REPORT OF REVIEW TENSAR ARES MECHANICALLY STABILIZED EARTH (MSE) WITH PRECAST CONCRETE PANELS PAIRED WITH EXTENSIBLE SOIL REINFORCEMENTS

April 2026

HIGHWAY INNOVATIONS, DEVELOPMENTS, ENHANCEMENTS AND ADVANCEMENTS (IDEA)

The Tensar ARES 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

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Review Summary

Following its initial review of the Tensar ARES 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 ARES Retaining Wall System submittal for projects where the Tensar ARES System is proposed.

Submittal Checklist

The checklist used from the IDEA protocol for this evaluation is C3 – Initial Technical Evaluation Checklist for Precast Concrete Panel Paired with Extensible Reinforcement. This is the second evaluation of the Tensar ARES System the first was performed by the predecessor to the IDEA program, the Highway Innovative Technology Evaluation Center (HITEC) in 1997.

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 ARES System no information has been designated by the applicant as confidential.

System Description

Components

The major components of the ARES Retaining Wall System are Precast Concrete Panels mechanically connected to extensible soil reinforcements (Tensar Uniaxial HDPE geogrids), and select granular backfill. Ares standard panels can be 5 ft high by 5 ft wide, 5 ft high by 9 ft wide, 5 ft high by 10 ft wide, or full height panels. Other standard components include a concrete leveling pad, EPDM bearing pads, and a geotextile filter across the vertical and horizontal joints between adjacent panels. The soil reinforcing consists of Tensar uniaxial geogrid (i.e., UX1100MSE, UX1400MSE, UX1500MSE, UX1600MSE, UX1700MSE) connected to the facing. The connection between the precast panels and the geogrid is achieved by mechanically connecting UX1700 geogrid tabs cast in the panels to the soil reinforcement using a "Bodkin Connection".

System History

The ARES system has been evaluated by the Highway Innovative Technology Evaluation Center (HITEC) in November 1997. The current ARES System submitted herein differs from the system evaluated by HITEC in 1997 in the way that the facing panels are attached to the geogrid backfill reinforcement. In the HITEC-evaluated system, the geogrid layers were connected into a narrow slot cast into the back face of the panels. That slot connection is shown in the HITEC report. The current system uses short tabs of Tensar UX1700 Geogrid cast into the concrete facing panels. The layers of backfill reinforcement are spliced to the tabs during construction. This connection method (using the bodkin bar):

a) Provides advantages in panel casting and handling.

b) Increases bending strength of the panels compared to the slotted connection.

c) Permits the use of a wider, more efficient range of geogrid reinforcement.

The system uses Tensar UX1400MSE, UX1500MSE, UX1600MSE and UX1700MSE geogrids. This current series of geogrids UX****MSE are similar, but more efficient materials that differ from former versions in the increased length of the rib elements and apertures as well as the long-term strength. These differences result in relatively minor changes to design properties.

System Properties

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

Soil Reinforcement Ultimate (Nominal) Tensile Strength and Reduction Factors

The soil reinforcing used in the ARES Retaining Wall System are uniaxial extensible high density polyethylene (HDPE) geogrids manufactured by Tensar. The geogrid used in the ARES 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 and is included 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 (UX1500MSE, UX1600MSE, AND UX1700MSE) to 2.68 (UX1100MSE, and UX1400), 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, the 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 long-term observations, default reduction factors as low as 1.1 have been used successfully by state departments of transportation. The 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 indicate 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 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 2.54 to 2.73 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.

Soil Reinforcement-Precast Panel Connection Capacity

The three components of the connection must be analyzed to determine the controlling mechanism of failure.

The controlling failure mechanism of the connection is the concrete/tab insert connection. The creep reduction factor for this mode of failure is the creep reduction factor for the tab connector (UX1700 = 2.54).

$$T_{al} = \frac{T_{ult-MARV} * CRu}{RF_{CRtab} x RF_D x RF_{ID}}$$

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, the 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 6ft below the top of the 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 a C_i factor of 0.8 for crushed stone and coarse sand meeting AASHTO backfill specifications.

Retaining Wall Design

The design methodology for an ARES 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 ARES Precast Concrete Panels 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 ARES Precast Panel Facing Units and Extensible Reinforcement Retaining Wall System for projects where the ARES System is proposed.

Closing

An updated technical evaluation should be performed for the ARES Precast Panel Facing Units and Extensible Reinforcement Retaining Wall System after five years (i.e., April 2031) or upon notice that a significant modification of the system has been made. For details regarding updated 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 ARES Retaining wall System

Geogrid Soil Reinforcement - Data/Geogrid		UX1100	UX1400	UX15000	UX16000	UX17000
$T_{ULT-MARV}$ (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		Varies[c]	Varies[c]	Varies[c]	Varies[c]	Varies[c]
Friction coefficient along geogrid-soil interface, ρ	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:						
	$T_{ult\ Bodkin}$ (lb/ft)	11990	11990	11990	11990	11990
	$CR_u = T_{ult\text{-}connection} / T_{ult\text{-}geogrid}$	0.93	0.93	0.93	0.93	0.93
	$RF_{CR\ Connection}$ 75 yrs	2.54	2.54	2.54	2.54	2.54
	$RF_{CR\ Connection}$ 100 yrs	2.57	2.57	2.57	2.57	2.57
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 compactor. Project-specific (or representative) installation damage testing and RF_{id} quantification is recommended for MSE fills with a maximum aggregate size greater than ¾" (19mm); [b] Predominant material; [c] Varies based on the panel size (i.e., 90% for 5x5, 96% for 5x9 and 87% for 5x10 panel sizes)

Tensar ARES® System

Initial Technical Evaluation Checklist for Precast Concrete Panel Paired with Extensible Reinforcement

Introduction....4
1.1 Facing Unit....6
- 1.1.1 Innovation....6
- 1.1.2 Types of Facing Units....6
- 1.1.3 Specifications....6
- 1.1.4 Dimensions, Tolerances, and Reinforcement Schedule....6
- 1.1.5 Compressive Strength....6
- 1.1.6 Percent Air Range....6
- 1.1.7 Mix Design....6
- 1.1.8 Shear, Alignment, or Bearing Devices....6
- 1.1.9 Filter....7
- 1.1.10 Aesthetic Facing Options....7
- 1.1.11 Limits on Facing Units....7

1.2 Extensible Reinforcement....8
- 1.2.1 Innovation....8
- 1.2.2 Reinforcement Styles/Types....8
- 1.2.3 Specifications....8
- 1.2.4 NTPEP Report....8
- 1.2.5 Facing Unit-Reinforcement Connection....9
- 1.2.6 Connection Strength Tests....10
- 1.2.7 Reinforcement Pullout Tests....10
- 1.2.8 Soil-Geosynthetic Interface Shear Tests....11

1.3 Other Components....11
- 1.3.2 Reinforced Soil....12
- 1.3.3 Drainage....12
- 1.3.4 Coping....12
- 1.3.5 Traffic Barriers....13
- 1.3.6 Slip Joints....13

Section 2: ERS Design....13
2.1 Design Methodology....13
- 2.1.1 Innovation....13

- 2.1.2 Design Methodology Description 13
- 2.1.3 Obstructions 14

2.2 Design Example 14

Section 3: Construction 14

3.1 Construction Procedures 14

- 3.1.1 Innovation 14
- 3.1.2 Construction Manual 15
- 3.1.3 Facing Unit Installation 15
- 3.1.4 Earth Reinforcement Installation 15
- 3.1.5 Alignment 15
- 3.1.6 Soil Installation 15
- 3.1.7 Erosion Control 15
- 3.1.8 Contractor Qualifications 16
- 3.1.9 Soil Installation (Repeated) 16

Section 4: Quality Control 16

4.1 Manufacturing 16

- 4.1.1 Facing Units 16
- 4.1.2 Earth Reinforcement 16
- 4.1.3 Other Devices 16

4.2 Construction 17

- 4.2.1 Construction Quality Control 17

Section 5: Performance 17

5.1 Performance History 17

- 5.1.1 Development and Usage History 17
- 5.1.2 Oldest Structures: 17
- 5.1.3 Tallest Structures: 19
- 5.1.4 User Approvals and Contact Information: 21

Section 6: Other 21

6.0 Other Information 21

- 6.1 Additional Information 21

Introduction

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

Tensar Response:

The ARES® Retaining Wall (ARES) is a trademarked system developed and supplied by Tensar International Corporation (TIC). The ARES system is a mechanically stabilized earth (MSE) retaining wall system. The system consists of precast concrete panel facing elements with connection devices used to mechanically connect the geogrid soil reinforcement to create an MSE retaining wall.

The ARES System features standard precast concrete facing panels mechanically connected to Tensar Uniaxial HDPE geogrids. The ARES Standard panels can be 5ft high x 5ft wide, 5ft high x9ft wide, 5ft high by 10ft wide or full height panels. Variable dimensions for non-standard panels can be used at top, bottom and end of walls. Tensar HDPE geogrids are connected to the concrete panels by splicing each layer to a Tensar UX1700 geogrid tab. The UX1700 tab is cast into the back side of the facing panel. The connection of the geogrid reinforcement layers to the UX1700 tab is made by overlapping the layer and the tab, inserting the ribs of each piece through the apertures of the other piece, weaving an HDPE bar between alternating ribs, and tightening the splice such that the transverse bars of the two pieces of the geogrid bear against the bar. This splicing mechanical connection is known as the "Bodkin Connection". The bodkin connection concept was first developed in 1986 in the United Kingdom and was first used in the United States in 1986 on FHWA Experimental Project called Tanque Verde-Wrightstown-Pantano Roads Intersection in Tucson, Arizona. At the time, the geogrid tab were formed by embedding a short section of the geogrid into the wall panel during pre-casting to form a "C-shaped" tab protruded from the back face of the panel. A schedule 40 PVC pipe was used to bodkin the tab to the primary reinforcement. The Tanque Verde ARES panel wall continue to perform well as of today after 40 years in service.

The ARES system has been evaluated by the Highway Innovative Technology Evaluation Center (HITEC) in November 1997. The current ARES System submitted herein differs from the system evaluated by HITEC in 1997 in the way that the facing panels are attached to the geogrid backfill reinforcement. In the HITEC-evaluated system, the geogrid layers were connected into a narrow slot casted into the back face of the panels. That slot connection is shown in the HITEC report. The current system uses short tabs of Tensar UX1700 Geogrid cast into the concrete facing panels. The layers of backfill reinforcement are spliced to the tabs during construction. This connection method (using the bodkin bar):

a) Provides advantages in panel casting and handling.
b) Increases bending strength of the panels.
c) Permits the use of a wider, more efficient range of geogrid reinforcement.

The system uses Tensar UX1400MSE, UX1500MSE, UX1600MSE and UX1700MSE geogrids. This current series of geogrids UX****MSE are similar, but more efficient materials that differ from former versions in the increased length of the rib elements and apertures. These differences result in minor changes to design properties.

Refer to Appendix 3.1.2, wherein the full wall system Installation Guide and Construction Manual is presented. In addition, refer to Appendix 0 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: Ares precast concrete panel facing elements.

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

  Tensar Response: Refer to the Ares Precast Quality Control Guide in Appendix – 1.1.3.a for concrete requirements of the facing elements. The reinforcement in the concrete facing elements is designed in accordance to AASHTO LRFD and is attached in Appendix –1.1.3.b.

- 1.1.4 Dimensions, Tolerances, and Reinforcement Schedule**: Provide standard dimensions, tolerances and typical steel reinforcement schedule (if any is used) for each type of unit** (e.g. standard, crest, corner, base, etc.) in **plan and section drawings**.

  Tensar Response: Refer to page 5 of Appendix – 1.1.4.a and page 6 of Appendix – 1.1.4.b and Appendix – 1.1.4.c for standard dimensions and typical steel reinforcement details.

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

  Tensar Response: The target 28-day minimum compressive strength is 4000 psi in 28 days.

- 1.1.6 Percent Air Range**: Provide the target percent air range**.

  Tensar Response: The target air content for freeze–thaw exposure shall be 6% ± 1.5% in accordance with AASHTO Construction Specifications unless otherwise required by the governing Agency/Project Specifications.

- 1.1.7 Mix Design**:** Producers will change mix design to accommodate state requirements.

  Tensar Response: The mix design for precast concrete panels must comply with state-specific requirements. Apart from this, the only other requirement is achieving a minimum strength of 4,000 psi after 28-days.

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

Tensar Response: Hardwood wedges are to be utilized during construction to aid panel alignment as described in the Installation Guide and Construction Manual in Appendix - 3.1.2. The quantity of bearing pads required is contingent upon panel dimensions and wall height. For a 5'x5' panel system, 4 bearing pads are required for wall heights up to 44 ft and 6 bearing pads for wall height over 44 ft. For a 5'x9' and 5'x10' panel systems, 8 bearing pads are required for wall heights up to 49 ft and 10 bearing pads for wall height over 49 ft. Refer to Appendix – 1.1.8.a for number of bearing pads determination and Appendix – 1.1.8.b for bearing pad specifications. Page 3 of Appendix – 1.1.4.a and page 4 of Appendix – 1.1.4.b and Appendix – 1.1.4.c show typical drawings of the bearing pads that are placed in between panels for the 3 typical panel sizes.

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

  Tensar Response: For the prevention of soil migration Class 1 8oz. Non-Woven Geotextile is used at the vertical and horizontal joints of the precast concrete panels. Refer to Appendix – 1.1.9 for the non-woven geotextile specifications. Page 3 of Appendix – 1.1.4.a and page 4 of Appendix – 1.1.4.b and Appendix – 1.1.4.c show typical placement of the geotextile at the horizontal and vertical joints for the 3 typical panel sizes.

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

  Tensar Response: The ARES system is adaptable to various aesthetic facing options. Typically form liners are required to create the architectural patterns including but not limited to murals, fractured fin, and stacked block. For illustrations, refer to Appendix – 1.1.10, which showcases several aesthetic facing options.

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

  Tensar Response: The ARES system can accommodate gentle horizontal curves, as shown in page 12 of Appendix – 1.1.4.a and page 13 of Appendix – 1.1.4.b and Appendix – 1.1.4.c. The minimum radius is 58' for 5x5, 96' for 5x9 and 103' for 5x10 panel system. A tighter radius of 24' can be formed with 2ft wide narrow panels – refer to said appendices.

  Minor bend along the wall can be accommodated by placing the panel joint at the bend point and therefore does not require a corner element. On the other hand, a corner element is required when there is a corner or a significant inflection point along the wall. The standard corner is 90 degrees. Any angle between 45 degrees to 330 degrees can be achieved with the use of a corner element. Any angle less than 45 degrees will require special treatment.

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: The current ARES System connection utilizes one aperture of the UX1700 geogrid tabs cast into the concrete facing panels and spliced to the geogrid reinforcement in the backfill with a HDPE bodkin bars during construction. The specification for the bodkin connector can be found in Appendix – 1.2.5.b The UX1700 geogrid tab can be fabricated in one of two ways:

  1) Cutting one aperture directly from the manufactured UX1700MSE geogrid roll. This geogrid tab is called UX1700MSE tab.
  2) Cutting in the middle of the widened transverse bar of the UX1700MSE geogrid roll. This geogrid tab is called UX1700ARES (UX1700AR) tab.

  The UX1700MSE geogrid tab is being utilized in situations where the geogrid tab is to be field cast, normally, at the backwall of bridge abutments and when more than one aperture of geogrid tab is required to skew the geogrid around to avoid obstructions.

  The MPDS for UX1700MSE and UX1700AR are the same product properties and can be found in Appendix – 1.2.5.a. UX1700AR is a direct extension of the UX1700MSE geogrid where it is manufactured in a roll form with exactly the same starting sheet, same punches, same orientation process and resin. With UX1700AR, the feed spacing through the punch press is adjusted to create a wider transverse bar to allow for cut in the middle of the transverse bar. This fabrication process allows close monitoring of the production of each connection tab for bow and skew. In addition, sharp edges created by cutting through the ribs to create UX1700MSE tab that are undesirable from the handling and safety perspective are eliminated through this fabrication process.

  Based on the internal development testing and as verified with the long-term connection test attached in Appendix – 1.2.6, the minimum and maximum required pre-cut CMD bar thickness are determined. Product specifications and quality control requirements for pre-cutting the UX1700AR tabs are produced as shown in Appendix – 4.1.3. The target width of the pre-cut CMD bar is set to 16mm with an allowable tolerance of ±4mm.

- 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: The current ARES System connection utilizes one aperture of the UX1700 geogrid tabs (UX1700MSE or UX1700AR) cast directly into the concrete facing panels. The tabs are spliced to the geogrid reinforcement in the backfill with an HDPE bodkin bar during construction.

  A long-term connection test program was carried out in the Tensar Research and Development Laboratory in Morrow, Georgia to evaluate the long-term creep performance of the entire ARES system connection details. The connection test set up and test data are summarized in the long-term connection test report dated May 16th, 2011 provided in Appendix – 1.2.6.a. Based on the long-term connection test, the entire ARES system connection detail has a long-term creep connection reduction factor of 2.73 (CR_{cr} = 0.366). The calculated long-term connection strength is higher than the long-term design strength of the geogrid as shown in the report. The ARES system design is therefore controlled by the geogrid design strength, not the connection capacity. However, refer to Appendix – 1.2.6.c for the connection capacity analysis.

  The potential pullout of the geogrid tab from the concrete were evaluated by casting the geogrid tab 2.5" into the concrete blocks and allowed to cure close to 4000 psi compressive strength prior to testing. Once the concrete reached strength close to 4000 psi, short-term quick tests (10% per minute speed) were performed to measure the ultimate concrete-geogrid tab connection strength and the failure mode for each test. Detailed test set up and data is summarized in the "ARES Concrete Block-to-UX1700AR (UX1700MSE) Tab Connection Strength Test Report" dated August 25th, 2011 provided in Appendix – 1.2.6.b. Based on the testing, all geogrid tabs ruptured close to the concrete blocks instead of pulling out from the concrete blocks. The concrete-geogrid tab connection strength is calculated to be 100% of the geogrid ultimate strength.

- 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 ARES 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.

  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 ARES 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. See page 11 of Appendix –1.1.4.a and page 12 of Appendix – 1.1.4.b and Appendix – 1.1.4.c for details.

- 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 ARES panel. See sheet 10 Appendix – 1.1.4.a and sheet 11 of Appendix – 1.1.4.b and c for example of traffic barrier on top of ARES panel. Guardrail post can be installed or driven into the geogrid but should be placed 3ft away from the wall face. See Appendix – 1.3.5 for guardrail post on top of ARES wall.

- 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 page 6 of Appendix – 1.1.4.a and page 7 of Appendix – 1.1.4.b and Appendix – 1.1.4.c for standard slip joint details in plan and elevation view. Slip joints are precast reinforced concrete with the same height as the precast panels. If needed, slip joints within Tensar Ares retaining 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 Ares retaining 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. See sheets 7, 9 and 10 in Appendix – 1.1.4.a for standard details for 5'x5' panels. Also see sheets 8, 10 and 11 in Appendix – 1.1.4.b and Appendix – 1.1.4.c for 5'x9' and 5'x10' panels to overcome obstructions.

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: The simplified method based on AASHTO 2017-2020 was used to perform design example calculations. 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®

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 Ares 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: See Appendix – 3.1.2 for the construction manual for the Ares wall 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 1.1.11 for limitations of the facing unit installation at curved sections of the wall.

- 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: Tensar prefers to use corner panels to address curve sections of the ARES wall. Two ways to address overlapping geogrids:

  a) The UX geogrid can be cast and angled up to 15 degrees against the back face of the concrete panel to avoid or minimize the overlapping layers of reinforcement. See sheet 7 of Appendix – 1.1.4.a and sheet 8 of Appendix – 1.1.4 b and Appendix – 1.1.4.c for the skew geogrid details.
  b) Provide a 3-inch soil cover between overlapping geogrid layers. Sheet 6 of Appendix – 1.1.4.a and sheet 7 of Appendix – 1.1.4 b and c show a plan view of geogrid overlapping.

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

  Tensar Response: Measures that are required to maintain the vertical and horizontal alignment of the wall face during construction can be found in Appendix – 3.1.2, section "Panel Placement" on page 12 of 24.

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

  Tensar Response: For procedures on soil installation in the reinforced zone refer to Appendix – 3.1.2, section "Reinforced Fill Placement" on page 17 of 24.

- 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, section "Excavation, Foundation and Drainage" on page 11 of 24.

- 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 ARES 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.a 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: See Appendix – 1.1.8.b for the required quality control measures for the manufacturing of bearing pads. Refer to Appendix – 4.1.3 for the required quality control measures for the UX1700AR geogrid tabs. Also, refer to Appendix – 4.1.2 for Tensar geogrids quality control and quality assurance manual.

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: See Appendix – 3.1.2 for the Installation and Construction quality control measures for the Ares wall system.

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.

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

  - **Tanque Varde Project**: This project was located in Tucson, Arizona, USA. It was constructed between the years of 1984-1985, and it consists of the construction of 46 walls to separate grades at Tanque Verde – Wrightstown – Pantano. See Appendix – 5.1.2.0 for additional projects.

Figure 1 - Oldest ARES Retaining Wall System: Tanque Verde, Tucson, AZ

This ARES project used a geogrid tab/bodkin connection, at that moment was called geogrid tab/"bodkin joint" connection. For additional information refer to FHWA report for ARES Tanque Varde Project in Appendix – 5.1.2.1 and a Transportation Research Record report for ARES Tanque Verde Project in Appendix – 5.1.2.2.

- **Overlander Bridge Access**: This project was located in Kamloops, British Columbia, Canada, and it consists of 8 walls for bridge approaches, abutments, and mainline embankments. It was constructed in 1999. The two bridge was founded on spread footing on top of the reinforced soil mass. See Appendix – 5.1.2.0 for additional projects.

Figure 2 - Overlander Bridge Access Looking Southeast beneath the bridge deck

In addition, refer to paper about Overlander Bridge Access Project in Appendix – 5.1.2.3.

- **I-25 - 70th Street Ramp:** This project was located in Denver, CO, and it consist of two walls support ramp to bridge over I-25. It was constructed in 1993. The panels for this project had a "Glenwood Canyon" Striation and were stained. See Appendix – 5.1.2.0 for additional projects.

- **Harrisville Blvd CNR Overpass:** This project was located in Moncton, New Brunswick, Canada, and it consist of two walls supporting a single span bridge over two rail lines. Maximum height of 36 feet and it was constructed in 1998. See Appendix – 5.1.2.0 for additional projects.

- 5.1.3 Tallest Structures: The **tallest three structures**.

  Tensar Response:

  - **Loop 375 Border Highway West Extension Project:** This project was located in El Paso, Texas and consists of a maximum height of 42 feet. See Appendix – 5.1.2.0 for additional projects.

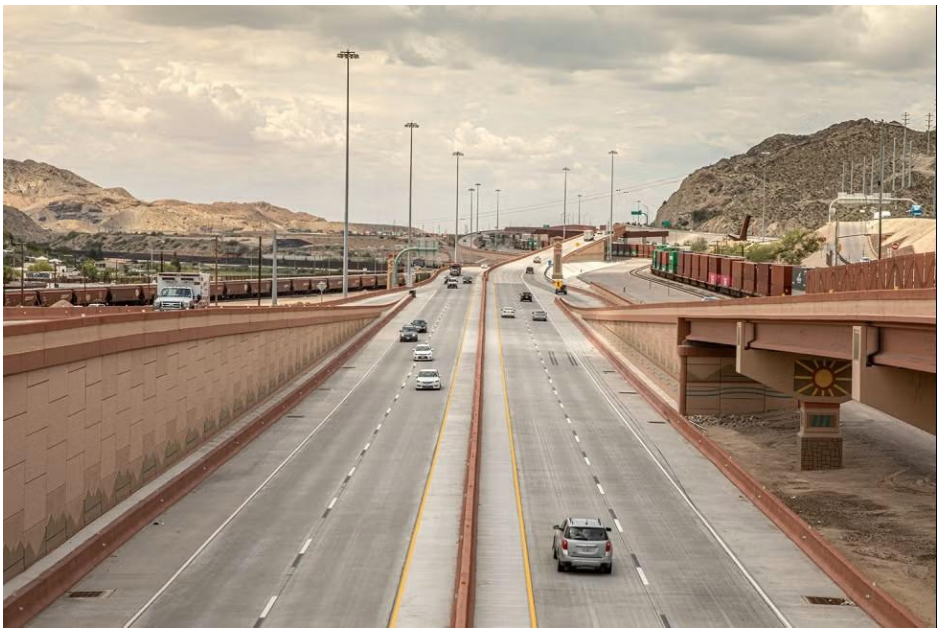


Figure 3 - Loop 375 Border Highway West Extension Project

Figure 4 - Loop 375 Border Highway West Extension Project

- **Memorial Causeway:** This project was located in Clearwater Beach, FL, and consists of a maximum height of 42 feet. See Appendix – 5.1.2.0 for additional projects.

Figure 5 - Memorial Causeway, FLDOT project, Pinellas County, FL: 42 feet, constructed in 2002

- **County Road 124 Overpass:** This project was located in Delaware County, OH, and consists of a maximum height of 40 feet. Refer to Appendix – 5.1.2.4 for a case study about this project.

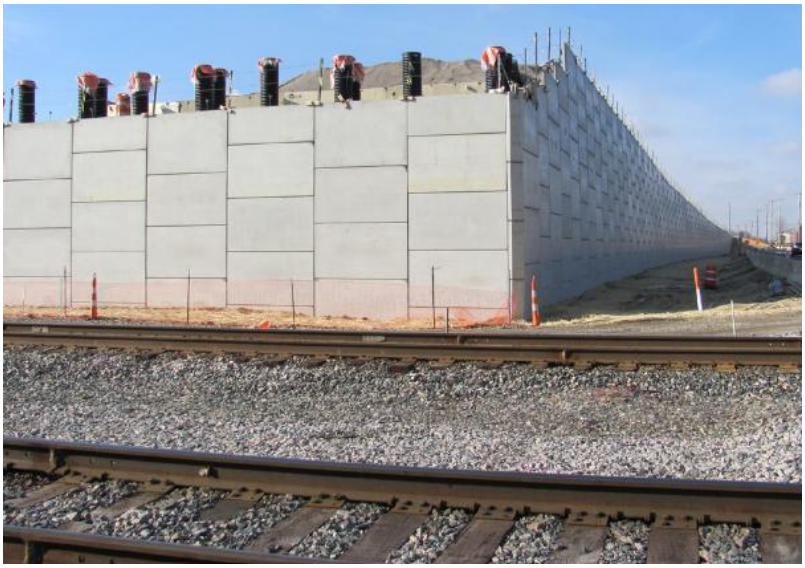


Figure 6 - County Road 124 Overpass

- 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 ARES Retaining Wall Systems have been widely approved and used by various public organizations. Here are some notable users and contact information for further inquiries:

1. **Eastgate Road Extension, Henderson, NV**
   a. **Contact**: Scott Fieldler
   b. **Phone**: (702) 267-3064
2. **SH 133, US 36 LYONS and E470, Denver, CO**
   a. **Contact**: Trevor Wang
   b. **Phone**: (303) 512-4072
3. **41st Street Overcrossing and NW 10th Street, WA**
   a. **Contact**: Tony Allen
   b. **Phone**: (306) 709-5450
4. **I-15 Auxiliary Lanes, 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. **US Army Crop of Engineers, Baltimore District, Baltimore, MD**
   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 Ares wall system, as provided by TIC.

Appendix 0

Tensar Grade Separation Solution Brochure

ARES® MSE Retaining Wall System

Back to main document

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.

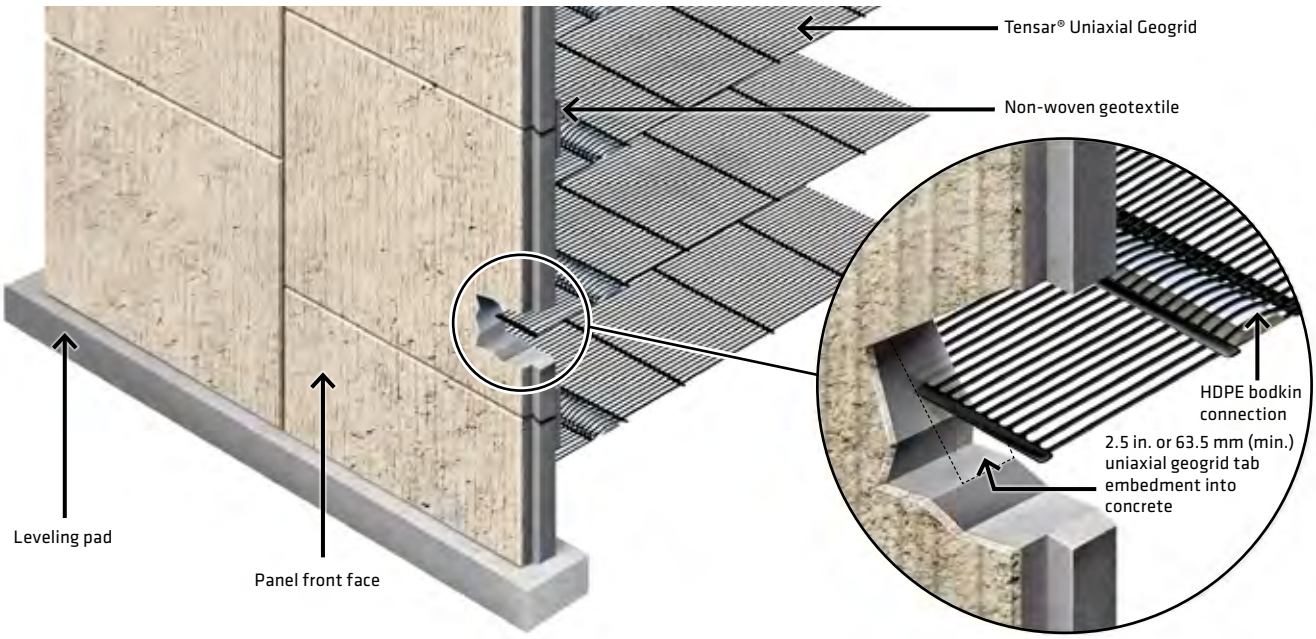
Tanque Verde Interchange – Tucson, Arizona

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.

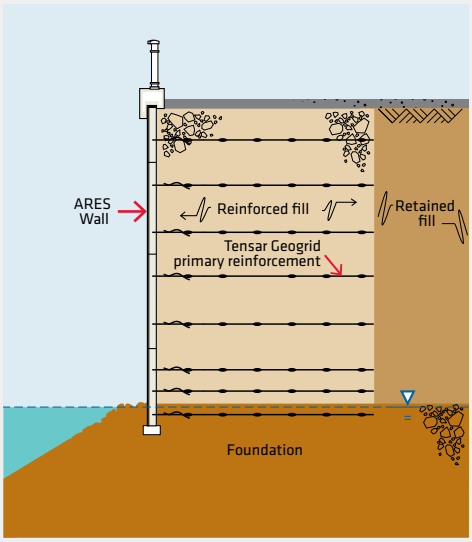
In addition to the various facing options available, contractors benefit from the time and labor costs saved when installing ARES Walls. The systems' panels have significant facing area, while the reinforcement is lightweight and easy to connect – no nuts or bolts to tighten.

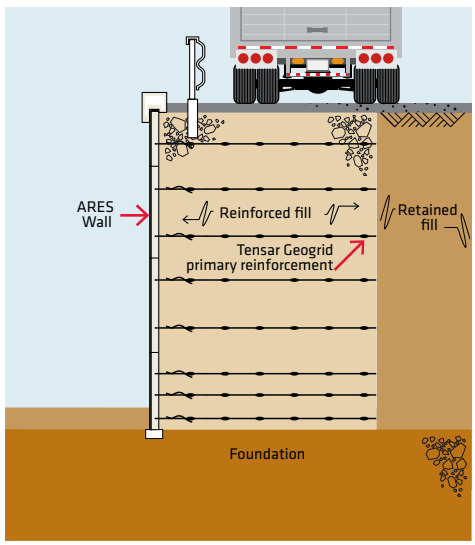
Memorial Causeway – Clearwater, Florida

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.

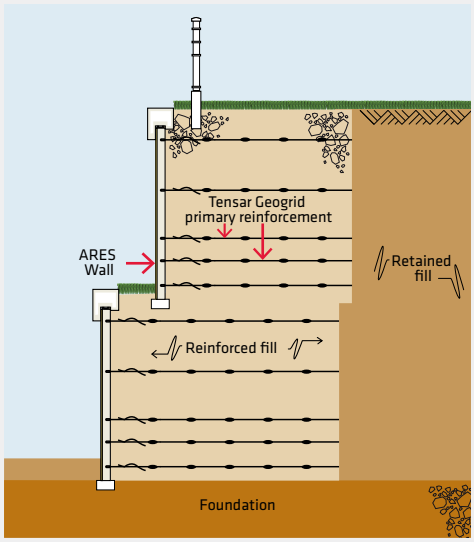




NOTE: *All details referenced in this brochure represent typical cross sections.*

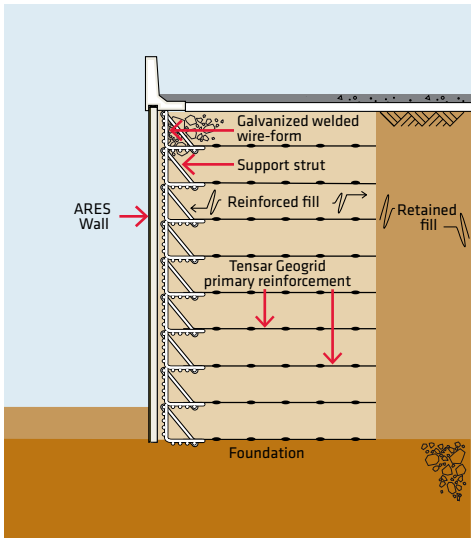
West Reynolds Road – Lexington, Kentucky

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



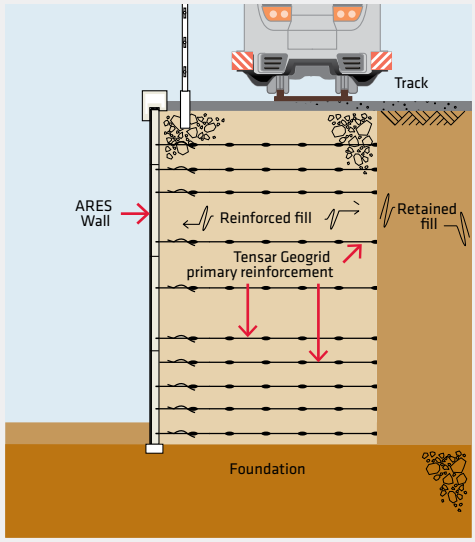
Third Street – Auburn, Washington

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.

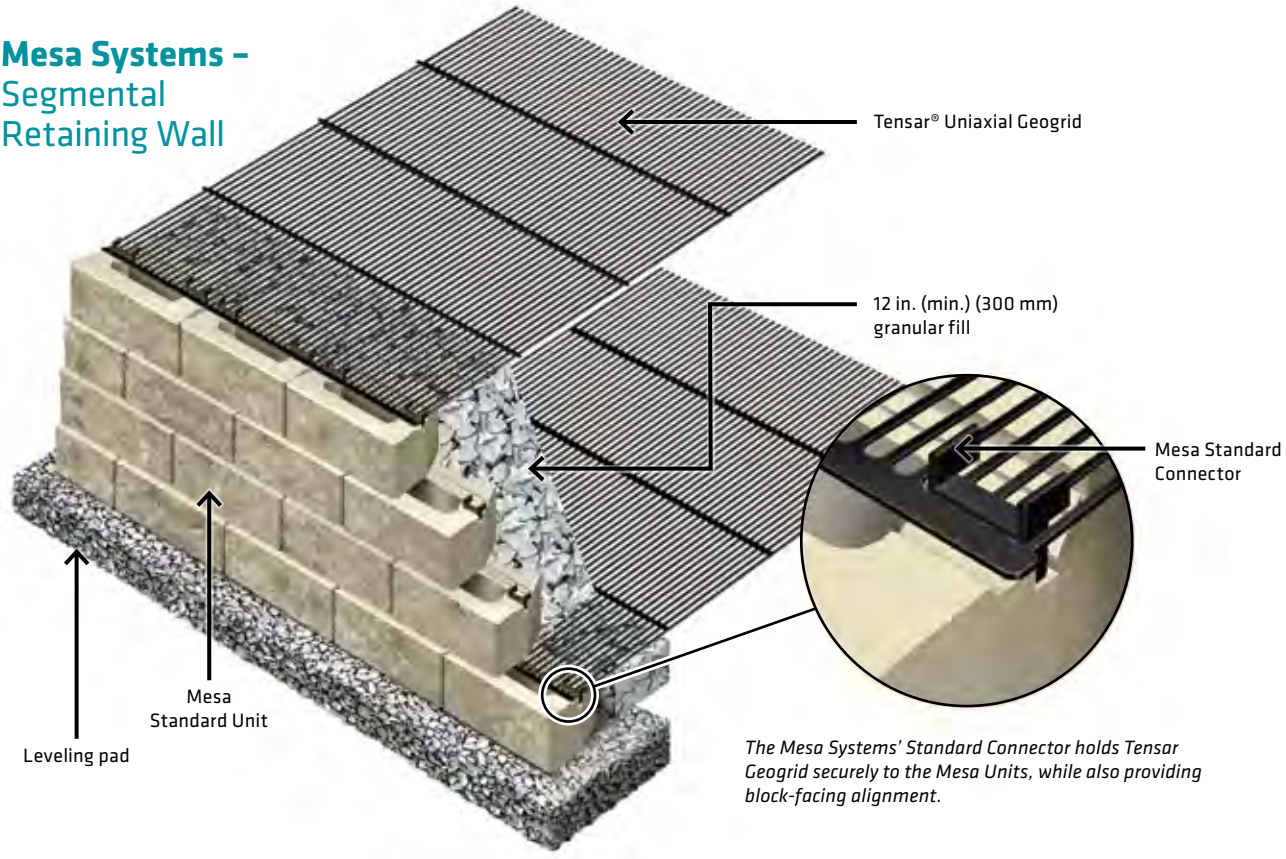
CREATIVE COMMERCIAL APPLICATIONS

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

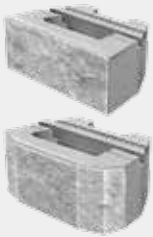
If your site requires a retaining wall, why not make it as attractive and useful as possible? In today's competitive and ever-changing wall market, architects and owners are continually looking for new ways to build walls that look as good as they perform.

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).

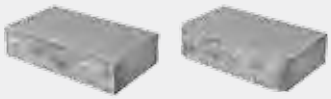
Standard Unit: 8" h x 18" w x 11" d nom./75 lbs (20 cm x 46 cm x 28 cm/34 kg)

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.

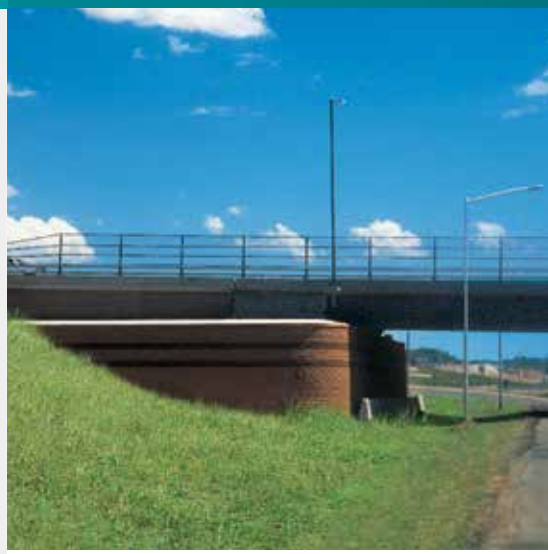
8" h x 18" w x 9" d nom./60 lbs (20 cm x 46 cm x 23 cm/27 kg)



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

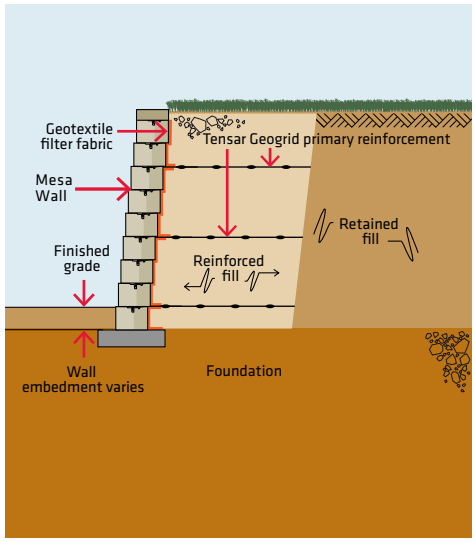
4" h x 18" w x 12" d nom./40 lbs (10 cm x 46 cm x 28 cm/18 kg)

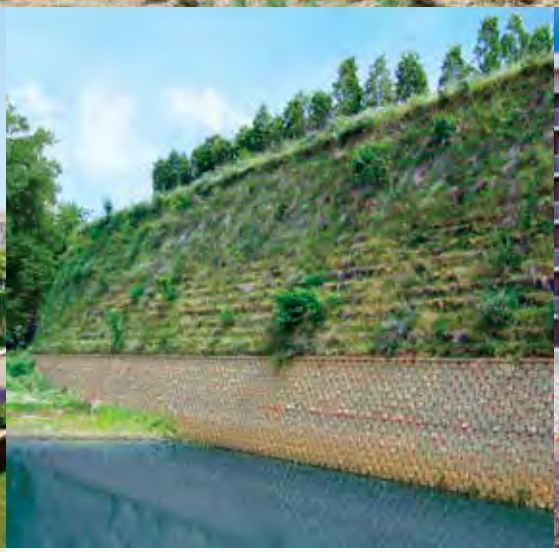
NOTE: *Actual dimensions may vary slightly depending on location.*



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

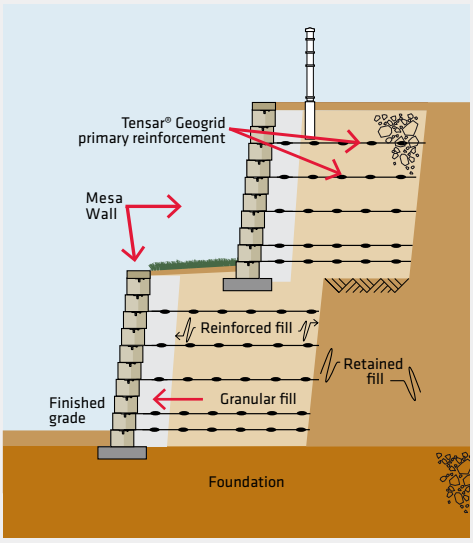
Varying elevations and tight property constraints posed a challenge for the residential site. A design-build solution provided over 48,000 sq ft (4,459 m²) of Mesa® Walls with heights exceeding 30 ft (9 m).

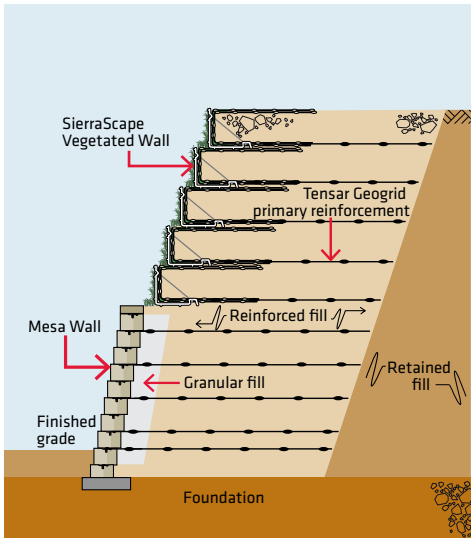
Brookworth Plaza – West Chester, Pennsylvania

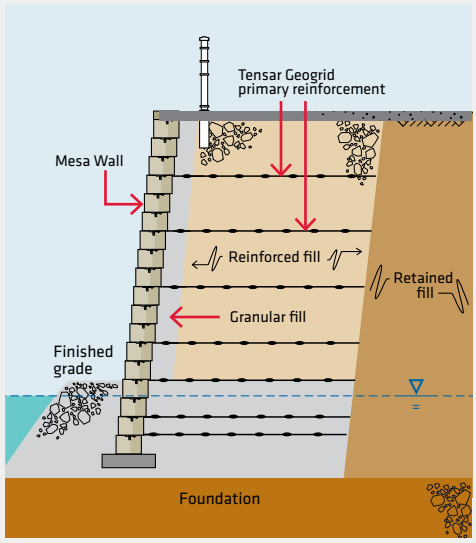
A combined installation of Mesa Walls and SierraScape® Walls provided a solution to the owner's request for green sightlines to the detention pond.

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

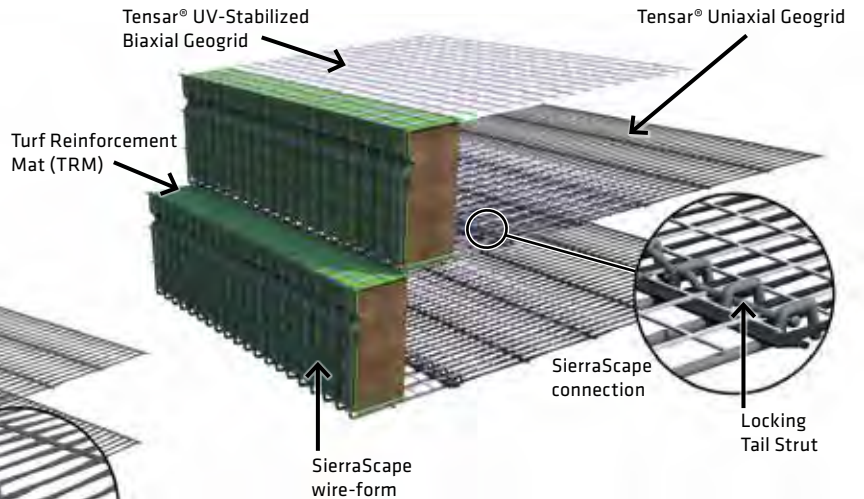
Unlike the stark appearance of most concrete wall systems, the SierraScape® System offers various facing options to meet your design needs. Options include:

- 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.

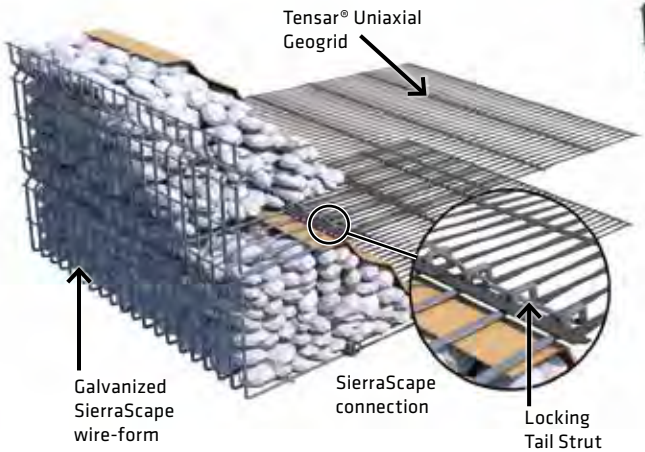
EASE OF INSTALLATION KEEPS COSTS DOWN

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

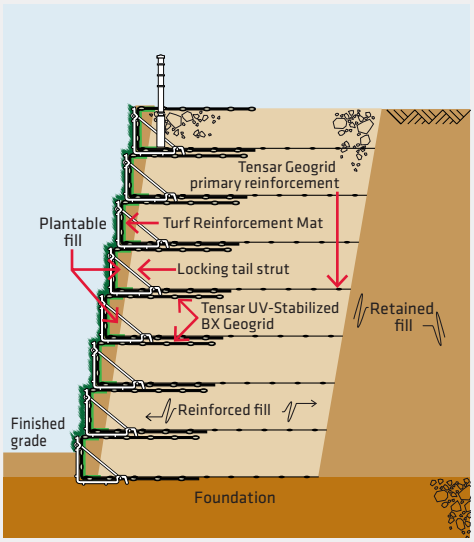
The SierraScape® System has increasingly become the system of choice for grade separation projects in the transportation, industrial, commercial and residential markets. By combining beauty and elegance with efficiency and performance, SierraScape Walls are built to stand the test of time.

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.

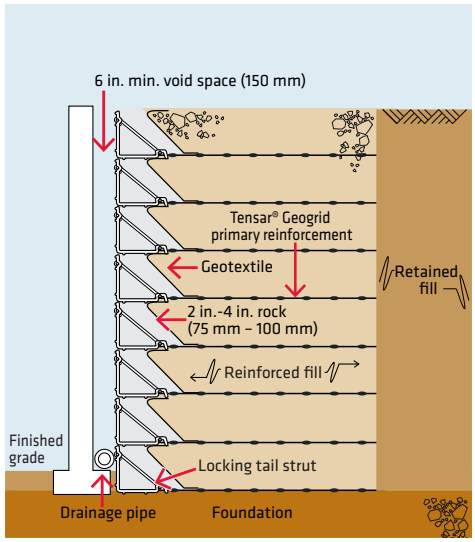
Post River Apartments – Atlanta, Georgia

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.



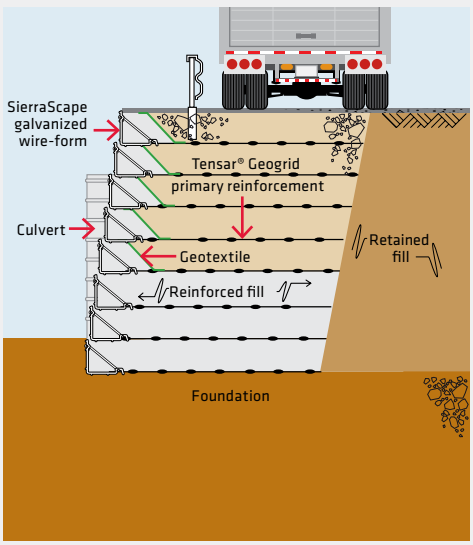
Brentwood Towne Square – Pittsburgh, Pennsylvania

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



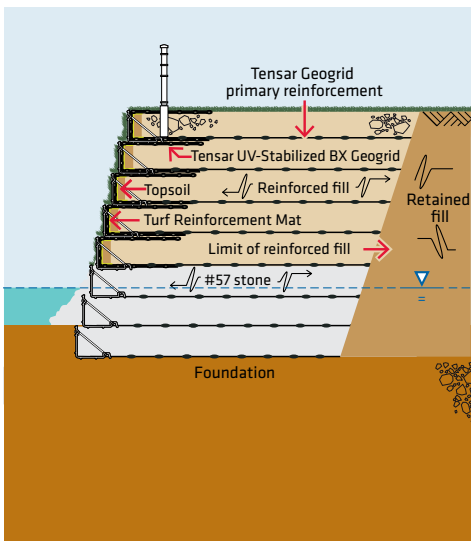
Vulcan Materials Plant – Columbia, South Carolina

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



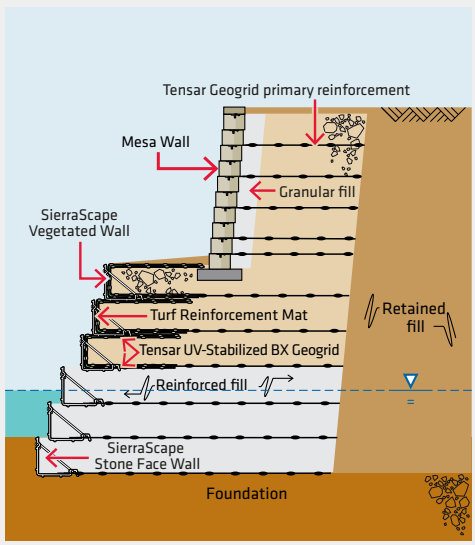
Quail Ridge – Kelowna, British Columbia

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

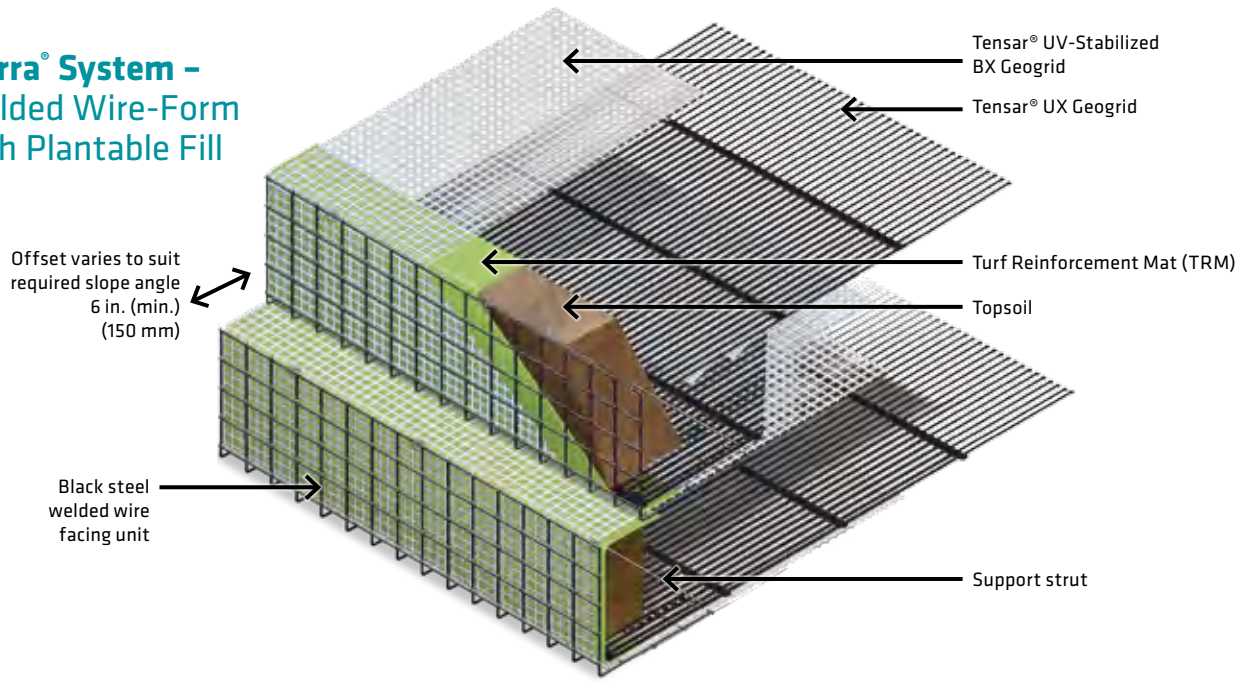
AESTHETICALLY PLEASING

- 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

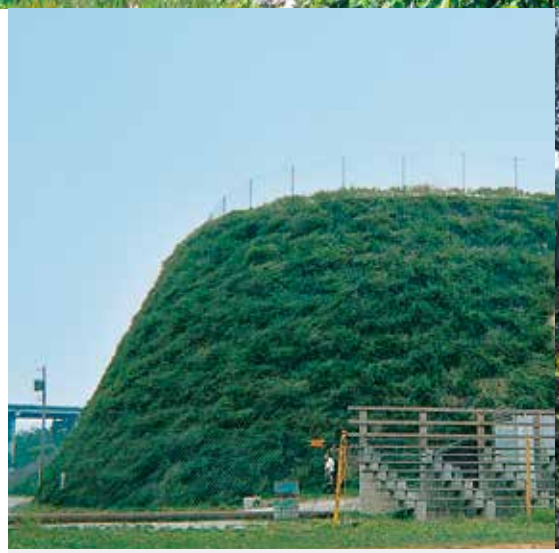


Optimal Designs Based on Site Constraints

SITE-SPECIFIC SOLUTIONS

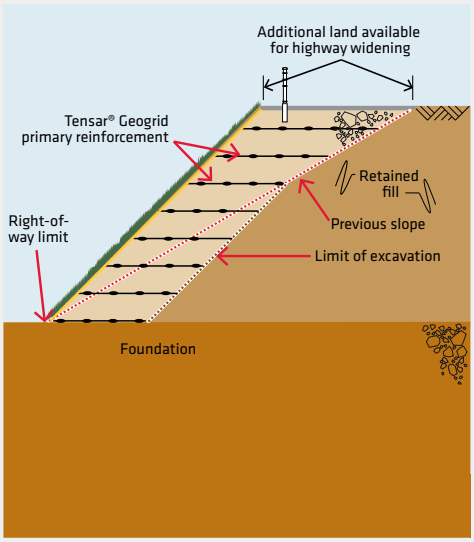
The Sierra® System was developed to provide designers and specifiers countless options for layout and slope angles. The flexibility of the system makes it an ideal choice for solving grade separation challenges.

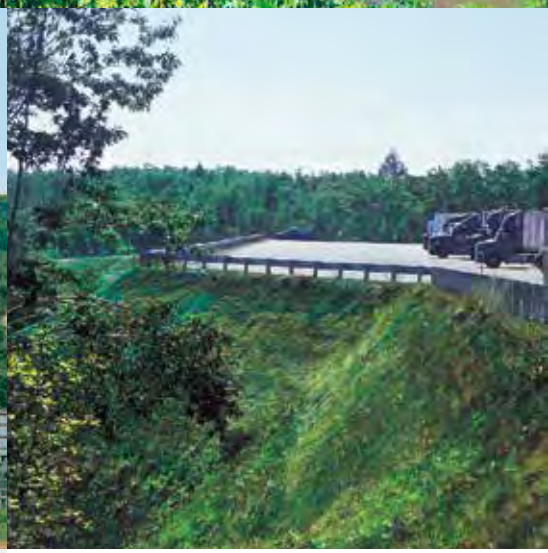
Designers and owners are continually faced with unique property and site constraints that require innovative solutions. Some projects may require a very steep grade change in which a green, vegetated face is the right solution. Other projects may incorporate complementary grade separation systems, such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



Bridge Street – Irwin, Pennsylvania

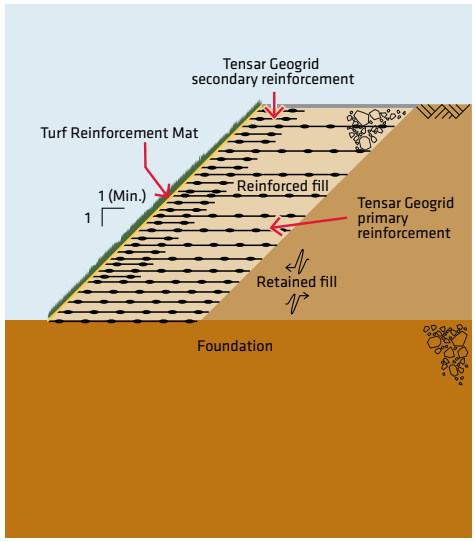
This project demonstrates how DOTs can use the Sierra System to stay within a restricted right-of-way while maximizing land use.





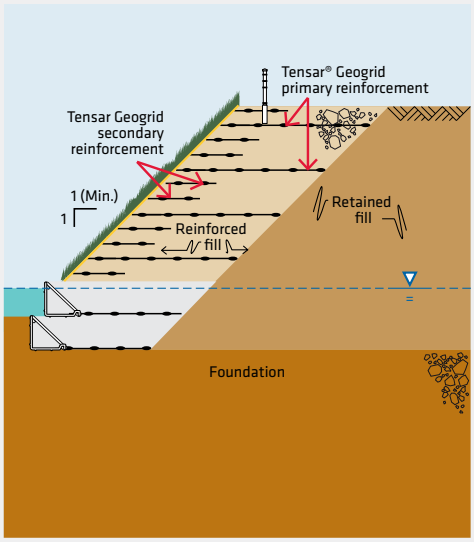
Poland Spring Plant Expansion – Poland Spring, Maine

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



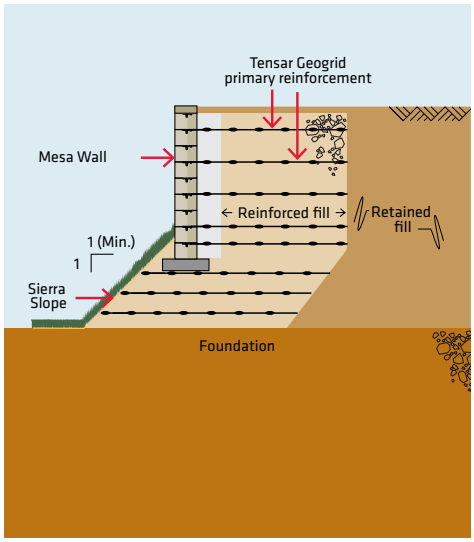
Marriott Riverchase – Hoover, Alabama

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.



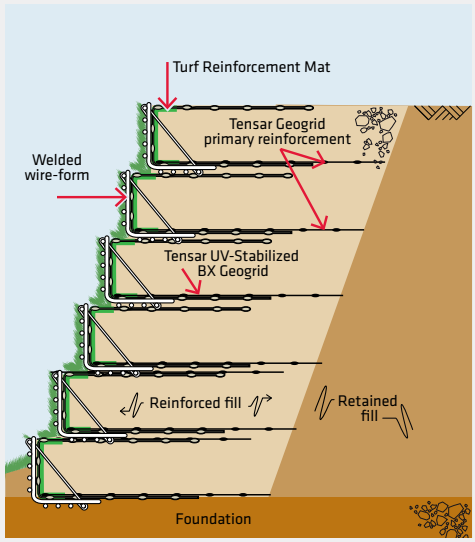
Pearl Street – Braintree, Massachusetts

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.



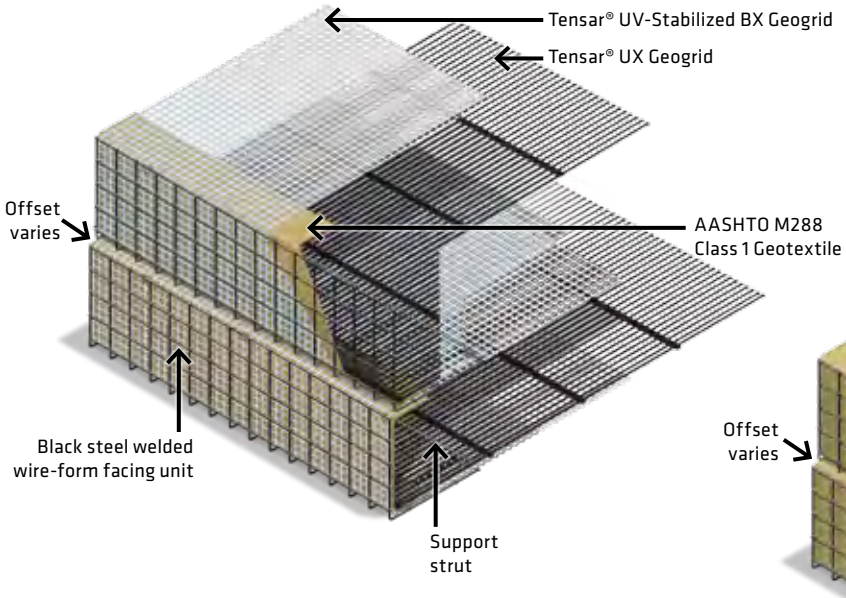
With lower cost materials, reduced labor and the use of lightweight installation equipment, a Tensar® Temporary Wall is the economical alternative.

TENSAR® TEMPORARY WALL

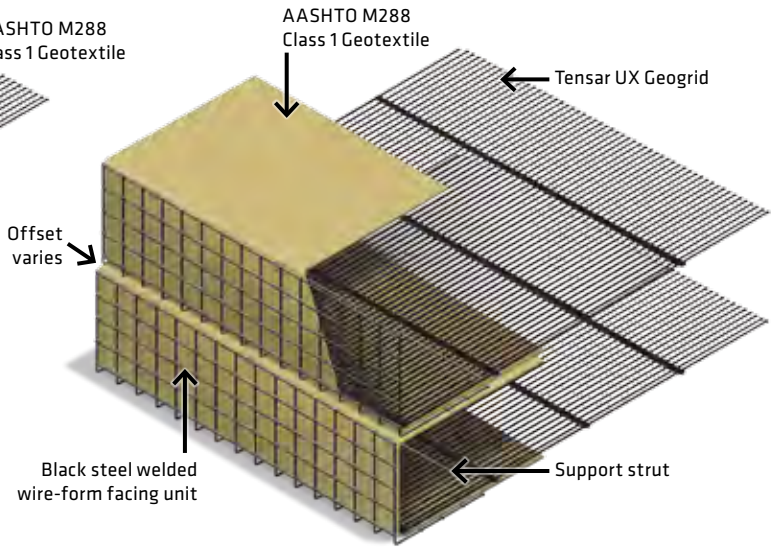
Temporary walls are a necessity for many types of staged construction, but the conventional means for installing them are expensive, requiring heavy lifting and pile-driving equipment. Structures such as soldier piles and lagging walls or sheet piling typically require toe penetration equal to or greater 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.

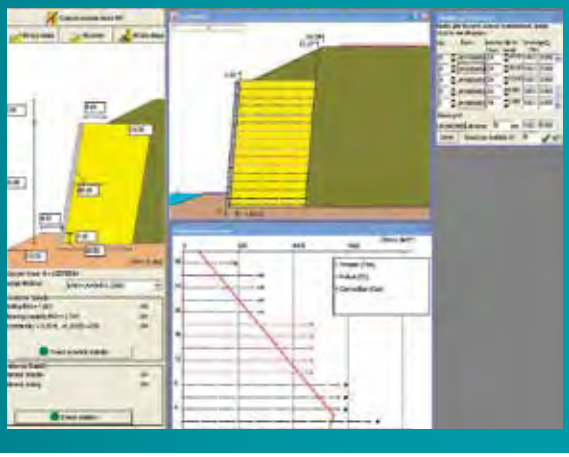
Fortunately, there is a proven technology that allows you to build temporary walls without the challenges and expense of traditional techniques. The Tensar Temporary Retaining Wall System uses an inexpensive wire-form facing system along with Tensar® Geogrids to reinforce the fill. As a result, Tensar 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.

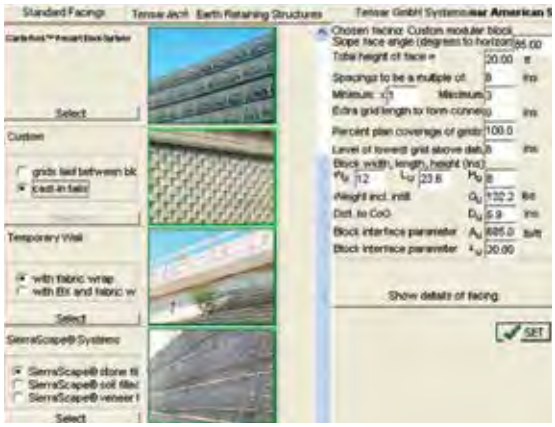
TENSARSOIL-PRO SOFTWARE IS COMPATIBLE WITH ALL MAJOR DESIGN METHODOLOGIES

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.

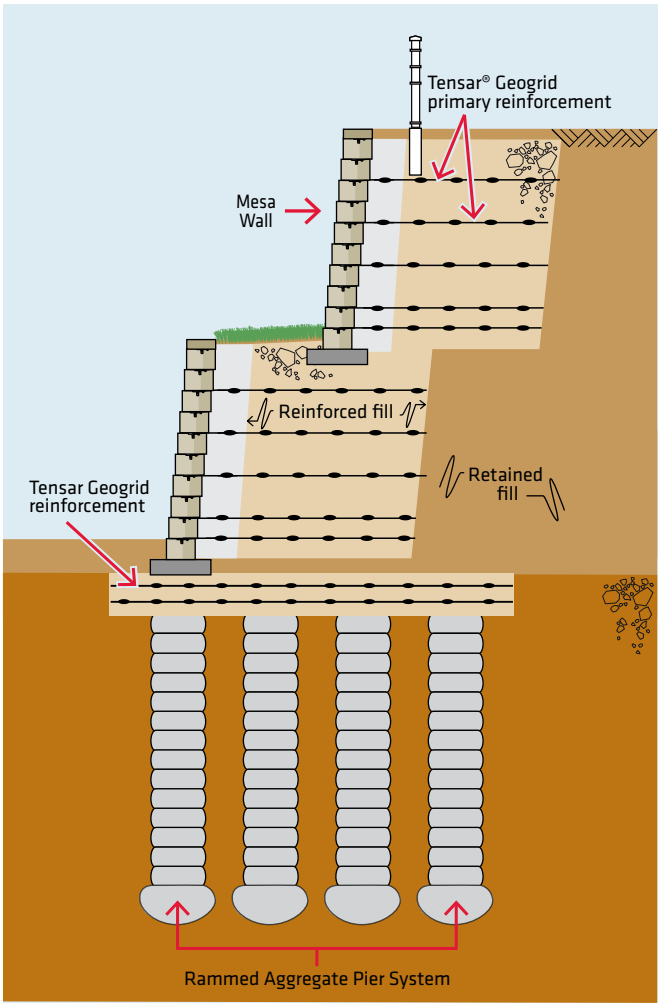
Tensar Corporation Provides Complete Site Solutions

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.



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GSS_BRO_6.14

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

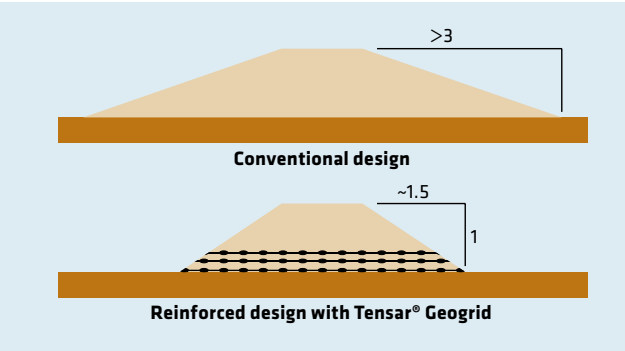
These projects are expensive and time-consuming when built using traditional means. However, with its composite soil and geogrid structure, the Prism System creates a reliable and cost-effective embankment foundation, either independently or in conjunction with other methods.

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 1.1.3.a

Ares Precast Quality Control Guide

ARES® MSE Retaining Wall System

Back to main document

ARES®
RETAINING WALL SYSTEMS

PRECASTER MANUAL

I. PURPOSE

This document is intended to supply the Precaster and Quality Control / Inspection Staff with the guidelines and criteria required to facilitate the casting and quality control of the concrete facing panels as well as proper maintenance of the forms and accessories supplied by Tensar Earth Technologies, Inc. (TET). To this end this Manual should be supplied to the Precaster and the Quality Control / Inspection Staff.

II. DEFINITIONS

"Project Purchase Agreement" where referenced below includes the Purchase Order issued by TET to the Precaster, and where applicable, the Precaster Contract between TET and the Precaster.

"Project Contract Documents" where referenced below includes the Project Plans and Specifications issued by the Project Owners or their representative.

III. RESPONSIBILITIES

The Precaster is responsible for all work and materials set forth in this document and the Project Purchase Agreement, including the following:

- Providing materials as set forth in Section III.B, below,
- Providing all labor, materials, equipment and accessories not specifically supplied by TET, as required to clean and maintain the forms, and complete the finished panel. This includes form setup, concrete placement and vibration, stripping, curing, storage, protection and shipping.
- Providing finished product conforming to the dimensions, quality, finish, and other requirements of TET's drawings and the Project Contract Documents,
- Assuring acceptability of the finished product to the Owner and the Owner's Engineer,
- Coordinating with the Contractor regarding the delivery of the finished product to the job site,
- Properly maintaining and storing the forms in accordance with this document and as necessary to prevent damage to the forms or deterioration of the forms.

TET is responsible for the following:

- Providing materials as set forth in Section III.A, below,

- Providing shop drawings including all dimensions and details required for casting each type of panel.
- If requested, providing technical assistance by a TET Technical Advisor at the start of the project to assist the Precaster in the initial set up and to advise him of the recommended procedure within the scope of this document. Note: The TET Technical Advisor is **not** a member of the Quality Control / Inspection Staff.

III. MATERIALS

A. Items Supplied by TET (FOB the Precaster's Plant)

- Panel forms complete with base plate, top and bottom rails, and moveable side rails.
- Geogrid tabs to be embedded in panels

B. Items supplied by the Precaster

- Concrete as specified in the contract documents (4,000 PSI minimum or per the Project Contract Documents, whichever is more stringent).
- Testing or certification of the concrete as required in the Project Contract Documents.
- Reinforcing steel (wire mesh or bars) as required in the contract documents or as shown in the Shop Drawings.
- Lifting inserts for setting the panels in the field, two (2) ton Burke Rapid Lifts per panel.
- Rubber mallets to aid in form release. **Steel hammers shall not be used on the forms.**
- Bulkheads for special panels.
- Materials required for leveling and raising forms to desired working elevation.

Note: If an architectural finish is required, the liners and other materials required for that purpose shall be provided by TET, unless specifically shown otherwise on TET's Purchase Order to the Precaster.

IV. DELIVERY OF THE TET SUPPLIED MATERIALS

A. Forms

- The forms will be delivered on a flatbed trailer and can be removed using a forklift. Forms are provided based on nominal 5x5 panels and 5x9 panels, each in two standard sizes. For the 5x5 panels, the forms weigh approximately 1500 lbs. and 2000 lbs. respectively. The out to out dimensions are as follows: standard form, 6'-0" long by 6'-0" wide by 1'-6" deep; and the large form, 7'-9.5" long by 6'-0" wide by 1'-6" deep. For the 5x9 panels, the forms weigh approximately 2000 lbs. and 2500 lbs. respectively. The out to out dimensions are as follows: standard form, 10'-9" long by 6'-9" wide by 1'-6" deep; and the large form, 10'-9" long by 8'-3" wide by 1'-6" deep.

- The forms are fitted with four short legs, approximately 4" high or tubular steel bases which are structurally capable of supporting the weight of the form bed and the concrete. The forms must be handled by placing the lifting devices adjacent to these members.
- The forms will be delivered in a ready to use condition and may have a protective coating. If so, the protective coating must be removed by the Precaster before placing the concrete.
- It is the Precaster's responsibility to inspect the forms prior to unloading. The Precaster shall note any damage to the forms on the shippers bill of lading and immediately notify TET (1-800-292-4459).

B. Geogrid Tabs

The geogrid tabs will be shipped in pre-cut lengths. The tabs should be placed such that one transverse bar is embedded in the concrete and one transverse bar is outside the panel face in order to make the Bodkin connection. The tabs should be placed such that the embedment is uniform across the length of the tab. In the event that pre-cut tabs are not allowed by the Project Specifications, full rolls of geogrid will be sent and will be required to be cut by the Precaster.

C. Liners

Architectural form liners, if required, will be supplied by TET. All liners are the property of TET and should be returned at the finish of the project.

D. Incidentals

All incidental items listed in section III shall be shipped direct from TET. All items must be cataloged by the Precaster and returned to TET at the finish of the project.

V. SETUP REQUIRED PRIOR TO CASTING

A. Setting up the casting beds

- The forms should be placed such that there is sufficient working room between them and to permit the forklift to approach the panel parallel to the tabs. This will enable the panels to be removed from the form using the tabs as the lifting points.
- The forms must be leveled at the legs or the tubular bases to prevent warping of the form. **A warped form will result in a warped product.**

B. Preparation of the form

- The tops and sides of the forms are bolted to the base plate. **It is not necessary to torque the bolts.** When the tool stops freely rotating, one small twist will assure proper contact and allow for easy removal upon completion of the pour.
- A **light film** of form oil must be applied to the base plate and sides rails, using a saturated cloth, before each pour. Spraying of form oil

can create excess accumulation of the oil and will often stain the panel face which can be cause for rejection.

C. Installation of bulkheads

- Top panels shall be smooth on the top and will not have the associated lip. Bulkheads shall be used and installed at the proper location and braced to prevent distortion. If required by the shop drawings, rebar dowels shall be installed through the bulkhead into the top of the panel.
- Bottom panels shall be smooth on the bottom and will not have the associated lip. Bulkheads shall be used and installed at the proper location and braced to prevent distortion.
- Cut panels, which are narrower than the full width of the standard or large panel and also require a smooth edge with no lip on one or possibly two sides. Forming for cut panels can be done with bulkheads.
- Narrow panels are similar to cut panels with the exception of having lips on both sides. For narrow panels, bulkheads can be fabricated or one side rail can be moved in and clamped or braced at the proper dimension.

All dimensions required to cast each panel will be included on the Shop Drawings provided by TET.

VI. CASTING, STRIPPING OF THE CONCRETE FACING

A. Placement of reinforcing steel or welded wire mesh

- Reinforcing shall be provided, cut, and tied by the Precaster.
- Reinforcing requirements for each panel shall be as provided on the shop drawing.
- The reinforcing shall be located as shown on the shop drawings and secured in the form prior to the placement of the concrete. Reinforcement can be hung from the geogrid tabs.

B. Placement and consolidation of the concrete

- The concrete shall be placed in the form and consolidated using internal or external vibrators as determined by the precaster. The quality of the resulting consolidation is the sole criteria for the methodology selected.
- The concrete in the panel shall be screeded to provide a level concrete finish across the back of the panel.
- Complete the float finishing of the panel.
- Additional lifting inserts should be paced in the back of the panel for removing the panel from the firm.

C. Stripping and removing the concrete facing

- Assure that the concrete has attained its stripping strength.

- Remove the form bolts from necessary side form panels and place in their holding slots.
- Rap the side rails and slot blade assembly with the rubber mallet to aid in release.
- Remove the necessary side rails.

The precaster shall stencil the panel identification on the back of the panel. It is also recommended that the panel identification be written on the side of the panel with a marker.

- The panels can be removed from the form base by using the lifting inserts in the back of the panel.
- Clean the side panels and base plates, paying particular attention to the area where latence can form between the base plate and the chamfer on the side forms.
- Reassemble the form and repeat the process.

VII. PANEL HANDLING

A. Storage

- Panels should be stored in the same position in which they were poured. The storage yard should be stabilized so that rain will not stain the face as a result of splash.
- The panels should not be stacked more than 5 units high. The dunnage between each panel must line up with the panel below it. If not, the panels can warp or crack which could be cause for rejection. It is recommended by TET that a plastic or styrofoam pad be attached to the top of the wooden dunnage to prevent staining of the finished face.

B. Shipping and Delivery

- The precaster shall coordinate delivery of the panels to the job site with the contractor.
- The precaster shall load panels and assure that proper dunnage and straps are used to prevent damage to the panels during transport.

Delivery tickets should note that the Contractor will be responsible for returning the dunnage on subsequent shipments.

- Precaster shall allow two hours of unloading time by the Contractor.

VIII. COMPLETION OF THE PROJECT

After casting for the project has been completed the precaster must perform the following operations:

1. Completely clean all form surfaces and remove any masonry build-up on the form substructure.

2. Note any repairs that are required prior to the next use of the form and forward to TET prior to the release of final payment.
3. Lightly coat all of the form surfaces that come in contact with concrete with form oil.
4. Reinstall the top, bottom and side forms and assure that all the bolts are in place.
5. Band the backform and other accessories and place them with the forms.
6. Forms may be stored by stacking them in the yard provided the dunnage has been placed to provide proper support as noted in the discussion above. Forms must be covered to protect them from the elements.
7. When forms are to be stored for an extended period of time, all exposed surfaces must be coated to prevent rust.

End of Document

Appendix 1.1.3.b

Panel Steel Design

ARES® MSE Retaining Wall System

Back to main document

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Alpharetta, GA 30009
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February 15, 2026

IDEA Evaluation Review Team
Geo-Institute
American Society of Civil Engineers

Subject: Reinforcing steel design for Precast Panels:
Pre-Submittal Review Phase for the ARES application
Tensar ARES MSE Retaining Wall System
Review Comments dated on September 4, 2025

Per your request, Tensar International Corporation (TIC) is pleased to submit this brief report and calculations for the subject project. The purpose of this work was to design the reinforcing steel in precast concrete wall panels that are a part of a Tensar Mechanically Stabilized Earth (MSE) retaining wall system for the subject project.

Our structural reinforcing steel design is based on the following technical information:

- Panel geometry and dimension presented in Shop drawings attached.
- Our design was based on applicable sections of the AASHTO-LRFD Bridge Design Specifications, 10th Edition 2022.
- Concrete compressive strength f'c of 4,000 psi. Panel thickness = 5.5".
- Reinforcing Steel shall be ASTM A615 Grade 60 and welded wire fabric shall be ASTM A82 minimum yield of 60 ksi. The size and location for the two panel considered are listed below:

Panel Type	Max. Soil Pressure (psf)	Vertical Rebar	Horizontal Rebar	Wire Mesh Alt.
Standard (A)	1,300	#4 Bar @ 14" o.c.	#4 Bar @ 16" o.c.	6 x 6 – W7.5 x W8.5
Special (AG14)	2,000	#4 Bar @ 14" o.c.	#4 Bar @ 16" o.c.	6 x 6 – W7.5 x W8.5

Sincerely,
Tensar International Corporation

Jorge Peralta, E.I.T.
Design Engineer, Walls & Slopes

cc: Willie Liew, P.E., Chief Engineer
cc: Jong, Lee, P.E., Engineering Manager

TABLE OF CONTENTS

Design Summary 2
Applicable Codes & Load Cases 3
Loading Analysis ..... 3 - 4
Calculations ..... 4 – 10
Conclusions 11
Panel Shop Drawings 12-16

Design Summary

This calculation report provides a structural design analysis of the reinforcing steel design for Precast concrete MSE panels manufactured by TIC. Existing calculations for the panel reinforcing steel design were performed in 2007 by TEC Services, Inc. Due to the fact that this company is not providing these services anymore, TIC is performing an updated calculations for the panel design to be compliant with AASHTO 2022 Standard (AASHTO LRFD 10th Edition.) The panels have updated parameters including a concrete strength of 4000 psi, panel thickness of 5.5 inches, and maximum horizontal geogrid line load of 3,382 lb/ft. The analysis is performed for two ARES panels, the standard panel (A) with standard geogrid tab spacing, and the special panel (AG14) with geogrids spaced more closely. The updated reinforcing steel shall be ASTM A615 Grade 60, and the welded wire fabric shall be ASTM A82 with minimum yield strength of 60 ksi. The design of the reinforcing steel of each panel is as follows:

Panel Type	Max. Soil Pressure (psf)	Vertical Rebar	Horizontal Rebar	Wire Mesh Alt.
Standard (A)	1,300	#4 Bar @ 14” o.c.	#4 Bar @ 16” o.c.	6 x 6 – W7.5 x W8.5
Special (AG14)	2,000	#4 Bar @ 14” o.c.	#4 Bar @ 16” o.c.	6 x 6 – W7.5 x W8.5

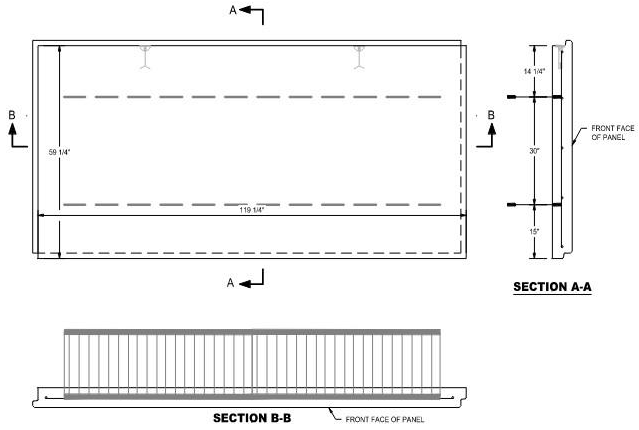


Figure 1 - Panel A

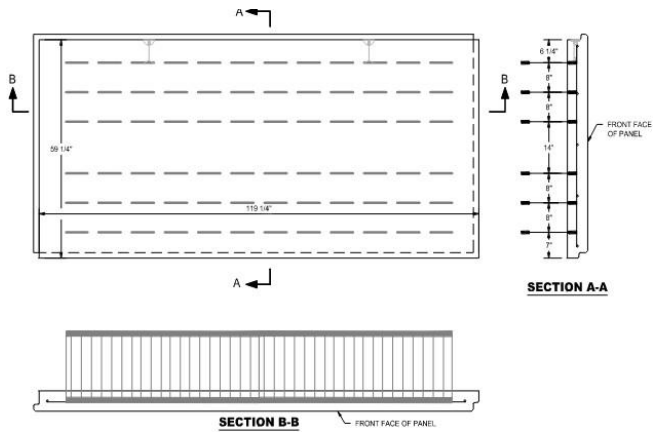


Figure 2 - Panel AG14

Applicable codes & Load Cases

Codes and design manuals applicable to the project are as follows:

- AASHTO LRFD Bridge Design Specifications, 10th Edition
- Building Code Requirements for Structural Concrete (ACI 318-19)

Loading Analysis

The structural reinforcing steel design calculations were based on the panel geometry, dimensions, and geogrid line spacings in shop drawings. The calculations were performed with geogrid horizontal line factored load capacity of 3,328 lb/ft (applying a ultimate load factor of 0.9D). To determine the structural steel reinforcement required, maximum soil pressure that would cause the geogrid to reach its line load capacity was determined. From the maximum soil load for the given geogrid configuration, the maximum moment and shear demands on the MSE panel were calculated. Vertical and horizontal reinforcement were designed accordingly to have sufficient capacity for the moment and shear demands. The steel reinforcement was designed to be compliant with AASHTO LRFD 10th Edition's minimum specifications for temperature and shrinkage of the panel, minimum tensile reinforcement for flexure, and applicable spacing requirements. The required minimum area of reinforcement was used to provide sufficiently sized steel wire mesh as an alternative to steel rebar design.

<u>Design Narrative:</u>

- Previous panel structural calculations need update to be compliant with AASHTO LRFD 10th Edition
- New design parameters should include:
    - Concrete strength, f'c = 4,000 psi
    - Panel thickness, tp = 5.5 in
    - Panel Types:
        - Standard Panel (A): Standard tab spacing
        - Special Panel (AG14): Geogrid tabs spaced ~ 8"
- Check panel for flexural & shear demand
- Determine size & spacing of rebar & check capacity of wall
- Allowable anchor load = 3,382 lb/ft
- Design will be performed using AASHTO LRFD 10th Edition
- Span of panel = 119.25" ; Height = 59.25"
- With As_min, provide suitable wire mesh alternative

Panel Dimensions:

Geogrid Edge Distance:
X1 = 15" (Standard)
X2 = 14-1/4" (Standard)

Total Panel Span, L = 59.25"
Panel Width, B = 12" (1' strip)
Panel Thickness, t_p = 5.5"

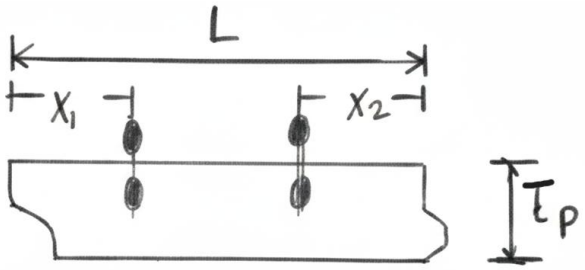


Figure 3 – Panel Dimensions

Concrete Properties:

f'c = 4,000 psi = 4.00 ksi
E_c = 57,000 * sqrt(f'c) — Assume normal weight concrete
E_c = 3,604,997 psi

Steel Properties:

f_y = 60,000 psi = 60 ksi
E_s = 29,000,000 psi = 29,000 ksi

Section Properties:

Rectangle Area Moment of Inertia
I = b*d^3 / 12 = (12")*(5.5"^3)/12 = 160.4 in^4

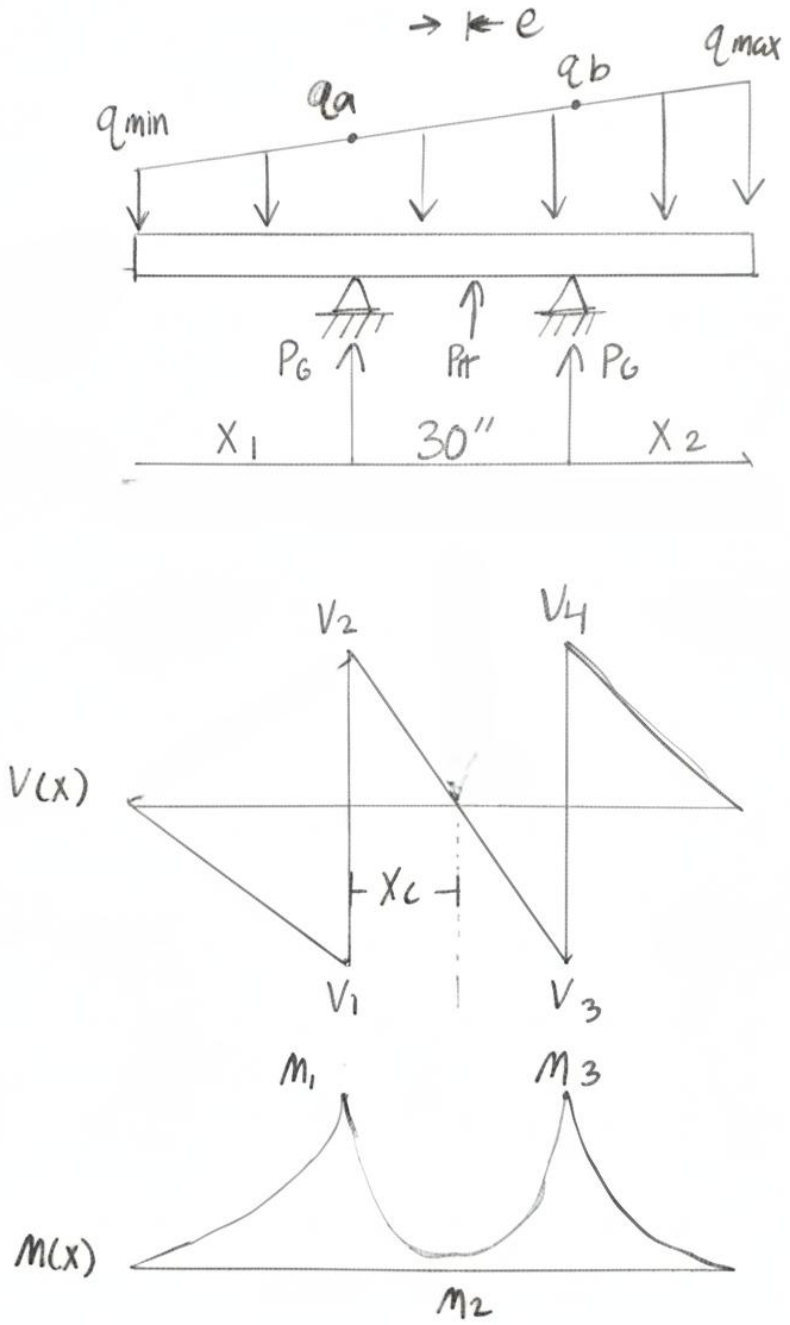


Figure 4 – Loading, Shear, and moment diagrams

Excel calculations to determine Max. Shear and Moment Demand are presented below:

Description	Parameter: Standard Panel	Value	Units	Notes
Panel Member Dimensions	Span_L	59.25	in	Given
Panel Member Dimensions	Span_L	4.94	ft	Given
Panel Member Dimensions	Strip_b	12.00	in	Given
Panel Member Dimensions	Strip_b	1.00	ft	Given
Panel Member Dimensions	Thickness_t	5.50	in	Given
Panel Member Dimensions	Thickness_t	0.46	ft	Given
Panel Member Dimensions	Geogrid_X1	15.00	in	Given
Panel Member Dimensions	Geogrid_X1	1.25	ft	Given
Panel Member Dimensions	Geogrid_X2	14.25	in	Given
Panel Member Dimensions	Geogrid_X2	1.19	ft	Given
Panel Member Dimensions	Anchor Capacity_Pg	3,382.00	lb	Given (analyzing 1' strip)
Material Properties	Concrete_f'c	4,000.00	psi	Concrete Strength
Material Properties	Ec	3,604,996.53	psi	Conc. Modulus of Elasticity
Material Properties	Ec	3,605.00	ksi	Conc. Modulus of Elasticity
Material Properties	Steel_fy	60,000.00	psi	Steel Yield Strength
Material Properties	Es	29,000,000.00	psi	Steel Modulus of Elasticity
Material Properties	Es	29,000.00	ksi	Steel Modulus of Elasticity

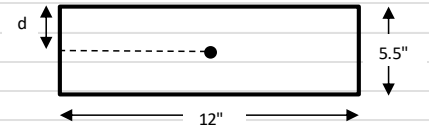
Description	Quantity	Formula	Value	Units
Resultant force	P_t	2 * P_g	6,764.00	lb
Eccentricity	e	(L - x1 - x2)/2 + x1 - L/2	0.0300	ft
Max distributed load	q_max	[P_t * (1 + 6e/L)] / (B * L)	1,419.12	plf
Min distributed load	q_min	[P_t * (1 - 6e/L)] / (B * L)	1,319.34	plf
Distributed load slope	Slope	(q_max - q_min) / L	20.20	plf/ft
Load at x1	q_a	(q_max - q_min) * (x1/L) + q_min	1,344.59	plf
Load at L-x2	q_b	(q_max - q_min) * ((L - x2)/L) + q_min	1,395.09	plf
Shear segment 1	V1	- q_min*x1 + 0.5*x1*(q_a - q_min)	-1,633.39	plf
Shear after anchor 1	V2	V1 + P_g	1,748.61	plf
Shear segment 4	V4	q_b*x2 + 0.5*x2*(q_max - q_b)	1,674.45	plf
Shear before anchor 2	V3	V4 - P_g	-1,707.55	plf
Zero shear location	x_c	solves 0 = V2 - q_a*x_c - 0.5*Slope*x_c^2	1.29	ft
Moment end seg 1	M1	q_min*x1*(1/2*x1) + 0.5*(q_a - q_min)*x1*(1/3*x1)	1,037.31	lb*ft
Moment end seg 3	M3	q_b*x2*(1/2*x2) + 0.5*(q_max - q_b)*x2*(2/3*x2)	999.14	lb*ft
Moment at zero shear region	M2	0.5*[(M1 - 0.5*V2*x_c) + (M3 - 0.5*V3*(L - x1 - x2 - x_c))]	136.66	lb*ft
Peak shear magnitude	**V_max**	max(|V1|, |V2|, |V3|, |V4|)	**1,748.61**	lb
Peak moment magnitude	**M_max**	max(|M1|, |M2|, |M3|)	**1,037.31**	lb*ft
Factored Shear (1.5)	**V_u**	1.5 * V_max (AASHTO Table 3.4.1-2)	**2,622.91**	lb
Factored Moment (1.5)	**M_u**	1.5 * M_max (AASHTO Table 3.4.1-2)	**1,555.96**	lb*ft

Excel calculations to determine Moment Capacity, Min. Tensile Reinforcement, Shear Capacity, and Temperature & Shrinkage check are presented below:

Input - Description	Value	Units	Notes
Concrete strength f_c (psi)	4,000.00	psi	
Steel yield fy (ksi)	60.00	ksi	
Thickness tp (in)	5.50	in	
Width per strip b (in)	12.00	in	
Demand Mu (lb-ft)	1,555.00	lb-ft	
Demand Vu (lb)	2,627.00	lb	
Flexure Coefficient, ϕf	0.90	-	For Flexure with normal weight concrete
Shear Coefficient, ϕs	0.75	-	For shear

Moment Capacity of Setion – Checking #4 bars @ 14" O.C. (vertical rebar option):

Parameter	*Symbol*	*Value*	*Formula/Notes*
#4 bar area (in^2)	A_bar	0.196	default #4
Spacing s (in)	s	14	
Effective depth (in)	d	2.75	tp/2
As per foot (in^2/ft)	As@14	0.168	A_bar*(12/@s)
a (in)	a@14	0.247	As*fy/(0.85*fc*b)
Mn (lb-ft)	Mn@14	2,206.24	As*fy*(d-a/2)/12
ϕfMn (lb-ft)	ϕf*Mn@14	1,985.61	ϕf*Mn
Flexure OK?		OK	compare to Mu
Max. spacing 3*tp (in)		16.5	Controlled by Temp. & Shrinkage per AASHTO 5.10.6-2
Spacing check		OK	s < 3 tp
Check if steel yields:	Cna	0.290657	a@14/0.85
	εs	0.0254	εs = 0.003 (d/Cna-1)
Yields OK?		OK	εs > 0.002



Minimum Tensile Reinforcement (cracking) – Checking #4 bars @ 14" O.C. : AASHTO sect. 5.6.3.3

Parameter	*Symbol*	*Value*	*Formula/Notes*
Modulus of rupture fr (ksi)	fr	0.48	0.24*√(fc) with fc in psi
Compresive stress prestress forces (ksi)	fcpe	0	Does not apply
Dead Load moment (kpi-in)	Mdnc	0	in the same way, Snc=0
Concrete density mod. Factor	λ	1.0	For normal weight concrete
Section modulus S (in^3)	Sc	60.5	Sc=b*tp^2/6
Flexual Cracking Varibility Factor	γ1	1.2	For precast segmental structures
Prestress Varibility Factor	γ2	0.0	Does not apply
Min. Yield Strength Factor	γ3	0.67	For Grade 60 reinforcement
Cracking Moment, Mcr (lb-ft)	Mcr_lbft	1,945.68	Mcr= γ3 [(γ1*fr+γ2*fcpe)*Sc-Mdnc(Sc/Snc-1)](kip-in → lb-ft)
Check: ϕfMn ≥ Mcr?		OK	

Shear Capacity of Section – Checking #4 bars @ 14" O.C. :

Parameter	*Symbol*	*Value*	*Formula/Notes*
Vc (lb)	Vc14	4,174.21	2*λ*√fc * b * d
ϕsVc (lb)	phiVc14	3,130.65	ϕs*Vc
Shear OK?		OK	compare to Vu

Temperature & Shrinkage (**horizontal**) – Checking #4 bars @ 16" O.C. : AASHTO 5.10.6

Parameter	*Symbol*	*Value*	*Formula/Notes*
(a) Ash_min (in^2/ft)	(a)Ashmin_T&S	0.0409	Ashmin_T&S= 1.30*b*tp/[2(b+tp)fy]; (fy in ksi); AASHTO 5.10.6-1
(b) Ash_min (in^2/ft)	(b)Ashmin_T&S	0.1188	Ashmin_T&S= 0.0018*b*tp; per ACI 318-14 & AASHTO 5.10.6-1
Horizontal bar area (in^2)	A_hbar	0.196	default #4
Horizontal spacing (in)	s_h	16	
As_@16 (in^2/ft)	As@16	0.147	A_hbar*(12/s_h)
T&S OK?		OK	As@16 > (b)Ashmin_T&S
Max. spacing 3*tp (in)		16.5	Max. Spacing = 3*tp; Per AASHTO 5.10.6-2
Spacing check		OK	s_h < 3 tp

Alt. Case – 6"x6" W7.5 x W8.5 (WWR), W7.5 vertical:			
Description	*Symbol*	*Value*	*Formula/Notes*
W-number vertical	Wv	7.5	default vertical
W-number horizontal	Wh	8.5	
Wire spacing (in)	s_w	6	
As_vert (in^2/ft)	As_v	0.15	(Wv/100)*(12/s_w)
As_horz (in^2/ft)	As_h	0.17	(Wh/100)*(12/s_w)
Effective depth (in)	d	2.75	tp/2
a_vert (in)	a_v	0.221	As*fy/(0.85*fc*b)
Mn_vert (lb-ft)	Mn_v	1,979.78	As*fy*(d-a/2)/12
ϕf*Mn_vert (lb-ft)	ϕfMn_v	1,781.80	
Flexure OK? (vert as main)		OK	compare to Mu
Vc (lb)	Vc	4,174.21	2√fc * b * d
ϕs*Vc (lb)	ϕsVc	3,130.65	
Shear OK?		OK	compare to Vu
Alt. Case – 6"x6" W7.5 x W8.5 (WWR) - Crack Control (Frosch-based, AASHTO service):			
Parameter	*Symbol*	*Value*	*Formula/Notes*
Allowable crack width (in)	wc	0.017	editable; 0.013 in typical for Class 2
Steel modulus (ksi)	Es	29,000.00	typical
Cover to tension face (d, in)	d	2.75	distance from steel centroid to tension face; default tp/2
beta factor, β	β	1.22	β = 1 + 0.08*d
fs (ksi)	fs_use	36	AASHTO simplification fs≈0.6*fy
s_max (in)	s_max	11.22	s_max = wc*Es/(β*fs)
Crack control @ 6"?		OK	

Shear and Moment for Special panel (AG14) – Check with soil pressure of 2,000 lb/ft:

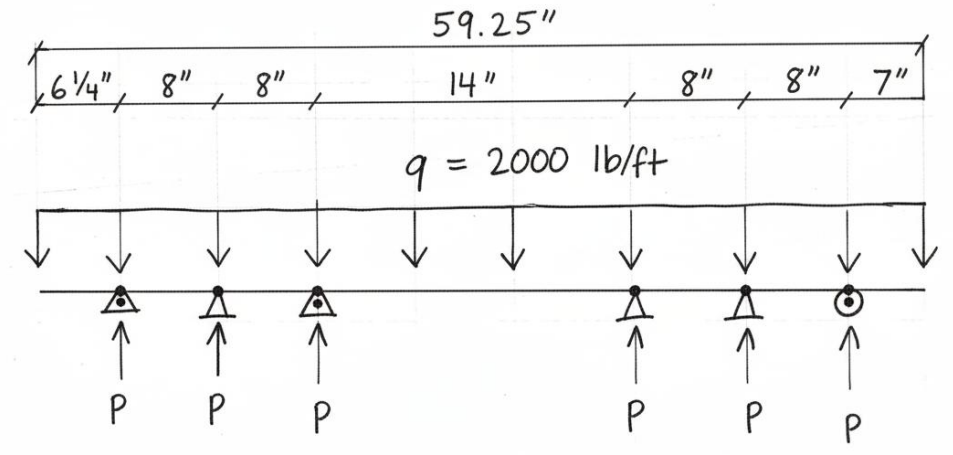


Figure 5 - Panel G14 Loading Diagram

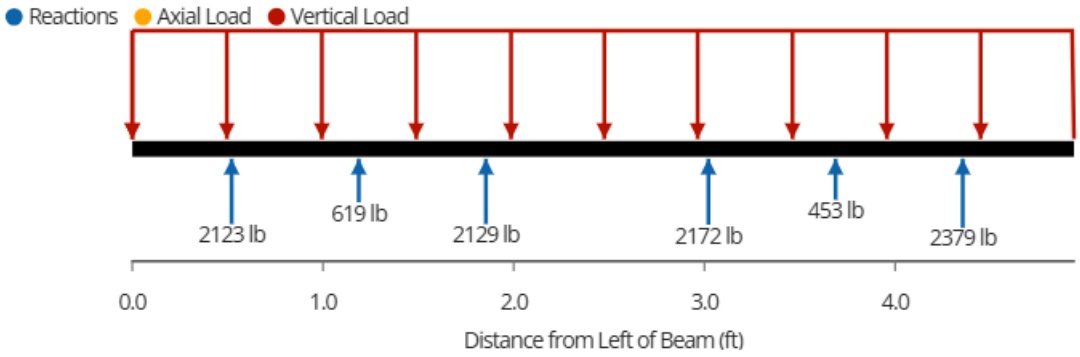


Figure 6 - Panel G14 Reactions

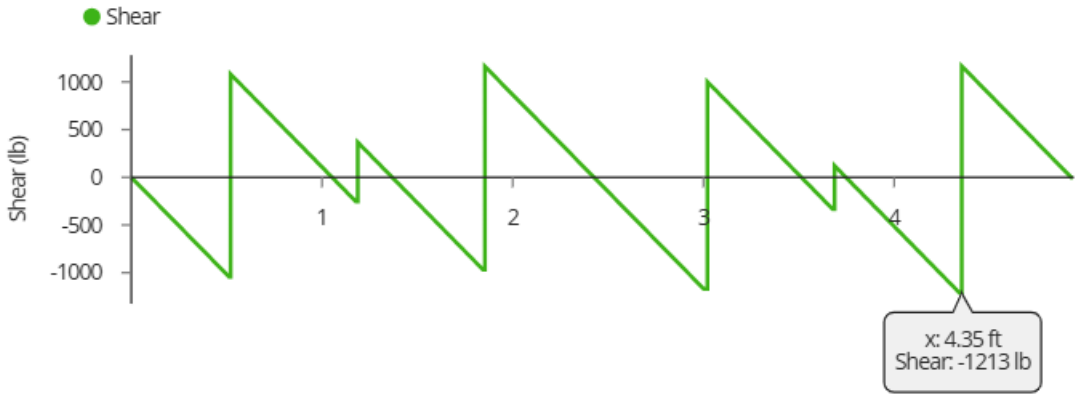


Figure 7 - Panel G14 Shear Demand

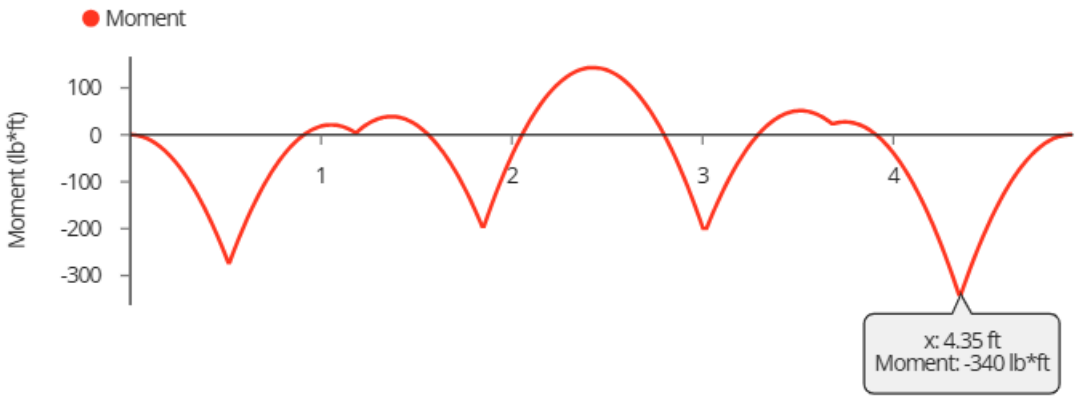


Figure 8 - Panel G14 Moment Demand

Summary:

Max. Factored Shear demand = 1,213 * 1.5 = 1,819.5 lb < ϕShear capacity = 3,130 lb → OK

Max. Factored Moment demand = 340 * 1.5 = 510 lb.ft < ϕMoment capacity = 1,985 lb.ft → OK

By inspection, the moment and shear demands for Special panel AG14 configuration are less than standard A panel configuration. As a result, no further calculations are necessary.

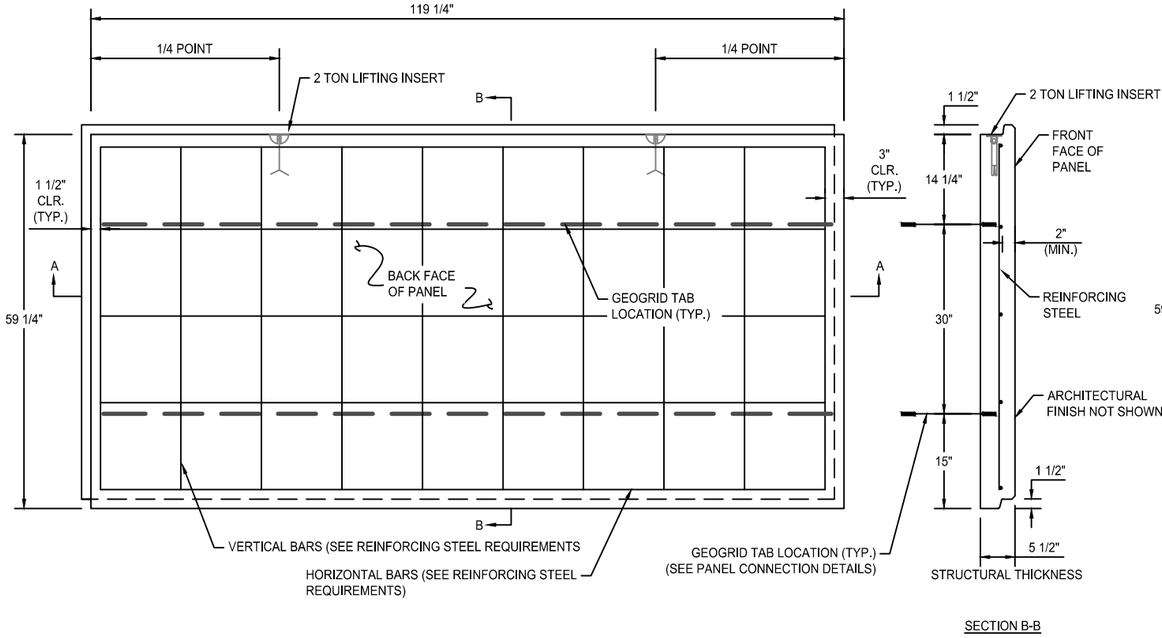
TENSAR Panel design conclusions

The structural reinforcing steel design to follow these recommendations:

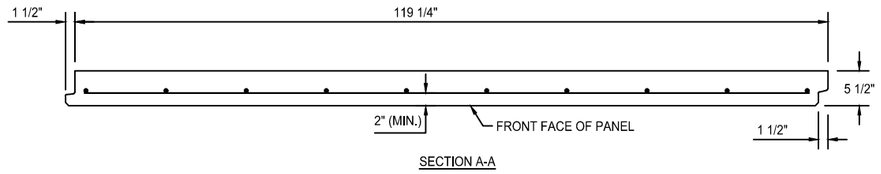
- Vertical reinforcement: #4 bar @ 14" o.c.
- Horizontal reinforcement: #4 bar @ 16" o.c.
- Use same bar configuration for both panel types due to spacing requirements
- Max soil loading on special panel (AG14) = 2,000 psf
- Wire mesh can be used to substitute rebar
- Chart for steel options for TENSAR panels:

Panel Type	Vertical Rebar	Horizontal Rebar	Wire Mesh Alt.
Standard (A)	#4 Bar @ 14” o.c.	#4 Bar @ 16” o.c.	6” x 6” – W7.5 x W8.5
Special (AG14)	#4 Bar @ 14” o.c.	#4 Bar @ 16” o.c.	6” x 6” – W7.5 x W8.5

STANDARD PRECAST PANEL SHOP DRAWINGS



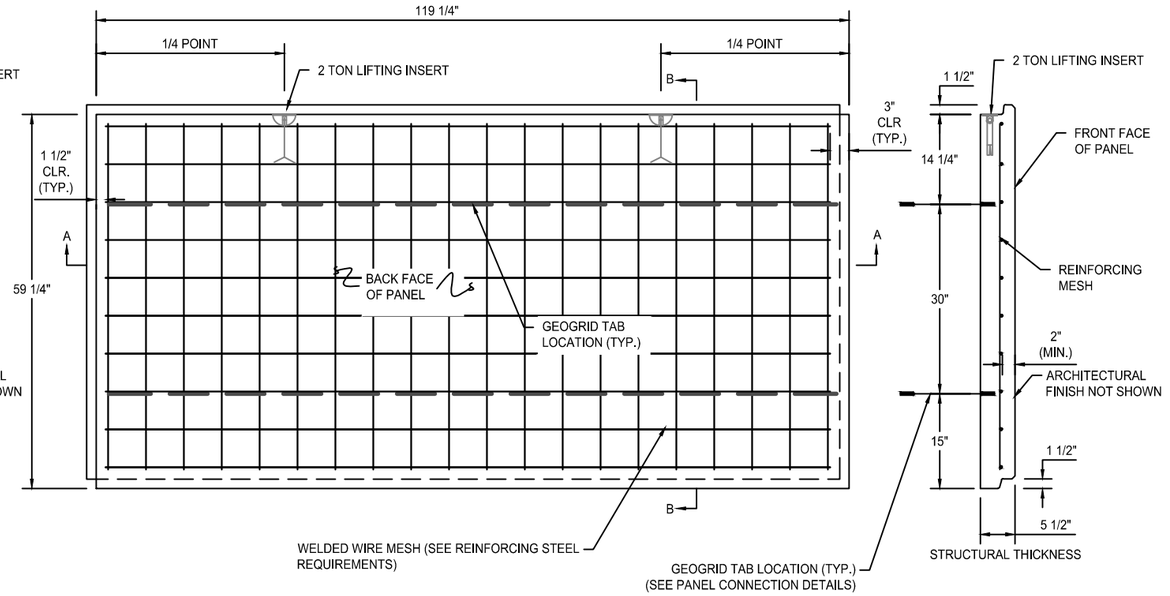
SECTION B-B



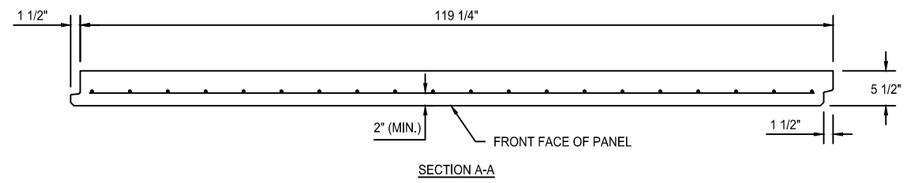
SECTION A-A

TYPICAL PANEL DETAILS - STANDARD A PANEL SHOWING REBAR REINFORCING

NOT TO SCALE



SECTION B-B



SECTION A-A

TYPICAL PANEL DETAILS - STANDARD A PANEL SHOWING MESH REINFORCING

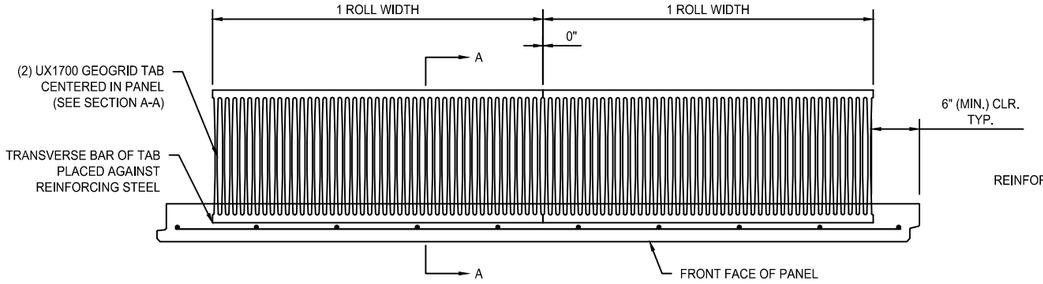
NOT TO SCALE

NOTES:

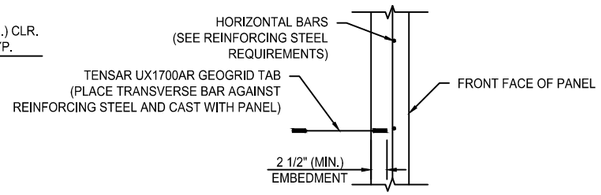
1. PANEL CONCRETE REQUIREMENTS:
   - 1.1. CONCRETE SHALL CONFORM TO THE REQUIREMENTS LISTED IN TXDOT ITEM 423, "RETAINING WALLS".
   - 1.2. CONCRETE SHALL HAVE A 28-DAY MINIMUM COMPRESSIVE STRENGTH OF 4,000 PSI.
2. PANEL REINFORCING STEEL REQUIREMENTS:
   - 2.1. ALL REINFORCING STEEL MUST MEET ASTM A-615 GRADE 60 REQUIREMENTS.
   - 2.2. ALL REINFORCING STEEL SHALL HAVE A MINIMUM CLEAR COVER OF 1 1/2" FROM ALL PANEL EDGES UNLESS OTHERWISE NOTED.
   - 2.3. REBAR SIZE AND SPACING REQUIREMENTS:
     - 2.3.1. HORIZONTAL: #4 BARS AT 16" O.C. (MAX.)
     - 2.3.2. VERTICAL: #4 BARS AT 14" O.C. (MAX.)
   - 2.4. WIRE MESH SIZE AND SPACING REQUIREMENTS:
     - 2.4.1. WELDED WIRE MESH: ASTM 1064 (A497) 6"x6" - W7.5xW8.5

NOTES (CONT.):

3. PANEL TOLERANCES:
   - 3.1. OVERALL DIMENSIONS: 1/2-INCH
   - 3.2. GEOGRID LOCATIONS: 1-INCH
   - 3.3. LIFTING INSERT LOCATIONS: 1/8-INCH
   - 3.4. ELEMENT SQUARENESS: 1/2-INCH (BETWEEN DIAGONALS)
   - 3.5. SURFACE FINISH (TEXTURED): 5/16-INCH IN 5-FEET
4. PANEL FINISH:
   - 4.1. REFER TO TENSAR PRECAST PANEL ARCHITECTURAL SUBMITTAL FOR MSE PANEL FINISH.
   - 4.2. ALL CORNER PANELS AND SLIP JOINTS SHALL HAVE A SMOOTH FINISH.



BOTTOM FACE VIEW

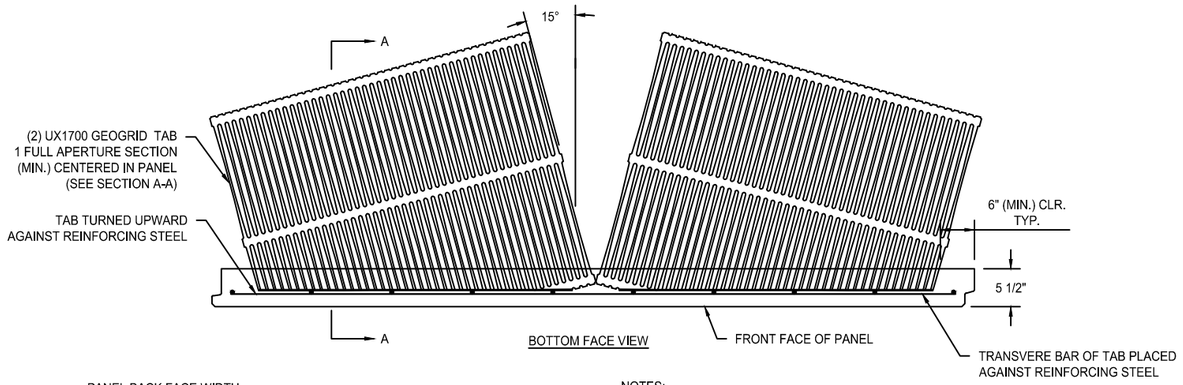


SECTION A-A

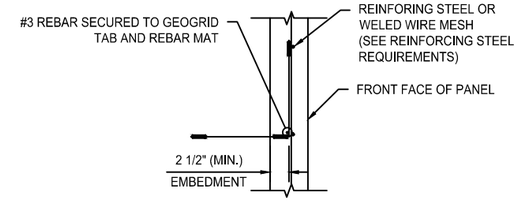
NOTE: TYPICAL ALL PANELS EXCEPT WHERE SHOWN

PANEL CONNECTION DETAIL (87% MAX. COVERAGE)

Scale: N.T.S.



BOTTOM FACE VIEW

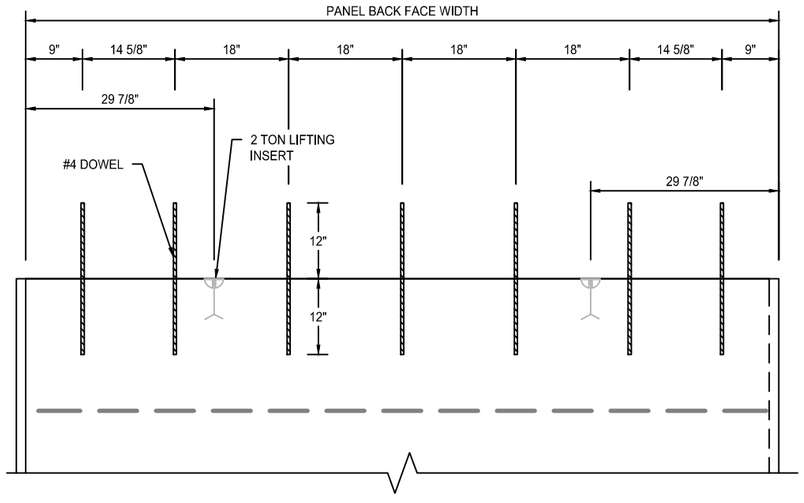


SECTION A-A

NOTES:
1. GEOGRID TAB EXTENDING ALONG REINFORCING STEEL AS NECESSARY TO PROVIDE MINIMUM EMBEDMENT OF FULL WIDTH OF GEOGRID TAB IN CONCRETE.

PANEL CONNECTION DETAIL AT 15° GEOGRID SKEW

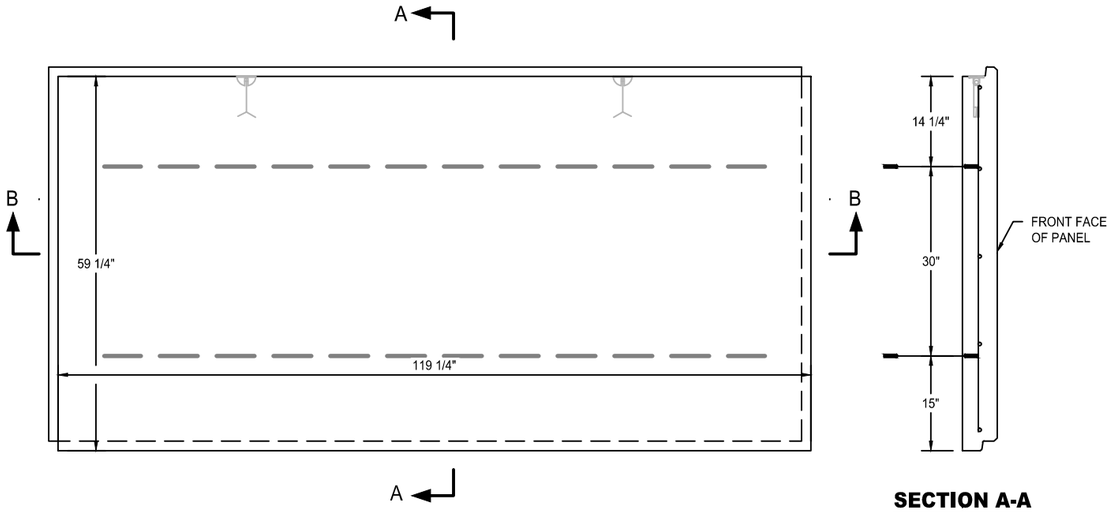
Scale: N.T.S.



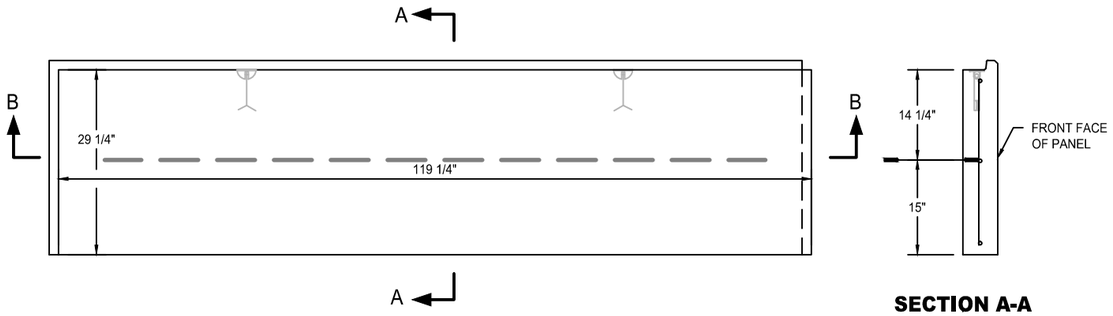
NOTE: QUANTITY AND DIMENSIONS TYPICAL UNLESS OTHERWISE NOTED ON INDIVIDUAL PANEL.

TYPICAL DOWEL AND LIFTING INSERT DETAIL

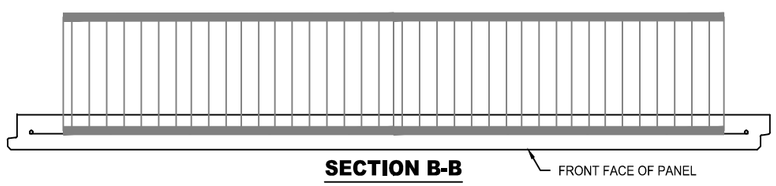
NOT TO SCALE



SECTION A-A



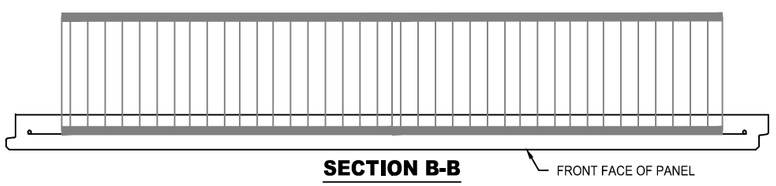
SECTION A-A



SECTION B-B

REINFORCING STEEL REQUIREMENTS		
	QTY	TYPE AND SPACING
HORIZONTAL:	5	#4 BARS (60 KSI) 16" O.C. (MAX.)
VERTICAL:	10	#4 BARS (60 KSI) 14" O.C. (MAX.)
OR 6" x 6" - W7.5xW8.5 WELDED WIRE MESH PER ASTM A-82, FABRICATION PER ASTM A-185		

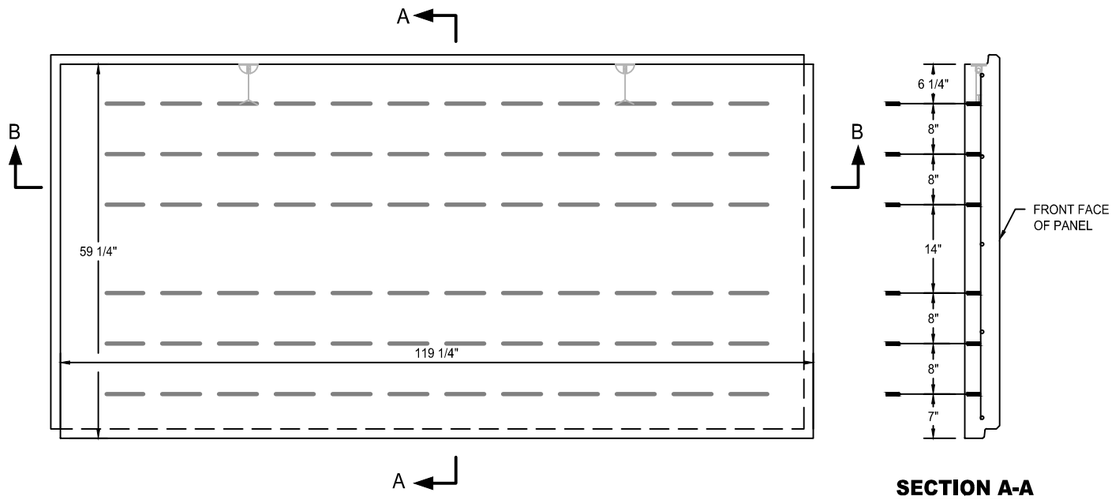
PANEL A



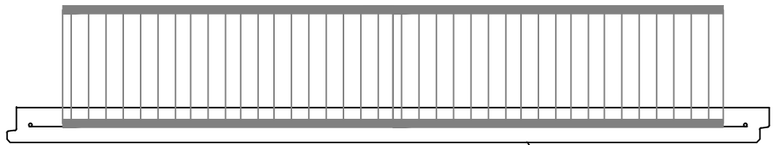
SECTION B-B

REINFORCING STEEL REQUIREMENTS		
	QTY	TYPE AND SPACING
HORIZONTAL:	3	#4 BARS (60 KSI) 16" O.C. (MAX.)
VERTICAL:	10	#4 BARS (60 KSI) 14" O.C. (MAX.)
OR 6" x 6" - W7.5xW8.5 WELDED WIRE MESH PER ASTM A-82, FABRICATION PER ASTM A-185		

PANEL B1



SECTION A-A



SECTION B-B

REINFORCING STEEL REQUIREMENTS		
	QTY	TYPE AND SPACING
HORIZONTAL:	5	#4 BARS (60 KSI) 16" O.C. (MAX.)
VERTICAL:	10	#4 BARS (60 KSI) 14" O.C. (MAX.)
OR 6" x 6" - W7.5xW8.5 WELDED WIRE MESH PER ASTM A-82, FABRICATION PER ASTM A-185		

PANEL AG14

NOTES:
1. REFER TO PANEL REINFORCEMENT DETAILS FOR PANEL THICKNESS, TYPICAL DIMENSIONS AND ADDITIONAL REQUIREMENTS.
2. 2 TON LIFTING INSERTS SHALL BE EQUALLY SPACED AT 1/4 POINT LOCATION FROM EDGE OF BACK FACE OF PANEL TO CENTER OF INSERT UNLESS OTHERWISE NOTED.

elevation view section view 2 TON LIFTING INSERTS (SEE NOTE 2)

TENSAR GEOGRID TAB LOCATION

Appendix 1.1.4.a

Ares 5x5 Standard Details

ARES® MSE Retaining Wall System

Back to main document

Tensar®
A Division of CMC

ARES®
RETAINING WALL SYSTEMS

MSE STANDARD DETAILS
PREPARED FOR

ARES (5'X5') RETAINING WALL SYSTEM

INDEX	
SHEET NUMBER	DESCRIPTION
1	Title Sheet
2	Standard Detail (1 of 12)
3	Standard Detail (2 of 12)
4	Standard Detail (3 of 12)
5	Standard Detail (4 of 12)
6	Standard Detail (5 of 12)
7	Standard Detail (6 of 12)
8	Standard Detail (7 of 12)
9	Standard Detail (8 of 12)
10	Standard Detail (9 of 12)
11	Standard Detail (10 of 12)
12	Standard Detail (11 of 12)
13	Standard Detail (12 of 12)

Plotted on February 25, 2026
C:\WP\IDEA\ARES\IDEA_CAD DETAILS\5 X 5 STANDARD DETAILS.DWG

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ARES®
RETAINING WALL SYSTEMS

PROJECT NAME AND LOCATION

----, ----

OWNER

OWNER PROJECT No.

CLIENT

IDEA

TIC PROJECT No. ----

DRAWN BY: V. JUAREZ
DESIGNED BY: ----
CHECKED BY : J. PERALTA

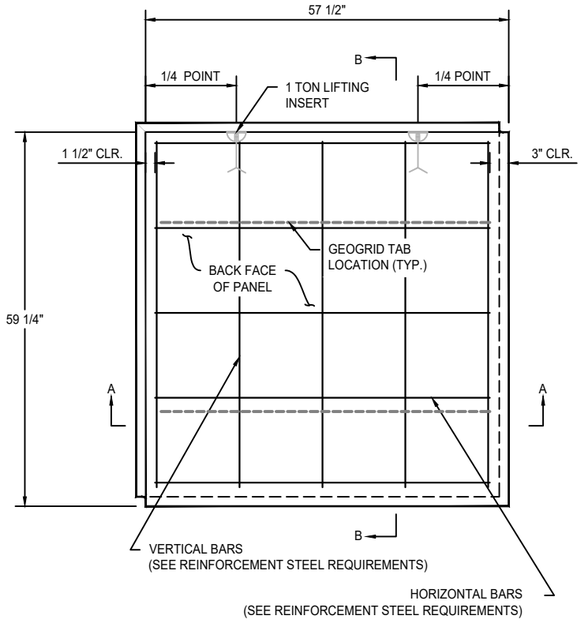
ENGINEER OF RECORD (MSE STRUCTURE ONLY):
---- P.E. NO.

NO.	DATE	DESCRIPTION	BY
0	06/25/2025	ISSUED FOR REVIEW	VJ

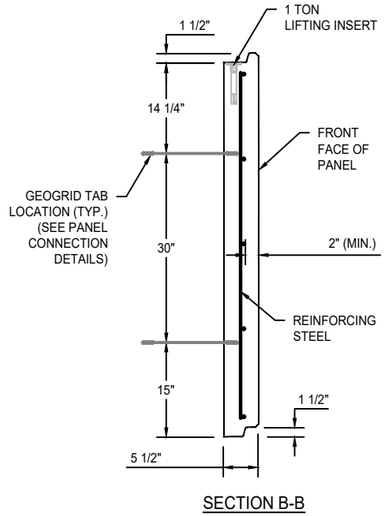
REVISION / ISSUE

SHEET TITLE

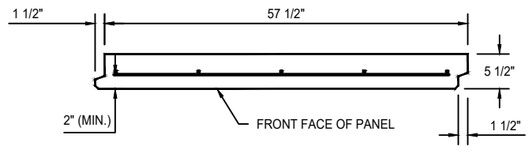
TITLE SHEET



BACK FACE VIEW



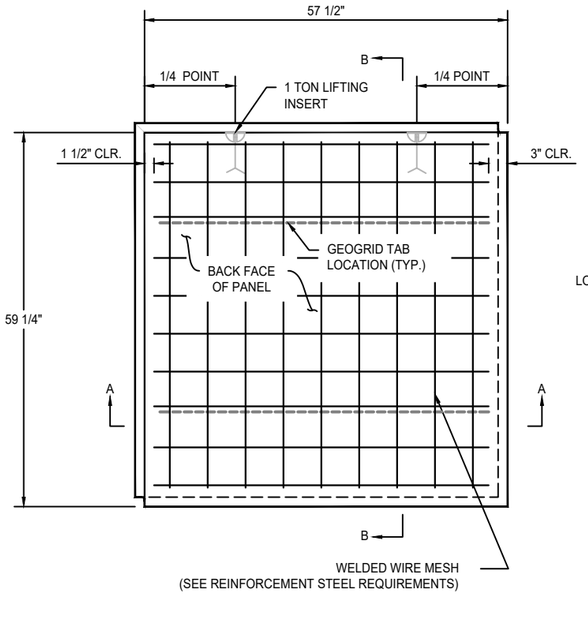
SECTION B-B



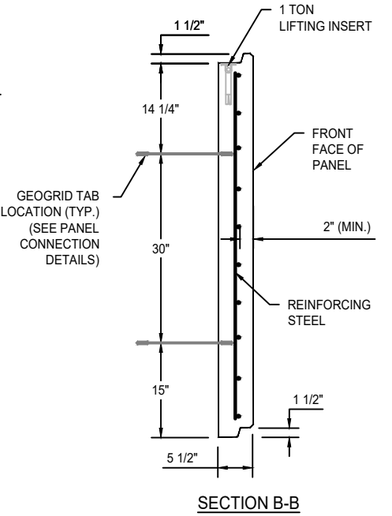
SECTION A-A

TYPICAL PANEL DETAILS - STANDARD A PANEL (REBAR REINFORCING)

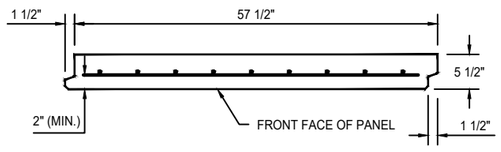
NOT TO SCALE



BACK FACE VIEW



SECTION B-B



SECTION A-A

TYPICAL PANEL DETAILS - STANDARD A PANEL (MESH REINFORCING)

NOT TO SCALE

NOTES:

1. CONCRETE SHALL HAVE A 28 DAY MINIMUM COMPRESSIVE STRENGTH OF 4,000 PSI.
2. REINFORCING STEEL REQUIREMENTS:

   REBAR SIZE AND SPACING :
   HORIZONTAL:#4 BARS @ 16" O.C. (MAX.)
   VERTICAL:#4 BARS @ 14" O.C. (MAX.)

   WIRE MESH SIZE AND SPACING :
   6"x6" - W8.5xW8.5 WELDED WIRE MESH PER ASTM A-82, FABRICATION PER ASTM A-185. (ASTM A1064)
3. REINFORCING STEEL SHALL HAVE A MINIMUM CLEAR COVER OF 1-1/2" FROM ALL PANEL EDGES UNLESS OTHERWISE NOTED.
4. PANEL TOLERANCES:
   - 4.1. OVERALL DIMENSIONS: 1/4-INCH
   - 4.2. GEOGRID LOCATIONS: 1-INCH
   - 4.3. LIFTING INSERT LOCATIONS: 1/8-INCH
   - 4.4. ELEMENT SQUARENESS: 1/2-INCH (BETWEEN DIAGONALS)
5. ALL PANELS SHOWN BACK FACE OF PANEL.
6. PANEL DIMENSIONS AND TOLERANCES SHALL BE IN ACCORDANCE WITH SUPPLEMENTED SPECIFICATIONS.

Tensar.
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ARES® RETAINING WALL SYSTEMS

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CLIENT

IDEA

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DRAWN BY: V. JUAREZ
DESIGNED BY: ----
CHECKED BY : J. PERALTA
ENGINEER OF RECORD (MSE STRUCTURE ONLY):
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NO.	DATE	DESCRIPTION	BY
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STANDARD DETAIL (1 OF 12)

Plotted on February 25, 2026
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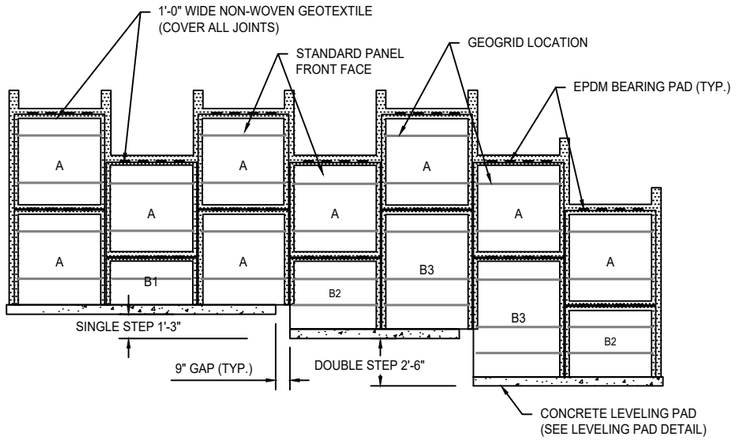
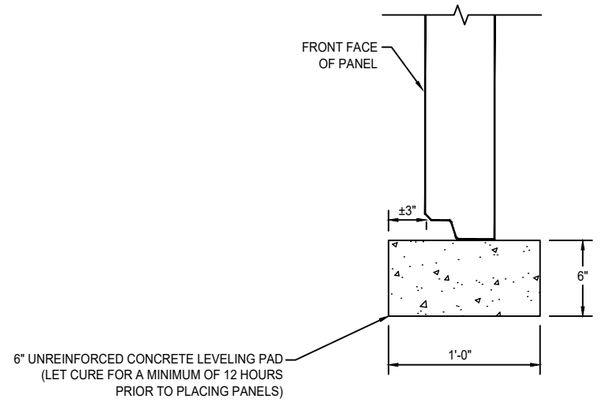


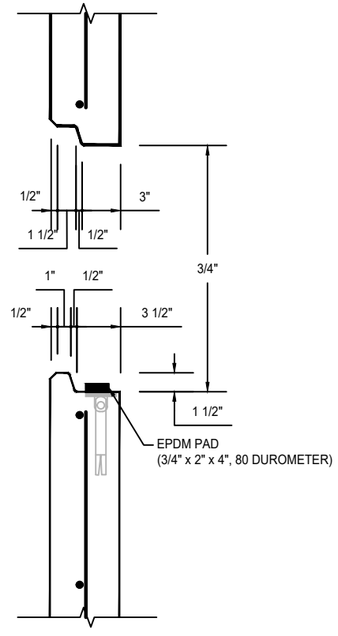
TYPICAL LEVELING PAD STEP AND GEOTEXTILE COVERAGE DETAIL

NOT TO SCALE



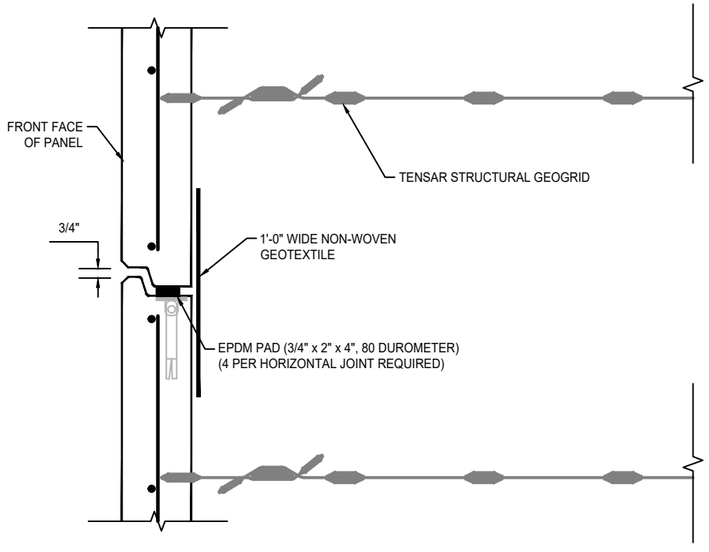
LEVELING PAD DETAIL

NOT TO SCALE



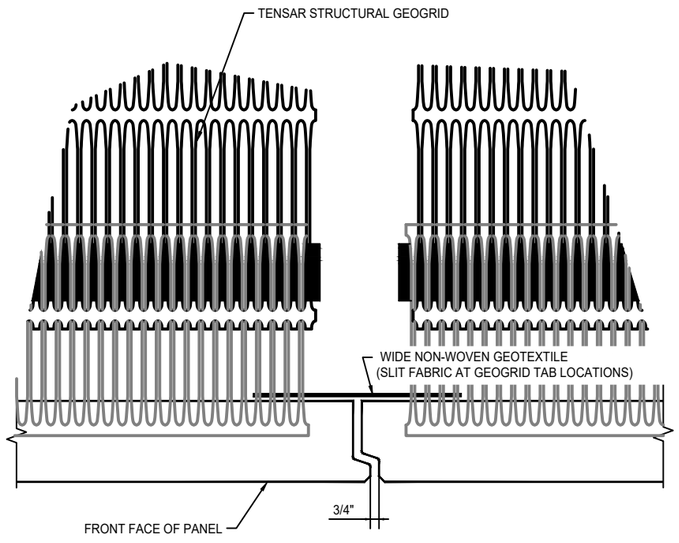
PANEL JOINT DETAIL

NOT TO SCALE



HORIZONTAL JOINT DETAIL

NOT TO SCALE



PLAN VIEW

VERTICAL JOINT DETAIL

NOT TO SCALE

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CHECKED BY : J. PERALTA

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SHEET TITLE

STANDARD DETAIL (2 OF 12)

Plotted on: February 25, 2026
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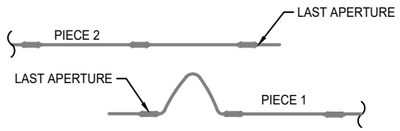
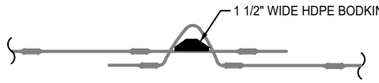
TO FORM A BODKIN CONNECTION:

1. BEND THE LAST APERTURE OF ONE PIECE OF GEOGRID IN HALF.



2. PASS THE BENT APERTURES THROUGH THE APERTURES OF THE SECOND PIECE OF GEOGRID AND INSERT THE BODKIN BAR INTO THE CAVITY BETWEEN THE GEOGRIDS.



3. PULL BOTH PIECES OF GEOGRID IN OPPOSITE DIRECTIONS TO COMPLETE CONNECTION.

NOTE:

THE SPLICED GEOGRID PIECE ON EITHER SIDE OF THE BODKIN CONNECTION SHALL BE AT LEAST 6 FEET LONG UNLESS THE GEOGRID TERMINATES IN A FIXED CONNECTION.

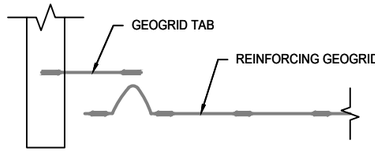
CROSS-SECTION

BODKIN CONNECTION

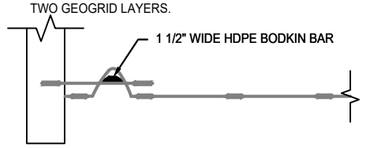
NOT TO SCALE

TO FORM A PANEL BODKIN CONNECTION FOR CONNECTING TO FACING PANEL:

1. BEND THE LAST APERTURE OF REINFORCING GEOGRID AS SHOWN.



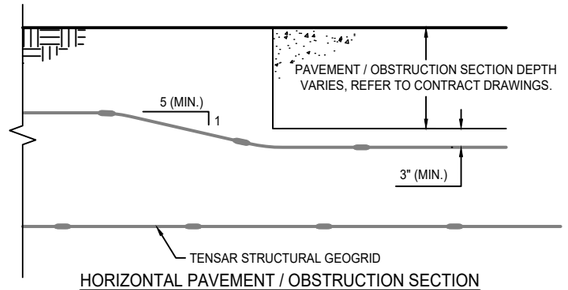
2. PASS THE RIBS OF THE BENT APERTURES THROUGH THE RIBS OF THE GEOGRID TAB AND INSERT THE BODKIN BAR INTO THE SPACE BETWEEN THE TWO GEOGRID LAYERS.



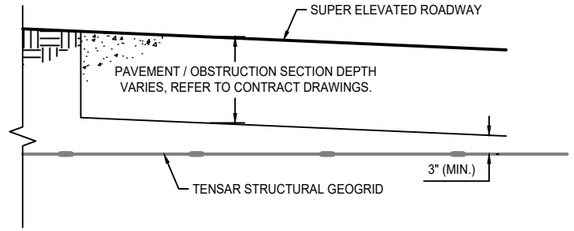
3. PULL REINFORCING GEOGRID TAUT TO TENSION CONNECTION.

PANEL BODKIN CONNECTION

NOT TO SCALE



HORIZONTAL PAVEMENT / OBSTRUCTION SECTION



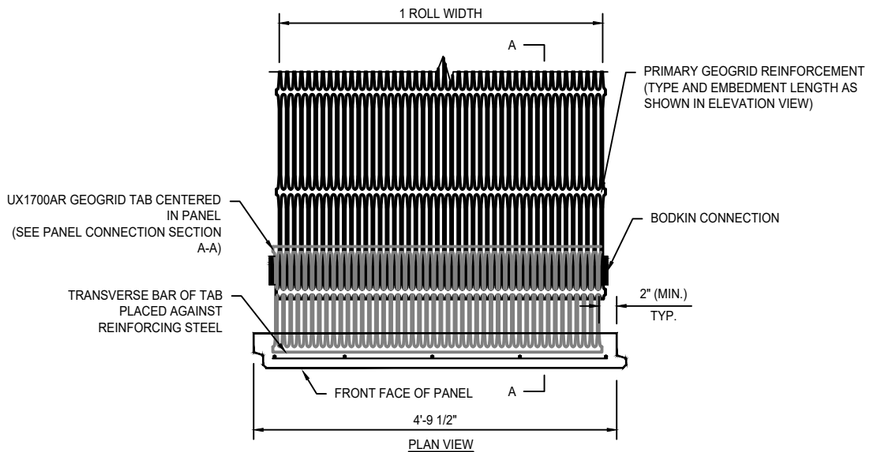
SUPERELEVATED PAVEMENT / OBSTRUCTION SECTION

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 SECTION

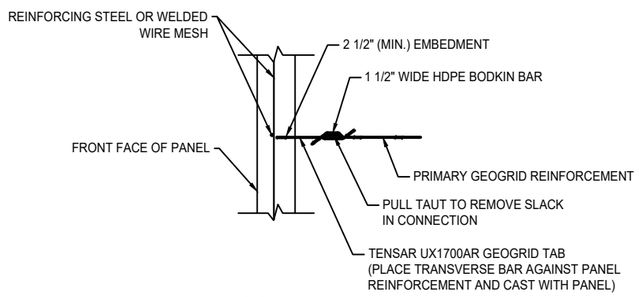
NOT TO SCALE



PLAN VIEW

CONNECTION DETAIL PLAN VIEW (90% COVERAGE)

NOT TO SCALE



SECTION A-A

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CHECKED BY : J. PERALTA

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SHEET TITLE

STANDARD DETAIL (3 OF 12)

Plotted on February 25, 2025
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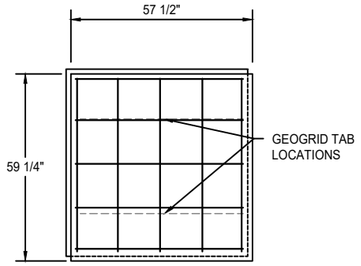
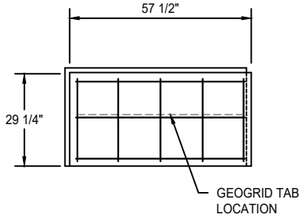
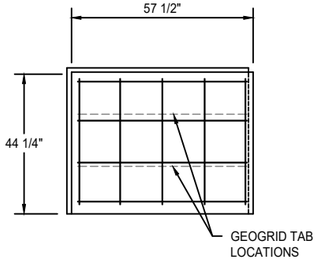


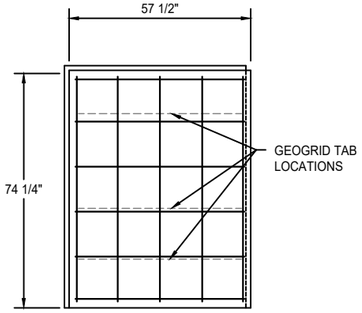
STANDARD A PANEL



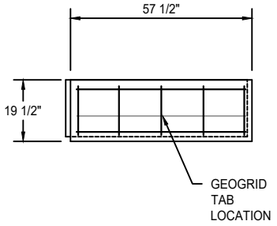
STANDARD B1 PANEL



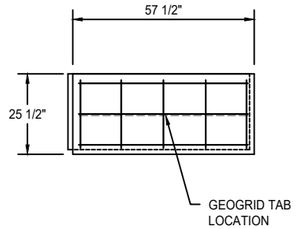
STANDARD B2 PANEL



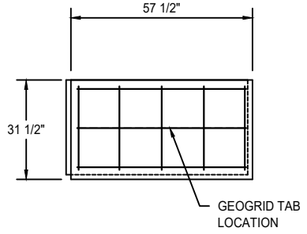
STANDARD B3 PANEL



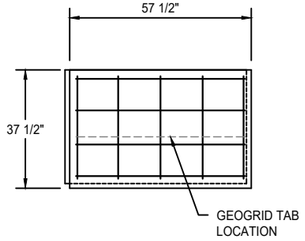
STANDARD T18 PANEL



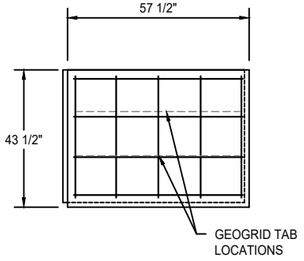
STANDARD T24 PANEL



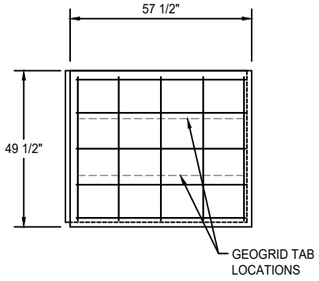
STANDARD T30 PANEL



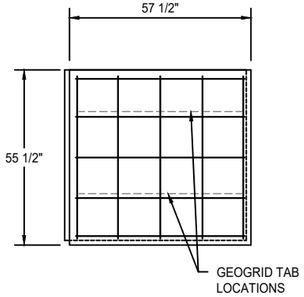
STANDARD T36 PANEL



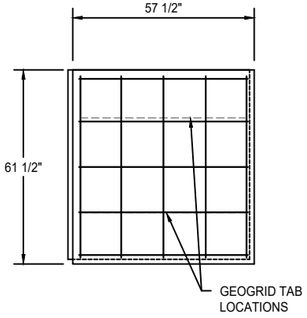
STANDARD T42 PANEL



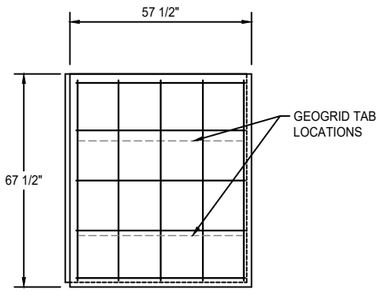
STANDARD T48 PANEL



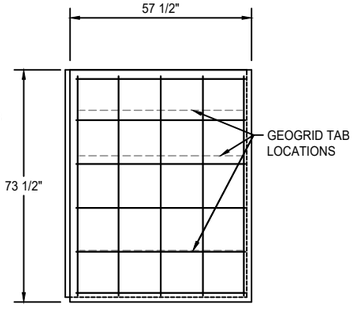
STANDARD T54 PANEL



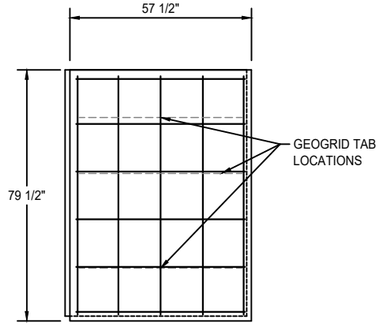
STANDARD T60 PANEL



STANDARD T66 PANEL



STANDARD T72 PANEL



STANDARD T78 PANEL

ALL PANELS ARE SHOWN BACK FACE VIEW

NOTES:

1. PANEL CONCRETE REQUIREMENTS:
   - 1.1. PRECAST CONCRETE PANEL SHALL CONFORM TO THE REQUIREMENTS OF SECTION 701, 703, 705 AND 709 OF ODOT STANDARD SPECIFICATIONS.
   - 1.2. CONCRETE SHALL HAVE A 28-DAY MINIMUM COMPRESSIVE STRENGTH OF 4,000 PSI.

2. PANEL REINFORCING STEEL REQUIREMENTS:
   - 2.1. ALL REINFORCING STEEL SHALL HAVE A MINIMUM CLEAR COVER OF 2" FROM ALL PANEL EDGES UNLESS OTHERWISE NOTED.
   - 2.2. REBAR SIZE AND SPACING REQUIREMENTS:
     - 2.2.1. HORIZONTAL: #4 BARS AT 16" O.C. (MAX.)
     - 2.2.2. VERTICAL: #4 BARS AT 14" O.C. (MAX.)
   - 2.3. WIRE MESH SIZE AND SPACING REQUIREMENTS
     - 24.1 WELDED WIRE MESH 6"x6" - W8.5xW8.5
   - 2.5. PANEL DIMENSIONS AND TOLERANCES SHALL BE IN ACCORDANCE WITH SUPPLEMENTED SPECIFICATIONS.

STANDARD 5' X 5' PANELS

NOT TO SCALE

Plotted on February 25, 2024
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CHECKED BY : J. PERALTA

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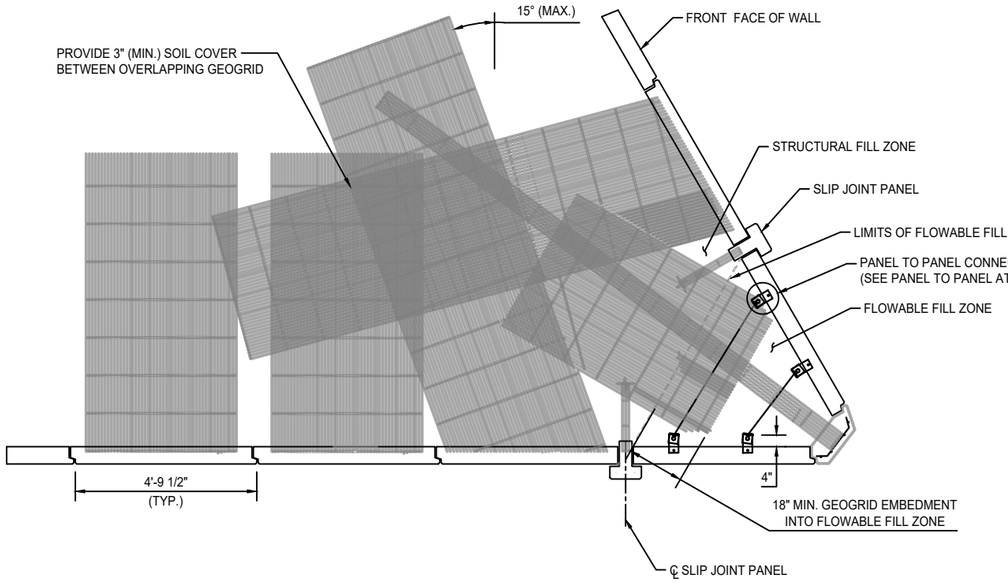
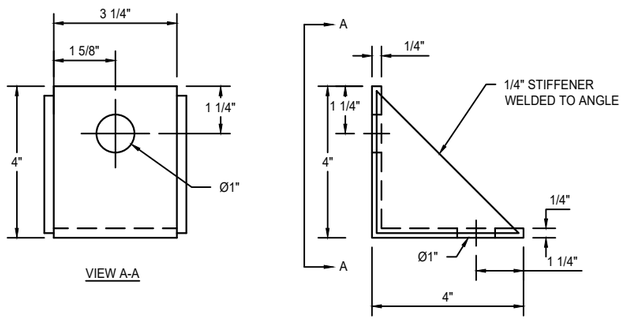
SHEET TITLE

STANDARD DETAIL (4 OF 12)



ACUTE CORNER LESS THAN 70°

NOT TO SCALE

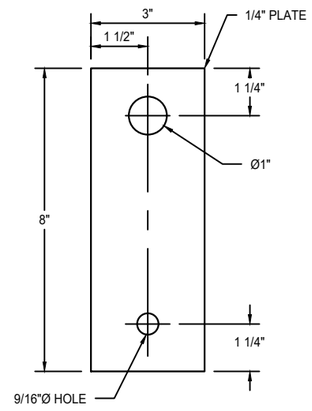


CONNECTION BOX

NOT TO SCALE

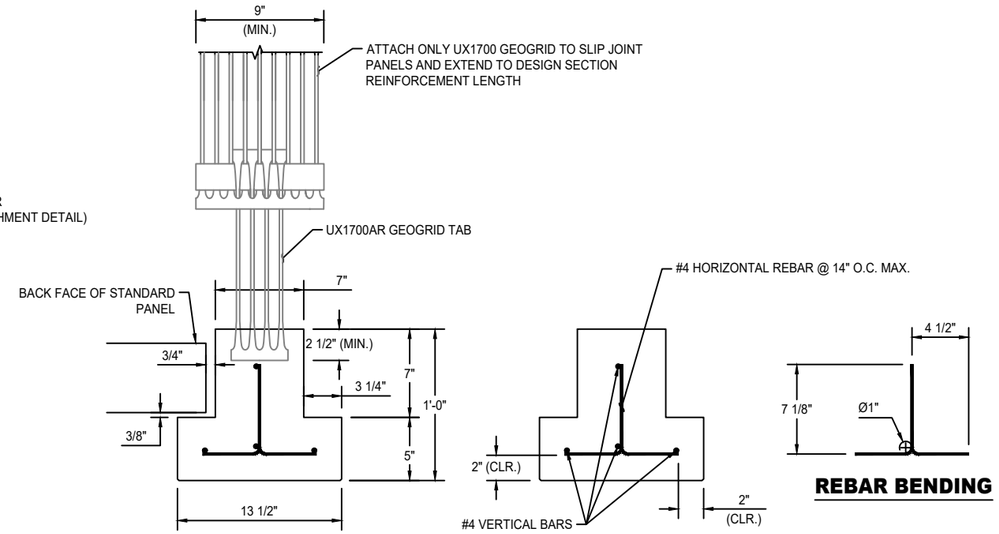
FABRICATION ATTACHMENT STEEL NOTES:

1. ALL STEEL ANGLES AND PLATES SHALL BE PER ASTM A36.
2. ALL FABRICATED STEEL PARTS SHALL BE HOT DIP GALVANIZED AFTER FABRICATION.
3. ALL DIMENSIONS ARE MINIMUM REQUIRED.



ATTACHMENT PLATE

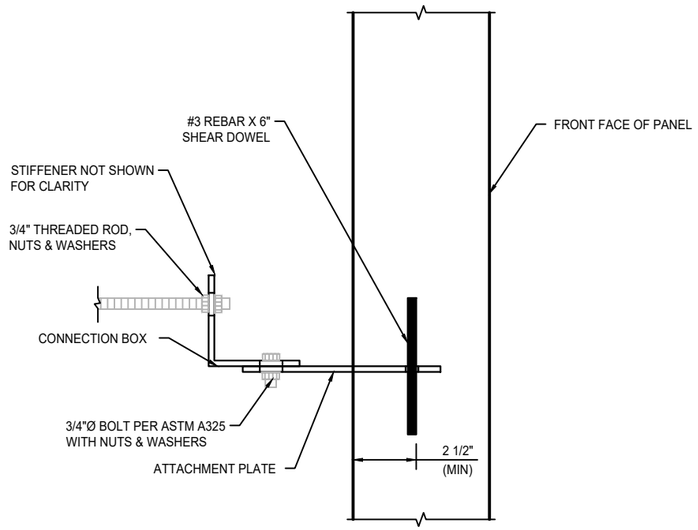
NOT TO SCALE



SLIP JOINT DETAIL

NOT TO SCALE

REBAR BENDING



PANEL TO PANEL ATTACHMENT

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ARES® RETAINING WALL SYSTEMS

PROJECT NAME AND LOCATION

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OWNER

OWNER PROJECT No.

CLIENT

IDEA

TIC PROJECT No. ----

DRAWN BY: V. JUAREZ

DESIGNED BY: ----

CHECKED BY : J. PERALTA

ENGINEER OF RECORD (MSE STRUCTURE ONLY):

---- P.E. NO.

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SHEET TITLE

STANDARD DETAIL (5 OF 12)

Plotted on February 25, 2025
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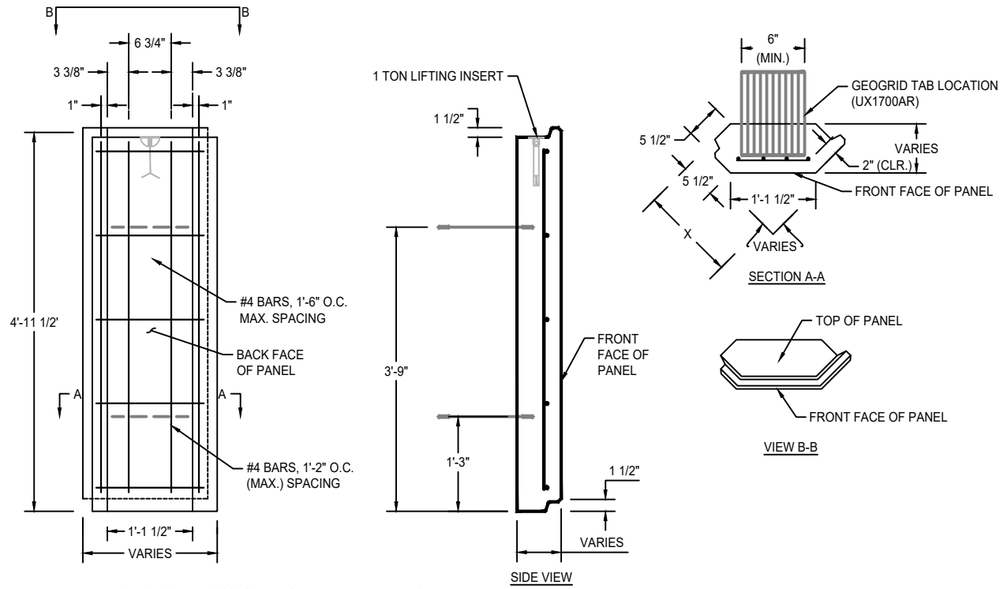
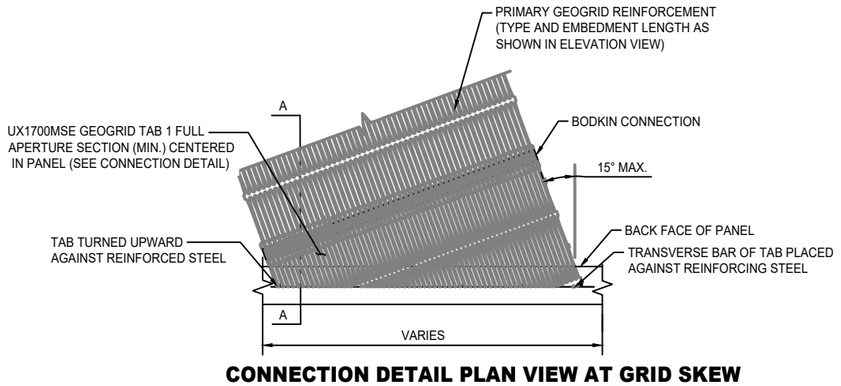


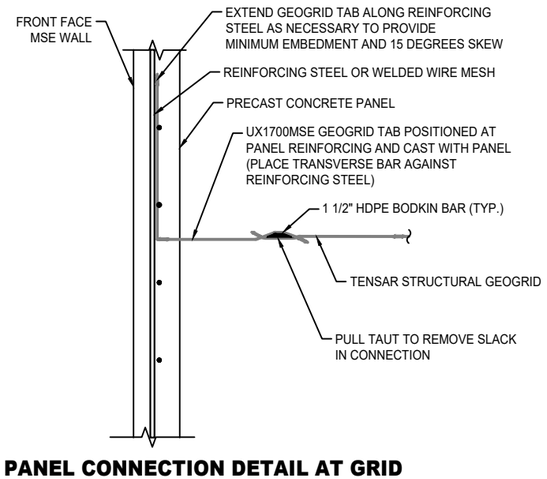
TYPICAL CORNER ELEMENT DETAIL

NOT TO SCALE



CONNECTION DETAIL PLAN VIEW AT GRID SKEW

NOT TO SCALE



PANEL CONNECTION DETAIL AT GRID SKEW (SECTION A-A)

NOT TO SCALE

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STANDARD DETAIL (6 OF 12)

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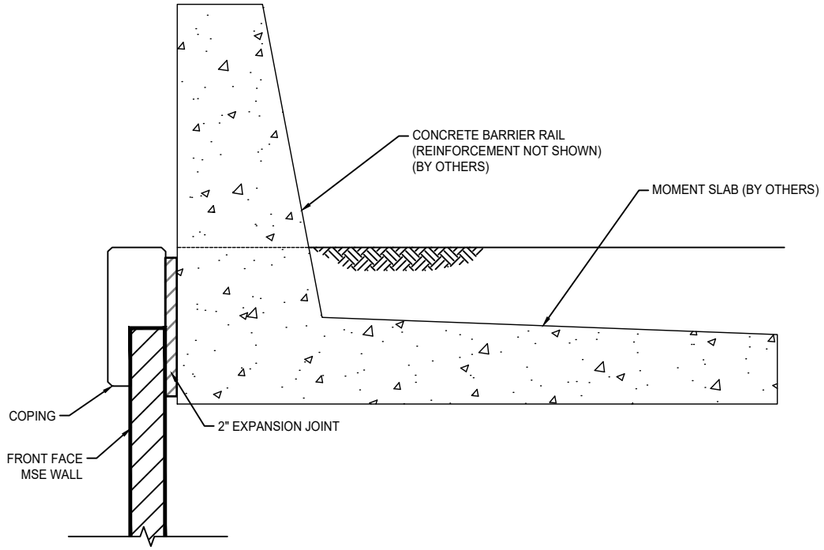
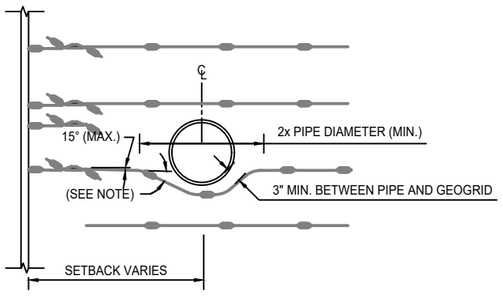


CONCRETE BARRIER RAIL WITH MOMENT SLAB ON MSE WALL

NOT TO SCALE

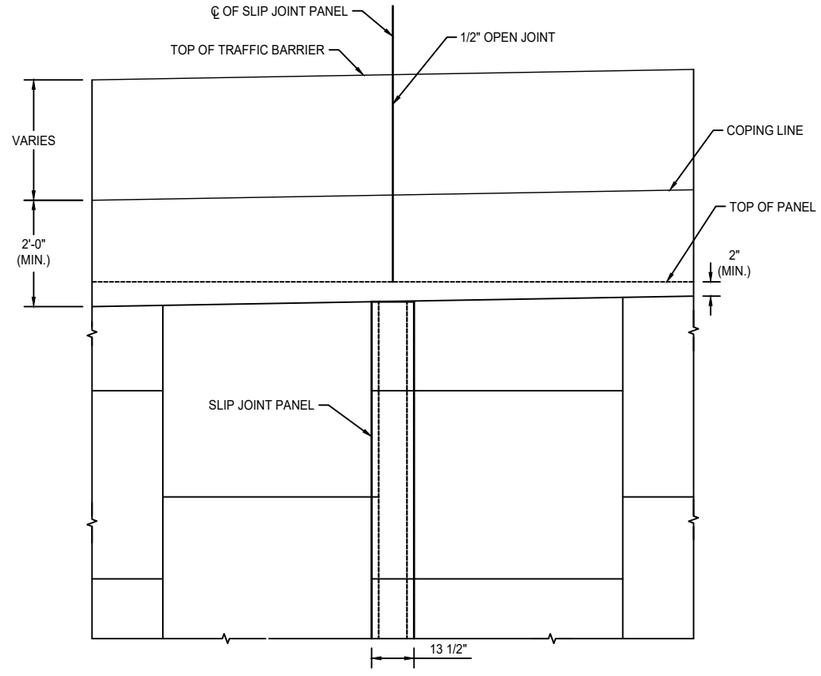


NOTE:

IF GEOGRID MUST BE MORE THAN 15° FROM HORIZONTAL, EXTEND GEOGRID 4'-0" LONGER THAN SHOWN ON ELEVATION VIEW.

TYPICAL OBSTRUCTION AVOIDANCE DETAIL

NOT TO SCALE



C.I.P. TRAFFIC BARRIER OVER SLIP JOINT PANEL

NOT TO SCALE

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STANDARD DETAIL (7 OF 12)

Plotted on February 25, 2025
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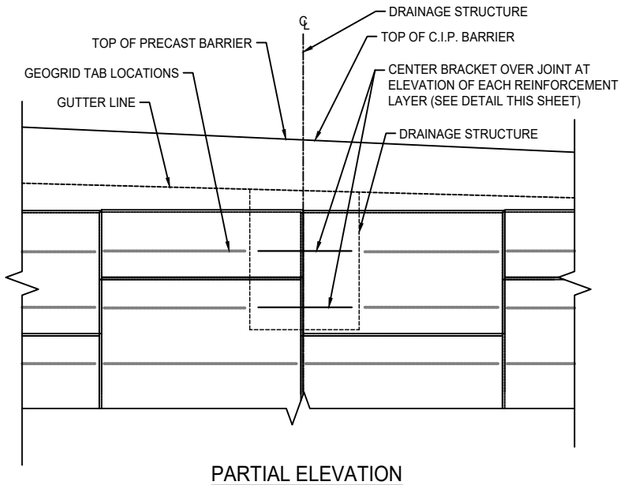
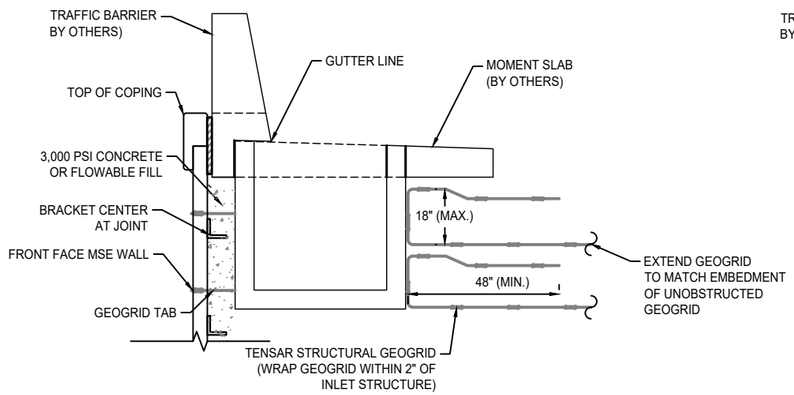
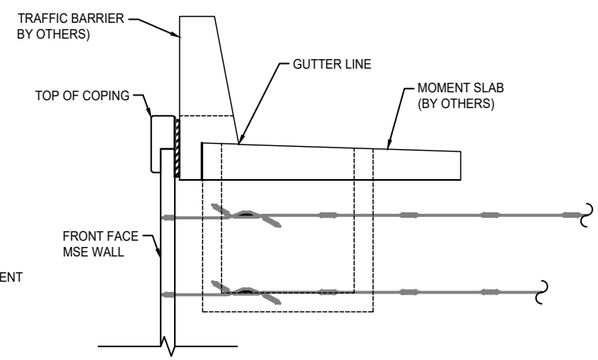


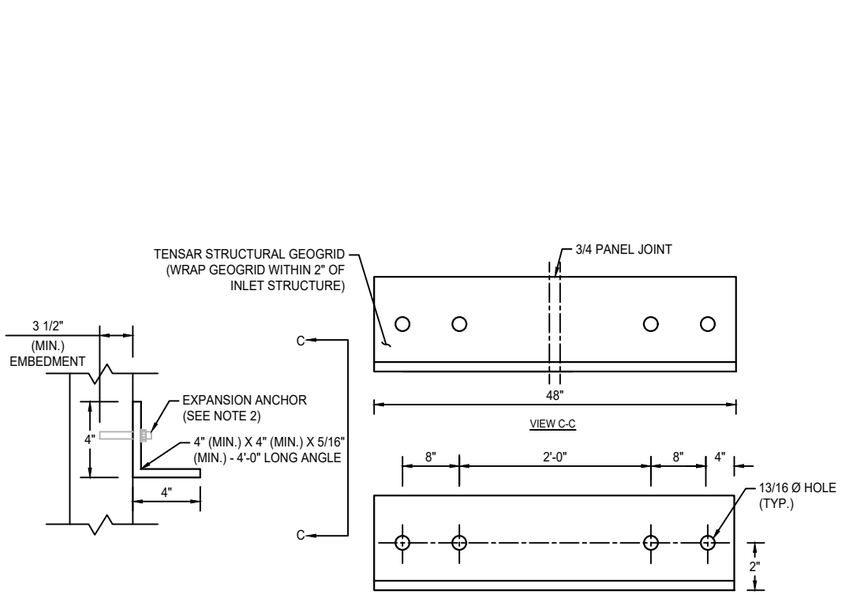
PARTIAL ELEVATION



SECTION A-A



SECTION B-B

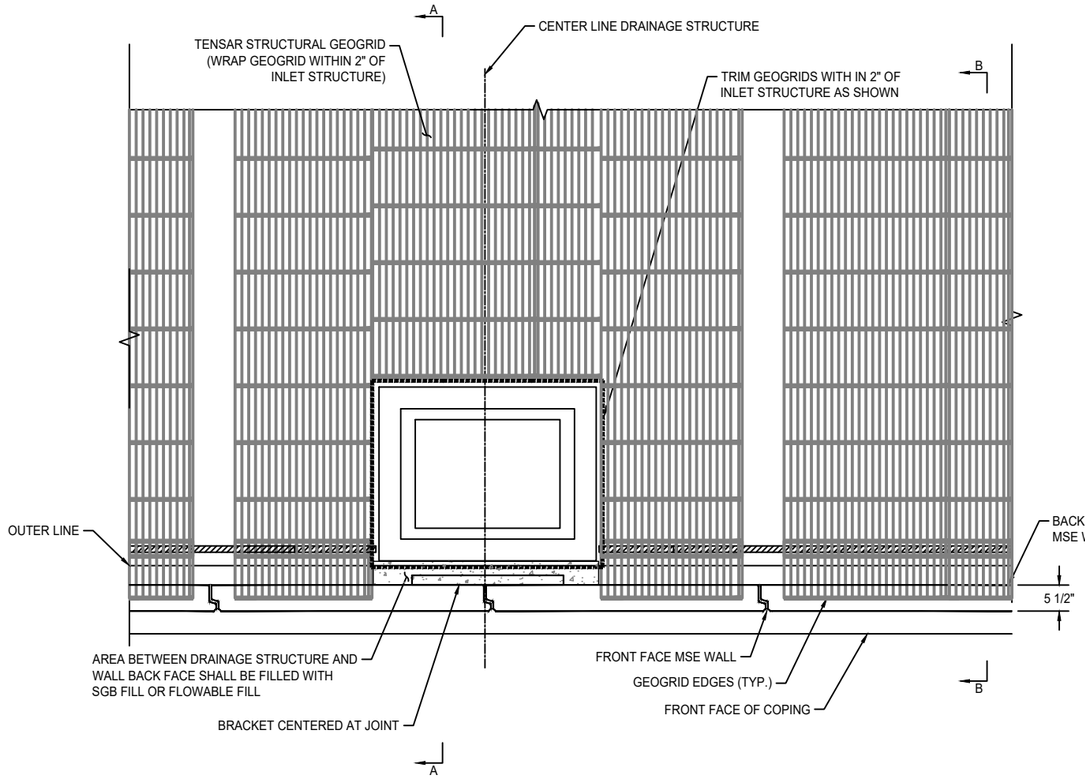


NOTES:

1. ALL FABRICATED STEEL PARTS SHALL BE HOT DIP GALVANIZED AFTER FABRICATION.
2. ANCHOR SHALL BE Ø 5/8" KIWIK BOLT 3 HOT-DIP GALVANIZED (HDG) EXPANSION ANCHOR OR APPROVED EQUAL.

BRACKET CENTERED OVER JOINT DETAIL

NOT TO SCALE



PARTIAL PLAN

MSE WALL AT DRAINAGE INLET

NOT TO SCALE

Plotted on February 25, 2026
DWG: V:\IDEA\TARES\TARES IDEA CAD DETAILS\5 X 5 STANDARD DETAILS.DWG

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CHECKED BY : J. PERALTA

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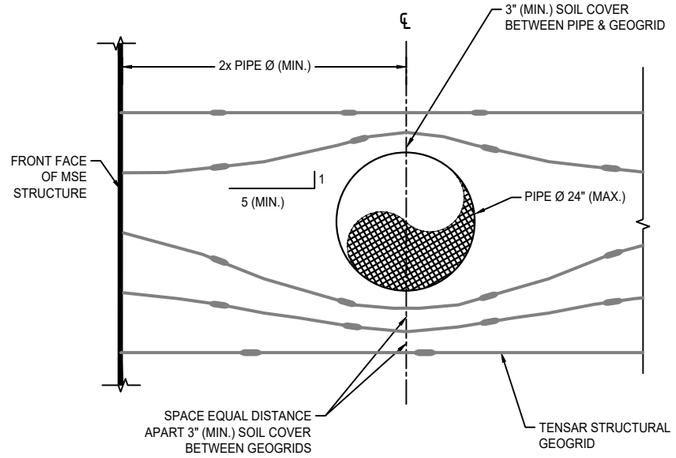
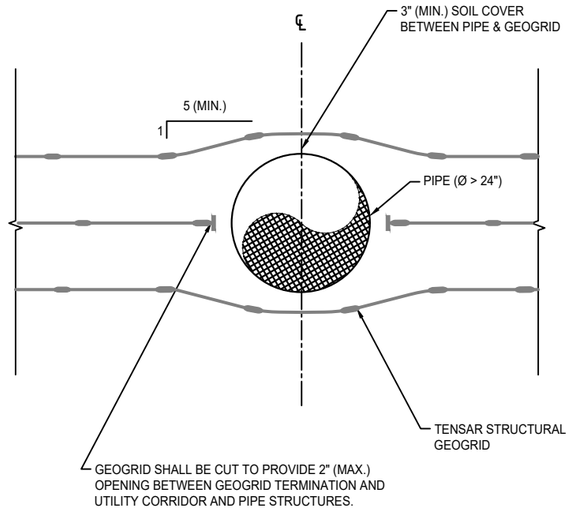
STANDARD DETAIL (8 OF 12)



CROSS-SECTION

GEOGRID PLACEMENT AT PIPE PARALLEL TO MSE STRUCTURE FACE

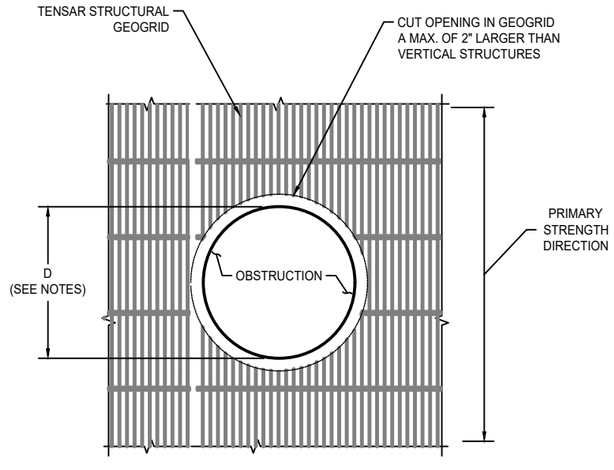
NOT TO SCALE



ELEVATION VIEW

GEOGRID PLACEMENT AT PIPE PERPENDICULAR TO MSE STRUCTURE FACE

NOT TO SCALE



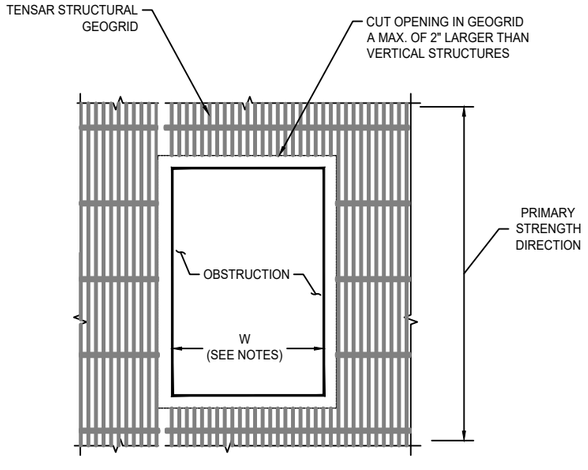
PLAN VIEW

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



PLAN VIEW

NOTES:

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

GEOGRID PENETRATION DETAIL-PLAN VIEW

NOT TO SCALE

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STANDARD DETAIL (9 OF 12)

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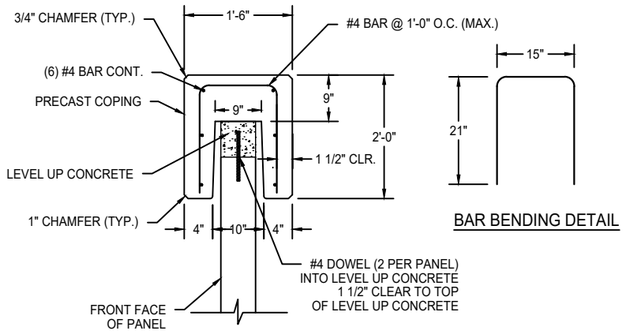
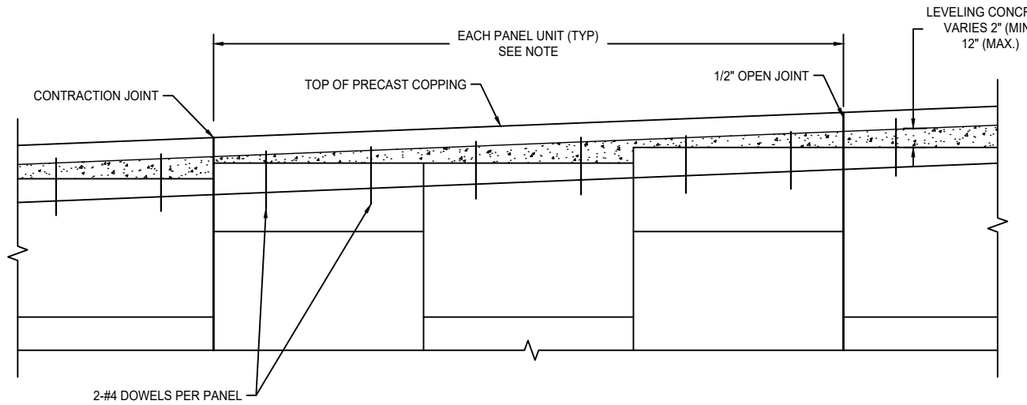


PRECAST COPING SECTION

NOT TO SCALE

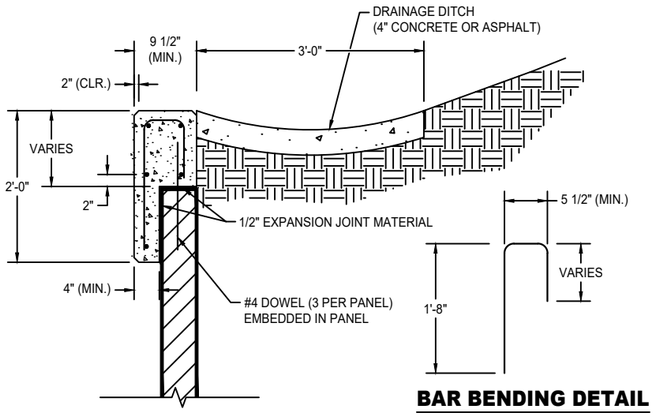
NOTE:

EXPANSION JOINTS IN PRECAST COPING SHALL BE AT 4 PANEL INTERVALS. COPING JOINTS MUST COINCIDE WITH PANEL JOINTS ON FRONT FACE AND SHALL BE PERPENDICULAR TO SLOPE LINE. REINFORCING STEEL SHALL BE STOPPED 2" SHORT OF EITHER SIDE OF THE EXPANSION JOINT. CONTRACTIONS JOINTS IN BETWEEN THE EXPANSION JOINTS SHALL BE PLACED TWO PANELS FROM EXPANSION JOINT.



PRECAST COPING PARTIAL ELEVATION VIEW

NOT TO SCALE

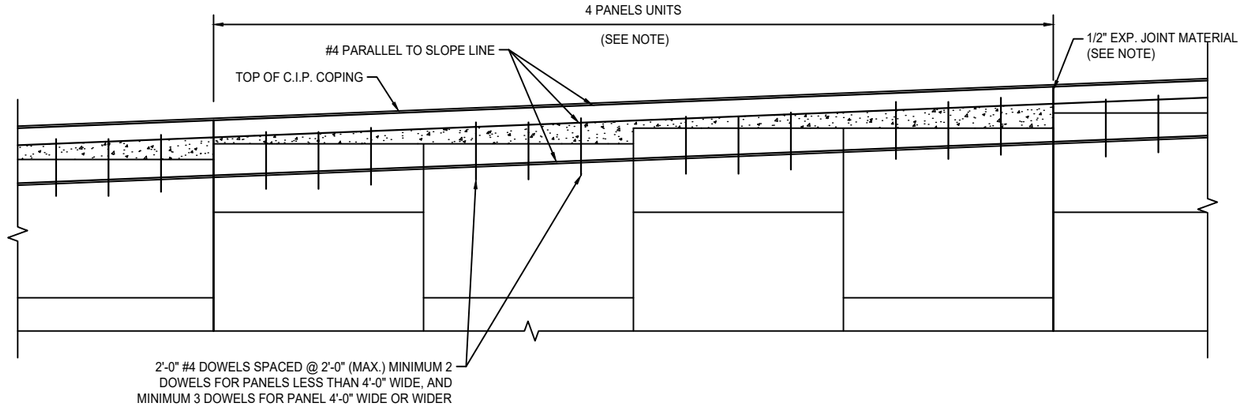


C.I.P COPING WITH SWALE

NOT TO SCALE

NOTE:

EXPANSION JOINTS IN C.I.P COPING SHALL BE AT 4 PANEL INTERVALS (NO MORE THAN 20'-0"). COPING JOINTS MUST COINCIDE WITH PANEL JOINTS ON FRONT FACE AND 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

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STANDARD DETAIL (10 OF 12)

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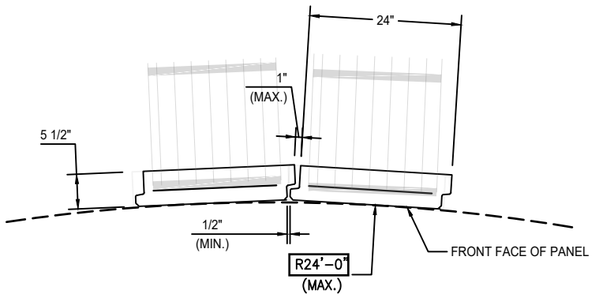
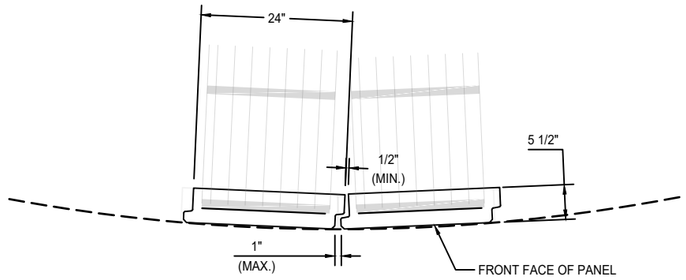
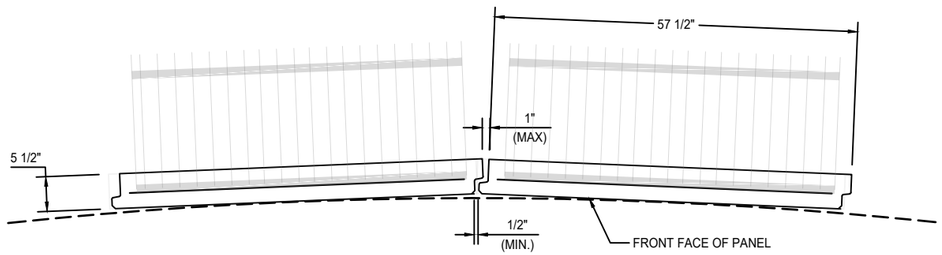


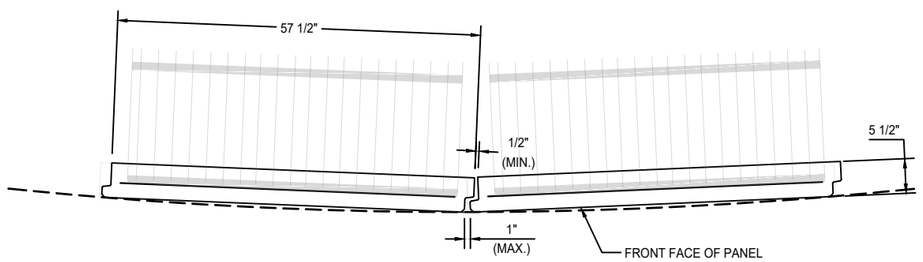
24" MIN. NARROW PANEL
INSIDE RADII
NOT TO SCALE



24" MIN. NARROW PANEL
OUTSIDE RADII
NOT TO SCALE



5X5 INSIDE RADII
NOT TO SCALE



5X5 PANEL OUTSIDE RADII
NOT TO SCALE

Plotted on February 25, 2026
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DESIGNED BY: ---
CHECKED BY : J. PERALTA
ENGINEER OF RECORD (MSE STRUCTURE ONLY):
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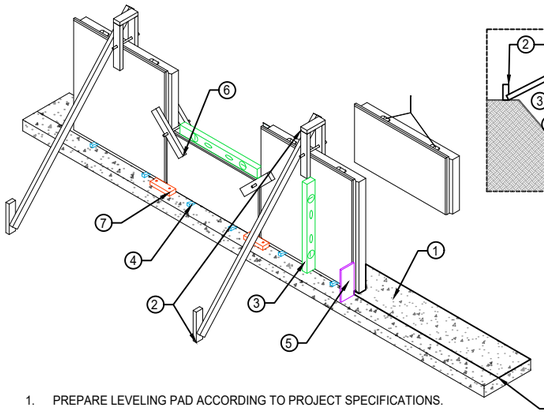
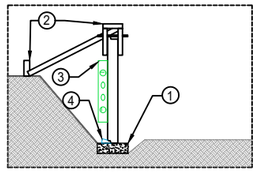
STANDARD DETAIL (11 OF 12)

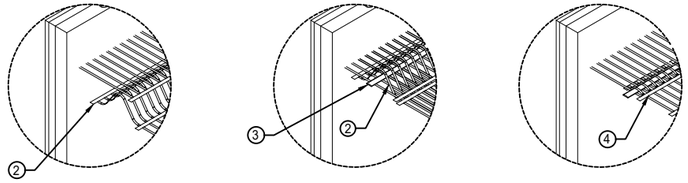




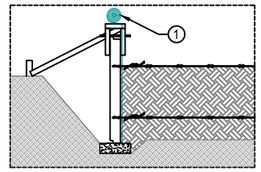
SIDE VIEW

1. PREPARE LEVELING PAD ACCORDING TO PROJECT SPECIFICATIONS.
2. THE FIRST ROW OF PANELS ARE PLACED ON THE LEVELING PAD AND BRACED.
3. THE PANELS SHOULD BE SET ACCORDING TO THE PANEL BATTER RECOMMENDATION.
4. DRIVE WEDGES AT THE QUARTER POINTS OF THE BOTTOM FRONT OF THE PANELS TO MAINTAIN BATTER.
5. USE A 3/4 IN. (19 MM) WOODEN SPACER TO ACHIEVE 3/4 IN. (19 MM) VERTICAL JOINTS.
6. ADJACENT HALF PANELS SHOULD BE CLAMPED TOGETHER.
7. AS AN OPTIONAL SOLUTION, NAIL 2X4 WOODEN BLOCKS AT THE JOINT OF THE PANELS TO PROVIDE ADDITIONAL SUPPORT.

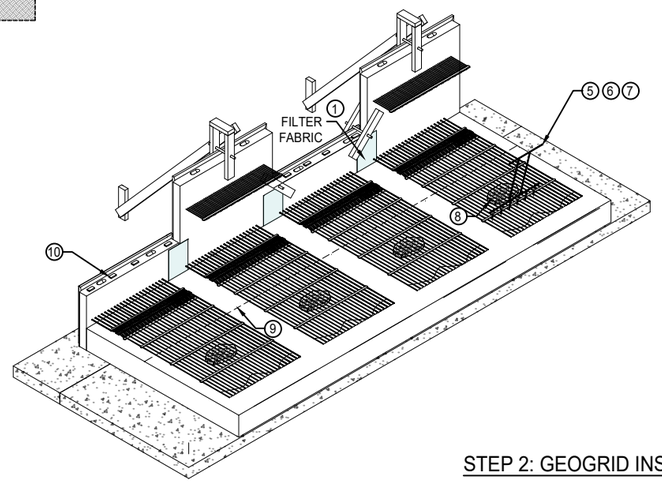
STEP 1: INSTALLING FIRST ROW OF PANELS



GEOGRID INSTALLATION/BODKIN CONNECTION (STEPS 1-3)

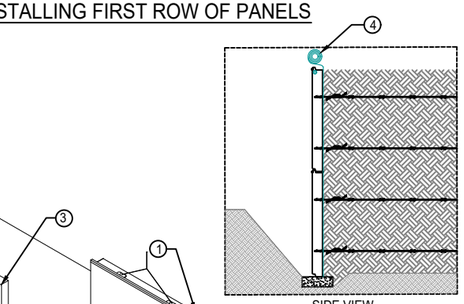


SIDE VIEW

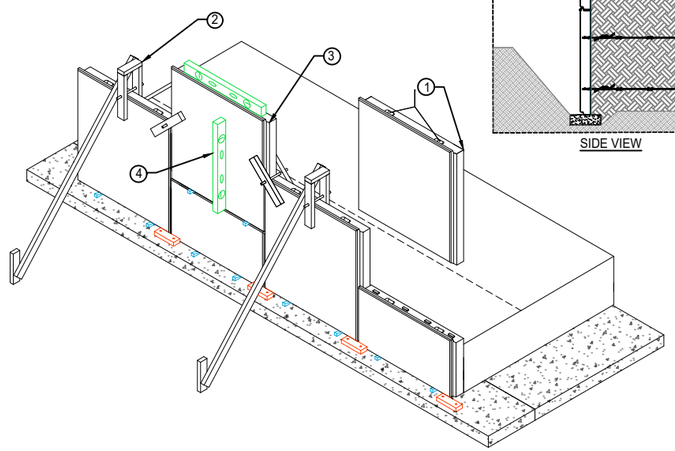


1. PLACE GEOTEXTILE FILTER FABRIC CENTERED ON VERTICAL AND HORIZONTAL JOINTS. SECURE TO PANEL WITH ADHESIVE.
2. BEND THE GEOGRID ROLL SUCH THAT RIBS FORM A TUNNEL WITH THE EMBEDDED GEOGRID TAB.
3. SLIDE THE BODKIN BAR THROUGH THE TUNNEL FORMED BY THE GEOGRID ROLL AND THE GEOGRID TAB.
4. PULL THE GEOGRID ROLL SNUG BY HAND.
5. WHEN THE FILL IS READY TO BE PLACED, USE A TENSIONING DEVICE TO TIGHTEN THE CONNECTION.
6. PLACE THE DEVICE AT LEAST 6' FROM THE BACK OF THE PANEL, ENSURING TO ENGAGE THE TRANSVERSE BAR ACROSS THE WIDTH OF THE GEOGRID.
7. TENSION THE GEOGRID EVENLY SO THAT THE BODKIN CONNECTION IS TAUT FOR ITS ENTIRE WIDTH.
8. PLACE AT LEAST 6 IN. (150 MM) OF SELECTED GRANULAR BACKFILL.
9. THE BACKFILL IS PLACED PARALLEL TO THE WALL STARTING APPROXIMATELY 3 FT (900 MM) FROM THE BACK OF THE PANELS. THE FILL IS THEN SPREAD TOWARD THE GEOGRID ENDS.
10. PLACE THE BEARING PADS EQUALLY SPACED ALONG EACH HORIZONTAL JOINT BETWEEN PANELS.

STEP 2: GEOGRID INSTALLATION, JOINT FABRIC & BACKFILL PLACEMENT

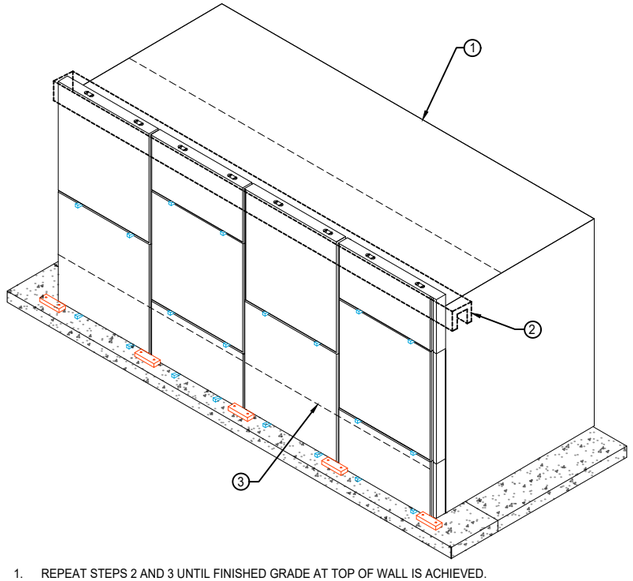


SIDE VIEW



1. WHEN THE SECOND ROW OF PANELS ARE READY TO BE SET, REMOVE CLAMPS FROM FIRST ROW OF PANELS.
2. PLACE THE SECOND ROW OF PANELS IN THE "WINDOW" CREATED BY THE FIRST ROW OF PANELS ONLY AFTER THE BACKFILL HAS REACHED 6 IN. BELOW THE HALF PANELS.
3. CHECK WALL BATTER AND ADJUST AS NEEDED. INSTALL WEDGES AT QUARTER POINTS AND CLAMP SECOND ROW OF PANELS TO THE ADJACENT/FIRST ROW OF PANELS.
4. CONTINUE INSTALLATION PF GEOTEXTILE FILTER FABRIC ALONG VERTICAL AND HORIZONTAL JOINTS.

STEP 3: INSTALLING SUBSEQUENT PANELS



1. REPEAT STEPS 2 AND 3 UNTIL FINISHED GRADE AT TOP OF WALL IS ACHIEVED.
2. INSTALL TOP OF WALL TREATMENT (E.G., COPING, BARRIER, ETC.) PER PROJECT SPECIFICATIONS.
3. AS SOON AS POSSIBLE, PLACE AND COMPACT BACKFILL TO ESTABLISH FINISHED GRADE AT THE FACE OF THE WALL.

STEP 4: FINALIZING WALL INSTALLATION

LEGEND

JOINT FABRIC	WEDGE
REINFORCED FILL	SPACER
EXISTING GROUND	BLOCK
	LEVEL

NOTES:

1. INSTALLATION SEQUENCE SHOWN FOR ILLUSTRATIVE PURPOSES ONLY.
2. DETAILS APPLY TO 5X5, 5X9 AND 5X10 ARES PANEL INSTALLATION SIMILARLY. ARES 5X5 PANELS SHOWN FOR CLARITY.

ARES INSTALLATION SEQUENCE

NOT TO SCALE

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ARES® RETAINING WALL SYSTEMS

PROJECT NAME AND LOCATION

----, ----

OWNER

OWNER PROJECT No.

CLIENT

IDEA

TIC PROJECT No. ----

DRAWN BY: V. JUAREZ
DESIGNED BY: ----
CHECKED BY : J. PERALTA

ENGINEER OF RECORD (MSE STRUCTURE ONLY):
---- P.E. NO.

NO.	DATE	DESCRIPTION	BY
0	06/25/2025	ISSUED FOR REVIEW	VJ

REVISION / ISSUE

SHEET TITLE

STANDARD DETAIL (12 OF 12)

SHEET 13 OF 13

Plotted on February 25, 2026
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Appendix 1.1.4.b

Ares 5x9 Standard Details

ARES® MSE Retaining Wall System

Back to main document

Tensar®
A Division of CMC

ARES®
RETAINING WALL SYSTEMS

MSE STRUCTURE SHOP DRAWINGS
PREPARED FOR

ARES (5'X9') RETAINING WALL SYSTEM

INDEX	
SHEET NUMBER	DESCRIPTION
1	Title Sheet
2	Standard Detail (1 of 13)
3	Standard Detail (2 of 13)
4	Standard Detail (3 of 13)
5	Standard Detail (4 of 13)
6	Standard Detail (5 of 13)
7	Standard Detail (6 of 13)
8	Standard Detail (7 of 13)
9	Standard Detail (8 of 13)
10	Standard Detail (9 of 13)
11	Standard Detail (10 of 13)
12	Standard Detail (11 of 13)
13	Standard Detail (12 of 13)
14	Standard Detail (13 of 13)

PLOTTED ON: 6/25/2025 - 2:02pm
D:\VF_TIDEA\ARES\ARES_IDEA_CAD_DETAILS\A9 ARES 5X9 STANDARD DETAILS.DWG

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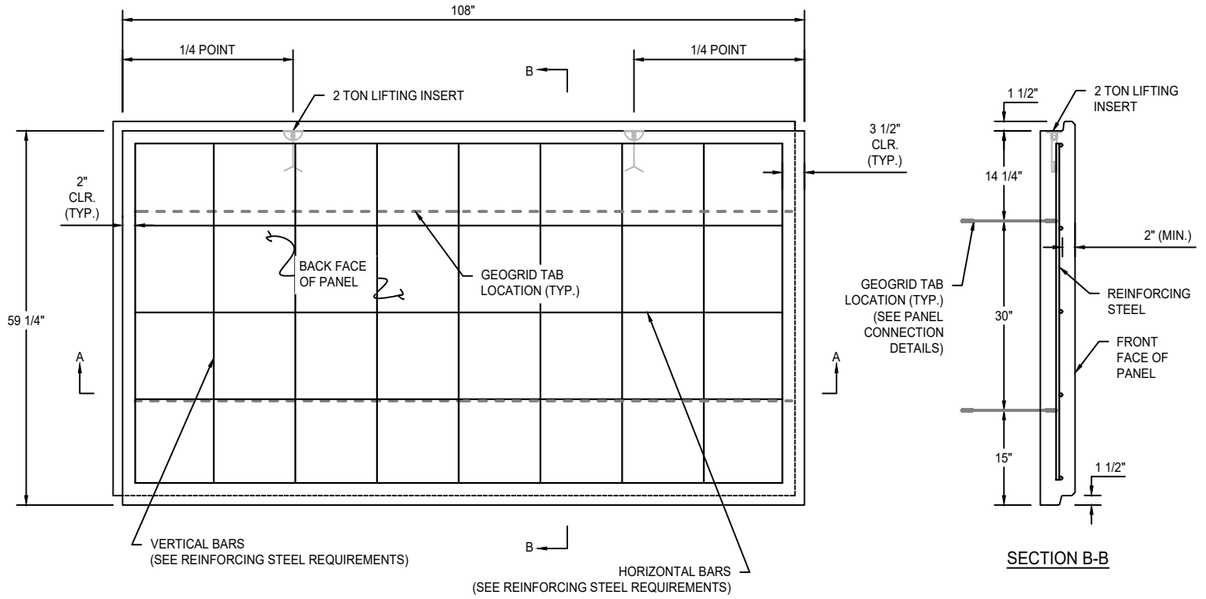
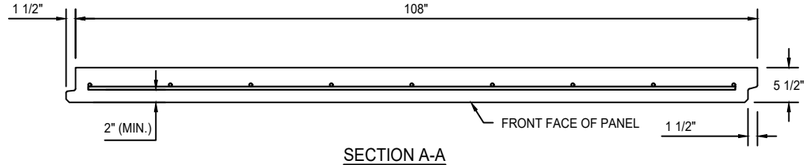
SHEET TITLE

TITLE SHEET



SECTION B-B



SECTION A-A

STANDARD A PANEL SHOWING REBAR REINFORCING

NOT TO SCALE

NOTES:

1. CONCRETE SHALL HAVE A 28 DAY MINIMUM COMPRESSIVE STRENGTH OF 4,000 PSI.

2. REINFORCING STEEL REQUIREMENTS:

   REBAR SIZE AND SPACING :
   HORIZONTAL:#4 BARS @ 16" O.C. (MAX.)
   VERTICAL:#4 BARS @ 14" O.C. (MAX.)

   WIRE MESH SIZE AND SPACING :
   6"x6" - W8.5xW8.5 WELDED WIRE MESH PER ASTM A-82, FABRICATION PER ASTM A-185. (ASTM A1064)

3. REINFORCING STEEL SHALL HAVE A MINIMUM CLEAR COVER OF 1-1/2" FROM ALL PANEL EDGES UNLESS OTHERWISE NOTED.

4. PANEL TOLERANCES:
   - 4.1. OVERALL DIMENSIONS: 1/4-INCH
   - 4.2. GEOGRID LOCATIONS: 1-INCH
   - 4.3. LIFTING INSERT LOCATIONS: 1/8-INCH
   - 4.4. ELEMENT SQUARENESS: 1/2-INCH (BETWEEN DIAGONALS)

5. ALL PANELS SHOWN BACK FACE OF PANEL.

6. PANEL DIMENSIONS AND TOLERANCES SHALL BE IN ACCORDANCE WITH SUPPLEMENTED SPECIFICATIONS.

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ARES® RETAINING WALL SYSTEMS

PROJECT NAME AND LOCATION

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OWNER

OWNER PROJECT No.

CLIENT
IDEA

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DESIGNED BY: ----
CHECKED BY : J. PERALTA

ENGINEER OF RECORD (MSE STRUCTURE ONLY):
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REVISION / ISSUE

SHEET TITLE

STANDARD DETAIL (1 OF 13)

Plotted on February 25, 2026
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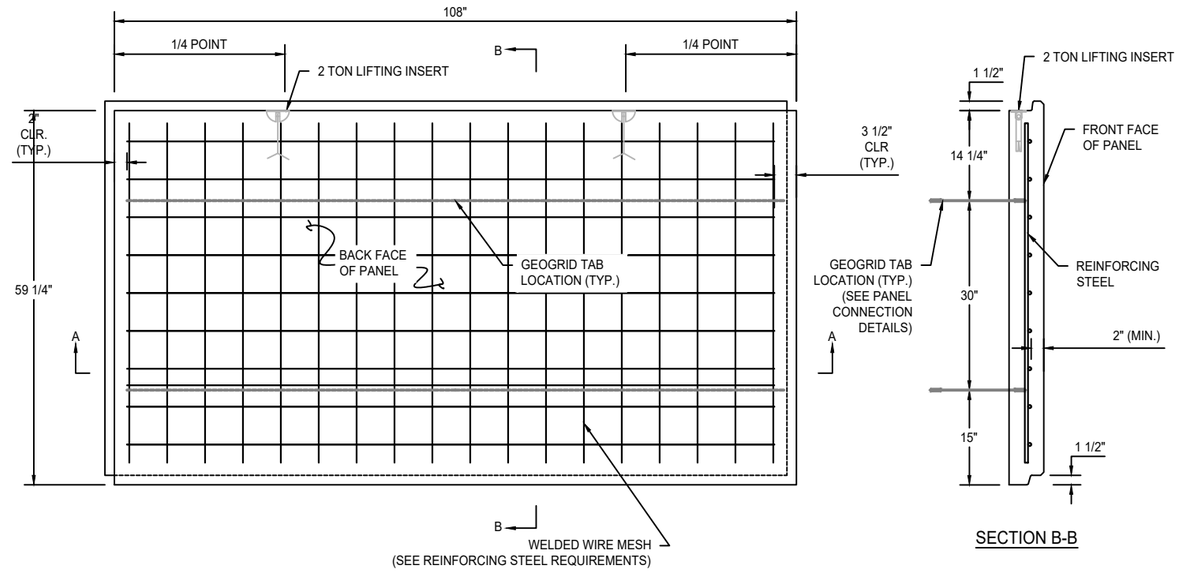
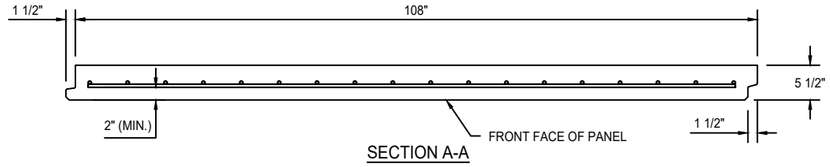


SECTION B-B



SECTION A-A

STANDARD A PANEL SHOWING MESH REINFORCING

NOT TO SCALE

NOTES:

1. CONCRETE SHALL HAVE A 28 DAY MINIMUM COMPRESSIVE STRENGTH OF 4,000 PSI.

2. REINFORCING STEEL REQUIREMENTS:

   REBAR SIZE AND SPACING :
   HORIZONTAL:#4 BARS @ 16" O.C. (MAX.)
   VERTICAL:#4 BARS @ 14" O.C. (MAX.)

   WIRE MESH SIZE AND SPACING :
   6"x6" - W8.5xW8.5 WELDED WIRE MESH PER ASTM A-82, FABRICATION PER ASTM A-185. (ASTM A1064)

3. REINFORCING STEEL SHALL HAVE A MINIMUM CLEAR COVER OF 1-1/2" FROM ALL PANEL EDGES UNLESS OTHERWISE NOTED.

4. PANEL TOLERANCES:
   - 4.1. OVERALL DIMENSIONS: 1/4-INCH
   - 4.2. GEOGRID LOCATIONS: 1-INCH
   - 4.3. LIFTING INSERT LOCATIONS: 1/8-INCH
   - 4.4. ELEMENT SQUARENESS: 1/2-INCH (BETWEEN DIAGONALS)
5. ALL PANELS SHOWN BACK FACE OF PANEL.

6. PANEL DIMENSIONS AND TOLERANCES SHALL BE IN ACCORDANCE WITH SUPPLEMENTED SPECIFICATIONS.

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STANDARD DETAIL (2 OF 13)

Plotted on February 25, 2026
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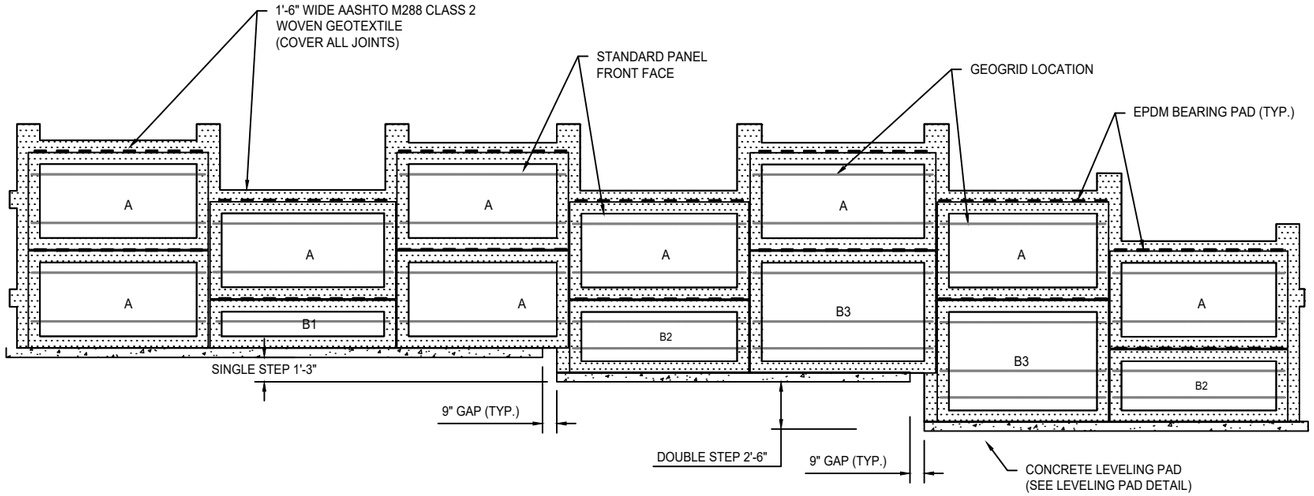
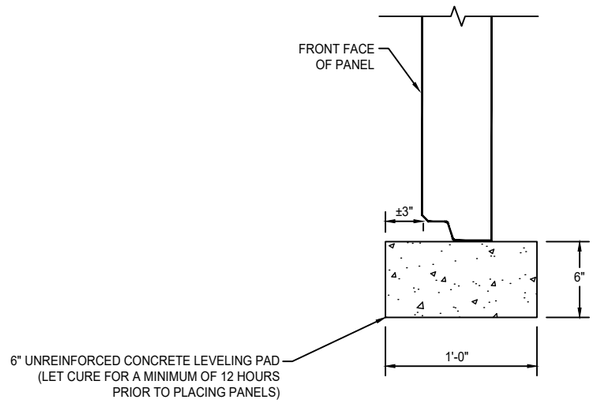


TYPICAL LEVELING PAD STEP AND GEOTEXTILE COVERAGE DETAIL

NOT TO SCALE



LEVELING PAD DETAIL

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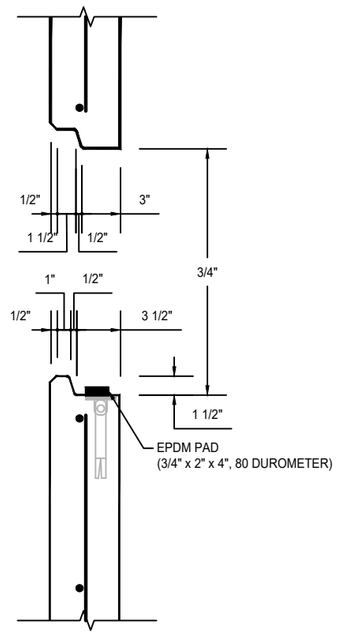

PANEL JOINT DETAIL

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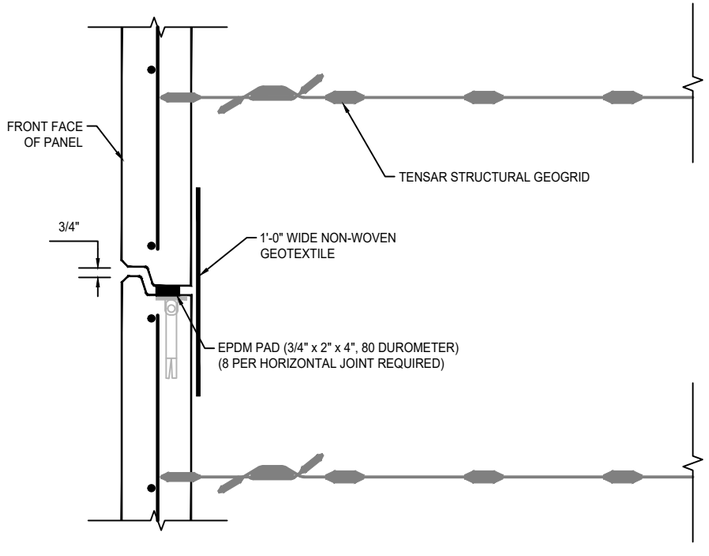

HORIZONTAL JOINT DETAIL

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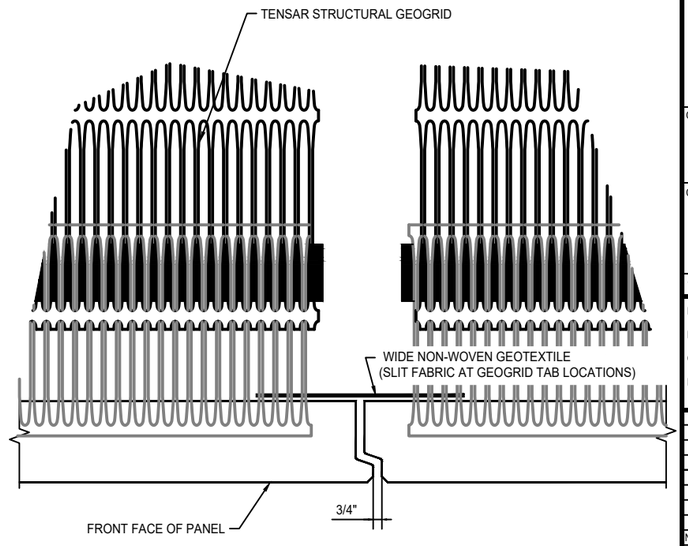

PLAN VIEW

VERTICAL JOINT DETAIL

NOT TO SCALE

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DRAWN BY: V. JUAREZ

DESIGNED BY: ----

CHECKED BY : J. PERALTA

ENGINEER OF RECORD (MSE STRUCTURE ONLY):

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SHEET TITLE

STANDARD DETAIL (3 OF 13)

Plotted on February 25, 2026
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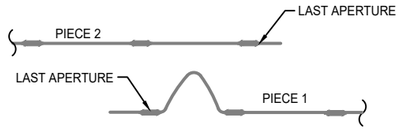
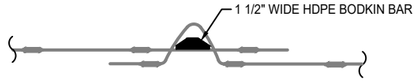
TO FORM A BODKIN CONNECTION:

1. BEND THE LAST APERTURE OF ONE PIECE OF GEOGRID IN HALF.



2. PASS THE BENT APERTURES THROUGH THE APERTURES OF THE SECOND PIECE OF GEOGRID AND INSERT THE BODKIN BAR INTO THE CAVITY BETWEEN THE GEOGRIDS.



3. PULL BOTH PIECES OF GEOGRID IN OPPOSITE DIRECTIONS TO COMPLETE CONNECTION.

NOTE:
THE SPLICED GEOGRID PIECE ON EITHER SIDE OF THE BODKIN CONNECTION SHALL BE AT LEAST 6 FEET LONG UNLESS THE GEOGRID TERMINATES IN A FIXED CONNECTION.

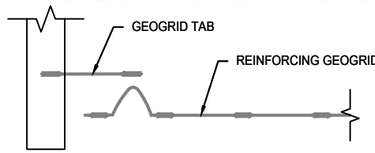
CROSS-SECTION

BODKIN CONNECTION

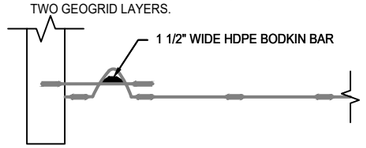
NOT TO SCALE

TO FORM A PANEL BODKIN CONNECTION FOR CONNECTING TO FACING PANEL:

1. BEND THE LAST APERTURE OF REINFORCING GEOGRID AS SHOWN.



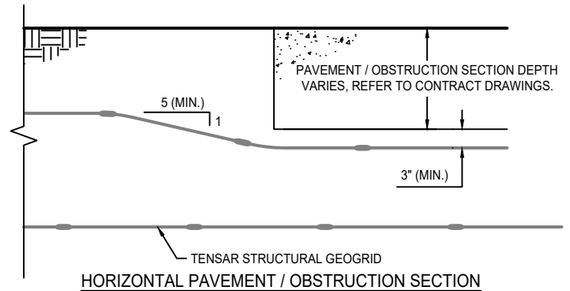
2. PASS THE RIBS OF THE BENT APERTURES THROUGH THE RIBS OF THE GEOGRID TAB AND INSERT THE BODKIN BAR INTO THE SPACE BETWEEN THE TWO GEOGRID LAYERS.



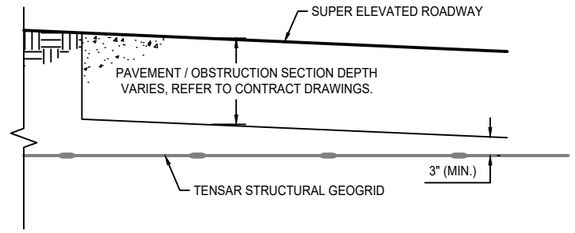
3. PULL REINFORCING GEOGRID TAUT TO TENSION CONNECTION.

PANEL BODKIN CONNECTION

NOT TO SCALE



HORIZONTAL PAVEMENT / OBSTRUCTION SECTION



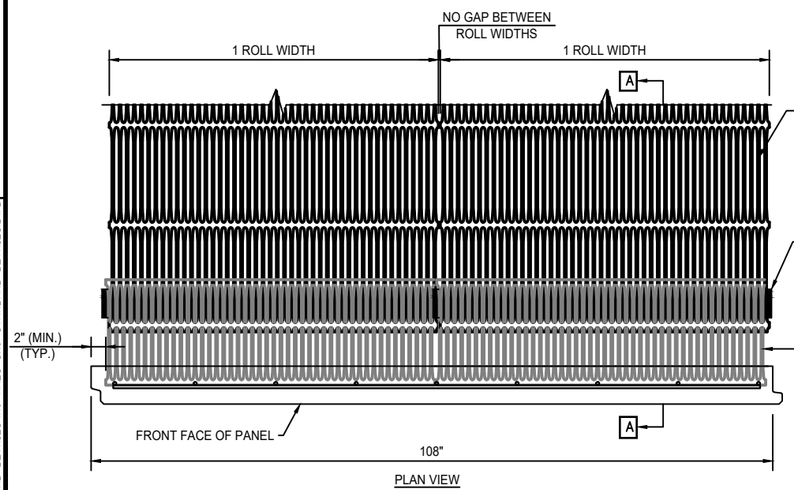
SUPERELEVATED PAVEMENT / OBSTRUCTION SECTION

NOTE:

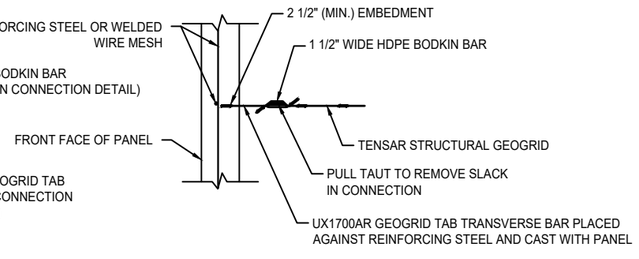
CONTRACTOR IS RESPONSIBLE TO COORDINATE THE PLACEMENT OF THE GEOGRID TO AVOID CONFLICT WITH THE CONTRACT PAVEMENT / OBSTRUCTION SECTION. GEOGRID MUST BE SEPARATED FROM THE PAVEMENT / OBSTRUCTION SECTION BY A MINIMUM OF 3".

GEOGRID PLACEMENT AT PAVEMENT / OBSTRUCTION SECTION

NOT TO SCALE



PLAN VIEW



SECTION A-A

CONNECTION DETAIL PLAN VIEW (96% COVERAGE)

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ARES® RETAINING WALL SYSTEMS

PROJECT NAME AND LOCATION

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CLIENT

IDEA

TIC PROJECT No. ----

DRAWN BY: V. JUAREZ

DESIGNED BY: ----

CHECKED BY : J. PERALTA

ENGINEER OF RECORD (MSE STRUCTURE ONLY):

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SHEET TITLE

STANDARD DETAIL (4 OF 13)

Plotted on February 25, 2026
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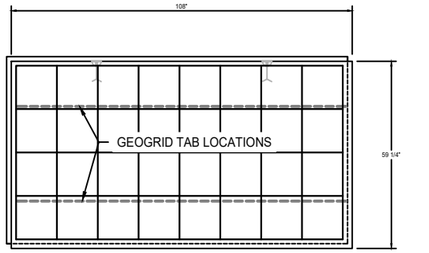
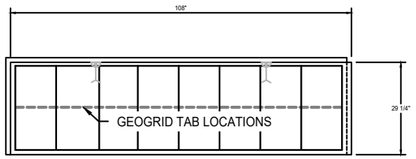
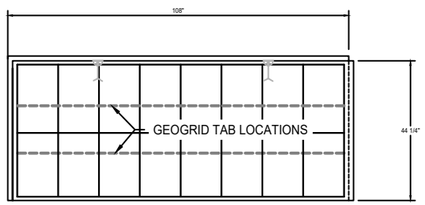


STANDARD A PANEL



STANDARD B1 PANEL



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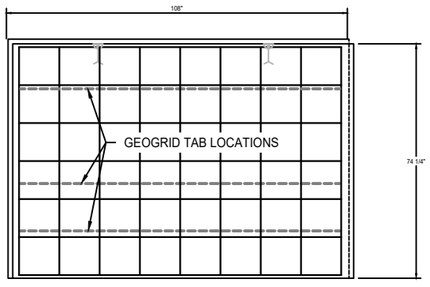
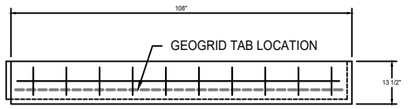
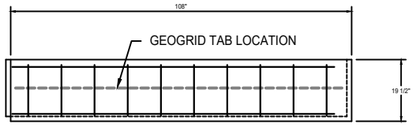


STANDARD B3 PANEL



STANDARD T12 PANEL



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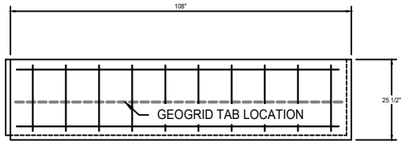
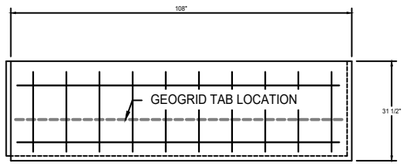
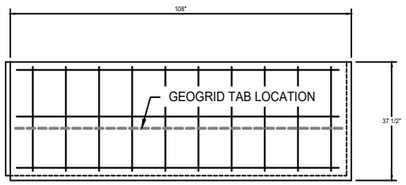


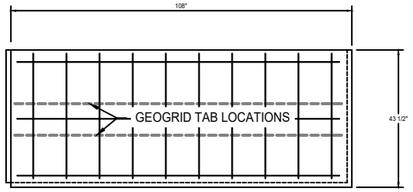
STANDARD T24 PANEL



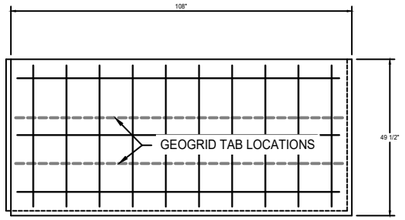
STANDARD T30 PANEL



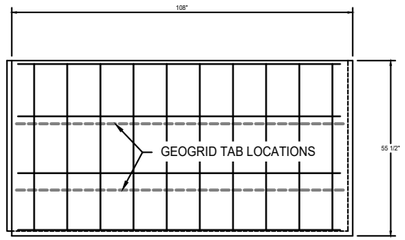
STANDARD T36 PANEL



STANDARD T42 PANEL



STANDARD T48 PANEL



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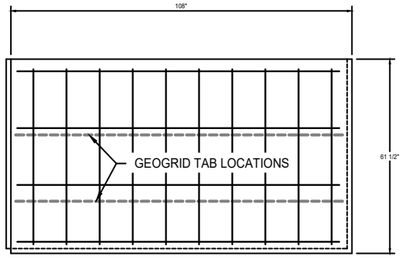
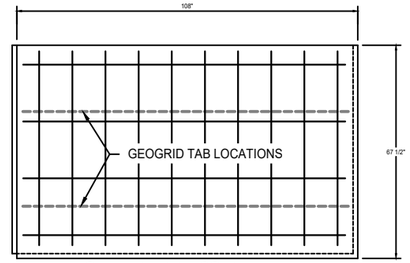
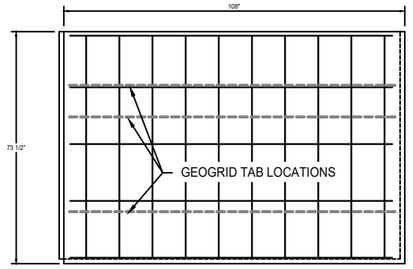


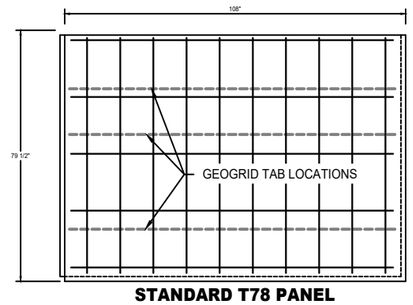
STANDARD T60 PANEL



STANDARD T66 PANEL



STANDARD T72 PANEL



STANDARD T78 PANEL

ALL PANELS SHOWN BACK FACE

NOTES:

1. PANEL CONCRETE REQUIREMENTS:
   - 1.1. CONCRETE SHALL HAVE A 28-DAY MINIMUM COMPRESSIVE STRENGTH OF 4,000 PSI.
2. PANEL REINFORCING STEEL REQUIREMENTS:
   - 2.1. ALL REINFORCING STEEL SHALL HAVE A MINIMUM CLEAR COVER OF 1-1/2" FROM ALL PANEL EDGES UNLESS OTHERWISE NOTED.
   - 2.2. REBAR SIZE AND SPACING REQUIREMENTS:
     - 2.2.1. HORIZONTAL: #4 BARS AT 16" O.C. (MAX.)
     - 2.2.2. VERTICAL: #4 BARS AT 14" O.C. (MAX.)
   - 2.3. WIRE MESH SIZE AND SPACING REQUIREMENTS
     - 24.1 WELDED WIRE MESH 6"x6" - W8.5xW8.5
   - 2.5. PANEL DIMENSIONS AND TOLERANCES SHALL BE IN ACCORDANCE WITH SUPPLEMENTED SPECIFICATIONS.

STANDARD 5' X 9' PANELS

NOT TO SCALE

Tensar.
A Division of CMC

Tensar International Corporation

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

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ARES® RETAINING WALL SYSTEMS

PROJECT NAME AND LOCATION

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CLIENT

IDEA

TIC PROJECT No. ----

DRAWN BY: V. JUAREZ

DESIGNED BY: ----

CHECKED BY : J. PERALTA

ENGINEER OF RECORD (MSE STRUCTURE ONLY):

---- P.E. NO.

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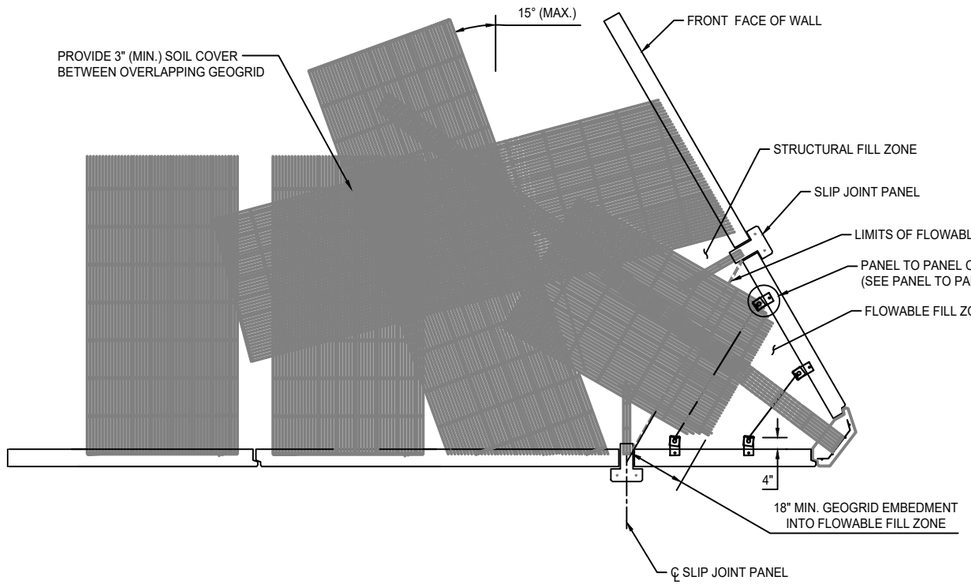
0	06/25/2025	ISSUED FOR REVIEW	VJ

REVISION / ISSUE

SHEET TITLE

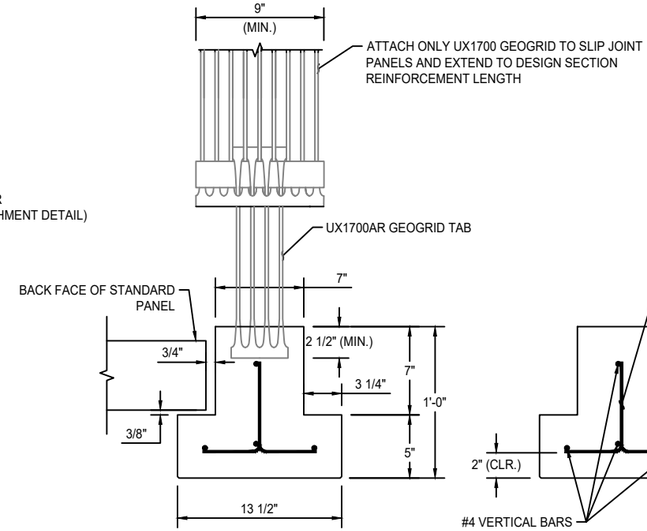
STANDARD DETAIL (5 OF 13)

Plotted on: February 25, 2026
C:\VP\IDEA\ARES\ARES IDEA_CAD DETAILS\A9 ARES 5X9 STANDARD DETAILS.DWG



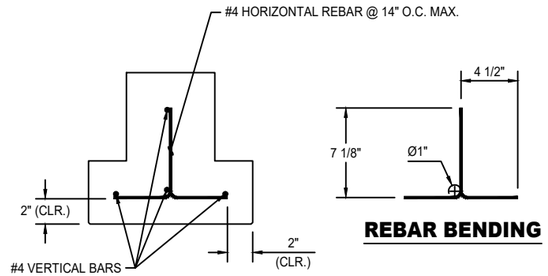
ACUTE CORNER LESS THAN 70°

NOT TO SCALE

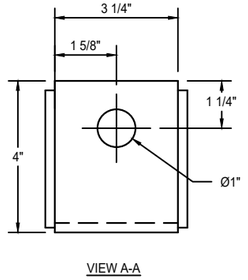


SLIP JOINT DETAIL

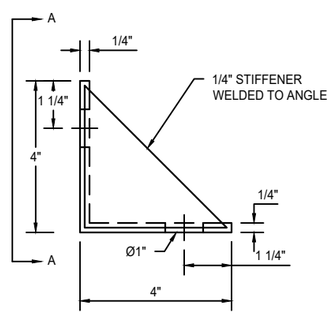
NOT TO SCALE



REBAR BENDING



VIEW A-A

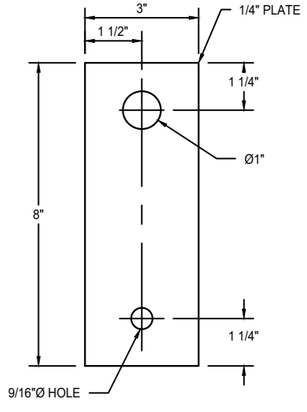


CONNECTION BOX

NOT TO SCALE

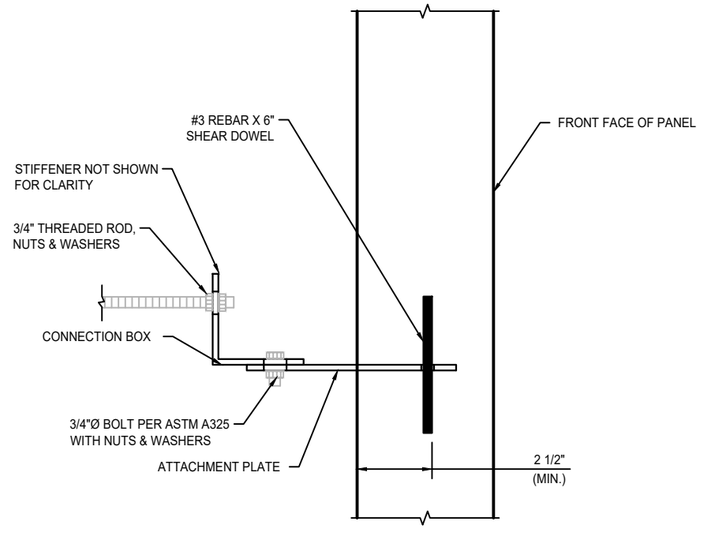
FABRICATION ATTACHMENT STEEL NOTES:

1. ALL STEEL ANGLES AND PLATES SHALL BE PER ASTM A36.
2. ALL FABRICATED STEEL PARTS SHALL BE HOT DIP GALVANIZED AFTER FABRICATION.
3. ALL DIMENSIONS ARE MINIMUM REQUIRED.



ATTACHMENT PLATE

NOT TO SCALE



PANEL TO PANEL ATTACHMENT

NOT TO SCALE

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CLIENT

IDEA

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DRAWN BY: V. JUAREZ

DESIGNED BY: ----

CHECKED BY : J. PERALTA

ENGINEER OF RECORD (MSE STRUCTURE ONLY):

---- P.E. NO.

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REVISION / ISSUE

SHEET TITLE

STANDARD DETAIL (6 OF 13)

Plotted on February 25, 2026
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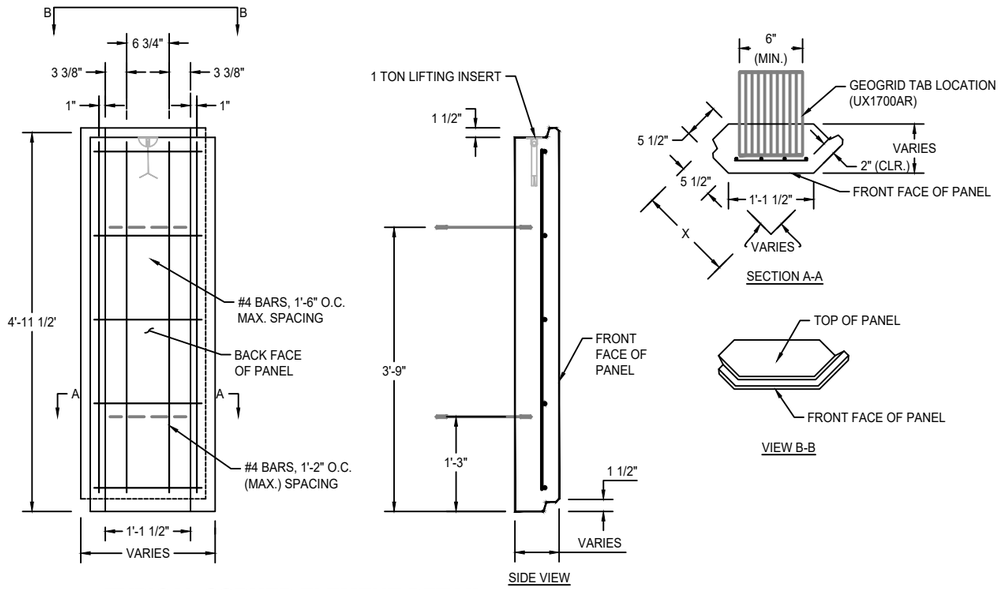
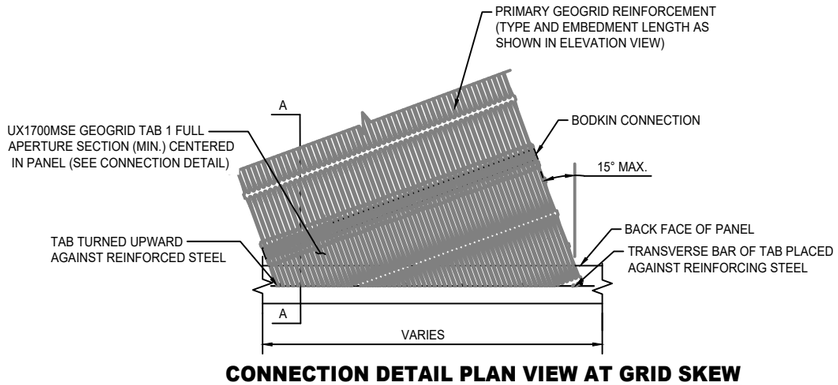


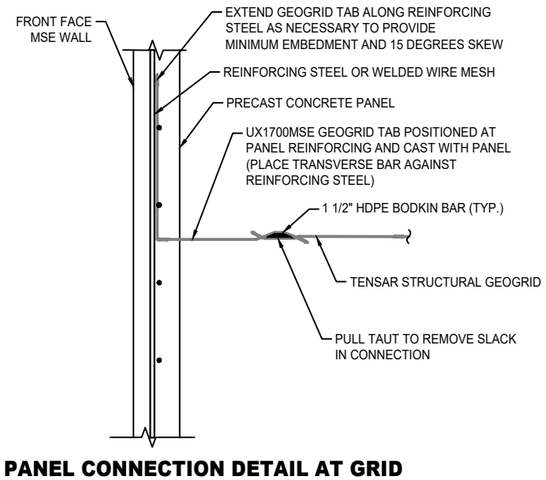
TYPICAL CORNER ELEMENT DETAIL

NOT TO SCALE



CONNECTION DETAIL PLAN VIEW AT GRID SKEW

NOT TO SCALE



PANEL CONNECTION DETAIL AT GRID SKEW (SECTION A-A)

NOT TO SCALE

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SHEET TITLE

STANDARD DETAIL (7 OF 13)

SHEET 8 OF 14

Plotted on February 25, 2026
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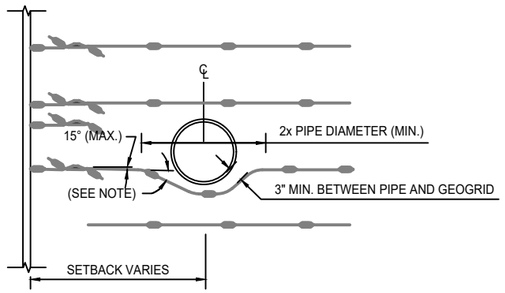
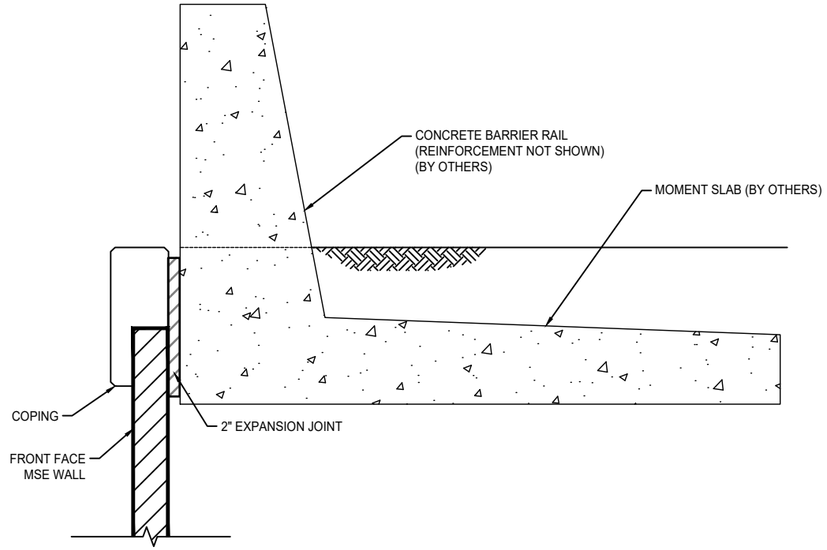


NOTE:

IF GEOGRID MUST BE MORE THAN 15° FROM HORIZONTAL, EXTEND GEOGRID 4'-0" LONGER THAN SHOWN ON ELEVATION VIEW.

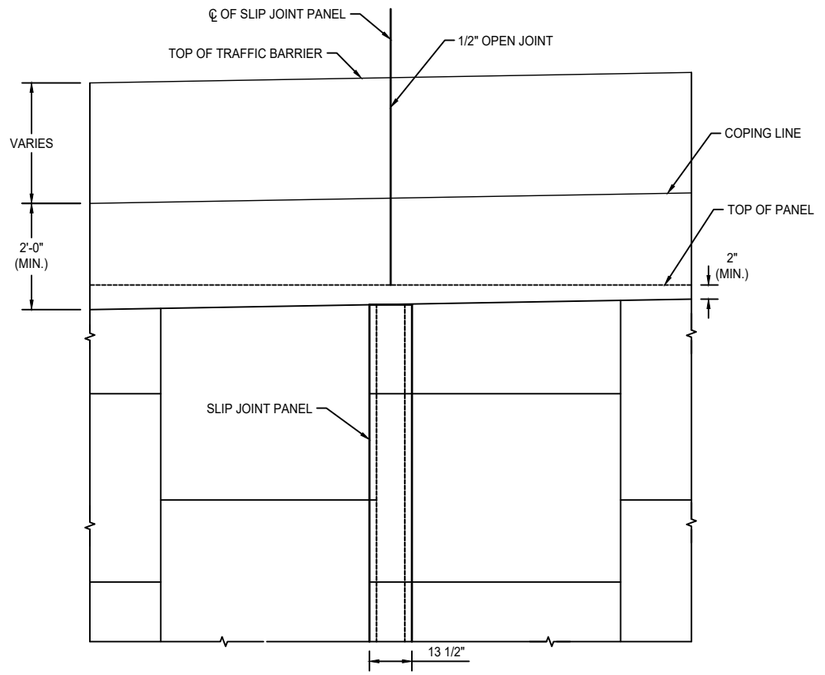
TYPICAL OBSTRUCTION AVOIDANCE DETAIL

NOT TO SCALE



CONCRETE BARRIER RAIL WITH MOMENT SLAB ON MSE WALL

NOT TO SCALE



C.I.P. TRAFFIC BARRIER OVER SLIP JOINT PANEL

NOT TO SCALE

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SHEET TITLE

STANDARD DETAIL (8 OF 13)

Plotted on February 25, 2025
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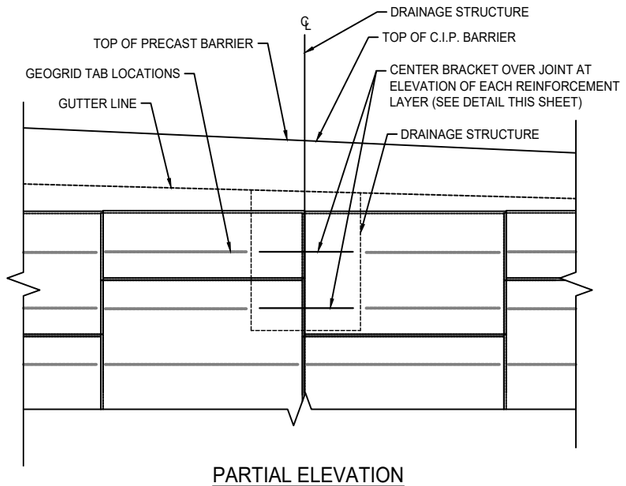
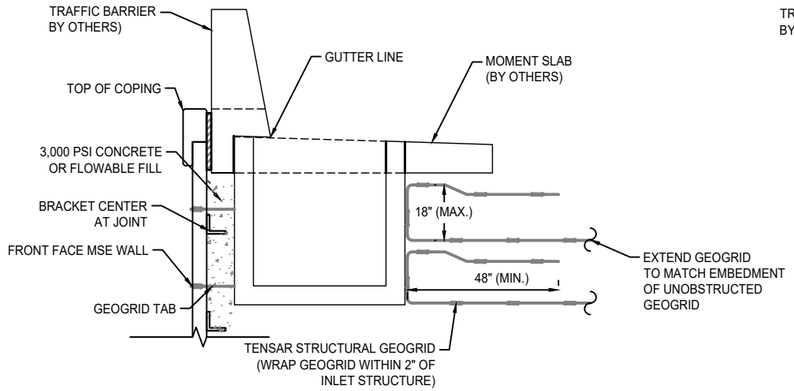
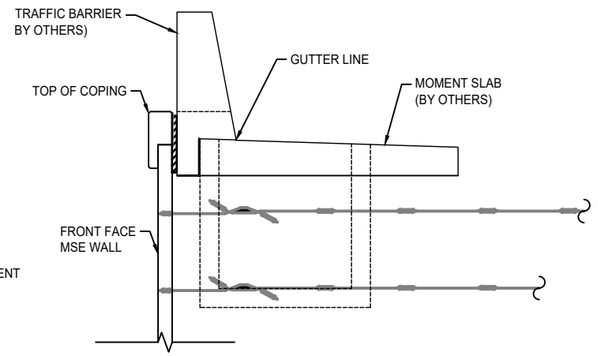


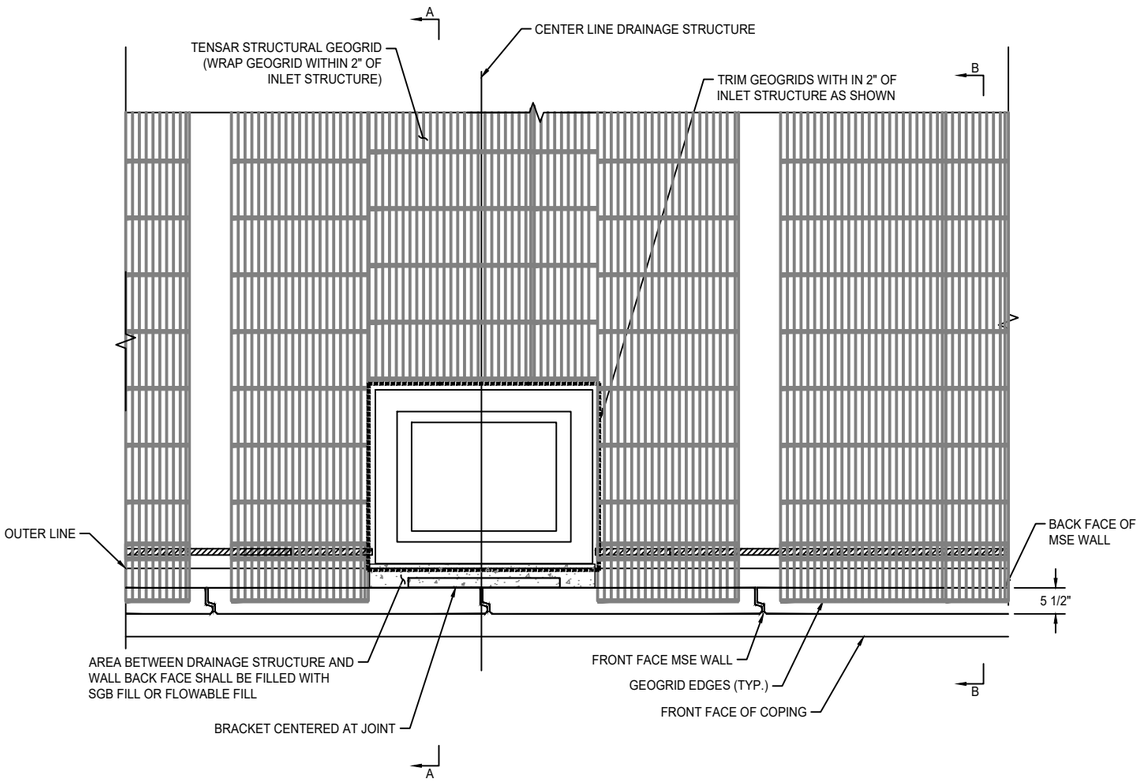
PARTIAL ELEVATION



SECTION A-A



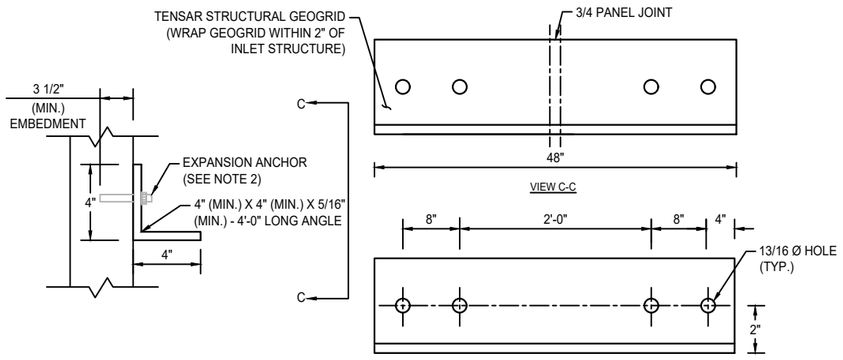
SECTION B-B



PARTIAL PLAN

MSE WALL AT DRAINAGE INLET

NOT TO SCALE



NOTES:

1. ALL FABRICATED STEEL PARTS SHALL BE HOT DIP GALVANIZED AFTER FABRICATION.
2. ANCHOR SHALL BE Ø 5/8" KIWIK BOLT 3 HOT-DIP GALVANIZED (HDG) EXPANSION ANCHOR OR APPROVED EQUAL.

BRACKET CENTERED OVER JOINT DETAIL

NOT TO SCALE

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

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SHEET TITLE

STANDARD DETAIL (9 OF 13)

Plotted on February 25, 2025
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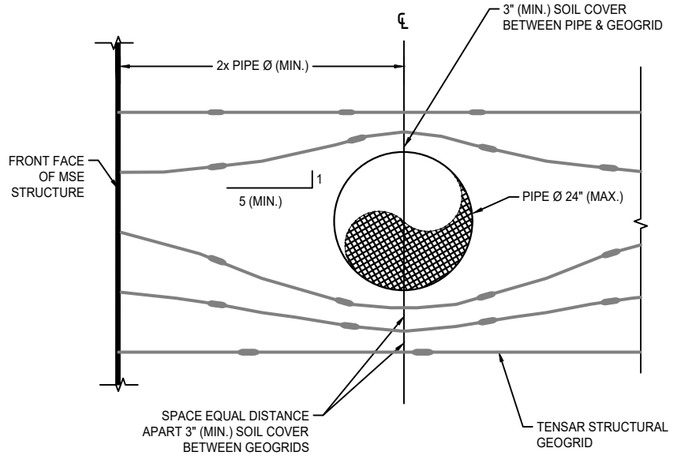
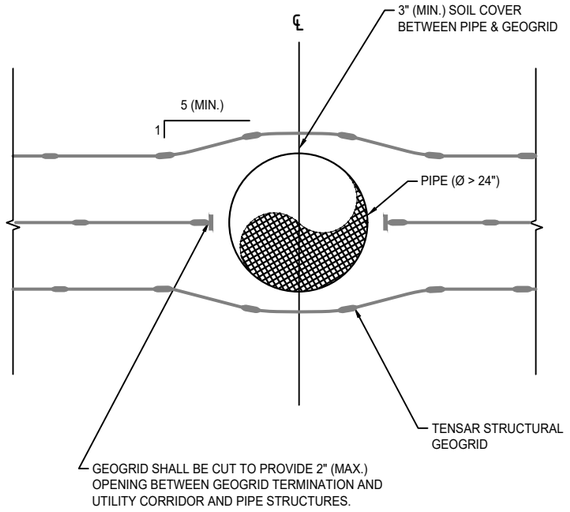


CROSS-SECTION

GEOGRID PLACEMENT AT PIPE PARALLEL TO MSE STRUCTURE FACE

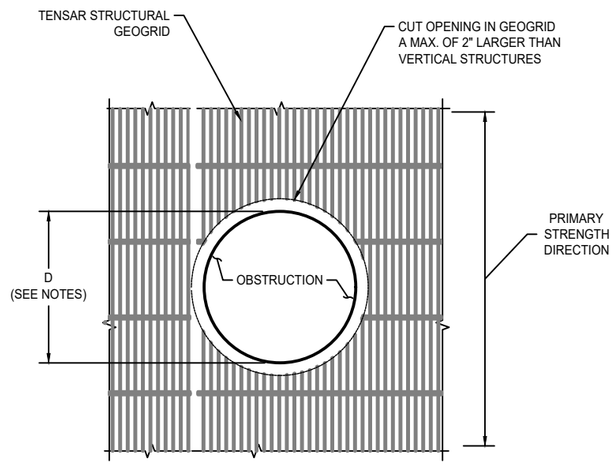
NOT TO SCALE



ELEVATION VIEW

GEOGRID PLACEMENT AT PIPE PERPENDICULAR TO MSE STRUCTURE FACE

NOT TO SCALE



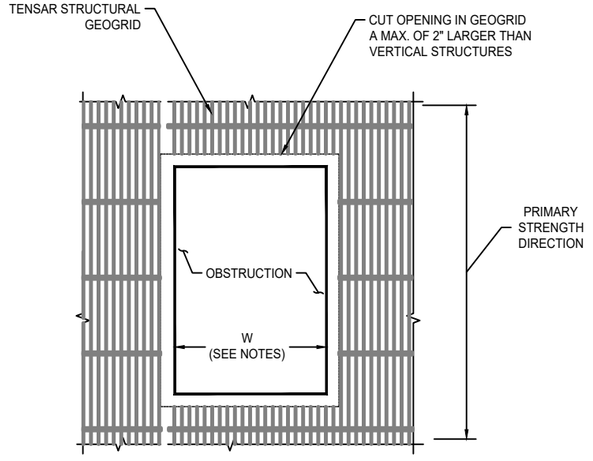
PLAN VIEW

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



PLAN VIEW

NOTES:

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

GEOGRID PENETRATION DETAIL-PLAN VIEW

NOT TO SCALE

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ARES® RETAINING WALL SYSTEMS

PROJECT NAME AND LOCATION

----, ----

OWNER

OWNER PROJECT No.

CLIENT

IDEA

TIC PROJECT No. ----

DRAWN BY: V. JUAREZ

DESIGNED BY: ----

CHECKED BY : J. PERALTA

ENGINEER OF RECORD (MSE STRUCTURE ONLY):

---- P.E. NO.

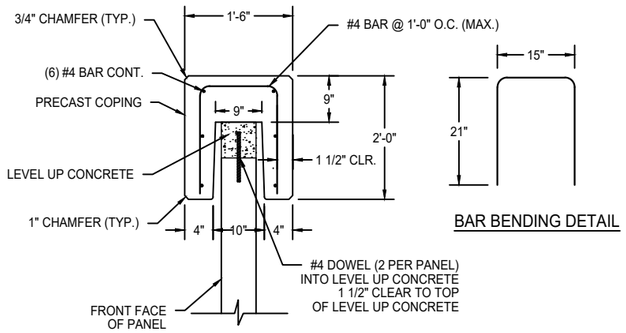
NO.	DATE	DESCRIPTION	BY
0	06/25/2025	ISSUED FOR REVIEW	VJ

REVISION / ISSUE

SHEET TITLE

STANDARD DETAIL (10 OF 13)

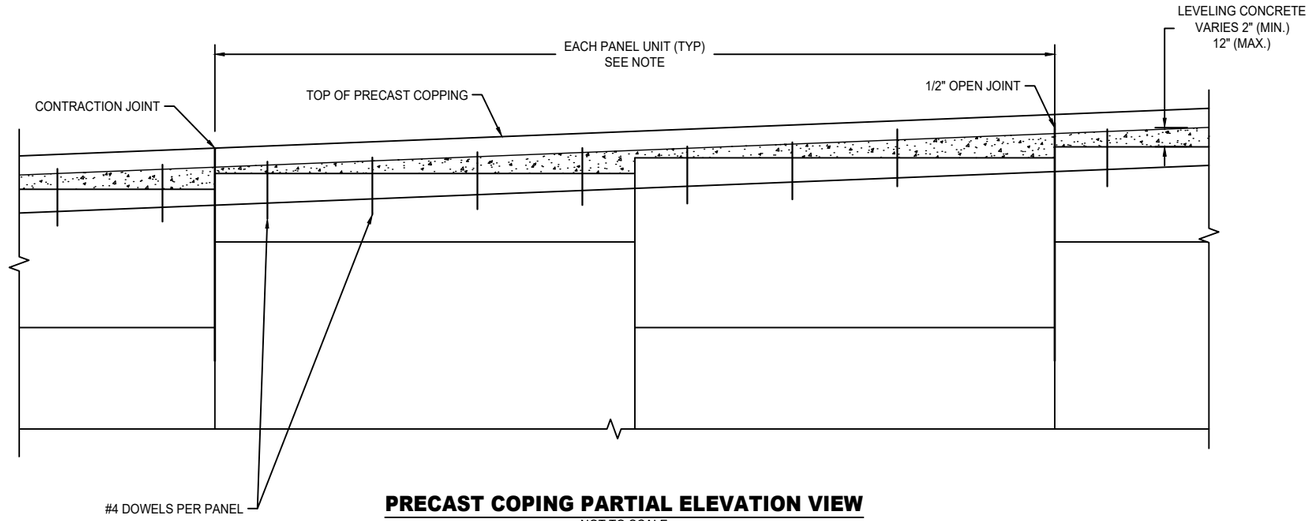
Plotted on February 25, 2026
C:\VJ\IDEA\ARES\ARES IDEA.CAD DETAILS\A9 ARES 5X9 STANDARD DETAILS.DWG



PRECAST COPING SECTION

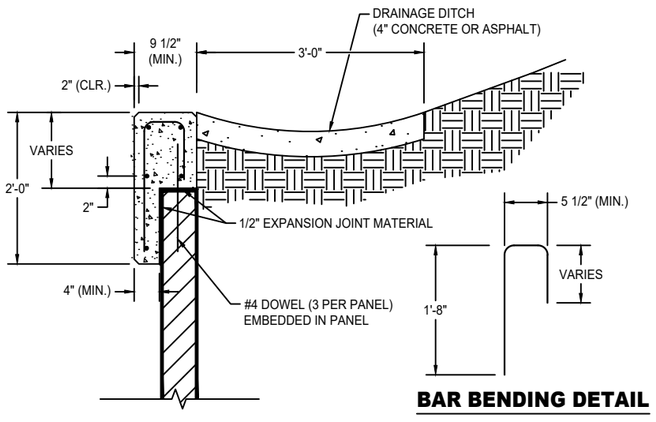
NOT TO SCALE

NOTE:
EXPANSION JOINTS IN PRECAST COPING SHALL BE AT 4 PANEL INTERVALS. COPING JOINTS MUST COINCIDE WITH PANEL JOINTS ON FRONT FACE AND SHALL BE PERPENDICULAR TO SLOPE LINE. REINFORCING STEEL SHALL BE STOPPED 2" SHORT OF EITHER SIDE OF THE EXPANSION JOINT. CONTRACTIONS JOINTS IN BETWEEN THE EXPANSION JOINTS SHALL BE PLACED TWO PANELS FROM EXPANSION JOINT.



PRECAST COPING PARTIAL ELEVATION VIEW

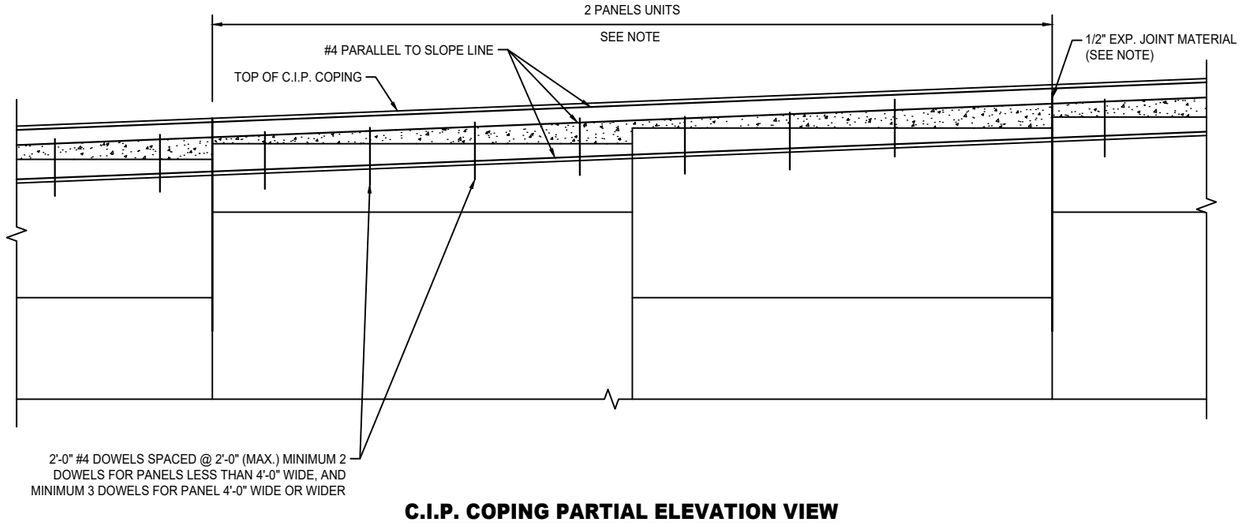
NOT TO SCALE



C.I.P COPING WITH SWALE

NOT TO SCALE

NOTE:
EXPANSION JOINTS IN C.I.P COPING SHALL BE AT 2 PANEL INTERVALS (NO MORE THAN 20'-0"). COPING JOINTS MUST COINCIDE WITH PANEL JOINTS ON FRONT FACE AND 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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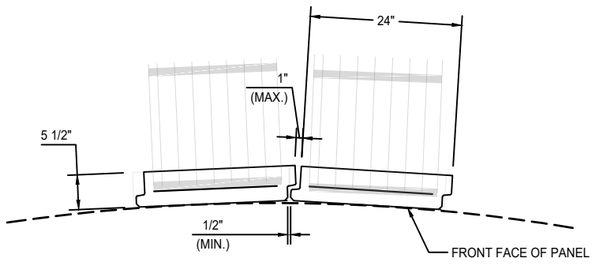
REVISION / ISSUE

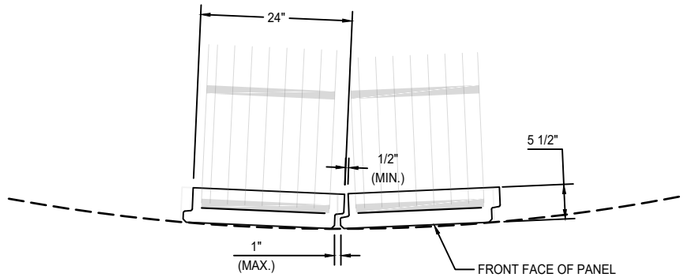
SHEET TITLE

STANDARD DETAIL (11 OF 13)

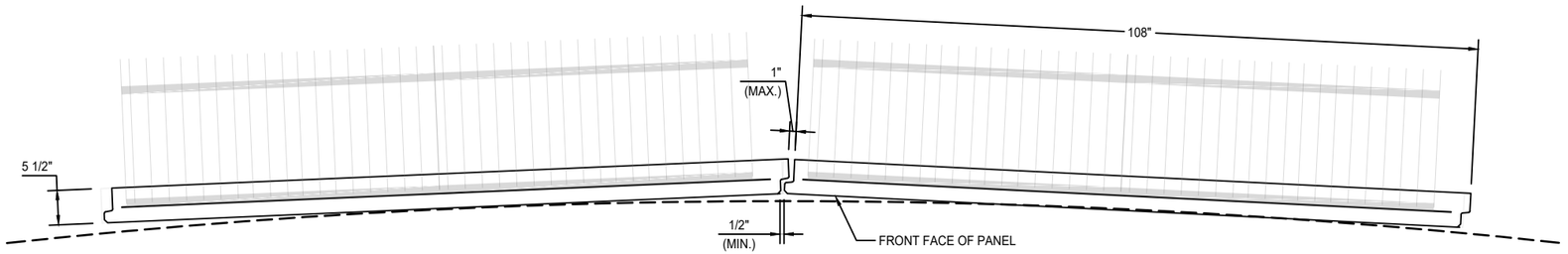
Plotted on February 25, 2026 - C:\VJ_IDEA\ARES\ARES IDEA.CAD DETAILS\A9 ARES 5X9 STANDARD DETAILS.DWG



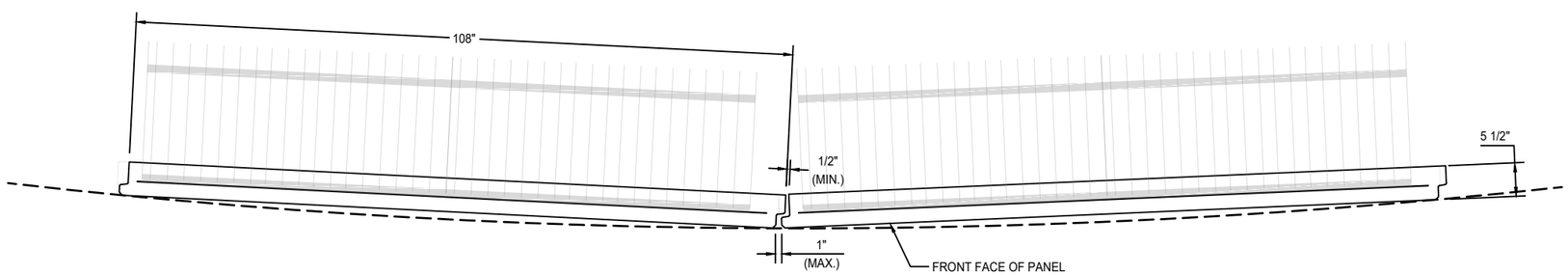
24" MIN. NARROW PANEL
INSIDE RADII
NOT TO SCALE



24" MIN. NARROW PANEL
OUTSIDE RADII
NOT TO SCALE



5X9 INSIDE RADII
NOT TO SCALE



5X9 OUTSIDE RADII
NOT TO SCALE

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Alpharetta, Georgia 30009 | 770-344-2090

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CLIENT
IDEA

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

SHEET TITLE
STANDARD DETAIL (12 OF 13)

Plotted on February 25, 2026
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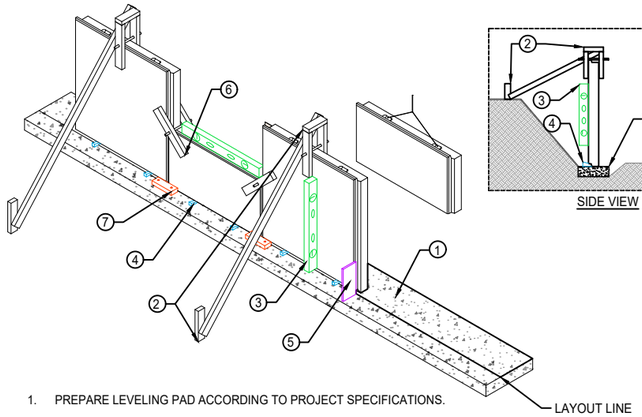
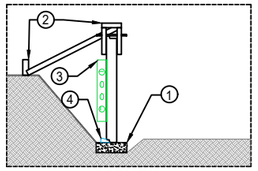
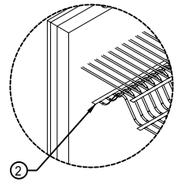


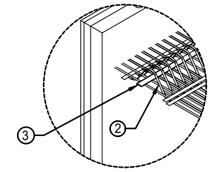


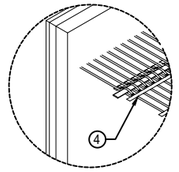
SIDE VIEW

1. PREPARE LEVELING PAD ACCORDING TO PROJECT SPECIFICATIONS.
2. THE FIRST ROW OF PANELS ARE PLACED ON THE LEVELING PAD AND BRACED.
3. THE PANELS SHOULD BE SET ACCORDING TO THE PANEL BATTER RECOMMENDATION.
4. DRIVE WEDGES AT THE QUARTER POINTS OF THE BOTTOM FRONT OF THE PANELS TO MAINTAIN BATTER.
5. USE A 3/4 IN. (19 MM) WOODEN SPACER TO ACHIEVE 3/4 IN. (19 MM) VERTICAL JOINTS.
6. ADJACENT HALF PANELS SHOULD BE CLAMPED TOGETHER.
7. AS AN OPTIONAL SOLUTION, NAIL 2X4 WOODEN BLOCKS AT THE JOINT OF THE PANELS TO PROVIDE ADDITIONAL SUPPORT.

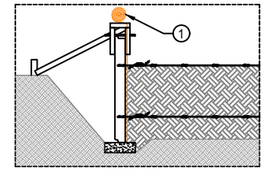
STEP 1: INSTALLING FIRST ROW OF PANELS



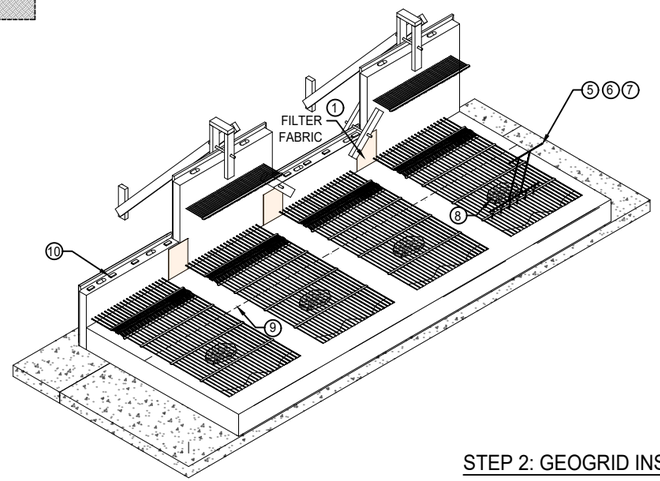




GEOGRID INSTALLATION/BODKIN CONNECTION (STEPS 1-3)

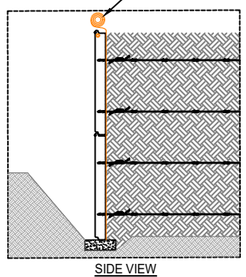


SIDE VIEW

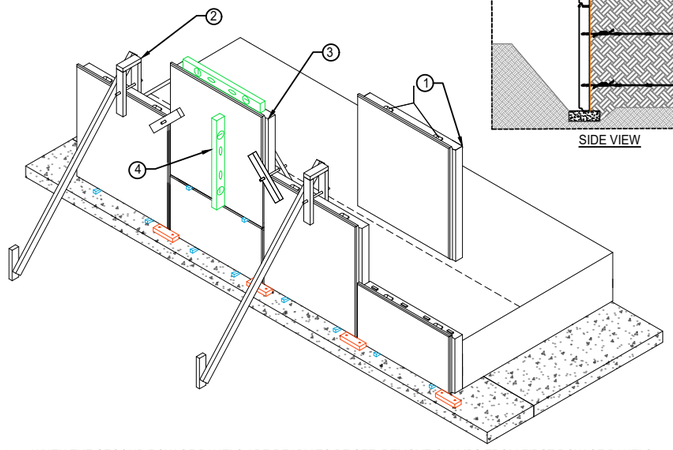


1. PLACE GEOTEXTILE FILTER FABRIC CENTERED ON VERTICAL AND HORIZONTAL JOINTS. SECURE TO PANEL WITH ADHESIVE.
2. BEND THE GEOGRID ROLL SUCH THAT RIBS FORM A TUNNEL WITH THE EMBEDDED GEOGRID TAB.
3. SLIDE THE BODKIN BAR THROUGH THE TUNNEL FORMED BY THE GEOGRID ROLL AND THE GEOGRID TAB.
4. PULL THE GEOGRID ROLL SNUG BY HAND.
5. WHEN THE FILL IS READY TO BE PLACED, USE A TENSIONING DEVICE TO TIGHTEN THE CONNECTION.
6. PLACE THE DEVICE AT LEAST 6' FROM THE BACK OF THE PANEL, ENSURING TO ENGAGE THE TRANSVERSE BAR ACROSS THE WIDTH OF THE GEOGRID.
7. TENSION THE GEOGRID EVENLY SO THAT THE BODKIN CONNECTION IS TAUT FOR ITS ENTIRE WIDTH.
8. PLACE AT LEAST 6 IN. (150 MM) OF SELECTED GRANULAR BACKFILL.
9. THE BACKFILL IS PLACED PARALLEL TO THE WALL STARTING APPROXIMATELY 3 FT (900 MM) FROM THE BACK OF THE PANELS. THE FILL IS THEN SPREAD TOWARD THE GEOGRID ENDS.
10. PLACE THE BEARING PADS EQUALLY SPACED ALONG EACH HORIZONTAL JOINT BETWEEN PANELS.

STEP 2: GEOGRID INSTALLATION, JOINT FABRIC & BACKFILL PLACEMENT

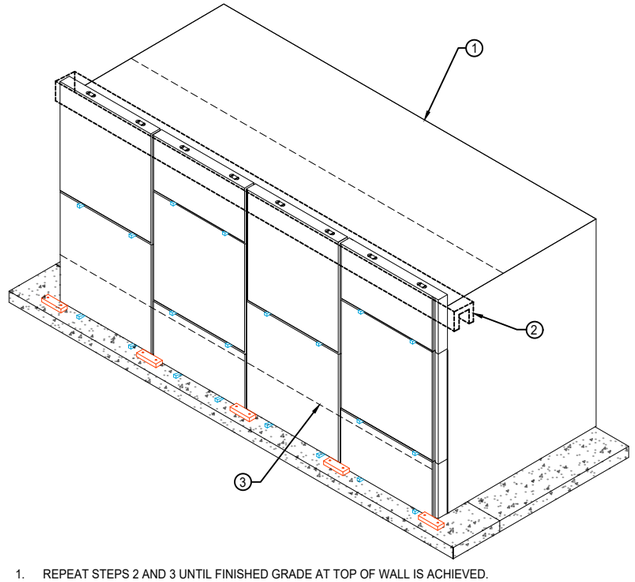


SIDE VIEW



1. WHEN THE SECOND ROW OF PANELS ARE READY TO BE SET, REMOVE CLAMPS FROM FIRST ROW OF PANELS.
2. PLACE THE SECOND ROW OF PANELS IN THE "WINDOW" CREATED BY THE FIRST ROW OF PANELS ONLY AFTER THE BACKFILL HAS REACHED 6 IN. BELOW THE HALF PANELS.
3. CHECK WALL BATTER AND ADJUST AS NEEDED. INSTALL WEDGES AT QUARTER POINTS AND CLAMP SECOND ROW OF PANELS TO THE ADJACENT/FIRST ROW OF PANELS.
4. CONTINUE INSTALLATION PF GEOTEXTILE FILTER FABRIC ALONG VERTICAL AND HORIZONTAL JOINTS.

STEP 3: INSTALLING SUBSEQUENT PANELS



1. REPEAT STEPS 2 AND 3 UNTIL FINISHED GRADE AT TOP OF WALL IS ACHIEVED.
2. INSTALL TOP OF WALL TREATMENT (E.G., COPING, BARRIER, ETC.) PER PROJECT SPECIFICATIONS.
3. AS SOON AS POSSIBLE, PLACE AND COMPACT BACKFILL TO ESTABLISH FINISHED GRADE AT THE FACE OF THE WALL.

STEP 4: FINALIZING WALL INSTALLATION

LEGEND

JOINT FABRIC
REINFORCED FILL
EXISTING GROUND
WEDGE
SPACER
BLOCK
LEVEL

NOTES:

1. INSTALLATION SEQUENCE SHOWN FOR ILLUSTRATIVE PURPOSES ONLY.
2. DETAILS APPLY TO 5X5, 5X9 AND 5X10 ARES PANEL INSTALLATION SIMILARLY. ARES 5X5 PANELS SHOWN FOR CLARITY.

ARES INSTALLATION SEQUENCE

NOT TO SCALE

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CHECKED BY : J. PERALTA

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REVISION / ISSUE

SHEET TITLE

STANDARD DETAIL (13 OF 13)

Plotted on February 25, 2026
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Appendix 1.1.4.c

Ares 5x10 Standard Details

ARES® MSE Retaining Wall System

Back to main document

Tensar®
A Division of CMC

ARES®
RETAINING WALL SYSTEMS

MSE STRUCTURE SHOP DRAWINGS
PREPARED FOR

ARES (5'X10') RETAINING WALL SYSTEM

INDEX	
SHEET NUMBER	DESCRIPTION
1	Title Sheet
2	Standard Detail (1 of 13)
3	Standard Detail (2 of 13)
4	Standard Detail (3 of 13)
5	Standard Detail (4 of 13)
6	Standard Detail (5 of 13)
7	Standard Detail (6 of 13)
8	Standard Detail (7 of 13)
9	Standard Detail (8 of 13)
10	Standard Detail (9 of 13)
11	Standard Detail (10 of 13)
12	Standard Detail (11 of 13)
13	Standard Detail (12 of 13)
14	Standard Detail (13 of 13)

PLOTTED ON: 6/25/2025 - 2:02pm
C:\VP_TIDEAWARES\ARES_IDEA_CRD_DETAILS\5 X 10 STANDARD DETAILS.DWG

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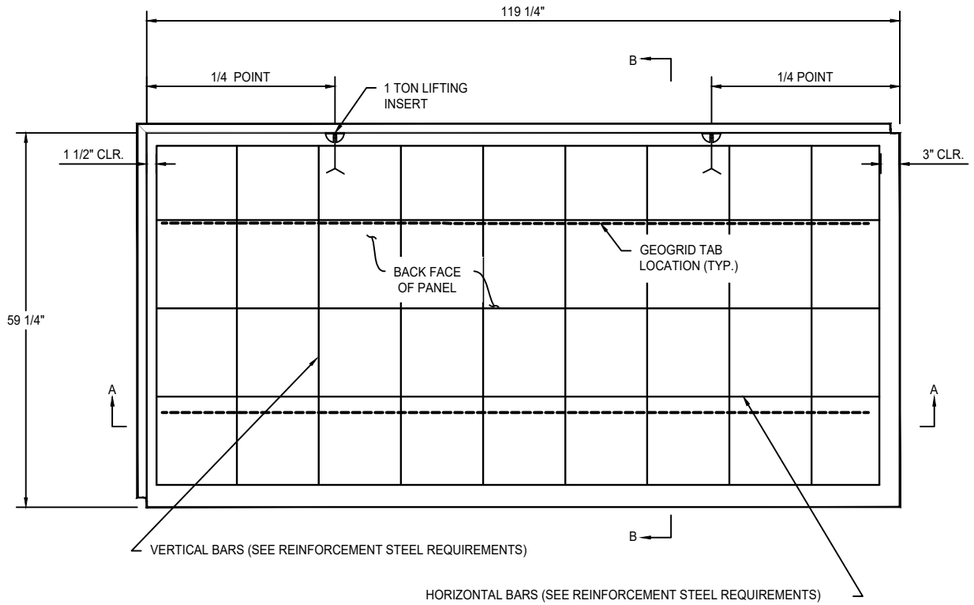
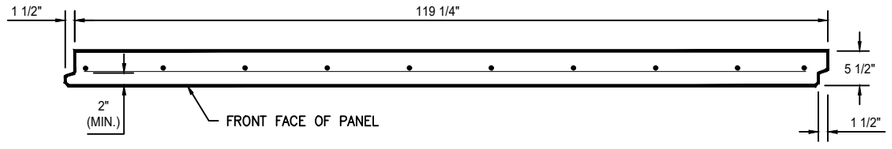
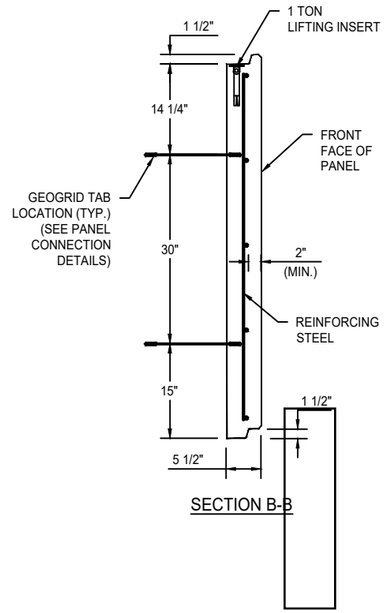
TITLE SHEET



BACK FACE VIEW



SECTION A-A



SECTION B-B

TYPICAL PANEL DETAILS - STANDARD A PANEL (REBAR REINFORCING)

NOT TO SCALE

NOTES:

1. CONCRETE SHALL HAVE A 28 DAY MINIMUM COMPRESSIVE STRENGTH OF 4,000 PSI.
2. REINFORCING STEEL REQUIREMENTS:

   REBAR SIZE AND SPACING :
   HORIZONTAL:#4 BARS @ 16" O.C. (MAX.)
   VERTICAL:#4 BARS @ 14" O.C. (MAX.)

   WIRE MESH SIZE AND SPACING :
   6"x6" - W8.5xW8.5 WELDED WIRE MESH PER ASTM A-82, FABRICATION PER ASTM A-185. (ASTM A1064)

3. REINFORCING STEEL SHALL HAVE A MINIMUM CLEAR COVER OF 1-1/2" FROM ALL PANEL EDGES UNLESS OTHERWISE NOTED.
4. PANEL TOLERANCES:
   - 4.1. OVERALL DIMENSIONS: 1/4-INCH
   - 4.2. GEOGRID LOCATIONS: 1-INCH
   - 4.3. LIFTING INSERT LOCATIONS: 1/8-INCH
   - 4.4. ELEMENT SQUARENESS: 1/2-INCH (BETWEEN DIAGONALS)
5. ALL PANELS SHOWN BACK FACE OF PANEL.
6. PANEL DIMENSIONS AND TOLERANCES SHALL BE IN ACCORDANCE WITH SUPPLEMENTED SPECIFICATIONS.

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SHEET TITLE

STANDARD DETAIL (1 OF 13)

Plotted on: May 7, 2026
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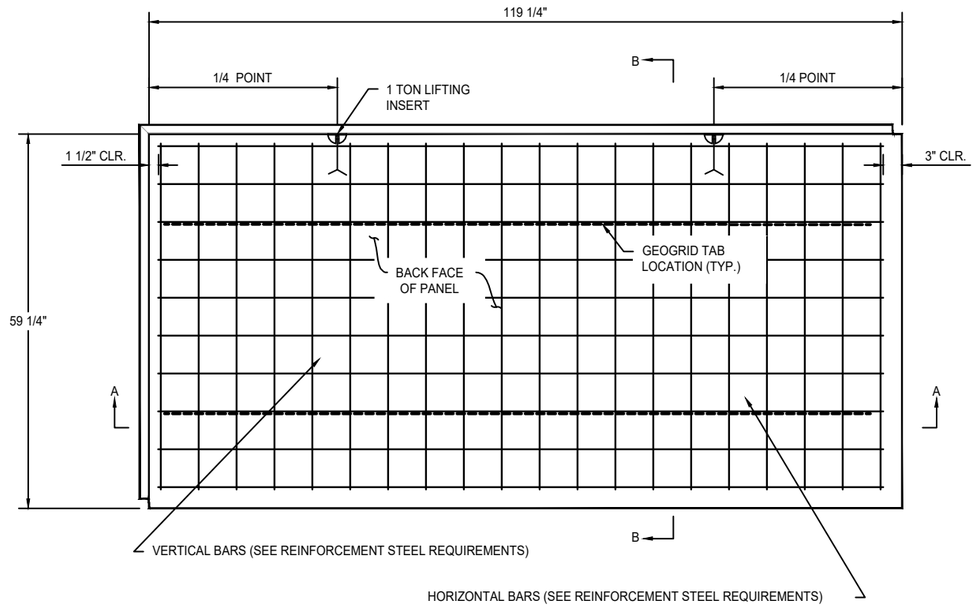
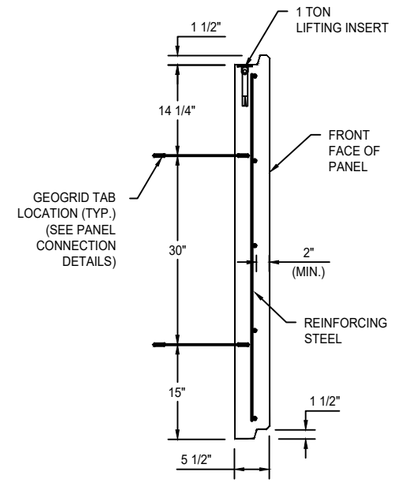
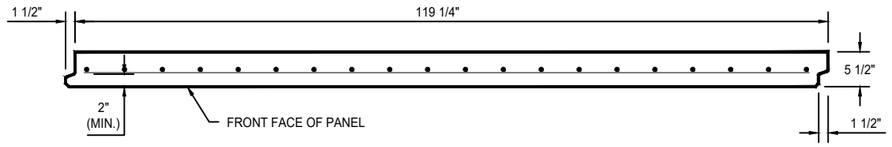


BACK FACE VIEW



SECTION B-B



SECTION A-A

TYPICAL PANEL DETAILS - STANDARD A PANEL (REBAR REINFORCING)

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   VERTICAL:#4 BARS @ 14" O.C. (MAX.)

   WIRE MESH SIZE AND SPACING :
   6"x6" - W8.5xW8.5 WELDED WIRE MESH PER ASTM A-82, FABRICATION PER ASTM A-185. (ASTM A1064)

3. REINFORCING STEEL SHALL HAVE A MINIMUM CLEAR COVER OF 1-1/2" FROM ALL PANEL EDGES UNLESS OTHERWISE NOTED.

4. PANEL TOLERANCES:
   - 4.1. OVERALL DIMENSIONS: 1/4-INCH
   - 4.2. GEOGRID LOCATIONS: 1-INCH
   - 4.3. LIFTING INSERT LOCATIONS: 1/8-INCH
   - 4.4. ELEMENT SQUARENESS: 1/2-INCH (BETWEEN DIAGONALS)

5. ALL PANELS SHOWN BACK FACE OF PANEL.

6. PANEL DIMENSIONS AND TOLERANCES SHALL BE IN ACCORDANCE WITH SUPPLEMENTED SPECIFICATIONS.

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ARES® RETAINING WALL SYSTEMS

PROJECT NAME AND LOCATION

----, ----

OWNER

OWNER PROJECT No.

CLIENT

IDEA

TIC PROJECT No. ----

DRAWN BY: V. JUAREZ
DESIGNED BY: ----
CHECKED BY : J. PERALTA

ENGINEER OF RECORD (MSE STRUCTURE ONLY):

P.E. NO.

NO.	DATE	DESCRIPTION	BY
0	06/25/2025	ISSUED FOR REVIEW	VJ

REVISION / ISSUE

SHEET TITLE

STANDARD DETAIL (2 OF 13)

Plotted on: May 7, 2026
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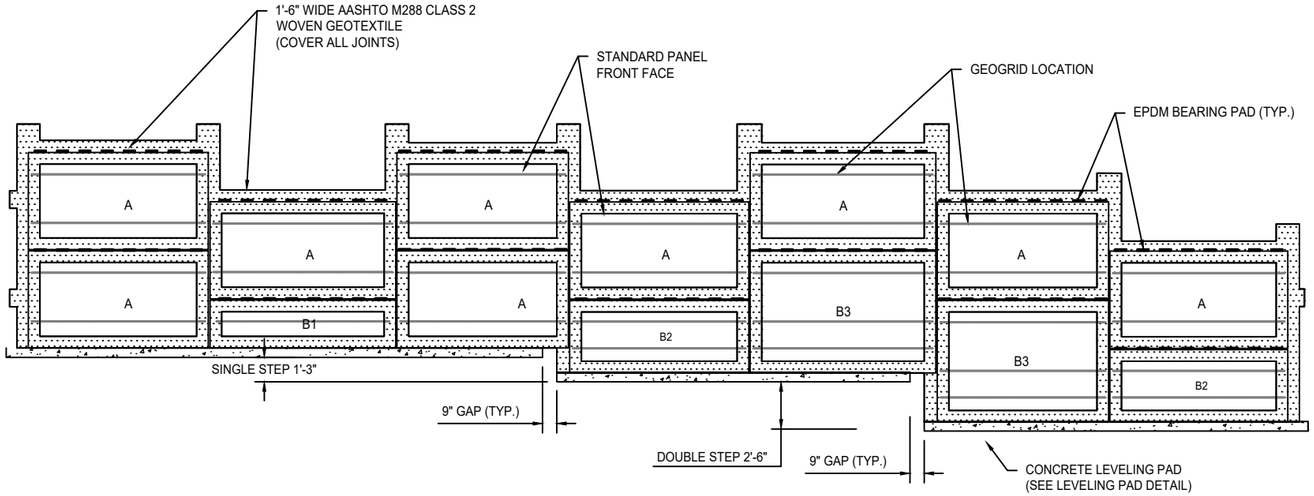
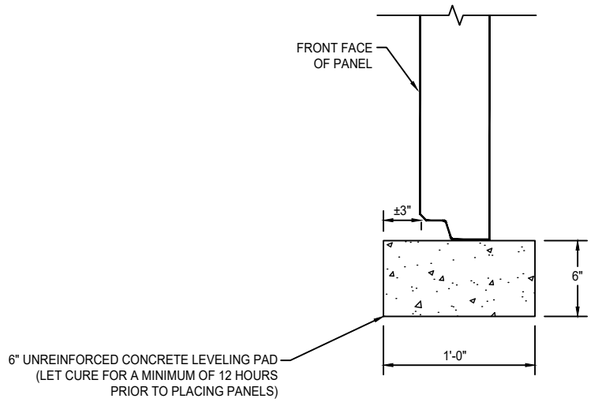


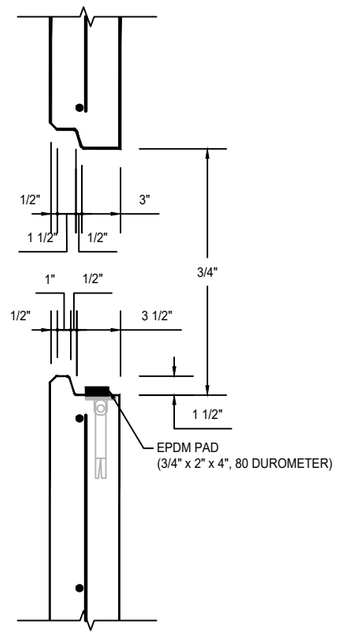
TYPICAL LEVELING PAD STEP AND GEOTEXTILE COVERAGE DETAIL

NOT TO SCALE



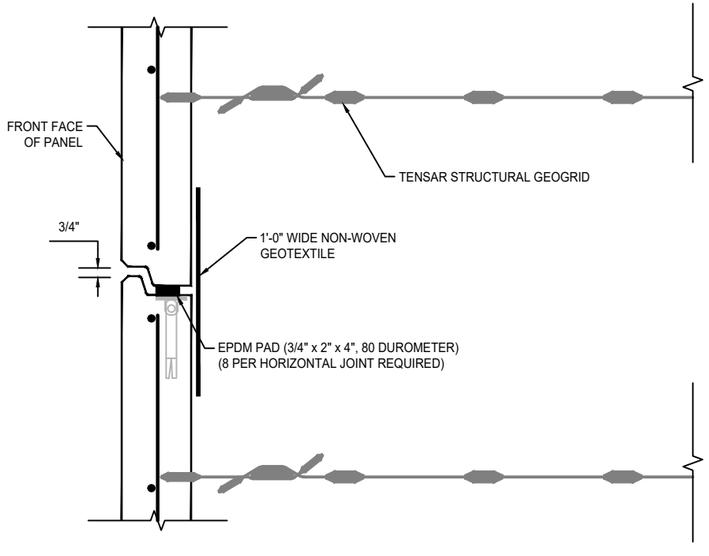
LEVELING PAD DETAIL

NOT TO SCALE



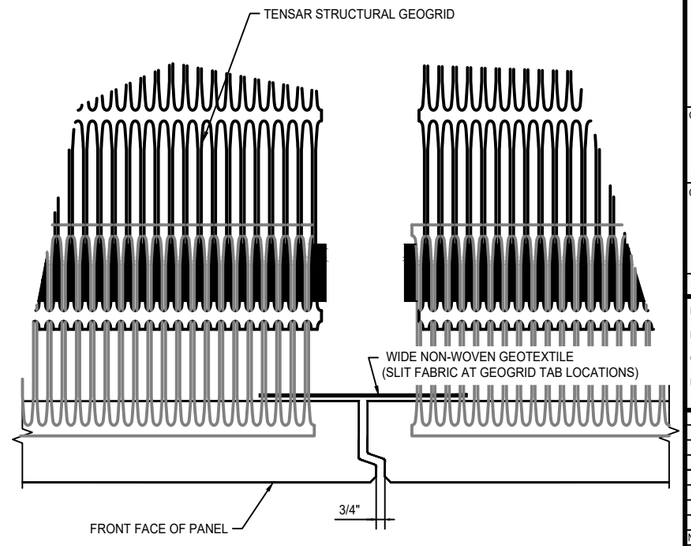
PANEL JOINT DETAIL

NOT TO SCALE



HORIZONTAL JOINT DETAIL

NOT TO SCALE



PLAN VIEW

VERTICAL JOINT DETAIL

NOT TO SCALE

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DRAWN BY: V. JUAREZ

DESIGNED BY: ----

CHECKED BY : J. PERALTA

ENGINEER OF RECORD (MSE STRUCTURE ONLY):

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NO.	DATE	DESCRIPTION	BY
0	06/25/2025	ISSUED FOR REVIEW	VJ

REVISION / ISSUE

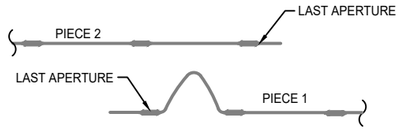
SHEET TITLE

STANDARD DETAIL (3 OF 13)

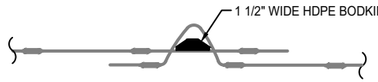
Plotted on February 25, 2026
C:\VP_IDEA\ARES\ARES IDEA.CAD DETAILS\5 X 10 STANDARD DETAILS.DWG

TO FORM A BODKIN CONNECTION:

1. BEND THE LAST APERTURE OF ONE PIECE OF GEOGRID IN HALF.



2. PASS THE BENT APERTURES THROUGH THE APERTURES OF THE SECOND PIECE OF GEOGRID AND INSERT THE BODKIN BAR INTO THE CAVITY BETWEEN THE GEOGRIDS.



3. PULL BOTH PIECES OF GEOGRID IN OPPOSITE DIRECTIONS TO COMPLETE CONNECTION.

NOTE:

THE SPLICED GEOGRID PIECE ON EITHER SIDE OF THE BODKIN CONNECTION SHALL BE AT LEAST 6 FEET LONG UNLESS THE GEOGRID TERMINATES IN A FIXED CONNECTION.

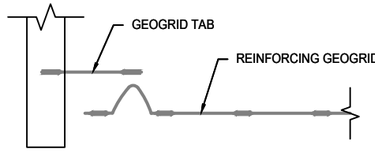
CROSS-SECTION

BODKIN CONNECTION

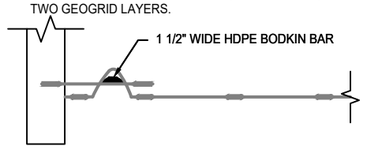
NOT TO SCALE

TO FORM A PANEL BODKIN CONNECTION FOR CONNECTING TO FACING PANEL:

1. BEND THE LAST APERTURE OF REINFORCING GEOGRID AS SHOWN.



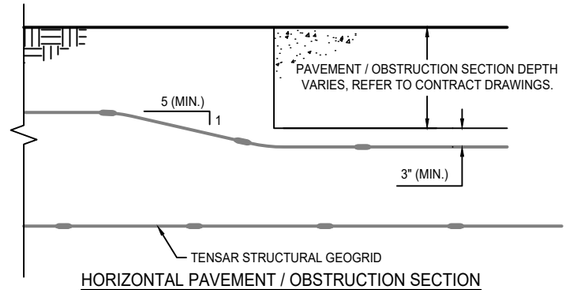
2. PASS THE RIBS OF THE BENT APERTURES THROUGH THE RIBS OF THE GEOGRID TAB AND INSERT THE BODKIN BAR INTO THE SPACE BETWEEN THE TWO GEOGRID LAYERS.



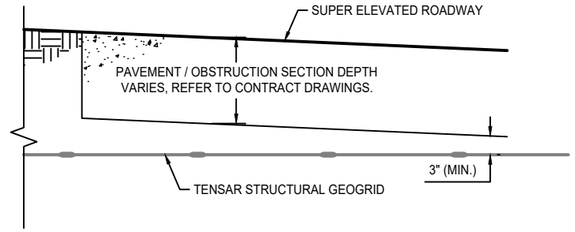
3. PULL REINFORCING GEOGRID TAUT TO TENSION CONNECTION.

PANEL BODKIN CONNECTION

NOT TO SCALE



HORIZONTAL PAVEMENT / OBSTRUCTION SECTION



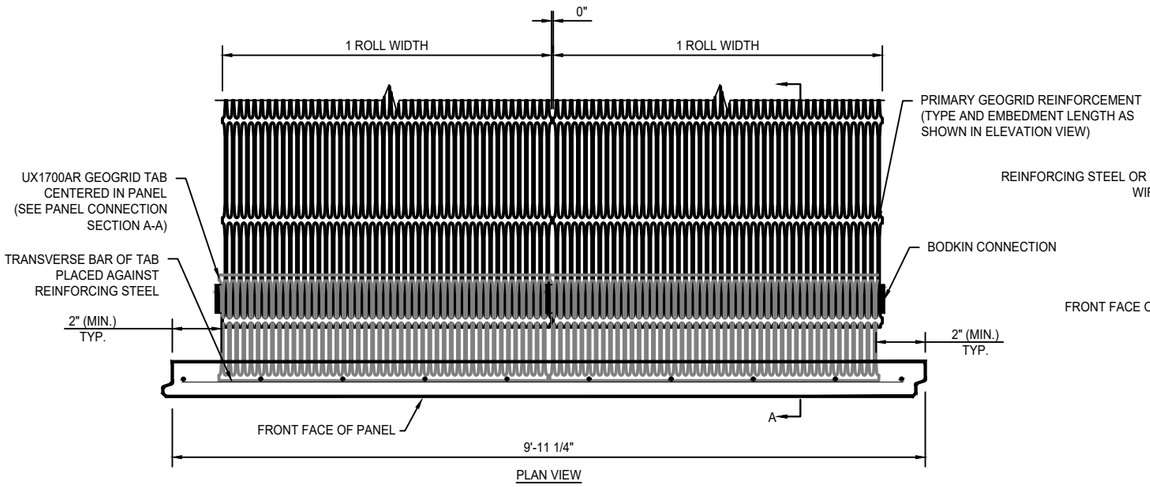
SUPERELEVATED PAVEMENT / OBSTRUCTION SECTION

NOTE:

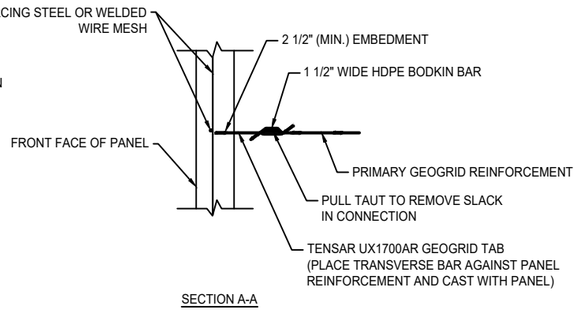
CONTRACTOR IS RESPONSIBLE TO COORDINATE THE PLACEMENT OF THE GEOGRID TO AVOID CONFLICT WITH THE CONTRACT PAVEMENT / OBSTRUCTION SECTION. GEOGRID MUST BE SEPARATED FROM THE PAVEMENT / OBSTRUCTION SECTION BY A MINIMUM OF 3".

GEOGRID PLACEMENT AT PAVEMENT / OBSTRUCTION SECTION

NOT TO SCALE



PLAN VIEW



SECTION A-A

CONNECTION DETAIL PLAN VIEW (90% COVERAGE)

NOT TO SCALE

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ARES® RETAINING WALL SYSTEMS

PROJECT NAME AND LOCATION

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DESIGNED BY: ----

CHECKED BY : J. PERALTA

ENGINEER OF RECORD (MSE STRUCTURE ONLY):

---- P.E. NO.

NO.	DATE	DESCRIPTION	BY
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REVISION / ISSUE

SHEET TITLE

STANDARD DETAIL (4 OF 13)

SHEET 5 OF 14

Plotted on February 25, 2026
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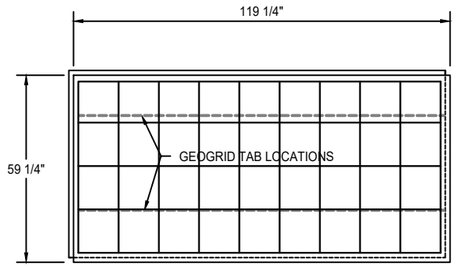
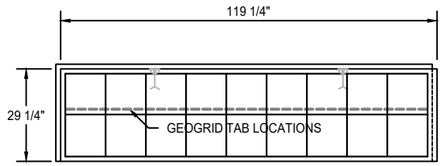
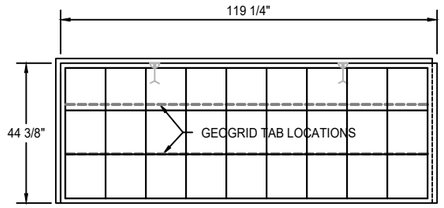


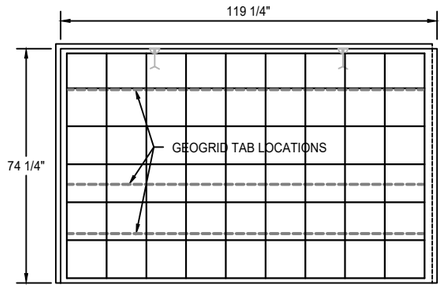
STANDARD A PANEL



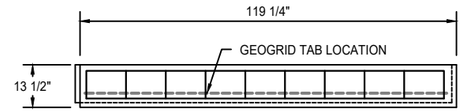
STANDARD B1 PANEL



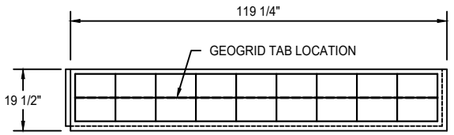
STANDARD B2 PANEL



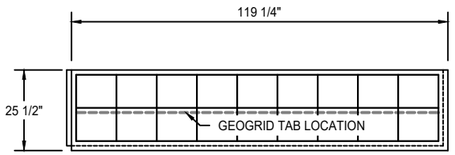
STANDARD B3 PANEL



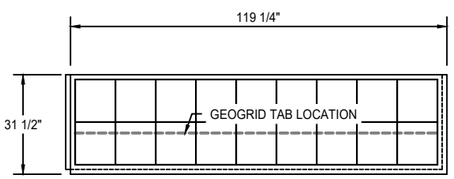
STANDARD T12 PANEL



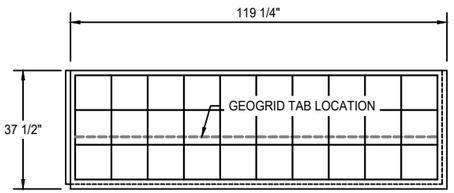
STANDARD T18 PANEL



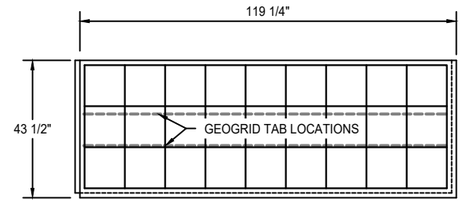
STANDARD T24 PANEL



STANDARD T30 PANEL



STANDARD T36 PANEL



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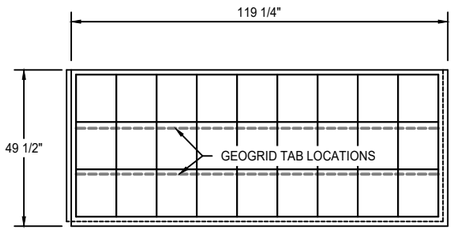
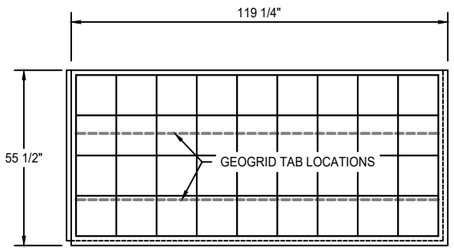
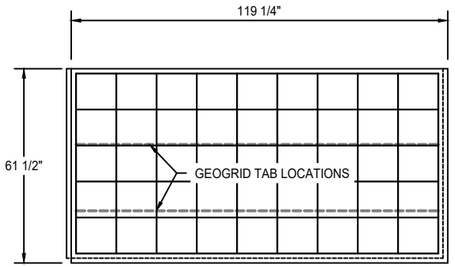


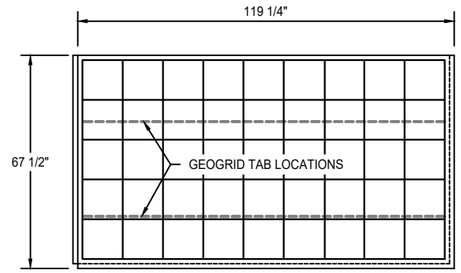
STANDARD T48 PANEL



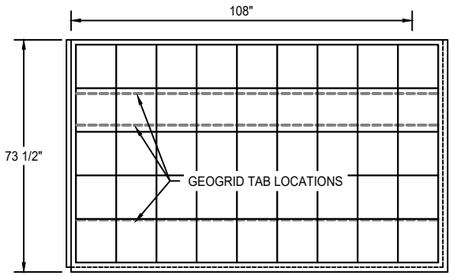
STANDARD T54 PANEL



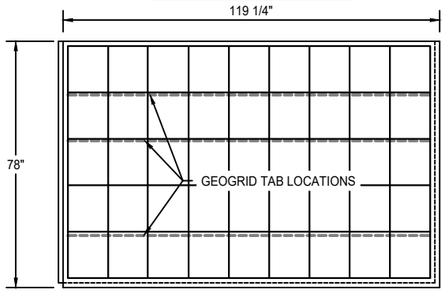
STANDARD T60 PANEL



STANDARD T66 PANEL



STANDARD T72 PANEL



STANDARD T78 PANEL

ALL PANELS ARE SHOWN BACK FACE VIEW

NOTES:

1. PANEL CONCRETE REQUIREMENTS:
   - 1.1. PRECAST CONCRETE PANEL SHALL CONFORM TO THE REQUIREMENTS OF SECTION 701, 703, 705 AND 709 OF ODOT STANDARD SPECIFICATIONS.
   - 1.2. CONCRETE SHALL HAVE A 28-DAY MINIMUM COMPRESSIVE STRENGTH OF 4,000 PSI.
2. PANEL REINFORCING STEEL REQUIREMENTS:
   - 2.1. ALL REINFORCING STEEL SHALL HAVE A MINIMUM CLEAR COVER OF 2" FROM ALL PANEL EDGES UNLESS OTHERWISE NOTED.
   - 2.2. REBAR SIZE AND SPACING REQUIREMENTS:
     - 2.2.1. HORIZONTAL: #4 BARS AT 16" O.C. (MAX.)
     - 2.2.2. VERTICAL: #4 BARS AT 14" O.C. (MAX.)
   - 2.3. WIRE MESH SIZE AND SPACING REQUIREMENTS
     - 24.1 WELDED WIRE MESH 6"x6" - W8.5xW8.5
   - 2.5. PANEL DIMENSIONS AND TOLERANCES SHALL BE IN ACCORDANCE WITH SUPPLEMENTED SPECIFICATIONS.

STANDARD 5' X 10' PANELS

NOT TO SCALE

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ARES® RETAINING WALL SYSTEMS

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DRAWN BY: V. JUAREZ
DESIGNED BY: ----
CHECKED BY : J. PERALTA

ENGINEER OF RECORD (MSE STRUCTURE ONLY):
---- P.E. NO.

NO.	DATE	DESCRIPTION	BY
0	06/25/2025	ISSUED FOR REVIEW	VJ

REVISION / ISSUE

SHEET TITLE

STANDARD DETAIL (5 OF 13)

SHEET 6 OF 14

Plotted on February 25, 2026
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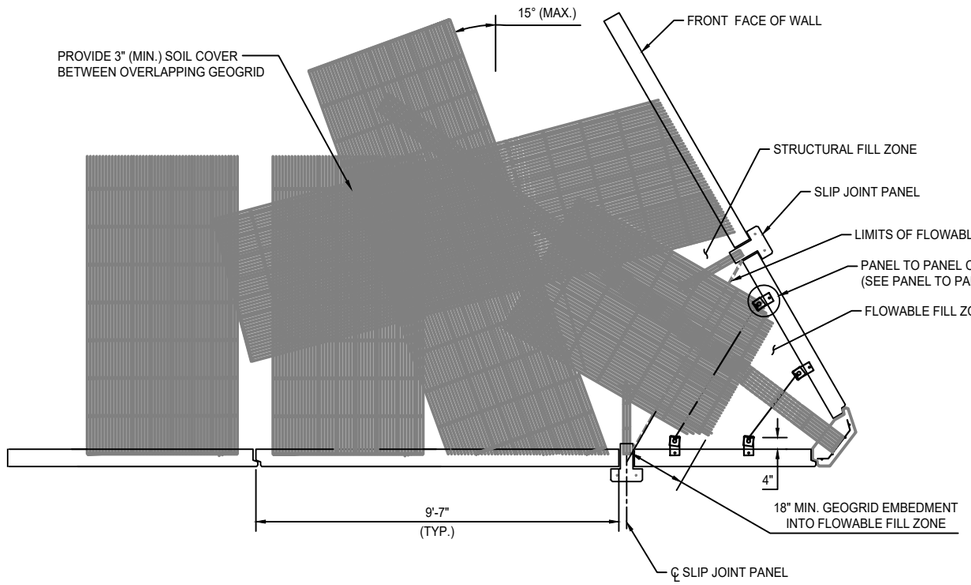
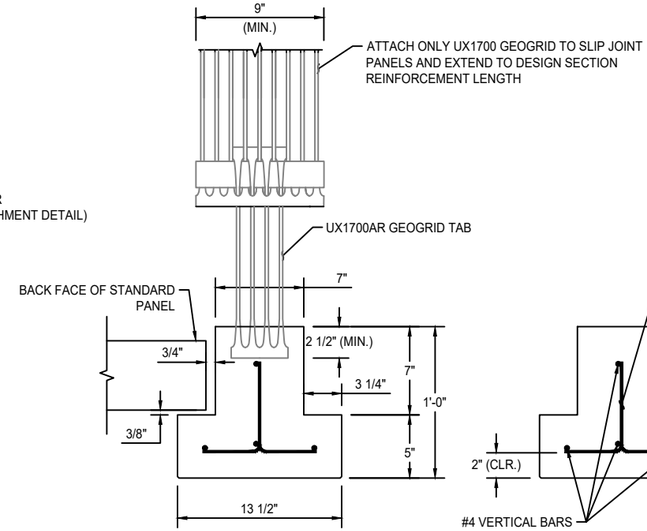


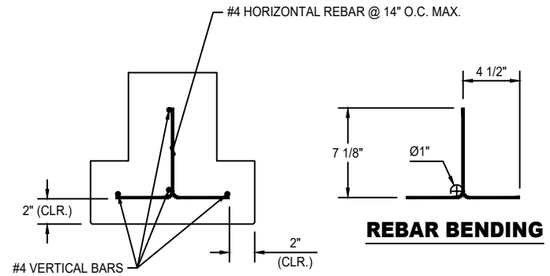
ACUTE CORNER LESS THAN 70°

NOT TO SCALE

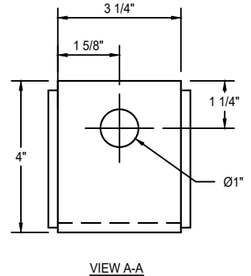


SLIP JOINT DETAIL

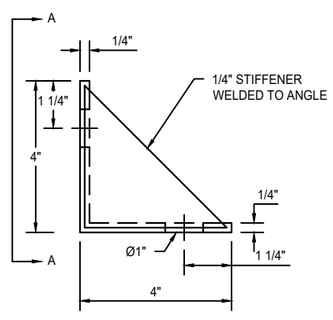
NOT TO SCALE



REBAR BENDING

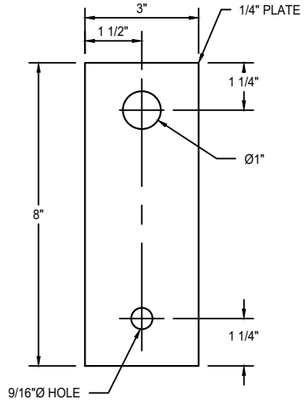


VIEW A-A



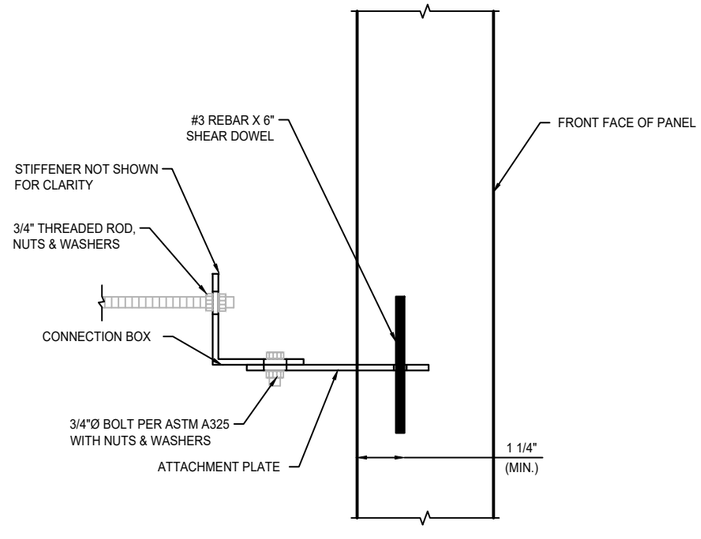
CONNECTION BOX

NOT TO SCALE



ATTACHMENT PLATE

NOT TO SCALE



PANEL TO PANEL ATTACHMENT

FABRICATION ATTACHMENT STEEL NOTES:

1. ALL STEEL ANGLES AND PLATES SHALL BE PER ASTM A36.
2. ALL FABRICATED STEEL PARTS SHALL BE HOT DIP GALVANIZED AFTER FABRICATION.
3. ALL DIMENSIONS ARE MINIMUM REQUIRED.

Plotted on February 25, 2026
C:\VF_IDEA\ARES\ARES IDEA.CAD DETAILS\5 X 10 STANDARD DETAILS.DWG

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PROJECT NAME AND LOCATION

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OWNER PROJECT No.

CLIENT

IDEA

TIC PROJECT No. ----

DRAWN BY: V. JUAREZ

DESIGNED BY: ----

CHECKED BY : J. PERALTA

ENGINEER OF RECORD (MSE STRUCTURE ONLY):

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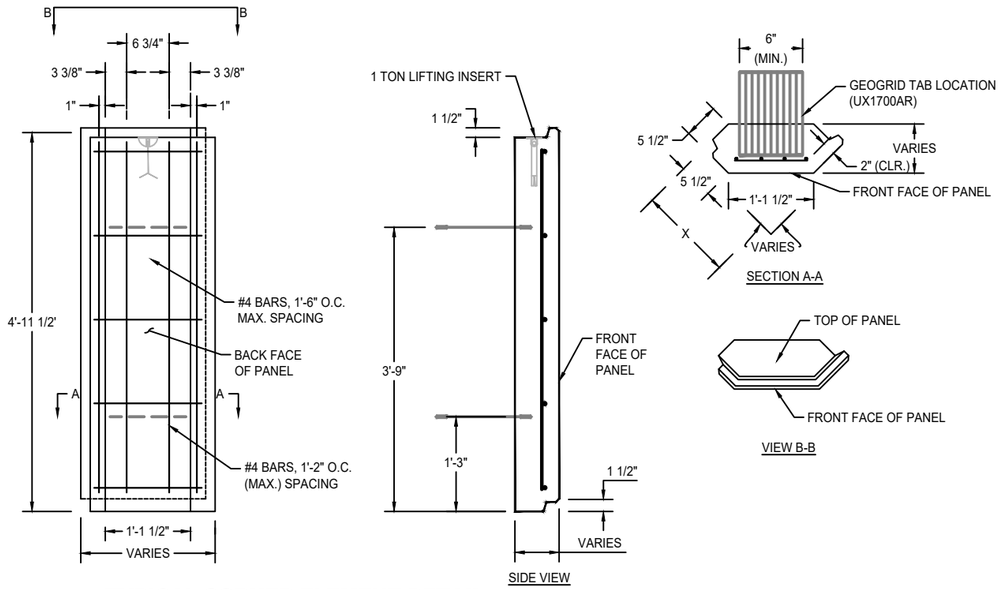
REVISION / ISSUE

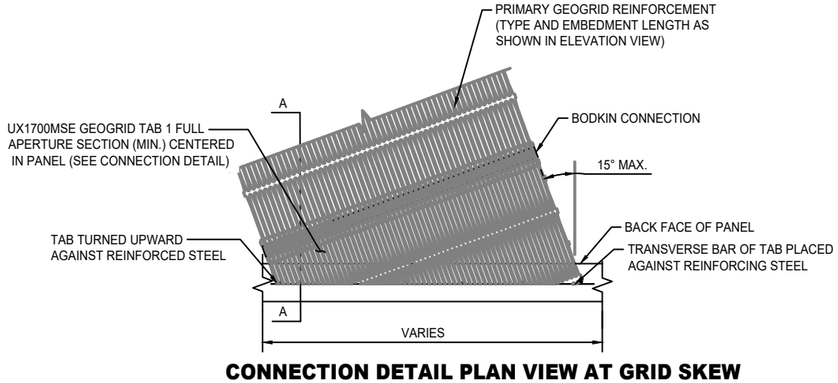
SHEET TITLE

STANDARD DETAIL (6 OF 13)



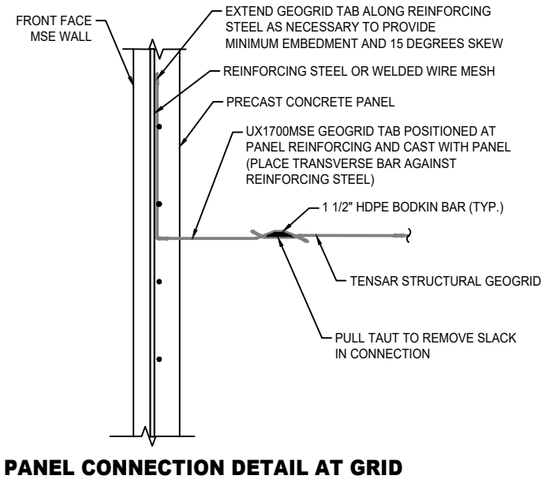
TYPICAL CORNER ELEMENT DETAIL

NOT TO SCALE



CONNECTION DETAIL PLAN VIEW AT GRID SKEW

NOT TO SCALE



PANEL CONNECTION DETAIL AT GRID SKEW (SECTION A-A)

NOT TO SCALE

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SHEET TITLE

STANDARD DETAIL (7 OF 13)

SHEET 8 OF 14

Plotted on February 25, 2026
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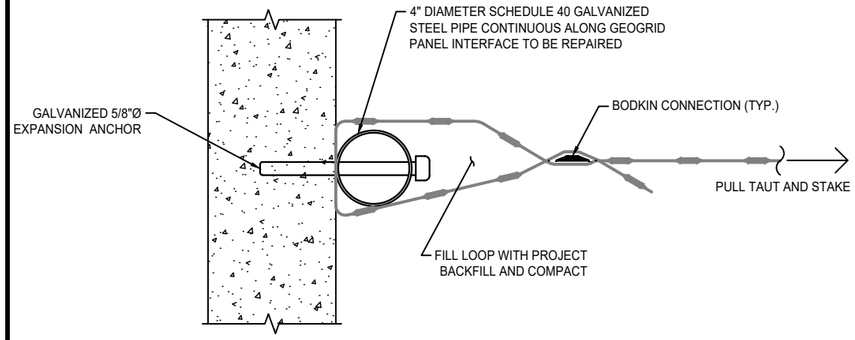
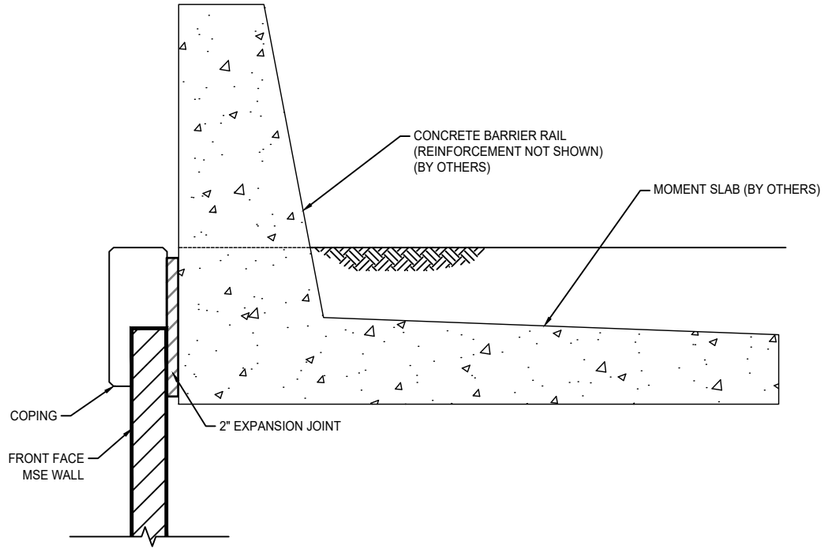
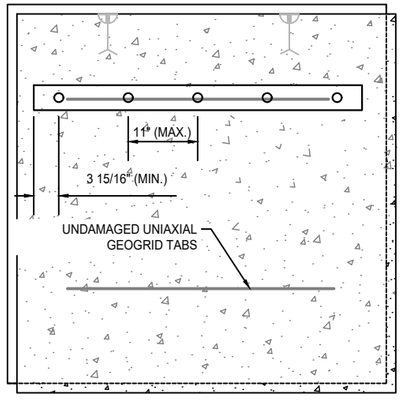


SECTION VIEW



CONCRETE BARRIER RAIL WITH MOMENT SLAB ON MSE WALL

NOT TO SCALE



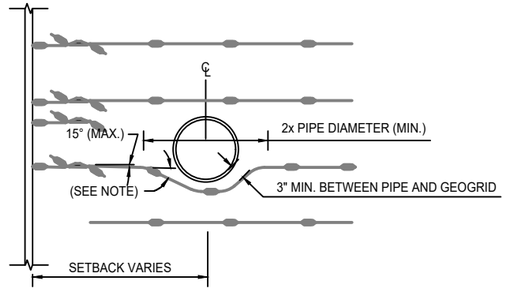
ELEVATION VIEW

NOTE:

ONLY DAMAGED SECTIONS NEED TO BE REPAIRED.
MINIMUM SECTION LENGTH = 11" - 2 BOLTS MINIMUM
REQUIRED PER SECTION.

REPAIR OF DAMAGED GEOGRID CONNECTION

NOT TO SCALE

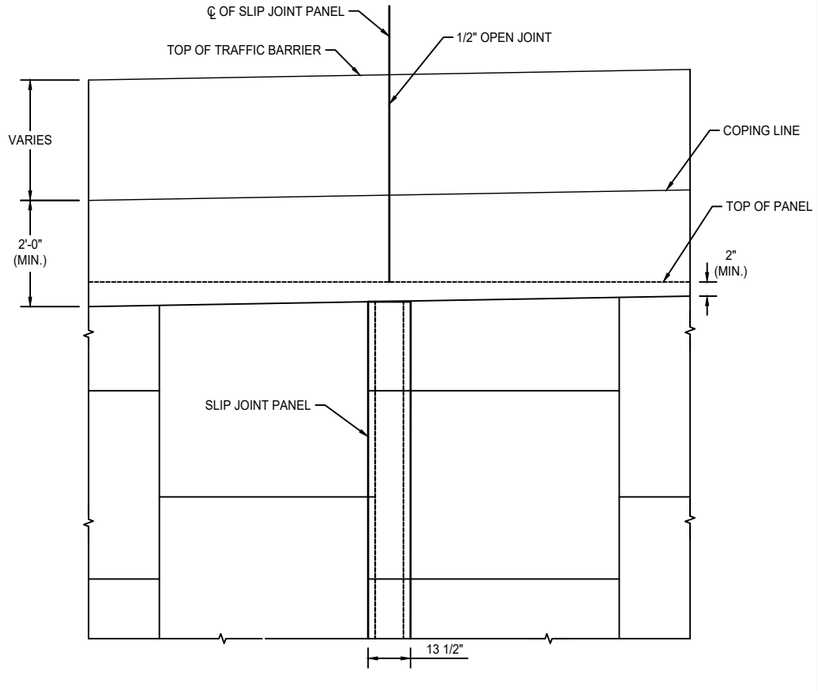


NOTE:

IF GEOGRID MUST BE MORE THAN 15° FROM
HORIZONTAL, EXTEND GEOGRID 4'-0" LONGER
THAN SHOWN ON ELEVATION VIEW.

TYPICAL OBSTRUCTION AVOIDANCE DETAIL

NOT TO SCALE



C.I.P. TRAFFIC BARRIER OVER SLIP JOINT PANEL

NOT TO SCALE

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SHEET TITLE

STANDARD DETAIL (8 OF 13)

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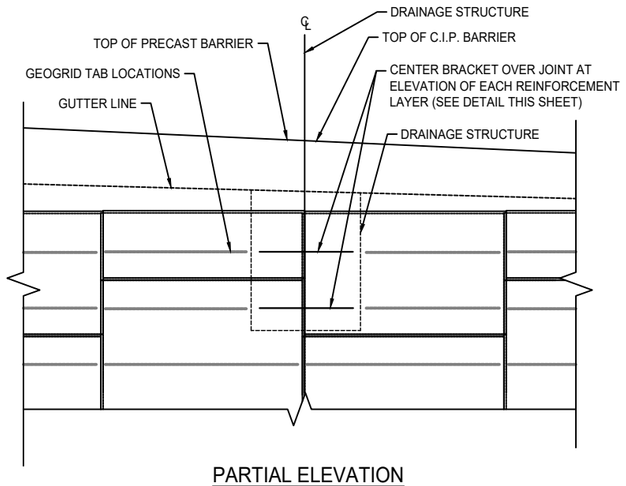
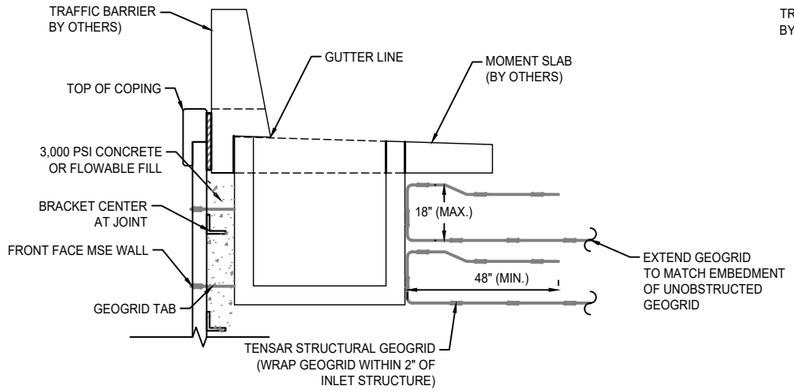
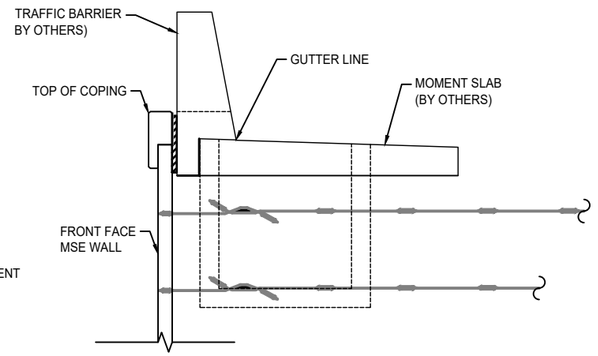


PARTIAL ELEVATION



SECTION A-A



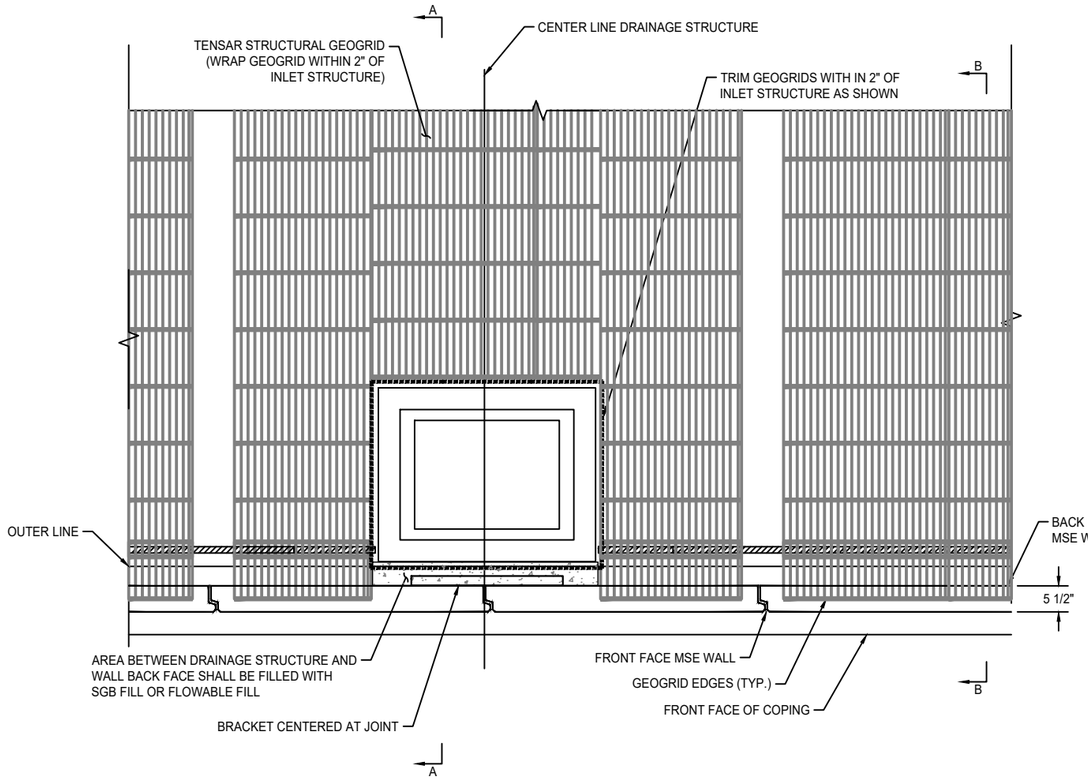
SECTION B-B

NOTES:

1. ALL FABRICATED STEEL PARTS SHALL BE HOT DIP GALVANIZED AFTER FABRICATION.
2. ANCHOR SHALL BE Ø 5/8" KIWIK BOLT 3 HOT-DIP GALVANIZED (HDG) EXPANSION ANCHOR OR APPROVED EQUAL.

BRACKET CENTERED OVER JOINT DETAIL

NOT TO SCALE



PARTIAL PLAN

MSE WALL AT DRAINAGE INLET

NOT TO SCALE

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DESIGNED BY: ----

CHECKED BY : J. PERALTA

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STANDARD DETAIL (9 OF 13)

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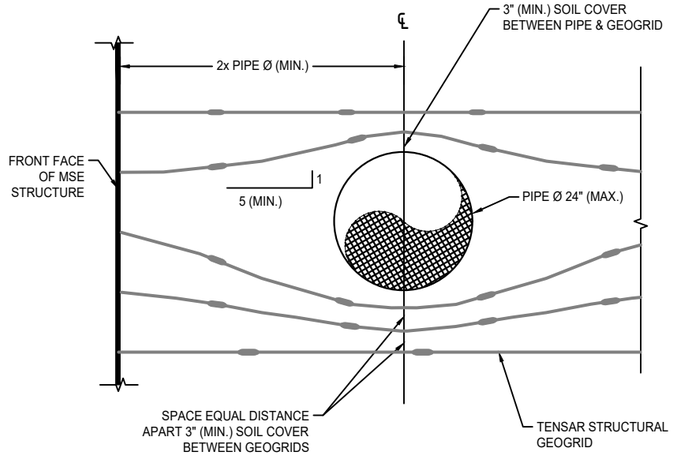
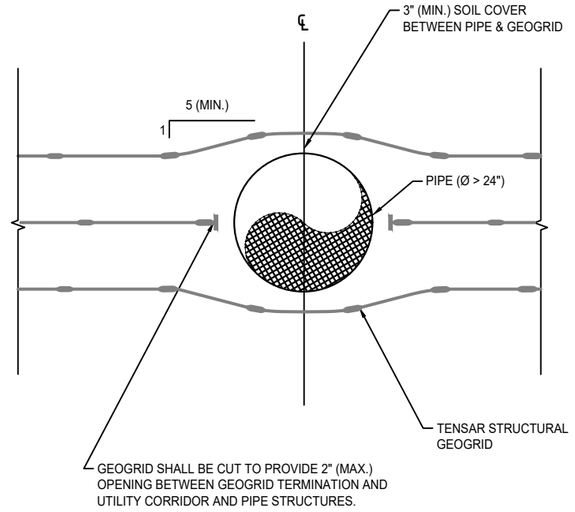


CROSS-SECTION

GEOGRID PLACEMENT AT PIPE PARALLEL TO MSE STRUCTURE FACE

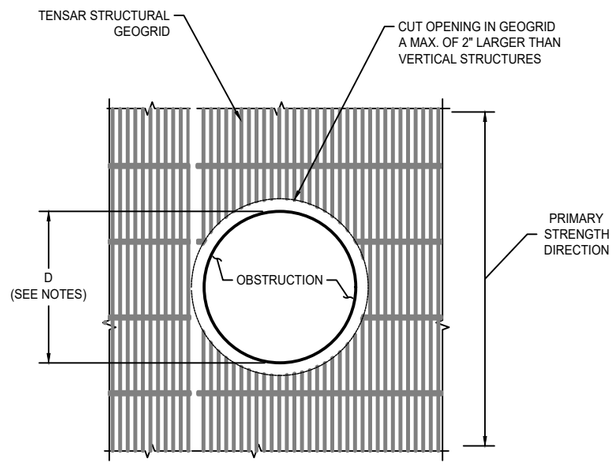
NOT TO SCALE



ELEVATION VIEW

GEOGRID PLACEMENT AT PIPE PERPENDICULAR TO MSE STRUCTURE FACE

NOT TO SCALE



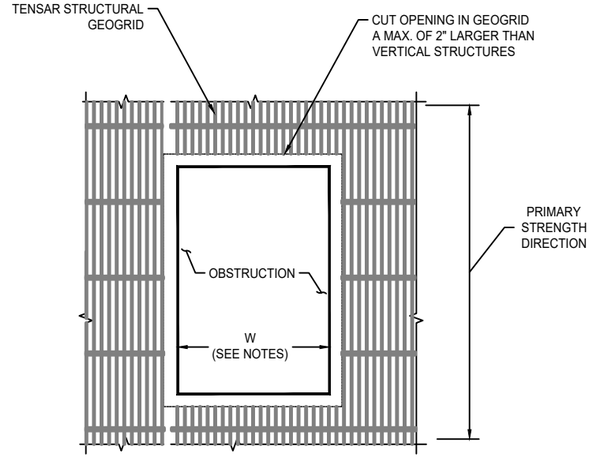
PLAN VIEW

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



PLAN VIEW

NOTES:

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

GEOGRID PENETRATION DETAIL-PLAN VIEW

NOT TO SCALE

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SHEET TITLE

STANDARD DETAIL (10 OF 13)

Plotted on February 25, 2026
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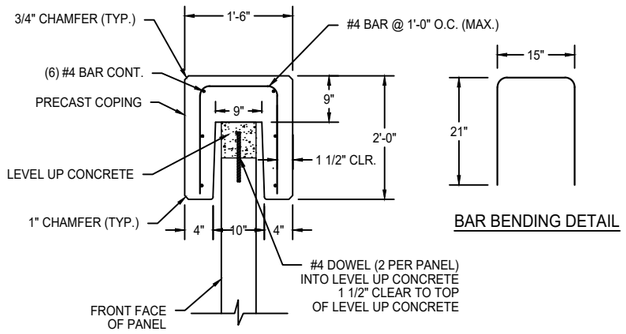
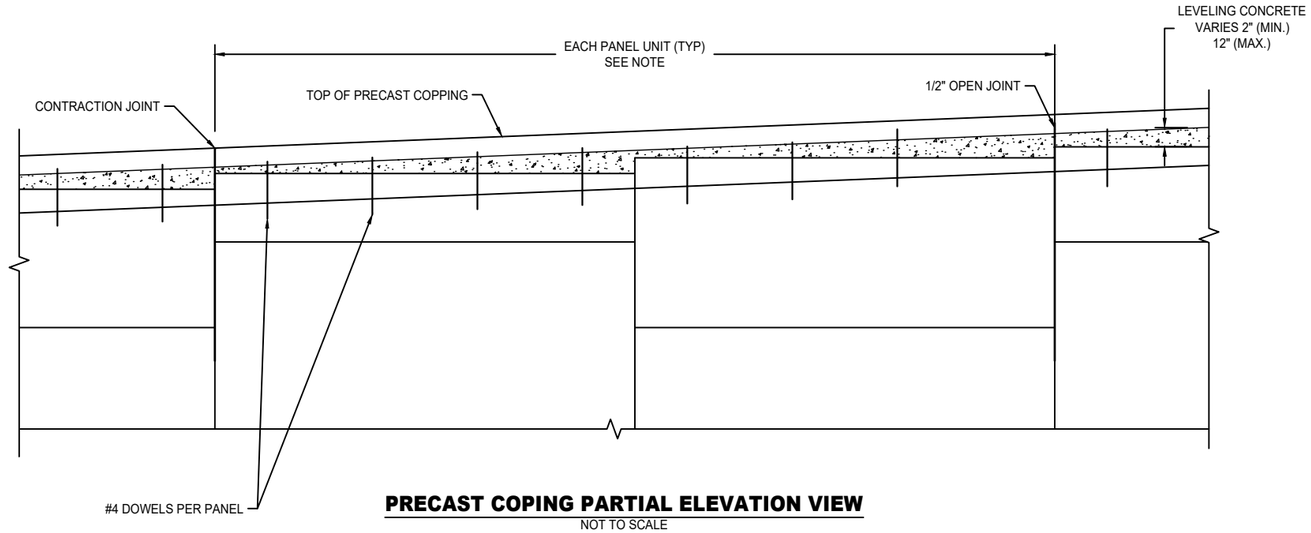


PRECAST COPING SECTION

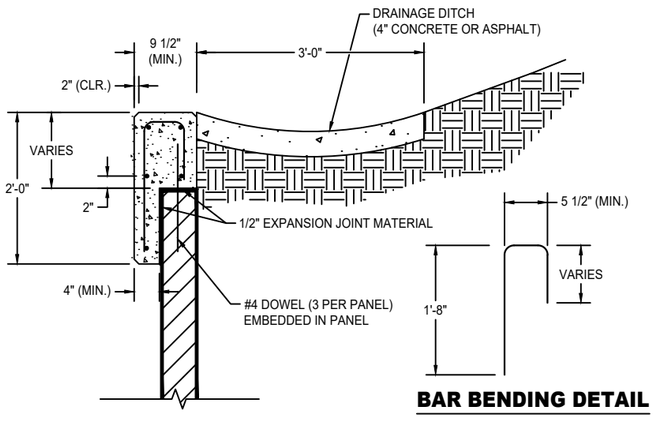
NOT TO SCALE

NOTE:
EXPANSION JOINTS IN PRECAST COPING SHALL BE AT 4 PANEL INTERVALS. COPING JOINTS MUST COINCIDE WITH PANEL JOINTS ON FRONT FACE AND SHALL BE PERPENDICULAR TO SLOPE LINE. REINFORCING STEEL SHALL BE STOPPED 2" SHORT OF EITHER SIDE OF THE EXPANSION JOINT. CONTRACTIONS JOINTS IN BETWEEN THE EXPANSION JOINTS SHALL BE PLACED TWO PANELS FROM EXPANSION JOINT.



PRECAST COPING PARTIAL ELEVATION VIEW

NOT TO SCALE

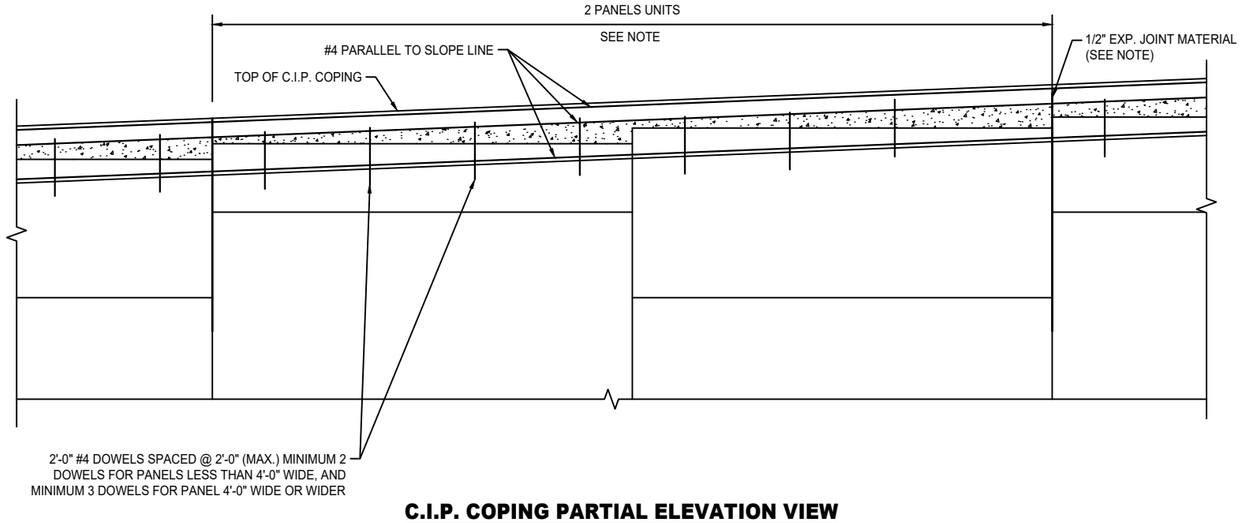


BAR BENDING DETAIL

C.I.P COPING WITH SWALE

NOT TO SCALE

NOTE:
EXPANSION JOINTS IN C.I.P COPING SHALL BE AT 2 PANEL INTERVALS (NO MORE THAN 20'-0"). COPING JOINTS MUST COINCIDE WITH PANEL JOINTS ON FRONT FACE AND 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

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STANDARD DETAIL (11 OF 13)

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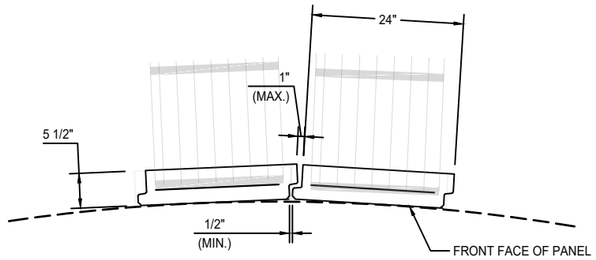
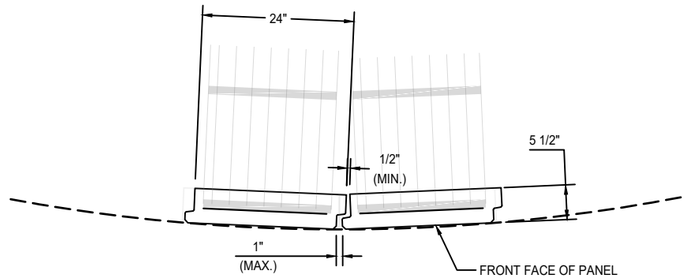
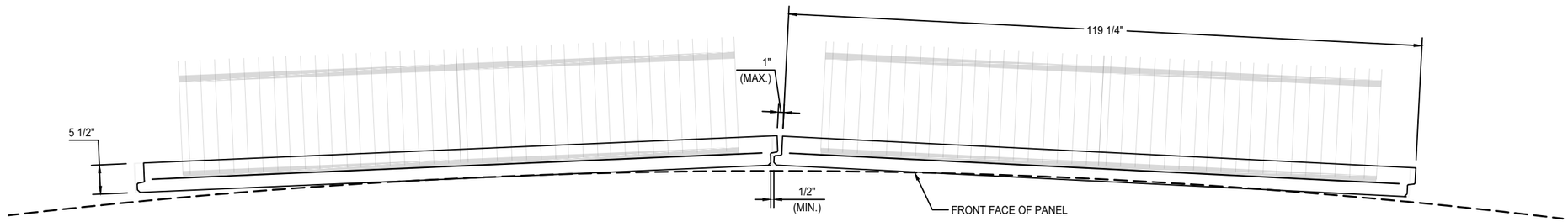


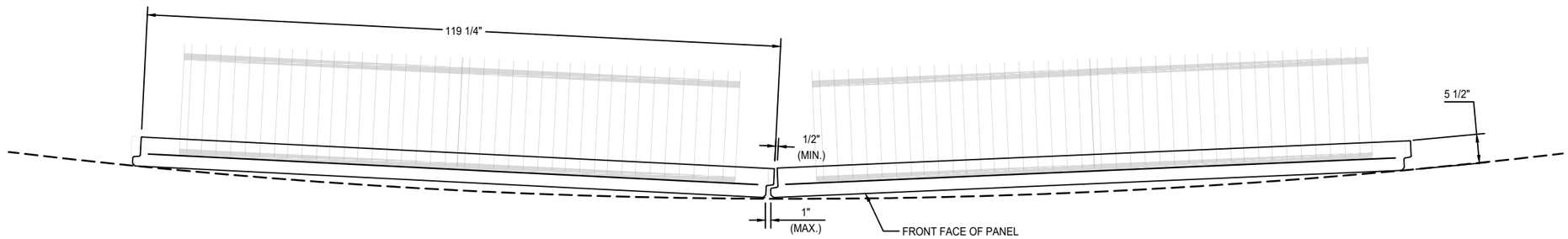
24" MIN. NARROW PANEL
INSIDE RADII
NOT TO SCALE



24" MIN. NARROW PANEL
OUTSIDE RADII
NOT TO SCALE



5X10 INSIDE RADII
NOT TO SCALE



5X10 OUTSIDE RADII
NOT TO SCALE

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CHECKED BY : J. PERALTA
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SHEET TITLE
STANDARD DETAIL (12 OF 13)

SHEET 13 OF 14

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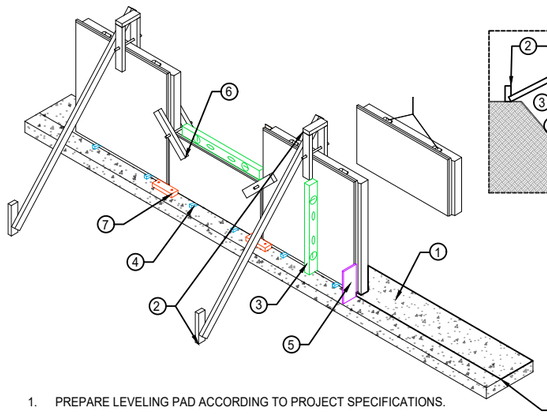
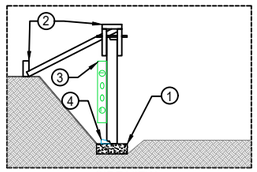
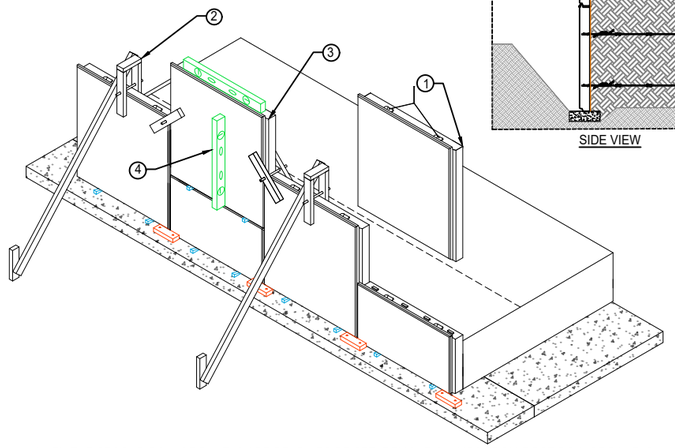


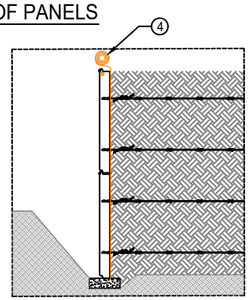


SIDE VIEW

1. PREPARE LEVELING PAD ACCORDING TO PROJECT SPECIFICATIONS.
2. THE FIRST ROW OF PANELS ARE PLACED ON THE LEVELING PAD AND BRACED.
3. THE PANELS SHOULD BE SET ACCORDING TO THE PANEL BATTER RECOMMENDATION.
4. DRIVE WEDGES AT THE QUARTER POINTS OF THE BOTTOM FRONT OF THE PANELS TO MAINTAIN BATTER.
5. USE A 3/4 IN. (19 MM) WOODEN SPACER TO ACHIEVE 3/4 IN. (19 MM) VERTICAL JOINTS.
6. ADJACENT HALF PANELS SHOULD BE CLAMPED TOGETHER.
7. AS AN OPTIONAL SOLUTION, NAIL 2X4 WOODEN BLOCKS AT THE JOINT OF THE PANELS TO PROVIDE ADDITIONAL SUPPORT.

STEP 1: INSTALLING FIRST ROW OF PANELS

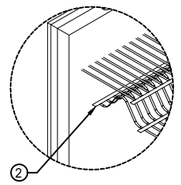


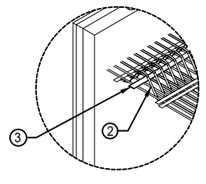


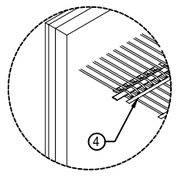
SIDE VIEW

1. WHEN THE SECOND ROW OF PANELS ARE READY TO BE SET, REMOVE CLAMPS FROM FIRST ROW OF PANELS.
2. PLACE THE SECOND ROW OF PANELS IN THE "WINDOW" CREATED BY THE FIRST ROW OF PANELS ONLY AFTER THE BACKFILL HAS REACHED 6 IN. BELOW THE HALF PANELS.
3. CHECK WALL BATTER AND ADJUST AS NEEDED. INSTALL WEDGES AT QUARTER POINTS AND CLAMP SECOND ROW OF PANELS TO THE ADJACENT/FIRST ROW OF PANELS.
4. CONTINUE INSTALLATION PF GEOTEXTILE FILTER FABRIC ALONG VERTICAL AND HORIZONTAL JOINTS.

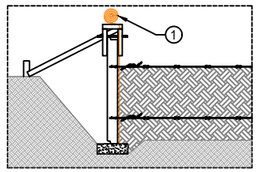
STEP 3: INSTALLING SUBSEQUENT PANELS



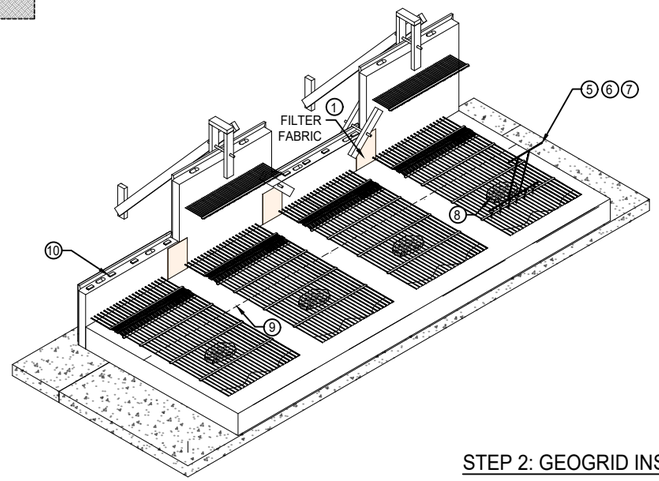




GEOGRID INSTALLATION/BODKIN CONNECTION (STEPS 1-3)

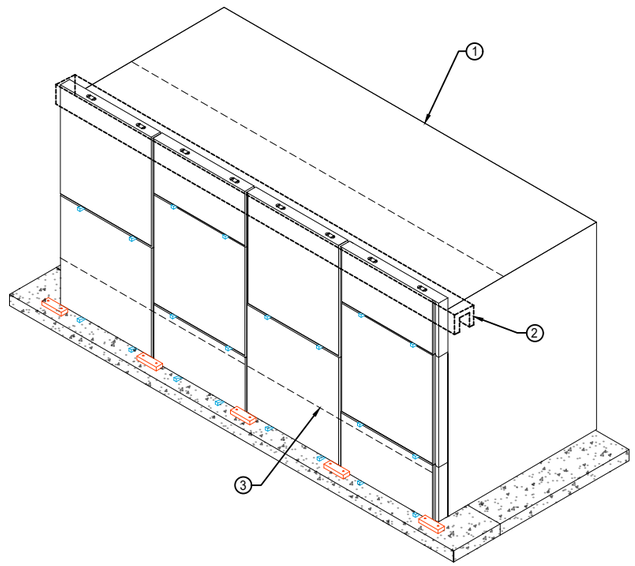


SIDE VIEW



1. PLACE GEOTEXTILE FILTER FABRIC CENTERED ON VERTICAL AND HORIZONTAL JOINTS. SECURE TO PANEL WITH ADHESIVE.
2. BEND THE GEOGRID ROLL SUCH THAT RIBS FORM A TUNNEL WITH THE EMBEDDED GEOGRID TAB.
3. SLIDE THE BODKIN BAR THROUGH THE TUNNEL FORMED BY THE GEOGRID ROLL AND THE GEOGRID TAB.
4. PULL THE GEOGRID ROLL SNUG BY HAND.
5. WHEN THE FILL IS READY TO BE PLACED, USE A TENSIONING DEVICE TO TIGHTEN THE CONNECTION.
6. PLACE THE DEVICE AT LEAST 6' FROM THE BACK OF THE PANEL, ENSURING TO ENGAGE THE TRANSVERSE BAR ACROSS THE WIDTH OF THE GEOGRID.
7. TENSION THE GEOGRID EVENLY SO THAT THE BODKIN CONNECTION IS TAUT FOR ITS ENTIRE WIDTH.
8. PLACE AT LEAST 6 IN. (150 MM) OF SELECTED GRANULAR BACKFILL.
9. THE BACKFILL IS PLACED PARALLEL TO THE WALL STARTING APPROXIMATELY 3 FT (900 MM) FROM THE BACK OF THE PANELS. THE FILL IS THEN SPREAD TOWARD THE GEOGRID ENDS.
10. PLACE THE BEARING PADS EQUALLY SPACED ALONG EACH HORIZONTAL JOINT BETWEEN PANELS.

STEP 2: GEOGRID INSTALLATION, JOINT FABRIC & BACKFILL PLACEMENT



1. REPEAT STEPS 2 AND 3 UNTIL FINISHED GRADE AT TOP OF WALL IS ACHIEVED.
2. INSTALL TOP OF WALL TREATMENT (E.G., COPING, BARRIER, ETC.) PER PROJECT SPECIFICATIONS.
3. AS SOON AS POSSIBLE, PLACE AND COMPACT BACKFILL TO ESTABLISH FINISHED GRADE AT THE FACE OF THE WALL.

STEP 4: FINALIZING WALL INSTALLATION

ARES INSTALLATION SEQUENCE

NOT TO SCALE

LEGEND

JOINT FABRIC	WEDGE
REINFORCED FILL	SPACER
EXISTING GROUND	BLOCK
	LEVEL

NOTES:

1. INSTALLATION SEQUENCE SHOWN FOR ILLUSTRATIVE PURPOSES ONLY.
2. DETAILS APPLY TO 5X5, 5X9 AND 5X10 ARES PANEL INSTALLATION SIMILARLY. ARES 5X5 PANELS SHOWN FOR CLARITY.

Plotted on February 25, 2025
C:\WP_VUE\ARES\ARES_IDEA_LRD DETAILS\5 X 10 STANDARD DETAILS.DWG

Tensar.
A Division of CMC

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

C.O.A. NO.

THIS PRELIMINARY DRAWING, DESIGN NOTES AND ASSOCIATED CALCULATIONS HAVE BEEN PREPARED BY TENSAR INTERNATIONAL CORPORATION FOR PRELIMINARY DESIGN PURPOSES ONLY AND SHALL NOT BE USED OR RELIED UPON FOR FINAL DESIGN OR CONSTRUCTION.

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PROJECT NAME AND LOCATION

----, ----

OWNER

OWNER PROJECT No.

CLIENT
IDEA

TIC PROJECT No. ----

DRAWN BY: V. JUAREZ
DESIGNED BY: ----
CHECKED BY : J. PERALTA
ENGINEER OF RECORD (MSE STRUCTURE ONLY):
---- P.E. NO.

NO.	DATE	DESCRIPTION	BY
0	06/25/2025	ISSUED FOR REVIEW	VJ

REVISION / ISSUE

SHEET TITLE
STANDARD DETAIL (13 OF 13)

Appendix 1.1.8.a

Bearing Pad Design

ARES® MSE Retaining Wall System

Back to main document

Bearing Pad Calculations: Date: April 2025

80 Durometer Bearing Pad
Size: 2" x 4" x 0.815"
Panel Size = 5' x 5'

Example calcs: at equivalent wall height pressure = 40ft

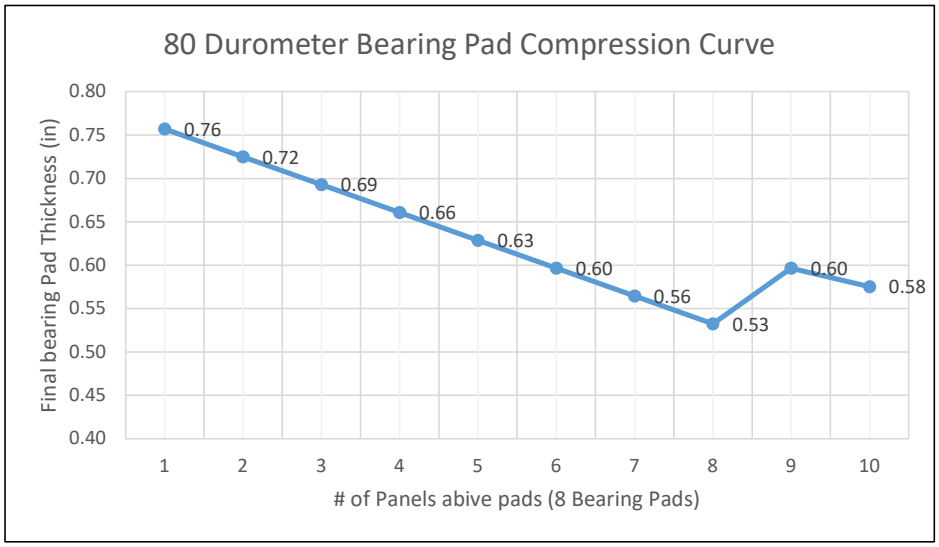
Concrete panel weight = 5ft x 5ft x 150 pcf x 0.5ft = 1875 lbs per panel
# of panel weight at 40ft = 40ft/5ft = 8
2X # of panel weight at = 8 x 2 = 16
Total concrete panel weight = 16ft x 1875lbs = 30,000 lbs
Number of pad per 5' x 5' panel = 4
Therefore load on each bearing pad = 30,000/4 = 7,500 lbs.

Based on the compression test results at 7500 lbs load, total compression of the bearing pad = 0.29"
Therefore, joint height after compression with 40ft equivalent panel load = 0.815" - 0.29" = 0.525"

Use 4 bearing pads per panel for wall height up to 44ft. Use 6 bearing pads per panel for wall height above 44ft.

Tabulation of Wall Height vs Bearing Pad Compression

Height above pads (ft)	# of Panels	Weight of panel x2 for downdrag (lbs)	# of bearing pads (ea)	Weight per bearing pad (lbs)	Bearing pad compression (in)	Final bearing pad thickness (in)
5	1	3,750	4	937.5	0.06	0.76
10	2	7,500	4	1,875.0	0.09	0.72
15	3	11,250	4	2,812.5	0.12	0.69
20	4	15,000	4	3,750.0	0.15	0.66
25	5	18,750	4	4,687.5	0.19	0.63
30	6	22,500	4	5,625.0	0.22	0.60
35	7	26,250	4	6,562.5	0.25	0.56
40	8	30,000	4	7,500.0	0.28	0.53
45	9	33,750	6	5,625.0	0.22	0.60
50	10	37,500	6	6,250.0	0.24	0.58



Bearing Pad Calculations: Date: April 2025

80 Durometer Bearing Pad
Size: 2" x 4" x 0.815"
Panel Size = 5' x 9'

Example calcs: at equivalent wall height pressure = 40ft

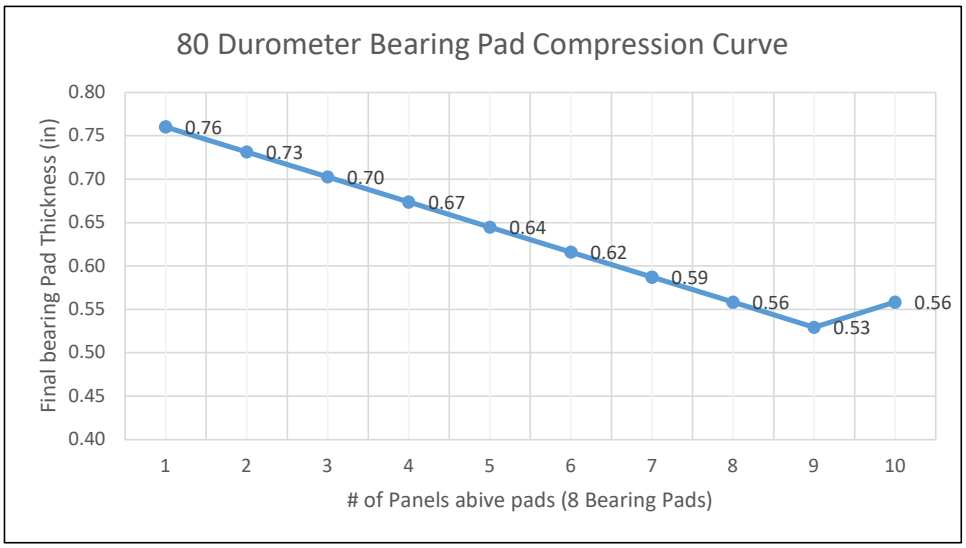
Concrete panel weight = 5ft x 9ft x 150 pcf x 0.5ft = 3375 lbs per panel
# of panel weight at 40ft = 40ft/5ft = 8
2X # of panel weight at = 8 x 2 = 16
Total concrete panel weight = 16ft x 3375lbs = 54,000 lbs
Number of pad per 5' x 9' panel = 8
Therefore load on each bearing pad = 54,000/8 = 6,750 lbs.

Based on the compression test results at 6750 lbs load, total compression of the bearing pad = 0.26"
Therefore, joint height after compression with 40ft equivalent panel load = 0.815" - 0.26" = 0.55"

Use 8 bearing pads per panel up for wall height up to 49ft. Use 10 bearing pads per panel for wall height above 50ft.

Tabulation of Wall Height vs Bearing Pad Compression

Height above pads (ft)	# of Panels	Weight of panel x2 for downdrag (lbs)	# of bearing pads (ea)	Weight per bearing pad (lbs)	Bearing pad compression (in)	Final bearing pad thickness (in)
5	1	6,750	8	843.8	0.05	0.76
10	2	13,500	8	1,687.5	0.08	0.73
15	3	20,250	8	2,531.3	0.11	0.70
20	4	27,000	8	3,375.0	0.14	0.67
25	5	33,750	8	4,218.8	0.17	0.64
30	6	40,500	8	5,062.5	0.20	0.62
35	7	47,250	8	5,906.3	0.23	0.59
40	8	54,000	8	6,750.0	0.26	0.56
45	9	60,750	8	7,593.8	0.29	0.53
50	10	67,500	10	6,750.0	0.26	0.56



Bearing Pad Calculations: Date: April 2025

80 Durometer Bearing Pad
Size: 2" x 4" x 0.815"
Panel Size = 5' x 10'

Example calcs: at equivalent wall height pressure = 40ft

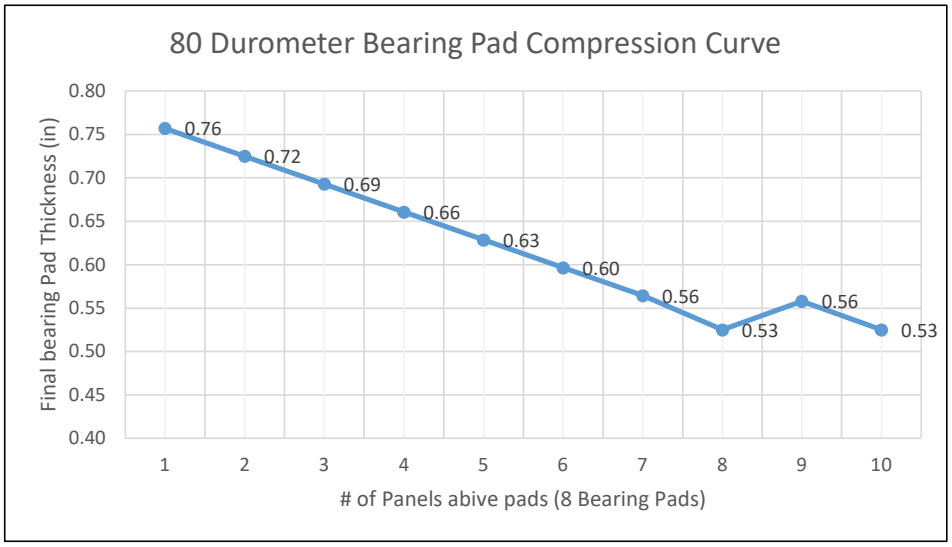
Concrete panel weight = 5ft x 10ft x 150 pcf x 0.5ft = 3750 lbs per panel
# of panel weight at 40ft = 40ft/5ft = 8
2X # of panel weight at = 8 x 2 = 16
Total concrete panel weight = 16ft x 3750lbs = 60,000 lbs
Number of pad per 5' x 10' panel = 8
Therefore load on each bearing pad = 60,000/8 = 7,500 lbs.

Based on the compression test results at 7500 lbs load, total compression of the bearing pad = 0.29"
Therefore, joint height after compression with 40ft equivalent panel load = 0.815" - 0.29" = 0.525"

Use 8 bearing pads per panel up for wall height up to 44ft. Use 10 bearing pads per panel for wall height above 44ft.

Tabulation of Wall Height vs Bearing Pad Compression

Height above pads (ft)	# of Panels	Weight of panel x2 for downdrag (lbs)	# of bearing pads (ea)	Weight per bearing pad (lbs)	Bearing pad compression (in)	Final bearing pad thickness (in)
5	1	7,500	8	937.5	0.06	0.76
10	2	15,000	8	1,875.0	0.09	0.72
15	3	22,500	8	2,812.5	0.12	0.69
20	4	30,000	8	3,750.0	0.15	0.66
25	5	37,500	8	4,687.5	0.19	0.63
30	6	45,000	8	5,625.0	0.22	0.60
35	7	52,500	8	6,562.5	0.25	0.56
40	8	60,000	8	7,500.0	0.29	0.53
45	9	67,500	10	6,750.0	0.26	0.56
50	10	75,000	10	7,500.0	0.29	0.53



Appendix 1.1.8.b

Bearing Pad Specifications

ARES® MSE Retaining Wall System

Back to main document

Phoenix National Laboratories, Inc.
2837 E. Chambers Street, Phoenix, AZ 85040
P: 602.431.8887 · F: 602.431.8889 · www.pnltest.com

LABORATORY TEST REPORT
EPDM BEARING PADS FOR MSE WALL
Corrected Rev 1

CLIENT	CLIENT PROJECT REF. NO.	CLIENT ORDER NO.
Tensar International	Unknown	Per Service Agreement

STATE	STATE PROJECT NO.	PROJECT DESCRIPTION
NC	N/A	Testing of EPDM Bearing for MSE Wall per NCDOT

MATERIAL SUBMITTED BY	DATE SUBMITTED	PNL PROJECT NO.	S.O. NO.	PNL LAB NO.	REPORT DATE
Client	03/14/2016	26-160103	001	ML825982	03/21/2016

SPECIFICATION	DUROMETER	RUBBER TYPE	LOT NO.	PAD SIZE (TxWxD)
FHWA-NHI-10-024	Not Specified	EPDM	None	0.815" x 2" x 4"

SUMMARY OF RESULTS

STANDARD	DESCRIPTION	REQUIREMENT	TEST VALUE	RESULT
AASHTO M251	**COMPRESSION DEFLECTION TEST: 5625 lbf**	Record Results	0.23	N/A
AASHTO M251	**COMPRESSION DEFLECTION TEST: 7500 lbf**	Record Results	0.29	N/A

PNL warrants that the above services and report were performed under the appropriate standard of care, including the skill and judgement that is reasonably expected from similarly situated technical personnel. No other warranty, guaranty, or representation, either expressed or implied is included or intended.

REVIEWED BY

Phoenix National Laboratories, Inc.
2837 E. Chambers Street, Phoenix, AZ 85040
P: 602.431.8887 · F: 602.431.8889 · www.pnltest.com

LABORATORY TEST REPORT
EPDM BEARING PADS FOR MSE WALL

Registered Professional Engineer (Mechanical) CERTIFICATE NO. 53776 KYLE J. FLEEGE DATE SIGNED 03-22-2016 ARIZONA U.S.A.
Expires 09/30/2018

CLIENT	STATE PROJECT NO.	CLIENT ORDER NO.
Tensar International	N/A	Per Service
PROJECT DESCRIPTION		PNL LAB NO.
Testing of EPDM Bearing for MSE		ML825982
LOT NO.	PAD SIZE	REPORT DATE
None	0.815" x 2" x 4"	03/21/2016

SPECIFIC TEST DATA

COMPRESSION STRAIN - AASHTO M251-06 Sect. 9.1							
TEST LOAD, lbf	**5% LOAD READING A (in.)**	**100% LOAD READING B (in.)**	**TOTAL COMPRESSIVE DEFLECTION B - A (in.)**	**ERT (in.)**	**STRAIN (B-A)/ERT**	**REQUIREMENT**	**RESULT**
5625	0.023	0.212	0.189	0.815	0.232	Not Specified	N/A
PAD THICKNESS AFTER COMPRESSION (ERT - 100% LOAD READING):						0.603	

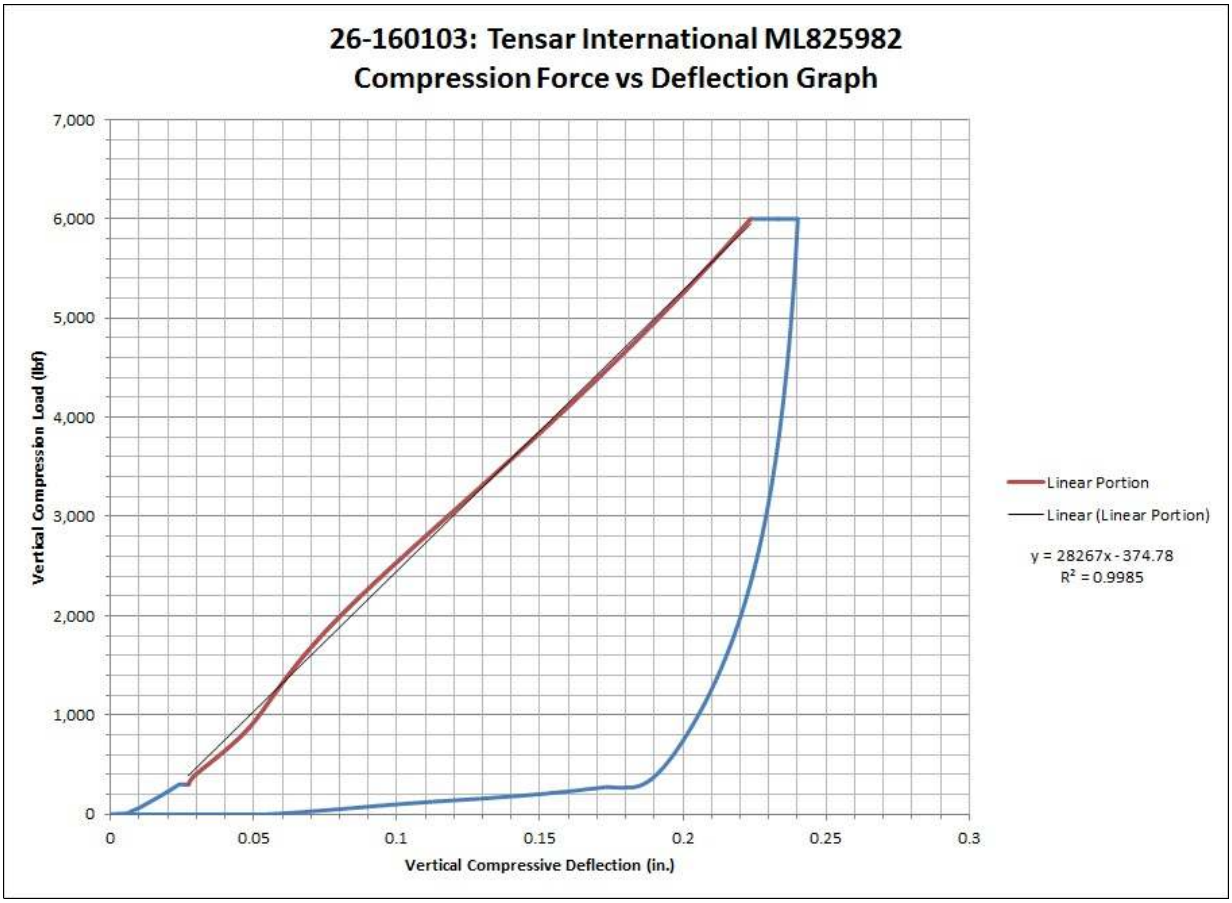


Figure 1: ML825982 Compressive Load vs Deflection Graph with linear regression analysis of load segment

Phoenix National Laboratories, Inc.
2837 E. Chambers Street, Phoenix, AZ 85040
P: 602.431.8887 · F: 602.431.8889 · www.pnltest.com

LABORATORY TEST REPORT
EPDM BEARING PADS FOR MSE WALL

CLIENT	STATE PROJECT NO.	CLIENT ORDER NO.
Tensar International	N/A	Per Service
PROJECT DESCRIPTION		PNL LAB NO.
Testing of EPDM Bearing for MSE		ML825982
LOT NO.	PAD SIZE	REPORT DATE
None	0.815" x 2" x 4"	03/21/2016

SPECIFIC TEST DATA

COMPRESSION STRAIN - AASHTO M251-06 Sect. 9.1							
TEST LOAD, lbf	**5% LOAD READING A (in.)**	**100% LOAD READING B (in.)**	**TOTAL COMPRESSIVE DEFLECTION B - A (in.)**	**ERT (in.)**	**STRAIN (B-A)/ERT**	**REQUIREMENT**	**RESULT**
7500	0.038	0.276	0.239	0.815	0.293	Not Specified	N/A
PAD THICKNESS AFTER COMPRESSION (ERT - 100% LOAD READING):						0.539	

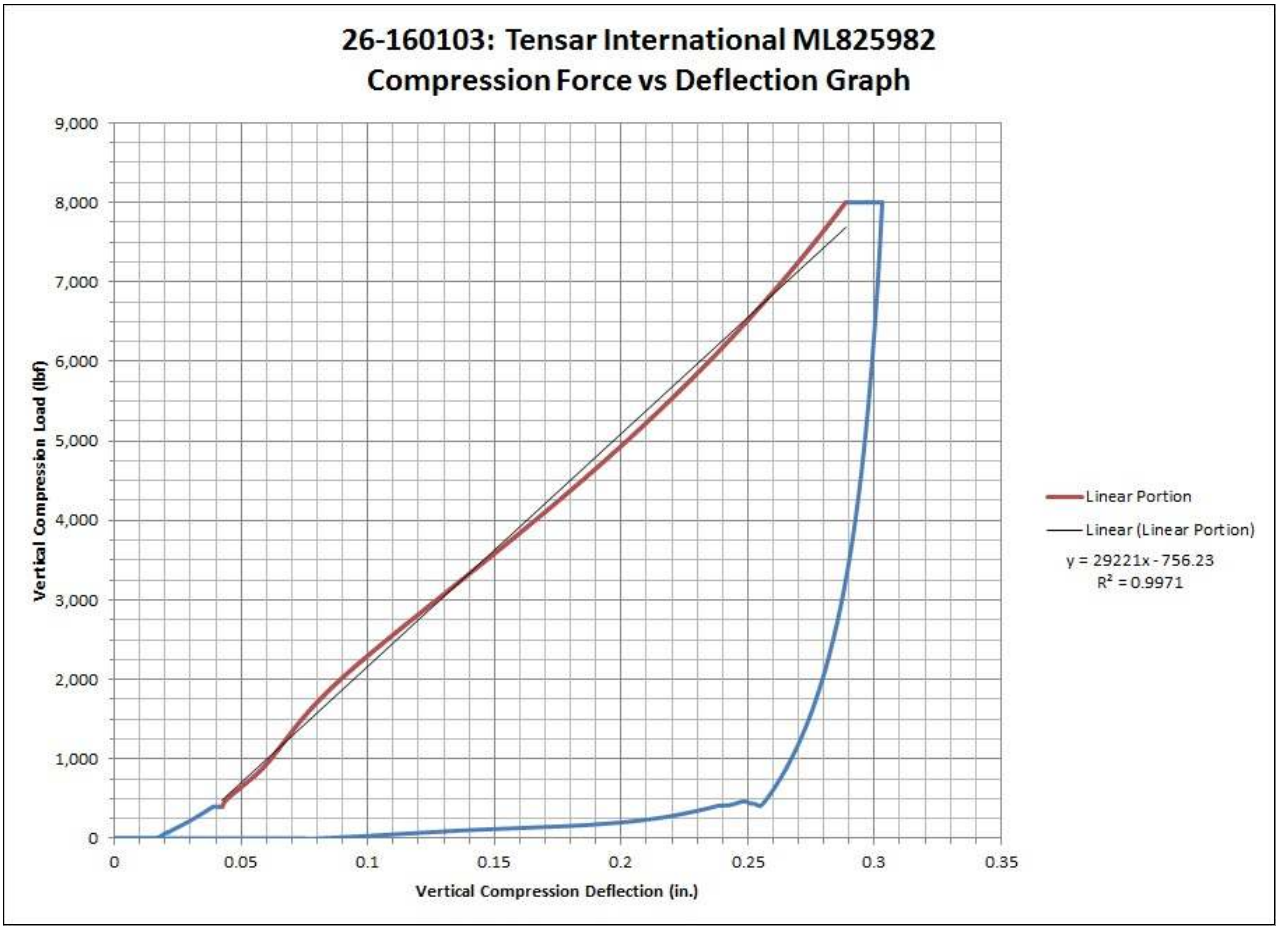


Figure 2: ML825982 Compressive Load vs Deflection Graph with linear regression analysis of load segment

Appendix 1.1.9

Geotextile Specifications Sheet

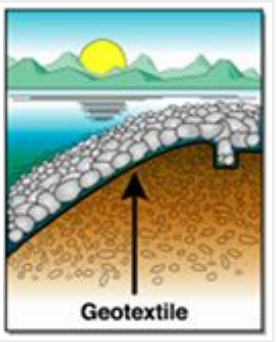
ARES® MSE Retaining Wall System

Back to main document

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 1.1.10

Architectural Facing Finish

ARES® MSE Retaining Wall System

Back to main document

Various Architectural Facing Finish for Tensar ARES System

Ashlar pattern

Fractured fin pattern

Granite block pattern

Large block pattern

Random relief pattern

Stacked block pattern

Appendix 1.2.3.a

75yr-Tensar UX Geogrid

Strength Design Properties

ARES® 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

ARES® MSE Retaining Wall System

Back to main document

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

ARES® MSE Retaining Wall System

Back to main document

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

AASHTO
THE VOICE OF TRANSPORTATION

Report Issued: May 2014
Report Expiration Date: May 2023
Next Quality Assurance Update Report: 2020

American Association of State Highway and Transportation Officials (AASHTO)

Executive Office: 444 North Capitol Street, NW, Suite 249 • Washington, DC • 20001
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2014 NTPEP Report Series

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.

Guidelines for Proper Use of NTPEP Results:

- The User is urged to carefully read any introductory notes at the beginning of this Report, and also to consider any special clauses, footnotes or conditions which may apply to any test reported herein. Any of these notes may be relevant to the proper use of NTPEP test data.
- 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

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Table of Contents

Executive Summary 4
1.0 Product Line Description and Testing Strategy 7
1.1 Product Description 7
1.2 Product Line Testing Approach 7
2.0 Product Polymer, Geometry, and Manufacturing Information……..……11
2.1 Product/Polymer Descriptors 11
2.2 Geometric Properties of Geogrids 11
2.3 Product Production Data and Manufacturing Quality Control 11
3.0 Wide Width Tensile Strength Data 12
4.0 Installation Damage Data (RF_{ID}) 13
4.1 Installation Damage Test Program 13
4.2 Installation Damage Full Scale Field Exposure Procedures and Materials Used 14
4.3 Summary of Installation Damage Full Scale Field Exposure Test Results 17
4.4 Estimating RF_{ID} for Specific Soils or for Products not Tested 18
4.5 Laboratory Installation Damage Test Results per ISO/EN 10722 21
5.0 Creep Rupture Data (RF_{CR}) 23
5.1 Creep Rupture Test Program 23
5.2 Baseline Tensile Strength Test Results 24
5.3 Creep Rupture Test Results 26
5.3.1 Statistical Validation to Allow the Use of Composite Rupture Envelope for Product Line ... 27
5.3.2 Validation of Shift Factors used for Accelerated Creep Rupture Tests 27
5.4 Creep Rupture Envelope Development and Determination of RF_{CR} 28
6.0 Long-Term Durability Data (RF_D) 31
6.1 Durability Test Program 31
6.2 Durability Test Results 32
7.0 Low Strain Creep Stiffness Data 34
7.1 Low Strain Creep Stiffness Test Program 34
7.2 Ultimate Tensile Test Results for Creep Stiffness Test Program 34
7.3 Creep Stiffness Test Results 34
Appendix A: NTPEP Oversight Committee A-1
Appendix B: Product Geometric and Production Details B-1
B.1 Product Geometric Information B-2
B.2 Product Production Information B-10
B.3 Product Manufacturing Quality Control Program B-10
Appendix C: Tensile Strength Detailed Test Results C-1
Appendix D: Installation Damage Detailed Test Results D-1
Appendix E: ISO/EN Laboratory Installation Damage Detailed Test Results E-1
E.1 ISO/EN Laboratory Installation Damage Test Program E-2
Appendix F: Creep Rupture Detailed Test Results F-1
Appendix G: Durability Detailed Test Results G-1
Appendix H: Creep Stiffness Detailed Test Results H-1

Tables

Table 1-1. Product designations included in product line. 7
Table 3-1. Wide width tensile strength, T_{ult}, for the UX-MSE / UX-HS product line. 12
Table 4-1. Independent installation damage testing required for NTPEP qualification. 13
Table 4-2. Summary of installation damage tensile test results. 17
Table 4-3. Measured RF_{ID}. 17
Table 4-4. Summary of laboratory (ISO procedure) installation damage test results. 22
Table 5-1. Independent creep rupture testing required for NTPEP qualification. 23
Table 5-2. Independent creep rupture testing required for NTPEP qualification. 24
Table 5-3. Ultimate tensile strength (UTS) and associated strain. 25
Table 5-4. Creep rupture test results for all tests conducted. 26
Table 5-5. RF_{CR} value for MSE/HS series geogrids for a 75 yr period of loading/use. 28
Table 6-1. Independent durability testing required for NTPEP qualification. 32
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. 33
Table 7-1. Ultimate tensile strength (UTS) & associated strain. 34
Table 7-2. Summary of creep stiffness test results. 35

Figures

Figure 1-1. Photo of UX1100MSE (machine direction is perpendicular to ruler shown). 8
Figure 1-2. Photo of UX1400MSE (machine direction is perpendicular to ruler shown). 8
Figure 1-3. Photo of UX1500MSE (machine direction is perpendicular to ruler shown). 9
Figure 1-4. Photo of UX1600MSE (machine direction is perpendicular to ruler shown). 9
Figure 1-5. Photo of UX1700MSE (machine direction is perpendicular to ruler shown). 10
Figure 4-1. Test soil grain size distribution. 15
Figure 4-2. Installation damage Type 1 test aggregate..... 15
Figure 4-3. Installation damage Type 2 test aggregate..... 16
Figure 4-4. Installation damage Type 3 test aggregate..... 16
Figure 4-5. UX-MSE / UX-HS product line installation damage as a function of soil d_{50} size. . 19
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). 19
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). 20
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). 20
Figure 5-1. Composite creep rupture data/envelope for the UX1100MSE/HS and X1400MSE/HS geogrid products. 29
Figure 5-2. Composite creep rupture data/envelope for the UX1500MSE/HS, UX1600MSE/HS and UX1700MSE/HS geogrid products..... 30
Figure 7-1. UX-MSE / UX-HS creep stiffness for 2 % strain @ 1000 hours. 35

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^{o} F (20^{o} 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

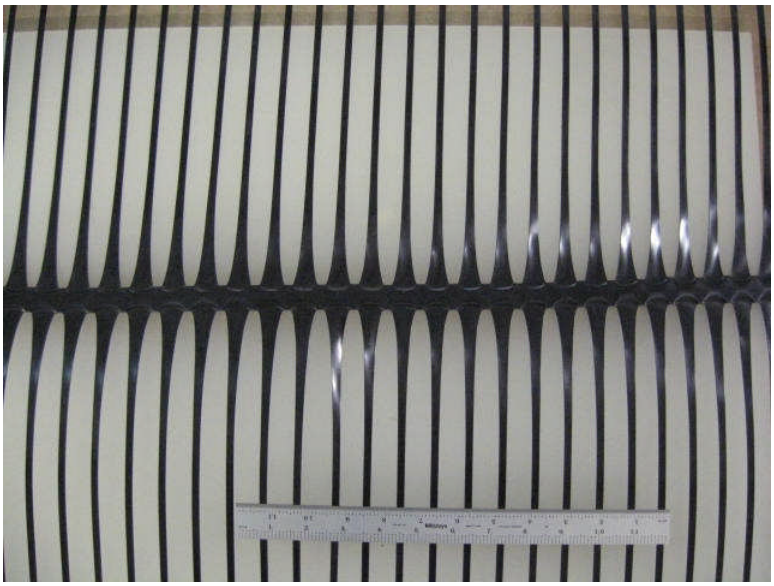


Figure 1-1. Photo of UX1100MSE (machine direction is perpendicular to ruler shown).

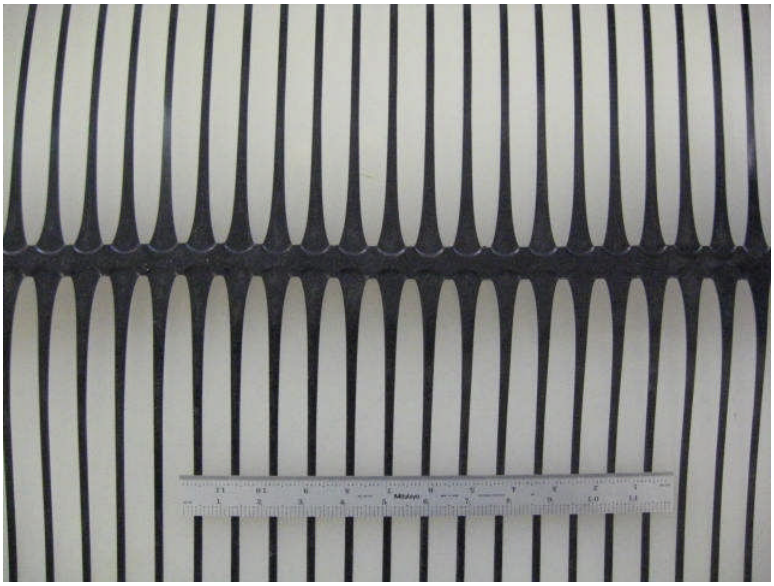


Figure 1-2. Photo of UX1400MSE (machine direction is perpendicular to ruler shown).

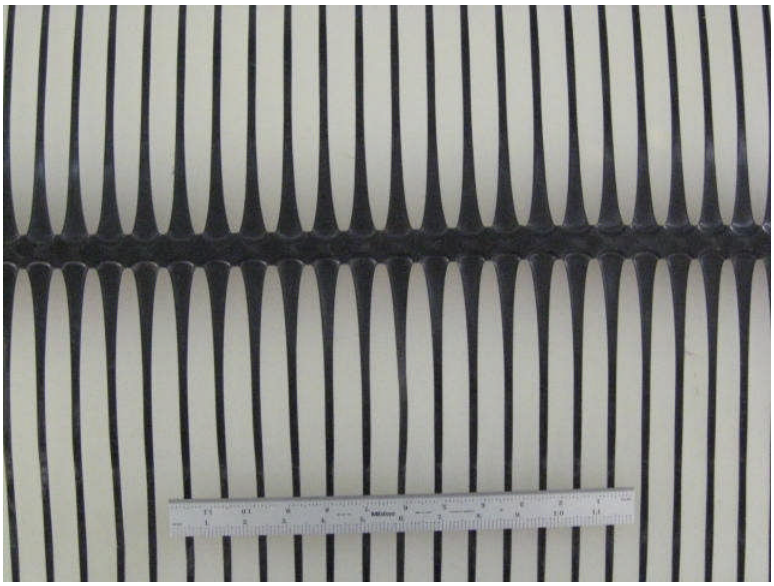


Figure 1-3. Photo of UX1500MSE (machine direction is perpendicular to ruler shown).

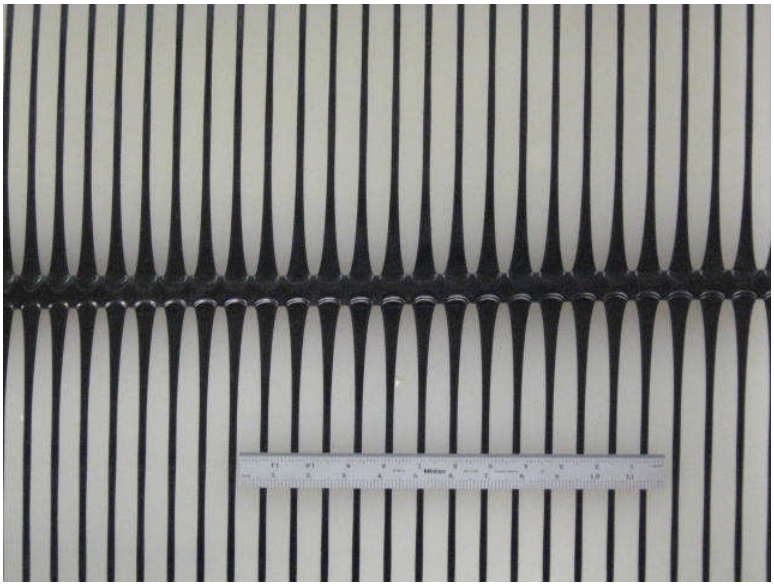


Figure 1-4. Photo of UX1600MSE (machine direction is perpendicular to ruler shown).

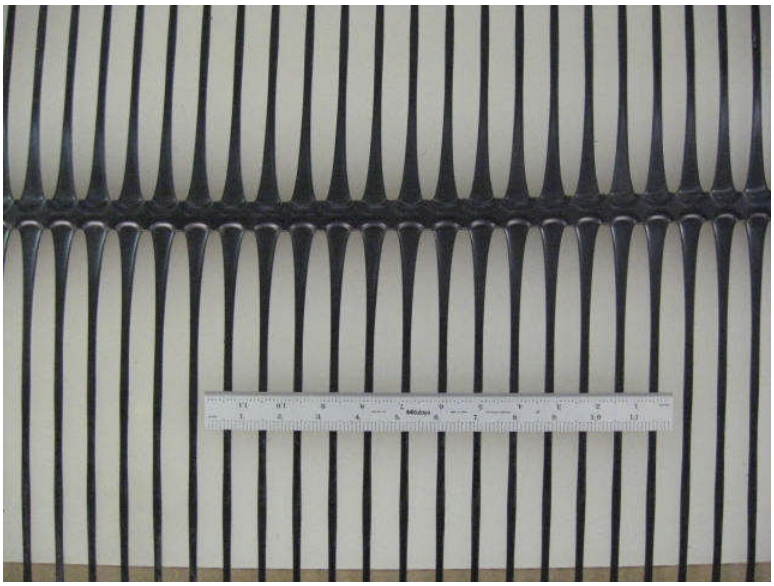


Figure 1-5. Photo of UX1700MSE (machine direction is perpendicular to ruler shown).

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.

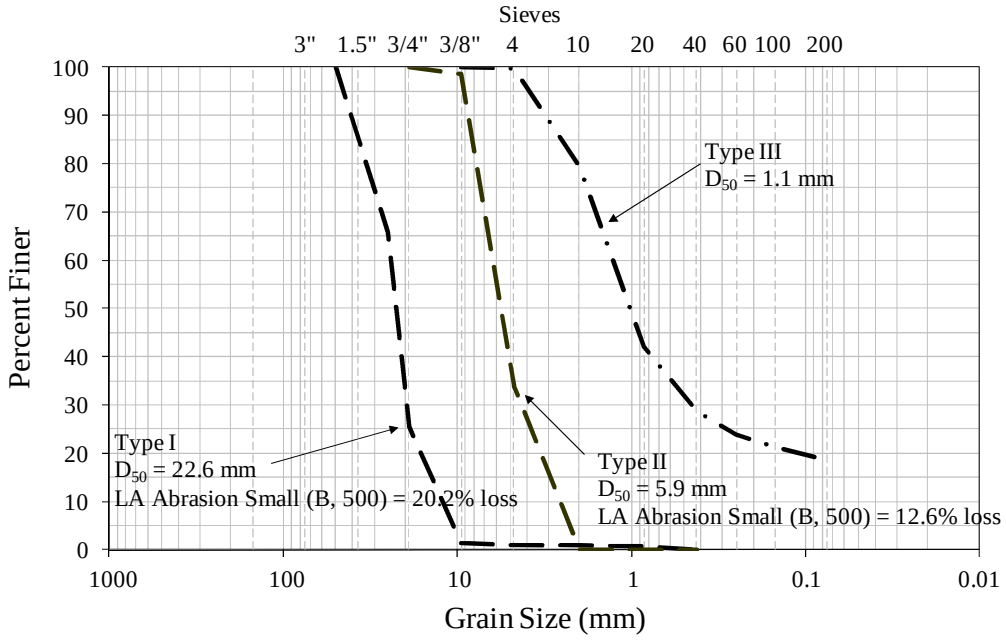


Figure 4-1. Test soil grain size distribution.

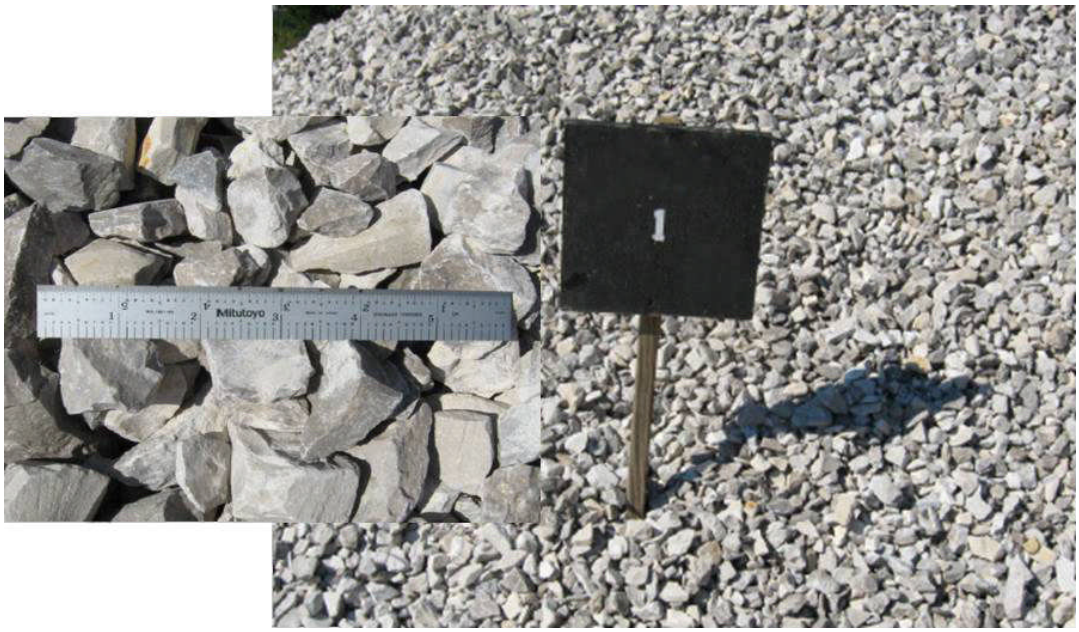


Figure 4-2. Installation damage Type 1 test aggregate.

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}.

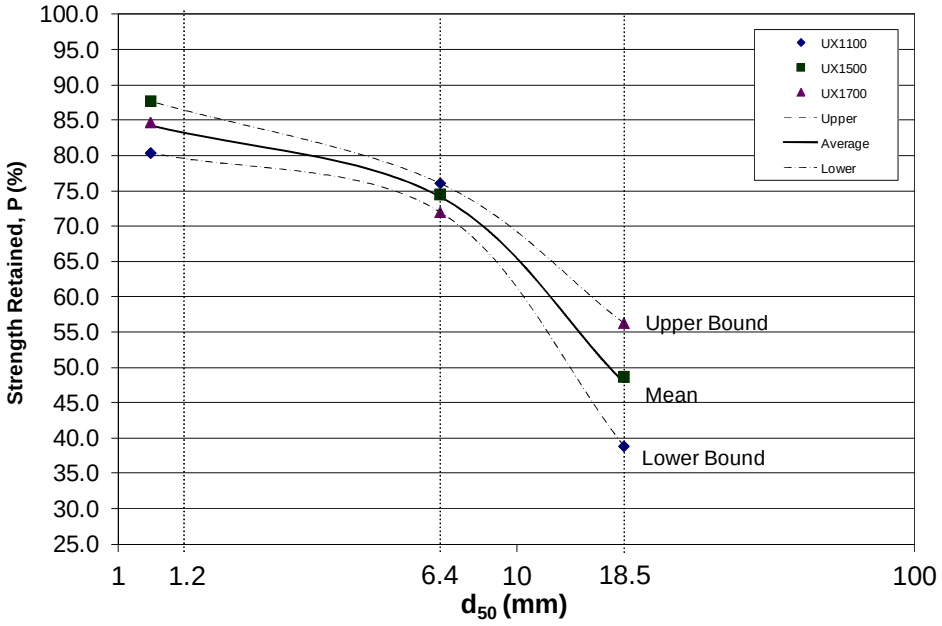
Style	Mass / Area ($oz./yd^2$)	Type 1 Coarse Gravel		Type 2 Sandy Gravel		Type 3 Silty Sand	
		% Retained	RF_{ID}	% Retained	RF_{ID}	% Retained	RF_{ID}
UX1100	8.55	38.9	2.57	76.1	1.31	80.4	1.24
UX1500	14.11	48.7	2.05	74.5	1.34	87.8	1.14
UX1700	23.72	56.3	1.78	72.0	1.39	84.8	1.18

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.

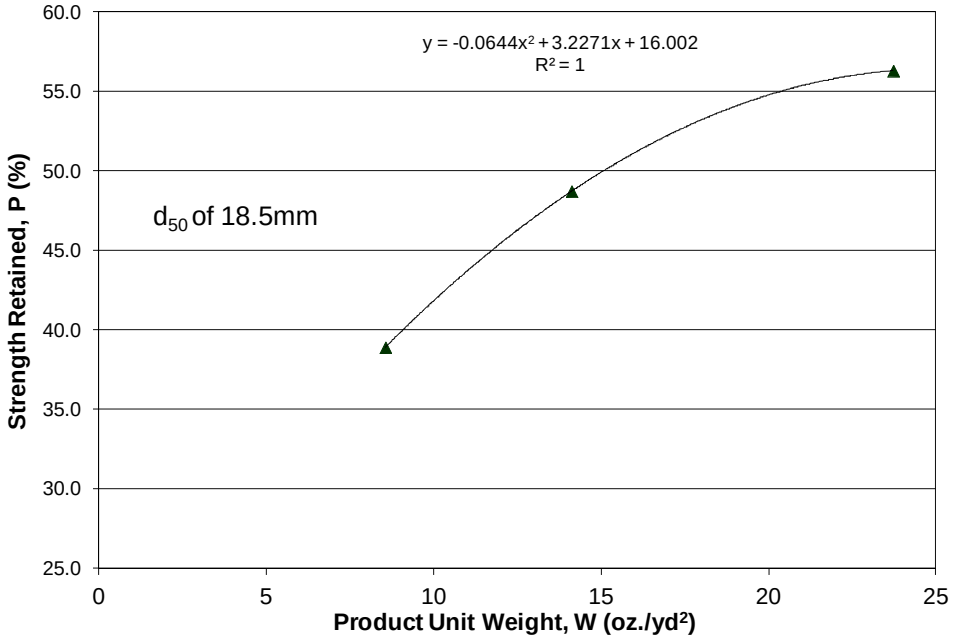


Figure 4-6. UX-MSE / UX-HS product line installation damage as a function of product unit weight for type 1 soil (coarse gravel - GP).

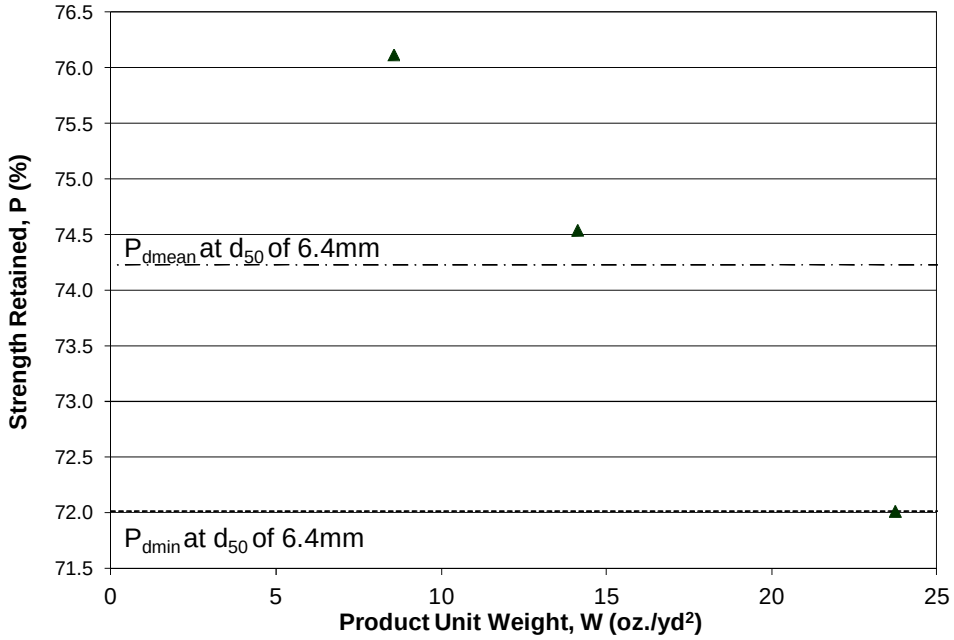


Figure 4-7. UX-MSE / UX-HS product line installation damage as a function of product unit weight for type 2 soil (sandy gravel - GP).

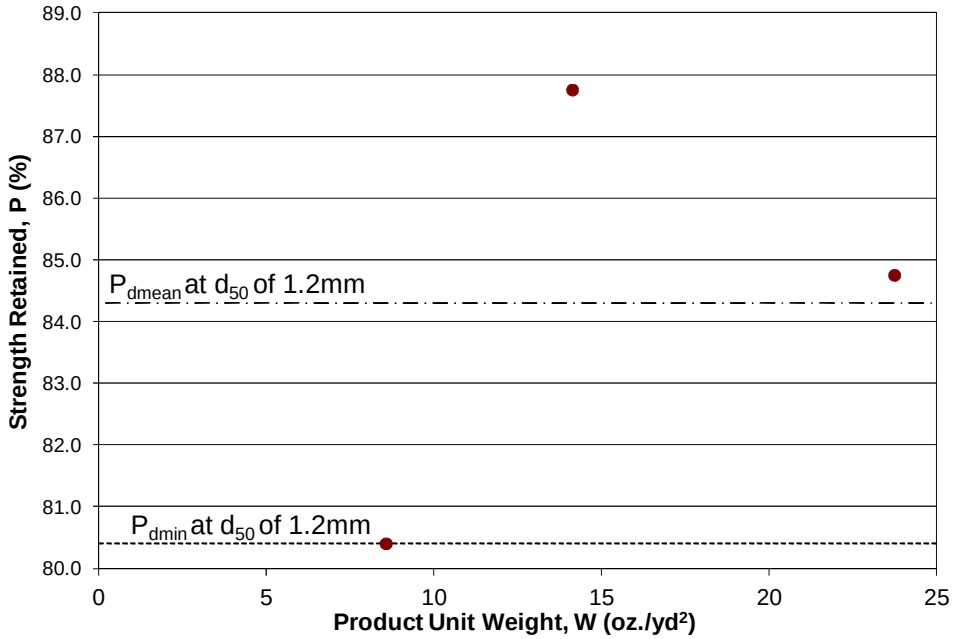


Figure 4-8. UX-MSE / UX-HS product line installation damage as a function of product unit weight for type 3 soil (silty sand – SM).

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

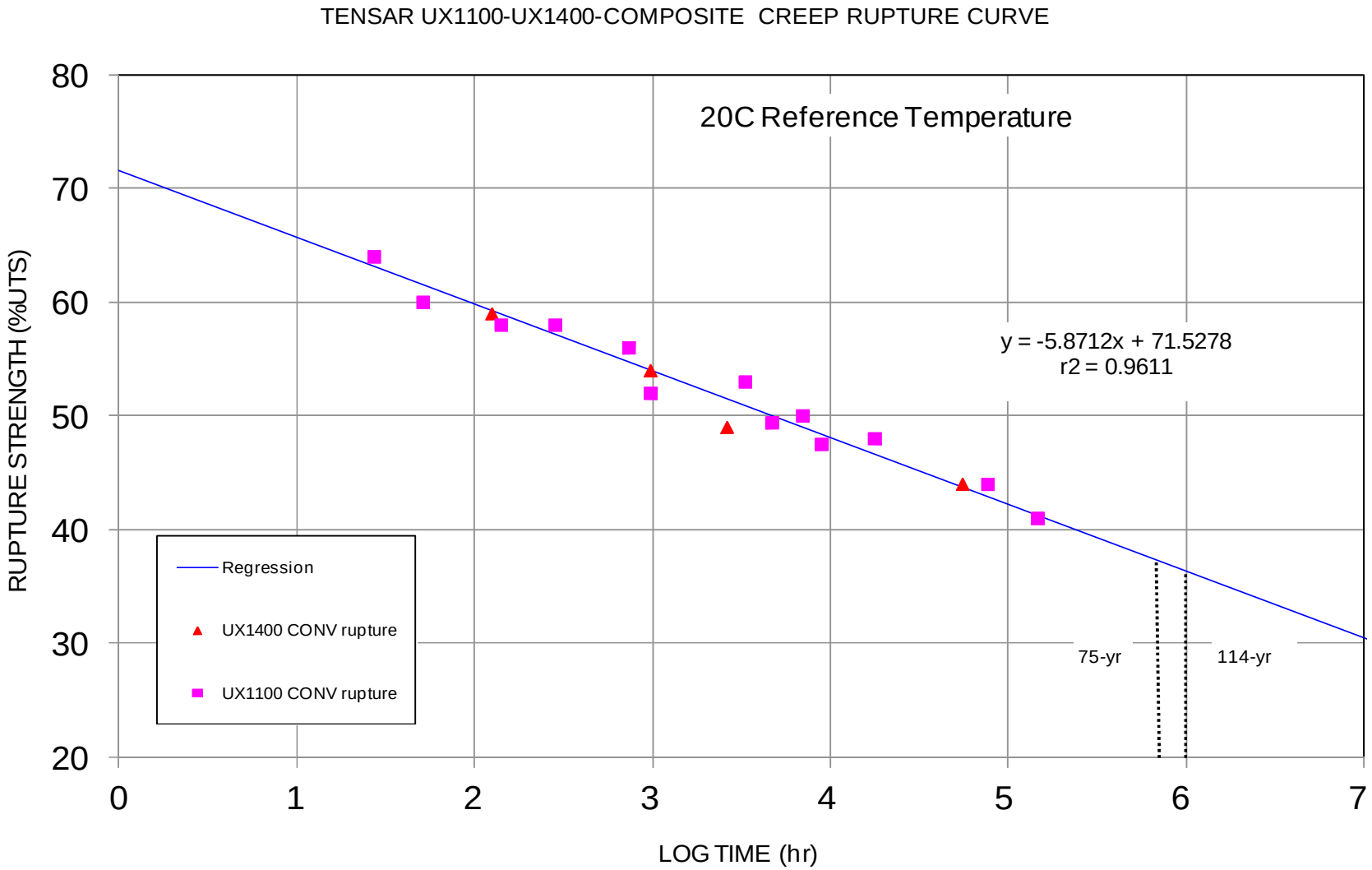


Figure 5-1. Composite creep rupture data/envelope for the UX1100MSE/HS and X1400MSE/HS geogrid products.

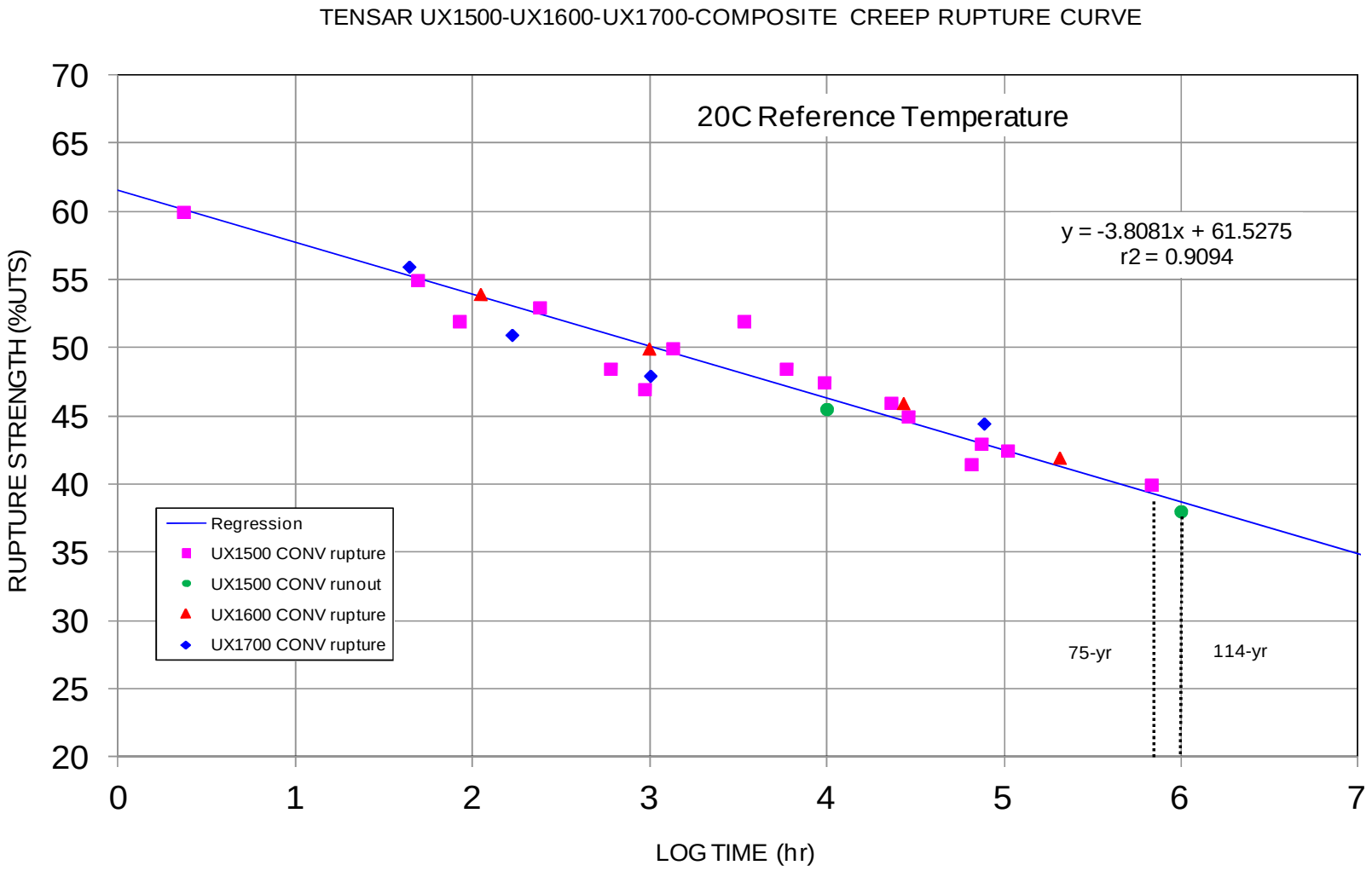


Figure 5-2. Composite creep rupture data/envelope for the UX1500MSE/HS, UX1600MSE/HS and UX1700MSE/HS geogrid products.

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)

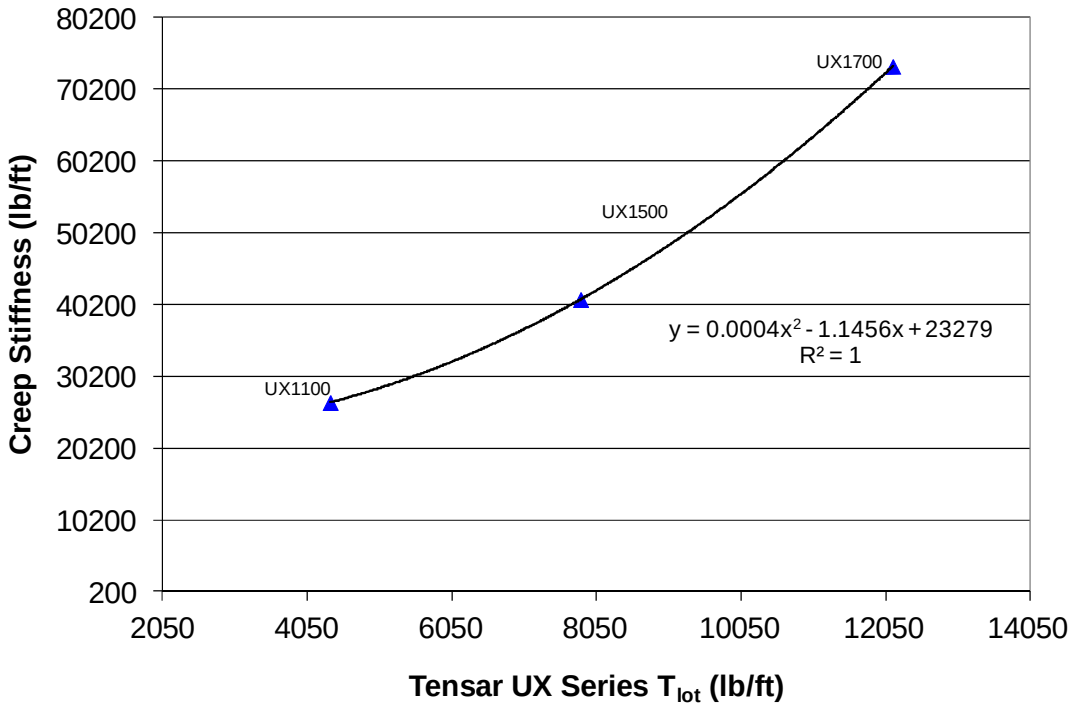


Figure 7-1. UX-MSE / UX-HS creep stiffness for 2 % strain @ 1000 hours.

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
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Lostumbo, John	j.lostumbo@tencategeo.com	TenCate Geosynthetics	Other	None
Gee, Bryan	bgee@tensarcorp.com	Tensar North American Green	Other	None
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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.

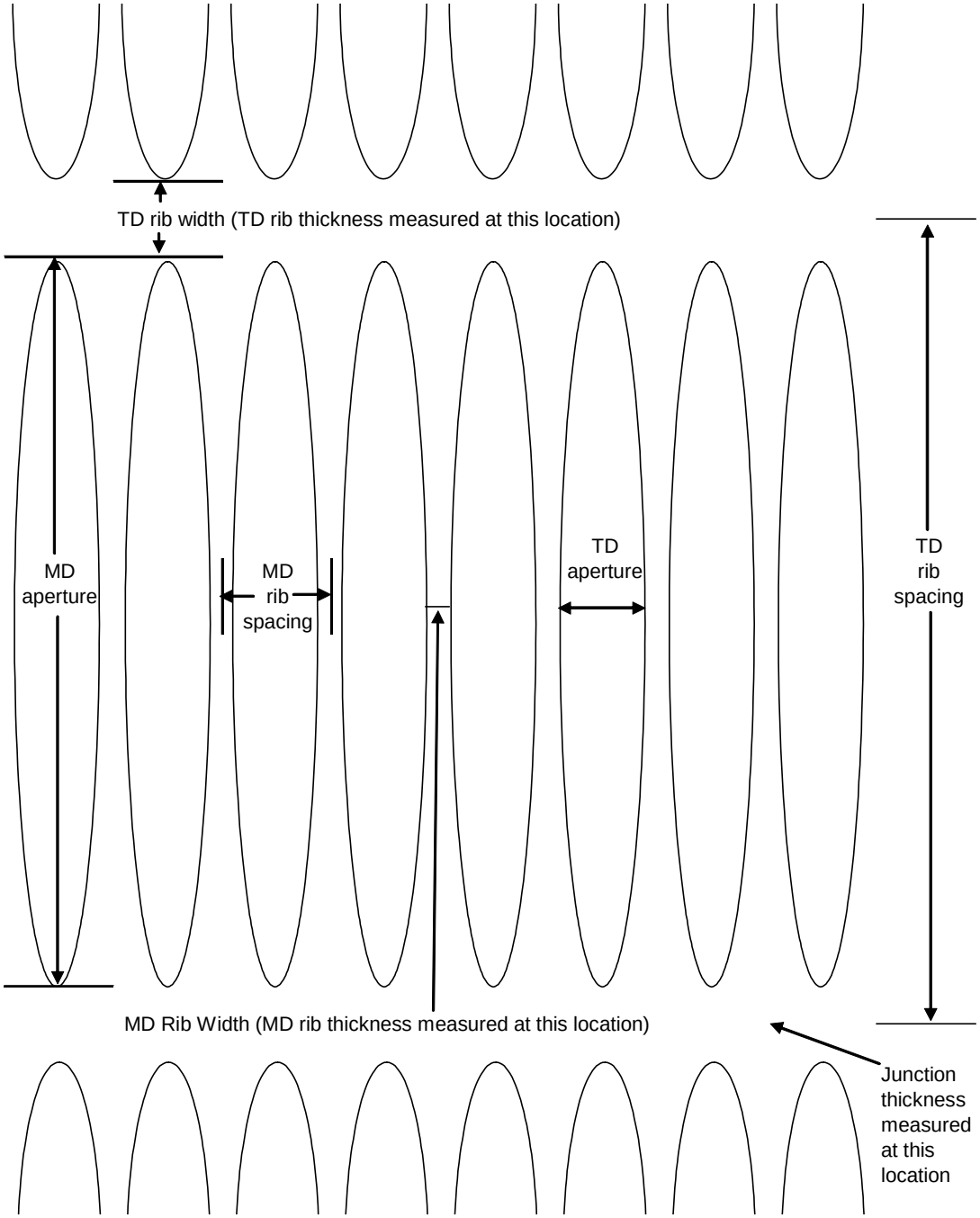

Figure B-1. Diagram of the geometric properties measurement locations.

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.

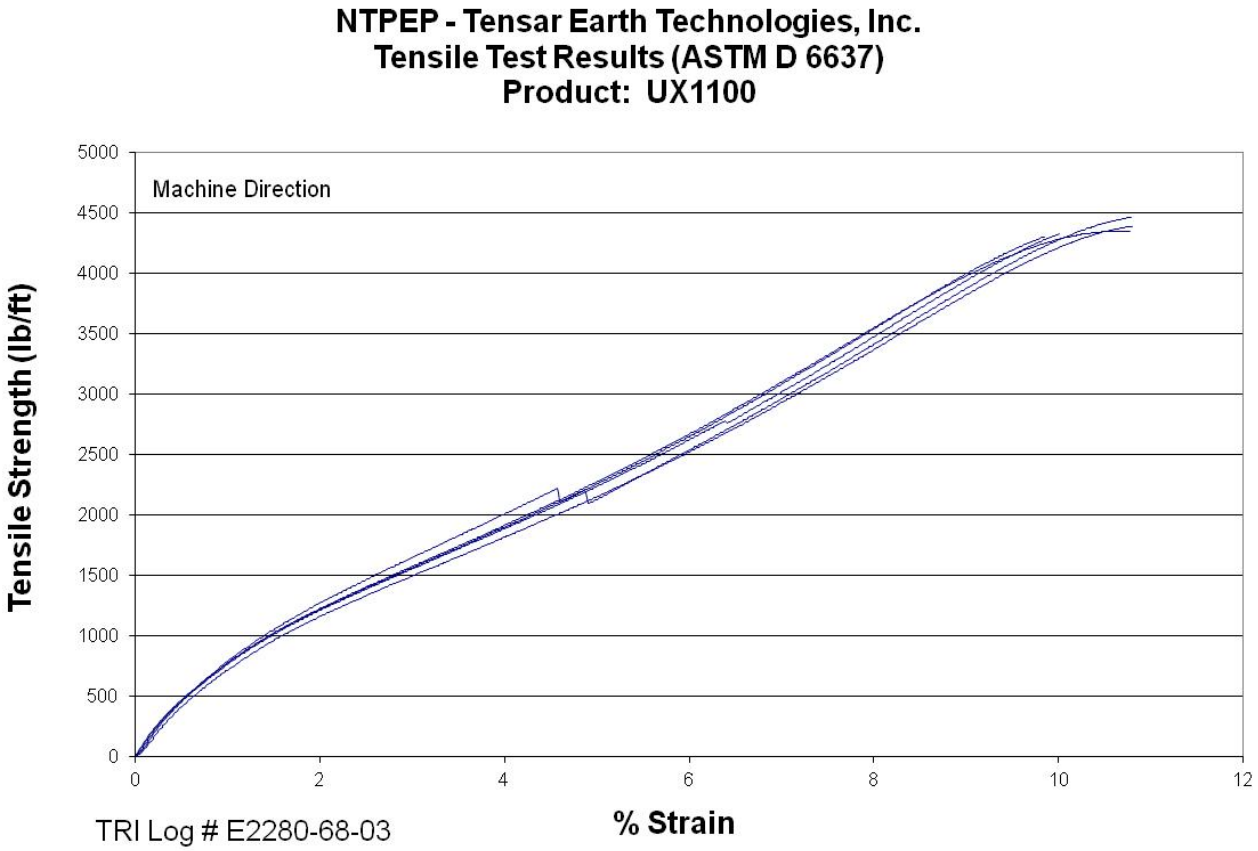

Figure C-1: Geogrid wide width tensile test load-strain curve for Tensar UX1100

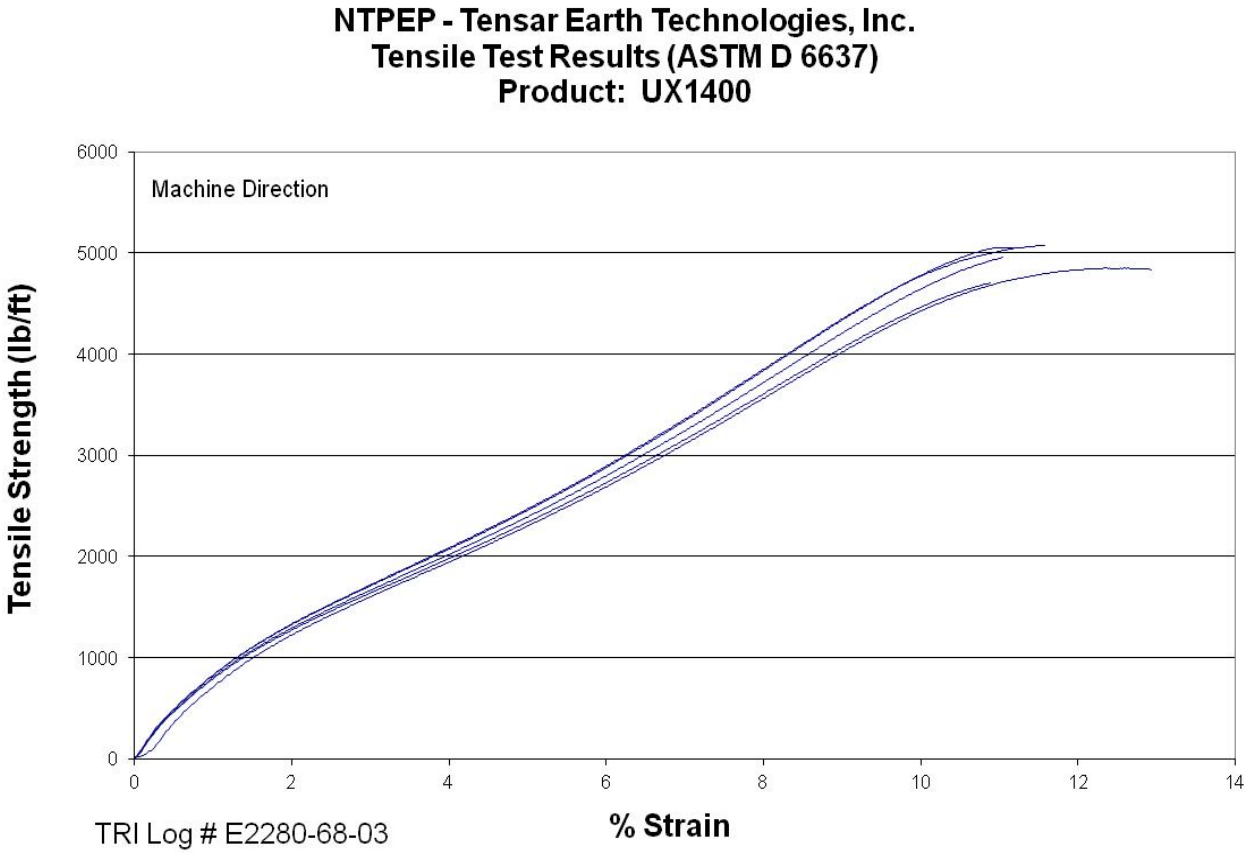

Figure C-2: Geogrid wide width tensile test load-strain curve for Tensar UX1400

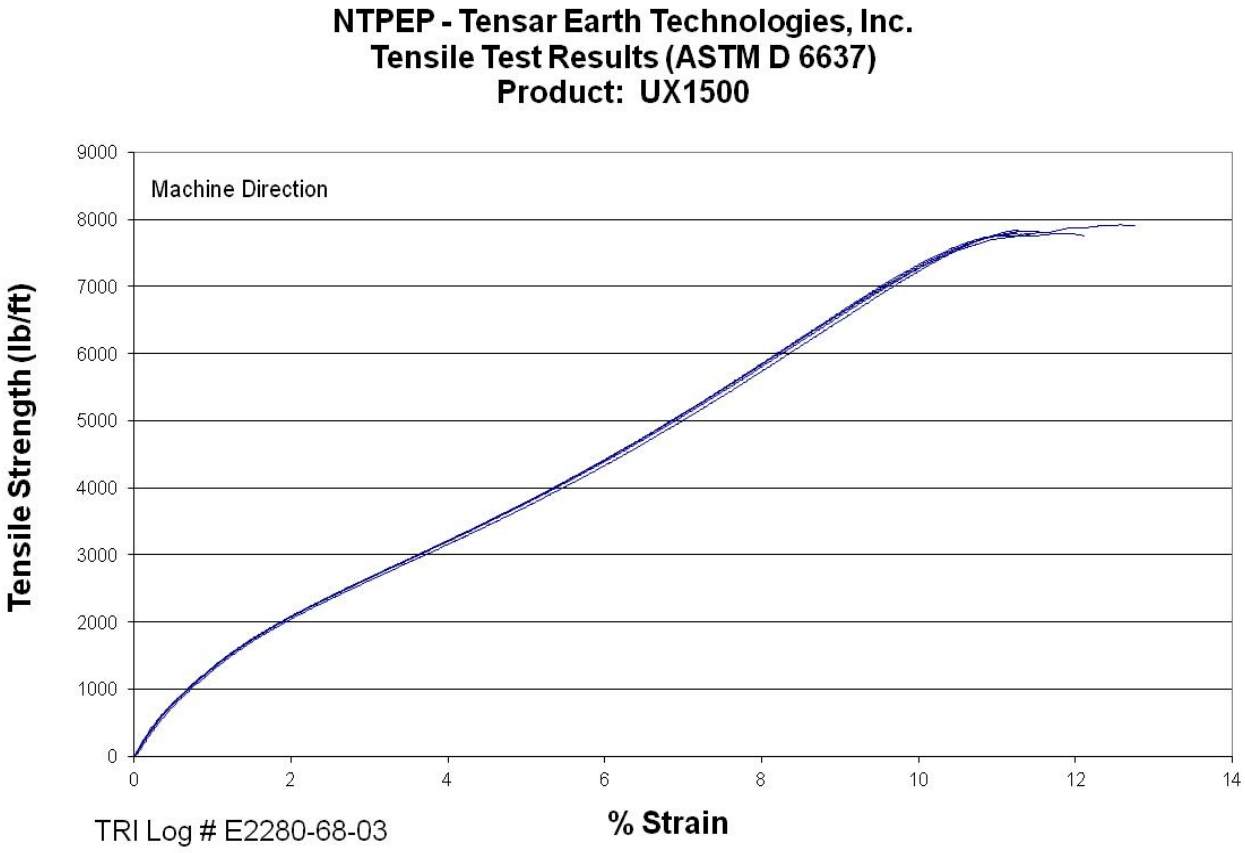

Figure C-3: Geogrid wide width tensile test load-strain curve for Tensar UX1500

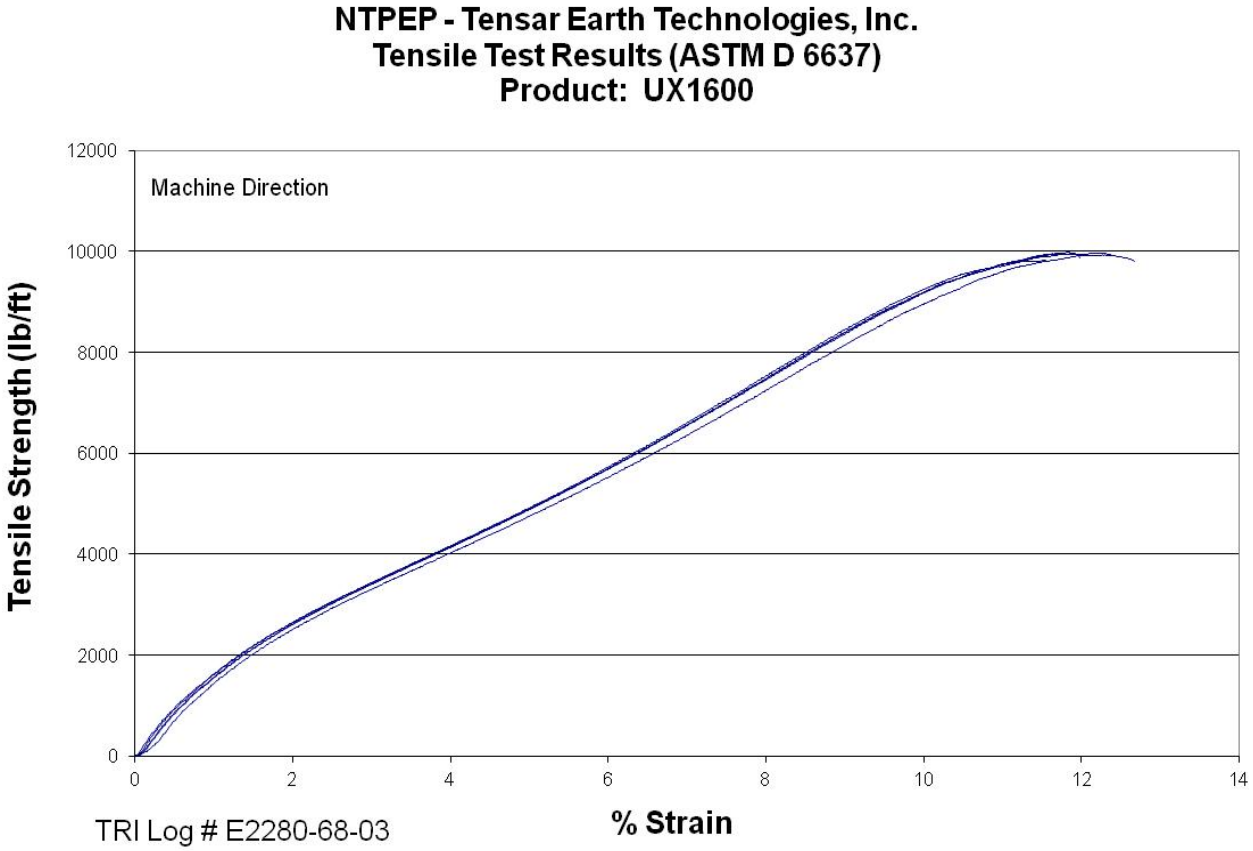

Figure C-4: Geogrid wide width tensile test load-strain curve for Tensar UX1600

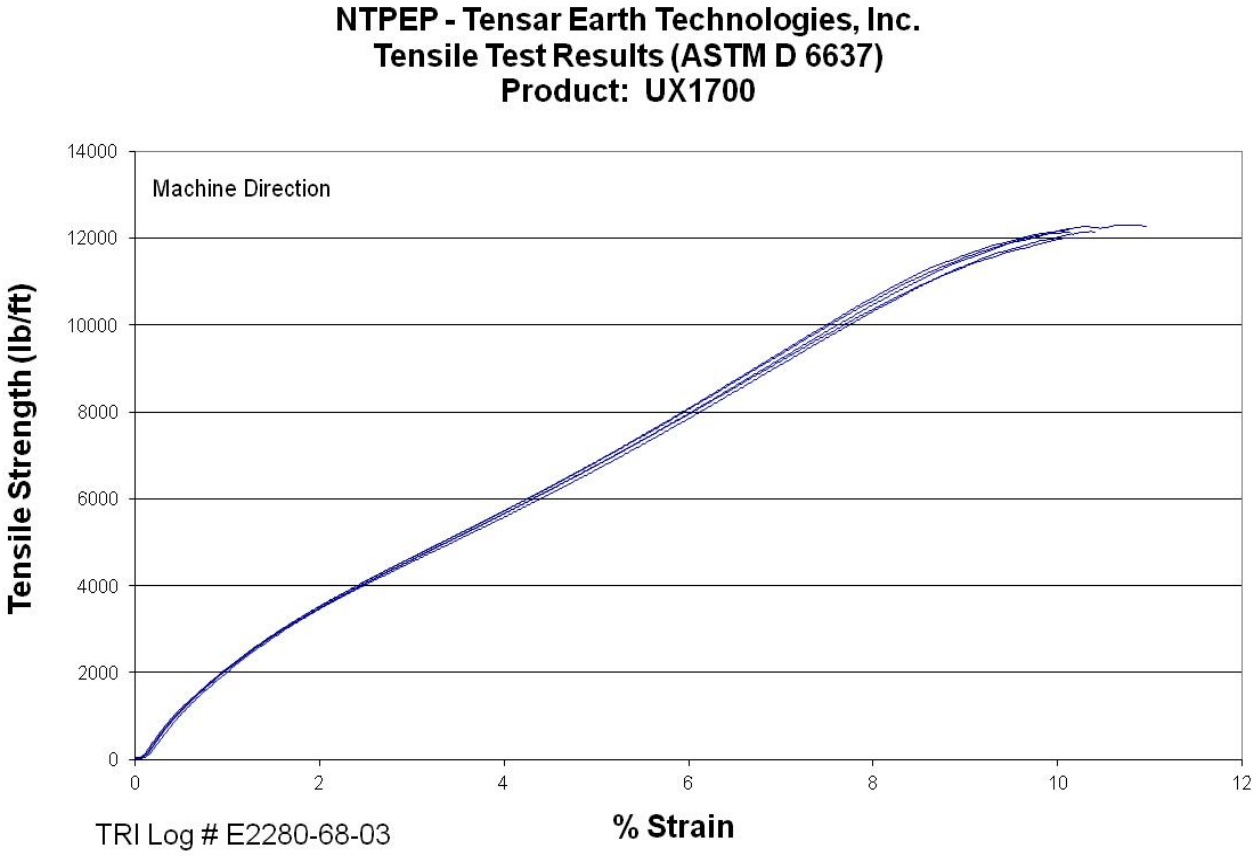

Figure C-5: Geogrid wide width tensile test load-strain curve for Tensar UX1700

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.

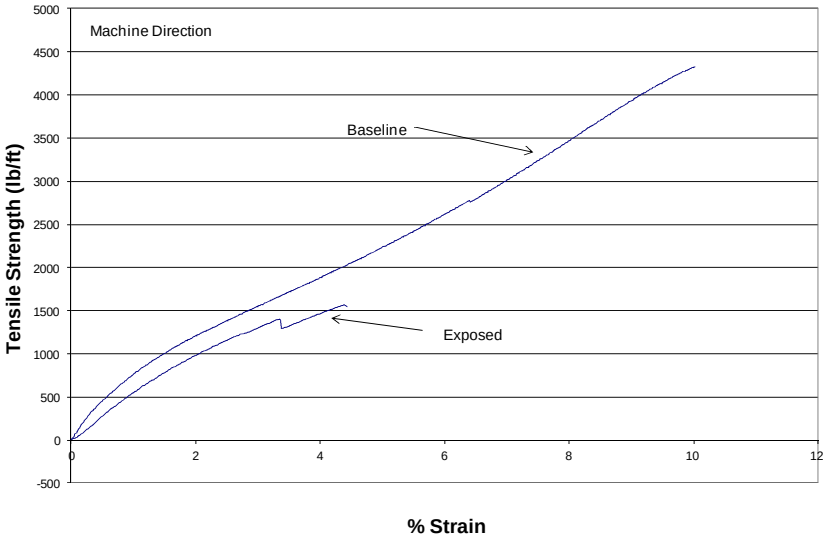

Figure D-1. Example baseline and exposed wide width tensile test load-strain curves for UX1100MSE geogrid installed in soil gradation 1.

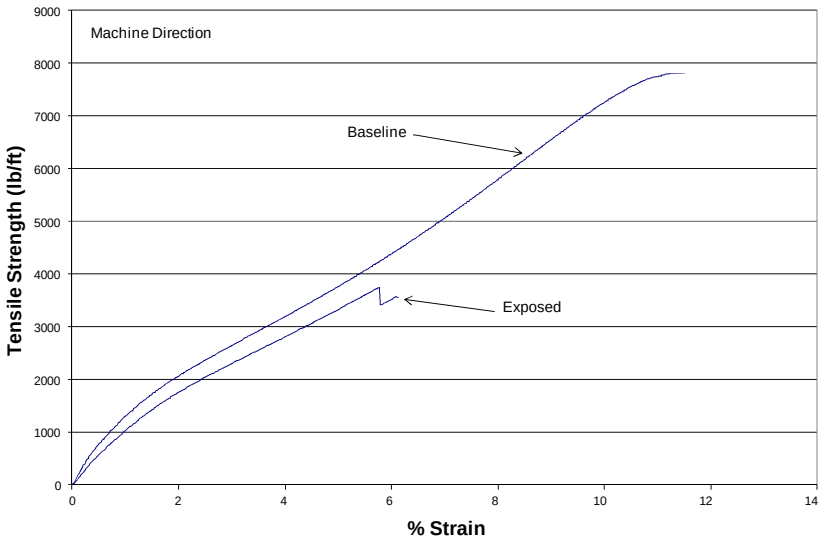

Figure D-2. Example baseline and exposed wide width tensile test load-strain curves for UX1500MSE geogrid installed in soil gradation 1.

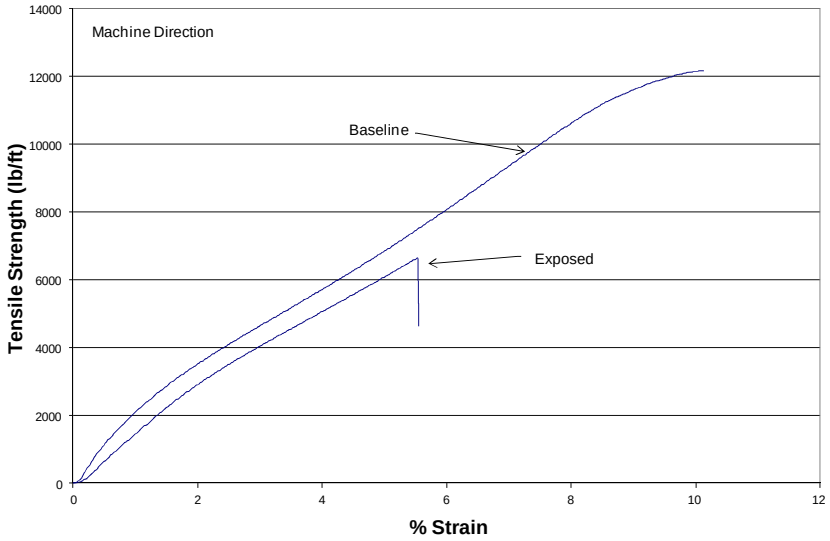

Figure D-3. Example baseline and exposed wide width tensile test load-strain curves for UX1700MSE geogrid installed in soil gradation 1.

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

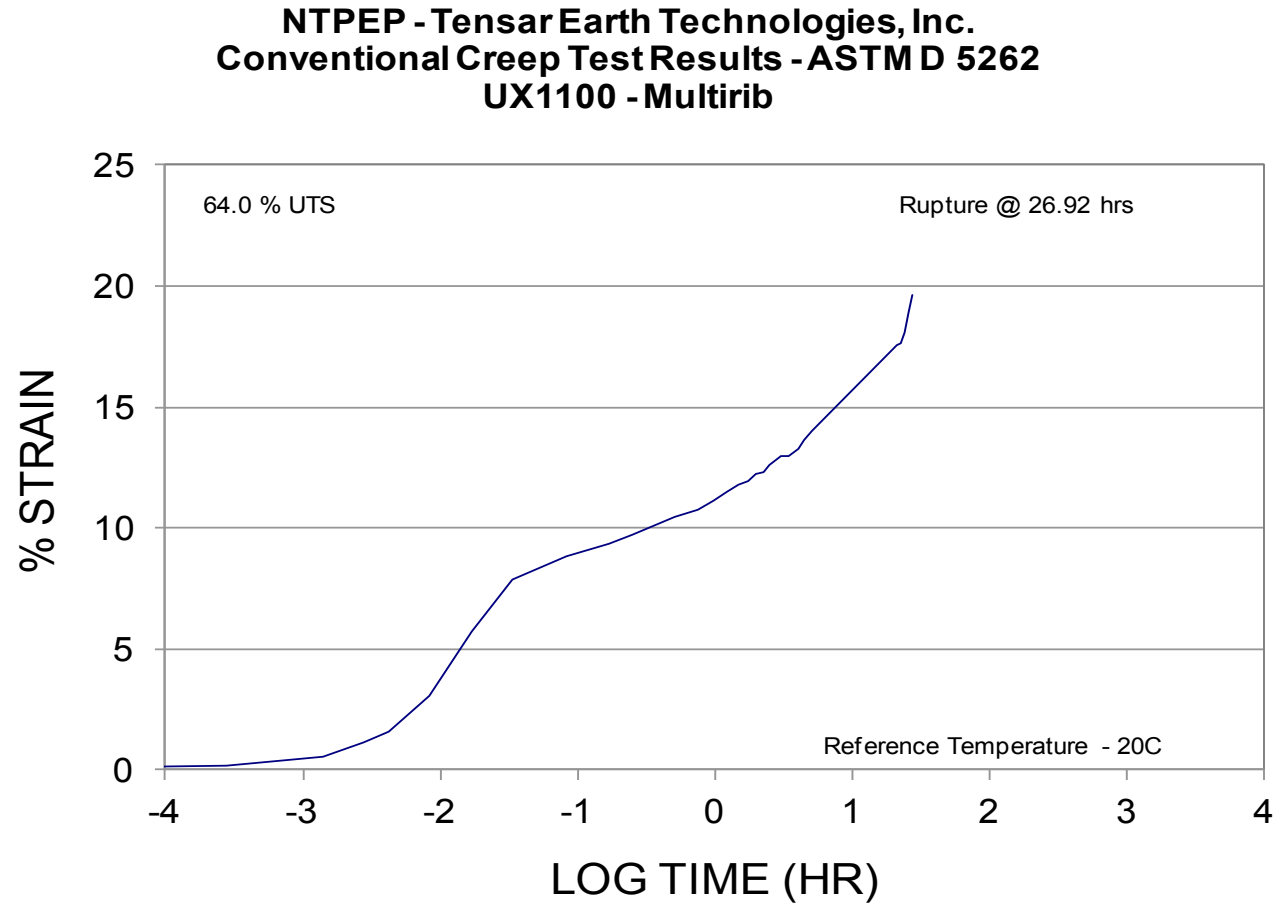
PARAMETER	TEST REPLICATE NUMBER 1	2	3	4	5	MEAN	STD. DEV.	COEF. VARI.	PERCENT RETAINED
Laboratory Installation Damage (ISO/EN 10722)									
Strength Retained measured via wide width tensile (ISO/EN 10319)									
MD Number of Ribs per Specimen:	7								
MD Number of Ribs per foot:	14.04								
MD - 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)

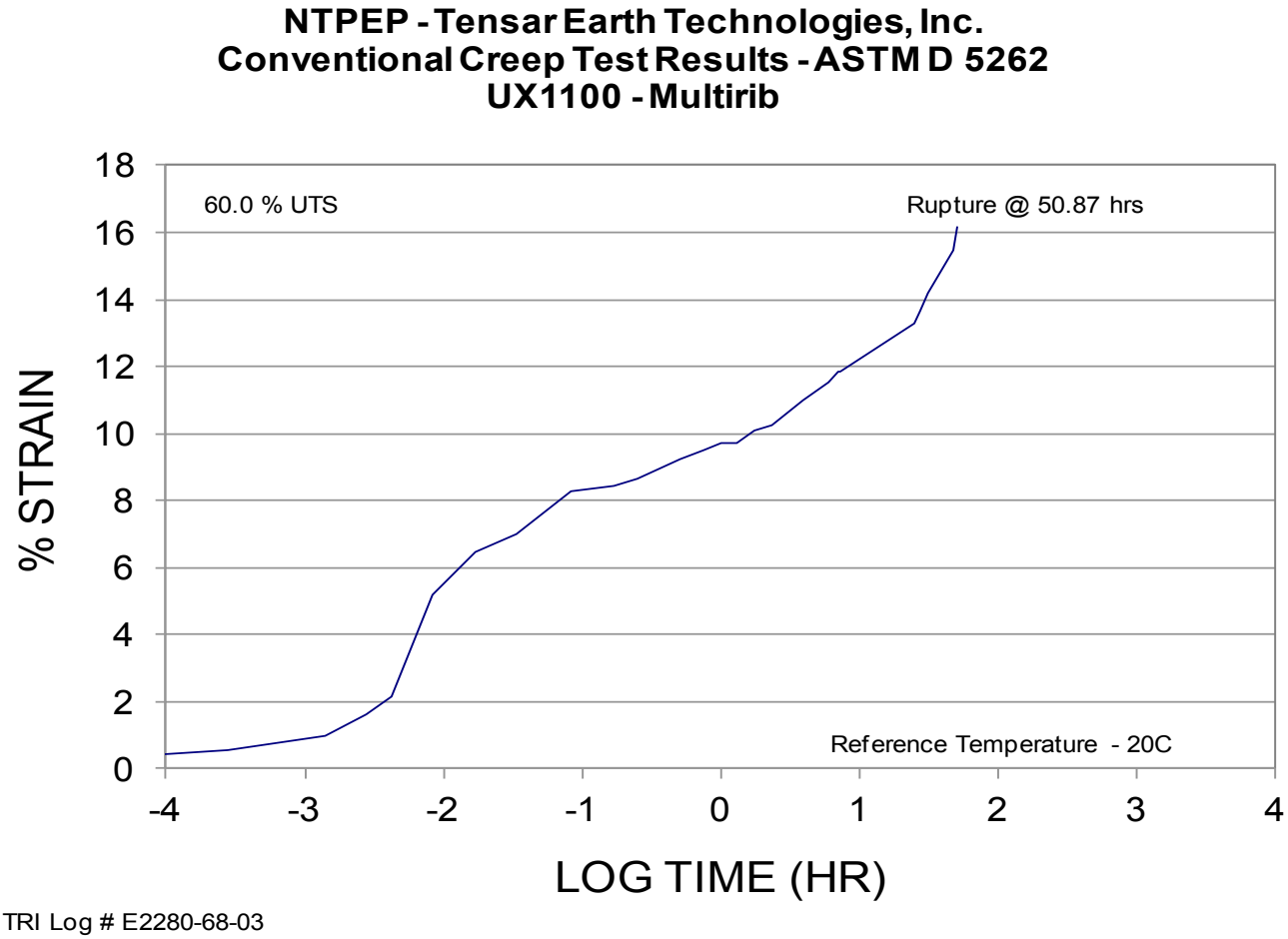
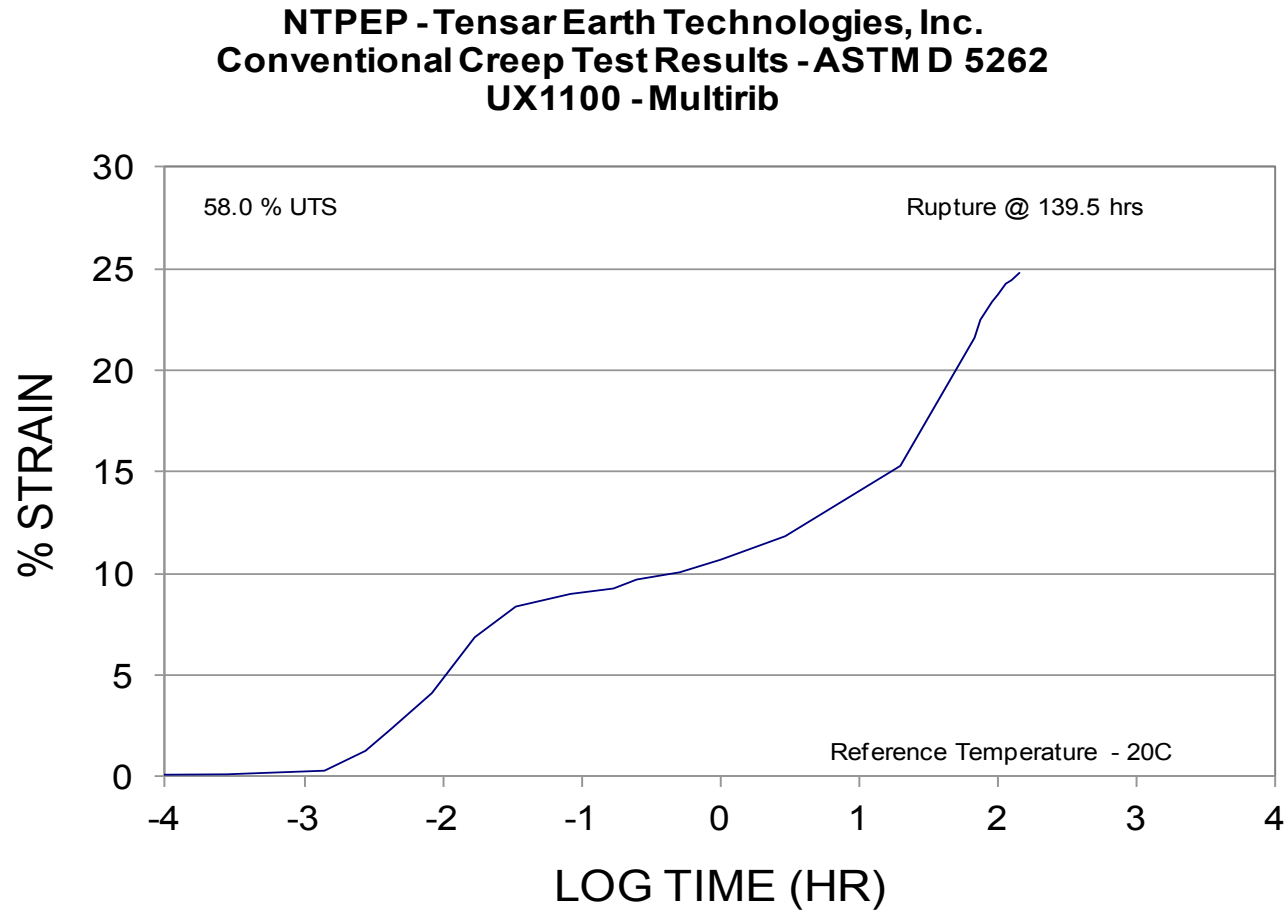
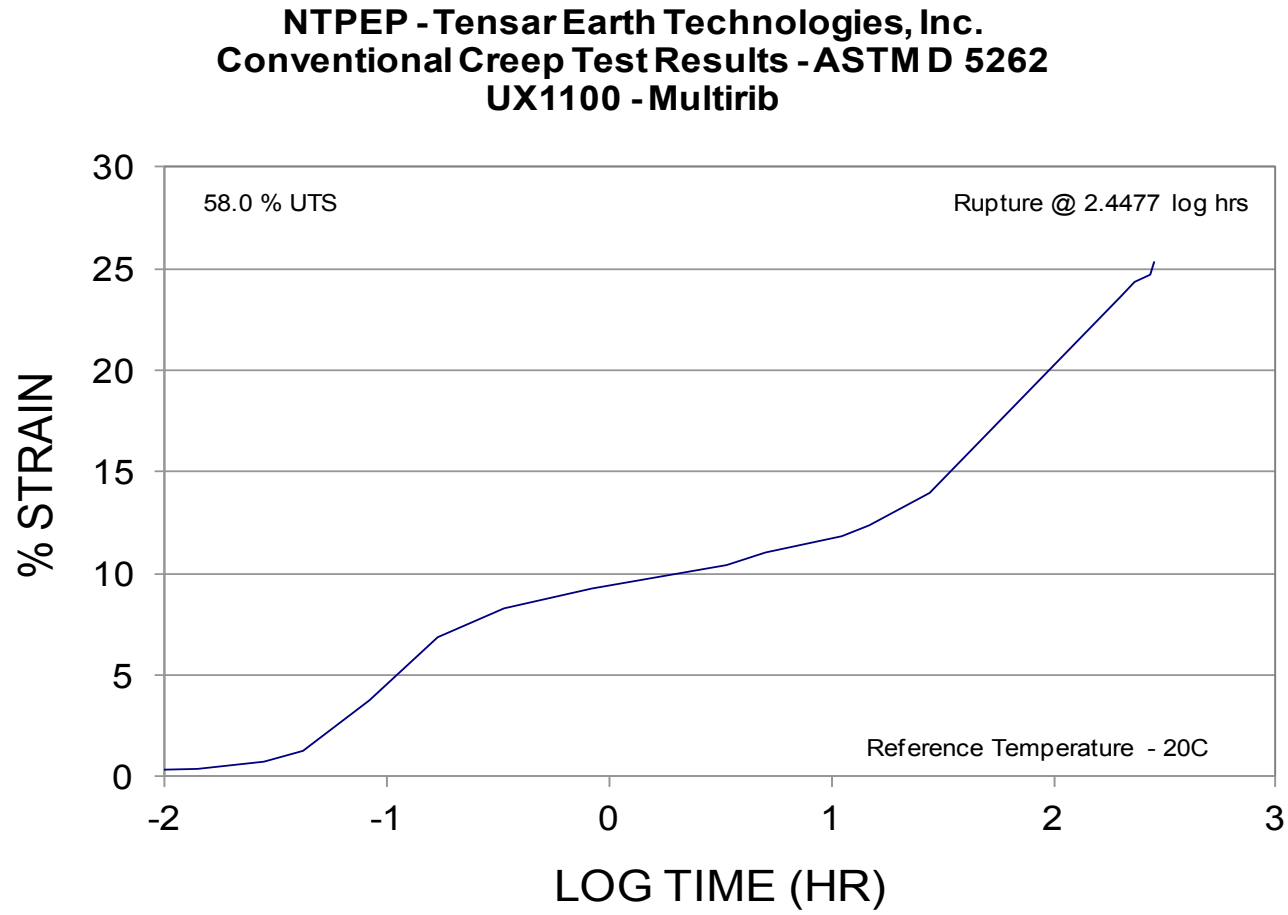


Figure F-2. Creep data/curve per ASTM D5262, multirib, for Tensar UX1100 at a load level of 60.0% UTS and 68°F(20°C)



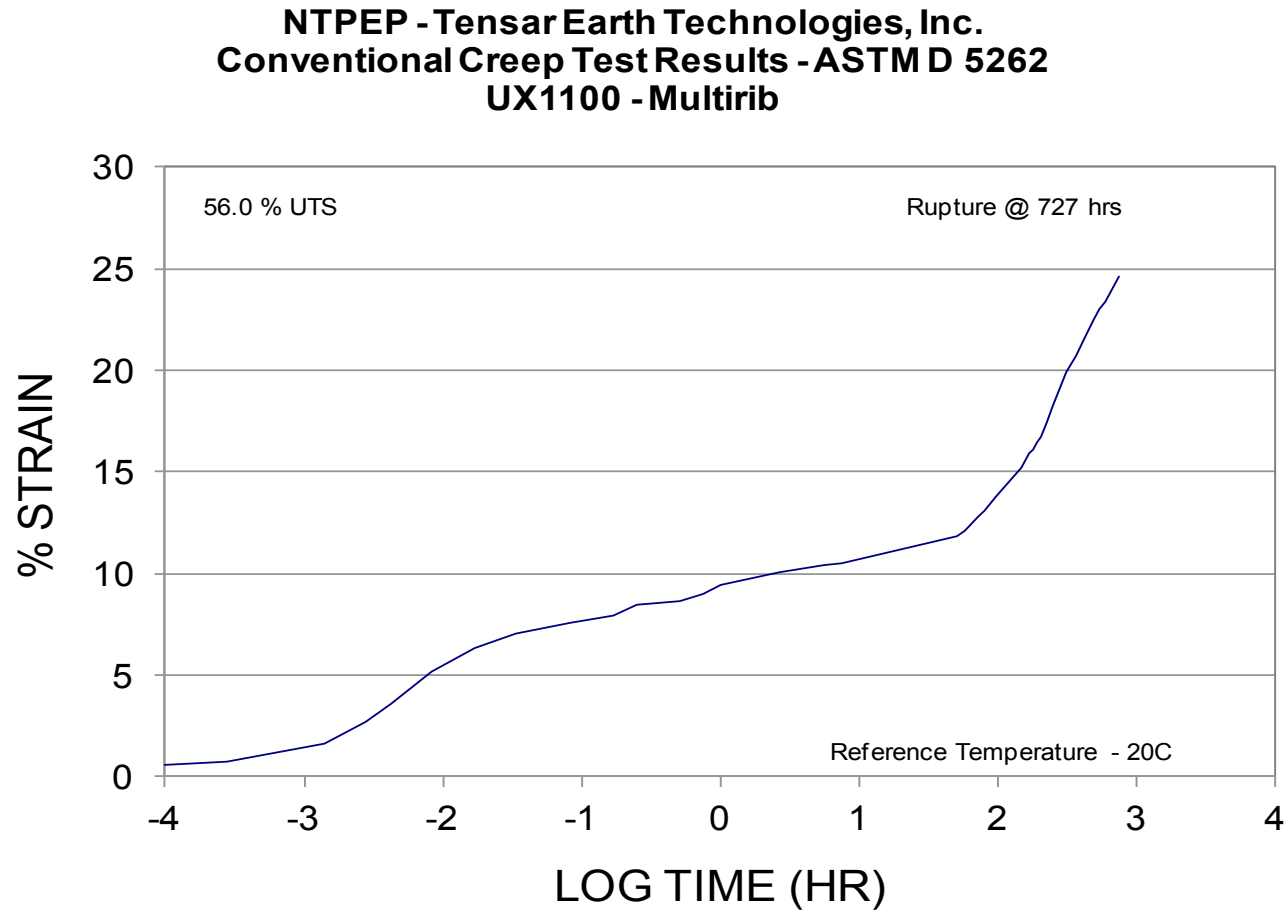
TRI Log # E2280-68-03

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)



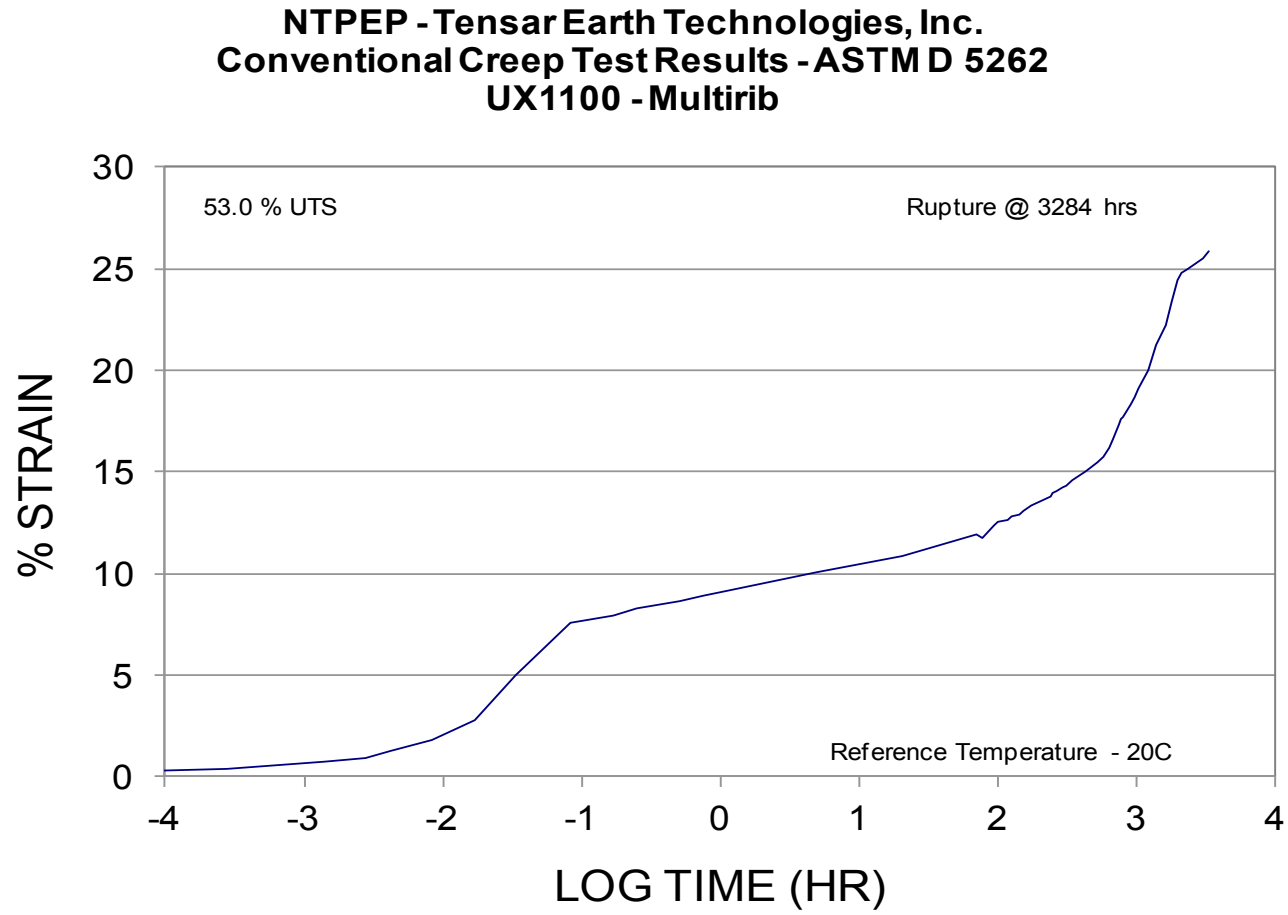
TRI Log # E2280-68-03

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)



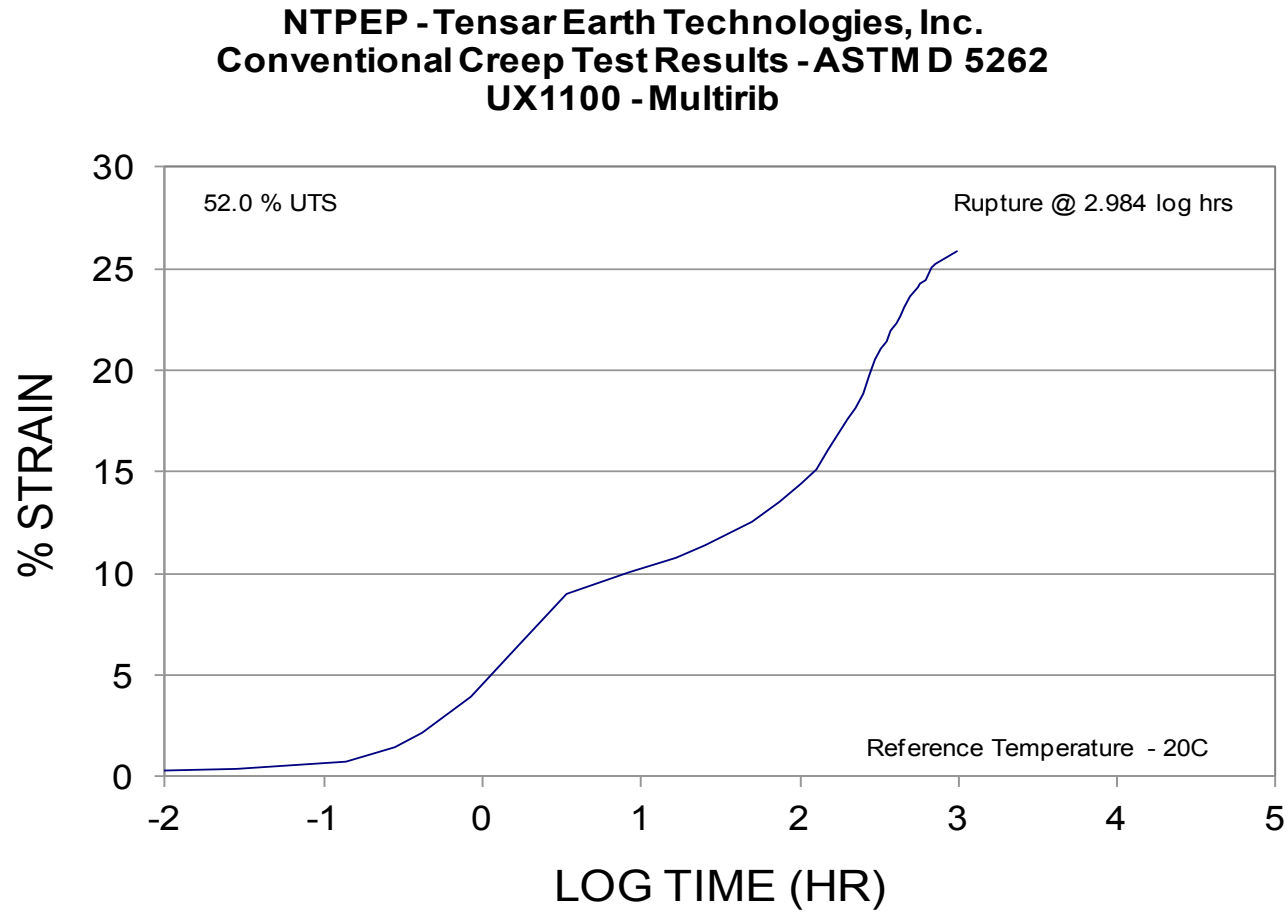
TRI Log # E2280-68-03

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)



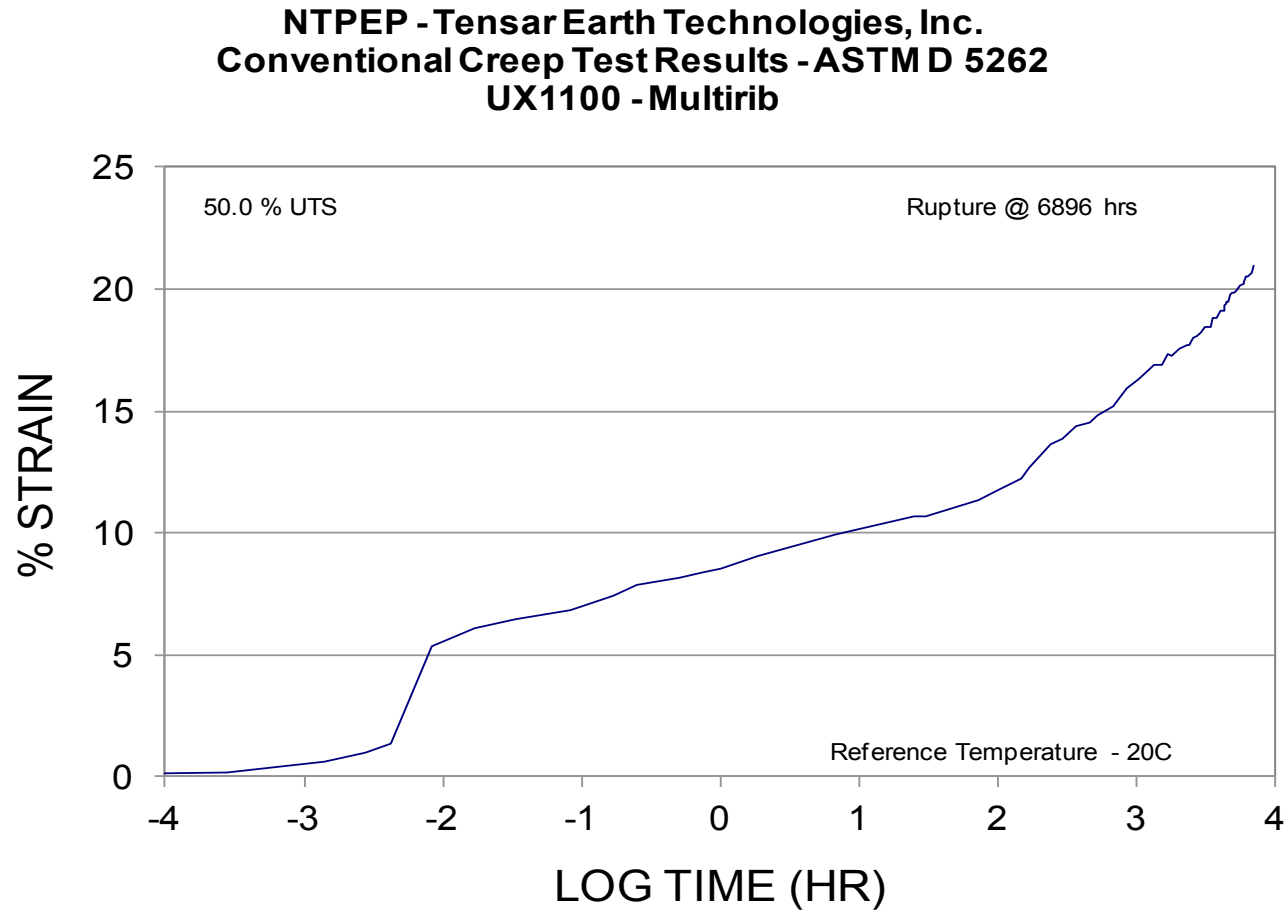
TRI Log # E2280-68-03

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)



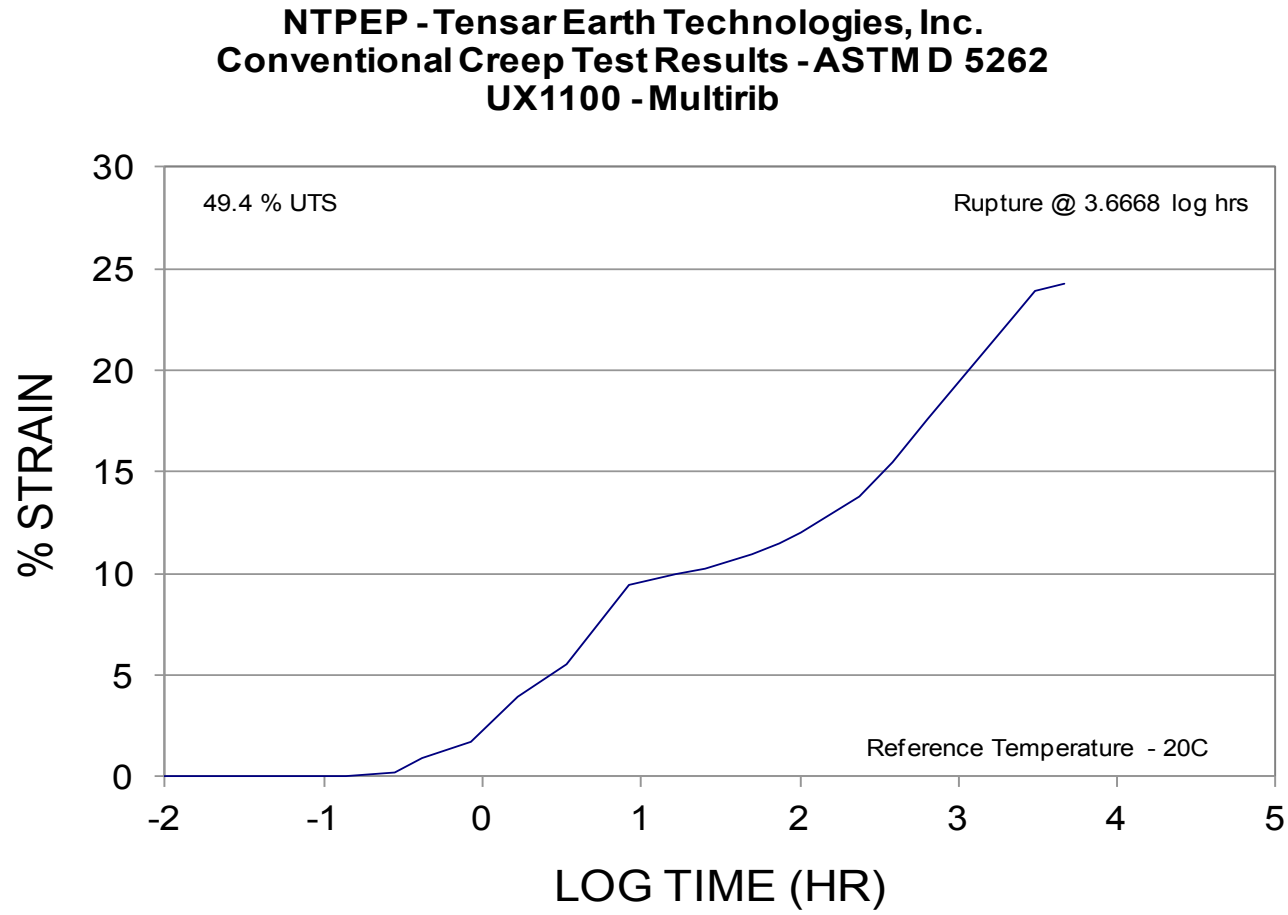
TRI Log # E2280-68-03

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)



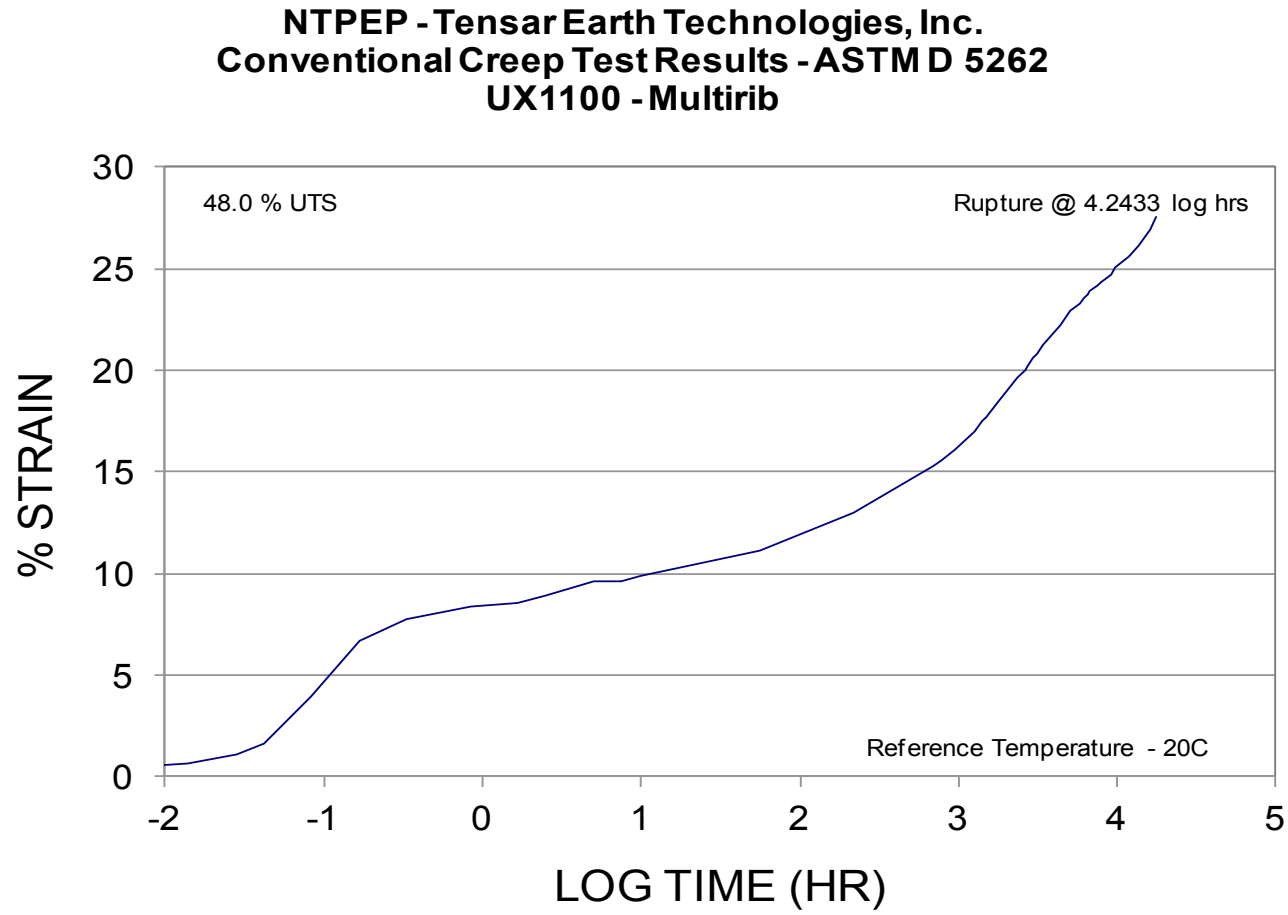
TRI Log # E2280-68-03

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)



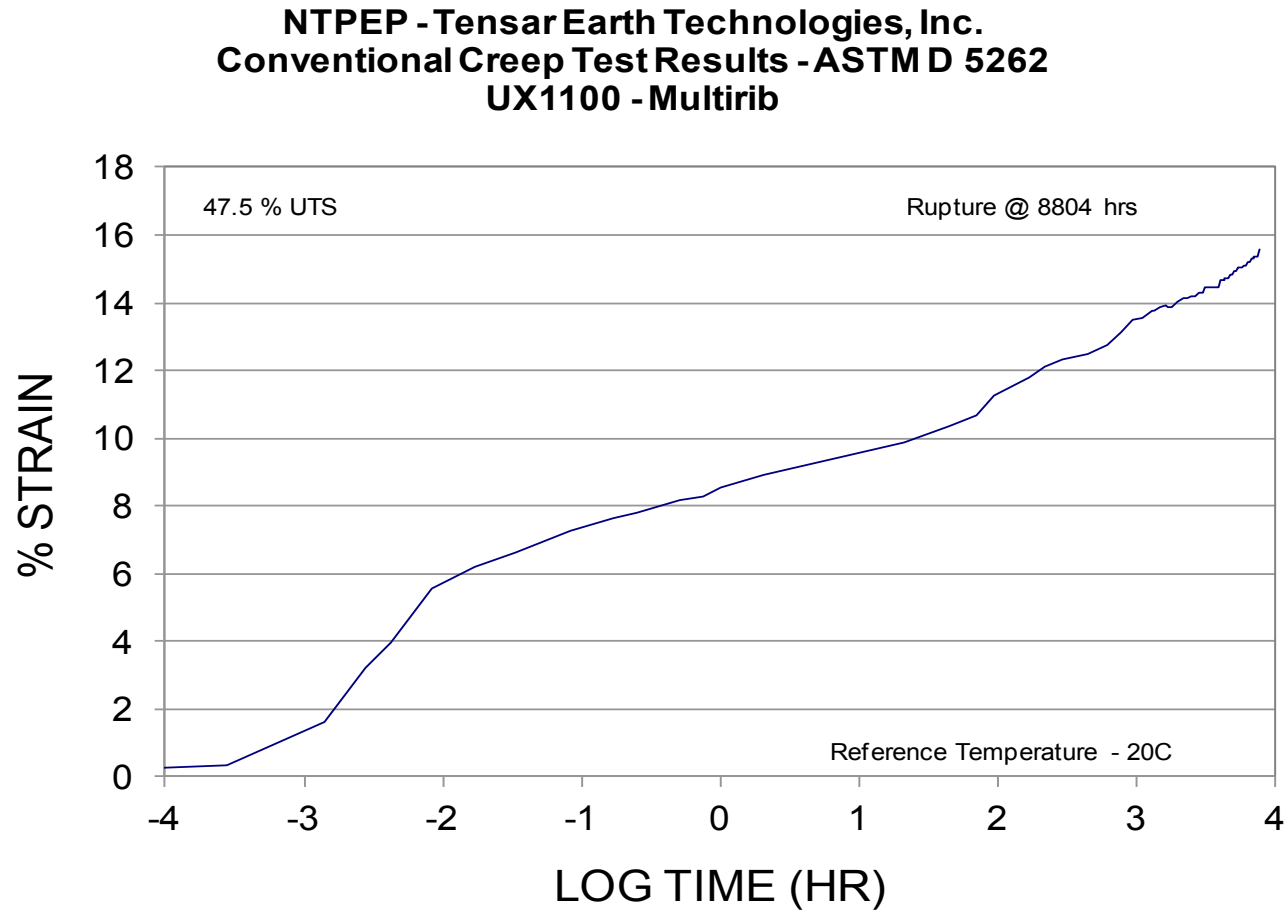
TRI Log # E2280-68-03

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)



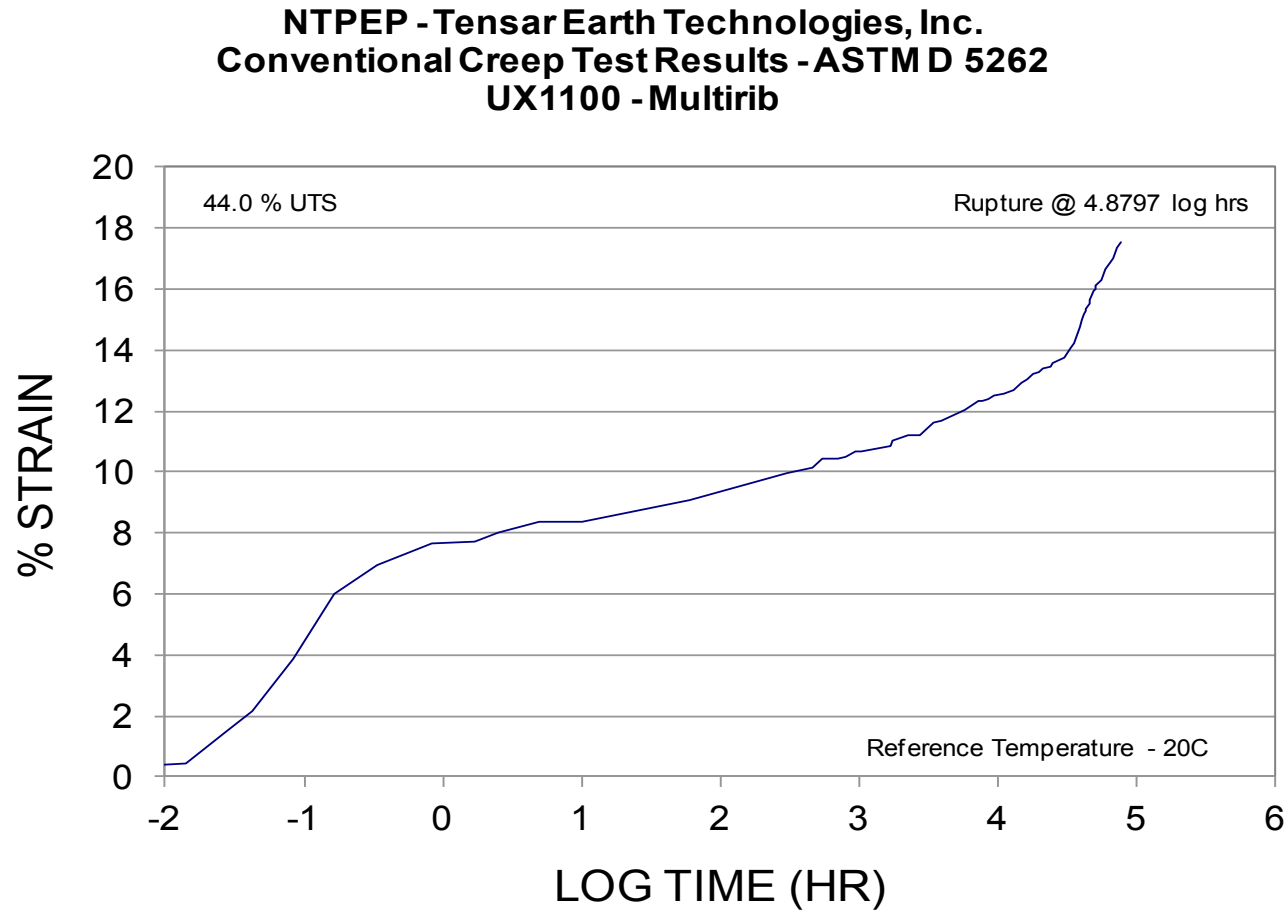
TRI Log # E2280-68-03

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)



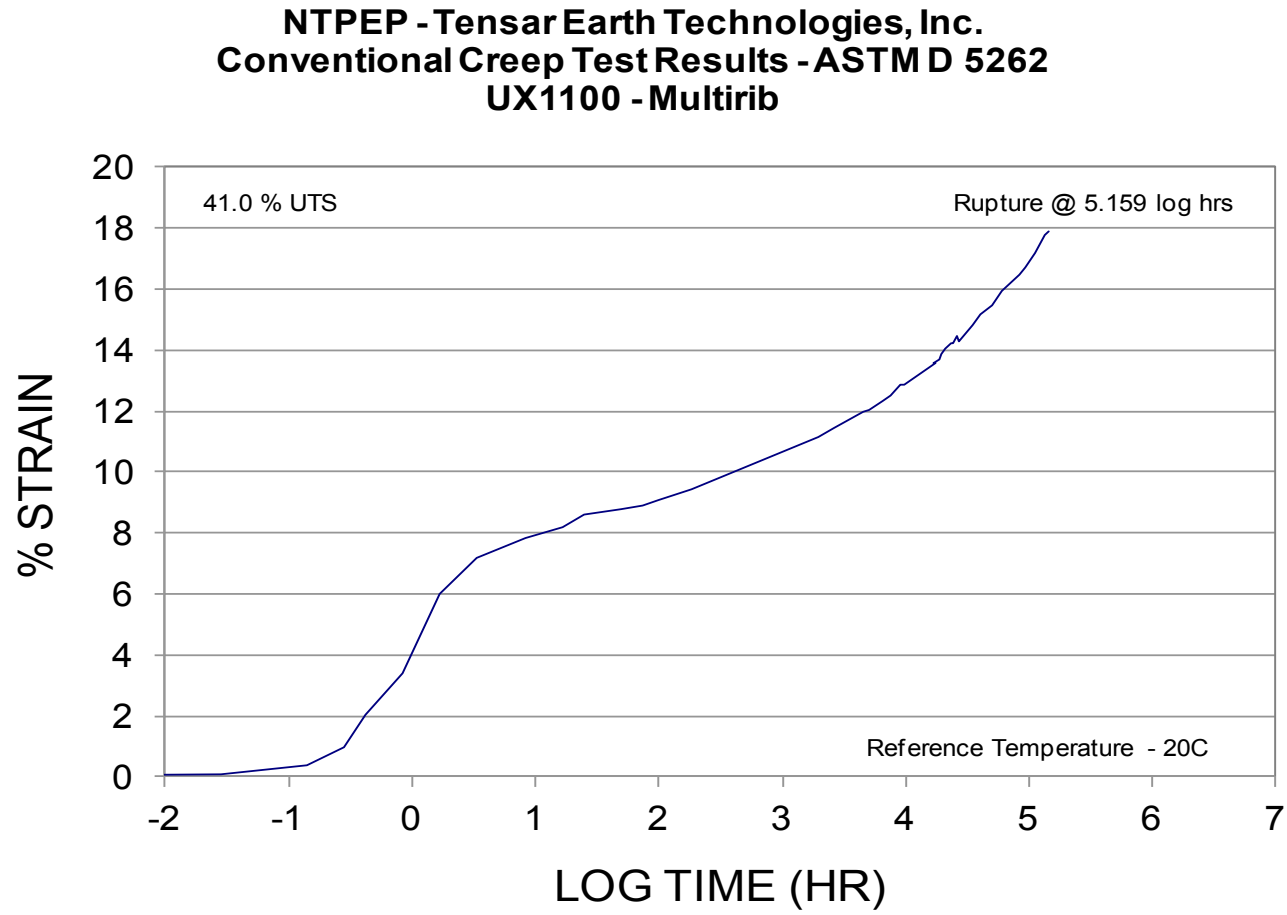
TRI Log # E2280-68-03

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)

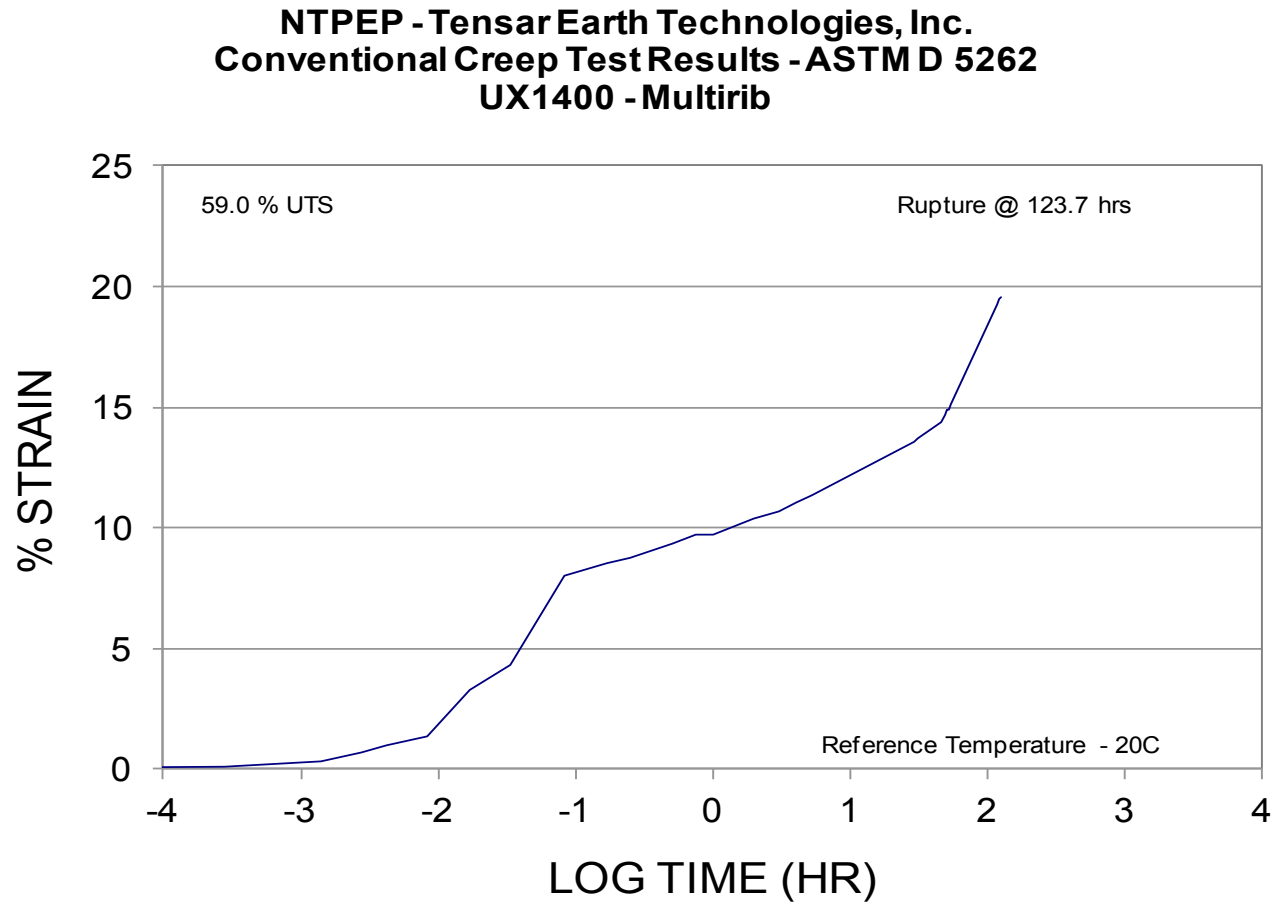
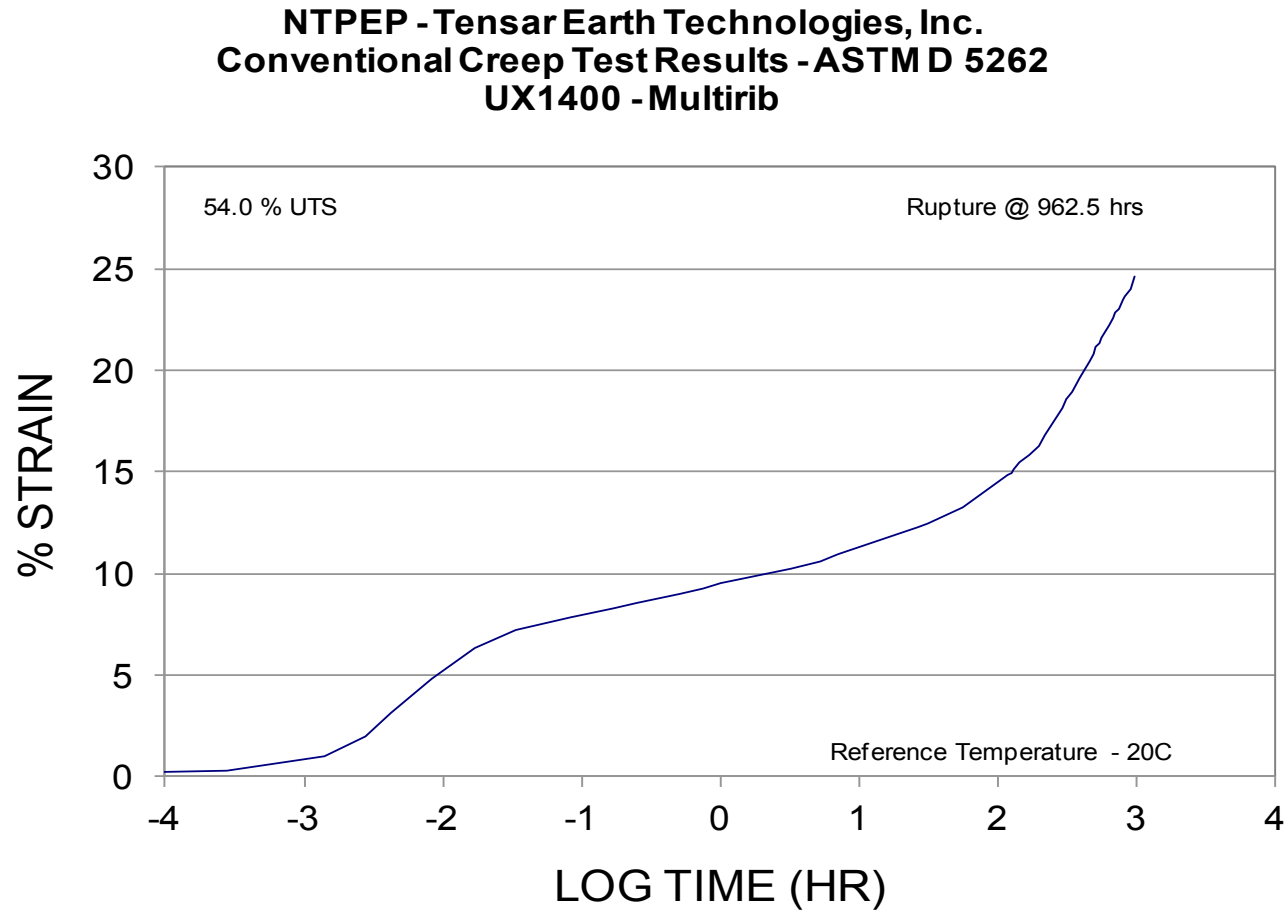
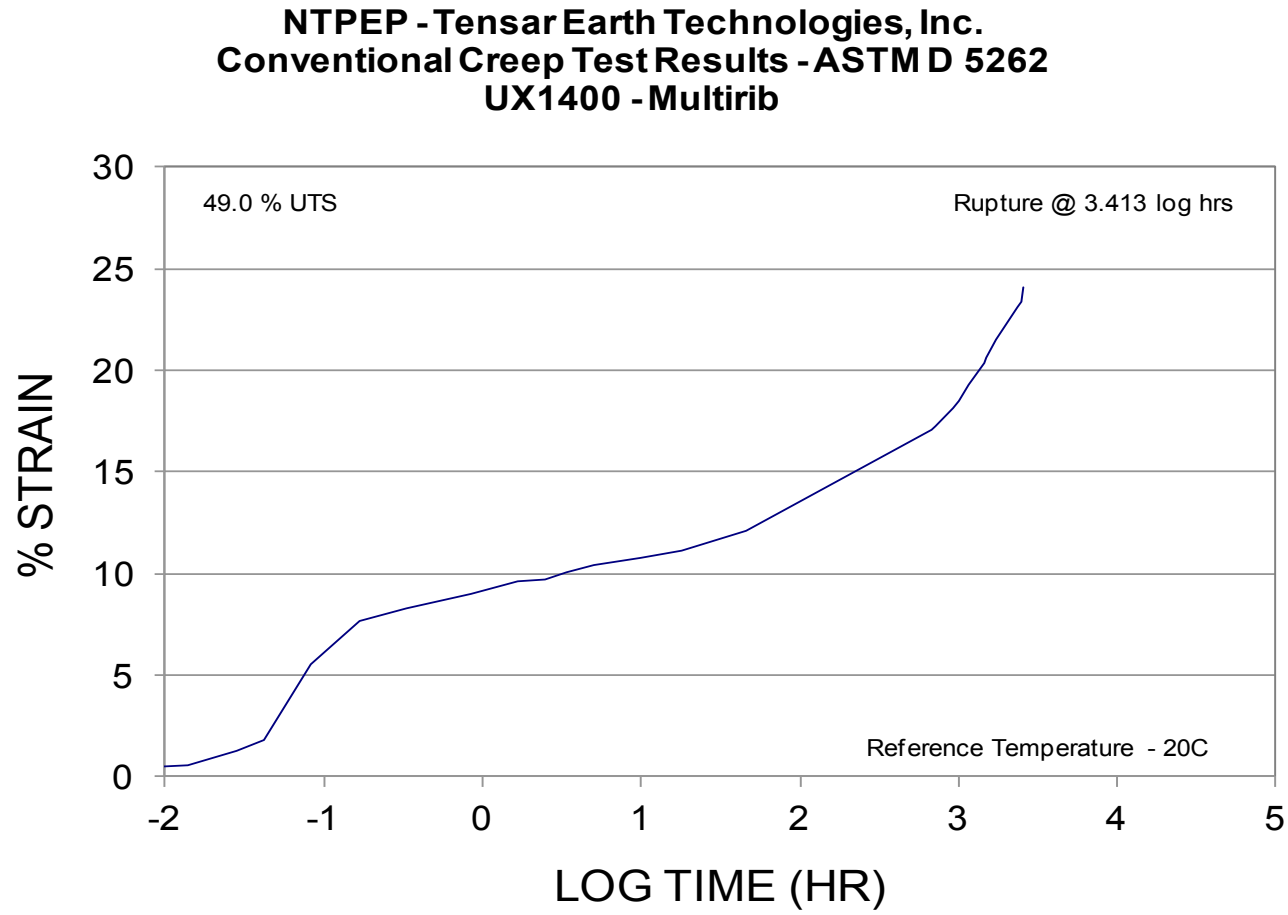


Figure F-14. Creep data/curve per ASTM D5262, multirib, for Tensar UX1400 at a load level of 59.0% UTS and 68°F(20°C)



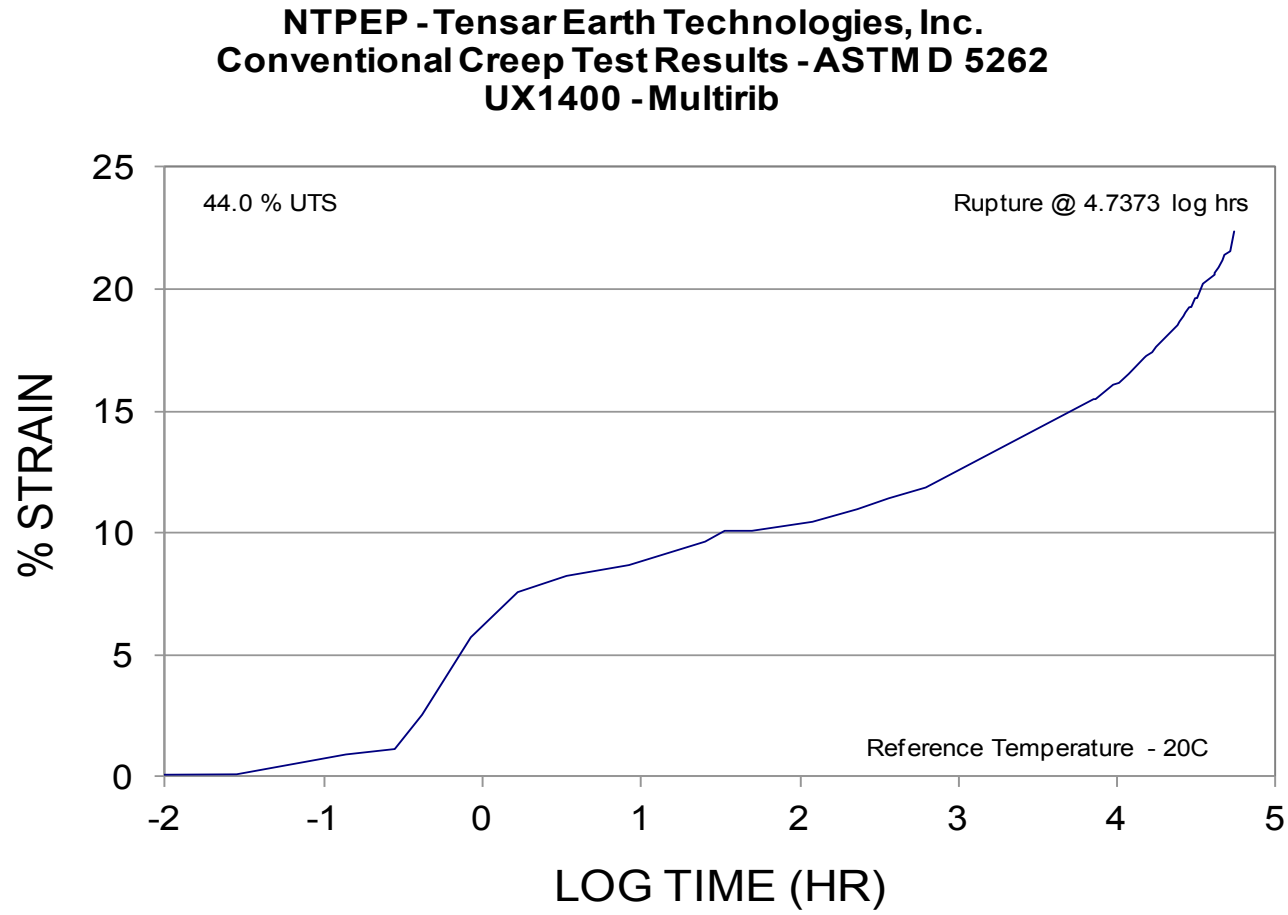
TRI Log # E2280-68-03

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)



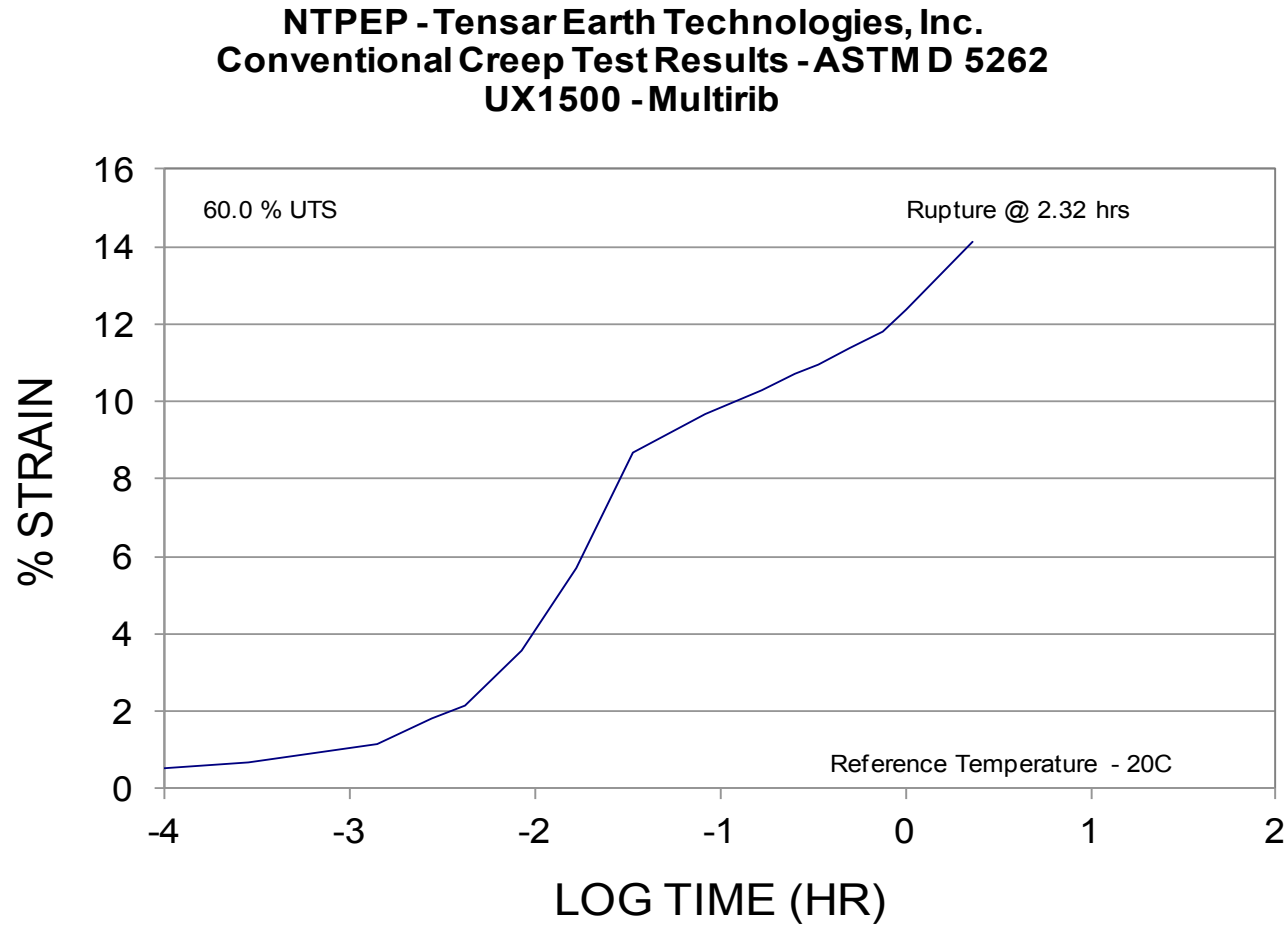
TRI Log # E2280-68-03

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)



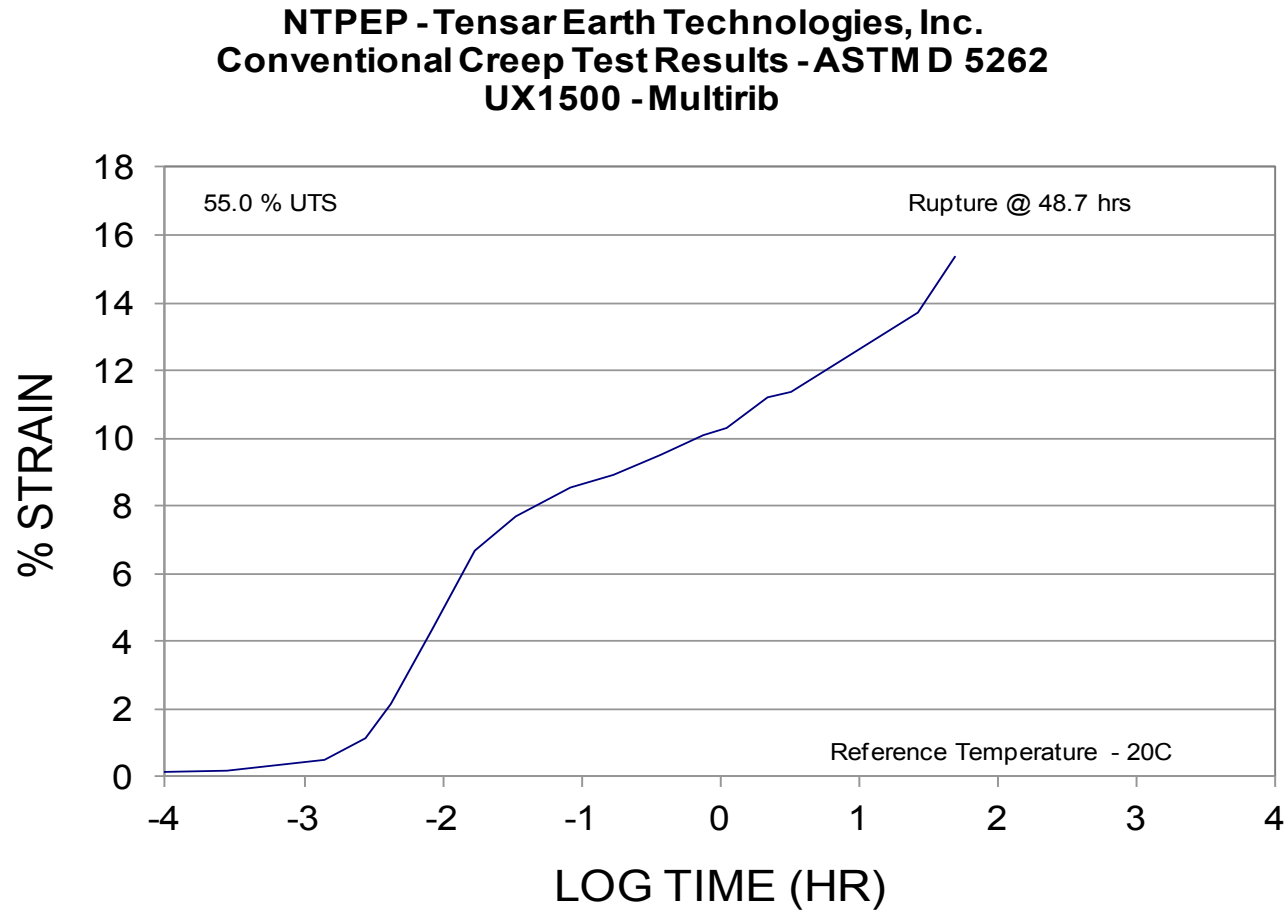
TRI Log # E2280-68-03

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)



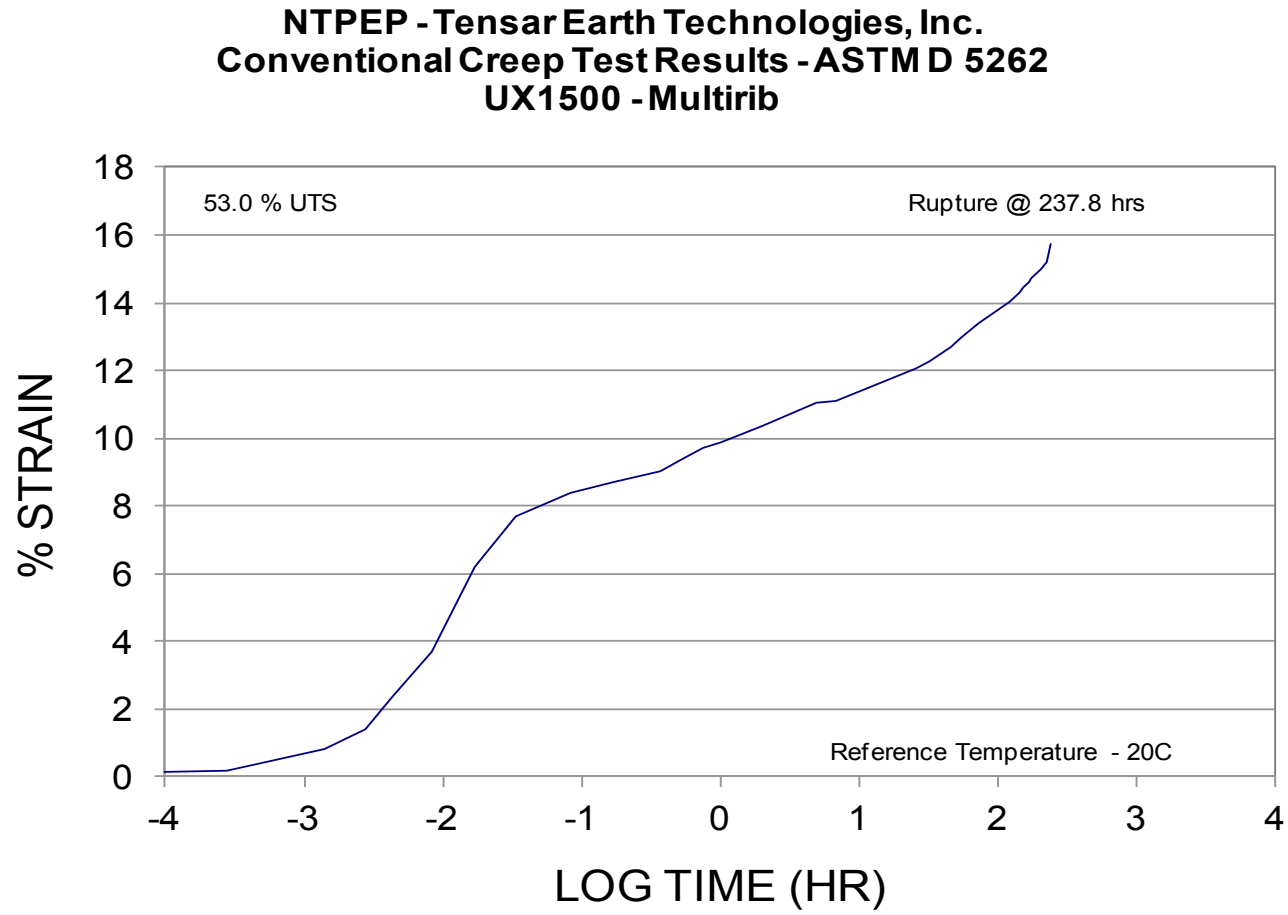
TRI Log # E2280-68-03

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)



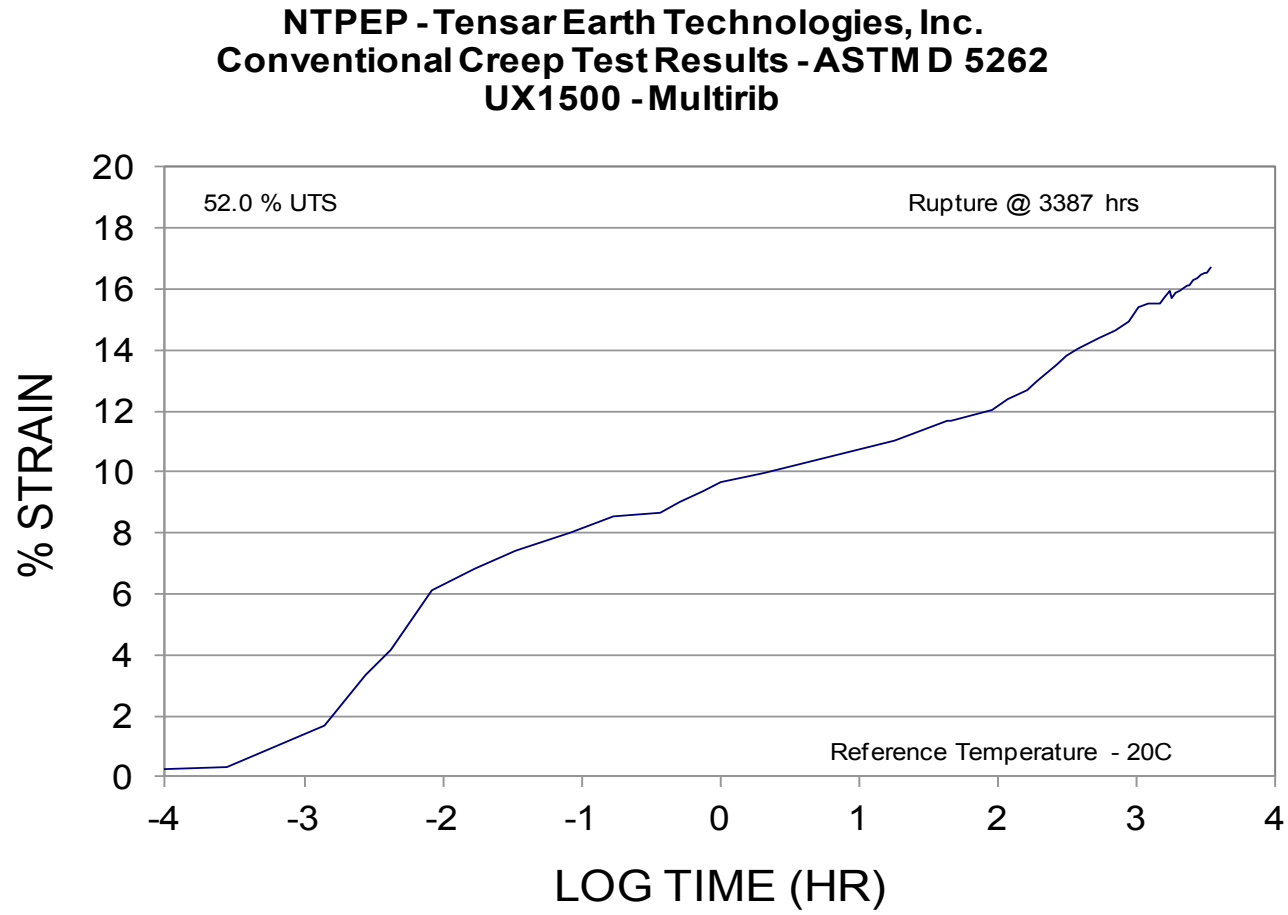
TRI Log # E2280-68-03

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)



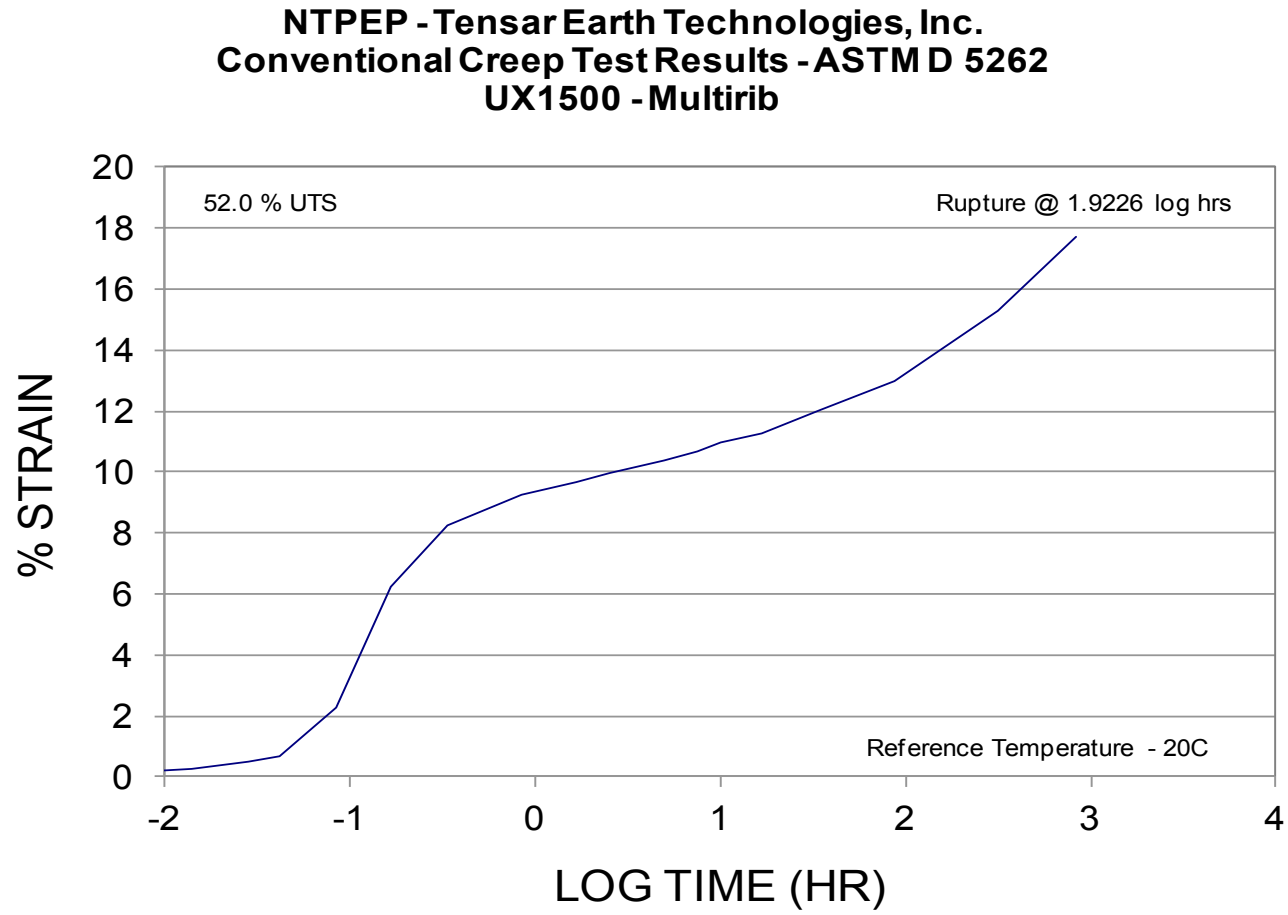
TRI Log # E2280-68-03

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)



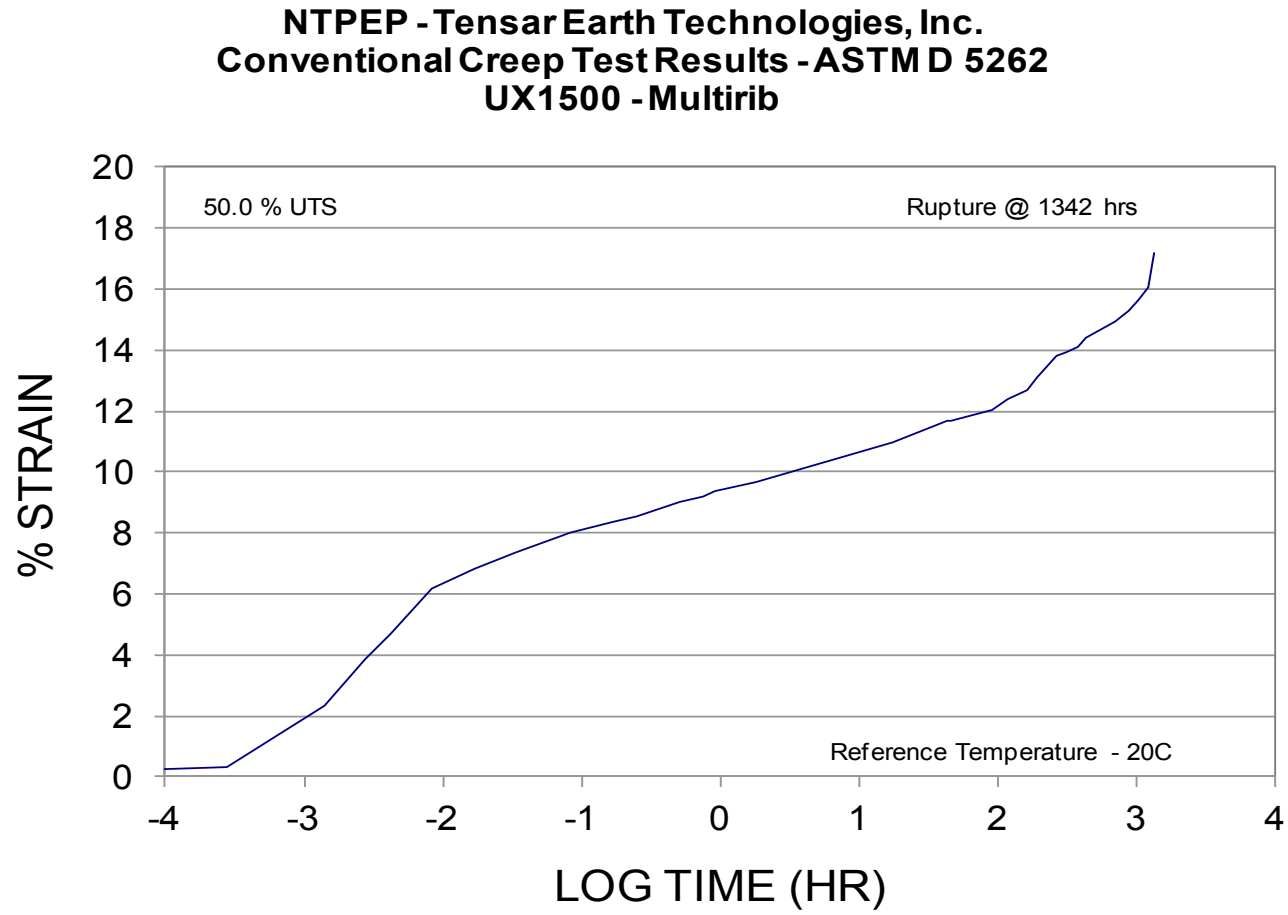
TRI Log # E2280-68-03

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)



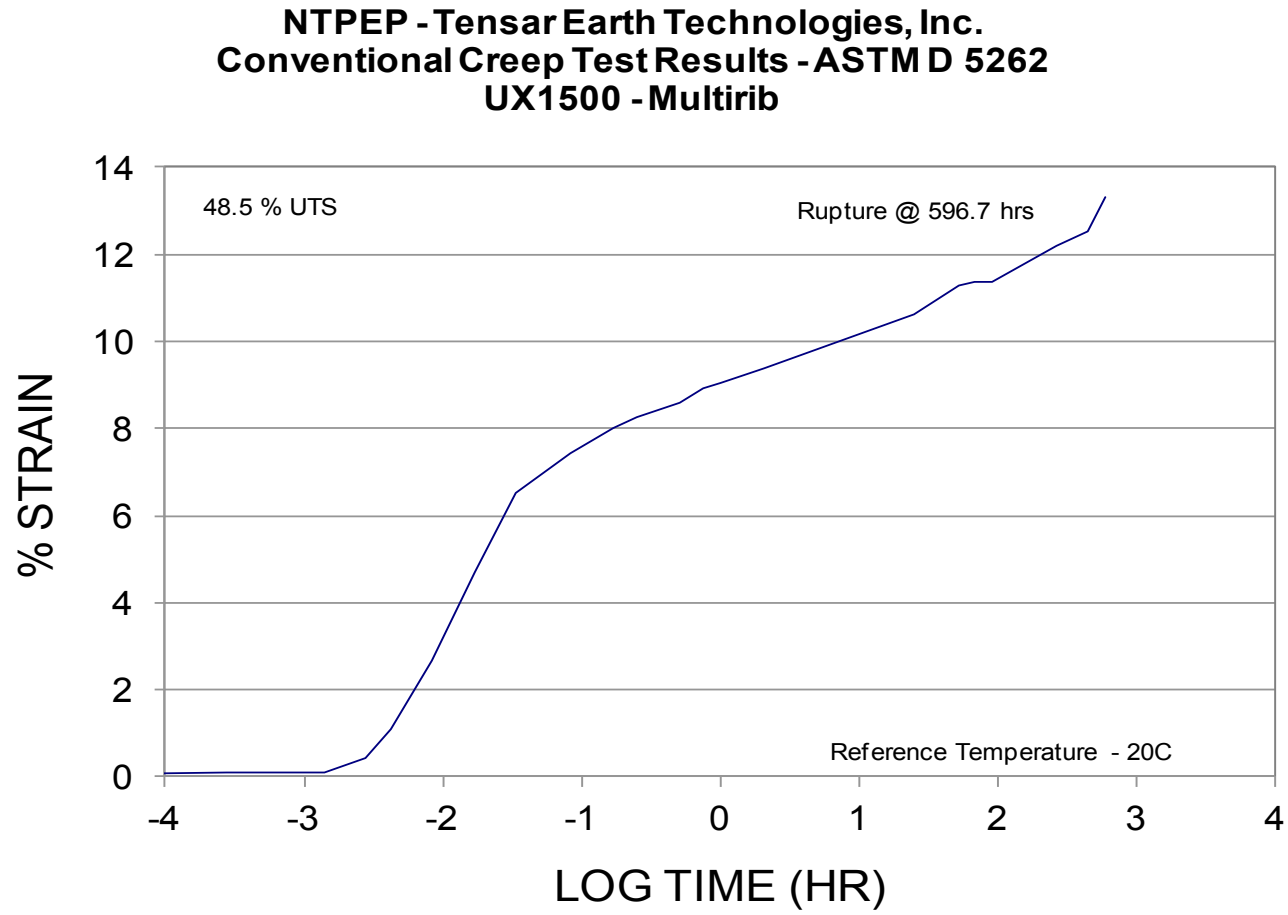
TRI Log # E2280-68-03

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)



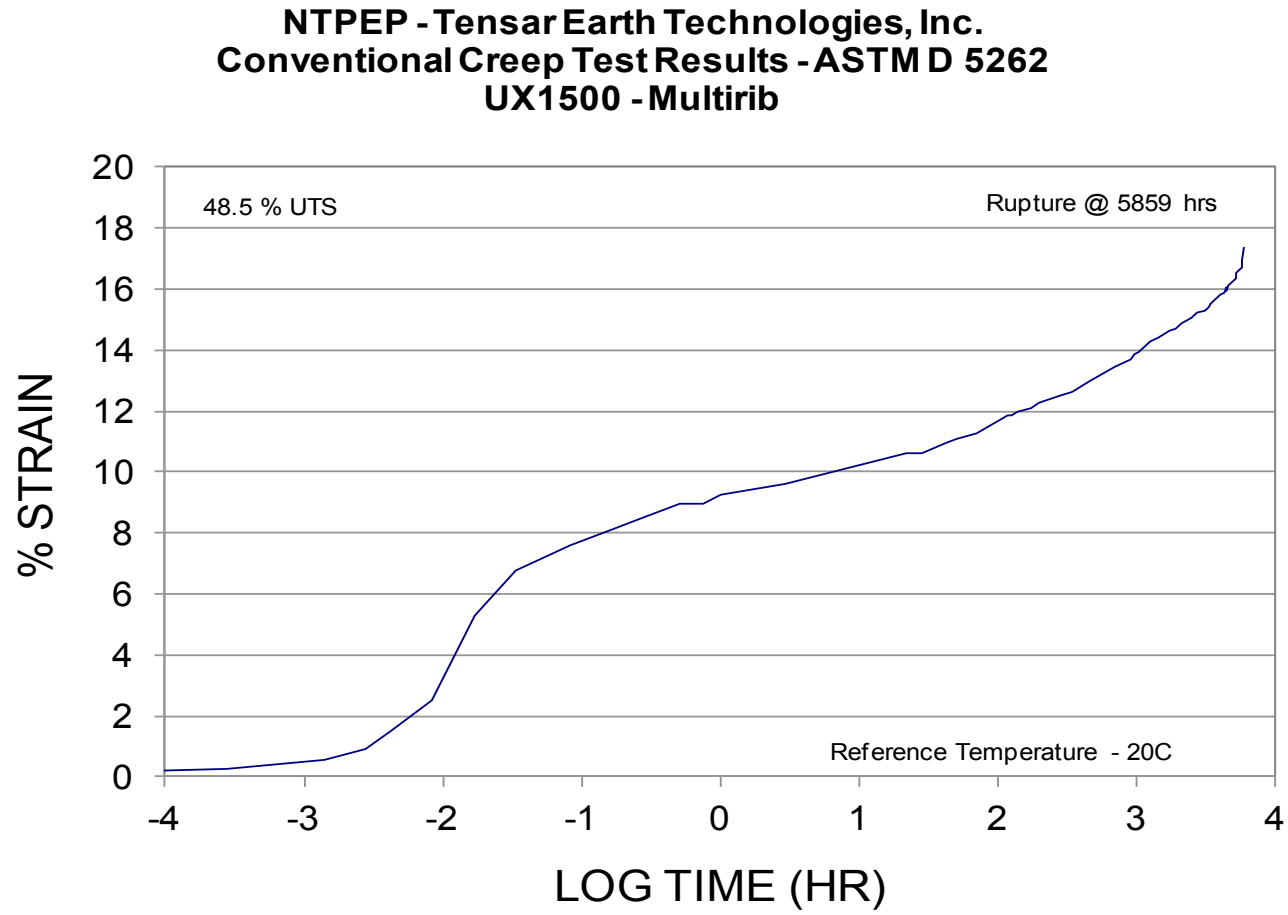
TRI Log # E2280-68-03

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)



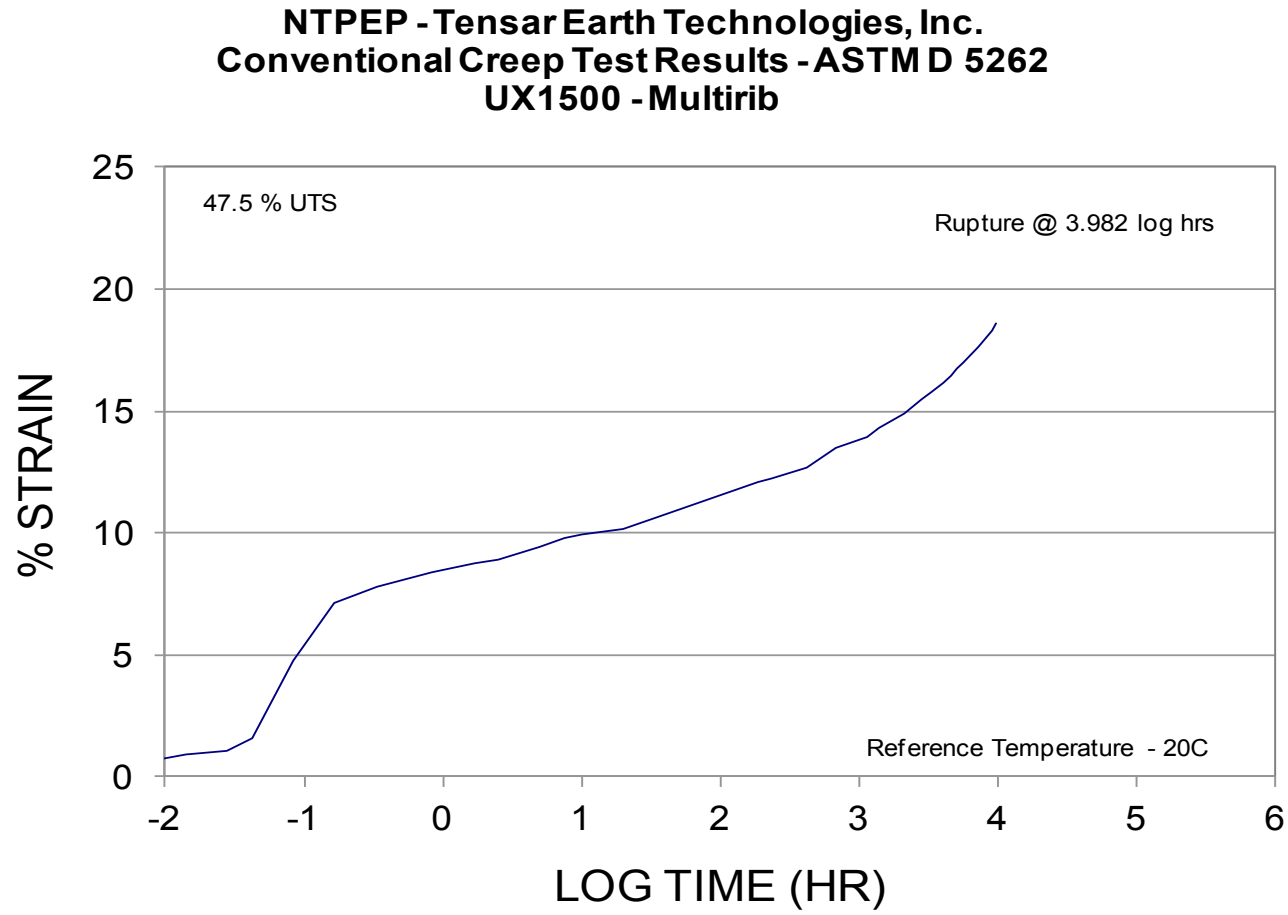
TRI Log # E2280-68-03

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)



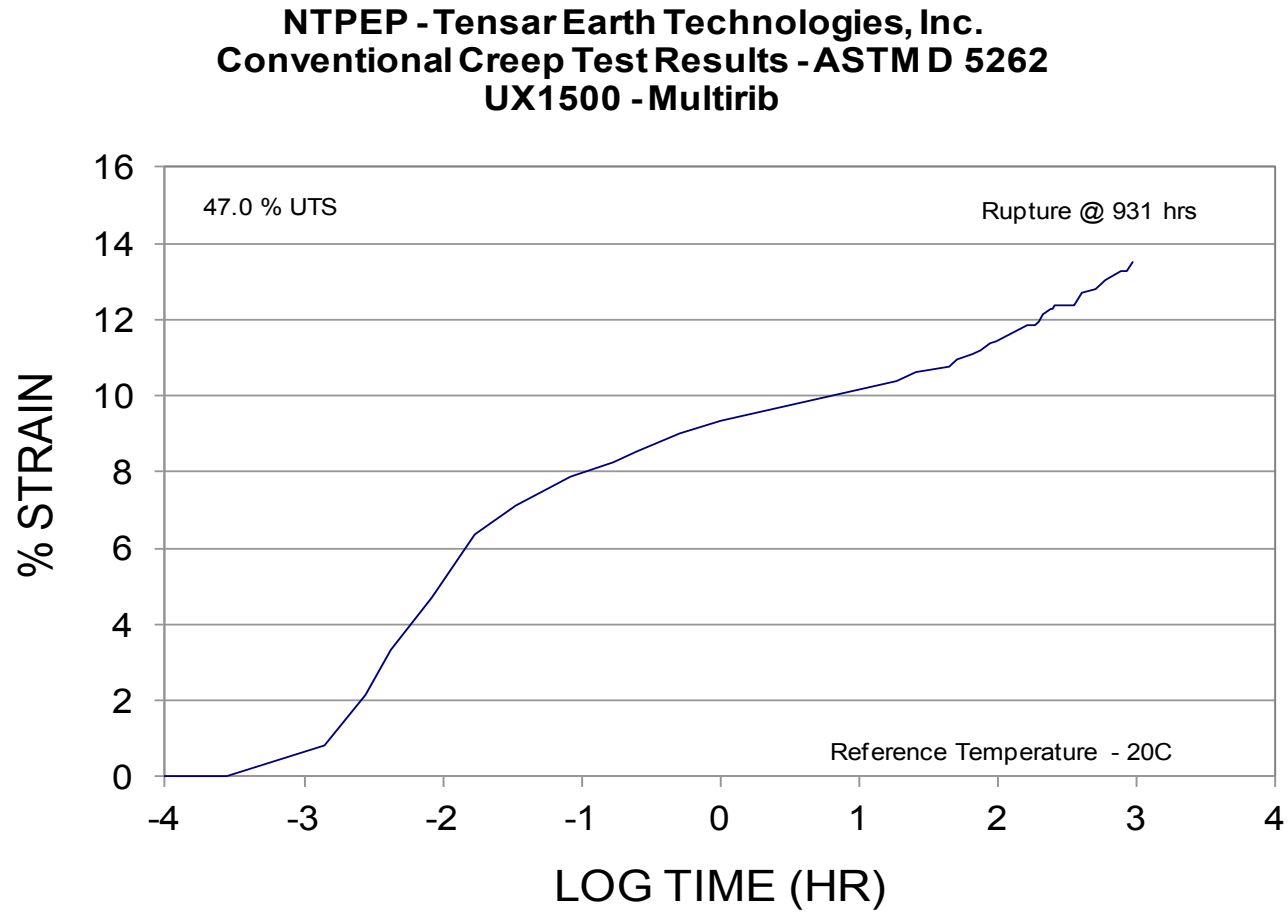
TRI Log # E2280-68-03

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)



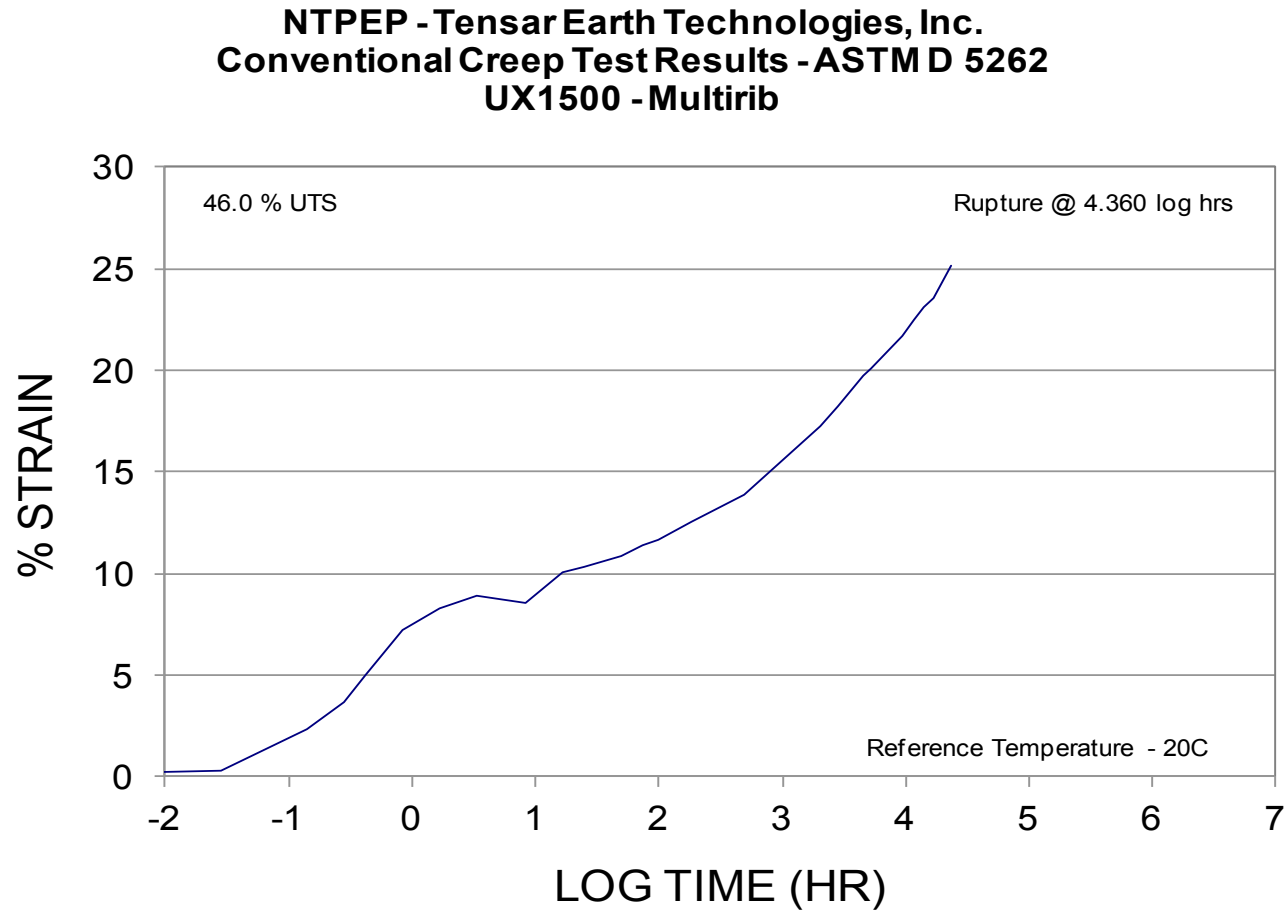
TRI Log # E2280-68-03

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)



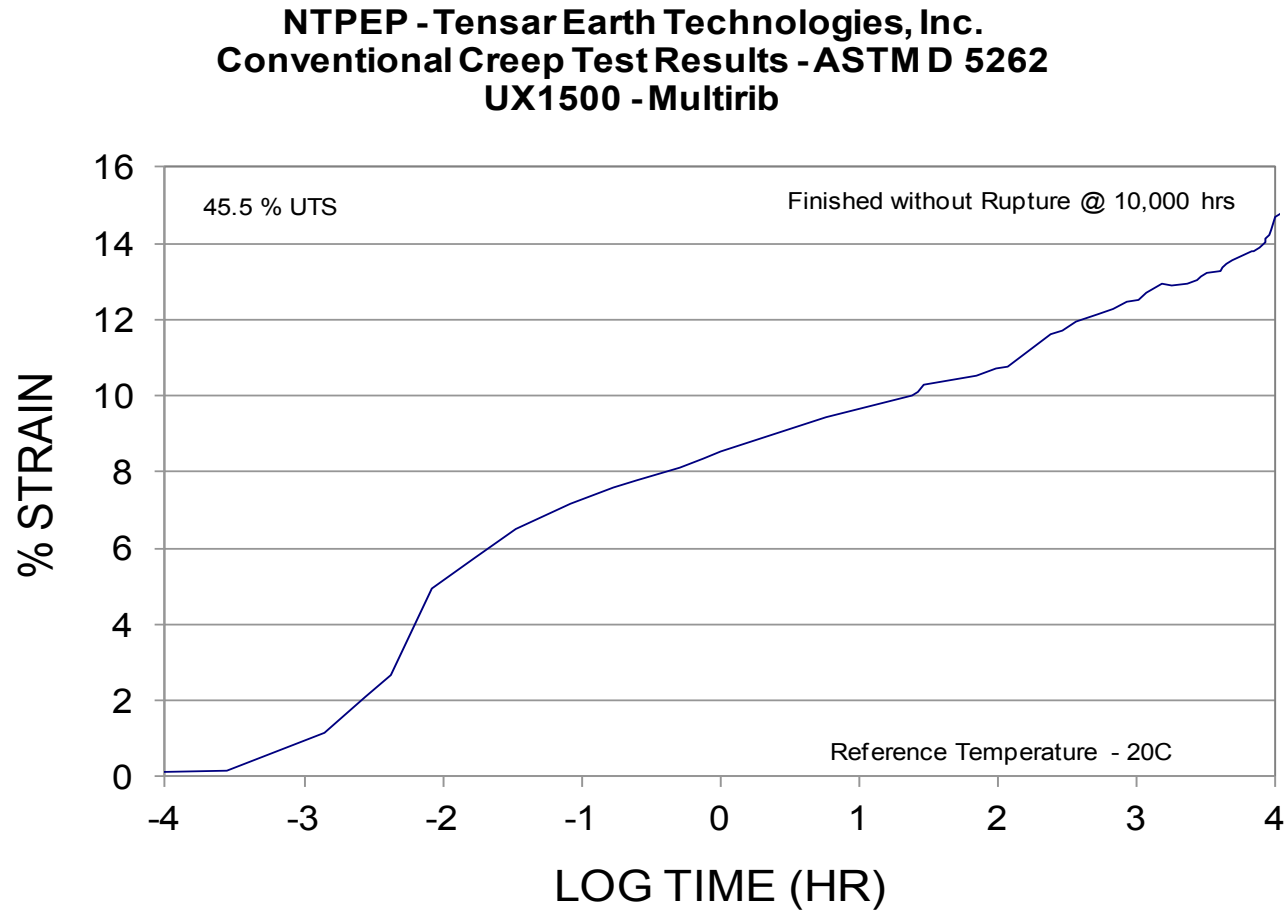
TRI Log # E2280-68-03

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)



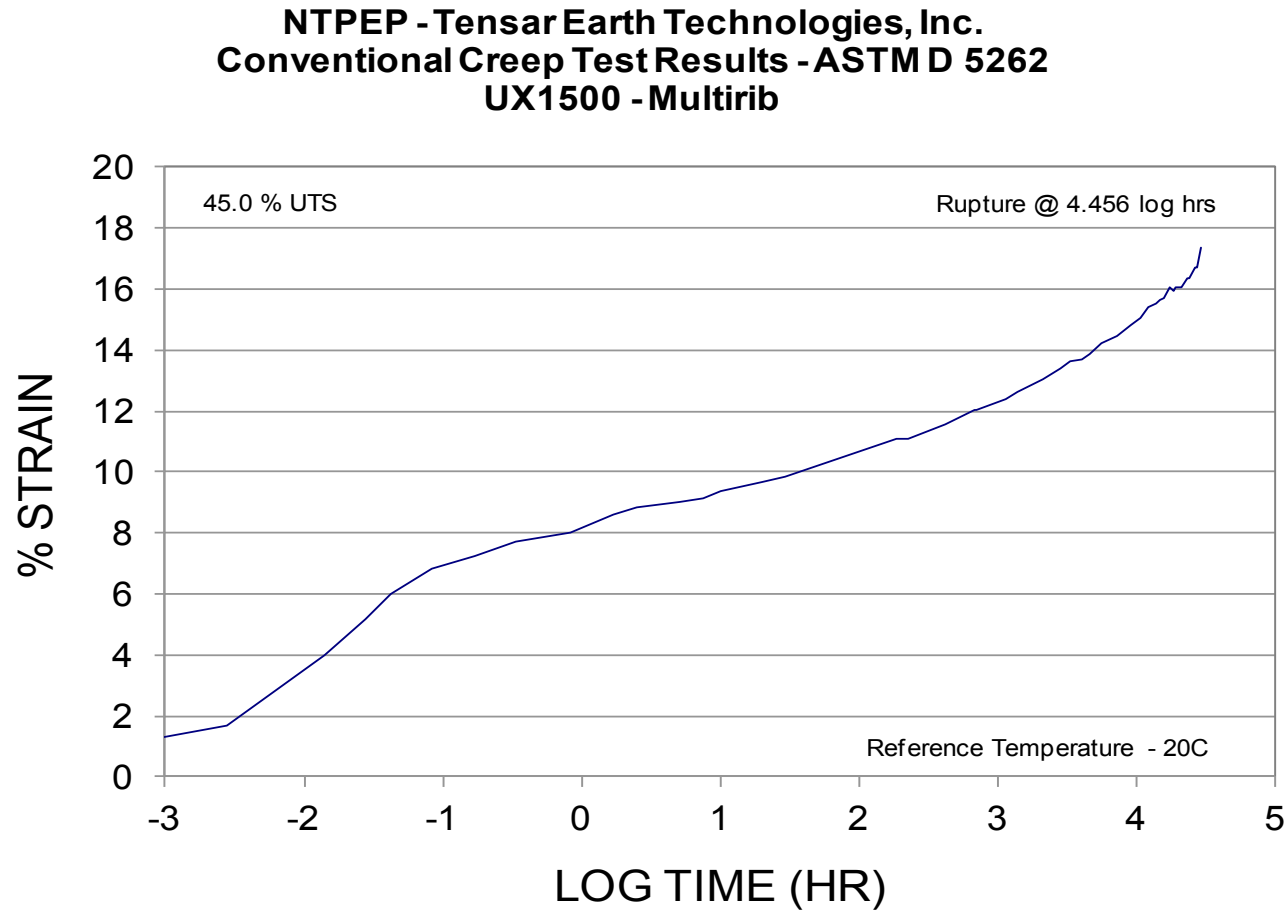
TRI Log # E2280-68-03

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)



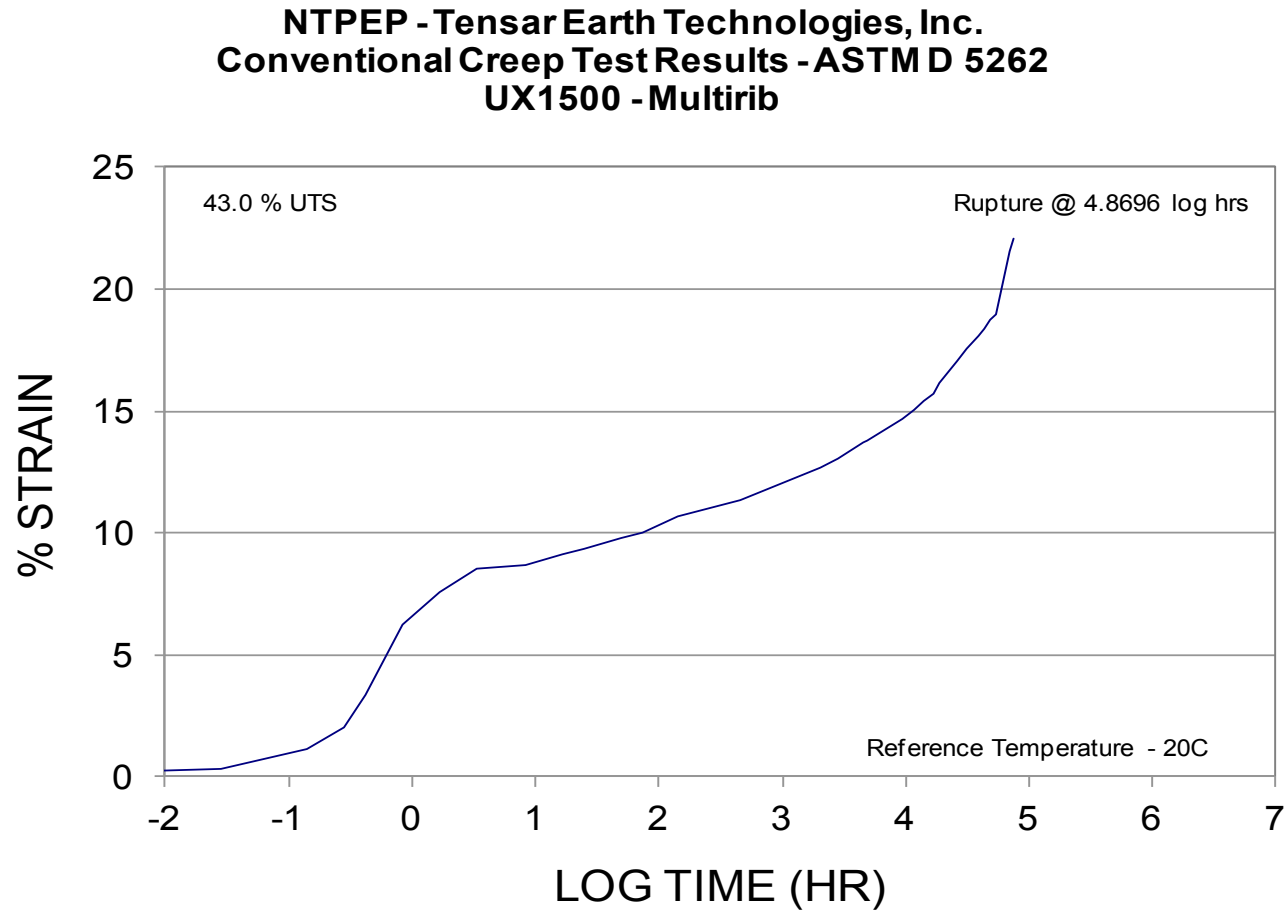
TRI Log # E2280-68-03

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)



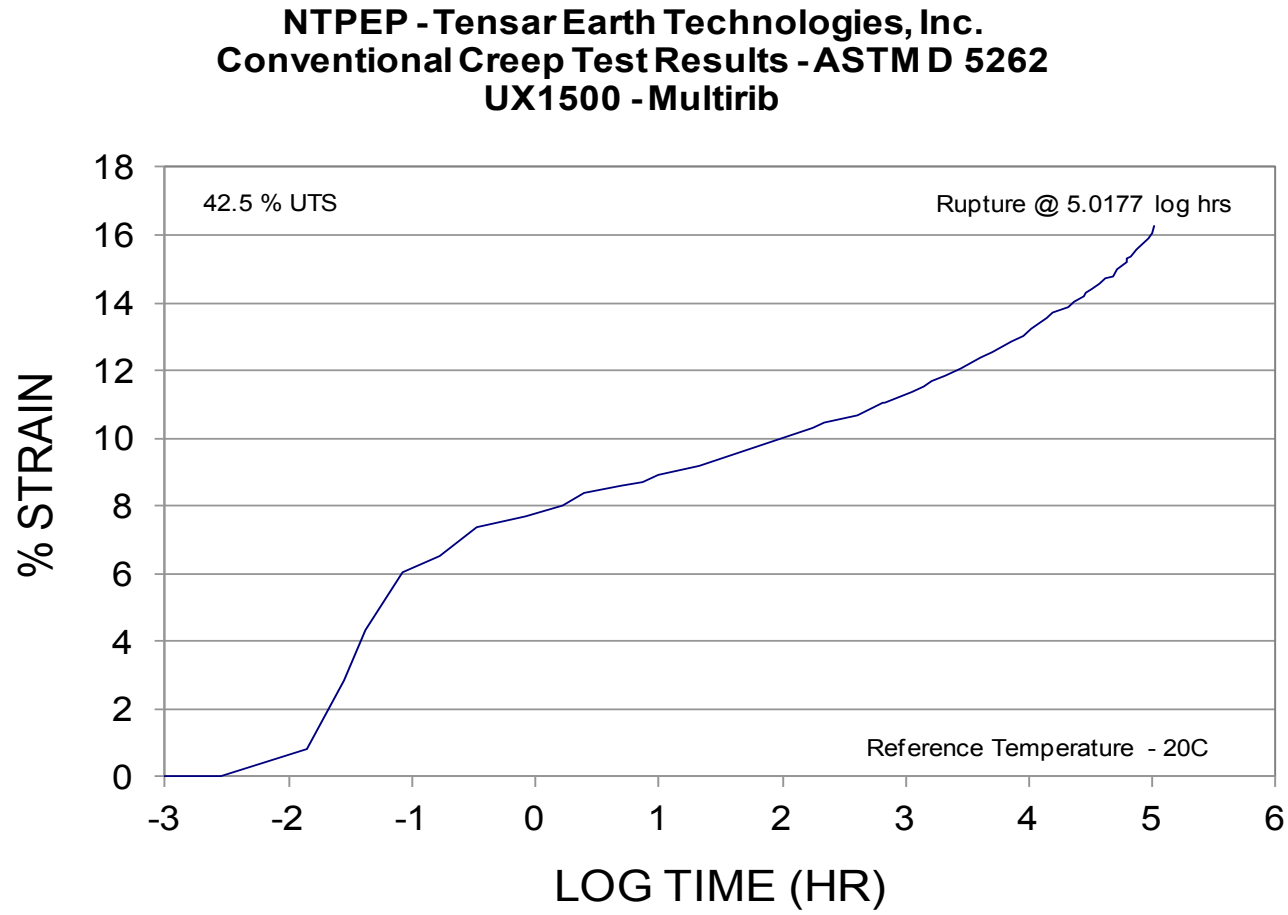
TRI Log # E2280-68-03

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)



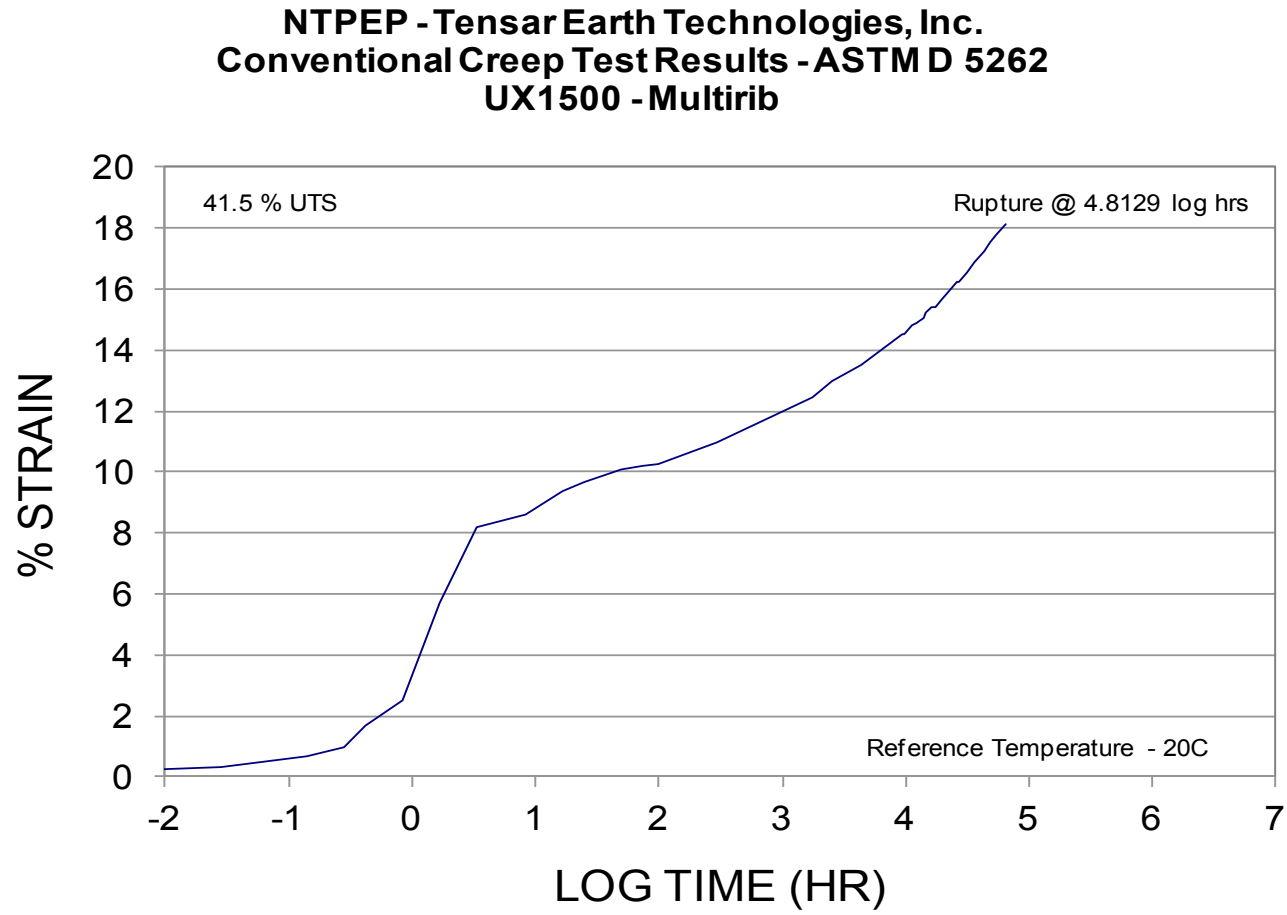
TRI Log # E2280-68-03

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)



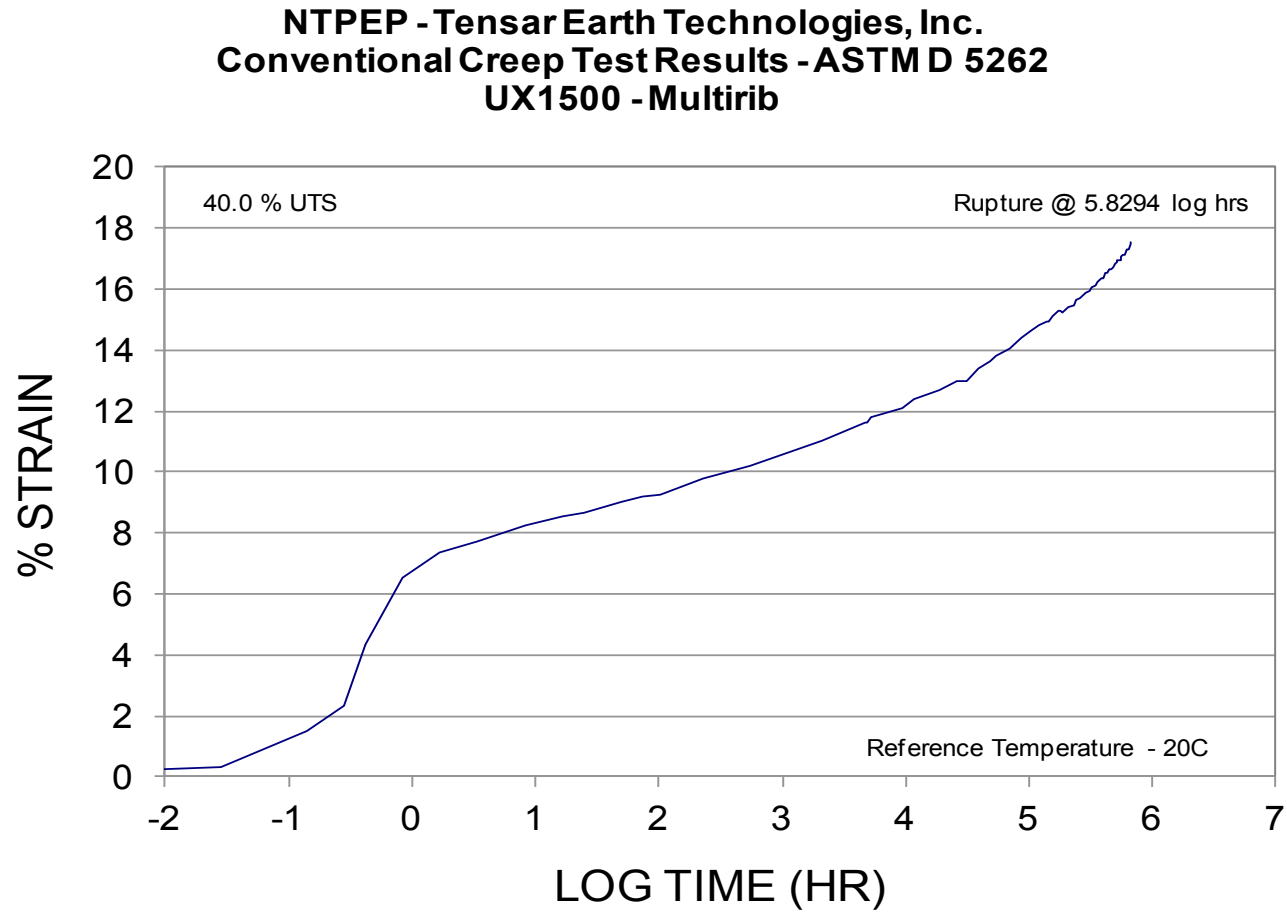
TRI Log # E2280-68-03

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)



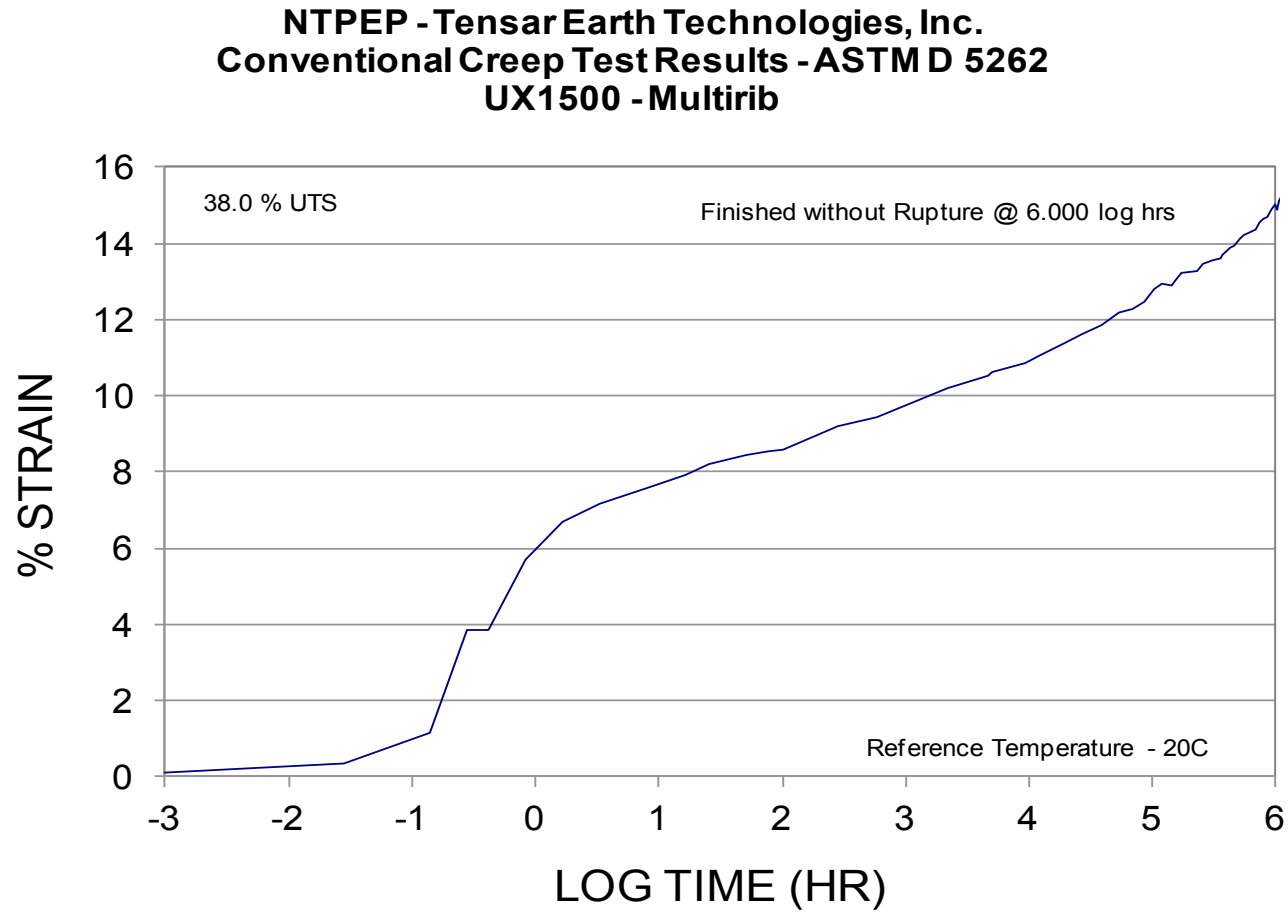
TRI Log # E2280-68-03

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)

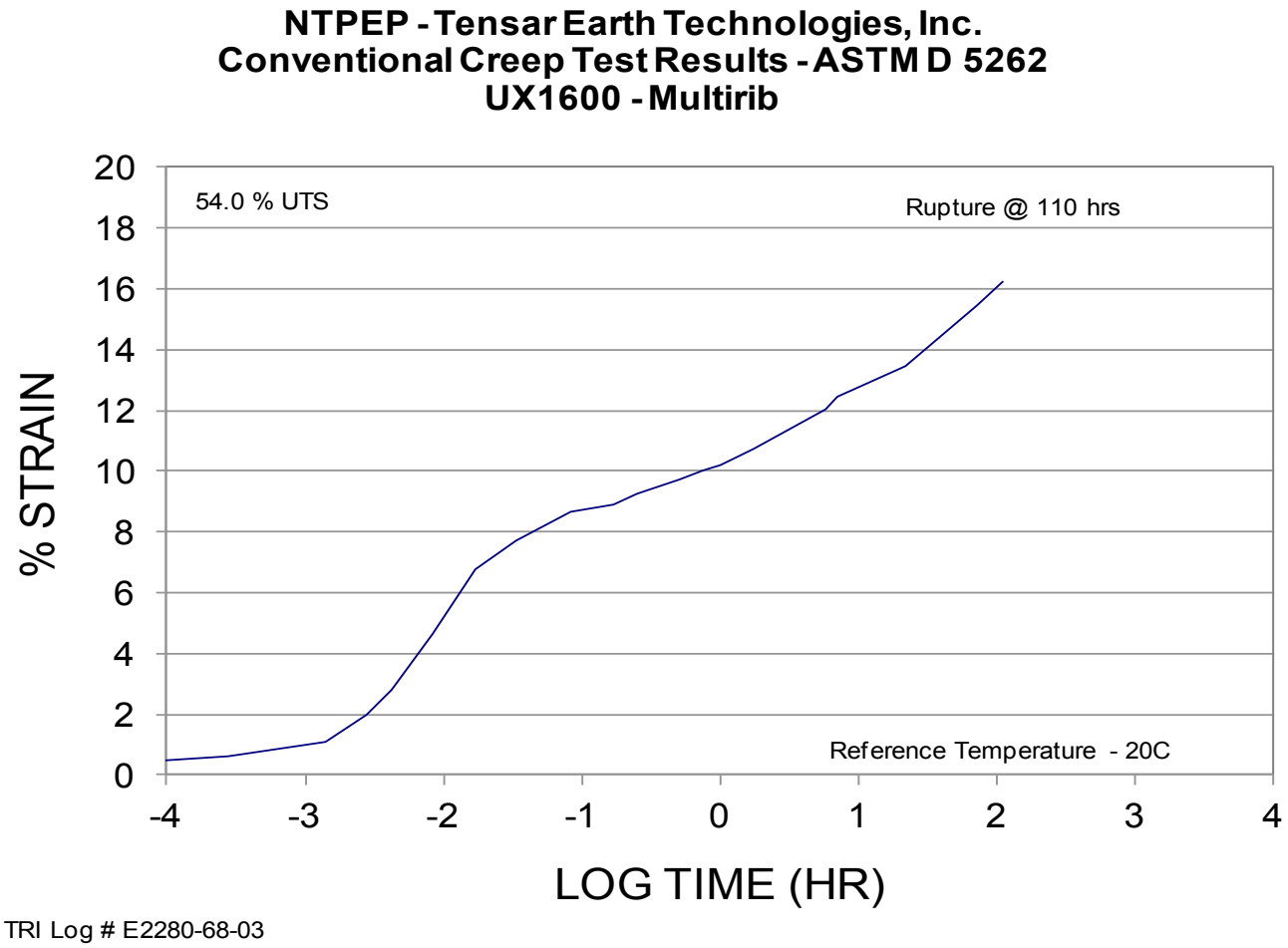
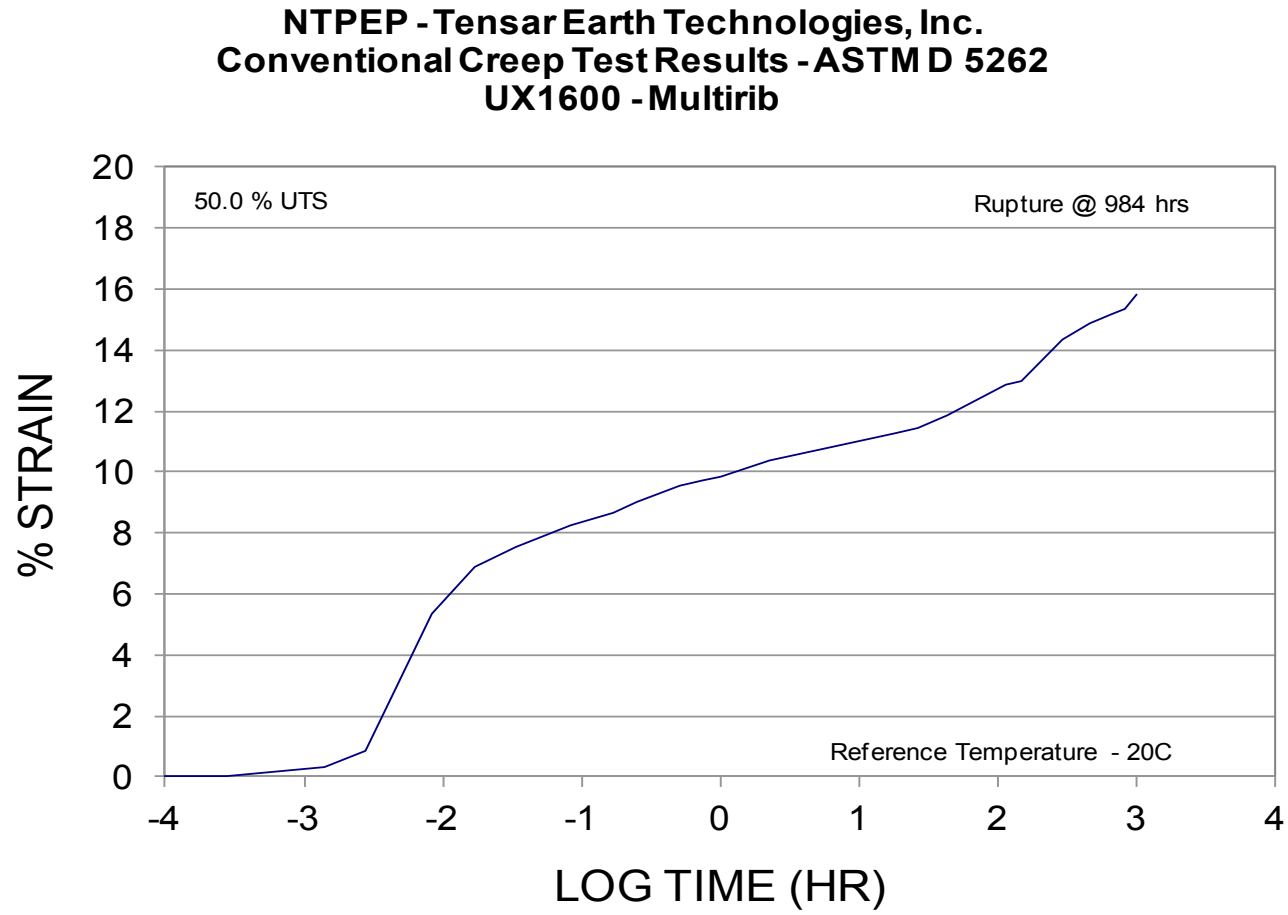
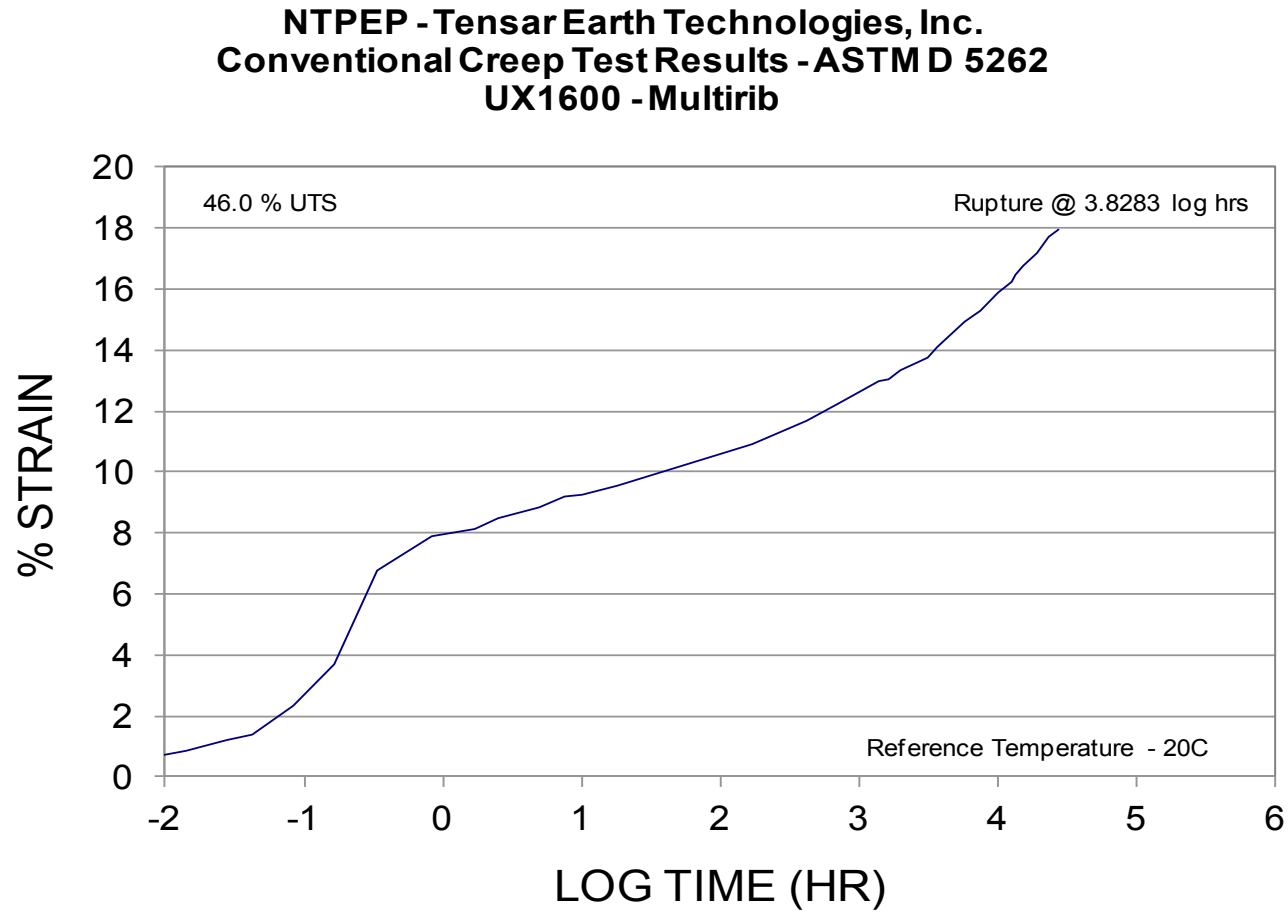


Figure F-36. Creep data/curve per ASTM D5262, multirib, for Tensar UX1600 at a load level of 54.0% UTS and 68°F(20°C)



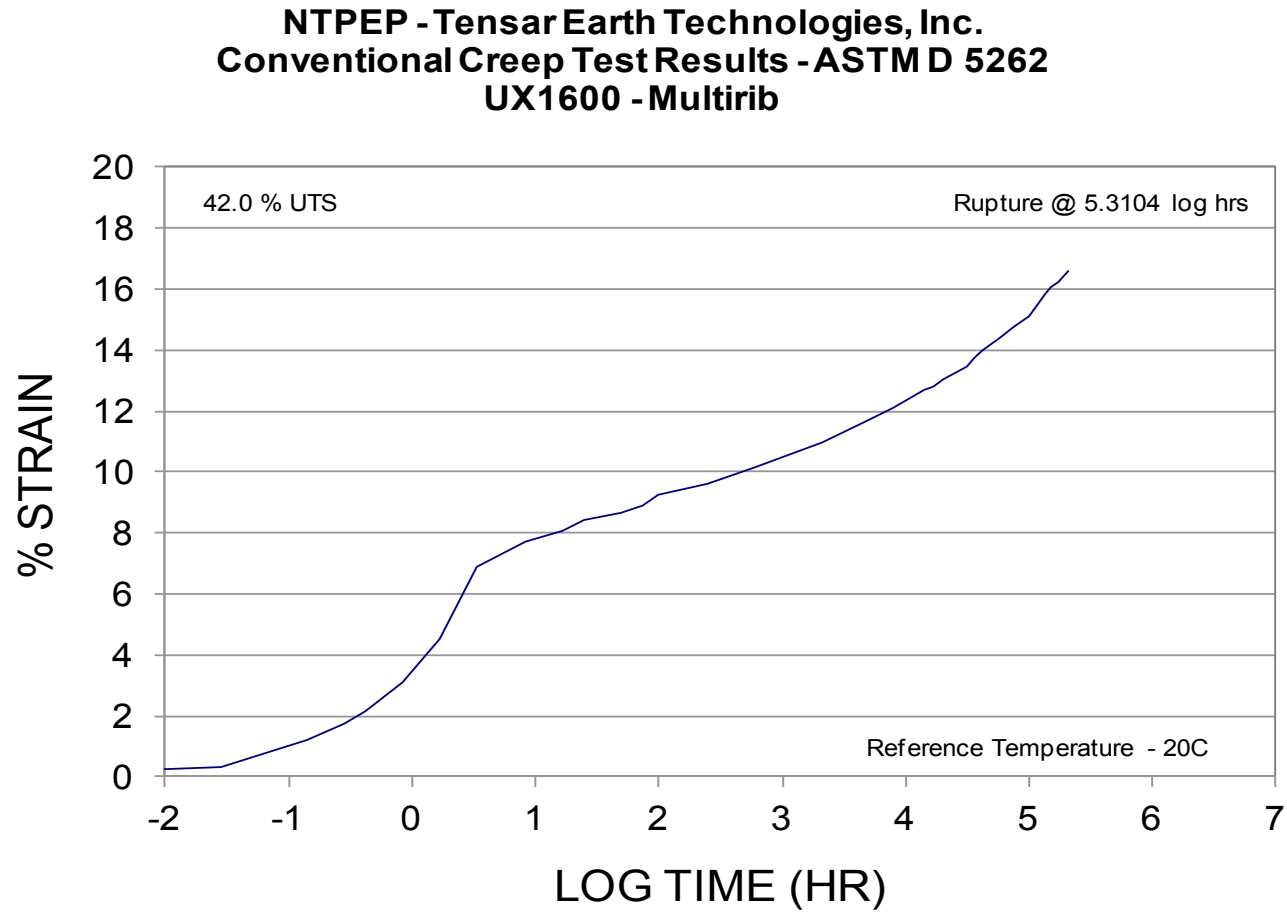
TRI Log # E2280-68-03

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)



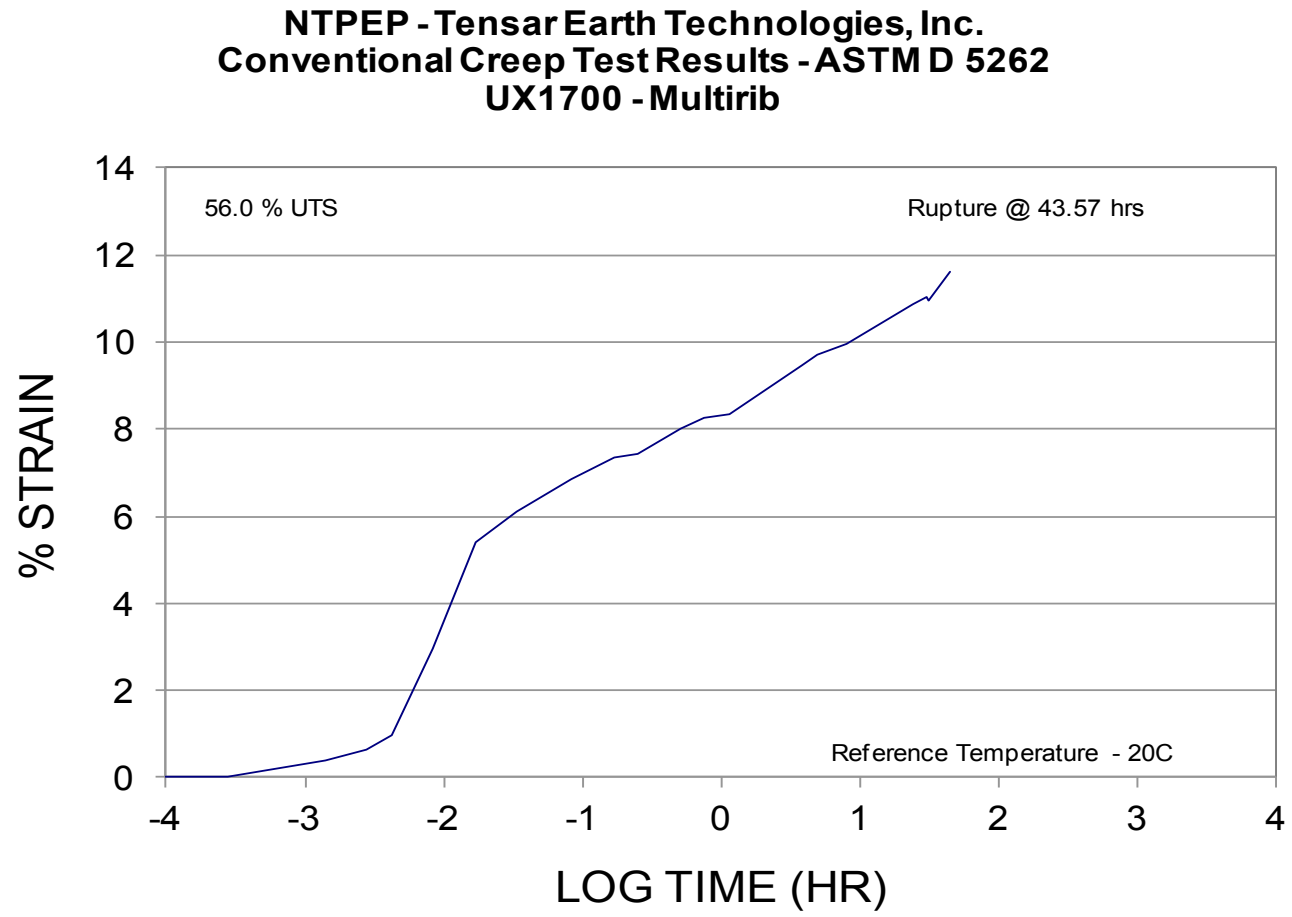
TRI Log # E2280-68-03

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)



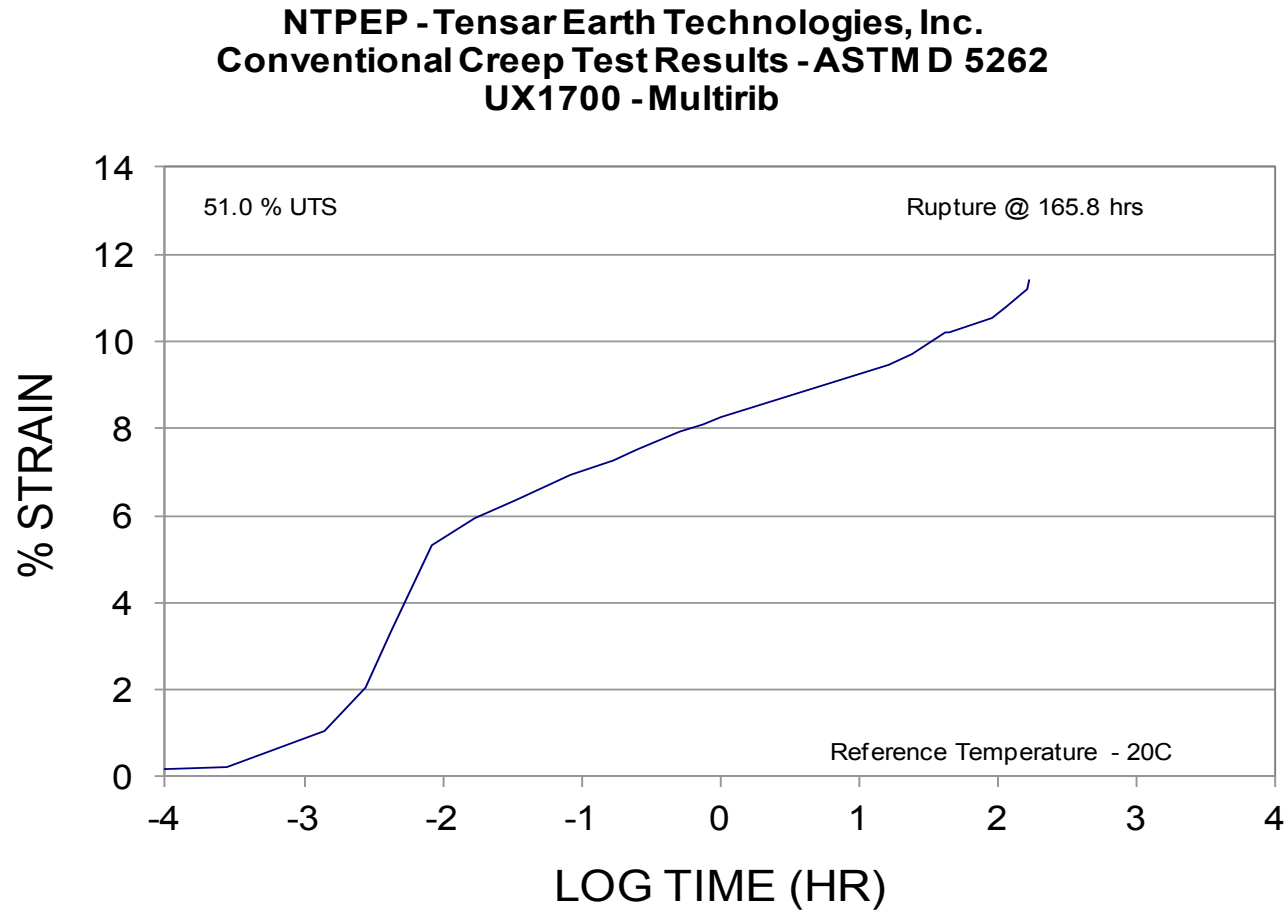
TRI Log # E2280-68-03

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)



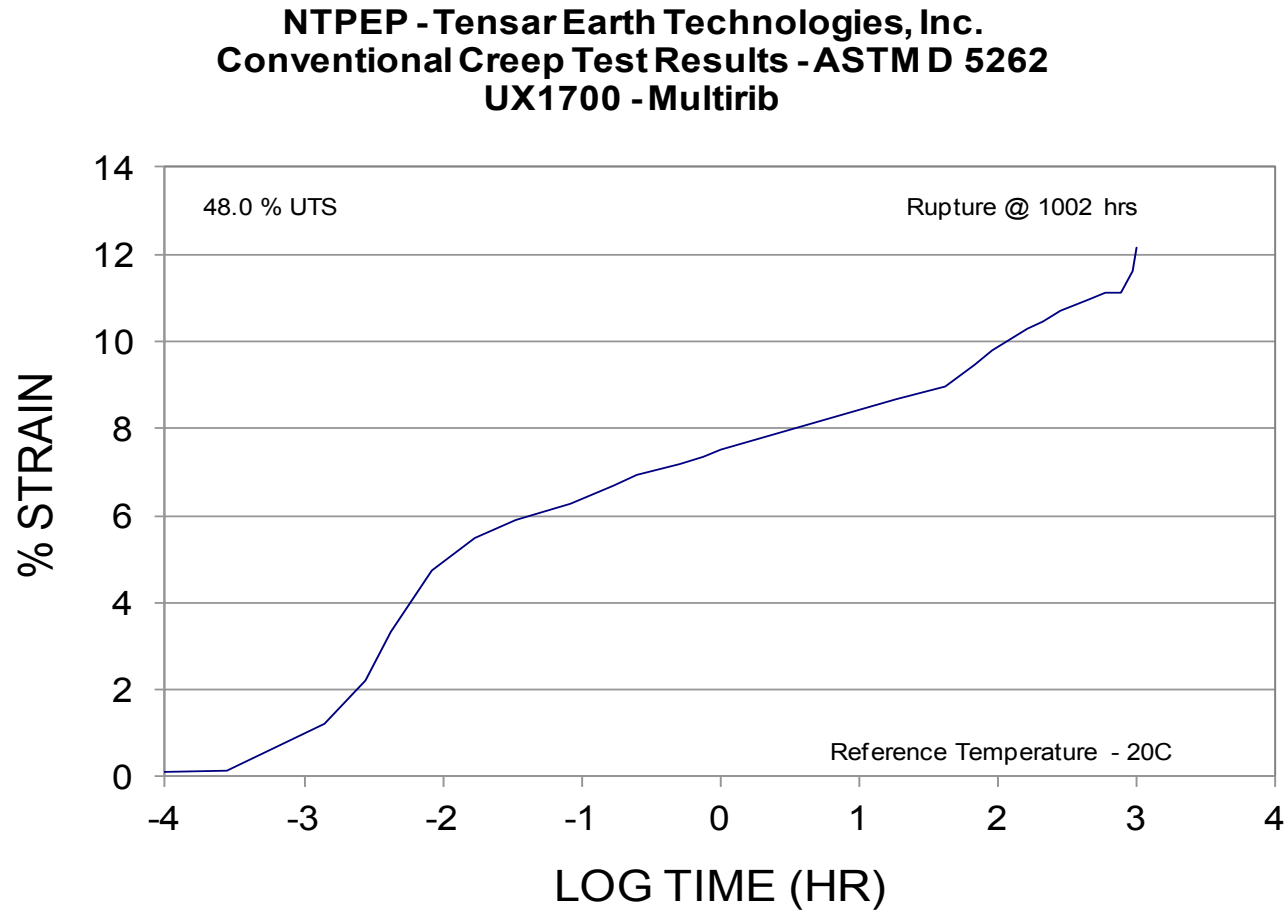
TRI Log # E2280-68-03

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)



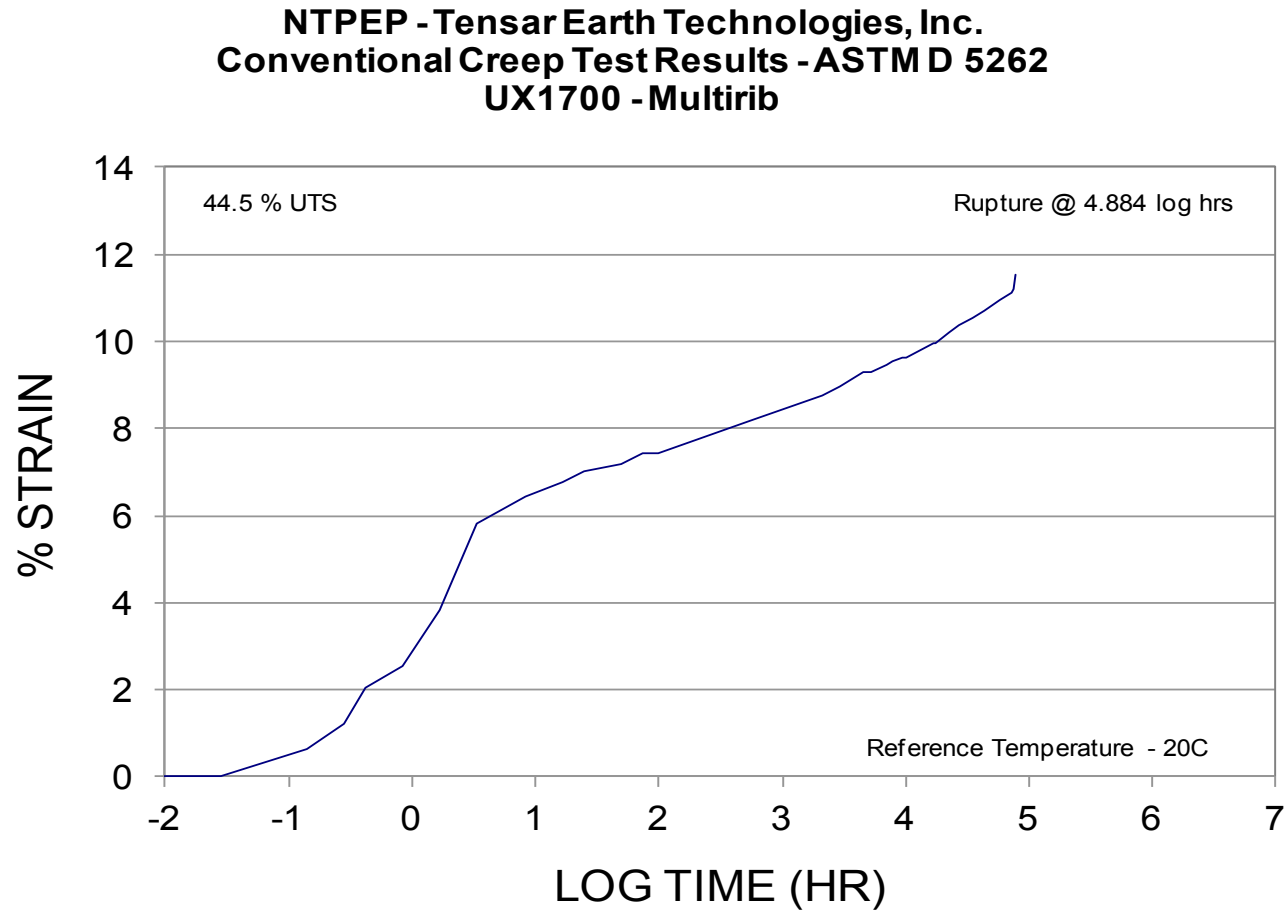
TRI Log # E2280-68-03

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

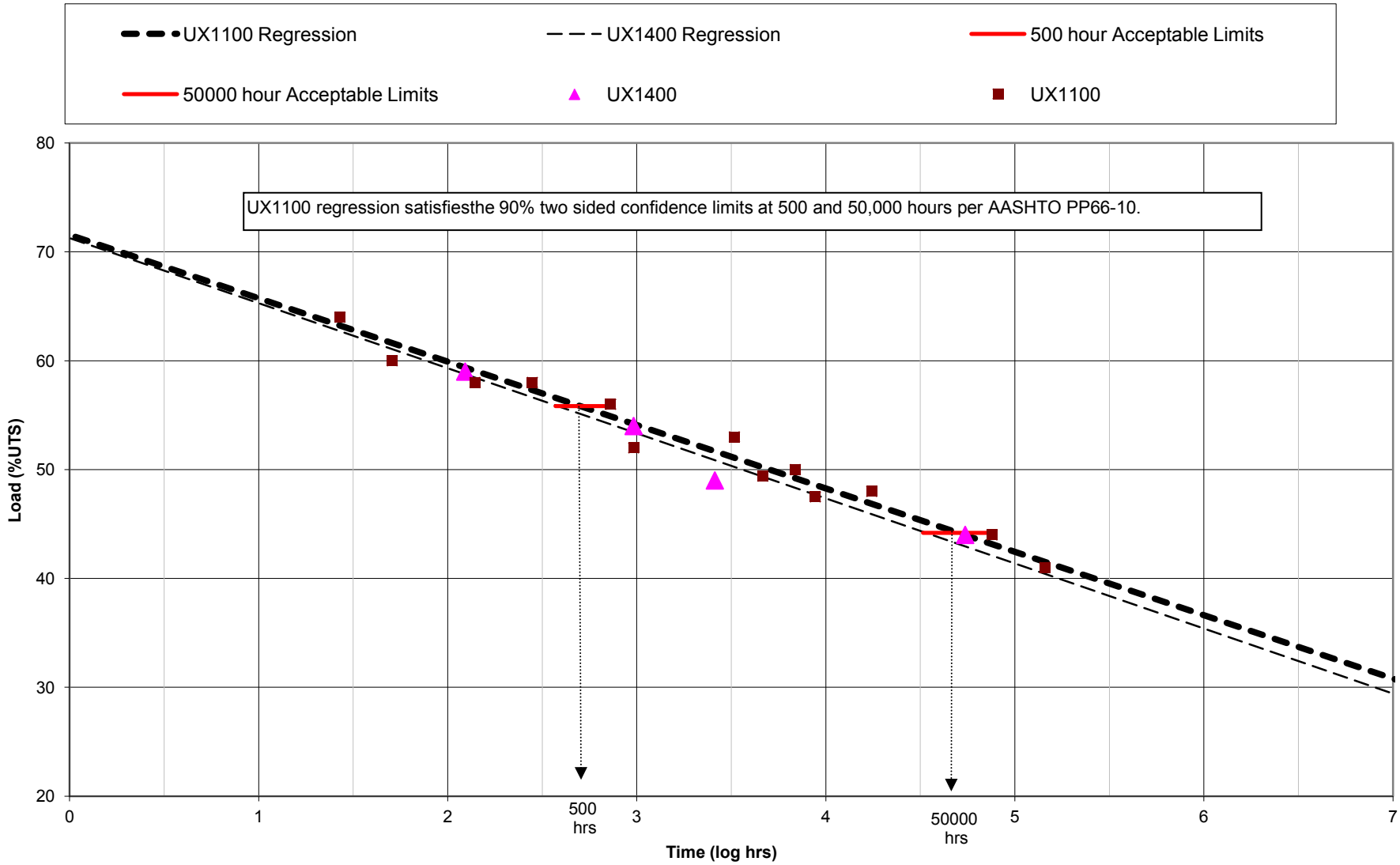


Figure F-44. Statistical evaluation results for determining validity of creating composite creep rupture envelope for the UX-MSE / UX-HS geogrids (UX1100MSE and UX1400MSE comparision) .

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

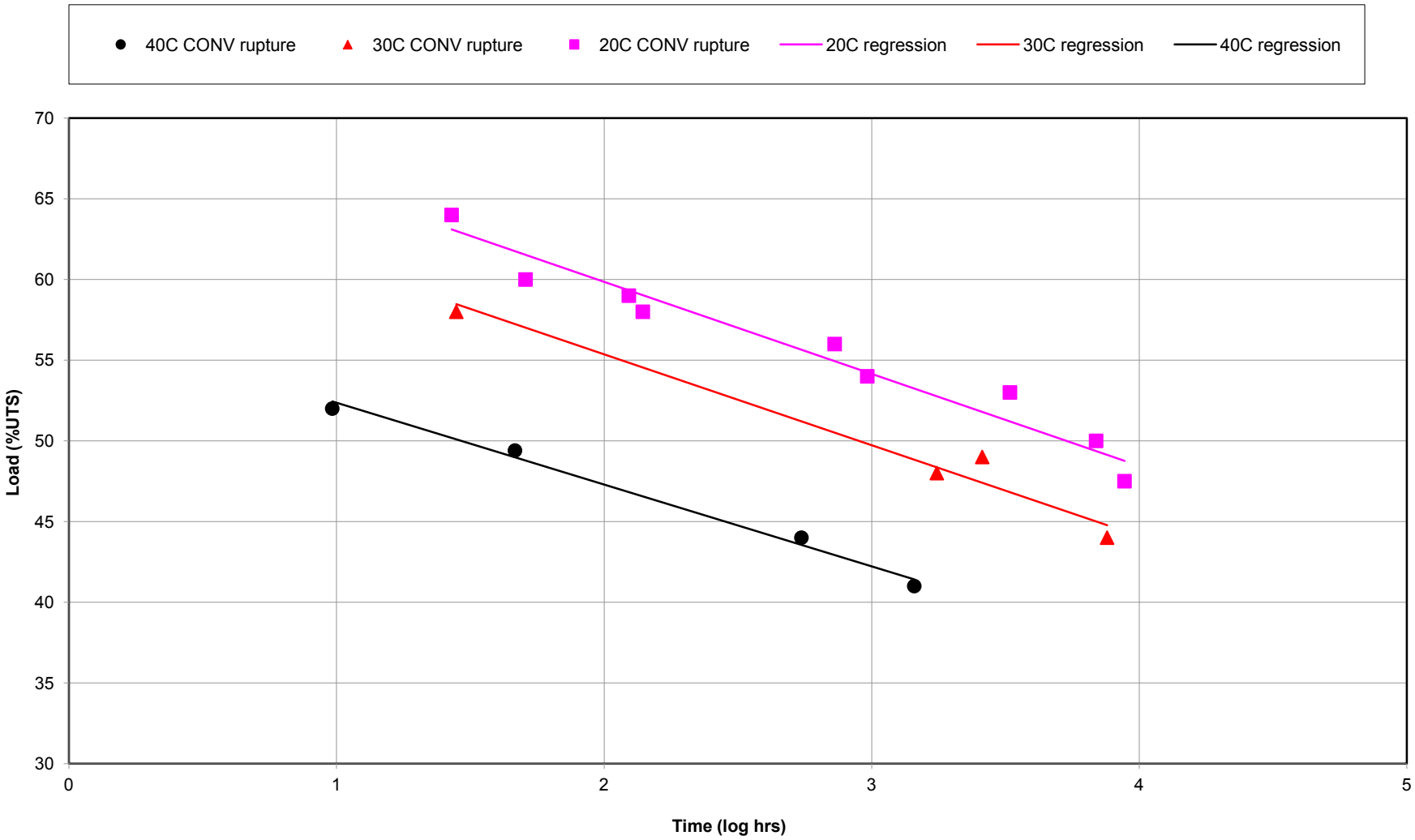


Figure F-45. Actual time to rupture data and regressions for verification of the best fit shift factors of 'x10' for 30°C and 'x100' for 40°C for UX1100 & UX1400.

TENSAR UX1100 - UX1400 - COMPOSITE CREEP RUPTURE CURVE

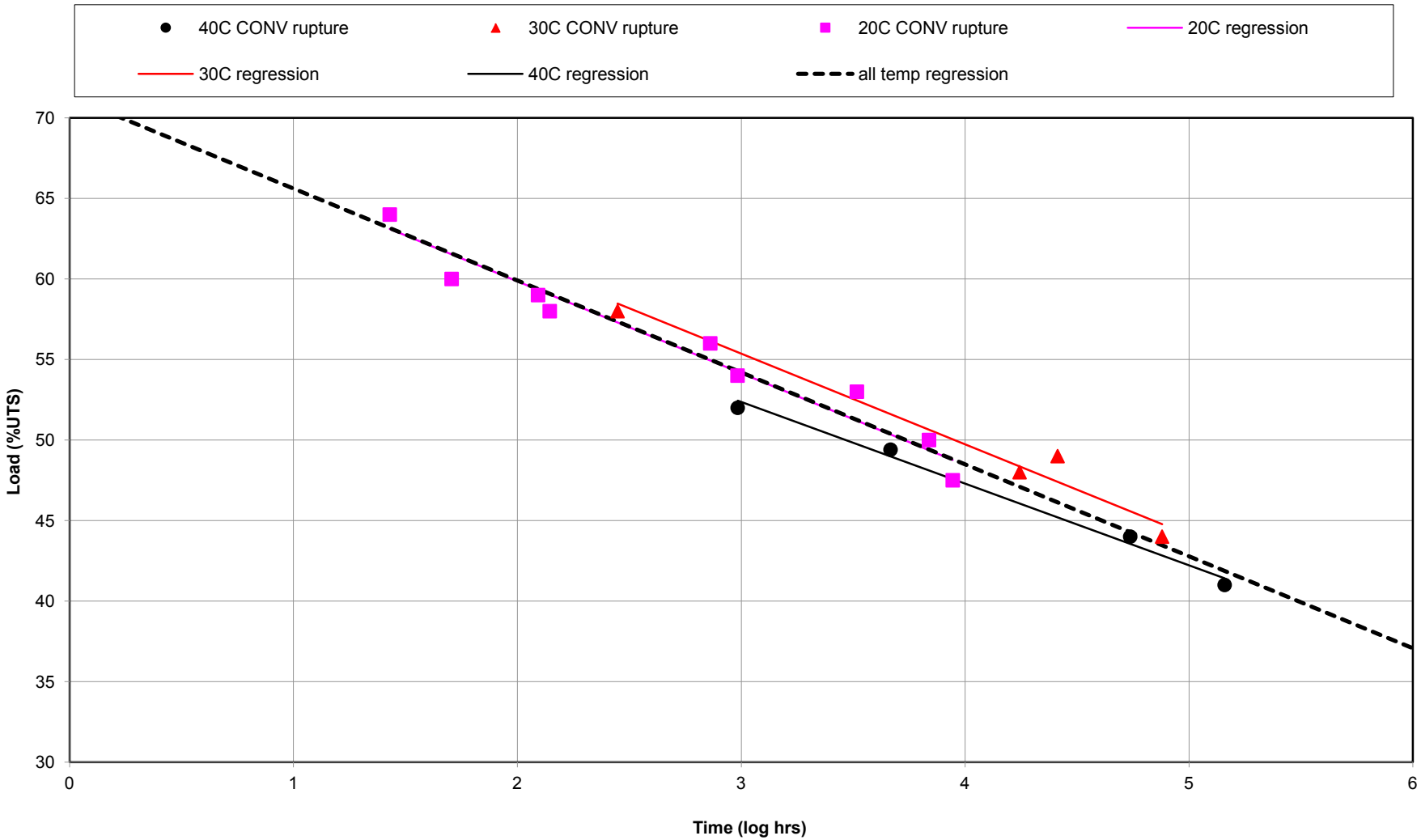


Figure F-46. Time shifted data and regressions for verification of the best fit shift factors of 'x10' for 30°C and 'x100' for 40°C for UX1100 & UX1400.

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	

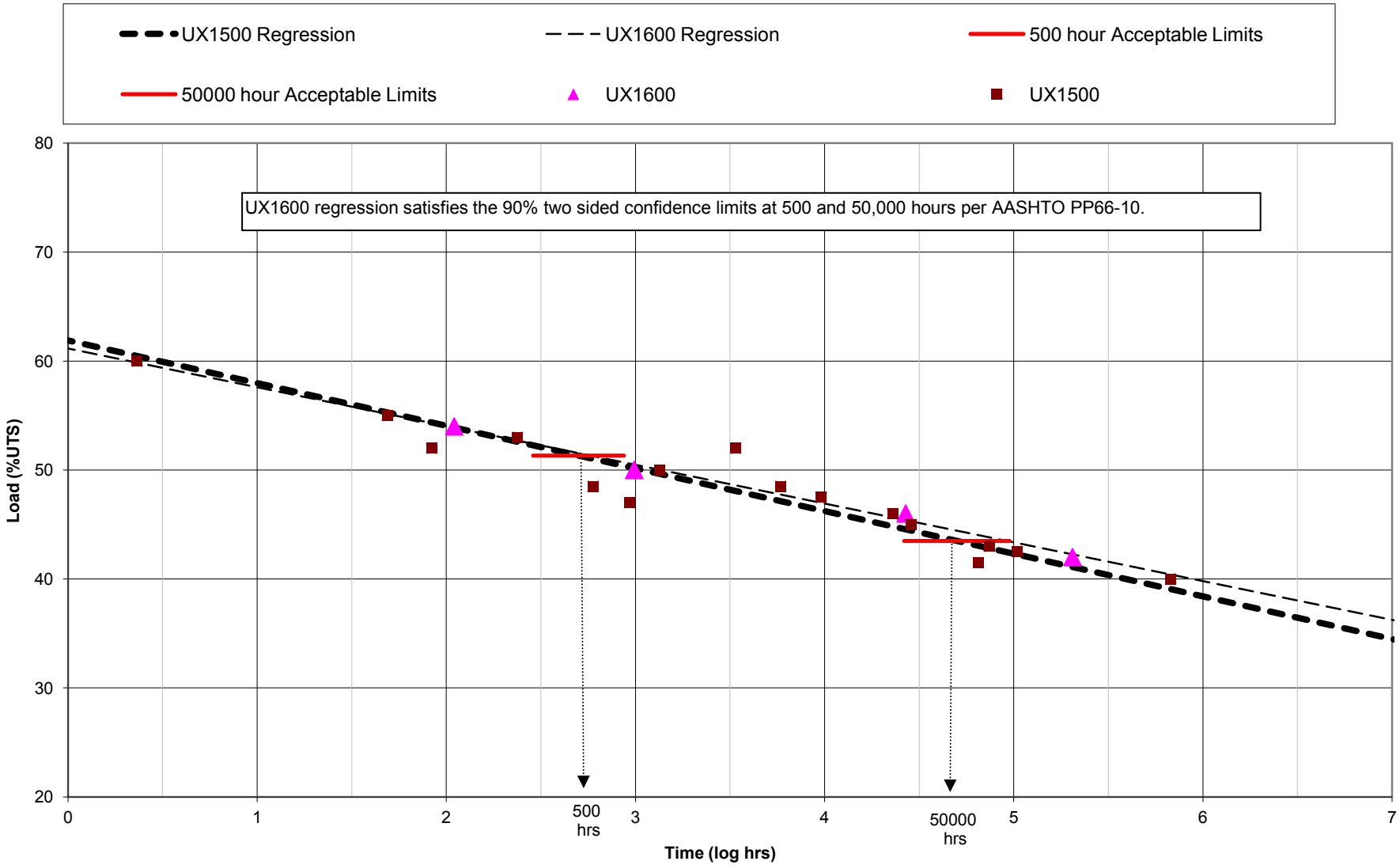


Figure F-47. Statistical evaluation results for determining validity of creating composite creep rupture envelope for the UX-MSE / UX-HS geogrids (UX1500MSE and UX1600MSE comparision) .

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

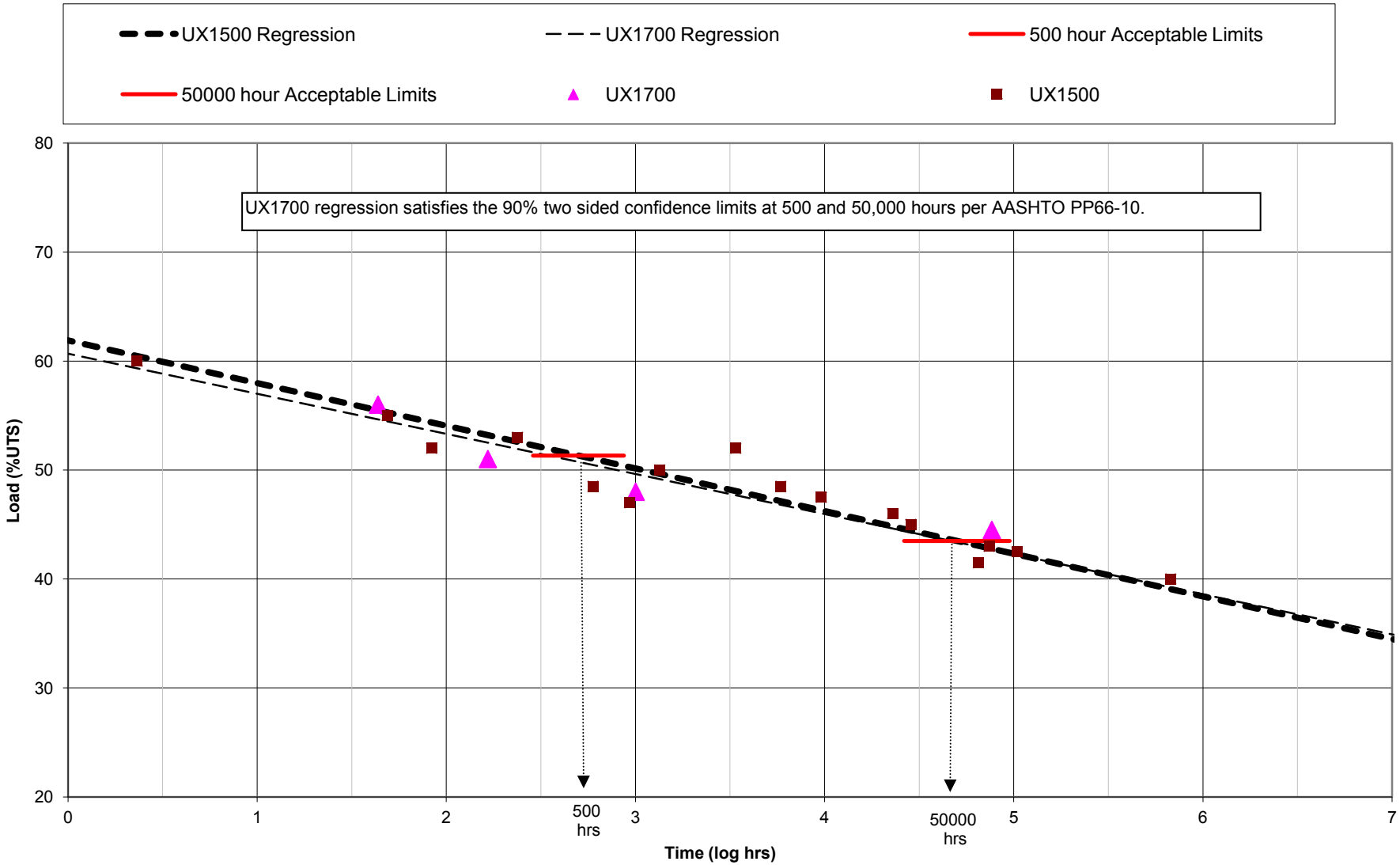


Figure F-48. Statistical evaluation results for determining validity of creating composite creep rupture envelope for the UX-MSE / UX-HS geogrids (UX1500MSE and UX1700MSE comparision) .

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

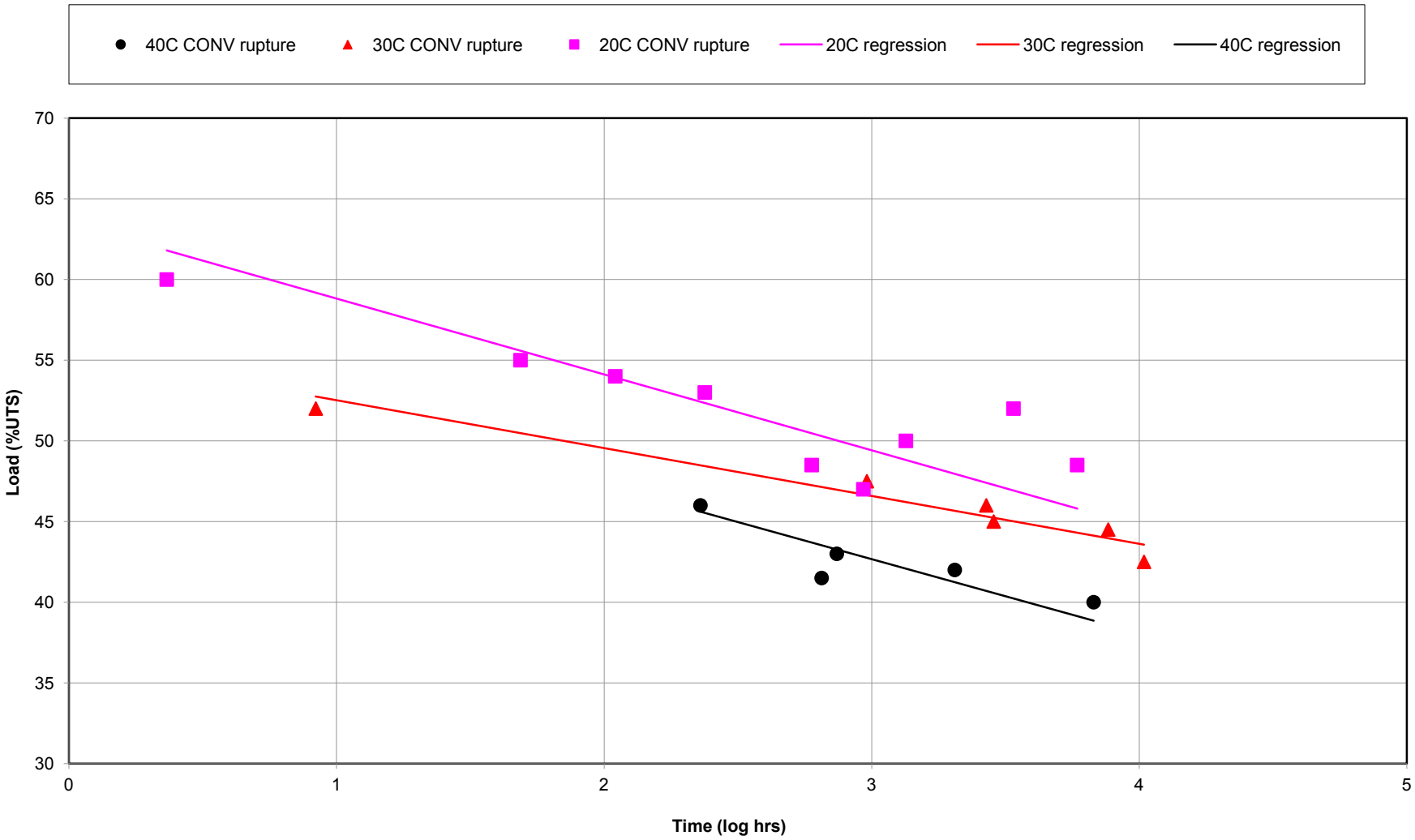


Figure F-49. Actual time to rupture data and regressions for verification of the best fit shift factors of 'x10' for 30°C and 'x100' for 40°C for UX1500, UX1600 & UX1700.

TENSAR UX 1500 - UX1600 - UX1700 - COMPOSITE CREEP RUPTURE CURVE

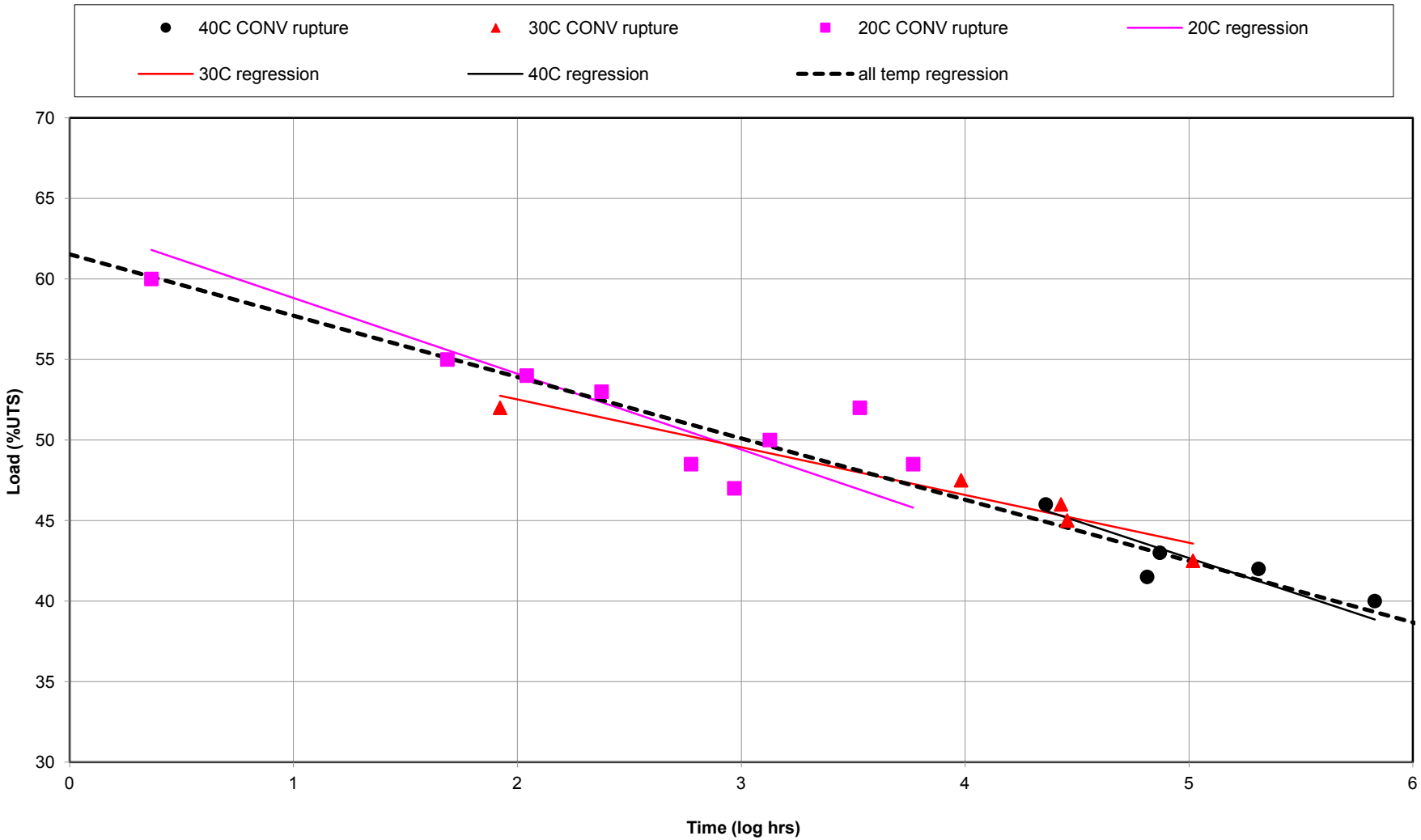


Figure F-50. Time shifted data and regressions for verification of the best fit shift factors of 'x10' for 30°C and 'x100' for 40°C for UX1500, UX1600 & UX1700.

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					MEAN	STD. DEV.	PERCENT RETAINED	AASHTO R69-15 SPEC
	1	2	3	4	5				
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

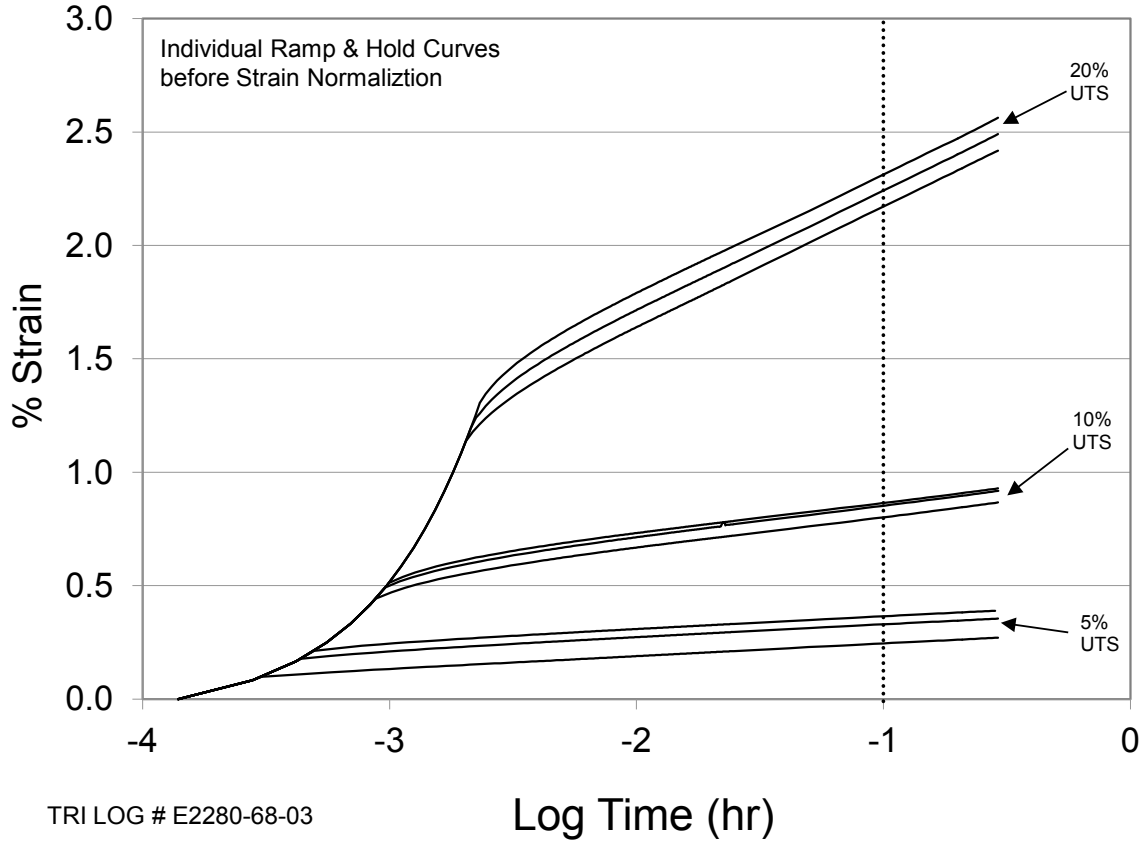


Figure H-1. Low strain ramp and hold tests for Tensar UX1100 geogrid, before strain normalization.

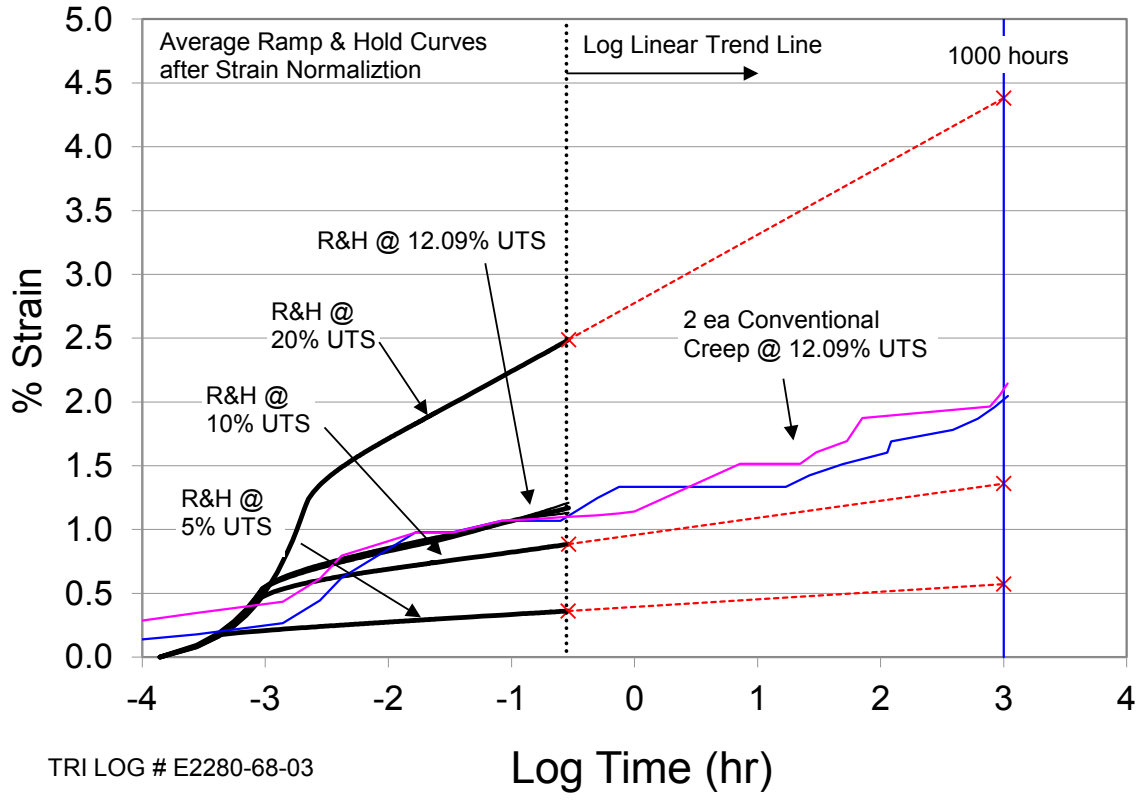


Figure H-2. Low strain ramp and hold tests for Tensar UX1100 geogrid, after strain normalization, with 1000 hour low strain creep tests.

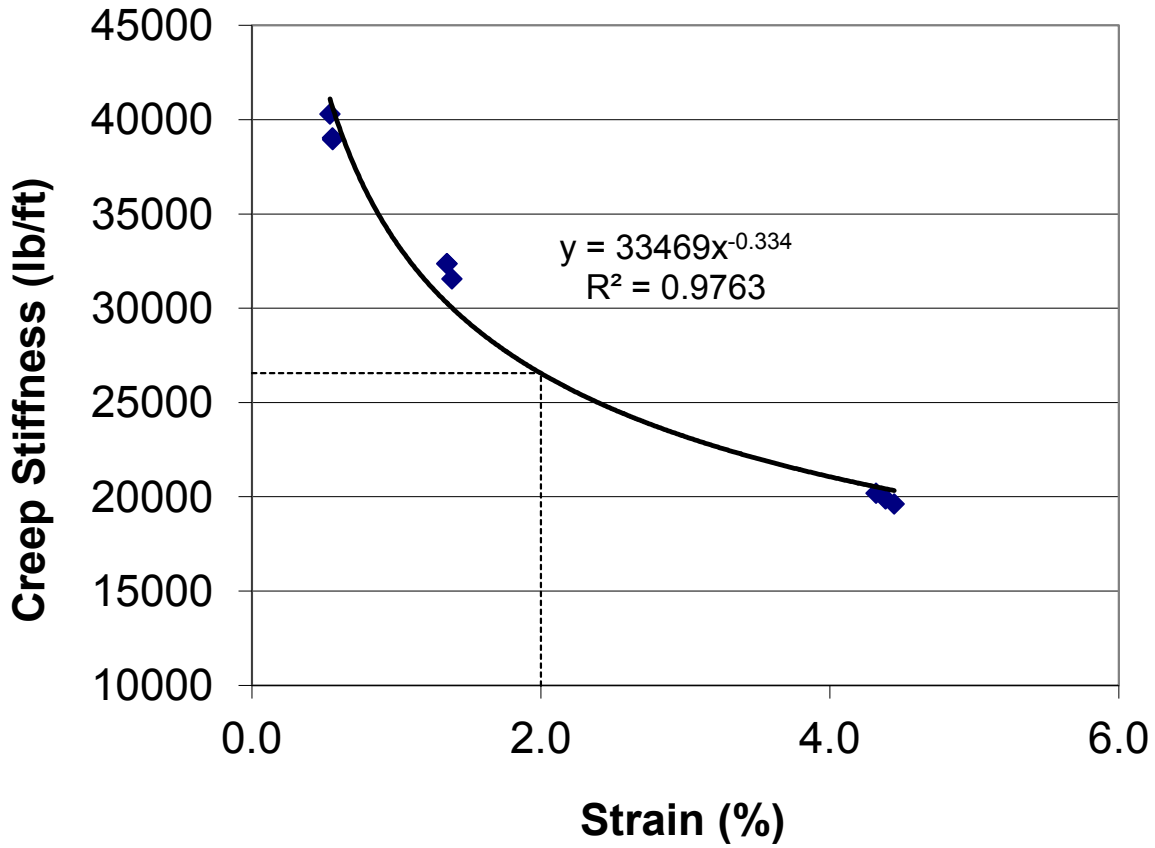
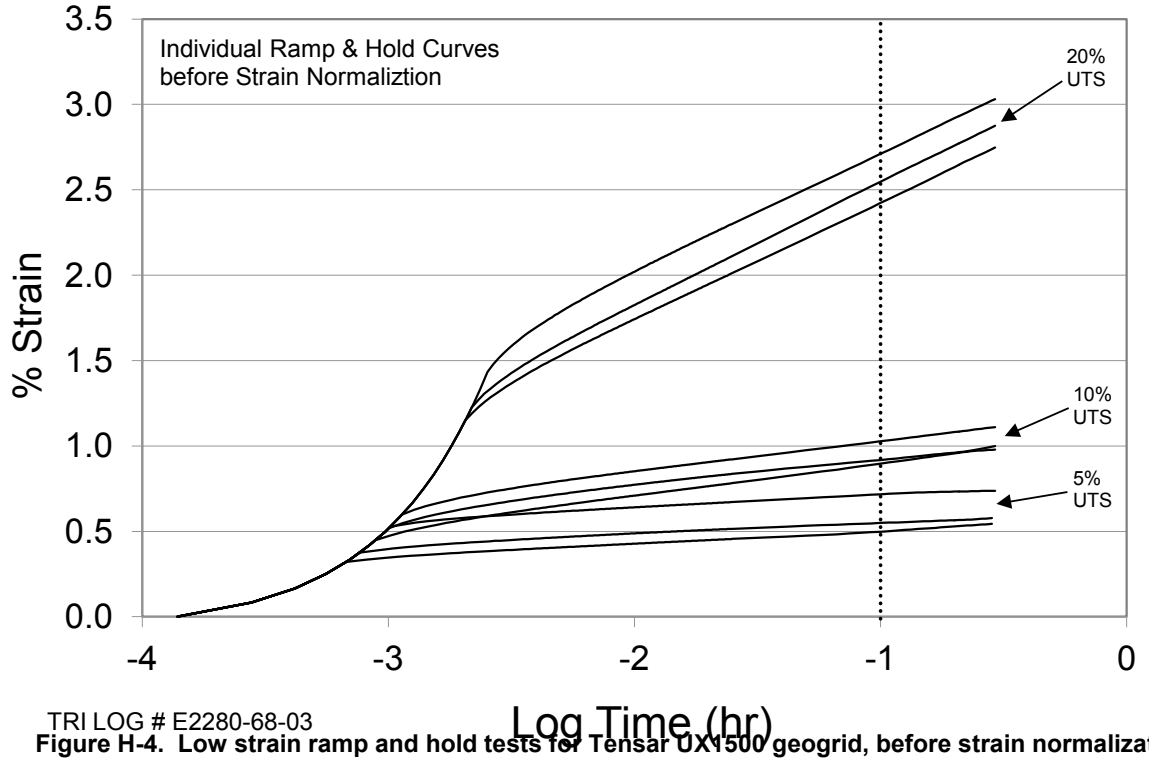


Figure H-3. Creep stiffness versus strain at 1,000 hours for Tensar UX1100 geogrid.



TRI LOG # E2280-68-03

Figure H-4. Low strain ramp and hold tests for Tensar UX1500 geogrid, before strain normalization.

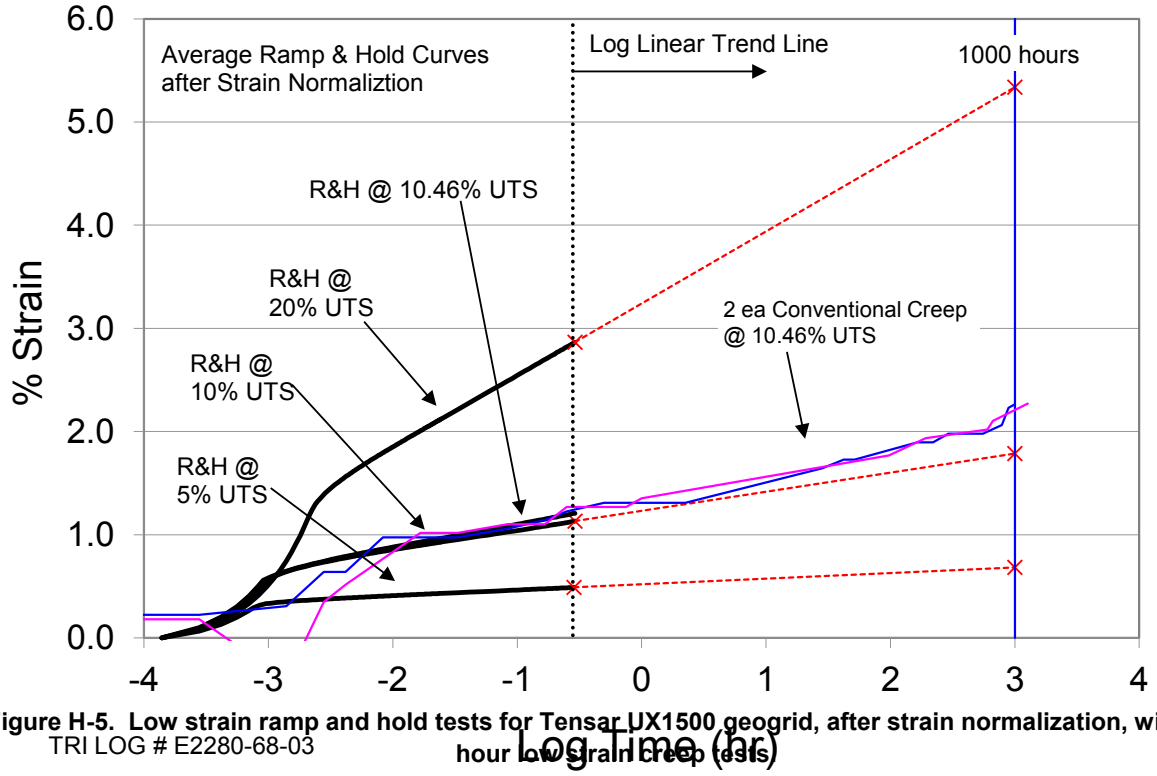


Figure H-5. Low strain ramp and hold tests for Tensar UX1500 geogrid, after strain normalization, with 1000 hour low strain creep tests
TRI LOG # E2280-68-03

Log Time (hr)

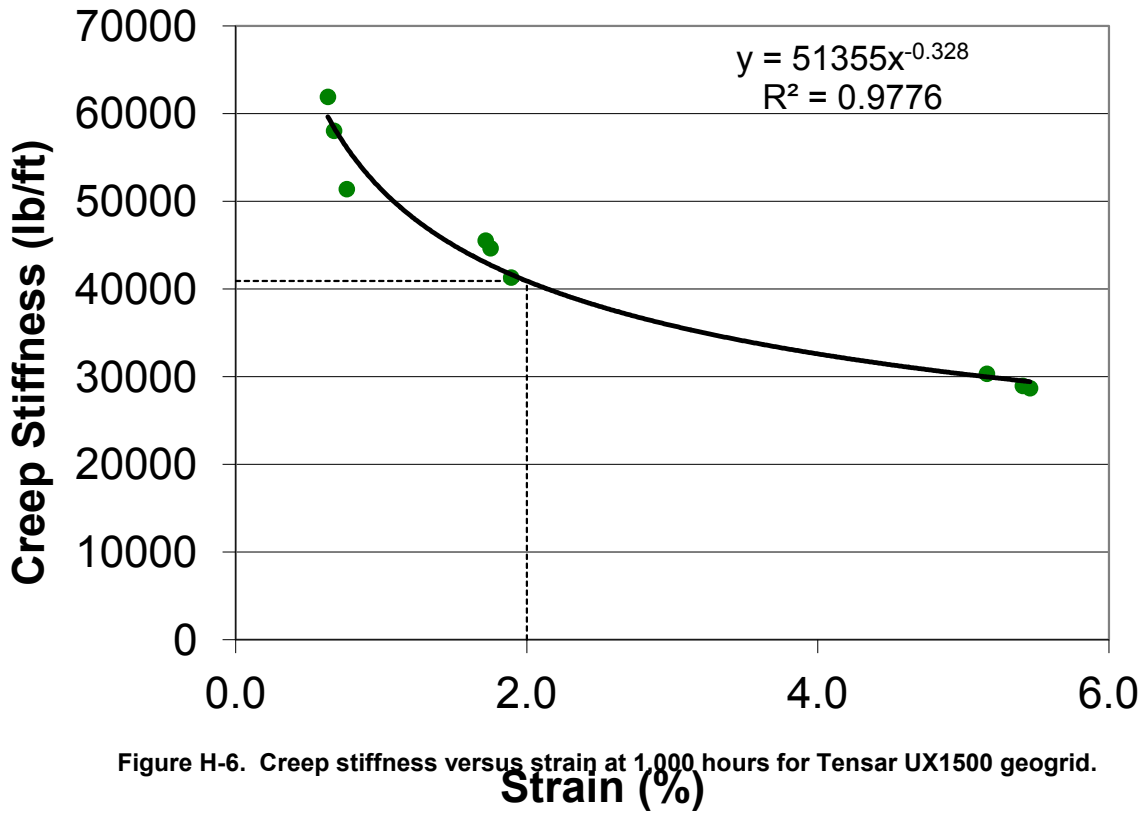


Figure H-6. Creep stiffness versus strain at 1,000 hours for Tensar UX1500 geogrid.

Strain (%)

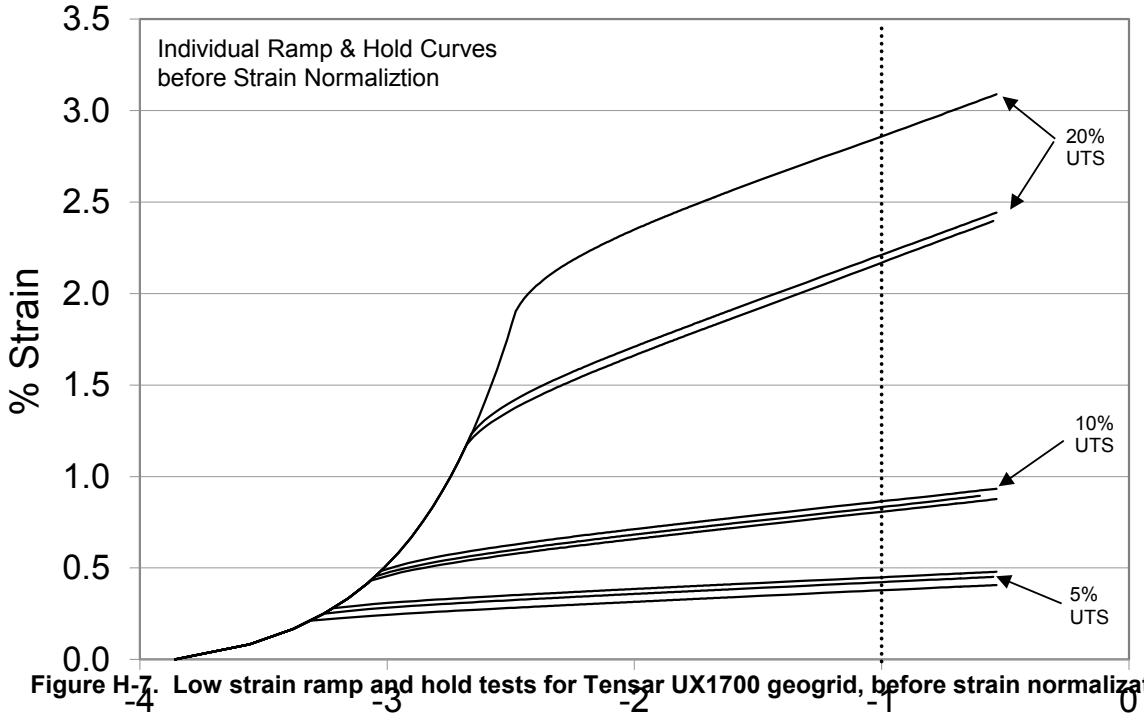


Figure H-7. Low strain ramp and hold tests for Tensar UX1700 geogrid, before strain normalization.

-4 -3 -2 -1 0

TRI LOG # E2280-68-03

Log Time (hr)

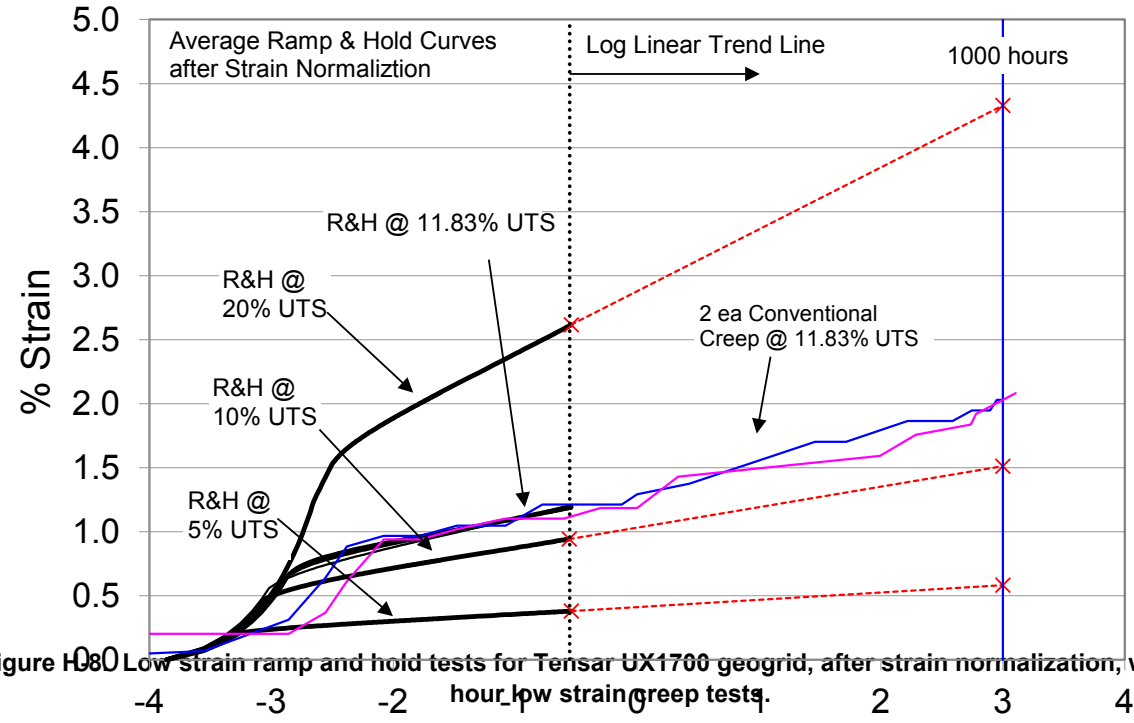


Figure H-8 Low strain ramp and hold tests for Tensar UX1700 geogrid, after strain normalization, with 1000 hour low strain creep tests.

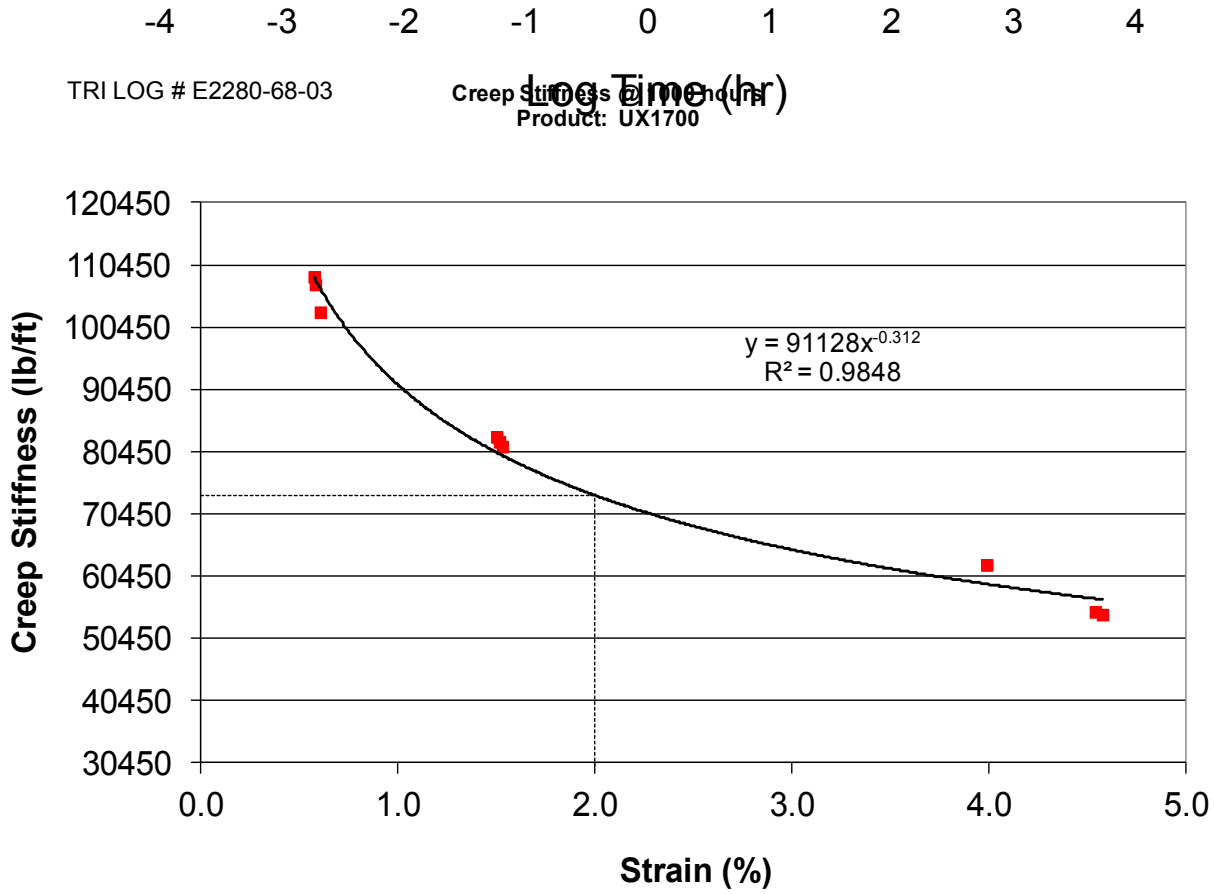


Figure H-9. Creep stiffness versus strain at 1,000 hours for Tensar UX1700 geogrid.

"The National Transportation Product Evaluation Program (NTPEP) was established by the American Association of State Highway and Transportation Officials (AASHTO) in early 1994. The program pools the professional and physical resources of the AASHTO member departments in order to test materials, products and devices of common interest. The primary goals of the program are to provide cost-effective evaluations for the states by eliminating duplication of routine testing by the states; and to reduce duplication of effort by the manufacturers who produce and market commonly used proprietary, engineered products." **NTPEP**

-- Rick Smutzer (IN), former NTPEP Chairman

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call 1.202.624.5800
fax 1.800.525.5469
online www.NTPEP.ORG

ITEM: NTPEP Report 2012-01-002

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Appendix 1.2.5.a

ARES Tab Product Specifications Sheet

ARES® MSE Retaining Wall System

Back to main document

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

Product Specification - Structural Geogrid UX1700AR

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.

Product Type: **Integrally Formed Structural Geogrid Tab**
Polymer: **High Density Polyethylene**
Load Transfer Mechanism: **Positive Mechanical Interlock**
Recommended Applications: **ARES System (Panel Walls)**

Product Properties

Index Properties	Units	MD Values[1]
▪ Tensile Strength @ 5% Strain[2]	kN/m (lb/ft)	75 (5,140)
▪ Ultimate Tensile Strength[2]	kN/m (lb/ft)	175 (11,990)
▪ Junction Strength[3]	kN/m (lb/ft)	160 (10,970)
▪ Flexural Stiffness[4]	mg-cm	9,075,000
▪ Minimum Carbon Black Content	%	2.0
▪ Minimum Transverse Bar Width	mm (in)	12 (0.47)
Durability		
▪ Resistance to Long Term Degradation[5]	%	100
▪ Resistance to UV Degradation[6]	%	95
Connection Capacity		
▪ Maximum Long-term Connection Capacity for 120-year Design Life[7]	kN/m (lb/ft)	57 (3,907)
Recommended Allowable Strength Reduction Factors		
▪ Reduction Factor for Creep for 120-year Design Life (RF_{CR}) [8]		2.60
▪ Minimum Reduction Factor for Durability (RF_D) [9]		1.10

Dimensions and Delivery

The structural geogrid tab shall be delivered to the jobsite in box form with 100 pieces per box. The geogrid tab is measuring 1.32 meters (4.33 feet) in width nominal and one aperture in length.

Notes:

1. Unless indicated otherwise, values shown are minimum average roll values determined in accordance with ASTM D4759-02. Brief descriptions of test procedures are given in the following notes.
2. True resistance to elongation when initially subjected to a load measured via ASTM D6637-10 Method A without deforming test materials under load before measuring such resistance or employing "secant" or "offset" tangent methods of measurement so as to overstate tensile properties.
3. Load transfer capability determined in accordance with ASTM D7737-11.
4. Resistance to bending force determined in accordance with ASTM D7748-12, using one meter long specimen.
5. Resistance to loss of load capacity or structural integrity when subjected to chemically aggressive environments in accordance with EPA 9090 immersion testing.
6. Resistance to loss of load capacity or structural integrity when subjected to 500 hours of ultraviolet light and aggressive weathering in accordance with ASTM D4355-05.
7. Maximum connection capacity based on long-term connection test of UX1700AR connected to UX1700MSE with 1.5” wide HDPE bodkin bar.
8. Reduction factor for Creep determined for 120-year design life and in-soil temperature of 20°C using standard extrapolation techniques to creep rupture data obtained following the test procedure in ASTM D5262-04. Actual design life of the completed structure may differ.
9. Minimum durability reduction factor per AASHTO/FHWA. Recommended reduction factor is 1.0 for non public application.

Tensar International Corporation warrants that at the time of delivery the geogrid 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 geogrid 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 May 1, 2017.

Appendix 1.2.5.b

ARES Bodkin Bar Product Specifications Sheet

ARES® MSE Retaining Wall System

Back to main document

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

Product Specification – Bodkin Bar/Rod

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.

Bodkin bar/rod is profile extruded and comprised of a select high density polyethylene polymer. The bodkin is used to joint sections of Tensar geogrids together to form a strong connection. This connection will maintain at least 90% of the tensile strength expected when using a continuous section of geogrid. The bodkin is fully stabilized to resist ultraviolet degradation.

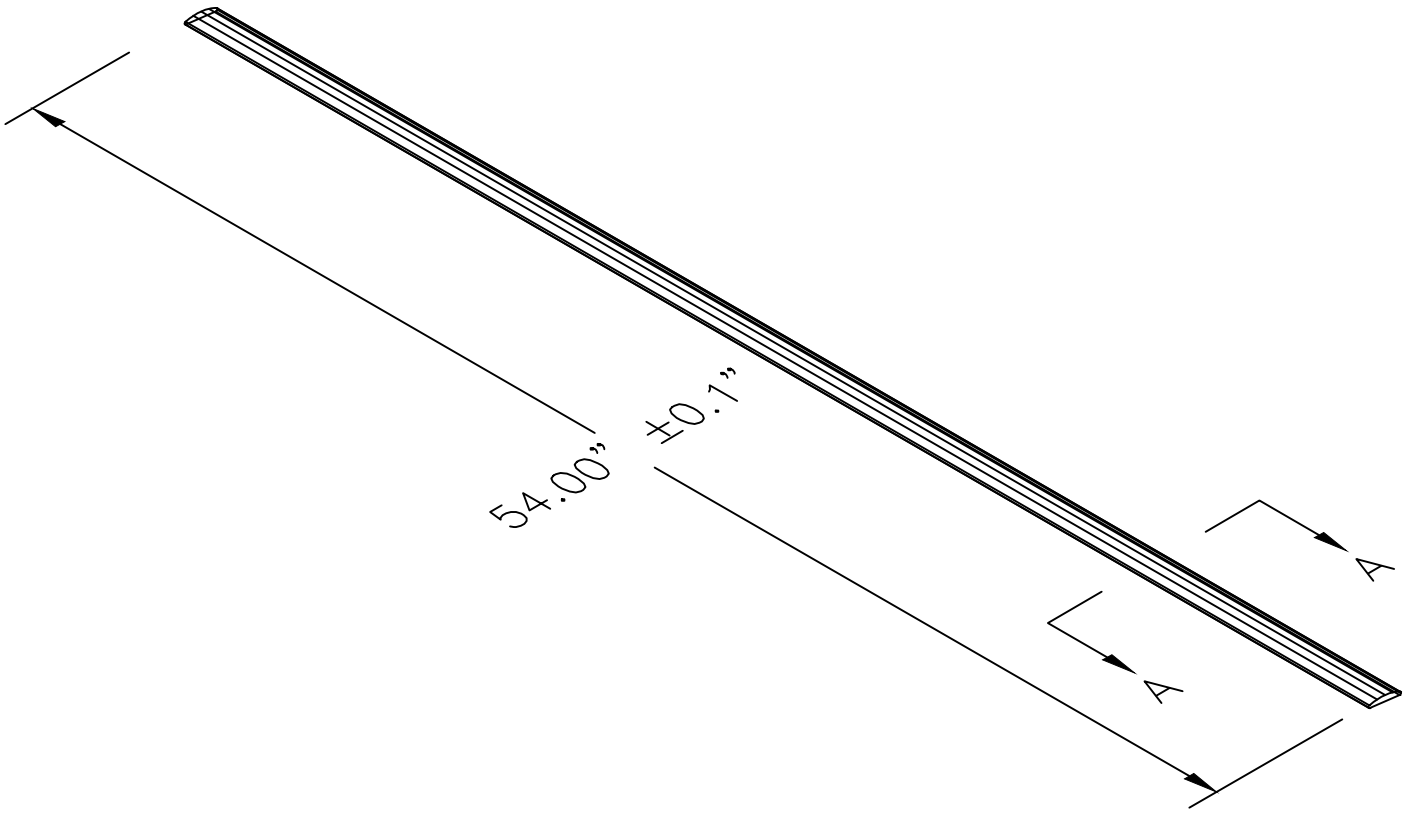
Product Properties

Index Properties	Test Method	Units	Values
▪ Tensile Strength[1]	ASTM D6637	%	90 (min)
Material			
▪ High Density Polyethylene	ASTM D1248 Type III/Class A/Grade 5	%	97 (min)
▪ Carbon Black[2]	ASTM D4218	%	2.0 (min)
Dimensions (Product Code)			
▪ OP135000	Measured	in.	0.375 dia. x 54.0L
▪ OP150000	Measured	in.	1.5W x 54.0L x 0.236t
▪ OP157000	Measured	in.	4.5W x 54.0L x 0.197t

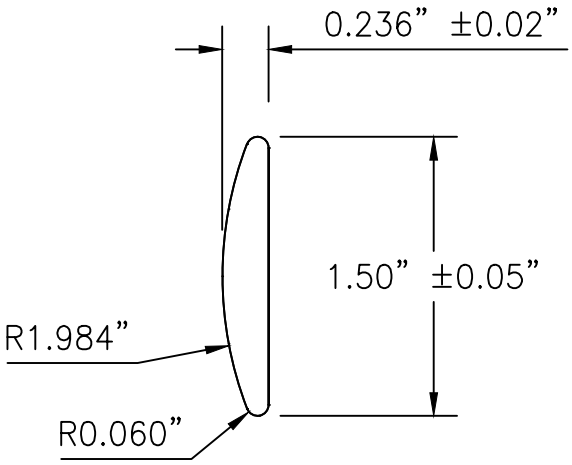
Notes:
1. Use of Bodkin bar/rod, installed as directed, will maintain at least 90% of the tensile strength of a continuous, non-joined geogrid section.
2. Carbon black added as a UV stabilizer.

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.



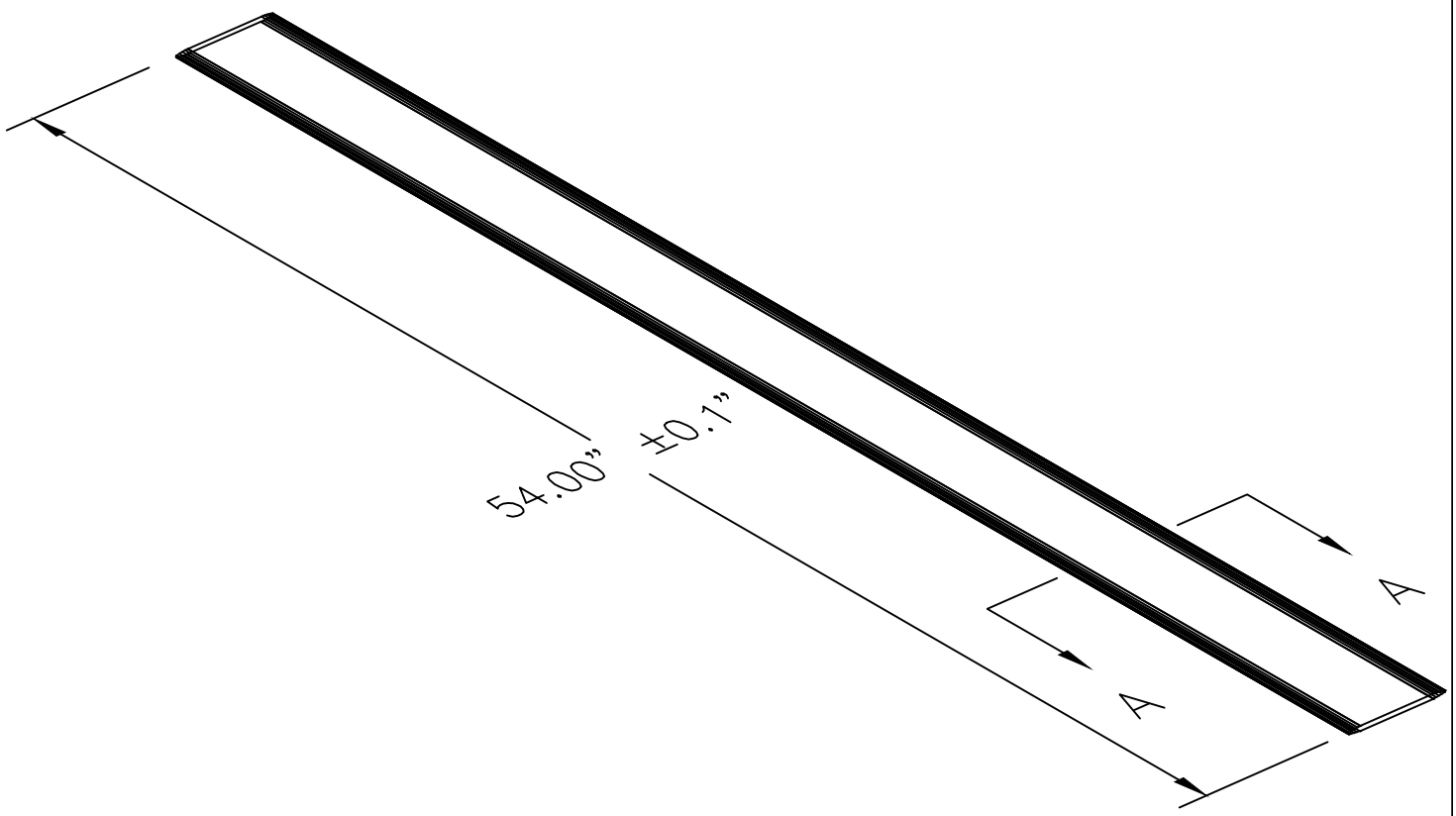
ISOMETRIC VIEW



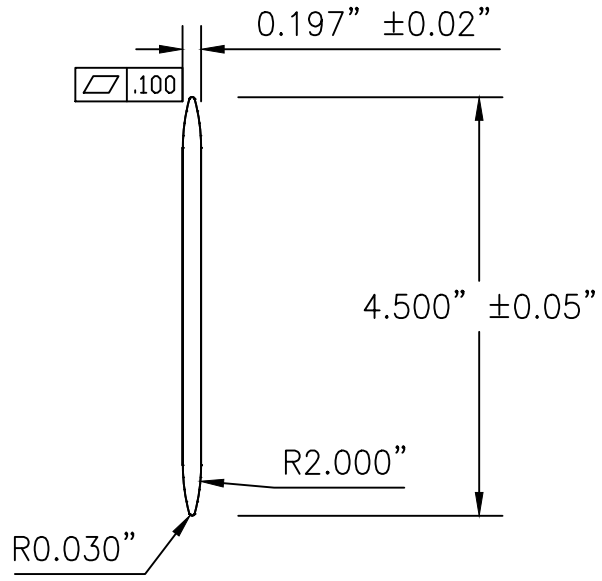
SECTION VIEW A–A

MATL: HDPE
ALL DIMENSIONS IN INCHES

Tensar.	DESCRIPTION: 1.5in. HDPE BODKIN BAR	PRODUCT CODE: OP150000
	FILE NAME: TIC3D_AR BODKIN BAR 1.5IN.DWG	THIS DRAWING IS BASED UPON SPECIFIC PROPERTIES OF TENSAR PRODUCTS (GEOGRIDS, DRAINAGE COMPOSITES AND EROSION MEDIA), WHICH ARE PROPRIETARY TO TENSAR CORPORATION 1210 CITIZENS PARKWAY, MORROW, GA, 30260 AND ARE PROTECTED BY US AND INTERNATIONAL PATENTS, AND LICENSING AGREEMENTS. YOUR USE OF THE INFORMATION CONSTITUTES YOUR ACKNOWLEDGMENT OF THE PROPRIETARY NATURE OF THESE DRAWINGS AND THE TECHNOLOGY.
	EFFECTIVE DATE: 02/01/2012	



ISOMETRIC VIEW



SECTION VIEW A–A

MATL: HDPE
ALL DIMENSIONS IN INCHES

Tensar.	DESCRIPTION: 4.5in. HDPE BODKIN BAR	PRODUCT CODE: OP157000
	FILE NAME: TIC3D_AR BODKIN BAR 4.5IN.DWG	THIS DRAWING IS BASED UPON SPECIFIC PROPERTIES OF TENSAR PRODUCTS (GEOGRIDS, DRAINAGE COMPOSITES AND EROSION MEDIA), WHICH ARE PROPRIETARY TO TENSAR CORPORATION 1210 CITIZENS PARKWAY, MORROW, GA, 30260 AND ARE PROTECTED BY US AND INTERNATIONAL PATENTS, AND LICENSING AGREEMENTS. YOUR USE OF THE INFORMATION CONSTITUTES YOUR ACKNOWLEDGMENT OF THE PROPRIETARY NATURE OF THESE DRAWINGS AND THE TECHNOLOGY.
	EFFECTIVE DATE: 02/01/2012	

Appendix 1.2.6.a

Long-term Connection Strength Tests

ARES® MSE Retaining Wall System

Back to main document

Long-term Connection Strength Test

ARES System Composite Connection

Prepared by Willie Liew, PE

dated May 16th, 2011

A long-term connection test program for Tensar ARES system is performed in the temperature controlled rooms at the Tensar Research and Development Laboratory in Morrow, Georgia. The test set up is comprised of UX1700 ARES tab (UX1700AR) wet-cast 2.5" minimum into concrete and then connected to UX1700MSE geogrid or UX1500MSE with HDPE bodkin bar. The long-term connection test set up is shown in Figure 1 and Figure 2. The concrete blocks are 4.25" long, 2.5" wide and 6" tall, cast with 5000 psi minimum concrete mix at Seminole Precast Manufacturing, Inc. Metal bars were placed at the bodkin connection and lightly clamped together to simulate overburden confining pressure in soil and to prevent the bodkin bar from rotating when creep load is applied.

For UX1700AR tab to UX1700MSE geogrid connection, two outer ribs of the UX1700AR tab and UX1700MSE geogrid specimens are cut so the middle 3 ribs of the UX1700AR tab are connected and under load along with the middle 4 ribs of the UX1700MSE geogrid. For UX1700AR tab to UX1500MSE geogrid connection, two outer ribs of the UX1500MSE geogrid are cut so that middle the 4 ribs are connected and under load along with 5 ribs of UX1700AR tab.

The test is performed with the 1.5" wide HDPE bodkin bar instead of the standard 4.5" wide HDPE bodkin bar to evaluate the long-term performance of the reduced size bodkin. The results to date and photographs of the test setup are enclosed. A total of 15 tests are set up: 11 tests with UX1700AR-UX1700MSE geogrid and 4 tests with UX1700AR-UX1500MSE geogrid. Five out of the eleven tests with UX1700AR-UX1700MSE and two of the four tests with UX1700AR-UX1500MSE are performed in the 40°C room. All other tests are performed in the 20°C room. To date, eleven tests have ruptured in both the 20°C and 40°C rooms with ruptured time ranged from 144 hours to 133,900 hours extrapolated.

All other tests are now well over 10,000 hours in real time and over 1,000,000 hours extrapolated as shown in Table 1 and Table 2. Using the conservative "x100" time shift factor determined from stress rupture tests for UX****MSE geogrid tested at 40°C, the tests at 40°C currently extrapolate to over 1,000,000 hours. The failure mode for each test is provided in Table 1 and Table 2, and also shown in Figure 3 and Figure 4.

Conservatively, as allowed by the standard of practice, using the on-going test times over 10,000 hours as "time-to-failure", the 75-year projected connection creep factor is 2.73 (CR_{cr} = 0.366) as shown in Figure 5.

Based on this long-term connection data, the long-term connection strength (T_{ac}) is conservatively calculated as

$T_{ac} = T_{ult} * CR_{cr} / RF_d$

For UX1700AR-UX1700MSE:

= 11990 lb/ft * 0.366 / 1.1

= 3989 lb/ft

For UX1700AR-UX1500MSE:

= 7810 lb/ft * 0.366 / 1.1

= 2599 lb/ft

The long-term connection strength calculated based on the long-term test data is also compared to the best case of long-term design strength (LTDS) of the geogrid which is

$T_{all} = T_{ult} / (RF_{cr} * RF_{id} * RF_d)$

For UX1700MSE:

= 11990 / (2.56*1.1*1.1)

= 3871 lb/ft

For UX1500MSE:

= 7810 / (2.56*1.1*1.1)

= 2521 lb/ft

Conclusions:

1. All long-term connection test results are higher than the long-term design strength (LTDS) of the geogrid. Ultimately, the long-term design strength of geogrid controls the design of the ARES connection system.

2. The connection strength is ultimately controlled by the geogrid rib. The geogrid samples are either rupturing close to the concrete block or at bodkin connection area. None of the geogrid samples is pulling out from the concrete block. The connection strength is not controlled by the 1.5” wide bodkin and/or the transverse bar since all tests failed on the geogrid ribs.

Table 1: Long-Term Connection Test with UX1700AR-UX1700MSE and 1.5” Wide Bodkin Bar

Test No.	T (°C)	Load (lb/ft)	T_{ult} (lb/ft)	$\%T_{ult}$	Shift factor	Time (hrs)	Shifted Time (hrs)	Test Status
1365	20	4852	12768	38.0	1	15124	15124	On-going
1366	20	5745	12768	45.0	1	11377	11377	Ruptured of UX1700AR tabs on ribs at bodkin connection
1374	20	5426	12768	42.5	1	6475	6475	Ruptured of UX1700AR tabs close to concrete block
1375	20	6783	12768	53.1	1	144	144	Ruptured of UX1700AR tabs on ribs at bodkin connection
1376	20	6384	12768	50.0	1	568	568	Ruptured of UX1700AR tab on ribs close to the block
1378	20	6065	12768	47.5	1	3545	3545	Ruptured of UX1700AR tabs on ribs at bodkin connection
1368	40	4501	12768	35.3	100	15124	1512400	On-going
1369	40	4852	12768	38.0	100	14983	1498300	On-going
1372	40	4150	12768	32.5	100	14691	1469100	On-going
1379	40	5426	12768	42.5	100	669.87	66987	Ruptured at UX1700AR tab on ribs close to the block
1380	40	5745	12768	45.0	100	352	35200	Ruptured of UX1700AR tab on ribs close to the block

Table 2: Long-Term Connection Test with UX1700AR-UX1500MSE and 1.5” Wide Bodkin Bar

Test No.	T (°C)	Load (lb/ft)	T_{ult} (lb/ft)	$\%T_{ult}$	Shift factor	Time (hrs)	Shifted Time (hrs)	Test Status
1382	20	3900	8013.5	48.7	1	549.87	549.87	Ruptured at UX1500MSE geogrid ribs
1381	40	3285	8013.5	41.0	100	591.95	59195	Ruptured at UX1500MSE geogrid ribs
1383	20	3695.5	8013.5	46.1	1	3192	3192	Ruptured at UX1500MSE geogrid ribs
1384	40	3120.6	8013.5	38.9	100	1339.6	133960	Ruptured at UX1500MSE geogrid ribs

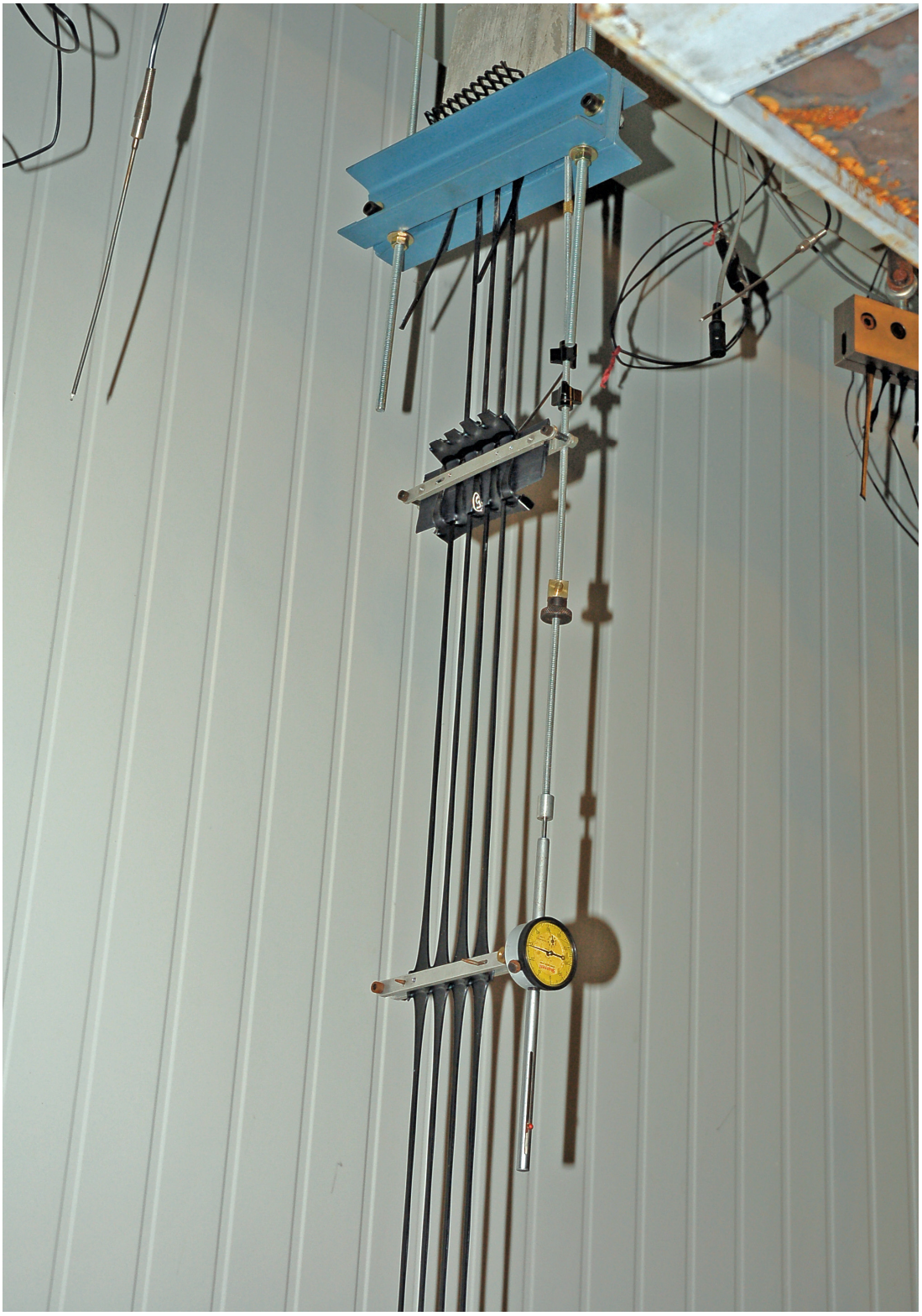


Figure 1: ARES Long-term composite connection test setup: UX1700AR tab wet-cast 2.5” into the concrete and connected to UX1700MSE or UX1500MSE geogrid with a 1.5” wide HDPE bodkin bar

Figure 2: Detailed view of UX1700AR tab connected to UX1700MSE geogrid with 1.5” wide HDPE bodkin bar

Figure 3: Typical ruptured test for UX1700AR-UX1700MSE connection

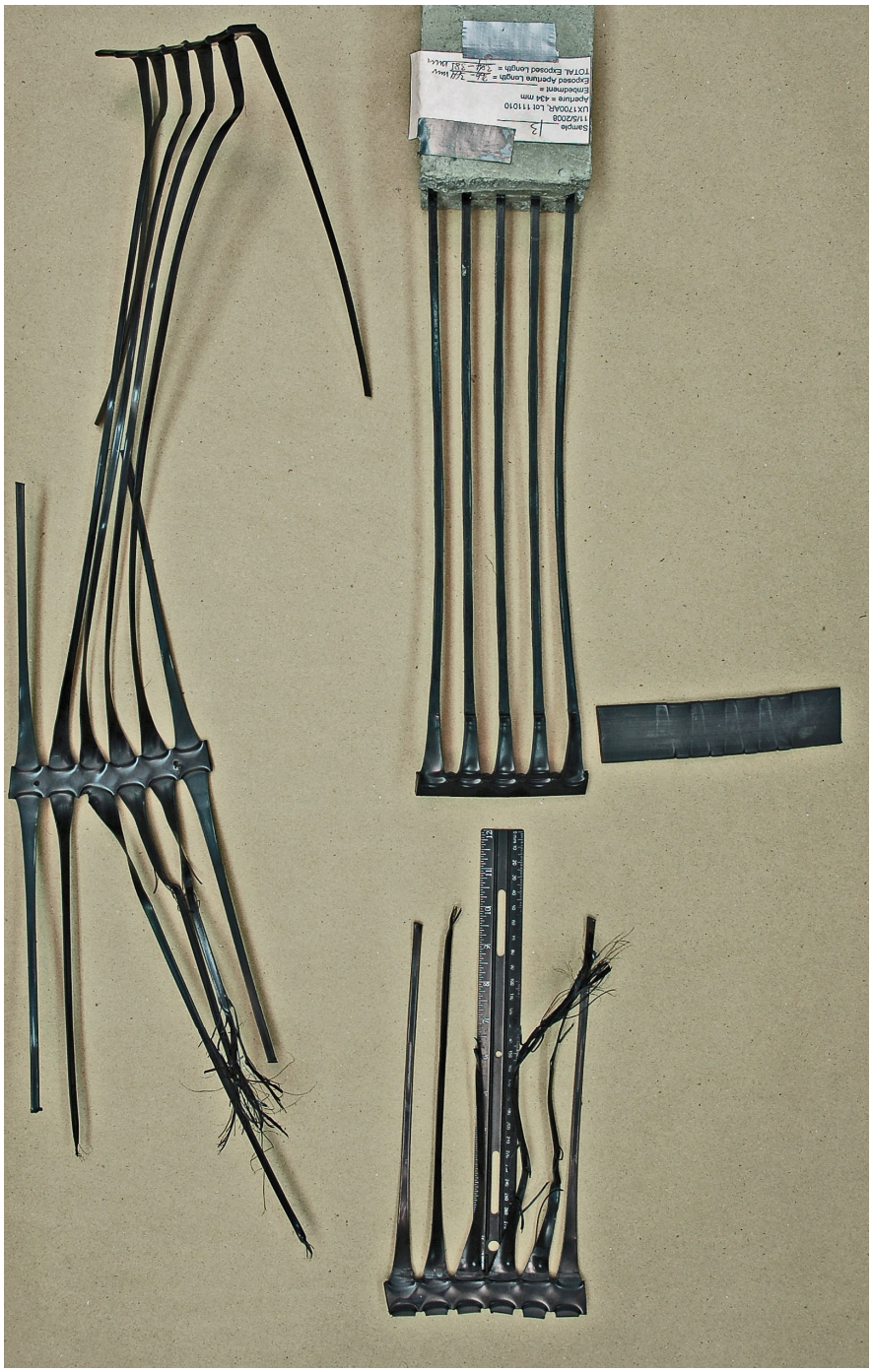


Figure 4: Typical ruptured test for UX1700AR-UX1500MSE connection

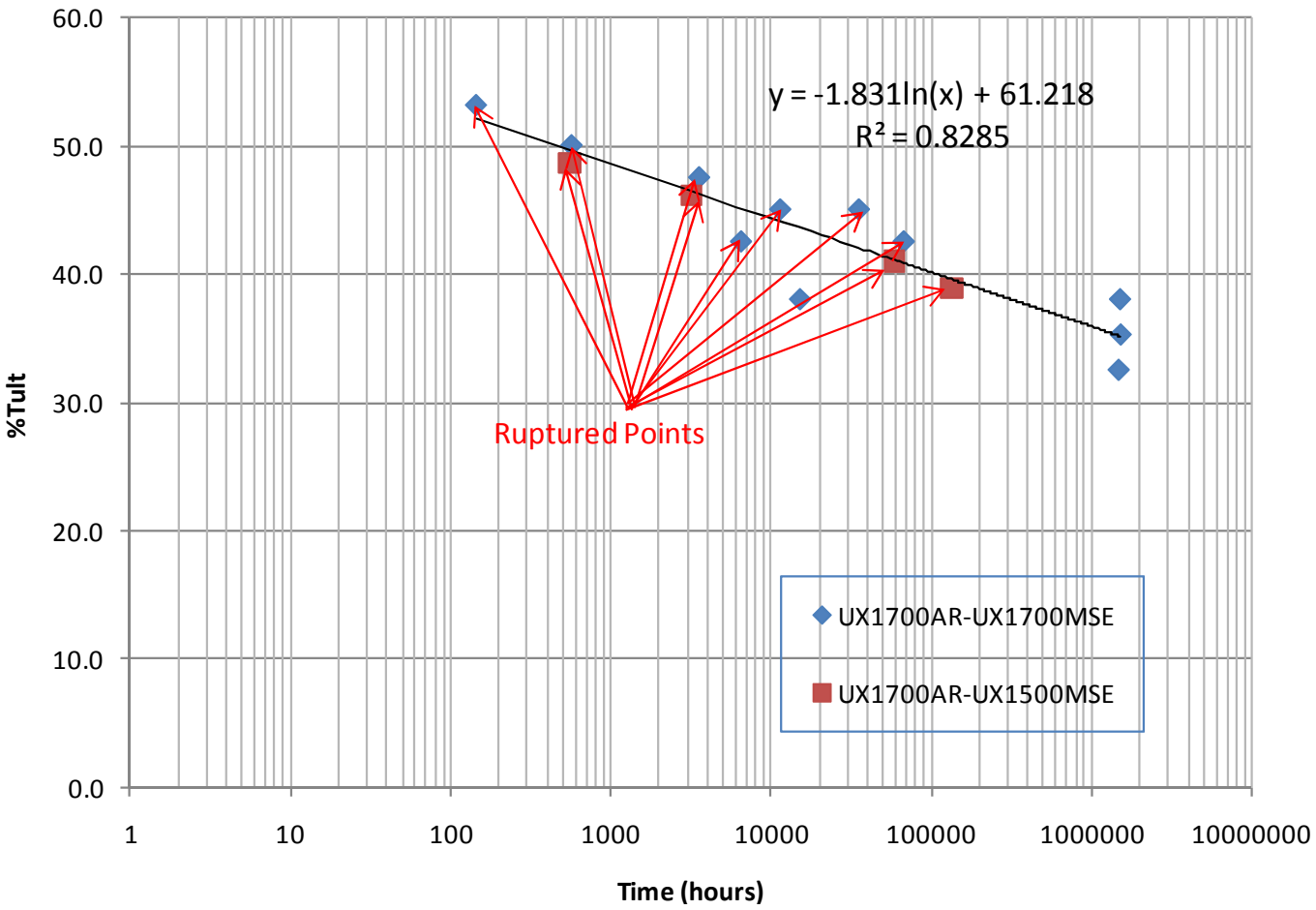


Figure 5: Long-term Bodkin Connection Creep Rupture Test Curve

Appendix 1.2.6.b

Ultimate Connection Strength Tests

ARES® MSE Retaining Wall System

Back to main document

ARES® CONCRETE BLOCK-to-UX1700AR (UX1700MSE) TAB
CONNECTION STRENGTH TESTS REPORT

by Willie Liew, P.E. August 25th, 2011

Six concrete blocks were wet cast at Seminole Precast Manufacturing, Inc. in Macon, Georgia on August 9th, 2011 with UX1700AR connection tab (Equivalent to UX1700MSE tab). The concrete blocks were 4.5" by 2.5" by 6" high. The tabs had 3 ribs and a single aperture that was embedded 2.5" in the concrete block. The concrete was a 5000 psi mix. However, the concrete blocks were allowed to cure to near 4000 psi compressive strength prior to perform the testing.

The tensile tests were performed at The Tensar Corporation QC Lab in Morrow on August 12th, 2011 when the concrete strength reached 3850 psi. The concrete cylinder information for this lot of concrete is provided in Appendix A. The sample of UX1700AR came from Lot # 310911 which has an ultimate lot tensile strength, $T_{ult\ lot}$ = 12178 plf and minimum average roll value (MARV) ultimate tensile strength, $T_{ult\ marv}$ = 11990 plf.

The MTS testing machine was modified to clamp the concrete block on the base as shown in Photographs 1 and 2. All 3 geogrid ribs of the tabs are loaded as shown in these photos. The gauge length of the sample was measured from the clamped upper CMD bar to the top of the concrete block. The samples were loaded at a strain rate of 10% per minute.

Tensile test data and load deformation plots are attached in Appendix B. Typical ruptured sample is shown in Photograph 3. As shown in the photograph, the ribs of the tabs either ruptured at or within the concrete or at the lower edge of the bodkin bar. The majority were at or in the concrete.

The average Concrete-to-UX1700AR ultimate connection strength was 11,297 plf. This is 92.8% of the ASTM D6637 ultimate geogrid tensile strength of this lot of geogrid. At 75 years design life with creep reduction factor RF_{cr} = 2.56 and durability reduction factor of RF_d = 1.1, the long-term connection strength of Concrete-UX1700AR is therefore 3951 plf (0.928 * $T_{ultMARV}$ / (RF_{cr} * RF_d). The Concrete-to-UX1700AR long-term connection strength is therefore higher than the long-term design strength of the geogrid tab at 3871 plf.

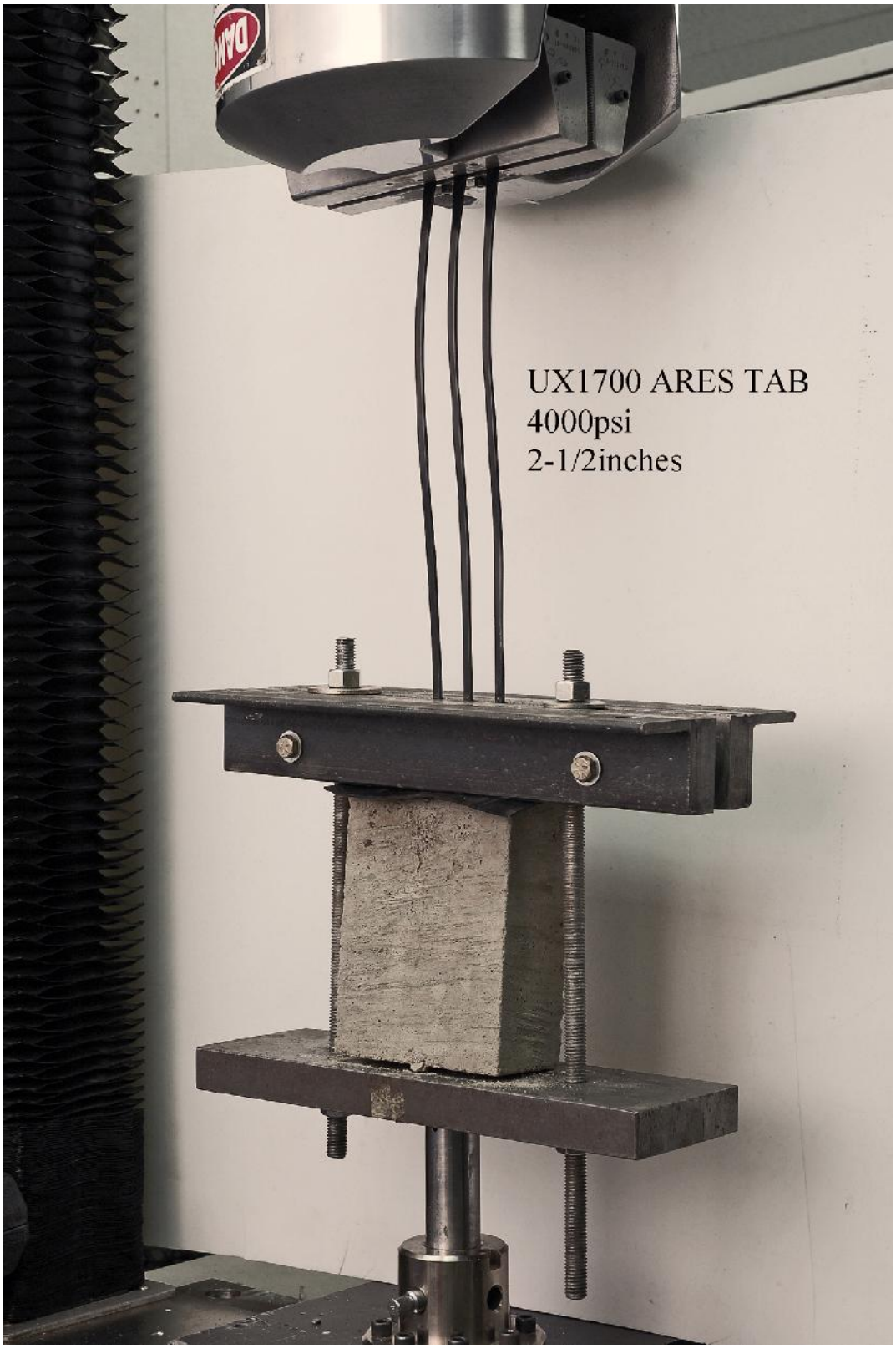


Photo 1 Front view of the 3-rib UX1700AR tab cast into concrete simulated facing unit holding down with clamp for tensile testing.

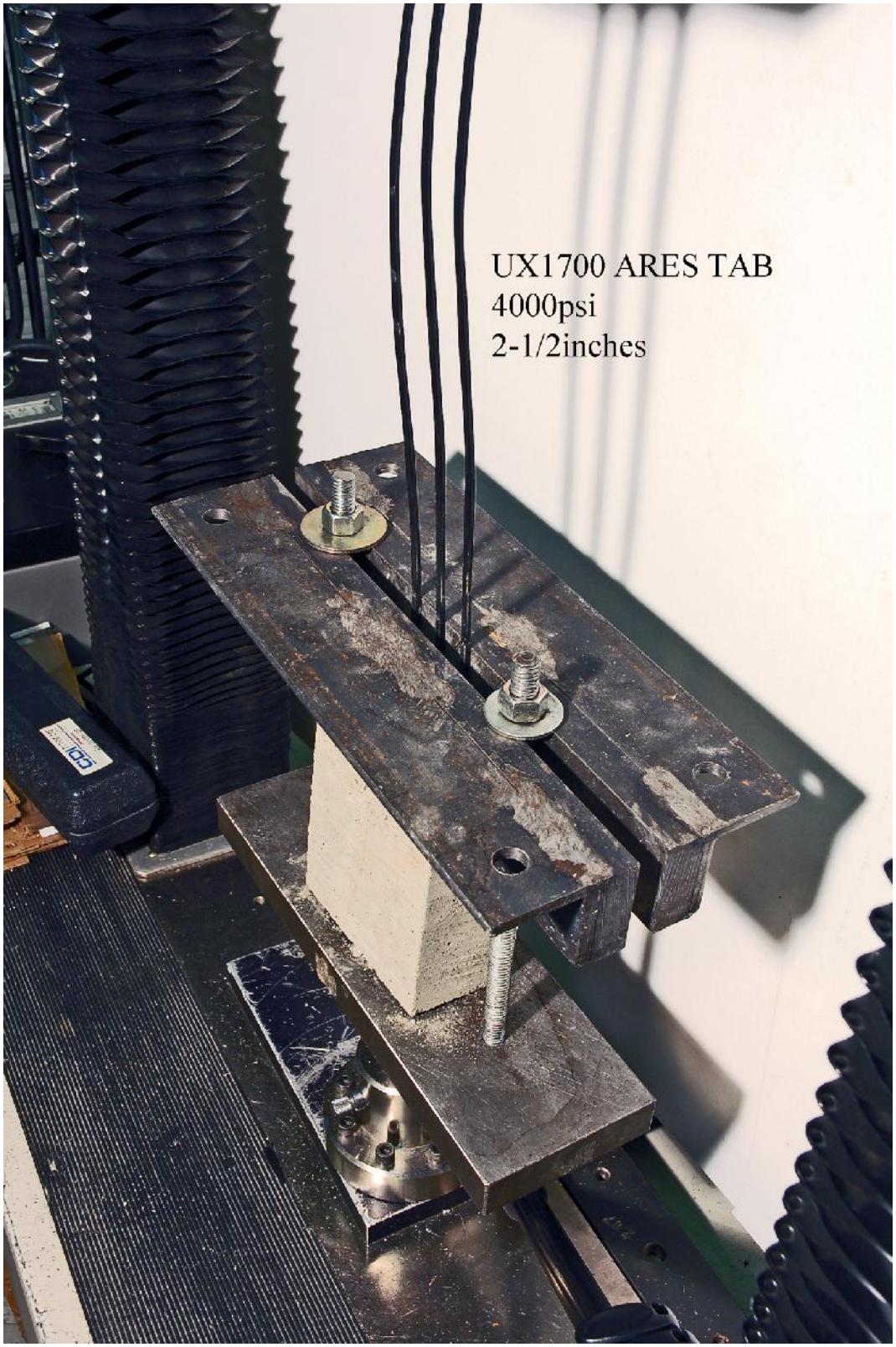


Photo 2 Side view of the concrete block with UX1700AR tab.

Photo 3 Ruptured samples from the tensile test

APPENDIX A

CONCRETE CYLINDER LOT INFORMATION

Seminole Precast Manufacturing, Inc.

220 Cargill Road Connector
Macon, Georgia 31216

Phone: (478) 781-2090
Fax: (478) 781-2098

8-25-2011

- **Slump 7 ¾”**
- **Air 4.0%**
- **Concrete Temp 90F**
- **Ambient Temp 96F**
- **Lot # 1011**
- **24 Hr Break 2327/2146**
- **48 Hr Break 3189**
- **72 Hr Break 3850**
- **7 Day Break 4563**

APPENDIX B

TENSILE TEST DATA

8/12/2011

Sample ID:	4000psiconcrete_UX1700tab2.5inch.mss	Test Date:	8/12/2011
Method:	WW Slack PreLoad being altered for save flow.msm	Operator:	MTS

Sample Results:

Name	Value	Units
Multiplier	15.360	
2% Tens Mod	1789.573	kN/m

Specimen Results:

Specimen #	Max Tens Str kN/m	Peak Load kN	Elongation at Peak in	Load @ 2% kN/m kN/m	Load @ 5% kN/m kN/m	Load@2%k N kN	Load@5%k N kN
1	167.306	10.892	2.035	34.971	65.864	2.277	4.288
2	167.013	10.873	1.949	37.262	69.086	2.426	4.498
3	163.557	10.648	1.904	36.830	70.028	2.398	4.559
4	169.099	11.009	2.135	31.488	65.455	2.050	4.261
5	161.124	10.490	2.326	3.820	38.202	0.249	2.487
6	161.181	10.494	1.932	36.497	68.024	2.376	4.429
Mean	164.880	10.734	2.047	30.144	62.777	1.963	4.087
Std. Dev.	3.399	0.221	0.160	13.067	12.170	0.851	0.792

Specimen #	%Strain At Peak %						
1	14.238						
2	13.638						
3	13.322						
4	14.936						
5	16.274						
6	13.522						
Mean	14.322						
Std. Dev.	1.123						

Specimen Comments:

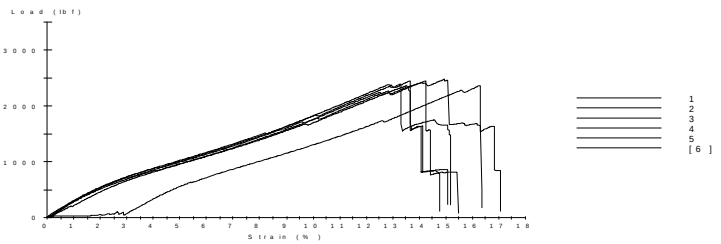
Specimen #	Comments
1	
2	
3	
4	
5	
6	

Calculation Inputs:

Name	Value	Units
Nominal Gage Length	363.000	mm
Slack Pre-Load	5.000	lbf
Slope Segment Length	20.000	%

Test Inputs:

Name	Value	Units
Break Sensitivity	90	%
Break Threshold	5.500	lbf
Data Acq. Rate	10.0	Hz
Sample Information 12		
Secondary Speed	2.0	in/min



Appendix 1.2.6.c

Connection Strength Analysis

ARES® MSE Retaining Wall System

Back to main document

ARES Connection Capacity Analysis

Connection testing for extensible soil reinforcement must consider the effects of installation damage on the connection strength whenever require by the agency or the project's owner. Connection strength tests that include placement and compaction of the reinforced fill (i.e., modular block geosynthetic reinforcement) as part of the connection test inherently have installation damage effects on the connection results. However, Connection tests performed in isolation (i.e., no reinforced fill) must include the potential for installation damage when determining the connection capacity. This is the case for the ARES system where the three components of the connection; 1.) long-term strength of the geogrid, 2.) long-term strength of the bodkin connection and 3.) long-term strength of the concrete panel to geogrid tab are all tested in isolation. The three components of the connection must be analyzed to determine the controlling mechanism of failure.

In accordance with AASHTO, the long-term design strength of the geogrid reinforcement ($LTDS_{geogrid}$) is determined as follows:

$$LTDS_{geogrid} = \frac{T_{ult}}{RF_D * RF_{ID} * RF_{CR}}$$

where:

- T_{ult} = Ultimate (or yield tensile strength) from wide width tensile strength tests (ASTM D 4595), based on minimum average roll value (MARV) for the product.
- RF_D = Durability reduction factor. It is dependent on the susceptibility of the geosynthetic to attack by microorganisms, chemicals, thermal oxidation, hydrolysis and stress cracking.
- RF_{ID} = Installation damage reduction factor.
- RF_{CR} = Creep reduction factor is the ratio of the ultimate strength (T_{ult}) to the creep limit strength obtained from laboratory creep tests for each product.

Long-term Bodkin Connection Strength ($LTDS_{bodkin}$) is determined as follows:

$$LTDS_{Bodkin} = \frac{T_{ult}}{RF_D * RF_{ID} * RF_{CRbodkin}}$$

where:

$RF_{CRBodkiin}$= Creep reduction factor is the ratio of the ultimate strength (T_{ult}) to the creep limit strength obtained from laboratory creep tests for the bodkin connection and each grid type product (2.73).

Long-term Connection Strength ($LTDS_{concrete/tab}$) Concrete Panel to Tab Insert is determined as follows:

$$LTDS_{concrete/tab} = \frac{T_{ult} * CR_u}{RF_D * RF_{ID} * RF_{CRtab}}$$

RF_{CRtab} = Creep reduction factor is the ratio of the ultimate strength (T_{ult}) to the creep limit strength obtained from laboratory creep tests for the tab insert (UX1700MSE) (2.54).

CR_u = Ultimate Connection/Ultimate- geogrid from testing = 0.93

Table 1 presents the results of the analysis of the components on the connection. The controlling mechanism for the connection strength is the strength of concrete panel/tab insert, as demonstrated by this analysis.

Table 1. Tensar International MSE Geogrid Strength Parameters (75 year Design Life)

Reinforcement	T_{ult} (lbs/ft)	RF_{CR}	RF_D	RF_{ID}	$LTDS_{geogrid}$ (lbs/ft)	$RF_{CRBodkin}$	$LTDS_{Bodkin}$ (lbs/ft)	CR_u	RF_{CRtab}	$LTDS_{concrete/tab}$ (lbs/ft)
Reinforced Fill Silty Sand										
UX1400MSE	4,800	2.68	1.1	1.15	1,416	2.73	1,390	93	2.54	1,390
UX1500MSE	7,810	2.54	1.1	1.14	2,452	2.73	2,281	93	2.54	2,281
UX1600MSE	9,870	2.54	1.1	1.15	3,072	2.73	2,858	93	2.54	2,857
UX1700MSE	11,990	2.54	1.1	1.18	3,637	2.73	3,384	93	2.54	3,382
Reinforcement	T_{ult} (lbs/ft)	RF_{CR}	RF_D	RF_{ID}, Sandy Gravel	*LTDS* (lbs/ft)	$RF_{CRBodkin}$	$LTDS_{Bodkin}$ *(lbs/ft)*	CR_u	RF_{CRtab}	$LTDS_{concrete/tab}$ (lbs/ft)
Reinforced Fill Sandy Gravel										
UX1400MSE	4,800	2.68	1.1	1.35	1,206	2.73	1,184	93	2.54	1,184
UX1500MSE	7,810	2.54	1.1	1.34	2.086	2.73	1,941	93	2.54	1,941
UX1600MSE	9,870	2.54	1.1	1.35	2,617	2.73	2,434	93	2.54	2,434
UX1700MSE	11,990	2.54	1.1	1.39	3,087	2.73	2,872	93	2.54	2,872
Reinforcement	T_{ult} (lbs/ft)	RF_{CR}	RF_D	RF_{ID}, 57 Stone	*LTDS* (lbs/ft))	$RF_{CRBodkin}$	$LTDS_{Bodkin}$ *(lbs/ft)*	CR_u	RF_{CRtab}	$LTDS_{concrete/tab}$ (lbs/ft)
Reinforced Fill #57 Stone										
UX1400MSE	4,800	2.68	1.1	2.57	634	2.73	622	93	2.54	622
UX1500MSE	7,810	2.54	1.1	2.05	1,364	2.73	1,269	93	2.54	1,269
UX1600MSE	9,870	2.54	1.1	1.86	1,899	2.73	1,767	93	2.54	1,767
UX1700MSE	11,990	2.54	1.1	1.78	2,411	2.73	2,242	93	2.54	2,242

MSEW does not include the direct ability to include installation damage in the determination of the connection capacity. It is therefore necessary to modify the input into MSEW to account for installation damage. The terms used in MSEW are reduction factor for degradation, RF_D, reduction factor for creep, RF_{CR}, and CR_u. The degradation reduction factor is consistent for all modes of connection failure and should be 1.1. The controlling failure mechanism of the connection is the concrete/tab insert connection. The creep reduction factor for this mode of failure if the creep reduction factor for the tab connector (UX1700 = 2.54). In order to account for the installation damage, the CR_u is divided by the appropriate installation damage reduction factor for the gradation of the reinforced fill being used (i.e. CR_u/RF_{ID}).

Appendix 1.2.7

Pullout Tests in Soil Report

Tensar UX****MSE Geogrids

ARES® 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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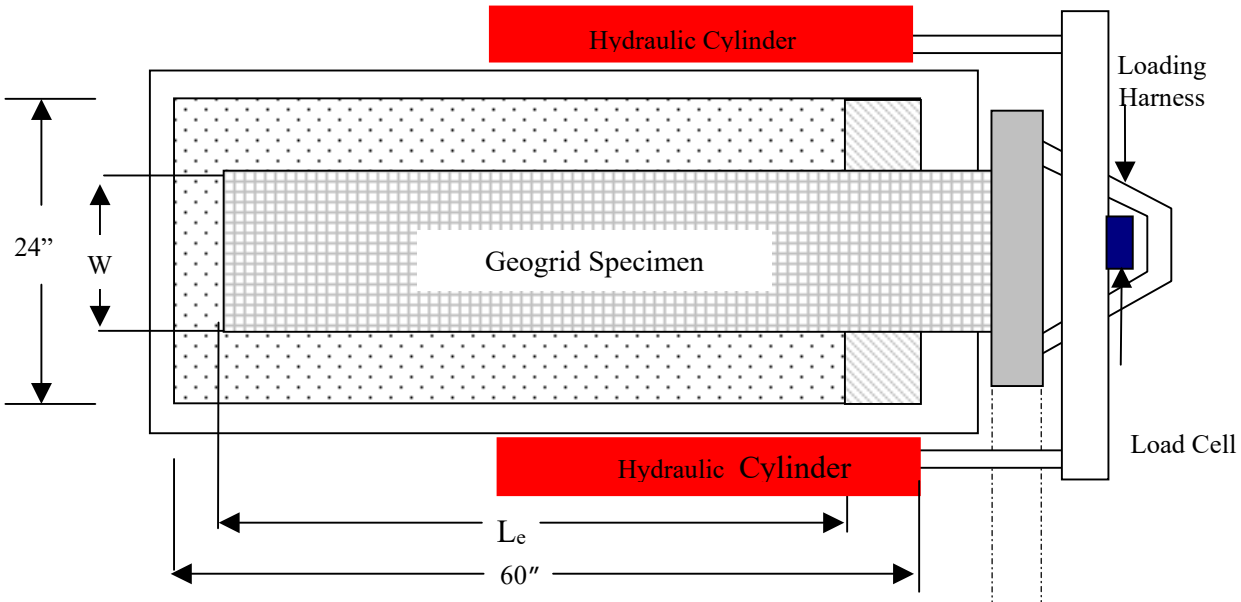

Figure 1. Plan view of pullout test setup (Note: not to scale).

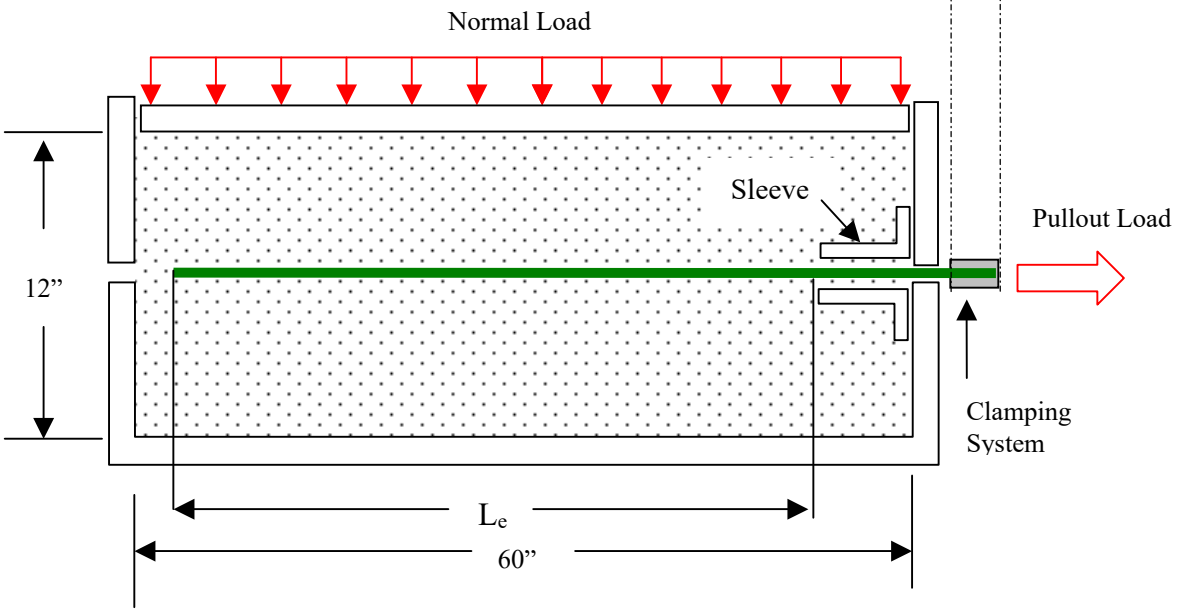

Figure 2. Cross-section of pullout test setup (Note: not to scale).

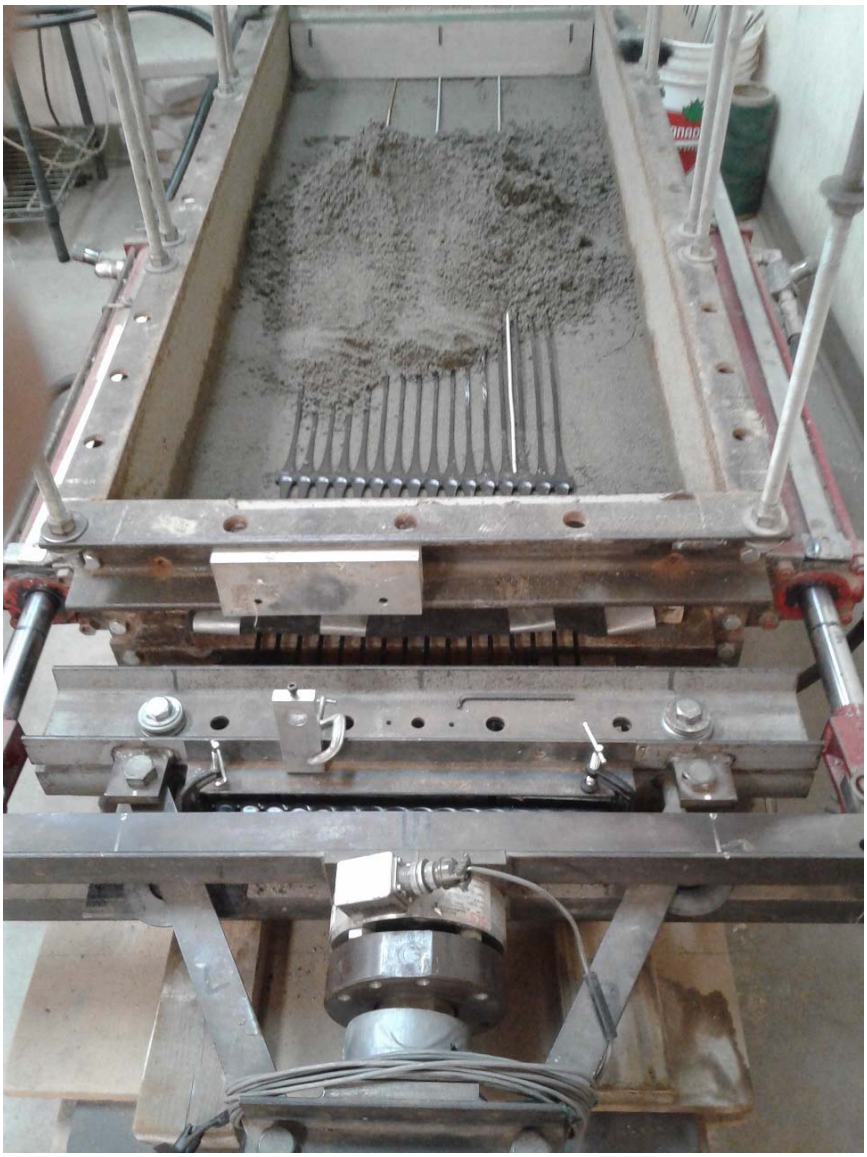

Figure 3. UX1700HS geogrid placed on compacted concrete sand.

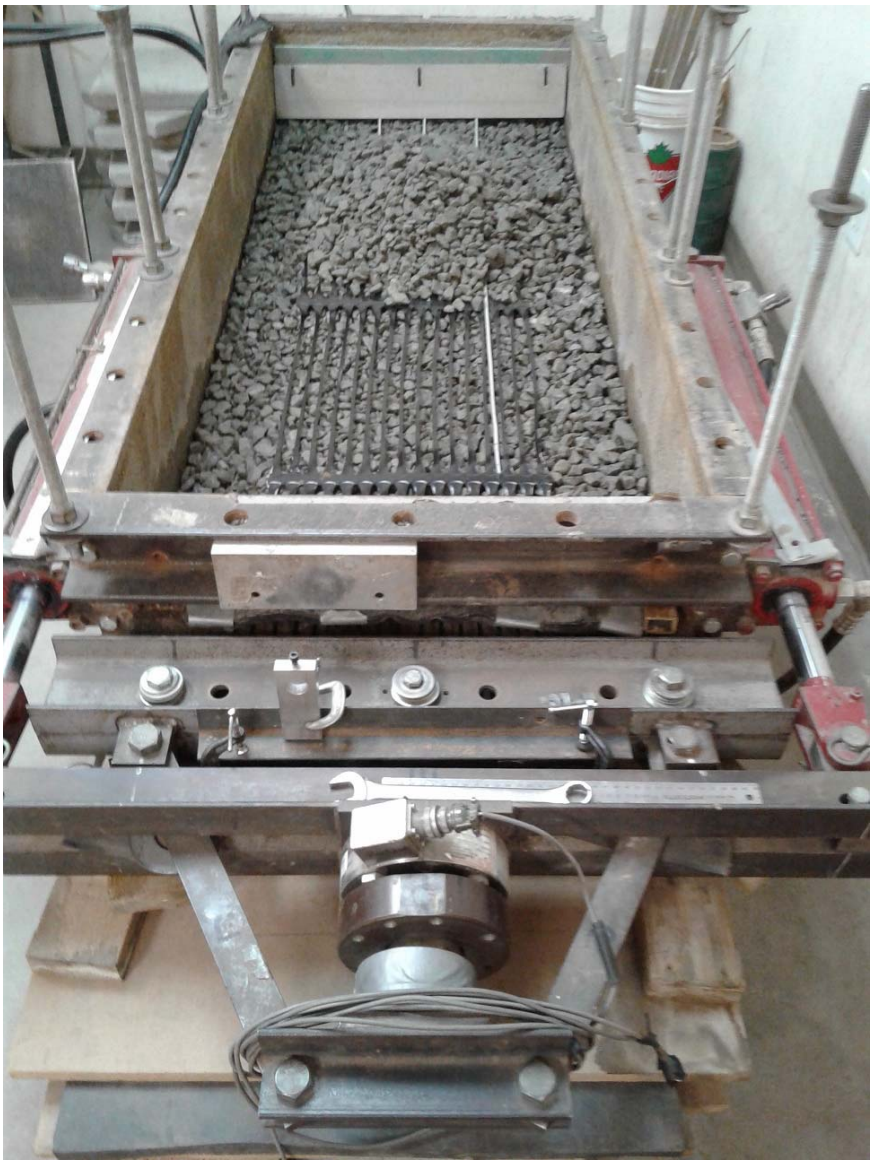

Figure 4. UX1700HS geogrid placed on compacted AASHTO #57 stone.

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

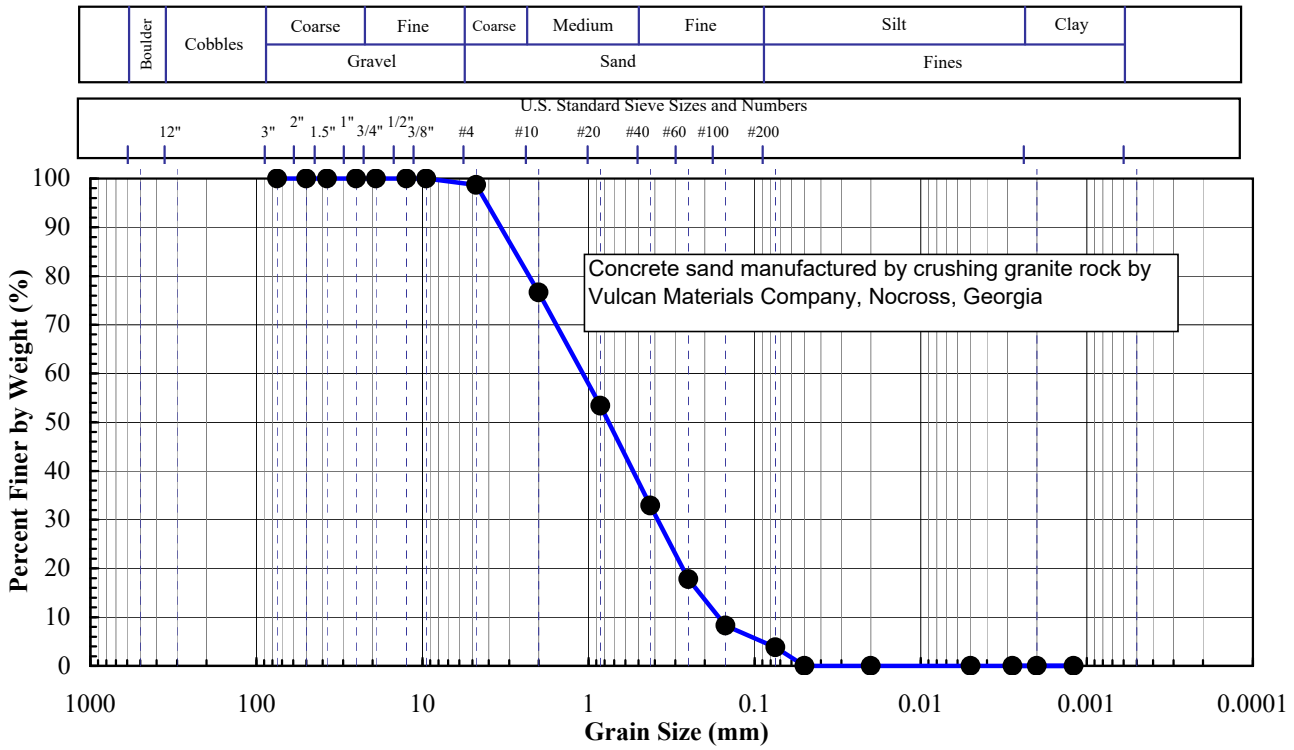
4405 International Blvd., Suite B-117, Norcross, GA 30093
Ph: (770) 931 8222 Fax: (770) 931 8240

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

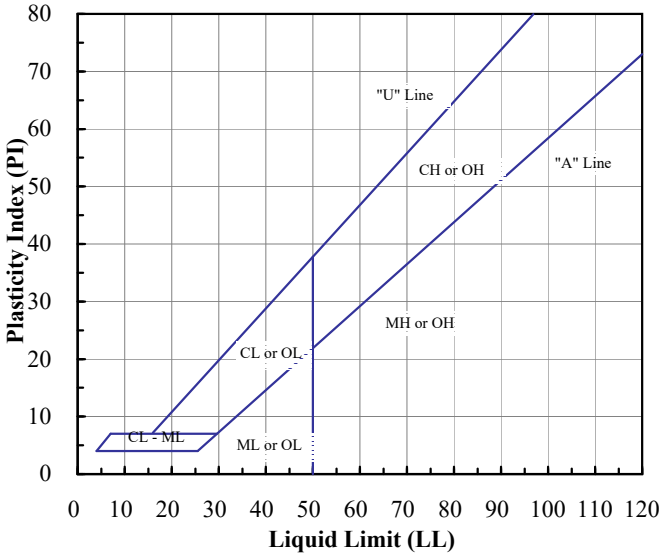


Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
1/2"	12.5	100.0
3/8"	9.5	100.0
#4	4.75	98.7
#10	2.00	76.6
#20	0.850	53.4
#40	0.425	32.9
#60	0.250	17.8
#100	0.150	8.3
#200	0.075	3.8

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)

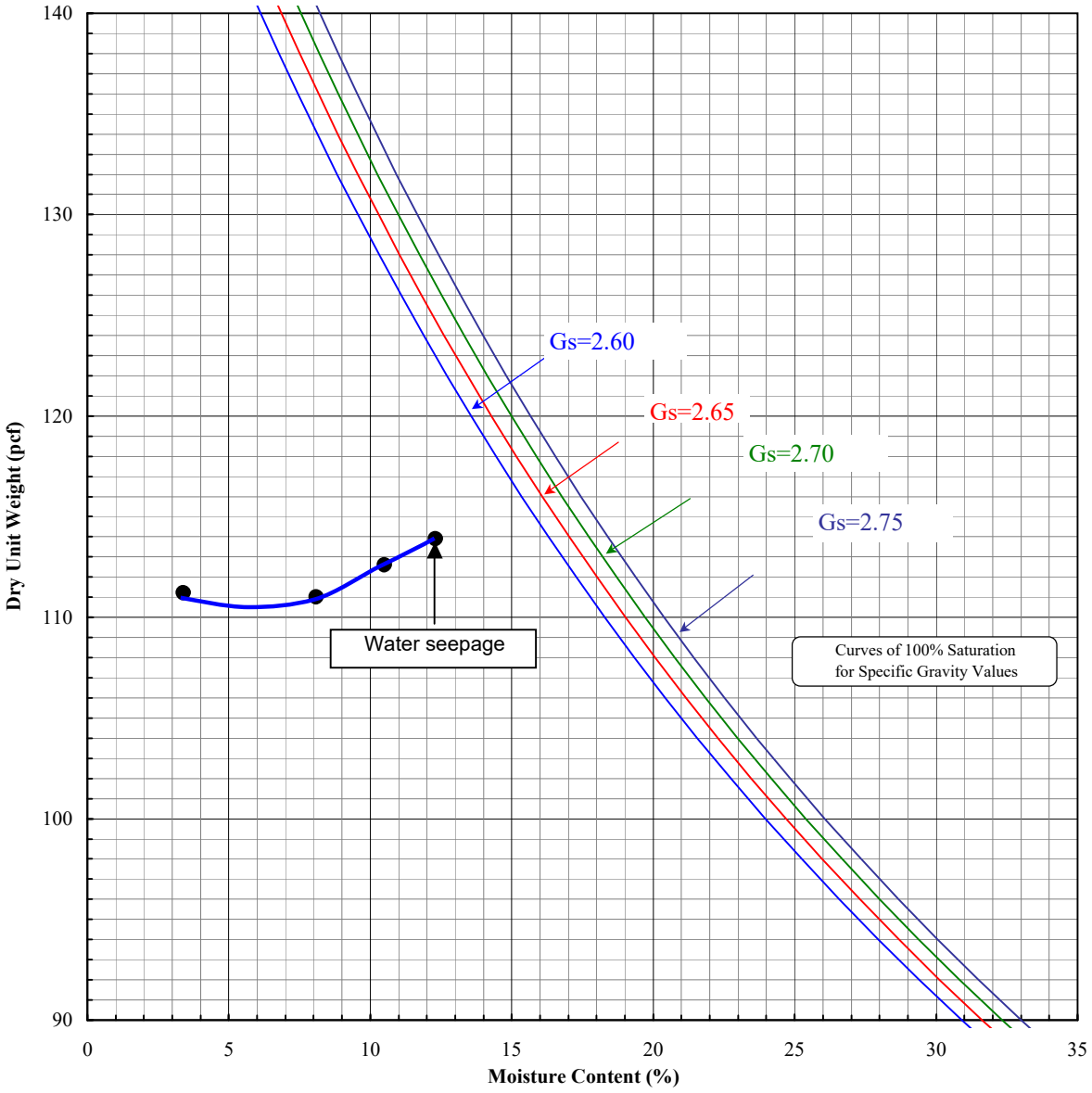
Note(s):

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)

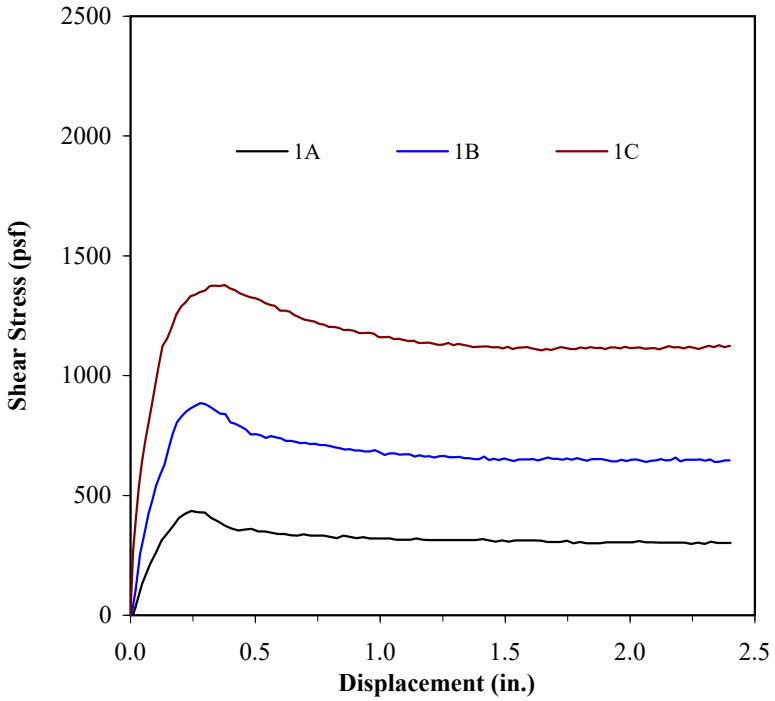


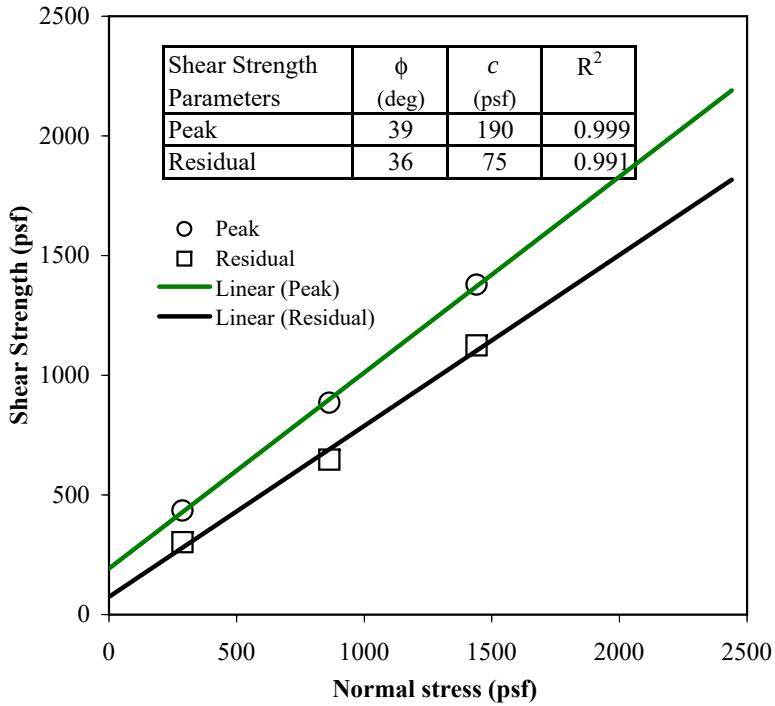
Client Sample ID.	SGI Sample ID NO.	Compaction Data		Maximum Dry Unit Weight and Optimum Moisture Content			
				Measured		Corrected	
		ω_i (%)	γ_d (pcf)	ω_{om} (%)	γ_{dmax} (pcf)	ω_{om} (%)	γ_{dmax} (pcf)
Concrete Sand	SGISP	3.4	111.2	12.0	114.0		
		8.1	111.0				
		10.5	112.6				
		12.3	113.9				

% 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

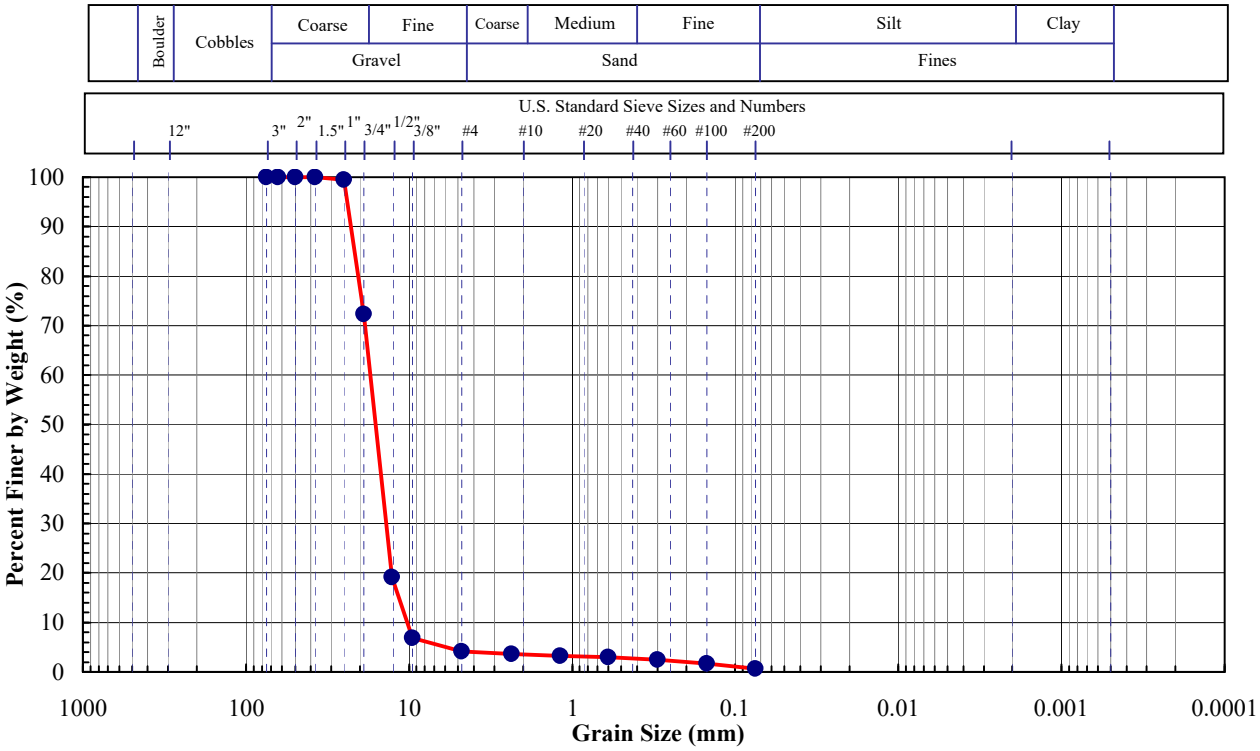
4405 International Blvd., Suite B-117, Norcross, GA 30093
Ph: (770) 931-8222 Fax: (770) 931-8240

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

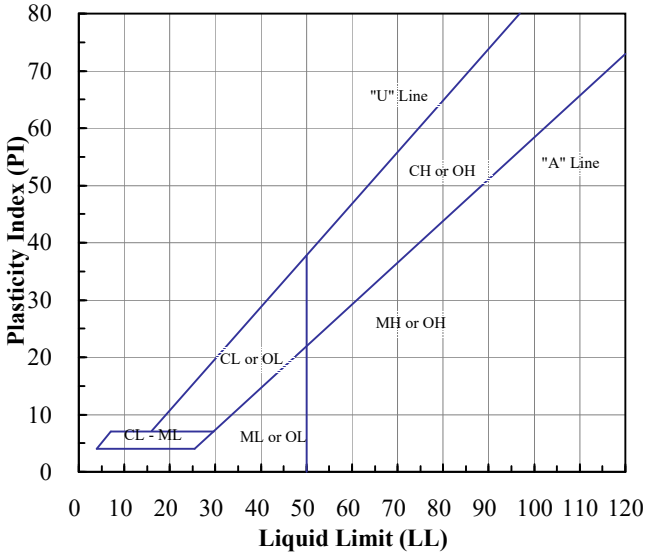


Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	99.5
3/4"	19	72.3
1/2"	12.5	19.2
3/8"	9.5	6.8
#4	4.75	4.1
#8	2.36	3.6
#16	1.180	3.2
#30	0.600	3.0
#50	0.300	2.5
#100	0.150	1.7
#200	0.075	0.7

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 (%):	

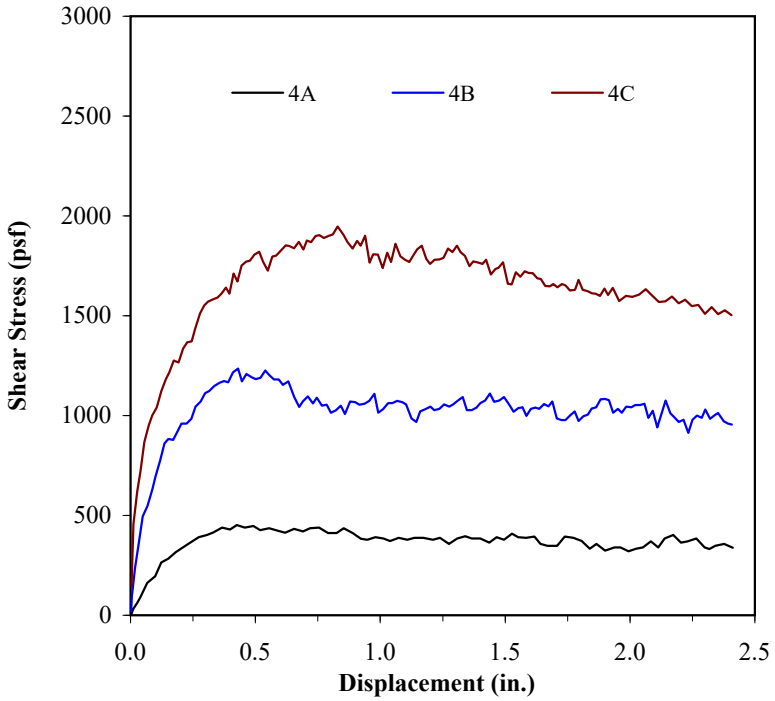
Coeff. Unif. (Cu):	1.7
Coeff. Curv. (Cc):	1.1

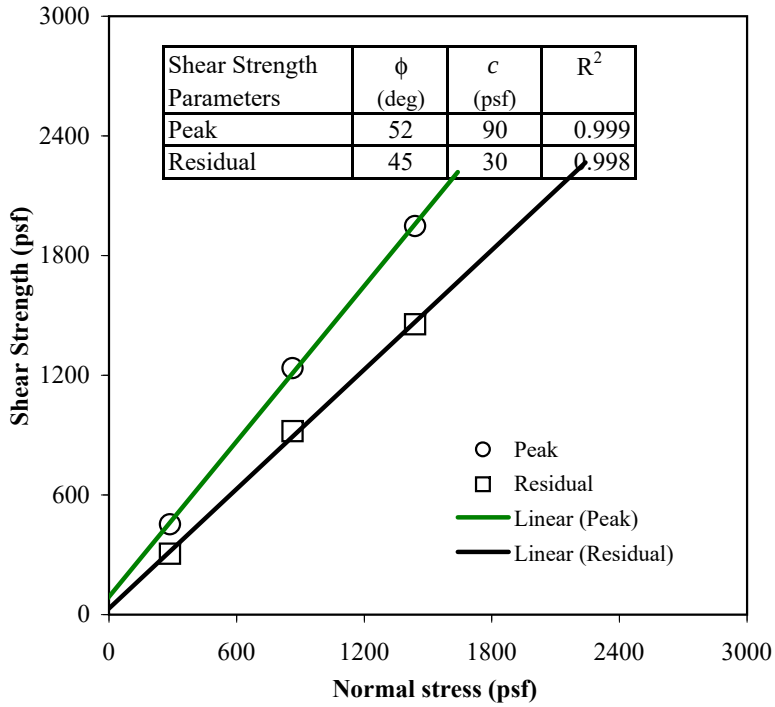


Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits LL (%)	Atterberg Limits PL (%)	Atterberg Limits PI (-)	Engineering Classification
AASHTO #57 Stone	SGIGP		0.7				GP

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

SGI TESTING SERVICES, LLC

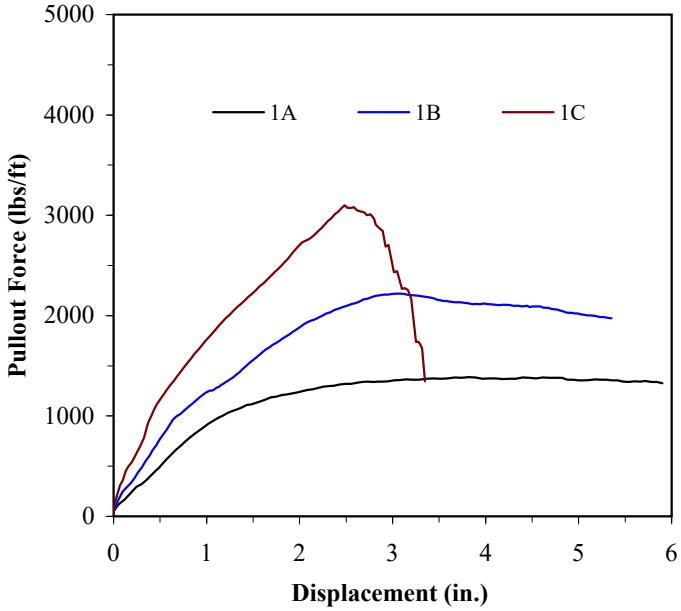
FIGURE NO.	A-5
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

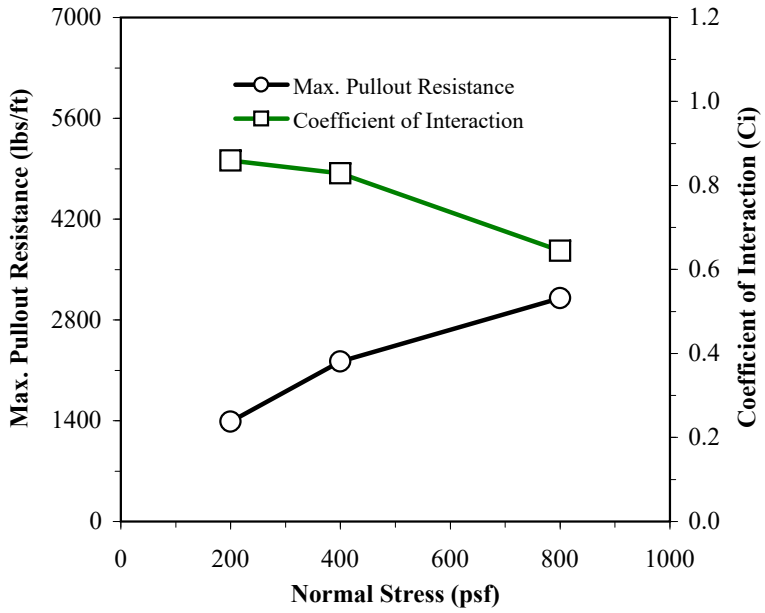
APPENDIX B

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.

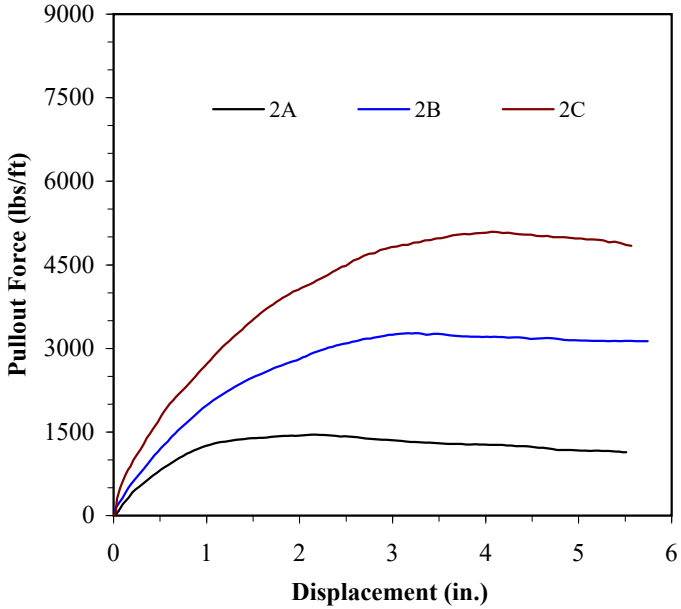
DATE REPORTED: 6/21/2022

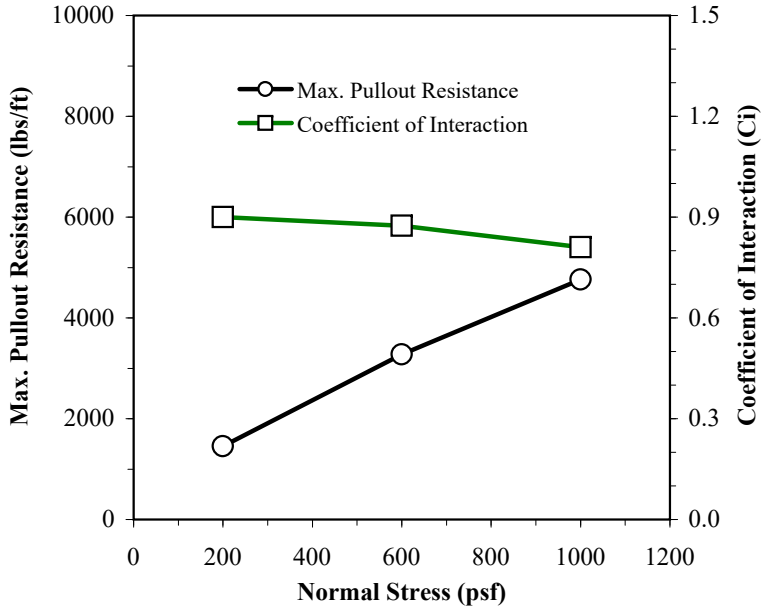
SGI TESTING SERVICES, LLC

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.

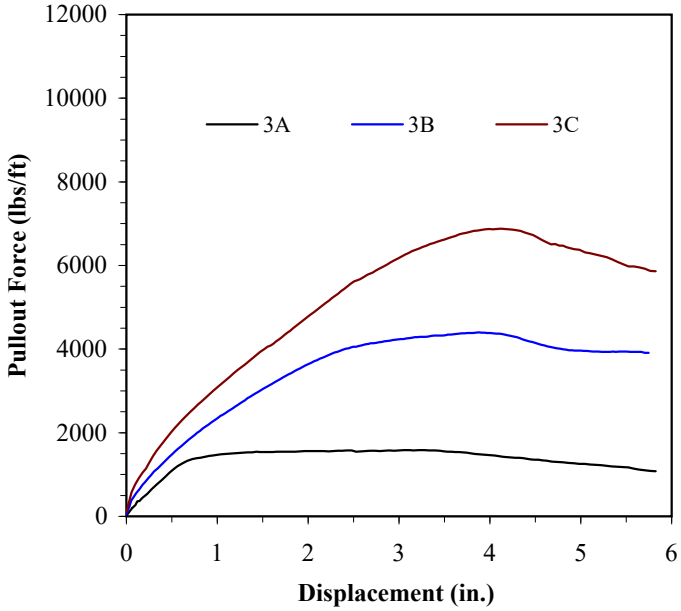
DATE REPORTED: 6/21/2022

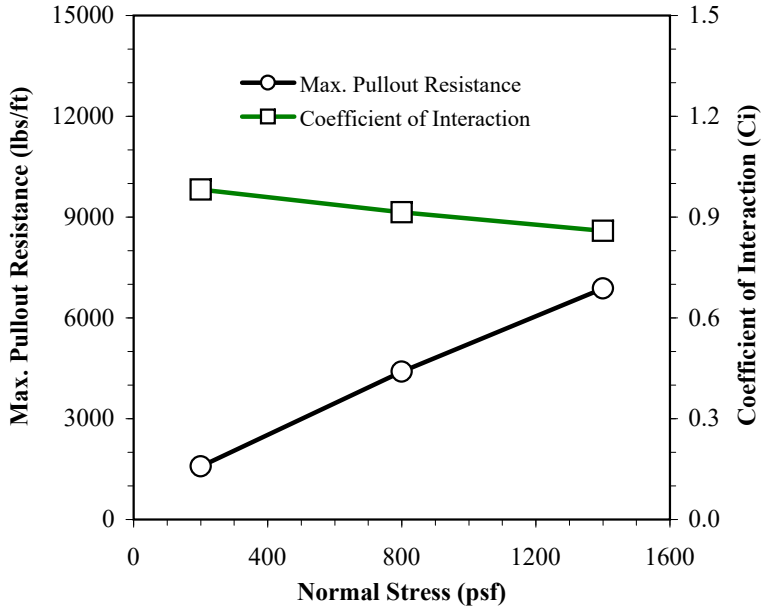
SGI TESTING SERVICES, LLC

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.

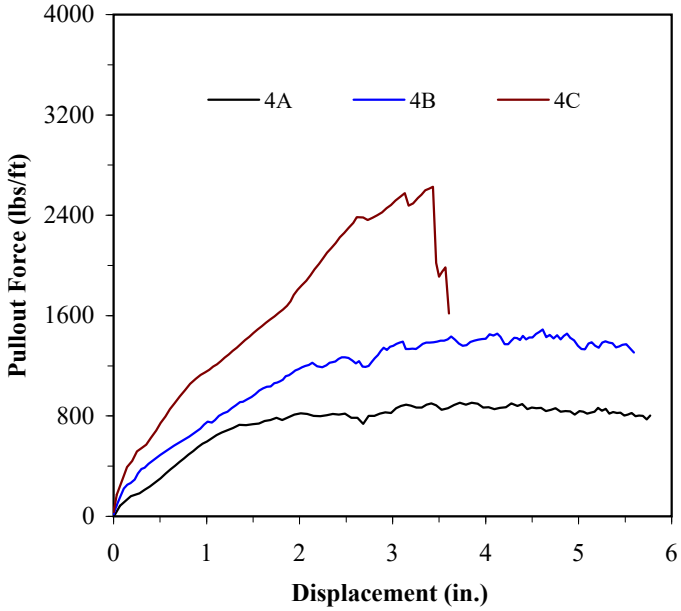
DATE REPORTED: 6/21/2022

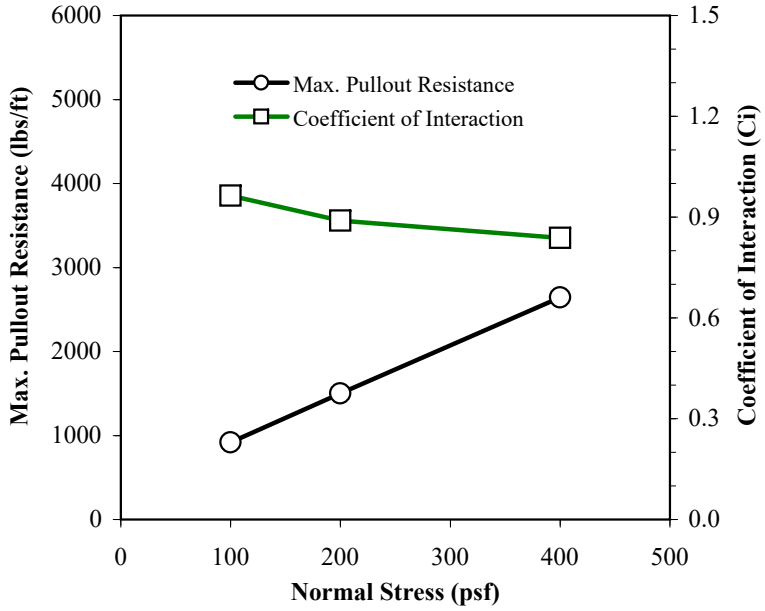
SGI TESTING SERVICES, LLC

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.

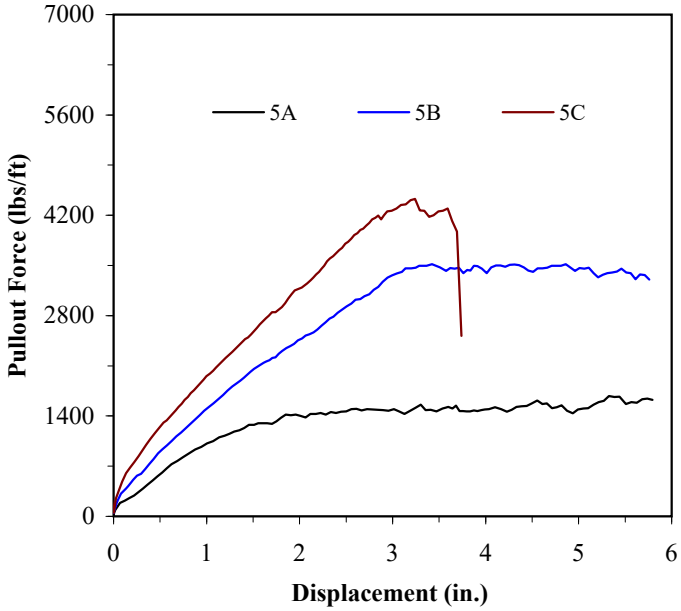
DATE REPORTED: 6/21/2022

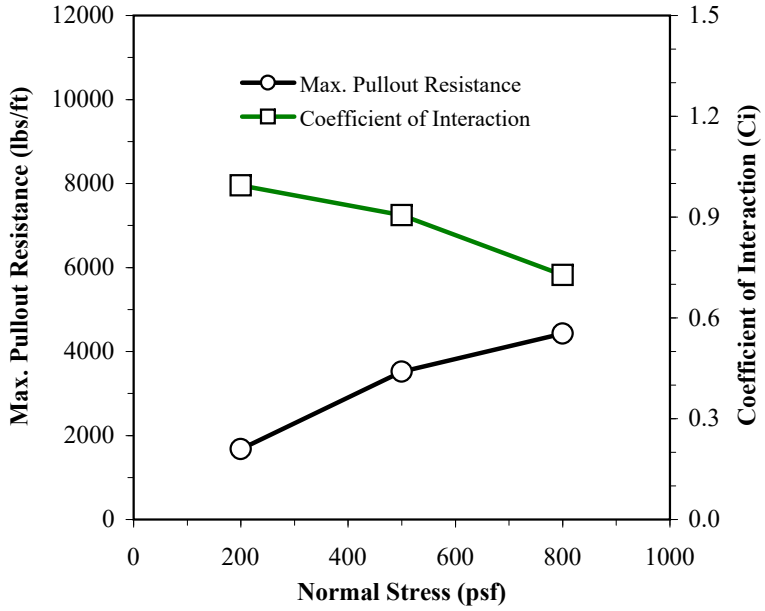
SGI TESTING SERVICES, LLC

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.

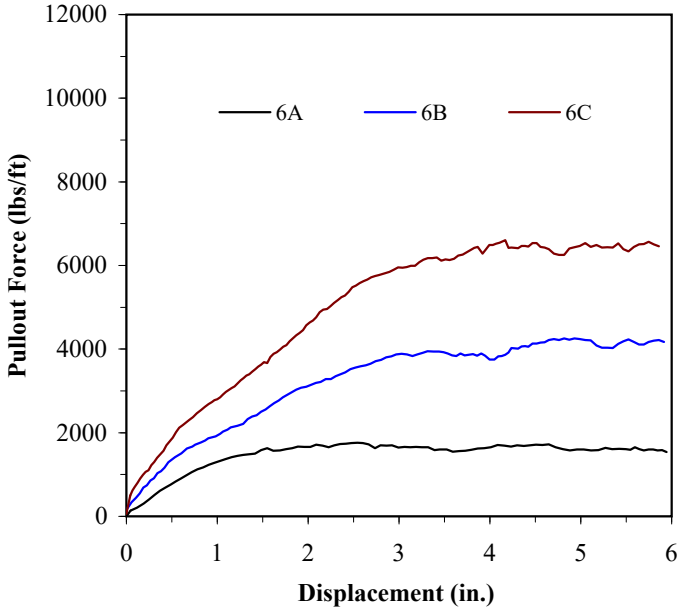
DATE REPORTED: 6/21/2022

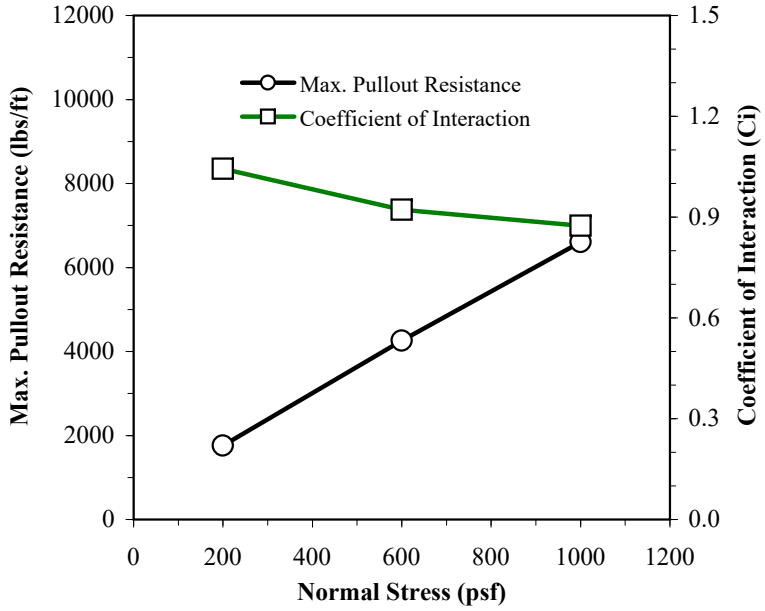
SGI TESTING SERVICES, LLC

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.

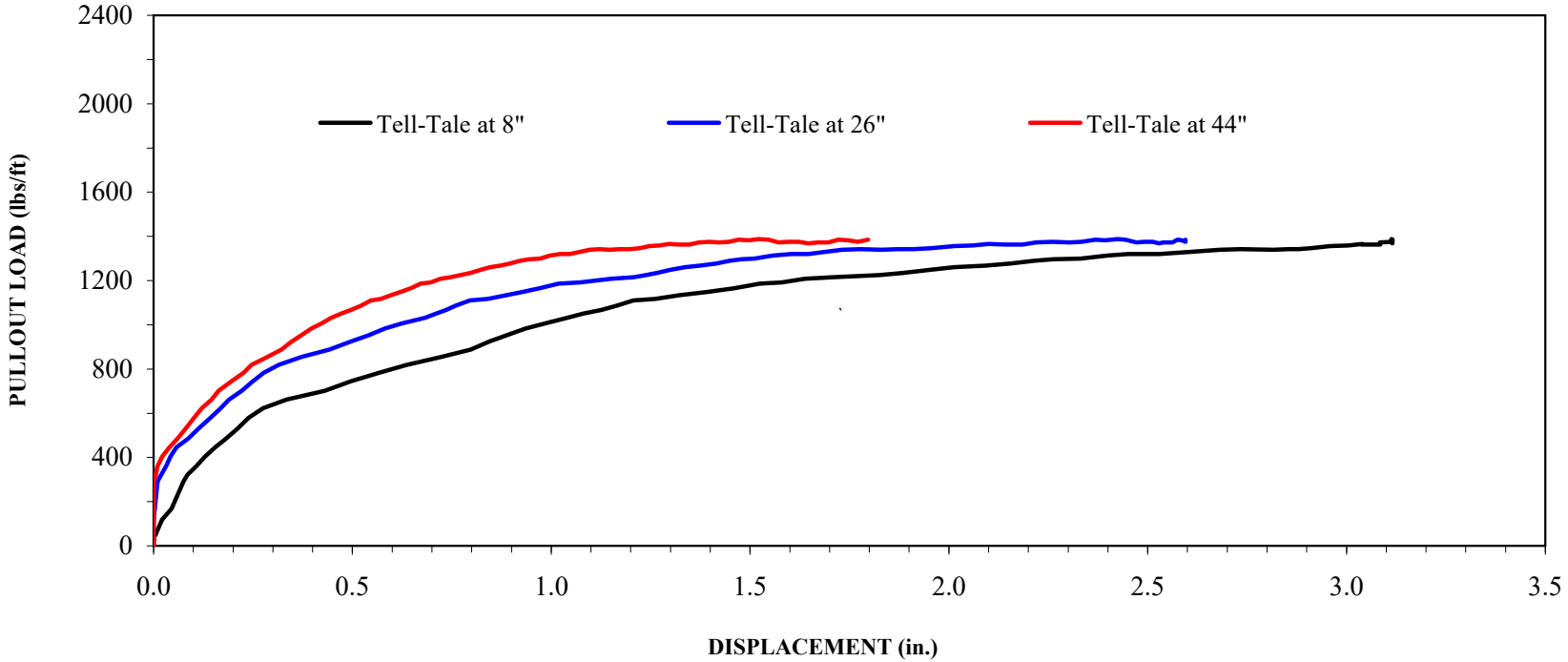
DATE REPORTED:	6/21/2022
FIGURE NO.	6
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

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

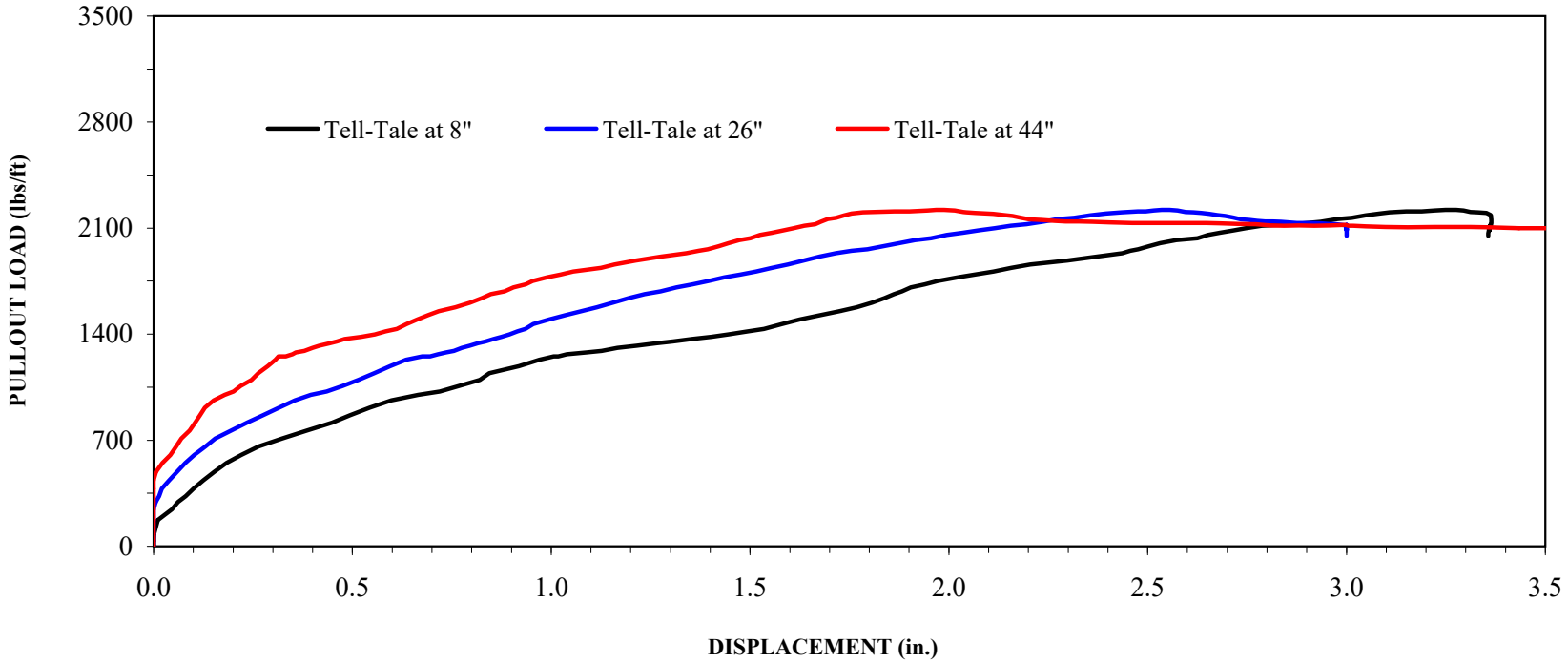
NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout 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



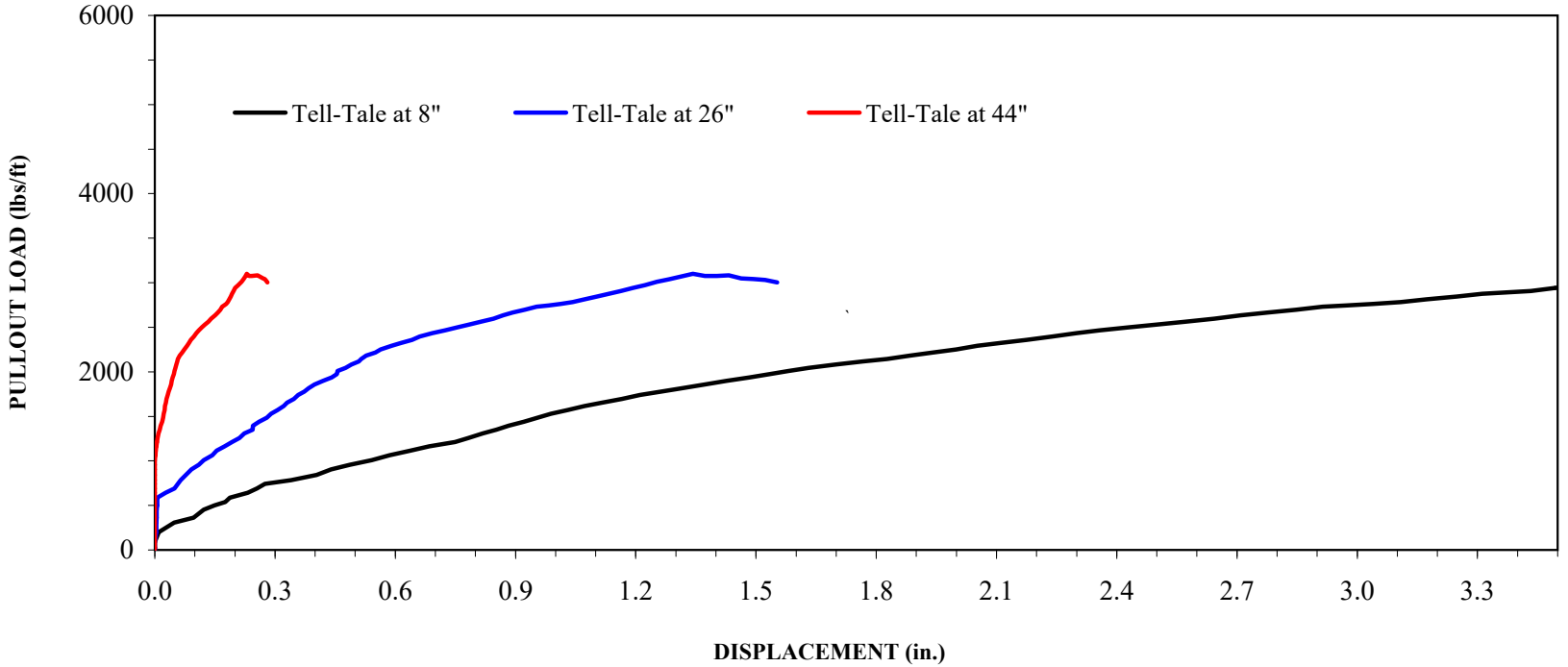
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
1B	14.0	44.0	400	8 26 44	44 144 244	86 292 492	7 24 41

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

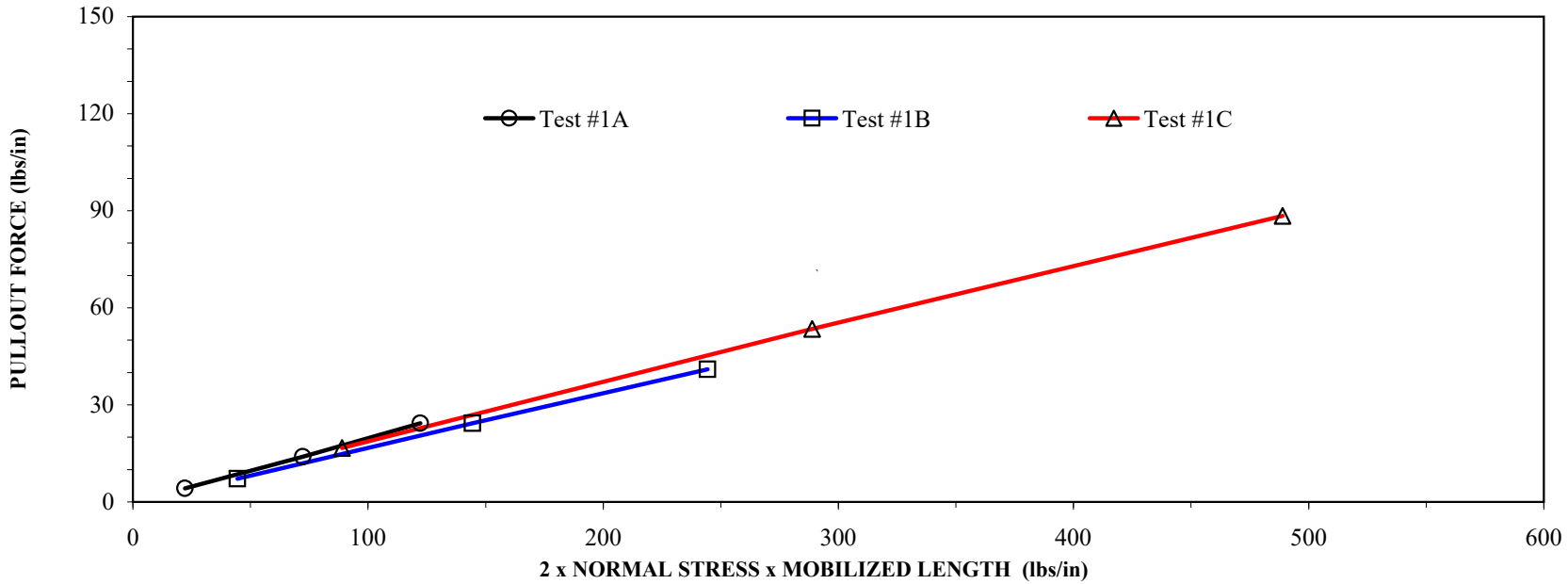
NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout 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 $\alpha^{(1)}$ (-)	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		

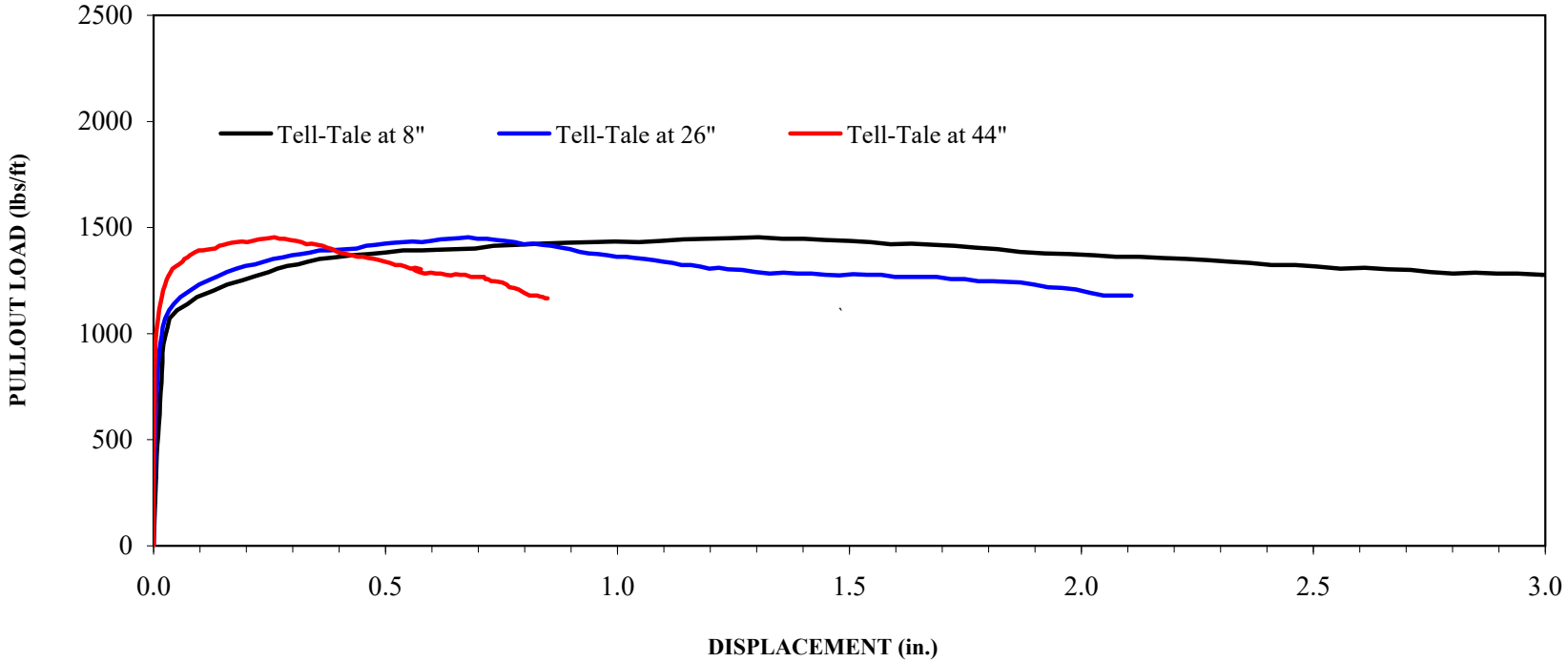
NOTE: (1) If the calculated α factor > 1 by the FHWA method (Appendix A FHWA-NHI-00-043), the reported α factor value is corrected to 1.0.

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



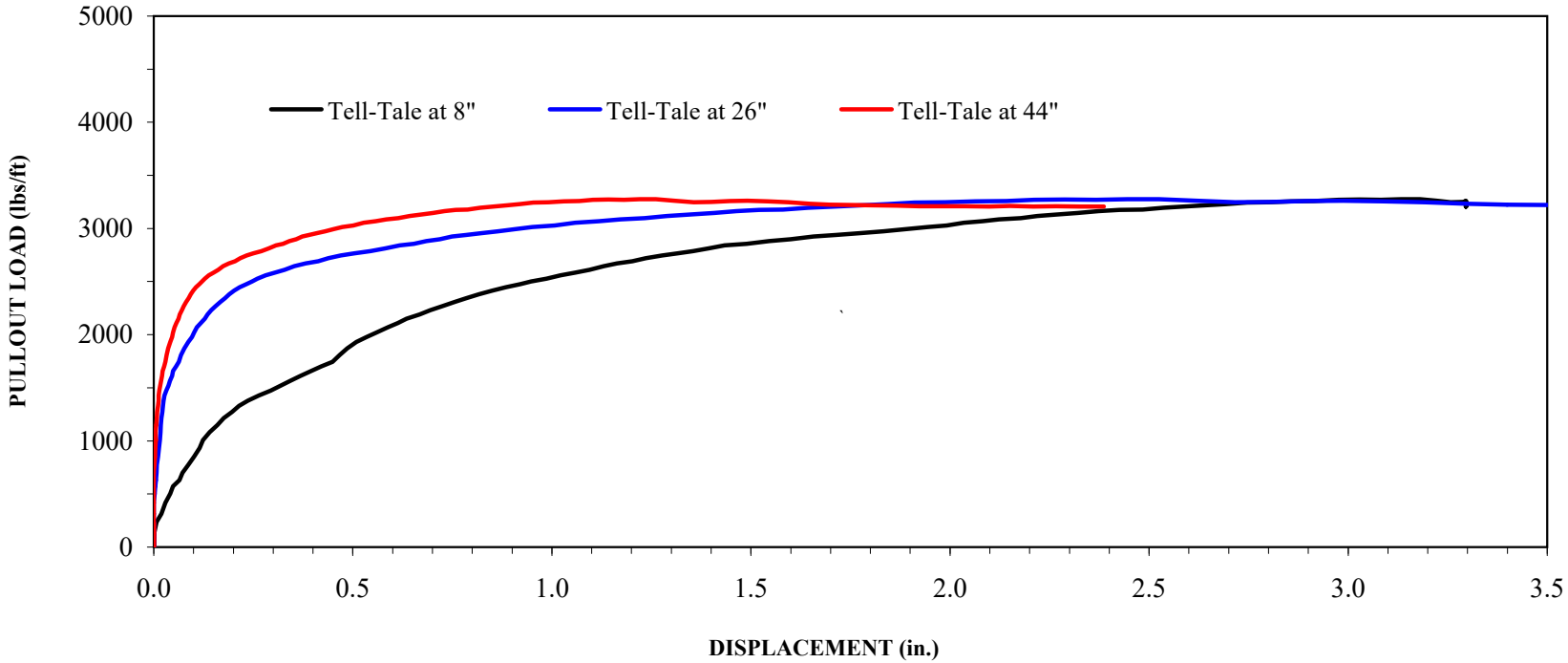
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
2A	15.0	44.0	200	8 26 44	22 72 122	86 322 472	7 27 39

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

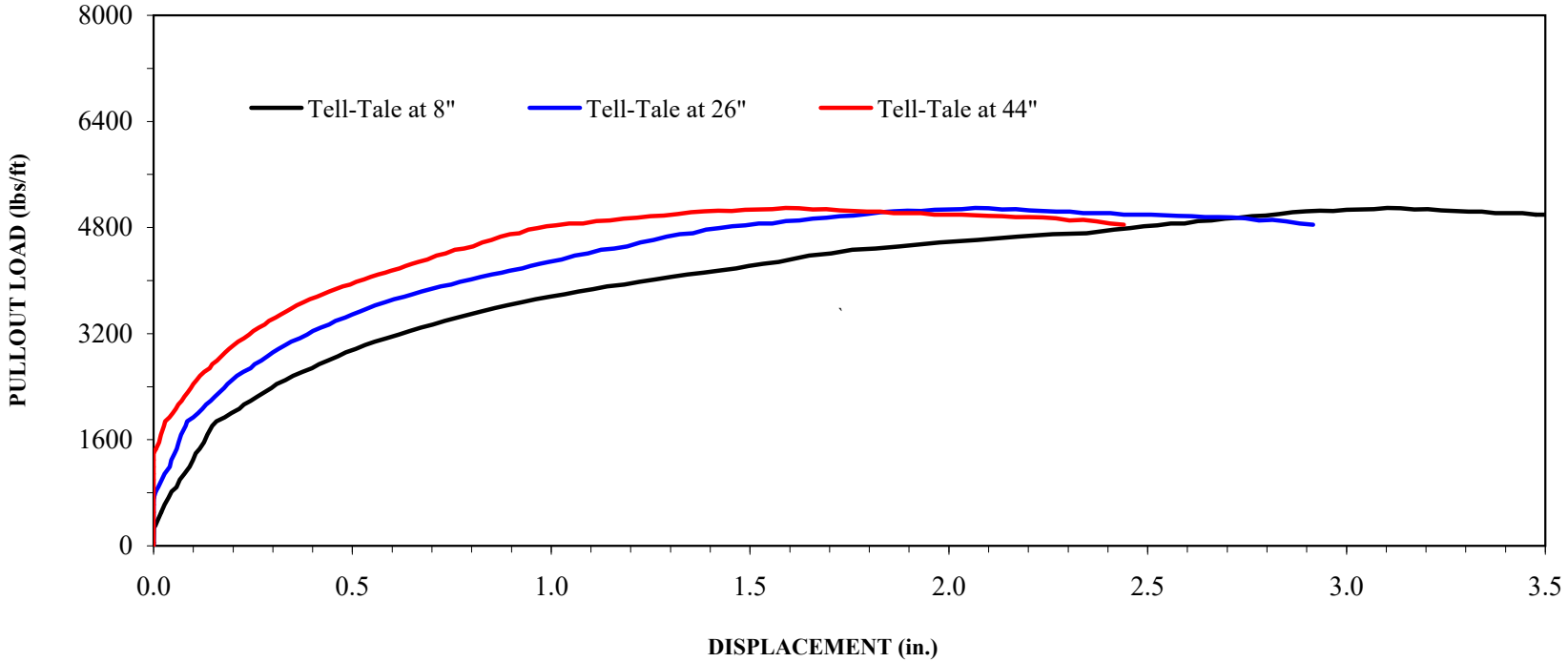
NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout 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

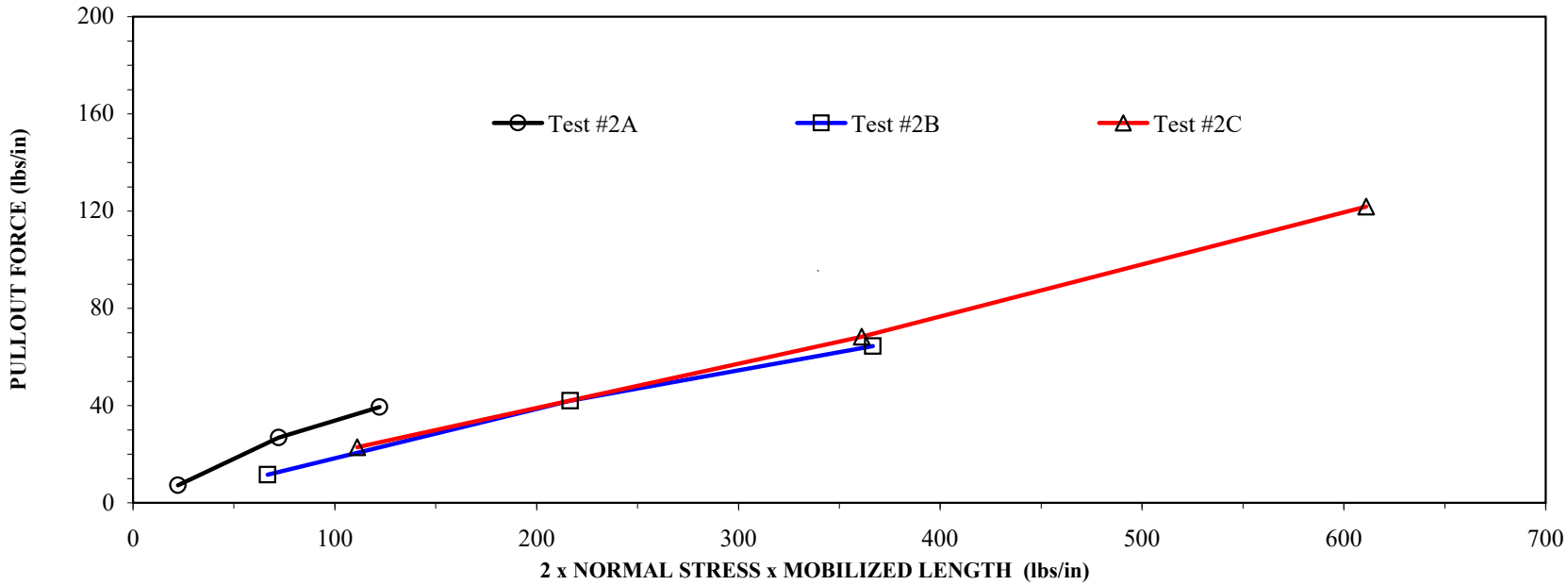
NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout 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 $\alpha^{(1)}$ (-)	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**

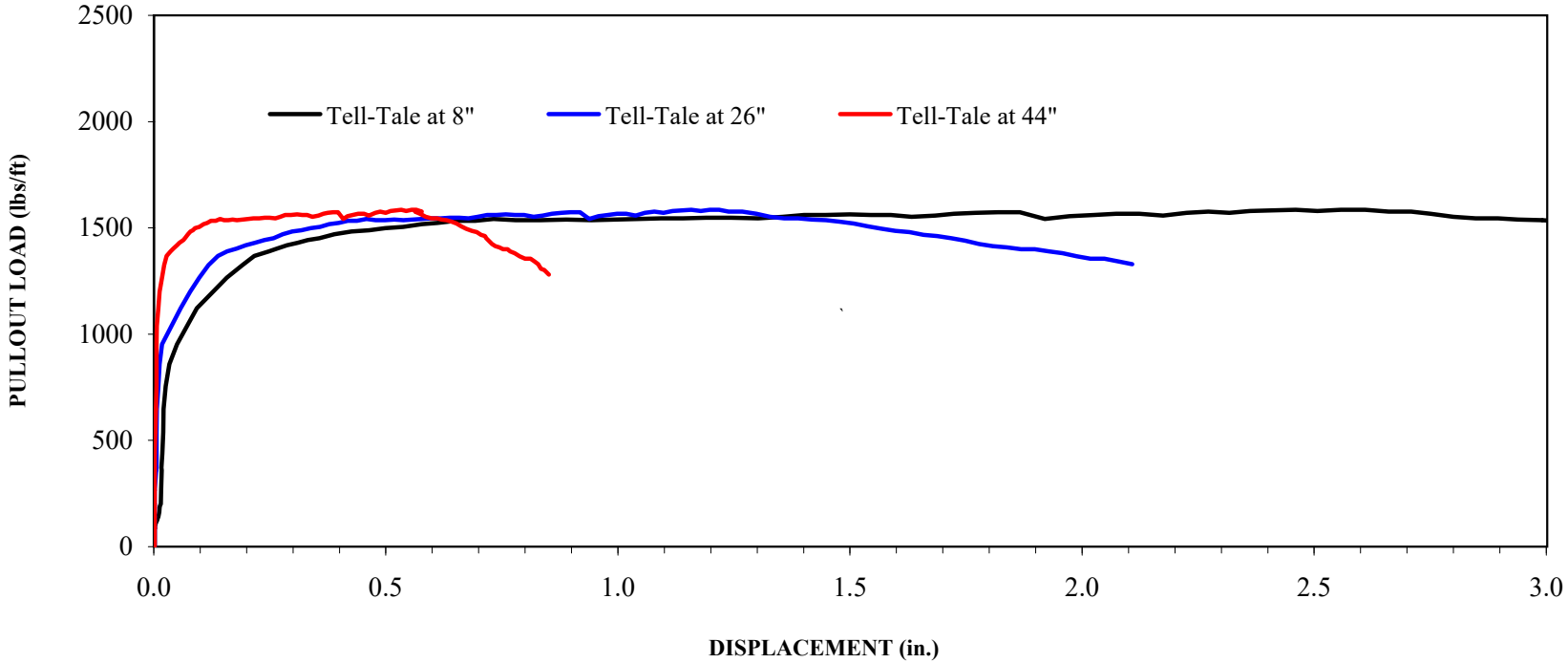
NOTE: (1) If the calculated α factor > 1 by the FHWA method (Appendix A FHWA-NHI-00-043), the reported α factor value is corrected to 1.0.

DATE REPORTED: 7/25/2022

SGI TESTING SERVICES, LLC

FIGURE NO.	8
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 3A at σ_n = 200 psf



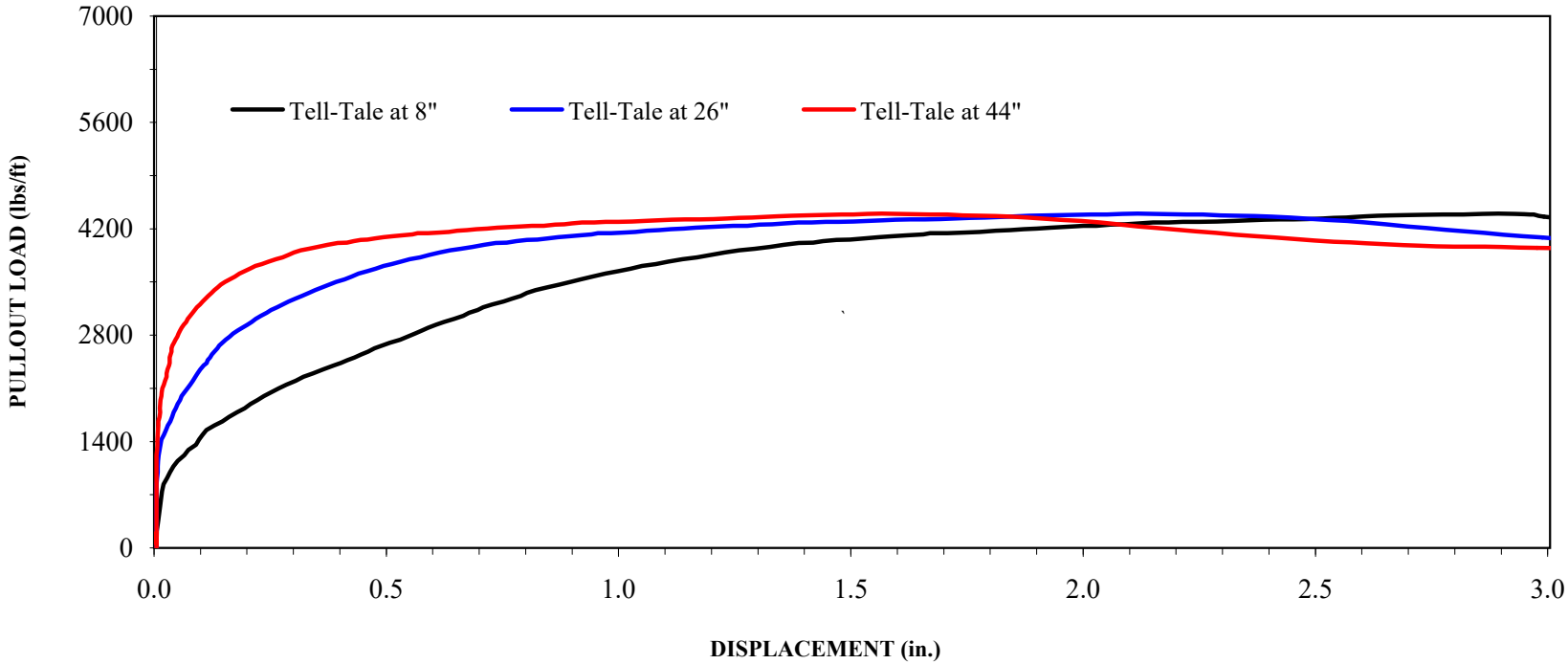
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
3A	15.0	44.0	200	8 26 44	22 72 122	101 266 538	8 22 45

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



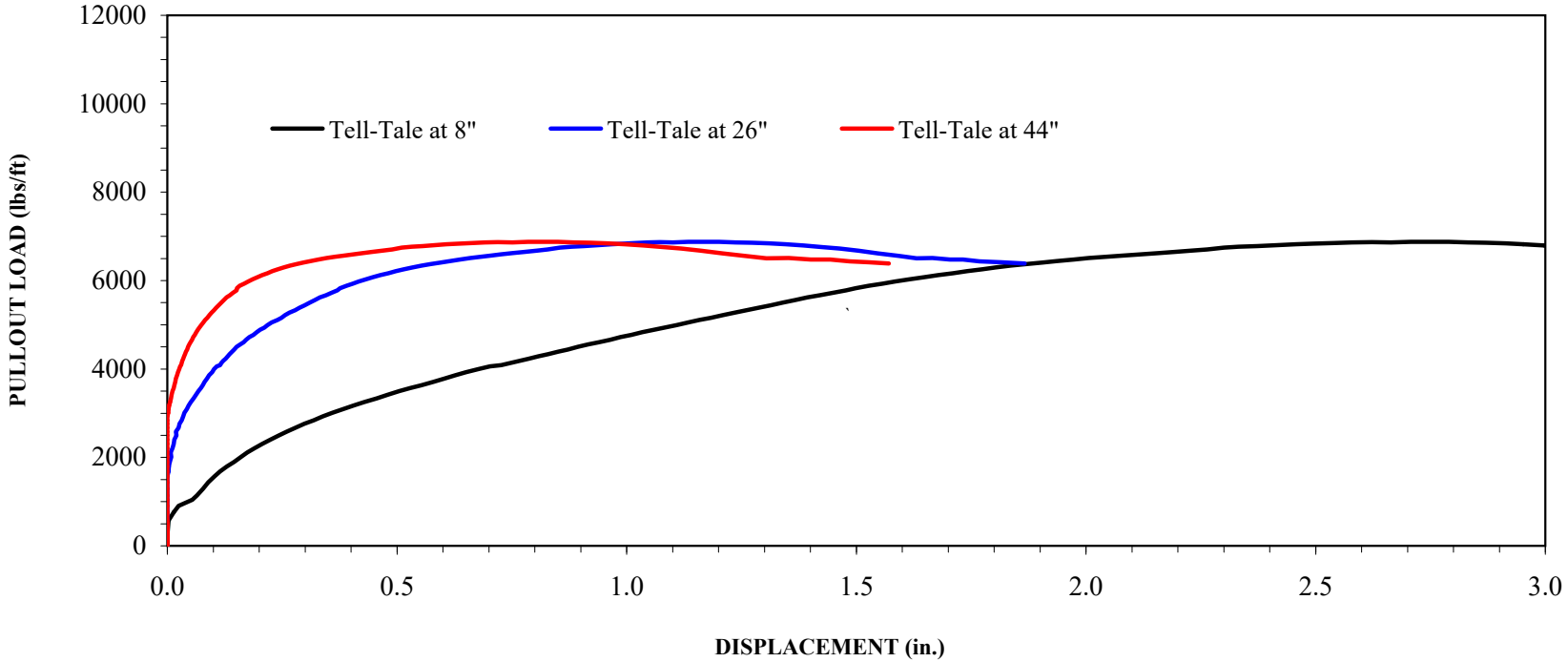
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[1] L_p (inch)	Total Normal Force[2] $2N = 2\sigma_n L_p$ (lbs/in)	Pullout Force[3] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
3B	15.0	44.0	800	8	89	226	19
				26	289	921	77
				44	489	1291	108

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	156	570	48
				26	506	1686	141
				44	856	3008	251

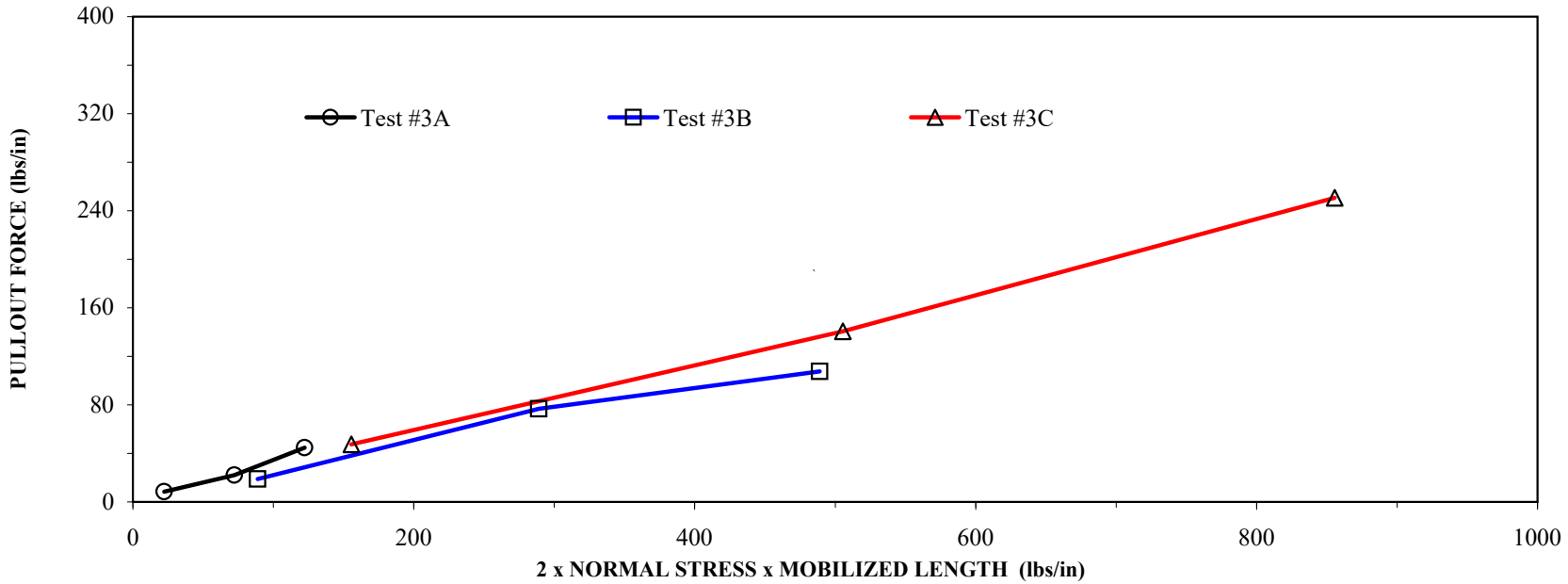
NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout 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	22	101	8	0.38		
				26	72	266	22	0.31		
				44	122	538	45	0.37		
									0.97	**0.97**
3B	15.0	44.0	800	8	89	226	19	0.21		
				26	289	921	77	0.27		
				44	489	1291	108	0.22		
									1.04	**1.00**
3C	15.0	44.0	1400	8	156	570	48	0.31		
				26	506	1686	141	0.28		
				44	856	3008	251	0.29		
									0.96	**0.96**

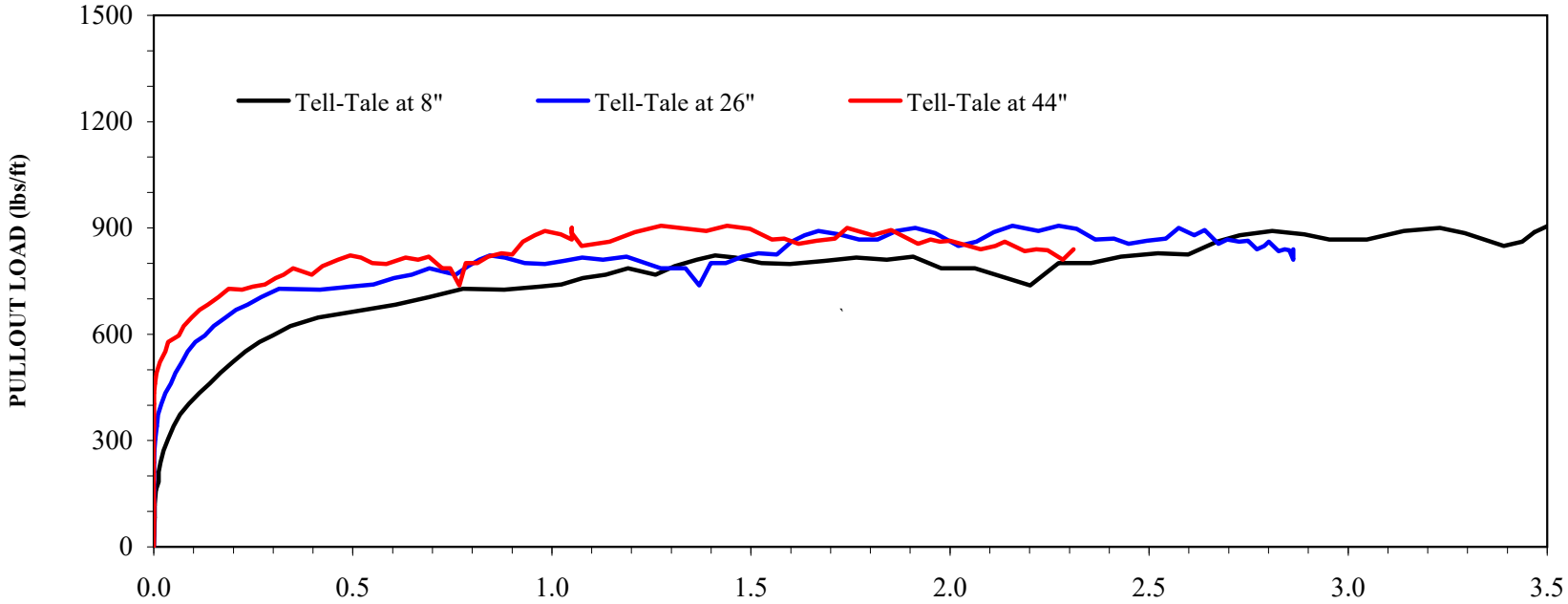
NOTE: (1) If the calculated α factor > 1 by the FHWA method (Appendix A FHWA-NHI-00-043), the reported α factor value is corrected to 1.0.

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



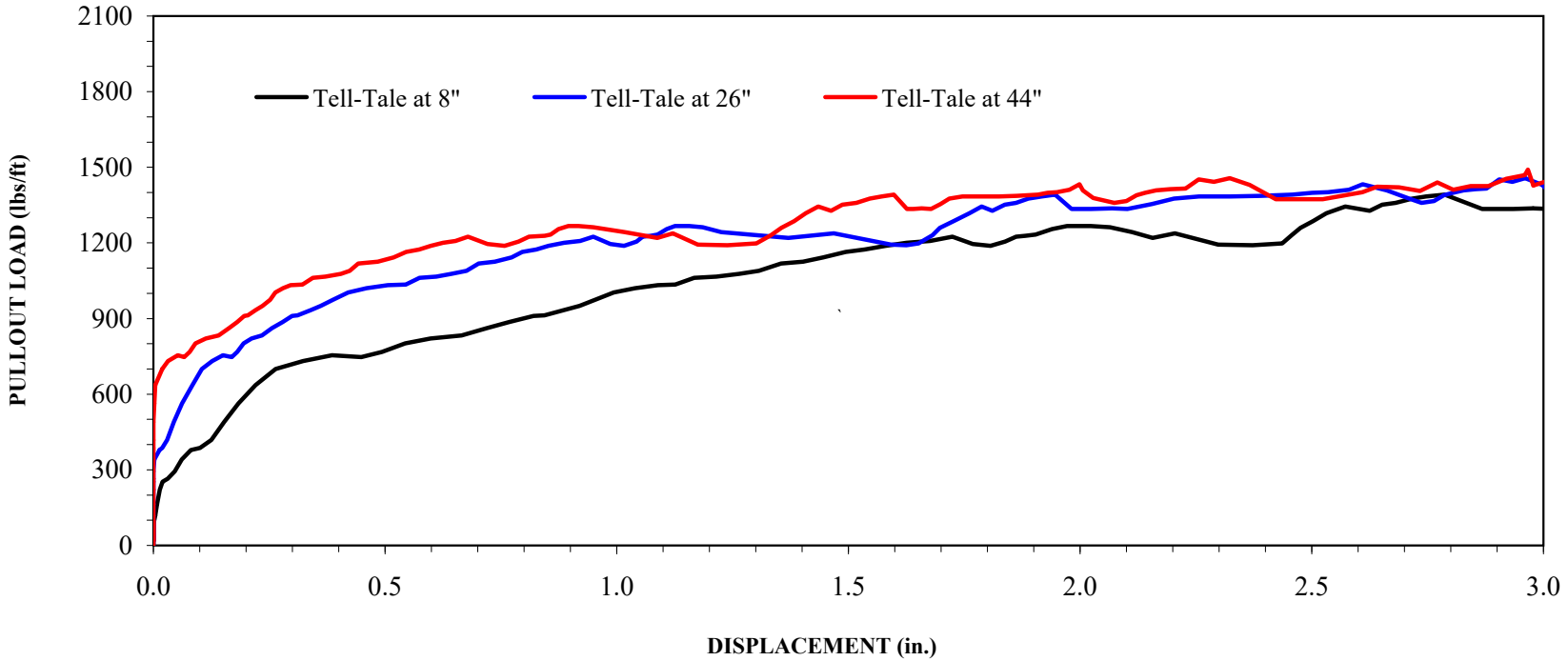
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
4A	15.0	44.0	100	8 26 44	11 36 61	84 271 461	7 23 38

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
FIGURE NO.	13
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

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	22	118	10
				26	72	378	32
				44	122	636	53

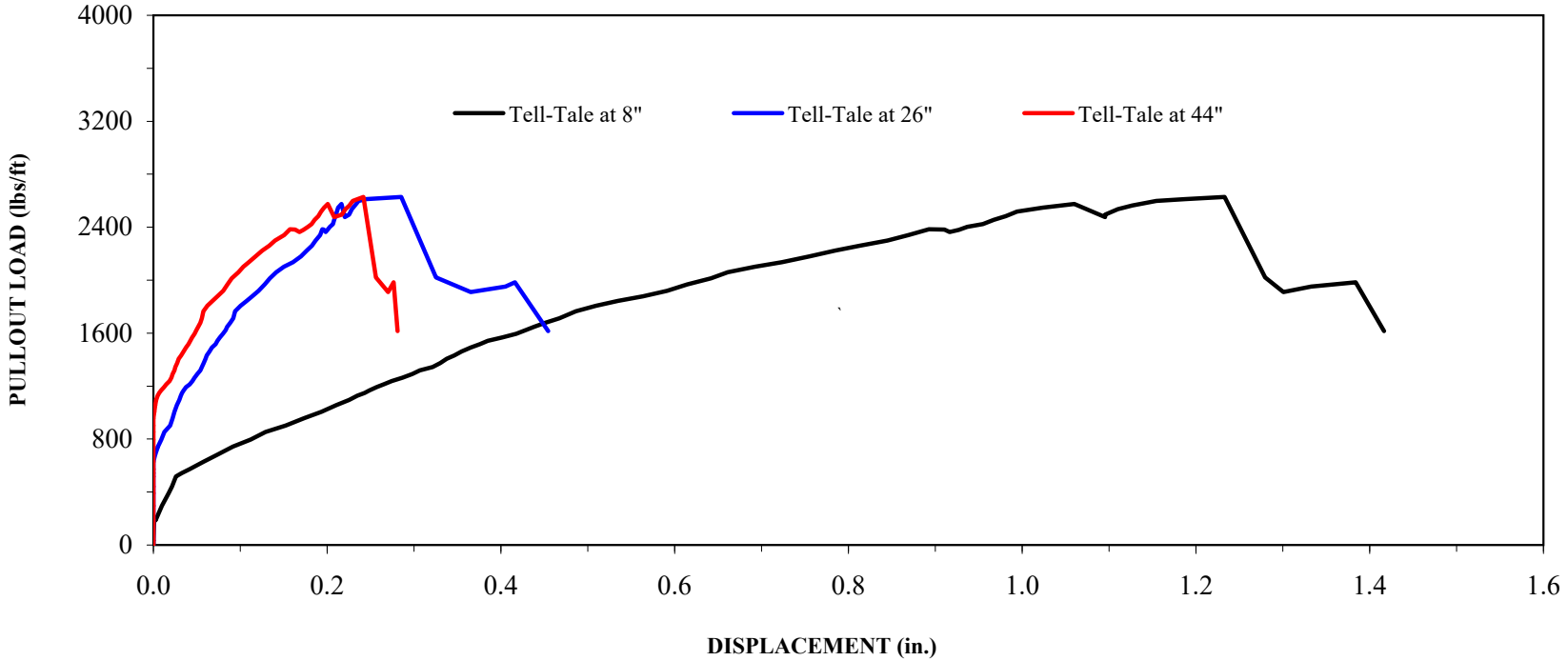
NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout 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

SGI TESTING SERVICES, LLC

FIGURE NO.	14
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

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 26 44	44 144 244	190 682 1005	16 57 84

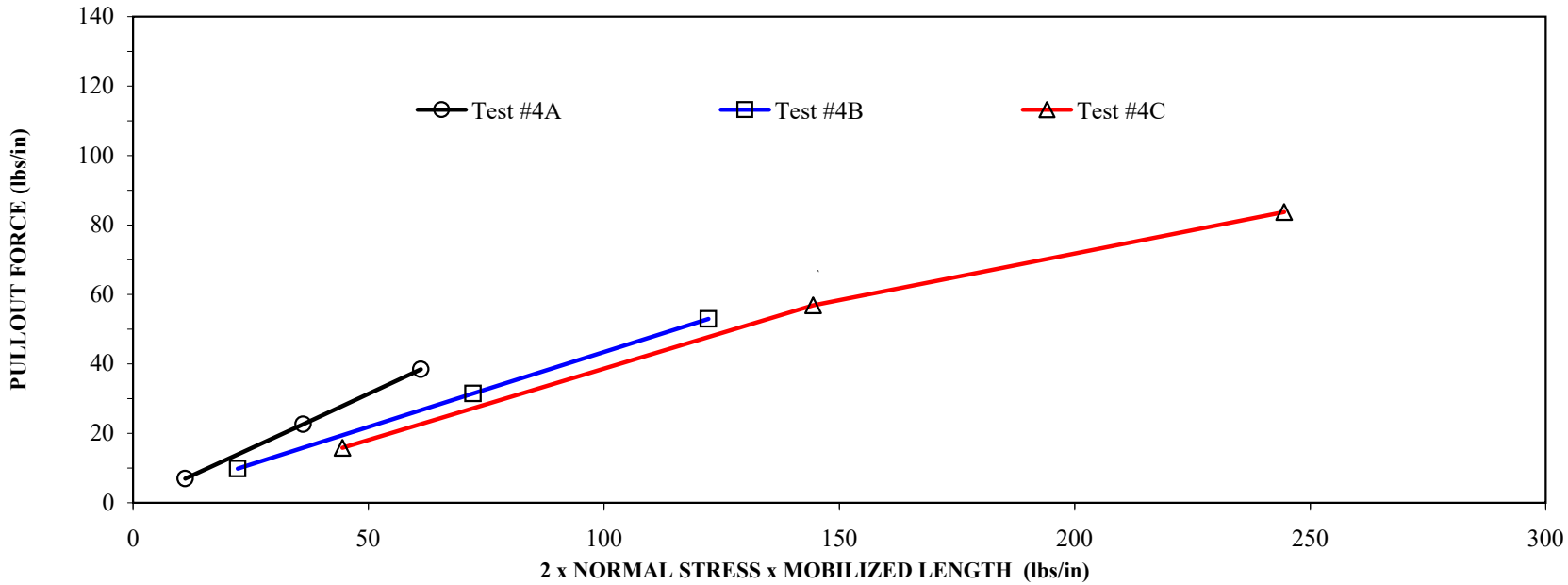
NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout 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
FIGURE NO.	15
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 $\alpha^{(1)}$
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		

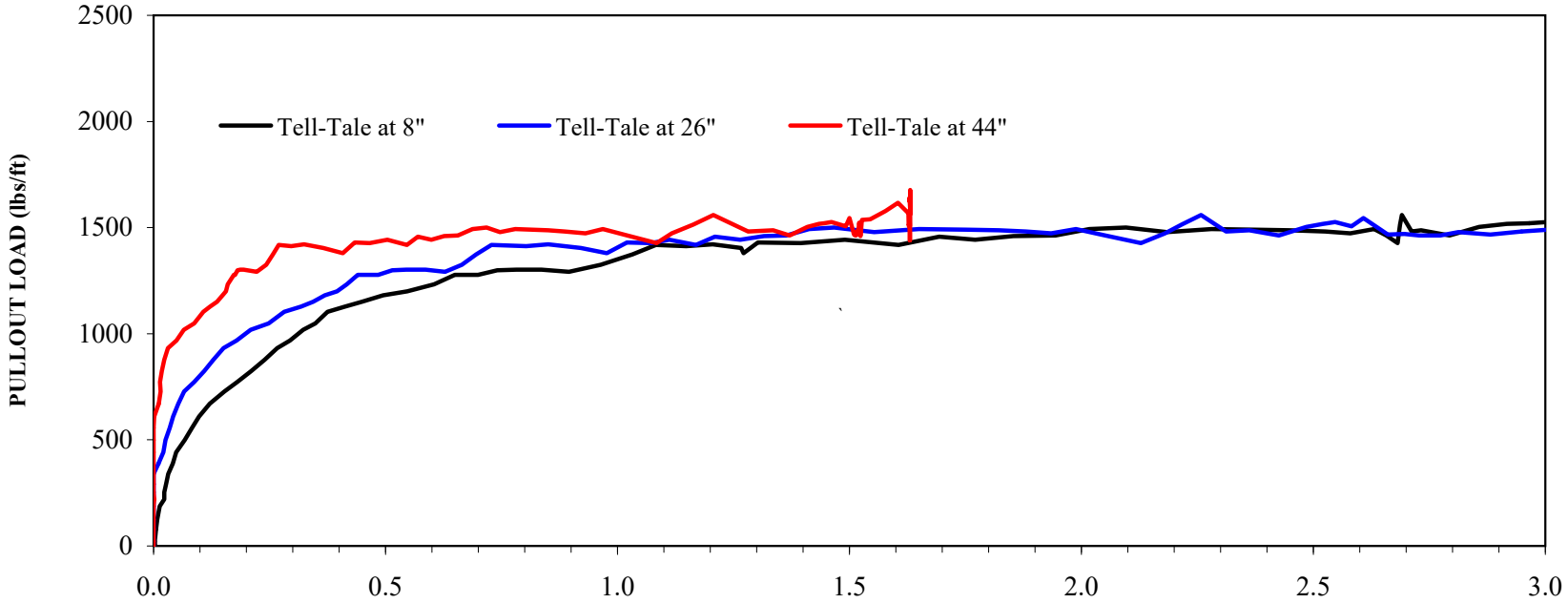
NOTE: (1) If the calculated α factor > 1 by the FHWA method (Appendix A FHWA-NHI-00-043), the reported α factor value is corrected to 1.0.

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



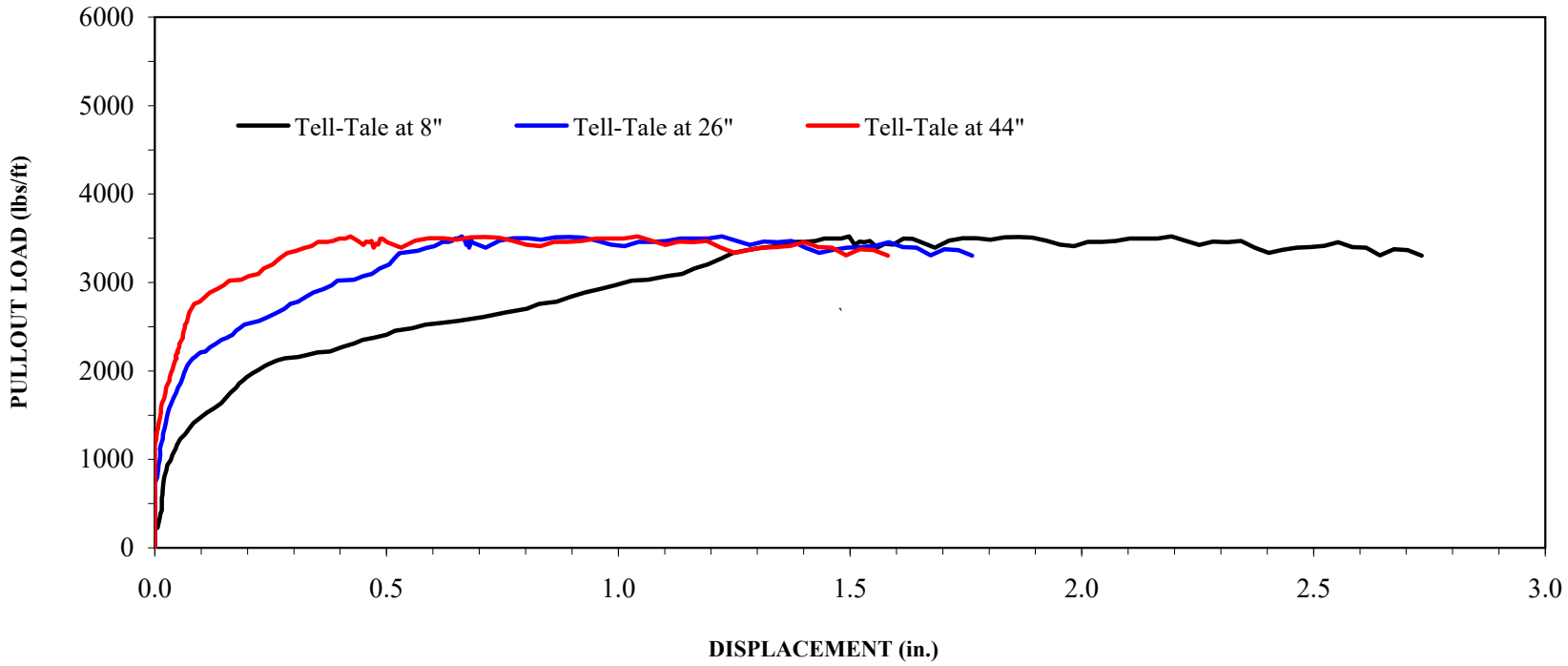
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
5A	15.0	44.0	200	8	22	125	10
				26	72	388	32
				44	122	670	56

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
FIGURE NO.	17
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 5B at σ_n = 500 psf



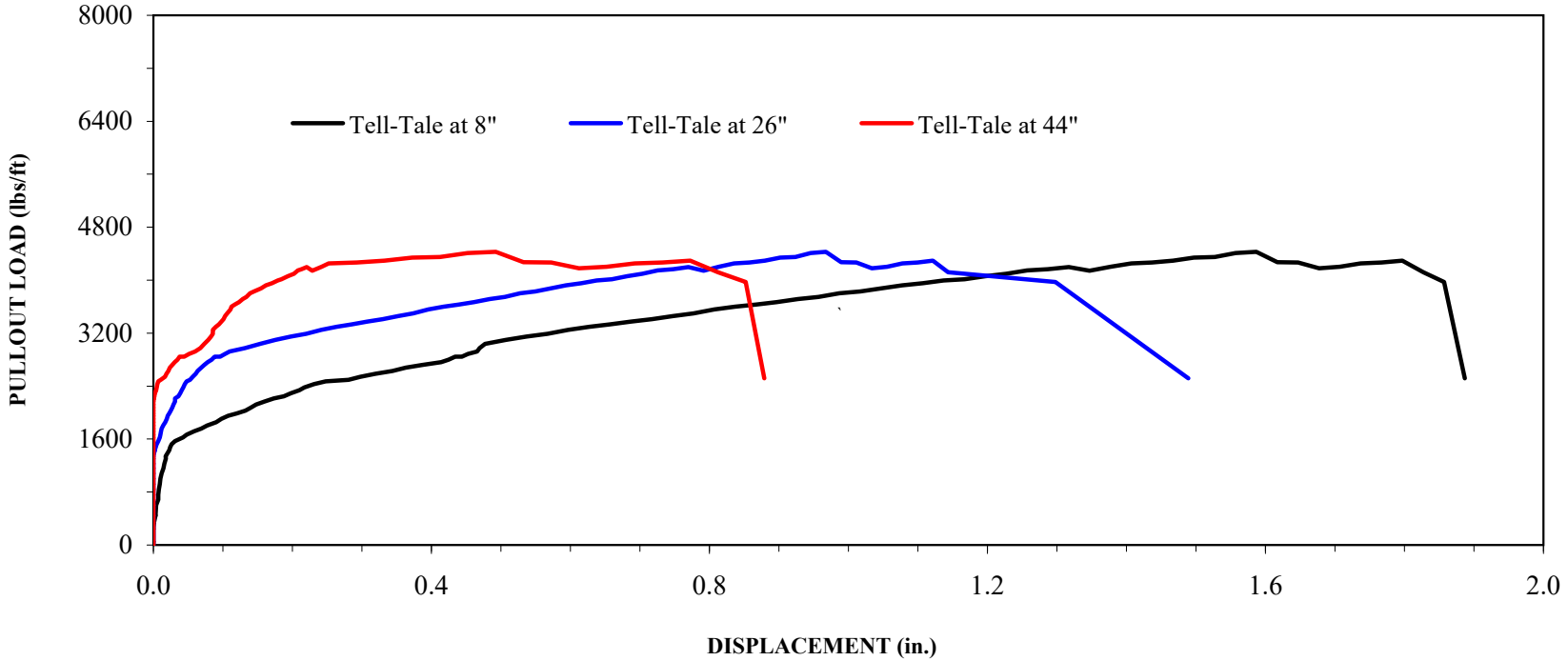
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
5B	15.0	44.0	500	8 26 44	56 181 306	232 801 1229	19 67 102

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



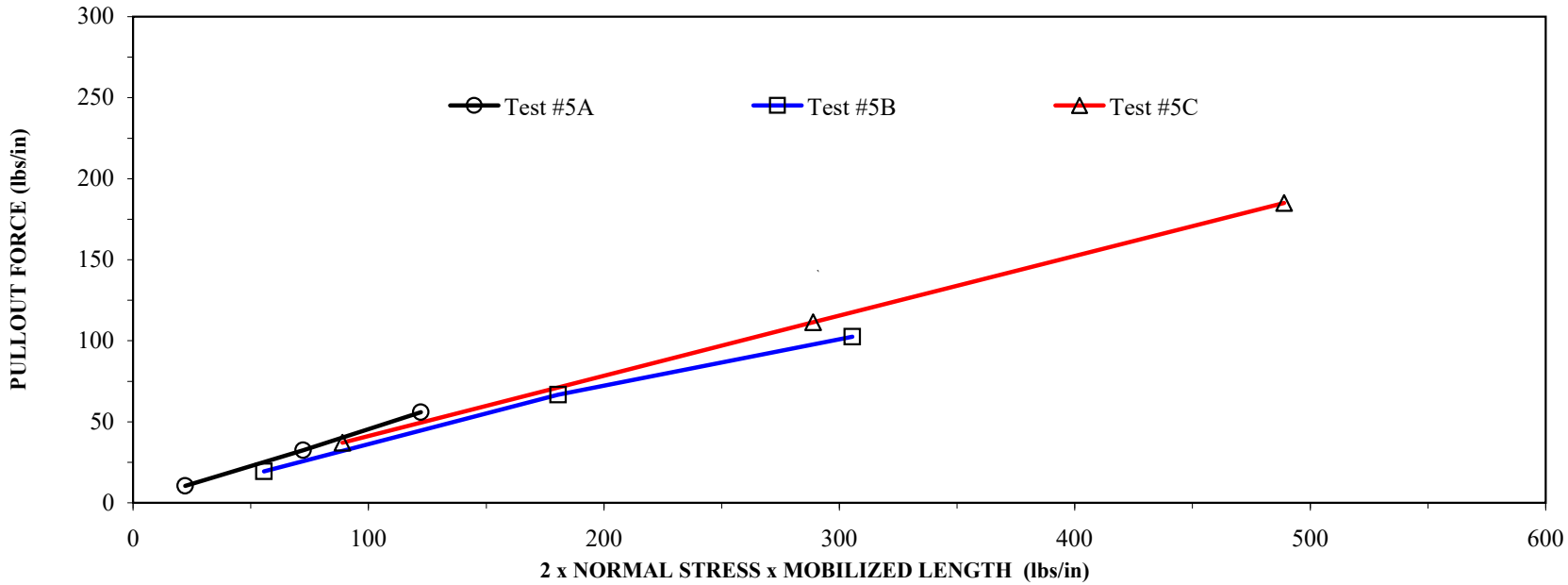
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
5C	15.0	44.0	800	8	89	446	37
				26	289	1338	112
				44	489	2221	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

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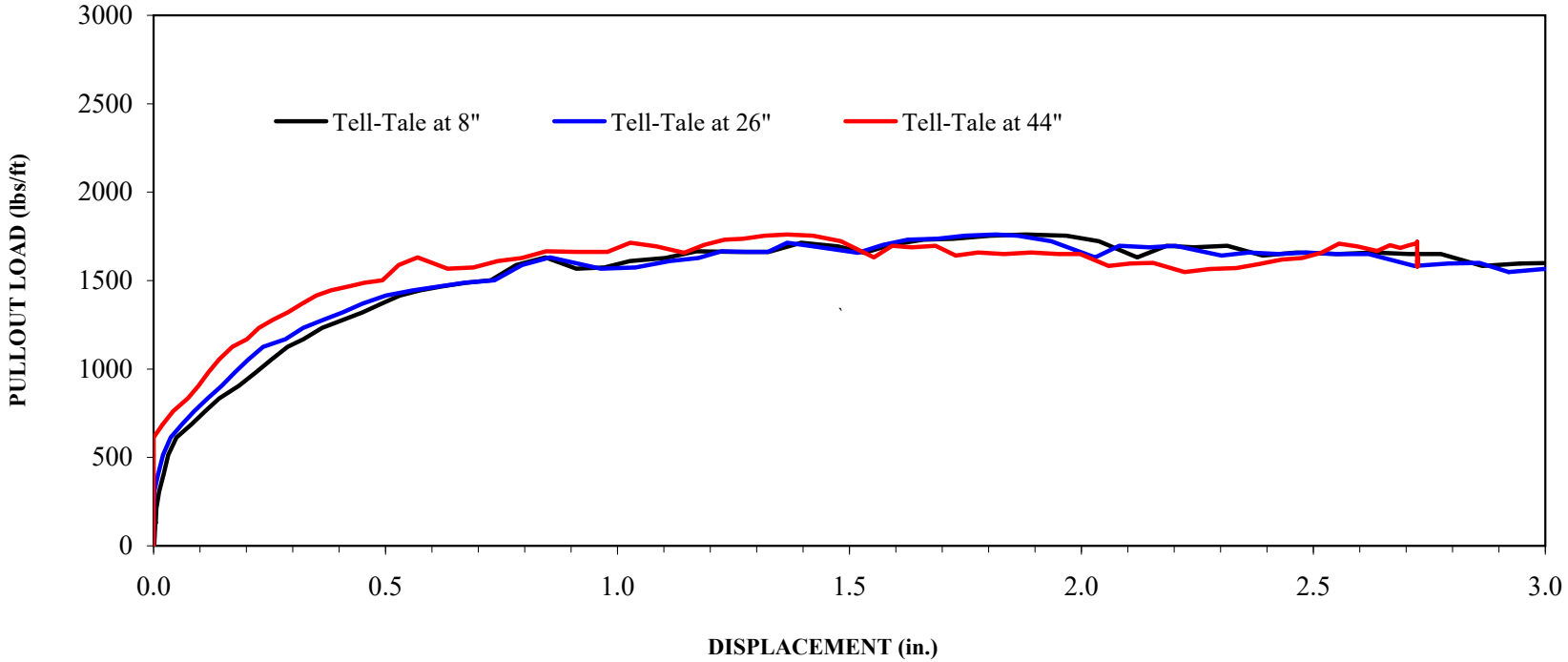
FIGURE NO.	20
PROJECT NO.	SGI22016
DOCUMENT NO.	
FILE NO.	

TENSAR INTERNATIONAL CORPORATION
PULLOUT TESTING (ASTM D 6706)
Test No.: 6A at σ_n = 200 psf



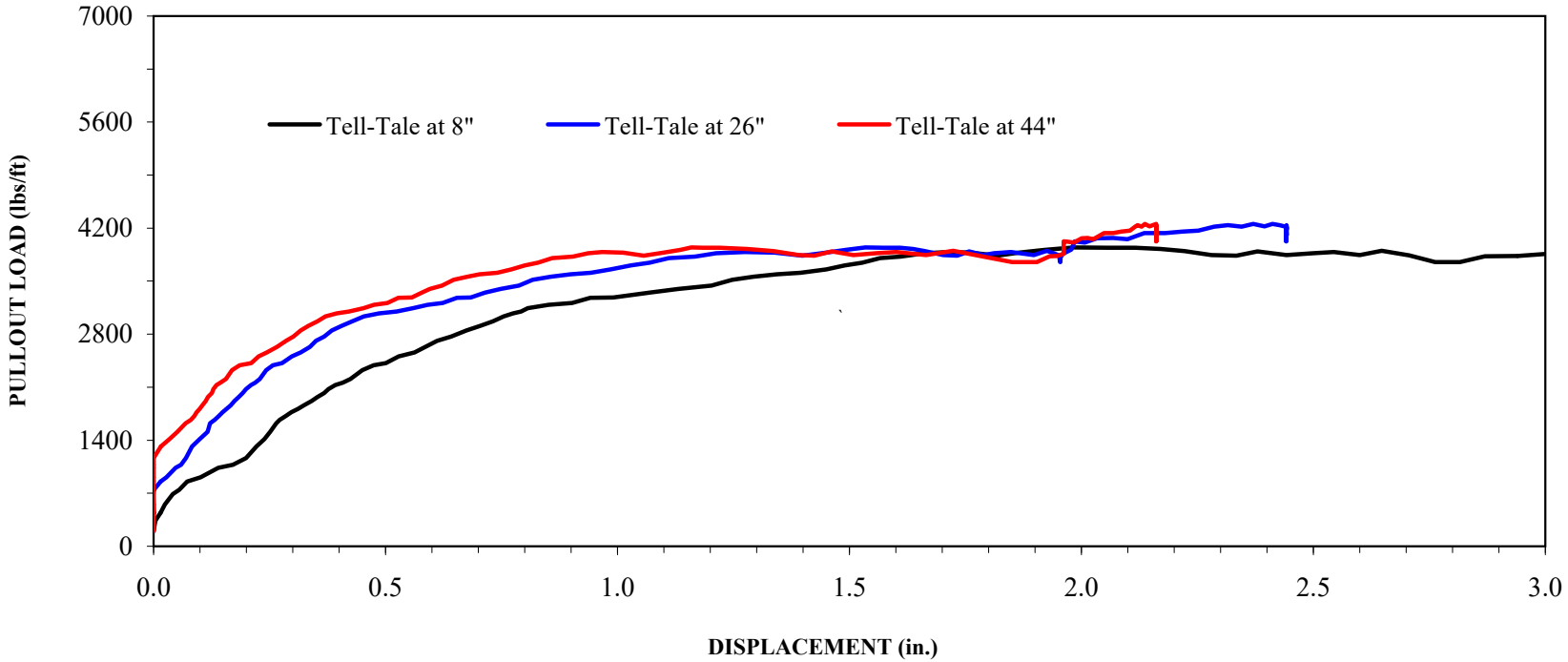
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
6A	15.0	44.0	200	8 26 44	22 72 122	130 405 686	11 34 57

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



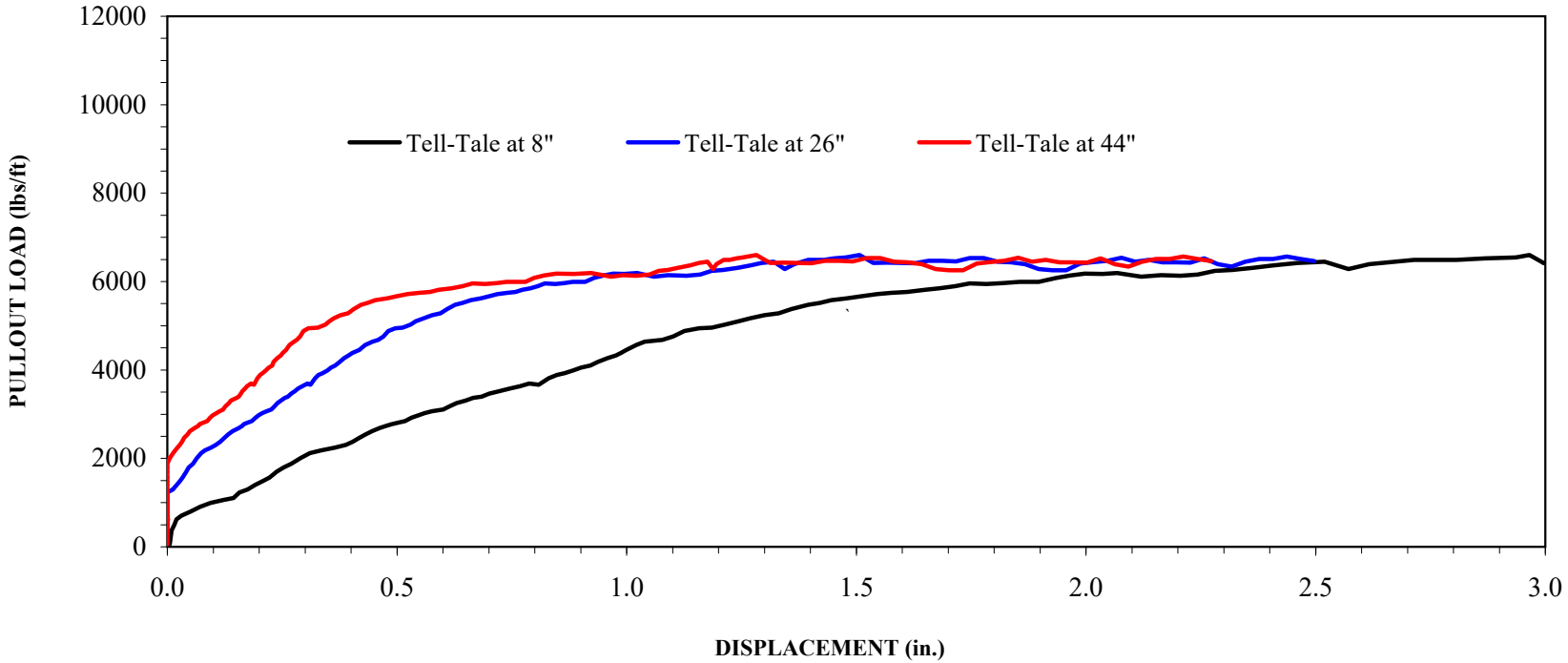
Test No.	Test Specimen Width W (in.)	Embedment Length s (in.)	Normal Stress σ_n (psf)	Mobilized Length[(1)] L_p (inch)	Total Normal Force[(2)] $2\sigma_n L_p$ (lbs/in)	Pullout Force[(3)] P (lbs/ft)	Pullout Resistance $P_r = P/W$ (lbs/in)
6B	15.0	44.0	600	8 26 44	67 217 367	251 855 1317	21 71 110

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

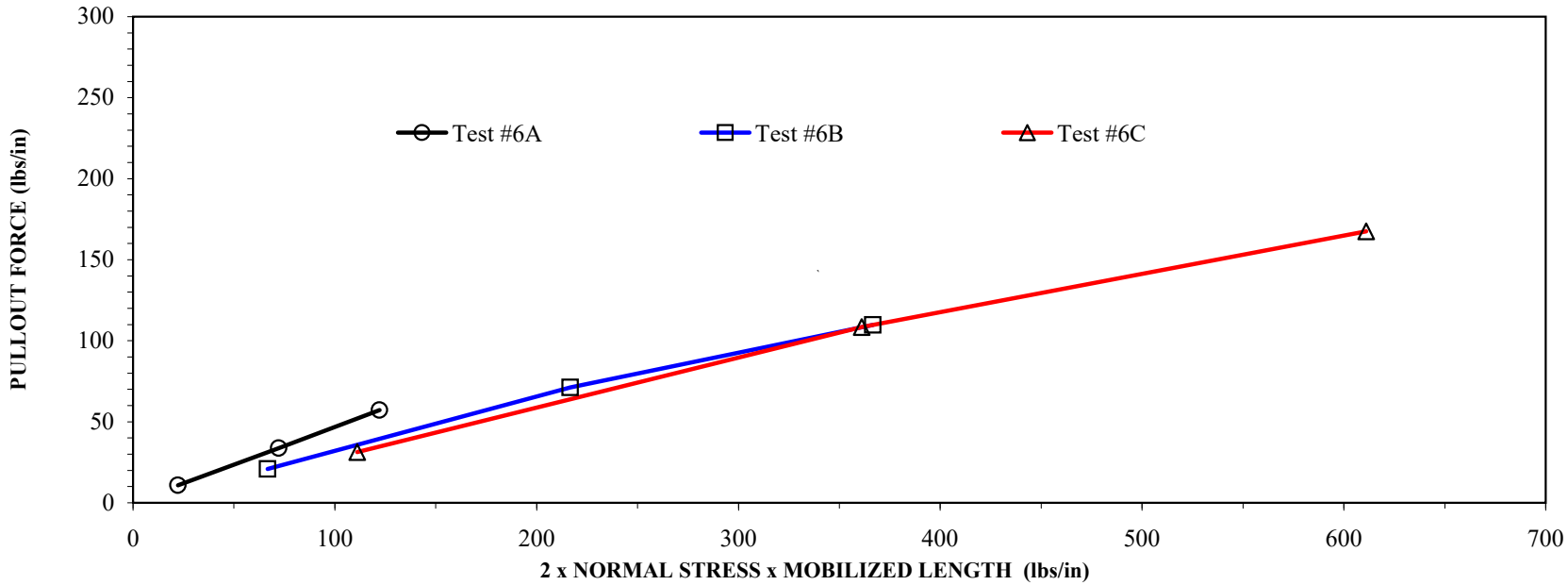
NOTES:
(1) The mobilized length is defined as the section length of reinforcement along which displacements > 0 at all the points except the rear point of the section where displacement = 0.
(2) Normal load acting on top and bottom of the reinforcement per unit width of the reinforcement.
(3) Pullout 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

ARES® 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.

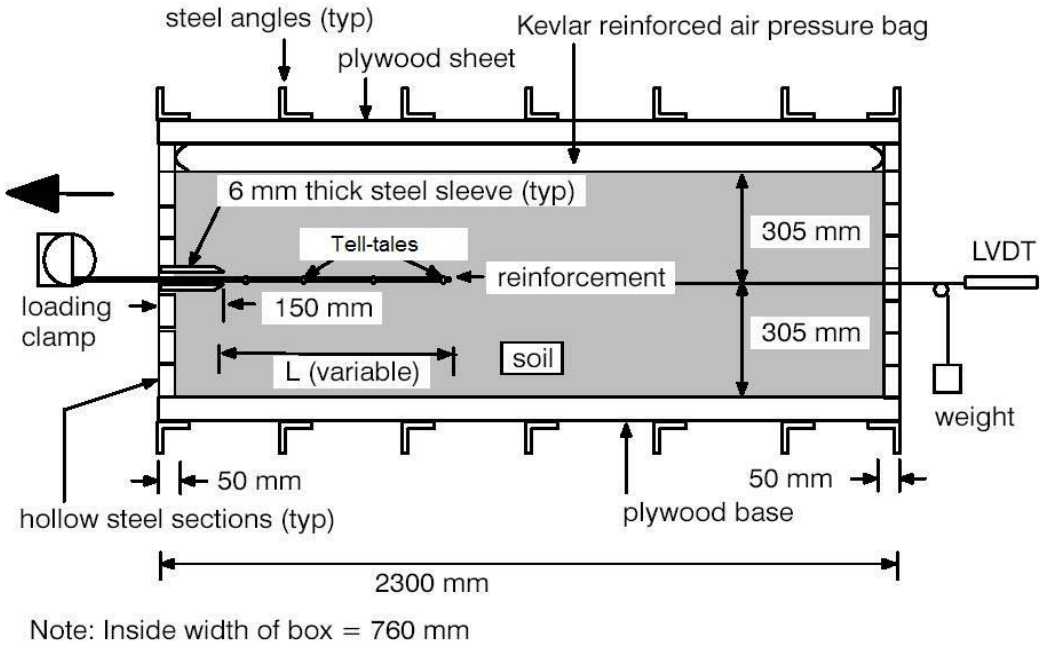


Figure 2: Cross-section view of in-soil pullout box apparatus and general arrangement of soil reinforcement.

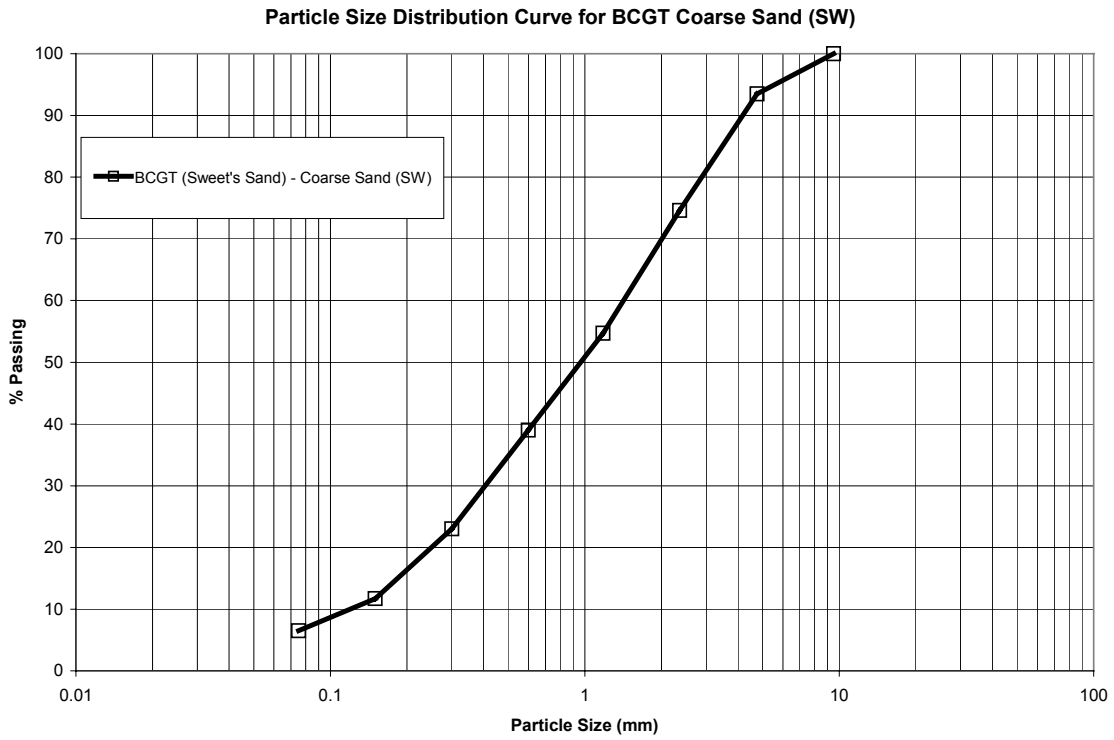


Figure 3: Particle size distribution for coarse sand (SW).

Figure 4: Photograph of a 900 mm-long Tensar UX1100MSE geogrid specimen showing the tell-tale locations.

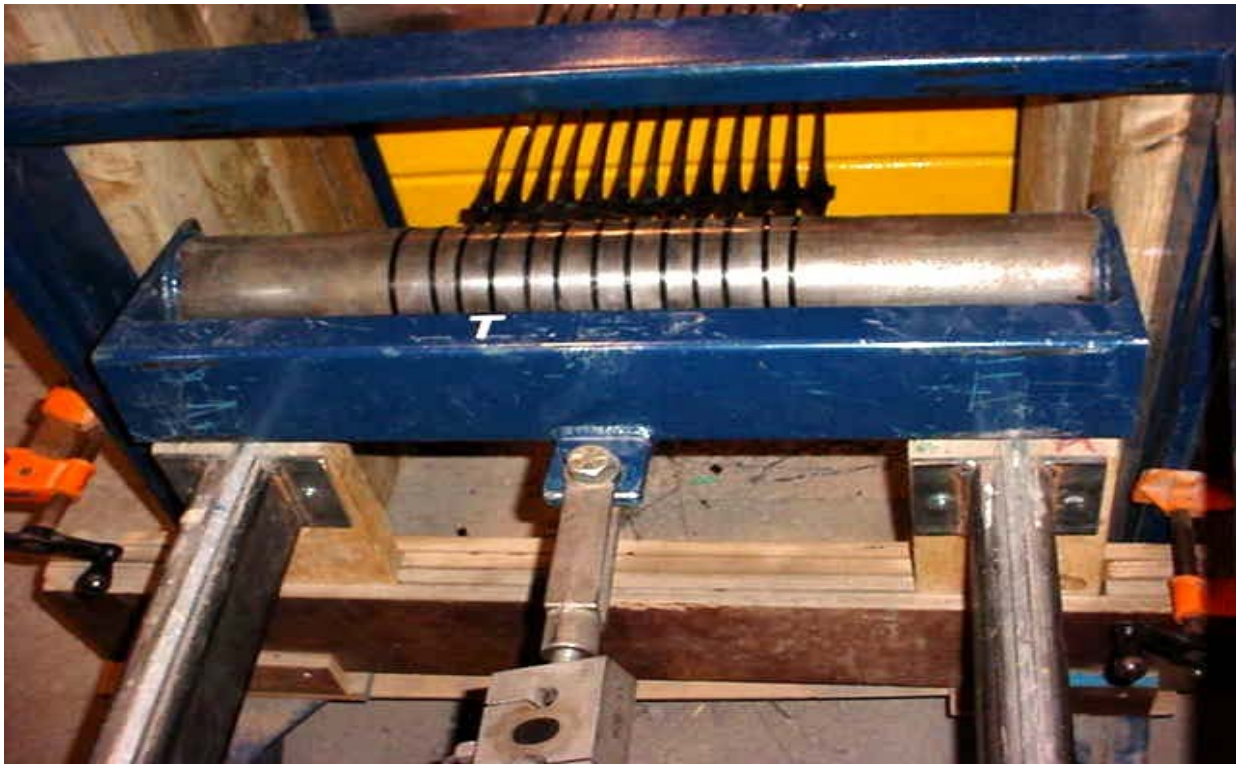


Figure 5: View of clamping arrangement at front of pullout box.

Figure 6: Pullout Capacity vs Confining Pressure for Tensar UX1100MSE Geogrid with Coarse Sand (SW)

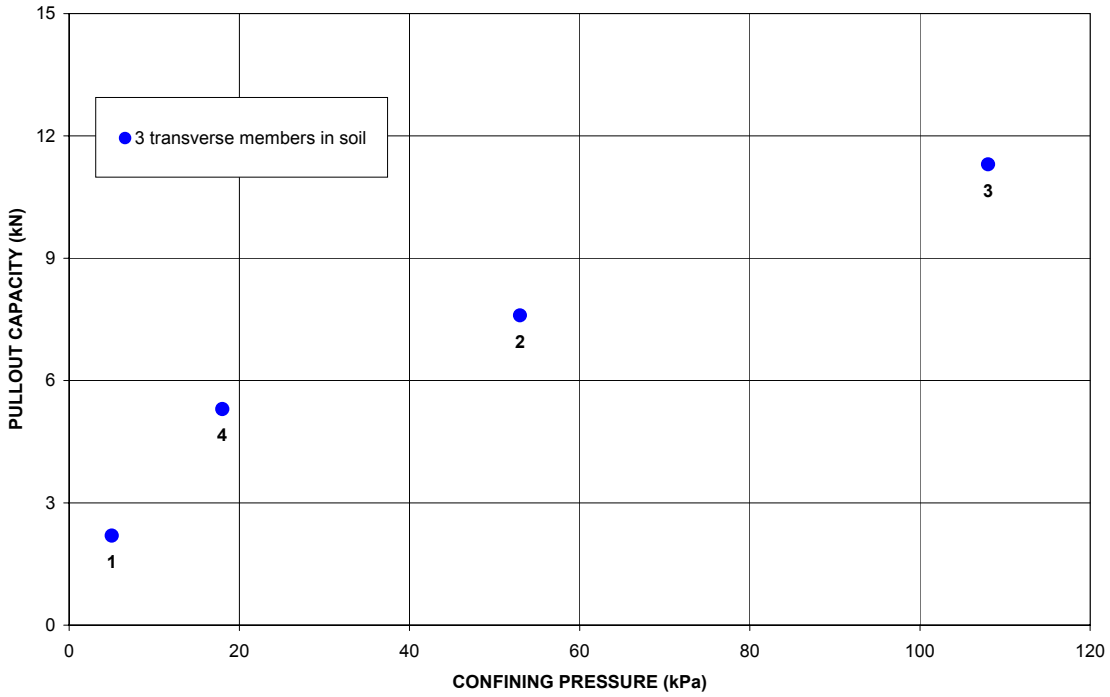


Figure 7: Exhumed Tensar UX1100MSE geogrid specimen (900 mm long) from Test 1, after testing under a confining pressure of 5 kPa.

Figure 8: Exhumed Tensar UX1100MSE geogrid specimen (900 mm long) from Test 2, after testing under a confining pressure of 53 kPa.

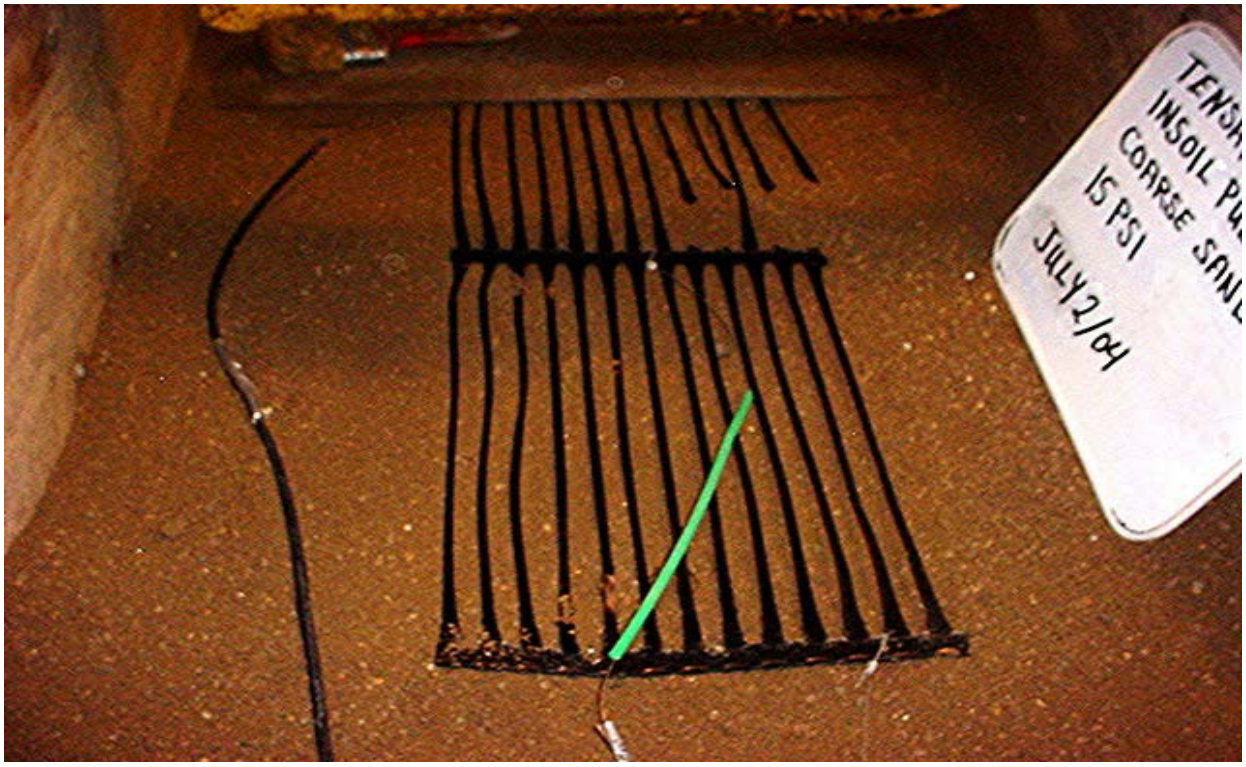


Figure 9: Exhumed Tensar UX1100MSE geogrid specimen (900 mm long) from Test 3, after testing under a confining pressure of 108 kPa.

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)

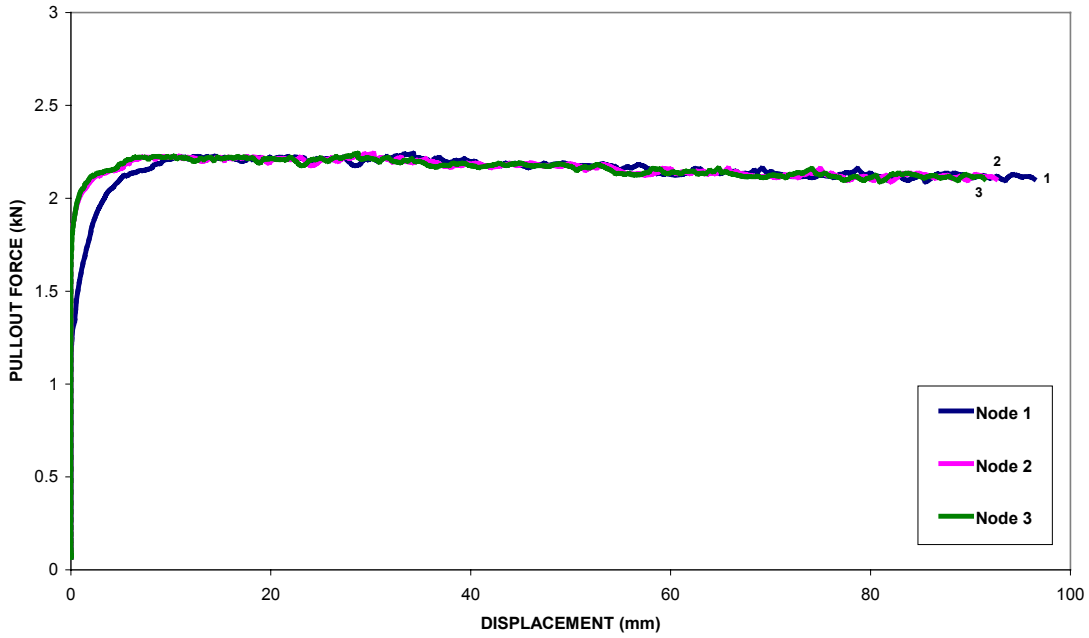


Figure A2: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with Coarse Sand (SW) Test 2 (53 kPa Confining Pressure)

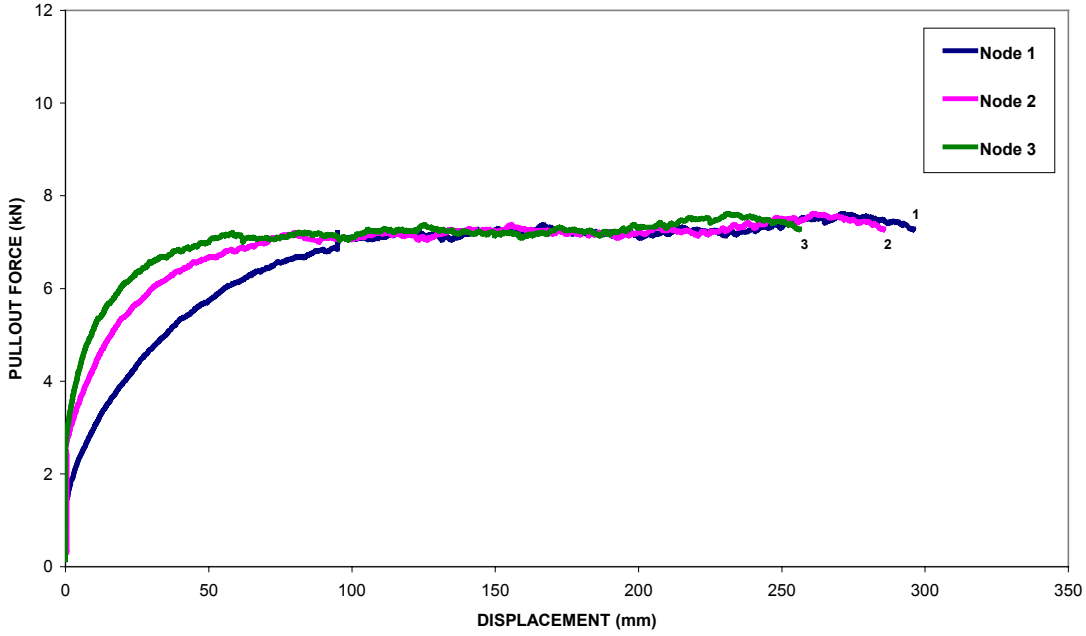


Figure A3: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with Coarse Sand (SW) Test 3 (108 kPa Confining Pressure)

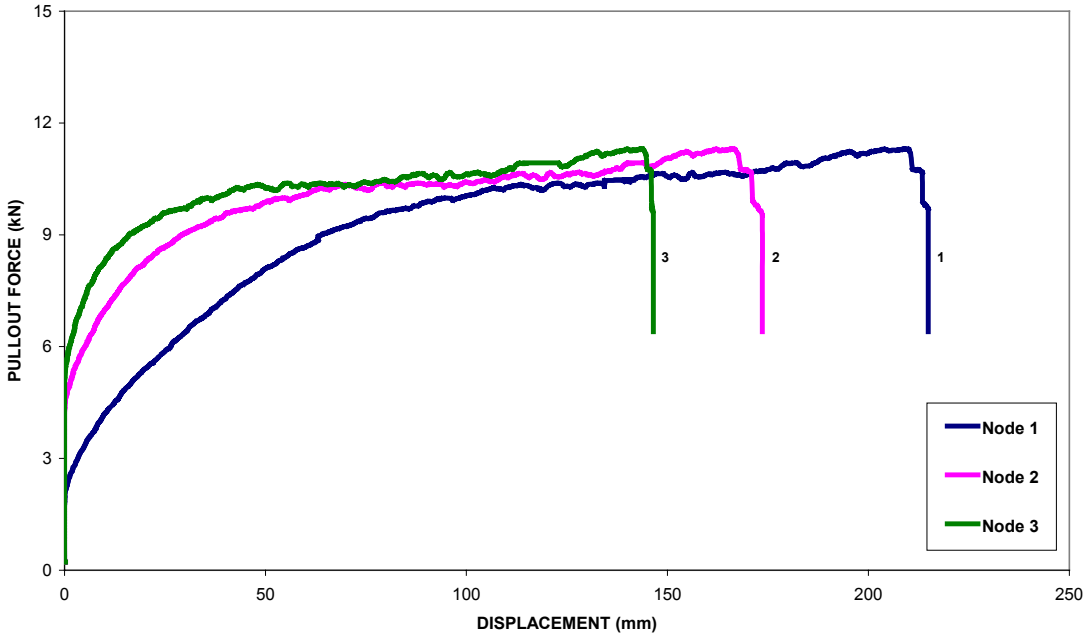
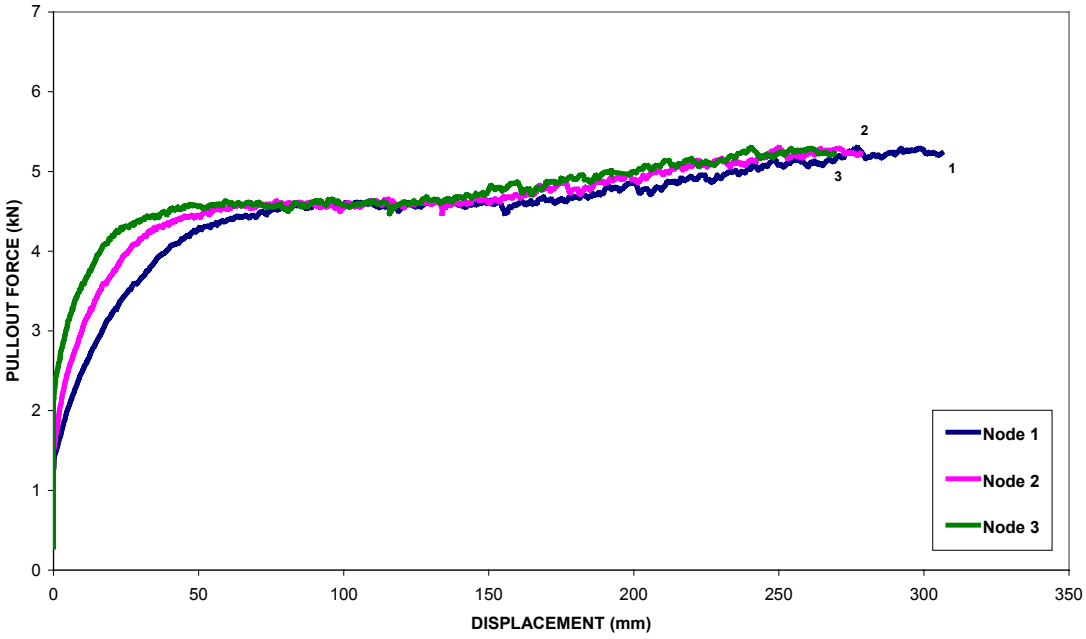


Figure A4: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with Coarse Sand (SW) Test 4 (18 kPa Confining Pressure)



Appendix 1.2.7.b

Pullout Tests in Soil Report

Tensar UX1100MSE Geogrids / Crushed Stone

ARES® 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.

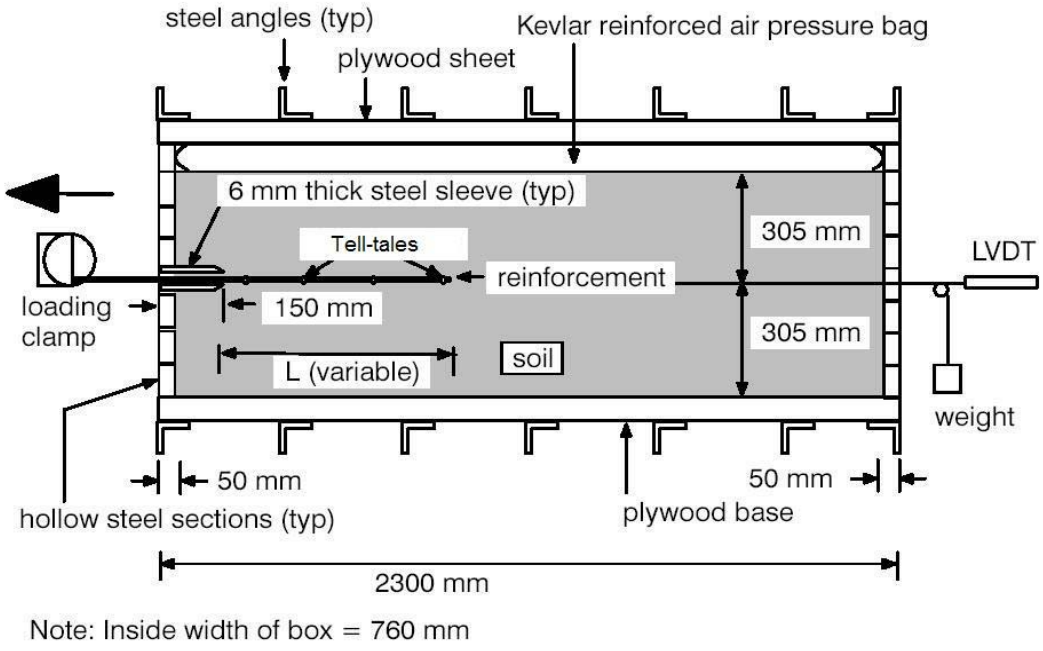


Figure 2: Cross-section view of in-soil pullout box apparatus and general arrangement of soil reinforcement.

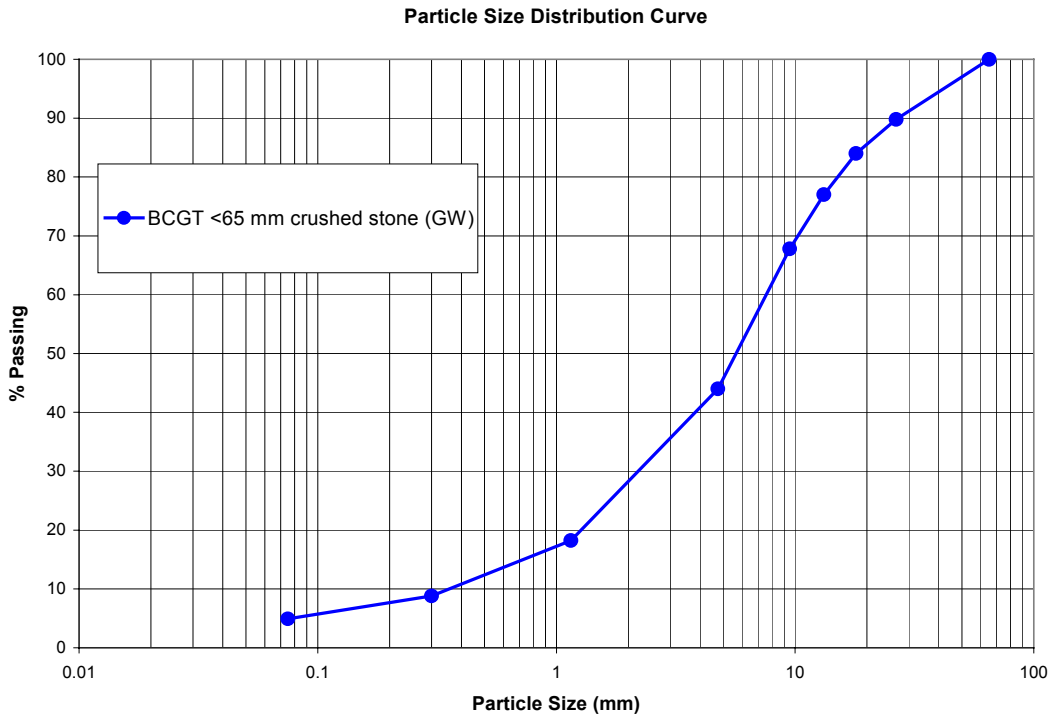


Figure 3: Particle size distribution for <65 mm crushed stone (GW).

Figure 4: Photographs of 900 mm-long (Left) and 1400 mm-long (Right) Tensar UX1100MSE geogrid specimens used in this investigation.

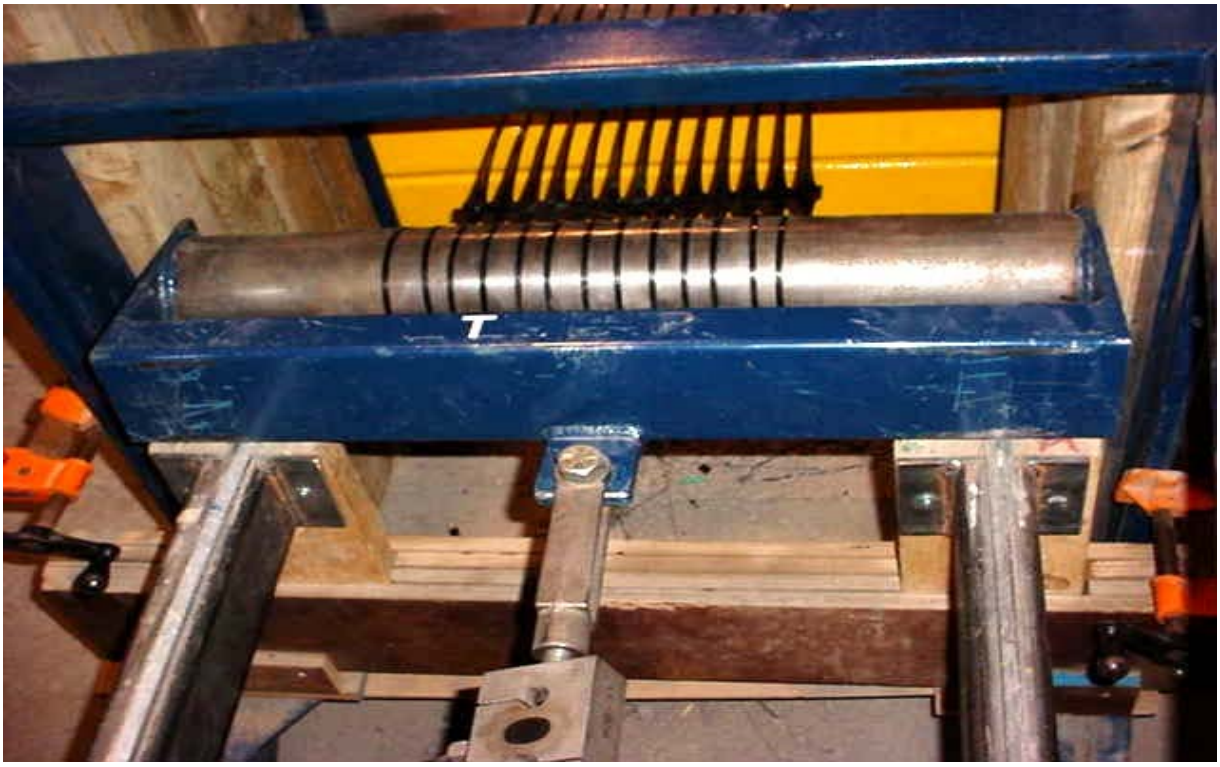


Figure 5: View of clamping arrangement at front of pullout box.

Figure 6: Pullout Capacity vs Confining Pressure for Tensar UX1100 Quint Geogrid with <65 mm Crushed Stone (GW)

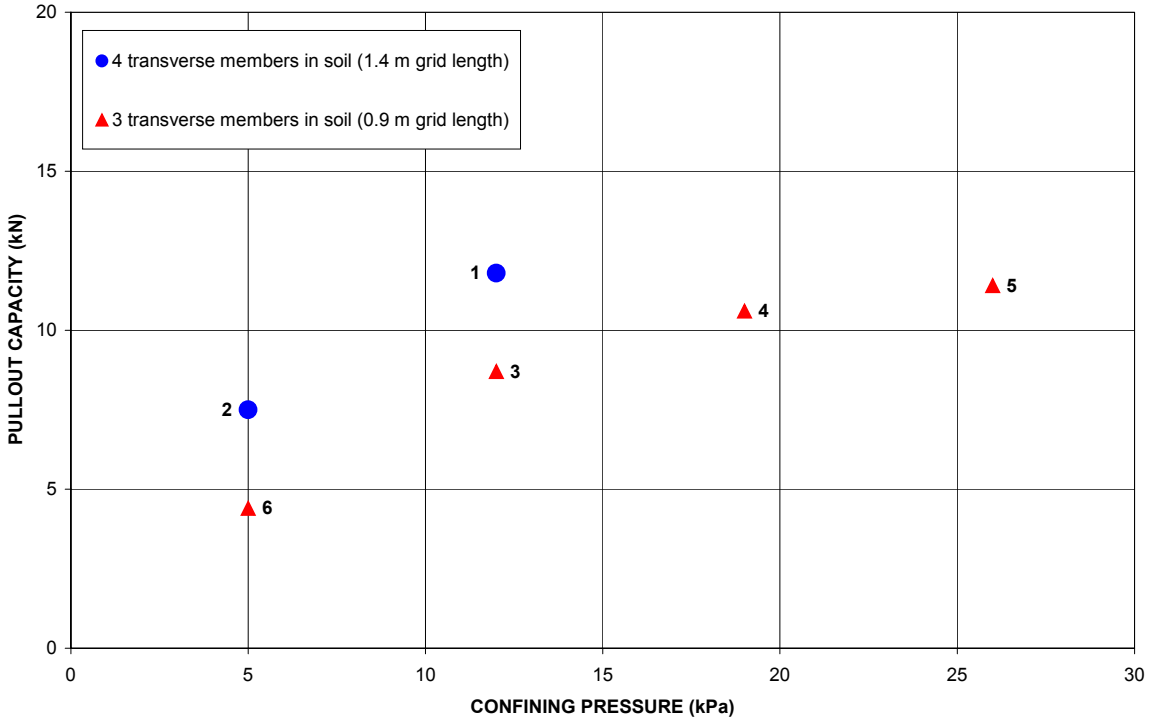


Figure 7: Exhumed Tensar UX1100MSE geogrid specimen (1400 mm long) from Test 1, after testing under a confining pressure of 12 kPa.

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)

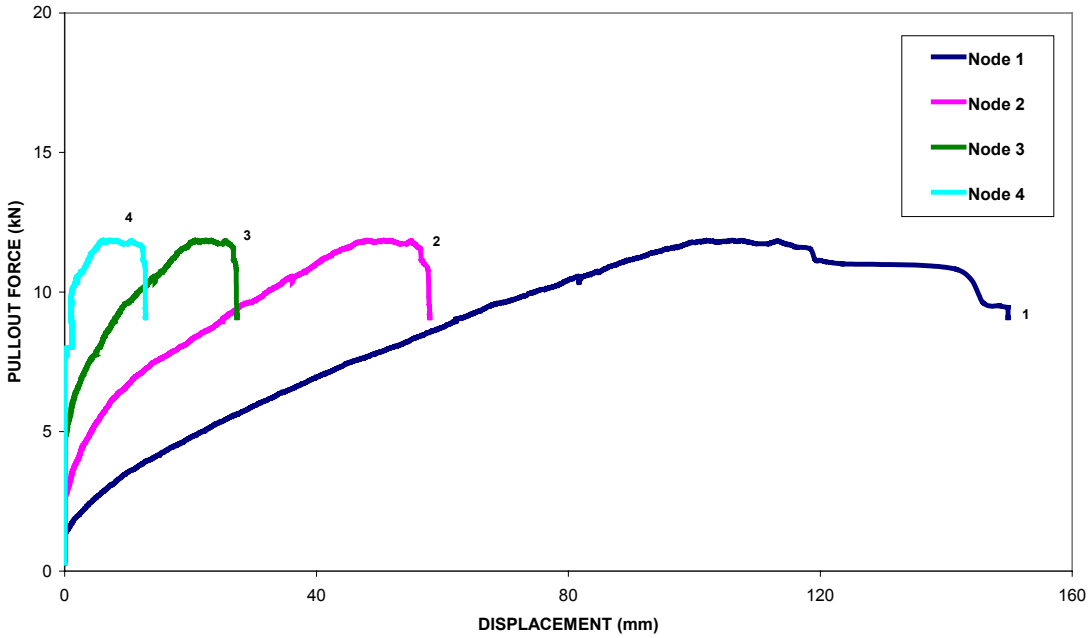


Figure A2: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with <65 mm Crushed Stone (GW) Test 2 (5 kPa Confining Pressure)

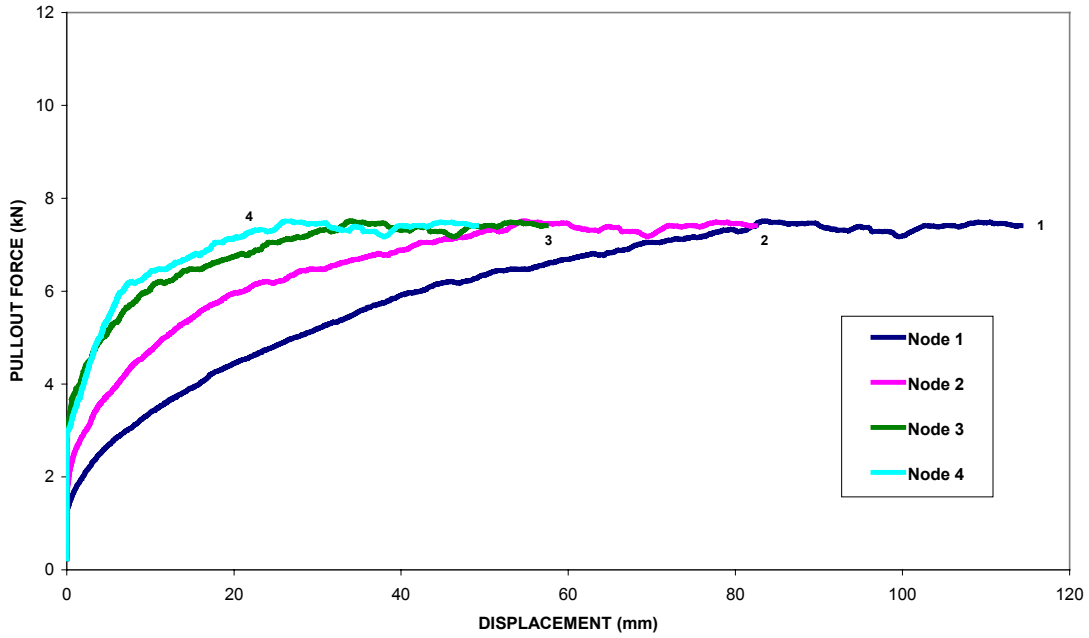


Figure A3: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with <65 mm Crushed Stone (GW) Test 3 (12 kPa Confining Pressure)

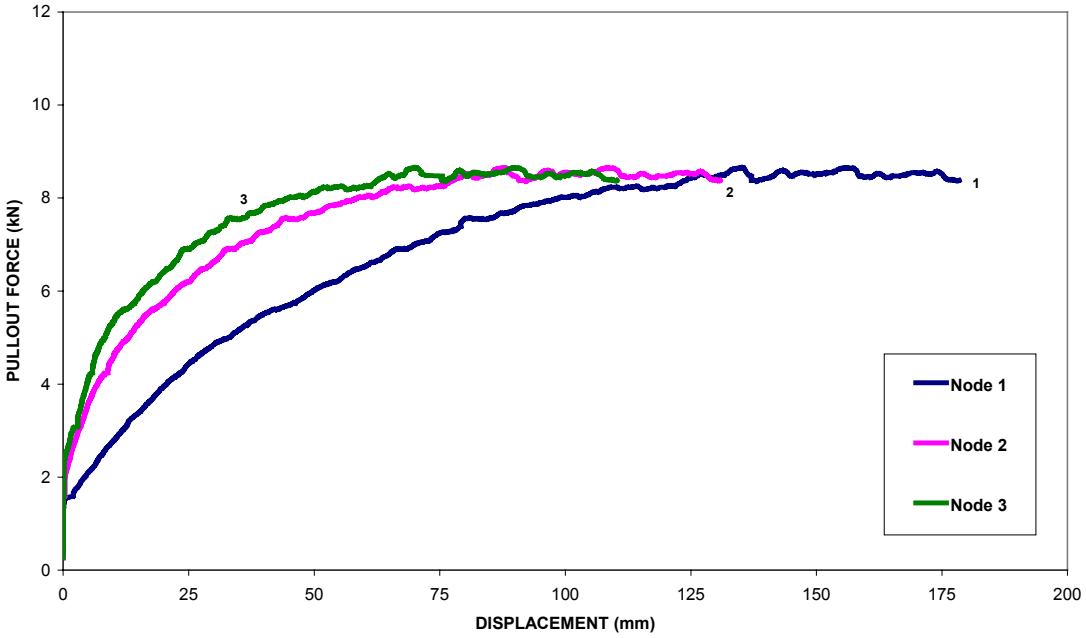


Figure A4: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with <65 mm Crushed Stone (GW) Test 4 (19 kPa Confining Pressure)

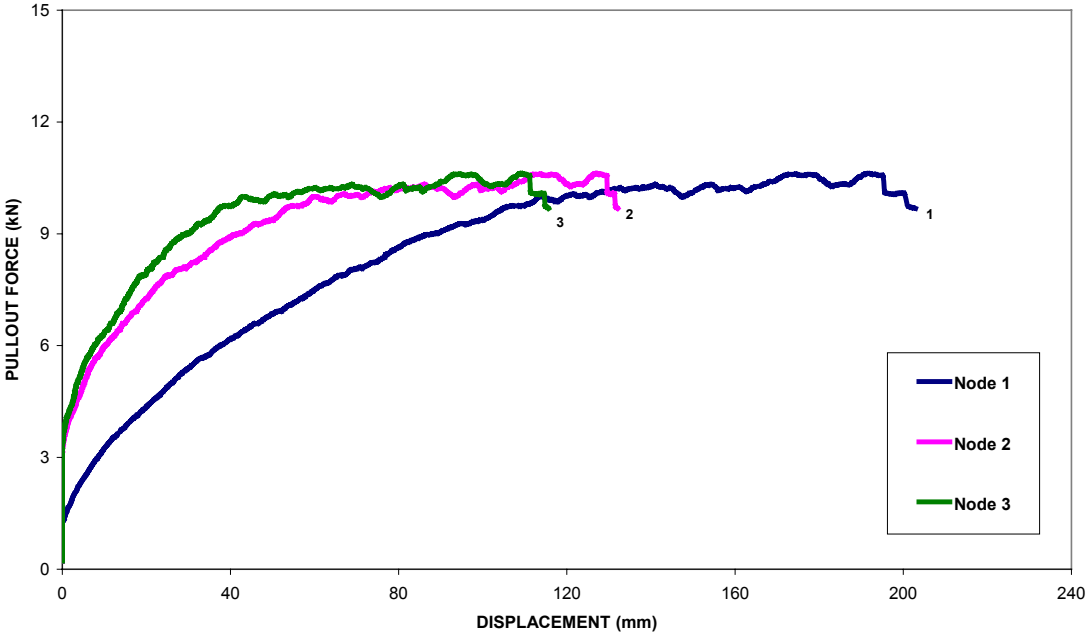


Figure A5: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with <65 mm Crushed Stone (GW) Test 5 (26 kPa Confining Pressure)

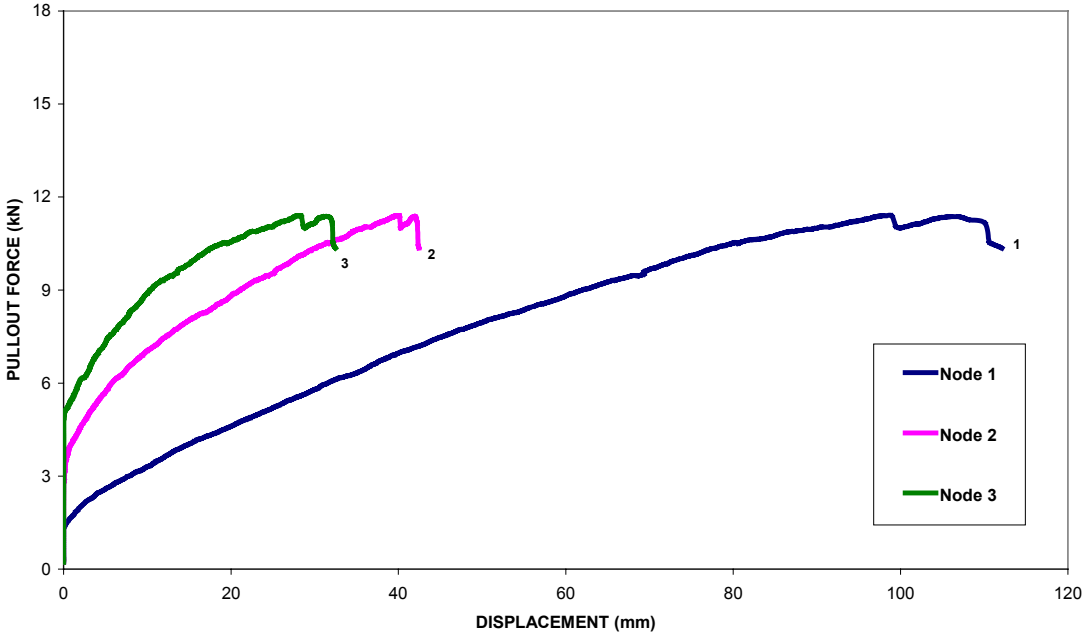
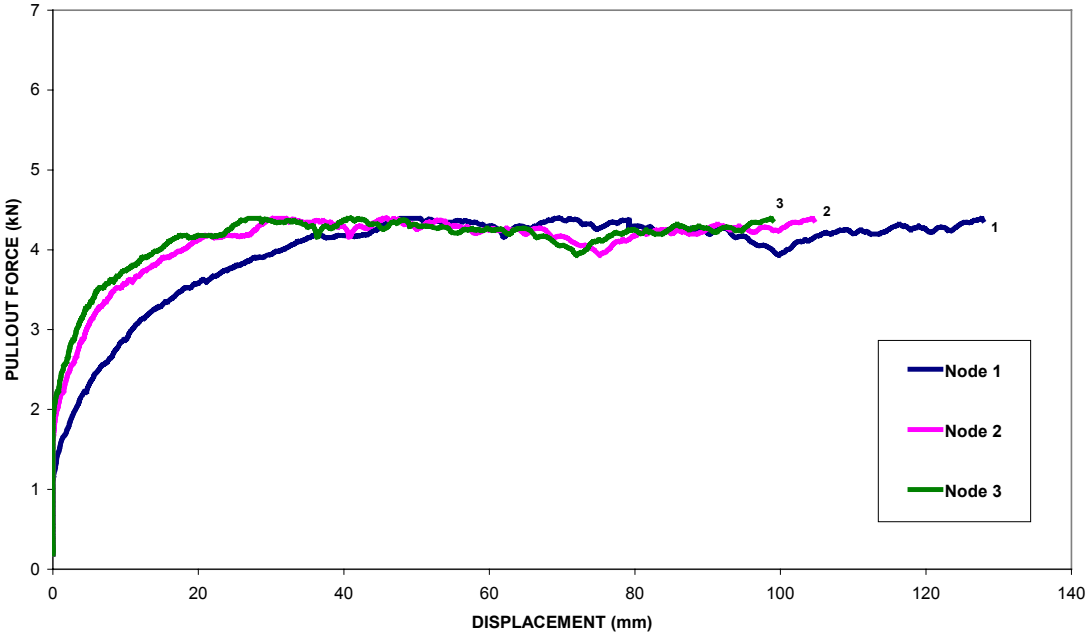


Figure A6: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with <65 mm Crushed Stone (GW) Test 6 (5 kPa Confining Pressure)



Appendix 1.2.7.c

Pullout Tests in Soil Report

Tensar UX1100MSE Geogrids / Fine Silty Sand

ARES® MSE Retaining Wall System

Back to main document

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

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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 <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.

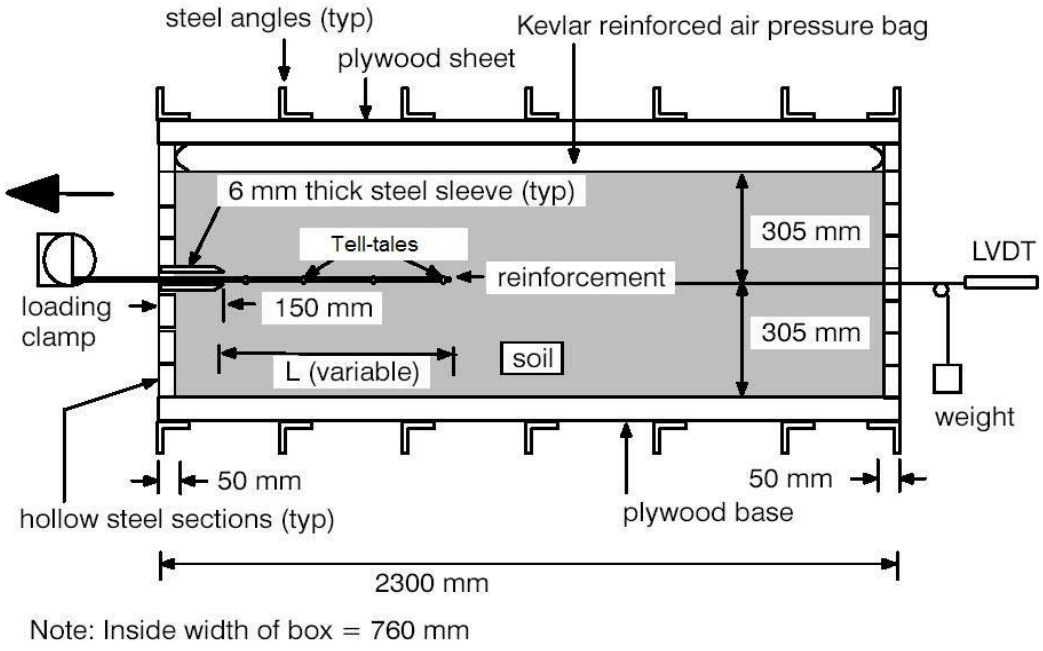


Figure 2: Cross-section view of in-soil pullout box apparatus and general arrangement of soil reinforcement.

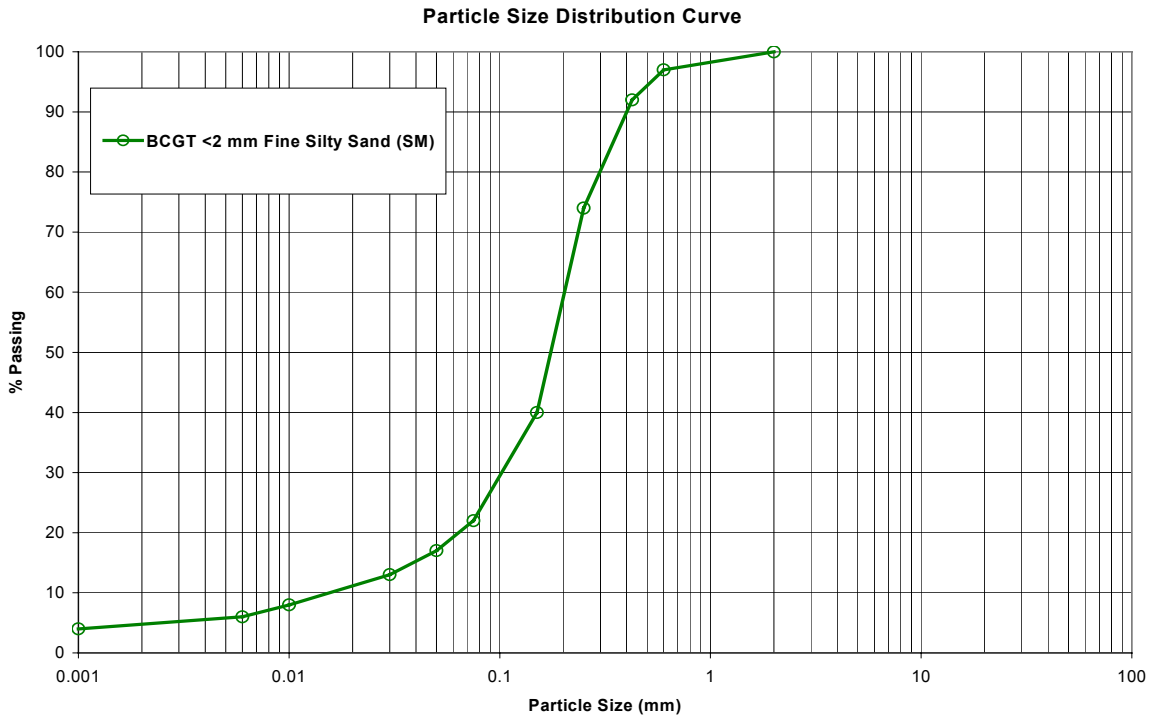


Figure 3: Particle size distribution for <2 mm fine silty sand (SM).

Figure 4: Photograph of a 900 mm-long Tensar UX1100MSE geogrid specimen showing the tell-tale locations.

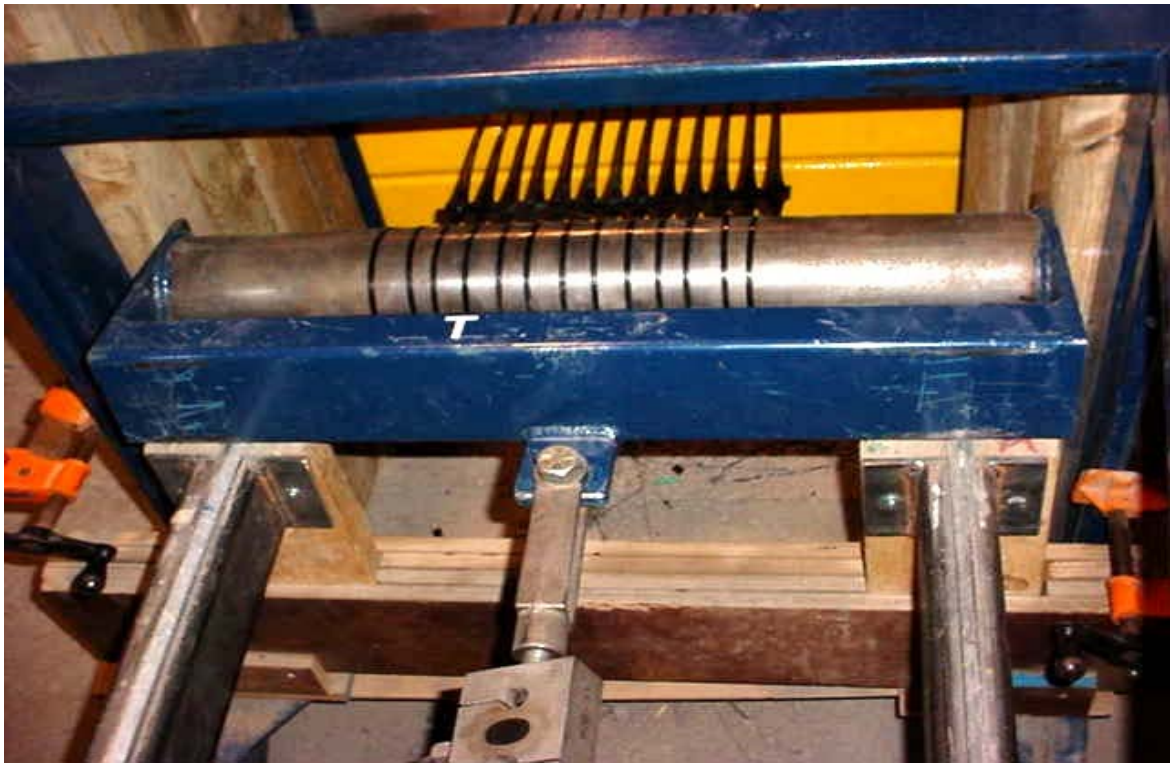


Figure 5: View of clamping arrangement at front of pullout box.

Figure 6: Pullout Capacity vs Confining Pressure for Tensar UX1100MSE Geogrid with <2 mm Fine Silty Sand (SM)

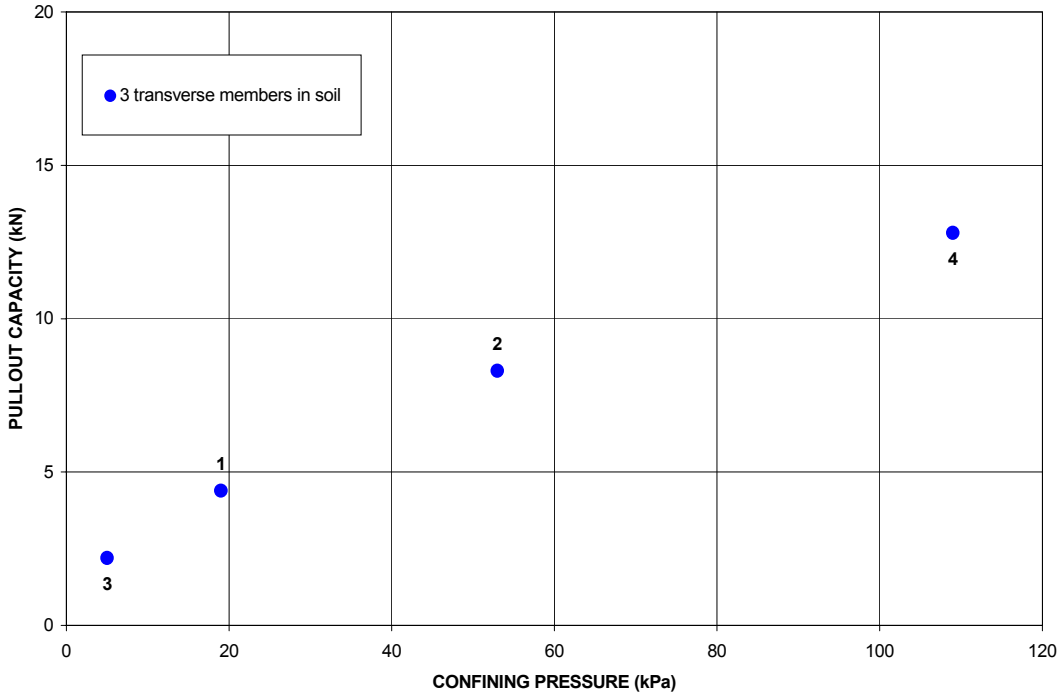


Figure 7: Exhumed Tensar UX1100MSE geogrid specimen (900 mm long) from Test 1, after testing under a confining pressure of 19 kPa.

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)

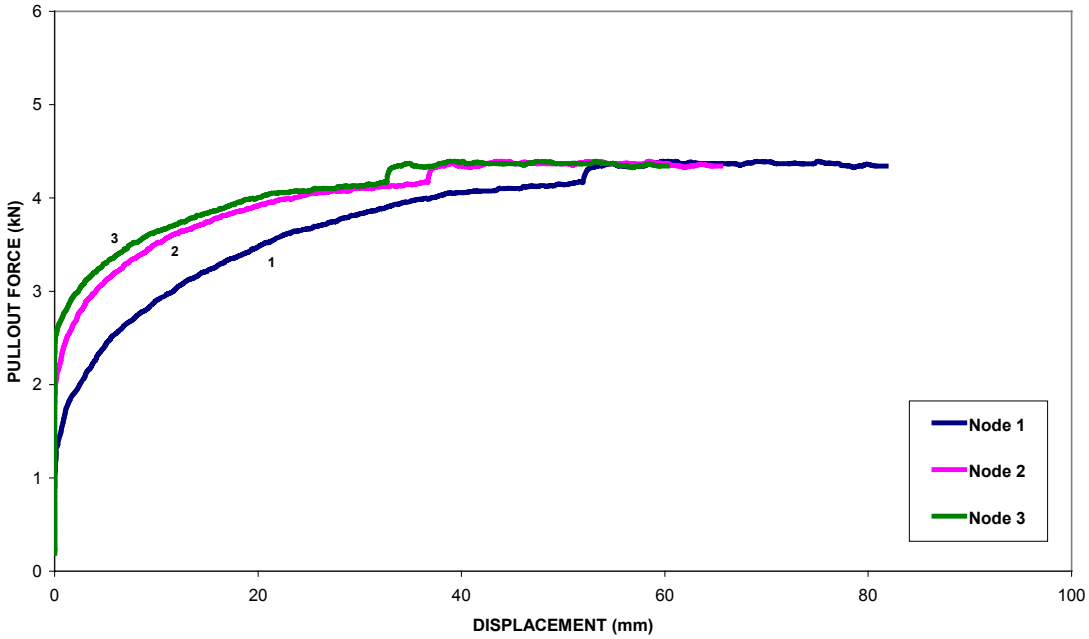


Figure A2: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with <2 mm Fine Silty Sand (SM) Test 2 (53 kPa Confining Pressure)

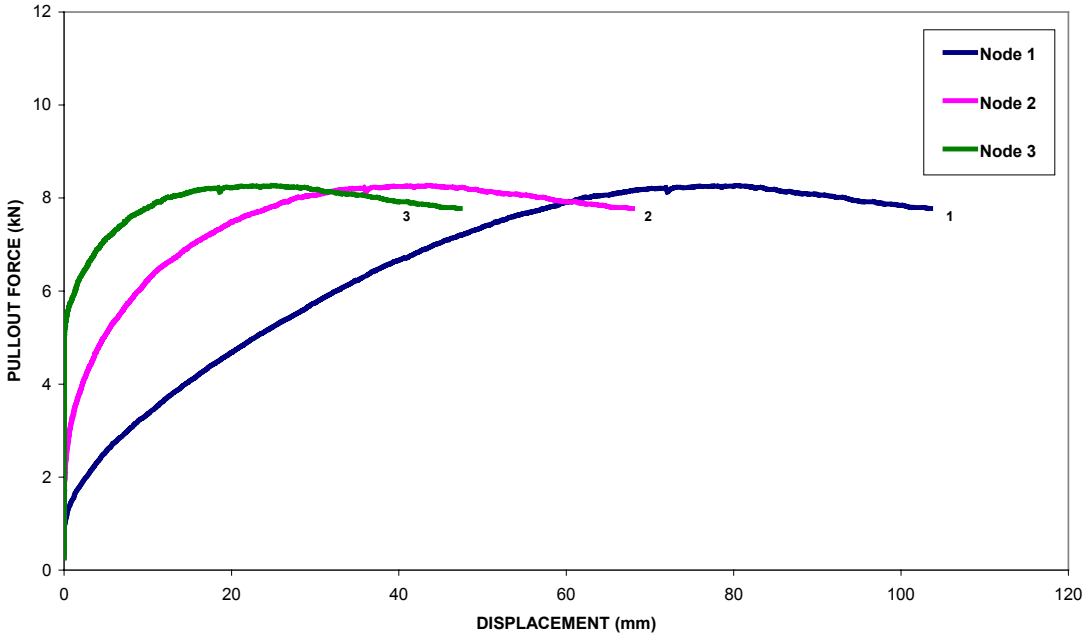


Figure A3: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with <2 mm Fine Silty Sand (SM) Test 3 (5 kPa Confining Pressure)

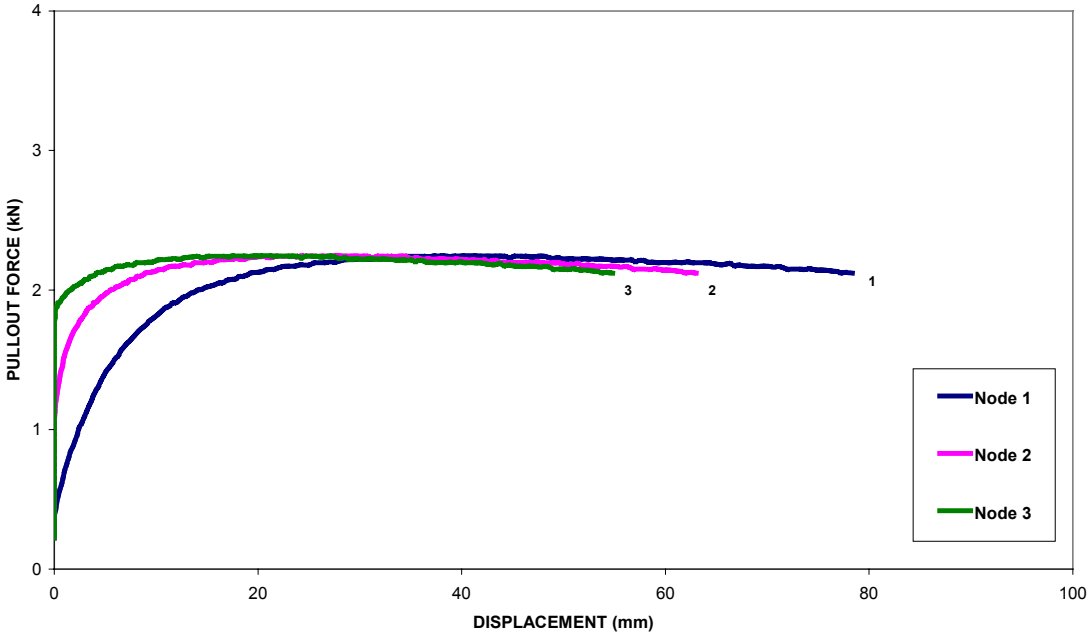
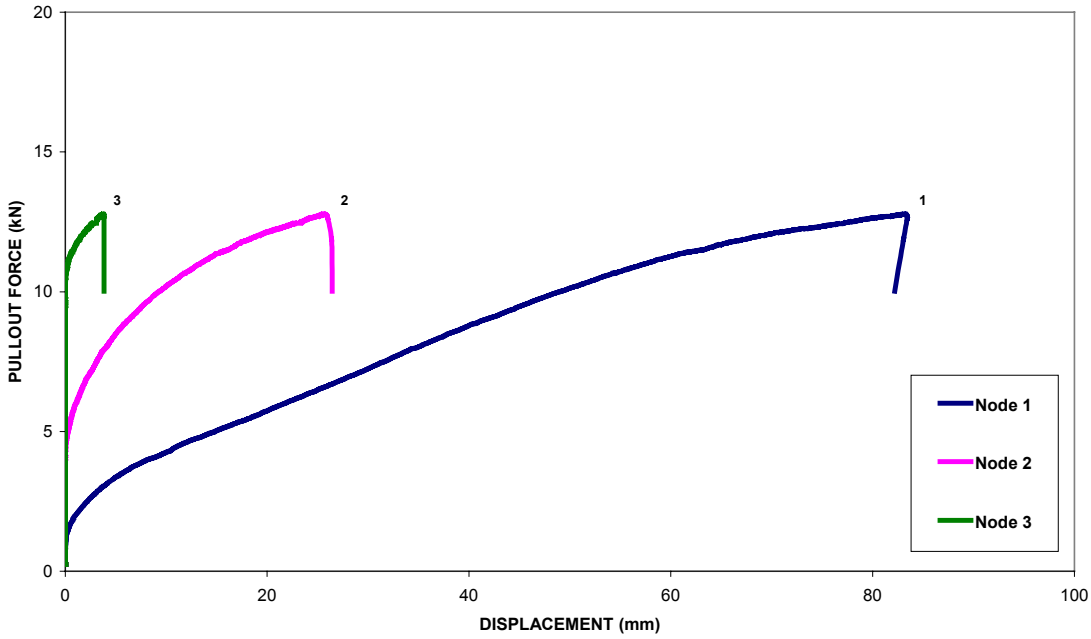


Figure A4: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1100MSE Geogrid with <2 mm Fine Silty Sand (SM) Test 4 (109 kPa Confining Pressure)



Appendix 1.2.7.d

Pullout Tests in Soil Report

Tensar UX1700MSE Geogrids / Course Sand

ARES® MSE Retaining Wall System

Back to main document

FINAL REPORT

RESULTS OF

IN-SOIL PULLOUT TESTING

WITH

Tensar UX1700MSE Geogrid and Coarse Sand (SW)

Submitted to

Tensar Earth Technologies Inc.

CONFIDENTIAL

Distribution:

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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 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.

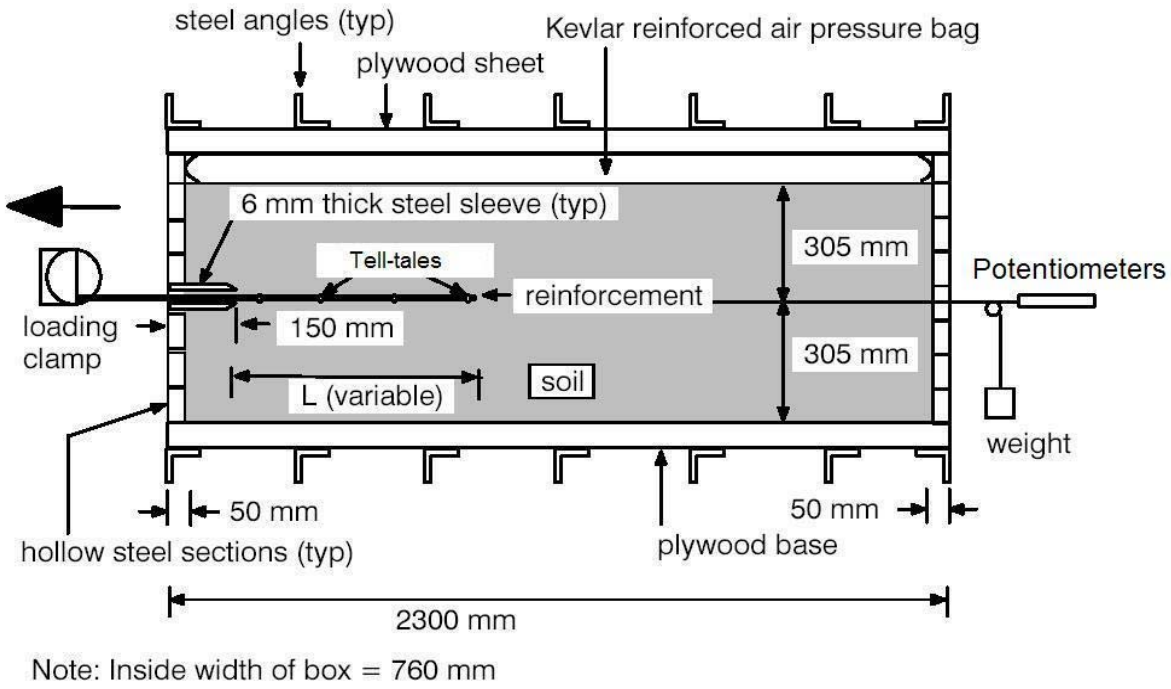


Figure 2: Cross-section view of in-soil pullout box apparatus and general arrangement of soil reinforcement.

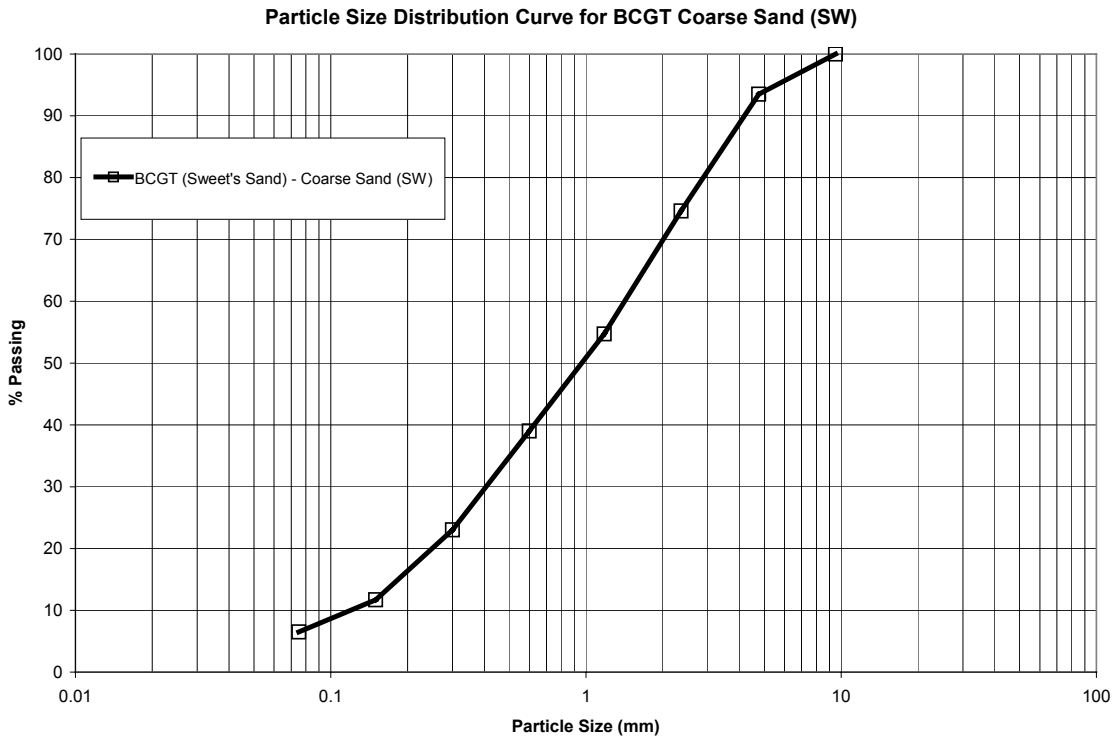


Figure 3: Particle size distribution for coarse sand (SW).

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)

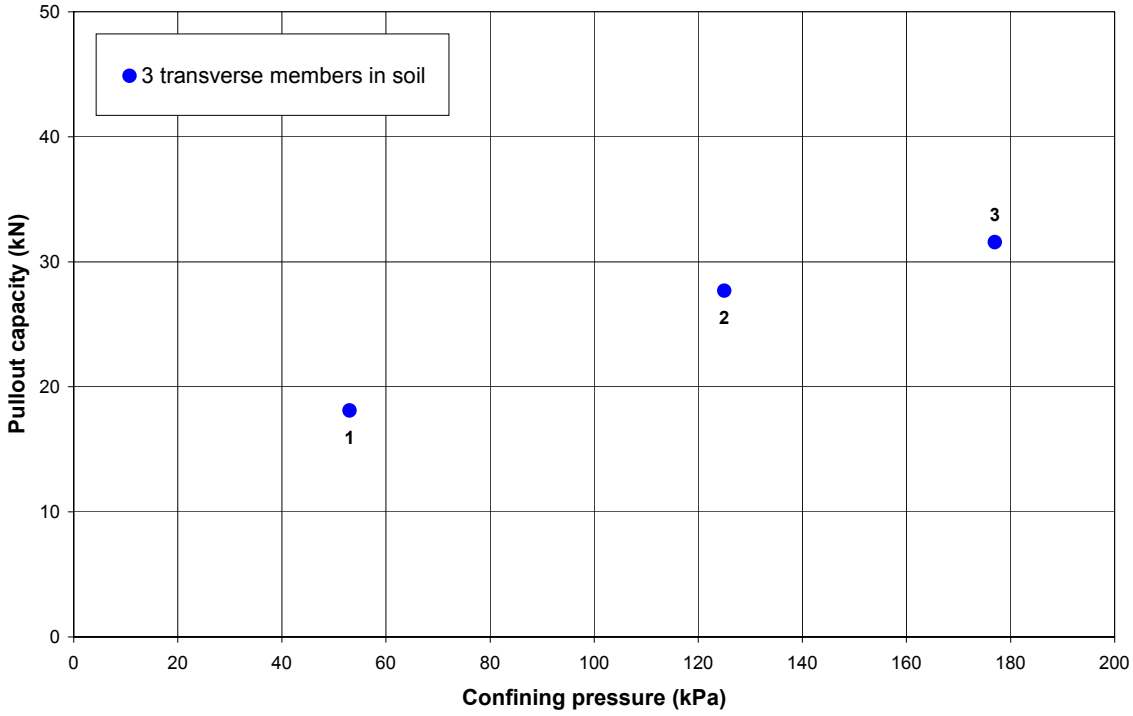


Figure 7: Exhumed Tensar UX1700MSE geogrid specimen (900 mm long) from Test 1, after testing under a confining pressure of 53 kPa.

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)**

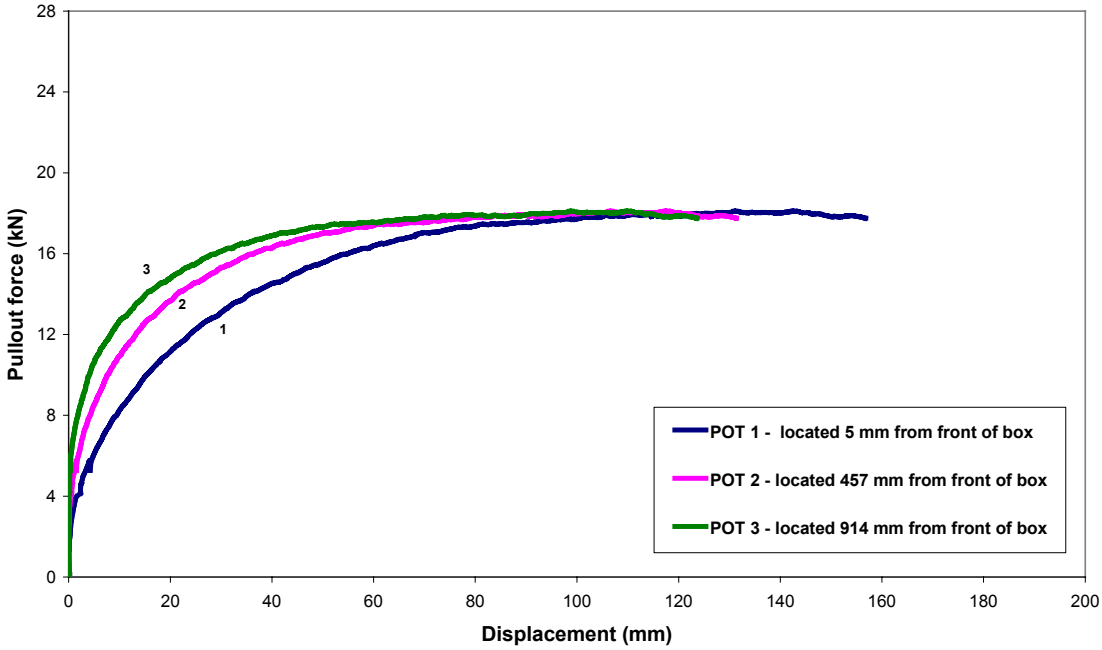


**Figure A2: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1700MSE Geogrid with Coarse Sand (SW)
Test 2
(125 kPa Confining Pressure)**

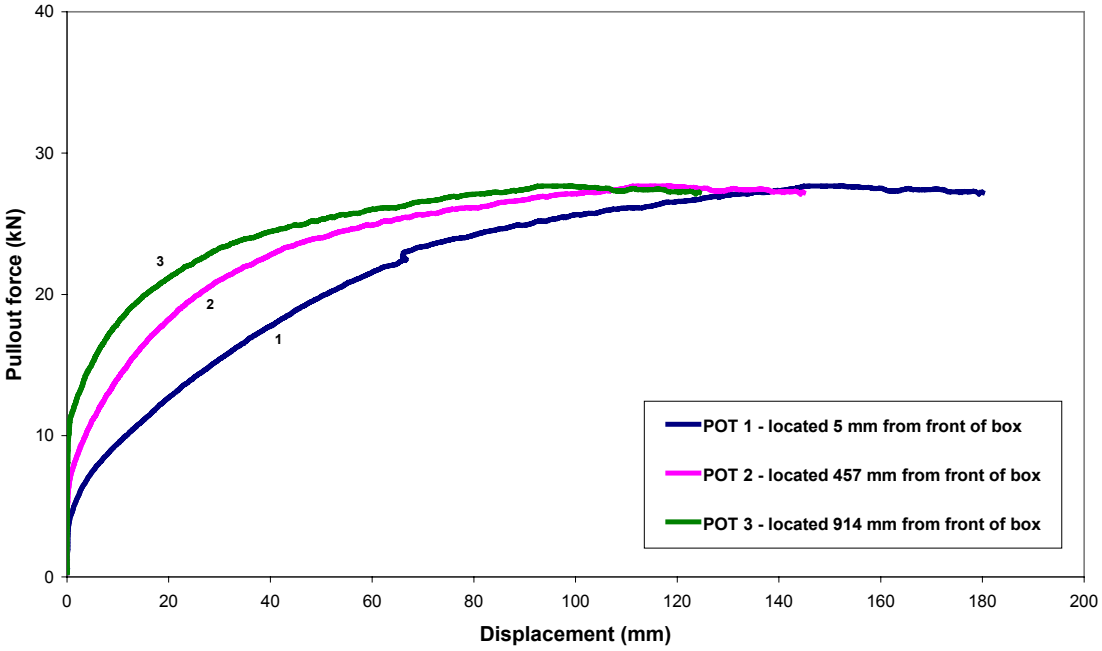
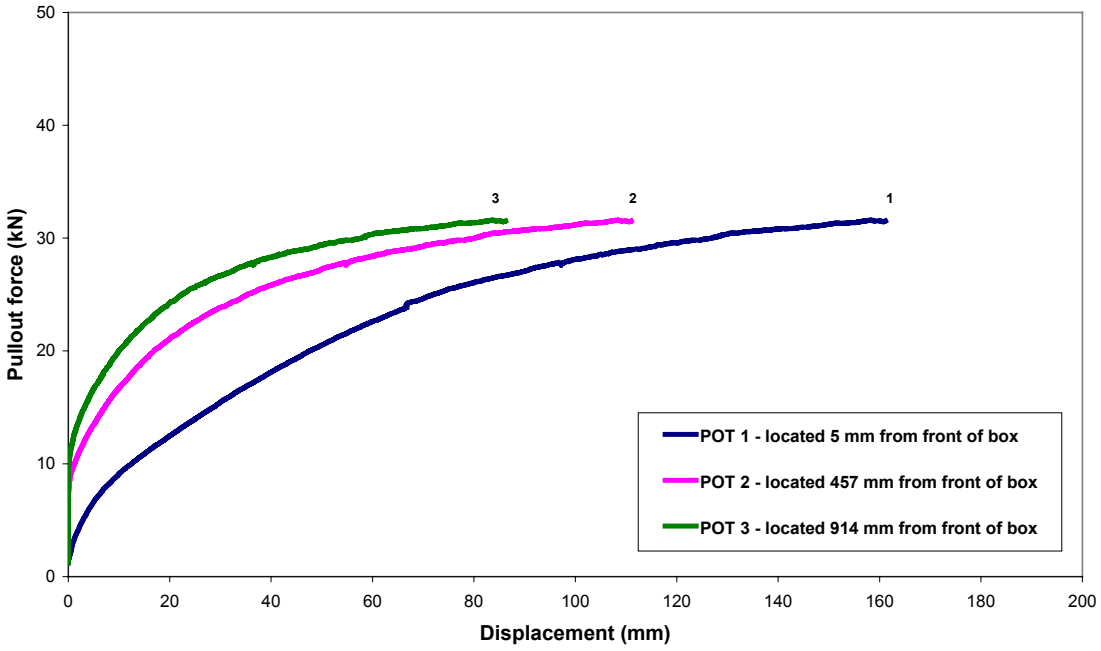


Figure A3: Relationship between Pullout Force and Displacement for Short Term Pullout Test: Tensar UX1700MSE Geogrid with Coarse Sand (SW) Test 3 (177 kPa Confining Pressure)



Appendix 1.2.8

Interface Direct Shear Tests Report

ARES® MSE Retaining Wall System

Back to main document

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

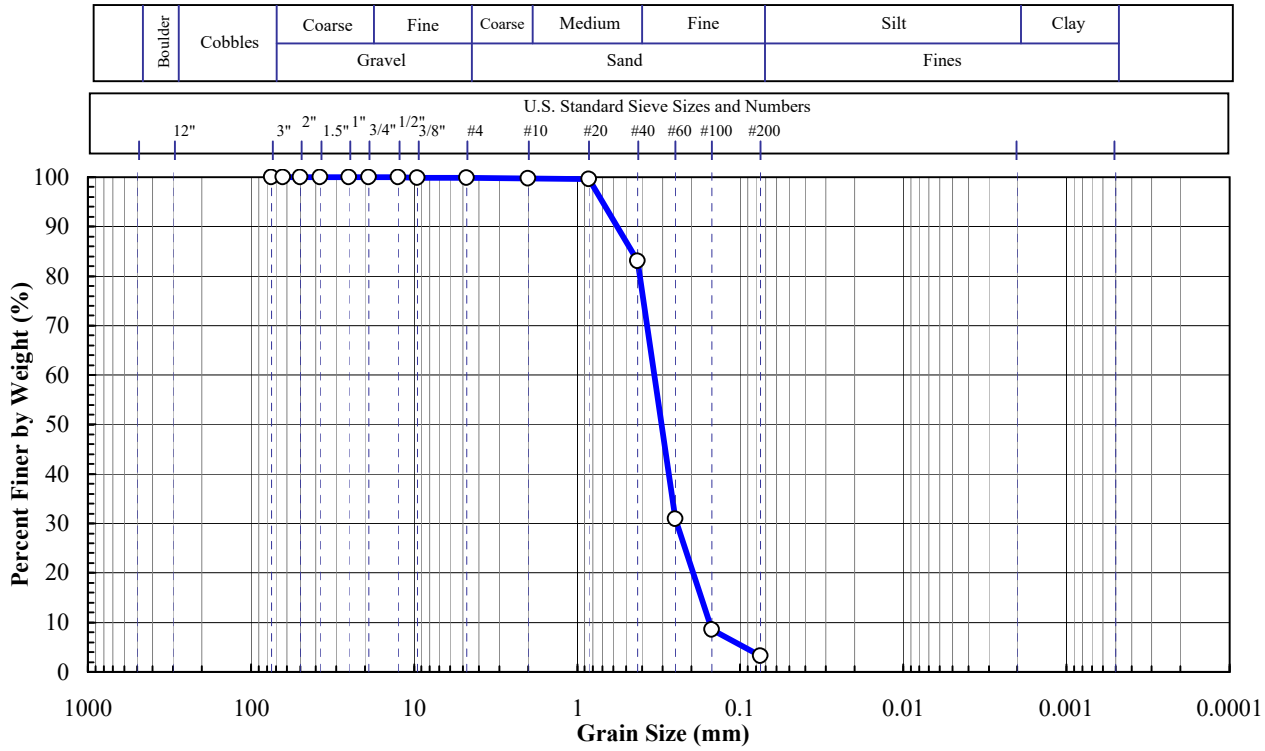
4405 International Blvd., Suite B-117, Norcross, GA 30093
Ph: (770) 931-8222 Fax: (770) 931-8240

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

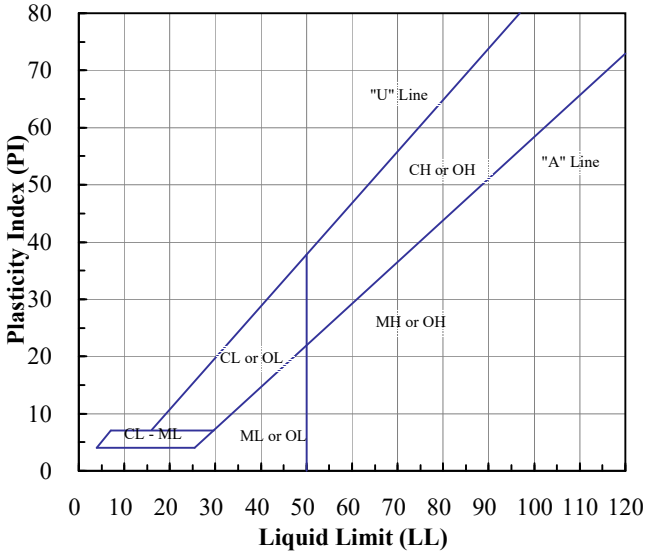


Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
1/2"	12.5	100.0
3/8"	9.5	99.9
#4	4.75	99.9
#10	2.00	99.8
#20	0.850	99.6
#40	0.425	83.0
#60	0.250	30.9
#100	0.150	8.5
#200	0.075	3.2

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



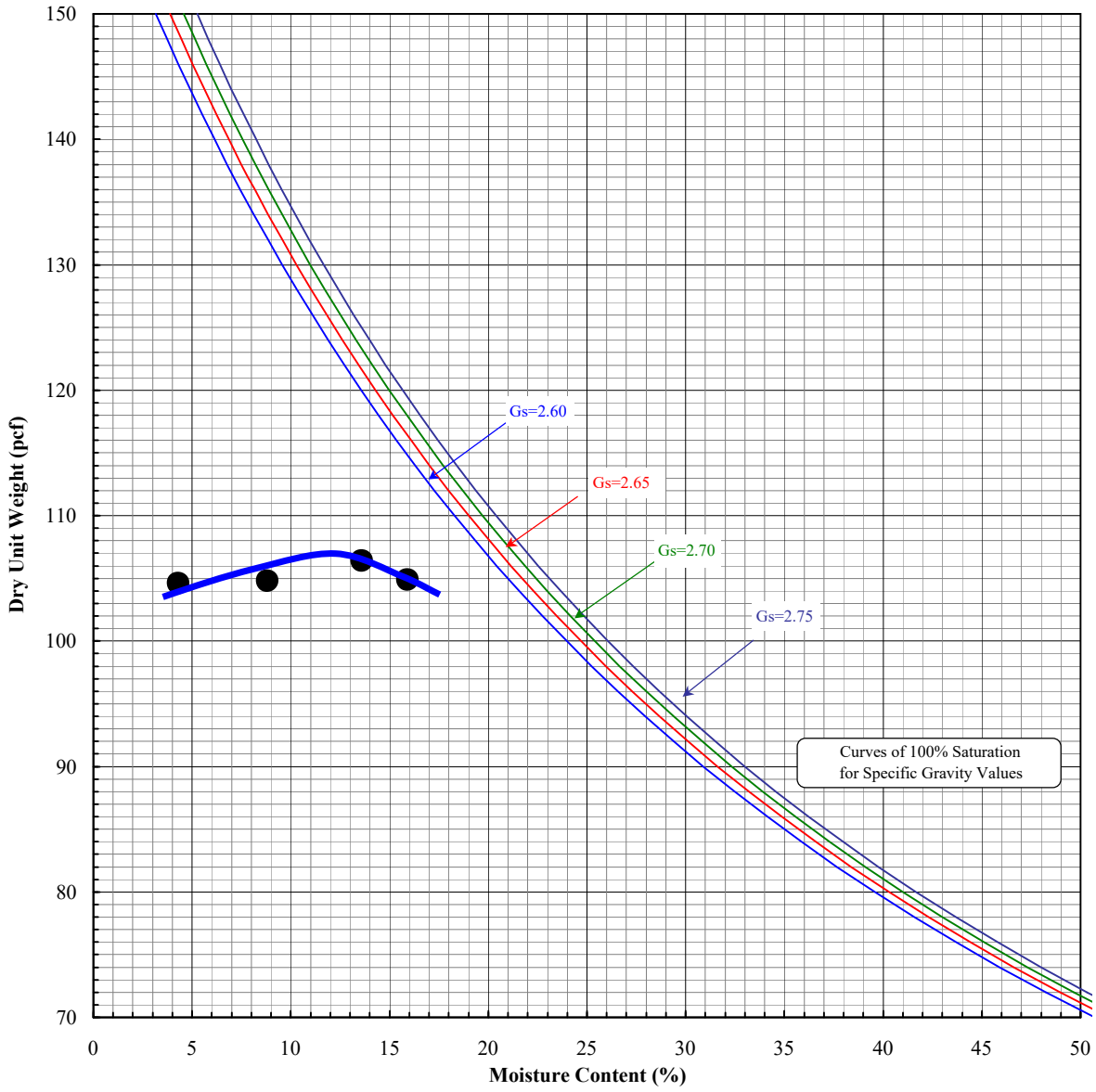
Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits LL (%)	Atterberg Limits PL (%)	Atterberg Limits PI (-)	Engineering Classification
Florida Sand	S23812		3.2				SP

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**



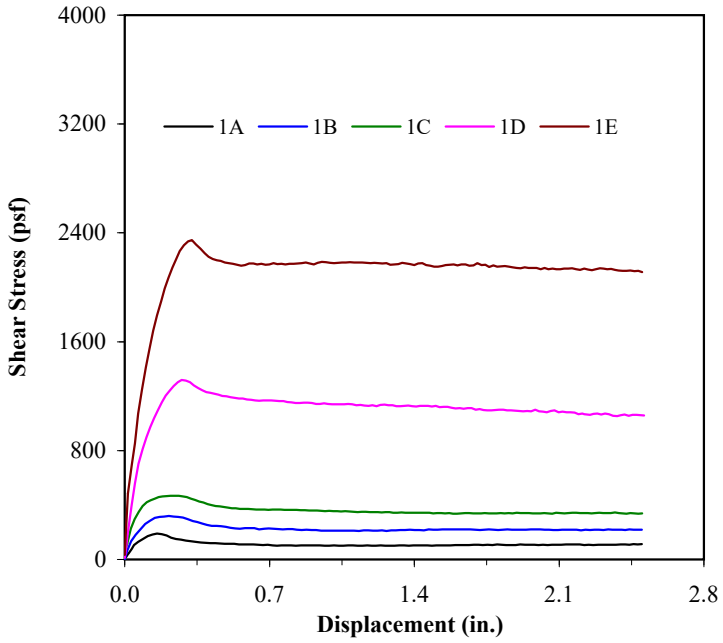
Client Sample ID.	SGI Sample ID NO.	Compaction Data		Maximum Dry Unit Weight and Optimum Moisture Content			
				Measured		Corrected	
		ω_i (%)	γ_d (pcf)	ω_{om} (%)	γ_{dmax} (pcf)	ω_{om} (%)	γ_{dmax} (pcf)
Florida Sand	S23812	4.3	104.6	12.4	106.6		
		8.8	104.8				
		13.6	106.4				
		15.9	104.9				

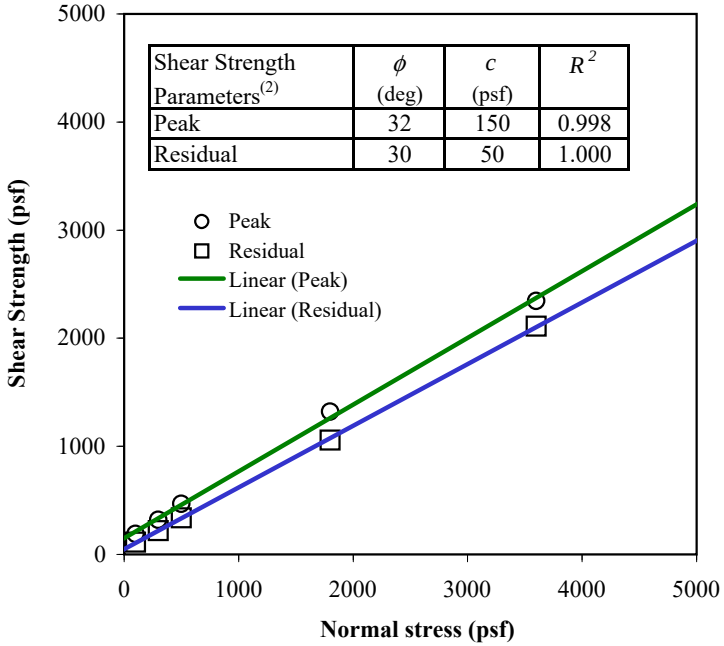
% Retained on #4 Sieve: 0.1
% Retained on #3/8" Sieve:
% Retained on 3/4" Sieve:
% Retained on 2.0" Sieve:

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%)





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.	

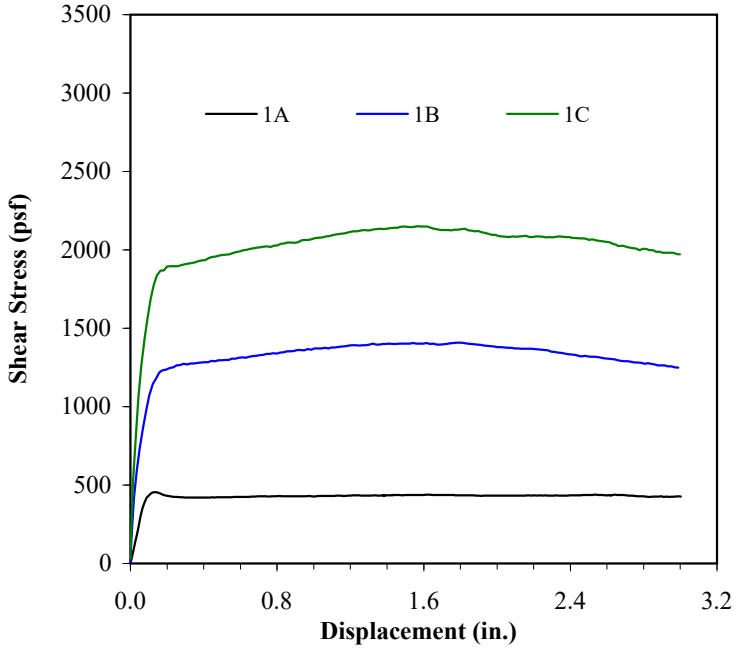
SGI TESTING SERVICES, LLC

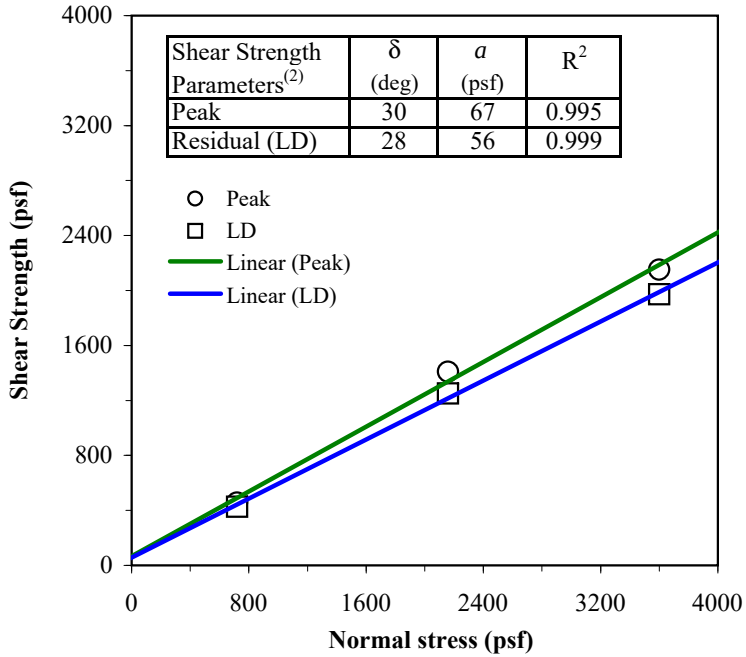
APPENDIX B

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.

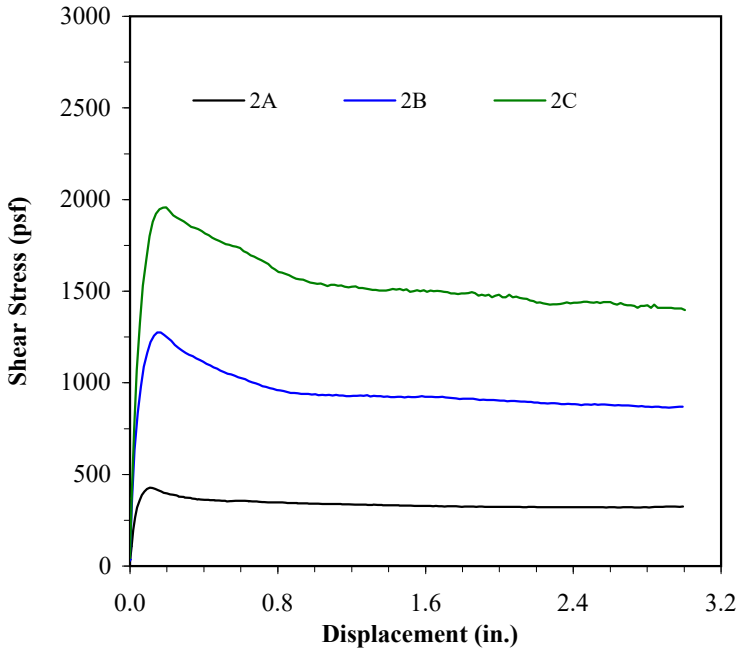
DATE OF REPORT: 1/31/2024

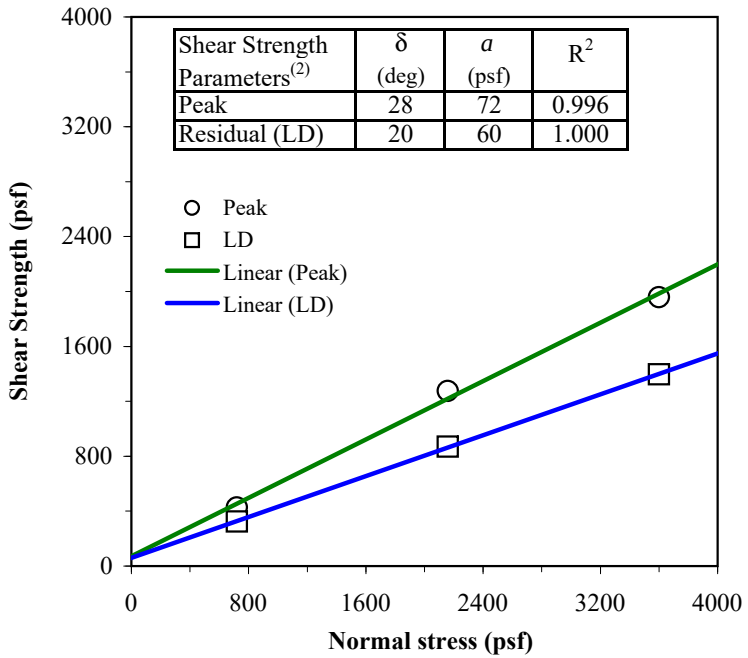
SGI TESTING SERVICES, LLC

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)	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}
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.

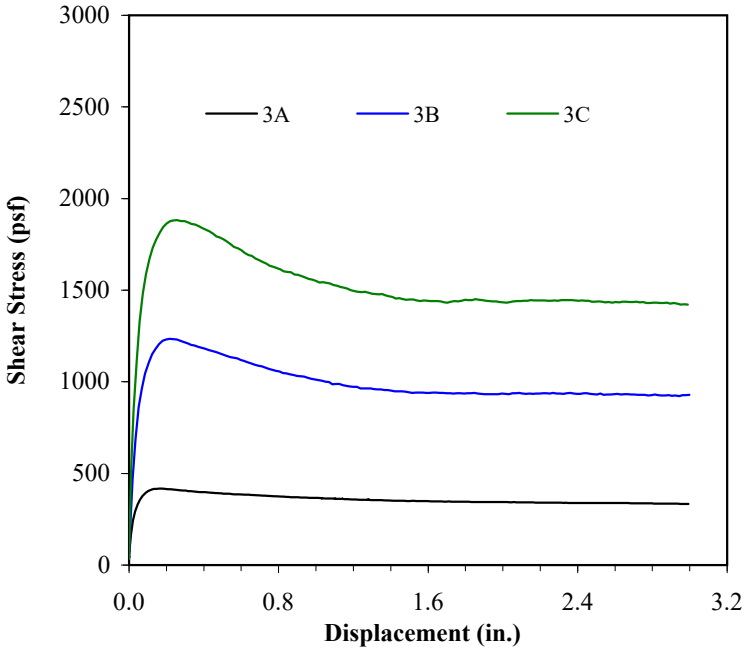
DATE OF REPORT: 1/31/2024

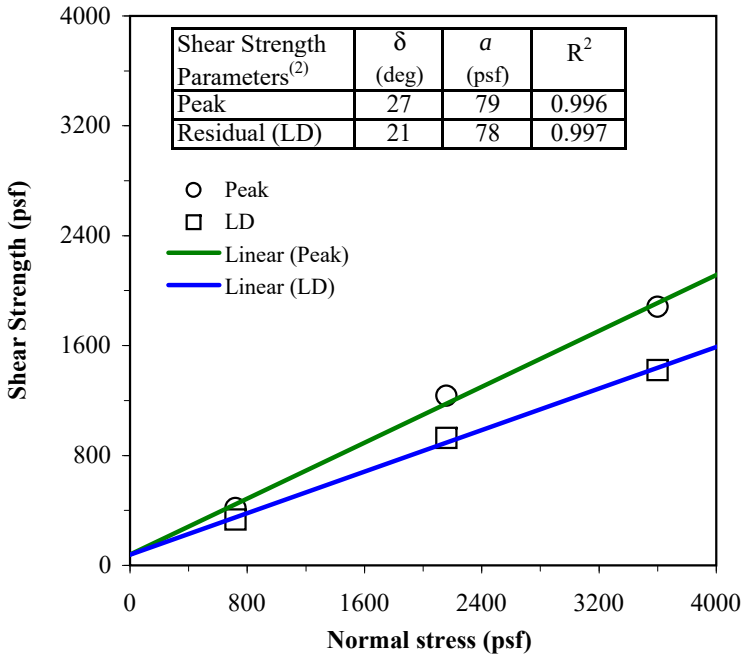
SGI TESTING SERVICES, LLC

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

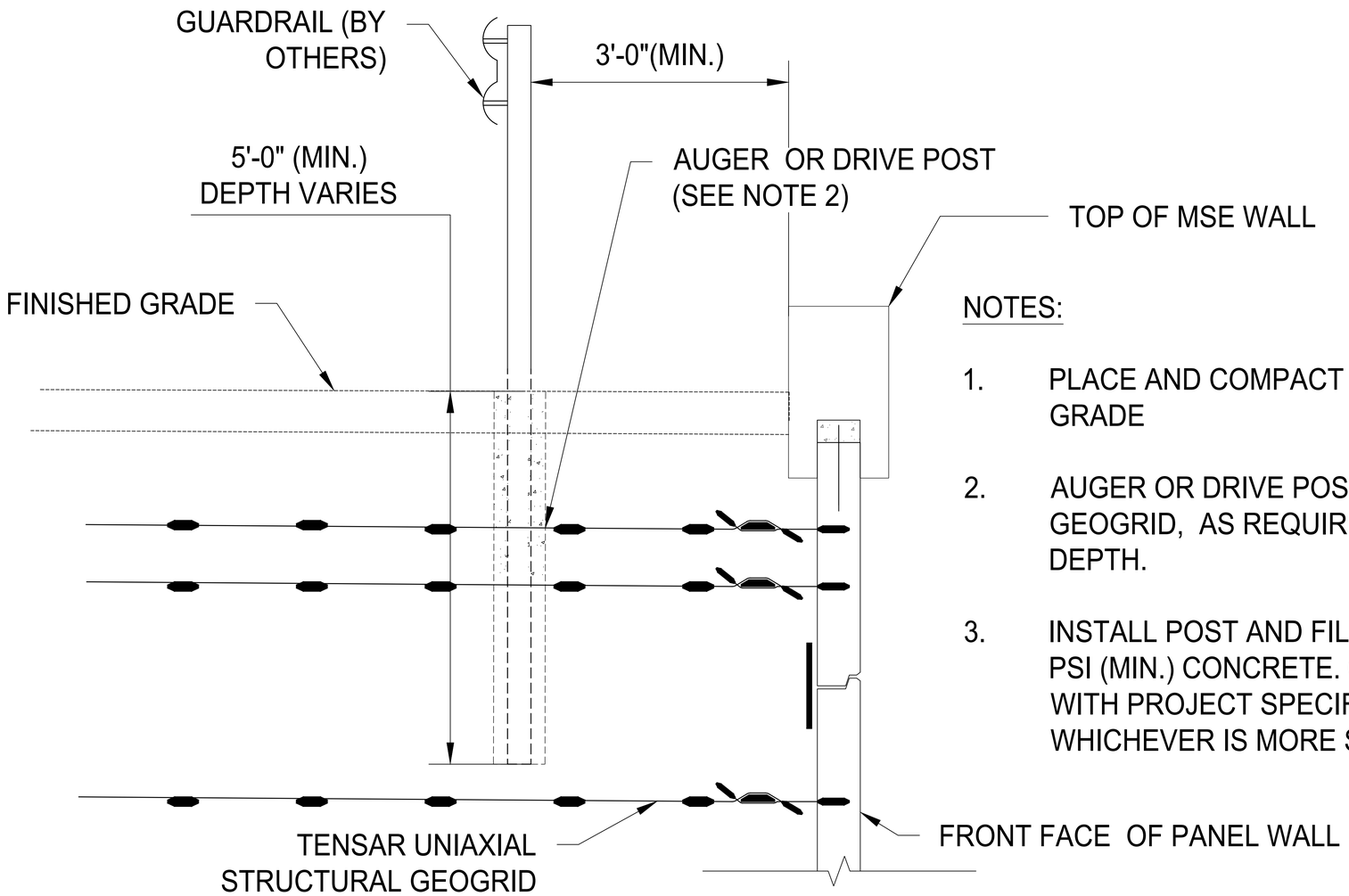
FIGURE NO.	3
PROJECT NO.	SGI22006
DOCUMENT NO.	
FILE NO.	

Appendix 1.3.5

Guardrail Detail on Top of ARES wall

ARES® MSE Retaining Wall System

Back to main document



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.

GUARD RAIL AT TOP OF ARES WALL DETAIL

NOT TO SCALE

Plotted on February 24, 2025
C:\WP\IDEA\ARES\ARES IDEA CAD DETAILS\GUARDRAIL DETAIL.DWG

Tensar
A Division of CMC

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

C.O.A. NO.

THIS DESIGN IS BASED UPON SPECIFIC PROPERTIES OF THOSE SPECIFIC TENSAR PRODUCTS INCORPORATED THEREIN WHICH ARE PROPRIETARY TO TENSAR. ANY SUBSTITUTION OF THE SPECIFIED PRODUCTS WILL INVALIDATE THIS DESIGN. THIS DRAWING IS BEING FURNISHED FOR USE ON THIS SPECIFIC PROJECT ONLY. ANY PARTY ACCEPTING THIS DOCUMENT DOES SO IN CONFIDENCE AND AGREES THAT IT SHALL NOT BE DUPLICATED WHOLE OR IN PART, NOR DISCLOSED TO OTHERS, WITHOUT THE CONSENT OF TENSAR INTERNATIONAL CORPORATION.

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PROJECT NAME AND LOCATION

OWNER

OWNER PROJECT No.

CLIENT
IDEA

TIC PROJECT No.

DRAWN BY: V. Juarez
DESIGNED BY:
CHECKED BY : J. Peralta
ENGINEER OF RECORD (MSE STRUCTURE ONLY):
P.E. NO.

NO.	DATE	DESCRIPTION	BY
0	07/03/2025	ISSUED FOR REVIEW	VJ

REVISION / ISSUE

SHEET TITLE

GUARDRAIL DETAIL

Appendix 2.2.1.0

Panel geogrid spacing

ARES® MSE Retaining Wall System

Back to main document

ARES Geogrid Elevations (Based on 59.25" A Panel)								
A/B1 Combination					B2/B3 Combination			
# Standard Layers	English		Metric		# Standard Layers	English		Metric
	inches	feet	meters			inches	feet	meters
	7	0.58	0.178			7	0.58	0.178
1	15	1.25	0.381		1	15	1.25	0.381
	23	1.92	0.584			22	1.83	0.558
	37	3.08	0.94		2	30	2.50	0.762
2	45	3.75	1.143			38	3.17	0.965
	53	4.42	1.346			52	4.33	1.321
	67	5.58	1.702		3	60	5.00	1.524
3	75	6.25	1.905			68	5.67	1.727
	83	6.92	2.108			82	6.83	2.083
	97	8.08	2.464		4	90	7.50	2.286
4	105	8.75	2.667			98	8.17	2.489
	113	9.42	2.87			112	9.33	2.845
	127	10.58	3.226		5	120	10.00	3.048
5	135	11.25	3.429			128	10.67	3.251
	143	11.92	3.632			142	11.83	3.607
	157	13.08	3.988		6	150	12.50	3.81
6	165	13.75	4.191			158	13.17	4.013
	173	14.42	4.394			172	14.33	4.369
	187	15.58	4.75		7	180	15.00	4.572
7	195	16.25	4.953			188	15.67	4.775
	203	16.92	5.156			202	16.83	5.131
	217	18.08	5.512		8	210	17.50	5.334
8	225	18.75	5.715			218	18.17	5.537
	233	19.42	5.918			232	19.33	5.893
	247	20.58	6.274		9	240	20.00	6.096
9	255	21.25	6.477			248	20.67	6.299
	263	21.92	6.68			262	21.83	6.655
	277	23.08	7.036		10	270	22.50	6.858
10	285	23.75	7.239			278	23.17	7.061
	293	24.42	7.442			292	24.33	7.417
	307	25.58	7.798		11	300	25.00	7.62
11	315	26.25	8.001			308	25.67	7.823
	323	26.92	8.204			322	26.83	8.179
	337	28.08	8.56		12	330	27.50	8.382
12	345	28.75	8.763			338	28.17	8.585
	353	29.42	8.966			352	29.33	8.941
	367	30.58	9.322		13	360	30.00	9.144
13	375	31.25	9.525			368	30.67	9.347
	383	31.92	9.728			382	31.83	9.703
	397	33.08	10.084		14	390	32.50	9.906
14	405	33.75	10.287			398	33.17	10.109
	413	34.42	10.49			412	34.33	10.465
	427	35.58	10.846		15	420	35.00	10.668
15	435	36.25	11.049			428	35.67	10.871
	443	36.92	11.252			442	36.83	11.227
	457	38.08	11.608		16	450	37.50	11.43
16	465	38.75	11.811			458	38.17	11.633
	473	39.42	12.014			472	39.33	11.989
	487	40.58	12.37		17	480	40.00	12.192
17	495	41.25	12.573			488	40.67	12.395
	503	41.92	12.776			502	41.83	12.751
	517	43.08	13.132		18	510	42.50	12.954
18	525	43.75	13.335			518	43.17	13.157
	533	44.42	13.538			532	44.33	13.513
	547	45.58	13.894		19	540	45.00	13.716
19	555	46.25	14.097			548	45.67	13.919
	563	46.92	14.3			562	46.83	14.275
	577	48.08	14.656		20	570	47.50	14.478
20	585	48.75	14.859			578	48.17	14.681
	593	49.42	15.062			592	49.33	15.037
	607	50.58	15.418		21	600	50.00	15.24

Appendix 2.2.1.1a

Problem 1 with Silty Sand

ARES® MSE Retaining Wall System

Back to main document

AASHTO 2017-2020
ARES Panel Wall System

MSEW+: Update # 2021.12

PROJECT IDENTIFICATION

Title:	ARES Panel Wall System
Project Number:	IDEA-Problem 1
Client:	IDEA
Designer:	JP
Station Number:	ARES - AASHTO

Description:

ARES System - 75 yrs Design Life. Silty Sand backfill material

Company's information:

Name: Tensar International Corporation
Street: 2500 Northwinds Parkway
Suite 500
Alpharetta, GA 30009
Telephone #: (770) 344-2090
Fax #: (770) 344-2084
E-Mail: Jorge.Peralta@cmc.com

File path and name: C:\WF\IDEA\ARES\Final\ARES Submittal Documents\Supporti.....
.....in bodkin)_75yr.BENp

Original date and time of creating this file: May 2025

PROGRAM MODE:

ANALYSIS
of a SIMPLE STRUCTURE
using GEOGRID as reinforcing material.

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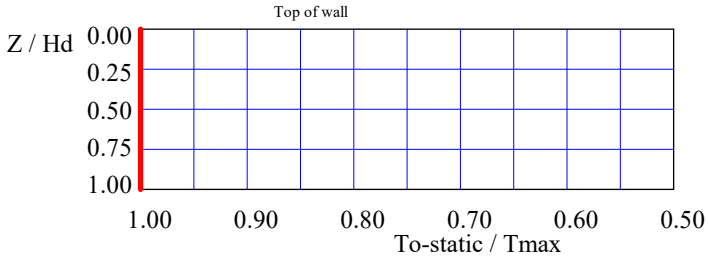
Version MSEW+ Version MSEW+ Version MSEW+

Version MSEW+ Version MSEW+ Version MSEW+

INPUT DATA: Facia and Connection (Analysis)

FACIA type: Segmental precast concrete panels.
Depth of panel is 0.49 ft. Horizontal distance to Center of Gravity of panel is 0.25 ft.
Average unit weight of panel is γ_f = 152.78 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

D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	UX1400MSE..	UX1500MSE..	UX1600MSE..	UX1700MSE..	N/A
Durability reduction factor, RFd	1.10	1.10	1.10	1.10	N/A
Creep reduction factor, RFc	2.77	2.77	2.77	2.77	N/A
Capacity Demand Ratio: connection break, CDR	N/A	N/A	N/A	N/A	N/A
Capacity Demand Ratio: connection pullout, CDR	N/A	N/A	N/A	N/A	N/A
CRu = Tult-connection/Tult-geogrid	0.93	0.93	0.93	0.93	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 }

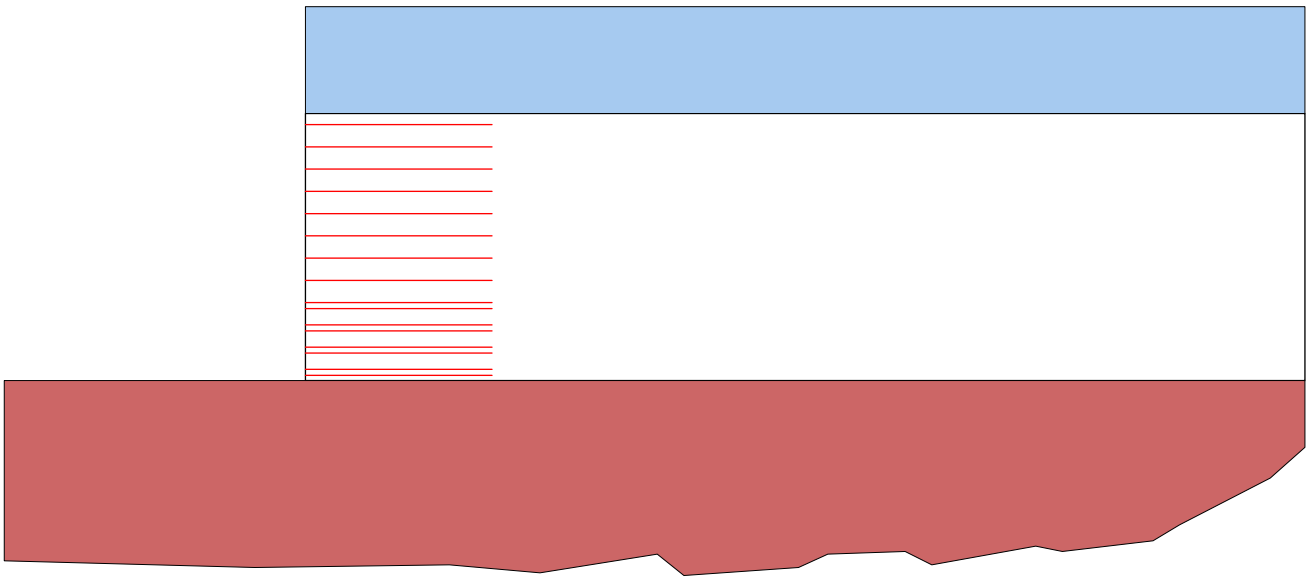
Soil in front of wall is Horizontal.

Batter, ω 0.0 [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	1.20

Resistance factor for reinforcement tension in connectors	ϕ	Static	Combined static/seismic
Geogrid:		0.90	1.20

Resistance factor for geosynthetic pullout	ϕ	0.90	1.20

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}):	$(\gamma_{p\text{-}EH})_{EQ}$	1.10
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	1.00	1.00
Reinforced Soil and Reinforcement	ϕ_τ	1.00	1.00

Resistance factor for bearing capacity of shallow foundation		Static	Combined Static/Seismic
	ϕ_b	0.65	0.65

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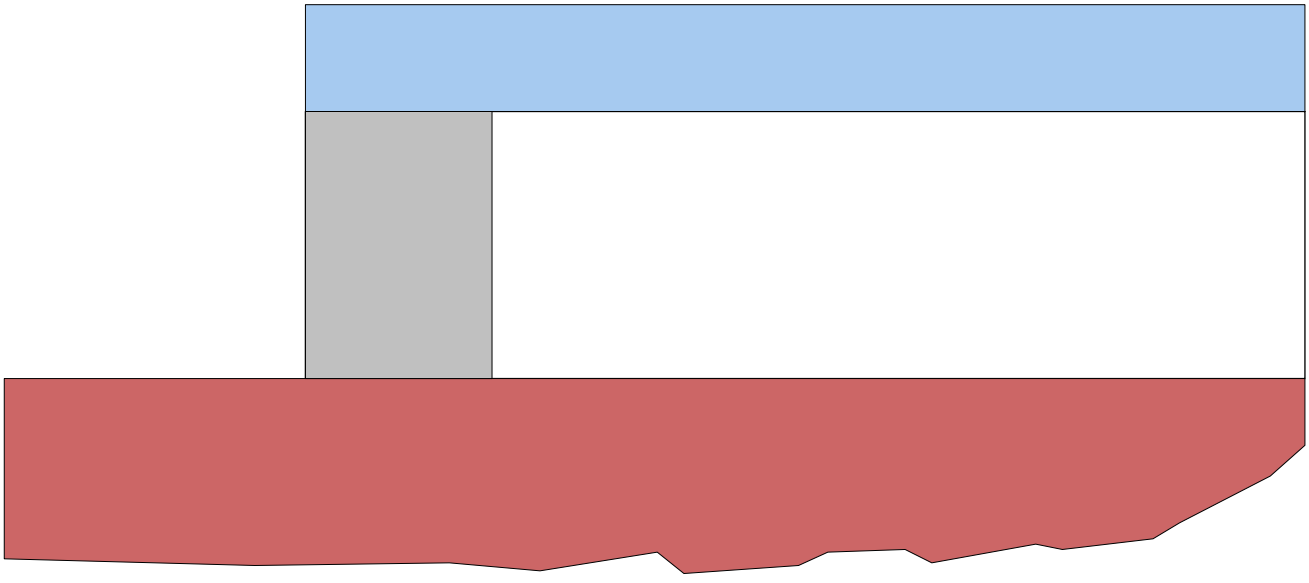
BEARING CAPACITY for GIVEN LAYOUT – Using AASHTO 2017-2020 method

	STATIC	SEISMIC	UNITS
(Water table does not affect bearing capacity)			
Factored bearing resistance. q-n	15982	N/A	[lb/ft ²]
Factored bearing load, σ_V	7302.1	N/A	[lb/ft ²]
Eccentricity, e	1.35	N/A	[ft]
Eccentricity, e/L	0.064	N/A	
CDR calculated	2.19	N/A	
Base length	21.00	N/A	[ft]

Unfactored applied bearing pressure = (Unfactored R) / [L - 2 * (Unfactored e)] =
Unfactored R = 96553.71 [lb/ft], L = 21.00, Unfactored e = 1.21 [ft], and Sigma = 5195.51 [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 = 2.077

#	Geogrid Elevation [ft]	Geogrid Length [ft]	CDR Static	CDR Seismic	Geogrid Type #	Product name
1	0.58	21.00	2.020	N/A	2	UX1500MSE-AR
2	1.25	21.00	2.057	N/A	3	UX1600MSE-AR
3	3.08	21.00	2.164	N/A	3	UX1600MSE-AR
4	3.75	21.00	2.206	N/A	3	UX1600MSE-AR
5	5.58	21.00	2.331	N/A	2	UX1500MSE-AR
6	6.25	21.00	2.381	N/A	2	UX1500MSE-AR
7	8.08	21.00	2.530	N/A	2	UX1500MSE-AR
8	8.75	21.00	2.590	N/A	3	UX1600MSE-AR
9	11.25	21.00	2.843	N/A	4	UX1700MSE-AR
10	13.75	21.00	3.155	N/A	4	UX1700MSE-AR
11	16.25	21.00	3.552	N/A	3	UX1600MSE-AR
12	18.75	21.00	4.072	N/A	2	UX1500MSE-AR
13	21.25	21.00	4.783	N/A	2	UX1500MSE-AR
14	23.75	21.00	5.814	N/A	2	UX1500MSE-AR
15	26.25	21.00	7.443	N/A	1	UX1400MSE-AR
16	28.75	21.00	10.406	N/A	1	UX1400MSE-AR

ECCENTRICITY for GIVEN LAYOUT

At interface with foundation: e/L static = 0.0910; Overturning: CDR-static = 3.89

#	Geogrid Elevation [ft]	Geogrid Length [ft]	e / L Static	e / L Seismic	Geogrid Type #	Product name
1	0.58	21.00	0.0871	N/A	2	UX1500MSE-AR
2	1.25	21.00	0.0827	N/A	3	UX1600MSE-AR
3	3.08	21.00	0.0711	N/A	3	UX1600MSE-AR
4	3.75	21.00	0.0670	N/A	3	UX1600MSE-AR
5	5.58	21.00	0.0564	N/A	2	UX1500MSE-AR
6	6.25	21.00	0.0527	N/A	2	UX1500MSE-AR
7	8.08	21.00	0.0432	N/A	2	UX1500MSE-AR
8	8.75	21.00	0.0399	N/A	3	UX1600MSE-AR
9	11.25	21.00	0.0285	N/A	4	UX1700MSE-AR
10	13.75	21.00	0.0185	N/A	4	UX1700MSE-AR
11	16.25	21.00	0.0101	N/A	3	UX1600MSE-AR
12	18.75	21.00	0.0032	N/A	2	UX1500MSE-AR
13	21.25	21.00	-0.0021	N/A	2	UX1500MSE-AR
14	23.75	21.00	-0.0058	N/A	2	UX1500MSE-AR
15	26.25	21.00	-0.0079	N/A	1	UX1400MSE-AR
16	28.75	21.00	-0.0082	N/A	1	UX1400MSE-AR

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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 break, CRu	Reduction factor for connection pullout, CRs	Available connection strength, Tc-break criterion [lb/ft]	Available connection strength, Tc-pullout criterion [lb/ft]	Available Geogrid strength, Tavailable [lb/ft]	CDR connection break		CDR connection pullout		CDR Geogrid strength		Product name
								Specified	Actual	Specified	Actual	Specified	Actual	
1	0.58	1506	0.93	N/A	2149	N/A	2452	N/A	1.40	N/A	N/A	N/A		UX1500MSE-AR
2	1.25	1988	0.93	N/A	2715	N/A	3072	N/A	1.34	N/A	N/A	N/A		UX1600MSE-AR
3	3.08	1907	0.93	N/A	2715	N/A	3072	N/A	1.40	N/A	N/A	N/A		UX1600MSE-AR
4	3.75	1827	0.93	N/A	2715	N/A	3072	N/A	1.46	N/A	N/A	N/A		UX1600MSE-AR
5	5.58	1746	0.93	N/A	2149	N/A	2452	N/A	1.21	N/A	N/A	N/A		UX1500MSE-AR
6	6.25	1666	0.93	N/A	2149	N/A	2452	N/A	1.27	N/A	N/A	N/A		UX1500MSE-AR
7	8.08	1585	0.93	N/A	2149	N/A	2452	N/A	1.33	N/A	N/A	N/A		UX1500MSE-AR
8	8.75	1894	0.93	N/A	2715	N/A	3072	N/A	1.41	N/A	N/A	N/A		UX1600MSE-AR
9	11.25	2724	0.93	N/A	3299	N/A	3637	N/A	1.19	N/A	N/A	N/A		UX1700MSE-AR
10	13.75	2402	0.93	N/A	3299	N/A	3637	N/A	1.35	N/A	N/A	N/A		UX1700MSE-AR
11	16.25	2080	0.93	N/A	2715	N/A	3072	N/A	1.28	N/A	N/A	N/A		UX1600MSE-AR
12	18.75	1758	0.93	N/A	2149	N/A	2452	N/A	1.20	N/A	N/A	N/A		UX1500MSE-AR
13	21.25	1436	0.93	N/A	2149	N/A	2452	N/A	1.47	N/A	N/A	N/A		UX1500MSE-AR
14	23.75	1114	0.93	N/A	2149	N/A	2452	N/A	1.89	N/A	N/A	N/A		UX1500MSE-AR
15	26.25	792	0.93	N/A	1321	N/A	1416	N/A	1.55	N/A	N/A	N/A		UX1400MSE-AR
16	28.75	470	0.93	N/A	1321	N/A	1416	N/A	2.61	N/A	N/A	N/A		UX1400MSE-AR

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Appendix 2.2.1.1b

Problem 1 with Sandy Gravel

ARES® MSE Retaining Wall System

Back to main document

AASHTO 2017-2020
ARES Panel Wall System

MSEW+: Update # 2021.12

PROJECT IDENTIFICATION

Title: ARES Panel Wall System
Project Number: IDEA-Problem 1
Client: IDEA
Designer: JP
Station Number: ARES - AASHTO

Description:

ARES System - 75 yrs Design Life. Sandy Gravel backfill material

Company's information:

Name: Tensar International Corporation
Street: 2500 Northwinds Parkway
Suite 500
Alpharetta, GA 30009
Telephone #: (770) 344-2090
Fax #: (770) 344-2084
E-Mail: Jorge.Peralta@cmc.com

File path and name: C:\WF\IDEA\ARES\Final\ARES Submittal Documents\Supporti.....
.....yr(SandyGravel).BENp

Original date and time of creating this file: May 2025

PROGRAM MODE:

ANALYSIS
of a SIMPLE STRUCTURE
using GEOGRID as reinforcing material.

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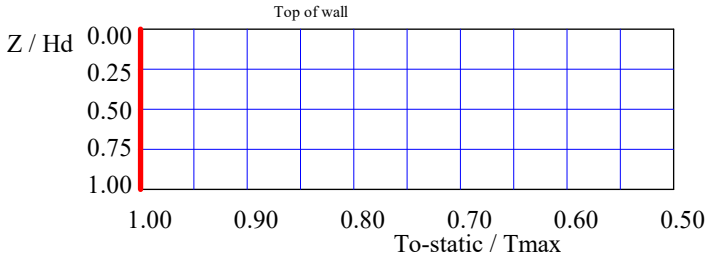
Version MSEW+ Version MSEW+ Version MSEW+

Version MSEW+ Version MSEW+ Version MSEW+

INPUT DATA: Facia and Connection (Analysis)

FACIA type: Segmental precast concrete panels.
Depth of panel is 0.49 ft. Horizontal distance to Center of Gravity of panel is 0.25 ft.
Average unit weight of panel is γ_f = 152.78 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

D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	UX1400MSE..	UX1500MSE..	UX1600MSE..	UX1700MSE..	N/A
Durability reduction factor, RFd	1.10	1.10	1.10	1.10	N/A
Creep reduction factor, RFc	2.77	2.77	2.77	2.77	N/A
Capacity Demand Ratio: connection break, CDR	N/A	N/A	N/A	N/A	N/A
Capacity Demand Ratio: connection pullout, CDR	N/A	N/A	N/A	N/A	N/A
CRu = Tult-connection/Tult-geogrid	0.93	0.93	0.93	0.93	N/A

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License number MSEW-401084

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 }

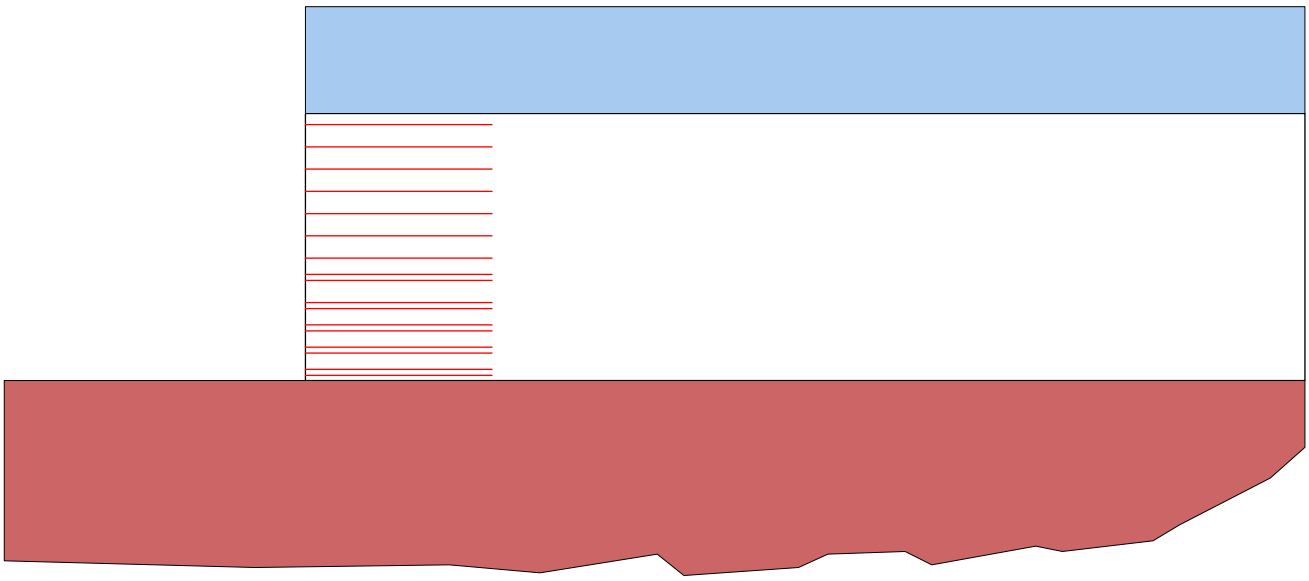
Soil in front of wall is Horizontal.

Batter, ω 0.0 [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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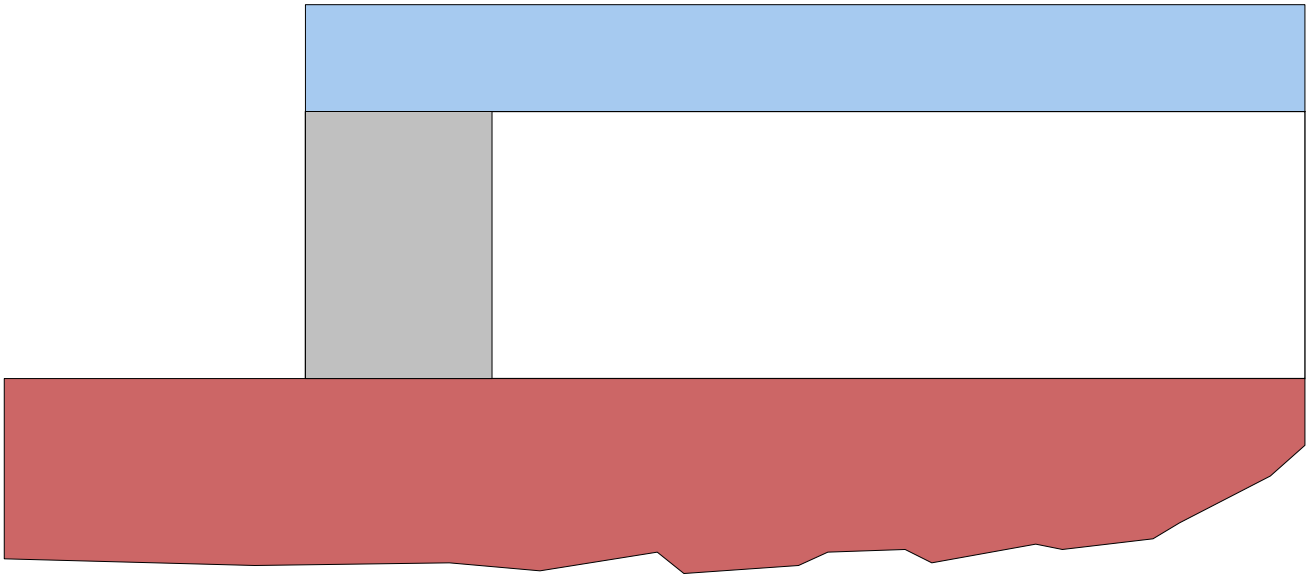
BEARING CAPACITY for GIVEN LAYOUT – Using AASHTO 2017-2020 method

	STATIC	SEISMIC	UNITS
(Water table does not affect bearing capacity)			
Factored bearing resistance. q-n	15982	N/A	[lb/ft ²]
Factored bearing load, σ_V	7302.1	N/A	[lb/ft ²]
Eccentricity, e	1.35	N/A	[ft]
Eccentricity, e/L	0.064	N/A	
CDR calculated	2.19	N/A	
Base length	21.00	N/A	[ft]

Unfactored applied bearing pressure = (Unfactored R) / [L - 2 * (Unfactored e)] =
Unfactored R = 96553.71 [lb/ft], L = 21.00, Unfactored e = 1.21 [ft], and Sigma = 5195.51 [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 = 2.077

#	Geogrid Elevation [ft]	Geogrid Length [ft]	CDR Static	CDR Seismic	Geogrid Type #	Product name
1	0.58	21.00	2.020	N/A	2	UX1500MSE-AR
2	1.25	21.00	2.057	N/A	3	UX1600MSE-AR
3	3.08	21.00	2.164	N/A	3	UX1600MSE-AR
4	3.75	21.00	2.206	N/A	3	UX1600MSE-AR
5	5.58	21.00	2.331	N/A	3	UX1600MSE-AR
6	6.25	21.00	2.381	N/A	2	UX1500MSE-AR
7	8.08	21.00	2.530	N/A	2	UX1500MSE-AR
8	8.75	21.00	2.590	N/A	3	UX1600MSE-AR
9	11.25	21.00	2.843	N/A	3	UX1600MSE-AR
10	11.92	21.00	2.920	N/A	3	UX1600MSE-AR
11	13.75	21.00	3.155	N/A	3	UX1600MSE-AR
12	16.25	21.00	3.552	N/A	3	UX1600MSE-AR
13	18.75	21.00	4.072	N/A	3	UX1600MSE-AR
14	21.25	21.00	4.783	N/A	2	UX1500MSE-AR
15	23.75	21.00	5.814	N/A	2	UX1500MSE-AR
16	26.25	21.00	7.443	N/A	1	UX1400MSE-AR
17	28.75	21.00	10.406	N/A	1	UX1400MSE-AR

ECCENTRICITY for GIVEN LAYOUT

At interface with foundation: e/L static = 0.0910; Overturning: CDR-static = 3.89

#	Geogrid Elevation [ft]	Geogrid Length [ft]	e / L Static	e / L Seismic	Geogrid Type #	Product name
1	0.58	21.00	0.0871	N/A	2	UX1500MSE-AR
2	1.25	21.00	0.0827	N/A	3	UX1600MSE-AR
3	3.08	21.00	0.0711	N/A	3	UX1600MSE-AR
4	3.75	21.00	0.0670	N/A	3	UX1600MSE-AR
5	5.58	21.00	0.0564	N/A	3	UX1600MSE-AR
6	6.25	21.00	0.0527	N/A	2	UX1500MSE-AR
7	8.08	21.00	0.0432	N/A	2	UX1500MSE-AR
8	8.75	21.00	0.0399	N/A	3	UX1600MSE-AR
9	11.25	21.00	0.0285	N/A	3	UX1600MSE-AR
10	11.92	21.00	0.0257	N/A	3	UX1600MSE-AR
11	13.75	21.00	0.0185	N/A	3	UX1600MSE-AR
12	16.25	21.00	0.0101	N/A	3	UX1600MSE-AR
13	18.75	21.00	0.0032	N/A	3	UX1600MSE-AR
14	21.25	21.00	-0.0021	N/A	2	UX1500MSE-AR
15	23.75	21.00	-0.0058	N/A	2	UX1500MSE-AR
16	26.25	21.00	-0.0079	N/A	1	UX1400MSE-AR
17	28.75	21.00	-0.0082	N/A	1	UX1400MSE-AR

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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	0.58	1689.7	1506.0	N/A	N/A	1.122	N/A	N/A	UX1500MSE-AR
2	1.25	2119.5	1987.6	N/A	N/A	1.066	N/A	N/A	UX1600MSE-AR
3	3.08	2119.5	1907.1	N/A	N/A	1.111	N/A	N/A	UX1600MSE-AR
4	3.75	2119.5	1826.6	N/A	N/A	1.160	N/A	N/A	UX1600MSE-AR
5	5.58	2119.5	1746.1	N/A	N/A	1.214	N/A	N/A	UX1600MSE-AR
6	6.25	1689.7	1665.6	N/A	N/A	1.014	N/A	N/A	UX1500MSE-AR
7	8.08	1689.7	1585.1	N/A	N/A	1.066	N/A	N/A	UX1500MSE-AR
8	8.75	2119.5	1894.1	N/A	N/A	1.119	N/A	N/A	UX1600MSE-AR
9	11.25	2119.5	1764.7	N/A	N/A	1.201	N/A	N/A	UX1600MSE-AR
10	11.92	2119.5	1300.4	N/A	N/A	1.630	N/A	N/A	UX1600MSE-AR
11	13.75	2119.5	2061.8	N/A	N/A	1.028	N/A	N/A	UX1600MSE-AR
12	16.25	2119.5	2080.4	N/A	N/A	1.019	N/A	N/A	UX1600MSE-AR
13	18.75	2119.5	1758.4	N/A	N/A	1.205	N/A	N/A	UX1600MSE-AR
14	21.25	1689.7	1436.3	N/A	N/A	1.176	N/A	N/A	UX1500MSE-AR
15	23.75	1689.7	1114.3	N/A	N/A	1.516	N/A	N/A	UX1500MSE-AR
16	26.25	976.9	792.3	N/A	N/A	1.233	N/A	N/A	UX1400MSE-AR
17	28.75	976.9	470.2	N/A	N/A	2.078	N/A	N/A	UX1400MSE-AR

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	0.58	0.900	1506.0	N/A	20.69	0.31	71836.0	N/A	47.701	N/A	N/A	N/A
2	1.25	0.900	1987.6	N/A	20.34	0.66	68991.4	N/A	34.711	N/A	N/A	N/A
3	3.08	0.900	1907.1	N/A	19.36	1.64	61502.4	N/A	32.249	N/A	N/A	N/A
4	3.75	0.900	1826.6	N/A	19.01	1.99	58868.1	N/A	32.228	N/A	N/A	N/A
5	5.58	0.900	1746.1	N/A	18.03	2.97	51954.4	N/A	29.755	N/A	N/A	N/A
6	6.25	0.900	1665.6	N/A	17.68	3.32	49530.4	N/A	29.738	N/A	N/A	N/A
7	8.08	0.900	1585.1	N/A	16.70	4.30	43191.8	N/A	27.249	N/A	N/A	N/A
8	8.75	0.900	1894.1	N/A	16.35	4.65	40978.3	N/A	21.635	N/A	N/A	N/A
9	11.25	0.900	1764.7	N/A	15.02	5.98	33211.6	N/A	18.820	N/A	N/A	N/A
10	11.92	0.900	1300.4	N/A	14.66	6.34	31264.7	N/A	24.043	N/A	N/A	N/A
11	13.75	0.900	2061.8	N/A	13.69	7.31	26248.4	N/A	12.731	N/A	N/A	N/A
12	16.25	0.900	2080.4	N/A	12.36	8.64	20050.0	N/A	9.638	N/A	N/A	N/A
13	18.75	0.900	1758.4	N/A	11.03	9.97	14637.2	N/A	8.324	N/A	N/A	N/A
14	21.25	0.900	1436.3	N/A	9.70	11.30	10009.8	N/A	6.969	N/A	N/A	N/A
15	23.75	0.900	1114.3	N/A	8.37	12.63	6174.9	N/A	5.542	N/A	N/A	N/A
16	26.25	0.900	792.3	N/A	7.04	13.96	3115.8	N/A	3.933	N/A	N/A	N/A
17	28.75	0.900	470.2	N/A	5.71	15.29	842.2	N/A	1.791	N/A	N/A	N/A

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Appendix 2.2.1.2a

Problem 2 with Silty Sand

ARES® MSE Retaining Wall System

Back to main document

AASHTO 2017-2020
ARES Panel Wall System

MSEW+: Update # 2021.12

PROJECT IDENTIFICATION

Title: ARES Panel Wall System
Project Number: IDEA - Problem 2
Client: IDEA
Designer: JP
Station Number: ARES - AASHTO

Description:

ARES System - 75 yrs Design Life. Silty Sand backfill material.

Company's information:

Name: Tensar International Corporation
Street: 2500 Northwinds Parkway
Suite 500
Alpharetta, GA 30009
Telephone #: (770) 344-2090
Fax #: (770) 344-2084
E-Mail: Jorge.Peralta@cmc.com

File path and name: C:\WF\IDEA\ARES\Final\ARES Submittal Documents\Supporti.....
.....75yr(SiltySand).BENp

Original date and time of creating this file: May 2025

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using GEOGRID as reinforcing material.

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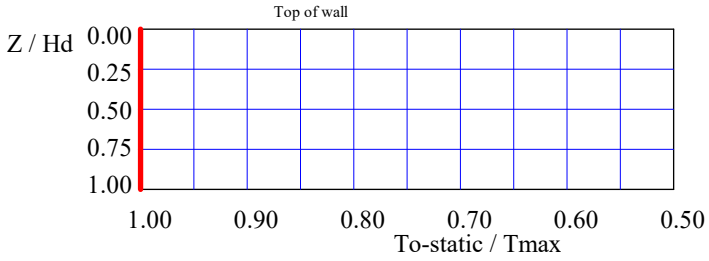
INPUT DATA: Facia and Connection (Analysis)

FACIA type: Segmental precast concrete panels.
Depth of panel is 0.49 ft. Horizontal distance to Center of Gravity of panel is 0.25 ft.
Average unit weight of panel is γ_f = 152.78 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

D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	UX1400MSE..	UX1500MSE..	UX1600MSE..	UX1700MSE..	N/A
Durability reduction factor, RFd	1.10	1.10	1.10	1.10	N/A
Creep reduction factor, RFc	2.77	2.77	2.77	2.77	N/A
Capacity Demand Ratio: connection break, CDR	N/A	N/A	N/A	N/A	N/A
Capacity Demand Ratio: connection pullout, CDR	N/A	N/A	N/A	N/A	N/A
CRu = Tult-connection/Tult-geogrid	0.93	0.93	0.93	0.93	N/A

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INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

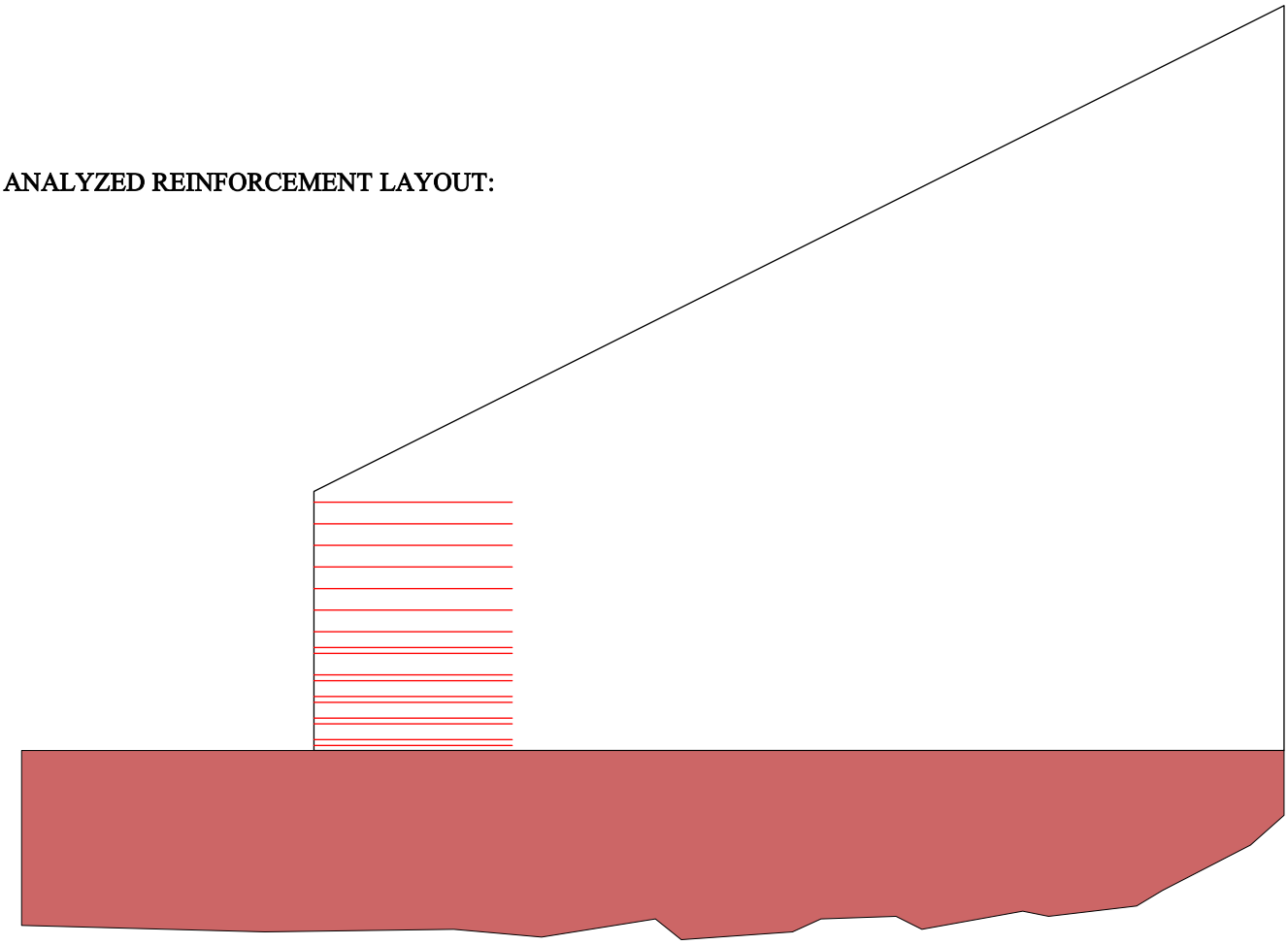
Design height, Hd 30.00 [ft] { Embedded depth is E = 0.00 ft, and height above top of finished bottom grade is H = 30.00 ft }

Soil in front of wall is Horizontal.

Batter, ω 0.0 [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:

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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	1.20

Resistance factor for reinforcement tension in connectors	ϕ	Static	Combined static/seismic
Geogrid:		0.90	1.20

Resistance factor for geosynthetic pullout	ϕ	0.90	1.20

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}):	$(\gamma_{p\text{-}EH})_{EQ}$	1.10
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	1.00	1.00
Reinforced Soil and Reinforcement	ϕ_τ	1.00	1.00

Resistance factor for bearing capacity of shallow foundation	Static	Combined Static/Seismic
ϕ_b	0.65	0.65

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DIRECT SLIDING for GIVEN LAYOUT (for GEOGRID reinforcements)

Along reinforced and foundation soils interface: CDR-static = 1.049

#	Geogrid Elevation [ft]	Geogrid Length [ft]	CDR Static	CDR Seismic	Geogrid Type #	Product name
1	0.58	23.00	1.015	N/A	2	UX1500MSE-AR
2	1.25	23.00	1.026	N/A	3	UX1600MSE-AR
3	3.08	23.00	1.059	N/A	3	UX1600MSE-AR
4	3.75	23.00	1.071	N/A	3	UX1600MSE-AR
5	5.58	23.00	1.107	N/A	3	UX1600MSE-AR
6	6.25	23.00	1.121	N/A	3	UX1600MSE-AR
7	8.08	23.00	1.161	N/A	2	UX1500MSE-AR
8	8.75	23.00	1.176	N/A	3	UX1600MSE-AR
9	11.25	23.00	1.239	N/A	3	UX1600MSE-AR
10	11.92	23.00	1.256	N/A	2	UX1500MSE-AR
11	13.75	23.00	1.308	N/A	3	UX1600MSE-AR
12	16.25	23.00	1.386	N/A	3	UX1600MSE-AR
13	18.75	23.00	1.474	N/A	3	UX1600MSE-AR
14	21.25	23.00	1.571	N/A	2	UX1500MSE-AR
15	23.75	23.00	1.677	N/A	2	UX1500MSE-AR
16	26.25	23.00	1.784	N/A	2	UX1500MSE-AR
17	28.75	23.00	1.870	N/A	1	UX1400MSE-AR

ECCENTRICITY for GIVEN LAYOUT

At interface with foundation: e/L static = 0.2093; Overturning: CDR-static = 1.88

#	Geogrid Elevation [ft]	Geogrid Length [ft]	e / L Static	e / L Seismic	Geogrid Type #	Product name
1	0.58	23.00	0.2021	N/A	2	UX1500MSE-AR
2	1.25	23.00	0.1940	N/A	3	UX1600MSE-AR
3	3.08	23.00	0.1721	N/A	3	UX1600MSE-AR
4	3.75	23.00	0.1642	N/A	3	UX1600MSE-AR
5	5.58	23.00	0.1432	N/A	3	UX1600MSE-AR
6	6.25	23.00	0.1356	N/A	3	UX1600MSE-AR
7	8.08	23.00	0.1154	N/A	2	UX1500MSE-AR
8	8.75	23.00	0.1082	N/A	3	UX1600MSE-AR
9	11.25	23.00	0.0817	N/A	3	UX1600MSE-AR
10	11.92	23.00	0.0748	N/A	2	UX1500MSE-AR
11	13.75	23.00	0.0562	N/A	3	UX1600MSE-AR
12	16.25	23.00	0.0314	N/A	3	UX1600MSE-AR
13	18.75	23.00	0.0068	N/A	3	UX1600MSE-AR
14	21.25	23.00	-0.0184	N/A	2	UX1500MSE-AR
15	23.75	23.00	-0.0455	N/A	2	UX1500MSE-AR
16	26.25	23.00	-0.0774	N/A	2	UX1500MSE-AR
17	28.75	23.00	-0.1213	N/A	1	UX1400MSE-AR

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Appendix 2.2.1.2b

Problem 2 with Sandy Gravel

ARES® MSE Retaining Wall System

Back to main document

AASHTO 2017-2020
ARES Panel Wall System

MSEW+: Update # 2021.12

PROJECT IDENTIFICATION

Title: ARES Panel Wall System
Project Number: IDEA - Problem 2
Client: IDEA
Designer: JP
Station Number: ARES - AASHTO

Description:

ARES System - 75 yrs Design Life. Sandy Gravel backfill material.

Company's information:

Name: Tensar International Corporation
Street: 2500 Northwinds Parkway
Suite 500
Alpharetta, GA 30009
Telephone #: (770) 344-2090
Fax #: (770) 344-2084
E-Mail: Jorge.Peralta@cmc.com

File path and name: C:\WF\IDEA\ARES\Final\ARES Submittal Documents\Supporti.....
.....yr(SandyGravel).BENp

Original date and time of creating this file: May 2025

PROGRAM MODE: ANALYSIS
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using GEOGRID as reinforcing material.

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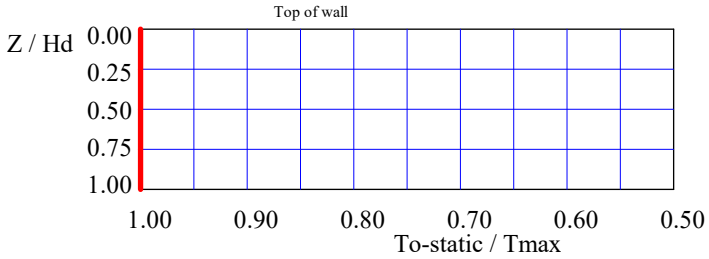
INPUT DATA: Facia and Connection (Analysis)

FACIA type: Segmental precast concrete panels.
Depth of panel is 0.49 ft. Horizontal distance to Center of Gravity of panel is 0.25 ft.
Average unit weight of panel is γ_f = 152.78 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

D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	UX1400MSE..	UX1500MSE..	UX1600MSE..	UX1700MSE..	N/A
Durability reduction factor, RFd	1.10	1.10	1.10	1.10	N/A
Creep reduction factor, RFc	2.77	2.77	2.77	2.77	N/A
Capacity Demand Ratio: connection break, CDR	N/A	N/A	N/A	N/A	N/A
Capacity Demand Ratio: connection pullout, CDR	N/A	N/A	N/A	N/A	N/A
CRu = Tult-connection/Tult-geogrid	0.93	0.93	0.93	0.93	N/A

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INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

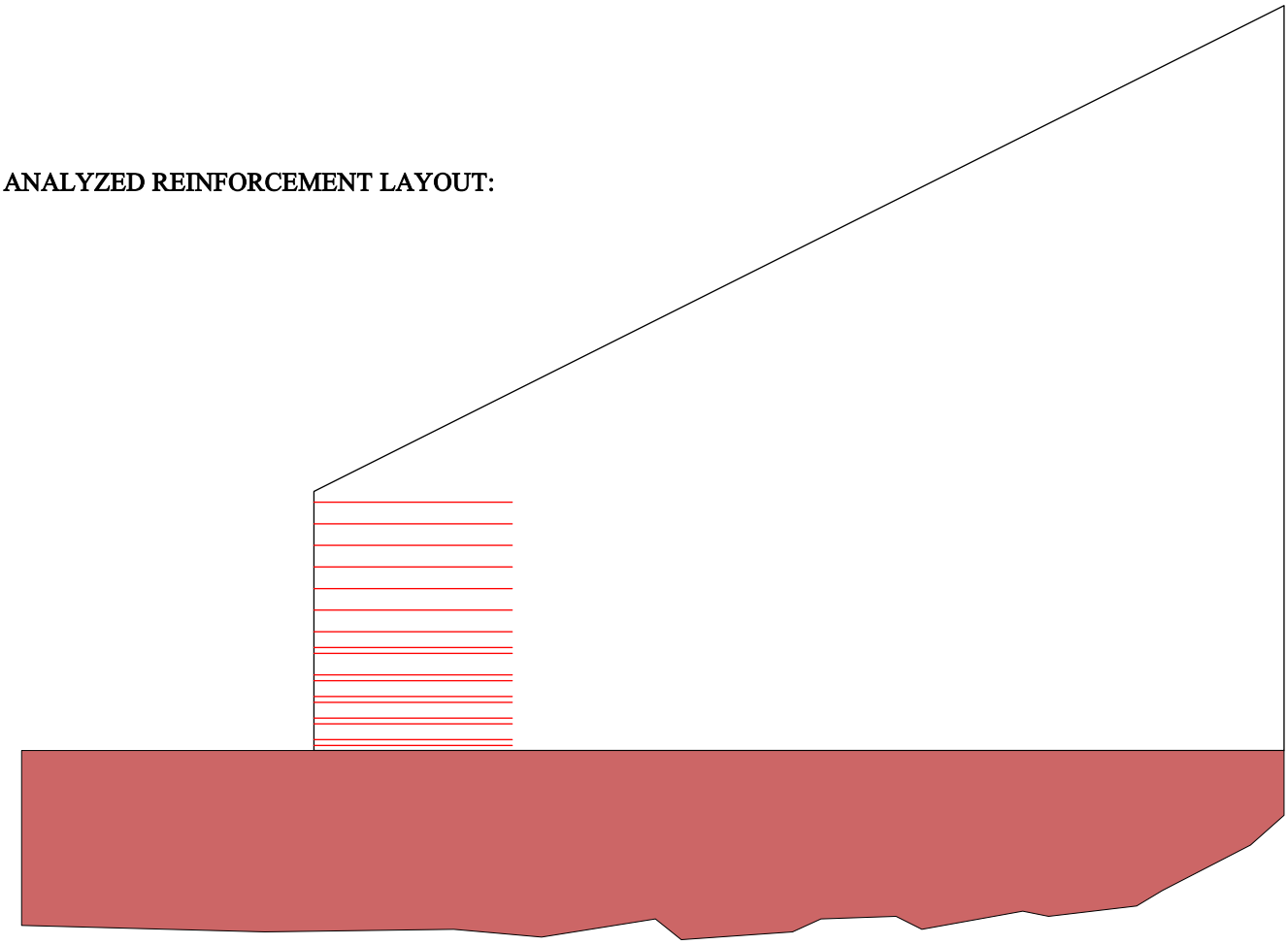
Design height, Hd 30.00 [ft] { Embedded depth is E = 0.00 ft, and height above top of finished bottom grade is H = 30.00 ft }

Soil in front of wall is Horizontal.

Batter, ω 0.0 [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:

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DIRECT SLIDING for GIVEN LAYOUT (for GEOGRID reinforcements)

Along reinforced and foundation soils interface: CDR-static = 1.049

#	Geogrid Elevation [ft]	Geogrid Length [ft]	CDR Static	CDR Seismic	Geogrid Type #	Product name
1	0.58	23.00	1.015	N/A	2	UX1500MSE-AR
2	1.25	23.00	1.026	N/A	4	UX1700MSE-AR
3	3.08	23.00	1.059	N/A	3	UX1600MSE-AR
4	3.75	23.00	1.071	N/A	3	UX1600MSE-AR
5	5.58	23.00	1.107	N/A	3	UX1600MSE-AR
6	6.25	23.00	1.121	N/A	3	UX1600MSE-AR
7	8.08	23.00	1.161	N/A	3	UX1600MSE-AR
8	8.75	23.00	1.176	N/A	4	UX1700MSE-AR
9	11.25	23.00	1.239	N/A	3	UX1600MSE-AR
10	11.92	23.00	1.256	N/A	2	UX1500MSE-AR
11	13.75	23.00	1.308	N/A	4	UX1700MSE-AR
12	16.25	23.00	1.386	N/A	4	UX1700MSE-AR
13	18.75	23.00	1.474	N/A	4	UX1700MSE-AR
14	21.25	23.00	1.571	N/A	3	UX1600MSE-AR
15	23.75	23.00	1.677	N/A	2	UX1500MSE-AR
16	26.25	23.00	1.784	N/A	2	UX1500MSE-AR
17	28.75	23.00	1.870	N/A	1	UX1400MSE-AR

ECCENTRICITY for GIVEN LAYOUT

At interface with foundation: e/L static = 0.2093; Overturning: CDR-static = 1.88

#	Geogrid Elevation [ft]	Geogrid Length [ft]	e / L Static	e / L Seismic	Geogrid Type #	Product name
1	0.58	23.00	0.2021	N/A	2	UX1500MSE-AR
2	1.25	23.00	0.1940	N/A	4	UX1700MSE-AR
3	3.08	23.00	0.1721	N/A	3	UX1600MSE-AR
4	3.75	23.00	0.1642	N/A	3	UX1600MSE-AR
5	5.58	23.00	0.1432	N/A	3	UX1600MSE-AR
6	6.25	23.00	0.1356	N/A	3	UX1600MSE-AR
7	8.08	23.00	0.1154	N/A	3	UX1600MSE-AR
8	8.75	23.00	0.1082	N/A	4	UX1700MSE-AR
9	11.25	23.00	0.0817	N/A	3	UX1600MSE-AR
10	11.92	23.00	0.0748	N/A	2	UX1500MSE-AR
11	13.75	23.00	0.0562	N/A	4	UX1700MSE-AR
12	16.25	23.00	0.0314	N/A	4	UX1700MSE-AR
13	18.75	23.00	0.0068	N/A	4	UX1700MSE-AR
14	21.25	23.00	-0.0184	N/A	3	UX1600MSE-AR
15	23.75	23.00	-0.0455	N/A	2	UX1500MSE-AR
16	26.25	23.00	-0.0774	N/A	2	UX1500MSE-AR
17	28.75	23.00	-0.1213	N/A	1	UX1400MSE-AR

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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	0.58	1689.7	1640.2	N/A	N/A	1.030	N/A	N/A	UX1500MSE-AR
2	1.25	2500.7	2171.1	N/A	N/A	1.152	N/A	N/A	UX1700MSE-AR
3	3.08	2119.5	2090.5	N/A	N/A	1.014	N/A	N/A	UX1600MSE-AR
4	3.75	2119.5	2010.0	N/A	N/A	1.054	N/A	N/A	UX1600MSE-AR
5	5.58	2119.5	1929.5	N/A	N/A	1.098	N/A	N/A	UX1600MSE-AR
6	6.25	2119.5	1849.0	N/A	N/A	1.146	N/A	N/A	UX1600MSE-AR
7	8.08	2119.5	1768.5	N/A	N/A	1.198	N/A	N/A	UX1600MSE-AR
8	8.75	2500.7	2126.7	N/A	N/A	1.176	N/A	N/A	UX1700MSE-AR
9	11.25	2119.5	1997.3	N/A	N/A	1.061	N/A	N/A	UX1600MSE-AR
10	11.92	1689.7	1483.8	N/A	N/A	1.139	N/A	N/A	UX1500MSE-AR
11	13.75	2500.7	2379.5	N/A	N/A	1.051	N/A	N/A	UX1700MSE-AR
12	16.25	2500.7	2447.3	N/A	N/A	1.022	N/A	N/A	UX1700MSE-AR
13	18.75	2500.7	2125.2	N/A	N/A	1.177	N/A	N/A	UX1700MSE-AR
14	21.25	2119.5	1803.2	N/A	N/A	1.175	N/A	N/A	UX1600MSE-AR
15	23.75	1689.7	1481.2	N/A	N/A	1.141	N/A	N/A	UX1500MSE-AR
16	26.25	1689.7	1159.2	N/A	N/A	1.458	N/A	N/A	UX1500MSE-AR
17	28.75	976.9	837.1	N/A	N/A	1.167	N/A	N/A	UX1400MSE-AR

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	0.58	0.900	1640.2	N/A	22.69	0.31	94364.3	N/A	57.531	N/A	N/A	N/A
2	1.25	0.900	2171.1	N/A	22.34	0.66	91351.5	N/A	42.077	N/A	N/A	N/A
3	3.08	0.900	2090.5	N/A	21.36	1.64	83362.6	N/A	39.876	N/A	N/A	N/A
4	3.75	0.900	2010.0	N/A	21.01	1.99	80532.2	N/A	40.065	N/A	N/A	N/A
5	5.58	0.900	1929.5	N/A	20.03	2.97	73042.1	N/A	37.855	N/A	N/A	N/A
6	6.25	0.900	1849.0	N/A	19.68	3.32	70394.2	N/A	38.071	N/A	N/A	N/A
7	8.08	0.900	1768.5	N/A	18.70	4.30	63440.0	N/A	35.872	N/A	N/A	N/A
8	8.75	0.900	2126.7	N/A	18.35	4.65	60973.7	N/A	28.670	N/A	N/A	N/A
9	11.25	0.900	1997.3	N/A	17.02	5.98	52195.5	N/A	26.133	N/A	N/A	N/A
10	11.92	0.900	1483.8	N/A	16.66	6.34	49960.6	N/A	33.670	N/A	N/A	N/A
11	13.75	0.900	2379.5	N/A	15.69	7.31	44098.6	N/A	18.532	N/A	N/A	N/A
12	16.25	0.900	2447.3	N/A	14.36	8.64	36683.0	N/A	14.989	N/A	N/A	N/A
13	18.75	0.900	2125.2	N/A	13.03	9.97	29948.6	N/A	14.092	N/A	N/A	N/A
14	21.25	0.900	1803.2	N/A	11.70	11.30	23917.9	N/A	13.264	N/A	N/A	N/A
15	23.75	0.900	1481.2	N/A	10.37	12.63	18543.3	N/A	12.519	N/A	N/A	N/A
16	26.25	0.900	1159.2	N/A	9.04	13.96	13850.0	N/A	11.948	N/A	N/A	N/A
17	28.75	0.900	837.1	N/A	7.71	15.29	9838.0	N/A	11.752	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 break, CRu	Reduction factor for connection pullout, CRs	Available connection strength, Tc-break criterion [lb/ft]	Available connection strength, Tc-pullout criterion [lb/ft]	Available Geogrid strength, Tavailable [lb/ft]	CDR connection break		CDR connection pullout		CDR Geogrid strength		Product name
								Specified	Actual	Specified	Actual	Specified	Actual	
1	0.58	1640	0.93	N/A	2149	N/A	2086	N/A	1.29	N/A	N/A	N/A		UX1500MSE-AR
2	1.25	2171	0.93	N/A	3299	N/A	3087	N/A	1.49	N/A	N/A	N/A		UX1700MSE-AR
3	3.08	2091	0.93	N/A	2715	N/A	2617	N/A	1.27	N/A	N/A	N/A		UX1600MSE-AR
4	3.75	2010	0.93	N/A	2715	N/A	2617	N/A	1.33	N/A	N/A	N/A		UX1600MSE-AR
5	5.58	1930	0.93	N/A	2715	N/A	2617	N/A	1.38	N/A	N/A	N/A		UX1600MSE-AR
6	6.25	1849	0.93	N/A	2715	N/A	2617	N/A	1.44	N/A	N/A	N/A		UX1600MSE-AR
7	8.08	1769	0.93	N/A	2715	N/A	2617	N/A	1.51	N/A	N/A	N/A		UX1600MSE-AR
8	8.75	2127	0.93	N/A	3299	N/A	3087	N/A	1.52	N/A	N/A	N/A		UX1700MSE-AR
9	11.25	1997	0.93	N/A	2715	N/A	2617	N/A	1.33	N/A	N/A	N/A		UX1600MSE-AR
10	11.92	1484	0.93	N/A	2149	N/A	2086	N/A	1.42	N/A	N/A	N/A		UX1500MSE-AR
11	13.75	2380	0.93	N/A	3299	N/A	3087	N/A	1.36	N/A	N/A	N/A		UX1700MSE-AR
12	16.25	2447	0.93	N/A	3299	N/A	3087	N/A	1.32	N/A	N/A	N/A		UX1700MSE-AR
13	18.75	2125	0.93	N/A	3299	N/A	3087	N/A	1.52	N/A	N/A	N/A		UX1700MSE-AR
14	21.25	1803	0.93	N/A	2715	N/A	2617	N/A	1.48	N/A	N/A	N/A		UX1600MSE-AR
15	23.75	1481	0.93	N/A	2149	N/A	2086	N/A	1.42	N/A	N/A	N/A		UX1500MSE-AR
16	26.25	1159	0.93	N/A	2149	N/A	2086	N/A	1.82	N/A	N/A	N/A		UX1500MSE-AR
17	28.75	837	0.93	N/A	1321	N/A	1206	N/A	1.47	N/A	N/A	N/A		UX1400MSE-AR

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Appendix 3.1.2

ARES Installation Guide and Construction Manual

ARES® MSE Retaining Wall System

Back to main document

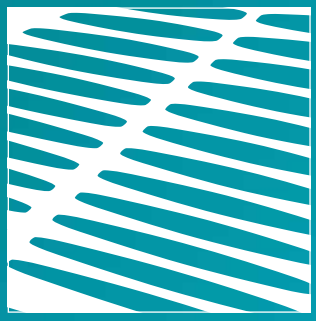
ARES®
RETAINING WALL SYSTEMS

ARES®
RETAINING WALL SYSTEMS

INSTALLATION GUIDE AND CONSTRUCTION MANUAL

Tensar®

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



Tensar® Geogrids

The ARES Retaining Wall 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**.

ARES® Retaining Wall Systems

DOTs, Contractors and Engineers have long appreciated the many advantages of panel walls. Their wide range of appearances and finishes, combined with the simplicity and speed of construction, make them attractive when compared to other types of wall systems. Unfortunately, limitations imposed by the behavior of reinforcing materials and a very narrow and expensive range of acceptable backfill materials have restricted their use until the introduction of Geogrids manufactured by Tensar International Corporation (Tensar). By mechanically connecting Tensar Geogrids with the advantages of panels, the fully integrated ARES® Retaining Wall Systems now offer a high-performance, cost-effective and aesthetically pleasing solution.

NO METAL – NO CORROSION

With soil reinforcement that is 100% polymeric, ARES Retaining Wall Systems are proven concrete panel wall solutions that eliminate corrosion concerns of soil reinforcement. ARES Systems offer cost advantages over conventional panel walls while eliminating the risks associated with the long term exposure to chlorides, sulfates, low-resistivity soils or stray electric current potential. This makes the system the logical choice for "hot" backfill soils, transformer platform areas and electrified rail systems.

As testimony to the durability of the ARES Systems, one of the first Tensar-reinforced panel walls was built as a seawall on the Gaspe Peninsula in Canada. After 30 years of North Atlantic storms and constant exposure to salt water, there are no signs of deterioration of the soil reinforcement. In fact, some of the first ARES installations were instrumented and carefully observed to verify the effectiveness and long-term performance of the systems. As part of an FHWA study at the Tanque Verde project in Arizona, the Tensar Geogrid behind sections of one such ARES wall was excavated to validate its durability. Thirty years after the original installation, the walls continue to perform as designed with no maintenance issues.

PURPOSE OF THIS DOCUMENT

This document is intended to provide the Owner, Engineer, Contractor and the Inspector with the guidelines and criteria required to facilitate construction and quality control of the ARES Precast Panel Retaining Wall System.

ARES® Systems' Components

COMPONENT	FUNCTION
Tensar Geogrids	High-density polyethylene (HDPE) structural geogrids internally reinforce the fill materials. Inert to chemical degradation, they can be used with different backfill materials, even crushed 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 3.0 m) or can be customized for full height construction.
Bodkin Connector	HDPE Connector for high connection efficiency without the concern for corrosion.
Full Engineering and Construction Services	Detailing, design, site assistance and stamped drawings for each ARES project upon request.

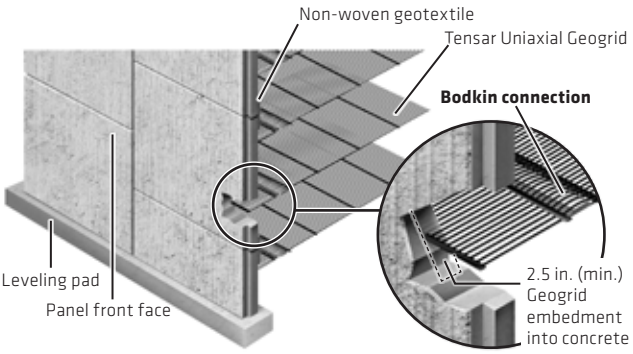


FIGURE 1: *ARES® Retaining Wall System – incremental 5 ft x 5 ft panel*

Wall Component Definitions

The following are standard terms that will be used for the ARES® Retaining Wall Systems. Refer to Figures 1 and 2 for a typical cross-section and the associated terms.

- **Bearing Pads** – Wall panel spacers are typically ribbed elastomeric or polymeric pads. They are inserted at the horizontal joint between panels to help provide the proper spacing. Proper spacing keeps the panels from having point contact and spalling the concrete.
- **Bodkin Connection** – This is where the connection is made between the wall facing panel and the soil reinforcement.
- **Concrete Leveling Pad** – The leveling pad is unreinforced cast-in-place concrete for precast panel facing to sit on a level foundation.
- **Coping** – The coping is used to tie in the top of the wall panels and to provide a pleasing finish to the wall top. It is cast-in-place or precast.
- **Filter Fabric** – Typically a non-woven geotextile fabric is used to cover the joint between panels. It is placed on the backside of the panel joints. This keeps the soil from piping through the joints and allows excess water to flow out.
- **Random Backfill (retained)** – Random backfill is the backfill that is retained.
- **Select Backfill** – Select granular backfill within the reinforced mass that meets the gradation, unit weight, internal friction angle and any other requirements.
- **Temporary Wooden Wedges** – Are used to help hold the panels at the correct batter during the backfilling operation.
- **Tensar Structural Geogrid** – Soil reinforcement that holds the wall facing panels in position and provides reinforcement for the soil.
- **Wall Facing Panel** – Wall facing panels are used to hold the soil in position at the face of the wall. The panels are made of precast concrete.

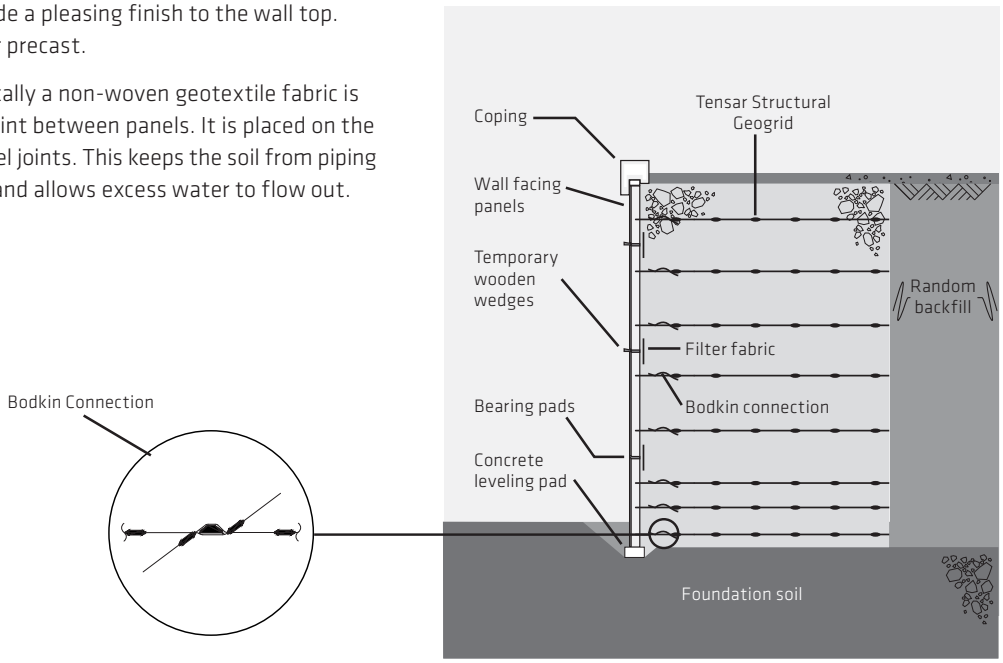


FIGURE 2: *Typical Cross Section*

MSE Wall Construction Best Practices

- Confirm receipt of the Tensar approved construction drawings.
- Confirm backfill material has been tested and approved before it is brought to the job site.
- Review the approved construction drawings.
- Ensure the Contractor's field supervisor has a copy of the approved construction drawings as well as Tensar Installation Guide and is familiar with them.
- Confirm the foundation soils are in accordance with project specifications.
- Confirm the leveling pad elevations, alignment and step locations prior to pouring the concrete.
- Notify the Tensar project manager of the expected start date for panel installation.
- Ensure panels, geogrid and accessories are properly stored to prevent damage.
- Inspect geogrid, accessories and panels for damage. Notify Tensar of any materials that are not in compliance with the plans and specifications.
- Install panels in accordance with the plans and specifications.
- Use corner panels at all corners. If corner panels are not indicated on the Tensar approved construction drawings, Tensar should be notified.
- Ensure wedges are installed on each course of panels. Use hardwood wedges.
- Check the batter of the panels daily (at a minimum) and adjust the initial batter accordingly. The vertical alignment of the panels below the panels being installed may be affected by the compaction of the soil behind those panels.
- When installing the filter fabric to the panel apply the adhesive to the panel and then apply the fabric.
- Place and compact fill in accordance with the plans and specifications. If fill lift thickness is not included in the plans and specifications, do not exceed fill lifts thicker than 10 in. (250 mm) loose. Thick lifts may cause the panels to move out of alignment.
- Ensure the geogrid reinforcement can be installed around all obstructions without skewing the geogrid more than 15 degrees from normal. Notify Tensar of any obstructions not shown on the Tensar approved construction drawings.

ARES® modular panels provide significant face area while the Tensar® Geogrid reinforcement is lightweight and easy to field connect.

Responsibilities for Construction Compliance

- The Contractor is responsible to provide construction in accordance with the contract documents and to coordinate the wall construction with related work.
- The Contractor is responsible for using the most recent set of approved construction drawings to perform the work and for verifying line, grade and offset needed to establish wall location according to the contract documents.
- The Contractor is responsible for monitoring material supply and ensuring that adequate lead time is provided with each request for delivery and that ordered quantities are available to prevent construction delays.
- The Contractor is responsible for unloading and inspecting materials upon delivery to the job site, and to provide proper storage and protection of materials.
- The Engineer is responsible for enforcing the requirements of the contract documents and the approved construction drawings.
- The Tensar technical advisor will be available at the start of the project to advise the Contractor's project team of the recommended construction procedures within the scope of this manual. The Tensar technical advisor is not a member of the inspection or quality control staff on the project.

Work provided by the Contractor includes:

- All wall site preparation and survey layout
- Forming and pouring the leveling pad
- Wall construction in its entirety according to approved construction drawings
- Installation of the top-of-wall treatment where required

If requested, services provided by Tensar include:

- Wall construction drawings
- On-site technical assistance at the start of construction

Materials

TYPICAL MATERIALS SUPPLIED BY TENSAR

- Precast concrete facing panels
- HDPE Tensar® Uniaxial (UX) Geogrid
- Filter fabric
- Bearing pads
- Bodkin bars

Materials supplied by Tensar are typically delivered in full truck load quantities. Off-loading is scheduled by the Contractor.

Any damage to the materials or discrepancies in quantities must be noted by the Contractor on the delivery ticket at the time of delivery and reported promptly to Tensar. The materials must be properly stored in such a manner and location to avoid damage or theft.

HDPE Tensar Uniaxial (UX) Geogrids can be color-coded to differentiate roll types.

ARES® panels typically arrive at the job site on flatbed trucks.

Palletized component materials upon arrival at site.

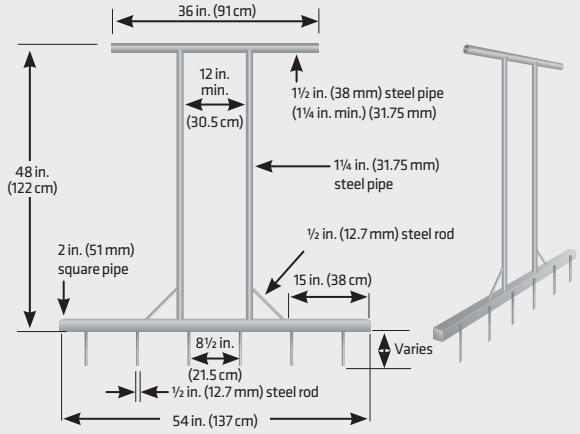


FIGURE 3: *Sample configuration of steel rake for tensioning geogrid.*

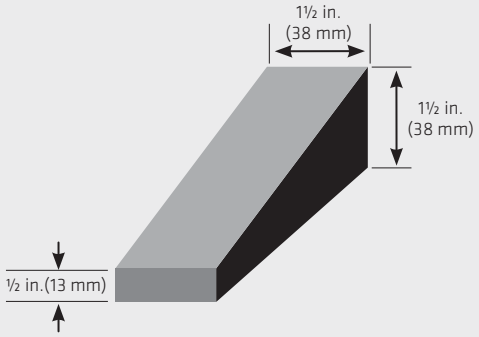


FIGURE 4: *Temporary Wooden Wedge, two per horizontal joint required on 5 x 5 and four per horizontal joint required on 5 x 9 and 5 x 10.*

Materials (continued)

MATERIALS AND TOOLS PROVIDED BY THE CONTRACTOR:

- Nylon slings for unloading panels
- Equal length cables with shackles to connect the lifting devices
- Devices for lifting panels by the embedded lifting inserts
- Rake for tensioning geogrid (Figure 3)
- Lumbers for bracing, staking, and fabrication of clamps, as well as threaded rod, washers and nuts
- Hardwood wedges (Figure 4)
- Standard and Header Clamps (Figures 5 and 6)
- ¾ in. (19 mm) plywood spacer to set panel vertical joints gap
- Spray paint for marking geogrid and panels
- Standard grade construction adhesive such as Liquid Nail or 3M 77 to attach the filter fabric to the panels at the joints
- Crowbars, 4 ft (1.2 m) long
- Wrenches for clamp bolts
- Sledge and Claw hammers
- Broom for sweeping the leveling pad
- Sharp blade or scissors to cut the filter fabric
- 4 ft (1.2 m) level
- Chalk line
- Plumb bob
- Concrete, steel, and forming materials for leveling pad and top of the wall treatment as required
- Backfill materials

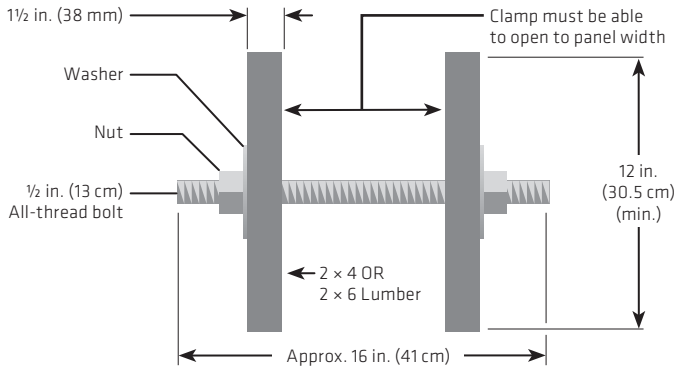


FIGURE 5:
Standard Clamp: *One per vertical joint*

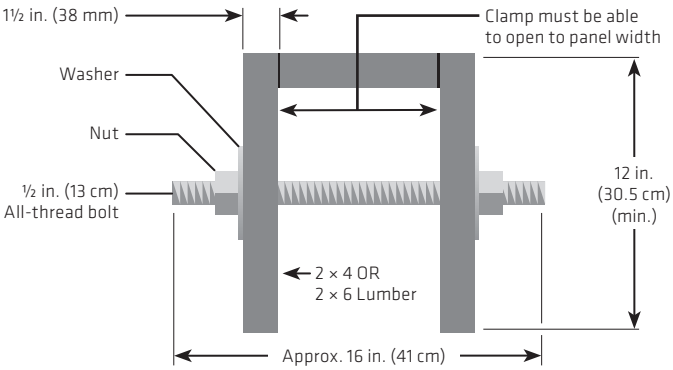


FIGURE 6:
Bracing Header Clamp: *For initial course only*

Materials (continued)

EQUIPMENT PROVIDED BY CONTRACTOR

- Panel handling and setting equipment – excavator, loader or similar equipment capable of properly lifting and placing the precast concrete facing panels.

  NOTE: *Typical weight of largest standard panel type is approximately 3,000 lbs (1,360 kg) for projects using nominal 5 ft (1.5 m) wide, 5.5 in. (140 mm) thick panels, approximately 5,500 lbs (2,495 kg) for projects using nominal 9 ft (2.75 m) wide, 5.5 in. (140 mm) thick panels, and approximately 6,000 lbs (2,720 kg) for 10 ft (3 m) wide, 5.5 in. (140 mm) thick panels.*

- Equipment to transport the select fill to the wall site.
- Equipment, such as a rubber-tired loader and small track dozer, preferably with angle blade, for placing and spreading the select fill.
- Large, smooth-drum roller for mass compaction
- Small, hand-operated, vibratory plate tamper or roller for compaction within 3 ft (0.9 m) of the back face of wall panel.

  NOTE: *Use of Jumping Jack is not recommended.*

IMAGE A:
ARES® panels should be stacked on a level, stable surface provided by the Contractor. Dunnage shall be carefully selected to allow panel separation and placed to avoid panel cracking.

Handling Materials Supplied by Tensar

All materials supplied by Tensar shall be properly stored in a secure location to prevent damage or theft.

PRECAST CONCRETE FACING PANELS

- It is the responsibility of the Contractor to schedule delivery of the panels in accordance with the installation schedule. Proper coordination will help avoid delays by having the precasting performed in harmony with the wall construction schedule.
- ARES® panels are usually delivered on flatbed trailers. The Contractor must provide a level, stable area to unload and stage panels. The acceptability of this access is at the discretion of the driver or his employer. The Contractor is allowed one hour to unload each truck, unless specifically agreed otherwise in writing with Tensar.
- The Contractor must take care to protect the panels from staining due to rain splash or damage due to improper placement of the dunnage. The number of panels in a stack shall not exceed five 5 x 5 panels, four 5 x 9 panels or four 5 x 10 panels as shown in Image A.
- The dunnage shall be properly spaced to avoid uneven loading in the panel stacks. (See Figure 7.) **All dunnages are the property of Tensar or its precaster and should be stacked by the Contractor for loading on a subsequent panel delivery truck.**
- Delivery tickets are included with each shipment and indicate the panel types furnished in that load. It is the responsibility of the Contractor to confirm the accuracy of the tickets and to note any damage that is visible prior to accepting delivery. Tensar or its precaster must be notified immediately if any panels have been damaged.

BEARING PADS FOR HORIZONTAL JOINTS BETWEEN PANELS

- Bearing pads will be delivered in cardboard cartons.
- The quantity of these cartons shall be noted on the delivery ticket and confirmed by the Contractor.

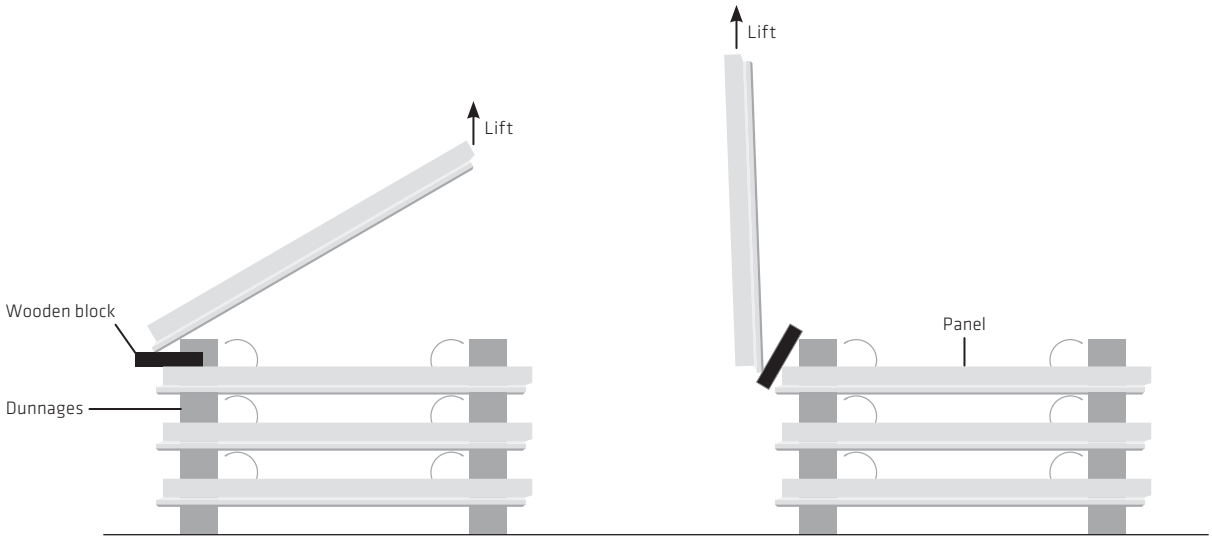


FIGURE 7: *Suggested stacking and lifting procedures for ARES segmental panels.*

Color-code the geogrid tabs on each panel to correspond with the geogrid rolls.

GEOGRID REINFORCEMENT

- Geogrid reinforcement shall be delivered in rolls and shall be labeled by type. These labels must be protected until the geogrid has been color-coded at the job site. The Contractor should retain any certifications included with the packing slip for the Engineer.
- The Contractor should immediately color code each of the geogrid types using spray paint on the edges and ends of the rolls (Images B and C). The Contractor may choose to highlight geogrid types on the approved construction drawings using corresponding colors.
- The Contractor is responsible for cutting the geogrid to length in the field. Precut geogrid should then be tagged for length and type. The first transverse bar (at the connection) on each section of geogrid should be trimmed neatly to expedite making the connection of the geogrid to the wall facing. Do not cut into the transverse bar of the geogrid (Figure 8 and Image D).

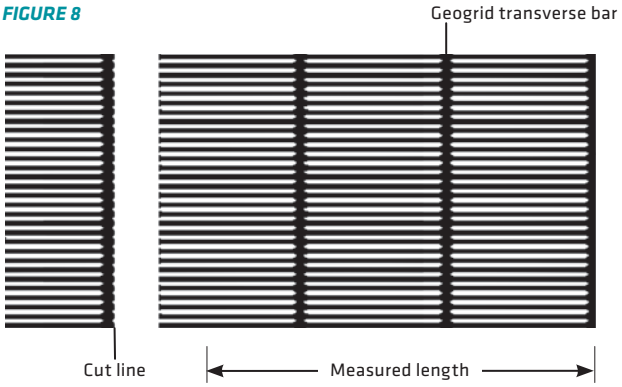
IMAGE B: *Uniaxial geogrid rolls should be color-coded prior to removing roll labels.*

IMAGE C: *Color coding of geogrid panels allows for quick, accurate identification even after labels are removed.*

FILTER FABRIC

- The filter fabric will be delivered in rolls and must be covered to protect it from direct sunlight.

FIGURE 8



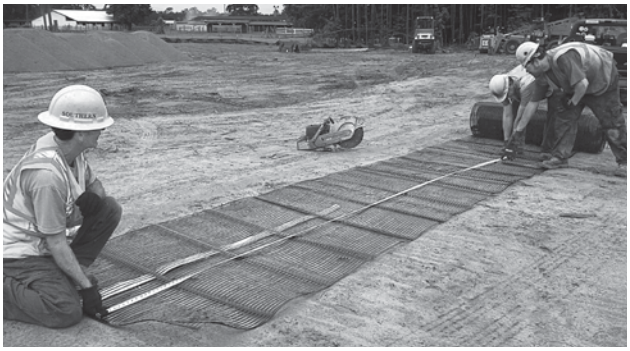


IMAGE D: *Safe cutting of geogrid may be accomplished with a variety of cutting tools. Care should be taken to avoid cutting the transverse bar.*

As with any wall system, the foundation is crucial to the wall's performance.

Construction Procedures for ARES® Retaining Wall Systems

EXCAVATION, FOUNDATION AND DRAINAGE

- Excavation will be performed to the lines and grade required for the installation of the entire wall system.
- The Contractor will be responsible for supporting the wall excavation. All work to support the excavation or to fill the void behind the wall will be the responsibility of the Contractor.
- Evaluation and approval of foundation suitability is the responsibility of the Engineer. Any foundation soils found to be unsuitable by the Engineer shall be removed and replaced with material suitable to the Engineer. The material shall be compacted to the density necessary to obtain the bearing pressure required by the Contract Documents, including the Project Plans and Specifications.
- Foundation shall be prepared according to contract documents and project specifications. The foundation is crucial to the performance of any panel wall system.
- The wall drainage system shall be installed as required in the Contract Documents.

CAST-IN-PLACE LEVELING PAD

- Once the foundation is prepared and approved by the Engineer, an unreinforced concrete pad is constructed. The purpose of this pad is to serve as a guide for the wall panel construction. This leveling pad is not intended for 'significant' structural foundation support in the final configuration of the wall. There is significant construction panel loading on the leveling pad, and it must be properly constructed and on a firm foundation in order to minimize potential wall movement during the construction of the wall.
- The leveling pad is important to the overall construction of the wall and the horizontal and vertical alignment of the wall. It must be in the correct horizontal position, level and at the correct grade.
- Unless otherwise shown in the contract documents or approved in writing by the Engineer, the leveling pad shall consist of 6 in. (150 mm) thick by 12 in. (300 mm) wide unreinforced concrete which shall be formed and poured in place. The concrete strength shall be in accordance with the contract documents or a minimum of 2,500 psi (17MPa), whichever is greater. The leveling pad must cure a minimum of 12 hours prior to the placement of the panels.

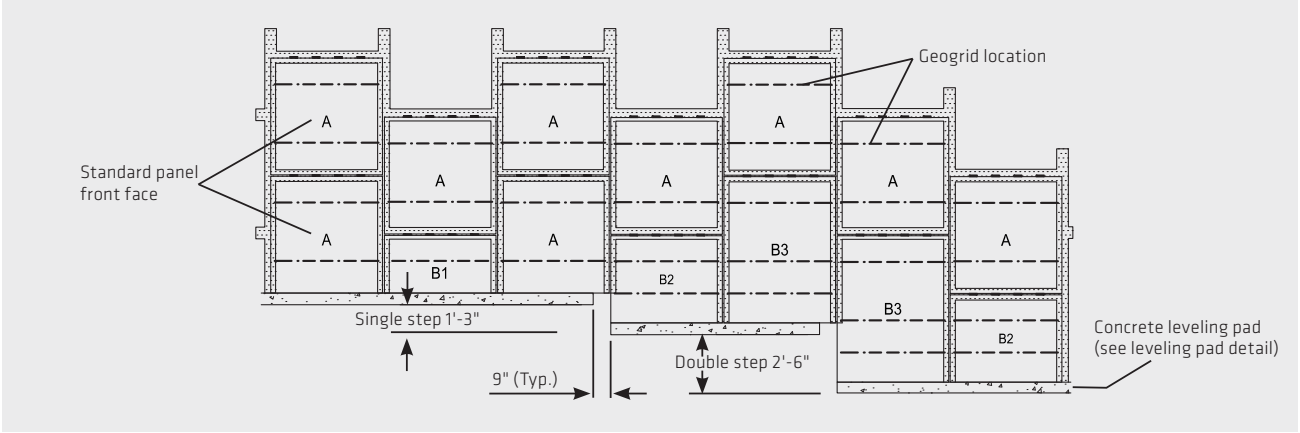


FIGURE 9: *Example of leveling pad and typical panels configuration.*

IMAGE E: *Panel placement commonly begins at corner locations. Placement and bracing of concrete panels.*

- The leveling pad shall have formed sides and a smooth, level surface set to the grades as shown on the approved construction drawings and shall be finished such that the elevation variance is less than +⅛ in. (3 mm) or -¼ in. (6 mm). A leveling pad that is not placed accurately will create problems with wall alignment and joint spacing during the construction process.
- Where steps in the leveling pad are shown on the approved construction drawings, the actual location of each step should be located and the bulkhead for the upper leveling pad step set back 9 in. (225 mm), nominal. (See Figure 9 and Image G.)
- After the concrete has cured and the forms are removed, lay out the front face of the wall and establish the location on the leveling pad by striking a chalk line at the front face of the bottom course of panels. (See Image F.) For panels with an architectural finish, the location and chalk line should be based on the face of the structural, non-architectural portion of the panel, i.e. immediately behind the architectural relief.
- Do not allow any overhanging of the panels off the leveling pad. If this happens, stop the construction and investigate the problem.

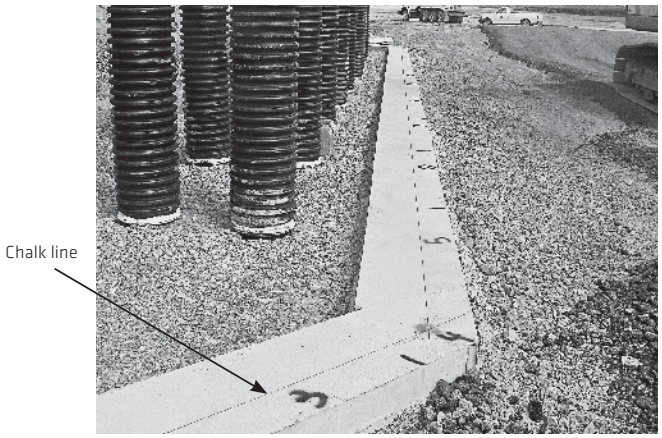


IMAGE F: *Recommended location of chalk line on leveling pad.*

PANEL PLACEMENT – BOTTOM COURSE

- At this point, the foundation has been constructed, drainage has been added, the materials have been checked and the leveling pad has been constructed. Shop drawings must be checked to ensure that the correct panels are being used in the correct location along the wall.
- It is generally preferable to start a wall at the lowest leveling pad elevation and at the location of any fixed point such as a corner and/or existing structure. (See Image E.)
- The bottom course is made up of alternating tall panels (A or B3) and half panels (B1 or B2), with the tall A panels above the half panels as shown in Figure 9.
- The alignment of the first course of panels will determine to a large degree the resulting appearance of the wall. Considerable attention must be paid to the setting and positioning of these panels.
- Remove the panels from the stack using proper lifting devices. Wood blocking must be placed under the bottom of the panel prior to lifting. (See Figure 7.) This protects the face of the panel being lifted from being scarred by the lower panel.
- Prior to setting any panel, sweep off the top of the leveling pad or lower panel, and the bottom of the panel being set, to assure that no foreign material will potentially be trapped under the panel, which could affect horizontal level. Bearing pads are not required between the leveling pad and the panels of the bottom course.

IMAGE G: *Upper leveling pad offset 9 in.*

IMAGE H: *Bracing of ARES® panels.*

IMAGE I: *Establishing batter of ARES® panels.*

Below is the suggested sequence for placing panels on the bottom course: It is important to note that the panel should NOT be released prior to the step specifically calling for that action in the following sequence:

- Lower the panel into position on the pad, using one person on each end of the panel. (See Image H.)
- Using crowbars, position the base of the panel so that it matches the chalk line.
- Use a temporary spacer to assure that the ¾ in. (19 mm) space across the vertical joint is consistently provided between panels. (See Image J.) Without the correct joint spacing, panel corners may crack and spall.
- Check the panel for horizontal level; shim if required. (See Figure 11.) If it is not level, shims are placed under the panel in order to make the panel level. Galvanized metal washers or rubber shims are allowed. A maximum ⅜ in. (9.5 mm) in total shim height at any location is allowed. If more shims are required then the leveling pad is not level.
- Set the batter on the panel. (See "An Important Note on Batter" on pg. 15.)

NOTE: *Shims shall consist of permanent material that will not deteriorate.*

- Using the 4 ft (1.2 m) level with a predetermined blocking attached to one end, push the top of the panel back until the level reads plumb. (See Image I.)
- On the taller panels, install a header clamp and brace and tighten the clamp securely. Drive a stake in front of the wall at the midpoint of the panel for adequate bracing. (See Figure 10.) Nail the bottom of the brace into the stake. Check the batter and then nail the brace to the header clamp.
- On the half-panels, the header clamp and staking are not necessary; the half-panels should be held in place by clamping to the adjacent taller panels. At every vertical joint, position a standard clamp at the top of the half-panel (such that it will result in one clamp on either end of the half-panel) and loosely fasten it. (See Figure 10.)
- The panel may be released at this point.
- Tighten the clamps, pulling the half-panel to the same batter as the taller panel, and recheck the panels for alignment, batter and level.
- Drive wedges at the quarter points of the bottom front of the panel to maintain the batter.
- Nail 2 x 4 wooden blocks at the joint of the panel to prevent sliding during backfill of the first course. (See Figure 10.)

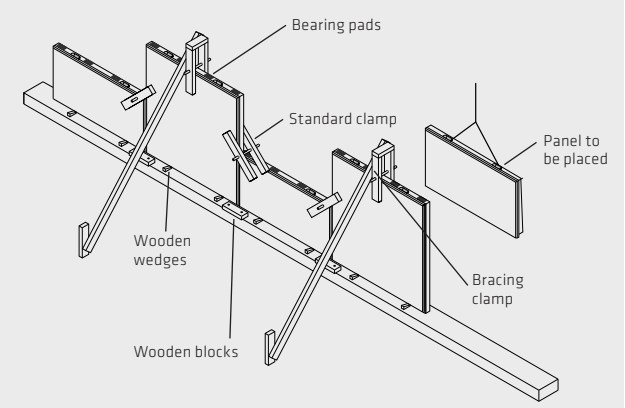


FIGURE 10: *Layout of bearing pads, wedges and clamps.*

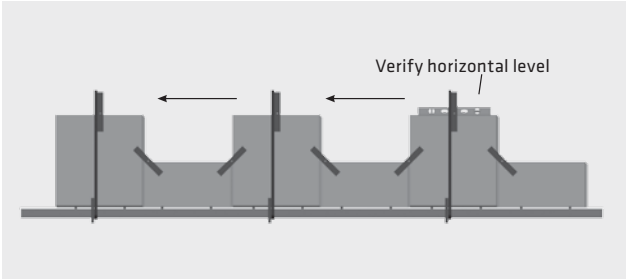


FIGURE 11: *Verifying horizontal level.*

IMAGE J: *Placement of 5 x 5 panels.*

Subsequent Course: *Panels placed, positioned, aligned, clamped, battered and wedged: all but the top course have been backfilled.*

Below is the suggested sequence for placing panels on all subsequent courses: It is important to note that the panel should NOT be released prior to the step specifically calling for that action in the following sequence:

- Subsequent panel rows are placed between panels that were previously placed. The ability to properly space and align these rows relies on the proper placement of the lower rows. All of the error produced by the lower rows is propagated upward and is difficult to correct. The same leveling, joint spacing, vertical and horizontal alignment applies to all of these rows as well.
- Prior to placing a panel on a subsequent course, the panel below should be backfilled to the point that the uppermost layer of geogrid attached to it is covered with at least one lift of compacted fill.
- Check the batter of the panel below the panel being set. Constant attention to the amount of rotation that is occurring in the adjacent panels and compensating in the following panels will yield the best results.
- Place bearing pads on lower panel. (See Images J and M.)
- Lower the panel into position on the bearing pads on the lower panel, using one person on each end of the panel.
- Using crowbars, position the panel and visually align it with the adjacent panels. (See Figure 12.)
- Check that the ¾ in. (19 mm) space across the vertical joint is consistently provided between panels. (See Image J.)
- Check the panel for horizontal level and shim if required. This is particularly important for taller walls to prevent alternate opening and closing of the vertical joints.
- Wedges may be temporarily placed in the vertical joints to maintain alignment until another panel is placed on top.
- Position and loosely fasten a standard clamp on each side of the new panel.
- The panel may be released at this point.
- Set the batter on the panel. (See “An Important Note on Batter” on pg. 15.)
- Tighten both side clamps and recheck the panel for alignment, batter and level.
- Drive the hardwood wedges at the quarter points between the top of the lower panel and the bottom of the new panel to assist in maintaining batter. These wedges should be checked during compaction and re-driven if they become loose.

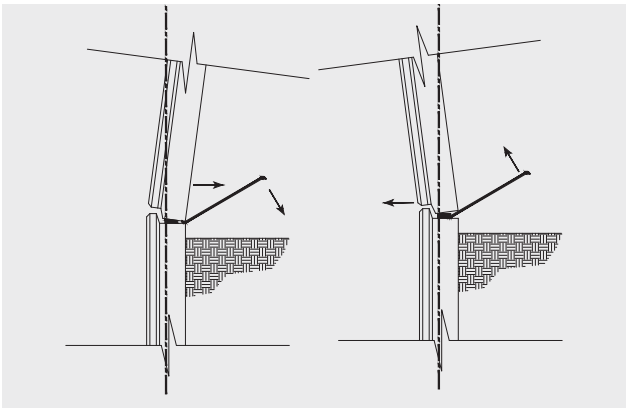


FIGURE 12: *Use of crowbars to align ARES® panels.*

IMAGE K: *Preslit fabric at geogrid elevations.*

IMAGE L: *Placement of filter fabric at ARES® vertical joint.*

Construction Procedures for ARES® Retaining Wall Systems (continued)

AN IMPORTANT NOTE ON BATTER

The amount of batter to which the panel is set is a function of the type, gradation and moisture content of the select fill. It is recommended that the batter in sands be initially set at 1 in. (25 mm) in 4 ft (1.2 m), and for coarser material be set at ¾ in. (19 mm) in 4 ft (1.2 m). The batter on subsequent rows of panels should be adjusted based on the results of the previous courses of panels when they have been backfilled to the top. Note also, particularly in sand backfill, that the required batter tends to be less on panels with more than two layers of geogrid and spaced vertically less than 30 in. (750 mm) apart. Guidance for determining and adjusting the batter is available from the Tensar technical advisor.

NOTE: *Failure to properly use wedges and clamps may result in excessive rotation of the panel.*

The lowest wedges in the column should be removed from the panels after three levels of wedges are in place above. Failure to remove wedges at this time can make subsequent removal difficult and may cause spalling of the concrete.

The vertical alignment of the overall wall should be checked daily using a plumb bob. These checks should be used to adjust the batter to which the panel is set. For example, if a batter of ¾ in. (19 mm) was used initially and after backfilling the batter measured with a plumb bob is ¼ in. then the next course should be set with ½ in. (13 mm) batter. If the panel gets a negative batter of ¼ in. (6 mm) after backfill is placed, the next course should be set with an additional ¼ in. batter. If for any reason the backfill source changes this process should be repeated. Monitoring and adjusting the batter of the panels will help maintain the vertical wall tolerances as required by the contract documents.

PLACING THE JOINT MATERIALS

- The Contractor should place the required bearing pads equally spaced along each horizontal joint between panels.
- Filter fabric is placed across the joints so that the backfill does not pipe through the joints to the outside of the wall. The minimum lap on each side of the joint is 6 in. and 1 ft along any cut piece of fabric along the joint. These requirements apply to horizontal and vertical joints. (See Image N.)
- The filter fabric is typically provided in a 12 in. (300 mm) wide strip and should be centered over all panel-to-panel joints and at special locations as shown in the contract documents where the wall abuts to other structures. The fabric should be slit around the embedded geogrid tabs.

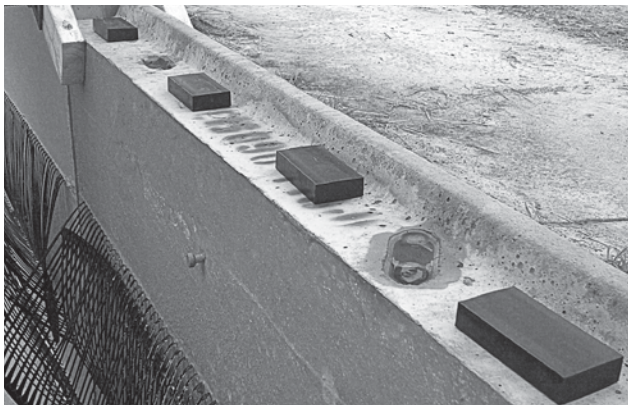


IMAGE M: *Common location of bearing pads.*

IMAGE N: *Filter fabric at vertical joint of ARES panels.*

IMAGE O: *Uniaxial geogrid shall be tensioned prior to fill placement.*

IMAGE P: *Placement of fill on tensioned geogrid.*

The following procedures are recommended for filter fabric preparation and installation:

- Once the filter fabric is cut, lay the filter fabric down and mark the location of the geogrid tab on the fabric.
- Pre-slit the fabric at the marked location to fit around the geogrid tabs. ***NOTE:*** *Use a sharp blade.*
- Use construction adhesive to hold the filter fabric in place prior to select fill placement. Adhesive to be applied surrounding the fabric about 2 in. from the edges including the area where the filter fabric is slit. The filter fabric needs to fully engage the back of the ARES® panels at all locations to ensure that the backfill does not leak through the joints.

GEOGRID IDENTIFICATION

- The Contractor is recommended to highlight the geogrid types on the construction drawings and spray paint the geogrid tab cast-in the back of the panels before fill placement using colors corresponding to colors of geogrid roll edges. (See pg. 10.)
- It is also recommended to label the geogrid embedment length for the particular section of the panel on the back of the panel face to ease installation prior to fill placement. (See pg. 10.)

PLACEMENT OF THE GEOGRID

- Install the UX Geogrid called for on the approved construction drawings, using the type, width and length of geogrid shown for each location within the wall. Installation of the geogrid must be coordinated with the panel and fill placement. The geogrid must be connected to the panels using the Bodkin connection as shown on the approved construction drawings.
- At the Contractor's discretion, prior to fill placement, the geogrid may be connected to the panels and then the geogrid may be temporarily flipped over the front face of the wall. The select fill shall be brought up to the level of the geogrid connection after compaction and shall be compacted and level for the entire geogrid embedment length prior to placing the geogrid.
- The Contractor should take care to ensure the level of the compacted fill is flush to the back face of the panel and up to the level of the geogrid connection.
- The geogrid shall not be placed on the grade until the necessary testing and acceptance of the in-place fill material has been obtained from the Engineer.
- The geogrid shall be positioned near perpendicular to the face of the panel and such that the tension is relatively uniform across the width of the connection and such that the geogrid lays flat on the grade for the entire embedment length.
- A tensioning rake is then inserted in front of one of the transverse bars to provide adequate tensioning from the panel and pushed down into the select fill (See Image O). The geogrid is then pulled with sufficient force to remove all slack. Proper technique is important to apply and maintain proper tension.
- While maintaining tension on the geogrid, select fill should be placed on the geogrid between the rake and the back of the panels (preferably immediately beyond the 3 ft (0.9 m) zone behind the panels). (See Image P.)
- The rake may be withdrawn immediately after initial placement of about one cubic yard (.76 m^3) loose, nominal or more of select fill on the section of geogrid.

IMAGE Q: Geogrid tensioning and fill placement process.

IMAGE R: Use only lightweight equipment to compact fill within 3 ft of ARES® panel.

FOR 5 FT X 9 FT PANELS AND 5 FT X 10 FT PANELS

- Unless otherwise noted on the approved construction drawings, two full widths of the proper type and length of geogrid shall be attached to each standard-width panel at each elevation requiring geogrid.

REINFORCEMENT FILL PLACEMENT

- Fill placement shall be performed in a manner that prevents the development of slack in the UX Geogrid. The select fill should be spread in a direction away from or parallel to the face of the wall. In this way, any slack that does develop will tend to be shoved toward the free (back) end of the geogrid. Further care should be taken during fill placement to avoid shoving the geogrid panels and causing them to shift sideways.
- Place and compact the select fill in accordance with the approved construction drawings and the contract documents. The select fill shall be compacted to a minimum of 95% of the maximum dry density as determined in accordance with AASHTO T-99 or as required by the contract documents, whichever is more stringent. Unless otherwise directed by the Engineer, the select fill lift thickness shall not exceed 10 in. (250 mm) loose. The lift thickness allowed is at the discretion of the Engineer, provided the Contractor can meet compaction requirements and maintain proper alignment.
- Static rolling is typically adequate for achieving the required compaction; heavy vibratory equipment may cause movement of wall components and potential misalignment of the wall facing, particularly in sand fill. The actual procedure used should be determined based on field trial results.
- Only hand-operated lightweight compaction equipment shall be used within 3 ft (0.9 m) of the back face of the panel. (See Image R.) Lightweight vibratory equipment and/or lightweight roller may be used for this purpose. The use of a Jumping Jack is not recommended.
- Tracked construction equipment shall not be operated directly on the geogrid. A minimum of 6 in. (150 mm) of fill is required between the tracks and the geogrid. Rubber-tired equipment may be operated directly on the geogrid, provided the subgrade is not pumping or rutting. Turning of all equipment shall be minimized to prevent dislocation or damage to the geogrid. The equipment must travel slowly and with sufficient care to avoid dislocating the geogrid.

IMAGE S: Bodkin connection used to connect geogrid panels to geogrid tab.

IMAGE T: Compacted select fill brought up to the level of the geogrid.

- At the end of each day, the Contractor must ensure that the reinforced fill zone is compacted and graded to drain away from the face of the wall and that berms or ditches are in place and functioning to prevent the entrance of runoff into the wall construction site.
- **Proper installation and tensioning of the geogrid and select fill is critical to the alignment, appearance and performance of the ARES® Retaining Wall Systems. Care should be taken to ensure that the geogrid is properly tensioned and select fill is properly placed.**

WALL TOLERANCES

Unless otherwise noted on the approved construction drawings or in the contract documents, ensure the following:

- Deviation in vertical and horizontal alignment does not exceed ¾ in. (19 mm) when measured with a 10 ft (3 m) straightedge. Offsets (measured perpendicular to wall face) at the joints between panels do not exceed ¾ in. (19 mm).
- Gaps at horizontal and vertical joints between adjacent panels are not less than ½ in. (12 mm) and not more than 1¼ in. (32 mm).
- Deviation in the final overall verticality of the completed wall (plumbness from top to bottom) does not exceed ½ in. per 10 ft (4 mm per m) of wall height.

THE ARES® SYSTEMS ADVANTAGE

For more than 30 years industry professionals have been using Tensar® Geogrids to build economical, long-lasting structures. With clear advantages in performance, design and installation, ARES® Systems offer a proven technology for addressing the most challenging projects.

For more information on ARES Systems, call **800-TENSAR-1**, visit **www.tensarcorp.com** or send an e-mail to **info@tensarcorp.com**. We are happy to supply you with additional information, system specifications, design details, conceptual designs, preliminary cost estimates, and much more.

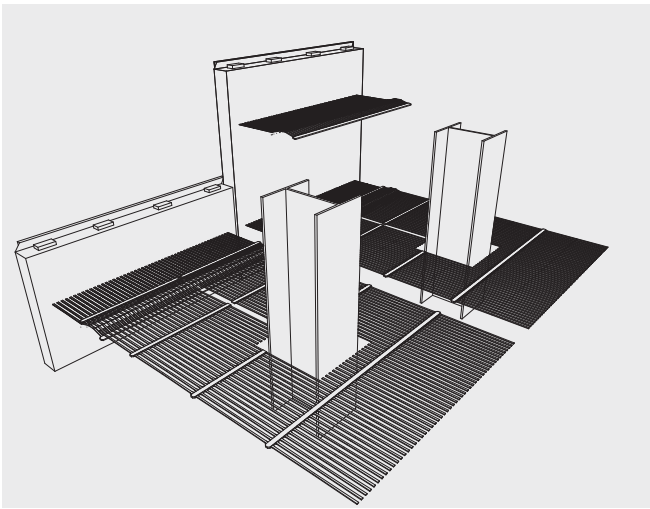
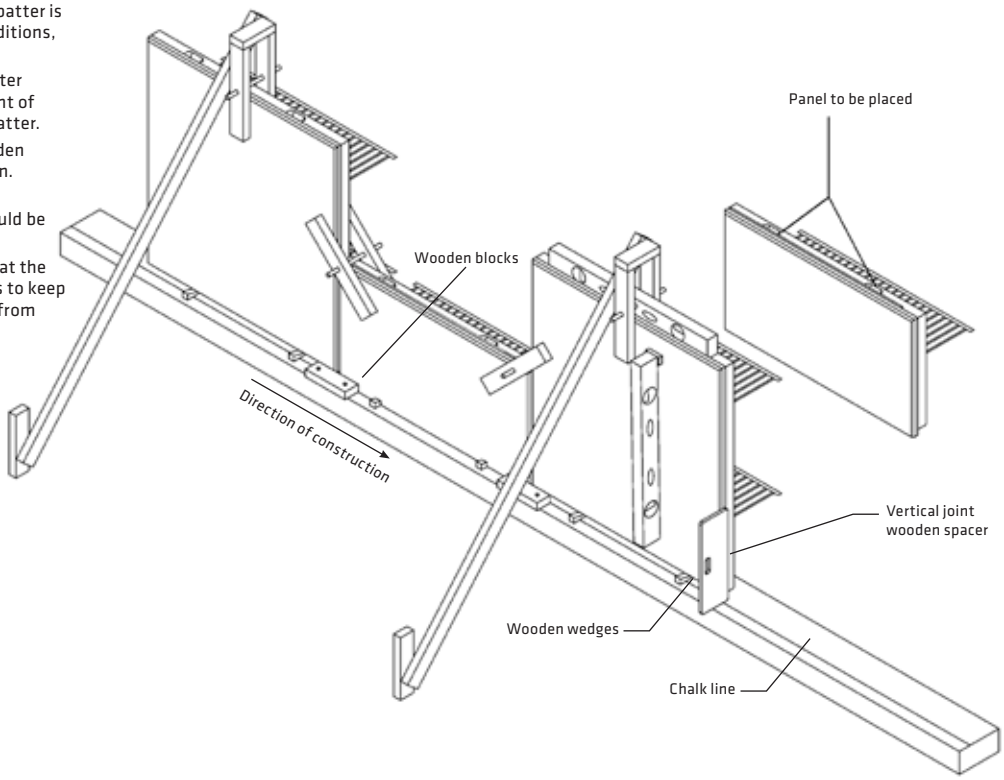
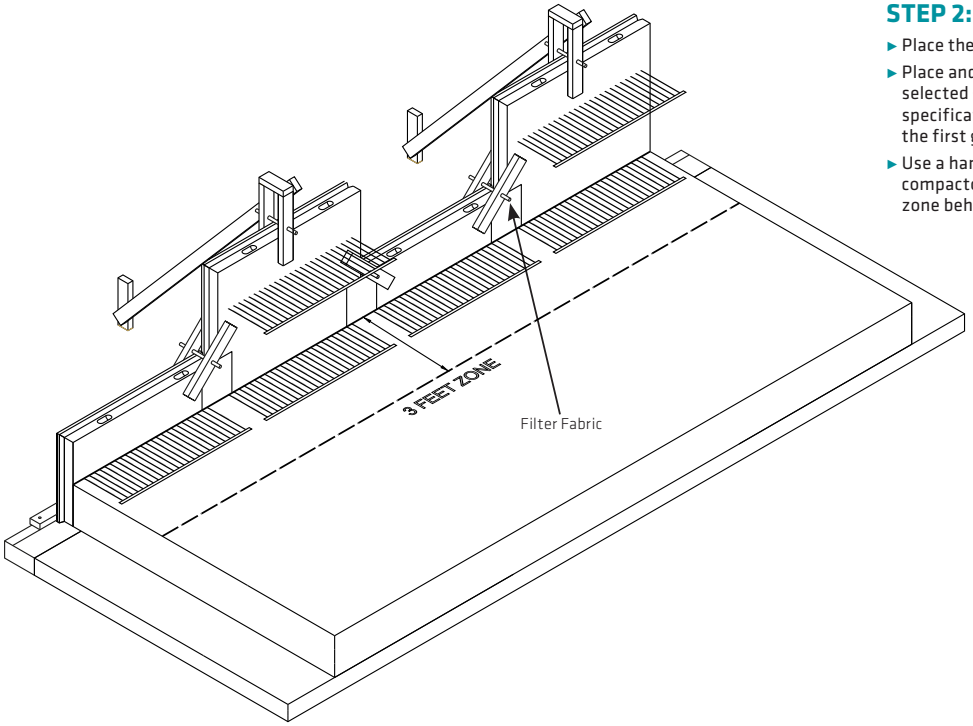


IMAGE U: *Placement of geogrid panels around and adjacent to vertical penetrations.*

STEP 1:

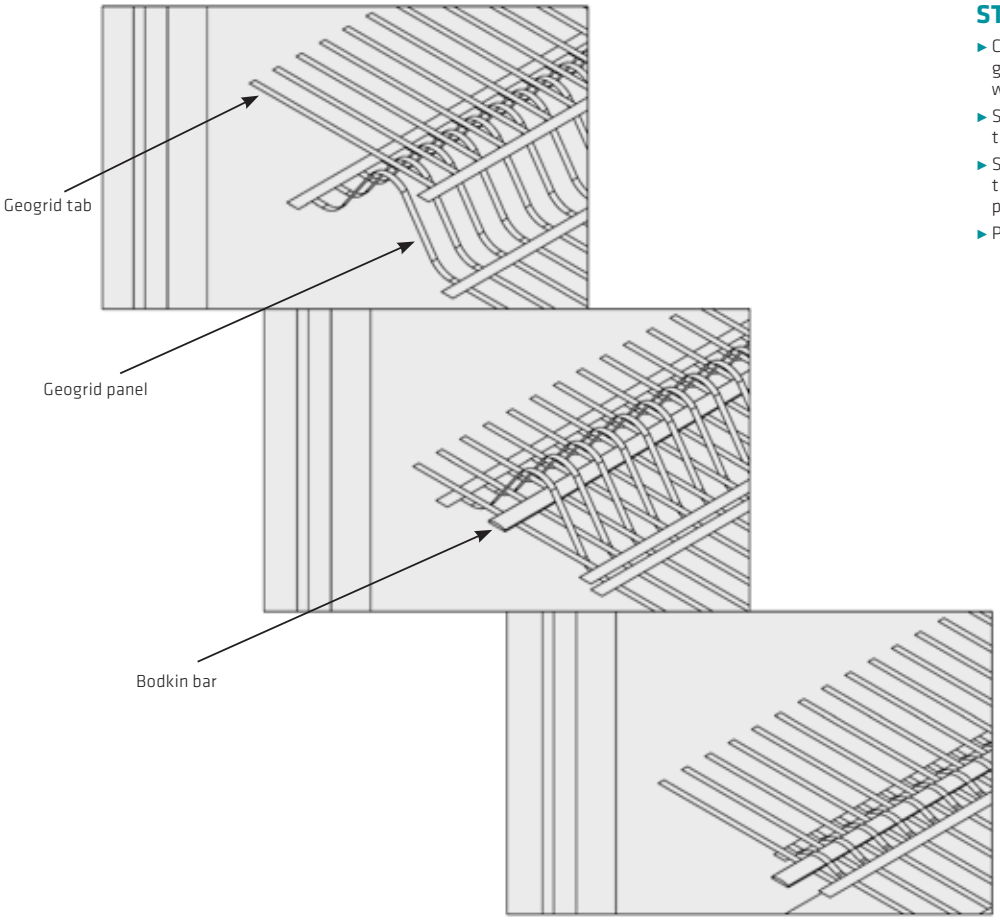
- The first row of panels are placed on the leveling pad and braced.
- The panels should be set with a backward batter according to the panel batter recommendation on page 15. Important: The batter is adjusted for the site conditions, e.g. backfill properties.
- Drive wedges at the quarter points of the bottom front of the panels to maintain batter.
- Use a ¾ in. (19 mm) wooden spacer to achieve the ¾ in. (19 mm) vertical joints.
- Adjacent half panels should be clamped together.
- Nail 2 x 4 wooden blocks at the joint of the panels; this is to keep the bottom of the panel from "kicking out."





STEP 2:

- Place the filter fabric over vertical joints.
- Place and compact initial lifts of selected granular backfill, per project specifications, up to the bottom of the first geogrid tab.
- Use a hand-operated vibratory compactor in the 3 ft (900 mm) zone behind the panels.

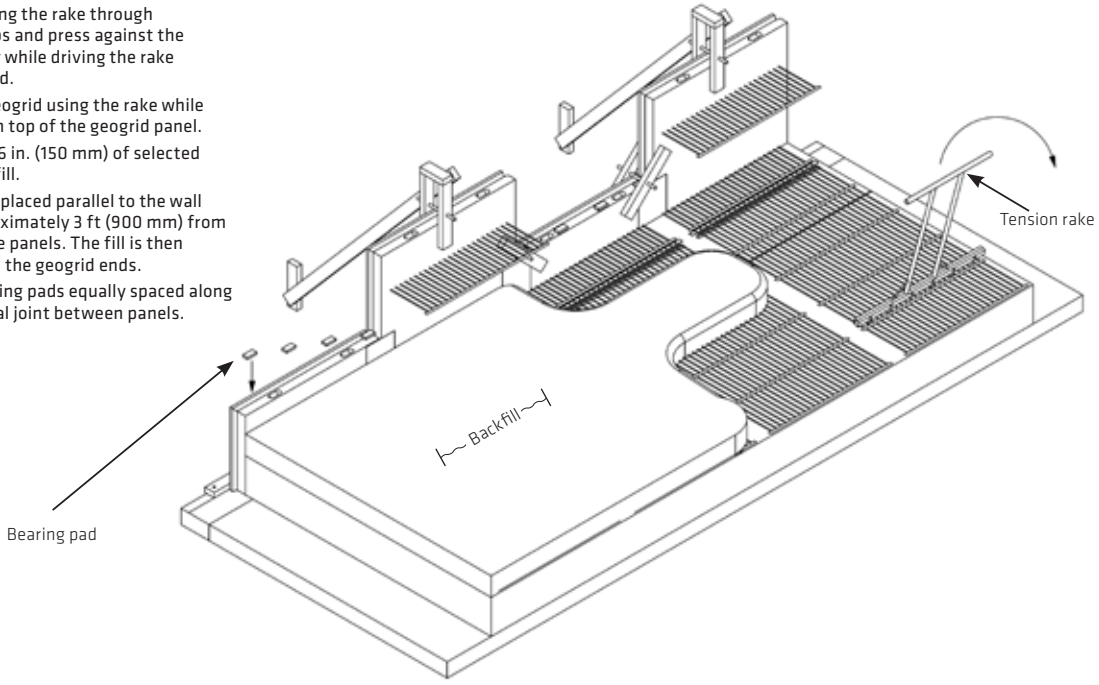


STEP 3:

- Connect each geogrid panel to the geogrid tab embedded in the panel with the Bodkin bar.
- Start by bending the geogrid through the tab to create a tunnel.
- Slide the Bodkin bar through the tunnel that was formed by the geogrid panel and the georid tab.
- Pull the geogrid snug by hand.

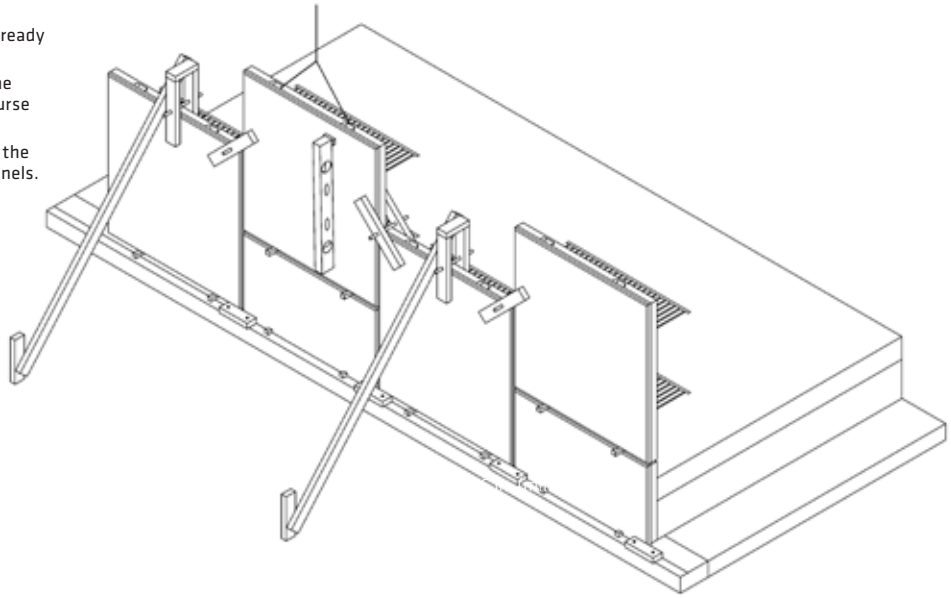
STEP 4:

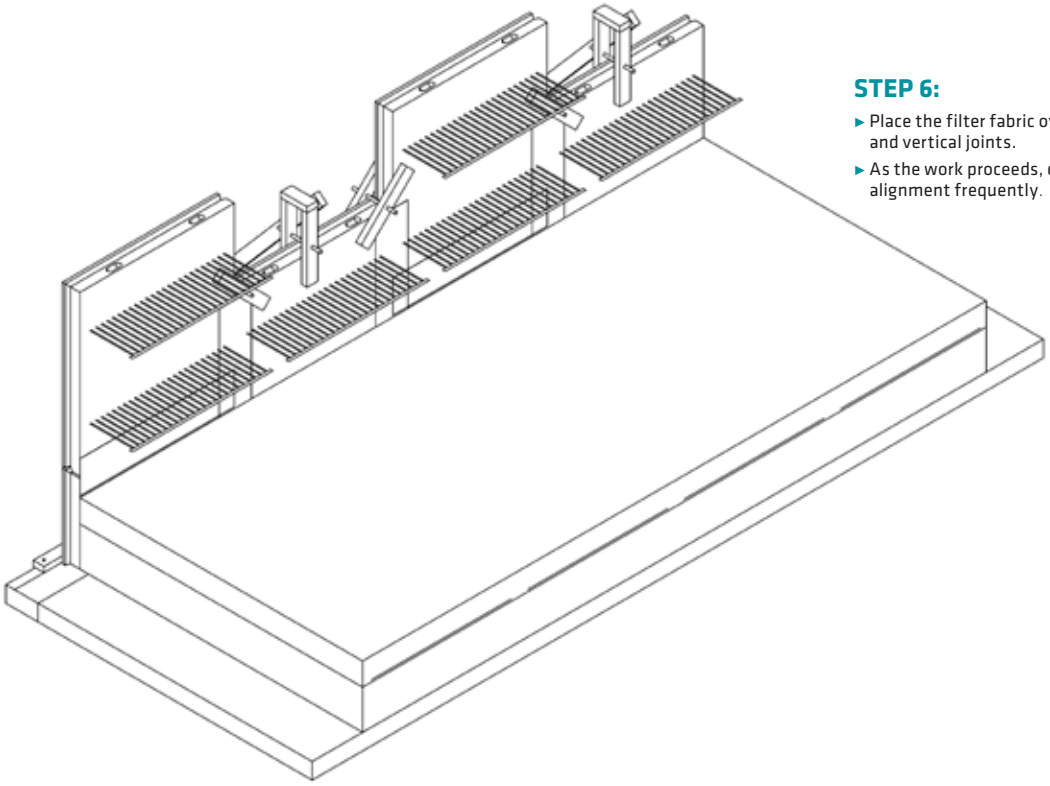
- When the fill is ready to be placed, tension the geogrid with the tension rake.
- Start by pushing the rake through the geogrid ribs and press against the transverse bar while driving the rake into the ground.
- Tension the geogrid using the rake while fill is placed on top of the geogrid panel.
- Place at least 6 in. (150 mm) of selected granular backfill.
- The backfill is placed parallel to the wall starting approximately 3 ft (900 mm) from the back of the panels. The fill is then spread toward the geogrid ends.
- Place the bearing pads equally spaced along each horizontal joint between panels.



STEP 5:

- Place the second row of panels only after backfill has reached 6 in. below the half panels.
- When the next row of panels are ready to be set, remove the clamps.
- Place the next row of panels in the "window" created by the first course of panels.
- Check the batter and then install the wedges and clamp to adjacent panels.



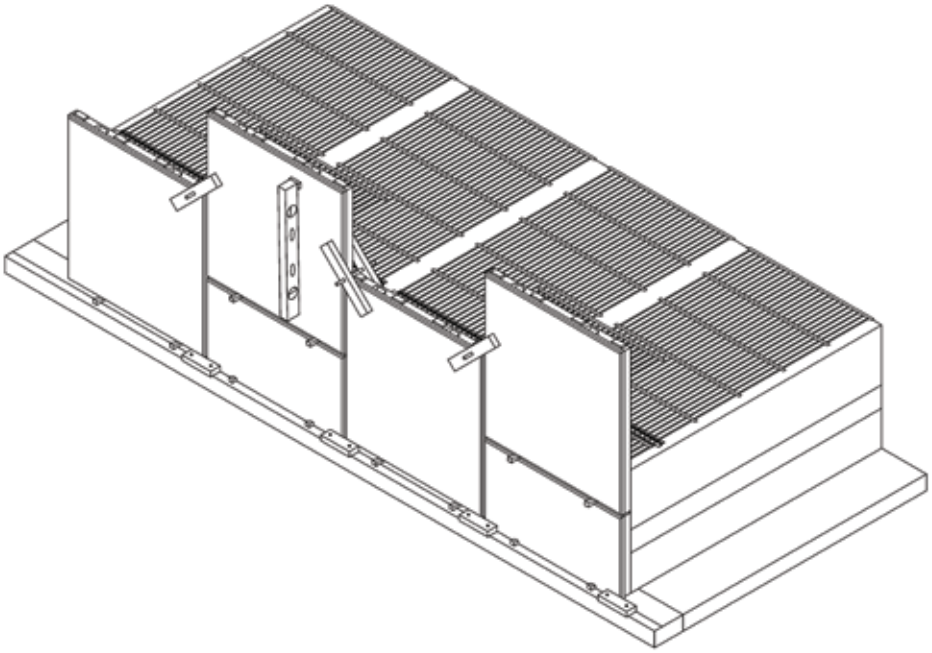


STEP 6:

- Place the filter fabric over the horizontal and vertical joints.
- As the work proceeds, check the panel's alignment frequently.

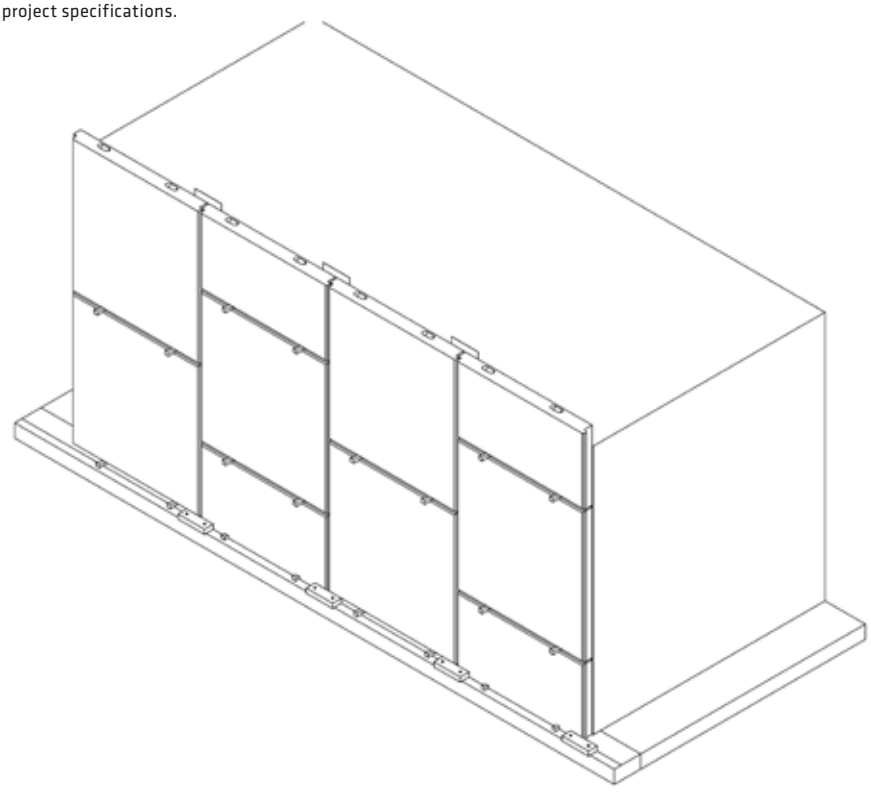
STEP 7:

- Place selected granular backfill in lifts per project specifications.
- After backfilling, recheck the batter and alignment of the panels.



STEP 8:

- Connect and install geogrid panels as described on step 3.
- Repeat the process from steps 4 to 7.
- As soon as practical, the front of the wall should be backfilled.
- The top wall treatment is then placed per project specifications.



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 the plans and specifications, the conditions and provisions of the agreement, including any addenda and other modifications issued prior to or after the bid and the execution of the original 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.

Engineer
The Owner's representative with authoritative charge over the inspection and acceptance of the wall construction in accordance with the contract documents.

Inspector
An authorized representative of the Owner assigned to see that the workmanship and materials are in accordance with the terms of the contract.

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 plans, profiles, typical cross-sections, working drawings and supplemental drawings, or exact reproductions thereof, which show the location, character, dimensions and details of the work to be performed.

Precaster
Every precast panel manufacturer under contract with Tensar.

Specifications
The part of the contract documents consisting of 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 Technical Advisor
An authorized representative of Tensar that is available on site at the start of the project to advise the Contractor recommended construction procedures within the scope of this document. This person is not an inspector or member of the quality control staff on the project.

Work
All work items to be performed by the Contractor under the terms and conditions of the contract that are necessary to fulfill the obligations of said contract.

Tensar®

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Appendix 4.1.2

Tensar Geogrids QC-QA Manual

Latest report: September 2024

ARES® 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.

TABLE OF CONTENTS

		Page
FORWARD		2
EXECUTIVE SUMMARY		3
LIST OF FIGURES		5
LIST OF TABLES		6
1.0	**INTRODUCTION**	7
2.0	**STATEMENT ON QUALITY**	7
3.0	**THE COMPANY**	7
4.0	**PRODUCT HISTORY**	7
5.0	**GENERAL PRODUCT DESCRIPTION**	7
6.0	**QUALITY PROCEDURES THROUGHOUT THE MANUFACTURING PROCESS**	8
6.1	Overall Concept and Approach	8
6.2	Quality Assurance of Base Materials	9
6.2.1	Resin	9
6.2.2	Masterbatch	10
6.2.3	Regrind Resin	10
6.2.4	Resin Formulation	10
6.3	Quality Control/Assurance of Work-In-Process	11
6.3.1	Sheet Extrusion	11
6.3.2	Sheet Perforation	14
6.3.3	Sheet Orientation	18
6.3.4	Finished Products	23
6.3.5	Line Checks	26
7.0	**CONCLUSIONS**	26
8.0	**GLOSSARY OF TERMS**	28
8.1	Base Material	28
8.2	Process Terms	29
8.3	Product Definitions	30
8.4	Quality Definitions	30
9.0	**GLOSSARY OF TEST STANDARDS**	31

LIST OF FIGURES

FIGURE	TITLE	Page
1	Sheet Thickness Measurement	13
2	Measurement of Extruded Sheet Width	13
3	Examples of High-Quality Extruded Sheet	14
4	Measuring Aperture Pitch in MD of Punched Sheet	17
5	Measuring the MD Aperture Length of a Uniaxial Geogrid	22
6	Aperture measurements for (a) Biaxial, (b) TriAx (c) HX/NX	22
7	Assessing Degree of Bow and Skew in a Uniaxial Geogrid	22
8	Tensile Test on a Single Rib of Uniaxial Geogrid	24

LIST OF TABLES

TABLE	TITLE	Page
1	Quality Assurance of Qualified Resins	9
2	Quality Assurance of Masterbatch	10
3	Quality Control/Assurance of Work-In-Process: Sheet Extrusion Principal Parameters and Their Purpose	11
4	Quality Control/Assurance of Work-In-Process: Sheet Extrusion Method, Frequency, and Corrective Action	12
5	Quality Control of Work-In-Process: Sheet Perforation Principal Parameters and Their Purpose	15
6	Quality Control of Work-In-Process: Sheet Perforation Method, Frequency, and Corrective Action	16
7	Quality Control of Work-In-Process: Sheet Orientation Principal Parameters and Their Purpose	18
8	Quality Control of Work-In-Process: Uniaxial Sheet Orientation Method, Frequency, and Corrective Action	19
9	Quality Control/Assurance of work-in-process: Multi-Axial Sheet Orientation Method, frequency, and corrective action	20
10	Quality Control/Assurance of Finished Biaxial Geogrids	21
11	Quality Assurance of Finished Products: Uniaxial Geogrids	23
12	Quality Assurance of Finished Products: Multi-Axial Geogrids	25
13	Quality Assurance of On-Line Production Quality Control	27

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

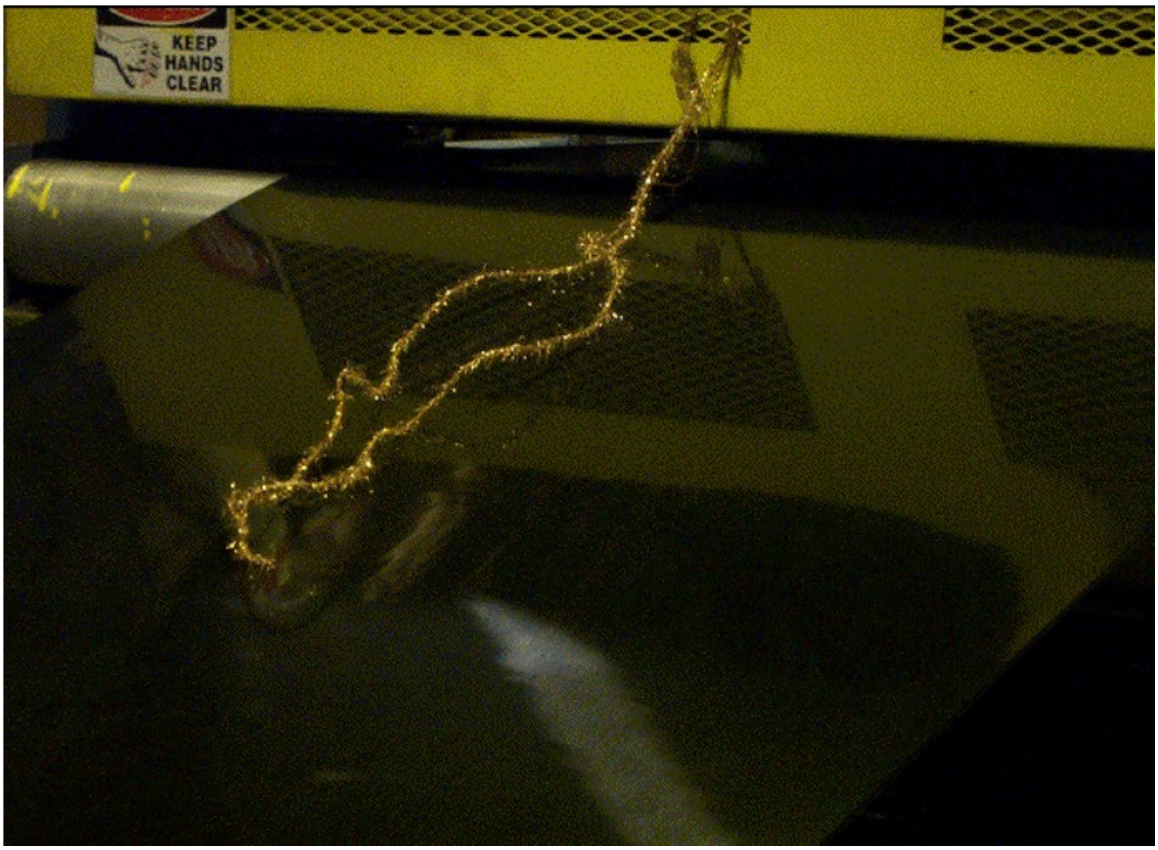


Figure 3 Example of High Quality, Extruded Sheet

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

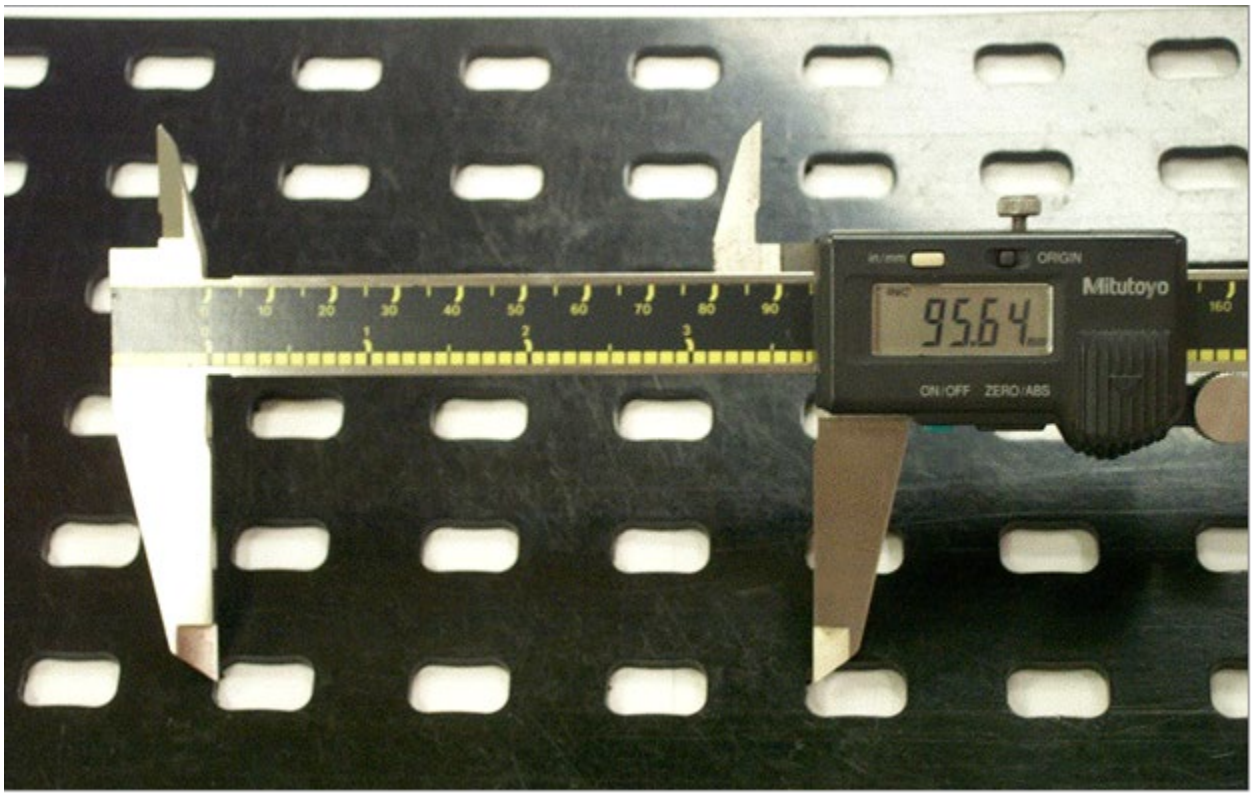


Figure 4 Measuring aperture pitch in the MD

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

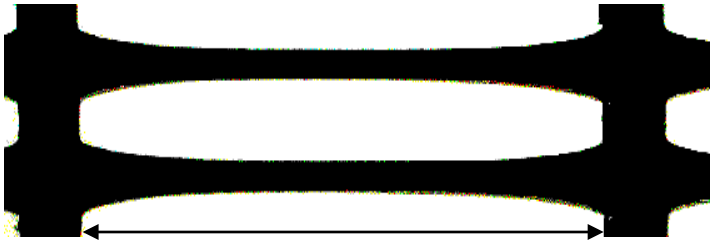
Parameter	Method	Frequency	Corrective Action
Product Width	Tape Measure	20-25 Rolls	Adjust Edge Trim
MD Aperture Length	Tape Measure	Every Lot	Adjust MD Draw
CMD Aperture Length	Tape Measure	Every Lot	Adjust CMD Draw
MD Rib Count per meter width	Tape Measure Direct Count	Every Lot	Adjust CMD Draw
CMD 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

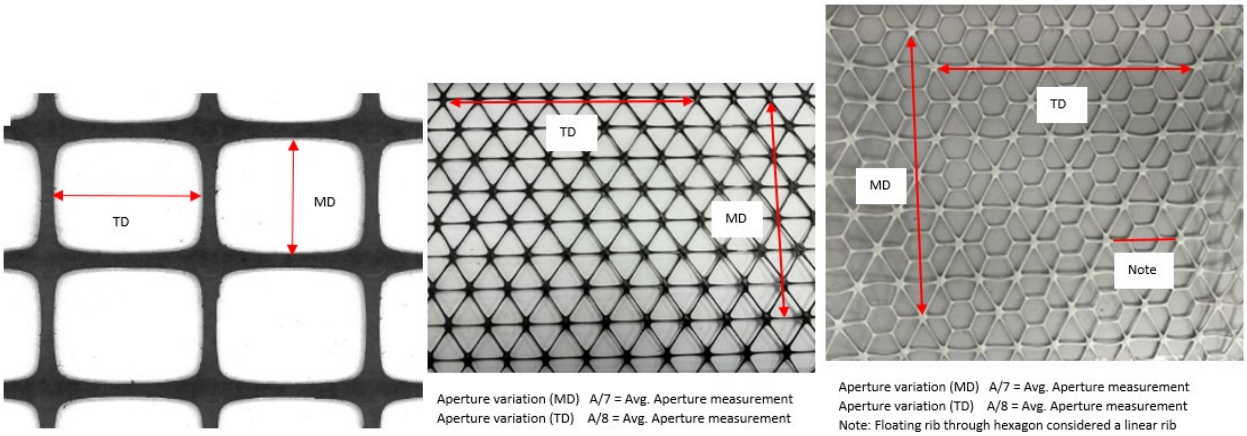
* Sampling per ASTM D4354: A production lot is defined as the summation of all batches (sub lots) which are produced on a daily basis across a fiscal year (September 1-August 31)
Note 1- Once / sub lot for wide width testing and once per sub lot for single rib in accordance with sampling requirements 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

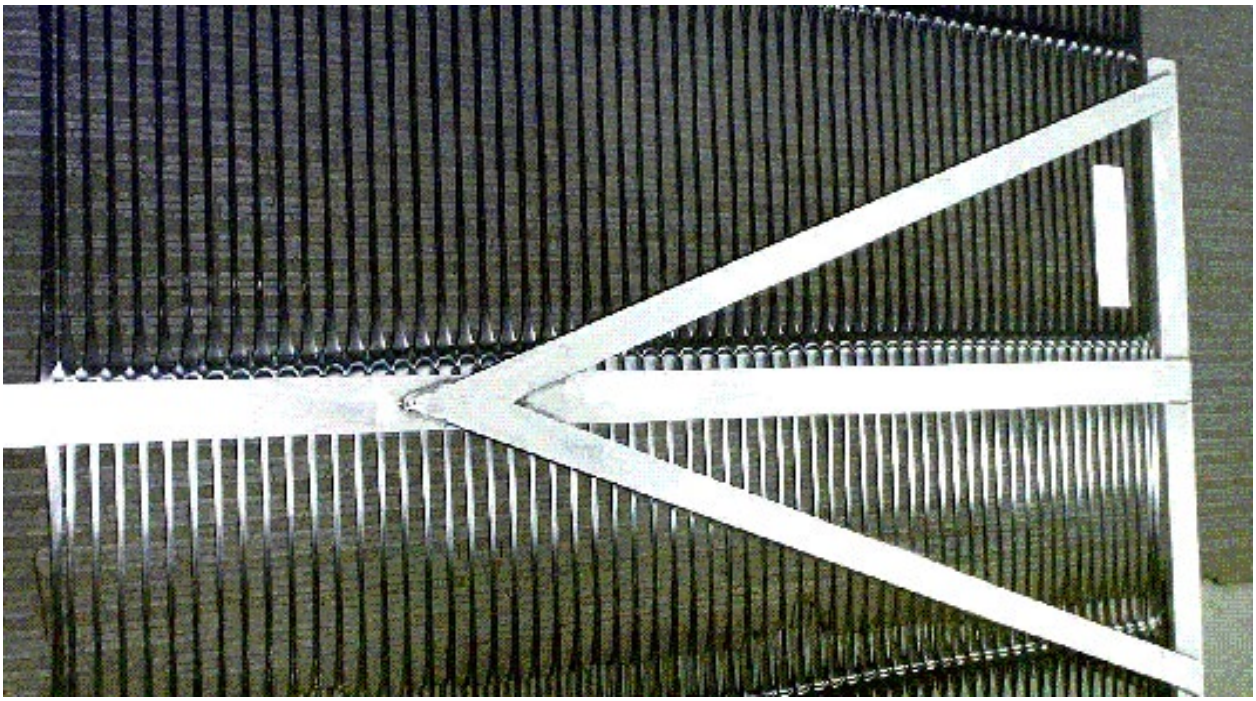


Figure 7 Assessing the Degree of Bow and Skew in a Uniaxial Geogrid.

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

ARES Tab Specifications

ARES® MSE Retaining Wall System

Back to main document

Specification for ARES® Geogrid Tabs

Description:

ARES Geogrid Tabs shall be cut from rolls of Tensar® geogrid. The geogrid will be UX1700AR geogrid. *Transverse* cuts shall be made to create tabs which are one aperture long in the machine (roll) direction. These cuts will bisect each transverse bar on each end of tab. *Longitudinal* cuts shall be made to remove one or more outside ribs on each side of each tab to achieve the acceptable tab width.

Quantity:

The quantity of accepted tabs will vary depending on the number of rolls provided, the number of apertures per roll and the number of tabs rejected.

To Be Provided by Tensar:

1. Rolls of geogrid
2. Product labels
3. Pickup and delivery of rolls necessary for conversion, finished product, waste, and regrind between vendor's location and Tensar warehouse (T1 at 1210 Citizens Parkway, Morrow, GA 30260).

To Be Provided by Vendor:

1. Cutting and measuring equipment
2. Plastic ties for tab bundles
3. Labels for regrind material
4. Boxes and packing tape

Tolerances and Rejection Criteria:

The finished units shall conform to the following requirements:

1. The width of the transverse bar on each end of the tab will be 12 mm minimum and 34mm maximum as shown in Figure 1.
2. The finished width of each tab will be 52 inches, with an allowable tolerance of ± 1 inch as shown in Figure 2.
3. Bow of piece: ½ inch maximum. Units not meeting this requirement shall be rejected.
4. Skew of piece: 1 inch maximum. Units not meeting this requirement shall be rejected.
5. Damage and defects: Units with damage or defects shall be rejected.

Figure 1: UX1700AR Tab Transverse Bar Width and Tolerance

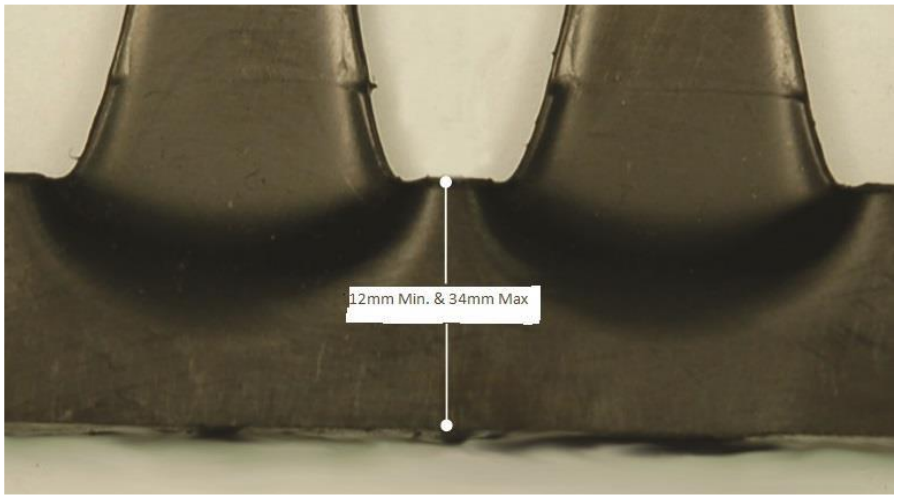
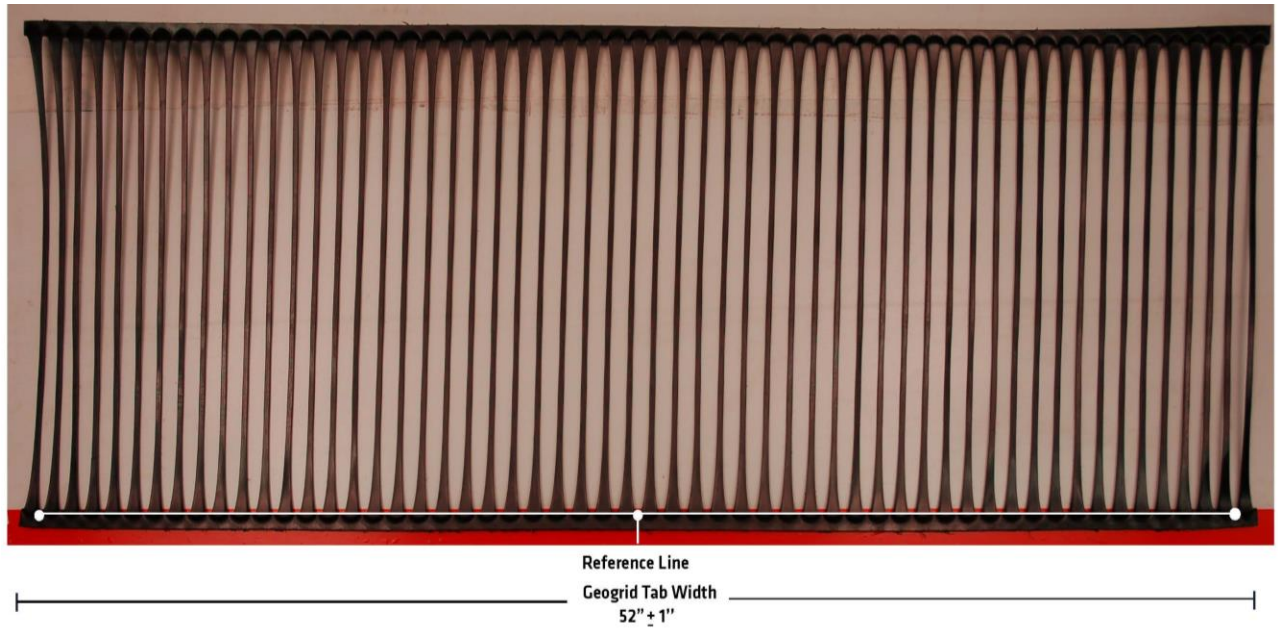


Figure 2: UX1700AR Tab Finished Width and Tolerance



Measuring Bow and Skew:

Refer to Figure 3 and Figure 4.

1. Establish a reference line along the inner edge of the transverse bar as shown in Figure 2, starting from the end of the last aperture on the left side of the grid piece and ending at the end of the last aperture on the right side of the grid piece. If the outermost rib on each edge of the grid piece has not already been removed, then ignore the outermost aperture and use instead the next-to-last aperture on each edge of the grid piece.

2. Press or clamp the tab down flat for bow and skew measurements
3. Measure bow as the greatest perpendicular distance from this reference line to the inner edge of the transverse bar at any point along the width of the grid tab. Bow may be above the reference line, as shown, or below the reference line, or both, on any given section of geogrid.
4. Establish a theoretical perpendicular line and measure skew as the distance from the reference line to a theoretical perpendicular line across the grid width (measured at maximum = the edge of the grid).
5. Repeat steps 1 through 3 for each transverse bar.

Figure 3: Determining Bow of UX1700AR Tab

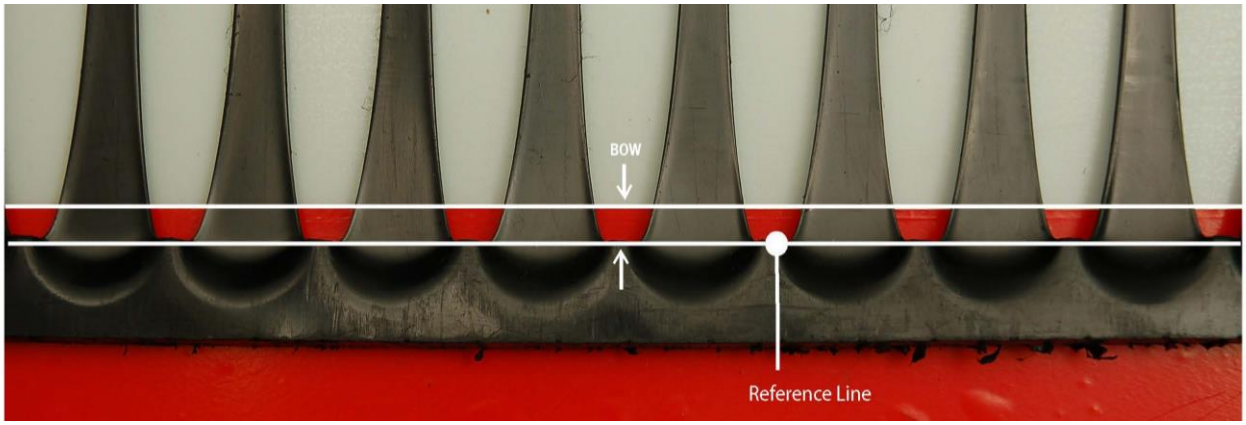
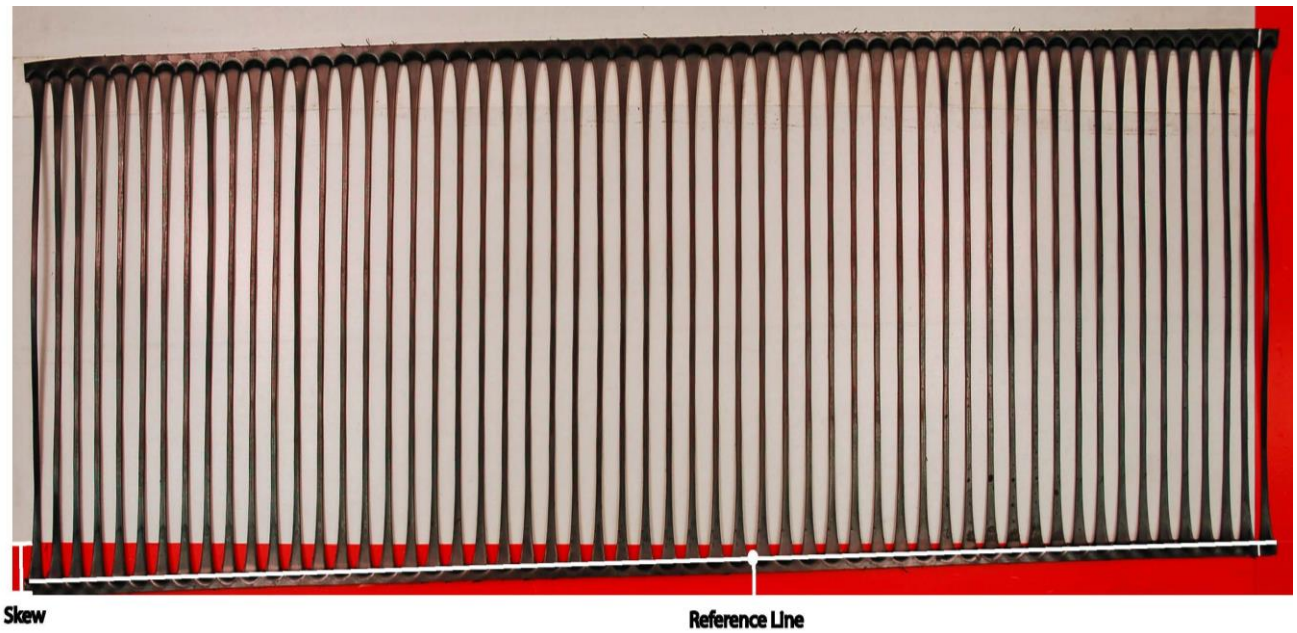


Figure 4: Determining Skew of UX1700AR Tab



Tracking:

Concurrent with production, Vendor shall record, tabulate and report daily roll #'s consumed, the yield per roll (tabs accepted/rejected from each roll, including actual lot number) and the nature of any excess waste.

Vendor's invoice will clearly itemize quantities of accepted tabs by lot #.

Packaging and Labeling:

Vendor shall:

1. Make bundles of 25 units each; label each bundle using lot labels provided by Tensar.
2. Package 4 bundles per box, provided by Vendor.
3. Label each box with a lot label appropriate as to the contents of the box, using labels provided by Tensar. Manually record production date, quantity per lot, and box quantity.
4. Palletize boxes on 4-ft x 5-ft wooden pallets, 4 boxes per pallet, or other quantity as necessary.
5. Organize regrind material in pallets wrapped with shrinkwrap and/or gaylords and mark as "RHDPE - Regrind ARES Tabs".

Rejected/Regrind Materials:

Tensar intends to have the rejected geogrid returned for regrind. Vendor shall keep these geogrid materials separate from other materials, including other Tensar geogrids, until Tensar has sent them to regrind or otherwise disposed of them.

Separation from Other Materials and Other Lots:

It is critical that the geogrid tabs be produced from the UX1700AR rolls provided by Tensar for that purpose. In producing the geogrid tabs, vendor shall not mix geogrid from different Tensar production lots of other type materials.

Certification:

As a requirement for payment, Vendor will provide written certification that all accepted tabs conform to these specifications.

Appendix 5.1.2.0

Historic Projects

ARES® MSE Retaining Wall System

Back to main document

ARES® Featured Abutment Wall Projects

1. Tanque Verde; Tucson, AZ
2. Overlander Bridge Access; Kamloops, British Columbia, Canada
3. I-25 – 70th Street Ramp; Denver, CO
4. Harrisville Blvd CNR Overpass, Moncton, New Brunswick, Canada
5. Pile Demo Wall, E-470; Brighton, CO
6. E-470 Phase IV Extension; Brighton, CO
7. SR 401; Cape Canaveral, FL
8. Memorial Causeway; Clearwater Beach, FL
9. SR 93 / I 275; St. Petersburg, FL
10. US 71 Cass County; Rabun, MO
11. U.S. Highway No. 36, Boulder County, Colorado
12. 41st Street Overcrossing, Everett, Washington
13. Beach Boulevard; Jacksonville Beach, FL
14. Powers Blvd. – Pine Creek, El Paso County, Colorado
15. SH 133 – Garfield County, Colorado
16. CR124 – Delaware County, Ohio
17. SR 22 – Hillside, New Jersey
18. Dutchess County Rail Trail, Dutchess County, New York

ARES Project Details

Project Name	**Tanque Verde Project**
Year Constructed	1984-1985
Location	Tucson, Arizona, USA
Design Engineer	Dames & Moore
Maximum Height	6 meters (19.7 ft)
Wall Length	1600 m (5248 ft)
Facing	Full Height Panels (geotab/bodkin connection)
Description	46 Walls were constructed to separate grades at Tanque Verde – Wrightstown - Pantano roads, and included ramps to two bridges. Full-height panels 150 mm (6 inches) thick and 3 m (10 ft) were used. The soil is reinforced with layers of TENSAR SR2 geogrid.
References	1. Berg, R.R., Bonaparte, R., Anderson, R.P. and Chouery, V.E., *Design, Construction, and Performance of Two Geogrid Reinforced Soil Retaining Walls*, Proceedings 3rd International Conference on Geotextiles, Vol. II, Vienna, Austria, 1986, pp. 401-406. 2. FHWA, *Tensar Geogrid-Reinforced Soil Wall, Grade-Separation Structures on the Tanque Verde-Wrightstown-Pantano Roads Intersection, Tucson, Arizona*, Experimental Projects Program, Ground Modification Techniques, U.S. Department of Transportation, Federal Highway Administration, Office of Highway Operations – Demonstration Projects Division, FHWA-EP-90-001-005, 133 p. 3. Fishman, K.L., Desai, C.S. and Berg, R.R., *Geosynthetic Reinforced Soil Wall: 4-year History*, Transportation Research Record No. 1330, January 1991. 4. Berg, R.R. and Chouery-Curtis, V.E., *Geogrid Reinforced Walls for Grade Separation, Tanque Verde Road, Tucson, USA,* Geosynthetic Case Histories, Editors - G.P. Raymond and J.P. Giroud, International Society for Soil Mechanics and Foundation Engineering, 1993, pp.262-263. 5. Wayne, M.H., Bright, D.G., Berg, R.R. and Fishman, K., Tanque Verde Retaining Wall Structure: Revisited after 11 Years," *10th Annual GRI Conference*, Drexel University, Philadelphia, PA, December 10-11, 1996.

Project	**Overlander Bridge Access**
Year Constructed	1999
Location	Kamloops, British Columbia, Canada
Design Engineer	Tensar Earth Technologies, Inc.
Maximum Height	9 meters (29.5 feet)
Wall Area	1500 square meters (16,000 square feet)
Facing	Full Height Panels
Description	Eight walls for bridge approaches, abutments, and mainline embankments. The two bridge was founded on spread footing on top of the reinforced soil mass
References	Brendan Sheppard, Dan Alzamora

Project	**I-25 – 70th Street Ramp**
Year Constructed	**1993**
Location	Denver, CO
Design Engineer	Tensar Earth Technologies, Inc.
Maximum Height	7.3 meters (24 feet)
Wall Area	600 square meters (6500 square feet)
Facing	Full Height Panels
Description	Two walls support ramp to bridge over I-25. The panels had a “Glenwood Canyon” Striation and were stained.
References	Bob Miller, Ron Anderson

Project	**Harrisville Blvd CNR Overpass**
Year Constructed	**1998**
Location	Moncton, New Brunswick, Canada
Design Engineer	Tensar Earth Technologies, Inc.
Maximum Height	11 meters (36 feet)
Facing	Full Height Panels
Description	Two abutment walls supporting a single span bridge over two rail lines. Bridge foundations were spread footings placed on the reinforced soil. The lower 17 foot high face section was cast-in-place with geogrid tabs cast in the back. These sections were about 2 feet thick to resist the impact of derailed trains. The upper 19 foot high panels were 6 inches thick. Upon completion of the CIP sections, panels were set in 2 days and backfilled in 7 days.
References	Bob Miller, Phil Perzia, Eugene Holt (Arrow 503 452 8178)

Project	Pile Demo Wall, E-470 Phase IV Extension
Year Constructed	2000
Location	Brighton, CO
Design Engineer	Tensar Earth Technologies, Inc.
Maximum Height	4.6 meters (15 feet)
Wall Length	8.2 meters (27 feet)
Wall Area	38 square meters(405 square feet)
Facing	Full Height Panels, 15 feet high by 9 feet wide
Description	A small wall was constructed prior to the start of the E-470 project construction. It was designed and built by TET and Slaton Bros. Custom Retaining Walls to demonstrate the viability of driving 12 inch H-piles through Tensar UXMESA2, USMESA3 and UXMESA6 geogrids. The piles were driven 2.5 to 4.5 feet behind the panels. Instrumentation was monitored during construction and after pile driving. There was no problem driving the piles. There was no damage to, or significant movement of, the panels. The piles cut cleanly through the grid.
References	Tensar Earth Technologies, Inc. "Report on Demonstration of Pile Driving through the HDPE Geogrid Reinforced Soil Fill of Full-Height Precast Concrete Panel Faced, Mechanically Stabilized Earth Wall, Colorado E-470 Project" April 2001.

	E-470 Phase IV Extension
Year Constructed	2001
Location	Brighton, CO
Design Engineer	Tensar Earth Technologies, Inc.
Maximum Height	12.8 meters (42 feet)
Wall Length	
Wall Area	11,600 square meters (125,000 square feet)
Facing	Full Height Panels
Description	A dozen walls for bridge approaches and abutments on this project. As of August 8, 2001. 3 walls are completed, 1 is under construction, 5 awaiting start of construction, and 3 yet to be designed. Maximum height of nine-foot wide panels is 34 feet. 8 foot high panels are set on top to create 42 foot sections. Piles are driven through the abutment walls geogrid. 4 foot diameter caissons within the reinforced fill are used for 1 bridge.
References	Brendan Sheppard, Nazar Elwazir

	SR 401
Year Constructed	**2002**
Location	Cape Canaveral, FL
Engineer of Record	Kevin Lee Tensar Earth Technologies 404-250-1290
Maximum Height	27 feet
Wall Length	4,000 ft
Wall Area	40,000 SF
Facing	ARES 5x5 Panels
Description	Abutment founded on large concrete piles.
TIC References	Jeremiah Riggio 404-250-1290
Wall Installer	Bonn-J Contracting 407-977-7666
GC	Eby Construction 321-784-5233
Owner's Rep	Canaveral Port Authority 321-783-7831 FDOT 321-690-3250

	Memorial Causeway
Year Constructed	**2002**
Location	Clearwater Beach, FL
Engineer of Record	Kevin Lee Tensar Earth Technologies 404-250-1290
Maximum Height	42 feet
Wall Length	2,700 ft
Wall Area	60,000 SF
Facing	ARES 5x5 Panels
Description	Abutment founded on 48" drilled shafts.
TIC References	Jeremiah Riggio 404-250-1290
Wall Installer	MSE Systems 407-931-1221
GC	PCL 727-462-9949
Owner's Rep	FDOT 727-725-7950

	SR 93 / I 275
Year Constructed	**2003**
Location	St. Petersburg, FL
Engineer of Record	Ken Akins Tensar Earth Technologies 404-250-1290
Maximum Height	38 feet
Wall Length	3,700 ft
Wall Area	80,000 SF
Facing	ARES 5x5 Panels
Description	Abutment founded on H- piles. Acute corners as tight as 45 degrees
TIC References	Jeremiah Riggio 404-250-1290
Wall Installer	Bonn-J Contracting 407-977-7666
GC	Jones Bros. 727-577-6200
Owner's Rep	FDOT 813-233-3836

	US 71 Cass County
Year Constructed	**2008**
Location	Rabun, MO
Engineer of Record	Brendan Sheppard Tensar International Corporation 404-250-1290
Maximum Height	23 feet
Wall Length	564 ft
Wall Area	9,000 SF
Facing	ARES 5x5 Panels
Description	Abutment founded on H- piles.
TIC References	Jeremiah Riggio 404-250-1290
Wall Installer	Philips Grading 680-834-3030
GC	Ideker 816-364-3970
Owner's Rep	MODOT 816-622-6500

	U.S. Highway No. 36
Year Constructed	**2007**
Location	Boulder County, Colorado
Engineer of Record	Brendan Sheppard Tensar International Corporation 404-250-1290
Maximum Height	18 feet
Wall Length	106 feet
Wall Area	1800 square feet
Facing	Ares 5'x9' panels
Description	Abutment founded on 3-foot diameter bridge columns. Skewed geogrid utilized on project.
TIC References	George Charalambous 404-250-1290
Wall Installer	Slaton Bros., Inc. 303-806-8160
GC	Hamon Contractors 303-297-0340
Owner's Rep	Colorado Department of Transportation 303-757-9309

	41st Street Overcrossing
Year Constructed	**2005**
Location	City of Everett, Washington
Engineer of Record	Gerry Kehler Tensar International Corporation 404-250-1290
Maximum Height	30 feet
Wall Length	188 feet
Wall Area	3800 square feet
Facing	Ares 5'x5' panels
Description	Bridge approach slab founded on reinforced backfill volume and sleeper slab.
TIC References	George Charalambous 404-250-1290
Wall Installer	Wilder Construction 425-259-9175
GC	Wilder Construction 425-259-9175
Owner's Rep	City of Everett Public Works Department 425-257-8800

	Beach Boulevard
Year Constructed	**2006**
Location	Jacksonville Beach, FL
Engineer of Record	Brendan Sheppard Tensar Earth Technologies 404-250-1290
Maximum Height	12 feet
Wall Length	1,000 ft
Wall Area	18,000 SF
Facing	ARES 5x5 Panels
Description	Abutment founded on concrete piles with tiered walls.
TIC References	Jeremiah Riggio 404-250-1290
Wall Installer	Bonn-J Contracting 407-977-7666
GC	Superior Construction 904-292-4240
Owner's Rep	FDOT 800-207-8236

	Powers Blvd. – Pine Creek
Year Constructed	**2003**
Location	El Paso County, Colorado
Engineer of Record	Ken Akins Tensar Earth Technologies, Inc. 404-250-1290
Maximum Height	32 feet
Wall Length	1400 feet
Wall Area	20,200 square feet
Facing	Ares Full Height Panels
Description	2 abutment walls and 8 wing walls were constructed in 2003. The two bridges were founded on spread footing on top of the reinforced soil mass.
TIC References	Jong Lee 404-250-1290
Wall Installer	Slaton Brothers, Inc. 303-806-8160
GC	URS 719-460-5318
Owner's Rep	Colorado Department of Transportation 303-757-9309

	SH 133
Year Constructed	**2008**
Location	Garfield County, Colorado
Engineer of Record	Brendan Sheppard Tensar International Corporation 404-250-1290
Maximum Height	17.5 feet
Wall Length	198 feet
Wall Area	2768 square feet
Facing	ARES 5x5 Panels
Description	1 Abutment wall. Abutment founded on micropiles.
TIC References	Phil Law 303-429-9511
Wall Installer	SEMA Construction 303-627-2600
GC	SEMA Construction 303-627-2600
Owner's Rep	Colorado Department of Transportation 303-757-9309

	OhioDOT – CR124
Year Constructed	**2010**
Location	Delaware County, Phio
Engineer of Record	Brendan Sheppard Tensar International Corporation 770-344-2090
Maximum Height	44 feet
Wall Length	Approximately 1,800 feet
Wall Area	44,275 square feet
Facing	ARES 5x9 Panels
Description	2 False Abutment wall
TIC References	Jeremiah Riggio 770-344-2090
Wall Installer	Trucco Construction 740-881-5140
GC	Trucco Construction 740-881-5140
Owner's Rep	Ohio Department of Transportation (614) 275-1318

	Route 22
Year Constructed	**2010-2011**
Location	Hillside, NJ
Engineer of Record	Brendan Sheppard Tensar International Corporation 770-344-2090
Maximum Height	30 feet
Wall Length	2,400 feet
Wall Area	60,000 square feet
Facing	ARES 5x5 Panels
Description	4 Abutment walls. Abutment founded on spread footing
TIC References	Jeremiah Riggio 770-344-2090
Wall Installer	Union Paving Construction 908-232-4100
GC	Union Paving Conctruction 908-232-4100
Owner's Rep	New Jersey Department of Transportation 609-530-4231

	Dutchess County Rail Trail
Year Constructed	**2010**
Location	Fishkill, New York
Engineer of Record	Brendan Sheppard Tensar International Corporation 770-344-2090
Maximum Height	22 feet
Wall Length	1000 feet
Wall Area	15,000 square feet
Facing	ARES 5x9 Panels
Description	5 Abutment walls. Abutment founded on spread footing
TIC References	Jeremiah Riggio 770-344-2090
Wall Installer	A. Colarusso & Sons, Inc. 518-828-3218
GC	A. Colarusso & Sons, Inc. 518-828-3218
Owner's Rep	Dutchess County Department of Transportation 845-486-2121

Appendix 5.1.2.1

FHWA report for ARES Tanque Varde Project

ARES® MSE Retaining Wall System

Back to main document

PB90258237

U.S. Department of Transportation
Federal Highway Administration

Experimental Projects Program

EXPERIMENTAL PROJECT 1
Ground Modification Techniques

Technology Transfer

Tensar Geogrid-Reinforced Soil Wall
Grade-Separation Structures on the
Tanque Verde-Wrightstown-Pantano Roads
Intersection
Tucson, Arizona

OFFICE OF HIGHWAY OPERATIONS
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16. Abstract

This report documents relevant information on the design, construction, and performance of Tensar® geogrid-reinforced soil retaining walls used in grade-separation structures on the Tanque Verde-Wrightstown-Pantano Roads Intersection project in Tucson, Arizona. The project represents the first use of geogrid reinforcement in mechanically stabilized earth (MSE) retaining walls in a major transportation-related application in North America.

The reinforced soil walls, which were constructed in 1985, have full height concrete facia panels and range up to 21.9 feet in height. Performance of the walls has been evaluated through an instrumentation and monitoring program funded by the FHWA Demonstration Projects Division.

Five main topics are discussed in the report: Tensar® SR2 geogrid reinforcement; design of the retaining walls including connections between the geogrid and facia panels; construction steps including pretensions of the geogrid layers; instrumentation, both during and after construction; and performance based on field observations.

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FOREWORD

Although the contents of this report describe the performance of Tensar Geogrid materials, the intent of publication is to demonstrate that geogrid-reinforcement soil wall construction is a viable approach. There are many geogrid-reinforcement products available that have similar properties and behaviors that should be considered, as well as Tensar, by the end user.

Additional information may be obtained from AASHTO-AGC-ARTBA Joint Committee, Task Force 27 report entitled, "In Situ Soil Improvement Techniques." Chapters 1 and 2 of this report provide design guidelines and construction specifications for the use of inextensible reinforcements for mechanically stabilized earth walls in permanent applications. Another publication worth consideration is FHWA research report number FHWA-RD-89-043 entitled, "Design and Construction Guidelines for Reinforced Soil Structures."

SI CONVERSION FACTORS (1)

DIMENSION	APPROXIMATE CONVERSIONS TO SI MEASURES			APPROXIMATE CONVERSIONS FROM SI MEASURES		
	WHEN YOU KNOW	MULTIPLY BY	TO FIND	WHEN YOU KNOW	MULTIPLY BY	TO FIND
Length	inches (in.)	2.5	centimeters (cm)	centimeters (cm)	0.39	inches (in.)
	feet (ft)	0.30	meters (m)	meters (m)	3.3	feet (ft)
Area	square inches (in^2)	6.5	square centimeters (cm^2)	square centimeters (cm^2)	0.16	square inches (in^2)
	square feet (ft^2)	0.93	square meters (m^2)	square meters (m^2)	11	square feet (ft^2)
Weight	pounds (lb)	4.5	Newtons (N)	Newtons (N)	0.22	pounds (lb)
	short tons (ton)	8.9	kiloNewtons (kN)	kiloNewtons (kN)	0.11	short tons (ton)
Tension	pounds per foot (lb/ft)	0.015	kiloNewtons per meter (kN/m)	kiloNewtons per meter (kN/m)	69	pounds per foot (lb/ft)
Stress	pounds per square inch (lb/in^2)	0.00033	Pascal (Pa)	Pascal (Pa)	3000	pounds per square inch (lb/in^2)
	pounds per square inch (lb/in^2)	0.048	Pascal (Pa)	Pascal (Pa)	21	pounds per square foot (lb/ft^2)
Density	pounds per cubic foot (lb/ft^3)	0.16	kiloNewtons per cubic meter (kN/m^3)	kiloNewtons per cubic meter (kN/m^3)	6.4	pounds per cubic foot (lb/ft^3)
Temperature	degrees Fahrenheit (°F)	0.56 (after subtracting 32)	degrees Celsius (°C)	degrees Celsius (°C)	1.8 (then add 32)	degrees Fahrenheit (°F)

Note: (1) More exact conversions can be found in ASTM E 380.

DESIGN, CONSTRUCTION, AND PERFORMANCE

OF

TENSAR GEOGRID-REINFORCED SOIL WALLS

AT

TANQUE VERDE-WRIGHTSTOWN-PANTANO ROADS

TUCSON, ARIZONA

January 1989

TABLE OF CONTENTS

EXECUTIVE SUMMARY.....i

1. INTRODUCTION.....1

1.1 Project Background.....1
1.2 Project Overview.....2
1.3 Report Overview.....3

2. SELECTION OF GEOGRID PROPERTIES FOR DESIGN.....5

2.1 Introduction.....5
2.2 Selection of Allowable Design Tension.....5
2.3 Temperatures in Reinforced Soil Walls.....6

2.3.1 Method of Analysis.....6
2.3.2 Assumptions.....6
2.3.3 Temperature Evaluation.....7
2.3.4 Computation for Wall 26-30.....8

2.4 Effect of Temperature on Geogrid Properties.....8

3. WALL DESIGN.....16

3.1 Introduction.....16
3.2 Preliminary Design Steps.....16

3.2.1 Problem Definition.....16

3.2.1.1 Geometry.....16
3.2.1.2 Loading.....18
3.2.1.3 Performance Requirements.....18
3.2.1.4 Design Life.....19

3.2.2 Design Assumptions Regarding Material Properties.....19
3.2.3 Parameters to be Determined.....20

3.3 Stability Analyses.....20
3.4 Internal Stability Analysis.....21

3.4.1 Method of Analysis.....21
3.4.2 Selection of Analysis Method.....22

3.4.3 Determination of Horizontal Stresses.....23
3.4.4 Selection of Reinforcement Spacing.....25
3.4.5 Verification of Pullout Resistance.....27

3.4.5.1 Available Anchorage Length.....27
3.4.5.2 Required Anchorage Length.....28
3.4.5.3 Final Reinforcement Length.....29

3.4.6 Geogrid Layout.....29

3.5 External Stability Analysis.....29

3.5.1 Introduction.....29
3.5.2 Failure Due to Sliding.....30
3.5.3 Failure Due to Overturning.....31
3.5.4 Failure by Lack of Bearing Capacity.....32
3.5.5 Rotational Slip Surface Instability.....32
3.5.6 Final Geogrid Layout.....32

3.6 Design of Structural Components.....32

3.6.1 Introduction.....32
3.6.2 Development of Connection System.....33

4. WALL CONSTRUCTION.....42

4.1 Wall Components.....42
4.2 Construction Sequence.....43

5. INSTRUMENTATION PROGRAM.....57

5.1 Introduction.....57
5.2 Location of Instruments.....57
5.3 Description of Instruments.....57

5.3.1 Pneumatic Load Cells.....57
5.3.2 Resistance Strain Gages.....58
5.3.3 Inductance Coils.....58
5.3.4 Surveying Devices.....58
5.3.5 Resistance Thermometers.....58

5.4 Calibration of Instruments.....59
5.5 Preparation of Instruments.....59

5.6 Installation of Instruments.....60
5.7 Monitoring of Instruments.....60

6. INSTRUMENTATION RESULTS.....71

6.1 Introduction.....71
6.2 Soil Stresses.....71

6.2.1 Vertical Soil Stresses.....71
6.2.2 Horizontal Soil Stresses.....72

6.3 Soil Strains.....73

6.3.1 Vertical Soil Strains.....73
6.3.2 Horizontal Soil Strains.....74

6.4 Geogrid Strains.....74

6.4.1 Reinforcement Strains During Construction.....74
6.4.2 Adjusted Reinforcement Strains.....75
6.4.3 Connection Strains.....76

6.5 Panel Movements.....76
6.6 Temperature.....77

7. DISCUSSION OF WALL PERFORMANCE.....102

7.1 Introduction.....102
7.2 Facing Panel Movements.....102
7.3 Tension and Strain in the Geogrid.....102
7.4 Deformation Analysis.....103
7.5 Time Effects.....104
7.6 Temperature Effects.....105
7.7 Conclusions.....105

8. RECENT DEVELOPMENTS IN DESIGN AND CONSTRUCTION.....112

8.1 Introduction.....112
8.2 Selection of Allowable Design Tension.....112
8.3 Wall Design.....114
8.4 Wall Construction.....115

REFERENCES.....117

LIST OF TABLES

Table 2-1. TENSAR SR2 geogrid specification.....11

Table 5-1. Load cell locations.....62

Table 5-2. Resistance strain gage locations.....63

Table 5-3. Inductance coil locations.....64

Table 5-4. Resistance thermometer locations.....65

Table 5-5. Estimated field accuracy of each of the instruments.....66

Table 5-6. Instrument monitoring schedule.....67

Table 6-1. Measured strains in the looped geogrid tab.....79

Table 6-2. Movement of survey points on wall 26-30.....80

Table 6-3. Movement of survey points on wall 26-32.....81

Table 7-1. Maximum geogrid strain in walls 26-30 and 26-32 within 2.5 ft. of the wall face.....107

LIST OF FIGURES

Figure 1-1. Plan view of the Tanque Verde Road project site...4

Figure 2-1. Illustration of TENSAR SR2 geogrid geometry.....12

Figure 2-2. Estimated temperature distribution behind a retaining wall concrete facing using a one-dimensional transient heat flow analysis....13

Figure 2-3. Temperature increases on the surface of exposed concrete panels due to solar radiation.....14

Figure 2-4. Distribution of predicted maximum temperature for wall 26-30, Tanque Verde Road Project, Tucson, Arizona.....15

Figure 3-1. Design steps for the reinforced soil wall.....35

Figure 3-2. Geometric and loading conditions of a reinforced soil wall.....36

Figure 3-3. Potential failure surfaces considered in the internal stability analysis of the reinforced soil walls.....36

Figure 3-4. Three possible stress distributions at the base of a reinforced soil wall.....37

Figure 3-5. Family of curves giving the minimum required spacing between reinforcement layers as a function of depth below the top surface for a series of tentative reinforcement lengths for wall 26.....37

Figure 3-6. Illustration of allowable anchorage length for a Rankine failure surface.....38

Figure 3-7. Final geogrid layout for wall panels 26-30 and 26-32.....39

Figure 3-8. Failure mechanisims considered in an external stability anslysis of a reinforced soil wall....40

Figure 3-9. Development of the facing panel-to-geogrid connection system.....41

Figure 4-1. Four different wall geometries used on the Tanque Verde Road project.....47

Figure 4-2. Components of the Tanque Verde Road MSE wall system.....48

Figure 4-3. Illustration of geogrid looped connection used on the Tanque Verde Road project.....48

Figure 4-4. Leveling bolt detail for the Tanque Verde Road project.....49

Figure 4-5. Vertical joint detail for the Tanque Verde Road project.....49

Figure 4-6. Concrete leveling pad detail for the Tanque Verde Road project.....50

Figure 4-7. Erection brace detail for the Tanque Verde Road project.....50

Figure 5-1. Locations of instruments for wall panel 26-30....68

Figure 5-2. Locations of instruments for wall panel 26-32....69

Figure 5-3. Location of survey points on wall faces.....70

Figure 6-1. Comparison of measured vertical soil stresses in wall 26-32 at a distance of 5 ft. from the leveling pad with the stresses obtained from design calculations.....82

Figure 6-2. Comparison of measured horizontal soil stresses in wall 26-30 with the stresses obtained from design calculations.....83

Figure 6-3. Measured vertical soil strains obtained from magnetic inductance coils in wall 26-30 at a distance of 3.5 ft. from the leveling pad.....84

Figure 6-4. Measured vertical soil strains obtained from magnetic inductance coils in wall 26-32 at a distance of 3.5 ft. from the leveling pad.....85

Figure 6-5. Measured horizontal soil strains obtained from magnetic inductance coils in wall 26-30 at a distance of 8 ft. from the leveling pad.....86

Figure 6-6. Measured horizontal soil strains obtained from magnetic inductance coils in wall 26-32 at a distance of 8 ft. from the leveling pad.....87

Figure 6-7. Measured geogrid strains in wall 26-30 at a distance of 1.5 ft. from the leveling pad.....88

Figure 6-8. Measured geogrid strains in wall 26-30 at a distance of 4.5 ft from the leveling pad.....89

Figure 6-9. Measured geogrid strains in wall 26-30 at a distance of 14.5 ft. from the leveling pad.....90

Figrue 6-10. Adjusted measured geogrid strains in wall 26-30 on 20 October 1985. The strains were measured with magnetic inductance coils.....91

Figure 6-11. Adjusted measured geogrid strains in wall 26-32 on 20 October 1985. The strains were measured with magnetic inductance coils.....92

Figure 6-12. Adjusted measured geogrid strains in wall 26-30 on 20 October 1985. The strains were measured with foil-type resistance strain gages.....93

Figure 6-13. Adjusted measured geogrid strains in wall 26-32 on 20 October 1985. The strains were measured with foil-type resistance strain gages.....94

Figure 6-14. Adjusted measured geogrid strains in wall 26-30 on 8 November 1986. The strains were measured with magnetic inductance coils.....95

Figure 6-15. Adjusted measured geogrid strains in wall 26-32 on 8 November 1986. The strains were measured with magnetic inductance coils.....96

Figure 6-16. Adjusted measured geogrid strains in wall 26-30 on 8 November 1986. The strains were measured with foil-type resistance strain gages.....97

Figure 6-17. Adjusted measured geogrid strains in wall 26-32 on 8 November 1986. the strains were measured with foil-type resistance strains gages.....98

Figure 6-18. Measured temperatures in wall 26-30.....99

Figure 6-19. Measured temperatures in wall 26-32.....100

Figure 6-20. Comparison of temperatures measured on the Tucson wall project on 15 June 1986 with those predicted using a one-dimensional transient heat flow analysis.....101

Figure 7-1. Isochronous tension-strain curve for TENSAR SR2 geogrid.....108

Figure 7-2. Plot of maximum strain versus wall location for walls 26-30 and 26-32 on 8 November 1986.....109

Figure 7-3. Estimated deformations of wall facing panel 26-30.....110

Figure 7-4. Estimated deformations of wall facing panel 26-32.....111

LIST OF PHOTOGRAPHS

Photograph 4-1. Casting of geogrid tabs.....51

Photograph 4-2. Precast facing panels with geogrid tabs embedded in the back face.....51

Photograph 4-3. Hoisting of the precast facing panels.....52

Photograph 4-4. Placement of the precast facing panels.....52

Photograph 4-5. Leveling bolts used to align the facing panels.....53

Photograph 4-6. Adjustable erection braces used to support and align facing panels during construction.....53

Photograph 4-7. Placement of concrete grout between the facing panel and leveling pad.....54

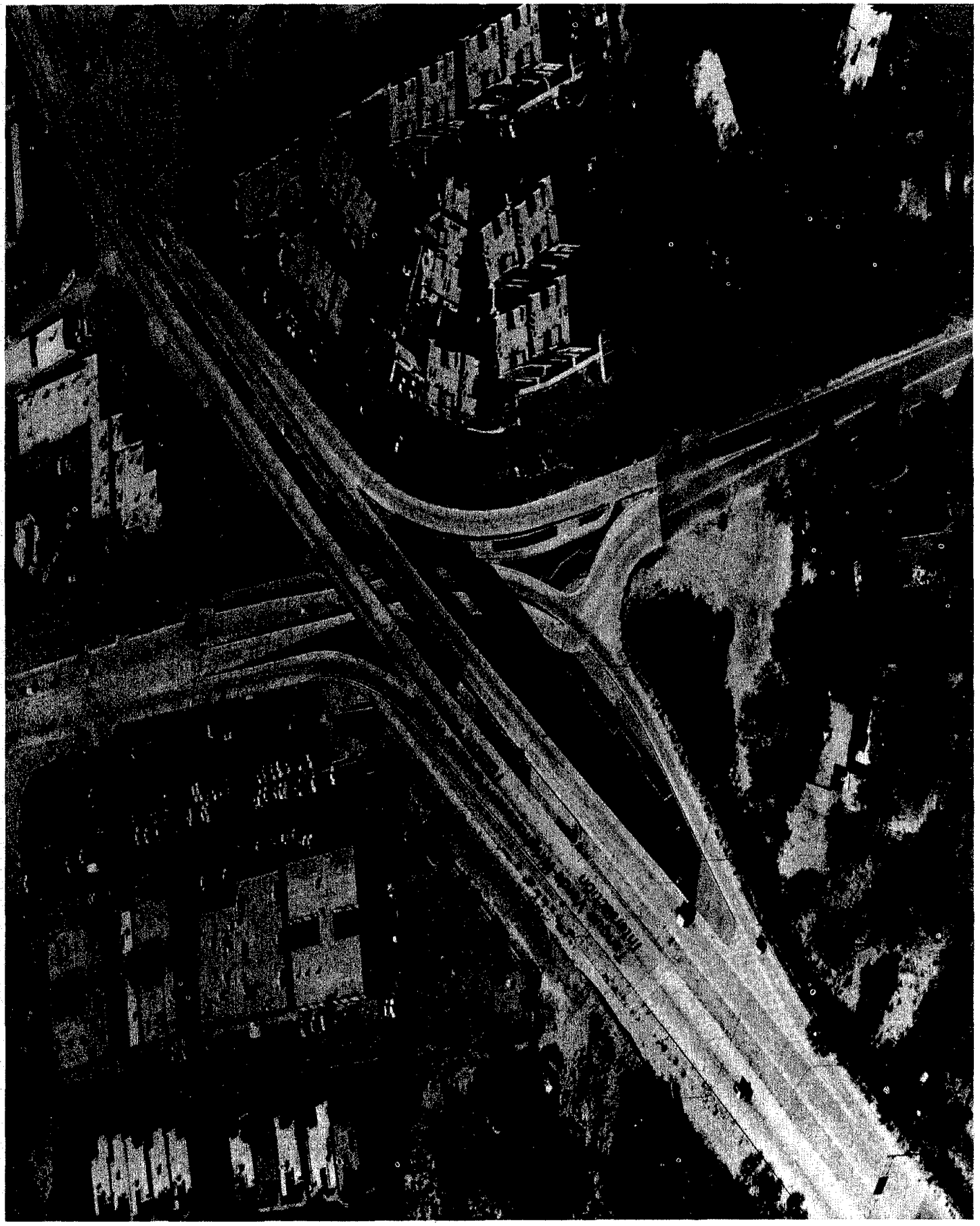
Photograph 4-8. Connection of geogrid tabs and geogrid reinforcement using PVC pipe connector.....54

Photograph 4-9. Wood wedge used to pretension the geogrid looped connection.....55

Photograph 4-10. Steel forks used to pretension the geogrid reinforcement.....55

Photograph 4-11. Placement of granular fill over a layer of geogrid.....56

Photograph 4-12. Finished Tanque Verde Road project MSE wall with sandblasted panel finish.....56



EXECUTIVE SUMMARY

The purpose of this report is to compile in one document, relevant information on the design, construction, and performance of the TENSAR ® geogrid-reinforced soil retaining walls used for grade-separated interchanges on the Tanque Verde-Wrightstown-Pantano Roads project (Tanque Verde Road project) in Tucson, Arizona. This project represents the first use of geogrid reinforcement in mechanically stabilized earth (MSE) retaining walls in a major transportation-related application in North America. The reinforced soil walls, which were constructed in 1985, have full-height concrete facing panels and range up to 21.9 ft in height. Performance of the walls has been evaluated through an instrumentation and monitoring program funded by the Federal Highway Administration (FHWA) Demonstration Projects Program.

Five main topics are addressed in this report: geogrid reinforcement, design, construction, instrumentation, and performance.

Geogrid Reinforcement. TENSAR SR2 geogrid was used as the soil reinforcement material for the Tanque Verde Road project. The report describes the selection of the allowable design tension for this geogrid, including the influence of time-dependent creep and construction-induced damage. Also evaluated is the effect of the summertime temperatures of the Sonoran Desert environment of Tucson on the properties of the polymer geogrid.

Design. The report presents the design of the retaining walls, including the analysis of internal and external stability. In addition, the report briefly discusses design of the facing panels and the connections between the geogrid reinforcement and the facing panels.

Construction. The principal components of the walls and the construction steps are discussed in this report. The wall components include geogrid reinforcement, concrete facing panels, leveling pads, geogrid tabs, and PVC pipe connections. The report also describes the main construction steps: preparing the foundation, casting the leveling pads, precasting the full-height concrete facing panels, placing and bracing the facing panels, connecting geogrid reinforcement to the facing panels, pretensioning the geogrid layers, backfilling over the geogrid reinforcement, compacting the fill, and, lastly, casting traffic barriers on top of the panels.

Instrumentation. The report documents the results of monitoring instrumentation installed in the soil and on the reinforcement behind two wall panels constructed in September 1985. The reports presents an evaluation of the performance of the geogrid-reinforced soil walls through measured stresses, strains, and displacements of the instrumented wall sections, both during and after construction.

Performance. In the report, the following conclusions are drawn from field observations and from the results of the instrumentation and monitoring program.

- The wall performance has been satisfactory.

- The maximum measured (corrected) strain in the geogrid reinforcement is very low (about 0.8%) and corresponds to a geogrid tension which is approximately 10% of the allowable design tension (i.e., the factor of safety against geogrid reinforcement rupture is 10). Thus, the wall design appears to have been conservative.

- Measured time-dependent geogrid strains (i.e., geogrid creep) are negligible. Predicted time-dependent strains are negligible for the design life.

- An initial wall batter of 1H:60V appears to be appropriate for full-height, concrete-faced, geogrid-reinforced soil walls similar to those constructed on this project.

- Panel movements during construction (i.e., 1H:60V) appear to be due more to initial slack in the geogrid reinforcement-to-panel connection behind the wall facing panel than to geogrid reinforcement strain. Thus, consistent tensioning (to remove initial slack) of the connection is an important construction step.

- The evidence to date indicates that the elevated summertime temperatures of the Sonoran Desert environment have no significant effect on the performance of the TENSAR geogrid reinforcement behind the wall facing.

1. INTRODUCTION

1.1 Project Background

This report was prepared by Ms. B.A. Gross, E.I. and Dr. R. Bonaparte, P.E. of GeoServices Inc. Consulting Engineers (GeoServices), Norcross, Georgia. The goal of the report is to compile in one document, relevant information on the design, construction, and performance of the TENSAR ® geogrid-reinforced soil retaining walls used on the Tanque Verde-Wrightstown-Pantano Roads project in Tucson, Arizona (Tanque Verde Road project). This report draws heavily on the early reports of Desert Earth Engineering (Desert Earth), Tucson, Arizona [1986, 1987], Dames and Moore [1985], Nelton [1984], and Bonaparte [1987a].

For the Tanque Verde Road project, forty-three geogrid-reinforced soil walls were constructed. These walls represent the first use of geogrid reinforcement along with precast concrete facing panels in a major transportation-related, mechanically-stabilized earth (MSE) application in North America. Under the auspices of the Federal Highway Administration (FHWA) Demonstration Projects Program (Work Order No. DTFH71-85-501-AZ-02), FHWA and The Tensar Corporation (Tensar), Morrow, Georgia, jointly funded a grant to monitor construction and performance of the walls. The grant was made to the Pima County Department of Transportation and Flood Control District (Pima County) who in turn contracted with Desert Earth to perform an instrumentation and monitoring program and prepare a report summarizing the results. The actual field instrumentation program was carried out by Desert Earth with major assistance from Professor C.S. Desai and his graduate assistant, K. Fishman, of the University of Arizona, Tucson.

The Tanque Verde Road project history currently spans more than four years, beginning with the project's design in 1984 and progressing through the latest monitoring program activities in 1988. The project was initiated in 1984 when personnel from Tensar approached the M.M. Sundt Corporation (Sundt), Tucson, Arizona, the General Contractor on the Tanque Verde Road project, with a request to consider geogrid-reinforced MSE retaining walls on the project in lieu of the originally specified steel-reinforced MSE walls. Sundt prepared a value-engineering submittal to Pima County requesting that geogrid-reinforced MSE walls be substituted for the steel-reinforced MSE walls. Tensar retained the Earth Structures Design Division of Dames and Moore (Dames and Moore), Phoenix, Arizona, to design the

geogrid-reinforced MSE walls for the value-engineering submittal. Dames and Moore subsequently became the Engineer-of-Record for the retaining walls. Dames and Moore retained Western Technologies Inc. (WTI) to conduct specialized laboratory and field studies of the geogrids. In particular, under the direction of Dames and Moore, WTI performed laboratory tests on geogrid "connections" to evaluate the tension-displacement characteristics of the connections and field studies to investigate how hot the thermoplastic geogrids might become in the fill behind the concrete facing panel during the summertime in the Sonoran Desert environment. (The laboratory tests performed by WTI were also used to verify the adequacy of the pullout resistance of the geogrid tabs embedded in the concrete panels.) In addition, WTI and Desert Earth performed the original geotechnical studies for the project. The general Civil Engineer for the project was Holban, Martin and Meza, Tucson, Arizona.

Several reports were prepared in connection with the Tanque Verde Road project. These reports are included in the References section at the end of this report. All of these reports have been carefully reviewed, and the most germane information from the reports has been compiled for this document.

1.2 Project Overview

Forty-three separate MSE (mechanically-stabilized earth) walls were constructed on the Tanque Verde Road project. The walls are situated within limited rights-of-way at two grade-separated interchanges at the intersections of Tanque Verde Road with both Wrightstown and Pantano Roads in northeast Tucson, Arizona. A plan view of the project site is shown in Figure 1-1. The walls extend for approximately 5200 lineal ft and vary in height from 1.1 to 21.9 ft. The wall facings on the project are full-height, precast concrete panels. For the forty-three walls, a total of 561 individual concrete panels were cast and used. The walls on the Tanque Verde Road project were built between November 1984 and October 1985.

Two of the wall panels on the Tanque Verde Road project were instrumented in September 1985 and have been monitored by Desert Earth since. The goals of the instrumentation program are to determine stresses and strains within the reinforced soil mass, movements of the wall panels, external and internal wall temperatures, and to make an overall assessment of the performance of the wall system. Wall panels 26-30 and 26-32 were selected for instrumentation because they are the highest wall sections on the project

that are independent of three-dimensional effects near the bridge abutment areas and changes in foundation elevations. The location of these panels is shown in Figure 1-1.

1.3 Report Overview

The report presents a description of the design, construction, and performance of the TENSAR geogrid-reinforced MSE walls constructed on the Tanque Verde Road project in Tucson, Arizona. The report begins with the design of the walls, then addresses construction, and finally, performance. At the end of the report, a brief summary is provided of developments in geogrid-reinforced soil wall technology since the time of the Tanque Verde Road project. The organization of the report is outlined below.

Chapters 2 and 3 are concerned with the design of the MSE walls. Chapter 2 presents the selection of geogrid properties for design and considers the effects of elevated temperatures and creep on geogrid tension-strain characteristics. Chapter 3 presents the design of the reinforced soil walls carried out by Dames and Moore [1984] using a design methodology presented in Netlon [1984].

Chapter 4 presents the construction of the reinforced soil walls and includes a description of the wall components and a summary of the wall construction sequence.

Chapter 5 presents the wall instrumentation program that was implemented and documented by Desert Earth [1986]. The chapter includes a description of the instruments and discussions of the locations, preparation, installation, and monitoring of the instruments.

Chapter 6 presents the results of the instrumentation program.

Chapter 7 presents a discussion on the performance of the walls and the geogrid reinforcement based on the instrumentation results.

Chapter 8 summarizes developments in design and construction of geogrid-reinforced soil walls since the time of the Tanque Verde Road project. Chapter 8 also presents recommendations for the design, construction, and instrumentation of future geogrid-reinforced soil walls.

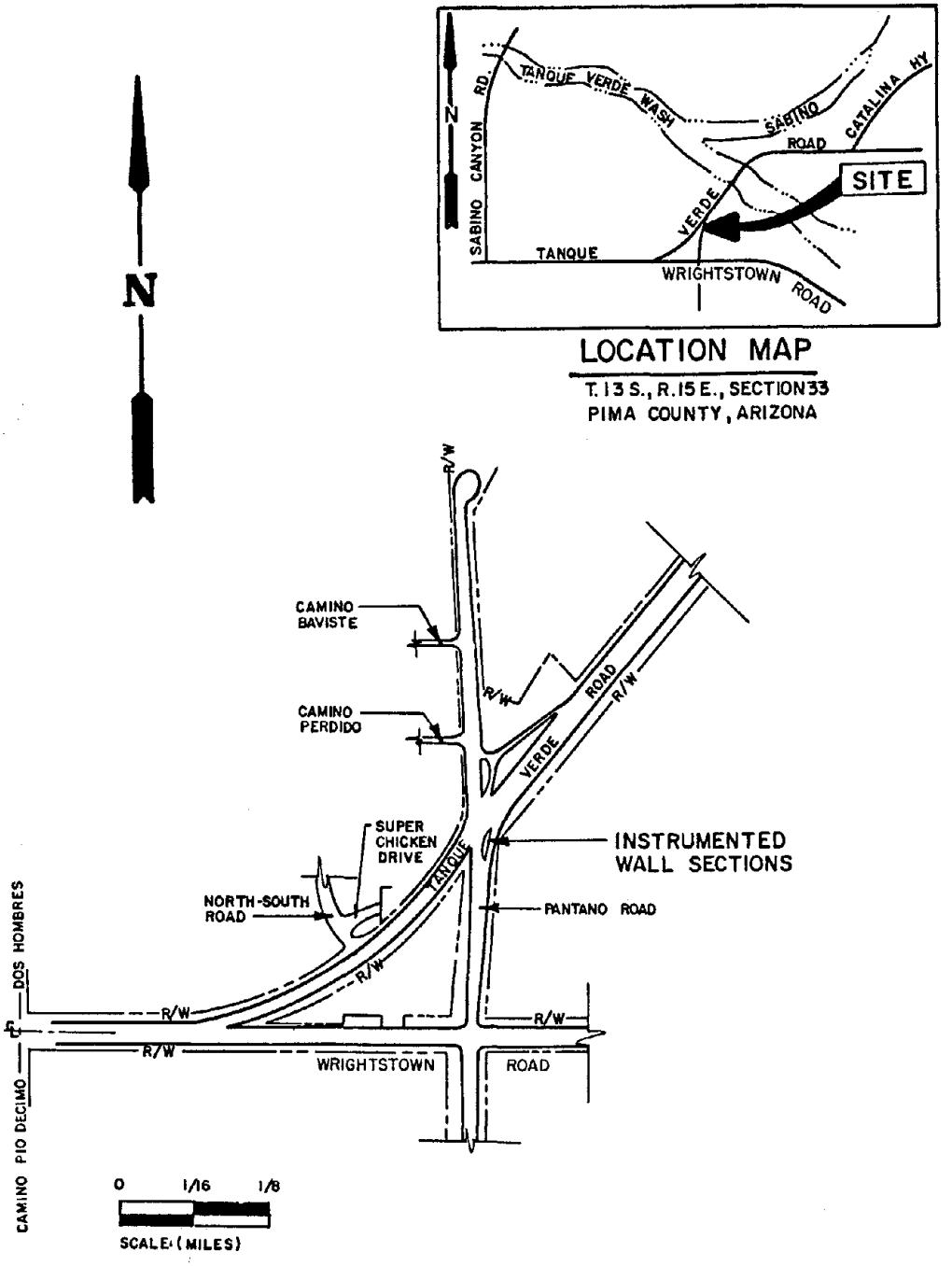


Figure 1-1. Plan view of the Tanque Verde Road project site.

2. SELECTION OF GEOGRID PROPERTIES FOR DESIGN

2.1 Introduction

TENSAR SR2 geogrid was used as the soil reinforcement material in Sundt's value-engineering submittal. The TENSAR SR2 geogrid is a uniaxial product (Figure 2-1) that is manufactured from high density polyethylene (HDPE) stabilized with about 2.5% carbon black to provide resistance to attack by ultraviolet (UV) light. This product has an index tensile strength (i.e., short-term tensile strength) of about 5400 lb/ft based on the wide strip tensile test (ASTM D4595). A specification sheet for this product is given in Table 2-1.

The index tensile strength of the geogrid is not suitable for design calculations for several reasons. First, HDPE is a thermoplastic polymer which undergoes time-dependent deformations (i.e., "creep") at high stress levels. Therefore, the tension in the geogrid must be kept below a creep threshold. Second, the influence of construction-induced geogrid damage must be considered.

The steps taken by Dames and Moore [1984] to estimate the allowable design tension of TENSAR SR2 geogrid are described in Section 2.2. The procedure used by Dames and Moore was based on the guidelines presented in Netlon [1984].

A further consideration on the Tanque Verde Road project was the influence of the summertime temperatures in the Sonoran Desert environment of Tucson on the HDPE geogrid properties. The effect of temperature on geogrid properties is described in Sections 2.3 and 2.4.

2.2 Selection of Allowable Design Tension

For the Tanque Verde Road project, the allowable design tension of the geogrid was based on creep considerations, which are a function of the geogrid polymer type and manufacturing process, and on the effects of construction on the geogrid, which is a function of the wall fill characteristics (i.e., mean grain size, angularity, etc.). The allowable design tension was determined by dividing the allowable tension based on creep considerations by a factor of safety (FS_t) to account for construction effects (i.e., damage associated with the construction procedure). This

factor of safety also accounted for any uncertainties in the structure geometry, soil properties, and the surcharge magnitude.

For TENSAR SR2 geogrid, the allowable tension based on creep considerations is reported to be 1,986 lb/ft at 10% strain after 120 years, based on tests performed by McGown, et al. [1984]. This long-term allowable tension value is approximately 37% of the short-term strength of the geogrid (i.e., 5400 lb/ft). The factor of safety was taken as FS_t = 1.5, which is appropriate for a clean gravelly sand based on guidelines given by Netlon [1984]. Therefore, the allowable design tension of the geogrid was:

$$1,986/1.5 = 1,324 \text{ lb/ft}$$

Since the time of design for the Tanque Verde Road project, Bonaparte and Berg [1987] have presented a more formal procedure to determine the allowable design tension. This method is discussed in Section 8.2.

2.3 Temperatures in Reinforced Soil Walls

2.3.1 Method of Analysis

Given the relatively high temperatures in Tucson in the summertime and the thermoplastic nature of HDPE, there was concern about the effects of elevated temperatures on the HDPE geogrid. The work done on this subject by Tensar for the Tanque Verde Road project was subsequently summarized by Bonaparte [1987a].

Tensar used a one-dimensional transient heat flow analysis to estimate temperatures in the reinforced soil wall. The analysis results gave a prediction of temperatures within the reinforced soil mass as a function of ambient air temperature, solar radiation, and the thermal properties of soil and concrete. With these predicted temperatures, the performance of the geogrid was evaluated, based on the results of elevated-temperature tension tests.

2.3.2 Assumptions

Several simplifying assumptions were made in the analysis to predict the temperatures behind the retaining walls.

- The heat flow within the reinforced soil mass was assumed to be entirely horizontal. This assumption ignores heating of the reinforced soil mass along its upper horizontal boundary.

- The concrete facing panels were assumed to have the following properties: thickness = 6 in.; moist unit weight = 155 lb/ft^3; thermal conductivity = 1.5 $Btu/hr/ft^2/°F$; and volumetric heat = 0.23 Btu/lb/°F. (The thermodynamic properties of concrete were taken as the average values reported in Townsend [1981] for twenty-five different concrete dams.)

- The granular wall backfill was assumed to have the following characteristics: moist unit weight = 125 lb/ft^3; dry unit weight = 120 lb/ft^3; moisture content = 4%; thermal conductivity = 1.07 $Btu/hr/ft^2/°F$; and volumetric heat = 0.21 Btu/lb/°F. (The soil thermal conductivity was estimated from equations provided in Mitchell [1976]. The soil volumetric heat was estimated from data presented in Mitchell [1976].)

- The variation of ambient air temperature with time of day was simplified into two 8-hour step functions centered around the monthly average temperature. One step corresponded to daytime heating, the other corresponded to nighttime cooling.

2.3.3 Temperature Evaluation

The governing equation for one-dimensional transient heat flow was solved by making the assumptions given in Section 2.3.2. The complete solution can be found in Bonaparte [1987a]. The analysis results are summarized in Figure 2-2. The curves correspond to a "general" solution and require two boundary conditions to develop a site-specific solution. The two boundary conditions are: (i) the concrete surface temperature equals T_s; and (ii) the average monthly ambient temperature equals T_{ave}. Figure 2-2 was used to estimate the temperature distribution within the reinforced soil walls on the Tanque Verde Road Project.

2.3.4 Computation for Wall 26-30

For the design, the temperature distribution corresponding to the peak summertime conditions was predicted. As an illustration of how this was done, wall panel 26-30 is considered here. A vector normal to this wall facing panel would point approximately due west.

The concrete surface temperature, T_s, was assumed to be equal to the mean monthly maximum daily temperature, T_{max}, plus an additional temperature increment due to solar heating of the concrete panels, ΔT. From the U.S. Department of Commerce [1988], the mean monthly maximum daily temperature in Tucson, Arizona in July is T_{max} = 98°F and the average monthly temperature is T_{ave} = 86°F. From Figure 2-3, the temperature increase on the surface of a westward facing, vertical concrete panel due to solar radiation is ΔT = 6°F. Therefore, the maximum temperature on the surface of the concrete panel was predicted to be $T_s = T_{max} + \Delta T$ = 98°F + 6°F = 104°F.

Using Figure 2-2, and the boundary conditions described above, a predicted temperature profile was drawn. This profile is shown in Figure 2-4. From this figure, the maximum predicted soil temperature for the considered conditions was about 95°F. The average predicted temperature in the first 6 in. of soil behind the wall facing was about 92°F.

2.4 Effect of Temperature on Geogrid Properties

In the previous section, it was shown that geogrid temperatures in the range of 90°F to 95°F were anticipated on the Tanque Verde Road project. As previously noted, the HDPE geogrid exhibits viscoelastic behavior. The strength and tension-strain behavior of an HDPE geogrid are, therefore, time and temperature dependent.

The TENSAR geogrid strength properties reported in Section 2.2 were from tests conducted at approximately 68°F. The effects of higher temperatures on this geogrid's strength properties were evaluated using results from three types of laboratory tests:

(1) short-term tests at elevated temperatures;

(2) short-term tests after prolonged exposure to elevated temperatures; and

(3) long-term creep tests at elevated temperatures.

These tests and the associated results have been described in detail by Bonaparte [1987a]. The results are briefly summarized below.

Short-term geogrid tension tests at elevated temperatures show that there is a decrease in short-term strength with increasing temperature. This decrease is approximately 20% for geogrid temperatures of about 100°F.

Short-term geogrid tension tests performed after prolonged geogrid exposure to elevated temperatures show that there is no change in geogrid strength after up to 30 days exposure to a temperature of 158°F.

Long-term geogrid creep tests carried out at 104°F (rather than the standard temperature of 68°F), show that creep behavior is influenced by temperature. The higher the temperature, the larger the strain at a given load and load duration. In addition, the results appear to show that the reduction in long-term allowable geogrid tension at a given elevated temperature is about the same as the reduction in short-term geogrid strength at the same temperature.

The results of the test data show that the strength and modulus of TENSAR geogrids decrease with increasing temperature. For a given duration of load application, the reduction in strength is approximately 0.67% for every degree Fahrenheit. However, because the geogrids are exposed to the elevated temperatures for relatively short periods (i.e., 6 to 12 hours a day for several months out of the year), their creep potential is limited. In the analysis of the test data [Bonaparte, 1987a], it was shown that the allowable geogrid tension at 68°F for 120 years (i.e., 1986 lb/ft) is about equivalent to the same allowable geogrid tension at 95°F for 210 hours, 90°F for 1300 hours, or 85°F for 7200 hours.

Based on the above analysis, it was concluded that the high temperatures of Tucson, Arizona should have no significant impact on the safety or performance of the geogrid-reinforced soil walls. However, given the limited

data base upon which this conclusion was drawn, it was judged worthwhile to attempt to define any relationship between temperature and geogrid strain in the reinforced soil wall monitoring program.

Table 2-1. TENSAR SR2 geogrid specification from The Tensar Corporation, Morrow, GA.

PROPERTY	TEST METHOD	VALUE
Reinforcement and Interlock		
o tensile modulus		
o MD dimension	ASTM D 638 Modified(1)	75,000 lb/ft (min)
o apertures		
o MD dimension	I.D. Calipered	3.9 in (nom)
o CMD dimension	I.D. Calipered	0.6 in (nom)
o open area	COE Method Modified(2)	55 % (nom)
o junctions		
o thickness	O.D. Calipered	0.18 in (nom)
o strength	ASTM D 638 Modified(3)	80 % (min)
o long term design load		
o MD dimension	Note (4)	2000 lb/ft (nom)
Material		
o high density polyethelene	ASTM D 1248 Type III/Class A/Grade 5	97.5 % (nom)
o carbon black	ASTM 4218	2.5 % (nom)
Dimensions		
o roll length		98 ft
o roll width		3.3 ft & 4.6 ft
o roll weight		61 lb & 85 lb

Notes:

1. Secant modulus at 2% elongation measured by tensile loading test ASTM D 638 modified to clamp single ribs of the grid structure at junctions and apply a constant rate of extension of the rib of 2 inches per minute at a temperature of 68° F. No offset allowances are made in caluclating secant modulus. MD dimension is along roll length. CMD dimension is across roll width.

2. Percent open area measured without magnification by Corps of Engineers method as specified in CW 02215 Civil Works Construction Guide, November 1977.

3. Junction strength is an index measure of the ability of the junction of a grid structure to resist shear and tensile stresses. Junction strength is measured as a percent of ultimate single rib tensile strength by tensile loading test ASTM D 638 modified to clamp the horizontal and vertical ribs of a "T" shaped specimen, with the grid junction forming the cross of the "T", and with a constant rate of extension of the specimen applied across the junction at a rate of 2 inches per minute at a temperature of 68° F.

4. Long term design load is the maximum sustained tensile force recommended for the geogrid in permanent engineered structures to ensure that deformation of the structure remains within acceptable limits over its service life. It is determined by measuring deformation of the geogrid at various loadings in 1000 hour constant load tensile tests which correlate to deformations expected over a 100 year service life.

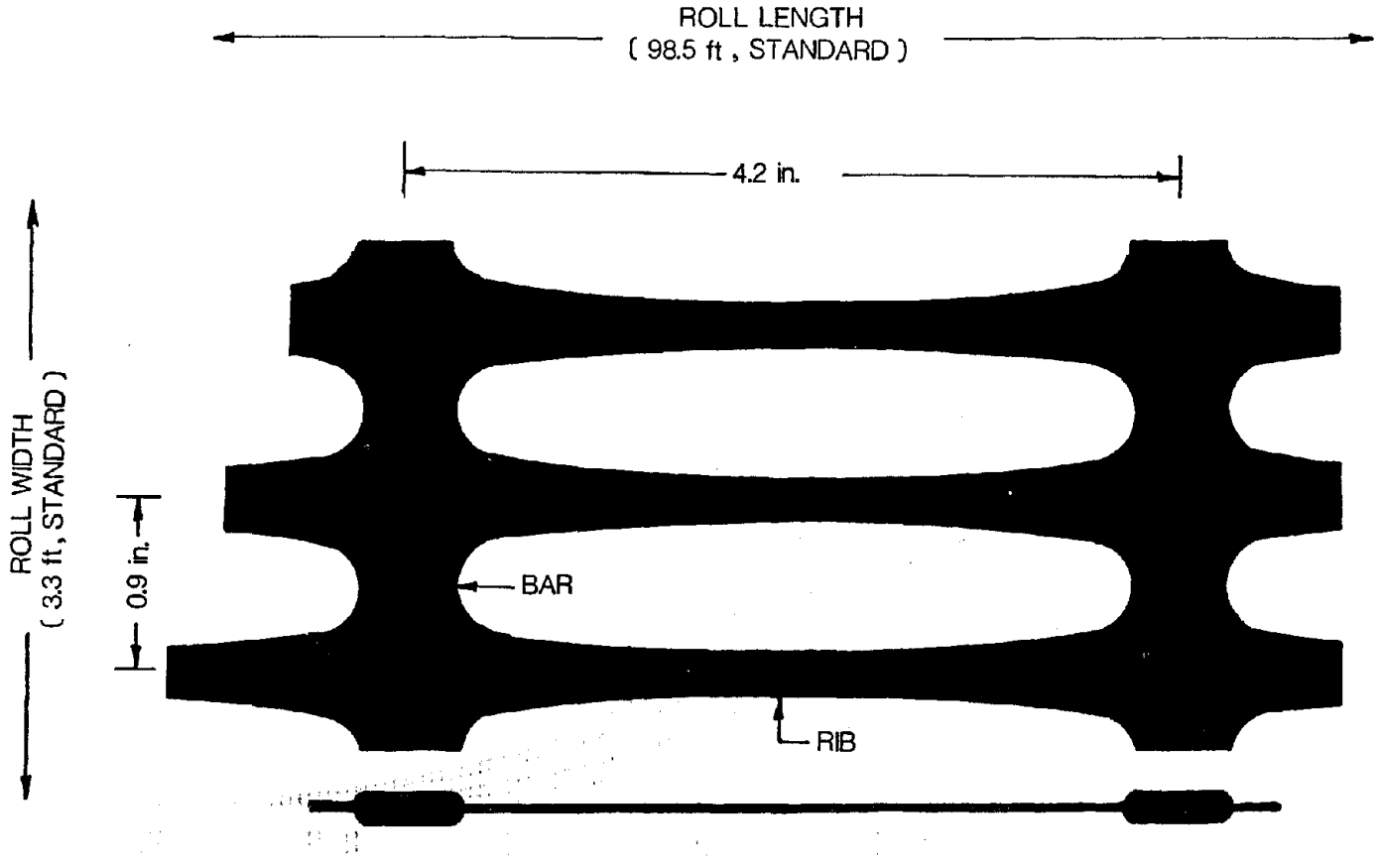


Figure 2-1. Illustration of TENSAR SR2 geogrid geometry. (Drawing is full scale; dimensions are approximate.)

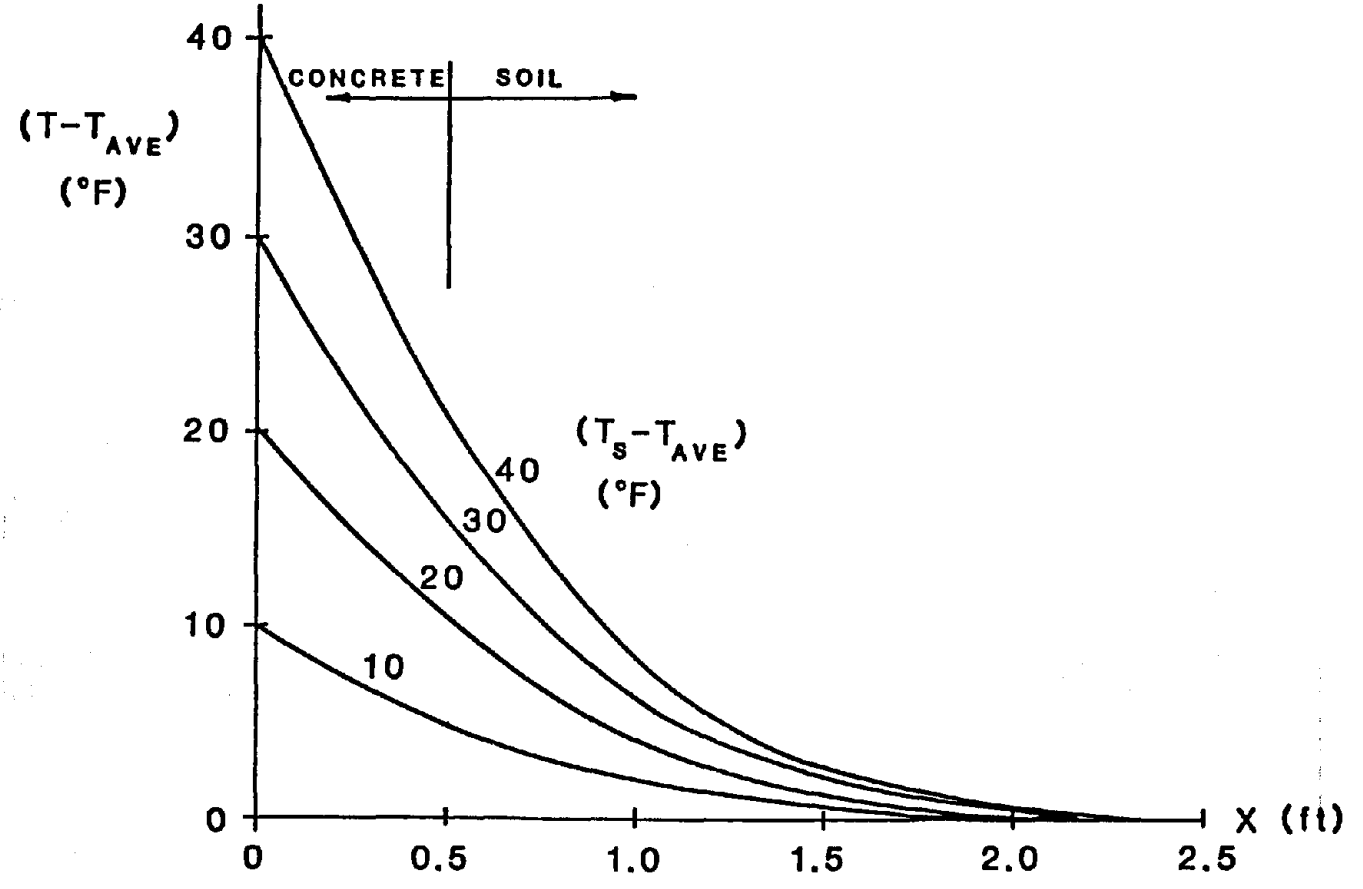


Figure 2-2. Estimated temperature distribution behind a retaining wall concrete facing using a one-dimensional transient heat flow analysis [from Bonaparte, 1987a].

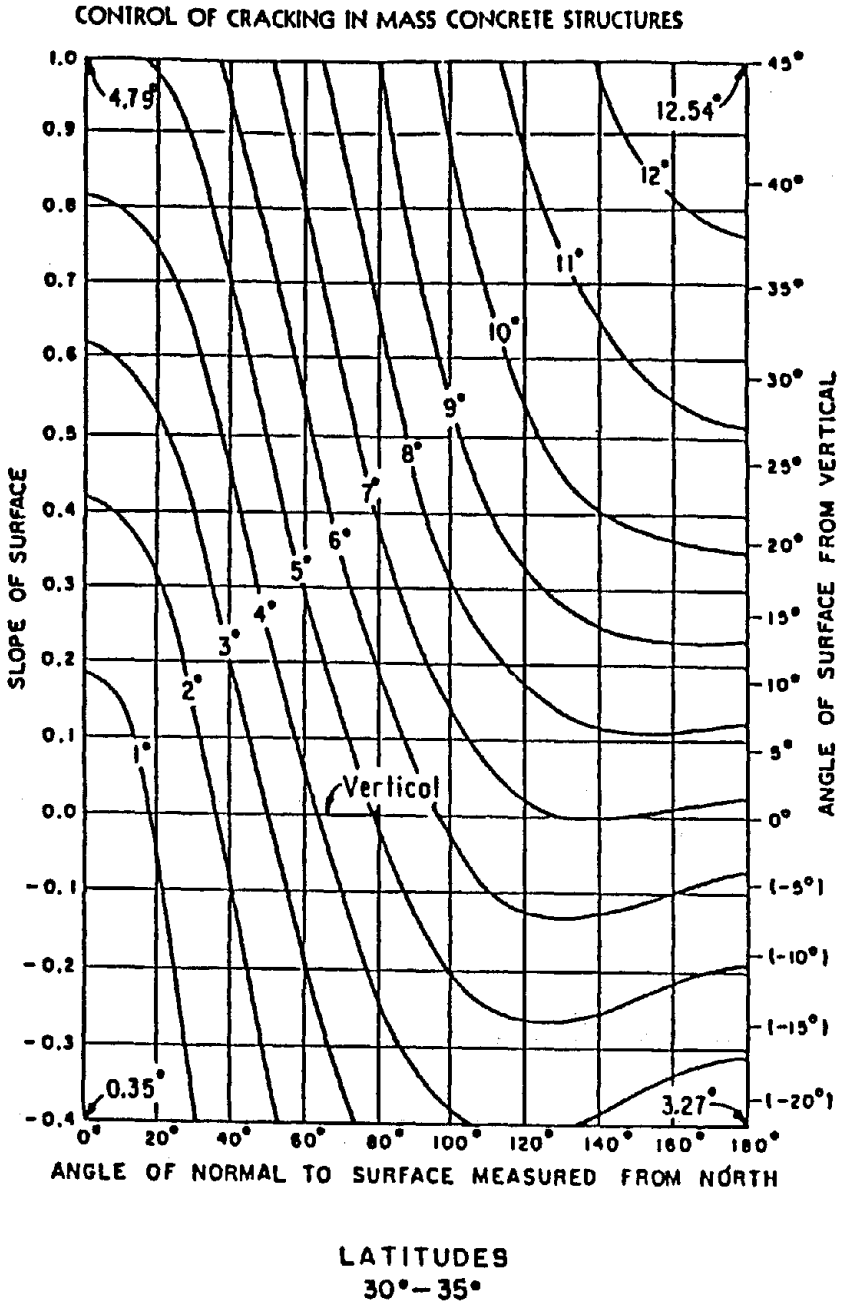


Figure 2-3. Temperature increases on the surface of exposed concrete panels due to solar radiation [from Townsend, 1981].

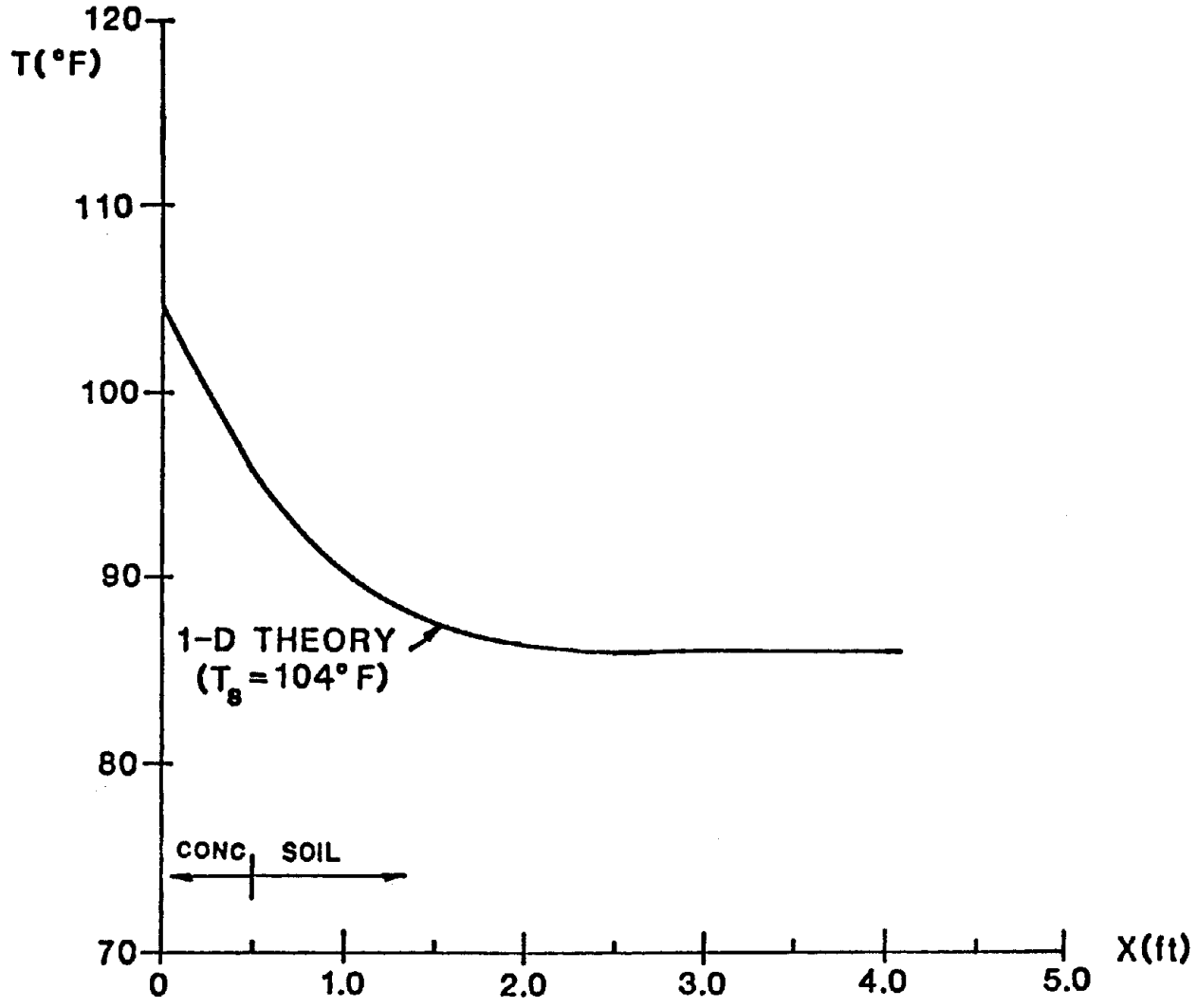


Figure 2-4. Distribution of predicted maximum temperature for wall 26-30, Tanque Verde Road project, Tucson, Arizona [from Bonaparte, 1987a].

3. WALL DESIGN

3.1 Introduction

This chapter presents the original design of panels 26-30 and 26-32 (the instrumented panels) of the MSE walls for the Tanque Verde Road project. A summary of the design steps followed by the Engineer-of-Record, Dames and Moore, is shown in Figure 3-1. The preliminary design steps are discussed in Section 3.2. Sections 3.3, 3.4, and 3.5 present the stability analyses of the considered wall sections and, subsequently, the final geogrid layout. The design of the facing panels and the connections between the geogrid reinforcement and the facing panels are briefly discussed in Section 3.6. Recent developments in design of MSE walls are discussed in Section 8.3.

3.2 Preliminary Design Steps

The preliminary steps in the design of the Tanque Verde Road reinforced soil wall included:

- problem definition (Section 3.2.1);
- design assumptions regarding material properties (Section 3.2.2); and
- list of parameters to be determined (Section 3.2.3).

3.2.1 Problem Definition

The first step in the design of wall panels 26-30 and 26-32 was to collect all of the information necessary to carry out the design. This included wall geometry, loading, performance requirements, and design life.

3.2.1.1 Geometry

The geometric variables which had to be defined are illustrated in Figure 3-2 and are discussed below.

Wall Height, H. The height of a wall is defined as the distance from the top of the wall panel to the top of the wall leveling pad. The maximum

height of wall panel 26-30 is 15.6 ft, and the maximum height of wall panel 26-32 is 16.1 ft.

Depth of Embedment, D. The depth of embedment is the distance from the ground surface in front of a wall to the top of the leveling pad. The depth of embedment for wall panels 26-30 and 26-32 is a minimum of 1.5 ft. This equates to a depth of embedment approximately equal to 10 percent of the wall height.

Face Inclination, θ. The inclination of the face of a wall is usually vertical (i.e., 90° from the horizontal plane), but it may be slightly inclined (e.g., between 80° to 90°). This inclination excludes wall batter to compensate for construction movements and for mobilization of the fill shearing resistance and the reinforcement tensile forces necessary for equilibrium. The inclination of wall panels 26-30 and 26-32 is $\theta = 90°$.

Slope of Soil Surface, β. The slope of the soil surface must be defined since the soil between the soil surface and the horizontal imposes a surcharge on the reinforced fill. The slope of the soil surface for wall panels 26-30 and 26-32 is $\beta = 0°$.

Spacing Requirements for Reinforcement. Vertical spacing requirements are governed by design requirements, which depend on factors such as the reinforcement tensile strength, and by practical considerations, which depend on factors such as the type of facing panels and facing panel connections, the locations of connections between reinforcement and facing panels, and compatibility with compacted fill lift thicknesses. Based upon the above considerations, the spacings of the geogrid reinforcement were required to be in multiples of 0.5 ft (to be compatible with 6-in. compacted fill lifts) with a minimum spacing of 1.0 ft.

Reinforcement Length. The final length of reinforcement was determined by internal and external stability considerations (Sections 3.4 and 3.5, respectively). For this project, there were additional requirements that the reinforcement length be at least 6 ft and that all reinforcement layers behind any wall panel have the same length.

3.2.1.2 Loading

A uniform vertical surcharge caused by a 9.5-in. thick precast concrete pavement and normal highway traffic on the pavement was considered in the design of wall 26. According to calculations performed by Dames and Moore [1984], this resulted in an equivalent surcharge load of q = 359 lb/ft^2.

The external stability of walls with traffic barriers was also calculated under a horizontal impact load of 1000 lb/ft. This case was considered in the design of wall panels 26-30 and 26-32.

3.2.1.3 Performance Requirements

The two performance requirements considered in the design of the reinforced soil walls were adequate stability and acceptable deformations. The primary requirement was adequate stability. The required minimum factors of safety for the considered stability failure mechanisms were [Dames and Moore, 1984]:

External Stability Failure Mechanisms

- sliding, $FS_s = 2.0$
- overturning, $FS_o = 2.0$
- bearing capacity failure, $FS_{bc} = 2.0$
- rotational slip-surface instability, $FS_r = 1.5$

Internal Stability Failure Mechanisms

- reinforcement pullout, $FS_p = 1.5$

The above factors of safety do not include a factor of safety against tensile rupture of the reinforcement, FS_t. In the design of reinforced soil walls, FS_t can be applied to the calculated tensions to maintain equilibrium or to the allowable reinforcement tension. For this project, the factor of safety, FS_t, was applied to the allowable reinforcement tension, as described in Section 2.2.

A secondary performance requirement was a limit on maximum wall deformations, i.e., maximum horizontal reinforced fill displacement and

maximum vertical reinforced fill settlement. Horizontal and vertical displacements are not normally calculated in design. Rather, they are estimated from field experience and compensated for in the construction of a wall. Horizontal wall movements are accounted for by an inward wall batter. For this project, the wall batter was 1H:96V. This wall batter had been used previously in the United Kingdom on a very limited number of geogrid-reinforced soil walls. Vertical wall movements can be controlled by constructing the wall on a firm foundation and by using well-compacted granular backfill. For this project, the foundation soil was excavated to a predetermined depth and recompacted to a dense state. As will be described subsequently, this wall batter was too small for the conditions at the Tanque Verde Road project.

3.2.1.4 Design Life

Because the Tanque Verde Road project MSE walls are permanent, critical structures, the design life was taken as 75 years based on requirements for highway structures given in AASHTO [1983]. As previously noted, the allowable design geogrid tension was based on a 120-year design life.

3.2.2 Design Assumptions Regarding Material Properties

Soils. The friction angles and unit weights of the fill in the reinforced zone, retained fill, and foundation soil were selected by Dames and Moore [1984] based on the limiting values for granular fills in guidelines by FHWA [1983]. Unit weights were based on previous experience with similar soils. The soil properties that were used to design the wall are as follows:

Material	Friction Angle	Unit Weight
Reinforced Fill	$\phi'_r = 34°$	$\gamma_r = 125$ lb/ft^3
Retained Fill	$\phi'_b = 30°$	$\gamma_b = 115$ lb/ft^3
Foundation Soil	$\phi'_f = 30°$	$\gamma_f = 110$ lb/ft^3

It should be noted that the actual friction angles of the fill materials were believed to be higher (i.e., the soil was stronger) than the limiting values given by FHWA [1983]. Berg, et al. [1986] estimated the actual friction angle of the granular fill used on the Tanque Verde Road project to be in the range of 36° to 38°.

Dames and Moore [1984] also conducted a limited analysis of the influence of the assumed unit weight on the design results. They found that the design results were not significantly affected when the value of γ_b was assumed to be in the range of 125 to 135 lb/ft^3 (rather than 115 lb/ft^3).

Reinforcement Material. The geogrid geometric characteristics, strength properties, and reinforcement-fill interaction properties used to design the wall are as follows [Netlon, 1984]:

Width of geogrid layer	$b = 3.28$ ft
Allowable design strength (Section 2.2)	$T_d = 1324$ lb/ft
Coefficient of reinforcement-fill interaction	$\mu = 0.9$

3.2.3 Parameters to be Determined

From a review of the geometric, loading, and material parameters given in Sections 3.2.1.1, 3.2.1.2, and 3.2.2, the wall height, the properties of the soil materials, and the reinforcement type were given. The design parameters to be evaluated were the spacing and length of the geogrid reinforcement, the size of the concrete wall facing panels, and the construction sequence. The design procedures used by Dames and Moore [1984] to determine these unknown parameters are presented in Sections 3.3 to 3.6.

3.3 Stability Analyses

To design the reinforced soil wall, Dames and Moore [1984] performed internal and external stability analyses of the wall using semi-empirical or analytical design methods that were performed manually. Most of the analyses were based on recommendations from Netlon [1984], which in turn were based

largely on design methods developed for Reinforced Earth® MSE walls [e.g., Lee, et al., 1973; Schlosser, 1978], for geotextile-reinforced MSE walls [e.g., Bell, et al., 1975; Broms, 1978], and from early research and field trials conducted with TENSAR geogrids [e.g., Jewell and Jones, 1981; Jones, 1984].

In the Dames and Moore design, the reinforced soil wall was first designed for internal stability. This resulted in the determination of the vertical reinforcement spacing and selection of tentative reinforcement length (Section 3.4). After the internal stability analysis, external stability calculations were carried out to verify that the tentative reinforcement length was adequate (Section 3.5). It should be noted, as discussed in Section 8.3, that current design procedures recommend that the external stability analysis be performed before the internal stability analysis.

3.4 Internal Stability Analysis

3.4.1 Method of Analysis

The internal stability analysis consists of evaluating the risk of internal failure of the reinforced soil mass. A failure surface is assumed to divide the reinforced soil mass into two zones, an active (or unstable) zone and a resistant (or stable) zone. The active zone tends to slide away from the resistant zone. The reinforcement tends to keep the two zones together. Failure occurs if the reinforcement cannot prevent slippage of the active zone, which can happen if:

- the tensile strength of the reinforcement is insufficient and the reinforcement ruptures; or
- the reinforcement pulls out of either the active zone or the resistant zone.

To evaluate the risk of rupture or pullout, the design tensile forces at the various reinforcement levels must be known. The design tensile forces are a function of the horizontal stresses in the reinforced soil mass and the reinforcement spacing. Once the horizontal stresses are determined, the vertical reinforcement spacing to prevent reinforcement rupture and the horizontal reinforcement length to prevent pullout can be determined.

The methodology used by Dames and Moore [1984] for the internal stability analysis can be summarized by the following steps:

(1) selection of analysis method;

(2) determination of horizontal stresses;

(3) selection of reinforcement spacing; and

(4) verification of pullout resistance.

These steps are discussed in Sections 3.4.2 through 3.4.5.

3.4.2 Selection of Analysis Method

The two common methods of determining horizontal stresses are the tie-back wedge analysis and the coherent gravity analysis. The main differences between the two methods originate from the basic assumptions made about the shape of the failure surface, the rotation of the wall facing, and the horizontal stresses acting within the soil in the reinforced zone.

Since both the tie-back wedge and coherent gravity methods were considered by Dames and Moore [1984], both methods are summarized below. However, as discussed in Section 8.3, the tie-back wedge method is currently used predominantly for the design of MSE walls with geosynthetic reinforcement, while the coherent gravity method is currently used predominantly for the design of MSE walls with steel reinforcement.

Tie-Back Wedge Analysis. The tie-back wedge method of analysis makes three basic assumptions:

(1) the critical potential failure surface (i.e., location of maximum horizontal stresses) is the classical Rankine active failure surface (Figure 3-3a);

(2) the assumed kinematic mechanism for wall movement is rotation about the toe of the wall; and

(3) the horizontal stresses acting along the potential failure surface are calculated using an active earth pressure coefficient, K_a.

Coherent Gravity Analysis. The coherent gravity method of analysis makes three basic assumptions:

(1) the critical potential failure surface is formed by the intersection of the classical Rankine active failure surface and a vertical line offset 0.3H behind the wall, where H is the wall height (Figure 3-3b);

(2) the assumed kinematic mechanism for wall movement is rotation about the top of the wall; and

(3) the horizontal stresses acting within the soil in the reinforced zone are calculated using a coefficient of earth pressure that varies linearly from an at-rest condition, K_0, at the ground surface to an active condition, K_a, at a depth of 20 ft and then remains at an active condition, K_a, with increasing depth.

3.4.3 Determination of Horizontal Stresses

When the internal stability design is carried out before the external stability design, an initial tentative reinforcement length must initially be assumed. For wall 26-30, which is 15.6 ft high, the initial reinforcement length was assumed to be 10 ft, which is approximately 0.65H. The horizontal stresses in the reinforced soil mass were determined using the following two-step approach:

(1) vertical stresses at each reinforcement level were calculated; and

(2) horizontal stresses at each reinforcement level were calculated by multiplying the vertical stresses at the considered level by a coefficient of earth pressure K_r:

$$\sigma'_h = K_r \sigma'_v \qquad \text{(Equation 3-1)}$$

where: σ'_h = horizontal stress (lb/ft^2);

σ'_v = vertical stress (lb/ft^2); and

K_r = coefficient of earth pressure (dimensionless).

The values of K_r used in the design were as follows:

- for the tie-back wedge method, $K_r = K_{ar}$ (coefficient of active earth pressure) = $\tan^2(45° - \phi'_r/2)$; and
- for the coherent gravity method, K_r decreases linearly from K_{or} (coefficient of at-rest earth pressure) at the ground surface to K_{ar} at a depth of 20 ft; below 20 ft, $K_r = K_{ar}$.

The vertical stresses were assumed to be caused by the weight of the reinforced fill, the uniform vertical surcharge, and the thrust exerted by the retained fill on the reinforced soil mass. An assumption was also required with respect to the effect of the retained fill thrust on the vertical stress distribution in the reinforced soil mass. The three most common assumptions are (Figure 3-4):

- the retained fill thrust has no effect and the vertical stress at any elevation is uniform and equal to the overburden pressure plus the uniform vertical surcharge (Figure 3-4a);
- the retained fill thrust is assumed to cause a trapezoidal vertical stress distribution (Figure 3-4b); and
- the retained fill thrust is assumed to change the vertical stress distribution in a manner similar to that proposed by Meyerhof for the stress distribution beneath eccentrically loaded footings (Figure 3-4c).

A trapezoidal vertical stress distribution was assumed for the tie-back wedge analysis, and a Meyerhof stress distribution was assumed for the coherent gravity analysis [Dames and Moore, 1984].

The calculated horizontal stress, σ'_h, in the reinforced soil based on the tie-back wedge method for any depth, z, below the crest of the wall is given by the equation (for $\beta = 0°$):

$$\sigma'_h = K_r \left[\gamma_r z + q + K_{ab} (\gamma_b z + 3 q)(z/L_t)^2\right] \qquad \text{(Equation 3-2)}$$

The horizontal stress, σ'_h, in the reinforced soil based on the coherent gravity method for any depth, z, is given by the equation (for $\beta = 0°$):

$$\sigma'_h = \frac{K_r (\gamma_r z + q)}{1 - \dfrac{K_{ab} z^2 (\gamma_b z + 3q)}{3L_t^2 (\gamma_r z + q)}} \qquad \text{(Equation 3-3)}$$

where:

K_r = coefficient of earth pressure related to the soil in the reinforced zone as described by Equation 3-1 (dimensionless);

K_{ab} = coefficient of active earth pressure related to the retained fill, equal to $\tan^2(45° - \phi'_b/2)$ (dimensionless);

γ_r = moist unit weight of the reinforced fill (lb/ft^3);

γ_b = moist unit weight of the retained fill (lb/ft^3);

q = uniform vertical surcharge (lb/ft^2);

z = distance from the wall crest to the considered reinforcement layer (ft); and

L_t = tentative reinforcement length (ft).

3.4.4 Selection of Reinforcement Spacing

The reinforcement spacing was determined using the equation:

$$v_z = T_d R_c/\sigma'_h \qquad \text{(Equation 3-4)}$$

where:

T_d = allowable design tension of geogrid reinforcement, as defined in Section 2.2 (lb/ft);

R_c = coverage ratio defined below by Equation 3-5 (dimensionless);

v_z = maximum vertical spacing between reinforcement layers at depth z below the wall crest (ft); and

σ'_h = horizontal stress from Equation 3-2 or 3-3 (lb/ft^2).

The coverage ratio is defined as follows:

$$R_c = b/d \qquad \text{(Equation 3-5)}$$

where:

b = width of geogrid strip (ft); and

d = center-to-center horizontal distance between geogrid strips (ft).

(Note: $R_c = 1$ in the case of continuous reinforcement, i.e., the reinforcement layers cover the entire horizontal surface of the reinforced soil mass at each reinforcement level. If there are spaces between reinforcing strips $R_c < 1$.)

Using Equations 3-2, 3-3, 3-4, and 3-5, and the allowable design tension of the TENSAR SR2 geogrid, two tables were developed for the maximum spacing, v_z, as a function of depth below the surface of the reinforced soil mass. Each table was developed for a range of panel heights and reinforcement lengths [Dames and Moore, 1984]. The first table was developed using the tie-back wedge method and assuming a trapezoidal vertical stress distribution (Equation 3-2). The second table was developed using the coherent gravity method and assuming a Meyerhof stress distribution (Equation 3-3).

From the two tables, a family of curves giving the theoretical spacing as a function of depth below the wall crest was developed for a series of reinforcement lengths, using the more critical (i.e., smaller) spacing values

based on comparison of the two tables. The family of curves developed by Dames and Moore [1984] for wall panels 26-30 and 26-32 is shown in Figure 3-5.

After a tentative reinforcement length of 10 ft was selected, the maximum spacing of the reinforcement was taken directly from the curve corresponding to that reinforcement length. At any given level, the actual vertical spacing was smaller than the maximum spacing and compatible with the practical requirements imposed by the panel geometry and fill lift thicknesses.

3.4.5 Verification of Pullout Resistance

3.4.5.1 Available Anchorage Length

As the final step of the internal stability analysis, Dames and Moore [1984] verified that the tentative reinforcement length of 10 ft selected in Section 3.4.4 was adequate.

The pullout resistance was only analyzed for a Rankine failure surface (tie-back wedge analysis) because, by inspection, the available anchorage length of reinforcement beyond the assumed line of maximum tension (i.e., the assumed critical potential failure surface) was shorter for the Rankine failure surface than for the bilinear failure surface of the coherent gravity method. In addition, only the pullout resistance of the uppermost geogrid layer was analyzed because, by inspection, the anchorage length was shorter for this uppermost layer than for any of the lower layers.

The available anchorage length is the portion of the reinforcement length which is outside the active (or unstable) zone (Figure 3-6). For this project, the available anchorage length, L_{aa} (ft), is given by:

$$L_{aa} = L_t - [(H - z) \tan(45° - \phi'_r/2)] \qquad \text{(Equation 3-6)}$$

where:

L_t = tentative length of the reinforcement (ft);

H = wall height (ft);

z = distance from the wall crest to the considered reinforcement layer (ft); and

ϕ'_r = friction angle of the reinforced fill (dimensionless).

The available anchorage length of wall panels 26-30 and 26-32 was calculated from Equation 3-6, with a tentative reinforcement length L_t = 10 ft, wall height H = 16 ft, and distance from the wall crest to the uppermost geogrid z = 1.5 ft, as L_{aa} = 2.29 ft [Dames and Moore, 1984].

3.4.5.2 Required Anchorage Length

The required anchorage length, L_{ar} (ft), was determined by relating the pullout resistance of the reinforcement to the tension in the reinforcement. The relevant equation, taken from Netlon [1984], is:

$$L_{ar} = \frac{FS_p \, \sigma'_h \, V_z}{2 \, \mu \, \tan\phi'_r \, (\gamma_r \, z + q)} \qquad \text{(Equation 3-7)}$$

where:

FS_p = factor of safety against pullout (dimensionless);

σ'_h = horizontal stress (lb/ft^2);

V_z = vertical reinforcement spacing at depth z below the wall crest (ft);

μ = coefficient of reinforcement-fill interaction (dimensionless);

ϕ'_r = friction angle of the reinforced fill (dimensionless);

γ_r = moist unit weight of the reinforced fill (lb/ft^3);

z = distance from the wall crest to the considered reinforcement layer (ft); and

q = uniform vertical surcharge (lb/ft^2).

The required anchorage length of wall panels 26-30 and 26-32 was calculated from Equation 3-7, with a tentative reinforcement length L_t = 10 ft and wall height H = 16 ft, as L_{ar} = 2.46 ft.

3.4.5.3 Final Reinforcement Length

The final reinforcement length, L_f, can be determined by comparing the available anchorage length, L_{aa}, to the required anchored length, L_{ar}:

- if $L_{aa} \geq L_{ar}$, then $L_f = L_t$; and
- if $L_{aa} < L_{ar}$, then $L_f = L_t + (L_{ar} - L_{aa})$.

The available anchorage length and required anchorage length of wall panels 26-30 and 26-32 were determined in Sections 3.4.5.1 and 3.4.5.2 as L_{aa} = 2.29 ft and L_{ar} = 2.46. Since $L_{aa} < L_{ar}$, $L_f = L_t + (L_{ar} - L_{aa})$ = 10 + (2.46 - 2.29) = 10.17 ft. This was less than the 10 ft geogrid reinforcement plus the 0.25 ft looped connection. Therefore, the final reinforcement length selected by Dames and Moore [1984] was L_f = 10 ft.

3.4.6 Geogrid Layout

The geogrid layout for wall panels 26-30 and 26-32 after the internal stability analysis is shown in Figure 3-7. The external stability analysis for these wall panels and for a 10 ft reinforcement length are described in Section 3.5. If the external stability analysis indicate an inadequate factor of safety for any of the external stability failure mechanisms discussed in the next section, the reinforcement length would have to be increased.

3.5 External Stability Analysis

3.5.1 Introduction

Dames and Moore [1984] analyzed the external stability of the reinforced soil wall by considering failure of the wall by the following potential failure mechanisms [Netlon, 1984]:

- sliding along the base of the reinforced soil mass;
- overturning about the toe of the soil mass;
- bearing capacity failure of the foundation soil; and
- rotational slip-surface failure.

These four mechanisms are illustrated in Figure 3-8.

All of these potential failure mechanisms were analyzed using classical soil mechanics methods.

3.5.2 Failure Due to Sliding

Sliding failure occurs when the reinforced soil mass, acting as a rigid body, is laterally displaced by the horizontal component of the thrust exerted by the retained fill on the reinforced fill.

For this project the factor of safety against sliding was required to be at least $FS_s = 2.0$. Dames and Moore [1984] calculated the sliding factor of safety using the following equation modified from Netlon [1984]:

$$FS_s = \frac{2\mu \tan\phi'_r (\gamma_r H + q)]}{K_{ab} (\gamma_b H + 2q)(H/L_f) + I} \qquad \text{(Equation 3-8)}$$

where:

L_f = reinforcement length from internal stability analysis (ft);

γ_r = moist unit weight of reinforced fill (lb/ft^3);

γ_b = moist unit weight of retained fill (lb/ft^3);

H = wall height (ft);

K_{ab} = active earth pressure coefficient of the retained fill, equal to $\tan^2(45° - \phi'_b/2)$ (dimensionless);

ϕ'_r = friction angle of the reinforced fill (dimensionless);

ϕ'_b = friction angle of the retained fill (dimensionless);

q = uniform vertical surcharge (lb/ft^2);

I = impact load, equal to 0 if the wall does not have a traffic barrier (lb/ft);

μ = coefficient of reinforcement-fill interaction (dimensionless).

Using Equation 3-8, the design parameters presented in Section 3.2, and a reinforcement length L_f = 10 ft, Dames and Moore calculated a factor of safety against sliding of 1.8 for a 16-ft high wall panel. This factor of safety is less than the required factor of safety FS_s = 2.0. Therefore, the reinforcement length was increased to L_f = 12 ft. This resulted in an acceptable factor of safety against sliding of 2.16.

3.5.3 Failure Due to Overturning

Overturning occurs when the entire reinforced soil mass, acting as a rigid body, rotates about its toe as a result of the thrust exerted by the retained fill on the reinforced soil mass.

For this project, the factor of safety against overturning was required to be at least FS_o = 2.0. Dames and Moore [1984] calculated the factor of safety against overturning using the following equation modified from Netlon [1984]:

$$FS_o = \frac{3\ (\gamma_r\ H + q)L_f^2}{K_{ab}\ (3q + \gamma_b\ H)H^2 + I\ H} \qquad \text{(Equation 3-9)}$$

where: all parameters are as defined in Equation 3.8.

Using Equation 3-9, the design parameters presented in Section 3.2, and a reinforcement length L_f = 10 ft, Dames and Moore calculated a factor of safety against overturning of 2.84 for a 16-ft high wall panel.

3.5.4 Failure by Lack of Bearing Capacity

Failure by lack of bearing capacity occurs when the stress at the base of the reinforced soil mass exceeds the bearing capacity of the foundation soil. Dames and Moore [1984] calculated the bearing pressures exerted by the wall on the foundation soil using classical soil mechanics methods. The maximum bearing pressure for the 21.9 ft wall panel was 3970 psf. This information was provided by Dames and Moore to Pima County, who independently verified that the foundation had adequate bearing capacity. It is noted that, independent of the wall design, Pima County's geotechnical consultant recommended that the foundation soil be excavated to a depth equal to the lesser of the particular wall panel height or 7 ft and then recompacted in place. This was required because pockets of the soil in the vicinity of the reinforced soil mass exhibited collapse potential when wetted.

3.5.5 Rotational Slip Surface Instability

The stability of all walls were deemed safe by inspection [Dames and Moore, 1984] with respect to the recommended factor of safety against slip-surface failure of 1.5. This appears to have been a safe assumption given the granular nature of the reinforced soil, retained soil, and foundation soil.

3.5.6 Final Geogrid Layout

The final geogrid layout for wall panels 26-30 and 26-32 after the internal and external stability analyses is shown in Figure 3-7. As illustrated in this figure, external stability governed the reinforcement length.

3.6 Design of Structural Components

3.6.1 Introduction

The traffic barrier, wall panels, and erection bracing system were designed using conventional structural analyses by Dames and Moore [1984] in accordance with design criteria established by Pima County Transportation and

Flood Control District and Holben, Martin, and Meza for this project. A system for connecting the geogrid reinforcement to the facing panels was initially proposed by Tensar. The initially proposed system, however, was revised during the design process, analyzed in detail, and tested in the laboratory. The development of the connection system is discussed below and illustrated in Figure 3-9.

3.6.2 Development of Connection System

The initial design of the connection between the facing panel and the geogrid reinforcement is shown in Figure 3-9a. However, the Pima County Transportation and Flood Control District believed that a sliding connection would be desirable, since a sliding connection could withstand limited differential panel and fill movements without overstressing either the facing panel or the connection. Therefore, the connection was redesigned as shown in Figure 3-9b.

The performance of this sliding connection (Figure 3-9b) was examined by Dames and Moore [1984] at Western Technologies, Inc. (WTI) by loading the connection to failure. Test results indicated that the connection exhibited relatively high deflection under load. This deflection was largely due to the combined interaction of the relatively small circumference No. 4 steel bar, the geometry of the looped connection, and the stiffness of the geogrid. In addition, the ribs of the steel bar created stress concentrations which resulted in cracking of the ribs of the geogrid. The WTI tests revealed the need for a modified connection that would better distribute the loads within the ribs of the geogrid, exhibit more favorable load-deflection characteristics, and reduce stress concentrations and, hence, eliminate cracking within the ribs of the geogrid. Therefore, the sliding connection was modified as shown in Figure 3-9c.

Test results on the modified sliding connection with a 1-in. diameter, Schedule 40 polyvinyl chloride (PVC) pipe (Figure 3-9c) revealed that the PVC pipe was more geometrically and physically compatible with the geogrid than the steel bar. Connection deflection under load was found to be within tolerable limits. Use of the PVC pipe facilitated seating of the load in comparison to the steel bar connection.

The modified sliding connection, as it was finally constructed in the field, is shown in Figure 3-9d.

START DESIGN

PRELIMINARY DESIGN STEPS (Section 3.2)

Problem Definition (Section 3.2.1)
Design Assumptions Regarding Material Properties (Section 3.2.2)
List of Parameters to be Determined (Section 3.2.3)

STABILITY ANALYSES (Section 3.3)

Internal Stability Analysis (Section 3.4)

- Selection of Analysis Method (Section 3.4.2)
- Determination of Horizontal Stresses (Section 3.4.3)
- Selection of Reinforcement Spacing (Section 3.4.4)
- Verification of Pullout Resistance (Section 3.4.5)

External Stability Analysis (Section 3.5)

- Failure Due to Sliding (Section 3.5.2)
- Failure Due to Overturning (Section 3.5.3)
- Failure by Lack of Bearing Capacity (Section 3.5.4)
- Rotational Slip Surface Instability (Section 3.5.5)

FINAL GEOGRID LAYOUT (Section 3.5.6)

DESIGN OF FACING UNITS AND CONNECTIONS (Section 3.6)

DESIGN FINISHED

Figure 3-1. Design steps for the reinforced soil wall, as conducted by Dames and Moore [1984], based on procedures recommended by Netlon [1984].

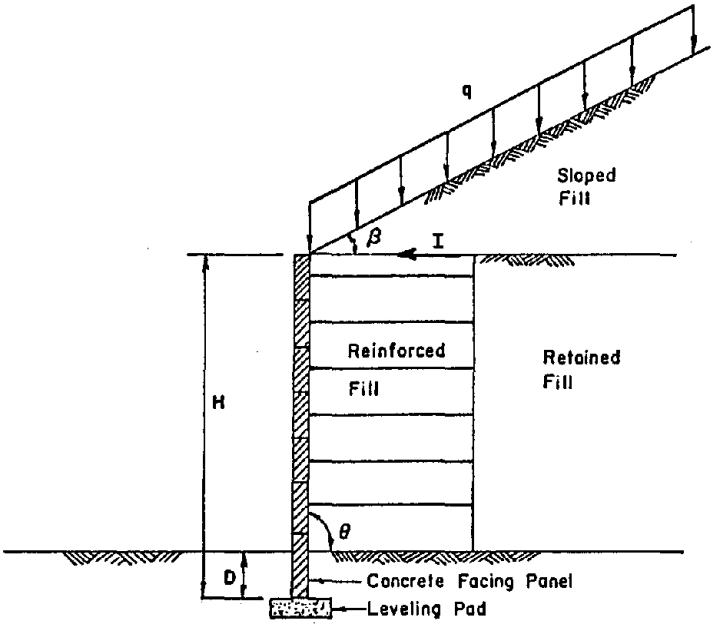


Figure 3-2. Geometric and loading conditions of a reinforced soil wall.

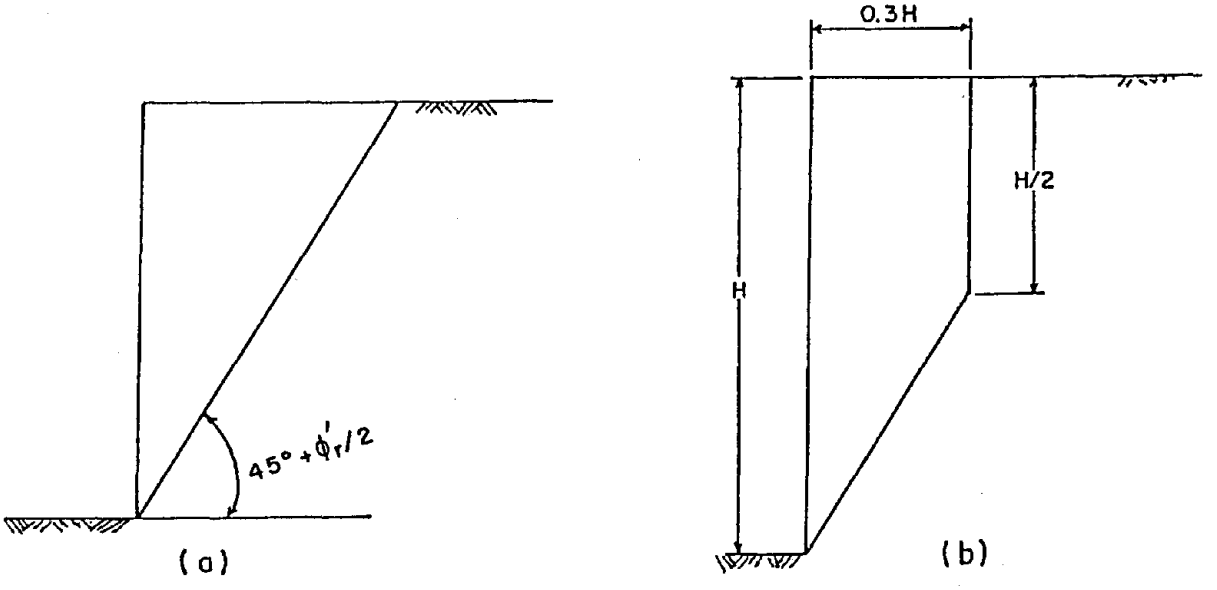


Figure 3-3. Potential failure surfaces considered in the internal stability analysis of reinforced soil walls: (a) Rankine-type surface used in the tie-back wedge analysis; and (b) bilinear surface used in the coherent gravity analysis.

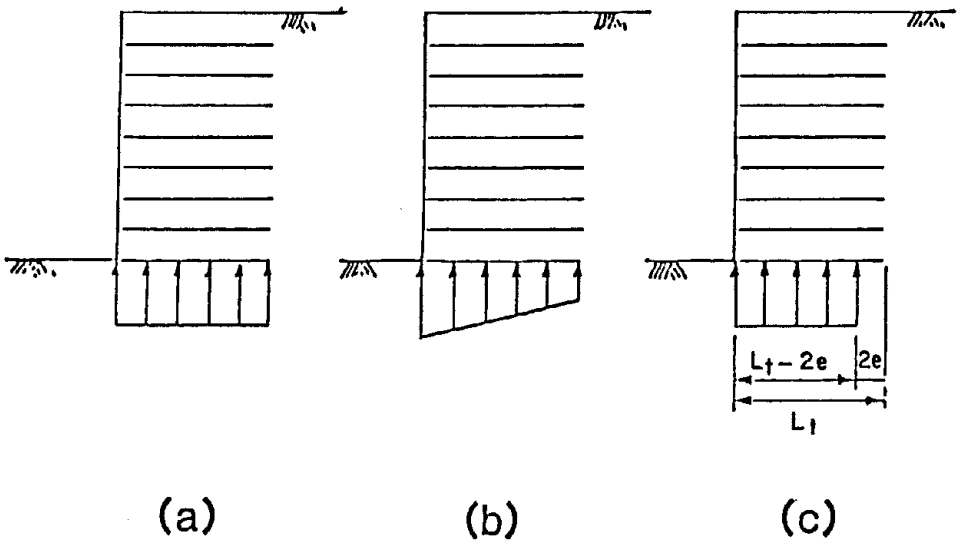


Figure 3-4. Three possible stress distributions at the base of a reinforced soil wall: (a) uniform; (b) trapezoidal; and (c) Meyerhof.

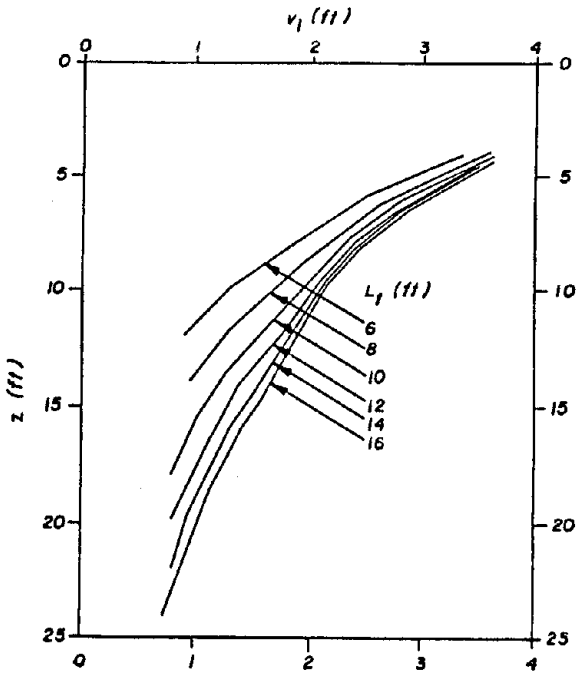


Figure 3-5. Family of curves giving the minimum required spacing, v, between reinforcement layers, as a function of depth, z, below the top surface for a series of tentative reinforcement lengths, L_t, for wall 26 [from Dames and Moore, 1984].

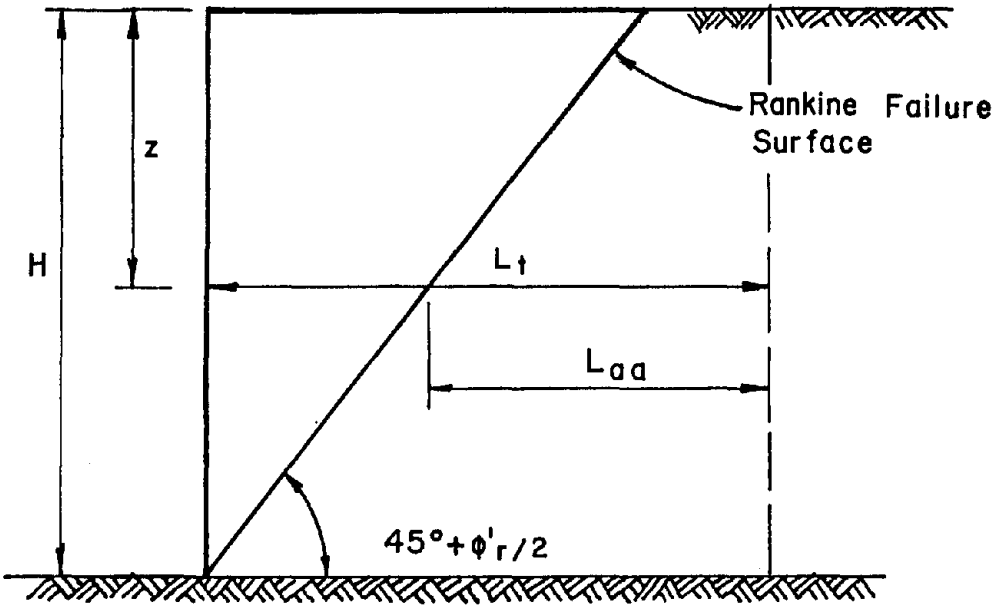


Figure 3-6. Illustration of allowable anchorage length for a Rankine failure surface (tie-back wedge analysis).

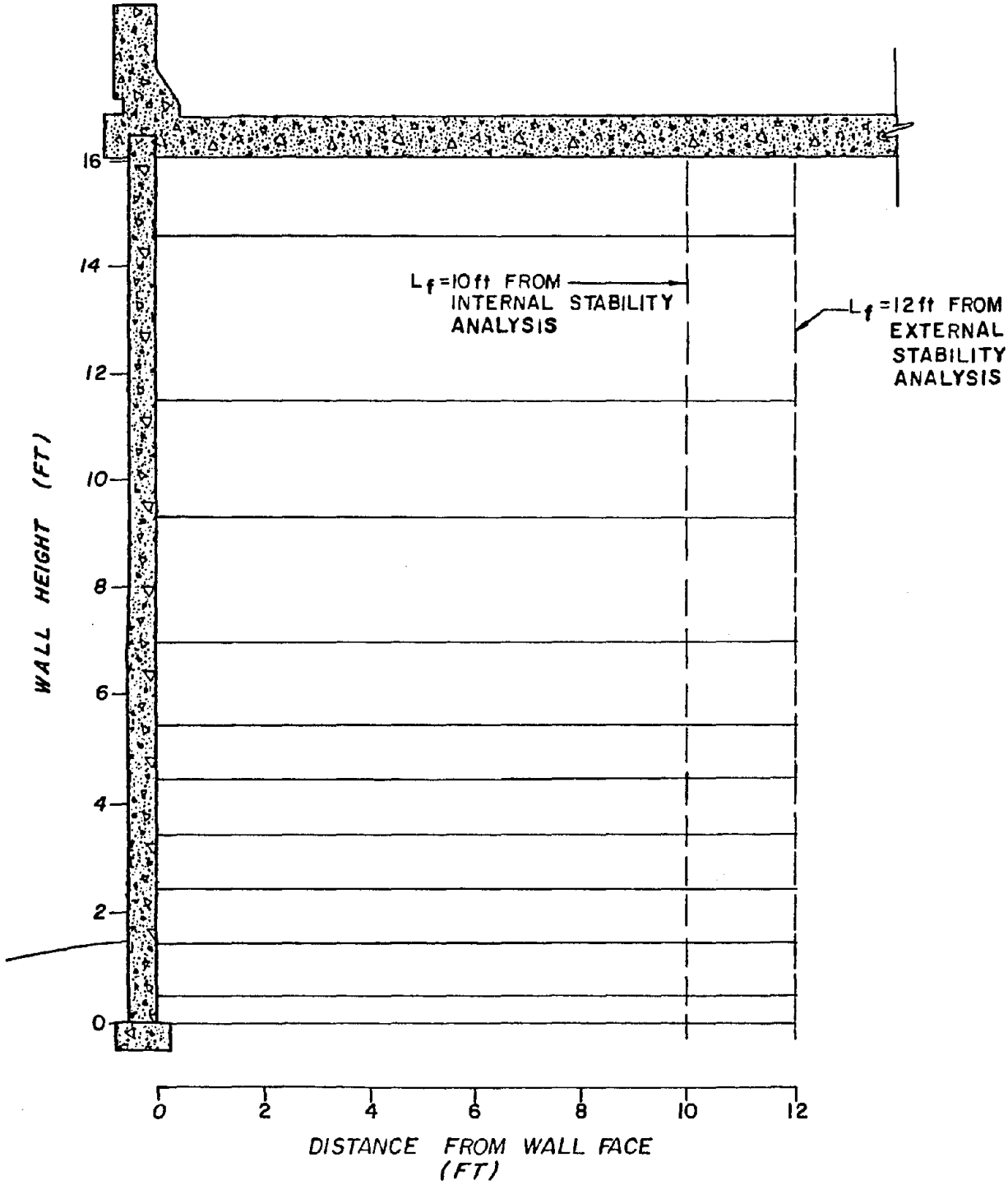


Figure 3-7. Final geogrid layout for wall panels 26-30 and 26-32.

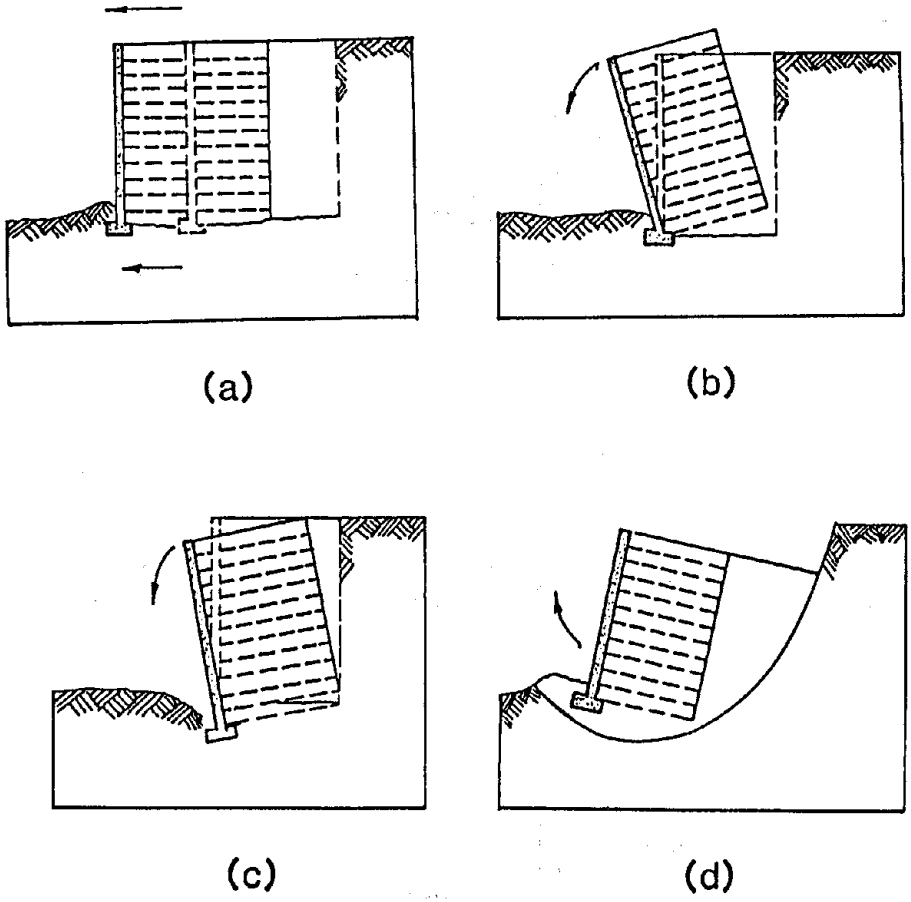


Figure 3-8. Failure mechanisms considered in an external stability analysis of a reinforced soil wall: (a) sliding; (b) overturning; (c) bearing capacity; and (d) rotational slip-surface failure.

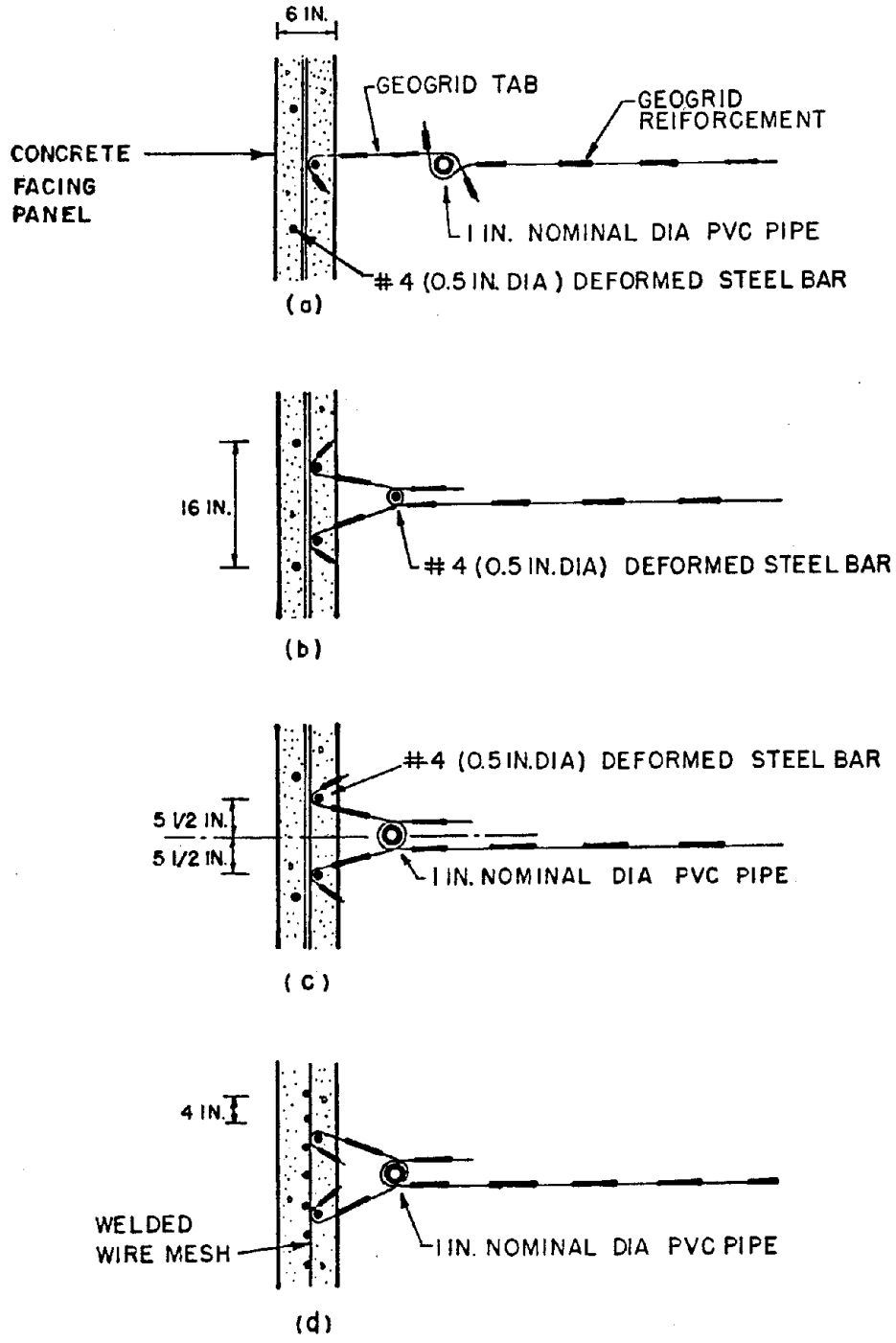


Figure 3-9. Development of the facing panel-to-geogrid connection system: (a) initial connection which had been used in the United Kingdom; (b) sliding connection; (c) modified sliding connection; and (d) modified sliding connection as it was constructed.

4. WALL CONSTRUCTION

4.1 Wall Components

Four different wall geometries were used on the Tanque Verde Road project, as illustrated in Figure 4-1. Wall 26, which has the two instrumented sections (panels 26-30 and 26-32), is a "type C" wall. The principal components of these MSE walls are illustrated in Figure 4-2 and are listed below.

Geogrid Reinforcement. TENSAR SR2 uniaxial geogrid was used as reinforcement. Geogrid layers were connected to concrete facing panels and extended 6 to 16 ft into the granular fill, depending on wall height.

Concrete Facing Panels. The reinforced soil mass was faced with full-height, precast, reinforced-concrete panels that were 6-in. thick and 10-ft wide. The concrete panels were originally designed with #4 steel bar reinforcement placed at 16-in. centers in the horizontal and vertical directions. However, Sundt submitted a change order that called for 6 in. x 6 in., W7.5 x W7.5 welded wire mesh instead of #4 bars. Additional reinforcement was placed around lift hooks (steel hooks cast into the concrete panels to allow the panels to be hoisted by a crane) of the larger panels. During the precasting operations, geogrid tabs were cast into the backs of concrete panels at predetermined locations (Photographs 4-1 and 4-2). The ends of the geogrid tabs were wrapped around #4 steel bars (which were tied to the welded wire mesh) and held in place with wire ties for casting. The geogrid tabs held up the reinforcement within the concrete panel form, thus making the tabs taut and eliminating the need to support the reinforcement during concrete placement and curing. In the field, the main geogrid reinforcement was connected to the tabs using PVC pipe as shown in Figure 4-3. The bottom of the panels were temporarily supported on two 1-in. diameter leveling bolts that were cast into the leveling pad (Figure 4-4). The bolts fit into two recesses cast into the bottom of the panels.

Leveling Pads. The concrete cast-in-place leveling pads were 1-ft wide by 8-in. thick and extended the length of a facing panel. The leveling pads were placed at foundation elevation with vertical cold joints between adjacent pads (Figure 4-5). The leveling pads were unreinforced unless they

supported facing panels that were 15 ft or more high. In these cases, the leveling pads were reinforced with two #4 steel bars located as shown in Figure 4-6. The functions of the leveling pads were to provide a level surface on which to place the panels and to control wall alignment. With the full-height facing panels, the leveling pads served the additional function of acting as footings to distribute the weight of the panels to the foundation soil.

Geogrid Tab and PVC Pipe Connections. The concrete facing panels were attached to the geogrid reinforcement by a connection system, illustrated in Figure 4-3, that was composed of a geogrid tab cast into the facing panel, a PVC pipe connector, and a strip of the main geogrid reinforcement. The geogrid tabs were formed by embedding a short section of geogrid into the wall panels during precasting such that a "c"-shaped tab protruded from the wall face (Photographs 4-1 and 4-2). The main geogrid reinforcement was overlapped with the geogrid tabs and secured by a 1-in. nominal diameter Schedule 40 PVC pipe which was passed through the loop made by the geogrid tabs and main geogrid reinforcement. The connections were specifically designed for this project, as discussed in Section 3.6, to withstand limited differential panel and fill movements without overstressing either the facing panel or the geogrid-to-panel connection.

4.2 Construction Sequence

This section presents a summary of the sequential construction of the Tanque Verde Road project MSE walls.

Step 1. Foundation soil was excavated to a depth equal to the lesser of the particular wall panel height or 7 ft and recompacted to at least 90% of maximum dry density based on the modified Proctor test (ASTM D1557). This step was necessary because an investigation by Pima County's geotechnical consultant indicated that the foundation soil had pockets of alluvial soil which were potentially collapsible when wetted.

Step 2. Leveling pads were cast-in-place and stepped to minimize panel height and wall fill earthwork volumes. Simultaneously, concrete facing panels were cast on site and stockpiled until they were needed. The concrete used for the facing panels was colored, as specified by the Architect. The

facing panels were cast with lift hooks so that they could be hoisted by a crane. Panels weighed between 1,350 and 16,500 lbs. A few damaged concrete panels were used as reaction blocks for adjustable temporary erection braces, which were used to support the wall panels during construction. Otherwise, cast blocks of concrete were used. The erection braces were pinned to the reaction blocks and wall panels as shown in Figure 4-7.

Step 3. Concrete facing panels were hoisted by their lift hooks (Photograph 4-3) with a crane and set vertically on the leveling pads (Photograph 4-4). Panel alignment was achieved by placing the panels such that the leveling bolts cast into the leveling pads penetrated the precast recesses in the bottoms of the panels (Photograph 4-5). The wall panels were held in place with the adjustable erection braces which were set on 10 ft centers in front of the vertical wall joints. Initially, the wall panels were battered inwards at 1H:96V (Photograph 4-6). After the erection braces were placed, the leveling bolts were adjusted to improve panel alignment.

Step 4. At the start of wall construction, no provision was made to clamp abutting facing panels at their tops so as to enforce uniform construction movements between panels. During the early stages of construction, movements of the panels (due to rotations about the leveling bolts) were observed. At the tops of the panels, differential movements between adjacent panels on the order of 1 in. were observed. To mitigate this problem, Sundt initiated clamping of adjacent panels at their tops. The clamps consisted of two wood boards, a threaded steel bar and two nuts. The clamps were positioned at the tops of the vertical joints between adjacent panels and tightened.

Step 5. Strips of nonwoven geotextile (Trevira S1120 manufactured by Hoescht Fibers Industries, Spartanburg, South Carolina) were glued to the facing panels and leveling pads at vertical wall joints. The geotextile is intended to function as a filter to keep the wall fill from washing out between the joints. Trevira S1120 is a polyester needlepunched nonwoven geotextile composed of spunbonded continuous filaments. Index properties of this material include: mass per unit area, 6 oz/yd^2; apparent opening size (AOS), U.S. standard sieve size 50 to 70; and grab strength, 155 lbs. Polymer foam strips were placed in the 0.75 in. vertical wall joints to protect the geotextile from ultraviolet degradation and vandalism.

Step 6. Concrete grout was troweled into the space between the bottom of each facing panel and the top of the leveling pad (Photograph 4-7). The purpose of the grout is to support the wall panels should the steel leveling bolts rust.

Step 7. Granular wall fill (clean, moderately well-graded gravelly sand) was placed up to the first geogrid level by a front end loader. The granular fill was compacted to at least 95% of the maximum dry density based on the modified Proctor test (ASTM D1557). Fill was compacted near the wall (within 3 ft of the wall face) by hand-operated lightweight vibratory tampers or hand-operated vibratory steel drum rollers. Further from the wall, an operator-driven, 1-ton, vibratory steel drum roller was used.

Step 8. At each required level of reinforcement, a continuous layer of geogrid was placed on the granular fill and secured to the geogrid tab embedded in the wall panel using the PVC pipe connector (Photograph 4-8).

Step 9. The geogrid reinforcement-facing panel connection was pretensioned by placing a timber wedge between the PVC pipe and wall facing to remove slack between the wall facing and the connection, as shown in Photograph 4-9. Any remaining slack was pulled out of the connection by inserting steel "T" forks through the apertures of the geogrid and into the ground and slowly pulling away from the wall (Photograph 4-10). In addition to removing slack from the connection, this procedure produced a small pretension in the geogrid. Once the geogrid was pretensioned, the fork was driven further into the ground to maintain the tension until sufficient wall fill had been placed on top of the geogrid to prevent loss of the tension. This pretensioning procedure worked well with the clean, gravelly sand fill.

Step 10. Granular wall fill was placed on the geogrid layer and spread with a front-end loader (Photograph 4-11). The typical thickness of the lifts was 8-in. loose and 6-in. compacted. However, the compacted lift thicknesses for the instrumented sections of the walls were thicker (on the order of 1.0 to 1.5 ft) to protect the instruments from construction-induced damage. After a lift of fill was placed, the steel "T" forks used to pretension the geogrid and the timber wedges used at the wall connection were removed. Fill was compacted as described in Step 7.

Step 11. As the height of the fill increased, the erection braces on the outside of the wall were loosened, allowing the wall to deflect outward and the load to be transferred to the geogrid reinforcement.

Step 12. Steps 8 through 11 were repeated for each geogrid layer until the fill reached the top of the wall. In a few cases, the erection braces were removed when the reinforced fill reached a level equal to about two-thirds of the total wall height. Usually, the braces were left attached until the fill reached the full wall height.

Step 13. Retained granular backfill behind the reinforced zone was compacted to at least 95% of the maximum dry density based on the modified Proctor test (ASTM D1557).

Step 14. A concrete traffic barrier was cast-in-place at the top of the wall. Prior to casting the traffic barrier, the connector clamps between panels were removed.

Step 15. After the completion of wall construction, the wall facings were sandblasted or, in the case of abutments, lightly jackhammered to produce an exposed aggregate finish (Photograph 4-12). This procedure was enhanced through the use of a setting inhibitor mixed into the concrete used to form the panels.

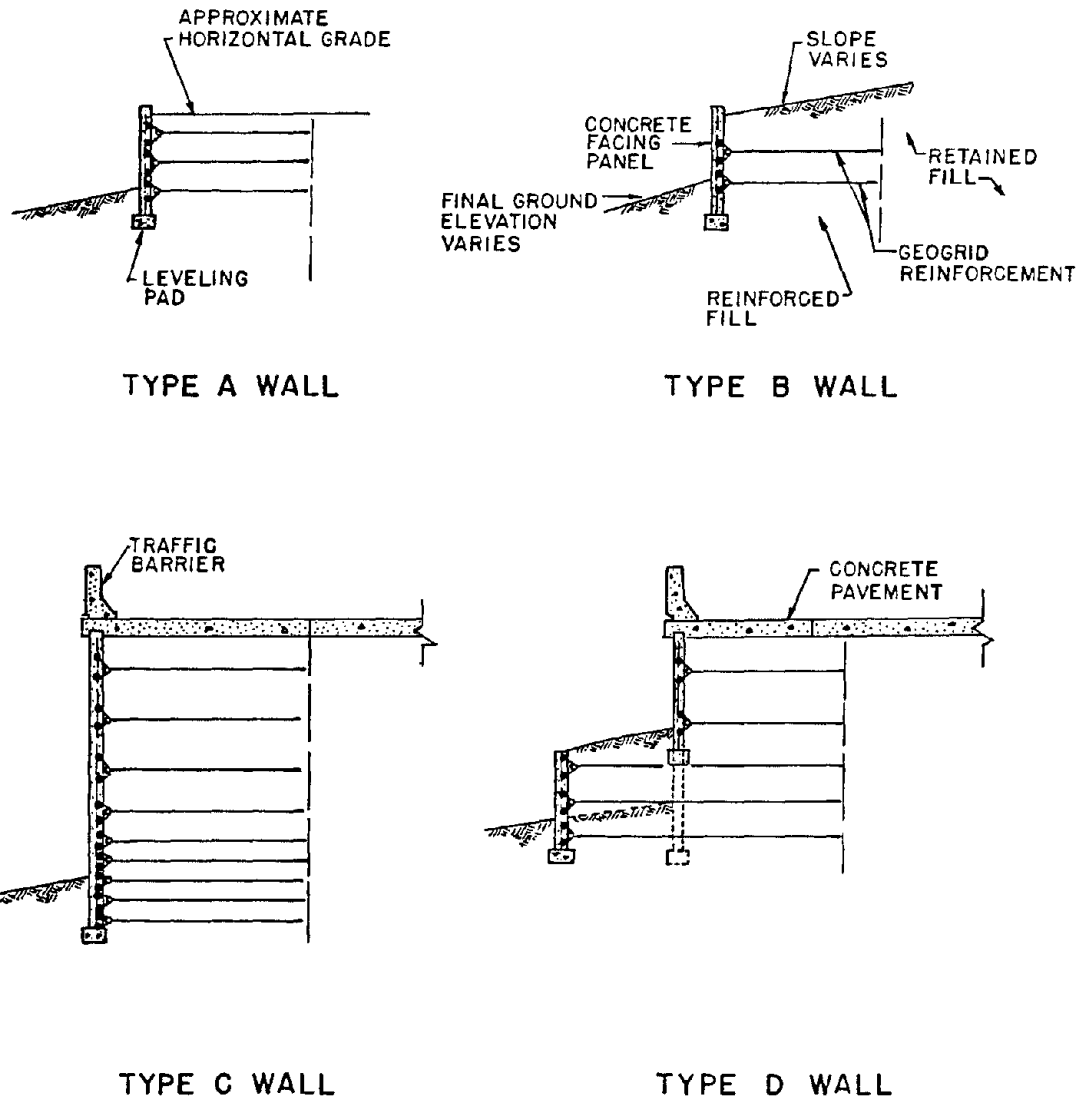


Figure 4-1. Four different wall geometries used on the Tanque Verde Road project [from Dames and Moore, 1985].

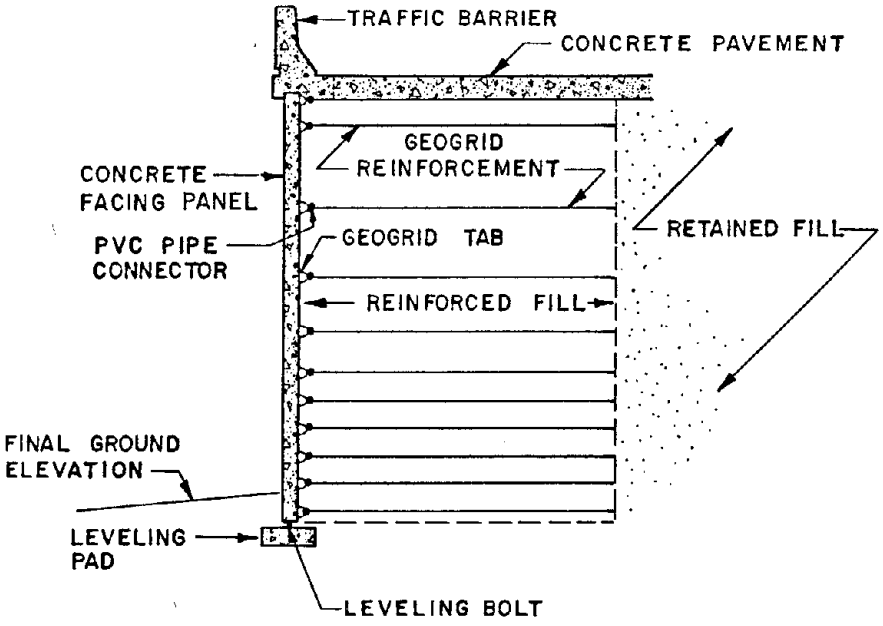


Figure 4-2. Components of the Tanque Verde Road MSE wall system.

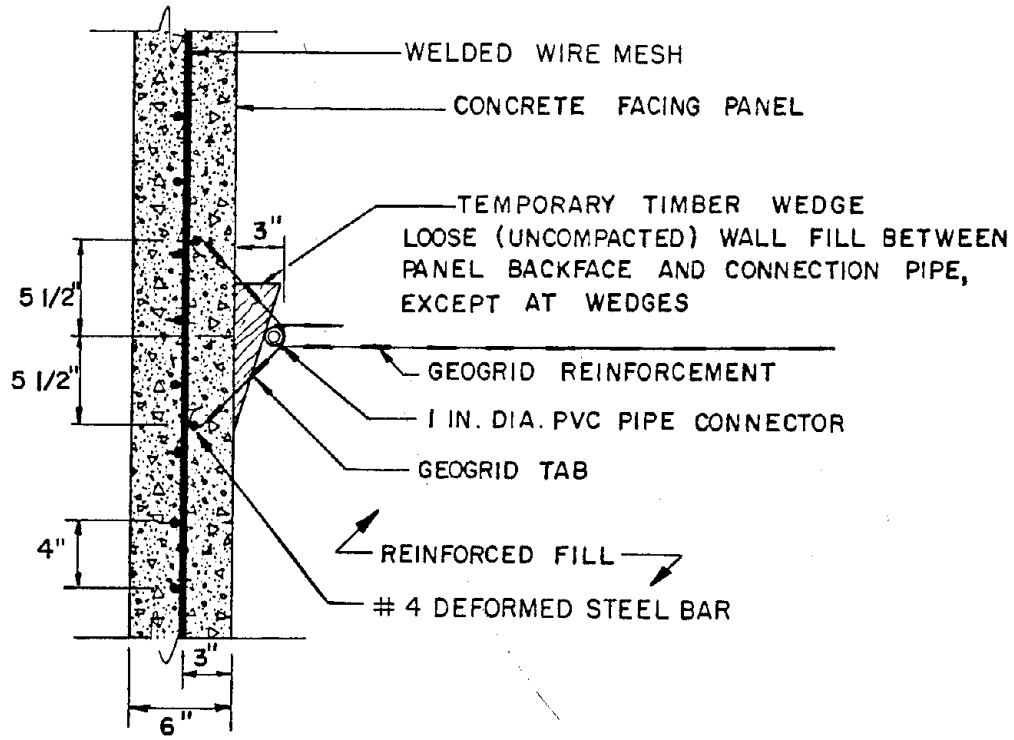


Figure 4-3. Illustration of geogrid looped connection used on the Tanque Verde Road project.

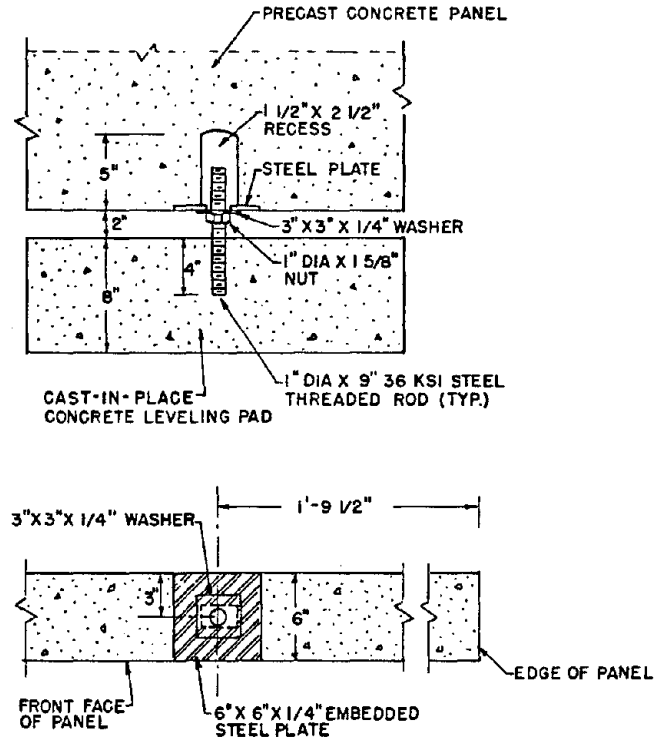


Figure 4-4. Leveling bolt detail for the Tanque Verde Road project [from Dames and Moore, 1985].

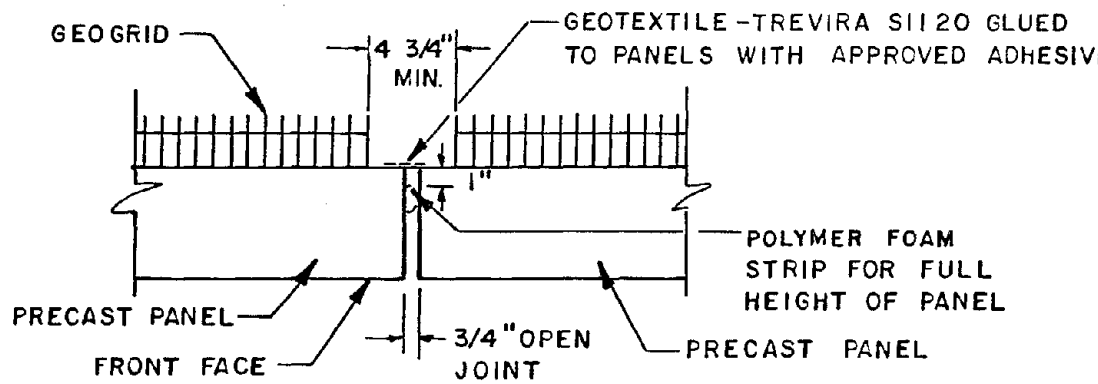


Figure 4-5. Vertical joint detail for the Tanque Verde Road project [from Dames and Moore, 1985].

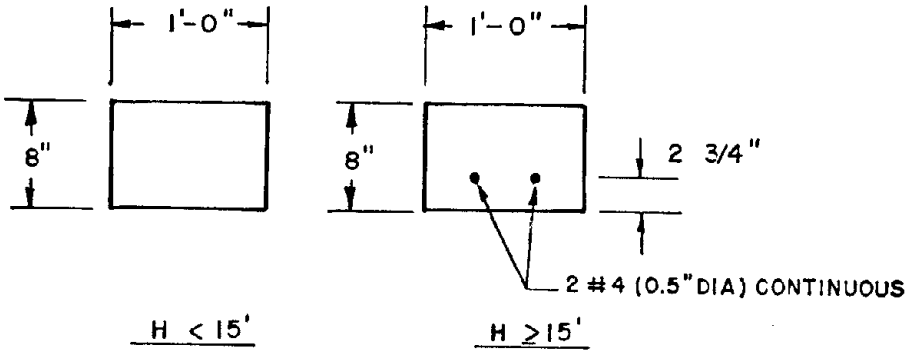


Figure 4-6. Concrete leveling pad detail for the Tanque Verde Road project [from Dames and Moore, 1985].

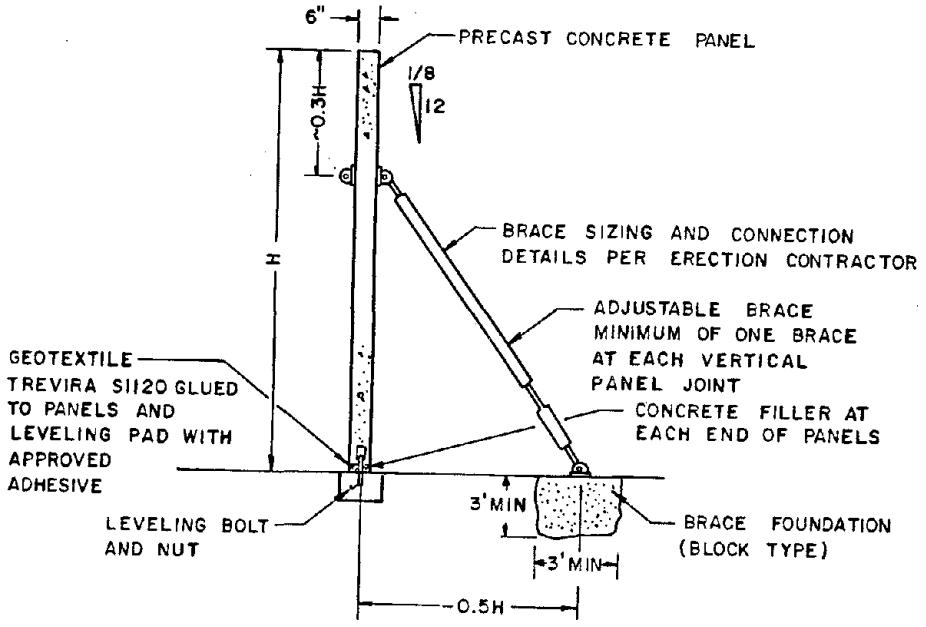
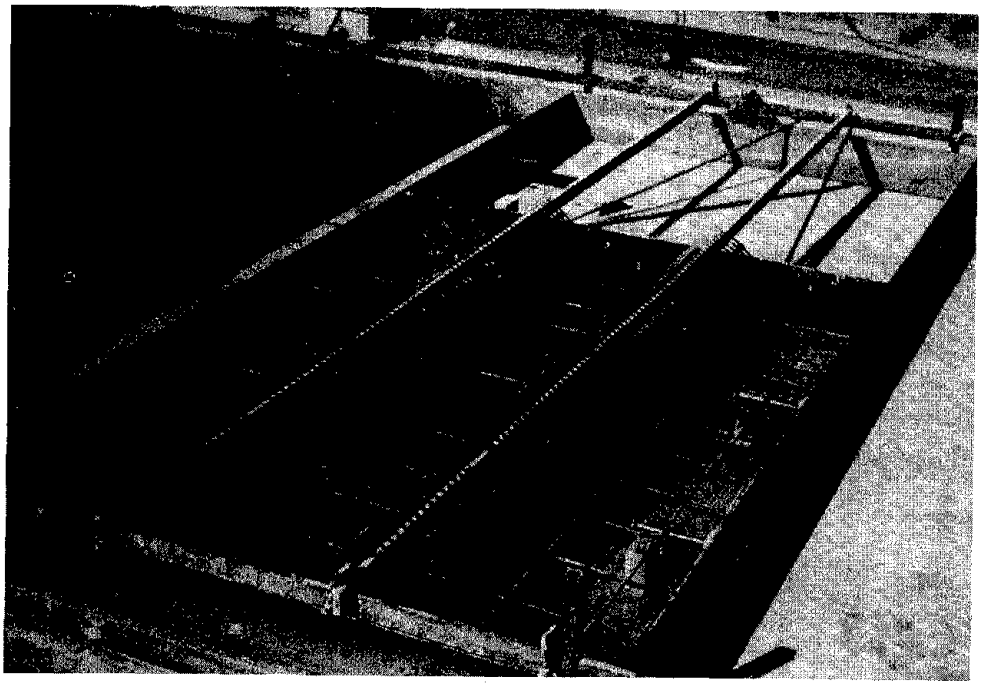


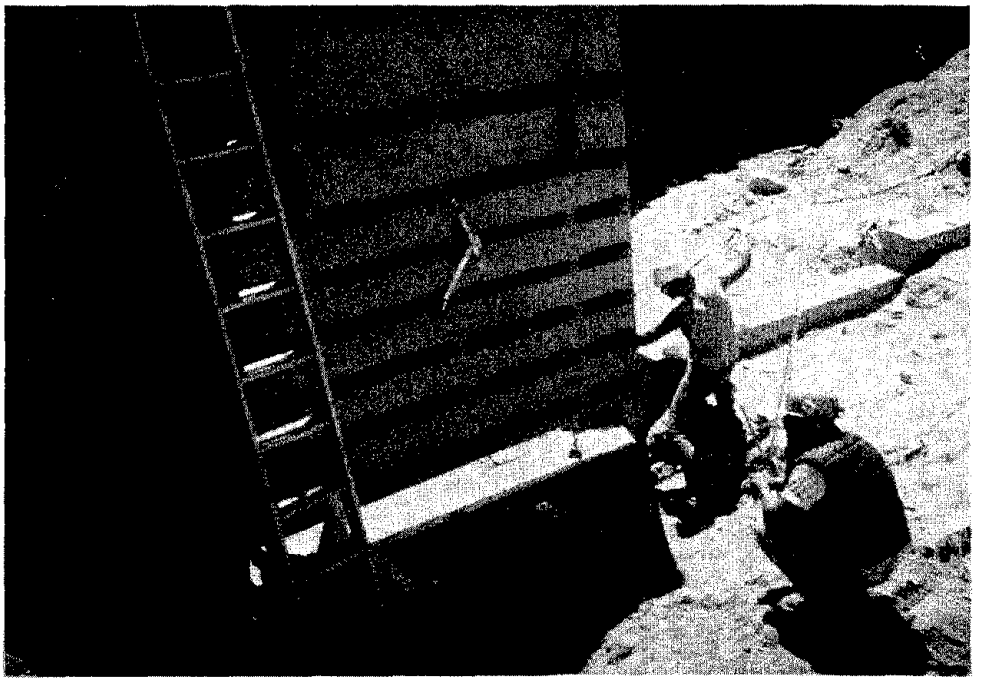
Figure 4-7. Erection brace detail for the Tanque Verde Road project [from Dames and Moore, 1985].



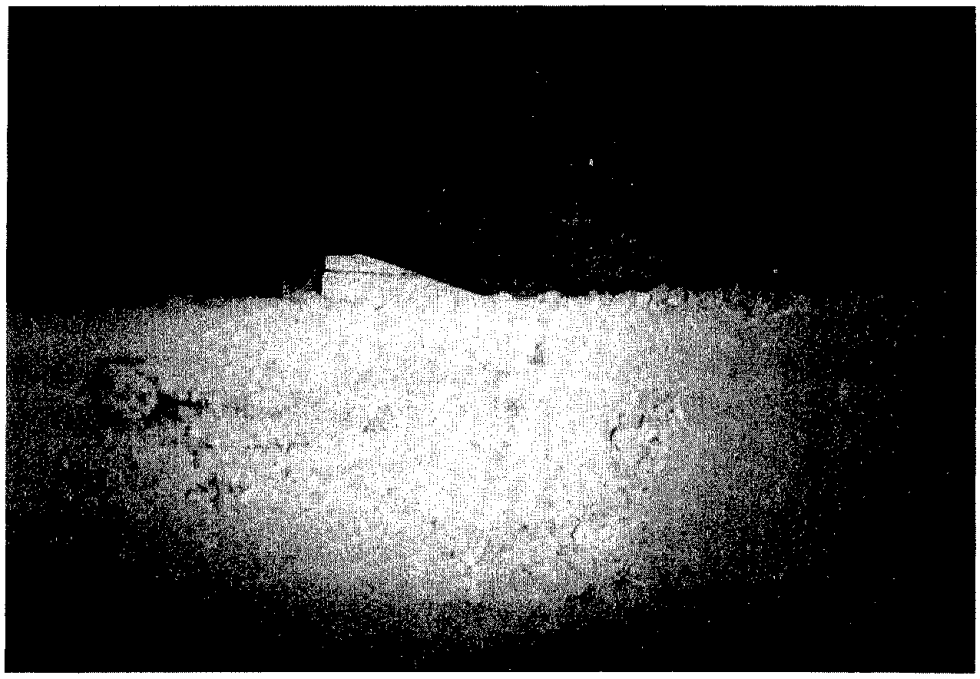
Photograph 4-1. Casting of geogrid tabs (i.e., embedded geogrid loop) into facing panels).

Photograph 4-2. Precast facing panels with geogrid tabs embedded in the back face.

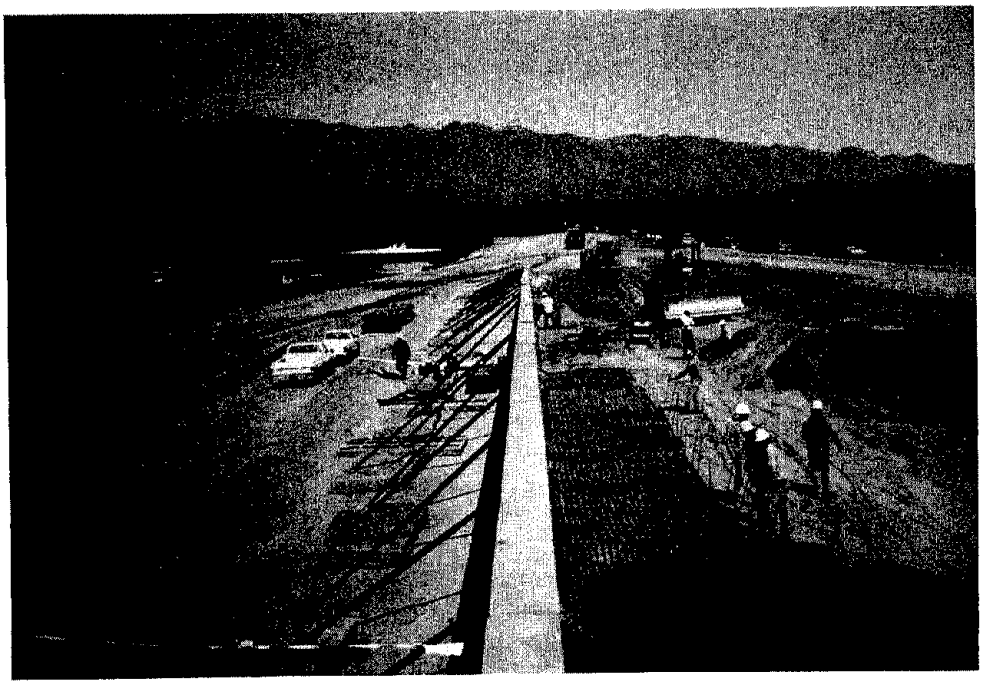
Photograph 4-3. Hoisting of the precast facing panels.



Photograph 4-4. Placement of the precast facing panels.

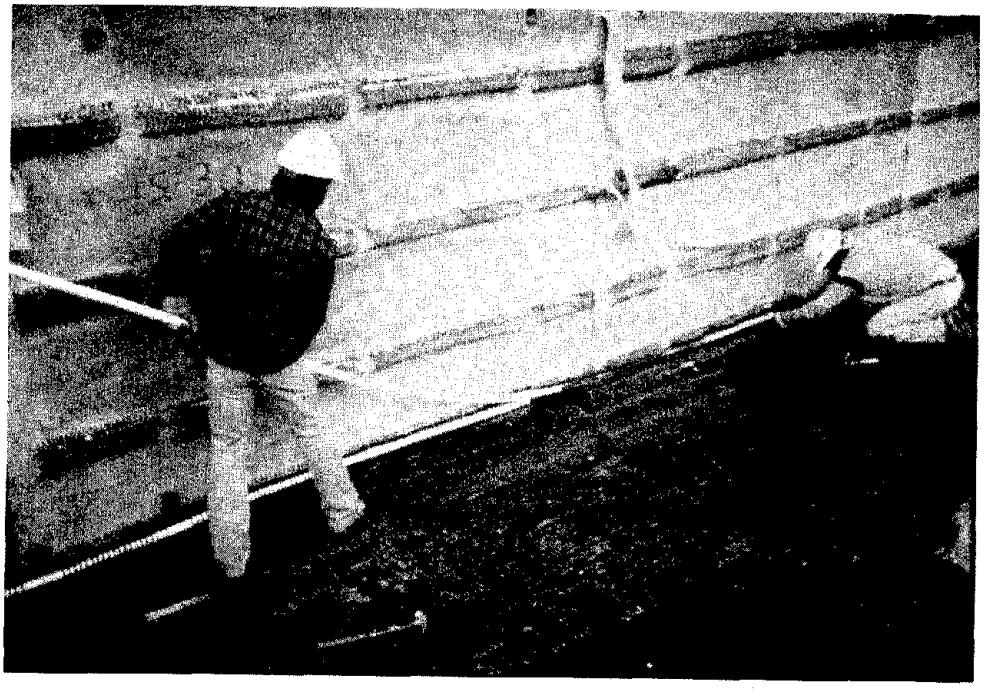


Photograph 4-5. Leveling bolts used to align the facing panels.

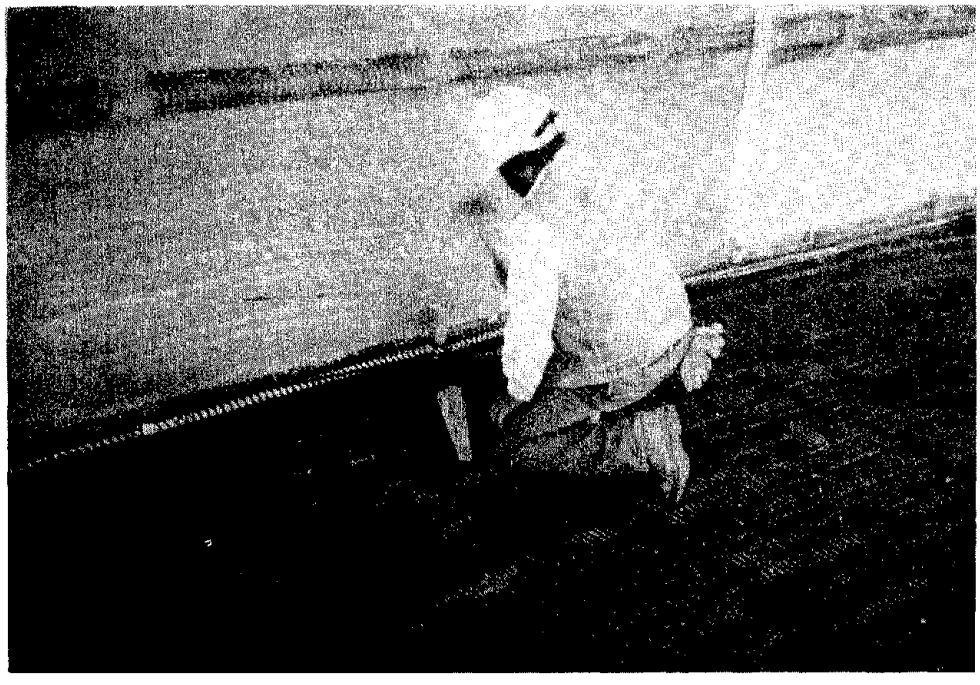


Photograph 4-6. Adjustable erection braces used to support and align facing panels during construction.

Photograph 4-7. Placement of concrete grout between the facing panel and leveling pad.

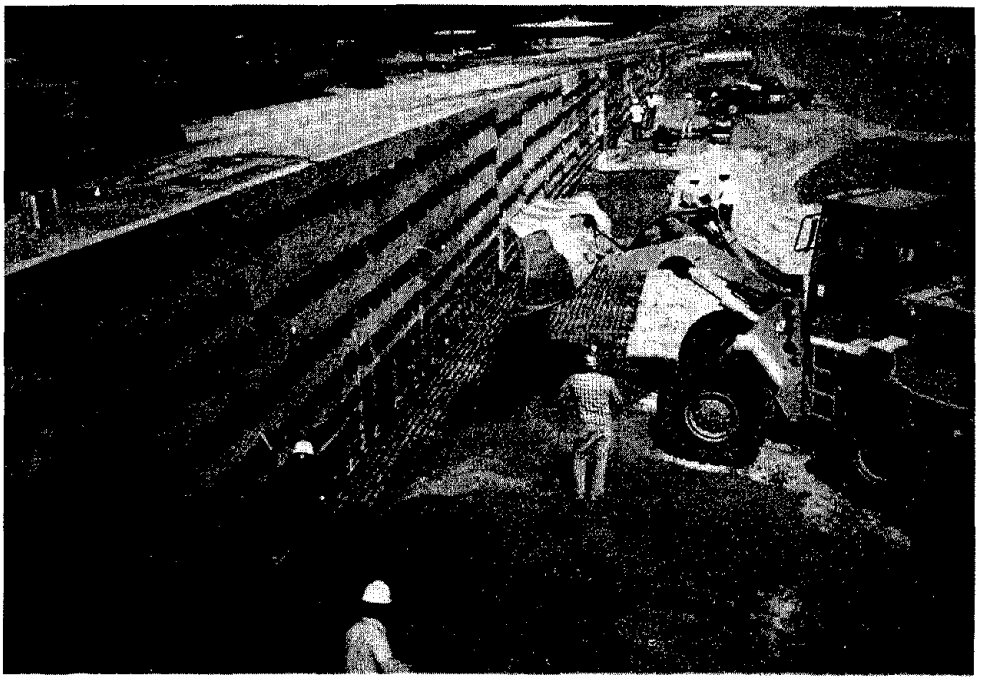


Photograph 4-8. Connection of geogrid tabs and geogrid reinforcement using PVC pipe connector.

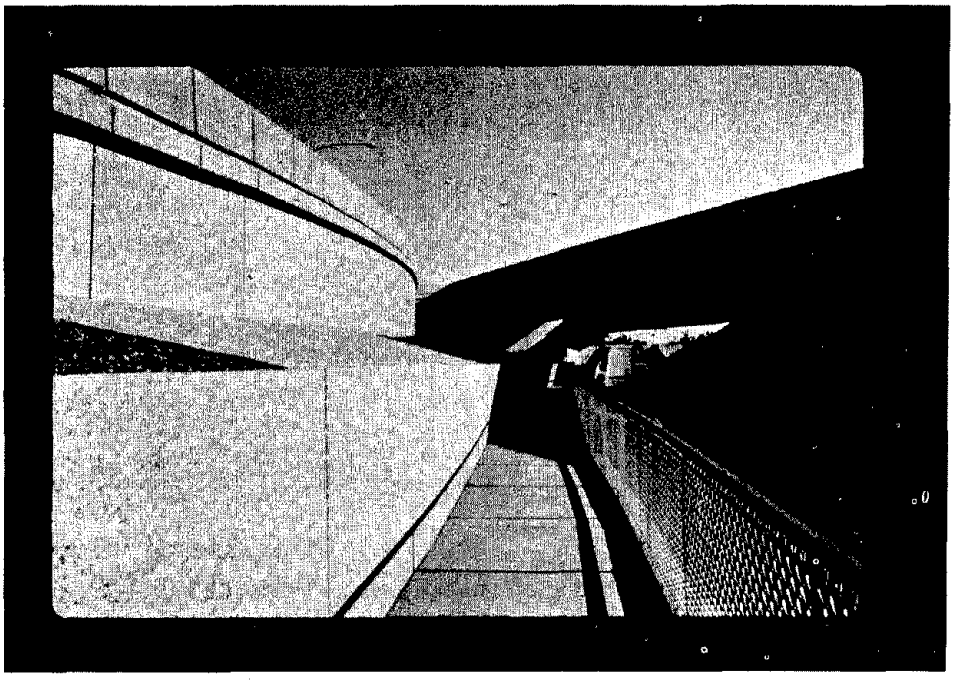


Photograph 4-9. Wood wedge used to pretension the geogrid looped connection.

Photograph 4-10. Steel forks used to pretension the geogrid reinforcement.



Photograph 4-11. Placement of granular fill over a layer of geogrid.



Photograph 4-12. Finished Tanque Verde Road project MSE wall with sandblasted panel finish.

5. INSTRUMENTATION PROGRAM

5.1 Introduction

An instrumentation program was developed with the goal of evaluating the performance of two sections of the geogrid-reinforced soil walls constructed for the Tanque Verde Road project. This goal was achieved by measuring the stresses, strains, and displacements of the two instrumented sections during and after construction. The external and internal temperatures of the wall were also monitored. The locations of the various instruments are described in Section 5.2. The types of instruments used on the project are described in Section 5.3. The calibration, preparation, installation, and monitoring of the instruments are summarized in Sections 5.4 through 5.7, respectively. A detailed description of the instrumentation program is presented in Desert Earth [1986].

5.2 Location of Instruments

Wall panels 26-30 and 26-32 were selected for instrumentation because they are the highest wall sections that are independent of three-dimensional effects (near the bridge abutment areas) and changes in foundation elevation. The location of these panels is shown in Figure 1-1. The locations of the instrumentation are shown in Figures 5-1 to 5-3 and listed in Tables 5-1 to 5-4. As previously noted, the panels are west facing. The inductance coils are located 3.3 ft from the south edge of the facing panels; the load cells, strain gages, and thermometers are located 3.3 ft from the north edge of the panels.

5.3 Description of Instruments

5.3.1 Pneumatic Load Cells

Eight SINCO Model 51482 pneumatic-type total pressure cells with a loading range of 0 to 300 psi were used to measure vertical stresses in the soil in the reinforced zone and horizontal stresses in the soil adjacent to the wall facing panel. Measurements of pressure readings were obtained using a SINCO Model 51421A pneumatic indicator with a loading range of 0 to 30 psi.

5.3.2 Resistance Strain Gages

Tensile strains in the geogrid reinforcement and geogrid tabs were measured with fifty Micromeasurement Model CEA-06-250-UN-350 foil-type resistance strain gages. Twelve similar gages bonded to small pieces of geogrid that were placed in the fill were used as temperature compensating gages. Strain readings were taken using a Micromeasurement Model P3500 strain indicator. The strain gages were wired to this unit through Micromeasurement Model SB10 switch boxes.

5.3.3 Inductance Coils

Tensile strains in the geogrid reinforcement and horizontal and vertical strains in the reinforced fill were deduced from displacements measured by fifty-two Bison magnetic inductance coils, grouped in twenty pairs and four triplets. These strains were determined with a Bison Model 4101A strain indicator.

5.3.4 Surveying Devices

Surveying points were located on the wall face as shown in Figure 5-3. Horizontal movements of the facing panels with respect to the leveling pads were monitored during construction and up to one month after construction. After this period, the wall faces were sand blasted or lightly jackhammered (to create an exposed aggregate finish) and, consequently, the surveying points were destroyed. The type of surveying instrument used to survey the wall face is not known. Therefore, the calibration, preparation, or instrumentation of the surveying devices is not discussed in the following sections.

5.3.5 Resistance Thermometers

Eighteen SINCO Carlson Model TM-1 resistance thermometers with a temperature range of 0° to 180°F were used to monitor the external and internal temperatures of the reinforced soil wall.

5.4 Calibration of Instruments

The instruments were calibrated in the laboratory, and the field accuracies of the instruments were estimated for the range of pressures, strains, or temperatures that the instruments measure. The estimated field accuracies of the instruments, presented in Table 5-5, are based on the inherent inaccuracies of the instruments, the limitations of the laboratory calibration techniques, and a limited review of literature on instrumentation. The Desert Earth [1986] report presents a thorough discussion of instrumentation calibration and field accuracy.

5.5 Preparation of Instruments

Each instrument was connected via an electrical or pneumatic cable to a switchbox or terminal post in a secure weatherproof enclosure at the base of panel 26-31. To avoid damage from sunlight, weather, vandalism, sandblasting, and other construction operations, the cables were routed through the fill behind the wall, under panel 26-31, and through protective conduits to the enclosures. To permit routing of the cables under the wall, troughs were molded by Sundt in the leveling pad under panel 26-31, when the pad was formed.

To save time in the field and ensure proper connections and placements of instruments, the electrical or plumbing connections, and grouping of instruments were done in advance of the start of construction, whenever possible. Thus, three harnesses were assembled, containing load cells, inductance coils, and resistance thermometers, all attached to cables grouped in bundles. These harnesses were transported to the site, and the cables were routed through the troughs in the leveling pad. The free ends of the cables were then connected to the associated switchboxes or terminal posts in the enclosure. The sensors were hung up out of the way on the inside of the wall. During the placement of the fill, the personnel installing the instruments needed only to bring the appropriate sensors, with cables attached, down from the wall and place them at the proper locations in the fill. Since a given instrument was connected in advance, the instruments could be monitored during the construction process.

Because the resistance strain gages, with cables attached, were bonded to the geogrid reinforcement in advance of the construction, the above procedure could not be followed with them. Instead, the cables from the

resistance strain gages were pulled through an empty conduit (run against the inside of panel 26-31, under the panel, and up to the enclosure) and connected to the switchboxes as each grid was placed. The necessity of making these electrical connections during the placement of the lifts made the installation of the strain-gaged geogrids troublesome and time consuming.

5.6 Installation of Instruments

All instruments were placed during construction in conjunction with installation of the uninstrumented geogrid reinforcement. The load cells used to measure horizontal stresses were either bonded directly to the wall panel with contact cement or placed about 12 in. away, oriented parallel to the wall panel, with their sensitive sides away from the wall. The load cells used to measure vertical stresses were buried with the sensitive side facing up. The resistance strain gages were bonded to predetermined strips of geogrid reinforcement before construction. At the appropriate lift levels, the geogrids were carefully carried to their locations and unrolled in place. As previously noted, temperature compensating strain gages were mounted to "dummy" pieces of geogrid material that were installed in the vicinity of the active gages. Inductance coils used to measure horizontal tensile strains in the geogrids were bolted to predrilled holes in the geogrids. The inductance coils used to measure vertical and horizontal soil strains were placed directly in the soil. Temperature probes were attached outside of the reinforced soil wall and placed at predetermined locations in the wall.

The sensors within the reinforced soil wall were placed at predetermined levels and covered with sand. Once all instruments and leads were in place on a particular lift, fill was placed on top of the lift with a front end loader and carefully pushed out over the instruments. As previously mentioned, the lift thicknesses for the instrumented panels were 1.0 to 1.5 ft, rather than the usual 6 in., to minimize damage to the instruments.

5.7 Monitoring of Instruments

During construction of wall panels 26-30 and 26-32, data was acquired as the wall fill was placed. At the beginning of each construction day, an initial daily reading of all the instruments in place was taken. Instrument readings were then taken after each of the following stages: geogrid pretensioning, fill dumping and spreading, and lift compaction. After the

two instrumented wall sections were completed, readings were taken at the times shown on Table 5-6.

Table 5-1. Load cell locations.

Facing Panel	Vertical Distance from Leveling Pad (ft)	Horizontal Distance from Facing Panel (ft)	Measurement
26-30	2.0	0.0	horizontal
	4.0	0.0	pressure
	8.0	0.0	
	12.5	0.0	
26-32	5.0	0.5	vertical
	5.0	2.5	pressure
	5.0	5.0	
	5.0	8.0	

Table 5.2. Resistance strain gage locations.

Facing Panel	Vertical Distance from Leveling Pad (in.)	Horizontal Distance from Facing Panel (in.)
26-30	18	12.5
	"	20.5
	"	29.3
	"	54.5
	"	102.3
	"	128.0
	54	13.0
	"	30.0
	"	42.5
	"	55.5
	"	81.0
	174	15.0
	"	56.5
	"	69.0
	"	77.0
	"	85.0
	"	90.0
	"	124.0
26-32	18	13.0
	"	21.0
	"	29.0
	"	55.0
	"	102.0
	"	125.0
	54	13.5
	"	30.0
	"	38.0
	"	43.0
	"	56.0
	"	80.0*
	138	15.0*
	"	56.0*
	"	76.0*
	"	84.0*
	"	126.0*

* Strain gage locations have been estimated from plotted strain gage results.

Table 5-3. Inductance coil locations.

Facing Panel	Vertical Distance from Leveling Pad (ft)	Horizontal Distance from Facing Panel (ft)	Measurement
26-30	0.5	0.5	geogrid
	0.5	2.5	strain
	3.5	0.5	
	3.5	3.0	
	8.0	0.5	
	8.0	3.0	
	8.0	6.0	
	8.0	9.0	
	14.5	0.5	
	14.5	4.0	
	14.5	7.0	
	14.5	10.0	
26-32	0.5	0.5	
	0.5	2.5	
	3.5	0.5	
	3.5	3.0	
	8.0	0.5	
	8.0	3.0	
	8.0	6.0	
	8.0	9.0	
	11.5	0.5	
	11.5	4.0	
	11.5	7.0	
	11.5	10.0	
26-30	3.5	0.5	soil
	3.5	3.0	strain
26-32	3.5	0.5	
	3.5	3.0	

Table 5-4. Resistance thermometer locations.

Facing Panel	Vertical Distance from Leveling Pad (ft)	Horizontal Distance from Facing Panel (ft)
26-30	2.5	surface
	9.0	surface
	2.5	0.0
	2.5	7.0
	9.0	0.5
	9.0	4.5
	14.5	0.5
	14.5	4.0
	14.5	8.0
26-32	2.5	surface
	9.0	surface
	2.5	0.0
	2.5	6.0
	9.0	0.5
	9.0	4.5
	14.5	0.5
	14.5	4.0
	14.5	8.0

Table 5-5. Estimated field accuracy of each of the instruments.

INSTRUMENT	ESTIMATED FIELD ACCURACY
Pneumatic Load Cells	
SINCO Model 51482 Total Pressure Cell SINCO Pneumatic Indicator Model 51421A	± 20%
Resistance Strain Gages	
Micromeasurement Model CEA-06-250-UN-350 gages Micromeasurement Model SB10 Switch Box Micromeasurement Model P3500 Digital Strain Indicator	± 0.05%
Inductance Coils	
Bison Inductance Coils Bison Model 4101A Strain Indicator	± 20%
Resistance Thermometers	
Carlson Resistance Thermometers TM-1	± 1°F

Table 5-6. Instrument monitoring schedule.

Recording Time	Date
During Placement of Wall Fill	9/23/85 to 10/7/85
2 Days after Placement of Wall Fill	10/9/85
1 Week after Placement of Wall Fill	10/13/85
2 Weeks after Placement of Wall Fill	10/20/85
3 Weeks after Placement of Wall Fill	10/27/85
1 Day after Placement of Pavement	10/30/85
3 Days after Placement of Pavement	11/1/85
1 1/2 Weeks after Placement of Pavement	11/13/85
1 Month after Placement of Pavement	12/1/85
2 1/2 Months after Placement of Pavement	1/15/86
Approach Opened to Traffic for 2 Weeks	2/13/86
Approach Opened to Traffic for 1 1/2 Months	3/19/86
Every 3 Months	6/15/86
	9/13/86
	11/8/86
	9/23/87

Note: The surveying points were monitored during the placement of the wall fill and about one month after placement of the wall fill. After this period, the wall faces were sandblasted or lightly jackhammered and, consequently, the surveying points were destroyed.

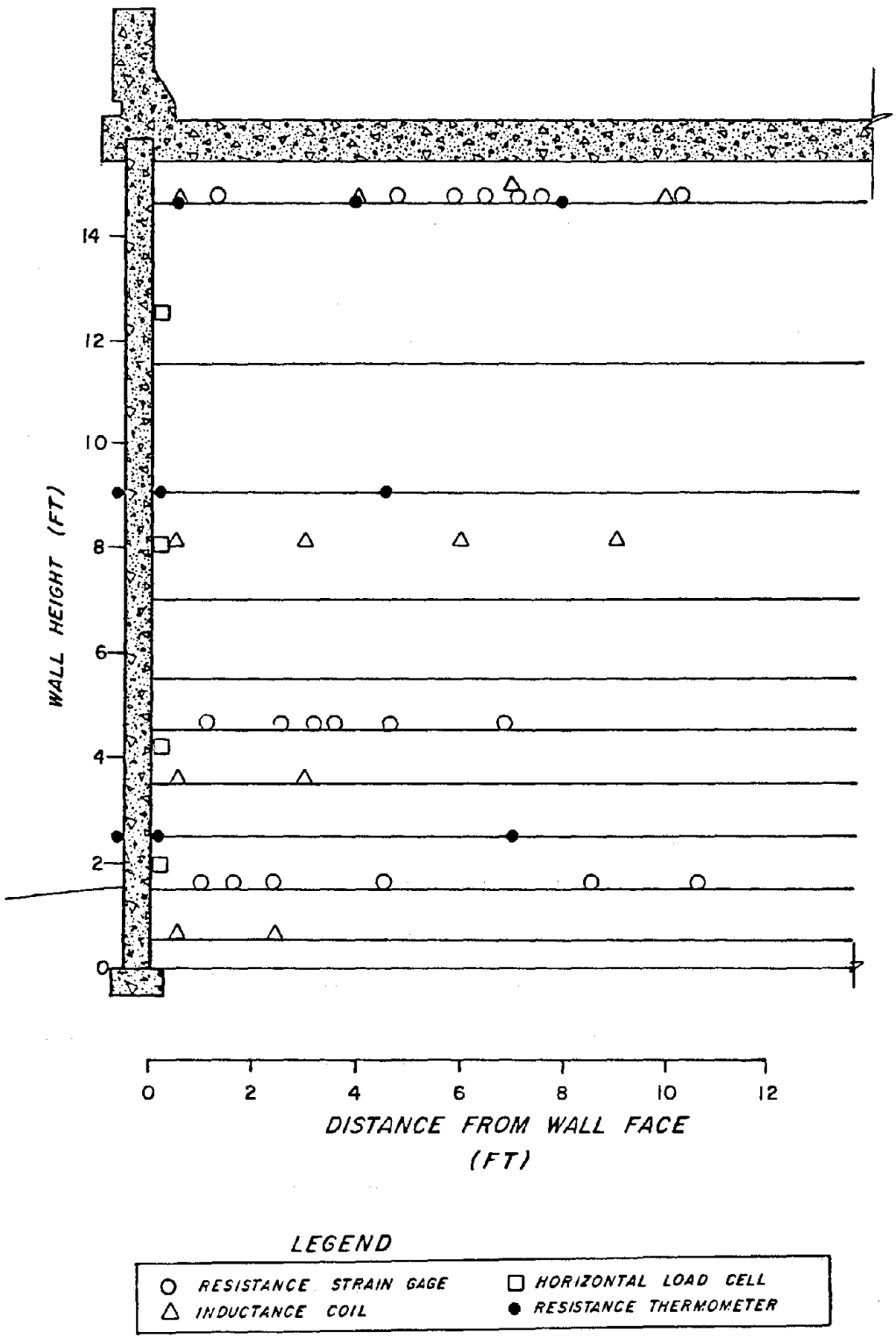


Figure 5-1. Locations of instruments for wall panel 26-30.

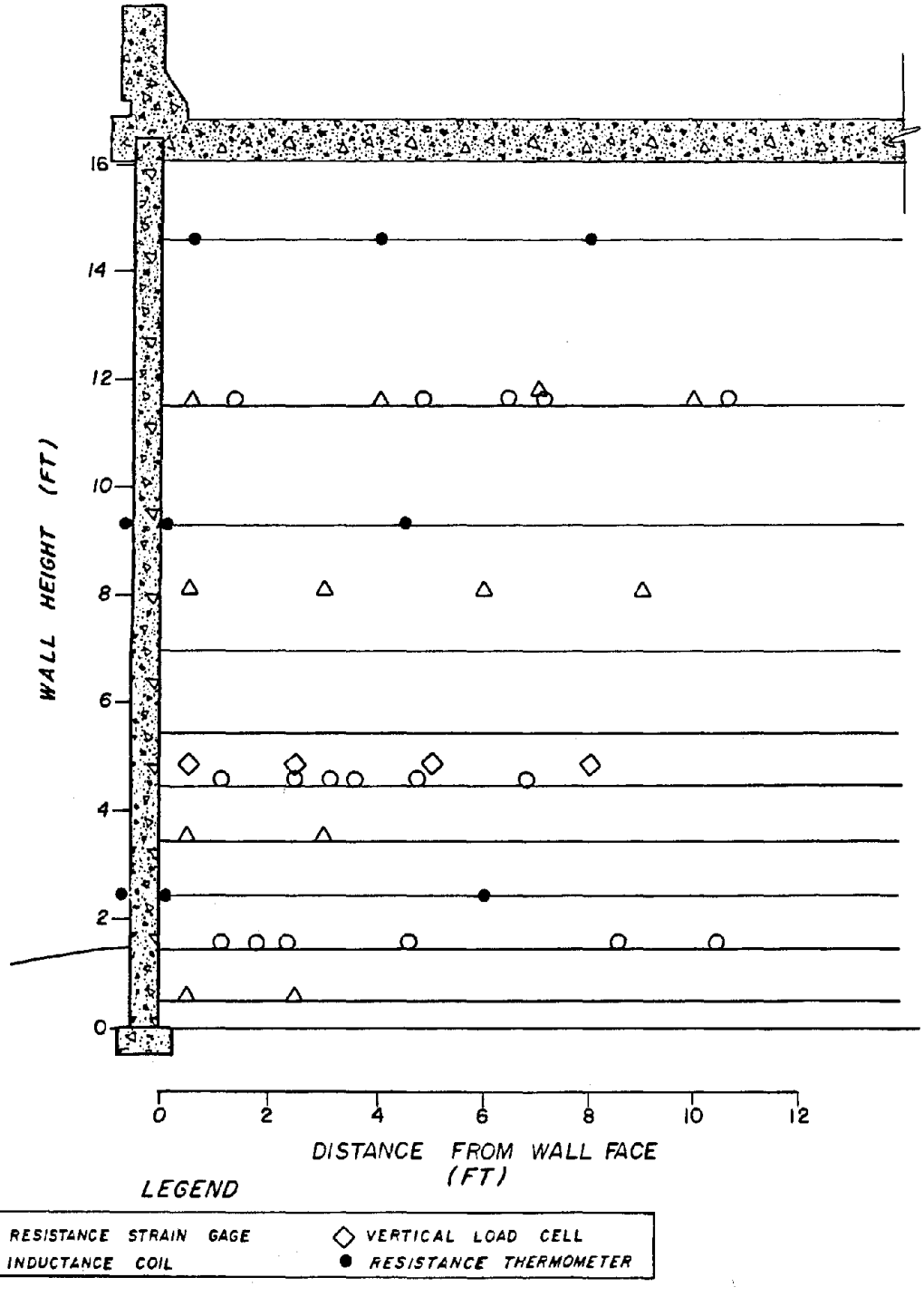


Figure 5-2. Locations of instruments for wall panel 26-32.

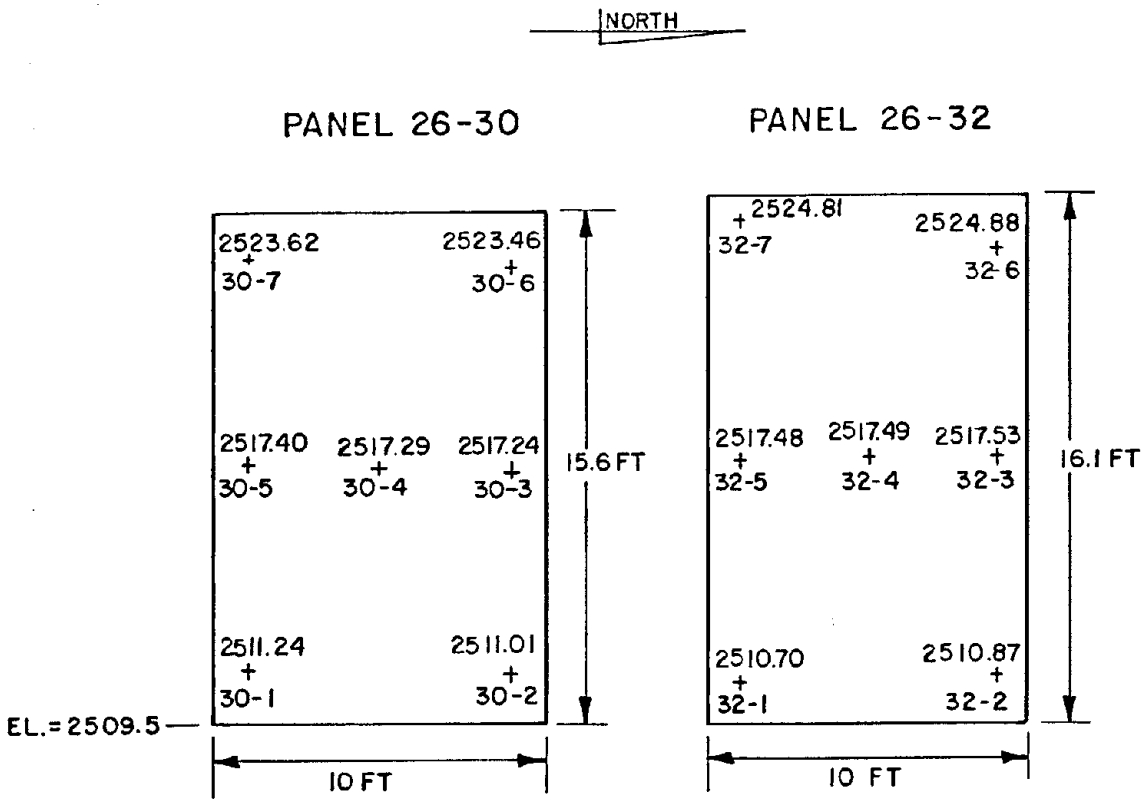


Figure 5-3. Location of survey points on wall faces.

6. INSTRUMENTATION RESULTS

6.1 Introduction

This chapter presents the results of the instrumentation measurements obtained by Desert Earth up to 8 November 1986. Limited results from measurements taken on 23 September 1987 are also presented. Section 6.2 discusses the horizontal and vertical stresses in the soil, measured with the pneumatic load cells, and compares them to the design stresses obtained from the Dames and Moore calculations. The vertical and horizontal soil strains deduced from the magnetic inductance coils are presented in Section 6.3. Section 6.4 presents the geogrid strains measured with the foil-type strain gages and those deduced from the magnetic inductance coils. In particular, the geogrid reinforcement strains during construction, approximately two weeks after construction, and approximately one year after construction and the geogrid tab strains are discussed. The panel movements measured with survey equipment are presented in Section 6.5. Section 6.6 presents the temperatures at the wall face and within the reinforced soil mass measured with resistance thermometers and compares them to the temperatures predicted in Section 2.3.4.

A discussion of the instrumentation results and the performance of the instrumented wall sections is presented in Chapter 7.

6.2 Soil Stresses

6.2.1 Vertical Soil Stresses

Vertical soil stresses in wall 26-32 were measured and compared with vertical stresses obtained from calculations based on the three previously discussed assumptions regarding the vertical stress distribution (Figure 3-4). The vertical soil stress measurements all correspond to a distance of 5 ft above the leveling pad. As shown in Figure 6-1, the measured vertical stresses are not well represented by any of the assumed stress distributions. Measured stresses near the wall face are quite low. In contrast, measured stresses in the central portion of the reinforced soil mass are, on average, larger than those obtained using any of the assumed stress distributions. The small measured vertical stress values near the wall face are probably due

to a combination of upward shear stresses in the soil imparted by wall friction, vertical load transfer from the reinforcement to the facing panels by tensioned membrane action, and soil arching over the load cells. The large measured vertical stress values in the central portion of the reinforced soil mass, relative to the calculated design values, may be due to factors such as instrumentation inaccuracies and variations in soil properties. The largest vertical stresses were measured on 15 June 1986, 13 September 1986, and 23 September 1987. These dates correspond to the three hottest days during which measurements were taken. It can be hypothesized that the large measured stresses may be due to a real temperature effect on the wall system (e.g., the high temperature causes the system to try to expand slightly, thereby increasing the stresses) or, instead, to the effect of temperature on the instrumentation. To determine the real reason(s) for the large measured stresses would require additional work.

6.2.2 Horizontal Soil Stresses

Horizontal soil stresses near the face of wall 26-30 were measured to evaluate the magnitude of the stresses exerted on the wall facing panels and the tensions in the geogrid connections. These measured values were also compared with the calculated horizontal stresses obtained using three different analysis or design procedures: (i) Rankine analysis; (ii) coherent gravity design method with a Meyerhof type of vertical stress distribution; and (iii) tie-back wedge design method with a trapezoidal vertical stress distribution. As shown in Figure 6-2, the measured horizontal stresses are close to the active Rankine values near the toe of the wall. Elsewhere, the measured horizontal stresses are, on average, substantially less than the maximums obtained with the various analysis or design methods. It is interesting that low horizontal stresses were measured in the vicinity of the facing panel, where low vertical stresses were also measured. Therefore, the low measured horizontal stresses may be due simply to the low vertical stresses (since $\sigma_n' = K_r\sigma_r'$). Alternatively, the low horizontal stresses may be due, in part, to soil arching between the reinforcement layers. As with the vertical stress measurements, the largest horizontal stresses were measured on 15 June 1986, 13 September 1986, and 23 September 1987. Thus, again, there appears to be a correlation between temperature and the measured stresses. As with the measured vertical stresses, the causes of the variations in horizontal stresses with temperature are unknown.

6.3 Soil Strains

6.3.1 Vertical Soil Strains

Figures 6-3 and 6-4 present results of vertical soil strain measurements for walls 26-30 and 26-32. These measured strains were adjusted to exclude strains induced by placement and compaction of the first lift of fill above the inductance coils. The strain measurements all correspond to a distance of 3.5 ft above the leveling pad. The limited vertical soil strain data show an increase in vertical strain with an increase in distance from the wall face. This result is consistent with the measured vertical stresses which showed an increase in vertical stress from the wall face to the central portion of the reinforced soil mass. The average vertical strain measurements between the two walls are consistent. However, the vertical soil strain for each wall varies erratically over time. The reason for this variation is not known; however, it is felt to be due to instrumentation factors, such as the inaccuracy of the inductance coil readings at low strains (less than 1%), rather than to a real physical effect. The vertical strain measurements do not show the temperature effect that is found in the stress measurements. This suggests that the stress increases observed during periods of high temperature are due to a temperature dependency in the load cells not accounted for in the calibrations.

Assuming the readings from 20 October 1985 are the most reliable, and considering only these, the maximum measured vertical soil strain in wall 26-30 was about 0.6% and the maximum vertical strain in wall 26-32 was about 0.4%, as shown in Figures 6-3 and 6-4. These vertical strain readings include the strain induced by the placement and compaction of all fill above the instrumentation except for the first fill lift (i.e., strain induced by placement and compaction of the first 1.0 ft of fill above the inductance coils). The vertical strains due to placement and compaction of the first fill lift above the inductance coils are quite variable (in the range of 0 to 2%) and reflect localized construction and compaction effects. The strains due to placement and compaction of the first fill lift can be found in Desert Earth [1986] and are not repeated here.

6.3.2 Horizontal Soil Strains

Horizontal soil strains measured using the magnetic inductance coils are shown in Figures 6-5 and 6-6. As with the vertical soil strain measurements, the results shown in Figures 6-5 and 6-6 were adjusted to exclude strains induced during placement and compaction of the first lift of fill over the inductance coils. The horizontal soil strain measurements correspond to a distance of 8 ft above the leveling pad. Again assuming that the readings from 20 October 1985 are the most reliable, and considering only these, the maximum horizontal strain at 0.5 ft from the wall face was about 1.7% in wall 26-30 and 2.0% in wall 26-32. These strains decrease to a relatively constant average value of about 0.7% at 3 ft from the wall face. The relatively large soil strains near the wall face, which occur after the placement and compaction of the first lift of fill over the inductance coils, are believed to be associated with movement of the wall panels and extension of the geogrid connection resulting from removal of the temporary erection braces and elongation of the geogrid-to-panel connections.

6.4 Geogrid Strains

6.4.1 Reinforcement Strains During Construction

Geogrid reinforcement strains were measured during the construction of walls 26-30 and 26-32 to determine the effects of various construction events (e.g., pretensioning of geogrid, placement of first lift of fill over the geogrid, compaction of first lift of fill over the geogrid, and loosening of temporary erection braces) on geogrid strain. Figures 6-7 to 6-9 show, respectively, the geogrid strains measured with strain gages during construction for wall 26-30 at elevations of 1.5 ft, 4.5 ft, and 14.5 ft above the leveling pad. Although the geogrid strain during construction was measured at all strain gages, only the results from three strain gages are reported at each level. [A complete presentation of geogrid strains measured during construction in wall 26-30 with inductance coils and in wall 26-32 with strain gages and inductance coils can be found in Desert Earth [1986] and is not repeated here.]

Geogrid strains measured in walls 26-30 and 26-32 after pretensioning the geogrid range from -0.1 to 2.4%. Comparison of strains measured by inductance coils and strain gages show that the strains deduced from the

inductance coils ranged from -0.1 to 2.4% after pretensioning, whereas the strains measured by the strain gages ranged from -0.1 to 0.1% after pretensioning. The large variation of strains deduced from the inductance coil readings is possibly a result of instrumentation factors, such as the inaccuracy of the inductance coils at low strain values (less than 1%). It is believed that the strain gage measurements are more reliable than the inductance coil measurements. Therefore, only strains measured with strain gages are considered during the construction period.

The strains measured with strain gages after placement and compaction of the first lift of soil over the various geogrid layers ranged from -0.2 to 0.2% for both walls. The negative geogrid strain values indicate that localized bending of the geogrid took place during compaction. The figures show that as additional fill lifts were placed, geogrid strains increased as the wall facing yielded and shear stresses at the soil-geogrid interface caused the geogrid to go into tension.

The strains measured with strain gages after the erection braces were first loosened ranged from -0.1 to 0.2% for both walls. It should be noted that loosening the temporary erection braces had little effect on the measured strains.

Overall, the geogrid strains measured during construction were small and showed only minor localized construction and placement effects. However, Desert Earth felt that these minor effects could be significant when compared to the small magnitude of geogrid strains that were measured. Hence, Desert Earth adjusted geogrid strain results measured after the wall construction period. The geogrid strains were adjusted by subtracting the cumulative strains that had occurred after compaction of the first lift of fill over the considered geogrid layer (i.e., the strains induced by pretensioning the geogrid and placement and compaction of the first lift of fill over the geogrid) from the measured strain results. As discussed above, the cumulative strains measured with strain gages after compaction of the first lift of fill ranged from -0.2 to 0.2% for both walls.

6.4.2 Adjusted Reinforcement Strains

Results of adjusted geogrid strain measurements recorded from inductance coils and strain gages on 20 October 1985 (about two weeks after the walls

were constructed) and 8 November 1986 (about one year after the walls were constructed) are presented in Figures 6-10 to 6-17. As previously discussed, these adjusted strains values do not include strains associated with geogrid placement and compaction of the first fill lift over the geogrid. Comparison of adjusted strain measurements between walls 26-30 and 26-32 on 20 October 1985 and 8 November 1986 and comparison of strain measured by inductance coils and by strain gages show the following: geogrid strains do not appear to be affected by geogrid elevation; geogrid strains were greatest near the wall face; the geogrid in wall 26-30 strained less than the geogrid in wall 26-32; measured strains showed little change from 20 October 1985 to 8 November 1986; the inductance coils measured approximately the same adjusted strains as the strain gages; and the inductance coils showed more variation in strain measurements than the strain gages. The adjusted strains measured on 20 October 1985 with inductance coils varied from 0.2 to 0.8% and averaged about 0.4%; the adjusted strains measured with strain gages varied from 0 to 0.5% and averaged about 0.3%. The strains measured on 8 November 1986 with inductance coils varied from 0.1% to 0.8% and averaged about 0.3%; the strains measured with strain gages varied from 0% to 0.4% and averaged about 0.2%. The larger variation of inductance coil measurements is believed to result from inaccuracy of inductance coils at low strains and rotation of the inductance coils caused by geogrid bending near the wall face.

6.4.3 Connection Strains

Connection strains were measured with strain gages located on the geogrid tabs above and below the PVC pipe connector. Unadjusted results from strain measurements of the geogrid tabs are presented in Table 6-1. These results show that the tensile strains in the top of the tabs are significantly greater than those in the bottom of the tabs, which indicates that the tabs were being pulled down by the geogrid reinforcement. Table 6-1 also shows that the maximum strains in the geogrid tabs do not occur as the connection is pretensioned using the wood wedges, but rather after wall construction. The maximum connection strain at the end of wall construction was about 1%.

6.5 Panel Movements

Horizontal movements of the survey points of walls 26-30 and 26-32, which were positioned on the wall panels as shown in Figure 5-3, are presented in Tables 6-2 and 6-3 and have been analyzed by Berg, et al. [1986]. The wall face movements were monitored during the construction period and up to one

month after the construction period. The measured deformations of the two panels were very consistent. During the construction period, the tops of both wall sections moved out approximately 3.0 in., while the toes of both walls remained stationary. The majority of this movement occurred between 30 September 1985 and 2 October 1985, which coincides with the time period in which the erection braces were removed. The total amount of movement exceeded the initial wall batter of about 2.0 in. (1H:96V) by about 1.0 in.

6.6 Temperature

Temperature measurements on the wall face and within the reinforced soil mass taken on 19 March 1986 and 15 June 1986 are presented in Figures 6-18 and 6-19. The highest temperature recorded on the wall face during the monitoring program was 124°F and occurred on 15 June 1986.

The temperatures measured on the hottest day (15 June 1986) show that the temperatures in the wall fill near the top layer of geogrid are controlled by heat transfer through the pavement. The maximum temperature in the fill at this level was 101°F. The bottom-most level of temperature readings show that the temperature at this level is influenced by the close proximity of natural ground. The maximum recorded temperature at this level was 86°F just behind the wall face and 77°F at a distance of about 6 ft behind the wall face.

The temperature recorded at the base of the wall just behind the wall face is lower than would be predicted using the results from the heat flow analysis presented in Section 2.3. Apparently, the natural ground is acting as a heat sink, drawing heat out of the backfill, and thereby lowering backfill temperatures near the base of the wall. This is a useful observation as the lower geogrid layers in a reinforced soil wall are usually the most highly stressed. It is also interesting to note the high values of wall face temperature measured at the bottom of the wall. These high values (compared to the ones measured at higher elevations) are probably due to the localized effect of radiative heat transfer from the ground to the wall panel and surrounding air.

The thermometer readings at approximately the mid-height of the wall appear to be the most representative for considering the influence of temperature on wall behavior. The temperature distribution from measurements on the middle level of thermometers from Figures 6-18 and 6-19 are plotted

in Figure 6-20 and compared to the temperature distribution shown in Figure 2-4, which was predicted from a one-dimensional heat flow analysis. From Figure 6-20, the predicted temperatures are underestimated because the actual temperatures at the surface of the concrete panels of 113°F and 115°F are higher than the temperature estimated in Section 2.3 of 104°F. Figure 6-20 also shows the calculated temperature distribution for a wall face temperature of 114°F and an average monthly temperature of 86°F, and the measured temperature distribution. This calculated distribution is in good agreement with the measured temperatures.

Several practical conclusions can be culled from the temperature measurements:

1) Even with temperatures up to 115°F at the surface of the concrete panels (which corresponds to an ambient air temperature of 104°F), temperatures within the reinforced soil mass do not exceed about 95°F to 100°F.

2) The temperature within the reinforced soil wall drops quickly with distance from the wall.

The effect of temperature on geogrid strains was also investigated by examining strain measurements recorded on the hottest day (15 June 1986) and on the following reading (13 September 1986). Any adverse effects of the elevated temperatures would be expected to be seen as increased geogrid strains. As will be shown in Section 7.6, no significant increase was observed.

Table 6-1. Measured strains in the looped geogrid tab.

		Vertical Distance from Leveling Pad (ft)	Top Strain (%)	Bottom Strain (%)
Measurements After Connection Pretensioning	Wall 26-30	1.5 4.5 14.5	0.645 0.121 -	-0.084 -0.016 0.021
	Wall 26-32	1.5 4.5 5.5* 14.5	0.060 - 0.041 0.194	- -0.068 0.091 0.002
Measurements on 20 October 1985	Wall 26-30	1.5 4.5 14.5	1.017 0.689 -	0.109 0.262 -0.035
	Wall 26-32	1.5 4.5 5.5* 14.5	0.118 - 0.238 0.512	- 0.067 0.523 0.016

* Extra strain gages attached to looped geogrid tab.

Table 6-2. Movement of survey points on wall 26-30.

	Movement of Survey Points (ft)						
Date	Point 1	Point 2	Point 3	Point 4	Point 5	Point 6	Point 7
9/24	0.00	0.02	0.01	0.01	-0.01	0.00	0.00
9/25	0.00	0.01	0.02	0.01	0.01	0.02	0.02
9/26	0.00	0.01	0.03	0.02	0.02	0.03	0.02
9/27	0.00	0.02	0.04	0.04	0.03	0.05	0.05
9/30	0.01	0.02	0.05	0.04	0.04	0.06	0.06
10/2	0.02	0.02	0.09	0.08	0.08	0.14	0.14
10/3	0.02	0.03	0.11	0.10	0.09	0.17	0.17
10/4	0.02	0.03	0.11	0.10	0.10	0.17	0.18
10/7	0.02	0.03	0.12	0.11	0.11	0.21	0.22
10/8	0.03	0.03	0.13	0.12	0.12	0.24	0.25
11/5	0.00	0.03	0.13	0.12	0.12	0.25	0.26

Table 6-3. Movement of survey points on wall 26-32.

	Movement of Survey Points (ft)						
Date	Point 1	Point 2	Point 3	Point 4	Point 5	Point 6	Point 7
9/24	0.01	0.00	0.01	0.02	-0.01	-0.03	-0.01
9/25	0.00	-0.01	0.01	0.01	0.01	0.01	0.01
9/26	0.00	-0.01	0.01	0.02	0.02	0.01	0.02
9/27	0.01	0.00	0.03	0.03	0.03	0.02	0.03
9/30	0.01	0.00	0.04	0.04	0.04	0.05	0.06
10/2	0.01	0.00	0.08	0.08	0.08	0.13	0.14
10/3	0.01	0.01	0.08	0.08	0.09	0.16	0.16
10/4	0.02	0.01	0.09	0.09	0.09	0.17	0.17
10/7	0.01	0.00	0.11	0.11	0.11	0.22	0.22
10/8	0.01	0.01	0.12	0.12	0.12	0.24	0.26
11/5	0.01	0.00	0.12	0.09	0.12	0.28	0.27

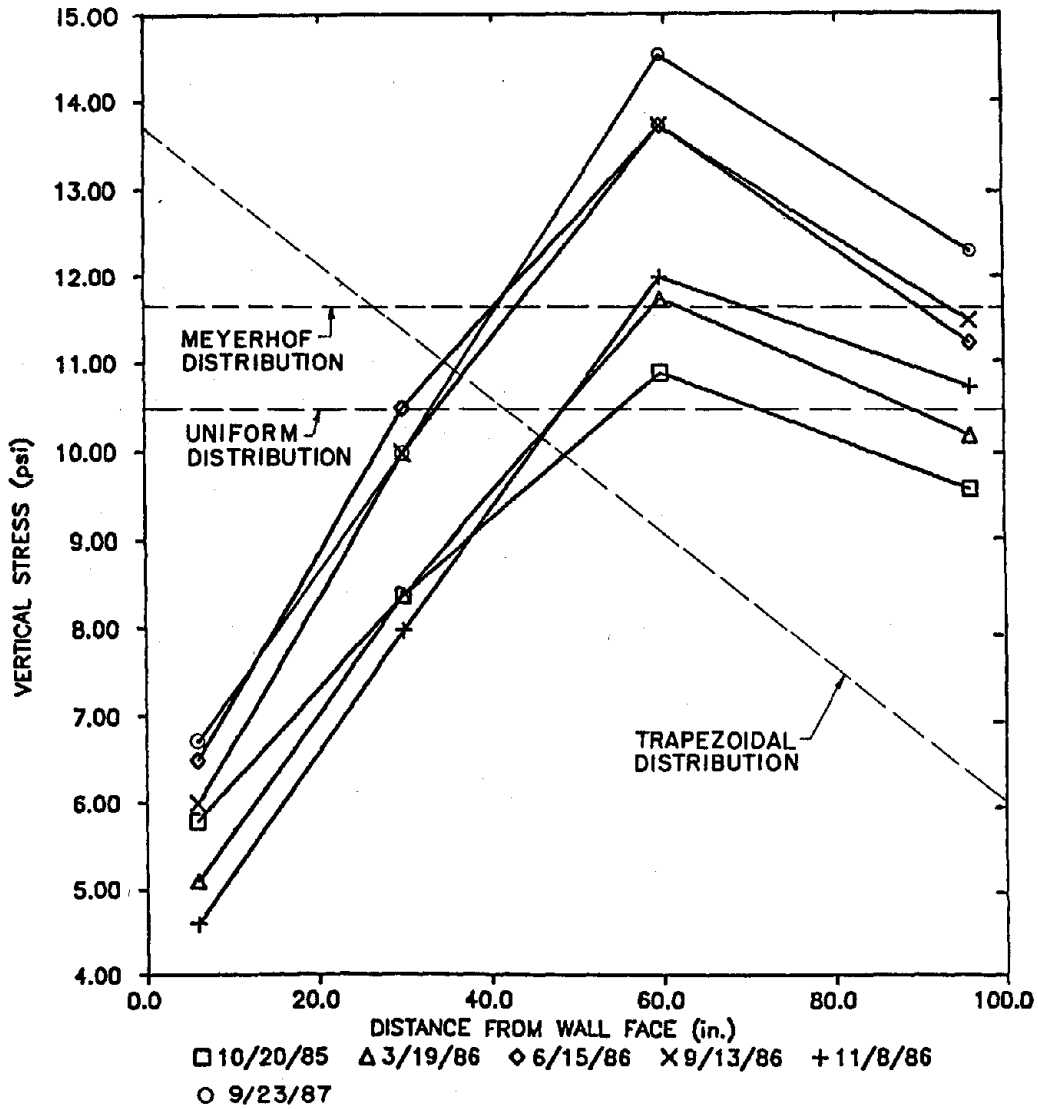


Figure 6-1. Comparison of measured vertical soil stresses in wall 26-32 at a distance of 5 ft from the leveling pad with the stresses obtained from design calculations [from Desert Earth, 1987].

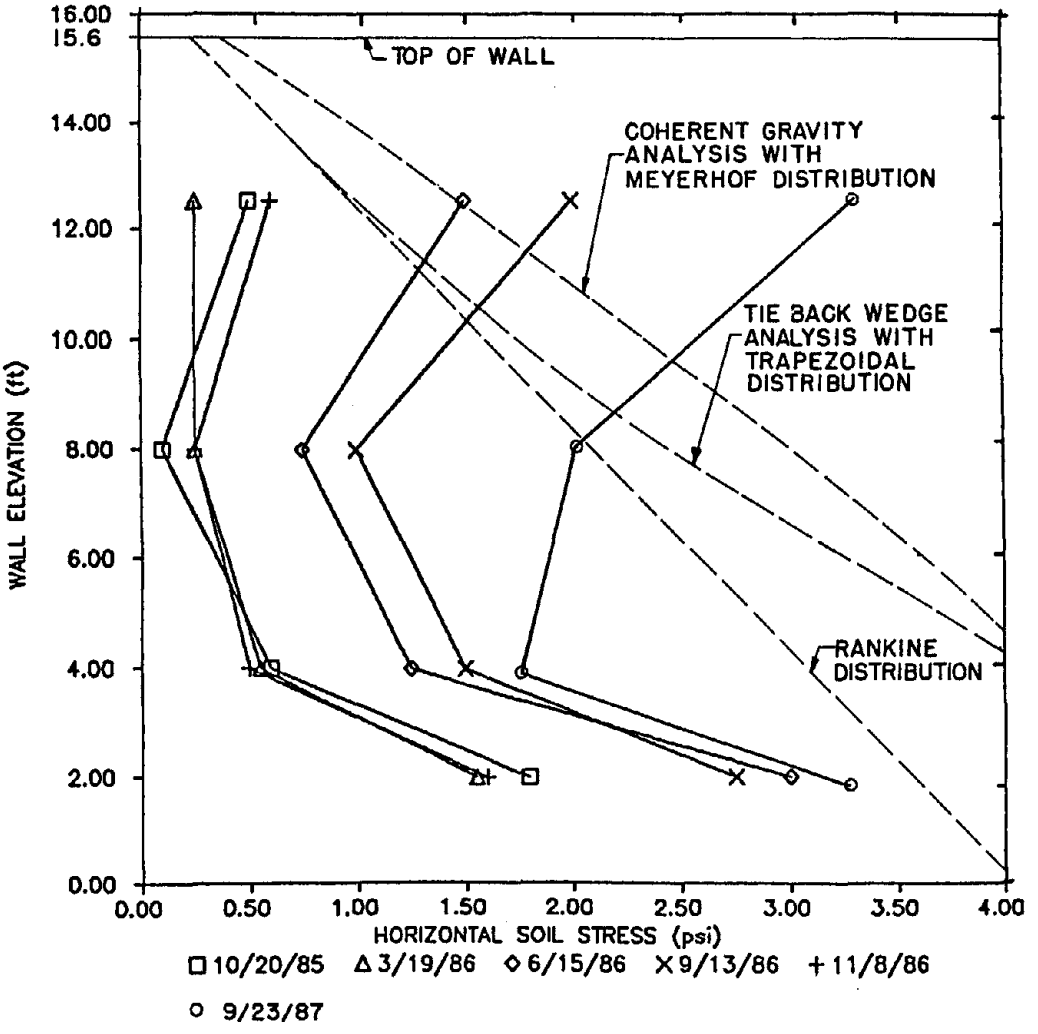


Figure 6-2. Comparison of measured horizontal soil stresses near the face of wall 26-30 with the stresses obtained from design calculations [from Desert Earth, 1987].

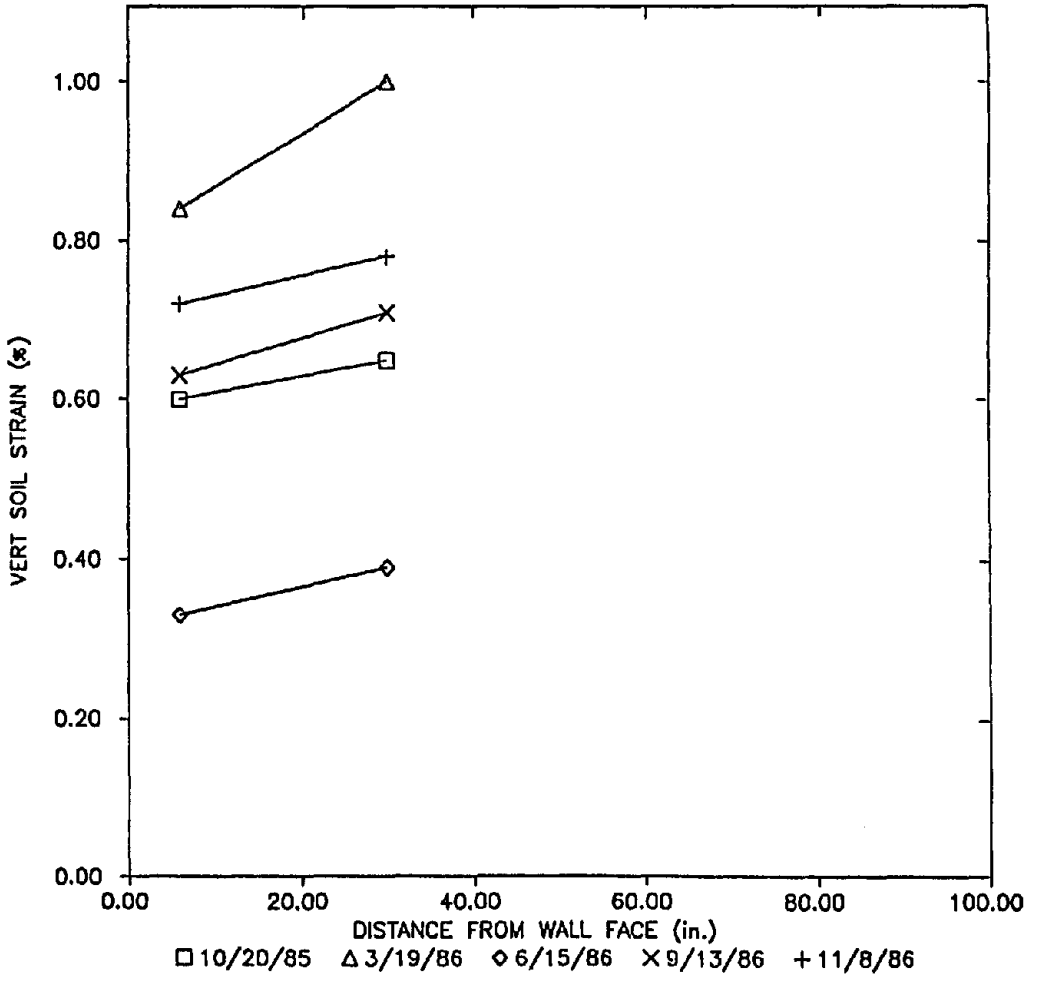


Figure 6-3. Measured vertical soil strains obtained from magnetic inductance coils in wall 26-30 at a distance of 3.5 ft from the leveling pad [from Desert Earth, 1987].

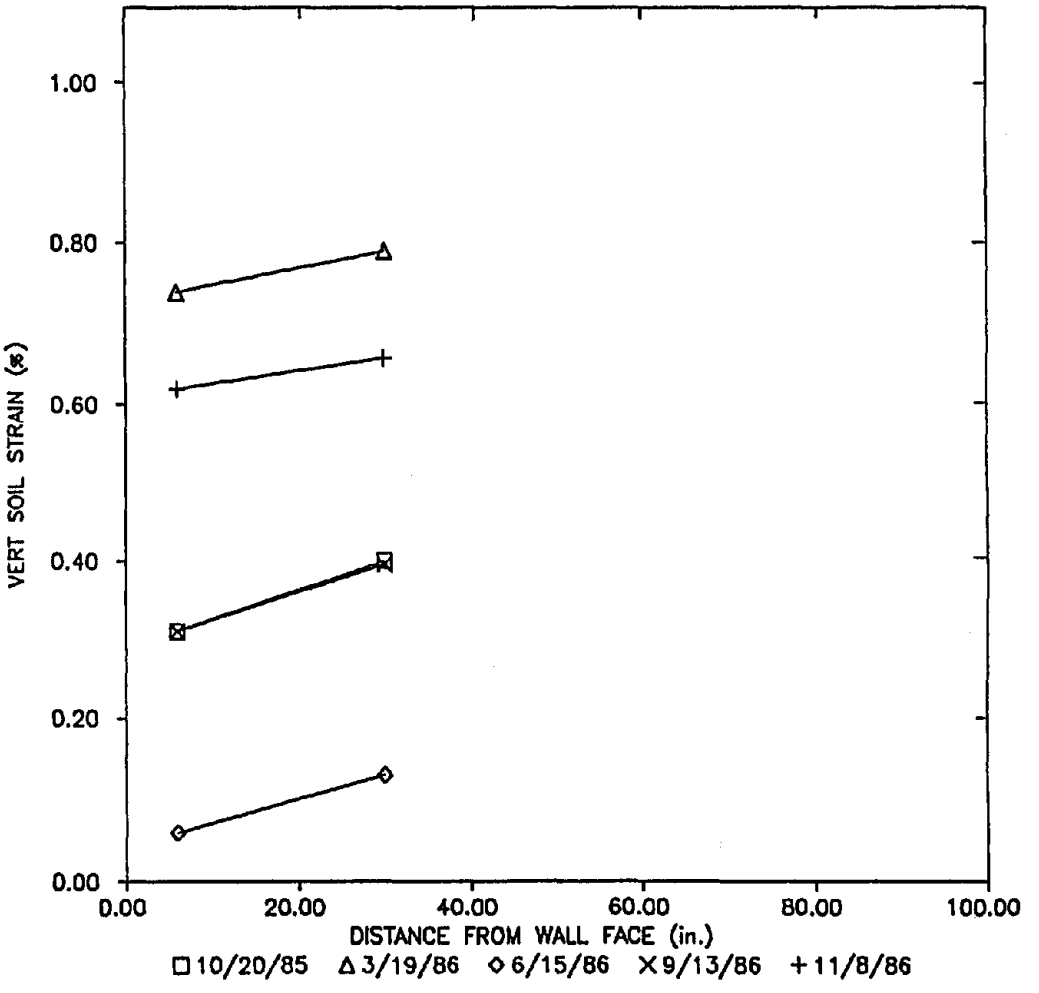


Figure 6-4. Measured vertical soil strains obtained from magnetic inductance coils in wall 26-32 at a distance of 3.5 ft from the leveling pad [from Desert Earth, 1987].

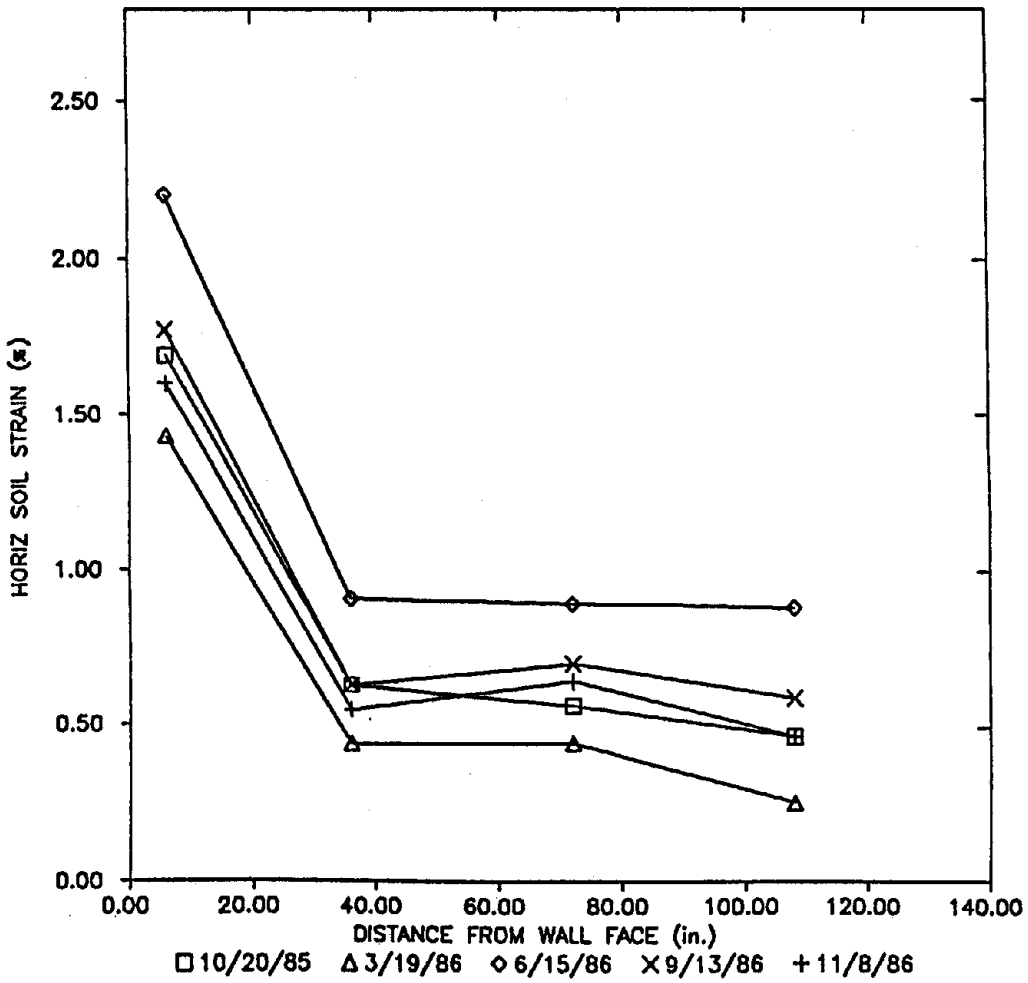


Figure 6-5. Measured horizontal soil strains obtained from magnetic inductance coils in wall 26-30 at a distance of 8 ft from the leveling pad [from Desert Earth, 1987].

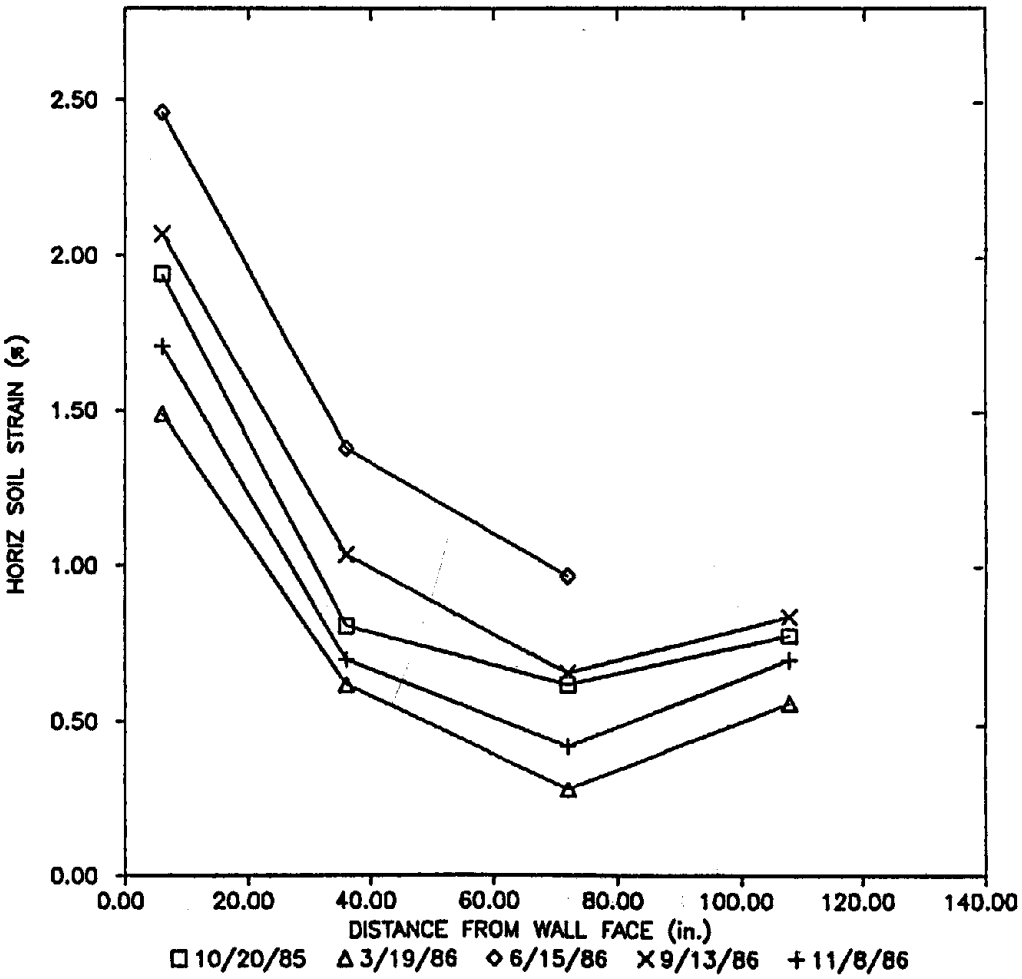


Figure 6-6. Measured horizontal soil strains obtained from magnetic inductance coils in wall 26-32 at a distance of 8 ft from the leveling pad [from Desert Earth, 1987].

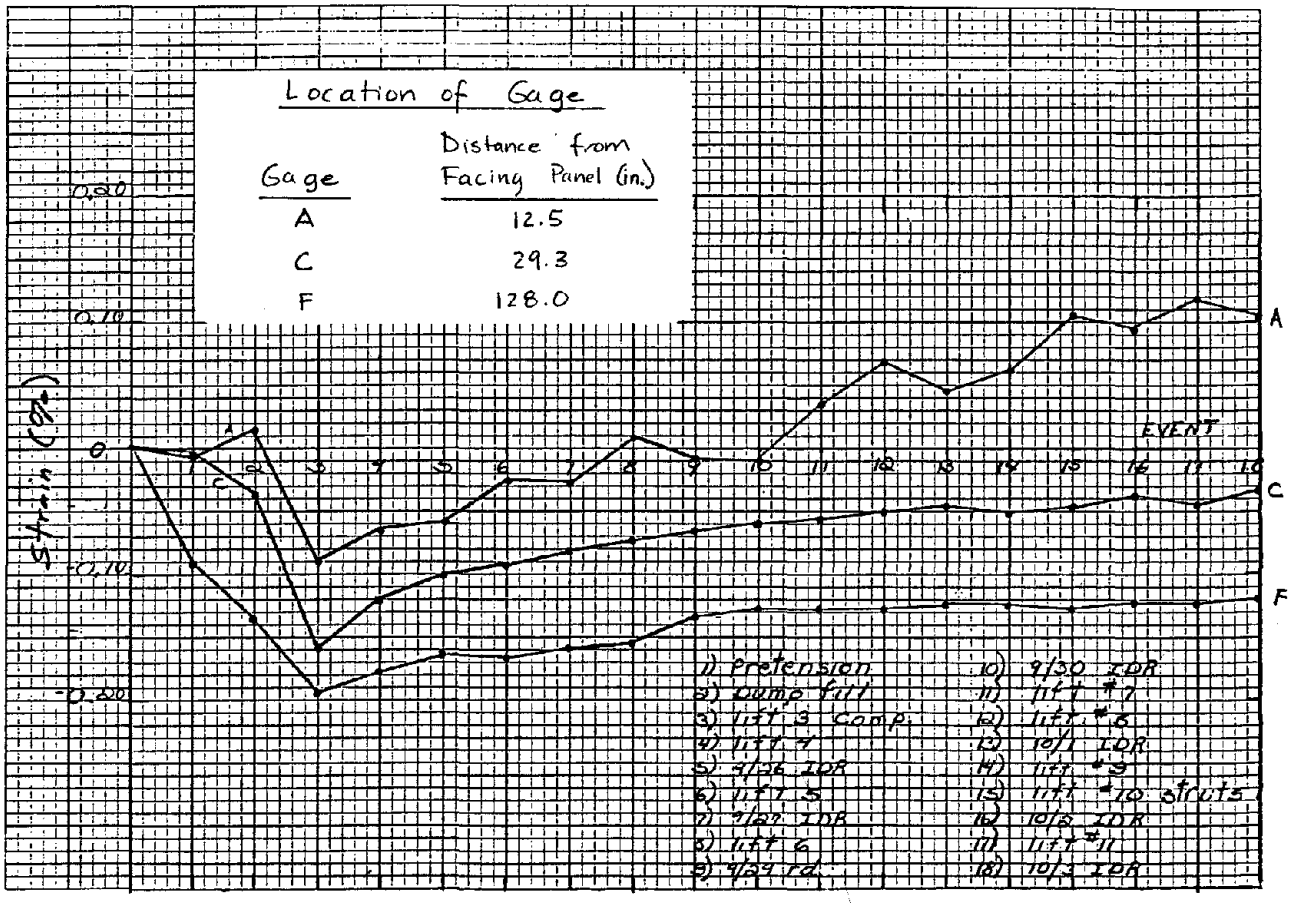


Figure 6-7. Measured geogrid strains in wall 26-30 at a distance of 1.5 ft from the leveling pad. The strains were measured during construction with strain gages [from Desert Earth, 1986].

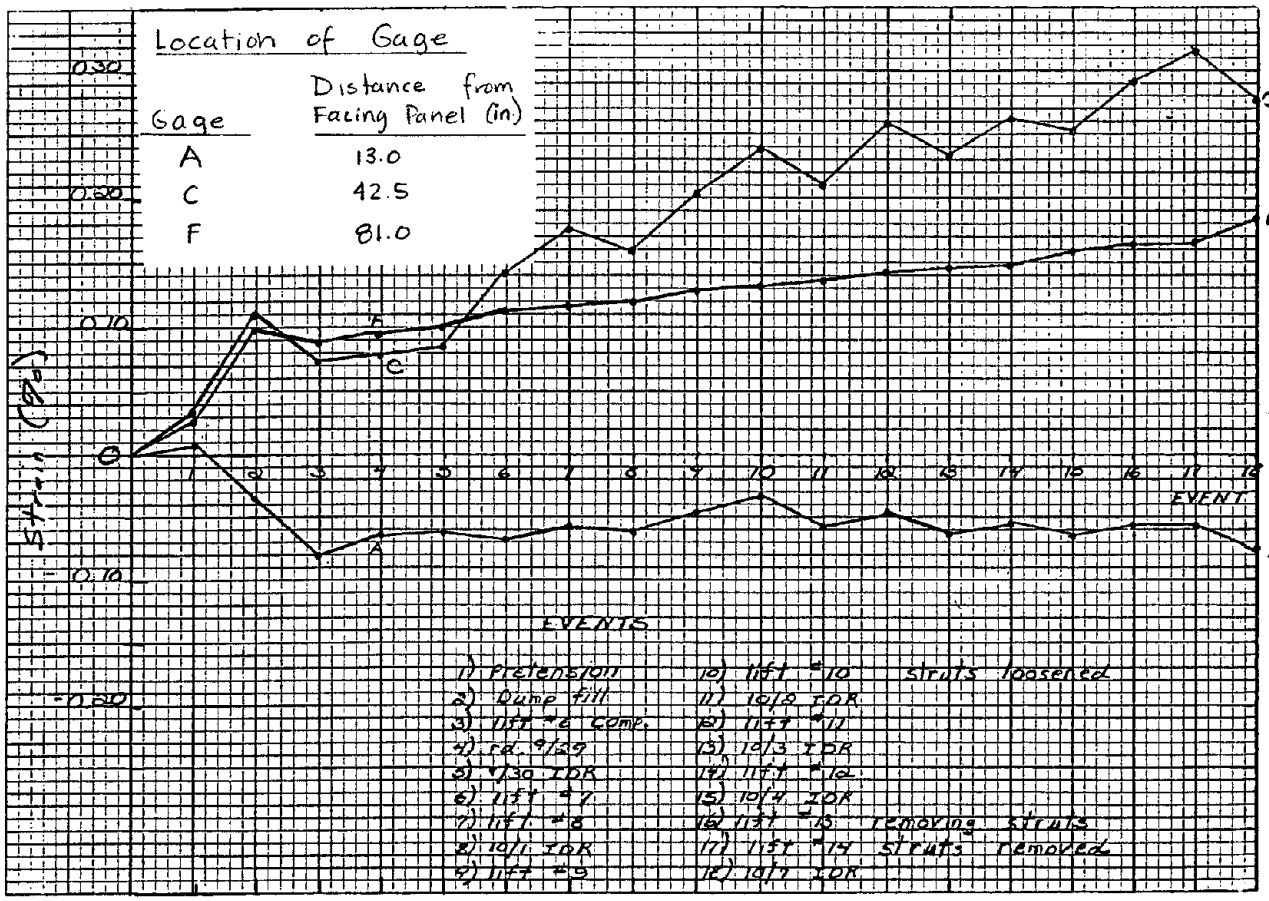


Figure 6-8. Measured geogrid strains in wall 26-30 at a distance of 4.5 ft from the leveling pad. The strains were measured during construction with strain gages [from Desert Earth, 1986].

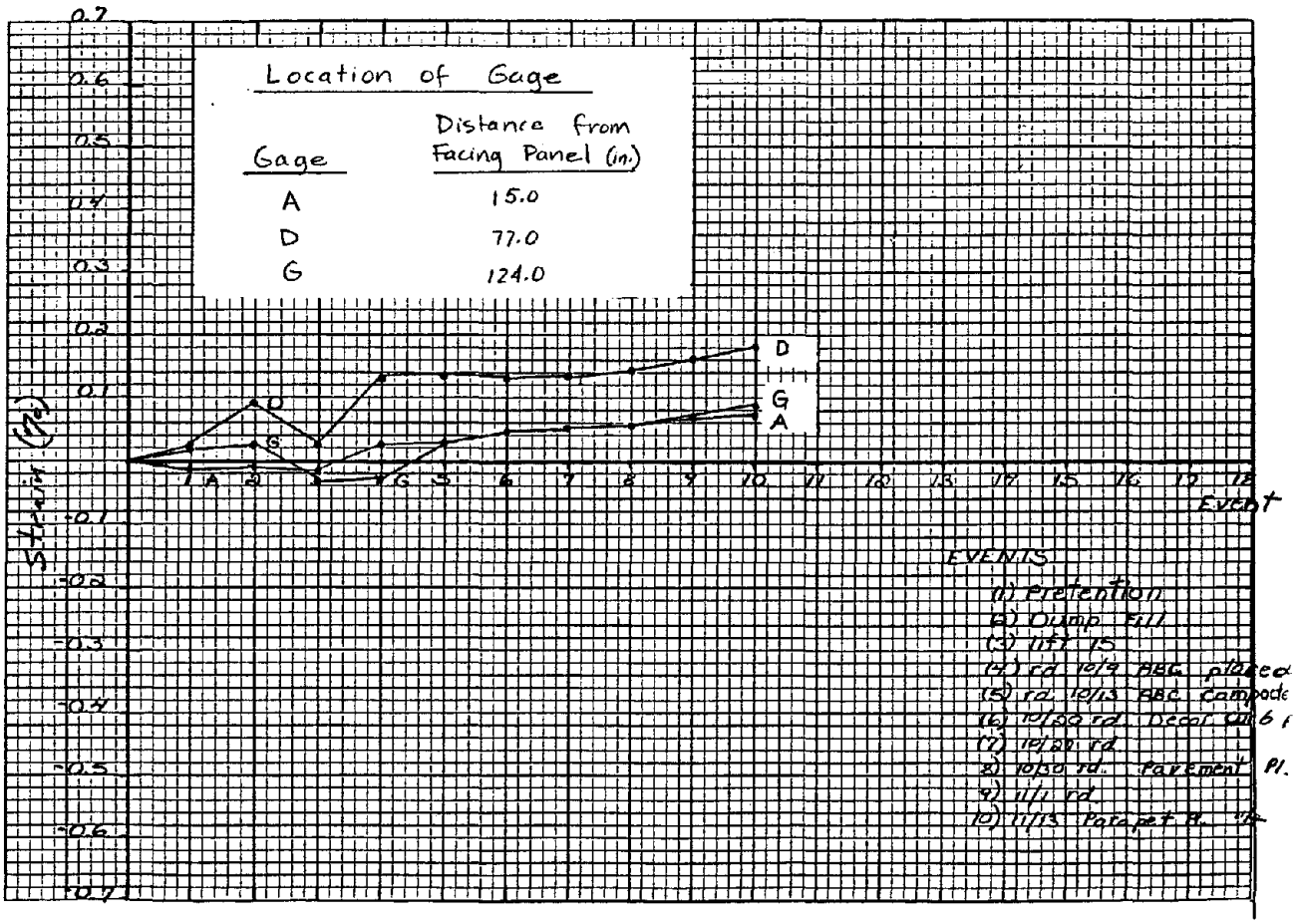


Figure 6-9. Measured geogrid strains in wall 26-30 at a distance of 14.5 ft from the leveling pad. The strains were measured during construction with strain gages [from Desert Earth, 1986].

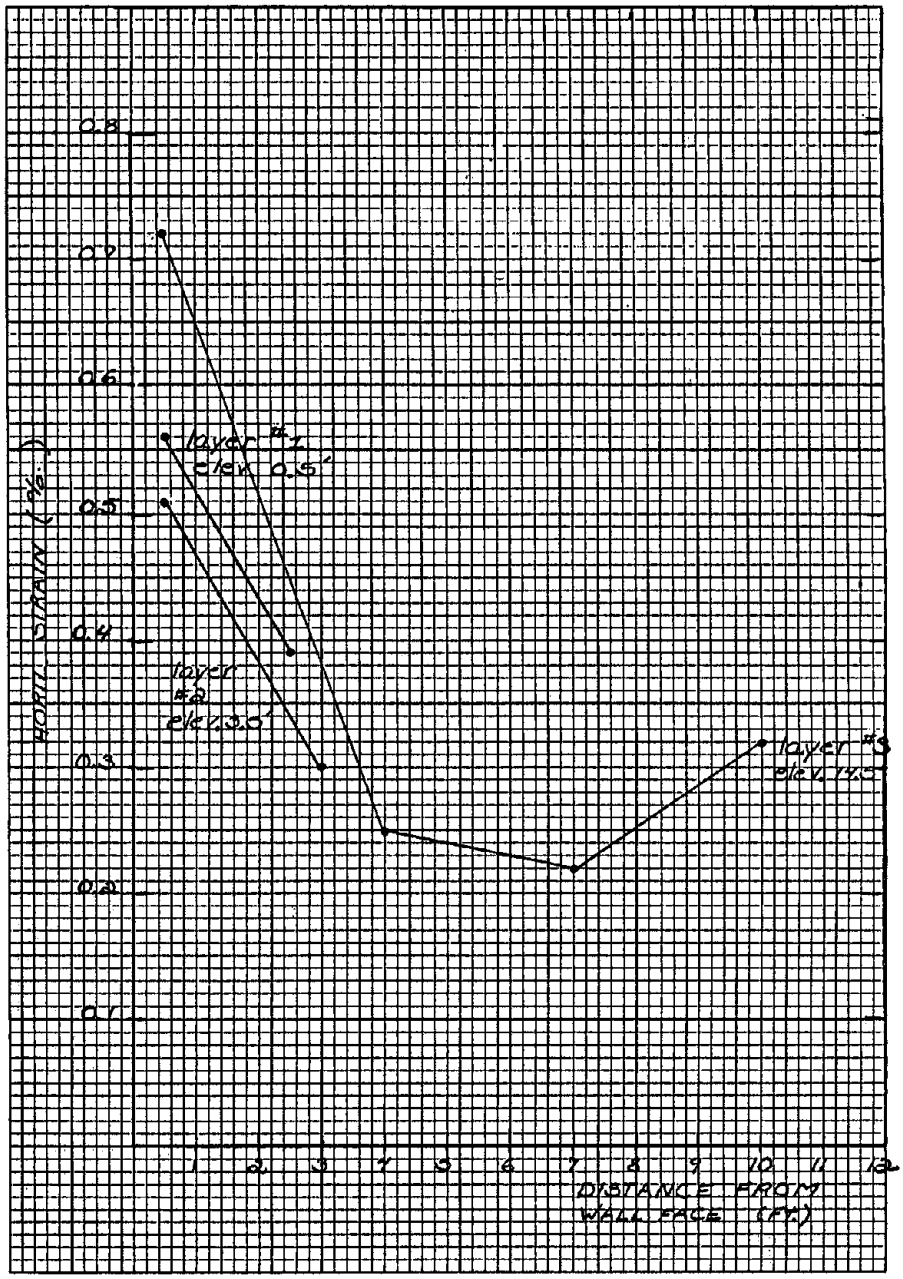


Figure 6-10. Adjusted measured geogrid strains in wall 26-30 on 20 October 1985. The strains were measured with magnetic inductance coils [from Desert Earth, 1986].

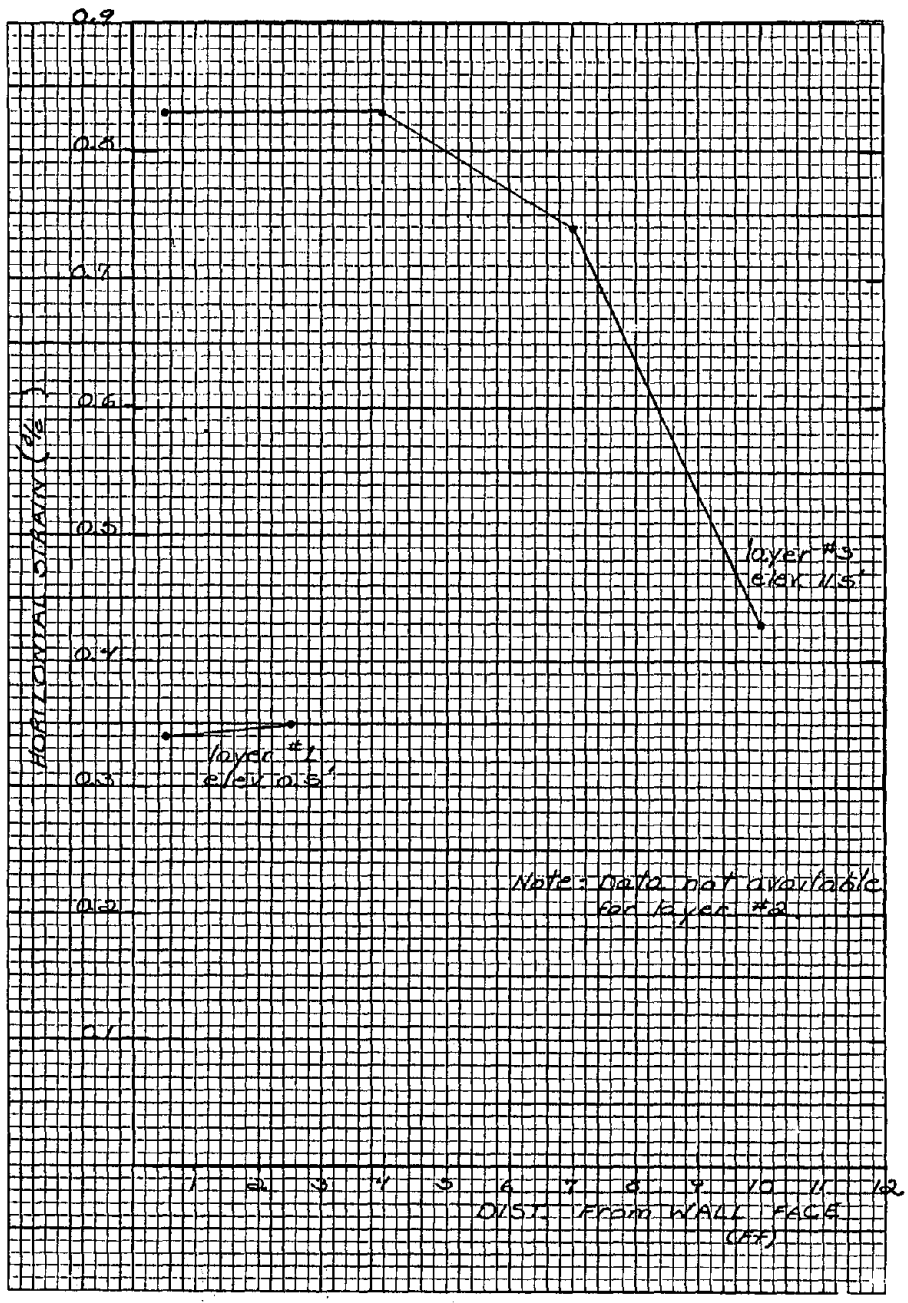


Figure 6-11. Adjusted measured geogrid strains in wall 26-32 on 20 October 1985. The strains were measured with magnetic inductance coils [from Desert Earth, 1986].

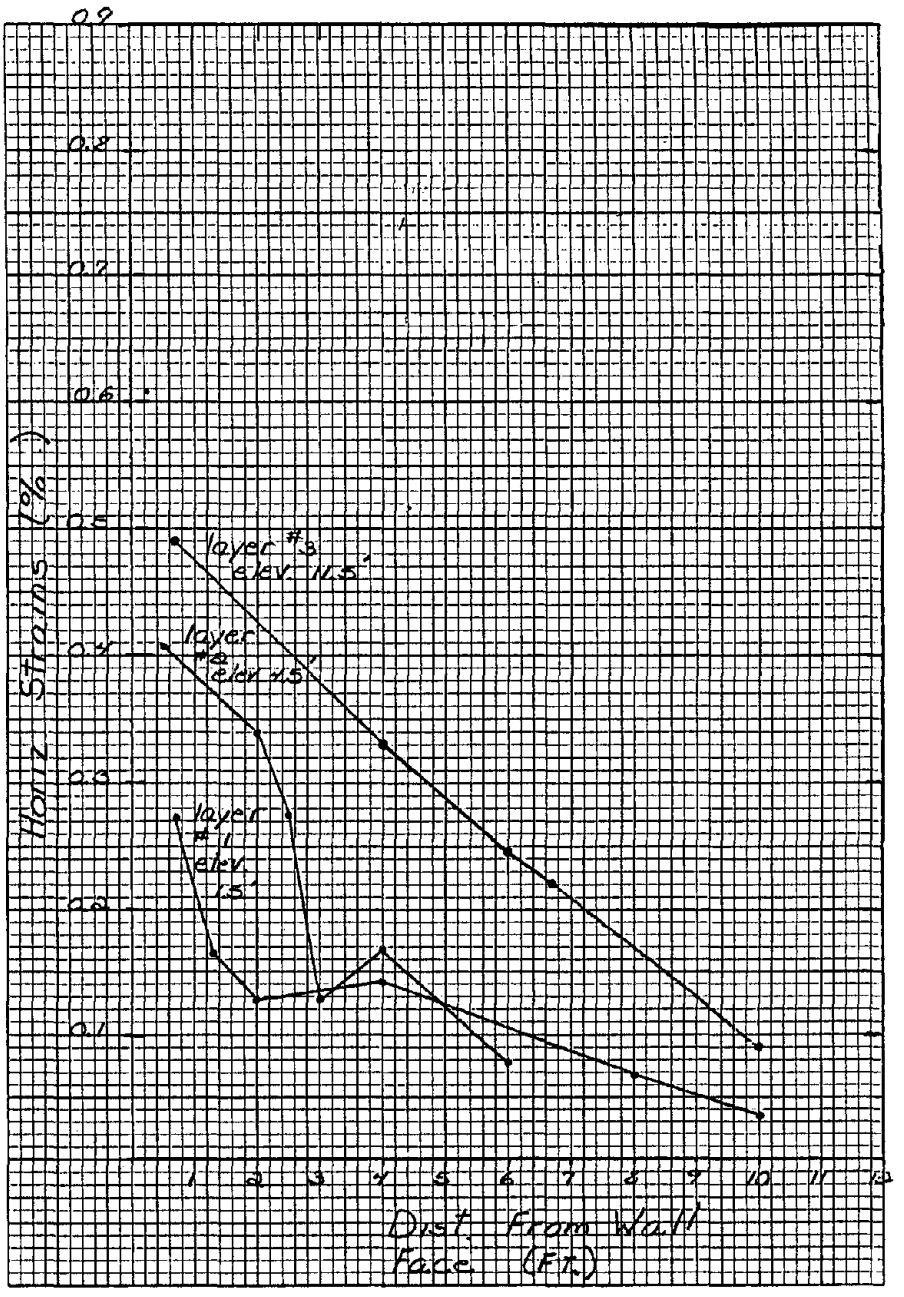


Figure 6-12. Adjusted measured geogrid strains in wall 26-30 on 20 October 1985. The strains were measured with foil-type resistance strain gages [from Desert Earth, 1986].

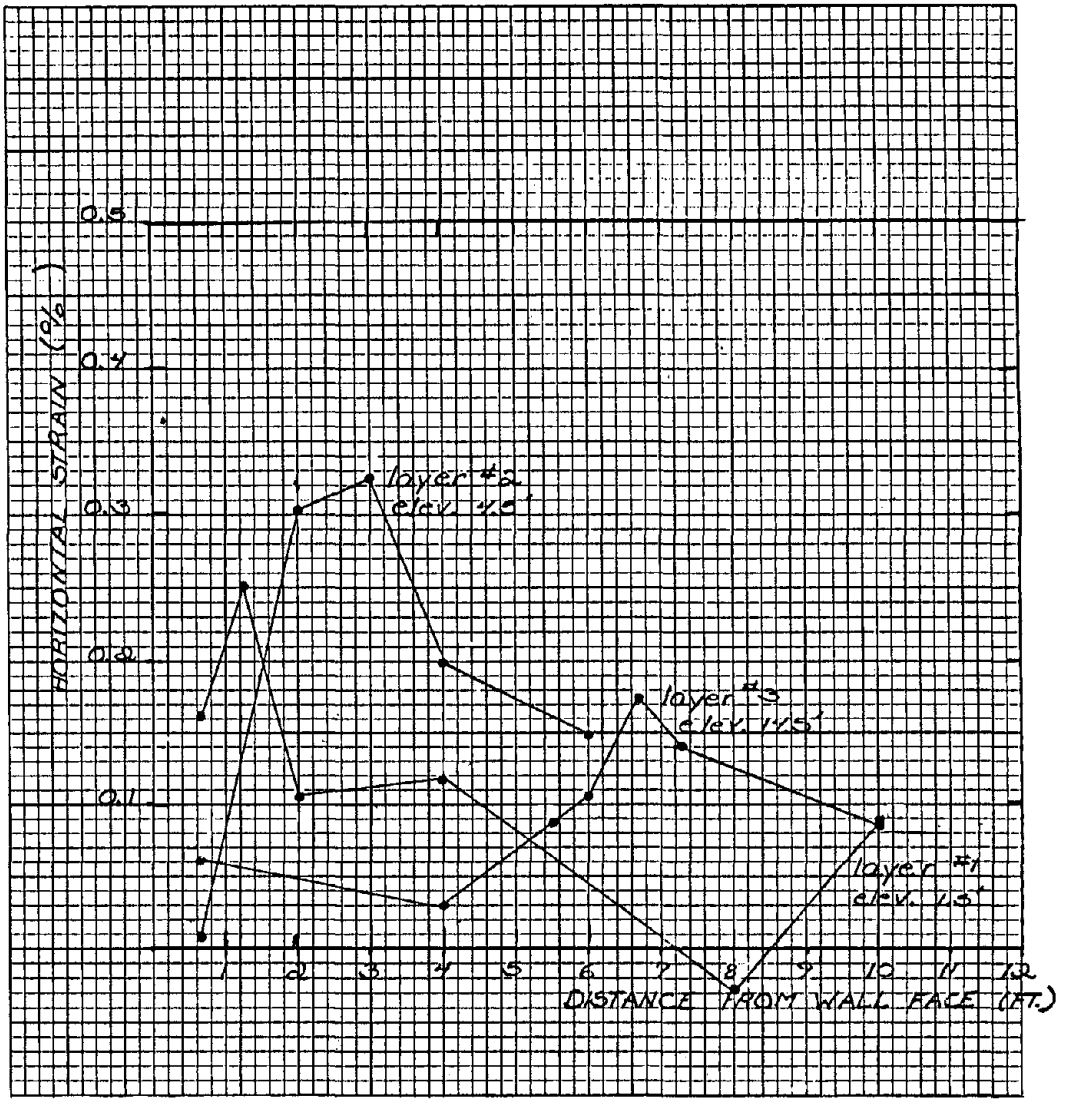


Figure 6-13. Adjusted measured geogrid strains in wall 26-32 on 20 October 1985. The strains were measured with foil-type resistance strain gages [from Desert Earth, 1986].

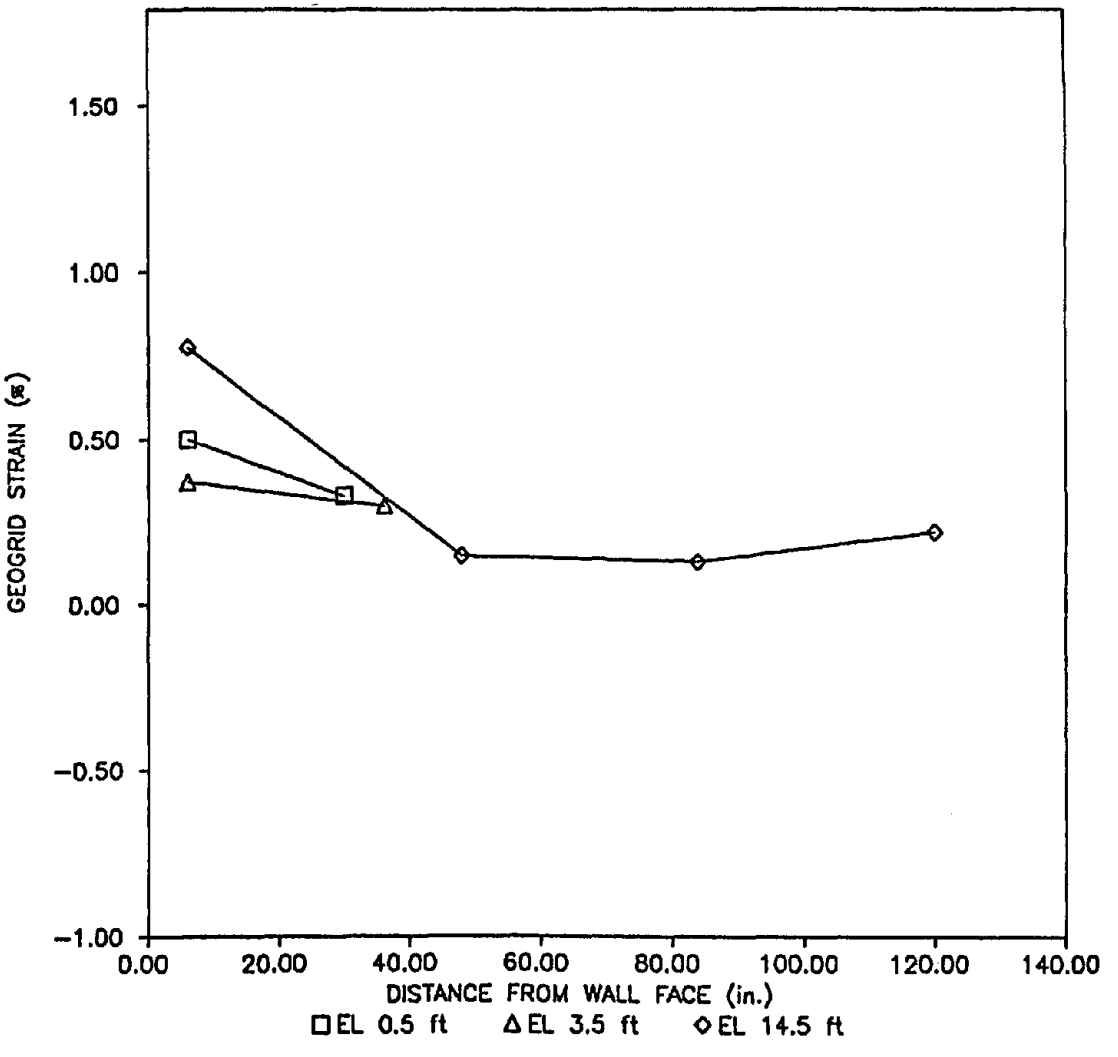


Figure 6-14. Adjusted measured geogrid strains in wall 26-30 on 8 November 1986. The strains were measured with magnetic inductance coils [from Desert Earth, 1987].

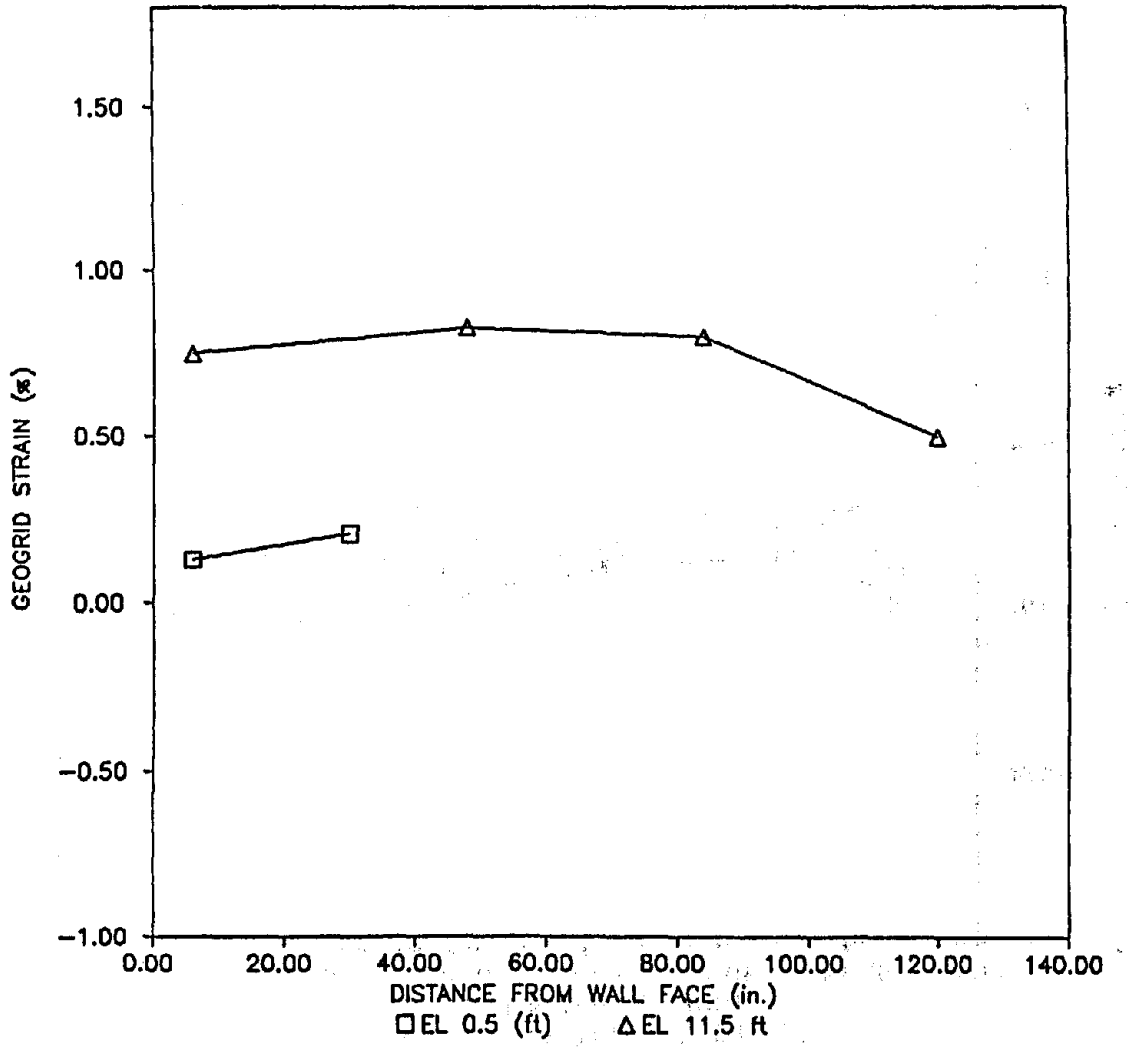


Figure 6-15. Adjusted measured geogrid strains in wall 26-32 on 8 November 1986. The strains were measured with magnetic inductance coils [from Desert Earth, 1987].

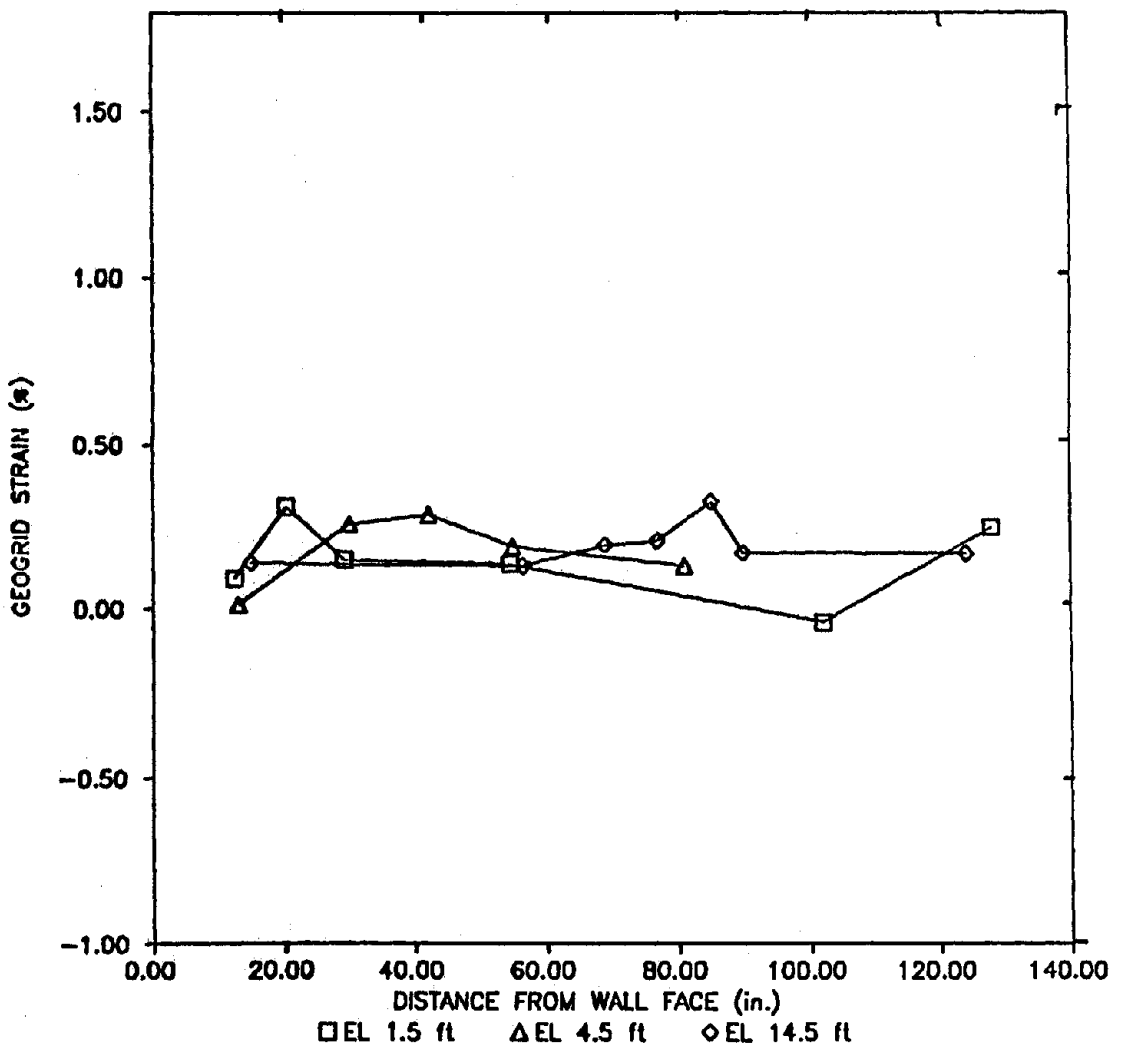


Figure 6-16. Adjusted measured geogrid strains in wall 26-30 on 8 November 1986. The strains were measured with foil-type resistance strain gages [from Desert Earth, 1987].

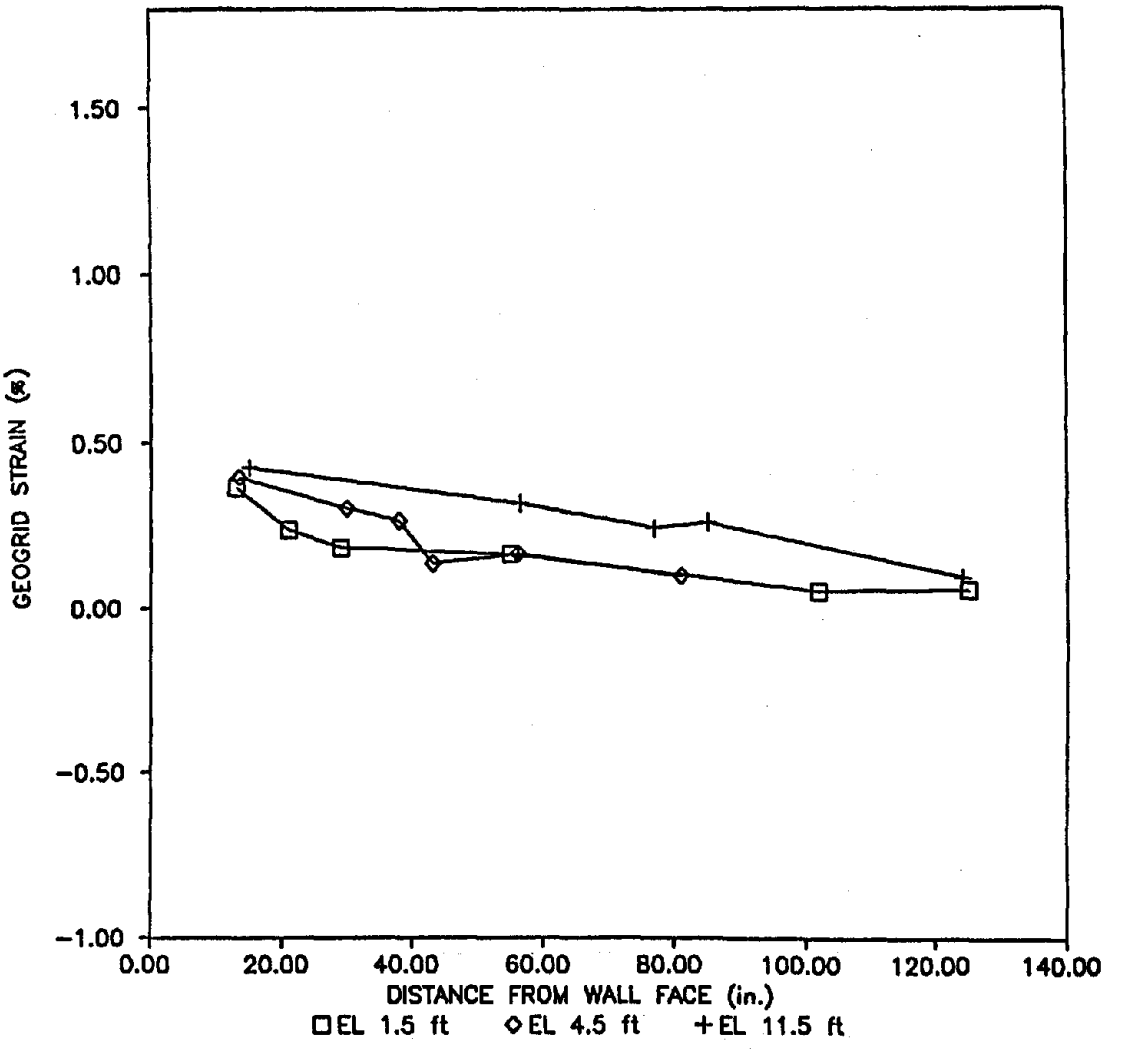


Figure 6-17. Adjusted measured geogrid strains in wall 26-32 on 8 November 1986. The strains were measured with foil-type resistance strain gages [from Desert Earth, 1987].

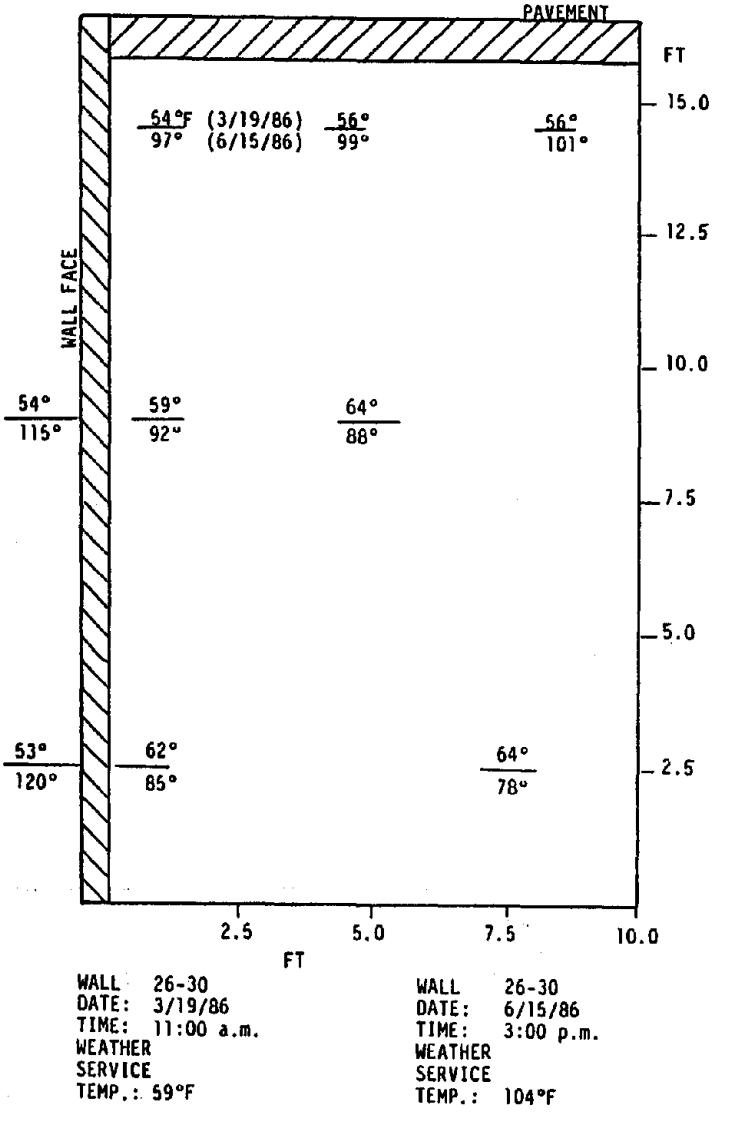


Figure 6-18. Measured temperatures in wall 26-30 [from Desert Earth, 1987].

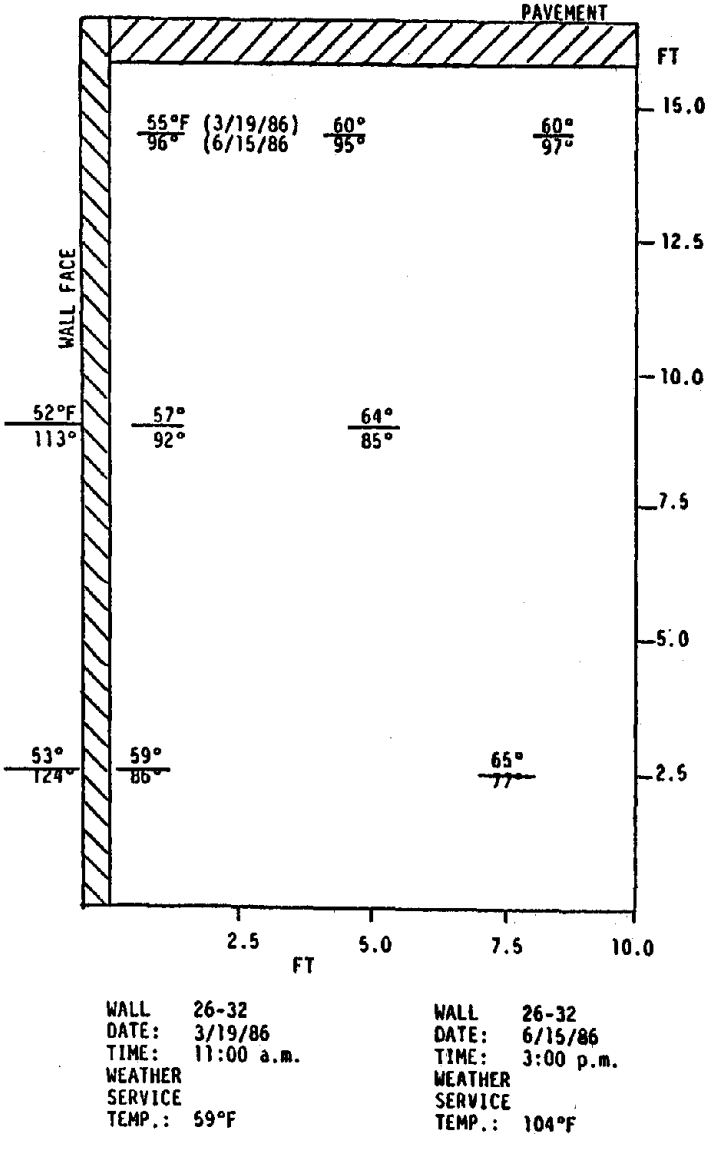


Figure 6-19. Measured temperatures in wall 26-32 [from Desert Earth, 1987].

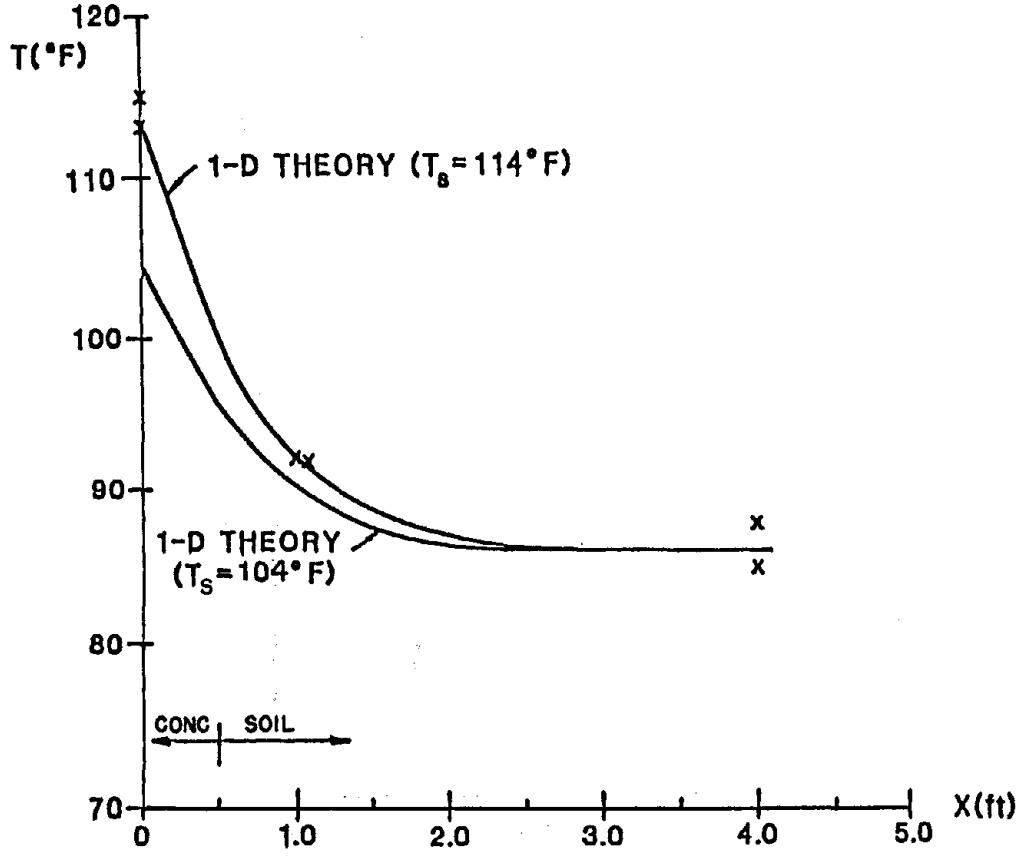


Figure 6-20. Comparison of temperatures measured in walls 26-30 and 26-32 on 15 June 1986 with those predicted using a one-dimensional transient heat flow analysis [from Bonaparte, 1987].

7. DISCUSSION OF WALL PERFORMANCE

7.1 Introduction

In this chapter the observed performance of the Tanque Verde Road project, geogrid-reinforced MSE walls is briefly reviewed. Section 7.2 presents the facing panel movements. Section 7.3 addresses the tension and strain in the geogrid reinforcement. Section 7.4 compares the results of the different strain and deformation measurements presented in Section 6. Sections 7.5 and 7.6 review the instrumentation program results with respect to the effects of time and temperature on wall performance. Section 7.7 draws relevant conclusions.

7.2 Facing Panel Movements

As discussed in Section 6.5, outward deflections of the tops of wall panels 26-30 and 26-32 at the end of construction are 3 in. The wall panels 26-30 and 26-32 are both approximately 16-ft high. The 3.0-in. outward deformation for "hinged" wall panels of this type corresponds to an angular rotation of the panels of about 1°. To compensate for this rotation an initial wall batter of about 1H:60V is necessary. This is larger than the actual wall batter of 1H:96V used on the project. For future projects, with full-height precast panels and geogrid reinforcement, a 1H:60V wall batter should be specified. This batter assumes that the external wall bracing is not removed until the wall is backfilled to at least two-thirds of its height.

7.3 Tension and Strain in the Geogrid

As indicated in Section 6.4, the maximum measured geogrid strain on 8 November 1986 was about 0.8% (excluding strain induced during placement and compaction of the first lift of fill over the geogrid) and the average strain was about 0.3%. These values are significantly less than the performance strain limit of 10% for polymer soil reinforcements that was established by McGown, et al. [1984].

The maximum tension in the geogrid reinforcement corresponds to the maximum geogrid strain. The maximum tension can be estimated from the tension-strain curve corresponding to long-term loading for TENSAR SR2 geogrid. This curve is approximated using isochronous curves derived from

long-term creep tests [McGown, et al., 1984]. The isochronous curve for TENSAR SR2 geogrid is shown in Figure 7-1 [Netlon, 1984]. Using this figure, and a load duration of 10,000 hours (the 8 November 1986 measurements were recorded about one year after construction of the walls), the maximum tension in the geogrid is estimated to be about 3.0 kN/m, which is equal to about 200 lbs/ft. This value of tension is only 10 percent of the long-term allowable tension for TENSAR SR2 geogrid, as provided in Section 2.2. Looked at another way, the factor of safety of the geogrid reinforcement against rupture during the design life of the structure is on the order of 10.

Maximum geogrid strains (based on the strain gage measurements) recorded on 8 November 1986 for walls 26-30 and 26-32 have been plotted in Figure 7-2 versus the locations of the readings. From Figure 7-2, an indication of the locations of the line of maximum geogrid tensions can be obtained. For wall 26-30, the line of maximum tensions follows a shape more or less consistent with an active Rankine plastic state in the soil. The line of maximum tensions for wall 26-32 is very close to the wall face. The reasons for the differences in the locations of the line of maximum tensions for walls 26-30 and 26-32 are unknown. However, because the magnitudes of the strains are so low, the locations of the line are not very significant.

7.4 Deformation Analysis

A review of the data shown in Section 6 reveals that the relationship between horizontal soil strain, geogrid strain, and panel movements can be very complex. To better understand this complex relationship, Figures 7-3 and 7-4 were developed. These figures were developed using the unadjusted strain gage, inductance coil, and survey results corresponding to 20 October 1985, which include the cumulative strains developed up to and including the compaction of the first lift of fill over the instruments. (The unadjusted values of geogrid strain and horizontal soil strain measured on 20 October 1985 are not included in this report and can be found in Desert Earth [1986]. Adjusted strain measurements are discussed in Sections 6.3.2, 6.4.1, and 6.4.2.) To start with, both figures show the relationship between the surveyed panel movements and height above the leveling pad. The two figures also show estimates of the facing panel movements based on numerical integration of the measured geogrid strains and horizontal soil strains. If all of the panel movement was due to elongation of the geogrids under the applied tensions, calculated movements based on numerical integration of the geogrid strains should be in reasonable agreement with the measured panel

movements obtained from the survey results. From these figures, it can be seen that the calculated panel movements based on integration of the geogrid strains are much less than the measured panel movements. This means that the measured panel movements are due largely to factors other than the straining of the geogrids under the applied tensions. The factors other than geogrid strain that may have contributed to the panel movements include:

- Elongation of the connection system immediately behind the wall face. Initial slack in the geogrid-to-panel connection probably accounted for some portion of the wall movement.

- Rigid body movement of the soil in the reinforced zone may also cause facing panel displacements to be larger than horizontal soil displacements.

- The geogrid reinforcement could have moved during the fill placement and compaction process, after the steel pretensioning forks were removed. This possibility is supported by the fact that for both wall panels, the horizontal soil strain near the wall face is more than twice the geogrid strain. During the period of time between when the pretensioning forks were removed and a significant amount of fill was in place on top of the geogrids, some movement ("slippage") of the geogrids could have occurred due to the vibratory action of overlying compaction equipment.

7.5 Time Effects

The main effect of time on the Tanque Verde Road project is the influence of prolonged loading on strains in the geogrid. From the isochronous curve in Figure 7-1 several features of the time-dependent deformation of TENSAR SR2 geogrid can be observed. First, below the 10% strain limit for this product, most time-dependent deformation occurs in the first 100 hours after load application. As a practical consequence, most time-dependent movement would occur during the construction period. Second, at the low strain levels measured on the Tanque Verde Road project (i.e., 0.8% maximum), time-dependent strains are negligible. This observation from laboratory tests is confirmed by the results shown in Table 7-1. From the table, it can be seen that within a zone close to the wall face, geogrid strains did not increase significantly in the time period from October 1985 until November 1986. The maximum measured increase in strain over an elapsed time of 13 months was

0.1%. The average increase in strain for the six sets of results reported in Table 7-1 was 0.006%. These magnitudes of increases are judged to be insignificant. It should also be remembered that the accuracy of the strain gage readings is believed to be ± 0.05%, as indicated in Table 5-5.

7.6 Temperature Effects

Table 7-1 can also be used to judge the effect of the Tucson summertime temperatures on the geogrid reinforcement. Comparison of the readings from 19 March 1985 (maximum temperature of 59°F) and 15 June 1986 (maximum temperature of 104°F) shows that there was no measurable increase in soil strain near the wall face at the time of the 15 June reading.

As discussed in Section 6.2, it appears that temperature affected the load cell readings. The reason for this is unknown. As pointed out in Section 6, the high readings may be due to a real physical effect, or simply to an unaccounted effect of temperature on the instrumentation. The latter possibility is supported by soil strain measurements, which do not appear to be affected by temperature. When the soil stress increases, we would expect a corresponding increase in soil strain. However, this was not observed. Unfortunately, the load cell was not calibrated at elevated temperatures. Therefore, we cannot determine the effect of the elevated temperatures on the load cell.

7.7 Conclusions

The following conclusions were drawn from field observations and from the results of the instrumentation and monitoring program.

- Overall, performance of the Tanque Verde Road project retaining walls has been satisfactory.

- The maximum measured (corrected) strain in the geogrid reinforcement is very low (about 0.8%). This strain corresponds to a geogrid tension which is approximately 10% of the allowable design tension (i.e., the factor of safety against geogrid reinforcement rupture is 10). This indicates that the material property assumptions (i.e., ϕ'_r = 34° for the reinforced fill) and the design methods (i.e., the tie-back wedge method and the coherent gravity method) applied to the Tanque Verde Road MSE walls were conservative.

- Measured time-dependent geogrid strains (i.e., geogrid creep) are negligible to date. Predicted time-dependent strains for the design life are negligible.

- An initial wall batter of 1H:60V appears to be appropriate for full-height, concrete-faced, geogrid-reinforced soil walls similar to those constructed on the Tanque Verde Road project. This batter assumes that the external wall bracing is not removed until the wall is backfilled to at least two-thirds of its height.

- Panel movements during construction (i.e., 1H:60V) appear to be due more to initial slack in the geogrid reinforcement-to-panel connection behind the wall facing panel than to geogrid reinforcement strain. Thus, consistent tensioning (to remove initial slack) of the connection is an important construction step.

- The evidence to date indicates that the elevated summertime temperatures of the Sonoran Desert environment have no significant effect on the performance of the geogrid reinforcement behind the wall facing.

Table 7-1. Maximum geogrid strain in walls 26-30 and 26-32 (measured with strain gages) within 2.5 ft of the wall face.

	Date	10/20/85	3/19/86	6/15/86	9/13/86	11/8/86
	Estimated T_{max}	80°F	59°F	104°F	93°F	69°F
Facing Panel	Distance from Leveling Pad (ft)	Geogrid Strain* (%)				
26-30	1.5	0.25	0.28	0.28	0.28	0.32
	4.5	0.31	0.29	0.16	0.23	0.26
	11.5	0.06	0.08	0.09	0.10	0.14
26-32	1.5	0.27	0.33	0.33	0.33	0.37
	4.5	0.41	0.42	0.39	0.36	0.40
	14.5	0.48	0.45	0.42	0.38	0.42

*Geogrid strains do not include compaction-induced strains (measurements are strains after the first lift of fill was placed on the considered geogrid layer and compacted).

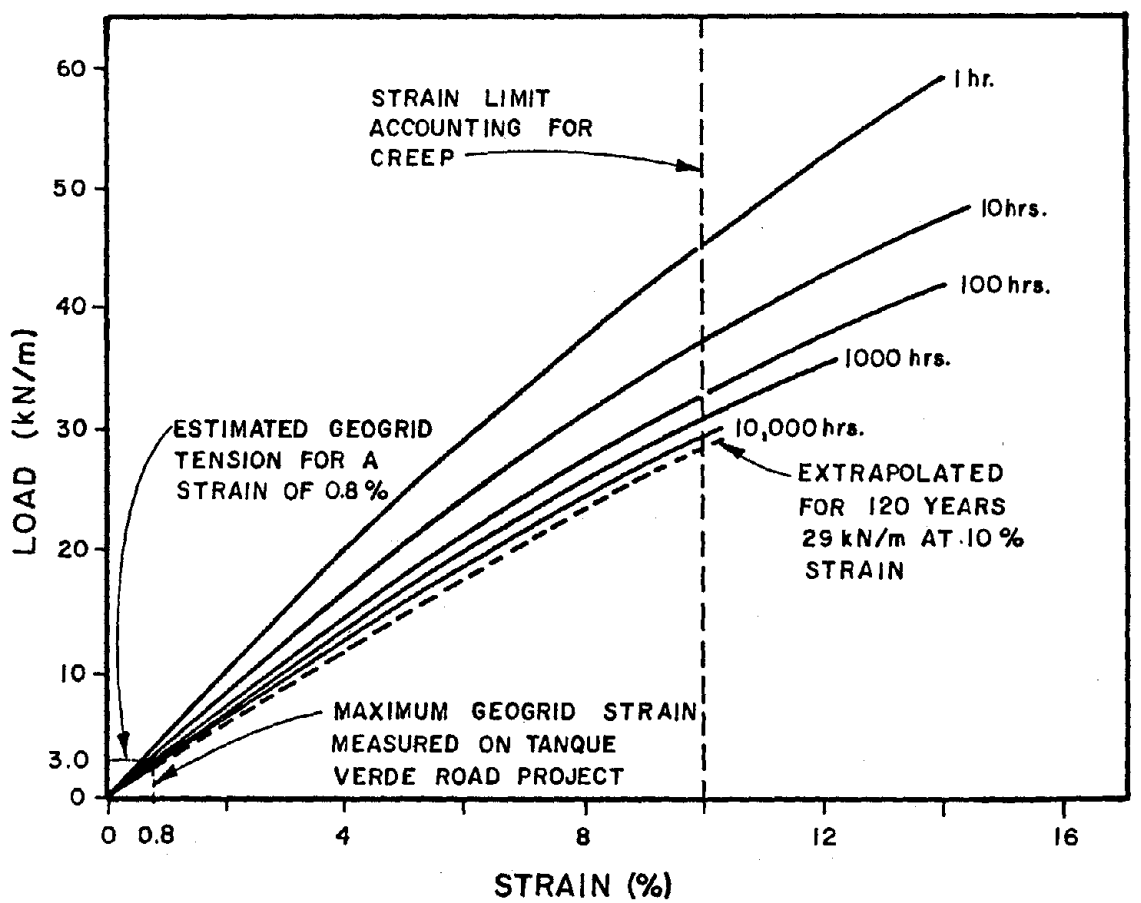


Figure 7-1. Isochronous tension-strain curve for TENSAR SR2 geogrid [Netlon, 1984]. This figure is used to estimate the geogrid tension for a strain of 0.8%.

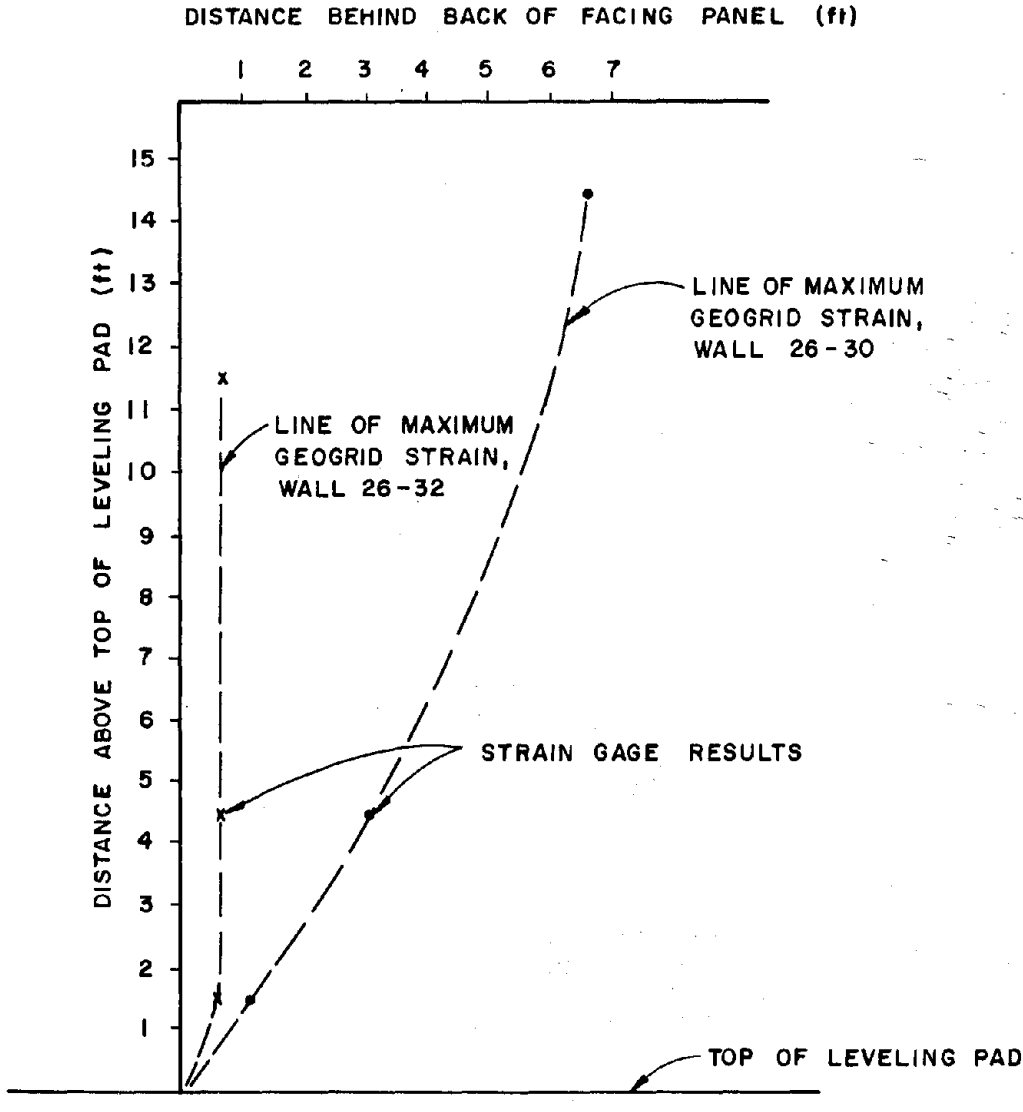


Figure 7-2. Plot of maximum geogrid strain (obtained from strain gage results) versus wall location for walls 26-30 and 26-32 on 8 November 1986.

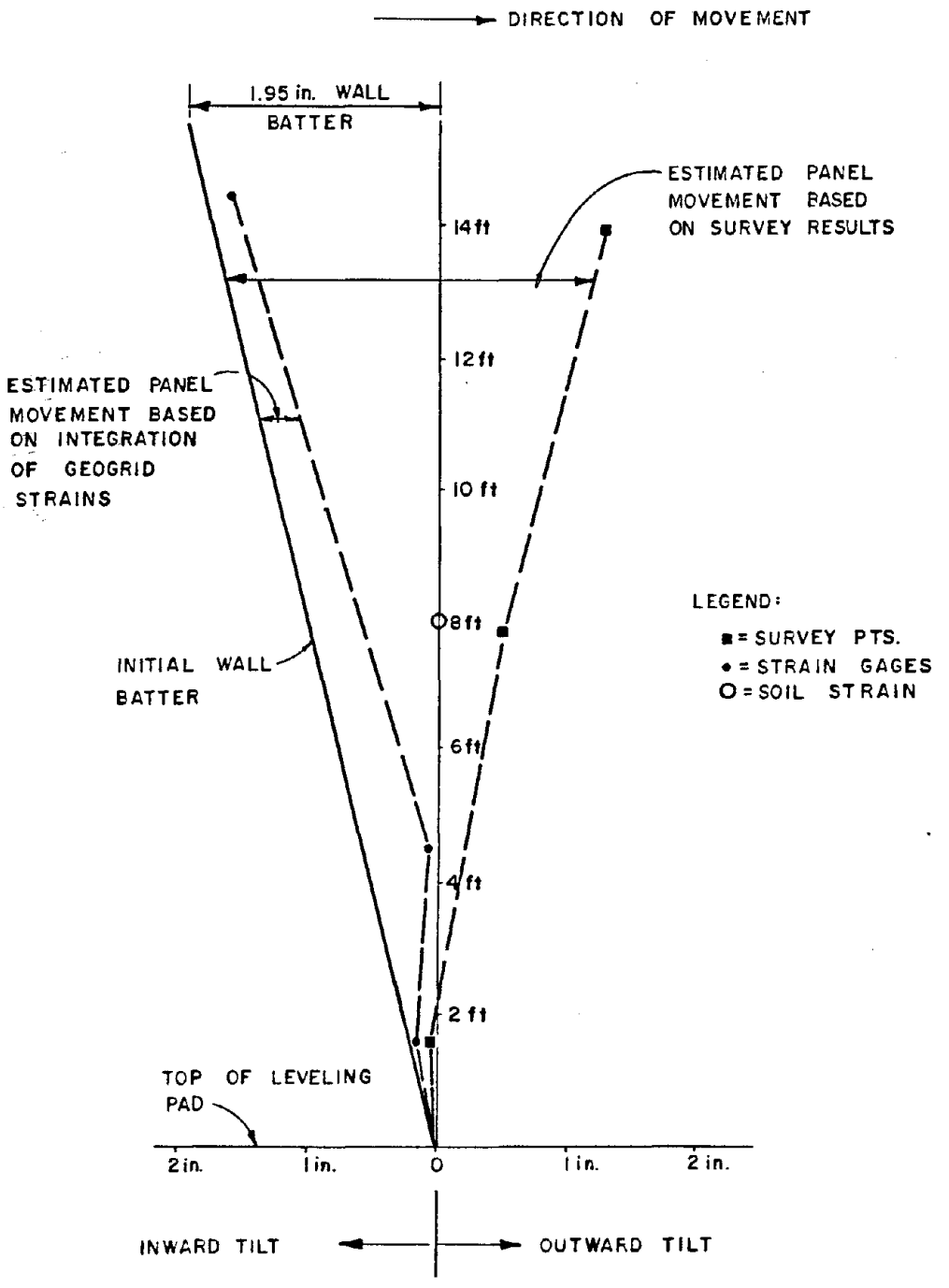


Figure 7-3. Estimated deformations of wall facing panel 26-30 on 20 October 1985 based on movements of survey markers and numerical integration of geogrid strains and horizontal soil strains.

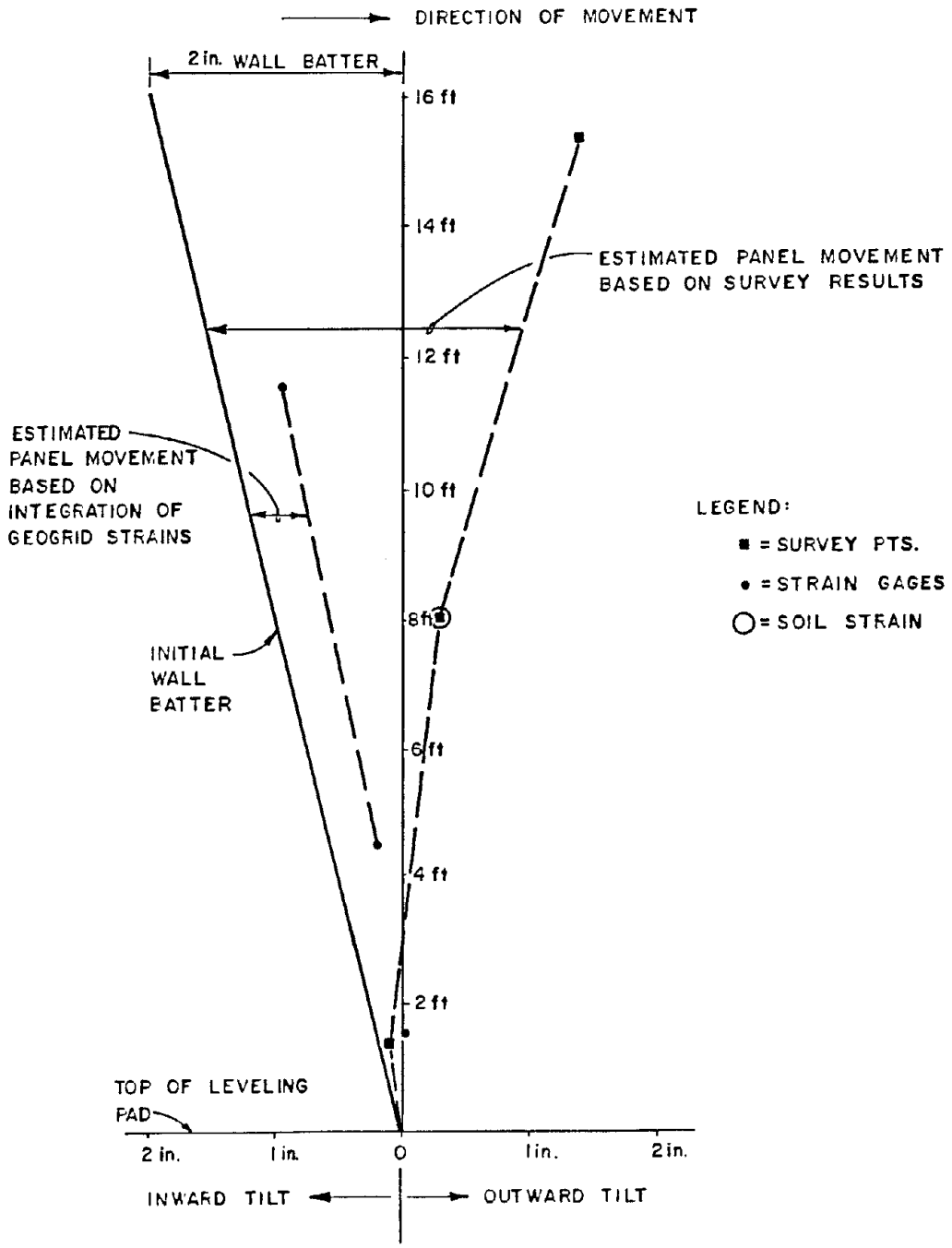


Figure 7-4. Estimated deformations of wall facing panel 26-32 on 20 October 1985 based on movements of survey markers and numerical integration of geogrid strains and horizontal soil strains.

8. RECENT DEVELOPMENTS IN DESIGN AND CONSTRUCTION

8.1 Introduction

Since the design and construction of the Tanque Verde Road project and through April 1987, approximately twenty-nine additional concrete panel-faced TENSAR geogrid-reinforced soil walls have been constructed in North America [Yako and Christopher, 1987]. The maximum height of these walls is about 40 ft, with most being much lower, typically in the range of 20 ft. From field observations of these walls and refinements in wall design procedures, geogrid-reinforced soil wall technology has advanced considerably. This section of the report briefly addresses some of the developments in design and construction of geogrid-reinforced soil walls since the time of the Tanque Verde Road project. Section 8.2 presents a method of selecting the allowable design tension of geosynthetic (i.e., geogrid and geotextile) reinforcement. Section 8.3 discusses developments in design procedures. Section 8.4 describes refinements in construction procedures and details.

8.2 Selection of Allowable Design Tension

A method of determining the allowable design strength of geosynthetic reinforcement that considers long-term geosynthetic creep, long-term polymer degradation, as well as construction-induced reinforcement damage was proposed by Bonaparte and Berg [1987]. This method is described below.

The allowable design tension in the reinforcement, T_a, is the minimum value of the allowable limit-state tension, T_{al}, and the allowable serviceability-state tension, T_{as}.

The allowable limit-state tension is given by:

$$T_{al} = \frac{T_l}{FS_t \; FS_d \; FS_c} \qquad \text{(Equation 8-1)}$$

where:

T_l = long-term tension based on a creep limit for the reinforcement;

FS_t = overall factor of safety to account for uncertainties in structure geometry, fill properties, reinforcement properties, and externally applied loads;

FS_d = durability factor of safety, which is dependent on the susceptibility of the geosynthetic to attack by micro-organisms and chemicals, thermal oxidation, and environmental stress cracking; and

FS_c = construction damage factor of safety.

The allowable serviceability-state tension is given by:

$$T_{as} = \frac{T_s}{FS_d\ FS_c} \qquad \text{(Equation 8-2)}$$

where:

T_s = long-term tension based on a strain limit for the reinforced soil structure;

FS_d = durability factor of safety; and

FS_c = construction damage factor of safety.

The allowable design tension in the reinforcement, T_a, is the minimum value of T_{al} and T_{as}.

Selection of appropriate tensions and factors of safety to be used with the above equations has been discussed by Bonaparte and Berg [1987].

8.3 Wall Design

This section summarizes advances in wall design related to factors of safety and design procedures.

Factors of Safety. At present, the most commonly used factors of safety for the internal and external stability design of reinforced soil walls are as follows:

- sliding, $FS_s = 1.5$;
- overturning, $FS_o = 2.0$;
- bearing capacity failure, $FS_{bc} = 2.0$;
- rotational slip-surface stability, $FS_r = 1.5$;
- reinforcement pullout, $FS_p = 1.5$; and
- reinforcement tensile rupture, $FS_t = 1.25$ (which is applied to the design tensile of the geogrid reinforcement as discussed in Section 8.2).

The Tanque Verde Road project was designed conservatively with factors of safety that are slightly greater than or equal to the above factors of safety.

Sequence of Design Steps. The sequence of steps that is commonly followed in the design of reinforced soil walls is to analyze the external wall stability before the internal wall stability. This is advantageous because external stability frequently controls the required reinforcement length. Thus, the reinforcement length calculated during the external stability analysis usually satisfies the internal stability design requirements. Knowing the reinforcement length at the start simplifies the internal stability design.

Analysis of Internal Stability Design. The tie-back wedge method is the most commonly used design method for the internal stability of geogrid-reinforced soil walls [Bonaparte, et al., 1987]. Today, the assumption regarding vertical stress most often used with the tie-back wedge method is that the vertical stress is equal to the weight of the overlying materials plus the stresses caused by any surcharges. The trapezoidal and Meyerhof type vertical stress distributions are no longer recommended. As discussed in Sections 3.4.2 and 3.4.3, Dames and Moore [1984] analyzed internal wall

stability using both the tie- back wedge method with the trapezoidal vertical stress distribution and the coherent gravity method with the Meyerhof type vertical stress distribution. The effect of using both design methods and then taking the more conservative of the two results was to provide a conservative wall design compared to the design obtained using the approach recommended today.

8.4 Wall Construction

This section presents current developments in wall construction components and construction procedures.

Geogrid Reinforcement. At the time of construction, the strongest geogrid product available from Tensar was SR2 which had a long-term design tension of 1986 lb/ft. Today, stronger products are available that allow larger vertical geogrid spacings and construction of reinforced soil walls to greater heights. Tensar reports that these new products have long-term design strengths up to 3000 lb/ft (compared to about 2000 lb/ft for the product used on the Tanque Verde Road project). At present, several geogrid products from other manufacturers are either commercially available or under development.

Connection System. The looped sliding connection between the facing panel and the geogrid reinforcement used in the Tanque Verde Road project was developed for the reasons discussed in Section 3.6 and is not commonly used today. The most common connection system for TENSAR geogrid-reinforced soil walls is a modified version of the straight bodkin (pinned) connection system shown in Figure 3-9a.

Connection tests were performed by Tensar to measure the efficiency of straight bodkin joints with different pin connectors. The efficiency of a joint is defined at the peak tension sustained by the jointed sample divided by the peak tension sustained by the unjointed sample. The connection tests indicate that the bodkin joints with the following pin connectors have efficiencies of 95% or greater and, therefore are considered to perform well [Bonaparte, 1987b]: 0.12 in. flat galvanized steel bar, 0.18 in. flat HDPE bar, 0.5 in. rectangular HDPE dowel, 0.5 in. diameter steel rod, 0.5 in. diameter steel pipe, and 0.24 in. flat galvanized steel bar. A straight bodkin connection with a 1 in. diameter PVC pipe connector (the same as used on the Tanque Verde Road project) has a 62% efficiency. All the bodkin

joints tested showed extensions at the design strength from 0.8 to 1.4 in. These extensions show the importance of pretensioning the bodkin joint in the field prior to backfilling over the joint. Pretensioning would reduce the connection extension by removing slack and prestraining the geogrid. As a result, facing panel movements will be reduced.

Facing Panels. As discussed in Section 4.1, the concrete facing panels were supported on two leveling bolts. Since the bottom of the facing panels were fixed, outward panel movement was accompanied by panel bending as illustrated in Figure 7-3 and 7-4. The panel bending and stress concentrations caused by the pinned connections are thought to have been at least partially responsible for the observed "spider" cracking of the concrete. Today, the bolting connection detail at the Tanque Verde Road project is not being used. Currently, alignment of the panels is usually achieved using temporary wood wedges or temporary clamps.

Since the Tanque Verde Road project, several of other facing systems have been used in TENSAR geogrid-reinforced soil walls in North America. These facings include articulated concrete panels, similar to the panels used in the Georgia Stabilized Earth (GASE) [Berg, et al., 1987], and full- height, lightweight precast "waffle" type panels [Tensar, 1986]. Other facing types that have been used with both geotextiles and geogrids include masonry-block, precast concrete blocks, gunite, timber railroad ties, gabions, and geotextile or geogrid wrap-around facings [Yako and Christopher, 1987].

Wall Batter. Observed panel movements on the Tanque Verde Road project wall indicated that the 1H:96V wall batter was not sufficient and that a batter of 1H:60V would have been more appropriate. Today, the wall batter used with full-height panels and geogrid reinforcement ranges from 1H:60V to 1H:48V.

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Appendix 5.1.2.2

Transportation Research Record report for ARES Tanque Verde Project

ARES® MSE Retaining Wall System

Back to main document

Durability of Geosynthetic Soil Reinforcement Elements in Tanque Verde Retaining Wall Structures

DONALD G. BRIGHT, JAMES G. COLLINS, AND RYAN R. BERG

The findings of an investigation into the stability and durability of high-density polyethylene (HDPE) geogrid soil reinforcing elements used in grade separation structures on a project at the Tanque Verde–Wrightstown–Pantano Roads intersection in Tucson, Arizona, is documented. The project represents the first use of geogrid reinforcement in concrete-faced, mechanically stabilized earth retaining walls in a major transportation-related application in North America. The reinforced soil walls were constructed in 1984 and 1985, and the geogrids have been in service for 8 to 9 years in an elevated temperature environment that accelerates the mechanisms of degradation. The combined effects of age and temperature exposure made this project a candidate for evaluation of geosynthetic reinforcement stability and durability. A sample retrieved from the project was subjected to a series of laboratory procedures and tests. Results of the tests are presented as topological analysis, ultimate tensile strength, ultimate tensile strain, 1,000-hr creep response, melt rheology, melt temperature range, crystallinity, and oxidative induction times. Test results are compared to archived geogrid values. Soil samples also were retrieved and analyzed. It is concluded that after more than 8 years of exposure to an elevated temperature environment, the exhumed HDPE geogrid has not experienced any significant change in the physical and performance properties of the geogrid or the morphological properties of the HDPE.

The durability of a specific geogrid soil reinforcing element used in a specific project is examined. The geogrid elements have been in service for between 8 and 9 years in concrete-faced, mechanically stabilized earth (MSE) retaining wall structures. The project is located in the southwestern United States and is exposed to high summer temperatures.

A summary description of the project is presented and references providing detailed design and performance information are noted. The mechanical instrumentation program and monitored results are referenced. The procedure used for exhumation of a geogrid sample is described, and a description and illustration of the geogrid reinforcing element is presented. The test methods used in this durability evaluation and a rationale are summarized along with tabulated test results and a discussion of the findings. Conclusions are stated regarding the durability of the geogrid based on interpretation of test results from archived and exhumed samples.

PROJECT DESCRIPTION

Forty-six geogrid-reinforced retaining walls were constructed to created grade separations for the Tanque Verde–Wrightstown–Pantano Roads intersection in Tucson, Arizona (Figure 1). Approximately 1600 lineal meters (m) of wall were constructed between December 1984 and September 1985. The walls vary in height from 1 to 6 m.

The precast concrete wall face panels are 0.15 m thick, 3 m wide, and full height. Four cross-section wall geometries, as illustrated in Figure 2, were used for the 46 wall structures. Comprehensive details of the wall design are presented in a value engineering study, the design report (*1*), and in the project construction drawings (*2*). Two of the Type D wall sections were instrumented in September 1985 and have been monitored since then. The goal of the instrumentation program is to assess the performance of the wall structures. Stresses and strains in the soil and strain in the geogrid are monitored with pneumatic load cells, inductance coils, and resistance strain gauges. Internal and external wall temperatures are monitored with resistance thermometers. Panel displacements during and immediately after construction were monitored using optical survey techniques. Brief descriptions of the instrumentation program and instrumentation readings during and immediately after construction have been presented by Berg et al. (*3*). Reports of the post-construction monitoring program have been presented by Desert Earth Engineering (*4,5*).

A comprehensive report on the project, incorporating the Dames and Moore design report and the Desert Earth Engineering monitoring results and an interpretation, has been published by the FHWA (*6*) and by Fishman et al. (*7*). A summary of monitoring results through the first 4 years in service was presented at the 1991 TRB Annual Meeting (*8*). A summary of monitoring results through the first 8 years in service was presented at the Seiken Symposium in Japan by Collin and Berg (*9*).

Based on resistance thermometer readings from one of the two monitored Type D wall sections, transient temperatures up to approximately 49°C (120°F) on the concrete wall face panels were recorded, and up to approximately 38°C (100°F) behind the panels. Figure 3 shows the temperature readings at various elevations within a Type D wall section as recorded in March and June 1986.

Mechanical response over time is the focus of the still-active instrumentation program. Based on readings of the two monitored wall sections, the geogrid reinforcement has experienced low tensile loads, resulting in maximum strains of less than 1 percent. Typical strains of 0.5 percent and 1.0 percent have been recorded for the two instrumented sections.

Although instrument readings taken as recently as September 1992 indicate no significant change in performance over time, an investigation into the stability of the geogrid soil reinforcing elements was initiated in 1993. An analysis of the geogrids used in this project was deemed appropriate because it is the oldest struc-

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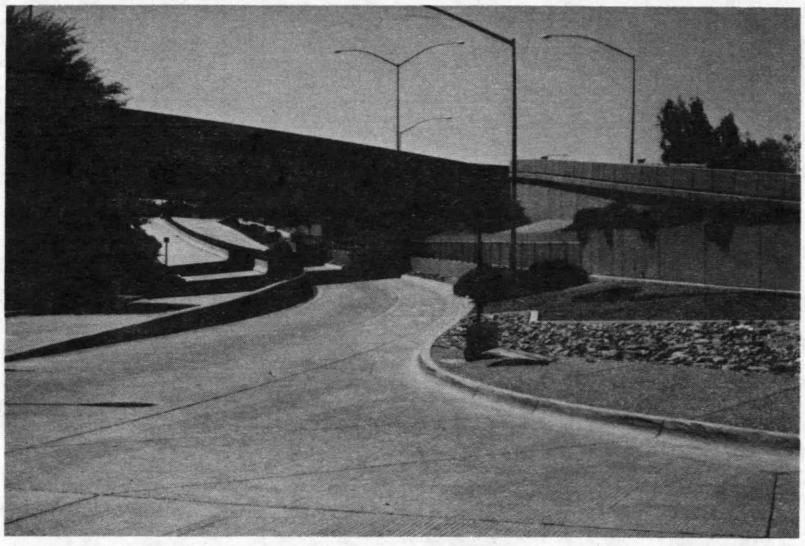

FIGURE 1 Tanque Verde–Wrightstown–Pantano Roads intersection.

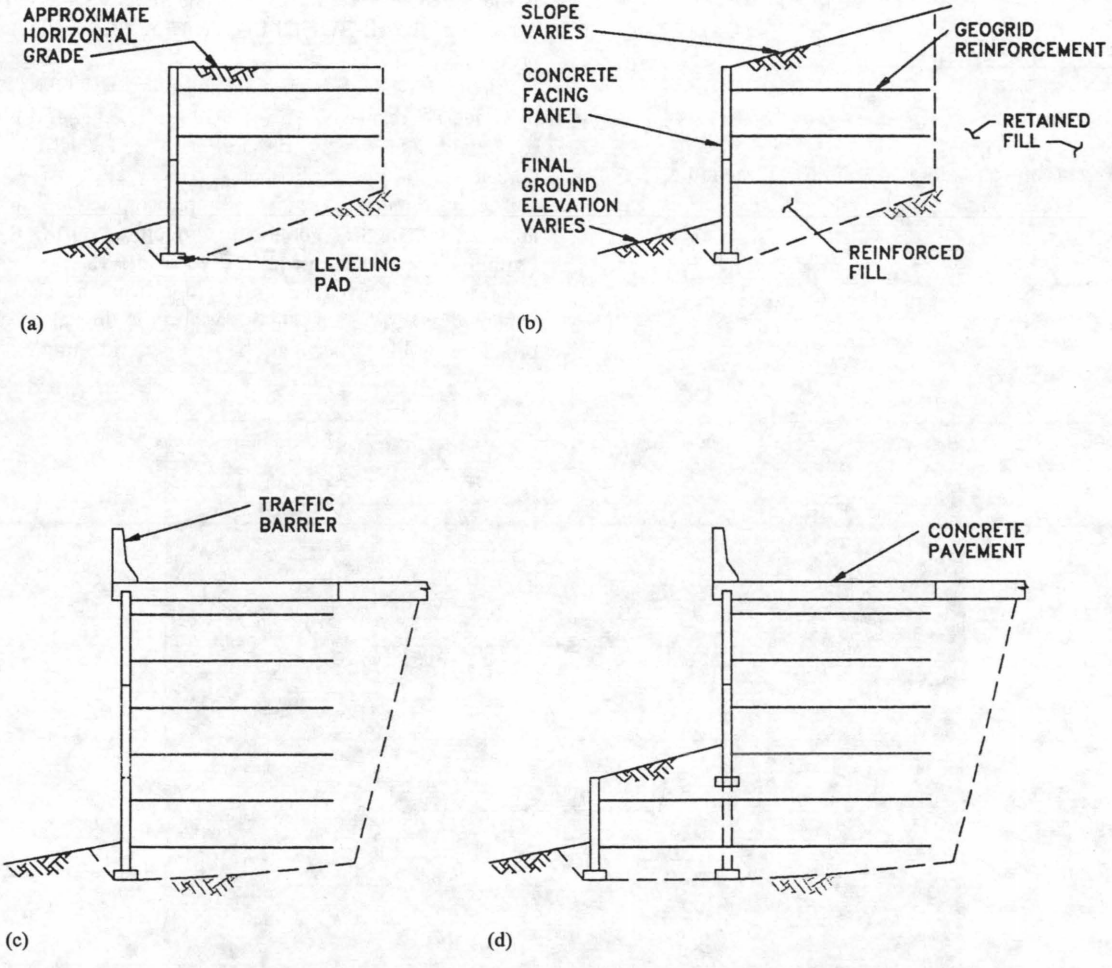

FIGURE 2 Four wall geometries used in Tanque Verde retaining wall structures. *a*, Type A wall; *b*, Type B wall; *c*, Type C wall; *d*, Type D wall.

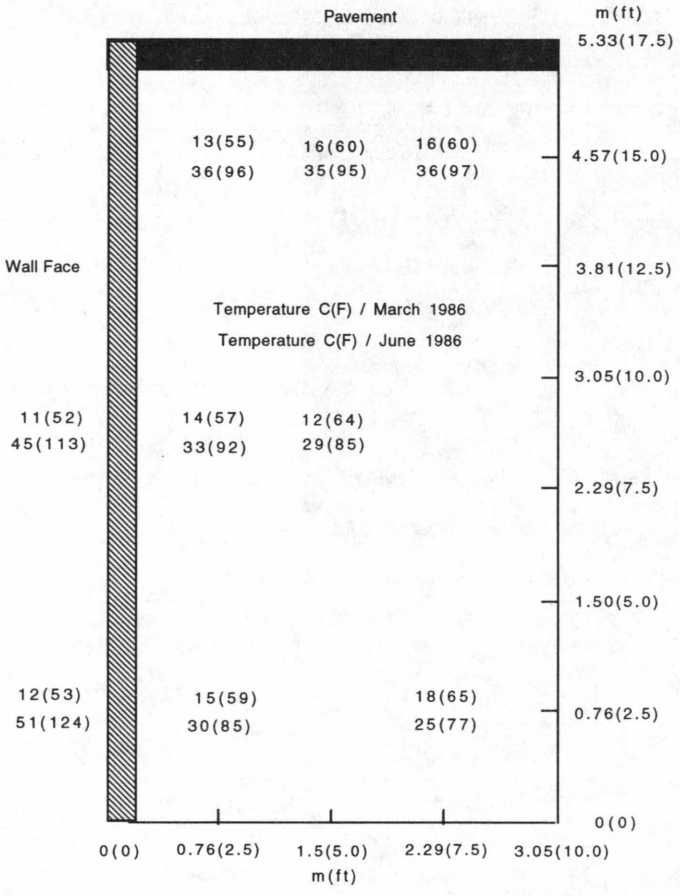

FIGURE 3 Temperature readings within a wall section.

ture of this type in service and because of the elevated temperatures associated with the project's location in the Sonoran Desert of Arizona.

EXHUMATION

A sample 1 m wide by 2.5 m long of the geogrid soil reinforcing elements from the Tanque Verde project was exhumed in August 1993. The sample was hand excavated from a Type A wall (Figure 2) adjacent to Tanque Verde Road (Figure 4). Figure 5 shows the ground surface before excavation; Figure 6 shows the texture and composition of the excavated reinforced fill. The sample was located 0.3 to 0.4 m beneath the finished grade behind the wall face, as shown in Figure 7. Once the bulk of reinforced fill was removed with shovels, the geogrid was brushed clean with a small whisk broom to avoid exhumation damage. Figure 8 shows the geogrid brushed clean before its removal. Figure 9 is a representative close-up of the geogrid before its removal. Also this geogrid sample was exhumed from above a very active ant nest.

GEOGRID MATERIAL

The soil reinforcing element used on this project is a Tensar SR2 geogrid. It is a uniaxial product fabricated by punching, reheating, and drawing an extruded sheet of high molecular weight, high-density polyethylene (HMW HDPE). The geometry of the geogrid is illustrated in Figure 10. Drawing increases the molecular orientation of the HMW HDPE, enhancing the tensile and modulus properties of the geogrid.

This HMW HDPE is classified as a Type III, Class A, Category 5, Grade E5 resin per ASTM D1248. The geogrid composition, by weight, and constituents are 97+ percent HMW HDPE, 2+ percent carbon black, plus antioxidants.

The specimens used for comparison testing are from an archived roll of SR2 geogrid manufactured during the same era (within 1 year) as the geogrid used in the Tanque Verde project. Because the archived geogrid is not from the same lot as the exhumed geogrid, some small variation in the values of measured properties is likely between the two geogrid samples.

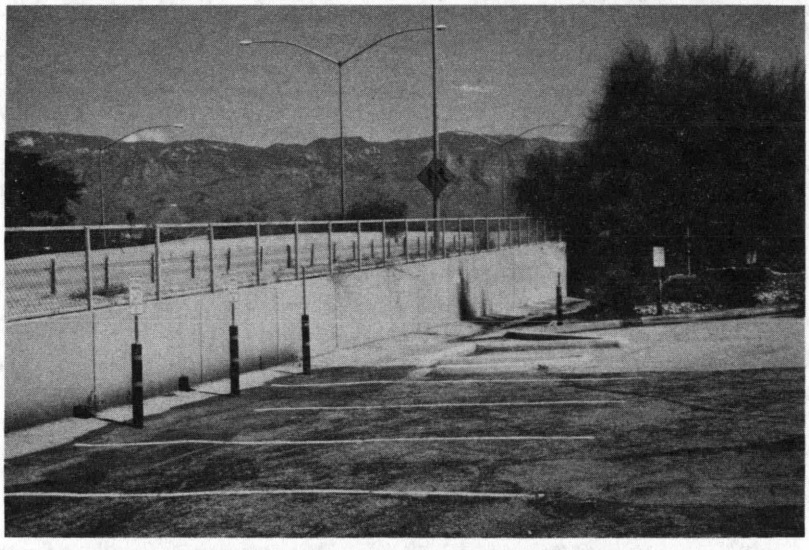

FIGURE 4 Type A wall where SR2 geogrid sample was exhumed.

FIGURE 5 Ground surface at excavation site of SR2 geogrid sample.

FIGURE 6 Representative texture and composition of excavated reinforced fill.

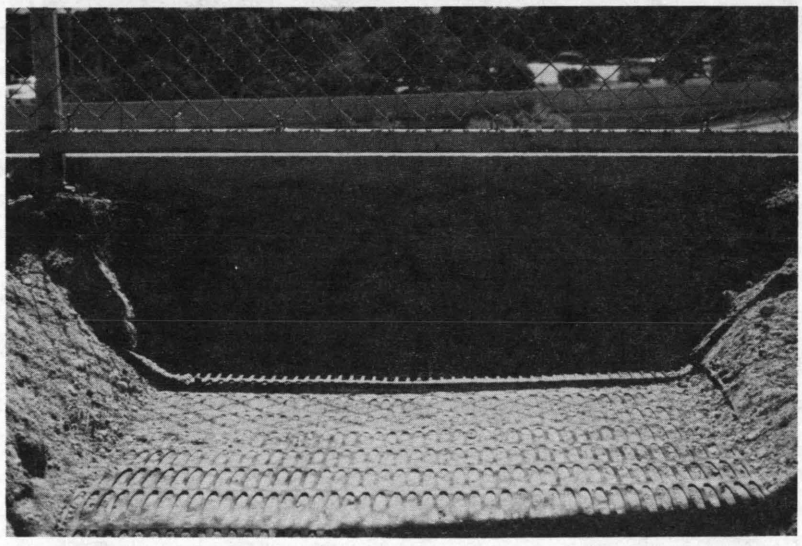

FIGURE 7 Uncovered SR2 geogrid toward Type A wall.

FIGURE 8 SR2 geogrid brushed clean, showing no evidence of physical damage.

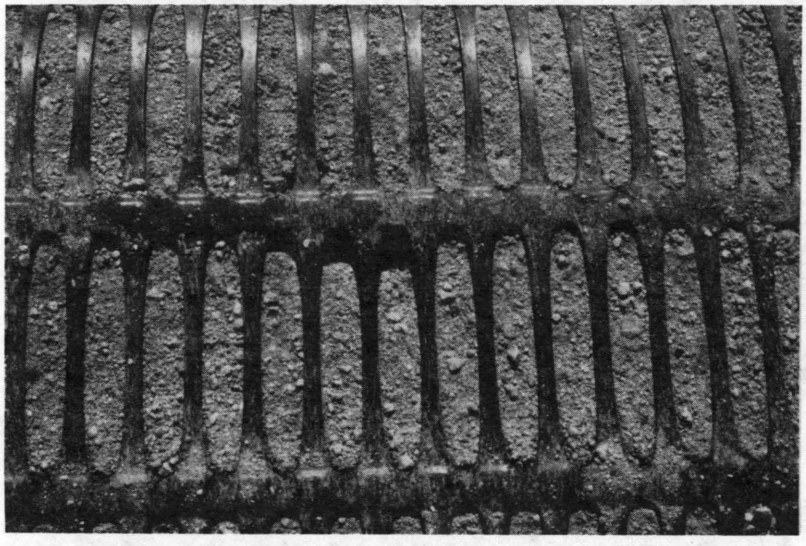

FIGURE 9 Close-up of SR2 geogrid, showing no evidence of physical damage and presence of a glossy surface.

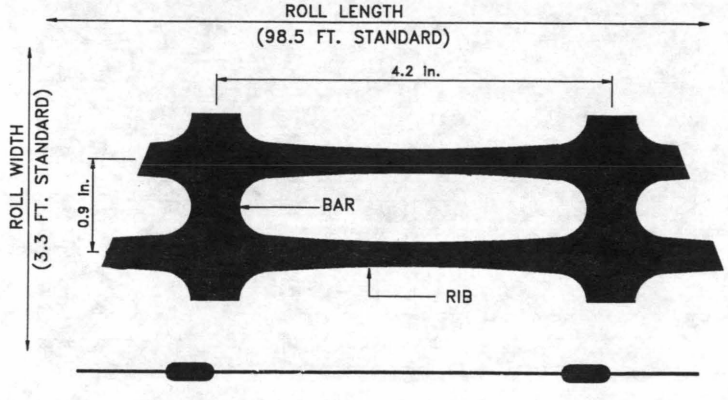

FIGURE 10 Geometry of SR2 geogrid

TEST PROTOCOL AND STANDARDS

Deterioration of geosynthetics may occur due to physical damage (i.e., installation damage), mechanical deformation (i.e., dimensional change, tensile and elongation behavior, creep response), thermal degradation (i.e., transition temperatures, crystallinity, oxidation), and biological degradation (i.e., attack by macro- and microorganisms). Thus visual, physical, mechanical, thermal, and compositional tests have been performed with specimens from the archived and exhumed geogrid samples.

Visual analysis, using a photographic record, assesses the presence and extent of installation, exhumation, and macroorganism damage across the surface topology. Scanning electron micrographs show the extent of such damage, that is, surface degradation due to attack by oxidation (surface dullness), soil chemistries, and microorganisms. Physical tests assess dimensional stability due to annealing, as well as subsequent densification via geogrid shrinkage due to prolonged exposure to an elevated temperature environment. Mechanical tests assess retention of tensile and elongation properties and the behavioral response to a constant sustained load. But these tests cannot necessarily differentiate between the mechanisms of mechanical deformation. Thermal tests, however, may assess any significant changes in the morphological status of the HMW HDPE that may relate to changes in mechanical properties. Composition tests indicate the residual amount of the principal additive, carbon black. Comparison with original formulations documents any concentration changes, thus indicating the duration of long-term stability.

The parameters evaluated, the related tests and applicable standards, and the number of specimens (i.e., archived and exhumed) per test are summarized in Table 1. The ASTM standards are taken from the *1993 Annual Book of ASTM Standards* published by ASTM, Philadelphia. The GRI standards are taken from *Test Methods and Standards* published by the Geosynthetic Research Institute at Drexel University in Philadelphia. The industrial standard for oxidative induction times (OITs) stipulates that a specimen be heated in a calorimeter at a rapid rate in an inert atmosphere (i.e., nitrogen) to a temperature usually above the melt range of the polymer (e.g., 200°C), allowed to thermally equilibrate, and then be switched to an oxygen atmosphere and timed and recorded to the onset of an exotherm. An exotherm indicates oxidative degradation in progress.

TEST RESULTS

Physical and mechanical test results and resin properties are summarized in Table 2. For tests using multiple specimens, results in Table 2 are reported as arithmetic averages with standard deviation. Figure 11 shows the unconfined tension creep evaluation of the exhumed SR2 geogrid in progress. Figure 12 shows the creep response of specimens taken from two rolls of SR2 geogrid manufactured in the 1984–1985 era, but within 1 year, and loaded to 31.7 kN/m, equivalent to 2,170 lb/ft. One roll was chosen for quality control (QC) testing following its manufacture; the other roll went into the Tanque Verde project and was exhumed more than 8 years later for evaluation. Figure 13 shows the creep response of specimens taken from two rolls of SR2 geogrid manufactured in the 1984–1985 era also, and within 1 year; both specimens are loaded to the design loading of 29 kN/m (2,000 lb/ft). One roll was archived; the other roll is the same as in Figure 12. Results of thermal analysis of geogrid components are summarized in Table 3. Melt transition and crystallinity sample sizes ranged from 9.4 to 10.2 mg. Only one thermogram was run per component because specimens were taken at random and results were typical of HMW HDPE.

A soil sample was taken from the exhumation site at Tanque Verde in Tucson, Arizona, for analysis; the soil fractions were 15.5/81.8/2.7, by weight percent, for gravel, sand, and fines, respectively. Soil pH was 8.0 and 8.7 in distilled water, and 7.6 and 7.8 in a 0.01 molal solution of $CaCl_2$, respectively.

DISCUSSION OF RESULTS

Visual Assessment

Photographs of the exhumation site at the Tanque Verde–Wrightstown–Pantano Roads intersection are presented as Figures 4 and 5. Figure 6 shows the texture and nature in the reinforced

TABLE 1 Properties and Related Tests and Specific Standards

Tests	Parameters	Standards	Number of Specimens	
			Archived Sample	Exhumed Sample
Physical	Mass / Area	ASTM D 3776	1	1
	Thickness, Rib	ASTM D 5199	10	10
	Thickness, Node	ASTM D 5199	10	10
Mechanical	Rib Strength	GRI GG1	10	10
	Junction Strength	GRI GG2	10	10
	Wide Width Strength	ASTM D 4595	10	10
	Tension Creep Behavior	ASTM D 5262	1	1
Thermal	Transition Temperatures	ASTM D 3418	3	3
	Heat of Crystallization	ASTM D 3417	3	3
	Oxidative Induction Time	Industry	3	3
Resin	Density/Specific Gravity	ASTM D 0792	2	2
	Melt Flow Index	ASTM D 1238	2	2
	Carbon Black Content	ASTM D 4218	1	1
Soil	pH	ASTM D 4972	n/a	2

TABLE 2 Physical and Mechanical Test Results and Resin Properties

Parameters	Number of Specimens		Number of Specimens	
	Average Value	Standard Deviation	Average Value	Standard Deviation
Mass / Area (g/m²)	912.	-	930.	-
Rib Thickness (in)	0.054	0.009	0.054	0.004
Junction Thickness (in)	0.178	0.002	0.177	0.001
Single Rib Strength (kN/m)				
Max. Load @ % Strain	85.0/15.2	1.79/1.15	85.0/14.0	0.45/0.64
Load @ 5 % Strain	46.8	0.85	48.6	0.35
Load @ 2 % Strain	26.2	0.65	26.5	0.29
Junction Strength (kN/m)				
Max. Load @ % Strain	84.6/16.4	0.63/0.59	83.7/16.0	0.42/0.55
Load @ 5 % Strain	46.1	0.40	47.0	0.57
Load @ 2 % Strain	26.1	0.42	25.9	0.41
Wide Width Strength (kN/m)				
Max. Load @ % Strain	78.0/15.3	3.0/1.28	78.0/14.0	2.2/1.21
Load @ 5 % Strain	44.6	0.40	43.3	0.30
Load @ 2 % Strain	25.6	0.45	23.6	0.37
Density (g/cc)	0.9530	0.0005	0.9595	0.0002
Melt Flow Index (g/10 min)	0.225	0.007	0.207	0.004
Carbon Black Concentration (%)	2.06	0.020	2.96	0.031

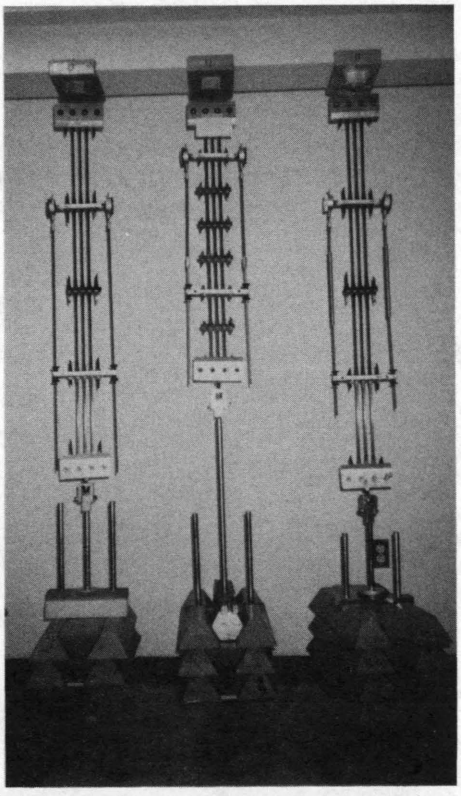

FIGURE 11 Unconfined tension creep evaluation of exhumed SR2 geogrid (center).

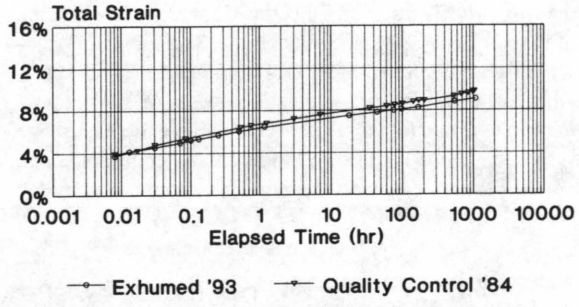

FIGURE 12 Creep response of quality control and exhumed SR2 geogrids at 31.7 kN/m.

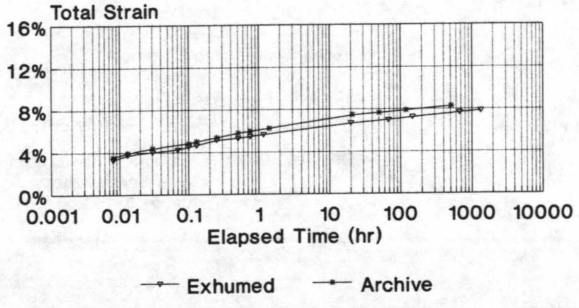

FIGURE 13 Creep response of archived and exhumed SR2 geogrids at 29 kN/m.

TABLE 3 Thermal Analysis Results

Parameter	Number of Specimens			Number of Specimens		
	Rib @ Mid Length	Node	In-Between Nodes	Rib @ Mid Length	Node	In-Between Nodes
Melt Range (°C)	90-139	93-134	91-136	90-134	93-133	92-133
Melt Peak (°C)	130	130	130	130	130	130
Crystallinity (%)	58	54	57	58	55	56
OIT (minutes)	6.7	8.0	7.4	4.7	4.3	3.9

fill, which was well-compacted and uniform. Figure 8 shows no broken or cut ribs over the surface of the exhumed geogrid sample; thus this sample experienced no significant installation or exhumation damage. There was no evidence of surface degradation due to attack by the resident ant colony; apparently the ants simply were not interested in a geogrid of HMW HDPE. Typically oxidation starts on the surface and progresses inward. An oxidized surface of a polyolefin, like polyethylene, will appear dull or tarnished. But surface quality is glossy, as evident in Figure 9, indicating no oxidative degradation in progress. Thus topological analysis by scanning electron microscopy was not warranted.

Physical Tests

Physical test results show no significant change in dimensional properties throughout 8-plus years. Rib and junction thicknesses exhibit zero change. The change recorded for mass per unit area is within the variance of specification for SR2 geogrid.

Mechanical Test Results

Test results show no significant change in tensile strength measured at 2 percent and 5 percent strain levels and at maximum loading and corresponding strain between the two samples of geogrid. The ultimate tensile strength values for a single rib are above the 79.0-kN/m product specification. The average tensile strength values plus one standard deviation for the exhumed sample fall within, or significantly overlap, the corresponding average values plus one standard deviation for the archived sample. Thus there is no significant change in single-rib, junction, or wide-width tensile strengths between archived and exhumed geogrid manufactured during the same era (within 1 year). There is also no indication of loss in ductility, or embrittlement, of the exhumed geogrid with time.

Creep Response

Figure 12 shows the creep response of two SR2 geogrid specimens manufactured in the 1984–1985 era; one taken from a roll selected for QC assessment and the other exhumed from the Tanque Verde project, both at the same loading of 31.7 kN/m. After 1,000 hr, both specimens exhibited a parallel response to the same constant, sustained loading, indicating that the mechanism by which creep occurs is the same. This behavior indicates that more than 8 years of exposure to an elevated temperature environment has not changed the creep response of aged versus production SR2 geogrid. Figure 13 shows the creep response of exhumed and archived SR2 geogrid specimens at the design loading, 29 kN/m for a total strain response of less than 10 percent. The archived geogrid has been in storage since its manufacture in 1984. At about 1000 hours, the two specimens are becoming asymptotic to less than 8.5 percent total strain. The two response curves are essentially parallel, indicating that the mechanism by which creep occurs is the same within the two geogrid specimens. Although these two geogrids are from different lot numbers, as discussed earlier, their response with time to a constant, sustained load is essentially the same; in addition, the mechanism by which the geogrid specimens respond is identical and thus has not changed over more than 8 years' duration and exposure to different aging temperatures, ambient and elevated, as documented in Figure 3.

Resin Properties

Resin density was determined from the extrudate from a melt flow index tester; no significant changes in morphology occurred over the duration. Melt flow index values indicate no change in the molecular weight of the resin over the duration. Any significant change in molecular weight would be reflected in corresponding changes in mechanical strength, for which there were no changes. However a 0.022-g/10-min difference does indicate that the samples came from different production lots, also indicated by the values on carbon black (CB) concentration.

CB specification for SR2 was 1 to 3 percent by weight in 1984, and 2+ percent is known and accepted to be sufficient to retard long-term degradation of HDPE due to exposure to ultraviolet light, which is of no concern here. The difference in CB concentration has not affected the mechanical properties of the exhumed geogrid relative to the archived geogrid; a higher CB concentration, if anything, would slightly lower mechanical properties initially. Any significant change in ductility or embrittlement would increase strength values with a corresponding decrease in strain values. As discussed earlier, this has not occurred.

Thermal Test Results

Melt range, peak temperature, and crystallinity data indicate no significant changes in the morphological state of the HMW HDPE geogrid, at various locations within the geogrid configuration, over the duration of the project in an elevated temperature environment. Crystallinity data verify that there was no significant change in the resin density reported in Table 2, because for semicrystalline thermoplastics, changes in morphological density are usually reflected in corresponding crystallinity. Any significant changes in either density or crystallinity should be reflected in mechanical property results, which is not the case. Such changes would not necessarily reflect a change in molecular weight.

OITs for the archived and exhumed samples are given in Table 3. A single test was run for each specimen identified by location within the basic geogrid configuration. OIT values of the archived SR2 sample are 7 to 8 min, typical of values reported by the resin supplier for the antioxidant package used in 1984 and evaluated at 200°C. OIT values for the exhumed sample are 4 to 5 min. For an average time duration of 6 ± 2 min, the OIT values of the archived and exhumed specimens are, within experimental reproducibility and significance for this test, essentially equal. The mass/area, melt flow index, and CB values clearly indicate that the test samples are from different production lots, as do the OIT values. Thus some difference in OIT values is expected between the archived and exhumed samples.

CONCLUSIONS

The Tanque Verde and archived geogrid samples were produced about 1984 but were from different production lots. The HMW HDPE geogrid experienced no significant installation or exhumation damage and exhibited no evidence of biological attack or surface oxidation. No significant change in physical, mechanical, thermal, or resin properties occurred throughout 8 or more years in service. The creep behavior of the archived and exhumed Tanque Verge geogrids is essentially identical and indicates that the mechanism by which creep occurs has not changed throughout the years in service. Thus assessment has shown that the first project using a geogrid of HMW HDPE as reinforcement in a concrete-faced, MSE retaining wall in a major transportation-related application in North America has not experienced any significant change in the performance and physical properties of the geogrid or in the morphological properties of its resin in more than 8 years of exposure to an elevated temperature environment.

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Appendix 5.1.2.3

Paper about Overlander Bridge Access Project

ARES® MSE Retaining Wall System

Back to main document

Mechanically Stabilized Earth Retaining Walls, Overlander Bridge Access Improvements, Kamloops

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J. Lockwood
Western Concrete Products, Armstrong, British Columbia, Canada

ABSTRACT
The "City of Kamloops" selected an optimized ARES MSE system comprised of Full Height Panels coupled with HDPE Geogrid Soil Reinforcement. This paper will discuss the innovative use of geogrid soil reinforcement for what is commonly referred to as "true abutment support", whereby the bridge seat is cast directly upon the compacted MSE mass. The project was challenging from the settlement perspective as founding materials quickly transitioned from bedrock to fills. This paper will focus on the key aspects associated with extensible geogrid design for true abutment support. In addition, it will emphasize the efficiencies associated with full height panel construction in comparison to other alternatives.

RÉSUMÉ
La ville de Kamloops a choisi un système optimisé ARES MSE consistant de panneaux de pleine hauteur accompagnée de géogrille de renforcement de PEHD. Ce document discutera l'utilisation innovatrice du renforcement de sol avec les géogrilles pour ce qui est généralement conçu sous le nom de "véritable appui de butée", par lequel le siège du pont soit coulé directement sur la masse MSE compactée. Le défi de ce projet relevait des affaisements possible car il y avait une transition rapide des matériaux d'infrastructure de roc en place au remblai. Ce document se concentrera sur les aspects clé liés à la conception des géogrilles extensibles pour les véritables appuis de butée. En outre, il soulignera les efficacités liées à la pleine construction de panneaux de pleine hauteur comparativement à d'autres techniques.

1 INTRODUCTION

This true bridge abutment project is located within the City of Kamloops, just northwest of the downtown core. In 1998 the City opted to tender this project as bridge access improvements at the Overlander Bridge.

The original tender condition allowed the use of conventional MSE modular panels complete with extensible reinforcement. During the tender period the successful contractor (BA Blacktop) opted to employ Tensar's Full Height ARES concrete wall system. This wall system utilizes extensible High Density Polyethylene HDPE geogrid mechanically attached to the back of the precast concrete facing.

This project was innovative as it was one of the first Geogrid HDPE MSE Abutment supporting walls whereby the bridge seat footing is supported on the top of MSE backfill (without any piles). The project was tendered by the City of Kamloops in 1998 and construction proceeded during the summer of 1999. This structure has now been in service for 10 years, and this paper will discuss the technical highlights.

Figure 1. Looking West along approach ramp

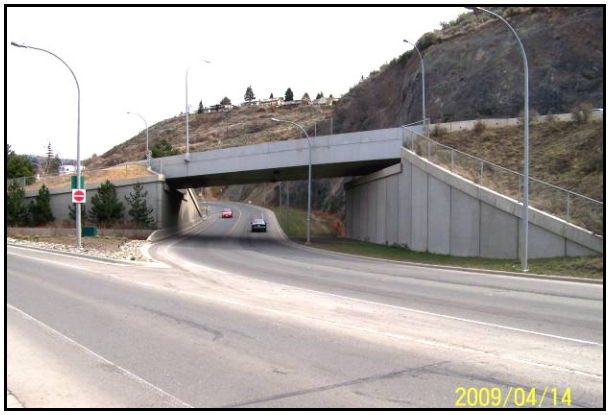

Figure 2. Looking Southeast beneath the bridge deck.

2 GEOTECHNICAL ASPECTS

The original geotechnical investigation was completed for the owner by Amec Earth and Environmental of Kamloops, BC.

2.1 Stratigraphy

In summary their findings revealed bedrock and shallow bedrock at the western extent of the project. Along the northeastern part of the site the presence of fills in the range of one metre deep were noted at some locations during initial geotechnical investigation. These fills were underlain by varying sands/gravels, silts/clays, in turn underlain by bedrock.

It was suggested that surficial organic materials be removed, and the native soils be compacted to 98% standard proctor density. Any further soft zones delineated during compaction would then be excavated and replaced with granular material. Deeper excavations were not possible along this corridor due to the presence of the existing road network connecting North and South Kamloops, as well as the adjacent rail line.

For both abutment walls it was anticipated that the abutment supporting structure would be founded upon bedrock on the west and shallow bedrock on the east. Estimated settlements were anticipated to be in the range of approximately 15 mm beneath the abutment zone.

The underlying bedrock surface was anticipated to dip down toward the north and east direction (towards the long approach wall seen in Figure 1). Settlement along the North approach wall was anticipated to be upwards of 90 to 150 mm.

2.2 West Abutment Proximal to Bedrock

The west abutment (Figure 3) was located at an existing bedrock outcrop at its southern extremity.

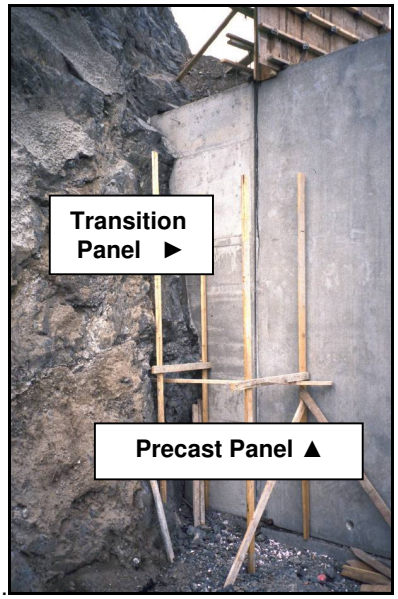

Figure 3. Precast panel c/w C.I.P. transition panel

2.3 Special measures at base of wall panel

For the approach ramps, when not directly bearing on rock, a frost-trench was over excavated and replaced with non frost susceptible aggregate. Refer to Figure 4 below for the sketch of the frost trench beneath level pad.

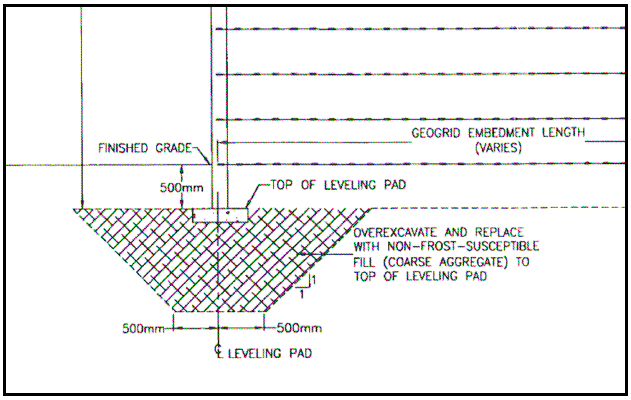

Figure 4. Frost trench with 1:1 sideslopes

3 DESIGN APPROACH WITH HDPE GEOGRID

A 100 year service life was specified by the Owner's Engineer, Stanley Consulting Group Ltd, of Kamloops. Design codes at that time consisted of the following:

CAN/CSA-S6
AASHTO where CSA-S6 did not govern
ATC-6 Seismic design guidelines.

3.1 HDPE Geogrid Types Utilized

For the purpose of this project the following geogrid types manufactured by Tensar were employed as follows:
Table 1. Geogrid Properties

Grid Designation	T ult kN/m
UX 1400HS	64.1
UX 1500HS	100.6
UX 1600HS	131.3
UX 1700HS	157.3

T ultimate is the ultimate strength of the geogrid before applying reduction factors. The load is measured in accordance with ASTM D6637.

3.2 HDPE Vertical Geogrid Spacing

Geogrid soil reinforcement was designed at vertical spacing increments of 0.5 m to 1.0 m.

The closer spacing of 0.5 m attributed to the abutment walls, where the tallest panels were located. The wider spacing of geogrid was often employed on the low panel heights such as the approach ramps for example.

On the panel noted in Figure 5 both grid spacing's are evident. Closer grid spacing at the top of panel was also required to allow for impact loads from traffic barriers at top of wall.

Figure 5. Geogrid Tabs at 0.5 and 1.0 m intervals

3.3 Reduction Factors on HDPE Design

This proprietary Tensar design implemented the following reduction parameters.

Durability reduction factors RFd = 1.1 based on clean granular structural backfill within the (reinforced zone).

Installation damage factors ranging from 1.10 to 1.20 were applied dependant on specific grid type. Creep reduction factors ranging from 2.33 to 2.48 were applied.

3.4 Owners Geotechnical Design Inputs

The soil parameters for design inputs were provided by the owner's geotechnical engineer as follows:

Reinforced backfill φ = 35° and 19.5 kN/m^3
Reinforced backfill φ = 30° and 19.5 kN/m^3
Reinforced backfill φ = 30° and 19.5 kN/m^3

Seismic acceleration of 0.05g
Max. applied bearing pressures of 250 kPa.
Traffic surcharge of 12 kPa

3.5 Internal Design

Within the MSE mass, stability is achieved using the strength of the soil being reinforced in conjunction with the tensile force of the Tensar Geogrid.

Design methods ensured that the geogrid were long enough to avoid pull out of the fill behind the Rankine failure plane, and that geogrid was well distributed within the reinforced mass, with sufficient tensile stress to preclude rupture (either short or long term).

Grid lengths on the abutment faces ranged from 75% to 95% of the total height (as measured from base of wall to top of road surface). Shorter grid lengths were evident on the West Abutment adjacent to rock. Grid lengths were maintained at 2.5 m or greater as per the owner's requirements, and AASHTO guidelines.

3.6 External Design

The following minimum factors of safety were maintained as follows:

Table 2. External Factors of Safety.

Condition Checked	Static	Seismic
Sliding on Base	1.5	1.1
Overturning	2.0	1.5

3.7 External Design Rock Cut Considerations

Due to the presence of bedrock at the West Abutment, an additional analysis of the external stability for rock failure along the critical slip plane was completed.

It was determined the critical slip plane was oriented at 42.5 ° from horizontal for the sliding condition. For this condition a Factor of Safety for sliding of 5.46 was calculated. During the assessment of the overturning condition a critical slip plane of 46.4 ° was determined, and yielded a 3.5 Factor of Safety.

4 MANUFACTURE WITH HDPE GEORID

4.1 Precast Panel Geometry

Panels were cast to full wall height with a nominal 20 mm joint between adjacent precast elements. This joint width is typical of the MSE precast industry.

Panels shown in Figure 7 were of standard size of 2765 mm wide and 170 mm thick.

4.2 HDPE Geogrid Tabs in Precast Placement

Grid tabs are simply set within the back face of precast panel at time of casting. The tab is embedded a minimum of 100 mm as shown in Figure 6.

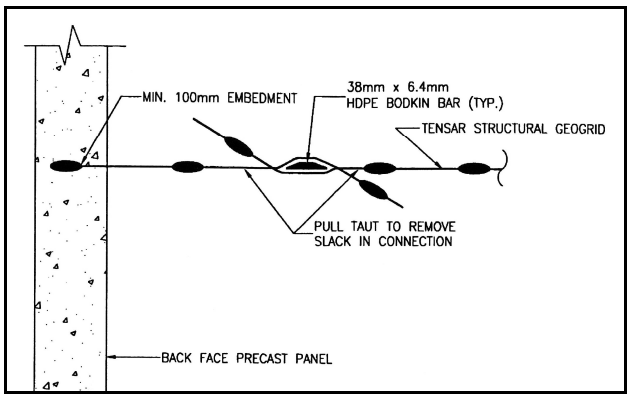

Figure 6. Grid tab embedment of 100 mm

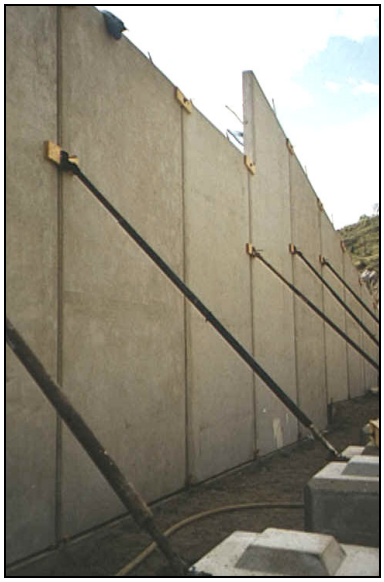

Figure 7. Transition Panels Abutment to Wingwall

4.3 Concrete Panel Specifications

Panels were cast with the following specifications.

Type 10 Cement
Air Content of 5 to 8%
Strength @ stripping of 20 MPa
Strength @ 28 days of 35 MPa

Precasting was completed by a CSA certified precaster at that time formerly known as Western Concrete Products.

4.4 Special Precast and Joint Detail for Settlement

The owner's geotechnical report had cited the possibility of settlement of up to 150 mm of total settlement. Since settlement could not be entirely ruled out, a special precast joint needed to be implemented to mitigate the settlement of adjacent panels.

After extensive review of this potential settlement concern it was determined the best solution from a precast point of view was to place a settlement joint at a midway location on the long approach wall. If the reader recalls earlier in this report this was the area where the previous fills had been predominantly noted. In reference to Figure 8 the special joint has been flagged with the arrow pointer. This allowed for a joint width at this location larger than the standard 20mm joint width.

The "backup panel" as it was referred to is noted proud of the front face of precast, allowing the designer to aesthetically hide an imperfect joint as settlement progressed during the short-term.

In the photo below one will also note that a larger joint was also required at the corresponding coping location.

Figure 8. Precast settlement joint shown above

This wall settled up to 150 mm over this 10.9 meter long section, in the range of 1.4% settlement.

4.5 HDPE Grid for Corner Elements

For standard 90 ° corner elements small width geogrid was cast in the backside of the corner elements. The design methodology for the corner element is identical to that of the larger panels. This ensures that the corner elements are not just aesthetically pleasing, but more importantly; structural load carrying precast elements.

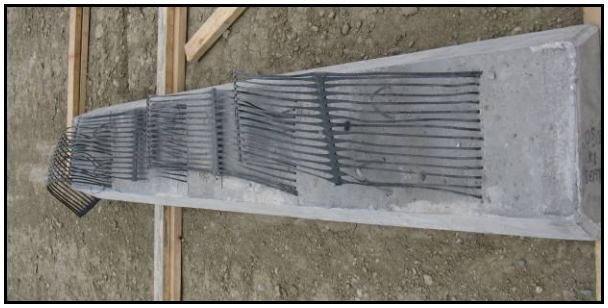

Figure 9. Special precast corner c/w geogrid tab

4.6 Initial Panel Placement

In Figure 11. full height panels are in the process of being rotated to a near vertical position. The panel is lifted by four pick- points on the back surface of panel.

In the lower portion of this photo (Figure 11) Richard Lockwood of Western Concrete Products has carefully planned and prepared the site by pre-positioning Lock Blocks to facilitate the efficient erection of panels as later described in this paper.

In Figure 12 the reader can see the precast panels securely braced with steel support struts at the 2/3 rd panel height location. The panels at this stage are placed and secured with a pre-defined batter in readiness for backfilling.

Figure 11. Panel lift from 4 pick points

Figure 12. Placement of Approach ramp panels

4.7 Bridge Seat Supported Directly On HDPE Geogrid

In an identical fashion to the more traditional method of MSE design, the bridge seat is cast directly atop the HDPE geogrid mass.

Figure 13. illustrates the seat cast directly atop the geogrid mass with no need for piling.

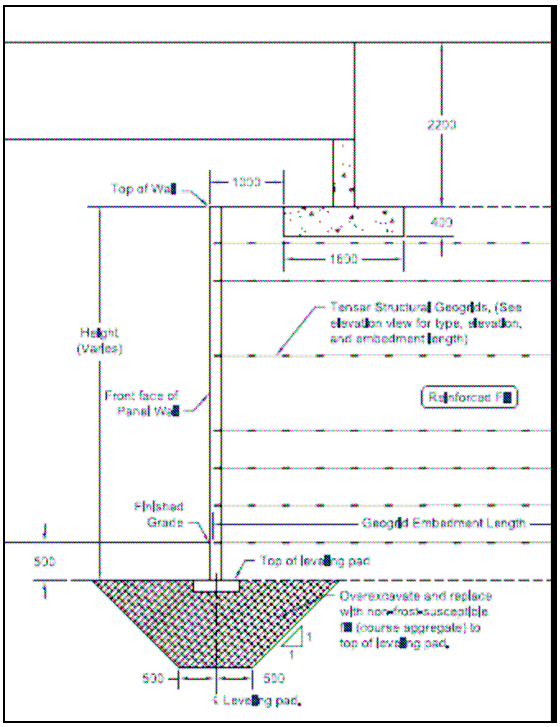

Figure 13. Abutment Seat is cast directly on HDPE Mass

4.8 Bridge Seat atop HDPE Tensar Geogrid

Figure 14 shows the concrete pour in progress at the West Abutment. The rock outcrop is noted immediately in the background.

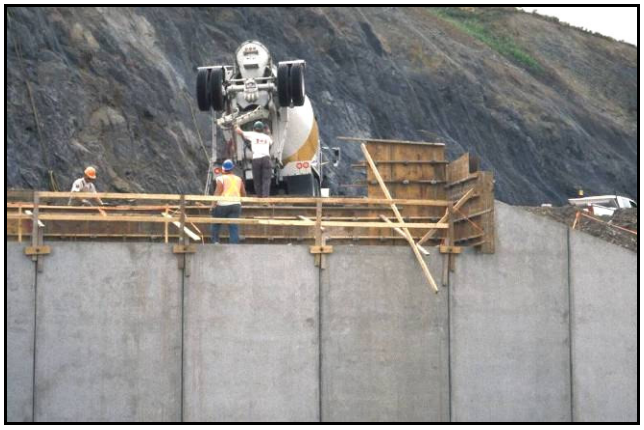

Figure 14. Casting of Bridge Seat atop HDPE Mass

5 INSTALLING HDPE REINFORCEMENT

5.1 Quick Connect Bodkin Bar

The attachment of panel grid-tabs to engage the primary soil reinforcement grid is easily accomplished with no special tools required. As illustrated in Figure 15. the bodkin bar is simply slipped through both ends of geogrid to make a positive connection.

5.2 Tensioning of Geogrid

Figure 16. illustrates how the geogrid is pulled taut at the tail-end to remove the slack over the geogrid length. This tensions the bodkin connection to ensure a positive connection and load transfer. This positive connection coupled with the inherent junction strength of the HDPE geogrid within a compacted engineered mass enables the load transfer mechanism to take effect.

Tension on the grid is applied manually with a grid-rake. The excavator end-dumps granular backfill onto the grid, so tension in the grid is maintained while compaction proceeds. This procedure is repeated from bottom to top of wall until the engineered mass is completed.

5.3 Compaction of the Geogrid Mass

As evidenced in Figure 17 compaction is achieved through the use of a vibratory roller compactor, excavator, and smaller plate tamper immediately behind the wall.

Backfill was placed in maximum lift thicknesses of 250 mm where compacted by riding roller. For fills placed within 1 metre of the back of wall and compacted by plate tamper the lift thickness was reduced to 150 mm.

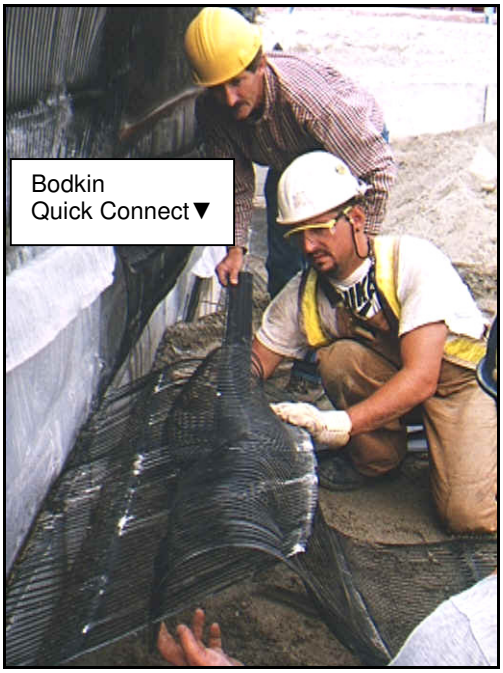

Figure 15. Grid Tab Connection to Primary Soil Reinforcement Grid

Figure 16. Use of a grid rake for tensioning of geogrid

The minimum compactive effort required was 95% of standard proctor dry density. Full height panel construction with HDPE geogrid allows the contractor to have un-impeded access to his entire backfill operation. This allows the backfill operation to be entirely independent of panel placement.

Figure 17. Vibratory riding roller for compaction

Figure 18. East Abutment & Wings Erected in 10 hours

6 HDPE AND FULL HEIGHT ADVANTAGES

6.1 HDPE Geogrid where De-icing Salts are present

As cited in AASHTO, impermeable liners are often suggested where conventional steel soil reinforcement is utilized and de-icing salts are present, as are used in many Canadian cities. For this reason backfill used in the presence of steel reinforcement should be tested for pH, Resistivity, Chlorides, and Sulphates.

By comparison backfill used in the presence of HDPE soil reinforcements requires testing of pH alone, and does not require testing of Resistivity, Chlorides, or Sulphates.

The choice of HDPE geogrid allows the client to source a wider range of backfill options, since the backfill parameters for HDPE geogrid are less restrictive as cited above.

6.2 High Panel Erection Rates

Figure 18 illustrates the East Abutment and associated wing walls being erected as noted below:

1 panel set every 12 to 18 minutes.
32 panels set in 10 hours or 445 m2 of facing.

7 CONCLUSION

The Full Height ARES system provided by Tensar and its distributor, Nilex Inc., has been successfully in service for 10 years at the time of this paper.

It has demonstrated excellent performance for a true abutment supporting application on HDPE geogrid. With varying foundation conditions and settlements up to 1.4% ARES has performed exceedingly well, and consistent with the performance of any other modular precast system on the marketplace today

At the time of this paper the writer Mr. Dan MacDonald, contacted the owner's yearly bridge inspection engineer, Watson Engineering of Kamloops. Andrew Watson indicated to the writer that they have not observed any maintenance concerns with respect to this structure over its current 10 year service life to date.

Interestingly it was pointed out to the writer that this structure is also known locally as "Mission Flats Road Overpass" by the local community in Kamloops.

ACKNOWLEDGEMENTS

The writers would like to acknowledge the following individuals who contributed to the overall success of this project. The City of Kamloops and the general contractor BA Blacktop for selecting the ARES wall system as their preferred wall system on this project.

Stanley Consulting Group Ltd., of Kamloops, BC as the owner's engineering representative for the structure. The construction and site expertise provided by Bob Miller of Tensar International in the erection of the ARES panels.

The concrete precast company (formerly Western Concrete Products) for the ingenuity in precast fabrication and erection services.

Andrew Waston of Watson Engineering of Kamloops for his current inspection comments on the structure to date.

Terra Engineering Ltd of Kamloops for the detailed summary construction report they prepared on this project, invaluable in writing the paper 10 years later.

Metz & Associates, Inc Structural Engineers Orlando, Florida for their assistance on precast design. The Nilex Inc. employees Ian Wilson & Gurch Sekhon who were involved in the initial phase of the project.

Appendix 5.1.2.4

Case Study County Road 124 Overpass project

ARES® MSE Retaining Wall System

Back to main document

ARES®
RETAINING WALL SYSTEMS

County Road 124 Overpass

Delaware County, Ohio

APPLICATION: The Ohio Department of Transportation (ODOT) wanted to bridge the heavily traveled County Road 124 over the existing CSX railway and was looking for an economical solution for the proposed retaining walls.

THE CHALLENGE: Tight budgets and limited backfill options presented challenges to the success of the project. Because the type of aggregate that is required for steel reinforcement was not locally available and would have to be hauled in from a further distance, additional transportation and material charges would be incurred.

A specified plain smooth form finish for the MSE retaining wall panels left little room for error caused by panel damage or misalignment.

SITE CONDITIONS: A tight site envelope was created since the overpass was being constructed directly adjacent to the existing road and in front of an active railroad. Also, close proximity of utilities associated with the CSX railroad required the contractor to alter the construction sequence.

ALTERNATIVE SOLUTIONS: While other MSE walls were considered for the job, Tensar offered the only MSE wall system with non-metallic soil reinforcing elements.

THE SOLUTION: The ARES Retaining Wall System was specified as an approved MSE wall solution, and was ultimately chosen by Trucco Construction Co. Inc. for use on the project. Many factors went into the decision to install the ARES walls, but the overwhelming advantage was the ability to use locally available backfill material with the extremely inert polymer geogrids, providing a cost savings for the overall project.

"By utilizing the Tensar ARES Retaining Wall System, we were able to take advantage of backfill material from a local source," said Randy King, vice president of Trucco Construction. "The ability to use this local material enabled us to be more competitive at bid time. We trucked in approximately 100,000 tons of granular backfill, which equated to around 5,000 loads," said King. "Given this quantity, utilizing less trucking was definitely a key component to the success of the project."

PROJECT HIGHLIGHTS

Project:
County Road 124 Overpass

Location:
Delaware County, Ohio

Installation:
Fall 2010

Product/System:
Tensar® UX Geogrid
ARES® Retaining Wall Systems

Quantity:
45,000 Sq ft of ARES Panel Walls

Owner/Developer:
Ohio Department of Transportation

Design Engineer:
American Consulting, Inc.

General Contractor:
Trucco Construction Co., Inc.

Materials Supplier:
Geogrid: Tensar International
Precast panels: United Precast

Two retaining walls were designed for the site totaling 45,000 square feet, with maximum heights reaching 40 feet and combined lengths of approximately 1,600 linear feet. Among the challenges for the project were double rows of tightly spaced piles supporting the bridge abutments, and specified plain smooth form finish for the panels. Smooth panels can be a challenge for any MSE retaining wall mainly because there is so little room for error aesthetically. The panels can be damaged easily, and the slightest misalignment, even if the wall remains within tolerance, is more visually apparent.

ARES Systems, utilizing Tensar Uniaxial Geogird reinforcement, can be used in a wide variety of backfill materials, including pH extremes, sulfates and where resistivity is an issue. This advantage allows for use of locally available materials to save on construction costs.

The onsite assistance provided by Tensar proved to be a valuable aspect in the installation process.

"A line item was set up in the proposal which enabled Tensar to send a technical representative, Jeremiah Riggio, to the site for five days," said King. "This created a team atmosphere in which Trucco Construction, along with ODOT, could work closely with Tensar to ensure that the walls were being constructed per the plan requirements, as well as the manufacturer's specifications. It resulted in a smooth process for all parties."

"Trucco's overall professionalism and attention to detail during every stage of construction was impressive," Riggio said, "and resulted in one of the best ARES installations I've seen."

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.

Double rows of tightly spaced piles support the abutments for the Delaware County Road 124 bridge over a CSX railway line.

For more information on ARES Systems or other Tensar Systems, call 800-TENSAR-1, email info@tensarcorp.com or visit www.tensarcorp.com

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