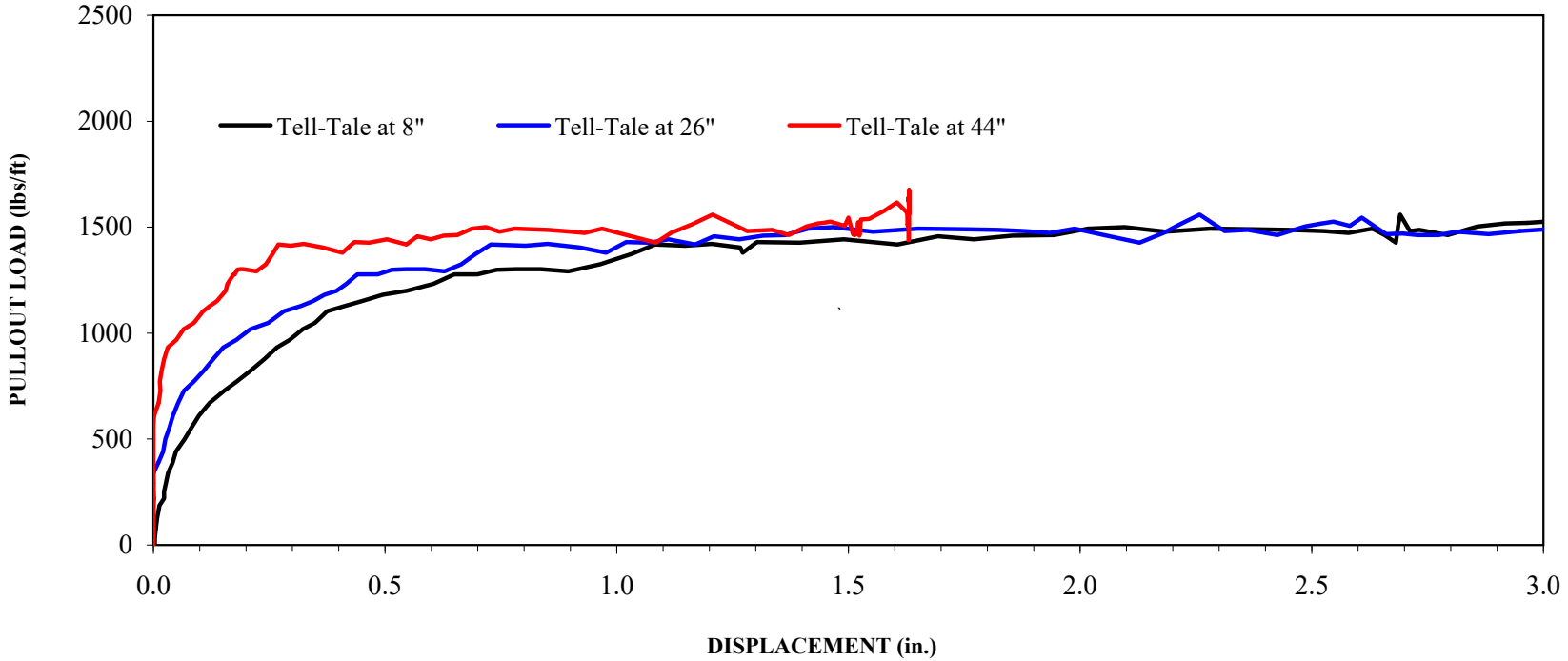
NOTE: (1) If the calculated α factor > 1 by the FHWA method (Appendix A FHWA-NHI-00-043), the reported α factor value is corrected to 1.0.

DATE REPORTED: 7/25/2022

SGI TESTING SERVICES, LLC

FIGURE NO.	16
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 5A at σ_n = 200 psf



Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
5A	15.0	44.0	200	8	22	125	10
				26	72	388	32
				44	122	670	56

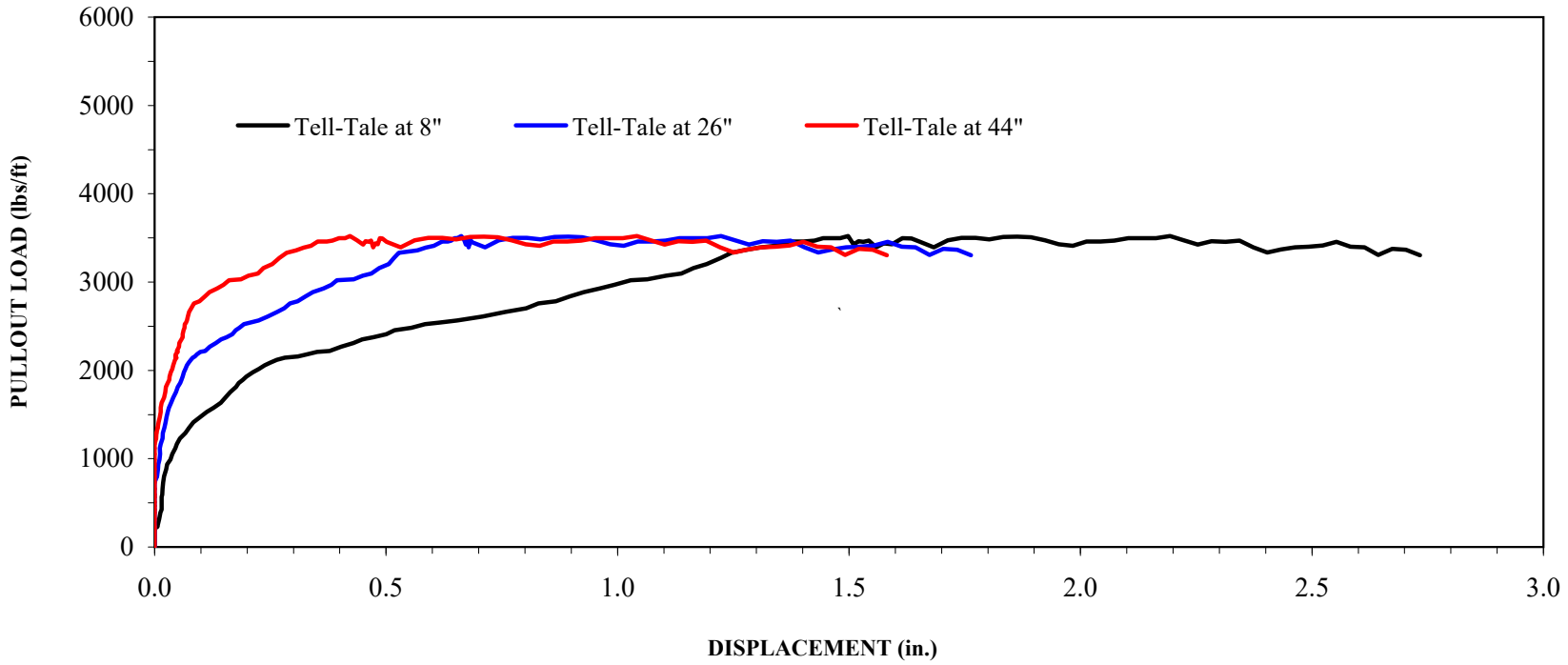
NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout force at the front of the specimen caused the very 1st movement of the tell-tale gage attached to the rear end of section #1, #2, or #3.

DATE REPORTED: 7/25/2022

SGI TESTING SERVICES, LLC

FIGURE NO.	17
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 5B at σ_n = 500 psf



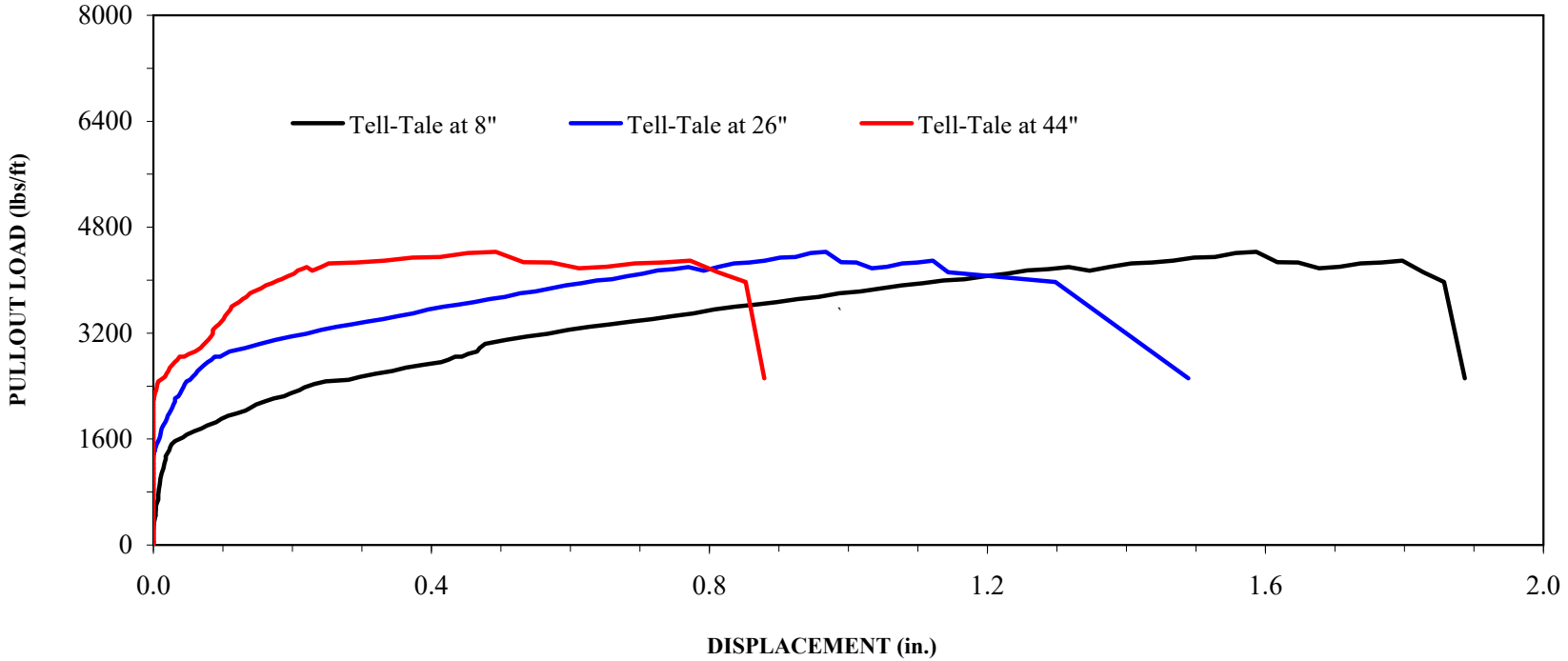
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
5B	15.0	44.0	500	8 26 44	56 181 306	232 801 1229	19 67 102

NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout force at the front of the specimen caused the very 1st movement of the tell-tale gage attached to the rear end of section #1, #2, or #3.

DATE REPORTED:	7/25/2022
FIGURE NO.	18
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 5C at σ_n = 800 psf



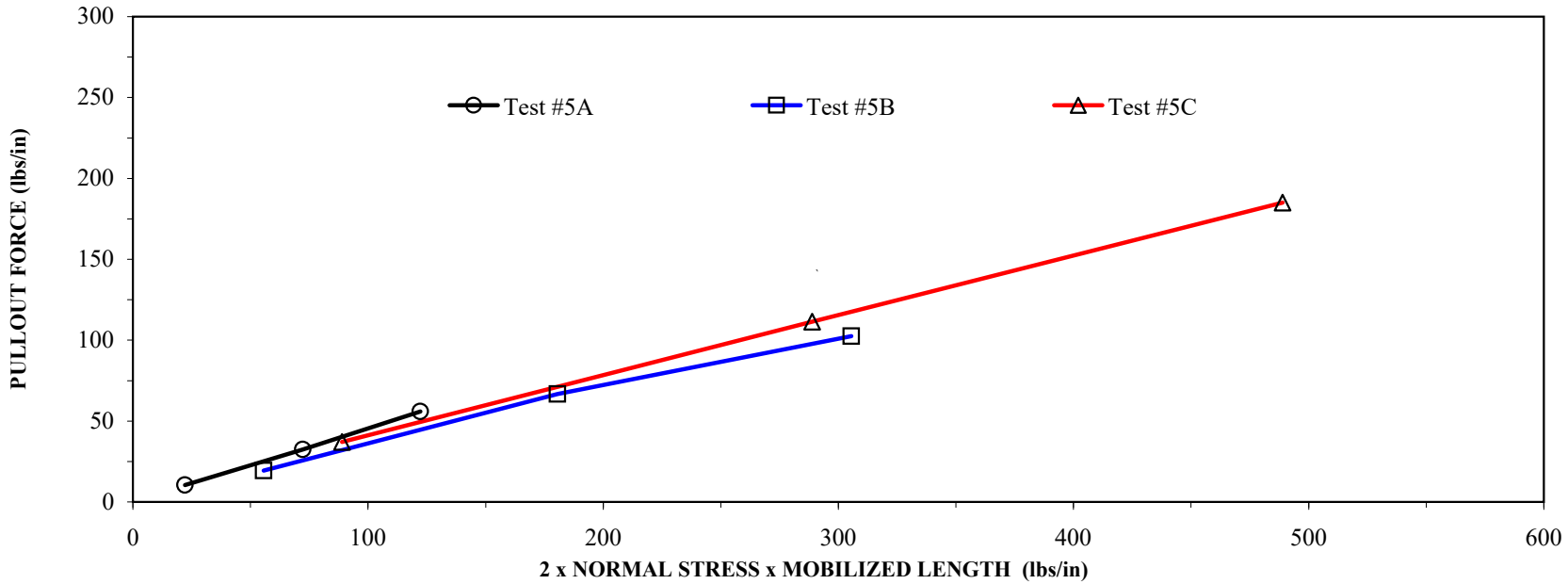
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
5C	15.0	44.0	800	8	89	446	37
				26	289	1338	112
				44	489	2221	185

NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout force at the front of the specimen caused the very 1st movement of the tell-tale gage attached to the rear end of section #1, #2, or #3.

DATE REPORTED:	7/25/2022
FIGURE NO.	19
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

TENSAR INTERNATIONAL CORPORATION
SUMMARY OF SCALE EFFECT CORRECTION FACTORS (α) BASED ON TELL-TALE WIRE GAGE DATA



Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length L_p (inch)	Total Normal Force $2N=2\sigma_n L_p$ (lbs/in)	Pullout Force P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)	Pullout Resistance Factor $F^* = P_r/(2N)$ (-)	Scale Effect Correction Factor $\alpha = F^*_{res}/F^*_{peak}$ Calculated $\alpha^{(1)}$ (-)	Reported $\alpha^{(1)}$
5A	15.0	44.0	200	8	22	125	10	0.47		
				26	72	388	32	0.45		
				44	122	670	56	0.46		
									0.97	**0.97**
5B	15.0	44.0	500	8	56	232	19	0.35		
				26	181	801	67	0.37		
				44	306	1229	102	0.34		
									0.96	**0.96**
5C	15.0	44.0	800	8	89	446	37	0.42		
				26	289	1338	112	0.39		
				44	489	2221	185	0.38		

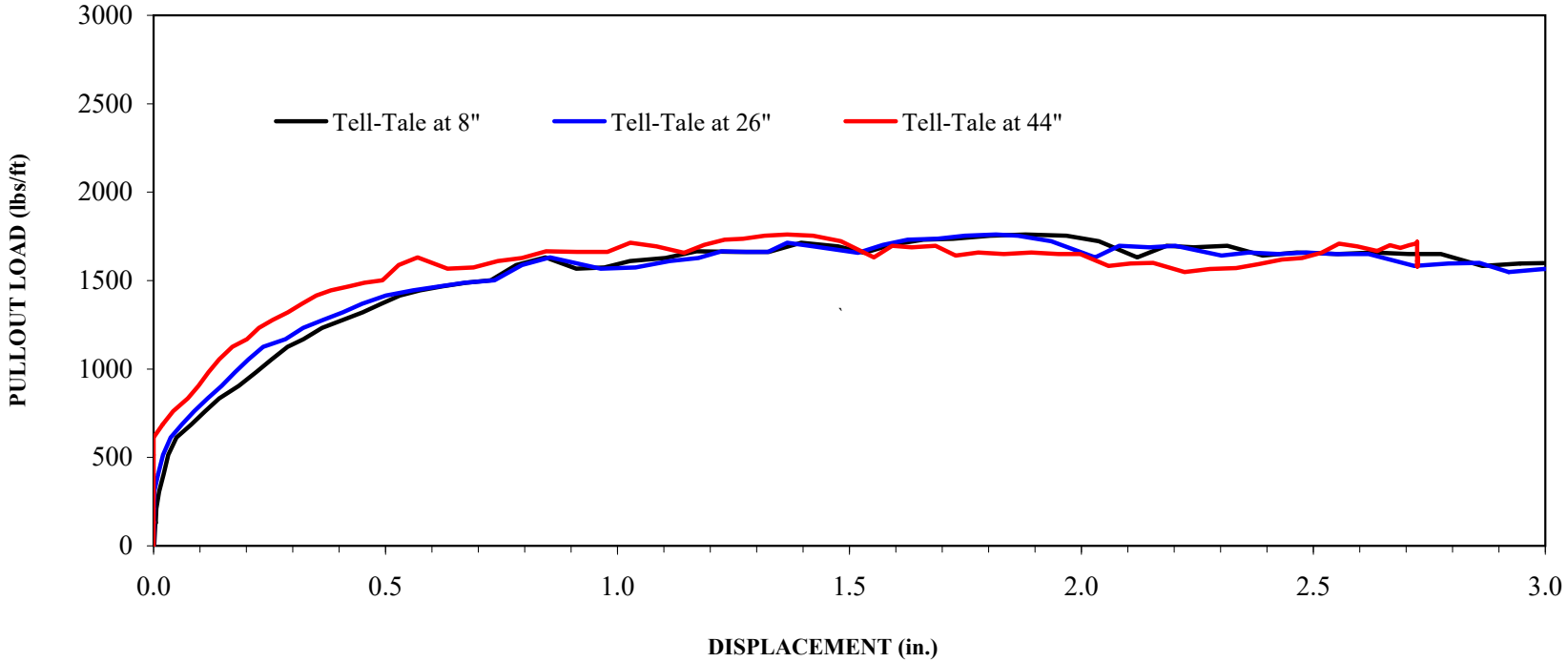
NOTE: (1) If the calculated α factor > 1 by the FHWA method (Appendix A FHWA-NHI-00-043), the reported α factor value is corrected to 1.0.

DATE REPORTED: 7/25/2022

SGI TESTING SERVICES, LLC

FIGURE NO.	20
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 6A at σ_n = 200 psf



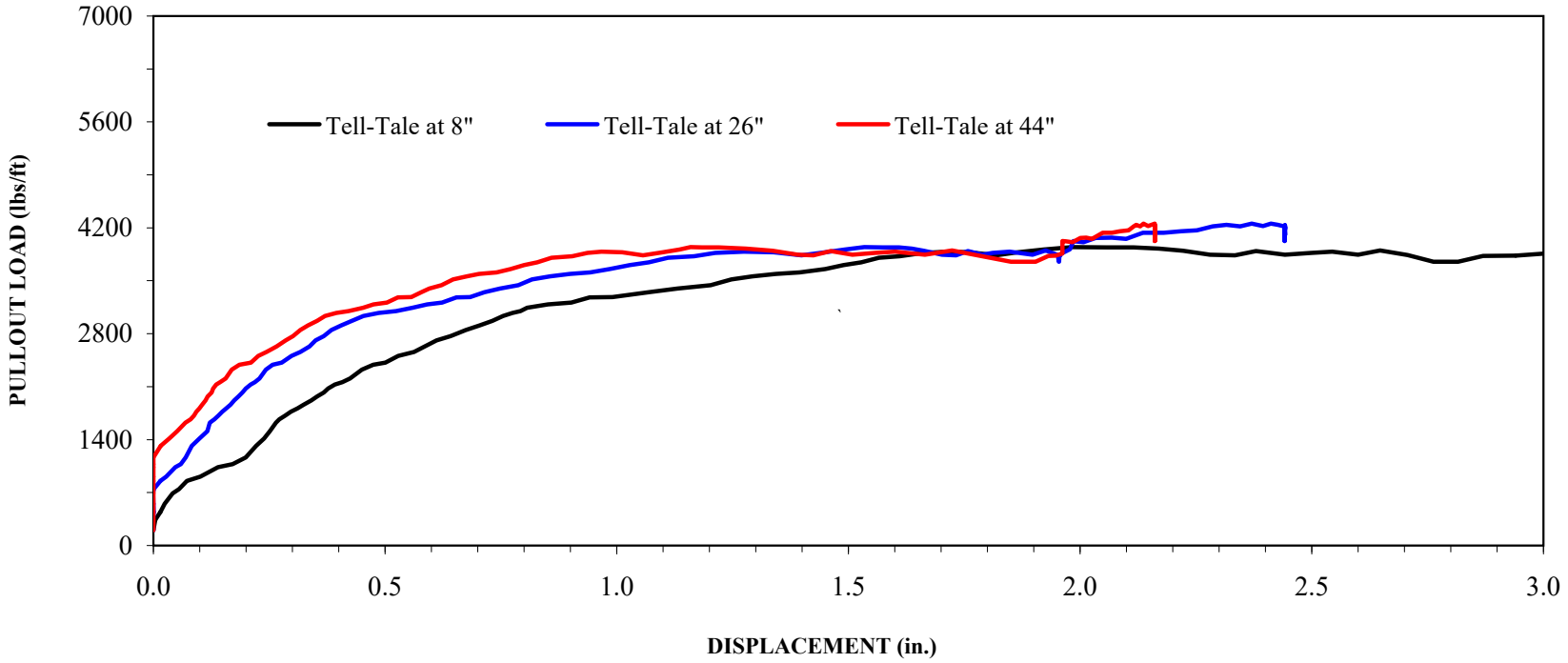
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
6A	15.0	44.0	200	8 26 44	22 72 122	130 405 686	11 34 57

NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout force at the front of the specimen caused the very 1st movement of the tell-tale gage attached to the rear end of section #1, #2, or #3.

DATE REPORTED:	7/25/2022
FIGURE NO.	21
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 6B at σ_n = 600 psf



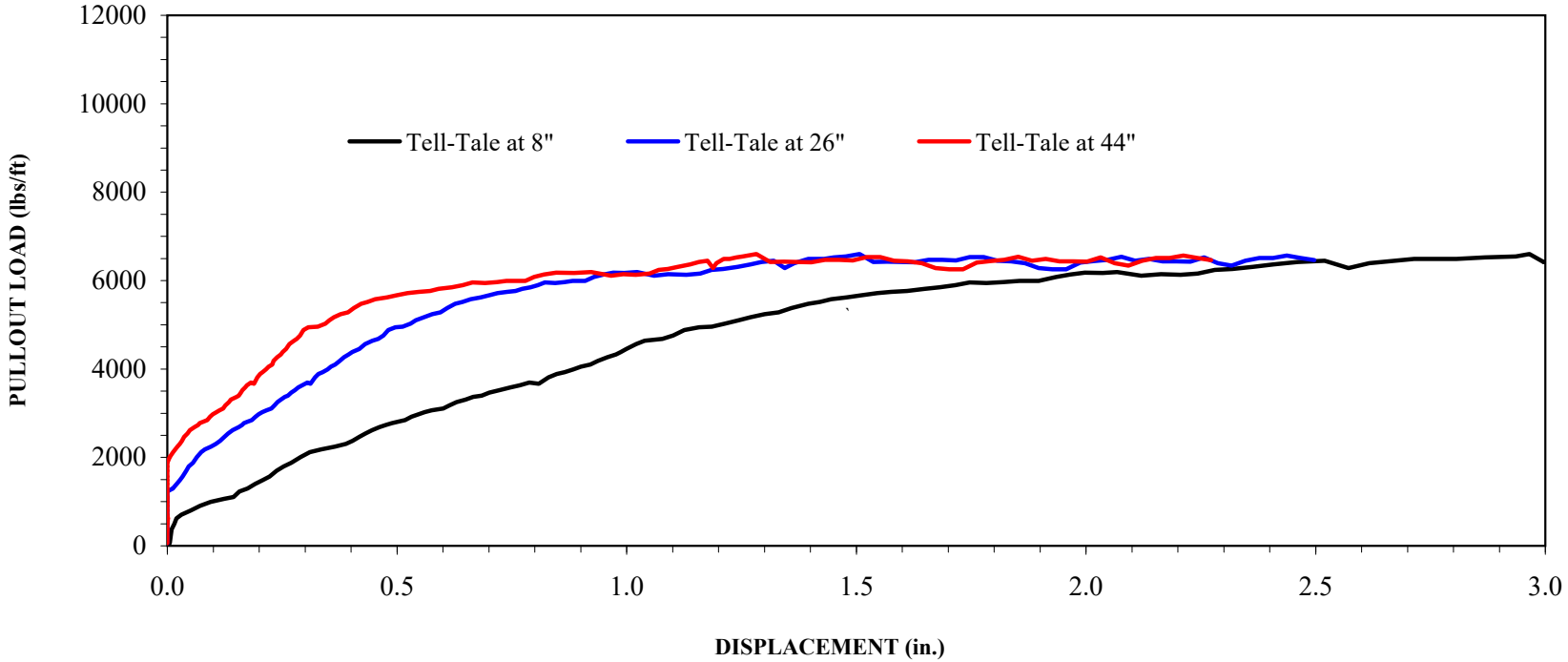
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
6B	15.0	44.0	600	8	67	251	21
				26	217	855	71
				44	367	1317	110

NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout force at the front of the specimen caused the very 1st movement of the tell-tale gage attached to the rear end of section #1, #2, or #3.

DATE REPORTED:	7/25/2022
FIGURE NO.	22
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 6C at σ_n = 1000 psf



Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
6C	15.0	44.0	1000	8	111	375	31
				26	361	1301	108
				44	611	2011	168

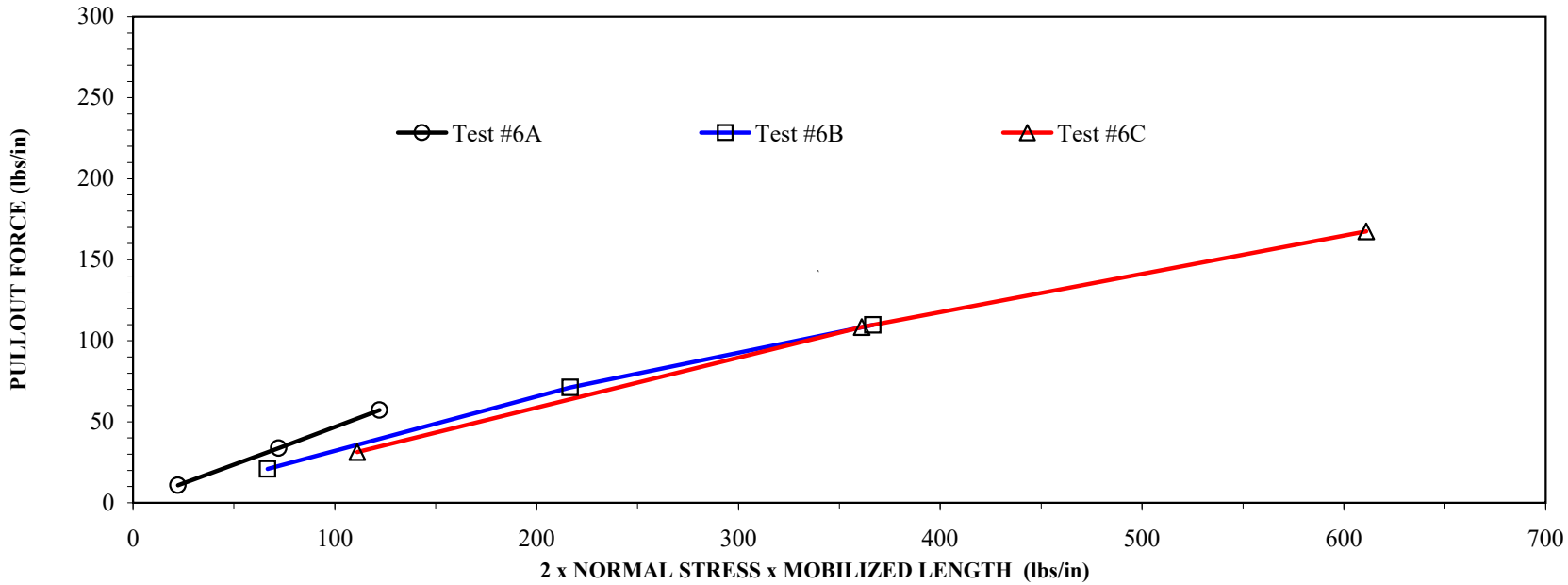
NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout force at the front of the specimen caused the very 1st movement of the tell-tale gage attached to the rear end of section #1, #2, or #3.

DATE REPORTED:	7/25/2022
FIGURE NO.	23
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

TENSAR INTERNATIONAL CORPORATION

SUMMARY OF SCALE EFFECT CORRECTION FACTORS (α) BASED ON TELL-TALE WIRE GAGE DATA



Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length L_p (inch)	Total Normal Force $2N = 2\sigma_n L_p$ (lbs/in)	Pullout Force P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)	Pullout Resistance Factor $F^* = P_r/(2N)$ (-)	Scale Effect Correction Factor $\alpha = F^*_{res}/F^*_{peak}$ Calculated $\alpha^{(1)}$ (-)	Scale Effect Correction Factor $\alpha = F^*_{res}/F^*_{peak}$ Reported $\alpha^{(1)}$
6A	15.0	44.0	200	8	22	130	11	0.49		
				26	72	405	34	0.47		
				44	122	686	57	0.47		
									0.96	**0.96**
6B	15.0	44.0	600	8	67	251	21	0.31		
				26	217	855	71	0.33		
				44	367	1317	110	0.30		
									0.95	**0.95**
6C	15.0	44.0	1000	8	111	375	31	0.28		
				26	361	1301	108	0.30		
				44	611	2011	168	0.27		
									0.98	**0.98**

NOTE: (1) If the calculated α factor > 1 by the FHWA method (Appendix A FHWA-NHI-00-043), the reported α factor value is corrected to 1.0.

DATE REPORTED: 7/25/2022

SGI TESTING SERVICES, LLC

FIGURE NO.	24
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

Appendix 1.2.7.a

Pullout Tests in Soil Report

Tensar UX1100MSE Geogrids / Course Sand

MESA® MSE Retaining Wall System

Back to main document

FINAL REPORT

RESULTS OF

IN-SOIL PULLOUT TESTING

WITH

Tensar UX1100MSE Geogrid

In Coarse Sand (SW)

Submitted to

Tensar Earth Technologies Inc.

Distribution:

2 copies Tensar Earth Technologies Inc.
5883 Glenridge Dr., Ste. 200
Atlanta GA 30328
USA

2 copies Bathurst, Clarabut Geotechnical Testing, Inc.
1167 Clyde Court, Kingston, Ontario
K7P 2E4
CANADA

This report shall not be reproduced except in full, without the prior written approval of Bathurst, Clarabut Geotechnical Testing, Inc.

Introduction

This report gives the results of an in-soil pullout testing program that was carried out to evaluate the in-soil performance of Tensar UX1100MSE geogrid soil reinforcement in coarse sand (SW). The test program was initiated in response to a verbal authorization to proceed from Mr. Andy Anderson of Tensar Earth Technologies Inc., received 15 March 2004. The tests were carried out at the laboratories of Bathurst, Clarabut Geotechnical Testing, Inc. in Kingston, Ontario, under the supervision of Mr. Peter Clarabut.

Objectives of test program

The principal objective of the testing was to evaluate the in-soil pullout capacity of Tensar UX1100MSE geogrid embedded in coarse sand (SW), while subjected to a range of confining pressures. The pullout tests were performed using the large pullout test apparatus shown in **Figures 1** and **2**.

Materials

Tensar UX1100MSE geogrid is a uni-axial geogrid with a reported MARV strength of 58.0 kN/m in the machine direction in accordance with GRI GG1 and tested at a constant rate of displacement of 10% of the gage length per minute. Specimens were cut from Roll/Lot #18673-127 (Tensar Designation) received at our laboratory on 15 March 2004. Based on tests performed in our laboratory a tensile strength of 61.1 kN/m was determined.

The particle size distribution curve for the soil used in this investigation is shown in **Figure 3**. The constant volume (residual) friction angle of the soil is 35 degrees.

Pullout box and test methodology

The test apparatus and method of test was in accordance with Draft ASTM Method of Test - Measuring Geosynthetic Pullout Resistance in Soil - ASTM Designation Number Z2467Z Draft No. 8 dated January 2001.

The pullout box is approximately 2290 mm long by 710 mm high (outside dimension). The inside width of the box is 760 mm (**Figure 2**). The soil surface was surcharged using a Kevlar reinforced polymer air bag to simulate variable depths of overburden soil above the test specimen. The bag allows a uniform surcharge pressure of up to 200 kPa to be applied to the surface of the soil. The pullout box surcharging arrangement is self-reacting using a system of steel angles that encompass the box. Columns of 50 mm-wide hollow steel sections slotted into each end of the test apparatus are used to confine the soil at the front and back of the pullout box. An adjustable pair of steel sleeves is used to pass the test specimen through the front end of the pullout box. The sidewalls of the pullout box are constructed from 25 mm-thick plywood that is covered on the inside (soil side) with a friction-reducing polymer liner.

For each test, a 286 mm-wide (13 longitudinal strands) geogrid specimen was used and extended approximately 900 mm into the soil box (3 transverse members). Each specimen was placed over a 305 mm-deep layer of compacted soil. Steel wire tell-tales were attached to at least 3 nodal points along the length of each specimen (see **Figure 4**) and displacements recorded using potentiometer-type displacement transducers. For all tests the nodal points were located at 25 mm, 480 mm, and 900 mm from the inside front sleeve of the pullout box. The entire specimen was then covered by a 305 mm-depth of soil and compacted. The soil above and below the specimen elevation was compacted in 150 mm lifts using an electric Kango impact hammer and compacting plate. The compacted unit densities and moisture content were recorded using a nuclear density meter. Average measurement results are reported in **Table 1**.

The reinforcement clamp (**Figure 5**) was attached to a computer-controlled electro-mechanical screw jack actuator with a maximum capacity of about 50 kN. The rate of displacement of the front clamp was kept constant at 1 mm/minute. The tensile load and displacements recorded by the actuator load cell and the potentiometers were monitored continuously by a computer-controlled data acquisition system.

Test results

As-compacted soil properties for the coarse sand (SW) are summarized in **Table 1**.

Peak pullout capacities are summarized in **Table 2**.

A pullout capacity versus confining pressure plot is presented in **Figure 6**.

Photographs of exhumed test specimens are shown in **Figures 7** through **10**.

Pullout force versus displacement plots for each test are presented in **Figures A1** through **A4** of **Appendix A**.

In **Test 3** rupture of the geogrid specimen occurred.

Pullout of the geogrid specimen occurred in **Tests 1, 2** and **4**.

Coefficient of interaction (C_i) values were calculated as follows based on recommendations in the GRI GG5 protocol:

$$C_i = P/2WL\sigma_n \tan\phi \leq 1.0$$

where:

P = pullout load (kN)
W = width of specimen (m)
L = length of specimen (m)
σ_n = normal (confining) pressure (kPa)
ϕ = residual friction angle of soil (degrees)

Using the expression above, all tests that ended in reinforcement pullout gave $C_i \leq 1.0$ as shown in **Table 2**.



Richard J. Bathurst Ph.D., P.Eng.

REFERENCES

GRI Test Method GG5 “Geogrid Pullout” dated 30 January 1991

Draft ASTM Method of Test - Measuring Geosynthetic Pullout Resistance in Soil - ASTM Designation Number Z2467Z Draft No. 8 dated January 2001

Table 1: As-compacted soil properties

Soil	Bulk Unit Density (kg/m^3)	Moisture Content (%)	Residual Friction Angle
Coarse sand (SW)	1800	2.5	35°

Table 2:
Coarse sand (SW) – Tensar UX1100MSE Geogrid in-soil pullout test results

Test No.	File Name	Date	Max Pullout Resistance (kN)	Confining Pressure (kPa)	Specimen Length in Soil (m)	Failure Type	C_i
1	TUX11C1	Jun 25/04	2.2	5	0.9	Pullout	1.0
2	TUX11C3	Jun 29/04	7.6	53	0.9	Pullout	0.4
3	TUX11C4	Jul 2/04	11.3	108	0.9	Rupture	n/a
4	TUX11C5	Jul 5/04	5.3	18	0.9	Pullout	0.8



Figure 1: Pullout box apparatus.

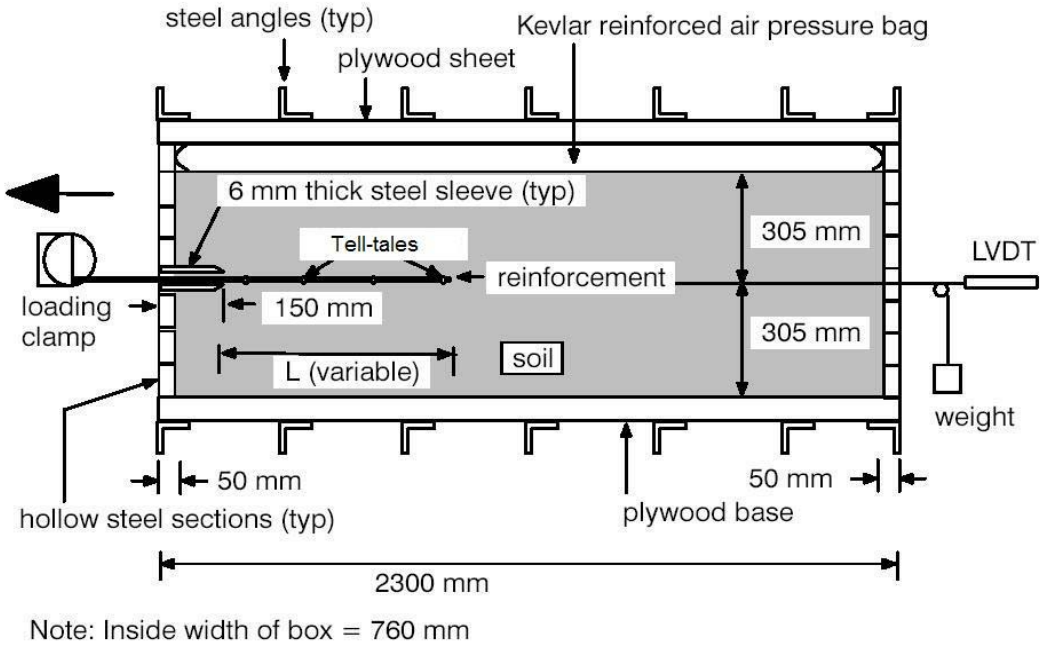


Figure 2: Cross-section view of in-soil pullout box apparatus and general arrangement of soil reinforcement.

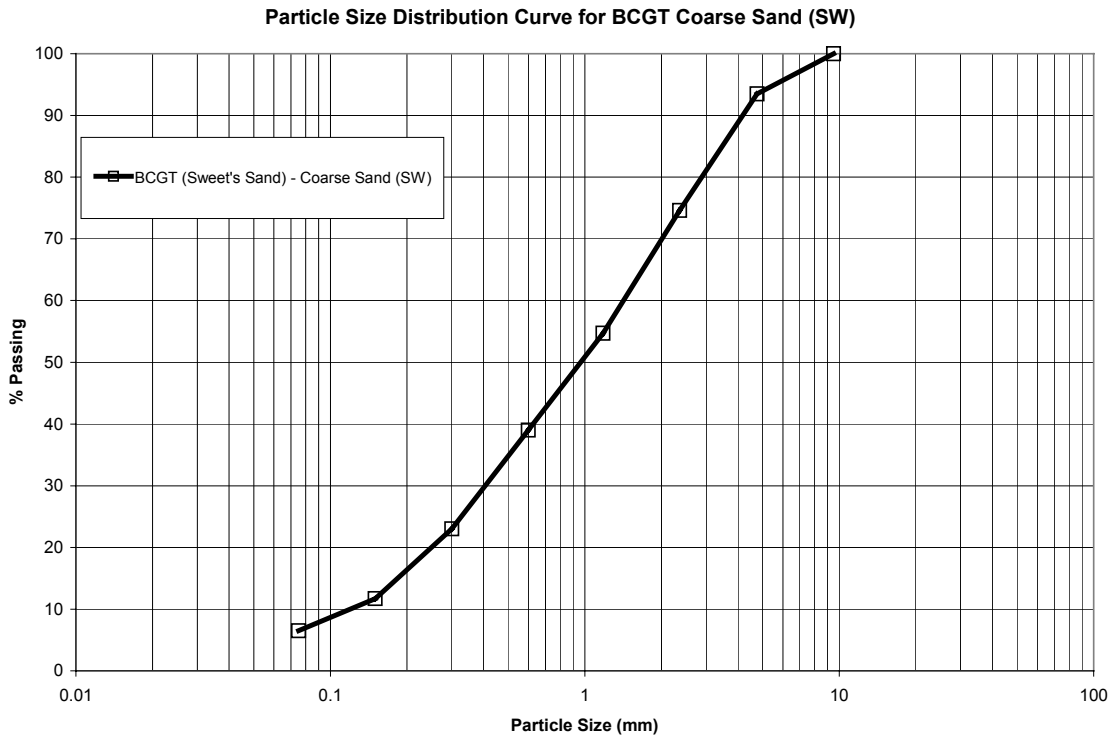


Figure 3: Particle size distribution for coarse sand (SW).



Figure 4: Photograph of a 900 mm-long Tensar UX1100MSE geogrid specimen showing the tell-tale locations.

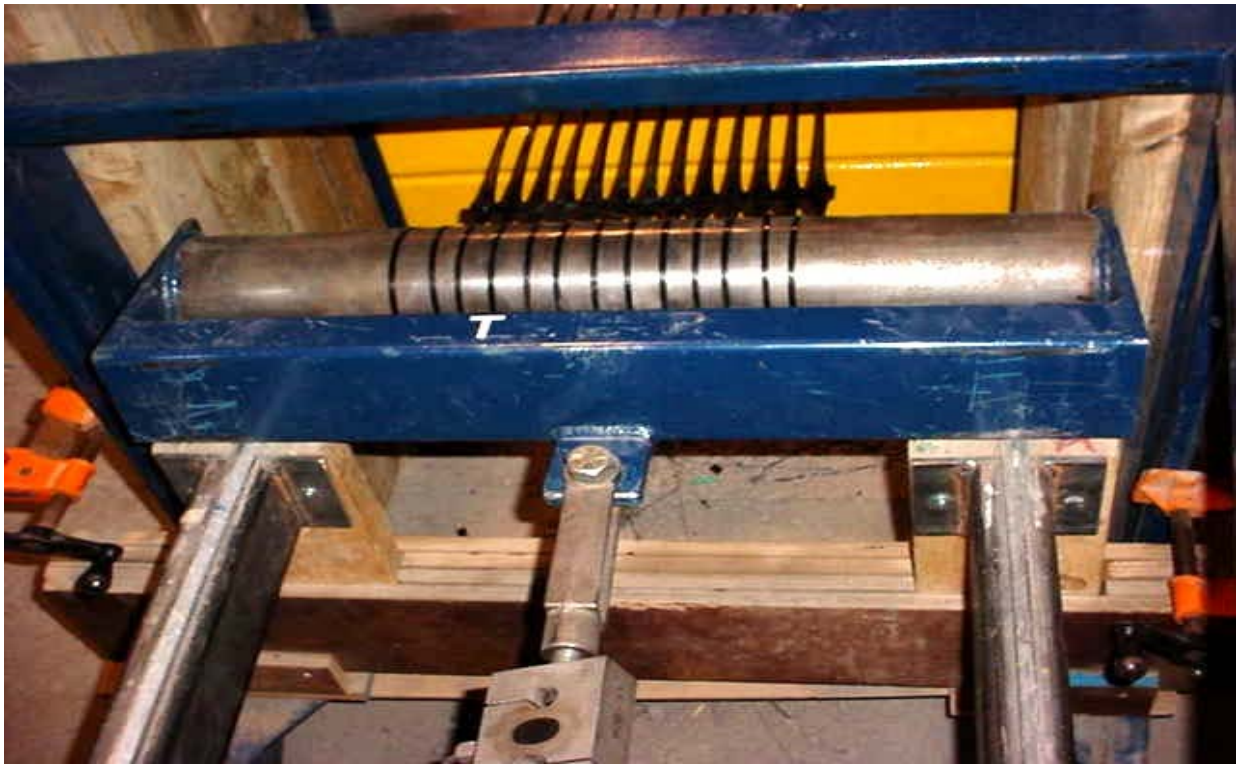


Figure 5: View of clamping arrangement at front of pullout box.

Figure 6: Pullout Capacity vs Confining Pressure for Tensar UX1100MSE Geogrid with Coarse Sand (SW)

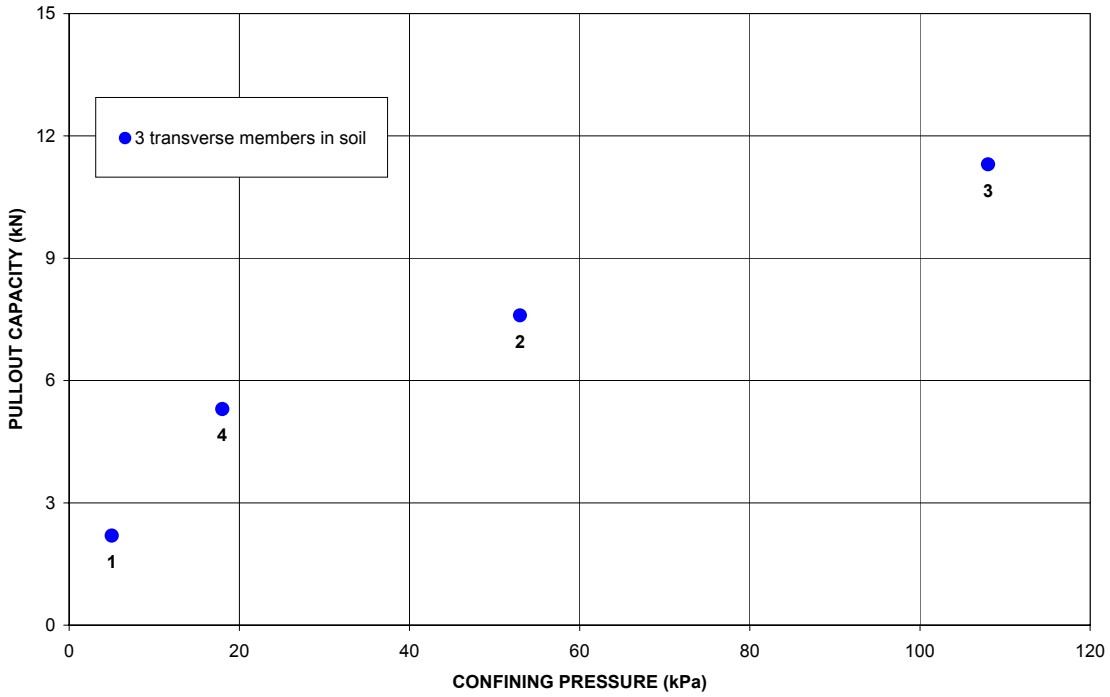




Figure 7: Exhumed Tensar UX1100MSE geogrid specimen (900 mm long) from Test 1, after testing under a confining pressure of 5 kPa.



Figure 8: Exhumed Tensar UX1100MSE geogrid specimen (900 mm long) from Test 2, after testing under a confining pressure of 53 kPa.

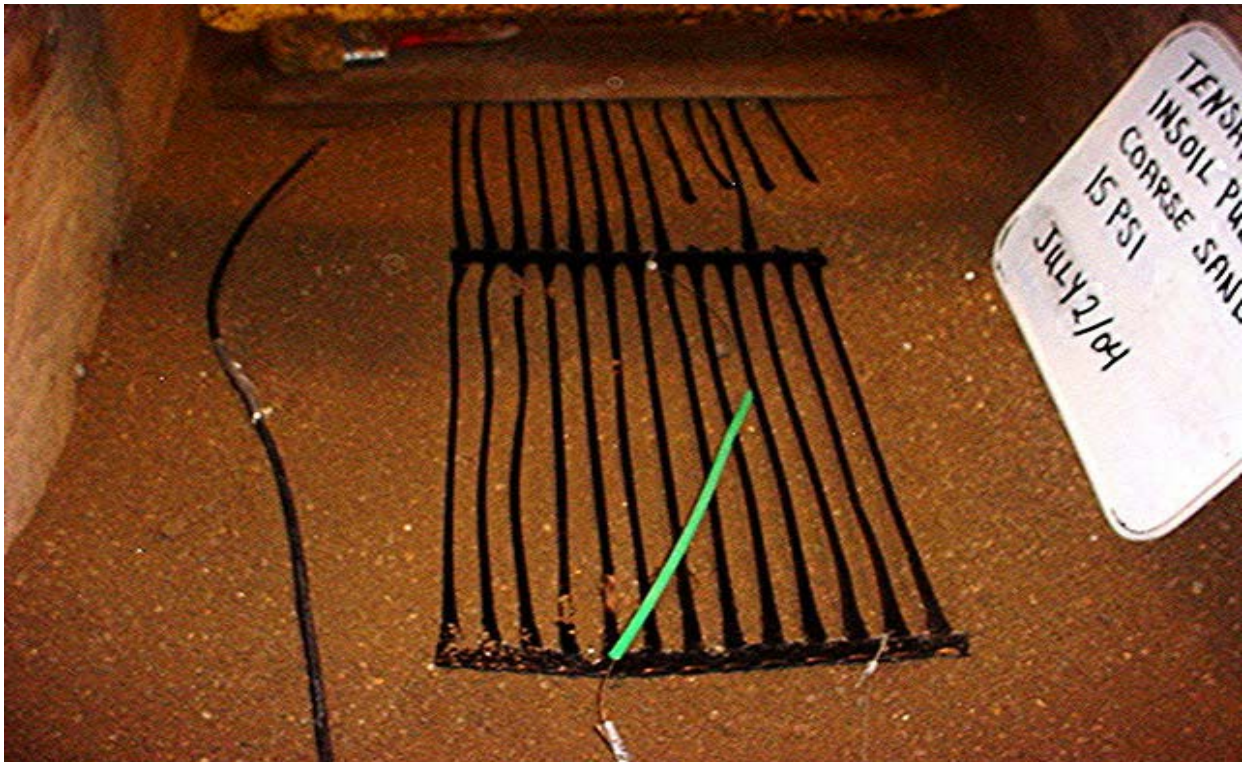


Figure 9: Exhumed Tensar UX1100MSE geogrid specimen (900 mm long) from Test 3, after testing under a confining pressure of 108 kPa.



Figure 10: Exhumed Tensar UX1100MSE geogrid specimen (900 mm long) from Test 4, after testing under a confining pressure of 18 kPa.

APPENDIX A

Figure A1: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with Coarse Sand (SW) Test 1 (5 kPa Confining Pressure)

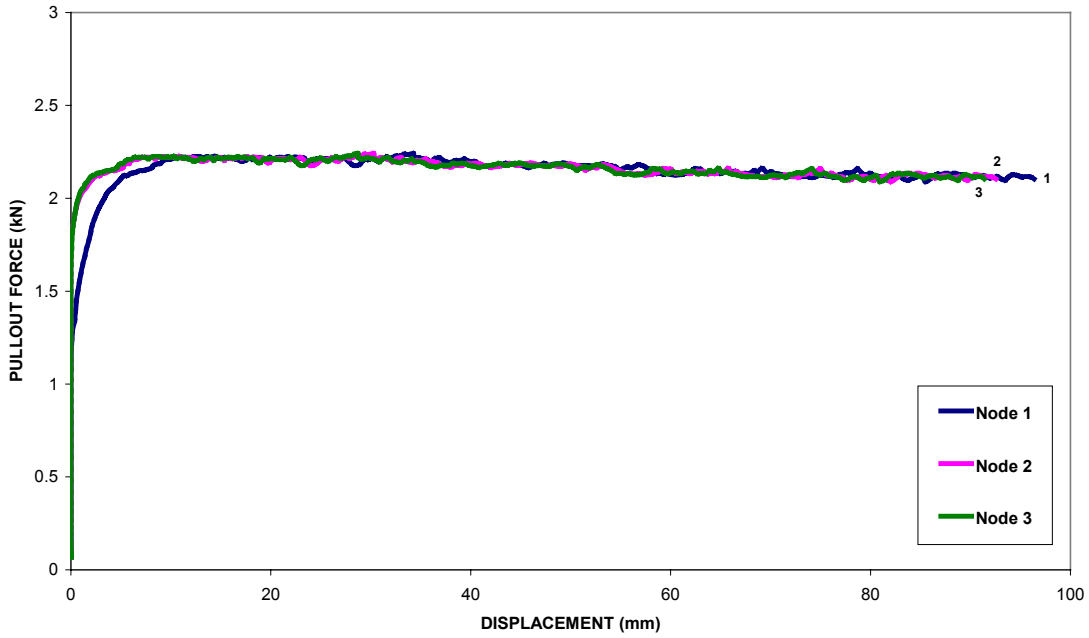


Figure A2: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with Coarse Sand (SW) Test 2 (53 kPa Confining Pressure)

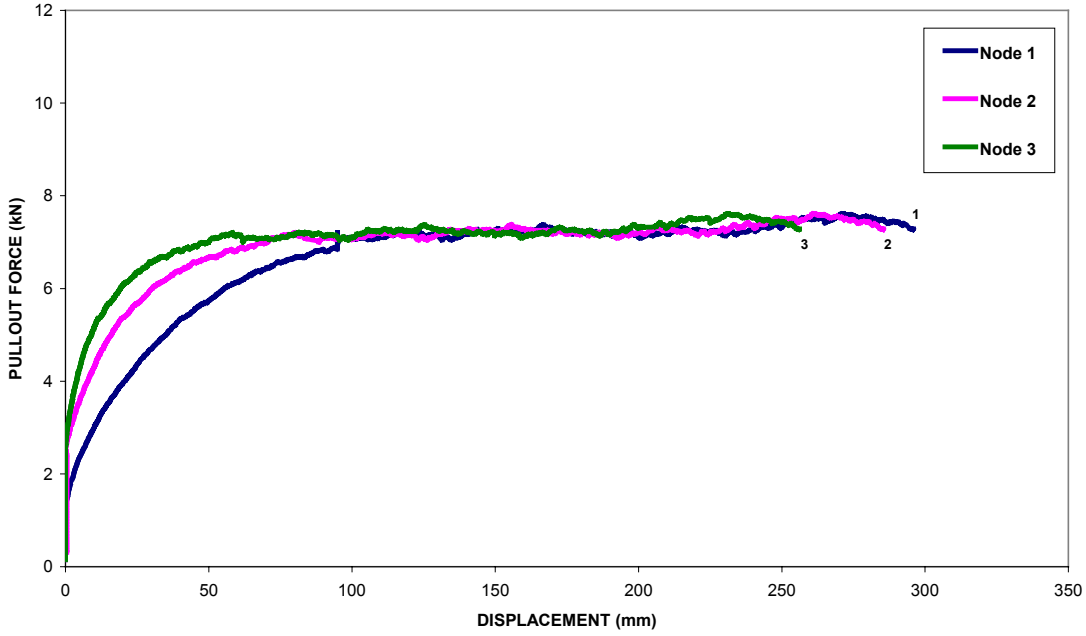


Figure A3: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with Coarse Sand (SW) Test 3 (108 kPa Confining Pressure)

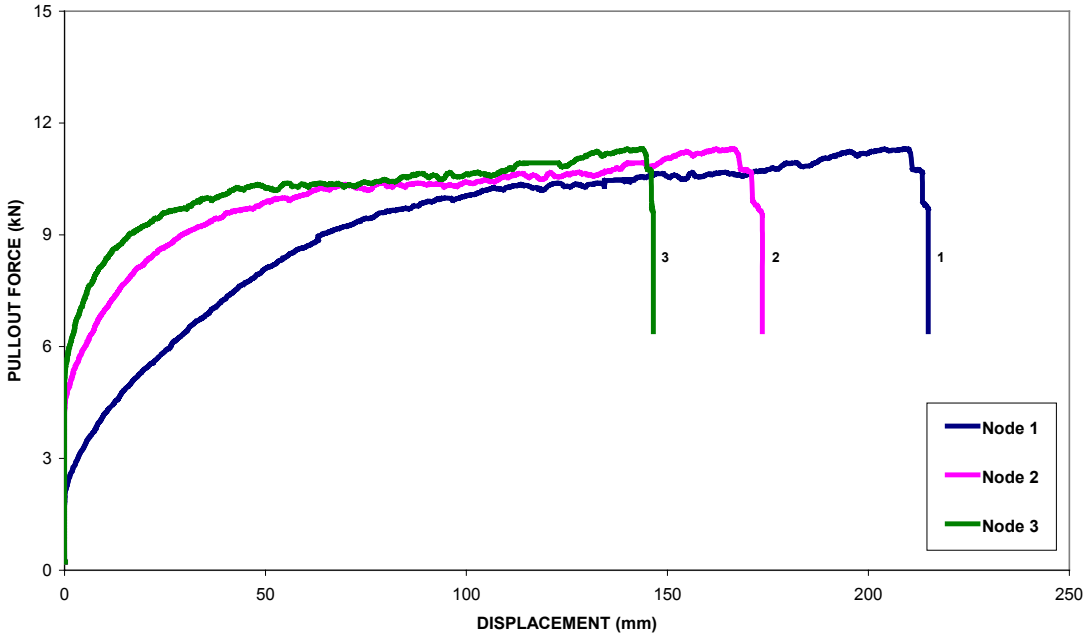
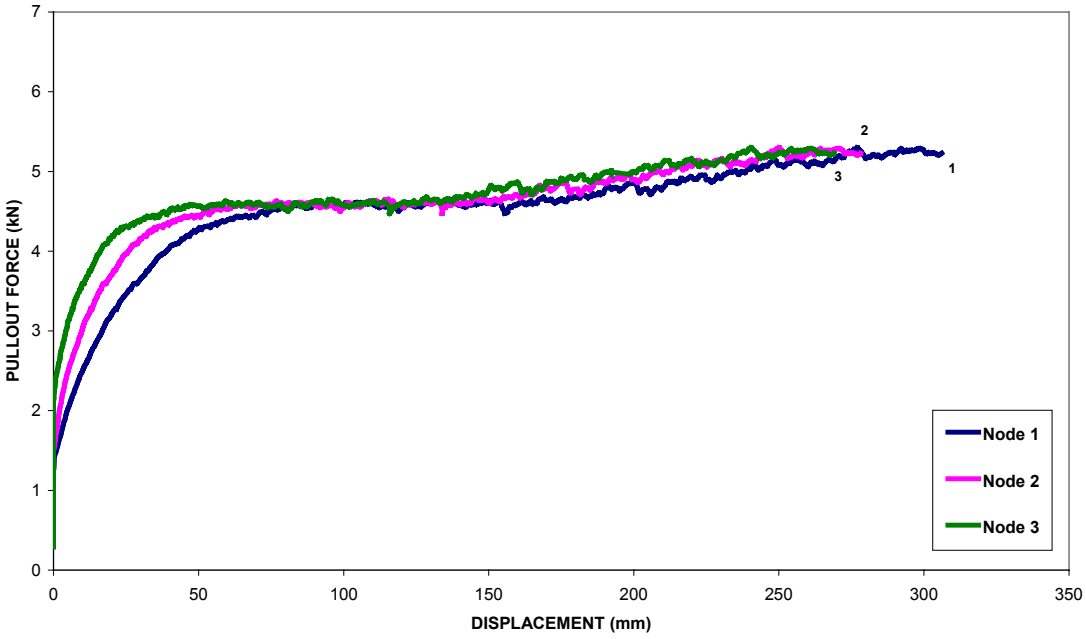


Figure A4: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with Coarse Sand (SW) Test 4 (18 kPa Confining Pressure)



Appendix 1.2.7.b

Pullout Tests in Soil Report

Tensar UX1100MSE Geogrids / Crushed Stone

MESA® MSE Retaining Wall System

Back to main document

FINAL REPORT

RESULTS OF

IN-SOIL PULLOUT TESTING

WITH

Tensar UX1100MSE Geogrid

In < 65 mm Crushed Stone (GW)

Submitted to

Tensar Earth Technologies Inc.

Distribution:

2 copies — Tensar Earth Technologies Inc.
5883 Glenridge Dr., Ste. 200
Atlanta GA 30328
USA

2 copies — Bathurst, Clarabut Geotechnical Testing, Inc.
1167 Clyde Court, Kingston, Ontario
K7P 2E4
CANADA

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Introduction

This report gives the results of an in-soil pullout testing program that was carried out to evaluate the in-soil performance of Tensar UX1100MSE geogrid soil reinforcement in <65 mm crushed stone (GW). The test program was initiated in response to a verbal authorization to proceed from Mr. Andy Anderson of Tensar Earth Technologies Inc., received 15 March 2003. The tests were carried out at the laboratories of Bathurst, Clarabut Geotechnical Testing, Inc. in Kingston, Ontario, under the supervision of Mr. Peter Clarabut.

Objectives of test program

The principal objective of the testing was to evaluate the in-soil pullout capacity of Tensar UX1100MSE geogrid embedded in <65 mm crushed stone (GW), while subjected to a range of confining pressures. The pullout tests were performed using the large pullout test apparatus shown in **Figures 1** and **2**.

Materials

Tensar UX1100MSE geogrid is a uni-axial geogrid with a reported MARV strength of 58.0 kN/m in the machine direction in accordance with GRI GG1 and tested at a constant rate of displacement of 10% of the gage length per minute. Specimens were cut from Roll/Lot #18673-127 (Tensar Designation) received at our laboratory on 15 March 2004. Based on tests performed in our laboratory a tensile strength of 61.1 kN/m was determined.

The particle size distribution curve for the soil used in this investigation is shown in **Figure 3**. The constant volume (residual) friction angle of the soil is 36 degrees.

Pullout box and test methodology

The test apparatus and method of test was in accordance with Draft ASTM Method of Test - Measuring Geosynthetic Pullout Resistance in Soil - ASTM Designation Number Z2467Z Draft No. 8 dated January 2001.

The pullout box is approximately 2290 mm long by 710 mm high (outside dimension). The inside width of the box is 760 mm (**Figure 2**). The soil surface was surcharged using a Kevlar reinforced polymer air bag to simulate variable depths of overburden soil above the test specimen. The bag allows a uniform surcharge pressure of up to 200 kPa to be applied to the surface of the soil. The pullout box surcharging arrangement is self-reacting using a system of steel angles that encompass the box. Columns of 50 mm-wide hollow steel sections slotted into each end of the test apparatus are used to confine the soil at the front and back of the pullout box. An adjustable pair of steel sleeves is used to pass the test specimen through the front end of the pullout box. The sidewalls of the pullout box are constructed from 25 mm-thick plywood that is covered on the inside (soil side) with a friction-reducing polymer liner.

For each test, a 286 mm-wide (13 longitudinal strands) geogrid specimen was used. In **Tests 1** and **2** the reinforcement extended into the soil box a distance up to approximately 1400 mm (4 transverse members). In **Tests 3** through **6** the geogrid specimen extended approximately 900 mm into the soil box (3 transverse members). For all tests, each specimen was placed over a 305 mm-deep layer of compacted soil. Steel wire tell-tales were attached to at least 3 nodal points along the length of each specimen (see **Figure 4**) and displacements recorded using potentiometer-type displacement transducers. For **Tests 1** and **2** the nodal points were located at 25 mm, 480 mm, 935 mm and 1375 mm from the inside front sleeve of the pullout box. For **Tests 3** through **6** the nodal points were located at 25 mm, 480 mm, and 935 mm from the inside front of the pullout box. The entire specimen was then covered by a 305 mm-depth of soil and compacted. The soil above and below the specimen elevation was compacted in 150 mm lifts using an electric Kango impact hammer and compacting plate. The compacted unit densities and moisture content were recorded using a nuclear density meter. Average measurement results are reported in **Table 1**.

The reinforcement clamp (**Figure 5**) was attached to a computer-controlled electro-mechanical screw jack actuator with a maximum capacity of about 50 kN. The rate of displacement of the front clamp was kept constant at 1 mm/minute. The tensile load and displacements recorded by the actuator load cell and the potentiometers were monitored continuously by a computer-controlled data acquisition system.

Test results

As-compacted soil properties for the <65 mm crushed stone (GW) are summarized in **Table 1**.

Peak pullout capacities are summarized in **Table 2**.

A pullout capacity versus confining pressure plot is presented in **Figure 6**.

Photographs of exhumed test specimens are shown in **Figures 7** through **11**.

Pullout force versus displacement plots for each test are presented in **Figures A1** through **A6** of **Appendix A**.

In **Tests 1 and 4** pullout/rupture of the geogrid specimen occurred.

In **Test 5** rupture of the geogrid specimen occurred.

Pullout of the geogrid specimen occurred in **Tests 2, 3** and **6**.

Coefficient of interaction (C_i) values were calculated as follows based on recommendations in the GRI GG5 protocol:

$$C_i = P/2WL\sigma_n \tan\phi \leq 1.0$$

where:

P = pullout load (kN)
W = width of specimen (m)
L = length of specimen (m)
σ_n = normal (confining) pressure (kPa)
ϕ = residual friction angle of soil (degrees)

Using the expression above, all tests that ended in reinforcement pullout gave $C_i = 1.0$ as shown in **Table 2**.



Richard J. Bathurst Ph.D., P.Eng.

REFERENCES

GRI Test Method GG5 “Geogrid Pullout” dated 30 January 1991

Draft ASTM Method of Test - Measuring Geosynthetic Pullout Resistance in Soil - ASTM Designation Number Z2467Z Draft No. 8 dated January 2001

Table 1: As-compacted soil properties

Soil	Bulk Unit Density (kg/m^3)	Moisture Content (%)	Residual Friction Angle
<65 mm crushed stone (GW)	1756	2.2	36°

Table 2:
<65 mm crushed stone (GW) – Tensar UX1100MSE Geogrid in-soil pullout test results

Test No.	File Name	Date	Max Pullout Resistance (kN)	Confining Pressure (kPa)	Specimen Length in Soil (m)	Failure Type	C_i
1	TUX1101	Mar 19/04	11.8	12	1.4	Pullout/Rupture	n/a
2	TUX1102	Mar 22/04	7.5	5	1.4	Pullout	1.0
3	TUX1103	Mar 23/04	8.7	12	0.9	Pullout	1.0
4	TUX1104	Mar 24/04	10.6	19	0.9	Pullout/Rupture	n/a
5	TUX1105	Mar 25/04	11.4	26	0.9	Rupture	n/a
6	TUX1106	Mar 26/04	4.4	5	0.9	Pullout	1.0



Figure 1: Pullout box apparatus.

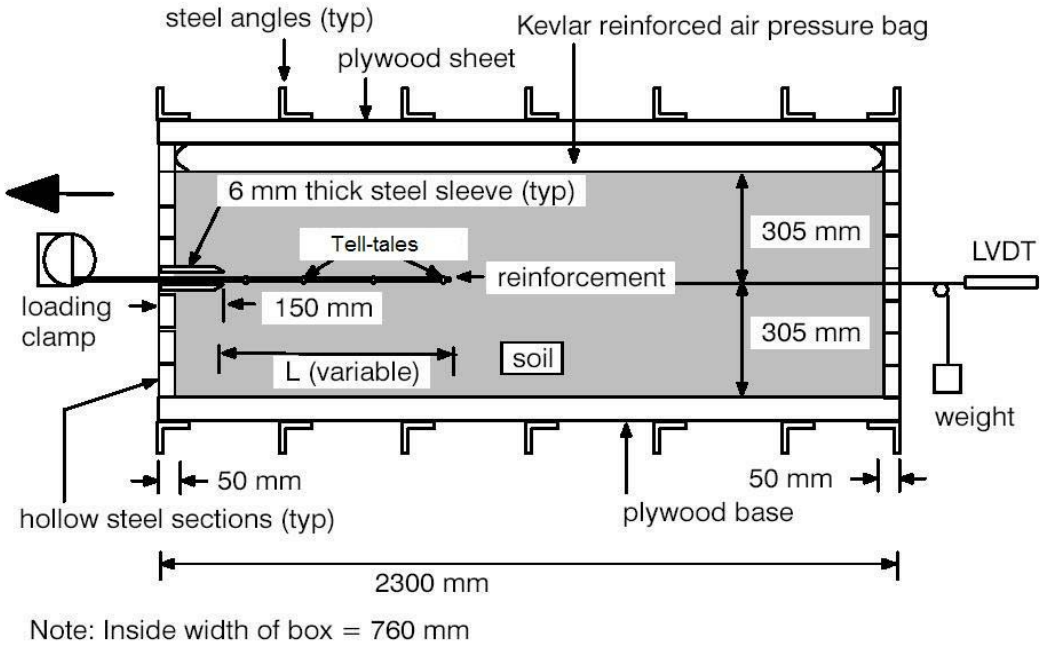


Figure 2: Cross-section view of in-soil pullout box apparatus and general arrangement of soil reinforcement.

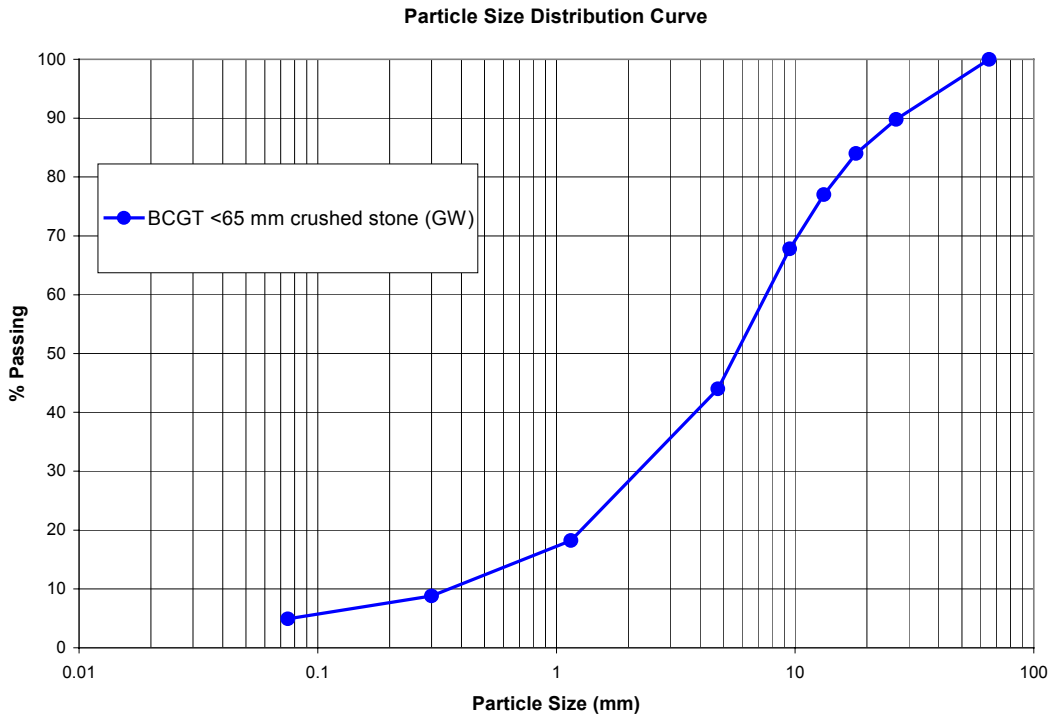


Figure 3: Particle size distribution for <65 mm crushed stone (GW).



Figure 4: Photographs of 900 mm-long (Left) and 1400 mm-long (Right) Tensar UX1100MSE geogrid specimens used in this investigation.

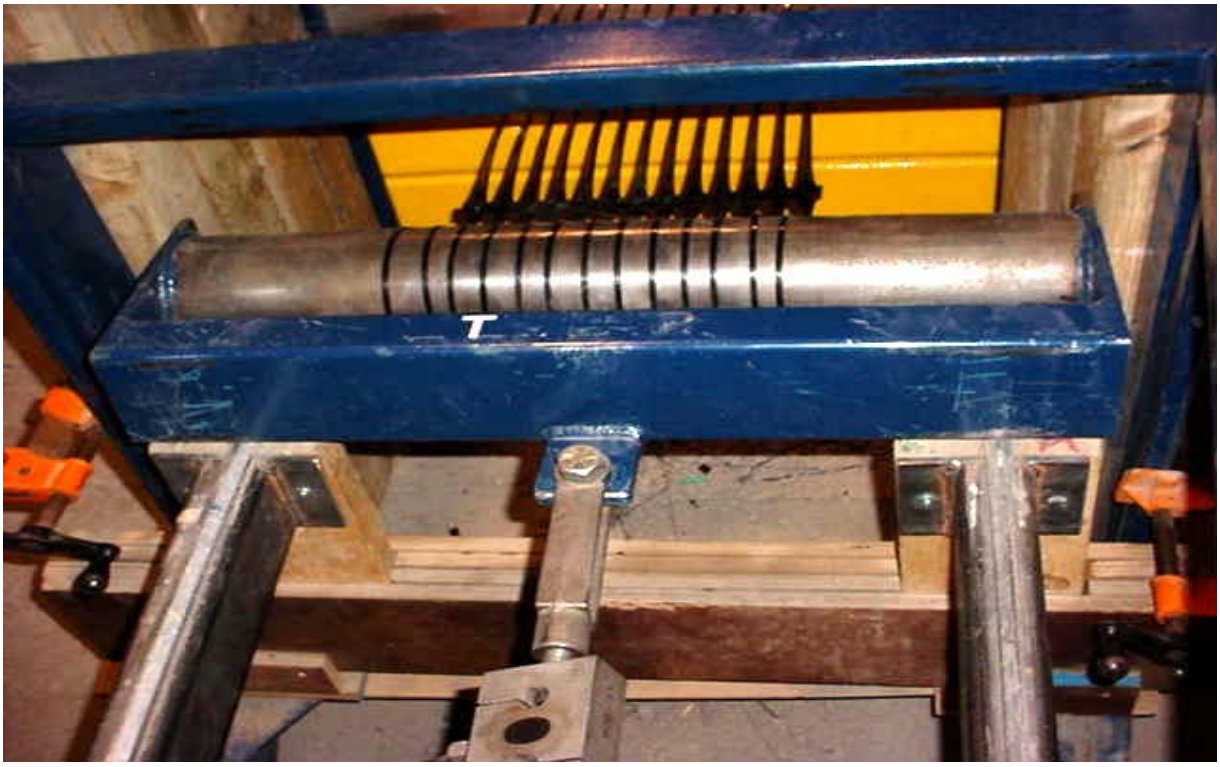


Figure 5: View of clamping arrangement at front of pullout box.

Figure 6: Pullout Capacity vs Confining Pressure for Tensar UX1100 Quint Geogrid with <65 mm Crushed Stone (GW)

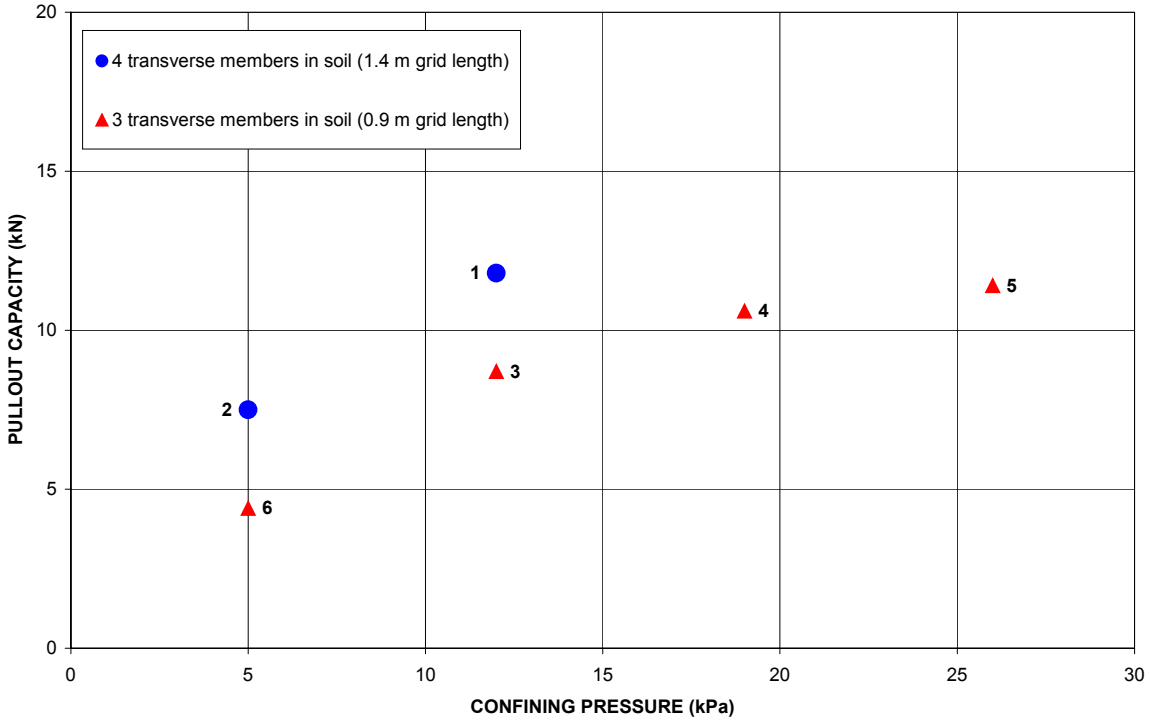




Figure 7: Exhumed Tensar UX1100MSE geogrid specimen (1400 mm long) from Test 1, after testing under a confining pressure of 12 kPa.



Figure 8: Exhumed Tensar UX1100MSE geogrid specimen (1400 mm long) from Test 2, after testing under a confining pressure of 5 kPa.



Figure 9: Exhumed Tensar UX1100MSE geogrid specimen (900 mm long) from Test 3, after testing under a confining pressure of 12 kPa.



Figure 10: Exhumed Tensar UX1100MSE geogrid specimen (900 mm long) from Test 4, after testing under a confining pressure of 19 kPa.



Figure 11: Exhumed Tensar UX1100MSE geogrid specimen (900 mm long) from Test 5, after testing under a confining pressure of 26 kPa.

APPENDIX A

Figure A1: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with <65 mm Crushed Stone (GW) Test 1 (12 kPa Confining Pressure)

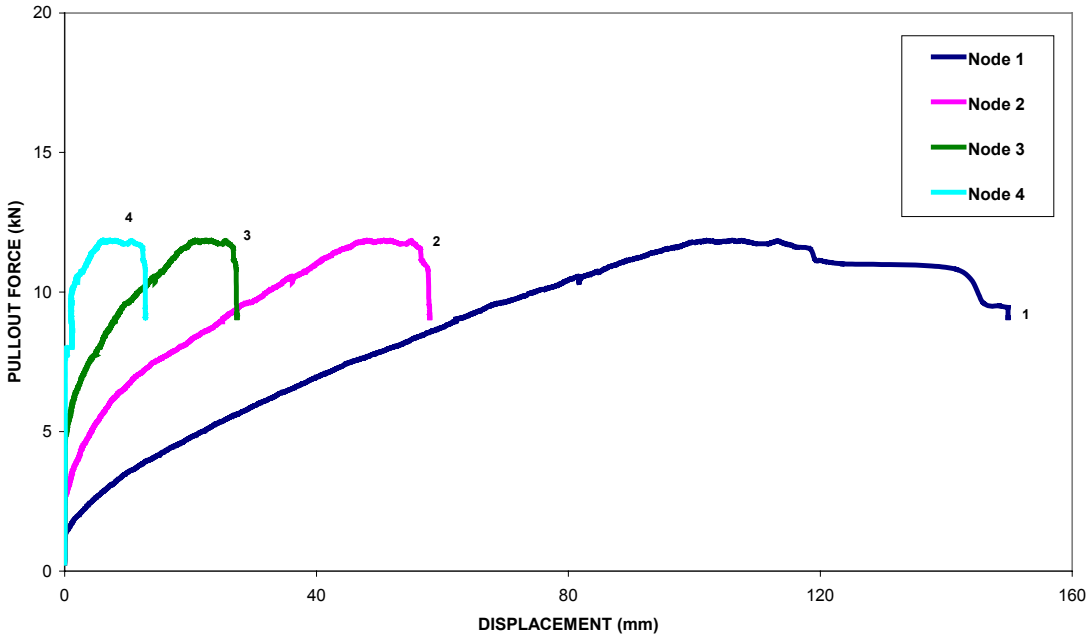


Figure A2: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with <65 mm Crushed Stone (GW) Test 2 (5 kPa Confining Pressure)

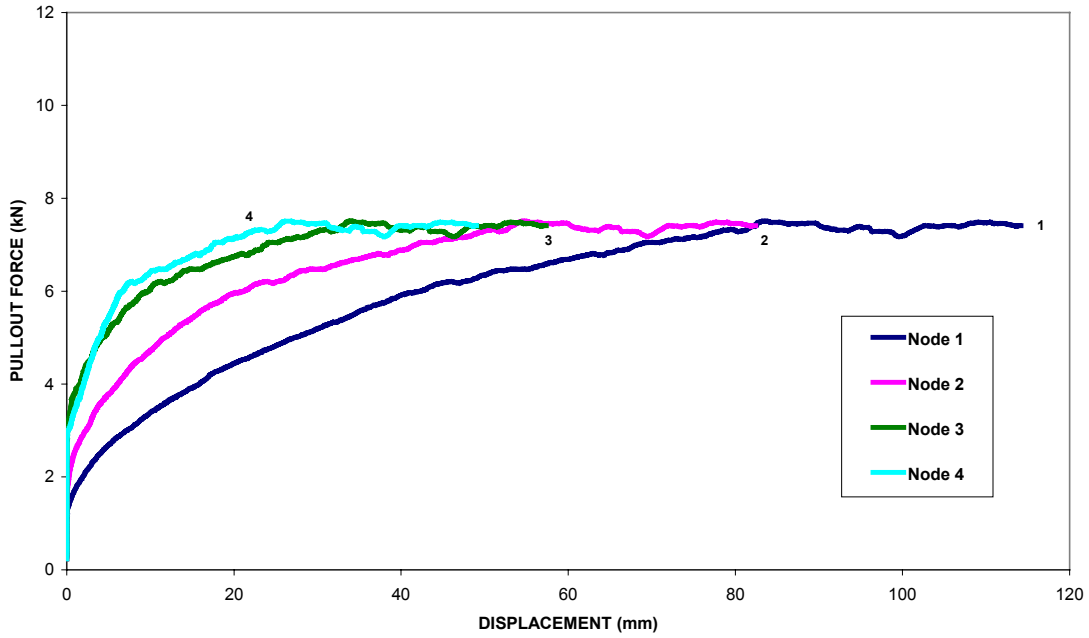


Figure A3: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with <65 mm Crushed Stone (GW) Test 3 (12 kPa Confining Pressure)

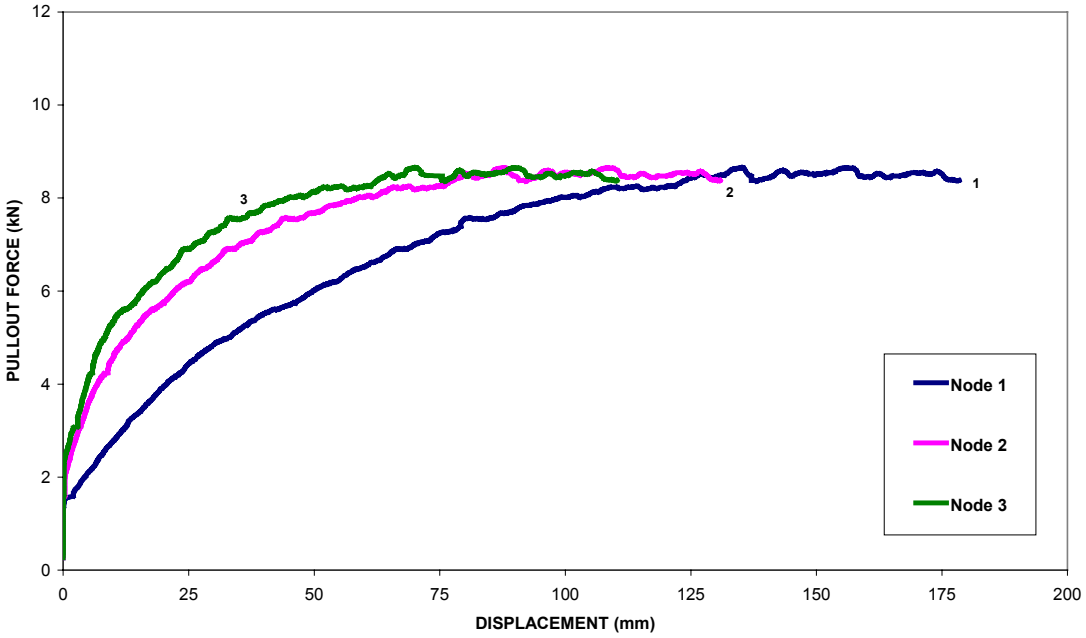


Figure A4: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with <65 mm Crushed Stone (GW) Test 4 (19 kPa Confining Pressure)

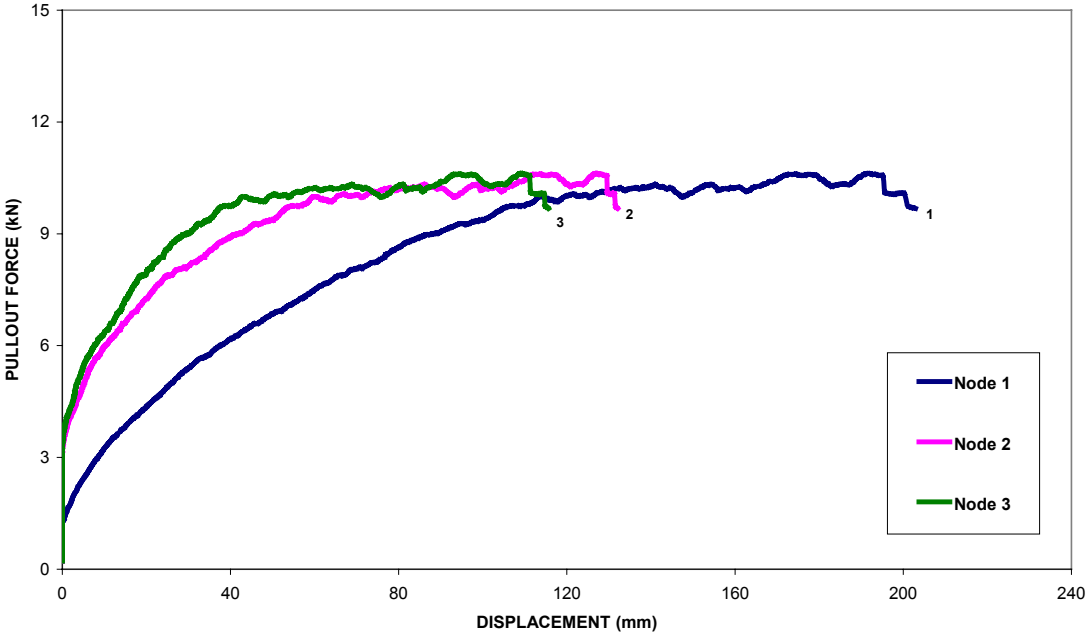


Figure A5: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with <65 mm Crushed Stone (GW) Test 5 (26 kPa Confining Pressure)

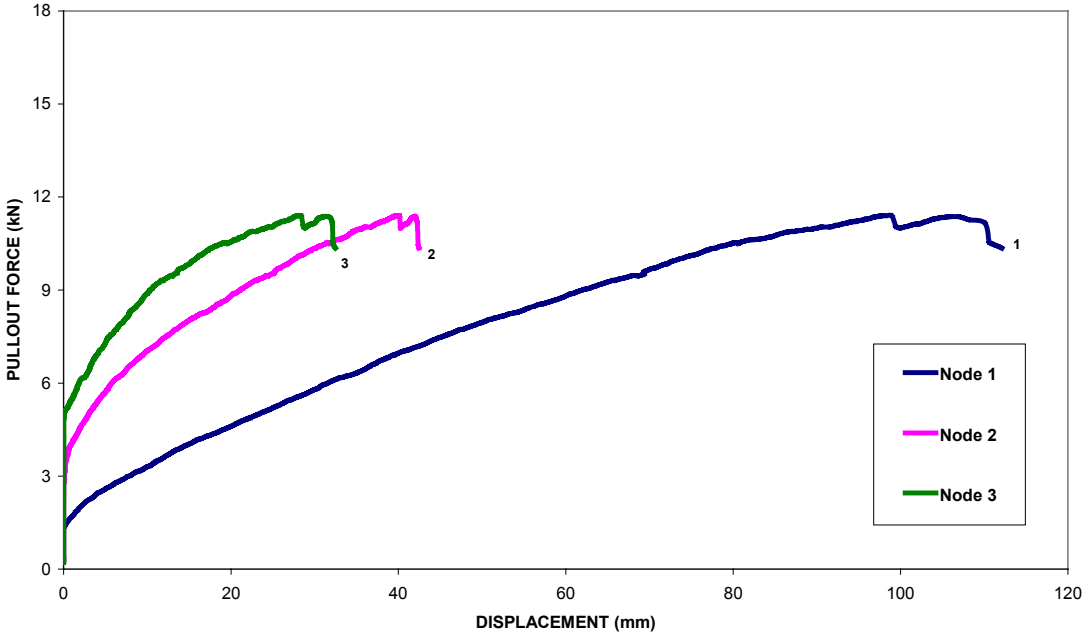
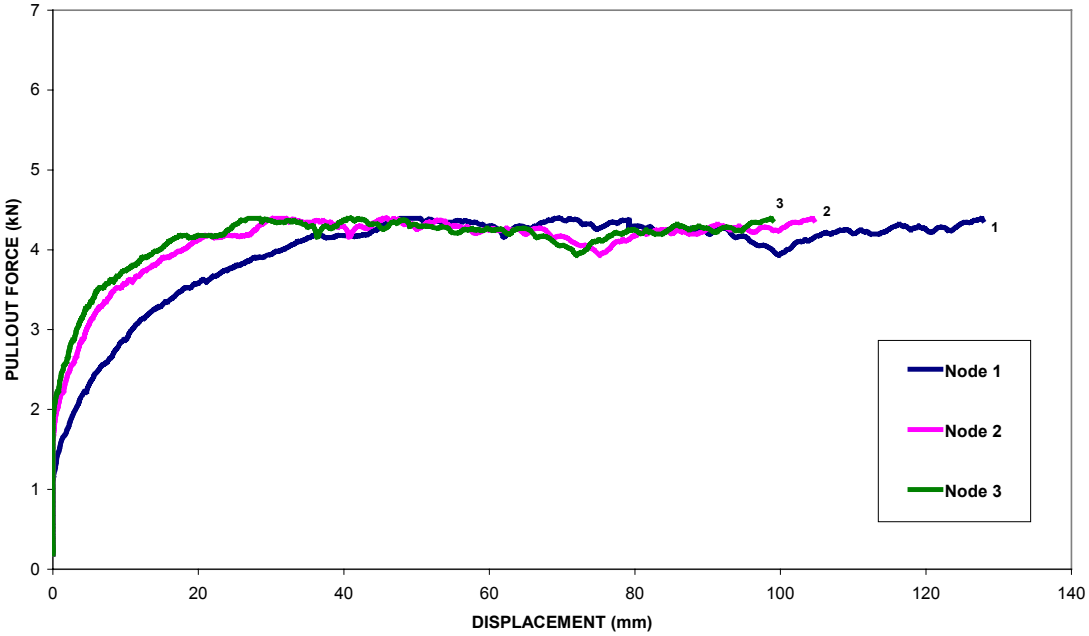


Figure A6: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with <65 mm Crushed Stone (GW) Test 6 (5 kPa Confining Pressure)



Appendix 1.2.7.c

Pullout Tests in Soil Report

Tensar UX1100MSE Geogrids / Fine Silty Sand

MESA® MSE Retaining Wall System

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FINAL REPORT

RESULTS OF

IN-SOIL PULLOUT TESTING

WITH

Tensar UX1100MSE Geogrid

In < 2 mm Fine Silty Sand (SM)

Submitted to

Tensar Earth Technologies Inc.

Distribution:

2 copies Tensar Earth Technologies Inc.
5883 Glenridge Dr., Ste. 200
Atlanta GA 30328
USA

2 copies Bathurst, Clarabut Geotechnical Testing, Inc.
1167 Clyde Court, Kingston, Ontario
K7P 2E4
CANADA

This report shall not be reproduced except in full, without the prior written approval of Bathurst, Clarabut Geotechnical Testing, Inc.

Introduction

This report gives the results of an in-soil pullout testing program that was carried out to evaluate the in-soil performance of Tensar UX1100MSE geogrid soil reinforcement in <2 mm fine silty sand (SM). The test program was initiated in response to a verbal authorization to proceed from Mr. Andy Anderson of Tensar Earth Technologies Inc., received 15 March 2004. The tests were carried out at the laboratories of Bathurst, Clarabut Geotechnical Testing, Inc. in Kingston, Ontario, under the supervision of Mr. Peter Clarabut.

Objectives of test program

The principal objective of the testing was to evaluate the in-soil pullout capacity of Tensar UX1100MSE geogrid embedded in <2 mm fine silty sand (SM), while subjected to a range of confining pressures. The pullout tests were performed using the large pullout test apparatus shown in **Figures 1** and **2**.

Materials

Tensar UX1100MSE geogrid is a uni-axial geogrid with a reported MARV strength of 58.0 kN/m in the machine direction in accordance with GRI GG1 and tested at a constant rate of displacement of 10% of the gage length per minute. Specimens were cut from Roll/Lot #18673-127 (Tensar Designation) received at our laboratory on 15 March 2004. Based on tests performed in our laboratory a tensile strength of 61.1 kN/m was determined.

The particle size distribution curve for the soil used in this investigation is shown in **Figure 3**. The constant volume (residual) friction angle of the soil is 27 degrees.

Pullout box and test methodology

The test apparatus and method of test was in accordance with Draft ASTM Method of Test - Measuring Geosynthetic Pullout Resistance in Soil - ASTM Designation Number Z2467Z Draft No. 8 dated January 2001.

The pullout box is approximately 2290 mm long by 710 mm high (outside dimension). The inside width of the box is 760 mm (**Figure 2**). The soil surface was surcharged using a Kevlar reinforced polymer air bag to simulate variable depths of overburden soil above the test specimen. The bag allows a uniform surcharge pressure of up to 200 kPa to be applied to the surface of the soil. The pullout box surcharging arrangement is self-reacting using a system of steel angles that encompass the box. Columns of 50 mm-wide hollow steel sections slotted into each end of the test apparatus are used to confine the soil at the front and back of the pullout box. An adjustable pair of steel sleeves is used to pass the test specimen through the front end of the pullout box. The sidewalls of the pullout box are constructed from 25 mm-thick plywood that is covered on the inside (soil side) with a friction-reducing polymer liner.

For each test, a 286 mm-wide (13 longitudinal strands) geogrid specimen was used and extended approximately 900 mm into the soil box (3 transverse members). Each specimen was placed over a 305 mm-deep layer of compacted soil. Steel wire tell-tales were attached to at least 3 nodal points along the length of each specimen (see **Figure 4**) and displacements recorded using potentiometer-type displacement transducers. For all tests the nodal points were located at 25 mm, 480 mm, and 910 mm from the inside front sleeve of the pullout box. The entire specimen was then covered by a 305 mm-depth of soil and compacted. The soil above and below the specimen elevation was compacted in 150 mm lifts using an electric Kango impact hammer and compacting plate. The compacted unit densities and moisture content were recorded using a nuclear density meter. Average measurement results are reported in **Table 1**.

The reinforcement clamp (**Figure 5**) was attached to a computer-controlled electro-mechanical screw jack actuator with a maximum capacity of about 50 kN. The rate of displacement of the front clamp was kept constant at 1 mm/minute. The tensile load and displacements recorded by the actuator load cell and the potentiometers were monitored continuously by a computer-controlled data acquisition system.

Test results

As-compacted soil properties for the <2 mm fine silty sand (SM) are summarized in **Table 1**.

Peak pullout capacities are summarized in **Table 2**.

A pullout capacity versus confining pressure plot is presented in **Figure 6**.

Photographs of exhumed test specimens are shown in **Figures 7** through **11**.

Pullout force versus displacement plots for each test are presented in **Figures A1** through **A4** of **Appendix A**.

In **Test 4** pullout / rupture of the geogrid specimen occurred.

Pullout of the geogrid specimen occurred in **Tests 1, 2** and **3**.

Coefficient of interaction (C_i) values were calculated as follows based on recommendations in the GRI GG5 protocol:

$$C_i = P/2WL\sigma_n \tan\phi \leq 1.0$$

where:

P = pullout load (kN)
W = width of specimen (m)
L = length of specimen (m)
σ_n = normal (confining) pressure (kPa)
ϕ = residual friction angle of soil (degrees)

Using the expression above, all tests that ended in reinforcement pullout gave $C_i \leq 1.0$ as shown in **Table 2**.



Richard J. Bathurst Ph.D., P.Eng.

REFERENCES

GRI Test Method GG5 “Geogrid Pullout” dated 30 January 1991

Draft ASTM Method of Test - Measuring Geosynthetic Pullout Resistance in Soil - ASTM Designation Number Z2467Z Draft No. 8 dated January 2001

Table 1: As-compacted soil properties

Soil	Bulk Unit Density (kg/m^3)	Moisture Content (%)	Residual Friction Angle
<2 mm Fine Silty Sand (SM)	1750	3.3	27°

Table 2:
<2 mm fine silty sand (SM) – Tensar UX1100MSE Geogrid in-soil pullout test results

Test No.	File Name	Date	Max Pullout Resistance (kN)	Confining Pressure (kPa)	Specimen Length in Soil (m)	Failure Type	C_i
1	TUX10M1	Apr 27/04	4.4	19	0.9	Pullout	0.9
2	TUX10M2	Apr 28/04	8.3	53	0.9	Pullout	0.6
3	TUX10M3	Apr 28/04	2.2	5	0.9	Pullout	1.0
4	TUX10M4	Apr 29/04	12.8	109	0.9	Pullout/Rupture	n/a



Figure 1: Pullout box apparatus.

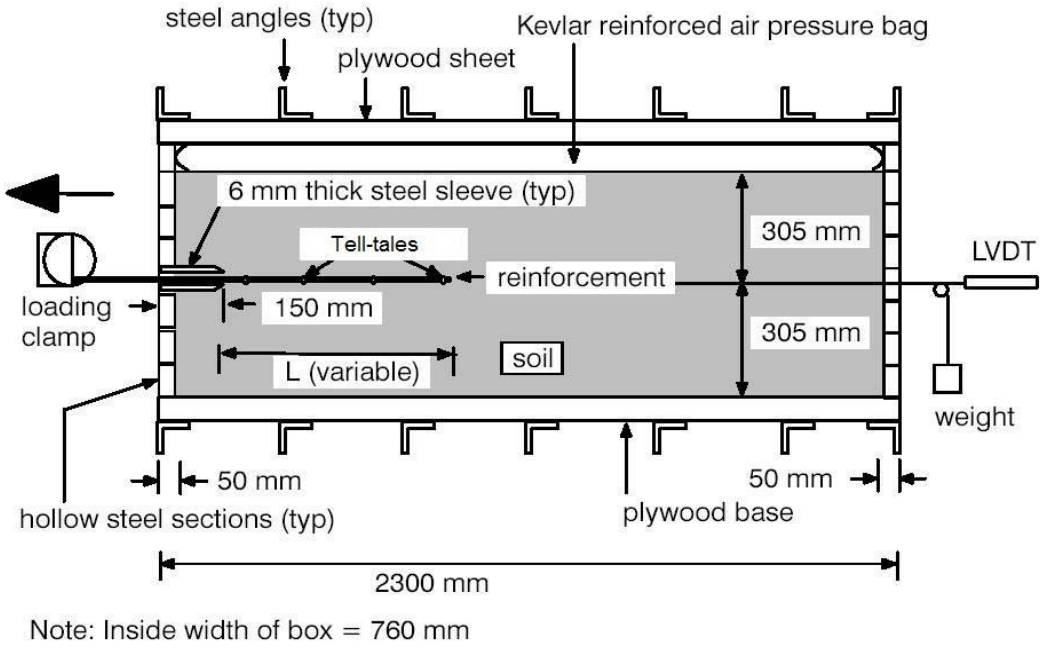


Figure 2: Cross-section view of in-soil pullout box apparatus and general arrangement of soil reinforcement.

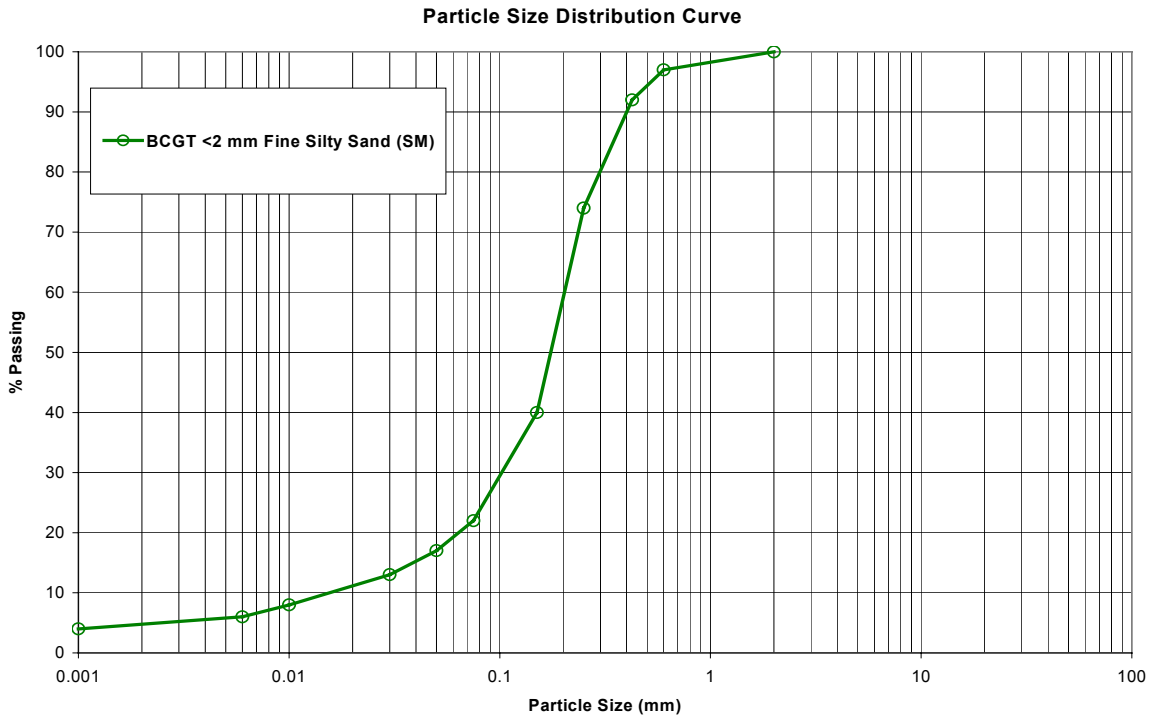


Figure 3: Particle size distribution for <2 mm fine silty sand (SM).



Figure 4: Photograph of a 900 mm-long Tensar UX1100MSE geogrid specimen showing the tell-tale locations.

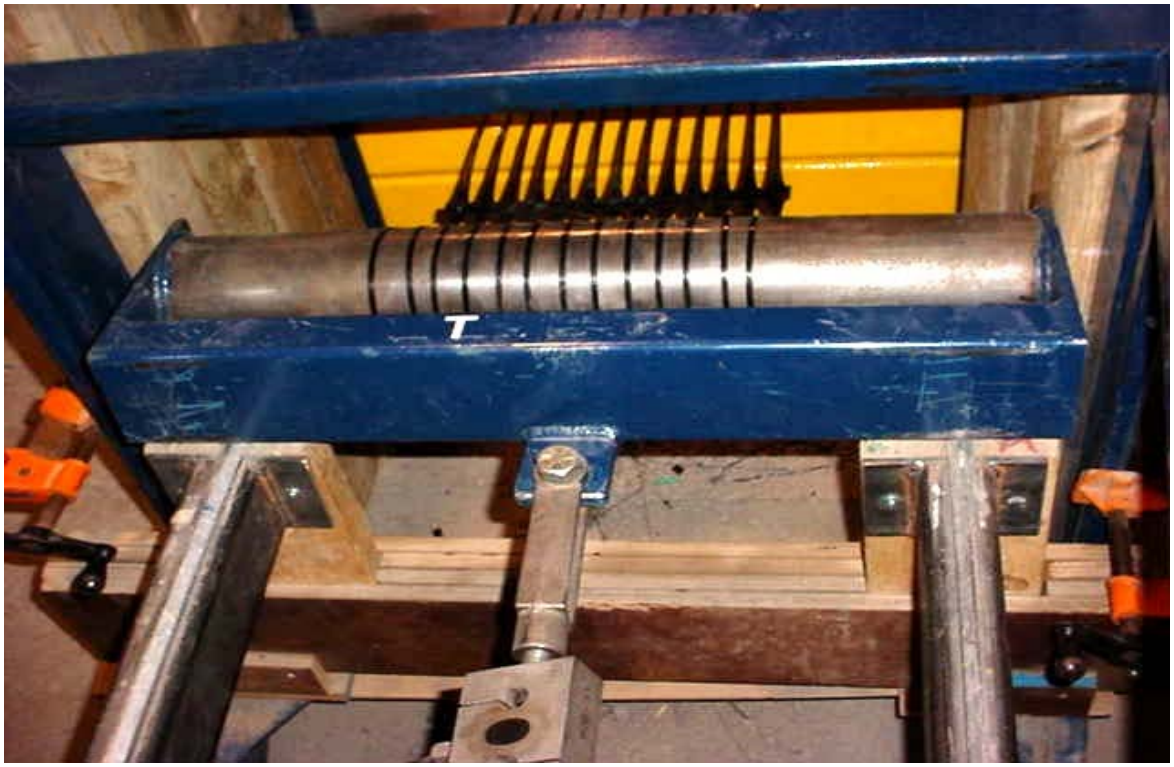


Figure 5: View of clamping arrangement at front of pullout box.

Figure 6: Pullout Capacity vs Confining Pressure for Tensar UX1100MSE Geogrid with <2 mm Fine Silty Sand (SM)

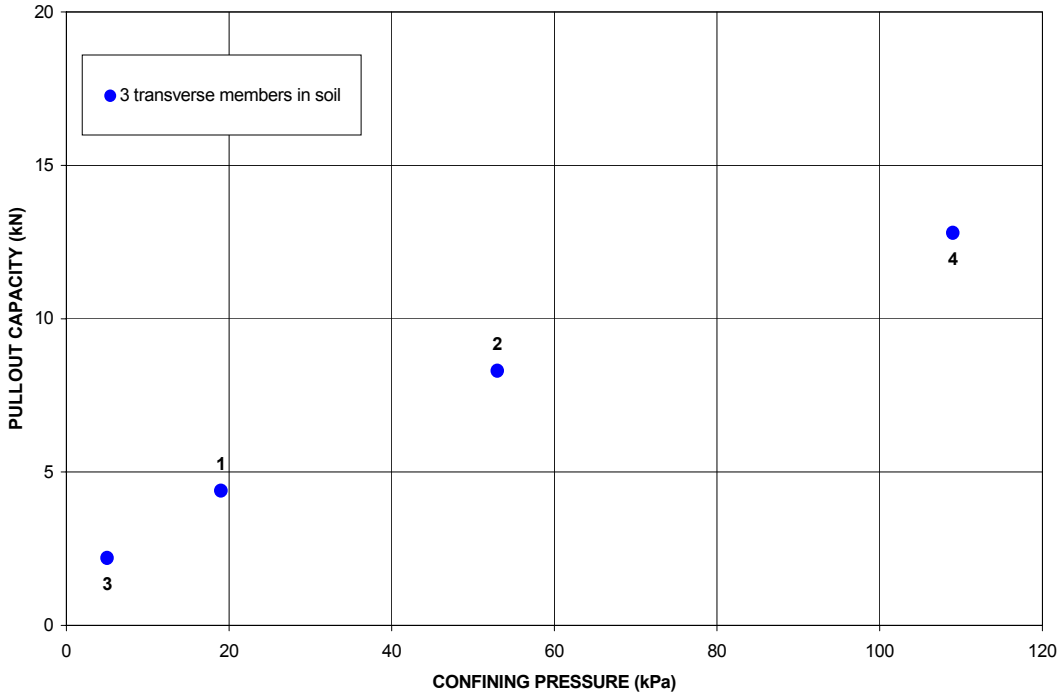




Figure 7: Exhumed Tensar UX1100MSE geogrid specimen (900 mm long) from Test 1, after testing under a confining pressure of 19 kPa.



Figure 8: Exhumed Tensar UX1100MSE geogrid specimen (900 mm long) from Test 2, after testing under a confining pressure of 53 kPa.



Figure 9: Exhumed Tensar UX1100MSE geogrid specimen (900 mm long) from Test 3, after testing under a confining pressure of 5 kPa.



Figure 10: Exhumed Tensar UX1100MSE geogrid specimen (900 mm long) from Test 4, after testing under a confining pressure of 109 kPa.



Figure 11: Exhumed Tensar UX1100MSE geogrid specimen (900 mm long) from Test 4, showing the rupture of 2 longitudinal strands after testing under a confining pressure of 109 kPa.

APPENDIX A

Figure A1: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with <2 mm Fine Silty Sand (SM) Test 1 (19 kPa Confining Pressure)

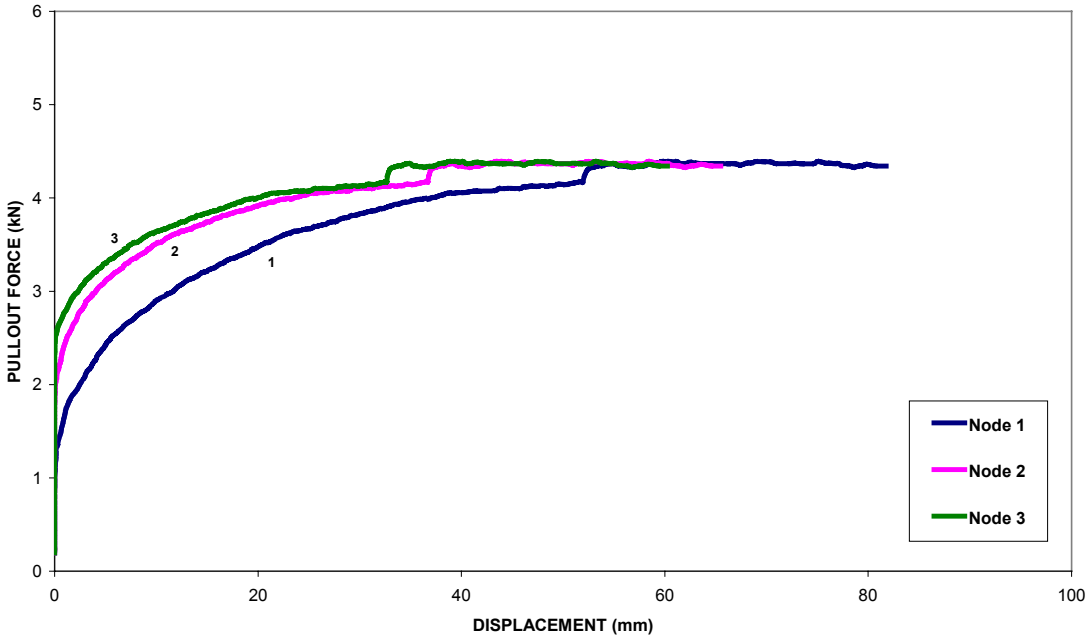


Figure A2: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with <2 mm Fine Silty Sand (SM) Test 2 (53 kPa Confining Pressure)

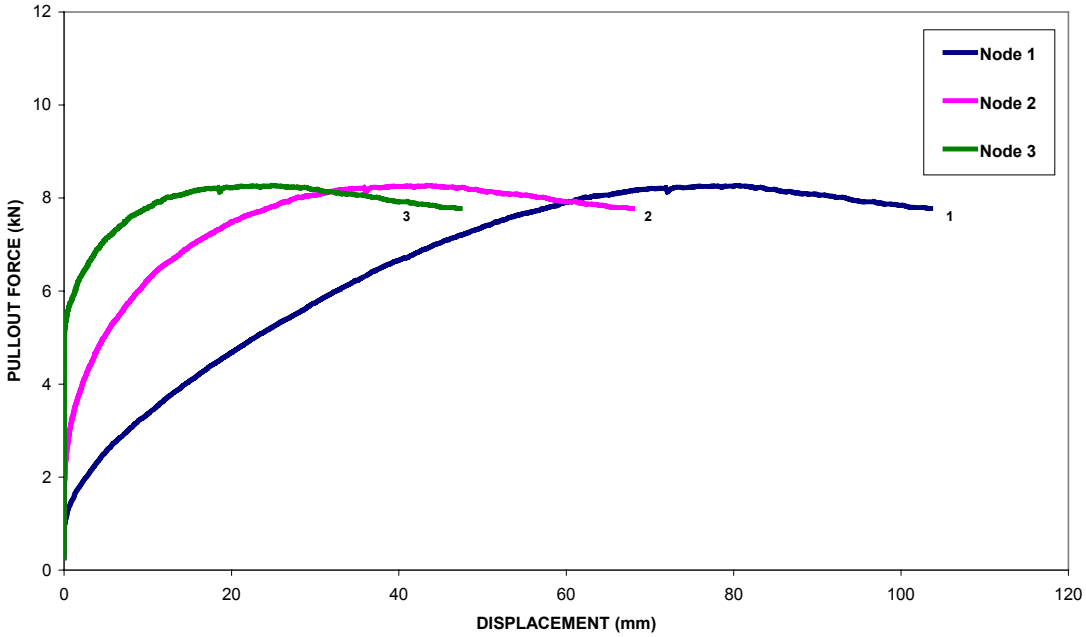


Figure A3: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with <2 mm Fine Silty Sand (SM) Test 3 (5 kPa Confining Pressure)

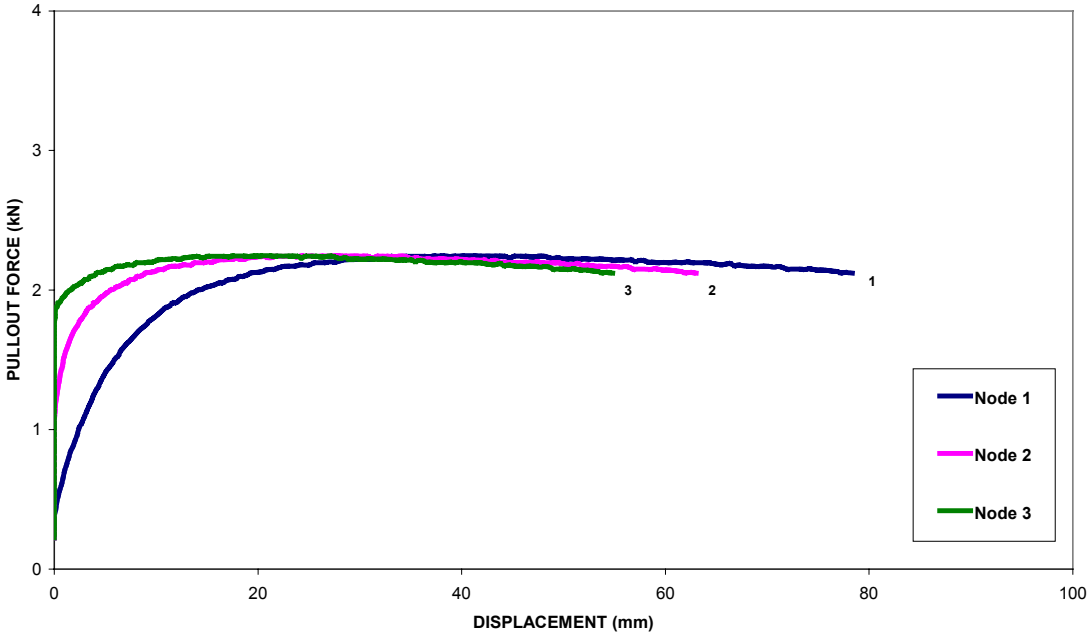
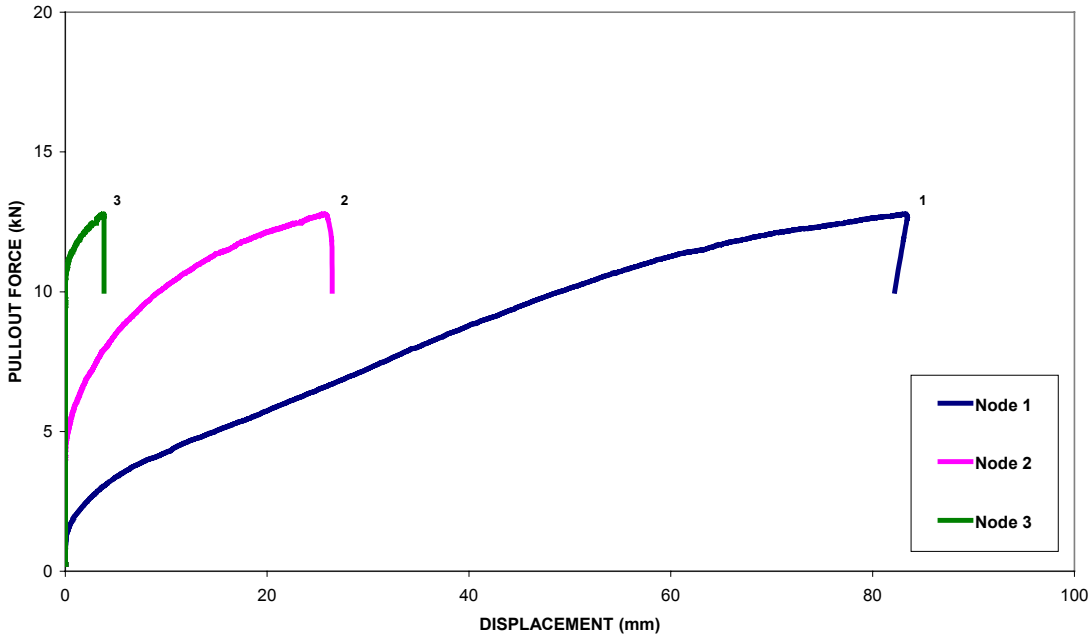


Figure A4: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with <2 mm Fine Silty Sand (SM) Test 4 (109 kPa Confining Pressure)



Appendix 1.2.7.d

Pullout Tests in Soil Report

Tensar UX1700MSE Geogrids / Course Sand

MESA® MSE Retaining Wall System

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FINAL REPORT

RESULTS OF

IN-SOIL PULLOUT TESTING

WITH

Tensar UX1700MSE Geogrid and Coarse Sand (SW)

Submitted to

Tensar Earth Technologies Inc.

CONFIDENTIAL

Distribution:

2 copies — Tensar Earth Technologies Inc.
5883 Glenridge Dr., Ste. 200
Atlanta GA 30328
USA

2 copies — Bathurst, Clarabut Geotechnical Testing, Inc.
1167 Clyde Court, Kingston, Ontario
K7P 2E4
CANADA

This report shall not be reproduced except in full, without the prior written approval of Bathurst, Clarabut Geotechnical Testing, Inc.

Introduction

This report gives the results of an in-soil pullout testing program that was carried out to evaluate the in-soil performance of Tensar UX1700MSE geogrid soil reinforcement in coarse sand (SW). The test program was initiated in response to a verbal authorization to proceed from Mr. Andy Anderson of Tensar Earth Technologies Inc., received 4 October 2004. The tests were carried out at the laboratories of Bathurst, Clarabut Geotechnical Testing, Inc. in Kingston, Ontario, under the supervision of Mr. Peter Clarabut.

Objectives of test program

The principal objective of the testing was to evaluate the in-soil pullout capacity of Tensar UX1700MSE geogrid embedded in coarse sand (SW), while subjected to a range of confining pressures. The pullout tests were performed using the large pullout test apparatus shown in **Figures 1** and **2**.

Materials

Tensar UX1700MSE geogrid is a uni-axial geogrid with a tensile strength of 175 kN/m in the machine direction (based on ASTM D 6637 method of test, and reported in the 2003 GFR Specifier's Guide, published December 2002). Specimens were cut from Roll/Lot #18941 (Tensar Designation) received at our laboratory on 4 October 2004.

The particle size distribution curve for the soil used in this investigation is shown in **Figure 3**. The constant volume (residual) friction angle of the soil is 35 degrees.

Pullout box and test methodology

The test apparatus and method of test was in accordance with Draft ASTM Method of Test - Measuring Geosynthetic Pullout Resistance in Soil - ASTM Designation Number Z2467Z Draft No. 8 dated January 2001.

The pullout box is approximately 2290 mm long by 710 mm high (outside dimension). The inside width of the box is 760 mm (**Figure 2**). The soil surface was surcharged using a Kevlar reinforced polymer air bag to simulate variable depths of overburden soil above the test specimen. The bag allows a uniform surcharge pressure of up to 200 kPa to be applied to the surface of the soil. The pullout box surcharging arrangement is self-reacting using a system of steel angles that encompass the box. Columns of 50 mm-wide hollow steel sections slotted into each end of the test apparatus are used to confine the soil at the front and back of the pullout box. An adjustable pair of steel sleeves is used to pass the test specimen through the front end of the pullout box. The sidewalls of the pullout box are constructed from 25 mm-thick plywood that is covered on the inside (soil side) with a friction-reducing polymer liner.

For each test, a 300 mm-wide (13 longitudinal strands) geogrid specimen was used and extended approximately 900 mm into the soil box (3 transverse members). Each specimen was placed over a 305 mm-deep layer of compacted soil. Steel wire tell-tales were attached to at least 3 nodal points along the length of each specimen (see **Figure 4**) and displacements recorded using potentiometer-type displacement transducers. For all tests the nodal points were located at 5 mm, 457 mm, and 914 mm from the inside front sleeve of the pullout box. The entire specimen was then covered by a 305 mm depth of soil and compacted. The soil above and below the specimen elevation was compacted in 150 mm lifts using an electric Kango impact hammer and compacting plate. The compacted unit densities and moisture content were recorded using a nuclear density meter. Average measurement results are reported in **Table 1**.

The reinforcement clamp (**Figure 5**) was attached to a computer-controlled electro-mechanical screw jack actuator with a maximum capacity of about 50 kN. The rate of displacement of the front clamp was kept constant at 1 mm/minute. The tensile load and displacements recorded by the actuator load cell and the potentiometers were monitored continuously by a computer-controlled data acquisition system.

Test results

As-compacted soil properties for the coarse sand are summarized in **Table 1**.

Peak pullout capacities are summarized in **Table 2**.

A pullout capacity versus confining pressure plot is presented in **Figure 6**.

Photographs of exhumed test specimens are shown in **Figures 7** through **9**.

Pullout force versus displacement plots for each test are presented in **Figures A1** through **A3** of **Appendix A**.

Pullout of the geogrid specimen occurred in all tests.

Coefficient of interaction (C_i) values were calculated as follows based on recommendations in the GRI GG5 protocol:

$$C_i = P/2WL\sigma_n \tan\phi \leq 1.0$$

where:

P = pullout load (kN)
W = width of specimen (m)
L = length of specimen (m)
σ_n = normal (confining) pressure (kPa)
ϕ = residual friction angle of soil (degrees)

The calculated results gave $C_i \leq 1.0$ for all tests as shown in **Table 2**.



Richard J. Bathurst Ph.D., P.Eng.

REFERENCES

ASTM D 6637-01. Standard Test Method for Determining Tensile Properties of Geogrids by the Single or Multi-Rib Tensile Method, American Society for Testing and Materials, West Conshohocken, PA 19428-2958 USA.

GRI Test Method GG5 “Geogrid Pullout” dated 30 January 1991

Draft ASTM Method of Test - Measuring Geosynthetic Pullout Resistance in Soil - ASTM Designation Number Z2467Z Draft No. 8 dated January 2001

Table 1: As-compacted soil properties

Soil	Bulk Unit Density (kg/m^3)	Moisture Content (%)	Residual Friction Angle
Coarse sand (SW)	1800	2.5	35°

Table 2:
Coarse sand (SW)– Tensar UX1700MSE Geogrid in-soil pullout test results

Test No.	File Name	Date (D/M/Y)	Max Pullout Resistance (kN)	Confining Pressure (kPa)	Specimen Length in Soil (m)	Failure Type	C_i
1	TUX17C1	05/10/04	18.1	53	0.9	Pullout	0.9
2	TUX17C2	07/10/04	27.7	125	0.9	Pullout	0.6
3	TUX17C3	14/10/04	31.6	177	0.9	Pullout	0.5

Width of specimens = 0.3 m



Figure 1: Pullout box apparatus.

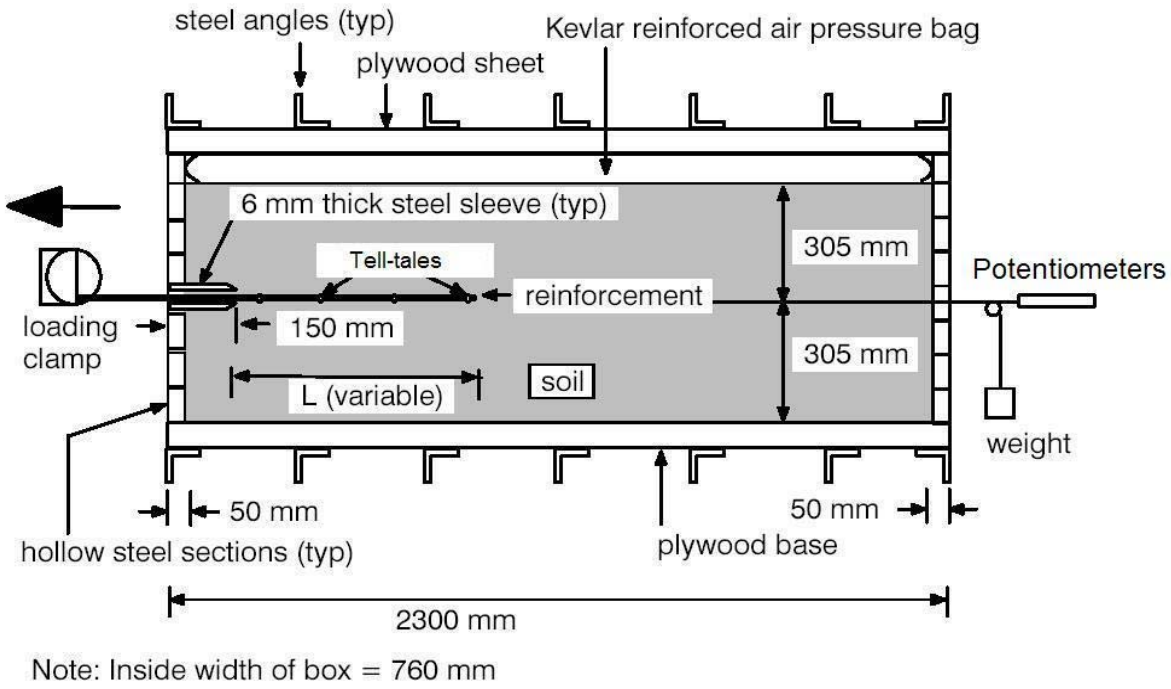


Figure 2: Cross-section view of in-soil pullout box apparatus and general arrangement of soil reinforcement.

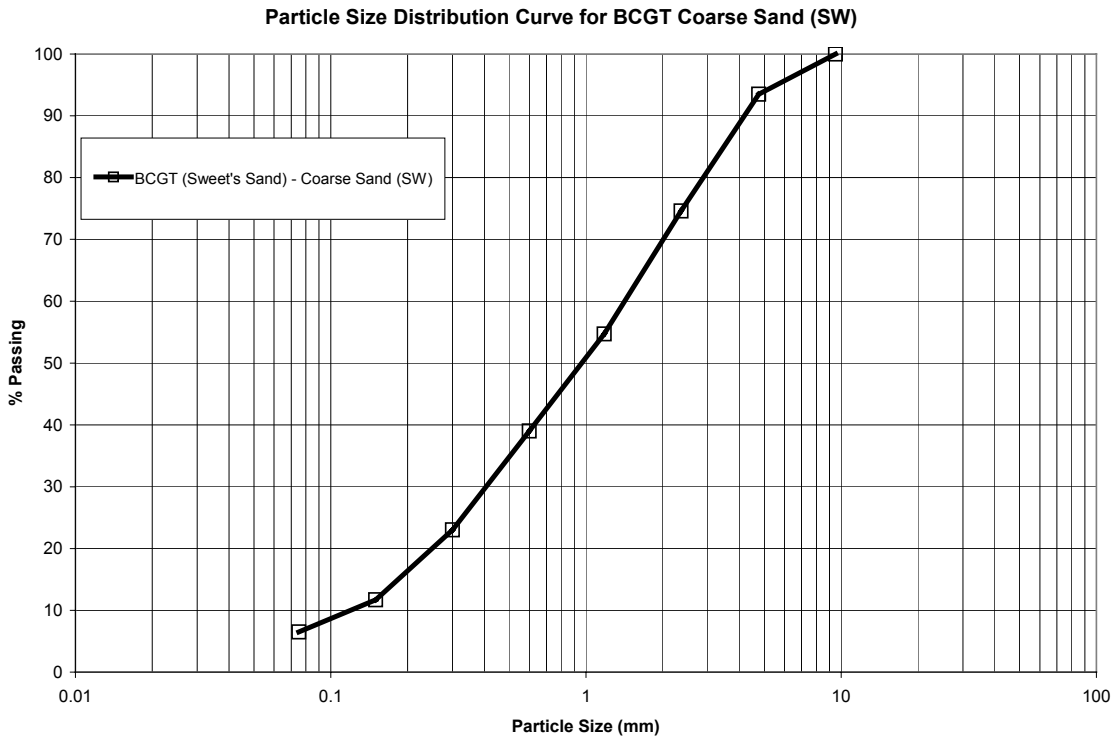


Figure 3: Particle size distribution for coarse sand (SW).



Figure 4: Photograph of a 900 mm-long Tensar UX1700MSE geogrid specimen showing the tell-tales.



Figure 5: View of clamping arrangement at front of pullout box.

Figure 6: Pullout Capacity vs Confining Pressure
Tensar UX1700MSE Geogrid - Coarse Sand (SW)

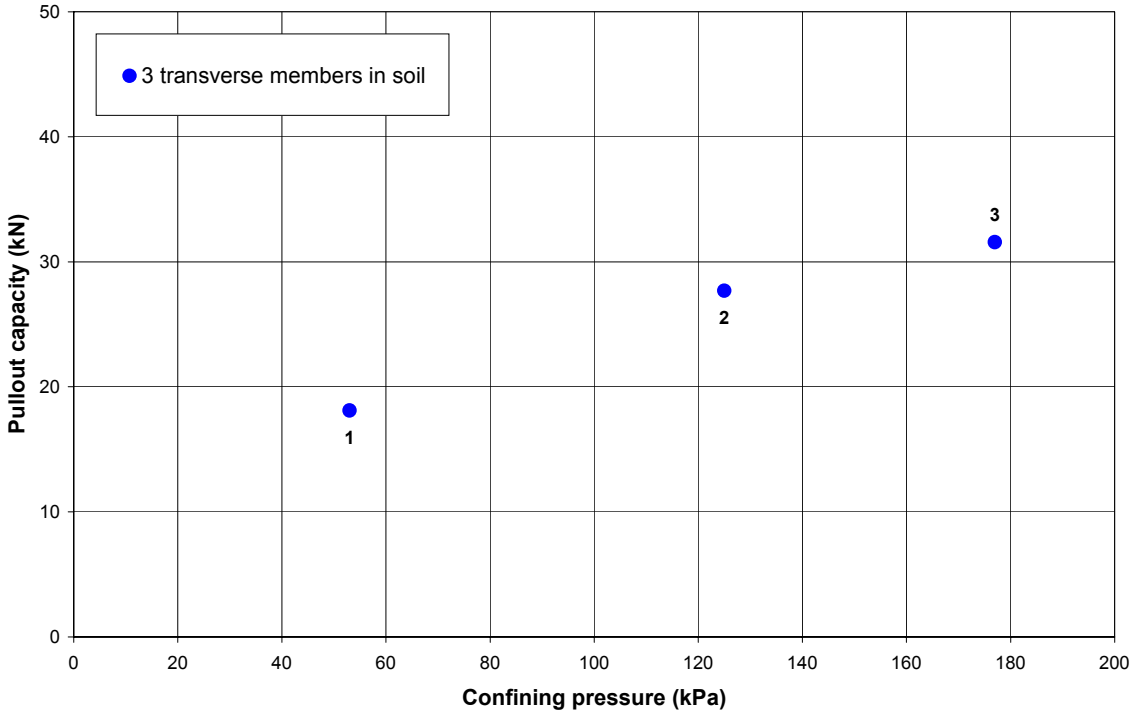




Figure 7: Exhumed Tensar UX1700MSE geogrid specimen (900 mm long) from Test 1, after testing under a confining pressure of 53 kPa.



Figure 8: Exhumed Tensar UX1700MSE geogrid specimen (900 mm long) from Test 2, after testing under a confining pressure of 122 kPa.



Figure 9: Exhumed Tensar UX1700MSE geogrid specimen (900 mm long) from Test 3, after testing under a confining pressure of 177 kPa.

APPENDIX A

**Figure A1: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1700MSE Geogrid with Coarse Sand (SW)
Test 1
(53 kPa Confining Pressure)**

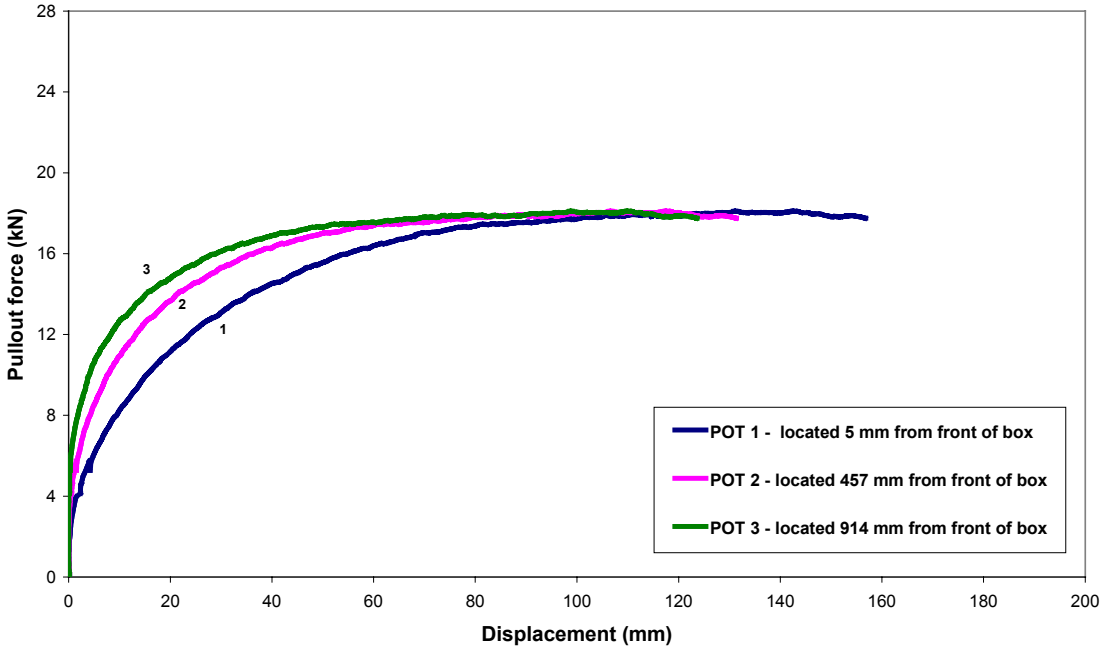


**Figure A2: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1700MSE Geogrid with Coarse Sand (SW)
Test 2
(125 kPa Confining Pressure)**

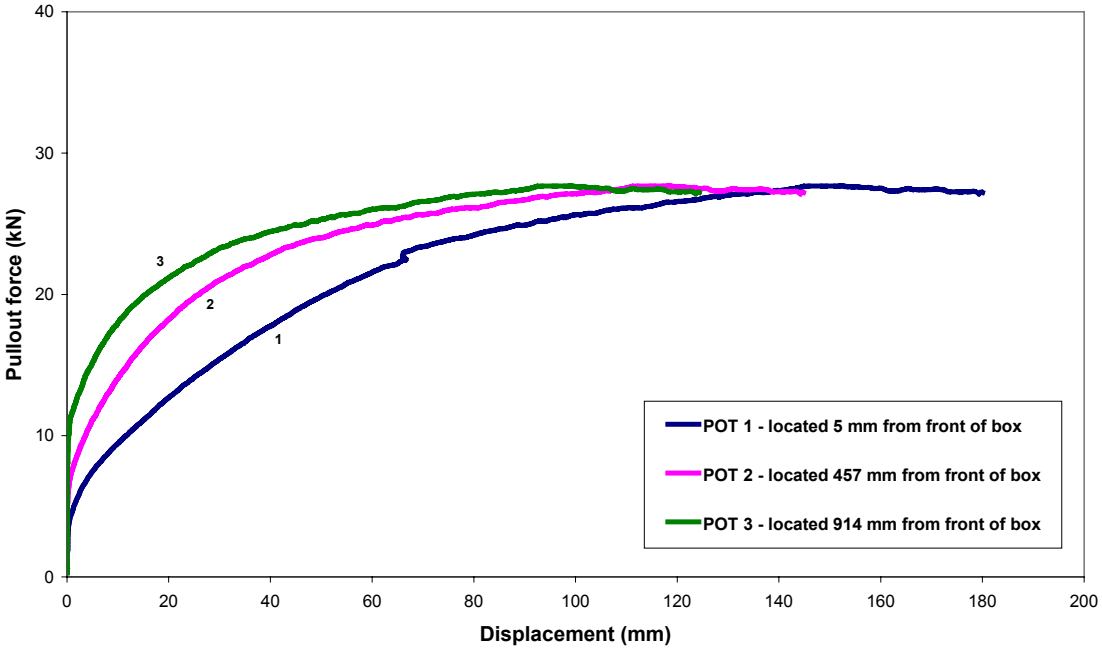
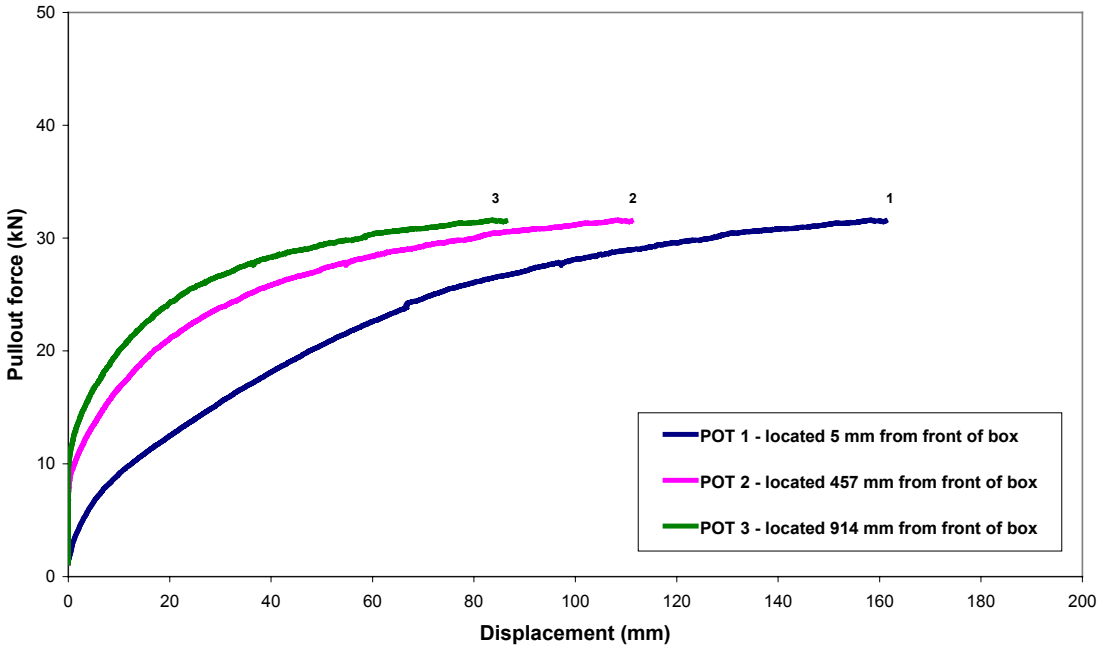


Figure A3: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1700MSE Geogrid with Coarse Sand (SW) Test 3 (177 kPa Confining Pressure)



Appendix 1.2.8

Interface Direct Shear Tests Report

MESA® MSE Retaining Wall System

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Prepared for:

Tensar International Corporation
2500 Northwinds Pkwy, Suite 500
Alpharetta, GA 30009

FINAL REPORT

INTERFACE DIRECT SHEAR TESTING FLORIDA SAND AGAINST TENSAR UX AND BX GEOGRIDS

Prepared by:

SGI TESTING SERVICES, LLC

4405 International Blvd., Suite B-117
Norcross, GA 30093

Project Number SGI23016

31 January 2024

CAVEAT

The reported results apply only to the materials and test conditions used in the laboratory testing program. The results do not necessarily apply to other materials or test conditions. The test results should not be used in engineering analysis unless the test conditions model the anticipated field conditions. The testing was performed in accordance with general engineering testing standards and requirements. This testing report is submitted for the exclusive use of the client to whom it is addressed.

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1. INTRODUCTION

SGI Testing Services, LLC (SGI) conducted a laboratory testing program to evaluate the shear strength of interfaces between Florida sand and Tensar uniaxial and biaxial geogrids. The sample preparation procedures and testing conditions used in the testing program were specified Mr. Willie Liew, P.E. of Tensar International Corporation. All of the pullout tests were conducted at SGI located in Norcross, Georgia, United States.

2. TEST MATERIALS

Two types of materials were used in this testing program. Descriptions of these materials are given below:

- *Reinforcement Material:* Tensar BX112040 biaxial geogrid, UX1400MSE, and UX1700MSE uniaxial geogrids; and
- *Soil Materials*: Florida sand. Particle size analysis, compaction, and direct shear strength testing were conducted on Florida sand, and the test results are presented in Appendix A.

Tensar BX112040, UX1400MSE, and UX1700MSE geogrids were provided by Tensar International Corporation. Bulk samples of Florida sand used in this testing program were obtained from a project site in Florida.

3. INTERFACE DIRECT SHEAR TEST EQUIPMENT

The interface direct testing device used in this testing consisted of an upper and lower shear box. The upper shear box measured 12 inch by 12 inch in plan and 3 inch in depth. The lower shear box measured 12 inch by 15 inch in plan and 3 inch in depth. Normal (vertical) stresses were applied to the testing specimen by using dead weight for low normal stress tests or an air bladder system for high normal stress tests, and shear loads were applied to the test specimen through an automatically-controlled motor system.

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4. TEST METHOD AND PROCEDURES

The interface direct shear tests were performed in accordance with the ASTM Standard Test Method D 5321, *"Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method"*. For each interface direct shear test, the test specimen was set up in accordance with the following procedures and tested under the specific conditions as described below:

- For each test, Florida sand was compacted into the lower shear box by hand tamping to approximately 95% of maximum standard Proctor dry unit weight at optimum moisture content;

- A geogrid specimen was trimmed from the as-received geogrid sample and placed on the compacted soil and attached to the lower shear box. The geogrid specimen was oriented so that the direction of shearing was parallel to the machine direction of the geogrid specimen;

- For each test, Florida sand was compacted on top of the geogrid into the upper shear box by hand tamping to approximately 95% of maximum standard Proctor dry unit weight at optimum moisture content;

- A specific normal stress was applied to the test specimen by using dead weight for low normal stress tests (720 psf), or through an air bladder system for high normal stress tests; and

- After the application of the normal stress, the test specimen was sheared at a constant shear displacement rate of 0.04 inch/min. Shearing was continued until a minimum total shear displacement of 3.0 inch was achieved.

5. INTERFACE DIRECT SHEAR TEST RESULTS

Three series of interface direct shear tests were performed in this testing program. For each test series, the test results are presented in a summary page in

Appendix B. The summary page includes:

- Shear force versus displacement figure;
- Shear strength versus normal stress figure; and
- A table that summarizes test conditions, peak and residual shear strengths, and direct sliding coefficients at peak and residual (i.e., large-displacement or LD) shear strengths.

The peak coefficient of direct sliding (C_{dsp}) was calculated using the equation as follows:

$$C_{dsp} = \frac{\tau_p}{\sigma_n \tan\phi_p + c_p} \quad (1)$$

where:

τ_p = Peak interface shear strength at σ_n;
σ_n = Total normal stress applied to the test interface;
ϕ_p = Peak total-stress friction angle of soil; and
c_p = Peak total-stress cohesion of soil.

The residual coefficient of direct sliding (C_{dsr}) was calculated using the equation as follows:

$$C_{dsr} = \frac{\tau_r}{\sigma_n \tan\phi_r + c_r} \quad (2)$$

where:

τ_r = Residual interface shear strength at σ_n;
σ_n = Total normal stress applied to the test interface;
ϕ_r = Residual total-stress friction angle of soil; and
c_r = Residual total-stress cohesion of soil.

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6. CLOSURE

SGI appreciates the opportunity to provide laboratory testing services to Tensar International Corporation. Should you have any questions regarding this report, or if you require additional information, please do not hesitate to contact the undersigned.

Sincerely,



Zehong Yuan, Ph.D., P.E.
Laboratory Manager

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Figure 1. Tensar BX112040 geogrid placed on compacted Florida sand with the MD parallel to the direction of shearing.

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Figure 2. Tensar UX1400MSE geogrid placed on compacted Florida sand with the MD parallel to the direction of shearing.

APPENDIX A

SUMMARY OF
SOIL TEST RESULTS

SGI Testing Services, LLC

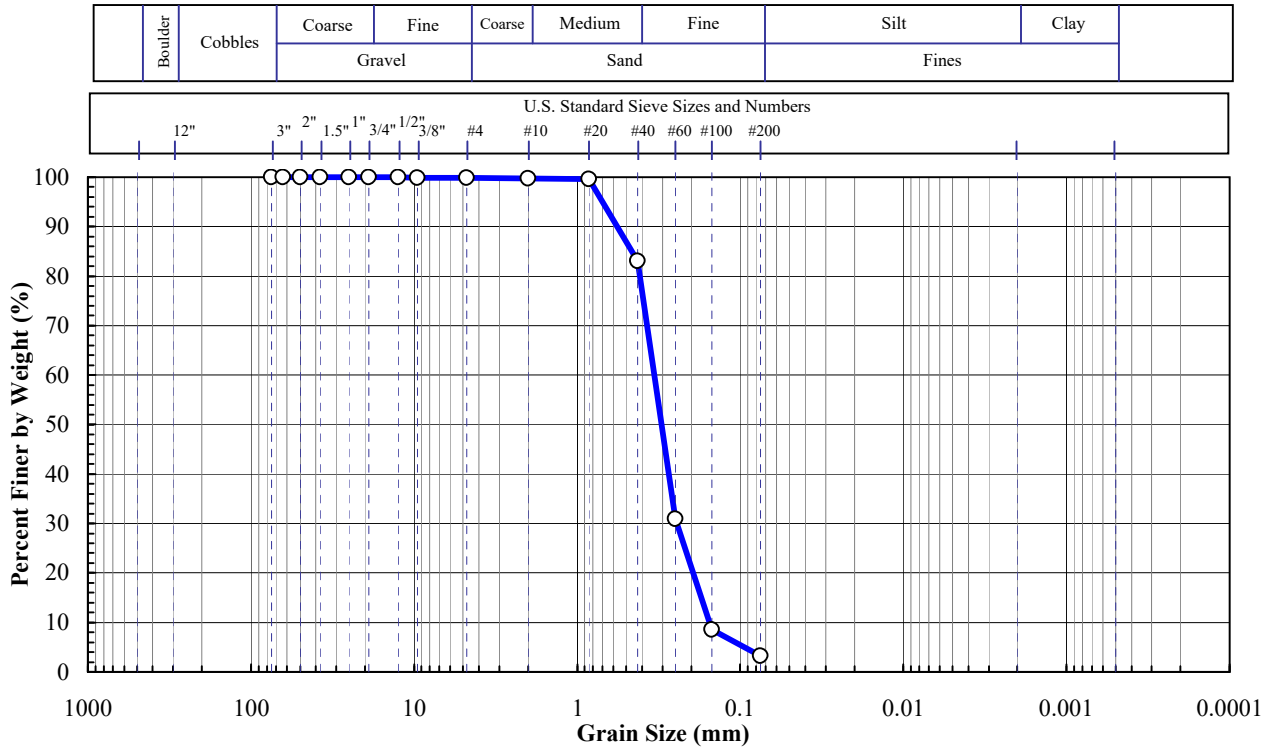
4405 International Blvd., Suite B-117, Norcross, GA 30093
Ph: (770) 931-8222 Fax: (770) 931-8240

Project Name:	SGI23016
Project No:	SGI23016
Client Sample ID:	Florida Sand
Lab Sample No:	S23812

ASTM D 2216, D 1140, D 422, C 136, D 4318, D 2487

SOIL INDEX PROPERTIES

Moisture Content, Grain Size, Atterberg Limits, Classification

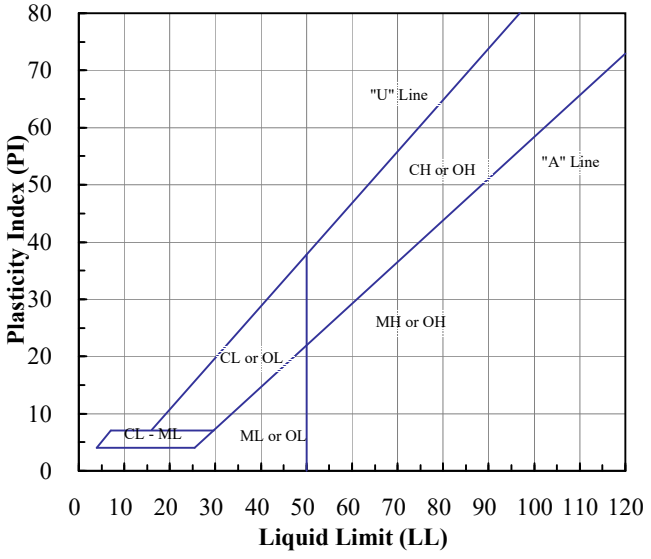


Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
1/2"	12.5	100.0
3/8"	9.5	99.9
#4	4.75	99.9
#10	2.00	99.8
#20	0.850	99.6
#40	0.425	83.0
#60	0.250	30.9
#100	0.150	8.5
#200	0.075	3.2

Hydrometer Particle Diameter (mm)	% Finer
0.0350	
0.0230	
0.0067	
0.0033	
0.0014	

Gravel (%):	.1
Sand (%):	96.7
Fines (%):	3.2
Silt (%):	
Clay (%):	

Coeff. Unif. (Cu):	2.2
Coeff. Curv. (Cc):	1.1



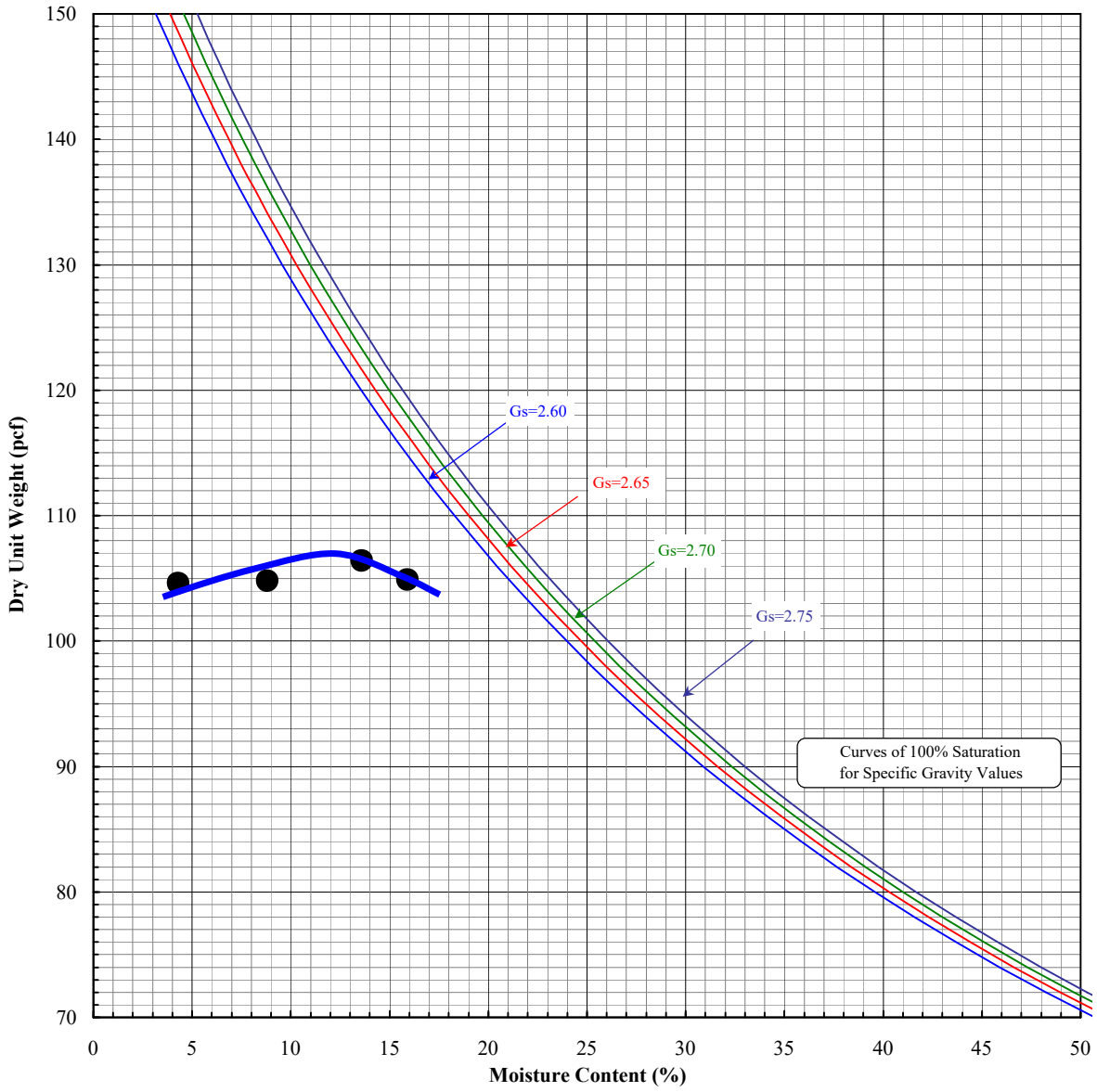
Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits LL (%)	Atterberg Limits PL (%)	Atterberg Limits PI (-)	Engineering Classification
Florida Sand	S23812		3.2				SP

SGI Testing Services, LLC

4405 International Blvd., Suite B-117, Norcross, GA 30093
Ph: (770) 931 8222 Fax: (770) 931 8240

Project Name:	SGI23016
Project No:	SGI23016
Client Sample ID:	Florida Sand
Lab Sample No:	S23812

ASTM D698 **COMPACTION MOISTURE-DENSITY RELATIONSHIP** **Standard - Method A**



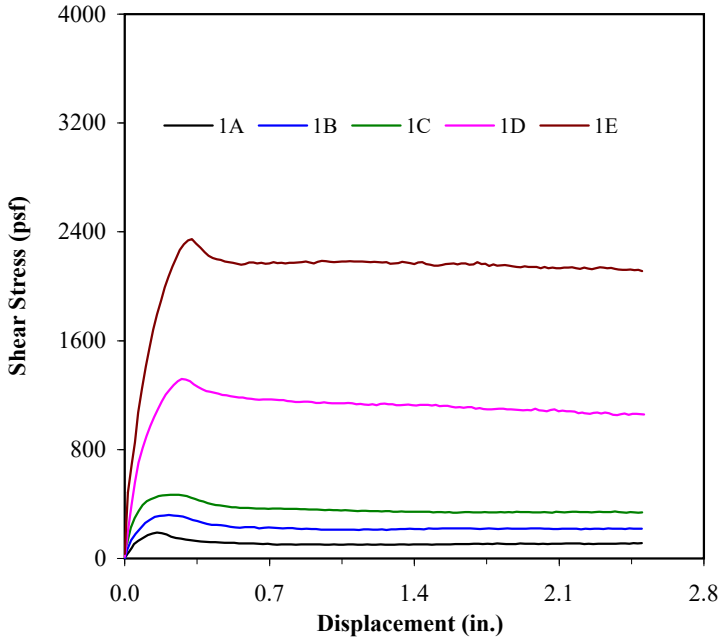
Client Sample ID.	SGI Sample ID NO.	Compaction Data ω_i (%)	Compaction Data γ_d (pcf)	Maximum Dry Unit Weight and Optimum Moisture Content — Measured ω_{om} (%)	Measured γ_{dmax} (pcf)	Corrected ω_{om} (%)	Corrected γ_{dmax} (pcf)
Florida Sand	S23812	4.3	104.6	12.4	106.6		
		8.8	104.8				
		13.6	106.4				
		15.9	104.9				

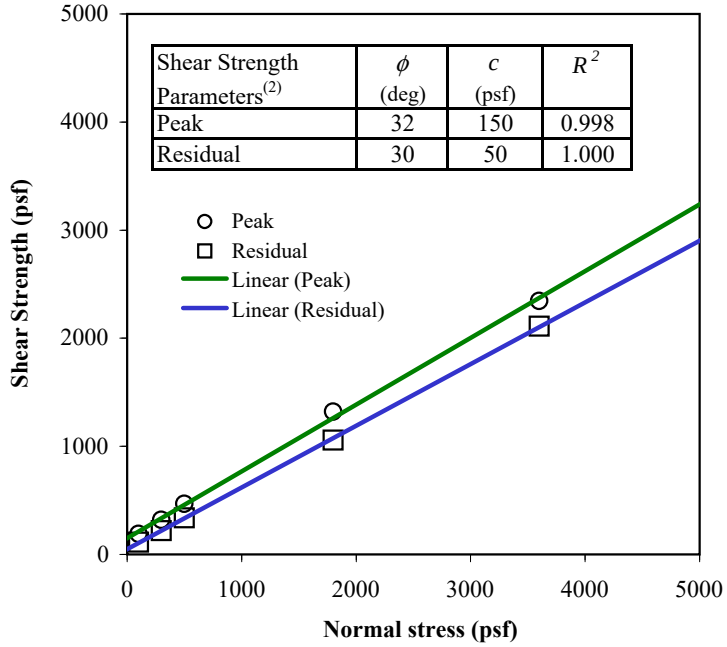
% Retained on #4 Sieve: 0.1
% Retained on #3/8" Sieve:
% Retained on 3/4" Sieve:
% Retained on 2.0" Sieve:

TENSAR INTERNATIONAL CORPORATION

DIRECT SHEAR TESTING (ASTM D 3080)

TEST SERIES NO. 1: Internal shear strength of Florida sand compacted to approximately 95% of max standard Proctor dry density at optimum moisture content (γ_{dmax} = 106.6 pcf, OMC = 12.4%)





Shear Strength Parameters[2]	ϕ (deg)	c (psf)	R^2
Peak	32	150	0.998
Residual	30	50	1.000

Test No.	Shear Box Size (in x in)	Normal Stress (psf)	Shear Rate (in/min)	Soaking Stress (psf)	Soaking Time (hour)	Consolidation Stress (psf)	Consolidation Time (hour)	Taget Soil Compaction Dry Density (pcf)	Taget Soil Compaction Moisture Content (%)	Target Soil Compaction Dry Density (pcf)	Target Soil Compaction Moisture Content (%)	Shear Strength τ_p (psf)	Shear Strength τ_r (psf)	Secant Angle ϕ_p (deg)	Secant Angle ϕ_r (deg)	Failure Mode
1A	12 x 12	100	0.04	-	-	100	0.5	101.3	12.4			190	112	62	48	(1)
1B	12 x 12	300	0.04	-	-	300	0.5					319	218	47	36	(1)
1C	12 x 12	500	0.04	-	-	500	0.5					469	339	43	34	(1)
1D	12 x 12	1800	0.04	-	-	1800	0.5					1320	1058	36	30	(1)
1E	12 x 12	3600	0.04	-	-	3600	0.5					2346	2111	33	30	(1)

NOTES:

(1) Sliding (i.e., shear failure) was forced to occur along the shear plane between the upper and lower shear box.

(2) The reported total-stress parameters of friction angle and cohesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The residual shear strength was calculated using the shear force measured at the end of the test.

DATE OF REPORT:	1/31/2024
FIGURE NO.	A-3
PROJECT NO.	SGI23016
DOCUMENT NO.	
FILE NO.	

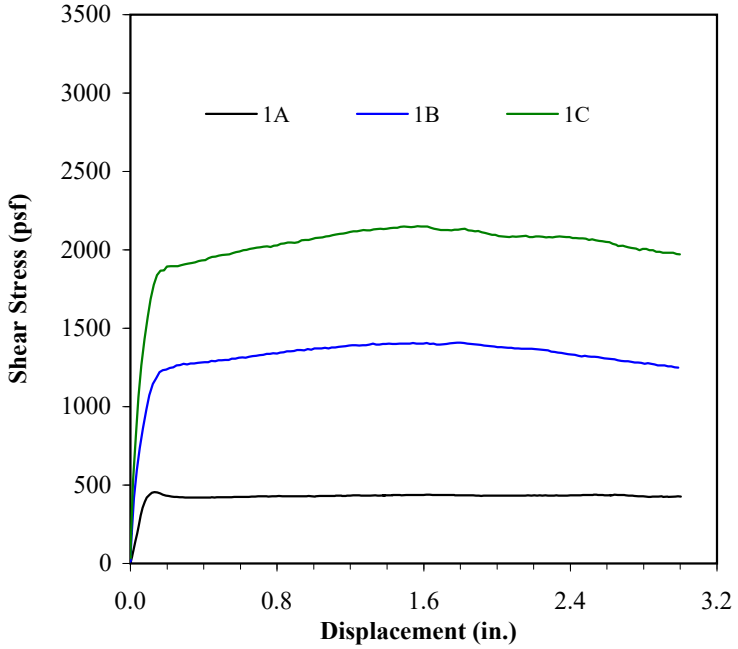
SGI TESTING SERVICES, LLC

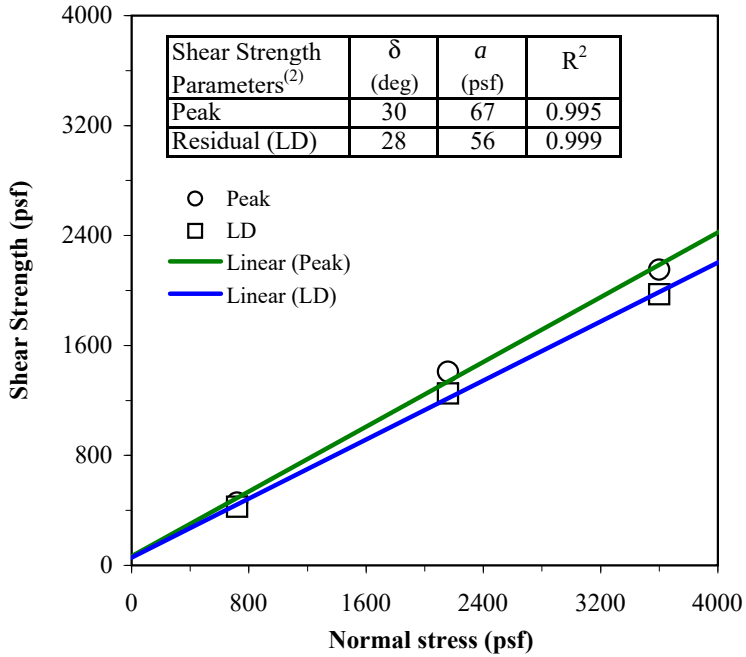
APPENDIX B

SUMMARY OF
INTERFACE DIRECT SHEAR TEST RESULTS

TENSAR INTERNATIONAL CORPORATION
INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Upper Shear Box: Florida sand compacted to approximately 95% of max standard Proctor dry density at OMC (γ_{dmax} = 106.6 pcf, OMC = 12.4%) against Tensar BX112040 geogird Lot #124675-18 in MD (direction of shearing parallel to MD) against
Lower Shear Box: Florida sand compacted to approximately 95% of max standard Proctor dry density at OMC (γ_{dmax} = 106.6 pcf, OMC = 12.4%)





Shear Strength Parameters[2]	δ (deg)	a (psf)	R^2
Peak	30	67	0.995
Residual (LD)	28	56	0.999

Test No.	Shear Box Size (in x in)	Normal Stress (psf)	Shear Rate (in/min)	Soaking Stress (psf)	Soaking Time (hrs)	Consolidation Stress (psf)	Consolidation Time (hrs)	Target Soil Compaction γ_d (pcf)	Target Soil Compaction ω_i (%)	Target Soil Compaction ω_f (%)	Soil Shear Strength Parameters ϕ_P (deg)	c_P (psf)	ϕ_R (deg)	c_R (psf)	Shear Strength[1] τ_P (psf)	τ_R (psf)	Coefficient of Direct Sliding C_{dsp}	C_{dsr}
1A	12 x 12	720	0.04	-	-	720	0.25	101.3	12.4	-	32	150	30	50	455	426	0.76	0.91
1B	12 x 12	2160	0.04	-	-	2160	0.25			-					1409	1250	0.94	0.96
1C	12 x 12	3600	0.04	-	-	3600	0.25			-					2151	1972	0.90	0.93

NOTE:
(1) Sliding (i.e., shear failure) occurred at the interface between Florida sand and geogrid.
(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The residual (τ_r) was calculated using the shear force measured at the end of the test.

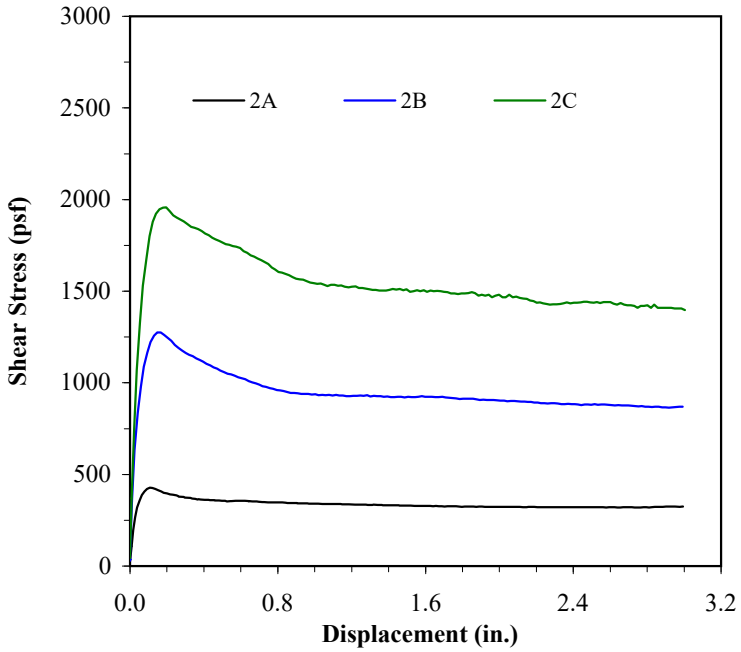
DATE OF REPORT: 1/31/2024

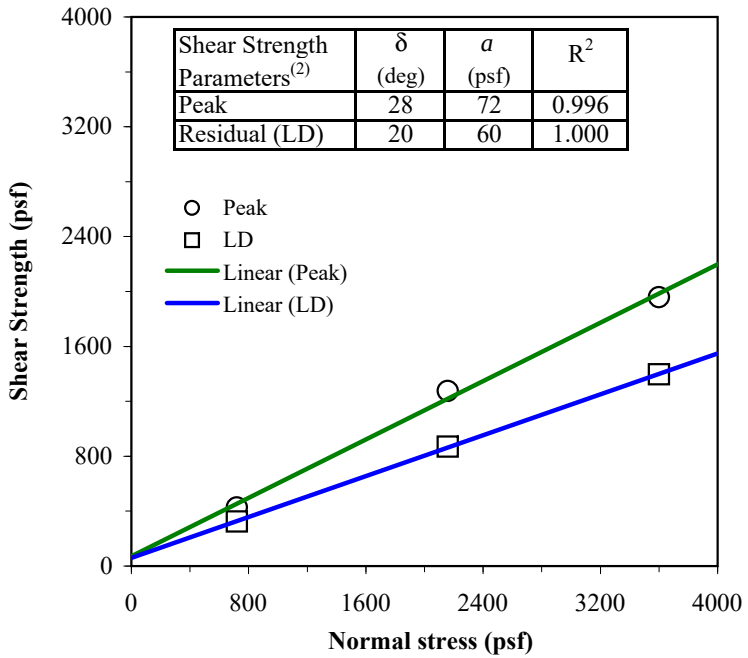
SGI TESTING SERVICES, LLC

FIGURE NO.	1
PROJECT NO.	SGI23016
DOCUMENT NO.	
FILE NO.	

TENSAR INTERNATIONAL CORPORATION
INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Upper Shear Box: Florida sand compacted to approximately 95% of max standard Proctor dry density at OMC (γ_{dmax} = 106.6 pcf, OMC = 12.4%) against Tensar UX1400MSE geogrid Lot #319097-88 in MD (direction of shearing parallel to MD) against
Lower Shear Box: Florida sand compacted to approximately 95% of max standard Proctor dry density at OMC (γ_{dmax} = 106.6 pcf, OMC = 12.4%)





Shear Strength Parameters[2]	δ (deg)	a (psf)	R^2
Peak	28	72	0.996
Residual (LD)	20	60	1.000

Test No.	Shear Box Size (in x in)	Normal Stress (psf)	Shear Rate (in/min)	Soaking Stress (psf)	Soaking Time (hrs)	Consolidation Stress (psf)	Consolidation Time (hrs)	Target Soil Compaction γ_d (pcf)	Target Soil Compaction ω_i (%)	Target Soil Compaction ω_f (%)	Soil Shear Strength Parameters ϕ_P (deg)	c_P (psf)	ϕ_R (deg)	c_R (psf)	Shear Strength[1] τ_P (psf)	τ_R (psf)	Coefficient of Direct Sliding C_{dsp}	C_{dsr}
2A	12 x 12	720	0.04	-	-	720	0.25	101.3	12.4	-	32	150	30	50	427	325	0.71	0.70
2B	12 x 12	2160	0.04	-	-	2160	0.25			-					1275	869	0.85	0.67
2C	12 x 12	3600	0.04	-	-	3600	0.25			-					1958	1397	0.82	0.66

NOTE:
(1) Sliding (i.e., shear failure) occurred at the interface between Florida sand and geogrid.
(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The residual (τ_r) was calculated using the shear force measured at the end of the test.

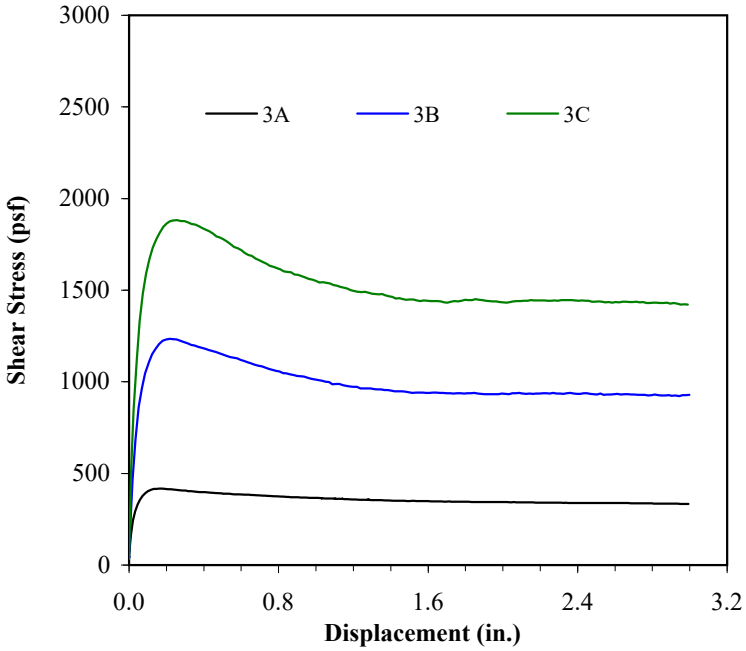
DATE OF REPORT: 1/31/2024

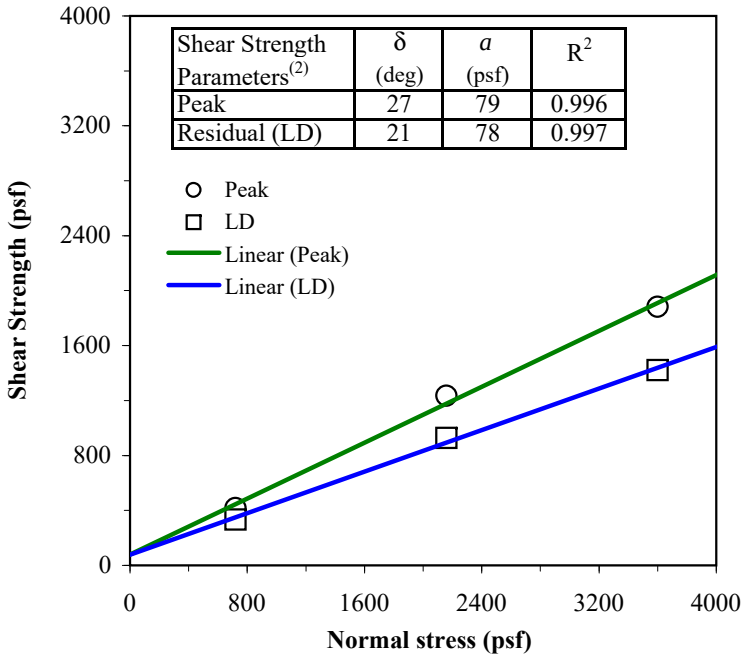
SGI TESTING SERVICES, LLC

FIGURE NO.	2
PROJECT NO.	SGI23016
DOCUMENT NO.	
FILE NO.	

TENSAR INTERNATIONAL CORPORATION
INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Upper Shear Box: Florida sand compacted to approximately 95% of max standard Proctor dry density at OMC (γ_{dmax} = 106.6 pcf, OMC = 12.4%) against Tensar UX1700MSE geogird Lot # 318845-5 in MD (direction of shearing parallel to MD) against
Lower Shear Box: Florida sand compacted to approximately 95% of max standard Proctor dry density at OMC (γ_{dmax} = 106.6 pcf, OMC = 12.4%)





Test No.	Shear Box Size (in x in)	Normal Stress (psf)	Shear Rate (in/min)	Soaking Stress (psf)	Soaking Time (hrs)	Consolidation Stress (psf)	Consolidation Time (hrs)	Target Soil Compaction γ_d (pcf)	Target Soil Compaction ω_i (%)	Target Soil Compaction ω_f (%)	Soil Shear Strength Parameters ϕ_P (deg)	Soil Shear Strength Parameters c_P (psf)	Soil Shear Strength Parameters ϕ_R (deg)	Soil Shear Strength Parameters c_R (psf)	Shear Strength(1) τ_P (psf)	Shear Strength(1) τ_R (psf)	Coefficient of Direct Sliding C_{dsp}	Coefficient of Direct Sliding C_{dsr}
3A	12 x 12	720	0.04	-	-	720	0.25	101.3	12.4	-	32	150	30	50	417	334	0.70	0.72
3B	12 x 12	2160	0.04	-	-	2160	0.25			-					1235	928	0.82	0.72
3C	12 x 12	3600	0.04	-	-	3600	0.25			-					1883	1422	0.79	0.67

NOTE:
(1) Sliding (i.e., shear failure) occurred at the interface between Florida sand and geogrid.
(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The residual (τ_r) was calculated using the shear force measured at the end of the test.

DATE OF REPORT: 1/31/2024

SGI TESTING SERVICES, LLC

FIGURE NO.	3
PROJECT NO.	SGI22006
DOCUMENT NO.	
FILE NO.	

Appendix 2.2.1.1a

Problem 1 with Silty Sand

MESA® MSE Retaining Wall System

Back to main document

AASHTO 2017-2020
Mesa Wall System

MSEW+: Update # 2026.01

PROJECT IDENTIFICATION

Title: Mesa Wall System
Project Number: Problem 1 Rev1
Client: IDEA
Designer: JP
Station Number: MESA - AASHTO

Description:

MESA System, DH = 30.0 ft; 75 yrs Design Life; Silty Sand backfill material

Company's information:

Name: Tensar International Corporation
Street: 2500 Northwinds Pkwy.
Suite 500
Alpharetta, GA 30009
Telephone #: (770) 344-2090
Fax #: (770) 344-2084
E-Mail: Jorge.Peralta@cmc.com

File path and name: C:\Users\P01000987\OneDrive - CMC\Desktop\MESA IDEA com..... ..... 75yr_SiltySand.BENp

Original date and time of creating this file: May 2026

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using GEOGRID as reinforcing material.

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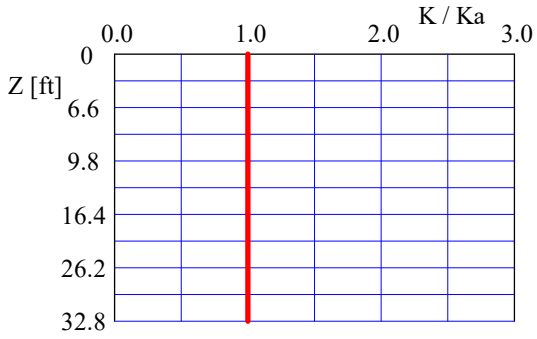
Version MSEW+ Version MSEW+ Version MSEW+

INPUT DATA: Geogrids (Analysis)

D A T A	Geogrid type #1	Geogrid type #2	Geogrid type #3	Geogrid type #4	Geogrid type #5
Tult [lb/ft]	3970.0	4800.0	7810.0	9870.0	11990.0
Durability reduction factor, RFd	1.10	1.10	1.10	1.10	1.10
Installation-damage reduction factor, RFid	1.24	1.15	1.14	1.15	1.18
Creep reduction factor, RFc	2.68	2.68	2.54	2.54	2.54
CDR for strength	N/A	N/A	N/A	N/A	N/A
Coverage ratio, Rc	1.000	1.000	1.000	1.000	1.000
Friction angle along geogrid-soil interface, ρ	28.35	28.35	28.35	28.35	28.35
Pullout resistance factor, F*	0.80·tanϕ	0.80·tanϕ	0.80·tanϕ	0.80·tanϕ	0.80·tanϕ
Scale-effect correction factor, α	1.0	1.0	1.0	1.0	1.0

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.00
3.3 ft	1.00
6.6 ft	1.00
9.8 ft	1.00
13.1 ft	1.00
16.4 ft	1.00
19.7 ft	1.00

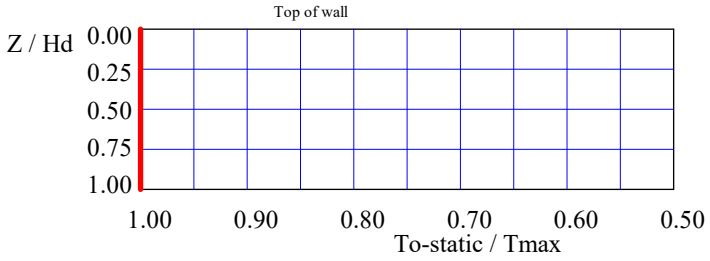


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INPUT DATA: Facia and Connection (according to revised Demo 82) (Analysis)

FACIA type: Facing enabling frictional connection of reinforcement (e.g., modular concrete blocks, gabions)
Depth/height of block is 0.92/0.69 ft. Horizontal distance to Center of Gravity of block is: 0.41 ft.
Average unit weight of block is: γ_f = 81.99 lb/ft ³

Z / Hd	To-static / Tmax
0.00	1.00
0.25	1.00
0.50	1.00
0.75	1.00
1.00	1.00



Connection strength, T-lot, is related to T-ult

Geogrid Type #1		Geogrid Type #2		Geogrid Type #3		Geogrid Type #4		Geogrid Type #5	
N-1	Tult-conn	N-2	Tult-conn	N-3	Tult-conn	N-4	Tult-conn	N-5	Tult-conn
0.0	3353.86	0.0	4055.04	0.0	6597.89	0.0	8338.18	0.0	10129.15
936.0	3353.86	936.0	4055.04	3350.0	6597.89	936.0	8338.18	936.0	10129.15

Geogrid Type #1		Geogrid Type #2		Geogrid Type #3		Geogrid Type #4		Geogrid Type #5	
σ	CRu	σ	CRu	σ	CRu	σ	CRu	σ	CRu
0.0	0.84	0.0	0.84	0.0	0.84	0.0	0.84	0.0	0.84
1021.1	0.84	1021.1	0.84	1021.1	0.84	1021.1	0.84	1021.1	0.84

(1) σ = Confining stress in between stacked blocks [lb/ft ²]
(2) CRult = Tc-ult / Tult
(3) CRcr = Tcre / Tult

D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	UX1100MSE..	UX1400MSE..	UX1500MSE..	UX1600MSE..	UX1700MSE..
Connection strength reduction factor, RFd	1.10	1.10	1.10	1.10	1.10
Creep reduction factor, RFc	N/A	N/A	N/A	N/A	N/A

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INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd 30.00 [ft] { Embedded depth is E = 0.00 ft, and height above top of finished bottom grade is H = 30.00 ft }

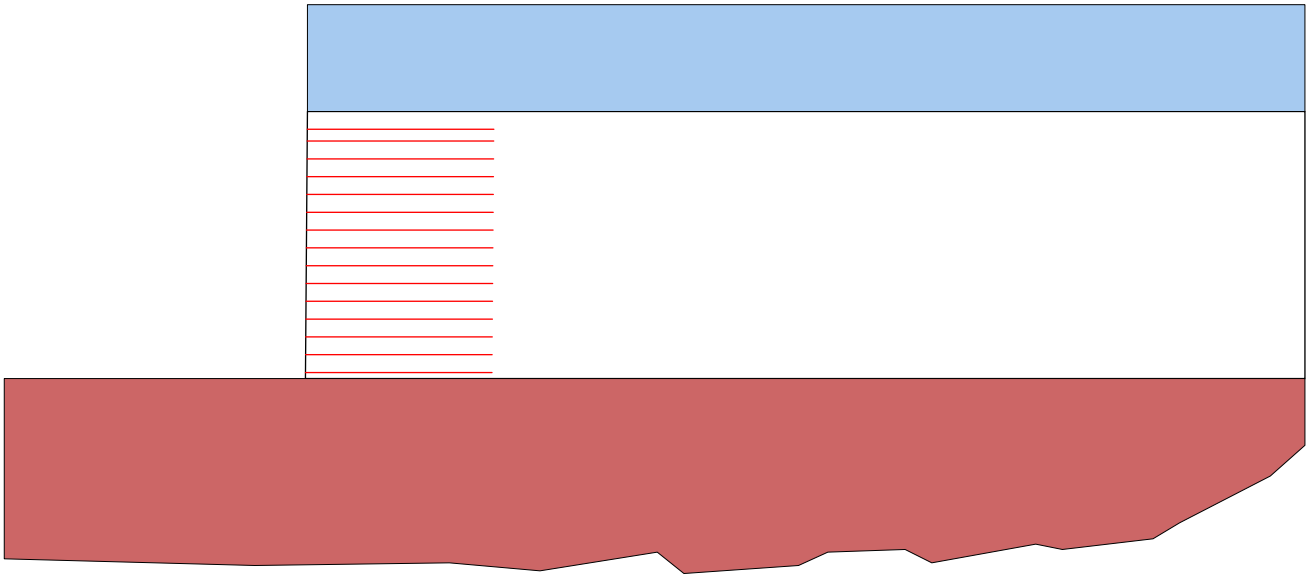
Soil in front of wall is Horizontal.

Batter, ω 0.5 [deg]
Backslope, β 0.0 [deg]
Backslope rise 0.0 [ft] Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

U N I F O R M S U R C H A R G E

Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 250.0 [lb/ft ²]

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 5 10 15 20 25 30 [ft]

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AASHTO 2017-2020 – Load and Resisting Factors

INTERNAL STABILITY

Load factor for vertical earth pressure, EV:	$\gamma_{p\text{-}EV}$	1.35
Load factor for earthquake loads, EQ:	$\gamma_{p\text{-}EQ}$	1.00
Load factor for live load surcharge, LS: (Same as in External Stability).	$\gamma_{p\text{-}LS}$	1.75
Load factor for dead load surcharge, ES: (Same as in External Stability).	$\gamma_{p\text{-}ES}$	1.50

Resistance factor for reinforcement tension	ϕ	Static	Combined static/seismic
Geogrid:		0.90	1.00

Resistance factor for reinforcement tension in connectors	ϕ	Static	Combined static/seismic
Geogrid:		0.90	1.00

Resistance factor for geosynthetic pullout	ϕ	0.90	1.00

EXTERNAL STABILITY

Load factor for vertical earth pressure, EV		Static	Combined Static/Seismic	
Sliding and Eccentricity	$\gamma_{p\text{-}EV}$	1.00	$\gamma_{p\text{-}EQ}$	1.00
Bearing Capacity	$\gamma_{p\text{-}EV}$	1.35	$\gamma_{p\text{-}EQ}$	1.35

Load factor of active lateral earth pressure, EH	$\gamma_{p\text{-}EH}$	1.50
Load factor of active lateral earth pressure during earthquake (does not multiply P_{AE} and P_{IR}):	$(\gamma_{p\text{-}EH})_{EQ}$	1.00
Load factor for earthquake loads, EQ (multiplies P_{AE} and P_{IR}):	$\gamma_{p\text{-}EQ}$	1.00

Resistance factor for shear resistance along common interfaces		Static	Combined Static/Seismic
Reinforced Soil and Foundation	ϕ_τ	0.90	1.00
Reinforced Soil and Reinforcement	ϕ_τ	0.90	1.00

Resistance factor for bearing capacity of shallow foundation		Static	Combined Static/Seismic
	ϕ_b	0.65	0.65

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ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, CDR = 2.23, factored bearing load = 7221 lb/ft².
Foundation Interface: Direct sliding, CDR = 1.860, Eccentricity, e/L = 0.0889, CDR-overturning = 3.89

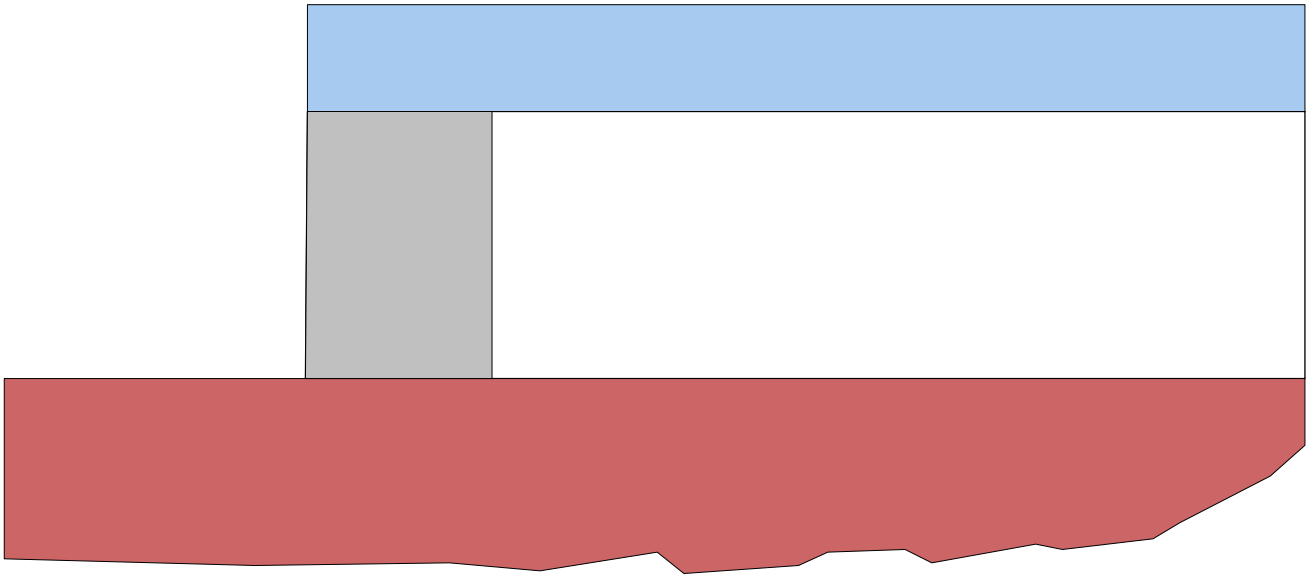
GEOGRID #	Elevation [ft]	Length [ft]	Type #	CONNECTION CDR [connection strength]	Geogrid strength CDR	Pullout resistance CDR	Direct sliding CDR	Eccentricity e/L	Product name
1	0.67	21.00	5	1.19	1.205	29.231	1.769	0.0844	UX1700MSE-MD
2	2.67	21.00	5	1.06	1.068	22.919	1.869	0.0716	UX1700MSE-MD
3	4.67	21.00	5	1.14	1.145	21.564	1.982	0.0597	UX1700MSE-MD
4	6.67	21.00	4	1.01	1.043	20.194	2.110	0.0487	UX1600MSE-MD
5	8.67	21.00	4	1.09	1.131	18.818	2.259	0.0386	UX1600MSE-MD
6	10.67	21.00	4	1.19	1.235	17.434	2.432	0.0294	UX1600MSE-MD
7	12.67	21.00	3	1.04	1.085	16.052	2.636	0.0212	UX1500MSE-MD
8	14.67	21.00	3	1.16	1.208	14.646	2.881	0.0139	UX1500MSE-MD
9	16.67	21.00	3	1.30	1.361	13.222	3.179	0.0077	UX1500MSE-MD
10	18.67	21.00	3	1.49	1.560	11.773	3.551	0.0024	UX1500MSE-MD
11	20.67	21.00	2	1.02	1.054	10.294	4.028	-0.0019	UX1400MSE-MD
12	22.67	21.00	2	1.23	1.271	8.744	4.662	-0.0051	UX1400MSE-MD
13	24.67	21.00	1	1.28	1.227	7.082	5.544	-0.0072	UX1100MSE-MD
14	26.67	21.00	1	2.01	1.932	6.060	6.857	-0.0083	UX1100MSE-MD
15	28.00	21.00	1	1.99	1.907	3.238	8.154	-0.0085	UX1100MSE-MD

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BEARING CAPACITY for GIVEN LAYOUT – Using AASHTO 2017-2020 method

	STATIC	SEISMIC	UNITS
(Water table does not affect bearing capacity)			
Factored bearing resistance. q-n	16072	N/A	[lb/ft ²]
Factored bearing load, σ_V	7221.0	N/A	[lb/ft ²]
Eccentricity, e	1.30	N/A	[ft]
Eccentricity, e/L	0.062	N/A	
CDR calculated	2.23	N/A	
Base length	21.00	N/A	[ft]

Unfactored applied bearing pressure = (Unfactored R) / [L - 2 * (Unfactored e)] =
Unfactored R = 96017.66 [lb/ft], L = 21.00, Unfactored e = 1.16 [ft], and Sigma = 5138.24 [lb/ft ²]



SCALE:

0 5 10 15 20 25 30 [ft]

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RESULTS for STRENGTH

Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Factored LTDS [lb/ft]	Factored Tmax [lb/ft]	Tmd [lb/ft]	Specified minimum CDR Static	Actual calculated CDR Static	Specified minimum CDR seismic	Actual calculated CDR seismic	Product name
1	0.67	3273.1	2716.1	N/A	N/A	1.205	N/A	N/A	UX1700MSE-MD
2	2.67	3273.1	3063.7	N/A	N/A	1.068	N/A	N/A	UX1700MSE-MD
3	4.67	3273.1	2857.6	N/A	N/A	1.145	N/A	N/A	UX1700MSE-MD
4	6.67	2764.6	2651.5	N/A	N/A	1.043	N/A	N/A	UX1600MSE-MD
5	8.67	2764.6	2445.4	N/A	N/A	1.131	N/A	N/A	UX1600MSE-MD
6	10.67	2764.6	2239.3	N/A	N/A	1.235	N/A	N/A	UX1600MSE-MD
7	12.67	2206.8	2033.2	N/A	N/A	1.085	N/A	N/A	UX1500MSE-MD
8	14.67	2206.8	1827.1	N/A	N/A	1.208	N/A	N/A	UX1500MSE-MD
9	16.67	2206.8	1621.0	N/A	N/A	1.361	N/A	N/A	UX1500MSE-MD
10	18.67	2206.8	1414.9	N/A	N/A	1.560	N/A	N/A	UX1500MSE-MD
11	20.67	1274.3	1208.8	N/A	N/A	1.054	N/A	N/A	UX1400MSE-MD
12	22.67	1274.3	1002.7	N/A	N/A	1.271	N/A	N/A	UX1400MSE-MD
13	24.67	977.4	796.6	N/A	N/A	1.227	N/A	N/A	UX1100MSE-MD
14	26.67	977.4	506.0	N/A	N/A	1.932	N/A	N/A	UX1100MSE-MD
15	28.00	977.4	512.6	N/A	N/A	1.907	N/A	N/A	UX1100MSE-MD

RESULTS for PULLOUT

Live Load included in calculating Tmax

NOTE: Live load is not included in calculating the overburden pressure used to assess pullout resistance.

#	Geogrid Elevation [ft]	Coverage Ratio	Factored: Stat./Seis. Tmax [lb/ft]	Tmd [lb/ft]	Le (see NOTE)	La [ft]	Avail.Static Pullout, Pr [lb/ft]	Specified Static CDR	Actual Static CDR	Avail.Seism. Pullout, Pr [lb/ft]	Specified seismic CDR	Actual seismic CDR
1	0.67	1.000	2716.1	N/A	20.65	0.35	79393.4	N/A	29.231	N/A	N/A	N/A
2	2.67	1.000	3063.7	N/A	19.60	1.40	70216.7	N/A	22.919	N/A	N/A	N/A
3	4.67	1.000	2857.6	N/A	18.55	2.45	61622.0	N/A	21.564	N/A	N/A	N/A
4	6.67	1.000	2651.5	N/A	17.51	3.49	53544.4	N/A	20.194	N/A	N/A	N/A
5	8.67	1.000	2445.4	N/A	16.46	4.54	46017.5	N/A	18.818	N/A	N/A	N/A
6	10.67	1.000	2239.3	N/A	15.41	5.59	39041.3	N/A	17.434	N/A	N/A	N/A
7	12.67	1.000	2033.2	N/A	14.36	6.64	32637.1	N/A	16.052	N/A	N/A	N/A
8	14.67	1.000	1827.1	N/A	13.32	7.68	26759.9	N/A	14.646	N/A	N/A	N/A
9	16.67	1.000	1621.0	N/A	12.27	8.73	21433.5	N/A	13.222	N/A	N/A	N/A
10	18.67	1.000	1414.9	N/A	11.22	9.78	16657.7	N/A	11.773	N/A	N/A	N/A
11	20.67	1.000	1208.8	N/A	10.17	10.83	12444.2	N/A	10.294	N/A	N/A	N/A
12	22.67	1.000	1002.7	N/A	9.12	11.88	8767.4	N/A	8.744	N/A	N/A	N/A
13	24.67	1.000	796.6	N/A	8.08	12.92	5641.4	N/A	7.082	N/A	N/A	N/A
14	26.67	1.000	506.0	N/A	7.03	13.97	3066.1	N/A	6.060	N/A	N/A	N/A
15	28.00	1.000	512.6	N/A	6.33	14.67	1659.5	N/A	3.238	N/A	N/A	N/A

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RESULTS for CONNECTION (static conditions)

Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Connection force, To [lb/ft]	Reduction factor for connection (short-term strength) CRult	Reduction factor for connection (long-term strength) CRcr	Available connection strength [lb/ft]	Available Geogrid strength, Tavailable [lb/ft]	CDR connection strength Specified	CDR connection strength Actual	CDR Geogrid strength Specified	CDR Geogrid strength Actual	Product name
1	0.67	2716	0.84	0.30	2943	3637	N/A	1.19	N/A		UX1700MSE-MD
2	2.67	3064	0.84	0.30	2943	3637	N/A	1.06	N/A		UX1700MSE-MD
3	4.67	2858	0.84	0.30	2943	3637	N/A	1.14	N/A		UX1700MSE-MD
4	6.67	2652	0.84	0.30	2422	3072	N/A	1.01	N/A		UX1600MSE-MD
5	8.67	2445	0.84	0.30	2422	3072	N/A	1.09	N/A		UX1600MSE-MD
6	10.67	2239	0.84	0.30	2422	3072	N/A	1.19	N/A		UX1600MSE-MD
7	12.67	2033	0.84	0.33	2109	2452	N/A	1.04	N/A		UX1500MSE-MD
8	14.67	1827	0.84	0.33	2109	2452	N/A	1.16	N/A		UX1500MSE-MD
9	16.67	1621	0.84	0.33	2109	2452	N/A	1.30	N/A		UX1500MSE-MD
10	18.67	1415	0.84	0.33	2109	2452	N/A	1.49	N/A		UX1500MSE-MD
11	20.67	1209	0.84	0.33	1296	1416	N/A	1.02	N/A		UX1400MSE-MD
12	22.67	1003	0.84	0.33	1296	1416	N/A	1.23	N/A		UX1400MSE-MD
13	24.67	797	0.84	0.33	1072	1086	N/A	1.28	N/A		UX1100MSE-MD
14	26.67	506	0.84	0.33	1072	1086	N/A	2.01	N/A		UX1100MSE-MD
15	28.00	513	0.84	0.33	1072	1086	N/A	1.99	N/A		UX1100MSE-MD

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Appendix 2.2.1.1b

Problem 1 with Sandy Gravel

MESA® MSE Retaining Wall System

Back to main document

AASHTO 2017-2020
Mesa Wall System

MSEW+: Update # 2026.01

PROJECT IDENTIFICATION

Title: Mesa Wall System
Project Number: Problem 1 Rev1
Client: IDEA
Designer: JP
Station Number: MESA - AASHTO

Description:

MESA System, DH = 30.0 ft; 75 yrs Design Life; Sandy Gravel backfill material

Company's information:

Name: Tensar International Corporation
Street: 2500 Northwinds Pkwy.
Suite 500
Alpharetta, GA 30009
Telephone #: (770) 344-2090
Fax #: (770) 344-2084
E-Mail: Jorge.Peralta@cmc.com

File path and name: C:\Users\P01000987\OneDrive - CMC\Desktop\MESA IDEA com.....
.....5yr_SandyGravel.BENp

Original date and time of creating this file: May 2026

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using GEOGRID as reinforcing material.

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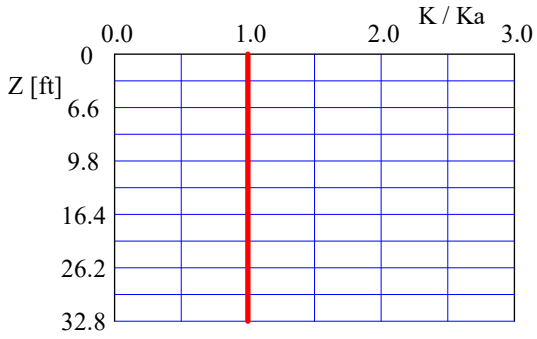
Version MSEW+ Version MSEW+ Version MSEW+

INPUT DATA: Geogrids (Analysis)

D A T A	Geogrid type #1	Geogrid type #2	Geogrid type #3	Geogrid type #4	Geogrid type #5
Tult [lb/ft]	3970.0	4800.0	7810.0	9870.0	11990.0
Durability reduction factor, RFd	1.10	1.10	1.10	1.10	1.10
Installation-damage reduction factor, RFid	1.31	1.35	1.34	1.35	1.39
Creep reduction factor, RFc	2.68	2.68	2.54	2.54	2.54
CDR for strength	N/A	N/A	N/A	N/A	N/A
Coverage ratio, Rc	1.000	1.000	1.000	1.000	1.000
Friction angle along geogrid-soil interface, ρ	28.35	28.35	28.35	28.35	28.35
Pullout resistance factor, F*	0.80·tanϕ	0.80·tanϕ	0.80·tanϕ	0.80·tanϕ	0.80·tanϕ
Scale-effect correction factor, α	1.0	1.0	1.0	1.0	1.0

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.00
3.3 ft	1.00
6.6 ft	1.00
9.8 ft	1.00
13.1 ft	1.00
16.4 ft	1.00
19.7 ft	1.00

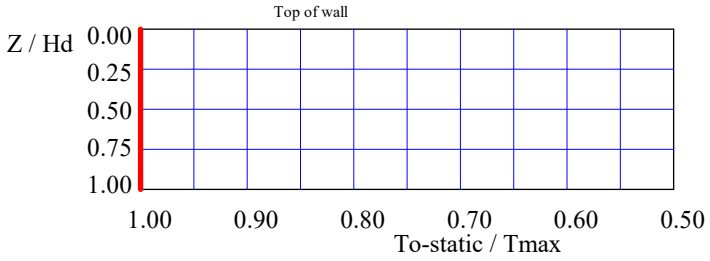


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INPUT DATA: Facia and Connection (according to revised Demo 82) (Analysis)

FACIA type: Facing enabling frictional connection of reinforcement (e.g., modular concrete blocks, gabions)
Depth/height of block is 0.92/0.69 ft. Horizontal distance to Center of Gravity of block is: 0.41 ft.
Average unit weight of block is: γ_f = 81.99 lb/ft ³

Z / Hd	To-static / Tmax
0.00	1.00
0.25	1.00
0.50	1.00
0.75	1.00
1.00	1.00



Connection strength, T-lot, is related to T-ult

Geogrid Type #1		Geogrid Type #2		Geogrid Type #3		Geogrid Type #4		Geogrid Type #5	
N-1	Tult-conn	N-2	Tult-conn	N-3	Tult-conn	N-4	Tult-conn	N-5	Tult-conn
0.0	3353.86	0.0	4055.04	0.0	6597.89	0.0	8338.18	0.0	10129.15
936.0	3353.86	936.0	4055.04	3350.0	6597.89	936.0	8338.18	936.0	10129.15

Geogrid Type #1		Geogrid Type #2		Geogrid Type #3		Geogrid Type #4		Geogrid Type #5	
σ	CRu	σ	CRu	σ	CRu	σ	CRu	σ	CRu
0.0	0.84	0.0	0.84	0.0	0.84	0.0	0.84	0.0	0.84
1021.1	0.84	1021.1	0.84	1021.1	0.84	1021.1	0.84	1021.1	0.84

(1) σ = Confining stress in between stacked blocks [lb/ft ²]
(2) CRult = Tc-ult / Tult
(3) CRcr = Tcre / Tult

D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	UX1100MSE..	UX1400MSE..	UX1500MSE..	UX1600MSE..	UX1700MSE..
Connection strength reduction factor, RFd	1.10	1.10	1.10	1.10	1.10
Creep reduction factor, RFc	N/A	N/A	N/A	N/A	N/A

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INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd 30.00 [ft] { Embedded depth is E = 0.00 ft, and height above top of finished bottom grade is H = 30.00 ft }

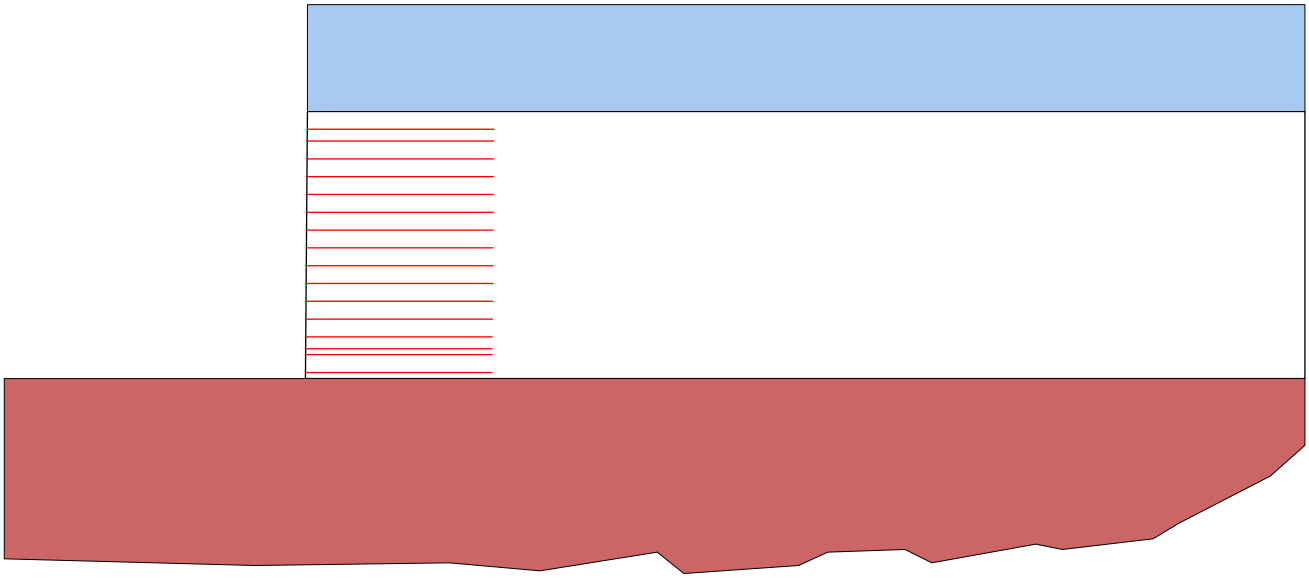
Soil in front of wall is Horizontal.

Batter, ω 0.5 [deg]
Backslope, β 0.0 [deg]
Backslope rise 0.0 [ft] Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

U N I F O R M S U R C H A R G E

Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 250.0 [lb/ft ²]

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 5 10 15 20 25 30 [ft]

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AASHTO 2017-2020 – Load and Resisting Factors

INTERNAL STABILITY

Load factor for vertical earth pressure, EV:	$\gamma_{p\text{-}EV}$	1.35
Load factor for earthquake loads, EQ:	$\gamma_{p\text{-}EQ}$	1.00
Load factor for live load surcharge, LS: (Same as in External Stability).	$\gamma_{p\text{-}LS}$	1.75
Load factor for dead load surcharge, ES: (Same as in External Stability).	$\gamma_{p\text{-}ES}$	1.50

Resistance factor for reinforcement tension	ϕ	Static	Combined static/seismic
Geogrid:		0.90	1.00

Resistance factor for reinforcement tension in connectors	ϕ	Static	Combined static/seismic
Geogrid:		0.90	1.00

Resistance factor for geosynthetic pullout	ϕ	0.90	1.00

EXTERNAL STABILITY

Load factor for vertical earth pressure, EV		Static		Combined Static/Seismic
Sliding and Eccentricity	$\gamma_{p\text{-}EV}$	1.00	$\gamma_{p\text{-}EQ}$	1.00
Bearing Capacity	$\gamma_{p\text{-}EV}$	1.35	$\gamma_{p\text{-}EQ}$	1.35

Load factor of active lateral earth pressure, EH	$\gamma_{p\text{-}EH}$	1.50
Load factor of active lateral earth pressure during earthquake (does not multiply P_{AE} and P_{IR}):	$(\gamma_{p\text{-}EH})_{EQ}$	1.00
Load factor for earthquake loads, EQ (multiplies P_{AE} and P_{IR}):	$\gamma_{p\text{-}EQ}$	1.00

Resistance factor for shear resistance along common interfaces		Static	Combined Static/Seismic
Reinforced Soil and Foundation	ϕ_τ	0.90	1.00
Reinforced Soil and Reinforcement	ϕ_τ	0.90	1.00

Resistance factor for bearing capacity of shallow foundation		Static	Combined Static/Seismic
	ϕ_b	0.65	0.65

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ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, CDR = 2.23, factored bearing load = 7221 lb/ft².
Foundation Interface: Direct sliding, CDR = 1.860, Eccentricity, e/L = 0.0889, CDR-overturning = 3.89

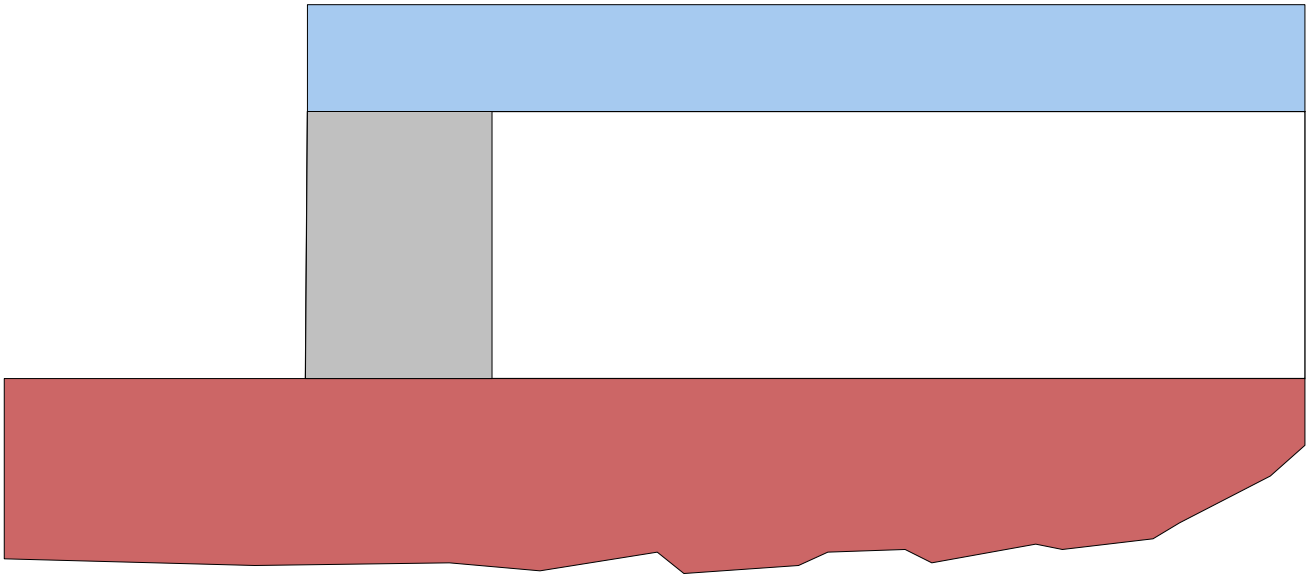
GEOGRID #	Elevation [ft]	Length [ft]	Type #	CONNECTION CDR [connection strength]	Geogrid strength CDR	Pullout resistance CDR	Direct sliding CDR	Eccentricity e/L	Product name
1	0.67	21.00	5	1.19	1.023	29.231	1.769	0.0844	UX1700MSE-MD
2	2.67	21.00	4	1.29	1.139	33.956	1.869	0.0716	UX1600MSE-MD
3	3.34	21.00	3	1.42	1.261	45.192	1.905	0.0675	UX1500MSE-MD
4	4.67	21.00	5	1.37	1.175	26.060	1.982	0.0597	UX1700MSE-MD
5	6.67	21.00	5	1.22	1.048	20.194	2.110	0.0487	UX1700MSE-MD
6	8.67	21.00	5	1.33	1.136	18.818	2.259	0.0386	UX1700MSE-MD
7	10.67	21.00	4	1.19	1.052	17.434	2.432	0.0294	UX1600MSE-MD
8	12.67	21.00	4	1.31	1.158	16.052	2.636	0.0212	UX1600MSE-MD
9	14.67	21.00	3	1.16	1.028	14.646	2.881	0.0139	UX1500MSE-MD
10	16.67	21.00	3	1.30	1.158	13.222	3.179	0.0077	UX1500MSE-MD
11	18.67	21.00	3	1.49	1.327	11.773	3.551	0.0024	UX1500MSE-MD
12	20.67	21.00	3	1.75	1.553	10.294	4.028	-0.0019	UX1500MSE-MD
13	22.67	21.00	2	1.23	1.083	8.744	4.662	-0.0051	UX1400MSE-MD
14	24.67	21.00	1	1.28	1.161	7.082	5.544	-0.0072	UX1100MSE-MD
15	26.67	21.00	1	2.01	1.829	6.060	6.857	-0.0083	UX1100MSE-MD
16	28.00	21.00	1	1.99	1.805	3.238	8.154	-0.0085	UX1100MSE-MD

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BEARING CAPACITY for GIVEN LAYOUT – Using AASHTO 2017-2020 method

	STATIC	SEISMIC	UNITS
(Water table does not affect bearing capacity)			
Factored bearing resistance. q-n	16072	N/A	[lb/ft ²]
Factored bearing load, σ_V	7221.0	N/A	[lb/ft ²]
Eccentricity, e	1.30	N/A	[ft]
Eccentricity, e/L	0.062	N/A	
CDR calculated	2.23	N/A	
Base length	21.00	N/A	[ft]

Unfactored applied bearing pressure = (Unfactored R) / [L - 2 * (Unfactored e)] =
Unfactored R = 96017.66 [lb/ft], L = 21.00, Unfactored e = 1.16 [ft], and Sigma = 5138.24 [lb/ft ²]



SCALE:

0 5 10 15 20 25 30 [ft]

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RESULTS for CONNECTION (static conditions)

Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Connection force, To [lb/ft]	Reduction factor for connection (short-term strength) CRult	Reduction factor for connection (long-term strength) CRcr	Available connection strength [lb/ft]	Available Geogrid strength, Tavailable [lb/ft]	CDR connection strength Specified	CDR connection strength Actual	CDR Geogrid strength Specified	CDR Geogrid strength Actual	Product name
1	0.67	2716	0.84	0.30	2943	3087	N/A	1.19	N/A		UX1700MSE-MD
2	2.67	2068	0.84	0.30	2422	2617	N/A	1.29	N/A		UX1600MSE-MD
3	3.34	1489	0.84	0.33	2109	2086	N/A	1.42	N/A		UX1500MSE-MD
4	4.67	2365	0.84	0.30	2943	3087	N/A	1.37	N/A		UX1700MSE-MD
5	6.67	2652	0.84	0.30	2943	3087	N/A	1.22	N/A		UX1700MSE-MD
6	8.67	2445	0.84	0.30	2943	3087	N/A	1.33	N/A		UX1700MSE-MD
7	10.67	2239	0.84	0.30	2422	2617	N/A	1.19	N/A		UX1600MSE-MD
8	12.67	2033	0.84	0.30	2423	2617	N/A	1.31	N/A		UX1600MSE-MD
9	14.67	1827	0.84	0.33	2109	2086	N/A	1.16	N/A		UX1500MSE-MD
10	16.67	1621	0.84	0.33	2109	2086	N/A	1.30	N/A		UX1500MSE-MD
11	18.67	1415	0.84	0.33	2109	2086	N/A	1.49	N/A		UX1500MSE-MD
12	20.67	1209	0.84	0.33	2109	2086	N/A	1.75	N/A		UX1500MSE-MD
13	22.67	1003	0.84	0.33	1296	1206	N/A	1.23	N/A		UX1400MSE-MD
14	24.67	797	0.84	0.33	1072	1028	N/A	1.28	N/A		UX1100MSE-MD
15	26.67	506	0.84	0.33	1072	1028	N/A	2.01	N/A		UX1100MSE-MD
16	28.00	513	0.84	0.33	1072	1028	N/A	1.99	N/A		UX1100MSE-MD

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Appendix 2.2.1.2a

Problem 2 with Silty Sand

MESA® MSE Retaining Wall System

Back to main document

AASHTO 2017-2020
Mesa Wall System

MSEW+: Update # 2026.01

PROJECT IDENTIFICATION

Title:	Mesa Wall System
Project Number:	Problem 2 Rev1
Client:	IDEA
Designer:	JP
Station Number:	MESA - AASHTO

Description:

MESA System, DH = 30.0 ft; 75 yrs Design Life; Silty Sand backfill

Company's information:

Name: Tensar International Corporation
Street: 2500 Northwinds Pkwy.
Suite 500
Alpharetta, GA 30009
Telephone #: (770) 344-2090
Fax #: (770) 344-2084
E-Mail: Jorge.Peralta@cmc.com

File path and name: C:\Users\P01000987\OneDrive - CMC\Desktop\MESA IDEA com..... ..... 75yr_SiltySand.BENp

Original date and time of creating this file: May 2026

PROGRAM MODE:

ANALYSIS
of a SIMPLE STRUCTURE
using GEOGRID as reinforcing material.

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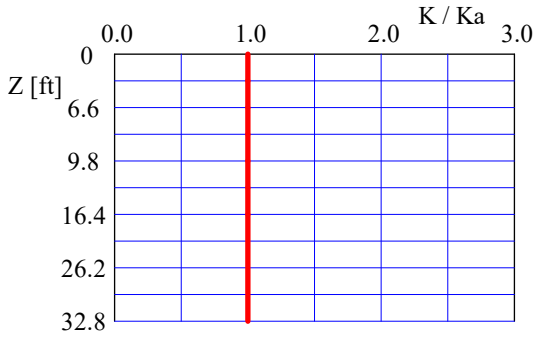
Version MSEW+ Version MSEW+ Version MSEW+

INPUT DATA: Geogrids (Analysis)

D A T A	Geogrid type #1	Geogrid type #2	Geogrid type #3	Geogrid type #4	Geogrid type #5
Tult [lb/ft]	3970.0	4800.0	7810.0	9870.0	11990.0
Durability reduction factor, RFd	1.10	1.10	1.10	1.10	1.10
Installation-damage reduction factor, RFid	1.24	1.15	1.14	1.15	1.18
Creep reduction factor, RFc	2.68	2.68	2.54	2.54	2.54
CDR for strength	N/A	N/A	N/A	N/A	N/A
Coverage ratio, Rc	1.000	1.000	1.000	1.000	1.000
Friction angle along geogrid-soil interface, ρ	28.35	28.35	28.35	28.35	28.35
Pullout resistance factor, F*	0.80·tanϕ	0.80·tanϕ	0.80·tanϕ	0.80·tanϕ	0.80·tanϕ
Scale-effect correction factor, α	1.0	1.0	1.0	1.0	1.0

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.00
3.3 ft	1.00
6.6 ft	1.00
9.8 ft	1.00
13.1 ft	1.00
16.4 ft	1.00
19.7 ft	1.00

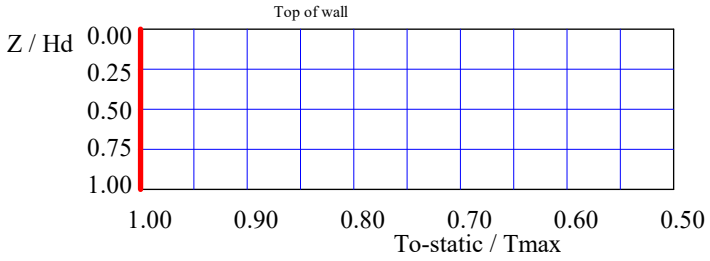


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INPUT DATA: Facia and Connection (according to revised Demo 82) (Analysis)

FACIA type: Facing enabling frictional connection of reinforcement (e.g., modular concrete blocks, gabions)
Depth/height of block is 0.92/0.69 ft. Horizontal distance to Center of Gravity of block is: 0.41 ft.
Average unit weight of block is: γ_f = 81.99 lb/ft ³

Z / Hd	To-static / Tmax
0.00	1.00
0.25	1.00
0.50	1.00
0.75	1.00
1.00	1.00



Connection strength, T-lot, is related to T-ult

Geogrid Type #1		Geogrid Type #2		Geogrid Type #3		Geogrid Type #4		Geogrid Type #5	
N-1	Tult-conn	N-2	Tult-conn	N-3	Tult-conn	N-4	Tult-conn	N-5	Tult-conn
0.0	3353.86	0.0	4055.04	0.0	6597.89	0.0	8338.18	0.0	10129.15
936.0	3353.86	936.0	4055.04	3350.0	6597.89	936.0	8338.18	936.0	10129.15

Geogrid Type #1		Geogrid Type #2		Geogrid Type #3		Geogrid Type #4		Geogrid Type #5	
σ	CRu	σ	CRu	σ	CRu	σ	CRu	σ	CRu
0.0	0.84	0.0	0.84	0.0	0.84	0.0	0.84	0.0	0.84
1021.1	0.84	1021.1	0.84	1021.1	0.84	1021.1	0.84	1021.1	0.84

(1) σ = Confining stress in between stacked blocks [lb/ft ²]
(2) CRult = Tc-ult / Tult
(3) CRcr = Tcre / Tult

D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	UX1100MSE..	UX1400MSE..	UX1500MSE..	UX1600MSE..	UX1700MSE..
Connection strength reduction factor, RFd	1.10	1.10	1.10	1.10	1.10
Creep reduction factor, RFc	N/A	N/A	N/A	N/A	N/A

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INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

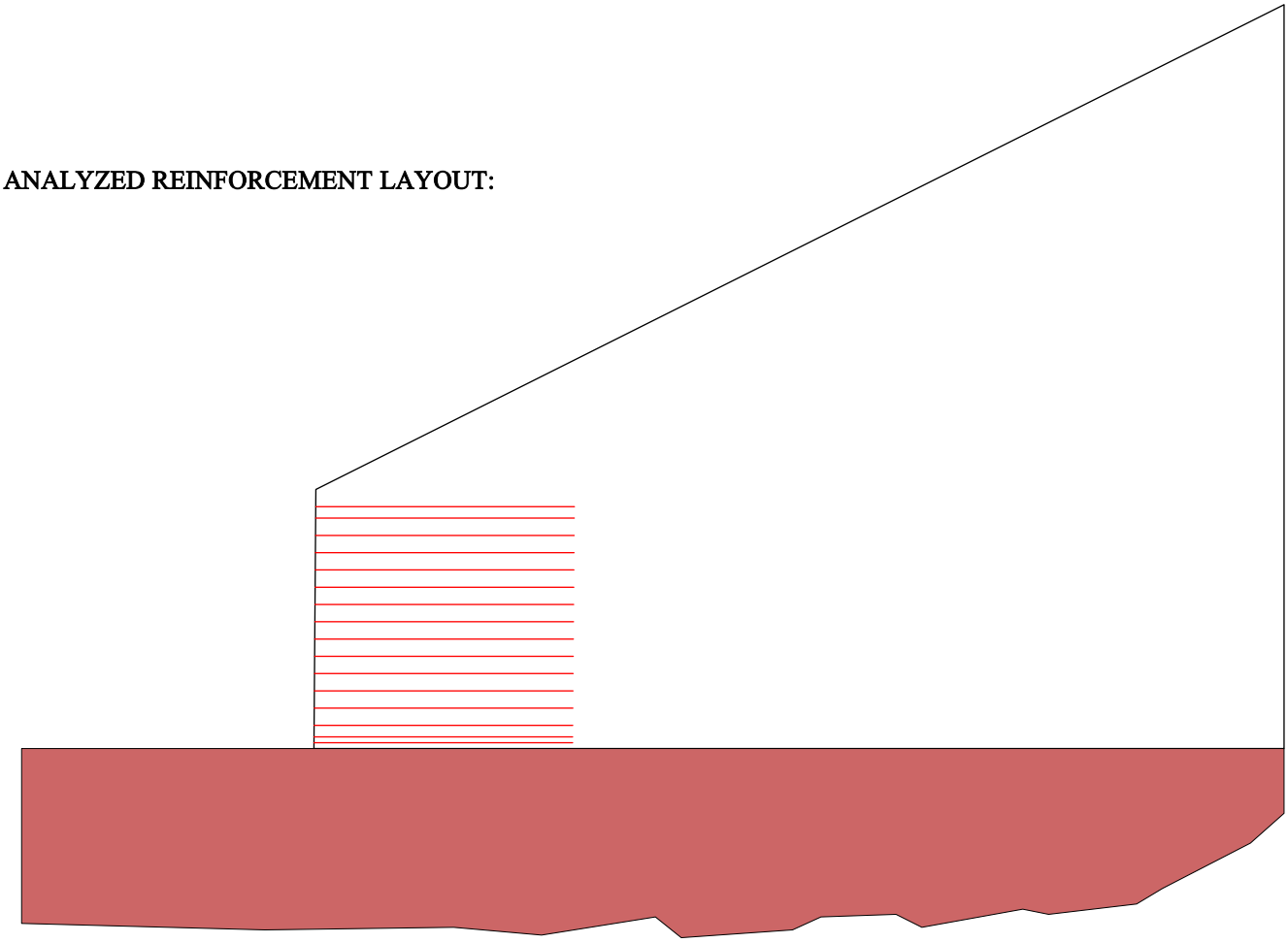
Design height, Hd 30.00 [ft] { Embedded depth is E = 0.00 ft, and height above top of finished bottom grade is H = 30.00 ft }

Soil in front of wall is Horizontal.

Batter, ω 0.5 [deg]
Backslope, β 26.6 [deg]
Backslope rise 60.0 [ft] Broken back equivalent angle, I = 26.56° (see Fig. 25 in DEMO 82)

U N I F O R M S U R C H A R G E
Uniformly distributed dead load is 0.0 [lb/ft ²]

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 5 10 15 20 25 30 [ft]

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AASHTO 2017-2020 – Load and Resisting Factors

INTERNAL STABILITY

Load factor for vertical earth pressure, EV:	$\gamma_{p\text{-}EV}$	1.35
Load factor for earthquake loads, EQ:	$\gamma_{p\text{-}EQ}$	1.00
Load factor for live load surcharge, LS: (Same as in External Stability).	$\gamma_{p\text{-}LS}$	1.75
Load factor for dead load surcharge, ES: (Same as in External Stability).	$\gamma_{p\text{-}ES}$	1.50

Resistance factor for reinforcement tension	ϕ	Static	Combined static/seismic
Geogrid:		0.90	1.00

Resistance factor for reinforcement tension in connectors	ϕ	Static	Combined static/seismic
Geogrid:		0.90	1.00
Resistance factor for geosynthetic pullout	ϕ	0.90	1.00

EXTERNAL STABILITY

Load factor for vertical earth pressure, EV			Static	Combined Static/Seismic	
Sliding and Eccentricity		$\gamma_{p\text{-}EV}$	1.00	$\gamma_{p\text{-}EQ}$	1.00
Bearing Capacity		$\gamma_{p\text{-}EV}$	1.35	$\gamma_{p\text{-}EQ}$	1.35

Load factor of active lateral earth pressure, EH	$\gamma_{p\text{-}EH}$	1.50
Load factor of active lateral earth pressure during earthquake (does not multiply P_{AE} and P_{IR}):	$(\gamma_{p\text{-}EH})_{EQ}$	1.00
Load factor for earthquake loads, EQ (multiplies P_{AE} and P_{IR}):	$\gamma_{p\text{-}EQ}$	1.00

Resistance factor for shear resistance along common interfaces			Static	Combined Static/Seismic	
Reinforced Soil and Foundation		ϕ_τ	0.90		1.00
Reinforced Soil and Reinforcement		ϕ_τ	0.90		1.00

Resistance factor for bearing capacity of shallow foundation		Static	Combined Static/Seismic
	ϕ_b	0.65	0.65

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ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, CDR = 2.24, factored bearing load = 9590 lb/ft².
Foundation Interface: Direct sliding, CDR = 1.067, Eccentricity, e/L = 0.1253, CDR-overturning = 2.54

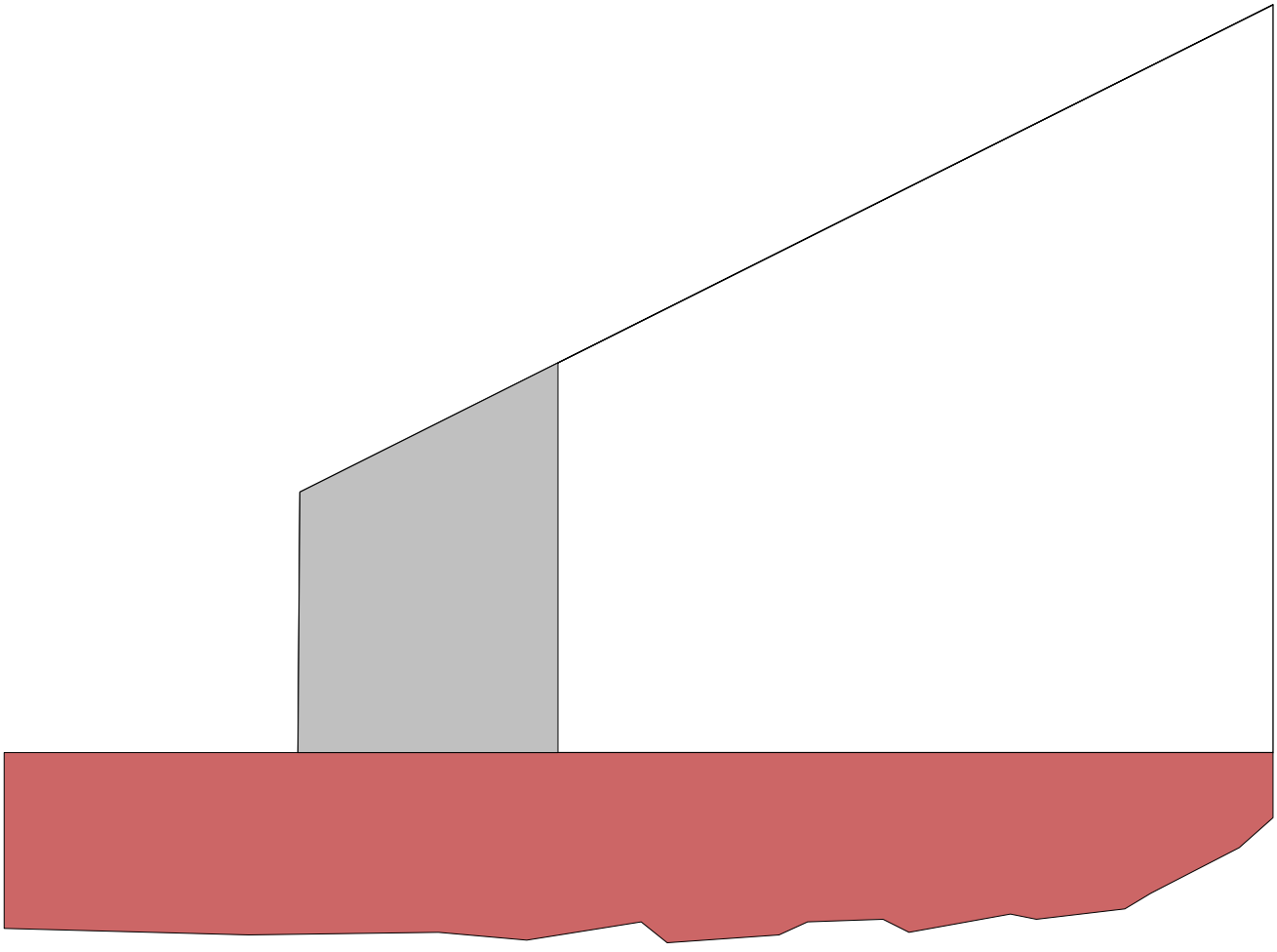
GEOGRID #	Elevation [ft]	Length [ft]	Type #	CONNECTION CDR [connection strength]	Geogrid strength CDR	Pullout resistance CDR	Direct sliding CDR	Eccentricity e/L	Product name
1	0.67	30.00	5	1.80	1.819	79.500	1.007	0.1197	UX1700MSE-MD
2	1.34	30.00	5	1.87	1.883	80.030	1.018	0.1141	UX1700MSE-MD
3	2.67	30.00	5	1.17	1.177	47.288	1.038	0.1032	UX1700MSE-MD
4	4.67	30.00	5	1.03	1.039	38.226	1.072	0.0870	UX1700MSE-MD
5	6.67	30.00	5	1.10	1.111	37.300	1.107	0.0713	UX1700MSE-MD
6	8.67	30.00	5	1.18	1.195	36.408	1.145	0.0559	UX1700MSE-MD
7	10.67	30.00	4	1.05	1.092	35.560	1.186	0.0407	UX1600MSE-MD
8	12.67	30.00	4	1.15	1.188	34.784	1.231	0.0257	UX1600MSE-MD
9	14.67	30.00	3	1.00	1.041	34.061	1.278	0.0107	UX1500MSE-MD
10	16.67	30.00	3	1.10	1.153	33.432	1.328	-0.0044	UX1500MSE-MD
11	18.67	30.00	3	1.24	1.292	32.929	1.382	-0.0198	UX1500MSE-MD
12	20.67	30.00	3	1.41	1.469	32.627	1.438	-0.0359	UX1500MSE-MD
13	22.67	30.00	3	1.63	1.702	32.570	1.496	-0.0534	UX1500MSE-MD
14	24.67	30.00	2	1.13	1.169	32.930	1.554	-0.0730	UX1400MSE-MD
15	26.67	30.00	1	1.36	1.303	40.054	1.607	-0.0964	UX1100MSE-MD
16	28.00	30.00	1	1.13	1.082	29.259	1.635	-0.1155	UX1100MSE-MD

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BEARING CAPACITY for GIVEN LAYOUT – Using AASHTO 2017-2020 method

	STATIC	SEISMIC	UNITS
(Water table does not affect bearing capacity)			
Factored bearing resistance. q-n	21523	N/A	[lb/ft ²]
Factored bearing load, σ_V	9590.0	N/A	[lb/ft ²]
Eccentricity, e	2.68	N/A	[ft]
Eccentricity, e/L	0.089	N/A	
CDR calculated	2.24	N/A	
Base length	30.00	N/A	[ft]

Unfactored applied bearing pressure = (Unfactored R) / [L - 2 * (Unfactored e)] =
Unfactored R = 172584.83 [lb/ft], L = 30.00, Unfactored e = 2.36 [ft], and Sigma = 6824.91 [lb/ft ²]



SCALE:

0 5 10 15 20 25 30 [ft]

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DIRECT SLIDING for GIVEN LAYOUT (for GEOGRID reinforcements)

Along reinforced and foundation soils interface: CDR-static = 1.067

#	Geogrid Elevation [ft]	Geogrid Length [ft]	CDR Static	CDR Seismic	Geogrid Type #	Product name
1	0.67	30.00	1.007	N/A	5	UX1700MSE-MD
2	1.34	30.00	1.018	N/A	5	UX1700MSE-MD
3	2.67	30.00	1.038	N/A	5	UX1700MSE-MD
4	4.67	30.00	1.072	N/A	5	UX1700MSE-MD
5	6.67	30.00	1.107	N/A	5	UX1700MSE-MD
6	8.67	30.00	1.145	N/A	5	UX1700MSE-MD
7	10.67	30.00	1.186	N/A	4	UX1600MSE-MD
8	12.67	30.00	1.231	N/A	4	UX1600MSE-MD
9	14.67	30.00	1.278	N/A	3	UX1500MSE-MD
10	16.67	30.00	1.328	N/A	3	UX1500MSE-MD
11	18.67	30.00	1.382	N/A	3	UX1500MSE-MD
12	20.67	30.00	1.438	N/A	3	UX1500MSE-MD
13	22.67	30.00	1.496	N/A	3	UX1500MSE-MD
14	24.67	30.00	1.554	N/A	2	UX1400MSE-MD
15	26.67	30.00	1.607	N/A	1	UX1100MSE-MD
16	28.00	30.00	1.635	N/A	1	UX1100MSE-MD

ECCENTRICITY for GIVEN LAYOUT

At interface with foundation: e/L static = 0.1253; Overturning: CDR-static = 2.54

#	Geogrid Elevation [ft]	Geogrid Length [ft]	e / L Static	e / L Seismic	Geogrid Type #	Product name
1	0.67	30.00	0.1197	N/A	5	UX1700MSE-MD
2	1.34	30.00	0.1141	N/A	5	UX1700MSE-MD
3	2.67	30.00	0.1032	N/A	5	UX1700MSE-MD
4	4.67	30.00	0.0870	N/A	5	UX1700MSE-MD
5	6.67	30.00	0.0713	N/A	5	UX1700MSE-MD
6	8.67	30.00	0.0559	N/A	5	UX1700MSE-MD
7	10.67	30.00	0.0407	N/A	4	UX1600MSE-MD
8	12.67	30.00	0.0257	N/A	4	UX1600MSE-MD
9	14.67	30.00	0.0107	N/A	3	UX1500MSE-MD
10	16.67	30.00	-0.0044	N/A	3	UX1500MSE-MD
11	18.67	30.00	-0.0198	N/A	3	UX1500MSE-MD
12	20.67	30.00	-0.0359	N/A	3	UX1500MSE-MD
13	22.67	30.00	-0.0534	N/A	3	UX1500MSE-MD
14	24.67	30.00	-0.0730	N/A	2	UX1400MSE-MD
15	26.67	30.00	-0.0964	N/A	1	UX1100MSE-MD
16	28.00	30.00	-0.1155	N/A	1	UX1100MSE-MD

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RESULTS for STRENGTH

Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Factored LTDS [lb/ft]	Factored Tmax [lb/ft]	Tmd [lb/ft]	Specified minimum CDR Static	Actual calculated CDR Static	Specified minimum CDR seismic	Actual calculated CDR seismic	Product name
1	0.67	3273.1	1799.2	N/A	N/A	1.819	N/A	N/A	UX1700MSE-MD
2	1.34	3273.1	1738.6	N/A	N/A	1.883	N/A	N/A	UX1700MSE-MD
3	2.67	3273.1	2780.5	N/A	N/A	1.177	N/A	N/A	UX1700MSE-MD
4	4.67	3273.1	3151.1	N/A	N/A	1.039	N/A	N/A	UX1700MSE-MD
5	6.67	3273.1	2945.0	N/A	N/A	1.111	N/A	N/A	UX1700MSE-MD
6	8.67	3273.1	2738.9	N/A	N/A	1.195	N/A	N/A	UX1700MSE-MD
7	10.67	2764.6	2532.8	N/A	N/A	1.092	N/A	N/A	UX1600MSE-MD
8	12.67	2764.6	2326.7	N/A	N/A	1.188	N/A	N/A	UX1600MSE-MD
9	14.67	2206.8	2120.6	N/A	N/A	1.041	N/A	N/A	UX1500MSE-MD
10	16.67	2206.8	1914.5	N/A	N/A	1.153	N/A	N/A	UX1500MSE-MD
11	18.67	2206.8	1708.4	N/A	N/A	1.292	N/A	N/A	UX1500MSE-MD
12	20.67	2206.8	1502.3	N/A	N/A	1.469	N/A	N/A	UX1500MSE-MD
13	22.67	2206.8	1296.2	N/A	N/A	1.702	N/A	N/A	UX1500MSE-MD
14	24.67	1274.3	1090.1	N/A	N/A	1.169	N/A	N/A	UX1400MSE-MD
15	26.67	977.4	750.3	N/A	N/A	1.303	N/A	N/A	UX1100MSE-MD
16	28.00	977.4	903.7	N/A	N/A	1.082	N/A	N/A	UX1100MSE-MD

RESULTS for PULLOUT

Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Coverage Ratio	Factored: Stat./Seis. Tmax [lb/ft]	Tmd [lb/ft]	Le [ft]	La [ft]	Avail.Static Pullout, Pr [lb/ft]	Specified Static CDR	Actual Static CDR	Avail.Seism. Pullout, Pr [lb/ft]	Specified seismic CDR	Actual seismic CDR
1	0.67	1.000	1799.2	N/A	29.65	0.35	143040.7	N/A	79.500	N/A	N/A	N/A
2	1.34	1.000	1738.6	N/A	29.30	0.70	139144.2	N/A	80.030	N/A	N/A	N/A
3	2.67	1.000	2780.5	N/A	28.60	1.40	131484.6	N/A	47.288	N/A	N/A	N/A
4	4.67	1.000	3151.1	N/A	27.55	2.45	120454.5	N/A	38.226	N/A	N/A	N/A
5	6.67	1.000	2945.0	N/A	26.51	3.49	109849.2	N/A	37.300	N/A	N/A	N/A
6	8.67	1.000	2738.9	N/A	25.46	4.54	99720.4	N/A	36.408	N/A	N/A	N/A
7	10.67	1.000	2532.8	N/A	24.41	5.59	90068.3	N/A	35.560	N/A	N/A	N/A
8	12.67	1.000	2326.7	N/A	23.36	6.64	80932.4	N/A	34.784	N/A	N/A	N/A
9	14.67	1.000	2120.6	N/A	22.32	7.68	72231.0	N/A	34.061	N/A	N/A	N/A
10	16.67	1.000	1914.5	N/A	21.27	8.73	64006.2	N/A	33.432	N/A	N/A	N/A
11	18.67	1.000	1708.4	N/A	20.22	9.78	56258.1	N/A	32.929	N/A	N/A	N/A
12	20.67	1.000	1502.3	N/A	19.17	10.83	49016.4	N/A	32.627	N/A	N/A	N/A
13	22.67	1.000	1296.2	N/A	18.12	11.88	42219.0	N/A	32.570	N/A	N/A	N/A
14	24.67	1.000	1090.1	N/A	17.08	12.92	35898.2	N/A	32.930	N/A	N/A	N/A
15	26.67	1.000	750.3	N/A	16.03	13.97	30054.0	N/A	40.054	N/A	N/A	N/A
16	28.00	1.000	903.7	N/A	15.33	14.67	26441.5	N/A	29.259	N/A	N/A	N/A

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Version MSEW+ Version MSEW+ Version MSEW+

Appendix 2.2.1.2b

Problem 2 with Sandy Gravel

MESA® MSE Retaining Wall System

Back to main document

AASHTO 2017-2020
Mesa Wall System

MSEW+: Update # 2026.01

PROJECT IDENTIFICATION

Title: Mesa Wall System
Project Number: Problem 2 Rev1
Client: IDEA
Designer: JP
Station Number: MESA - AASHTO

Description:

MESA System, DH = 30.0 ft; 75 yrs Design Life; Sandy Gravel backfill material

Company's information:

Name: Tensar International Corporation
Street: 2500 Northwinds Pkwy.
Suite 500
Alpharetta, GA 30009
Telephone #: (770) 344-2090
Fax #: (770) 344-2084
E-Mail: Jorge.Peralta@cmc.com

File path and name: C:\Users\P01000987\OneDrive - CMC\Desktop\MESA IDEA com.....
.....5yr_SandyGravel.BENp

Original date and time of creating this file: May 2026

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using GEOGRID as reinforcing material.

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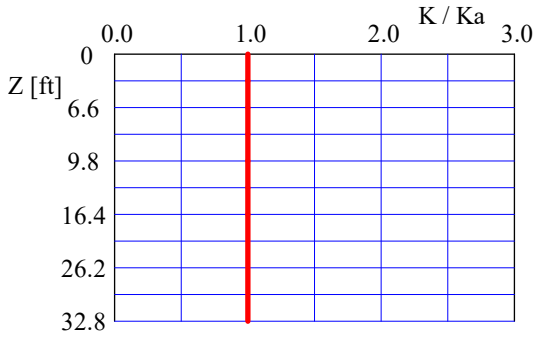
Version MSEW+ Version MSEW+ Version MSEW+

INPUT DATA: Geogrids (Analysis)

D A T A	Geogrid type #1	Geogrid type #2	Geogrid type #3	Geogrid type #4	Geogrid type #5
Tult [lb/ft]	3970.0	4800.0	7810.0	9870.0	11990.0
Durability reduction factor, RFd	1.10	1.10	1.10	1.10	1.10
Installation-damage reduction factor, RFid	1.31	1.35	1.34	1.35	1.39
Creep reduction factor, RFc	2.68	2.68	2.54	2.54	2.54
CDR for strength	N/A	N/A	N/A	N/A	N/A
Coverage ratio, Rc	1.000	1.000	1.000	1.000	1.000
Friction angle along geogrid-soil interface, ρ	28.35	28.35	28.35	28.35	28.35
Pullout resistance factor, F*	0.80·tanϕ	0.80·tanϕ	0.80·tanϕ	0.80·tanϕ	0.80·tanϕ
Scale-effect correction factor, α	1.0	1.0	1.0	1.0	1.0

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.00
3.3 ft	1.00
6.6 ft	1.00
9.8 ft	1.00
13.1 ft	1.00
16.4 ft	1.00
19.7 ft	1.00

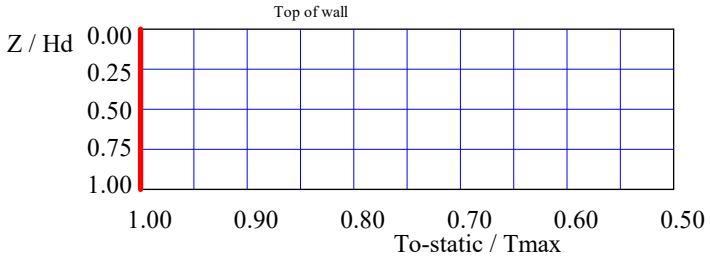


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INPUT DATA: Facia and Connection (according to revised Demo 82) (Analysis)

FACIA type: Facing enabling frictional connection of reinforcement (e.g., modular concrete blocks, gabions)
Depth/height of block is 0.92/0.69 ft. Horizontal distance to Center of Gravity of block is: 0.41 ft.
Average unit weight of block is: γ_f = 81.99 lb/ft ³

Z / Hd	To-static / Tmax
0.00	1.00
0.25	1.00
0.50	1.00
0.75	1.00
1.00	1.00



Connection strength, T-lot, is related to T-ult

Geogrid Type #1		Geogrid Type #2		Geogrid Type #3		Geogrid Type #4		Geogrid Type #5	
N-1	Tult-conn	N-2	Tult-conn	N-3	Tult-conn	N-4	Tult-conn	N-5	Tult-conn
0.0	3353.86	0.0	4055.04	0.0	6597.89	0.0	8338.18	0.0	10129.15
936.0	3353.86	936.0	4055.04	3350.0	6597.89	936.0	8338.18	936.0	10129.15

Geogrid Type #1		Geogrid Type #2		Geogrid Type #3		Geogrid Type #4		Geogrid Type #5	
σ	CRu	σ	CRu	σ	CRu	σ	CRu	σ	CRu
0.0	0.84	0.0	0.84	0.0	0.84	0.0	0.84	0.0	0.84
1021.1	0.84	1021.1	0.84	1021.1	0.84	1021.1	0.84	1021.1	0.84

(1) σ = Confining stress in between stacked blocks [lb/ft ²]
(2) CRult = Tc-ult / Tult
(3) CRcr = Tcre / Tult

D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	UX1100MSE..	UX1400MSE..	UX1500MSE..	UX1600MSE..	UX1700MSE..
Connection strength reduction factor, RFd	1.10	1.10	1.10	1.10	1.10
Creep reduction factor, RFc	N/A	N/A	N/A	N/A	N/A

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INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

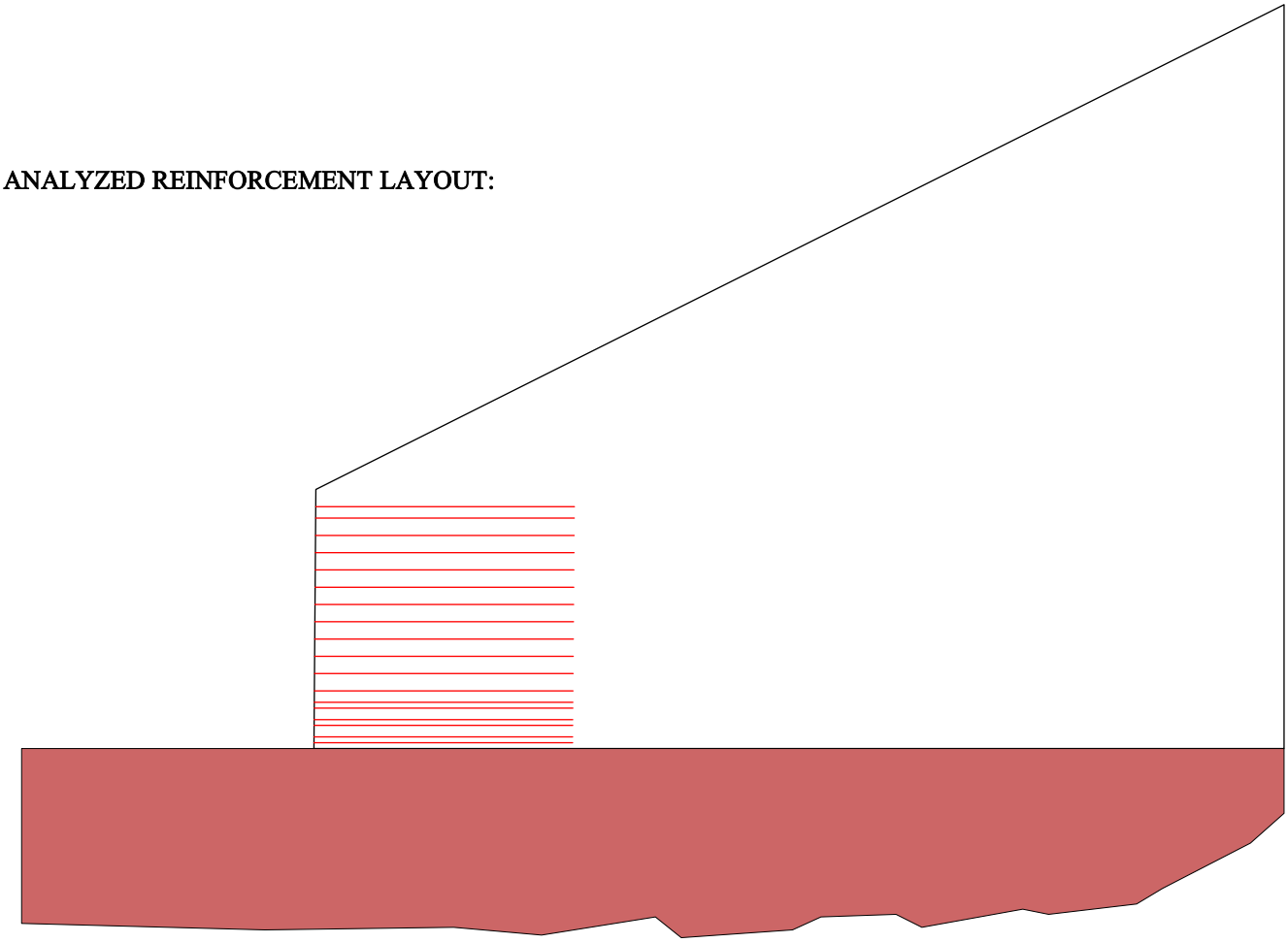
Design height, Hd 30.00 [ft] { Embedded depth is E = 0.00 ft, and height above top of finished bottom grade is H = 30.00 ft }

Soil in front of wall is Horizontal.

Batter, ω 0.5 [deg]
Backslope, β 26.6 [deg]
Backslope rise 60.0 [ft] Broken back equivalent angle, I = 26.56° (see Fig. 25 in DEMO 82)

U N I F O R M S U R C H A R G E
Uniformly distributed dead load is 0.0 [lb/ft ²]

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 5 10 15 20 25 30 [ft]

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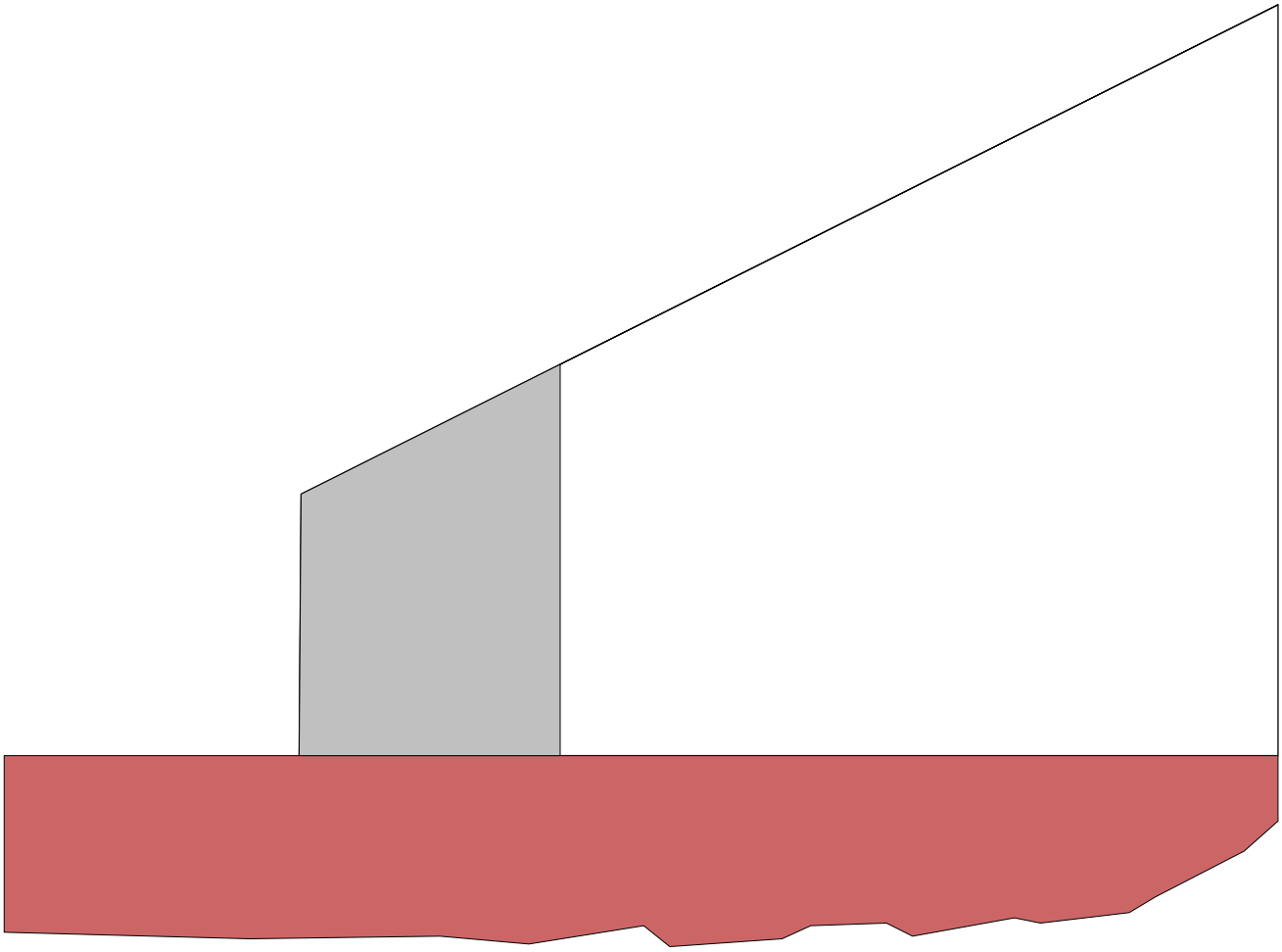
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BEARING CAPACITY for GIVEN LAYOUT – Using AASHTO 2017-2020 method

	STATIC	SEISMIC	UNITS
(Water table does not affect bearing capacity)			
Factored bearing resistance. q-n	21523	N/A	[lb/ft ²]
Factored bearing load, σ_V	9590.0	N/A	[lb/ft ²]
Eccentricity, e	2.68	N/A	[ft]
Eccentricity, e/L	0.089	N/A	
CDR calculated	2.24	N/A	
Base length	30.00	N/A	[ft]

Unfactored applied bearing pressure = (Unfactored R) / [L - 2 * (Unfactored e)] =
Unfactored R = 172584.83 [lb/ft], L = 30.00, Unfactored e = 2.36 [ft], and Sigma = 6824.91 [lb/ft ²]



SCALE:

0 5 10 15 20 25 30 [ft]

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DIRECT SLIDING for GIVEN LAYOUT (for GEOGRID reinforcements)

Along reinforced and foundation soils interface: CDR-static = 1.067

#	Geogrid Elevation [ft]	Geogrid Length [ft]	CDR Static	CDR Seismic	Geogrid Type #	Product name
1	0.67	30.00	1.007	N/A	3	UX1500MSE-MD
2	1.34	30.00	1.018	N/A	3	UX1500MSE-MD
3	2.67	30.00	1.038	N/A	3	UX1500MSE-MD
4	3.34	30.00	1.049	N/A	3	UX1500MSE-MD
5	4.67	30.00	1.072	N/A	3	UX1500MSE-MD
6	5.34	30.00	1.083	N/A	3	UX1500MSE-MD
7	6.67	30.00	1.107	N/A	5	UX1700MSE-MD
8	8.67	30.00	1.145	N/A	5	UX1700MSE-MD
9	10.67	30.00	1.186	N/A	5	UX1700MSE-MD
10	12.67	30.00	1.231	N/A	4	UX1600MSE-MD
11	14.67	30.00	1.278	N/A	4	UX1600MSE-MD
12	16.67	30.00	1.328	N/A	4	UX1600MSE-MD
13	18.67	30.00	1.382	N/A	3	UX1500MSE-MD
14	20.67	30.00	1.438	N/A	3	UX1500MSE-MD
15	22.67	30.00	1.496	N/A	3	UX1500MSE-MD
16	24.67	30.00	1.554	N/A	3	UX1500MSE-MD
17	26.67	30.00	1.607	N/A	1	UX1100MSE-MD
18	28.00	30.00	1.635	N/A	1	UX1100MSE-MD

ECCENTRICITY for GIVEN LAYOUT

At interface with foundation: e/L static = 0.1253; Overturning: CDR-static = 2.54

#	Geogrid Elevation [ft]	Geogrid Length [ft]	e / L Static	e / L Seismic	Geogrid Type #	Product name
1	0.67	30.00	0.1197	N/A	3	UX1500MSE-MD
2	1.34	30.00	0.1141	N/A	3	UX1500MSE-MD
3	2.67	30.00	0.1032	N/A	3	UX1500MSE-MD
4	3.34	30.00	0.0977	N/A	3	UX1500MSE-MD
5	4.67	30.00	0.0870	N/A	3	UX1500MSE-MD
6	5.34	30.00	0.0817	N/A	3	UX1500MSE-MD
7	6.67	30.00	0.0713	N/A	5	UX1700MSE-MD
8	8.67	30.00	0.0559	N/A	5	UX1700MSE-MD
9	10.67	30.00	0.0407	N/A	5	UX1700MSE-MD
10	12.67	30.00	0.0257	N/A	4	UX1600MSE-MD
11	14.67	30.00	0.0107	N/A	4	UX1600MSE-MD
12	16.67	30.00	-0.0044	N/A	4	UX1600MSE-MD
13	18.67	30.00	-0.0198	N/A	3	UX1500MSE-MD
14	20.67	30.00	-0.0359	N/A	3	UX1500MSE-MD
15	22.67	30.00	-0.0534	N/A	3	UX1500MSE-MD
16	24.67	30.00	-0.0730	N/A	3	UX1500MSE-MD
17	26.67	30.00	-0.0964	N/A	1	UX1100MSE-MD
18	28.00	30.00	-0.1155	N/A	1	UX1100MSE-MD

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RESULTS for STRENGTH

Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Factored LTDS [lb/ft]	Factored Tmax [lb/ft]	Tmd [lb/ft]	Specified minimum CDR Static	Actual calculated CDR Static	Specified minimum CDR seismic	Actual calculated CDR seismic	Product name
1	0.67	1877.4	1799.2	N/A	N/A	1.043	N/A	N/A	UX1500MSE-MD
2	1.34	1877.4	1738.6	N/A	N/A	1.080	N/A	N/A	UX1500MSE-MD
3	2.67	1877.4	1687.1	N/A	N/A	1.113	N/A	N/A	UX1500MSE-MD
4	3.34	1877.4	1635.6	N/A	N/A	1.148	N/A	N/A	UX1500MSE-MD
5	4.67	1877.4	1584.1	N/A	N/A	1.185	N/A	N/A	UX1500MSE-MD
6	5.34	1877.4	1532.5	N/A	N/A	1.225	N/A	N/A	UX1500MSE-MD
7	6.67	2778.6	2437.4	N/A	N/A	1.140	N/A	N/A	UX1700MSE-MD
8	8.67	2778.6	2738.9	N/A	N/A	1.014	N/A	N/A	UX1700MSE-MD
9	10.67	2778.6	2532.8	N/A	N/A	1.097	N/A	N/A	UX1700MSE-MD
10	12.67	2355.0	2326.7	N/A	N/A	1.012	N/A	N/A	UX1600MSE-MD
11	14.67	2355.0	2120.6	N/A	N/A	1.111	N/A	N/A	UX1600MSE-MD
12	16.67	2355.0	1914.5	N/A	N/A	1.230	N/A	N/A	UX1600MSE-MD
13	18.67	1877.4	1708.4	N/A	N/A	1.099	N/A	N/A	UX1500MSE-MD
14	20.67	1877.4	1502.3	N/A	N/A	1.250	N/A	N/A	UX1500MSE-MD
15	22.67	1877.4	1296.2	N/A	N/A	1.448	N/A	N/A	UX1500MSE-MD
16	24.67	1877.4	1090.1	N/A	N/A	1.722	N/A	N/A	UX1500MSE-MD
17	26.67	925.2	750.3	N/A	N/A	1.233	N/A	N/A	UX1100MSE-MD
18	28.00	925.2	903.7	N/A	N/A	1.024	N/A	N/A	UX1100MSE-MD

RESULTS for PULLOUT

Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Coverage Ratio	Factored: Stat./Seis. Tmax [lb/ft]	Tmd [lb/ft]	Le [ft]	La [ft]	Avail.Static Pullout, Pr [lb/ft]	Specified Static CDR	Actual Static CDR	Avail.Seism. Pullout, Pr [lb/ft]	Specified seismic CDR	Actual seismic CDR
1	0.67	1.000	1799.2	N/A	29.65	0.35	143040.7	N/A	79.500	N/A	N/A	N/A
2	1.34	1.000	1738.6	N/A	29.30	0.70	139144.2	N/A	80.030	N/A	N/A	N/A
3	2.67	1.000	1687.1	N/A	28.60	1.40	131484.6	N/A	77.934	N/A	N/A	N/A
4	3.34	1.000	1635.6	N/A	28.25	1.75	127746.4	N/A	78.104	N/A	N/A	N/A
5	4.67	1.000	1584.1	N/A	27.55	2.45	120454.5	N/A	76.041	N/A	N/A	N/A
6	5.34	1.000	1532.5	N/A	27.20	2.80	116825.3	N/A	76.230	N/A	N/A	N/A
7	6.67	1.000	2437.4	N/A	26.51	3.49	109849.2	N/A	45.069	N/A	N/A	N/A
8	8.67	1.000	2738.9	N/A	25.46	4.54	99720.4	N/A	36.408	N/A	N/A	N/A
9	10.67	1.000	2532.8	N/A	24.41	5.59	90068.3	N/A	35.560	N/A	N/A	N/A
10	12.67	1.000	2326.7	N/A	23.36	6.64	80932.4	N/A	34.784	N/A	N/A	N/A
11	14.67	1.000	2120.6	N/A	22.32	7.68	72231.0	N/A	34.061	N/A	N/A	N/A
12	16.67	1.000	1914.5	N/A	21.27	8.73	64006.2	N/A	33.432	N/A	N/A	N/A
13	18.67	1.000	1708.4	N/A	20.22	9.78	56258.1	N/A	32.929	N/A	N/A	N/A
14	20.67	1.000	1502.3	N/A	19.17	10.83	49016.4	N/A	32.627	N/A	N/A	N/A
15	22.67	1.000	1296.2	N/A	18.12	11.88	42219.0	N/A	32.570	N/A	N/A	N/A
16	24.67	1.000	1090.1	N/A	17.08	12.92	35898.2	N/A	32.930	N/A	N/A	N/A
17	26.67	1.000	750.3	N/A	16.03	13.97	30054.0	N/A	40.054	N/A	N/A	N/A
18	28.00	1.000	903.7	N/A	15.33	14.67	26441.5	N/A	29.259	N/A	N/A	N/A

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RESULTS for CONNECTION (static conditions)

Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Connection force, To [lb/ft]	Reduction factor for connection (short-term strength) CRult	Reduction factor for connection (long-term strength) CRcr	Available connection strength [lb/ft]	Available Geogrid strength, Tavailable [lb/ft]	CDR connection strength Specified	CDR connection strength Actual	CDR Geogrid strength Specified	CDR Geogrid strength Actual	Product name
1	0.67	1799	0.84	0.33	2109	2086	N/A	1.17	N/A		UX1500MSE-MD
2	1.34	1739	0.84	0.33	2109	2086	N/A	1.22	N/A		UX1500MSE-MD
3	2.67	1687	0.84	0.33	2109	2086	N/A	1.25	N/A		UX1500MSE-MD
4	3.34	1636	0.84	0.33	2109	2086	N/A	1.29	N/A		UX1500MSE-MD
5	4.67	1584	0.84	0.33	2109	2086	N/A	1.33	N/A		UX1500MSE-MD
6	5.34	1533	0.84	0.33	2109	2086	N/A	1.38	N/A		UX1500MSE-MD
7	6.67	2437	0.84	0.30	2943	3087	N/A	1.33	N/A		UX1700MSE-MD
8	8.67	2739	0.84	0.30	2943	3087	N/A	1.18	N/A		UX1700MSE-MD
9	10.67	2533	0.84	0.30	2943	3087	N/A	1.28	N/A		UX1700MSE-MD
10	12.67	2327	0.84	0.30	2423	2617	N/A	1.15	N/A		UX1600MSE-MD
11	14.67	2121	0.84	0.30	2423	2617	N/A	1.26	N/A		UX1600MSE-MD
12	16.67	1915	0.84	0.30	2423	2617	N/A	1.39	N/A		UX1600MSE-MD
13	18.67	1708	0.84	0.33	2109	2086	N/A	1.24	N/A		UX1500MSE-MD
14	20.67	1502	0.84	0.33	2109	2086	N/A	1.41	N/A		UX1500MSE-MD
15	22.67	1296	0.84	0.33	2109	2086	N/A	1.63	N/A		UX1500MSE-MD
16	24.67	1090	0.84	0.33	2109	2086	N/A	1.94	N/A		UX1500MSE-MD
17	26.67	750	0.84	0.33	1072	1028	N/A	1.36	N/A		UX1100MSE-MD
18	28.00	904	0.84	0.33	1072	1028	N/A	1.13	N/A		UX1100MSE-MD

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Appendix 3.1.2.1

MESA Installation Guide and Construction Manual

MESA® MSE Retaining Wall System

Back to main document

MESA®
RETAINING WALL SYSTEMS

INSTALLATION AND SPECIAL CONSIDERATIONS MANUAL



Tensar®

The Mesa® Retaining Wall Systems can easily accommodate a variety of design considerations.

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Endless aesthetic options with
The Connection You Can Count On™



TENSAR® GEOGRIDS

The **Mesa® Systems** owe their long-term performance and durability to high-strength **Tensar® Uniaxial (UX) Geogrids**. Due to their stiff interlocking capabilities, these geogrids stand the test of time, outperforming other commercially available geosynthetics.
For more information, visit **www.tensarcorp.com**.

Introduction

THE CONNECTION YOU CAN COUNT ON™

The Mesa® Retaining Wall Systems from Tensar International Corporation (Tensar) offer superior and cost-effective solutions for all of your retaining wall needs. Unlike other segmental retaining wall (SRW) systems, the Mesa Systems incorporate a positive, mechanical connection between the wall face and the Tensar® Geogrid reinforcement. This provides unsurpassed structural integrity that greatly reduces the chance of wall failure, even under the most severe conditions.

Designed as a truly integrated solution, Mesa Walls are also the only SRW system that comes from a single source of supply and demand. Every wall component has been specifically created to work together for optimum efficiency and performance. High strength, low absorption concrete units, corrosive resistant connectors and Tensar Geogrids all work together to create a mechanically stabilized earth (MSE) system that meets or exceeds every industry standard.

ENDLESS DESIGN OPTIONS

No matter which consideration is needed, the Mesa Systems have a solution. Curved walls blend with natural contours, inside and outside corners complement traditional angular looks of existing structures and steps and stairs provide functionality, allowing for limitless wall designs. From structural walls to tiered gardens, the Mesa Systems blend effortlessly with the natural surroundings of any site. Available throughout the United States, Latin America and Canada, the Mesa Systems are produced and sold locally through a large network of authorized distributors.

The following pages of this manual provide a general guideline to assist you in the construction of Mesa Retaining Wall Systems for a wide variety of special considerations. Whether you require stairs, 90-degree corners or convex and concave curves, the Mesa Systems can easily accommodate any project's design consideration.

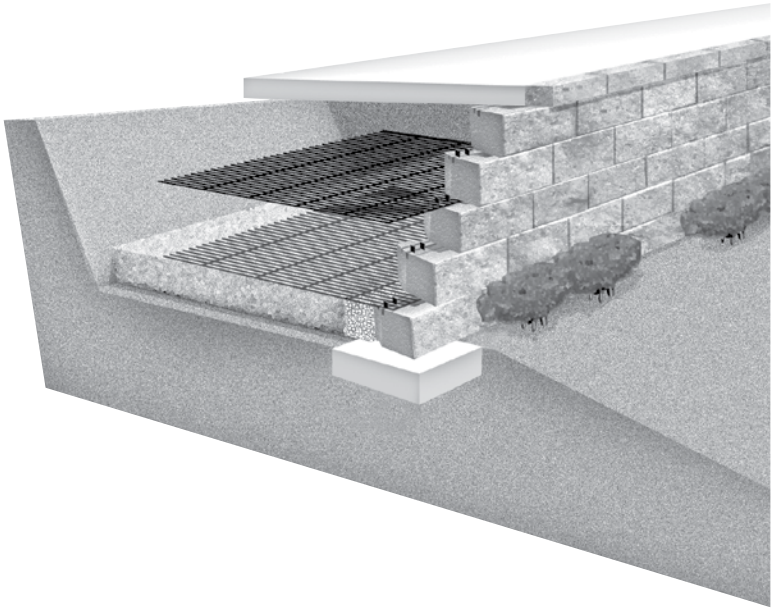
Mesa® Systems' Components	
Component	**Function**
Tensar Geogrids	High-density polyethylene (HDPE) structural geogrids internally reinforce fill materials. Inert to chemical degradation, they can be used with different backfills, including recycled concrete.
Mesa Segmental Units	High-strength concrete block with a compressive strength that exceeds the National Concrete Masonry Association (NCMA) standards (>4,000 psi) (27.6 mpa).
Mesa Connectors	Unique locking connectors are designed to mechanically connect the Tensar Geogrids to the Mesa Units. They provide a low-strain, end-bearing connection that is not dependent on friction for structural integrity and allow walls to be built near vertical or with a ⅝ in. (1.6 cm) setback.
Full Engineering and Construction Services	Detailing, design, site assistance and stamped drawings available for each Mesa project upon request.



Mesa® Units and Connectors

To further meet the aesthetic needs of any property, Mesa® Units are offered in a straight or radius face and are locally available in a wide variety of colors and textures. Custom colors may also be produced for higher-profile installations, and the innovative design of the unique Mesa Connector allows units to be installed with a near-vertical face or a ⅝ in. (1.6 cm) setback option – profiles not all SRW systems can provide.

To get in touch with your local authorized Mesa wall distributor, please call **800-TENSAR-1**.



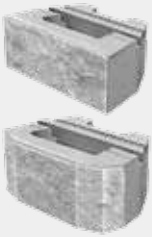
Mesa Standard and DOT Connectors



The Mesa Standard Connectors positively connect Tensar® Geogrid to the Mesa Facing Unit, often eliminating the need for core fill.



The Mesa DOT Connector engages the geogrid across the entire block width.



Standard Units
The most popular Mesa Unit in our full line of products. The Standard Unit can be used for most retaining wall needs.

8" h x 18" w x 11" d nom./75 lbs
(20 cm x 46 cm x 28 cm/34 kg)



Ashford™ Units
Create randomly patterned walls using the Mesa® Ashford™ System. Units are available in three sizes for a variety of facing options (available only with a straight split).

Standard Unit: 8" h x 18" w x 11" d nom./75 lbs (20 cm x 46 cm x 28 cm/34 kg)

Medium Unit: 8" h x 12" w x 11" d nom./60 lbs (20 cm x 30 cm x 28 cm/27 kg)

Tall Unit: 16" h x 6" w x 11" d nom./60 lbs (40 cm x 15 cm x 28 cm/27 kg)



Corner Units
Used to create walls with clean and precise 90° corners.

8" h x 18" w x 9" d nom./60 lbs (20 cm x 46 cm x 23 cm/27 kg)



Cap Unit
Used at the top of the wall for a finished look.

4" h x 18" w x 12" d nom./40 lbs (10 cm x 46 cm x 28 cm/18 kg)

NOTE: *Actual dimensions and weights may vary slightly depending on location.*



FIGURE 1: *Prepare for wall construction by clearing the site and cutting and color-coding the Tensar® Geogrid.*

Construction and Quality Control

1.1 CONSTRUCTION RESPONSIBILITIES

This installation section of the manual provides general guidelines for construction and quality control of the installation. This section should be provided to the owner's engineer, the construction quality assurance inspector and the contractor. Specific construction details and procedures depend on individual site conditions and other considerations, which are the responsibility of the owner and engineer.

NOTE: *All installation instructions apply to the Mesa® Standard Unit except where otherwise stated.*

The contractor must provide construction in accordance with the contract documents, plans and specifications. The contractor is also responsible for the verification of line, grade and other physical features as well as surface water drainage control.

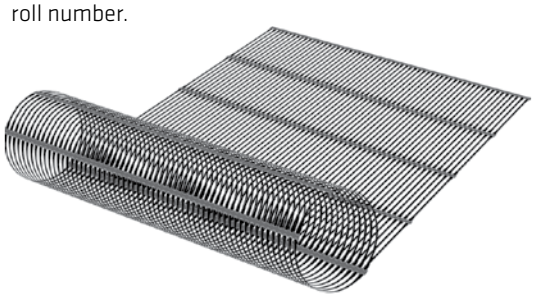
1.2 MATERIALS AND HANDLING

Materials Supplied

- Mesa Units and Connectors
- Mesa Cap Units (where required)
- Tensar® Uniaxial (UX) and/or Biaxial (BX) Geogrids
- Drainage composite and piping (where required)
- Geotextile filter materials (where required)

Handling Wall Materials

- The concrete facing units are delivered on pallets and off-loaded by the contractor. Transporting equipment must have firm ground and a stable, level area to off-load. A forklift is normally used to handle pallets. If pallets are the property of the block manufacturer, they must be stored by the contractor for pick up. The contractor must provide protection from staining or discoloration of the units by using wood dunnage and polyethylene sheet film, or similar.
- The Mesa Connectors are shipped in cartons and should be stored in a secure and dry location.
- Tensar Geogrids are shipped in roll form; the contractor is responsible for off-loading.
- Geogrid rolls should be stored in a secure area. Each roll will be labeled by type, lot number and roll number.



- Standard Tensar UX Geogrid roll sizes are 4.36 ft wide x 200 or 250 ft long (1.33 m x 61 m or 76 m). Rolls weigh between 87 and 170 lbs (40 and 77 kg) depending on type. Before labels have been removed, rolls should be color-coded with spray paint to help identify geogrid type (Figure 1).

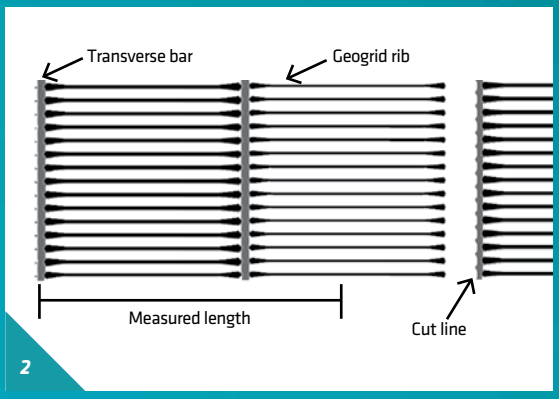




FIGURE 2: *Cut Tensar Geogrid at nearest transverse bar beyond the measured length.*

FIGURE 3: *The base course of Mesa® Units.*

- It is the contractor's responsibility to verify the quantities shipped and condition of the materials. The contractor will inventory materials supplied to assure sufficient quantities have been delivered.
- The contractor will be allowed a limited amount of time to off-load materials.
- If certifications are required, the contractor must provide a written request prior to shipment of the material. The contractor will ensure that all information, including product type, roll/lot number, etc. is furnished to the engineer.

Contractor Supplied Materials

- Dead blow hammer
- 2- or 4-ft (.61 or 1.22 m) levels
- Utility saw and/or grinder
- Masonry string and chalk line
- All cast-in-place concrete and structural components
- Stone filter medium
- Reinforced or select fill
- Pitchfork, stakes and/or rods used to remove slack from geogrid
- All labor, equipment and supervision necessary to perform the total Mesa® Wall construction

1.3 WALL CONSTRUCTION PREPARATORY WORK

- Verify approval of Mesa Units, Tensar® Geogrids and the specified reinforced fill.
- Review drawings to plan geogrid layout. Review drawings and site plans to consider surface water drainage control both during and after construction.
- Prepare subgrade by excavating vertically to plan elevation and horizontally to design geogrid lengths. If a rock face is shown, it is the responsibility of the Engineer to determine the competency of the rock at the limits of excavation shown on those plans.

NOTE: *Any deviation in the location of the rock face with respect to the face of the retaining wall may require an adjustment to the Tensar Geogrid design and the Engineer of Record must be notified by the contractor prior to proceeding with the wall construction.*

- The subgrade shall be approved before proceeding with wall construction. Any foundation soils found unsuitable by the engineer shall be treated in a manner approved by the engineer.
- To speed up construction on larger projects, we suggest cutting the Tensar Geogrids in advance. Cut geogrid at the nearest transverse bar beyond the measured length (illustrated in Figure 2) or several inches in front of the transverse bar to provide "finger shims" (Figure 8 on page 9).

LEVELING PAD CONSTRUCTION

- The leveling pad must be flat and level to assure the first course of Mesa Units will provide uniform support to the courses above it. Non-uniform support will induce tensile stresses and shear stresses in the Mesa Units above the first course that can result in cracking.
- The leveling pad (Figure 3) may be constructed with unreinforced concrete or compacted ¾ in. (1.9 cm) minus, well-graded aggregate. It is typically 12 in. (30 cm) wider than the Mesa Units, providing 6 in. (15 cm) in front of and behind the Mesa Unit allowing for wall curvature and minor alignment adjustments. It is generally 6 in. (15 cm) deep. For unreinforced concrete leveling pads, steel or wood forms are generally required to assure that the top of the leveling pad is flat and level. For aggregate leveling

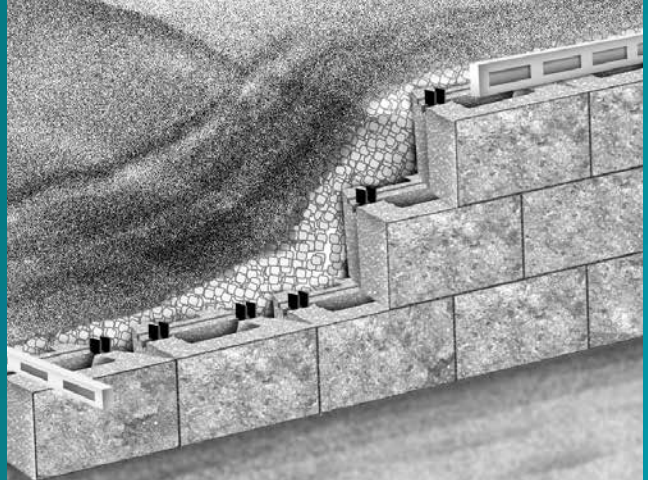


FIGURE 5: *As units are placed, it is important that the units remain level from front-to-back and side-to-side. This level should be checked as each course is placed. If the units become more than 1/8 in. out of level from front-to-back, block-to-block or side-to-side, they should be brought to level by grinding and/or shimming.*

pads, the aggregate is generally overfilled, compacted and then carefully trimmed down to near plan elevation. The Mesa® Units are then seated into the aggregate with a rubber mallet to the plan elevation and then leveled front-to-back and side-to-side.

- Steps in the leveling pad are required to change elevation. It is important that the height of the step is equal to the height of the number of Mesa Unit courses. In practice, the steps may be slightly more or less due to the thickness of the Tensar® Geogrid reinforcement connected between courses and to the normal dimensional tolerances of the units. Where a concrete step is not at the same elevation of the adjacent courses, shimming or grinding will be required to match the elevation and provide a flat and level bearing surface for the next course of units. As shown in Figure 4, such grinding or shimming will be required over some distance (5 ft [1.52 m] min.) from the step to assure that the top surface of the entire course of Mesa Units placed on the stepped section of the leveling pad is flat and level. Ribs or transverse bars of the Tensar Geogrid may be used as shims if their thickness is correct.
- If contract documents indicate the wall has a battered face, the contractor shall ensure that the ⅝ in. (1.6 cm) setback is accounted for at each leveling pad step. We recommend that the elevation of the leveling pad does not exceed four courses.
- The top of the Mesa Units should be adjusted as required for leveling. Shimming and/or grinding may be required. (For instructions, see the "Shimming Mesa Units" section on page 9).

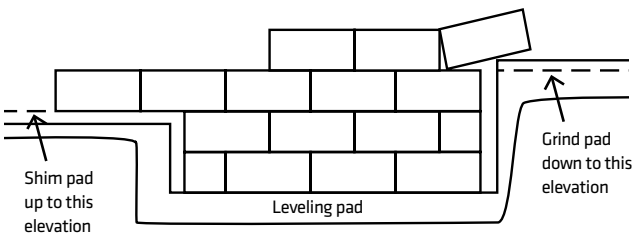


FIGURE 4

1.4 WALL CONSTRUCTION

- Wall line shall be established using a chalk or string line. Chalk line should be placed on the concrete pad along the tails of the Mesa Units. A string line can be used in lieu of a chalk line and will be necessary where the leveling pad consists of aggregate. (Alignment based on the split-faced Mesa Units may render an uneven wall face.)
- Once the leveling pad is complete (Figure 5), the first course can be installed. Place each Mesa Unit so that their sides touch and their textured face is outward. The first course must be accurately placed, carefully spaced and leveled to facilitate construction and enhance the appearance of the entire wall.
- Prior to the installation of the second course and each successive course, the tops of the units on which the course is to be placed must be swept clean. Failure to do this will result in problems with seating and leveling the Mesa Units and may also increase the likelihood of cracks developing in the units due to load concentrations as additional courses are set.
- The Mesa Units are stacked in a running bond pattern, similar to standard masonry wall construction. A limit for the shift from a perfect running bond is needed. On straight wall sections, it should be a ½ in. (1.28 cm) to ensure all standard connector teeth are squarely in the slots. Once placed, advance the units forward toward the face of the wall until they make contact with the connectors. The connectors do allow units to slide from side to side, so check vertical joint alignment frequently (max. 20 units) to ensure that a running bond pattern is maintained within the limits stated above.
- Drainage fill is placed to the limits shown on the drawings. Drainage and/or core fill may not be required for structural walls (see "Definitions" section under "Drainage Fill" and "Core Fill"). Proper installation of drainage materials is critical to overall wall performance. Drainage materials must be installed properly and protected during construction.



FIGURE 8: *Tensar® UX Geogrid and Mesa® Connector placement on Mesa Units.*

SHIMMING MESA® UNITS

- It is important that the courses of Mesa® Units are level front-to-back and side-to-side. To achieve this in the front-to-back plane, it may be necessary to grind the units or use shims between some of the courses to correct for:
  1. The thickness of the Tensar® Geogrid reinforcement ribs that cross the tails of the units or
  2. Slight differences in the height at the front and back of the units.
- For courses placed on a geogrid elevation, shims may be required on the unit's front face shell (Figure 6). The shims should be the same thickness of the geogrid rib. The material can be trimmed from the same roll of grid that is used in that particular course.

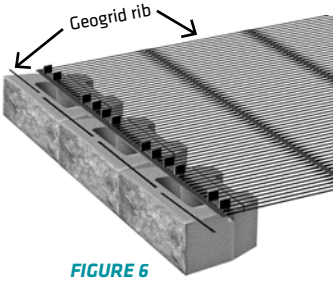


FIGURE 6

- Another alternative for shimming is to cut the geogrid so that the ribs extend approximately 1 in. (2.5 cm) onto the unit's front face shell (Figure 7).

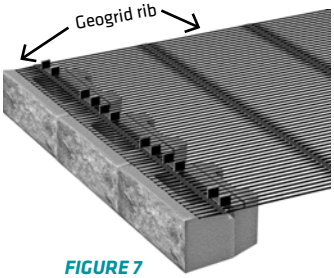
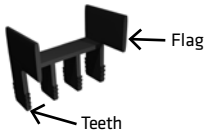


FIGURE 7

- To correct for slight differences in unit heights, a geogrid rib may be cut and placed on successive courses to bring the face and/or tail back to level. The "rib shim" and "finger shim" should be positioned on the inside of the unit's front face shell, generally within ½ in. (1.27 cm) of the unit's core (see shimming Figures 6 and 7). This will place less stress on the underlying block than would exist if the shim was placed near the unit's front face.
- Tensar Geogrids may be draped over the upper courses of the wall until the reinforced fill reaches the level of the geogrids. To ensure that the Mesa Wall's geometry is being maintained, a string line should be pulled after each course is set. The string can be referenced from the connector slot or tail of the Mesa Unit.

NOTE: *To ensure proper installation, the geogrid transverse bar must be pulled to contact with the teeth of the Mesa Connector.*



1.5 GEOGRID AND CONNECTOR PLACEMENT

The following section covers the installation for both Tensar Geogrids and Mesa Connectors (Figure 8) based on the use of both the Mesa Standard and Mesa XL Units only.

- At the end of each day, the Contractor must ensure the reinforced backfill is graded to drain water away from the face of the wall. Berms and/or ditches must also be in place and functioning to prevent the entrance of runoff into the wall construction site.

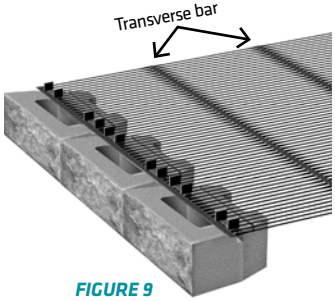


FIGURE 9

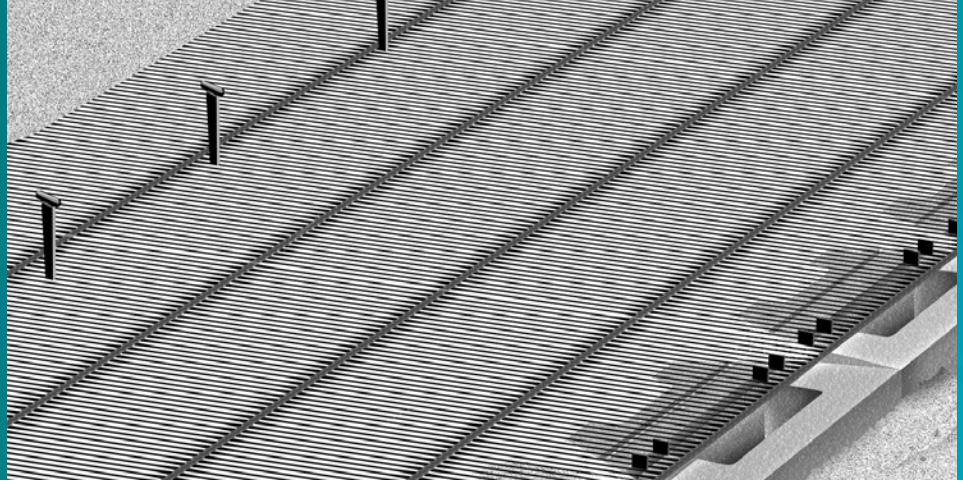


FIGURE 10: *Pinning Tensar® Geogrids.*

- At the location and elevation shown on the plans, install the appropriate Tensar® UX Geogrid (type and length specified) ensuring the transverse bar of the grid is in the right location (Figure 9 on page 9). A connector should be placed in each connector slot of the Mesa Units. On courses with geogrid, all four connector's teeth must penetrate through the grid apertures and into both connector slots. To align the connectors over the slots, the transverse bar may be cut midway between the connectors. The transverse bar should be snug against the connector teeth before final seating of the connector.
- Connector flags must be oriented in the proper direction for a "battered" or "near-vertical" wall as per the contract documents. Connector placement with flags forward (toward the wall face) will render a near-vertical wall. Connector placement with the flags pointed away from the wall face will provide an automatic ⅝ in. (1.6 cm) setback.
- The reinforced fill must meet specification, be compacted to the required density and be graded reasonably smooth. On courses that require geogrid, the reinforced fill must be at the top of the course's same elevation for a distance of 3 ft (.91 m) before the fill can be gradually sloped.
- Remove slack from geogrid by pulling it taut and then anchoring it with stakes and/or rods (Figure 10). The stakes used to position the geogrid tail can be withdrawn once the fill is in place and then reused on subsequent lifts. A pitchfork can also help remove slack by inserting it at the tail of the geogrid and pulling until the fill has been placed on the grid. Removal of any geogrid slack will help prevent wall face movement caused by translation of the slack toward the wall face.
- Fill placement shall be performed to minimize geogrid slack development. Fill should meet specification and be spread away from or parallel to the wall face. By doing this, any slack that may develop will be pushed toward the free (unconnected) end of the geogrid. Unless the contract documents are stringent, the loose lifts of the reinforced fill must not exceed 6 in. (15 cm) where hand compaction equipment is used or 10 in. (25 cm) where heavy compaction equipment is used. Thicknesses may vary depending on the approved project's specific soil types. The fill shall be compacted to 95% of ASTM D-698 or as required by the contract documents. Only hand-operated compaction equipment shall be used within 3 ft (.91 m) of the tails of the units.
- Heavy equipment should not be used within 3 ft (.91 m) of the back of the facing units, and tracked construction equipment should not be operated directly on Tensar Geogrids. Rubber tired equipment may be operated on the geogrid providing the subgrade is not pumping or rutting. Turning of tired equipment should be minimized to prevent dislocation or damage to the geogrid or facing units. Equipment must travel slowly and with sufficient care to avoid dislocating the geogrid or facing units.
- Install the Cap Units, if required, by attaching them to the Mesa Units below using an approved exterior concrete adhesive. Cap Units may be placed with a nominal 1 in. (2.54 cm) overhang or flush with the wall face.
- Wall penetrations may be accommodated by cutting the Mesa Units to fit with a utility saw and a mason's hammer. The small voids, less than 1 in., (2.54 cm) can then be closed with a cement and sand mix or other methods that have been approved by the engineer.

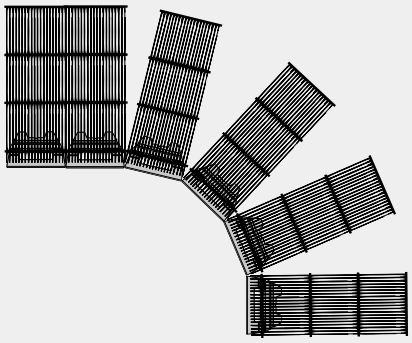
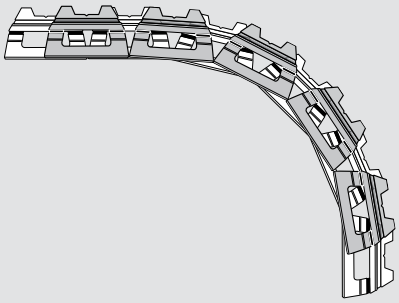
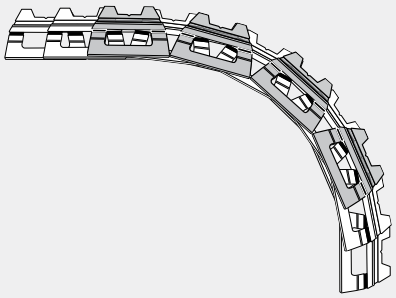


Mesa® Curves and Corners

2.1 CONCAVE CURVES

When possible begin a Mesa® Wall with a concave curve from the center of the curve, alternating left and right of the center unit. When building with a 5/8 in. (1.6 cm) setback, each Mesa Unit falls behind on a concave curve relative to any units below.

NOTE: *Refer to the "Construction and Quality Control" section of this manual for proper installation procedures of connectors, units and geogrids for all courses.*

1st Course	2nd Course	3rd Course
		
Follow standard installation instructions for preparing the subgrade and leveling pad. When placing the first course,* it is suggested that a flex pipe be placed on the unit's tail to ensure a smooth curve. If using a 5/8 in. (1.6 cm) setback, overlap corners of units on the base course. The overlap amount will vary based on the curve size. The radius becomes larger as the wall becomes taller, therefore gapping will occur (max. gap size 1/2 in. [1.27 cm]). If the maximum gap is exceeded, one flag may be removed from each connector to close the gap.	Follow standard installation instructions for backfilling and course placement. On the second course, set the first unit on halfbond to the base course in the center of the concave curve where possible. Work in both directions to minimize movement off bond.	Follow standard installation procedures for backfilling and course placement. On the third course, set the first unit on halfbond to the second course in the center. Work in both directions to minimize movement off bond. Ensure that all four teeth of the connector are positioned in the connector slot. At the ends of the curve, the running bond must be re-established to develop full connection capacity. This may require cutting or grinding of the facing units.

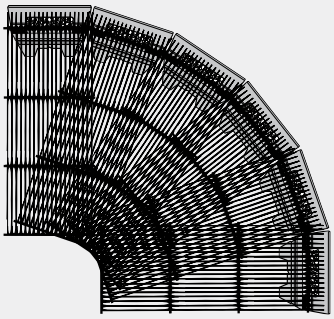
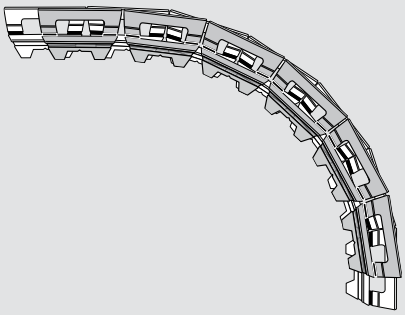
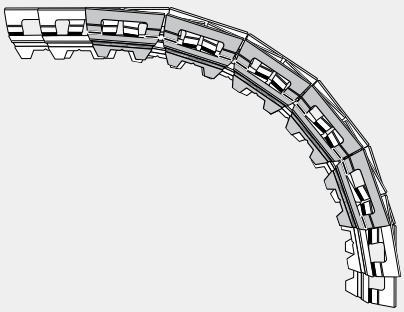
NOTE: *Course 1 – On tight curves, grid may be cut lengthwise to the width of the Mesa Units to ensure the transverse bar engages both connectors.*



2.2 CONVEX CURVES

As with concave walls, begin a wall with a convex curve from the center of the curve, alternating left and right of the center unit. When building with a 5/8 in. (1.6 cm) setback, each unit reduces in radius on a convex curve relative to the unit below. Conversely to concave, a convex curve's radius gets smaller with each additional course.

NOTE: *Refer to the "Construction and Quality Control" section of this manual for proper installation procedures of connectors, units and geogrids for all courses. Minimum recommended radius at top of wall is 55 in. (140 cm) for convex curves.*

1st Course	2nd Course	3rd Course
		
Follow standard installation instructions for preparing the subgrade and leveling pad. When placing the first course,* it is suggested that a flex pipe be placed on unit's tail to ensure a smooth curve. If using a 5/8 in. (1.6 cm) setback, gap the units on the base course no more than 1/2 in. (1.27 cm). The radius becomes smaller as the wall becomes taller, therefore binding will occur. If binding on any course begins to occur, trim the sides of the units with a concrete saw to realign.	Follow standard installation instructions for backfilling and course placement. On the second course, set the first unit on halfbond to the base course in the center of the convex curve where possible. Work in both directions to minimize movement off bond.	Follow standard installation procedures for backfilling and course placement for the third course. On the third course, set the first unit on halfbond to the second course in the center. Work in both directions to minimize movement off bond. Ensure that all four teeth of the connector are positioned in the connector slot. At the ends of the curve, the running bond must be re-established to develop full connection capacity. This may require cutting or grinding of the facing units.

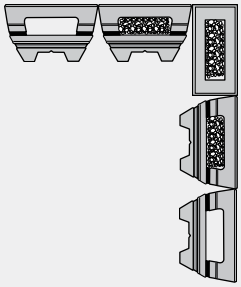
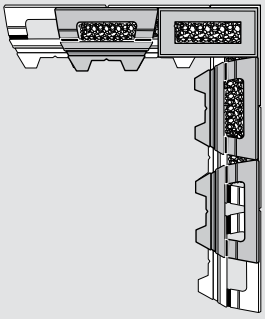
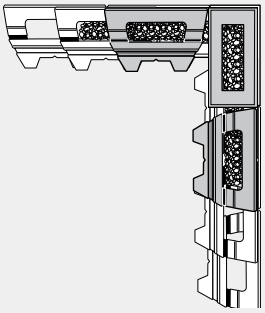
**__NOTE:__ Course 1 – On tight curves, grid may be cut lengthwise to the width of the Mesa® Units to ensure the transverse bar engages both connectors. Provide 3 in. (7.6 cm) (min.) soil cover between overlapping layers of Tensar® Geogrid reinforcement.*



2.3 90-DEGREE OUTSIDE CORNERS

Corner Units simplify installation and provide a sharply defined and visually appealing component to any Mesa® Wall. Installation is simple, quick and requires no special equipment or specialized personnel.

NOTE: *Refer to the "Construction and Quality Control" section of this manual for proper installation procedures of connectors, units and geogrids for all courses.*

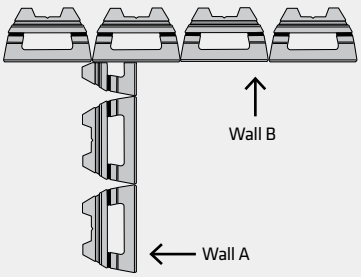
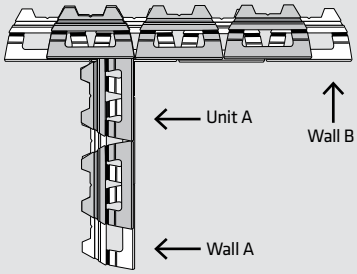
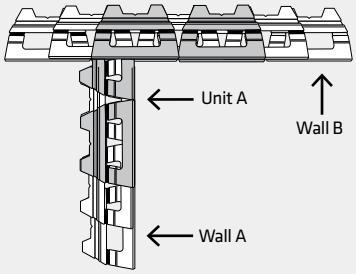
1st Course	2nd Course	3rd Course
		
Follow standard installation instructions for preparing the subgrade and leveling pad. Working with the Corner Unit, place units tightly against each other on the first course. Granular fill must be placed in the Mesa Corner Unit as well as in each Mesa Unit to either side of the Corner Unit.	Follow standard installation instructions for backfilling and course placement. When building a Mesa Wall with a 5/8 in. (1.6 cm) setback, the shorter 9 in. (23 cm) side of the Corner Unit should be field cut to account for the setback and to maintain a running bond. On the second course, alternate the direction of the Corner Unit and set units on halfbond to the base units. Secure the Corner Unit to the unit below using an approved concrete exterior adhesive. Granular fill shall be placed in the corner unit as well as in each Mesa Unit to either side of the Corner Unit.	Follow standard installation procedures for backfilling and course placement. When building a Mesa Wall with a 5/8 in. (1.6 cm) setback, the shorter 9 in. (23 cm) side of the Corner Unit should be field cut to account for the setback and to maintain a running bond. On the third course, alternate the direction of the Corner Unit and set units on halfbond to the second course. Secure the Corner Unit to the unit below using an approved concrete exterior adhesive. Granular fill must be placed in the corner unit as well as in each Mesa Unit to either side of the Corner Unit.



2.4 90-DEGREE INSIDE CORNERS

90-degree inside corners require some preparation and planning for results that are sharply defined. Once the initial sizing is determined, installation is quick and simple. For the most visually appealing walls, the field cuts illustrated below should be applied to the shorter (Wall A) of the two walls.

NOTE: *Refer to the "Construction and Quality Control" section of this manual for proper installation procedures of connectors, units and geogrids for all courses.*

1st Course	2nd Course	3rd Course
		
Follow standard installation instructions for preparing the subgrade and leveling pad. First course construction can begin at the corner or at a point beyond the corner. At the inside corner, begin with a Mesa® Unit cut to approximately 1/3 of the original face length. From the Corner Unit, position the Mesa Units tightly against each other.	Follow standard installation procedures for backfilling and course placement. Refer to the above illustration for proper unit placement on the second course. When building this course, begin with Wall B, and construct Wall B past the extent of Wall A. Measure and field cut Unit A to provide a symmetric running bond over the first course.	Follow standard installation procedures for backfilling and course placement. Refer to the above illustration for proper unit placement on the third course. When building this course, begin with Wall B, and construct Wall B past the extent of Wall A. Measure and field cut Unit A, such that the third course units are centered over the units in the first course.



Mesa® Steps and Stairs

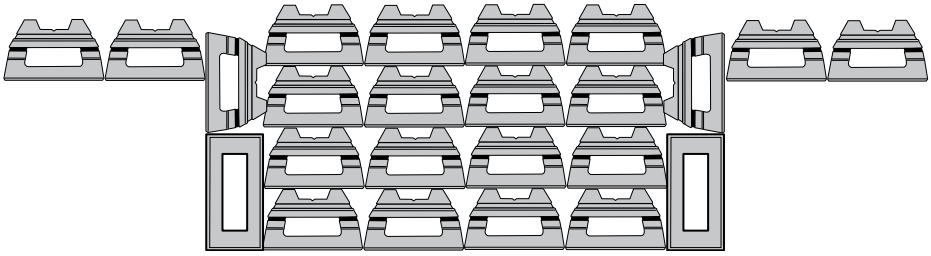
3.1 STEPS IN FRONT OF WALL

When installing steps, simply combine Mesa® Standard, Corner and Cap Units to realize virtually any wall design.

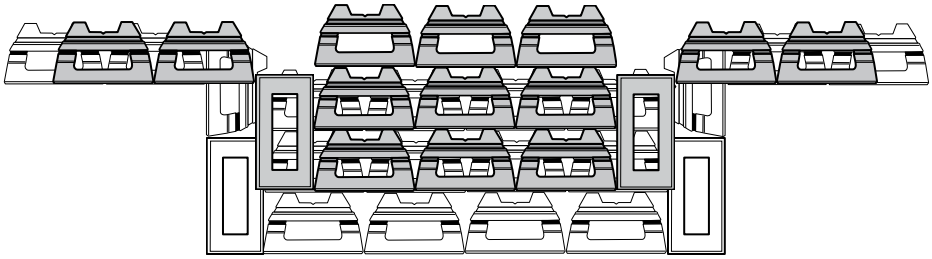
Follow the installation techniques previously stated in this manual. Different riser heights can be achieved by varying the location of the base Standard Unit (see detail on page 16). Mesa Standard Units used for step construction should be filled with granular fill or concrete.

NOTE: *Refer to the "Construction and Quality Control" section of this manual for proper installation procedures of connectors, units and geogrids for all courses.*

1st Course



2nd Course

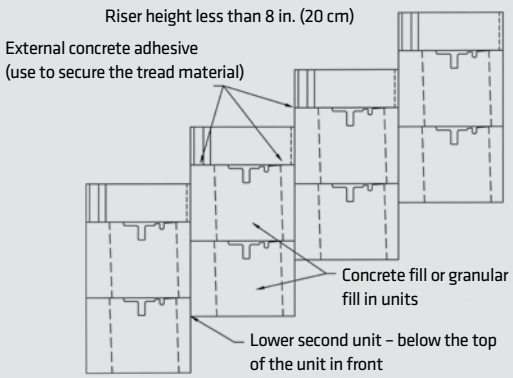


NOTE: *Various tread material can be used to cap steps and stairs including, but not limited to: Mesa Cap Units, pavers, natural stone and concrete. When required by the engineer, an approved exterior concrete adhesive should be used. Please consult your local Mesa Representative for more information.*



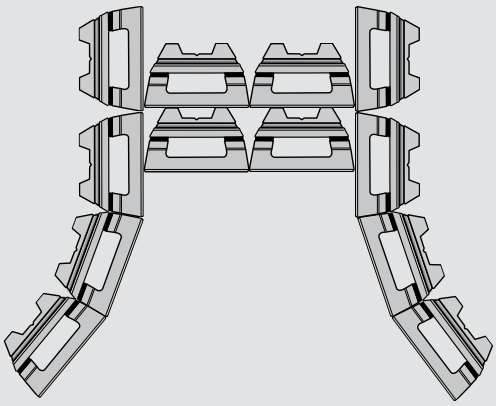
3.2 STEPS IN WALL

Riser Height Detail

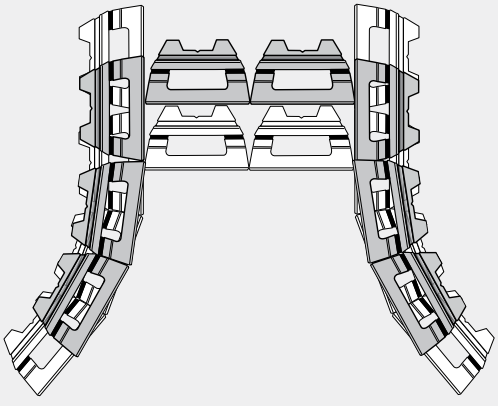


NOTE: *Refer to the "Construction and Quality Control" section of this manual for proper installation procedures of connectors, units and geogrids for all courses.*

1st Course



2nd Course





Tiered and Top of Wall Finishes

4.1 TIERED WALLS

Tiered Mesa® Walls offer a visually pleasing and less obtrusive alternative to conventional wall construction. On sites that provide sufficient land area for this application, these walls are typically designed with greenspace between the tiers. For tiered walls to be designed separately, the distance between the lower Wall A (at its top) and the upper Wall B (at its base) must be at least twice the height of the lower Wall A (Figure 11). If this condition does not exist, the lower Wall A must be designed to account for the additional loading of the upper Wall B.

NOTE: *Refer to the "Construction and Quality Control" section of this manual for proper installation procedures of connectors, units and geogrids for all courses.*

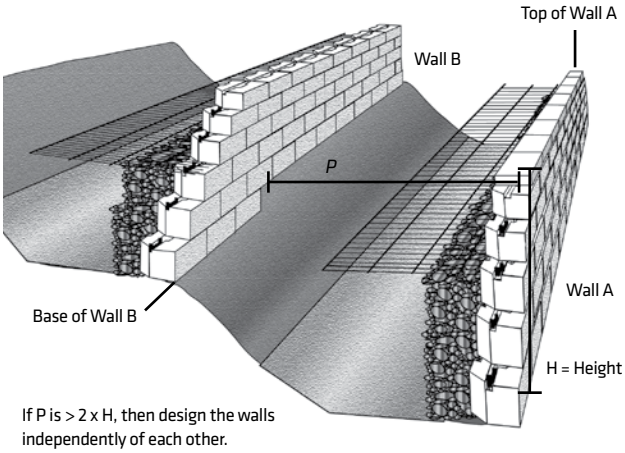


FIGURE 11

4.2 TRAFFIC BARRIERS

Mesa Retaining Walls can be easily capped with reinforced concrete traffic barriers. Following the wall installation, set and secure forming materials along the top course of the Mesa Wall according to standard procedures. When the design calls for an overhanging barrier, form the front edge by attaching an "L" shape forming material to the face of the Mesa Wall. Construct the barrier as designed, including, but not limited to, the use of the control joints (Figure 12).

NOTE: *Refer to the "Construction and Quality Control" section of this manual for proper installation procedures of connectors, units and geogrids for all courses.*

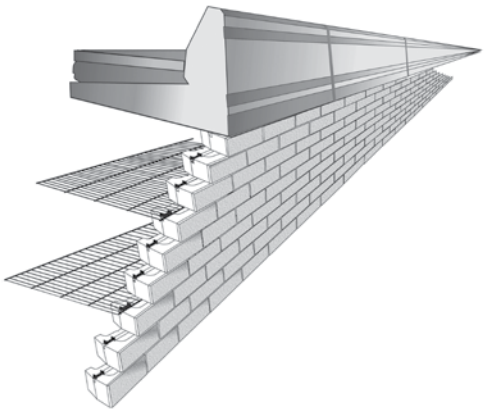


FIGURE 12



4.3 GUIDE RAILS

Guide rails can be easily placed along the top of a Mesa Wall according to the standard installation procedures. Guide rail posts may be placed by cutting holes in the Tensar Geogrid reinforcement, or the post may be driven through the geogrid. The guide rail post location, design and impact on the Mesa Retaining Wall shall be taken into account by the design engineer. Guide rails can also be placed into sonotubes positioned during wall construction and then filled with concrete. Tensar Geogrids can be cut to accommodate the sonotubes.

4.4 HANDRAILS AND FENCE POSTS

Similar to guide rail installation, handrails and fence posts can be placed behind the Mesa Units as described in section 4.3. Placement and spacing of the handrail or fence will be determined by the project specific loading and geometry of the structure. The typical detail provided on page 21 addresses handrail installation for a 50 pound (23 kg) per lineal foot horizontal force applied at 42 in. (107 cm) above the finished grade behind the Mesa Units. For loading conditions other than this, the wall designer should perform additional calculations to confirm the stability of the affected portion of the wall. For additional information please contact Tensar Engineering at **800-TENSAR-1**.

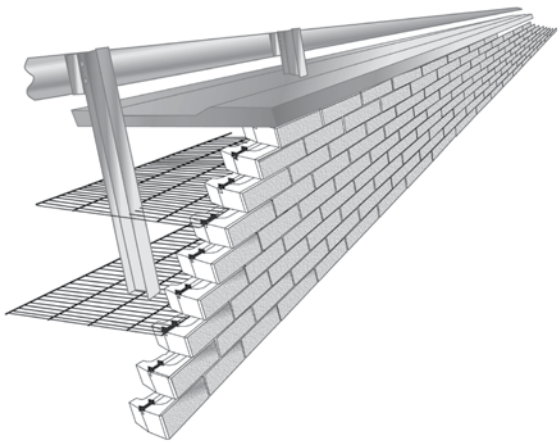


FIGURE 13

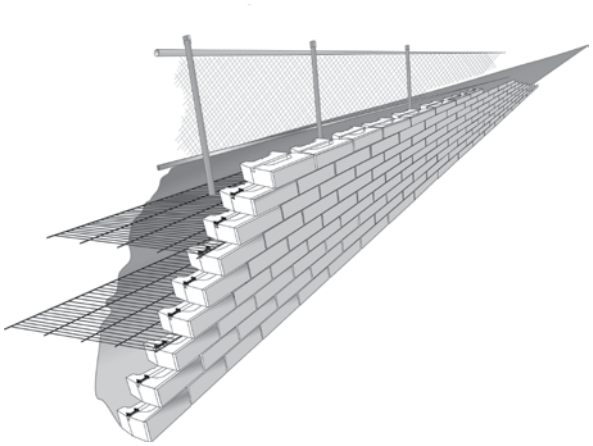


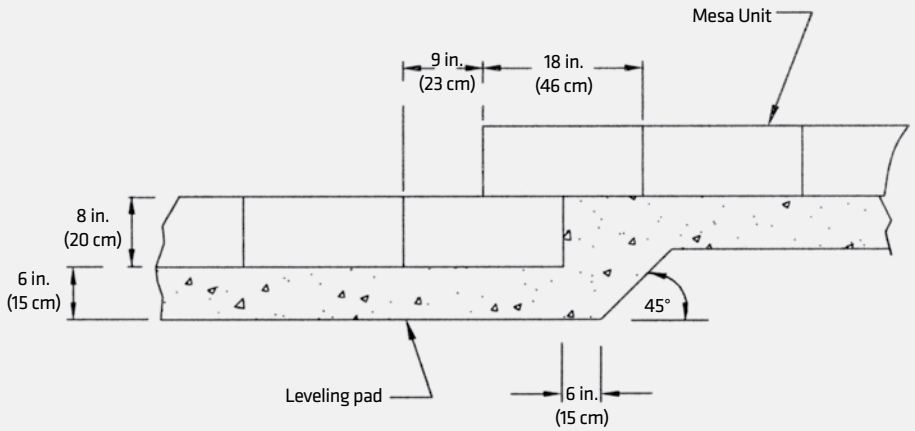
FIGURE 14



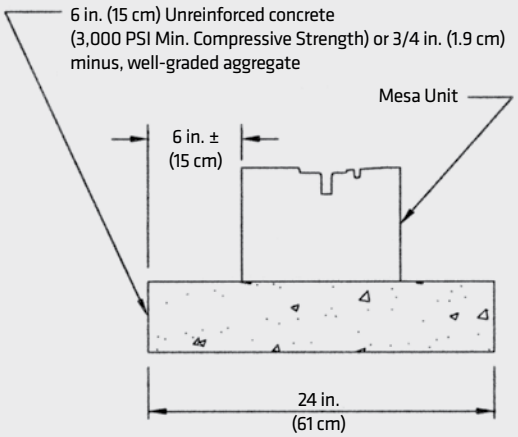
Mesa® Wall Details

The following section contains the standard details for designing generic Mesa Walls with special considerations. Please refer to the "Construction and Quality Control" section of this manual for proper installation procedures of connectors, units and geogrid. Specific details for specific walls are the responsibility of the owner and the engineer.

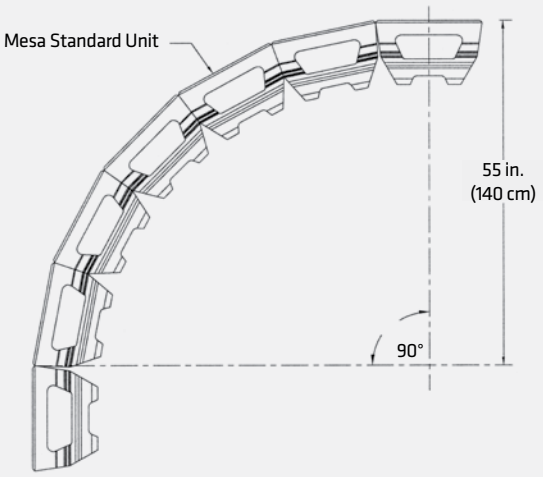
Typical Leveling Pad Step Detail



Typical Leveling Pad Detail

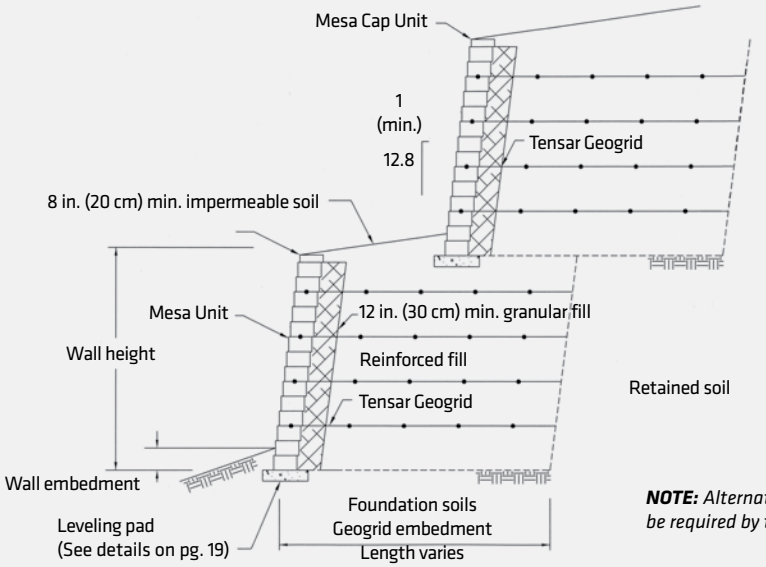


Typical 90-Degree Curve Detail



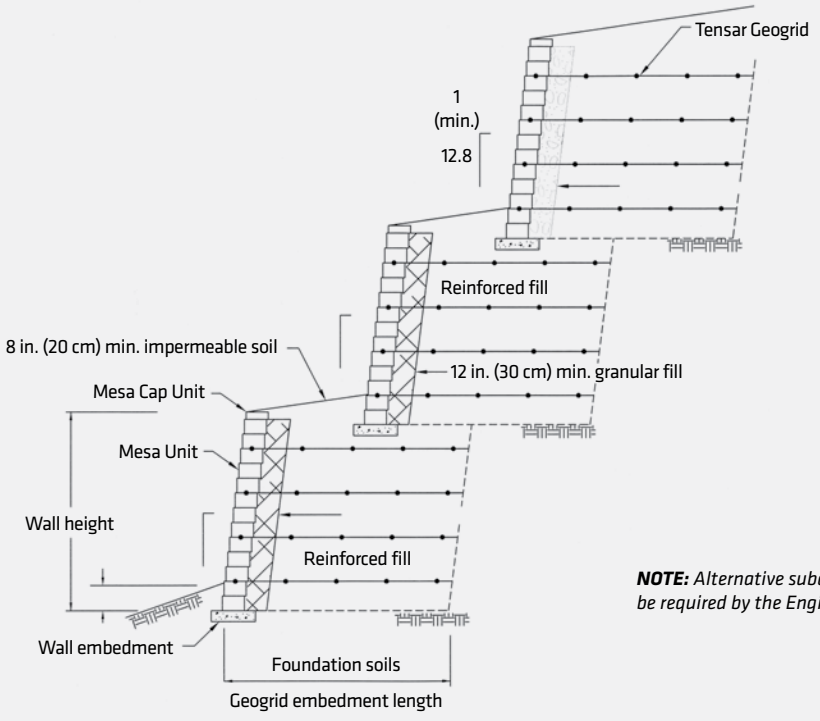


Typical Tiered Detail – 2 Tiers



NOTE: *Alternative subdrain system may be required by the Engineer of Record.*

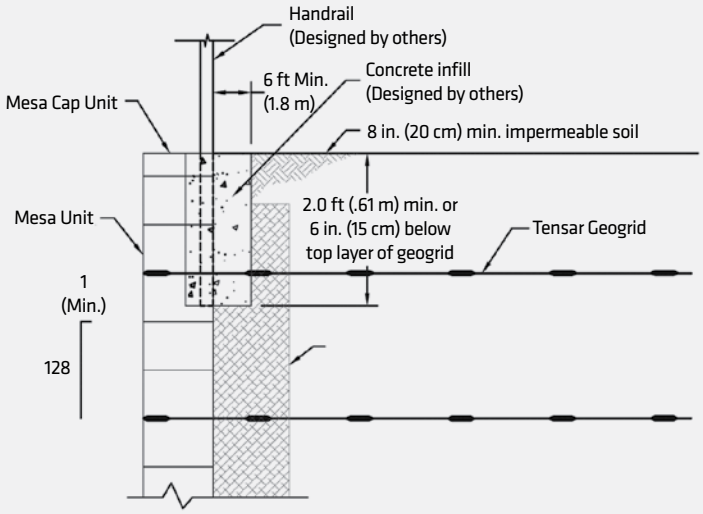
Typical Tiered Detail – 3 Tiers



NOTE: *Alternative subdrain system may be required by the Engineer of Record.*



Handrail on Top of Wall

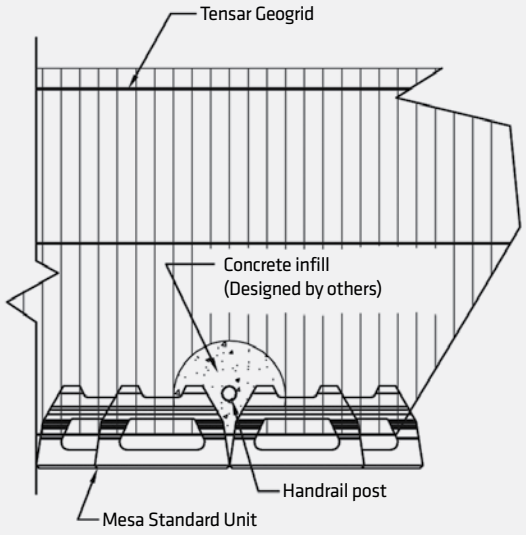


Step 1: *Place granular fill to bottom of handrail.*

Step 2: *Place top layer of Tensar® Geogrid and remaining Mesa® Units above it.*

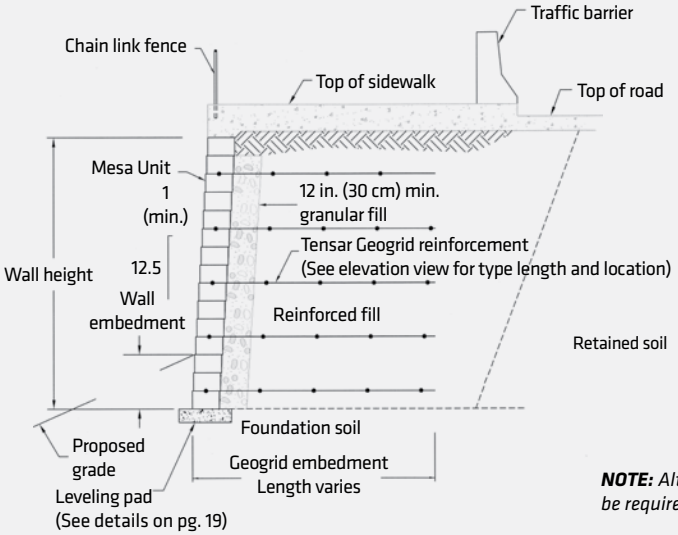
Step 3: *Cut Tensar Geogrid and then set rail post. Form and pour concrete infill at tail of the Mesa Units.*

Plan View – Detail on Handrail on Top of Wall



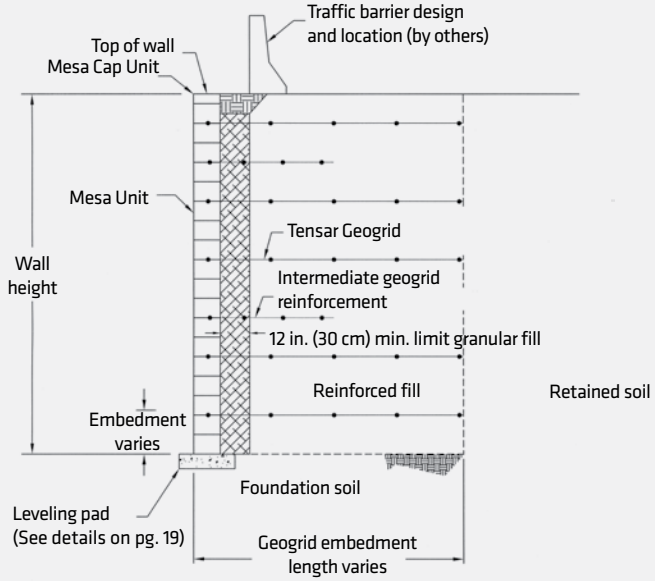


Typical Detail with Chain Link Fence



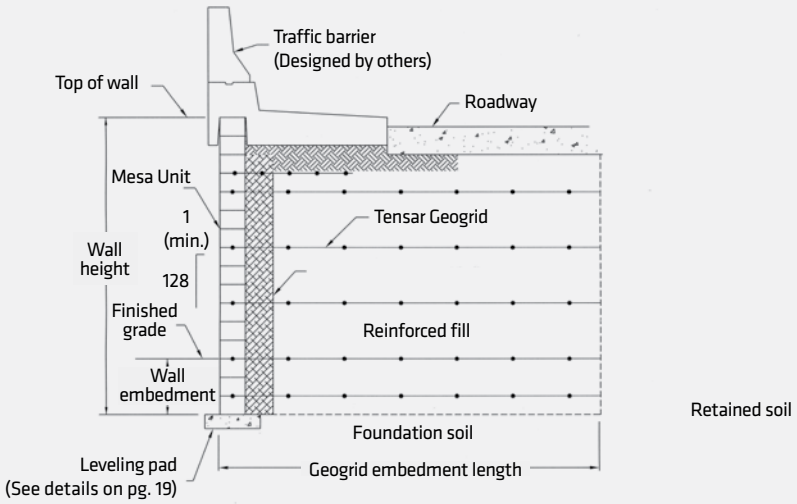
NOTE: *Alternative subdrain system may be required by the Engineer of Record.*

Typical Detail of Traffic Barrier Behind Wall

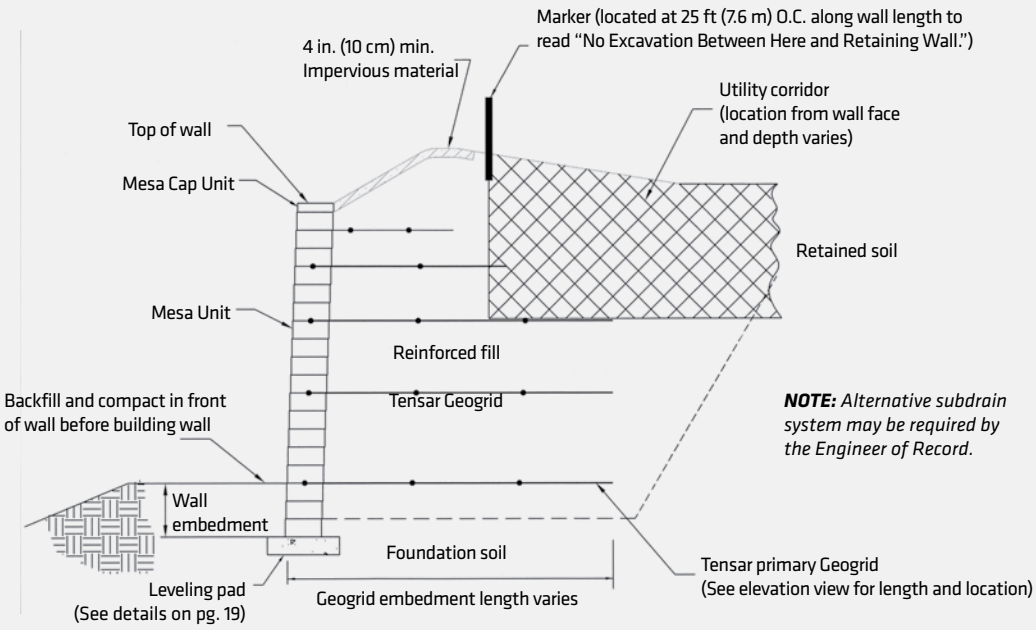




Typical Detail of Traffic Barrier on Top of Wall



Typical Detail with Utility Corridor Behind Wall





General Terms

Approved Construction Drawings
The final wall drawings provided by Tensar to the Contractor for submittal to the Owner/Owner's Engineer and subsequently approved by the Owner/Owner's Engineer for construction.

Contract Documents
The Agreement between the Owner and the Contractor, including conditions of the contract drawings, specifications and the provisions of the Agreement between the Contractor and the supplier of the Mesa® Systems. These documents should also include addenda and other modifications issued prior to the execution of the Contract.

Contractor
The individual, firm or corporation acting directly through its agents or employees to undertake the execution of the work under terms of the contract.

Core Fill
Free-draining, coarse-grained soil that is placed within the empty cores of the segmental concrete facing units. Core fill may not be required within the Mesa Unit if the Contractor can provide the Engineer and/or Architect with connection testing performed without core fill verifying that the connection strength of the system exceeds the requirements set forth in the design data.

Engineer
The Owner's representative with authoritative charge over the inspection and acceptance of the wall construction in accordance with the contract documents.

Granular Fill
Free-draining, coarse-grained soil which is placed behind the Mesa Segmental Concrete Facing Units and in the openings between the Mesa Units as specified on the plans. The Engineer and/or Architect may specify a nonwoven geotextile which meets AASHTO M288-96 subsurface drainage requirements in lieu of drainage fill.

Inspector
The owner's authorized representative assigned to see that the workmanship and materials are in accordance with the terms of the contract.

Mesa Connector
A proprietary mechanical connection device made of high density polymer with fiberglass inclusions to positively connect the Tensar® Geogrid to the Mesa Units.

Owner
The Owner of the project with whom a contract has been made for payment for the work performed under the terms of the contract.

Plans
The part of the contract documents consisting of the approved plans, profiles, typical cross sections, working drawings and supplemental drawings, or exact reproduction thereof, which shows the location, character, dimensions and details of the work to be performed.

Reinforced Fill
The specified soil material that interacts with the geogrid reinforcement to create a flexible gravity mass. Its limits extend from the back of the facing element or granular medium to the tails of the soil reinforcement or as indicated in the plans.

Setback (Batter)
The rearward offset from the vertical plane between two adjacent block courses created by the orientation of the flags on the Mesa Connectors.

Specifications
A description of the quality and quantity of the materials and workmanship that will be required of the Contractor in the execution of the work under the contract between the owner and the Contractor.

Tensar Geogrids
A proprietary polymeric grid formed by a regular network of integrally connected tensile elements with apertures of sufficient size to allow interlocking with surrounding soil, rock or earth and connection to Mesa Facing Units that function as tensile reinforcement.

Notes

Notes

Tensar®

Tensar International Corporation
2500 Northwinds Parkway, Suite 500
Alpharetta, Georgia 30009

tensarcorp.com
800-TENSAR-1

Distributed by:

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MESA_IG_SPEC_2.16

Appendix 4.1.2

Tensar Geogrids QC-QA Manual

Latest report: September 2024

MESA® MSE Retaining Wall System

Back to main document

Tensar
A Division of CMC

THE QC-QA MANUAL ON THE MANUFACTURE OF TENSAR® GEOGRIDS

2024

Document Name/Ref:	QA-QC Manual Morrow
Document holder:	Mark Wayne
Date:	27 September 2024

Revision	Prepared	Reviewed	Approved	Date	Description
NA	M Wayne	D Hall	B Reall	9/27/2024	Global alignment

FOREWORD

TENSAR® geogrids are highly regular structures possessing consistent properties and interactive performance with soil and aggregate materials in field service. The end users of TENSAR® geogrids desire and are entitled to expect consistent quality and performance. Thus, The Tensar Corporation, LLC employs quality procedures throughout the manufacturing process to ensure its geogrids meet its published specifications.

Furthermore, adherence to quality procedures results in a more efficient manufacturing operation, thus controlling costs, and ensuring value to the end user – TENSAR customers.

This manual documents the stepwise, in-house quality procedures implemented by Tensar in the manufacture of TENSAR® geogrids. It is offered to our customers as evidence that quality procedures are in place and followed. This revision supersedes all previous versions.

EXECUTIVE SUMMARY

The Tensar Corporation develops and supplies specialist engineering technologies, technical services, and high-performance polymeric geogrids that it offers as value enhancing alternatives to traditional materials and practices in the building and construction industries. These markets include transportation, commercial and industrial site development, waste containment, natural resource extraction, and coastal and waterway protection industries.

This manual document the in-house quality control procedures employed in the manufacture of TENSAR® geogrids at The Tensar Corporation, LLC plant facility located in Morrow, Georgia. The objective herein is to quantify these stepwise procedures from the receipt of raw materials through the manufacturing process to the finished products.

The process is basically the extrusion of sheet followed by its perforation with a precise arrangement of holes and subsequent stretching, or drawing, into the finished grid configuration. For uniaxial geogrids, drawing is unidirectional, and for multi-axial geogrids drawing is multi-directional.

Quality procedures begin with incoming raw materials, continue through each workstation in the manufacturing process, and conclude with characterization of each lot of finished product. Specific parameters are monitored at each workstation. Each parameter is evaluated according to a prescribed test method or procedure, and frequency. There is a corrective action and disposition for every non-conformance. Descriptive and inferential statistical analyses of all data recorded for each geogrid product is conducted periodically to verify compliance to specifications. Tensar's sampling and testing procedures follow AASHTO National Transportation Product Evaluation Program REGEO and SSGEO workplans.

This manual quantifies the procedures in place to ensure that TENSAR® geogrids will consistently meet the expectations of our customers in field performance in geotechnical and civil construction applications. It is Tensar's policy to test and report data using SI units.

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1.0 INTRODUCTION

The purpose of this quality manual is to document the quality procedures employed in the manufacture of Tensar® geogrids at The Tensar Corporation, LLC plant facility located in Morrow, Georgia.

The objective is to quantify the stepwise procedures in quality control and quality assurance from the receipt of raw materials through the manufacturing process to the finished products.

This scope includes the quality procedures in place for the manufacture of uniaxial and multi-axial geogrids used in the geotechnical and civil construction industry.

2.0 STATEMENT ON QUALITY

The Tensar Corporation, LLC is dedicated to the manufacture of high quality TENSAR® geogrids. Tensar's intent is to meet the expectations of its customer's needs in product quality and field performance in the geotechnical and civil construction applications.

3.0 THE CORPORATION

The Tensar Corporation develops and supplies specialist engineering technologies, technical services, and high-performance polymeric products that it markets as value enhancing alternatives to traditional materials and practices in the building and construction industries. Tensar's innovative technologies, and manufactured products and services are promoted and sold as integrated engineering systems throughout the world. Tensar International Corporation (TIC), a subsidiary, markets and sells the corporation's products and engineering services worldwide to the transportation, commercial and industrial site development, waste containment, natural resource extraction, and coastal and waterway protection industries. Although the Tensar International Corporation has worldwide marketing and sales responsibility, it focuses mainly on the western hemisphere, whereas its subsidiary, Tensar International Limited (TIL), focuses on the eastern hemisphere.

4.0 PRODUCT HISTORY

The manufacturing procedures for uniaxial and multi-axial geogrids are based upon the Tensar process technology developed and patented by the late Dr. F. Brian Mercer of Netlon Limited located in Blackburn, United Kingdom. Netlon Limited is now Tensar International Limited (TIL).

Many patents protect the Tensar products, processes, and application technologies. The Tensar Corporation holds perpetual exclusive rights to these technologies worldwide.

5.0 GENERAL PRODUCT DESCRIPTION

TENSAR® geogrids are integrally connected structures in a grid or lattice configuration that is contiguous throughout the repeating structure and possessing high tensile strength. The Tensar process technology achieves this end result by aligning long-chain, polymeric molecules through the mechanical stretching of perforated sheet that imparts high tensile strength and stiffness throughout the repeating structure.

6.0 QUALITY PROCEDURES THROUGHOUT THE MANUFACTURING PROCESS

6.1 Overall Concept and Approach

Quality procedures begin with the incoming base materials, continue through each work station in the manufacturing process, and conclude with characterization of each lot of finished product. Tensar utilizes a four (4) step sequence assuring a high level of quality in its finished products:

1. Quality assurance (QA) begins with verification that base materials meet in-house specifications.

2. Production personnel implement on-line quality control (QC) at each work station, viz., extrusion, punch, and stretching. Principal parameters are measured at prescribed frequencies on the work-in-process (WIP) at each station. Parameter values are recorded in Quality Windows by product code and lot number for later retrieval and analysis. Any WIP not in compliance with product specifications is physically "tagged" as nonconforming and documented on the paper work that follows the material lot to the next workstation. If nonconforming WIP is deemed beyond correction or segregation at the current or next workstation, the lot is designated as regrind and further processing ceases. A non-conformance (NC) report is issued documenting the disposition and the corrective action(s) to be taken to prevent re-occurrence.

   At a prescribed frequency, QA technicians perform on-line checks at each workstation to verify QC activity and property values recorded.

3. Each lot of finished product undergoes a prescribed physical, mechanical, and analytical tests following industry accepted standards (e.g., ASTM, GRI). Procedurally, physical properties are measured on the production floor; mechanical and analytical properties, in the QA laboratory. Any finished product determined as being out-of-specification is "tagged" as nonconforming product. Then the segregation of conforming product from nonconforming product begins. A non-conformance report is issued for documenting the disposition and corrective action(s) to be taken to prevent re-occurrence. Electronic "tagging" in the database prevents non-conforming, finished product from being shipped inadvertently.

   Parameter values are recorded in a database for subsequent retrieval by product identification and lot number for purposes of certification.

4. Descriptive and inferential statistical management review analysis of all data recorded for each Tensar® geogrid is conducted at least once a year to ensure an adequate frequency of testing. The Goal of the management review is to insure consistent testing methodology is followed and repetitive. The objective of the management review is to provide consistent and repetitive testing procedures. The action plan of the management review is to monitor procedures and results semi-annually to insure consistency. An internal audit of the entire quality system with findings is conducted and documented annually.

5. Any external conformance testing performed on Tensar products is performed by Third party laboratories who are GAI-LAP accredited, unless the customer has requirements for a specific laboratory.

6.2 Quality Assurance of Base Materials

6.2.1 Resins

A resin becomes a qualified resin for the manufacture of Tensar® geogrids after several lots of product have been manufactured successfully, met all physical, mechanical, and analytical specifications. The quality assurance of incoming qualified resins includes verification of resin identification and characterization of its melt point and melt index.

Plastic resins are received by railroad hopper cars or bulk truck. There are typically four (4) compartments within each rail car and bulk truck. At least one sample is taken from each compartment. Due to the capacity of railroad hopper cars, samples are taken from the bottom of each compartment. Vendor resin identification and production lot number are recorded and submitted along with the vendor's quality control data to the QA Laboratory for a compliance check of all samples as prescribed in Table 1.

Table 1

Quality Assurance of Qualified Plastic Resins

Parameter	Method	Frequency	Corrective Action
Resin Identification	Vendor's Documentation	Once per Resin Lot	Non-Compliance Rejection
Melting Point	Fisher-Johns Melt Apparatus	Once per Sample	Non-Conformance Rejection
Melt Index	ASTM D 1238 Procedure B	Once per Sample	Non-Conformance Rejection

6.2.2 Masterbatch

A masterbatch is a qualified plastic resin compounded to a high concentration of a color pigment and/or special additive(s). The quality assurance of an incoming masterbatch includes characterization of melt point and pigment concentration according to the test methods and frequency shown in Table 2.

Table 2

Quality Assurance of Masterbatch

Parameter	Method	Frequency	Corrective Action
Melting Point	Fisher-Johns Melt Apparatus	Once per Lot	Non-Conformance Rejection
Pigment Concentration	ASTM D 4218	Once per Lot	Non-Conformance Adjust Material Blend Formulation

6.2.3 Regrind Resin

Regrind resin is the edge-trim from extruded and oriented sheet, punch-outs from sheet perforation, and finished product not meeting minimum specifications that is reclaimed for granulation and re-worked back into the resin formulation. Thus, regrind resin is comprised of qualified plastic resin, masterbatch, earlier regrind. The quality control is primarily the segregation of resin type (i.e., PE or PP) or additive package (i.e., carbon black, pigment, or flame retardant).

6.2.4 Resin Formulation

A resin formulation is a specific blend of qualified plastic resin, masterbatch, and regrind as prescribed by product specifications.

6.3 Quality Control/Assurance of Work-In-Process

6.3.1 Sheet Extrusion

The principal parameters ensuring quality in extruded sheet are resin formulation of base materials and pigment loading; sheet thickness, profile, width, and surface quality; and weight per unit length. The purpose of monitoring these parameters is given in Table 3.

Table 3.

Quality Control/Assurance of Work-In-Process: Sheet Extrusion
Principal Parameters and Their Purpose

Parameter	Purpose
Material Blend	To ensure the proper formulation for tensile properties.
Pigment Concentration	To ensure UV stability and/or color.
Sheet MD Thickness Profile	To ensure uniformity in stretching, tensile strength properties, and weight / area.
CMD Width	To ensure proper width for successive steps in the manufacturing process.
Sheet Surface Quality	To alert potential processing problems / conditions during sheet orientation.
Weight / Unit Length	Sheet Weight Uniformity

The method of measurement, frequency, and corrective action for each sheet extrusion parameter is summarized in Table 4. A non-conformance(s) is issued against the lot for pigment concentration, sheet thickness, or width being out-of-specification.

Table 4.

Quality Control/Assurance of Work-In-Process: Sheet Extrusion
Method, Frequency, and Corrective Action

Parameter	Method	Frequency	Corrective Action
Material Blend	Weight Measure	Twice / Shift	Adjust blend ratio
Pigment Concentrate	ASTM D 4218	Once per Lot	Adjust Masterbatch Blend
Sheet MD Thickness Profiles	Micrometer	Three Measurements per hour	Check lip die Check roll gap setting Check roll speeds Make adjustments Flag NC area
CMD Width	Tape Measure	Once per Hour	Adjust edge trim
Sheet Surface Quality	Visual Inspection	Continuous	Adjust Temperatures Clean Rollers
Weight/Length QA	Net Sheet Weight & Length	Once per Lot	Record

Non-conformance(s) are documented on the paperwork that follows the reel of sheet to the next workstation. If a non-conformance is deemed beyond correction or segregation at the next workstation, the lot is designated as regrind or scrap and further processing ceases.

Figures 1 and 2 show how sheet thickness and width are measured, respectively. Figure 3 shows an example of high quality, extruded sheet.



Figure 1

Sheet Thickness Measurement



Figure 2

Measurement of Extruded Sheet Width

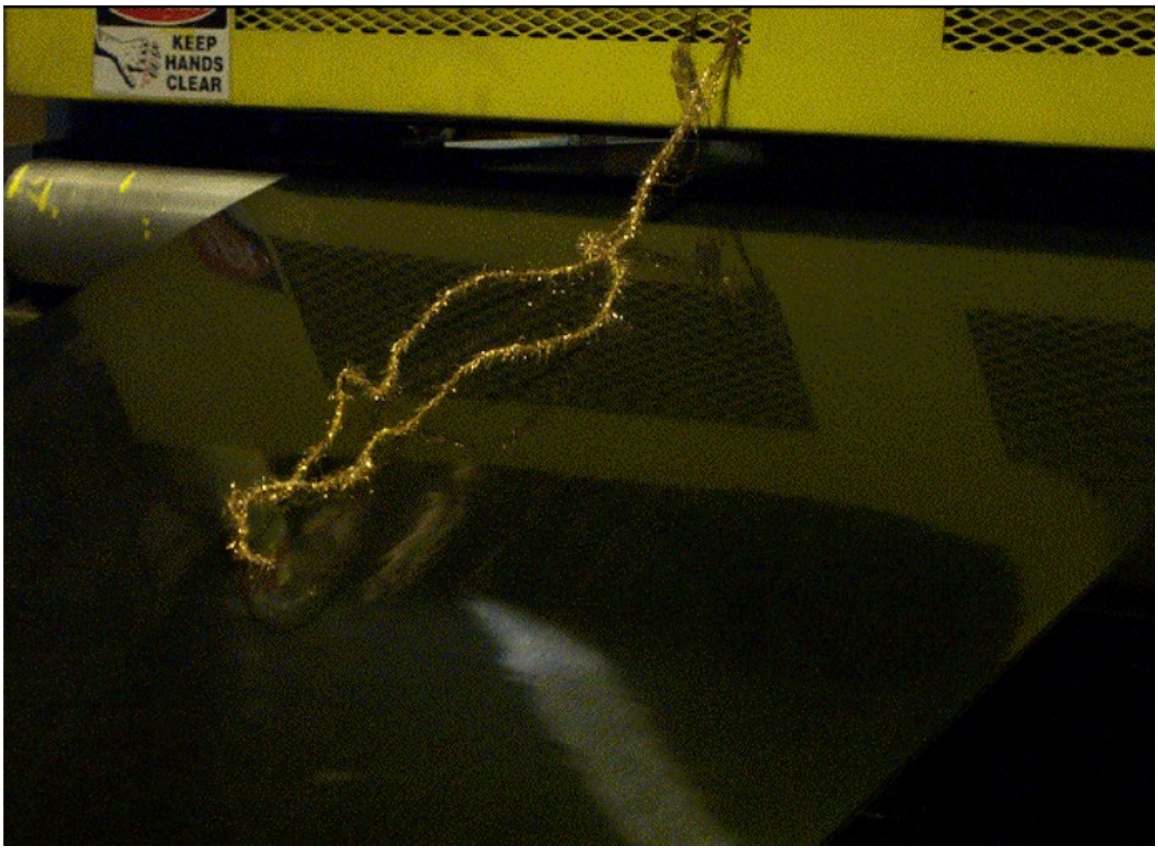


Figure 3 Example of High Quality, Extruded Sheet

6.3.2 Sheet Perforation

The principal parameters ensuring quality in punched sheet are sheet width, punch configuration and alignment, and aperture pitch, squareness, and quality. The purpose of monitoring these parameters is given in Table 5.

Table 5

Quality Control of Work-In-Process: Sheet Perforation
Principal Parameters and Their Purpose

Parameter	Purpose
Sheet Width	To ensure proper width for successive steps in the manufacturing process.
MD Aperture Pitch	To ensure consistent spacing of apertures in the MD.
MD Aperture Alignment	To ensure that each MD row of apertures is parallel to its neighboring rows.
CMD Aperture Pitch & Alignment	To monitor punch tool settings and wear
Squareness	To control the bow and skew in the finished product.
Aperture Quality	To ensure consistent stretching.

Pitch is the number of apertures per unit length in the MD. Pitch controls the amount of material between consecutive apertures within a given MD row in both the perforated sheet and finished product. Aperture alignment is the arrangement of all apertures to within a single MD row with all rows being parallel across the width of the sheet. In concert with punch tool setup, aperture alignment controls the amount of material between adjacent MD rows of apertures across the sheet width (CMD).

Squareness relates to the degree of right-angle orientation of a CMD row of apertures to the MD edge of the sheet. Squareness impacts the degree of bow and skew in the finished product.

The method of measure, frequency, and corrective action for each parameter in sheet perforation is itemized in Table 6. Figure 4 shows the measurement of aperture pitch in the MD.

Table 6

Quality Control of Work-In-Process: Sheet Perforation
Method, Frequency, and Corrective Action

Parameter	Method	Frequency	Corrective Action
Sheet Width	Tape Measure	Every 20 Minutes	Adjust Edge Trim Record Meter Count Flag Area
MD Aperture Pitch & Alignment	Calipers & Straight Edge	Every 20 Minutes	Adjust Press Advance Tool Alignment Flag Area Issue Non Conformance
CMD Aperture Pitch & Alignment	Punch Tool	Punch Tool Setup	Rework Punch Tool
Squareness	T-Square & Tape Measure	Once per punch tool setup	Adjust Punch Tool Alignment Flag Area
Aperture Quality	Visual Inspection	Every 20 Minutes	Sharpen Punches

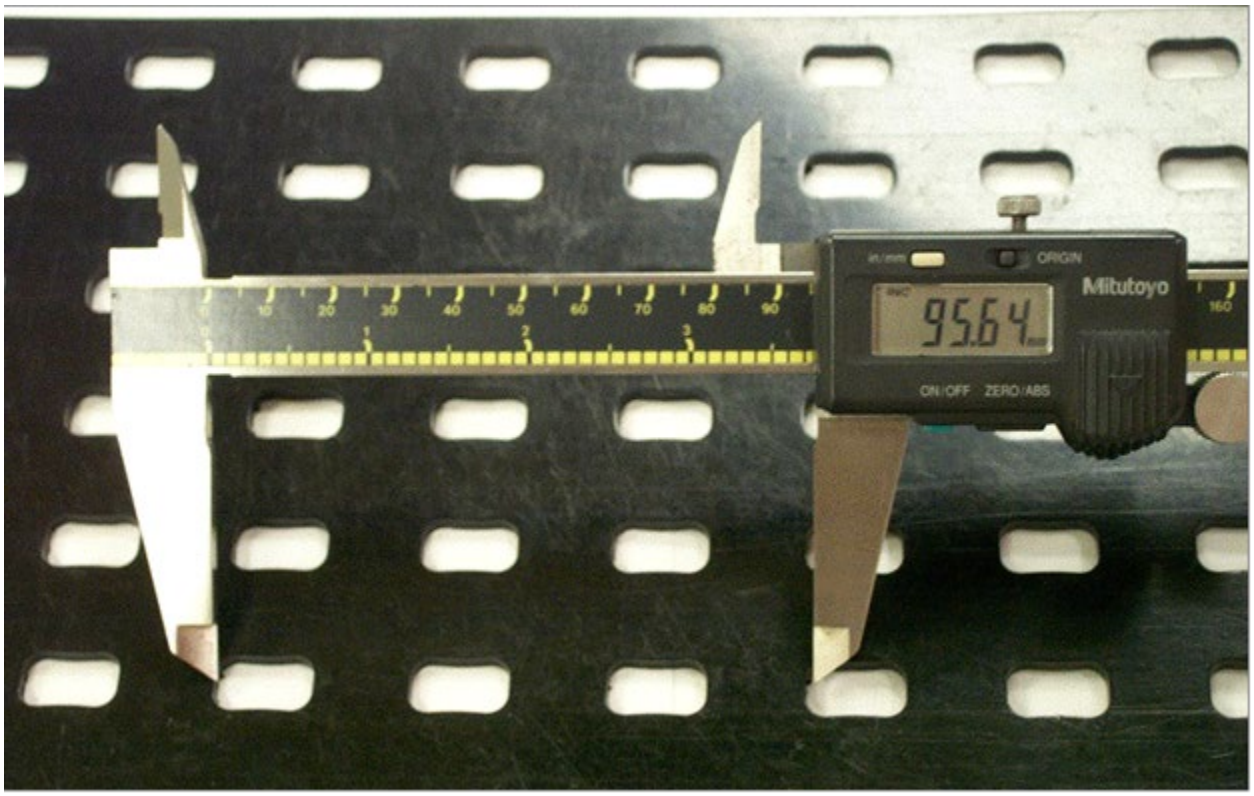


Figure 4 Measuring aperture pitch in the MD

6.3.3 Sheet Orientation

The principal physical parameters ensuring the quality and achieving the specifications in finished product are finished width, minimum aperture length, rib counts, bow and skew, weight per unit length, and roll length and weight. The purpose of monitoring these parameters is given in Table 7.

Table 7

Quality Control of Work-In-Process: Sheet Orientation
Principal Parameters and Their Purpose

Parameter	Purpose
Product Width	Minimum Product Width
MD Aperture Length	Sufficient Orientation
Bow	Squareness
Skew	Squareness
Rib Pitch- longitudinal (Multi-Axial)	Sufficient Orientation
Mid-Rib Depth (Multi-Axial)	In Soil Performance
Mid-Rib Width (Multi-Axial)	In Soil Performance
Nodal Thickness (Multi-Axial)	In Soil Performance

The method of measurement, frequency, and corrective action for each physical parameter out-of-specification is itemized in Tables 8 and 9 for uniaxial and multi-axial products, respectively. Appropriate line adjustments are made to correct any parameter being out-of-specification. Only when the finished product meets all physical property specifications, as measured by manufacturing quality control, is a sample submitted to the QA laboratory for evaluation of mechanical properties.

Table 8

Quality Control of Work-In-Process: Uniaxial Sheet Orientation
Method, Frequency, and Corrective Action

Parameter	Method	Frequency	Corrective Action
Product Width	Tape Measure	Every 5 Rolls	Adjust MD Draw
MD Aperture Length	Tape Measure	Every 5 Rolls	Adjust MD Draw
Bow (1)	T Square Tape Measure	Every 5 Rolls	Adjust Temperatures Heat Distribution
Skew (1)	T Square Tape Measure	Every 5 Rolls	Adjust Temperatures Heat Distribution

(1) For the uniaxial 1700 ARES product, the bow and skew are measured on every roll.

Table 9
Quality Control/Assurance of Work-In-Process: Multi-axial Sheet Orientation
Method, Frequency, and Corrective Action

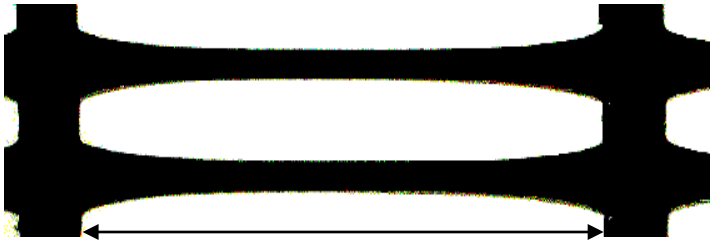
Parameter	Method	Frequency	Corrective Action
Product Width	Tape Measure	20-25 Rolls	Adjust Edge Trim
MD Aperture Length	Tape Measure	Every Lot	Adjust MD Draw
CMD Aperture Length	Tape Measure	Every Lot	Adjust CMD Draw
MD Rib Count per meter width	Tape Measure Direct Count	Every Lot	Adjust CMD Draw
CMD Rib Count per meter length	Tape Measure Direct Count	Every Lot	Adjust MD Draw
Bow	T Square Tape Measure	Every Lot	Adjust Temperatures Heat Distribution
Skew	T Square Tape Measure	Every Lot	Adjust Temperatures Heat Distribution

Table 10
Quality Control/Assurance of Finished Biaxial Products

Parameter	Method	Frequency	Corrective Action
MD Tensile Strength at 2% Strain	**ASTM D 6637	*Once / sub lot	Nonconformance - Regrind
MD Tensile Strength at 5% Strain	**ASTM D 6637	*Once / sub lot	Nonconformance - Regrind
MD Tensile Strength - Ultimate	**ASTM D 6637	*Once / sub lot	Nonconformance - Regrind
TD Tensile Strength at 2% Strain	**ASTM D 6637	*Once / sub lot	Nonconformance - Regrind
TD Tensile Strength at 5% Strain	**ASTM D 6637	*Once / sub lot	Nonconformance - Regrind
TD Tensile Strength - Ultimate	**ASTM D 6637	*Once / sub lot	Nonconformance - Regrind
Carbon Black Content	ASTM D 4128	*Once / sub lot	Nonconformance – Regrind
MD Junction Strength	ASTM D 7737	*Once / sub lot	Nonconformance – Regrind
TD Junction Strength	ASTM D 7737	*Once / sub lot	Nonconformance – Regrind
Rib and Junction Thickness	Direct Measure	*Once / sub lot	Nonconformance – Regrind
Percent Open Area	Direct Measure	*Once / sub lot	Nonconformance – Regrind
UV Testing	ASTM D4355	Annual for select grades	Nonconformance – Regrind
Flexural Rigidity	ASTM D 7748	Once / production order	Nonconformance – Regrind

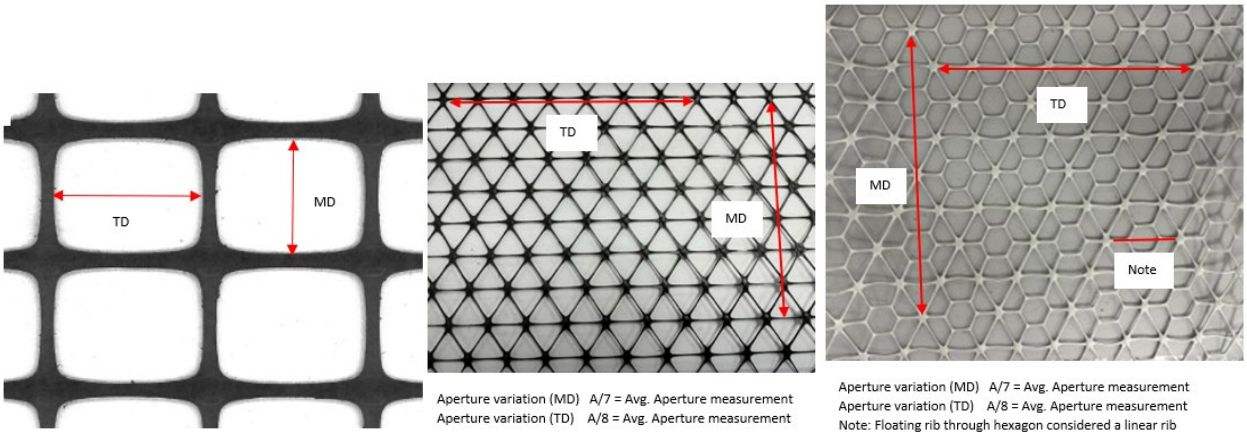
* Sampling per ASTM D4354: A production lot is defined as the summation of all batches (sub lots) which are produced on a daily basis across a fiscal year (September 1-August 31)
Note 1- Once / sub lot for wide width testing and once per sub lot for single rib in accordance with sampling requirements per ASTM D4354

Figure 5 shows how MD aperture length of a uniaxial geogrid is measured. Figure 6 shows how biaxial, TriAx and HX/NX apertures are measured. Figure 7 shows the assessment of the degree of bow and skew in a uniaxial geogrid.



MD Aperture Length

Figure 5 Measuring the MD Aperture Length of a Uniaxial Geogrid



(a) (b) (c)

Figure 6 Aperture measurements for (a) Biaxial, (b) TriAx (c) HX/NX

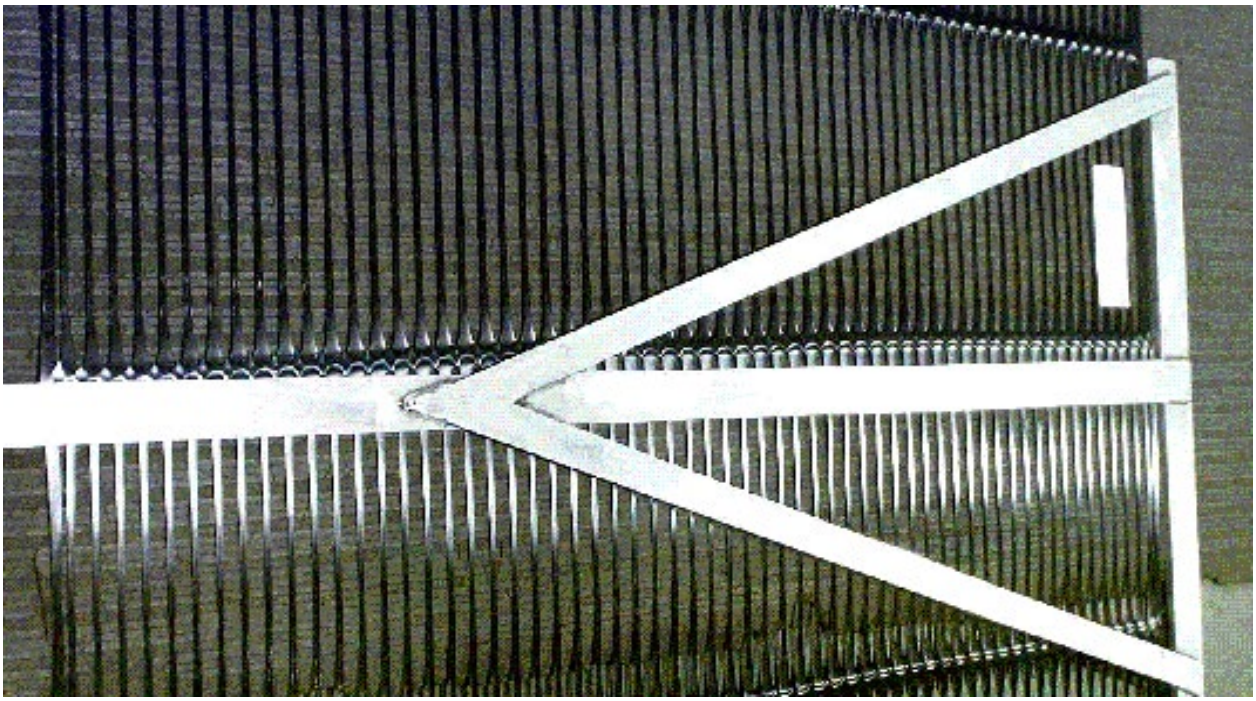


Figure 7 Assessing the Degree of Bow and Skew in a Uniaxial Geogrid.

6.3.4 Finished Products

For uniaxial and multi-axial geogrids, the physical properties are measured at the frequency given in Tables 8 and 9, respectively. For any parameter out-of-specification, process adjustment(s) are made to bring the property(s) back into specification. All rolls out-of-specification are designated as regrind. After about ten (10) rolls are manufactured meeting physical property specifications, a sample is sent to the QA laboratory for evaluation of mechanical properties (Tables 10 and 11).

Table 11

Quality Assurance of Finished Products: Uniaxial Geogrids

Parameter	Method	Frequency	Corrective Action
MD Tensile Strength @ 2% Strain	ASTM D 6637 Instron Software Bluehill 2	Once per Lot	Non-Conformance Regrind
MD Tensile Strength @ 5% Strain	ASTM D 6637 Instron Software Bluehill 2	Once per Lot	Non-Conformance Regrind
Ultimate MD Tensile Strength	ASTM D 6637 Instron Software Bluehill 2	Once per Lot	Non-Conformance Regrind
MD Junction Strength	ASTM D 7737 Instron Software Bluehill 2	Once per Lot	Non-Conformance Regrind
Weight / Length	Tape Measure Scales	Once per Lot	Record
CMD Brittleness	45° Bend Test	Once per lot	Non-Conformance Regrind

If a mechanical property is out-of-specification, the production personnel at the orientation workstation are notified, and samples are taken from the proceeding and following rolls relative to the nonconforming roll sample for measurement of mechanical properties. This procedure is repeated until both samples meet mechanical specifications. All rolls' in-betweens are designated as regrind. Figure 8 shows the tensile testing for MD ribs of uniaxial geogrids.



Figure 8

Tensile Test on Single Rib of Uniaxial Geogrid

Table 12 Quality Assurance of Finished Products: Multi-Axial Geogrids

Parameter	Method	Frequency	Corrective Action
Weight / Length	Tape Measure Scales	Once per Lot	Record
Flame Retardancy (Applicable Products)	UL 94	Every 10 rolls	Non-Conformance Regrind
Radial Stiffness at low strain	ASTM D 6637 Instron Software Bluehill 2	Once per Lot	Non-Conformance Regrind

6.3.5 Line Checks

The intent and purpose of periodic line checks by QA technicians is to verify the activity of on-line quality control and the accuracy and recording of measurements. Line checks are conducted once within every twelve (12) hour shift. Each workstation has a prescribed checklist of items. Measurements by QA technicians are compared against product specification sheets and production records. Any discrepancy(s) must be resolved with the lead operator for that work station and the shift supervisor. For any work-in-process out-of-specification, a non-conformance is issued.

A line checklist by workstation for the manufacture of geogrids is provided in Table 12.

7.0 CONCLUSIONS

This Quality Manual documents the quality and stepwise procedures employed in the manufacture of uniaxial, and multi-axial Tensar® geogrids. Quality procedures start with raw materials and follow through the manufacturing process to the finished products. There is assurance that raw materials meet in-house specifications, continuous control of work-in-process, quantification of performance properties of the finished product, and a system for reporting and tracking non-conformances to disposition with corrective action.

The intent of The Tensar Corporation, LLC is to illustrate conclusively that its geogrid products will consistently exceed its published specifications and the expectations of its customers and field performance in the geotechnical and civil construction applications.

Table 13

Quality assurance of on-line production quality control

Work Station	Process Parameter
Extrusion	Product W-I-P Code Lot Number Blender Ratios Sheet Meter Count Sheet Thickness Sheet Width Sheet Appearance
Punch Press	Product W-I-P Code Lot Number Aperture Quality Aperture Pitch Groove depth Sheet Width
Uniaxial Orientation	Finished Product Code Lot Number Roll Number Bow Skew MD Rib Length Product Width Outer Rib Width Labeling
Multi-Axial Orientation	Finished Product Code Lot Number Roll Number MD Aperture Length CMD Aperture Length Product Width Labeling

8.0 GLOSSARY OF TERMS

8.1 Base Materials

Carbon Black, n – material consisting of elemental carbon in the form of near-spherical colloidal particles and coalesced particle aggregates of colloidal size, obtained by partial combustion or thermal decomposition of hydrocarbons.

Copolymer, n – A polymer of two or more different monomers.

Flame Retardant, n –additive that reduces the tendency of plastics to burn that is incorporated into a blend during compounding.

High-Density Polyethylene (PE), n - family of resins obtained by polymerizing ethylene gas, $H_2C=CH_2$. HDPE exhibits a density ranging from about .941 to .965.

Masterbatch, n - resin compound containing high percentages of pigments and/or other additives, to be added in small amounts to batches [blends] during compounding.

Polydispersity, n - the breadth of the molecular-weight distribution of a polymer. A measure of polydispersity is the ratio of the weight-average and number-average molecular weights (Mw/Mn).

Polymer Material (Resin), n - any plastic compound or resin that is utilized in the process of manufacture.

Polyethylene (PE), n - family of resins obtained by polymerizing ethylene gas, $H_2C=CH_2$. By varying the catalysts and methods of polymerization, properties such as density, melt-flow index, crystallinity, degree of branching and cross-linking, the molecular weight and polydispersity can be regulated over wide ranges.

Polypropylene (PP), n - any of several types of a large family of thermoplastic resins made by polymerizing propylene, H2C=CHCH3. By varying the catalysts and methods of polymerization, properties such as density, melt-flow index, crystallinity, degree of branching and cross-linking, the molecular weight and polydispersity can be regulated over wide ranges.

Regrind, n - waste material such trimmings from sheet extrusion, punch outs, and rejected product that has been reclaimed by granulating. Regrind is usually mixed with polymer material at a predetermined ratio for reprocessing.

8.2 Process Terms

Cross Machine Direction (CMD or XMD), n – The direction perpendicular to the machine direction (MD).

Drawing, n - the process of stretching a thermoplastic causing molecular orientation and change in its physical and mechanical properties.

Extruder, n - machine for producing more or less continuous lengths of plastics having constant cross sections, such as sheets, tubes, etc.

Extrusion, n - process of shaping, by temperature and pressure, a resin through a profile die (i.e., a sheet die).

Extrudate, n - molten resin exiting an extruder die.

Quality Assurance production lot, n - reel of continuously extruded sheet wound onto a reel cart. Lot numbers are assigned by the order of extrusion. (ASTM D4354)

Quality Control production lot, n – The summation of all batches (sub lots) which are produced on a daily basis across a fiscal year (September 1-August 31). This information is used to compute the minimum average roll value (MARV) applicable to a geosynthetic's intrinsic physical properties. The MARV is not appropriate for performance or durability properties (i.e., creep testing). (ASTM D4354)

Machine Direction (MD), n – direction in which a product moves through the manufacturing process.

Orientation, n - the process of stretching a hot thermoplastic sheet to align the molecular chains in the direction(s) of stretching, thus improving modulus and tensile strength in the respective direction(s).

Pitch, n - distance from the leading edge of one aperture to the leading edge of the next aperture.

Rib count, n - number of ribs in a section of sample divided by the width across the sample section, as measured from the **outer edge** of the first rib to the **outer edge** of the last rib across the width of the section. For example, the MD rib count is the number of ribs in the sample in the transverse direction divided by the length of the transverse section, expressed in ribs/meter.

Stretching, n – see orientation.

Multi-Axial Orientation, n – when the orientation is applied in three directions, thus improving modulus and tensile strength in multi-directions.

Work-In-Process (W-I-P), n – material progressing through a work station on the way to a finished product.

Uniaxial Orientation, n - when the orientation is applied in one direction, thus improving modulus and tensile strength in that direction.

8.3 Product Definitions

Aperture, n - the open space between the repeating members of a geogrid.

Aperture length, n - the dimension, of the aperture opening, which runs parallel with the machine direction of the geogrid.

Bow, n – extent of bend or curvature off a straight line in the direction of the ribs.

Diagonal rib, n – tensile load carrying member of the geogrid. A member of the geogrid that runs at a 60 degree and 120 degree angle to the cross machine direction.

Geogrid, n – a geosynthetic formed by a regular network of integrally connected elements with apertures greater than 6.35 mm (1/4 in.) to allow interlocking with surrounded soil, rock, earth and other surrounding materials to function primarily as reinforcement. (ASTM D 4439, D 35)

Geosynthetic, n – a planar product manufactured from polymeric material used with soil, rock, earth, or other geotechnical engineering related material as an integral part of a man-made project, structure, or system. (ASTM D 4439, D 35)

Junction, n - point where the elements of a geogrid intersect.

Longitudinal rib, n - tensile load carrying member of the geogrid. A member of the geogrid that runs between two junctions in a line perpendicular to the cross machine direction.

Skew, n - deviation away from a straight line in the direction of the ribs.

Transverse rib, n **-** soil bearing member of the geogrid. The member, of the geogrid product, which runs between two junctions in a line perpendicular to the machine direction.

8.4 Quality Definitions

Quality Control, n - the operational techniques and the activities which sustain a quality of material, product, system, or service that will satisfy given needs; also, the use of such techniques and activities. (ASTM D4439, D35)

Quality Assurance, n – all those planned or systematic actions necessary to provide adequate confidence that a material, product, system, or service will satisfy given needs. (ASTM D4439, D35)

9.0 GLOSSARY OF TEST STANDARDS

ASTM International

ASTM D 1238	Flow Rates of Thermoplastics by Extrusion Plastometer (Procedure B)
ASTM D 1248	Polyethylene Plastics Molding and Extrusion Materials
ASTM D 1600	Abbreviated Terms Relating to Plastics
ASTM D 4101	Propylene Plastic Injection and Extrusion Materials
ASTM D 4218	Determination of Carbon Black Content in Polyethylene Compounds by the Muffle-Furnace Technique
ASTM D 4439	Terminology for Geosynthetics
ASTM D 4873	Identification, Storage, and Handling of Geosynthetic Rolls and Samples
ASTM D 5262	Evaluating the Unconfined Tension Creep and Creep [Stress] Rupture Behavior of Geosynthetics
ASTM D 6637	Determining Tensile Properties of Geogrids by the Single or Multi-Rib Tensile Method
ASTM D 7737	Individual Geogrid Junction Strength

Geosynthetic Research Institute (GRI) Test Methods and Standards

GRI GG2 Geogrid Junction Strength (2005)

GRI GG4 Determination of Long-Term Design Strength of Stiff Geogrids (1991)

Underwriters Laboratory, Inc.

UL 94 Test for Flammability of Plastic Materials for Parts in Devices and Appliances

Appendix 4.1.3

MESA Connector Compliance

MESA® MSE Retaining Wall System

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PREMIER PLASTIC
MOLDING & TOOLING, INC.

719 Patrick Industrial Lane
Winder, Georgia 30680

May 01, 2025

CERTIFICATE OF COMPLIANCE

The following described shipment, to the best of our knowledge, complies with materials, processes and customer specifications supplied to us:

Product Description:	Standard Mesa Connector
Customer Part #	OP474015L-US-Box
Resin Type & Lot #	Black Polifil GFPPCC-3010 MFG Lot # 31397
Lot #	4502778331
Material Description	PP 30% GF Black
Colorant Part	N/A
Colorant Description	N/A
Quantity Shipped	1058 Boxes (317,400 Parts)
Other Information	(12) Cavity Mold

By: Philip R. Spark
(Signature)

Title: OWNER / CFO
(Print or Type)

Date: 5/1/25

DOING THE NEEDFUL SINCE 1973

March 10, 2025

PREMIER PLASTIC MOLDING & TOOL
719 PATRICK INDUSTRIAL LANE
WINDER, GA 30680

To whom it may concern:

We hereby certify that THE PLASTICS GROUP has manufactured the following Polifil® product to meet or exceed all product specifications:

PRODUCT:	**BLACK POLIFIL GFPPCC-3010**	**Lot Number:**	**31397**
PO Number:	**9361/LOT# 31397**	**Weight:**	**19,200**
Packaging:	**POLIFIL GAYLORDS**	**Mfg Date:**	**3/7/2025**

Properties	Test Method	Conditions	Units	Specification	Results
Ash	TPG WI	565C 20 min	%	29 to 33	30.7
Melt	D-1238	PP-230/2.16	g/10 min	4 to 12	9.75

The property values listed above have been obtained using laboratory controlled test methods. They are offered without guarantee since conditions under which the product is used are beyond our control. Mold shrinkage is intended as a guide only, as specific shrinkage is affected by part design, mold design and molding conditions. Therefore, The Plastics Group disclaims any liability for loss or damage incurred in connection with the use of this product.

Sincerely,

THE PLASTICS GROUP OF AMERICA

David Johnson

David Johnson
Plant Manager

1112 River Street, Woonsocket, RI 02895
Tel 401-765-1000 • 800-984-4TPG(4874)
www.plasticsgroup.com



Appendix 4.2.1

MESA DOT Installation Manual

MESA® MSE Retaining Wall System

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Mesa Construction & Quality Control Manual

This "Installation" manual provides general guidelines for construction and quality control of the installation. This shall be provided to the owner's Engineer, the Construction Quality Assurance Inspector, and the Contractor. This manual's contents provide general guidelines and examples. Specific construction details and procedures depend on individual site conditions and other considerations which are the responsibility of the Owner and the Engineer.

All installation instructions shall apply to the Mesa Standard Unit, the Mesa XL Unit, and the Mesa Landscape Unit, except where otherwise noted.

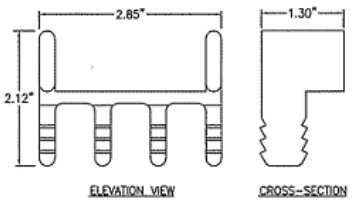
1.1 Responsibilities for Construction Compliance

- The Contractor must provide construction in accordance with the contract documents, plans, and specifications. The Contractor is also responsible for the verification of line, grade, and other physical features, as well as surface water drainage control.

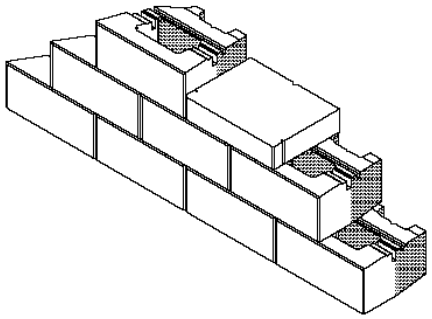
1.2 Materials and Handling

MATERIALS SUPPLIED

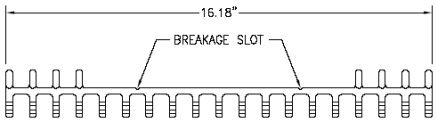
- Mesa Segmental Concrete Facing Units and Mesa Standard and DOT Connectors
- Mesa Cap Units (where required)
- Tensar® Uniaxial (UX) and Biaxial (BX) Geogrids

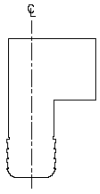


MESA STANDARD CONNECTOR
NOT TO SCALE



MESA STANDARD UNIT & CAP UNIT





DOT CONNECTOR
NOT TO SCALE

HANDLING WALL MATERIALS

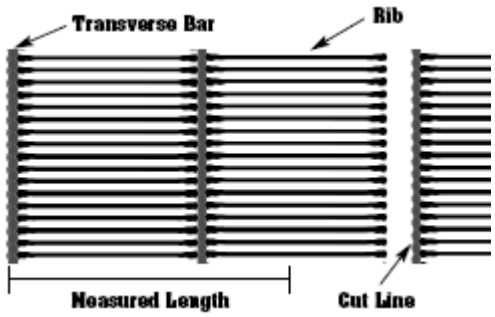
- The concrete facing units are delivered on pallets and off-loaded by the Contractor. Transporting equipment must have firm ground and a stable, level area to off-load. A forklift is normally used to handle pallets. If pallets are the property of the block manufacturer, they must be stored by the Contractor for pickup. The Contractor must provide protection from staining or discoloration of the units by using wood dunnage and polyethylene sheet film.
- The Mesa Standard and DOT Connectors are shipped in cartons and shall be stored in a secure and dry location.
- The Tensar Geogrids are shipped in roll form with the Contractor responsible for off-loading.
- Rolls should be stored in a secure area. Each roll will be labeled as to its type, its lot number, and its roll number.
- Standard roll sizes of Tensar UX Geogrid are 4.36 feet wide by 200 or 250 feet long. Rolls weigh between 87 and 170 pounds depending on type. Before labels have been removed, it is suggested that rolls be color-coded with spray paint on the ends of the rolls to identify geogrid type. The plans may be color coded to match the paint color chosen for each type.
- It is the Contractor's responsibility to verify the quantities shipped and condition of the materials. The Contractor will inventory materials supplied to assure sufficient quantities have been delivered.
- The Contractor will be allowed a limited amount of time to off-load materials.
- If certifications are required, the Contractor must provide a written request prior to shipment of the material. The Contractor will ensure that all information, including product type, roll/lot number, etc., is furnished to the Engineer.

CONTRACTOR SUPPLIED MATERIALS

- Dead blow hammer
- 2-foot or 4-foot levels
- Utility saw and/or grinder
- Plumb bob or other means of checking wall batter
- Masonry string and chalk line
- Pitchfork (optional to remove slack from Tensar Geogrid)
- All cast-in-place concrete and structural components
- Stone filter medium
- Approved shimming materials if geogrid is not used
- Drainage composite and piping (where required)
- Geotextile filter materials (where required)
- Reinforced or select fill
- Stakes and/or rods used to remove the slack from the geogrid unless pitchfork is used
- All labor, equipment, and supervision necessary to perform the total Mesa Wall construction

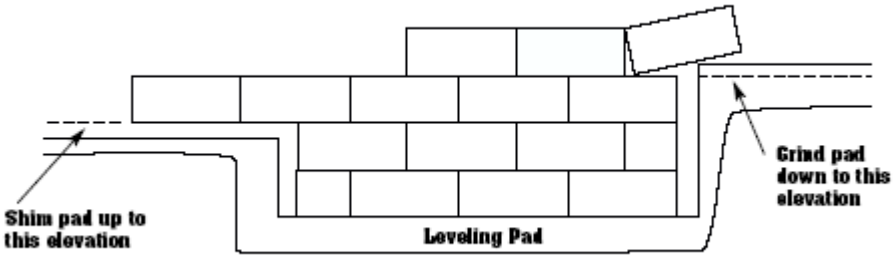
1.3 Preparatory Work for Wall Construction

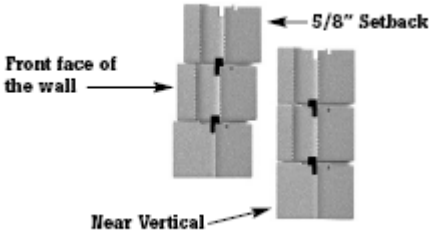
- Verify approval of Mesa Units, Tensar Geogrid, and the specific reinforced fill
- Review drawings to plan Tensar Geogrid layout
- Review drawings and site plans to consider surface water drainage control both during and after construction.
- Prepare subgrade by excavating vertically to plan elevation and horizontally to design geogrid lengths. If a rock interface exists, it is the responsibility of the Engineer to determine the competency of the rock at the limits of excavation shown on the wall plans. ***Any deviation in the location of the rock interface with respect to the face of the retaining wall may require an adjustment to the Tensar Geogrid design and the designer of record must be notified by the contractor prior to proceeding with the wall construction.***
- The subgrade shall be approved by the Engineer before proceeding with wall construction. Any foundation soils found unsuitable by the Engineer shall be treated in a manner approved by the Engineer.
- On larger projects it is suggested that Tensar Geogrid be cut in advance to speed wall construction.
- Cut geogrid flush at the nearest transverse bar beyond measured length, as shown below, or two to three inches in front of the transverse bar to provide “finger shims” as is illustrated on page 6.

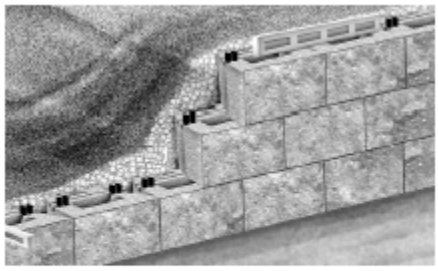


LEVELING PAD CONSTRUCTION

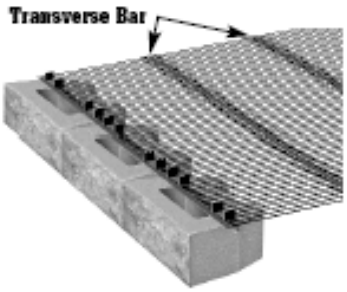
- The leveling pad must be properly surveyed and located in accordance with the contract plans. The leveling pad must be flat and level to assure that the first course of Mesa Units will provide uniform support to the courses above it. Non-uniform support will induce tensile stresses and shear stresses in the Mesa Units above the first course that can result in cracking.
- If contract documents indicate the wall has a battered face, the Contractor shall ensure that the 5/8-inch setback is accounted for at each leveling pad step. It is recommended that the elevation changed of the leveling pad does not exceed four courses. If the contract documents indicate a near-vertical face some adjustment to the leveling pad may be required if the bottom of the wall has many steps. Each course is setback 1/16-inch in the near vertical configuration. (See details below)
- The leveling pad may be constructed with unreinforced concrete or compacted, 3/4-inch minus, well-graded aggregate. It is typically 12 inches wider than the Mesa Units, providing 6 inches in front of and behind the Mesa Unit, allowing for wall curvature and minor alignment adjustments. It is generally 6 inches deep. For unreinforced concrete leveling pads, steel or wood forms are required to assure that the top of the leveling pad is flat and level. For aggregate leveling pads, the aggregate is generally overfilled, compacted, and then carefully trimmed down to near plan elevation. The Mesa Units are then seated into the aggregate with a rubber mallet to the plan elevation, and leveled front to back and side to side.
- Steps in the leveling pad are required to change elevation. It is important that the height of the step is equal to the height of the number of Mesa Unit courses. In practice, the steps may be slightly more or less due to the thickness of the Tensar Geogrid reinforcement connected between courses and to the normal dimensional tolerances of the units. Where a concrete step is not at the same elevation of the adjacent courses, shimming or grinding will be required to match the elevation and provide a flat and level bearing surface for the next course of units. As shown in the figure below, such grinding or shimming will be required over some distance (5 feet min.) from the step to assure that the top surface of the entire course of Mesa Units placed on the stepped section of the leveling pad is flat and level. Ribs of the Tensar Geogrid may be used as shims if their thickness is correct.







As subsequent courses of Mesa Units are placed, it is important that the units remain level from front to back and side to side. This level should be checked as each course is placed. If the units are more than 1/8-inch out of level from front to back or side to side, or if the units are not level from back to front, they should be brought to level by grinding and/or shimming



1.4 Wall Construction

- Wall line shall be established using a chalk or string line. Chalk line should be placed on the concrete pad along the tails of the Mesa Units. A string line can be used in lieu of a chalk line. (Alignment based on the split faced Mesa Units may render an uneven wall face.)
- Once the leveling pad is complete, the first course can be installed. Place the Mesa Unit with the sides touching and the textured face outward. The first course must be accurately placed, carefully spaced, and leveled to facilitate construction and to enhance the appearance of the wall.
- Prior to the installation of the second course and each successive course, the top of the Mesa Units should be adjusted as necessary to be level. Shimming and/or grinding may be required (see "Shimming Mesa Units" section on pages 6 for instructions). The tops of the Units must be swept clean. Failure to do this will result in problems with seating and leveling the Mesa Units and increase the likelihood of cracks developing in the units due to load concentrations as additional courses are set.
- The Mesa Units are stacked in a running bond fashion, similar to standard masonry wall construction. A limit for the shift from a perfect running bond is needed. On straight wall sections, it should be a 1/2 inch to assure all connector teeth are squarely in the slots. The units, once placed, shall be advanced forward toward the face of the wall until they make contact with the Mesa Connector. The connectors do allow for the units to slide from side to side, therefore, the vertical joint alignment should be checked frequently (max. 20 units) to ensure that a running bond pattern is maintained within the limits stated above.
- Drainage fill is placed to the limits shown on the drawings. Drainage and/or core fill may not be required for structural walls (refer to the Definitions section under "Drainage Fill" and "Core Fill"). Proper installation of drainage materials is critical to overall wall performance. Drainage materials must be installed properly and protected during construction.

SHIMMING MESA UNITS

- It is important that the courses of Mesa Units are level front to back and side to side. To achieve this in the front to back plane, it may be necessary to grind the units or use shims between some of the courses to correct for:
  1. the thickness of the geogrid reinforcement ribs that cross the tails of the units, or
  2. for slight differences in the height at the front and back of the units.
- For courses placed on a geogrid elevation, shims may be required on the front face shell of the Mesa Unit. The shims should be the same thickness of the geogrid rib. The shim material can be a rib trimmed from the same roll of Tensar Geogrid that is placed on top of the front face shell of the Unit. An alternative is to cut the geogrid so that the ribs extend approximately 1 inch onto the front face shell. These methods are shown in the drawing below.

Figure 1: Geogrid Rib Shim

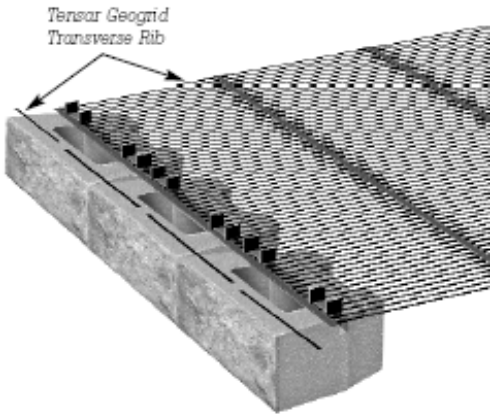
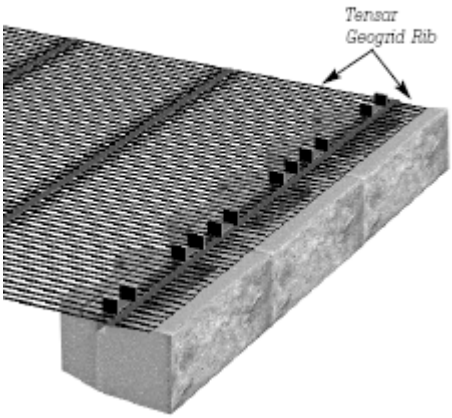


Figure 2: Geogrid Finger Shim

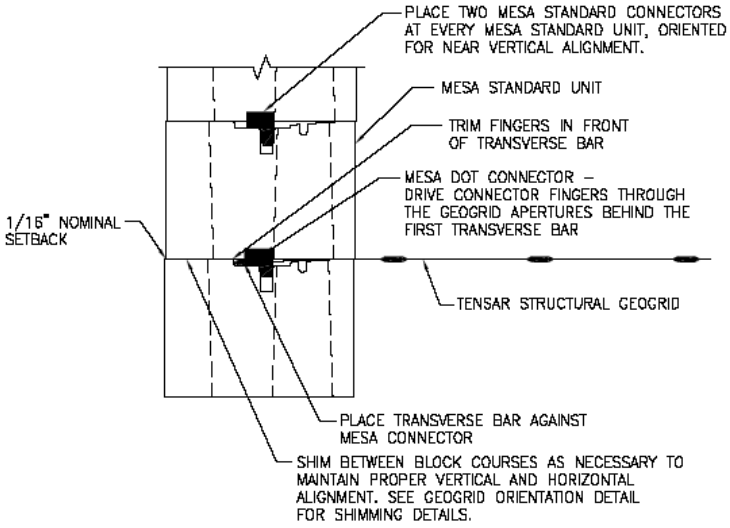


- To correct for slight differences in unit heights, a geogrid rib may be cut and placed on successive courses to bring the face and/or tail back to level. As shown in Figure 1 and Figure 2 (above), the "Rib Shim" and "Finger Shim" should be positioned on the inside of the front face shell of the unit, generally within a 1/2 inch of the core. This results in less stress on the underlying block. Nylon rope or degradable materials may not be used to shim Mesa units.
- Front to back level and side to side level should be check on the first course of Mesa Units placed and every two to three courses above that level. If corrections of more than 1/8" are required to bring the units back level it is suggested that this be accomplished over several wall courses. In addition to checking the level of the units, the batter must be checked periodically. After the first four feet of wall construction, it is recommended that the batter of the wall be spot checked every two feet. Adjustments to the wall batter should be made to achieve the overall wall batter.
- The Tensar Geogrid may be draped over the upper courses of the wall until the reinforced fill reaches the level of the Tensar Geogrid. To ensure the Mesa Wall's geometry is being maintained, a string line should be pulled after each course has been set. The string can be referenced from the connector slot or tail of the Mesa Unit.
- Unless the specifications require tighter tolerances, the wall should be constructed with a final overall batter variance of no more than ½-inch per 10-feet of wall height from the design batter. Horizontal and Vertical alignment variance should be no more ¾-inch over 10-feet.

1.5 Geogrid & Connector Placement

The following section covers the installation for both geogrids and connectors based on use of the Standard, XL, and/or Landscape Units.

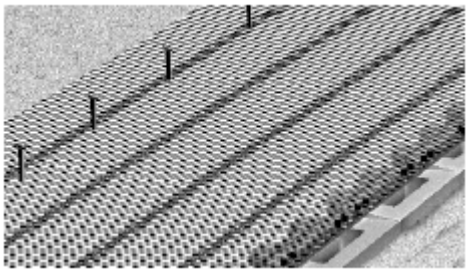
- At the location and elevation shown on the plans, install the Tensar Geogrid of the type and length specified, ensuring the transverse bar of the Tensar UX Geogrid is in the location indicated in the illustration below. The teeth of the Mesa DOT Connector must penetrate through the Tensar Geogrid apertures into the Connector slot on each side of the top of the Mesa Units and into the core of the unit. Where necessary to align the teeth of the DOT connector with the geogrid apertures, the connector may be broken into two or three pieces. Break points have been scored into the top of the connector. The transverse bar of the geogrid should be snug against the connector teeth before final seating of the connector. On block courses that do not receive geogrid, a Mesa Standard Connector shall be placed in each of the two connector slots. All four teeth of the connectors should be positioned in the slots. To aid in the seating of the standard connector a tool may be fashioned from a 2'x4' and used in conjunction with a hammer to seat the connector.



MESA STANDARD UNIT AND GEOGRID
CONNECTION DETAIL FOR NEAR VERTICAL ALIGNMENT
NOT TO SCALE

- Flags on top of the Mesa Connector must be oriented in the proper direction for a "battered" or "near vertical" wall as per the contract documents. Placement of the connector with flags forward (toward the face of the wall) will render a near vertical wall. Placing the connectors with the flags pointed away from the wall face will provide a 5/8-inch setback.
- The fill on which the geogrid is placed must be compacted to meet the project specifications and graded reasonably smooth. On block courses that require geogrid, the reinforced fill must be at the same elevation as the top of the block for a distance of three feet before the fill can be gradually sloped. Remove slack from the geogrid by pulling it

taut, and then anchor it using stakes and/or rods. The stakes used to position the tail of the geogrid can be withdrawn once the fill has been placed and then reused on subsequent lifts. Alternatively, a pitchfork can be inserted at the tail of the geogrid and slack removed by prying until the fill has been placed on the geogrid. The removal of the slack from the geogrid prior to fill placement will help prevent any movement of the wall face due to translation of the slack toward the wall face. It is recommended that at least two to five courses of blocks be placed above the geogrid prior to backfilling.



1.6 Fill Placement

- Fill placement shall be performed to minimize development of slack in the geogrid. The fill should be placed at the wall face and graded away from or parallel to the face of the wall. Any slack that does develop in the geogrid will be pushed toward the free (unconnected) end. The reinforced fill lift thickness and compaction requirements should follow the project specifications. Maximum lift thickness after compaction shall not exceed twelve inches where heavy compaction equipment is used. The contractor shall decrease this lift thickness, if necessary, to obtain the specified density or to maintain wall alignment. Where hand-operated compaction equipment is used, maximum lift thickness after compaction shall not exceed six inches. The reinforced fill shall be compacted to 95% of ASTM D-698 (AASHTO T-99) or as required by the contract specifications, whichever is more stringent. Only hand-operated compaction equipment shall be used within three feet of the facing unit.
- Heavy equipment shall not be used within three feet of the back of the facing units. Tracked construction equipment shall not be operated directly on the geogrid. Rubber-tired equipment may be operated on the geogrid providing the subgrade is not pumping or rutting. Turning of rubber-tired equipment shall be minimized to prevent dislocation or damage to the geogrid or facing units. The equipment must travel slowly and with sufficient care to avoid dislocating the geogrid or the facing units.
- At the end of each day, the Contractor shall ensure the reinforced backfill is graded to drain water away from the face of the wall. Berms and/or ditches must also be in place and functioning to prevent the entrance of surface runoff into the wall construction site. Delays in wall construction during rainy periods should be avoided to minimize the likelihood of saturation of the backfill.

1.7 Other

- The Cap Units, if required, are installed by attaching them to the units below using an approved exterior concrete adhesive. The Cap Units may be placed such that a nominal 1 inch overhang is achieved or flush with the face of the wall.
- Wall penetrations may be accommodated by cutting the Mesa Units to fit using a utility saw and a mason's hammer. Small voids can then be closed with a cement and sand mix or other methods that have been approved by the Engineer.

1.8 Definitions

Contract Documents: The Agreement between the Owner and the Contractor including conditions of the contract drawings, specifications, and the provisions of the Agreement between the Contractor and the Supplier of the Mesa Systems. These documents shall also include addenda and other modifications issued prior to the execution of the Contract.

Core Fill: Free-draining, coarse-grained soil which is placed within the empty cores of the Segmental Concrete Facing Units. Core Fill may not be required within the Mesa Unit if the Contractor can provide the Engineer and/or Architect with connection testing performed without Core Fill verifying that the connection strength of the system exceeds the requirements set forth in the design data.

Drainage Fill: Free-draining, coarse-grained soil which is placed behind the Mesa Segmental Concrete Facing Units and in the openings between the Mesa Units as specified on the Plans. The Engineer and/or Architect may specify a nonwoven geotextile which meets AASHTO M288-96 subsurface drainage requirements in lieu of Drainage Fill.

Inspector: The Owner's Authorized Representative assigned to see that the workmanship and materials are in accordance with the terms of the Contract.

Mesa Connector: A proprietary mechanical connection device made of high-density polymer with fiberglass inclusions to positively connect the Tensar Geogrid to the Mesa Units.

Plans: The part of the Contract documents consisting of the approved plans, profiles, typical cross sections, working drawings and supplemental drawings, or exact reproduction thereof, which shows the location, character, dimensions, and details of the Work to be performed.

Reinforced Fill: The specified soil material that interacts with the Geogrid reinforcement to create a flexible gravity mass. Its limits extend from the back of the facing element or granular medium to the tails of the soil reinforcement or as indicated on the Plans.

Setback (Batter): The rearward offset from the vertical plane between two adjacent block courses created by the orientation of the flags on the Mesa Connectors.

Specifications: A description of the quality and quantity of the materials and workmanship that will be required of the Contractor in the execution of the Work under the Contract between the Owner and the Contractor.

Tensar Geogrids: A proprietary polymeric grid formed by a regular network of integrally connected tensile elements with apertures of sufficient size to allow interlocking with surrounding soil, rock, or earth and connection to Mesa facing units that function as tensile reinforcement.

Appendix 5.1.1.1

MESA Project Overview

MESA® MSE Retaining Wall System

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MESA®
RETAINING WALL SYSTEMS

For Use in Abutment Applications



Tensar®

Atlanta, Georgia

Prepared: February 13, 2009

MESA® System Highlight Project List

1. Monroe Street Viaduct; Baltimore, MD
2. Memorial Parkway; Huntsville, AL
3. SR 397 (Ainsworth Avenue), City of Pasco, Washington
4. Puente Gemelo El Trebol (Guatemala City, Guatemala)
5. Paso a Desnivel San Cristobal (Guatemala City, Guatemala)
6. I-25 Adams County, CO
7. Powers Blvd, El Paso County, CO

MESA Project Details

	Monroe Street Viaduct
Year Constructed	**2005**
Location	Baltimore, MD
Engineer of Record	Steven Valero Tensar Earth Technologies 404-250-1290
Maximum Height	17 feet
Wall Length	540 feet
Wall Area	5,300 SF
Facing	Mesa Standard Units with DOT3 connection
Description	MESA wall with corner units and H-piles founded on a LTP
TIC References	Jeremiah Riggio 404-250-1290
Wall Installer	Joseph B. Fay Co. 410-244-0437
GC	Joseph B. Fay Co. 410-244-0437
Owner's Rep	Batimore DOT 410-396-3835 State Highway Administration of Maryland (MDSHA) 888-228-6971

	Memorial Parkway
Year Constructed	**2008**
Location	Huntsville, AL
Engineer of Record	Brendan Sheppard Tensar International Corporation 404-250-1290
Maximum Height	30 feet
Wall Length	5,100 feet
Wall Area	100,500 SF
Facing	Mesa Standard Units with DOT connection
Description	MESA wall with radius corner and 48" drilled shafts
TIC References	Jeremiah Riggio 404-250-1290
Wall Installer	J.A. Manning 410-244-0437
GC	Reed Contracting 256-533-0505
Owner's Rep	Alabama DOT 334-242-6208

	SR 397 (Ainsworth Avenue)
Year Constructed	**2006**
Location	City of Pasco, Washington
Engineer of Record	Gerry Kehler Tensar International Corporation 404-250-1290
Maximum Height	33 feet
Wall Length	1870 feet
Wall Area	42000 square feet
Facing	Mesa Standard Units with DOT connection
Description	Mesa walls with 90-degree corners, grouted units, and 4 foot x 4 foot pier column obstructions.
TIC References	George Charalambous 404-250-1290
Wall Installer	Apollo, Inc. 509-586-1104
GC	Apollo, Inc. 509-586-1104
Owner's Rep	City of Pasco, Washington 509-544-3080

	Puente Gemelo El Trebol
Year Constructed	**2006**
Location	Guatemala City, Guatemala
Engineer of Record	Jose Perez Precon Guatemala City , Guatemala
Maximum Height	24.3 ft
Wall Length	197 ft
Wall Area	3,760 sf
Facing	Mesa Standard Units with Mesa Standard connectors
Description	Rounded corners. Main bridge beam on the reinforced fill
TIC References	Reinaldo Vega-Meyer 404-250-1290
Wall Installer	Precon (Guatemala) 502-2-328-8000
GC	Precon (Guatemala) 502-2-328-8000
Owner's Rep	

	Paso a Desnivel San Cristobal
Year Constructed	**2006**
Location	Guatemala City, Guatemala
Engineer of Record	Jose Perez Precon Guatemala City , Guatemala
Maximum Height	27.5 ft
Wall Length	820 ft
Wall Area	23,680 sf
Facing	Mesa Standard Units
Description	Rounded corners. Bridge supported on piles. Piles inside the reinforced fill.
TIC References	Reinaldo Vega-Meyer 404-250-1290
Wall Installer	Precon (Guatemala) 502-2-328-8000
GC	Precon (Guatemala) 502-2-328-8000
Owner's Rep	

	I-25 Adams County
Year Constructed	**2008**
Location	Adams County, Colorado
Engineer of Record	Brendan Sheppard Tensar International Corporation 404-250-1290
Maximum Height	12 feet
Wall Length	334 feet
Wall Area	3672 SF
Facing	Mesa Ashford Units with HP2 connectors
Description	2 Abutment walls. Abutment founded on piles.
TIC References	Phil Law 303-429-9511
Wall Installer	Slaton Bros 303-806-8160
GC	
Owner's Rep	Colorado Department of Transportation

	Powers Blvd
Year Constructed	
Location	El Paso County, Colorado
Engineer of Record	Brendan Sheppard Tensar International Corporation 404-250-1290
Maximum Height	30 feet (3 tiers)
Wall Length	2250 feet
Wall Area	23300 SF
Facing	Mesa Standard Units with DOT connection
Description	3 tiers MESA wall and abutment founded on piles.
TIC References	Jeremiah Reggio 404-214-5364
Wall Installer	Slaton Bros 303-806-8160
GC	
Owner's Rep	Colorado Department of Transportation

Appendix 5.1.1.2

MESA Aesthetic Pictures

MESA® MSE Retaining Wall System

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ABUTMENT PROJECT PHOTOS – MESA® SYSTEM



Helper Junction, UT



South Academy, CO



Powers Woodmen, CO



I-25 Weld City,



128th Over I-25,



Founders Meadow, CO



Metrolink, MO



Pearl Street, MA



SR240, WA



Ainswoth, WA

Appendix 5.1.2.1a

and

Appendix 5.1.2.1b

Pearl St Rail Project: photos and Report

MESA® MSE Retaining Wall System

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MESA AND SIERRA SOLVE SPACE CONSTRUCTION PROBLEMS FOR THE MBTA

Mark H. Wayne, Ph.D.
Tensar Earth Technologies, USA

Bob Miller
Tensar Earth Technologies, USA

Proceedings of GRI-9 Conference on
"Geosynthetics in Infrastructure
Enhancement and Remediation,"
R. M. Koerner & G. R. Koerner, Eds., 1996
GII Publ., Philadelphia, Pennsylvania, USA

ABSTRACT

Thousands of people commute to and from Boston everyday, many of them using the "T," subway and commuter rail system, owned and operated by the Massachusetts Bay Transit Authority (MBTA). To provide better service to a greater number of people, the MBTA decided to reopen two old railway commuter lines, the Plymouth and Middleboro, that have not carried passengers since the late 1930's[1]. This paper focuses on the unique manner in which geosynthetic reinforced earth systems were utilized to eliminate a grade crossing located at the line's intersection with Pearl Street. This project is unique in that it is the first project to utilize mechanically stabilized earth (MSE) slopes and unitized segmental block walls to support heavy rail traffic, (E-80 loading).

INTRODUCTION

A key element in the reopening of the Plymouth and Middleboro commuter lines was the elimination of the Pearl Street grade crossing. A map of the area is included as Figure 1. To perform this task, MBTA engineers determined that a bridge would be needed to provide the grade separation between rail and street traffic. Sverdrup Corporation, as a consultant to the MBTA, was commissioned to redesign the area. The existing embankment needed to be raised to carry the track over the road, and six precast panel retaining walls and one tie back structure were originally specified to keep the embankments within the existing right-of-way. The project scope involved retaining the earth around both the bridge abutments and the area under the rail lines after crossing the bridge. Beyond the bridge, the tracks split into a "Y," heading in the directions of Plymouth and Middleboro. In addition to the six walls specified by Sverdrup for the rail line, an additional three cast-in-place walls were required to accommodate vehicular traffic on a realigned Pearl Street.

Tensar Earth Technologies, under the contractor designed retaining wall section of the project, redesigned the specified precast panel walls and the tie back structure. These were replaced by three Sierra® MSE slopes, one Mesa® unitized segmental retaining wall, and three combination

slope/wall MSE structures. The three cast-in-place walls along Pearl Street were also replaced with Mesa walls.

ADVANTAGES OVER TRADITIONAL SYSTEMS

There are many advantages to using this particular reinforced embankment (Slope & Wall System). Based purely on aesthetics, the system surpasses the appearance of cast-in-place or panel wall systems. An oversteepened geogrid reinforced slope can be used in place of or in combination with a segmental retaining wall to reduce costs and increase right-of-way. The concrete face is either fully or partially eliminated, creating either a "green" wall, or an architectural block face wall/ green wall combination. The oversteepened slope concept can easily accommodate rapid changes in grade which can result from unexpected field conditions. In addition, the ability to utilize readily available site soils as a construction fill material over select soils furthers the cost advantage.

In areas where space restrictions require the use of vertical wall systems, a segmental block retaining wall system again is an aesthetic alternative to cast-in-place or panel wall systems. Unlike traditional segmental block walls, this new system is the only geogrid reinforced MSE segmental block wall system which combines polymeric reinforcement, high compression block, and a patented mechanical connector, to create a fully integrated system. A typical detail of the block and FS2.0™ Connector is shown as Figure 2. The high compression block is specified at a 27,580 kPa (4000 PSI) minimum. As such, it is more resistant to the effects of differential settlement and freeze/thaw action than the more commonly used 13,790-20,690 kPa (2500 to 3000 PSI) landscape blocks. The FS2.0 mechanical connector locks the structural geogrid to the block and provides the only positive mechanical connection between segmental concrete facing units and structural geogrid reinforcement in the industry today. Table 1 presents connection strength test results for the block and various geogrids. These tests were conducted at very low confining pressures, 6.9 kPa (1 psi) and 13.8 kPa (2 psi), which is equivalent to approximately 0.5 m (1.5 feet) and 1 m (3 feet) of overburden, respectively. The ultimate strength of the connection for UX1400 is over 2.5 times greater than the long term design strength of the geogrid for both confining pressures. The connection strength at 19 mm (0.75 inch) displacement is also over 2.5 times the long term design of the geogrid for both confining pressures. The results for the UX1500, UX1600, and UX1700 are similar to that of the UX1400. The ultimate strength was achieved at approximately 19 mm (0.75 inch) of displacement and is approximately equal to or greater than twice the long term design strength of the geogrid. This information is important when considering the use of segmental block wall systems in the design of railway transit systems. This is mainly due to the fact that E-80 train loading results in high lateral loads at the top of the wall, where confining pressures are low. Table 1 confirms the fact that the block/geogrid connector provides adequate connection strength even at low confining pressures.

Table 1 - Mesa/Tensar Structural Geogrid Connection Test Results

Geogrid	Long-Term Design Strength kN/m (lbs/ft)	Confining Pressure kN/m^2 (psi)	Ultimate Connection Strength kN/m (lbs/ft)	F.S. on Ultimate Connection Strength	Connection Strength @ 19 mm (3/4 inch) Displacement kN/m (lbs/ft)	F.S. on Connection Strength @ 19 mm (3/4 inch) Displacement
UX1400	14.8 (1016)	6.9 (1)	45.5 (3120)	3.1	36.9 (2528)	2.5
UX1400	14.8 (1016)	13.8 (2)	38.8 (2659)	2.6	38.5 (2638)	2.6
UX1500	24.6 (1683)	13.8 (2)	69.4 (4756)	2.8	59.4 (4071)	2.4
UX1600	32 (2190)	13.8 (2)	78.9 (5404)	2.5	64.1 (5340)	2.4
UX1700	38.9 (2666)	13.8 (2)	85.3 (5845)	2.2	85.3 (5845)	2.2

DESIGN PROGRAMS

This project was designed through the use of the Tenswal[2] and Tenslo[3] computer programs. The Tenswal program uses the tie-back wedge method of analysis (which is the standard practice in the United States) to determine the geogrid type, location, and lengths required to satisfy both internal and external stability. The slope stability program, Tenslo, uses a modified bishop method of analysis. The geometry, soil boundaries and strengths, phreatic surface, seismic coefficients, geogrid strengths, and geogrid layout are input into the program. The program then uses a predetermined pattern of failure circle centers as specified by the user to calculate driving and resisting moments of the section. The increased resistance against failure due to geogrids intersected by a circular slip plane is modeled as a resisting moment based on the depth of overburden, geogrid strength and length of reinforcement behind the failure surface. The program determines the critical radius for each circle center analyzed. After all circle centers have been analyzed the program then determines the lowest factor of safety and summarizes the information for this circle center. Based on a railroad train loading equation provided by MBTA, a railroad train spreadsheet calculation was used to analyze train loading conditions and reinforcement requirement.

SURCHARGE LOADING

Several different surcharge loads were used in the design of the walls and slopes for the Pearl Street project. A 28.7 kN/m^2 (600 psf) surcharge was applied to take into account construction loads. A 55.8 kN/m^2 (1165 psf) railroad train surcharge was determined in accordance with equations supplied by the MBTA. The equation supplied by the MBTA stated that the surcharge of an E-80 train is equivalent to approximately 2.9 m (9.5 feet) of soil with a unit weight of 18.9 kN/m^3 (120 pcf). In addition, a 1.2 kN/m^2 (25 psf) surcharge was included to account for the track. The summation of these two surcharge loads was used in design. For those track sections where 1.5 m (5 feet) or more of open space existed between the tracks and edge of a wall or

slope, a 12 kN/m^2 (250 psf) surcharge was added to account for possible truck traffic. For the temporary wall system, an additional analysis considered a 12 kN/m^2 (250 psf) traffic surcharge, along with 500 mm (20 inches) of ballast at 18.9 kN/m^3 (120 pcf). This equated to a dead load surcharge of 21.5 kN/m^2 (450 psf). Typical cross sections for the steep slopes, unitized segmental block retaining wall, and slope/wall combinations are depicted as Figures 3, 4, and 5, respectively.

GEOGRID DESIGN STRENGTHS

Since this project involved phased construction, design analysis included phased, temporary, and permanent walls. Phased and temporary walls were designed on the basis of a six month service life. Due to the viscoelastic nature of geosynthetic materials, the tensile strength is dependent upon the rate of loading of the material. Based on this fact, along with supporting documentation from external studies, geogrid strengths were increased (from their long-term, 100 year design strength) for all short term loading conditions.

GENERALIZED DESIGN PROCEDURE

1) The TENSWAL[2] design program was used to evaluate the internal stability of the reinforced wall system. A 28.7 kN/m^2 (600 psf) construction surcharge and a factor of safety for uncertainties of 1.5 or greater was considered in the analysis. For the temporary wall system, an additional analysis considered a 12 kN/m^2 (250 psf) traffic surcharge and 500 mm (20 inches) of ballast at 18.9 kN/m^3 (120 pcf). This equated to a dead load surcharge of 21.5 kN/m^2 (450 psf). Since ballast would not be added in the case of the phased construction wall only a traffic surcharge of 12 kN/m^2 (250 psf) was considered in this analysis.

2) The TENSLO[3] design program was used to perform global stability analysis. Two cases were considered in this analysis. Global stability analysis took into consideration a 55.8 kN/m^2 (1165 psf) railroad train surcharge and 12 kN/m^2 (250 psf) surcharge for truck traffic adjacent to the track. In addition, a 28.7 kN/m^2 (600 psf) construction surcharge was considered separately. For both cases, a global factor of safety of 1.5 or greater was used in the analysis.

3) The TENSLO program was also used to perform seismic stability analysis on the various cross sections. Based on a short duration loading of less than 1 hour, the strength of the structural geogrid was increased. Per MBTA, a peak horizontal acceleration of 0.17 g was considered along with an acceleration coefficient of 0.10 g as per AASHTO guidelines[4]. An overall factor of safety equal to or greater than 1.13 was used in this analysis.

4) Railroad train loads were analyzed with the train spreadsheet calculation. Distances from the appropriate cross sections developed by MBTA were used to establish: a) distance from the wall face to the first railroad tie; b) width of railroad ties; and c) distances between the two railroad ties.

CONCLUSIONS

This paper focused on the unique manner in which geosynthetic reinforced earth systems were utilized to eliminate an at grade crossing located at the line's intersection with Pearl Street. This project was unique in respect to the fact that it is the first project to utilize mechanically stabilized earth (MSE) slopes and unitized segmental block walls to support heavy railroad traffic (E-80 loading). This project also demonstrated the unique manner in which a mechanically stabilized earth slope and unitized segmental block wall were easily combined into a single MSE structure. The wall and slope work for the Pearl Street project is expected to be completed in October of 1995, and the entire project is scheduled for completion by September 1996. With its completion, the Plymouth and Middleboro commuter lines will carry passengers once again.

ACKNOWLEDGMENTS

The authors would like to thank Sean Wokasien, Joe Emberson, and Dolly Cockroft of the TET design group for their assistance with the preparation of this paper. Special thanks to Francis Clark and Kathy Frederick for their invaluable article on this topic. The authors would also like to express their gratitude to the Sverdrup Corporation and the general contractor, DeMatteo Construction, for considering the use of the Tensar MSE systems.

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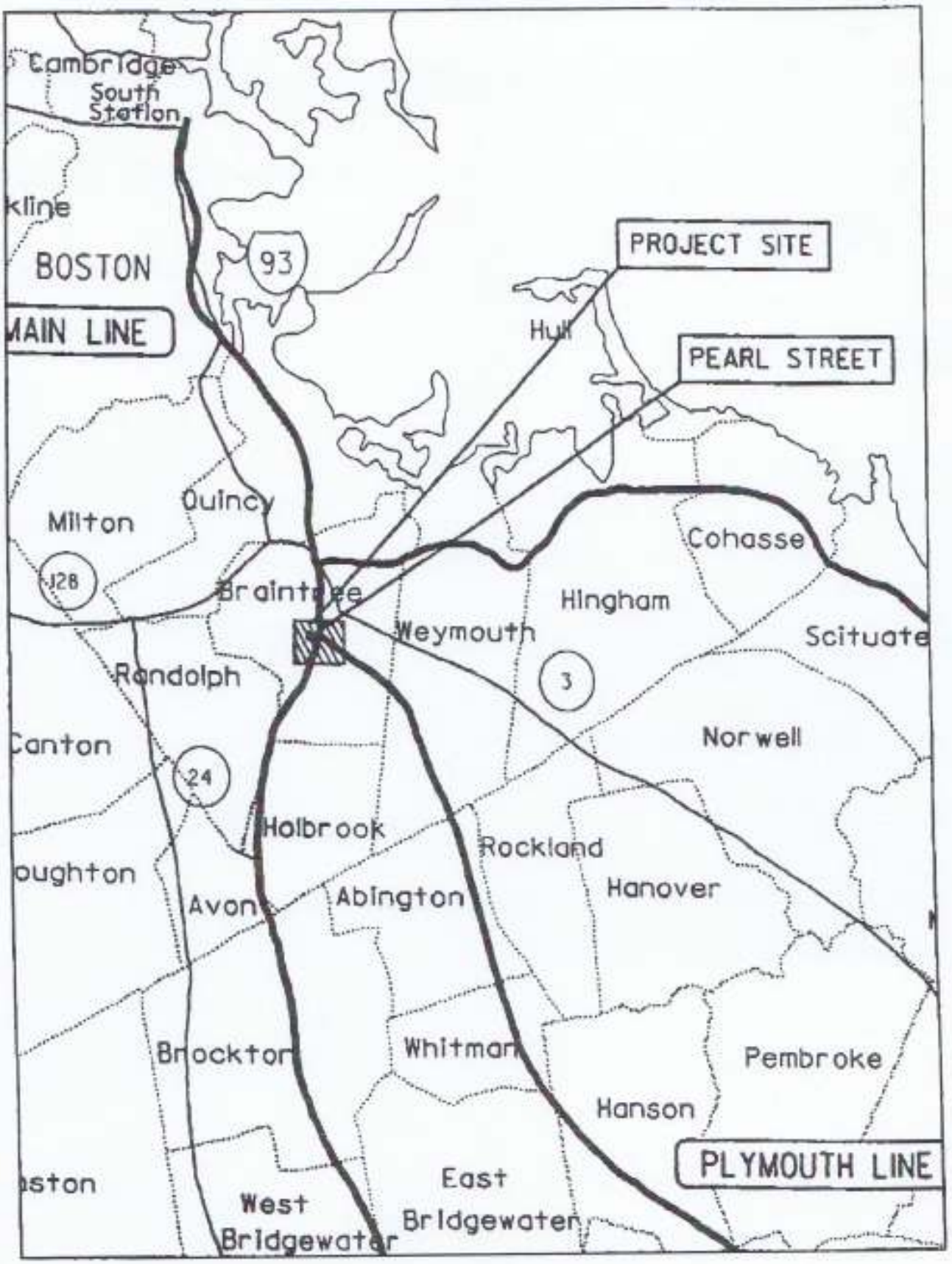


Figure 1: General Project Location Map

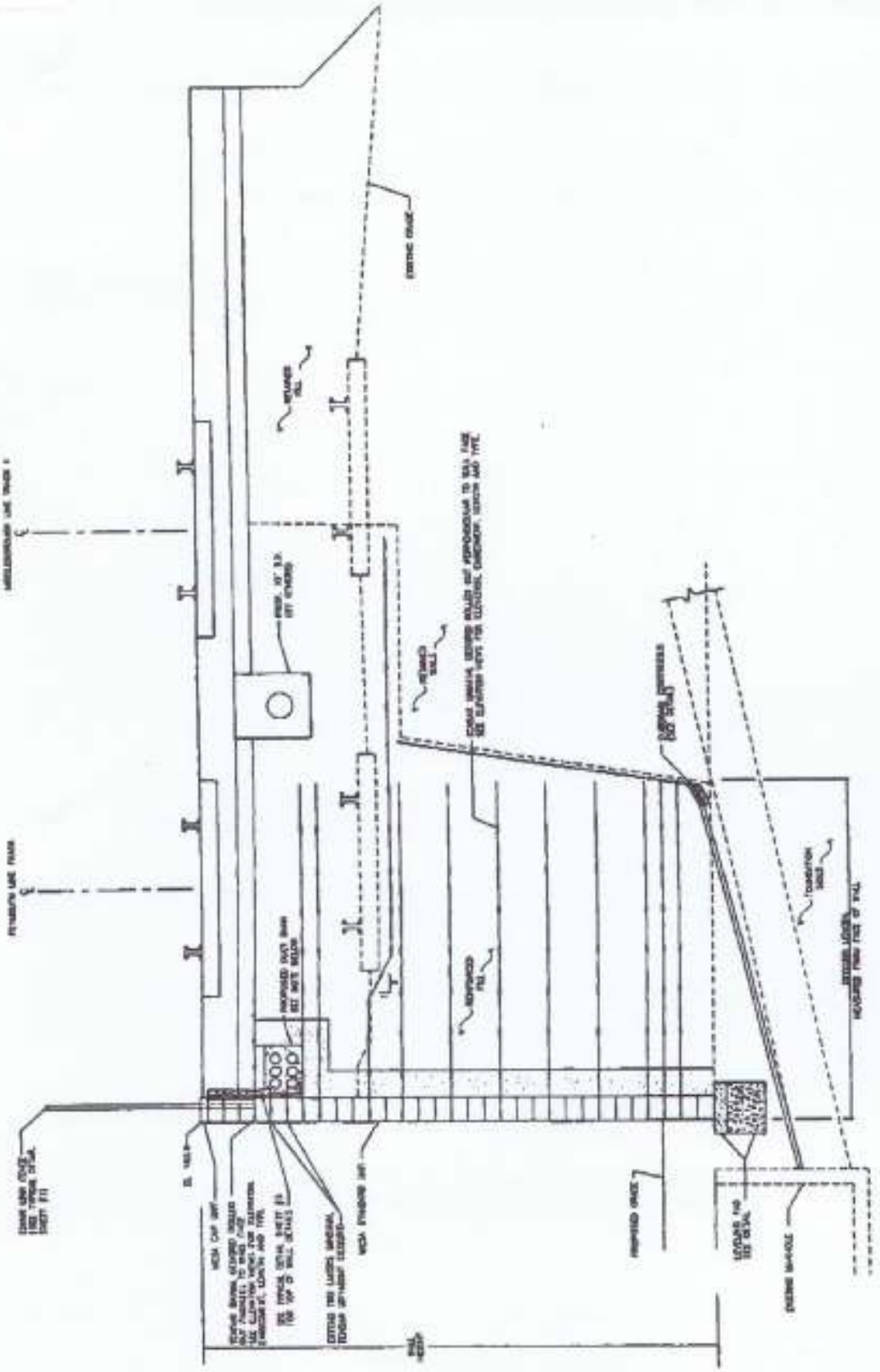


FIGURE 4 - TYPICAL SEGMENTAL BLOCK WALL DETAIL

Appendix 5.1.2.2

Brattleboro - Vermont Project

MESA® MSE Retaining Wall System

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Tensar Mesa™
Retaining Wall System
Initial Report 2006-5
August 2006

Reporting on Work Plan 2001-S-9
Category II Experimental Project

State of Vermont
Agency of Transportation
Materials and Research

Neale Lunderville, Secretary of Transportation
Richard Tetreault, Director of Program Development
William Ahearn, Materials and Research Engineer

Prepared by:

Drew Gelfenbein, P.E.
Civil Engineer III

Reviewed by:

Christopher C. Benda, P.E.

Date: 12/12/06

Introduction

In 2003 the Vermont Agency of Transportation constructed its first Tensar Mesa™ retaining wall system as part of the Brattleboro NH010-2(2) project. The wall was constructed as the northwest wing wall of the railroad bridge over RT 9.

Tensar Earth Technologies, Inc provided the shop drawings for the project, based upon design criteria provided by CLD Consulting Engineers. Since this was the State of Vermont's first time using this particular type of MSE wall, it was designated a Category II Experimental Feature.

The Tensar Mesa™ retaining wall system is an economical and aesthetically customizable product that worked reasonably well for this project. The wall is approximately 9 meters high and 17 meters long, for a total wall area of approximately 98 square meters (1052 square feet).

The Tensar Mesa™ retaining wall system was thought to be beneficial for several reasons:

- Complete details of the wall system would be solicited in advance and incorporated into the contract documents. This would allow contractors in this area not familiar with this type of construction to become better acquainted with the construction requirements.

- The design could be reviewed in advance by the Agency of Transportation. This would allow the Agency to resolve any problems it had with computations, allowable stresses, design loads, construction details and specifications, before bid letting.

- An MSE wall can be constructed to greater heights compared to a conventionally reinforced concrete abutment.

- In accordance with the Agency's "Policy on Earth Retaining Structures" dated November 1995, successful completion and satisfactory performance of this wall in the field would allow the addition of another MSE system to the Agency's Approved Product List and more competitive bidding of future projects.

- An MSE wall would be more tolerant of differential settlement than a conventional reinforced concrete wall.

This report documents our observations during and post construction, laboratory testing performed on the facing elements and provides a summary of our recommendations.

Product Description

The retaining wall system supplied by Tensar achieves its structural integrity through the use of geogrid, placed within layers of compacted fill material, which are attached to concrete blocks to form a reinforced earthen embankment, wall or abutment (Figure 1). The blocks are formed with grooves which are then fitted with plastic clips to provide a means of securing the geogrid to the blocks during construction (Figure 2). The geogrid is laid out horizontally behind the panels, secured to the blocks and covered with compacted select backfill. The layered system of geogrid forms a mass which is sufficiently stable to provide structural support without the use of piles.



Figure 1: Geogrid being placed in layers of compacted fill.

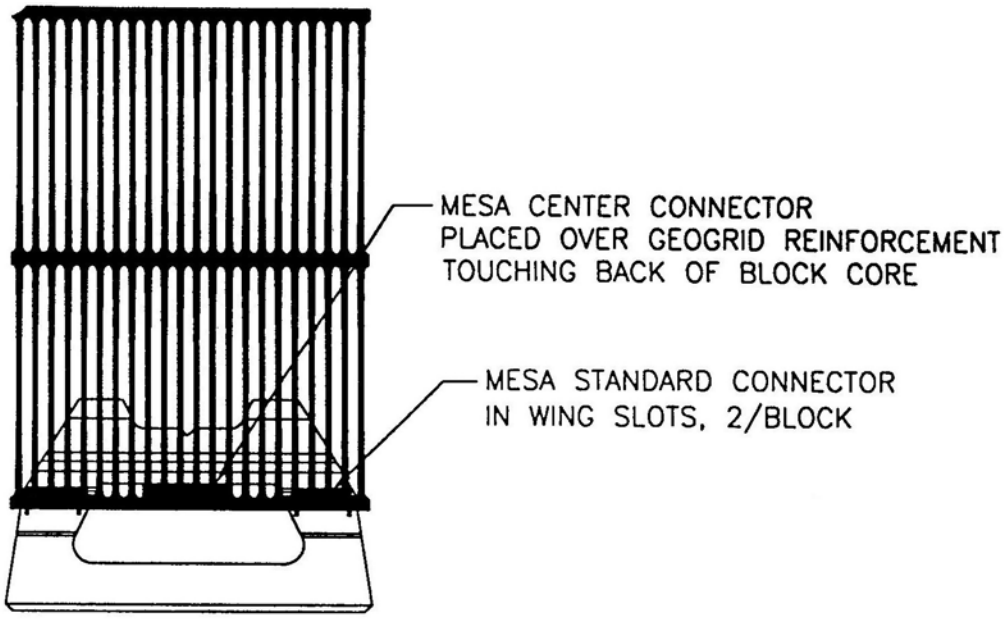


Figure 2: Geogrid Connection to Concrete Blocks

Reinforcement Geogrid

The geogrid is made from high density polyethylene (HDPE). The length and strength of the geogrid would be project specific, based upon the design requirements.

Wall Units

Each unit has a face dimension of 203 mm (8 inches) high and 457 mm (18 inches) wide with a minimum thickness of 279 mm (8 inches). Each unit weighs approximately 35 kilograms (77 pounds). The split face block can be seen in Figure 3.



Figure 3: Split Face Block

Leveling Pad

The leveling pad is the base for the Tensar Mesa™ retaining wall system. It is constructed of non-reinforced concrete with a minimum 28 day strength of 20.7 MPa (3000 psi). The leveling pad had nominal dimensions of 150-mm (6 inches) thickness and 610-mm width (24 inches).

Design Considerations

The height of the wall needed was approximately 7.5 meters (24 feet). A standard cast in place concrete wall would have required a wide footing to resist the overturning forces of the retained material. The size of the footing would have made a concrete wall very expensive.

The Tensar Mesa™ retaining wall system does not need a large footing. A small leveling pad is poured and approximately 1 meter (4 feet) of the wall is constructed below grade. For this reason this option is much more cost effective.

Construction

The Tensar Mesa™ retaining wall system was assembled by the J.A. McDonald Construction company from Lyndon Center, Vermont. See Figure 4 for Typical Cross-Section.

The first stage of construction involved placing the 610mm by 150mm non-reinforced concrete leveling pad on which the blocks rest. The geogrid is laid out and attached to every third course of blocks

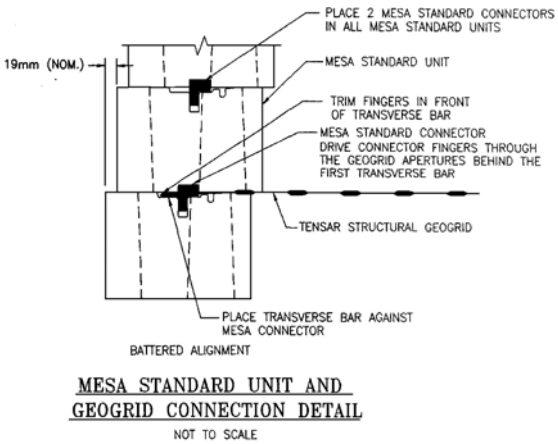


Figure 4: Typical Cross section of block and geogrids connection

using the plastic clips provided (Figure 5) by the wall supplier. After securing the geogrid to the blocks, the backfill material was placed on the geogrid and spread out with CAT 325c excavator. The backfill material was placed in maximum lifts of 250mm (10 inches) compacted with an Ingersoll-Rand SD-450 roller and a plate compactor. The plate compactor was used in any area within 3 feet of the wall face to limit vibration transmitted to the wall.

For this wall, 4 different strength geogrids were used. The strongest geogrid was used at the bottom of the wall. At the suggestion of Tensar's representative the different types of geogrid were color coded using spray paint and flagging. The color coding system allowed for the different strength geogrids to be easily distinguishable and prevented the wrong geogrid from being used.



Figure 5: Clips installed through geogrid into modular block.

Observations

- In the courses where the geogrid was placed, the blocks had to be shimmed. The geogrid was placed under the back edge of the block and tipped the block slightly forward. If the blocks were not shimmed the batter of the wall would be lost in these courses.

- The batter of the wall is designed to be 19mm ($^5/_8$ inch) per course of blocks. This is achieved by placing the block back firmly against the plastic clip set in the block below it. The batter in the field was closer to 13mm (½ inch) per course.

- The dimensions varied from block to block. In order to keep the seams of the block lined up gaps were left between some of the blocks. (Figure 3) The variation in dimensions also made it hard to keep the wall face straight. (Figure 6)



Figure 6 – Wall Face

- A number of the plastic clips were broken. The cause was not seen, but it could have been due to tight fit between the blocks and the clips or due to careless installation of the blocks and/or clips. Supposedly no broken clips were used in rows where geogrid was placed.

Freeze-Thaw Testing

During the construction of the Tensar Mesa™ retaining wall system in the fall of 2003 a random concrete block used for the wall facing was taken from the construction site and brought to the Vermont Agency of Transportation's Materials and Research Testing Laboratory. Two rectangular cubes were cut from the concrete block. The initial dimensions of the samples can be found in Table 1. The two samples were tested according to AASHTO T 161-00 (ASTM C 666-97), Resistance of Concrete to Rapid Freezing and Thawing. The test was conducted with a freeze temperature of 0^{o}F and a thaw temperature of 40^{o}F. A complete freeze-thaw cycle time of approximately 4 hours was used. Prior to the beginning of the test the fundamental transverse frequencies and initial weights were measured on the samples and then performed weekly, after approximately 50 cycles. The State of Vermont requires one modification of the standard method. Due to the severity of the winters in the region and the high use of salt on the roadways the samples were tested in a 3% salt water solution.

Test Results:

Sample 1 - Dimensions 2 11/16" x 2 15/16" x15 5/8"

Date	Cycle	Weight (g)	% Weight Loss	Frequency	Relative Dynamic Modulus of Elasticity	Recorder Comments
02/18/2004	0	4744.1	-	1613		
02/27/2004	63	4677.4	1.41	1430	78.60	A little mortar loss on ends
03/04/2004	103	4524.0	4.64	1466	82.60	Lots of loss, top cracked
03/12/2004	153	3847.5	18.90	-		4 pieces
03/19/2004	194	-				

Table 1a: Test Results - Sample 1

Sample 2 - Dimensions 2 7/8" x 2 3/4" x15 11/16"

Date	Cycle	Weight (g)	% Weight Loss	Frequency	Relative Dynamic Modulus of Elasticity	Recorder Comments
02/18/2004	0	4572.7	-	1554		
02/27/2004	63	4472.1	2.20	1470	89.48	A little mortar loss on ends
03/04/2004	103	4050.8	11.41	-	-	Lots of loss on ends
03/12/2004	153	-	-	-	-	Broken into 4 small pieces, much mortar loss

Table 1b: Test Results – Sample 2

Both samples performed very poorly. The test method requires up to 300 cycles or failure, whichever occurs first. Neither sample could continue the test past 200 cycles, as they had fallen apart. Figure 7 shows Sample 1 when testing was terminated after 194 cycles.



Figure 7a: Sample prior to removal from freeze-thaw apparatus.



Figure 7b: Drained sample.



Figure 7c: Sample removed from tray.

Observations January 2006

In January 2006 a site visit was performed as the final follow up to the construction of the Tensar Mesa MSE wall. The wall appeared to be functioning well with the following few exceptions:

Two blocks (Figure 8) appeared to have vertical stress fractures. The blocks were not located in the same vicinity.



Figure 8a – Vertical fracture



Figure 8b – Vertical fracture

There was one area of 5 blocks (Figure 9) where they all seemed to have a corner chipped away. This condition could possibly have existed from construction or occurred after construction.



Figure 9 – Blocks with chipped corners

Recommendations

- The tolerances for block height and width should be tightened in order to provide a more uniform block.
- The specification should be written in such a manner as to allow the inspector more authority to reject blocks. The inspectors commented that some blocks were chipped or cracked and should not have been used.
- The blocks performed extremely poorly under the AASHTO T 161-00 (ASTM C 666-97), Resistance of Concrete to Rapid Freezing and Thawing test. A sealant should be specified to be applied to the wall face after construction and as part of a periodic maintenance schedule.

In conclusion, it is recommended that the Tensar Mesa™ retaining wall system for use on Agency projects be withheld until the manufacturer can clearly demonstrate that the facing units will not deteriorate in a freeze-thaw environment in the presence of de-icing chemicals. The authors feel that this important issue has not been adequately addressed through prior research and field studies.

It is also recommended that this project be monitored into the future for any adverse changes and the changes are reported in future updates.

Appendix 5.1.2.3

Founders-Meadows CDOT Brige Project

MESA® MSE Retaining Wall System

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Report No. CDOT-DTD-R-2001-12

Performance of Geosynthetic-Reinforced Walls Supporting the Founders/Meadows Bridge and Approaching Roadway Structures

Report 2: Assessment of the Performance and Design of the Front GRS Walls and Recommendations for Future GRS Abutments

Naser Abu-Hejleh
Jorge G. Zornberg
Trevor Wang
Michael McMullen
William (Skip) Outcalt



October 2001

COLORADO DEPARTMENT OF TRANSPORTATION RESEARCH BRANCH

The contents of this report reflect the views of the authors, who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views of the Colorado Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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7. Author(s) Naser Abu-Hejleh, Jorge G. Zornberg, Trevor Wang, Michael McMullen, and William (Skip) Outcalt	**8. Performing Organization Report No.** CDOT-DTD-R-2001-12
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15. Supplementary Note

16. Abstract The Founders/Meadows structure is the first major bridge in the United States built on footings supported directly by geosynthetic-reinforced soil (GRS) walls, eliminating the traditional use of deep foundations altogether. The performance of the front GRS walls, which support the bridge structure and embankment behind the abutment wall, was investigated by collecting data for the movements of the wall facing, settlement of the bridge footing, distributions of the vertical earth pressures and geogrid tensile strains inside the front GRS walls, and lateral earth pressures against the wall facing. Monitoring data was collected during six construction stages and while the structure was in service. This report provides a summary and analysis of the collected data, assessment of the performance and design of the front wall, and recommendations for design and construction of future GRS abutments. Compaction operations created large loads in the reinforcements and against the wall facing during interim construction stages of the front wall. The front GRS walls showed excellent performance because: (i) the monitored movements were significantly smaller than those expected in design or allowed by performance requirements, (ii) post-construction movements and geogrid strains became negligible after an in-service period of 1 year, (iii) measured loads in the reinforcements, connections, and on the wall facing were less than or around 50% of those estimated in the design, (iv) there is not any potential for overturning the structure (due to the flexibility of GRS wall system, resulting in the reduction of loads developed behind and against the wall facing), and (v) the measured bearing pressures were well below the allowable soil bearing capacity. The design employs a high creep reduction factor for the geogrid reinforcements although little if any long-term creep was observed.

Implementation The excellent performance of the Founders/Meadows GRS walls and other GRS abutment structures reported in the literature suggests that the use of GRS walls to support both the bridge and approaching roadway structure should be considered by CDOT design engineers as a standard alternative in future bridge abutment projects. The deformations and performance monitored at the Founders/Meadows GRS walls can be used as a reference for future projects. The features, limitations, and recommendations for the design and construction of future GRS abutments are presented. Recommendations to waive the design requirements for facing connection strength and to employ larger percent of the geosynthetic ultimate strength in the design are furnished.

17. Key Words	18. Distribution Statement
GRS abutments, design, performance, creep, connection, walls, MSE	

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price

CONVERSION TABLE

U. S. Customary System to SI to U. S. Customary System

(multipliers are approximate)

Multiply (symbol)	by	To Get (symbol)	Multiply	by	To Get
		LENGTH			
Inches (in)	25.4	millimeters (mm)	mm	0.039	in
Feet (ft)	0.305	meters (m)	m	3.28	ft
yards (yd)	10.914	meters (m)	m	1.09	yd
miles (mi)	1.61	kilometers (km)	m	0.621	mi
		AREA			
square inches (in^2)	645.2	square millimeters (mm^2)	mm^2	0.0016	in^2
square feet (ft^2)	0.093	square meters (m^2)	m^2	10.764	ft^2
square yards (yd^2)	0.836	square meters (m^2)	m^2	1.195	yd^2
acres (ac)	0.405	hectares (ha)	ha	2.47	ac
square miles (mi^2)	2.59	square kilometers (km^2)	km^2	0.386	mi^2
		VOLUME			
fluid ounces (fl oz)	29.57	milliliters (ml)	ml	0.034	fl oz
gallons (gal)	3.785	liters (l)	l	0.264	gal
cubic feet (ft^3)	0.028	cubic meters (m^3)	m^3	35.71	ft^3
cubic yards (yd^3)	0.765	cubic meters (m^3)	m^3	1.307	yd^3
		MASS			
ounces (oz)	28.35	grams (g)	g	0.035	oz
pounds (lb)	0.454	kilograms (kg)	kg	2.202	lb
short tons (T)	0.907	megagrams (Mg)	Mg	1.103	T
		TEMPERATURE (EXACT)			
Farenheit (°F)	5(F-32)/9 (F-32)/1.8	Celcius (° C)	° C	1.8C+32	° F
		ILLUMINATION			
foot candles (fc)	10.76	lux (lx)	lx	0.0929	fc
foot-Lamberts (fl)	3.426	candela/m (cd/m)	cd/m	0.2919	fl
		FORCE AND PRESSURE OR STRESS			
poundforce (lbf)	4.45	newtons (N)	N	.225	lbf
poundforce (psi)	6.89	kilopascals (kPa)	kPa	.0145	psi

Performance of Geosynthetic-Reinforced Walls Supporting the Founders/Meadows Bridge and Approaching Roadway structures

Report 2: Performance and Assessment of the Design of the Front GRS Walls and Recommendations for Future GRS Abutments

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Report No. CDOT-DTD-R-2001-12

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Research Branch

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Colorado Department of Transportation
Research Branch
4201 E. Arkansas Ave.
Denver, CO 80222
(303) 757-9506

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EXECUTIVE SUMMARY

The Founders/Meadows structure is the first major bridge in the United States built on footings supported directly by geosynthetic-reinforced soil (GRS) walls, eliminating the use of traditional deep foundations altogether. The first report of this study (Abu-Hejleh et al., 2000) presented the design, materials, construction, and instrumentation of these GRS walls. The performance of the front GRS walls, which support the bridge structure and embankment behind the abutment wall, was investigated by collecting data for the movements of the wall facing, settlement of the bridge footing, distributions of the vertical earth pressures and geogrid tensile strains inside the front GRS walls, and lateral earth pressures against the wall facing. Monitoring data was collected during six construction stages and while the structure was in service. This 2nd report presents a summary and analysis of the collected data, assessment of the performance and design of the front GRS wall based on the reliable collected data, and recommendations for design and construction of future GRS abutments.

The maximum geogrid tensile loads, connection loads, and lateral earth pressures against the wall facing measured during placement and compaction of 1 meter of backfill are up to twice those estimated in the design. Measures to alleviate/contain the compaction influence during interim construction stages of the wall were furnished in Chapter 6.

The Founders/Meadows front GRS walls experienced excellent performance because:

- The monitored movements were smaller than those expected in design and less than a third of those allowed by AASHTO performance requirements. Post-construction movements and geogrid tensile strains became negligible after an in-service period of 1 year. Preliminary results suggest no signs of development of bridge bump problem.
- The measured maximum geogrid tensile loads range from 43% to 57% of the values estimated in design.
- The measured connection loads range from 19% to 47% of the values estimated in design. The measured lateral earth pressures against the wall facing range from 4.2 % to 36.4% of the values estimated in design. The measured vertical earth pressures behind the wall facing range from 2% to 67% of the vertical earth pressure values employed in design. The

placement of bridge superstructure had a small influence on the loads developed behind and against the upper zone of the wall facing.

- The measured eccentricity values of the resulting vertical forces acting at different horizontal levels inside the front GRS wall were negligible (i.e., there is not potential for overturning the Founders/Meadows structure). This was attributed to the flexibility/movements of the front GRS wall system that reduced the loads developed behind and against the wall facing.
- The measured bearing pressures at the base of the reinforced fill and below the bridge footing were below the expected design and allowable values of the soil bearing capacity.

If the post-construction geogrid strains were developed due to traffic loads only (not from creep), the design overestimated the reinforcement loads due to traffic load by almost two times. If the post-construction geogrid strains were developed due to creep (not from traffic load), they are relatively very small (zero to 0.09%), and they leveled out and became constant from January to June 2000. However, CDOT design procedure employs a creep reduction factor of 2.7 to determine the geogrid long-term design strength from the geogrid ultimate strength.

The findings of this study agree well with the research findings on three other GRS bridge supporting structures (FHWA, 2000; see Section 1.5.1). First, all four structures experienced acceptable level of movements under service surcharge pressure up to 200 kPa. Second, all structures showed negligible long-term lateral creep deformation under service load. Third, lateral earth loads against the relatively flexible wall facings and connections were small.

Implementation Statement

The excellent performance of the Founders/Meadows GRS walls and other GRS abutment structures reported in the literature suggests that the use of GRS walls to support both the bridge and approaching roadway structure should be considered by CDOT design engineers as a standard alternative in future bridge abutment projects. This application works well for multiple spans bridges and allows for construction in stages and within a small working area. GRS abutments can be considered at any time scour is not a significant problem. GRS abutments may be advantageous from cost basis whenever 1) a fill retaining structure is needed at an abutment and pile driving or drilled shafts are difficult or expensive, and 2) as a competitive measure to

alleviate the bridge bump problem resulting from *moderate* post-construction settlements (25 mm to 75 mm) of the approach slab. A settlement below 25 mm can be more economically dealt with by using an approach slab approximately 9 m long. A settlement of 75 mm would be a typical abutment settlement limit for a superstructure of a medium span bridge (see Chapter 2). When expected structure settlements exceed 75 mm, deep footings should be considered to support the bridge abutment. In all cases, the foundation soil for the leveling pad of the wall facing should be firm.

Future GRS abutments constructed and designed as in the Founders/Meadows structure are expected to experience a satisfactory performance. Abu-Hejleh et al. (2000) presented the layout, design, and material and construction specifications of the front GRS walls of the Founders/Meadows structures (see also Chapter 1). The recommendations that could be considered in the design and construction of future GRS abutments to enhance the performance and reduce the project costs are:

1) The maximum tension line can be assumed to be bilinear. It starts at the toe of the wall and extends through a straight line to the back edge of the bridge footing at the mid height of the wall, and from there extends vertically to the back edge of the bridge footing.
2) The displacements reported for the Founders/Meadows structure provide a reference of the order of magnitude of displacements anticipated in future GRS abutment projects.
3) To minimize wall deformations to the lowest level, it is preferred to: a) place backfill behind the abutment wall before placing the girders, b) place the GRS backfill during the warm and dry season, and c) the well-compacted granular backfill should have a friction angle of 40 degrees.
4) The Founders/Meadows project verifies this application for applied surcharge bearing pressures up to 150 kPa. Higher surcharge pressures up to 200 kPa are allowed that could require reinforcement with closer spacing than that employed in the Founders/Meadows structure.

Based on the performance of the Founders/Meadows structure and the findings of other research studies (see Section 1.5.1), the following additional recommendations could be made:

1) Place the blocks and reinforcements as in the Founders/Meadows structure but without mechanical connections between blocks (i.e., waive the requirements for facing connection strength). In this case, it may be preferred to utilize blocks having a raised lip along the block's outside face and to use setbacks 50% larger than those employed with the mechanical connections. In order to control/minimize the wall facing movements in the upper 1.6 m of the wall below the surcharge load, it is recommended for that zone to: 1) place reinforcements with a wrapped-around procedure, 2) attach the facing blocks to the reinforced soil mass through tail reinforcements that extend 1 m into the reinforced soil mass, and 3) dowel and grout with cement the top block layers (in lieu of gravel filled block cells).

2) Employ any type of geosynthetic reinforcements (not necessary geogrid) having a weight per area larger than 271 g/m^2 (from CDOT specification) that meet the design requirements and:

   a. Reinforcement spacing of 0.2 m (not 0.4 m as in the Founders/Meadows structure).

   b. Reinforcement stiffness consistent with those used in the Founders/Meadows structure in order to maintain small loads in reinforcements and an acceptable level of wall movements. In other words, for a vertical reinforcement spacing of 0.2 m, the tensile force at 1% of lateral tensile strain should be larger than 1000 kN/m, measured in accordance with ASTM D 4595 test method.

   c. Reinforcements long-term design strength estimated as high as 35% of the ultimate strength (17% employed in the Founders/Meadows structure for the geogrid). To reliably use this high value, it may be necessary to perform confined creep tests (FHWA, 1997; Wu and Helwany, 1996) on soil-geosynthetic specimens (materials and testing conditions as expected in the project) and show that the GRS composite will experience negligible long-term creep deformations.

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1.0 BACKGROUND

1.1 Overview

The technology of geosynthetic-reinforced soil (GRS) systems has been used extensively in transportation systems to support the self-weight of the backfill soil, roadway structures, and traffic loads. The increasing use and acceptance of soil reinforcement has been triggered by a number of factors, including cost savings, aesthetics, simple and fast construction techniques, good seismic performance, and the ability to tolerate large differential settlement without structural distress. A comparatively new use of this technology is the use of GRS abutments in bridge applications, in which the reinforced soil mass would directly support both the bridge and approaching roadway structures. In this case, the reinforcement tensions and soil stresses are mobilized in a different manner than in the case of GRS walls supporting small surcharge loads. When compared to typical systems involving the use of deep foundations to support bridge structures, the use of geosynthetic-reinforced systems has the potential of alleviating the "bump at the bridge" problem caused by differential settlements between the bridge abutment and approaching roadway.

The most prominent GRS abutment for bridge support in the U.S. is the new Founders/Meadows Parkway structure, located 20 miles south of downtown Denver, Colorado. It carries Colorado State Highway 86 over U.S. Interstate 25. Figure 1.1 shows the segmental retaining wall system located at the southeast side of the new Founders/Meadows Bridge structure. This figure shows the bridge superstructure supported by the "front GRS wall," which extends around a 90-degree curve into a "lower GRS wall" supporting the "wing wall" and a second tier, "upper GRS wall." Figure 1.2 shows a plan view of the completed two-span bridge and approaching roadway structures. Each span of the new bridge is 34.5 m long and 34.5 m wide in order to accommodate six traffic lanes and sidewalks on both sides of the bridge. Figure 1.3 shows a typical cross-section through the "front GRS wall" and "abutment GRS wall." The figure illustrates that the front GRS wall provides direct support for the bridge and approaching roadway structures. The centerline of the bridge abutment wall and front edge of the foundation are located 3.1 m and 1.35 m, respectively, from the rear of the block facing of the front GRS wall (Figure 1.3). A short reinforced concrete abutment wall and two wing walls, resting on the spread foundation,

confine the reinforced backfill soil behind the bridge abutment (Figures 1.1, 1.2, and 1.3) and support the bridge approach slab.

The Founders/Meadows structure was the first major bridge in the United States built on footings supported directly by a geosynthetic-reinforced soil system, eliminating the use of traditional deep foundations (piles and caissons) altogether. A key element in the design was the need to support the high concentrated loads from the bridge footing and to alleviate the bridge bump problem. In addition, the construction allows for construction in stages and comparatively smaller construction working areas. The competent claystone bedrock formation below the base of the reinforced backfill and the use of an extended reinforced zone (Figure 1.3) significantly enhanced the overall stability and minimized settlements of the front GRS wall structure. The reasons listed above, other perceived advantages of GRS structures, and excellent performance of full-scale geosynthetic reinforced abutments and piers (see Section 1.5.1) convinced Mr. Trevor Wang and other Colorado DOT engineers to select GRS walls to support the Founders/Meadows superstructure. CDOT designed this structure in 1996, before FHWA published preliminary design details for bridge superstructures directly supported by MSE walls with panel facings (not blocks) and steel reinforcement (not geosynthetic reinforcement) in 1997 (Elias and Christopher 1997).

The performance of bridge superstructures supported by GRS abutments has not been tested under actual service conditions to merit acceptance without reservation in highway construction. Consequently, the Founders/Meadows structure was considered experimental and comprehensive material testing, instrumentation, and monitoring programs were incorporated into the construction operations. Three sections of the GRS system were instrumented to provide information on the structure movements, distribution of vertical earth pressures, lateral earth pressure on the wall facing, geogrid strains, and soil temperatures and moisture content. Performance data were collected during six construction stages and after opening the structure to traffic. Monitoring will continue until the structure's long-term movement becomes negligible. The overall objectives of this investigation are:

- To assess the performance of the structure (front and abutment GRS walls) under service loads using short- and long-term movement data.
- To evaluate the suitability of CDOT and AASHTO design procedures and assumptions for the use of front GRS wall as a measure to support the bridge footings, and, in conjunction with the abutment GRS wall, to alleviate the bridge bump problem.
- To collect performance data for future calibration and validations of numerical models.

The first report of this study (Abu-Hejleh et al., 2000) presented the design, materials, construction, and instrumentation of the Founders/Meadows structure. The focus of this second report is on the performance of the front GRS wall that supports the bridge structure and the embankment behind the abutment wall (see Figure 1.3). This report compiled herein presents a summary and analysis of all the data collected in the monitoring program for the front GRS wall, and assessment of the performance and design of this wall. The report recommendations for future design and construction of GRS abutments supporting directly bridge and approaching roadway structures were presented previously in the Executive Summary. This report is organized as follows:

- The rest of this chapter will present a brief description of the front GRS wall's design (Section 1.2), materials (Section 1.3), and instrumentation (Section 1.4). See Abu-Hejleh et al. (2000) for more detailed description of these issues. Findings of previous research studies referred to in the discussion of the results presented in this report are presented in Section 1.5.
- Chapter 2 presents a summary and discussion of the construction and post-construction induced movement data of the front wall facing and settlement of the bridge footing and comparison with the design and tolerable values.
- Chapter 3 presents a summary, analysis, and discussion of the collected measurements for the distribution of vertical earth pressure within the front GRS wall during six construction stages and one post-construction stage. The measured and analyzed data are compared with the design values and discussed.

- Chapter 4 presents a summary, analysis, and discussion of the collected measurements for lateral earth pressure against the wall facing during all monitored stages. The measured and analyzed data are compared with the design data and discussed.
- Chapter 5 presents a summary, analysis, and discussion of the collected measurements for the distribution of geogrid strains within the front GRS wall during six construction stages and one post-construction stage. The measured and analyzed data are compared with the design data and discussed.
- Chapter 6 presents a summary of all the study findings regarding the response and performance of the front GRS wall and an assessment of CDOT design of the front GRS wall.

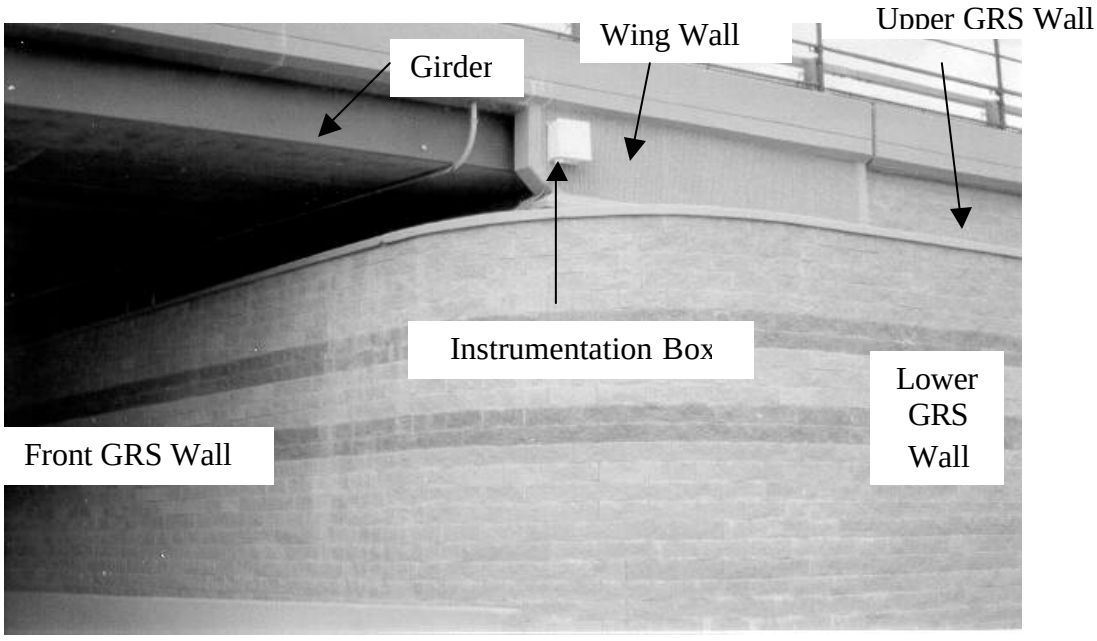


Figure 1.1 View of the Southeast Side of the Completed Founders/Meadows Bridge.

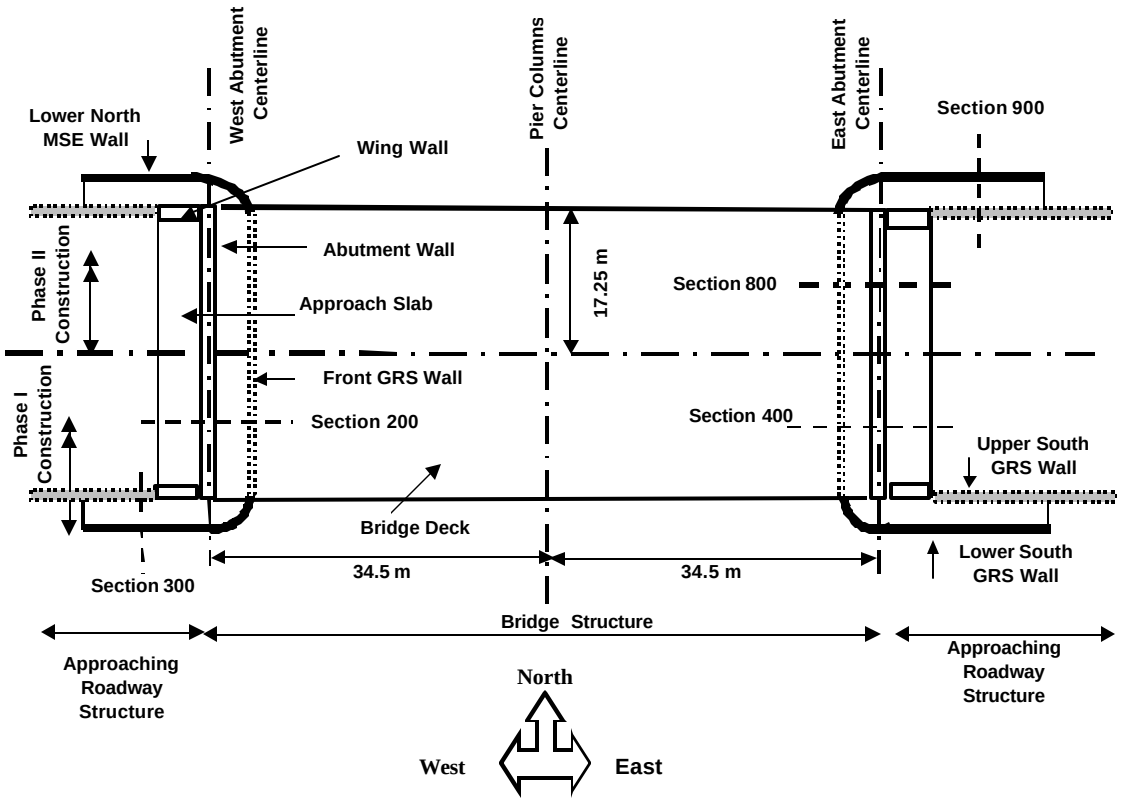


Figure 1.2 Plan View of the Completed Two-span Founders/Meadows Bridge and Approaching Roadway Structures.

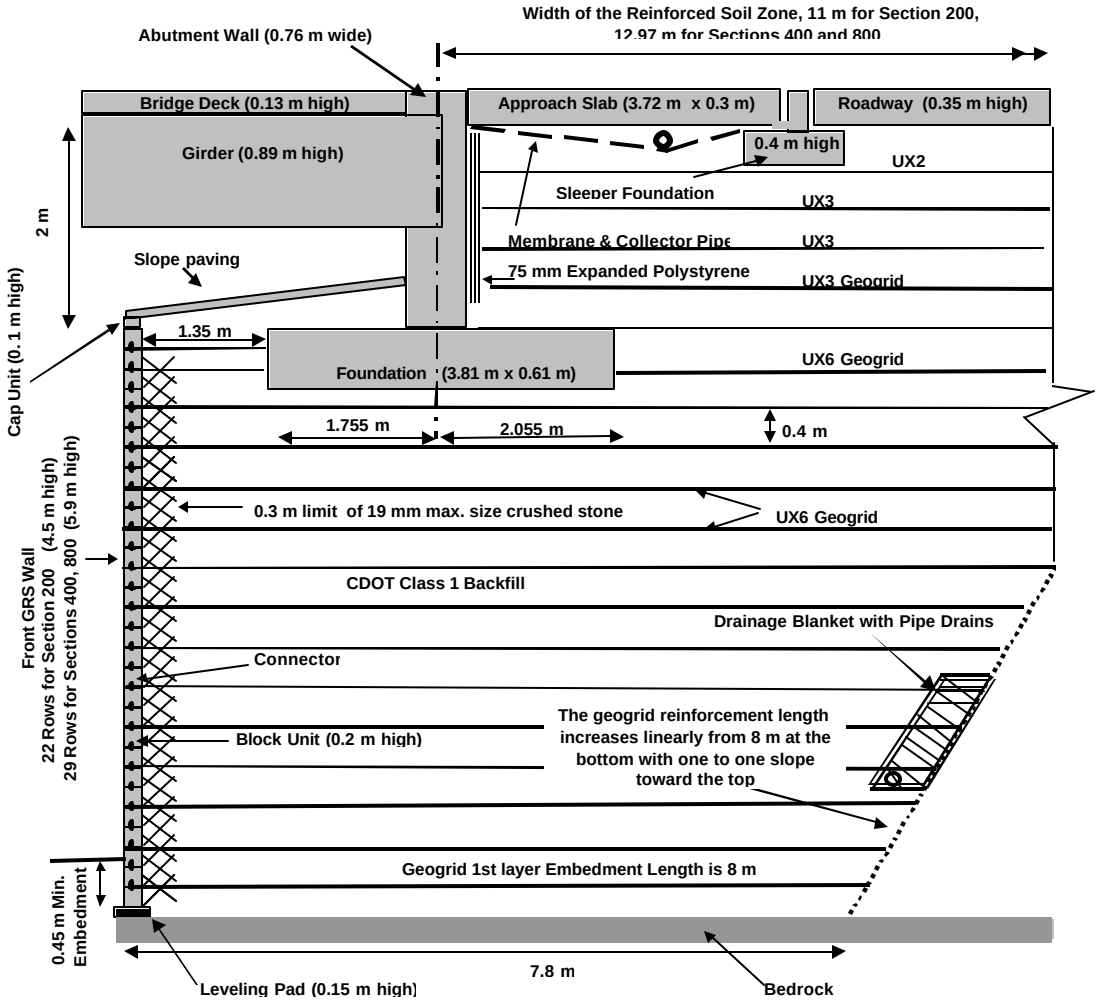


Figure 1.3. Typical Monitored Cross-Section (Sections 200, 400, and 800 in Figure 1.2) through the Front and Abutment GRS Walls.

1.2 Design of the Front GRS Wall

A comparatively long reinforced soil zone below the bridge and approaching roadway structure was considered (Figure 1.3) in order to address four design issues. First, integrating the roadway approach embankment and the bridge footing with an extended reinforced soil zone may alleviate the differential settlement problem. Second, enhancing the overall stability of the reinforced structure. Third, providing an additional margin of safety to alleviate concerns regarding a potential shear strength loss due to soaking of the claystone bedrock. Fourth, and in conjunction with the firm claystone foundation, significantly minimizing the settlement of the reinforced soil structure. The expected small settlements of the front GRS wall and the no scour potential made the design and construction of this structure possible.

The vertical earth pressure, $s_{v,}$ and lateral earth pressure, $s_{h,}$ employed for the calculation of the reinforcements maximum forces along the potential failure line at a depth z below the bridge footing (Elias and Christopher, 1997) are:

$$s_v = g\, z + Ds_v \qquad (1.1)$$

$$s_h = K_a\, s_v \qquad (1.2)$$

Where g is the backfill unit weight, K_a is the active earth pressure coefficient, and Ds_v is the stress increment induced within the soil mass by concentrated surcharge loads applied on the bridge footing. The value of K_a was estimated in the design as 0.31 for a backfill soil with friction angle of 34 degrees and a surcharge slope angle of 14 degrees (to account for backfill behind abutment wall). The maximum tensile force, $T_{max,}$ in each geogrid reinforcement layer (100% coverage) per unit width of wall was estimated as:

$$T_{max} = s_h\, S \qquad (1.3)$$

Where S is the reinforcement vertical spacing. For MSE walls supporting high surcharge load, Elias and Christopher (1997) recommended the use of Equations 1.2 to estimate the lateral earth pressures on the wall facing, s_{ho}, and Equation 1.3 to estimate the connection loads per unit width of wall, $T_{o,}$

$$\sigma_{ho} = K_a \sigma_v \quad (1.4)$$

$$T_o = \sigma_h S \quad (1.5)$$

1.3 Materials of the Front GRS Wall

The backfill soil used in this structure includes fractions of gravel (35%), sand (54.4%), and fine-grained soil (10.6%). The backfill met the material and construction (i.e., compaction level) requirements for CDOT Class 1 backfill (Abu-Hejleh et al., 2000). A friction angle of 34° and zero cohesion were assumed for the backfill material in the design of the front GRS walls. To evaluate the suitability of these design parameters for the gravelly backfill, conventional direct shear tests and large-size direct shear and triaxial tests were conducted. In the conventional tests, the 35% gravel portion was removed from the specimens per the test standards, but in the large-size triaxial and direct shear tests, the backfill soil specimens included the gravel portion. The results of conventional direct shear tests and large-size direct shear and triaxial tests indicate that assuming zero cohesion in the design procedure and removing the gravel portion from the test specimens lead to significant underestimation of the actual shear strength of the backfill. Hyperbolic model constitutive parameters were determined from the results of the large-size triaxial tests.

CDOT specifications imposed a global reduction factor of 5.82 to determine the long-term design strength (LTDS) of the geogrid reinforcements from their ultimate strength. This global reduction factor accounts for reinforcement tensile strength losses over the design life period due to creep, durability, and installation damage. It also includes a factor of safety to account for uncertainties. The Tensar Corporation manufactured the geogrid reinforcements. The long-term-design strength (LTDS) of the UX 6 geogrid reinforcements employed beneath the bridge footing (Figure 1.3) is 27 kN/m. Facing mechanical connectors between blocks layers and between blocks and reinforcements were employed in the front GRS wall (Figure 1.3). The measured connection strength for this system of connectors mobilized at a horizontal movement of 19 mm (service state), conducted in accordance with NCMA Test Method SRWU-1, was 57.7 kN/m. All of CDOT requirements for the geogrid reinforcements and connections were met (Abu-Hejleh et al., 2000).

1.4 Instrumentation and Monitoring Program

A number of reliable instruments and techniques were employed in this study to monitor the performance of the front GRS wall. Surveying was employed to determine the movements of the facing of the front GRS wall and settlement of the bridge footing. An inclinometer (Geokon Model 6000) was employed to measure lateral movement of the fill material behind the facing of the front GRS wall, both parallel and or perpendicular to the wall. Geokon Model 4800 pressure cells were used to measure the distribution of vertical earth pressure inside the front GRS wall. Geokon Model 4810 pressure cells were employed to measure profiles of lateral earth pressure against the rear facing of the front GRS wall. Geokon Model 4420 Crackmeters and Geokon Model 4050 strain gages were employed to measure distribution of geogrid strains inside the front GRS wall. Refer to Geokon manuals and Abu-Hejleh et al. (2000) for a detailed description of these gages and reduction of the gages data.

The instrumentation program was conducted in two phases: Phases I and II, which correspond, respectively, to the construction of the Phase I Structure (from July to December 1998) and Phase II Structure (from January to June 1999). Construction was implemented in two phases to accommodate traffic needs (Figure 1.2). Monitored Sections 200 and 400 are located at the center of the Phase I Structure and Section 800 (identical to section 400) is located at the center of the Phase II Structure (Figure 1.3). The layouts of the instrumented Sections 200, 400, and 800 are shown in Figures 1.4, 1.5, and 1.6, respectively. All these sections were instrumented with survey targets as shown in Figures 1.4, 1.5, and 1.6. As a pilot investigation, Section 400 of Phase I structure was additionally instrumented with two pressure cells, two crackmeters, and one inclinometer (Figures 1.4 and 1.5). Abu-Hejleh et al (2000) presented the results of these four gages. There were some concerns with geogrid strain results obtained from the crackmeters (Abu-Hejleh et al., 2000) and so their results are not presented in this report.

Along Phase II Structure, Section 800 also was heavily instrumented with pressure cells and strain gauges (Geokon 4050 gages) along four critical Location Lines: Location Line A close to the wall facing, Location Line B close to the centerline of the bridge abutment wall, Location Line C close to the back edge of the bridge footing, and Location Line D behind the bridge footing (see Figure 1.6). Table 1.1 lists the location of these gages, where Y is the height of the

gage above the leveling pad, and X is the distance of the gage from the rear side of the block facing. The gages in Table 1.1 are designated through one or two digits and two or three letters (e.g., 10VBN, 11HN, and 6SBN). The first digit indicates the number of the closest geogrid layer to the gage (Figure 1.6). The first letter indicates the gage type: V= pressure cell to measure vertical pressure, H= pressure cell to measure lateral earth pressure, and S= Strain gage. The 2nd letter indicates the closest Location Line to the gage. The third letter (optional, N or S) is used when two gages are placed at the same location, one north of the control section (N) and one south of the control section (S). Two gages were placed at the same location to check on the consistency of the measured data and the suitability of assuming plain strain conditions. The 1st (reference) readings for each gage and the date and fill height when these reading were collected are also summarized in Table 1.1. For most of the gages, reference readings were collected when there was no backfill over the gage (i.e., gage location in Table 1.1 matches the fill height when the 1st reading was collected).

The collected performance data is organized according to the loading sequence (Figures 1.4 and 1.5) as follows:

Stage I. Construction of the front GRS wall up to the bridge footing elevation. The Stage I structure provides support for the bridge and approaching roadway structures.

Stage II. Placement of the bridge footing and girders seat.

Stage III. Placement of girders.

Stage IV. Placement of the reinforced backfill behind the abutment wall from the bridge footing elevation to the bottom of the sleeper footing.

Stage V. Placement of the of bridge deck.

Stage VI. Placement of the approaching roadway structure (including approach slab) and other minor structures. By the end of this stage, the total average vertical pressure exerted directly underneath the bridge footing was estimated as 115 kPa. This stage was completed on December 16, 1998 for the Phase I Structure and on June 30, 1999 for the Phase II Structure.

Stage VII. Post-Construction Stage until June 2000 (lasted 1 year for the Phase I Structure and 18 months for the Phase II Structure). The total vertical contact pressure exerted directly underneath the bridge footing during this stage was estimated to be 150 kPa.

Table 1.2 shows the start and completion date of each stage along Sections 200 and 400 (Phase I) and Section 800 (Phase II). Note that Stage IV occurred before Stage III on the Phase I Structure. The estimated average applied vertical earth pressures underneath the bridge footing and on the base of the reinforced fill (7.5 m wide) along Sections 400 and 800 are summarized in Table 1.3. Data from Section 800 gages were collected automatically every 6 minutes during Stage I and every half hour during Construction Stages II to VI. After opening the structure to traffic (Stage VII), data was collected every 10 minutes over the first two months and every 36 hours after that. Data for all gages were lost from days 320 to 355 and from days 439 to 483 (days counted from January 1, 1999). All the instrumentation results collected until the end of Stage VII (June 2000) are presented and discussed in this report. Subsequent publications will summarize the results from long-term monitoring results obtained after June 2000.

Table 1.1 Location and Reference Readings for Gages Placed along Section 800.

Gage #	Data logger Channel	Gage Location		Information for Reference Readings			
		X	Y	Date of Collection (# of Days from Jan. 1, 1999)	Height of Fill (m)	Gauge Reading	Temp., C
Pressure Cells to Measure Vertical Earth Pressure							
0VA	3	1.00	-0.15	23	-0.15	9644	0.2
0VD	5	7.50	-0.15	23	-0.15	8888	-0.1
3VA	11	0.60	1.52	28	1.52	8976	11.5
6VA	16	0.60	2.33	36	2.33	8988	14.4
6VBN	17	3.10	2.33	36	2.33	9026	17.2
6VBS	18	3.10	2.33	36	2.33	8995	21.1
6VC	19	5.50	2.33	36	2.33	8952	18.3
6VD	20	7.50	2.33	36	2.33	8420	17.8
10VA	28	0.60	3.95	49	3.95	9190.8	-0.9
10VBN	29	3.30	3.95	49	3.95	8787	-2.7
10VBS	30	3.30	3.95	49	3.95	8753	-0.2
10VC	31	5.00	3.95	49	3.95	9362	-0.5
10VD	32	7.60	3.95	49	3.95	8507	-0.7
12VD	41	7.30	4.95	51	5.28	9326	0.7
13VA	42	1.85	5.05	55	5.28	9090	2
13VB	43	3.25	5.05	55	5.28	9046	2.9
13VC	44	4.65	5.05	55	5.28	8889	3.7
Pressure Cells to Measure Lateral Earth Pressure on the Facing							
7H	21	0	4.27	46	4.27	9171	3.43
9H	22	0	3.75	48	3.75	9558	9.70
11HN	33	0	4.57	50	4.57	9101	10.8
11HS	34	0	4.57	50	4.57	9269	10.3
12H	40	0	4.97	55	5.28	8615	4.42
Strain Gages to Measure Lateral Geogrid Strain							
2SA	7	0.39	0.6	23	0.9	7076	3.1
2SB	8	3.6	0.6	23	0.9	7373	3.3
2SC	9	5.8	0.6	23	0.9	8122	3.3
6SA	12	0.38	2.23	35	2.23	4120	20.2
6SBN	13	3.35	2.23	35	2.23	4560	12.3
6SBS	14	3.35	2.23	35	2.23	3328	17.2
6SC	15	5.5	2.23	35	2.23	3923	14.6
10SA	23	0.33	3.85	50	3.85	2741	4.9
10SB	25	3.25	3.85	50	3.85	3080	4
10SC	26	5	3.85	50	3.85	3640	4
10SD	27	7.6	3.85	50	3.85	3576	3.9
12SB	36	3.17	4.67	51	4.67	2845	10.2
12SD	38	7.43	4.67	51	4.67	3375	8.3

Table 1.2. Time Progress of the Monitored Construction and Post-Construction Stages.

Monitored Stages	Phase I Structure		Phase II Structure, Section 800	
	Section 200 Date	Section 400 Date	Starting Date	# Days from Jan. 1, '99
Leveling Pad	7/16/98	8/15/98	1/19/99	19
Stage I Construction	8/ 15/98	9/12/98	2/ 24/99	19-55
Stage II Construction	9/12/98	9/26/98	3/8/99	55-67
Stage III Construction	10/6/98	10/12/98	3/10/99	67-69
Stage IV Construction	9/19/98	10/3/98	3/26/99	69-85
Stage V Construction	11/25/98		5/25/99	85-125
Stage VI Construction	12/15/98		6/29/99	125-179
Post-Construction Stage (VII)	12/16/98		6/30/99	180-545

Table 1.3. Estimated Average Applied Vertical Earth Pressure Beneath the Bridge Footing and on the Base of the Reinforced Fill along Sections 400 and 800.

	Estimated Average Applied Vertical Earth Pressure (kPa)	
	Bridge Footing	Base of the Reinforced Fill
Stage I Construction	0	117
Stage II Construction	22	134
Stage III Construction	64	155
Stage IV Construction	84	180
Stage V Construction	101	189
Stage VI Construction	115	199
Stages VII and VIII (Bridge in service)	150	223

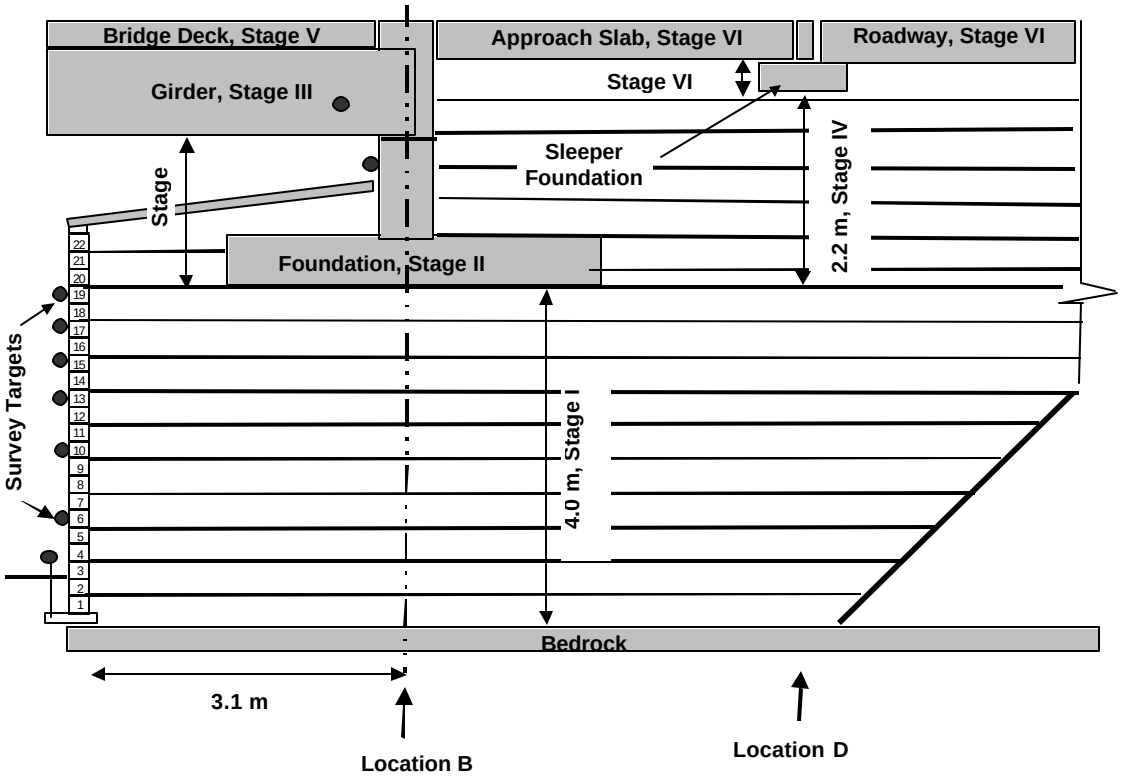


Figure 1. 4 Instrumented Section 200 Indicating Construction Stages.

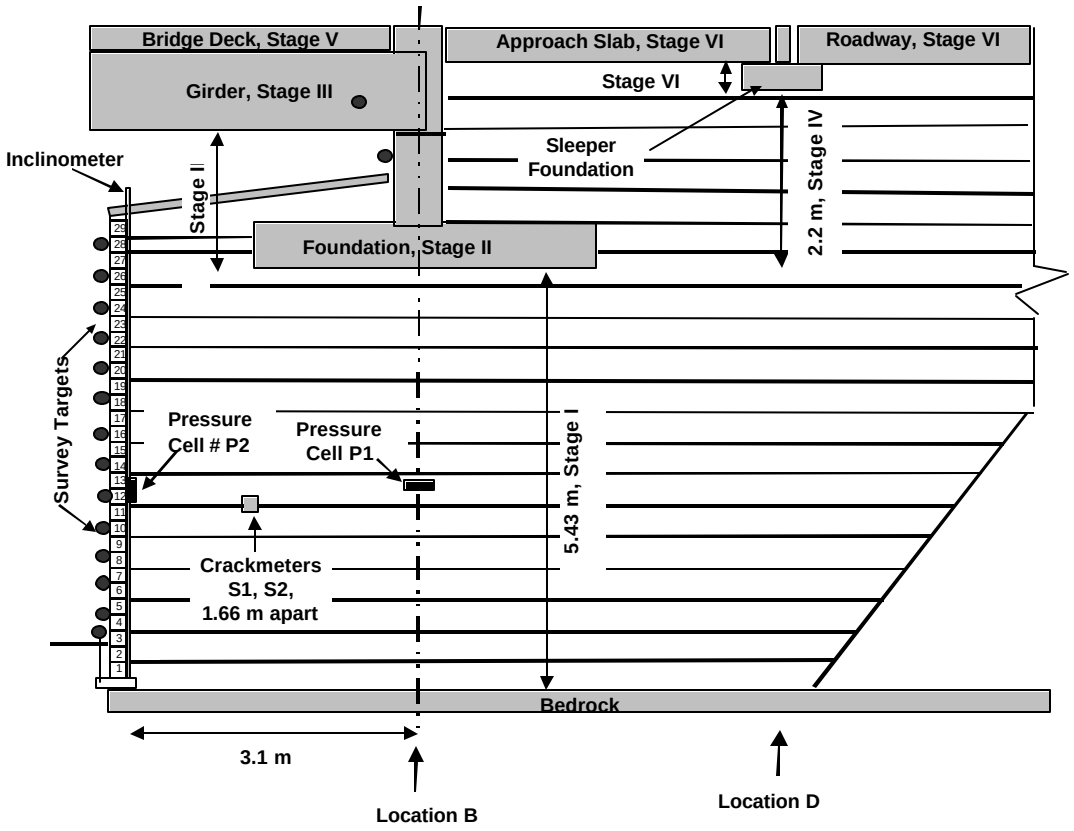


Figure 1.5. Instrumented Section 400 Indicating Construction Stages.

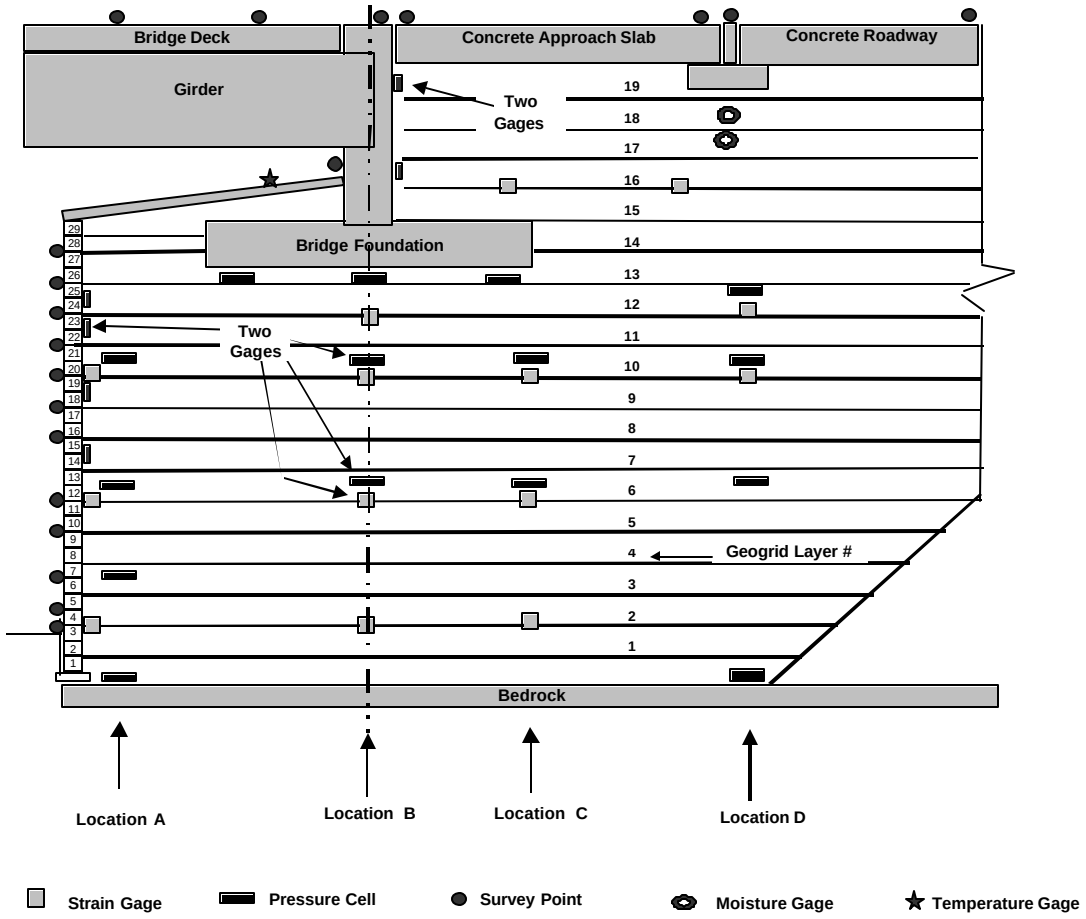


Figure 1.6 Layout of Instrumented Section 800.

1.5. Summary of Relevant Findings from Previous Research Studies

This section will present findings of previous research studies referred to in the discussion of the results presented in this report.

1.5.1 Use of GRS Wall Structures as Bridge Support

A recently published FHWA report (FHWA, 2000) describes three studies on GRS bridge supporting structures: load test of the Turner-Fairbank pier, load test of the Havana Yard piers and abutment in Denver, Colorado, and pre-loading of the Black Hawk abutment. Findings and conclusions of each study were presented. The findings of these studies that relevant to the study presented herein on the Founders/Meadows GRS walls are as follows:

- GRS abutments constructed using closely spaced (0.2 m to 0.3 m) and sufficiently stiff geosynthetic reinforcements, well-compacted granular backfill, and strong blocks will experience satisfactory performance under an average surcharge pressure of 200 kPa. The long-term creep deformations under service load of such GRS abutment structures were negligible. Small vertical reinforcement spacing and good compaction of the granular backfill significantly contribute to the satisfactory performance of the GRS bridge supporting structures.
- The use of concrete blocks as facing without mechanical connections between blocks results in satisfactory performance of the structures under service loads (i.e., the requirements for the connection strength could be waived). In this case, reinforcement layers were extended between block layers and the hollow concrete blocks were filled with uniform size gravel. The facing system developed its connection capacity by interface friction between blocks and reinforcement and between reinforcements and gravel.
- GRS abutments are clearly viable and adequate alternatives to bridge abutments supported by deep foundations or by metallic reinforced soil abutments.

Although the performance of the Turner Fairbank GRS pier and the Havana GRS abutment were excellent, this was not the case for the loaded Havana GRS pier. The GRS pier loaded in Havana Yard in Denver was a relatively slender structure when compared to the Turner Fairbank GRS pier (Abu-Hejleh et al., 2001). Four to five months after the Havana pier was loaded, excessive

movements of the top several block layers and severe cracking of the block facing were noticed. Therefore, it was decided to remove the applied surcharge load, dismantle the large-scale GRS abutment and pier structures, and perform a forensic investigation and a facing connection stability analysis. Abu-Hejleh et al. (2001) summarized the results of this forensic and stability investigations and identified possible causes for the excessive deformation and cracking experienced by the loaded GRS pier structure.

Abu-Hejleh et al. (2001) concluded that the friction-based connection strength for the facing of the Havana loaded pier was adequate for levels deeper than 1.6 m from the level of surcharge load. For future construction of piers and abutments with a high surcharge load, Abu-Hejleh et al. (2001) recommended the following construction measures to prevent the excessive facing movements in the upper 1.6 m of the structure: 1) place reinforcements with a wrapped-around procedure behind the facing, 2) attach the block facing to the reinforced soil mass through placement of tail reinforcements between block layers that extend 1 meter into the reinforced soil mass, and 3) dowel and grout with cement the block layers (in lieu of gravel filled block cells).

1.5.2 Influence of Temperature and Seasonal Changes on GRS Walls

Buttry et al. (1996) reported the influence of temperature and seasonal changes on the response of a 3.5 m high GRS segmental retaining wall constructed at the University of Wisconsin. This wall was instrumented to measure movements, earth pressures, forces between segmental units, temperatures and strains in the geogrid reinforcements. Monitoring was done during construction (October to December of 1993) and after construction. The observed behavior of the wall can be divided into two stages:

- From October of 1993 until March of 1994. The measured normal forces acting on the facing unit were about twice the weight of units above the load cell by the end of construction (December 1994) and remained until March of 1994. The difference was attributed to the downward frictional forces applied by the backfill soil to the rear of the block facing units. The geogrid strains increased during construction but leveled out and remained constant or increased slowly until April 1994.

- From March to October of 1994. Starting March of 1994, the normal forces began to decrease until August 1994 when the normal forces measurements approached the weight of the units. In April of 1994, the geogrid strains readings increased significantly at all locations and continued increasing through the summer months and into October 1994. Also during this time period, the wall facing moved outward approximately 2 mm. One explanation for these observations (Buttry et al., 1996) is that the wall system was rigid during the winter season and began to move, settle, and adjust during the spring thaw, when any ice in the backfill was melting and the soil may have been temporarily wet. As the wall moved and reached a more stable form, the downward frictional forces applied by the backfill soil to the rear of the block facing units dissipated, thus reducing the normal forces between facing units.

The front GRS wall of the Founders/Meadows structure along Section 800 also experienced a response that seems to be related to the seasonal changes of moisture and temperature (presented in Chapters 2, 3, 4, 5). Construction Stages I, II, III, and IV of section 800 occurred during the winter season and Construction Stages V and VI occurred during the spring season.

1.5.3 Strains and Stresses in GRS Wall Structures

At the at rest conditions (i.e., the lateral soil strain is zero) the lateral earth pressure is at its highest level. For this case, the ratio of lateral to vertical earth pressure, K_o, for a soil with friction angle of 34 degrees (assumed for the soil of the front wall) was estimated as 0.44. At the active limit state, the soil mass stretches in the lateral direction and the lateral tensile soil strain reaches the tensile strain at failure. This fully mobilizes the friction resistance of the backfill, thus reducing the horizontal lateral earth pressure to its lowest level. For this case, the ratio of lateral to vertical earth pressure for the soil of the front GRS wall was estimated as 0.31 (see section 1.2).

In a reinforced soil system, the measured reinforcement strains and forces give insight into changes of the lateral soil strains and stresses (see McGown et al., 1998). A common assumption made for geosynthetic reinforcement, due to their high level of bond, is that the tensile strains in the soil and reinforcement are equal in the direction of the reinforcement (Jewel, 1985). This

means that the geogrid reinforcement and the surrounding soil, both initially resting at zero strains, will expand to the same level of lateral tensile strains. When a soil element tends to expand in the lateral direction, it is resisted by both the reinforcement and the adjacent soil. At rest conditions (zero lateral soil strain), the lateral boundary of the soil element is fully confined by the adjacent soil elements. In this case, upper limit of lateral earth pressure is applied on the reinforced soil system, fully resisted by soil, and zero tensile force is mobilized by reinforcement. This is approximately the case for soil elements located far away from the wall facing (e.g., along Location Line D) where very small geogrid strains are measured. When the soil element stretches in the lateral direction, some of the reduced lateral (compression) earth pressure will be carried/supported by the reinforcement tensile stresses. If the soil yields sufficiently in the lateral direction, it is possible that the reinforcement tensile forces will balance all the lateral earth pressure. This provides the lower limit case for lateral earth pressure supported by adjacent soil element (theoretically zero) and the upper limit case for the reinforcement tensile force.

According to McGown et al. (1998), two limiting cases must be considered for lateral earth pressure on the facing of MSE walls. First, if the lateral boundary of the wall is not allowed to yield, the resulting pressure will be equal to or greater than those obtained under at-rest conditions. This case produces the upper limit of the lateral earth pressure on the wall. Secondly, if the lateral boundary is allowed to yield sufficiently to mobilize large tensile resistance in the reinforcement, and if the required forces and available forces balance, theoretically there will be no lateral earth pressure acting on the wall. This provides the lower limit case. However, even for this lower limit case, the soil masses between the reinforcing layers may have a tendency to produce localized stresses near the facing. These develop because each soil layer between reinforcements tends to act separately, causing the wall to be subjected to active horizontal pressure over the depth of that layer. For the front GRS wall of the Founders/Meadows structure, this lower limit earth pressure is expected to be smaller than 3 kPa.

Ingold (1979) investigated the effects of compaction on soil backfill behind rigid retaining walls. It was found that due to compaction stresses, the assumption of at-rest lateral earth pressures at the back of rigid retaining walls may be an underestimation of the actual horizontal stresses

within the soil mass. Compaction induces significant lateral earth pressures that are locked–in after removal of the vertical compaction stresses. Thus, due to compaction, ratios between horizontal to vertical earth pressures exceed those at the at-rest condition (larger than 0.44 in our case). Ingold (1979) developed an analytical model to estimate lateral earth pressure caused by compaction at the back of rigid retaining walls.

In GRS Walls, compaction also induces locked-in strains in the reinforcements after removal of the compaction loads. This is called *Static Interlock* based on the soil particles being locked into the geogrid due to static load (McGown et. al., 1998). Another mechanism called *Dynamic Interlock* (McGown et al., 1998) is also available and could be developed by repeated compaction loading in soil masses containing geogrid reinforcements with integral junctions (as used in the GRS walls of the Founders/Meadows structure). During compaction of a backfill soil containing a geogrid with integral junction, the compaction load forces soil particles into the apertures of the grids. When the compaction load is released, the grid attempts to return to its initial conditions, but is resisted by the particles within apertures. These locked-in strains have an effect similar to a confining stress on the soil (i.e., they increase the strength of the soil and reduce the lateral earth pressure).

The research conducted by Andraws and Yogarajah (1994) indicated that for stiff connection between the reinforcement elements and the facing, the tensile strain distribution was linear with maximum strains occurring close to the facing. For flexible connection, the maximum tensile strain occurred far from the facing. For the later case, larger shear resistance was mobilized in the soil and this resulted in reduced lateral earth pressure on the facing units.

2. MOVEMENTS OF THE FRONT GRS WALL STRUCTURE

2.1 Introduction

Direct measurements of the movements of the facing of the front GRS wall and settlement of the bridge footings were obtained from surveying and an inclinometer. Indirect measurements for the outward facing displacement of the front wall were obtained from the results of strain gages at geogrid layers 6 and 10 of Section 800 (Figure 1.6). Average geogrid strains at layers 6 and 10 of Section 800 were calculated using the geogrid strain values measured along Location Lines A, B, C, and D (Figure 1.6). The geogrid outward displacements at the facing were obtained by integrating the geogrid strains measured along each layer, and assuming that the retained backfill did not move and that there was no reinforcement slippage. The displacements calculated in this way were then compared to direct measurements of the wall outward displacements from surveying and the inclinometer. Complete details of the geogrid strain measurements are presented in Chapter 5.

Instrumented sections 200 and 400 are located at the center of Phase I Structure and Section 800 is located at the center of Phase II Structure (see Figures 2.2 to 2.6). The height of the front GRS wall (i.e., elevation above leveling pad) is 5.9 m for Sections 400 and 800 (identical), and 4.5 m for section 200. The bridge footing is located 5.28 m above leveling pad for Sections 400 and 800 and 3.86 m above the leveling pad for Section 200. The collected displacement data is organized according to the loading sequence, as follows: construction of the front GRS wall (Stage I), placement of the bridge superstructure (Stages II to VI), and post-construction stage (VII). Movements induced during wall construction (Stage I) can be compensated during wall construction (i.e., before placement of the bridge superstructure).

2.2 Facing Outward Displacements during Wall Construction (Stage I)

Monitoring data on the outward displacements of the wall induced during construction of the front GRS wall is summarized in Figure 2.1. The figure shows the outward wall displacements measured by surveying Section 400 along the lower 14 facing block layers (up to 2.75 m above leveling pad). These displacements resulted from construction of the front GRS wall from elevation 3.65 to 5.5 m. Figure 2.1

also shows the outward wall displacements measured by surveying Section 800 along the lower 10 facing block layers (up to 2.0 m above the leveling pad). These displacements resulted from construction of the front GRS wall from elevation 2.44 to 5.5 m. Estimated outward wall displacements along Section 800 inferred from strain gage results collected along geogrid layers 6 and 10 are shown in Figure 2.1. The inferred outward displacement obtained for geogrid layer 6 resulted from construction of the front GRS wall from elevation 2.23 m to 5.28 m; while the inferred outward displacement obtained for geogrid layer 10 resulted from construction of the GRS wall from elevation 3.85 m to 5.28 m. It is important to note that the sets of movement data shown in Figure 2.1 were not collected during construction of the same reinforcement lifts and, consequently, a direct comparison is not possible. Nevertheless, they show a consistent trend and provide an order of magnitude of the expected outward displacements during construction of GRS walls. The maximum outward wall displacements measured during construction of the front GRS wall of sections 400 and 800 were 8.5 mm, and 11.5 mm, respectively.

Additional insight on the characteristics of the outward wall displacements can be gained from the strain gage measurements collected during construction of the walls. Figure 2.2 shows the estimated average geogrid lateral strains and the outward displacements of the front wall facing obtained along geogrid layers 6 and 8 of Section 800. The information is presented as a function of the estimated vertical earth pressure applied on these layers (Equation 1.1) during all construction stages. Construction of the front wall itself (before placement of the bridge structure) corresponds to the first three data points shown in Figures 2.2a and 2.2b.

Figure 2.2a shows a good agreement between the average geogrid strains at different depths when the strain values are plotted as a function of the applied vertical earth pressures. However, for the same level of applied vertical earth pressure, Figure 2.2b shows that the wall outward displacements for layer 10 are higher than for layer 6. This is an expected behavior because the width of the active zone (defined by the locus of maximum tension line) increases with the height above the leveling pad. The second data point in Figures 2.2a and 2.2b was collected after compaction and placement of approximately 1 m of backfill (corresponding to approximately 20 kPa of vertical earth pressure) over

the gages. Strain gage monitoring results shown in Figure 2.2 indicate that, in spite of the surcharge loads due to the bridge superstructure, the largest components of wall outward displacements occurred during placement and compaction of a few lifts of soil above the geogrid layers (i.e., approximately 2 m of soil or 40 kPa).

2.3 Facing Outward Displacements Induced by Placement of Bridge Superstructure (Stages II to VI)

Monitoring data on the outward wall displacements induced during placement of the bridge superstructure is summarized in Figure 2.3. The data was obtained from surveying and strain gage records. As observed in the figure, the maximum wall outward displacements experienced along sections 200, 400, and 800 during placement of the bridge were approximately 7, 9, and 10 mm, respectively. The maximum outward displacements occurred within the upper third of the wall, directly below the bridge footing. In spite of the different height of the three sections (Section 200 is 4.5 m high, while Sections 400 and 800 are 5.9 m high), all three sections show a similar pattern. A maximum outward displacement of approximately 9 mm was induced by placement of the bridge superstructure (Figure 2.3). Although the outward displacements obtained for Sections 400 and 800 are of the same order of magnitude (these sections have identical configuration), it can be observed that displacements induced in Section 800 are somehow higher. Possible explanations for the difference in outward displacements between these two sections are:

- *Different construction season.* Most of the Phase I Structure (Section 400) was constructed during a warm season while the front GRS wall of Phase II Structure (Section 800) was constructed during a cold season (Table 1.1). Placement of the bridge superstructure along Section 800 occurred mostly in March and April of 1999 when thawing and wet seasons started (Table 1.1). This may have led to softening of the backfill and comparatively larger deformations.

- *Different construction sequence*. The backfill behind the abutment wall was placed before placement of the girders during construction of Section 400. Instead, the girders were placed before

placing backfill behind the abutment wall during construction of Section 800 (Table 1.1). This induced, most probably, larger lateral displacements and reinforcement strains within the GRS backfill along Section 800.

Additional insight on the outward movements induced by placement of the bridge superstructure can also be gained from assessment of the strain gage measurements shown in Figure 2.2. The label shown next to each data point in Figure 2.2 indicates the construction stage to which the data point corresponds (4^{th} data point corresponds to Stage II, 5^{th} corresponds to Stage III and so on). During Stages II to VI, the GRS system responded with comparatively small deformations to the increasing vertical earth pressures. A possible reason for this behavior is the influence of compaction experienced in the previous stage (Stage I). An additional potential justification is the fact that Construction Stages II to IV took place during the winter season. Buttry et al. (1996) reported a comparatively more rigid behavior for a GRS structure during the winter season (Section 1.5.2). During Stages V and VI (last three data points in Figures 2.2a and 2.2b), the GRS system appears to have responded with comparatively large strains and displacements to the increasing level of applied vertical earth pressures. Thawing and wetting of the backfill, as well as disappearance of the compaction influence, may have led to softening of the backfill in these stages (see Section 1.5.2).

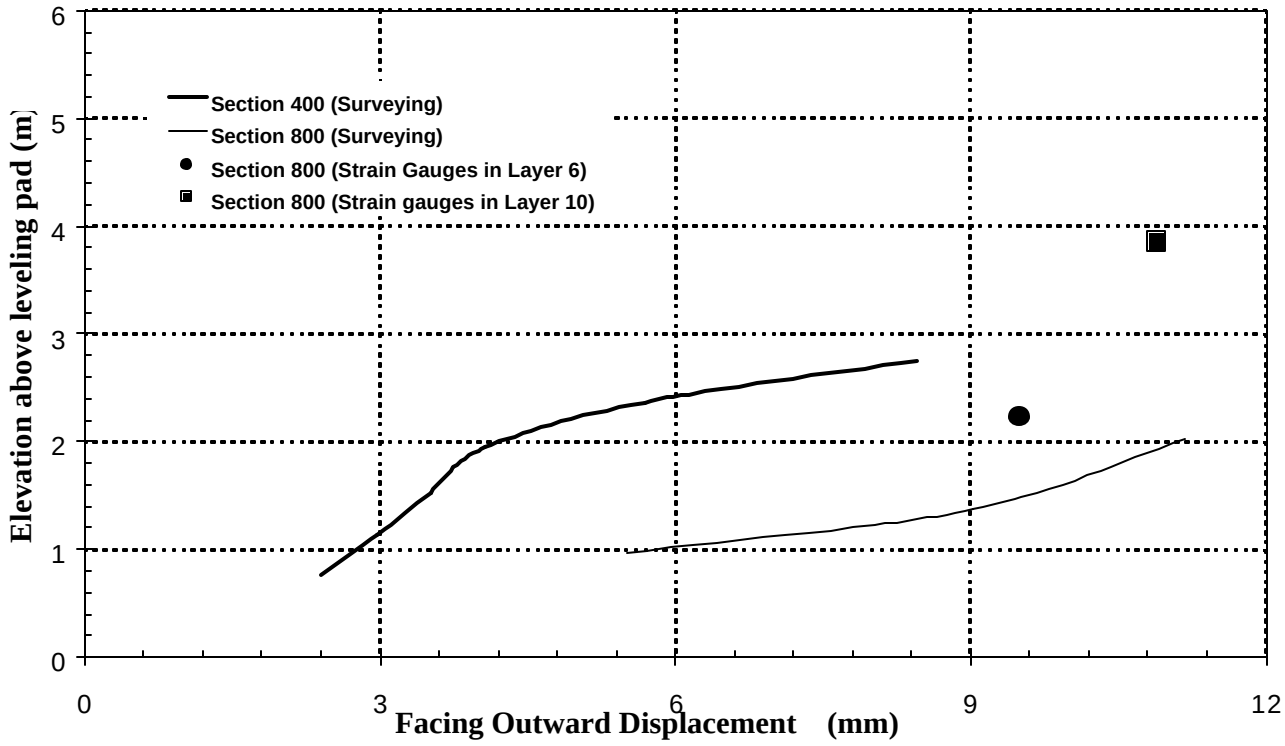


Figure 2.1. Measured Outward Displacements of the Front Wall Facing Induced during Construction of the Front GRS Wall. Note: sets of data correspond to different loading conditions.

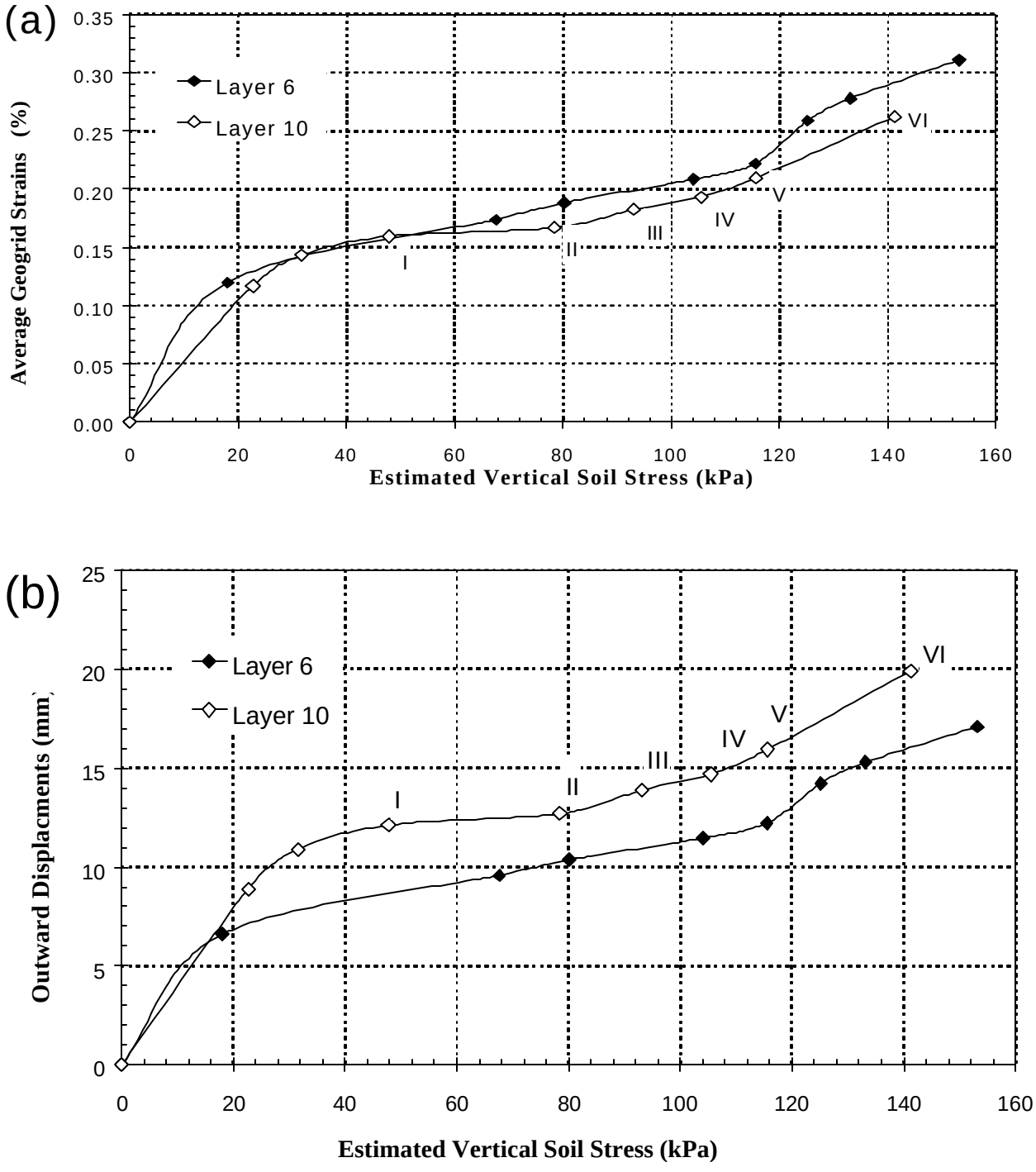


Figure 2.2. Strain Gage Results from Geogrid Layers 6 and 8 of Section 800 during all Construction Stages: a) Average Geogrid Strain; b) Estimated Outward Displacements at the Facing. Note: The label shown next to each data point indicates the construction Stage (I to VI) to which the data point corresponds (shown for Layer 10 data only).

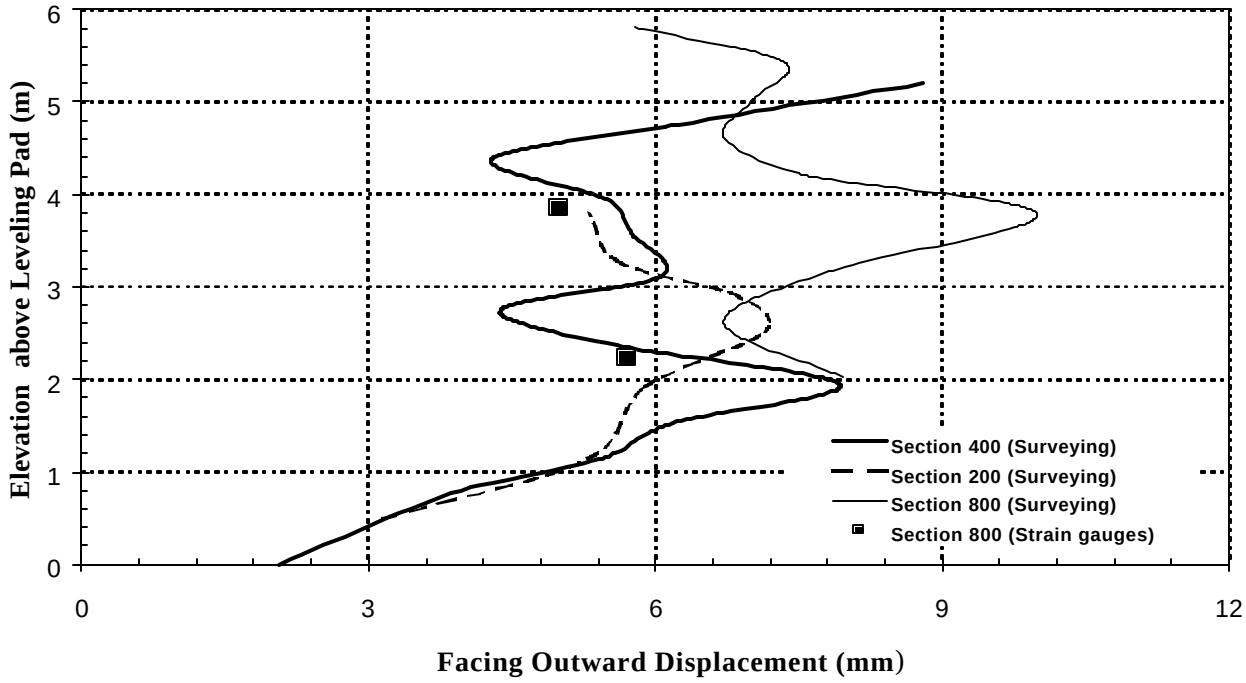


Figure 2.3. Measured Outward Displacements of the Front Wall Facing Induced by Placement of the Bridge Superstructure.

2.4 Facing Outward Displacements Induced while Bridge in Service

Monitoring Stage VII, which corresponds to the movements induced after opening the bridge to traffic, covers the period from December 1998 to June 2000 for the Phase I Structure (Sections 200 and 400) and from June 1999 to June 2000 for Section 800 of the Phase II Structure (see Table 1.1). The outward displacements of the front wall measured from surveying during Stage VII along Sections 200 and 800 are shown in Figures 2.4a and 2.4b, respectively. Figure 2.4c shows the outward displacement profile along Section 400 measured from the inclinometer at different times during Stage VII. The data obtained from the inclinometer along Section 400 represent the movements of the wall relative to the leveling pad (i.e., not the total "absolute" movements of the wall). However, since the outward displacements of the leveling pad during placement of the bridge superstructure were comparatively small (see Figure 2.3), leveling pad displacements were neglected in the interpretation of inclinometer data. Surveying results for Section 400 from December 1998 to June 1999 are not presented because

the surveying control point used in Section 400 was relocated during this period. Surveying results for Section 400 displacements from June 1999 to January 2000 and from June 1999 to June of 2000 are shown in Figure 2.4d. Figure 2.4d indicates good correlation between the surveying and inclinometer movement results obtained over similar monitoring periods.

The data shown in Figure 2.4 suggests that the outward wall displacements induced while the bridge was in service tended to decrease toward the leveling pad. The maximum outward wall displacement seemed to occur directly below the level of the bridge footing. The results in Figures 2.4 indicate that the structures continued to move after opening the bridge to traffic (Monitoring Stage VII). Although movements were not negligible, these displacements were relatively small for Sections 200 (Figure 2.4a) and 800 (figure 2.4b) and comparatively larger for Section 400 (Figures 2.4c and 24.d). Possible causes for these post-construction movements are traffic load, creep, and seasonal and temperature changes.

Figure 2.5 shows the time history of some of the collected geogrid strain measurements. The strain data shown in the figure was collected from strain gages placed in Section 800 during one year after opening the structure to traffic (i.e., June 1999 to June 2000 or days 180 to 545 from Jan. 1, 1999). Excellent agreement can be observed between the displacements inferred from strain gages placed in geogrid layers 6 and 10 and the displacements obtained from surveying (see Figure 2.4b). Geogrid strain results shown in Figure 2.5 support observations made regarding the outward displacements shown in Figure 2.4, which showed relatively small (though not negligible) post-construction displacements from June 1999 to January 2000, and negligible post-construction movements from January to June 2000. As shown in Figure 2.5, the geogrid strains measured from January to June 2000 (approximately days 365 to 540 from Jan. 1, 1999) leveled out and remained approximately constant.

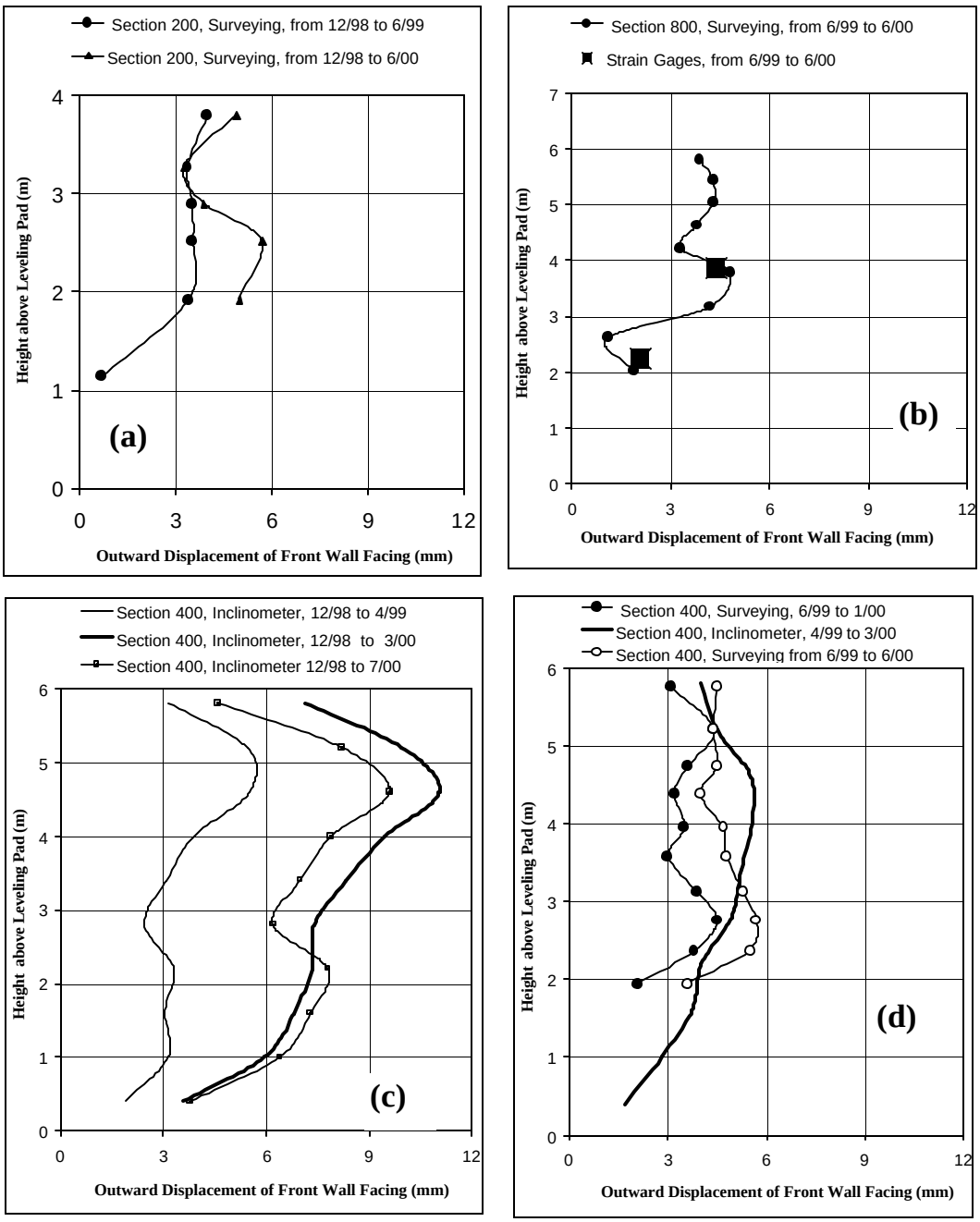


Figure 2.4. Measured Outward Displacements of the Front Wall Facing Induced while the Structure was in Service: (a) Section 200 from Surveying; (b) Section 800 from Surveying and Strain Gages; (c) Section 400 from the Inclinometer; (d) Section 400 from the Inclinometer and Surveying.

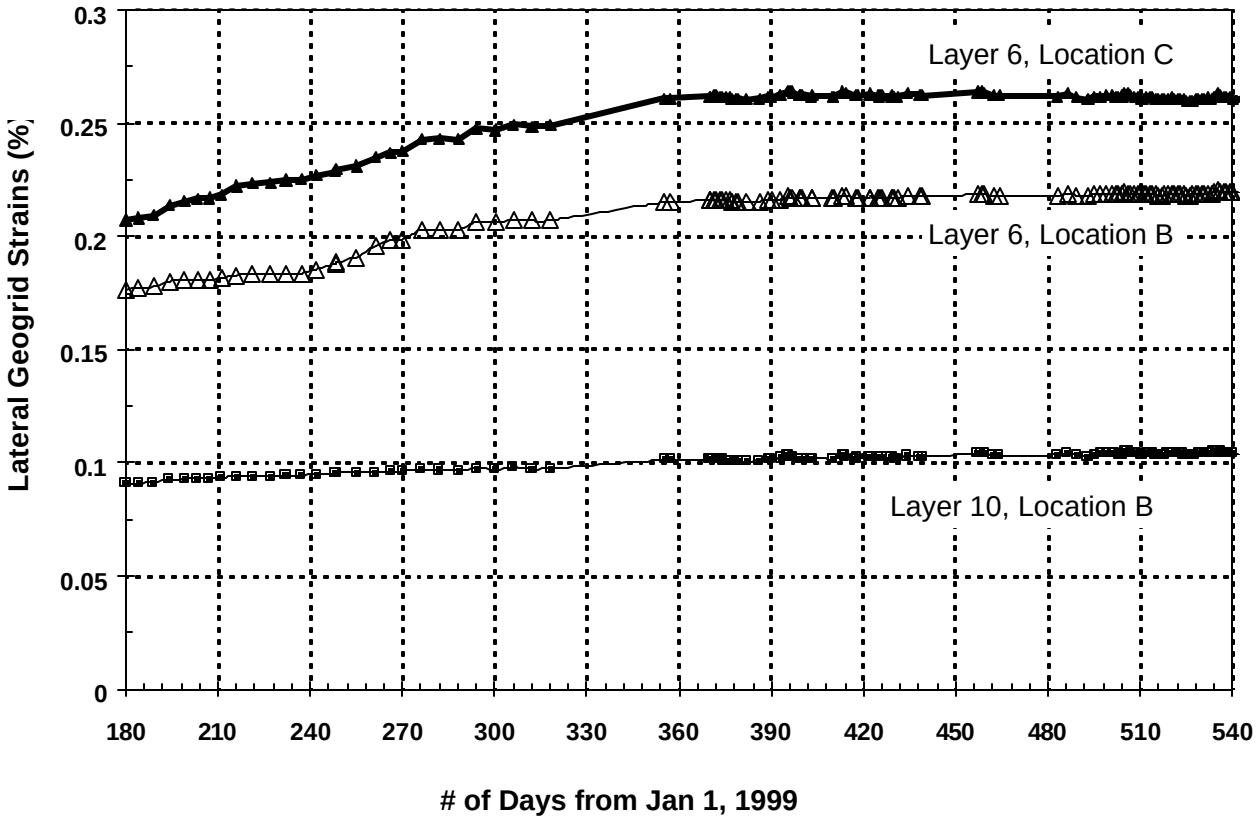


Figure 2.5. Geogrid Strain Gage Results Obtained along Section 800 below the Bridge Footing while Bridge was in Service (Stage VII). Note: The period shown in the horizontal axis ranges from June 1999 (180 days from Jan 1 1999) to June 2000 (540 days from January 1, 1999).

2.5 Settlement of the Bridge Footing

The maximum measured vertical settlements of the bridge footing, due to the placement of the bridge superstructure (Stages II to VI), were 13 and 12 mm on Sections 200 and 800, respectively. The maximum measured vertical settlements of the bridge footing induced while the bridge was in service (Stage VII) on Sections 200, 400, and 800 were 7, 11, and 10 mm respectively. The post-construction settlements of the bridge footing of the Phase I Structure on Section 400 took place during the first year in service (December 1998 to January 2000), but became negligible from approximately January to

June 2000. This correlates very well with the measured negligible front wall outward displacements along Section 400 during the same period of time (Figures 2.4c and 2.4d).

2.6 Summary and Discussion of the Front GRS Structure Movements

Table 2.1 summarizes the maximum recorded outward displacements and vertical settlements of the front GRS wall facing along Sections 200, 400 and 800, obtained from surveying, inclinometer, and strain gages. Surveying results for the bridge footing settlements along sections 200, 400, and 800 are also summarized in Table 2.1. The movements shown in the table were induced during construction of the front GRS wall (Stage I), during placement of the bridge superstructure (Stages II to VI), and during different periods after opening the structure to traffic (Stage VII). Although not shown in the table, the component of the facing displacements parallel to the wall, measured from surveying during all stages, was also collected. These displacements were essentially negligible. The more sensitive inclinometer results along Section 400 also indicated negligible displacements parallel to the wall. These findings support the assumption of a plain strain condition at the middle of the Phase I and Phase II Structures. The vertical settlement of the wall facing was approximately the same at different heights. This indicates that most of the wall vertical settlements were due to the settlement of the leveling pad and compression of the joint materials located between the leveling pad and first row of facing blocks.

The maximum displacement values obtained from all monitoring techniques and along all monitored sections during different stages are also shown in Table 2.1. These movements are normalized with respect to the wall design height. Careful analysis of the summary in Table 2.1 and Figure 2.2 leads to the following observations regarding the overall deformation response of the front GRS wall:

- Movements induced during construction of the front GRS wall. The wall experienced comparatively large movements and geogrid strains during this stage. This was attributed primarily to the influence of compaction that created large locked-in lateral geogrid and soil strains after removal of the compaction vertical loads. Strain gages results for geogrid layers 6 and 10 suggest that, out of the total wall outward displacements induced during all monitored stages (Stages I to VII),

approximately 50% occurred during placement and compaction of the initial 2 m of backfill (approximately 40 kPa vertical earth pressure) above these geogrid layers (Figure 3.2 and Table 2.1). The maximum measured front wall outward displacement induced by wall construction was 12 mm, which corresponds to 0.2 % of the front wall height. The measured settlement of the leveling pad supporting the front wall facing was approximately 8 mm.

- Movements induced during placement of the bridge superstructure. The total average vertical contact pressure directly underneath the bridge footing due to placement of the bridge superstructure was estimated as 115 kPa. The maximum front wall outward displacement and bridge footing settlement induced by placement of the bridge superstructure were 10 and 13 mm, respectively. These movements correspond, respectively, to 0.17% and 0.29 % of the front wall height (0.29% is normalized in relation to a wall height of 4.5 m). The measured settlement of the leveling pad supporting the front wall facing was approximately 7 mm.

- Movements induced after opening of the structure to traffic. The total average vertical contact pressure directly underneath the bridge footing due to placement of the bridge superstructure and the traffic load was estimated as 150 kPa. The maximum front wall outward displacement and bridge footing settlement induced while the bridge was in service (Stage VII) were 13 and 10 mm, respectively. These movements correspond, respectively, to 0.22 % and 0.17 % of the front wall height. The measured settlement of the leveling pad supporting the front wall facing was approximately 5 mm. From the time of opening the bridge to traffic (December 1998 for Phase I Structure and June 1999 for Phase II Structure) until January 2000, the structure showed continued movements. However, from January 2000 to June of 2000, the monitoring results suggest that the front GRS wall and bridge footing experienced negligible movements. Geogrid strain readings leveled out and remained approximately constant during this period.

CDOT engineers expected that the maximum outward displacement of the front GRS wall and settlement of the bridge footing due to placement of the bridge superstructure would not exceed, respectively, 20 mm (measured maximum displacement is 11 mm) and 25 mm (measured maximum

displacement is 13 mm). According to guidelines from the American Association of State Highway and Transportation Officials (AASHTO 1996), the two-span Founders/Meadows Bridge supported at its abutments by GRS walls could safely tolerate a maximum long-term differential settlement (due to placement of the bridge superstructure and after opening the bridge to traffic) of 70 mm without serious structural distress. An additional consideration was the need to maintain a 4.95 m minimum clearance between I-25 and the bottom of the bridge superstructure, which implies that the settlement of the bridge footing should not exceed 100 mm. The maximum settlement recorded for the bridge footing is 23 mm (13 mm induced by placement of bridge superstructure plus 10 mm induced after 18 months of service, as indicated in Table 2.1). This is roughly one-third the tolerable differential settlement (70 mm).

Based on the overall recorded deformation response so far, it can be concluded that the Founders/Meadows structure has showed excellent short- and long-term performance. Specifically, the monitored movements were significantly smaller than those expected in design or allowed by performance requirements, there were no signs of any structural damage, and post-construction movements became negligible after an in-service period of one year.

Table 2.1. Summary of the Maximum Movements of the Front Wall Facing and of the Settlements of the Bridge Abutment Footing

	Induced *Only* by GRS Wall Construction (Stage I)	Induced *Only* by Placement of Bridge Superstructure (Stages II to VI) [1]	Induced *Only* While Bridge was in Service (Stage VII) [2]		
			6 months	12 months	18 months
A. Maximum Outward Displacement (mm) of the Front Wall Facing					
Section 200, Surveying		7	4		6
Section 400, Surveying	9	9	8 [3]	12 [3]	13 [3]
Section 400, Inclinometer			6	11	10
Section 800, Surveying	12	10		5	
Section 800, Strain Gages	11	6	4	4	
Maximum Displacements (mm) (% of Wall Height)	12 (0.2 %)	10 (0.17 %)	8	12	13 (0.22 %)
B. Settlement (mm) of the Leveling Pad Supporting the Front Wall Facing					
Section200, Surveying		7	4		5
Section 400, Surveying	6	7	2	5	5
Section 800, Surveying	8	3		3	
Maximum Settlement (mm)	8	7	4	5	5
C. Bridge Abutment Footing Settlement (mm)					
Section 200, Surveying		13	7		6
Section 400, Surveying			7	11	10
Section 800, Surveying		12		10	
Maximum Settlement (mm) (% of Wall Height)		13 (0.29 %)	7	11	10 (0.17 %)

Notes: [1] Estimated surcharge is 115 kPa; [2] Months in service are counted from December 1998 for Sections 200 and 400, and from June 1999 for Section 800. Estimated surcharge is 150 kPa; [3] Displacements are estimated based on surveying and inclinometer data

3.0 VERTICAL EARTH PRESSURES IN THE FRONT GRS WALL

3.1 Introduction

Figure 1.6 and Table 1.3 show the locations of all pressure cells placed along Section 800 (Phase II Structure) to measure distribution of vertical earth pressures inside the front GRS wall below the bridge footing during all monitored stages. Gages placed between the 6th and 7th geogrid layers will be referred as layer 6 gages (Figure 1.6). Layers 0, 6, 10, and 13 gages are located at depths of 5.43 m, 2.95 m, 1.33 m, and 0.23 m, respectively, below the bottom of the bridge footing. Layer 0 gages were placed at the base of the reinforced fill (depth of 5.43 m). Layer 13 gages were placed directly below the bridge footing (at depth of 0.23 m). One Gage (3VA) was placed close to the facing (location line A) between geogrid layers 3 and 4 (Figure 1.6).

3.2 Typical Vertical Earth Pressure Results during Placement and Compaction of the Backfill

Figure 3.1 shows the vertical earth pressures measured during construction of the front GRS wall (Stage I) from layer 6 gages (Figure 3.1a) and from layer 10 gages (3.1b). The measured data shown in Figure 3.1 was collected every six minutes to monitor the soil response to compaction and construction operations of the wall. Note that in Figure 3.1, 51.5 day refers to the midday of the 52nd day from Jan. 1, 1999 and 52.0 refers to the midnight of that day. As expected, changes in the measured vertical earth pressures occurred during the day working hours and no changes occurred during the evening and night hours. The sudden changes of the measured vertical earth pressures (20 to 35 kPa) shown in this figure are most likely due to compaction-applied loads. Figure 3.1 also shows the estimated average vertical earth pressures (from Eq. 1.1) at the level of the measuring pressure cells at different stages. The measured response from the pressure cells correlates very well with the applied backfill self-weight loads during construction of the front wall (Figure 3.1).

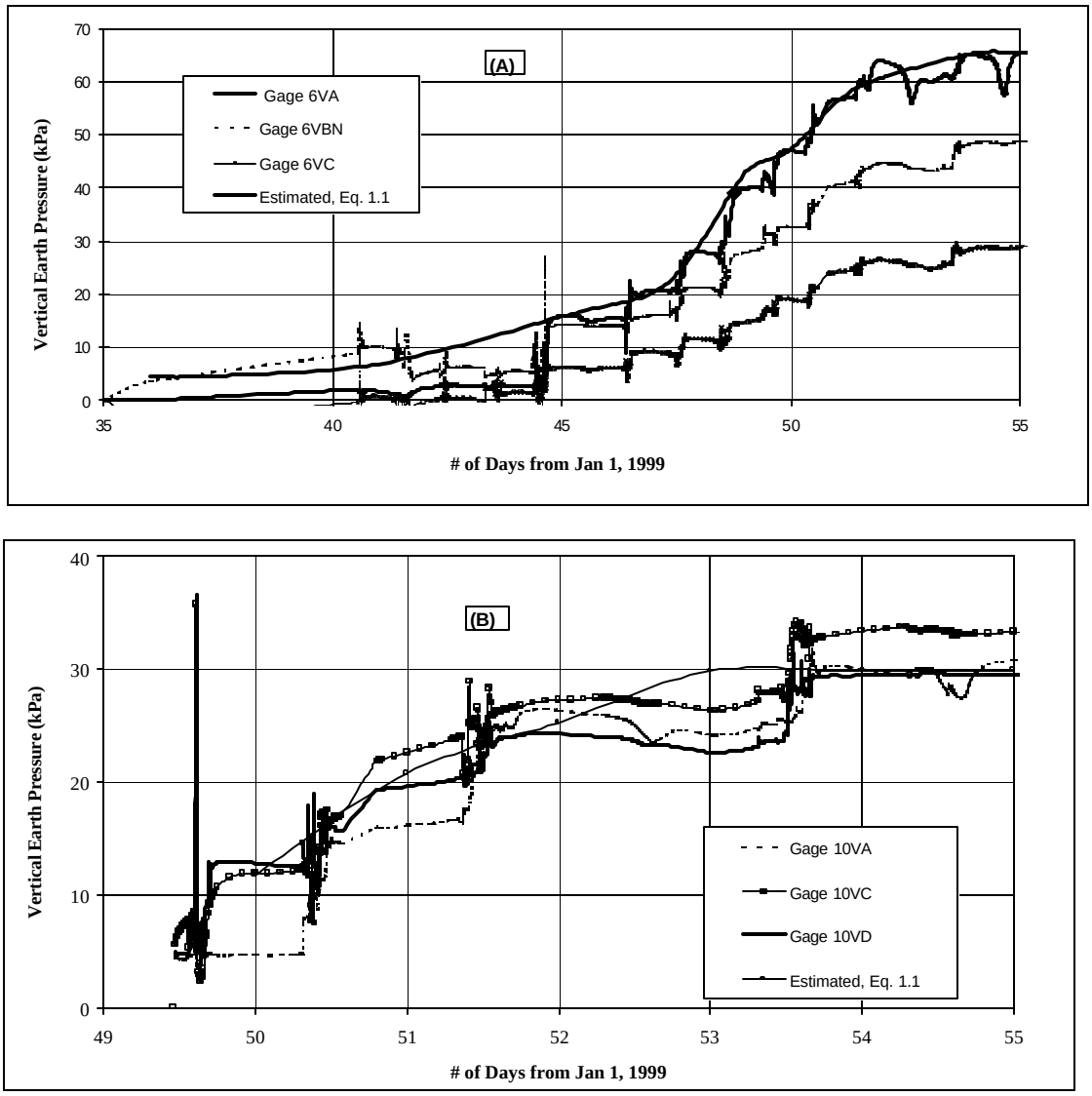


Figure 3.1. Typical Measured and Estimated Vertical Earth Pressure during Placement and Compaction of the Backfill from: (a) Layer 6 Gages, and (b) Layer 10 Gages.

3.3 Results of Vertical Earth Pressures during all Construction Stages

3.3.1 Measured Vertical Earth Pressures

Figure 3.2 shows time records of the vertical earth pressures measured by the pressure cells placed at the base of the reinforced soil mass along location lines A and D during all construction stages (Stages I to VI, till 180 days from Jan. 1, 1999). At the base of the reinforced fill, no pressure cells were placed at location lines B and C (Figure 1.6). Vertical earth pressures value at these missing locations were estimated by vertical and horizontal extrapolation of the measured vertical earth pressures around these two missing locations. Horizontal profiles of the vertical earth pressures at the base of the reinforced fill by end front wall construction (Stage I) and end of all construction stages (Stage VI) are given in Figure 3.3.

Figures 3.4 and 3.5 show time records and horizontal profiles of the measured vertical earth pressures from layer 6 gages during all construction stages. Figures 3.6 and 3.7 show time records and horizontal profiles of the measured vertical earth pressures from layer 10 gages during all construction stages. Figure 3.8 shows horizontal profiles of the measured changes in the vertical earth pressures from layers 6 and 10 gages induced by placement of the bridge superstructure (Stages II to VI, changes from Stage I values).

Figure 3.9 show time records of the vertical earth pressures measured directly beneath the bridge footing (from layer 13 gages) during placement of the bridge superstructure (Stages II to VI). The data of Figure 3.9 was recorded from pressure cells placed (see Figure 1.6) at location line A (close to the exterior edge of the bridge footing from Gage 13VA), location line B (from Gage 13VB), location line C (from Gage 13VC), and location line D (from gage 12VD). Horizontal profiles of the measured vertical earth pressures beneath bridge footing during placement of the bridge superstructure (Stages II to VI) are given in Figure 3.10.

Time records for the vertical earth pressures measured from all gages placed close to the wall facing (location line A) during all construction stages are summarized in Figure 3.11.

Table 3.1 summarizes the vertical earth pressures measured from all gages during all construction stages. Table 3.1 also summarizes changes in measured vertical earth pressures developed due to the placement of the bridge superstructure (Stages II to VI, like in Figure 3.8), referred to as "changes from Stage I." The measured changes in vertical earth pressures along different location lines (A, B, C, and D) induced by placement of the bridge superstructure (Stages II to VI) are summarized in Figure 3.12.

3.3.2 Discussion of the Results measured during Construction

By the end of Stage I (front wall construction) at depths of 5.43 m (layer 0) and 2.95 m (layer 6), the vertical earth pressures close to the wall facing exceeded those at other locations (see Figures 3.3 and 3.5). The reinforced fill at a depth of 1.33 m (layer 10) showed almost uniform distribution of vertical earth pressures at the end of Stage I Construction (Figure 3.7).

Sharp increases in measured vertical earth pressures were observed at all levels along location lines A, B, and C during Stage III construction (placement of girders) in two days (see Figures 3.2, 3.4, 3.6, and 3.9). As expected, the pressure cell placed along location line D at the base of the fill (Gage 0VD) recorded small changes in vertical earth pressure (Figure 3.2 and Figure 3.12d), but the pressure cell along location line D beneath the bridge footing (Gage 12VD) recorded no change in the vertical earth pressure (Figure 3.9 and Figure 3.12d). During Stages II and III, measured vertical earth pressures beneath the edges of the bridge footing are higher than at the center of the footing (Figure 3.10). This is a typical pressure diagram for rigid footing resting on any material. The results of gages placed at deeper depths (layers 0, 6 and, 10) during stages II and III suggest the following:

- Location line B along the center of the bridge abutment footing experienced the highest increase in vertical earth pressures during these stages followed by location C (Figures 3.8 and 3.12).
- The changes in vertical earth pressures along location lines B and C during these stages decreased with the depth below the bridge footing as expected (Figures 3.8 and Figure 3.12), but such a trend was not noticed for changes in vertical earth pressures along location lines A (close to the facing, Figure 3.12a) and D (Figure 3.12d).

The pressure cell placed along location lines D and C responded to loading during Construction Stage IV, when the overlying backfill was placed behind the abutment wall (see Figures 3.2 to 3.9 and Figures 3.12c and 3.12d). As expected also, location line B showed a small response near to the bridge footing (Gage 12VB) and showed larger responses at deeper levels (Figure 3.12b). Location line A showed a response to loading during Stage IV only in the zone close to the base of the fill as expected (Figure 3.12a).

During Stage V, there was a severe and reduction in the measured vertical earth pressures close to the wall facing along location line A (see Figures 3.2 to 3.8 and 3.11) and in the zone beneath the bridge footing (Figures 3.9 and 3.10). This can be attributed to the comparatively large lateral movements of the front wall facing (see Chapters 2 and 5) and possible settlement of the bridge footing during Stage V. The results along location line A behind the wall facing shown in Figures 3.11 and 3.12a and Table 3.1 indicate that the largest reduction occurred at the base of the fill (40 kPa, Figure3.12a), the reduction was 23 kPa, 12 kPa, and 13 kPa at depths 3.76 m (Gage 3VA), 2.95 m (Layer 6), and 1.33 m (Layer 10), respectively. Directly below the bridge footing (depth 0.23 m), the measured reduction was 10 kPa at location line C and 17 kPa at location line A (Figure 3.10). Note that although the vertical earth pressure at these locations dropped down, the total vertical earth pressures remained in compression (Table 3.1). During Stage V, the pressure cells placed along location line D showed almost no change beneath the bridge footing (from Gage 12VD, see Figure 3.12d) and small increases at layers 6 and 10 gages (Figure 3.12d). As the vertical earth pressures close to the wall facing were dropping for layers 6 and 10 gages during Stage V, the vertical earth pressures along location line B (along the bridge abutment centerline) and to a lesser degree along location line C were increasing at a relatively high rate (Figure 3.4 to 3.8 and Figure 3.12). This behavior suggests transfer of vertical earth pressures at layers 6 and 10 from the zone close to the wall facing to a zone deeper in the reinforced soil.

The loading response at different locations during Stage VI is very similar to that presented for Stage IV.

By the end of all construction stages (Stage VI), the measured vertical earth pressures at location line A became very close to or even less than the values measured by the end of Stage I Construction (see Figures 3.11 and 3.12a). In other words, the measured changes in vertical earth pressure behind the wall facing that were induced by placement of the bridge superstructure were very small (less than 15 kPa). The distribution of vertical earth pressures directly beneath the bridge footing was almost uniform: 28 kPa along location line B, 34 kPa along location line A, and 39 kPa along location line C (Figure 3.10). At other levels (layers 0, 6, and 10), the vertical earth pressure distribution within the reinforced soil mass below the bridge footing differed significantly from location to location (Figures 3.3, 3.5, and 3.7). The lowest vertical earth pressure occurred close to the facing and the highest vertical earth pressure occurred along location line B, the centerline of the bridge abutment. This distribution was close to symmetrical around the center of the reinforced soil zone (~3.75 m from rear side of the facing), especially for layers 0 and 10 (Figure 3.3 and 3.7).

Table 3.1: Measured Vertical Earth Pressures (kPa) during all Construction Stages.

Stage #	I	II		III		IV		V		VI	
		Ch. = Change from Stage I (Wall construction) Values									
Gage #		Ch.	Total	Ch.	Total	Ch.	Total	Ch.	Total	Ch.	Total
Layer 0 Gages											
0VA	136	13	149	25	161	36	172	-3	133	2	138
0VD	75	6	81	12	87	37	112	50	125	62	138
3VA*	61	7	68	12	73	13	74	-11	51	-11	50
Layer 6 Gages											
6VA	66	13	79	24	90	26	92	15	81	9	75
6VBN	49	21	70	54	103	77	126	122	171	135	184
6VBS	36	20	56	47	83	63	99	94	130	99	135
6VC	29	6	35	18	47	39	68	58	87	71	100
6VD	47	3	50	7	54	41	88	51	98	66	113
Layer 10 Gages											
10VA	31	8	39	14	45	14	45	0	31	-1	30
10VBN	18	28	46	59	77	70	88	98	116	92	110
10VBS	29	38	67	85	114	100	129	142	171	139	168
10VC	33	10	43	30	63	62	95	86	119	106	139
10VD	30	-1	29	-2	28	36	66	39	69	54	84
Layer 13 Gages**											
12VD**	0	1	1	0	0	18	18	19	19	27	27
13VA	0	16	16	53	53	55	55	38	38	34	34
13VB	0	13	13	29	29	32	32	39	39	28	28
13VC	0	8	8	33	33	41	41	30	30	39	39

*Gage 3VA placed between geogrid layers 3 and 4

** Gage 12VD placed between geogrid layers 12 and 13

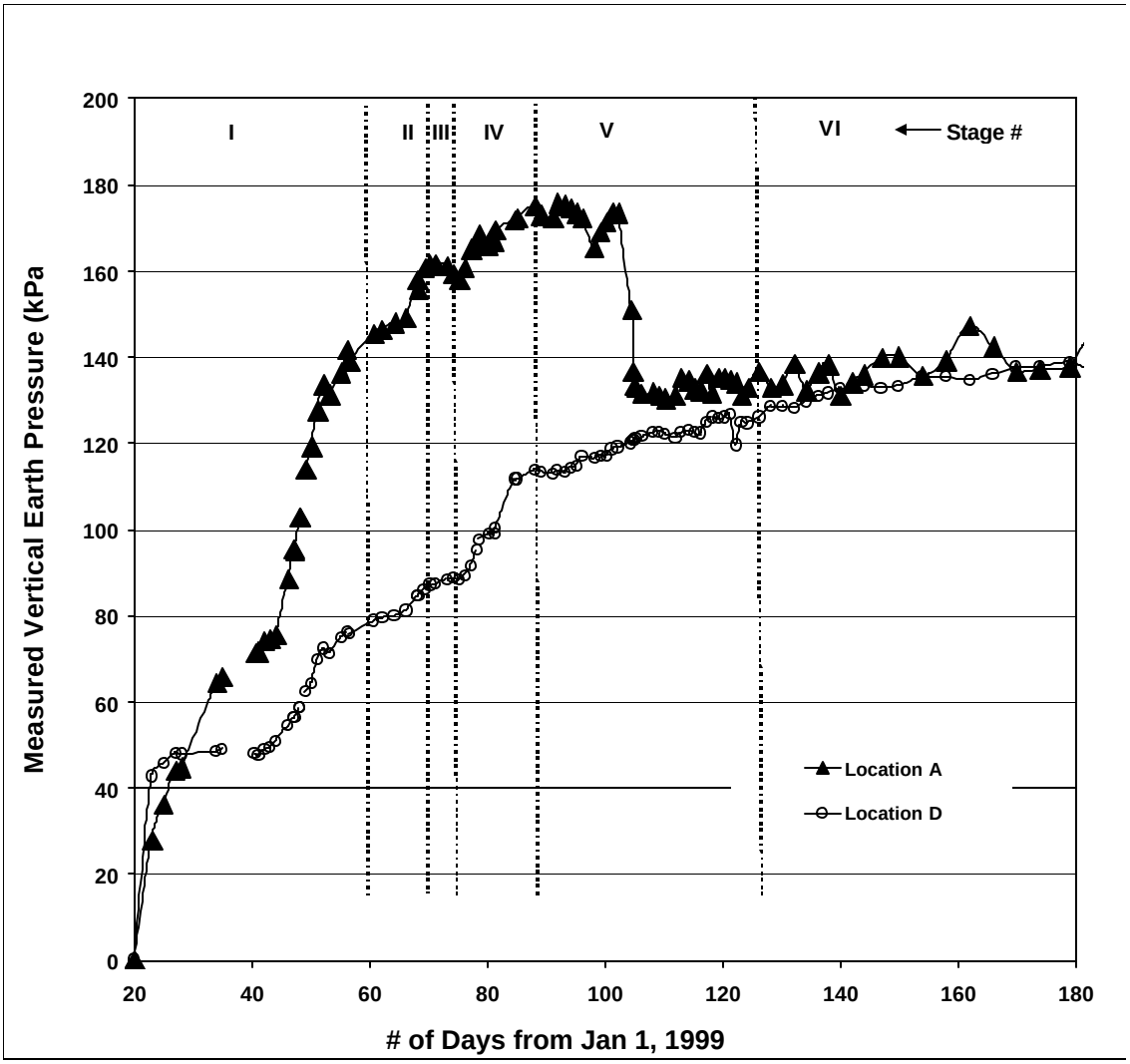


Fig. 3.2. Time Records for the Vertical Earth Pressure Measured from Gages Placed at the Base of the Fill during all Construction Stages.

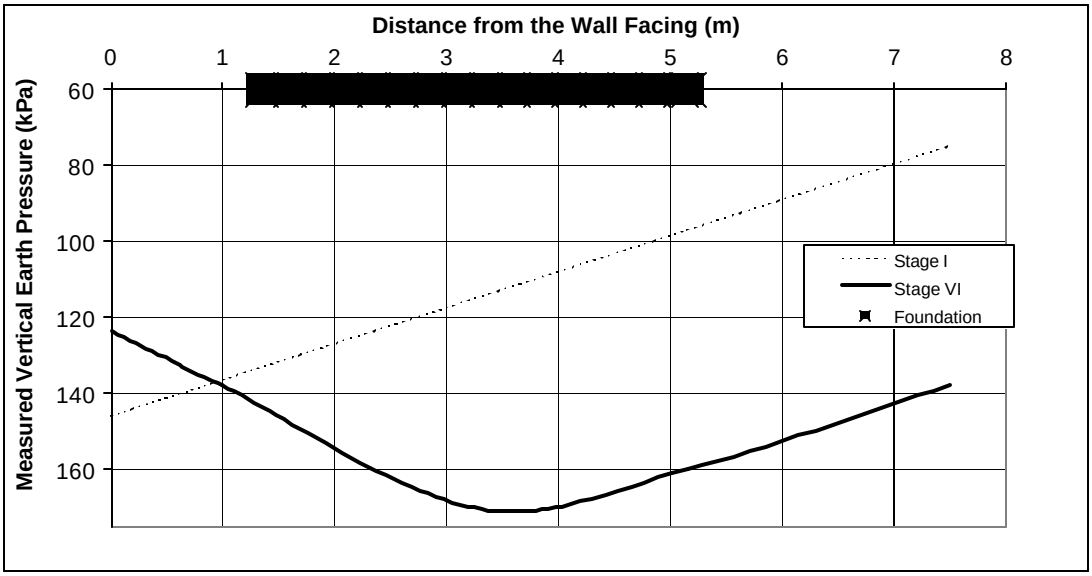


Fig. 3.3. Horizontal Profiles of the Vertical Earth Pressures Measured at the Base of the Reinforced Fill during Construction Stages.

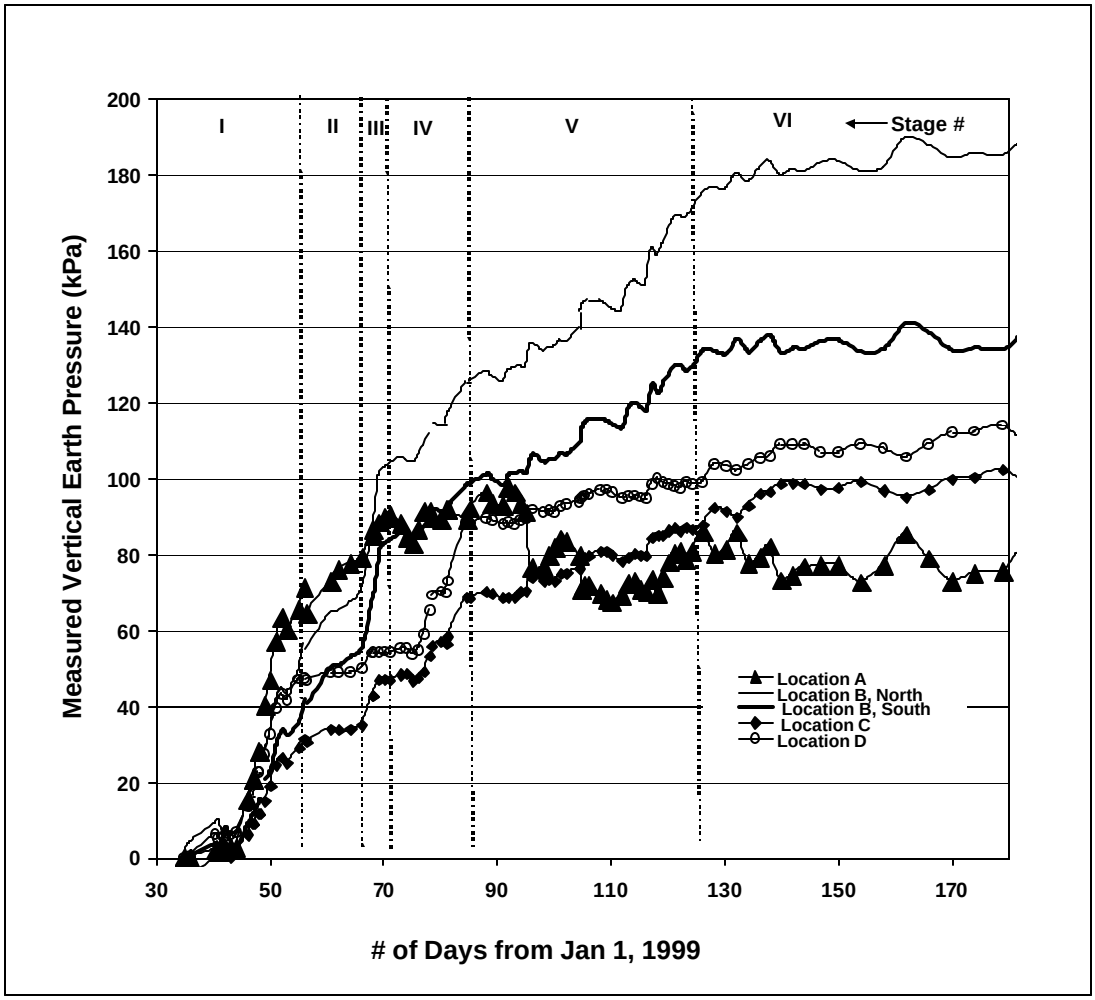


Fig. 3.4. Time Records for the Vertical Earth Pressures Measured from Layer 6 Gages during all Construction Stages.

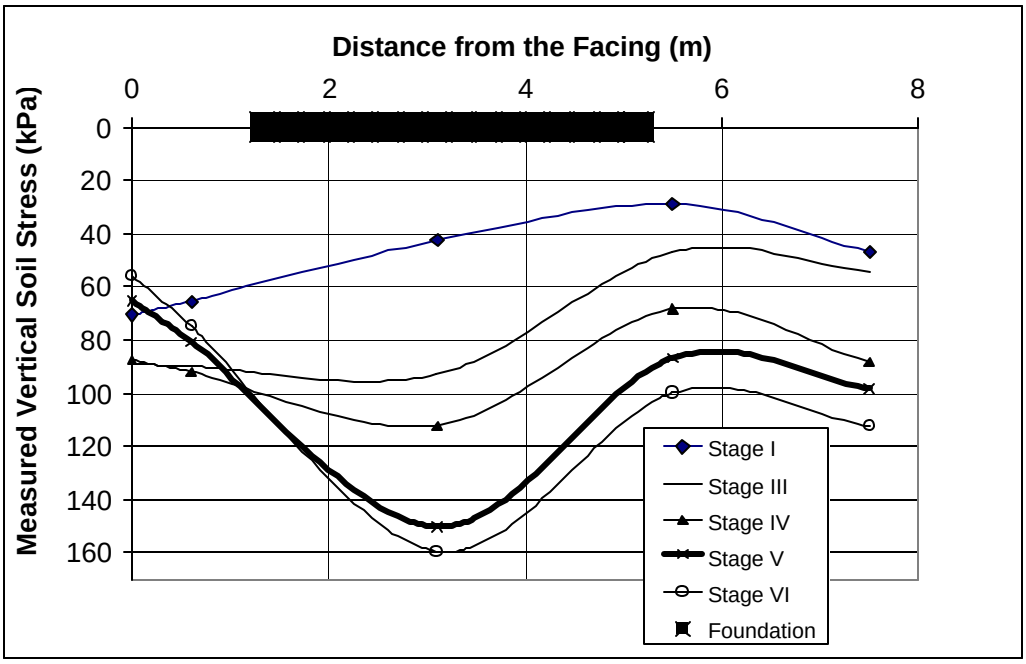


Fig. 3.5. Horizontal Profiles of the Vertical Earth Pressures Measured from Layer 6 Gages during Construction Stages.

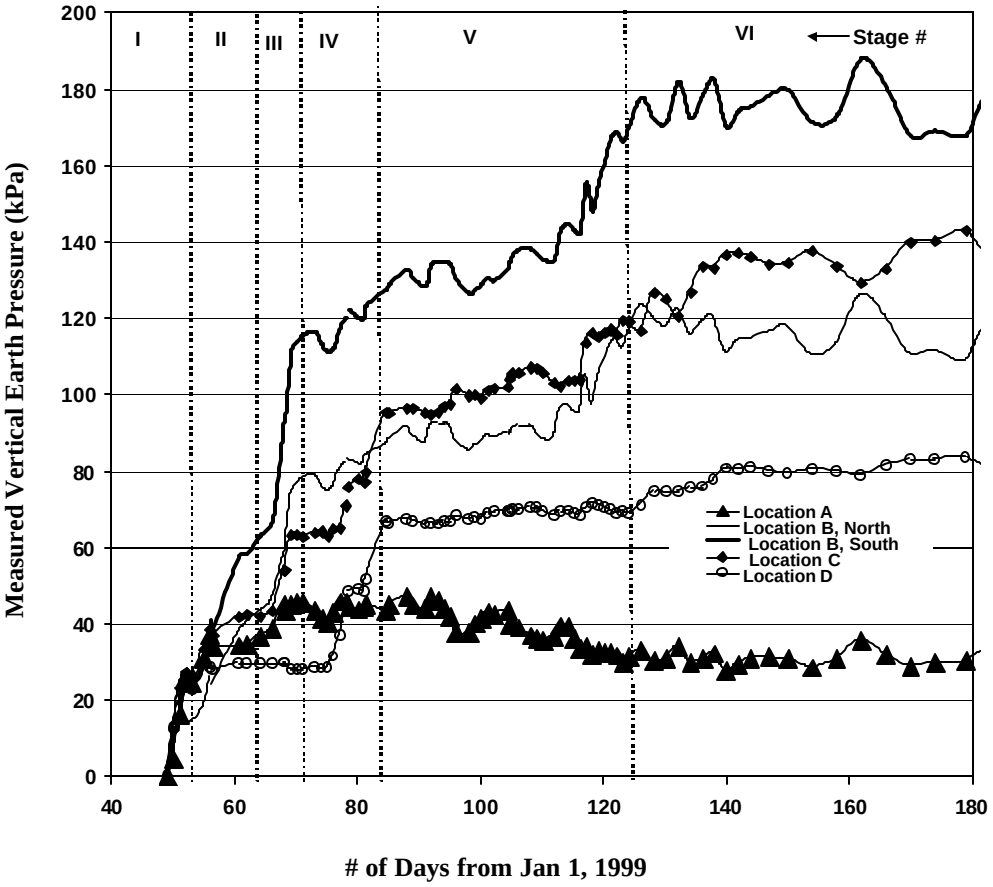


Fig. 3.6. Time Records for the Vertical Earth Pressures Measured from Layer 10 Gages during all Construction Stages.

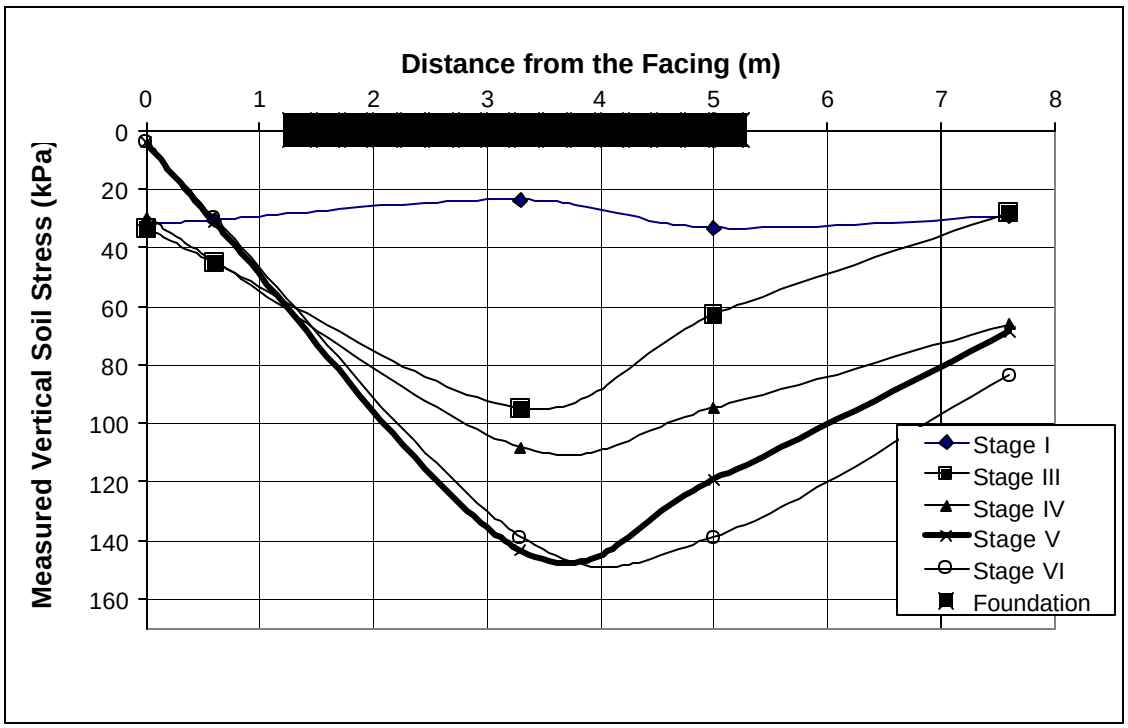


Fig. 3.7. Horizontal Profiles of the Vertical Earth Pressures Measured from Layer 10 Gages during Construction Stages.

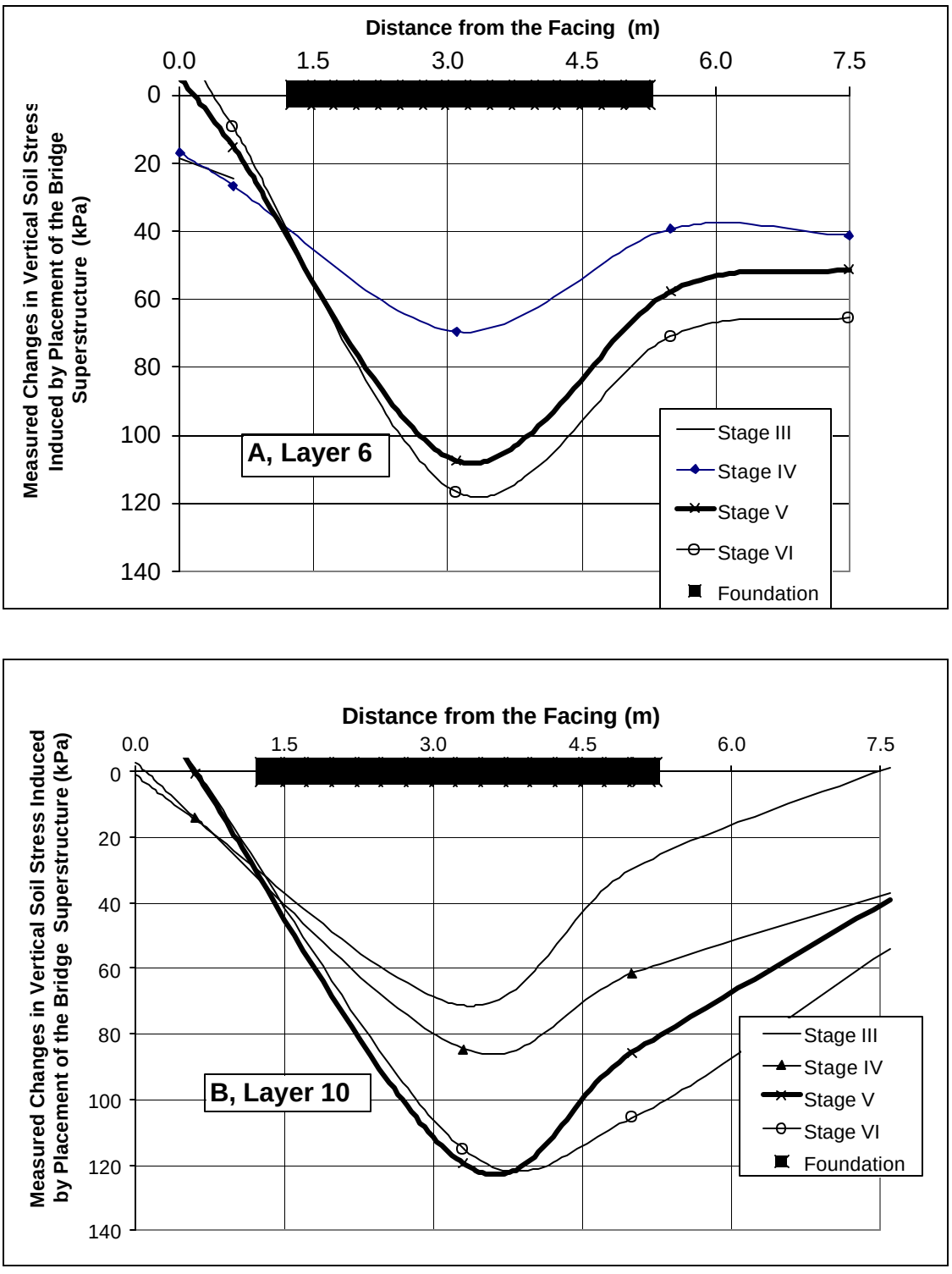


Fig. 3.8. Horizontal Profiles of Measured Changes in Vertical Earth Pressure Induced by Placement of the Bridge Superstructure (Stages II to VI from Stage I).

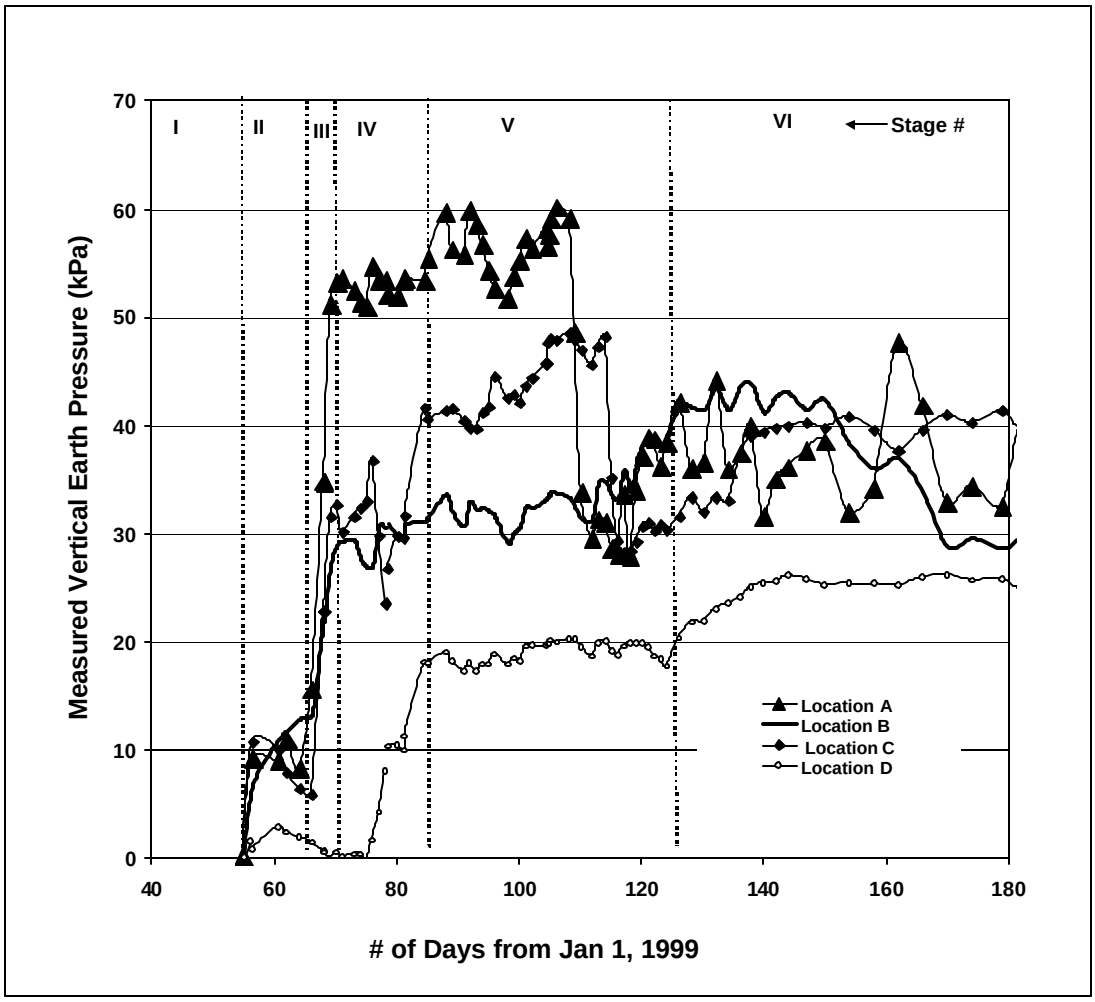


Fig. 3.9. Time Records for the Vertical Earth Pressures Measured from Layer 13 Gages Placed Beneath Bridge Footing during Construction Stages.

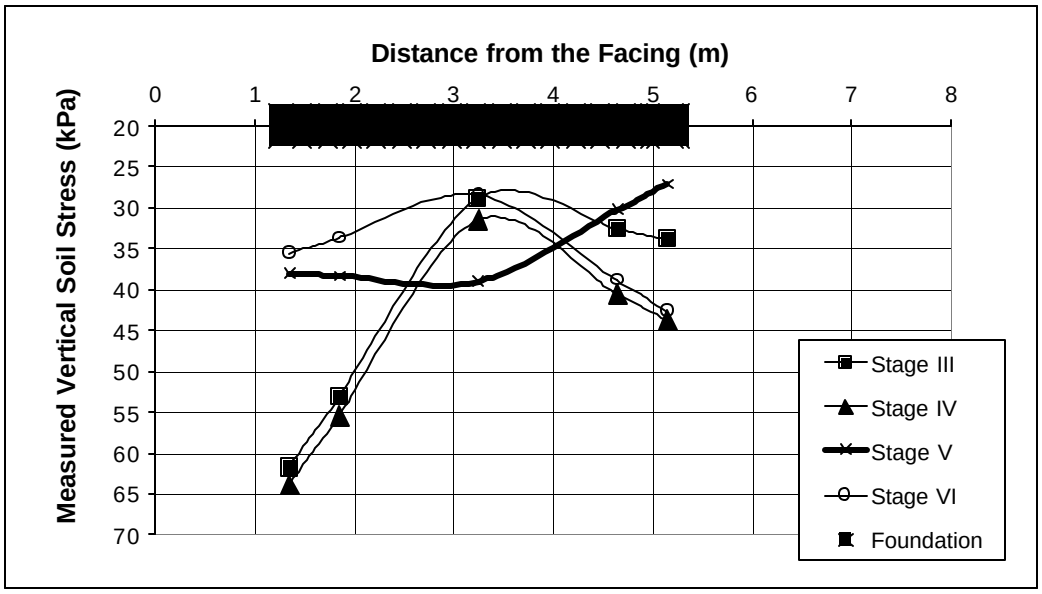


Fig. 3.10. Horizontal Profiles of the Vertical Earth Pressures Measured from Layer 13 Gages Placed Beneath Bridge Footing during Construction Stages.

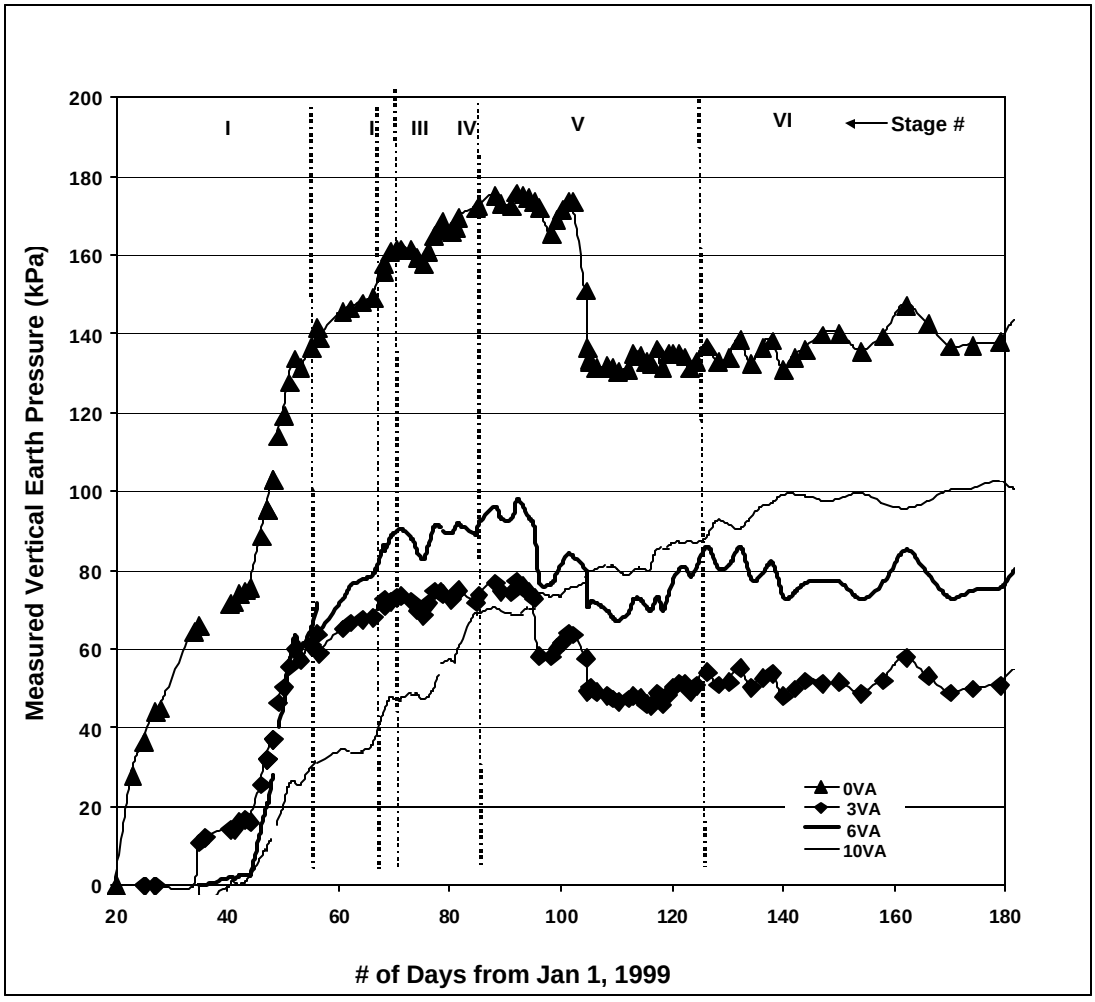


Fig. 3.11. Time Records for the Vertical Earth Pressure Measured during all Construction Stages from Gages Placed along Location Line A Close to the Wall Facing.

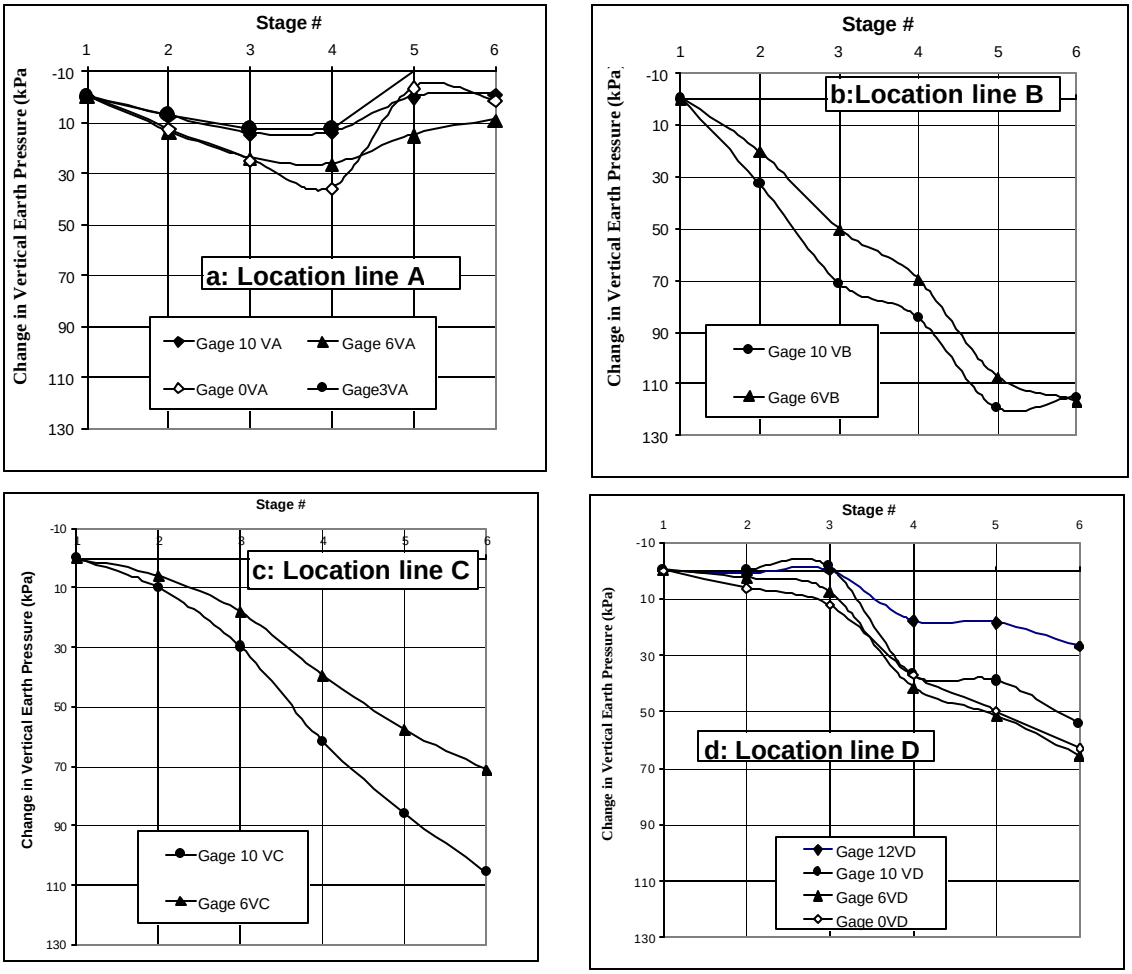


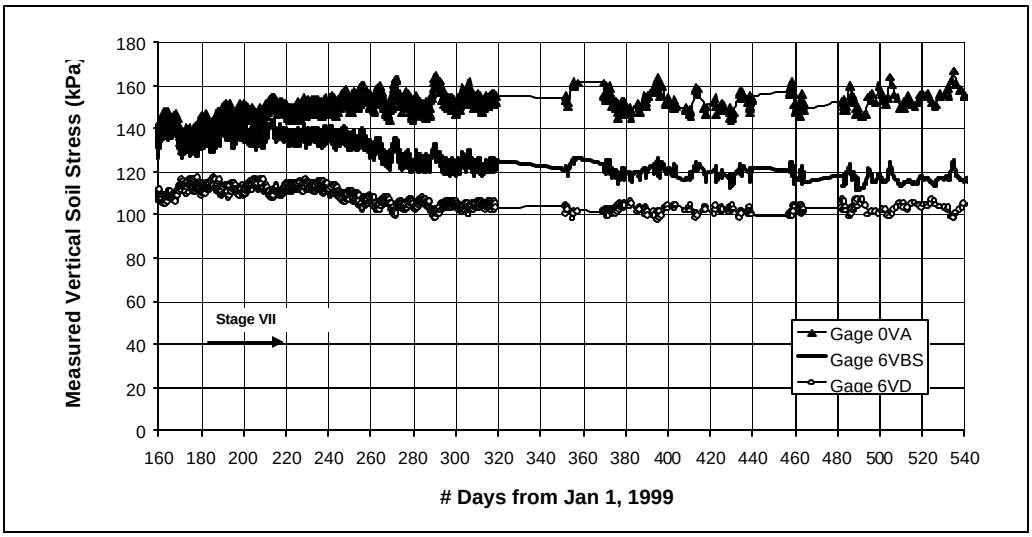
Fig. 3.12. Measured Vertical Earth Pressure along Different Location Lines due to Placement of the Bridge Superstructure.

3.4 Results of Vertical Earth Pressures while the Structure was in Service

Figure 3.13 shows selected samples of time records of measured vertical earth pressures after opening the structure to traffic during Stage VII (from days 180 to 545 from Jan. 1, 1999). Table 3.2 summarizes the vertical earth pressures measured by the end of the construction stages (Stage VI) and after opening the bridge to traffic (Stage VII). Table 3.2 also summarizes changes in vertical earth pressure developed after opening the bridge to traffic, referred to as "changes from Stage VI." As seen in Table 3.2, only six gages registered small changes in the measured vertical earth pressures after opening the structure to traffic (0VA, 6VBS, 6VD, 10VBS, 13VA, 10VC) ranging from -11.3 kPa (a decrease) to 14.5 kPa (an increase). These small changes occurred immediately after opening the bridge to traffic (see Figure 3.13), possibly due to placement of minor structures on the bridge structure (even after opening the bridge to traffic). All other gages (15 gages) showed very small change in the measured vertical earth pressures after opening structure to traffic (Table 3.2). As expected, live loads from traffic do not seem to leave behind "locked-in" vertical earth pressures.

Table 3.2: Measured Vertical Earth Pressure while Structure was in Service.

Monitored Stage #	End of Construction (Stage VI)	While Structure in Service (Stage VII)	
Gage #		Change from Stage VI	Total
0VA	138	14	152
0VD	138	-4	134
3VA	50	5	55
6VA	75	3	78
6VBN	184	-1	183
6VBS	135	-11	124
6VC	100	-6	94
6VD	113	-8	105
10VA	30	2	32
10VBN	110	7	117
10VBS	168	15	183
10VC	139	-11	128
10VD	84	-3	81
12VD	27	-1	26
13VA	34	8	42
13VB	28	0	28
13VC	39	-2	37



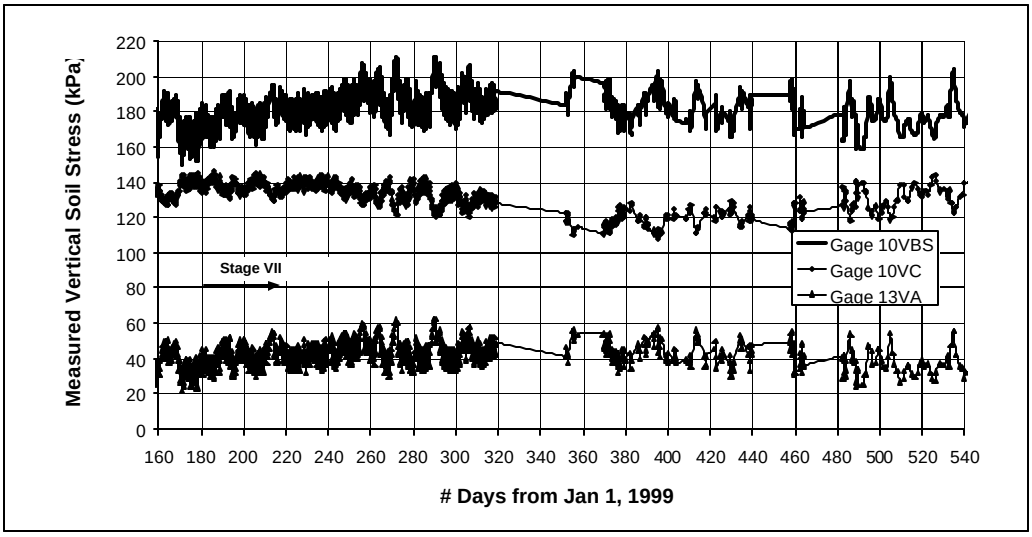


Fig. 3.13. Time Records for Measured Vertical Earth Pressures While Bridge was in Service (Stage VII).

3.5 Consistency of the Measured Data

The results from pressure cell gages placed at the same location (North and South in Figures 3.4 and 3.6) ran close to each other during Stage I when uniform backfill self-weight loading was applied, and almost parallel to each other during Stages IV through VI. The parallel behavior indicates almost similar response to different levels of loading by these two gages. The difference between these two gages ran up to almost 30%. The placement of girders on the abutment wall of Phase II Structure in two days starting from the south side, and the variability in soil properties between these two locations (e.g., variable soil stiffness due to variable compaction) might have contributed to this difference.

The measured response from the pressure cells correlates very well with the applied backfill self-weight loads during construction of the wall and during all subsequent construction stages. Sharp increases in measured vertical earth pressures were registered during Stage III construction (placement of girders). As expected, the pressure cells placed along location line D responded to loading during Construction Stages IV and VI when the overlying backfill and approach slab were placed, but showed very small response when the bridge structure was placed (Stages II, III and V). There is a close agreement between the results of pressure Cell P1 (118 kPa by end of Stage VI) placed along Section 400 of the Phase I Structure (Figure 1.5) and Gage 6VBS (135 kPa by end of Stage VI), both placed at the same elevation and distance from wall facing. Results of the next sections will demonstrate excellent agreement between the measured and applied total earth loads on geogrid layers 6 and 10 during different construction stages and along location line D. As will also be discussed in the next two chapters, the measurements for vertical earth pressures are strongly correlated with the measured lateral earth pressures against the wall facing and geogrid lateral strains. Based on the above discussion, it is fair to conclude that the measured results for the vertical earth pressures during construction are consistent and representative of the actual vertical stresses experienced by the structure.

3.6 Analysis and Design Implication of the Measured Results

When two gages are placed at the same location, their average value is used in analyzing the results.

The width of the reinforced soil zone is approximately 7.5 m at the bottom of the reinforced fill and increases on 1H: 1V slope toward the top (see Figure 1.3). To simplify the data analysis, the width of the reinforced soil zone is taken as 7.5 m (from the rear side of the facing to approximately location line D, see Figure 1.6). Vertical earth pressure data across the reinforced soil zone were obtained from pressure cells placed along location lines A, B, C, and D (Figure 1.6).

Very small change in the measured vertical earth pressures occurred while the structure was in service. Live loads due to traffic loading means accounting for a scenario such as a line of fully loaded trucks suddenly stopping on top of the bridge during a traffic jam. If this scenario happened during Stage VII, the instruments certainly did not capture it. Therefore, the collected data while the structure was in service cannot verify or be employed to investigate design assumptions regarding the effect of live load on the vertical loading of the structure. Therefore, these data will not be utilized herein for assessment of the design procedure. The data measured by end of construction stages will be utilized in the assessment of the design procedure of the front GRS wall. These data were proven reliable in the previous section.

3.6.1 Overturning Stability Analysis of the Front GRS Wall

The center location of the reinforced soil zone assumed in the analysis is 3.75 m from the facing. Positive eccentricity means that the location of the resulting vertical force acting on the reinforced soil zone is shifted from the center location of the reinforced soil zone toward the wall. The horizontal profiles of the vertical earth pressures measured from layer 0, 6, and 10 gages (Figures 3.3, 3.5, and 3.7) were employed to determine the eccentricity values of the resulting vertical earth forces acting, respectively, at: 1) the base of the reinforced fill, 2) the horizontal level within the reinforced soil zone at depth of 2.95 m (Layer 6), and 3) the horizontal level within the reinforced soil zone at depth of 1.33 m (Layer 10). The horizontal profiles of the vertical earth pressures measured just beneath the bridge footing (from front edge to back edge) from Layer 13 gages (Figure 3.10) were employed to determine the eccentricity value of the resulting vertical earth force acting on the bridge footing. Results for these measured eccentricity values at different stages and locations are shown in Figure 3.14 and Table 3.3. Also shown in Table 3.3 are the maximum eccentricity values measured during all construction stages.

By the end of Stage I Construction, the reinforced fill at depth 1.33 m (layer 10) experienced almost uniform distribution of vertical earth pressures but for layers 0 and 6 larger vertical earth pressures developed close to the wall facing (Figures 3.3, 3.5 and 3.7). This caused very small eccentricity on layer 10 (close to zero) and larger eccentricity values for layers 0 and 6 (see Figure 3.14). Eccentric loading of the bridge footing during Stages II and III was felt mostly beneath the bridge footing and at depth of 1.33 m (Figure 3.14). The placement of backfill behind the bridge abutment during Stage IV increased the loads in the interior side of the reinforced soil zone. This stabilized the reinforced soil mass against overturning and reduced the eccentricity values at all levels below the bridge footing (Figure 3.14). With the redistribution of vertical earth pressure that occurred during Stage V (i.e., reduction of vertical earth pressure close to the facing and increase of vertical earth pressure along the bridge abutment centerline), the eccentricity values were further reduced during Stage V (Figure 3.14). By the end of all construction stages, the eccentricity values of the resulting vertical forces at most locations were around zero. This is because the distributions of vertical earth pressures at different horizontal levels were uniform or close to symmetrical around the center of the reinforced soil zone as was discussed in a previous section (see Section 3.3.2 and Figures 3.3, 3.5, 3.7 and 3.10).

To meet the overturning stability requirements, the eccentricity of the resulting force must be less than L/4 for rock and L/6 for soil, where L is the width of the footing (Christopher and Elias, 1997). The allowable eccentricity is 0.64 m for the bridge footing (3.8/6), 1.25 m for the resulting force acting on the reinforced soil mass (7.5/6), and 1.88 m for the rocky base of the fill (7.5/4). Based on this information and the measured eccentricity values (Figure 3.14 and Table 3.3), the following can be concluded:

- The eccentricity values of the resulting vertical forces acting within and at the base of the front GRS wall is almost zero, well below the tolerable and design values (i.e., the factor of safety against overturning of the Founders/Meadows Bridge is very high, significantly exceeding the value estimated in the design).
- The most critical stage for the overturning stability occurred at the end of Stage III (the placement girders). The backfill behind the bridge (placed during Stage IV) should be placed before placement of the girders to increase the forces resisting potential overturning of the reinforced soil structure.

- The movement of the front GRS wall during Stage V (see Chapters 2 and 5) resulted in the reduction of high vertical earth pressure developed behind the wall facing. This enhanced the overturning stability of the Founders/Meadows Bridge structure.

Table 3.3. Measured Eccentricity Values for the Resulting Vertical Forces Acting below the Bridge Footing, Inside the Reinforced Fill, and at the Base of the Reinforced Fill.

Location	Measured Eccentricity by end of Construction, Stage VI, (m)	Maximum Measured Eccentricity during all Stages (m)
Bridge Footing	-0.05	0.20
Layer 10, Depth of 1.33 m	-0.43	0.23
Layer 6, Depth of 2.95 m	-0.1	0.4
Base of the Fill	0	0.4

3.6.2 Bearing Capacity Analysis for the Reinforced Soil Mass

The horizontal profiles of vertical earth pressures measured from layers 0, 6, and 10 gages across the 7.5 m wide reinforced soil zone (Figures 3.3, 3.5, and 3.7) were employed to determine the average measured value of vertical earth pressure acting on these layers. The corresponding estimated applied average vertical earth pressures at the horizontal level of these layers were estimated using the information summarized in the third column of Table 1.3. Applied and measured average vertical earth pressures on these layers during various construction stages are shown in Figure 3.15. The average measured vertical earth pressure values give insight into the loads resisted by the reinforced soil mass. The difference between the applied (estimated) and measured values provides insight into amount of vertical soil loads transferred through side shear to the structures alongside the reinforced soil mass. Most of this side shear is developed due to the settlement of the reinforced soil mass relative to the surrounding structures. By the end of Stage I Construction, the applied and measured average vertical earth pressures matched at a depth of 1.33 m, were very close at the base of the fill, and around 20-kPa difference occurred at depth of 2.95 m (Figure 3.15). During all subsequent construction stages, the measured changes in average vertical earth pressures for layers 6 (depth 2.95 m) and 10 (depth 1.33 m) were very close to those estimated (Figure 3.15). This implies that most of the applied vertical loads during placement of the bridge structure at depths of 1.33 m to 2.95 m below the bridge footing

were supported by the reinforced soil mass (i.e., very small loads if any were transferred to the wall facing).

Allowable bearing capacity of 480 kPa was recommended for the claystone bedrock supporting the front GRS walls (Abu-Hejleh et al., 2000). The measured average vertical earth pressure on the base of the reinforced fill (depth of 5.43 m) by the end of Construction Stage VI was 154 kPa, less than the estimated value of 199 kPa (Figure 3.15). To account for eccentric loading and calculate the bearing pressure, the Meyerhof approach assumes that eccentric loading results in a uniform redistribution of pressure over a reduced width equal to the width of the footing (L) less twice the eccentricity (Elias and Christopher, 1997). The measured eccentricity of the resulting force acting on the base of the fill by the end of the construction stages was zero (see Table 3.3). Thus, it can be concluded that the measured bearing pressure at the base of the reinforced fill is below the expected design value and well below the tolerable soil bearing capacity.

3.6.3 Bearing Capacity Analysis of the Bridge Footing

The horizontal profiles of vertical earth pressures measured from the three gages placed 0.23 m beneath the bridge footing (Figure 3.10) were employed to determine the average measured value of vertical earth pressure beneath the bridge footing. The corresponding estimated applied average vertical earth pressures were estimated using the information summarized in the second column of Table 1.3 and Equation 1.1. The estimated applied and measured vertical earth pressures beneath the bridge footing during different construction stages are shown in Figure 3.16. This figure indicates there is a 40-kPa difference between the applied and measured vertical earth pressures by the end of Stage IV. During subsequent stages, especially Stages V and VI, the difference between the measured and applied loads beneath the bridge footing increased significantly (Figure 3.16). It is possible that the difference between the applied and measured loads beneath the bridge footing was transferred to the wall facing through side shear due to the settlement of the reinforced soil mass beneath the bridge footing relative to the more rigid wall facing.

Allowable bearing capacities on the reinforced soil masses of 240 kPa and 200 kPa were recommended by, respectively, CDOT engineers (Abu-Hejleh et al., 2000) and the FHWA (Elias and Christopher, 1997). The measured average vertical earth pressure beneath the bridge footing by the end of the construction stages was 34 kPa, significantly below the estimated value of 119 kPa. The measured eccentricity of the resulting force acting on the bridge footing by the end of the construction stages was very close to zero (see Table 3.3). Thus, it can be concluded that the measured bearing pressure below the bridge footing is way below the expected value from the design and the allowable bearing capacity value.

3.6.4 Vertical Earth Pressure Distributions for Internal and External Stability Analyses

It was found from the results of strain gages (Section 5.4.4) that the potential failure line extends from the toe of the wall to the back edge of bridge footing (location line C). The vertical earth pressure employed for the calculation of maximum reinforcements forces along the potential failure line was estimated through Equation 1.1 (see Figure 3.17). Also shown in Figure 3.17 are three vertical profiles of measured vertical earth pressures 1) average of earth pressures from the wall facing to location line C (back edge of the bridge footing), 2) maximum of earth pressures measured along location line B, and 3) vertical earth pressures directly behind the wall facing. The results shown in Figure 3.17 suggest:

- The vertical earth pressure distribution within the potential failure zone differs significantly from location to location. The lowest vertical earth pressure occurred close to the wall facing and the highest vertical earth pressure occurred along the centerline of the bridge abutment (see Figures3.5 and 3.7).
- Equation 1.1 used in the design procedure provides good estimation of the average vertical earth pressure acting within the potential failure zone, not along the potential failure line. The design procedure might underestimate the vertical earth pressures acting on the potential failure line beneath the bridge footing by approximately 20%. The design procedure overestimates the vertical earth pressure acting on the potential failure line close to the toe of the wall by approximately 33%.

- The measured vertical earth pressure behind the wall facing is smaller than the estimated vertical earth pressure from Equation 1.1.by approximately 33% (close to the bottom of the wall) to 98 % (at a depth of 1.33 m below the footing).

The external stability analysis requires the estimation of vertical earth pressures developed at plane at the back end of the reinforced soil mass (location line D). These vertical pressures were obtained from measured vertical earth pressures along location line D. Figure 3.18 shows a profile of measured and assumed vertical earth pressures along location line D by the end of the construction stages (VI). Excellent agreement between the measured and predicted design values occurred at depths of 1.33 m and 2.95 m. Therefore, it can be concluded that the design procedure provides reasonable estimations for the vertical earth pressures developed along the back of the reinforced soil zone.

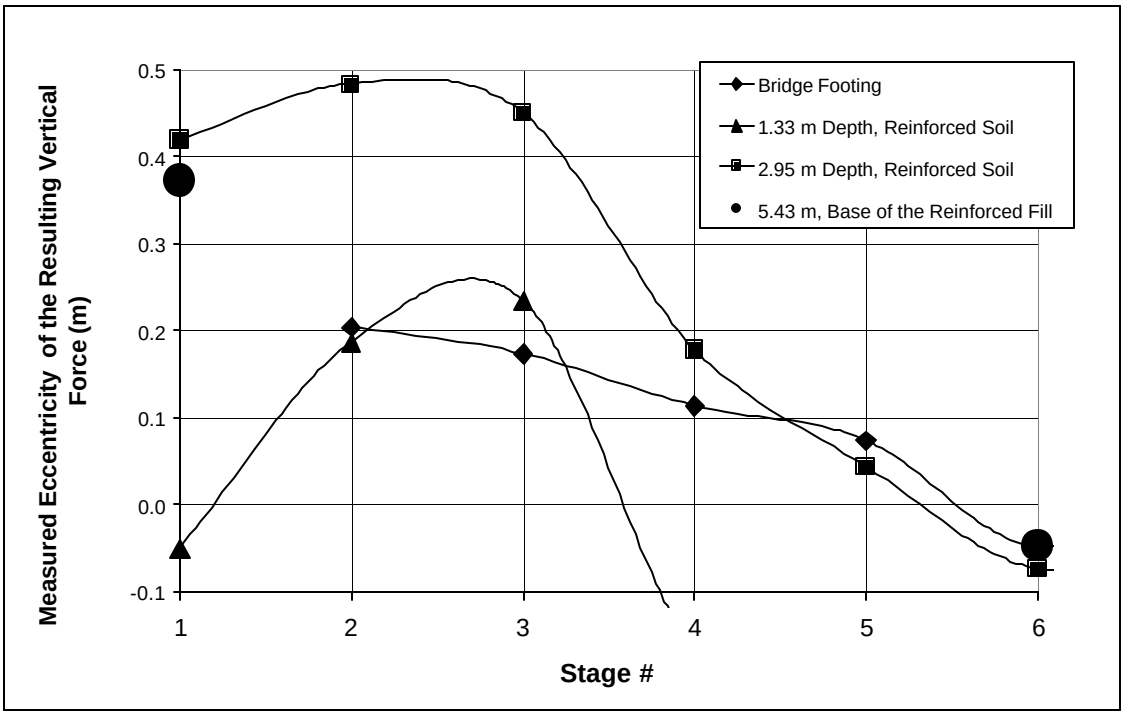


Fig. 3.14. Eccentricity Values of the Resulting Vertical Forces Acting at Different Horizontal Levels inside the Front GRS Wall.

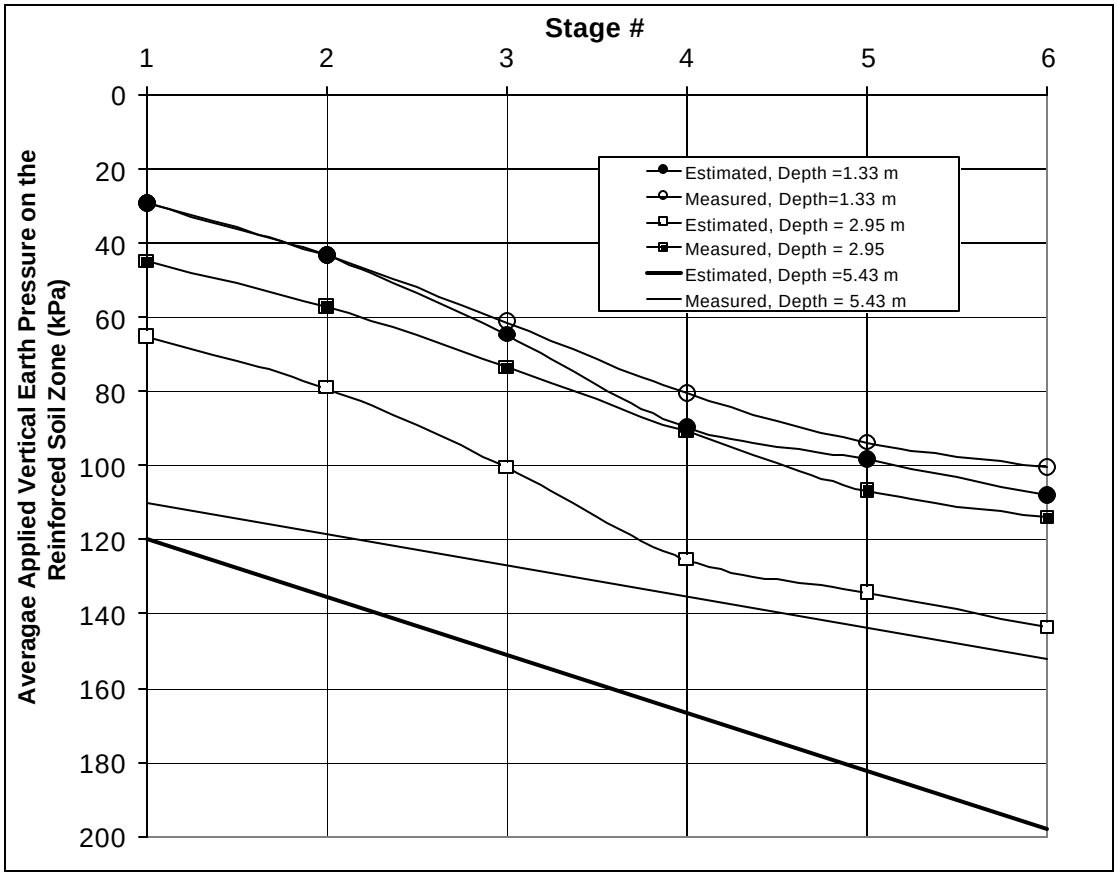


Fig. 3.15. Measured and Applied Average Vertical Earth Pressures at Different Horizontal Levels inside the Front GRS Wall.

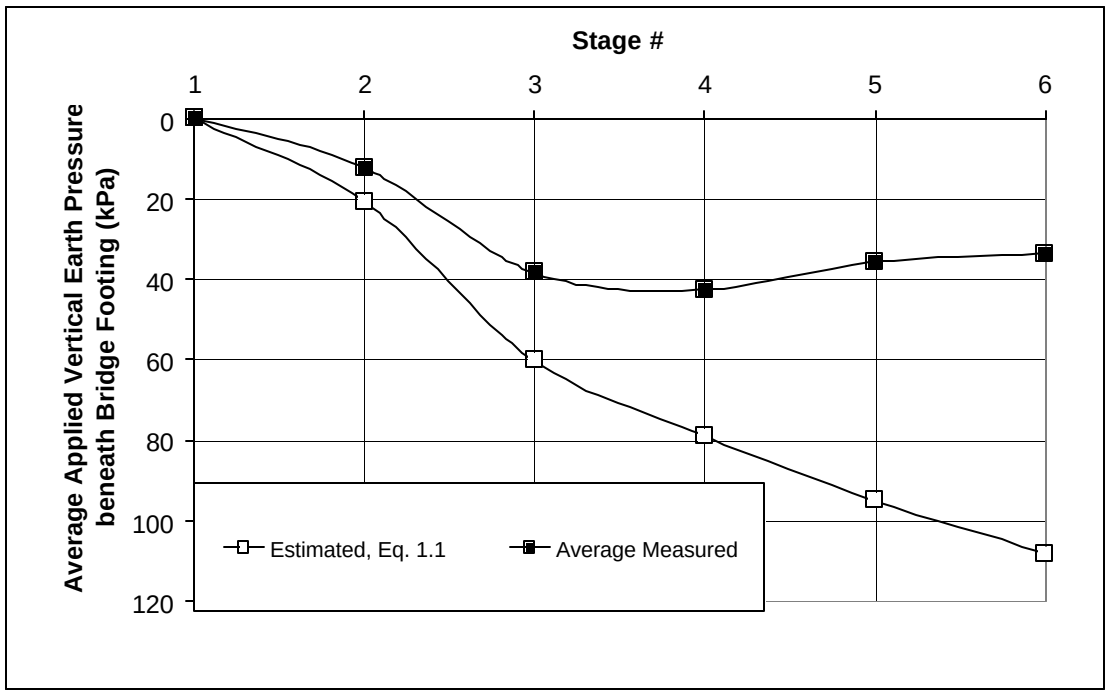


Fig. 3.16. Measured and Estimated Applied Vertical Earth Pressures Beneath the Bridge Footing.

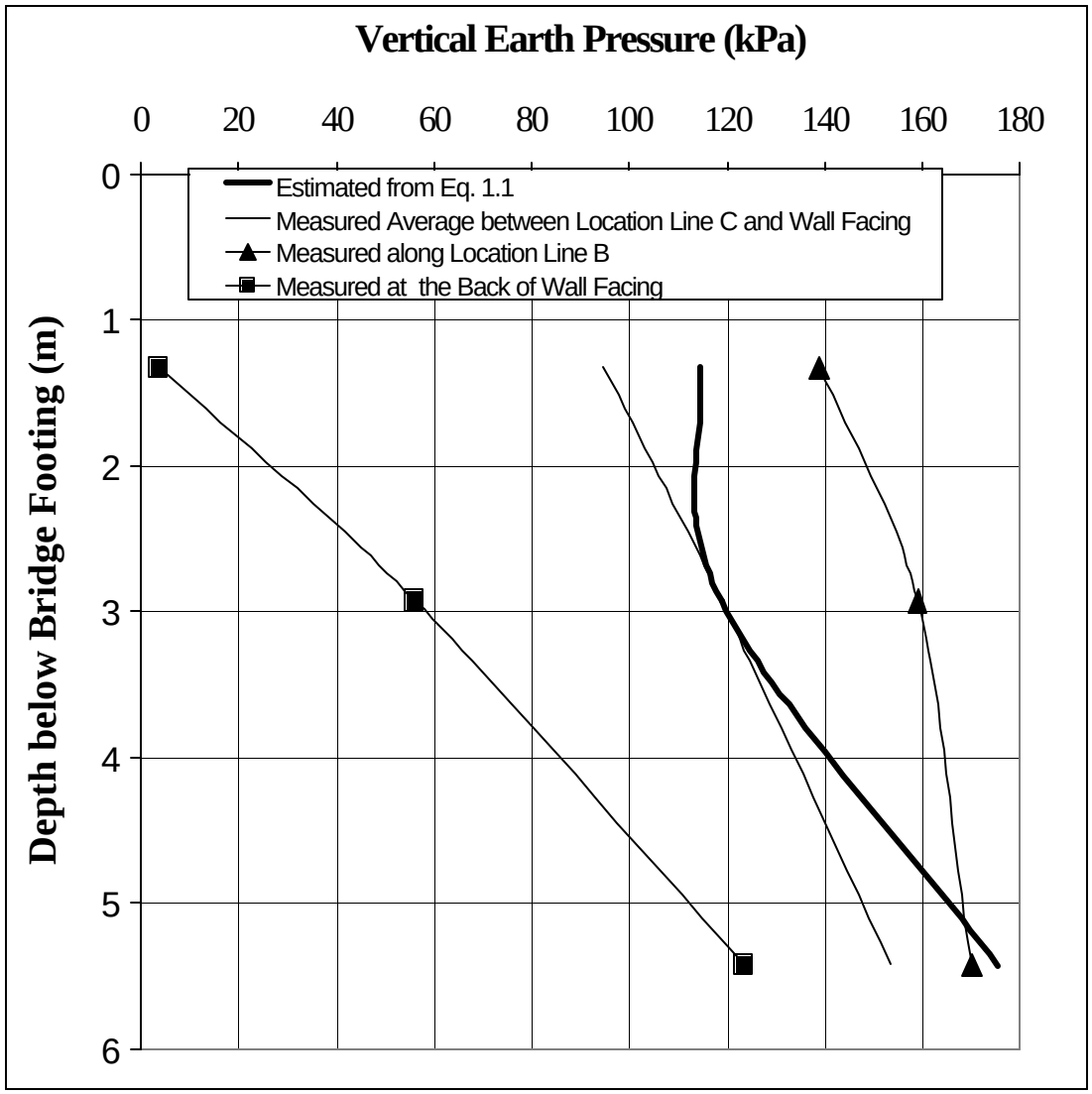


Fig. 3.17. Assumed and Measured Vertical Earth Pressures Beneath the Bridge Footing by the End of Construction.

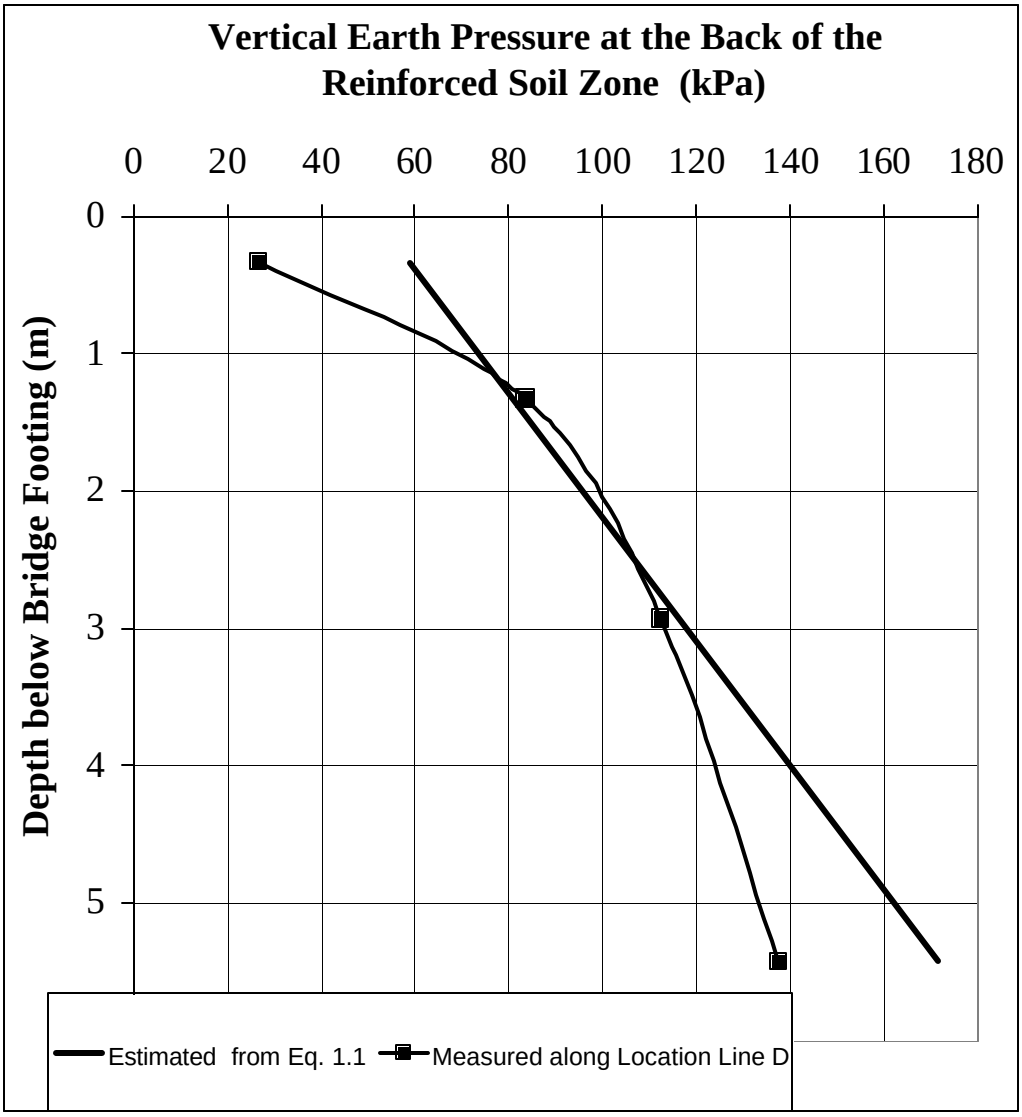


Fig. 3.18. Assumed and Measured Vertical Earth Pressures at the Back of the Reinforced Soil Zone by the End of Construction Stages.

4 LATERAL EARTH PRESSURES AGAINST THE FRONT WALL FACING

4.1 Introduction

Six gages were employed to measure distribution of lateral earth pressures against the facing of the front GRS wall (below the bridge footing) during all monitored stages. One pressure cell was placed along Section 400 of the Phase I Structure (P2, Figure 1.5) and five pressure cells were placed along Section 800 of the Phase II Structure (Table 1.1 and Figure 1.6).

4.2 Results of Pressure Cell P2 Placed along Section 400 during Construction

Measurements for lateral earth pressure on the wall facing of Section 400 (Phase I Structure) from pressure cell P2 were collected manually during all construction stages. Table 4.1 and Figure 4.1 summarize these measurements and the corresponding estimated horizontal and vertical earth pressures at the location of P2 gage (Equations 1.4 and 1.1). Ratios of measured lateral earth pressure to estimated vertical earth pressure are also listed in Table 4.1. This ratio was 2.01 for the first measurement, when just 0.1 m of backfill had been placed and compacted on top of P2 gage, much higher than the design estimated ratio of 0.31 (Table 4.1 and Figure 4.1). As reported in the literature (Section 1.5.3), it seems that compaction operations created lateral earth pressure on the facing that were locked-in after the compaction load was released. The measured lateral earth pressures and the ratio decreased in the subsequent two measurements when an additional 0.8 m of the backfill (vertical earth pressure of 17.7 kPa) was placed (Table 4.1). The decrease in horizontal earth pressure on the facing can be attributed to a small outward displacement of the facing (wall yielding). As additional backfill was placed, the lateral earth pressure on the facing increased but the ratio decreased. A constant ratio of approximately 0.11 was achieved from the end of Stage I to Stage VI. This ratio (0.11) is much less than the estimated design value of 0.31. This implies that current design procedures overestimate the loads carried by the facing.

Table 4.1. Measured and Estimated Lateral Earth Pressures against the Front Wall Facing of Section 400 (Measured from Pressure Cell P2).

Stage #	Estimated Vertical Earth Pressure from Eq. 1.1. (kPa)	Measured Lateral Earth Pressure on the Wall Facing (kPa)	Ratio of Lateral to Vertical Earth Pressure	Estimated Lateral Earth Pressure from Equation 1.4 (kPa)
I	0	0		0
I	2.3	4.5	2.01	0.7
I	11.3	2.8	0.25	3.49
I	20.3	2.7	0.13	6.28
I	24.7	4.8	0.19	7.67
I	47.2	5.6	0.12	14.64
I	62.9	7.3	0.12	19.5
II	75.7	8.5	0.11	23.45
IV	111.6	11.4	0.10	34.6
VI	129.6	14.7	0.11	40.4
VII	149.9	15.0	0.10	46.5

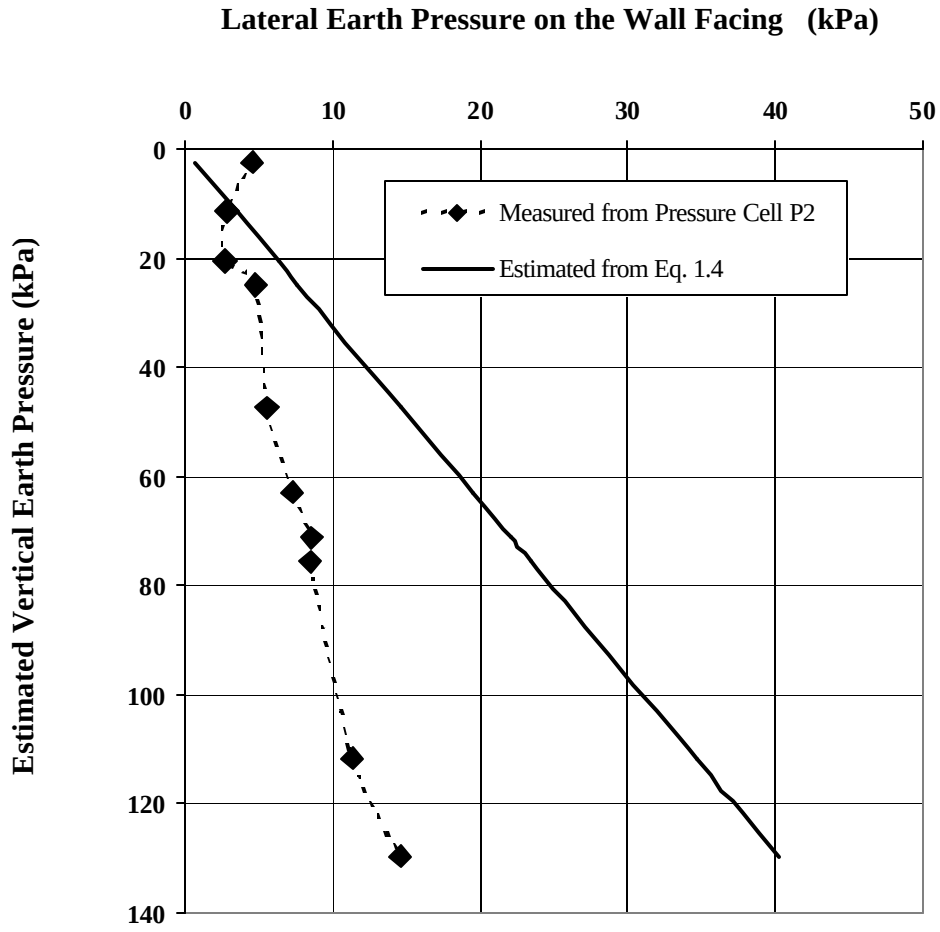


Figure 4.1. Measured and Estimated Lateral Earth Pressures against the Front Wall Facing of Section 400 during Construction (Measured from Pressure Cell P2).

4.3 Results of Pressure Cells Placed along Section 800 during Construction

Figures 4.2 and 4.3 show the time records of the measured lateral earth pressure against the wall facing of Section 800 during all construction stages (i.e., from days # 55 to day # 180 from Jan. 1., 1999). Continuous abrupt (not smooth) changes in the measured data (Figures 4.2 and 4.3) can be attributed to changes in air and backfill temperature and continuous movements of the wall facing. When 0.7 m of

backfill was placed and compacted over gages 11HN and 11HS during wall construction (Stage I), a sharp increase in the horizontal lateral earth pressure of 5.5 kPa was developed (Figure 4.3). The corresponding ratio between lateral earth pressure and vertical earth pressure is 0.611, higher than the estimated design ratio of 0.31. This could be attributed to the locked-in influence of the compaction operations as discussed before.

The measured lateral earth pressures against the wall facing at the end of each construction stage from each gage are summarized in Table 4.2 and Figure 4.4. Figure 4.5 shows vertical profiles of the measured and estimated lateral earth pressure against the wall facing by the end of front wall construction (Stage I) and by the end of all construction stages (Stage VI). These profiles are erratic showing no clear trend with the depth. This can be attributed to differences in the movement of the segmental wall at different levels. Figure 4.5a indicates that the compaction-induced stresses by the end of wall construction (Stage I Construction) remained at the top layer of the wall.

Gage 12H, located close to the top of the wall directly beneath the bridge footing, registered very small lateral earth pressure during all construction stages. This suggests the wall facing directly below the bridge footing did not feel the surcharge loads applied during placement of the bridge superstructure (Stages II to VI). It is possible that the top reinforced soil zone of the wall experienced relatively large lateral displacements during placement of the bridge superstructure, allowing the geogrid reinforcements to be mobilized and support most of the lateral earth pressure loads, thus taking the lateral earth pressure off the wall facing.

Most of the gages responded well to loading during Stages II to IV, especially gages 9H and 11HS (Figures 4.2 to 4.4 and Table 4.2). Sharp increases in the measured lateral earth pressures can be noticed in Figures 4.2 and 4.3 around days 69 and 70 when girders were placed (Stage III). It seems that the wall facing supported relatively large lateral earth pressures during these stages because the wall system was relatively rigid during Stages II, III, and IV (see Chapters 2 and 5).

During Stage V, the measured lateral earth pressure on the wall facing decreased for gages 9H, 11HN, and 11HS and showed no change for Gage 7H (see Figure 4.4 and Table 4.2). The reduction in the lateral earth pressures can be attributed to the relatively large lateral movements of the reinforced soil wall system during Stage V (see Chapter 2). These movements allowed the geogrid reinforcements to be mobilized and support a larger proportion of the lateral earth pressure loads, thus taking off the lateral earth pressure from the wall facing. This behavior also correlates very well with the measured vertical earth pressure (Chapter 3) and geogrid strains (Chapter 5) close to the wall facing during Stage V.

As increasing vertical loads were applied during Stage VI, the lateral earth pressures against the wall facing increased again (Figure 4.4). A ratio between the lateral and vertical earth pressure of approximately 0.04 to 0.08 was achieved towards the end of all construction stages (Stage VI). This ratio is much smaller than the ratio of 0.31 assumed in the design process. Figure 4.5b indicates clearly that the design procedures overestimate considerably the lateral earth loads carried by the facing.

Except for gage 7H, changes in the lateral earth pressure on the wall facing induced by placements of the bridge superstructures (Stages II to VI) were small (Figure 6.4). This response correlates very well with the small increase in the measured vertical earth pressure close to the wall facing due to the placement of the bridge superstructure (Chapter 3) and could be attributed to the large wall movements during Stage V (see Chapter 2).

Table 4.2. Measured Lateral Earth Pressures against the Front Wall Facing of Section 800 at the End of Each Monitored Stage.

Stage	I	II	III	IV	V	VI	VII
Gage #							
7H*	0.5	0.5	1.0	2.0	2.0	4.4	5.9
9H	1	1.5	4.0	6.0	0.5	2.4	3.1
11HN	6.0	6.5	6.7	7.0	6.0	8.5	9.3
11HS	5.0	6.5	8.0	9.0	6.0	7.8	8.4
12H	0	0.5	0.5	0.5	0.5	1.5	0.6
*Gage placed at elevation of 2.74 m above leveling pad but first data was collected at elevation of 4.27 m (see Table 2.5)							

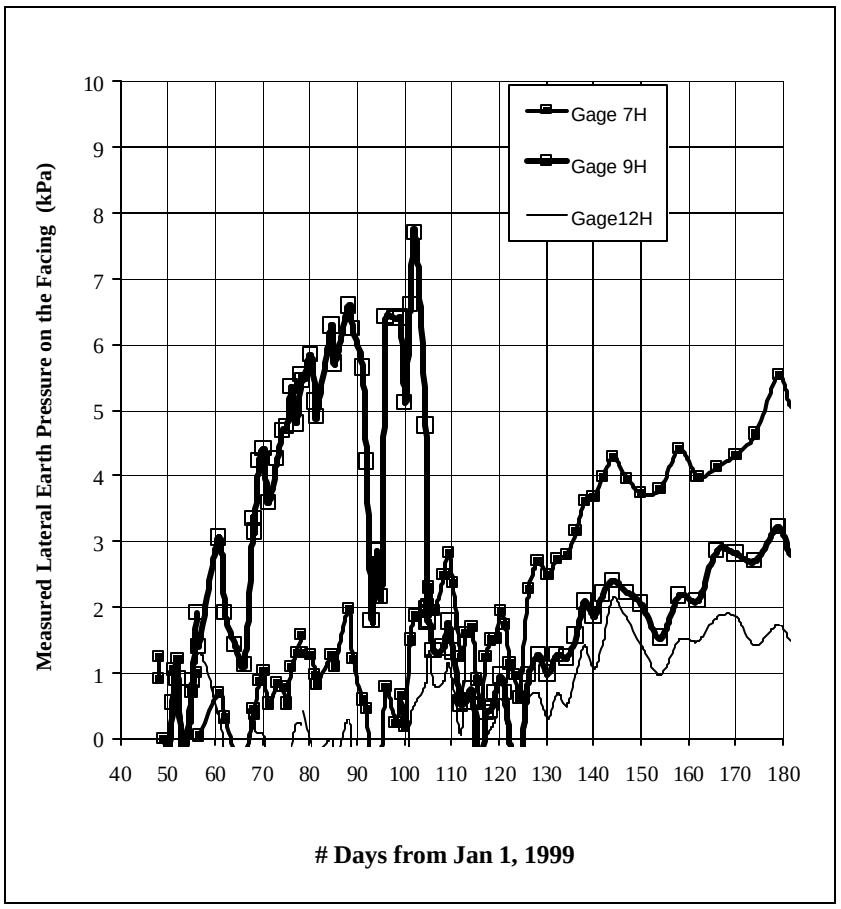


Figure 4.2. Time Records for the Measured Lateral Earth Pressure against the Wall Facing of Section 800 during all Construction Stages (from Gages 7H, 9H, and 12 H).

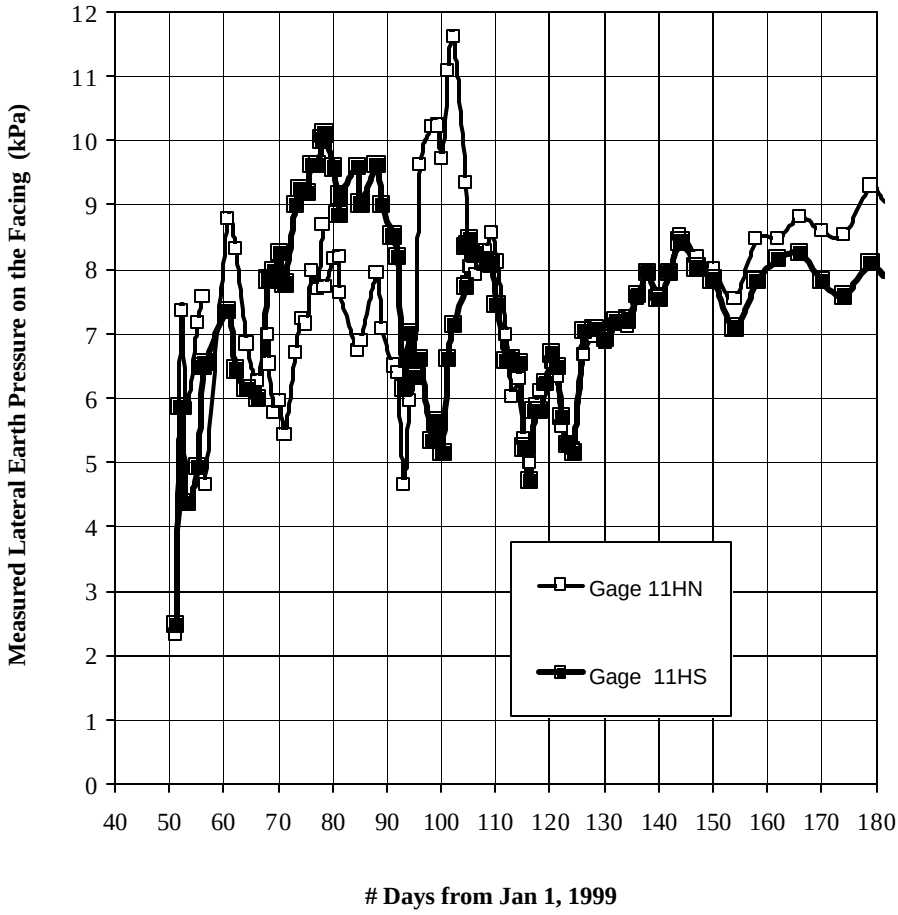


Figure 4.3. Time Records for the Measured Lateral Earth Pressure against the Front Wall Facing of Section 800 during all Construction Stages (from Gages 11 HN and 11HS).

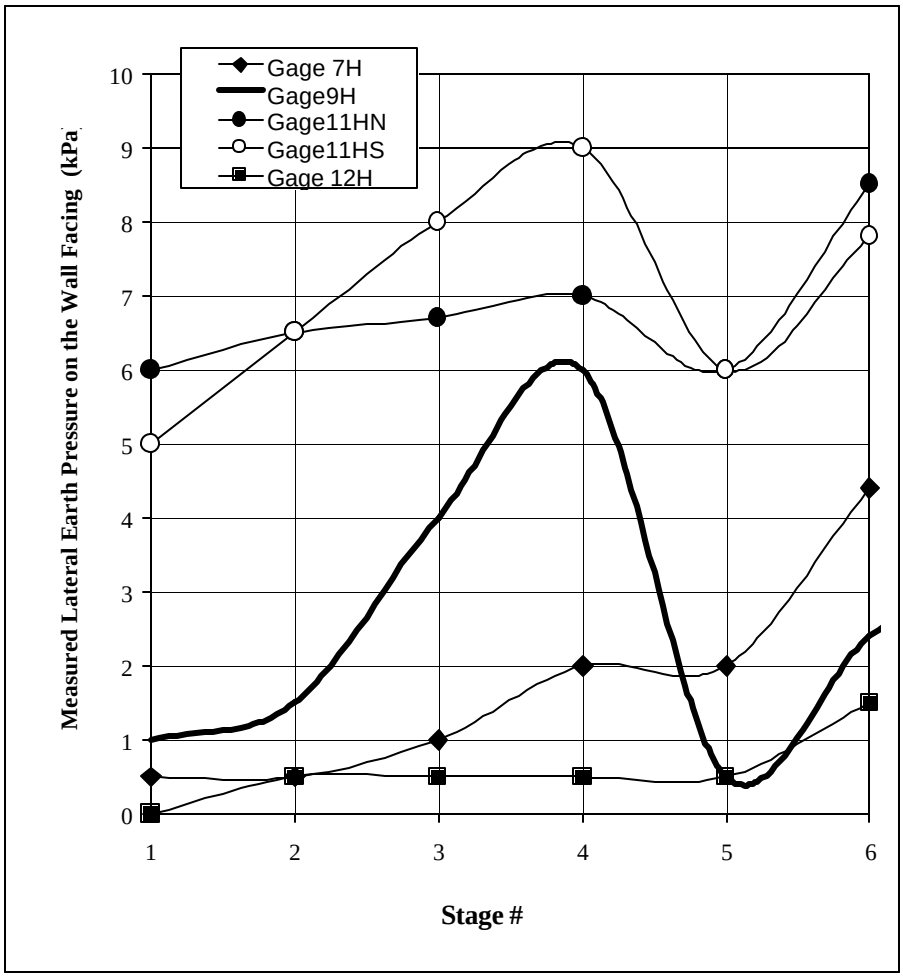


Figure 4.4. Lateral Earth Pressure against the Front Wall Facing of Section 800 Measured by the End of each Construction Stage.

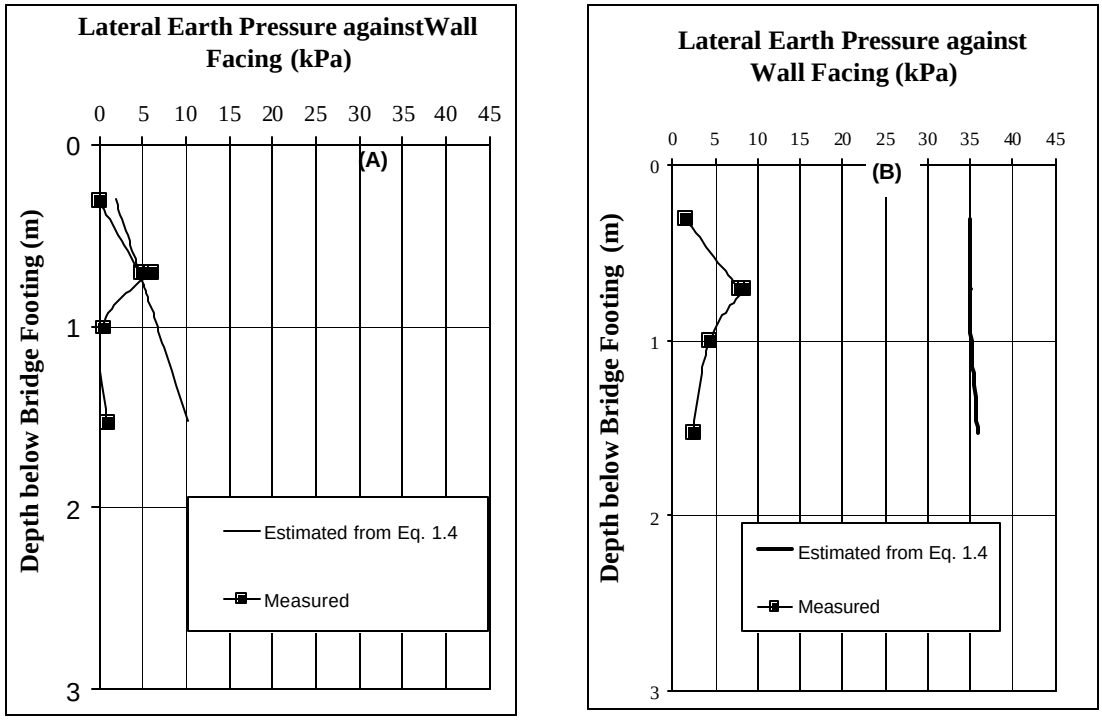


Figure 4.5. Measured and Estimated Profiles of Lateral Earth Pressure against the Front Wall Facing of Section 800: a) by the End of Front GRS Wall Construction (Stage I), and b) by the End of all Construction Stages (Stage VI).

4.4 Results while Structure was in Service (Stage VII)

Typical time records for the measured lateral earth pressure against the wall facing while the bridge was in service for one year (Stage VII, days 180 to 545 from Jan. 1, 1999) are shown in Figure 4.6. The fluctuations in the measured values can be attributed to changes in air and backfill temperature and continuous movements of the wall facing. Tables 4.1 and 4.2 list the values of the measured lateral earth pressures for Stage VII. The Stage VII results in the tables represent average values of data measured over a year while the bridge was in service. Along Section 400 (Table 4.1) the measured increase in lateral earth pressure during Stage VII was a very small value of 0.3 kPa. Along Section 800 (Table 4.2) the measured increase in lateral earth pressure was also small for most of the gages (less than 0.7 kPa, except Gage 7H). The increase in lateral earth pressure while the structure was in service was relatively large at the location of Gage 7H (1.5 kPa, see Table 4.2). The small increase in the measured lateral earth pressure during Stage VII along Section 800 occurred immediately after opening the bridge to traffic (see Figure 4.6), possibly due to placement of minor structures on the bridge structure (even after opening the bridge to traffic).

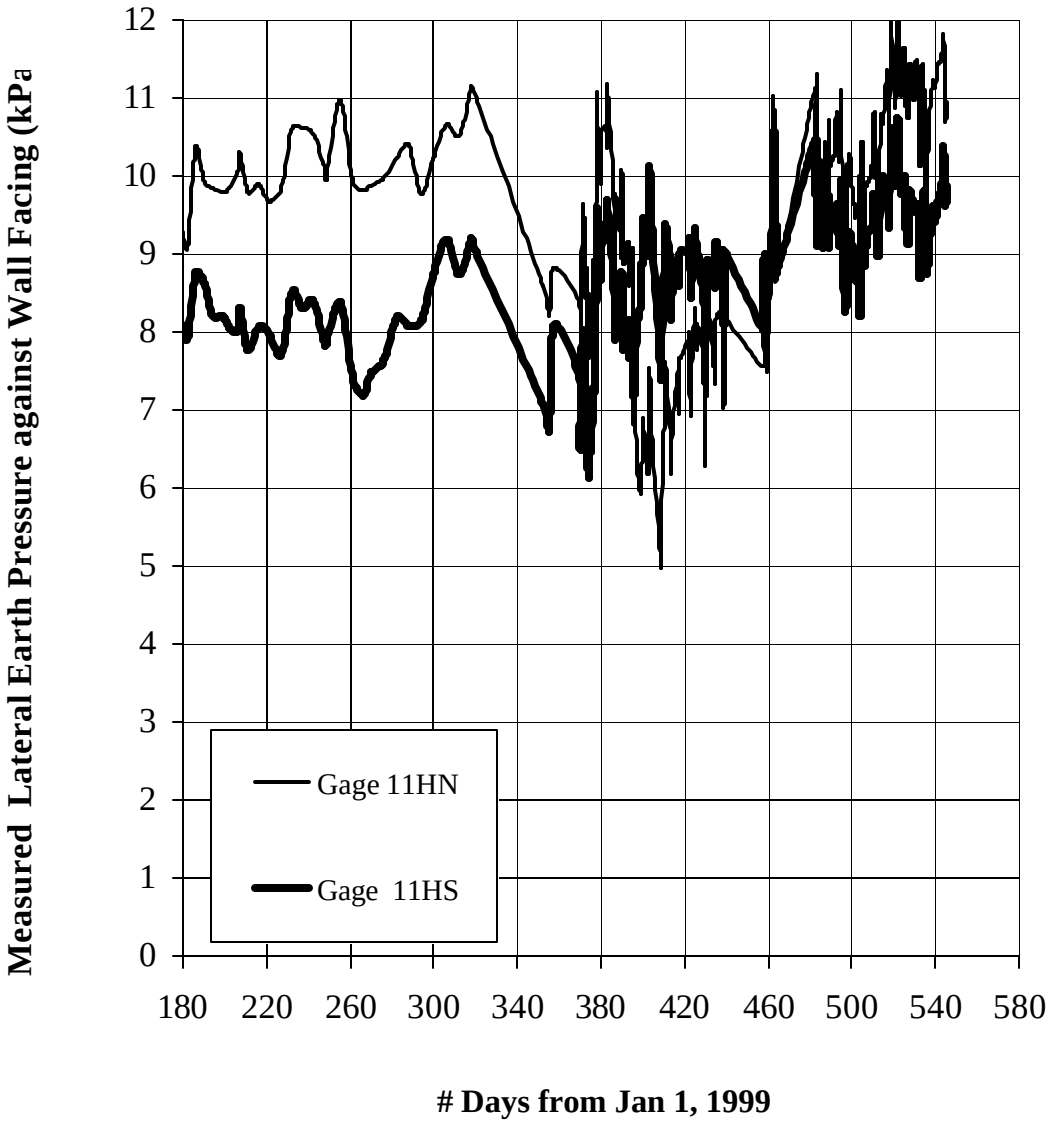


Figure 4.6. Typical Time Records for the Measured Lateral Earth Pressure against the Front Wall Facing of Section 800 while the Structure was in Service.

4.5 Consistency of the Measured Data

The consistency of the measured data is investigated by comparing the results of Gages 11HN and 11HS, placed at the same elevation. Figure 4.6 indicates that the results from these two gages run parallel and very close to each other. The measured lateral earth pressures from gages 11HN and 11HS and other gages are not seem to be influenced by seasonal changes in temperatures (see Figure 4.6). The measured lateral earth pressures on the facing correlate with the applied loads during various construction stages as discussed in previous sections. Sharp increases in the measured lateral earth pressures can be noticed in Figures 4.2 and 4.3 around days 69 and 70 when girders were placed (Stage III). As will be summarized in Chapter 6, the measured lateral earth pressures correlate well with the vertical earth pressures and geogrid lateral strain measured close to the wall facing. Based on the above discussion, it is fair to conclude that the measured results for the lateral earth pressure on the wall facing are reliable and consistent during the construction stages.

4.6 Summary and Assessment of the Design Procedure

Compaction operations induced comparatively large earth pressure against the wall facing. Some of the compaction induced pressure remained locked-in after removal of the vertical compaction loads. Large ratios of lateral to vertical earth pressure of 2.01 and 0.61 were measured along sections 400 and 800, respectively, during initial stages of placement and compaction of backfill over the gages (assumed value in the design is 0.31). The design procedure (Equation 1.4) does not account for the influence of compaction. It therefore underestimated the developed lateral earth pressure against the wall facing during initial stages of backfill placement and compaction. For facing with low connection strength or low interface shear strength, the compaction loads might cause excessive deformation or improper alignment to the facing. It is estimated that the compaction-induced lateral earth pressures on the wall facing exceed the assumed active earth pressure over a depth, approximately, less than 1 m. The ratio between lateral to vertical earth pressure dropped to an almost constant value of 0.11 (Table 4.1) after almost 2 meters of backfill were placed and compacted over the gage.

The measured changes in lateral earth pressure against the wall facing while the structure was in service were small and much lower than the values estimated in the design. Live loads due to traffic loading

account for a scenario such as a line of fully loaded trucks suddenly stopping on top of the bridge during a traffic jam. It seems this scenario did not occur during Stage VII because the instruments certainly did not capture it. In other words, the collected data cannot verify design assumptions regarding the effect of live traffic load on the horizontal loading of the wall facing and therefore will not be utilized for assessment of the design procedure. The lateral earth pressures against wall facing measured by end of construction will be utilized for assessment of the design procedure. These data were proven reliable in the previous section.

Table 4.3 summarizes the measured and estimated (Eq. 1.4) lateral earth pressure by the end of all construction stages from gages placed along Sections 400 and 800, and the ratio between measured to estimated values. The measured lateral earth pressures against the wall facing range from 4.2 % to 36.4% of the values estimated in the design. The ratio between lateral to vertical earth pressure at the end of construction stages was 0.11 for section 400 and 0.04 to 0.08 for Section 800 (estimated ratio in the design is 0.31). Hence, it can be concluded the current design procedure overestimates the lateral earth pressures against the front GRS wall facing by almost three times.

The very low earth pressure measured against the wall facing suggests a behavior close to the behavior of a GRS wall with a yielding lateral boundary rather than with a rigid lateral boundary (McGown et al., 1998, see Section 1.5.4). The relatively flexible wall system allowed for the comparatively larger mobilization of the friction resistance of the backfill and the support of more lateral earth pressure by reinforcements, thus taking more lateral earth pressure off the wall facing. Smaller lateral earth pressures were measured against the wall facing along Section 800 than along Section 400. The large difference occurred, most likely, because Section 800 experienced more lateral movements than Section 400 (see Chapter 2). The lateral earth pressures against the wall facing of section 800 declined significantly during Stage V construction when the wall facing experienced large outward displacements (see Chapter 2).

The design procedure may incorrectly estimate the lateral earth pressure against the wall facing because:

- The true shear strength of the backfill is higher than what is assumed in the design (see section 1.5.2). A backfill with higher shear strength will produce smaller lateral earth pressure.

- The wall behavior and response was close to the behavior of GRS walls with yielding lateral boundary, thus reducing the earth pressure against the wall facing as discussed previously.

Table 4.3. Measured and Estimated Lateral Earth Pressure (kPa) against the Front Wall Facing by the End of Construction.

Gage #	By the end of Construction (End of Stage VI)		
	Estimated	Measured	Percentage Ratio of Measured to Estimated Values (%)
7H	38.8	4.4	11.3
9H	35.9	2.4	6.7
11HN	34.9	8.5	24.3
11HS	34.9	7.8	22.3
12H	35	1.5	4.2
P2	40.4	14.7	36.4

5. GEOGRID STRAINS IN THE FRONT GRS WALL

5.1 Introduction

Figure 1.6 and Table 1.3 show the locations of all strain gages placed along Section 800 (Phase II Structure) to measure distribution of geogrid lateral strains inside the front GRS wall (below the bridge footing) during all monitored stages. Strain gages attached to geogrid layers # 2, # 6, # 10, and #12 (Figure 1.6) will be referred to as, respectively, layer 2 gages, layer 6 gages, layer 10 gages, and layer 12 gages. Layers 12, 10, 6, and 2 gages were placed at depths of 0.61 m, 1.43 m, 3.05 m, and 4.68 m, respectively, below the bridge footing. Layer 12 gages were placed 0.61 m below the bridge footing. The outward displacements of the front GRS wall facing inferred from strain gages were presented and discussed in Chapter 2.

5.2 Typical Geogrid Strains during Placement and Compaction of the Backfill

Figure 5.1 shows time records of the geogrid strains measured from layer 6 gages during construction of the front GRS wall (Stage I). These data were collected every six minutes to monitor the development of geogrid strains during construction activities. It is interesting to notice that changes to the geogrid strains occurred during working hours and no changes occurred during the evening and night hours (when no work was taking place). These changes seemed to be sharp and occurred in a very small short period (i.e., changes were not distributed over the entire working hours during which backfill was placed and compacted). These changes did not fluctuate.

Figure 5.2 shows the measured strains from geogrid layer 6 during construction of the front GRS wall (Stage 1) versus the estimated applied vertical earth pressure over geogrid layer 6 (Eq. 1.1). The changes in geogrid strains correlate very well with the applied load. Significant increases in geogrid strains occurred during placement and compaction of the initial 1 m of backfill over the gages (~20 kPa in Figure 5.2), especially close to the wall facing (along Location Line A). Comparatively, smaller geogrid strains developed during the rest of Stage 1 Construction. During the compaction and

placement of 1 m of backfill, the responses in geogrid strains were different from Gage 6SBN and Gage 6SBS, which were placed at the same location (Figure 5.2). However, the response from these two gages in subsequent stages was almost parallel. The large increases in geogrid strains and differences in the measured geogrid strains for gages placed at the same location following placement and compaction of backfill over the gages can be attributed to several factors:

- Locked-in strains induced by compaction loads (see Section 1.5.3). The influence of compaction loading might vary across the geogrid layers.
- Presence of slack in the geogrid and differences in initial tensioning of the geogrid.
- Non-stable or erroneous 1st reading (reference) due to the sensitivity of the gages and flexibility of the geogrid layers.
- Bending of the geogrid. If the geogrid layer is placed on uneven ground or experiencing any flexural loading (may be due to uneven backfill settlement or settlement of backfill relative to the facing) one side of the geogrid (top or bottom) will be in tension as the other will be in compression.

The strain gages placed along Section 800 were ordered with high sensitivity (Geokon Model 4050) because the measured geogrid strains on Phase I Structure were small (Abu-Hejleh et al., 2000). During construction stages, several strain gages went off the scale and maxed out yielding no reading. The combination of gage sensitivity, flexibility of the geogrid (i.e., can easily experience vertical deformation), installation, and backfill compaction operation produced the loss of several strain gages.

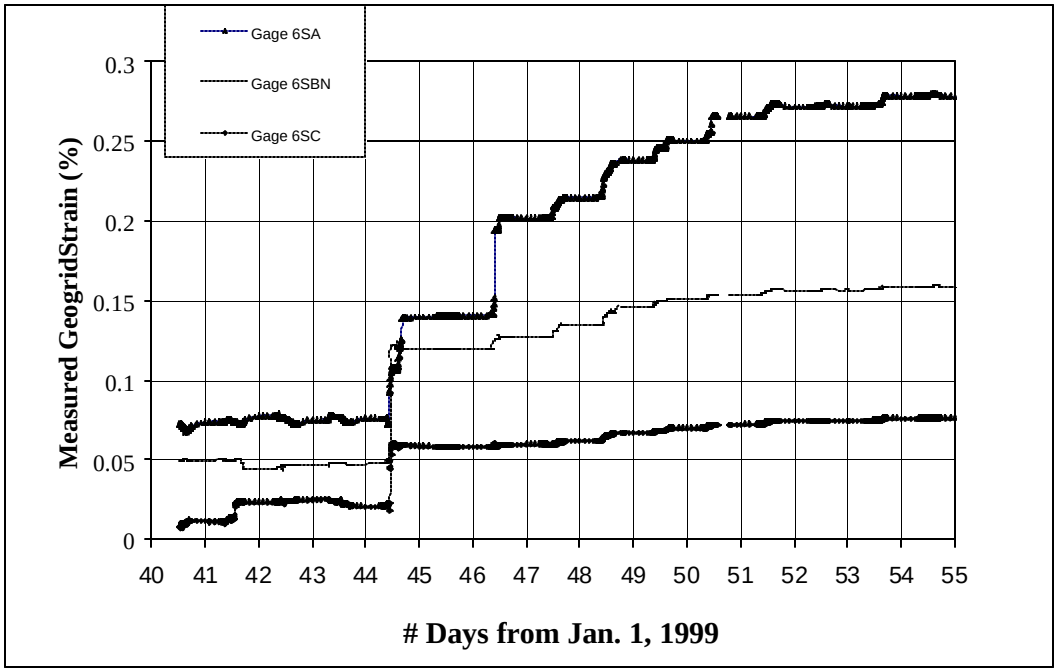


Fig. 5.1. Typical Time Records for Measured Geogrid Strains during Construction of the Front GRS Wall (Stage I).

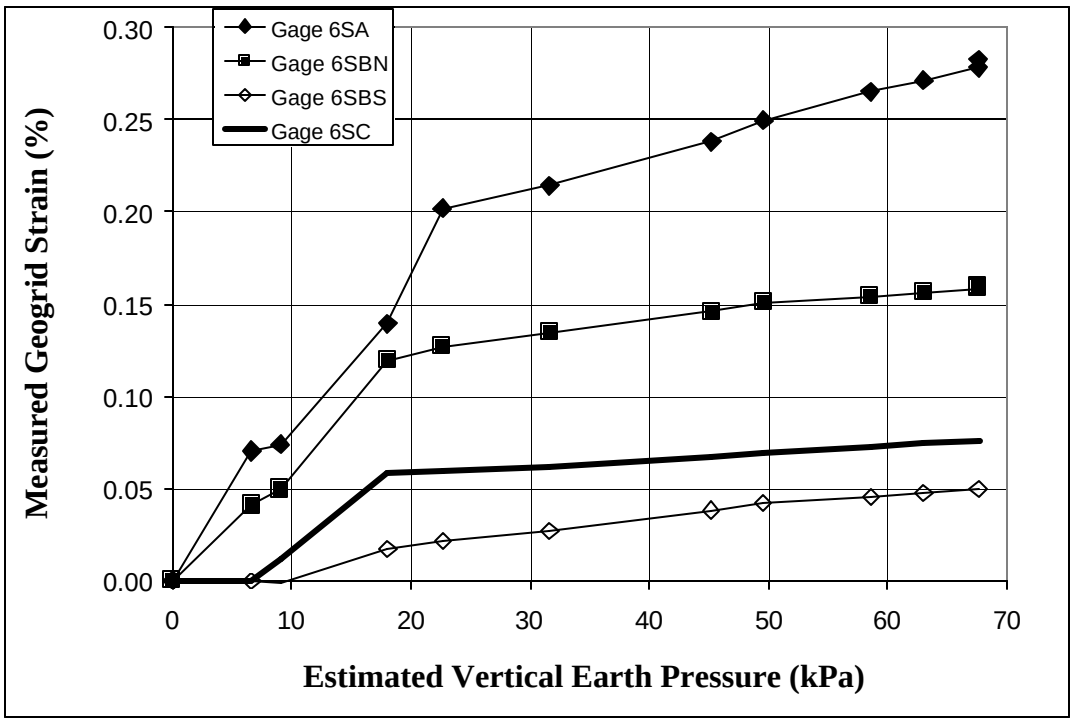


Fig. 5.2. Typical Measured Geogrid Strains vs. Vertical Earth Pressure during Construction of the Front GRS Wall (Stage I).

5.3 Results of Layer 2 Strain Gages during all Construction Stages

Figures 5.3 shows time records for the geogrid strain results measured during all construction stages at geogrid layer 2 (Figure 1.3). Sharp increases of geogrid strains occurred following placement and compaction of the backfill along Location Lines A and B. In all subsequent construction stages, small changes in geogrid strains occurred for gages placed along Location Lines B and C. The measured geogrid strains along Location Line A continued to increase and exceeded those measured along Location Lines B and C (Figure 5.3). Just before the end of Stage I Construction, Gage 2SA went off the scale and maxed out (Figure 5.3). Transmission from Gage 2SB was also lost by the end of construction Stage VI (Figure 5.3).

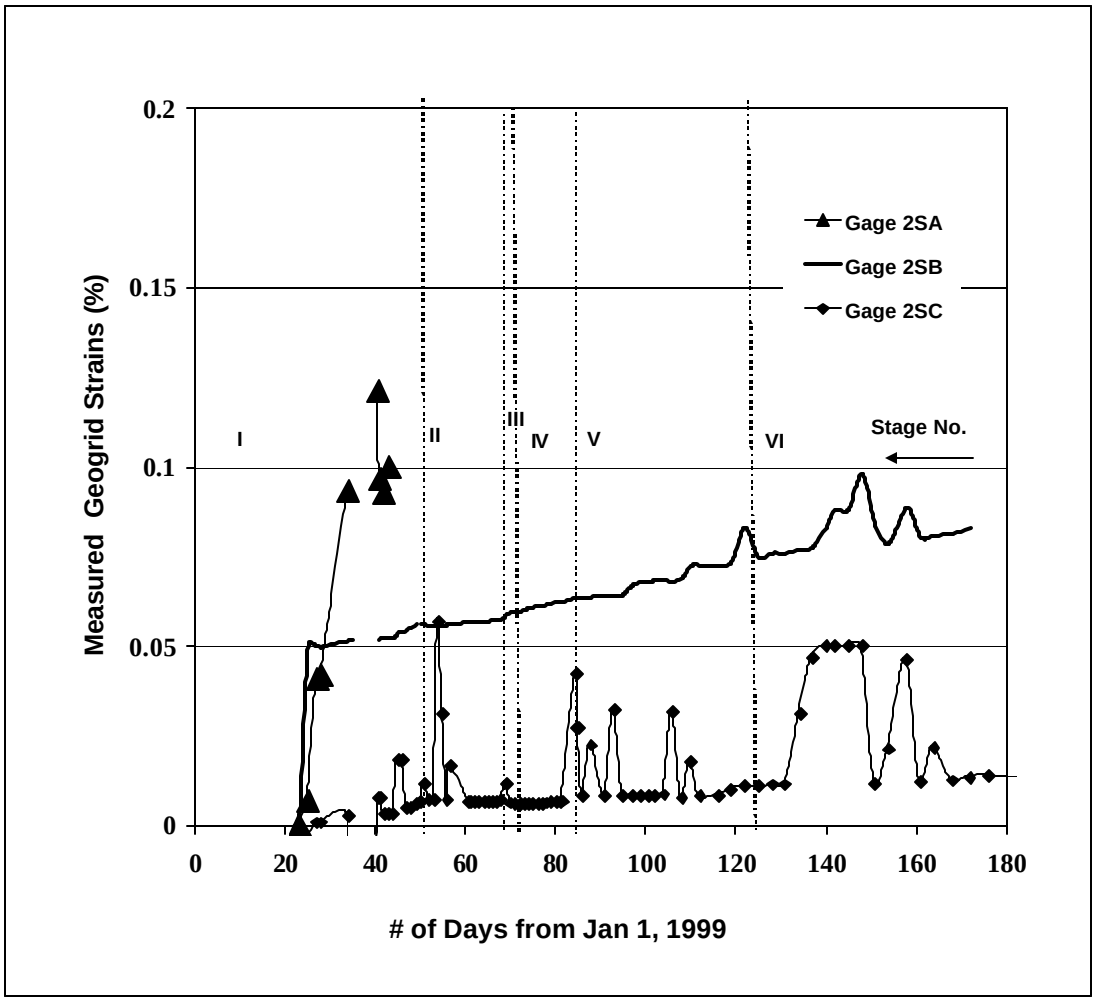


Fig. 5.3. Measured Strains along Geogrid Layer 2 during all Construction Stages.

5.4 Results of Layers 6 and 10 Gages during all Construction Stages

Figures 5.4 and 5.5 show time records for the geogrid strain results measured during all construction stages from geogrid layers 6 and 10, respectively. Gage 6SBN exceeded the measuring range at the end of Stage VI (Figure 5.4). The strain readings from Gage 6SA exceeded the measuring range during Stage IV, seemed to fall down and become within the measuring range at the end of Stage V and most of Stage VI, but exceeded the measuring range by the end of Stage VI (Figure 5.4). The missing strain data for Gages 6SBN and 6SA at the end of Stage VI were estimated based on extrapolation from their data at previous stages and from strain data obtained from Gage 6SBS. A summary of the obtained strains along geogrid layers 6 and 10 at different construction stages is shown in Figure 5.6. The geogrid strain measurements at different stages as a function of the applied vertical earth pressures over geogrid layers 6 and 10 (Eq. 1.1) are shown in Figure 5.7. The geogrid strain measurements at locations lines A and B were extrapolated to estimate the geogrid strains at the rear side of the wall facing. Horizontal profiles of the total geogrid strains, and *change*s in geogrid strains developed due to the placement of the bridge superstructure (Stages II to VI from Stage I), measured from layers 6 and 10 gages, are given in Figures 5.8 and 5.9, respectively.

Gage 10SD registered the development of a very small compressive strain in geogrid following the initial placement and compaction backfill over that gage (Figure 5.5 and 5.6). This could be attributed to bending of the geogrid layer and/or non-stable or erroneous 1st reading (reference reading). It is possible also that the compressive strains measured in the geogrid were real. It is difficult to imagine the development of compression strains in the geogrid while it was isolated in the air, but it could be possible when geogrid became an integral part of the reinforced soil mass. This gage experienced an increase of geogrid tensile strains when additional backfill was placed and during all subsequent construction stages, but registered very small compressive strains in the geogrid by end of construction stages. This, as discussed before, could be attributed to erroneous reference readings.

During Stages II to VI, geogrid layers 6 and 10 at different Location Lines responded with comparatively small deformations and strains to the increasing vertical earth pressures (Figures 5.6 and

5.7, see Chapter 2). The highest strains occurred close to the wall facing during Construction Stages I through IV (Figure 5.8).

During Stages V and VI, the GRS system along layers 6 and 10 appears to have responded with comparatively large strains and displacements to the increasing level of applied vertical earth pressures (Figures 5.6 and 5.7, see Chapter 2). As shown in Figure 5.9, the change in geogrid tensile strains at Location Line A (behind wall facing) due to the placement of the bridge superstructure (Stage II to VI from Stage I) slightly increased along geogrid layer 6 and declined sharply along geogrid layer 10. Most of the geogrid tensile straining during Stage V occurred inside the interior portion of the reinforced soil mass along Location Line B for layer 6 (Figure 5.4 and 5.6) and along Location Lines C and B for layer 10 (Figures 5.5 and 5.6). By the end of Stage VI, the maximum strain occurred at Location Line C for layer 10 (Figure 5.8b and 5.9b). Along geogrid layer 6, the maximum strain occurred at Location Line A (0.4%) which was slightly higher than that measured along Location Line B (0.37%) as demonstrated in Figure 5.8a. However, the increase in geogrid tensile strains due to the placement of the bridge superstructure (Stages II to VI) for layer 6 (Figure 5.9a) at Location Line B (0.23%) was much larger than that at Location Line A (0.097%).

Along layer 10, the geogrid tensile strains at Location Line A dropped during Stages V and VI, while the strains at Location Lines C and B were increasing at a comparatively very high rate during these two stages (Figure 5.5 and 5.6). This severe reduction in geogrid strain at Location Line A close to the wall facing measured from gage 10SA correlates very well with the reduction of both the measured lateral earth pressure against the wall facing from Gage 11HN (see previous Chapter) and the measured vertical earth pressures behind the wall facing from Gage10VA (see Chapter 3). The significant increase of geogrid tensile strain along Location Lines B and C from Gage 10SB and 10SC during Stage V correlates well with the significant increase in vertical earth pressures measured from Gages 10VBN and 10VC (Figure 3.6).

Along geogrid layer 6, a very small increase in the geogrid tensile strains occurred at Location Line A during Stage V (Figure 5.6a). This seems to correlate well with the no change in the measured lateral

earth pressure against the wall facing from Gage 7H during this stage. At the same time, the strains at Location Line B were increasing at a comparatively high rate (Figure 5.4). This correlates well with the comparatively high rate of increases in the measured vertical earth pressure from Gage 6VBN during this stage (Figure 3.4).

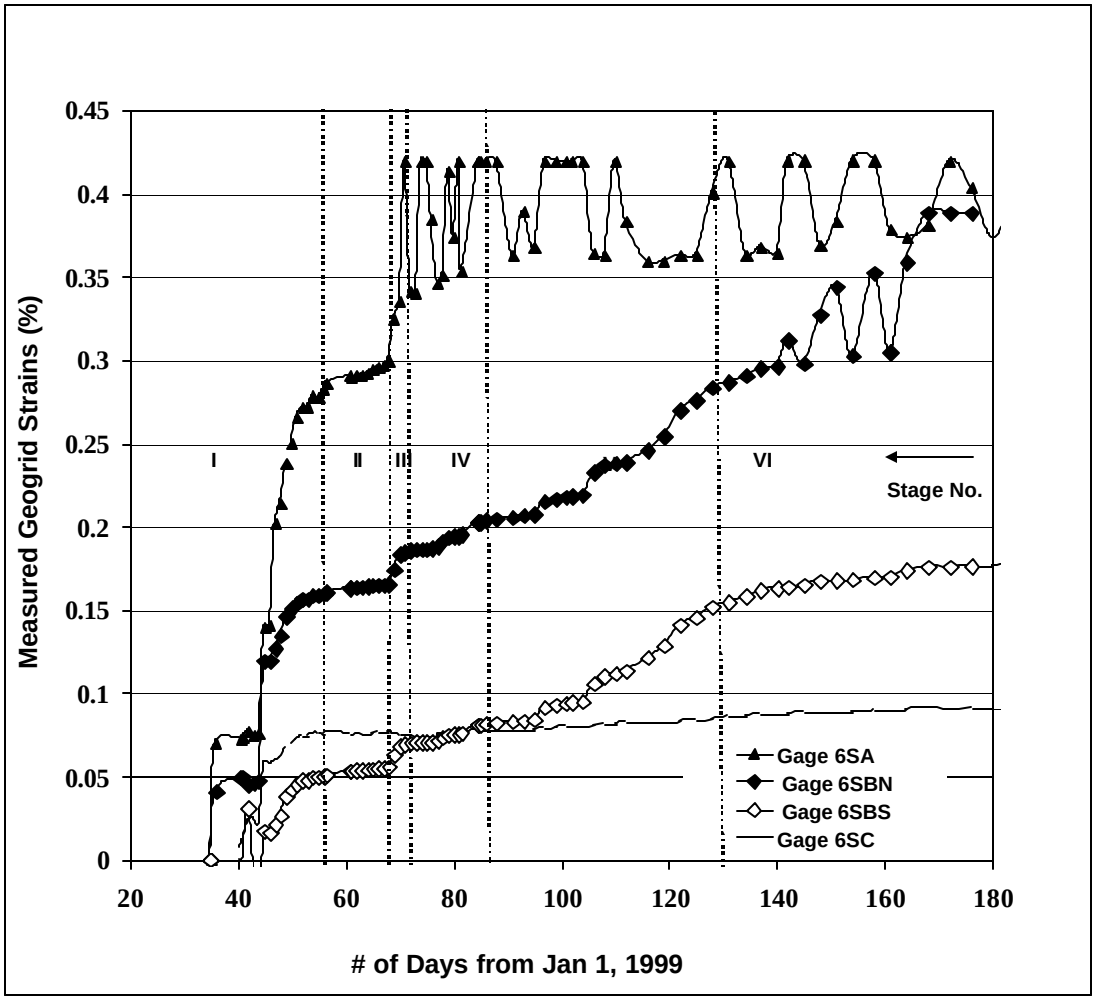


Fig. 5.4. Measured Strains along Geogrid Layer 6 during all Construction Stages.

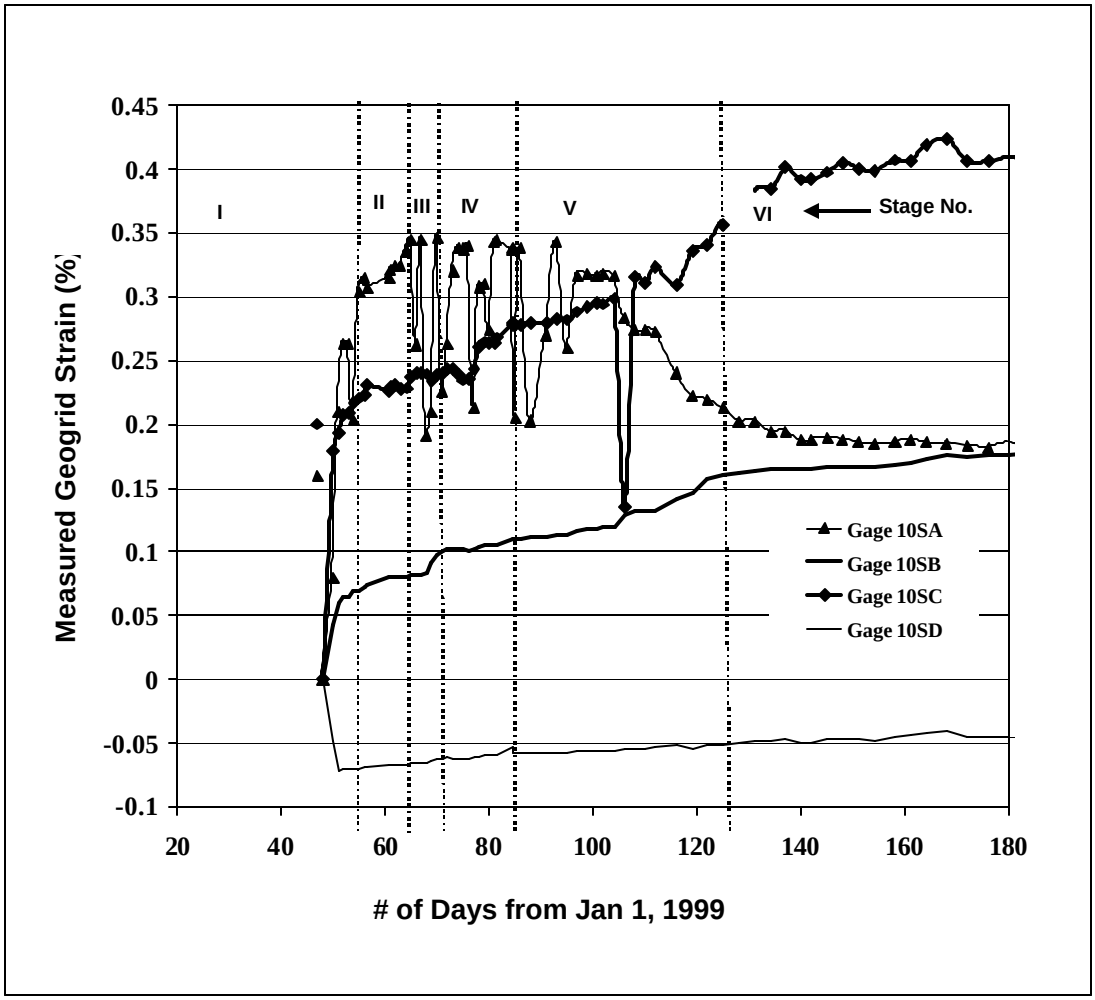


Fig. 5.5. Measured Strains along Geogrid Layer 10 during all Construction Stages.

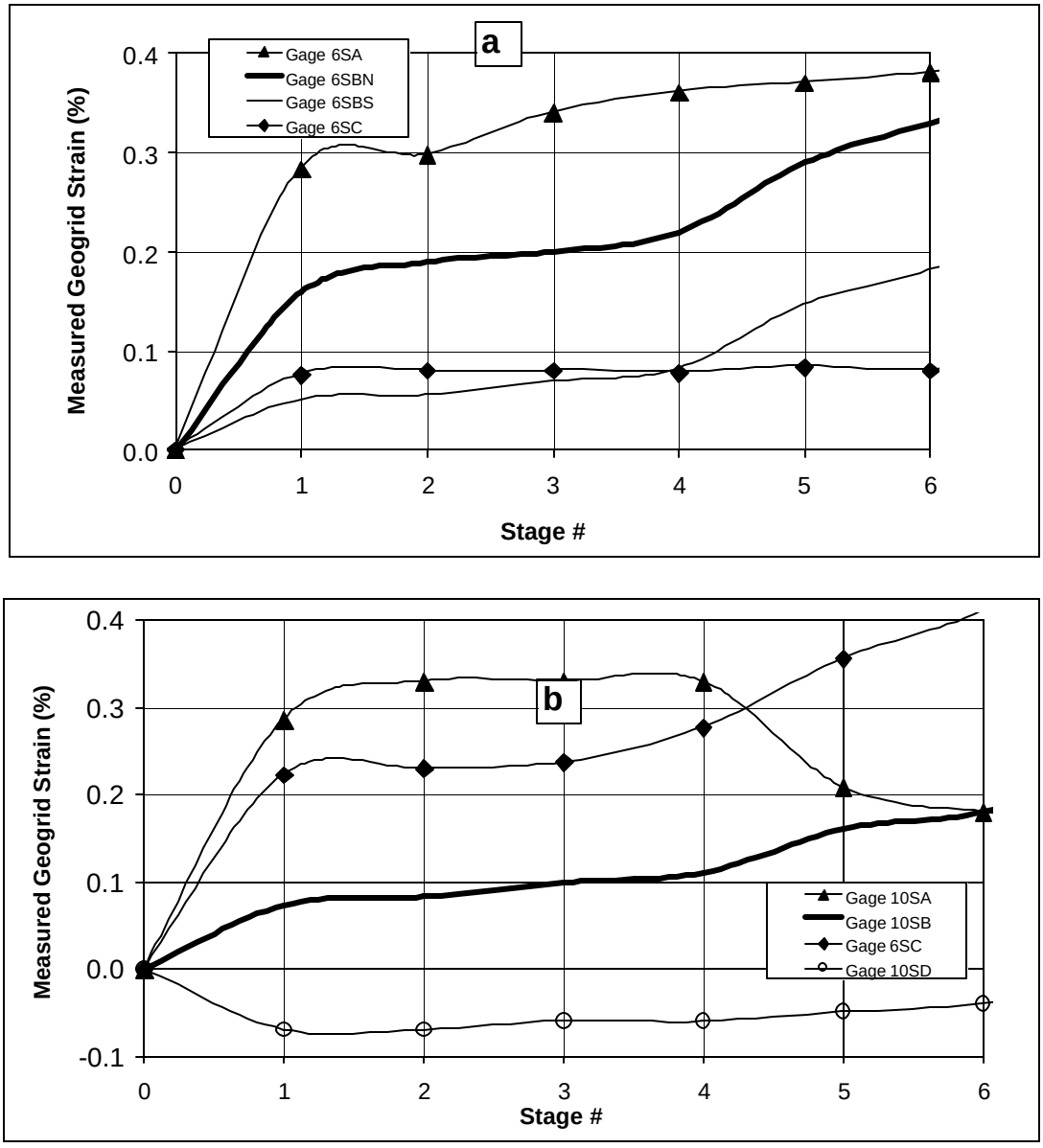


Fig. 5.6. Measured Geogrid Strains at Various Construction Stages along a) Geogrid Layers 6, and b) Geogrid Layer 10.

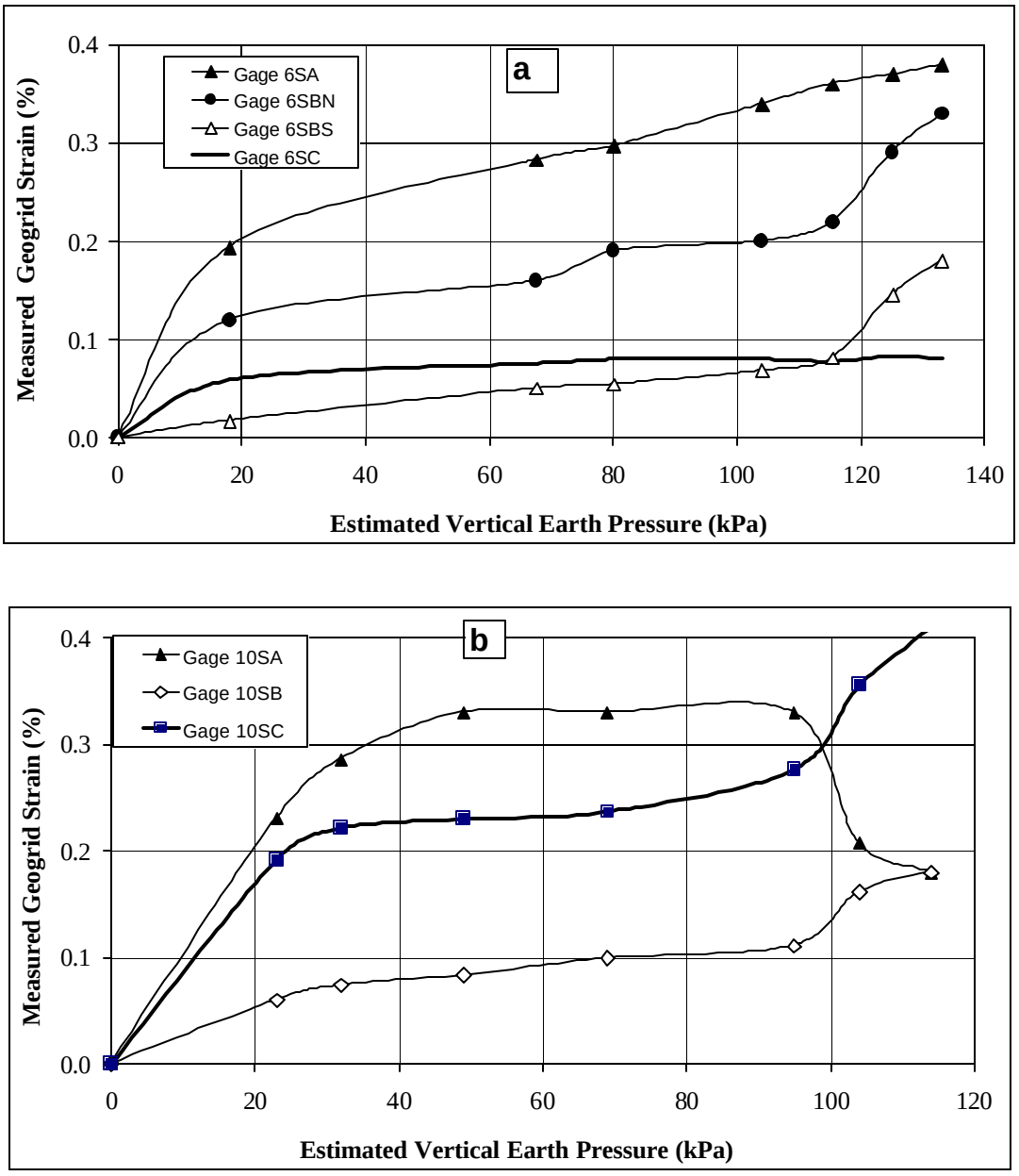


Fig. 5.7. Measured Geogrid Strains vs. Vertical Earth Pressure during all Construction Stages along a) Geogrid Layer 6, and b) Geogrid Layer 10. Note : 3rd data point refers to Stage I Construction, 4th data to Stage II and so on.

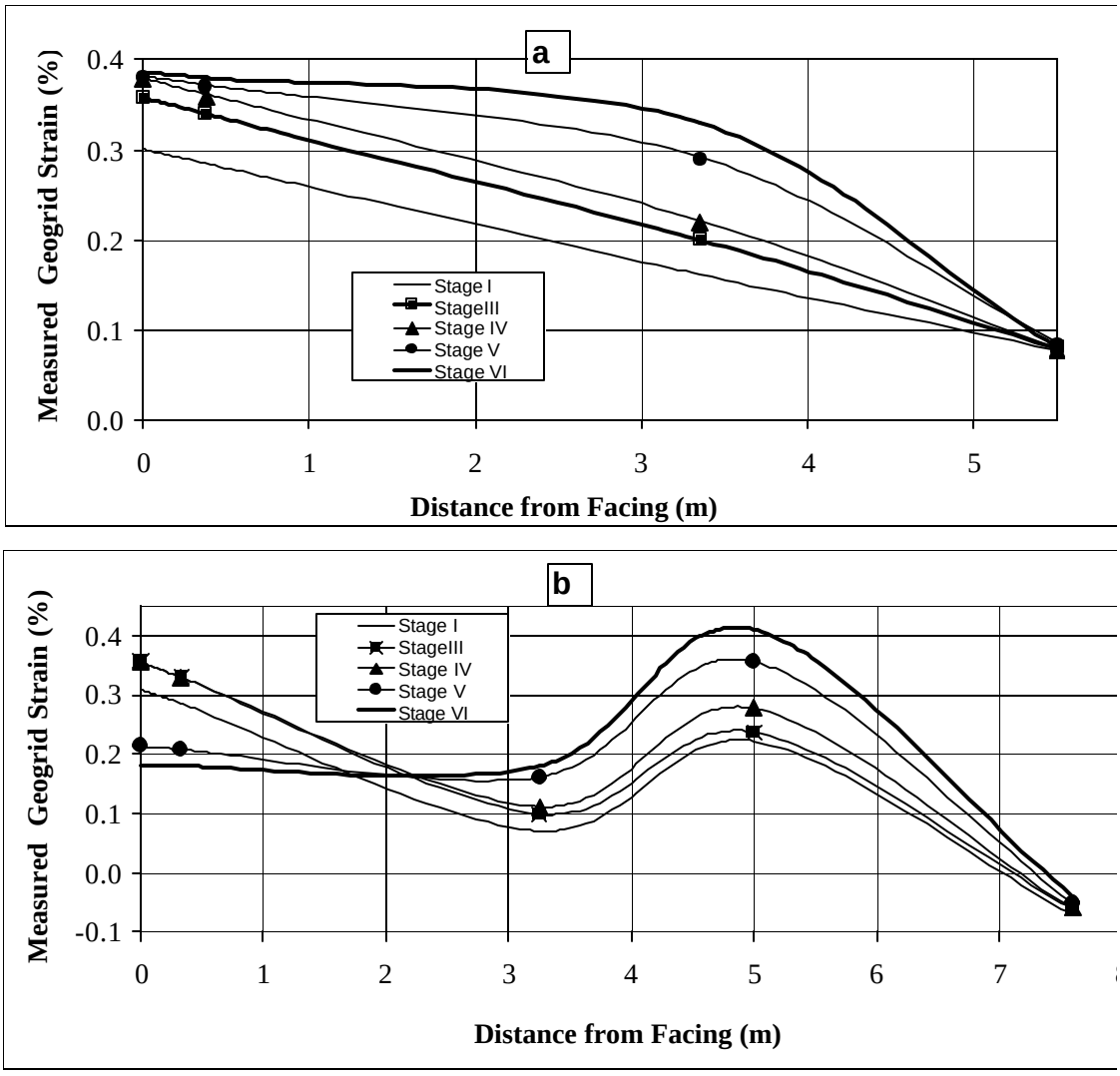


Fig. 5.8. Horizontal Profiles of the Measured Total Geogrid Strains during Various Construction Stages along: a) Geogrid Layer 6, and b) Geogrid Layer 10.

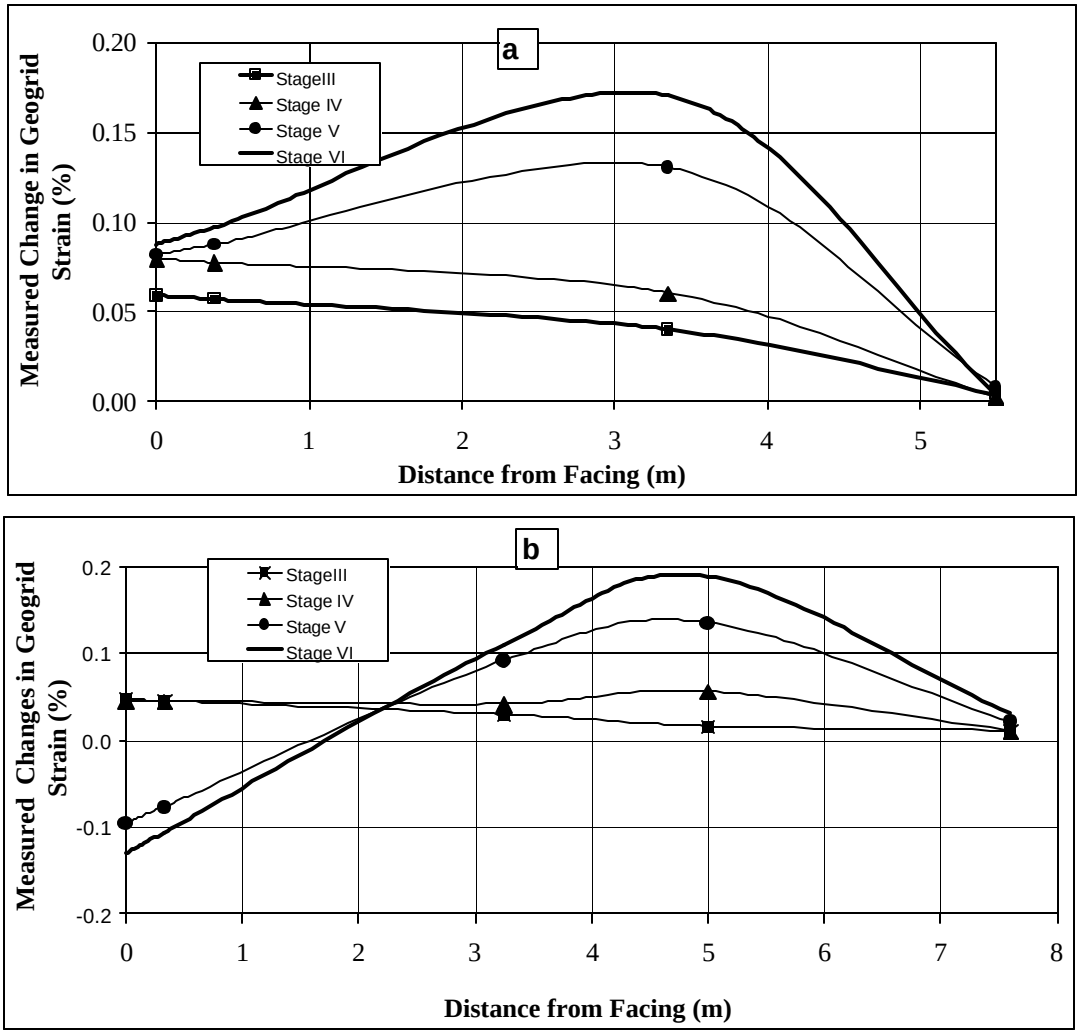


Fig. 5.9. Horizontal Profiles of the Measured Changes in Geogrid Strains during Placement of the Bridge Superstructure (Stages II to VI) from Stage I along: a) Geogrid Layer 6, and b) Geogrid Layer 10.

5.5 Results of Layer 12 Gages during all Construction Stages

Figures 5.10 shows time records for the measured strain data along geogrid layer 12 during all construction stages along Location Lines B and D. Due to the placement and compaction of 0.61 m of backfill over the gages (end of Stage I Construction), very sharp and significant increases in geogrid strains were measured: 0.44% along Location Line B and 0.18% along Location Line D (Figure 5.10). These large changes are questionable and unlikely to be representative of the axial/lateral trains experienced by the geogrid beneath the bridge footing. Large changes in the measured geogrid strains occurred during Stages III (placement of girders) and IV (placement of backfill behind the abutment wall). These changes are larger than those that occurred for geogrid layers 6 and 10 during the same stages, as expected. Very sharp reduction of the tensile geogrid strains was observed during Stage V (0.5% for Gage 12SB and 0.39% for Gage 12SD). This correlates very well with the observed sharp reduction of vertical earth pressures beneath the bridge footing during Stage V (see Chapter 3). Geogrid layer 12 was subjected to increasing tensile strains at Location Lines B and D during the final construction stage (Stage VI, Figure 5.10). Gage 12SD registered final very small compressive strains in the geogrid by the end of the construction stages, which could be attributed to erroneous reference readings.

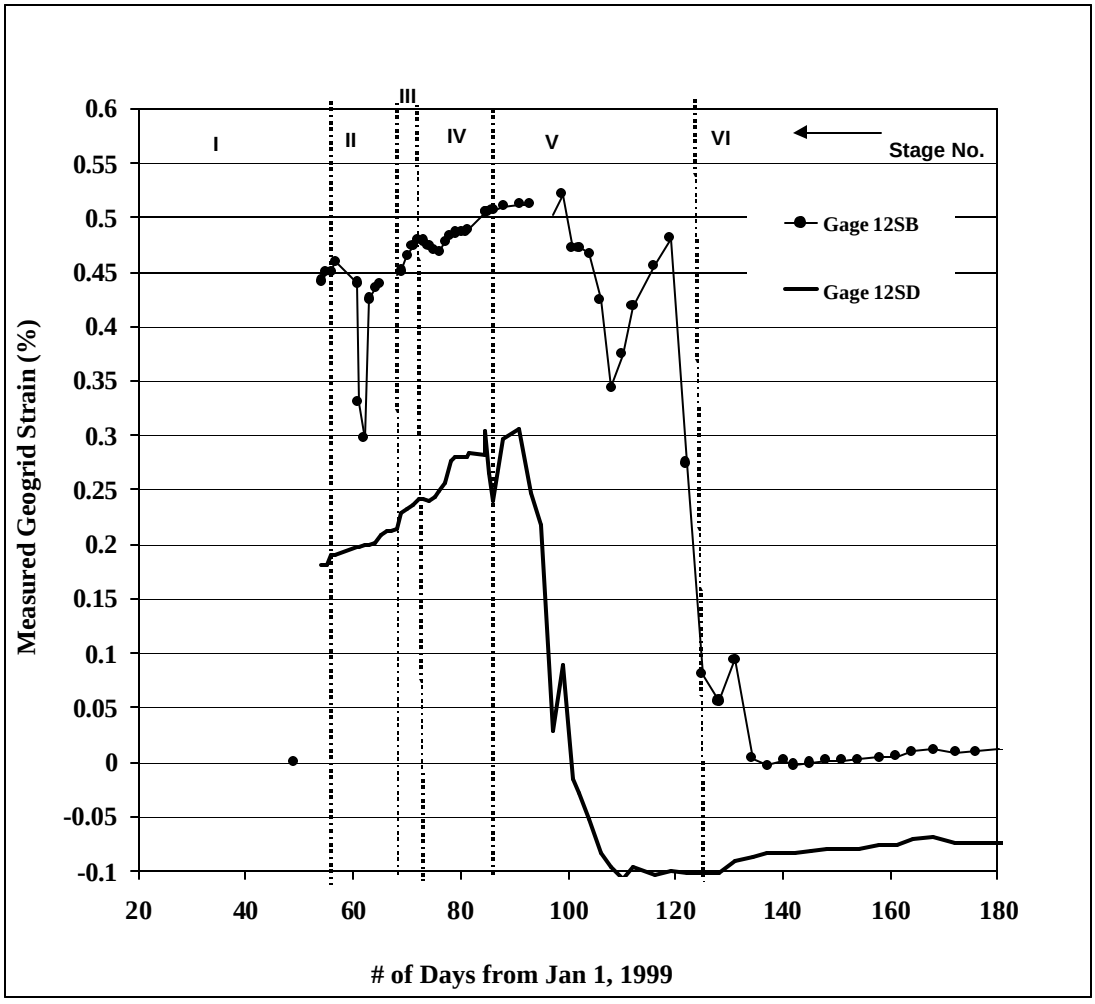


Fig. 5.10. Measured Strains along Geogrid Layer 12 during all Construction Stages.

5.6 Measured Results while the Structure was in Service (Stage VII)

Figures 5.11, 5.12, and 5.13 show time history records of the collected geogrid strain measurements along geogrid layers 6, 10, and 12, respectively, after opening the structure to traffic during Stage VII (days 180 to 545 from Jan. 1., 1999). Gage 10SC exceeded the measuring range during Stage VII and therefore was lost (Figure 5.12). The developed post-construction geogrid tensile strains (Figures 5.11 to 5.13) for all monitored geogrid layers were very small from approximately June 1999 to January 2000 and became negligible from approximately January 2000 to June 2000 (i.e., strains leveled out and remained approximately constant during this period). The geogrid strain measurements compare very favorably with the movements of the facing column of the front GRS wall obtained through

surveying and inclinometer, which showed relatively small (though not negligible) post-construction displacements from June 1999 to January 2000, and negligible post-construction movements from January to June 2000 (see Chapter 2).

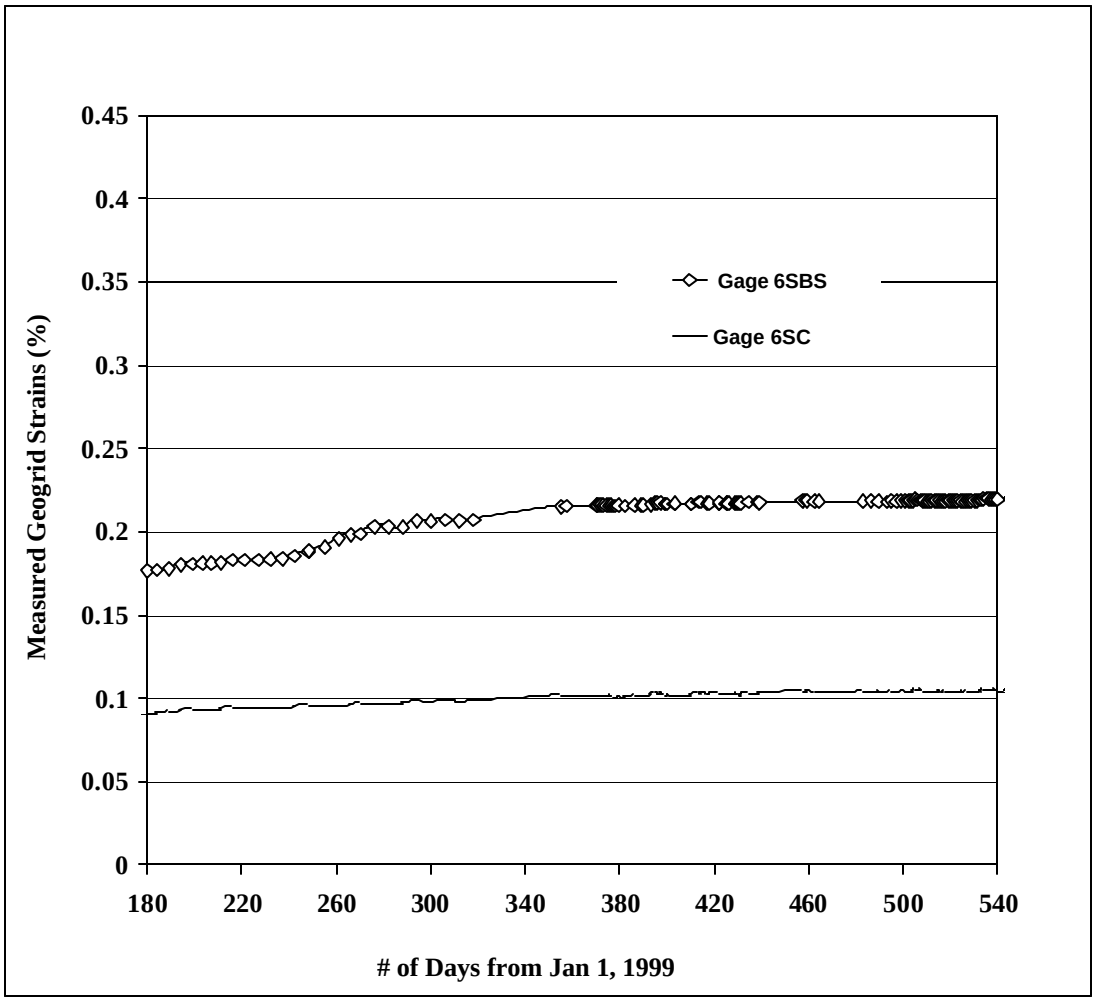


Fig. 5.11. Measured Strains along Geogrid Layer 6 while Structure was in Service (Stage VII).

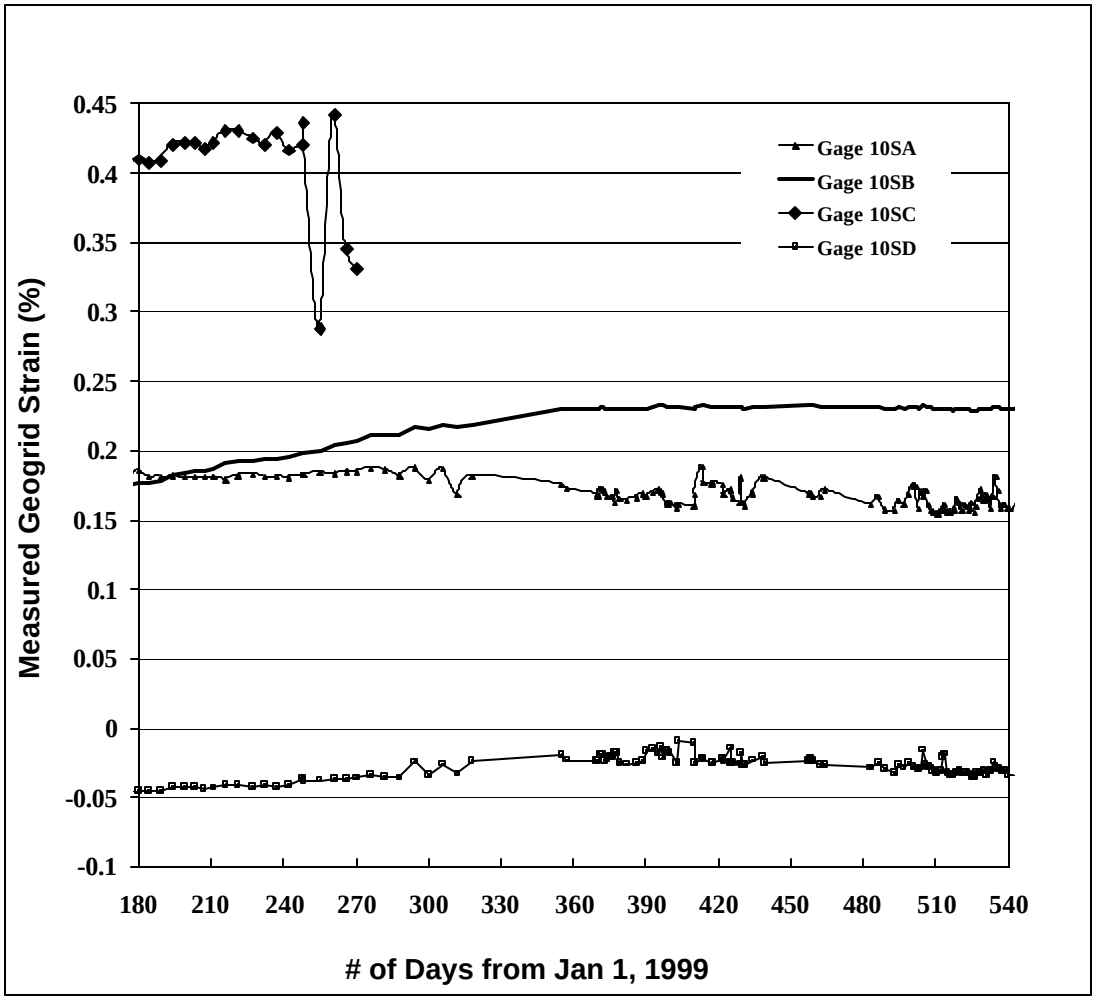


Fig. 5.12. Measured Strains along Geogrid Layer 10 while Structure was in Service.

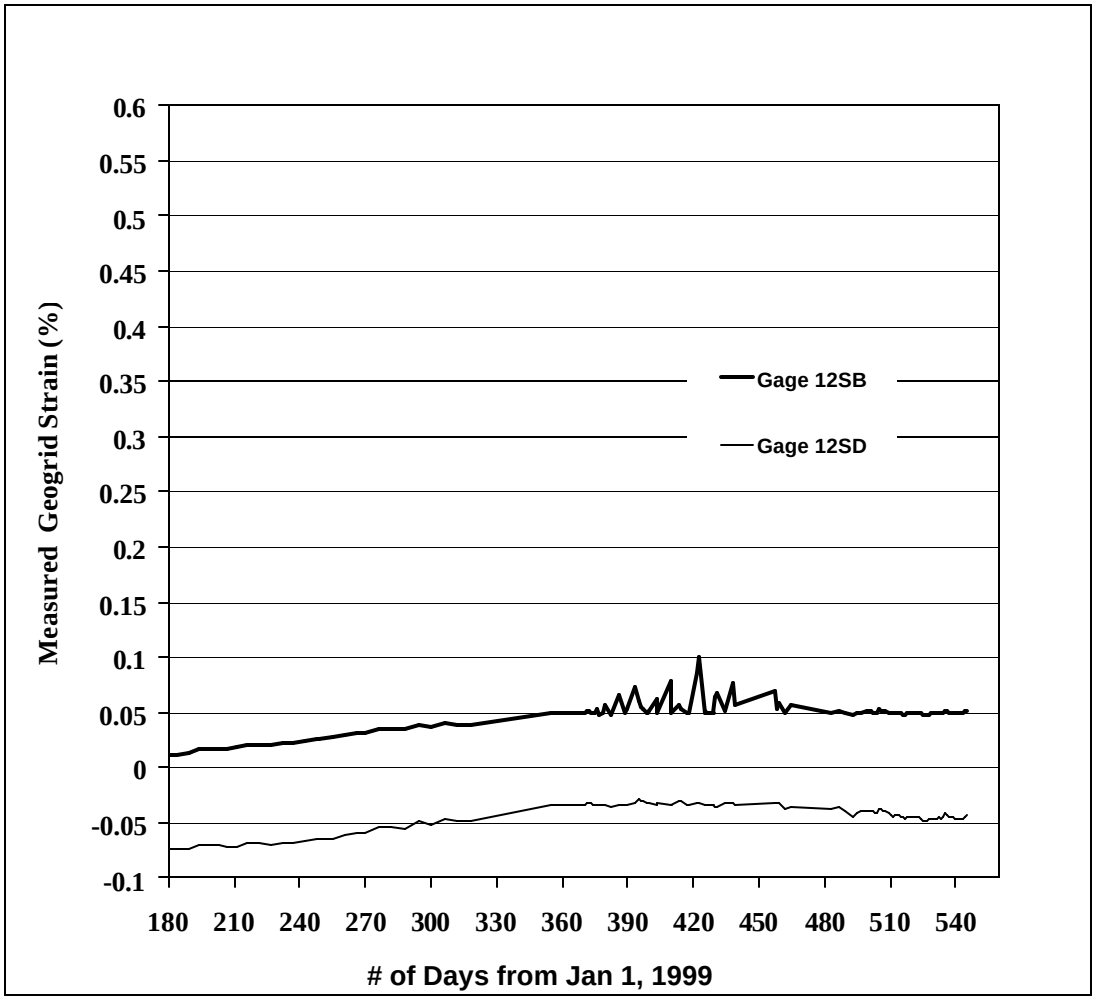


Fig. 5.13. Measured Strains along Geogrid Layer 12 While Structure was in Service.

5.7 Summary and Discussion of the Measured Results

Table 5.1 summarizes the geogrid strains measured from all gages during all construction stages. Table 5.1 also summarizes the geogrid strains developed due to the placement of the bridge superstructure (Stages II to VI), referred to as "changes from Stage I." Table 5.2 summarizes the geogrid strains measured by the end of the construction stages (Stage VI) and after opening the bridge to traffic (Stage VII). The data listed for Stage VII correspond to 6 months and 12 months after opening the bridge structure to traffic. Table 5.2 also summarizes the geogrid strains developed after opening the bridge to traffic, referred to as "changes from Stage VI." The data obtained by extrapolation are highlighted in Tables 5.1 and 5.2. For geogrid layer 2, located in the lower zone of the wall, the highest strains occurred at Location Line A (close to the wall facing) during all construction and post-construction stages (Table 5.1). At Location Lines B and C of geogrid layer 2, Location Line C of geogrid layer 6 (located in the middle zone of the wall), and Location Line D of geogrid layer 10 (located in the upper zone of the wall), very small changes in geogrid tensile strain were measured during all monitored stages (Table 5.1). This is because the lateral boundary of the soil along these locations was fully confined by the adjacent soil, allowing no loads to be supported by the geogrid. The geogrid layers along different locations and depths experienced increasing geogrid tensile strains and forces during and after construction (Tables 5.1 and 5.2). The maximum reliable registered geogrid strain was 0.5% (Table 5.2).

Sharp increases in geogrid tensile strains were measured following placement and compaction of the first two meters of backfill over gages. Strain gages results for geogrid layers 6 and 10 suggest that, out of the total geogrid strains induced during all monitored stages, approximately 50% occurred during placement and compaction of the initial 2 m of backfill (approximately 40 kPa vertical earth pressure) above these geogrid layers (see Figure 5.2 and Tables 5.1 and Table 5.2). This was attributed primarily to the influence of compaction that created large locked-in lateral geogrid and soil strains after removal of the compaction vertical loads. Other factors that might contribute to these large increases in geogrid strains are presence of slack, bending of the geogrid reinforcement and non-stable or erroneous reference reading (reference reading). To eliminate/alleviate these error causes, it is suggested to take

the reference readings for geogrid strain data after approximately 0.4 m of backfill is placed and compacted.

During Stages II to VI, the GRS system along geogrid layers 6 and 10 responded with comparatively small deformations and strains to the increasing vertical earth pressures. A possible reason for this behavior is the influence of compaction experienced in the previous stage. An additional potential justification is the fact that Construction Stages II to IV of Section 800 took place during the winter season. Buttry et al. (1996) reported a comparatively more rigid behavior for a GRS structure during the winter season (see Section 1.5.2). For geogrid layers 6 and 10, the highest strains occurred close to the wall facing during Construction Stages I through IV. This could be attributed to the relatively stiff connection between the facing and geogrid reinforcements during these stages and rigid behavior of the wall system (Section 1.5.3). Beneath the bridge footing, changes in the measured geogrid strains during Stages II to VI were larger than those that occurred for layers 6 and 10. This could be a sign of comparatively larger lateral movements beneath the bridge footing, resulting in relatively larger mobilization of the soil friction resistance and tensile resistance of the geogrid reinforcements.

During Stages V and VI, the GRS system along layers 6 and 10 responded with comparatively large strains and displacements to the increasing level of applied vertical earth pressures. Thawing and wetting of the backfill, as well as disappearance of the compaction influence, may have led to softening of the backfill during these stages (see observations reported by Buttry et al., 1996 in Section 1.5.2). Most of the geogrid tensile straining of layers 6 and 10 occurred inside the interior portion of the reinforced soil mass along Location Line B for layer 6 and along Location Line C for layer 10, where measured geogrid strains were increasing at a comparatively very high rate The stretching movement of the reinforced soil mass at these locations allowed for more of the lateral earth pressure to be supported by the geogrid reinforcements at these locations.

The geogrid tensile strains dropped during Stage V at locations behind the wall facing along geogrid layer 10 and beneath the bridge footing along Location Lines B and D. This is a sign of the reinforced soil mass subjected to lateral compressive loads at these locations that reduced the geogrid tensile loads. This response is strongly correlated with the measured vertical and lateral earth pressures from

other gages, suggesting it is reliable. This response could also be attributed to the lateral movement of the reinforced soil mass and possible settlement and rotation of the bridge footing relative to the more rigid wall facing during Stage V. This movement, most likely, led to the redistribution (increase and decrease) of geogrid tensile loads and vertical earth pressure loads in the reinforced soil mass.

The developed post-construction (1 year) geogrid tensile strains were very small (0 to 0.09%, see Table 5.2) and become negligible from approximately January 2000 to June 2000.

5.8 Consistency of the Measured Geogrid Strain Data

More research is needed to explain the reduction of geogrid tensile strain data and vertical earth pressure beneath the bridge footing that occurred during Stage V. Also, there is no measured geogrid strain data along Location Lines A and C beneath the bridge footing so that a complete picture of the geogrid strain distribution beneath the bridge footing could be obtained. For these reasons, the measured lateral strain data along geogrid layer 12 will not be analyzed further.

Previous sections concluded that the measured changes in lateral strains of geogrid layers 6 and 10 correlate very well with the applied loads and the measured response from pressure cells employed to measure vertical and lateral earth pressures. The geogrid strain measurements compare very favorably with the movements of the facing column of the front GRS wall obtained through surveying and inclinometer and show excellent agreement with the movements obtained after opening the structure to traffic (see Chapter 2). Based on the above discussion, it is fair to conclude that the measured results for the changes in geogrid lateral strains for layers 6 and 10 are consistent and representative of the actual response. Therefore, these data are found appropriate for comparison with the assumed design values.

Table 5.1: Measured Geogrid Strains (%) during all Construction Stages.

Stage #	I	II		III		IV		V		VI	
		Ch. = Change from Stage I (Wall construction) Values									
Gage #		Ch.	Total	Ch.	Total	Ch.	Total	Ch.	Total	Ch.	Total
Layer 2 Gages											
2SA	(0.24)										
2SB	0.06	0	0.06	0	0.06	0	0.06	0.02	0.08	0.02	0.08
2SC	0.01	0	0.01	0	0.01	0	0.01	0	0.01	0	0.01
Layer 6 Gages											
6SA	0.28	0.02	0.3	0.06	0.34	(0.08)	(0.36)	0.09	0.37	0.1	0.38
6SBN	0.16	0.03	0.19	0.04	0.20	0.06	0.22	0.13	0.29	0.17	0.33
6SBS	0.05	0.01	0.06	0.02	0.07	0.03	0.08	0.1	0.15	0.13	0.18
6SC	0.08	0	0.08	0	0.08	0	0.08	0	0.08	0	0.08
Layer 10 Gages											
10SA	0.29	0.04	0.33	0.04	0.33	0.04	0.33	-.08	0.21	-.11	0.18
10SB	0.07	0.01	0.08	0.03	0.10	0.04	0.11	0.09	0.16	0.11	0.18
10SC	0.22	0.01	0.23	0.02	0.24	0.06	0.28	0.14	0.36	0.19	0.41
10SD	-.07	0	-.07	0.01	-.06	0.01	-.06	0.02	-.05	0.03	-.04
Layer 12 Gages											
12SB	0.44	0	0.44	0.14	0.58	0.18	0.62	-.33	0.11	-.32	0.12
12SD	0.18	0.02	0.20	0.06	0.24	0.11	0.29	-.28	-.10	-.25	-.07

Data between parenthesis are for missing data estimated with extrapolation

Table 5.2: Measured Geogrid Strains (%) while Structure was in Service.

Monitored Stage #	End of Construction (Stage VI)	While Structure in Service (Stage VII)			
		After 6 months		After 12 months	
		Change from Stage VI	Total	Change from Stage VI	Total
Gage #	**Layer 6 Gages**				
6SA	0.38	(0.02)	(0.40)	(0.02)	(0.40)
6SBN	0.33	(0.04)	(0.37)	(0.04)	(0.37)
6SBS	0.18	0.04	0.22	0.04	0.22
6SC	0.08	0.02	0.10	0.03	0.11
	Layer 10 Gages				
10SA	0.18	-0.01	0.17	-0.01	0.17
10SB	0.18	0.05	0.23	0.05	0.23
10SC	0.41	(0.09)	(0.50)	(0.09)	(0.50)
10SD	-0.04	0.03	-0.01	0.01	-0.03
	Layer 12 Gages				
12SB	0.12	0.05	0.17	0.04	0.16
12SD	-0.07	0.04	-.03	0.03	-.04

Note: Data between parenthesis are for missing data estimated with extrapolation

5.9 Internal Stability Analysis of the Front GRS Wall

The analysis of the measured geogrid strain data requires the determination of the value and location of the maximum tensile forces in each geogrid layer to gain insight into the potential breakage of geogrid reinforcements. The maximum measured geogrid tensile strains across geogrid layers 6 and 10 at different construction stages were used to estimate the maximum geogrid tensile forces. The analysis also requires the determination of the connection forces between blocks and geogrid layer to gain insight in the potential breakage of connections. The geogrid strains at the rear side of the wall facing for geogrid layers 6 and 10 at different construction stages were used to estimate the connection forces between the blocks and geogrid. The geogrid tensile loads were obtained directly from the load-strain curve measured for UX 6 Tensar geogrid using ASTM D4595 method. Neglecting the time-dependent response of the geogrid or creep in estimating the geogrid loads will be validated in the next section. For the strain range measured in our study (less than 1%) the load-strain curve was linear and the applied geogrid tensile force was 2000 kN/m @1% strain.

5.9.1 Tensile Forces in the Geogrid Layers and Connections

Results for the maximum tensile forces and connection forces along geogrid layers 6 and 10 versus the applied vertical earth pressure on these layers during different construction stages are shown in Figures 5.14a and 5.14b. Predictions for the maximum tensile forces and connection forces in geogrid layers 6 and 10 as given in AASHTO 5.8.4.1C (using Equations 1.3 and 1.5) are also shown in Figure 5.14. If AASHTO predictions are reasonable, all measured curves shown in Figure 5.14 should be close to the AASHTO curve. Tables 5.3 and 5.4 summarize the measured maximum geogrid axial tensile forces and connection forces, respectively, by the end of the construction stages and 12 months after opening the bridge to traffic, along with the estimated values from the design (Equations 1.3 and 1.5). The ratio (%) of the measured geogrid maximum tensile forces and connection forces to the AASHTO estimated values are also shown in Tables 5.3 and 5.4. The results of Figure 5.14 and Tables 5.3 and 5.4 suggest that:

- The design procedure did not account for the compaction-induced stresses developed in the geogrid reinforcements and connections during interim construction stages of the front wall. The geogrid maximum tensile forces and connection forces measured at any horizontal level during placement and compaction of 1 m of backfill over that level are up to two times of those estimated in the design (Figure 5.14).

- Figure 5.14a and Table 5.3 show that the design procedure overestimated the required maximum geogrid tensile forces measured by the end of the construction stages. The measured maximum geogrid tensile forces range from 43% to 57% of the forces estimated in the design. This could be attributed to true shear strength of the backfill being higher than assumed in the design. It is also possible that the facing supported some of the applied lateral earth pressure, which was not accounted for in the design.

- Figure 5.14b and Table 5.4 show that the design procedure overestimated the required connection forces measured at the end of the construction stages. The measured connection forces range from 19% to 47% of the connection loads estimated in the design. The difference could be attributed to

two factors: (1) actual shear strength of the backfill was higher than assumed in the design, and (2) relatively flexible facing and connection that allowed for comparatively larger mobilization of the friction resistance of the backfill and the support of more lateral earth pressure by reinforcements, thus taking more of the lateral earth pressure off the facing and connections. Results of Table 5.4 suggest that the lateral boundary of the wall yielded more at geogrid layer 10 than at layer 6. The more flexible connection at layer 10 could be attributed to smaller vertical loads (smaller restrain) acting on the block facing of layer 10.

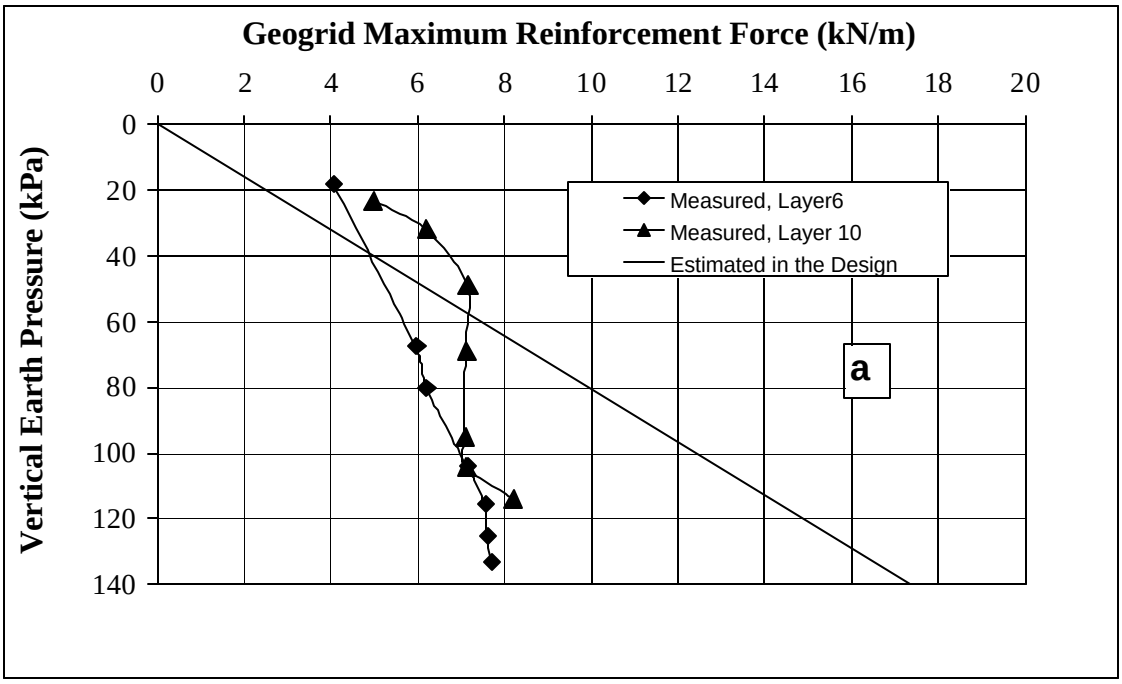
- The post-construction geogrid tensile strains may be developed due to traffic loads or/and creep. If the post-construction geogrid strains were developed due to traffic loads only (not from creep), then the developed maximum geogrid tensile forces while the structure was in service increased by 1.8 kN/m for layer 10 and 0.4 kN/m for layer 6 (Table 5.3). These measured forces range from 16% to 57% of the reinforcement loads estimated in design. Hence, the design overestimated the reinforcement loads due to traffic load by almost two times (assuming no creep-induced strains). If the post-construction geogrid strains were developed due to creep (no traffic load), they are very small (zero to 0.09%) when compared to the creep limit for the geogrid reinforcement, and they leveled out and become constant from January to June 2000 (long-term creep deformations are negligible). However, CDOT design procedure employs a creep reduction factor of 2.7 to determine the geogrid long-term design strength from the geogrid ultimate strength. This is a very high reduction factor that should be adjusted or estimated more rationally.

Table 5.3: The Design and Measured Maximum Geogrid Tensile Forces.

Geogrid Layer #	Estimated Max. Geogrid Tensile Force (kN/m)	Measured Max. Geogrid Tensile Force (kN/m)	Percentage Ratio of Measured to Estimated
End of Construction			
6	16.5	7.7	47%
10	14.3	8.2	57%
After Opening Bridge to Traffic			
6	19.0	8.1	43%
10	17.5	10	57%

Table 5.4: Design and Measured Connection Forces.

Geogrid Layer #	Estimated Connection Force (kN/m)	Measured Connection Force (kN/m)	Percentage Ratio of Measured to Estimated
End of Construction			
6	16.5	7.7	47%
10	14.3	3.6	25
After Opening Bridge to Traffic			
6	19.0	8.1	43%
10	17.5	3.3	19



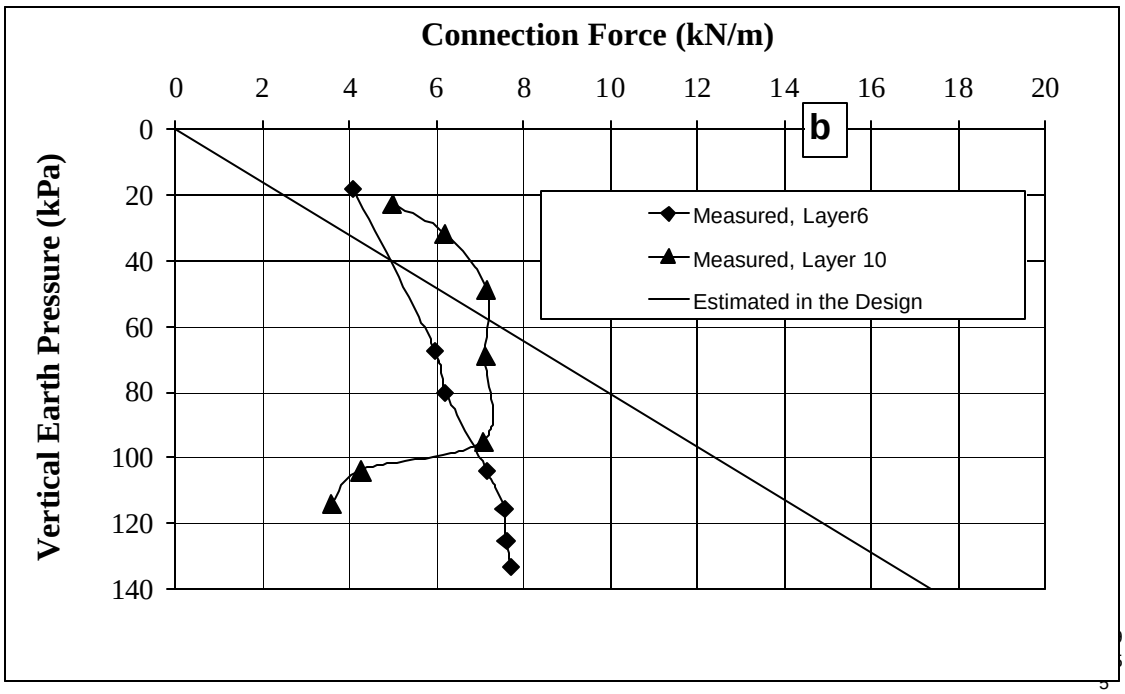


Fig. 5.14. Measured and AASHTO Predicted Results along Geogrid Layers 6 and 10: a) Maximum Geogrid Tensile Forces, and b) Connection Forces.

5.9.2 Reinforcements Internal Stability with Respect to Pullout Failure

Reinforcement stability requires that the reinforcement pullout resistance be larger than the geogrid applied maximum tensile force. Information on the measured geogrid maximum tensile forces was presented in the previous section. During construction, the comparatively large compaction-induced tensile geogrid forces might exceed the relatively low pullout capacity of the upper geogrid layers.

Location of the maximum tensile force line is needed to estimate the embedment length of the reinforcement in the resistance zone to reinforcement pullout failure. The reinforcement pullout resistance is linearly related to the reinforcement embedment length. Location of the potential failure surface is assumed by AASHTO to coincide with the maximum tensile forces line. The location of this line for GRS walls, with no surcharge load and constructed with extensible reinforcements is shown in Figure 5.15 (Elias and Christopher, 1997). For a structure like the Founders/Meadows structure, where a reinforced soil wall directly supports a high surcharge load, AASHTO's 1996 specifications recommend the embedment length beneath the bridge footing to be the length of reinforcements beyond the back edge of the bridge footing.

The locations of the maximum geogrid strains across geogrid layers 2, 6, and 10 were employed to determine the location of the maximum tensile forces line (i.e., the locus of the maximum tensile force in each reinforcement layer). For geogrid layer 6, the obtained maximum geogrid strain by the end of Stage VII (see Table 5.1) at Location Line A (0.40%) was very close to that at Location Line B (0.37%). However, the increases in geogrid tensile strains due to the placement of the bridge superstructure (more reliable strains are obtained in this range) were higher at Location Line B (0.17%) than at Location Line A (0.097%, see Table 5.1). Therefore, Location Line B was selected as the location where the maximum tensile strains occurred for layer 6. The results, as summarized in Figure 5.15, suggest that the maximum tension line can be assumed to be bilinear. It starts at the toe of the wall and extends through a straight line to the back edge of the bridge footing at the mid height of the wall, and from there extends vertically to the back edge of the bridge footing. This finding is in support of AASHTO recommendation presented before.

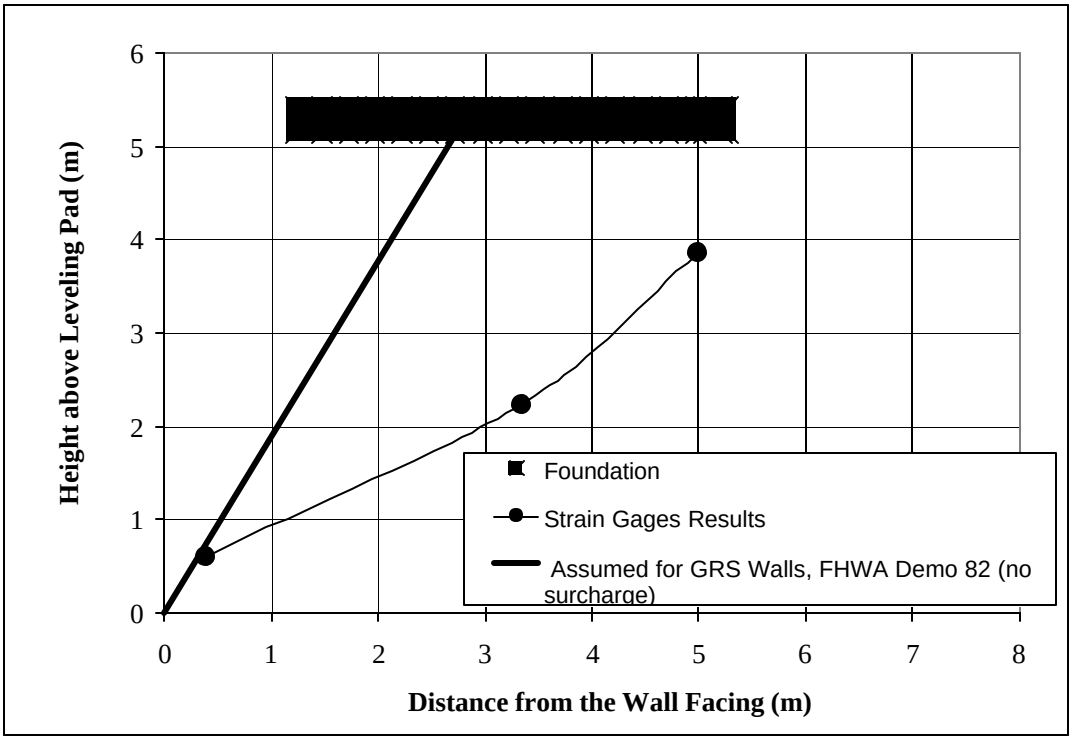


Fig. 5.15. Measured Location of the Maximum Tension Line (i.e., Potential Failure Line), Obtained from the Measured Geogrid Strain Data.

6. SUMMARY AND ASSESSMENT OF THE DESIGN AND PERFORMANCE OF THE FRONT GRS WALL

6.1 Overview

CDOT successfully completed the construction of the new Founders/Meadows Bridge in June of 1999. The Founders/Meadows structure is the first major bridge in the United States built on footings supported directly by a geosynthetic-reinforced soil system, eliminating the use of traditional deep foundations (piles and caissons) altogether. Figure 6.1 shows a typical cross-section through the "front GRS wall" and "abutment GRS wall," illustrating that the front GRS wall provides direct support for the bridge and embankment behind the abutment wall. A key element in the design was the need to support the high concentrated loads from the bridge footing and to alleviate the bridge bump problem. In addition, this kind of bridge support allows for construction in stages and comparatively smaller construction working areas. The Founders/Meadows Bridge was considered experimental, and comprehensive material testing, instrumentation, and monitoring programs were incorporated into the construction operations. These programs would allow assessment of the overall structure performance and evaluation of the suitability of CDOT and AASHTO design procedures and assumptions for the use of the front GRS wall as a measure to support the bridge footings, and, in conjunction with the abutment GRS wall, to alleviate the bridge bump problem.

The first report of this study (Abu-Hejleh et al., 2000) presented the design, materials, construction, and instrumentation of the Founders/Meadows structure. This second report presents a summary and analysis of the collected instrumentation data and assessment of the performance and design for the front GRS wall. This wall supports directly the bridge structure and the embankment behind the bridge wall (Figure 6.1). Recommendations for design and construction of future GRS abutments are furnished in the Executive Summary.

The overall instrumentation program was conducted in two phases: Phases I and II, which correspond, respectively, to the construction of the Phase I Structure (from July to December 1998) and Phase II

Structure (from January to June 1999). Instrumented sections 200, 400 (Figure 6.1) are located at the center of Phase I Structure and Section 800 (Figure 6.2) is located at the center of Phase II Structure. These sections were instrumented with survey targets, an inclinometer, and strain gages to measure movements of the facings of the front GRS wall and settlement of the bridge footing along Phase I and Phase II Structures (Figures 6.1 and 6.2). One gage (P2) was placed in the middle zone of the front wall (Figure 6.1) to obtain data for lateral earth pressure against the front wall facing of Section 400. The front GRS wall along Section 800 of the Phase II Structure was heavily instrumented with pressure cells and strain gages (Figure 6.2). Five gages to measure lateral earth pressure against the front wall facing were placed in the middle and upper zones of the front wall facing (Figure 6.2). Gages to measure distributions of vertical earth and geogrid strain were placed along three zones inside the front GRS wall (Figure 6.2): the upper zone of the wall (near the bridge footing, around geogrid layers 12 and 13), the middle zone of the wall (around geogrid layers 6 and 10), and the lower zone (near the base of the wall). These gages were placed along four critical vertical Location Lines (Figure 6.2): vertical Location Line A is very close to the wall facing, vertical Location Line B is close to the centerline of the bridge abutment wall, vertical Location Line C is close to the back edge of the bridge footing, and vertical Location Line D is behind the bridge footing (~7.5 m from the facing). The performance data presented in this report for the front GRS walls were collected during construction of the front GRS wall (Stage I, see Figure 6.1), during placement of the bridge superstructure (Stages II to VI, see Figure 6.1), and while the structure was in service for 18 months (Stage VII).

6.2 Measured Front Wall Response during Construction of the Front Wall

Construction of the front GRS wall went from July 16, 1998 to September 12 (warm season) for the Phase I Structure (Sections 200 and 400), and from January 19, 1999 to February 24, 1999 (cold season) for the Phase II Structure (Section 800). The maximum measured front wall outward displacement induced by wall construction was 12 mm, which corresponds to 0.2 % of the front wall height (Table 2.1). The measured settlement of the leveling pad supporting the front wall facing was approximately 8 mm (Table 2.1).

Measurements for vertical earth pressure inside the reinforced soil mass of Section 800 suggest that compaction induced vertical earth pressures ranging from 20 to 35 kPa (Figure 3.1). During placement and compaction of the first 1 m of backfill over the gages, sharp increases in geogrid strains (Figures 5.1 and 5.2) and lateral earth pressure against the wall facing (Table 4.1, Figure 4.3) were measured by these gages. Ratios of the measured lateral earth pressures to the design estimated values were as high as two. This response was attributed primarily to the influence of compaction that created large geogrid strains and earth pressures against the wall facing. Some or all of the compaction-induced pressures and strains remained locked-in after removal of the compaction vertical loads. Strain gage results for geogrid layers 6 and 10 suggest that, out of the total wall outward displacements and maximum geogrid strains induced during all monitored stages, approximately 50% occurred during placement and compaction of the initial 2 meters of backfill above these geogrid layers (see Figures 5.2 and 2.2, and Tables 2.1, Tables 5.1 and 5.2). The measured tensile strains of geogrid layers 6 and 10 (Section 800) dropped to the values estimated in the design after almost two to three meters of backfill were placed and compacted (Figure 5.14). The ratio between lateral to vertical earth pressure dropped to the ratio estimated in the design (0.31) after one meter of backfill was placed and compacted and it further dropped to a value of 0.11 after almost two meters of backfill were placed and compacted (from Gage P2 in Section 400 shown in Figure 6.1, see Table 6.1).

6.3 Measured Front Wall Response during Placement of the Bridge Superstructure

Monitoring stages (Figure 6.1) include placement of the bridge footing and girders seat (Stage II), placement of girders (Stage III), placement of reinforced backfill behind the abutment wall (Stage IV), placement of the bridge deck (Stage V), and placement of minor structures (Stage VI). These stages were completed on December 16, 1998 for the Phase I Structure (Sections 200 and 400), and on June 30, 1999 for the Phase II Structure (Section 800). The total average vertical contact stress directly underneath the bridge footing by the end of Stage VI was estimated as 115 kPa.

The maximum front wall outward displacement and bridge footing settlement induced by placement of the bridge superstructure were 10 and 13 mm, respectively. These movements correspond, respectively, to 0.17% and 0.29 % of the front wall height (Table 2.1). The measured settlement of the leveling pad supporting the front wall facing was approximately 7 mm (Table 2.1). It was observed that displacements induced in Section 800 were higher than those at Section 400 (Figure 2.3). This was attributed to a different construction season for both sections (summer to fall for Section 400, and winter to spring for Section 800) and a different construction sequence (Stage IV before Stage III for Section 400).

The measured response of the front GRS wall along Section 800 of Phase II Structure during placement of the bridge superstructure during Stages II to IV was significantly different from the measured response during Stage V, as detailed below.

6.3.1 Wall Response along Section 800 during Construction Stages II to IV

During these stages, the front GRS wall system along geogrid layers 6 and 10 (middle zone of the wall, Figure 6.2) responded with comparatively small deformations and strains to the increasing vertical earth pressures (Figures 2.2 and 5.7). A possible reason for this behavior is the influence of compaction experienced in the previous stage. An additional potential justification is the fact that Construction Stages II to IV of Section 800 took place during the winter season. Due to the relatively rigid behavior of the wall system during these stages, the wall facing supported relatively large lateral earth pressures against the wall facing (Table 4.2, Figure 4.4) and the maximum geogrid tensile strains occurred close to the wall facing (Figure 5.8). Along the levels of geogrid layers 6 and 10, there was excellent agreement between the applied and measured average vertical earth pressure during these stages, suggesting no transfer of bridge vertical loads to the wall facing at these levels (Figure 3.15).

Beneath the bridge footing (upper zone of the front wall), changes in the measured geogrid strains during Stages II to IV were larger than those that occurred at geogrid layers 6 and 10 (Table 5.1, Figure 5.10). There was almost 40 kPa difference between the applied and measured average vertical earth pressure in that zone (Figure 3.16). At the level beneath the bridge footing, the measured lateral earth

pressures against the wall facing were negligible (Gage 12H, Table 4.2). It is speculated that the reinforced soil zone beneath the bridge footing experienced significant lateral deformations (as evidenced by large increases of geogrid tensile strains) and settlement during these stages. This movement, most likely, caused the transfer of portion of the applied bridge vertical loads to the relatively more rigid wall facing, and minimized the lateral earth pressure against the wall facing.

6.3.2 Wall Response along Section 800 during Construction Stages V and VI

During these stages, the front GRS wall along layers 6 and 10 responded with comparatively large strains and displacements to the increasing level of applied vertical earth pressures (Figures 2.2 and 5.7). Thawing and wetting of the backfill, as well as disappearance of the compaction influence, may have led to softening of the backfill during these stages. The following additional observations were noticed during Stage V:

- The measured vertical earth pressures directly behind the wall facing dropped sharply at all levels (lower, middle, and upper zones, Figure 3.11 and Table 3.1). The measured lateral earth pressures against the wall facing (Table 4.2, Figure 4.4) and geogrid tensile strains (Table 5.1 and Figure 5.6) dropped for locations close to geogrid layer 10 (close to the upper zone of the wall) and remained approximately constant at locations close to geogrid layer 6 (middle zone of the wall).
- Location Lines B and C of geogrid layers 6 and 10 (the middle, interior portion of the front wall) experienced large increases of vertical earth pressures (Table 3.1) and geogrid tensile strains (Table 5.1). Most of the geogrid tensile straining occurred along vertical Location Line B for geogrid layer 6 and along vertical Location Line C for layer 10. The increase in vertical earth pressure was higher at vertical Location Line B than at C. The difference between the applied and measured average vertical earth pressure was very small (Figure 3.15). This response suggests that the reinforced soil mass between geogrid layers 6 and 10 supported most of the applied loads (i.e., no transfer of load to the wall facing).
- Both the geogrid tensile strains (Figure 5.10 and Table 5.1) and vertical earth pressure beneath the bridge footing (Figure 3.9 and Table 3.1) dropped sharply during Stage V. The difference between the applied and measured average vertical earth pressure beneath the bridge footing increased from

40 kPa to 60 kPa (Figure 3.16). At the level beneath the bridge footing, the measured lateral earth pressure against the wall facing remained negligible (Gage 12H, Table 4.2). It is speculated that some of the applied bridge loads at the horizontal level directly below the bridge footing were transferred to the wall facing.

By the end of all construction stages (Stage VI), the measured vertical earth pressures (Table 3.1) and geogrid tensile strains (Table 5.1) behind the wall facing (vertical Location Line A), and the lateral earth pressure against the wall facing (Table 4.2) became very close to, or even less, than the values measured by the end of Stage I Construction. In other words, the measured changes in loads behind the wall facing due to the placement of the bridge superstructure were very small. Geogrid strains were highest at vertical Location Line C for geogrid layer 10, at vertical Location Line B for layer 6 (see Figures 5.8 and 5.9), and behind the wall facing for geogrid layer 2 (Figure 5.3). The distribution of vertical earth pressures directly beneath the bridge footing was almost uniform (Figure 3.10). At other levels (geogrid layers 0, 6, and 10), the vertical earth pressure distribution differed significantly from location to location (Figures 3.3, 3.5, and 3.7 and Table 3.1). The lowest vertical earth pressure occurred close to the wall facing and the highest vertical earth pressure occurred along vertical Location Line B, the centerline of the bridge abutment. This distribution was approximately symmetrical around the center of the reinforced soil zone (~3.75 m from rear side of the facing). Ratios between measured lateral earth pressures against the wall facing in the upper half of the wall and estimated vertical earth pressure range from 0.04 to 0.11, way below the value estimated in the design (0.31). The maximum registered geogrid tensile strain by the end of all construction stages was 0.41% (Table 5.1), almost half the value expected in the design.

It can be concluded that the comparatively large movements of the front GRS structure experienced during Stage V significantly influenced the distribution of stresses and strains inside the reinforced soil mass of the front wall, especially close to the wall facing and directly beneath the bridge footing. This movement mobilized the backfill shear resistance and reinforcement tensile resistance (see Section 1.5.3), and led to:

- Reduction of the comparatively large: vertical earth pressure developed behind the facing, lateral earth pressure against the wall facing, and connection loads between reinforcement and blocks.
- Transfer of the large vertical earth pressures and reinforcement loads developed behind the wall facing to locations deeper inside the reinforced soil zone (except for the zone beneath the bridge footing).
- For the uppermost zone of the front wall below the bridge footing, it is speculated that a large portion of the applied bridge loads were transferred to the wall facing due to differential settlement between the reinforced backfill and the more rigid wall facing.

6.4 Measured Front Wall Response while the Structure was in Service

The post-construction stage (Stage VII) was completed on June 2000 (lasted 1 year for Phase II Structure and 18 months for Phase I Structure). The total average vertical contact stress directly underneath the bridge footing during this stage was estimated as 150 kPa.

The maximum registered front wall outward displacement and bridge footing settlement induced while the bridge was in service (Stage VII) were 13 and 10 mm, respectively. These movements correspond, respectively, to 0.22% and 0.17% of the front wall height (Table 2.1). The measured settlement of the leveling pad supporting the front wall facing was approximately 5 mm (Table 2.1). From the time of opening the bridge to traffic (December 1998 for the Phase I Structure and June 1999 for the Phase II Structure) until January 2000, the structure showed continued movements (Table 2.1 and Figure 2.4). However, from January 2000 to June of 2000, the measured movement results along all sections from different techniques suggest that the front GRS wall and bridge footing experienced negligible movements (Table 2.1 and Figure 2.4). The developed post-construction geogrid tensile strains for all monitored geogrid layers along Section 800 were very small from approximately June 1999 to January 2000 (0 to 0.09%, Table 5.2) and became negligible from approximately January 2000 to June 2000 (Figures 5.11, 5.12 and 5.13). For Section 800 of Phase II Structure, the monitored influence of traffic loads on the vertical loading of the reinforced soil mass (Table 3.2 and Figure 3.13) and horizontal loading of the wall facing (Figure 4.6, Tables 3.1 and 3.2) was very small.

6.5 Assessment of the Performance and Design of the Front GRS Wall

The measured changes in the vertical earth pressure in the reinforced soil mass and lateral earth pressure against the wall facing while the structure was in service were much lower than the values estimated in the design. Live loads due to traffic loading account for a scenario such as a line of fully loaded trucks suddenly stopping on top of the bridge during a traffic jam. If this scenario happened during Stage VII, our instruments certainly did not capture it. These performance data cannot be used to investigate design assumptions regarding the effect of live loads due to traffic. It was demonstrated in this study that all measured performance data for the structure during the construction stages are consistent and representative of the overall actual response experienced by the front GRS wall. The performance data that will be utilized in the assessment of the design and performance of the front GRS wall are:

- The vertical earth pressures in the reinforced soil mass and lateral earth pressures against the wall facing measured by the end of construction stages (Stage VI). These data were mostly collected along Section 800.
- The measured geogrid tensile strains (Section 800) and facing movement (Sections 200, 400, and 800) measured by the end of construction (Stage VI) and by the end of the post-construction stage (Stage VII, while the structure was in service).

Assessment of the external stability of the front GRS structure (overturning, sliding, and bearing capacity) assumed the reinforced soil zone to extend from the facing to vertical Location Line D, located approximately 7.5 m from the facing. Connection loads between reinforcements and facing blocks were obtained from geogrid strain measurements at the facing.

6.5.1 Movements of the Front GRS Wall

- The measured maximum outward displacement of the front GRS wall and settlement of the bridge footing due to placement of the bridge superstructure were, respectively, 11 mm (20 mm expected in the design) and 13 mm (25 mm expected in the design).
- The maximum *total* settlement recorded for the bridge footing was 23 mm, which is roughly a third of the AASHTO specified maximum tolerable *differential* settlement (70 mm) and a fourth of the

value assigned in the design for adequate clearance between I-25 and the bottom of the bridge superstructure (100 mm).

- The long-term outward displacement of the front wall facing and settlement of the bridge footing ceased from January to June 2000 (Table 2.1).

6.5.2 Internal Stability of the Reinforced Soil Mass

- The maximum tension line can be assumed to be bilinear. It starts at the toe of the wall and extends through a straight line to the back edge of the bridge footing at the mid height of the wall, and from there extends vertically to the back edge of the bridge footing (Figure 5.15).
- The design procedure provides a good estimation of the average vertical earth pressure acting within the potential failure zone. Along the potential failure line, the design underestimates the vertical earth pressures at locations beneath the bridge footing by approximately 20%, and overestimates them at locations close to the toe of the wall by approximately 33% (Figure 3.17).
- The design procedure did not account for the compaction-induced stresses developed in the geogrid reinforcements during interim construction stages of the front wall. The geogrid maximum tensile forces measured during placement and compaction of 1 meter of backfill are up to twice those estimated in the design (Figure 5.14a and Figure 5.2). During construction, the comparatively large compaction-induced tensile geogrid forces might exceed the relatively low pullout capacity of the upper geogrid layers. In order to anchor the reinforcements against potential reinforcement pullout during the compaction operations, it is suggested to drive steel bars through the back of the reinforcements into the backfill.
- The design procedure overestimated the maximum geogrid tensile forces for geogrid layers 6 and 10 measured after construction was completed (Figure 5.14a and Table 5.3). The measured maximum geogrid tensile forces range from 43% to 57% of the design values. This could be attributed to the fact that actual backfill shear strength is higher than the assumed value in the design. It is also possible that the facing supported some of the applied lateral earth pressure, which was not accounted for in the design.
- The post-construction geogrid tensile strains may be developed due to traffic loads or/and creep. If the post-construction geogrid strains were developed due to traffic loads only (not from creep), then

the measured maximum geogrid forces ranges from 16% to 57% of the reinforcement loads estimated in the design (Table 5.3). Hence, the design overestimated the reinforcement loads due to traffic load by almost two times (assuming no creep-induced strains). If the post-construction geogrid strains were developed due to creep (not from traffic loads), these strains are relatively very small (zero to 0.09%, Table 5.2), and they leveled out and became constant (Figures 5.11, 5.12, and 5.13) from January to June 2000 (i.e., long-term creep deformations were negligible). However, CDOT design procedure employs a very large creep reduction factor of 2.7 to determine the geogrid long-term design strength from the geogrid ultimate strength. The small post-construction creep geogrid strain could be attributed to two factors: 1) confinement of geogrid materials in well-compacted granular material that has no creep potential; and 2) the relatively small tensile loads in the geogrid reinforcements under working load conditions (maximum geogrid strain of 0.43% measured by the end of the construction stages).

6.5.3 Internal Stability of Connections and Wall Facing

- The design procedure did not account for the compaction-induced stresses developed at the connections and against the wall facing during interim construction stages of the front wall. The connection loads (Figure 5.14b) and lateral earth pressure against the wall facing (Figure 4.1 and Table 4.1) measured during placement and compaction of 1 meter of backfill are up to twice those estimated in the design. For wall facings with low connection strength or low interface shear strength, the compaction loads might cause excessive deformation or improper alignment to the facing. To prevent this unacceptable performance, it is recommended to: 1) fill the hollow core of the concrete blocks with gravel and drive temporary steel bars through block cells, 2) place a zone of crushed stone for a distance of 0.3 m behind the facing blocks, and 3) lightly compact the 3 feet behind the facing with one pass of a hand-operated compactor and keep heavy machinery 2 feet from the facing.
- The design procedure overestimated the connection forces (Figure 5.14b and Table 5.4) and lateral earth pressure against the wall facing (Table 4.3) measured after construction was completed. The measured connection forces range from 19% to 47% of the design values. The measured lateral earth pressures against the wall facing range from 4.2% to 36.4% of the design values. The

difference could be attributed to two factors: (1) actual shear strength of the backfill is higher than the assumed value in the design, and (2) relatively flexible facing and connection that allowed for comparatively larger mobilization of the shear resistance of the backfill and the support of more lateral earth pressure by reinforcements, thus taking more of the lateral earth pressure off the facing and connections.

- The measured vertical earth pressure behind the wall facing measured after construction was completed ranges from 2% (at a depth of 1.33 m below the footing) to 67% (at the bottom of the wall) of the vertical earth pressures employed in the design (Figure 3.17).
- The placement of the bridge superstructure had a small influence on the loads developed behind and against the upper half of the wall facing (Section 800, see Tables 3.1, 4.2, and 5.1).

6.5.4 External Stability of the Front GRS Wall

- The eccentricity values of the resulting vertical forces acting at different horizontal levels inside the front GRS wall measured after construction was completed are negligible (Table 3.3 and Figure 3.14). Therefore, there is not any potential for overturning the Founders/Meadows structure.
- The flexibility of the front GRS wall exhibited during Stage V (significant movement occurred to the structure) significantly enhanced the overturning stability of the structure by reducing the loads developed behind and against the wall facing.
- The most critical stage for the overturning occurred at the end of Stage III (placement of girders, see Figure 3.14). The backfill behind the bridge (placed during Stage IV) should be placed before placement of the girders to increase the forces resisting potential overturning of the reinforced soil structure.
- The measured bearing pressure at the base of the reinforced fill (Figure 3.15) by the end of construction (154 kPa) is below the expected design value (~199 kPa) and well below the tolerable bearing capacity (480 kPa).
- The measured bearing pressure below the bridge footing (Figure 3.16, 34 kPa) is way below the expected design value (115 kPa) and the allowable value (200 kPa). It is speculated that a large portion of the applied bridge loads beneath the bridge footing were transferred to the wall facing due to differential settlement between the reinforced backfill and the more rigid wall facing.

- The design procedure provides a reasonable estimate of the vertical earth pressures developed along the back of the reinforced soil zone (Figure 3.18).

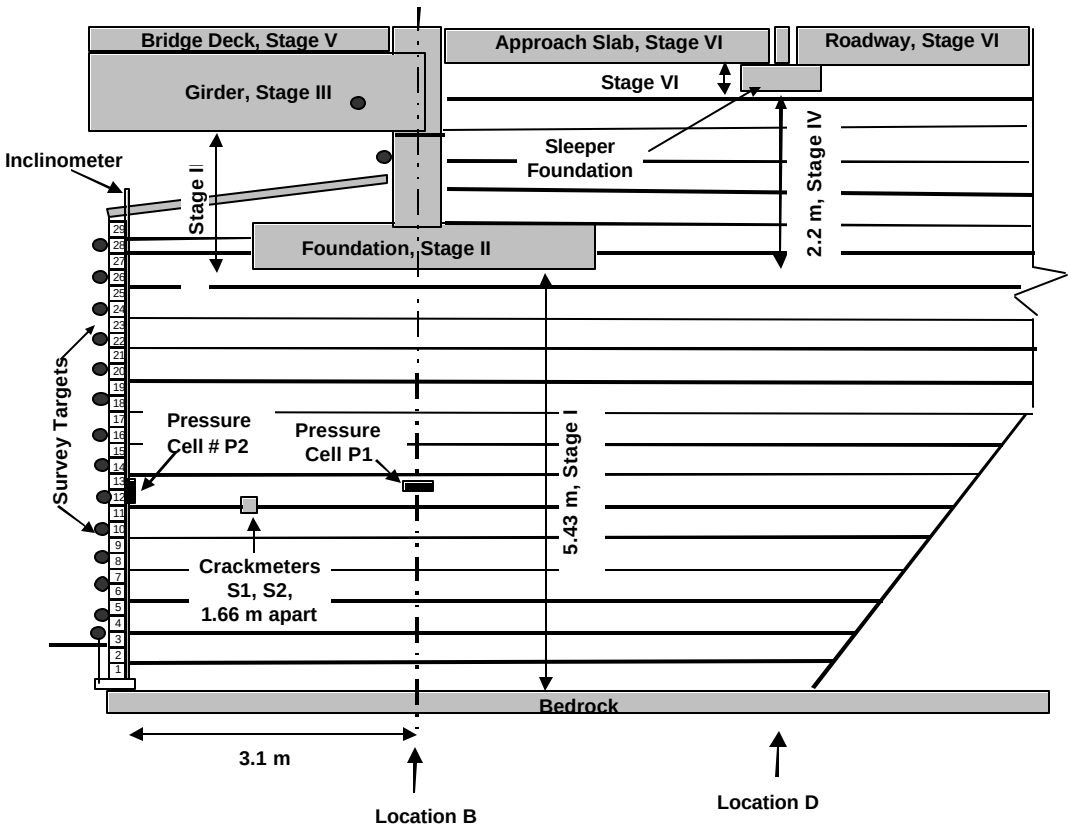


Figure 6.1. Instrumented Section 400 Showing all Monitored Construction Stages.

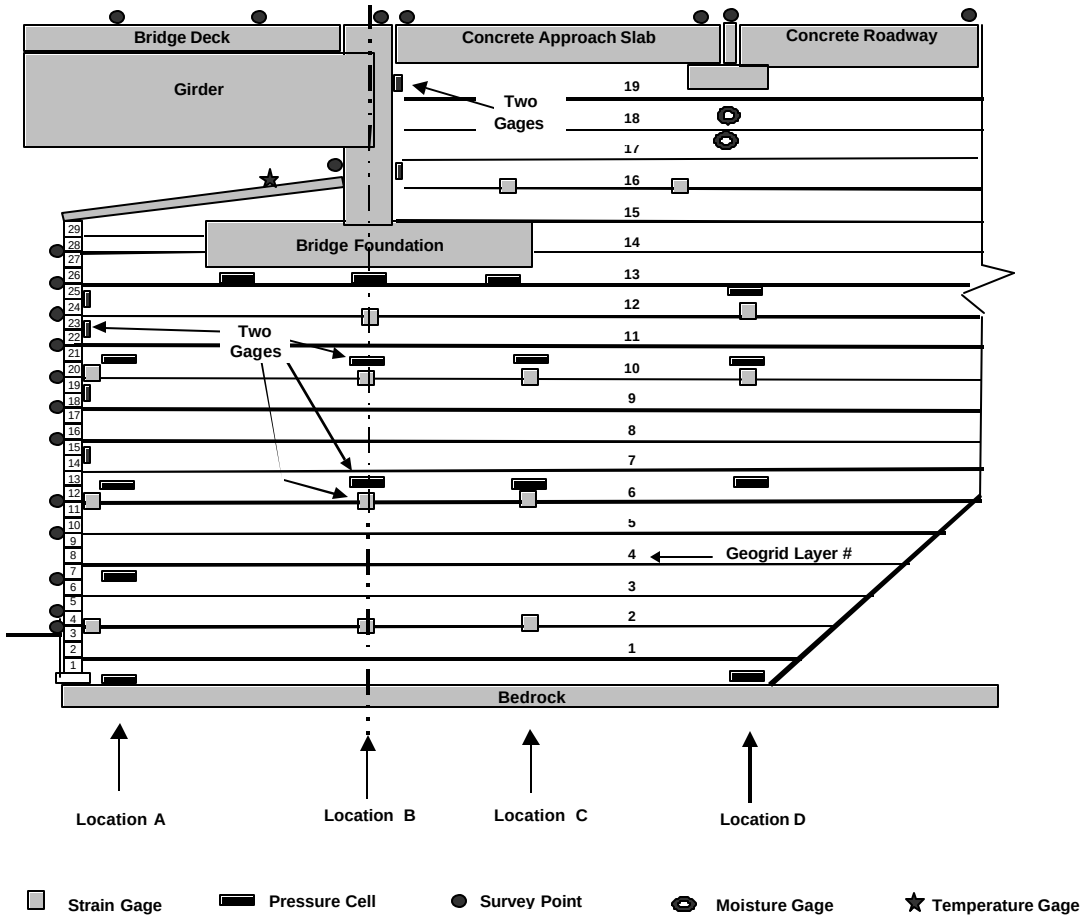


Figure 6.2. Layout and Instrumentation Program for Section 800.

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Appendix 5.1.3.2

SH 65 CDOT Project

MESA® MSE Retaining Wall System

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MESA®
RETAINING WALL SYSTEMS

Mesa County, Colorado

PROJECT
State Highway 65

LOCATION
Mesa County, Colorado

APPLICATION
Mesa Retaining Wall System

QUANTITY
Wall Face Area – 10,000 square feet

OWNER
Colorado Department of Transportation

SUB-CONTRACTOR
MAX Retaining Walls, Inc.

ENGINEER
Ground Engineering

BLOCK SUPPLIER
Basalite Concrete Products, LLC

INSTALLATION DATE
Summer 2008

PROJECT DETAILS

At this remote, mountainous location, an active landslide continued to push the existing highway down the slope. Repaving was a short-term fix, so the Colorado Department of Transportation sought a more permanent solution, a soil-nail wall faced with a mechanically stabilized earth wall. The Mesa Retaining Wall System was the system of choice for the MSE portion of the project, and MAX Retaining Walls was more than qualified for the installation. Avalanches adjacent to the site postponed the initial construction start due to safety concerns, but installation resumed in the Summer of 2008 with completion in the Fall of that same year.







Tensar INTERNATIONAL

For information please call:
Tensar International Corporation
800-TENSAR-1
www.tensar-international.com

MAX Retaining Walls

BASALITE® CONCRETE PRODUCTS, LLC



Appendix 5.1.3.3

Briarcliff Apartments Project

MESA® MSE Retaining Wall System

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MESA®
RETAINING WALL SYSTEMS

Briarcliff Apartments

Kansas City, Missouri

APPLICATION: Installation of a retaining wall to reinforce a steep bluff. The bluff supports a parking lot and swimming pool at a master-planned apartment community.

THE CHALLENGE: Access to the installation site and the resulting impact on logistics were the primary challenges, according to Randy Grego, General Manager of BC Hardscapes. "There was no access from the front, and not a lot of area to store block and materials, so we brought them in as needed," Grego explained.

Mike Stein, P.E., with Shafer, Kline & Warren (SKW), added that the upper portion and lower portion of the wall were each situated on different limestone rock formations.



The corrosive resistant properties of Tensar Geogrids allowed the use of recycled concrete for the backfill material, providing overall project savings.

SITE CONDITIONS: SKW's Stein, who designed the wall, noted that the site had been a borrow-and-dump site, requiring removal of a mixture of shale materials and soil in the excavated area. BC Hardscapes' Grego added that the cut at the back of the reinforced zone was unstable, and intermittent sliding occurred. It was eventually overexcavated and treated with fly ash. In addition, an unusual amount of rainfall resulted in some erosion behind the wall during construction.

ALTERNATIVE SOLUTIONS: A retaining wall system incorporating a polyester geogrid had been originally specified. When the environmentally conscious owner/developer decided to use recycled concrete for the backfill, the wall designer approached Tensar International and switched to Mesa Systems, due to Tensar® Geogrid's ability to withstand the concrete's alkalinity.

THE SOLUTION: Because of the site access challenge, the wall was built from the top of the bluff down. Twenty thousand tons of recycled concrete that had been crushed and screened for reuse as gravel aggregate were purchased and brought to the site. The installer set up a conveyor station at the top of the bluff to position the aggregate behind the wall. Aggregate was loaded onto the conveyor belt, dropping into the reinforced zone. Bobcats were then used to spread the backfill.

A 4 in. thick concrete leveling pad was poured at the foot of the wall, and Mesa Units in a blend of almond, buff and gray colors were placed. To support the

PROJECT HIGHLIGHTS

Project:
Briarcliff Apartments

Location:
Kansas City, Missouri

Installation:
Spring – Fall 2010

Quantity:
28,164 square feet Mesa Standard Units; Maximum wall height 40 feet.

Owner/Developer:
Briarcliff Development Company, Inc.

Engineer/Wall Designer:
Shafer, Kline & Warren, Inc.

Installation Contractor:
BC Hardscapes, LLC

Materials Supplier:
Midwest Block & Brick, Inc.

residential development at the top of the bluff, 12 piers within the reinforced zone were installed in sections as the project was being completed. Grego credited the geogrid and Mesa Systems' positive mechanical connection as key to the project's success.

THE MESA SYSTEMS ADVANTAGE: Inert to chemical degradation, Tensar® UX® Geogrids feature polymer resins that successfully resist high pH soils. They join versatile, high-strength Mesa Units and a unique locking connector as the only single-source, fully integrated segmental retaining wall (SRW) system with a positive mechanical connection. That connection assures full load transfer from wall face to reinforcement, resulting in unsurpassed structural integrity, efficiency and performance.

ADDITIONAL INFORMATION AND SERVICES: Tensar International Corporation specializes in solutions for site development problems such as grade changes requiring retaining walls and poor soil conditions affecting the cost of roadways, parking lots and building structures. Our solutions use proprietary engineered systems and our own unique products, services and application technologies. Our products and technologies, backed by the most thorough quality assurance practices, are at the forefront of the industry. Highly adaptable, cost-effective and installation-friendly, they provide exceptional, long-term performance under the most demanding conditions. Our support services include site evaluation, design consulting and site construction assistance.

For innovative solutions to your engineering challenges, rely on the experience, resources and expertise that have set the industry standard for nearly three decades.



Inert to chemical degradation, Tensar HDPE Geogrids are able to withstand the high alkalinity found in recycled concrete.



The random placement of three different colored Mesa Units gave the finished wall a unique appearance.

For more information on the Mesa Systems or other Tensar Systems, call 800-TENSAR-1, email info@tensarcorp.com or visit www.tensarcorp.com

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2500 Northwinds Parkway
Suite 500
Alpharetta, GA 30009

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Appendix 5.1.4

MESA DOT Contacts

MESA® MSE Retaining Wall System

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List of References for the Mesa® Retaining Wall System for Transportation Projects*

Utah-
Jon Bischoff, Darin Sjoblom, Jim Higbee
Utah Department of Transportation
4501 South 2700 West
Salt Lake City, UT 84114
Main Ph (801) 965-4000

Colorado-
Shing-Chun (Trevor) Wang, Mark Vessely, Dr. Hsing-Cheng Liu
Colorado Department of Transportation
4201 East Arkansas Ave.
Denver, CO 80222
Main Ph (303) 757-9011

Washington –
Tony Allen
Washington Department of Transportation
PO Box 47365
Olympia, WA 98504-7365
Main Ph (360) 709-5450

Idaho –
Tri Buu
Idaho Department of Transportation
3311 W State St.
PO Box 7129
Boise, ID 83707
Main Ph (208) 334-8448

Tennessee –
Mr. Houston Walker or Mr. Len Oliver
Tennessee Department of Transportation
505 Deaderick St.
James K. Polk Blsg./Ste. 1100
Nashville, TN 37243
Main Ph (615) 741-3351

Alabama –
Ms. Lyndi Blackburn
Alabama Department of Transportation
1409 Coliseum Blvd.
Montgomery, AL 36110
Main Ph (334) 206-2203

North Carolina -
Scott Hidden
North Carolina Department of Transportation
1589 Mail Service Center
Raleigh, NC 27699
(919) 250-4088

Maryland –
Mr. Raplh Manna
Maryland State Highway Authority
707 N Calvert St.
Baltimore, MD 21202
Main Ph (410) 545-8333

Connecticut. –
Leo Fontaine
Connecticut Department of Transportation
2800 Berlin Turnpike
P.O.Box 317546
Newington, CT 06131
Main Ph (860) 594-3180

West Virginia –
Mr. Lovell Facemire
West Virginia Department of Transportation
1900 Kanawha Blvd East
Charleston, WV 25305
Main Ph (304) 558-9752

Private consultants involved in FHWA design procedure and guidelines for MSE structures

Mr Jim Collin
The Collin Group
7445 Arlington Rd.
Bethesda, MD 20814
Main Ph (301) 907-9501

Mr Ryan Berg
Ryan Berg & Associates
2190 Leyland Alcove
Woodbury, MN 55125
Main Ph (651) 735-7622



Tensar International Corporation
5883 Glenridge Dr, Suite 200
Atlanta, GA 30329
(800) TENSAR-1

* By listing these individuals as a reference does not imply endorsement of the Mesa Retaining Wall System by any individual, the company or agency listed.