REPORT OF REVIEW THE REINFORCED EARTH® MECHANICALLY STABILIZED EARTH WITH PRECAST CONCRETE FACING PANELS AND INEXTENSIBLE SOIL REINFORCEMENTS

October 2022

HIGHWAY INNOVATIONS, DEVELOPMENTS, ENHANCEMENTS AND ADVANCEMENTS (IDEA)

The Reinforced Earth® MSE Precast Concrete Facing and Inextensible Reinforcement 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 Reinforced Earth® Precast Concrete Facing and Inextensible Reinforcement 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 Reinforced Earth® Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System submittal for projects where the Reinforced Earth® (RECo) Retaining Wall System is proposed.

Submittal Checklist

The checklist used from the IDEA protocol for this evaluation is C4 – Initial Technical Evaluation Checklist for Concrete Panel Paired with Inextensible Reinforcement. This is the first evaluation of the Reinforced Earth® Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System by either the IDEA, or the predecessor HITEC, evaluation programs.

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 Reinforced Earth® Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System no information has been designated by the applicant as confidential.

System Description

Components

The major components of the Reinforced Earth® Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System are precast concrete facing panels, inextensible steel soil reinforcements, and select granular backfill. Other standard components include a concrete leveling pad, EPDM bearing pads, shims, panel joint geotextile, adhesive, tie-strips and bolt sets for connecting soil reinforcement to panel tie strip connections. The precast concrete facing that has a minimum thickness of 5 ½ inches. Standard precast panel shapes include cruciform, square, hexagonal, and rectangular. The soil reinforcing consists of either discrete steel ribbed strips (i.e., High Adherence (HA) Strip) or a ladder strip (i.e., High Adherence (HA) Ladder) connected to the facing. The connector is embedded into the panel, has dual steel plates protruding from the back of the panel, and facilitates a double-shear type bolted connection to the discrete steel soil reinforcement strip/ladder.

System History

The Reinforced Earth system was developed during the 1950s and patented in 1963 by the inventor of modern mechanically stabilized earth, Henri Vidal. He introduced the system to the United States in 1971. Continued evolution and improvement of the facing, soil reinforcements, and other components have taken place. The modern system with details and components as described in this document can be dated to 1979.

The first structure was built in 1971 for Caltrans and used a steel facing with flat steel reinforcing strips bolted to the back face. The cruciform-shape concrete facing was used on the projects immediately following, while the steel facing was phased out in the US during the 1970s.

The ribbed reinforcing strips were introduced in 1978. Further research and development led to optimizing and adjustments of the strip dimensions and grade of steel to those presented in this document. Ladder soil reinforcements were introduced in the 1980's. HA ladders as described in this document were introduced in 2000. EPDM Bearing Pads were introduced in 1984.

The oldest structure in the U.S using the ribbed strip reinforcement was constructed in 1978 on I-66 at Glebe Road, Arlington VA. The oldest structure using the HA Ladder was constructed in 2000 on I-65, Jeffersonville, IN. The tallest structure was constructed in 2005 for the Seattle-Tacoma Airport and consists of three tiers with a total height of 134.5 feet. Ribbed steel strips were used as the reinforcement. The tallest single tier wall was constructed in 2018 as part of SoFi Stadium. The wall is 110 feet tall and uses ribbed steel strips.

System Properties

The following properties are reported by the applicant for Reinforced Earth® Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System.

Soil Reinforcement

The RECo HA ribbed steel strip soil reinforcements are discrete, inextensible steel strips with transverse ribs formed in the top and bottom surface. The standard strips are 2 inches (50 mm) wide and 0.157 inches (4 mm) thick. They are also available in 2 inch (50 mm) wide and 0.24 inch (6 mm) thick strips. The HA ribbed strip is manufactured with low-carbon steel in conformance with ASTM

A572 and are hot rolled from bars into the strips. This soil reinforcement is protected from corrosion through the application of a zinc coating, applied by hot-dip galvanizing in conformance with ASTM A123.

The RECo HA ladders are made up of two longitudinal W10 wires, welded to a connector plate on one end, spaced 2-inches apart. Four-inch long transverse wires are welded to the longitudinal wires and spaced every 6 inches over the length of the ladder. HA ladder Reinforcement is cold drawn W10 steel wire milled to the required shape and dimensions. This soil reinforcement is protected from corrosion through the application of a zinc coating, applied by hot-dip galvanizing in conformance with ASTM A123.

Soil Reinforcement-Facing Panel Connection Capacity

Both strips and ladders are connected to the facing with a double shear type connection using a single bolt to a tie strip protruding from the back face of the panel. The tie strip consists of two parallel plates, with aligned 9/16-inch holes. The strips and ladders are provided with a 9/16-inch diameter hole on one end and fit in between the two plates of the tie strip. The bolt is ½-inch in diameter. The tie strip and bolt are protected from corrosion through the application of a zinc coating, applied by hot-dip galvanizing in conformance with ASTM A123.

Soil Reinforcement Design Tensile Strength

The design tensile strength of the HA ribbed strip and HA ladder are controlled by the tensile strength of the reinforcement. The HA ribbed strip soil reinforcement has tensile strengths of F_u = 80 ksi and F_y = 65 ksi. The HA ladder soil reinforcement has tensile strengths of F_u = 75 ksi and F_y = 65 ksi. The connection between the reinforcement and facing panel has been designed to have a higher tensile resistance then the soil reinforcement.

Pullout Design Parameters

Based on the data available for AASHTO specified backfill gradations, RECo recommends the following relationship for determining f* for RECo HA ribbed strips for granular backfills meeting AASHTO requirements for gradation, shear strength and durability:

$$f^* = 1.5 + \ln(C_u) < 4.50 \text{ @ Overburden press up to 2,500 psf}$$

$$f^* = \tan\varphi \text{ @ Overburden pressure greater than 2,500 psf (i.e., depth of 20 feet)}$$

where: C_u = coefficient of uniformity = D_{60}/D_{10}

φ = effective friction angle of reinforced fill

Retaining Wall Design

The design methodology of a Reinforced Earth wall considers both the external and internal stability of the structure. Internal stability is determined using the Coherent Gravity Method (CGM). The CGM is provided in Section 2.1.2 of the attached final submittal and in detail in the AASHTO LRFD 9th Edition (2020).

The design examples in this submittal were performed using both MSEW+ and a proprietary software developed by RECo. The scope of this review did not provide for a comprehensive evaluation of the RECo software. However, the applicant states that the design of the RECo MSE Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System is consistent with current AASHTO specifications. This assertion is supported by the results of the analyses of example problems in the submittal, performed with both the proprietary software and the computer program MSEW+.

Agencies may check designs with a commercial program, with the design properties listed in the table attached to this review report.

System Innovations

This IDEA evaluation concurs with RECo that their system provides the following innovations:

- Soil Reinforcement Splay Angle - The allowable splay angle of reinforcing strips has long been designated as 15 degrees, based on early recommendations of Terre Armée (RECo's parent company) in the 1970's. However, this recommendation was based on a simple reduction (cos 15° = 0.965, or roughly a 3% reduction) and assuming all reinforcing strips in the zone of interest were splayed. Recent full-scale pullout testing has shown that individual reinforcing strips may be splayed upwards of 30 degrees without a significant loss in pullout capacity. In addition, testing of panel tie strip connections with splayed reinforcing strips show minimal reduction in capacity. For vertical obstructions, the IDEA evaluation suggests that a maximum of one splayed reinforcing strip per row of reinforcement for a nominal 5 ft wide panel and a maximum of two splayed reinforcing strips for a nominal 10 wide panel be splayed up to 30 degrees.

- Results from extensive pullout testing has been performed on both the HA Strip and the HA Ladder indicate a distinct improvement of the friction factor, F*, compared to the default values provided for steel ribbed reinforcement in the AASHTO LRFD Bridge Design Specifications and FHWA *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Vol 1* (Berg et al. 2009).

- Acute Corner Design - Using a bin-wall design for arranging the facing and soil reinforcements to accommodate an acute corner. RECo has performed numerical modeling that justifies a threshold of <60 degrees for acute corner design, rather than AASHTO recommended 70 degrees..

- Shored MSE Design – RECo often utilizes "sandwich" reinforcements for SMSE walls with aspect ratios less than 0.5 in order to control deformation and provide reinforcements against potential compound failure planes at the shoring wall and MSE fill interface.

- MSE Walls on Marginal Soils - The inherent flexibility of MSE walls allows for construction on sites with soft soils. In fact, the very first RE wall constructed in the US settled over 3 ft. Single-stage Reinforced Earth walls can be constructed on marginal foundations with the use of phased construction, surcharges, lightweight fills, ground improvement, or some combination of these methods; along with engineering evaluation of anticipated total and differential settlements across the wall structure(s).

Reviewer Comments

Following its initial review of the RECo MSE Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System submittal, the review team provided the applicant with 55 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 RECo MSE Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System submittal for projects where the RECo System is proposed.

Closing

An update technical evaluation should be performed for the RECo MSE Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System in five years (i.e., August 2027) or upon notice that a significant modification of the system has been made. For details regarding update technical evaluations and other guidance for the use of technical evaluations by transportation agencies, go to https://www.geoinstitute.org/special-projects/idea.

Summary Table MSEW+ Input Parameters for Reinforced Earth® Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System

Reinforcement	Design Life	F_y (ksi)	b (inches)	t (inches)	α	f*		Factored Tensile Resistance (kips)
						Overburden Pressure up to 2,500 psf	Overburden Pressure 2500 psf	
HA ribbed strip (4 mm)	75	65	1.97	0.157	1.00	$1.5 + \ln(C_u) < 4.50$	tan φ	9.76
HA ribbed strip (4 mm)	100	65	1.97	0.157	1.00	$1.5 + \ln(C_u) < 4.50$	tan φ	7.50
HA ribbed strip (6 mm)	75	65	1.97	0.240	1.00	$1.5 + \ln(C_u) < 4.50$	tan φ	17.01
HA ribbed strip (6 mm)	100	65	1.97	0.240	1.00	$1.5 + \ln(C_u) < 4.50$	tan φ	14.73
			Width (inches)	Diameter (inches)				
HA ladder (W10)	75	65	4.00	0.357	1.00	1.2+log (Cu) < 3.0	tan φ	5.78
HA ladder (W10)	100	65	4.00	0.357	1.00	1.2+log (Cu) < 3.0	tan φ	6.95

[1] The factored tensile resistance considers metal loss over the design life due to corrosion

IDEA

Highway Innovations, Developments, Enhancements and Advancements

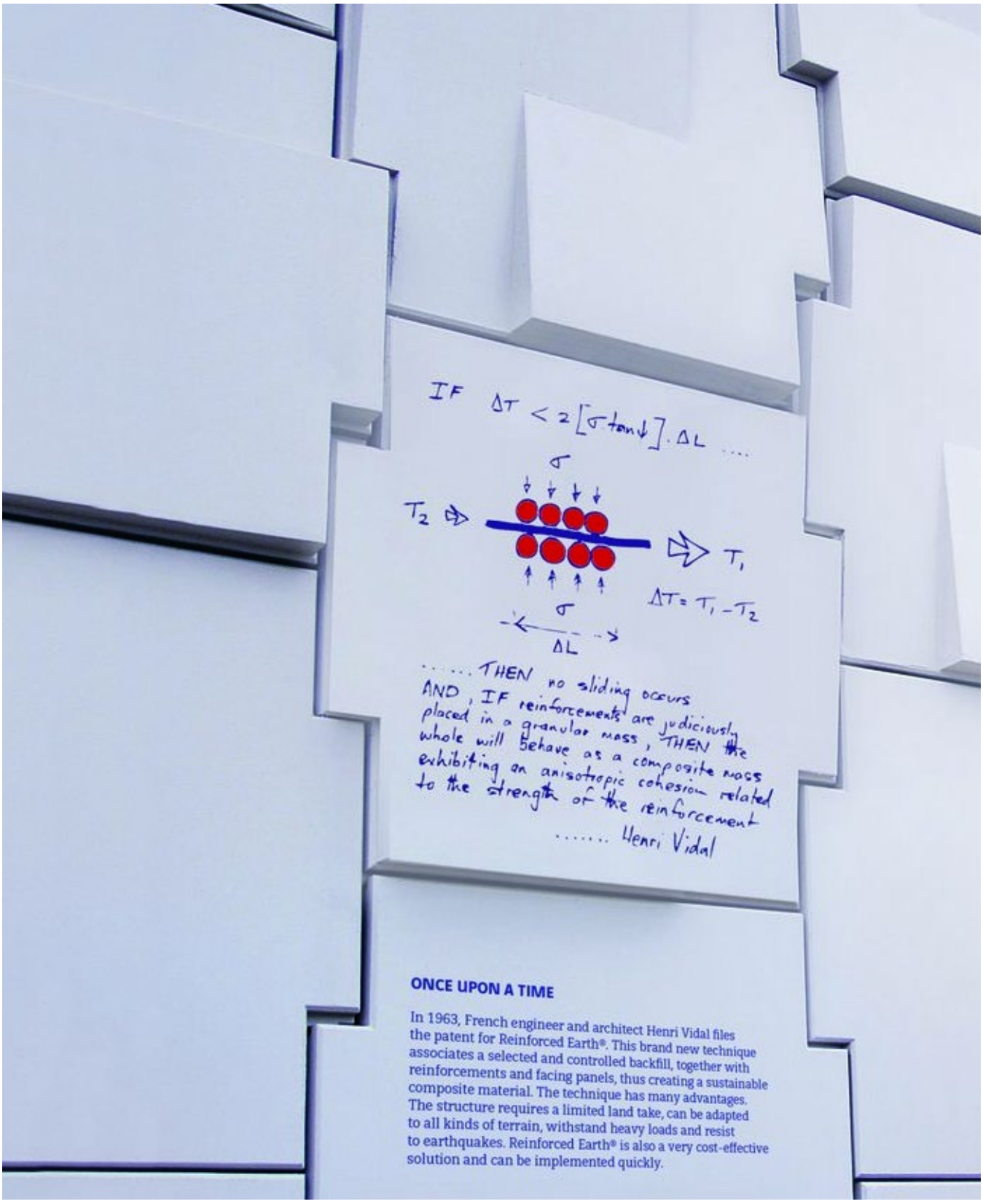
Reinforced Earth®
Mechanically Stabilized Earth with Precast Concrete Facing Panels and Inextensible Soil Reinforcements

September 30,2022

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Rev.	Date	Modification	Design by	Checked by	Approved by
3	9/30/2022	Final Comments	JZ	KPB	KPB
2	6/27/2022	Review Comments	JZ	KPB	KPB
1	2/8/2022	TOC hyperlinks	JZ	KPB	KPB
0	1/14/2022		JZ	KPB	KPB

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Forward

The following information in this document is organized to adhere to the checklist in the IDEA Appendix C4 titled, "Initial Technical Evaluation Checklist for Precast Concrete Panel Paired with Inextensible Reinforcement."

This submittal specifically covers The Reinforced Earth Company's (RECo) Mechanically Stabilized Earth (MSE) retaining wall system, trademarked as Reinforced Earth®. This document will refer to the system as "Reinforced Earth" or "RE."

INTRODUCTION

The Reinforced Earth® MSE Wall System is composed of the following components:

1. Reinforced concrete facing panels
    a. Cruciform Panels (Nominally 4.92'x4.92')
    b. Square Panels (Nominally 4.92'x4.92' or 5'x5')
    c. Rectangular Panels (Nominally 4.92' x9.84' or 5'x10')
    d. Hexagonal Panels (Nominally 5'x5')
2. Galvanized steel tie strips with bolt sets
    a. 10 gage standard tie strip thickness
    b. 7 gage optional tie strip thickness
    c. ½" diameter x 1 ¼" long A449 bolt with nut and washer
3. Galvanized steel "HA" reinforcing strips or "HA" ladders
    a. High Adherence (HA) Reinforcing Strips: 50mm wide x 4mm or 6mm thick
    b. High Adherence (HA) Ladders: W10 Wire size (longitudinal and transverse), with 10 gage connector plate
4. Facing panel joint material (geotextile and bearing pads)

1.0 ERS Components

The major components of the Reinforced Earth wall are precast concrete facing panels, inextensible steel soil reinforcements, and select granular backfill. Other standard components include a concrete leveling pad, EPDM bearing pads, shims, panel joint geotextile, adhesive, tie-strips and bolt sets for connecting soil reinforcement to panel tie strip connections

Traffic Barrier, coping, slip joints, corner elements, and added drainage components may also be included depending on the requirements and criteria of the project.

All of these components will be described in the appropriate section of this document.

1.1 Facing Unit

Reinforced Earth® concrete panel facing units come in four basic shapes – Cruciform, Square, Hexagonal and Rectangular. Each has a typical structural thickness of 5.5 inches to meet AASHTO requirements,

however thinner panels are feasible, and we have successfully designed, fabricated and supplied panels with 5-inch structural thickness. Each type has a standard “A” size, with varied height alternatives used along the top and bottom of wall. Slip joint facing units and corner elements are also used where required. In this document, the term “facing unit” may be interchanged with the term “panel.”

1.1.1 Facing Unit Innovations

Concrete Facing Units may be cast with low carbon concrete.

1.1.2 Facing Unit Types

The following types of standard facing units are used:

- Cruciform shaped “A” panel
- Square shaped “A” panel
- Hexagonal shaped “A” panel
- Rectangular shaped “A” panel

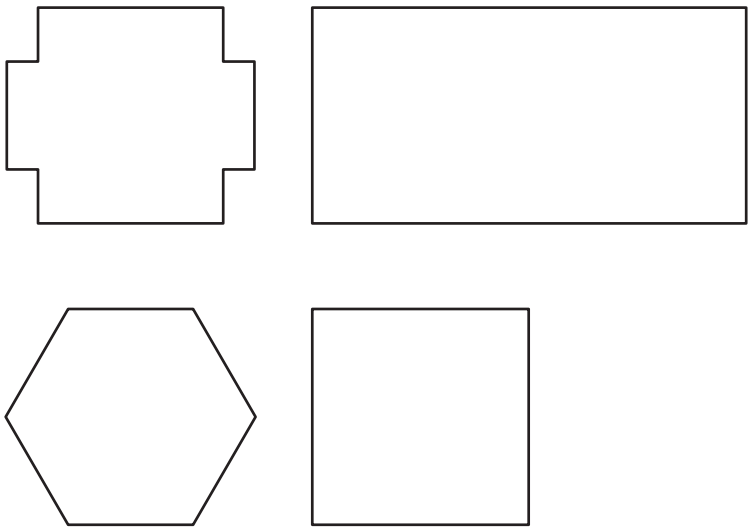


Figure 1.1.2-1: Front face view showing relative size of each standard facing unit shape.

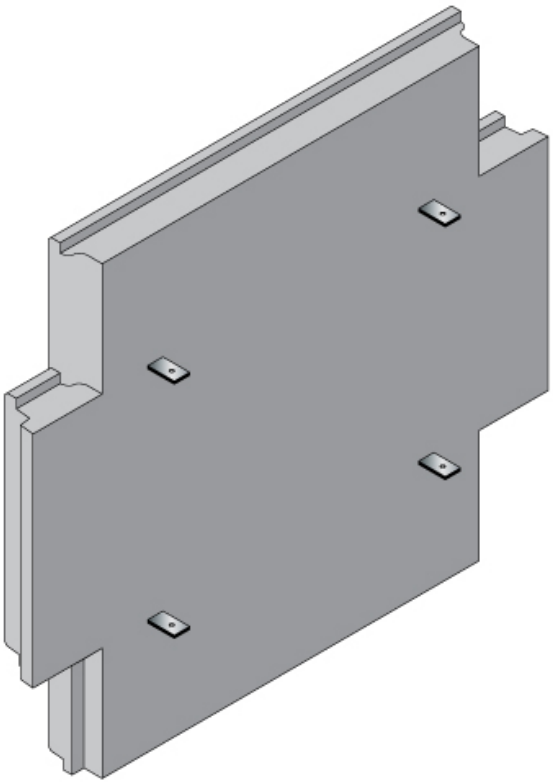


Figure 1.1.2-2: Isometric view of the back face of a cruciform shape "A" panel.

The facing panels are stacked in columns with the horizontal joints of adjacent columns staggered. Only one of the four facing panel shapes are used on a single wall, except in rare special design conditions.



Figure 1.1.2-3: Example of a typical Reinforced Earth highway wall using rectangular shape facing units. North Tarrant Express Segment 3A.

Other standard facing unit types such as slip joints and corner elements are used when needed and are discussed in **Sections 1.1.11, 1.3.6, and 3.1.3**.

1.1.3 Facing Unit Specifications

The specifications for each facing unit are included in **Appendix A1.1.3**.

1.1.4 Facing Unit Dimensions, Tolerances and Typical Steel Reinforcement Schedule

The dimensions, tolerances and typical steel reinforcement for the facing units listed above are included in **Appendix A1.1.4**.

1.1.5 Compressive Strength

Concrete for panels shall have a minimum compressive strength after 28 days of 4,000 PSI in accordance with AASHTO Class A requirements. Minimum strength will be increased if required by the specifications of a particular project. Due to the need to cycle forms daily, most precast mixes used by RECo result in 28 day strengths exceeding 5,500 psi. As such, data and design values will be presented for both 4,000 psi and 5,000 psi design strengths.

Curing time depends on the local mix design, ambient temperature, and state DOT specifications. Generally, a 12 hour cure time is adequate to reach stripping (form removal) strength and units may be

shipped once reaching 28 day required strength (and in some cases sooner, on the order of 85% of 28 day strength) if mix design has proven to adequately reach strength under normal conditions.

Generally, for smooth form finish panels, the piece may be removed from the form after 12 hours and after attaining a compressive strength of 2,000 psi.

Refer to **Appendix A1.1.5** for more detailed information.

1.1.6 Air Entrainment

The percentage of air in the precast concrete will vary depending on project specifications. In general, panels constructed in accordance with AASHTO Specification Class A (AE) require an air content range of 6 % +/- 1.5%.

1.1.7 Mix Design

The concrete mix design is developed to meet the requirements set by the Owner of the project, and varies from plant to plant depending on the owner requirements and locally available cement and aggregates. Typical Mix designs comply with AASHTO Class A or A(AE).

Per AASHTO LRFD Bridge Construction Specifications, Table 8.2.2-1:

Class of Concrete	Min. Cement Content	Max. Water / Cementitious Material Ratio	Air Content Range	Size of Course Aggregate Per AASHTO M43	Size Number per AASHTO M43	Specified Compressive Strength at 28 days
	lb /yd^3	lb per lb	%	Nominal Size		Psi
A	611	0.49	-	1.0 in. to No. 4	57	4,000
A(AE)	611	0.45	6 +/- 1.5	1.0 in. to No. 4	57	4,000

Table 1.1.7-1: AASHTO Mix Design Requirements

Additional recommendations for Facing Unit Mix Design can be found at the National Precast Concrete Association (NPCA): https://precast.org/wp-content/uploads/2014/09/Retaining-Wall-Best-Practices-Manual.pdf

1.1.8 Shear, Alignment and Bearing Devices

Shear between the facing units is not a factor in the stability of the facing. Shear in this manner is considered for segmental or large block facing, not the concrete panel facing used for the RE wall system.

Both horizontal and vertical alignment of the facing is achieved by setting panels with a typical ¾-inch open joint between units. All facing units form a shiplap or tongue and groove joint with adjacent panels on all sides of the front face.

Bearing devices are in the form of 1-inch or ¾-inch thick EPDM r bearing pads ASTM D-2000 M2AA 807, having a minimum durometer hardness of 80 (+/- 5) . Typically, two or four pads are set in notches on the top of a panel, and the next course of panels is set on top of the bearing pads. The bearing pads

ensure there is no concrete-to-concrete contact, assist in the alignment of panels during construction and over the life of the wall, and allow the wall face to accommodate fill settlements.

Bearing Pads are designed to compress to assure the facing will compress compatibly with the elastic compression of the select backfill, to allow the facing panels to be nearly as flexible as the MSE embankment itself, and to prevent concrete to concrete point loading. The nature of bearing pad compression is complex and is a function of bearing pad geometry and thickness, backfill type and compression during compaction, and any settlement of the facing panels due to compression of the foundation soils beneath the wall. There is a potential for minimal compression of bearing pads when conditions are optimal, and more significant (i.e., approaching the design load and final design thickness) when foundation conditions are poor and/or MSE wall fiil is sand.

RECo has developed various bearing pad geometries for our various facing systems and adjust the size and quantity of bearing pads depending on the wall height and loading conditions.

See **Appendix A1.1.8** for bearing pad calculations and details.

There are no additional permanent devices used for alignment. However, there are temporary wedges, clamps and braces and construction methods as outlined in RECo's construction manual. Refer to **Section 3.0**.

1.1.9 Migration of Fill Prevention

Migration of fill through the facing is prevented with the use of a geotextile spanning all joints between facing units. Adhesive is used to adhere the geotextile to the back face of concrete facing units. The shiplap or tongue and groove joints on the facing units prevent UV light from degrading the geotextile.



Figure 1.1.9-1: Geotextile at Panel Joints

Generally, a 12" or 18" wide AASHTO Class 3 filtration non-woven geotextile is utilized for joints. In submerged conditions where drawdown conditions are present, an AASHTO Class 1 woven

monofilament geotextile is specified. Geotextiles utilized at joints are modified to comply with specific owner specifications when required or where particular conditions of the structure will require it.

Details and Specifications for typical geotextile can be found in **Appendix A1.1.9**.

1.1.10 Aesthetics



Figure 1.1.10-1: An example of stone pattern.

There is a very wide range of aesthetic options for precast concrete facing panels, including plain smooth concrete, repeating patterns, textures, and replicated materials such as stone, brick and wood. Detailed custom designs can also be cast into the concrete. The architectural finish is cast using a form liner that increases the thickness of the panel beyond the structural thickness. The front face of the facing units can accommodate varying reliefs of a fraction of an inch to several inches. The concrete mix can include integral color or the wall can be stained after installation. It is RECo's opinion that stain provides a more uniform color than integral color, as concrete color will vary over a typical production period.

Refer to **Appendix A1.1.10** for photos, details, and a marketing brochure.

1.1.11 Curved Alignment Limitations



Figure 1.1.11-1: A pedestrian ramp wall on a curve.

The curvature of a Reinforced Earth wall is formed by a series of short chords, depending on the width of the panels. If no special modifications to the facing are made, the RE wall system has the following minimum radius for inside and outside curvature depending on facing unit type:

- Cruciform shaped panels: 50-ft minimum radius
- Square shaped panels: 50-ft minimum radius
- Rectangular shaped panels: 100-ft minimum radius
- Hexagonal shaped panels: 100 ft minimum radius

The minimum radius is typically limited by the allowable bend between two panels, which is controlled by the recommended tolerance of the open vertical joint size and shiplap or tongue and grove geometry.

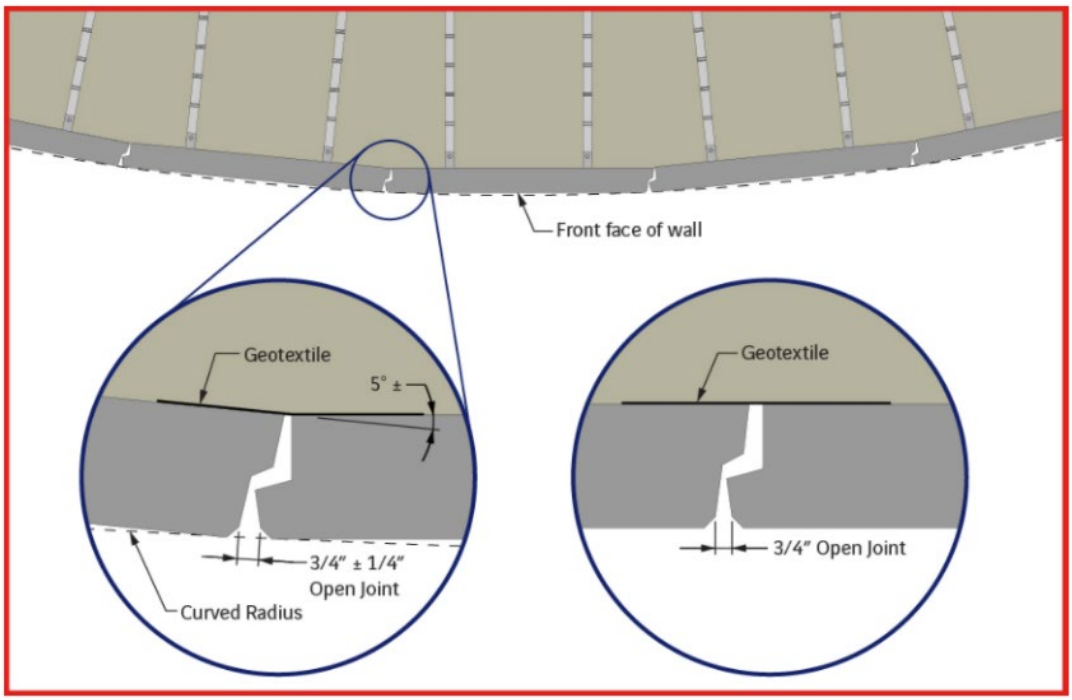


Figure 1.1.11-2: Partial plan of 5-ft x 5-ft panels on a 50-ft radius vs. a straight line

No modifications are required to be made to the facing units to achieve the minimum curves. To form curves with smaller radii, modifications to the facing units can be made, such as mitered and truncated ends. RECo has successfully designed and supplied panels as narrow as 2 ft wide for curved alignments and radii as small as 20 ft. Additional information on curved alignments can be found on our website: https://reinforcedearth.com/blog/curved-retaining-walls/

The typical method of creating a corner in the wall is by using one of RECo's standard corner element units. The units are designed to accept a facing panel on either side, providing overlap on the back face that functions as a shiplap joint. Tie strips are cast into the corner elements, and soil reinforcements are attached in the field. The four corner element types each accommodate a range of angles:

- Type 1: 63° to 89°
- Type 2: 90° to 117°
- Type 3: 117° to 162°
- Type 4: 30° to 62°

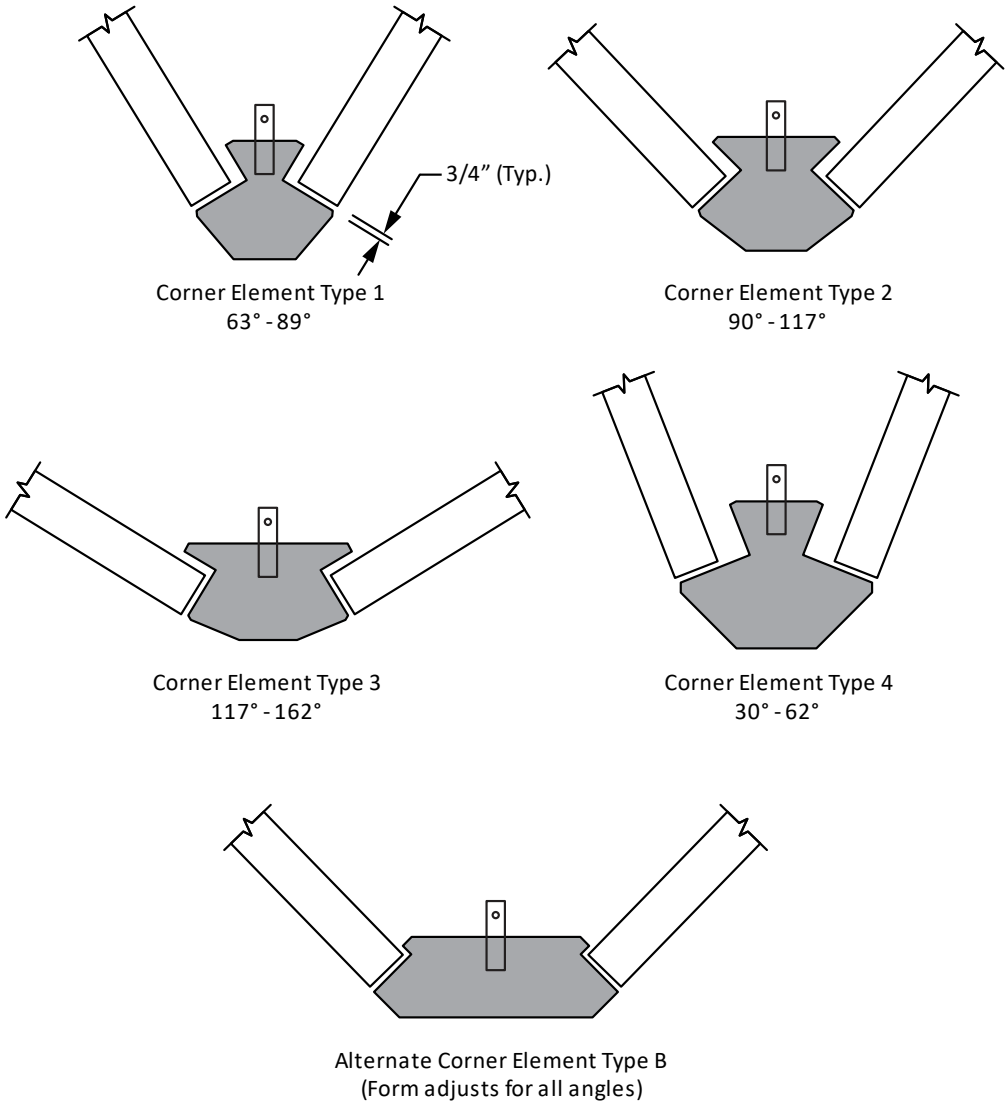


Figure 1.1.11-3: Plan view of standard corner elements

For inside corners, angles smaller than 60 degrees require a special bin-wall design method, (see **Appendix A2.1**). Since RECo uses discrete, inextensible reinforcements that may be splayed upwards of 30 degrees (with proper analysis), RECo can design walls with interior angles of 60 degrees or greater without the need to tie across and create a tied bin structure. According to numerical analysis by RECo, interior angles as low as 60 degrees can be constructed with skewed/splayed reinforcement rather than a tied across bin design.

Reinforced Earth walls with acute angles as small as 20 degrees have been successfully constructed, however in some cases acute angles less than 30 degrees may not be feasible depending on factors such as wall height, loading, and obstructions in the backfill.

Outside corners generally have no limitations on the angle that can be achieved. Outside corners are typically created by reversing a corner element or butting the end of a facing panel to the front face of an adjacent panel. Refer to **Appendix A1.1.11** for standard corner details.

1.2 Inextensible Reinforcement

Reinforced Earth walls are designed using discrete steel strip soil reinforcements or discrete steel ladder soil reinforcements. Both types are inextensible and have advantages depending on a variety of factors for a given wall.



Figure 1.2-1: Reinforced Earth High Adherence (HA) Strip.

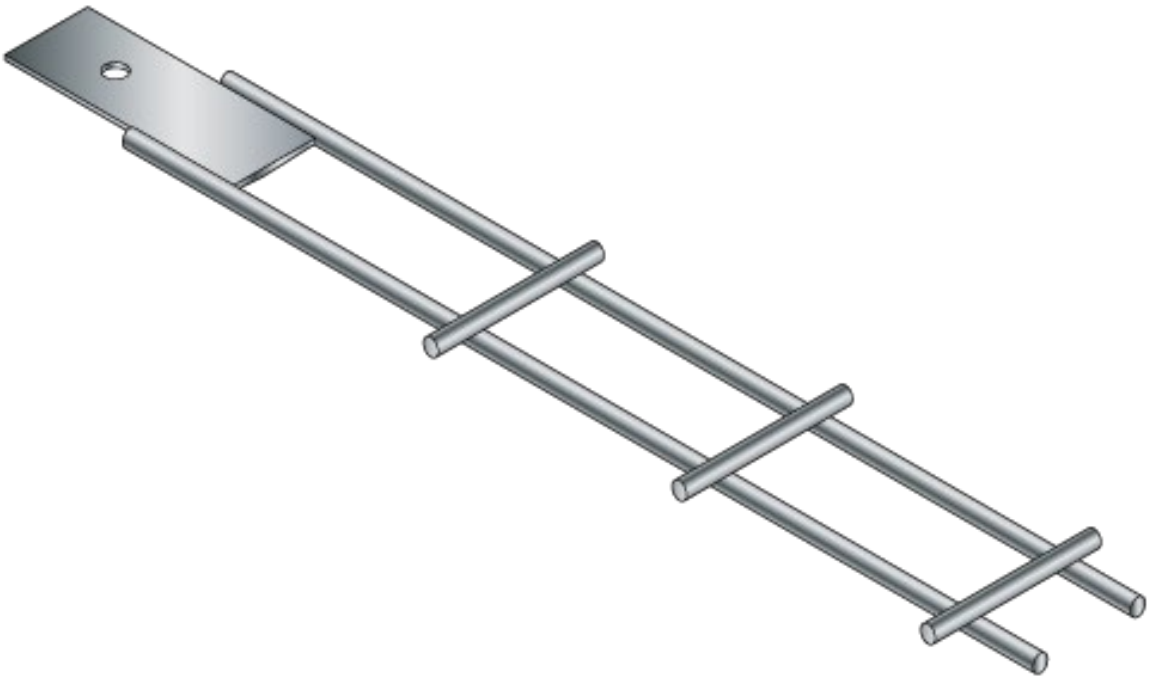


Figure 1.2-2: Reinforced Earth High Adherence (HA) Ladder.

1.2.1 Innovations

There are two specific innovations related to RECo's inextensible steel reinforcement:

- The apparent coefficient of friction between the reinforcement and the soil (f*). RECo has continually researched the interaction between soil reinforcements and backfill and improved upon recommendations for the optimal f* value depending on the project criteria. Recommended f* values, based upon research specific to these two RE reinforcements, are greater than the default values used for generic steel strip reinforcements.
    - See **Section 1.2.16**
    - Refer to **Appendix A1.2.1** for RECo's Tech Bulletin 2019-01, which describes the most up to date practice for the use of f* for Ribbed Steel Strips.
    - Refer to **Appendix A1.2.1** for MSE Pullout Testing for RECo HA Ladder and HA Strip Reinforcements, Texas Tech University, December 21, 2011

- The allowable splay angle of reinforcing strips has long been designated as 15 degrees, based on early recommendations of Terre Armée (RECo's parent company) in the 1970's. However, this recommendation was based on a simple reduction (cos 15° = 0.965, or roughly a 3% reduction) and assuming all reinforcing strips in the zone of interest were splayed. Recent full-scale pullout testing has shown that individual reinforcing strips may be splayed upwards of 30 degrees without a significant loss in pullout capacity. In addition, testing of panel tie strip connections with splayed reinforcing strips show minimal reduction in capacity, with an ultimate capacity exceeding the factored design capacity.
    - See **Appendix A1.2.1** for research paper by Jayawickrama, et al "Effect of Skewing and Splaying on Pullout Capacity of Steel MSE Reinforcement"
    - See **Appendix A1.2.15** for report by SGI regarding Pullout Testing of RECo Tie Strip Connectors.
- Based on the research by RECo and Jayawickrama, reinforcing strips may be splayed up to 30 degrees. See **Sections 1.2.15** and **2.1.3** for additional information.

1.2.2 Types of Soil Reinforcements

There are two types of soil reinforcements commonly used for RE walls:

- High-Adherence (HA) Ribbed Strips, referred to in this document as "strips".
    - Strips are 50mm wide by 4 (or 6) mm thick in section. Elevated transverse ribs are spaced along the top and bottom over the full length of the strip.
- High-Adherence Ladders, referred to in this document as "ladders".
    - Ladders are made up of two longitudinal W10 wires, welded to a connector plate on one end, spaced 2-inches apart. Four-inch long transverse wires are welded to the longitudinal wires and evenly spaced over the length of the ladder.

Refer to **Appendix A1.2.2** for dimensions of each type of reinforcement.

1.2.3 Physical Property Specifications

High Adherence (HA) Ribbed Reinforcement Strips shall be hot rolled from bars to the required shape and dimensions. Their physical and mechanical properties shall conform to ASTM A572/A572M-15, Table 1, Gr. 65, Tables 2 and 3 'Chemical Requirements / Alloy Content', Type 1, 2 or 3 (discretion of the manufacturer), and Table 4 minimum yield 65ksi, tensile 80ksi.

HA Ladder Reinforcement, shall be cold drawn W10 steel wire milled to the required shape and dimensions. The physical and mechanical properties shall conform to ASTM A1064/A1064M-17, (AASHTO M 32M/M 32) plain wire Gr. 65, Minimum yield 65ksi, tensile 75ksi, longitudinal and transverse wires shall be of equal size. Welded into the finished ladder configuration in accordance with Section 8.3.1, minimum shear strength of welds shall be 35ksi.

Connector plate material shall comply with ASTM A1011/A1011M-15, SS Type 1, Gr.50 or HSLAS Class 1, Gr. 50, minimum yield 50ksi, tension 65ksi. Welded to the ladder configuration by a certified welder (as detailed), type E70S-3 electrode, minimum yield 67ksi, tensile 79.2ksi, welding shall comply with provisions listed within the latest edition of ANSI/AWS D1.1, A5.18.

HA Strips: Yield strength (F_y) = 65 ksi; Ultimate strength (F_u) = 80 ksi

HA Ladders: Yield strength (F_y) = 65 ksi; Ultimate strength (F_u) = 75 ksi

1.2.4 Corrosion Protection

Hot-dipped galvanization is the most common corrosion protection for strips and ladders. The minimum galvanization thickness for permanent structures (typically 75 year design life) is 2 oz. per square foot (3.4 mils) on all[1] surfaces.

For HA Strips, the hot-dipped galvanizing process follows ASTM A-123 specification, Section 6.1, Table 2, Coating Grade 85, (mass) 2.0 oz/ft², (thickness) 86µm, 3.4 mils. (AASHTO M 111M/M 111).

For HA Ladders Hot-Dip Galvanized coating, shall be applied after the ladder and connector plate are fabricated and shall conform to the minimum requirements of ASTM A641/A641M-09a (Reapproved 2014), Table 1, Class B, minimum weight (mass) 2 oz/ft² or (thickness) 3.4 mils., 86µm, (AASHTO M 111M/M 111)[1].

1.2.5 Sacrificial Steel Thickness

Per AASHTO 11.10.6.4.2a – Steel Reinforcements:

The structural design of steel soil reinforcements and connections shall incorporate a thickness, E_c , as follows:

$E_c = E_n - E_s$ (11.10.6.4.2a-1)

Where:

E_c = thickness (mil.) of metal reinforcement at end of service life as shown in Figure 11.10.6.4.1-1.

E_n = nominal thickness (mil.) of steel reinforcement at construction.

E_s = sacrificial thickness (mil.) of metal expected to be lost by uniform corrosion during service life of structure.

For structural design, sacrificial thicknesses shall be computed for each exposed surface as follows, assuming that the soil backfill used is nonaggressive:

Loss of galvanizing = 0.58 mil./year (15 µm) for first two years

0.16 mil./year (4 µm) for subsequent years

Loss of carbon steel = 0.47 mil./year (12 µm) after zinc depletion

These metal loss rates shall be considered applicable to hot-dip galvanized steel placed within wall fill that meets AASHTO criteria for corrosivity, and not applicable to black steel or steel coated by any other process. Other carbon steel loss rates shall be used for black steel and for steel coated by any other material or process, or with fills that do not meet AASHTO criteria for corrosivity.

Soils shall typically be considered nonaggressive relative to corrosion of galvanized steel if they meet the following criteria:

- pH = 5 to 10
- Resistivity [3]3000 ohm-cm
- Chlorides ≤100 ppm
- Sulfates ≤200 ppm
- Organic Content ≤1 percent

The sacrificial steel thickness is based on the metal loss rates specified by AASHTO. For a 75-year design life, the sacrificial steel thickness is 0.708mm (0.0278 in.) per side. For a 100-year design life, the sacrificial steel thickness is 1.008mm (0.0397 in.) per side.

The sacrificial steel thickness is based on the metal loss rates specified by AASHTO. For a 75-year design life, the sacrificial steel reduction in diameter is 1.416mm For a 100-year design life, the sacrificial steel reduction in diameter is 2.016mm.

Reinforcement	Design Life	F_y (ksi)	b (inches)	t (inches) (E_n)	Sacrificial thickness (E_s) (2 sides) (in.)	Factored Tensile Resistance (kips) [1]
HA ribbed strip (4 mm)	75	65	1.969	0.1575	0.0557	9.76
HA ribbed strip (4 mm)	100	65	1.969	0.1575	0.0794	7.50
HA ribbed strip (6 mm)	75	65	1.969	0.236	0.0557	17.01
HA ribbed strip (6 mm)	100	65	1.969	0.236	0.0794	14.73

HA Ladders			Width (inches)	Diameter (inches)	Sacrificial thickness (E_s) (reduction in diameter) in.	
HA ladder (W10)	75	65	4.00	0.357	0.111	5.78
HA ladder (W10)	100	65	4.00	0.357	0.159	6.95

Table 1.2.5-1: Sacrificial Thickness and Factored Tensile Resistance

[1] The factored tensile resistance considers metal loss over the design life due to corrosion

1.2.6 Corrosion Test Results

For projects designed in accordance with AASHTO specifications, the metal loss rates specified in 11.10.6.4.2.a (noted in **Section 1.2.5** above) are conservative for backfills meeting the electrochemical criteria as specified by AASHTO (noted in **Section 1.2.5** above).

The Reinforced Earth Company has performed and/or participated in numerous corrosion test studies over the past 50 years. Following is a brief bibliography of metal loss research:

AMSE: Reduced Zinc Loss Rate for Design of MSE Structures (https://www.amsewalls.org/wp-content/uploads/2013/06/AMSE_Metal_Loss_White_Paper.pdf)

Anderson, Gladstone, Fishman, Withiam "Durability of Reinforcing Strips 30+ Years Experience with MSE", 2006. TRB 2006 Annual Meeting

Earth Reinforcement Testing / McMahon&Mann "Condition and Durability Study for RECo Reinforcements and Tie-Strip Connections Samples" Parts 1-5 (2011-2021). See **Appendix A1.2.6** for an excerpt from these reports.

American GeoTechnics. "Corrosion of Buried Steel Reinforcements, Glenns Ferry MSE Wall System" December 23, 2020.

Additional research that includes data from in service Reinforced Earth walls as well as references to dozens of technical papers on the subject can be found in the following publications:

FHWA NHI 09 087 – Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes. 2009.

Fishman K., Withiam J. NCHRP 24-28: LRFD Metal Loss and Service-Life Strength Reduction Factors for Metal-Reinforced Systems in Geotechnical Applications. NCHRP Report 675. Transportation Research Board. 2011

1.2.7 Dimensional Tolerances

The dimensional tolerances for strips and ladders are as follows. Details can be found in **Appendix A1.2.2**.

Dimensional tolerances for strips:

- Thickness: -0 mm (0 in.), +0.5 mm (1/64 in.)
- Width: ±1.5 mm (1/64 in.)
- Length: -0 mm (0 in.), +190.5 mm (7 ½ in.)

Dimensional tolerances for ladders:

- Wire lengths: ±1/2-inch
- Connector plate length: ±1/8-inch
- Connector plate width: +1/16-inch, -0".

1.2.8 Reinforcement Connection to Facing Units

Both strips and ladders are connected to the facing with a double shear type connection using a single bolt to a tie strip protruding from the back face of the panel. The tie strip consists of two parallel plates, with aligned 9/16-inch holes. The strips and ladders are provided with a 9/16-inch diameter hole on one end and fit in between the two plates of the tie strip. The bolt is ½-inch in diameter.

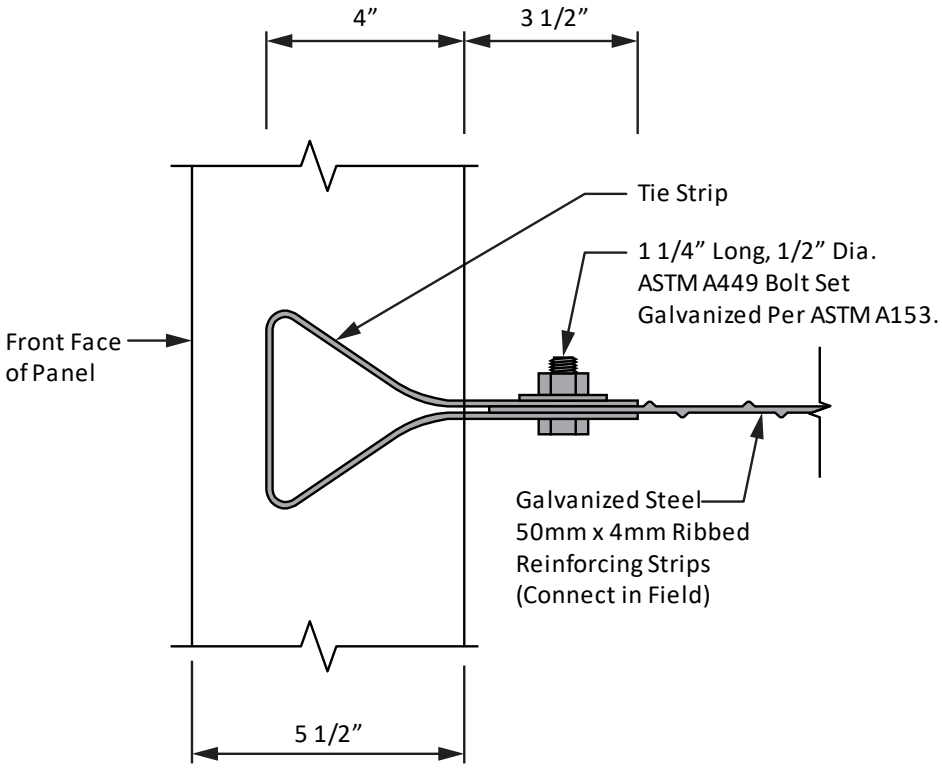


Figure 1.2.8-1: Tie Strip assembly with bolt, washer, nut and reinforcing strip. Ladder connection is similar.

1.2.9 Connection Devices

For connecting the soil reinforcements to the facing units, the following connection devices are used:

- Tie strip cast in the panel, protruding from the back face.

- One ½-inch diameter x 1 ¼" long bolt and nut set, with washer; all galvanized.

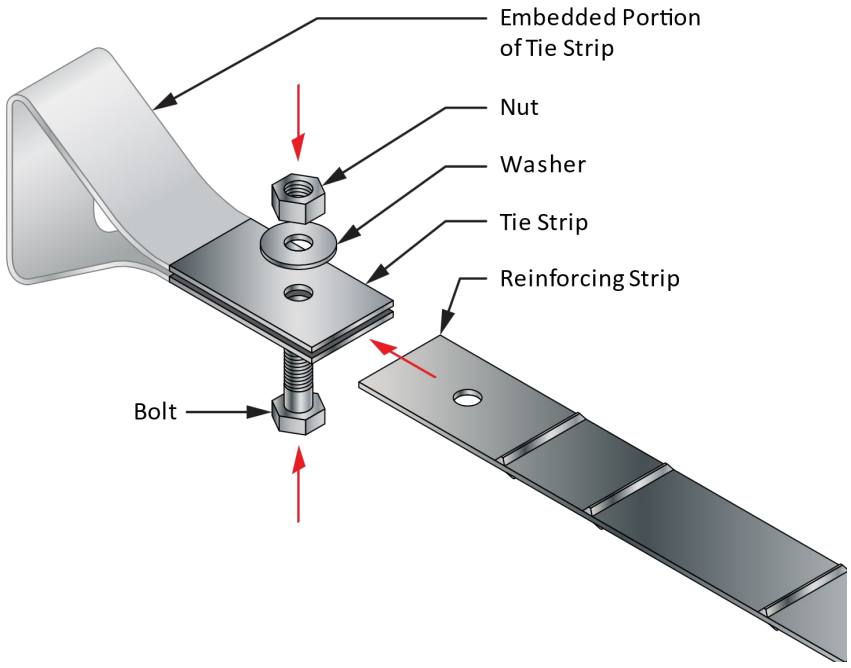


Figure 1.2.9-1: Tie Strip assembly with bolt, washer, nut and reinforcing strip

For splice-connecting HA Strips to increase the length, two connection types are used depending on the tension computed in the internal stability calculations:

- Lap splice connection.
- Splice plate connection.

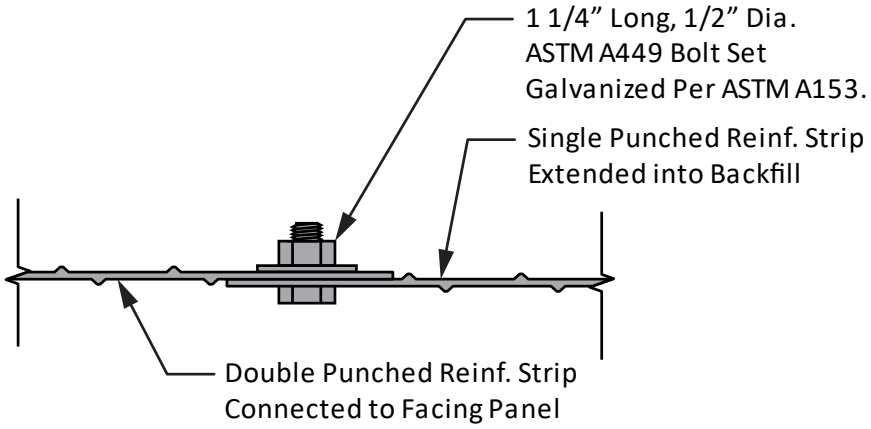


Figure 1.2.9-2: Lap Splice Connection

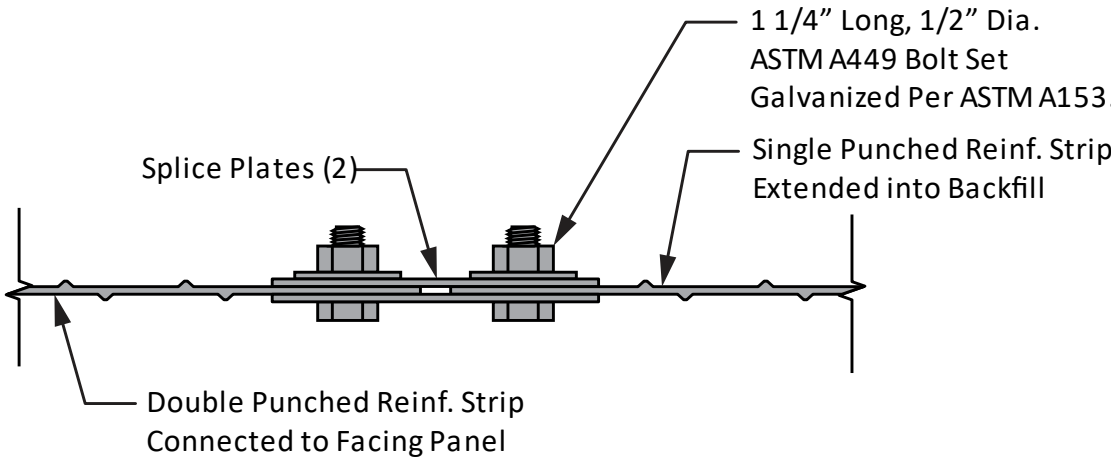


Figure 1.2.9-3: Splice Plate Connection

Case No. (See Appendix)	Connection Type	Plate Thickness (ga) [2,3]	Tie Strip / Splice Plate Dimensions (in)	Reinforcing Strip Thickness (mm) [4,5]	Max Design Load on Assembly (Kips)	Design Life 75/100 (years)
1	Tie Strip	10	2” x 7 ½”	4	9.76	75
2	Tie Strip	10	2” x 7 ½”	4	7.5	100
3	Tie Strip	7	2” x 7 ½”	6	17.01	75
4	Tie Strip	7	2” x 7 ½”	6	14.73	100

Table 1.2.9-1a: Tie strip assembly with bolt, washer, nut and reinforcing strips – connection strength.

Case No. (See Appendix)	Connection Type	Plate Thickness (ga) [2,3]	Tie Strip / Splice Plate Dimensions (in)	Reinforcing Strip Thickness (mm) [4,5]	Max Design Load on Assembly (Kips)	Design Life 75/100 (years)
5	Lap Splice	N/A	N/A	4	9.33	75
6	Lap Splice	N/A	N/A	4	7.5	100
7	Splice Plates	10	2” x 6 ½”	4	9.76	75
8	Splice Plates	10	2” x 6 ½”	4	7.5	100
9	Splice Plates	7	2” x 6 ½”	6	17.01	75
10	Splice Plates	7	2” x 6 ½”	6	14.73	100

Table 1.2.9-1b: Splice connection with bolt, washer, nut and reinforcing strips – connection strength.

Notes:

1. Design load based on 75 year and 100-year service life using AASHTO Carbon Steel Corrosion Rates for MSE Walls.
2. 10 ga = 3.42 mm (0.135 in.)
3. 7 ga = 4.55 mm (0.179 in.)
4. 4 mm = (0.157 in.)
5. 6 mm = (0.236 in.)
6. Tie Strip Pullout Capacity controls (See Section 1.2.15)
7. $\emptyset$ = 0.75
8. All Connections made with ½" Diameter A449 Bolt Set
9. Lap Splice Capacity Controlled by Bolt Shear
10. Strip yield limit state controls.

See **Appendix A1.2.9** for connection device (tie strip) fabrication details.

1.2.10 Connection Device Physical Property Specifications

The physical properties of the connection devices are as follows:

Tie strips and Splice Plates are made of hot rolled flat plate carbon steel. The standard tie strip and splice plate is 10 gauge, ASTM A1011, Grade 50, Class 1 steel and is hot-dip galvanized after fabrication. The overall dimensions are 7 ½" long x 4" deep x 2" wide. Fy = 50 ksi, Fu = 65 ksi. RECo also designs and supplies tie strips and swith the following dimensions: 7 gauge, 7 ½" long x 4" deep x 2" wide and 10 gauge, 7 ½" long x 4" deep x 4" wide.

The standard bolt is A449, ½-inch diameter and 1 ½" long. Fy = 92 ksi, Fu = 120 ksi. Note, A449 specification is used for the bolt set due to the special thread length that allows for the threads to be beyond the shear plane.

ASTM A449 covers headed bolts, rods, and anchor bolts in diameters ranging from 1/4" through 3" inclusive. It is a medium strength bolt manufactured from a medium carbon or alloy steel that develops its mechanical values through a heat-treating process. It is intended for general engineering applications.
ASTM A449 is virtually identical in chemistry and strength to ASTM A325 and SAE J429 grade 5. However, A449 is more adaptable in the sense that it covers a larger diameter range and is not restricted by a specific configuration. The components of the fastener include:

Heavy Duty Hex Head Structural Screw - conforming to minimum requirements of (ASTM F3125-15a), ASTM A449/A449M-14, Material Gr. 1035MR, Core Hardness (RC) 30.4, dimensions per ASME B18.2.6-2010.

Heavy Duty Hex nut - conforming to the minimum requirements of ASTM A563-07a, Material Gr. 1026L, Core Hardness (RC) 28.1.

Heavy Duty Structural Washer - conforming to minimum requirements of ASTM F436-11, Material Gr. F436, Core Hardness (RC) 36.6.

Spliced Reinforcement Hardware

For the purpose of lengthening a soil reinforcement beyond its manufactured length, as well as repairing damaged reinforcements and sections of RE walls, RECo has two splice connections (Single Shear and Double Shear with Splice Plates). The determination of which splice detail is required is a function of the stress in the reinforcement at the location of the splice.

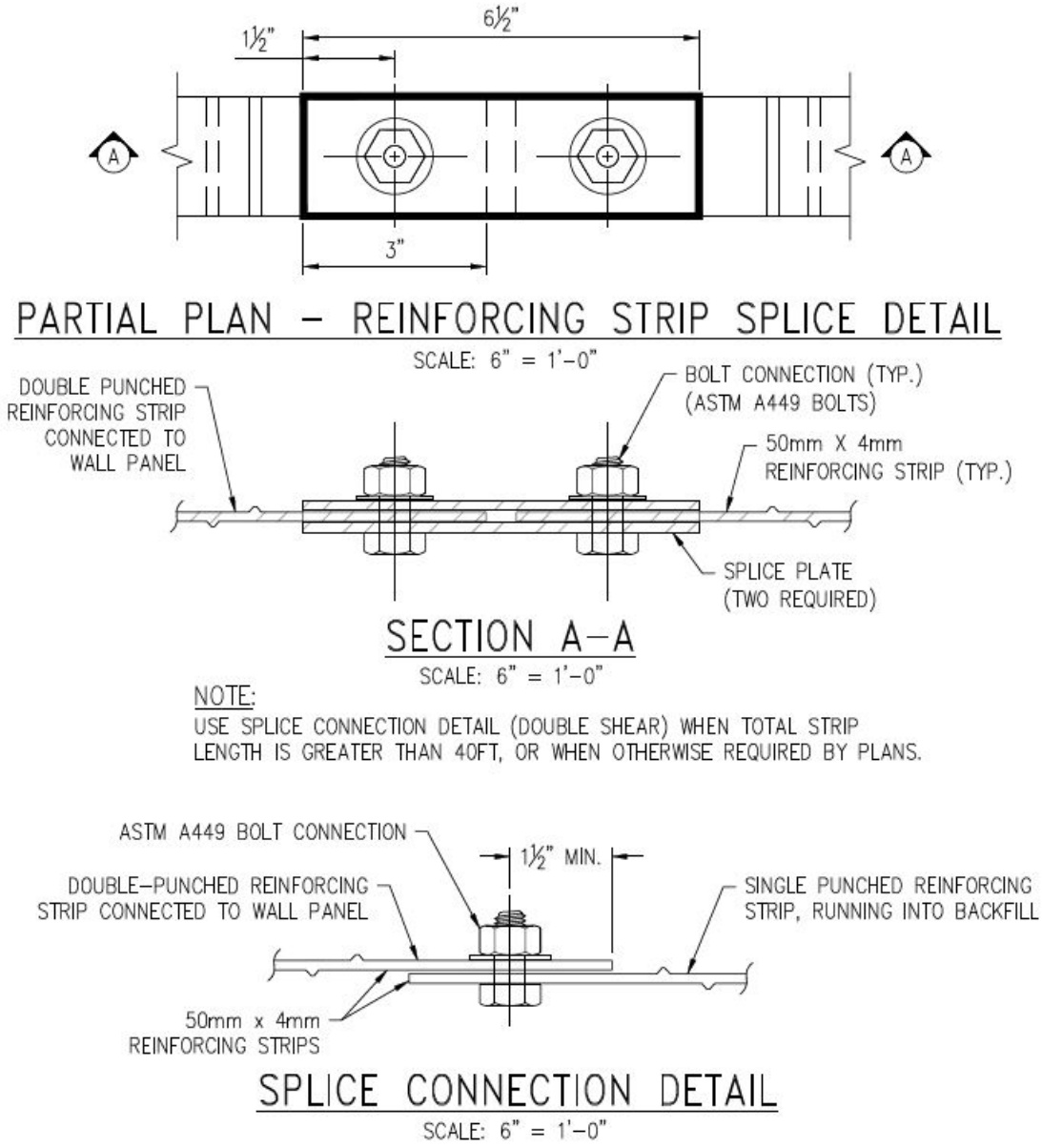


Figure 1.2.10-1: Soil reinforcement splice details

1.2.11 Connection Device Corrosion Protection

Tie Strips and Splice Plates utilize Hot-dipped galvanization. The minimum galvanization thickness for permanent structures (typically 75 year design life) is 2 oz. per square foot (3.4 mils, 86 µm)) on all[1]

surfaces. The hot-dipped galvanizing process follows ASTM A-123 specification, Section 6.1, Table 2, Coating Grade 85.

For Bolt Sets, Zinc coated Hot-Dip Galvanizing shall conform to requirements of ASTM F2329-13, minimum average thickness from 15 tests per batch 3.4 mils, 86µm.

1.2.12 Connection Device Sacrificial Steel Thickness

See Section 1.2.5 for discussion. The sacrificial steel thickness is based on the metal loss rates specified by AASHTO. For a 75-year design life, the sacrificial steel thickness is 0.708mm per exposed side. For a 100-year design life, the sacrificial steel thickness is 1.008mm per exposed side.

1.2.13 Connection Device Corrosion Test Results

See **Section 1.2.6** for discussion. In addition, FHWA NHI-09-087 section 2.4.c states the following:

> *"Some systems use tie strips embedded in precast concrete panels to connect the reinforcements at the wall face. The tie strips have top and bottom plates that sandwich the reinforcements; constituting a double shear connection. Within the tie-strip/strip interface there is very little loss of thickness; negligible compared to that of surfaces exposed to the soil. Thus, loss of thickness and sacrificial steel requirements are applied the gross cross section."*

This means that metal losses from corrosion are not significant for the faying surfaces of the reinforcements (within the connection), so only one side of a tie strip is exposed to the fill and subject to metal loss. The beneficial effect of the "sandwich" has been observed on numerous extracted soil reinforcement samples. Examples are included in **Appendix A1.2.13**.

1.2.14 Connection Device Dimensional Tolerances

See **Appendix A1.2.9** for connection device dimensional tolerances.

1.2.15 Facing Unit Connection Strength Test Results

RECo's standard Tie Strip connections have been tested periodically since the 1980's.

Following is a summary of 35 pullout test samples, conducted in 3 different labs between 1990 and 2021:

Test Series	No. of Samples	f'c (psi)	Average Failure Load (lbs)	Confinement	Skew (deg.)	$\frac{\sqrt{5{,}000}}{\sqrt{(f'c)}}$	$\frac{\sqrt{4{,}000}}{\sqrt{(f'c)}}$	Nu, 5,000	Nu, 4,000	ϕNu, 5,000	ϕNu, 4,000
AEL 1990	**5**	**4437**	**17,624**	**N**	**N**	106%	95%	18,708	16,733	14,031	12,550
STI 2020	**5**	**6270**	**17,397**	**N**	**N**	89%	80%	15,536	13,896	11,652	10,422
STI 2020	**5**	**5360**	**16,676**	**N**	**N**	97%	86%	16,069	14,406	12,080	10,805
SGI 2021	**5**	**7200**	**18,047**	**N**	**N**	83%	75%	15,039	13,451	11,279	10,088
SGI 2021	**5**	**8020**	**17,640**	**Y**	**N**	79%	71%	13,928	12,458	10,446	9,343
SGI 2021	**5**	**8040**	**16,052**	**Y**	**15**	79%	71%	12,659	11,322	9,494	8,492
SGI 2021	**5**	**8030**	**14,394**	**Y**	**30**	79%	71%	11,358	10,159	8,519	7,619
							AVG No Skew	15,856	14,189	11,898	10,642

Table 1.2.15-1: Tie Strip Connection Pullout Strength Test Data

Notes:

1. Reports included in **Appendix A1.2.15**

2. Φ = 0.75

3. **ϕ** Nu,5000 = Average Failure Load x $\frac{\sqrt{5{,}000}}{\sqrt{(f'c)}}$ x ϕ

In summary, the ultimate pullout capacity of Tie Strip Connectors exceeds the design steel strength of the connection assembly (accounting for metal losses) by a factor of 1.6 +/-.

Design Life	Factored Limiting Design Tension (Reinforcement) (kips)	Factored Tie Strip Tensile Capacity (kips)[1]	Factored Tie Strip Pullout Capacity * (Kips)	Factored Tie Strip Pullout Capacity ** (Kips)
75 Years	9.75	11.65	11.90	10.64
100 Years	7.50	8.16	11.90	10.64

Table 1.2.15-2: Connection Strength Summary

[1] 10 gage Tie Strip
*: Nu, 5,000
** Nu, 4,000

Note that pullout of the connection is not affected by metal loss, and as such the controlling factor for connection design is the loss of strength due to loss of sacrificial thickness, and not pullout.

See **Appendix A1.2.15** for connection unit test results, and design calculations.

Effect of splaying reinforcing strips and ladders on the pullout strength of connection

Results from skewed connection test program are summarized in **Table 2.1.3-2**.

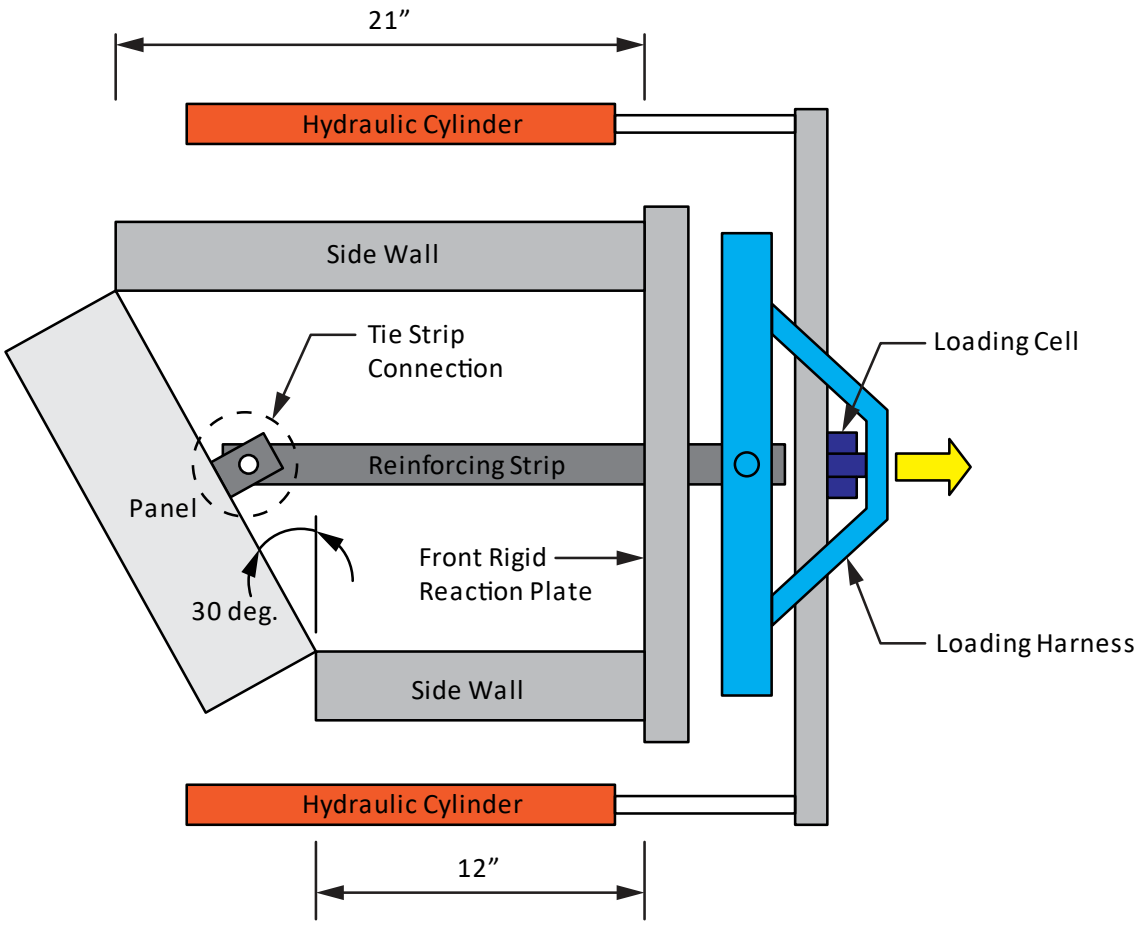


Figure 1.2.15-1: Pullout of Tie-Strip with splayed reinforcement. Schematic diagram of the confined connection test setup with the concrete panel at a skew angle α (30° or 15°)

Excerpt from SGI report: SGI121015.2021.01:

> *“As proven by the testing at SGI in 2021, splaying of reinforcing strips up to angles of 30 degrees results in a minor decrease in tie strip pullout capacity, but the connection*

strength is governed by the Reinforcing Strip Tensile Capacity (accounting for metal losses), and not the pullout strength."

1.2.16 Soil Reinforcement Pullout Test Results

High Adherence (HA) Strips

The traditional design model used by RECo and AASHTO for High adherence (ribbed) steel strips is f* = 1.2+log Cu. This model was based on the data available in the 1980's and included backfill types that do not meet AASHTO MSE backfill requirements for durability, minimum shear strength (34 degrees), maximum fines content (15% < #200), maximum percentage passing the No. 40 sieve (60%) and degree of compaction (95% standard Proctor: AASHTO T-99). Since the time of the initial issue of the AASHTO design specifications, RECo and other researchers have performed tests on hundreds of samples in varying backfills that meet AASHTO specifications.

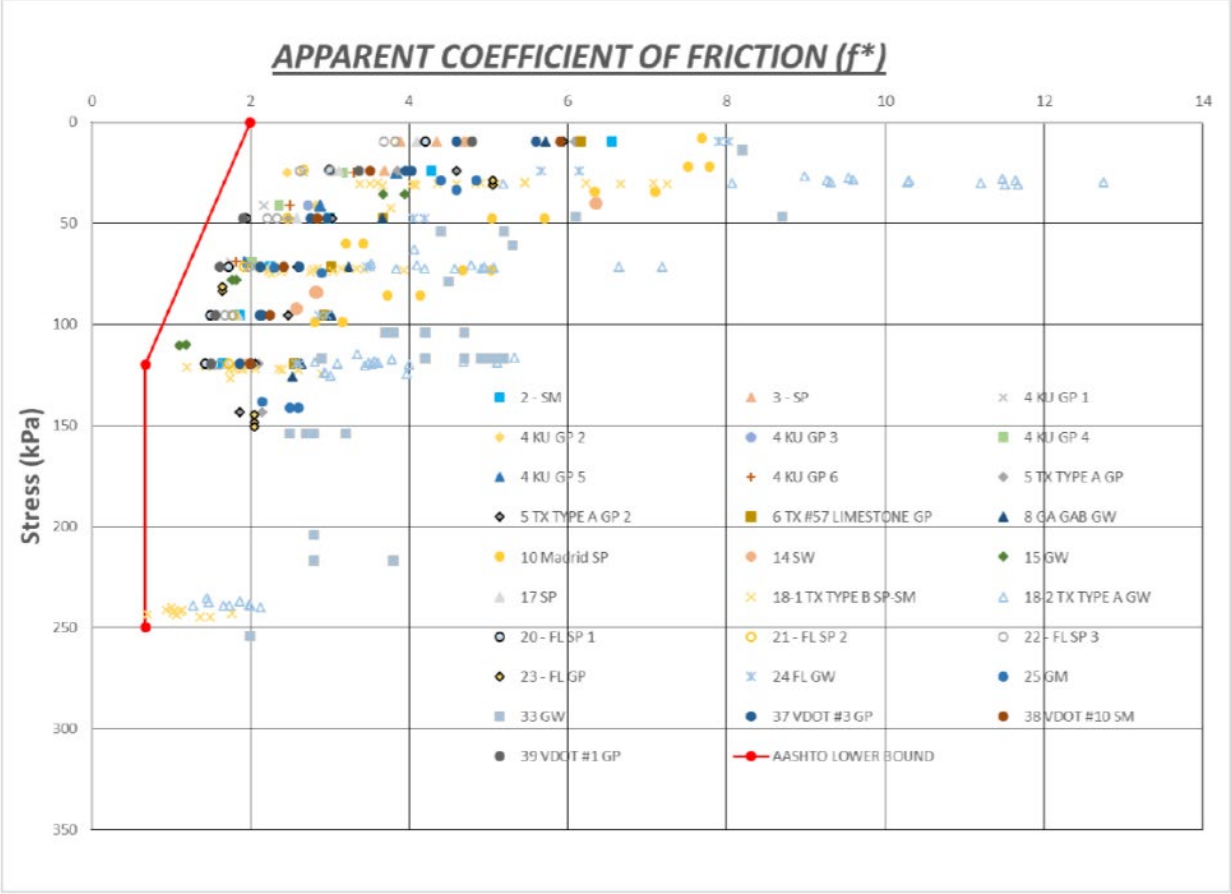


Figure 1.2.16-1 – f for HA Strips*

Based on the data available for AASHTO backfills, RECo recommends the following relationship for determining f* for RECo High Adherence (aka ribbed)strips for granular backfills meeting AASHTO requirements for gradation, shear strength and durability:

$$f^*_o = 1.5 + \ln(Cu) < 4.50 \qquad \textit{(Formula 1.2.16)}$$

The above relationship envelopes the 95% confidence limit of backfills meeting AASHTO specifications.

For Open Graded Coarse Aggregates: it was recognized early on by RECo that coarse aggregates with a low Cu do not fit the 1.2 + log Cu model, and per RECo Tech Bulletin MSE-6 (1995) the recommended f* value for coarse aggregates was 2.0 at the ground surface. Further testing has shown that the f* value for coarse aggregates far exceeds this value, therefore the current Cu relationship does not apply. See recommendations later in Tech Bulletin 2019-01 for Open Graded Coarse Aggregate backfills such as AASHTO #1 thru #7 aggregates.

Cu Calculation Note: For the purposes of f* calculations, the smallest Sieve Size used for the D10 calculation is #200 (0.075mm), so if a backfill has 15% passing the #200 sieve and 10% passing the 0.05mm, 0.075mm is used in the calculation.

Note: Figure 11.10.6.3.2-2-Default Values for the Pullout Friction Factor, shows the inflection point on f* = tan ∅ to be at a depth of Z_p = 20 ft. Research with conventional and light weight backfills has shown that this inflection point is related to overburden pressure, not necessarily depth, and as such should be ~2500 psf or 120 kPa. Further discussion and data related to this topic can be found in RECo Tech Bulletin 2019-01, located in **Appendix A1.2.1**.

Pullout test procedures and results are included in Technical Bulletin 2019-01 **Appendix A1.2.1.**

The pullout resistance of steel strip reinforcements has been studied since the early 1970's and there are numerous technical papers and articles dedicated to the subject. In addition to the test reports and papers included herein, following is a list of some additional references with information on the subject. These references may be provided upon request.

1978. Friction in Reinforced Earth. Schlosser and Elias. ASCE Symposium on Earth Reinforcement

1996. Comparative study and measurement of the pull-out capacity of extensible and inextensible reinforcements. Segrestin & Bastick. Earth Reinforcement (Ochai, asufuku & Omine (eds) 1996 Belkema, Rotterdam. ISBN 90 5410 833 9.

2013. Pullout Resistance of Mechanically Stabilized Reinforcements in Backfills Typically Used in Texas: Volumes 1, 2 and 3. Texas Tech University Multidisciplinary Research in Transportation. Report #0-6493-1.

2013. Pullout Resistance Factors for Steel Reinforcements Used in TXDOT MSE Walls. Lawson, et al. ASCE GeoCongress 2013

2015. Pullout Resistance of Mechanically Stabilized Earth Wall Steel Strip Reinforcement in Uniform Aggregate. University of Kansas Report No. K-TRAN: KU-14-7.

2016. Frictional Resistance of Closely Spaced Steel Reinforcement Strips used in MSE Walls. Strahler, Walters and Stuedlein. ASCE Journal of Geotechnical and Geoenvironmental Engineering. April 1, 2016.

High Adherence (HA) Ladders

The use of High Adherence ladder reinforcing strips (H.A. Ladders) to reinforce soil is an extension of Reinforced Earth technology. The discrete strips of galvanized steel are made of welded wire, manufactured into a ladder configuration. H.A. Ladders have two longitudinal wires spaced 2" apart with cross bars (the rungs of the ladder) at 6" on center. Each of the cross bars are 4" long and therefore the effective width of the H.A. Ladder is 4".

RECo performed extensive pullout testing during the development of HA Ladders in the late 1990's. Subsequent testing was performed by Texas Tech University in 2010 and the 2011 report is included in Appendix A1.2.16. An excerpt of the report is shown below in Figure 1.2.16-2.

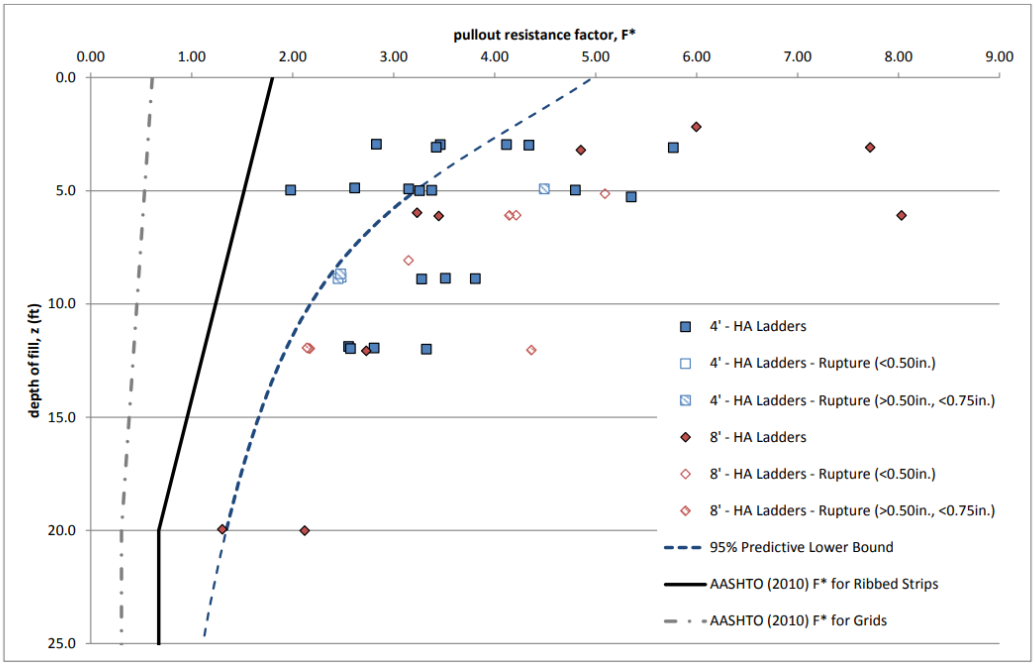


Figure 1.2.16-2 – f testing for HA Ladders (Texas Tech report figure 3.16)*

Summary f* design values for HA Strips and HA Ladders:

Reinforcement	b (inches)	α	f*	
			Overburden Pressure up to 2,500 psf	Overburden Pressure 2500 psf
HA ribbed strip (4 mm)	1.97	1.00	$1.5 + \ln(C_u) < 4.50$	tan φ
HA ribbed strip (4 mm)	1.97	1.00	$1.5 + \ln(C_u) < 4.50$	tan φ
HA ribbed strip (6 mm)	1.97	1.00	$1.5 + \ln(C_u) < 4.50$	tan φ
HA ribbed strip (6 mm)	1.97	1.00	$1.5 + \ln(C_u) < 4.50$	tan φ
	Width (inches)			
HA ladder (W10)	4.00	1.00	1.2+log(Cu) < 3.0	tan φ
HA ladder (W10)	4.00	1.00	1.2+log(Cu) < 3.0	tan φ

Table 1.2.16-1 – Summary f for HA Strips and HA Ladders*

1.2.17 Soil Reinforcement Interface Shear Test Results

N/A for discrete soil reinforcements.

1.3 Other Components

Included below are other components that are inherent to the wall system (reinforced soil and drainage measures), as well as components that may not be required on all projects (coping and traffic barrier top-outs, and slip joints).

1.3.1 Innovations

Low Density Cellular Concrete as MSE Wall backfill

The Reinforced Earth Company has extensive experience with the use of Low-Density Cellular Concrete (LDCC) as MSE wall backfill. Some of the specification parameters below for granular fills do not apply for LDCC. See **Appendix A1.3.1** for additional information.

1.3.2 Reinforced Soil Properties

The typical ranges of backfill for Reinforced Earth are within the same gradations recommended in AASHTO and FHWA for use with metallic reinforcement. Select granular backfill conforming to Contract Specifications must be used within the reinforced volume.

What are the general performance requirements for select backfill?

Typically, select backfill is to be in accordance with AASHTO specifications. State and or Project specifications for MSE select backfill are prepared by the Owner, or the Consultant, based on project-specific requirements. Backfill selection also requires consideration of the relative economies of imported versus locally available materials, previous experience with similar materials, and local construction practice. Reinforced Earth select backfill may be a natural soil and rock material, a recycled material, or an industrial by-product. All select backfill material, no matter what its origin, must meet the following general performance requirements:

- The material must be well drained,
- The material must not be prone to post-construction movement due to creep,
- The material must be durable and not break down or change its properties during construction, and
- The material must not be overly aggressive relative to corrosion of galvanized steel.

What are the minimum physio-chemical requirements for select backfill?

In general, select backfill materials should meet the requirements and conform to the specifications shown in **Tables 1.3.2-1** and **1.3.2-2**. In very special circumstances, exceptions may be made to these requirements after careful review by RECo and consultation with the owner's engineering team.

How does the behavior of a Reinforced Earth structure change if the amount of fines in the select backfill increases?

Although the standard specification for Reinforced Earth select backfill requires less than or equal to 15% passing the 0.075 mm (No. 200) sieve, materials with greater fines passing the 0.075 mm (No. 200) sieve *may be considered under limited circumstances and after careful assessment*.

The Owner/Consultant must weigh the potential cost advantage of using such fine-grained backfill against the possibly significant increase in the number and length of steel reinforcements required, as well as the resulting increase in the Reinforced Earth backfill volume. In order to justify using a soil with greater than 15% fines as select backfill, the designer must evaluate short term stability factors, including saturation/drainage behavior, and develop construction procedures appropriate to that material.

The select granular backfill material used in the mechanically stabilized earth structure shall be reasonably free from organic and otherwise deleterious materials and shall conform to the following gradation limits as determined by AASHTO T-27:

Sieve Size	Percent Passing
4 inches	100
3 inches	75 – 100
No. 40	0 – 60
No. 200	0 - 15

Table 1.3.2-1

(Reference AASHTO LRFD Bridge Construction Specifications section 7.3.6.3)

The friction angle of the select backfill used in the reinforced fill zone shall be 34° minimum, unless shown otherwise on the Plans. The friction angle shall be determined by the standard direct shear test, AASHTO T-236, utilizing a sample of the material compacted to 95 percent of AASHTO T-99, Methods C or D (with oversize correction, as outlined in Note 7), at optimum moisture content

In addition, the backfill shall conform to all of the following requirements:

(a) Plasticity Index -- The Plasticity Index (P.I.), as determined by AASHTO T-90, shall not exceed 6.

(b) Soundness -- The material shall be substantially free of shale or other soft, poor durability particles. The material shall have a magnesium sulfate soundness loss of less than 30 percent after four (4) cycles, as determined by AASHTO T-104.

(c) Organic Content – The material shall have an organic content less than one percent as determined by AASHTO T 267 on the portion of the material finer than the No. 10 sieve.

(d) Electrochemical Requirements -- The backfill material shall conform to the following electrochemical requirements:

Property	Requirement	Applicable Test Methods
Resistivity[1]	Minimum 3000 ohm-cm, at 100% saturation	AASHTO T-288-12 ASTM G-187-12a
pH	5 – 10	AASHTO T-289-91 ASTM G-51-95(2012)
Water Soluble Chlorides[2, 3]	≤ 100 ppm	AASHTO T-291-94 ASTM D-512-12 ASTM D-4327-11
Water Soluble Sulfates[2, 3]	≤ 200 ppm	AASHTO T-290-95 (2012) ASTM D-516-16 ASTM D-4327-11

Table 1.3.2-2

(Reference AASHTO LRFD Bridge Construction Specifications section 7.3.6.3 and AASHTO LRFD Bridge Design Specification section C11.10.6.4.2a)

Notes:

1. Resistivity testing shall be performed up to and including, but not exceeding, 100 % soil saturation. Note 6 of AASHTO T-288 (Note 5 in older editions) shall not be used.
2. If the minimum resistivity exceeds 5000 ohm-cm, at 100% saturation, the need for testing of chlorides and sulfates is waived.
3. ASTM D-4327-11 (Standard Test Method for Anions in Water by Suppressed Ion Chromatography) may be used in lieu of AASHTO T-291/ASTM D-512 and AASHTO T-290/ASTM D-516 for determining the concentration of Water-Soluble Chlorides and Sulfates.

1.3.3 Drainage Measures

The following drainage measures are inherent to the wall system:

- Panel joints: The internal drainage with relation to the panel joints and geotextile is discussed in **Section 1.1.9.**
- Backfill: Backfill gradation is typically well draining (< 15% fines). For situations where the wall is expected to be periodically inundated with water due to high water events from adjacent rivers, streams, and reservoirs, a free draining backfill (with less than 5% fines) should be used.

The following drainage measures are often incorporated into the MSE wall design if site conditions dictate and/or required by the Owner:

- Base drainage systems, specified and designed by the project Geotechnical Engineer, rather than the wall system supplier.
- It is highly recommended to direct any storm water away from the wall and into correctly designed collection systems. Surface drainage elements are designed by the project Civil

Engineer (and not the MSE wall system supplier). Surface drainage elements are not designed, supplied or specified by wall system supplier.

The drainage design for MSE walls should be coordinated between the Civil Engineer, Geotechnical Engineer and the MSE Wall Designer. External drainage features (swales, ditches, French drains, etc.) should be clearly shown in the contract documents. Internal drainage requirements (backfill type, panel joint geotextile types and any requirements for under-drains) should be specified in the contract documents. Underdrain outlet locations and tie-ins to site drainage elements should be considered by the Civil Engineer and coordinated between the Civil Engineer, Geotechnical Engineer, MSE Wall Designer and the contractor depending on site conditions. Refer to FHWA-NHI-10-024 (2009) Section 5.3 for more information.

Although panel joints provide adequate weeping area, weep holes can be cast into the facing units. Since the ¾" panel joints provide adequate weeping area, weep-holes are not typically specified unless required by the Owner's Engineer.

1.3.4 Coping Top-out



Figure 1.3.4-1: Precast Coping atop MSE Facing

Standard precast concrete coping units for RE walls are supplied in 10-ft lengths. In section, the coping unit is 2-ft tall and typically 1'-4" wide (For plain finish panels or panels with a shallow architectural relief). A 1'-3" deep "throat" in the bottom allows the unit to sit atop the facing units and leveling concrete. Cast-in-place coping design can be provided to match the precast section as necessary. Typical dimensions and details can be found in **Appendix A1.3.4**.

When a fence or handrail is attached to the coping, a structural connection to the facing may be required. With cast in place coping this is achieved using dowels protruding from the top of facing units,

and for precast coping a steel connection angle or Z plate is bolted to the back face of MSE panel and coping unit.

1.3.5 Traffic Barrier Top-out

A wide variety of traffic barrier sections can be applied to RECo's RE walls. The most common precast concrete option is shipped as a monolithically cast coping and parapet unit, with rebar protruding to accommodate a cast-in-place moment slab. A 1'-3" deep "throat" in the bottom allows the unit to sit atop the facing units and leveling concrete. All other dimensions of the traffic barrier unit are in accordance with the Owner's requirements. Cast-in-place coping design can be provided to match the precast section as necessary.

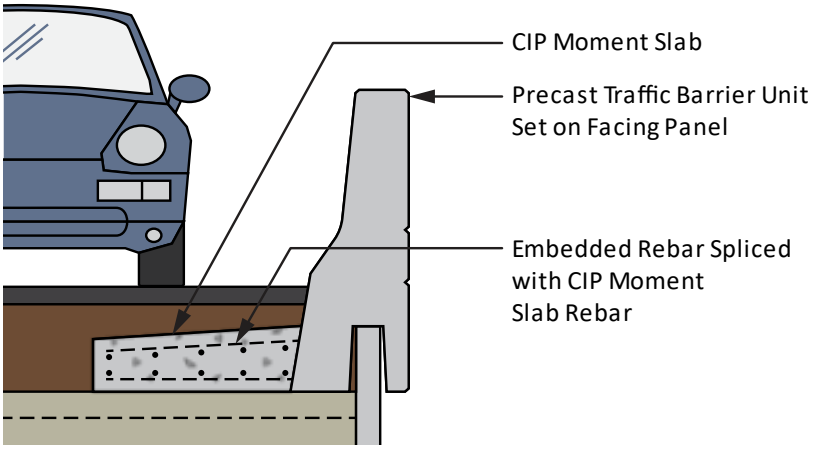


Figure 1.3.5-1: Precast Traffic Barrier and Cast in Place Moment Slab

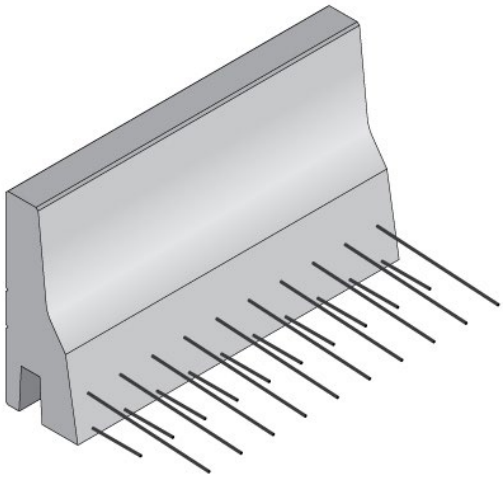


Figure 1.3.5-2: Precast Traffic Barrier Unit

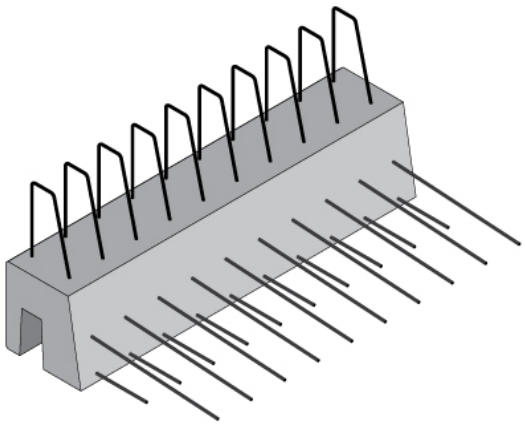


Figure 1.3.5-3: Precast Half Connector Unit

RECo does not supply materials for a post and beam type traffic barrier, but this option is easily accommodated by the RE wall. The Owner's design for post size, spacing and depth is coordinated with RECo to incorporate any necessary considerations into the design of the RE wall.

Dimensions and details for a typical concrete traffic barrier, as well as a typical post and beam alternative can be found in **Appendix A1.3.5**.

1.3.6 Slip Joints

Slip joints are installed at specified facing columns along a wall to increase the wall's capacity to handle differential settlement under three primary cases:

- To provide additional facing flexibility for severe differential settlement due to foundation conditions. (Design limiting differential settlement: 5'x5' panels = 1%, 5'x10' panels = 0.5%).
- To address the special case of differential settlement where wall foundation conditions change abruptly:
    - Rock foundation to soil.
    - Reinforced Earth wing-wall "steps off" a rigid concrete foundation such as the heel of the abutment onto the relatively more flexible soil foundation beyond the heel.
- To cover and add flexibility to the transition from a Reinforced Earth structure to a cast-in-place concrete structure, as when a retaining wall abuts the headwall of a culvert or the concrete wingwall of a bridge.

The slip joint consists of a gap in the facing units of up to 1'-6" wide, with a slip joint cover bridging the gap in the wall. RECo's standard slip joint covers are 1'-10" wide on the front face and include a narrow stem on the back face. The configuration allows for additional movement and rotation of the panel columns on either side of the slip joint cover in relation to each other. For typical slip joint details, refer to **Appendix A1.3.6**.

2.0 ERS Design

2.1 Design Methodology

The design methodology of a Reinforced Earth wall considers both the external and internal stability of the structure. Internal stability is determined using the Coherent Gravity Method (CGM). The CGM is provided in **Section 2.1.2** and in detail in the AASHTO LRFD 9th Edition (2020) and the 2022 FHWA MSE Wall Manual.

2.1.1 Innovations

Four innovations are claimed by RECo regarding design methodology:

1. Increased Pullout Resistance
2. Acute Corner Design
3. Shored MSE Walls
4. MSE Wall construction on marginal foundations (phased and 2 Stage Construction)

Note on Overall Design Methodology:

Reinforced Earth® was invented in 1957 by French engineer and architect Henri Vidal, who first published results of his research in 1963. After a brief period of skepticism by practicing engineers, the first significant structures were constructed in Europe in 1967. The new, patented technology was so versatile and cost effective that its use spread rapidly in the early 1970's to more than 30 countries throughout the world.

Vidal brought his technology to the United States in 1971, at the invitation of the Federal Highway Administration (FHWA), to solve a difficult landslide problem. With the encouragement of financial support from a Federal Highway demonstration project, several states constructed additional Reinforced Earth structures as slide buttresses and slide repair walls in mountainous areas, as bridge abutments, high retaining walls, and even a foundation reinforcement slab over a sinkhole-prone area. From these first experiences, it was recognized that this technology, with its unique high strength and flexibility, could be used to support extremely heavy loads, even on marginal foundation conditions. The Coherent Gravity Design Method was developed by The Reinforced Earth Company based on data from instrumented structures, and further validated by numerical modeling in the early 1980's.

Increased Pullout Resistance (f*):

The use of a higher f* compared to AASHTO default values (see **Section 1.2.16**).

Acute Corner Design:

Using a bin-wall design for arranging the facing and soil reinforcements to accommodate an acute corner. RECo has performed numerical modeling that justifies a threshold of <60 degrees for acute corner design, rather than AASHTO recommended 70 degrees. Refer to "Acute Corner Design" in **Appendix A2.1** for more information.

MSE walls on marginal foundations:

The inherent flexibility of Reinforced Earth MSE walls allow for construction on sites with soft soils. In fact, the very first RE wall constructed in the US settled over 3 ft. Single-stage Reinforced Earth walls can be constructed on marginal foundations with the use of phased construction, surcharges, lightweight fills, ground improvement, or some combination of these methods.

More information can be seen at https://reinforcedearth.com/blog/accommodating-foundation-soil-settlement-with-retaining-walls/

Refer to RECo Technical Memo 2020-01 in **Appendix A2.1** for further discussion.

Shored MSE Walls:

Since the 1970's RECo has provided unique solutions for non-conventional applications. Locations where reinforcement lengths less than 70% of the wall height have been employed in various applications. More recently, a specific sub-set of this technique has been designated as Shored MSE (SMSE) by FHWA. RECo follows the general recommendations of the FHWA SMSE Manual (February 2006), with some specific modifications and variations in design and detailing based on the use of inextensible reinforcements (rather than extensible reinforcements as researched for the FHWA SMSE manual).

Most notably, RECo often utilizes "sandwich" reinforcements for SMSE walls with aspect ratios less than 0.5 in order to control deformation and provide reinforcements against potential compound failure plane at the shoring wall and MSE fill interface.

2.1.2 Design Methodology Description

Definitions:

Internal Stability

"Internal Stability" shall mean check of design plans and drawings aiming at preventing the occurrence of purely internal failure of the structure, i.e., failure modes not engaging outside the volume limited by the foundation level at the bottom, the facing elements in the front and the end of the soil reinforcement or stems at the back. This check is limited to i) soil reinforcement tensile capacity; ii) soil reinforcement pull-out capacity; iii) facing elements structural capacity; and iv) structural capacity of connectors between soil reinforcement and facing elements.

External Stability

"Local External Stability" shall mean i) simple sliding along the base of MSE wall backfill at the interface with soils below the wall (top of leveling pad elevation), ii) determination of applied bearing pressure, and iii) overturning (limiting eccentricity).

"External Stability" shall mean the geotechnical design aimed at preventing the occurrence of failures engaging volumes at least partially outside the Structure. External Stability shall include without limitation i) sliding on the base; ii) bearing capacity of the foundation soil; iii) overturning, where

applicable; iv) compound sliding failure (failure surface engaging volumes both outside and inside the structure); and v) global sliding failure (failure surface engaging only volumes outside the structure).

These checks are carried out based on the geometry and design parameters provided by the Owner and assuming a correct execution of the works, in compliance with local regulations and the provisions of the Construction Manual.

Design Responsibilities

The external stability (bearing capacity, settlement, sliding, compound and global stability) analyses are performed by the Owner's or Design Builder's geotechnical engineer of record (GER), unless specified otherwise. Global stability is an important consideration based on characteristics of the deeper strata affecting both the MSE wall itself and surrounding area and should be investigated by the owner in the project scoping phase to confirm that the site is suitable for supporting the loads of a MSE wall. The External Stability Design of a MSE structure is performed utilizing the AASHTO LRFD 2020 specifications.

The Internal Stability design of a Reinforced Earth MSE structure is typically performed by RECo utilizing the Coherent Gravity Method detailed in the AASHTO LRFD 2020 specifications (Section C11.10.6.2.1). Where project specifications require, the Simplified Method (AASHTO LRFD 2020 specifications Appendix B11) is utilized for Internal Stability design.

Although it is not part of Internal Stability, Local External Stability (Sliding, Overturning/Limiting Eccentricity and Applied Bearing Pressure) is checked by RECo, using the parameters specified by the GER.

Coherent Gravity Method

Per "State of the Practice of MSE Wall Design for Highway Structures" (Anderson, Gladstone and Sankey, 2013),

> *"The Coherent Gravity Method was developed by postulating MSE structure behavior, observing actual structures, and interpreting observations in terms of the fundamentals of statics and soil mechanics (Anderson, et al., 2010). Three international symposia on soil reinforcement in 1978 and 1979, followed by publication of "Reinforced Earth Structures, Recommendations and Rules of the Art" (French Ministry of Transport, 1979), presented a substantial body of knowledge and defined the Coherent Gravity Method for the design of MSE structures. The method includes a bilinear envelope of maximum reinforcement tension, a state of stress varying with depth within the structure, high-pullout-resistance reinforcements (originally ribbed strips, with data becoming available later for welded wire mesh reinforcements), and minimal reinforcement movement and elongation. Design characteristics of the Coherent Gravity Method, as shown in Figure 2 [Figure 2.1.2-1], are listed below.*

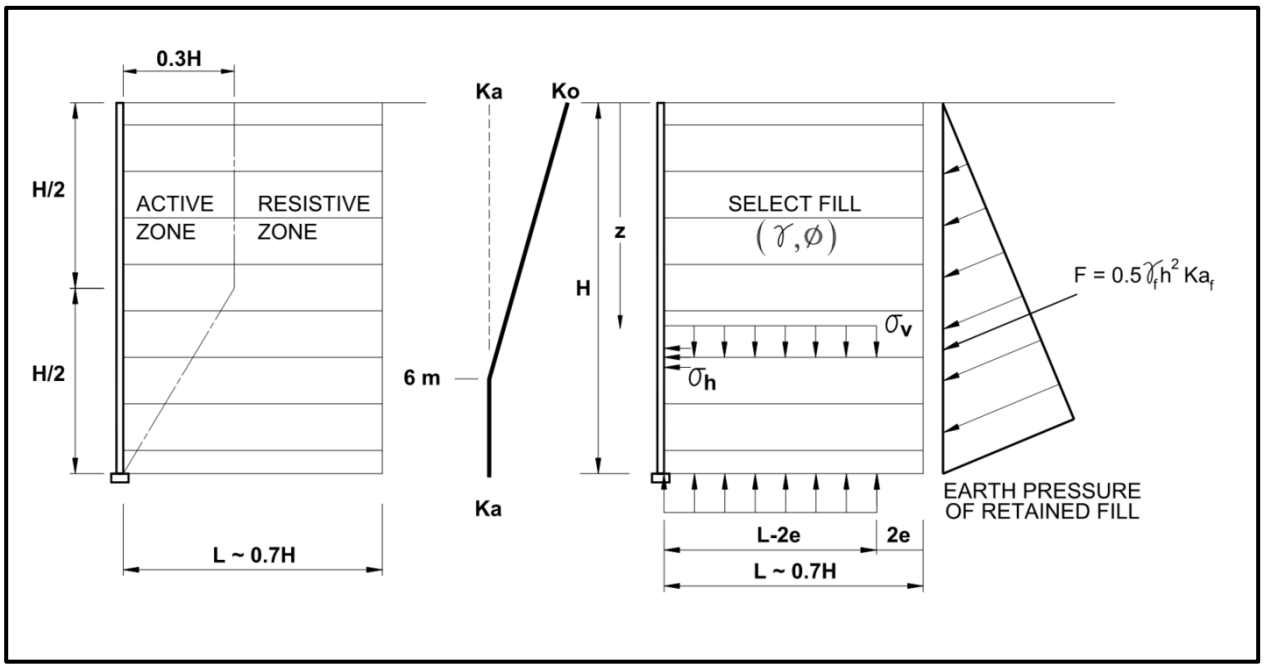


Figure 2.1.2-1

- *A rectangular cross section ("block") defined by the structure height, H, and the reinforcement length, L;*
- *Application of vertical and horizontal forces to the block, creating eccentric loading;*
- *A Meyerhof bearing pressure distribution at the base of the structure to determine foundation reactions and the repeated use of Meyerhof to determine the vertical earth pressure at each reinforcement level (Meyerhof, 1953);*
- *A state of stress decreasing from at rest (K_0) at the top of the structure to active (K_a) at a depth beyond 6 meters ;*
- *The resulting tensile forces in the reinforcements, determined from the horizontal earth pressure multiplied by the tributary area of the wall face restrained by the reinforcement at that level;*
- *The bilinear envelope of maximum reinforcement tension that separates the active from the resistive zone; and*
- *The inextensibility and high pullout resistance of the reinforcements which maintain the internal stability of the block.*

The effects of externally applied loads on the reinforced soil mass, and the tendency of those loads to increase vertical and horizontal stresses within the structure, were confirmed by and extensive finite element study of 6 meter and 10.5 meter high walls (Anderson, et al., 1983). The Coherent Gravity Method was reported in its entirety,

including worked example calculations, by Mitchell, et al., in NCHRP Report 290 (NCHRP, 1987).

Additional information regarding the Coherent Gravity Method can be found in the technical paper, Anderson, Gladstone and Withiam: "Coherent Gravity: The Correct Design Method for Steel Reinforced MSE Walls", ASCE Earth Retention Conference Proceedings, ER 2010.

The Coherent Gravity Method has been utilized by RECo since the 1980's. However, as LRFD methods evolved, the application of load and resistance factors has confused the fundamental design hypothesis that externally applied loads create eccentricity within the reinforced soil volume. The eccentricity that is calculated must be based on service loads. The resulting calculated vertical earth pressure be factored by the appropriate load factors to result in a factored design load that correlates to a factored earth pressure (σ_h).

Internal Stability – Coherent Gravity Method- Design Summary

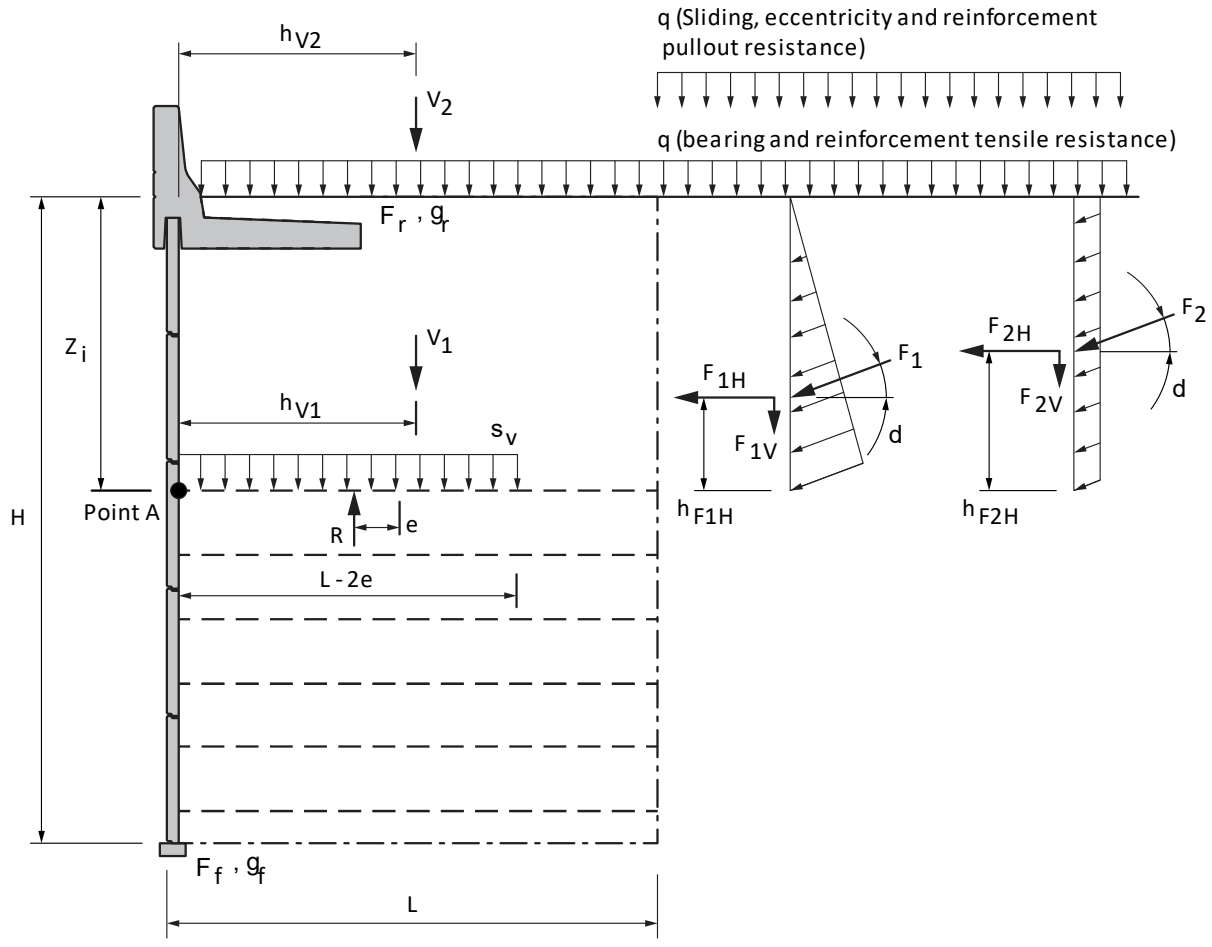


Figure 2.1.2-2

Vertical Soil Stress, σ_v, at Reinforcement Level:

Based on local equilibrium of all forces acting above that level (i).

$\sigma_v = \sum V / (L - 2e_i)$

The eccentricity is calculated based on service loads and the resulting vertical earth pressure is factored to result in a factored design load that correlates to a factored earth pressure σ_h.

Horizontal Soil Stress, σ_h, at Reinforcement Level:

Product of vertical soil stress and the appropriate K value for the overburden height at reinforcement level.

$\sigma_h = \sigma_v$ x K

Pullout Calculations

Consider only the effective length of reinforcement beyond the line of maximum tension.

Exclude any traffic surcharge applied above the reinforced soil mass in calculating the vertical stress and overburden for pullout resistance

For strip reinforcement, use:

R = 2 b γ Z f* Le Ø Where:

R = Factored Pullout Resistance of Reinforcing Strip

2 b = Two times the width of reinforcing strip (i.e. Top and Bottom)

γ = Unit Weight of Soil

Z = Depth to Reinforcing Strip Level

f* = Apparent Coefficient of Friction between reinforcement and soil

Le = Effective Length of Reinforcing Strip Beyond Line of Max Tension

Ø =0.9 (Reduction Factor for Pullout Resistance)

Reinforcing Strip Pullout CHECK

Effective Length Overdesign Factor = R N / (σ_{hb} A_t) Where:

N = Number of Reinforcing Strips Across Tributary Width

A_t = Tributary Area = Tributary Width x Reinforcing Strip Vertical Spacing

R = Factored Pullout Resistance of Reinforcing Strips

σ_{hb} = Factored Horizontal Soil Stress at reinforcement layer

Maximum Reinforcement Tension

The procedure for determining earth pressures at each level of reinforcement within the structure is the same as for reinforcing strip pullout, except that the live load surcharge directly over the reinforced zone is included in the calculation.

Once the horizontal stress is determined at each reinforcement level, the number of strips required at each level is calculated by the following equation:

$N = (\sigma_{ht} \times A_t) / T$

Where:

N = Number of strips per tributary wall area (rounded up to the nearest integer)

σ_{ht} = Factored Horizontal earth pressure at that level

A_t = Tributary wall area resisted by N strips

T = Factored reinforcing strip tensile resistance

The tributary area used in the calculations above is typically 2.46’ x 9.84’ (24.2 sq ft) for Reinforced Earth structures using standard precast concrete panels. This corresponds to vertical reinforcing strip spacing times [9.84 ft]. The number of strips required per tributary area is then provided across that area on the panels.

Internal Stability – Distribution of Reinforcements

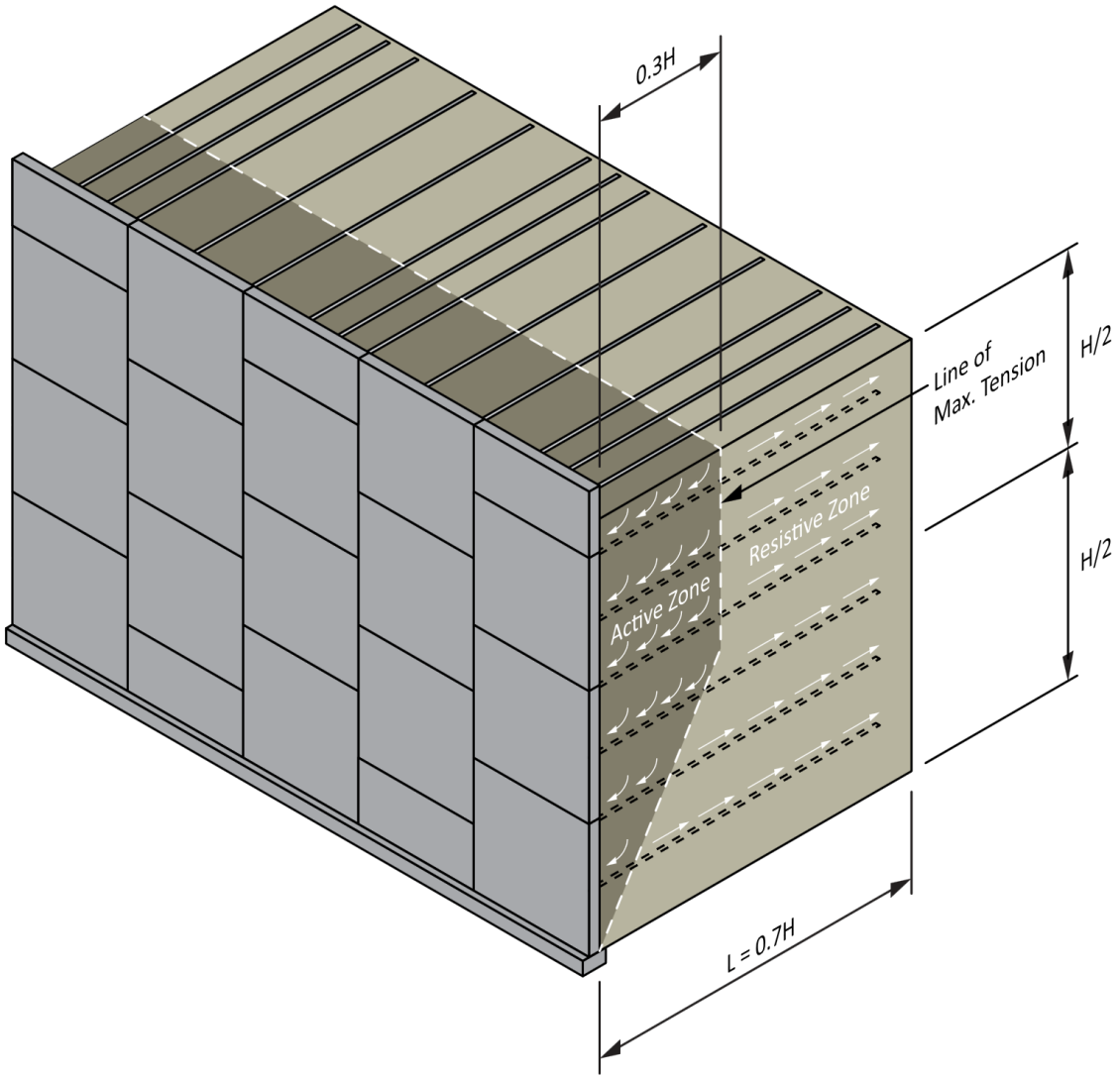


Figure 2.1.2-3: Isometric showing active and resistive zones

For a selected design section, the tension on all strips at a given level is equal because:

- The active zone is not acting over one panel width, but rather a continuous wedge acting over many panels along the length of wall.
- Since the width, length and coefficient of interaction for all of the strips on a given level resisting the active zone from moving out is the same, then the tension on all those reinforcements must also be the same, regardless of their spacing.
- The tension on the reinforcements is a result of the displacement of the active wedge

Internal Stability – Tributary Area

How is the number of strips per panel determined?

Once the horizontal stress transfer at a reinforcement level is determined, the number of strips required at that level is calculated for the applicable tributary area.

The tributary wall area used in the calculation above is typically 2.46' x 9.84' (24.2 sq ft) for Reinforced Earth structures using standard cruciform (4.92' x4.92'92'), square (4.92' x 4.92') or rectangular (4.92' x 9.84') panels. This corresponds to half a panel high (0.75 m [2.46 ft]), by two (4.92') panels, or one 9.84' wide [9.84 ft]. The number of strips required per tributary area is then provided across that area. Thus, as shown in Figure 2.1.2-3, a required number of strips N = 5 results in 3 reinforcements at the bottom of an A-5 panel and 2 across the top of an A-5 panel. For other panel types, the number of strips provided per panel is determined as the ratio of the actual panel area to the 24.2 SF tributary area.

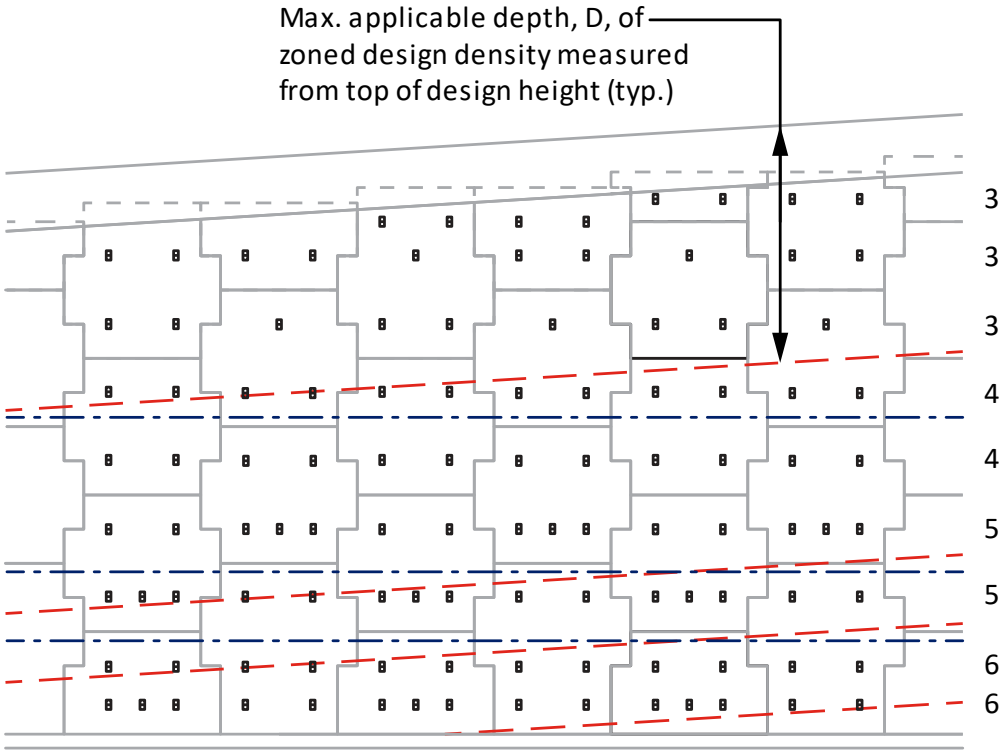


Figure 2.1.2-4: Reinforcing strip density based on tributary area

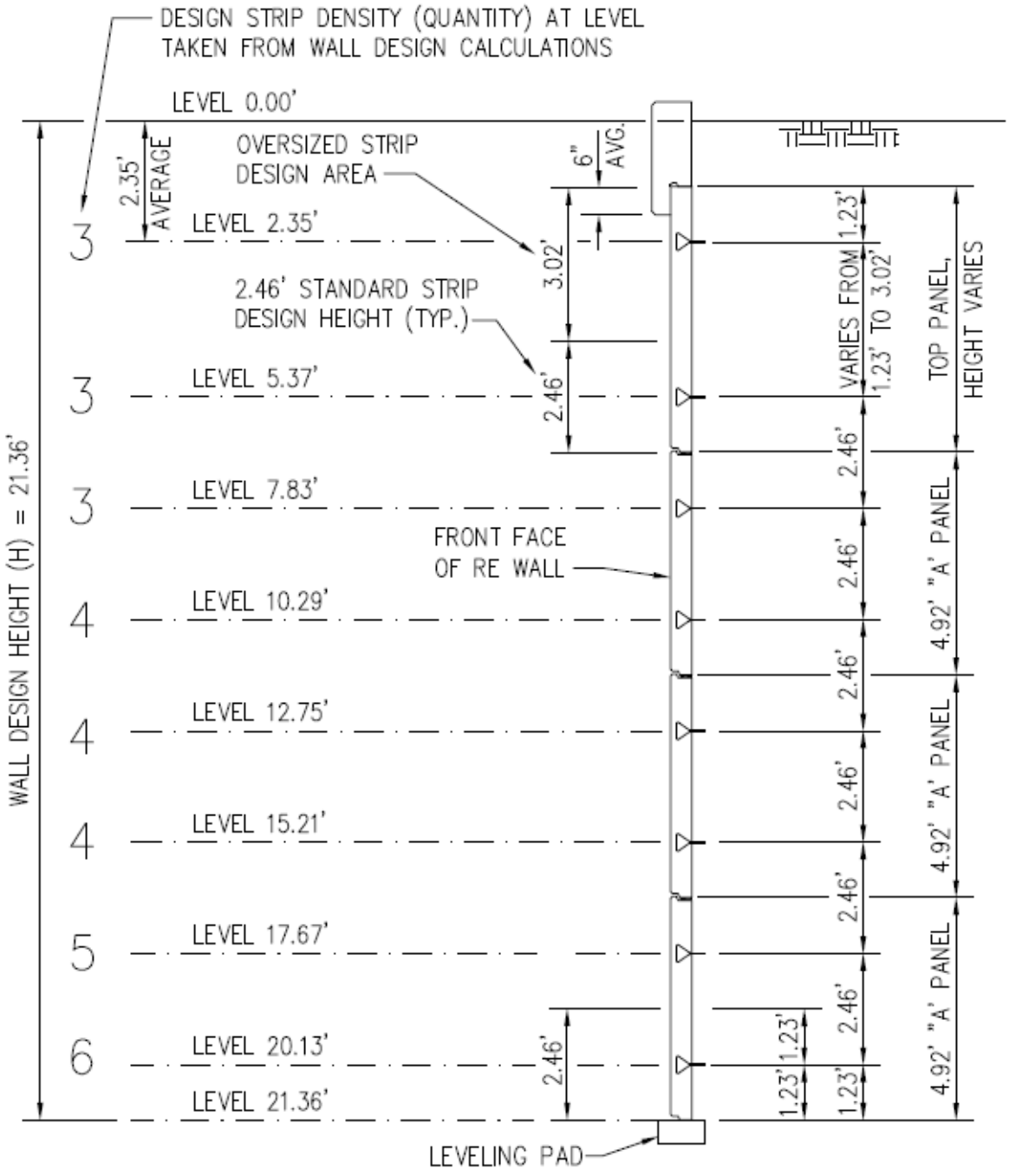


Figure 2.1.2-5: Reinforcing strip vertical spacing example

2.1.3 Obstructions in the Reinforced Soil Zone

Vertical and horizontal obstructions in the reinforced zone are accommodated in a variety of ways, depending on the type and geometry of the obstruction. Some common obstructions include:

- Drainage structures (inlets, pipes, etc.)

- Foundation elements (footings, piles, etc.)
- Railing, barrier, or fence posts behind the top of the wall.

Some typical methods of avoiding the above obstructions include:

- Splaying the soil reinforcement horizontally to the left or right. The single-bolt connection of the strips and ladders allow for a simple rotation of the reinforcement away from perpendicular to the wall face. If a significant number of strips or ladders are splayed, design calculations are provided to ensure that the wall's internal stability is maintained.
- Skewing the soil reinforcement vertically above or below the obstruction.
- Casting tie strips in non-standard locations on the facing units.
- Field-installing steel angles or "clip angles" on the back face of the panels.

Obstructions should be limited in size (whenever possible) as follows:

- Horizontal Obstructions (Pipes)
    - Limit outside diameter to 30". Keep in mind that AASHTO requires soil reinforcements to be spaced at a maximum vertical spacing of 33", so when pipes are greater than 30" in diameter, the presence of the pipe results in a conflict in reinforcement requirements.
- Vertical Obstructions
    - Deep Foundation Elements
        - Limit outside diameter to 36"
        - Provide a minimum of 18" (or 1.0 D), whichever is greater minimum clearance to back face of MSE wall
    - Inlet Structures
        - Limit Width to 6 ft

Obstructions with dimensions exceeding the above limits may be accommodated on a case by case basis, but should be avoided whenever possible.

Refer to **Appendix A2.1.3** for examples and additional details of the methods described in this section.

Effect of Splay Angle

In theory, splaying the soil reinforcement decreases the allowable tension that can be carried by the soil reinforcement . However, recent testing has shown that reinforcing strip splay up to 30 degrees has little effect on the capacity of the reinforcing strip and connection.

Historically, the splay of soil reinforcements has been limited to a maximum of 15 degrees with little effect. For larger splaying of the soil reinforcement, the allowable tension capacity is decreased. The allowable tension load is 9.76 Kips (75 year service Life) for soil reinforcements placed perpendicular to the panel. However, for a splayed soil reinforcements of 30 degees, the maximum allowable tension is decreased to 7.9 Kips. This value was determined from actual testing of the connection subject to loads from splayed soil reinforcement. The maximum tension load of 7.9 Kips is limited by the metal tie strip connection and not from a reduced pullout resistance of the reinforcement. In fact, data suggests that

skewing of reinforcements up to 30 degrees has little effect on the pullout resistance of the reinforcement.
Typically one soil reinforcement may be splayed up to 30 degrees per row (on a 5'x5' panel, or two per row on a 5'x10' panel), the remaining soil reinforcement should be placed perpendicular to the panel (see below):

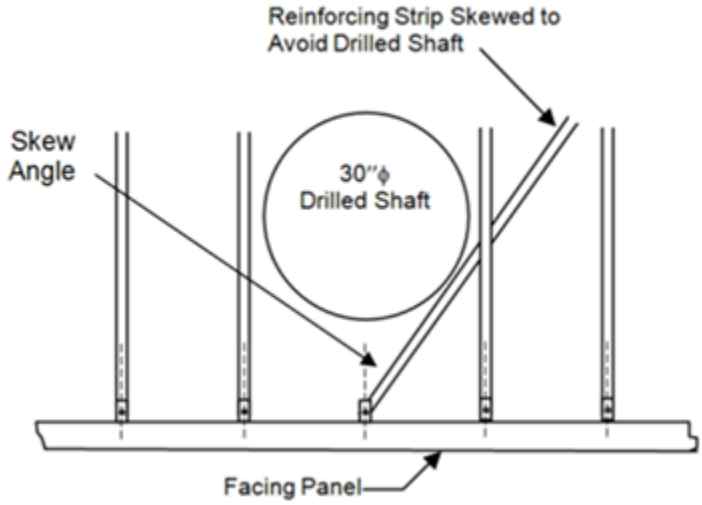


Figure 2.1.3-1: Reinforcing strip skew at drilled shaft

Per AASHTO 11.10.10.4:

> *3) If the soil reinforcements consist of discrete strips and depending on the size and location of the obstruction, it may be possible to splay the reinforcements around the obstruction…..For the third alternative, the splay angle, measured from a line perpendicular to the wall face, shall be small enough that the splaying does not generate moment in the reinforcement or the connection of the reinforcement to the wall face. The tensile resistance of the splayed reinforcement shall be reduced by the cosine of the splay angle.*
>
> *C11.10.10.4 Typically, the splay of reinforcements is limited to a maximum of 15 degrees*

Pullout capacity of Splayed Reinforcing Strips from backfill

Based on research by RECo and at the Texas Tech University (See **Appendix A1.2.1**), the reduction in pullout capacity for reinforcing strips skewed at 15 degrees or less does not need to be considered. In fact, due to passive resistance on the edge of the reinforcement, the pullout resistance often increases when a reinforcing strip is skewed. Data also suggests that skewing of reinforcements up to 30 degrees has little effect on the pullout resistance of the reinforcement.

	Pullout Resistance Factor, f* (Ribbed Reinforcement) - Summary of Test Results								
	Skew = 0 deg.			Skew = 15 deg.			Skew = 30 deg.		
DOF (ft)	No	Mean	S.D.	No	Mean	S.D.	No	Mean	S.D.
5	21	4.87	0.8	3	9.7	1.01	3	6.16	1.02
12	14	3.02	0.41	5	3.82	0.72	4	2.04	0.56
20	12	1.98	0.53	-	-	-	-	-	-
40	10	1.14	0.31	-	-	-	-	-	-

Table 2.1.3-1: Pullout Resistance of Straight and Skewed Strips (Ref: Effect of Skewing and Splaying on Pullout Capacity of Steel MSE Reinforcement, Jayawikrama, et al, 2011)

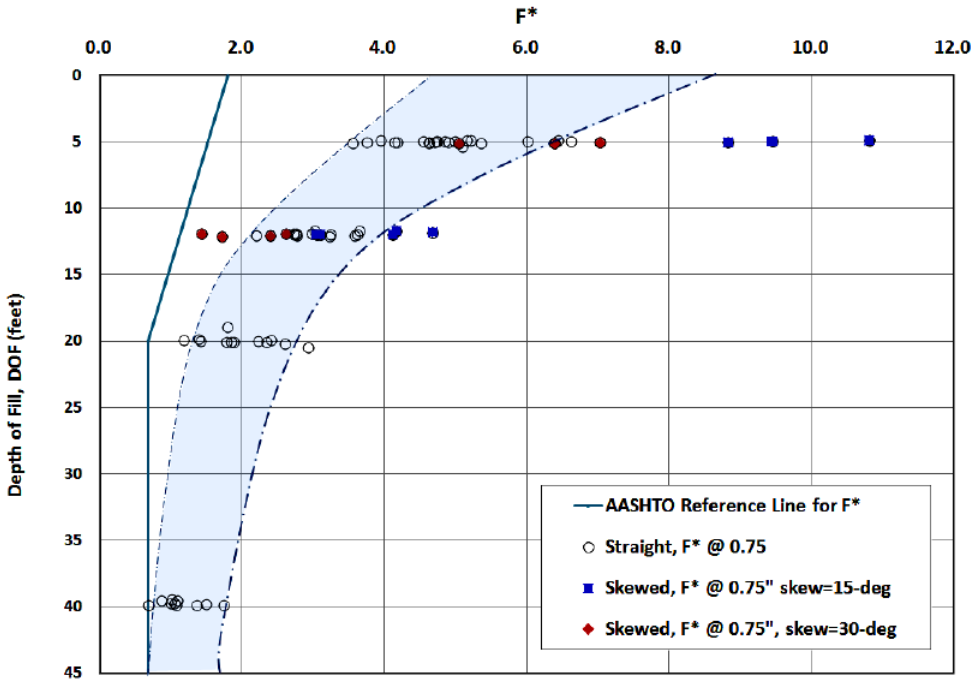


FIGURE 9. F* versus Depth of Fill: Ribbed Steel Strip Reinforcement

Figure 2.1.3-2: Pullout Resistance of Straight and Skewed Strips (Ref: Effect of Skewing and Splaying on Pullout Capacity of Steel MSE Reinforcement, Jayawikrama, et al, 2011)

Pullout capacity of Tie Strips from Concrete Panels – considering Splayed Reinforcing Strips

Test Series	No. of Samples	f'c (psi)	Avg. Failure Load (lbs)	Confinement	Skew	$\frac{\sqrt{5{,}000}}{\sqrt{(f'c)}}$	$\frac{\sqrt{4{,}000}}{\sqrt{(f'c)}}$	$N_{u,\,5,000}$	$N_{u,\,4,000}$	$\phi N_{u,\,5,000}$	$\phi N_{u,\,4,000}$
SGI 2021	5	7200	18047	N	N	83%	75%	15039	13451	11279	10088
SGI 2021	5	8020	17640	Y	N	79%	71%	13928	12458	10446	9343
SGI 2021	5	8040	16052	Y	15	79%	71%	12659	11322	9494	8492
SGI 2021	5	8030	14394	Y	30	79%	71%	11358	10159	8519	7619

Table 2.1.3-2: Pullout Resistance of Tie Strips for Straight and Splayed Reinforcing Strips

Based on test results, the ultimate pullout capacity of the tie strip is reduced by 9% when the reinforcing strip is skewed 15° (16,052/17,640) and 18% when the reinforcing strip is splayed 30° (14,394/17,640). Based on these test results, the effect of splayed reinforcements on pullout capacity of the tie strip only needs to be evaluated when the skew exceeds 15° and 28 day compressive strength of concrete is less than 5,000 psi. Note that RECo has conducted an audit of concrete mix designs used throughout the United States and approximately 90% exceed 5,000 psi 28 day compressive strength.

Potential for Bending Moment in Tie Strips due to Splayed Reinforcing Strips

With regard to the potential for a moment to be induced on the tie strip due to the splay of the reinforcement, additional tests were performed by SGI with special samples of tie strips where the exposed section was ground to a thickness of 1.8mm +/- per flange. The results of the pullout test show that the failure loads at 30-degree splay angle was only 2% less than the straight pull, which is within the range of variation associated with testing and thus a significant moment was not induced.

Test Series	No. of Samples	Average Failure Load (lbs)	Confinement	Skew	Comments
SGI 2021	5	10940	Y	N	Failure of Steel Section
SGI 2021	5	10698	Y	30	Failure of Steel Section

Table 2.1.3-3: Tie Strip Failure Loading – 1.8mm flange thickness





Straight Pull Failure at 10.9 kips 30 Degree Splay Failure at 10.7 kips

Figure 2.1.3-3: 1.8mm flange thickness with straight and 30 degree splayed reinforcing strips

Check Failure Stress – Net Section:

10.9 k / 0.20 in^2 = 54.5 ksi. This is reasonable given that the grinding operation was not exact and some parts of the section may have been less than 0.07" thick. A variation of 1/16" would lead to the reduced thickness that equates to a failure at 65 ksi.

Check Effect of Splayed Strip and Induced Bending:

Calculate the capacity of a tie strip with 1.8mm (0.07 in.) flange thickness, subject to a 30 degree splayed strip with 10.7 kips applied force. The following simplified approach for calculating the bending stress does not incorporate the effects of soil confining pressure on the connection and treats the bolted connection as fixed:

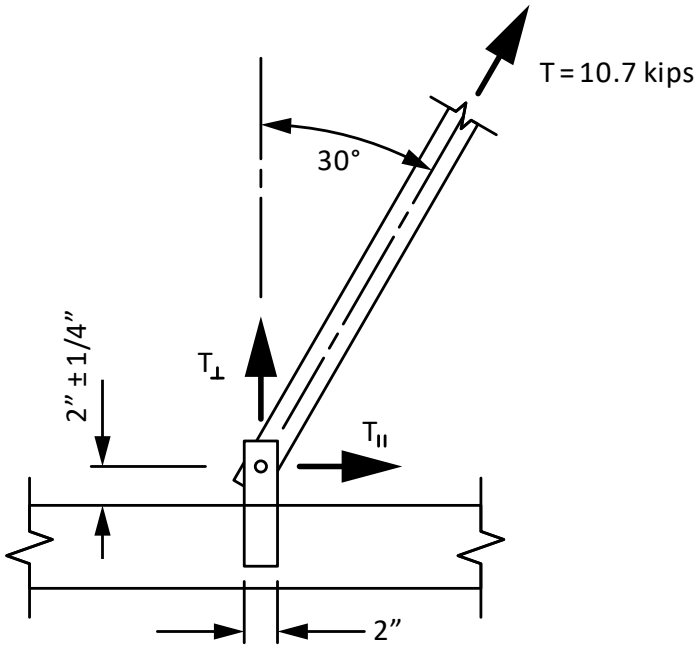


Figure 2.1.3-4

$T_{\perp}$ = 10.7 Cos 30° = 9.27 kips
T_{α} = 10.7 sin 30° = 5.35 kips
Tension = 9.27 kips; Shear = 5.35 kips
Moment at Failure= M_f = 5.35 kips x 2.25" * = 12.0 k-in.

Gross Sectional Area A_{sg} = 0.07 in x 2" x 2 Plates = 0.28 in^2
Net Sectional Area A_{sn} = 0.07 in x (2"- 9/16") x 2 Plates = 0.20 in^2
b = plate thickness = 0.07"

Section Modulus = $S = \frac{bt^2}{6}\ x\ 2\ plates = \frac{0.07\ x\ 2^2}{6}\ x\ 2 = 0.093\ in^3$

Stress at extreme fibers = $\sigma = \frac{T}{A} + \frac{M}{S} = \frac{9.27}{0.28} + \frac{12.0}{0.093}$ = 163 ksi > 50 ksi

Check Plastic Moment: Mn=Mp=FyZ < 1.6My (AASHTO LRFD 6.12.2.2.7-1)

Z = $\frac{bt^2}{4} = \frac{0.07\ x\ 2^2}{4}\ x\ 2 = 0.14\ in^3$

Mp = 50 ksi x 0.14 in^3 = 7 k-in < Calculated Moment at Failure (12.0 k-in) by 70%.

My = Fy S = 50 ksi x 0.093 in^3 = 4.65 k-in. Mp < 1.6 My.

** 2" is the design dimension. 2.25" was used to back calculate capacity based on actual test sample as built dimension (which falls within fabrication tolerance)*

The fact that ruptures were occurring at the bolt hole (with no potential moment) and at the gross section (with some potential moment) at about the same load, shows that the theoretical bending stress calculated above is too high. The simplified method of analysis is conservative because it does not consider the benefit of the soil confining pressure and the bolted (i.e, not fixed) connection.

For a standard 10 gage tie strip, 75-year service life. T = 9.76 k. t = 2mm (0.079") after metal loss

T_F = 9.76 Cos 30° =8.45 kips
T_α = 9.76 sin 30° = 4.88 kips
Tension = 8.45 kips; Shear = 4.88 kips
Moment = 4.88 kips x 2.0" = 9.76 k-in.

Gross Sectional Area A_{sg} = 0.079 in x 2" x 2 Plates = 0.315 in^2
Net Sectional Area A_{sn} = 0.079 in x (2"- 9/16") x 2 Plates = 0.227 in^2
b = plate thickness = 0.079"
Section Modulus = $S = \frac{bt^2}{6}\ x\ 2\ plates = \frac{0.079\ x\ 2^2}{6}\ x\ 2 = 0.105$ in^3

Check Plastic Moment: Mn=Mp=FyZ < 1.6My (AASHTO LRFD 6.12.2.2.7-1)

Z = $\frac{bt^2}{4} = \frac{0.079\ x\ 2^2}{4}\ x\ 2 = 0.158$ in^3

Mp = 50 ksi x 0.158 in^3 = 7.9 k-in

My = Fy S = 50 ksi x 0.105 in^3 = 5.27 k-in. Mp < 1.6 My (8.43 k-in)

Back Calculate the allowable tension for strips splayed @ 30 degrees:

7.9 k-in / 2 in = 3.95 k
3.95 k / sin 30° = **7.9 k**

Conclusion – Splayed Reinforcing Strips

To be in accordance with AASHTO requirements per section 6.12.2.2.7-1, limit the maximum tension on strips splayed 30 degrees to 7.9 kips (approximately 80% of maximum design strength 9.76 kips). This approach is conservative as it is shown above that the actual induced bending stress is less than calculated. If panel design concrete strength is less than 5,000 psi (4,000 psi Min.), then reduce design capacity to 7.6 kips per the data presented in Table 2.1.3-2)

2.2 Design Example

Two example design cases are included in the appendices. A level surcharge and a sloped surcharge design case, each with a 75-year design life for a 30-ft tall wall.

2.2.1 Problems 1 & 2

Complete calculations for Problems 1 and 2 can be found in **Appendix A2.2.1**. In addition to using MSEW+, a second set of calculations are provided using RECo's proprietary design software.

MSEW+ performs Coherent Gravity calculations in accordance with AASHTO 2017-2020. However, the following differences between MSEW+ and RECo's design methodology should be noted:

- For slope surcharges, MSEW uses the unit weight of select fill for the material on slope. RECo program uses the unit weight of retained fill for the material on slope. Normally, this is not a big difference, but it does result in some variance between the two calculation methods.
- The inflection point for K and f* = 19.7 (6m) in MSEW, whereas RECo inflection point is 20.
- There are often minor differences between MSEW and RECo in the calculated loads and resistances in the upper 2 layers of reinforcement and bottom layer of reinforcement. RECo uses a tributary area approach for each reinforcement layer. These minor differences are typically inconsequential but may result in an addition reinforcement on the bottom layer.

3.0 Construction

The following section describes the construction procedure for a typical Reinforced Earth wall.

3.1. Construction Procedures

In brief, the procedure for constructing a Reinforced Earth wall involves repeating the following steps:

1. Prepare the foundation.
2. Pour a 6"x12" unreinforced concrete leveling pad
3. Set and brace the first course of facing panels with a slight inward batter (typically ½" over 4').
4. Place and compact the select backfill in lifts.
5. When the backfill reaches a tie strip level on the panels, connect the row of reinforcing strips or ladders.
6. Compact the backfill on top of the row of installed strips or ladders.
7. Set and brace the next course of panels when required.
8. Repeat the above steps until the wall is complete.

Refer to **Appendix A3.1** for RECo's marketing flyer giving a brief overview of construction.

3.1.1 Innovations

One innovations are claimed for construction.

MSE walls on marginal foundations:

The inherent flexibility of Reinforced Earth MSE walls allow for construction on sites with soft soils. In fact, the very first RE wall constructed in the US settled over 3 ft. Single-stage Reinforced Earth walls can be constructed on marginal foundations with the use of phased construction, surcharges, lightweight fills, ground improvement, or some combination of these methods.

More information can be seen at https://reinforcedearth.com/blog/accommodating-foundation-soil-settlement-with-retaining-walls/

Refer to RECo Technical Memo 2020-01 in **Appendix A2.1** for further discussion.

3.1.2 Construction Manual

RECo has several versions of construction manuals for this system, depending on the panel type being used. For simplicity, we have included one manual, which outlines construction procedures for walls using square facing units. A complete construction manual for RECo's Reinforced Earth wall system can be found in **Appendix A3.1.2**. Updates to our manuals are posted on our website at https://reinforcedearth.com/resources/construction-manuals/

3.1.3 Construction Limitations for Curves and Corners

Where there are curves and corners, the wall can be installed according to the design without limitations other than the possibility that alternative backfill compaction methods and equipment may be required in locations with a tight inside radius or corner. Smaller compaction equipment may be needed, or it may be beneficial to use open-graded crushed stone backfill in these areas or for the entire structure. A crushed stone will require minimal compaction effort compared to a sandy or more well graded backfill.

Other limitations involving curves and corners are design-related and discussed in **Section 1.1.11**.

3.1.4 Construction Procedures for Curves and Corners

The general procedure for installing the earth reinforcement (strips and ladders) does not change based on the alignment of the wall. The strips and ladders are connected to the facing with the typical bolted connection described in **Section 1.2.8**.

Strip and ladder placement may vary depending on the horizontal alignment:

- Curved section of wall:
  - For a curved section of wall, the strips and ladders are placed perpendicular to the wall facing, similar to a conventional straight wall. Depending on the radius of the curve, the ends of the reinforcements may overlap on the free end.
- Corners (90 degree or wider):
  - The strips and ladders are once again placed perpendicular to the wall facing, similar to a conventional straight wall.
- Corners (Less than 90 degrees):
  - A corner of less than 90 degrees may require horizontal splaying of the reinforcements away from perpendicular to avoid the end of the reinforcement being obstructed by the wall facing on the other side of the corner.
- Acute Corners (Less than 60 degrees)
  - Soil reinforcement placement for acute corners typically require a bin-wall type design. The nose of the corner is connected to both sides of the corner for a horizontal distance away from the corner until the reinforcements no longer need to be splayed to avoid the other side of the facing.

- In all cases, it is not necessary to provide fill to separate overlapping reinforcing strips in corners. This is because any overlap between discrete reinforcing strips or ladders results in a minimal (less than 5%) reduction in surface contact with select backfill. Furthermore the beneficial effect of the transverse reinforcements (i.e. the reinforcements coming from the other wall face) increasing the composite shear strength of the reinforced soil mass is ignored in the internal stability design.

3.1.5 Vertical and Horizontal Alignment

The construction procedures include several measures to maintain the design vertical and horizontal alignment of the wall face. In general, properly installing the bottom course of panels provides the basis for alignment of the wall. The following measures are described in more detail in the Construction Manual provided in **Appendix A3.1.2**.

- Leveling pad
  - The leveling pad is unreinforced concrete (6" x 12" section) that is smoothed, leveled, and stepped where necessary. The leveling pad provides a smooth and level surface to set the bottom course of panels.
- Chalk or string line
  - This ensures that the first course of panels follows the horizontal alignment shown on the design drawings.
- EPDM (synthetic rubber) shims, wooden wedges, wooden clamps, and bracing
  - These items are construction equipment and tools used to ensure proper alignment of the bottom row of facing panels. Clamps are also used for subsequent rows.
- EPDM (synthetic rubber) bearing pads
  - Bearing pads are placed on top the of panels, providing consistent spacing for the horizontal joints, and adding flexibility to the wall face.
- Wooden spacers
  - Wooden spacers help align adjacent panels on the bottom row to ensure the vertical joints are consistent and wall does not shorten or elongate along the horizontal alignment
- Panel batter
  - When panels are set, they are battered toward the backfill. When the backfill is compacted behind the panel, the compaction energy and backfill consolidation pushes the panel out to its final vertical alignment, and engages the reinforcements.
- External Bracing on first course of panels
  - The first course of panels is braced externally, with batter achieved prior to nailing braces
- Wooden Clamps
  - Panels are clamped together at each course after placement and alignment. Clamps are removed prior to setting next course of panels after reinforcing strips have been backfilled and compaction completed
- Soil reinforcement placement

- Soil reinforcements are recommended to be placed with the free end approximately 2 inches higher than at the facing connection point. This ensures that differential settlement perpendicular to the wall does not affect the alignment of the wall facing.

- Compaction
  - Standard compaction requirements and methods are used in part to ensure that the wall alignment is consistent throughout the structure.

3.1.6 Reinforced Soil Installation

The reinforced soil (select granular backfill) is placed and compacted in lifts of over the full height of the wall. Lifts are placed with a loose thickness of no more than 12 inches. Compaction is achieved by using a large smooth-drum vibratory roller, except within a 3-ft zone directly behind the facing. In the 3-ft zone, a small hand-operated vibratory compactor must be used to control panel movement.

Select backfill installation steps are described in more detail in the Construction Manual provided in **Appendix A3.1.2**.

3.1.7 Erosion Prevention During Construction

During construction, it is the contractor's responsibility to control stormwater runoff and drainage in the vicinity of the wall to prevent erosion. In general, the site should be maintained in such a way that water is collected and discharged away from the wall facing and select backfill mass. Following are standard notes from RECo plans:

1. *The Contractor is responsible for controlling storm water drainage in the vicinity of the wall during construction. Storm water runoff is to be collected and discharged away from the wall and the reinforced backfill mass. MSE wall backfill shall not be placed or traveled upon when saturated. Compacted backfill shall be re-worked or allowed to dry when the moisture content exceeds optimum by 2% or more.*

2. *The finished grade backfill in front of the wall shall be placed and compacted as soon as possible (as construction sequence permits to allow for bottom panels to move towards a plumb position) to mitigate the potential for erosion and undermining of the concrete leveling pad. In any event, backfill in front of the wall shall be placed and compacted before wall construction exceeds a height of 20.0 feet. Finished grade backfill shall be placed in lifts and properly compacted as required by project specifications.*

3.1.8 Required Experience and Qualifications for Wall Installer

The Reinforced Earth Company does not prequalify contractors, as this is the responsibility of the owner. MSE wall installers shall meet the owner's requirements for licensing, insurance, experience, financial requirements, etc.

In addition to owner's prequalification requirements, the suggested minimum technical requirements for MSE wall installers are as follows:

The foreman (or other person in responsible charge of MSE wall installation) should have experience in earthworks, compaction methods and the ability to read and understand the plans and specifications. Experience with line and grade alignment of concrete formwork, pipe or precast unit installation is necessary for proper installation of precast facing panels.

The contractor's foreman should be provided the wall installation plans, project specifications and RECo construction manual at least 3 days prior to the scheduled start of construction to review the documents and prepare for installation.

The contractor should utilize a licensed surveyor for all wall layout working points, establishment of elevation data points, etc.

Project Specifications should include a pre-construction meeting requirement with the Wall Designer/Supplier, Wall Installer, General Contractor, and Owner's Representatives to discuss MSE wall plans, specifications, backfill and compaction requirements, phasing, storage and handling of materials, etc. The meeting can be virtual with a follow-up site visit by the wall designer/supplier representative when construction begins.

Finally, the owner shall provide inspection personnel to ensure the plans and specifications are followed.

4.0 Quality Control

Construction quality control consists of the following:

1. Alignment Control. By use of bracing and wedging, along with periodic survey control of wall alignment. Maintaining line and grade are key in any retaining wall construction activity, and daily checking of installation batter to ensure panels are achieving plumb during the construction process.
2. Compaction Control. By use of the proper lift thickness, moisture content and equipment for the type of backfill utilized. Compaction monitoring shall be performed in accordance with project and AASHTO requirements. Typically, a minimum of 95% Standard Proctor Density is required for select granular fill placed 3 ft or more behind the face of wall. For the 3 ft zone closest to the wall, compaction shall be achieved with at least 3 passes of a lightweight (less than 1000 lbs) walk behind plate compactor.
3. Plan and Specification Compliance. Note that The Reinforced Earth Company is not the inspector. The owner and/or designated third party inspection agency is responsible for quality assurance.
4. See the Construction Manual provided in **Appendix A3.1.2** for additional information.

MSE Walls are simple to construct but require consistent supervision of work by the contractor to ensure quality control and informed inspection by the owner to provide quality assurance. The Reinforced Earth Company recommends that Owner's inspectors should have experience in MSE wall inspection, and it is highly recommended for inspectors to take the FHWA MSE Wall inspection course.

4.1 Manufacturing

The concrete facing units are manufactured either by one of RECo's precast concrete facilities or by a precast partner within a reasonable distance to the project.

The HA Strip soil reinforcements are rolled at CMC (Magnolia, AR) and/or Nucor (Jackson, MS), then fabricated at RECo's plant in Houston, Texas. The 50mm wide ribbed steel strips arrive in lengths of 48.5' from the steel mill, and RECo then cuts the reinforcements to length and punches the holes at the ends of the reinforcement. Galvanization is performed by certified galvanizing plants.

The HA Ladder soil reinforcements are manufactured by a partner in Bellwood, Illinois.

All other components are manufactured by a third party.

4.1.1 Facing Unit Quality Control

The concrete utilized in the precast of the units is tested in accordance with the specifications of the state Department of Transportation where the project is located. Precasters working for the Reinforced Earth Company are at a minimum approved by the state DOT, and in many cases NPCA (National Precast Concrete Association) or PCI (Precast/Prestressed Concrete Institute) certified.

RECO Q/A requirements for precast concrete facing unit fabricators consist of the following:

1. Precast fabricators working for RECo are typically NPCA or PCI certified, and if the project is a state DOT project, the fabricator shall be DOT approved. Since we generally work with a network of established plants, non-DOT projects are also provided DOT quality precast.
2. Quality control of precast items is provided by the precaster. Certificates of compliance with project specifications are provided by the precaster as required by project specifications.
3. Quality assurance of precast items is provided by owner's inspector, or third-party inspector as required by project specifications.
4. Steel Forms for precast panels, corner elements, slip joints and copings are provided by RECo. Periodic inspection and routine maintenance are provided by precaster. When casting tolerances cannot be met, forms are removed from production and sent to RECo forms maintenance facility.
5. Precasters are provided a list of panels to be fabricated for the project, along with the wall installation drawings. Precast production is tracked using RECo on-line fabrication reporting software.
6. Unless panels are on DOT pre-approved list, precast fabricators are provided panel fabrication drawings for each project.
7. RECo provides precast manual for fabricators and in plant training for new precasters.
8. Periodic plant audits are made by RECo project managers and RECo National Plants Manager(s). The RECo Project Manager and National Plants Manager will review fabrication set up as well as fabricated products and will pursue corrective action with the precaster for any items not meeting specifications.

See **Appendix A4.1.1** for Precast Concrete Panel Quality Control Manual

4.1.2 Soil Reinforcement Quality Control

The manufacturer provides certificate of compliance as required by project specifications for both manufacturing of steel reinforcement and hot-dipped galvanization.

See **Appendix A4.1.2** for Quality Control of Soil Reinforcement.

4.1.3 Other Components Quality Control

Select backfill: Quality control is supplied by the contractor. Requirements of quality control depend on the state Department of Transportation where the project is located. The minimum compaction requirements shall be 95% of Standard Proctor Density with a target moisture content of 1 to 2% dry of optimum.

Backfill shall meet project specifications. Oversight of backfill suppliers is by the contractor and/or owner, not RECo.

Backfill test results are provided by the contractor to the owner for review. When questions arise regarding backfill suitability with respect to project specifications, RECo may be provided backfill test results to confirm design parameters for select fill are applicable for project backfill.

Other Components: Wall components not mentioned above are manufactured by a third party. The manufacturer provides certificate of compliance as required by project specifications for both manufacturing of steel reinforcement and hot-dipped galvanization.

4.2 Construction

4.2.1 Quality Control Measures During Construction

Quality control during construction is essential to ensure proper alignment and overall performance of the structure. Quality control measures for construction are included in the Construction Manual provided in **Appendix A3.1.2**.

In addition, various checklists have been developed by DOTs and FHWA for inspection. Following are links to some examples:

Florida DOT

https://www.fdot.gov/docs/default-source/construction/CONSTADM/Guidelist/InspectGuidelist/FY1213/MSE-Wall-Inspectors-Handbook.pdf

Delaware DOT

https://cpb-us-w2.wpmucdn.com/sites.udel.edu/dist/1/1139/files/2013/10/Rpt.-143-Mechanically-Stabilized-Earth-Wall-Final-Leshchinsky-yxyg38.pdf

5.0 Performance

The Reinforced Earth wall system performs as a coherent gravity retaining wall structure. The system has a unique level of flexibility and an effective and efficient load carrying ability. Over 50,000 Reinforced Earth MSE walls have been constructed in the US since 1971, and over 100,000 structures worldwide totaling over 700,000,000 SF.

https://reinforcedearth.com/who-we-are/news/reco-usa-50th-anniversary/

5.1 Performance History

Over the past 50 years there have been several opportunities to evaluate the long-term performance of Reinforced Earth walls. In addition to the excellent performance of RE walls in normal service conditions, RE walls have shown excellent performance in extreme events such as earthquakes, floods, fire, explosions, and on sites with significant foundation settlements.

In all cases where there have been issues with performance, the problem has been attributed to one or more of the following:

- Inadequate site drainage
- Improper Backfill
- Unforeseen foundation conditions
- Failure of utility within reinforced soil zone
- Poor construction quality
- Unanticipated super flood and/or tsunami

A sampling of the bibliography of performance related studies is found below:

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5.1.1 Development and Usage History

The Reinforced Earth system was developed during the 1950s and patented in 1963 by the inventor of modern mechanically stabilized earth, Henri Vidal. He introduced the system to the United States in 1971. Continued evolution and improvement of the facing, soil reinforcements, and other components have taken place. The modern system with details and components as described in this document can be dated to 1979.

The first structure was built in 1971 for Caltrans and used a steel facing with flat steel reinforcing strips bolted to the back face. The cruciform-shape concrete facing was used on the projects immediately following, while the steel facing was phased out in the US during the 1970s.

The first architectural finish on Reinforced Earth Panels was in 1973, with the use of timbers placed in forms on the Little River Road project in the Great Smokey Mountains National Park.

The first use of form liners on Reinforced Earth facing panels was in 1975 on the Clark Fork Highway (ID).

Special Curved Facing Units were used on the Vail Pass (CO) in 1975.

The ribbed reinforcing strips were introduced in 1978. Further research and development led to optimizing and adjustments of the strip dimensions and grade of steel to those presented in this document.

EPDM Bearing Pads were introduced in 1984.

Precast Traffic Barriers were introduced in 1985.

Wire Faced "Terratrel™" Reinforced Earth walls were introduced in 1981.

Rectangular Facing Panels were introduced in 1992.

Square Facing Panels were introduced in 1997.

Alignment pins on cruciform panels were phased out of use in the mid 1990's.

Ladder soil reinforcements were introduced in the 1980's. HA ladders as described in this document were introduced in 2000.

Design Guidance History

In addition to the internal design documents developed by Terre Armée and The Reinforced Earth Company, the following is a brief history of the major design guidance documents issued for Reinforced Earth Structures (For international references, we are only listing the original 1979 SETRA document, but note that several international codes now exist, most notably the British Standard and the Eurocode).

In 1979, the French Ministry of Transport (SETRA) issued "Reinforced Earth Structures, Recommendations and Rules of the Art".

In January of 1990, FHWA issued "Durability /Corrosion of Soil Reinforced Structures" (Elias)

In 1990, FHWA issued "Reinforced Soil Structures – Volume 1, Design and Construction Guidelines – Volume II, Summary of Research and Systems Information" (Christopher, Gill, Giroud, Mitchell, Schlosser and Dunnicliff). This document has been updated several times, most recently in 2009 (FHWA-NHI-09-083).

In August of 1990, AASHTO-AGC-ARTBA Joint Committee Subcommittee on New Highway Materials issued the Task Force 27 Report "In Situ Soil Improvement Techniques" which included: Guidelines for Design of MSE Walls and Construction Specifications for MSE Walls.

In 1991, AASHTO issued the first MSE Wall Specifications. Since that time, there have been many updates to the specification, and RECo has been involved in review and providing industry feedback to AASHTO as the specification is updated.

5.1.2 Oldest Three Structures

The following is a listing of the oldest RECo structures in the United States, as well as the oldest that use the present day soil reinforcements discussed in this document:

- Oldest structures:
  - RECo project numbers 1, 7, 14 and 16
- Oldest structures using current ribbed reinforcing strips:
  - RECo project numbers 478, 482 and 1584
- Oldest structures using HA ladder soil reinforcements:
  - RECo project numbers 7673 and 7856

Refer to **Appendix A5.1.2** for additional information and images for each project.

5.1.3 Tallest Three Structures

The following is a list of the three tallest tiered and vertical RE walls in the United States. See **Appendix A5.1.3** for more information.

- Tiered:
  - RECo project number 8079 (134.5 feet)
  - RECo project number 4130 (105 feet)
  - RECo project number 6253 (98.5 feet)
- Vertical
  - RECo project number 537 (110 feet)
  - RECo project number 18408 (110 feet)
  - RECo project number 7818 (79 feet)

For additional reference, the three tallest tiered and vertical RE walls outside the USA:

- Tiered:
  - South Africa, De Beers CDM Oranjemund "Tweepad", 1979, 138 feet tall (42.0 m)
  - South Africa, Mbabane Bypass – Mangwaneni Fill, 2008, 118 feet tall (36.0 m)
  - South Africa, Sishen Primary Crusher, 2005, 112 feet tall (34.0 m)
- Vertical:
  - Spain, VTE C-820 & C- 822.TR: Santiago Del Teide – Adeje, 2008, 74 feet tall (22.5 m)
  - Spain, Autopista del Atlantico Muro 1, 1991, 76 feet tall (23.3 m)
  - Saudi Arabia, King Abdulazziz Military Academy Riyadh, 1981, 72 feet tall (22.0 m)

5.1.4 Private and Public Approvals

The Reinforced Earth system is approved by every state DOT. In addition, dozens of other agencies (cities, counties, port authorities, etc.) have approved the Reinforced Earth MSE wall system. Private approvals tend to be on a project basis.

Federal Level public approvals:

- United States Army Corps of Engineers
- United States Department of Defense

State level public approvals:

Alabama Dept. of Transportation	**Missouri Dept. of Transportation**
Alaska Dept. of Transportation and Public Facilities	**Montana Dept. of Transportation**
Arizona Dept. of Transportation	**Nebraska Dept. of Transportation**
Arkansas Dept. of Transportation	**Nevada Dept. of Transportation**
California Dept. of Transportation (Caltrans)	**New Hampshire Dept. of Transportation**
Colorado Dept. of Transportation	**New Jersey Dept. of Transportation**
Connecticut Dept. of Transportation	**New Mexico Dept. of Transportation**
Delaware Dept. of Transportation	**New York State Dept. of Transportation**
District Dept. of Transportation (Washington, DC)	**North Carolina Dept. of Transportation**
Florida Dept. of Transportation	**North Dakota Dept. of Transportation**
Georgia Dept. of Transportation	**Ohio Dept. of Transportation**
Hawaii Dept. of Transportation	**Oklahoma Dept. of Transportation**
Idaho Dept. of Transportation	**Oregon Dept. of Transportation**
Illinois Dept. of Transportation	**Pennsylvania Dept. of Transportation**
Illinois State Toll Highway Authority	**Rhode Island Dept. of Transportation**
Indiana Dept. of Transportation	**South Carolina Dept. of Transportation**
Iowa Dept. of Transportation	**South Dakota Dept. of Transportation**
Kansas Dept. of Transportation	**Tennessee Dept. of Transportation**
Kentucky Transportation Cabinet	**Texas Dept. of Transportation**
Louisiana Dept. of Transportation and Development	**Utah Dept. of Transportation**
Maine Dept. of Transportation	**Vermont Agency of Transportation**
Maryland Dept. of Transportation State Highway Administration	**Virginia Dept. of Transportation**
Massachusetts Dept. of Transportation	**Washington State Dept. of Transportation**
Michigan Dept. of Transportation	**West Virginia Dept. of Transportation**
Minnesota Dept. of Transportation	**Wisconsin Dept. of Transportation**
Mississippi Dept. of Transportation	**Wyoming Dept. of Transportation**

Table 5.1.4

Other regional public agency approvals:

- Chicago Department of Transportation (CDOT)
- North Texas Tollway Authority (NTTA)
- Dallas Area Rapid Transit (DART)
- Harris County Tollway Authority (HCTRA)
- Central Texas Regional Mobility Authority (CTRMA)
- Charlotte Area Transit System (CATS)
- Massachusetts Bay Transportation Authority (MBTA)

6.0 Other Information

6.1 Any Other Helpful Information

This section includes information that has not been specifically requested. RECo has additional valuable information on patents and instrumentation of structures for near and long-term evaluation. In addition, RECo has input on the designation of responsible parties for different aspects of a successful RE wall project over the entire life of the structure, based on experience and collaboration with The Association for Mechanically Stabilized Earth (AMSE).

6.1.1 Patents

Terre Armee holds hundreds of worldwide patents. In the USA, Terre Armee and RECo have been granted dozens of patents related to Reinforced Earth MSE walls, including the following:

The original Reinforced Earth system consisted of elliptical shaped steel facing, to which, smooth steel strips were bolted. The first structure of this type was built in 1965 in Pragneres, France.

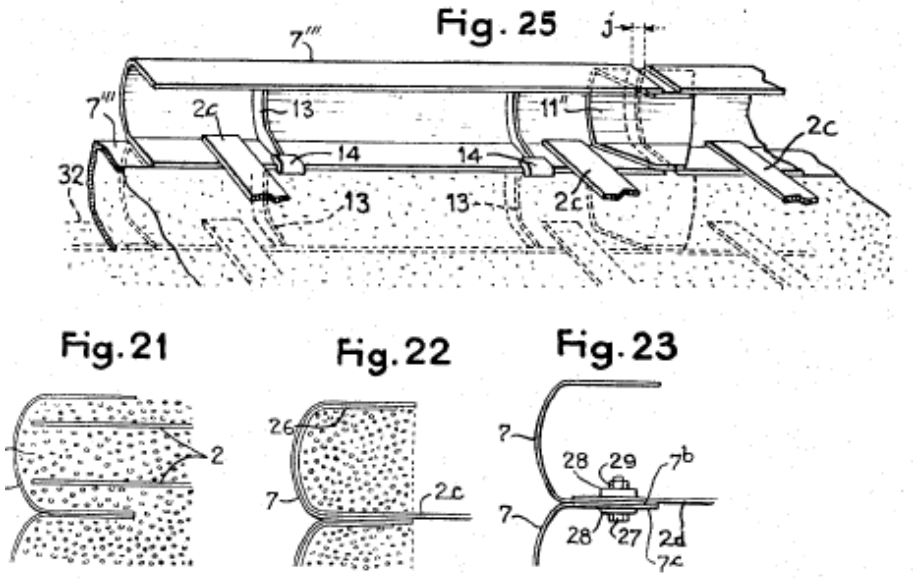


Figure 6.1.1-1: REF: Vidal Patent 3421326 (1969)

By 1969, the formal development of precast panels with Reinforced Earth had begun. The original panels consisted of 180mm thick *unreinforced* concrete panels with a steel "C" anchor embedded into the panel. 3 ½" protruding out the back face to create a single shear connection with a smooth reinforcing strip. The width of the reinforcing strip varied to develop the required pullout resistance. The first concrete panel wall was built in 1970 in Poitiers, France.

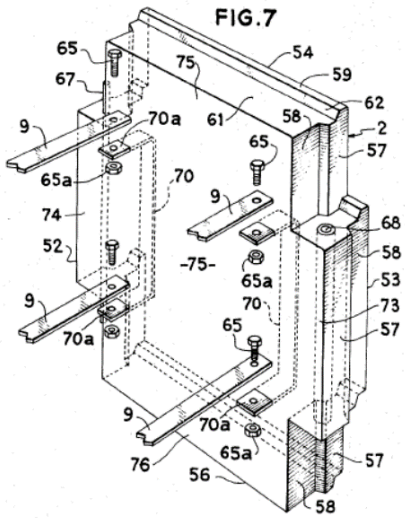


Figure 6.1.1-2: REF: Vidal Patent 3,686,873 (1972)

In the early 1970's, the grade of reinforcement was typically A36 (fy=36 ksi) and the amount of galvanization was not standardized. At the same time, RECo and Terre Armée experimented with stainless steel, aluminum alloys, epoxy coated steel and synthetic reinforcements. Horizontal joints between panels were separated with cork. Vertical joints were protected with foam filler.

In 1978, High Adherence Ribbed Strips were introduced in the United States after several years of testing in the US and Europe. The first structures in the US with ribbed strips were built soon thereafter.

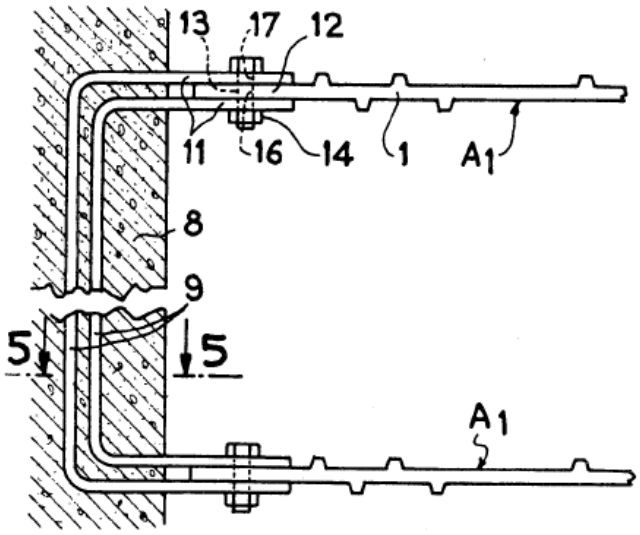


Figure 6.1.1-3: REF: Vidal Patent 4,116,010 (1978)

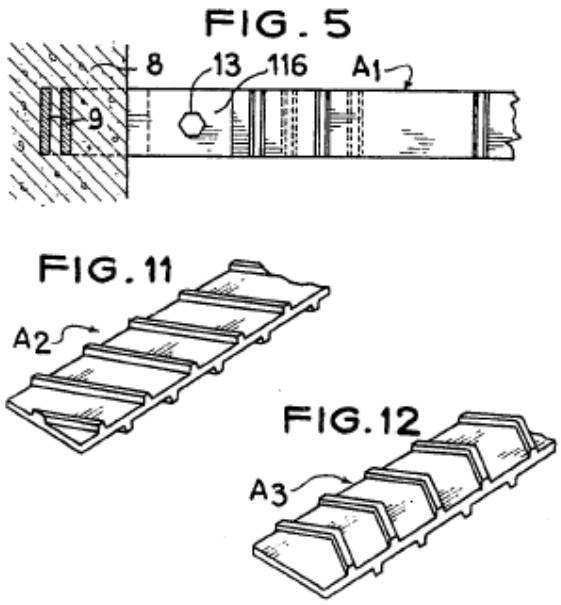


Figure 6.1.1-4: REF: Vidal Patent 4,116,010 (1978)

In 1983, the connection mechanism was innovated, and the panel thickness was reduced from 7" to 5 1/2". The "C" anchor was replaced with a much more economical galvanized tie strip as is presently used. Between 1978 and 1980, over 220 Reinforced Earth Panels were tested with various thickness, connection devices (tie strips) and concrete reinforcement layouts. A second series of full-scale panel load tests (additional 161 panels) were tested and reported on in 1982. In addition, individual tie strip pullout tests were conducted in 1975 (on various connection geometries, including one similar to the now "standard" tie strip and again 1990 on the "standard" tie strip used in Reinforced Earth panels.

In 1985, Precast Traffic Barriers (atop MSE walls) were introduced by The Reinforced Earth Company.

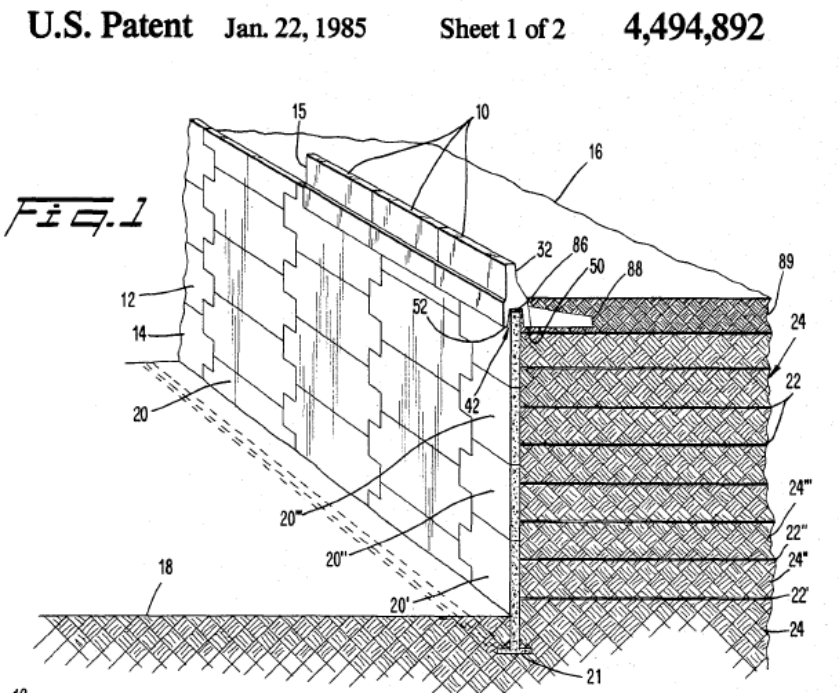


Figure 6.1.1-5: REF: Wojciechiowski Patent 4,494,892 (1985)

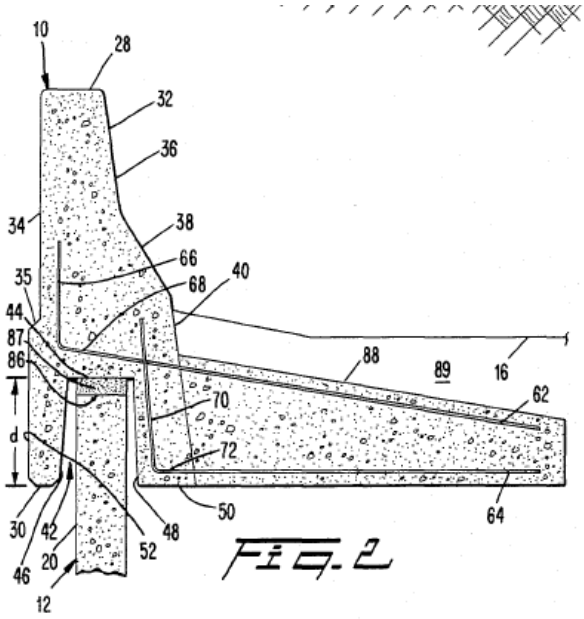


Figure 6.1.1-6: REF: Wojciechiowski Patent 4,494,892 (1985)

By 1986, the grade of steel for the Reinforcing Strips was standardized at fy=65 ksi. Cork at panel joints was replaced with EPDM Bearing Pads. Panel Joints were protected with geotextile rather than foam.

IN 1990, High Adherence (HA) Ladders were introduced by The Reinforced Earth Company.

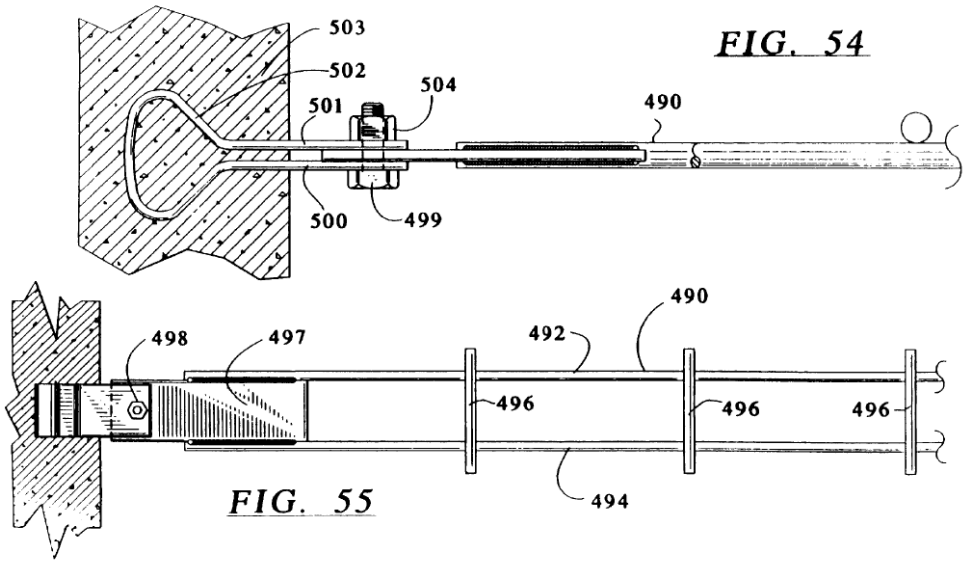


Figure 6.1.1-7: REF: Anderson Patent 6,079,908 (1990)

The preceding information is just a highlight of the innovations made throughout the history of Terre Armée and The Reinforced Earth Company, more recent patents include:

- Concealed Crash Walls for MSE structures (Lamberson, US 6,174,109)
- Two Stage Wall Connector (Timmons, US 6,802,675)
- Retaining Wall Construction and a Method for Construction of Such a Retaining Wall (2-Stage) (Pagano, 4,961,673)
- Facing Element with Integrated Compressibility (Freitag, US 9,422,686)
- Reinforced Structures in The Ground (Freitag, US 8,979,437)
- Continuous Fluid Tightness for a Civil Engineering Work (Freitag, US 8,985,900)
- Facing Element for Reinforced Soil Structure (Freitag, US 9,175,454)
- Stabilized Earth Structure and Method for Constructing It (Freitag, US 7,125,202)

6.1.2 Instrumented Structures

The Coherent Gravity Method is The Reinforced Earth Company's preferred method for design of MSE walls reinforced with inextensible (steel) reinforcements and is outlined in Section 11 of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2020). The Coherent Gravity Method is an empirical method, based on instrumented structure data.

Reinforcement Behavior:

In pullout tests, the reinforcements are placed between layers of compacted soil in a pullout box and an overburden load is applied to the soil by an air bladder or mechanical means. The pullout force is applied to the leading end of the soil reinforcement and the pullout force and resulting displacement of the reinforcement are measured at frequent intervals. The pullout resistance of the reinforcement should be determined at a displacement of 20 mm.

Inextensible (Steel) Reinforcement:

The applied pullout force and resulting displacement of the reinforcement are measured simultaneously. With inextensible reinforcements, the displacement at the leading end is nearly the same as the displacement at the free end because reinforcement strain is negligible, so displacement is measured at the leading end where the load is applied. The friction developed between the reinforcement and the soil is determined for a leading-edge displacement of 20 mm, and the transfer of load to the soil via friction is uniformly distributed over the full length of the reinforcement.

In an actual structure, the load is applied to the reinforcement by the soil within the active zone, which is trying to escape through the wall face. The magnitude of this earth pressure depends on the vertical stress and the coefficient of lateral earth pressure. Vertical stress is a function of the overburden pressure, which increases with depth in the structure, while the coefficient of lateral earth pressure varies from at rest (K_o) at the top of the structure to active (K_a) at a depth of 6 m and deeper. The horizontal earth pressure becomes tension in the reinforcements through the mechanism of friction.

The tension in the reinforcement is greatest at the line of maximum tension (Figure 6.1.2-1) and decreases gradually over the full reinforcement length until near the free end, where the tension decreases rapidly to zero. Significant tension is observed over the full length of the reinforcements.

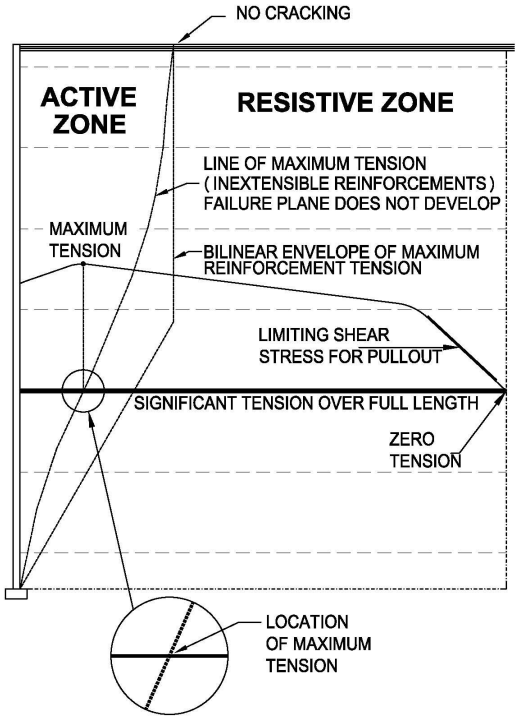


Figure 6.1.2-1: Inextensible reinforcement tension

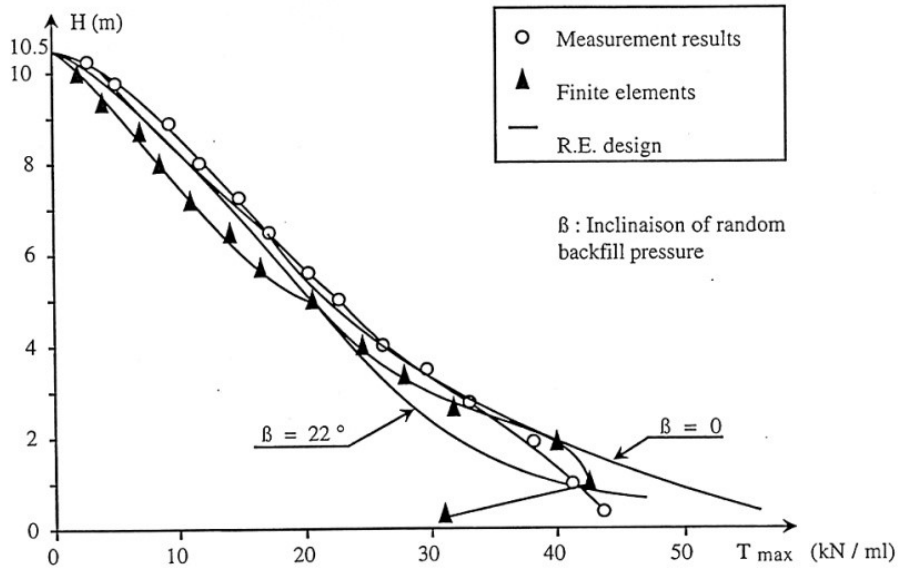


Figure 6.1.2-2: Measured loads in reinforcement

Figure 6.1.2.2 is plotted from a full-scale test structure and the matching Finite Element Model (FEM) (Bastick, et al., 1993) and shows the maximum measured reinforcement tension, the maximum tension calculated by FEM, and the tension calculated by the Coherent Gravity method. Note that all are in close agreement, and that all three clearly show the overturning effect as curvature of the line near the bottom of structure. In the United States we use the β = 0 line to represent that the lateral earth pressure acts on the mass horizontally, not at an angle of inclination. This is the more conservative design line.

As was shown in Figure 6.1.2-1, the tension is distributed over nearly the entire reinforcement length. The compressive strength and shear strength of the soil combine to make the MSE structure behave as a rigid body. This rigid body behavior is also evident in Figure 6.1.2-2, in the magnitude of the maximum reinforcement tension, which increases toward the bottom of the wall. The increase is magnified by the overturning effect of the externally applied loads. This overturning effect, determined by the Meyerhof (1953) calculation, is considered by the Coherent Gravity Method but is not considered by the Simplified Method.

Reference: Anderson P., Gladstone R., Withiam J. (2010). "Coherent Gravity: The Correct Design method for Steel-Reinforced MSE Walls". ASCE Earth Retention Conference 3. ISBN (print): 9780784411285

More than twenty Reinforced Earth structures have been instrumented worldwide over the past 50 years. See section 5.1 for bibliography of performance related studies which include many of the instrumented structures. Instrumented Structure Data may be provided upon request.

6.1.3 Design Coordination

The Reinforced Earth Company's design responsibility is generally limited to the internal stability of the Reinforced Earth structure only. External stability of the retaining wall system is typically excluded from our responsibility. The owner's geotechnical engineer should perform the necessary studies regarding

external stability in the plan development stage. The following note appears on each sheet of design drawings provided by The Reinforced Earth Company:

"The design contained on these drawings is based on information provided by the owner. On the basis of this information, The Reinforced Earth Company has designed, and is responsible for the internal stability of the structure only. External stability, including foundation and slope stability, is the responsibility of the owner"

In summary, The Reinforced Earth Company is not responsible for global stability of the wall, foundation settlement, both total and differential, and bearing capacity of the foundation soils. In addition, stability of both temporary and permanent slopes, during and after construction, is not the responsibility of The Reinforced Earth Company.

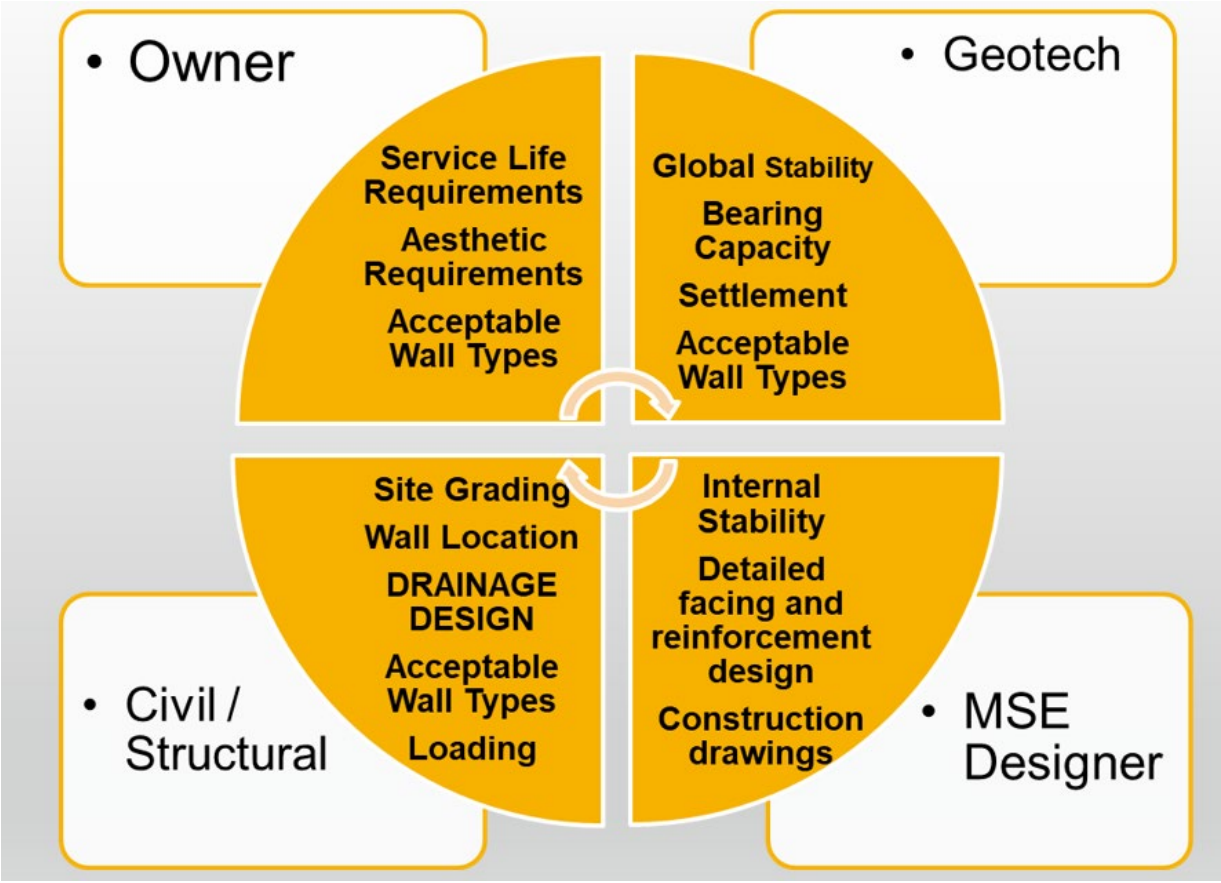


Figure 6.1.3-1: Typical responsibility matrix for MSE wall design

Appendix

Appendix

IDEA
Highway Innovations, Developments, Enhancements and Advancements

Reinforced Earth®
Mechanically Stabilized Earth with Precast Concrete Facing Panels and Inextensible Soil Reinforcements

September 30, 2022

The products, trademarks, data, innovations, and design values presented herein are the intellectual property of The Reinforced Earth Company. This Information is presented for the purposes of IDEA review and publication. The Reinforced Earth Company retains all rights to the information provided and its use for the design, supply and manufacturing of Mechanically Stabilized Earth structures.

Rev.	Date	Modification	Design by	Checked by	Approved by
3	9/30/2022	Final Panel Review Comments	JZ	KPB	KPB
2	6/27/2022	Panel Review Comments	JZ	KPB	KPB
1	2/8/2022	TOC hyperlinks	JZ	KPB	KPB
0	1/14/2022		JZ	KPB	KPB

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A4.1.3 Other Components Quality Control 878
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A1.0 ERS Components

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A1.1 Facing Unit

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A1.1.1 Facing Unit Innovations

This section is intentionally left blank.

A1.1.2 Facing Unit Types

This section is intentionally left blank.

A1.1.3 Facing Unit Specifications

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A1.1.4 Facing Unit Dimensions, Tolerances and Typical Steel Reinforcement Schedule

The following details show the typical dimensions and steel reinforcement for the standard facing unit shapes:

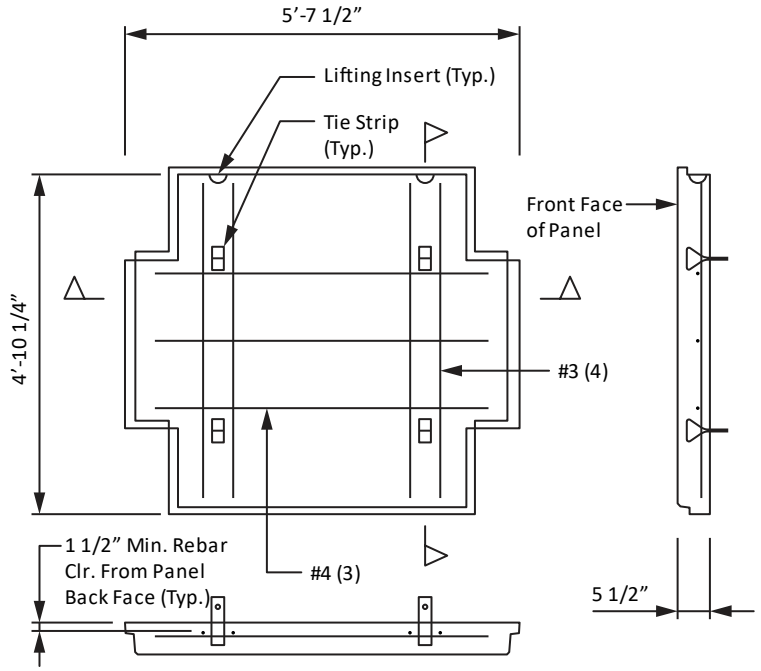


Figure A1.1.4a – Standard Cruciform facing unit with four tie strips. Back face view.

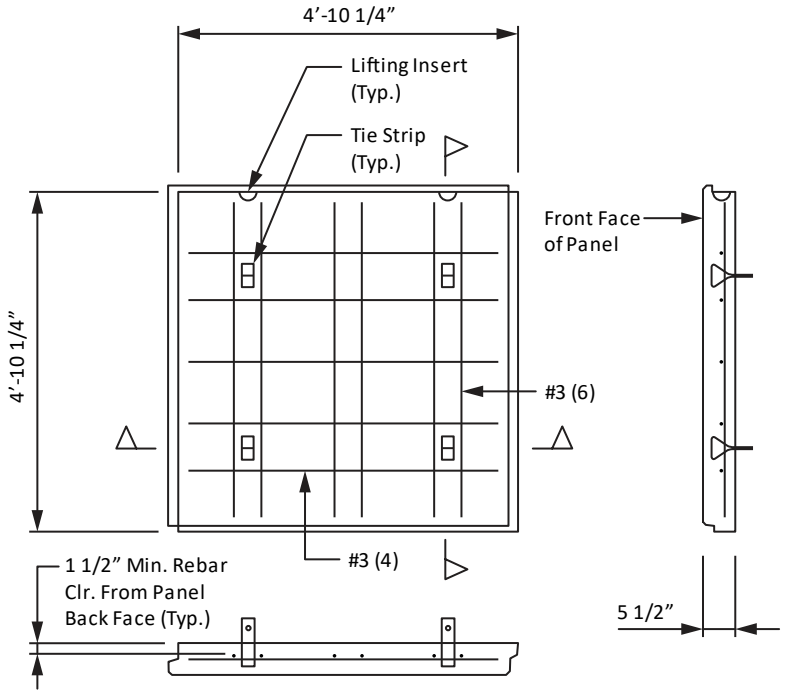


Figure A1.1.4b – Standard Rectangular facing unit with four tie strips. Back face view.

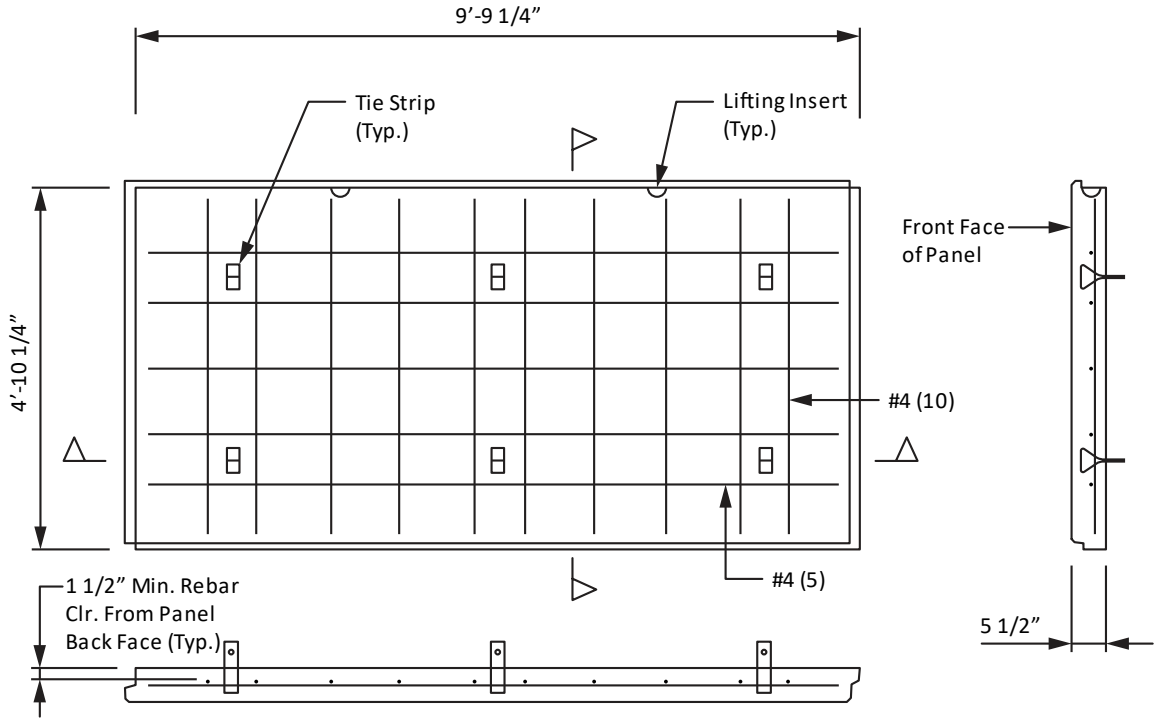


Figure A1.1.4c – Standard Rectangular facing unit with six tie strips. Back face view.

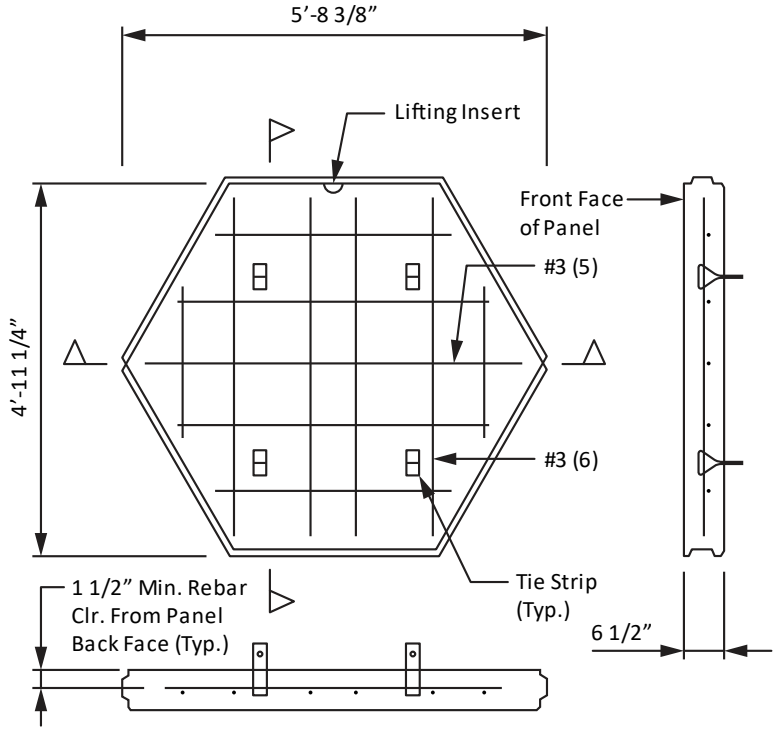


Figure A1.1.4d – Standard Hexagonal facing unit with four tie strips. Back face view.

Tolerances

All units shall be manufactured within the following tolerances with respect to the dimensions shown on the shop drawings:

(a) Attachment Device Locations and Alignment – The lateral position of reinforcing strip attachment devices (tie strip) shall be within 1 inch of dimension shown, and embedment measured from the back face of the panel shall be within +/-1/4 inch. See Figure A1.1.4e below.

(b) Panel Dimensions – Due to RECo providing steel form molds all panel dimensions shall be as stated within detail shop drawings. All additional required hardware embedded in the panel with the exception of attachment devices shall be within 1/4 inch (+/-) of dimension shown.

(c) Panel Squareness -- Squareness, as determined by the difference between the two diagonals, shall not exceed 1/2 inch.

(d) Panel Surface Finish -- Surface defects on smooth-formed surfaces, measured on a length of 5 feet, shall not exceed 1/4 inch. Surface defects on textured-finished surfaces, measured on a length of 5 feet, shall not exceed 5/16 inch.

(e) Panel Rebar – Panel rebar shall be placed within the following tolerances:
   a. Clear Cover: (+/-) 3/8" (REF ACI 318-14, 26.6.2.1)
   b. Location from Plan Dimension: (+/-) 2" horizontally or vertically from what is shown in shop drawings, noting that cover requirements control.
   c. Tie strip shall not be in contact with uncoated panel reinforcement steel.

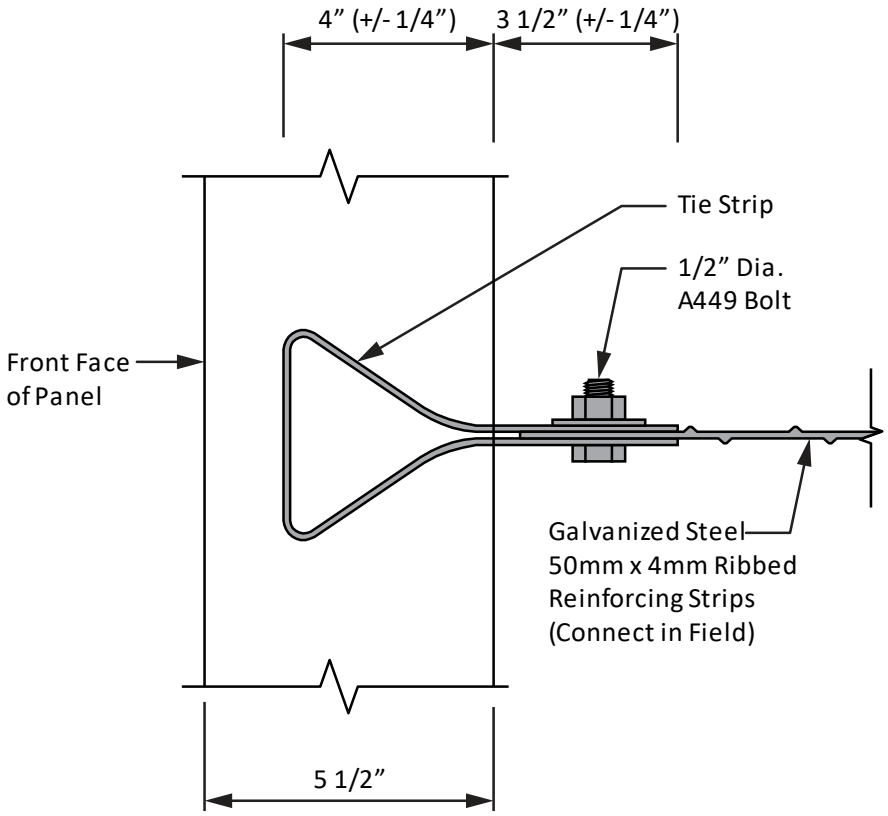


Figure A1.1.4e – Section: Reinforcing strip connection to facing.

A1.1.5 Compressive Strength

The following are RECo minimum requirements. When owner's requirements exceed RECo minimums, the owner requirements prevail.

Acceptance of the concrete panels, with respect to compressive strength, shall be determined based on production lots. A production lot is defined as a group of panels that shall be represented by a single set of compressive strength samples and shall consist of not more than 80 panels or a single day's production, whichever is less.

Compressive strength tests shall be performed on 6-inch diameter by 12-inch-tall cylinders prepared in accordance with AASHTO T-23. During the production of the concrete panels, the manufacturer shall randomly sample the concrete in accordance with AASHTO T-141. A single set of compressive strength samples, consisting of a minimum of four (4) cylinders, shall be made for every production lot.

For every compressive strength sample set, a minimum of two cylinders shall be cured in the same manner as the panels and tested at seven (7) days or less. The average compressive strength of these cylinders, when tested in accordance with AASHTO T-22, will determine the initial strength of the concrete. In addition, a minimum of two cylinders shall be cured in accordance with AASHTO T-23 and tested at 28 days. The average compressive strength of these cylinders, when tested in accordance with AASHTO T-22, will determine the compressive strength of the production lot.

If the initial strength test result indicates a compressive strength greater than or equal to 4,000 psi, then this test result will be utilized as the compressive strength test result for that production lot, and the requirement for testing at 28 days will be waived for that production lot.

Acceptance of a production lot will be made if the compressive strength test result is greater than or equal to 4,000 psi. If the compressive strength test result is less than 4,000 psi, the acceptance of the production lot will be based on its meeting the following acceptance criteria in its entirety:

(a) Ninety (90) percent of the compressive strength test results for the overall production shall exceed 4,150 psi.

(b) The average of any six (6) consecutive compressive strength test results, including the one in question, shall exceed 4,250 psi.

(c) No individual compressive strength test result shall fall below 3,600 psi.

If a production lot fails to meet the specified compressive strength requirements, the production lot shall be rejected. Such rejection shall prevail unless the manufacturer, at its own expense, obtains and submits evidence of a type acceptable to the Engineer that the strength and quality of the concrete placed within the panels of the production lot is acceptable. If such evidence consists of tests made on cores taken from the panels within the production lot, the cores shall be obtained and tested in accordance with AASHTO T-24.

Acceptance Criteria

Precast panels shall be accepted for use in wall construction provided the concrete strength meets or exceeds the minimum compressive strength requirement, the soil reinforcement connection devices and the panel dimensions are within tolerances and any chipping, cracks, honeycomb or other defects are within acceptable standards for precast concrete as determined by the Engineer.

It is recognized that certain cracks and surface defects are not detrimental to the structural integrity of the panel if properly repaired. The Engineer shall determine the need for and the proper method of such

repair. All repairs shall be approved by the Engineer prior to acceptance of the panel for use in wall construction.

A1.1.6 Air Entrainment

This section is intentionally left blank.

A1.1.7 Mix Design

This section is intentionally left blank.

A1.1.8 Shear, Alignment and Bearing Devices

RECo
Bearing Pad Calculations

Rev	Date	Modification	Drafted by	Checked by	Approved by
Original	04/06/2016	First issue	KPB	RL	KPB
REV 1	11/7/2016	Added REW	KPB	LY	KPB
REV 2	1/6/2017	Updated for Tech Memo	KPB	RL	KPB
REV 3	10/28/2021	Added BPWRLA	KPB	DLH	KPB
REV 4	04/22/2022	Removed Disclaimer for IDEA	KPB	KPB	KPB
REV 5	9/30/2022	Added clarification for BP34 design	KPB	KPB	KPB

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Introduction

RECo has tested five Bearing Pad types routinely used in RECo supplied MSE wall systems:

BP34 RIBBED (4" x 3 1/8" x 3/4") – Used for REW Panels

BP1A RIBBED (4 3/8" x 2 9/16" x 1") – Used for REW, SQR and RLA Panel Types

BP03 FLAT (4 ½" x 2 ¾" x ¾") – Used for REW and SQR and as supplemental pads for Corner Elements and Slip Joints

BP21 FLAT (10" x 2 ¾" x ¾") – Used for RLA panel types

BPWRLA (10" x 2 7/8" x 7/8") – Used for RLA Panel Types and heavily loaded SQR and REW walls

The BP34, BP1A, BP03, BPWRLA and BP21 pads are manufactured by REX-HIDE.

KEY:
REW = Cruciform Shape Panels (4.92'x4.92')
SQR = Square Shape Panels (4.92' x 4.92')
RLA = Rectangular Shape Panels (4.92'x9.84')

These calculations demonstrate the adequacy of the BP34, BP1A, BP03, BPWRLA and BP21 bearing pads as used for REW, SQR and RLA panels.

Discussion

Bearing Pads are designed to compress to assure the facing will compress compatibly with the elastic compression of the select backfill, and to allow the facing panels to be nearly as flexible as the MSE embankment itself. The nature of bearing pad compression is complex and is a function of bearing pad geometry and thickness, backfill type and compression during compaction, and any settlement of the facing panels due to compression of the foundation soils beneath the wall. There is a potential for minimal compression of bearing pads when conditions are optimal, and more significant (i.e. approaching the design load and final design thickness) when foundation conditions are poor and/or select backfill is sand.

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Analysis

The attached calculations are based on use of a conservative design factor of 2.0 times the Panel Weight, and a post construction design horizontal joint width of 1/2" (after settlement and bearing pad compression) in accordance with FHWA recommendations. The analysis assumes 6" thick panel thickness and that differential settlement does not exceed 1% for 5'x5' panels (REW+SQR) or 0.5% for 5'x10' panels (RLA). The analysis is also based on the assumption that the backfill meets standard AASHTO specifications and no large surcharge loading (beyond typical AASHTO design requirements) is applied near the panel face. If any of the above assumptions do not apply, then the bearing pad requirements should be assessed based on the specific conditions of the project.

Conclusions

The calculations with a Design Factor of 2.0 and a minimum joint thickness of 0.375" or 0.5" after compression of the bearing pads result in the following limits:

Panel Type	Bearing Pad Type	No. of Pads per Panel	Limiting Height above Joint (ft)
REW	BP34	2	30
SQR/REW	BP1A	2	27.5
SQR/REW	BP03	2	30
SQR/REW	BP21	2	60
SQR/REW	BPWRLA	2	65
RLA	BP1A	4	27.5
RLA	BP21	2	30
RLA	BP21	4	60
RLA	BPWRLA	2	30
RLA	BPWRLA	4	65

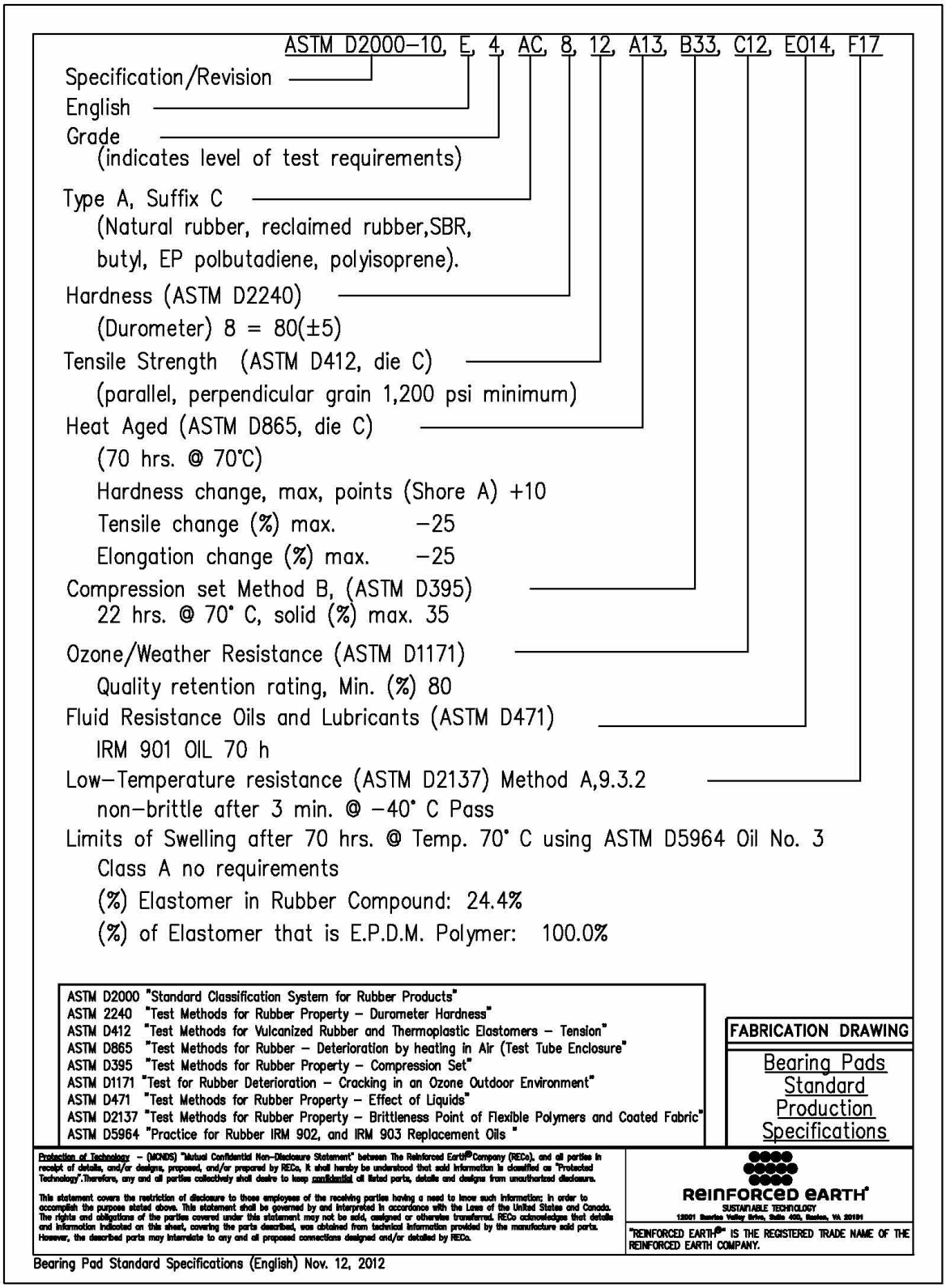
The calculations limit the use of two bearing pads (BP34, BP1A or BP03) per SQR/REW panel to wall heights of 27.5 to 30 ft. Likewise the calculations limit the use of two bearing pads (BP21 or BPWLRA) per RLA panel to wall heights of 30 ft. For taller walls, use of two BP21 or BPWRLA bearing pads per SQR/REW panel, or four BP21/BPWRLA bearing pads per RLA panel allows construction of walls up to 60 ft (65 ft with BPWRLA).

Attachments:

Bearing Pad Calculations (24 pages)

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ASTM D2000–10, E, 4, AC, 8, 12, A13, B33, C12, E014, F17
Specification/Revision
English
Grade
(indicates level of test requirements)
Type A, Suffix C
(Natural rubber, reclaimed rubber,SBR,
butyl, EP polbutadiene, polyisoprene).
Hardness (ASTM D2240)
(Durometer) 8 = 80(±5)
Tensile Strength (ASTM D412, die C)
(parallel, perpendicular grain 1,200 psi minimum)
Heat Aged (ASTM D865, die C)
(70 hrs. @ 70°C)
Hardness change, max, points (Shore A) +10
Tensile change (%) max. −25
Elongation change (%) max. −25
Compression set Method B, (ASTM D395)
22 hrs. @ 70° C, solid (%) max. 35
Ozone/Weather Resistance (ASTM D1171)
Quality retention rating, Min. (%) 80
Fluid Resistance Oils and Lubricants (ASTM D471)
IRM 901 OIL 70 h
Low–Temperature resistance (ASTM D2137) Method A,9.3.2
non–brittle after 3 min. @ −40° C Pass
Limits of Swelling after 70 hrs. @ Temp. 70° C using ASTM D5964 Oil No. 3
Class A no requirements
(%) Elastomer in Rubber Compound: 24.4%
(%) of Elastomer that is E.P.D.M. Polymer: 100.0%
ASTM D2000 "Standard Classification System for Rubber Products"
ASTM 2240 "Test Methods for Rubber Property – Durometer Hardness"
ASTM D412 "Test Methods for Vulcanized Rubber and Thermoplastic Elastomers – Tension"
ASTM D865 "Test Methods for Rubber – Deterioration by heating in Air (Test Tube Enclosure"
ASTM D395 "Test Methods for Rubber Property – Compression Set"
ASTM D1171 "Test for Rubber Deterioration – Cracking in an Ozone Outdoor Environment"
ASTM D471 "Test Methods for Rubber Property – Effect of Liquids"
ASTM D2137 "Test Methods for Rubber Property – Brittleness Point of Flexible Polymers and Coated Fabric"
ASTM D5964 "Practice for Rubber IRM 902, and IRM 903 Replacement Oils "
FABRICATION DRAWING
Bearing Pads
Standard
Production
Specifications
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Bearing Pad Standard Specifications (English) Nov. 12, 2012

REINFORCED EARTH

Bearing Pad Calculations

DATE	6/3/2015		**Bearing Pad Code BP34**
Engineer	KPB		

Dimensions (inches)	4	WIDTH (w)	
	3.125	LENGTH (b)	
	0.75	THICKNESS (t)	
Durometer	80 +/-5	SPEC	
(Shore A)	75 (MIN.)		

PCI Shape Factor $S = \frac{wb}{2(w+b)t}$

$S = 1.17$

PCI Allowable Compressive Stress = σ

$\sigma = 1000 + 100\,S <= 1500$

$\sigma = 1116.96$ psi

$\sigma = 885.6$ psi Per Design Load Below

Stress Strain Test Data — Terre Armee Internal Testing 2013

LBS	Compression
2400	0.125 Inch Compression
5170	0.25 Inch Compression
11240	0.375 Inch Compression

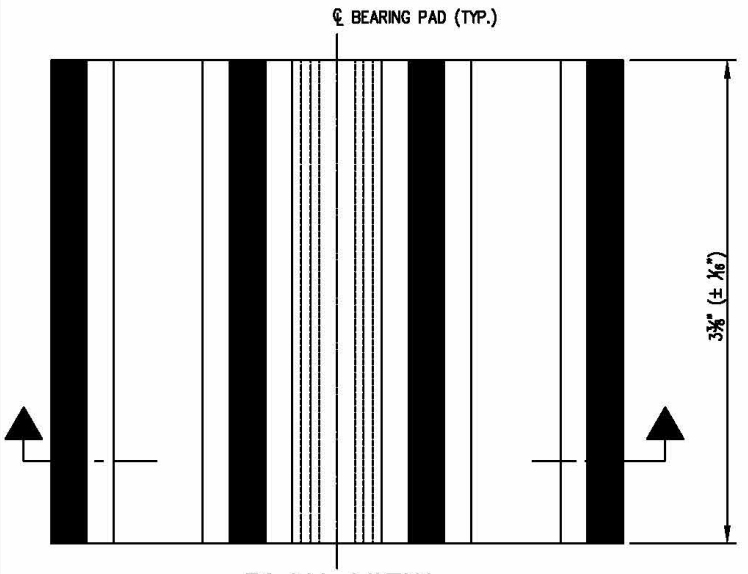
Note: The design joint width of 0.375" is unique to Cruciform Shape Panels using the "BP34" bearing pad. This tolerable width is based on over 30 year's experience and observation of constructed structures.

Design Joint	0.375	Inch	**= t After Compression**
Controlling Data	0.375	Inch Compression	11240 LBS Design Capacity
Design Load (per pad)	11070	LBS	= (2 x H x B x T x Y)/#

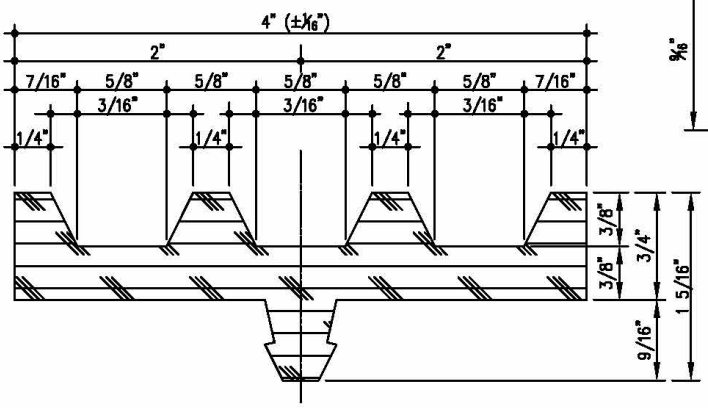
Height of Wall (H)	30	ft
Width of Panel (B)	4.92	ft
Number of Pads/Panel (#)	2	qty
Thickness of Panel (T)	0.5	ft
Unit Weight of Concrete (Y)	150	LBS/FT3
DESIGN FACTOR (x PANEL T)	2	

DESIGN LOAD VS. DESIGN CAPACITY

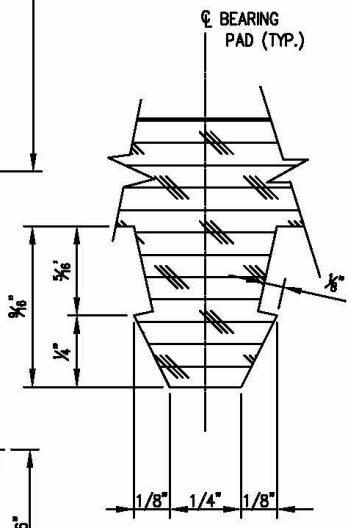
0.984875 ***OK***



PLAN VIEW



SECTION



DETAIL

Description:
¾" Thick Custom Molded E.P.D.M. Ribbed Bearing Pad
Item

General Information:

Manufacturer: (RECo Approved Vender or Approved Equal)

Material Specification: ASTM D2000-12; E, 2, BE, B, 15, A14(Shore A), B14, C12, E014, F17
For Explanation see Bearing Pads Standard Production Specification sheet .

Comments: Durable rubber compound integrated with high strength synthetic fibers, resulting in an integral pad with excellent physical properties and weather resistance. Durable dense rubber with excellent compressive strength, hardness, tear resistance and cold weather flexibility. "Green Product" molded from masticated rubber using discarded tires.

FABRICATION DRAWING

RECo Part #:
BP-34

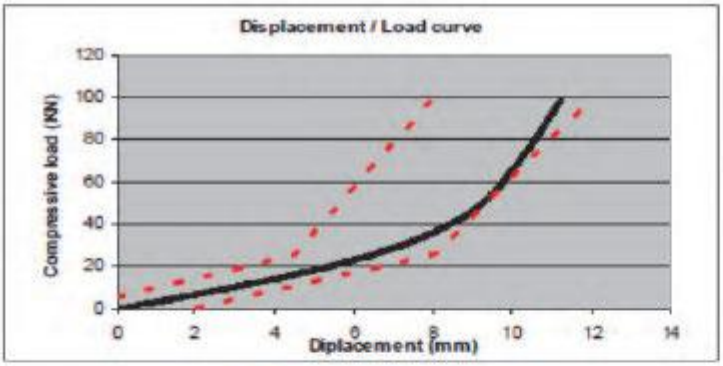
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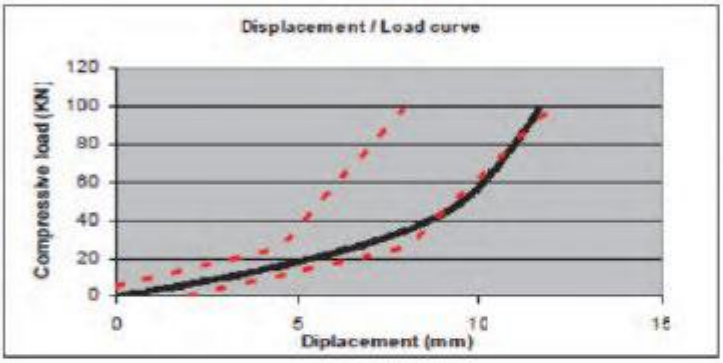
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¾" Thick Ribbed Bearing Pad (English) Nov. 17, 2010



Specimen n°1



Specimen n°2

Note: The red dashed lines indicate the desired envelope of displacement vs compressive load. The solid black line indicates the test results.

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Bearing Pad Calculations

DATE	6/2/2015		**Bearing Pad Code BP1A**
Engineer	KPB		

5x5 Panels

Dimensions (inches)	2.5625	WIDTH (w)	
	4.375	LENGTH (b)	
	1	THICKNESS (t)	
Durometer	80	+/-5	SPEC
(Shore A)	75	(MIN.)	

PCI Shape Factor $S = \frac{wb}{2(w+b)t}$

$S = 0.81$

PCI Allowable Compressive Stress = σ

$\sigma = 1000 + 100\,S <= 1500$

$\sigma = 1080.80$ psi

$\sigma = 905.1429$ psi Per Design Load Below

Stress Strain Test Data: TRI 5/1/2015 Report

LBS	Compression
1523	**0.125 Inch Compression**
3031	**0.25 Inch Compression**
5083	**0.375 Inch Compression**
10260	**0.5 Inch Compression**
20480	**0.625 Inch Compression**

Design Joint: **0.5** Inch **= t After Compression**

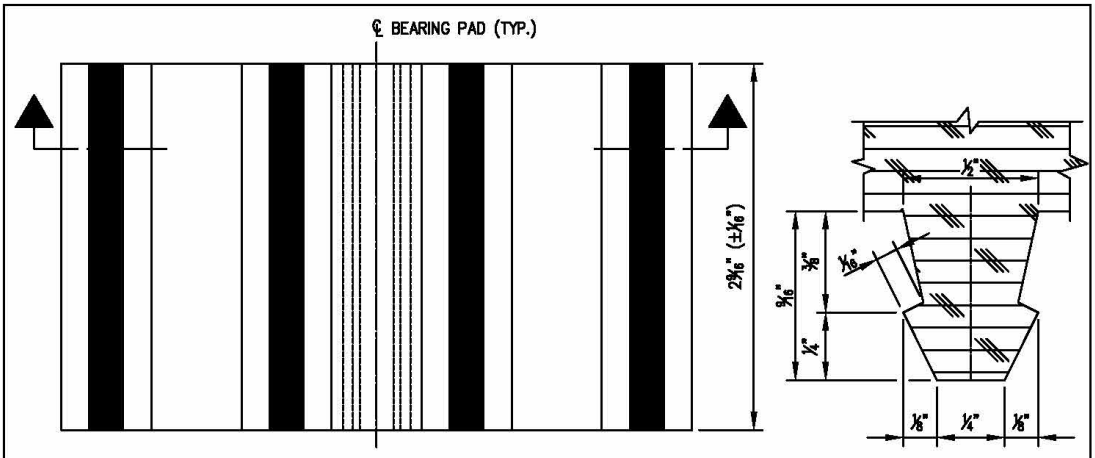
Controlling Data: 0.5 Inch Compression — 10260 LBS — Design Capacity

Design Load (per pad): 10147.5 LBS = (2 x H x B x T x Y)/#

Height above Joint (H)	27.5	ft
Width of Panel (B)	4.92	ft
Number of Pads/Panel (#)	2	qty
Thickness of Panel (T)	0.5	ft
Unit Weight of Concrete (Y)	150	LBS/FT3
DESIGN FACTOR (x PANEL T)	2	PER FHWA

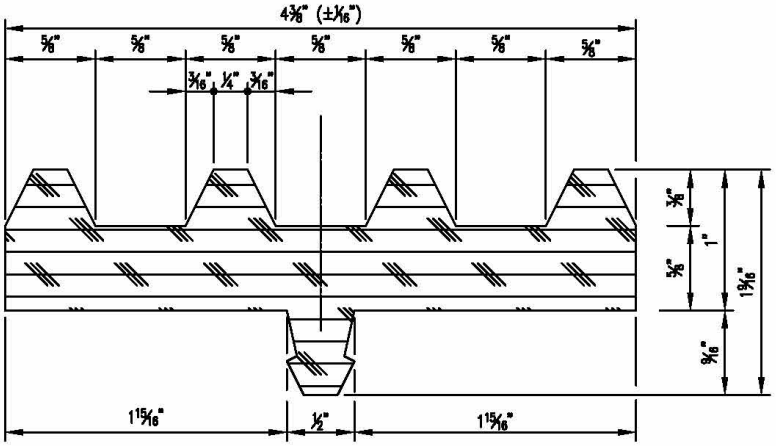
DESIGN LOAD VS. DESIGN CAPACITY

0.989035 ***OK***



PLAN VIEW

DETAIL



SECTION

Description:

1" Thick Custom Molded E.P.D.M. Ribbed Bearing Pad

Item

General Information:

Manufacturer: (RECo Approved Vender or Approved Equal)

Material Specification: ASTM D2000-12; E, 2, BE, B, 15, A14(Shore A), B14, C12, E014, F17
For Explanation see Bearing Pads Standard Production Specification sheet .

Comments: Durable rubber compound integrated with high strength synthetic fibers, resulting in an integral pad with excellent physical properties and weather resistance. Durable dense rubber with excellent compressive strength, hardness, tear resistance and cold weather flexibility. "Green Product" molded from masticated rubber using discarded tires.

FABRICATION DRAWING

RECo Part #:
BP-1A

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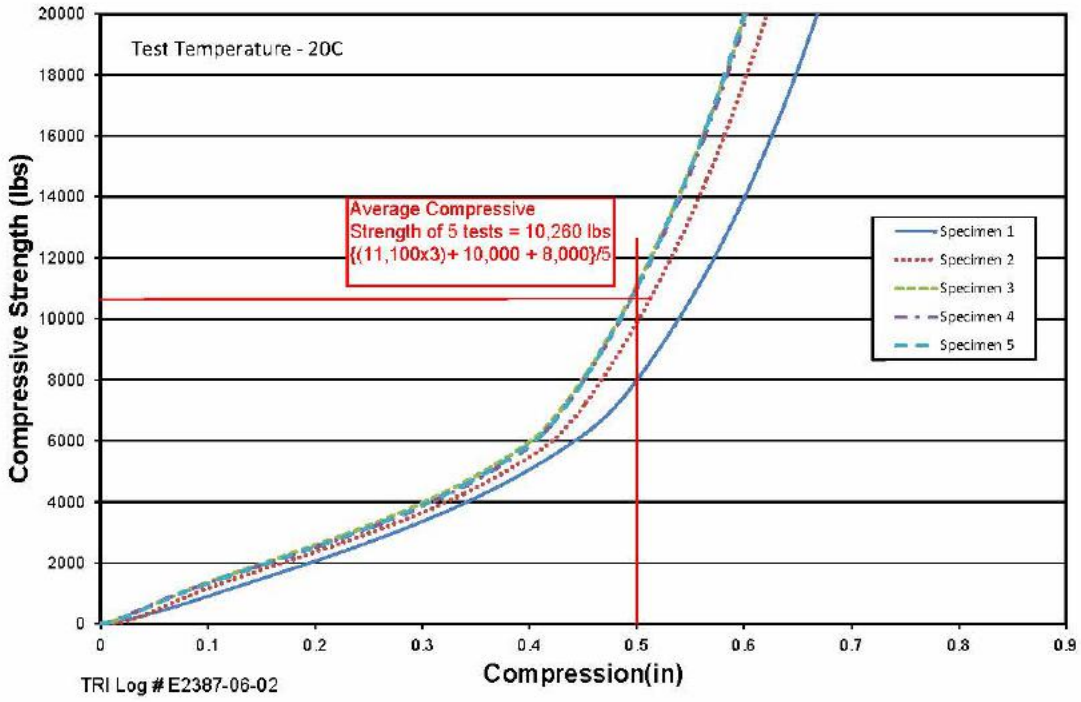
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1" Thick Ribbed Bearing Pad (English) Nov. 17, 2010

Compression Test Results
TRI Client : Reinforced Earth Co.
Product: Bearing Pad - BP1A



TRI Log # E2387-06-02

9063 Bee Caves Road / Austin, TX 78733 / 512-263-2101 / FAX 263-2558 / 800-880-TEST

REINFORCED EARTH®

Bearing Pad Calculations

DATE	6/2/2015		**Bearing Pad Code BP03**
Engineer	KPB		

5x5 Panels

Dimensions (inches)	2.75	WIDTH (w)	
	4.5	LENGTH (b)	
	0.75	THICKNESS (t)	
Durometer	80	SPEC	
(Shore A)	75	(MIN.)	

PCI Shape Factor $S = \frac{wb}{2(w+b)t}$

$S = 1.14$

PCI Allowable Compressive Stress = σ

$\sigma = 1000 + 100\,S <= 1500$

$\sigma = 1113.79$ psi

$\sigma = 894.5455$ psi Per Design Load Below

Stress Strain Test Data: TRI 5/1/2015 Report

LBS	Compression
4482	0.125 Inch Compression
11341	0.25 Inch Compression
23684	0.375 Inch Compression
40000	0.5 Inch Compression

Design Joint: **0.5** Inch **= t After Compression**

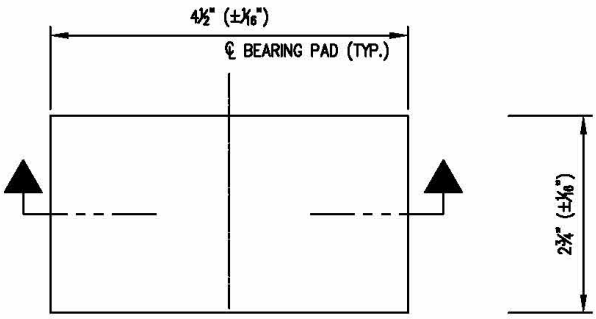
Controlling Data: 0.25 Inch Compression — 11341 LBS Design Capacity

Design Load (per pad): 11070 LBS = (2 x H x B x T x Υ)/#

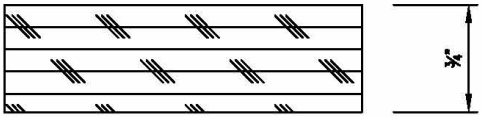
Height above Joint (H)	30	ft
Width of Panel (B)	4.92	ft
Number of Pads/Panel (#)	2	qty
Thickness of Panel (T)	0.5	ft
Unit Weight of Concrete (Υ)	150	LBS/FT3
DESIGN FACTOR (x PANEL T)	2	PER FHWA

DESIGN LOAD VS. DESIGN CAPACITY

0.976104 ***OK***



PLAN VIEW



SECTION

Description:

¾" Thick Solid E.P.D.M. Bearing Pad

Item

General Information:

Manufacturer:	(RECo Approved Vender or Approved Equal)
Material Specification:	ASTM D2000–12; E, 2, BE, B, 15, A14(Shore A), B14, C12, E014, F17 For Explanation see Bearing Pads Standard Production Specification sheet .
Comments:	Durable rubber compound integrated with high strength synthetic fibers, resulting in an integral pad with excellent physical properties and weather resistance. Durable dense rubber with excellent compressive strength, hardness, tear resistance and cold weather flexibility. "Green Product" molded from masticated rubber using discarded tires.

FABRICATION DRAWING

RECo Part #:
BP-03

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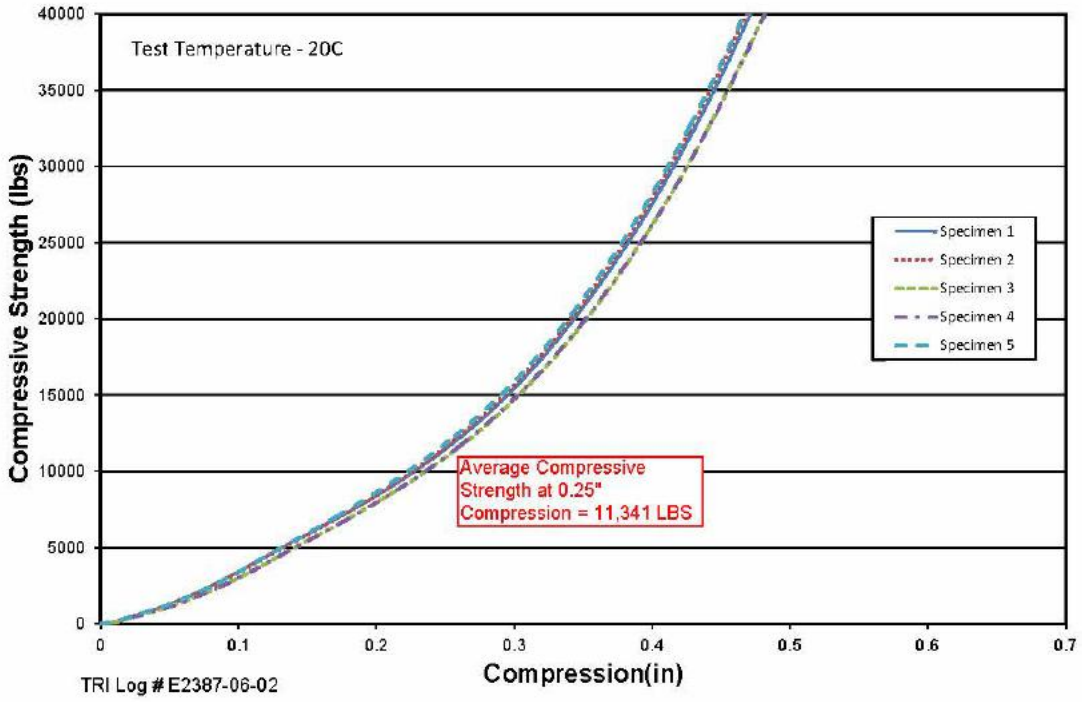
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¾" Thick Solid Bearing Pad (English) Nov. 12, 2010

Compression Test Results
TRI Client : Reinforced Earth Co.
Product: Bearing Pad - BP03



TRI Log # E2387-06-02

9063 Bee Caves Road / Austin, TX 78733 / 512-263-2101 / FAX 263-2558 / 800-880-TEST

REINFORCED EARTH

Bearing Pad Calculations

DATE 6/2/2015 **Bearing Pad Code BP21**
Engineer KPB

5x5 panels

Dimensions (inches)	2.75	WIDTH (w)	
	10	LENGTH (b)	
	0.75	THICKNESS (t)	
Durometer	80	SPEC	
(Shore A)	75	(MIN.)	

PCI Shape Factor $S = \frac{wb}{2(w+b)t}$ $S = 1.44$

PCI Allowable Compressive Stress = σ

$\sigma = 1000 + 100\,S <= 1500$

$\sigma = 1143.79$ psi

$\sigma = 805.0909$ psi Per Design Load Below

Stress Strain Test Data TRI 5/1/2015 Report

LBS	Compression
6231	**0.125 Inch Compression**
23026	**0.25 Inch Compression**
43545	**0.375 Inch Compression**
N.T.	**0.5 Inch Compression**

Design Joint **0.5** Inch **= t After Compression**

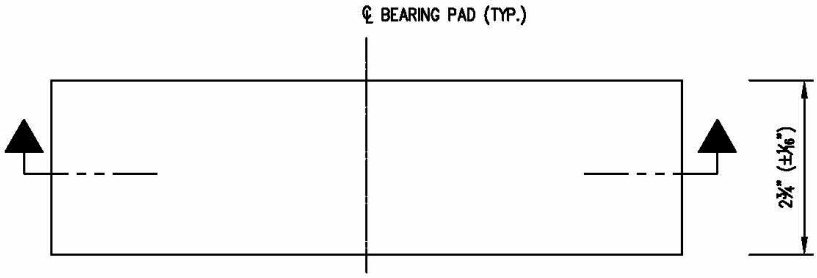
Controlling Data 0.25 Inch Compression 23026 LBS Design Capacity

Design Load (per pad) 22140 LBS = (2 x H x B x T x Y)/#

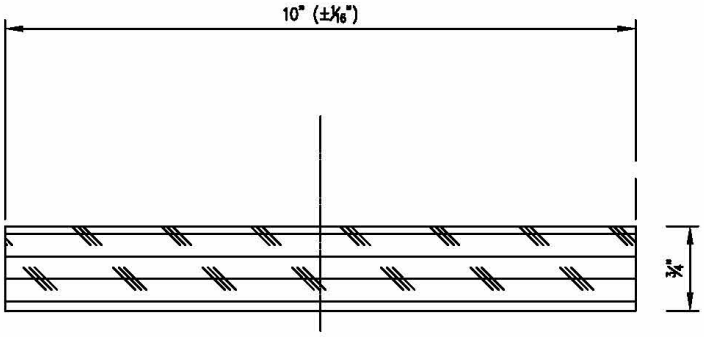
Height above Joint (H)	60	ft
Width of Panel (B)	4.92	ft
Number of Pads/Panel (#)	2	qty
Thickness of Panel (T)	0.5	ft
Unit Weight of Concrete (Y)	150	LBS/FT3
DESIGN FACTOR (x PANEL T)	2	PER FHWA

DESIGN LOAD VS. DESIGN CAPACITY

0.961522 ***OK***



PLAN VIEW



SECTION

Description:

¾" Thick Solid E.P.D.M. Bearing Pad

Item

General Information:

Manufacturer:	(RECo Approved Vender or Approved Equal)
Material Specification:	ASTM D2000–12; E, 2, BE, 8, 15, A14(Shore A), B14, C12, E014, F17 For Explanation see Bearing Pads Standard Production Specification sheet .
Comments:	Durable rubber compound integrated with high strength synthetic fibers, resulting in an integral pad with excellent physical properties and weather resistance. Durable dense rubber with excellent compressive strength, hardness, tear resistance and cold weather flexibility. "Green Product" molded from masticated rubber using discarded tires.

FABRICATION DRAWING

RECo Part #:
BP-21

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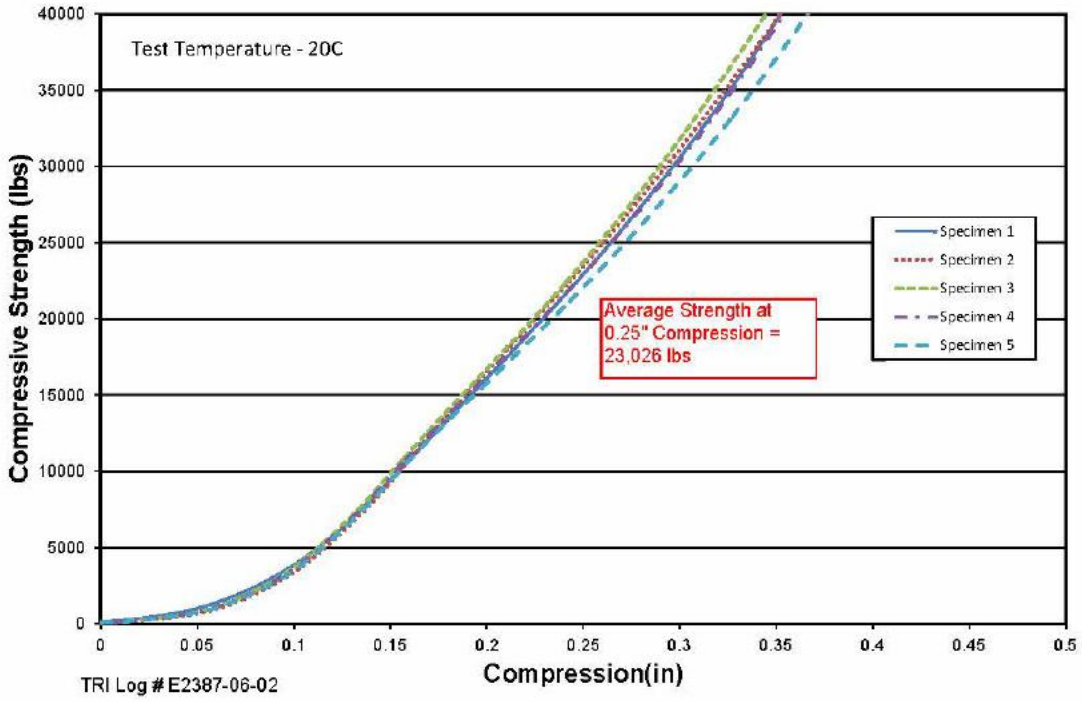
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Elongated ¾" Thick Solid E.P.D.M. Bearing Pad (English) Nov. 17, 2010

Compression Test Results
TRI Client : Reinforced Earth Co.
Product: Bearing Pad - BP21



TRI Log # E2387-06-02

9063 Bee Caves Road / Austin, TX 78733 / 512-263-2101 / FAX 263-2558 / 800-880-TEST

REINFORCED EARTH®

Bearing Pad Calculations

DATE	6/2/2015		**Bearing Pad Code BP1A**
Engineer	KPB		

5x10 Panels

Dimensions (inches)	2.5625	WIDTH (w)	
	4.375	LENGTH (b)	
	1	THICKNESS (t)	
Durometer	80 +/-5	SPEC	
(Shore A)	75 (MIN.)		

PCI Shape Factor $S = \frac{wb}{2(w+b)t}$

$S = 0.81$

PCI Allowable Compressive Stress = σ

$\sigma = 1000 + 100\,S <= 1500$

$\sigma = 1080.80$ psi

$\sigma = 905.1429$ psi Per Design Load Below

Stress Strain Test Data: TRI 5/1/2015 Report

LBS	Compression
1523	0.125 Inch Compression
3031	0.25 Inch Compression
5083	0.375 Inch Compression
10260	0.5 Inch Compression
20480	0.625 Inch Compression

Design Joint: 0.5 Inch = t After Compression

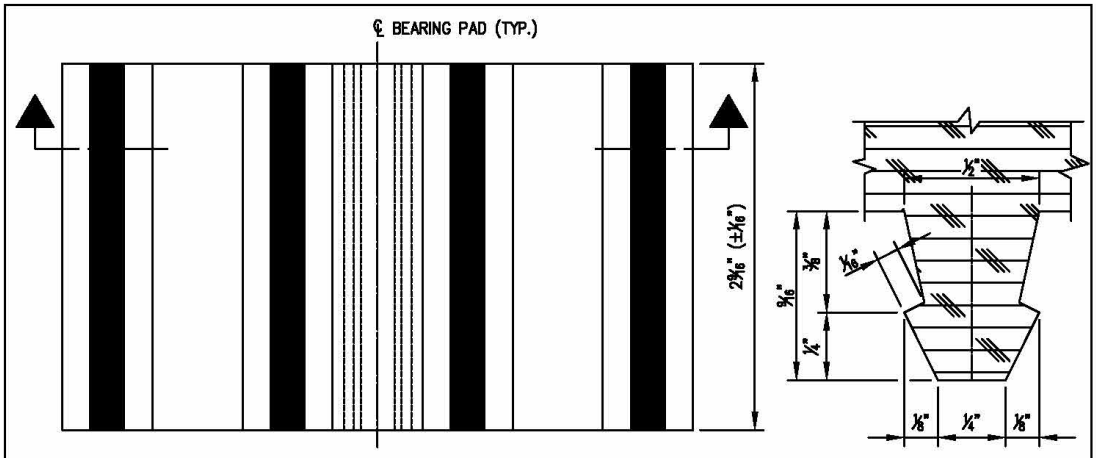
Controlling Data: 0.5 Inch Compression — 10260 LBS — Design Capacity

Design Load (per pad): 10147.5 LBS = (2 x H x B x T x ϒ)/#

Height above Joint (H)	27.5	ft
Width of Panel (B)	9.84	ft
Number of Pads/Panel (#)	4	qty
Thickness of Panel (T)	0.5	ft
Unit Weight of Concrete (ϒ)	150	LBS/FT3
DESIGN FACTOR (x PANEL T)	2	PER FHWA

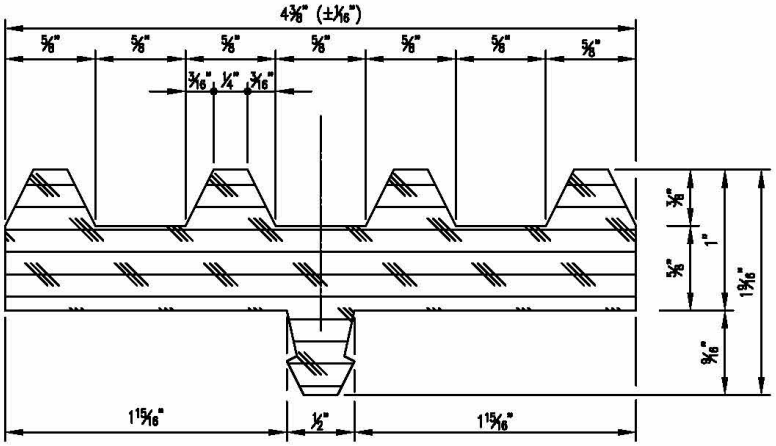
DESIGN LOAD VS. DESIGN CAPACITY

0.989035 ***OK***



PLAN VIEW

DETAIL



SECTION

Description:

1" Thick Custom Molded E.P.D.M. Ribbed Bearing Pad

Item

General Information:

Manufacturer: (RECo Approved Vender or Approved Equal)

Material Specification: ASTM D2000-12; E, 2, BE, B, 15, A14(Shore A), B14, C12, E014, F17
For Explanation see Bearing Pads Standard Production Specification sheet .

Comments: Durable rubber compound integrated with high strength synthetic fibers, resulting in an integral pad with excellent physical properties and weather resistance. Durable dense rubber with excellent compressive strength, hardness, tear resistance and cold weather flexibility. "Green Product" molded from masticated rubber using discarded tires.

FABRICATION DRAWING

RECo Part #:
BP-1A

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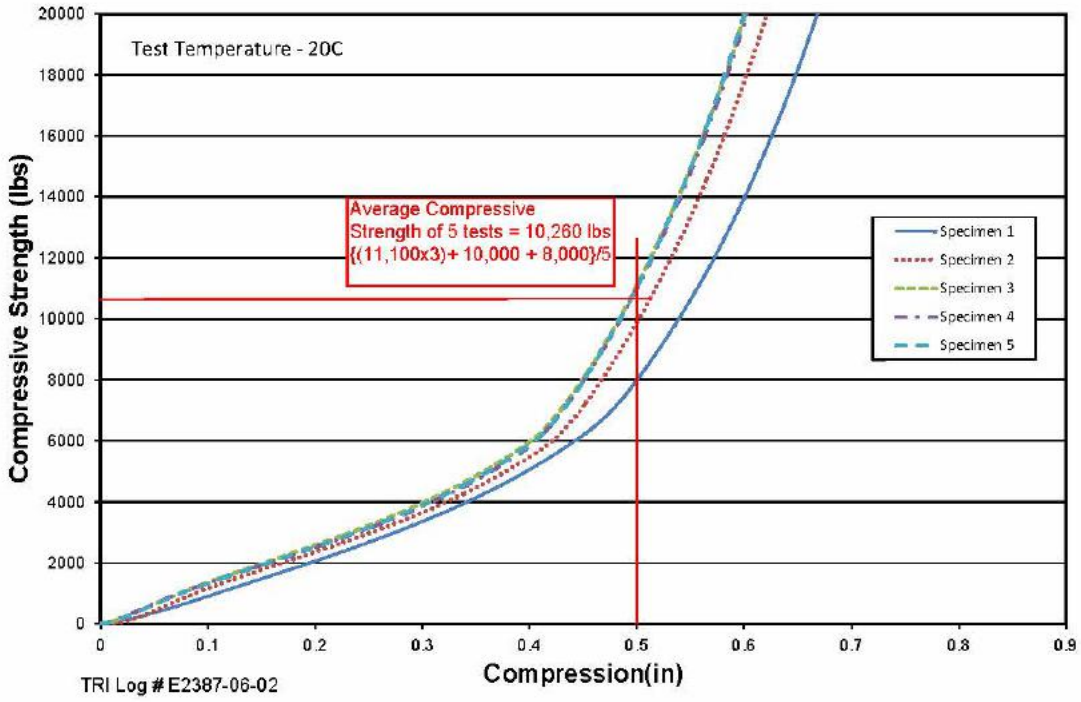
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1" Thick Ribbed Bearing Pad (English) Nov. 17, 2010

Compression Test Results
TRI Client : Reinforced Earth Co.
Product: Bearing Pad - BP1A



TRI Log # E2387-06-02

9063 Bee Caves Road / Austin, TX 78733 / 512-263-2101 / FAX 263-2558 / 800-880-TEST

REINFORCED EARTH

Bearing Pad Calculations

DATE	6/2/2015	**Bearing Pad Code BP21**
Engineer	KPB	

5x10 panels

Dimensions (inches)	2.75	WIDTH (w)	
	10	LENGTH (b)	
	0.75	THICKNESS (t)	
Durometer	80	SPEC	
(Shore A)	75	(MIN.)	

PCI Shape Factor $S = \frac{wb}{2(w+b)t}$

$S = 1.44$

PCI Allowable Compressive Stress = σ

$\sigma = 1000 + 100\,S <= 1500$

$\sigma = 1143.79$ psi

$\sigma = 805.0909$ psi Per Design Load Below

Stress Strain Test Data: TRI 5/1/2015 Report

LBS	Compression
6231	**0.125 Inch Compression**
23026	**0.25 Inch Compression**
43545	**0.375 Inch Compression**
N.T.	**0.5 Inch Compression**

Design Joint: **0.5** Inch **= t After Compression**

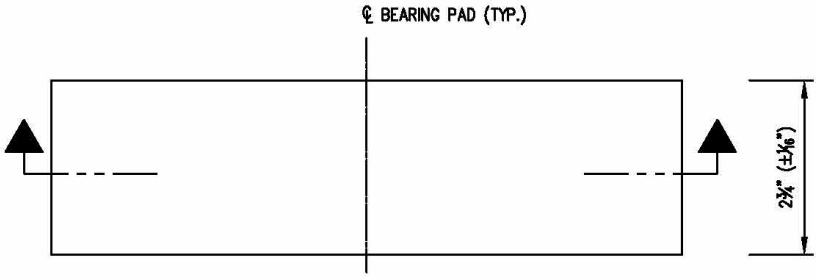
Controlling Data: 0.25 Inch Compression — 23026 LBS — Design Capacity

Design Load (per pad): 22140 LBS = (2 x H x B x T x Y)/#

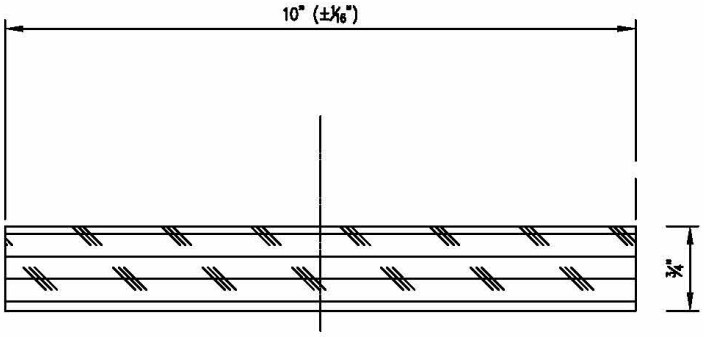
Height above Joint (H)	30	ft
Width of Panel (B)	9.84	ft
Number of Pads/Panel (#)	2	qty
Thickness of Panel (T)	0.5	ft
Unit Weight of Concrete (Y)	150	LBS/FT3
DESIGN FACTOR (x PANEL T)	2	PER FHWA

DESIGN LOAD VS. DESIGN CAPACITY

0.961522 ***OK***



PLAN VIEW



SECTION

Description:

¾" Thick Solid E.P.D.M. Bearing Pad

Item

General Information:

Manufacturer:	(RECo Approved Vender or Approved Equal)
Material Specification:	ASTM D2000–12; E, 2, BE, 8, 15, A14(Shore A), B14, C12, E014, F17 For Explanation see Bearing Pads Standard Production Specification sheet .
Comments:	Durable rubber compound integrated with high strength synthetic fibers, resulting in an integral pad with excellent physical properties and weather resistance. Durable dense rubber with excellent compressive strength, hardness, tear resistance and cold weather flexibility. "Green Product" molded from masticated rubber using discarded tires.

FABRICATION DRAWING

RECo Part #:
BP-21

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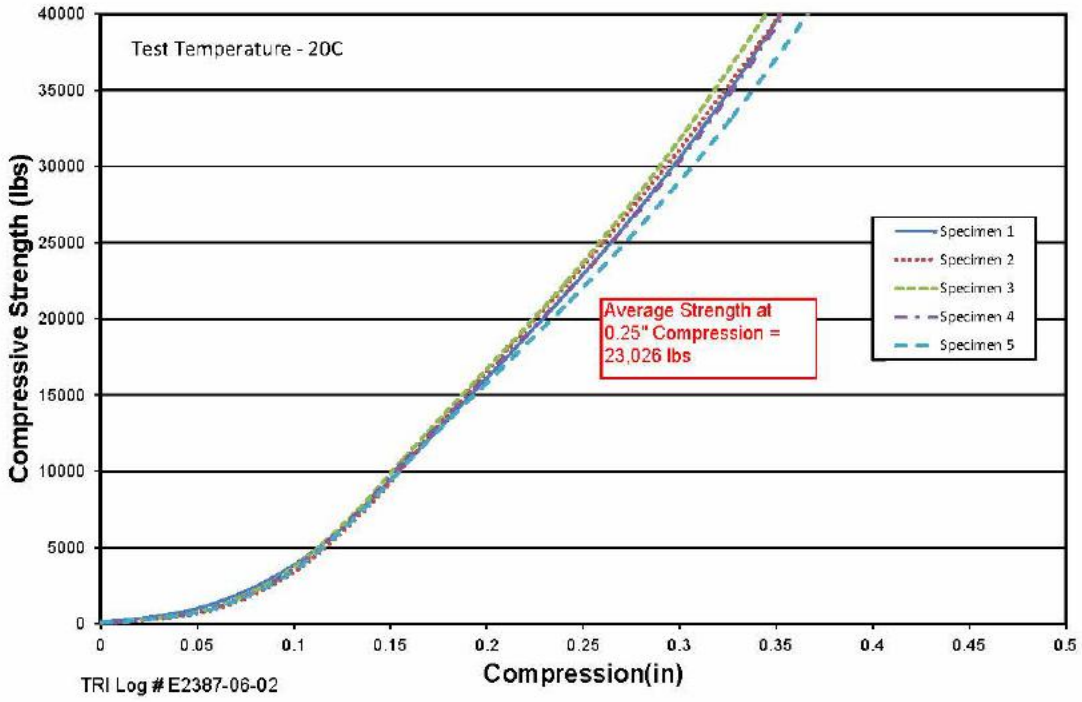
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Elongated ¾" Thick Solid E.P.D.M. Bearing Pad (English) Nov. 17, 2010

Compression Test Results
TRI Client : Reinforced Earth Co.
Product: Bearing Pad - BP21



TRI Log # E2387-06-02

9063 Bee Caves Road / Austin, TX 78733 / 512-263-2101 / FAX 263-2558 / 800-880-TEST

REINFORCED EARTH®

Bearing Pad Calculations

DATE	10/28/2021		**Bearing Pad Code BPWRLA**
Engineer	KPB		

5x5 Panels

Dimensions (inches)	2.875	WIDTH (w)	
	10	LENGTH (b)	
	0.875	THICKNESS (t)	
Durometer	80 +/-5	SPEC	
(Shore A)	75 (MIN.)		

PCI Shape Factor $S = \frac{wb}{2(w+b)t}$

$S = 1.28$

PCI Allowable Compressive Stress = σ

$\sigma = 1000 + 100\,S <= 1500$

$\sigma = 1127.60$ psi

$\sigma = 834.2609$ psi Per Design Load Below

Stress Strain Test Data: Per TRI Testing - Report Dated 8-5-2019

LBS	Compression
3752	**0.125 Inch Compression**
11491	**0.25 Inch Compression**
24710	**0.375 Inch Compression**
45000	**0.5 Inch Compression**
	0.625 Inch Compression

Design Joint: **0.5** Inch **= t After Compression**

Controlling Data: 0.375 Inch Compression — 24710 LBS Design Capacity

Design Load (per pad): 23985 LBS = (2 x H x B x T x Y)/#

Height above Joint (H)	65	ft
Width of Panel (B)	4.92	ft
Number of Pads/Panel (#)	2	qty
Thickness of Panel (T)	0.5	ft
Unit Weight of Concrete (Y)	150	LBS/FT3
DESIGN FACTOR (x PANEL T)	2	

DESIGN LOAD VS. DESIGN CAPACITY

0.97066 ***OK***

REINFORCED EARTH

Bearing Pad Calculations

DATE	10/2/2019		**Bearing Pad Code BPWRLA**
Engineer	KPB		

5x10 Panels

Dimensions (inches)	2.875	WIDTH (w)	
	10	LENGTH (b)	
	0.875	THICKNESS (t)	
Durometer	80 +/-5	SPEC	
(Shore A)	75 (MIN.)		

PCI Shape Factor $S = \frac{wb}{2(w+b)t}$

$S = 1.28$

PCI Allowable Compressive Stress = σ

$\sigma = 1000 + 100\,S <= 1500$

$\sigma = 1127.60$ psi

$\sigma = 847.0957$ psi Per Design Load Below

Stress Strain Test Data: Per TRI Testing - Report Dated 8-5-2019

LBS	Compression
3752	**0.125 Inch Compression**
11491	**0.25 Inch Compression**
24710	**0.375 Inch Compression**
45000	**0.5 Inch Compression**
	0.625 Inch Compression

Design Joint: **0.5** Inch **= t After Compression**

Controlling Data: 0.375 Inch Compression — 24710 LBS Design Capacity

Design Load (per pad): 24354 LBS = (2 x H x B x T x Y)/#

Height above Joint (H)	33	ft
Width of Panel (B)	9.84	ft
Number of Pads/Panel (#)	2	qty
Thickness of Panel (T)	0.5	ft
Unit Weight of Concrete (Y)	150	LBS/FT3
DESIGN FACTOR (x PANEL T)	2	

DESIGN LOAD VS. DESIGN CAPACITY

0.985593 ***OK***

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Bearing Pad Calculations

DATE	10/28/2021		**Bearing Pad Code BPWRLA**
Engineer	KPB		

5x10 Panels

Dimensions (inches)	2.875	WIDTH (w)	
	10	LENGTH (b)	
	0.875	THICKNESS (t)	
Durometer	80 +/-5	SPEC	
(Shore A)	75 (MIN.)		

PCI Shape Factor $S = \frac{wb}{2(w+b)t}$

$S = 1.28$

PCI Allowable Compressive Stress = σ

$\sigma = 1000 + 100\,S <= 1500$

$\sigma = 1127.60$ psi

$\sigma = 834.2609$ psi Per Design Load Below

Stress Strain Test Data: Per TRI Testing - Report Dated 8-5-2019

LBS	Compression
3752	**0.125 Inch Compression**
11491	**0.25 Inch Compression**
24710	**0.375 Inch Compression**
45000	**0.5 Inch Compression**
	0.625 Inch Compression

Design Joint: **0.5** Inch **= t After Compression**

Controlling Data: 0.375 Inch Compression — 24710 LBS Design Capacity

Design Load (per pad): 23985 LBS = (2 x H x B x T x Y)/#

Height above Joint (H)	65	ft
Width of Panel (B)	9.84	ft
Number of Pads/Panel (#)	4	qty
Thickness of Panel (T)	0.5	ft
Unit Weight of Concrete (Y)	150	LBS/FT3
DESIGN FACTOR (x PANEL T)	2	

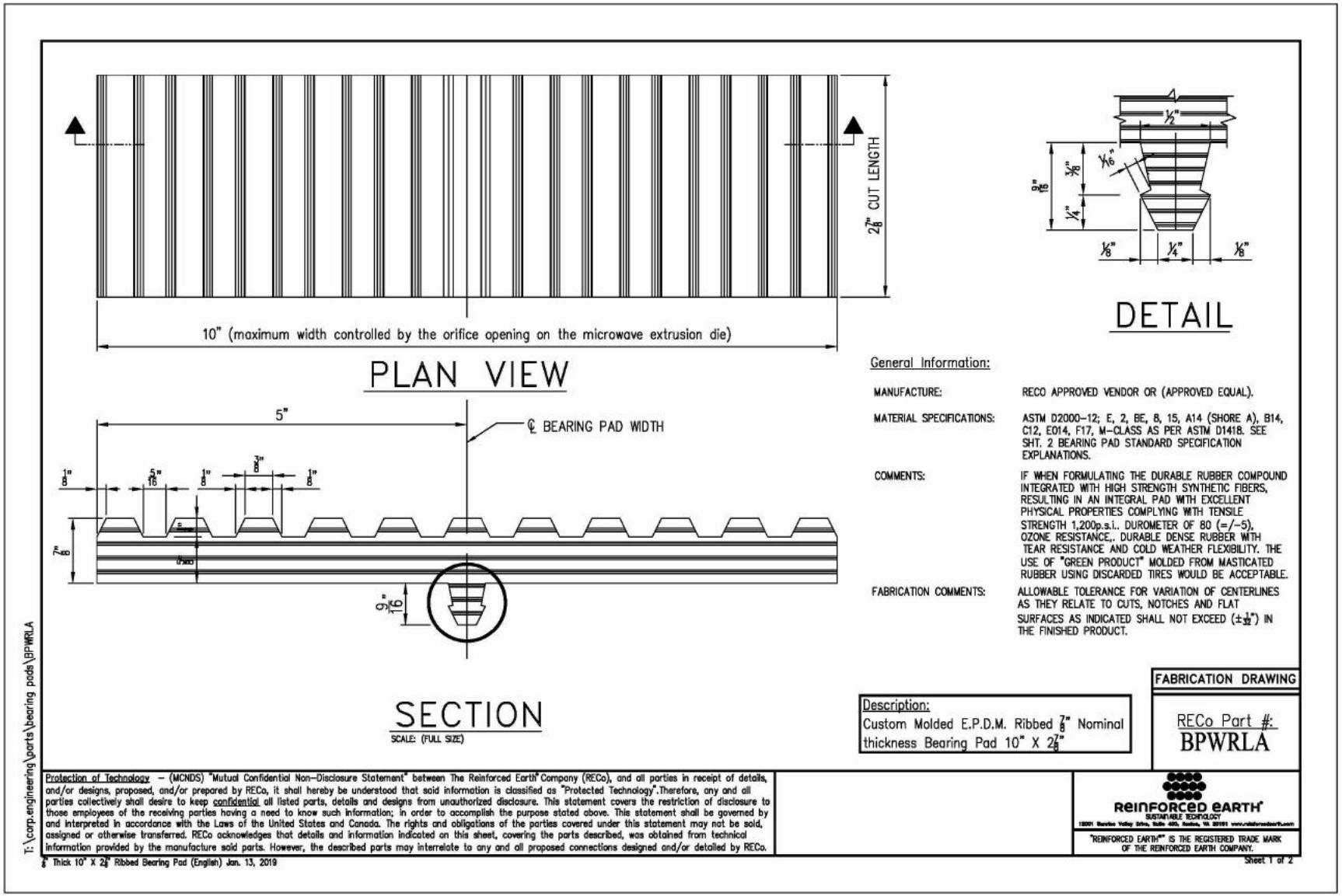
DESIGN LOAD VS. DESIGN CAPACITY

0.97066 ***OK***

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Sterling, VA 20166
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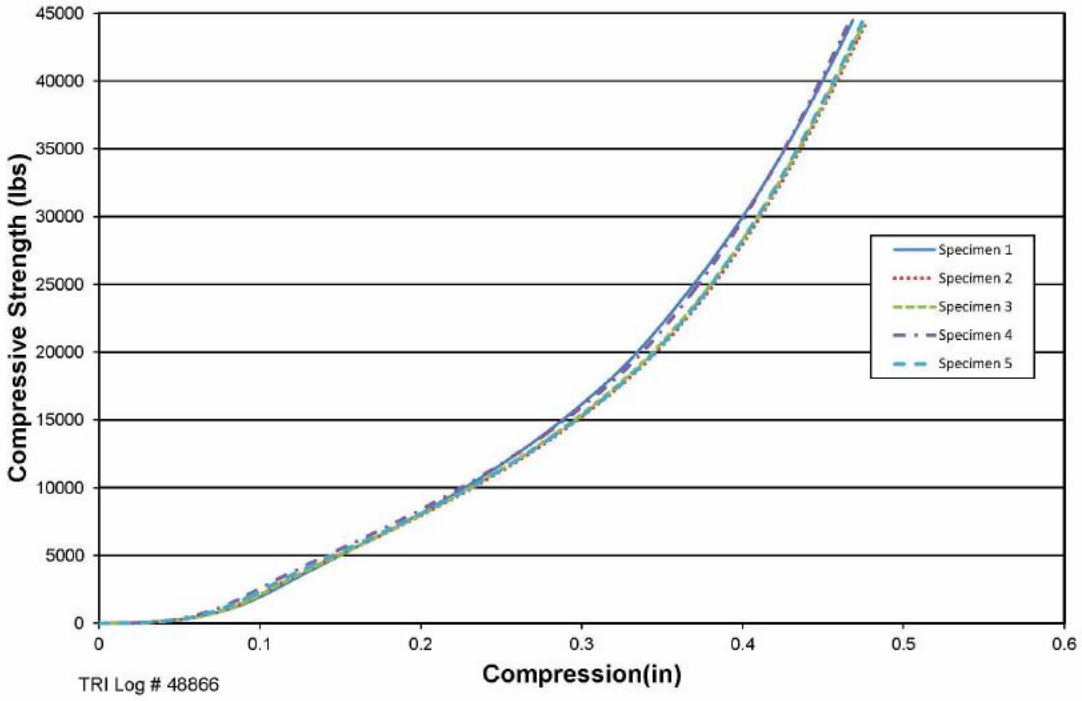
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Compression Test Results
TRI Client : Reinforced Earth Co.
Product: Bearing Pad - BPWRLA



TRI Log # 48866

TRI Environmental | 9063 Bee Cave Rd., Austin, TX 78733, USA | 512-263-5944 | www.tri-env.com

A1.1.9 Migration of Fill Prevention

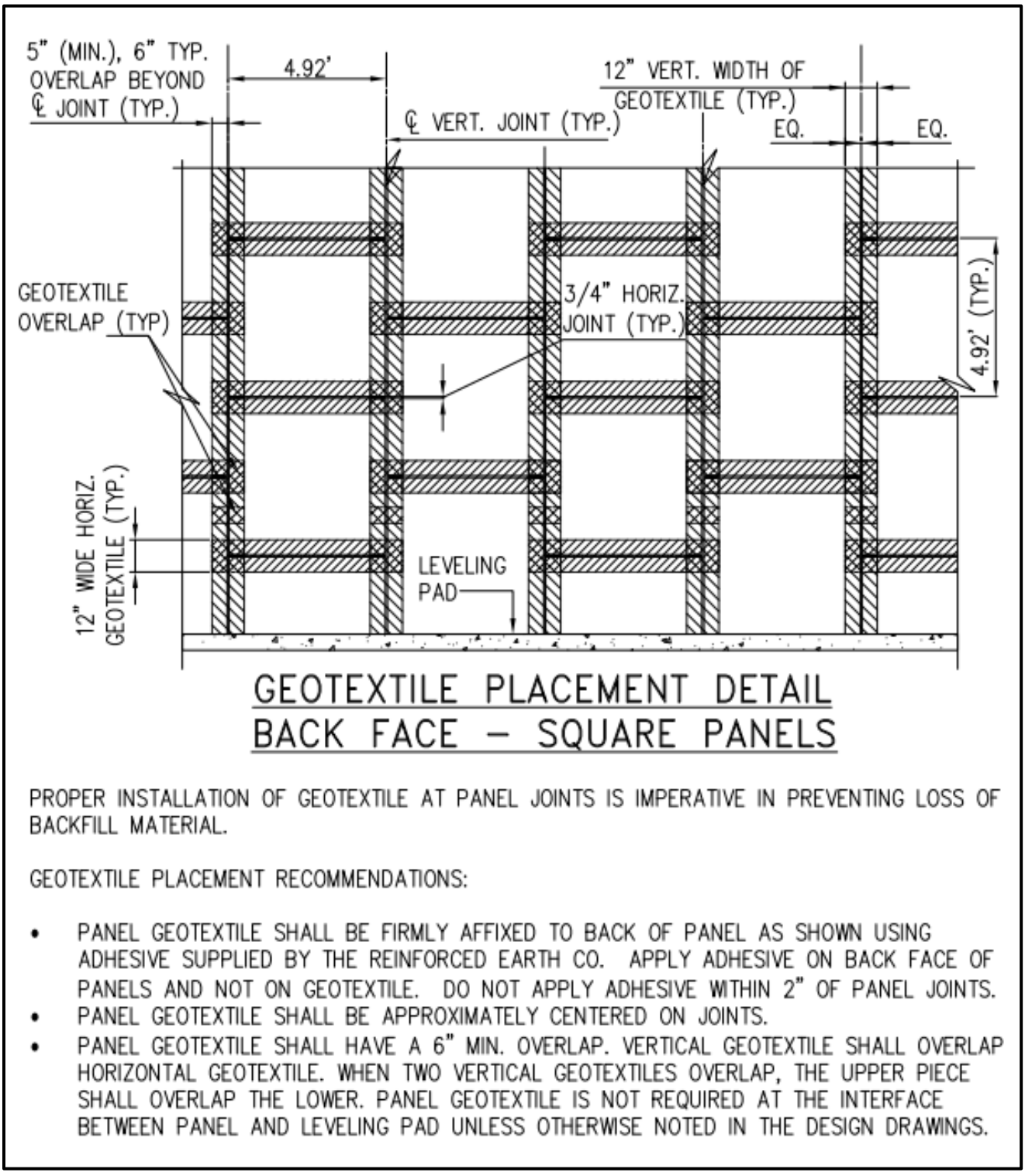


Figure A1.1.9-1: Geotextile Placement Detail – Square Panels

Square panels are shown above, for other panel types, the detail is similar.

Where required, as shown on the plans, horizontal and vertical joints between panels shall be covered by a geotextile. The geotextile may be either a non-woven needle punched polypropylene geotextile or a woven monofilament polypropylene geotextile as approved by the wall supplier. Adhesive used to

hold the geotextile filter fabric material to the rear face of the panels prior to backfill placement shall be approved by the wall supplier.

Geotextile for standard (not submerged) conditions shall (at a minimum) meet AASHTO M288 requirements for Class 3, Subsurface Drainage with the following requirements:

Requirements for panel joint geotextile – not submerged			
Backfill materials shall be granular, meeting the specifications herein.			
Minimal or no submerged conditions exist on the structure			
Compaction conditions near to the fascia panels are performed with light weight equipment as outlined in this specification.			
Geotextile is adhered to the panels by either mechanical or adhesive means.			
Non-woven polypropylene is recommended.			
Test Methods		Units	
Grab strength	ASTM D 4632	N	500
Tear strength	ASTM D 4533	N	180
Puncture strength	ASTM D 6241	N	990
Permittivity	ASTMD 4491	sec^{-1}	0.5
Apparent opening size	ASTM D 4751	Mm	0.425 max
Ultraviolet stability (retained strength after 500 hours' exposure)	ASTM D 4355	%	50 @

Table A1.1.9a

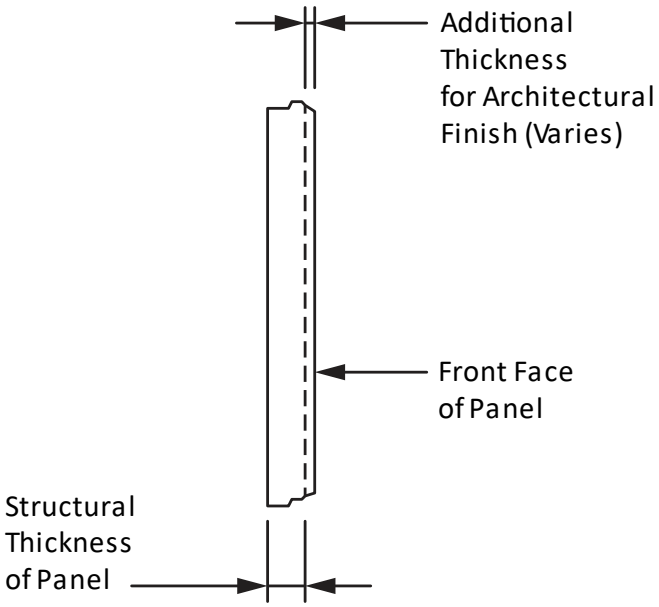
Requirements for panel joint geotextile SUBMERGED conditions			
Backfill materials shall be granular, meeting the specifications herein.			
MSE structure is considered Submerged if subjected to inundation for a period longer than 10 continuous days. The conditions in this case may include rapid drawdown, submerged or partially submerged sections of the wall.			
Scour depth of the toe of the MSE mass shall be designed AND protected as per the Engineer.			
Compaction conditions near to the fascia panels are performed with light weight equipment as outlined in this specification.			
Geotextile is adhered to the panels by either mechanical or adhesive means.			
Woven monofilament polypropylene is recommended.			
Test Methods		Units	
Grab strength	ASTM D 4632	N	800
Tear strength	ASTM D 4533	N	300
Puncture strength	ASTM D 6241	N	1650
Permittivity	ASTMD 4491	sec^{-1}	0.5
Apparent opening size	ASTM D 4751	mm	0.425 max
Ultraviolet stability (retained strength after 500 hours' exposure)	ASTM D 4355	%	50 @

Table A1.1.9b

All numeric values represent MARV in the weaker principal direction except AOS which represents the MAXIMUM average roll value.

A1.1.10 Aesthetics

The following detail shows the added architectural finish in relation to the structural thickness of a panel:



Section: Facing Panel

The following are examples of architectural finish types - plain smooth concrete, repeating patterns, textures, replicated materials, and custom murals:



Figure A1.1.10-1: Plain smooth concrete panel finish



Figure A1.1.10-2: Repeating pattern



Figure A1.1.10-3: Texture (fractured fin)



Figure A1.1.10-4: Replicated material (stacked stone)



Figure A1.1.10-5: Custom mural

The following is an 8-page marketing brochure with more examples of aesthetic options for Reinforced Earth walls:

Architectural Applications

REINFORCED EARTH



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Our Walls - Your Canvas

The Reinforced Earth Company (RECo) has developed a wealth of experience in combining art with engineering. Our large precast concrete facing panels provide a durable "canvas" for the artist, allowing for a design that stands the test of time. A Reinforced Earth® Mechanically Stabilized Earth (MSE) retaining wall is a versatile medium that is perfect for prominently displaying the unique history, values, interests, or brand of any community or property owner.

Collaborating with the architect or artist, Reinforced Earth's engineers will incorporate the desired artwork into the design plans of the retaining wall. Final renderings can show the client the end result of the project. Our precast division will transfer the artwork onto the facing panels with great precision, providing the desired three-dimensional relief and surface textures. A finishing touch of color can be added by the artist on site.

Retaining walls are abundant and serve a vital role in our infrastructure. However, they are often overlooked by the public or seen as purely functional. Murals and other forms of art help etch a community's identity into the minds of residents and passersby, bringing people together with a sense of pride.

Whether it's a simple repeating design or an elaborate mural by a local artist, The Reinforced Earth Company has the ability to turn your retaining walls into a visually pleasing work of art.



FanWall® Precast Noise Barrier System



Reinforced Earth® MSE Retaining Wall System



TechSpan® Precast Concrete Arch System

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IH-20 @ JBS Parkway, Odessa, TX



I-70 to Indianapolis Airport, Indianapolis, IN



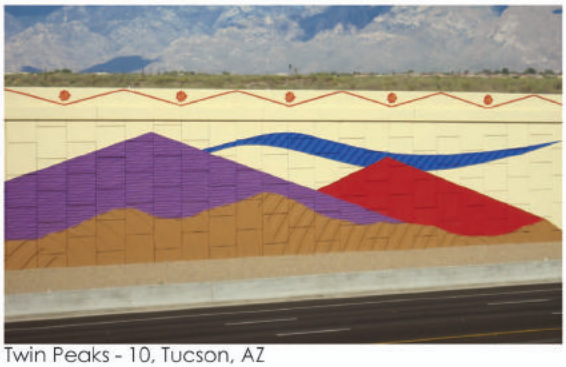
I-15 Design Build, Las Vegas, NV



Western Wake Freeway, Wake County, NC



146th Street over US-31, Hamilton County, IN



Twin Peaks - 10, Tucson, AZ



I-35 @ Tres Equis Blvd, Laredo, TX



Rt. 18, Sect. 5F, 7E & 11th, New Brunswick, NJ

Limitless Creative Possibilities

Design Solutions That Work for You

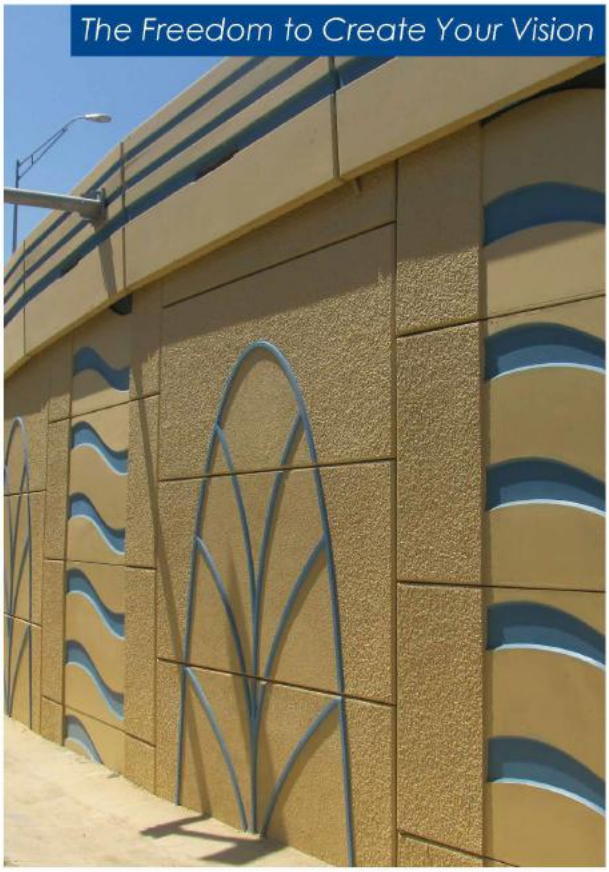
Reinforced Earth precast concrete panels offers the flexibility to create an amazing variety of effects.



Project Wide Theme

The Reinforced Earth Company has completed over 45,000 structures in the US, building a reputation for engineering excellence, architectural creativity, and an unyielding focus on quality and customer support. With over 45 years of experience, RECo brings structural, geotechnical and economic value to projects of all types, sizes and complexities

Reinforced Earth retaining structures are an economical way to meet ordinary and extraordinary earth retention and load support needs for highways and bridges, railroads and mass transit systems, waterfronts, airports, loading docks, industrial and mining facilities and commercial and residential developments. Each wall is a coherent gravity structure, custom-engineered by RECo to project-specific requirements including applied loading, foundation conditions, and aesthetics.





A First and Lasting Impression

Complimentary Design to Existing Structures

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Showcase Community Spirit





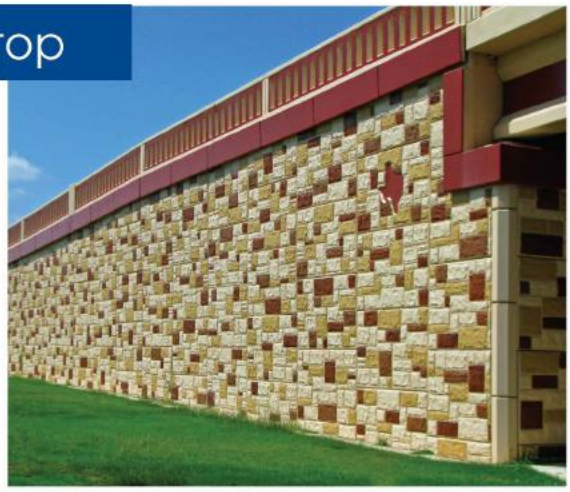
Enhance Existing Environments





Provide an Artistic Backdrop





Endless Architectural Options

Panels are erected in an offset alternate pattern, automatically creating a distinctive and pleasing appearance in every Reinforced Earth wall. You can further enhance the beauty of your structure by using one of our many standard architectural form liners, or we can customize a liner pattern to meet your needs. Several examples are shown here, but contact us for more information about the architectural result you desire.



Fractured Fin



Ashlar Stone



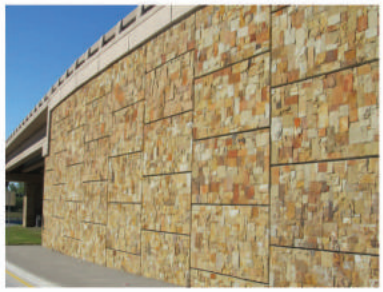
Raised Panel



Brick Texture



Geometric Design



Stone Texture



Smooth Panel / Fractured Fin



Ashlar Stone / Custom



Running Bond



Rustication



Custom Mural



Custom Graphics



RTD West Corridor, Denver, CO



Johnnie Dodds Boulevard, Mt. Pleasant, SC



US 80 @ Collins Road, Sunnyvale, TX



I225 & Colfax, Denver, CO



Marquette Interchange, Milwaukee, WI



I-275, Tampa, FL



SR 92 Timpanogos Hwy, Lehi, UT



I-235 / US 77 @ 36th Street, Oklahoma City, OK



US 62/83 @ BNSF Railroad, Childress, TX

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A1.1.11 Curved Alignment Limitations

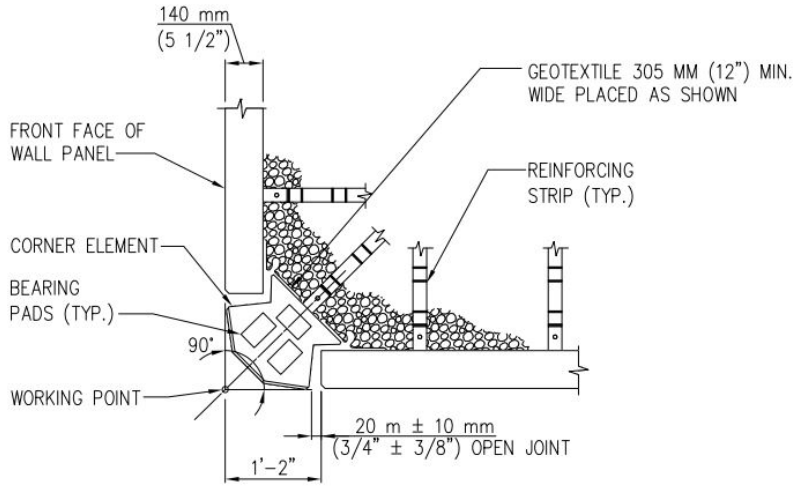


Figure A1.1.11-1: 90 deg. corner element detail

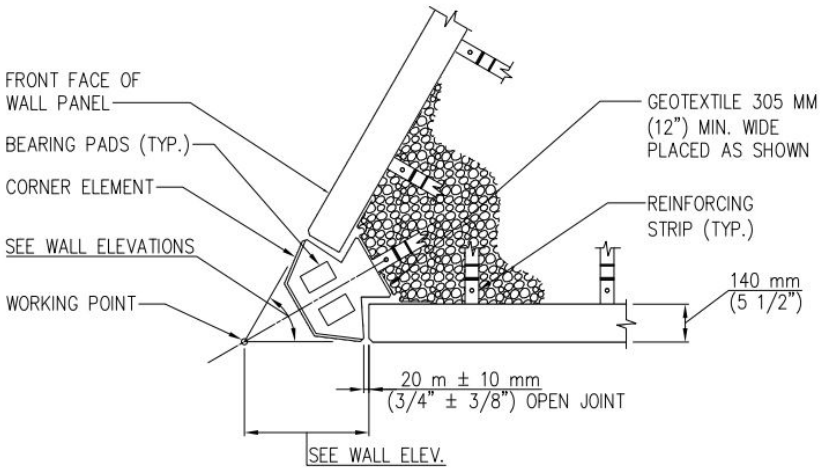


Figure A1.1.11-2: Acute corner element detail

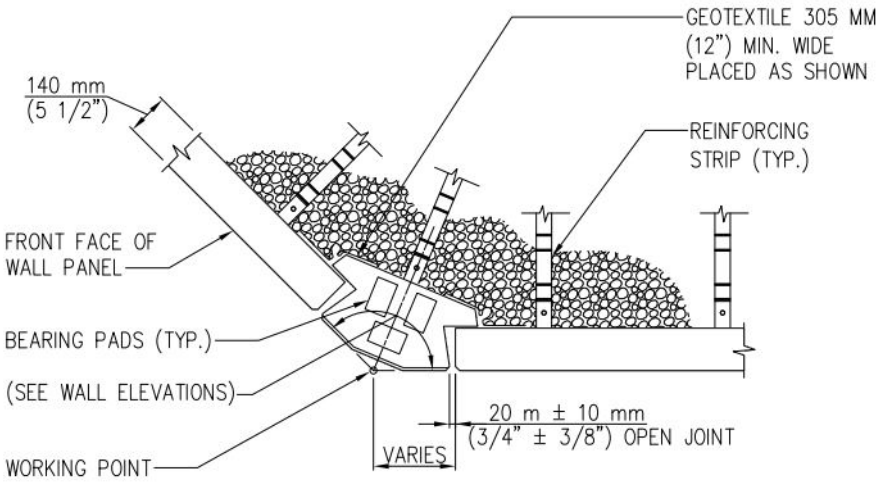


Figure A1.1.11-3: Obtuse corner element detail

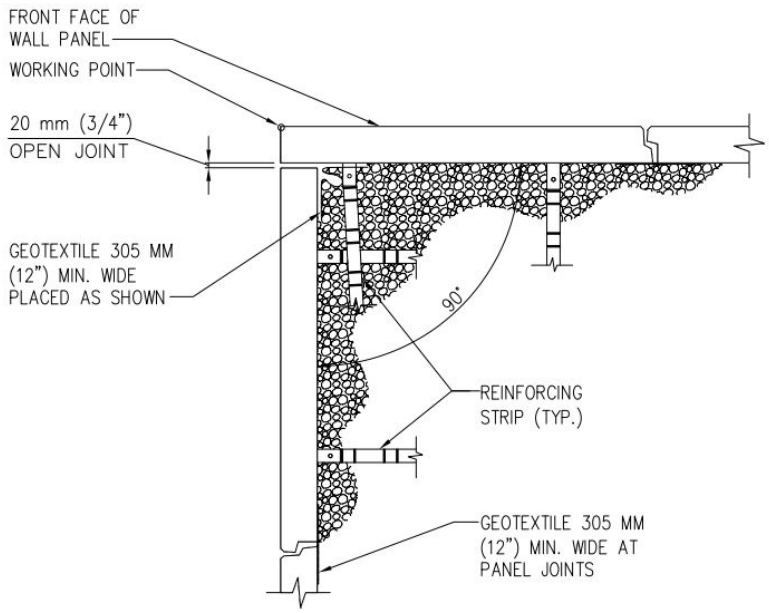


Figure A1.1.11-4: 90 deg. butt joint detail

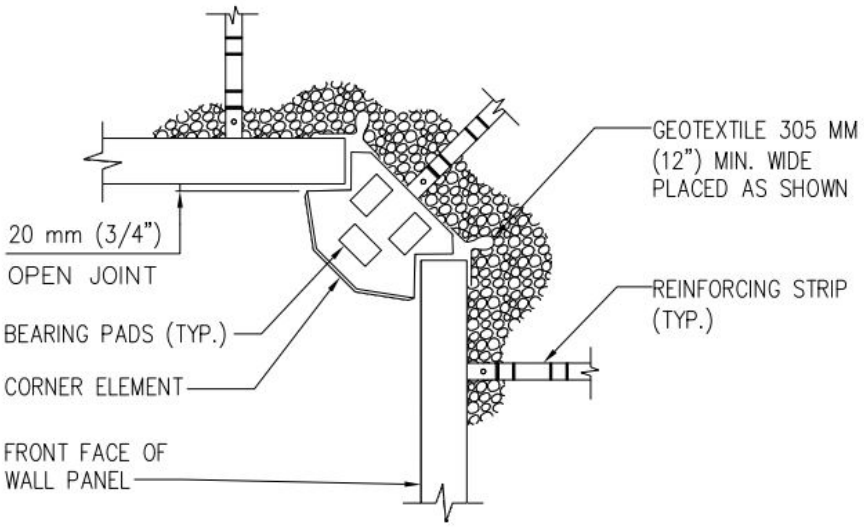


Figure A1.1.11-5: 270 deg. corner element detail

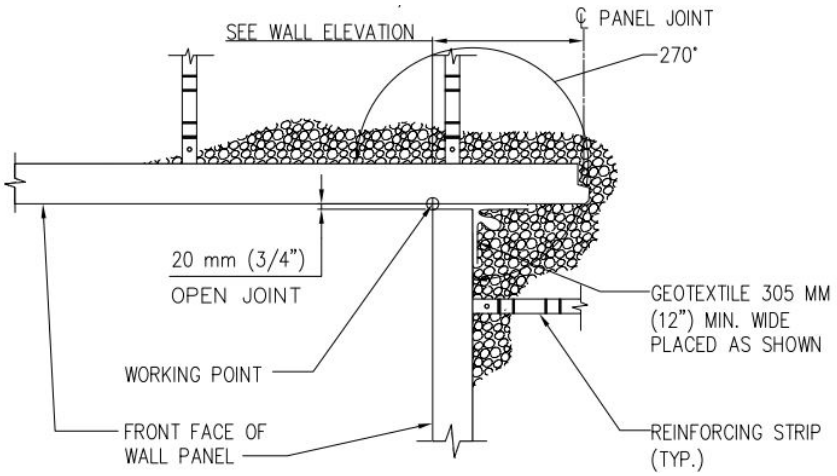


Figure A1.1.11-6: 270 deg. butt joint detail

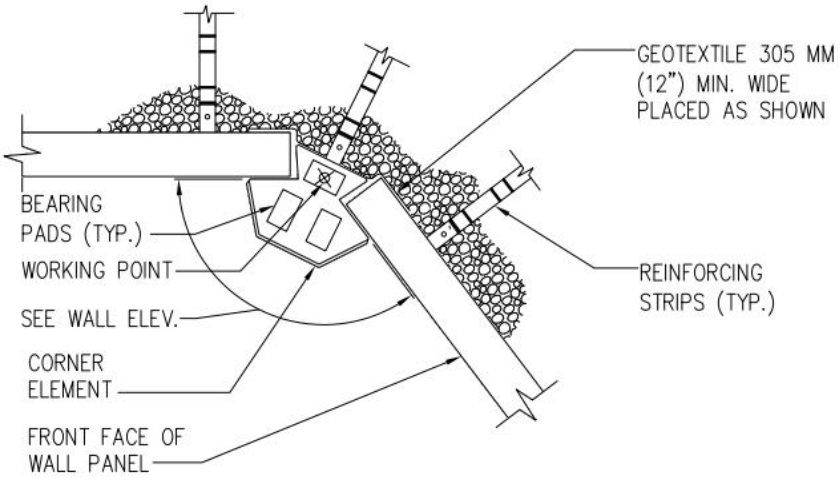


Figure A1.1.11-7: Obtuse inside corner element detail

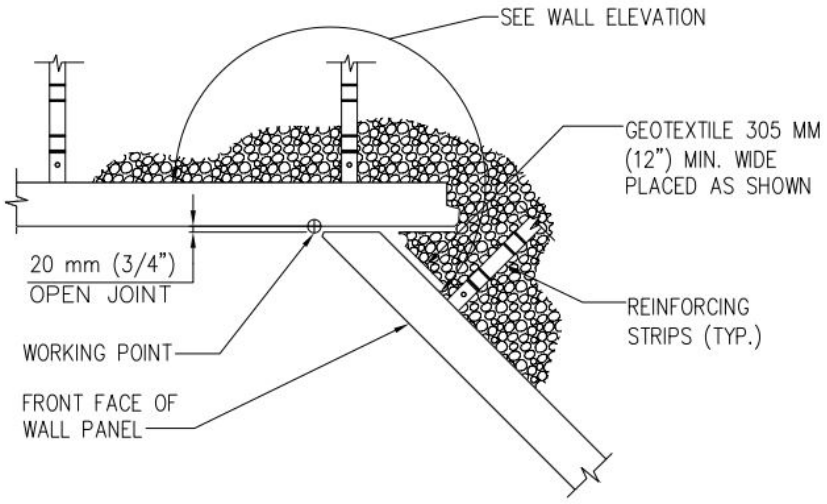


Figure A1.1.11-8: Obtuse inside butt joint detail

A1.2 Inextensible Reinforcement

This section is intentionally left blank.

A1.2.1 Innovations

The following documentation is included in this section:

- RECo's Tech Bulletin 2019-01, which describes the most up to date practice for the use of f* for Ribbed Steel Strips. (24 pages)
- MSE Pullout Testing for RECo HA Ladder and HA Strip Reinforcements, Texas Tech University, December 21, 2011. (290 pages)
- Research paper by Jayawickrama, et al "Effect of Skewing and Splaying on Pullout Capacity of Steel MSE Reinforcement". (19 pages)
- Reinforcing Strip splice details. (2 pages)

Pullout Resistance and Apparent Coefficient of Friction (f*) of High Adherence (HA) Reinforcing Strips In AASHTO MSE Backfill

Tech Bulletin 2019

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Rev.	Date	Modification	Design by	Checked by	Approved by
1	Jan 3, 2020	Updated Figures	KPB		
Orig	March 19, 2019	Original	KPB	LY, RL	KPB

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Pullout Resistance of Inextensible Mechanically Stabilized Earth Reinforcements

The internal stability design of a Mechanically Stabilized Earth (MSE) structure is composed of two design checks: (1) Pullout Resistance and (2) Reinforcement Strength. This document will focus on the development of Pullout Resistance Friction Factors (f*) for inextensible (steel) reinforcements for use in design of safe MSE walls.

As shown in Figure 1 (an MSE wall reinforced with inextensible steel reinforcements and level ground at the top of wall), the pullout of reinforcements in a MSE structure is resisted by the friction between the soil reinforcements and select fill in the "resistive zone". To determine the soil reinforcement pullout resistance, the "apparent coefficient of friction" (f*) needs to be understood. f* is a function of the soil reinforcement geometry, degree of compaction, shear strength and particle size(s) of the select granular backfill.

f* is determined by two methods: (1) default values in the AASHTO code (Figure 2) or (2) product specific pullout tests in different types of backfill. It should be noted that the default values in the AASHTO code are based on historical data with a multitude of backfill types (with the backfills defining the default envelope actually not meeting current AASHTO requirements for gradation, shear strength, PI and durability), resulting in a conservative envelope for design that can be used whenever the backfill type for the project is not known at the onset of MSE wall design. However, the purpose of this document is to show that higher values for f* may be used when the backfill type is known.

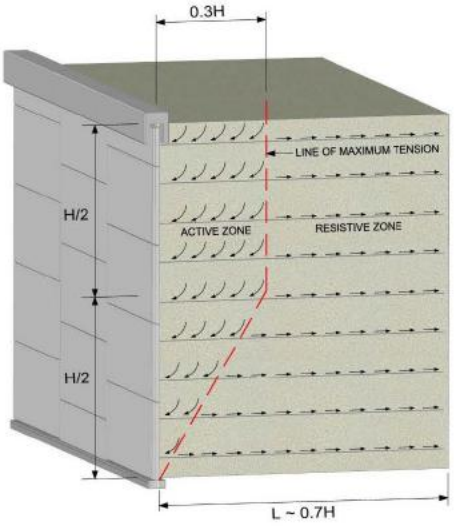


Figure 1

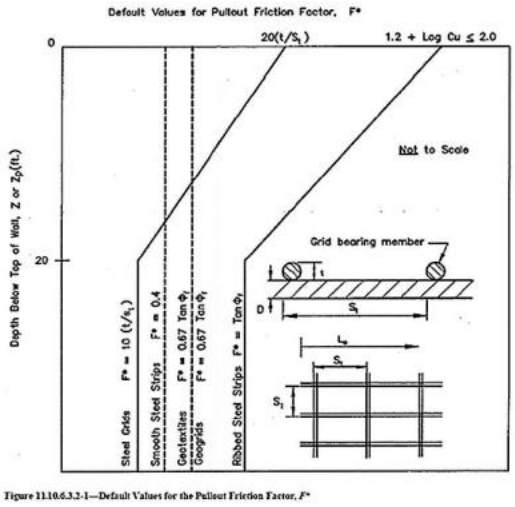


Figure 11.10.6.3.2-1—Default Values for the Pullout Friction Factor, F*

Figure 2

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When testing soil reinforcements for pullout, it is important to provide a test setup that is comparable to the field placement of reinforcements, i.e. the backfill should be placed and compacted, but not otherwise sculpted to compensate for inherent voids below the reinforcement that are a function of its geometry (none of which occur when using linear Reinforced Earth High Adherance Ribbed Strips). Once the reinforcements are placed on a compacted layer of fill, additional compacted fill layer(s) are placed and then an overburden stress is applied by jacks, weights or air bladders to simulate the normal load at the given depth of reinforcement below grade.



Photo 1

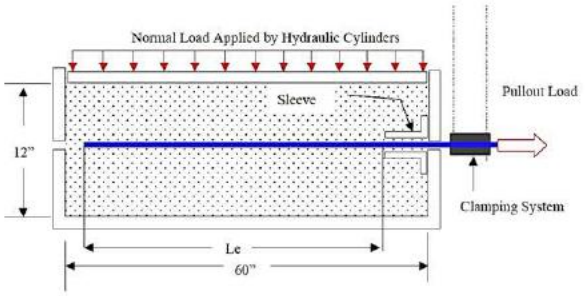


Figure 3

(Photo 1 and Figure 3 courtesy of SGI Testing Services, Atlanta, GA)

To determine f* values from pullout tests, an adequate number of tests are necessary at varying overburden pressures. For reinforcement types and/or backfill types not already extensively tested, at least two tests should be performed at each of the following overburden pressures: 200 psf, 600 psf, 1200 psf, 2400 psf and 4000 psf. Due to the statistical variance inherent in testing, it is important to perform enough tests to establish a confidence level in the data collected. Simply relying on a single test at one or two overburden pressures should only be done when confirming standard design parameters, not when proposing design values for new reinforcement types or backfills.

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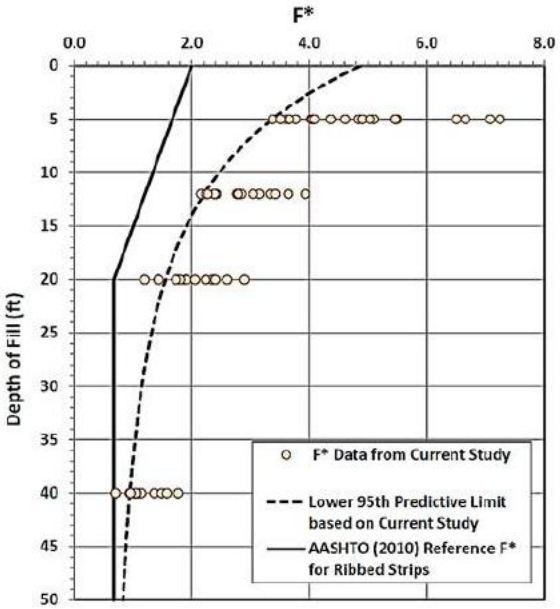


FIG. 4. Pullout resistance factor, F* vs. Depth of Fill for HA strip reinforcements in sandy backfill, properly compacted (95% Rel. Comp.)

Figure 4

(Figure 4 Reference: "Pullout Resistance Factors for Inextensible MSE Reinforcements Embedded in Sandy Backfill", Lawson, et. al, TRB 2013 Annual Meeting)

As shown in Figure 4 above, the pullout resistance of a Reinforced Earth® (RECo) 50mmx4mm ribbed reinforcing strip in a sandy backfill far exceeds the default value provided in the AASHTO code. The C_u of the sandy backfill tested was 4.7. According to the AASHTO default formula $f^*=1.2+\log C_u$, the f^*_o design value would be 1.87 (at the ground surface). However, testing shows that a safe and realistic design value is more on the order of $f^*_o = 4.0$ for this material. The design value at depths of 20 ft should be limited to tan Ø, (where Ø is the shear strength of the select backfill), regardless of reinforcement type and lab test data. The reason for this limitation is illustrated below in Figure 6 and discussed later in this document.

Like the behavior of sandy materials, angular crushed stone material has excellent pullout resistance with RECo ribbed reinforcements as compared to the AASHTO model (Figure 5). For crushed stone material, the AASHTO formula of $1.2+\log C_u$ is simply not valid (as the typical C_u ranges from 2 to 4). The angularity and interlock of aggregate provides for a significant frictional resistance, far more than the calculated value of $1.2+\log 2 = 1.5$. A realistic design value is more on the order of $f^*_o = 4.5$ (at ground surface), note a value of $f^*_o = 8.0$ as plotted on the envelope, while standard practice is to limit the design value to a maximum of 4.5.

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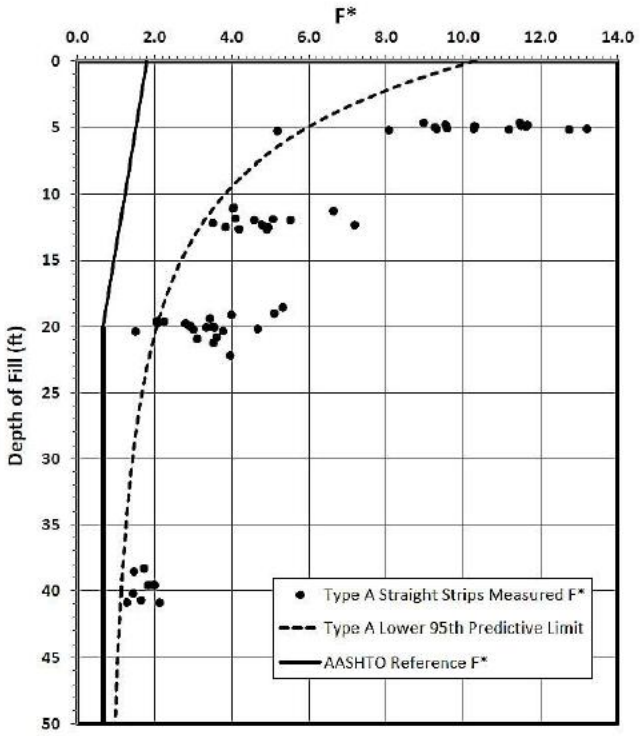


Figure 6.17 F* versus Depth of Fill Chart for Ribbed Strip Reinforcements Embedded in Type A Backfill (Ordinary Compaction, Skew Angle = 0°)

Figure 5

(Figure 5 Reference: "Jayawickrama, Surles, Wood, Lawson. Pullout Resistance Factors Mechanically Stabilized Reinforcements in Backfills Typically used in Texas, TXDOT Report 0-6493-1, August 2013.

The Reinforced Earth Company has sufficient pullout data from hundreds of tests performed in more than forty (40) different backfills that range from silty sands to open graded crushed stone. This wealth of data allows us to optimize our design for virtually any project.

Limiting f* to tan Ø at depths greater than 20 ft (2500 psf)

In a full-scale structure, as reinforcements are subjected to higher overburden pressures (>2500psf) with depth and the effects of dilation are diminished and at a depth greater than 20 ft, the reinforcement is relying mostly on the interface shear of the element and the select backfill. Reviewing engineers should take note of this fact as some MSE wall systems are (perhaps unknowingly) over-relying on lab test data and not correlating lab tests with field testing of full-scale structures. Furthermore, as many tested backfills have Ø angles greater than the design Ø angle of 34 degrees (typical for sands and gravels), lab tests at high overburdens will indicate f* values greater than the shear strength of the soil that is mobilized during the service life of an MSE wall.

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Observation– Ribbed strips exhibit f* values greater than tan Ø at 20 ft. depth

Ribbed strips take advantage of soil dilatancy under low confining stress. The effect of dilatancy is shown in figures 2,4,5,7 & 8 where the coefficient f* is maximum at the ground surface and decreases with depth. The following diagram (Figure 6) illustrates the phenomenon of dilatancy for ribbed strips. This phenomenon occurs with all steel reinforcement types (with smooth strips having the lowest dilatancy, and wire mats or other strips with other geometric configurations and ribbed strips exhibiting similar effects of dilatancy).

As the effects of dilatancy diminish with increasing depth of overburden, the design pullout resistance of ribbed (***and all other soil reinforcements***) are limited by the soil to soil shear strength which is defined by tan Ø at 20 ft. depth.

The pullout resistance in a test box always exceeds tan Ø (design) due to the following:

- Peak Shear Strength of the soil is mobilized during a pullout test. The peak shear strength is often significantly greater than the design shear strength (typically taken as 34 degrees for granular soils). For crushed stone backfills, the peak shear strength is on the order of 45 to 50 degrees, and the design shear strength is limited to 40 degrees per AASHTO.

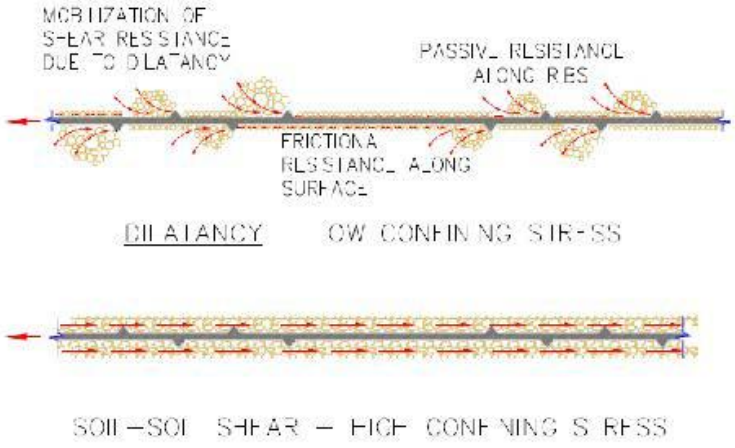


Figure 6 Ribbed Strips – Frictional Behavior with Depth

- The peak shear strength of the soil is not developed within an MSE structure because the deformations necessary for it to develop do not occur at service loading conditions in actual structures reinforced with inextensible steel soil reinforcements.
- *f* should be measured at ¾" displacement (this displacement is measured at the leading end for inextensible reinforcements). Extending the tests to 3" to 4" of displacement is incorrect and should be avoided because when large displacement occurs, another phenomenon occurs, namely* ***induced***

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***dilation**, which is the piling up of the soil at the face of the pullout test box, resulting in exaggerated pullout resistances that cannot develop in actual MSE structures*

Note: The 20 ft (6m) depth threshold was chosen nearly 30 years ago based on the information available at the time. More recent large-scale lab and full-scale instrumented field testing indicates the depth to consider limiting pullout resistance to tan Ø may be as deep as 30 to 40 ft. However, the AASHTO design requirements are clear. f* shall be limited to tan Ø at 20 ft. depth for normal weight fills (or 2500 psf overburden for lightweight fills)– FOR ALL SOIL REINFORCEMENT TYPES.

Pullout resistance in full scale structures – Ribbed Metallic Strips

Figure 7 shows the measured apparent coefficient of friction in a full-scale structure constructed with RECo ribbed strips. The backfill for this structure was a poorly graded sand with gravel, with a peak shear strength (Ø) of 38 degrees.

Figure 8 shows the measured apparent coefficient of friction in a large size pullout box (12'x12'x 4' deep), testing RECo ribbed strips. The backfill for this test was a fine to medium sand (SP-SM) with a peak shear strength (Ø) of 39 to 43 degrees.

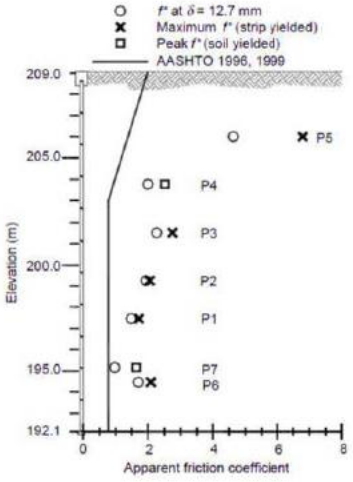


Figure 7

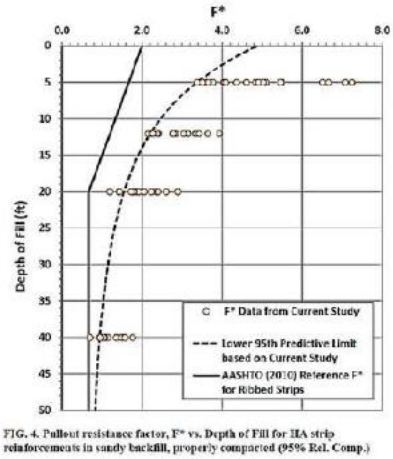


FIG. 4. Pullout resistance factor, F* vs. Depth of Fill for IIA strip reinforcements in sandy backfill, properly compacted (95% Rel. Comp.)

Figure 8

(Figure 7 Reference: Runser, Fox and Bourdeau, "Field Performance of a 17m High Reinforced Soil Retaining Wall", Geosynthetics International, 2001, Vol. 8, No. 5.)

(Figure 8 Reference: "Pullout Resistance Factors for Inextensible MSE Reinforcements Embedded in Sandy Backfill", Lawson, et. al, TRB 2013 Annual Meeting)

Note the similarities in f* with depth for the small-scale pullout testing (Figure 10), full-scale wall (Figure 7) and large-scale pullout box testing (Figure 8).

As shown above, Ribbed Strips exhibit measured Apparent Coefficient of Friction (f*) values greater than tan Ø at 20 ft depth. However, for design purposes, the f* value should be limited to tan Ø as this is the value set forth in AASHTO.

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AASHTO Requirements for Apparent Coefficient of Friction (f)*

(REF: AASHTO LRFD Bridge Design Specifications, Section 11.10.6.3.2 – Reinforcement Pullout Design)

"F and α shall be determined from product-specific pullout tests in the project backfill material or* ***equivalent soil****, or they can be estimated empirically/theoretically.*

For standard backfill materials (see AASHTO LRFD Bridge Construction Specifications, Article 7.3.6.3), except for uniform sands, i.e. coefficient of uniformity, Cu=D60/D10 <4. In absence of test data, it is acceptable to use conservative default values for F and α as shown in Figure 11.10.6.3.2-2 and Table 11.10.6.3.2-1. For ribbed steel strips, if the specific Cu for the wall backfill is unknown at the time of design, a Cu of 4.0 should be assumed for design to determine f*."*

The following information will show that The Reinforced Earth Company has adequate test data to characterize pullout resistance factors for nearly any backfill meeting the AASHTO gradation, shear strength and durability requirements, and thus may classify backfills as "equivalent soils" with respect to design values for f*.

AASHTO Backfill Recommendations

The AASHTO MSE Backfill recommendations were established with AASHTO Task Force 27 in 1990.

MSE Backfill

(REF: AASHTO LRFD Bridge Construction Specifications, Section 7.3.6.3)

"Structure backfill for mechanically stabilized earth walls shall conform to the following grading, internal friction angle and soundness requirements:

Sieve Size	*Percent Passing*
4.0 in (100 mm)	100
No. 40 (425 μm)	0 – 60
No. 200 (70 μm)	0-15

* Plasticity Index (PI), as determined by AASHTO T 90, shall not exceed 6.

The material shall exhibit an angle of internal friction of not less than 34 degrees, as determined by the Direct Shear Test, AASHTO T236 (ASTM D 3080), on a portion finer than the No. 10 (2.0 mm) sieve, utilizing a sample of the material compacted to 95 percent of AASHTO T99, Methods C or D (with oversized correction as outlined in Note 7) at optimum moisture content. No testing is required for backfills where 80 percent of sizes are greater than 0.75 in. (19 mm).

The materials shall be substantially free of shale or other soft, poor durability particles. The material shall have a magnesium sulfate soundness loss of less than 30 percent after four cycles."

The following table includes 26 different backfill gradations that are the basis of RECo's equivalent soils designations.

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Summary of Backfill Properties and f*o

Test ID	Backfill Description (ASTM D2487)	Avg. % Passing 4" (100mm)	Avg. % Passing 3/4" (20 mm)	Avg. % Passing No.4 (5 mm)	Avg. % Passing No.10 (2 mm)	Avg. % Passing No.40 (425 μm)	Avg. % Passing No.200 (70 μm)	D60	D10	Cu	Shear Strength	f*o
								Average			Φ avg	at surface
Case 2 - Gravelly Silty Sand	Silty Sand with Gravel (SM)	100	85.1	66.3	58.1	38.4	15.1	3	0.08	37.5	42	4.5
Case 3 - Concrete Sand	Concrete Sand (SW)	100	100	96.6	72.1	30.7	3.4	1.3	0.175	7.4	-	3.5
Case 4 - KU Gravel 1	Poorly Graded Gravel (GP)	100	82.4	0	0	0	0	16	11.5	1.4	46	2.9
Case 4 - KU Gravel 2	Poorly Graded Gravel (GP)	100	82.9	1	1	0.6	0.4	16	8	2.0	49	2.9
Case 4 - KU Gravel 3 #57	Poorly Graded Gravel (GP)	100	87.4	7.7	5	1.8	0.2	15	5	3.0	47	3.2
Case 4 - KU Gravel 4 #68	Poorly Graded Gravel (GP)	100	89.3	11.8	7	0.6	0.4	12	3	4.0	47	3.2
Case 4 - KU Gravel 5 #68	Well Graded Gravel with Sand (GW)	100	92.5	17.2	11	0.5	0.4	11	1.75	6.3	49	3.3
Case 4 - KU Gravel 6 #68	Well Graded Gravel with Sand (GW)	100	90.1	21	15	4.6	2.4	14	1	14.0	46	3.2
Case 5 - TX Type A Gravel #467	Poorly Graded Gravel (GP)	100	49.9	2.8	2	1	0.9	15	10	1.5	52	3.75
Case 6 - TX Crushed Limestone #57	1" crushed limestone (AASHTO #57) (GP)	100	84	9	0	0	0	15.7	6.25	2.5	52	5.4
Case 8 - GA Graded Aggregate	GADOT Graded Aggregate Base - Well Graded Gravel with Sand (GW)	100	76.25	55	36.25	25	9.25	6.35	0.9	7.1	42	5.4
Case 10 1.1 Madrid Fine Sand	(Fine Sand) - Poorly Graded Sand with Silt (SP-SM)	100	100	100	100	70	6	0.45	0.125	3.6	37	5.8
Case 15 - Triel Sand Gravel	Well Graded Gravel with Sand (GW)	100	82	47	39	14	2	9	0.35	25.7	38	4.7
Case 17 Gravelly Sand NJ	Gravelly Sand (SP)	100	99.9	71.6	61.1	20.1	0.1	2	0.29	6.8	38	3.7
Case 18 TX Type B Sand	TX Type B Sand (SP-SM)	100	-	99.2	-	66.3	9	0.37	0.08	4.6	41	4
Case 18 2 TX Type A Gravel	TX Type A Gravel (GW)	100	-	29	17	7	1	13	0.7	18.6	52	6.3
Case 20 - FL Sand 1	Poorly Graded Fine Sand (SP)	100	100	100	100	93.5	3.7	0.22	0.09	2.4	34	2.75
Case 21 - FL Sand 2	Poorly Graded Fine Sand (SP)	100	100	99.8	99.6	87.4	4.5	0.25	0.1	2.5	35	3.1
Case 22 - FL Sand 3	Poorly Graded Fine Sand (SP)	100	99.1	97.2	96	86	4.6	0.24	0.11	2.2	35	3.05
Case 23 - FL 3" Limestone	Poorly Graded Gravel (GP)	92.5	7	2	1.5	0.5	0	60	24	2.5	34	3
Case 24 - FL Miami Limestone	Well Graded Gravel (GW)	100	79.5	41.1	22	8	0.2	10.1	0.7	14.4	36.9	6.1
Case 25 - Manufactured Sand	Silty Gravel with Sand (GM)	88	53	41	31	21	13	28	0.07	400.0	35	4.8
Case 33 - Stahler, Walters	Well Graded Gravel (GW)	100	85	47	38	18	3	7	0.22	46.0	42	6.8
Case 37 - VDOT #3 Stone	Poorly Graded Gravel (GP)	100	0.1	0.1	0.1	0.1	0.1	48	30	1.6	50	4
Case 38 - VDOT #10 Screenings	Silty Sand (SM)	100	100	99.6	75.6	38.3	17.6	1.2	0.075	16.0	36	4.2
Case 39 - VDOT #1 Stone	Poorly Graded Gravel (GP)	100	1.7	0.7	0.5	0.35	0.1	65	40	1.6	50	2.5

Table 2 – Tested Backfill Properties

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C_u vs f* Relationship

The traditional design model used by RECo and AASHTO for ribbed steel strips is f* = 1.2+log C_u. This model was based on the data available in the 1980's and included backfill types that do not meet AASHTO MSE backfill requirements for durability, minimum shear strength (34 degrees), maximum fines content (15% < #200), maximum percentage passing the No. 40 sieve (60%) and degree of compaction (95% standard Proctor: AASHTO T-99). Since the time of the initial issue of the AASHTO design specifications, RECo and other researchers have performed tests on hundreds of samples in varying backfills that meet AASHTO specifications.

Based on the data available for AASHTO backfills, RECo recommends the following relationship for determining f* of AASHTO granular backfills:

$$f^*_o = 1.5 + \ln(C_u) < 4.5 \qquad \textit{(Formula 1)}$$

The above relationship envelopes the 95% confidence limit of backfills meeting AASHTO specifications.

Note on Open Graded Coarse Aggregates: it was recognized early on by RECo that coarse aggregates with a low C_u do not fit the 1.2 + log Cu model, and per RECo Tech Bulletin MSE-6 (1995) the recommended f value for coarse aggregates was 2.0 at the ground surface. Further testing has shown that the f* value for coarse aggregates far exceeds this value, therefore the current C_u relationship does not apply. See recommendations later in this report for Open Graded Coarse Aggregate backfills such as AASHTO #1 thru #7 aggregates.*

C_u Calculation Note: For the purposes of f calculations, the smallest Sieve Size used for the D10 calculation is #200 (0.075mm), so if a backfill has 15% passing the #200 sieve and 10% passing the 0.05mm, 0.075mm is used in the calculation.*

The following figure (Figure 9) illustrates the relationship with f* and C_u for the backfills referenced in this report. Only one backfill tested (KU Gravel #6) does not follow the trends and as such has been considered an outlier data point.

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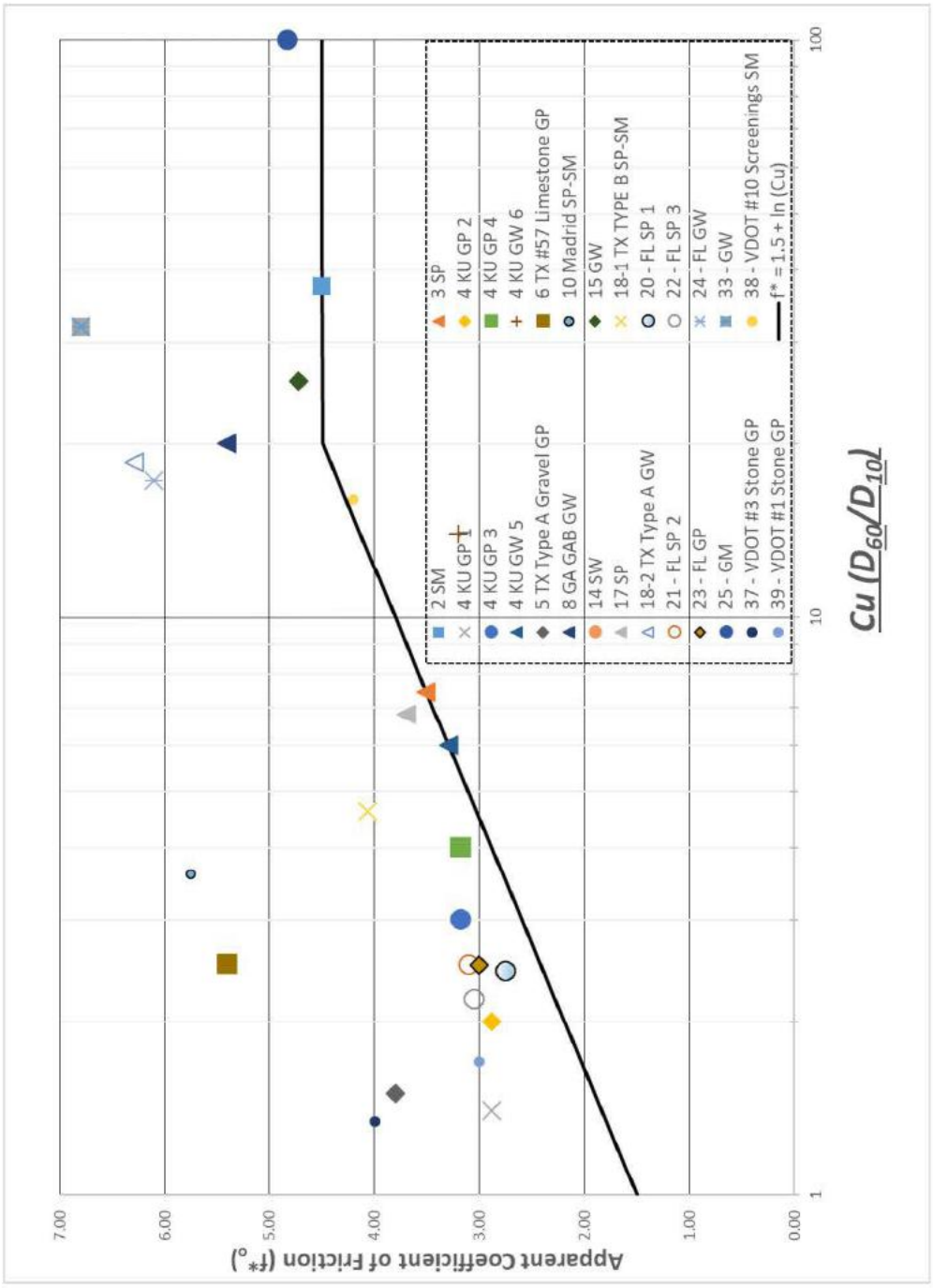


Figure 9

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Friction Factor (f*) to be used in the Design of Reinforced Earth Structures

The Reinforced Earth Company now has abundant data to allow us to use values greater than the AASHTO default values. The following is a summary of the current recommendations for design:

Unless direct pullout testing is performed, the selection of f^*_o will depend on the following backfill properties: Gradation, Shear Strength, Coefficient of Uniformity (C_u), and angularity (in the case of gravels). The Coefficient of Uniformity, C_u, is equal to the size of the sieve opening through which 60% of the material passes divided by the size of the sieve opening through which 10% of the material passes (D60/D10). As mentioned previously, the AASHTO default equation for $f^*_o = 1.2 + \log C_u$ may be used in design, but it is excessively conservative for all soils except uniform fine sands with a Cu<1.5. It is noted that the roundness or angularity of the backfill material also plays a significant role in the actual f^*_o value. For simplicity, five categories of f^*_o may be considered in a design based on soil type, shear strength and gradation as follows:

$f^*_o = 1.8$

This default value is to be used for projects constructed with backfills having a coefficient of uniformity $1.5 < C_u < 2$, i.e., materials such as *uniform rounded sands (SP) and materials such as pea gravel*, unless substantiated otherwise by pullout tests in representative backfill, and as noted below for angular course aggregates (AASHTO #1 thru #7 as discussed below).

$f^*_o = 2.5$

This default value is to be used for projects constructed with backfills meeting AASHTO shear strength and gradation requirements having a coefficient of uniformity $2 \leq Cu \leq 3$.

$f^*_o = 2.8$

This default value is to be used for projects constructed with backfills meeting AASHTO shear strength and gradation requirements having a coefficient of uniformity $3 \leq C_u \leq 4$, OR projects in Florida meeting FLDOT specifications for MSE fill and is based on extensive testing from multiple sources of Florida uniform sand backfills.

$f^*_o = 3.5$

This default value is to be used for projects constructed with well graded backfills meeting AASHTO gradation and shear strength requirements having a coefficient of uniformity $4 < C_u < 10$.

$f^*_o = 4.0$

This default value is to be used for projects constructed with well graded backfills meeting AASHTO gradation and shear strength requirements and having a coefficient of uniformity $C_u > 10$. This value should also be used for projects constructed with angular and sub angular gravels meeting AASHTO No. 3, 4, 5, 57, 6 or 7 stone or similar. Crushed gravel is widely available across the United States, and though it typically has a low coefficient of uniformity, pullout testing has confirmed that its angularity and durability gives high pullout resistance.

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The following "Equivalent Soils" are considered

Backfill Description (Based on ASTM – D2487 - United Soil Classification System)	Group Symbol(s)	% Passing 4"	% Passing ¾"	% Passing No. 4	% Passing No. 40	% Passing No. 200	C_u	Typical Shear Strength Φ (deg)	Default f* at ground surface
		Cobbles to Course Gravel	Medium to Fine Gravel	Course to Medium Sand	Fine Sand	Silt - Clay			
Poorly Graded Fine Sands (note 1)	SP, SP-SM SP-SC	100%	100	90-100	>50	0-15	1.5 to 2	30 to 34	1.8
Poorly Graded Fine Sands (note 1)	SP, SP-SM SP-SC	100%	100	90-100	>50	0-15	2 to 3	30 to 34	2.5 – 2.8 [1]
Silty Sands, Silty Sand with Gravel	SM, SW-SM SW-SC	100%				5-15	3 to 4	34 to 38	2.8
Well Graded Sands	SW	100%		>50%		0-5	4 to 10	34 to 38	3.5
Well Graded Sandy Gravels	GW-GM	100%		< 50%		5 to 12%	>10	36 to 45	4
Well Graded Fine Gravels (3/4" to No. 4)	GW	100%	0 to 90%		0 to 10%	0-5	4 to 10	38 to 45	3.5
Poorly Graded Fine Gravels (3/4" to No. 4)	GP	100%	0 to 90%		0 to 10%	0-5	2 to 4	38 to 45	3.0
Well Graded Course Gravels (4" to 3/4") - (AASHTO No. 3, 4, 5, 57, 6,67, 7)	GW	100%	0 to 15%		0 to 5%	0-5	2 to 6	45 +	4
Poorly Graded Course Gravels (4" to 3/4") - (AASHTO No. 1 & No. 2)	GP	100%	0 to 15%		0 to 5%	0-5	1.5 to 2	45 +	3.0

Notes:

1. Some Poorly Graded (i.e. uniform) Fine Sands may not meet AASHTO minimum shear strength of 34 degrees, however are allowed by FLDOT and WIDOT with a minimum shear strength of 30 degrees.

2. The gradation ranges, associated Cu and f* values shown above correlate to backfills tested by RECo for each classification

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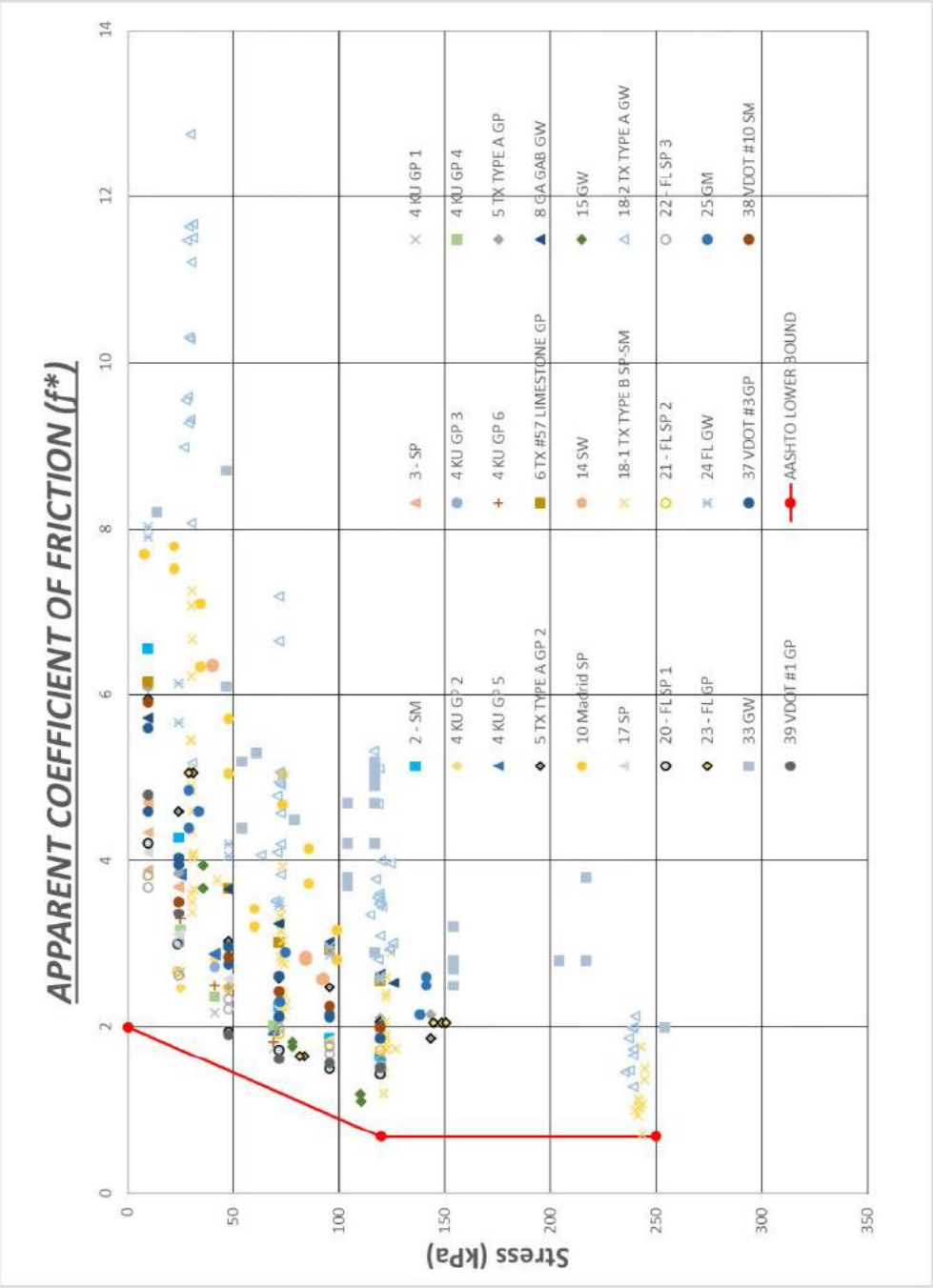
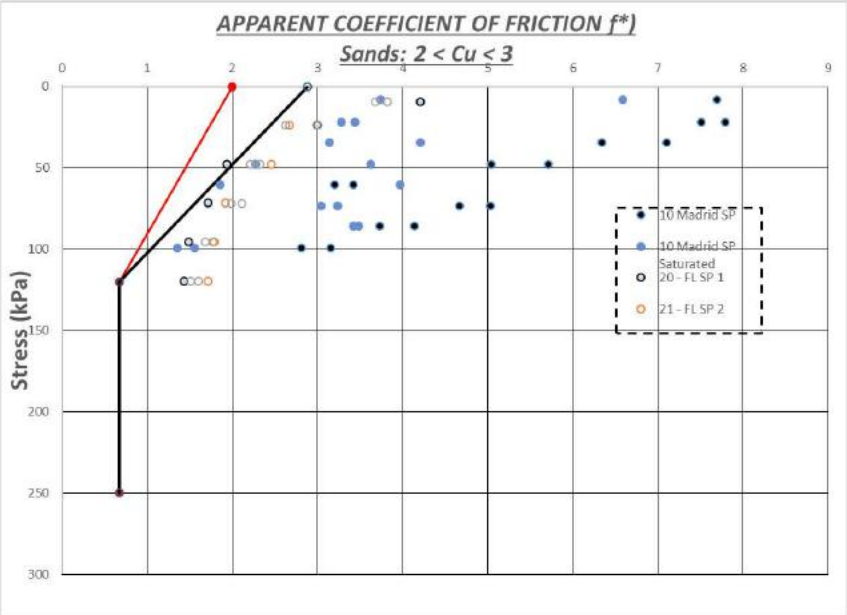
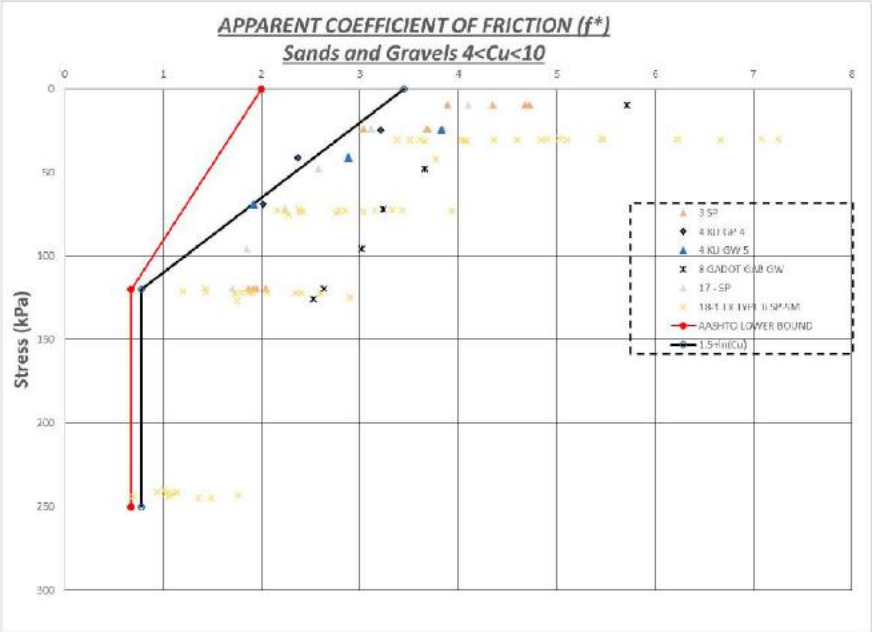


Figure 10

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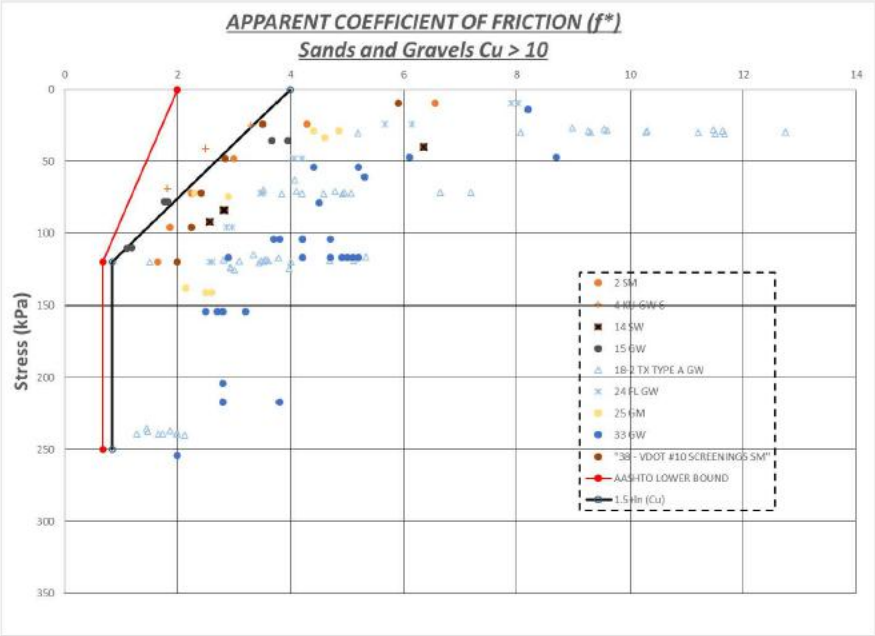
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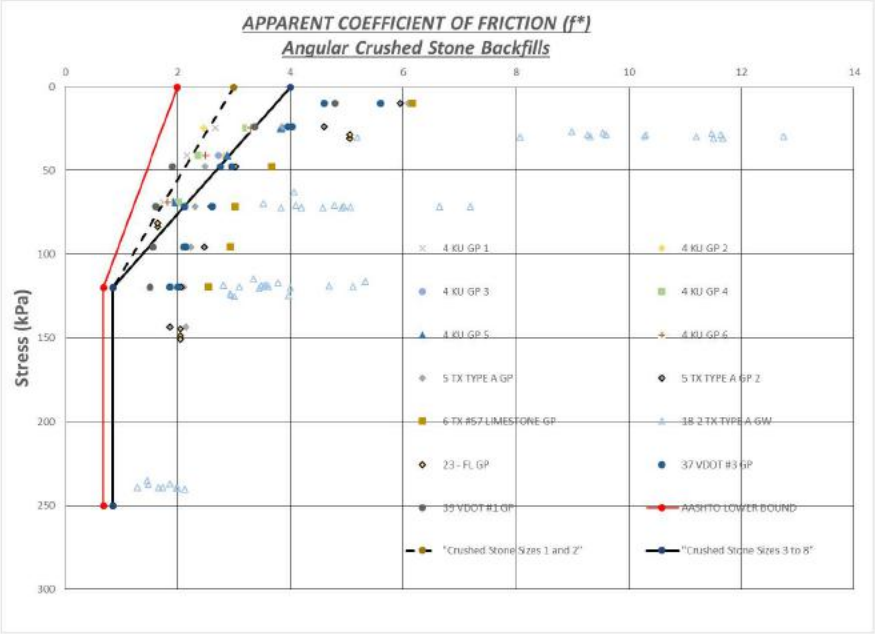




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Closing

The preceding information demonstrates that the AASHTO default values for f* are excessively conservative for RECO (HA) ribbed strips. Test data from backfills meeting AASHTO requirements reflects that conservatism and support the need for higher f* values (for depths of 0 to 20 ft) to be considered for design.

Test data is available upon request.

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Runser, Fox and Bourdeau, "Field Performance of a 17m High Reinforced Soil Retaining Wall", Geosynthetics International, 2001, Vol. 8, No. 5.)

Test References #18-1 & 18 -2: Final Research Report, MSE Pullout Testing for RECo HA Ladder and HA Strip Reinforcements. Center for Multidisciplinary Research in Transportation. Texas Tech University. 12-21-2011

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High Adherence (HA) = Ribbed Steel Strips

The pullout resistance of discrete steel strips is a function of the backfill type, overburden pressure, length, width and geometry of the strips. The data presented herein is specifically for Terre Armee / Reinforced Earth High Adherence (HA) Strips.

The dimensions of the standard 50mm wide HA Strips tested are shown below:

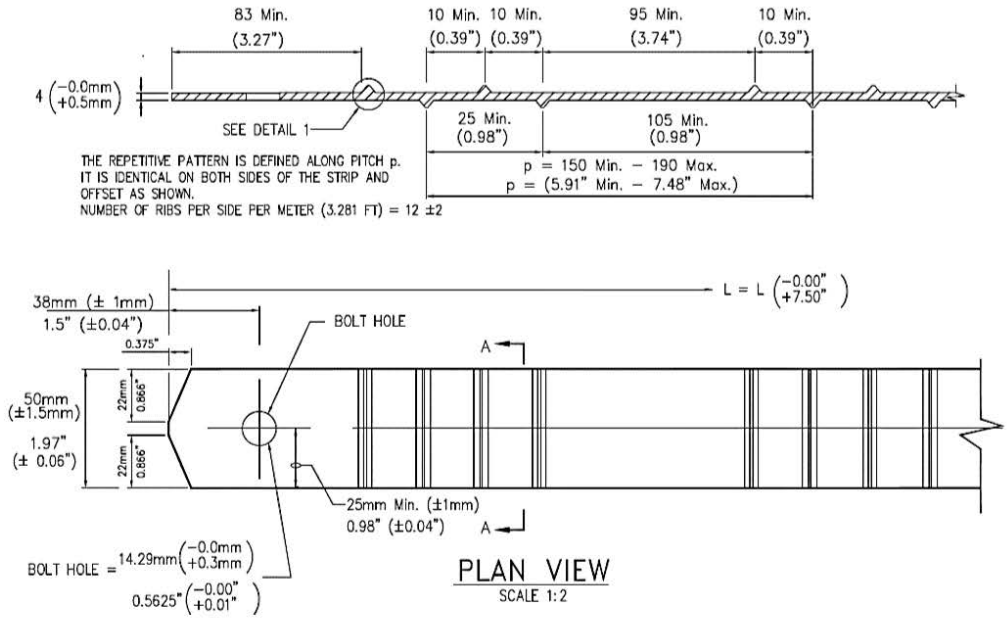


Figure A1 – High Adherence Strip Geometry

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Appendix B: Skewed Reinforcing Strips

Design considerations for Skewed / Splayed Ribbed Reinforcing Strips

The internal stability design of a Mechanically Stabilized Earth (MSE) structure is composed of two design checks: (1) Pullout Resistance and (2) Reinforcement Strength. This article will focus on the design considerations for Pullout Resistance and Structural Strength of Skewed/Splayed Ribbed H.A. (High Adherance) reinforcements and the effects at panel connections (i.e. tie strips).

As shown in Figure B1, Reinforcing Strips are structurally bolted to facing panels by means of a tie strip connection. Typically, the reinforcing strip is placed perpendicular to the facing panel. However, there are times when an obstruction in the backfill results in the need to skew reinforcements to clear the obstruction (Figure B2).



Figure B1

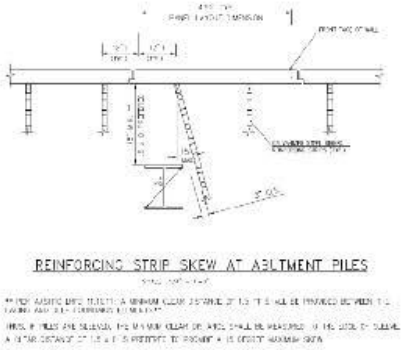


Figure B2

Historically, the skew angle of the reinforcing strips has been limited to 15 degrees. This value is based on an assumed reduction in pullout capacity based on a cos (α) x R (resistance) where α = skew angle, and an increase in reinforcement load based on 1 / cos (α). However, this assumption has been proven to be excessively conservative by a series of full-scale pullout tests with ribbed reinforcing strips placed on skew angles up to 30 degrees. When discrete ribbed reinforcing strips are placed on a skew, there is an added passive resistance provided on the edge of the strip which increases the total pullout resistance.

As shown in figure B3 below, the effect of skewing strips does not decrease the apparent coefficient of friction (f*) below the design value (which for this backfill f*o would be 3.5). As such, there is no need to reduce the pullout capacity by the cos(α) as noted above.

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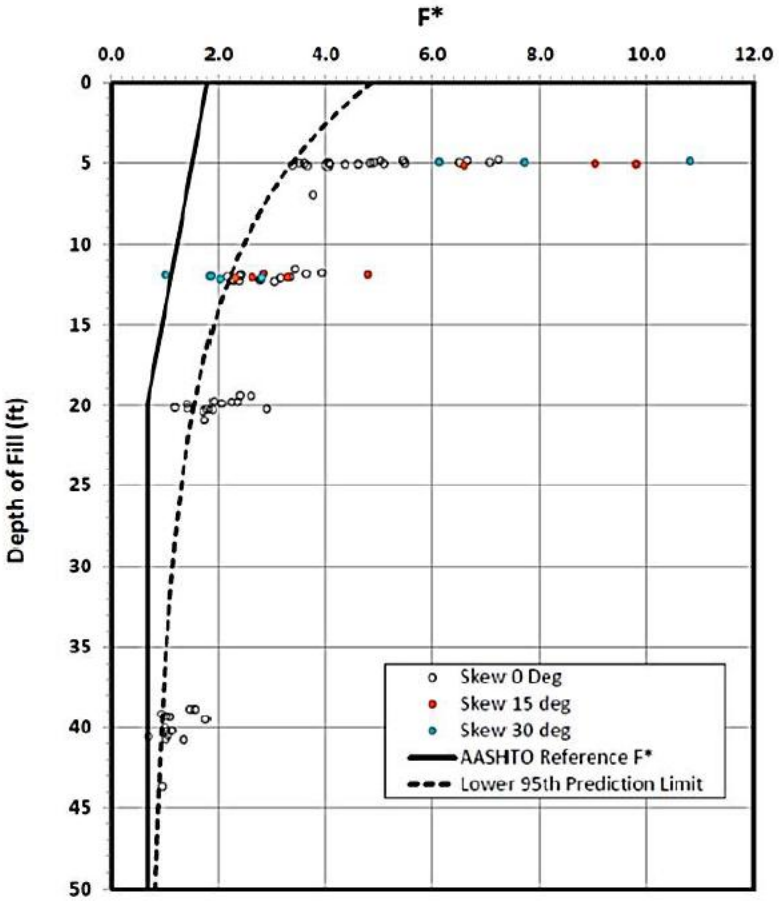


Figure 6.5 F* versus Depth of Fill Chart for Ribbed Strip Reinforcement Embedded in Type B Backfill (Relative Compaction = 95%): Effect of Skewing

Figure B-3 Effect of Skewing Strips on f*

(Figure B3 Reference: "Jayawickrama, Surles, Wood, Lawson. Pullout Resistance Factors Mechanically Stabilized Reinforcements in Backfills Typically used in Texas, TXDOT Report 0-6493-1, August 2013.

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Appendix C: Influence of Water on f*

Discussion of the Influence of Water on the f* Value Used in Design

Hundreds of Reinforced Earth structures have become fully saturated in service or remain permanently inundated throughout the service life of the structure. There is not a significant reduction in the friction developed between the reinforcements and the saturated soil for backfills meeting the strict gradation requirements for Mechanically Stabilized Earth (MSE) backfill.

The excellent performance of saturated steel reinforced MSE structures is due to the following:

a. The soils specified for construction are granular and are of high shear strength, generally $\phi > 36^{\circ}$, yet 34° shear strength is typically used in design.

b. The soils are compacted during construction to relatively high densities while at or slightly dry of optimum moisture content.

c. The soils are subjected to low strain levels in service and are normally stressed in plane strain.

d. There is no mechanism for excess pore pressures to develop because of the use of reasonably free draining backfills.

e. Galvanization and metal loss rates provided under AASHTO are conservative and saturated conditions do not result in actual metal losses exceeding these values where electrochemical conditions are met according to the code.

The following additional AASHTO requirements assure that the frictional characteristics of the soil, and therefore, the apparent coefficient of friction between soil and reinforcements, will remain essentially unaffected by saturation.

a. 0 – 15% passing the No. 200 sieve in accordance with AASHTO T27 or ASTM D422 (restricts the percentage of silt and clay particles in the soil thus ensuring a material that is relatively well draining).

b. A minimum shear strength of 34° determined by direct shear test per AASHTO T236 or ASTM D3080 conducted on a sample of material compacted to 95% of Standard Proctor density at optimum moisture content per AASHTO T99 or ASTM D698.

c. Plasticity index (PI) less than or equal to 6 per AASHTO T90 or ASTM D4318 (restricts the percentage of plastic clay particles).

Soils meeting these restrictive requirements, in addition to the required particle durability testing, will not be susceptible to a significant reduction in shear strength when becoming fully saturated. Of course, the same holds true for the apparent coefficient of friction, f*, between the strip and soil, since this parameter is directly related to the shear strength of the soil.

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In 1982, Terre Armee Internationale researched the effects of saturated conditions on the shear strength of soils and on the apparent coefficient of friction, f*. Both soils marginally exceed the AASHTO gradation or PI requirements (#1 exceeds the #40 sieve maximum of 60%, and #2 has a PI = 14, which is significantly greater than the maximum of 6 permitted by AASHTO). Nevertheless, the trends demonstrated by the test show the effects of saturation in fine grained soils, and reinforce the reasoning behind the AASHTO gradation and PI limitations.

The results of these tests are presented herein as follows:

a. Table C1 presents index properties and direct shear results for each soil at both optimum moisture content and in a saturated, undrained condition and the determination of the coefficient of uniformity, Cu, and the corresponding fo* value to be used in design.

b. Figures C1 & C2 present a plot of the apparent coefficient of friction determined by pullout tests at various heights of overburden for both optimum moisture and saturated conditions. In addition, the f* envelope that would be used for design, based on the material's gradation, is also shown.

When a soil sample is sheared under poorly drained conditions, the resulting shear strength is generally less than that obtained under well drained conditions. However, for soils meeting AASHTO MSE backfill specifications, the decrease in shear strength is not significant.

In addition, in the design of structures that become submerged, the forces to be resisted generally include the effects of a minimum drawdown (differential head). In the reinforced strip resistance calculation, reduced overburden on the reinforcing strips due to soil buoyancy is also included. Therefore, the influence of water in a saturated structure is already addressed in the design. There is no adjustment necessary of the fo* value used in the design.

Note, while neither soil meets the AASHTO specifications, the findings of the testing are of importance as they show that a high PI combined with saturated conditions results in a significant reduction of pullout resistance.

	Soil 1 Madrid Fine Sand (SP-SM)	Soil 2: Madrid Clayey Sand (SC)
Percent Passing ½"	100 %	100 %
Percent Passing #4 Sieve	100 %	96 %
Percent Passing #40 Sieve	70 %	60 %
Percent Passing #200 Sieve	6 %	15 %
Percent Finer than 15 µm (i.e. clay)	4 %	12 %
Coefficient of Uniformity (Cu = D60/D10)	3.6	13.3
Plasticity Index (PI)	Non-Plastic	14 *
Direct Shear Test on Soil at Optimum Moisture Content (Φ)	37°	43°
Triaxial Shear Test on Soil Saturated and Consolidated under 4000 psf	34°	34°
Maximum Apparent Coefficient of Friction at Ground Surface for Design (f*o)	2.8	1.8

Table C1

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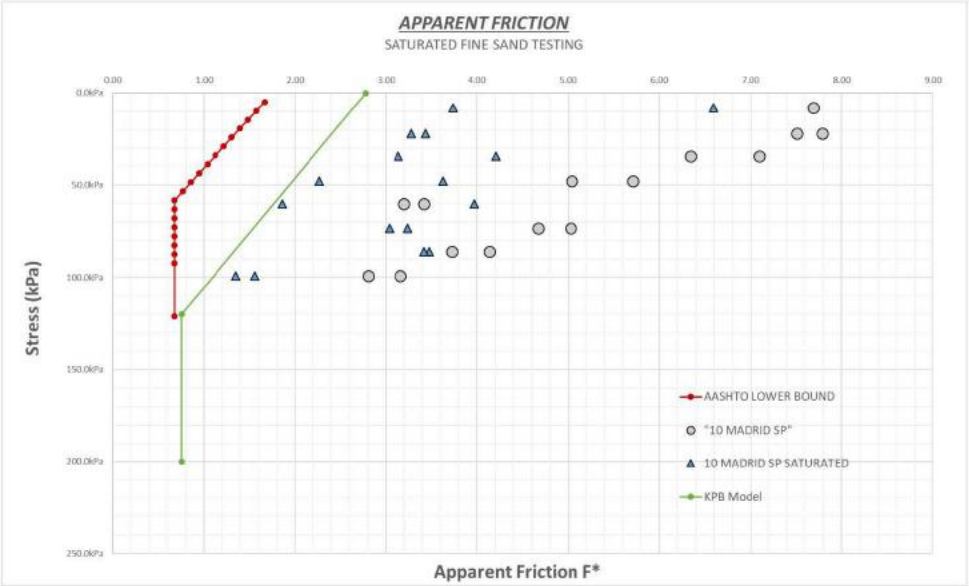


Figure C1 – FINE SAND WITH 6% PASSING #200 SIEVE

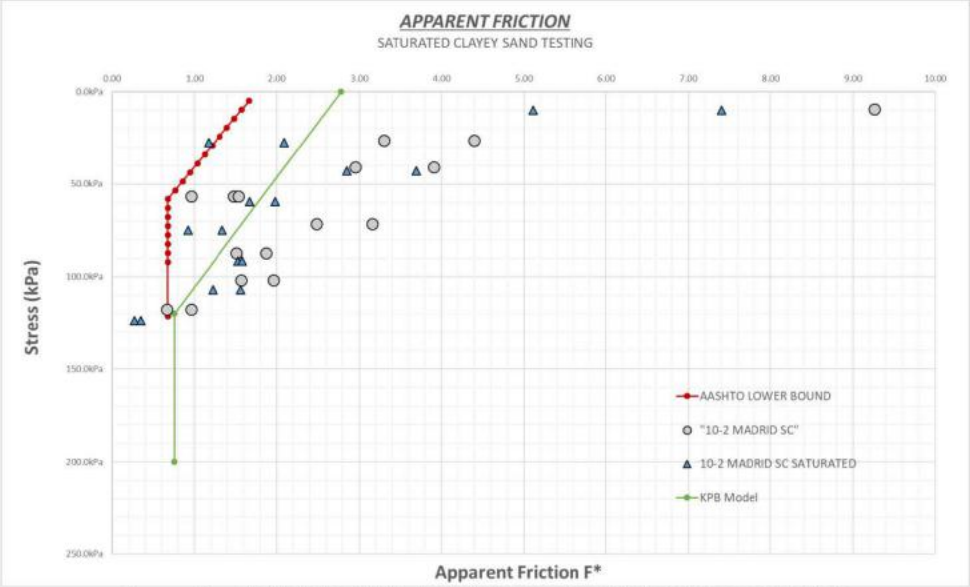


Figure C2 – FINE SAND WITH CLAY (SC), PI = 14, SATURATED CONDITIONS

(Note, this backfill does not meet AASHTO requirements, but test results are shown herein to demonstrate the loss of frictional resistance when clayey soils are saturated)

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Appendix D – AASHTO Course Aggregate Sizes

TABLE 703.01-1 SIZE OF COARSE AGGREGATE (AASHTO M 43)

Size No.	Nominal size square openings [1] inch	mm	Amounts finer than each laboratory sieve (square openings), percent by weight: 4 in. 100 mm	3 1/2 in. 90 mm	3 in. 75 mm	2 1/2 in. 63 mm	2 in. 50 mm	1 1/2 in. 37.5 mm	1 in. 25 mm	3/4 in. 19 mm	1/2 in. 12.5 mm	3/8 in. 9.5 mm	No. 4 4.75 mm	No. 8 2.36 mm	No. 16 1.18 mm	No. 50 300 µm	No. 100 150 µm
1	3 1/2 to 1 1/2	90 to 37.5	100	90 to 100		25 to 60		0 to 15		0 to 5							
2	2 1/2 to 1 1/2	63 to 37.5			100	90 to 100	35 to 70	0 to 15		0 to 5							
24	2 1/2 to 3/4	63 to 19			100	90 to 100		25 to 60		0 to 10	0 to 5						
3	2 to 1	50 to 25				100	90 to 100	35 to 70	0 to 15		0 to 5						
357	2 to No. 4	50 to 4.75				100	95 to 100		35 to 70		10 to 30		0 to 5				
4	1 1/2 to 3/4	37.5 to 19					100	90 to 100	20 to 55	0 to 15		0 to 5					
467	1 1/2 to No. 4	37.5 to 4.75					100	95 to 100		35 to 70		10 to 30	0 to 5				
5	1 to 1/2	25 to 12.5						100	90 to 100	20 to 55	0 to 10	0 to 5					
56	1 to 3/8	25 to 9.5						100	90 to 100	40 to 75	15 to 35	0 to 15	0 to 5				
57	1 to No. 4	25 to 4.75						100	95 to 100		25 to 60		0 to 10	0 to 5			
6	3/4 to 3/8	19 to 9.5							100	90 to 100	20 to 55	0 to 15	0 to 5				
67	3/4 to No. 4	19 to 4.75							100	90 to 100		20 to 55	0 to 10	0 to 5			
68	3/4 to No. 8	19 to 2.36							100	90 to 100		30 to 65	5 to 25	0 to 10	0 to 5		
7	1/2 to No. 4	12.5 to 4.75								100	90 to 100	40 to 70	0 to 15	0 to 5			
78	1/2 to No. 8	12.5 to 2.36								100	90 to 100	40 to 75	5 to 25	0 to 10	0 to 5		
8	3/8 to No. 8	9.5 to 2.36									100	85 to 100	10 to 30	0 to 10	0 to 5		
89	3/8 to No. 16	9.5 to 1.18									100	90 to 100	20 to 55	5 to 30	0 to 10	0 to 5	
9	No. 4 to 16	4.75 to 1.18										100	85 to 100	10 to 40	0 to 10	0 to 5	
10	No. 4 to 0 [2]	4.75 to 0 [2]										100	85 to 100				10 to 30

[1] Numbered sieves are those of the United States Standard Sieve Series.
[2] Screenings.
Where standard size of coarse aggregate designated by two or three digit numbers are specified, obtain the specified gradation by combining the appropriate single digit standard size aggregates by a suitable proportioning device which has a separate compartment for each coarse aggregate combined. Perform the blending as directed by the Laboratory.

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FINAL RESEARCH REPORT

MSE PULLOUT TESTING FOR RECO HA LADDER AND HA STRIP REINFORCEMENTS

For

The Reinforced Earth Company

Vienna, VA

Prepared by

Center for Multidisciplinary Research in Transportation

Texas Tech University

Lubbock, TX

December 21, 2011

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16. Abstract

This report presents findings of a research study to investigate the pullout resistance behavior of high adherence (HA) strip and HA-ladder reinforcements produced by the Reinforced Earth Company (RECo) for mechanically stabilized earth (MSE) structures. The test program included a total of 16 pullout tests on HA-strips and 52 pullout tests on HA-ladders. All MSE pullout tests for this study were performed in sandy backfill (coefficient of uniformity of 4.7) using a large-scale pullout resistance test system. Tests were conducted at overburden pressures corresponding to an equivalent depth of fill ranging from 3ft to 40ft. Selected HA-strip and HA-ladder reinforcements were instrumented with strain gages to obtain additional information about their pullout behavior. The most significant findings of this study are as follows: (a) Pullout resistance factors (F*) obtained for RECo HA-strips embedded in well-compacted sandy backfill were higher than those predicted by the AASHTO reference equation for default F* by a significant margin; (b) Pullout resistance factors (F*) calculated for HA ladders also showed a clear margin in comparison to AASHTO reference F* values for both ribbed strips and welded steel grids; (c) The data support the view that HA-ladders can be treated in the MSE design process as AASHTO ribbed strip elements with a width equal to the transverse bar length (4 inches); (d) Results obtained from strain gaged HA-strip and HA-ladder reinforcements indicate, in general, that a linear load distribution exists along the full length of the reinforcement. This supports the view that the HA-strip and HA-ladder reinforcements allow mobilization of pullout resistance uniformly along their entire length, a finding which is consistent with an inherent assumption made in the *coherent gravity method* of MSE wall design.

17. Key Words:	18. Distribution Statement:
pullout testing, pullout resistance factor, inextensible, ribbed strips, high-adherence strips, high-adherence ladders, mechanically stabilized earth, retaining walls, soil structure interaction	

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1 INTRODUCTION

1.1 The Research Problem

This report documents the findings from a research study that investigated the pullout resistance behavior of two types of mechanically stabilized earth (MSE) reinforcements produced by the Reinforced Earth Company (RECo); namely, high adherence (HA) strip reinforcements and high adherence (HA) ladder reinforcements. The research study consisted of two separate series of pullout tests that were conducted using the large scale MSE pullout resistance test system designed and constructed by the researchers at Texas Tech University. In addition to the pullout tests, selected HA-strip and HA-ladder reinforcements were instrumented with electrical resistance strain gages so that additional information on their pullout behavior could be obtained. Tests were conducted at overburden pressures corresponding to an equivalent depth of fill ranging from 3ft to 40ft. All pullout load tests were conducted using a sandy backfill that marginally satisfied particular gradation requirements specified by AASHTO, FHWA, and TxDOT (AASHTO 2010a, FHWA November 2009, TxDOT 2004).

1.2 Authorization and Scope

The research work described in this report was conducted under Texas Tech University Research Agreement No. 14506513 approved on June 8, 2011, and sponsored by The Reinforced Earth Company. This project complements a research study sponsored by the Texas Department of Transportation (TxDOT), *Project No.0-6493: Pullout Resistance of Mechanically Stabilized Earth Reinforcements in Backfills Typically Used in Texas*. The documentation provided in this report is limited to the work completed under the research contract with RECo. However, selected data obtained from the TxDOT-sponsored research study are summarized in this report and used for comparative purposes.

1.3 Research Objectives

The objectives of this research study were as follows:

a) Determine the pullout resistance factors (F* values) for HA-strip reinforcements and compare them against the ribbed steel strip guidelines provided in AASHTO for F* values to be used in the design of MSE walls

b) Determine the pullout resistance factors (F* values) for HA-ladder reinforcements and compare them against the ribbed steel strip guidelines provided in AASHTO for F* values to be used in the design of MSE walls

c) Instrument the reinforcements with strain gages in order to determine how the pullout resistance is distributed along the length of the reinforcement

d) Present the pullout resistance factors (F* values) for HA-strip type reinforcement within the context of the larger pullout data set collected from TxDOT project 0-6493.

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1.4 Report Organization

Chapter 1 introduces the research problem, documents the scope and authorization for this study, and identifies the research objectives. Chapter 2 presents the research method including detailed descriptions of the equipment and materials used in testing. Chapter 2 also describes the test procedures used as well as test matrix and data reduction and analysis methods. Chapter 3 of this report presents the results from MSE pullout tests for gaged and ungaged HA-strip and HA-ladder reinforcements. These findings summarize the detailed pullout test reports and strain gage data collected as part of this study and which are presented in Appendices A through D of this report. A brief discussion of the research findings is presented alongside collected data. The final chapter, Chapter 4, provides the summary and conclusions from the research study.

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2 METHOD

2.1 Overview

This chapter summarizes the research methods used for this study. Section 2.2 provides a description of the test equipment used. This is followed by a description of the materials used in this test program, both soil backfill and the MSE reinforcing, in Section 2.3. Section 2.4 describes the pullout test procedure including test setup preparation, application of overburden pressure, and application of pullout load. Section 2.5 presents the test program conducted for this study, that is, the test matrix of actual tests performed. The steps undertaken for pullout testing of gaged reinforcements is discussed separately in Section 2.6. The last section in this chapter, Section 2.7, describes the data analysis methods.

2.2 Large-Scale MSE Pullout Resistance Test System

MSE pullout tests for this study were performed using the large-scale pullout resistance test system at Texas Tech University, the centerpiece of which is the MSE Test Box shown in Figure 2.1. The dimensions of the MSE Test Box are 12ft by 12ft in plan area and 4ft in depth. The MSE Test Box can accommodate up to three layers of earth reinforcement in a single filling of the test box with reinforcement layers placed at heights of 1.0ft, 2.0ft and 3.0ft from the base. Reinforcements placed in the MSE Test Box at all three levels can be pulled to the south through slots cut in the south wall (primary test bulkhead) or to the east (secondary test bulkhead), allowing for optimum test space utilization.



Figure 2.1 Large Scale MSE Pullout Resistance Test System: Civil Engineering Department, Texas Tech University.

In this test system, the soil overburden pressures applied to the embedded earth reinforcement are simulated using a reaction frame assembly which consists of nine 4ft by 4ft pressure plates that are hydraulically jacked against three wide flange cross beams. This reaction

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frame assembly is designed to accommodate simulation of overburden pressures up to 40ft of fill. Vertical overburden pressure is monitored throughout the test using four earth pressure cells that are grouted into the bottom slab of the MSE Test Box. A second, independent measurement of the applied overburden is made using a pressure transducer that monitors the fluid pressure in the hydraulic jacks.

Pullout testing of each embedded earth reinforcement is accomplished, one at a time, by attaching a pullout load assembly to the reinforcement and then applying the pullout force using a 60-ton hollow core hydraulic jack. HA-strip and HA-ladder reinforcements are connected to the pull rod using a simple pin mechanism. Two independent systems are used to measure and record the pullout force. The first is an annular load cell mounted on the pull rod. The second is a pressure transducer that measures hydraulic pressure in the hollow-core jack. Figure 2.2 shows the load cell and hollow core jack.

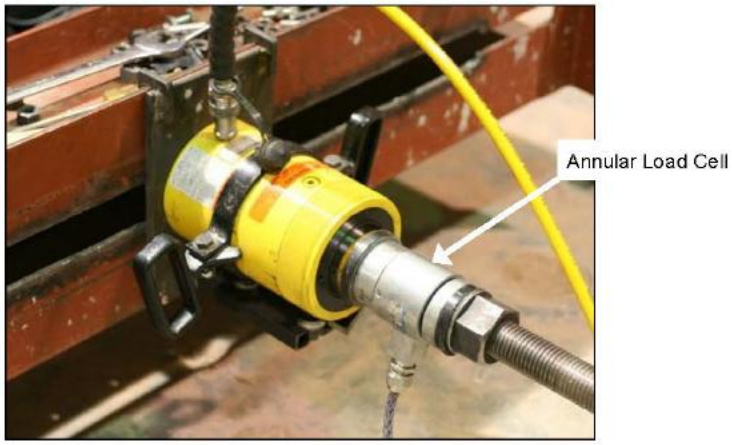


Figure 2.2 Hollow-core jack and annular load cell.

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Similarly, the displacement of the reinforcement during pullout testing is determined using two independent measurements. The first is an optical measurement of the displacement using a witness marker against a graduated scale. The second is a digital measurement of displacement that is made using an electronic displacement gage (Figure 2.3).

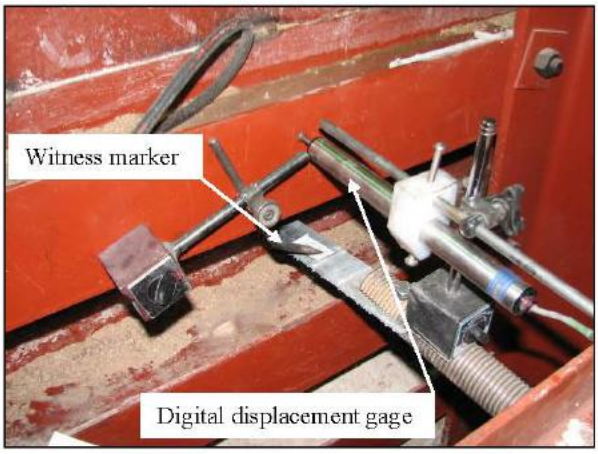


Figure 2.3 Witness marker and digital displacement gage used for displacement measurements.

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Figure 2.4 shows a single layer of HA-strip reinforcements laid out in the MSE Test Box for testing. Two of the HA-strips are 8ft long. Three of the HA-strips are 12ft long. All are instrumented with strain gages. Figure 2.5 show a similar arrangement of HA-ladder reinforcements. This particular layer consists of a total of eight HA-ladder reinforcements; four 4ft long ladders and four 8ft long ladders. As seen in the figure, three of these HA-ladders have been instrumented with strain gages.



Figure 2.4 Instrumented HA-strips laid out for pullout testing.

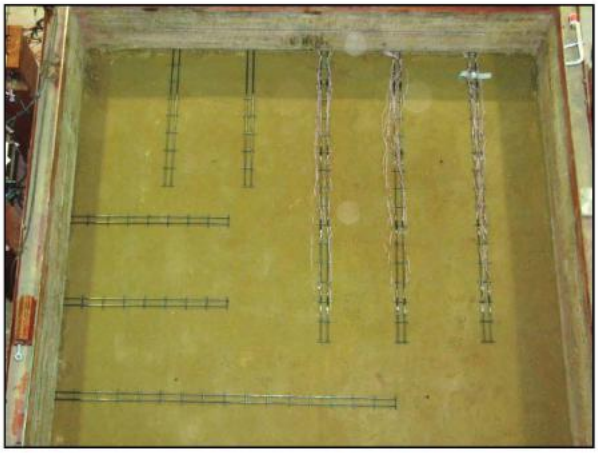


Figure 2.5 Instrumented and uninstrumented HA-ladder reinforcement laid out for pullout testing.

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2.3 Materials

2.3.1 Granular Backfill

The backfill material used in this test program was a fine to medium sand that marginally satisfies the requirements for MSE select backfill as specified in AASHTO LRFD Bridge Construction Specifications, FHWA Publication No. FHWA-10-024 and TxDOT Specification Item No. 423/Type B select backfill (AASHTO 2010a, FHWA November 2009, TxDOT 2004). Table 2.1 summarizes these gradation requirements, and Figure 2.6 presents the particle size distribution curve for the sandy backfill material used in this study. The material was determined to be non-plastic (ASTM D4318) and classifies as SP-SM according to the Unified Soil Classification System (ASTM D2487). The coefficient of uniformity, C_u of this sandy backfill is 4.7, which is close to the default C_u value of 4.0 recommended by AASHTO when the actual C_u for the wall backfill is not known at the time of the wall design (AASHTO 2010b). The sandy backfill used for this study has a maximum dry unit weight of 124.5pcf and an optimum water content of 7.8% as established through testing conducted in accordance with TxDOT standard test procedure, Tex-114-E, which uses the same compactive energy as the Standard Proctor compaction method (ASTM D698).

Table 2.1 Gradation requirements for select granular backfill.

U.S. Sieve Size	Percent Passing from FHWA-10-024	Percent Retained TxDOT Specification Item No. 423/Type B
4.0in (102 mm)	100	--
3.0in. (76mm)	--	0
No. 40 (0.425mm)	0-60	40-100
No. 200 (0.075mm)	0-15	85-100

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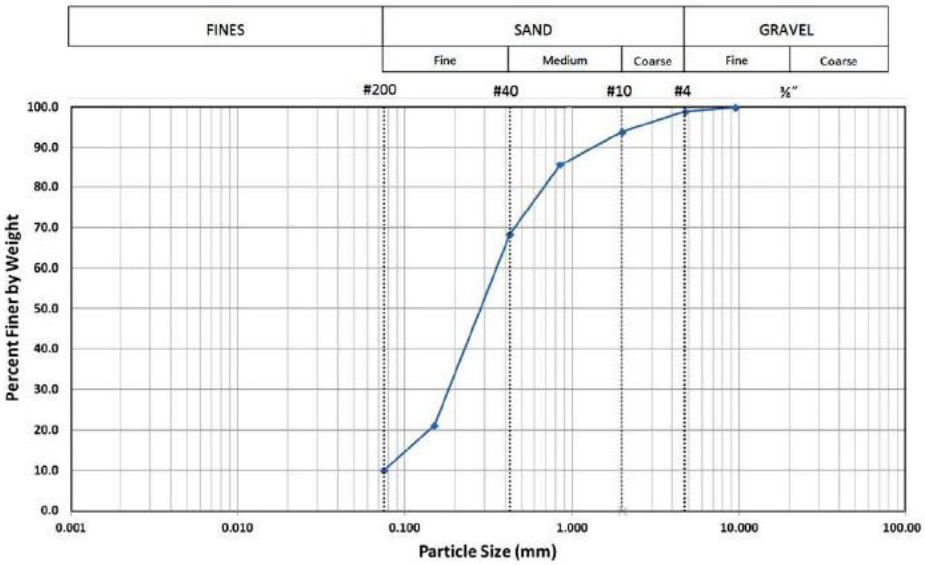


Figure 2.6 Particle size distribution curve for MSE wall backfill material used in pullout testing.

2.3.2 MSE Reinforcement

The test program for this research study evaluated HA-strip and HA-ladder reinforcements, both of which are produced by The Reinforced Earth Company. Figure 2.7 and Figure 2.8 provide detailed specifications used in the production of these earth reinforcing materials (The Reinforced Earth Company 2005).

HA-strip reinforcements have been used in the construction of MSE walls for more than 30 years (The Reinforced Earth Company n.d.). As a result, a significant body of data exists on the pullout resistance of this type of reinforcement. More importantly, AASHTO and FHWA-NHI provide guidance on default pullout resistance factors, F* to be used with HA-strip reinforcements (AASHTO 2010b, FHWA November 2009). In contrast, HA-ladder reinforcements represent a relatively new product for which limited data exist.

The RECo HA-strip elements tested in this study (16, total) were typical construction-ready soil reinforcements except that they were black (non-galvanized) to facilitate installation of strain gages. The RECo HA-ladder elements tested for this study were also typical construction-ready product. However, one set of HA-ladders (28, total) was galvanized, and one set of HA-ladders (24, total) was black (non-galvanized) to facilitate application of strain gages. The black HA-ladders were fitted with a thickened connection plate to facilitate pullout failure of the reinforcement as opposed to premature breakage at the bolt hole during testing.

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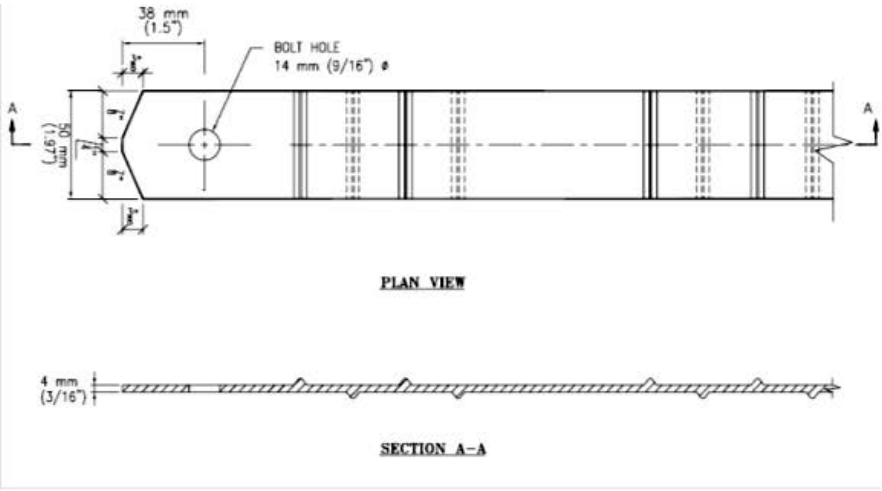


Figure 2.7 Schematic of HA-strip type reinforcement as tested. (The Reinforced Earth Company 2005)

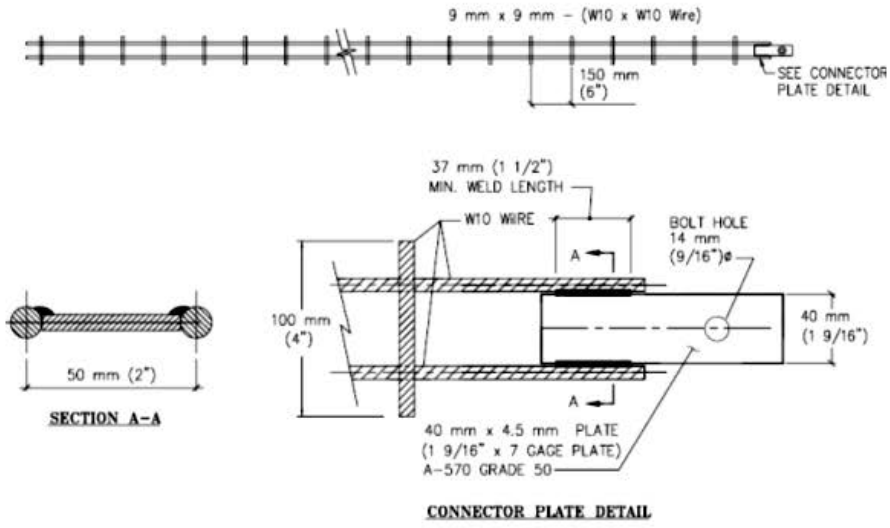


Figure 2.8 Schematic of HA-ladder type reinforcement. (The Reinforced Earth Company 2005)

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2.4 Pullout Test Procedure

This section describes the procedure used for pullout testing of MSE reinforcement. The test procedure may be divided into three separate stages: (a) preparation of the test setup, (b) application of the overburden pressure, and (c) application of the pullout load.

2.4.1 Test Setup Preparation

Test setup preparation involves filling the test box with the backfill and compacting the backfill material to achieve desired density while embedding the MSE reinforcements at appropriate depths within the compacted backfill. The MSE Test Box was designed with fixed permanent walls on three sides (east, west and south sides) and a removable wall on the north side to allow access for filling the box. During preparation of a test setup, the north wall is raised in 1-ft stages as backfill is placed inside the box. Backfill material is placed in the MSE Test Box in lifts of loose fill with approximate thickness of 4in. Each lift of material is then compacted with seven passes of a vibratory compactor to achieve a compacted thickness of about 3in. The compacted unit weight is measured using a nuclear density gage (ASTM D6938) to ensure a minimum relative compaction of 95% is achieved during compaction.

MSE reinforcements are embedded within the backfill at three levels located at heights of 1ft, 2ft and 3ft from the base slab. When placing reinforcements within the fill, the leading end of the reinforcement protrudes out of the MSE Test Box through a slot opening in either the south or the east wall. This leading end of the reinforcement is what is attached to the pullout load assembly for testing. Once the compacted fill reaches the 3ft 9in. height inside the MSE Test Box, one final lift of backfill material is added. This soil is placed and carefully graded to achieve a level surface, but this lift is not compacted because its primary purpose is to provide proper seating for the pressure plates which will be placed on top of this layer.

2.4.2 Application of the Vertical Overburden Pressure

The large-scale MSE pullout resistance test system is designed such that nine 4ft x 4ft stiffened steel plates are placed on top of the backfill covering the entire surface area of the MSE Test Box. Then, three wide flange reaction beams are positioned over the pressure plates and the beams bolted to support columns on either side of the box. Each reaction beam spans over three pressure plates. Hydraulic jacks are placed between the pressure plates and the reaction beams. The hydraulic jacks are then pressurized causing the jacks to apply downward pressure on the soil by pushing against the reaction beams. The hydraulic jacks are connected to a main hydraulic pump via a manifold so that all nine jacks are pressurized equally and simultaneously.

It should be noted that the target overburden pressure used to test each MSE reinforcement in a given test setup typically varies from reinforcement to reinforcement. Therefore, it is necessary to adjust the jack pressure before each pullout test to achieve the correct overburden pressure for that specific test. An appropriate allowance to account for the geostatic stress due to weight of the overlying backfill soil is made when calculating the jack-applied pressure.

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2.4.3 Application of the Pullout Load

The pullout load on the embedded reinforcements is applied by a hollow core jack that applies tension force to the MSE reinforcement by pushing against a stationary bearing surface, or bulkhead, attached to the MSE Test Box. The pull force is transmitted to the MSE reinforcement through a high tensile strength extension rod. Reinforcements are connected to the extension rod by a simple pin-joint to facilitate uniformly-applied, pure axial load without bending.

Once the extension rod is attached, an optical displacement marker (also called witness marker) is mounted on the wall of the MSE Test Box to provide visual measurement of the pullout displacement. The optical displacement marker consists of a pointer with a magnetic base. The location on the reinforcement to which the witness marker points at this juncture is referred to as "test zero". Test zero is the reference from which both optical and digital displacements during pullout testing are measured. Once "test zero" is established, a reference sticker with 0.125in. markings from (-0.25in to +1.50in) is affixed to the reinforcement. In addition to optical displacement measurements, the displacement of the reinforcements during pullout testing is also recorded using a digital displacement gage.

Once the entire pullout mechanism is set up, all sensors are activated, and this allows the data acquisition system to start recording data. Four channels of digital data are collected during each pullout test in addition to the data from the earth pressure cells. These four channels of data are collected using a Vishay 5000 data acquisition system connected to a personal computer and controlled by "Strain Smart" software. The four digital data channels are:

1. Pullout load from annular load cell
2. Pressure transducer in the hollow core hydraulic jack
3. Linear displacement transducer
4. Pressure transducer in the overburden stress application hydraulics

Data are collected at frequency of 10Hz on all of these channels. Data from the earth pressure cells are displayed on the computer screen together with the other four data channels during testing, but the actual recording of pressure cell data requires a Geokon data-logger which records earth pressure cell data continuously at a frequency of 4 times per minute.

Once the data acquisition system is activated, the vertical overburden pressure is applied up to the target pressure. Testing then begins by applying the pullout force via the hollow core hydraulic jack by pumping the jack manually. Reinforcements are pulled until an optical displacement of 1.5in is reached or until the reinforcement ruptures.

The above procedure is repeated until all MSE reinforcements for a test setup are tested. The test setup is then decommissioned which consists of archiving the test data, removing the fill from the MSE Test Box, removing and inspecting the condition of the tested earth reinforcements, and cleaning up the system in preparation for the next test setup.

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2.5 Pullout Test Matrix

2.5.1 Overview

The pullout test program commissioned for this study included a total of 16 pullout tests on HA-strips and 24 pullout tests on HA ladders. These pullout tests were completed in two separate test setups. In addition to these two setups, this report also presents pullout test data for 28 additional pullout tests on HA ladder reinforcements which were previously conducted during the course of TxDOT project 0-6493, but which were not part of that study.

2.5.2 Pullout Tests for HA-Strips

Pullout tests on 16 HA-strips were commissioned for this study. These tests were performed in one test setup as shown in Table 2.2. Two different lengths of HA-strips were tested: 8ft and 12ft. These reinforcements were tested under effective overburden depths of 5ft, 12ft, 20ft and 40ft. Each test was conducted in replicates of two. All strip reinforcements were provided with electrical strain gages at 2.0ft intervals along their length.

Table 2.2 Test matrix for HA-strips.

Overburden Depth	12ft HA-strips	8ft HA-strips
5ft	2 replicates	2 replicates
12ft	2 replicates	2 replicates
20ft	2 replicates	2 replicates
40ft	2 replicates	2 replicates

2.5.3 Pullout Tests for HA Ladders

Table 2.3 provides a summary of the test matrix for the 24 pullout tests on HA-ladders commissioned for this study. Two different lengths of HA-ladders were tested: 4ft and 8ft. These reinforcements were tested under overburden depths of 3ft, 5ft, 6ft, 12ft and 20ft, all of which were accomplished in one test setup. Due to some low-displacement ruptures, the depths of fill were modified as testing proceeded which resulted in an asymmetrical test matrix. Six HA-ladder reinforcements (8ft length) in the top and middle layers were provided with electrical strain gages.

Table 2.3 Test matrix for HA-ladders.

Overburden Depth	8ft HA-ladders	4ft HA-ladders
3ft	3 replicates	3 replicates
5ft	1 replicate	
6ft	5 replicates	3 replicates
12ft	3 replicates	3 replicates
20ft		3 replicates

In addition to the 24 pullout tests on HA ladders described in Table 2.3, the test program for HA-ladders also includes data for 28 additional pullout tests which were conducted during the

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course of TxDOT project 0-6493. Table 2.4 summarizes the test matrix for these 28 additional HA ladder pullout tests.

Table 2.4 Test matrix for additional HA-ladders

Overburden Depth	8ft HA-ladders	4ft HA-ladders
3ft	3 replicates	3 replicates
5ft	5 replicates	5 replicates
9ft	3 replicates	3 replicates
12ft	1 replicate	5 replicates

2.6 Pullout Testing of Strain-Gaged MSE Reinforcements

2.6.1 HA-Strips

As mentioned previously, selected HA-strip and HA-ladder reinforcements were instrumented with strain gages. The objective of these gaged pullout tests was to develop a better understanding of the mechanisms that control development of pullout resistance in MSE reinforcement.

The test program included a total of 16 pullout tests (See Table C in Appendix C) on strain-gaged ribbed strips, with eight of the strips at 8ft long and eight at 12ft. Figure 2.9 shows the strain gage layout used on a typical 12ft long ribbed steel reinforcement. Gages were installed at a distances of 1ft, 3ft, 5ft, 7ft, 9ft and 11ft from the front end of the pull face of the MSE Test Box. Gages were mounted in pairs (i.e., both top and bottom) of the strip at each gage location.

The instrumentation layout shown in Figure 2.9 was designed to facilitate evaluation of how the pullout load is distributed along the length of the reinforcement. A uniform soil stress distribution (expected) would suggest that the reinforcement is behaving in an inextensible manner with uniform slip between the reinforcement and soil at both front and back ends of the strip.

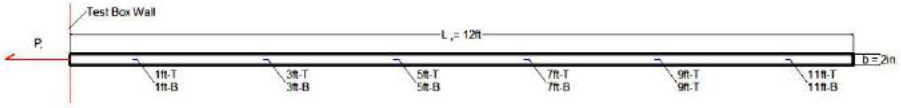


Figure 2.9 Strain gage plan for HA-strips.

The strain gages used for this study were general-purpose, prewired, Micro-Measurement SR-4, 350ohm SR-4 linear gages. Vishay Precision Group distributes these gages under item identification C2A-13-062LW-350. The gages were applied to the cleaned steel surface according to Vishay Micro-Measurements Instruction Bulletin B-127-14 using M-Bond 200, a cyanoacrylate bonding system (Vishay Precision Group 2010). The gages were then protected using M-Coat J, a hard-curing polysulfide protective coating system. The M-Coat J is the most robust strain gage protection system distributed by Vishay Precision Group.

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2.6.2 *HA-Ladders*

Selected HA-ladders were also instrumented with strain gages. A total of 6 pullout tests (see Table D in Appendix D) were performed on strain-gaged HA-ladders. All gaged ladders were 8ft long. Figure 2.10 shows the strain gage layout used on a typical 8ft long HA ladder reinforcement. Accordingly, gages were installed at distances of 1ft, 3ft, 5ft and 7ft from the pull-face of the MSE Test Box. Gages were mounted in pairs (i.e. both top and bottom) on each longitudinal bar at each gage location.

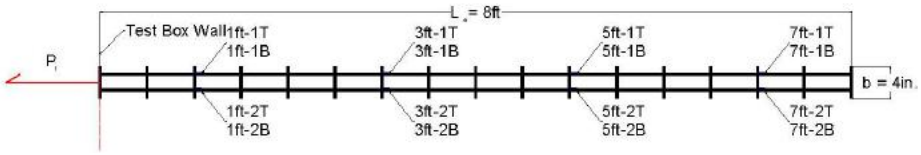


Figure 2.10 Strain gage plan for HA-ladders.

The gages used for HA-ladders are identical to those applied to the HA-strips and were applied using the same method with one exception. Each gage location was sanded flat using a belt sander with fine grit sand paper. The sanded plane was less than 0.25in. wide. This was done to avoid collapsing of the foil gage by bending it around the curved surface of the longitudinal bar (which would have rendered the gage unusable). As with the gaged strips, M-Coat J was used to provide durable protection for gages applied to HA-ladders.

2.7 Data Analysis

The information collected during each pullout test consists of three separate categories of data: (a) the pullout force measured using a load cell as well as by a pressure transducer attached to the hydraulic line of the jack, (b) the pullout displacement measured optically with the help of a witness marker against a graduated scale and digitally using displacement gages, and (c) the vertical overburden pressure measured using earth pressure cells and by measurement of fluid pressure in the hydraulic jacks used to apply the overburden stress. In the case of strain-gaged strips and ladders, data acquired also included strains measured at various locations and various levels of pullout load.

The independent measurements for each variable of interest were combined using the following equation (Lyons 1991).

$$a = \frac{\sum \frac{a_i}{\sigma_i^2}}{\sum \frac{1}{\sigma_i^2}} \qquad (1)$$

where: a = combined measurement
a_i = Measurement made with a specific measuring system
σ_i = Standard deviation associated with each measurement

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Equation 1 allows multiple independent measurements of the same variable to be combined in such a manner that the measuring system providing more consistent data (*i.e.*, less error) receives higher weight and the measuring system with less consistent data (*i.e.*, more error) receives lower weight. The overall standard deviation corresponding to the combined measurement is given by Equation 2 (Lyons 1991).

$$\frac{1}{\sigma^2} = \sum \frac{1}{\sigma_i^2} \qquad (2)$$

where: σ = combined standard deviation
σ_i = Standard deviation associated with each measurement

Figure 2.11 shows a typical load-displacement diagram obtained from a pullout test. This load-displacement curve represents a pullout load test conducted on an HA-strip reinforcement. The figure shows two separate plots of load versus displacement; one for the pull force calculated based on the hydraulic jack pressure transducer measurement and the other for the pull force calculated from load cell readings. Solid black circles represent the combined pull forces calculated using Equation 1 at displacements of 0.5in. and 0.75in., which are typical.

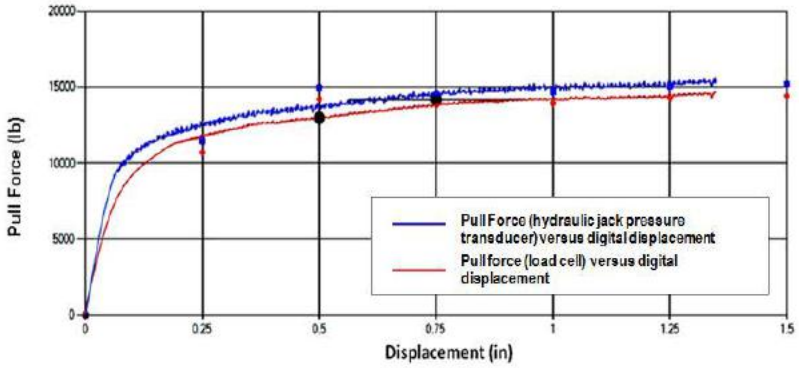


Figure 2.11 Typical load displacement curve for HA-strips.

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The pullout loads corresponding to 0.5in. and 0.75in. displacements are used to determine the pullout resistance factor, F* in the following equation (AASHTO 2010b):

$$F^* = \frac{P_r}{C\,\alpha\,L_e\,b\,\sigma_v} \quad (3)$$

where: P_r = pullout load corresponding to a specific displacement
L_e = Length of embedment of the reinforcement
b = Width of the reinforcement
σ_v = Vertical overburden pressure
C =2
α = 1.0

This F* calculation procedure was used for all MSE pullout tests conducted for this study, with data typically calculated for 0.5in. and 0.75in. deformations. In the case where rupture occurred prior to the established deformation thresholds, F* was calculated for the pullout load corresponding to rupture.

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3 RESULTS AND DISCUSSION

3.1 Pullout Test Results

3.1.1 HA-Strip Pullout Test Results

The appendices present MSE pullout test reports for each individual pullout test, along with a summary table of key test parameters for all tests. Appendix A presents pullout test reports on HA-strips as commissioned for this RECo study. Each pullout test report includes test information, a description of the reinforcement element, backfill compaction data, a tabular summary of the pullout test results, the load-displacement curve, overburden pressure data, strain rate data, and a summary of backfill material properties. These test reports should be referred to for detailed information about a particular pullout test.

At a summary level, Figure 3.1 and Figure 3.2 present the results from the 16 pullout tests performed on RECo HA-strips. These are plots of the pullout resistance factor, F*, versus depth of fill for each test. Charts are differentiated by the pullout load corresponding to 0.50in. and 0.75in. displacements. When calculating F*, it should be noted that according to accepted experimental procedures for inextensible reinforcements, a maximum displacement of 0.75-in is used to select the pullout load if the maximum value for pullout load or rupture of the specimen does not occur first (National Highway Institute 2001).

The data in each chart are differentiated by length and by rupture condition. In the case where the test specimen ruptured before the designated displacement, the F* value was calculated at the point of rupture.

HA-strip pullout results show a total of five ruptures prior to the designated displacement, two at displacements less than 0.5in. and three at displacements less than 0.75in. or 31 percent of the population. These ruptures occurred at pullout forces which ranged from 21,431lb to 24,051lb, average 22,454lb. This corresponds to the *de facto* rupture capacity of HA-strip reinforcement connections.

Figure 3.13 and Figure 3.14 place the RECo pullout test data collected for this study within the context of the larger set of HA-strip pullout test data obtained from TxDOT project 0-6493. These data are also differentiated by displacement and rupture condition.

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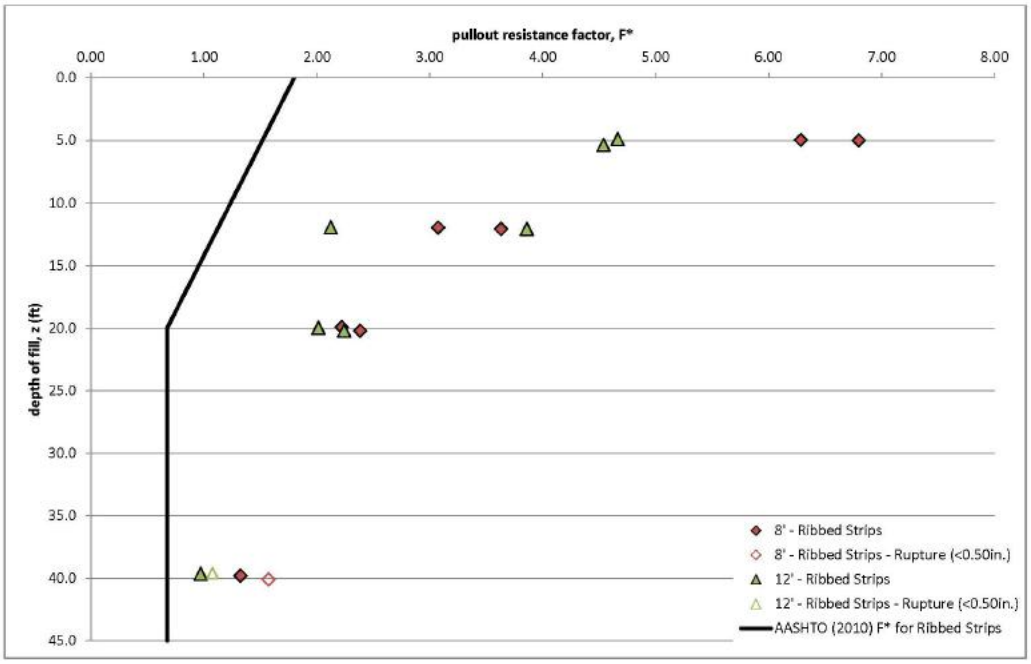


Figure 3.1 Pullout resistance factor, F* vs. depth of fill, z for RECo HA-strips; pullout load corresponding to 0.50in. displacement.

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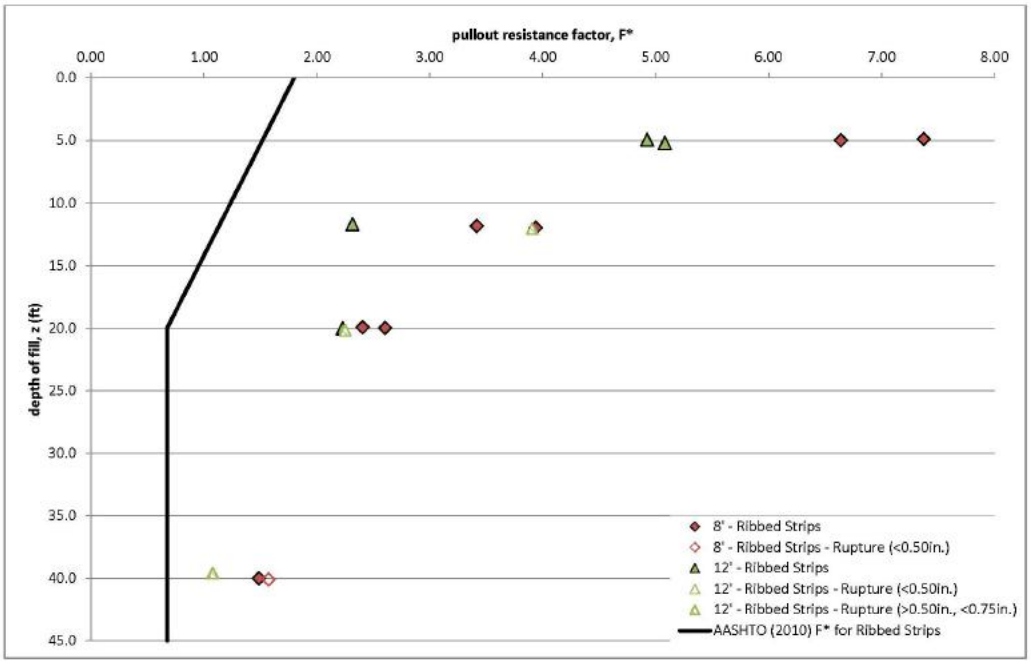


Figure 3.2 Pullout resistance factor, F* vs. depth of fill, z for RECo HA-strips; pullout load corresponding to 0.75in. displacement.

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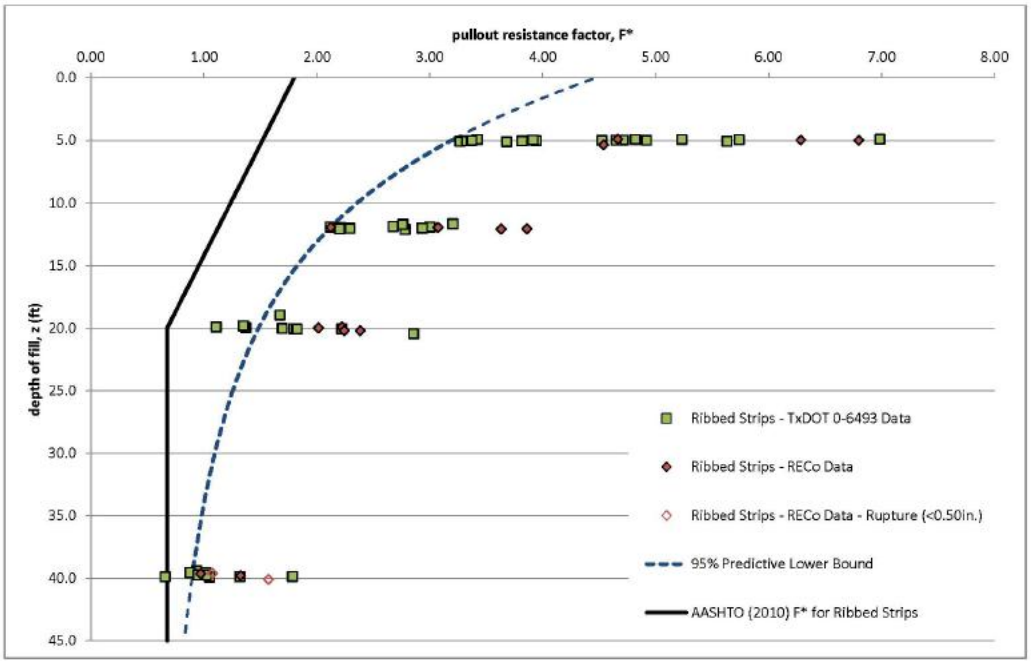


Figure 3.13 Pullout resistance factor, F* vs. equivalent overburden depth for RECo HA-strips; pullout load corresponding to 0.50in. displacement; RECo and TxDOT 0-6493 data.

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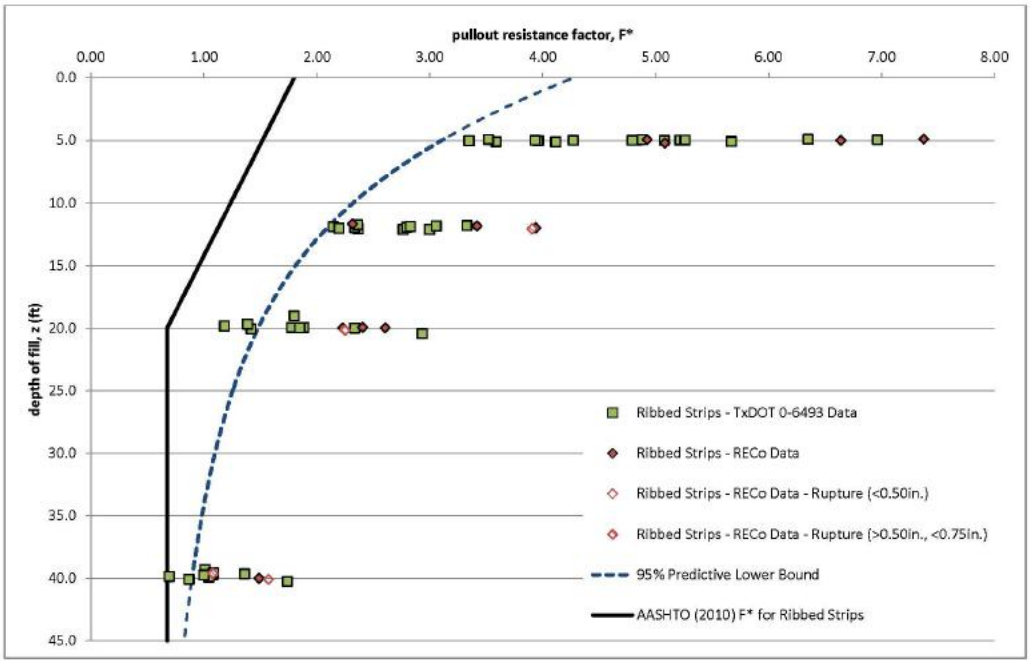


Figure 3.14 Pullout resistance factor, F* vs. equivalent overburden depth for RECo HA-strips; pullout load corresponding to 0.75in. displacement; RECo and TxDOT 0-6493 data.

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3.1.2 *Statistical Analysis of HA-Strip Pullout Test Data*

Figure 3.13 and Figure 3.14 show two reference lines for F*. The first line, designated by a solid (black) line, is the AASHTO reference line for default F*. This reference line represents the 95th percent lower limit for strip pullout data based on research conducted by the Florida DOT using uniform beach sand with a fine to medium grain size (The Reinforced Earth Company 1995). The second reference line on Figure 3.13 and Figure 3.14, designated by the dashed (blue) line, represents the 95th percent lower limit for the pullout test data obtained from this study. The techniques used to establish this lower-bound predictive limit are explained below.

Statistical procedures such as analysis of variance (ANOVA) and regression analysis require that the data set satisfies certain prerequisite conditions including: (a) normality, (b) homoscedasticity, and (c) independence. The requirement of homoscedasticity, also called uniformity of variance, was clearly not met as evidenced by non-uniform variance in the F* data. In other words, the variance in F* differed over the range of depths of fill considered, with higher variance at a depth of fill of 5ft and lower variance with increasing depth of fill. To address this, F* data were transformed into ln(F*) using a procedure known as the Box-Cox transformation (Osborne 2010), and this was found to be the optimal transformation to satisfy the homoscedasticity requirement. The *transformed* data set met all three prerequisite conditions identified above, thus allowing statistical evaluation.

In the next step, statistical analyses were performed on transformed F* – *i.e.*, ln(F*) – to develop an appropriate prediction model for the F*-depth of fill relationship. In this analysis, the F* data set for all HA-strips was treated as the *dependent variable*. The F* values calculated based on pullout loads measured at 0.50in and 0.75in displacements were considered separately because of the strong correlation that exists between these two variables.

Several *independent variables* were considered as possibly influencing F*. The most obvious among independent variables is the depth of fill. The dependence of F* on depth of fill has been clearly established through previous test data. A second variable that was of interest was the length of reinforcement. Current AASHTO specifications assume that F* is independent of the length of reinforcement, so our analyses included length of reinforcement among the independent variables so that the validity of the assumption could be tested. Also included among the independent variables were certain other variables that represent *test conditions* associated with pullout testing for a test setup. These variables included the position of the reinforcement within the MSE Test Box (top, middle or bottom layer), the order in which the tests were performed, maximum prior overburden stress, the number of times overburden stress on the reinforcement exceeded the test overburden stress, and the ratio between the maximum prior overburden stress and the test overburden stress. These test condition variables were included in the analysis so that any undue influence they might have on F* could be evaluated.

In the statistical analysis performed, the depth of fill emerged as the independent variable that has the most dominant influence on F*. The effect of length of reinforcement was determined to be statistically insignificant for lengths of 8ft and less. Reinforcements 12ft long showed possible influence, but this finding was not conclusive. Therefore, the reinforcement length was not incorporated as an independent variable in the final predictive model.

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Among the test condition variables examined, the test layer was identified as a significant variable influencing F*. The ratio between the maximum prior overburden stress and the test overburden stress was also found to be a significant variable. The final predictive model for F*, however, should be free of systematic influence from such test condition variables. Therefore, standard test conditions were defined to normalize the influence of these test conditions. Reinforcement placed in the middle layer of the MSE Test Box would experience minimum interference from potential boundary effects and therefore, the middle layer was selected as the standard (or reference) test position. Similarly, a reinforcement for which the overburden stress never exceeded test overburden stress (maximum prior overburden stress equal to test overburden stress) was chosen as the reference. With these test condition variables accounted for, the final predictive model was established for the data set, including the lower-bound 95th percent prediction limit, and the transformed model – $ln(F^*)$ – was converted back to the original scale – F*.

The reference line shown as a dashed (blue) line in Figure 3.13 and Figure 3.14 represents the 95th percent lower prediction limit for the F* data collected and analyzed in this study. It should be noted that the data shown on these figures – *i.e.*, individual test points – have not been adjusted to represent standard test conditions; however, the lower-bound prediction limit shown does represent F* values that would result from pullout tests performed under standard test conditions. One of the implications of this approach is that more than 5 percent of the data points appear to the left of the lower bound predictive limit.

3.1.3 Discussion of HA-Strip Pullout Test Results

The AASHTO reference line shown in summary data plots for ribbed strips is based on default design parameters for the backfill: *i.e.*, an internal friction angle of 34° and a uniformity coefficient of 4. This reference line represents the default F* vs. overburden relationship when actual soil parameters or pullout test data are not available for a given soil or soil-reinforcement combination. As such, the AASHTO reference line establishes a basis for comparison of the data. All pullout test data collected for this study plot to the right of the AASHTO reference line. This indicates that the pullout behavior of HA-strips embedded in the sandy backfill used for this study surpasses the AASHTO default value. The margin between actual pullout behavior and the AASHTO reference line is greatest for shallow overburden depths, and the margin decreases with increasing overburden depth.

Arguably, the data from this study are limited, consisting of only 16 tests on HA-strips. However, the combined dataset – consisting of pullout tests conducted for this study (16 HA-strips) plus pullout tests performed as part of TxDOT project 0-6493 (47 HA-strips) – totals 63 pullout tests. This larger dataset shows the same trend; namely, that a positive margin exists between actual pullout behavior and the AASHTO reference line, and this margin is greatest for shallow overburden depths but decreases with increasing overburden depth.

The margin between default and actual values represents a positive finding for pullout resistance of RECo HA-strips embedded in sandy backfill. If default values for pullout resistance are used for design, the margin represents an additional safety factor. Alternatively, use of higher pullout resistance values in the design corresponding to the 95th percent lower-bound predictive relationship depicted in Figure 3.13 and Figure 3.14 could result in cost savings for the constructed wall.

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Regarding a comparison of F* values as determined for pullout loads corresponding to deformations of 0.50in and 0.75in, the analysis indicates that a statistically-significant yet small difference exists. Mean F* values associated with 0.50in displacement are approximately 95.5 percent of the mean F* values associated with 0.75in displacement. This suggests the soil-structure response is sufficiently stiff to develop practically all of the pullout capacity of HA-strips by 0.50in displacement.

3.1.4 HA-Ladder Pullout Test Results

Appendix B presents pullout test reports on HA-ladders as commissioned for this study. Each pullout test report includes test information, a description of the reinforcement element, backfill compaction data, a tabular summary of the pullout test results, the load-displacement curve, overburden pressure data, strain rate data, and a summary of backfill material properties. These test reports should be referred to for detailed information about a particular pullout test.

Figure 3.15 and Figure 3.16 summarize pullout test data collected for RECo HA-ladders. Both charts present pullout resistance factor, F*, versus depth of fill for combined data from the 24 HA-ladders tested for this study and the 28 HA-ladders tested during the course of the TxDOT project. The charts are differentiated by the pullout force corresponding to displacements of 0.50in. and 0.75in.

The data presented on each chart are differentiated by length and by rupture condition. In the case where the test specimen ruptured before the designated displacement, the F* value was calculated at the point of rupture.

HA-ladder pullout results show a total of 16 ruptures prior to the designated displacement, nine at displacements less than 0.5in and seven at displacements less than 0.75in, or 31 percent of the population. These ruptures occur at pullout forces which range from 14,142lb to 18,234lb, average 16,247lb, which is consistent with the typical rupture capacity of HA-ladder reinforcements (The Reinforced Earth Company n.d.).

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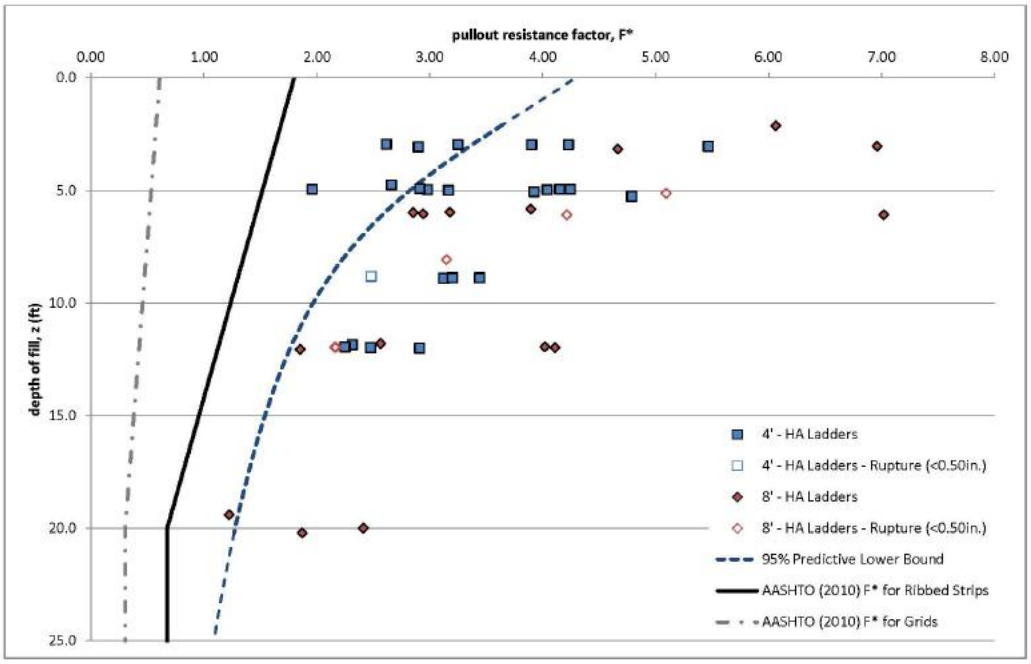


Figure 3.15 Pullout resistance factor, F* vs. equivalent overburden depth for RECo HA-ladders; pullout load corresponding to 0.50in. displacement.

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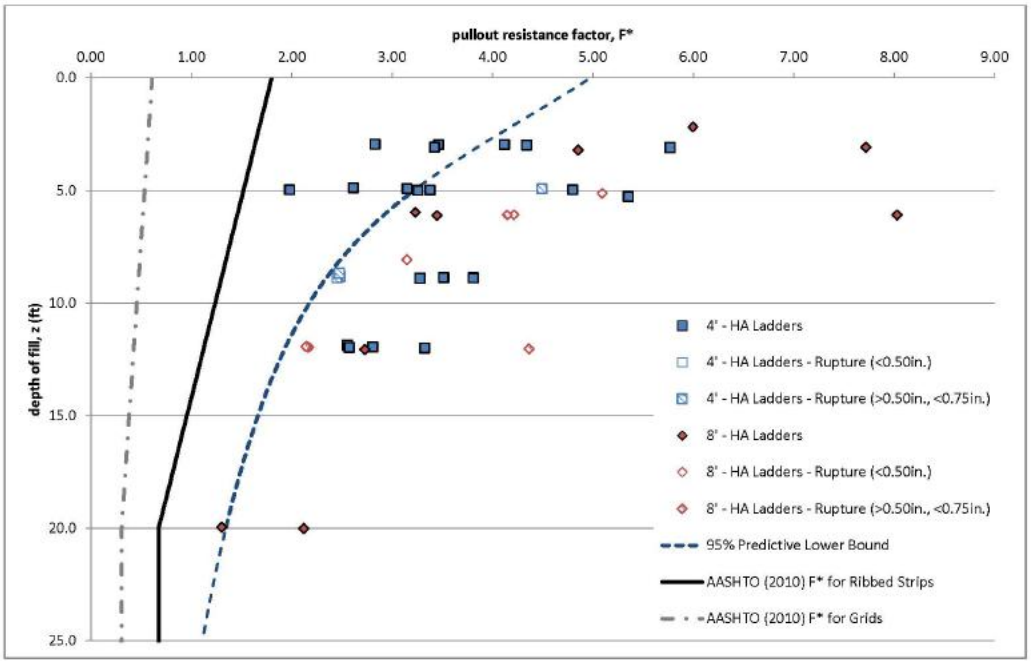


Figure 3.16 Pullout resistance factor, F* vs. equivalent overburden depth for RECo HA-ladders; pullout load corresponding to 0.75in. displacement.

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3.1.5 *Statistical Analysis of HA- Ladder Pullout Test Data*

The statistical analyses performed on F* data collected from pullout testing of HA-ladders followed the same approach as that described previously for HA-strip data. Once again, the primary objective in the statistical analyses was to establish the 95th percent lower prediction limit for each data set. The prediction limits obtained for F* values corresponding to 0.50-in and 0.75-in displacements are shown in Figure 3.15 and Figure 3.16 by dashed (blue) lines. It should be noted that these prediction limits are applicable to F* values obtained from pullout tests performed under standard test conditions. However, the F* data shown in these plots – *i.e.*, individual test points – are unadjusted values corresponding to actual test conditions. Again, one of the implications of this approach is that more than 5 percent of the data points appear to the left of the lower bound predictive limit.

3.1.6 *Discussion of HA-Ladder Pullout Test Results*

As was the case for HA-strips, statistical analysis of the RECo HA-ladder pullout test data shows a statistically-significant yet slight difference between the F* vs. overburden relationship as determined for pullout loads corresponding to deformations of 0.5in. and 0.75in. Mean F* values associated with 0.50in displacement are approximately 92.1 percent of the mean F* values associated with 0.75in displacement. This suggests the soil-structure response is sufficiently stiff to develop practically all of the pullout capacity of HA-ladders by 0.50in. displacement. Further, the pullout test data show that pullout resistance for HA-ladders is independent of length, which is the expected relationship.

The RECo HA-ladder consists of two W10 longitudinal bars spaced at 2in center-to-center and W10 transverse bars spaced at 6in on center (Figure 2.8). Each transverse bar is 4in long, resulting in a 1in tang on both sides of the longitudinal bars. However, unlike tangs for typical welded steel grids which are an irregular artifact of fabrication, the tangs for RECo HA-ladders are an intentional feature of the design. For this reason, a conceptual question exists as to whether HA-ladders should be analyzed as grid-type reinforcements where the width is equal to the centerline spacing of the longitudinal bars (*i.e.*, 2in) or as a ribbed strip type reinforcement where the width is equal to the length of the transverse bars (*i.e.*, 4in).

To evaluate this question, Figure 3.15 and Figure 3.16 each include two AASHTO reference lines. The solid (black) line is the AASHTO reference line for ribbed strip reinforcements determined based on an element width of 4in. The dotted (gray) line is the AASHTO reference line for a welded steel grid having a longitudinal bar spacing of 2in. The AASHTO reference line for grids appears overly conservative for ribbed strips.

Comparison of the HA-ladder pullout data to these AASHTO reference lines shows that the data plot to the right of both lines. Moreover, the 95th percent lower prediction limit established for the data collected in this study also lies to the right of the AASHTO reference lines. Thus, the pullout test data for this study support the view that HA-ladders can, at a minimum, be treated in the MSE design process as AASHTO ribbed strip elements with a width equal to the transverse bar length (4in.). This is the view promoted by RECo in that they refer to the HA-ladders as a "strip" type element.

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3.2 Strain Gage Results

3.2.1 Overview

Appendix C presents strain gage reports for RECo HA-strips, and Appendix D presents strain gage reports for RECo HA-ladders. Due to the inherently descriptive nature of this data, there is no simple way to summarize the findings. Rather, this section of the report describes the information provided in each strain gage test report using the HA-strip element TS29-S2-Z12-L12-T as a representative example for both types of reinforcement.

3.2.2 Gage Plan

Each strain gage report includes a small sketch of the gage plan showing the basic gage identifiers. Figure 3.7 is a representative sketch for a 12ft HA-strip, and Figure 3.8 is a representative sketch for an 8ft HA-ladder. The gage plans for both types of reinforcements use matched gages, top (T) and bottom (B), at each test location measured from the MSE Test Box wall. The use of gage pairs facilitates calculation of an average stress at each test location, thus helping to eliminate effects of incidental bending in the reinforcement. Additionally, since incidental bending is expected to be minor, the matched gages provide a degree of redundancy.

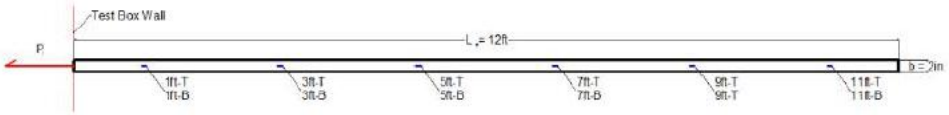


Figure 3.7 Strain gage plan for a 12ft HA-strip.

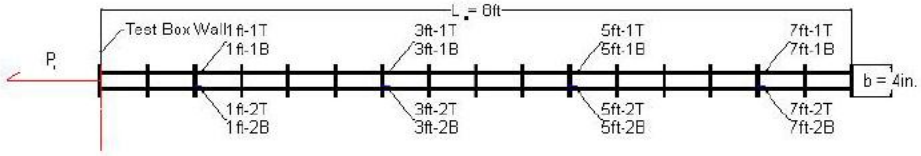


Figure 3.8 Strain gage plan for 8ft HA-ladder.

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3.2.3 Axial Load Data by Gage

Each strain gage report begins with a plot of axial load data by gage, a typical example being the chart shown in Figure 3.9. In this chart, the raw strain data have been converted into axial loads by multiplying the measured strain by the cross-sectional area of the element and the modulus of elasticity of steel. The data were filtered to show only the upper envelope of the strain data and to eliminate artifacts of the load application method (hydraulic pump). The data analysis process also included systematic filtering of results to exclude data when the strain gages began to fail. This approach errs on the side of preserving as much data as possible. This means that some of the strain data presented in the individual test plots may be unreliable, but this is preferable to unnecessarily or arbitrarily deleting what may be reliable data.

Figure 3.9 is a plot of axial pullout force measured via strain gages (vertical axis) versus total pullout force measured during the pullout test (horizontal axis). The black reference line corresponds to the pullout force measured at the MSE Test Box wall. The basic trend is that axial pullout forces measured farther from the test box wall are lower, as would be expected.

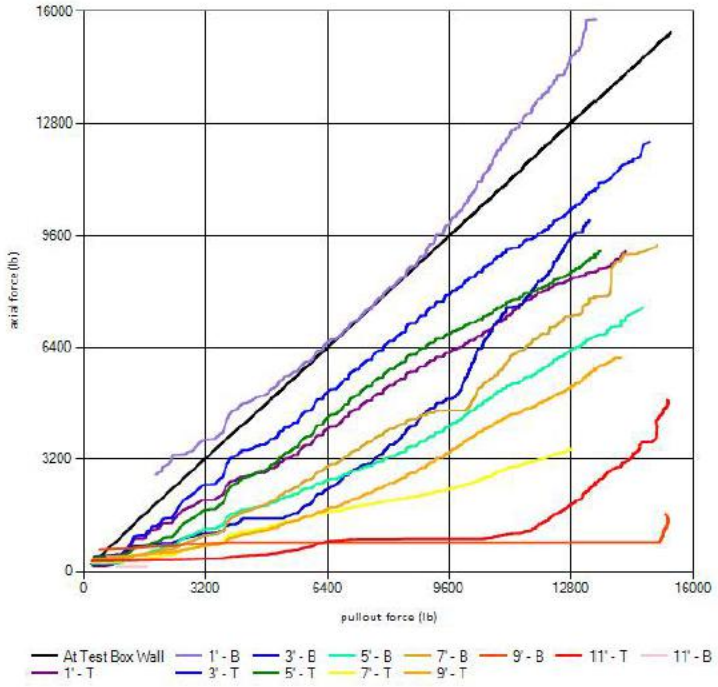


Figure 3.9 TS29-S2-Z12-L12-T axial load data by gage.

It should be noted that the strain gage analysis is carried out in the load domain unlike the pullout test reports which are reported in the displacement domain. The load domain was

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selected for several reasons. The primary reason is that the pullout force develops quickly under relatively small displacement. In the early displacement range, there is some question of digital displacement reliability due to movement during pull rod seating. Additionally, even with the M-Coat J heavy-duty strain gage protection, the strain gages tend to be scraped from the reinforcement after some displacement.

Figure 3.9 is the most basic strain gage data summary, and these plots will typically exhibit a fan appearance. It should be noted that Gage 9'-B (the bottom gage located 9ft from the MSE Test Box wall) is probably failing at a pullout load of about 15,000lb. Similarly, Gage 11'-T is possibly beginning to fail at loads greater than about 11,500lb. As noted above, the conservative filtering approach used to process the axial strain readings retains most of these data, and this has implications for subsequent plots.

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3.2.4 Axial Load Data by Location

Axial load data by location represent the next stage in processing strain gage data. For this step, the matched gage pairs are averaged to eliminate any incidental bending, the result being a plot like that shown in Figure 3.10. This plot identifies all gage locations with recorded data, and test locations with the most working gages are shown with the heaviest line weights. It should be noted that HA-strip data will typically provide an average for two gages; whereas, HA-ladder data will typically average up to four gages.

As gages fail, discontinuities appear in the data plot. Sometimes these discontinuities are marginal as seen in the 3ft gages in Figure 3.10. At other times the discontinuities are more pronounced as seen in the 1ft and 7ft gages. Sometimes the discontinuities result in obvious errors such as the 9ft gages for loads beyond 14,000lb.

The measured axial force is greatest near the test box wall (*i.e.* location of load application) and least at the strain gage location furthest from the wall as should be expected.

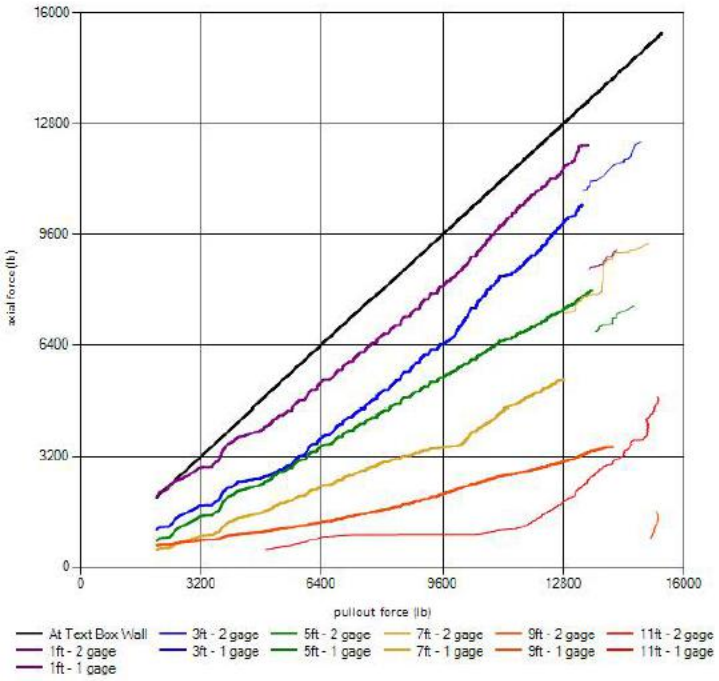


Figure 3.10 TS29-S2-Z12-L12-T axial load data by location.

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3.2.5 Axial Load along Length

The next step in processing the strain gage data is to determine the axial load along the length of the reinforcement. The report shows this information in the form of both a data table and a plot.

Table 3.1 is a typical data table. This data table presents axial load data by location at 1,000lb pullout force increments. This is achieved by sampling the continuous data presented in Figure 3.10 at discrete 1000lb loads. When the data are sampled from less than a complete set of gages, an asterisk or set of asterisks indicates the number of failed gages. Such data may not be unreliable but carries far less confidence. This can be seen in Table 3.1 for the example reinforcement.

Table 3.1 TS29-S2-Z12-L12-T axial load along reinforcement length at discrete, 1,000lb. loads.

Axial Load Results							
Pullout Resistance, P_r (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	401*	555	379	341	549	226	
2000	1923	1082	766	506	622	333*	
3000	2754	1673	1349	827	741	358*	
4000	3609	2331	2070	1315	916	430*	
5000	4144	2663	2538	1675	1043	524*	
6000	4909	3347	3180	2119	1212	730*	
7000	5762	4205	3836	2565	1412	918*	
8000	6672	5037	4482	3009	1653	937*	
9000	7554	5966	5100	3331	1920	937*	
10000	8515	6890	5727	3560	2265	937*	
11000	9690	8209	6397	4350	2587	1037*	0.29
12000	10742	8993	6918	4955	2819	1328*	0.38
13000	11735	10112	7561	7113*	3118	2043*	0.54
14000	8891*	11507*	7052*	8668*	2435	2956*	0.79
15000	No Data	12259*	No Data	9311*	835*	3897*	1.11

*calculated from 1 strain gage.

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Selected rows of data from the Table 3.1are plotted to visually show how axial load varies at each test location along the reinforcement, with Figure 3.11 providing a representative example. This type of plot is a graphical interpretation of the load distribution along the length of the reinforcing element.

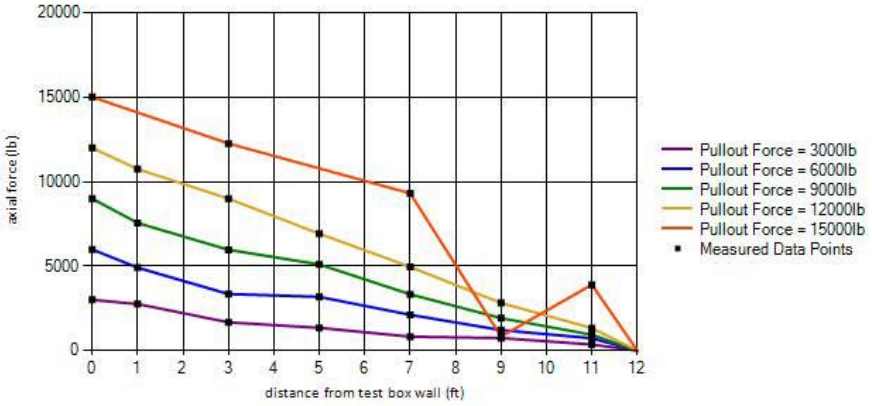


Figure 3.11 TS29-S2-Z12-L12-T axial load along length.

It is expected that the axial load in the reinforcement will equal the pullout force at the test box wall, and this axial force will decrease (linearly) to zero at the end of the element. The data obtained from this study tend to support this relationship with a few exceptions. Frequently, a localized increase in load appears at the 1ft location, and this is more pronounced at higher loads. This suggests a stress anomaly near the MSE Test Box wall; for example, a localized soil compression against the wall.

Other anomalies appear when the data become questionable due to failed gages. A representative example appears in Figure 3.11. Recall that for loads above 14,000lb, a questionable gage was noted in the previous plots for this test. In Figure 3.11, the failed gages become manifest as the irregular shape of the curve at 9ft from the test box wall under 15,000lb of pullout force. This type of error occurs more frequently at higher loads as more gages fail.

It is important to note that, in this plot, black symbols represent measured data. Locations with missing data symbols correspond to instrumentation points at which all the gages have failed. Examples of this can be seen in the data plot (Figure 3.11) at 1ft and 5ft locations corresponding to a pullout force of 15,000lb. When such data gaps occur, the data plot continues by connecting the data points on either side of the data gap. However, these types of data gaps degrade the reliability of the plot. For example, consider a reinforcing element for which all the strain gages have failed. A data plot for this reinforcement would be a straight line from the applied pullout force at the leading end of the reinforcement to zero at the unrestrained end. Such a data plot would appear to exhibit the expected (linear) trend perfectly, but in reality, it would not provide any information about the actual distribution of pullout load along the element. Consequently, the data are only analyzed when at least half of the gage locations have working gages.

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3.2.6 Average Shear Stress for Soil along Length

The final step in processing strain gage data was to determine how the average shear stress between soil and steel reinforcing material varies along the length of the reinforcement. For this purpose, the slope of each line segment in Figure 3.11 was calculated. These slopes, divided by the length of the reinforcement associated with that part of the plot (point to point), provides the average shear resistance between gage locations. Figure 3.12 and Table 3.2 present a sample data set.

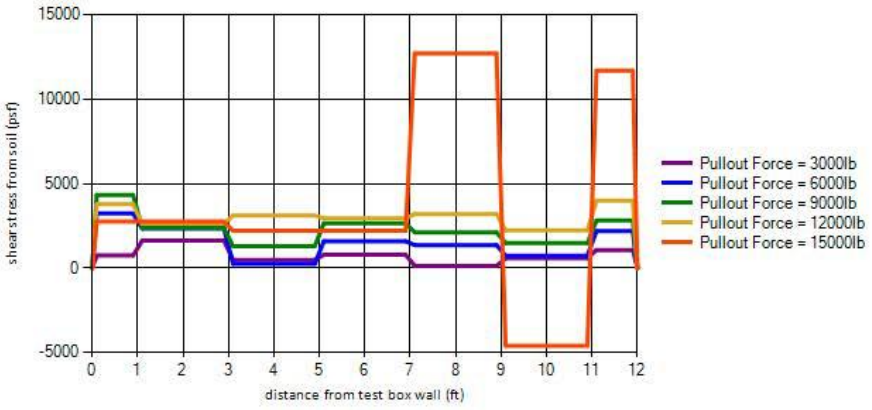


Figure 3.12 TS29-S2-Z12-L12-T average shear stress from soil along length.

Figure 3.12 indicates that, in general, the distribution of shear resistance along the length of the reinforcement is fairly uniform at each load threshold, which is the expected relationship. However, deviations from this trend can be noticed. Careful review of the data reveals that such deviations are often associated with anomalies. In fact, the anomalies observed in Figure 3.11 (axial load) become amplified when expressed in the soil shear resistance plot. For example, the axial force calculated at the 9ft location at a pullout force of 15000lb suggests probable gage failure at this load level. The result of this erroneous gage reading is clearly evident in the abnormally high and low shear resistance values seen in Figure 3.12 for 7-9ft and 9-11ft segments.

Whereas Figure 3.12 depicts the shear resistance relationship, Table 3.2 provides the average shear stress values for each segment of the reinforcement at 1,000lb load increments. This facilitates a more complete presentation of the data.

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Table 3.2 TS29-S2-Z12-L12-T average shear stress from soil along length at discrete, 1,000lb. loads.

Shear Stress from Soil Results									
Pullout Resistance, P_r (lb)	*Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall*								*Displacement (in.)*
	0-1ft	*1-3ft*	*3-5ft*	*5-7ft*	*7-9ft*	*9-11ft*	*11-12ft*	*Mean*	
1000	1848	-231	263	58	-312	485	677	254	
2000	221	1262	474	389	-174	433	1000	499	
3000	743	1622	487	782	129	575	1074	751	
4000	1200	1917	391	1133	599	728	1291	1002	
5000	2618	2222	187	1295	949	778	1571	1254	
6000	3228	2343	250	1591	1361	723	2190	1496	
7000	3669	2336	552	1907	1729	742	2753	1746	
8000	4019	2454	832	2210	2034	1074	2812	2003	
9000	4326	2382	1300	2653	2116	1475	2812	2249	
10000	4533	2437	1744	3250	1943	1991	2812	2506	
11000	3961	2221	2718	3072	2643	2325	3112	2753	0.29
12000	3778	2624	3112	2945	3204	2236	3985	3000	0.38
13000	3828	2435	3826	672	5993	1612	6129	3253	0.54
14000	15250	-3925	6684	-2424	9349	-781	8867	3494	0.79
15000	2748	2748	2211	2211	12714	-4592	11690	3752	1.11

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3.2.7 Summary Observations about the Strain Gage Data

The reports in Appendix C and Appendix D provide details of the strain gage results for individual pullout tests on HA-strips and HA-ladders. At a summary level, observations about the entire population of strain gage results are as follows:

- Strain gage data for both HA-strips and HA-ladder reinforcements indicate, in general, that a linear load distribution exists along the full length of the reinforcement element, varying from the pullout force at the leading end to zero at the free end.
- Strain gage data show this linear load distribution most consistently for pullout loads in the low to medium range, typically corresponding to displacements (movement of the reinforcement during the pullout test) of about ¼ inch or less.
- At higher pullout loads, the load relationship shows greater variability. This variability is likely due to the failure of strain gages as displacement of the reinforcement increases.
- It is not uncommon for the reinforcement to exhibit a localized load increase near the front of the element, just behind the MSE Test Box wall. The localized load increase is more pronounced at higher pullout loads.
- Because HA-ladders were gaged with four strain gages at each location (top and bottom of both longitudinal bars) and HA-strips were only gaged with two strain gages at each location (top and bottom), data for the ladders are less sensitive to loss of a gage and tend to be more consistent.

Overall, the strain gage data generally depict a linear load distribution along the full length of the HA-ladder and HA-strip reinforcements for the entire range of loads applied during pullout testing. As such, this load distribution more or less translates into a uniform distribution of soil resistance mobilized by the reinforcements. A uniform soil stress distribution indicates inextensible behavior of the reinforcement and is consistent with the coherent gravity design method.

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4 SUMMARY AND CONCLUSIONS

4.1 Summary

This report presents findings of a research study to investigate the pullout resistance behavior of HA-strip and HA-ladder reinforcements produced by the Reinforced Earth Company for mechanically stabilized earth (MSE) structures. The test program included a total of 16 pullout tests on HA-strips and 52 pullout tests on HA-ladders.

All MSE pullout tests for this study were performed in sandy backfill using the large-scale pullout resistance test system at Texas Tech University, the centerpiece of which is a 12ft by 12ft by 4ft MSE Test Box. Tests were conducted at overburden pressures corresponding to an equivalent depth of fill ranging from 3ft to 40ft. In addition to the pullout tests, selected HA-strip and HA-ladder reinforcements were instrumented with electrical resistance strain gages to obtain additional information about their pullout behavior.

Pullout test results for RECo HA-strips can be compared with the default F* vs. overburden relationship for ribbed strips published by AASHTO. The HA-strip test data plot to the right of the AASHTO reference line which indicates that the pullout behavior of HA-strips surpasses the AASHTO default value. The margin between actual pullout behavior and the AASHTO reference line is greatest for shallow overburden depths, and the margin decreases with increasing overburden depth. This margin can be appropriated into the MSE design process in the form of cost savings for constructed walls. The margin is a positive finding for the HA-strips tested for this study.

For HA-ladders tested in this study, comparison of pullout results to the AASHTO reference lines for both ribbed strips and welded steel grids shows that the data plot to the right of both reference lines. This finding helps to address the conceptual question as to whether HA-ladders should be analyzed as grid-type reinforcements where the width is equal to the centerline spacing of the longitudinal bars (*i.e.*, 2in) or as ribbed strip type reinforcements where the width is equal to the length of the transverse bars (*i.e.*, 4in). Because the pullout test data show a margin for HA-ladders relative to the ribbed strip AASHTO reference line, this supports the view that HA-ladders can be treated in the MSE design process as AASHTO ribbed strip elements with a width equal to the transverse bar length (4in).

The test program included affixing strain gages to 16 HA-strips and 6 HA-ladders in order to develop a better understanding of the mechanisms that control development of pullout resistance in MSE reinforcement. Results for both HA-strip and HA-ladder reinforcements typically show that the axial load in the MSE reinforcement equals the pullout force at the leading end, and this axial force decreases linearly to zero at the unrestrained end of the element. These data were used to establish the average soil shear stress mobilized along the length of the reinforcements for the full range of loads applied during pullout testing. This is a uniform stress distribution which is consistent with inextensible reinforcement behavior articulated in the coherent gravity method of MSE wall design.

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4.2 Conclusions

The findings of this research study support the following conclusions:

1. Statistical analysis of pullout test data for both HA-strips and HA-ladders show a slight increase in F* as determined for pullout loads corresponding to deformations of 0.75in compared to 0.50in. This suggests the soil-structure response is sufficiently stiff to develop practically all of the pullout capacity of HA-strips and HA-ladders by 0.50in displacement.

2. Approximately 31 percent of tests for both HA-strips and HA-ladders experienced rupture prior to the target deformations of 0.5in and 0.75in. In all cases of rupture, the pullout load corresponds to the typical maximum capacity of the reinforcement. Thus, rupture was expected.

3. Pullout behavior of RECo HA-strips in properly-compacted sandy backfill as expressed by the 95th percent lower-bound predictive limit is conservative compared to the default F* vs. depth of fill relationship for ribbed strips published by AASHTO. The margin between actual pullout behavior and the AASHTO reference line is greatest for shallow overburden depths, and the margin decreases with increasing overburden depth.

4. Pullout test data for RECo HA-ladders support the view that HA ladders can be treated in the MSE design process as AASHTO ribbed strip elements with a width equal to the transverse bar length (4in.).

5. Pullout behavior of RECo HA-ladders in properly-compacted sandy backfill as expressed by the 95th percent lower-bound predictive limit is conservative compared to the default F* vs. overburden relationship for ribbed strips published by AASHTO.

6. Results from strain gaged pullout tests for both HA-strip and HA-ladder reinforcements show that the axial load in the MSE reinforcement equals the pullout force at the leading end, and this axial force typically decreases linearly to zero at the unrestrained end of the element.

7. The observed axial force distribution in HA-strip and HA-ladder reinforcements produces a uniform soil shear stress mobilized along the full length of the reinforcements for the full range of loads applied during pullout testing. This is consistent with inextensible reinforcement behavior articulated in the coherent gravity method of MSE wall design.

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Appendix A:

Pullout Data for MSE Reinforcement Pullout Tests:

Reinforced Earth Company High Adherence Strips in Type B Backfill

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TABLE A: SUMMARY DATA FOR MSE REINFORCEMENT PULLOUT TESTS
REINFORCED EARTH COMPANY HIGH ADHERENCE STRIPS IN TYPE B BACKFILL

					0.50in. Displacement					0.75in. Displacement					Peak Displacement				
TestID	Length, Le (ft)	Width, b (in.)	Skew, β (°)	Relative Compaction, CR (%)	Displacement (in.)	Overburden, σv (psf)	DOF, z (ft)	Load, Pr (lb)	F*	Displacement (in.)	Overburden, σv (psf)	DOF, z (ft)	Load, Pr (lb)	F*	Displacement (in.)	Overburden, σv (psf)	DOF, z (ft)	Load, Pr (lb)	F*
TS29-S9-Z5-L8-M	8	2	0	97.8%	0.50	633	5.0	10610	6.29	0.75	637	5.0	11290	6.64	1.36	636		12382	
TS29-S13-Z5-L8-B	8	2	0	95.9%	0.50	636	5.0	11537	6.80	0.75	624	4.9	12261	7.37	1.34	653		13473	
TS29-S1-Z5-L12-T	12	2	0	99.9%	0.50	623	4.9	11629	4.67	0.75	629	4.9	12386	4.93	1.43	632		13761	
TS29-S5-Z5-L12-T	12	2	0	99.9%	0.50	683	5.4	12399	4.54	0.75	664	5.2	13492	5.08	1.40	686		15626	
TS29-S10-Z12-L8-M	8	2	0	97.8%	0.50	1540	12.1	14914	3.63	0.75	1529	12.0	16064	3.94	1.40	1537		17563	
TS29-S14-Z12-L8-B	8	2	0	95.9%	0.50	1524	12.0	12494	3.07	0.75	1509	11.8	13752	3.42	1.36	1488		15433	
TS29-S2-Z12-L12-T	12	2	0	99.9%	0.50	1523	11.9	12933	2.12	0.75	1491	11.7	13816	2.32	1.38	1513		15645	
TS29-S6-Z12-L12-M	12	2	0	97.8%	0.51	1538	12.1	23753	3.86						0.51	1539	12.1	24051	3.91
TS29-S11-Z20-L8-B	8	2	0	95.9%	0.50	2578	20.2	16396	2.39	0.75	2549	20.0	17709	2.61	1.45	2548		20881	
TS29-S15-Z20-L8-B	8	2	0	95.9%	0.50	2540	19.9	15061	2.22	0.75	2542	19.9	16315	2.41	1.46	2544		18018	
TS29-S3-Z20-L12-T	12	2	0	99.9%	0.50	2549	20.0	20547	2.02	0.75	2551	20.0	22766	2.23	0.99	2533		24494	
TS29-S7-Z20-L12-M	12	2	0	97.8%	0.51	2575	20.2	23101	2.24						0.52	2573	20.2	23160	2.25
TS29-S12-Z40-L8-B	8	2	0	95.9%	0.50	5073	39.8	17926	1.33	0.75	5100	40.0	20263	1.49	0.99	5096		21537	
TS29-S16-Z40-L8-B	8	2	0	95.9%											0.36	5112	40.1	21431	1.57
TS29-S4-Z40-L12-T	12	2	0	99.9%	0.50	5055	39.6	19665	0.97						0.73	5051	39.6	21871	1.08
TS29-S8-Z40-L12-M	12	2	0	97.8%											0.25	5051	39.6	21758	1.08

Table A

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 21, 2011
Test Identification:	TS29-S9-Z5-L8-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



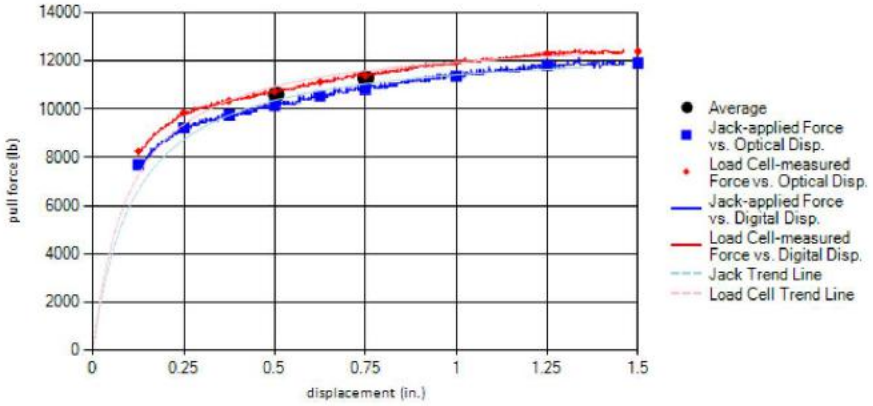
MSE Reinforcement	
Type:	Ribbed Strip
Length, L_e (ft):	8
Width, b (in.):	2
Skew Angle, β (°):	0

Backfill Material			
Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	121.7 ±3.3
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	6.5 ±0.8%
		Relative Compaction, C_R (%):	97.8 ±2.7%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	633 ±3	10,610 ±20	5.0 ±0.0	**6.29 ±0.03**
0.75	0.75 ±0.00	637 ±4	11,290 ±15	5.0 ±0.0	**6.64 ±0.05**
At Peak Load	1.36 ±0.00	636 ±5	12,382 ±22	No Data	**No Data**

Load-Displacement Curve



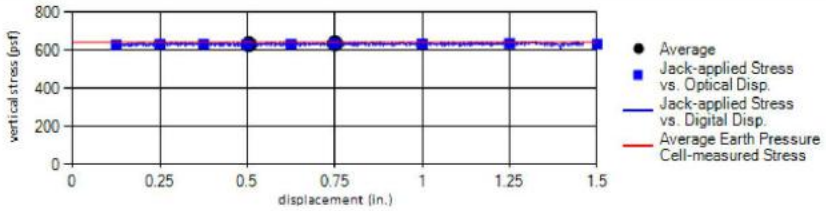
Comments	Personnel	
Strain gage data collected. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, YW, DH
	Prepared:	TW
	Checked:	WL, PJ

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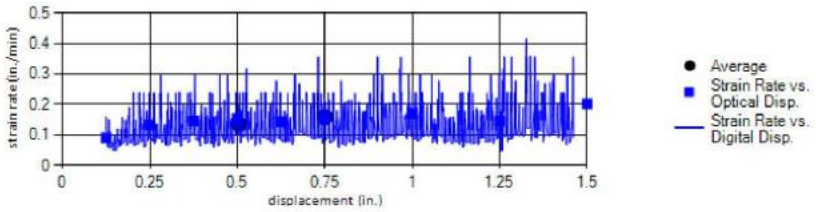
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
708 ±13	666 ±8	No Data	614 ±1	583 ±7	642 ±49	633 ±3	637 ±4	636 ±5



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.14 ±0.02	0.14 ±0.04	
0.75	0.16 ±0.04	0.16 ±0.04	0.15 ±0.06
At Peak Load	0.16 ±0.07	0.16 ±0.07	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S13-Z5-L8-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	8
Width, b (in.):	2
Skew Angle, β (°):	0

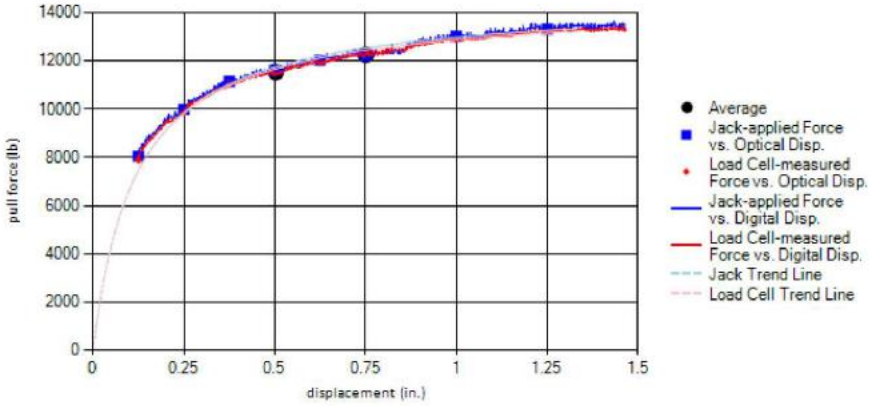
Backfill Material

Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	119.4 ±2.5
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	7.1 ±0.5%
		Relative Compaction, C_R (%):	95.9 ±2.0%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	636 ±4	11,537 ±25	5.0 ±0.0	**6.80 ±0.05**
0.75	0.75 ±0.00	624 ±7	12,261 ±35	4.9 ±0.1	**7.37 ±0.09**
At Peak Load	1.34 ±0.00	653 ±5	13,473 ±42	No Data	**No Data**

Load-Displacement Curve



Comments

Strain gage data collected. No incidental skew data collected. No incidental skew data.

Personnel

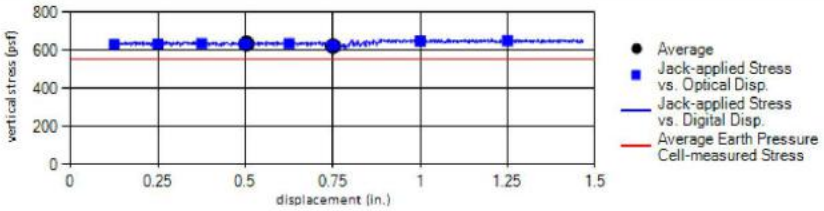
Tested:	TW, RH, AS
Prepared:	TW
Checked:	WL, PJ

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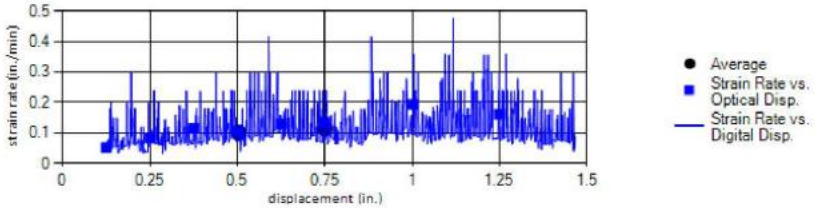
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
615 ±15	462 ±23	No Data	534 ±8	611 ±7	555 ±65	637 ±4	624 ±7	654 ±5



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.10 ±0.02	0.12 ±0.03	
0.75	0.11 ±0.03	0.15 ±0.04	0.13 ±0.07
At Peak Load	0.15 ±0.02	0.17 ±0.07	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 20, 2011
Test Identification:	TS29-S1-Z5-L12-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



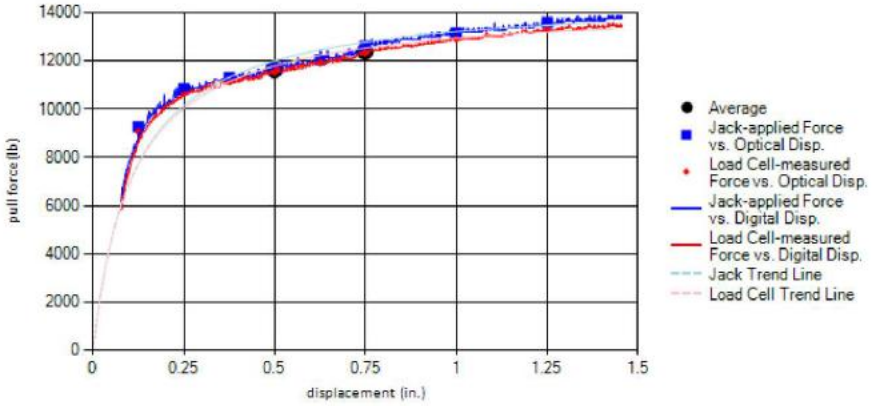
MSE Reinforcement	
Type:	Ribbed Strip
Length, L_e (ft):	12
Width, b (in.):	2
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	124.4	±1.5
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	5.8	±0.5%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	99.9	±1.2%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	623 ±6	11,629 ±36	4.9 ±0.0	**4.67 ±0.05**
0.75	0.75 ±0.00	629 ±2	12,386 ±19	4.9 ±0.0	**4.93 ±0.02**
At Peak Load	1.43 ±0.00	632 ±3	13,761 ±0	No Data	**No Data**

Load-Displacement Curve



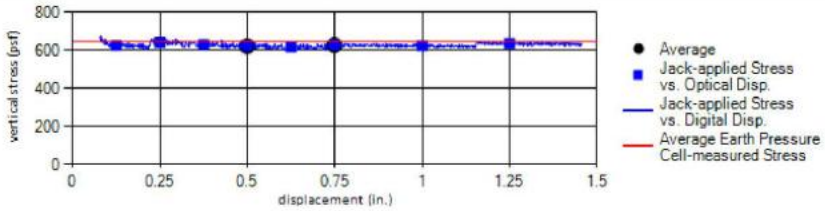
Comments	Personnel	
Strain gage data collected. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, RH, DH
	Prepared:	TW
	Checked:	WL, PJ

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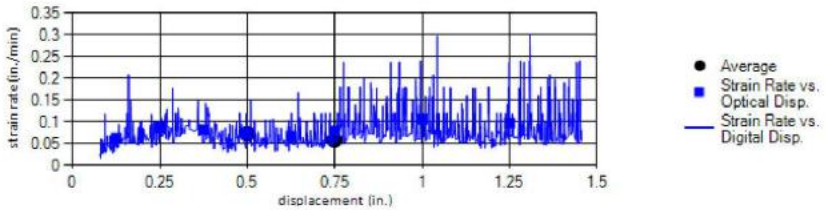
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
720 ±15	613 ±11	No Data	665 ±10	596 ±11	648 ±50	623 ±6	629 ±2	632 ±3



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.07 ±0.02	0.08 ±0.02	
0.75	0.06 ±0.00	0.07 ±0.02	0.08 ±0.04
At Peak Load	0.08 ±0.01	0.10 ±0.04	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E,	
Sieve Size	*Percent Retained*	*Percent Retained*	TEX-106-E, TEX-107-E)	
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S5-Z5-L12-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	12
Width, b (in.):	2
Skew Angle, β (°):	0

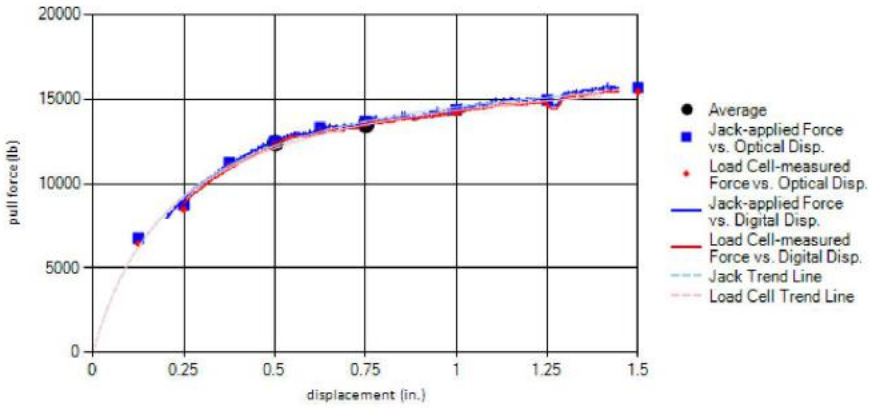
Backfill Material

Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	124.4 ±1.5
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	5.8 ±0.5%
		Relative Compaction, C_R (%):	99.9 ±1.2%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	683 ±4	12,399 ±0	5.4 ±0.0	**4.54 ±0.03**
0.75	0.75 ±0.00	664 ±6	13,492 ±37	5.2 ±0.0	**5.08 ±0.05**
At Peak Load	1.40 ±0.00	686 ±3	15,626 ±51	No Data	**No Data**

Load-Displacement Curve



Comments

Strain gage data collected. No incidental skew data collected. No incidental skew data.

Personnel

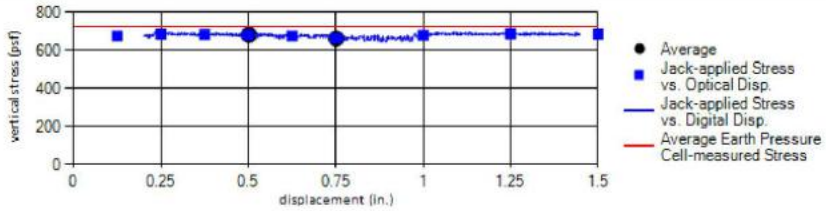
Tested:	TW, RH, AS
Prepared:	TW
Checked:	WL, PJ

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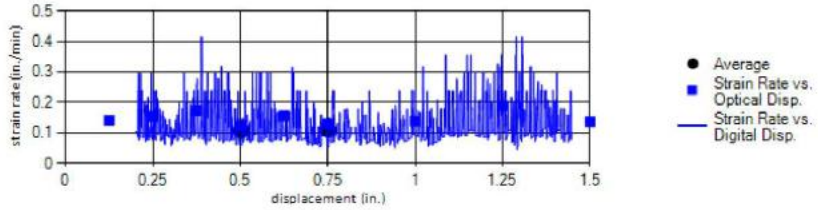
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
711 ±4	742 ±7	No Data	667 ±3	785 ±18	726 ±44	682 ±4	662 ±6	686 ±3



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.11 ±0.03	0.15 ±0.04	
0.75	0.11 ±0.03	0.14 ±0.04	0.15 ±0.06
At Peak Load	0.16 ±0.06	0.18 ±0.07	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 21, 2011
Test Identification:	TS29-S10-Z12-L8-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement	
Type:	Ribbed Strip
Length, L_e (ft):	8
Width, b (in.):	2
Skew Angle, β (°):	0

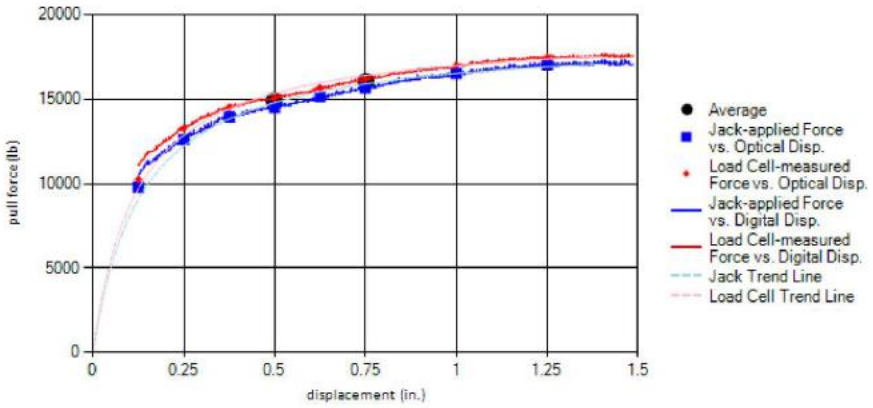
Backfill Material

Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	121.7 ±3.3
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	6.5 ±0.8%
		Relative Compaction, C_R (%):	97.8 ±2.7%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	1,540 ±3	14,914 ±21	12.1 ±0.0	**3.63 ±0.01**
0.75	0.75 ±0.00	1,529 ±2	16,064 ±20	12.0 ±0.0	**3.94 ±0.01**
At Peak Load	1.40 ±0.00	1,537 ±7	17,563 ±33	No Data	**No Data**

Load-Displacement Curve



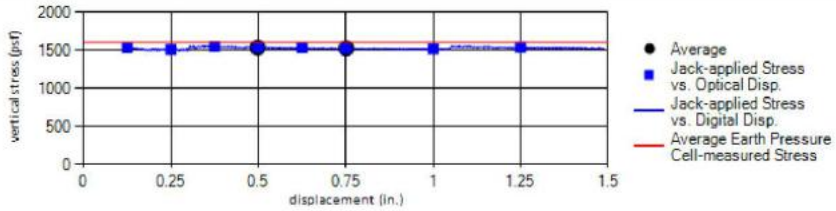
Comments	Personnel	
Strain gage data collected. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, YW, DH
	Prepared:	TW
	Checked:	WL, PJ

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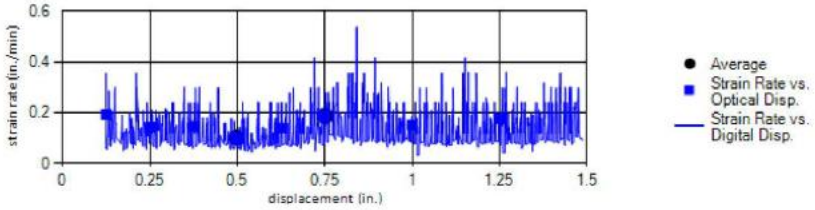
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
1,713 ±6	1,601 ±32	No Data	1,595 ±4	1,512 ±19	1,605 ±74	1,540 ±3	1,529 ±2	1,536 ±7



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.10 ±0.02	0.14 ±0.04	
0.75	0.19 ±0.05	0.13 ±0.04	0.16 ±0.07
At Peak Load	0.17 ±0.05	0.16 ±0.06	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E,	
Sieve Size	*Percent Retained*	*Percent Retained*	TEX-106-E, TEX-107-E)	
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S14-Z12-L8-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	8
Width, b (in.):	2
Skew Angle, β (°):	0

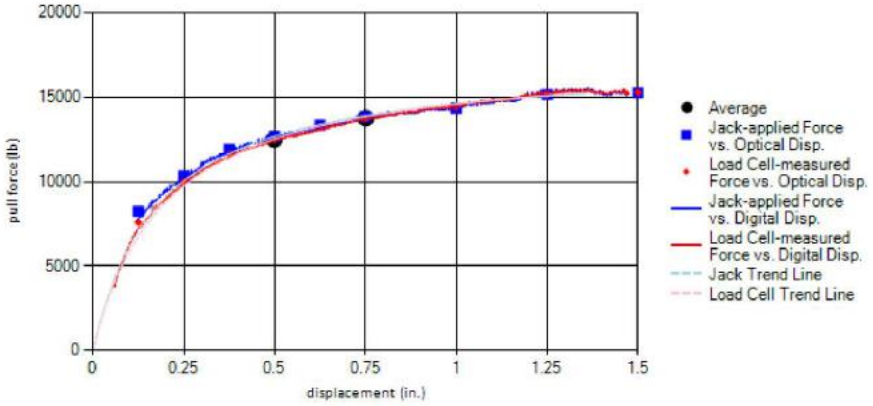
Backfill Material

Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	119.4 ±2.5
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	7.1 ±0.5%
		Relative Compaction, C_R (%):	95.9 ±2.0%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	1,524 ±2	12,494 ±21	12.0 ±0.0	**3.07 ±0.01**
0.75	0.75 ±0.00	1,509 ±3	13,752 ±25	11.8 ±0.0	**3.42 ±0.01**
At Peak Load	1.36 ±0.00	1,488 ±6	15,433 ±33	No Data	**No Data**

Load-Displacement Curve



Comments

Strain gage data collected. No incidental skew data collected. No incidental skew data.

Personnel

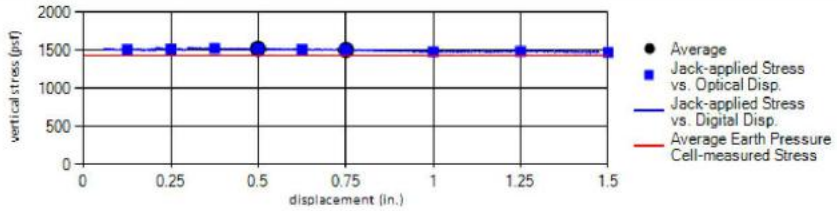
Tested:	TW, RH, AS
Prepared:	TW
Checked:	WL, PJ

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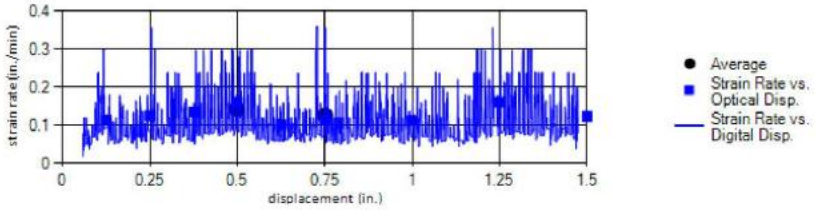
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
1,518 ±8	1,278 ±27	No Data	1,436 ±132	1,510 ±131	1,435 ±134	1,524 ±2	1,509 ±3	1,488 ±6



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.14 ±0.04	0.14 ±0.04	
0.75	0.13 ±0.05	0.12 ±0.04	0.13 ±0.06
At Peak Load	0.17 ±0.06	0.15 ±0.07	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 20, 2011
Test Identification:	TS29-S2-Z12-L12-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement	
Type:	Ribbed Strip
Length, L_e (ft):	12
Width, b (in.):	2
Skew Angle, β (°):	0

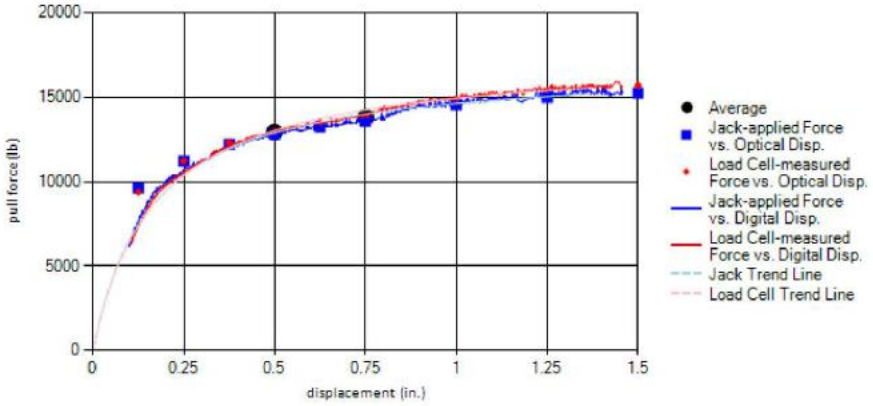
Backfill Material

Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	124.4	±1.5
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	5.8	±0.5%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	99.9	±1.2%

Pullout Test Results

Nominal	Measured				Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****	
0.50	0.50 ±0.00	1,523 ±3	12,933 ±39	11.9 ±0.0	**2.12**	**±0.01**
0.75	0.75 ±0.00	1,491 ±7	13,816 ±41	11.7 ±0.1	**2.32**	**±0.01**
At Peak Load	1.38 ±0.00	1,513 ±15	15,645 ±58	No Data	**No**	**Data**

Load-Displacement Curve



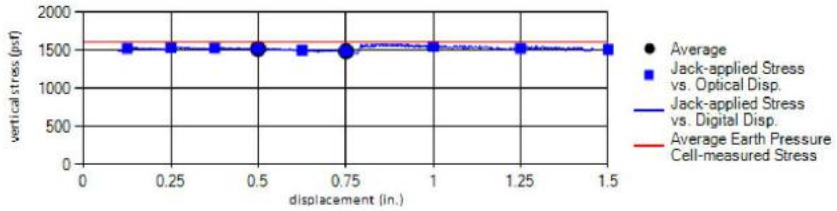
Comments	Personnel	
Strain gage data collected. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, RH, DH
	Prepared:	TW
	Checked:	WL, PJ

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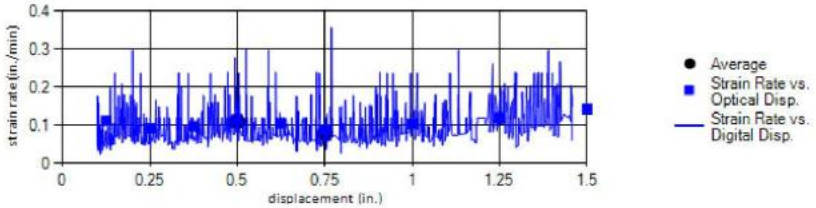
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
1,758 ±56	1,549 ±31	No Data	1,627 ±25	1,512 ±28	1,611 ±102	1,523 ±3	1,491 ±7	1,511 ±15



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.11 ±0.03	0.10 ±0.03	
0.75	0.08 ±0.02	0.10 ±0.03	0.10 ±0.05
At Peak Load	0.12 ±0.04	0.14 ±0.05	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S6-Z12-L12-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	12
Width, b (in.):	2
Skew Angle, β (°):	0

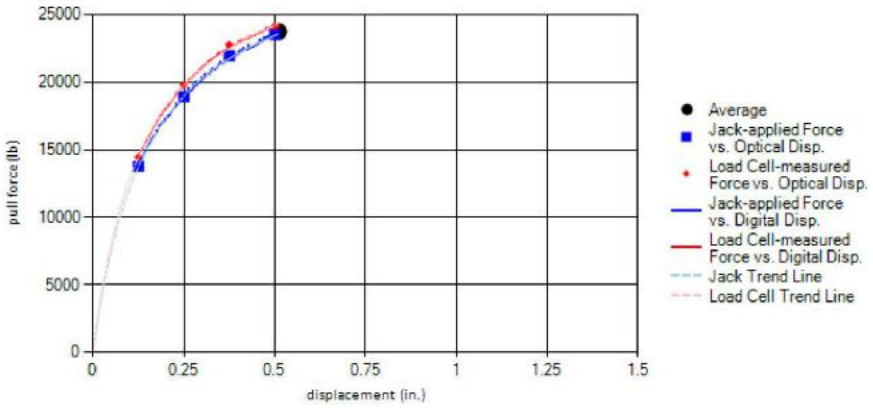
Backfill Material

Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	121.7 ±3.3
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	6.5 ±0.8%
		Relative Compaction, C_R (%):	97.8 ±2.7%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.51 ±0.00	1,538 ±2	23,753 ±26	12.1 ±0.0	**3.86 ±0.01**
At Peak Load	0.51 ±0.00	1,539 ±6	24,051 ±18	12.1 ±0.0	**3.91 ±0.01**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



Comments

Rupture of connection at 0.51in. Strain gage data collected. No incidental skew data collected. No incidental skew data.

Personnel

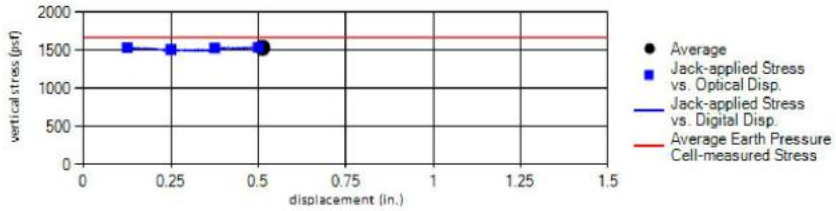
Tested:	TW, RH, DH
Prepared:	TW
Checked:	WL, PJ

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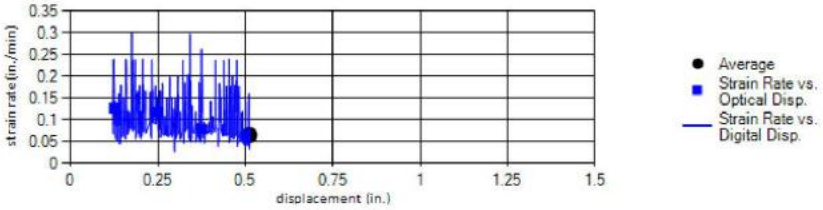
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(Peak)	Jack(0.75)
1,833 ±29	1,587 ±7	No Data	1,666 ±8	1,608 ±4	1,674 ±98	1,538 ±2	1,539 ±6	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.07 ±0.02	0.11 ±0.04	
At Peak Load	0.07 ±0.04	0.11 ±0.05	0.12 ±0.05
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
At Peak Load	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 21, 2011
Test Identification:	TS29-S11-Z20-L8-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement	
Type:	Ribbed Strip
Length, L_e (ft):	8
Width, b (in.):	2
Skew Angle, β (°):	0

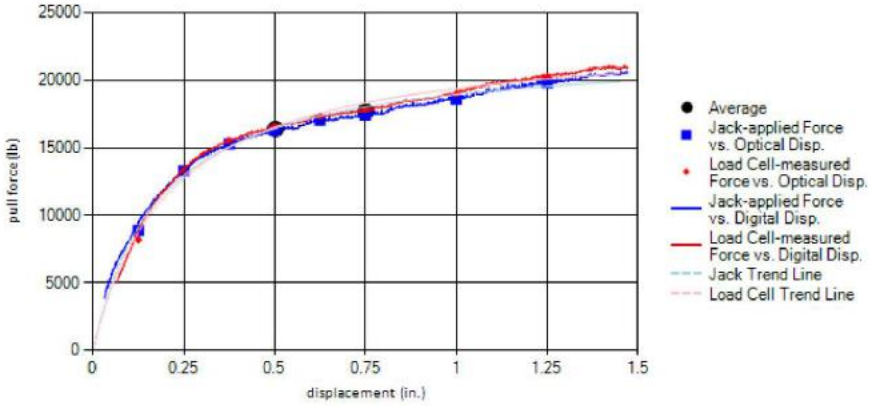
Backfill Material

Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	119.4 ±2.5
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	7.1 ±0.5%
		Relative Compaction, C_R (%):	95.9 ±2.0%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	2,578 ±2	16,396 ±31	20.2 ±0.0	**2.39 ±0.01**
0.75	0.75 ±0.00	2,549 ±3	17,709 ±20	20.0 ±0.0	**2.61 ±0.01**
At Peak Load	1.45 ±0.00	2,548 ±3	20,881 ±22	No Data	**No Data**

Load-Displacement Curve



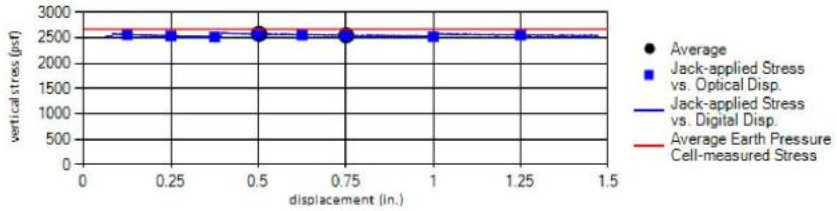
Comments	Personnel	
Strain gage data collected. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, RH, DH
	Prepared:	TW
	Checked:	WL, PJ

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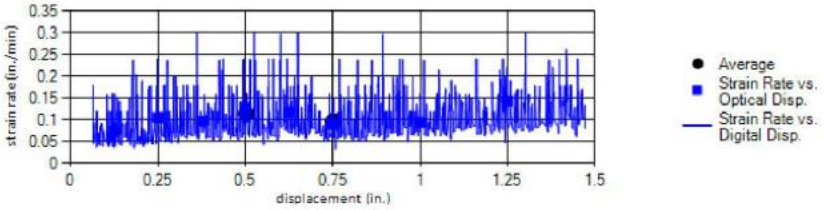
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
2,888 ±147	2,454 ±62	No Data	2,672 ±45	2,676 ±36	2,672 ±175	2,578 ±2	2,548 ±3	2,548 ±3



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.11 ±0.03	0.11 ±0.03	
0.75	0.10 ±0.02	0.11 ±0.03	0.11 ±0.05
At Peak Load	0.13 ±0.04	0.13 ±0.05	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E,	
Sieve Size	*Percent Retained*	*Percent Retained*	TEX-106-E, TEX-107-E)	
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S15-Z20-L8-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	8
Width, b (in.):	2
Skew Angle, β (°):	0

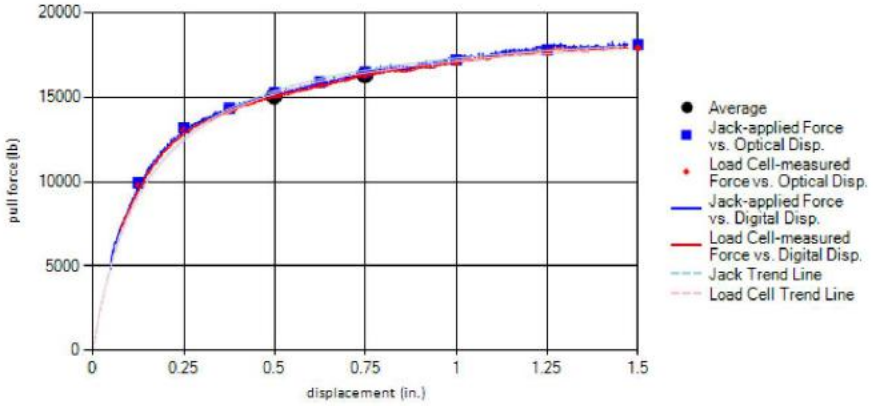
Backfill Material

Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	119.4 ±2.5
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	7.1 ±0.5%
		Relative Compaction, C_R (%):	95.9 ±2.0%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	2,540 ±4	15,061 ±23	19.9 ±0.0	**2.22 ±0.01**
0.75	0.75 ±0.00	2,542 ±3	16,315 ±25	19.9 ±0.0	**2.41 ±0.01**
At Peak Load	1.46 ±0.00	2,544 ±5	18,018 ±30	No Data	**No Data**

Load-Displacement Curve



Comments

Strain gage data collected. No incidental skew data collected. No incidental skew data.

Personnel

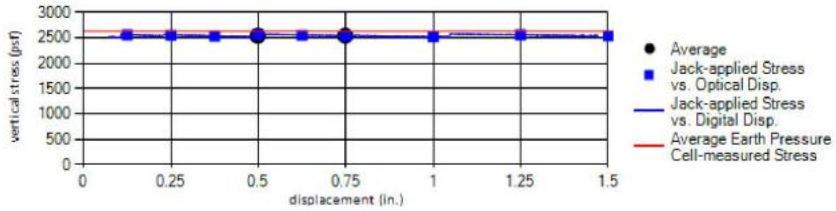
Tested:	TW, RH, AS
Prepared:	TW
Checked:	WL, PJ

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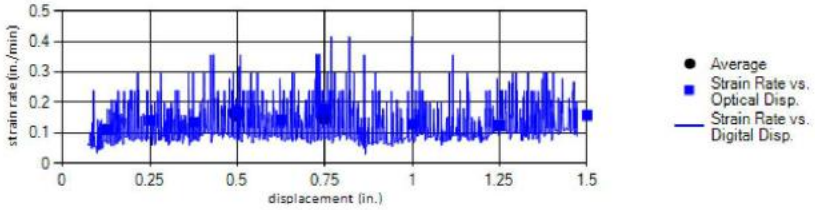
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
2,660 ±6	2,614 ±74	No Data	2,632 ±6	2,627 ±8	2,633 ±41	2,540 ±4	2,541 ±3	2,543 ±5



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.16 ±0.04	0.15 ±0.04	
0.75	0.15 ±0.04	0.16 ±0.04	0.14 ±0.06
At Peak Load	0.16 ±0.06	0.16 ±0.06	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 20, 2011
Test Identification:	TS29-S3-Z20-L12-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



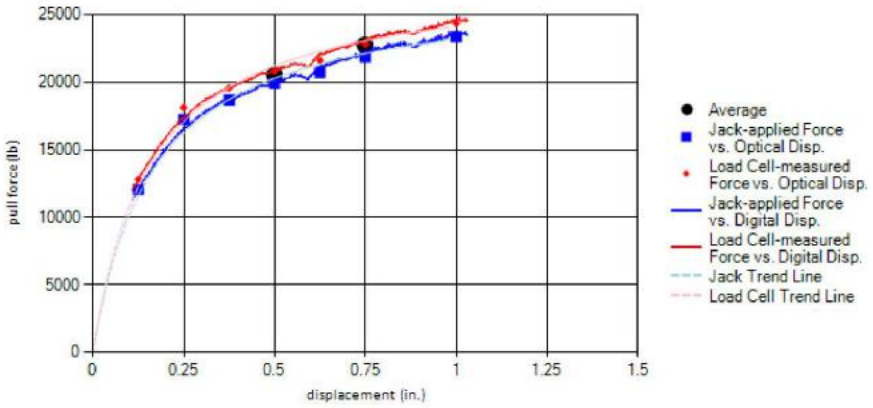
MSE Reinforcement	
Type:	Ribbed Strip
Length, L_e (ft):	12
Width, b (in.):	2
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	124.4	±1.5
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	5.8	±0.5%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	99.9	±1.2%

Pullout Test Results

Nominal	Measured				Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****	
0.50	0.50 ±0.00	2,549 ±4	20,547 ±42	20.0 ±0.0	**2.02**	**±0.01**
0.75	0.75 ±0.00	2,551 ±3	22,766 ±34	20.0 ±0.0	**2.23**	**±0.00**
At Peak Load	0.99 ±0.00	2,533 ±5	24,494 ±58	No Data	**No**	**Data**

Load-Displacement Curve



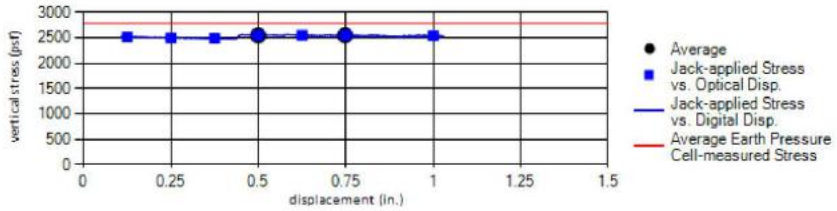
Comments	Personnel	
Rupture of connection at 1.03in. Strain gage data collected. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, RH, AJ
	Prepared:	TW
	Checked:	WL, PJ

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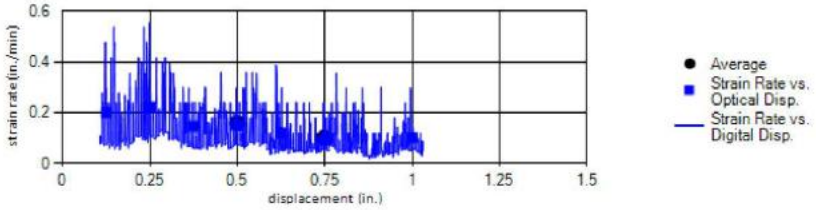
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
2,965 ±57	2,720 ±41	No Data	2,847 ±19	2,613 ±22	2,786 ±138	2,549 ±4	2,551 ±3	2,532 ±5



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.16 ±0.04	0.17 ±0.06	
0.75	0.10 ±0.03	0.13 ±0.05	0.14 ±0.09
At Peak Load	0.10 ±0.05	0.10 ±0.06	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S7-Z20-L12-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	12
Width, b (in.):	2
Skew Angle, β (°):	0

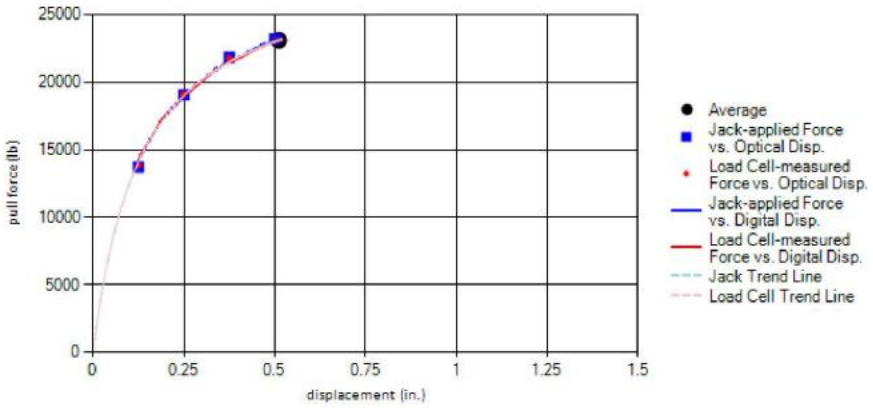
Backfill Material

Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	121.7 ±3.3
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	6.5 ±0.8%
		Relative Compaction, C_R (%):	97.8 ±2.7%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.51 ±0.00	2,575 ±2	23,101 ±18	20.2 ±0.0	**2.24 ±0.00**
At Peak Load	0.52 ±0.00	2,573 ±3	23,160 ±59	20.2 ±0.0	**2.25 ±0.01**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



Comments

Rupture of connection at 0.52in. Strain gage data collected. No incidental skew data collected. No incidental skew data.

Personnel

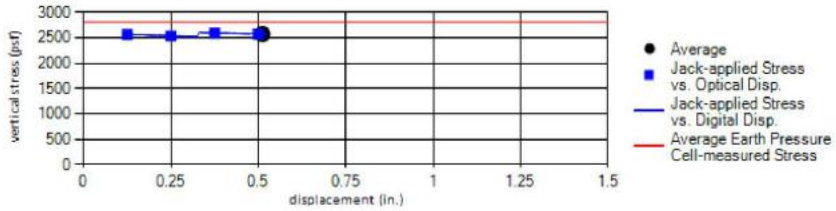
Tested:	TW, YW, DH
Prepared:	TW
Checked:	WL, PJ

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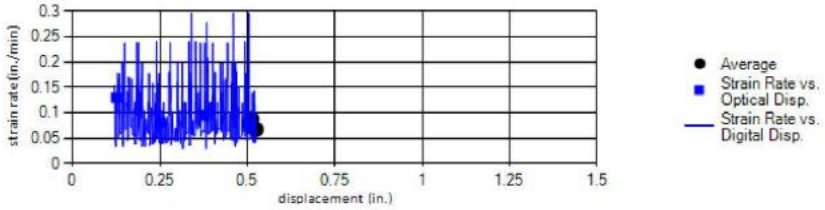
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(Peak)	Jack(0.75)
3,037 ±28	2,657 ±20	No Data	2,841 ±32	2,711 ±19	2,812 ±149	2,575 ±2	2,573 ±3	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.09 ±0.03	0.11 ±0.04	
At Peak Load	0.07 ±0.03	0.11 ±0.06	0.11 ±0.05
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
At Peak Load	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 21, 2011
Test Identification:	TS29-S12-Z40-L8-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement	
Type:	Ribbed Strip
Length, L_e (ft):	8
Width, b (in.):	2
Skew Angle, β (°):	0

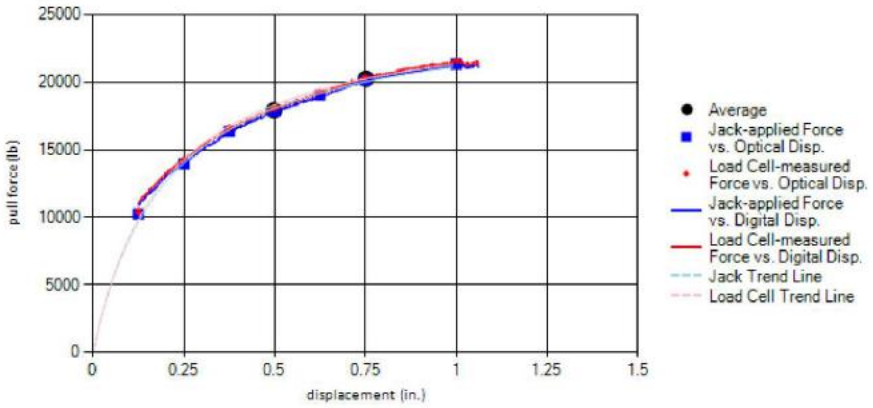
Backfill Material

Specification:	TxDOT Item 423 - Type B	In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)		*Dry Unit Weight, γ_d (pcf):*	119.4	±2.5
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Moisture Content, w (%):*	7.1	±0.5%
Optimum Moisture Content, OMC (%):	7.8%	*Relative Compaction, C_R (%):*	95.9	±2.0%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	5,073 ±5	17,926 ±21	39.8 ±0.0	**1.33 ±0.00**
0.75	0.75 ±0.00	5,100 ±2	20,263 ±30	40.0 ±0.0	**1.49 ±0.00**
At Peak Load	0.99 ±0.00	5,096 ±5	21,537 ±23	No Data	**No Data**

Load-Displacement Curve



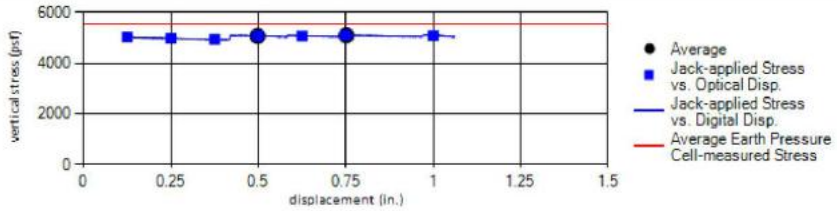
Comments	Personnel	
Rupture of connection at 1.06in. Strain gage data collected. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, RH, DH
	Prepared:	TW
	Checked:	WL, PJ

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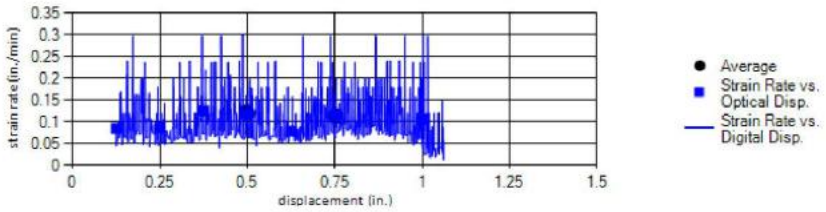
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
5,691 ±60	5,294 ±19	No Data	5,654 ±25	5,540 ±28	5,545 ±159	5,072 ±5	5,100 ±2	5,096 ±5



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.12 ±0.03	0.11 ±0.03	
0.75	0.11 ±0.03	0.11 ±0.03	0.11 ±0.05
At Peak Load	0.13 ±0.06	0.13 ±0.05	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 21, 2011
Test Identification:	TS29-S16-Z40-L8-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



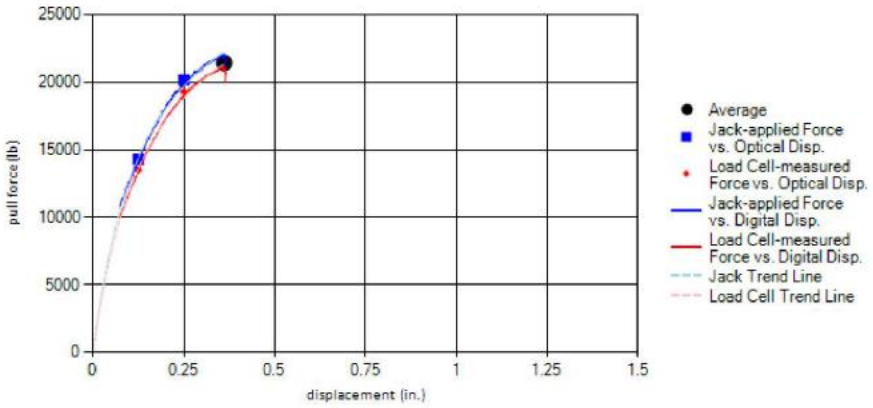
MSE Reinforcement	
Type:	Ribbed Strip
Length, L_e (ft):	8
Width, b (in.):	2
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	119.4	±2.5
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.1	±0.5%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	95.9	±2.0%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
At Peak Load	0.36 ±0.00	5,112 ±7	21,431 ±162	40.1 ±0.1	**1.57 ±0.01**
0.50	No Data	No Data	No Data	No Data	**No Data**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



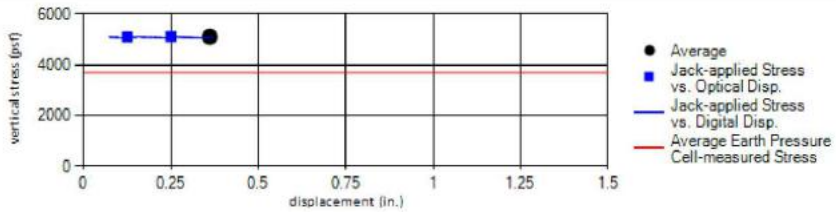
Comments	Personnel	
Rupture of connection at 0.37in. Strain gage data collected. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, RH, AS
	Prepared:	TW
	Checked:	WL, PJ

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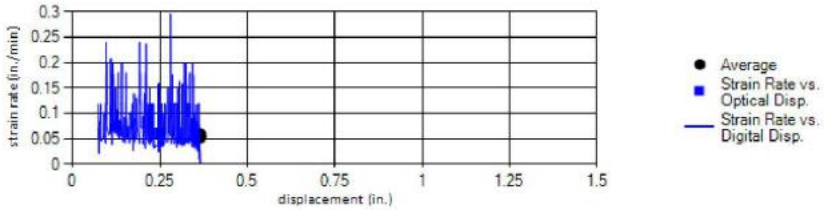
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(Peak)	Jack(0.50)	Jack(0.75)
3,899 ±2,531	3,599 ±2,365	No Data	3,662 ±2,598	3,622 ±2,500	3,695 ±2,476	5,112 ±7	No Data	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
At Peak Load	0.06 ±0.03	0.09 ±0.05	
0.50	No Data	No Data	0.09 ±0.05
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
At Peak Load		
0.50	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information

Test Date:	June 20, 2011
Test Identification:	TS29-S4-Z40-L12-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	12
Width, b (in.):	2
Skew Angle, β (°):	0

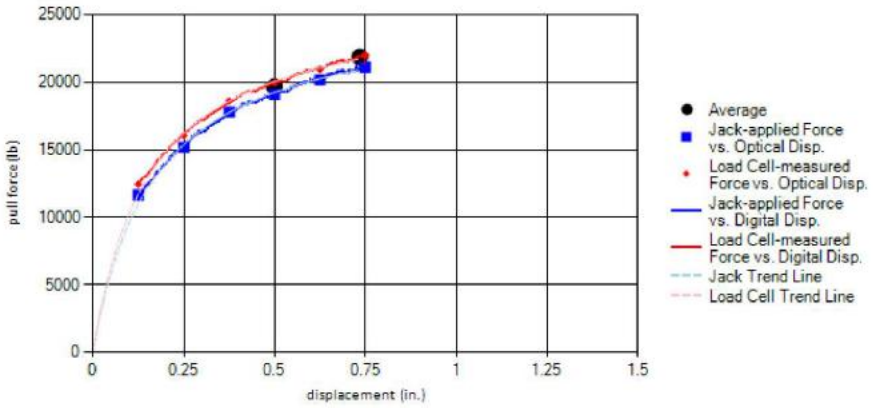
Backfill Material

Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	124.4	±1.5
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	5.8	±0.5%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	99.9	±1.2%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	5,055 ±3	19,665 ±25	39.6 ±0.0	**0.97 ±0.00**
At Peak Load	0.73 ±0.00	5,051 ±7	21,871 ±22	39.6 ±0.1	**1.08 ±0.00**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



Comments

Rupture of connection at 0.75in. Strain gage data collected. No incidental skew data collected. No incidental skew data.

Personnel

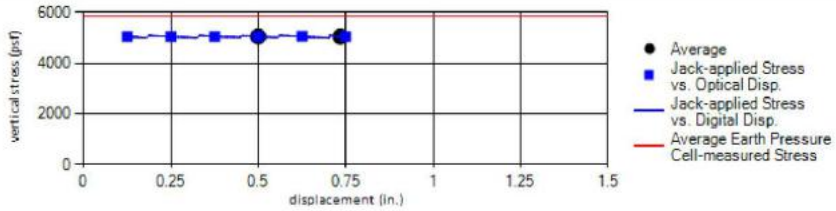
Tested:	TW, RH, AJ
Prepared:	TW
Checked:	WL, PJ

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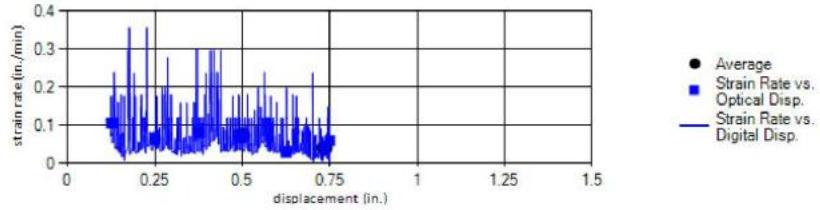
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(Peak)	Jack(0.75)
6,222 ±13	5,687 ±19	No Data	5,962 ±13	5,550 ±30	5,855 ±260	5,055 ±3	5,050 ±7	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.07 ±0.02	0.09 ±0.04	
At Peak Load	0.04 ±0.02	0.07 ±0.04	0.08 ±0.06
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
At Peak Load	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S8-Z40-L12-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	12
Width, b (in.):	2
Skew Angle, β (°):	0

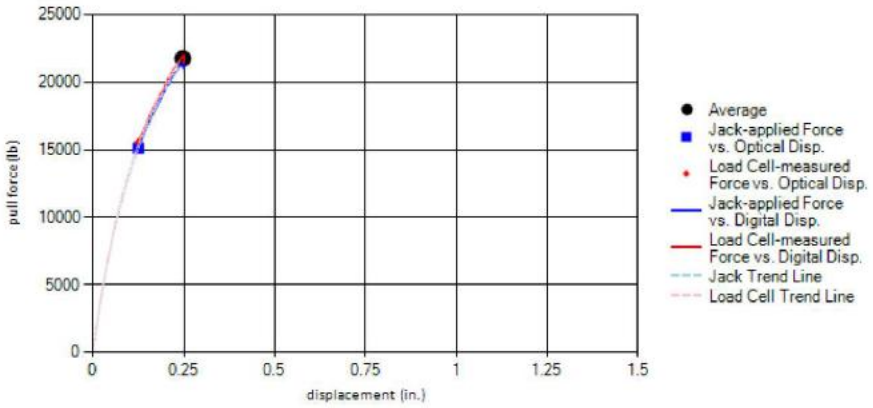
Backfill Material

Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	121.7 ±3.3
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	6.5 ±0.8%
		Relative Compaction, C_R (%):	97.8 ±2.7%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
At Peak Load	0.25 ±0.00	5,051 ±4	21,758 ±41	39.6 ±0.0	**1.08 ±0.00**
0.50	No Data	No Data	No Data	No Data	**No Data**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



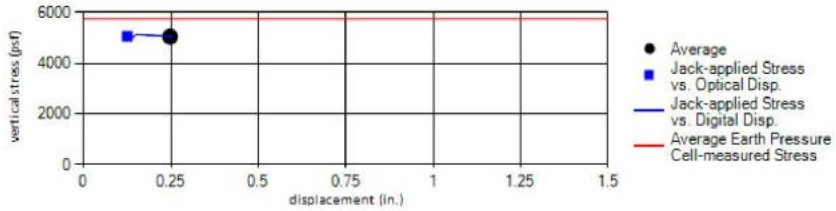
Comments	Personnel	
Rupture of connection at 0.25in. Strain gage data collected. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, YW, DH
	Prepared:	TW
	Checked:	WL, PJ

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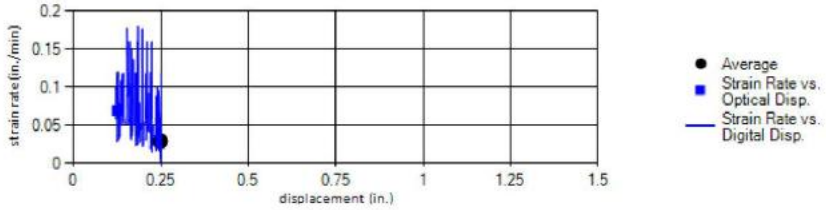
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(Peak)	Jack(0.50)	Jack(0.75)
6,193 ±12	5,423 ±7	No Data	5,832 ±9	5,536 ±11	5,746 ±303	5,050 ±4	No Data	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
At Peak Load	0.03 ±0.02	0.09 ±0.05	
0.50	No Data	No Data	0.08 ±0.05
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
At Peak Load		
0.50	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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Appendix B:

Pullout Data for MSE Reinforcement Pullout Tests:

Reinforced Earth Company Ladders in Type B Backfill

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TABLE B: SUMMARY DATA FOR MSE REINFORCEMENT PULLOUT TESTS
REINFORCED EARTH COMPANY LADDERS IN TYPE B BACKFILL

					0.50in. Displacement					0.75in. Displacement					Peak Displacement				
TestID	Length, Le (ft)	Width, b (in.)	Skew, β (°)	Relative Compaction, CR (%)	Displacement (in.)	Overburden, σv (psf)	DOF, z (ft)	Load, Pr (lb)	F*	Displacement (in.)	Overburden, σv (psf)	DOF, z (ft)	Load, Pr (lb)	F*	Displacement (in.)	Overburden, σv (psf)	DOF, z (ft)	Load, Pr (lb)	F*
TS7-L1-Z3-L4-T	4	4	0	94.4%	0.54	378	3.0	4264	4.23	0.75	381	3.0	4405	4.34	1.00	384		4605	
TS7-L2-Z3-L4-T	4	4	0	94.4%	0.52	378	3.0	3273	3.25	0.76	377	3.0	3484	3.46	0.97	381		3566	
TS7-L3-Z3-L4-T	4	4	0	94.4%	0.52	378	3.0	3938	3.90	0.76	377	3.0	4146	4.12	1.38	376		4388	
TS28-L4-Z3-L4-T	4	4	0	97.6%	0.50	403	3.2	5015	4.66	0.75	409	3.2	5290	4.85	1.48	419		5930	
TS28-L9-Z3-L4-M	4	4	0	97.6%	0.50	387	3.0	7183	6.96	0.75	393	3.1	8095	7.72	1.44	401		8690	
TS28-L16-Z3-L4-T	4	4	0	97.6%	0.50	271	2.1	4383	6.06	0.75	278	2.2	4439	6.00	1.41	293		4740	
TS7-L4-Z3-L8-T	8	4	0	94.4%	0.51	376	2.9	5245	2.62	0.74	376	2.9	5673	2.83	1.34	378		6001	
TS7-L5-Z3-L8-T	8	4	0	94.4%	0.49	391	3.1	6044	2.90	0.74	392	3.1	7158	3.42	1.04	395		7801	
TS7-L6-Z3-L8-M	8	4	0	94.4%	0.50	389	3.0	11336	5.47	0.75	395	3.1	12146	5.77	1.33	403		12700	
TS28-L1-Z3-L8-T	8	4	0	97.6%	0.50	400	3.1	12058	5.65	0.75	409	3.2	13128	6.01	1.29	426		14122	
TS28-L6-Z3-L8-M	8	4	0	97.6%	0.50	409	3.2	15589	7.15	0.75	420	3.3	16800	7.50	1.44	438		17449	
TS28-L18-Z3-L8-T	8	4	0	97.6%	0.50	394	3.1	11031	5.24	0.75	402	3.2	11531	5.38	1.39	415		12372	
TS6-L1-Z5-L4-T	4	4	0	95.2%	0.49	632	5.0	5028	2.98	0.75	637	5.0	5532	3.26	1.28	637		6522	
TS6-L2-Z5-L4-T	4	4	0	95.2%	0.52	606	4.8	4305	2.66	0.73	622	4.9	4338	2.61	0.63	606	4.8	4443	2.75
TS6-L3-Z5-L4-T	4	4	0	95.2%	0.49	637	5.0	5383	3.17	0.76	635	5.0	5717	3.38	0.89	636		5774	
TS6-L7-Z5-L4-M	4	4	0	95.2%	0.50	631	4.9	7142	4.24	0.76	634	5.0	8116	4.80	1.07	636		8683	
TS6-L8-Z5-L4-M	4	4	0	95.2%	0.50	671	5.3	8568	4.79	0.76	672	5.3	9600	5.35	1.24	679		10442	
TS6-L4-Z5-L8-T	8	4	0	95.2%	0.52	631	4.9	6593	1.96	0.75	634	5.0	6698	1.98	0.85	635		6791	
TS6-L5-Z5-L8-T	8	4	0	95.2%	0.49	627	4.9	9737	2.91	0.74	627	4.9	10538	3.15	1.46	632		11107	
TS6-L6-Z5-L8-M	8	4	0	95.2%	0.50	644	5.1	13485	3.92	0.00					0.69	645	5.1	14782	4.29

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					0.50in. Displacement					0.75in. Displacement					Peak Displacement				
TestID	Length, Le (ft)	Width, b (in.)	Skew, β (°)	Relative Compaction, CR (%)	Displacement (in.)	Overburden, σv (psf)	DOF, z (ft)	Load, Pr (lb)	F*	Displacement (in.)	Overburden, σv (psf)	DOF, z (ft)	Load, Pr (lb)	F*	Displacement (in.)	Overburden, σv (psf)	DOF, z (ft)	Load, Pr (lb)	F*
TS7-L13-Z5-L8-B	8	4	0	94.4%	0.44	631	4.9	13963	4.15						0.52	628	4.9	15043	4.49
TS7-L14-Z5-L8-B	8	4	0	94.4%	0.46	634	5.0	13671	4.04						0.58	635	5.0	16254	4.80
TS28-L11-Z5-L8-B	8	4	0	97.6%											0.39	654	5.1	17771	5.09
TS28-L5-Z6-L4-T	4	4	0	97.6%	0.50	760	6.0	6445	3.18	0.75	779	6.1	7162	3.45	1.49	772		8406	
TS28-L10-Z6-L4-M	4	4	0	97.6%	0.50	763	6.0	5803	2.85	0.75	761	6.0	6564	3.23	1.42	758		9934	
TS28-L15-Z6-L4-B	4	4	0	97.6%	0.50	776	6.1	14518	7.02	0.75	777	6.1	16629	8.03	0.95	776		17807	
TS28-L2-Z6-L8-T	8	4	0	97.6%	0.50	771	6.0	12093	2.94	0.75	764	6.0	12434	3.05	1.38	784		13503	
TS28-L7-Z6-L8-M	8	4	0	97.6%	0.50	743	5.8	15438	3.89	0.75	772	6.1	17030	4.14	0.72	776	6.1	17164	4.15
TS28-L8-Z6-L8-M	8	4	0	97.6%	0.50	763	6.0	16177	3.98						0.62	776	6.1	17432	4.21
TS28-L21-Z6-L8-M	8	4	0	97.6%															
TS28-L24-Z6-L8-B	8	4	0	97.6%											0.31	1030	8.1	17293	3.15
TS7-L7-Z9-L4-M	4	4	0	94.4%	0.49	1132	8.9	10388	3.44	0.76	1132	8.9	11503	3.81	1.04	1128		12174	
TS7-L8-Z9-L4-M	4	4	0	94.4%	0.50	1133	8.9	9668	3.20	0.76	1131	8.9	10594	3.51	1.35	1135		10994	
TS7-L9-Z9-L4-M	4	4	0	94.4%	0.51	1135	8.9	9459	3.13	0.75	1135	8.9	9924	3.28	1.18	1131		10942	
TS7-L10-Z9-L8-B	8	4	0	94.4%											0.38	1106	8.7	14601	2.48
TS7-L11-Z9-L8-B	8	4	0	94.4%											0.38	1124	8.8	14875	2.48
TS7-L12-Z9-L8-B	8	4	0	94.4%											0.27	1133	8.9	14814	2.45
TS6-L9-Z12-L4-M	4	4	0	95.2%	0.51	1511	11.9	9328	2.31	0.76	1514	11.9	10313	2.55	1.17	1524		11009	
TS6-L10-Z12-L4-B	4	4	0	95.2%	0.51	1522	11.9	9130	2.25	0.76	1527	12.0	10475	2.57	1.37	1519		12074	
TS6-L11-Z12-L4-B	4	4	0	95.2%	0.50	1527	12.0	10088	2.48	0.75	1523	11.9	11397	2.81	1.13	1520		12332	
TS6-L12-Z12-L4-B	4	4	0	95.2%	0.49	1531	12.0	11881	2.91	0.75	1529	12.0	13564	3.33	1.06	1524		14776	
TS6-L14-Z12-L4-B	4	4	0	95.2%											0.29	1527	12.0	14235	3.50
TS28-L14-Z12-L4-B	4	4	0	97.6%	0.50	1527	12.0	16738	4.11						0.66	1534	12.0	17860	4.37

Table B

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					0.50in. Displacement					0.75in. Displacement					Peak Displacement				
Test ID	Length, Le (ft)	Width, b (in.)	Skew, β (°)	Relative Compaction, CR (%)	Displacement (in.)	Overburden, σv (psf)	DOF, z (ft)	Load, Pr (lb)	F*	Displacement (in.)	Overburden, σv (psf)	DOF, z (ft)	Load, Pr (lb)	F*	Displacement (in.)	Overburden, σv (psf)	DOF, z (ft)	Load, Pr (lb)	F*
TS28-L19-Z12-L4-M	4	4	0	97.6%	0.50	1505	11.8	10296	2.57	0.75	1540	12.1	11212	2.73	1.43	1517		12289	
TS28-L22-Z12-L4-B	4	4	0	97.6%	0.50	1523	11.9	16334	4.02						0.71	1520	11.9	18234	4.50
TS6-L13-Z12-L8-B	8	4	0	95.2%											0.33	1526	12.0	14142	1.74
TS28-L3-Z12-L8-T	8	4	0	97.6%	0.50	1538	12.1	15196	1.85	0.75	1524	12.0	16972	2.09	0.75	1521	11.9	17385	2.14
TS28-L12-Z12-L8-B	8	4	0	97.6%											0.30	1526	12.0	17576	2.16
TS28-L13-Z12-L8-B	8	4	0	97.6%											0.29	1526	12.0	17674	2.17
TS28-L17-Z20-L4-T	4	4	0	97.6%	0.50	2475	19.4	8081	1.22	0.75	2544	20.0	8847	1.30	1.44	2505		9505	
TS28-L20-Z20-L4-M	4	4	0	97.6%	0.50	2577	20.2	12855	1.87	0.75	2552	20.0	14425	2.12	1.43	2575		16119	
TS28-L23-Z20-L4-B	4	4	0	97.6%	0.50	2549	20.0	16406	2.41						0.60	2572	20.2	17370	2.53

Table B

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 08, 2010
Test Identification:	TS7-L1-Z3-L4-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

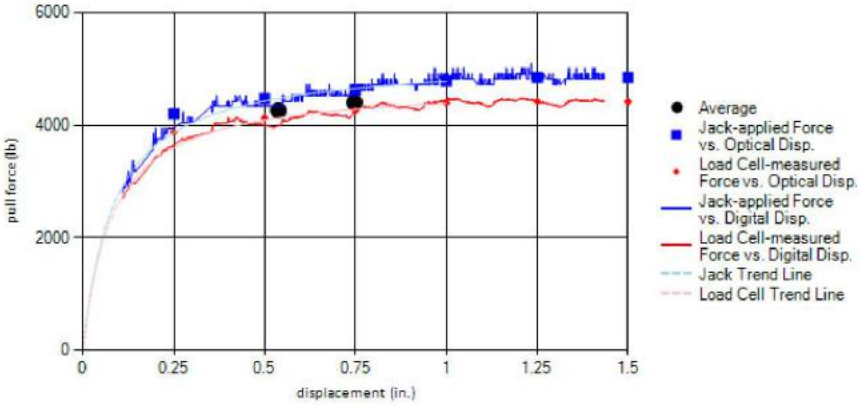
Backfill Material

Specification:	TxDOT Item 423 - Type B	In-Place Density (TEX-115-E)	
Moisture Density Relationship (TEX-114-E)		*Dry Unit Weight, γ_d (pcf):*	117.5 ±1.9
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Moisture Content, w (%):*	7.2 ±0.1%
Optimum Moisture Content, OMC (%):	7.8%	*Relative Compaction, C_R (%):*	94.4 ±1.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.54 ±0.06	378 ±3	4,264 ±43	3.0 ±0.0	**4.23 ±0.06**
0.75	0.75 ±0.06	381 ±3	4,405 ±43	3.0 ±0.0	**4.34 ±0.06**
At Peak Load	1.00 ±0.07	384 ±4	4,605 ±45	No Data	**No Data**

Load-Displacement Curve



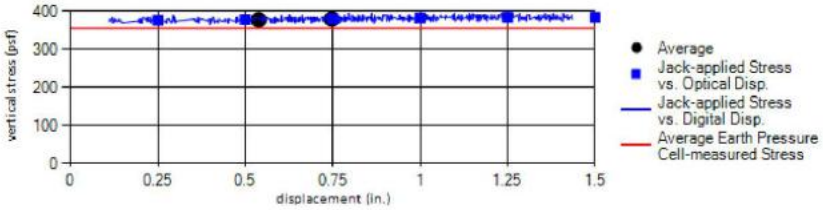
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, DH
	Prepared:	TW
	Checked:	WL, PJ

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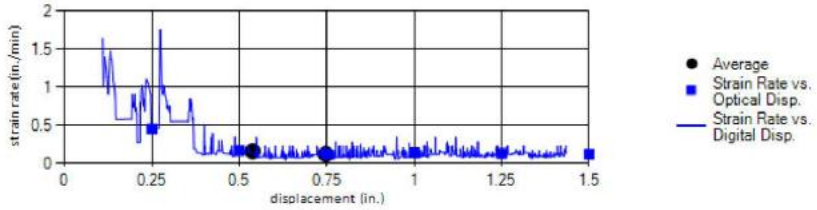
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
414 ±31	349 ±1	316 ±1	353 ±1	351 ±0	357 ±34	378 ±3	381 ±3	384 ±4



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.16 ±0.08	0.38 ±0.23	
0.75	0.12 ±0.03	0.12 ±0.03	0.19 ±0.22
At Peak Load	0.13 ±0.05	0.13 ±0.05	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 08, 2010
Test Identification:	TS7-L2-Z3-L4-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



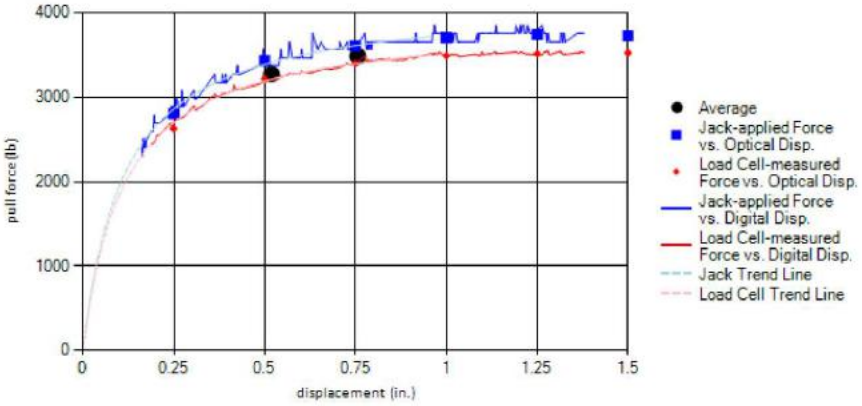
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material			
Specification:	TxDOT Item 423 - Type B	In-Place Density (TEX-115-E)	
Moisture Density Relationship (TEX-114-E)		*Dry Unit Weight, γ_d (pcf):*	117.5 ±1.9
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Moisture Content, w (%):*	7.2 ±0.1%
Optimum Moisture Content, OMC (%):	7.8%	*Relative Compaction, C_R (%):*	94.4 ±1.5%

Pullout Test Results

Nominal	Measured		Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.52 ±0.07	378 ±3	3,273 ±57	3.0 ±0.0	**3.25 ±0.06**
0.75	0.76 ±0.06	377 ±3	3,484 ±34	3.0 ±0.0	**3.46 ±0.04**
At Peak Load	0.97 ±0.07	381 ±4	3,566 ±23	No Data	**No Data**

Load-Displacement Curve



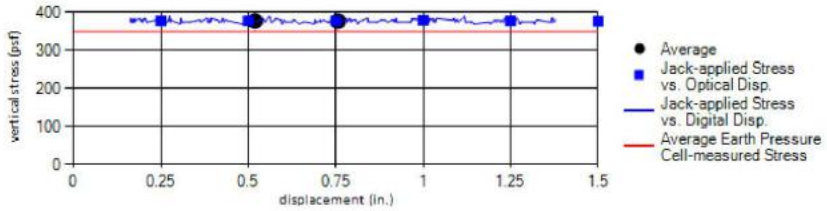
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	AS, MB, KH
	Prepared:	TW
	Checked:	WL, PJ

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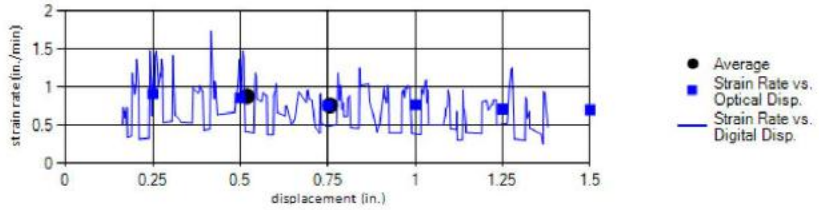
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
381 ±14	352 ±1	312 ±3	355 ±1	351 ±0	350 ±23	378 ±3	378 ±3	382 ±4



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.88 ±0.21	0.97 ±0.23	
0.75	0.76 ±0.15	0.78 ±0.17	0.83 ±0.28
At Peak Load	0.93 ±0.23	0.78 ±0.24	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E,	
Sieve Size	*Percent Retained*	*Percent Retained*	TEX-106-E, TEX-107-E)	
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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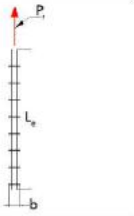
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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information

Test Date:	July 08, 2010
Test Identification:	TS7-L3-Z3-L4-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

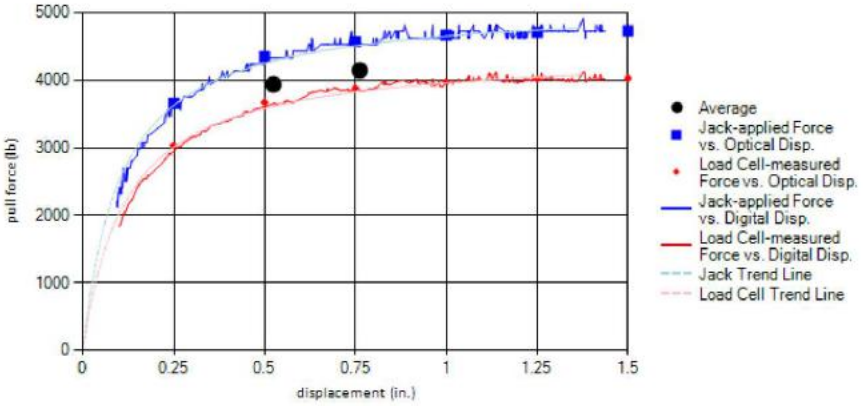
Backfill Material

Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	117.5 ±1.9
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	7.2 ±0.1%
		Relative Compaction, C_R (%):	94.4 ±1.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.52 ±0.06	378 ±3	3,938 ±77	3.0 ±0.0	**3.90 ±0.08**
0.75	0.76 ±0.07	377 ±4	4,146 ±37	3.0 ±0.0	**4.12 ±0.05**
At Peak Load	1.38 ±0.06	376 ±4	4,388 ±25	No Data	**No Data**

Load-Displacement Curve



Comments

No incidental skew data collected. No incidental skew data.

Personnel

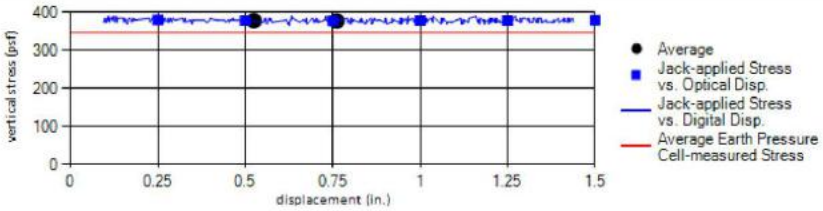
Tested:	AS, MB, KH
Prepared:	TW
Checked:	WL, PJ

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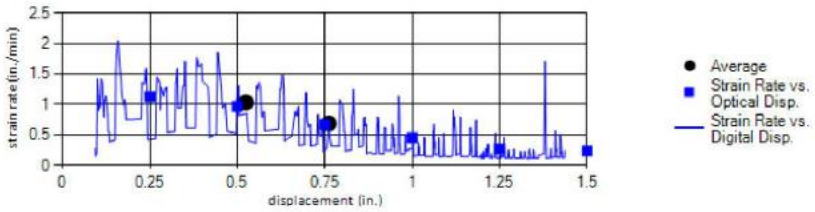
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
362 ±10	353 ±13	311 ±12	358 ±6	352 ±5	347 ±21	379 ±3	378 ±4	377 ±4



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	1.04 ±0.28	1.13 ±0.28	
0.75	0.69 ±0.24	0.84 ±0.26	0.59 ±0.45
At Peak Load	0.92 ±0.22	0.21 ±0.13	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 14, 2011
Test Identification:	TS28-L4-Z3-L4-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



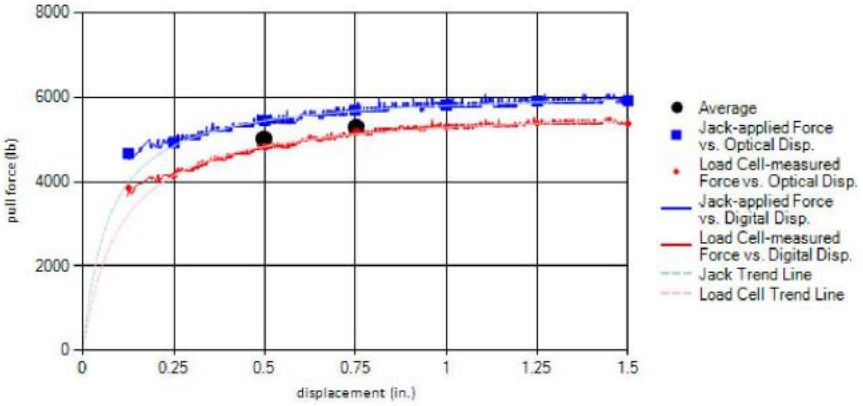
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	121.5	±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.9%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	97.6	±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	403 ±4	5,015 ±31	3.2 ±0.0	**4.66 ±0.06**
0.75	0.75 ±0.00	409 ±3	5,290 ±16	3.2 ±0.0	**4.85 ±0.04**
At Peak Load	1.48 ±0.00	419 ±3	5,930 ±56	No Data	**No Data**

Load-Displacement Curve



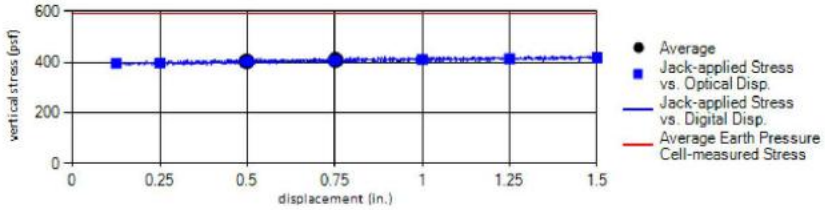
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AJ, RH
	Prepared:	TW
	Checked:	WL, PJ

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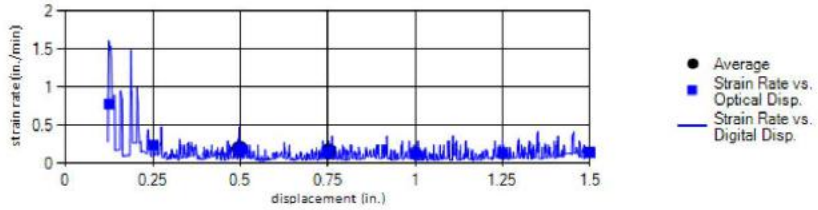
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
467 ±8	793 ±1	No Data	446 ±5	663 ±5	592 ±144	403 ±4	409 ±3	419 ±3



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.19 ±0.09	0.15 ±0.05	
0.75	0.16 ±0.06	0.11 ±0.04	0.19 ±0.29
At Peak Load	0.15 ±0.04	0.16 ±0.08	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E,	
Sieve Size	*Percent Retained*	*Percent Retained*	TEX-106-E, TEX-107-E)	
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 15, 2011
Test Identification:	TS28-L9-Z3-L4-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



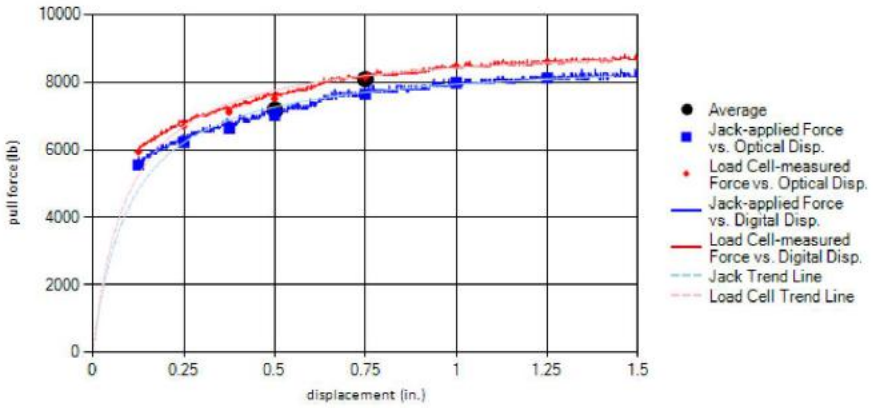
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material			
Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	121.5 ±3.2
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	7.2 ±0.9%
		Relative Compaction, C_R (%):	97.6 ±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	387 ±2	7,183 ±39	3.0 ±0.0	**6.96 ±0.06**
0.75	0.75 ±0.00	393 ±4	8,095 ±23	3.1 ±0.0	**7.72 ±0.09**
At Peak Load	1.44 ±0.00	401 ±6	8,690 ±29	No Data	**No Data**

Load-Displacement Curve



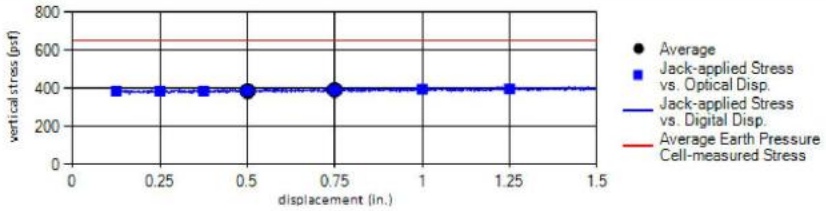
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, YW
	Prepared:	TW
	Checked:	WL, PJ

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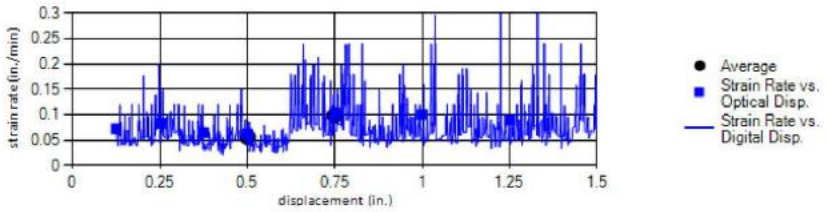
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
573 ±10	833 ±27	No Data	478 ±5	733 ±7	655 ±139	387 ±2	393 ±4	401 ±6



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.06 ±0.01	0.06 ±0.02	
0.75	0.10 ±0.03	0.08 ±0.03	0.09 ±0.11
At Peak Load	0.10 ±0.01	0.08 ±0.04	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E,	
Sieve Size	*Percent Retained*	*Percent Retained*	TEX-106-E, TEX-107-E)	
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 15, 2011
Test Identification:	TS28-L16-Z3-L4-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



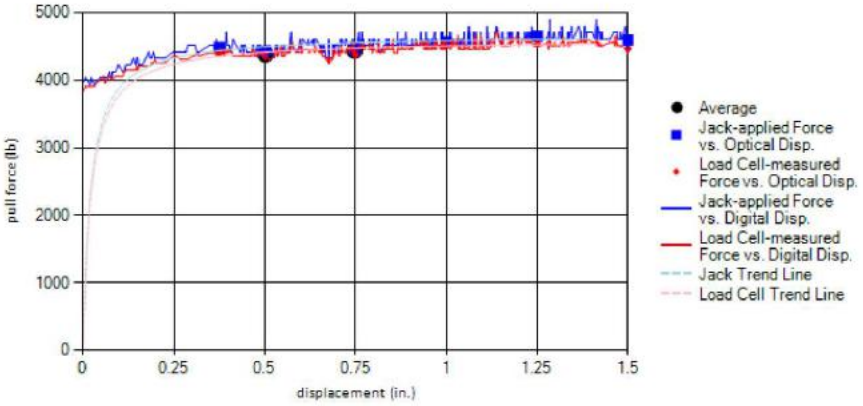
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	121.5	±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.9%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	97.6	±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	271 ±2	4,383 ±26	2.1 ±0.0	**6.06 ±0.07**
0.75	0.75 ±0.01	278 ±3	4,439 ±25	2.2 ±0.0	**6.00 ±0.07**
At Peak Load	1.41 ±0.00	293 ±3	4,740 ±149	No Data	**No Data**

Load-Displacement Curve



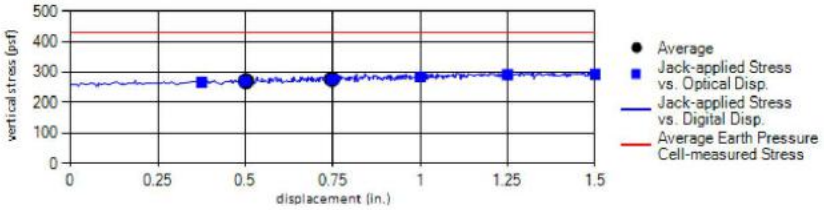
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	TW, RH, AS
	Prepared:	TW
	Checked:	WL, PJ

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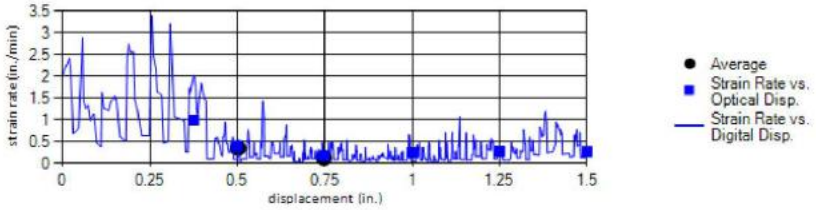
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
343 ±9	626 ±12	No Data	344 ±11	411 ±18	431 ±117	271 ±2	277 ±3	293 ±3



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.35 ±0.15	1.05 ±0.56	
0.75	0.13 ±0.01	0.32 ±0.17	0.42 ±0.49
At Peak Load	0.74 ±0.26	0.39 ±0.24	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 08, 2010
Test Identification:	TS7-L4-Z3-L8-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



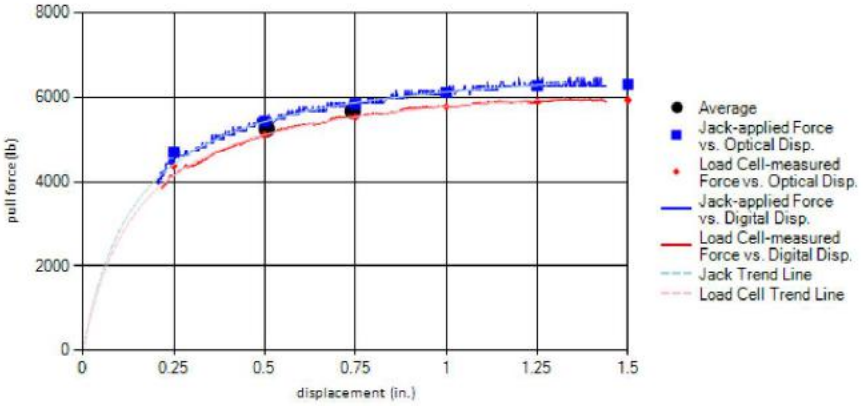
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	117.5	±1.9
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.1%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	94.4	±1.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.51 ±0.06	376 ±3	5,245 ±120	2.9 ±0.0	**2.62 ±0.06**
0.75	0.74 ±0.06	376 ±3	5,673 ±63	2.9 ±0.0	**2.83 ±0.04**
At Peak Load	1.34 ±0.07	378 ±4	6,001 ±21	No Data	**No Data**

Load-Displacement Curve



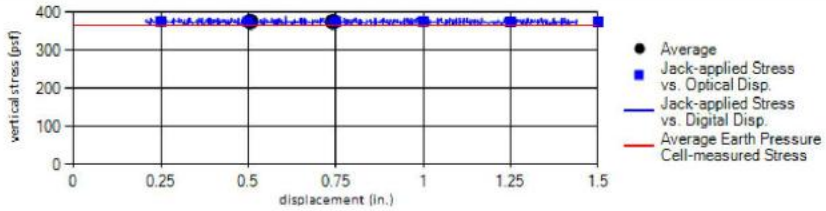
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	AS, MB, KH
	Prepared:	TW
	Checked:	WL, PJ

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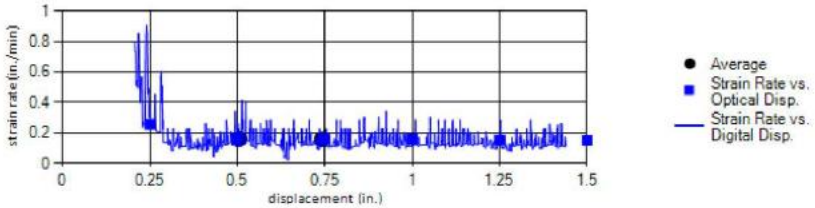
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
354 ±107	413 ±96	336 ±127	365 ±112	361 ±102	366 ±112	376 ±3	376 ±3	378 ±4



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.16 ±0.04	0.17 ±0.06	
0.75	0.16 ±0.04	0.17 ±0.04	0.18 ±0.12
At Peak Load	0.17 ±0.04	0.15 ±0.04	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 08, 2010
Test Identification:	TS7-L5-Z3-L8-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



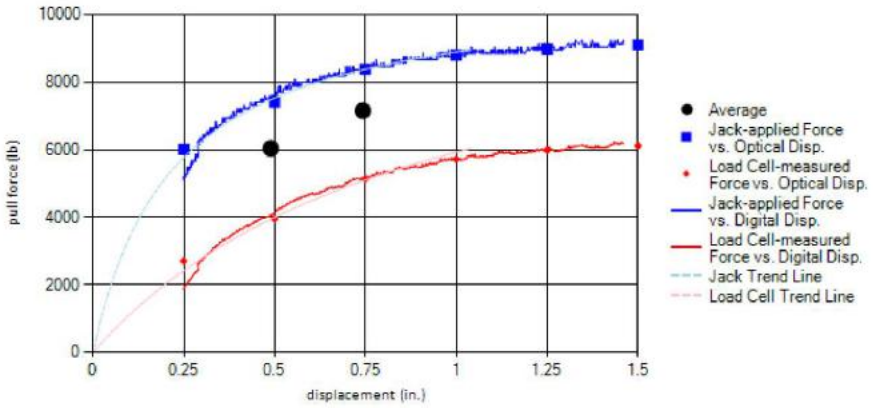
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	117.5	±1.9
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.1%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	94.4	±1.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.49 ±0.06	391 ±3	6,044 ±227	3.1 ±0.0	**2.90 ±0.11**
0.75	0.74 ±0.06	392 ±3	7,158 ±116	3.1 ±0.0	**3.42 ±0.06**
At Peak Load	1.04 ±0.08	395 ±4	7,801 ±75	No Data	**No Data**

Load-Displacement Curve



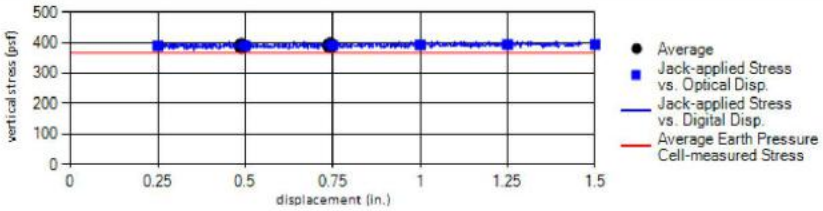
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	AS, MB, KH
	Prepared:	TW
	Checked:	WL, PJ

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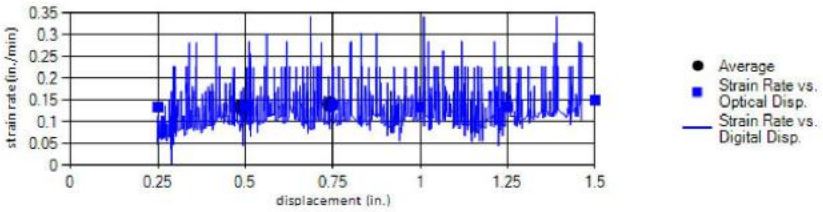
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
358 ±6	444 ±40	328 ±7	364 ±4	343 ±10	367 ±45	391 ±3	392 ±3	395 ±4



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.13 ±0.03	0.12 ±0.03	
0.75	0.14 ±0.03	0.14 ±0.03	0.14 ±0.05
At Peak Load	0.16 ±0.05	0.14 ±0.05	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 08, 2010
Test Identification:	TS7-L6-Z3-L8-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



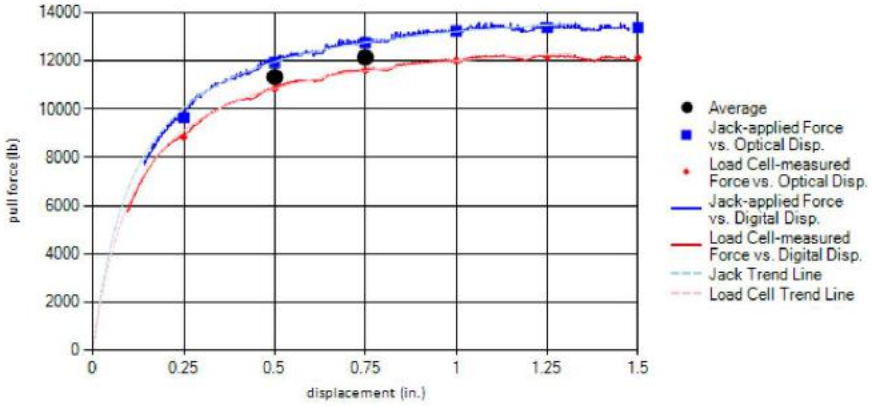
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	117.5	±1.9
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.1%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	94.4	±1.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.06	389 ±3	11,336 ±202	3.0 ±0.0	**5.47 ±0.11**
0.75	0.75 ±0.06	395 ±3	12,146 ±111	3.1 ±0.0	**5.77 ±0.07**
At Peak Load	1.33 ±0.07	403 ±4	12,700 ±46	No Data	**No Data**

Load-Displacement Curve



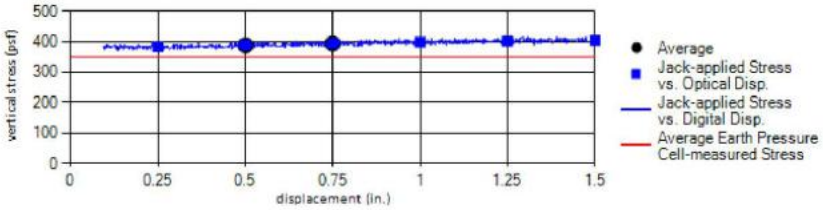
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	AS, MB, KH
	Prepared:	TW
	Checked:	WL, PJ

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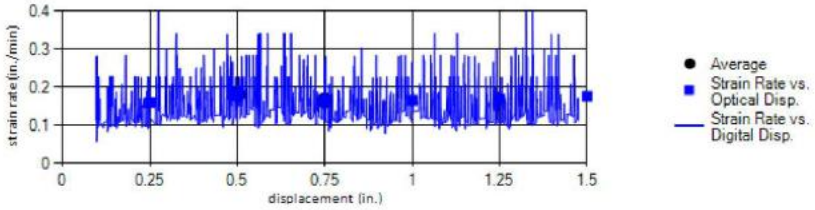
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
296 ±20	434 ±31	328 ±20	328 ±14	373 ±14	352 ±52	389 ±3	395 ±3	403 ±4



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.18 ±0.04	0.17 ±0.04	
0.75	0.17 ±0.04	0.18 ±0.04	0.18 ±0.09
At Peak Load	0.23 ±0.06	0.17 ±0.05	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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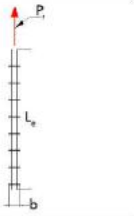
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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information

Test Date:	June 14, 2011
Test Identification:	TS28-L1-Z3-L8-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

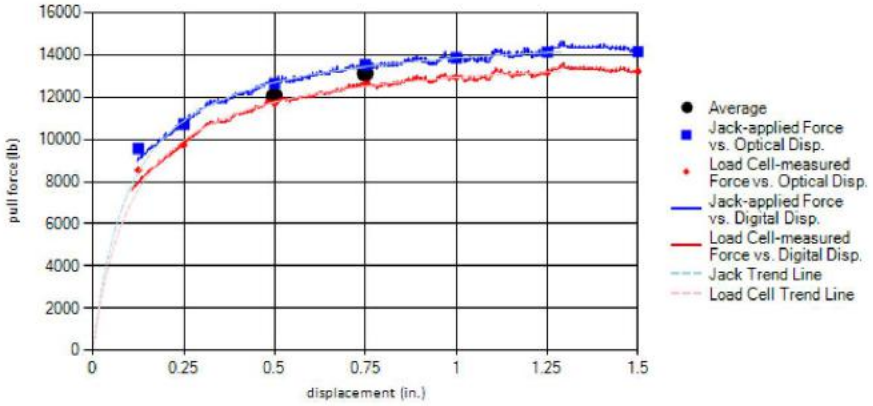
Backfill Material

Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	121.5 ±3.2
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	7.2 ±0.9%
		Relative Compaction, C_R (%):	97.6 ±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	400 ±4	12,058 ±22	3.1 ±0.0	**5.65 ±0.05**
0.75	0.75 ±0.00	409 ±3	13,128 ±28	3.2 ±0.0	**6.01 ±0.04**
At Peak Load	1.29 ±0.00	426 ±5	14,122 ±58	No Data	**No Data**

Load-Displacement Curve



Comments

Strain gage data collected. No incidental skew data collected. No incidental skew data.

Personnel

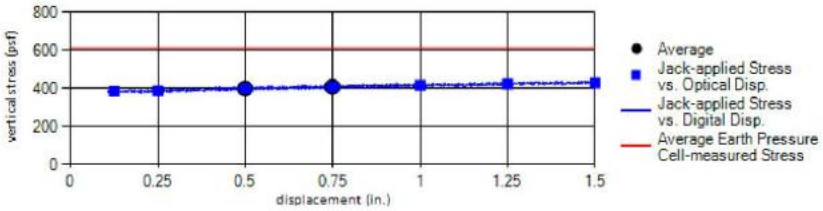
Tested:	TW, AJ, RH
Prepared:	TW
Checked:	WL, PJ

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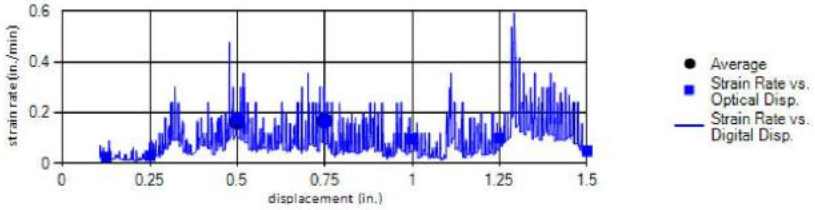
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
593 ±51	773 ±14	No Data	428 ±5	663 ±13	614 ±128	400 ±4	409 ±3	426 ±5



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.16 ±0.04	0.10 ±0.05	
0.75	0.17 ±0.05	0.13 ±0.04	0.11 ±0.08
At Peak Load	0.46 ±0.14	0.10 ±0.08	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 15, 2011
Test Identification:	TS28-L6-Z3-L8-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



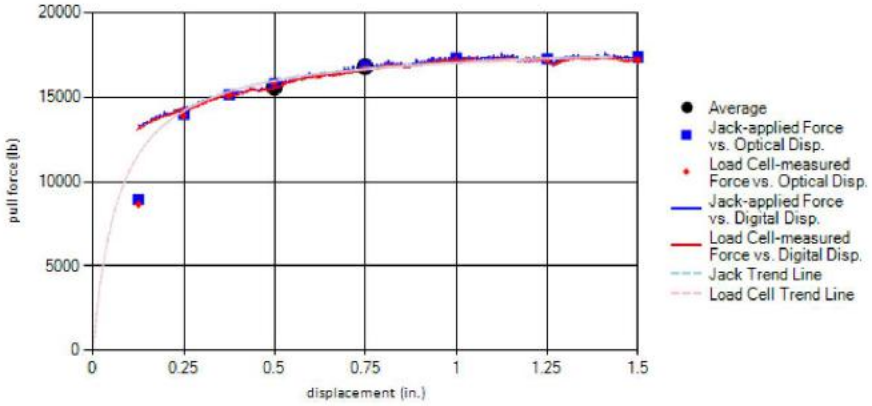
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	121.5	±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.9%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	97.6	±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	409 ±4	15,589 ±25	3.2 ±0.0	**7.15 ±0.06**
0.75	0.75 ±0.00	420 ±3	16,800 ±31	3.3 ±0.0	**7.50 ±0.06**
At Peak Load	1.44 ±0.00	438 ±3	17,449 ±59	No Data	**No Data**

Load-Displacement Curve



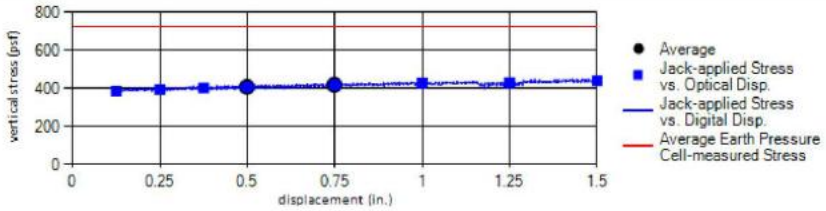
Comments	Personnel	
Strain gage data collected. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, YW
	Prepared:	TW
	Checked:	WL, PJ

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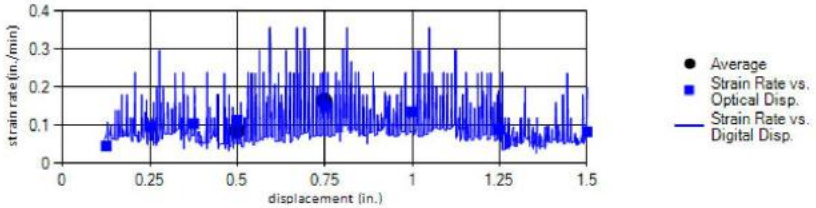
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
738 ±115	923 ±23	No Data	480 ±7	763 ±25	726 ±170	409 ±4	420 ±3	438 ±3



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.09 ±0.02	0.10 ±0.03	
0.75	0.16 ±0.04	0.15 ±0.05	0.11 ±0.06
At Peak Load	0.06 ±0.02	0.09 ±0.04	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E,	
Sieve Size	*Percent Retained*	*Percent Retained*	TEX-106-E, TEX-107-E)	
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 15, 2011
Test Identification:	TS28-L18-Z3-L8-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



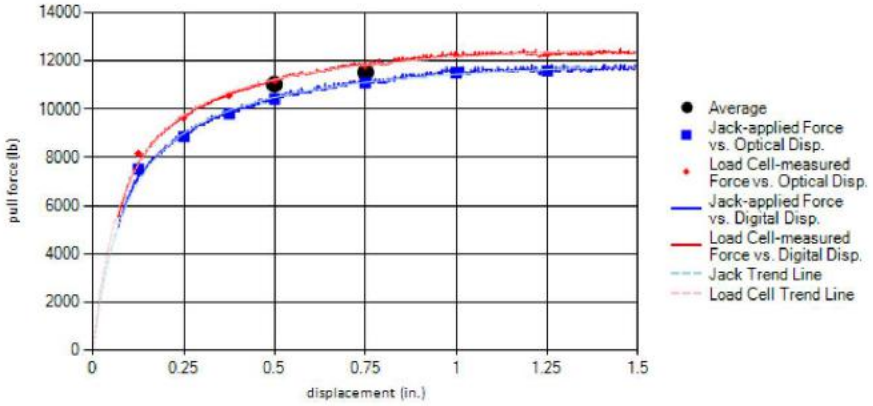
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	121.5	±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.9%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	97.6	±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	394 ±3	11,031 ±20	3.1 ±0.0	**5.24 ±0.05**
0.75	0.75 ±0.00	402 ±2	11,531 ±18	3.2 ±0.0	**5.38 ±0.03**
At Peak Load	1.39 ±0.00	415 ±8	12,372 ±50	No Data	**No Data**

Load-Displacement Curve



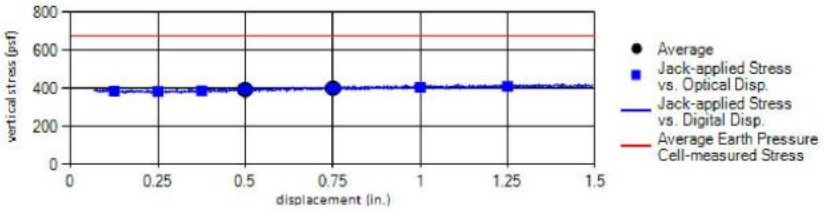
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, RH
	Prepared:	TW
	Checked:	WL, PJ

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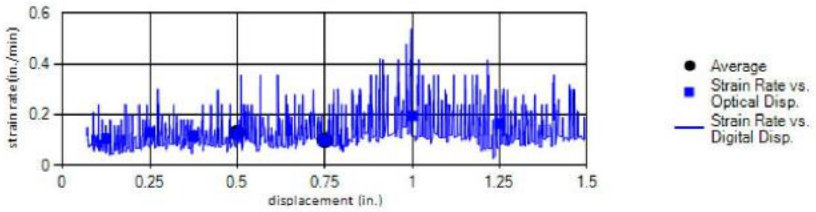
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
576 ±6	819 ±8	No Data	446 ±22	865 ±69	676 ±176	394 ±3	402 ±2	414 ±8



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.13 ±0.02	0.12 ±0.03	
0.75	0.10 ±0.01	0.15 ±0.04	0.16 ±0.08
At Peak Load	0.14 ±0.02	0.17 ±0.07	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 01, 2010
Test Identification:	TS6-L1-Z5-L4-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



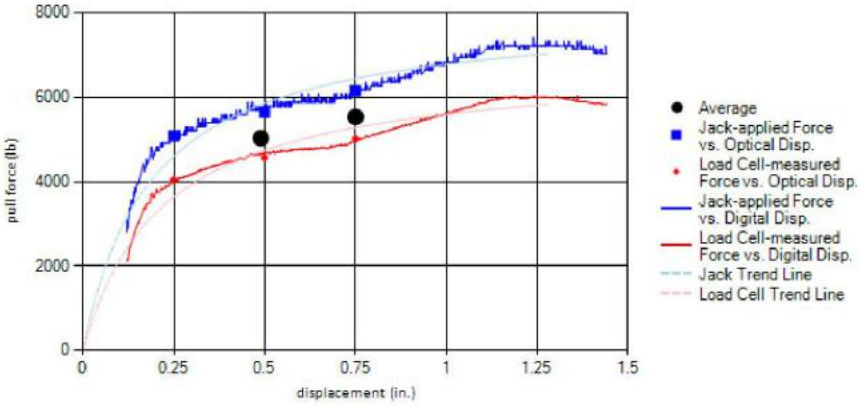
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material			
Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	118.5 ±0.3
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	7.2 ±0.2%
		Relative Compaction, C_R (%):	95.2 ±0.2%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.49 ±0.06	632 ±3	5,028 ±78	5.0 ±0.0	**2.98 ±0.05**
0.75	0.75 ±0.06	637 ±4	5,532 ±105	5.0 ±0.0	**3.26 ±0.07**
At Peak Load	1.28 ±0.08	637 ±4	6,522 ±38	No Data	**No Data**

Load-Displacement Curve



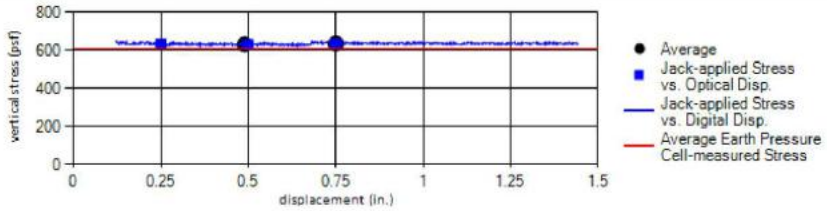
Comments	Personnel	
Rupture of connection at 1.44in. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, DH
	Prepared:	TW
	Checked:	WL, PJ

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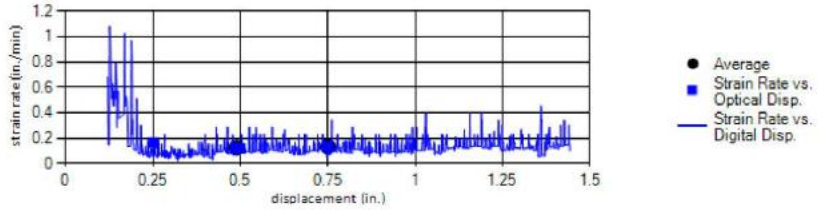
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
514 ±5	677 ±2	654 ±2	563 ±2	644 ±2	610 ±62	633 ±3	637 ±4	637 ±4



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.12 ±0.04	0.10 ±0.03	
0.75	0.13 ±0.03	0.14 ±0.03	0.15 ±0.11
At Peak Load	0.17 ±0.07	0.16 ±0.06	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E,	
Sieve Size	*Percent Retained*	*Percent Retained*	TEX-106-E, TEX-107-E)	
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 01, 2010
Test Identification:	TS6-L2-Z5-L4-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

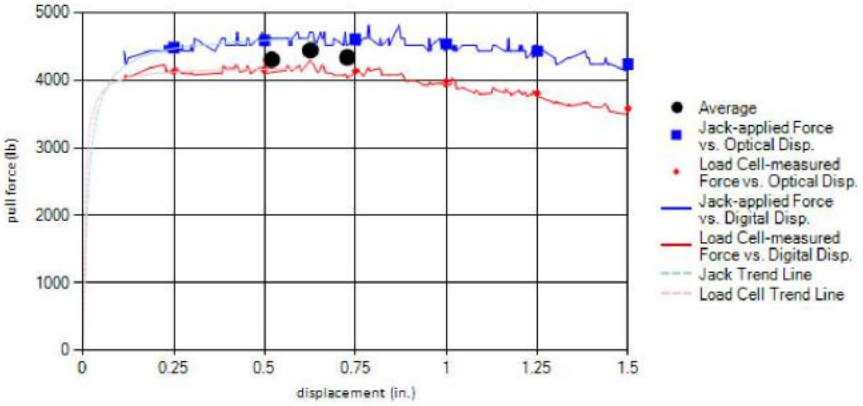
Backfill Material

Specification:	TxDOT Item 423 - Type B	In-Place Density (TEX-115-E)	
Moisture Density Relationship (TEX-114-E)		*Dry Unit Weight, γ_d (pcf):*	118.5 ±0.3
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Moisture Content, w (%):*	7.2 ±0.2%
Optimum Moisture Content, OMC (%):	7.8%	*Relative Compaction, C_R (%):*	95.2 ±0.2%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.52 ±0.06	606 ±4	4,305 ±28	4.8 ±0.0	**2.66 ±0.02**
At Peak Load	0.63 ±0.07	606 ±11	4,443 ±44	4.8 ±0.1	**2.75 ±0.06**
0.75	0.73 ±0.07	622 ±11	4,338 ±37	4.9 ±0.1	**2.61 ±0.05**

Load-Displacement Curve



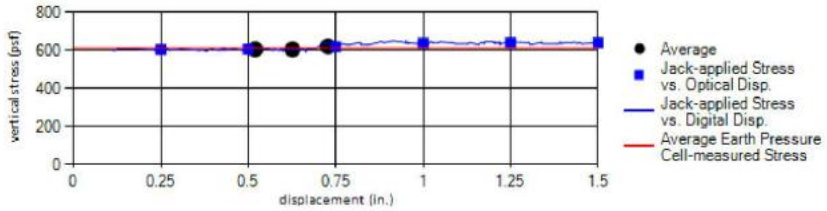
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, DH
	Prepared:	TW
	Checked:	WL, PJ

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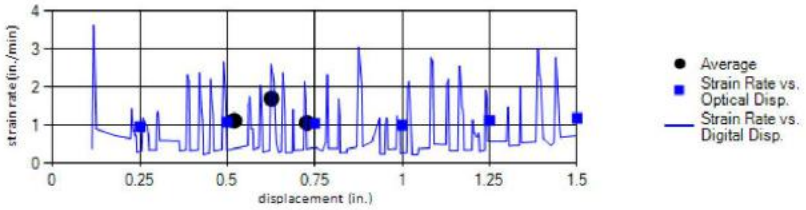
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(Peak)	Jack(0.75)
559 ±4	669 ±13	642 ±9	563 ±6	640 ±8	615 ±46	606 ±4	606 ±12	623 ±12



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	1.12 ±0.56	1.07 ±0.54	
At Peak Load	1.71 ±0.77	1.11 ±0.75	1.06 ±0.77
0.75	1.07 ±0.59	1.09 ±0.53	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
At Peak Load	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 01, 2010
Test Identification:	TS6-L3-Z5-L4-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

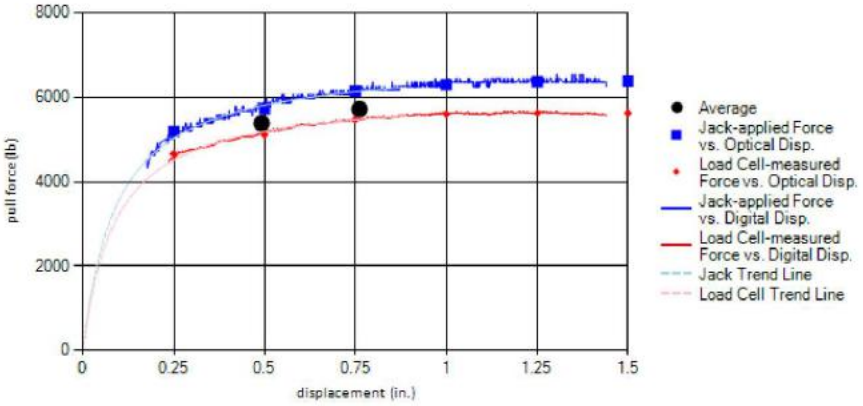
Backfill Material

Specification:	TxDOT Item 423 - Type B	In-Place Density (TEX-115-E)	
Moisture Density Relationship (TEX-114-E)		*Dry Unit Weight, γ_d (pcf):*	118.5 ±0.3
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Moisture Content, w (%):*	7.2 ±0.2%
Optimum Moisture Content, OMC (%):	7.8%	*Relative Compaction, C_R (%):*	95.2 ±0.2%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.49 ±0.06	637 ±4	5,383 ±89	5.0 ±0.0	**3.17 ±0.06**
0.75	0.76 ±0.06	635 ±3	5,717 ±41	5.0 ±0.0	**3.38 ±0.03**
At Peak Load	0.89 ±0.08	636 ±4	5,774 ±39	No Data	**No Data**

Load-Displacement Curve



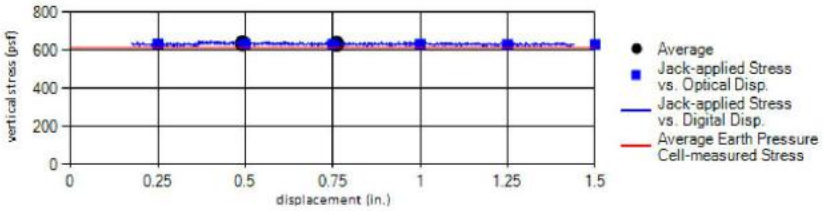
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, DH
	Prepared:	TW
	Checked:	WL, PJ

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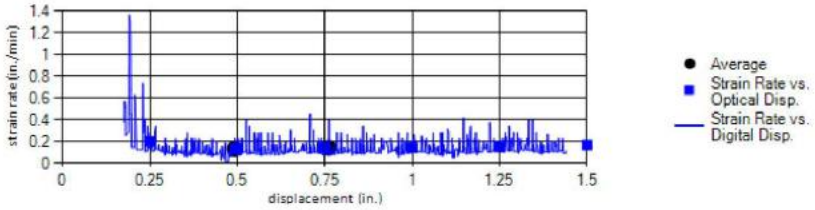
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
569 ±9	684 ±15	626 ±8	562 ±3	639 ±3	616 ±46	638 ±4	635 ±3	636 ±4



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.13 ±0.04	0.13 ±0.04	
0.75	0.14 ±0.04	0.15 ±0.04	0.16 ±0.12
At Peak Load	0.15 ±0.05	0.14 ±0.06	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information

Test Date:	July 02, 2010
Test Identification:	TS6-L7-Z5-L4-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

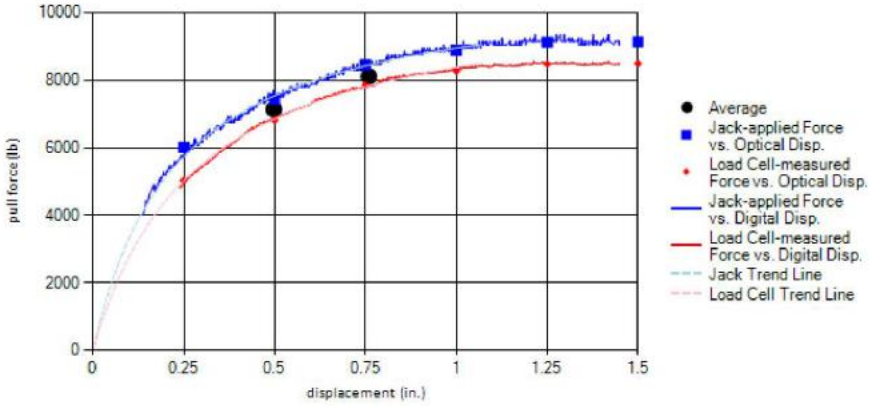
Backfill Material

Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	118.5 ±0.3
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	7.2 ±0.2%
		Relative Compaction, C_R (%):	95.2 ±0.2%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.06	631 ±3	7,142 ±226	4.9 ±0.0	**4.24 ±0.14**
0.75	0.76 ±0.06	634 ±3	8,116 ±129	5.0 ±0.0	**4.80 ±0.08**
At Peak Load	1.07 ±0.08	636 ±4	8,683 ±59	No Data	**No Data**

Load-Displacement Curve



Comments

No incidental skew data collected. No incidental skew data.

Personnel

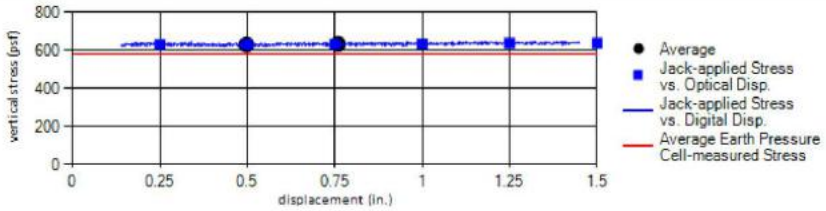
Tested:	TW, AS, DH
Prepared:	TW
Checked:	WL, PJ

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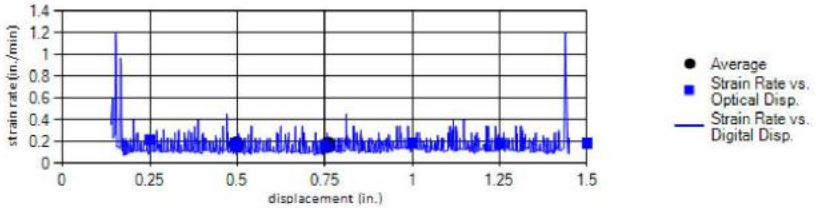
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
718 ±10	568 ±21	489 ±26	575 ±14	563 ±14	583 ±77	631 ±3	634 ±3	636 ±4



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.17 ±0.05	0.18 ±0.05	
0.75	0.17 ±0.04	0.16 ±0.04	0.19 ±0.11
At Peak Load	0.19 ±0.06	0.18 ±0.05	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 02, 2010
Test Identification:	TS6-L8-Z5-L4-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



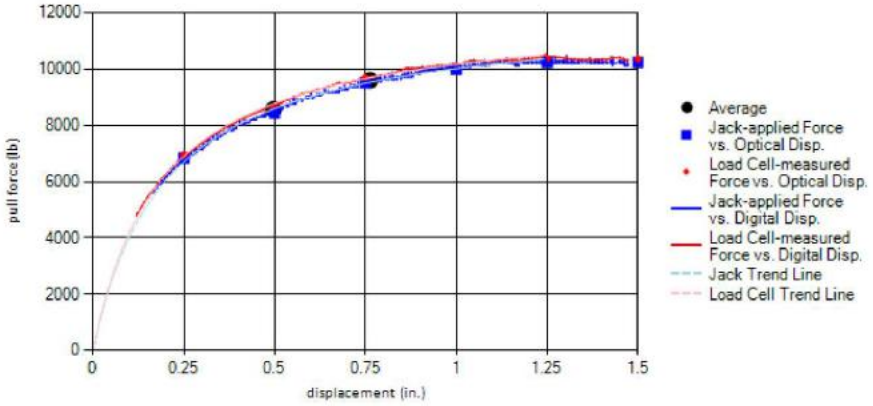
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material			
Specification:	TxDOT Item 423 - Type B	In-Place Density (TEX-115-E)	
Moisture Density Relationship (TEX-114-E)		*Dry Unit Weight, γ_d (pcf):*	118.5 ±0.3
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Moisture Content, w (%):*	7.2 ±0.2%
Optimum Moisture Content, OMC (%):	7.8%	*Relative Compaction, C_R (%):*	95.2 ±0.2%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.06	671 ±3	8,568 ±228	5.3 ±0.0	**4.79 ±0.13**
0.75	0.76 ±0.06	672 ±3	9,600 ±130	5.3 ±0.0	**5.35 ±0.08**
At Peak Load	1.24 ±0.08	679 ±4	10,442 ±39	No Data	**No Data**

Load-Displacement Curve



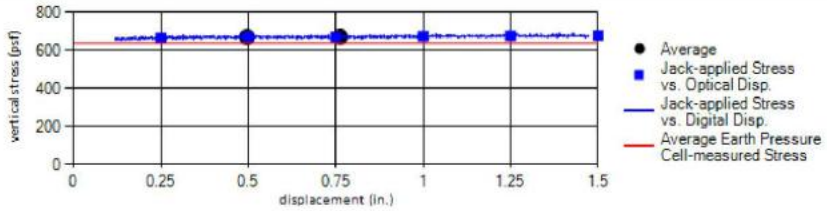
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, DH
	Prepared:	TW
	Checked:	WL, PJ

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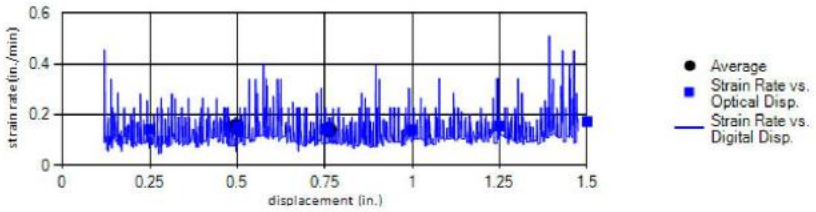
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
758 ±26	633 ±7	540 ±12	611 ±3	644 ±23	637 ±73	671 ±3	672 ±3	679 ±4



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.15 ±0.04	0.14 ±0.03	
0.75	0.14 ±0.04	0.16 ±0.04	0.16 ±0.10
At Peak Load	0.28 ±0.06	0.15 ±0.06	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E,	
Sieve Size	*Percent Retained*	*Percent Retained*	TEX-106-E, TEX-107-E)	
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 01, 2010
Test Identification:	TS6-L4-Z5-L8-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



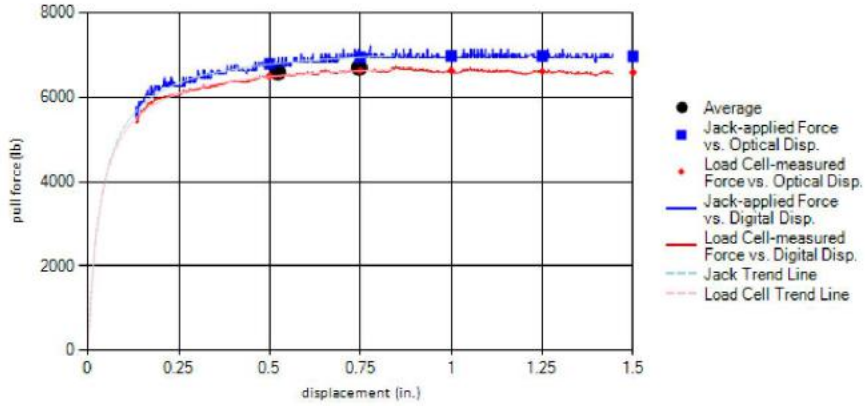
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material			
Specification:	TxDOT Item 423 - Type B	In-Place Density (TEX-115-E)	
Moisture Density Relationship (TEX-114-E)		*Dry Unit Weight, γ_d (pcf):*	118.5 ±0.3
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Moisture Content, w (%):*	7.2 ±0.2%
Optimum Moisture Content, OMC (%):	7.8%	*Relative Compaction, C_R (%):*	95.2 ±0.2%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.52 ±0.06	631 ±3	6,593 ±46	4.9 ±0.0	**1.96 ±0.02**
0.75	0.75 ±0.06	634 ±4	6,698 ±25	5.0 ±0.0	**1.98 ±0.01**
At Peak Load	0.85 ±0.08	635 ±5	6,791 ±26	No Data	**No Data**

Load-Displacement Curve



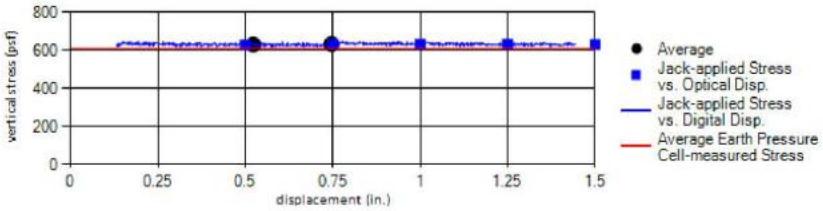
Comments	Personnel	
One pump stroke before recording started. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, DH
	Prepared:	TW
	Checked:	WL, PJ

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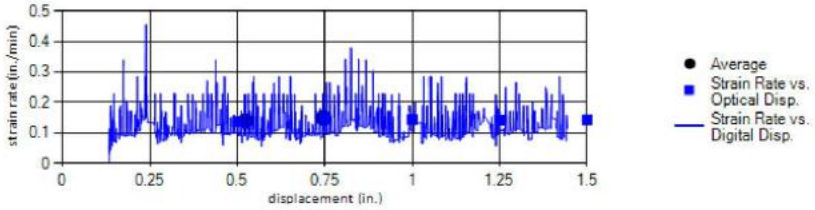
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
548 ±4	673 ±19	654 ±4	563 ±3	625 ±5	613 ±50	631 ±3	634 ±4	635 ±5



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.14 ±0.04	0.14 ±0.03	
0.75	0.15 ±0.04	0.13 ±0.03	0.14 ±0.05
At Peak Load	0.19 ±0.06	0.15 ±0.06	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 01, 2010
Test Identification:	TS6-L5-Z5-L8-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



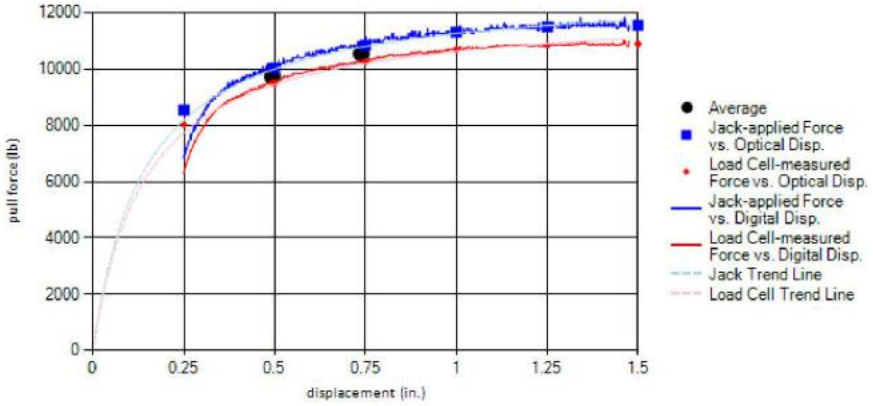
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	118.5	±0.3
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.2%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	95.2	±0.2%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.49 ±0.06	627 ±4	9,737 ±219	4.9 ±0.0	**2.91 ±0.07**
0.75	0.74 ±0.06	627 ±3	10,538 ±104	4.9 ±0.0	**3.15 ±0.03**
At Peak Load	1.46 ±0.05	632 ±4	11,107 ±33	No Data	**No Data**

Load-Displacement Curve



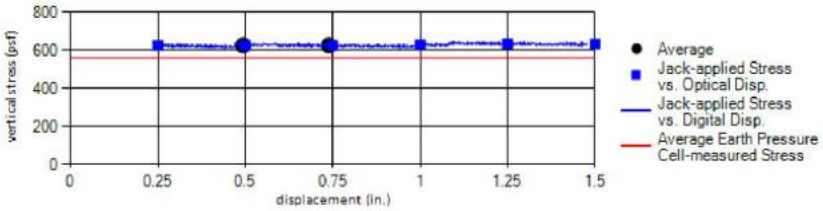
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, DH
	Prepared:	TW
	Checked:	WL, PJ

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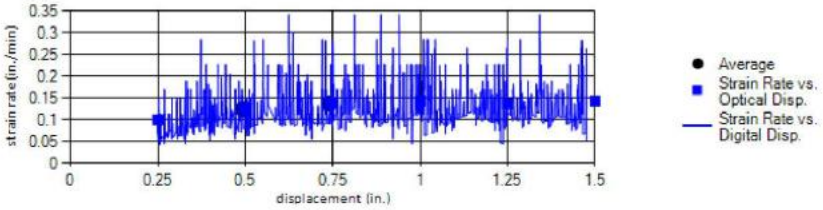
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
488 ±118	614 ±140	605 ±148	519 ±111	582 ±112	562 ±136	627 ±4	627 ±3	633 ±4



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.12 ±0.03	0.11 ±0.03	
0.75	0.14 ±0.03	0.13 ±0.03	0.13 ±0.05
At Peak Load	0.21 ±0.05	0.14 ±0.05	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E,	
Sieve Size	*Percent Retained*	*Percent Retained*	TEX-106-E, TEX-107-E)	
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 02, 2010
Test Identification:	TS6-L6-Z5-L8-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



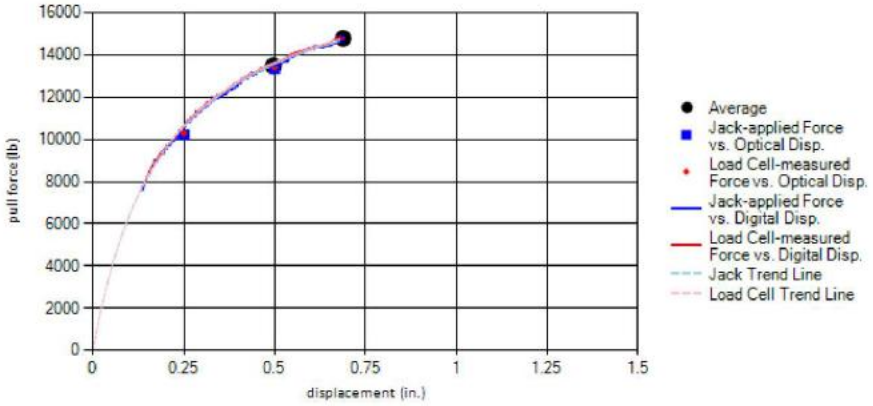
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	118.5	±0.3
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.2%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	95.2	±0.2%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.06	644 ±3	13,485 ±361	5.1 ±0.0	**3.92 ±0.11**
At Peak Load	0.69 ±0.04	645 ±4	14,782 ±164	5.1 ±0.0	**4.29 ±0.06**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



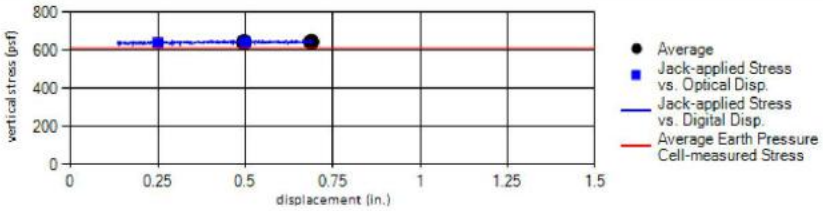
Comments	Personnel	
Rupture in connection at 0.69in. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, DH
	Prepared:	TW
	Checked:	WL, PJ

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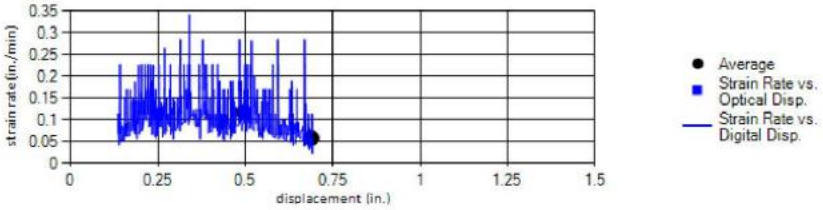
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(Peak)	Jack(0.75)
698 ±14	717 ±65	509 ±10	581 ±1	571 ±2	615 ±85	644 ±3	645 ±4	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.12 ±0.04	0.13 ±0.04	
At Peak Load	0.06 ±0.05	0.11 ±0.05	0.11 ±0.05
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
At Peak Load	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 08, 2010
Test Identification:	TS7-L13-Z5-L8-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

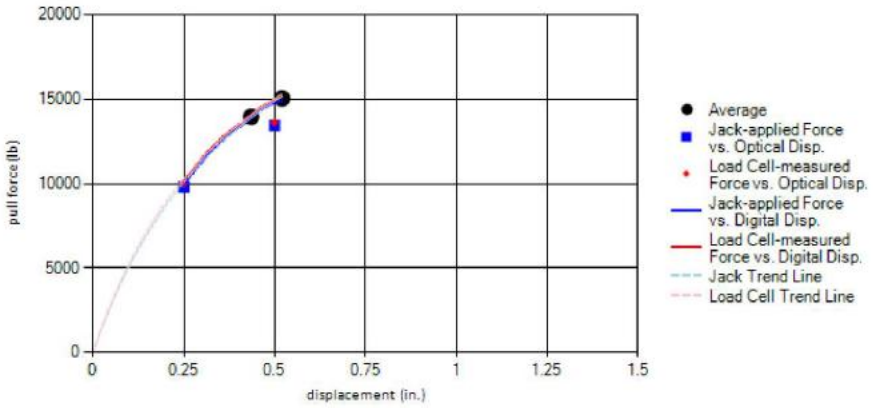
Backfill Material

Specification:	TxDOT Item 423 - Type B	In-Place Density (TEX-115-E)	
Moisture Density Relationship (TEX-114-E)		*Dry Unit Weight, γ_d (pcf):*	117.5 ±1.9
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Moisture Content, w (%):*	7.2 ±0.1%
Optimum Moisture Content, OMC (%):	7.8%	*Relative Compaction, C_R (%):*	94.4 ±1.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.44 ±0.04	631 ±3	13,963 ±396	4.9 ±0.0	**4.15 ±0.12**
At Peak Load	0.52 ±0.04	628 ±4	15,043 ±394	4.9 ±0.0	**4.49 ±0.12**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



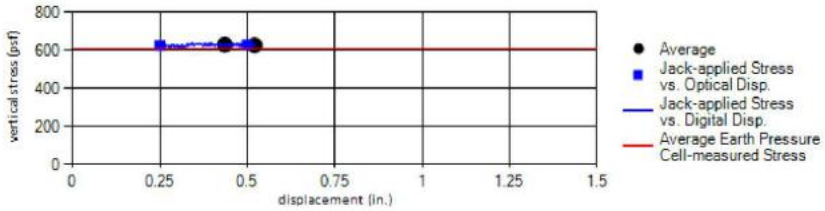
Comments	Personnel	
Rupture in connection at 0.52in. No incidental skew data collected. No incidental skew data.	*Tested:*	AS, MB, KH
	Prepared:	TW
	Checked:	WL, PJ

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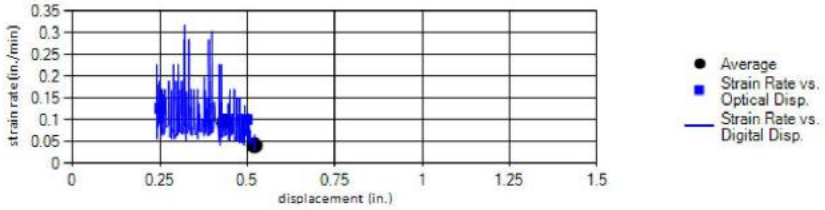
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(Peak)	Jack(0.75)
657 ±33	548 ±40	612 ±59	624 ±38	613 ±34	611 ±54	631 ±3	628 ±4	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.10 ±0.03	0.11 ±0.03	
At Peak Load	0.04 ±0.04	0.10 ±0.05	0.12 ±0.10
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
At Peak Load	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 08, 2010
Test Identification:	TS7-L14-Z5-L8-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



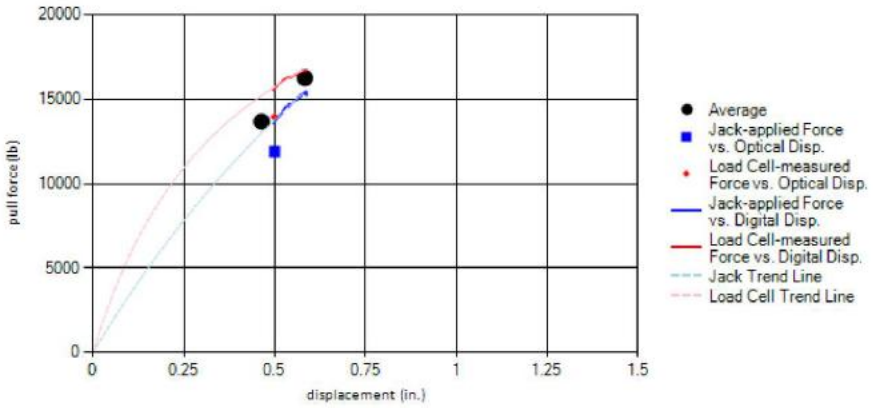
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	117.5	±1.9
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.1%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	94.4	±1.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.46 ±0.05	634 ±3	13,671 ±959	5.0 ±0.0	**4.04 ±0.28**
At Peak Load	0.58 ±0.04	635 ±4	16,254 ±542	5.0 ±0.0	**4.80 ±0.16**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



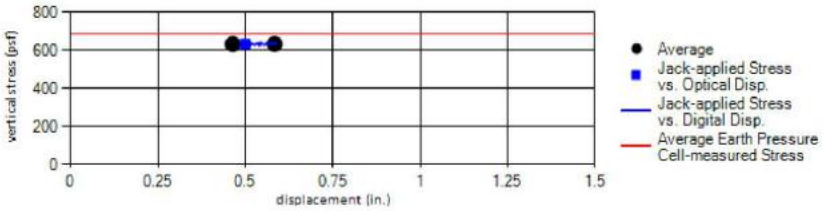
Comments	Personnel	
Rupture in connection at 0.59in. No incidental skew data collected. No incidental skew data.	*Tested:*	AS, MB, KH
	Prepared:	TW
	Checked:	WL, PJ

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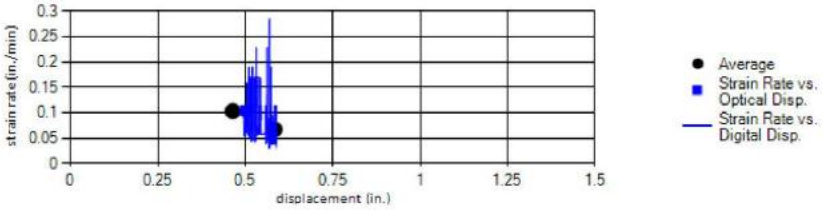
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(Peak)	Jack(0.75)
685 ±6	865 ±152	633 ±5	642 ±5	619 ±10	689 ±113	634 ±3	634 ±4	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.10 ±0.04	0.13 ±0.07	
At Peak Load	0.07 ±0.06	0.10 ±0.05	0.13 ±0.12
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
At Peak Load	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 14, 2011
Test Identification:	TS28-L11-Z5-L8-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



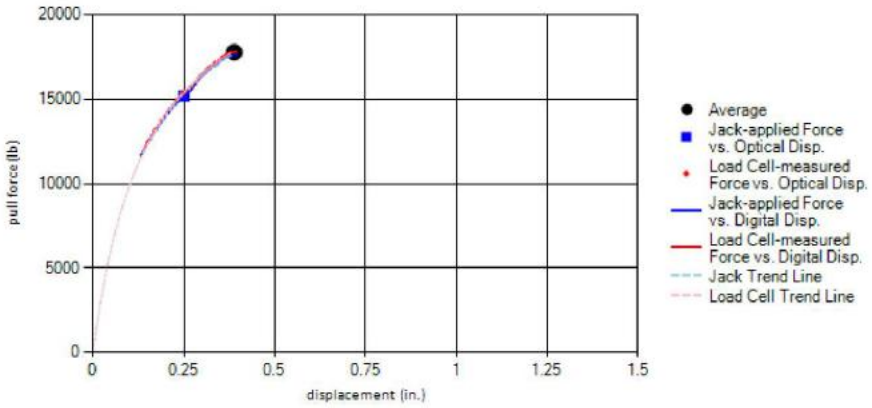
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, θ (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	121.5	±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.9%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	97.6	±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
At Peak Load	0.39 ±0.00	654 ±4	17,771 ±19	5.1 ±0.0	**5.09 ±0.03**
0.50	No Data	No Data	No Data	No Data	**No Data**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



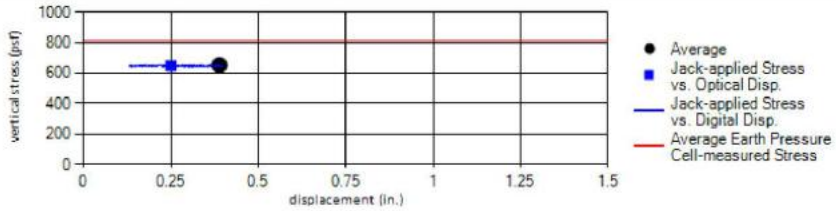
Comments	Personnel	
Rupture of longitudinal bar at 0.40in. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, YW, AJ
	Prepared:	TW
	Checked:	WL, PJ

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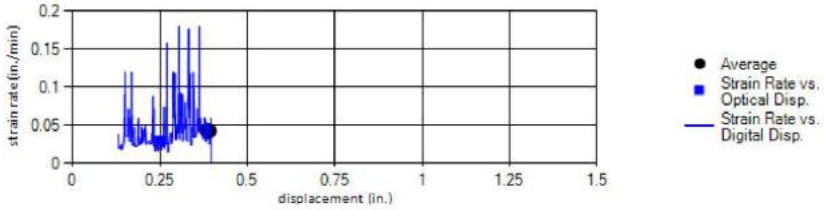
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(Peak)	Jack(0.50)	Jack(0.75)
737 ±52	881 ±1	No Data	692 ±5	952 ±0	815 ±109	654 ±4	No Data	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
At Peak Load	0.04 ±0.01	0.05 ±0.03	
0.50	No Data	No Data	0.06 ±0.05
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
At Peak Load		
0.50	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 14, 2011
Test Identification:	TS28-L5-Z6-L4-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



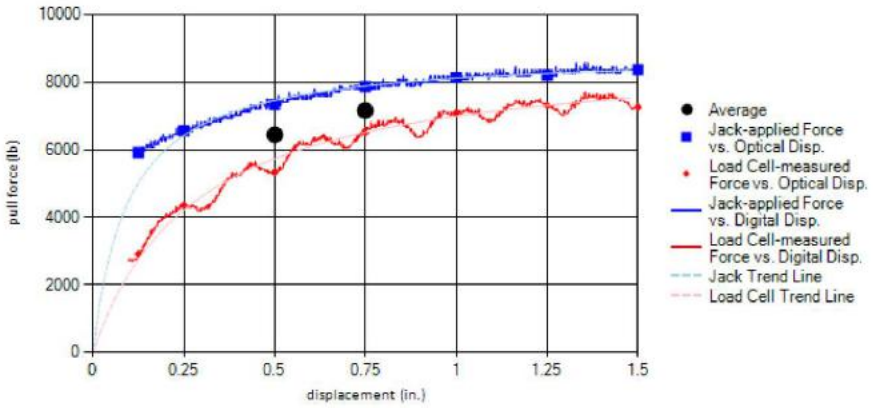
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material			
Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	121.5 ±3.2
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	7.2 ±0.9%
		Relative Compaction, C_R (%):	97.6 ±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	760 ±2	6,445 ±21	6.0 ±0.0	**3.18 ±0.02**
0.75	0.75 ±0.00	779 ±4	7,162 ±28	6.1 ±0.0	**3.45 ±0.02**
At Peak Load	1.49 ±0.00	772 ±5	8,406 ±46	No Data	**No Data**

Load-Displacement Curve



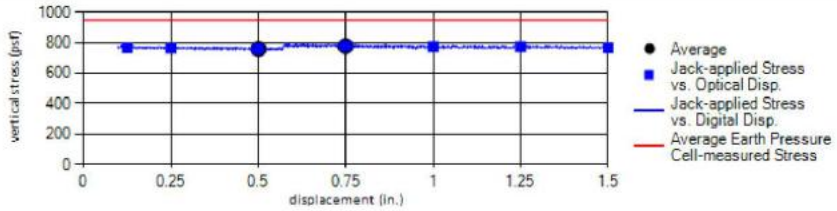
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AJ, RH
	Prepared:	TW
	Checked:	WL, PJ

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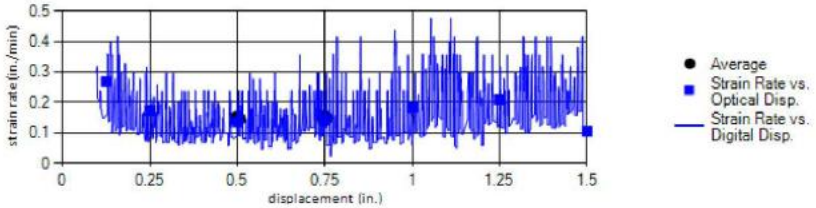
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
765 ±7	1,210 ±6	No Data	747 ±4	1,078 ±6	950 ±200	760 ±2	779 ±4	772 ±5



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.15 ±0.03	0.13 ±0.04	
0.75	0.15 ±0.03	0.15 ±0.04	0.19 ±0.11
At Peak Load	0.34 ±0.09	0.23 ±0.08	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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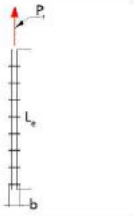
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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information

Test Date:	June 15, 2011
Test Identification:	TS28-L10-Z6-L4-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

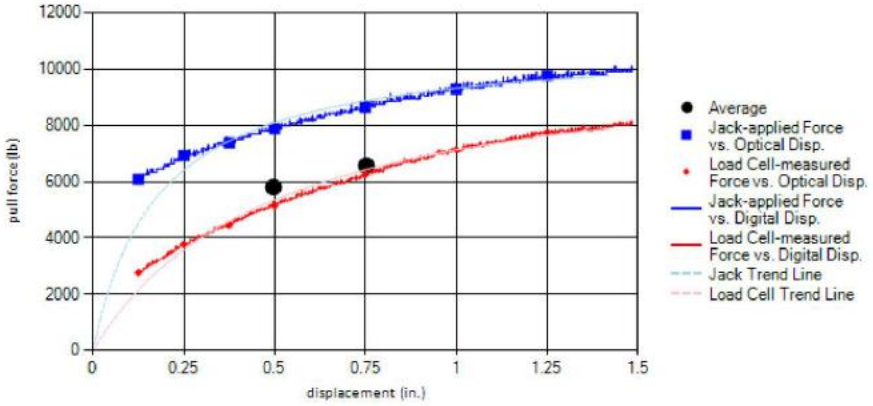
Backfill Material

Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	121.5 ±3.2
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	7.2 ±0.9%
		Relative Compaction, C_R (%):	97.6 ±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	763 ±3	5,803 ±19	6.0 ±0.0	**2.85 ±0.02**
0.75	0.75 ±0.00	761 ±2	6,564 ±25	6.0 ±0.0	**3.23 ±0.02**
At Peak Load	1.42 ±0.00	758 ±3	9,934 ±51	No Data	**No Data**

Load-Displacement Curve



Comments

No incidental skew data collected. No incidental skew data.

Personnel

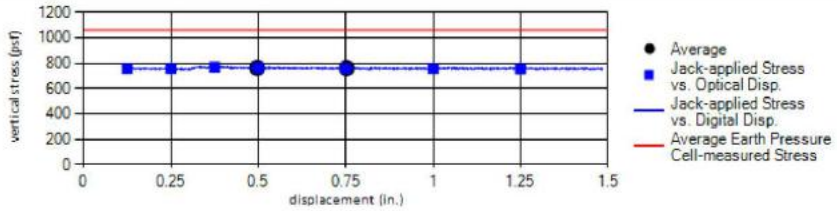
Tested:	TW, AS, RH
Prepared:	TW
Checked:	WL, PJ

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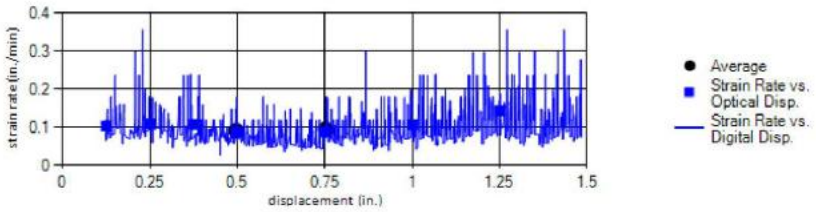
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
931 ±5	1,356 ±76	No Data	785 ±1	1,170 ±2	1,060 ±223	763 ±3	761 ±2	758 ±3



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.09 ±0.02	0.10 ±0.03	
0.75	0.09 ±0.02	0.08 ±0.02	0.12 ±0.16
At Peak Load	0.11 ±0.02	0.14 ±0.06	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 14, 2011
Test Identification:	TS28-L15-Z6-L4-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



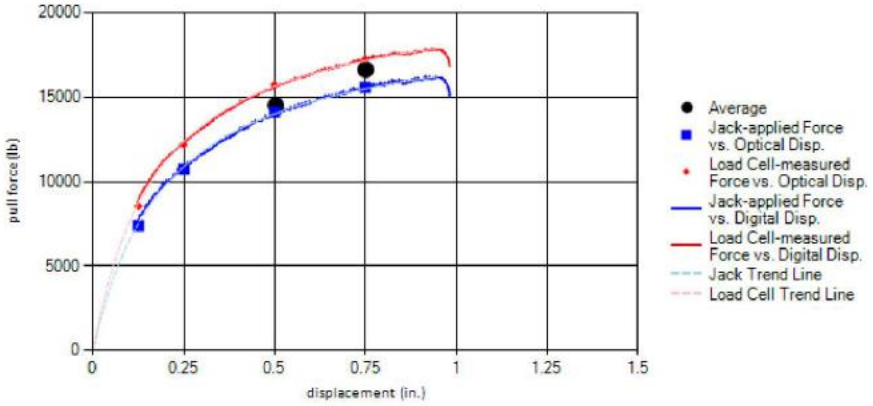
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, θ (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	121.5	±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.9%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	97.6	±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	776 ±2	14,518 ±38	6.1 ±0.0	**7.02 ±0.03**
0.75	0.75 ±0.00	777 ±3	16,629 ±24	6.1 ±0.0	**8.03 ±0.04**
At Peak Load	0.95 ±0.00	776 ±8	17,807 ±29	No Data	**No Data**

Load-Displacement Curve



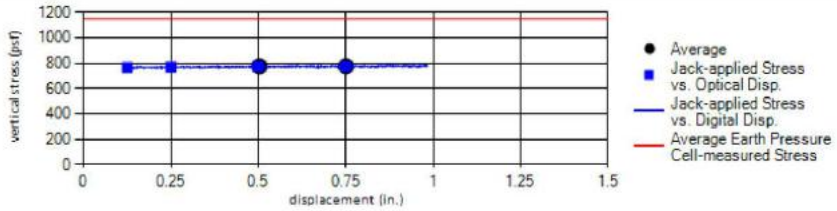
Comments	Personnel	
Rupture of both longitudinal bars at 0.98in. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AJ, RH
	Prepared:	TW
	Checked:	WL, PJ

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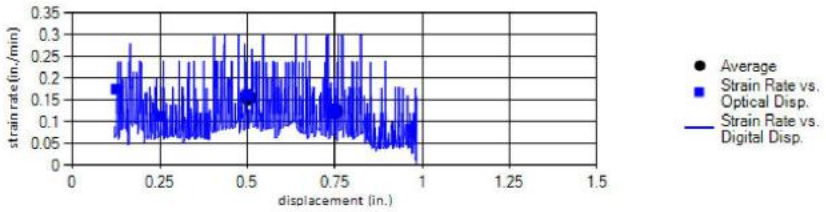
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
754 ±5	1,969 ±526	No Data	780 ±2	1,092 ±2	1,149 ±558	776 ±2	776 ±3	776 ±8



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.16 ±0.05	0.13 ±0.04	
0.75	0.12 ±0.03	0.14 ±0.04	0.13 ±0.06
At Peak Load	0.08 ±0.02	0.11 ±0.06	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):	SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*	8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*	7.9	*Internal Friction Angle, φ (deg.):*	43°

Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 15, 2011
Test Identification:	TS28-L2-Z6-L8-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



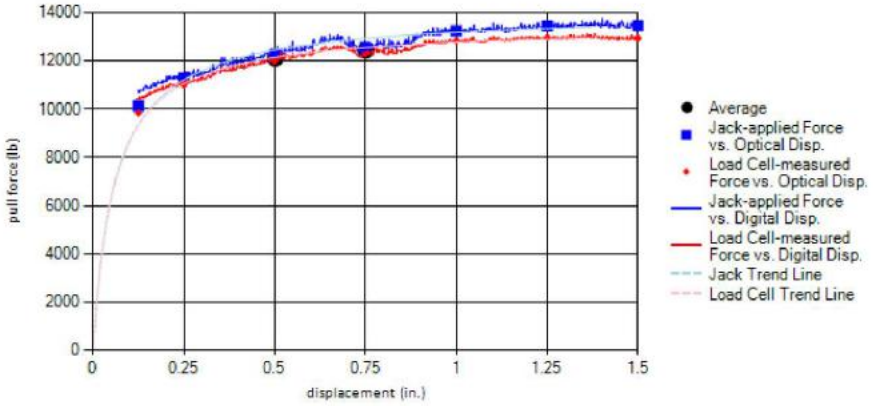
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, θ (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	121.5	±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.9%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	97.6	±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.01	771 ±7	12,093 ±25	6.0 ±0.1	**2.94 ±0.03**
0.75	0.75 ±0.00	764 ±5	12,434 ±44	6.0 ±0.0	**3.05 ±0.02**
At Peak Load	1.38 ±0.00	784 ±4	13,503 ±45	No Data	**No Data**

Load-Displacement Curve



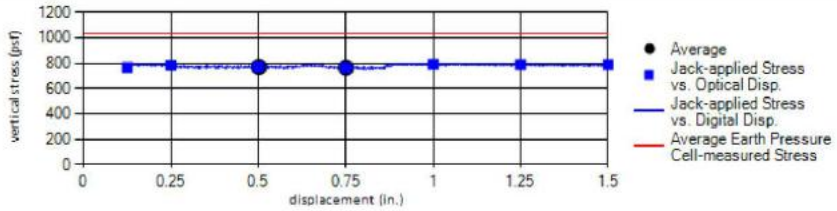
Comments	Personnel	
Strain gage data collected. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AJ, AS
	Prepared:	TW
	Checked:	WL, PJ

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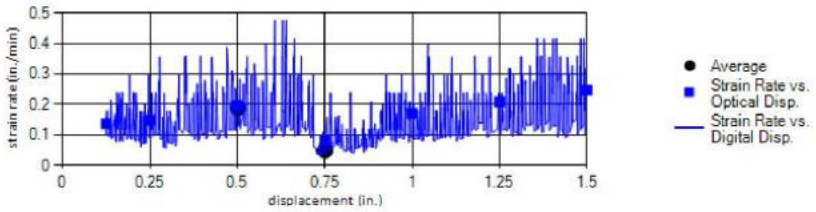
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
954 ±46	1,196 ±9	No Data	813 ±3	1,180 ±6	1,036 ±162	770 ±7	764 ±5	784 ±4



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.19 ±0.06	0.17 ±0.05	
0.75	0.05 ±0.01	0.19 ±0.08	0.17 ±0.08
At Peak Load	0.30 ±0.14	0.19 ±0.08	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E,	
Sieve Size	*Percent Retained*	*Percent Retained*	TEX-106-E, TEX-107-E)	
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 15, 2011
Test Identification:	TS28-L7-Z6-L8-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

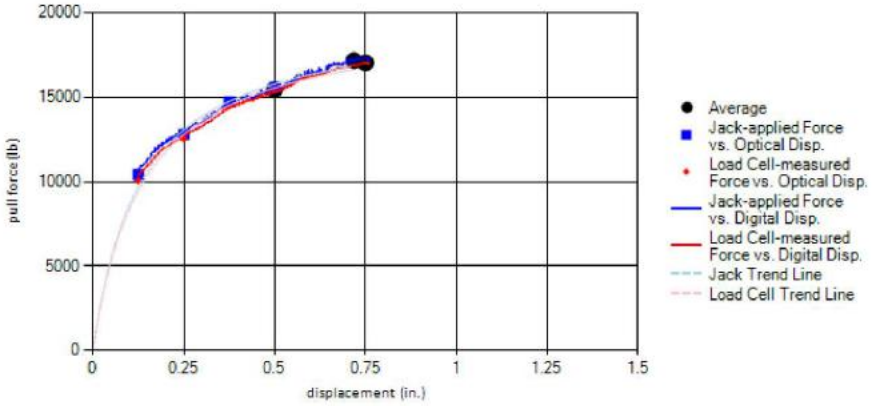
Backfill Material

Specification:	TxDOT Item 423 - Type B	In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)		*Dry Unit Weight, γ_d (pcf):*	121.5	±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Moisture Content, w (%):*	7.2	±0.9%
Optimum Moisture Content, OMC (%):	7.8%	*Relative Compaction, C_R (%):*	97.6	±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	743 ±6	15,438 ±32	5.8 ±0.0	**3.89 ±0.03**
At Peak Load	0.72 ±0.00	776 ±6	17,164 ±51	6.1 ±0.0	**4.15 ±0.03**
0.75	0.75 ±0.00	772 ±5	17,030 ±41	6.1 ±0.0	**4.14 ±0.03**

Load-Displacement Curve



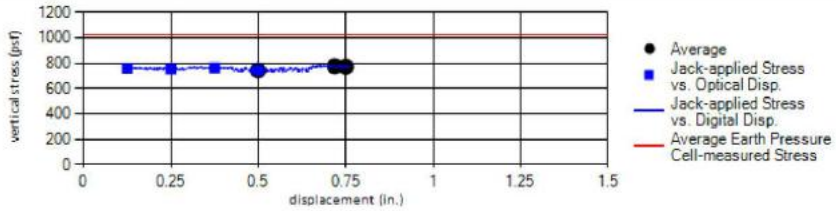
Comments	Personnel	
Rupture of longitudinal bar at 0.76in. Strain gage data collected. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, YW
	Prepared:	TW
	Checked:	WL, PJ

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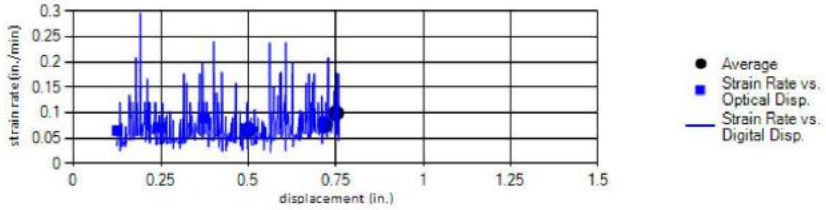
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(Peak)	Jack(0.75)
974 ±121	1,226 ±104	No Data	750 ±86	1,148 ±120	1,025 ±213	743 ±6	776 ±6	772 ±5



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.07 ±0.02	0.07 ±0.02	
At Peak Load	0.08 ±0.01	0.08 ±0.04	0.08 ±0.04
0.75	0.10 ±0.06	0.10 ±0.04	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
At Peak Load	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 16, 2011
Test Identification:	TS28-L8-Z6-L8-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



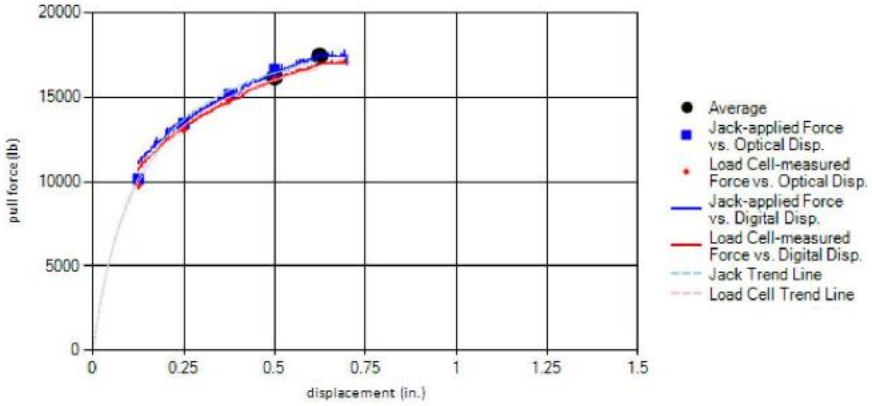
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material			
Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	121.5 ±3.2
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	7.2 ±0.9%
		Relative Compaction, C_R (%):	97.6 ±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	763 ±3	16,177 ±38	6.0 ±0.0	**3.98 ±0.02**
At Peak Load	0.62 ±0.00	776 ±5	17,432 ±40	6.1 ±0.0	**4.21 ±0.03**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



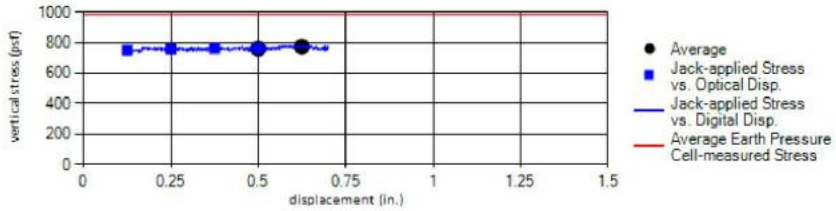
Comments	Personnel	
Rupture of longitudinal bar at 0.70in. Strain gage data collected. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, YW
	Prepared:	TW
	Checked:	WL, PJ

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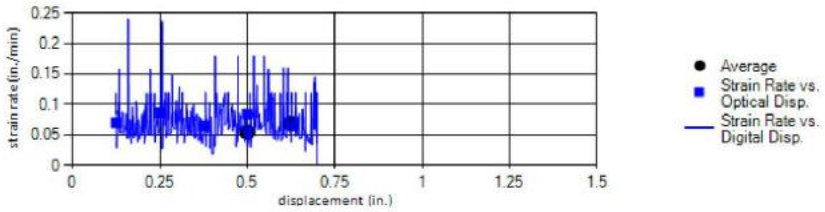
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(Peak)	Jack(0.75)
867 ±8	1,219 ±5	No Data	816 ±3	1,042 ±6	986 ±159	763 ±3	775 ±5	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.05 ±0.02	0.08 ±0.02	
At Peak Load	0.07 ±0.03	0.08 ±0.03	0.08 ±0.12
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
At Peak Load	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 15, 2011
Test Identification:	TS28-L21-Z6-L8-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material			
Specification:	TxDOT Item 423 - Type B	In-Place Density (TEX-115-E)	
Moisture Density Relationship (TEX-114-E)		*Dry Unit Weight, γ_d (pcf):*	121.5 ±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Moisture Content, w (%):*	7.2 ±0.9%
Optimum Moisture Content, OMC (%):	7.8%	*Relative Compaction, C_R (%):*	97.6 ±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	No Data	No Data	No Data	No Data	**No Data**
0.75	No Data	No Data	No Data	No Data	**No Data**
At Peak Load	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve

Comments	Personnel	
NO TEST. Displacement failures. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, RH, AS
	Prepared:	TW
	Checked:	WL, PJ

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Vertical Stress, σ_v (psf)								
EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
No Data	No Data	No Data	No Data	No Data	No Data	No Data	No Data	No Data

Strain Rate			
nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	No Data	No Data	
0.75	No Data	No Data	No Data
At Peak Load	No Data	No Data	

Incidental Skew		
nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties				
Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 15, 2011
Test Identification:	TS28-L24-Z6-L8-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



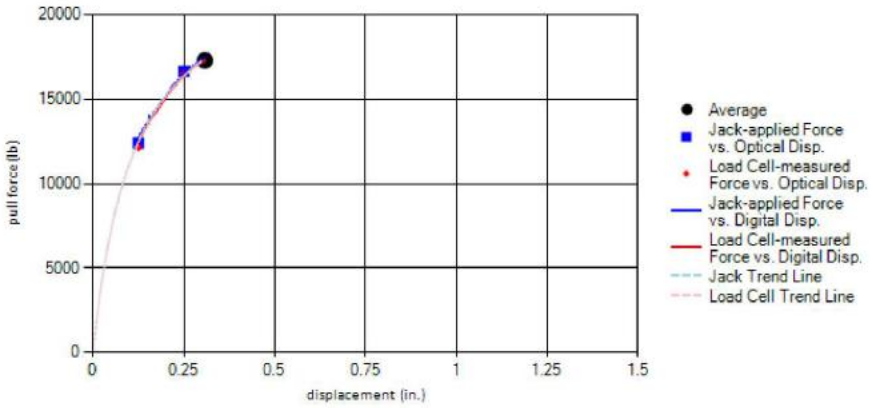
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	121.5	±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.9%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	97.6	±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
At Peak Load	0.31 ±0.00	1,030 ±3	17,293 ±40	8.1 ±0.0	**3.15 ±0.01**
0.50	No Data	No Data	No Data	No Data	**No Data**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



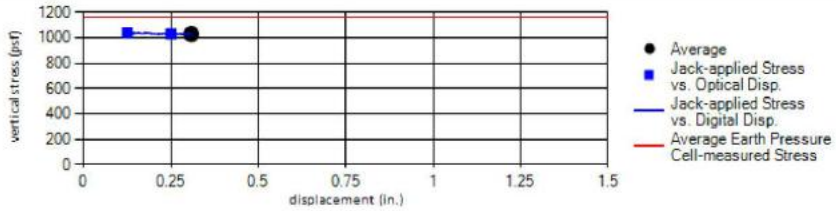
Comments	Personnel	
Rupture of longitudinal bar at 0.31in. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, RH
	Prepared:	TW
	Checked:	WL, PJ

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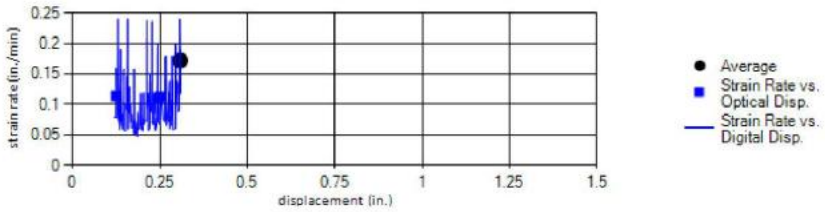
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(Peak)	Jack(0.50)	Jack(0.75)
1,003 ±2	1,370 ±6	No Data	1,001 ±9	1,276 ±63	1,163 ±168	1,030 ±3	No Data	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
At Peak Load	0.17 ±0.04	0.10 ±0.04	
0.50	No Data	No Data	0.10 ±0.04
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
At Peak Load		
0.50	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 08, 2010
Test Identification:	TS7-L7-Z9-L4-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



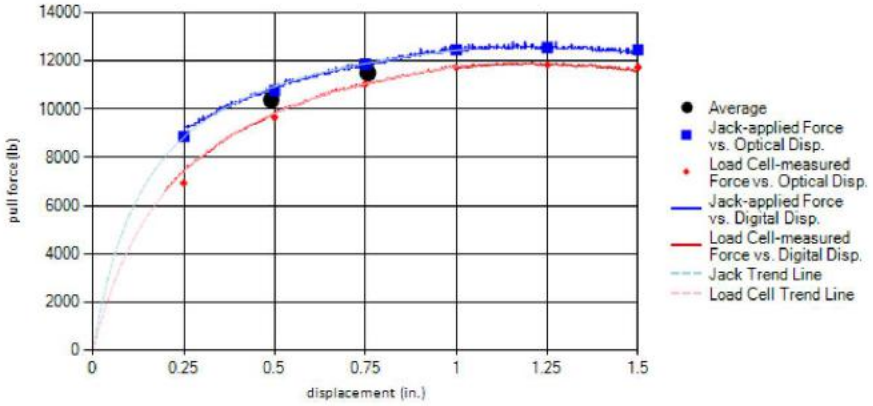
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, θ (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	117.5	±1.9
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.1%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	94.4	±1.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.49 ±0.06	1,132 ±4	10,388 ±266	8.9 ±0.0	**3.44 ±0.09**
0.75	0.76 ±0.06	1,132 ±3	11,503 ±143	8.9 ±0.0	**3.81 ±0.05**
At Peak Load	1.04 ±0.08	1,128 ±5	12,174 ±77	No Data	**No Data**

Load-Displacement Curve



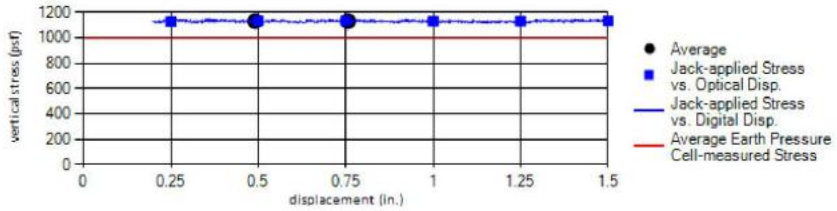
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	AS, MB, DH
	Prepared:	TW
	Checked:	WL, PJ

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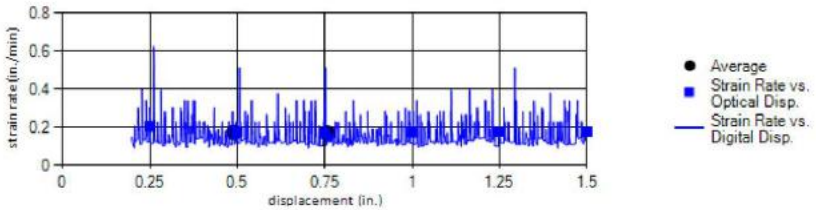
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
926 ±31	968 ±38	973 ±34	1,061 ±36	1,067 ±41	999 ±66	1,133 ±4	1,133 ±3	1,129 ±5



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.17 ±0.04	0.18 ±0.05	
0.75	0.17 ±0.04	0.18 ±0.05	0.18 ±0.07
At Peak Load	0.16 ±0.06	0.17 ±0.05	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 08, 2010
Test Identification:	TS7-L8-Z9-L4-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



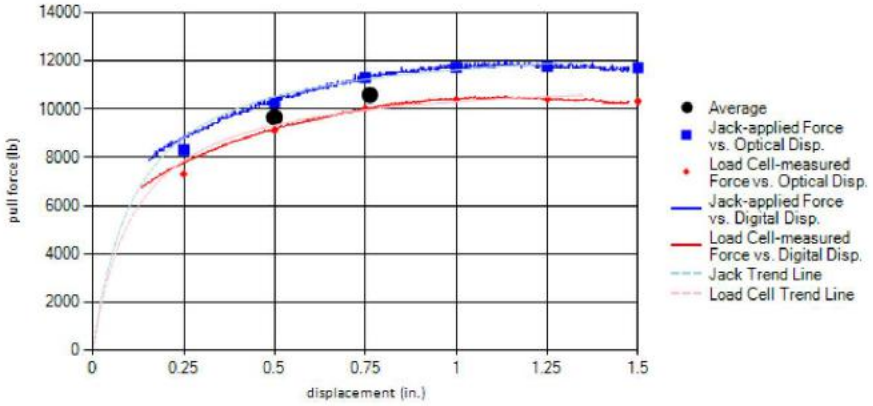
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, θ (°):	0

Backfill Material			
Specification:	TxDOT Item 423 - Type B	In-Place Density (TEX-115-E)	
Moisture Density Relationship (TEX-114-E)		*Dry Unit Weight, γ_d (pcf):*	117.5 ±1.9
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Moisture Content, w (%):*	7.2 ±0.1%
Optimum Moisture Content, OMC (%):	7.8%	*Relative Compaction, C_R (%):*	94.4 ±1.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.06	1,133 ±3	9,668 ±199	8.9 ±0.0	**3.20 ±0.07**
0.75	0.76 ±0.06	1,131 ±3	10,594 ±119	8.9 ±0.0	**3.51 ±0.04**
At Peak Load	1.35 ±0.08	1,135 ±4	10,994 ±50	No Data	**No Data**

Load-Displacement Curve



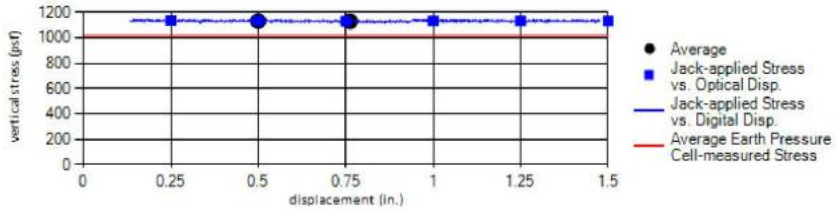
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	AS, MB, DH
	Prepared:	TW
	Checked:	WL, PJ

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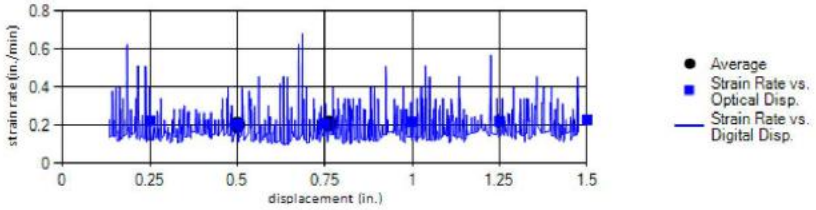
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
940 ±9	1,001 ±11	971 ±20	1,077 ±8	1,099 ±12	1,018 ±63	1,133 ±3	1,131 ±3	1,135 ±4



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.20 ±0.05	0.20 ±0.04	
0.75	0.21 ±0.06	0.20 ±0.06	0.22 ±0.09
At Peak Load	0.33 ±0.08	0.22 ±0.08	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 08, 2010
Test Identification:	TS7-L9-Z9-L4-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



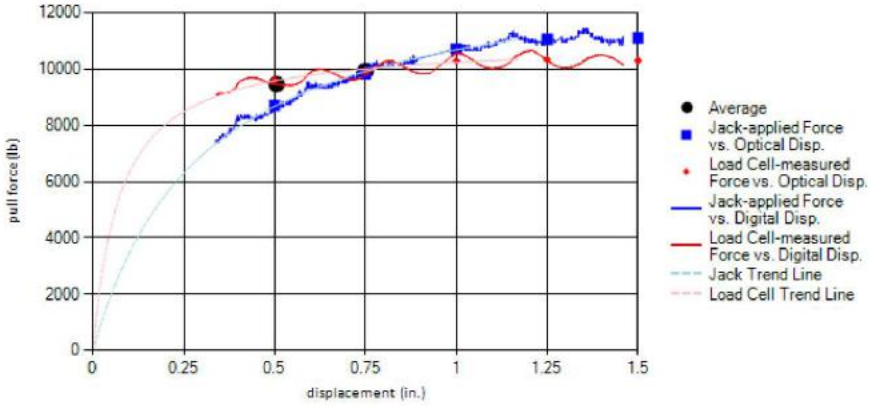
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material			
Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	117.5 ±1.9
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	7.2 ±0.1%
		Relative Compaction, C_R (%):	94.4 ±1.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.51 ±0.07	1,135 ±3	9,459 ±157	8.9 ±0.0	**3.13 ±0.05**
0.75	0.75 ±0.06	1,135 ±3	9,924 ±126	8.9 ±0.0	**3.28 ±0.04**
At Peak Load	1.18 ±0.07	1,131 ±4	10,942 ±119	No Data	**No Data**

Load-Displacement Curve

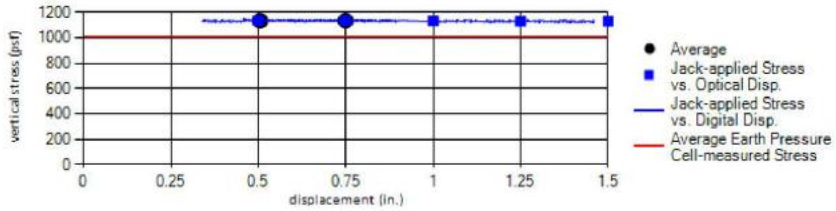


Comments	Personnel	
Loose LVDT connections in first 2 optical readings. No incidental skew data collected. No incidental skew data.	*Tested:*	AS, MB, KH
	Prepared:	TW
	Checked:	WL, PJ

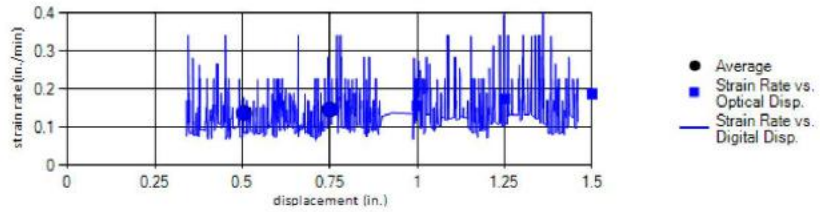
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Vertical Stress, σ_v (psf)								
EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
941 ±3	1,003 ±4	990 ±6	1,077 ±3	1,040 ±57	1,010 ±53	1,135 ±3	1,135 ±3	1,131 ±4



Strain Rate			
nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.14 ±0.03	0.14 ±0.04	
0.75	0.15 ±0.04	0.14 ±0.03	0.16 ±0.06
At Peak Load	0.15 ±0.06	0.17 ±0.06	



Incidental Skew		
nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties				
Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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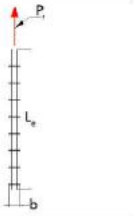
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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information

Test Date:	July 08, 2010
Test Identification:	TS7-L10-Z9-L8-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

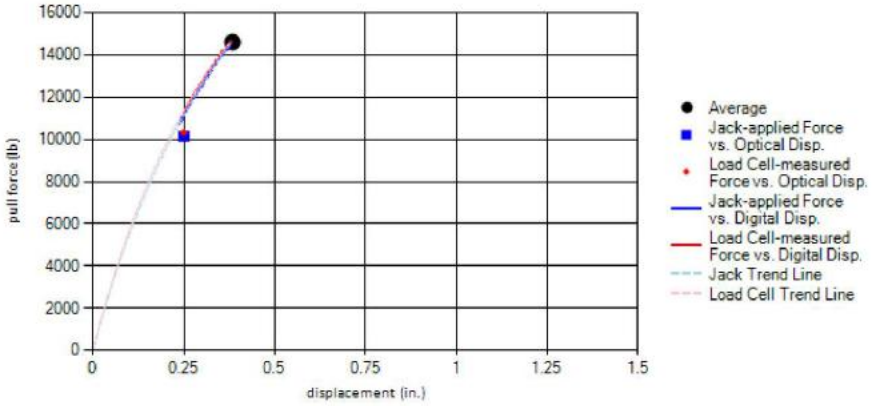
Backfill Material

Specification:	TxDOT Item 423 - Type B	In-Place Density (TEX-115-E)	
Moisture Density Relationship (TEX-114-E)		*Dry Unit Weight, γ_d (pcf):*	117.5 ±1.9
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Moisture Content, w (%):*	7.2 ±0.1%
Optimum Moisture Content, OMC (%):	7.8%	*Relative Compaction, C_R (%):*	94.4 ±1.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
At Peak Load	0.38 ±0.04	1,106 ±6	14,601 ±698	8.7 ±0.0	**2.48 ±0.12**
0.50	No Data	No Data	No Data	No Data	**No Data**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



Comments

Rupture in connection at 0.38in. No incidental skew data collected. No incidental skew data.

Personnel

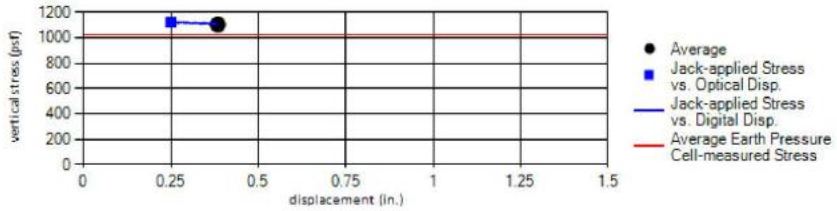
Tested:	AS, MB, KH
Prepared:	TW
Checked:	WL, PJ

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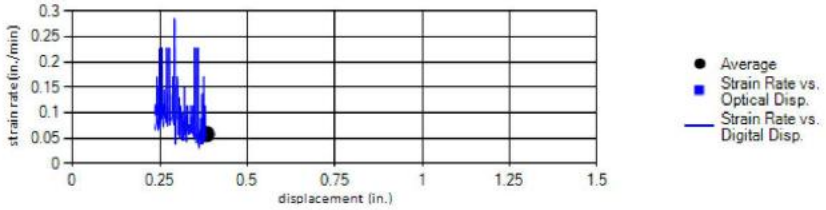
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(Peak)	Jack(0.50)	Jack(0.75)
1,077 ±200	973 ±111	1,070 ±159	1,018 ±129	976 ±123	1,023 ±153	1,106 ±6	No Data	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
At Peak Load	0.06 ±0.05	0.10 ±0.05	
0.50	No Data	No Data	0.11 ±0.06
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
At Peak Load		
0.50	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):	SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*	8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*	7.9	*Internal Friction Angle, φ (deg.):*	43°

Gradation *Sieve Size*	TEX-110-E *Percent Retained*	ASTM D422 D1140 *Percent Retained*	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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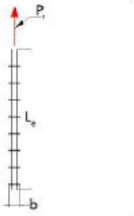
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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information

Test Date:	July 08, 2010
Test Identification:	TS7-L11-Z9-L8-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

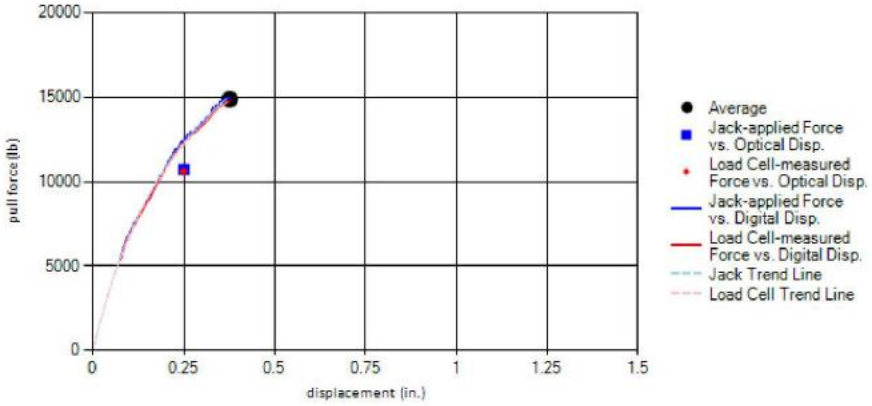
Backfill Material

Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	117.5 ±1.9
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	7.2 ±0.1%
		Relative Compaction, C_R (%):	94.4 ±1.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
At Peak Load	0.38 ±0.04	1,124 ±4	14,875 ±586	8.8 ±0.0	**2.48 ±0.10**
0.50	No Data	No Data	No Data	No Data	**No Data**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



Comments

Rupture in connection at 0.38in. No incidental skew data collected. No incidental skew data.

Personnel

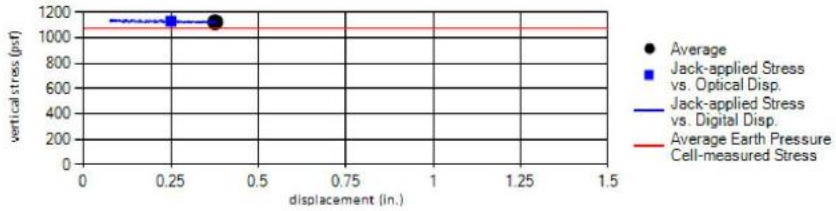
Tested:	AS, KH, DH
Prepared:	TW
Checked:	WL, PJ

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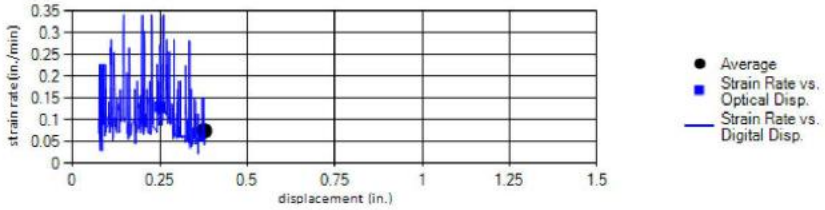
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(Peak)	Jack(0.50)	Jack(0.75)
1,000 ±18	1,036 ±21	1,161 ±22	1,113 ±22	1,057 ±22	1,074 ±61	1,124 ±4	No Data	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
At Peak Load	0.07 ±0.07	0.12 ±0.07	
0.50	No Data	No Data	0.14 ±0.11
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
At Peak Load		
0.50	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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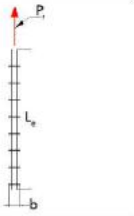
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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information

Test Date:	July 08, 2010
Test Identification:	TS7-L12-Z9-L8-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

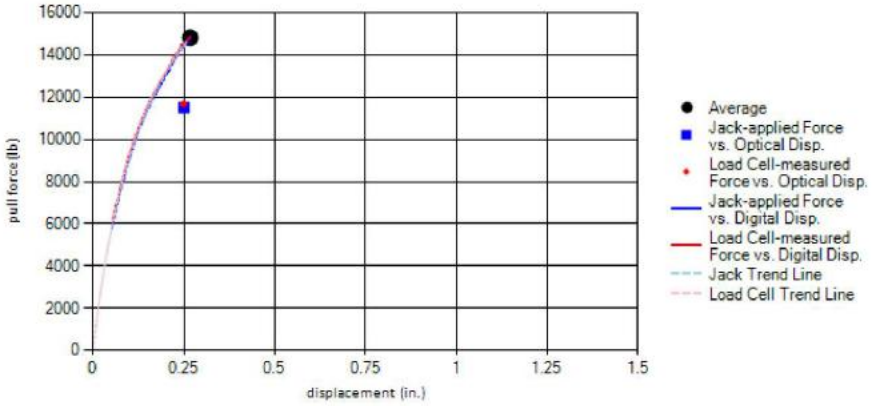
Backfill Material

Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	117.5 ±1.9
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	7.2 ±0.1%
		Relative Compaction, C_R (%):	94.4 ±1.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
At Peak Load	0.27 ±0.04	1,133 ±5	14,814 ±839	8.9 ±0.0	**2.45 ±0.14**
0.50	No Data	No Data	No Data	No Data	**No Data**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



Comments

Rupture in connection at 0.27in. No incidental skew data collected. No incidental skew data.

Personnel

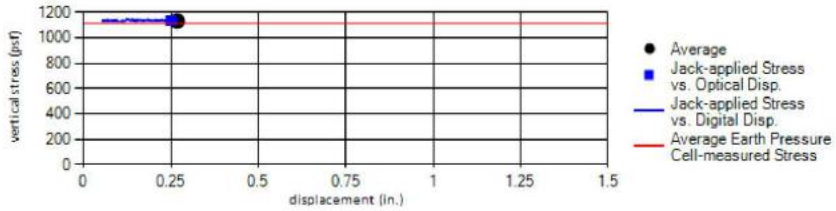
Tested:	AS, KH, DH
Prepared:	TW
Checked:	WL, PJ

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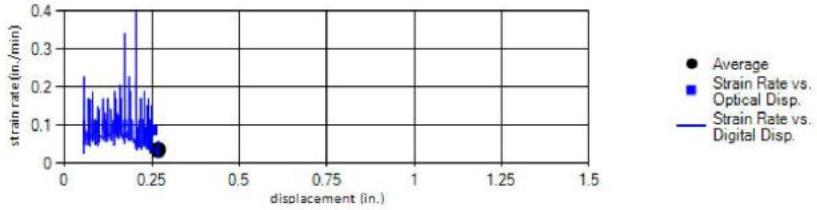
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(Peak)	Jack(0.50)	Jack(0.75)
1,056 ±15	1,051 ±13	1,254 ±164	1,134 ±16	1,074 ±15	1,114 ±106	1,133 ±5	No Data	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
At Peak Load	0.04 ±0.05	0.11 ±0.10	
0.50	No Data	No Data	0.11 ±0.10
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
At Peak Load		
0.50	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):	SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*	8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*	7.9	*Internal Friction Angle, φ (deg.):*	43°

Gradation *Sieve Size*	TEX-110-E *Percent Retained*	ASTM D422 D1140 *Percent Retained*	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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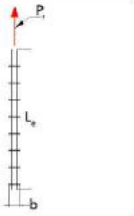
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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information

Test Date:	July 02, 2010
Test Identification:	TS6-L9-Z12-L4-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

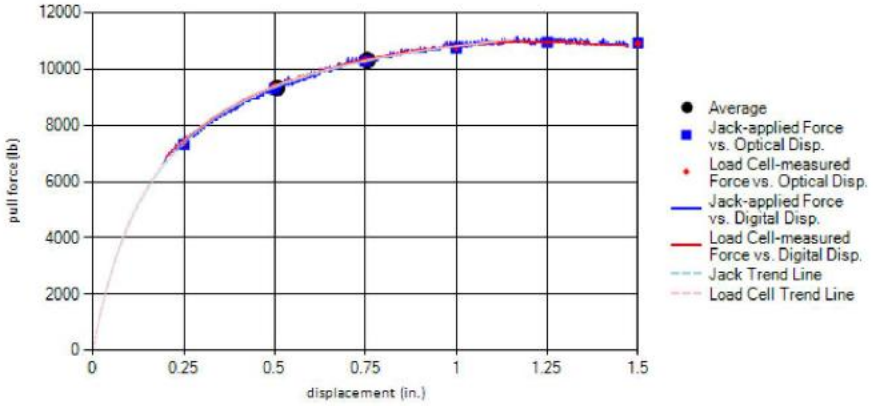
Backfill Material

Specification:	TxDOT Item 423 - Type B		
Moisture Density Relationship (TEX-114-E)		In-Place Density (TEX-115-E)	
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Dry Unit Weight, γ_d (pcf):*	118.5 ±0.3
Optimum Moisture Content, OMC (%):	7.8%	*Moisture Content, w (%):*	7.2 ±0.2%
		Relative Compaction, C_R (%):	95.2 ±0.2%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.51 ±0.06	1,511 ±3	9,328 ±231	11.9 ±0.0	**2.31 ±0.06**
0.75	0.76 ±0.06	1,514 ±9	10,313 ±130	11.9 ±0.1	**2.55 ±0.04**
At Peak Load	1.17 ±0.08	1,524 ±5	11,009 ±30	No Data	**No Data**

Load-Displacement Curve



Comments

No incidental skew data collected. No incidental skew data.

Personnel

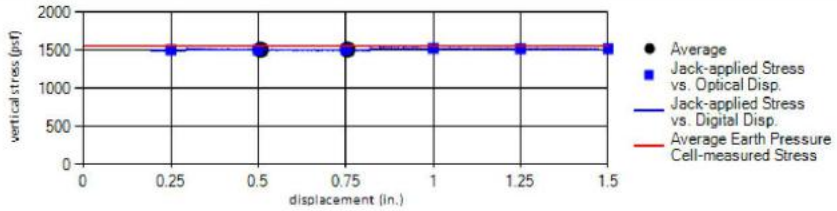
Tested:	TW, AS, DH
Prepared:	TW
Checked:	WL, PJ

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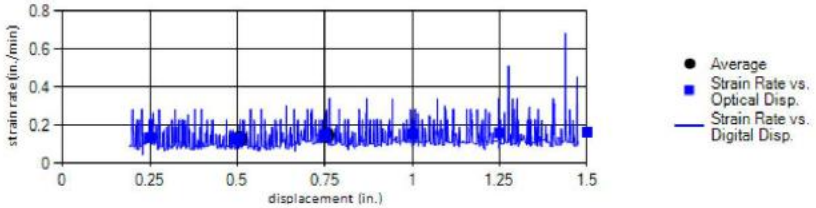
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
1,562 ±35	1,570 ±55	1,591 ±39	1,441 ±29	1,645 ±64	1,562 ±81	1,511 ±3	1,514 ±9	1,524 ±5



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.13 ±0.03	0.13 ±0.03	
0.75	0.15 ±0.04	0.14 ±0.04	0.15 ±0.08
At Peak Load	0.21 ±0.06	0.16 ±0.05	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E,	
Sieve Size	*Percent Retained*	*Percent Retained*	TEX-106-E, TEX-107-E)	
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 01, 2010
Test Identification:	TS6-L10-Z12-L4-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



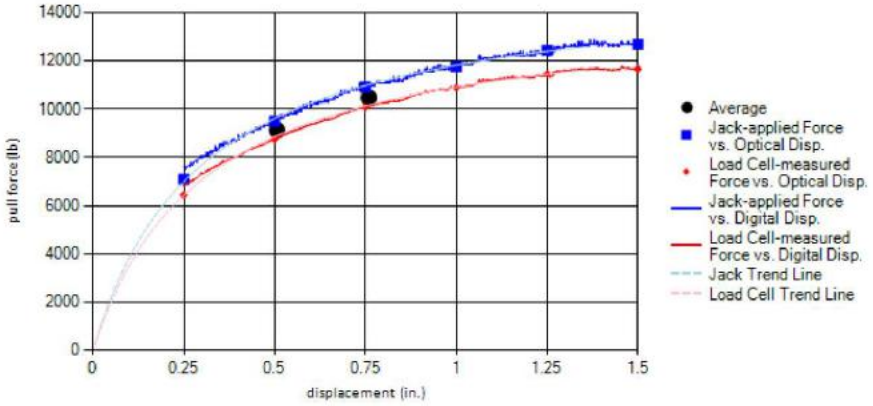
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	118.5	±0.3
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.2%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	95.2	±0.2%

Pullout Test Results

Nominal	Measured				Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****	
0.50	0.51 ±0.06	1,522 ±6	9,130 ±300	11.9 ±0.0	**2.25**	**±0.07**
0.75	0.76 ±0.07	1,527 ±4	10,475 ±196	12.0 ±0.0	**2.57**	**±0.05**
At Peak Load	1.37 ±0.08	1,519 ±5	12,074 ±84	No Data	**No**	**Data**

Load-Displacement Curve



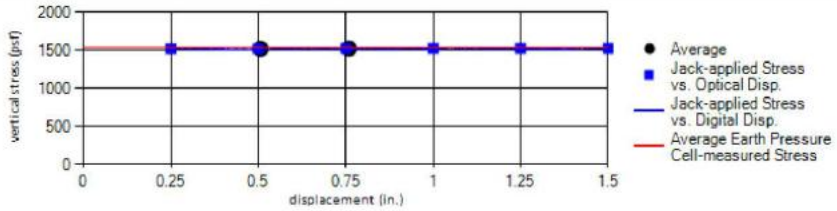
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, DH
	Prepared:	TW
	Checked:	WL, PJ

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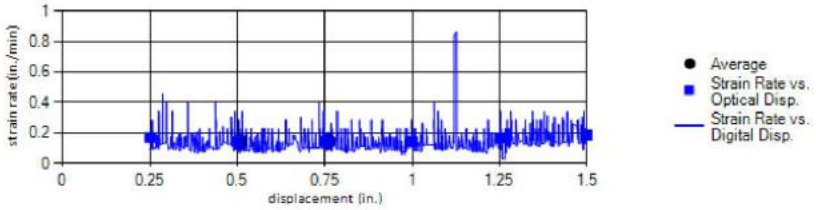
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
1,783 ±478	1,563 ±144	1,534 ±112	1,367 ±102	1,467 ±100	1,543 ±273	1,522 ±6	1,527 ±4	1,519 ±5



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.14 ±0.04	0.15 ±0.05	
0.75	0.14 ±0.04	0.14 ±0.04	0.16 ±0.12
At Peak Load	0.19 ±0.06	0.16 ±0.08	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E,	
Sieve Size	*Percent Retained*	*Percent Retained*	TEX-106-E, TEX-107-E)	
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 01, 2010
Test Identification:	TS6-L11-Z12-L4-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



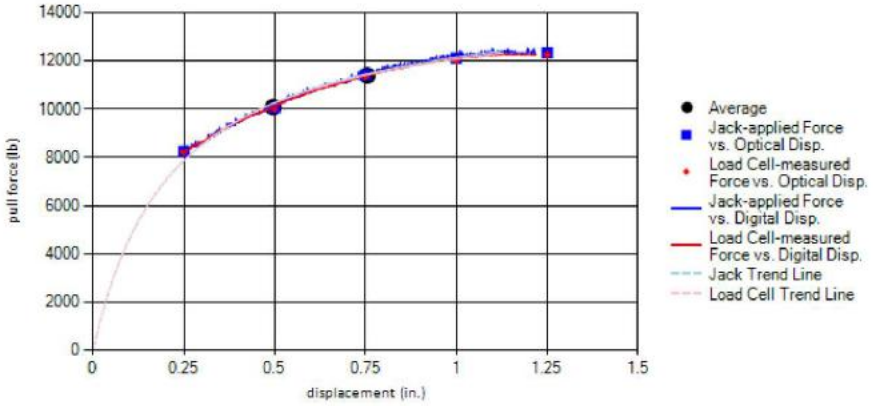
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	118.5	±0.3
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.2%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	95.2	±0.2%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.06	1,527 ±3	10,088 ±284	12.0 ±0.0	**2.48 ±0.07**
0.75	0.75 ±0.06	1,523 ±3	11,397 ±177	11.9 ±0.0	**2.81 ±0.04**
At Peak Load	1.13 ±0.06	1,520 ±4	12,332 ±42	No Data	**No Data**

Load-Displacement Curve



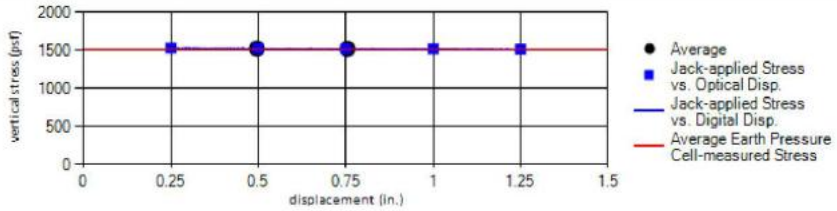
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, DH
	Prepared:	TW
	Checked:	WL, PJ

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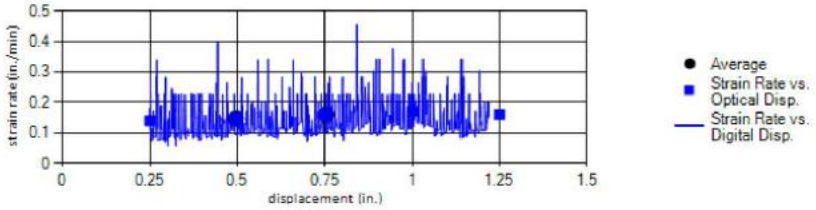
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
1,542 ±57	1,615 ±7	1,541 ±12	1,393 ±4	1,491 ±8	1,517 ±78	1,527 ±3	1,523 ±3	1,520 ±4



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.15 ±0.04	0.14 ±0.04	
0.75	0.16 ±0.04	0.15 ±0.04	0.16 ±0.10
At Peak Load	0.18 ±0.06	0.17 ±0.06	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E,	
Sieve Size	*Percent Retained*	*Percent Retained*	TEX-106-E, TEX-107-E)	
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 01, 2010
Test Identification:	TS6-L12-Z12-L4-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



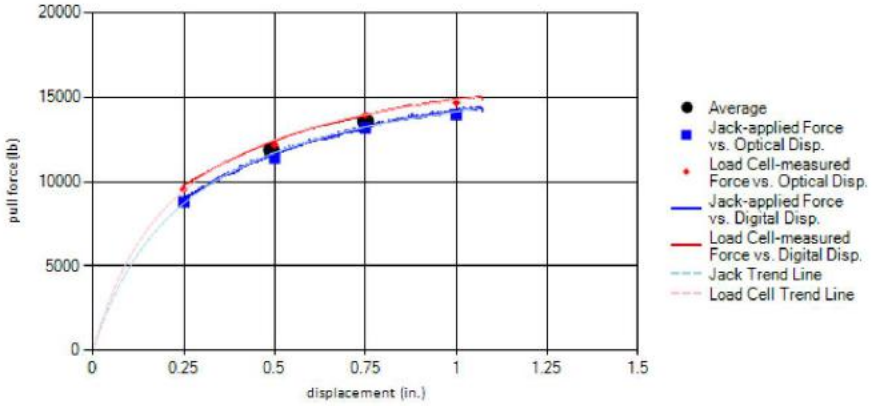
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	118.5	±0.3
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.2%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	95.2	±0.2%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.49 ±0.07	1,531 ±4	11,881 ±393	12.0 ±0.0	**2.91 ±0.10**
0.75	0.75 ±0.06	1,529 ±3	13,564 ±224	12.0 ±0.0	**3.33 ±0.06**
At Peak Load	1.06 ±0.05	1,524 ±4	14,776 ±90	No Data	**No Data**

Load-Displacement Curve



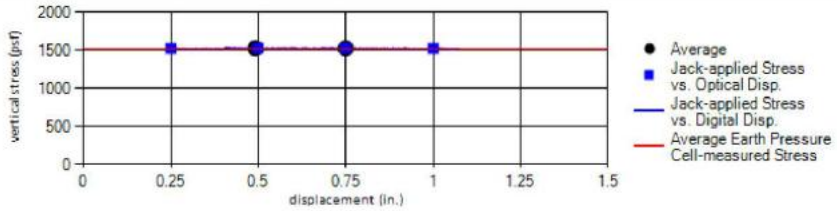
Comments	Personnel	
Rupture in connection at 1.07in. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, DH
	Prepared:	TW
	Checked:	WL, PJ

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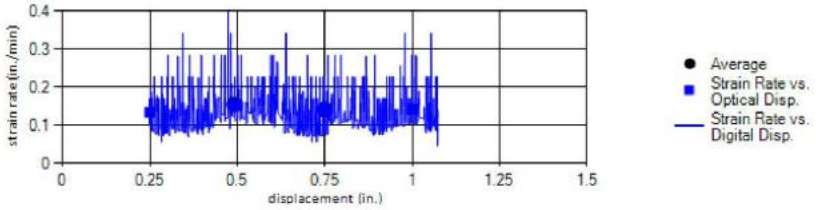
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
1,568 ±19	1,610 ±26	1,539 ±9	1,396 ±1	1,496 ±3	1,522 ±75	1,531 ±4	1,529 ±3	1,524 ±4



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.16 ±0.04	0.14 ±0.04	
0.75	0.14 ±0.04	0.15 ±0.04	0.14 ±0.06
At Peak Load	0.14 ±0.05	0.14 ±0.05	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E,	
Sieve Size	*Percent Retained*	*Percent Retained*	TEX-106-E, TEX-107-E)	
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 01, 2010
Test Identification:	TS6-L14-Z12-L4-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



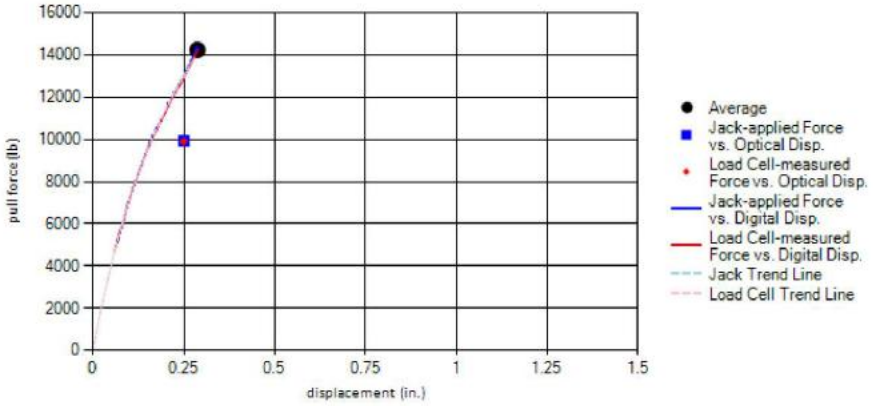
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	118.5	±0.3
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.2%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	95.2	±0.2%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
At Peak Load	0.29 ±0.04	1,527 ±4	14,235 ±970	12.0 ±0.0	**3.50 ±0.24**
0.50	No Data	No Data	No Data	No Data	**No Data**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



Comments	Personnel	
Rupture in connection at 0.29in. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, DH
	Prepared:	TW
	Checked:	WL, PJ

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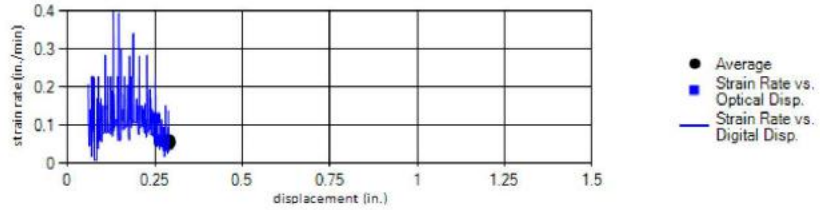
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(Peak)	Jack(0.50)	Jack(0.75)
1,600 ±3	1,822 ±104	1,607 ±3	1,402 ±0	1,521 ±6	1,590 ±146	1,527 ±4	No Data	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
At Peak Load	0.06 ±0.06	0.13 ±0.10	
0.50	No Data	No Data	0.13 ±0.10
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
At Peak Load		
0.50	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 14, 2011
Test Identification:	TS28-L14-Z12-L4-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



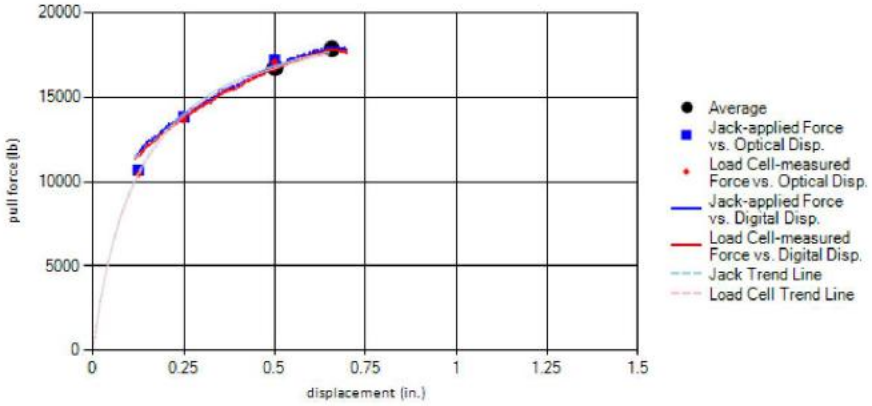
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	121.5	±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.9%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	97.6	±2.5%

Pullout Test Results

Nominal	Measured				Calculated		
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	1,527 ±4	16,738 ±23	12.0 ±0.0	**4.11 ±0.02**
At Peak Load	0.66 ±0.00	1,534 ±2	17,860 ±29	12.0 ±0.0	**4.37 ±0.01**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



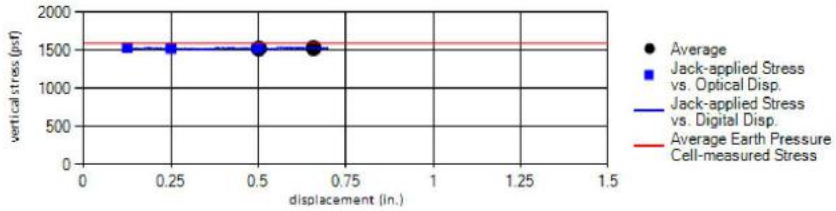
Comments	Personnel	
Rupture of longitudinal bar at 0.70in. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AJ, RH
	Prepared:	TW
	Checked:	WL, PJ

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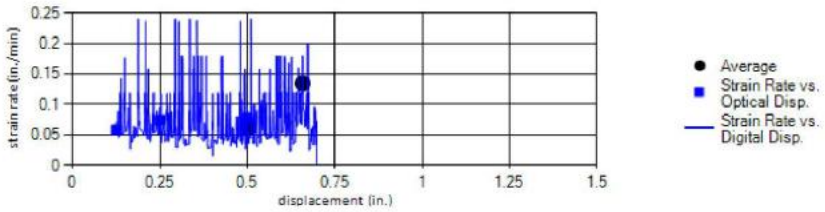
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(Peak)	Jack(0.75)
1,393 ±9	1,664 ±72	No Data	1,414 ±7	1,906 ±9	1,594 ±212	1,527 ±4	1,534 ±2	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.06 ±0.02	0.08 ±0.03	
At Peak Load	0.13 ±0.04	0.08 ±0.04	0.08 ±0.04
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
At Peak Load	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 15, 2011
Test Identification:	TS28-L19-Z12-L4-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



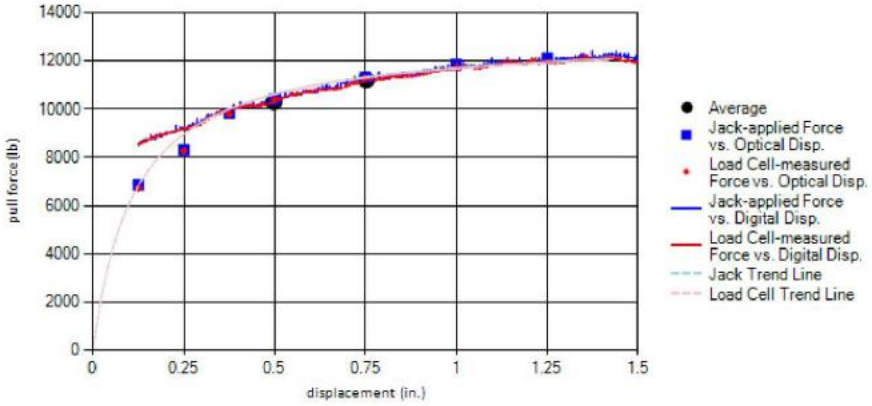
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	121.5	±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.9%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	97.6	±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	1,505 ±4	10,296 ±28	11.8 ±0.0	**2.57 ±0.01**
0.75	0.75 ±0.00	1,540 ±4	11,212 ±28	12.1 ±0.0	**2.73 ±0.01**
At Peak Load	1.43 ±0.00	1,517 ±6	12,289 ±59	No Data	**No Data**

Load-Displacement Curve



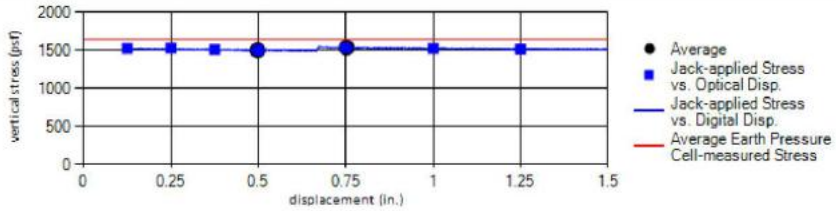
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, RH
	Prepared:	TW
	Checked:	WL, PJ

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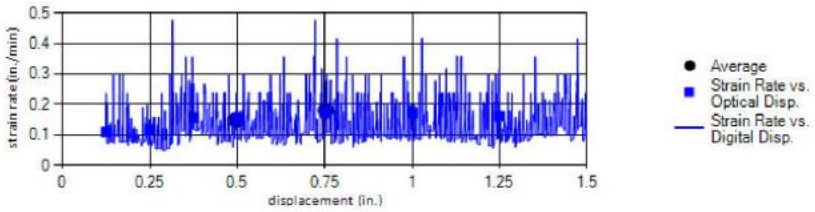
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
1,469 ±2	1,892 ±7	No Data	1,370 ±3	1,862 ±5	1,648 ±232	1,505 ±4	1,540 ±4	1,517 ±6



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.15 ±0.04	0.15 ±0.05	
0.75	0.18 ±0.05	0.17 ±0.05	0.16 ±0.07
At Peak Load	0.18 ±0.04	0.16 ±0.06	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 15, 2011
Test Identification:	TS28-L22-Z12-L4-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



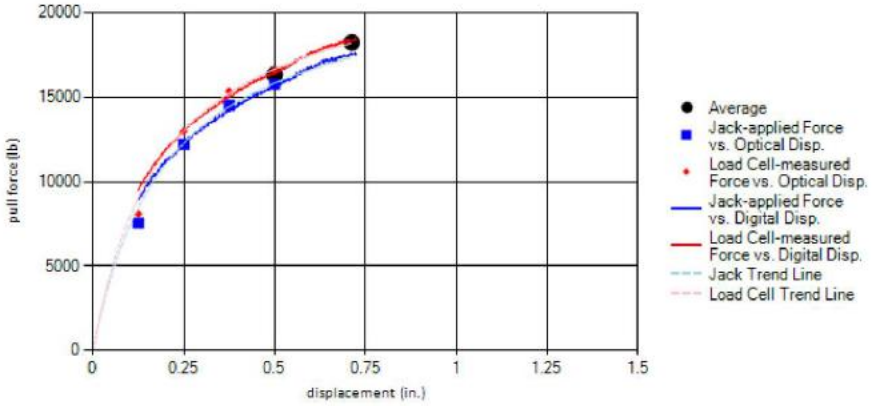
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	121.5	±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.9%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	97.6	±2.5%

Pullout Test Results

Nominal	Measured				Calculated		
Displacement (in.)	*Displacement (in.)*		*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*		*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50	±0.00	1,523 ±5	16,334	±23	11.9 ±0.0	**4.02 ±0.02**
At Peak Load	0.71	±0.00	1,520 ±2	18,234	±21	11.9 ±0.0	**4.50 ±0.01**
0.75	No	Data	No Data	No	Data	No Data	**No Data**

Load-Displacement Curve



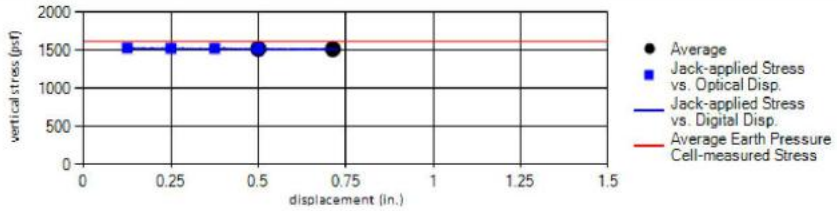
Comments	Personnel	
Rupture of longitudinal bar at 0.72in. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, RH
	Prepared:	TW
	Checked:	WL, PJ

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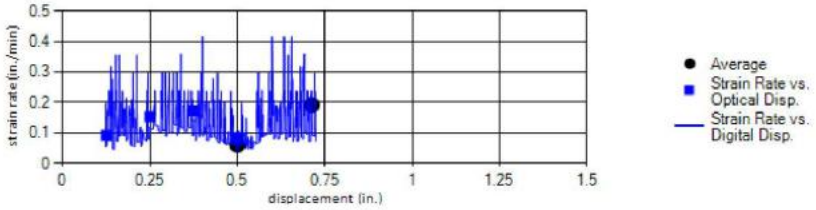
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(Peak)	Jack(0.75)
1,445 ±1	1,809 ±25	No Data	1,386 ±1	1,839 ±9	1,620 ±207	1,523 ±5	1,520 ±2	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.06 ±0.01	0.16 ±0.05	
At Peak Load	0.19 ±0.06	0.15 ±0.08	0.17 ±0.24
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
At Peak Load	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	July 01, 2010
Test Identification:	TS6-L13-Z12-L8-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



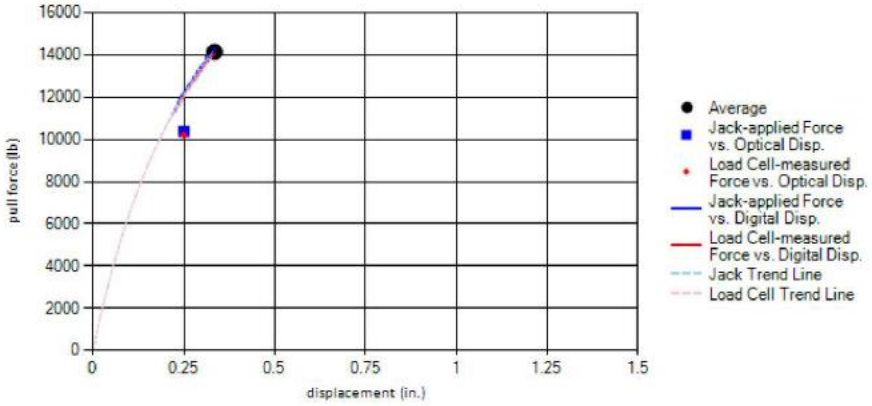
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	118.5	±0.3
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.2%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	95.2	±0.2%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
At Peak Load	0.33 ±0.04	1,526 ±4	14,142 ±750	12.0 ±0.0	**1.74 ±0.09**
0.50	No Data	No Data	No Data	No Data	**No Data**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



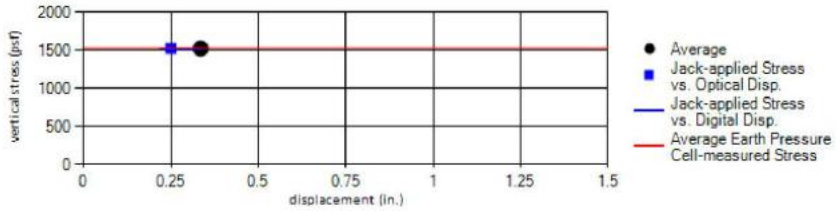
Comments	Personnel	
Rupture in connection at 0.34in. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, DH
	Prepared:	TW
	Checked:	WL, PJ

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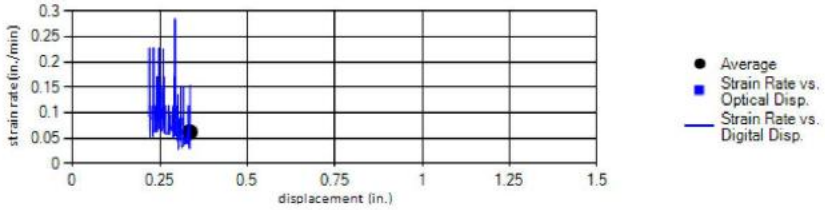
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(Peak)	Jack(0.50)	Jack(0.75)
1,600 ±4	1,630 ±51	1,560 ±4	1,396 ±0	1,497 ±3	1,536 ±87	1,526 ±4	No Data	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
At Peak Load	0.06 ±0.05	0.10 ±0.06	
0.50	No Data	No Data	0.10 ±0.06
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
At Peak Load		
0.50	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 16, 2011
Test Identification:	TS28-L3-Z12-L8-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

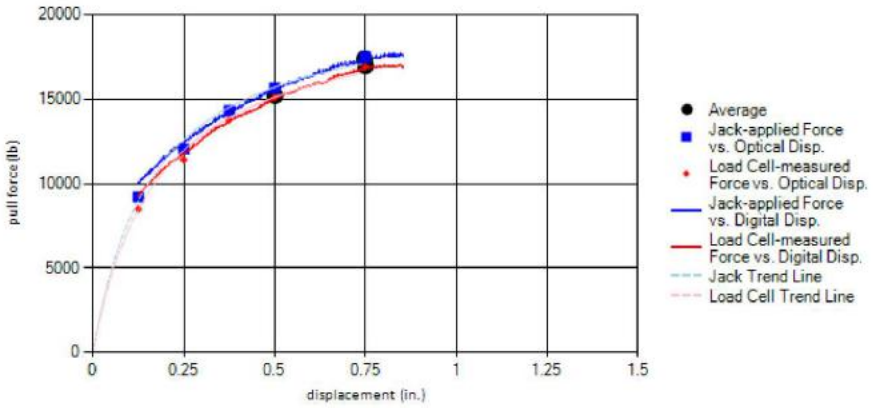
Backfill Material

Specification:	TxDOT Item 423 - Type B	In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)		*Dry Unit Weight, γ_d (pcf):*	121.5	±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Moisture Content, w (%):*	7.2	±0.9%
Optimum Moisture Content, OMC (%):	7.8%	*Relative Compaction, C_R (%):*	97.6	±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	1,538 ±3	15,196 ±23	12.1 ±0.0	**1.85 ±0.00**
At Peak Load	0.75 ±0.00	1,521 ±3	17,385 ±51	11.9 ±0.0	**2.14 ±0.01**
0.75	0.75 ±0.00	1,524 ±3	16,972 ±26	12.0 ±0.0	**2.09 ±0.01**

Load-Displacement Curve



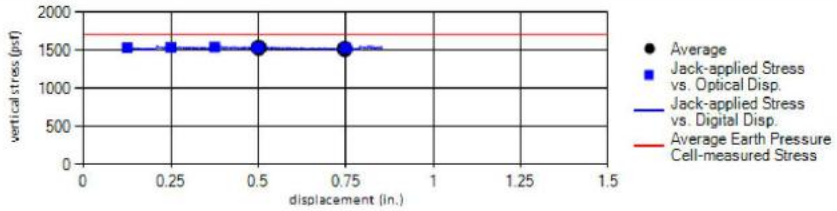
Comments	Personnel	
Rupture of longitudinal bar at 0.85in. Strain gage data collected. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, YW
	Prepared:	TW
	Checked:	WL, PJ

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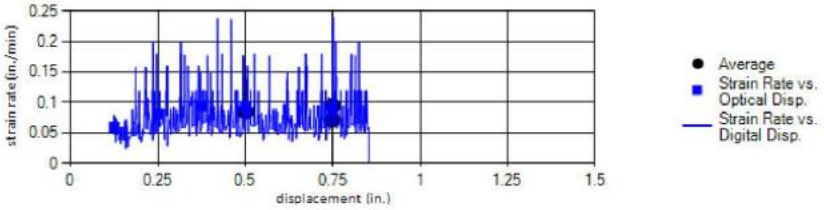
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(Peak)	Jack(0.75)
1,609 ±33	1,922 ±12	No Data	1,442 ±3	1,868 ±2	1,710 ±196	1,538 ±3	1,521 ±3	1,524 ±3



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.08 ±0.02	0.09 ±0.02	
At Peak Load	0.07 ±0.01	0.08 ±0.03	0.08 ±0.03
0.75	0.09 ±0.03	0.08 ±0.02	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
At Peak Load	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 13, 2011
Test Identification:	TS28-L12-Z12-L8-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

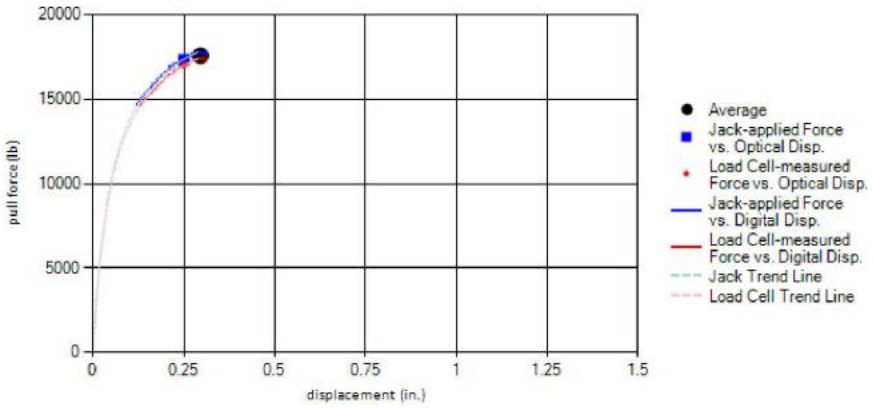
Backfill Material

Specification:	TxDOT Item 423 - Type B	In-Place Density (TEX-115-E)	
Moisture Density Relationship (TEX-114-E)		*Dry Unit Weight, γ_d (pcf):*	121.5 ±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Moisture Content, w (%):*	7.2 ±0.9%
Optimum Moisture Content, OMC (%):	7.8%	*Relative Compaction, C_R (%):*	97.6 ±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
At Peak Load	0.30 ±0.00	1,526 ±8	17,576 ±38	12.0 ±0.1	**2.16 ±0.01**
0.50	No Data	No Data	No Data	No Data	**No Data**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



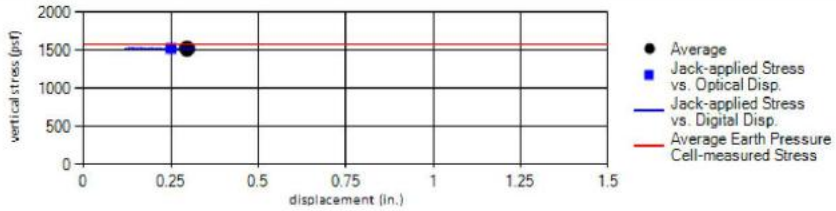
Comments	Personnel	
Rupture of longitudinal bar at 0.31in. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, AJ
	Prepared:	TW
	Checked:	WL, PJ

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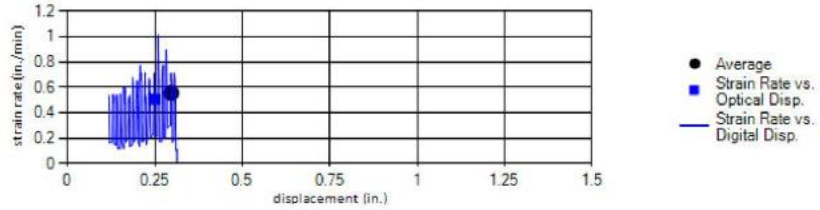
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(Peak)	Jack(0.50)	Jack(0.75)
1,433 ±31	1,659 ±15	No Data	1,438 ±14	1,799 ±16	1,582 ±157	1,526 ±8	No Data	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
At Peak Load	0.56 ±0.22	0.45 ±0.22	
0.50	No Data	No Data	0.44 ±0.24
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
At Peak Load		
0.50	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 16, 2011
Test Identification:	TS28-L13-Z12-L8-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



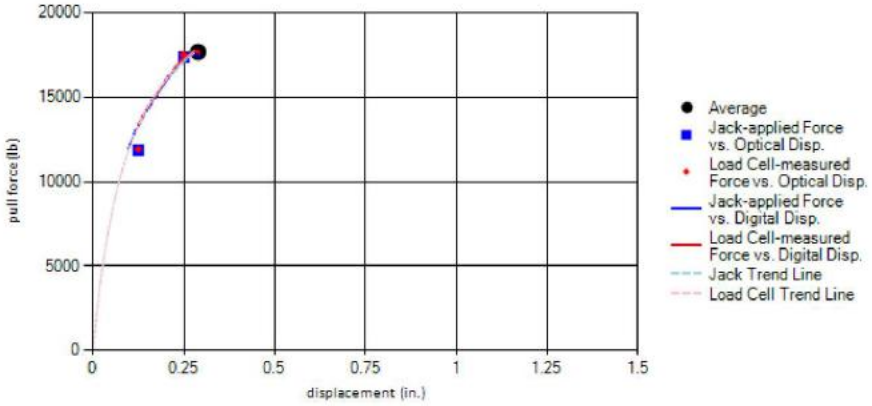
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	121.5	±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.9%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	97.6	±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
At Peak Load	0.29 ±0.00	1,526 ±4	17,674 ±36	12.0 ±0.0	**2.17 ±0.01**
0.50	No Data	No Data	No Data	No Data	**No Data**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



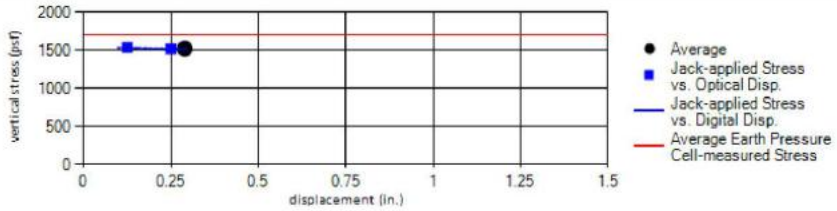
Comments	Personnel	
Rupture of longitudinal bar at 0.29in. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, YW
	Prepared:	TW
	Checked:	WL, PJ

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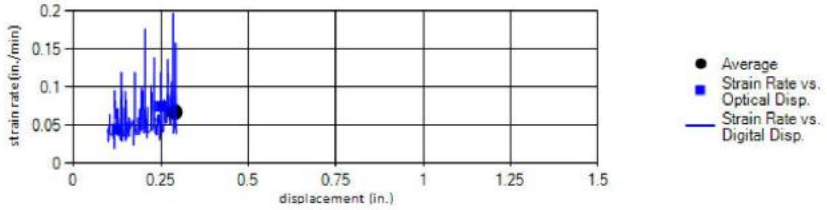
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(Peak)	Jack(0.50)	Jack(0.75)
1,529 ±5	1,916 ±10	No Data	1,474 ±3	1,904 ±3	1,706 ±206	1,526 ±4	No Data	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
At Peak Load	0.07 ±0.02	0.06 ±0.03	
0.50	No Data	No Data	0.06 ±0.03
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
At Peak Load		
0.50	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):	SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*	8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*	7.9	*Internal Friction Angle, φ (deg.):*	43°

Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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REPORT OF MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 15, 2011
Test Identification:	TS28-L17-Z20-L4-T
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



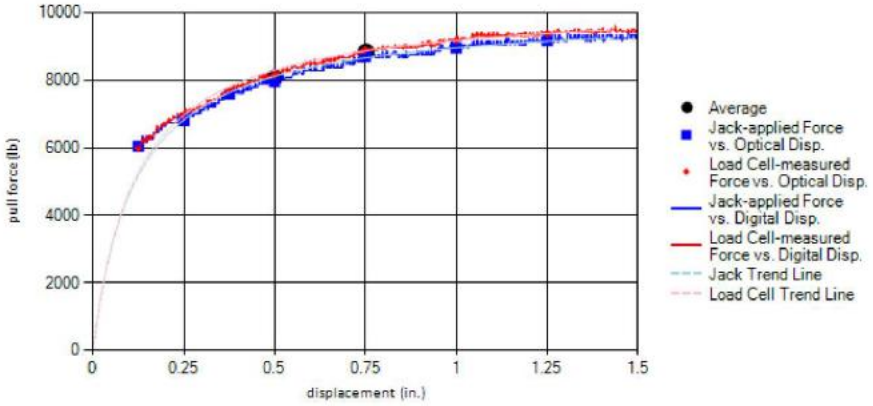
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	121.5	±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.9%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	97.6	±2.5%

Pullout Test Results

Nominal	Measured				Calculated		
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****	
0.50	0.50 ±0.00	2,475 ±3	8,081 ±27	19.4 ±0.0	**1.22**	**±0.01**
0.75	0.75 ±0.00	2,544 ±4	8,847 ±25	20.0 ±0.0	**1.30**	**±0.01**
At Peak Load	1.44 ±0.00	2,505 ±5	9,504 ±75	No Data	**No**	**Data**

Load-Displacement Curve



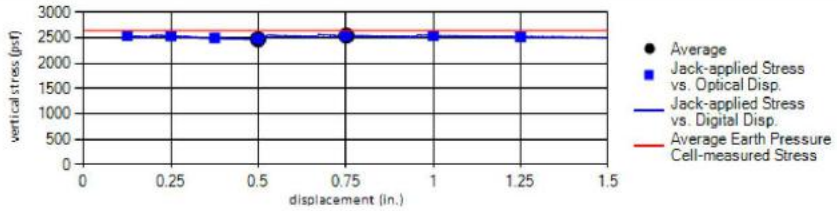
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	TW, RH, AS
	Prepared:	TW
	Checked:	WL, PJ

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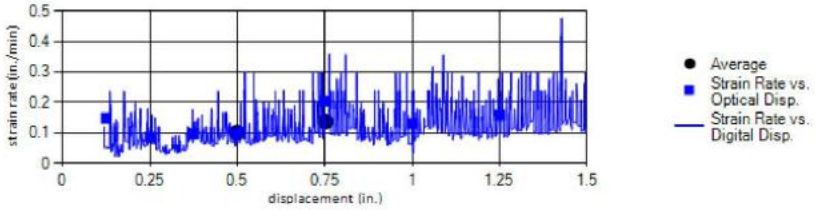
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
2,436 ±18	2,825 ±22	No Data	2,263 ±17	3,068 ±29	2,648 ±318	2,475 ±3	2,544 ±4	2,505 ±5



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.10 ±0.02	0.09 ±0.03	
0.75	0.14 ±0.03	0.14 ±0.04	0.14 ±0.07
At Peak Load	0.21 ±0.06	0.18 ±0.07	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 15, 2011
Test Identification:	TS28-L20-Z20-L4-M
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



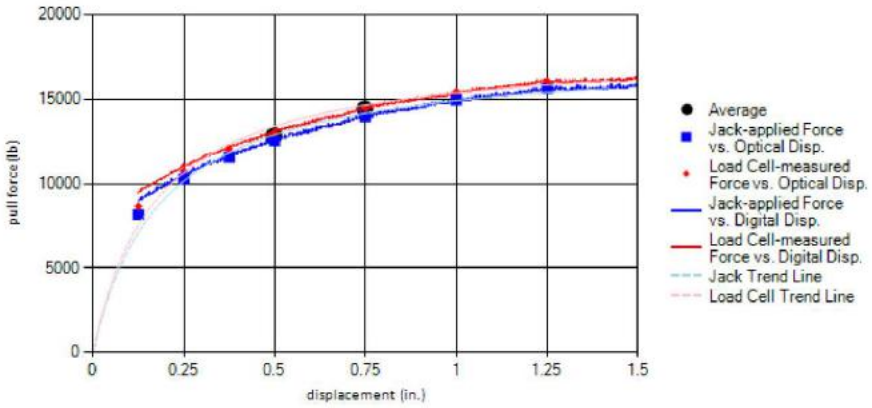
MSE Reinforcement	
Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

Backfill Material					
Specification:	TxDOT Item 423 - Type B		In-Place Density (TEX-115-E)		
Moisture Density Relationship (TEX-114-E)			*Dry Unit Weight, γ_d (pcf):*	121.5	±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5		*Moisture Content, w (%):*	7.2	±0.9%
Optimum Moisture Content, OMC (%):	7.8%		*Relative Compaction, C_R (%):*	97.6	±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	2,577 ±4	12,855 ±35	20.2 ±0.0	**1.87 ±0.01**
0.75	0.75 ±0.00	2,552 ±3	14,425 ±20	20.0 ±0.0	**2.12 ±0.01**
At Peak Load	1.43 ±0.00	2,575 ±5	16,119 ±33	No Data	**No Data**

Load-Displacement Curve



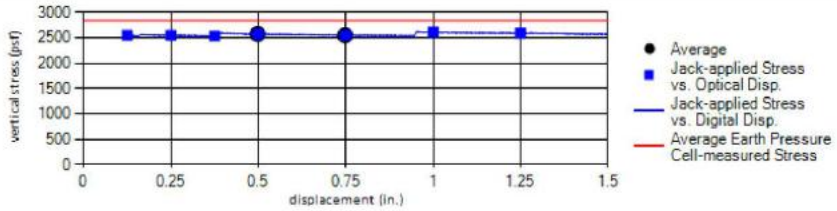
Comments	Personnel	
No incidental skew data collected. No incidental skew data.	*Tested:*	TW, AS, RH
	Prepared:	TW
	Checked:	WL, PJ

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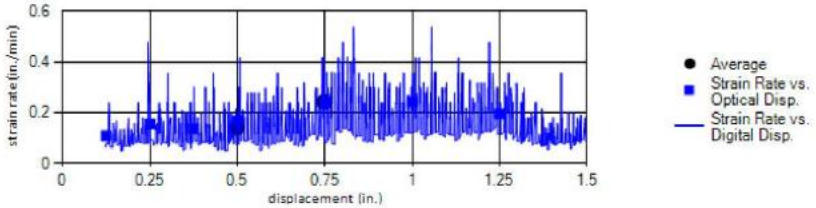
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(0.75)	Jack(Peak)
2,659 ±57	3,026 ±40	No Data	2,294 ±47	3,375 ±24	2,838 ±407	2,577 ±4	2,552 ±3	2,575 ±5



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.14 ±0.04	0.15 ±0.04	
0.75	0.24 ±0.07	0.18 ±0.05	0.17 ±0.08
At Peak Load	0.17 ±0.09	0.17 ±0.08	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
0.75	No Incidental Skew Data	
At Peak Load		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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TEXAS TECH UNIVERSITY
Center for Multidisciplinary Research in Transportation
Reinforced Earth Company: MSE Pullout

REPORT OF
MSE REINFORCEMENT PULLOUT TEST

Test Information	
Test Date:	June 15, 2011
Test Identification:	TS28-L23-Z20-L4-B
Test Facility:	12'x12'x4' MSE Test Box

Test Specimen Sketch



MSE Reinforcement

Type:	RECo HA Ladder
Length, L_e (ft):	4
Width, b (in.):	4
Skew Angle, β (°):	0

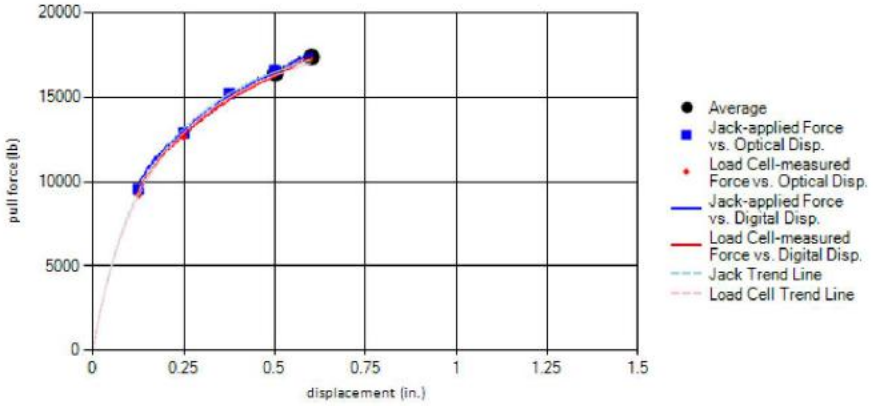
Backfill Material

Specification:	TxDOT Item 423 - Type B	In-Place Density (TEX-115-E)	
Moisture Density Relationship (TEX-114-E)		*Dry Unit Weight, γ_d (pcf):*	121.5 ±3.2
Maximum Dry Unit Weight, γ_{dmax} (pcf):	124.5	*Moisture Content, w (%):*	7.2 ±0.9%
Optimum Moisture Content, OMC (%):	7.8%	*Relative Compaction, C_R (%):*	97.6 ±2.5%

Pullout Test Results

Nominal	Measured			Calculated	
Displacement (in.)	*Displacement (in.)*	*Vertical Stress, σ_v (psf)*	*Pullout Resistance, P_r (lb)*	*Effective Fill Depth, z_{eff} (ft)*	***Pullout Resistance Factor, F****
0.50	0.50 ±0.00	2,549 ±2	16,406 ±26	20.0 ±0.0	**2.41 ±0.01**
At Peak Load	0.60 ±0.00	2,572 ±2	17,370 ±47	20.2 ±0.0	**2.53 ±0.01**
0.75	No Data	No Data	No Data	No Data	**No Data**

Load-Displacement Curve



Comments	Personnel	
Rupture of longitudinal bar at 0.61in. No incidental skew data collected. No incidental skew data.	*Tested:*	TW, RH, AS
	Prepared:	TW
	Checked:	WL, PJ

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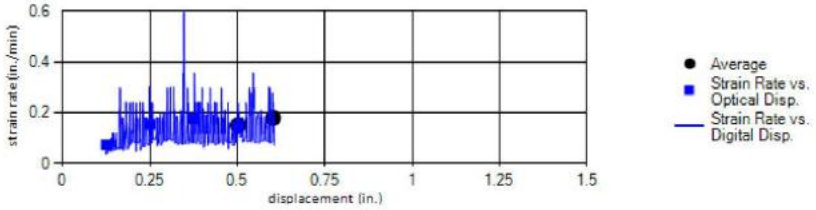
Vertical Stress, σ_v (psf)

EPC0	EPC1	EPC2	EPC3	EPC4	Ave. EPC	Jack(0.50)	Jack(Peak)	Jack(0.75)
2,445 ±10	2,918 ±16	No Data	2,284 ±9	2,931 ±27	2,644 ±287	2,549 ±2	2,572 ±2	No Data



Strain Rate

nominal displacement (in.)	*instantaneous rate (in./min)*	*rate for ¼" preceding displacement (in./min)*	*overall average rate (in./min)*
0.50	0.15 ±0.03	0.16 ±0.05	
At Peak Load	0.18 ±0.06	0.16 ±0.06	0.14 ±0.07
0.75	No Data	No Data	



Incidental Skew

nominal displacement (in.)	*skew angle (degrees)*	*skew direction*
0.50		
At Peak Load	No Incidental Skew Data	
0.75		

Backfill Material Properties

Unified Soil Classification (ASTM D 2487):		SP-SM	Shear Strength Properties (ASTM D 3080)	
Resistivity (TEX-129-E) *(ohm-cm):*		8,004	*Cohesion, c (psf):*	112
Soil pH (TEX-128-E)*:*		7.9	*Internal Friction Angle, φ (deg.):*	43°
Gradation	TEX-110-E	ASTM D422 D1140	Atterberg Limits (TEX-104-E, TEX-105-E, TEX-106-E, TEX-107-E)	
Sieve Size	*Percent Retained*	*Percent Retained*		
3"	0.0%	0.0%	*Liquid Limit, LL (%):*	non-plastic
No. 4	1.5%	0.8%	*Plastic Limit, PL (%):*	non-plastic
No. 40	28.3%	33.7%	*Plasticity Index, PI (%)*	non-plastic
No. 200	90.6%	91.4%	*Bar Linear Shrinkage, LS (%)*	2

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Appendix C:

Strain Gage Data for MSE Reinforcement Pullout Tests:

Reinforced Earth Company High Adherence Strips in Type B Backfill

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TABLE C: STRAIN GAGE DATA FOR MSE REINFORCEMENT PULLOUT TESTS
REINFORCED EARTH COMPANY HIGH ADHERENCE STRIPS IN TYPE B BACKFILL

TestID	Length, Le (ft)	Width, b (in.)	Skew, β (°)	Relative Compaction, CR (%)
TS29-S9-Z5-L8-M	8	2	0	97.8%
TS29-S13-Z5-L8-B	8	2	0	95.9%
TS29-S1-Z5-L12-T	12	2	0	99.9%
TS29-S5-Z5-L12-T	12	2	0	99.9%
TS29-S10-Z12-L8-M	8	2	0	97.8%
TS29-S14-Z12-L8-B	8	2	0	95.9%
TS29-S2-Z12-L12-T	12	2	0	99.9%
TS29-S6-Z12-L12-M	12	2	0	97.8%
TS29-S11-Z20-L8-B	8	2	0	95.9%
TS29-S15-Z20-L8-B	8	2	0	95.9%
TS29-S3-Z20-L12-T	12	2	0	99.9%
TS29-S7-Z20-L12-M	12	2	0	97.8%
TS29-S12-Z40-L8-B	8	2	0	95.9%
TS29-S16-Z40-L8-B	8	2	0	95.9%
TS29-S4-Z40-L12-T	12	2	0	99.9%
TS29-S8-Z40-L12-M	12	2	0	97.8%

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STRAIN GAGE REPORT FOR MSE REINFORCEMENT PULLOUT TEST

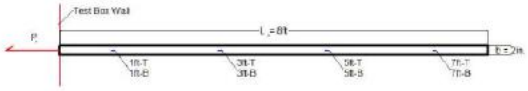
Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S9-Z5-L8-M
Test Facility:	12'x12'x4' MSE Test Box

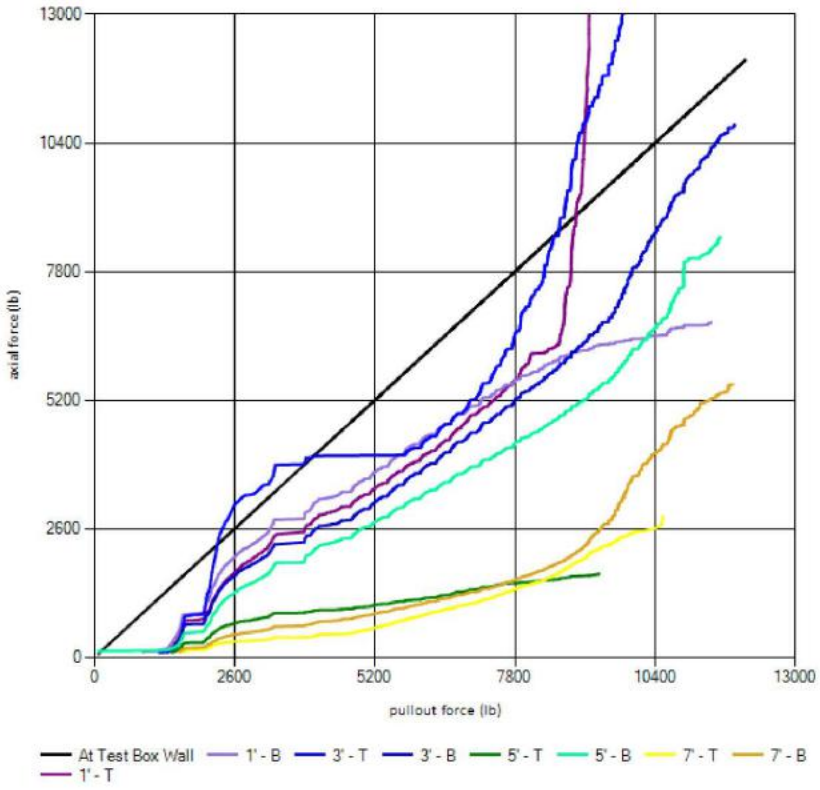
MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	8
Width, b (in.):	2
Skew Angle, θ (°):	0

Test Specimen Sketch



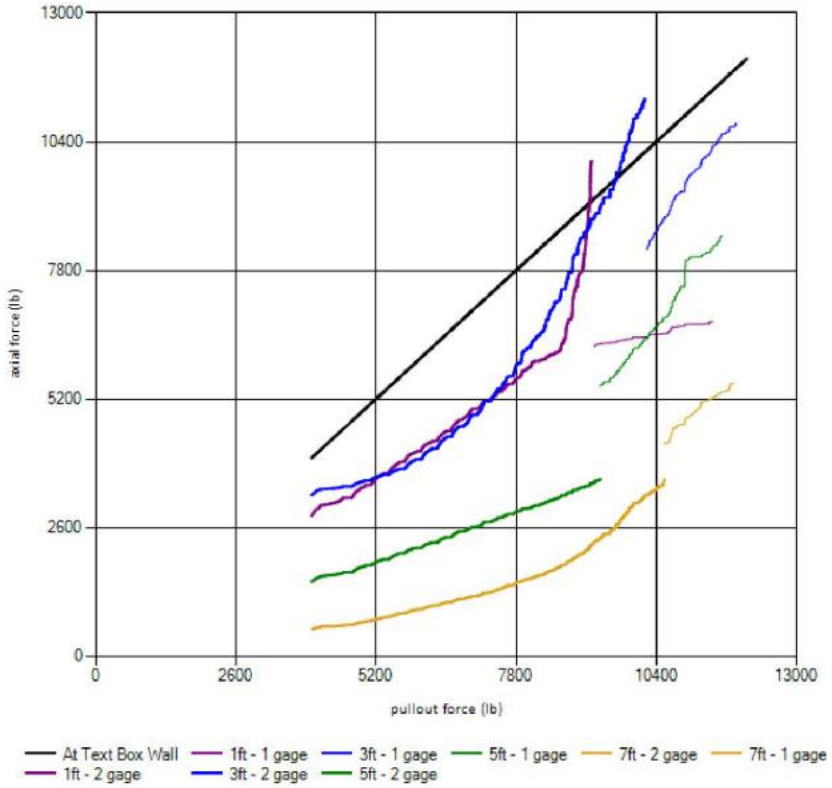
Axial Load-Pullout Load Data by Gage



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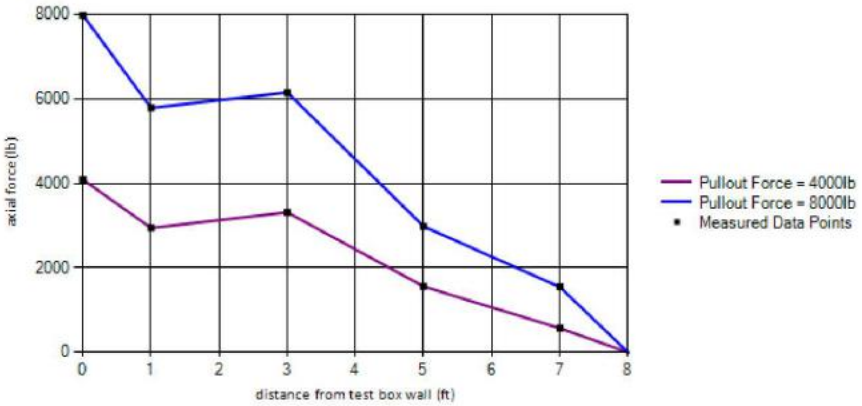
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

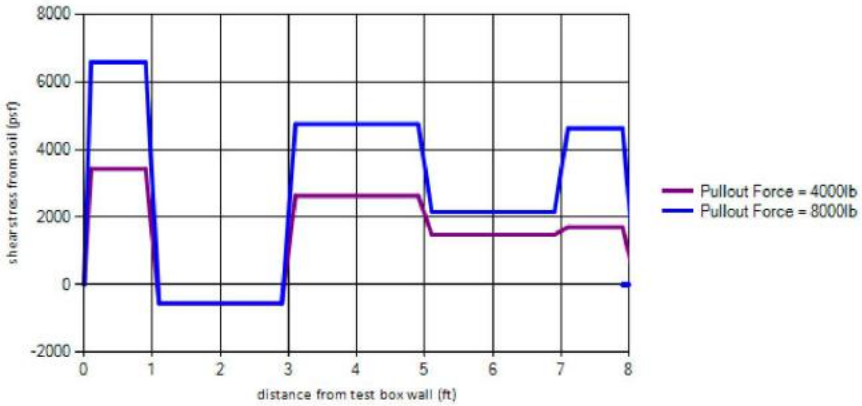
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	136*	104*	142*	111*			
2000	1002	1030	506	215			
3000	2203	2654	1166	440			
4000	2943	3314	1557	569			
5000	3436	3543	1821	698			
6000	4187	4023	2214	956			
7000	4973	4868	2606	1213			
8000	5785	6150	2980	1544			
9000	7946	8383	3406	2088			
10000	6426*	10543	6148*	3027			0.37
11000	6699*	9643*	7819*	4853*			0.71

*calculated from 1 strain gage.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	*Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall*								*Displacement (in.)*
	0-1ft	*1-3ft*	*3-5ft*	*5-7ft*	*7-9ft*	*9-11ft*	*11-12ft*	*Mean*	
1000	2923	49	-58	46	334			416	
2000	3066	-42	785	437	644			759	
3000	2397	-677	2232	1089	1320			1126	
4000	3429	-556	2635	1483	1706			1532	
5000	4699	-160	2584	1684	2093			1876	
6000	5479	246	2714	1887	2867			2255	
7000	6124	157	3394	2089	3639			2630	
8000	6586	-548	4755	2155	4631			2993	
9000	3209	-656	7466	1978	6264			3381	
10000	10644	-6175	6593	4681	9082			3740	0.37
11000	12933	-4416	2736	4450	14558			4129	0.71

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STRAIN GAGE REPORT FOR MSE REINFORCEMENT PULLOUT TEST

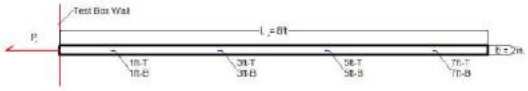
Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S13-Z5-L8-B
Test Facility:	12'x12'x4' MSE Test Box

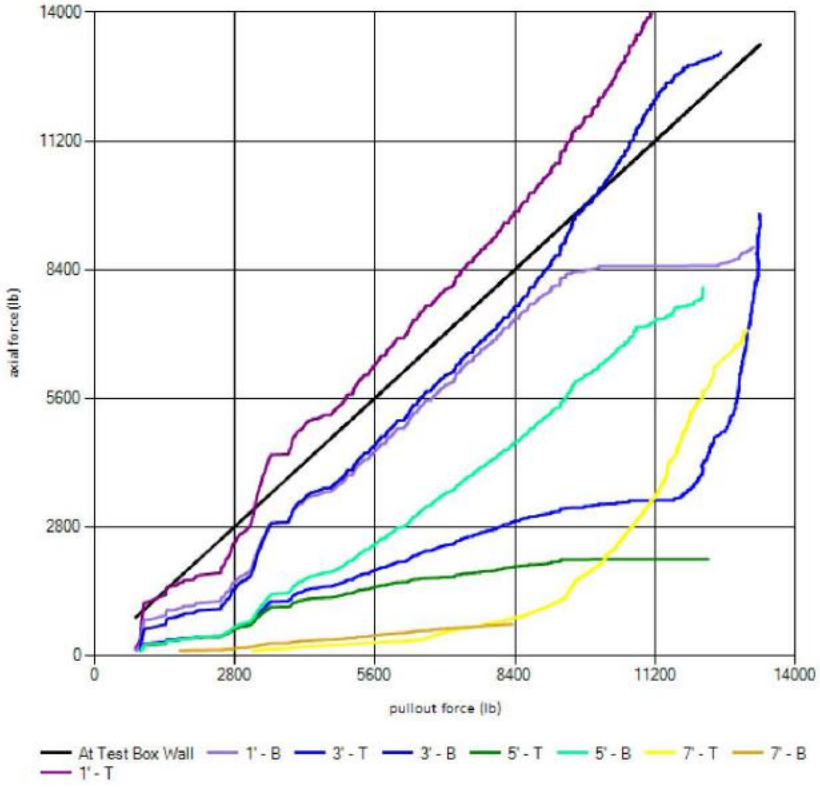
MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	8
Width, b (in.):	2
Skew Angle, θ (°):	0

Test Specimen Sketch



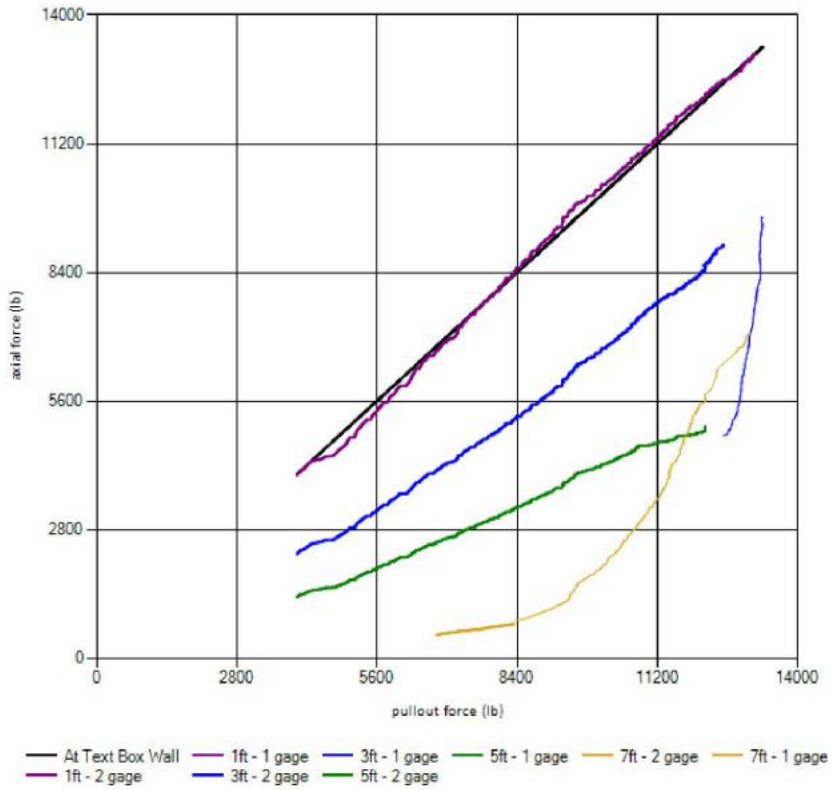
Axial Load-Pullout Load Data by Gage



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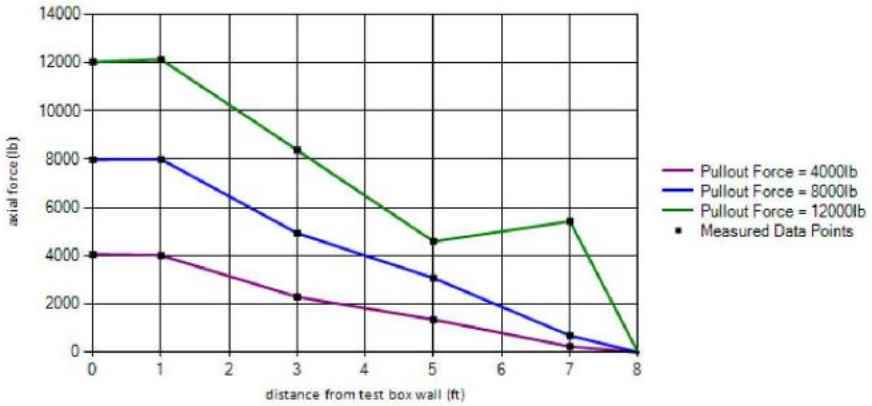
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

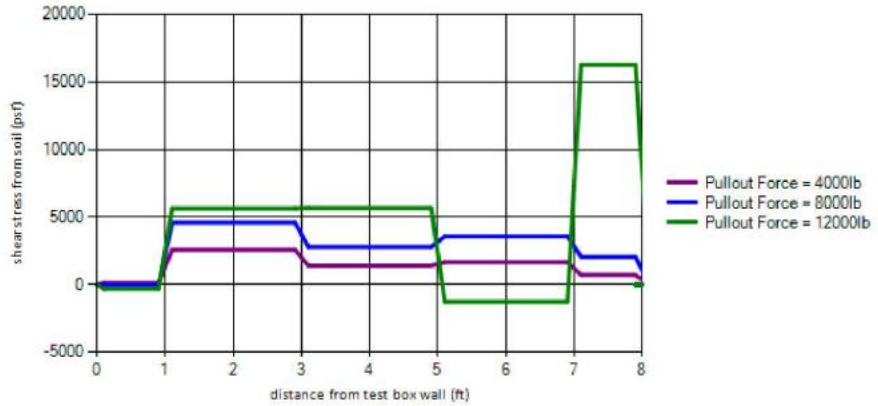
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	609	317	180	102*			
2000	1388	663	376	112*			
3000	2285	1174	683	159*			
4000	4014	2292	1353	238			
5000	4700	2771	1655	305			
6000	5810	3499	2138	394			
7000	6886	4220	2600	541			
8000	7995	4932	3073	689			
9000	9107	5707	3579	1035*			
10000	10208	6660	4174	1937*			0.26
11000	11115	7547	4633	3147*			0.38
12000	12134	8371	4589	5424*			0.68
13000	14144	6939*	No Data	6839*			1.17

*calculated from 1 strain gage.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall								Displacement (in.)
	0-1ft	1-3ft	3-5ft	5-7ft	7-9ft	9-11ft	11-12ft	Mean	
1000	991	439	205	117	306			352	
2000	1658	1088	430	396	337			728	
3000	2147	1667	735	787	476			1125	
4000	133	2583	1410	1671	715			1522	
5000	894	2893	1675	2024	916			1874	
6000	547	3466	2042	2616	1181			2247	
7000	305	3999	2429	3090	1622			2620	
8000	-36	4594	2788	3577	2066			2994	
9000	-395	5100	3192	3815	3106			3366	
10000	-564	5322	3728	3355	5812			3758	0.26
11000	-392	5351	4371	2230	9440			4119	0.38
12000	-326	5644	5674	-1253	16273			4509	0.68
13000	-3343	10808	75	75	20518			4886	1.17

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STRAIN GAGE REPORT FOR MSE REINFORCEMENT PULLOUT TEST

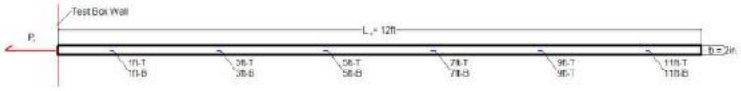
Test Information

Test Date:	June 20, 2011
Test Identification:	TS29-S1-Z5-L12-T
Test Facility:	12'x12'x4' MSE Test Box

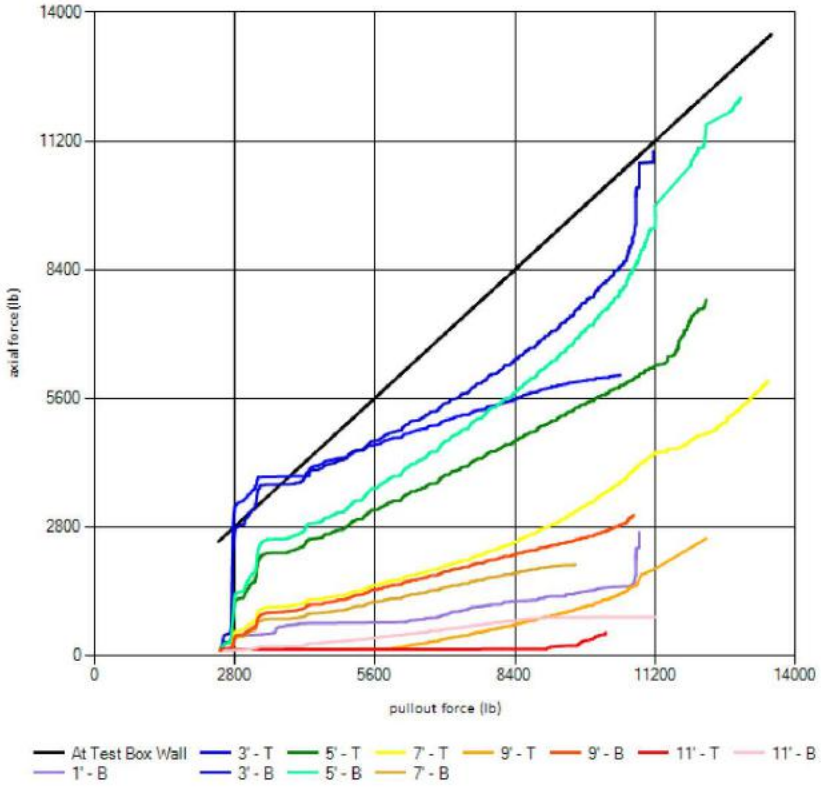
MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	12
Width, b (in.):	2
Skew Angle, θ (°):	0

Test Specimen Sketch



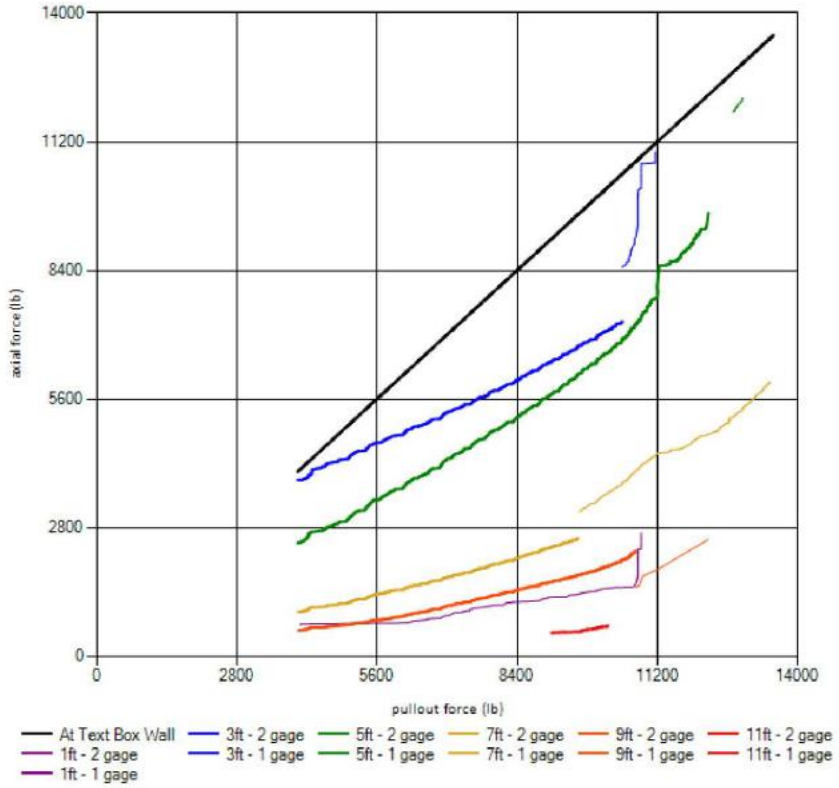
Axial Load-Pullout Load Data by Gage



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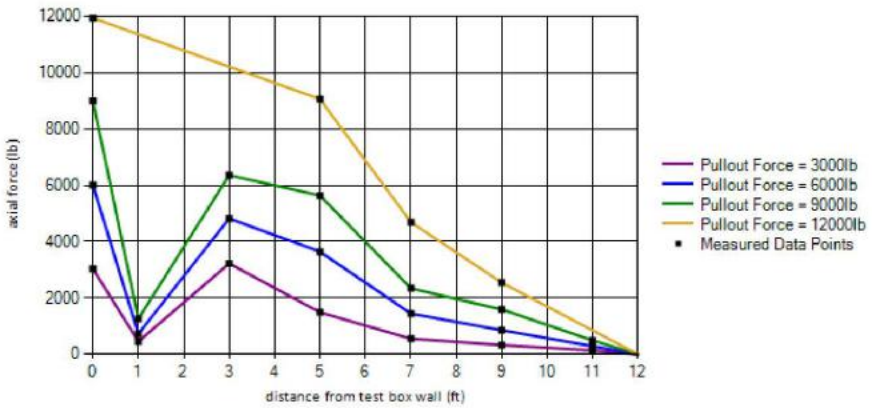
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

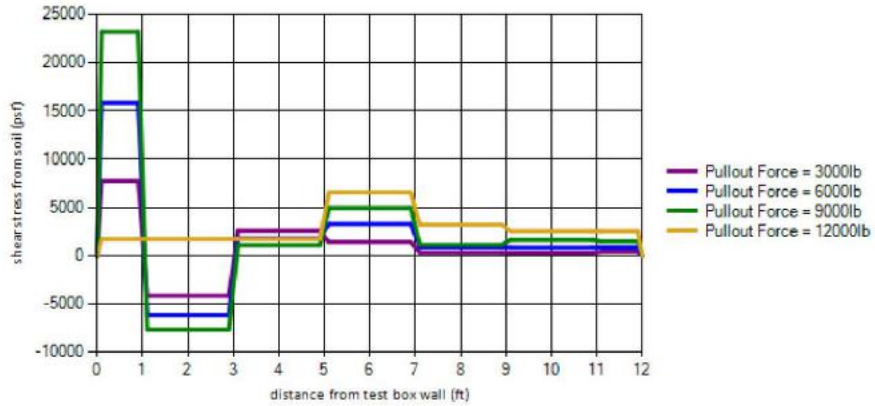
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	-1559*	127*	119*	102*	116	111	0.31
2000	-1559*	127*	119*	102*	116	111	0.31
3000	455*	3221	1488	552	330	138	
4000	709*	3874	2540	990	580	176	
5000	724*	4345	3014	1176	691	231	
6000	725*	4820	3642	1444	854	288	
7000	898*	5300	4282	1716	1092	356	
8000	1114*	5794	4934	2007	1337	425	
9000	1260*	6361	5635	2341	1593	491	
10000	1446*	6969	6448	3404*	1886	629	
11000	2037*	10334*	7501	4196*	1768*	835*	0.35
12000	No Data	No Data	9061	4690*	2543*	No Data	0.65

*calculated from 1 strain gage.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	*Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall*								*Displacement (in.)*
	0-1ft	*1-3ft*	*3-5ft*	*5-7ft*	*7-9ft*	*9-11ft*	*11-12ft*	*Mean*	
1000	12156	-2529	12	26	-21	7	334	623	0.31
2000	12156	-2529	12	26	-21	7	334	623	0.31
3000	7735	-4149	2600	1404	333	287	415	758	
4000	10253	-4747	2000	2325	616	606	527	1032	
5000	12892	-5431	1997	2756	729	690	692	1255	
6000	15844	-6142	1767	3298	885	849	864	1502	
7000	18373	-6604	1527	3850	937	1104	1068	1755	
8000	20628	-7020	1290	4390	1005	1368	1276	1997	
9000	23194	-7651	1088	4941	1123	1652	1474	2248	
10000	25685	-8284	781	4566	2277	1886	1887	2502	
11000	26731	-12445	4249	4957	3643	1398	2506	2737	0.35
12000	1727	1727	1727	6556	3221	2543	2543	2985	0.65

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STRAIN GAGE REPORT FOR MSE REINFORCEMENT PULLOUT TEST

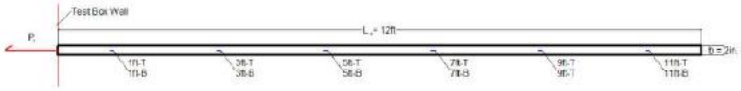
Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S5-Z5-L12-T
Test Facility:	12'x12'x4' MSE Test Box

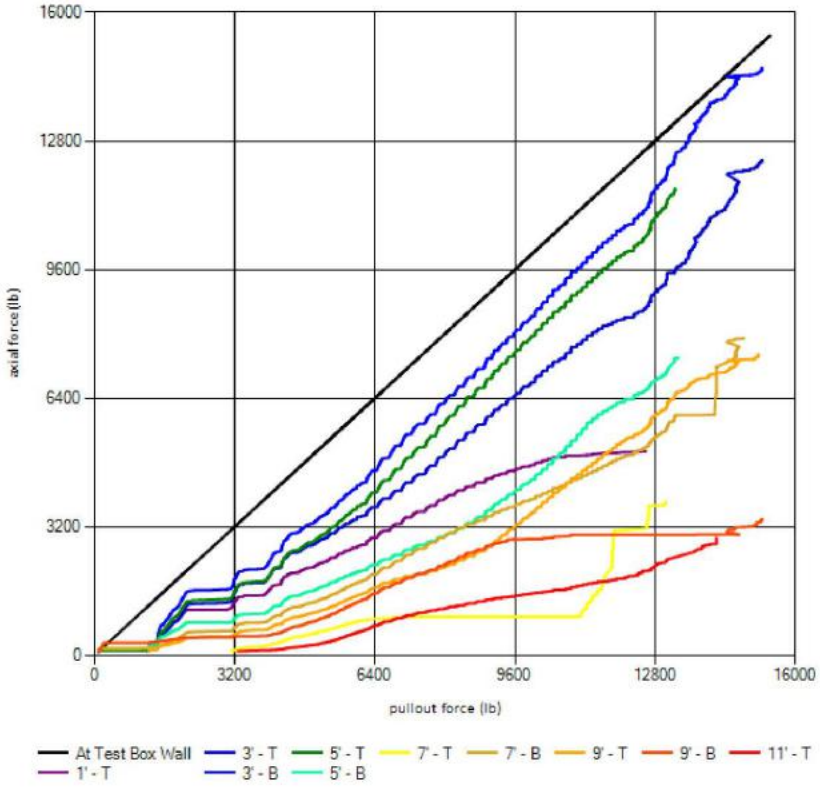
MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	12
Width, b (in.):	2
Skew Angle, θ (°):	0

Test Specimen Sketch



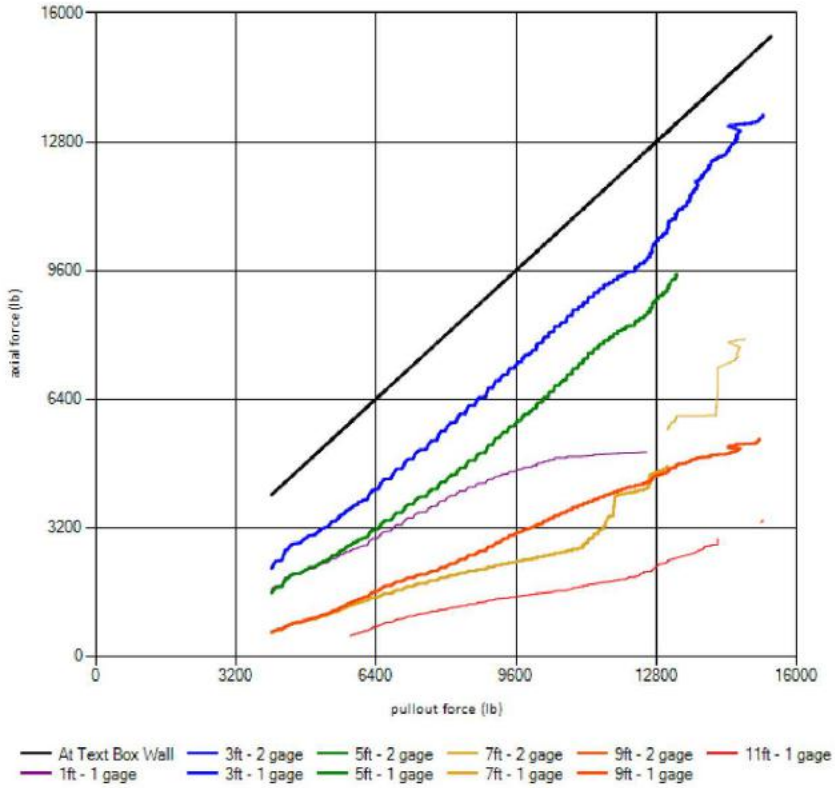
Axial Load-Pullout Load Data by Gage



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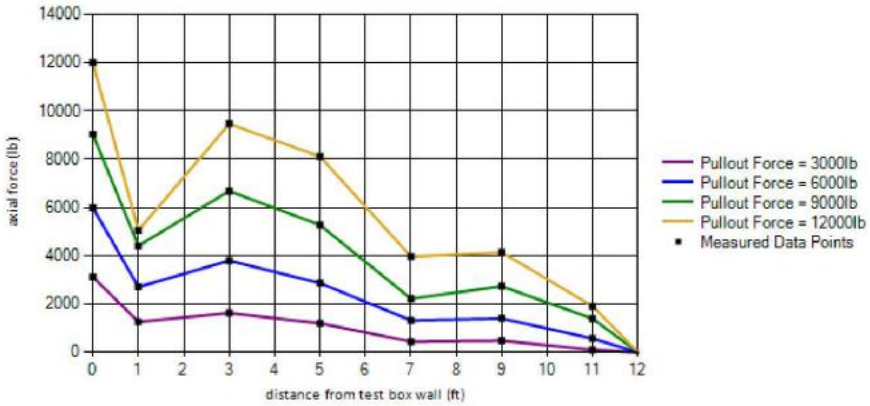
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

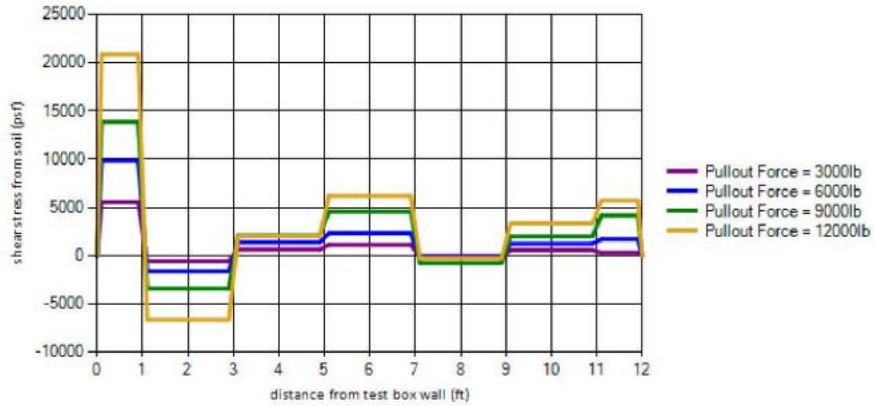
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	133*	134	156	104*	246	102*	
2000	984*	1189	925	481*	399	102*	
3000	1262*	1623	1199	444	484	102*	
4000	1653*	2193	1587	589	609	136*	
5000	2231*	3061	2252	945	962	268*	
6000	2704*	3793	2865	1320	1399	568*	
7000	3311*	4709	3611	1665	1871	929*	
8000	3916*	5669	4386	1970	2239	1177*	
9000	4408*	6680	5272	2219	2733	1391*	0.26
10000	4754*	7654	6187	2437	3233	1546*	0.31
11000	4974*	8667	7241	2715	3738	1720*	0.39
12000	5050*	9462	8109	3963	4139	1911*	0.47
13000	No Data	10652	9162	5234*	4606	2402*	0.65
14000	No Data	12144	No Data	6102*	4972	2727*	0.96

*calculated from 1 strain gage.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	*Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall*								*Displacement (in.)*
	0-1ft	*1-3ft*	*3-5ft*	*5-7ft*	*7-9ft*	*9-11ft*	*11-12ft*	*Mean*	
1000	3190	-1	-34	79	-213	216	306	299	
2000	3001	-307	396	666	122	446	306	496	
3000	5562	-541	636	1133	-59	572	306	779	
4000	7092	-809	908	1497	-30	710	408	1004	
5000	8346	-1245	1214	1960	-26	1041	805	1253	
6000	9838	-1634	1392	2319	-119	1246	1704	1496	
7000	10957	-2097	1647	2920	-310	1414	2787	1741	
8000	12249	-2630	1925	3625	-404	1593	3532	2000	
9000	13857	-3408	2113	4579	-771	2013	4174	2257	0.26
10000	15755	-4350	2201	5625	-1194	2530	4638	2502	0.31
11000	18152	-5540	2139	6789	-1534	3027	5159	2756	0.39
12000	20851	-6618	2030	6218	-264	3343	5732	3000	0.47
13000	2386	2386	2235	5892	942	3307	7205	3259	0.65
14000	1842	1842	4531	4531	1695	3368	8180	3496	0.96

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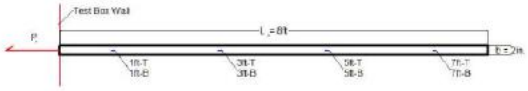
Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S10-Z12-L8-M
Test Facility:	12'x12'x4' MSE Test Box

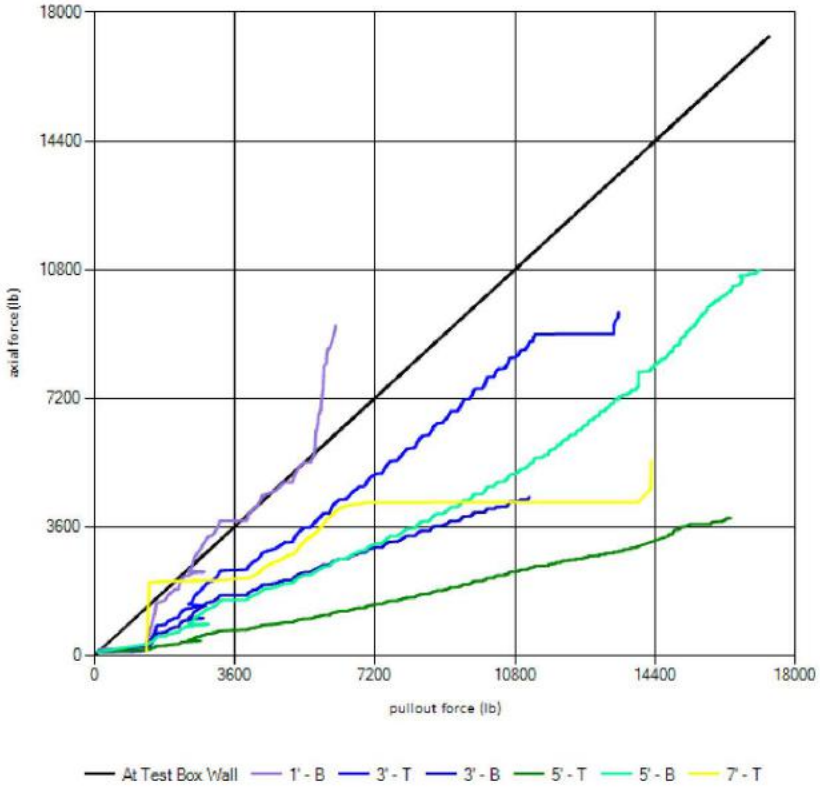
MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	8
Width, b (in.):	2
Skew Angle, θ (°):	0

Test Specimen Sketch



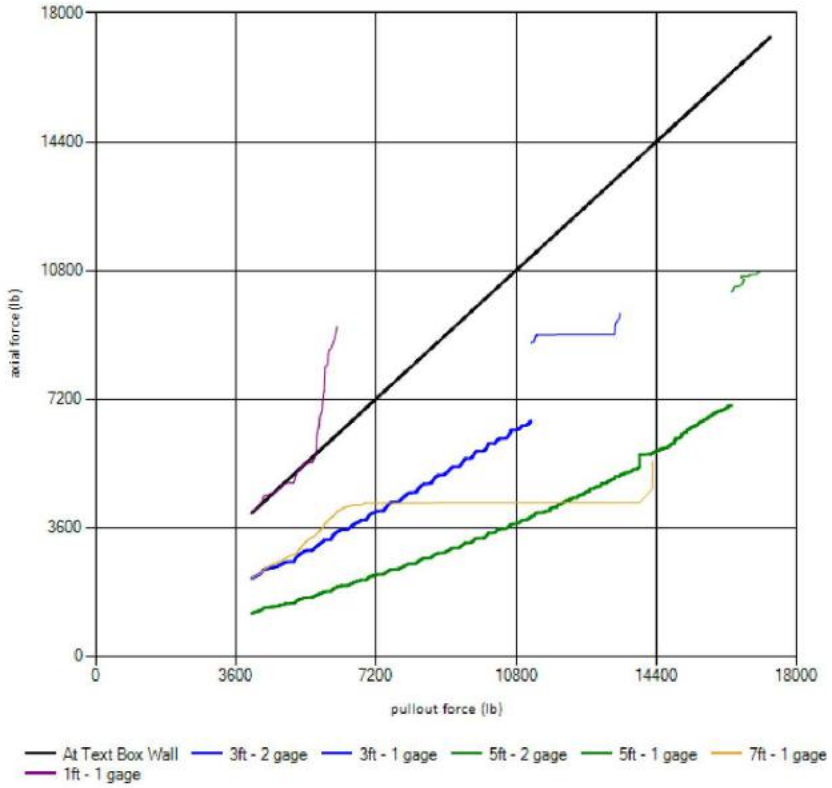
Axial Load-Pullout Load Data by Gage



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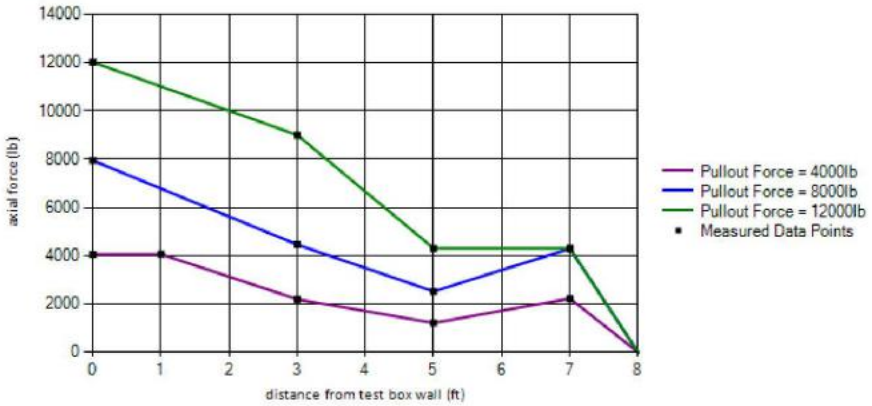
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

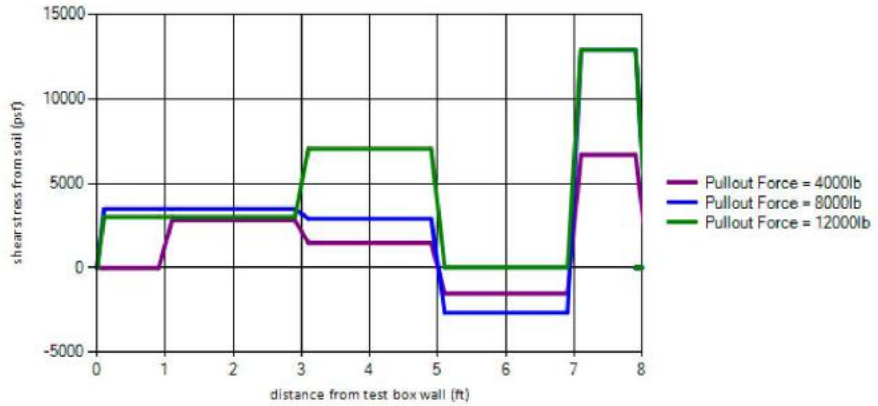
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	227*	175	223	102*			
2000	1823*	906	470	2080*			
3000	3163*	1679	912	2102*			
4000	4065*	2196	1216	2230*			
5000	4934*	2683	1494	2810*			
6000	8227*	3312	1842	3814*			
7000	No Data	3919	2198	4282*			
8000	No Data	4458	2520	4297*			
9000	No Data	5161	2946	4297*			
10000	No Data	5808	3354	4297*			
11000	No Data	6665	3810	4297*			
12000	No Data	9002*	4303	4297*			
13000	No Data	9002*	4817	4297*			0.26
14000	No Data	No Data	5249	4311*			0.33

*calculated from 1 strain gage.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall								Displacement (in.)
	0-1ft	1-3ft	3-5ft	5-7ft	7-9ft	9-11ft	11-12ft	Mean	
1000	2455	78	-72	181	306			392	
2000	638	1374	654	-2414	6240			763	
3000	-582	2225	1150	-1785	6307			1113	
4000	-23	2802	1471	-1521	6689			1521	
5000	202	3377	1784	-1975	8431			1876	
6000	-6667	7373	2205	-2959	11443			2252	
7000	3092	3092	2582	-3127	12847			2629	
8000	3482	3482	2907	-2666	12890			2978	
9000	3844	3844	3323	-2027	12890			3377	
10000	4175	4175	3680	-1414	12890			3743	
11000	4350	4350	4283	-730	12890			4131	
12000	3016	3016	7048	9	12890			4506	
13000	4026	4026	6277	780	12890			4885	0.26
14000	5172	5172	5172	1407	12934			5201	0.33

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STRAIN GAGE REPORT FOR MSE REINFORCEMENT PULLOUT TEST

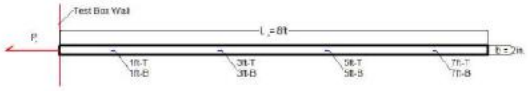
Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S14-Z12-L8-B
Test Facility:	12'x12'x4' MSE Test Box

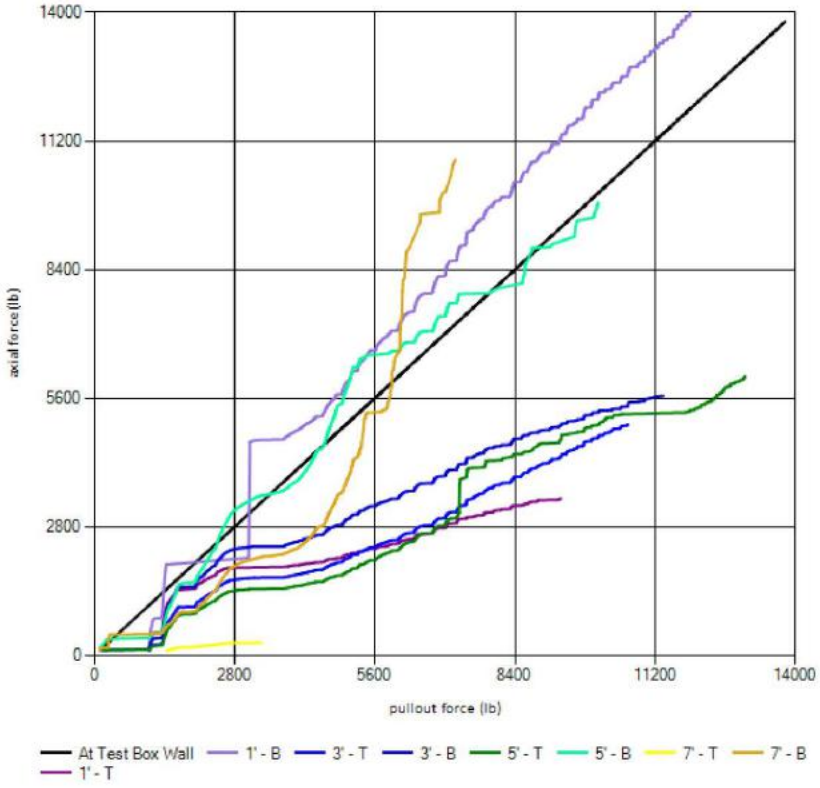
MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	8
Width, b (in.):	2
Skew Angle, θ (°):	0

Test Specimen Sketch



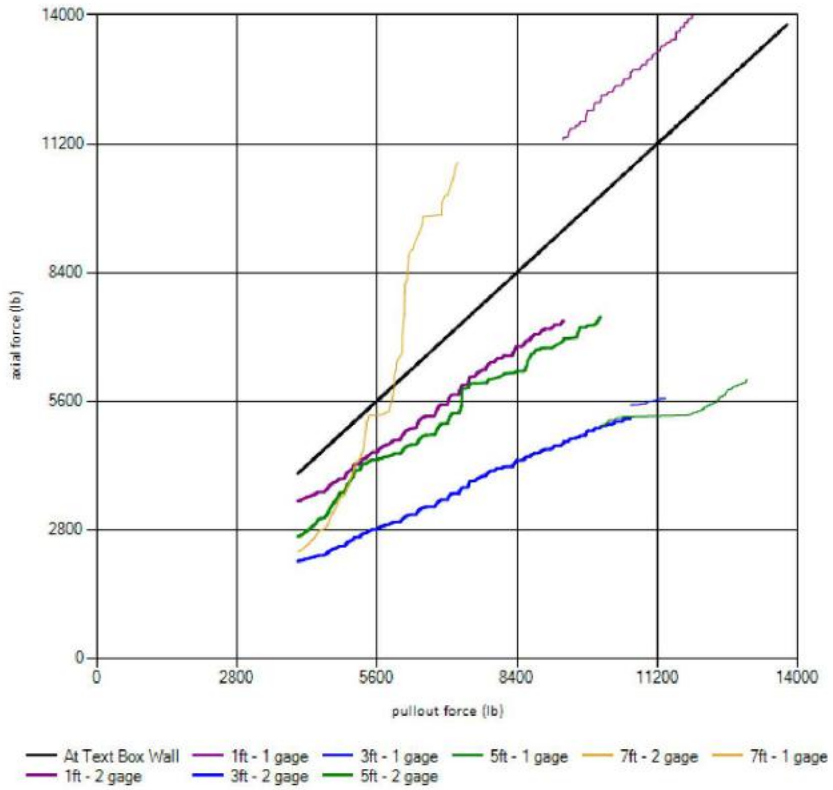
Axial Load-Pullout Load Data by Gage



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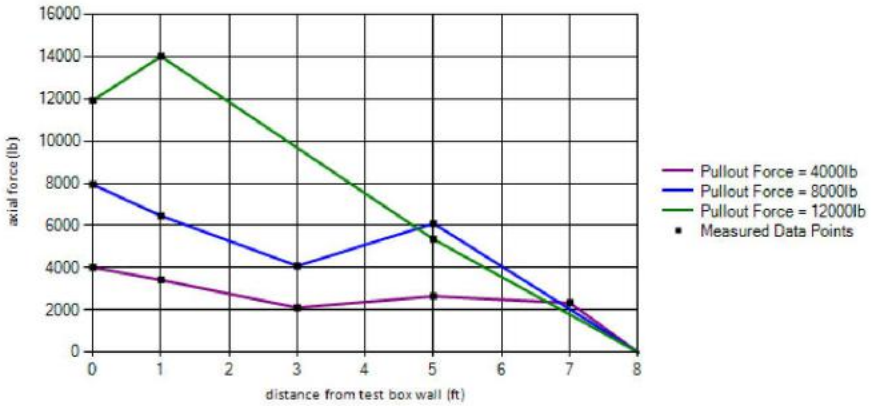
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

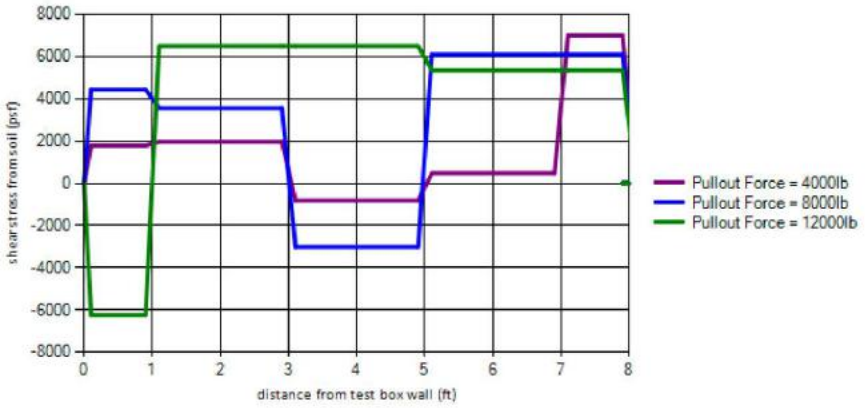
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	499	250	317	478*			
2000	1838	1406	1382	631			
3000	2430	2008	2362	1156			
4000	3423	2111	2652	2336*			
5000	4020	2507	3753	3731*			
6000	4802	3000	4492	6737*			
7000	5573	3478	5146	10052*			
8000	6458	4084	6096	No Data			
9000	7158	4540	6753	No Data			
10000	12054*	4992	6433	No Data			0.26
11000	13062*	5593*	5290*	No Data			0.33
12000	14011*	No Data	5347*	No Data			0.43

*calculated from 1 strain gage.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall								Displacement (in.)
	0-1ft	1-3ft	3-5ft	5-7ft	7-9ft	9-11ft	11-12ft	Mean	
1000	1983	374	-101	-243	1435			435	
2000	645	649	36	1126	1893			770	
3000	1682	633	-532	1810	3467			1122	
4000	1777	1968	-812	475	7008			1506	
5000	2904	2269	-1869	32	11194			1870	
6000	3667	2704	-2238	-3368	20212			2259	
7000	4209	3143	-2502	-7359	30156			2616	
8000	4443	3562	-3019	6096	6096			2977	
9000	5466	3926	-3320	6753	6753			3367	
10000	-6242	10593	-2162	6433	6433			3740	0.26
11000	-6012	11205	455	5290	5290			4147	0.33
12000	-6239	6498	6498	5347	5347			4474	0.43

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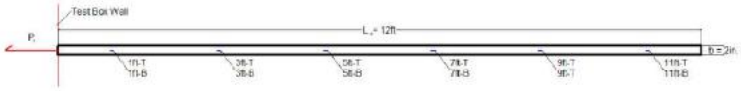
Test Information

Test Date:	June 20, 2011
Test Identification:	TS29-S2-Z12-L12-T
Test Facility:	12'x12'x4' MSE Test Box

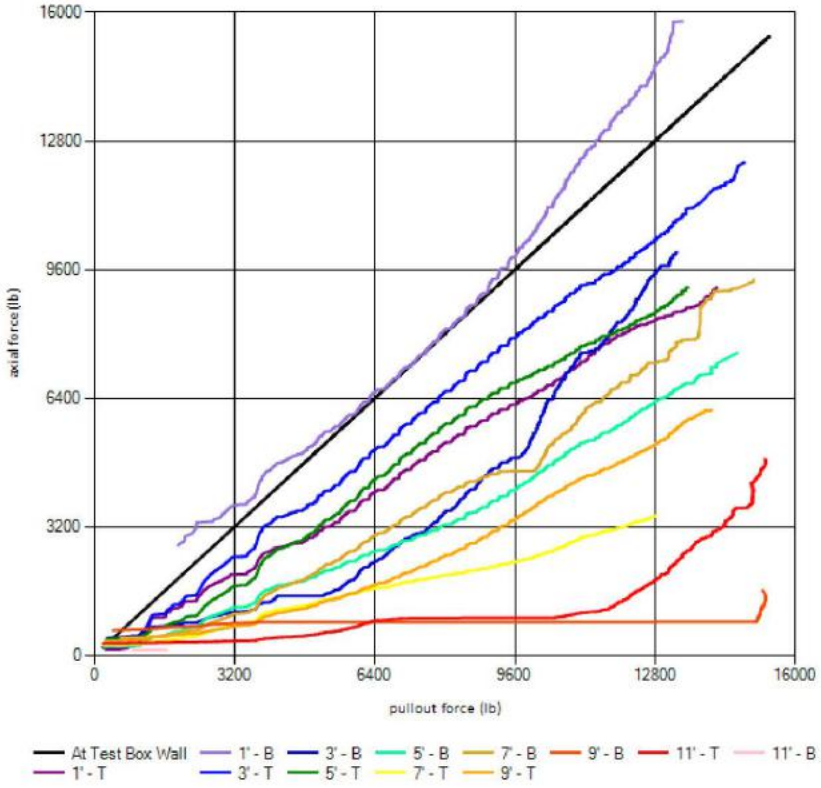
MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	12
Width, b (in.):	2
Skew Angle, θ (°):	0

Test Specimen Sketch



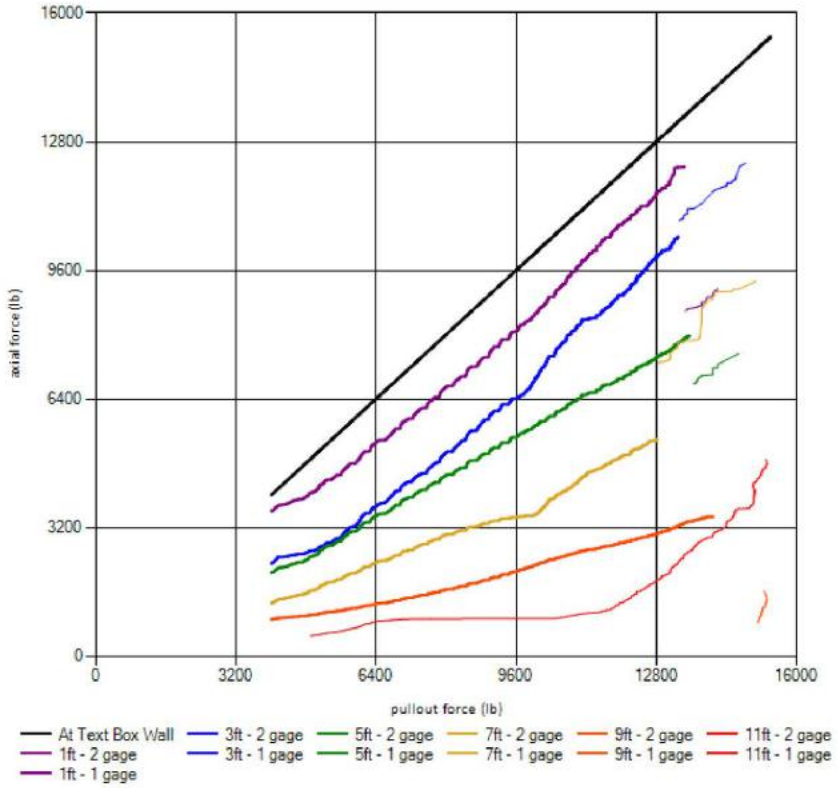
Axial Load-Pullout Load Data by Gage



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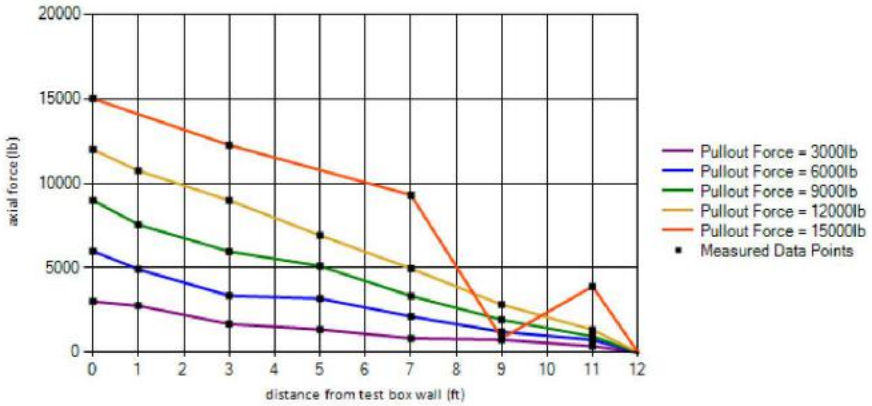
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

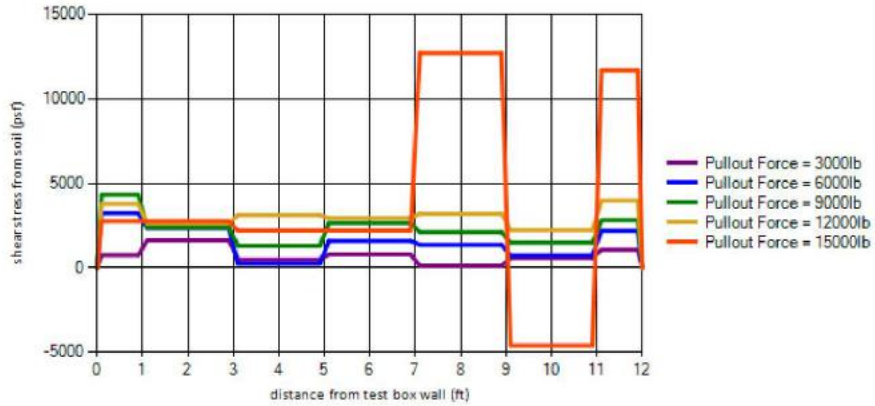
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	401*	555	379	341	549	226	
2000	1923	1082	766	506	622	333*	
3000	2754	1673	1349	827	741	358*	
4000	3609	2331	2070	1315	916	430*	
5000	4144	2663	2538	1675	1043	524*	
6000	4909	3347	3180	2119	1212	730*	
7000	5762	4205	3836	2565	1412	918*	
8000	6672	5037	4482	3009	1653	937*	
9000	7554	5966	5100	3331	1920	937*	
10000	8515	6890	5727	3560	2265	937*	
11000	9690	8209	6397	4350	2587	1037*	0.29
12000	10742	8993	6918	4955	2819	1328*	0.38
13000	11735	10112	7561	7113*	3118	2043*	0.54
14000	8891*	11507*	7052*	8668*	2435	2956*	0.79
15000	No Data	12259*	No Data	9311*	835*	3897*	1.11

*calculated from 1 strain gage.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	*Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall*								*Displacement (in.)*
	0-1ft	*1-3ft*	*3-5ft*	*5-7ft*	*7-9ft*	*9-11ft*	*11-12ft*	*Mean*	
1000	1848	-231	263	58	-312	485	677	254	
2000	221	1262	474	389	-174	433	1000	499	
3000	743	1622	487	782	129	575	1074	751	
4000	1200	1917	391	1133	599	728	1291	1002	
5000	2618	2222	187	1295	949	778	1571	1254	
6000	3228	2343	250	1591	1361	723	2190	1496	
7000	3669	2336	552	1907	1729	742	2753	1746	
8000	4019	2454	832	2210	2034	1074	2812	2003	
9000	4326	2382	1300	2653	2116	1475	2812	2249	
10000	4533	2437	1744	3250	1943	1991	2812	2506	
11000	3961	2221	2718	3072	2643	2325	3112	2753	0.29
12000	3778	2624	3112	2945	3204	2236	3985	3000	0.38
13000	3828	2435	3826	672	5993	1612	6129	3253	0.54
14000	15250	-3925	6684	-2424	9349	-781	8867	3494	0.79
15000	2748	2748	2211	2211	12714	-4592	11690	3752	1.11

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STRAIN GAGE REPORT FOR MSE REINFORCEMENT PULLOUT TEST

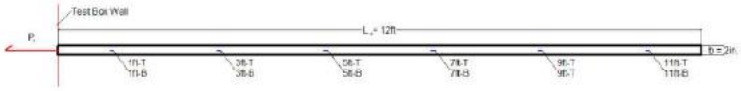
Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S6-Z12-L12-M
Test Facility:	12'x12'x4' MSE Test Box

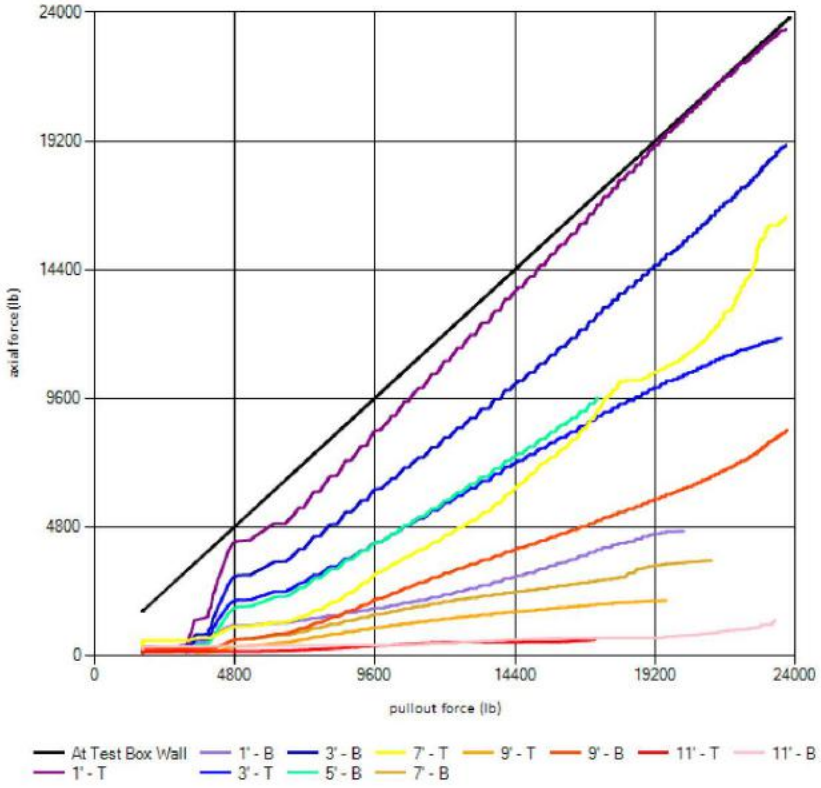
MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	12
Width, b (in.):	2
Skew Angle, θ (°):	0

Test Specimen Sketch



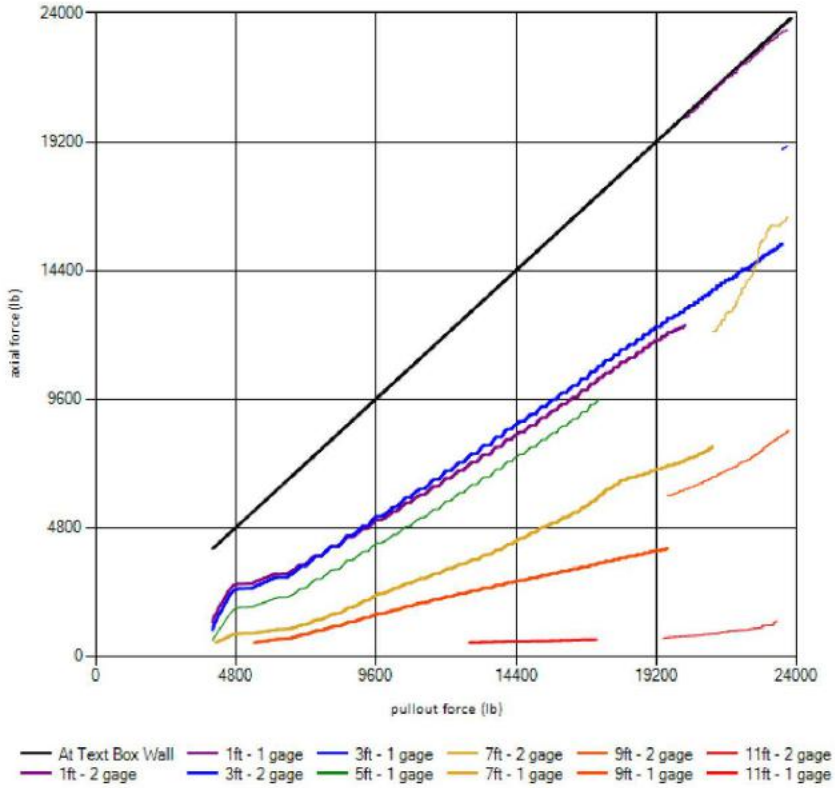
Axial Load-Pullout Load Data by Gage



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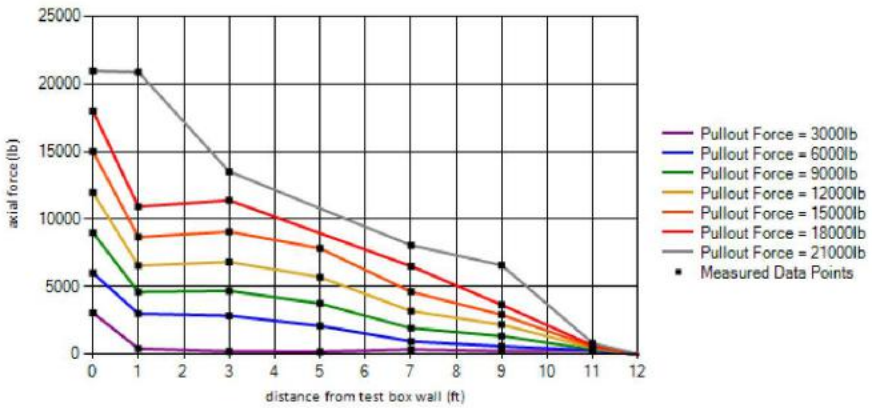
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

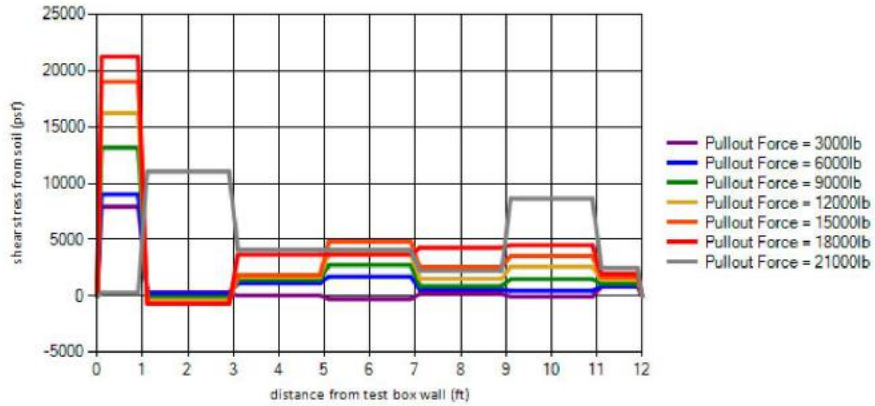
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	111*	102*	102*	121*	105*	183	
2000	353*	223*	176*	353	232*	274	
3000	420*	212	188*	353	232*	274	
4000	1397	1119	676*	480	217	274	
5000	2696	2518	1809*	838	475	277	
6000	3003	2857	2101*	956	594	285	
7000	3353	3272	2461*	1126	761	298	
8000	3976	3974	3100*	1486	1045	325	
9000	4612	4689	3755*	1933	1347	359	
10000	5257	5405	4385*	2422	1650	393	
11000	5916	6126	5049*	2818	1933	424	
12000	6560	6829	5708*	3197	2189	466	
13000	7287	7607	6443*	3613	2453	509	
14000	7974	8345	7136*	4080	2695	538	
15000	8665	9069	7838*	4623	2922	556	
16000	9390	9811	8567*	5129	3158	582	
17000	10159	10601	9364*	5770	3402	623	
18000	10924	11393	No Data	6519	3654	650*	
19000	11634	12119	No Data	6875	3884	650*	
20000	13178	12795	No Data	7227	6140*	729*	0.28
21000	20885*	13520	No Data	8063	6585*	818*	0.32
22000	21869*	14244	No Data	13245*	7122*	958*	0.37
23000	22840*	15000	No Data	15781*	7841*	1143*	0.44

*calculated from 1 strain gage.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	*Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall*								*Displacement (in.)*
	0-1ft	*1-3ft*	*3-5ft*	*5-7ft*	*7-9ft*	*9-11ft*	*11-12ft*	*Mean*	
1000	4656	14	No Data	-28	23	-116	548	416	
2000	7560	195	70	-264	181	-63	821	718	
3000	7926	313	35	-247	182	-63	821	766	
4000	7936	417	664	295	394	-85	821	1011	
5000	6801	267	1063	1457	544	296	831	1241	
6000	9016	219	1134	1718	543	463	854	1502	
7000	10896	122	1218	2002	548	694	895	1746	
8000	11972	3	1311	2421	661	1080	974	1992	
9000	13152	-114	1400	2733	878	1482	1077	2249	
10000	14283	-223	1531	2943	1159	1885	1179	2504	
11000	15185	-316	1617	3346	1328	2264	1271	2744	
12000	16232	-403	1680	3767	1513	2583	1399	2993	
13000	17170	-480	1745	4245	1741	2916	1528	3252	
14000	18093	-556	1814	4584	2076	3236	1614	3501	
15000	19009	-606	1847	4822	2552	3548	1668	3750	
16000	19830	-632	1866	5157	2957	3864	1746	4000	
17000	20581	-663	1856	5390	3553	4167	1870	4255	
18000	21237	-704	3656	3656	4296	4507	1949	4501	
19000	22066	-726	3933	3933	4486	4852	1949	4747	
20000	20451	574	4176	4176	1631	8116	2188	4999	0.28
21000	255	11048	4093	4093	2217	8650	2455	5243	0.32
22000	283	11438	749	749	9185	9246	2874	5491	0.37
23000	443	11759	-586	-586	11911	10047	3429	5747	0.44

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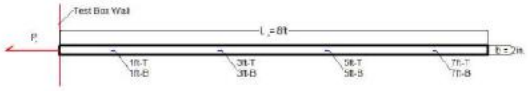
Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S11-Z20-L8-B
Test Facility:	12'x12'x4' MSE Test Box

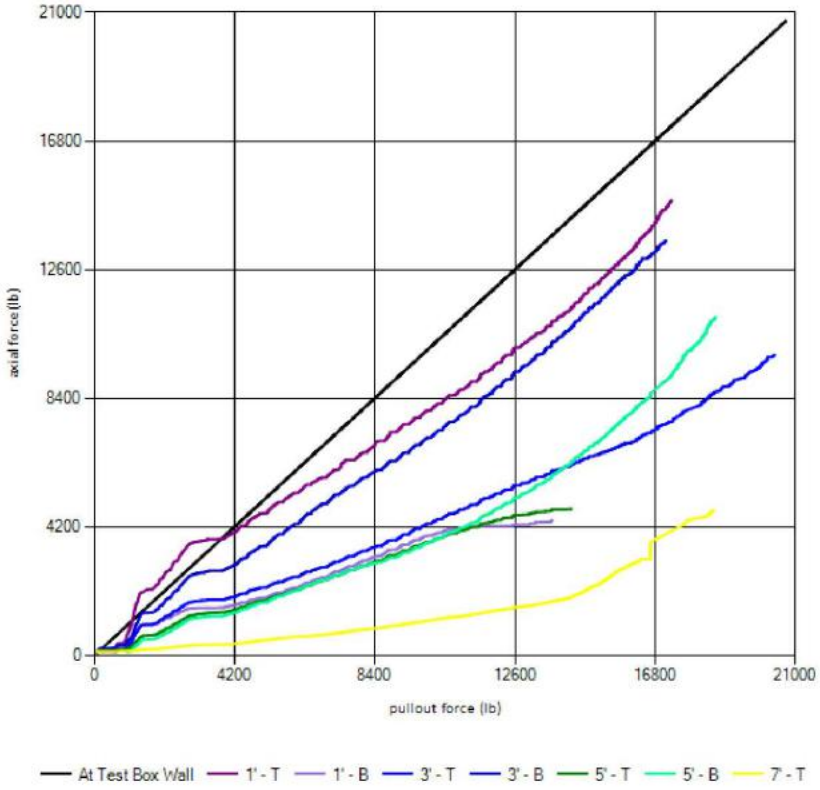
MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	8
Width, b (in.):	2
Skew Angle, θ (°):	0

Test Specimen Sketch



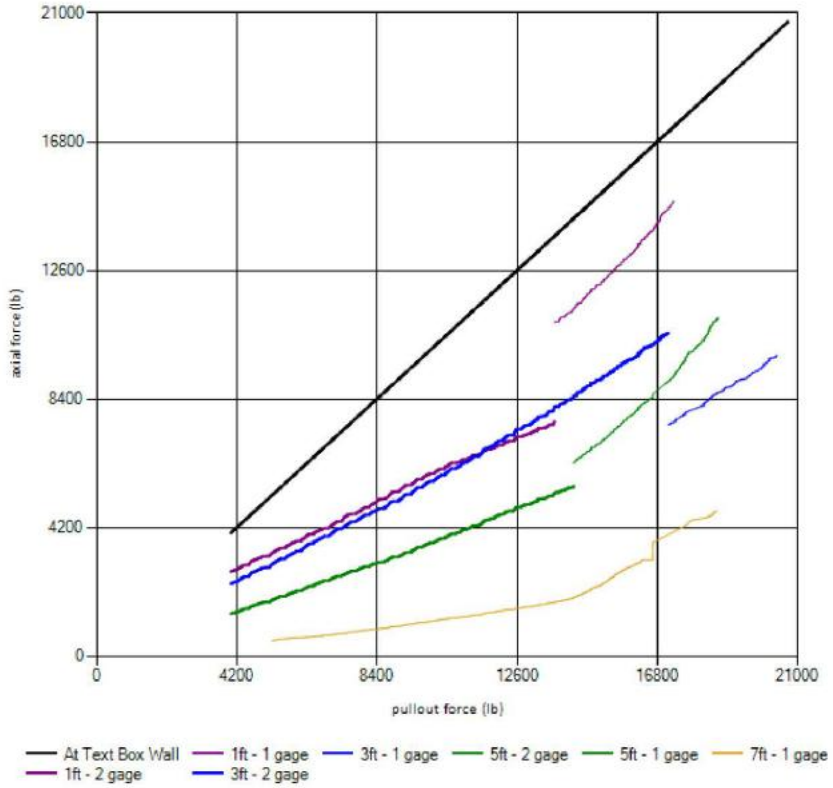
Axial Load-Pullout Load Data by Gage



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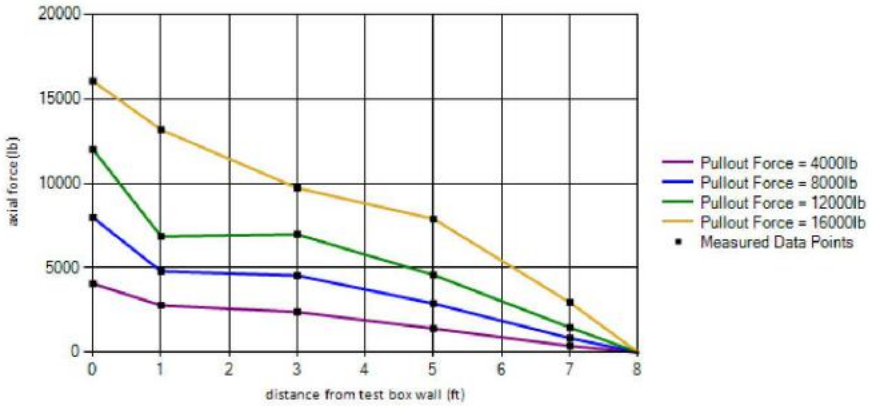
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

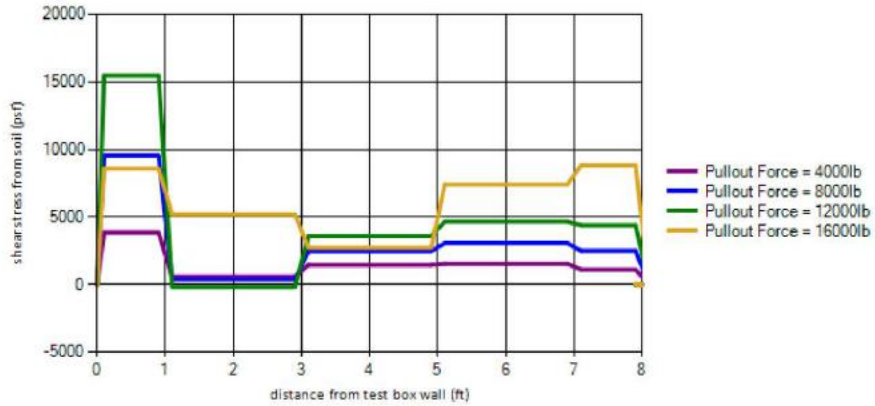
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	752	507	232	148*			
2000	1837	1431	728	230*			
3000	2612	2198	1265	337*			
4000	2775	2380	1402	373*			
5000	3234	2863	1734	480*			
6000	3750	3417	2116	603*			
7000	4280	4020	2531	700*			
8000	4791	4542	2884	834*			
9000	5416	5117	3275	981*			
10000	5949	5698	3689	1135*			
11000	6409	6301	4117	1284*			
12000	6856	6974	4577	1464*			
13000	7300	7619	4999	1629*			
14000	11106*	8279	5428	1827*			0.28
15000	12098*	8999	6913*	2276*			0.35
16000	13177*	9719	7892*	2945*			0.46
17000	14429*	9660	8931*	4260*			0.61
18000	No Data	8177*	10176*	4510*			0.83

*calculated from 1 strain gage.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall								Displacement (in.)
	0-1ft	1-3ft	3-5ft	5-7ft	7-9ft	9-11ft	11-12ft	Mean	
1000	956	368	413	125	445			402	
2000	582	609	1053	748	690			762	
3000	1186	621	1400	1391	1012			1128	
4000	3850	592	1467	1544	1118			1522	
5000	5183	557	1693	1881	1441			1861	
6000	6742	499	1951	2270	1810			2249	
7000	8268	389	2234	2747	2099			2638	
8000	9543	374	2487	3074	2502			2989	
9000	10834	448	2764	3440	2943			3385	
10000	12158	377	3013	3831	3406			3751	
11000	13798	162	3276	4250	3852			4128	
12000	15460	-178	3596	4670	4391			4503	
13000	17132	-478	3930	5055	4887			4879	
14000	8705	4241	4277	5401	5482			5253	0.28
15000	8709	4648	3128	6956	6829			5625	0.35
16000	8583	5187	2740	7422	8834			6014	0.46
17000	7661	7154	1093	7007	12779			6369	0.61
18000	9866	9866	-2998	8499	13530			6766	0.83

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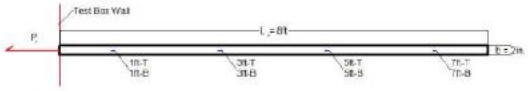
Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S15-Z20-L8-B
Test Facility:	12'x12'x4' MSE Test Box

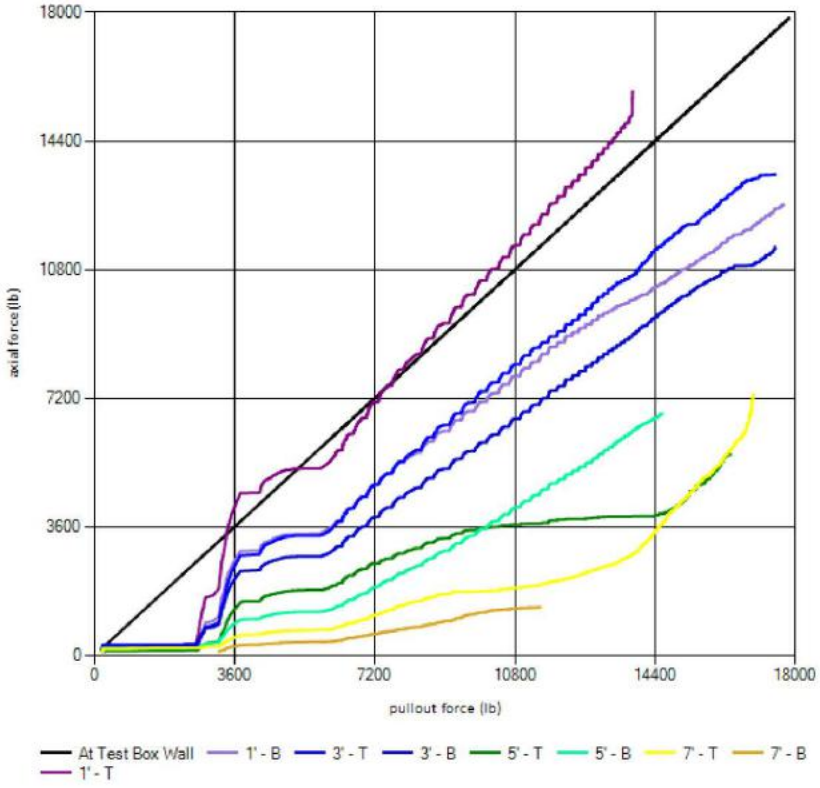
MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	8
Width, b (in.):	2
Skew Angle, θ (°):	0

Test Specimen Sketch



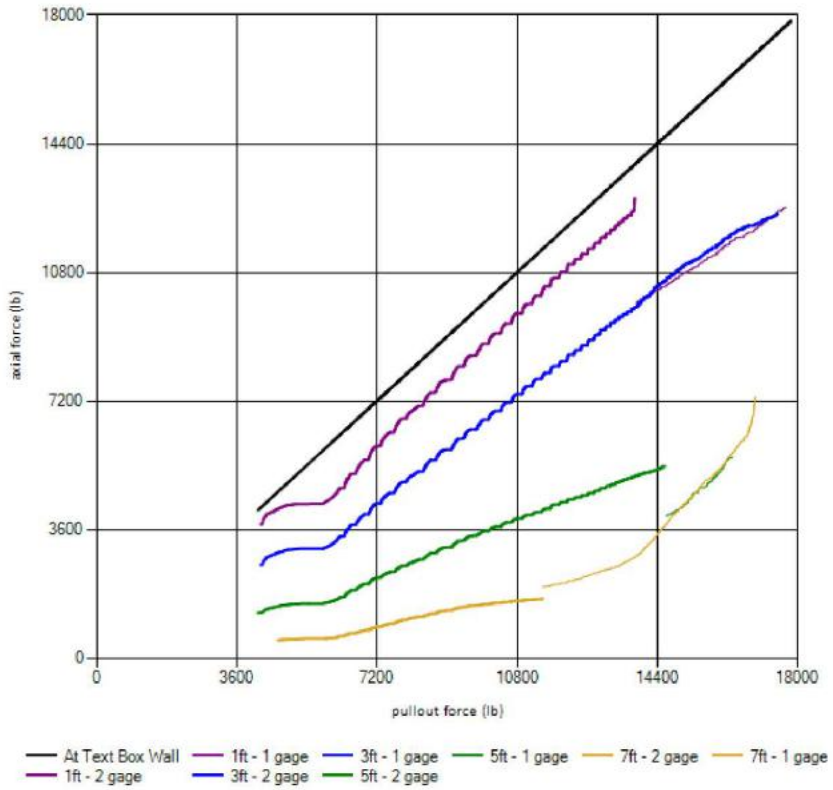
Axial Load-Pullout Load Data by Gage



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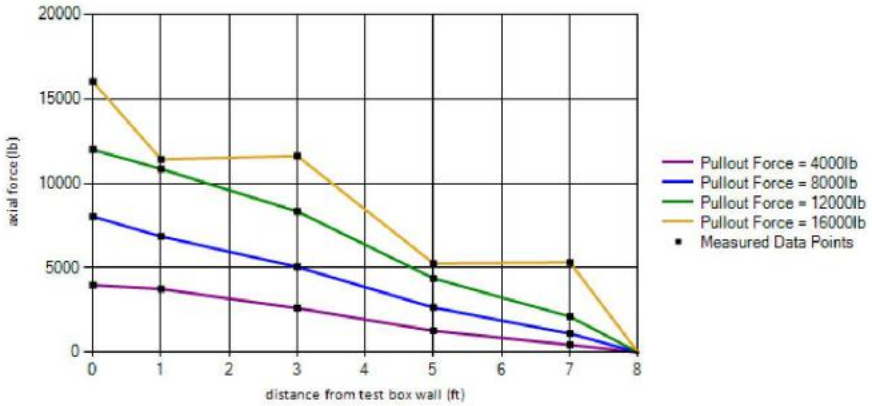
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

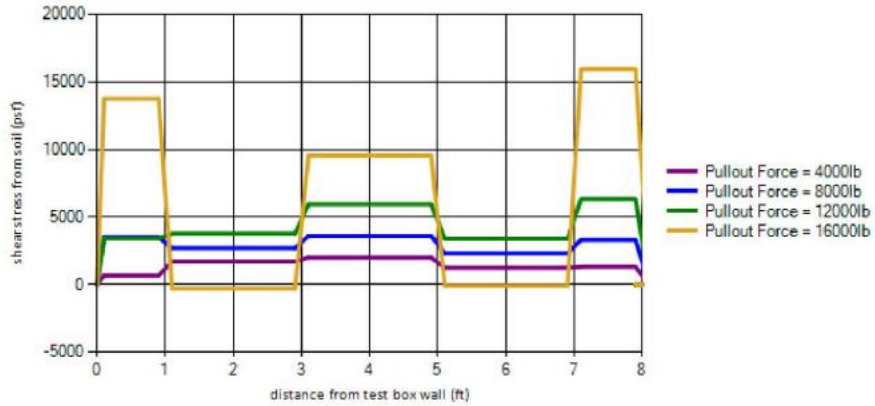
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	223*	278	167	204*			
2000	235*	278	172	213*			
3000	1346	831	353	206			
4000	3745	2608	1267	440			
5000	4288	3037	1502	528			
6000	4573	3257	1623	577			
7000	5672	4116	2128	803			
8000	6853	5051	2647	1105			
9000	7829	5861	3113	1337			
10000	8813	6680	3554	1508			
11000	9878	7575	3985	1627			
12000	10851	8337	4373	2110*			
13000	11762	9178	4752	2449*			0.27
14000	10574*	9996	5153	3009*			0.37
15000	10711*	10907	4224*	4208*			0.52
16000	11419*	11619	5253*	5315*			0.71
17000	12063*	12174	No Data	6819*			0.99

*calculated from 1 strain gage.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall								Displacement (in.)
	0-1ft	1-3ft	3-5ft	5-7ft	7-9ft	9-11ft	11-12ft	Mean	
1000	2611	-84	167	-56	612			410	
2000	5930	-65	160	-63	640			829	
3000	4898	773	717	221	617			1117	
4000	682	1705	2011	1241	1319			1490	
5000	2100	1876	2303	1461	1583			1870	
6000	4569	1975	2450	1569	1730			2286	
7000	3924	2334	2982	1988	2408			2617	
8000	3521	2703	3606	2314	3314			3010	
9000	3504	2951	4122	2665	4010			3374	
10000	3514	3200	4688	3069	4525			3744	
11000	3477	3454	5385	3538	4880			4139	
12000	3423	3771	5946	3394	6331			4497	
13000	3721	3875	6639	3455	7348			4876	0.27
14000	10299	867	7265	3215	9028			5253	0.37
15000	12850	-294	10025	24	12623			5623	0.52
16000	13769	-301	9550	-93	15945			6003	0.71
17000	14749	-167	4016	4016	20458			6367	0.99

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STRAIN GAGE REPORT FOR MSE REINFORCEMENT PULLOUT TEST

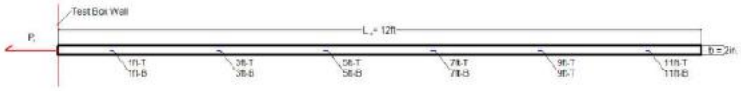
Test Information

Test Date:	June 20, 2011
Test Identification:	TS29-S3-Z20-L12-T
Test Facility:	12'x12'x4' MSE Test Box

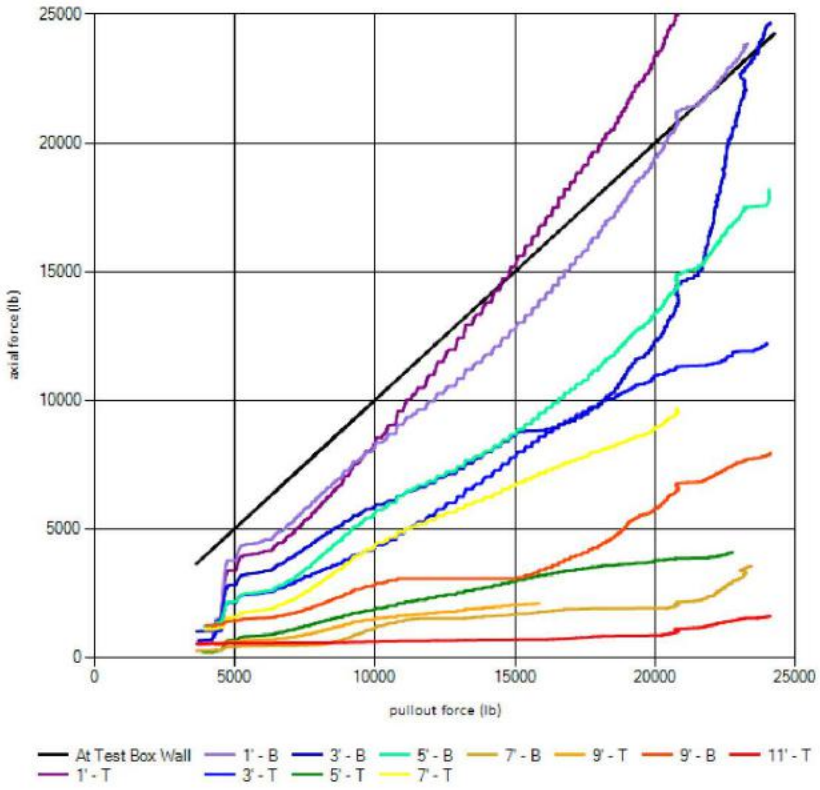
MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	12
Width, b (in.):	2
Skew Angle, θ (°):	0

Test Specimen Sketch



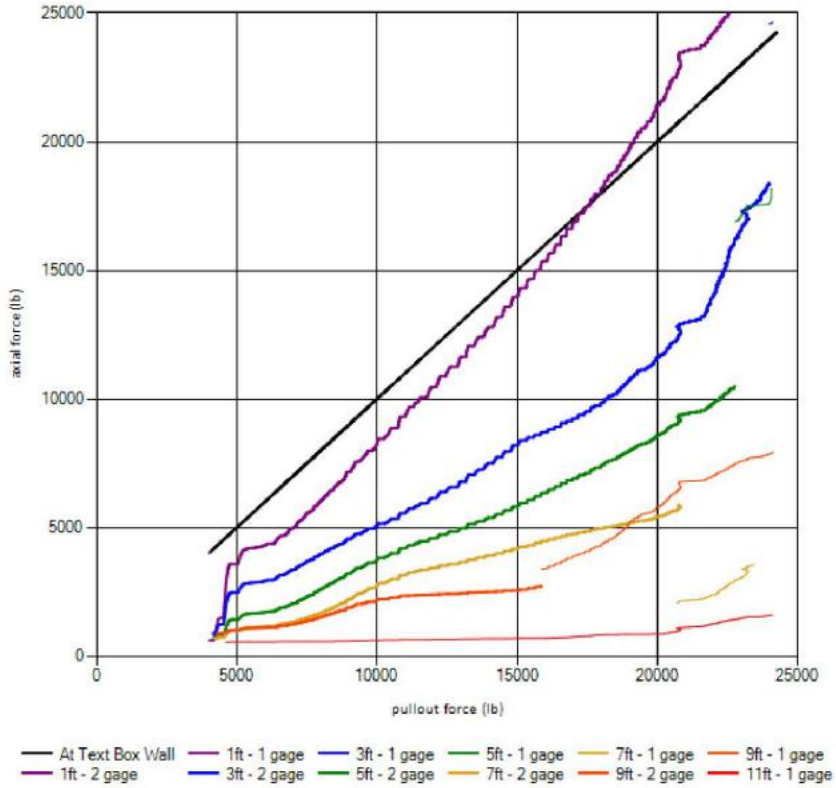
Axial Load-Pullout Load Data by Gage



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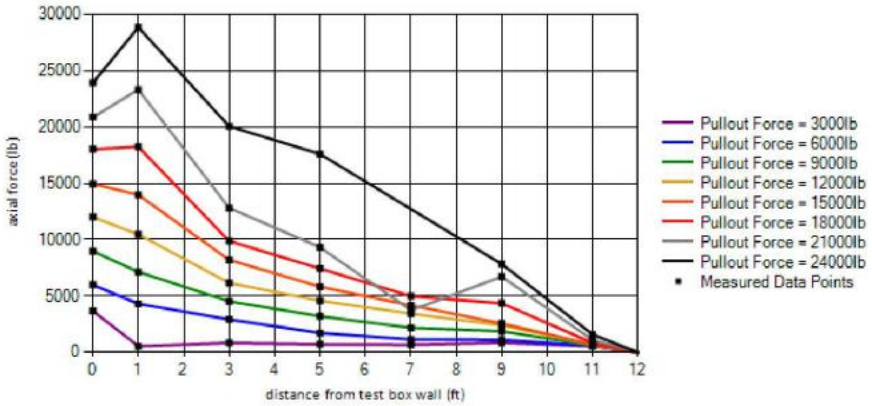
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

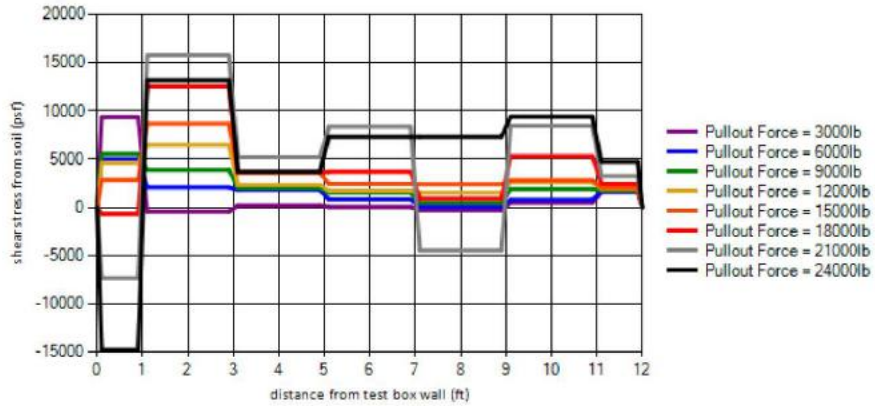
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	543	840	710	676	863	535*	0.5
2000	543	840	710	676	863	535*	0.5
3000	543	840	710	676	863	535*	0.5
4000	642	870	718	681	866	538*	0.42
5000	3706	2544	1453	1006	1021	557*	
6000	4306	2916	1707	1147	1109	566*	
7000	4999	3362	2037	1338	1242	566*	
8000	6030	3962	2599	1682	1519	575*	
9000	7128	4532	3196	2174	1867	590*	
10000	8232	5064	3729	2729	2162	616*	
11000	9372	5585	4201	3153	2339	638*	
12000	10462	6150	4584	3429	2396	648*	
13000	11627	6788	4972	3652	2445	665*	
14000	12738	7472	5362	3903	2503	677*	
15000	13985	8200	5816	4173	2559	687*	
16000	15369	8716	6352	4472	3198*	697*	
17000	16847	9234	6900	4751	3881*	745*	0.26
18000	18243	9871	7431	4976	4356*	823*	0.31
19000	19752	10743	7991	5171	5156*	851*	0.37
20000	21257	11527	8526	5392	5748*	855*	0.45
21000	23313	12811	9318	3733*	6710*	1081*	0.59
22000	24212	14039	9821	2394*	7012*	1253*	0.68
23000	25752	16806	16951*	3184*	7611*	1494*	0.83
24000	28846*	20065	17602*	No Data	7833*	1575*	0.97

*calculated from 1 strain gage.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	*Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall*								*Displacement (in.)*
	0-1ft	*1-3ft*	*3-5ft*	*5-7ft*	*7-9ft*	*9-11ft*	*11-12ft*	*Mean*	
1000	9353	-445	195	51	-281	492	1605	915	0.5
2000	9353	-445	195	51	-281	492	1605	915	0.5
3000	9353	-445	195	51	-281	492	1605	915	0.5
4000	10204	-342	228	56	-277	491	1615	1011	0.42
5000	3776	1742	1637	671	-23	697	1670	1241	
6000	5021	2085	1813	840	56	815	1698	1495	
7000	6010	2455	1988	1048	144	1014	1698	1751	
8000	5877	3102	2045	1375	244	1416	1726	1997	
9000	5547	3895	2004	1532	461	1914	1771	2244	
10000	5255	4752	2002	1499	851	2319	1848	2496	
11000	4780	5680	2076	1572	1220	2551	1914	2741	
12000	4610	6469	2349	1733	1549	2623	1943	3000	
13000	4225	7260	2724	1980	1809	2671	1994	3259	
14000	3774	7899	3165	2189	2100	2738	2032	3499	
15000	2866	8678	3576	2464	2421	2808	2062	3735	
16000	1827	9980	3546	2820	1911	3751	2090	3995	
17000	475	11419	3501	3223	1305	4703	2236	4251	0.26
18000	-667	12559	3660	3682	930	5299	2469	4505	0.31
19000	-2240	13514	4129	4229	22	6458	2553	4751	0.37
20000	-3860	14595	4500	4702	-534	7339	2565	4993	0.45
21000	-7320	15753	5240	8377	-4466	8443	3244	5218	0.59
22000	-6651	15259	6327	11140	-6927	8639	3759	5499	0.68
23000	-8116	13418	-217	20650	-6640	9175	4482	5762	0.83
24000	-14746	13171	3694	7327	7327	9387	4725	5983	0.97

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STRAIN GAGE REPORT FOR MSE REINFORCEMENT PULLOUT TEST

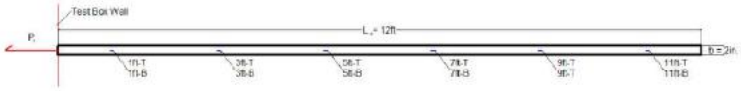
Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S7-Z20-L12-M
Test Facility:	12'x12'x4' MSE Test Box

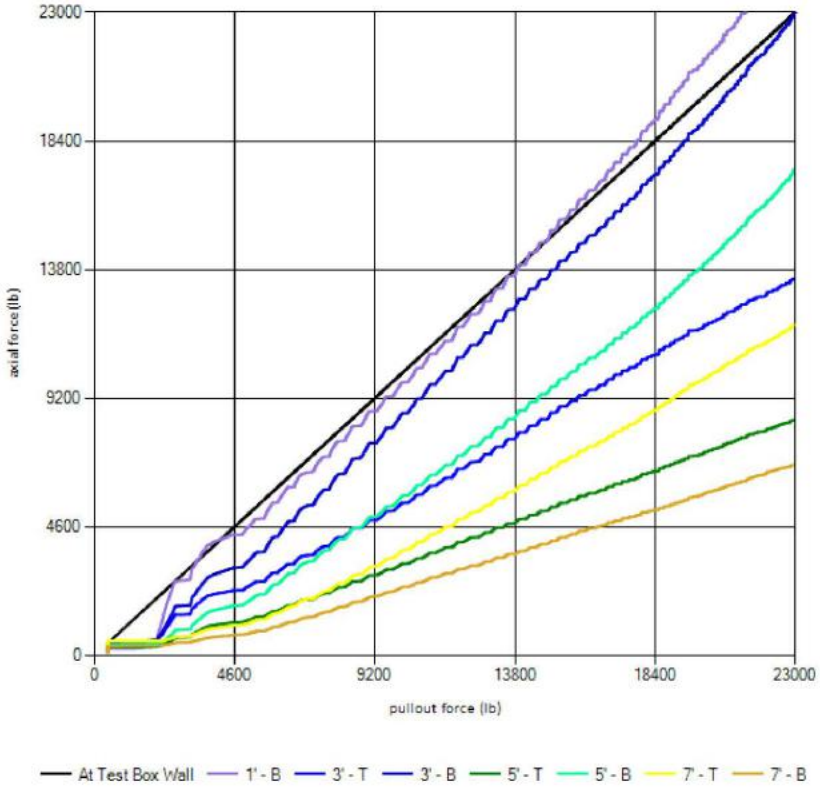
MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	12
Width, b (in.):	2
Skew Angle, θ (°):	0

Test Specimen Sketch



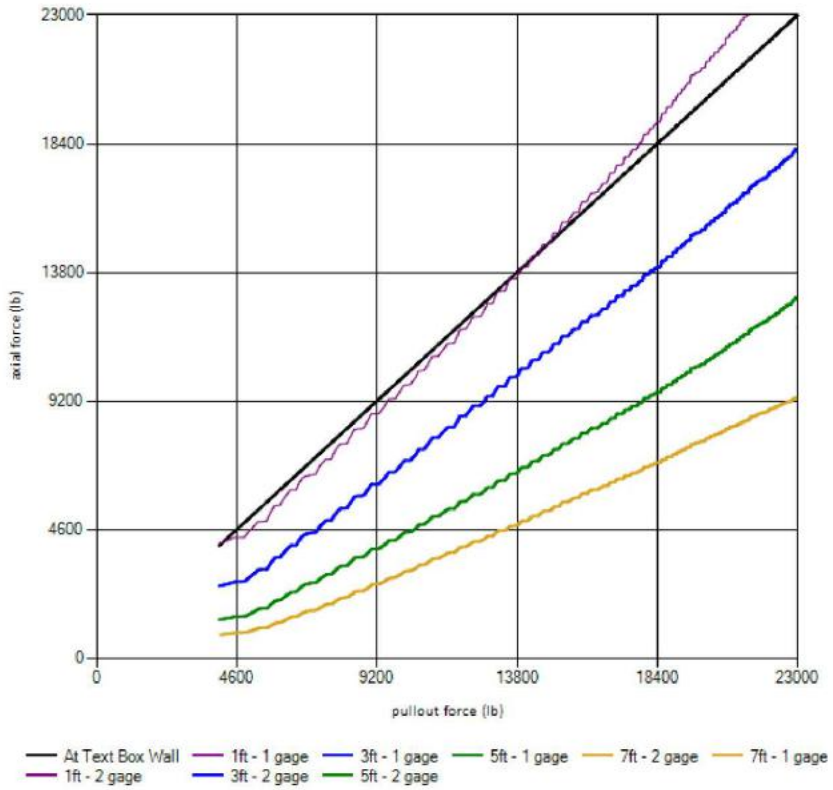
Axial Load-Pullout Load Data by Gage



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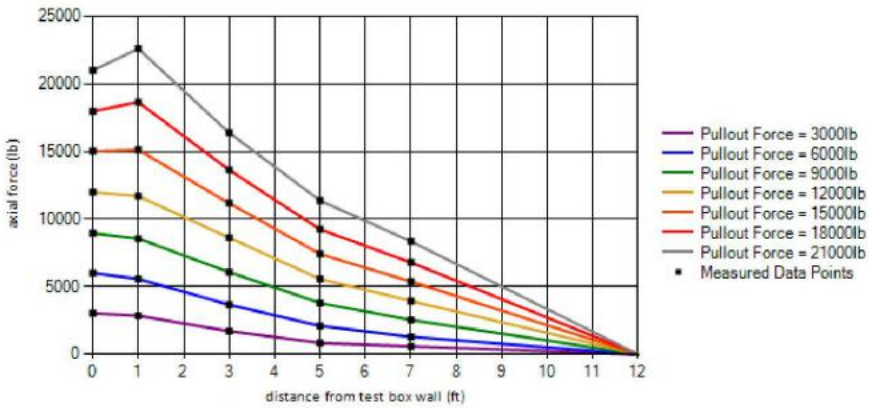
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

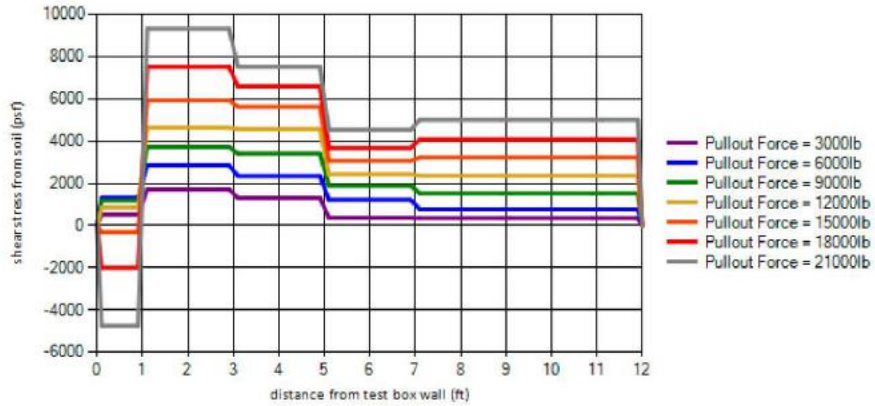
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	263*	464	353	439	No Data	No Data	
2000	866*	657	399	442	No Data	No Data	
3000	2854*	1712	834	588	No Data	No Data	
4000	4090*	2564	1365	830	No Data	No Data	
5000	4573*	2913	1595	971	No Data	No Data	
6000	5569*	3670	2102	1288	No Data	No Data	
7000	6541*	4457	2660	1680	No Data	No Data	
8000	7561*	5266	3225	2098	No Data	No Data	
9000	8554*	6072	3798	2542	No Data	No Data	
10000	9579*	6900	4378	3008	No Data	No Data	
11000	10619*	7741	4961	3471	No Data	No Data	
12000	11704*	8616	5569	3949	No Data	No Data	
13000	12858*	9507	6196	4432	No Data	No Data	
14000	13932*	10324	6787	4879	No Data	No Data	
15000	15122*	11173	7420	5379	No Data	No Data	
16000	16355*	12052	8070	5894	No Data	No Data	
17000	17392*	12775	8599	6301	No Data	No Data	
18000	18650*	13636	9241	6792	No Data	No Data	
19000	20081*	14601	9964	7336	No Data	No Data	0.26
20000	21224*	15407	10585	7819	No Data	No Data	0.3
21000	22599*	16383	11372	8352	No Data	No Data	0.35
22000	23987*	17263	12097	8840	No Data	No Data	0.42

*calculated from 1 strain gage.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	*Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall*								*Displacement (in.)*
	0-1ft	*1-3ft*	*3-5ft*	*5-7ft*	*7-9ft*	*9-11ft*	*11-12ft*	*Mean*	
1000	3274	-302	167	-130	264	264	264	339	
2000	3678	314	387	-66	265	265	265	523	
3000	519	1713	1317	370	353	353	353	757	
4000	-256	2289	1799	802	498	498	498	1001	
5000	1383	2490	1978	935	583	583	583	1259	
6000	1319	2849	2352	1220	773	773	773	1502	
7000	1291	3125	2696	1470	1008	1008	1008	1743	
8000	1238	3444	3061	1691	1259	1259	1259	1994	
9000	1194	3723	3411	1885	1525	1525	1525	2238	
10000	1222	4018	3783	2056	1805	1805	1805	2496	
11000	1014	4317	4169	2235	2083	2083	2083	2739	
12000	853	4632	4571	2430	2369	2369	2369	2997	
13000	470	5027	4967	2647	2659	2659	2659	3254	
14000	205	5411	5305	2863	2927	2927	2927	3500	
15000	-317	5924	5629	3061	3227	3227	3227	3754	
16000	-967	6453	5974	3264	3536	3536	3536	4008	
17000	-1344	6925	6265	3446	3781	3781	3781	4236	
18000	-2006	7520	6593	3673	4075	4075	4075	4495	
19000	-3091	8220	6956	3942	4402	4402	4402	4763	0.26
20000	-3758	8724	7234	4149	4691	4691	4691	4993	0.3
21000	-4744	9324	7517	4529	5011	5011	5011	5254	0.35
22000	-5939	10085	7750	4886	5304	5304	5304	5502	0.42

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STRAIN GAGE REPORT FOR MSE REINFORCEMENT PULLOUT TEST

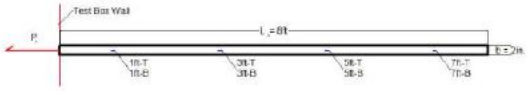
Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S12-Z40-L8-B
Test Facility:	12'x12'x4' MSE Test Box

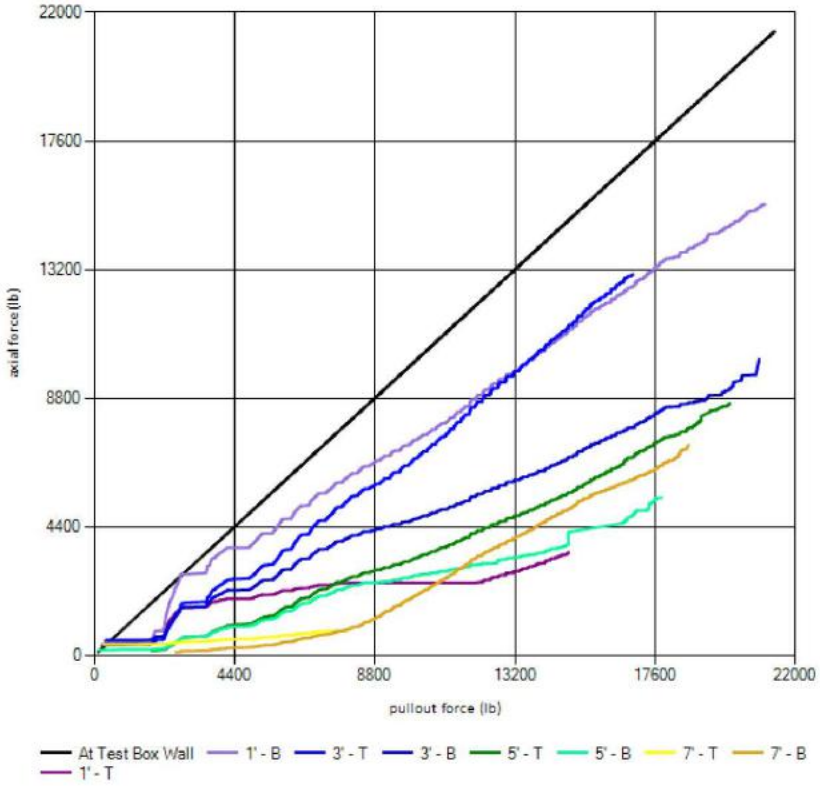
MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	8
Width, b (in.):	2
Skew Angle, θ (°):	0

Test Specimen Sketch



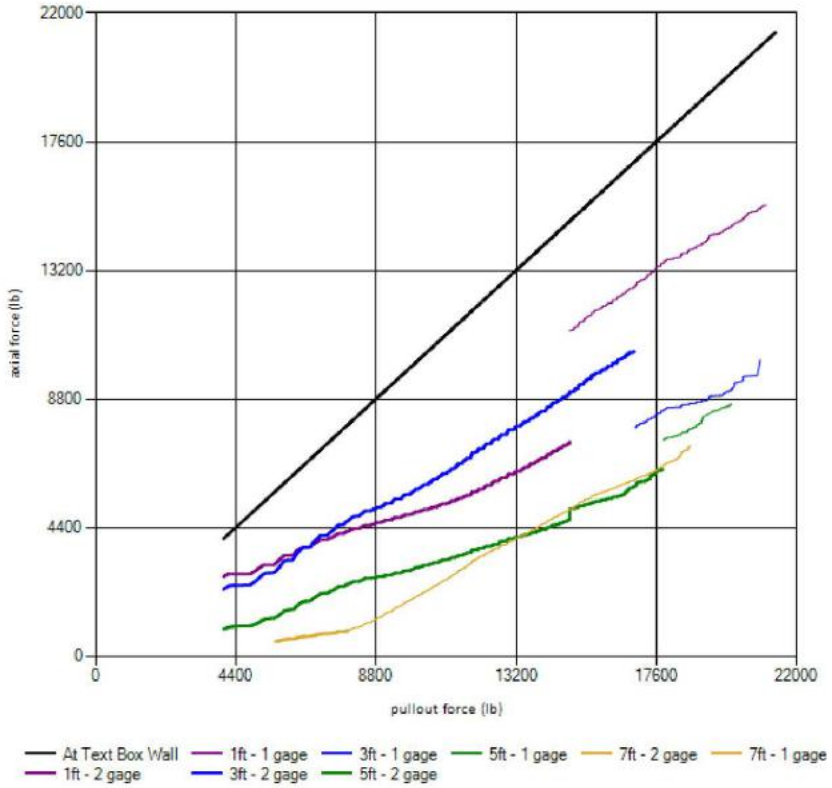
Axial Load-Pullout Load Data by Gage



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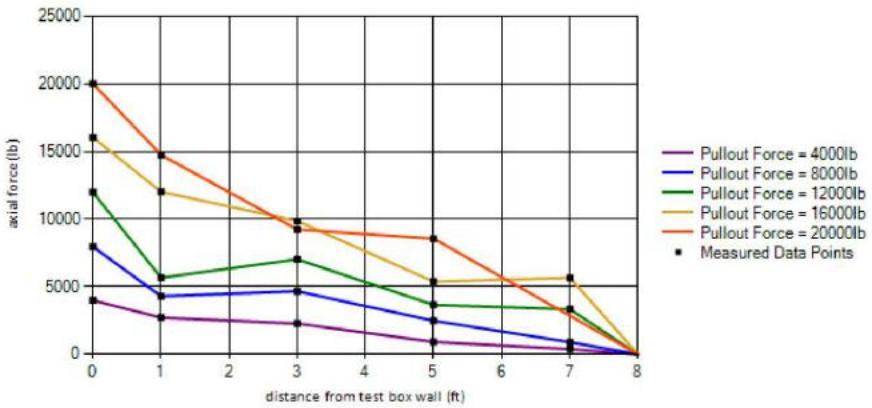
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

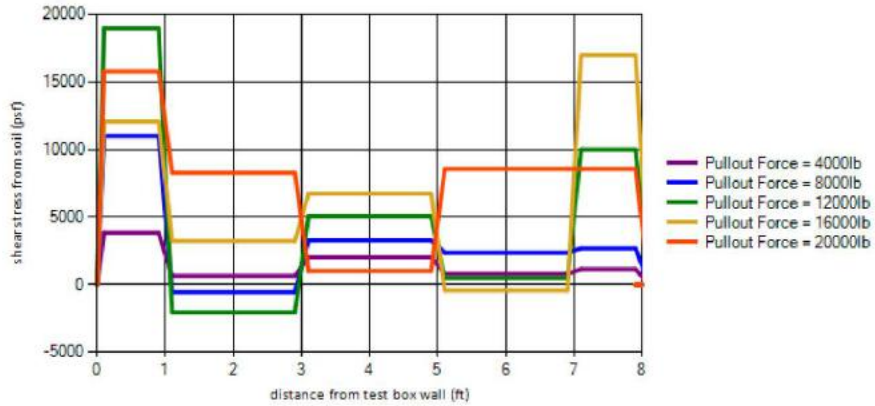
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	432	464	181	390*			
2000	714	574	204	395*			
3000	2236	1728	630	315			
4000	2709	2275	914	383			
5000	2946	2591	1116	441			
6000	3419	3233	1535	565			
7000	3895	3983	2044	723			
8000	4289	4661	2479	898*			
9000	4598	5136	2727	1363*			
10000	4911	5672	3008	2014*			
11000	5226	6267	3296	2614*			
12000	5662	7024	3655	3335*			
13000	6239	7756	4020	3946*			
14000	6746	8347	4307	4477*			
15000	10172*	9084	4964	5084*			0.3
16000	12020*	9861	5375	5655*			0.36
17000	12750*	8511*	5888	6080*			0.43
18000	13547*	8469*	7373*	6614*			0.52
19000	14093*	8713*	8051*	No Data			0.62
20000	14752*	9232*	8558*	No Data			0.74

*calculated from 1 strain gage.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall								Displacement (in.)
	0-1ft	1-3ft	3-5ft	5-7ft	7-9ft	9-11ft	11-12ft	Mean	
1000	2087	-49	425	-313	1169			423	
2000	3817	210	555	-287	1184			745	
3000	2158	762	1647	474	944			1108	
4000	3846	651	2042	796	1148			1497	
5000	6263	532	2213	1012	1322			1887	
6000	7667	279	2547	1456	1694			2241	
7000	9123	-132	2908	1982	2169			2601	
8000	10999	-558	3272	2372	2694			2983	
9000	13094	-806	3613	2046	4088			3361	
10000	15367	-1142	3996	1491	6043			3762	
11000	17268	-1561	4456	1024	7841			4118	
12000	18973	-2042	5053	481	10004			4495	
13000	20466	-2275	5604	111	11838			4898	
14000	21703	-2402	6061	-256	13432			5243	
15000	14451	1631	6181	-181	15253			5621	0.3
16000	12056	3238	6730	-420	16966			6015	0.36
17000	12784	6359	3934	-288	18240			6379	0.43
18000	13322	7617	1644	1139	19843			6745	0.52
19000	14711	8070	993	8051	8051			7124	0.62
20000	15777	8280	1012	8558	8558			7504	0.74

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STRAIN GAGE REPORT FOR MSE REINFORCEMENT PULLOUT TEST

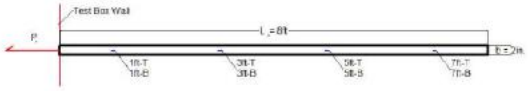
Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S16-Z40-L8-B
Test Facility:	12'x12'x4' MSE Test Box

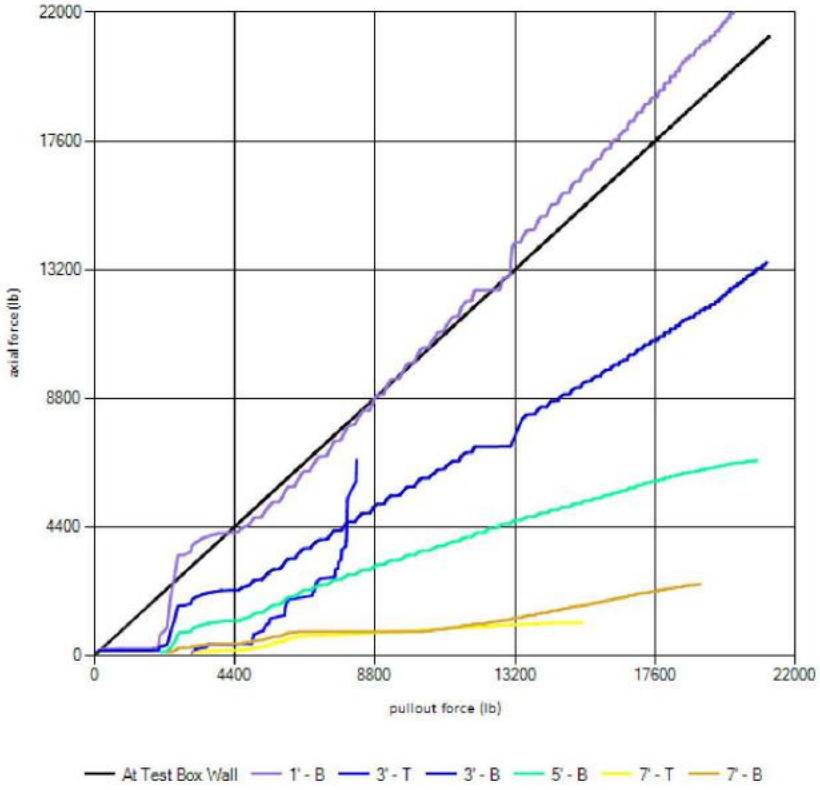
MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	8
Width, b (in.):	2
Skew Angle, θ (°):	0

Test Specimen Sketch



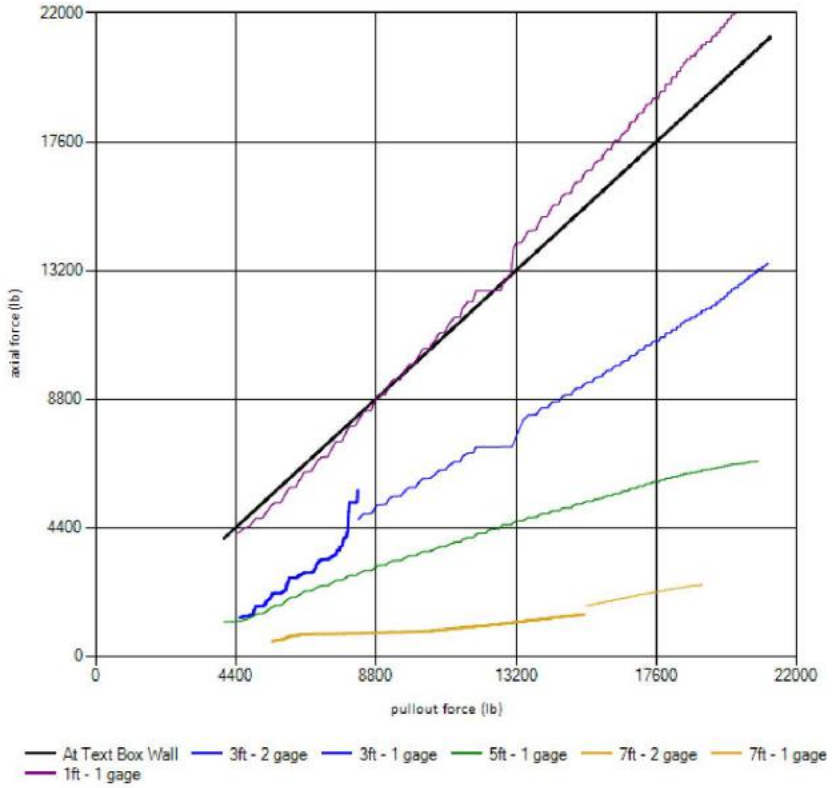
Axial Load-Pullout Load Data by Gage



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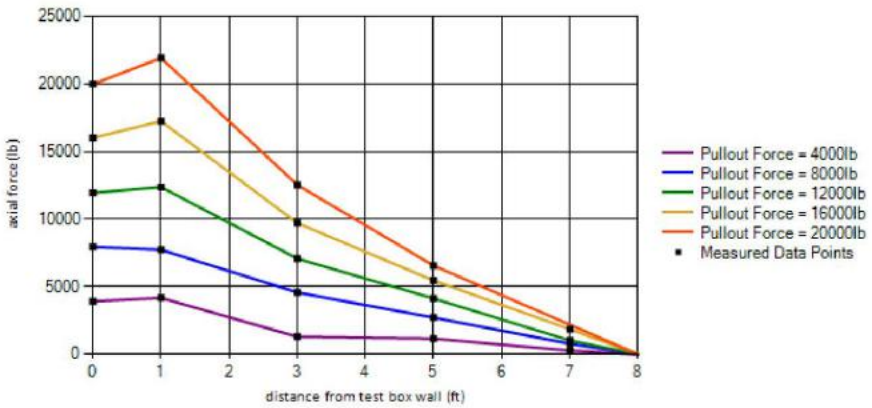
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

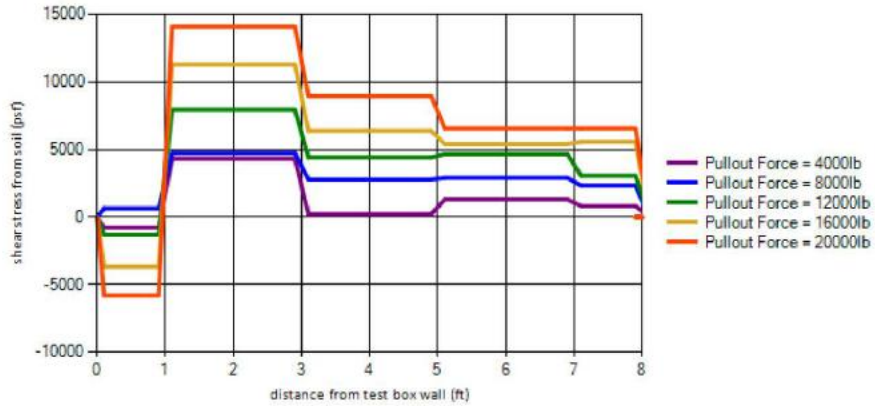
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	241*	136	124*	102*			
2000	563*	178	127*	102*			
3000	3706*	1024	899*	249*			
4000	4177*	1298	1152*	276			
5000	4612*	1551	1382*	362			
6000	5560*	2393	1884*	621			
7000	6619*	3102	2339*	755			
8000	7734*	4571	2713*	775			
9000	9017*	5235*	3128*	804			
10000	10156*	5863*	3476*	834			
11000	11306*	6493*	3809*	924			
12000	12372*	7076*	4128*	1019			
13000	13512*	7643*	4546*	1142			
14000	14910*	8432*	4836*	1265			
15000	16075*	9071*	5153*	1390			
16000	17240*	9710*	5455*	1856*			
17000	18458*	10388*	5777*	2094*			
18000	19667*	11092*	6095*	2291*			
19000	20794*	11739*	6326*	2430*			
20000	21919*	12527*	6556*	No Data			0.28
21000	23422*	13294*	6665*	No Data			0.34

*calculated from 1 strain gage.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall								Displacement (in.)
	0-1ft	1-3ft	3-5ft	5-7ft	7-9ft	9-11ft	11-12ft	Mean	
1000	3798	158	19	32	306			565	
2000	4429	577	76	38	306			765	
3000	-1957	4024	186	975	747			1145	
4000	-783	4319	218	1314	828			1468	
5000	1082	4592	253	1530	1085			1865	
6000	1259	4751	763	1895	1862			2242	
7000	980	5275	1145	2375	2265			2605	
8000	640	4743	2787	2907	2326			2980	
9000	4	5673	3161	3485	2413			3382	
10000	-366	6439	3581	3962	2503			3763	
11000	-754	7221	4025	4328	2772			4146	
12000	-1289	7944	4421	4665	3056			4478	
13000	-1450	8804	4646	5105	3427			4886	
14000	-2739	9716	5395	5356	3795			5249	
15000	-3190	10507	5877	5644	4171			5630	
16000	-3667	11295	6383	5397	5569			6007	
17000	-4374	12105	6917	5525	6281			6375	
18000	-4887	12864	7494	5706	6874			6764	
19000	-5455	13583	8120	5844	7290			7116	
20000	-5787	14088	8957	6556	6556			7496	0.28
21000	-7475	15192	9943	6665	6665			7849	0.34

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STRAIN GAGE REPORT FOR MSE REINFORCEMENT PULLOUT TEST

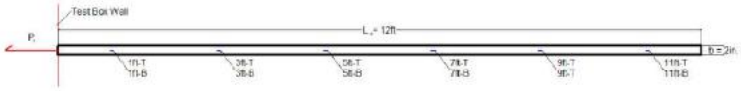
Test Information

Test Date:	June 20, 2011
Test Identification:	TS29-S4-Z40-L12-T
Test Facility:	12'x12'x4' MSE Test Box

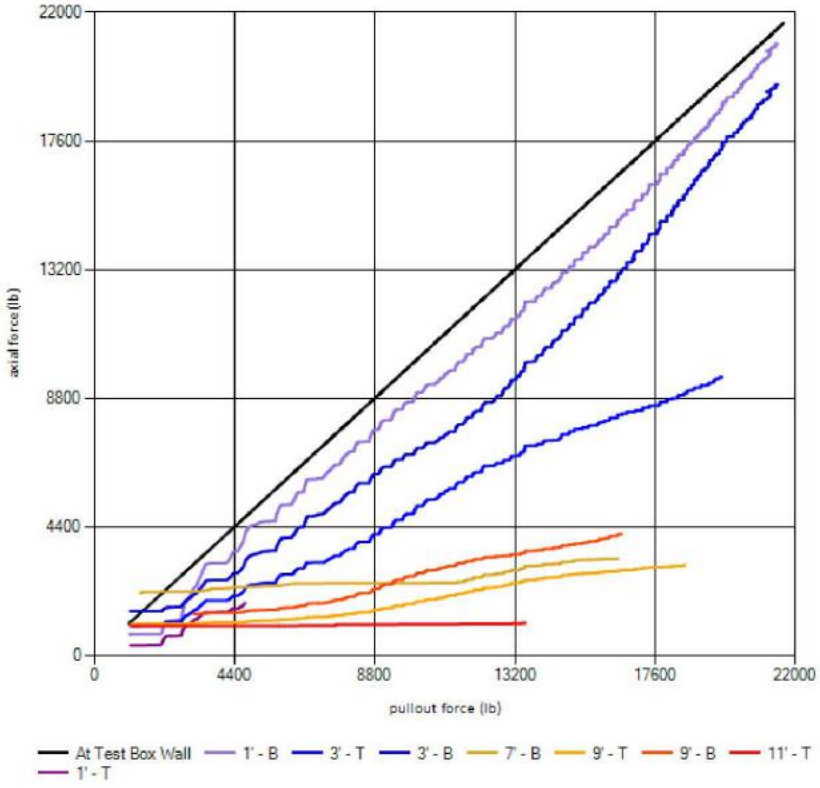
MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	12
Width, b (in.):	2
Skew Angle, θ (°):	0

Test Specimen Sketch



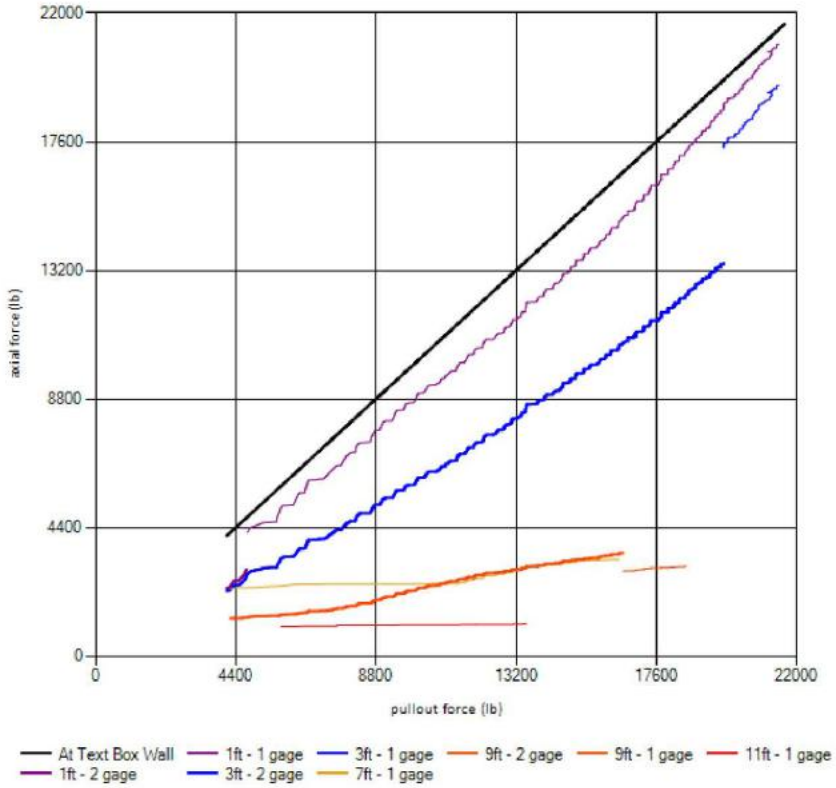
Axial Load-Pullout Load Data by Gage



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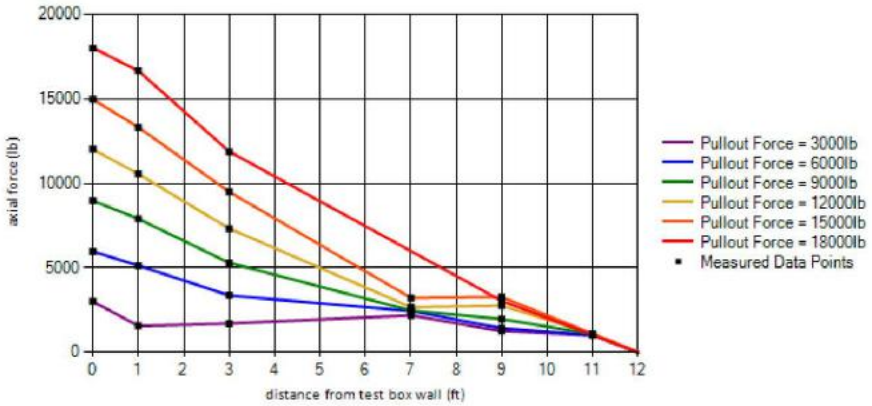
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

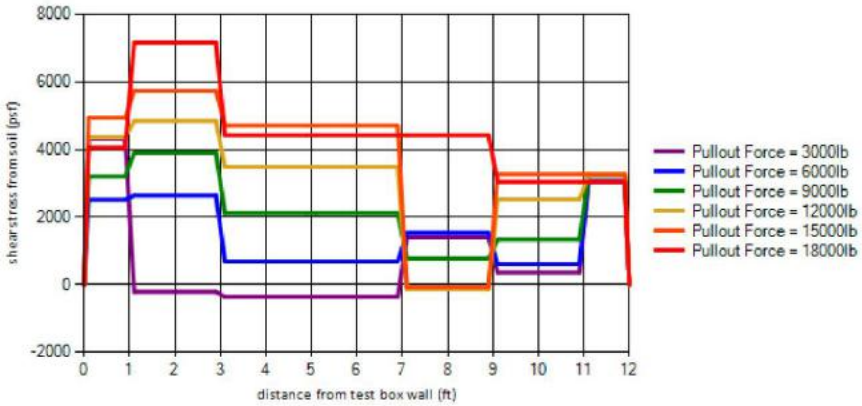
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	535	1267	No Data	2144*	1246	1008*	
2000	702	1311	No Data	2181*	1248	1012*	
3000	1560	1702	No Data	2186*	1252	1012*	
4000	2364	2258	No Data	2307*	1295	1012*	
5000	4435*	2924	No Data	2362*	1360	1012*	
6000	5130*	3367	No Data	2446*	1424	1019*	
7000	6084*	4012	No Data	2478*	1549	1021*	
8000	6942*	4603	No Data	2478*	1706	1076*	
9000	7899*	5297	No Data	2478*	1965	1076*	
10000	8798*	5949	No Data	2478*	2264	1076*	
11000	9554*	6530	No Data	2478*	2499	1076*	
12000	10556*	7326	No Data	2670*	2763	1082*	
13000	11303*	7949	No Data	2860*	2920	1087*	
14000	12335*	8768	No Data	3087*	3113	No Data	
15000	13320*	9496	No Data	3222*	3274	No Data	
16000	14320*	10215	No Data	3300*	3426	No Data	0.26
17000	15451*	11034	No Data	No Data	2943*	No Data	0.31
18000	16657*	11882	No Data	No Data	3040*	No Data	0.37

*calculated from 1 strain gage.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	*Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall*								*Displacement (in.)*
	0-1ft	*1-3ft*	*3-5ft*	*5-7ft*	*7-9ft*	*9-11ft*	*11-12ft*	*Mean*	
1000	1789	-1097	-658	-658	1347	357	3024	283	
2000	4271	-913	-652	-652	1399	355	3035	531	
3000	4324	-212	-363	-363	1401	360	3035	750	
4000	5480	159	-37	-37	1517	426	3035	1048	
5000	1672	2266	421	421	1503	523	3035	1248	
6000	2515	2644	691	691	1533	608	3057	1492	
7000	3081	3108	1151	1151	1394	792	3062	1778	
8000	3149	3509	1594	1594	1157	945	3229	1998	
9000	3210	3902	2115	2115	769	1333	3229	2242	
10000	3598	4273	2603	2603	321	1781	3229	2499	
11000	4218	4536	3039	3039	-32	2134	3229	2740	
12000	4370	4845	3492	3492	-140	2522	3245	3003	
13000	5017	5032	3816	3816	-89	2748	3262	3244	
14000	4941	5350	4261	4261	-39	3113	3113	3496	
15000	4936	5736	4706	4706	-78	3274	3274	3741	
16000	4962	6157	5187	5187	-189	3426	3426	3994	0.26
17000	4664	6626	4045	4045	4045	2943	2943	4251	0.31
18000	4055	7162	4421	4421	4421	3040	3040	4502	0.37

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STRAIN GAGE REPORT FOR MSE REINFORCEMENT PULLOUT TEST

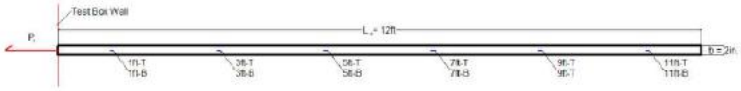
Test Information

Test Date:	June 21, 2011
Test Identification:	TS29-S8-Z40-L12-M
Test Facility:	12'x12'x4' MSE Test Box

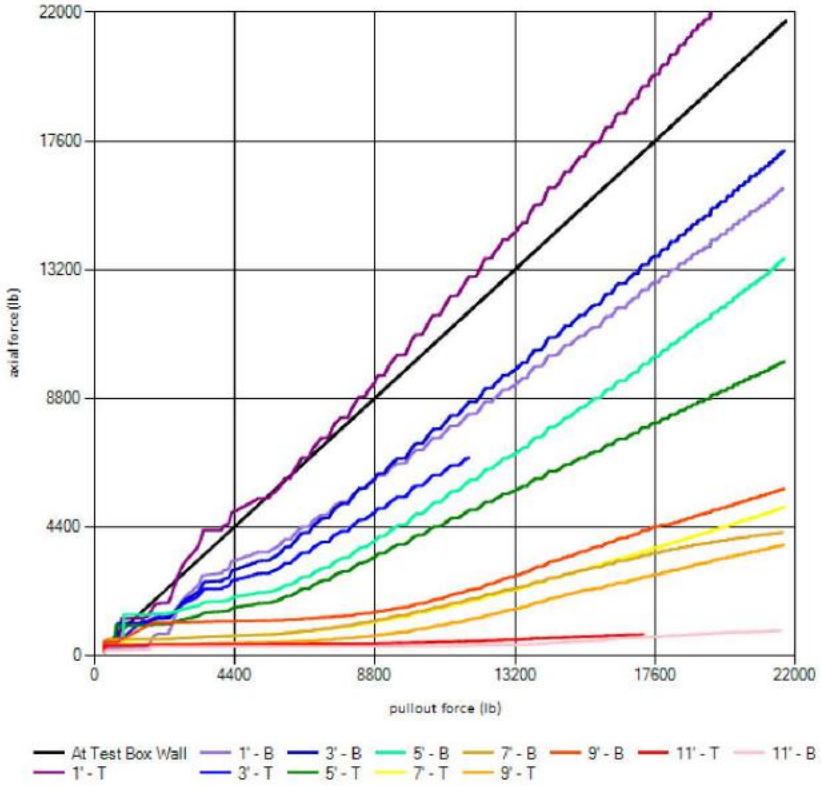
MSE Reinforcement

Type:	Ribbed Strip
Length, L_e (ft):	12
Width, b (in.):	2
Skew Angle, θ (°):	0

Test Specimen Sketch



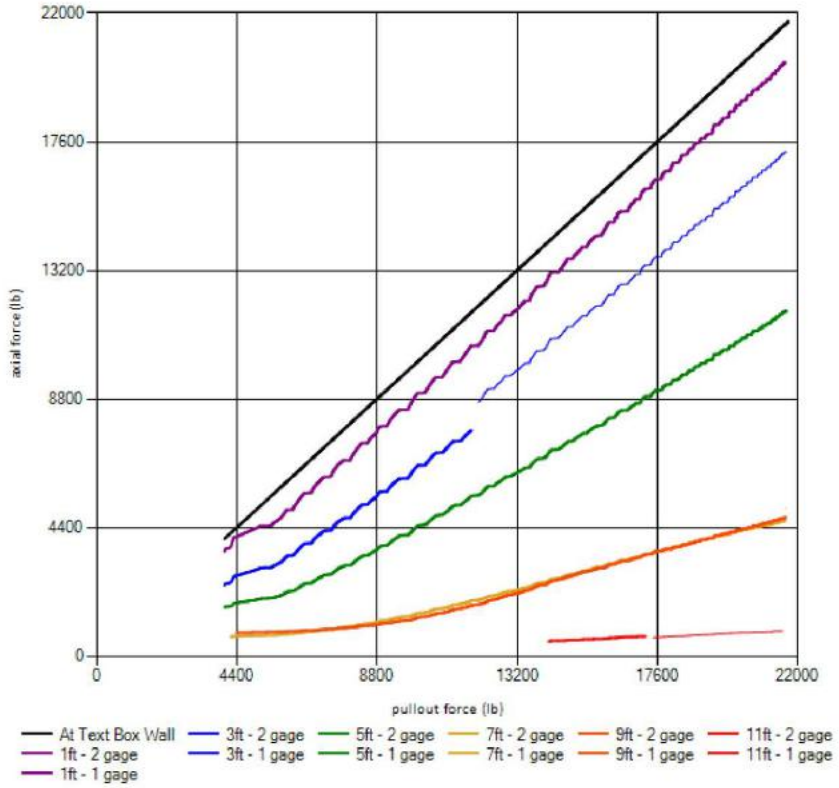
Axial Load-Pullout Load Data by Gage



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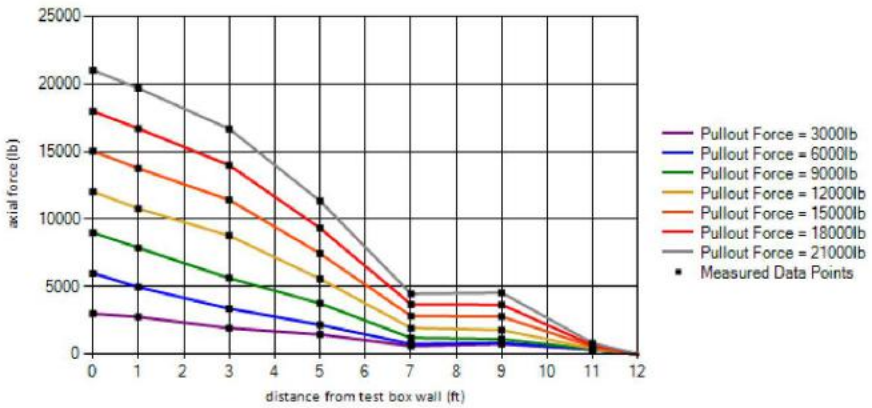
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

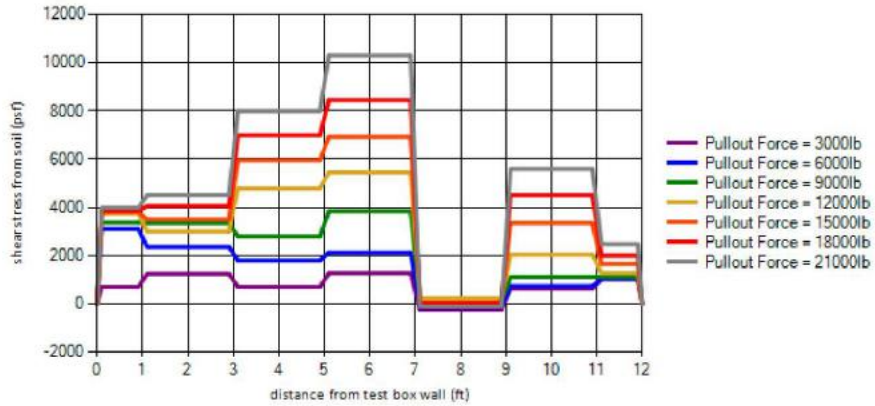
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	714	1132	1018	575	748	270	
2000	1181	1257	1243	575	755	337	
3000	2763	1933	1462	614	769	342	
4000	3642	2462	1690	658	798	344	
5000	4372	2953	1928	704	814	348	
6000	4961	3383	2180	758	843	349	
7000	5954	4138	2668	876	901	356	
8000	6892	4864	3193	1024	987	357	
9000	7875	5633	3768	1209	1114	373	
10000	8802	6367	4339	1421	1282	381	
11000	9835	7167	4988	1687	1533	397	
12000	10780	8777*	5581	1948	1788	427	
13000	11731	9652*	6182	2226	2105	449	
14000	12775	10556*	6821	2527	2447	498	
15000	13755	11421*	7453	2834	2787	554	
16000	14666	12220*	8035	3110	3067	602	
17000	15710	13126*	8689	3407	3384	663	
18000	16693	13990*	9332	3695	3673	669*	
19000	17696	14887*	10002	3970	3968	728*	
20000	18728	15800*	10677	4231	4266	793*	
21000	19686	16676*	11343	4480	4552	825*	

*calculated from 1 strain gage.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	*Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall*								*Displacement (in.)*
	0-1ft	*1-3ft*	*3-5ft*	*5-7ft*	*7-9ft*	*9-11ft*	*11-12ft*	*Mean*	
1000	701	-627	171	664	-260	717	811	237	
2000	2181	-113	20	1002	-269	627	1010	477	
3000	709	1245	706	1273	-233	641	1025	750	
4000	1201	1769	1159	1547	-209	680	1032	1010	
5000	1861	2128	1537	1837	-166	699	1044	1248	
6000	3110	2366	1805	2132	-127	741	1047	1499	
7000	3088	2724	2205	2689	-37	817	1067	1746	
8000	3395	3041	2507	3254	55	945	1072	2006	
9000	3375	3362	2798	3838	143	1111	1119	2250	
10000	3518	3652	3041	4378	208	1351	1144	2494	
11000	3578	4002	3268	4953	230	1705	1191	2757	
12000	3754	3004	4794	5449	240	2042	1280	3008	
13000	3834	3118	5205	5934	182	2483	1348	3252	
14000	3715	3328	5603	6442	119	2923	1495	3503	
15000	3816	3501	5953	6928	70	3350	1663	3757	
16000	3980	3669	6278	7388	64	3697	1806	3998	
17000	3883	3876	6656	7923	35	4082	1988	4251	
18000	3867	4054	6987	8455	33	4507	2006	4496	
19000	3897	4214	7327	9049	2	4861	2183	4749	
20000	3883	4392	7684	9670	-53	5210	2379	5006	
21000	3979	4515	7999	10295	-108	5590	2476	5253	

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Appendix D:

Strain Gage Data for MSE Reinforcement Pullout Tests:

Reinforced Earth Company Ladders in Type B Backfill

TABLE D: STRAIN GAGE DATA FOR MSE REINFORCEMENT PULLOUT TESTS
REINFORCED EARTH COMPANY LADDERS IN TYPE B BACKFILL

TestID	Length, Le (ft)	Width, b (in.)	Skew, β (°)	Relative Compaction, CR (%)
TS28-L1-Z3-L8-T	8	4	0	97.6%
TS28-L6-Z3-L8-M	8	4	0	97.6%
TS28-L2-Z6-L8-T	8	4	0	97.6%
TS28-L7-Z6-L8-M	8	4	0	97.6%
TS28-L8-Z6-L8-M	8	4	0	97.6%
TS28-L3-Z12-L8-T	8	4	0	97.6%

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STRAIN GAGE REPORT FOR MSE REINFORCEMENT PULLOUT TEST

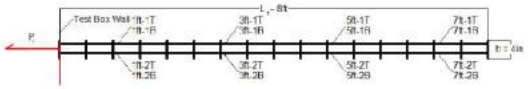
Test Information

Test Date:	June 14, 2011
Test Identification:	TS28-L1-Z3-L8-T
Test Facility:	12'x12'x4' MSE Test Box

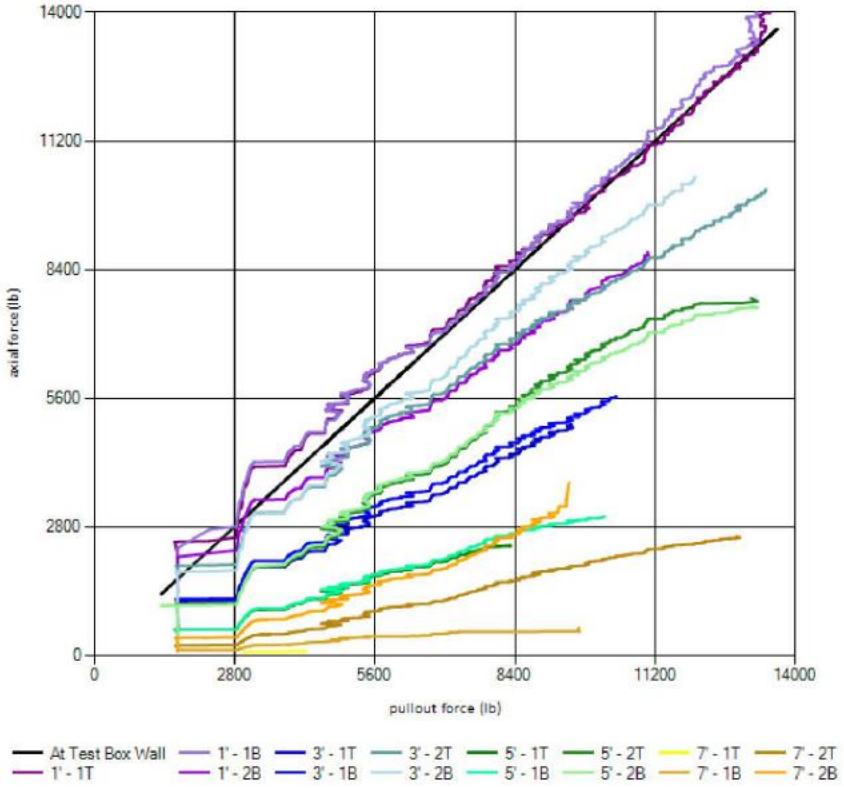
MSE Reinforcement

Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, θ (°):	0

Test Specimen Sketch



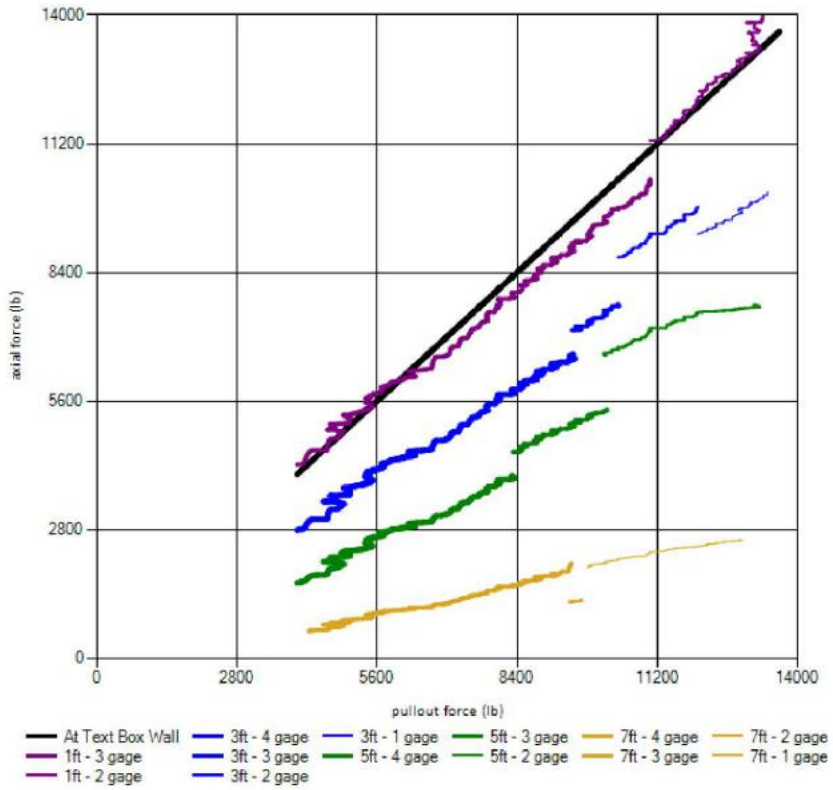
Axial Load-Pullout Load Data by Gage



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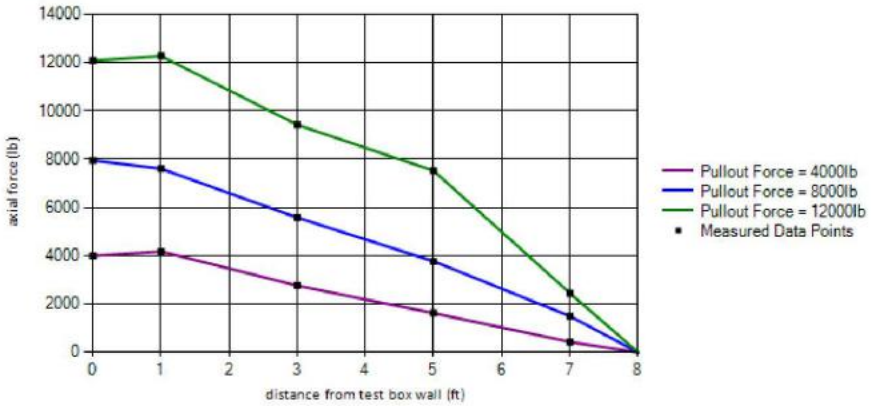
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

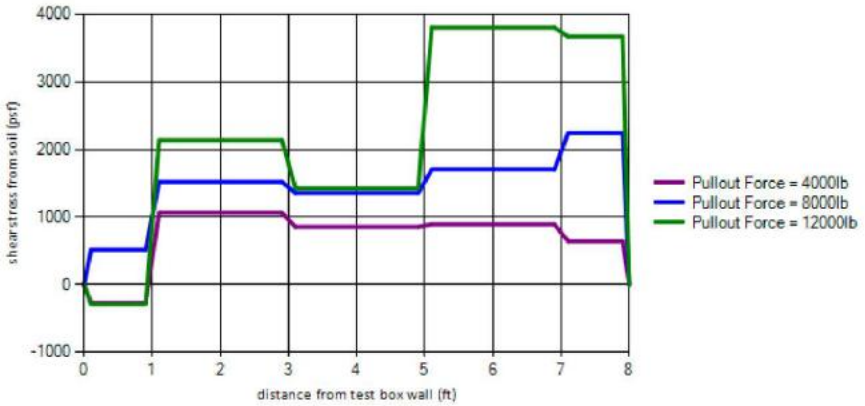
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	2287*	1375	676	177*			
2000	2507*	1550	839	245*			
3000	3486*	2210	1232	314			
4000	4179*	2760	1620	429			
5000	5167*	3588	2233	788*			
6000	6034*	4335	2814	1048*			
7000	6677*	4836	3182	1210*			
8000	7605*	5578	3769	1495*			
9000	8523*	6263	4844*	1770*			
10000	9349*	7389*	5524*	2033***			
11000	10453*	9125**	7074**	2272***			0.33
12000	12272**	9423***	7522**	2448***			0.53
13000	13356**	9929***	7640**	2577***			0.96

*calculated from 3 strain gage. **calculated from 2 strain gages. ***calculated from 1 strain gages.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall								Displacement (in.)
	0-1ft	1-3ft	3-5ft	5-7ft	7-9ft	9-11ft	11-12ft	Mean	
1000	-941	684	525	374	266			311	
2000	-632	718	533	446	367			391	
3000	-671	957	733	688	471			570	
4000	-270	1064	855	893	643			750	
5000	-276	1184	1016	1084	1182			934	
6000	-67	1274	1141	1325	1572			1123	
7000	544	1381	1240	1479	1815			1320	
8000	515	1520	1357	1705	2243			1490	
9000	673	1694	1064	2306	2655			1682	
10000	926	1470	1399	2618	3050			1869	
11000	834	996	1538	3602	3407			2064	0.33
12000	-286	2137	1426	3805	3673			2265	0.53
13000	-432	2570	1717	3797	3865			2450	0.96

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STRAIN GAGE REPORT FOR MSE REINFORCEMENT PULLOUT TEST

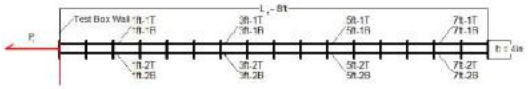
Test Information

Test Date:	June 15, 2011
Test Identification:	TS28-L6-Z3-L8-M
Test Facility:	12'x12'x4' MSE Test Box

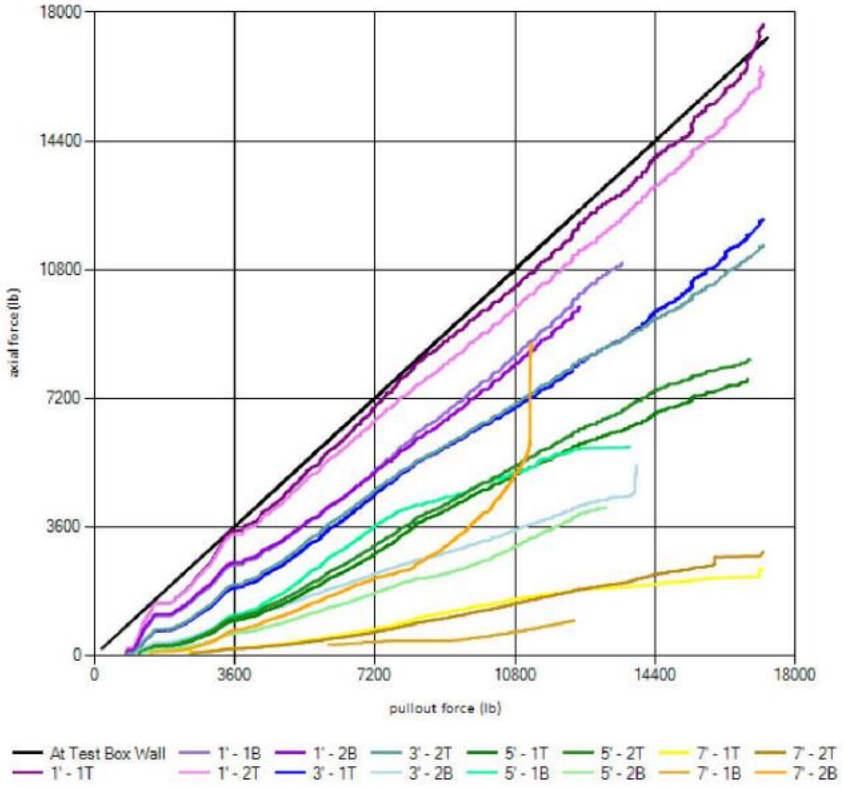
MSE Reinforcement

Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, θ (°):	0

Test Specimen Sketch



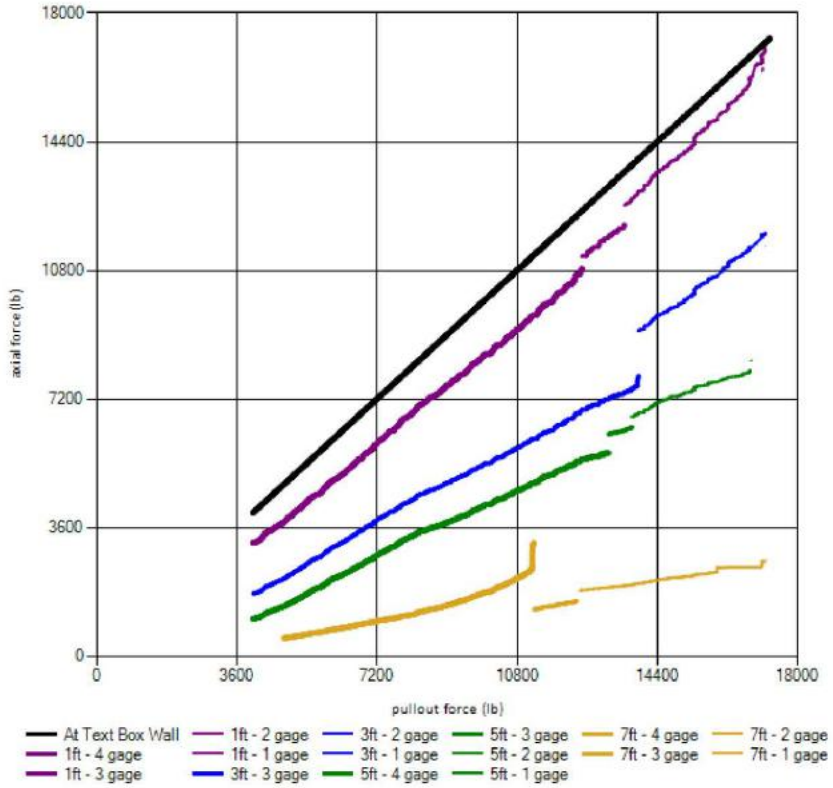
Axial Load-Pullout Load Data by Gage



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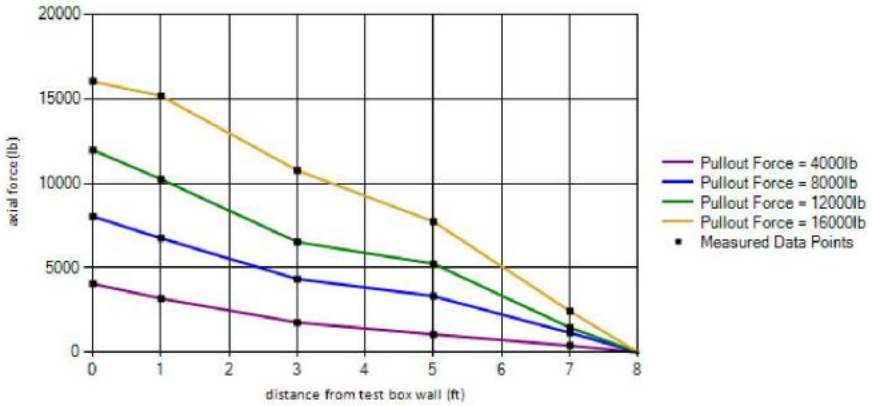
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

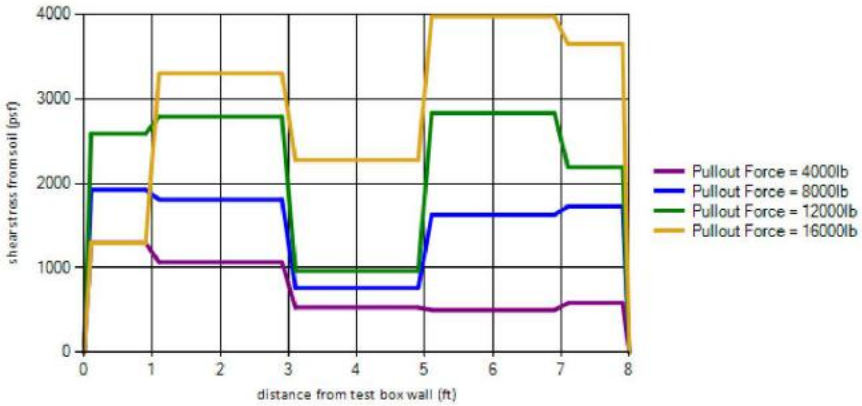
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	340	163**	88*	298***			
2000	1379	645*	276	212**			
3000	2347	1207*	619	235			
4000	3184	1763*	1056	389			
5000	3932	2293*	1499	533			
6000	4845	2992*	2110	726			
7000	5746	3656*	2689	937			
8000	6748	4337*	3322	1151			
9000	7567	4846*	3742	1423			
10000	8438	5385*	4230	1791			
11000	9353	5962*	4740	2315			
12000	10243	6523*	5238	1461*			
13000	11659*	7157*	5849	1915**			
14000	13081**	8611**	6839**	2052**			
15000	14049**	9908**	7331**	2216**			0.37
16000	15165**	10764**	7733**	2432**			0.6
17000	16564**	11660**	8171***	2578**			0.98

*calculated from 3 strain gage. **calculated from 2 strain gages. ***calculated from 1 strain gages.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall								Displacement (in.)
	0-1ft	1-3ft	3-5ft	5-7ft	7-9ft	9-11ft	11-12ft	Mean	
1000	1043	133	56	-158	448			194	
2000	992	550	277	48	317			383	
3000	1004	855	441	288	353			566	
4000	1295	1065	531	500	584			759	
5000	1590	1229	596	725	799			936	
6000	1743	1390	662	1038	1089			1126	
7000	1884	1568	725	1314	1405			1313	
8000	1924	1808	761	1628	1727			1506	
9000	2157	2041	829	1739	2135			1688	
10000	2357	2290	866	1829	2687			1877	
11000	2502	2544	916	1819	3473			2066	
12000	2588	2790	964	2832	2192			2244	
13000	2080	3376	982	2950	2872			2446	
14000	1407	3352	1329	3591	3077			2629	
15000	1482	3106	1933	3836	3325			2819	0.37
16000	1292	3301	2273	3975	3648			3005	0.6
17000	643	3678	2617	4194	3867			3186	0.98

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STRAIN GAGE REPORT FOR MSE REINFORCEMENT PULLOUT TEST

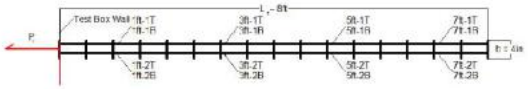
Test Information

Test Date:	June 15, 2011
Test Identification:	TS28-L2-Z6-L8-T
Test Facility:	12'x12'x4' MSE Test Box

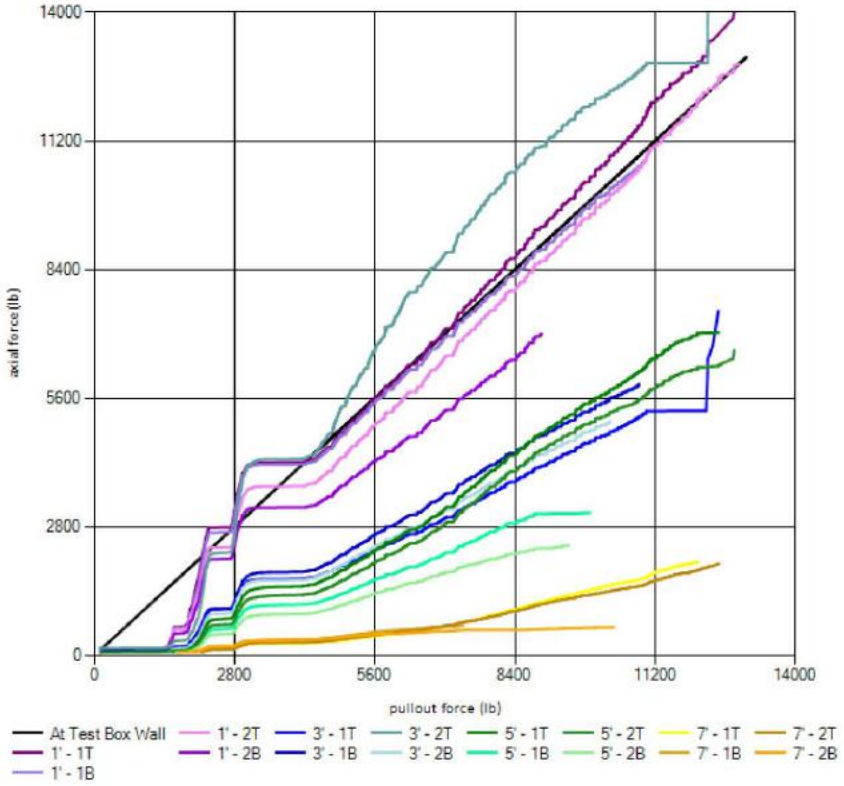
MSE Reinforcement

Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, θ (°):	0

Test Specimen Sketch



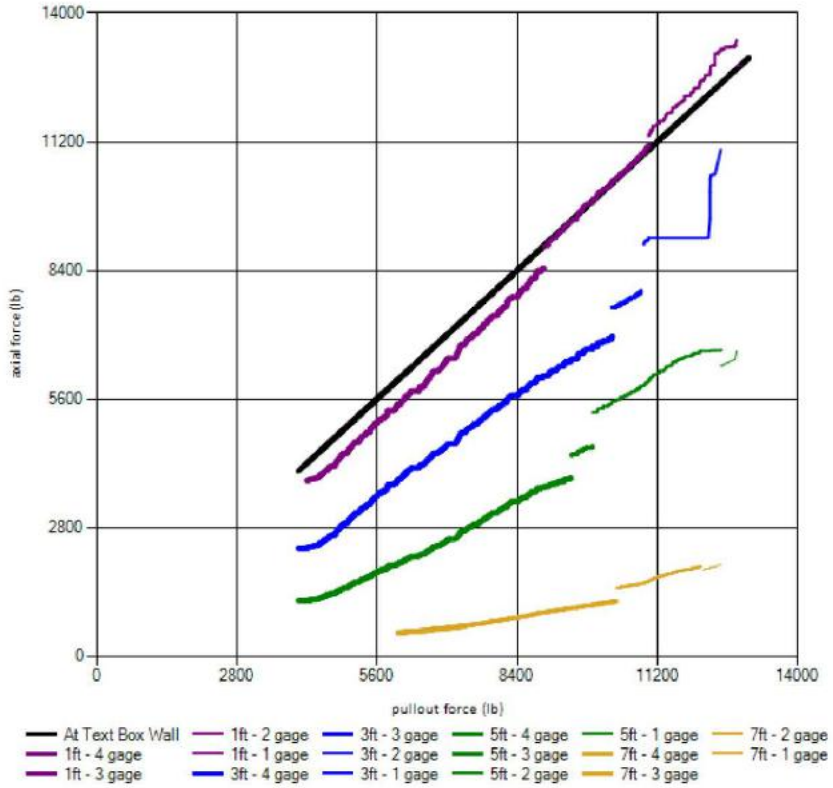
Axial Load-Pullout Load Data by Gage



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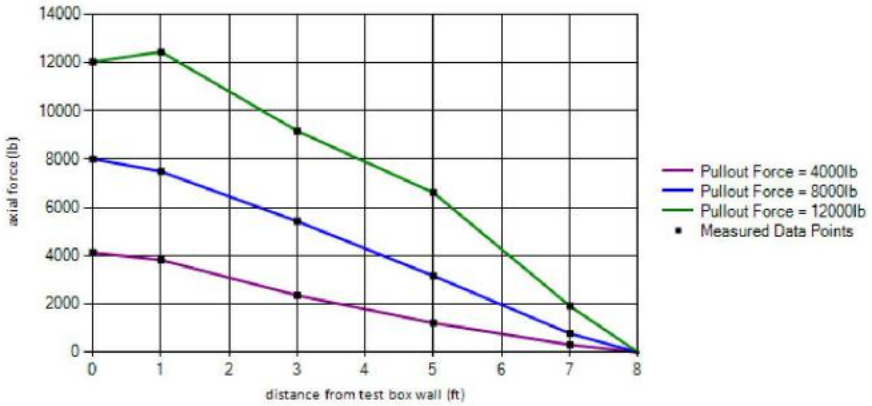
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

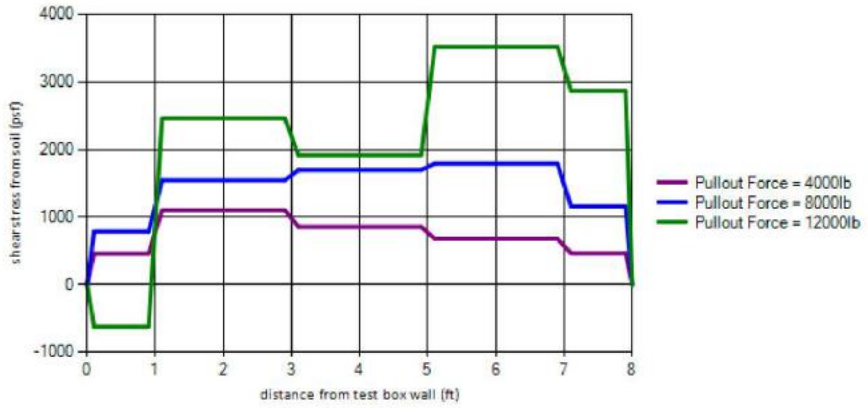
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	138	107	85***	67***			
2000	1384	578	249	92**			
3000	3510	2058	1010	255			
4000	3824	2357	1213	310			
5000	4451	2916	1503	383			
6000	5456	3823	2012	502			
7000	6411	4568	2500	608			
8000	7487	5424	3161	772*			
9000	8840*	6149	3709	968*			
10000	10043*	6786	5119**	1136*			
11000	11177*	8690**	5969**	1625**			
12000	12443**	9167**	6609**	1912**			0.52

*calculated from 3 strain gage. **calculated from 2 strain gages. ***calculated from 1 strain gages.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall								Displacement (in.)
	0-1ft	1-3ft	3-5ft	5-7ft	7-9ft	9-11ft	11-12ft	Mean	
1000	1795	23	16	14	100			250	
2000	988	605	246	118	138			383	
3000	-751	1089	786	566	383			564	
4000	460	1100	858	677	466			774	
5000	812	1151	1060	840	574			936	
6000	814	1224	1359	1133	753			1125	
7000	877	1382	1551	1419	913			1312	
8000	785	1548	1697	1791	1159			1502	
9000	264	2018	1830	2056	1452			1691	
10000	-42	2443	1250	2987	1703			1878	
11000	-288	1865	2041	3258	2438			2060	
12000	-621	2458	1918	3522	2868			2256	0.52

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STRAIN GAGE REPORT FOR MSE REINFORCEMENT PULLOUT TEST

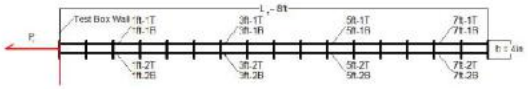
Test Information

Test Date:	June 15, 2011
Test Identification:	TS28-L7-Z6-L8-M
Test Facility:	12'x12'x4' MSE Test Box

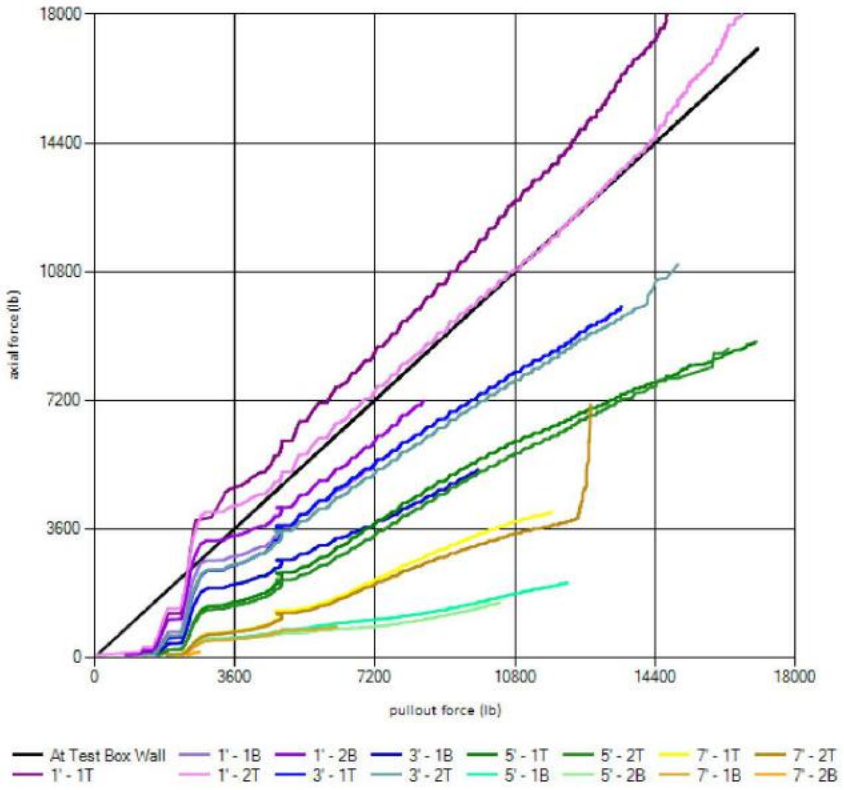
MSE Reinforcement

Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, θ (°):	0

Test Specimen Sketch



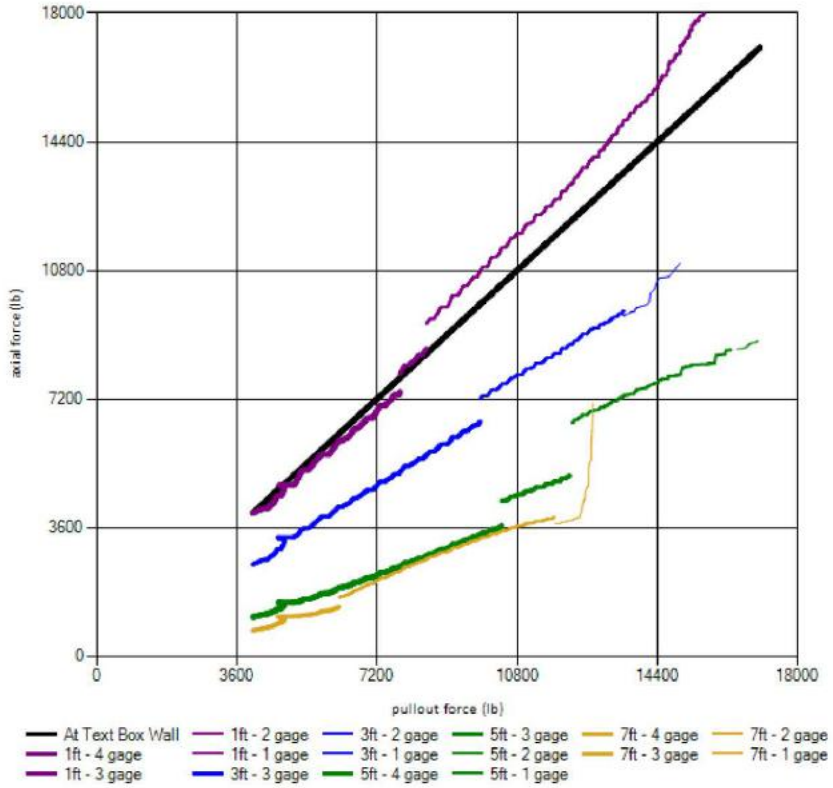
Axial Load-Pullout Load Data by Gage



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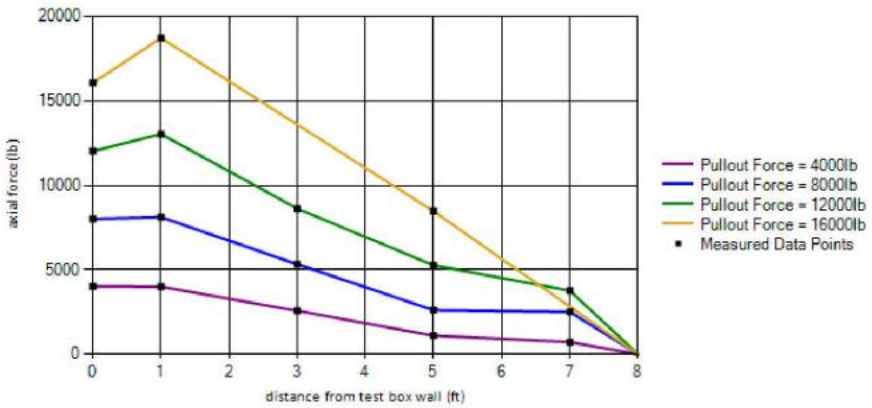
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

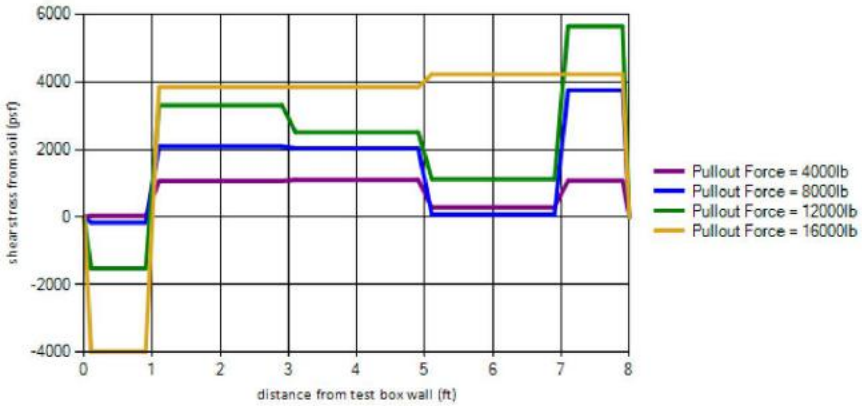
Pullout Resistance, P, (lb)	Axial Load, P (lb) calculated from strain data at distances from test box wall						Displacement (in.)
	1ft	3ft	5ft	7ft	9ft	11ft	
1000	125**	75***	74**	69***			
2000	1027	476*	148*	80**			
3000	3548	2255*	935	586*			
4000	3999	2572*	1100	722*			
5000	4784	3268*	1464	1053*			
6000	5789	4008*	1815	1312*			
7000	6663	4643*	2189	2007**			
8000	8114*	5326*	2612	2505**			
9000	9830**	5963*	3014	2928**			
10000	10890**	7102**	3440	3330**			
11000	11994**	7975**	4582*	3684**			
12000	13033**	8620**	5264*	3763***			
13000	14270**	9319**	6997**	No Data			0.28
14000	15468**	9818***	7480**	No Data			0.35
15000	17099**	10999***	8008**	No Data			0.47
16000	18720**	No Data	8463**	No Data			0.58

*calculated from 3 strain gage. **calculated from 2 strain gages. ***calculated from 1 strain gages.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall								Displacement (in.)
	0-1ft	1-3ft	3-5ft	5-7ft	7-9ft	9-11ft	11-12ft	Mean	
1000	1515	37	1	4	104			213	
2000	1412	414	246	51	119			369	
3000	-851	970	990	262	879			559	
4000	37	1070	1104	283	1083			754	
5000	253	1137	1353	308	1579			929	
6000	350	1336	1645	377	1968			1129	
7000	501	1515	1840	137	3010			1312	
8000	-159	2090	2036	80	3758			1501	
9000	-1297	2900	2212	64	4392			1681	
10000	-1392	2841	2747	82	4995			1868	
11000	-1502	3015	2545	673	5526			2061	
12000	-1521	3310	2517	1126	5644			2254	
13000	-1844	3714	1741	3498	3498			2445	0.28
14000	-2204	4238	1753	3740	3740			2625	0.35
15000	-3091	4576	2243	4004	4004			2820	0.47
16000	-3982	3847	3847	4231	4231			3012	0.58

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STRAIN GAGE REPORT FOR MSE REINFORCEMENT PULLOUT TEST

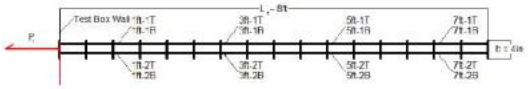
Test Information

Test Date:	June 16, 2011
Test Identification:	TS28-L8-Z6-L8-M
Test Facility:	12'x12'x4' MSE Test Box

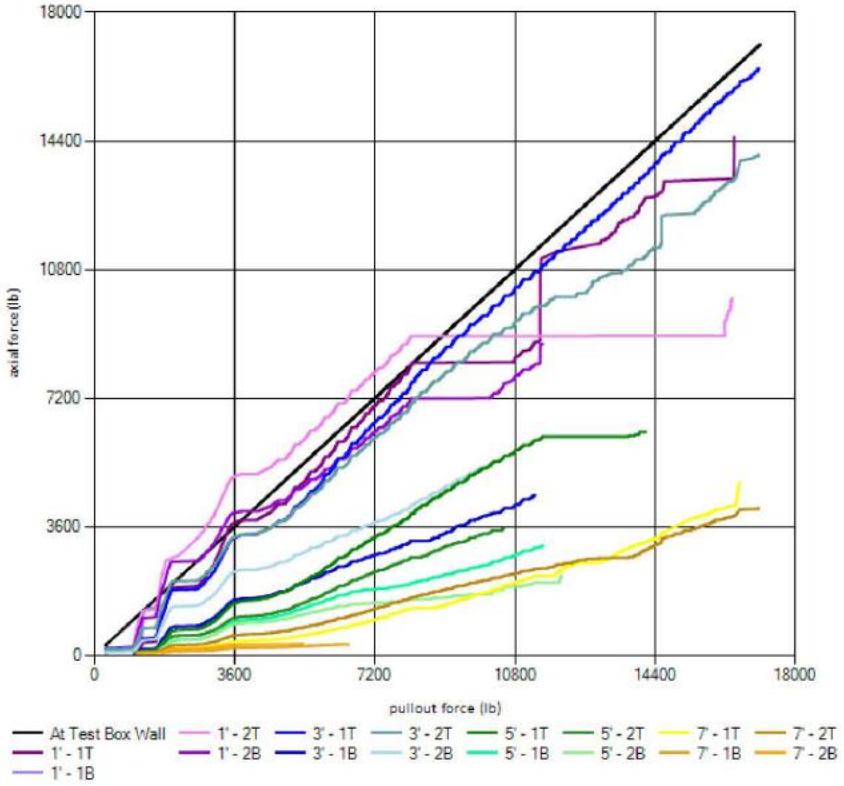
MSE Reinforcement

Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, θ (°):	0

Test Specimen Sketch



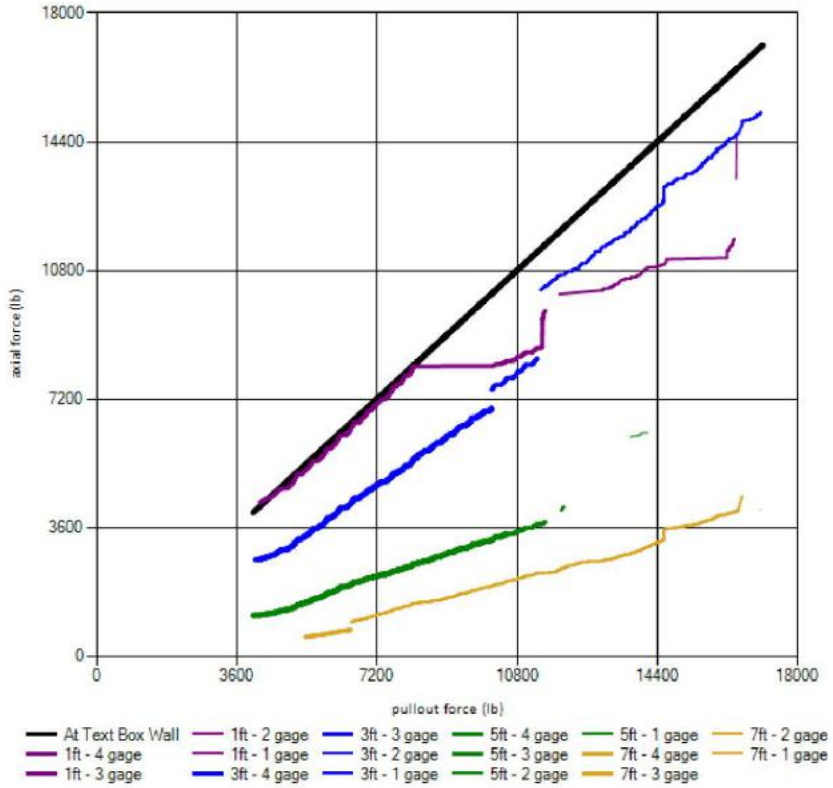
Axial Load-Pullout Load Data by Gage



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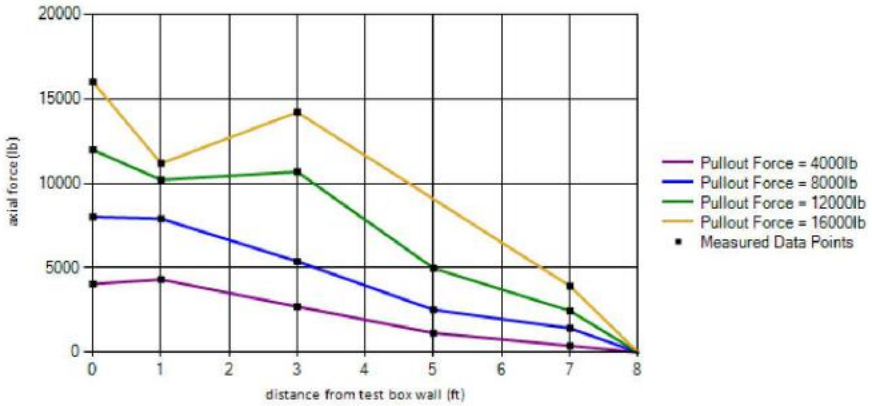
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

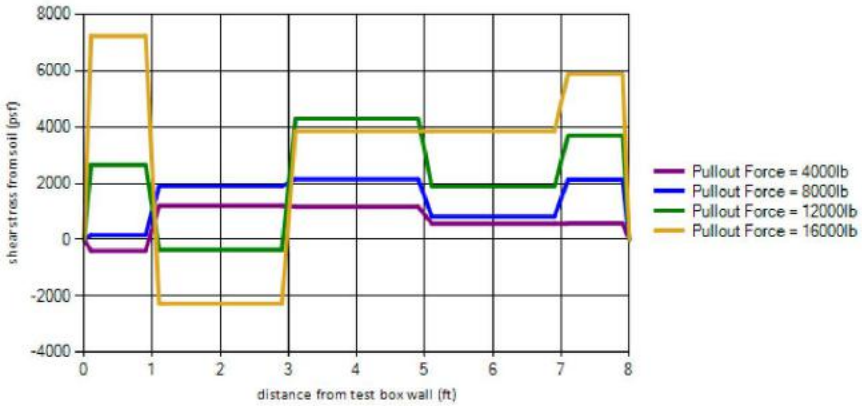
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	395*	238	90**	71*			
2000	2355*	1401	466	175			
3000	3107*	1843	679	243			
4000	4307*	2701	1139	383			
5000	4863*	3107	1354	442			
6000	5830*	3873	1777	641*			
7000	6848*	4639	2179	1088**			
8000	7903*	5379	2510	1421**			
9000	8120*	6135	2870	1628**			
10000	8125*	6995	3215	1923**			
11000	8447*	8061*	3551*	2216**			
12000	10205**	10683**	4968**	2455**			
13000	10274**	11441**	6127***	2683**			
14000	10775**	12229**	6212***	3007**			0.31
15000	11148**	13371**	No Data	3617**			0.4
16000	11176**	14207**	No Data	3920**			0.5
17000	No Data	15109**	No Data	4106***			0.61

*calculated from 3 strain gage. **calculated from 2 strain gages. ***calculated from 1 strain gages.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	*Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall*								*Displacement (in.)*
	0-1ft	*1-3ft*	*3-5ft*	*5-7ft*	*7-9ft*	*9-11ft*	*11-12ft*	*Mean*	
1000	948	118	111	14	107			193	
2000	-525	716	702	218	263			376	
3000	-173	948	873	328	364			561	
4000	-396	1205	1172	567	575			758	
5000	217	1317	1314	684	662			939	
6000	256	1468	1572	852	962			1125	
7000	232	1657	1845	818	1633			1313	
8000	170	1893	2151	817	2132			1503	
9000	1328	1489	2449	932	2441			1688	
10000	2804	847	2835	969	2884			1874	
11000	3805	289	3383	1001	3324			2059	
12000	2651	-358	4286	1885	3683			2245	
13000	4108	-875	3986	2583	4025			2440	
14000	4896	-1090	4513	2404	4511			2632	0.31
15000	5745	-1668	3658	3658	5426			2808	0.4
16000	7230	-2273	3858	3858	5880			2999	0.5
17000	907	907	4126	4126	6159			3173	0.61

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STRAIN GAGE REPORT FOR MSE REINFORCEMENT PULLOUT TEST

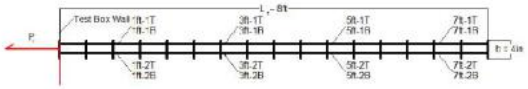
Test Information

Test Date:	June 16, 2011
Test Identification:	TS28-L3-Z12-L8-T
Test Facility:	12'x12'x4' MSE Test Box

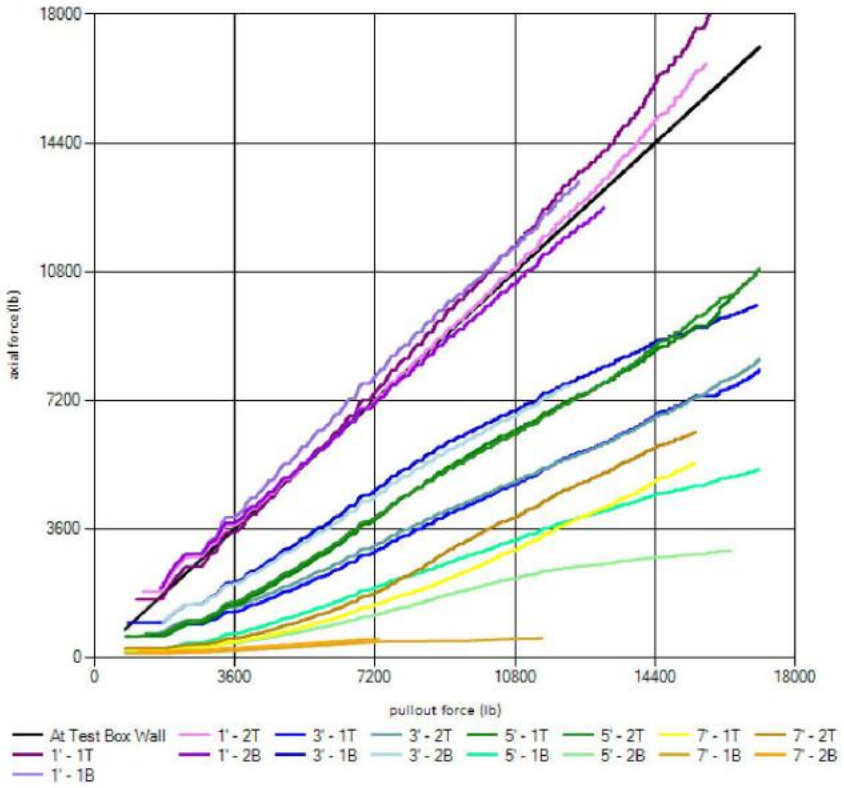
MSE Reinforcement

Type:	RECo HA Ladder
Length, L_e (ft):	8
Width, b (in.):	4
Skew Angle, θ (°):	0

Test Specimen Sketch



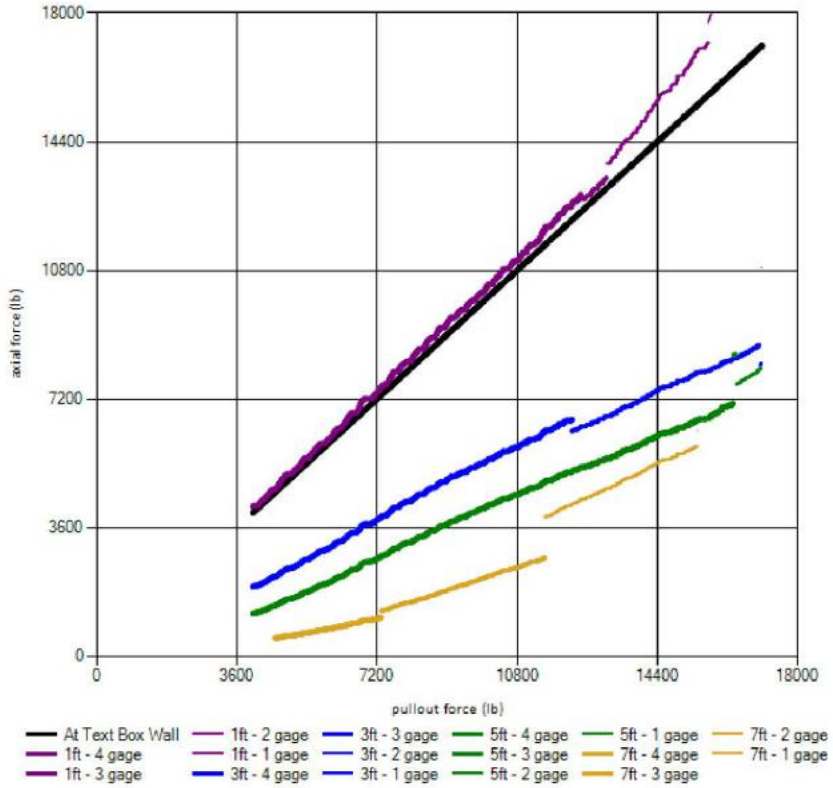
Axial Load-Pullout Load Data by Gage



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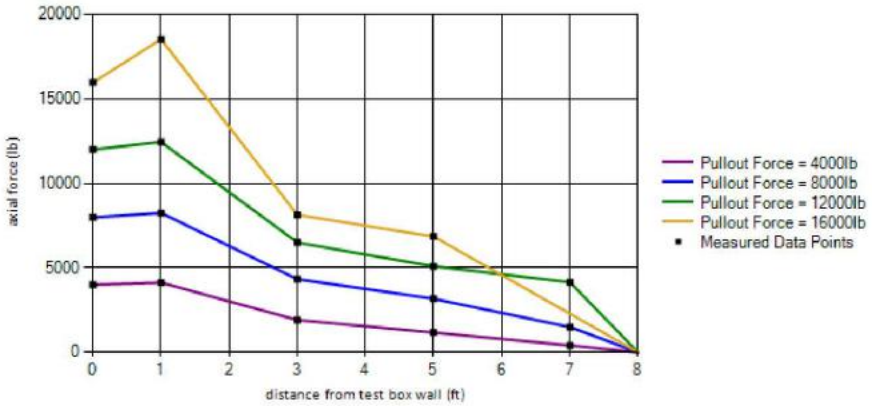
Axial Load-Pullout Load Data by Location



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Axial Load-Distance from Test Box Wall Plot



Axial Load Results

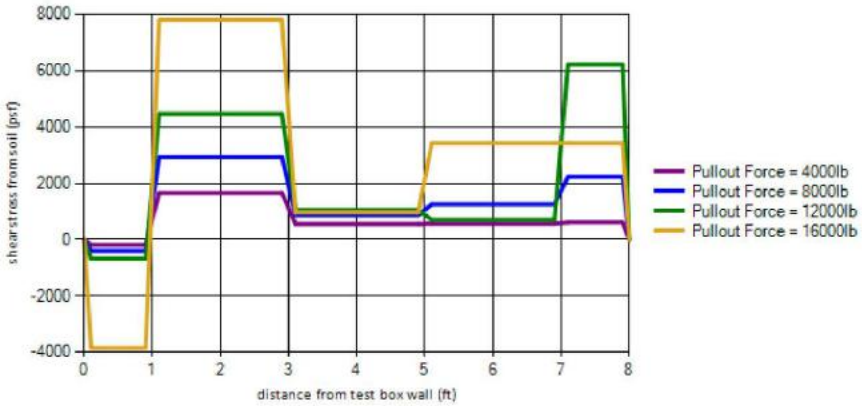
Pullout Resistance, P, (lb)	*Axial Load, P (lb) calculated from strain data at distances from test box wall*						*Displacement (in.)*
	1ft	*3ft*	*5ft*	*7ft*	*9ft*	*11ft*	
1000	1660***	821	410	183			
2000	2304	1000	500	204			
3000	3152	1402	776	281			
4000	4123	1910	1168	408			
5000	5155	2483	1621	578			
6000	6150	3064	2096	774			
7000	7219	3710	2640	1012			
8000	8247	4326	3170	1491*			
9000	9313	4930	3695	1842*			
10000	10293	5455	4178	2210*			
11000	11330	5975	4622	2566*			
12000	12448	6493	5088	4148**			0.25
13000	13385*	6659*	5481	4632**			0.31
14000	14953**	7196*	5952	5171**			0.39
15000	16276**	7708*	6399	5669**			0.49
16000	18529***	8133*	6850	No Data			0.61
17000	20965***	8526*	8198**	No Data			0.76

*calculated from 3 strain gage. **calculated from 2 strain gages. ***calculated from 1 strain gages.

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Shear Stress from Soil-Distance from Test Box Wall Plot



Shear Stress from Soil Results

Pullout Resistance, P, (lb)	Average Shear Stress from Soil (psf) calculated from strain data at distances from test box wall								Displacement (in.)
	0-1ft	1-3ft	3-5ft	5-7ft	7-9ft	9-11ft	11-12ft	Mean	
1000	-1089	629	309	170	274			175	
2000	-461	978	375	222	306			374	
3000	-186	1313	470	371	421			568	
4000	-189	1660	557	569	613			750	
5000	-220	2004	646	782	867			939	
6000	-206	2314	726	991	1162			1127	
7000	-336	2631	803	1221	1518			1311	
8000	-405	2940	868	1259	2237			1496	
9000	-486	3288	926	1390	2764			1686	
10000	-437	3629	958	1476	3314			1875	
11000	-473	4016	1015	1543	3849			2065	
12000	-678	4466	1054	705	6223			2249	0.25
13000	-577	5045	883	637	6948			2438	0.31
14000	-1403	5818	933	586	7757			2628	0.39
15000	-1887	6426	982	548	8504			2816	0.49
16000	-3845	7797	962	3425	3425			2994	0.61
17000	-6052	9330	246	4099	4099			3175	0.76

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EFFECT OF SKEWING AND SPLAYING ON PULLOUT CAPACITY OF STEEL MSE REINFORCEMENT

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EFFECT OF SKEWING AND SPLAYING ON PULLOUT CAPACITY OF STEEL MSE REINFORCEMENT

Priyantha Jayawickrama, William D. Lawson, Timothy A. Wood, James Surles and Asitha J. Senanayake

ABSTRACT

Structures embedded in the reinforced fill of Mechanically Stabilized Earth (MSE) retaining walls often prevent the soil reinforcements from being placed in their proper, design configurations. Such conflicts are generally addressed by using alternative reinforcement layouts that circumvent the obstructions. Among the strategies used most commonly are *skewing* of strip type reinforcements and *cutting and splaying* of grid type reinforcement. This research investigated the impact of skewing and splaying on the pullout resistance capacity of these two types of reinforcement. The research involved a series of pullout tests that were conducted in a specially designed large scale pullout resistance test system. This MSE pullout load test system included an MSE test box with dimensions 12ft by 12ft (3.66m by 3.66m) in plan area and 4ft (1.22m) in depth. In this test system, the soil overburden pressures acting on the embedded earth reinforcement is simulated by using a reaction frame assembly which consists of nine 4ft by 4ft (1.22m by 1.22m) pressure plates that are hydraulically jacked against three wide flange cross beams. The reaction frame assembly allows simulation of overburden pressures up to 40ft (12.2m) of fill. The pullout test program included: (a) tests conducted on strip type reinforcement with skew angles of 0°, 15° and 30° and (b) tests conducted on grid type reinforcement with splay angles of 0°, 15° and 30°. The pullout loads obtained are then compared to determine the impact of skewing and splaying on the pullout resistance.

Key Words: skewing, splaying, MSE reinforcement, pullout resistance

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INTRODUCTION

Conflict between obstructions located behind the wall facing and soil reinforcement is one of the most common problems encountered in MSE wall construction. Examples of such obstructions include foundations for bridge abutments, foundations for traffic sign structures and light poles, drainage inlets and storm sewers. These obstructions prevent the reinforcements from being placed in their proper design configuration. Ideally, such potential conflicts should be identified during preliminary planning stages and the obstructions moved away from reinforced fill and hence avoid the problem. However, it is not always possible to relocate structures to avoid conflicts. Therefore, wall designers and contractors must often rely on alternative reinforcement layouts to circumvent obstructions that are found within the reinforced fill. Such alternative reinforcement layouts have the potential to impact the wall performance in a number of ways. First, the alternative reinforcement may cause the lateral earth pressure distribution on the wall panels to be different from that used in the design. Secondly, the alternative reinforcement configuration can impact the pullout resistance offered by the MSE reinforcement. Little or no data is available in published literature on the influence of alternative reinforcement details on pullout resistance. This paper describes a pullout resistance test program conducted with a large scale test system to measure pullout resistance of skewed reinforcing strips and cut and splayed grid reinforcement.

ALTERNATIVE REINFORCEMENT LAYOUTS

Several different strategies are used when modifying soil reinforcement layout to avoid obstructions (*1*). Providing the wall facing with additional connectors and then shifting the soil reinforcing laterally to avoid the obstruction is one commonly used strategy. Another strategy involves the use of a structural frame around the obstruction. In this approach, the reinforcing elements near the obstruction are not connected to the facing directly; instead they are connected to the structural frame and then the frame attached to the facing panels.

This research is concerned with two other alternative reinforcement layouts commonly used to circumvent obstructions. The first, used with strip type reinforcement with nut and bolt connections, involves *skewing* of the reinforcement as shown in Figure 1 (*1, 2*). The second, used with grid type reinforcement, involves cutting the transverse bars of the grid and splaying the longitudinal bars around the obstruction. Figure 2 depicts the above cut-and-splay reinforcement configuration. When skewing or splaying is used, it is recommended that skew or splay angles do not exceed 15-deg. The skew or splay angle is defined as the angle measured from a line perpendicular to the wall face in a horizontal plane. It is further recommended that the tensile capacity of the reinforcement be reduced by multiplying it with the cosine of the skew (or splay) angle (*1*). The primary objective of this research study is to collect data that would check the validity of the above approach.

PULLOUT RESISTANCE OF SOIL REINFORCEMENT

Although the alternative reinforcement details described above are frequently used in MSE wall construction, the impact they may have on wall stability has not been studied in detail. This research focused on the impact of the alternative reinforcement details on one important aspect of wall design, i.e. stability against *pullout failure*.

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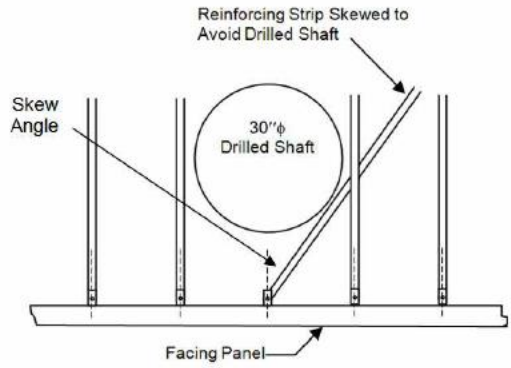


FIGURE 1. Skewing Strip Type Reinforcement to Avoid Vertical Obstructions

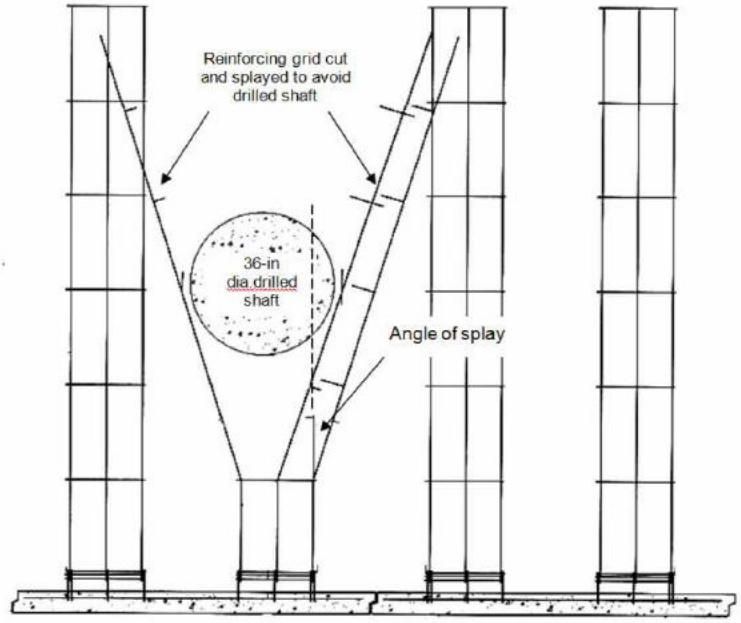


FIGURE 2. Cutting and Splaying Grid Type Reinforcement to Avoid Vertical Obstructions

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Analysis of MSE walls for pullout failure begins with an assumption regarding the location of the *line of maximum tensile stress* or *potential failure surface* within the reinforced backfill. This line of maximum tensile stress separates the reinforced fill into two zones: active zone and resistant zone. Data collected from instrumented MSE walls have shown that the location of the line of maximum stress varies depending on whether the type of reinforcement used is *inextensible* or *extensible*. Figure 3 shows the lines of maximum stress commonly used MSE wall design (*1*). When performing stability analysis with respect to reinforcement pullout, the tensile force, T_{max} carried by each reinforcement element is calculated first based on the lateral earth pressure acting on an appropriate tributary area. Next, the maximum pullout resistance that would develop along the length of reinforcement embedded within the resistant zone is determined. Finally, the factor of safety with respect to pullout failure is estimated as shown in Equation (1) below (*3*). The length of the reinforcement should be such that it provides a minimum factor of safety of 1.5.

$$FS_{PO} = \frac{F^* \sigma_v L_e C R_c \alpha}{T_{max}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where: FS_{PO} – Factor of safety against pullout failure, F^*– Pullout resistance factor, σ_v – Vertical soil overburden stress acting on the soil reinforcement, L_e–Effective length of reinforcement, R_c – Coverage ratio (width of reinforcing element/horizontal spacing between reinforcing elements), α – Scale correction factor (α =1 for inextensible reinforcements), C – Effective unit parameter (C = 2 for strips and grids) T_{max} – Maximum tensile load on soil reinforcement

The pullout resistance factor, F^* in the above equation depends on a number of factors. The most important among them are the type of reinforcement and the type of backfill used in wall construction. Ideally, F^* to be used in a given design should be obtained from pullout resistance tests conducted using the specific backfill-reinforcement combination.

In the absence of such data, the following equations provided in AASHTO LRFD Bridge Design Specifications may be used to estimate default F^* values (*3*).

For steel ribbed reinforcement:

$$F^* = 1.2 + \log \cdot C_u \leq 2.0 \text{ at the top of the wall} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2a)$$

$$F^* = \tan\phi \text{ at a depth 20ft and below} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2b)$$

where:

C_u = Coefficient of uniformity for backfill (Use C_u = 4.0 as default)

ϕ = Angle of internal friction for backfill (Use ϕ = 34° as default)

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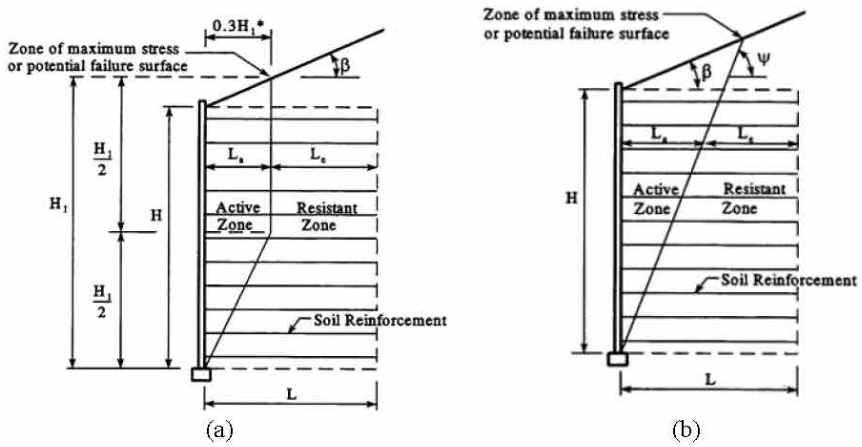


FIGURE 3. Line of Maximum Tensile Stress (a) Inextensible Soil Reinforcement and (b) Extensible Soil Reinforcement [Source: FHWA NHI-10-024, 2009]

For steel grid reinforcement:

$F^* = 20\left(\frac{t}{S_t}\right)$ at the top of the wall ..... (3a)

$F^* = 10\left(\frac{t}{S_t}\right)$ at a depth 20ft and below ..... (3b)

where:

t = Diameter of the transverse bar
S_t = Transverse bar spacing

PULLOUT RESISTANCE TESTING

A review of literature undertaken at the outset of this research identified a number of previous research studies that investigated pullout resistance behavior of MSE reinforcement in the laboratory. Many of these experimental studies involved pullout testing of steel strip and grid type reinforcement. These include Chang *et al.* (*4*), Peterson (*5*), Hannon and Forsyth (*6*), Nielsen and Anderson (*7*), Bergado *et al.* (*8, 9*) and Matsui *et al.* (*10*). The overall configuration of the test systems used by different researchers remained the same. The test systems included a soil box in which the reinforcing elements were embedded, a mechanism to apply large overburden pressures and mechanisms to apply and measure pullout forces and displacements of the reinforcement. The overall dimensions of the soil boxes varied from 4.0-5.0ft (1.22-1.52m) in length, 1.67-6.0ft (0.51-1.83m) in width and 1.5-4.0ft (0.46-1.22m) in height. While most of pullout testing had been conducted in soil boxes with enclosed side walls on all four sides, some

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of the pullout testing had been conducted with an open front face (*4*). Two separate mechanisms were used to simulate large overburden pressures at the interface between soil and reinforcement. The first of these involved a set of hydraulic rams and bearing plates that apply vertical pressure on the top soil surface by pushing against a stationary reaction frame. The second mechanism used air bags that were pressurized between the top soil surface and a cover plate to simulate additional overburden pressure. Pullout loads were applied by using hydraulic rams that were pressured against the front face of the soil box or a stationary reaction frame. Although the same basic mechanism was used in all research studies to apply the pullout load, variations were found in the loading procedures used. While displacement controlled loading with constant strain rate was found to be the most commonly used procedure, some research studies used load controlled approach in which pullout loads were applied in specified load increments (*5*, *7*). Variations were also observed in the use of a seating load to get rid of slack in the loading assembly. Generally, the measurement of pullout loads was achieved with load cells while LVDTs and extensometers were used to monitor displacements.

PULLOUT RESISTANCE TESTING PROGRAM

Large Scale Pullout Load Test System

FHWA Publication FHWA NHI-00-043 recommends that test boxes used for pullout resistance testing should have a minimum length of 4ft (1.21m), a minimum width of 2.5ft (7.60m) and a minimum depth of 1.5ft (0.45m) (*11*). However, a test box meeting those minimum dimension requirements will not be large enough for testing the pullout resistance of alternative reinforcement arrangements near obstructions. Therefore, a large scale pullout load test system with dimensions 12ft by 12ft (3.67m by 3.67m) in plan area and 4ft (1.22m) in depth was designed and built as a part of this research project. This large pullout load test system is shown in Figure 4. This MSE test box accommodated three layers of MSE reinforcement in a single filling of the text box. The reinforcement layers were placed at heights of 1.0ft (0.30m), 2.0ft (0.61m) and 3.0ft (0.92m) from the base. The reinforcements at all three levels can be pulled to the south through slots cut in the south wall (primary bulkhead) or to the east (secondary bulkhead), allowing for optimum test space utilization.

In this test system, the soil overburden pressures acting on the embedded earth reinforcement is simulated by using a reaction frame assembly which consists of nine 4ft by 4ft (1..22mx1.22m) pressure plates that are hydraulically jacked against three wide flange cross beams. The reaction frame assembly allows simulation of overburden pressures up to 40ft (12.2m) of fill. The vertical overburden pressure is monitored throughout the test using four earth pressure cells that are grouted into the bottom slab of the test box. An independent measurement of the applied overburden will be made by using a pressure transducer that monitors the fluid pressure in the hydraulic jacks.

Figure 5(a) shows skewed strip type reinforcements that have been placed in the test box for testing. Similarly, Figure 5(b) shows grid reinforcements that have been cut and splayed.

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FIGURE 4. Large Scale MSE Pullout Resistance Test System: Civil Engineering Department, Texas Tech University

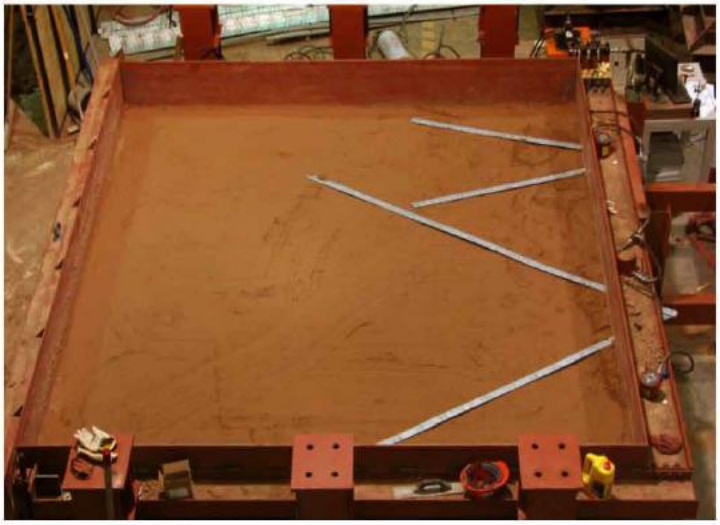
Pullout testing of each embedded earth reinforcement was accomplished, one at a time, by attaching a pullout load assembly to the reinforcement and then applying the pullout force using a 60-ton (534 kN) hollow core hydraulic jack. When testing strip type reinforcement, the pullout assembly used a simple pin joint mechanism to connect the reinforcement to the pull rod. When testing grid type reinforcement, the pullout assembly used a friction gripper that clamps on to each individual longitudinal bar of the steel grid (Figures 6b).

Two independent systems were used to record the pullout force. The first uses an annular load cell mounted on the pull rod (Figure 6a). The second was through a pressure transducer that measures hydraulic pressure in the hollow-core jack. Similarly, the displacement of the reinforcement was determined using two independent measurements. The first is a visual measurement of the displacement using a witness marker against a graduated scale. The second is a digital measurement of displacement with an electronic displacement gage (Figure 6b). To minimize potential errors due to *bias* in one measuring system with respect to the other, each pair of parallel measuring systems were calibrated against each other periodically throughout the test program.

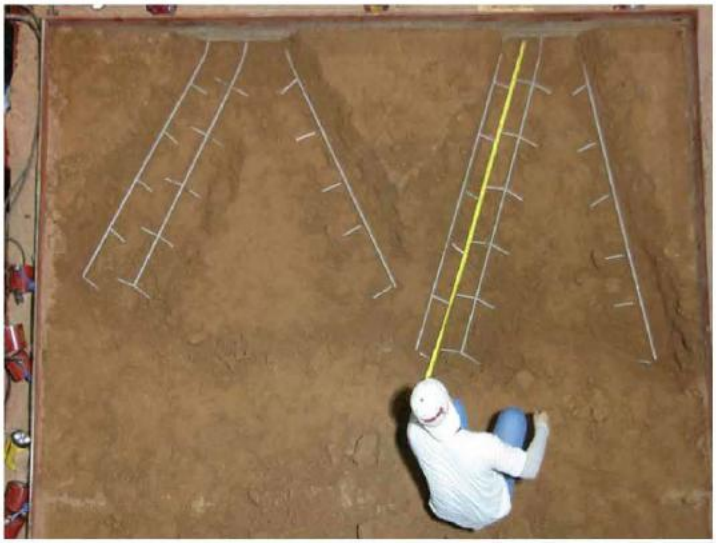
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(a)



(b)

FIGURE 5. (a) Skewed Reinforcing Strips in the Test Box, (b) Cut-and-Splayed Grid Reinforcement in the Test Box

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Backfill and Reinforcing Material

The backfill material used in the above test program was a fine to medium sand that marginally met the requirements listed for MSE select granular reinforced fill in FHWA Publication FHWA NHI-10-024 (*1*). The particle size distribution curve for the backfill material is shown in Figure 7. As seen in the figure, the particle size distribution analysis was repeated 3 times during the course of the test program to ensure that the material gradation did not undergo significant change. The material was determined to be a non-plastic material that classifies as SP-SM according to the Unified Soil Classification System (USCS). It has a maximum dry unit weight of 124.5pcf (19.6 kN/m^3) and an optimum water content of 7.8% as established through testing conducted in accordance with ASTM D-698. During testing, backfill material was placed in lifts of loose fill with approximate thickness of 5.0-in (125mm). This material was then compacted with 10 passes of a vibratory compactor to achieve a compacted thickness of about 4.0-in (100mm). The compacted density was measured using a nuclear density gage to make sure that a target relative compaction of 95% had been achieved during compaction. The average relative compaction calculated based on nuclear density gage readings obtained for all 29 test setups was 95.7% and the standard deviation was 6.8%. The average water content measured was 6.8% and the corresponding standard deviation was 4.4%.

The test program described in this paper included two different types of soil reinforcement: (a) 2-in wide (50mm), 160 mil (4mm) thick ribbed galvanized steel reinforcement of varying length, (b) welded galvanized steel grid reinforcement of varying width, length, transverse bar size and spacing, longitudinal bar size and spacing.

Scope of the Test Program

Table 1 summarizes the overall scope of the test program undertaken in this research for both types of MSE reinforcement. As shown in the table, the strip reinforcements were tested in lengths of 4ft (1.22m), 6ft (1.83m), 8ft (2.44m) and 12ft (3.66m). These reinforcements were subjected to pullout testing at zero skew angle (i.e. straight pulls) under a range of overburden pressures that correspond to 5ft (1.53m), 12ft (6.04m), 20ft (6.10m) and 40ft (12.20m) of soil fill. In addition, ribbed strip reinforcements were tested under skew angles of 15-deg and 30-deg under two different depths of overburden, 5ft (1.53m) and 12ft (6.04m).

The pullout test program on welded steel grids involved a much larger array of variables. In addition to reinforcement length and depth of overburden, the grid test program included four other variables: longitudinal bar spacing, transverse bar spacing, longitudinal bar size and transverse bar size. The complete array of variables used in grid testing is shown on the right hand side of Table 1. As seen in the table, the grid reinforcement tested under splay angles of 15 and 30-deg consisted of 9x12 grids with bar sizes W20xW11. Therefore, data collected on a subset of straight pull grids with matching parameters are presented in this paper and used for comparison.

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(a)



(b)

FIGURE 6. (a) Hollow-Core Jack and Annular Load Cell, (b) Friction Gripper used with Grid Type Reinforcement and Digital Displacement Gages

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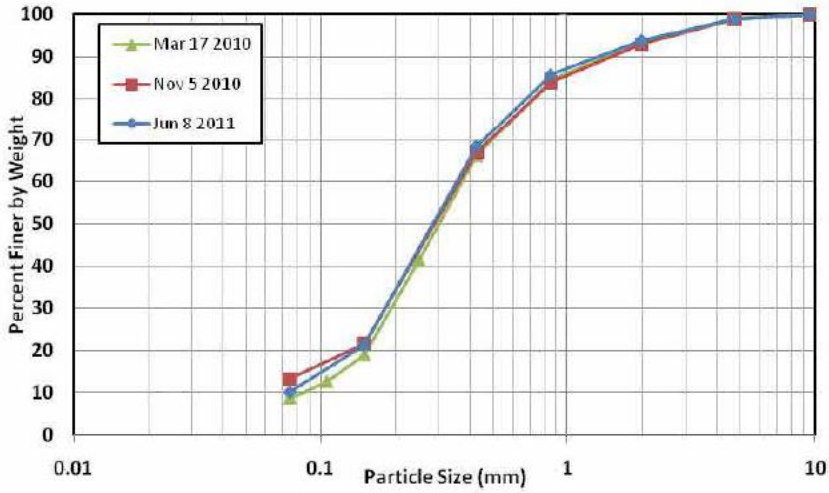


FIGURE 7. Particle Size Distribution Curve for MSE Wall Backfill Material Used in Testing Data Acquisition and Processing (1 inch = 25.4mm)

DATA ANALYSIS

The information collected during each pullout test consisted of data belonging to three separate categories: (a) Pullout force (measured using a load cell as well as a pressure transducer attached to the hydraulic line of the jack), (b) Pullout displacement (measured visually with the help of a witness marker against a graduated scale and using digital displacement gages), and (c) vertical overburden pressure (measured using earth pressure cells and through measurement of fluid pressure in the hydraulic jacks that were used to generate additional overburden. The independent measurements made of each variable of interest were then combined into one by using the following equation proposed by Lyons, 1991 (*12*).

$$\alpha = \frac{\sum \frac{\alpha_i}{\sigma_i^2}}{\sum \frac{1}{\sigma_i^2}} \qquad \text{(4a)}$$

where: α – Combined measurement, α_i – Measurement made with a specific measuring system, σ_i – Standard deviation associated with each measurement

Equation 4(a) allows multiple measurements of the same variable to be combined in such a manner that the measuring system providing more consistent data receives higher weight and the measuring system with less consistent data receives lower weight. The overall standard deviation corresponding to the combined measurement is given by Equation 4(b).

$$\frac{1}{\sigma^2} = \sum \frac{1}{\sigma_i^2} \qquad \text{(4b)}$$

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TABLE 1. Pullout Test Program

Ribbed Strip Reinforcement			Welded Grid Reinforcement					
Alignment with respect to Pullout Load Direction	**Overburden Depth (feet)[1]**	**Length of Reinforcement (feet)[a]**	**Overburden Depth (feet)[a]**	**Length (feet)[a]**	**Transverse Bar Spacing (in)[b]**	**Transverse Bar Size**	**Long. Bar Spacing (in)[b]**	**Long. Bar Size**
Parallel/Straight, β=0°	5, 12, 20, 40	4, 6, 8, 12	5, 12, 20, 40	3, 4.5, 6, 9, 12	6, 12, 18, 24	W7.5, W11, W15	2, 6, 9, 12	W9.5 W20
Skewed, β=15°	5, 12	4.5	5, 12	6	12	W11	9	W20
Skewed, β=30°	5, 12	8	5, 12	5	12	W11	9	W20

[a] 1 ft = 0.305m
[b] 1 in = 25.4 mm

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Figure 8 shows a typical load-displacement diagram obtained from a pullout test. This load displacement curve represents a pullout load conducted on a strip type reinforcement. The figure shows two separate plots of load versus displacement; one based on the pull force calculated based on hydraulic jack pressure transducer measurement and the other based on pull force calculated from annular load cell readings. Solid black circles represent the combined pull forces calculated using Eq. (4a) at displacements of 0.5-in (12.5mm) and 0.75-in (20mm).

The pullout loads at 0.5 and 0.75-in (12.5 and 20mm) are then used to determine the pullout resistance factor, F* by using the following equation:

$$F^* = \frac{P_r}{C\,\alpha\,L_e\,b\,\sigma_v} \qquad \text{.....} \qquad (5)$$

where: P_r – Pullout load corresponding to a specific displacement, L_e – Length of embedment of the reinforcement, b – Width of the reinforcement, σ_v – Vertical overburden pressure, C=2 and α = 1.0

When calculating F* corresponding to skewed and splayed reinforcement, the length of embedment, L_e used was the actual length of the reinforcement and not the projected length in the direction of the pull calculated by multiplying with cosine of skew or splay angle.

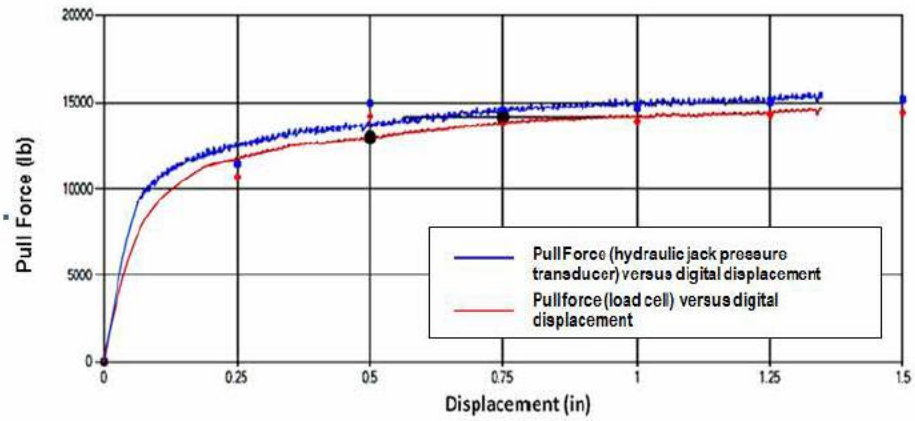


FIGURE 8. Typical Load-Displacement Curve

The database of pullout resistance factors (F*-values) developed as described above disregards potential interaction effects that may exist between reinforcements embedded in the same test setup as well as possible boundary effects. To investigate the influence of such extraneous variables, an Analysis of Covariance (ANCOVA) was performed on the dataset. ANCOVA analysis identified the vertical position of the reinforcement within the box (top, middle or bottom layer) as a significant variable affecting measured F*. The ratio between the

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maximum prior overburden stress and the test overburden stress was found to be a second important variable. Therefore, as the next step, the data set was *normalized* to represent standard test conditions by correcting the F* for the effect of these two variables. The standard test conditions used were reinforcement placed in mid layer (least interference and therefore minimum potential for boundary effects) and maximum prior overburden stress = test overburden stress.

RESULTS

Ribbed Steel Strip Reinforcement

In Figure 9, results obtained from a total of 72 pullout tests conducted on strip reinforcement are shown in the form of an F* versus Depth of Fill (DOF) plot. These F* values were calculated from pullout loads that correspond to a displacement of 0.75-in (20mm). The F* versus DOF plot for 0.5-in (12.5mm) displacement was almost identical to the one shown here. In Figure 9, F* values corresponding to zero skew strips are shown in open circles and F* values for skew angles of 15 and 30-deg are shown in solid symbols. The shaded zone represents the envelop for F* values obtained for strips with zero skew. The AASHTO reference line for default F* (Eq. 2a and 2b) corresponding to a soil friction angle, $\phi = 34°$ and coeff. of uniformity, C_u=4.0 is also shown on this plot.

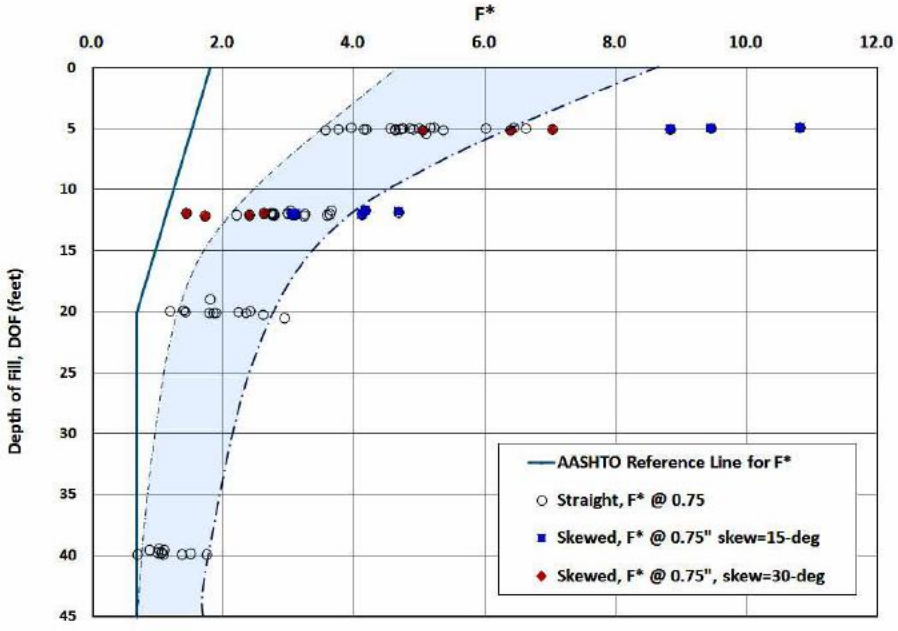


FIGURE 9. F* versus Depth of Fill: Ribbed Steel Strip Reinforcement

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Table 2 provides the summary statistics corresponding to the above data set. It shows the number of tests completed, the mean F* and the standard deviation for each depth of fill and skew angle.

Review of data provided in Table 2 and Figure 9 reveals following trends. The average F* obtained for the 15° skew angle is higher than the average F* for zero skew. This observation is true for both 5ft (1.52m) and 12ft (3.66m) depths. Accordingly, the data suggests that there is no reduction in pullout resistance factor for strip reinforcements placed at a skew angle of 15°. An Analysis of Variance (ANOVA) was conducted to determine whether the difference observed between F* values calculated for zero and 15-deg skew is statistically significant. ANOVA lead to the conclusion that this difference is significant at a p-value of 0.05. The difference between F* at a skew angle of 30° and zero was not as prominent, but nevertheless statistically significant at p=0.05. However, in this case, the trends observed at 5ft (1.52m) and 12ft (3.66m) depths of fill were found to be contradictory and therefore, no definite conclusion can be reached whether skew will have positive or negative influence on pullout resistance.

TABLE 2. Summary of Test Results

	Pullout Resistance Factor, F*; Ribbed Strip Reinforcement								
	Skew = 0°			Skew = 15°			Skew = 30°		
DOF(ft)[a]	No	Mean	S.D.	No	Mean	S.D.	No	Mean	S.D.
5	21	4.87	0.80	3	9.70	1.01	3	6.16	1.02
12	14	3.02	0.41	5	3.82	0.72	4	2.04	0.56
20	12	1.98	0.53	--	--	--	--	--	--
40	10	1.14	0.31	--	--	--	--	--	--
	Pullout Resistance Factor, F*(S_t/t); Welded Grid Reinforcement								
	Skew = 0°			Skew = 15°			Skew = 30°		
DOF(ft)[a]	No	Mean	S.D.	No	Mean	S.D.	No	Mean	S.D.
5	9	40.97	3.46	2	57.60	0.42	3	65.97	2.71
12	13	30.93	7.40	3	37.51	0.19	3	35.38	1.45
20	10	20.50	5.15	--	--	--	--	--	--
40	3	15.62	0.91	--	--	--	--	--	--

[a] 1 ft = 0.305m

Welded Steel Grid Reinforcement

The data obtained from 48 pullout tests conducted on welded steel grids are plotted in the form of a normalized F*, i.e. F*(S_t/t) versus Depth of Fill (Figure 10). According to AASHTO reference line for default F* (Equations 3a and 3b) to be used for inextensible grids is primarily a function of transverse bar spacing/transverse bar diameter ratio (i.e. t/S_t). Normalization of F*

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factor by dividing it with t/S_t allows direct comparison of F* values for grids with various bar sizes and spacings. In Figure 10, the normalized F* for cut-and-splayed grids are shown in bold symbols while normalized F* for straight grids are shown in open circles. The shaded area provides the envelop for normalized F* for grids with zero splay angle.

The summary statistics for the same data set is shown in Table 2. In the case of grid reinforcement, the average normalized F* values corresponding to skew angles of 15 and 30-deg are both higher than the average normalized F* for grids with zero skew. This trend is seen at both fill depths, 5ft (1.52m) as well as 12ft (3.66m). As in the case of strip reinforcement, ANOVA analysis performed on this data confirmed that influence of splaying on pullout resistance of grids is statistically significant at a p-value of 0.05.

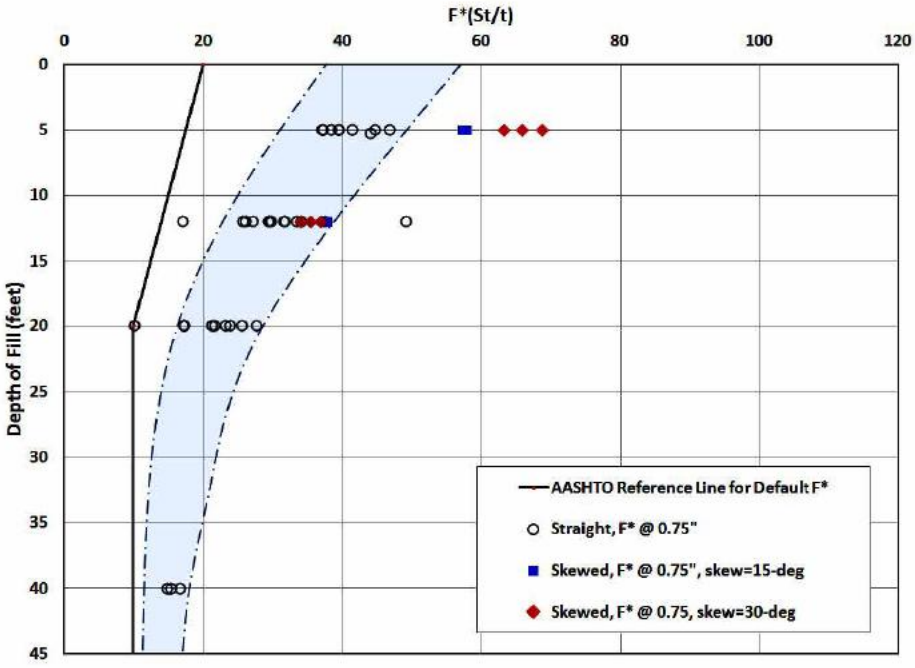


FIGURE 10. Normalized F* versus Depth of Fill: Welded Steel Grid Reinforcement

CONCLUSIONS

The primary objective of the research described above was to determine potential impact that skewing of strip type MSE reinforcement and cutting and splaying of grid reinforcement may have on the pullout resistance capacities of these reinforcements. The data collected and the analyses performed lead to the following useful conclusions.

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Data obtained for reinforcements laid with 15 and 30-deg skew or splay angles showed a noticeable shift in comparison to data obtained for reinforcement subjected to straight pulls. This trend was seen in both types of reinforcement tested (strips and grids), and for both 5ft and 12ft (1.52m and 3.66m) depths. ANOVA analysis conducted on each data set confirmed that the influence of skew and splay angle on the pullout resistance factors was significant at a p-value of 0.05.

Although, it is generally believed that skewing and splaying will affect the wall performance negatively, 15-deg skew/splay yielded higher average F*-values than the average F* obtained for strips and grids with no skew/splay. This finding, together with observations made during testing, suggest that passive resistance that develops on the outer edges of the reinforcement as the reinforcement tries to slice through the backfill likely offsets any loss in pullout resistance due to skewing or splaying.

In 3 out of 4 cases considered, 30-deg skew/splay also provided higher average F* in comparison to reinforcements with no skew/splay. However, the opposite trend was observed in strip reinforcement at 12ft (3.66m) depth. Therefore, a definite conclusion cannot be reached with respect to the influence of 30-deg skew on pullout resistance.

The data collected in this research support the current practice of limiting skew and splay angles to 15-deg. The data clearly demonstrate that the pullout resistance is not adversely affected by skew or splay at angles up to 15-deg. The data further suggest that reduction of pullout capacity by multiplying with cosine of the skew or splay angle may be redundant.

It is important to note that this research did not investigate potential impact that skewing and splaying may have on the distribution of lateral earth pressure on wall panels. This design consideration must be addressed through independent research. Also, it is worth noting that testing described in this research project were conducted using granular fill that has been compacted to achieve 95% relative compaction with respect to standard Proctor test. Compaction conditions in close vicinity of obstructions may not satisfy this requirement. Furthermore, this research did not investigate potential adverse impact skewing and splaying may have on other aspects of reinforcement performance. For example, large angles of splay in grid type reinforcement will cause significant bending stresses in the longitudinal bars of the grid as well as permanent (plastic) deformation. Although the deformed grid may provide adequate pullout resistance as evidenced in the data collected in this research, the permanent deformations caused by splaying may have adverse impact on its longevity. Such factors should be considered when determining acceptable levels of skew and splay in MSE reinforcements in the field.

ACKNOWLEDGEMENT

This research project was conducted in cooperation with the Texas Department of Transportation (TxDOT) and the Federal Highway Administration (FHWA). The authors wish to express their gratitude to the research sponsors. The authors also wish to thank the members of the project advisory panel for their guidance and assistance throughout the project.

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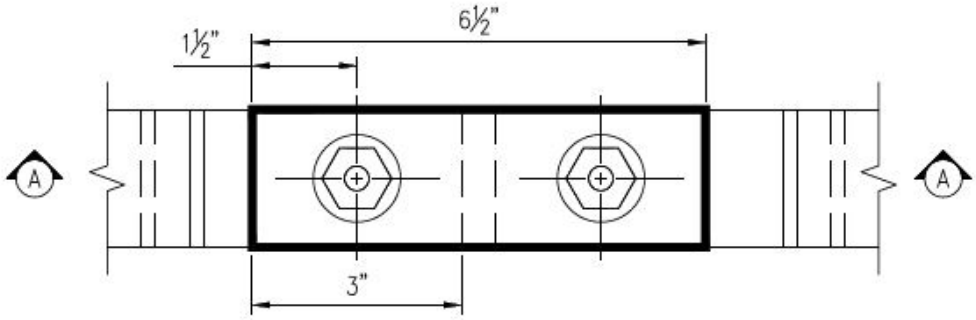
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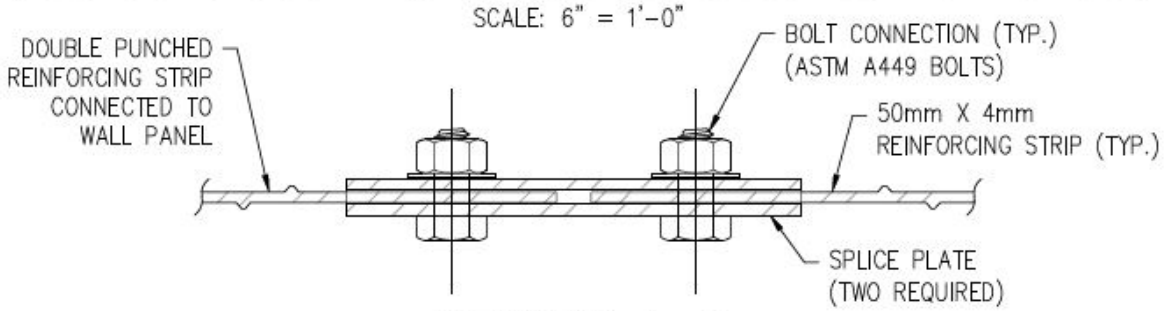
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PARTIAL PLAN – REINFORCING STRIP SPLICE DETAIL

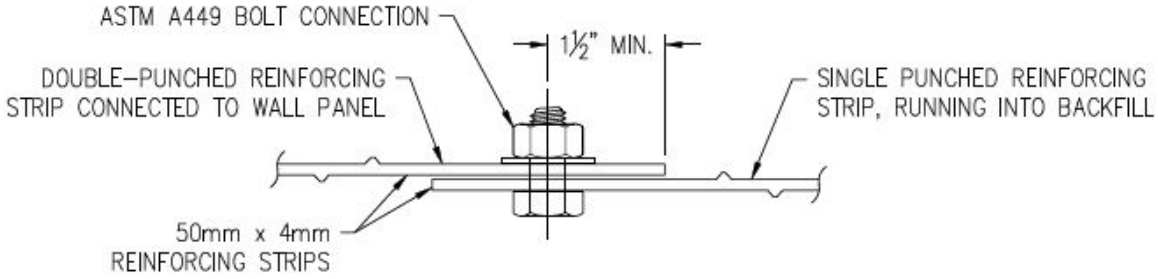
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SECTION A–A

SCALE: 6" = 1'–0"

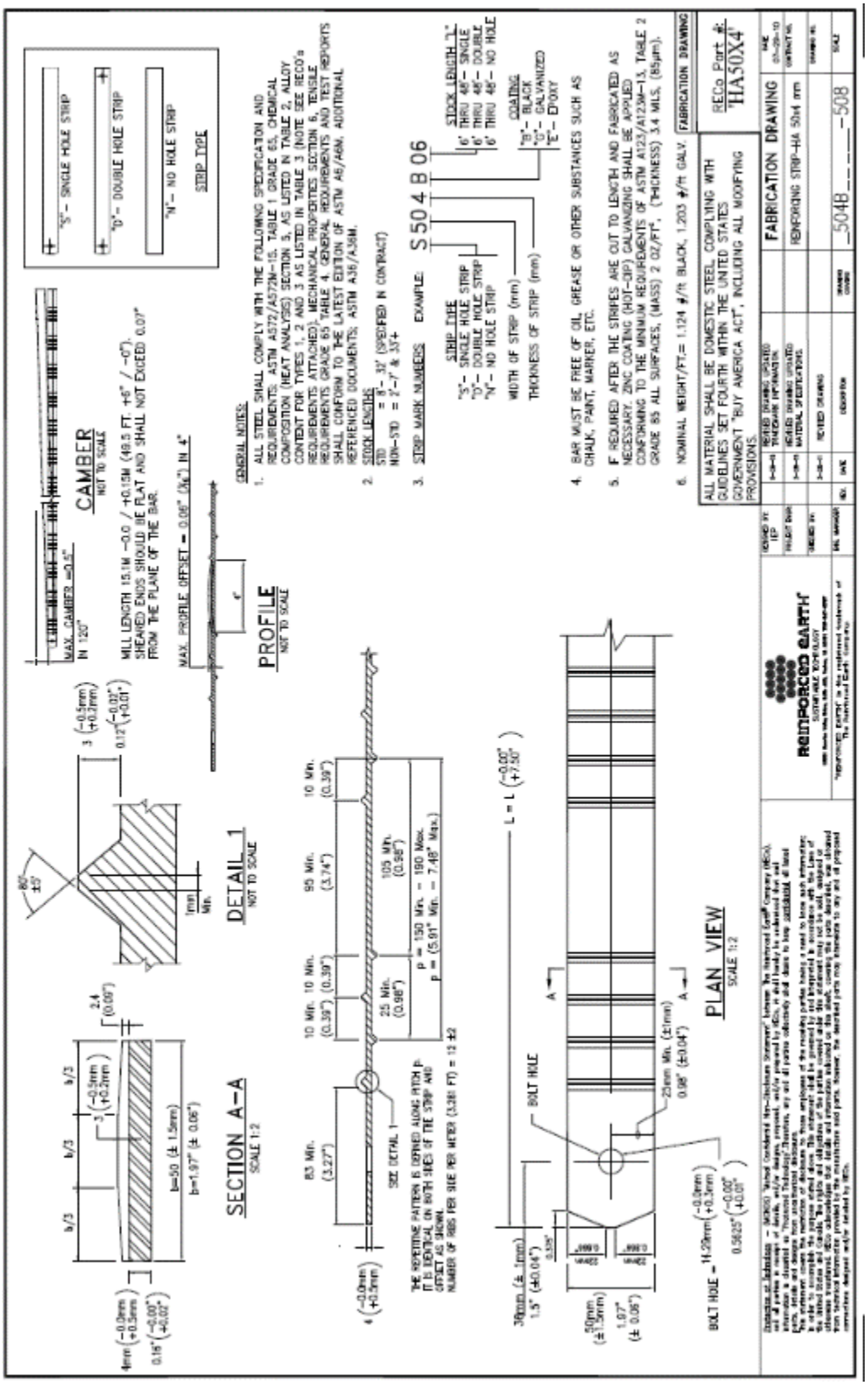
NOTE:
USE SPLICE CONNECTION DETAIL (DOUBLE SHEAR) WHEN TOTAL STRIP LENGTH IS GREATER THAN 40FT, OR WHEN OTHERWISE REQUIRED BY PLANS.



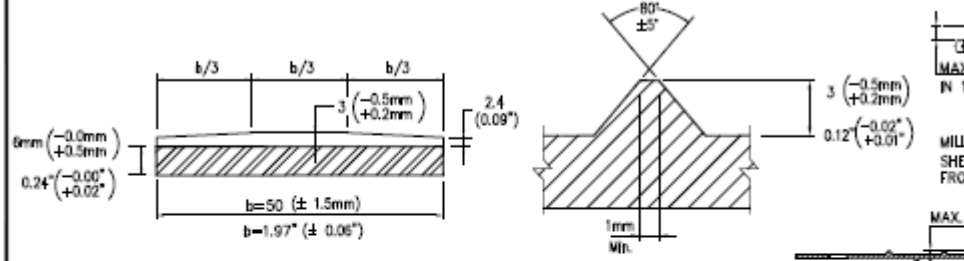
SPLICE CONNECTION DETAIL

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A1.2.2 Types of Soil Reinforcements

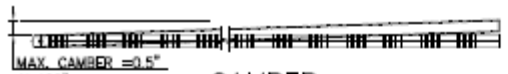


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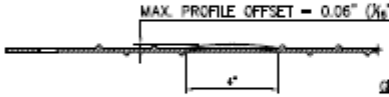
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SCALE 1:2

DETAIL 1
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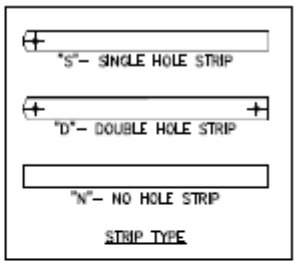


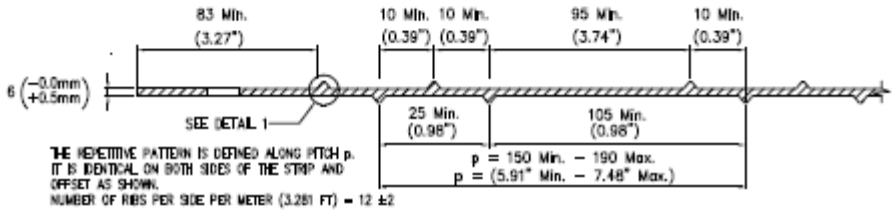
CAMBER
NOT TO SCALE

MILL LENGTH 15.1M −0.0 / +0.15M (49.5 FT. +6" / −0").
SHEARED ENDS SHOULD BE FLAT AND SHALL NOT EXCEED 0.07" FROM THE PLANE OF THE BAR.

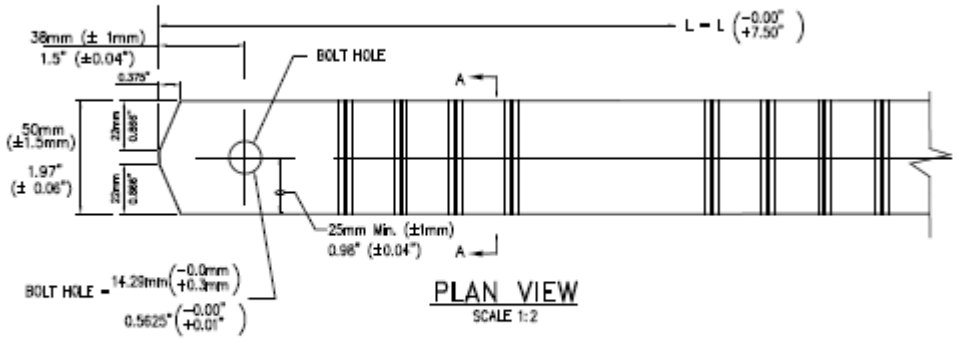


PROFILE
NOT TO SCALE





THE REPETITIVE PATTERN IS DEFINED ALONG PITCH p. IT IS IDENTICAL ON BOTH SIDES OF THE STRIP AND OFFSET AS SHOWN.
NUMBER OF RIBS PER SIDE PER METER (3.281 FT) = 12 ±2



PLAN VIEW
SCALE 1:2

GENERAL NOTES:

1. ALL STEEL SHALL COMPLY WITH THE FOLLOWING SPECIFICATION AND REQUIREMENTS: ASTM A572/A572M–15, TABLE 1 GRADE 65, CHEMICAL COMPOSITION (HEAT ANALYSIS) SECTION 5, AS LISTED IN TABLE 2, ALLOY CONTENT FOR TYPES 1, 2 AND 3 AS LISTED IN TABLE 3 (NOTE SEE RECO's REQUIREMENTS ATTACHED). MECHANICAL PROPERTIES SECTION 6, TENSILE REQUIREMENTS GRADE 65 TABLE 4. GENERAL REQUIREMENTS AND TEST REPORTS SHALL CONFORM TO THE LATEST EDITION OF ASTM A6/A6M. ADDITIONAL REFERENCED DOCUMENTS; ASTM A36/A36M.
2. STOCK LENGTHS
   STD = 8'– 32' (SPECIFIED IN CONTRACT)
   NON–STD = 2'–7' & 33'+
3. STRIP MARK NUMBERS EXAMPLE: S 50 6 B 06

   STRIP TYPE
   "S"– SINGLE HOLE STRIP
   "D"– DOUBLE HOLE STRIP
   "N"– NO HOLE STRIP
   WIDTH OF STRIP (mm)
   THICKNESS OF STRIP (mm)
   STOCK LENGTH "L"
   6' THRU 48'– SINGLE
   6' THRU 48'– DOUBLE
   6' THRU 48'– NO HOLE
   COATING
   "B"– BLACK
   "G"– GALVANIZED
   "E"– EPOXY
4. BAR MUST BE FREE OF OIL, GREASE OR OTHER SUBSTANCES SUCH AS CHALK, PAINT, MARKER, ETC.
5. IF REQUIRED AFTER THE STRIPES ARE CUT TO LENGTH AND FABRICATED AS NECESSARY. ZINC COATING (HOT–DIP) GALVANIZING SHALL BE APPLIED CONFORMING TO THE MINIMUM REQUIREMENTS OF ASTM A123/A123M–13, TABLE 2 GRADE 85 ALL SURFACES, (MASS) 2 OZ/FT². (THICKNESS) 3.4 MILS, (85µm).
6. NOMINAL WEIGHT/FT.= 1.685#/ft BLACK, 1.805#/ft GALV.

FABRICATION DRAWING

ALL MATERIAL SHALL BE DOMESTIC STEEL, COMPLYING WITH GUIDELINES SET FOURTH WITHIN THE UNITED STATES GOVERNMENT "BUY AMERICA ACT", INCLUDING ALL MODIFYING PROVISIONS.

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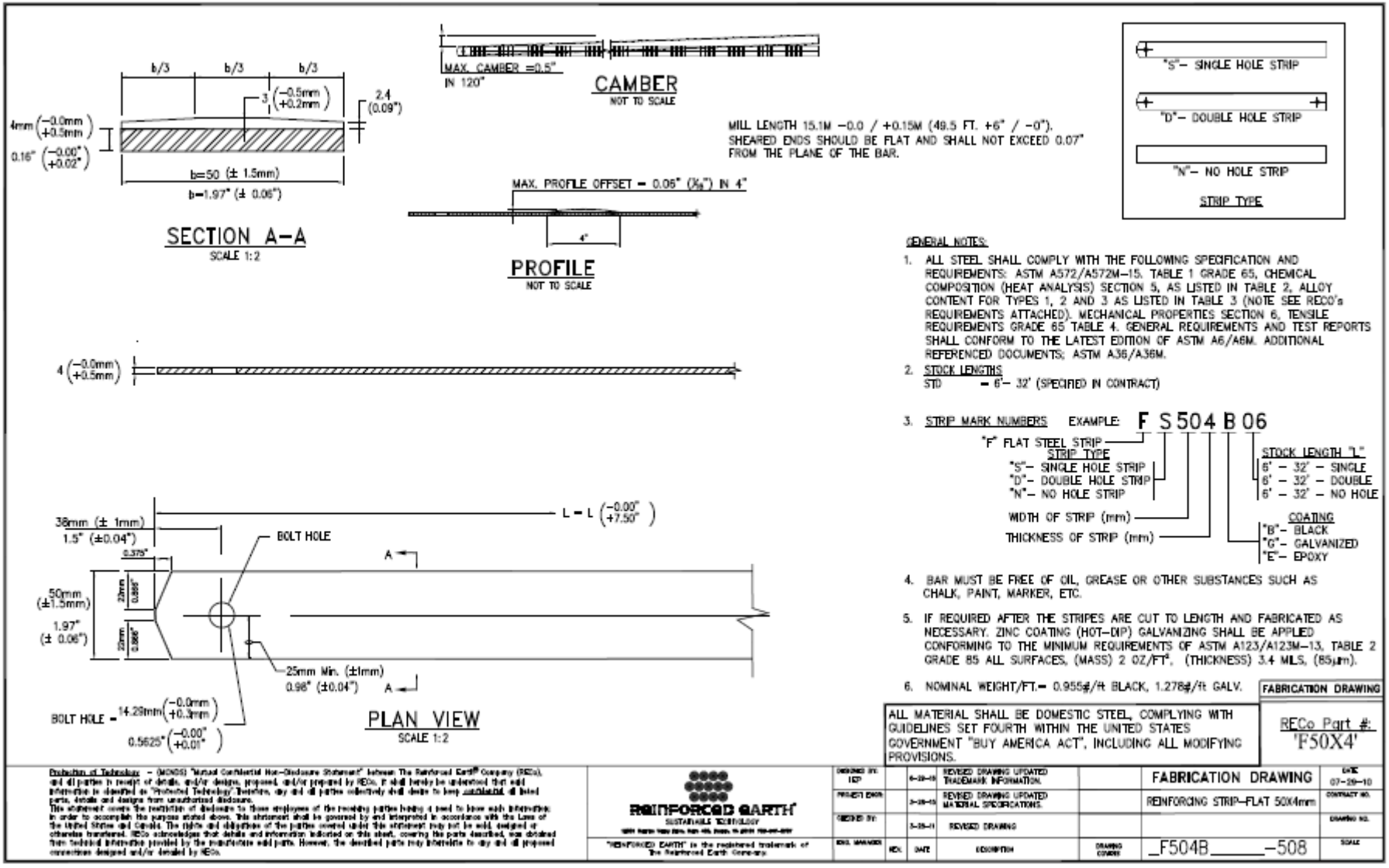
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PROJECT ENGR.		3–25–10	REVISED DRAWING UPDATED MATERIAL SPECIFICATIONS.		REINFORCING STRIP–HA 50x6mm	CONTRACT NO.
CHECKED BY:		3–25–10	REVISED DRAWING			DRAWING NO.
ENG. MANAGER	REV.	DATE	DESCRIPTION	DRAWING CHANGE	_506B_____–508	SCALE

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2. STOCK LENGTHS
   STD = 6'– 32' (SPECIFIED IN CONTRACT)
3. STRIP MARK NUMBERS EXAMPLE: F S 504 B 06
   - "F" FLAT STEEL STRIP
   - STRIP TYPE: "S"– SINGLE HOLE STRIP; "D"– DOUBLE HOLE STRIP; "N"– NO HOLE STRIP
   - WIDTH OF STRIP (mm)
   - THICKNESS OF STRIP (mm)
   - STOCK LENGTH "L": 6' – 32' – SINGLE; 6' – 32' – DOUBLE; 6' – 32' – NO HOLE
   - COATING: "B"– BLACK; "G"– GALVANIZED; "E"– EPOXY
4. BAR MUST BE FREE OF OIL, GREASE OR OTHER SUBSTANCES SUCH AS CHALK, PAINT, MARKER, ETC.
5. IF REQUIRED AFTER THE STRIPES ARE CUT TO LENGTH AND FABRICATED AS NECESSARY, ZINC COATING (HOT–DIP) GALVANIZING SHALL BE APPLIED CONFORMING TO THE MINIMUM REQUIREMENTS OF ASTM A123/A123M–13, TABLE 2 GRADE 85 ALL SURFACES, (MASS) 2 OZ/FT², (THICKNESS) 3.4 MILS, (85µm).
6. NOMINAL WEIGHT/FT.= 0.955#/ft BLACK, 1.278#/ft GALV.

FABRICATION DRAWING

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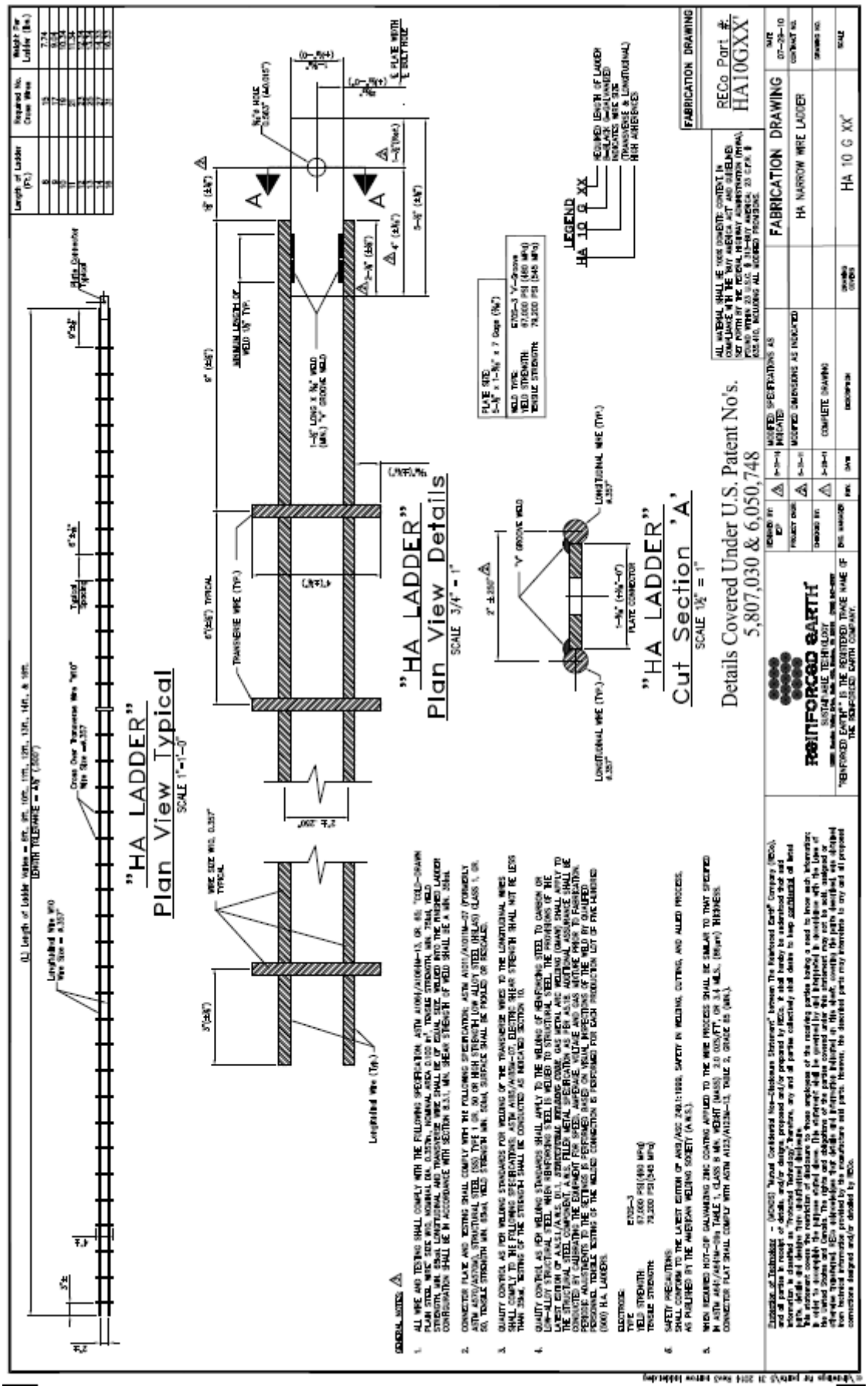
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This statement covers the restriction of disclosure to those employees of the receiving parties having a need to know such information in order to accomplish the purpose stated above. This statement shall be governed by and interpreted in accordance with the Laws of the United States and Canada. The rights and obligations of the parties covered under this statement may not be sold, assigned or otherwise transferred. RECo acknowledges that details and information indicated on this sheet, covering the parts described, was obtained from technical information provided by the manufacturer said parts. However, the described parts may interrelate to any and all proposed connections designed and/or detailed by RECo.

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PROJECT ENGR.		3–28–10	REVISED DRAWING UPDATED MATERIAL SPECIFICATIONS.		REINFORCING STRIP–FLAT 50X4mm	CONTRACT NO.
CHECKED BY:		3–25–11	REVISED DRAWING			DRAWING NO.
ENG. MANAGER	NO.	DATE	DESCRIPTION	DRAWING CHECKED	_F504B_____–508	SCALE

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A1.2.3 Physical Property Specifications

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A1.2.4 Corrosion Protection

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A1.2.5 Sacrificial Steel Thickness

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A1.2.6 Corrosion Test Results

The following documentation is included in this section:

- McMahon & Mann Consulting Engineering and Geology, PC, “Condition and Durability Study for RECo Reinforcements and Tie-Strip Connections Samples from Lexington, Kentucky” Parts 1-5 (2011-2021). (13 pages)

McMahon & Mann Consulting Engineering and Geology, P.C.

Donald R. McMahon, P.E., F.ASCE
Michael J. Mann, P.E.
Kenneth L. Fishman, Ph.D., P.E.
Shawn W. Logan, P.E.
Andrew J. Nichols, P.E.
Todd Swackhamer, P.E.
James J. Janora, P.G.
Susanne J.M. George, P.E.
Andrew J. Klettke, P.E.
Kaitlyn M. Murray, P.E.

February 9, 2021
File: 20-003
Sent via Email

Mr. Keith Brabant, P.E.
Chief Operating Officer
The Reinforced Earth Company
45610 Woodland Rd.
Suite 200
Sterling, VA 20166

RE: Condition and Durability Study for RECO Reinforcements and Tie-Strip Connections Samples from Lexington, KY

Dear Keith:

In accordance with our agreement executed February 10, 2020, the attached report presents results from our analysis of the reinforcements, tie strips, and fill samples retrieved from Lexington, Kentucky (RE:0475).

These observations support the conclusion that the life of the protective zinc coating along the surfaces of hot-dipped galvanized steel reinforcements is greater than that anticipated based on the AASHTO metal loss model used for design of MSE walls. This is true for the portions of the reinforcements exposed to fill as well as the portion included with the tie-strip connection. The results support the conclusion that the portion of the strip reinforcement that is sandwiched within the connection is protected by the tie-strip.

The KY Structure was 38 years old when the samples were exhumed and was constructed in 1980. At the time the wall was designed and constructed, RECo's reinforcements were manufactured with ASTM A 36 rather than Grade 50 steel, and ASTM A-570 Grade C (Fy = 33 ksi) was used for the tie strips and this is reflected in the results from tension tests performed on the samples.

2495 Main Street, Suite 432, Buffalo, NY 14214
Telephone: 716-834-8932 □ www.mmce.net

We appreciate the opportunity to perform this study and look forward to working with The Reinforced Earth Company in the future.

Sincerely yours,

McMAHON & MANN CONSULTING ENGINEERING AND GEOLOGY, P.C.

Kenneth L. Fishman

Kenneth L. Fishman, Ph.D., P.E.
Principal

Attachment

RE:0475, S. JEFFERSON ST. OVERPASS (KY)

This structure is located in Lexington, KY, and was designed in July 1979 and constructed during Fall/Winter 1979 - 1980. KYTRANS sampled this structure in 2018 (38-year-old samples). The samples include eight (8) hot-dip galvanized carbon steel ribbed soil reinforcements labeled as Sample Extra 1, Sample 2, Sample Extra 2, Sample 4, Sample 5, Sample 6, Sample 12 and Sample 15. Two (2) of the samples (Sample 4 and Sample 6) incorporated the tie strip connections from the bolted ends.

KYTRANS also collected two (2) buckets of soil samples and RECo shipped these to McMahon & Mann Consulting Engineering and Geology, P.C. (McMahon & Mann). One of the samples was collected from the rear of the structure, the other is a clean sample representative of the original source.

SOIL/AGGREGATE TESTING

Laboratory analyses of the fill samples include grain size analyses and measurements of resistivity, pH, chloride ion content, and sulfate ion content. Results from these tests were used to estimate the aggressiveness of the fill material relative to the corrosion of the metallic reinforcements. We engaged Geotechnics Geotechnical Engineering Laboratory (Geotechnics) for the soil testing.

We sent four samples to Geotechnics for testing that included:

1. Sample B. One bucket (24.4) collected from the rear of the structure from RE:0475, S. Jefferson St. Overpass, Lexington, KY. This sample included lightly cemented conglomerations of aggregates that could be broken up with a mortar and pestle.

2. Sample C. One bucket (16.4 lbs.) from RE:0475, S. Jefferson St. Overpass, Lexington, KY, including a clean sample representative of the original source. This sample appeared to be No. 57 stone with very little, if any, material passing a #10 sieve.

Sample B from the Jefferson Street site in Kentucky appeared to have a significant portion of particles passing the #10 sieve. Sample B included lightly cemented conglomerations of aggregates that could be broken up with a mortar and pestle to release finer particles that are less than 2 mm and pass the #10 sieve.

Sample C from the source used at Jefferson Street in KY is a coarse-grained sample that appears to be No. 57 stone. The sample has little material passing a #10 sieve. AASHTO tests cannot be applied to materials with little or no material passing a #10 sieve and an alternative test procedure (Tex-620-M) was performed that included the larger particle sizes within the test specimen. There was not enough sample for resistivity testing so testing only included measurements of sulfate ion content, chloride ion content, and pH.

Table 1 summarizes the results of the sieve analyses and electrochemical measurements performed on the soil samples. Samples A and B from the Illinois Tri-State Tollway and the

Jefferson Road site in Kentucky have high sulfate content (> 200 ppm) and low resistivity (< 3000 Ω-cm).

Table 1
Results from testing samples of fill and infill

Sample ID	% Passing			PI	Organics	ρ	pH	SO_4	Cl^-
	#4	#10	#200		%	Ω-cm		ppm	ppm
A	67	43.6	18.6	7	0.8	1450	8.4	492	33.4
B	42	27	6	NP	-	1500	9.65	470	155
C	3	2	1	NP	-	-	9.91	228	20
TX S1-N1[2]	61.5	52.6	27.4	NP	0.1	6200	9.17	<100	<10
TX N3[2]	41.4	30.4	7.8	NP	0.3	11,000	9.62	<100	<10

REINFORCEMENT SAMPLES

The loose scale and other contaminants from the surfaces of the samples were removed prior to making measurements and observations. Cleaning of the samples was accomplished using brass and steel wire brushes (depending on the ease of scale, zinc oxide, or rust removal). Special care was taken to avoid scratching or removing any remaining zinc on the surfaces of the samples.

The thicknesses of the remaining zinc coating, the base steel, and depths of the pits along each sample were measured using a zinc thickness gauge, a digital micrometer with an accuracy of ± 0.0001 inches, and a pit depth gage with an accuracy of ±0.001 in.

The following subsections discuss the results from thickness measurements obtained with an electromagnetic thickness gage (Elcometer), strip and weigh measurements, scanning electron microscopy with energy dispersive spectrometry (SEM/EDS) analyses, and tension tests performed on the reinforcement samples.

Thickness measurements from Elcometer

The remaining zinc thicknesses (z_r) were measured using an electromagnetic zinc thickness gauge (Elcometer) with a precision of ± 10 µm. Measurements were taken from both sides of the samples. The zinc thickness gauge determines the thickness of nonferrous (nonmagnetic) materials covering a ferrous (magnetic) substrate. Measurements of remaining zinc thickness included at least 5 sample points spaced along both the top and the bottom surfaces of the samples. A cluster of five measurements at each sample point was obtained, and the average of these measurements represents the remaining zinc thickness for the sample point. The thickness of remaining zinc from the portion of the reinforcement included within the tie strip was compared to the areas that have been exposed to the fill. These measurements describe the distribution of remaining zinc thickness.

Remaining zinc thickness for the portion of the reinforcements exposed to the fill.

Figure 1 shows the results of the zinc thickness measurements from the Elcometer for the portion of the galvanized steel strip samples that were exposed to the fill. These samples were retrieved

from three sites including Texas (29 years old), Illinois (27 to 28 years old), and Kentucky (38 years old). The bar graphs in this figure represent the average values from each side of a sample (side 1 and side 2). The error bars on the graph show the range for the minimum and the maximum measurements.

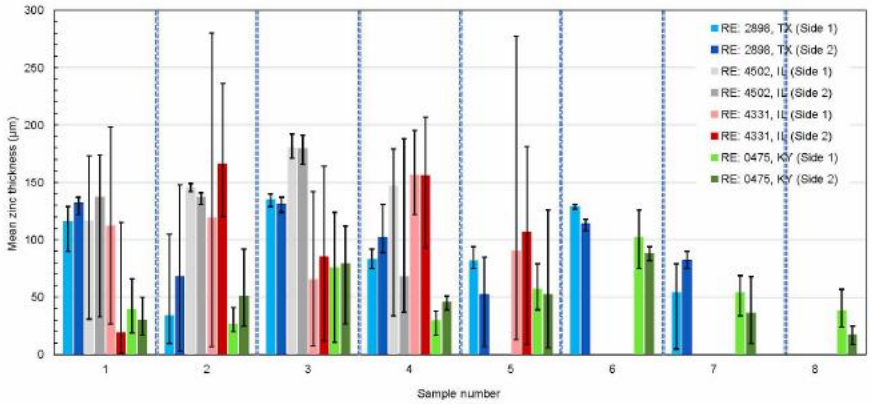


Figure 1 – Zinc thickness measurements from the portion of the reinforcements that were exposed to the fill. Note: KY Samples 1 -8 correspond to Samples Extra 1, 2, Extra 2, 4, 5, 6, 12 and 15, respectively.

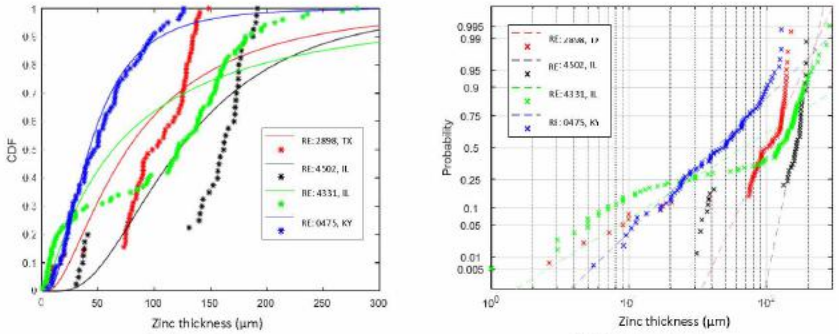
The following observations can be made from Figure 1:

- The least remaining zinc corresponds to samples from the site in KY. This is consistent with the low resistivity and high salt contents measured from the fill samples retrieved from the KY site. Samples from KY were retrieved after 38-years of exposure, so these are the oldest samples compared to the other sites. The lowest observation of remaining zinc from the KY samples was 17 µm with only 4 out of 16 measurements greater than 60 µm.

- Some samples from Texas and KY had spot measurements of remaining zinc thickness that were close to zero (i.e. zinc coating consumed). Based on the AASHTO metal loss model, zinc coating is expected to be consumed after 16 years and samples from Texas and KY had been in service for 29 and 38 years, respectively. The lowest spot thickness measurement from the Illinois samples was 33 µm.

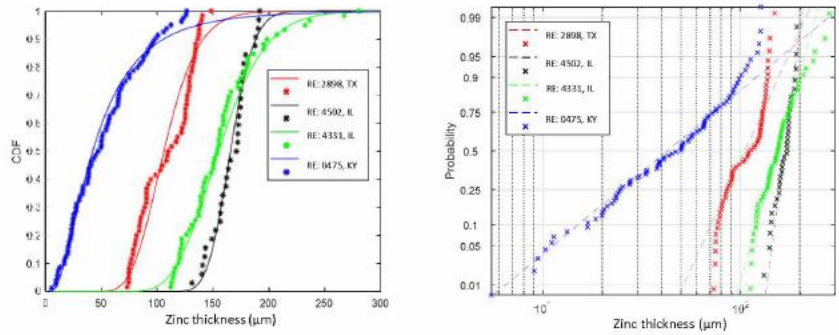
A log-normal distribution appeared to fit the data from each site. The cumulative distribution functions (CDF) for normal and log-normal distributions fitted to the zinc thickness measurements are shown in Figures 2 (a) and (b), respectively. The data from Kentucky was the most consistent and the entire data set could be modeled as a group. Data from Texas exhibited a gap between measurements ranging from 40 µm to 70 µm. Data from Illinois exhibited a gap between measurements ranging from 40 µm to 130 µm. To obtain a better distribution, the zinc thickness measurements smaller than 40 µm in Texas and Illinois sites were excluded from the data set shown in Figure 2, and the CDF for the remainder of the data are shown in Figure 3 (i.e., the data

sets were parsed into two groups shown in Figures 2 and 3). A good fit between log-normal distributions and the data sets can be observed from Figure 3. These data show the differences in remaining zinc observed between samples from these three sites.



(a) normal (b) lognormal

Figure 2 – CDF for all zinc thickness measurements



(a) normal (b) lognormal

Figure 3 – CDF after culling zinc thickness measurements

The following observations can be made from Figures 2 and 3:

- The mean of the remaining zinc from the samples retrieved from Kentucky is approximately 50 µm with a standard deviation of 32 µm.
- Compared to the other two sites, there is more variability in the observations of remaining zinc from the site in Kentucky. This is likely due to the variability in the character of the fill observed at this site. Fill samples taken from one area appeared to include more fines.

Remaining Zinc at Connections

Figure 4 shows the results of the zinc thickness measurements for the portion of the reinforcements that were within the connection (not directly exposed to the fill). These samples were retrieved from three sites including Texas, Illinois, and Kentucky. Similar to Figure 1, the bar graphs in this figure represent the average values, where each value is the average of 5 different measurements at a particular point on the strip sample. The error bars on the graph show the range for the minimum and the maximum of all average values.

Most connection locations have more than 80 µm of zinc remaining except for the samples from Kentucky, which have less and approximately 60 µm of zinc remaining. Only 3 measurements are less than 86 µm and 2 are from KY.

The remaining zinc is higher in the connection areas compared to portions of the samples that were exposed to fill during service. Compared to portions of the reinforcements that were exposed to fill, the connection areas have approximately 8 µm to 40 µm more remaining zinc. The variability of the measurements is described with the error bars on the bar chart in Figure 4. Compared to the measurements on the portions of the reinforcements exposed to fill (error bars in Figure 1), there appears to be less variability in the data describing remaining zinc on the connection ends of the reinforcements that were included within the tie strips.

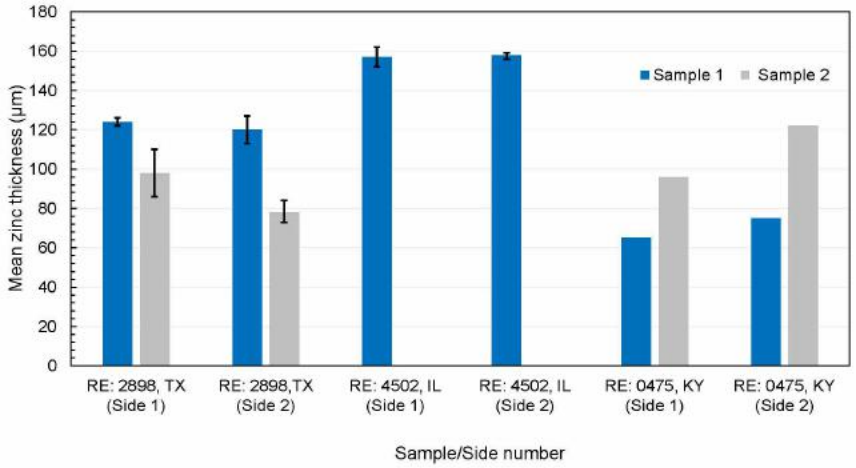


Figure 4 – Zinc thickness measurements from the portion of the reinforcements within the tie strips. Note: KY Sample 1 and Sample 2 in this chart correspond to Samples 4 and 6, respectively.

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Pit Depths

The pit depth measurements from different reinforcement samples retrieved from the Texas, Illinois, and Kentucky sites are presented in Figure 5. The bar graphs in this figure show the average pit depths and the error bars show the range for the minimum and the maximum of the measurements.

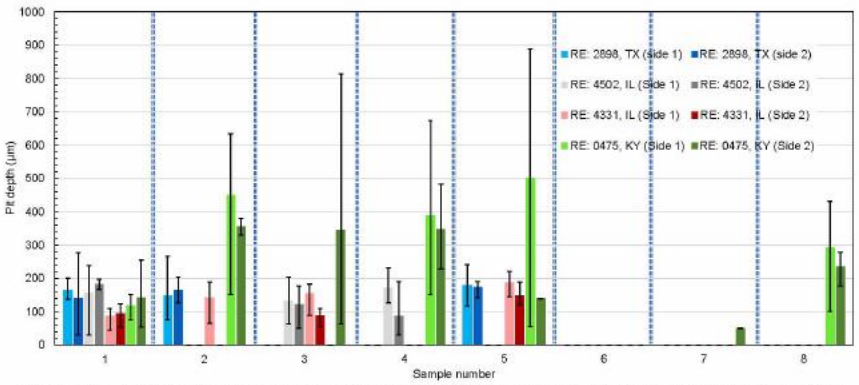


Figure 5 – Pit depth measurements. Note: KY Samples 1 -8 correspond to Samples Extra 1, 2, Extra 2, 4, 5, 6, 12 and 15, respectively.

In general, the deepest pits were observed from the samples retrieved from Kentucky. The average pit depths measured from each sample from Kentucky were quite variable and ranged between **50 µm and 500 µm, and the deepest pit was observed to be 869 µm**. The low resistivity and high chloride and sulfate ion contents of the fill sampled from Kentucky may have contributed to more aggressive pitting.

Strip and weigh measurements

The strip and weigh measurements were performed according to the ASTM A90/A90M (2018) by immersing the specimens in the stripping solution (dilute hydrochloric acid). The specimens selected for these measurements were three reinforcement samples (approximate size = 2 inches by 2 inches) from the sites located in Texas, Illinois, and Kentucky. Table 2 shows the results from the three samples.

Table 2
Results from the strip and weigh measurements

Site	Surface area (m^2)	Mass loss (g)	Coating weight (g/m^2)	Coating weight (oz/ft^2)	Coating thickness (µm)	Sample #	Elcometer Measurements from each side (Points E/J) (µm)
TX	0.005902	3.363	570	1.87	80	5	77/84
IL	0.004743	4.436	935	3.06	131	2	143/140
KY	0.004309	0.272	63	0.206	9	15	24/20

The last two columns on the right-hand side of Table 2 indicate the samples that were selected for strip and weigh measurements and the zinc thickness measurements from each side of the sample measured with the Elcometer. These values agree well with the results of the zinc thickness measurements using strip and weigh measurements.

SEM/EDS

Two samples obtained from the Kentucky site (KY – 4A and KY – 4B) were selected for the preparation of micrographs and analysis using SEM/EDS. The micrographs were used for optical measurements made via scanning electron microscopy (SEM) with elemental analysis via energy-dispersive spectroscopy (EDS). These measurements were used for comparison with the measurements from the electronic thickness gage, and to identify the elemental composition of the corrosion products, and remaining galvanization on the surfaces of the samples. McMahon & Mann engaged Electron Microscopy Innovation Technologies LLC. (EMIT) to prepare the micrographs and perform SEM and EDS measurements.

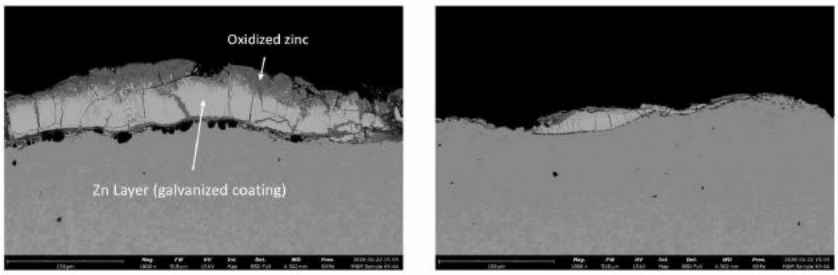
Figures 6, 7, and 8 depict the results from SEM measurements and EDS performed on the samples from Kentucky. One of the samples from KY was obtained at the connection (KY – 4A) and the other one was obtained from the portion of the same reinforcement strip that was exposed to the fill (KY – 4B). EMIT observed clear evidence of a zinc-rich galvanized surface layer on Sample KY – 4A, which ranged from about 50 µm - 100 µm in thickness. The average zinc thickness measured by Elcometer at the connection for strip samples from KY was about 90 µm, which agreed well with the above observation from SEM analyses.

Figure 6 (a) shows SEM images taken from Sample KY – 4A, where most of the surface is covered by the zinc coating but at some spots, the coating was spalled/chipped off of the surface. As indicated in this figure, the semi-quant of lighter gray on the zinc coating presents the corroded regions. EDS analyses for Sample KY – 4A (Figure 7) showed that the outer surface of the zinc-rich layer was partially oxidized. High levels of chloride ions (7.5% and 1.0% by weight) and sulfate ions (2.5% by weight) were observed in this layer. This is an unexpected result as this sample was obtained at the connection with limited exposure to the corrosive agents.

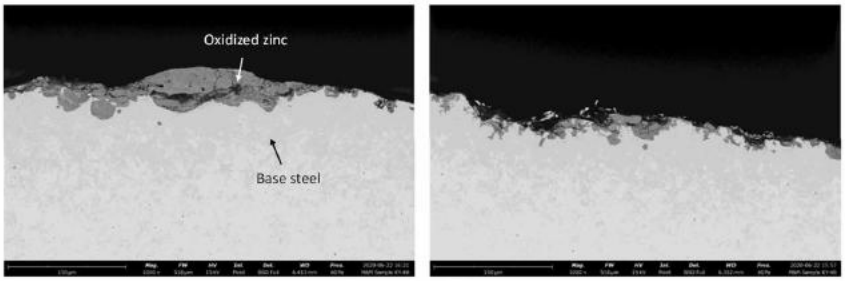
In Sample KY 4B, the zinc coating was nearly completely consumed as all the analyzed spots contained less than 0.5% by weight zinc. Figure 6 (b) presents SEM images taken from this specimen that shows the entire zinc coating on the surface is consumed. This observation may be compared with the minimum zinc thickness measurement for KY strip specimens (about 6 µm).



The results of the SEM analyses present local properties at specific locations and are not necessarily representative of the entire surface of the sample.



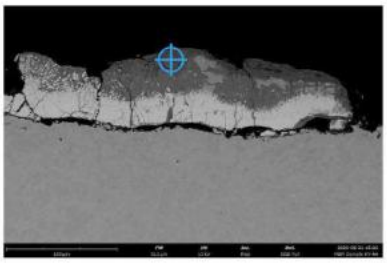
(a) Sample KY – 4A



(b) Sample KY – 4B

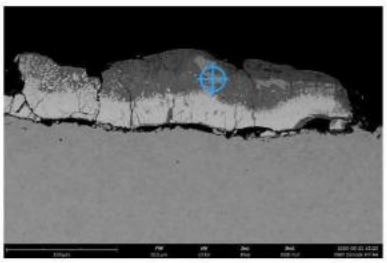
Figure 6 – SEM images of reinforcement strips

EDS analyses for Sample KY – 4B showed a thin band of zinc remained at the steel/oxide interface (Figure 8). The iron oxides that would form after the zinc is completely consumed, do not adhere well to steel substrates and will continue to exfoliate over time, which might explain why there is little evidence of remaining zinc on the surface. No chloride or sulfate ions were found on the surface of this sample, which is unexpected as the sample was heavily corroded and obtained from the exposed portion of the reinforcement strip. This may be explained by wetting-drying cycles, which may have flushed-out the corrosive agents on the surface of this sample. In addition, the easier access to oxygen along with the wetting-drying cycles accelerates the corrosion rate, which may be the reason for the corrosion observed in this sample.



Element Number	Element Symbol	Element Name	Atomic Conc.	Weight Conc.
8	O	Oxygen	43.46	17.50
16	S	Sulfur	3.10	2.50
17	Cl	Chlorine	8.40	7.50
25	Mn	Manganese	0.00	0.00
26	Fe	Iron	6.76	9.50
30	Zn	Zinc	38.28	63.00

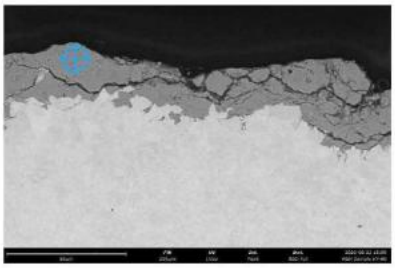
EDS semi-quant of partially oxidized Zn coating. Note the high levels of chlorine and sulfur detected



Element Number	Element Symbol	Element Name	Atomic Conc.	Weight Conc.
8	O	Oxygen	40.27	14.50
17	Cl	Chlorine	1.25	1.00
26	Fe	Iron	7.16	9.00
30	Zn	Zinc	51.32	75.50

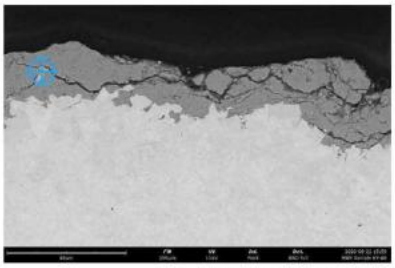
EDS semi-quant light gray region shows slightly less oxidation of the Zn coating and lower concentration of chlorine compared to the region above.

Figure 7 – EDS analysis for Sample KY – 4A



Element Number	Element Symbol	Element Name	Atomic Conc.	Weight Conc.
8	O	Oxygen	53.47	24.88
14	Si	Silicon	0.61	0.50
25	Mn	Manganese	0.31	0.50
26	Fe	Iron	45.34	73.63
30	Zn	Zinc	0.26	0.50

EDS semi-quant of oxide on surface shows low concentrations of Zn



Element Number	Element Symbol	Element Name	Atomic Conc.	Weight Conc.
8	O	Oxygen	39.20	15.58
14	Si	Silicon	0.00	0.00
25	Mn	Manganese	0.00	0.00
26	Fe	Iron	60.49	83.92
30	Zn	Zinc	0.31	0.50

EDS semi-quant of bright region in oxdie shows low concentrations of Zn

Figure 8 – EDS analysis for Sample KY – 4B.

Tension Test

Table 3 summarizes the results from tension tests performed on the samples. Specimens for tension testing were cut from each sample with lengths of 1 inch or 2 inches and with widths approximately one quarter of the specimen lengths (i.e., L/W ≈ 4). Thickness measurements were made on the samples (**Sample thickness range**) and on the specimens (**Spec. thick.**). The thickness of the specimens that were cut for tension testing were within 5 percent of the lowest thickness measured from the samples. The thicknesses measured from pitted samples/specimens appear to be less than those from the nonpitted samples.

Some differences are apparent in the properties observed from pitted compared to non-pitted samples. The pitted samples are observed to be more brittle compared to nonpitted samples from the same sites. Lower elongations at break and stiffnesses that are slightly higher are observed from pitted samples. The yield strengths of the pitted specimens also appear to be higher (up to 22%) compared to nonpitted samples, while the ultimate strengths are within approximately 8 percent. The largest differences in strengths are observed from the Kentucky samples.

In general, the yield and tensile strength from the Kentucky samples are less than those from Texas and Illinois. This may be due to the use of lower grade steel (Grade 36) when the walls in Kentucky were designed and constructed (before 1980). The lower grade steel may be affected by pitting differently compared to higher grades. The yield stress appears to be high for Grade 36 steel, but this is possible depending on the heat from which these reinforcements were manufactured.

Table 3
Results of uniaxial tensile tests

Type	ID	Sample thickness range (mm)	Spec. thick. (mm)	Condition	UTS[1] (ksi)	Elongation at break (%)	0.2% YS[2] (ksi)	E[3] (ksi)
Strips	KY-12	5.15 - 5.48	5.01	not pitted	73.2	26	51.6	23,200
	KY-15	4.17 – 4.64	3.97	pitted	79.4	13	62.8	24,900
	TX-7	4.36 – 4.68	4.18	not pitted	85.5	26	62.4	28,300
	TX-2	4.06 – 4.28	3.88	pitted	90.0	20	67.0	29,300
	IL-3	4.55 – 4.79	4,52	pitted	95.0	15	67.9	27,100

[1]UTS = ultimate tensile strength
[2]YS = yield strength
[3]E = modulus of elasticity

CONCLUSIONS

- Good comparisons between remaining zinc measurements from the Elcometer (zinc thickness gage), strip and weigh measurements and SEM were observed. This gives confidence to the measurements from the Elcometer, which include the bulk of the thickness measurements as presented in Figures 1 through 4.

- The performance of the strips exceeds current design requirements (as well as the original design). According to the AASHTO Metal Loss model, zinc is fully consumed in 16 years. Even though the backfill did not meet AASHTO requirements for resistivity and sulfate content, there is still zinc remaining after 38 years

- Regarding the strips within the connection, and from photographs of the samples from Kentucky, it appears that the tie strips were not completely tightened against the reinforcement. This could have affected the zinc loss within the connection during service.

- The KY Structure was 38 years old when the samples were exhumed and was constructed in 1980. At the time the wall was designed and constructed, RECo's reinforcements were manufactured with ASTM A 36 rather than Grade 50 steel, and ASTM A-570 Grade C (Fy = 33 ksi) was used for the tie strips This explains the lower yield and ultimate strengths from tension tests as described in Table 3.

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A1.2.7 Dimensional Tolerances

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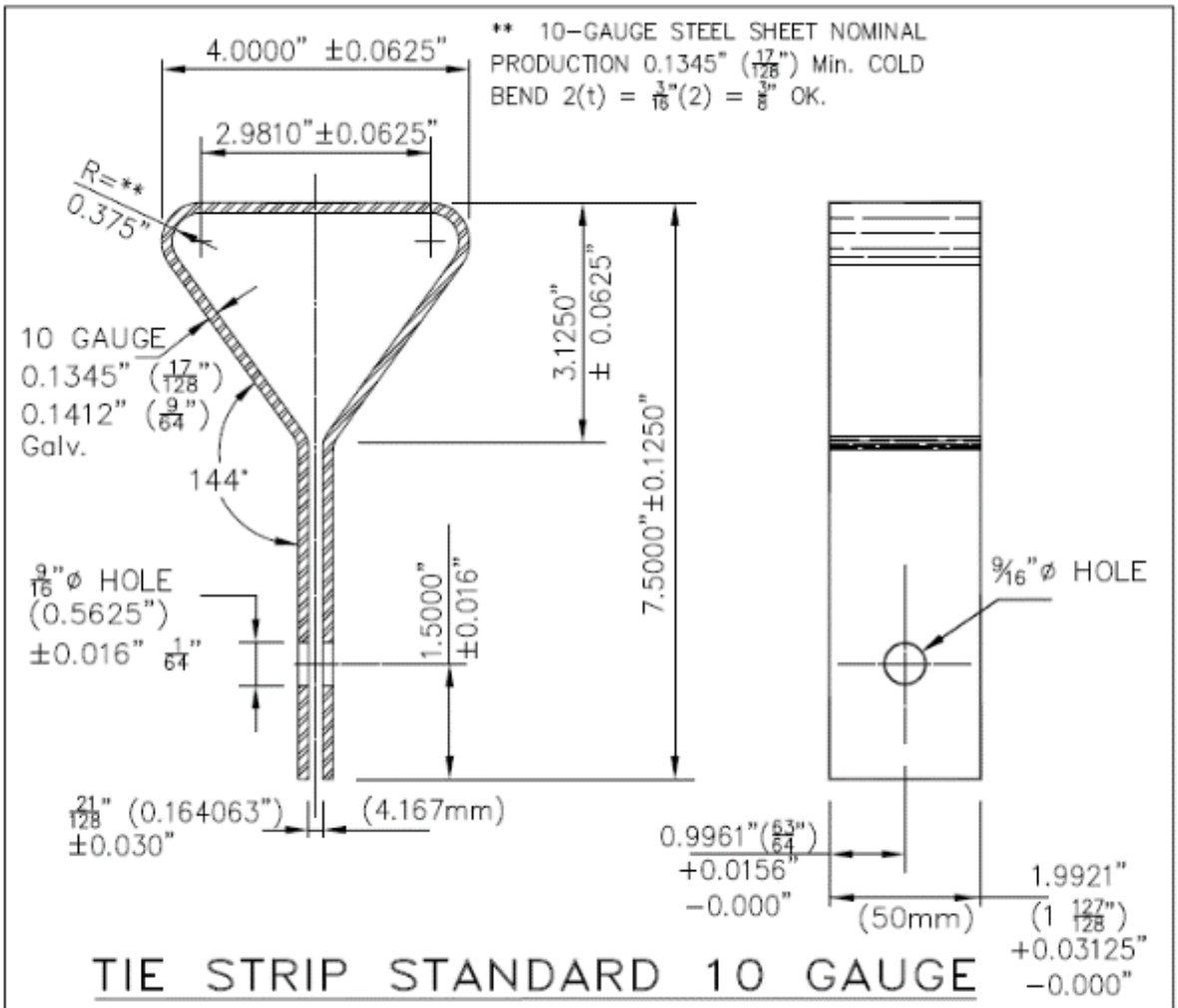
A1.2.8 Reinforcement Connection to Facing Units

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A1.2.9 Connection Devices

The following documentation is included in this section:

- Tie strip details (3 pages)
- Connection strength calculation cases (10 pages)



TIE STRIP STANDARD 10 GAUGE

All Material Shall Be 100% Domestic In Content, as Accordance With The 'Buy America Act', 'American Recovery and Reinvestment Act' (ARRA), and Statutory Provisions Set Forth By the Federal Highway Administration (FHWA), Found Within Title 23 U.S.C. § 313; Regulatory Policy Found in Title 23 C.F.R. § 635.410, Including All Modified Provisions and Rulings.

Description: 19½" LONG

Tie Strip	10 Gauge	50mm	Galvanized
Item	Material Thick.	Width	hot-dip zinc coating
	17/128"(0.1345")	127/128"(1.9921")	85µm (2) 0.006693"

General Information:

Manufacturer: (RECo Approved Vender or Approved Equal)

Material Specification: ASTM A1011/A1011M-13, 'STANDARD SPECIFICATION FOR STEEL, SHEET AND STRIP, HOT-ROLLED, CARBON, STRUCTURAL, HIGH-STRENGTH LOW-ALLOY, HIGH-STRENGTH LOW-ALLOY WITH IMPROVED FORMABILITY, AND ULTRA-HIGH STRENGTH' SS Gr.50 or HSLAS Gr.50, Class 1, Mech. Properties: Min. Tensile Strength 65ksi, Min. Yield Strength 50ksi.

Finish Specification: ZINC (HOT-DIP GALVANIZED) SHALL COMPLY WITH ASTM A123/A123M-13, TABLE 2, GRADE 85, MINIMUM THICKNESS 2.0 OZ/FT², 3.3 mils, 85µm, 600 g/m².

Fabrication Coments: NOTE: ALLOWABLE TOLERANCE FOR VARIATION OF CENTERLINES OF HOLES SHALL NOT EXCEED ± 1/32" IN THE FINISHED PRODUCT.

FABRICATION DRAWING

RECo Part #: TS10G

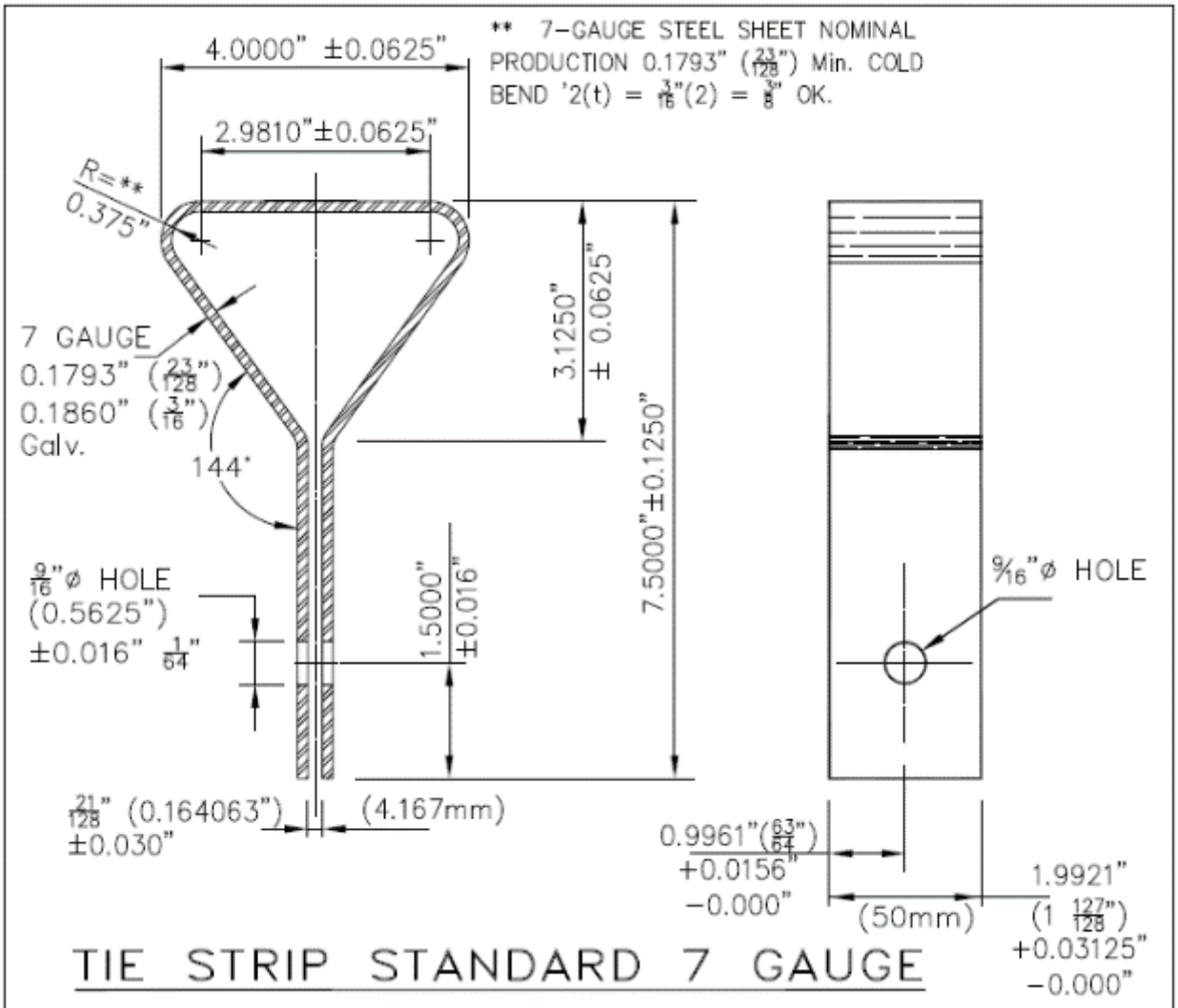
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TIE STRIP STANDARD 7 GAUGE

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Description: 19½" LONG

Item	Material Thick.	Width	Galvanized
Tie Strip	7 Gauge	50mm	hot-dip zinc coating
	23/128" (0.1793") (4.554mm)	1 31/32" (1.969")	85µm (2) 0.006693"

General Information:

Manufacturer: (RECo Approved Vender or Approved Equal)

Material Specification: ASTM A1011/A1011M-13, 'STANDARD SPECIFICATION FOR STEEL, SHEET AND STRIP, HOT-ROLLED, CARBON, STRUCTURAL, HIGH-STRENGTH LOW-ALLOY, HIGH-STRENGTH LOW-ALLOY WITH IMPROVED FORMABILITY, AND ULTRA-HIGH STRENGTH' SS Gr.50 or HSLAS Gr.50, Class 1, Mech. Properties: Min. Tensile Strength 65ksi, Min. Yield Strength 50ksi.

Finish Specification: ZINC (HOT-DIP GALVANIZED) SHALL COMPLY WITH ASTM A123/A123M-13, TABLE 2, GRADE 85, MINIMUM THICKNESS 2.0 OZ/FT², 3.3 mils, 85µm, 600 g/m².

Fabrication Coments: NOTE: ALLOWABLE TOLERANCE FOR VARIATION OF CENTERLINES OF HOLES SHALL NOT EXCEED ± 1/32" IN THE FINISHED PRODUCT.

FABRICATION DRAWING

RECo Part #:
TS07G

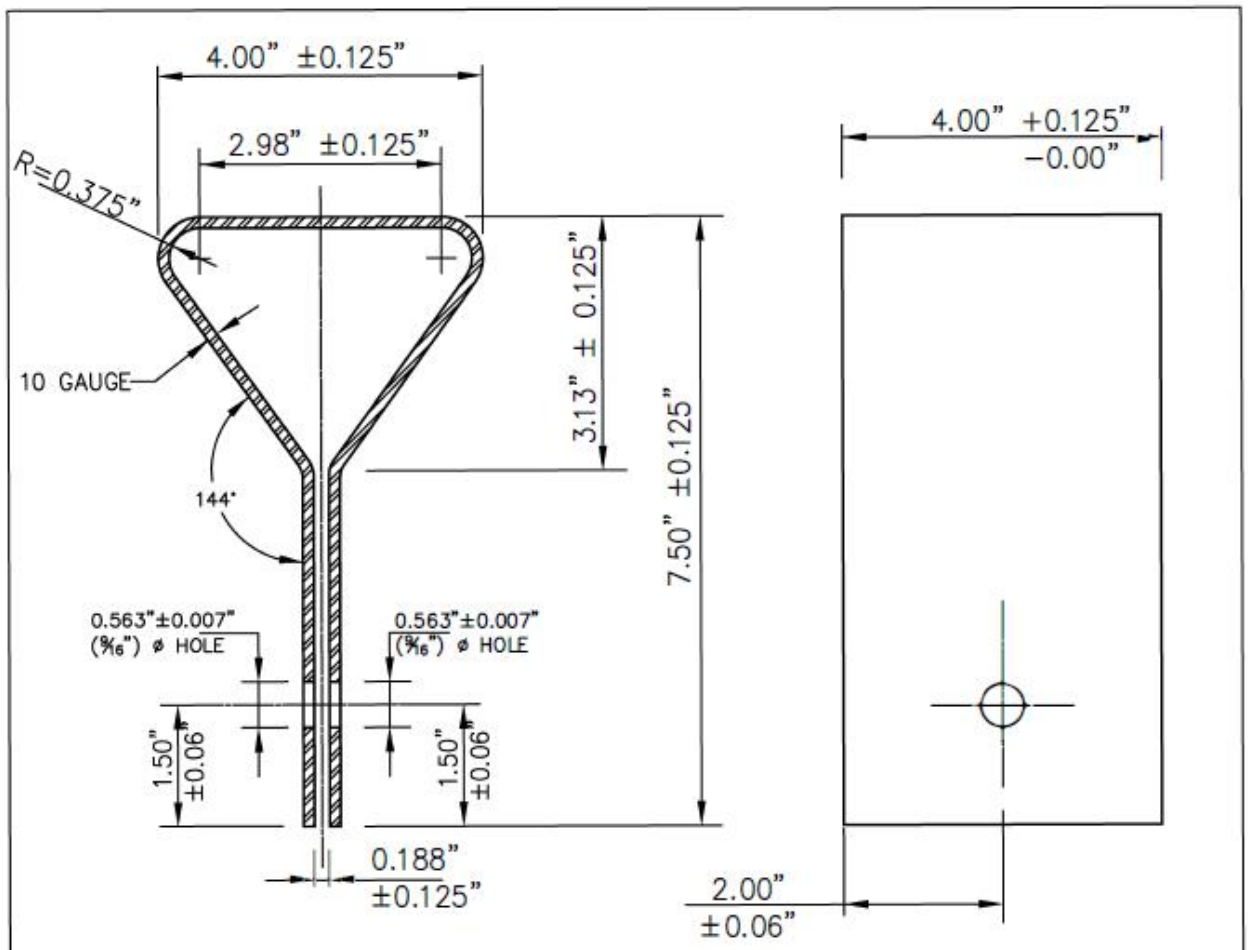
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REINFORCED EARTH
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REV. 05/2014



10 GAUGE 4" WIDE SPECIAL TIE STRIP DETAIL (ENGLISH)

ALL MATERIAL SHALL BE DOMESTIC STEEL, COMPLYING WITH GUIDELINES SET FOURTH WITHIN THE UNITED STATES GOVERNMENT "BUY AMERICAN ACT", INCLUDING ALL MODIFYING PROVISIONS.

Description:

10 Gauge X 4.00" X 19.48" Tie Strip

- Item
- Required Length
- Required Width
- Required Thickness

General Information:

Manufacturer:	(RECo Approved Vender or Approved Equal)
Material Specification:	ASTM A 1011/A 1011M–07 SS Gr.50 or HSLAS Gr.50, Class 1, (Formerly ASTM A–570), Mech. Properties: Min. Tensile Strength 65ksi, Min. Yield Strength 50ksi.
Finish Specification:	ASTM A–123/A 123M–02, Grade 85. hot–Dip Galvanized (Zinc Coating). thickness 3.3mils, 2.0 oz/s.f.
Fabrication Comments:	NOTE: ALLOWABLE TOLERANCE FOR VARIATION OF CENTERLINES OF HOLES SHALL NOT EXCEED ± 1/16" IN THE FINISHED PRODUCT.
Package Quanitity:	1 (EA.)

FABRICATION DRAWING

RECo Part #: TS4WG

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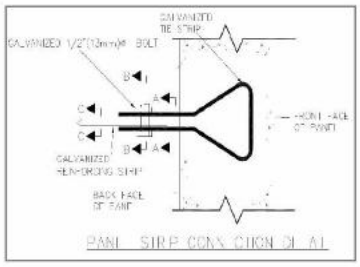
REV.

#1) 10 Gauge - 4 mm Tie Strip Designed for 75 Years Design Life

JOB NUMBER:	IDEA
JOB NAME:	IDEA
DESIGNED BY:	DE

Galvanization and Carbon Steel Loss Rates:

Design Life of Structure =	75	yrs
Zinc (first 2 years):	15	µm/yr
Zinc (subsequent years):	4	µm/yr
Carbon Steel:	12	µm/yr
Carbon Steel (75 - 100 yrs)	12	µm/yr



Mechanical Properties of Reinforcement Hardware

	HA Strip	Tie Strip
Thickness t (mm)=	4	3.42
Width w (mm)=	50	50
A (mm2) =	200	171
Fu (Ksi) =	80	65
Fy (Ksi) =	65	50

Thickness of Zinc Coat.

Zinc coat tz = 86 µm

Life of Zinc: 16 yrs

Bolt Set.

Diameter 12.7 mm

Fu = 120 Ksi

Bolt Hole= 14.29 mm

TENSILE RESISTANCE OF REINFORCING STRIP & CONNECTORS. LRFD METHOD

Tensile Resistance Resistance Factor for Metallic Reinforcement and connectors= Φ = **0.75**

Section A-A (Tie Strip Only), 2 plates (Yielding limit state)

At end of design life, carbon steel loss : **0.708** mm/side

Width(less- slot) 50 mm

Thickness 2.004 mm

Remaining Thickness /plate = 2.004 mm

As (2 plates) = 200.400 mm2

0.311 in^2

Factored Tensile Resistance= Φ FyAs = **11.65** Kips

Section B-B-1 (Reinforcing Strip at Bolt Holes)(Fracture Limit State)

At end of design life, carbon steel loss : **0** mm/side

Width(less- bolt hole) 35.71 mm

Thickness= 4 mm

Remaining Thickness /plate = 4 mm

Ae = 142.840 mm2

0.221 in^2

Factored Tensile Resistance= Φ AeFu = **13.28** Kips

Section B-B-2 (Bearing strength of strips @ Bolt Holes)

At end of design life, carbon steel loss : **0** mm/side

Lc(Clear edge =1-1/2"-9/32) 1.22 in

Remaining Thickness /plate = 4 mm

Factored Bearing Resistance at bolt hole=Φ. Rn= Φ(1.2Lc t Fu)<=Φ(2.4 d t Fu) **11.34** Kips

Section B-B-3 (Shear Resistance of Bolt) threads are excluded

At end of design life, carbon steel loss : **0** mm/side

Area of bolt = 0.19635 in2

AASHTO 6.13.2.7-1 Factored Shear Resistance =Φ Rn= Φ(0.56 Fu Ab Ns) = **19.79** Kips

Section B-B-4 (Tie Strip Connector@ Bolt Holes)(Fracture Limit State)

At end of design life, carbon steel loss : **0.708** mm/side

Width(less- bolt hole) 35.71 mm

Thickness= 2.712 mm

Remaining Thickness (one side)/plate = 2.712 mm

Ae(2Plates) = 193.691 mm2

0.300 in^2

Factored Tensile Resistance= Φ AeFu = **14.64** Kips

Section B-B-5 (Bearing strength of Tie Strip @ Bolt Holes)

At end of design life, carbon steel loss : **0** mm/side

Lc(Clear edge =1-1/2"-9/32) 1.22 in

Remaining Thickness(2plate =) 6.84 mm

Factored Bearing Resistance at bolt hole=Φ. Rn= Φ(1.2Lc t Fu)<=Φ(2.4 d t Fu) **15.75** Kips

Section C-C (Reinforcing Strip),(Yielding limit state)

At end of design life, carbon steel loss : **0.708** mm/side

Width(less- slot) 50 mm

Thickness 2.584 mm

Remaining Thickness strip= 2.584 mm

As = 129.200 mm2

0.200 in^2

Factored Tensile Resistance= Φ FyAs = **9.76** Kips

Controling Maximum Tension Load= **9.76** Kips

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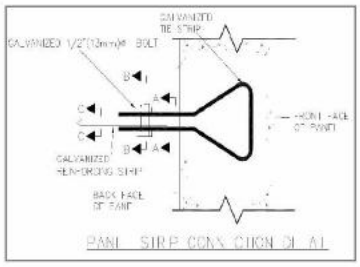
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#2) 10 Gauge - 4 mm Tie Strip Designed for 100 Years Design Life

JOB NUMBER: IDEA
JOB NAME: IDEA
DESIGNED BY: DE

Galvanization and Carbon Steel Loss Rates:

Design Life of Structure =	100	yrs
Zinc (first 2 years):	15	µm/yr
Zinc (subsequent years):	4	µm/yr
Carbon Steel:	12	µm/yr
Carbon Steel (75 - 100 yrs)	12	µm/yr



Mechanical Properties of Reinforcement Hardware

	HA Strip	Tie Strip
Thickness t (mm)=	4	3.42
Width w (mm)=	50	50
A (mm2) =	200	171
Fu (Ksi) =	80	65
Fy (Ksi) =	65	50

Thickness of Zinc Coat.

Zinc coat tz = 86 µm
Life of Zinc: 16 yrs

Bolt Set.

Diameter 12.7 mm
Fu = 120 Ksi

Bolt Hole= 14.29 mm

TENSILE RESISTANCE OF REINFORCING STRIP & CONNECTORS. LRFD METHOD

Tensile Resistance Resistance Factor for Metallic Reinforcement and connectors= Φ = **0.75**

Section A-A (Tie Strip Only), 2 plates (Yielding limit state)

At end of design life, carbon steel loss : **1.008** mm/side
Width(less- slot) 50 mm
Thickness 1.404 mm

Remaining Thickness /plate = 1.404 mm
As (2 plates) = 140.400 mm2
0.218 in^2
Factored Tensile Resistance= Φ FyAs = **8.16** Kips

Section B-B-1 (Reinforcing Strip at Bolt Holes)(Fracture Limit State)

At end of design life, carbon steel loss : **0** mm/side
Width(less- bolt hole) 35.71 mm
Thickness= 4 mm

Remaining Thickness /plate = 4 mm
Ae = 142.840 mm2
0.221 in^2
Factored Tensile Resistance= Φ AeFu = **13.28** Kips

Section B-B-2 (Bearing strength of strips @ Bolt Holes)

At end of design life, carbon steel loss : **0** mm/side
Lc(Clear edge =1-1/2"-9/32) 1.22 in

Remaining Thickness /plate = 4 mm
Factored Bearing Resistance at bolt hole=Φ. Rn= Φ(1.2Lc t Fu)<=Φ(2.4 d t Fu) **11.34** Kips

Section B-B-3 (Shear Resistance of Bolt) threads are excluded

At end of design life, carbon steel loss : **0** mm/side
AASHTO 6.13.2.7-1

Area of bolt = 0.19635 in2
Factored Shear Resistance =Φ Rn= Φ(0.56 Fu Ab Ns) = **19.79** Kips

Section B-B-4 (Tie Strip Connector@ Bolt Holes)(Fracture Limit State)

At end of design life, carbon steel loss : **1.008** mm/side
Width(less- bolt hole) 35.71 mm
Thickness= 2.412 mm

Remaining Thickness (one side)/plate = 2.412 mm
Ae(2Plates) = 172.265 mm2
0.267 in^2
Factored Tensile Resistance= Φ AeFu = **13.02** Kips

Section B-B-5 (Bearing strength of Tie Strip @ Bolt Holes)

At end of design life, carbon steel loss : **0** mm/side
Lc(Clear edge =1-1/2"-9/32) 1.22 in

Remaining Thickness(2plate =) 6.84 mm
Factored Bearing Resistance at bolt hole=Φ. Rn= Φ(1.2Lc t Fu)<=Φ(2.4 d t Fu) **15.75** Kips

Section C-C (Reinforcing Strip),(Yielding limit state)

At end of design life, carbon steel loss : **1.008** mm/side
Width(less- slot) 50 mm
Thickness 1.984 mm

Remaining Thickness strip= 1.984 mm
As = 99.200 mm2
0.154 in^2
Factored Tensile Resistance= Φ FyAs = **7.5** Kips

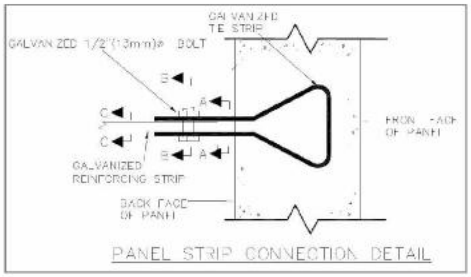
Controling Maximum Tension Load= **7.50** Kips

#3) 7 Gauge - 6 mm Tie Strip Designed for 75 Years Design Life

JOB NUMBER: IDEA
JOB NAME: IDEA
DESIGNED BY: DE

Galvanization and Carbon Steel Loss Rates:

Design Life of Structure =	75	yrs
Zinc (first 2 years):	15	µm/yr
Zinc (subsequent years):	4	µm/yr
Carbon Steel:	12	µm/yr
Carbon Steel (75 - 100 yrs)	12	µm/yr



PANEL STRIP CONNECTION DETAIL

Mechanical Properties of Reinforcement Hardware

	HA Strip	Tie Strip
Thickness t (mm)=	6	4.55
Width w (mm)=	50	50
A (mm2) =	300	227.5
Fu (Ksi) =	80	65
Fy (Ksi) =	65	50

Thickness of Zinc Coat.

Zinc coat tz = 86 µm
Life of Zinc: 16 yrs

Bolt Set.

Diameter 12.7 mm
Fu = 120 Ksi

Bolt Hole= 14.29 mm

TENSILE RESISTANCE OF REINFORCING STRIP & CONNECTORS. LRFD METHOD

Tensile Resistance Resistance Factor for Metallic Reinforcement and connectors= Φ = **0.75**

Section A-A (Tie Strip Only), 2 plates (Yielding limit state)

At end of design life, carbon steel loss : **0.708** mm/side
Width(less- slot) 50 mm
Thickness 3.134 mm
Remaining Thickness /plate = 3.134 mm
As (2 plates) = 313.400 mm2
0.486 in^2
Factored Tensile Resistance= Φ FyAs = **18.22** Kips

Section B-B-1 (Reinforcing Strip at Bolt Holes)(Fracture Limit State)

At end of design life, carbon steel loss : **0** mm/side
Width(less- bolt hole) 35.71 mm
Thickness= 6 mm
Remaining Thickness /plate = 6 mm
Ae = 214.260 mm2
0.332 in^2
Factored Tensile Resistance= Φ AeFu = **19.93** Kips

Section B-B-2 (Bearing strength of strips @ Bolt Holes)

At end of design life, carbon steel loss : **0** mm/side
Lc(Clear edge =1-1/2"-9/32) 1.22 in
Remaining Thickness /plate = 6 mm
Factored Bearing Resistance at bolt hole=Φ. Rn= Φ(1.2Lc t Fu)<=Φ(2.4 d t Fu) **17.01** Kips

Section B-B-3 (Shear Resistance of Bolt) threads are excluded

At end of design life, carbon steel loss : **0** mm/side
AASHTO 6.13.2.7-1
Area of bolt = 0.19635 in2
Factored Shear Resistance =Φ Rn= Φ(0.56 Fu Ab Ns) = **19.79** Kips

Section B-B-4 (Tie Strip Connector@ Bolt Holes)(Fracture Limit State)

At end of design life, carbon steel loss : **0.708** mm/side
Width(less- bolt hole) 35.71 mm
Thickness= 3.842 mm
Remaining Thickness (one side)/plate = 3.842 mm
Ae(2Plates) = 274.396 mm2
0.425 in^2
Factored Tensile Resistance= Φ AeFu = **20.73** Kips

Section B-B-5 (Bearing strength of Tie Strip @ Bolt Holes)

At end of design life, carbon steel loss : **0** mm/side
Lc(Clear edge =1-1/2"-9/32) 1.22 in
Remaining Thickness(2plate =) 9.1 mm
Factored Bearing Resistance at bolt hole=Φ. Rn= Φ(1.2Lc t Fu)<=Φ(2.4 d t Fu) **20.96** Kips

Section C-C (Reinforcing Strip),(Yielding limit state)

At end of design life, carbon steel loss : **0.708** mm/side
Width(less- slot) 50 mm
Thickness 4.584 mm
Remaining Thickness strip= 4.584 mm
As = 229.200 mm2
0.355 in^2
Factored Tensile Resistance= Φ FyAs = **17.32** Kips

Controling Maximum Tension Load= **17.01** Kips

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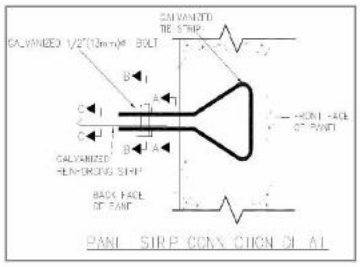
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#4) 7 Gauge - 6 mm Tie Strip Designed for 100 Years Design Life

JOB NUMBER: IDEA
JOB NAME: IDEA
DESIGNED BY: DE

Galvanization and Carbon Steel Loss Rates:

Design Life of Structure =	100	yrs
Zinc (first 2 years):	15	µm/yr
Zinc (subsequent years):	4	µm/yr
Carbon Steel:	12	µm/yr
Carbon Steel (75 - 100 yrs)	12	µm/yr



Mechanical Properties of Reinforcement Hardware

	HA Strip	Tie Strip
Thickness t (mm)=	6	4.55
Width w (mm)=	50	50
A (mm2) =	300	227.5
Fu (Ksi) =	80	65
Fy (Ksi) =	65	50

Thickness of Zinc Coat.

Zinc coat tz = 86 µm
Life of Zinc: 16 yrs

Bolt Set.

Diameter 12.7 mm
Fu = 120 Ksi

Bolt Hole= 14.29 mm

TENSILE RESISTANCE OF REINFORCING STRIP & CONNECTORS. LRFD METHOD

Tensile Resistance Resistance Factor for Metallic Reinforcement and connectors= Φ = **0.75**

Section A-A (Tie Strip Only), 2 plates (Yielding limit state)

At end of design life, carbon steel loss : **1.008** mm/side
Width(less- slot) 50 mm
Thickness 2.534 mm
Remaining Thickness /plate = 2.534 mm
As (2 plates) = 253.400 mm2
0.393 in^2
Factored Tensile Resistance= Φ FyAs = **14.73** Kips

Section B-B-1 (Reinforcing Strip at Bolt Holes)(Fracture Limit State)

At end of design life, carbon steel loss : **0** mm/side
Width(less- bolt hole) 35.71 mm
Thickness= 6 mm
Remaining Thickness /plate = 6 mm
Ae = 214.260 mm2
0.332 in^2
Factored Tensile Resistance= Φ AeFu = **19.93** Kips

Section B-B-2 (Bearing strength of strips @ Bolt Holes)

At end of design life, carbon steel loss : **0** mm/side
Lc(Clear edge =1-1/2"-9/32) 1.22 in
Remaining Thickness /plate = 6 mm
Factored Bearing Resistance at bolt hole=Φ. Rn= Φ(1.2Lc t Fu)<=Φ(2.4 d t Fu) **17.01** Kips

Section B-B-3 (Shear Resistance of Bolt) threads are excluded

At end of design life, carbon steel loss : **0** mm/side
AASHTO 6.13.2.7-1
Area of bolt = 0.19635 in2
Factored Shear Resistance =Φ Rn= Φ(0.56 Fu Ab Ns) = **19.79** Kips

Section B-B-4 (Tie Strip Connector@ Bolt Holes)(Fracture Limit State)

At end of design life, carbon steel loss : **1.008** mm/side
Width(less- bolt hole) 35.71 mm
Thickness= 3.542 mm
Remaining Thickness (one side)/plate = 3.542 mm
Ae(2Plates) = 252.970 mm2
0.392 in^2
Factored Tensile Resistance= Φ AeFu = **19.12** Kips

Section B-B-5 (Bearing strength of Tie Strip @ Bolt Holes)

At end of design life, carbon steel loss : **0** mm/side
Lc(Clear edge =1-1/2"-9/32) 1.22 in
Remaining Thickness(2plate =) 9.1 mm
Factored Bearing Resistance at bolt hole=Φ. Rn= Φ(1.2Lc t Fu)<=Φ(2.4 d t Fu) **20.96** Kips

Section C-C (Reinforcing Strip),(Yielding limit state)

At end of design life, carbon steel loss : **1.008** mm/side
Width(less- slot) 50 mm
Thickness 3.984 mm
Remaining Thickness strip= 3.984 mm
As = 199.200 mm2
0.309 in^2
Factored Tensile Resistance= Φ FyAs = **15.05** Kips

Controling Maximum Tension Load= **14.73** Kips

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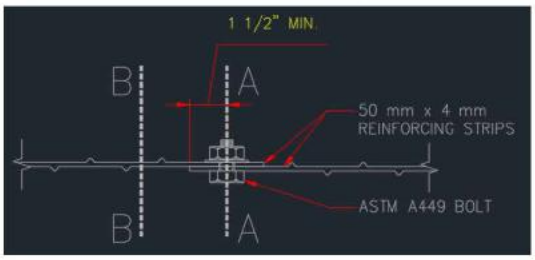
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#5) Single Splice Connection Designed for 75 Years Design Life

JOB NUMBER: IDEA
JOB NAME: IDEA
DESIGNED BY: DE

Galvanization and Carbon Steel Loss Rates:

Design Life of Structure =	75	yrs
Zinc (first 2 years):	15	µm/yr
Zinc (subsequent years):	4	µm/yr
Carbon Steel:	12	µm/yr
Carbon Steel (75 - 100 yrs)	12	µm/yr



Mechanical Properties of Reinforcement Hardware

HA Strip

Thickness t (mm)=	4
Width w (mm)=	50
A (mm2) =	200
Fu (Ksi) =	80
Fy (Ksi) =	65

Thickness of Zinc Coat.

Zinc coat tz	86	µm
Life of Zinc	16	yrs

Bolt Set.

Diameter	12.7	mm
Fu =	120	Ksi
Bolt Hole=	14.29	mm

Tensile Resistance Resistance Factor for Metallic Reinforcement and connectors= Φ = **0.75**

Section A-A (Reinforcing Strip at Bolt Holes)(Fracture Limit State): 1 SIDE EXPOSED

At end of design life, carbon steel loss : **0.708** mm/side — Remaining Thickness /plate = 3.292 mm

Width(less- bolt hole) 35.71 mm — Ae = 117.557 mm2

Thickness= 3.292 mm — 0.182 in^2

Factored Tensile Resistance= Φ AeFu = **10.93** Kips

Section A-A (Bearing strength of strips @ Bolt Holes): 1 SIDE EXPOSED

At end of design life, carbon steel loss : **0.708** mm/side — Remaining Thickness /plate = 3.292 mm

Lc(Clear edge =1-1/2"-9/32) 1.22 in

Factored Bearing Resistance at bolt hole=Φ. Rn= Φ(1.2Lc t Fu)<=Φ(2.4 d t Fu) **9.33** Kips

Section A-A (Shear Resistance of Bolt) **BOLT IN SINGLE SHEAR**

At end of design life, carbon steel loss : **0** mm/side — Area of bolt = 0.19635 in2

Factored Shear Resistance =Φ Rn= Φ(0.56 Fu Ab Ns) = **9.90** Kips

Section B-B (Reinforcing Strip),(Yielding limit state)

At end of design life, carbon steel loss : **0.708** mm/side — Remaining Thickness strip= 2.584 mm

Width 50 mm — As = 129.200 mm2

Thickness 2.584 mm — 0.200 in^2

Factored Tensile Resistance= Φ FyAs = **9.76** Kips

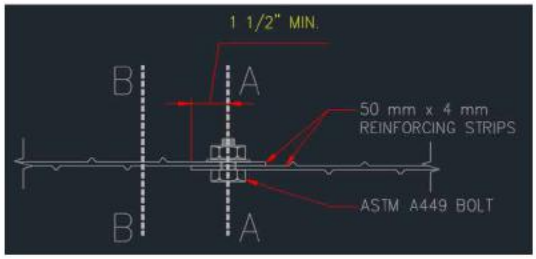
Controling Maximum Tension Load= 9.33 Kips

#6) Single Splice Connection Designed for 100 Years Design Life

JOB NUMBER: IDEA
JOB NAME: IDEA
DESIGNED BY: DE

Galvanization and Carbon Steel Loss Rates:

Design Life of Structure =	100	yrs
Zinc (first 2 years):	15	µm/yr
Zinc (subsequent years):	4	µm/yr
Carbon Steel:	12	µm/yr
Carbon Steel (75 - 100 yrs)	12	µm/yr



Mechanical Properties of Reinforcement Hardware

HA Strip

Thickness t (mm)=	4
Width w (mm)=	50
A (mm2) =	200
Fu (Ksi) =	80
Fy (Ksi) =	65

Thickness of Zinc Coat.

Zinc coat tz	86	µm
Life of Zinc	16	yrs

Bolt Set.

Diameter	12.7	mm
Fu =	120	Ksi
Bolt Hole=	14.29	mm

Tensile Resistance Resistance Factor for Metallic Reinforcement and connectors= Φ = **0.75**

Section A-A (Reinforcing Strip at Bolt Holes)(Fracture Limit State): 1 SIDE EXPOSED

At end of design life, carbon steel loss : **1.008** mm/side — Remaining Thickness /plate = 2.992 mm

Width(less- bolt hole) 35.71 mm — Ae = 106.844 mm2

Thickness= 2.992 mm — 0.166 in^2

Factored Tensile Resistance= Φ AeFu = **9.94** Kips

Section A-A (Bearing strength of strips @ Bolt Holes): 1 SIDE EXPOSED

At end of design life, carbon steel loss : **1.008** mm/side — Remaining Thickness /plate = 2.992 mm

Lc(Clear edge =1-1/2"-9/32) 1.22 in

Factored Bearing Resistance at bolt hole=Φ. Rn= Φ(1.2Lc t Fu)<=Φ(2.4 d t Fu) **8.48** Kips

Section A-A (Shear Resistance of Bolt) **BOLT IN SINGLE SHEAR**

At end of design life, carbon steel loss : **0** mm/side — Area of bolt = 0.19635 in2

Factored Shear Resistance =Φ Rn= Φ(0.56 Fu Ab Ns) = **9.90** Kips

Section B-B (Reinforcing Strip),(Yielding limit state)

At end of design life, carbon steel loss : **1.008** mm/side — Remaining Thickness strip= 1.984 mm

Width 50 mm — As = 99.200 mm2

Thickness 1.984 mm — 0.154 in^2

Factored Tensile Resistance= Φ FyAs = **7.5** Kips

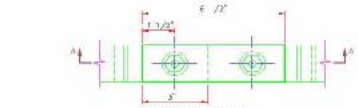
Controling Maximum Tension Load= 7.50 Kips

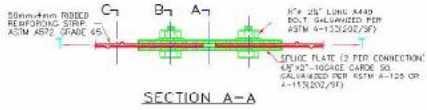
#7) Double Splice Connection With 10 Gauge Splice Plate Designed for 75 Years Design life

JOB NUMBER:	IDEA
JOB NAME:	IDEA
DESIGNED BY:	DE

Galvanization and Carbon Steel Loss Rates:

Design Life of Structure =	75	yrs
Zinc (first 2 years):	15	µm/yr
Zinc (subsequent years):	4	µm/yr
Carbon Steel:	12	µm/yr
Carbon Steel (75 - 100 yrs)	12	µm/yr





DOUBLE SPLICE PLATE CONNECTION DETAIL

Mechanical Properties of Reinforcement Hardware

	HA Strip	S-Plate
Thickness t (mm)=	4	3.42
Width w (mm)=	50	50
A (mm2) =	200	171
Fu (Ksi) =	80	65
Fy (Ksi) =	65	50

Thickness of Zinc Coat.
Zinc coat tz = 86 µm
Life of Zinc: 16 yrs

Bolt Set.
Diameter 12.7 mm
Fu = 120 Ksi
Bolt Hole= 14.29 mm

Tensile Resistance Resistance Factor for Metallic Reinforcement and connectors= Φ = **0.75**

Section A-A (SPLICE PLATES), 2 plates (Yielding limit state)
At end of design life, carbon steel loss : **0.708** mm/side Remaining Thickness /plate = 2.004 mm
Width 50 mm As (2 plates) = 200.400 mm2
Thickness 2.004 mm 0.311 in^2
Factored Tensile Resistance= Φ FyAs = **11.65** Kips

Section B-B-1 (Reinforcing Strip at Bolt Holes)(Fracture Limit State)
At end of design life, carbon steel loss : **0** mm/side Remaining Thickness /plate = **4** mm
Width(less- bolt hole) 35.71 mm Ae = 142.840 mm2
Thickness= 4 mm 0.221 in^2
Factored Tensile Resistance= Φ AeFu = **13.28** Kips

Section B-B-2 (Bearing strength of strips @ Bolt Holes)
At end of design life, carbon steel loss : **0** mm/side Remaining Thickness /plate = **4** mm
Lc(Clear edge =1-1/2"-9/32) 1.22 in
Factored Bearing Resistance at bolt hole=Φ. Rn= Φ(1.2Lc t Fu)<=Φ(2.4 d t Fu) **11.34** Kips

Section B-B-3 (Shear Resistance of Bolt)
At end of design life, carbon steel loss : **0** mm/side Area of bolt = 0.19635 in2
Factored Shear Resistance =Φ Rn= Φ(0.56 Fu Ab Ns) = **19.79** Kips

Section B-B-4 (SPLICE PLATE ConnectorS@ Bolt Holes)(Fracture Limit State)
At end of design life, carbon steel loss : **0.708** mm/side Remaining Thickness /plate = 2.712 mm
Width(less- bolt hole) 35.71 mm Ae(2Plates) = 193.691 mm2
Thickness= 2.712 mm 0.300 in^2
Factored Tensile Resistance= Φ AeFu = **14.64** Kips

Section B-B-5 (Bearing strength of Splice Plates @ Bolt Holes)
At end of design life, carbon steel loss : **0.708** mm/side Remaining Thickness(2plate =) 5.424 mm
Lc(Clear edge =1-1/2"-9/32) 1.22 in
Factored Bearing Resistance at bolt hole=Φ. Rn= Φ(1.2Lc t Fu)<=Φ(2.4 d t Fu) **12.49** Kips

Section C-C (Reinforcing Strip),(Yielding limit state)
At end of design life, carbon steel loss : **0.708** mm/side Remaining Thickness strip= 2.584 mm
Width 50 mm As = 129.200 mm2
Thickness 2.584 mm 0.200 in^2
Factored Tensile Resistance= Φ FyAs = **9.76** Kips

Controling Maximum Tension Load= 9.76 Kips

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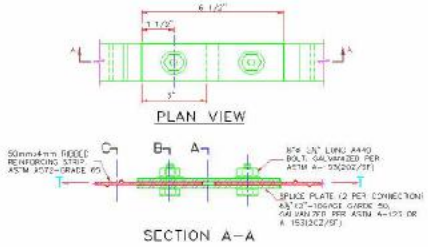
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#8) Double Splice Connection With 10 Gauge Splice Plate Designed for 100 Years Design life

JOB NUMBER:	IDEA
JOB NAME:	IDEA
DESIGNED BY:	DE

Galvanization and Carbon Steel Loss Rates:

Design Life of Structure =	100	yrs
Zinc (first 2 years):	15	µm/yr
Zinc (subsequent years):	4	µm/yr
Carbon Steel:	12	µm/yr
Carbon Steel (75 - 100 yrs)	12	µm/yr



DOUBLE SPLICE PLATE CONNECTION DETAIL

Mechanical Properties of Reinforcement Hardware

	HA Strip	S-Plate
Thickness t (mm)=	4	3.42
Width w (mm)=	50	50
A (mm2) =	200	171
Fu (Ksi) =	80	65
Fy (Ksi) =	65	50

Thickness of Zinc Coat.
Zinc coat tz = 86 µm
Life of Zinc: 16 yrs

Bolt Set.
Diameter 12.7 mm
Fu = 120 Ksi
Bolt Hole= 14.29 mm

Tensile Resistance Resistance Factor for Metallic Reinforcement and connectors= Φ = **0.75**

Section A-A (SPLICE PLATES), 2 plates (Yielding limit state)
At end of design life, carbon steel loss : **1.008** mm/side Remaining Thickness /plate = 1.404 mm
Width 50 mm As (2 plates) = 140.400 mm2
Thickness 1.404 mm 0.218 in^2
Factored Tensile Resistance= Φ FyAs = **8.16** Kips

Section B-B-1 (Reinforcing Strip at Bolt Holes)(Fracture Limit State)
At end of design life, carbon steel loss : **0** mm/side Remaining Thickness /plate = 4 mm
Width(less- bolt hole) 35.71 mm Ae = 142.840 mm2
Thickness= 4 mm 0.221 in^2
Factored Tensile Resistance= Φ AeFu = **13.28** Kips

Section B-B-2 (Bearing strength of strips @ Bolt Holes)
At end of design life, carbon steel loss : **0** mm/side Remaining Thickness /plate = 4 mm
Lc(Clear edge =1-1/2"-9/32) 1.22 in
Factored Bearing Resistance at bolt hole=Φ. Rn= Φ(1.2Lc t Fu)<=Φ(2.4 d t Fu) **11.34** Kips

Section B-B-3 (Shear Resistance of Bolt)
At end of design life, carbon steel loss : **0** mm/side Area of bolt = 0.19635 in2
Factored Shear Resistance =Φ Rn= Φ(0.56 Fu Ab Ns) = **19.79** Kips

Section B-B-4 (SPLICE PLATE ConnectorS@ Bolt Holes)(Fracture Limit State)
At end of design life, carbon steel loss : **1.008** mm/side Remaining Thickness /plate = 2.412 mm
Width(less- bolt hole) 35.71 mm Ae(2Plates) = 172.265 mm2
Thickness= 2.412 mm 0.267 in^2
Factored Tensile Resistance= Φ AeFu = **13.02** Kips

Section B-B-5 (Bearing strength of Splice Plates @ Bolt Holes)
At end of design life, carbon steel loss : **1.008** mm/side Remaining Thickness(2plate =) 4.824 mm
Lc(Clear edge =1-1/2"-9/32) 1.22 in
Factored Bearing Resistance at bolt hole=Φ. Rn= Φ(1.2Lc t Fu)<=Φ(2.4 d t Fu) **11.11** Kips

Section C-C (Reinforcing Strip),(Yielding limit state)
At end of design life, carbon steel loss : **1.008** mm/side Remaining Thickness strip= 1.984 mm
Width 50 mm As = 99.200 mm2
Thickness 1.984 mm 0.154 in^2
Factored Tensile Resistance= Φ FyAs = **7.5** Kips

Controling Maximum Tension Load= 7.50 Kips

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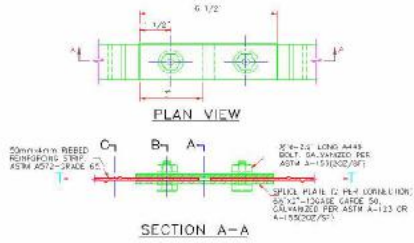
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#9) Double Splice Connection With 7 Gauge Splice Plate Designed for 75 Years Design life

JOB NUMBER:	IDEA
JOB NAME:	IDEA
DESIGNED BY:	DE

Galvanization and Carbon Steel Loss Rates:

Design Life of Structure =	75	yrs
Zinc (first 2 years):	15	µm/yr
Zinc (subsequent years):	4	µm/yr
Carbon Steel:	12	µm/yr
Carbon Steel (75 - 100 yrs)	12	µm/yr



DOUBLE SPLICE PLATE CONNECTION DETAIL

Mechanical Properties of Reinforcement Hardware

	HA Strip	S-Plate
Thickness t (mm)=	6	4.55
Width w (mm)=	50	50
A (mm2) =	300	227.5
Fu (Ksi) =	80	65
Fy (Ksi) =	65	50

Thickness of Zinc Coat.
Zinc coat tz = 86 µm
Life of Zinc: 16 yrs

Bolt Set.
Diameter 12.7 mm
Fu = 120 Ksi
Bolt Hole= 14.29 mm

Tensile Resistance Resistance Factor for Metallic Reinforcement and connectors= Φ = **0.75**

Section A-A (SPLICE PLATES), 2 plates (Yielding limit state)
At end of design life, carbon steel loss : **0.708** mm/side Remaining Thickness /plate = 3.134 mm
Width 50 mm As (2 plates) = 313.400 mm2
Thickness 3.134 mm 0.486 in^2
Factored Tensile Resistance= Φ FyAs = **18.22** Kips

Section B-B-1 (Reinforcing Strip at Bolt Holes)(Fracture Limit State)
At end of design life, carbon steel loss : **0** mm/side Remaining Thickness /plate = 6 mm
Width(less- bolt hole) 35.71 mm Ae = 214.260 mm2
Thickness= 6 mm 0.332 in^2
Factored Tensile Resistance= Φ AeFu = **19.93** Kips

Section B-B-2 (Bearing strength of strips @ Bolt Holes)
At end of design life, carbon steel loss : **0** mm/side Remaining Thickness /plate = 6 mm
Lc(Clear edge =1-1/2"-9/32) 1.22 in
Factored Bearing Resistance at bolt hole=Φ. Rn= Φ(1.2Lc t Fu)<=Φ(2.4 d t Fu) **17.01** Kips

Section B-B-3 (Shear Resistance of Bolt)
At end of design life, carbon steel loss : **0** mm/side Area of bolt = 0.19635 in2
Factored Shear Resistance =Φ Rn= Φ(0.56 Fu Ab Ns) = **19.79** Kips

Section B-B-4 (SPLICE PLATE ConnectorS@ Bolt Holes)(Fracture Limit State)
At end of design life, carbon steel loss : **0.708** mm/side Remaining Thickness /plate = 3.842 mm
Width(less- bolt hole) 35.71 mm Ae(2Plates) = 274.396 mm2
Thickness= 3.842 mm 0.425 in^2
Factored Tensile Resistance= Φ AeFu = **20.73** Kips

Section B-B-5 (Bearing strength of Splice Plates @ Bolt Holes)
At end of design life, carbon steel loss : **0.708** mm/side Remaining Thickness(2plate =) 7.684 mm
Lc(Clear edge =1-1/2"-9/32) 1.22 in
Factored Bearing Resistance at bolt hole=Φ. Rn= Φ(1.2Lc t Fu)<=Φ(2.4 d t Fu) **17.70** Kips

Section C-C (Reinforcing Strip),(Yielding limit state)
At end of design life, carbon steel loss : **0.708** mm/side Remaining Thickness strip= 4.584 mm
Width 50 mm As = 229.200 mm2
Thickness 4.584 mm 0.355 in^2
Factored Tensile Resistance= Φ FyAs = **17.32** Kips

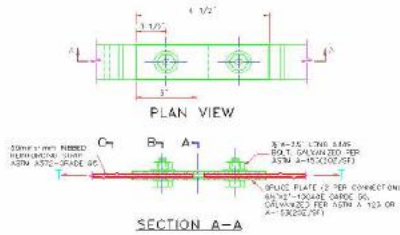
Controling Maximum Tension Load= 17.01 Kips

#10) Double Splice Connection With 7 Gauge Splice Plate Designed for 100 Years Design life

JOB NUMBER:	IDEA
JOB NAME:	IDEA
DESIGNED BY:	DE

Galvanization and Carbon Steel Loss Rates:

Design Life of Structure =	100	yrs
Zinc (first 2 years):	15	µm/yr
Zinc (subsequent years):	4	µm/yr
Carbon Steel:	12	µm/yr
Carbon Steel (75 - 100 yrs)	12	µm/yr



DOUBLE SPLICE PLATE CONNECTION DETAIL

Mechanical Properties of Reinforcement Hardware

	HA Strip	S-Plate
Thickness t (mm)=	6	4.55
Width w (mm)=	50	50
A (mm2) =	300	227.5
Fu (Ksi) =	80	65
Fy (Ksi) =	65	50

Thickness of Zinc Coat.
Zinc coat tz = 86 µm
Life of Zinc: 16 yrs

Bolt Set.
Diameter 12.7 mm
Fu = 120 Ksi
Bolt Hole= 14.29 mm

Tensile Resistance Resistance Factor for Metallic Reinforcement and connectors= Φ = **0.75**

Section A-A (SPLICE PLATES), 2 plates (Yielding limit state)
At end of design life, carbon steel loss : **1.008** mm/side Remaining Thickness /plate = 2.534 mm
Width 50 mm As (2 plates) = 253.400 mm2
Thickness 2.534 mm 0.393 in^2
Factored Tensile Resistance= Φ FyAs = **14.73** Kips

Section B-B-1 (Reinforcing Strip at Bolt Holes)(Fracture Limit State)
At end of design life, carbon steel loss : **0** mm/side Remaining Thickness /plate = 6 mm
Width(less- bolt hole) 35.71 mm Ae = 214.260 mm2
Thickness= 6 mm 0.332 in^2
Factored Tensile Resistance= Φ AeFu = **19.93** Kips

Section B-B-2 (Bearing strength of strips @ Bolt Holes)
At end of design life, carbon steel loss : **0** mm/side Remaining Thickness /plate = 6 mm
Lc(Clear edge =1-1/2"-9/32) 1.22 in
Factored Bearing Resistance at bolt hole=Φ. Rn= Φ(1.2Lc t Fu)<=Φ(2.4 d t Fu) **17.01** Kips

Section B-B-3 (Shear Resistance of Bolt)
At end of design life, carbon steel loss : **0** mm/side Area of bolt = 0.19635 in2
Factored Shear Resistance =Φ Rn= Φ(0.56 Fu Ab Ns) = **19.79** Kips

Section B-B-4 (SPLICE PLATE ConnectorS@ Bolt Holes)(Fracture Limit State)
At end of design life, carbon steel loss : **1.008** mm/side Remaining Thickness /plate = 3.542 mm
Width(less- bolt hole) 35.71 mm Ae(2Plates) = 252.970 mm2
Thickness= 3.542 mm 0.392 in^2
Factored Tensile Resistance= Φ AeFu = **19.12** Kips

Section B-B-5 (Bearing strength of Splice Plates @ Bolt Holes)
At end of design life, carbon steel loss : **1.008** mm/side Remaining Thickness(2plate =) 7.084 mm
Lc(Clear edge =1-1/2"-9/32) 1.22 in
Factored Bearing Resistance at bolt hole=Φ. Rn= Φ(1.2Lc t Fu)<=Φ(2.4 d t Fu) **16.32** Kips

Section C-C (Reinforcing Strip),(Yielding limit state)
At end of design life, carbon steel loss : **1.008** mm/side Remaining Thickness strip= 3.984 mm
Width 50 mm As = 199.200 mm2
Thickness 3.984 mm 0.309 in^2
Factored Tensile Resistance= Φ FyAs = **15.05** Kips

Controling Maximum Tension Load= 14.73 Kips

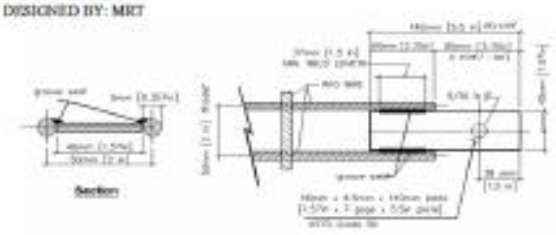
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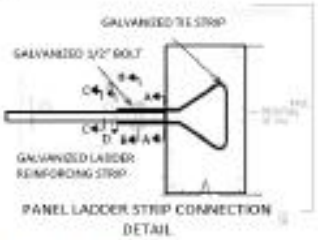
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#11) 10 Gauge Tiestrip - HA Ladder (W10) 75 Years Design Life

REINFORCED EARTH

JOB NUMBER: IDEA
JOB NAME: IDEA
DESIGNED BY: MRT





PANEL LADDER STRIP CONNECTION DETAIL

Galvanization and Carbon Steel Loss Rates:

Design Life of Structure =	**75**	**yrs**
Zinc (first 2 years):	15	µm/yr
Zinc (subsequent years):	4	µm/yr
Carbon Steel:	12	µm/yr
Carbon Steel (75 - 100 yrs)	12	µm/yr

Mechanical Properties of Reinforcement Hardware

	Plate	**Tie Strips**	**Ladder wire W10**	
Thickness t (mm)=	4.5	3.42	9.07	=dia.
Width w (mm)=	40	50		
A (mm^2) =	180	171		
Fu (Ksi) =	65	65	75	
Fy (Ksi) =	50	50	65	

Thickness of Zinc Coat.

			Bolt Sz.			
Zinc coat t =	86	µm	ϕ =	12.7	mm	= 0.50 in.
Life of Zinc:	16	yrs	Fu =	120	Ksi	

TENSILE RESISTANCE OF HIGH ADHERENCE LADDER REINFORCING STRIP & CONNECTORS, LRFD METHOD
Tensile Resistance Resistance Factor for Metallic Reinforcement and connectors= Φ = 0.75

Section A-A (Tie Strip Only), 2 Tie Strip plates
At end of design life, carbon steel loss : 0.708mm/side, Remaining Thickness (1 plate) = 3.420- 2 x 0.708 =2.004mm
As (2 plates) = 2.004in 2 x 50 =200.400mm2 or 0.311in2, Factored Tensile Resistance= Φ FyAs =11.65Kips

Section B-B (Tie Strip at Bolt Holes), 2 tie strip plates with 14.3mm bolt holes,
At end of design life, carbon steel loss : 0.708mm/side, Remaining Thickness (1 plate, corrosion on 1 side) = 3.420 - 0.708 =2.712mm
A (2 plates) = 2.712 x 2 x [50 - 14.3] = 193.637mm2 or 0.300in2
Factored Tensile Resistance=Φ Fu As =14.63Kips
Factored Bearing Resistance at bolt hole=Φ . Rn=Φ(2.4 d t Fu) =12.49Kips



Section B-B (Plate at Bolt Holes)
At end of design life, carbon steel loss : 0mm/side, Thickness = 4.50mm
As = 3.08 x 40.000=123.36mm2 or 0.191 in2
Factored Tensile Resistance= Φ. Fu As =8.74Kips
Factored Bearing Resistance at bolt hole= Φ. Rn= Φ(2.4 d t Fu) =10.36Kips

Section D-D (Plate Location Beyond Tie strip)
At end of design life, carbon steel loss : 0.708mm/side, Plate Remaining Thickness = 4.500- 2 x 0.708 =3.084mm
As = 3.08 x [40.000]=123.36mm2 or 0.19
Factored Tensile Resistance= Φ. Fy As =7.17Kips

Section B-B (Shear Resistance of Bolt)
At end of design life, carbon steel loss : 0mm/side, Area of Bolt = 126.68mm2= 0.196in2
Factored Shear Resistance =Φ Rn= Φ(0.56 Fu Ab Ns) =19.79Kips

Section C-C (Ladder Wire)
At end of design life, carbon steel loss : 0.708, Remaining Diameter = 9.068- 2 x 0.708 =7.652mm
As (2 wires) = 2 (7.652)^2\4 . =91.970mm2 or 0.143in2
Factored Tensile Resistance= Φ . Fy. As =6.95Kips

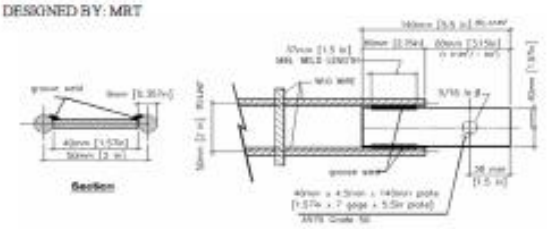
Shear Strength of groove welds Weld length = 1.50in Fexx = 67.00 Φw1 = 0.85
Weld size = a = 0.250in Fu = 79.20
At end of design life, carbon steel loss : 0.708mm
Remaining weld size = a_r = 0.222in
Factored Shear Resistance = $(0.60\,\Phi_{w1}\,F_{exx}\,.\,L\,.\,a_{r\,/\,2\ welds})$ 22.77Kip

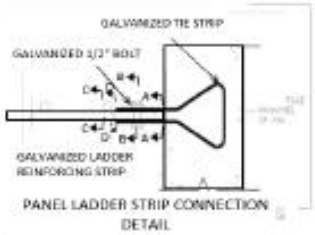
RESULTS:
Section A-A (Tie Strip)= 11.65Kips
Section B-B (TS at bolt hole), a) Factored Tensile Resistance= 14.63Kips
b) Factored Bearing Resistance @ bolt hole= 12.49Kips
Section B-B (Plate at bolt hole), a) Factored Tensile Resistance= 8.74Kips
b) Factored Bearing Resistance @ bolt hole= 10.36Kips
Section B-B (Bolt)= 19.79KipsKips
Section C-C (Ladder Reinf. Strip)= 6.95Kips, CONTROLS
Shear Strength of groove welds =22.77kips
Section D -D (Plate Beyond TS), a) Factored Tensile Resistance= 7.17Kips

#12) 10 Gauge Tiestrip - HA Ladder (W10) 100 Years Design Life

REINFORCED EARTH

JOB NUMBER: IDEA
JOB NAME: IDEA
DESIGNED BY: MRT





PANEL LADDER STRIP CONNECTION DETAIL

Galvanization and Carbon Steel Loss Rates:

Design Life of Structure =	100	yrs
Zinc (first 2 years):	15	µm/yr
Zinc (subsequent years):	4	µm/yr
Carbon Steel:	12	µm/yr
Carbon Steel (75 - 100 yrs)	12	µm/yr

Mechanical Properties of Reinforcement Hardware

	Plate	Tie Strips	Ladder wire W10	
Thickness t (mm)=	4.5	3.42	9.07	=dia.
Width w (mm)=	40	50		
A (mm^2) =	180	171		
Fu (Ksi)=	65	65	75	
Fy (Ksi)=	50	50	65	

Thickness of Zinc Coat.

Zinc coat t = 86 µm

Life of Zinc: 16 yrs

Bolt Set

Φ = 12.7 mm = 0.50 in.

Fu = 120 Ksi

TENSILE RESISTANCE OF HIGH ADHERENCE LADDER REINFORCING STRIP & CONNECTORS, LRFD METHOD

Tensile Resistance Resistance Factor for Metallic Reinforcement and connectors= Φ = 0.75

Section A-A (Tie Strip Only), 2 Tie Strip plates

At end of design life, carbon steel loss : 1.008mm/side, Remaining Thickness (1 plate) = 3.420- 2 x 1.008 =1.404mm

As (2 plates) = 1.404x 2 x 50 =140.400mm2 or 0.218in2, Factored Tensile Resistance= Φ FyAs =8.16Kips

Section B-B (Tie Strip at Bolt Holes), 2 tie strip plates with 14.3mm bolt holes,

At end of design life, carbon steel loss : 1.008mm/side, Remaining Thickness (1 plate, corrosion on 1 side) = 3.420 - 1.008 =2.412mm

A (2 plates) = 2.412 x 2 x [50 - 14.3] = 172.217mm2 or 0.267in2

Factored Tensile Resistance=Φ Fu As =13.01Kips

Factored Bearing Resistance at bolt hole=Φ . Rn=Φ(2.4 d t Fu) =11.11Kips



Section B-B (Plate at Bolt Holes)

At end of design life, carbon steel loss : 0mm/side, Thickness = 4.50mm

As = 2.48 x 40.000=99.36mm2 or 0.154 in2

Factored Tensile Resistance= Φ. Fu As =8.74Kips

Factored Bearing Resistance at bolt hole= Φ. Rn= Φ(2.4 d t Fu) =10.36Kips

Section D-D (Plate Location Beyond Tie strip)

At end of design life, carbon steel loss : 1.008mm/side, Plate Remaining Thickness = 4.500- 2 x 1.008 =2.484mm

As = 2.48 x (40.000)=99.36mm2 or 0.15

Factored Tensile Resistance= Φ. Fy As =5.78Kips

Section B-B (Shear Resistance of Bolt)

At end of design life, carbon steel loss : 0mm/side, Area of Bolt = 126.68mm2= 0.1963in2

Factored Shear Resistance =Φ Rn= Φ(0.56 Fu Ab Ns) =19.79Kips

Section C-C (Ladder Wire)

At end of design life, carbon steel loss : 1.008, Remaining Diameter = 9.068- 2 x 1.008 =7.052mm

As (2 wires) = 2 (7.052)^2/4 . π=78.112mm2 or 0.121in2

Factored Tensile Resistance= Φ. Fy. As =5.90Kips

Shear Strength of groove welds Weld length = 1.50in Fexx = 67.00 Φe1 = 0.85

Weld size = a = 0.250in Fu = 79.20

At end of design life, carbon steel loss : 1.008mm

Remaining weld size = a_r = 0.2103in

Factored Shear Resistance = ($0.60 \Phi_{e1} F_{exx} \cdot L \cdot a_{r\,[2\ welds]}$) 21.56Kip

RESULTS:

Section A-A (Tie Strip)= 8.16Kips

Section B-B (TS at bolt hole), a) Factored Tensile Resistance= 13.01Kips

b) Factored Bearing Resistance @ bolt hole= 11.11Kips

Section B-B (Plate at bolt hole), a) Factored Tensile Resistance= 8.74Kips

b) Factored Bearing Resistance @ bolt hole= 10.36Kips

Section B-B (Bolt)= 19.79KipsKips

Section C-C (Ladder Reinf. Strip)= 5.90Kips

Shear Strength of groove welds =21.56kips

Section D -D (Plate Beyond TS), a) Factored Tensile Resistance= 5.78Kips, CONTROLS

A1.2.10 Connection Device Physical Property Specifications

This section is intentionally left blank.

A1.2.11 Connection Device Corrosion Protection

This section is intentionally left blank.

A1.2.12 Connection Device Sacrificial Steel Thickness

This section is intentionally left blank.

A1.2.13 Connection Device Corrosion Test Results

Of the numerous soil reinforcement samples that have been extracted from existing structures and tested, the following is one example:

- Brookwood Interchange (I-75 @ I-85 near the 17th Street Bridge), Atlanta, GA
- RECo project # 846
- Original construction - 1982
- Reinforcing strips exhumed during demolition for widening project – May 2008
- Reinforcing strips in service for 26 years
- Connection with greatest corrosion analyzed



Inside of Tie Strip Plates

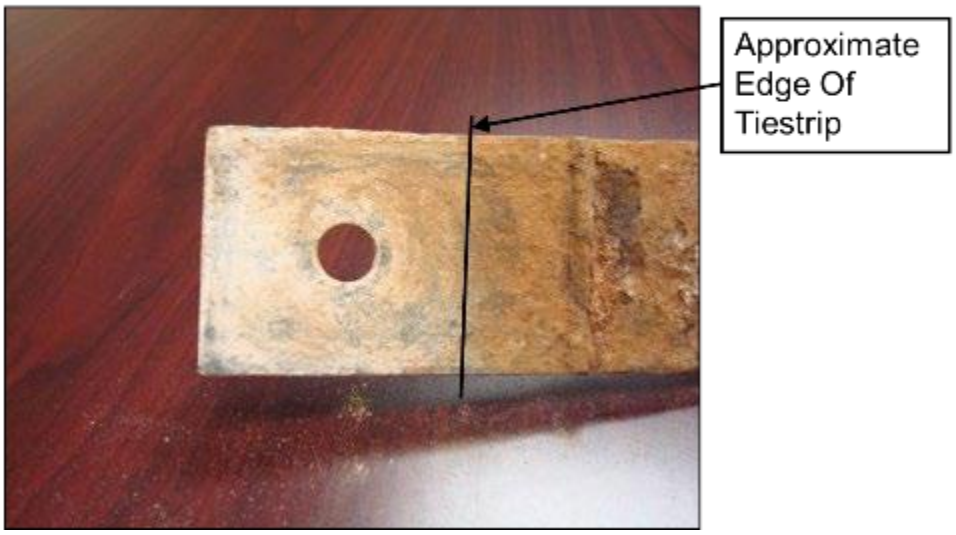
Top Plate



Bottom Plate



Reinforcing Strip Inside Connection



Note that the remaining zinc inside the connection versus the corroded strip outside connection. Sandwiching by tie strip plates has protected the reinforcing strip at the connection.

Bolt Shank is Also Protected



Note that there is no corrosion of the bolt shank sandwiched inside the connection.

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A1.2.14 Connection Device Dimensional Tolerances

Connection device dimensional tolerances can be found in **Appendix A1.2.9**.

A1.2.15 Facing Unit Connection Strength Test Results

The following documentation is included in this section:

- Report by ICC-ES, 2020 (6 pages)
- Report by STI, 2020 (33 pages)
- Report by SGI, 2021 (37 pages)
- Report by AEL, 1990 (10 pages)

ICC EVALUATION SERVICE

ICC-ES Evaluation Report

ESR-4391

Issued September 2020

This report is subject to renewal September 2021.

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DIVISION: 03 00 00—CONCRETE
Section: 03 15 19—Cast-in Concrete Anchors
Section: 03 16 00—Concrete Anchors

REPORT HOLDER:

THE REINFORCED EARTH COMPANY

EVALUATION SUBJECT:

STEEL TIE STRIP ANCHOR AND REINFORCING STRIP

1.0 EVALUATION SCOPE

Compliance with the following codes:

- 2018, 2015, 2012 and 2009 *International Building Code®* (IBC)

For evaluation for compliance with codes adopted by Los Angeles Department of Building and Safety (LADBS), see ESR-4391 LABC Supplement.

Property evaluated:

Structural

2.0 USES

The steel tie strip anchor is used as a cast-in place anchor in precast concrete panels and in conjunction with steel reinforcing strips to resist lateral soil pressures in mechanically stabilized earth retaining wall applications. The steel tie strip anchor is used to resist static and seismic (Seismic Design Categories A through F) tension loads in cracked and uncracked normal-weight concrete panels having a minimum compressive strength, f'_c, of 5,000 psi (34.4 MPa).

The steel tie strip anchor and reinforcing strip are an alternative to cast-in anchors described in Section 1901.3 of the 2018 and 2015 IBC, and Sections 1908 and 1909 of the 2012 and Sections 1911 and 1912 of the 2009 IBC.

3.0 DESCRIPTION

3.1 Steel Tie Strip Anchor:

The steel tie strip anchor (Anchors) consists of a bent shape steel plate as shown in Figure 1. The Anchors are made from No. 10 gage [0.135-inch-thick (3.42 mm)], ASTM A1011, SS Grade 50 or HSLAS Grade 50 Class 1 steel, having a minimum yield strength of 50 ksi (345 MPa) and ultimate tensile strength of 65 ksi (448 MPa). The Anchors are hot-dipped galvanized in accordance with ASTM A123 with minimum Grade 85. One end of the anchor comes with a predrilled hole used to receive a ½-inch-diameter bolt for steel reinforcing strips attachment as described in Section 3.2 of this report. The Anchors are cast-in place by the precast concrete panel fabricators approved by the Reinforced Earth Company. See Figure 1 for typical description of Anchors.

3.2 Steel Reinforcing Strips:

The steel reinforcing strips (Strips) consist of a ribbed steel plate as shown in Figure 2. The Strips are made from 0.157-inch (4mm) thick steel complying with ASTM A572, Grade 65 steel, having a minimum yield strength of 65 ksi (448 MPa) and ultimate tensile strength of 80 ksi (552 MPa). The Strips are hot-dipped galvanized in accordance with ASTM A123 with a minimum Grade 85. The Strips are connected in the field to the Anchors that are already embedded in the precast concrete panels using a ½-inch-diameter hex head bolt complying Section 3.4 of this report.

3.3 Precast Concrete Panels:

The precast concrete panels must be normal-weight concrete conforming to Sections 1903 and 1905 of the IBC and having a minimum compressive strength, f'_c, of 5,000 psi (34.4 MPa). The minimum precast concrete panel thickness must be 5½ inches (140 mm). The precast concrete panels must include reinforcement as shown in Table 1 and Figure 4 of this report. The location of the steel tie strip anchors on the precast concrete panels must comply with Table 1 and Figure 4 of this report. The precast concrete panels must be cast by fabricators approved by the Reinforced Earth Company. The design of the precast concrete panels is outside the scope of this report.

3.4 Bolts:

The bolts are supplied by others and must comply with ASTM F3125, Grade A325, Type 1 or 3 or ASTM A449 Type 1 or 3 with matching hex nuts complying with ASTM A563. The bolts must be hot-dipped galvanized to ASTM A153.

4.0 DESIGN AND INSTALLATION

4.1 Design:

The strength design of the Strips connected to the Anchors embedded in precast concrete panels is based on the lowest of the values determined from Section 4.2 and 4.3 of this report.

4.2 Anchors Strength Design:

4.2.1 General: Design strength of cast-in place Anchors as an alternative to anchors complying with the IBC must be determined in accordance with the provisions of this report. Design information of Anchors is provided in Table 2 for load

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combinations calculated in accordance with Section 1605.2 of the IBC and Section 5.3 of ACI 318-14, as applicable.

4.2.2 The strength design value of the Anchors in precast concrete panels have been calculated in Table 3 of this report based on the installation information in Table 1 and the design information in Table 2.

4.3 Strips Strength Design:

The strength design of the Strips provided in Table 4 is for load combinations in accordance with IBC Section 1605.2.

4.4 Installation:

The Anchors and Strips must be installed in accordance with this report, the manufacturer's published installation instruction and the approved plans. Where there is a conflict between these documents, the most restrictive governs. The Strips must be connected in the field to the Anchors using a ½-inch-diameter hex head bolt with matching nut complying with Section 3.4 of this report as shown in Figure 3.

4.5 Special Inspection:

Special inspection must be determined by the registered design professional in accordance with Chapter 17 of the IBC.

5.0 CONDITIONS OF USE

The Anchors and Strips described in this report comply with the codes listed in Section 1.0 of this report, subject to the following conditions:

5.1 Anchors and Strips must be installed in accordance with the manufacturer's printed installation instructions and this report. In case of conflict, this report governs.

5.2 Anchors must be limited to use in cracked and uncracked normal-weight precast concrete panels having a minimum compressive strength, f'_c, of 5,000 psi (34.4 MPa).

5.3 Strength design and allowable strength design values of the Anchors and Strips must be established in accordance with Section 4.1 of this report.

5.4 Anchors may be used to resist seismic forces in locations designated as Seismic Design Categories A, B, C, D, E and F, subject to the conditions of this report.

5.5 Calculations and details, justifying the use of the products is in accordance with the applicable code and this evaluation report, must be submitted to the code official for approval. The calculations and details must be prepared by a registered design professional where required by the statutes of the jurisdiction which the project is to be constructed.

5.6 The design and manufacturing of the precast concrete panels is outside the scope of this report.

5.7 The design of the mechanically stabilized earth walls as a retaining wall system and the soil capacity interaction with the Strips are outside the scope of this report.

5.8 Special inspection must be provided in accordance with Section 4.5.

5.9 Anchors and Strips are manufactured under an approved quality control program with inspections by ICC-ES.

6.0 EVIDENCE SUBMITTED

6.1 Tension tests in cracked and uncracked concrete in accordance with ASTM E488.

6.2 Calculations of the steel components in accordance with the AISC 360.

6.3 Quality control documentation.

7.0 IDENTIFICATION

7.1 The steel tie strip anchors and steel reinforcing strips are identified by packaging labeled with the manufacturer's name (The Reinforced Earth Company) and evaluation report number (ESR-4391).

7.2 The report holder's contact information is the following:

THE REINFORCED EARTH COMPANY
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STERLING, VIRGINIA 20166
(800) 446-5700
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TABLE 1—STEEL TIE STRIP ANCHOR PRECAST CONCRETE PANEL INSTALLATION INFORMATION

Minimum Concrete member thickness (inches)	Minimum concrete compressive strength (psi)	Minimum Reinforcement (See Figure 4 for details)	Overall Height of Steel Tie Strip Anchor (inches)	Effective Embedment (inches)	Minimum steel tie strip anchor clear edge distance (inches)	Minimum steel tie strip anchor spacing (inches)
5.5	5,000	No. 4 ASTM A615, Grade 60	7.5	4	8	8.5

For **SI:** 1 inch= 25.4 mm; 1 psi = 6.89 kPa.

TABLE 2—TIE STRIP ANCHOR DESIGN INFORMATION [1,2,3,4]

Characteristic	Symbol	Units	Value
Effective Steel Stress Area	A_{se}	in^2	0.394
Minimum Specified Ultimate Steel Strength	f_{ut}	psi	65,000
Steel Strength in Tension	N_{sa}	lbf	25,610
Strength Reduction Factor for Steel Failure Modes, Tension	ϕ_s	-	0.70
Nominal Embedment Depth	h_{nom}	in.	4
Effective Embedment Depth	h_{ef}	in.	3.86
Pullout Strength in Cracked or Uncracked Concrete	N_p	lbf	13,662
Pullout Strength, Seismic	N_{eq}	lbf	13,662
Strength Reduction Factor for Concrete Breakout and Pullout	ϕ_c	-	0.70

For **SI:** 1 in=25.4 mm, 1 in^2= 645 mm^2, 1 psi= 6.894 kPa, 1 lbf= 4.45 N.

[1]The tabulated data is to be used in conjunction with the design criteria given in ACI 318-14. The strength design values of the Anchors in precast concrete panels have been calculated in Table 3 below using the Table 2 values.
[2]All values of ϕ were determined from the load combinations of IBC Section 1605.2 or Section 5.3 of ACI 318-14, as applicable.
[3] The tie strip is a ductile element.
[4] Minimum compressive strength is 5,000 psi.

TABLE 3—STRENGTH DESIGN OF TIE STRIP ANCHOR

Tie Strip Anchor in Concrete	Strength Design in Tension, lbf
Steel Strength of Anchor in Tension	17,297
Pull-out Strength in Cracked or Uncracked Concrete, Condition B	9,563
Pull-out Strength, Seismic	9,563
Governing Strength	9,563

For **SI:** 1 lbf = 4.45 N

TABLE 4—STRENGTH DESIGN OF STEEL REINFORCING STRIP CONNECTION TO TIE STRIP[1,2,3]

Steel Component	Nominal Steel Thickness, inch	Corroded Steel Thickness, inch	Corroded Steel Area, in^2	Steel Yield Strength, psi	Nominal Strength, lbf	Strength Reduction Factor for Steel Connection	Strength Design, lbf
Tie Strip Anchor	0.135	0.079	0.31	50,000	15,500	0.75	11,625
Reinforcing Strip	0.157	0.101	0.20	65,000	13,000	0.75	9,750
For **SI:** 1-inch= 25.4 mm, 1 in2=645 mm2, 1 psi= 6.894 kPa, 1 lbf=4.45 N.						Governing Strength	9,750

[1]The steel tie strip anchor and connection to the reinforcing strip steel strength design is based on AISC 360 provisions for LRFD assuming corrosion thickness losses of 0.056-inch (1.416 mm).
[2]Installation must comply with Section 4.4 of this report.
[3]The bolt and nut must comply with Section 3.4 of this report.

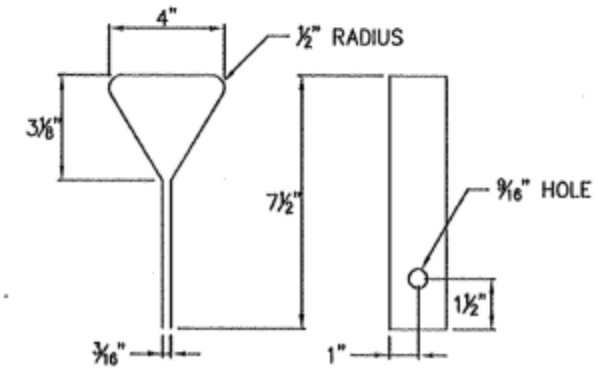


FIGURE 1—STEEL TIE STRIP ANCHOR

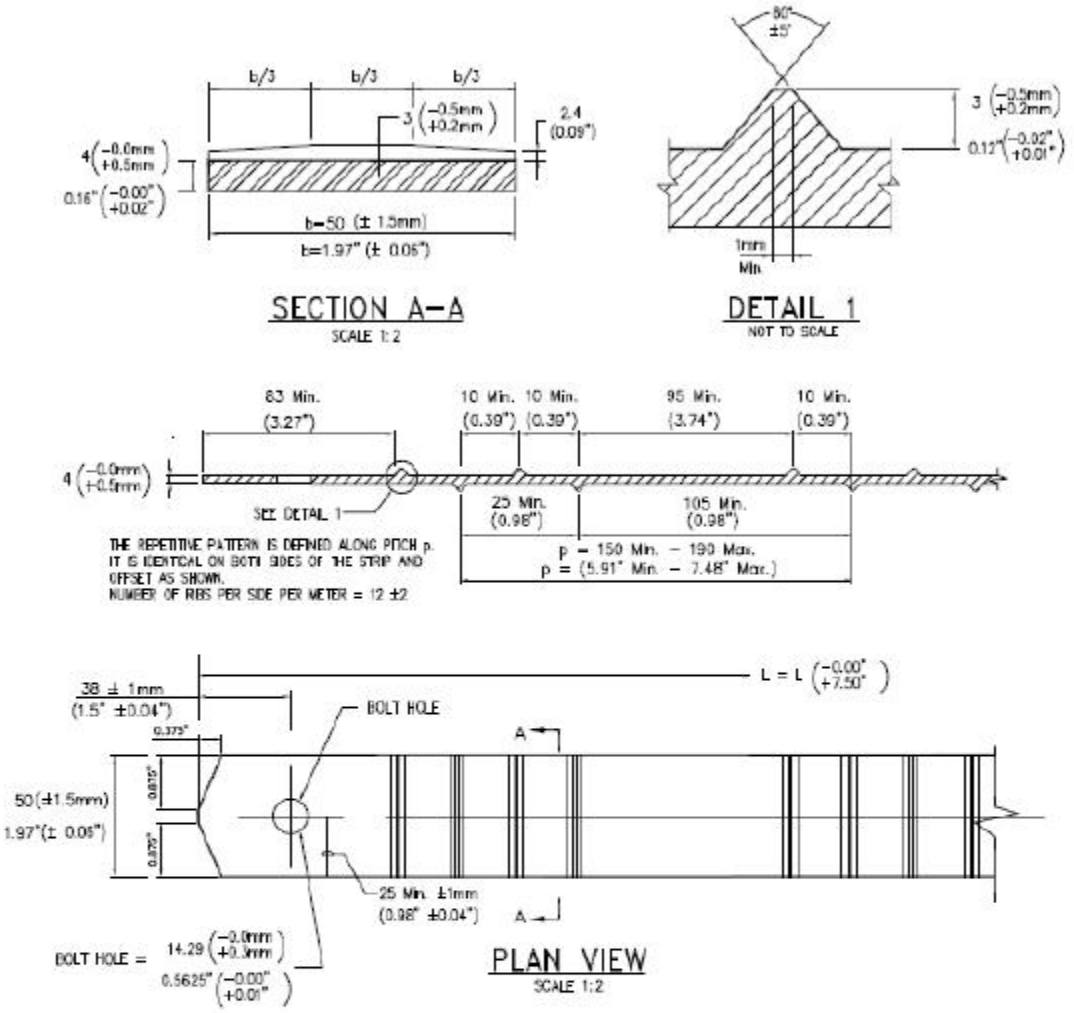


FIGURE 2—STEEL REINFORCING STRIP

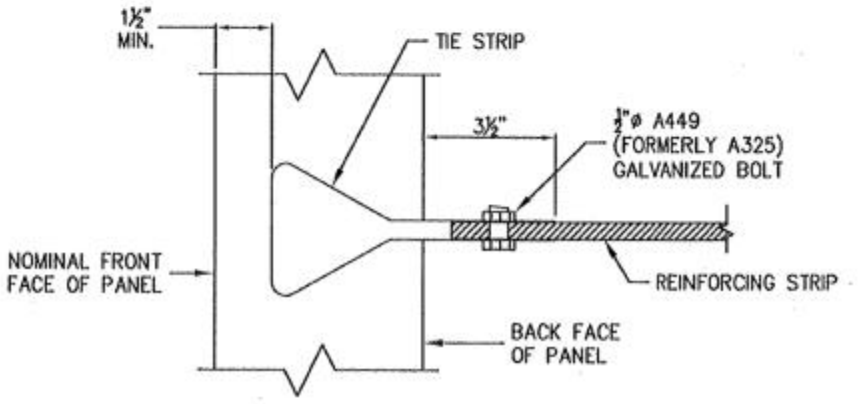
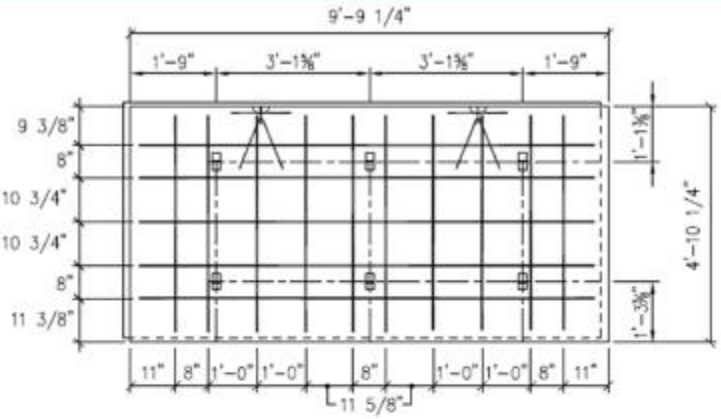
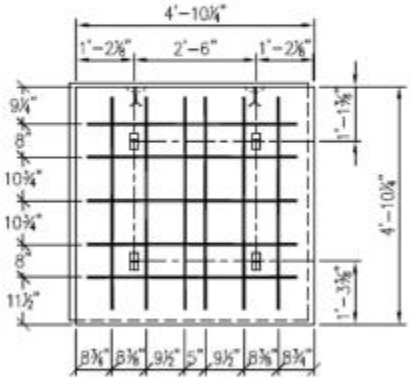


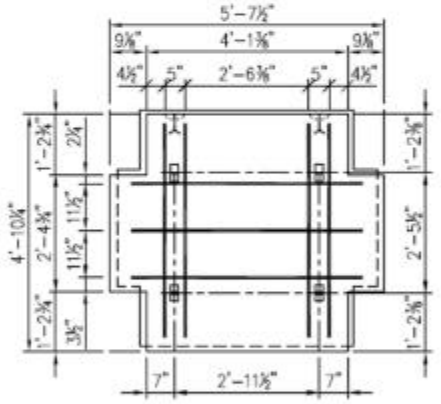
FIGURE 3—TYPICAL INSTALLATION DETAIL OF STEEL TIE STRIP ANCHOR TO STEEL REINFORCING STRIP



STD RLA PANEL A-6



STD SQR PANEL A-4



STD REW PANEL A-4

NOTES:
1. A STANDARD "A" PANEL IS SHOWN IN THESE DETAILS. ACTUAL PANEL SIZES VARY DEPENDING ON THE WALL GEOMETRY INCLUDING THE REQUIRED NUMBER OF TIE STRIPS.
2. ALL REBAR SHOWN IS #4.
3. WIRE MESH MAY BE SUBSTITUTED FOR THE REBAR SHOWN GIVEN AN EQUIVALENT AREA OF STEEL IS PROVIDED IN BOTH DIRECTIONS.

FIGURE 4—TYPICAL REINFORCEMENT DETAILS OF PRECAST CONCRETE PANELS

DIVISION: 03 00 00—CONCRETE
Section: 03 15 19—Cast-in Concrete Anchors
Section: 03 16 00—Concrete Anchors

REPORT HOLDER:

THE REINFORCED EARTH COMPANY

EVALUATION SUBJECT:

STEEL TIE STRIP ANCHOR AND REINFORCING STRIP

1.0 REPORT PURPOSE AND SCOPE

Purpose:

The purpose of this evaluation report supplement is to indicate that the steel tie strip anchor and reinforcing strip, described in ICC-ES evaluation report ESR-4391, have also been evaluated for compliance with the code noted below as adopted by the Los Angeles Department of Building and Safety (LADBS).

Applicable code editions:

- 2020 *City of Los Angeles Building Code* (LABC)

2.0 CONCLUSIONS

The steel tie strip anchor and reinforcing strip, described in Sections 2.0 through 7.0 of the evaluation report ESR-4391, comply with the LABC Chapter 19 and 22, and are subject to the conditions of use described in this supplement.

3.0 CONDITIONS OF USE

The steel tie strip anchor and reinforcing strip described in this evaluation report supplement must comply with all of the following conditions:

- All applicable sections in the evaluation report ESR-4391.
- The design, installation, conditions of use and identification of the steel tie strip anchor and reinforcing strip are in accordance with the 2018 *International Building Code®* (IBC) provisions noted in the evaluation report ESR-4391.
- The design, installation and inspection are in accordance with additional requirements of LABC Chapters 16 and 17, as applicable.

This supplement expires concurrently with the evaluation report, issued September 2020.

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Report No: STQA50757.1R2
Date: July 21, 2020
Page No: 1 of 26

Testing of Reinforced Earth Tie Strip Anchor

Prepared by:

Thomas A. Kolden, P.E.
Specialized Testing, Inc.

LICENSED PROFESSIONAL ENGINEER THOMAS A. KOLDEN C 61572 CIVIL STATE OF CALIFORNIA

Digitally signed by Thomas A. Kolden
Date: 2020.07.21 07:38:14 -05'00'

Rev. No.	Date	Description
R0	31 March 2020	Original Report
R1	16 June 2020	Added Min edge/spacing test plus allowable stress design information
R2	21 July 2020	Minor revisions to assessment on page 11 plus add final design table on page 11

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INTRODUCTION:

The objective of the test program was to determine the axial pull out capacity of the Reinforced Earth Company Tie Strip anchor for evaluation by ICC Evaluation Service, LLC. The test program was performed in general accordance with ASTM E488-18 *Standard Test Methods Strength of Anchors in Concrete Elements* and other criteria. The test project was undertaken under contract with the Reinforced Earth Company under contracts signed on 17 May 2019, 11 June 2019 and 2 December 2019.

Specialized Testing, Inc. (STI), dba Specialized Testing, was the laboratory of record for this test program. Specialized Testing, Inc. is accredited under ISO 17025 by the International Accreditation Service (IAS) as listed on IAS TL-228. STI is also accredited by the Miami-Dade Product Control Section as listed in Certification No. 19-0620.01 and the City of Los Angeles, Approval No. TA10155. All testing was performed at the laboratory facilities of Specialized Testing located in Santa Fe Springs, CA, except where noted otherwise.

The Reinforced Earth Company designs pre-cast concrete panels that are used in a type of mechanically stabilized earth retaining wall. As part of the overall design of the earth retaining structures, the concrete panels are attached to High Adherence Reinforcing Strips, High Adherence Reinforcing Ladders or Welded Bar Bats, whose function is to reinforce the backfill retained behind the panel structure and to secure the panel facing to that earthen structure. The connection of the concrete panels to the earth reinforcing component is accomplished through the Tie Strip anchors cast-in-place in the concrete panels. The Reinforced Earth Company system is described in their product brochure and on their website: www.reinforcedearth.com. A photograph of a tie strip specimen is shown in Photograph 1, below. A drawing of the device is also shown in Figure 1, next page.



Photo 1: Tie Strip Anchor

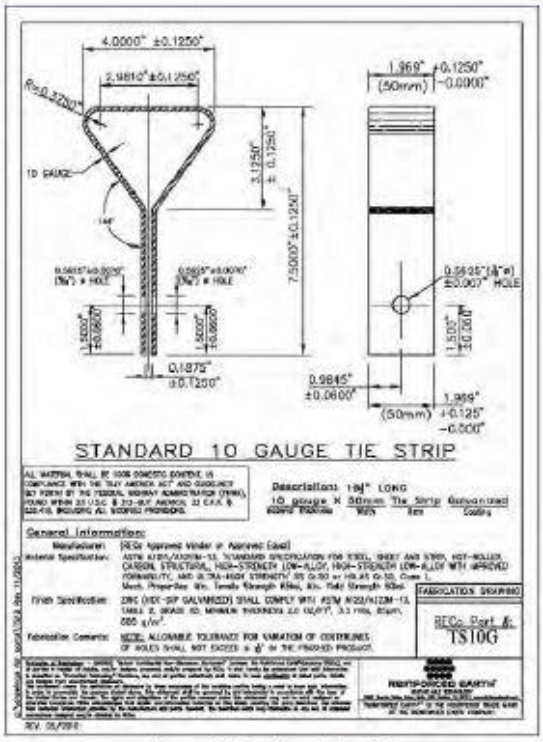


Figure 1: Drawing of Tie Strip

The essential purpose of the test program reported herein was to test the Tie Strip as a cast-in-place anchor, or insert, to determine the strength of the anchor-connection to the concrete. The test protocol was described in a Specialized Testing document dated June 6, 2019 that was presented to Mr. Yamil Moya, P.E., of ICC Evaluation Service, Inc. (ICC-ES) for prior review and approval for ICC-ES File 19-01-13 related to the application for an evaluation report by The Reinforced Earth Company. The test plan was prepared to address the requirements of ICC-ES Test Plan sent to Mr. Paul Frankenberger of The Reinforced Earth Company via email from Mr. Vince Yang, dated April 10, 2019, which referenced ICC-ES *Acceptance Criteria for Cast-In Assemblies in Concrete* (AC502) as the basis for testing of the Tie Strip device.

The project as conducted was intended to conform to the approved test plan. The project was initiated on May 17, 2019, with work commencing shortly thereafter. However, the results of the initial testing were deemed less than desired, and in consequence, some aspects of the test plan were redefined. The same tests were to be conducted, but the concrete member and embedment depth of the anchor were changed. The changes were outlined in a revised test plan dated November 8, 2019, which was submitted to ICC-ES for review. The testing described hereafter was in general conformance to the revised test plan. A copy of the test plan is included on pages 24 to 26 of this report.

PROJECT SCOPE:

As outlined in the test plan, four sets of five Strip Tie anchors were cast in concrete. The first set was for static pullout testing of a single anchor in un-cracked concrete; the second set for static pullout testing of a single anchor in cracked concrete; the third set for seismic tension testing of a single anchor in cracked concrete; and the final set of static tension testing of two anchors at minimum spacing in un-cracked concrete. For the tests of a single anchor, the tie strip anchor was cast in the approximate center of a concrete member. For the tests of two anchors, the tie strips were installed along the centerline of the concrete member and at minimum spacing. The nominal embedment of the anchor was 4 inches in all cases. All concrete members included No. 4 rebar reinforcing. Drawings of the concrete members and layout of rebar and anchor locations are presented on pages 15 to 18 of this report. The concrete thickness was 5 1/2 inches for all tests.

SAMPLING:

Representatives of Specialized Testing, Inc. traveled to Pro-Cast Products, Inc. on several occasions to sample the strip tie anchor specimens, witness the casting of the concrete and prepare compressive strength cylinders. Pro-Cast Products, Inc. is located at 27417 East 3rd Street, Highland, CA 92346. The precast company is licensed by the Reinforced Earth Company to fabricate the concrete panels, and therefore maintains an inventory of the Tie Strips for that purpose.

Anchor specimens were sampled on or about 15 November 2019, 14 January 2020, and 13 April 2020. Approximately twenty specimens were sampled each time and marked. There were several cartons of the anchors available containing over one hundred specimens. The samples were subsequently cast into concrete on 25 November 2019, 17 January 2020, and 13 April 2020, and was witnessed by STI each time. The concrete members were delivered to the laboratory facility on or about 2 December 2019, 29 January 2020, and 24 April 2020. Some of the sampled anchors, which were taken from the same carton, were transported back to the laboratory for further examination and testing.

SAMPLE IDENTIFICATION:

The samples were examined and found to be in general conformance to the product drawing (see Figure 1, page 3 of this report). Three of the specimens were randomly selected from the sampled quantity and submitted for material testing to determine the steel thickness and mechanical properties. This work was subcontracted to Atlas Testing Laboratory of Rancho Cucamonga, CA. The results are summarized in Table 1, below. The material specification is ASTM A1011, which has a minimum specified yield strength of 50,000 psi and minimum tensile strength of 65,000 psi. Three additional specimens were measured for various dimensions and the results listed in Table 2

Table 1: Tie Strip Anchor Material Properties

Specimen	Thickness (in.)	Yield (2% offset) (psi)	Ultimate (psi)	Elongation (%)	ROA (%)
1	0.139	69,100	81,700	30	70
2	0.142	67,100	80,200	31	71
3	0.142	66,000	77,900	30	71
Mean	0.141	67,400	79,900	30	71

Notes to Table 1:
1. Reference Atlas Testing Laboratory report presented on page 23 of this report

Table 2: Tie Strip Anchor Dimensions

Specimen Number	H_{Anchor} (in.)	W_{strap} (in.)	W_{Anchor} (in.)	H_{Bend} (in.)	End (in.)	Dia. (in.)	Gap (in.)
1	7.52	1.97	4.00	3.15	1.47	0.56	0.19
2	7.50	1.94	4.00	3.25	1.50	0.55	0.20
3	7.53	1.94	3.94	3.16	1.53	0.55	0.24
Ave.	7.52	1.95	3.98	3.18	1.50	0.55	0.21
Specified	7.500	1.969	4.000	3.125	1.500	0.563	0.188
+Tolerance	0.125	0.125	0.125	0.125	0.060	0.007	0.125
-Tolerance	0.125	0.000	0.125	0.125	0.060	0.007	0.125
Satisfactory	okay	n.g.	okay	okay	okay	n.g.	okay

CONCRETE:

The anchors were cast-in-place in concrete as set up and completed by Pro-Cast Products, Inc. The Pro-Cast concrete mix design is shown in Table 3, below. The aggregate gradation is shown in Table 4, below. The compressive strength of the concrete at the time of anchor testing was derived based on testing of cylinders at intervals. The data for the two cast dates are shown in Table 5, below, and Table 6 on the next page.

Table 3: Pro-Cast Products, Inc. Concrete Mix Design

Material Component	Miscellaneous Units	Batch Weight (lb)	S.G.	Volume (ft^3)
Cement		506	3.15	2.57
Fly Ash		169	2.33	1.16
Sand		1303	2.62	7.97
1 1/2" x 3/4"		-	2.64	0.00
1" x #4		1373	2.64	8.34
3/8" x #8		296	2.62	1.81
Water	36.0 Gal.	299.9	1.00	4.81
Entrapped Air	1.25%	-		0.34
Totals	-	3948	-	27.00

Notes to Table 3:
1. Cement: Type II Portland
2. Fly Ash: Class F, from Boral Resources
3. Sand and Aggregate: Robertson's Redlands
4. Additional Admixtures: 3.0 oz./cwt of WRDA 64 and 1-15 oz./cwt of ADVA 198
5. Additional information pertaining to the mix design is presented on page 16 of this report.

Table 4: Pro-Cast Products, Inc. Aggregate Gradation

Size	%	Aggregate Gradations											
		2"	1 1/2"	1"	3/4"	3/8"	#4	#8	#16	#30	#50	#100	#200
WCS	44	100	100	100	100	100	99	80	60	39	18	7	2
1 1/2"	0	100	99	30	6	1	1	0	0	0	0	0	0
1"	46	100	100	95	70	12	1	0	0	0	0	0	0
3/8"	10	100	100	100	100	96	18	2	1	1	0	0	0
Blend	-	100	100	96	75	27	4	0	0	0	0	0	0
Combined	100	100	100	98	86	59	46	35	27	17	8	3	1

Notes to Table 4:
1. WCS and aggregate source is Robertson's Redlands
2. Additional aggregate data presented on pages 17 to 19 of this report

Table 5: Compressive Strength Test Results, 11/25/2019 Cast Date

Date	Age	Strength (psi)	Cylinders
11/25/19	0	Cast	-
12/02/19	7	4970	3
12/10/19	15	6000	3
12/17/19	22	6240	3
12/24/19	29	6330	3

Notes to Table 5:
1. Strength is average of tests of three 4" diameter by 8" concrete cylinders
2. Cylinders prepared pursuant to ASTM C31 and field cured with concrete members
3. Cylinders tested pursuant to ASTM C39

Table 6: Compressive Strength Test Results, 01/24/2020 Cast Date

Date	Age	Strength (psi)	Cylinders
01/24/20	0	Cast	-
01/31/20	7	4250	3
02/04/20	11	4610	3
02/07/20	14	5090	3
02/11/20	18	5190	3
02/21/20	28	5480	3

Notes to Table 6:
1. Strength is average of tests of three 4" diameter by 8" concrete cylinders
2. Cylinders prepared pursuant to ASTM C31 and field cured with concrete members
3. Cylinders tested pursuant to ASTM C39

Table 7: Compressive Strength Test Results, 04/13/2020 Cast Date

Date	Age	Strength (psi)	Cylinders
04/13/20	0	Cast	-
04/24/20	11	3230	3
04/28/20	15	4160	3
05/04/20	21	4670	3
05/11/20	28	4760	3
05/22/20	39	5120	3
06/11/20	59	6140	3 cores

Notes to Table 7:
1. Strength is average of tests of three 4" diameter by 8" concrete cylinders
2. Cylinders prepared pursuant to ASTM C31 and field cured with concrete members
3. Cylinders tested pursuant to ASTM C39
4. The final set of tests were performed on cylinders cored from the concrete

TEST EQUIPMENT:

The loads were applied with a hydraulic ram actuated with an electrically powered hydraulic pump. The load and displacement data were measured using force and displacement transducers, with the data acquired using a proprietary data acquisition system. All measurement equipment was calibrated and traceable to NIST; and the most recent calibration date for each device is also listed on each of the data sheets presented on pages 12 to 14 of this report.

The seismic shear testing was subcontracted to Element Materials Technology (Element), 662 Cromwell Avenue, Saint Paul, Minnesota. The testing by Element was conducted on 21 February 2020, and was witnessed by the author of this report. Element issued a report with the pertinent test data, and the report is attached to the end of this report.

TEST PROCEDURES:

Test procedures were as prescribed in AC502, ASTM E488 and as described in the referenced approved test plan.

PERSONNEL:

The following STI personnel participated in this project are listed below.

Ricardo Flores: Senior Test Technician
Luis Gutierrez: Test Technician
Tom Kolden: Report Author

DISPOSITION OF SAMPLES & MATERIALS:

The remaining untested anchor specimens and residue are subject to disposal thirty days from the date of this report.

TEST DATA:

The test data sheets for each test series completed by STI are presented on pages 12 to 14 of this report. The data is summarized in Table 8, below. The results of Seismic Tension testing are presented in Element Report No. ESP033012P.1R0 dated 27 February 2020, which is appended to the end of this report.

Table 8: Summary of Test Results

Test Series	Test Number	Cast Date	Test Date	Concrete Age (days)	$f'_{c,test}$ (psi)	F_{test} (lbf)	$N_{u,5000}$ (lbf)	$N_{u,mean}$ (lbf)	COV	N_k (lbf)
Static Tensiion Un-Cracked	1	11/25/19	12/19/19	24	6270	17,946	16,026	15,536	0.044	13,185
	2	11/25/19	12/19/19	24	6270	16,815	15,016			
	3	11/25/19	12/19/19	24	6270	18,005	16,078			
	4	11/25/19	12/19/19	24	6270	16,156	14,427			
	5	11/25/19	12/19/19	24	6270	18,064	16,131			
Static Tensiion Cracked	1	01/24/20	02/17/20	24	5360	16,148	15,596	16,069	0.049	13,368
	2	01/24/20	02/17/20	24	5360	16,544	15,979			
	3	01/24/20	02/17/20	24	5360	15,416	14,889			
	4	01/24/20	02/19/20	26	5420	17,547	16,853			
	5	01/24/20	02/19/20	26	5420	17,726	17,025			
Seismic Tension - Residual	1	01/24/20	02/21/20	28	5480	13,573	12,965	12,882	0.018	n/a
	2	01/24/20	02/21/20	28	5480	13,827	13,208			
	3	01/24/20	02/21/20	28	5480	13,216	12,624			
	4	01/24/20	02/21/20	28	5480	13,203	12,612			
	5	01/24/20	02/21/20	28	5480	13,612	13,002			
Min Edge/Space	1	04/13/20	05/27/20	44	5570	23,799	22,548	22,954	0.057	n/a
	2	04/13/20	06/08/20	56	5930	24,471	22,470			
	3	04/13/20	06/10/20	58	5990	22,968	20,984			
	4	04/13/20	06/10/20	58	5990	26,440	24,156			
	5	04/13/20	06/10/20	58	5990	26,938	24,611			

Notes to Table 8:
1. Tests conducted pursuant to applicable sections of ASTM E488
2. Concrete strength, $f'_{c,test}$, determined by interpolation between test dates, as shown in Tables 5 and 6
3. F_{test} are the ultimate loads in the tests
4. $N_{u,norm} = F_{test} \cdot (5{,}000 \text{ psi} / f'_{c,test})^{0.5}$; normalize test results to a common compressive strength of 5,000 psi
5. $N_{u,mean}$ is the average of $N_{u,norm}$
6. COV denotes the coefficient of variation, which is the standard deviation divided by the mean
7. $N_k = N_{u,mean} \cdot (1 - kv)$, where v is the COV and k = 3.401 (Owens factor for five test replicates); characteristic result
8. For the seismic tension tests, the results are the residual static tension results after sucoessfully cycling pursuant to the test protocol described in ASTM E488; where $N_{eq,test}$ = 8,350 lbf, $N_{n,test}$ = 6,260 lbf and $N_{t,test}$ = 4,200 lbf.

Note that the characteristic result of static tests in cracked concrete and uncracked concrete are essentially identical (differences due to the typical variables of testing). The tie strip anchor is of a design that is most likely not significantly affected by the presence of cracks, and so the results testing would be expected to be generally similar.

PHOTOGRAPHS:



Photo 2: Concrete with 1 Tie Strip Anchor for Static Tension



Photo 3: Block with Induced Crack



Photo 4: Measuring Crack Width



Photo 5: Typical Setup



Photo 6: Typical Failure



Photo 7: Typical Failure – Double Anchor

ASSESSMENT:

The design strength of the Reinforced Earth Strip Tie can be calculated pursuant to the pertinent Sections of Chapter 17 of ACI 318-14 *Building Code Requirements for Structural Concrete* (and Commentary).

The tensile strength of the Reinforced Earth Strip Tie can be calculated pursuant to Section 17.4.1.2 together with equation 17.4.1.2.

$N_{sa} = A_{se,N} \cdot f_{uta}$

$A_{se,N}$ The effective cross-sectional area, which occurs at the hole in the upper part of the tie.

$A_{se,N}$ = 2 strips · 0.140" strip thickness · (1.969" strip width – 0.563" hole) = 0.394 in²

f_{uta} Specified minimum tensile strength of the strip tie steel, 65,000 psi

$N_{sa} = A_{se,N} \cdot f_{uta}$ = (0.394 in²)(65,000 lbf/in²) = 25,610 lbf

The Tie Strip anchor set at a nominal embedment (i.e., 4 inches) into the concrete has an effective embedment of approximately 3.860 inches (4 inches – nominal strip thickness of 0.140 inch). Assuming a point load, the breakout strength of the device may be calculated pursuant to Section 17.4.2.2 and equation 17.4.2.2a.

$N_b = k_c \cdot \lambda_a \cdot f'^{0.5}_c \cdot h_{ef}^{1.5}$

k_c Breakout factor, which is 24 for cast-in anchors, in cracked concrete

λ_a 1.0 for this anchor and anchoring condition in normal weight concrete

f'_c design concrete strength, taken as 5,000 psi

h_{ef} design effective embedment, taken as 3.86 inches

$N_b = k_c \cdot \lambda_a \cdot f'^{0.5}_c \cdot h_{ef}^{1.5} = (24)(1.0)(5000)^{0.5}(3.85)^{1.5}$ = 12,870 lbf

However, the configuration of the tie strip is similar to two anchors taken together in very close proximity (i.e., group anchor), and the "point" load above may be adjusted on the basis of Section 17.4.2.1 equation 17.6.2.1a (or b). This is out of the ordinary, but for the tie strip anchor being considered, is deemed a reasonable approach.

$N_{cbg} = (A_{Nc}/A_{Nco}) \cdot \Psi_{ec,N} \cdot \Psi_{ed,N} \cdot \Psi_{c,N} \cdot \Psi_{cp,N} \cdot N_b$

$A_{Nco} = 9h_{ef}^2 = 9(3.86 \text{ in.})^2 = 134.1 \text{ in}^2$ (assuming a "point" load)

$A_{Nc} = (1.5h_{ef} + 2 + 1.5h_{ef})(1.5h_{ef} + 4 + 1.5h_{ef})$
$= (1.5(3.86) + 2 + 1.5(3.86))(1.5(3.86) + 4 + 1.5(3.86)) = (15.58 \text{ in.})(13.58 \text{ in.})$
$= 211.2 \text{ in}^2$

$\Psi_{ec,N}$ = 1.0 for the condition considered; no eccentricity (reference Section 17.4.2.4)

$\Psi_{ed,N}$ = 1.0 for the condition considered; no near-edge (reference Section 17.4.2.5)

$\Psi_{c,N}$ = 1.0 for cracked concrete; or 1.25 for cast-in anchors in un-cracked concrete (Sect. 17.4.2.6)

$\Psi_{cp,N}$ = 1.0 for the condition considered, no edge affects (reference Section 17.4.2.7)

N_b = 12,870 lbf, as calculated above

$N_{cbg} = A_{Nco}/A_{Nc} \cdot \Psi_{ec,N} \cdot \Psi_{ed,N} \cdot \Psi_{c,N} \cdot \Psi_{cp,N} \cdot N_b$
= (211.2 in²)/(134.1 in²)(1.0)(1.0)(1.0 for cracked)(1.0)(12,870 lbf)
= 20,270 lbf for concrete in which cracking can occur

Or 20,270 lbf · 1.25 = 25,337 lbf for concrete in which cracking will not occur.

Pullout is determined on the basis of the tests that were conducted, using the characteristic values (N_k) that were normalized to the 5,000 psi concrete strength as listed in Table 7 on page 7 of this report.

$N_{p,cr}$ = 13,368 lbf $\qquad$ $N_{p,uncr}$ = 13,185 lbf

Comparing the values of N_{cbg}, N_{sa} and N_p, it is apparent the governing values are the test results, N_p. Note that the results of testing in cracked and un-cracked concrete are essentially identical, which demonstrates the tie strip anchor is not affected by cracked concrete. The two sets of data can be combined, which yields a combined mean of 15,082 lbf with a coefficient of variation of 0.053 (5.3%) and a characteristic of 13,662 lbf (using an Owen's factor of 2.568 for ten replicates). That is,

$N_{p,cr} = N_{p,uncr}$ = 13,662 lbf.

For Seismic Tension Testing, the required value of N_{eq} is one-half the average of the static tension tests in cracked concrete. The test was conducted pursuant to Section 9.5 of ACI 355.2-07.

N_{eq} = 0.50($F_{test,mean}$) = 0.50(16,676 lbf) = 8,338 lbf

$N_{eq,test}$ = 8,340 lbf

The results of the seismic test are assessed pursuant to Section 9.5.3 of ACI 355.2-07. The average residual tension load after cycling was 13,486 lbf. $1.6N_{eq}$ = 13,340 lbf. Since the average residual, 13,486 lbf, is greater than $1.6N_{eq}$, 13,340 lbf, the requirement for seismic performance is satisfied. The tie strip was successfully cycled at the required value of N_{eq}, therefore no reduction for seismic design and the residual tension strength is satisfactory, no reduction for seismic loading is indicated.

For the Minimum Edge/Spacing Test of two anchors, the edge distance tested was greater than the breakout edge distance of $1.5h_{ef}$ (1.5·3.86 inches; or 5.77 inches). So, the minimum edge tested does not require and edge adjustment. The tested spacing distance was 8 1/2 inches. The calculated load for two tie strip anchors at 12 inch edge and 8 1/2 inch spacing is as follows.

A_{Nco} = 211.2 in² for a single anchor (with its unique group configuration)

$A_{Nc} = (1.5h_{ef} + 4 + 1.5h_{ef})(1.5h_{ef} + 1 + 8.5 + 1 + 1.5h_{ef})$
= (1.5(3.86") + 4" + 1.5(3.96"))(1.5(3.86") + 1" + 8.5" + 1" + 1.5(3.86")
= (15.58 in²)(22.08 in²)
= 344.0 in²

$N_b = N_{p,uncr}$ = 13,662 lbf

N_{cbg} = (344.0 in²/211.2 in²)(13,185 lbf) = 1.629(13,662 lbf) = 22,255 lbf

The calculated value, 22,255 lbf, is less than the actual mean test result, 22,954 lbf, and so the 8 1/2" spacing is validated.

The load values above are service load values derived pursuant to ACI 318-14 and the 2018 International Building Code. Calculated pursuant to that criteria the value of steel tensile strength, N_{sa} = 25,610 lbf for which the ϕ factor for steel design is 0.70 (ACI 318 Section 17.3.3 (a)); the values for concrete breakout: $N_{b,cr}$ = 20,270 lbf and $N_{b,uncr}$ = 25,337 lbf; and the value for pullout strength: $N_{p,cr} = N_{p,uncr}$ = 13,662 lbf and the ϕ factor for concrete breakout and pullout is 0.70 (reference ACI 318 Section 17.3.3 (c), where anchor governed by pullout Condition B).

It should be noted, however, that the tie strip anchor is used as a component of a retaining wall system that is designed pursuant to American Association of State Highway and Transportation Official (AASHTO) standards and criteria and not the International Building Code. The service load combinations are as defined by AASHTO (reference especially *Standard Specifications for Highway Bridges*, 2002 and *AASHTO LRFD Bridge Design Specifications*, 2020. The resistance factors prescribed by AASHTO are 0.75 (1.0 for seismic). For design of earth retaining structures, the load factors are 1.35 for dead load (vertical earth pressures), 1.75 live load (traffic) and 1.0 (seismic).

Systems such as this are not ordinarily designed using Allowable Stress Design. Based on the provisions of AASHTO, the conversion of the governing strength design load (i.e., 13,662 lbf) to an allowable stress design value is based on the following: $T_{allow,ASD} = \phi N_p / \alpha$, where α is the conversion factor calculated as a weighted average of the load factors for the controlling load combination. In this case, the controlling load combination (not including seismic) is 1.35DL + 1.75 LL. For a retaining wall height of 20 ft with a 2 ft design surcharge (a reasonably conservative and typical condition), the weighted average value, α, would be 1.4. Therefore, the allowable stress value, $T_{allow,ASD}$ = 0.75(13,662 lbf)/1.4 = 7,319 lbf. An increase of 50% on this value is permitted for seismic load conditions; that is, $T_{allow,ASD,seis}$ = 10,979 lbf.

FINAL TABLE:

The following is the final design table based on strength design.

Design Table

Characteristic	Symbol	Units	Value
Effective Steel Stress Area	A_{se}	in^2	0.394
Minimum specifified Ultimate Steel Strength	f_{uta}	psi	65,000
Steel Strength in Tension	N_{sa}	lbf	25,610
Strength Reduction Factor for Steel Failure Modes, Tension	ϕ_s	-	0.70
Nominal Embedment Depth	h_{nom}	in.	4
Effective Embedment Depth	h_{ef}	in.	3.86
Pullout Strength In Cracked or Uncracked concrete	N_p	lbf	13,662
Pullout Strength, seismic	N_{eq}	lbf	13,662
Strength Reduction Factor for Concrete Breakout	ϕ_c	-	0.75
Strength Reduction Factor for Concrete Breakout and Pullout	ϕ_c	-	0.70

Notes:
1. Tie Strip is a ductile steel element
2. Minimum concrete compressive strength is 5,000 psi
3. Strength design reduction factors pursuant to ACI 318-14 Chapter 17

TEST DATA: Static Tension, Un-Cracked Concrete

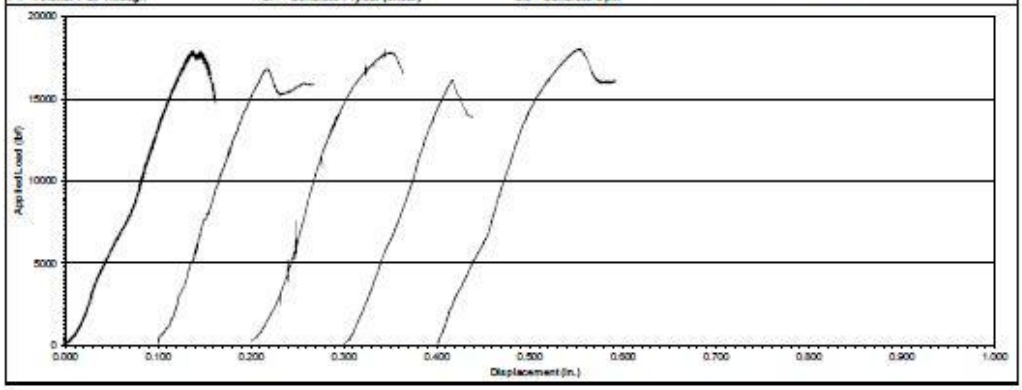
Specialized Testing, Inc. Project No. STQA50757	Sample Information and Test Standard Reinforced Earth Strap Tie Anchor ASTM E488-18 Pull Out	Test Series Static Tension Un-Cracked Concrete

Setup and Installation		Test Equipment	Asset Number	Calibration Date
Technicians	Luis Gutierrez	System Number	PDAQ	NA
		Load Cell	321765A	09/30/19
		LVDT	LSCT-1	07/31/19
		LVDT	LSCT-2	09/30/19

Test Specimen Data		Concrete Data		Calculations Based on Test Data	
Anchor System	Strap Tie Anchor	Mix #	Pro-Cast	Number of Test Samples, n	4
Anchor Material	Carbon Steel	Cast Date	11/25/2019	Average Load, $F_{Load,i}$ (lbf)	17397
		Anchor Location	Top of Deck	Standard Dev. (lbf)	865
		Member Dimension (in.)	5 1/2 x 24 x 24	COV	5.0%
		Concrete Strength (psi)	6270		

Test Data \ Test Number	1	2	3	4	5
Test Date	12/19/19	12/19/19	12/19/19	12/19/19	12/19/19
Ultimate Load (lbf)	17946	16815	18005	16156	18064
Failure Displacement (in.)	0.145	0.118	0.144	0.117	0.153
Failure Mode	CB	CB	CB	CB	CB
Test Duration (sec)	173	151	171	76	100
Initial Crack Width (in.)	n/a	n/a	n/a	n/a	n/a
Final Crack Width (in.)	n/a	n/a	n/a	n/a	n/a

Failure Mode Index			
PO - Anchor Pull Out	CB - Concrete Breakout	STT - Steel Tensile, Threads	SST - Steel Shear, Body
PT - Anchor Pull Through	CBE - Concrete Breakout Edge	STN - Steel Tensile, Neck	STN - Steel Shear, Neck
	CP - Concrete Pryout (shear)	CS - Concrete Split	



TEST DATA: Static Tension, Cracked Concrete

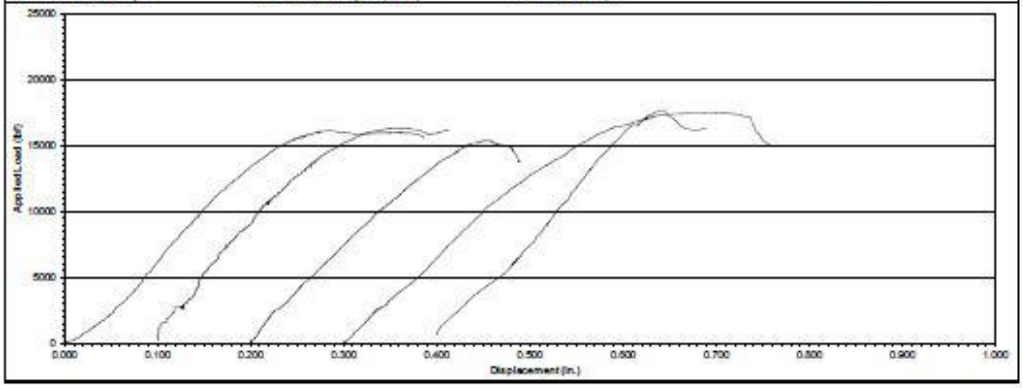
Specialized Testing, Inc. Project No. STQA50757	Sample Information and Test Standard: Reinforced Earth Strap Tie Anchor, ASTM E488-18 Pull Out	Test Series: Static Tension, Cracked Concrete

Setup and Installation		Test Equipment	Asset Number	Calibration Date
Technicians	Luis Gutierrez	System Number	PDAQ	NA
		Load Cell	321785A	03/30/19
		LVDT	LSCT-1	07/31/19
		LVDT	LSCT-2	03/30/19

Test Specimen Data		Concrete Data		Calculations Based on Test Data	
Anchor System	Strap Tie Anchor	Mix #	Pro-Cast	Number of Test Samples, n	4
Anchor Material	Carbon Steel	Cast Date	01/24/2020	Average Load, $F_{Load,i}$ (lbf)	16678
		Anchor Location	Top of Deck	Standard Dev. (lbf)	988
		Member Dimension (in.)	5 1/2 x 36 x 36	COV	5.8%
		Concrete Strength (psi)	5360 (1-3)		
			5420 (4-5)		

Test Data \ Test Number	1	2	3	4	5
Test Date	02/17/20	02/17/20	02/17/20	02/19/20	02/19/20
Ultimate Load (lbf)	16148	16544	15416	17547	17726
Failure Displacement (in.)	0.286	0.344	0.251	0.397	0.244
Failure Mode	CB	CB	CB	CB	CB
Test Duration (sec)	97	118	78	109	87
Initial Crack Width (in.)	n/a	n/a	n/a	n/a	n/a
Final Crack Width (in.)	n/a	n/a	n/a	n/a	n/a

Failure Mode Index			
PO - Anchor Pull Out	CB - Concrete Breakout	STT - Steel Tensile, Threads	SST - Steel Shear, Body
PT - Anchor Pull Through	CBE - Concrete Breakout Edge	STN - Steel Tensile, Neck	SSN - Steel Shear, Neck
	CP - Concrete Pryout (shear)	CS - Concrete Split	



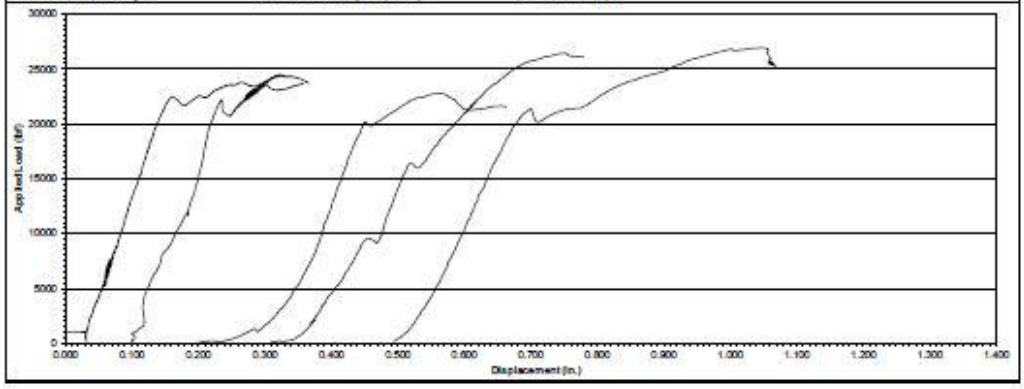
TEST DATA: Static Tension, Double Anchor, Un-Cracked Concrete

Specialized Testing, Inc. Project No. STQA50757A	Sample Information and Test Standard Reinforced Earth Strap Tie Anchor ASTM E488-18 Pull Out		Test Series Static Tension Min Edge/Spacing
Setup and Installation Technicians Ricardo Flores	**Test Equipment**	**Asset Number**	**Calibration Date**
	System Number	PDAQ	NA
	Load Cell	321785A (50K)	03/26/20
	LVDT	LVDT25	03/26/20

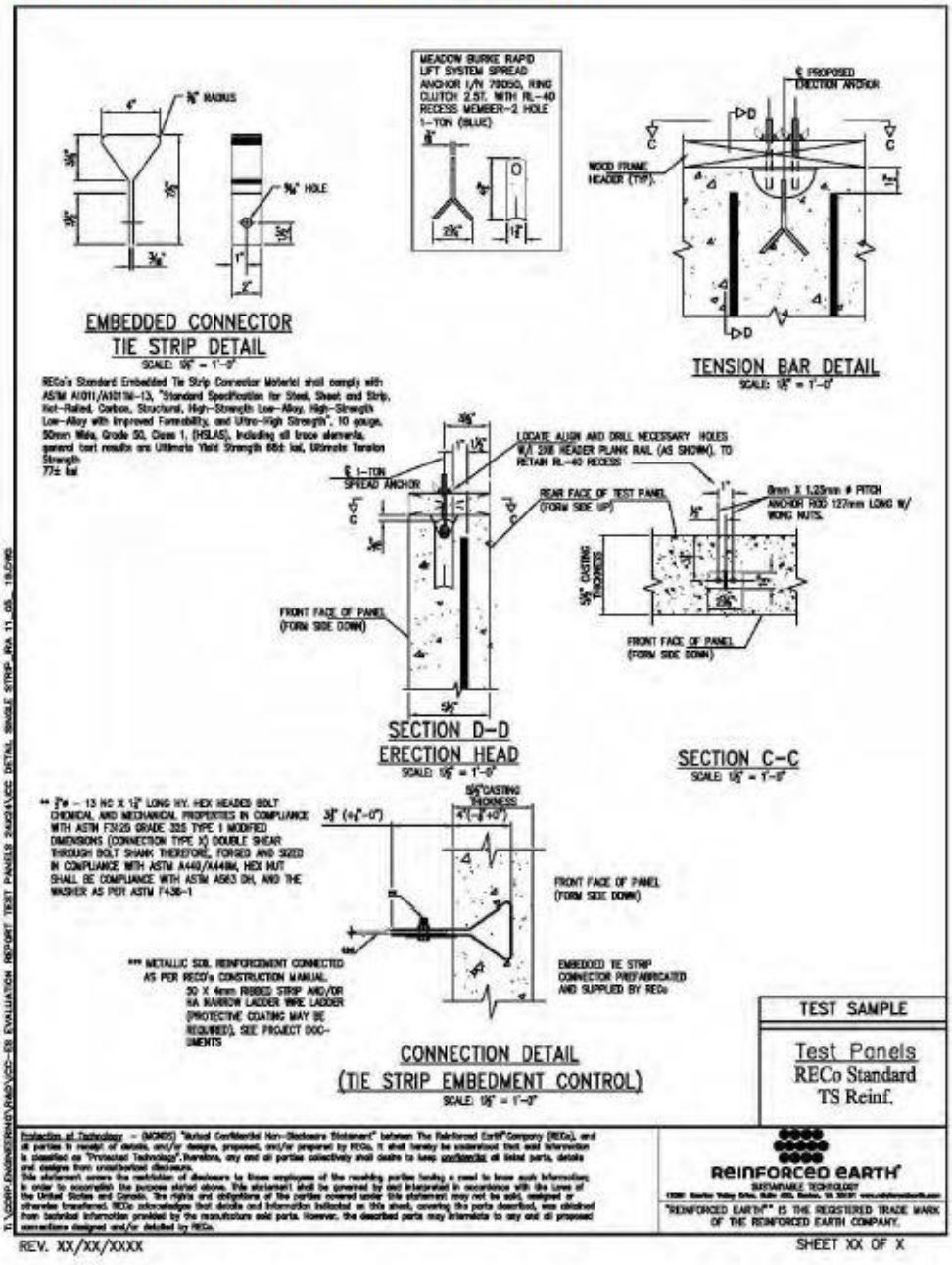
Test Specimen Data		Concrete Data		Calculations Based on Test Data	
Anchor System	Strap Tie Anchor	Mix #	Pro-Cast	Number of Test Samples, n	5
Anchor Material	Carbon Steel	Cast Date	04/13/2020	Average Load, $F_{n,test,i}$ (lbf)	24923
		Anchor Location	Top of Deck	Standard Dev. (lbf)	1707
		Member Dimension (in.)	6 x 36 x 36	COV	6.8%
		Concrete Strength (psi)	5570 (1), 5930 (2) and 5990 (3 - 5)	Note: Test results applicable to testing a pair of anchors in tension simultaneously	

Test Data \ Test Number	1	2	3	4	5
Test Date	05/27/20	06/08/20	06/10/20	06/10/20	06/10/20
Time	2:58 PM	6:52 AM	1:38 PM	2:23 PM	2:40 PM
Ultimate Load (lbf)	23799	24471	22968	26440	26938
Failure Displacement (in.)	0.265	0.221	0.542	0.452	0.646
Failure Mode	CB	CB	CB	CB	CB
Test Duration (sec)	218	254	298	306	188
Initial Crack Width (in.)	n/a	n/a	n/a	n/a	n/a
Final Crack Width (in.)	n/a	n/a	n/a	n/a	n/a

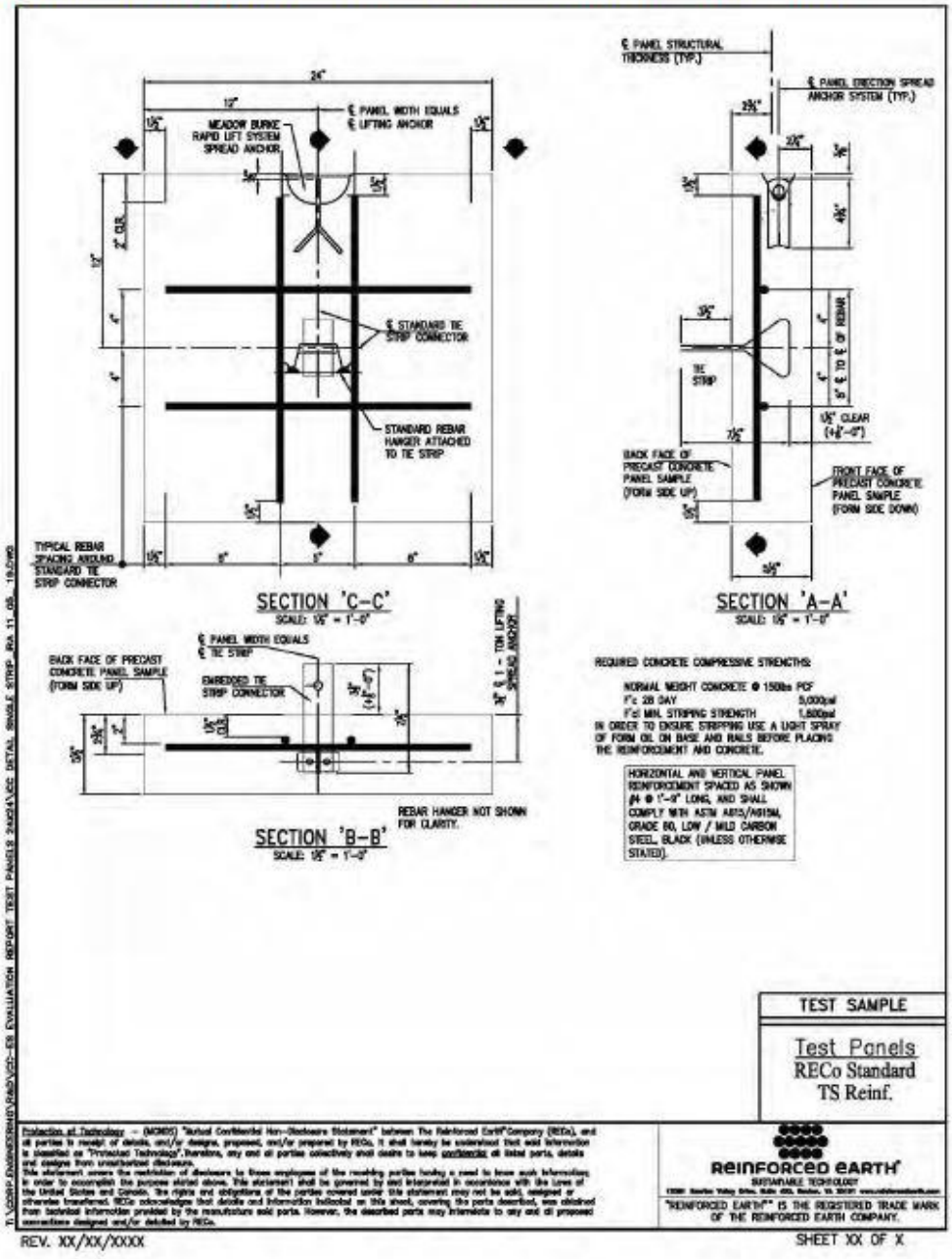
Failure Mode Index			
PO - Anchor Pull Out	CB - Concrete Breakout	STT - Steel Tensile, Threads	SST - Steel Shear, Body
PT - Anchor Pull Through	CBE - Concrete Breakout Edge	STN - Steel Tensile, Neck	STN - Steel Shear, Neck
	CP - Concrete Pryout (shear)	CS - Concrete Split	



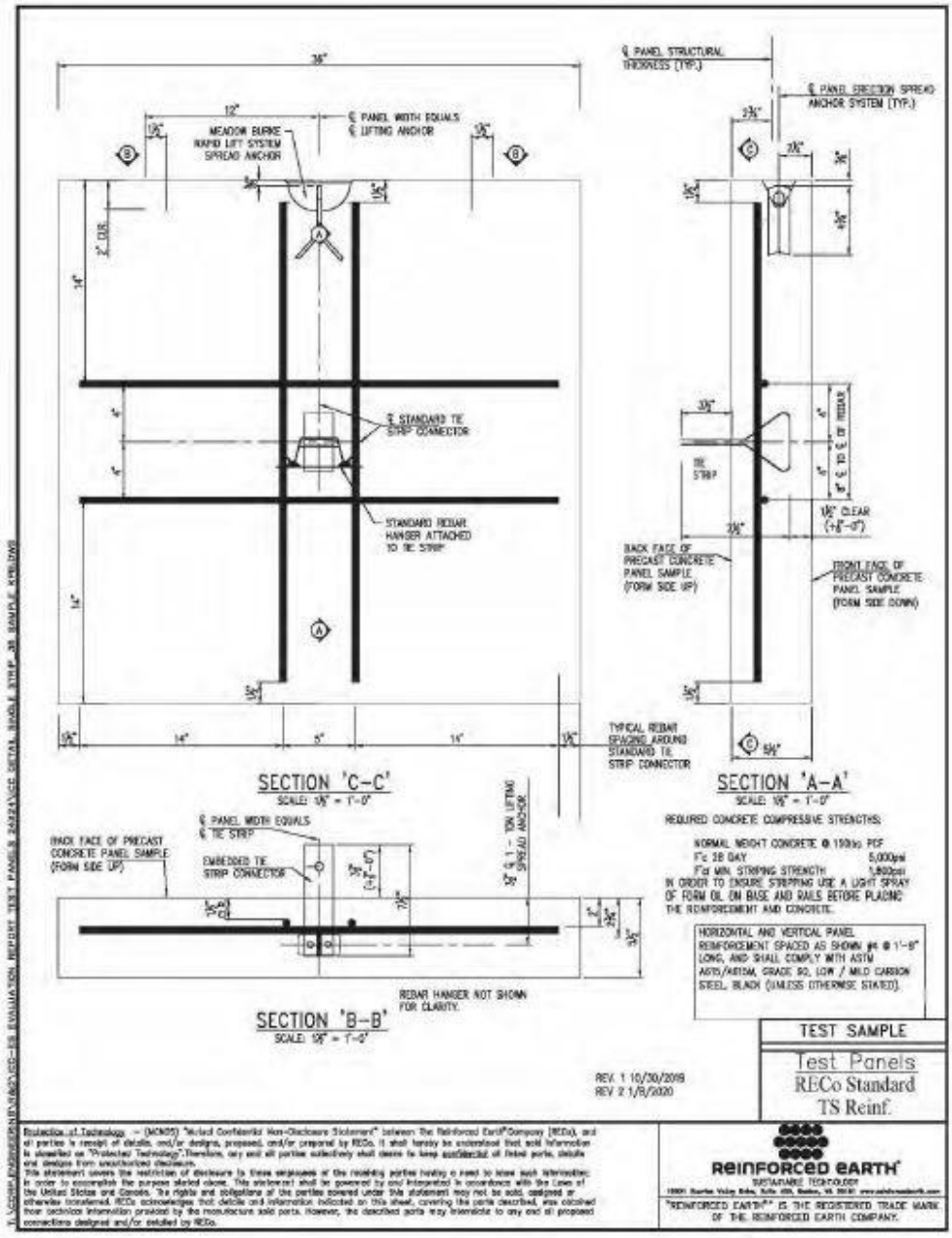
Reinforced Earth Casting Drawing: Tie Strip Installation Detail



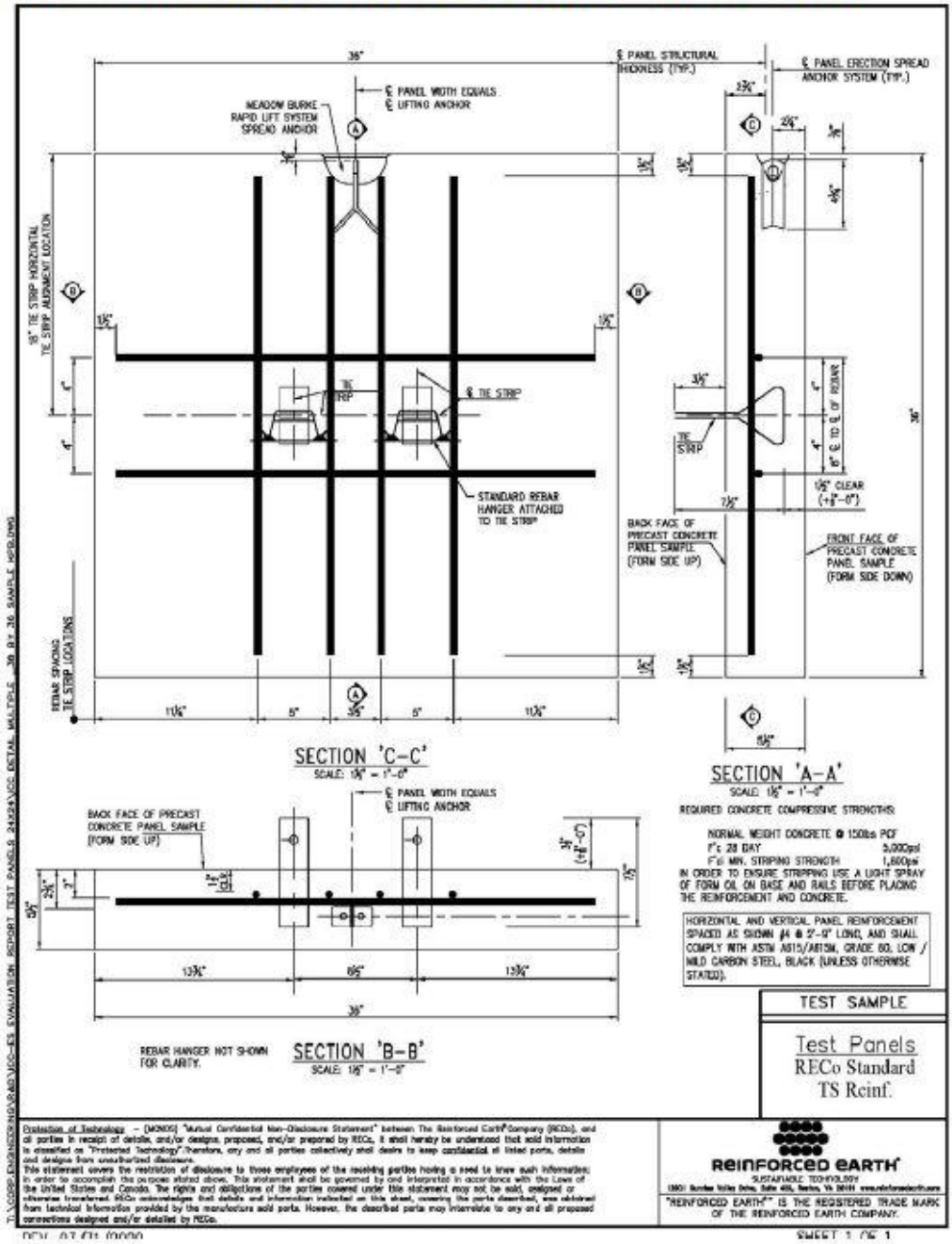
Reinforced Earth Casting Drawing: Single Anchor for Testing in Un-Cracked Concrete



Reinforced Earth Casting Drawing: Single Anchor for Testing in Cracked Concrete



Reinforced Earth Casting Drawing: Double Anchor for Testing in Un-Cracked Concrete



Pro-Cast Products, Inc., Concrete Mix Design

ROBERTSON'S

ROCK * SAND * BASE MATERIALS
READY MIX CONCRETE

Date:	6-24-19	Batch Plant: Redlands - 8353 Alabama - Redlands SMARA #91-36-0072	
Mix I.D. :	#SC675B38		
Project:	Pachappa Precast Lagging Panels		
Contractor:	Pro-Cast Products - 27417 East 3rd St - Highland		
Description:	Class I Mix - 675 lbs.	Contract No:	
Strength (f'c):	4060 psi 28 Mpa @ 28 days	W/C ratio:	0.44
Slump:	4 " +/- 1" (7" +/- 2" after addition of ADVA 190)	Sack Content:	7.18 sk.
Max. Size of Agg.:	1 "	Gal/sk.:	5.01
Pump Type:	Boom Pump (4" Line)	Un. Wt.:	145.2

MIX DESIGN PROPORTION

Contents		Batch Weights Lbs	Kg	%	Sp. Gr.	Volume Cb Ft	Cb Mt
Cement		505	300	75	3.15	2.57	0.095
Fly Ash-Class F		169	100	25	2.33	1.16	0.043
Sand		1303	773	44	2.62	7.97	0.295
1-1/2" x 3/4"		0	0	0	2.64	0.00	0.000
1" x #4		1373	815	46	2.64	8.34	0.309
3/8" x #8		296	176	10	2.62	1.81	0.067
Water	36.0 gal	299.9	178		1.00	4.81	0.178
Entrapped Air	1.25 %					0.34	0.013
	Wt.=	3948	2342		Vol. =	27.00	1.000

ADMIXTURES :

WRDA 64	3.0 oz./cwt	20.3 oz.	783.2 ML	
ADVA 198	2-15 oz./cwt	4-105 oz./yd	0 -2762 ML	

No. 16 :X=	60	3/4" X =	75
No. 30 :X=	35	3/8"X =	25
No. 50 :X=	18		

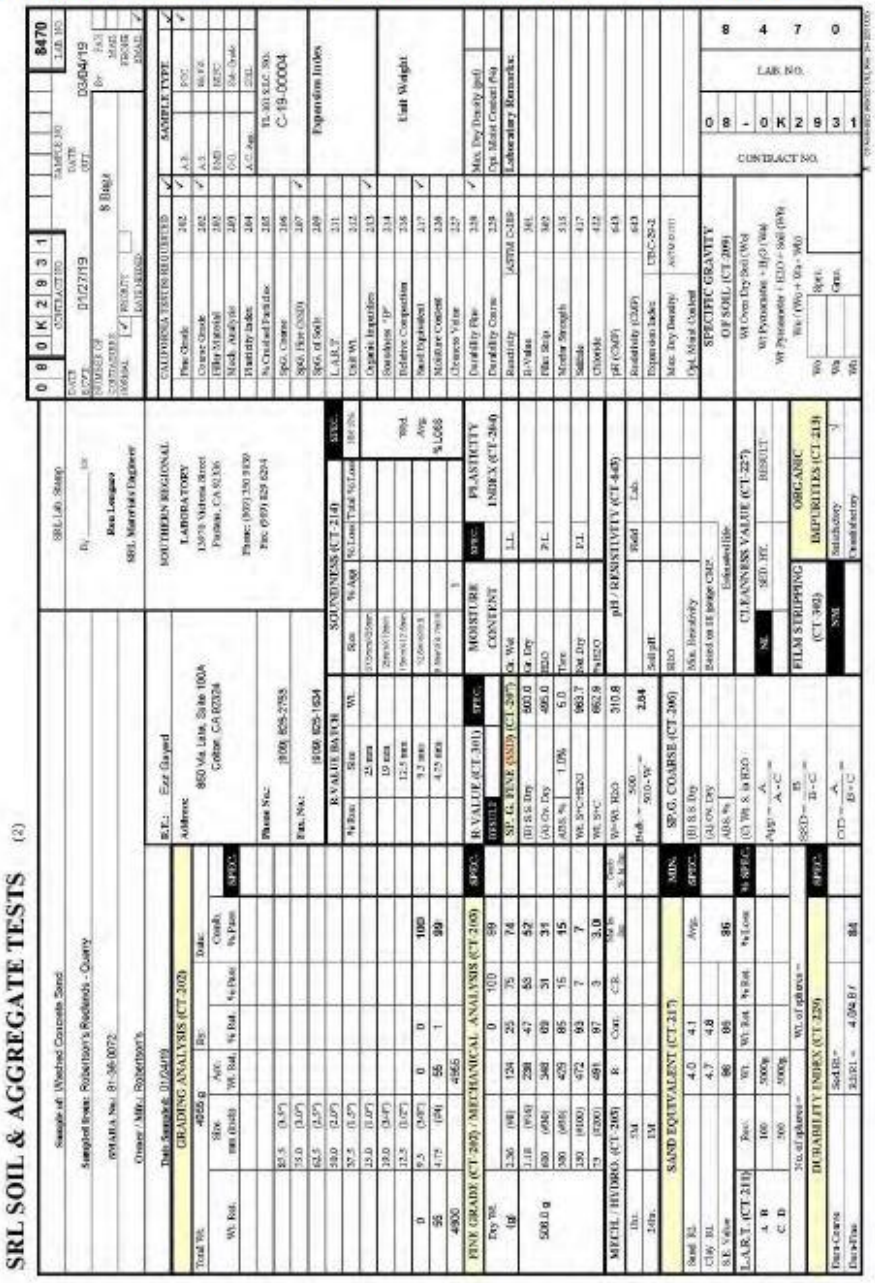
AGGREGATE GRADATIONS

Size	%	2"	1 1/2"	1"	3/4"	3/8"	No 4	No 8	No 16	No 30	No 50	No 100	No 200
WCS	44	100	100	100	100	100	99	80	60	39	18	7	2
1 1/2"	0	100	99	30	6	1	1	0	0	0	0	0	0
1"	46	100	100	95	70	12	1	0	0	0	0	0	0
3/8"	10	100	100	100	100	96	18	2	1	1	0	0	0
Blend		100	100	96	75	27	4	0	0	0	0	0	0
Combined	100	100	100	98	86	59	46	35	27	17	8	3	1
Caltrans Spec		100	100	90-100	55-100	45-75	35-60	27-45	20-35	12-25	5-15	1-8	0-4

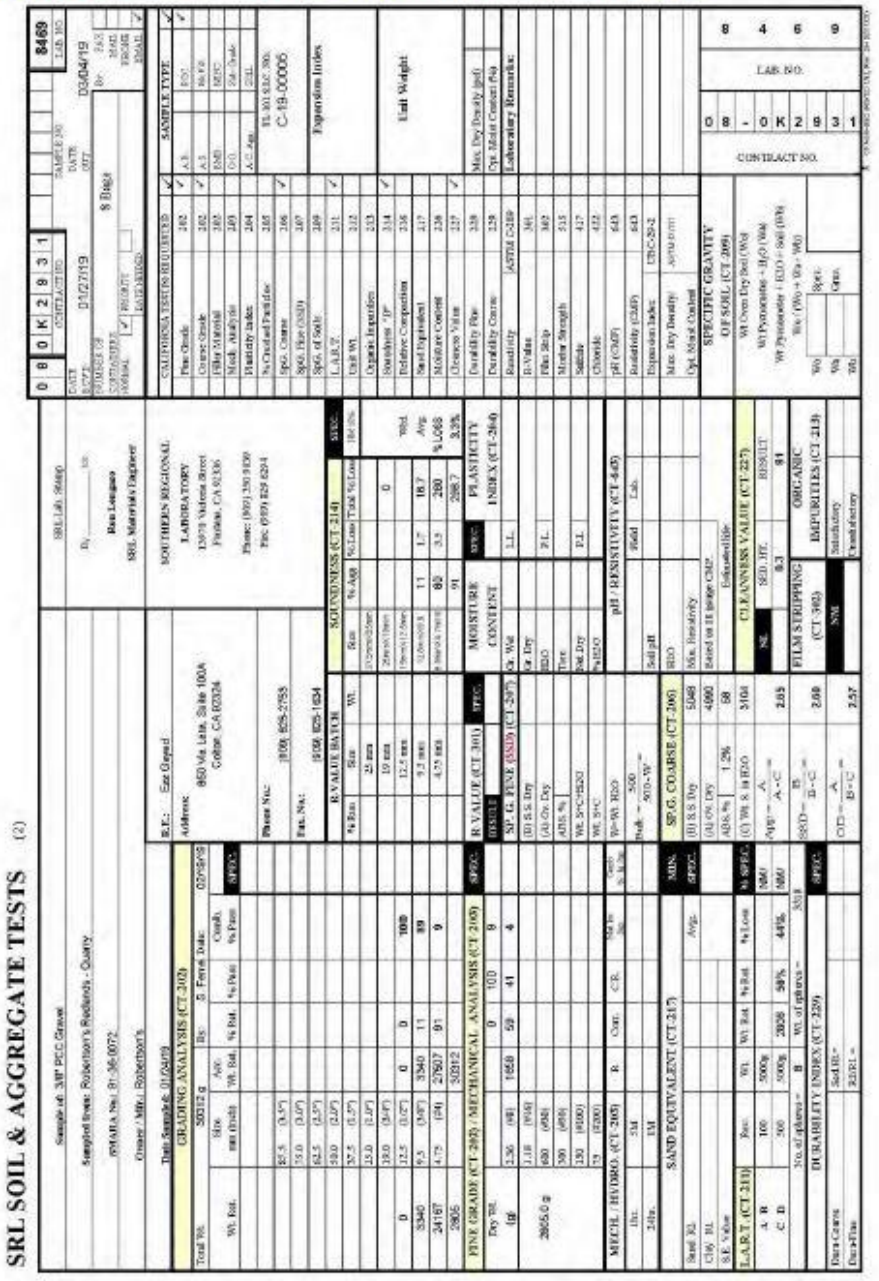
Sand Source : Robertson's Redlands
Rock Source : Robertson's Redlands
Cement : Portland Cement Type II
Flyash: Boral Resources

(951) 685-2200 ext 6381 P.O Box 3500, Corona, Ca., 92878 Fax (951) 280-1429

Aggregate Data: Washed Concrete Sand; Provided by Pro-Cast Products, Inc.



Aggregate Data: 3/8" PCC Gravel; Provided by Pro-Cast Products, Inc.



SRL SOIL & AGGREGATE TESTS (2)

Sample of: 3/8" PCC Gravel
Sampled from: Robertson's Redlands - Quarry
RMATA No.: 01-36-0072
Owner / Mfg.: Robertson's

SRL Lab Stamp

By: _____ Lic.
Dan Lavigne
SRL Materials Engineer

0 8 0 K 2 9 3 1 — 8469 (Lab. No.)
DATE: 01/27/19 — SAMPLED BY: DATE: 03/04/19
REPORT TO: — WORK ORDER NO.: — CONTRACT NO.:
SHIPPED TO: ✓ PROJECT: 8 Bags — By: — TRUCK NO.:
APPROVAL: — DELIVERED TO: — PHASE:

E.I.C.: Eric Gerard

SOUTHERN REGIONAL LABORATORY
Address: 850 Via Lata, Suite 100A, Colton, CA 92324
Phone No.: (909) 825-2788
Fax No.: (909) 825-1824

Test Sample#: 01/04/19

GRADING ANALYSIS (CT 202) — Sample Wt.: 30312 g — Dry Wt.: 30312 g — Ret. 5 Perch Date: 02/05/19

Wt. Ret.	Size mm (inch)	Acc. Wt. Ret.	% Ret.	% Pass	Cumul. % Pass	SPEC.
	63.5 (2.5")					
	50.0 (2.0")					
	37.5 (1.5")					
	25.0 (1.0")					
	19.0 (3/4")					
0	12.5 (1/2")	0	0	100	100	
3540	9.5 (3/8")	3540	11	89	89	
24167	4.75 (#4)	27707	91	9	9	
2605		30312				

FINE GRADE (CT 205) / MECHANICAL ANALYSIS (CT 2165) — SPEC.

Dry Wt. (g)			R	Corr.	CR	Adj. % Ret.	Cumul. % Ret.	SPEC.
2605.0 g	2.36	(#8)	1468	59	41	4		
	1.18	(#16)						
	600	(#30)						
	300	(#50)						
	150	(#100)						
	75	(#200)						

MECH. BY DMA (CT 2165) — Dish: — Wt.: 514 — Tot.: 514

SAND EQUIVALENT (CT 217) — MIN. SPEC.

		Avg.
Sand Eq.		
Clay Eq.		
S.E. Value		

L.A.R.T. (CT 211)	Rev.	Wt.	Wt. Ret.	% Ret.	% Loss	% SPEC.
A B	100	5000g				(Min)
C D	500	5000g	2928	58%	42%	(Min)

No. of spheres = B — Wt. of spheres = 3518

DURABILITY INDEX (CT 229) — SPEC.

Dust Grams	Sand Eq.*	
Dura-Plus	220/1 =	

R VALUE (CT 301) — SPEC. — 085018

SP. G. FINE (3550) (CT 207) — Wt. SSD: 500 — Bulk = 500/ — Wt. Pyc+Wtr: — Wt. Pyc+H2O: — Wt. SPC: — Wt. SPC+H2O: — Pyc+Wtr: — Pyc+H2O:

SP. G., COARSE (CT 206) — SPEC.

(1) S.S. Dry	5048	Min. Retained %
(2) Oven Dry	4990	Based on 1# Image Chart
Abs. %	1.2%	Estimated Size
(1) Wt. in H2O	3104	

Avg. = A =		
Avg. = A-C =	2.05	
SSD = B-C =	2.09	
OD = A =	2.57	
B-C =		

SOUNDNESS (CT 214) — SPEC.

	Size	Wt.	% Ret.	% Loss	Total % Loss	Retime
	25 mm					
	19 mm				0	
	12.5 mm					Wtd.
	9.5 mm		11	11	11.7	Avg.
	4.75 mm		89	3.5	280	% Loss
			91		288.7	3.3%

MOISTURE CONTENT — SPEC.

Cr. Wt.	
Cr. Dry	
H2O	
Tare	
Wet Dry	
% H2O	

pH / RESISTIVITY (CT 643)

Soil pH	
Min. Resistivity	

PLASTICITY INDEX (CT 204) — SPEC.

L.L.	
P.L.	
P.I.	

CLEANNESS VALUE (CT 227)

	SED. WT.	RESULT
NE	83	91

FILM STRIPPING (CT 302) — ORGANIC IMPURITIES (CT 213)

Satisfactory	
Unsatisfactory	

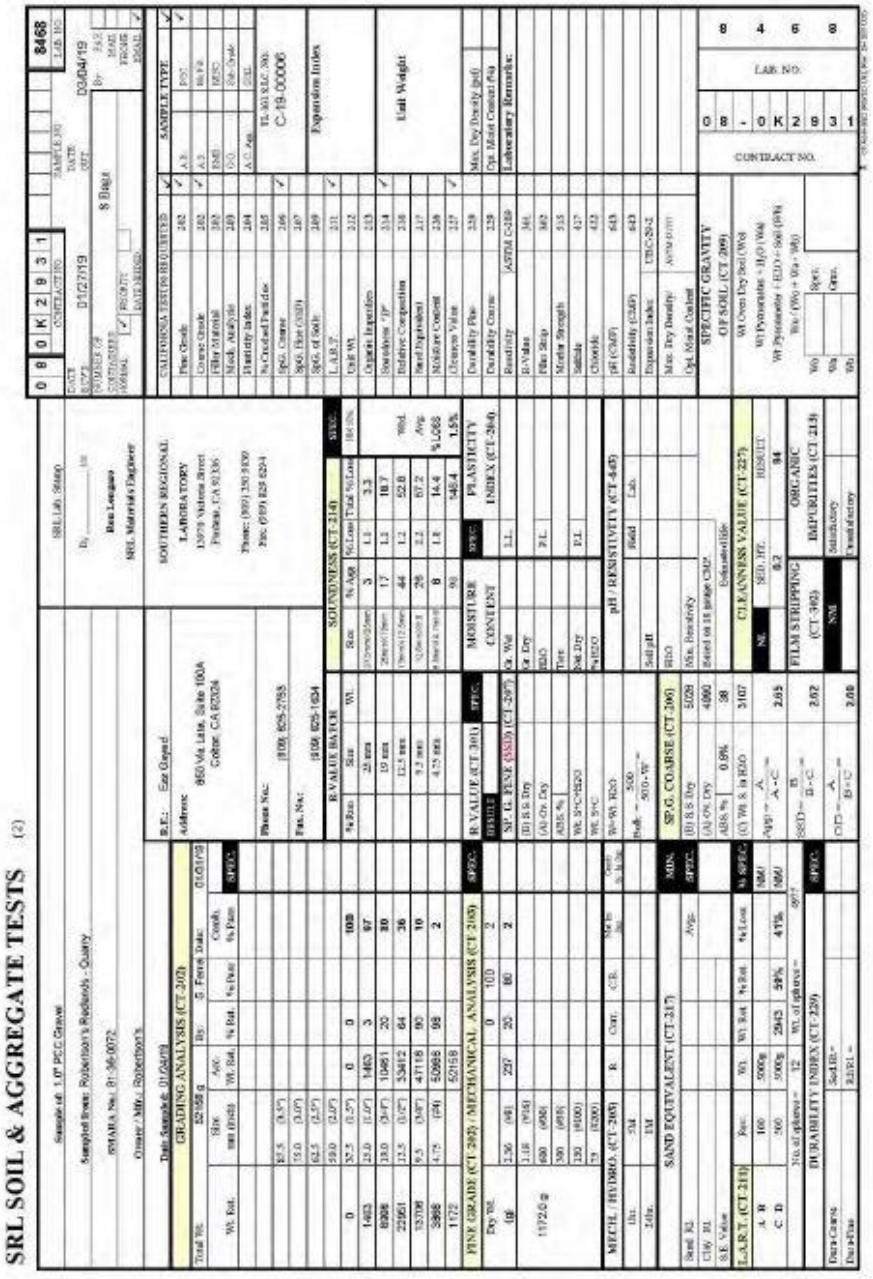
SAMPLE TYPE

Pre-Check	202	✓ A.B.
Coarse Grade	202	✓ A.S.
Fillet Material	203	DSD
Mech. Analysis	203	D/O
Durability Index	204	A.C. Agg.
% Crushed Particles	205	
Spec. Coarse	206	✓
Spec. Fine (3550)	207	
Spec. of Soils	209	
L.A.R.T.	211	✓
Unit Wt.	212	
Organic Impurities	213	
Soundness "IP"	214	✓
Relative Compaction	216	
Sand Equivalent	217	
Moisture Content	226	
Cleanness Value	227	✓
Durability Fine	229	
Durability Coarse	229	
R-Value	301	
Film Strip	302	
Mortar Strength	515	
Sulfide	417	
Chloride	422	
pH (643)	643	
Resistivity (532)	643	
Expansion Index	DSC-5h2	
Max. Dry Density	ASTM D1557	
Opt. Moist. Content		
SPECIFIC GRAVITY OF SOIL (CT 209)		
Wt Oven Dry Soil (Wo)		
Wt Pycnometer + H2O (Wa)		
Wt Pycnometer + H2O + Soil (Wb)		
Wo / (Wo + Wa - Wb)		
Spec. Grav.		

Test Sample#: 01/04/19

| | | | | | |

Aggregate Data: 1" PCC Gravel; Provided by Pro-Cast Products, Inc.



MATERIALS PROPERTIES TESTS: by Atlas Testing Laboratories

Sample (s): Customer Supplied

TEST REPORT

ATLAS TESTING LABORATORIES, INC.
9820 6th Street, Rancho Cucamonga, CA 91730 909-373-4130 FAX 909-373-4132

IG PO# 2799 DATE: 02/20/2020
ATL# 015896

CLIENT: SPECIALIZED TESTING
ATTN: TIM FOSTER
10600 PIONEER BLVD., SUITE G
SANTA FE SPRINGS, CA 90670

QTY: 3
WORK ORDER# STQA50726

ALLOY: STEEL

SPEC: ASTM-A-1011/18a, CUSTOMER REQUIREMENTS

MECHANICAL TEST, METHOD ASTM-A370/19[e1]

	INSPECTION					YIELD		ULTIMATE		ELONGATION			RDX OF AREA		
SPECIMEN ID	TEMP	VISUAL INSPECT OK?	THICK	WIDTH	ORIG. AREA	ACTUAL LOAD IN LBS	KSI	ACTUAL LOAD IN LBS	KSI	ORIG. GAGE LENGTH	ACTUAL ELONG	%	FINAL DIAM.	%	LOC. OF FRACT. (A)
1	RT	OK	0.1390	0.250	0.0348	2400	69.1	2840	81.7	1.00	0.30	30	.0650/.1620	70	1
2	RT	OK	0.1420	0.251	0.0356	2392	67.1	2857	80.2	1.00	0.31	31	.0630/.1640	71	1
3	RT	OK	0.1420	0.251	0.0356	2352	66.0	2777	77.9	1.00	0.30	30	.0620/.1650	71	1
REQUIRED, MINIMUM: CUST REQ.)							50		65			-		-	
REQUIRED, MINIMUM: ASTM-A-1011							50		60			22		-	

Strain Rate: 0.005 inch per inch per minute; Yield is at 0.2% Offset; Elongation calculated after fracture. ATE# 508

(A) 1 = Center Fracture; 2 = Outer Quarter Fracture; 3 = Fractured @ Gage Mark; 4 = Fractured @ Radius; 5 = Material Flaw/Defect

REMARKS: Material, as tested, conforms to requirements.

Respectfully Submitted:

Wendy P. Franco, Quality Assurance

Date: 2/20/2020

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Page 1 of 1

REFERENCED TEST PLAN (page 1 of 3):

SPECIALIZED TESTING
Committed to a Safer World

ICC Evaluation File 19-01-13 Test Plan (Version 3.0) November 8, 2019

Specialized Testing, Inc. (STI) entered into an agreement with The Reinforced Earth Company (client) to perform the testing of their tie strap device in accordance with Section 3.2 of the Test Plan prepared by Mr. Vince Yang of ICC-ES for Reinforced Earth on or about April 10, 2019.

STI completed testing pursuant to Version 2.0 of the test plan submitted in June of this year, but the results were not as expected. This was due to a combination of factors including using concrete strength that was less than what would typically be used, using relatively small concrete members in which to perform the tests, and testing at a minimum tolerance embedment in minimum tolerance concrete thickness.

In light of this, we propose changes to the test program as outlined below.

1. Contacts:

ICC Evaluation Service LLC	The Reinforced Earth Company
Mr. Yamil Moya, P.E. ICC Evaluation Service, LLC Western Regional Office 3060 Saturn Street, Suite 100 Brea, Califomial 92821	Paul Frankenberger, P.E. The Reinforced Earth Company 23161 Mill Creek Drive, Suite 315 Laguna Hills, CA 92653

The work to be performed is intended to primarily conform to ICC-ES AC502 in accord with the ICC-ES test plan. The follow plan details are intended to address the requirements outlined in Section 3.2 of the test plan. The testing stipulated in Mr. Yang's plan includes static tension of the Tie Strap in un-cracked concrete, static tension in cracked concrete and seismic tension in cracked concrete performed in accordance with Section 3.4 and 3.5 of AC502. In addition, the mechanical properties of the Tie Strap material will be tested in accordance with ASTM A370 to determine the yield and tensile strength as well as the elongation and reduction of area.

2. Sampling: STI will sample the specimens to be used from the precast company that produces the client's panels in San Bernardino, CA in accordance with Section 3.1 of ICC Evaluation Service AC85. We expect to obtain all specimens from a single lot and identify the lot number or equivalent lot identification information. The sampling will be documented as to time and place, how many specimens sampled, the quantity of the lot available for sampling, and the number of lots of specimens available at the time sampling is performed.

3. Product Identification/Verification: STI will provide for testing of specimens selected from the sampled set to check dimensions such as material thickness, width and hole size as well as overall dimensions and mechanical properties including tensile and yield strength, elongation and reduction of area) to confirm material satisfies requirements of the material type specified.

REFERENCED TEST PLAN (page 2 of 3):

SPECIALIZED TESTING • 10600 Pioneer Boulevard, Suite G • Santa Fe Springs, California 90670 • Office (562) 903-0032 • Fax (562) 903-3534

4 Concrete: The concrete will be cast by the precast company that produces the client's panels in San Bernardino, CA. The precast company will prepare the forms, set the reinforcing and the sampled ties. The tie straps will be cast-in-place. STI will monitor the forms to document the reinforcing type and locations as well as the tie specimens, and will witness the casting of the concrete. STI will also prepare cylinder specimens for subsequent testing to determine compressive strength.

The intent is to prepare a total of five (5) each for static tension in un-cracked concrete, five (5) each for static tension in cracked concrete, and five (5) each for seismic tension in cracked concrete with single strap tie specimens, with three (3) additional for contingencies; and to prepare seven (7) additional specimens with two anchors in each concrete member at minimum edge and spacing.

The concrete will be normal weight concrete having a minimum 28-day strength of 5,000 psi, which is the standard for such panels for use in California. The panels will be cast to a thickness of 5 1/2 inches, which is the standard thickness for such panels in California. The tie straps will be cast at a 4 inch embedment depth and the tolerance for installation will be reduced to -1/8 inch/+ 0 inch). The general layout will be as shown in the attached drawings.

g. STI will also prepare a quantity of 4" diameter by 8" cylinders from the same concrete used to prepare the forms and will test the cylinders during the curing process at intervals to monitor concrete compressive strength. Cylinders will be tested in sets of three at 7 days, 14 days, 21 days and 28 days, and possibly at other intermediate intervals as necessary.
h. All concrete blocks will be configured identically. A sketch of the form and block configurations are attached.

5. Cracked Concrete: For the purpose of cracking the concrete in order to perform anchor testing in cracked concrete, 1-inch diameter holes will be cored in selected blocks to insert splitting wedges, which will be hammered in until a crack is created. After the crack is created, the splitting wedges will be removed and then in preparation of the anchor test, reinstalled. The initial crack width will be taken as 0.000 inches wide, and will then be increased by hammering the splitters until the crack width, measured on either side of the anchor, is at an average of the target width, which is 0.012" for static tension testing and 0.020" for seismic tension testing. The intent is for the crack to propagate across the anchor center, between and parallel to the width of the straps; however, it is not unlikely that the crack will propagate along the edge of one of the two straps protruding from the concrete, as being the path of least resistance.

6. Anchor Testing: Anchor testing will be performed generally as outlined in ASTM E488-15. A minimum of five replicates will be performed for each of the test series to be performed. The loads will be applied using a hydraulic ram and electric pump, with the force and displacement measured using force and displacement transducers with the data acquired with a PC-based data acquisition system. The load-displacement data will be acquired and presented, together with the ultimate loads and failure modes. Photographs documenting setup and failures will also be taken.

For seismic tests, the requisite seismic cycled loads will be calculated based on the provision of Section 3.5 of AC502.

7. Test Report: STI will prepare an ICC-ES AC85 compliant report that includes the data outlined in Section 2.3 of AC502. Test values will be normalized to minimum concrete and/or steel properties pursuant to Section 3.6 of AC502. Normalization for concrete will be based on the square root of the ratio of minimum strength (5,000) to actual strength in test. The normalized mean and characteristic values, together with the coefficients of variation will be reported for each test. The results of the seismic tests will be assessed pursuant to the requirements

REFERENCED TEST PLAN (page 3 of 3):

delineated in AC502 as well.

8. Schedule: We anticipate the sampling and casting will be performed in completed in the first half of November, and testing will probably be performed in early December.

Respectfully submitted,

Thomas A. Kolden, P.E. (CA)
Director of Engineering

Attachments:

1. Drawing of Strap Tie (client)
2. Photograph of Strap Tie (client)
3. TS connection Detail (client)
4. Concrete Forms

element

Element Materials Technology
662 Cromwell Avenue
St Paul, MN
55114-1720 USA

P 651 645 3601
F 651 659 7348
T 888 786 7555
info.stpaul@element.com
element.com

Specialized Testing, Inc.
10600 Pioneer Blvd, Suite G
Santa Fe Springs, CA 90670

Report Number: ESP033012P.1R0
Date: February 27, 2020

Seismic Tension Testing of Strap Anchor in Accordance with ICC-ES AC193

I hereby certify that this plan, specification or report was prepared by me or under my direct supervision and that I am a duly Licensed Professional Engineer under the laws of the State of Minnesota.

Jason R. Steen
Registration No. 43491

Prepared By:

Jason Steen, P.E.
Staff Engineer
Phone: 651-659-7259

Reviewed By:

Luke Tavernit, P.E.
Senior Engineer
Phone: 651-659-7340

Revision	Report Date	Purpose
0	27 Feb. 2020	Original Report

It is our policy to retain components and sample remnants for 30 days from the report date, after which time they may be discarded. The data herein represents only the item(s) tested. This report shall not be reproduced, except in full, without the written approval of Element Materials Technology.

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EAR CONTROLLED

INTRODUCTION AND SCOPE OF WORK

This report presents the results of testing conducted on Reinforced Earth Tie Strap anchors in a seismic tension configuration. A total of 6 pre-installed Reinforced Earth Tie Strap anchors were submitted for testing. The Reinforced Earth Tie Strap anchors arrived pre-installed and pre-cracked in six individual concrete members measuring roughly 36" x 36" x 5-1/2".

The scope of our work was limited to conducting the tests and reporting the results.

Our work was authorized by Mr. Tom Kolden of Specialized Testing, Inc., on February 10, 2020 under purchase order 2801. Element is accredited for testing to ICC-ES AC193 by the American Association for Laboratory Accreditation (A2LA) as listed in their Scope of Accreditation Certificate Number 1479.01 revised 08/12/2019 and valid to 12/31/2020.

TEST METHODS

The seismic tension testing was conducted in accordance with ICC-ES AC193, "Acceptance Criteria for Mechanical Anchors in Concrete Elements" Table 4.2.

The strap anchors were tested on February 21, 2020 for seismic tension with cycle loads that were supplied by Specialized Testing. For this project no embedment depth or concrete compressive strength was given to Element. The seismic tension test performed were all done in individual concrete test members with the supplied loads and taken ultimate failure.

The equipment used to apply the loads included a hydraulic pump and cylinder. The equipment used to measure the loads and displacements are listed on the test summary sheets, along with the calibration dates. All measurement equipment is calibrated at least annually, and the calibrations are traceable to the National Institute of Standards and Technology (NIST). Measuring equipment used included load cells, string-pot transducers, linear variable differential transformers (LVDT), torque wrenches and dial calipers. In most cases, the data from the anchor tests was acquired using a proprietary data collection software system developed for Element.

REMARKS

The remaining untested and/or unused anchor specimens and the concrete members are subject to disposal thirty days from the date of this report.

CORPORATE
45610 Woodland Road, Suite 200
Sterling, VA 20166
www.reinforcedearth.com

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TEST RESULTS

Seismic Tension Test Summary

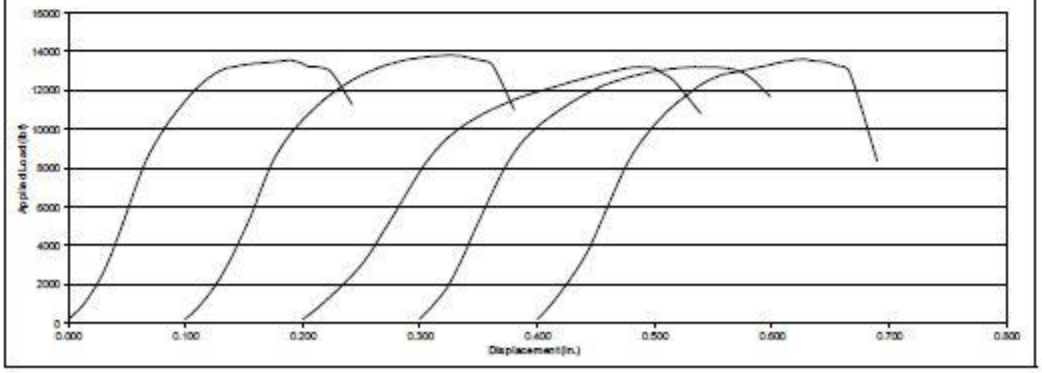
Element Materials Technology - St. Paul Project No. ESP033012P	Sample Information and Test Standard Specialized Testing, Inc. ICC-ES AC193 Table 4.2 Criteria	Test Series Series 18 Reinforced Earth Tie Strap

Setup and Installation				Test Equipment		Calibration Date
Technician(s)	J.Brown					
Drill	n/a			System Number	CME-SPC-069.1	06/18/19
Hammer Weight (lbs)	n/a			Load Cell	CME-SPC-069	06/18/19
Cleaning Method	n/a			String	n/a	n/a
Drill Bit Diameter (in.)	n/a			Dial Indicator	CME-SPC-062	04/10/19
Specified Drill Bit	n/a			Dial Indicator	CME-SPC-130	05/30/19
Bit Diameter Before (in.)	n/a	Install Temp, °F (Air, Conc.)	70,70	Caliper	n/a	n/a
Bit Diameter After (in.)	n/a	Test Temp, °F (Air, Conc.)	70,70			

Test Specimen Data		Concrete Data		Calculations Based on Test Data	
Anchor System	Reinforced Earth Tie Strap	Mix #	n/a	Number of Test Samples, n	5
Anchor Material	Galvanized steel	Cast Date	n/a	Average Load, $F_{u,test,i}$ (lbf)	13486
Anchor Size (in.)	n/a	Anchor Location	Trowel finish surface	Standard Deviation (lbf)	271
Lot Number	n/a	Member Dimension (in.)	36 x 36 x 5-1/2	COV	2.0%
Embedment, h_{ef} (in.)	n/a	Concrete Strength (psi)	n/a		
Spacing (in.)	1 per Block			N_{sb} =	8950
Edge (in.)	17			N_i =	8260
				N_{cb} =	4200

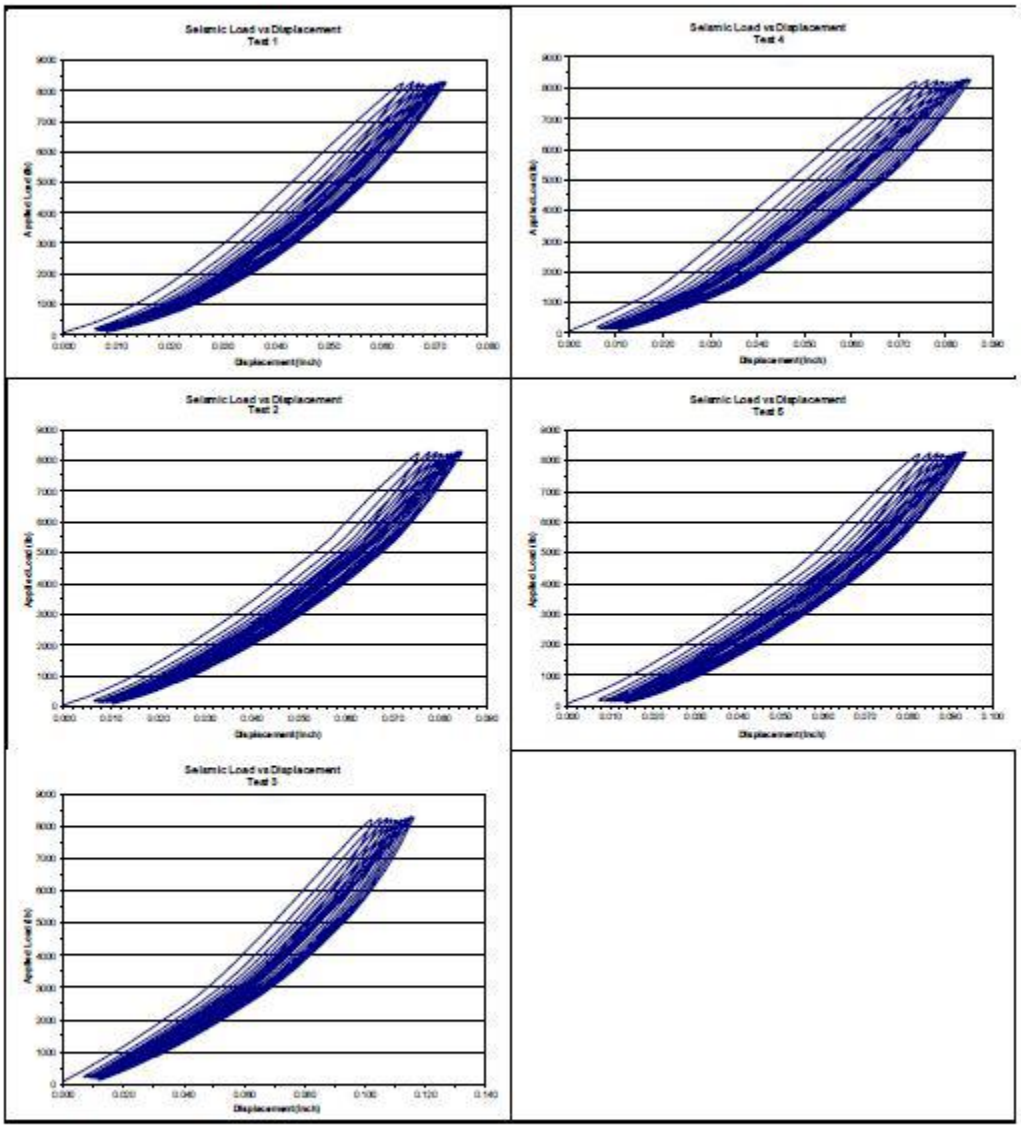
Test Number / Test Data	1	2	3	4	5					
Test Date	02/21/20	02/21/20	02/21/20	02/21/20	02/21/20					
Hammer Blows	n/a	n/a	n/a	n/a	n/a					
Installation Torque (ft-lb)	n/a	n/a	n/a	n/a	n/a					
Installation Turns	n/a	n/a	n/a	n/a	n/a					
Test Torque (ft lb)	n/a	n/a	n/a	n/a	n/a					
Ultimate Load (lbf)	13573	13827	13216	13203	13612					
Crack Before (in.)	0.020	0.020	0.020	0.020	0.020					
Crack After (in.)	n/a	n/a	n/a	n/a	n/a					
Failure Displacement (in.)	0.189	0.226	0.286	0.295	0.228					
Failure Mode	CC	CC	CC	CC	CC					
Test Duration (sec)	99	101	98	98	99					
$N_{u,i}$ (lbf)	13573	13827	13216	13203	13612					

Failure Mode Index					
PO - Anchor Pull Out	CC - Concrete Cone	SB - Steel, Body	BB - Borehole Bond
PT - Anchor Pull Through	CE - Concrete Edge	ST - Steel, Threads	BE - Bond Element
	CP - Concrete Pryout	SN - Steel, Neck	BA - Bond Anchor
	CS - Concrete Split	TN - Mating Element	TI - Internal Thread



EAR CONTROLLED

Cyclic Load Displacement Data



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PHOTOGRAPHS



Photo 1: Typical Test Setup



Photo 2: Typical Test Setup (Close)



Photo 3: Test #1 – Concrete Cone (CC) Failure Mode

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Photo 4: Test #2 - Concrete Cone (CC) Failure Mode



Photo 5: Test #3 – Concrete Cone (CC) Failure Mode

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Photo 6: Test #4 - Concrete Cone (CC) Failure Mode



Photo 7: Test #5 - Concrete Cone (CC) Failure Mode

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Prepared for:

The Reinforced Earth Company
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FINAL REPORT

CONNECTION STRENGTH FLAT AND RIBBED REINFORCING STRIPS CONNECTED TO CONCRETE PANEL VIA REGULAR AND GROUND TIE-STRIP

Prepared by:

SGI TESTING SERVICES, LLC

4405 International Blvd., Suite B-117
Norcross, GA 30093

Project Number SGI21015
Report Number SGI121015.2021.01RR
Date of Testing: October 2020 to March 2021
Date of Reporting: 27 November 2021

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 determine the connection strength of reinforcing strip to concrete panel via the Tie-Strip. The sample preparation procedures and testing conditions used in the testing program were specified by Mr. Robert Lozano of The Reinforced Earth Company (RECO). All of the pullout tests were conducted at SGI located in Norcross, Georgia.

2. TEST MATERIALS

Three types of materials were used in this testing program. Descriptions of these materials are given below:

- *Reinforcement:* 50 mm wide x 4.0 mm thick flat reinforcing strip and 50 mm wide x 6.0 mm thick ribbed reinforcing strip. Both types of strips have a 9/16 inch diameter connection hole at one end;
- *Concrete Panel:* 18 inch wide x 18 inch high x 5.5 inch thick precast concrete panels with the Tie-Strip cast at the center of panel as shown in Figure 1. There were two types of Tie-Strips: (i) 3.4 mm (10 gage) thick regular Tie-Strip, and (ii) 1.8 mm thick ground Tie-Strip.
- *Soil Material*: Concrete sand, manufactured by crushing granite rock by Vulcan Materials Company (Norcross). Particle-size analysis, standard Proctor compaction, and shear tests were conducted on the concrete sand. The test results are presented in Appendix A.

Reinforcing strips and concrete panels were provided by RECO. Concrete sand was obtained by SGI from Vulcan Materials Company (Norcross).

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3. TEST EQUIPMENT

The connection testing device used in this testing program had plan dimensions of 3 ft by 7 ft and an overall depth of 20 inch. Normal (vertical) stresses were applied to the testing specimen by using an air cylinder, and connection (horizontal) loads were applied to the test specimen through two hydraulic cylinders.

4. TEST METHOD AND PROCEDURES

4.1 Unconfined Connection Test

In unconfined connection tests (Test Series #4), the Tie-Strip connection and reinforcing strip were not confined in backfill material. As shown in Figure 1, the concrete panel was placed at the bottom of the pullout box and the back of concrete panel was placed against the rigid (block) reaction system. The end with the standard 0.5-inch diameter connection hole of the reinforcing strip was inserted into the gap between two Tie-Strip tabs with the connection holes in reinforcing strip and Tie-Strip aligned, and tightened with a 0.5-inch diameter bolt and nut as shown in Figure 1. The other end of the reinforcing strip was connected to the connection loading harness by using a special mechanical connection system. After completion of the test setup, the reinforcing strip was subjected to tensile loading by displacing it at a constant displacement rate of 0.04 inch/min as measured at the specimen clamp. The reinforcing strip was continually pulled until failure occurred.

4.2 Confined Test with Reinforcing Strip Perpendicular to Concrete Panel

In the confined connection test with the reinforcing strip perpendicular to the concrete panel (Test Series #1 and 5), the regular Tie-Strip connection and reinforcing strip were confined in backfill material. For each confined connection test, the test was set up in accordance with the following procedures as described below:

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- Construction of the Confined Space: The confined space is bounded by: (i) the front rigid reaction plate of the pullout box, (ii) the bottom of the pullout box, (iii) two sidewalls, and (iv) the concrete panel to form a rectangular prism space as shown in Figure 2. The front rigid wall was lined with smooth geomembrane, and two sidewalls were lined with smooth steel plates to minimize soil-wall friction and achieve uniform confining (normal) stress;

- The backfill soil was placed in the lower half of the confined space and compacted by hand tamping to form a 9-inch thick layer. The backfill soil was compacted to approximately 90% of maximum standard Proctor dry unit weight;

- The end with the standard 9/16-inch diameter connection hole of the reinforcing strip was inserted into the gap between two Tie-Strip tabs with the connection holes in reinforcing strip and tie-strip aligned, and tightened with a 0.5-inch diameter bolt and nut as shown in Figure 3. The other end of the reinforcing strip was connected to the connection loading harness using a special mechanical connection system;

- The backfill soil was placed in the upper half of the confined space and compacted by hand tamping to form a 9-inch thick layer. The backfill soil was compacted to approximately 90% of maximum standard Proctor dry unit weight;

- A load cell was then attached to the loading harness to measure the connection load of the reinforcing strip at the specimen clamp. An LVDT was fixed to the specimen clamp to measure the total pullout displacement at the specimen clamp and another LVDT was installed against the panel as shown Figure 14;

- A normal stress (1000 psf) was then applied to the test specimen through an air cylinder; and

- After completion of the test setup, the reinforcing strip was subjected to tensile loading by displacing it at a constant displacement rate of 0.04 inch/min as measured at the specimen clamp. The reinforcing strip was continually pulled until failure occurred.

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A confined connection test setup is shown by the test setup photos in Figures 4, 5, and 6.

4.3 Confined Test with Concrete Panel at 30° and 15° Skew Angles

In the confined connection test with the concrete panel at a 15° or 30° skew angle (Test Series #2, 3, and 6), the Tie-Strip connection and reinforcing strip were confined in backfill material and the panel was skewed at a 15° or 30° angle with respect to the normal panel orientation (i.e., reinforcing strip perpendicular to the panel surface) as shown in Figure 7. For each confined connection test with the panel at a skew angle, the test was set up in accordance with the same procedures as described Section 4.1 except the confined space was constructed differently.

The confined space is bounded by: (i) the front rigid reaction plate of the pullout box, (ii) the bottom of the pullout box, (iii) two sidewalls with different length, and (iv) the concrete panel at a skew angle to form a trapezoidal prism space as shown in Figures 8 and 9. The front rigid wall was lined with smooth geomembrane, and two sidewalls were lined with smooth steel plates to minimize soil-wall friction and achieve uniform confining (normal) stress.

5. TOTAL AND CONNECTION DISPLACEMENT

In the confined connection test, the confining soil is compressed in the horizontal direction by lateral earth pressure exerted by the panel as shown in Figures 17 and 18. The measured displacement at the clamp (total displacement) includes three components:

(i) Elongation of reinforcing steel strip (d_e),
(ii) Connection displacement d_c, and
(iii) Panel displacement d_p.

Assuming max elastic strain of the 20 inch long test reinforcing strip is 0.2%, then d_e at max connection load (in the elastic range) is less than 21*0.2% = 0.04 inch. This small

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amount (0.04 inch) is practically negligible. Therefore, the connection displacement is equal to the total displacement at the clamp minus the panel displacement, and mathematically expressed by:

$d_c = d_t - d_p$

For each test, the connection displacement is calculated by subtracting the total displacement at the clamp from the panel displacement.

6. CONNECTION TEST RESULTS

Six series of connection 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:

- Connection force versus connection displacement graph;
- Maximum connection strength versus test number graph; and
- A table that summarizes test conditions, connection forces at selected displacements, maximum connection strength, max connection displacement at maximum connection strength, and failure mode.

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7. CLOSURE

SGI appreciates the opportunity to provide laboratory testing services to The Reinforced Earth Company. 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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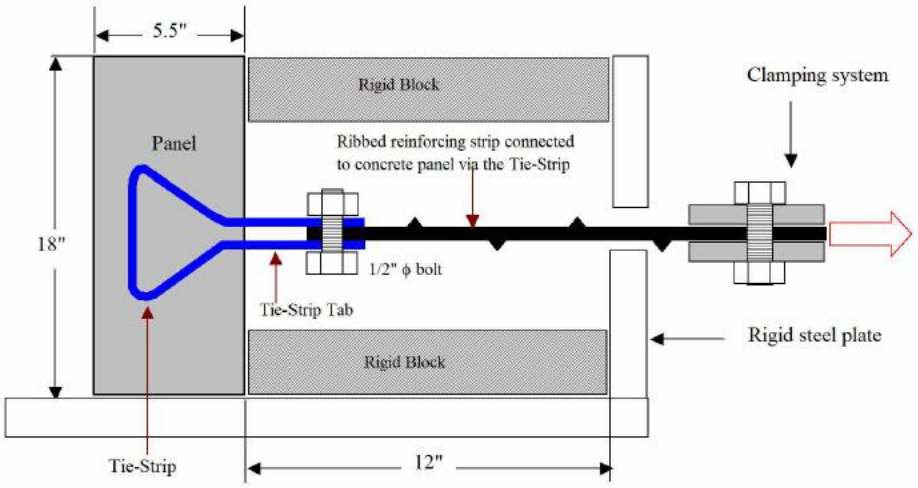


Figure 1. Unconfined connection test setup. The Tie-Strip tabs, connection bolt and nuts, and ribbed reinforcing strip were not confined in soil.

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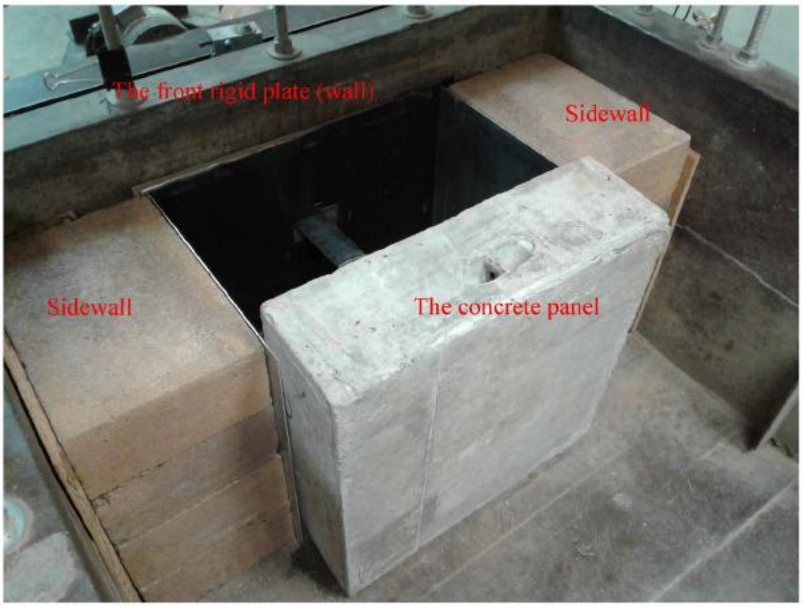


Figure 2. The confined space is defined as the space bounded by the front rigid reaction plate of the pullout box, the bottom of the pullout box, two sidewalls, and the back of the concrete panel.

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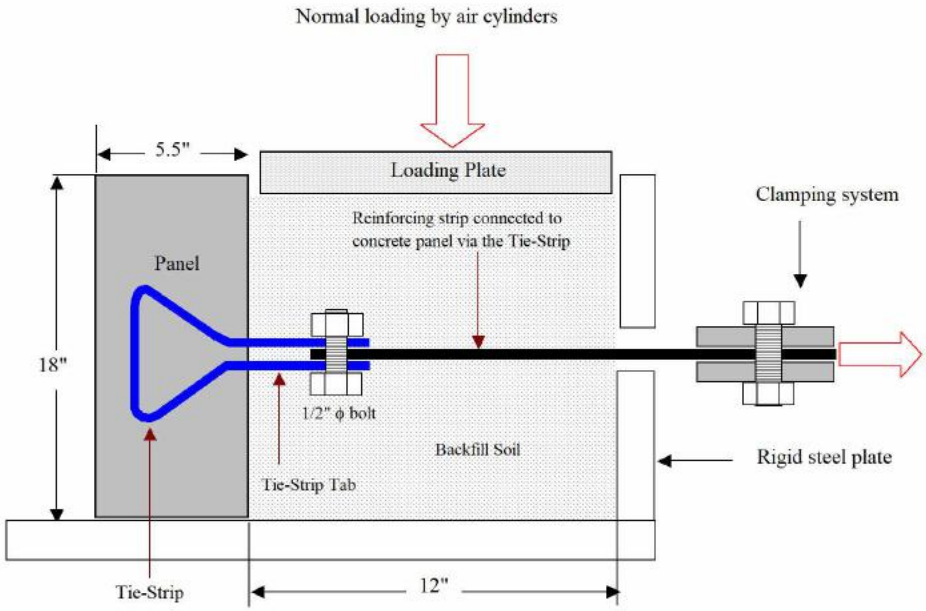


Figure 3. Schematic diagram of the confined connection test setup.

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Figure 4. Confined connection test setup step: place a panel on the bottom of the connection box, compact the backfill soil in the lower half of the confined space, and connect a reinforcing strip to the panel via the Tie-Strip connection.

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Figure 5. Compact the backfill soil in the upper half of the confined space

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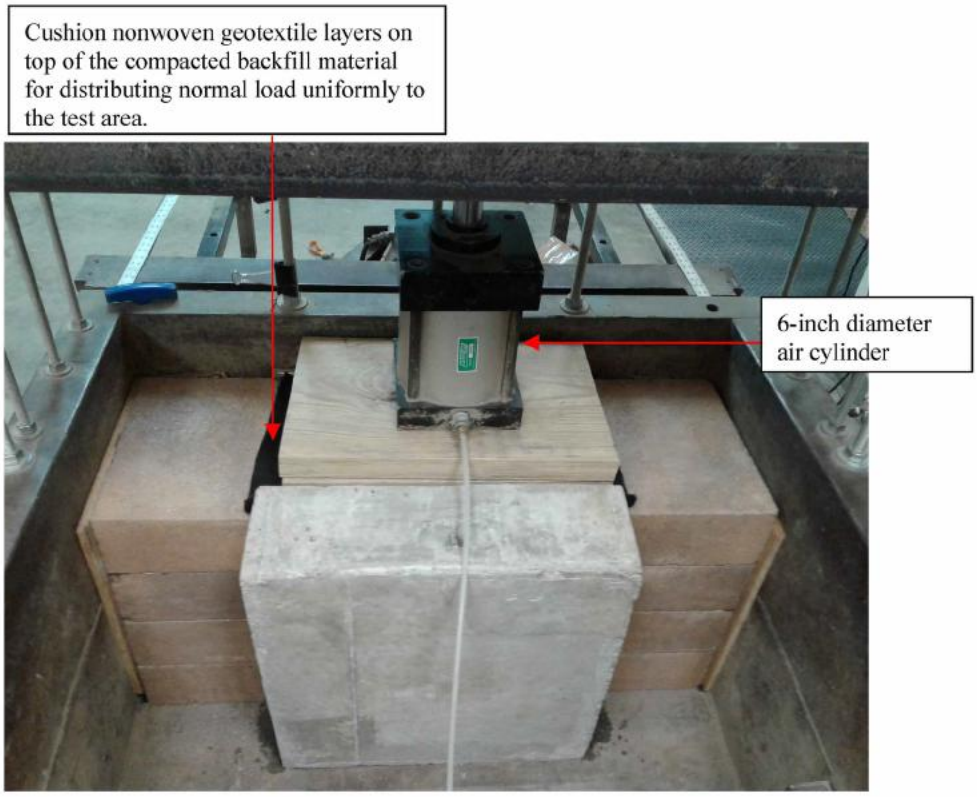


Figure 6. A completed confined connection test setup with the reinforcing strip perpendicular to the concrete panel (i.e., straight pull).

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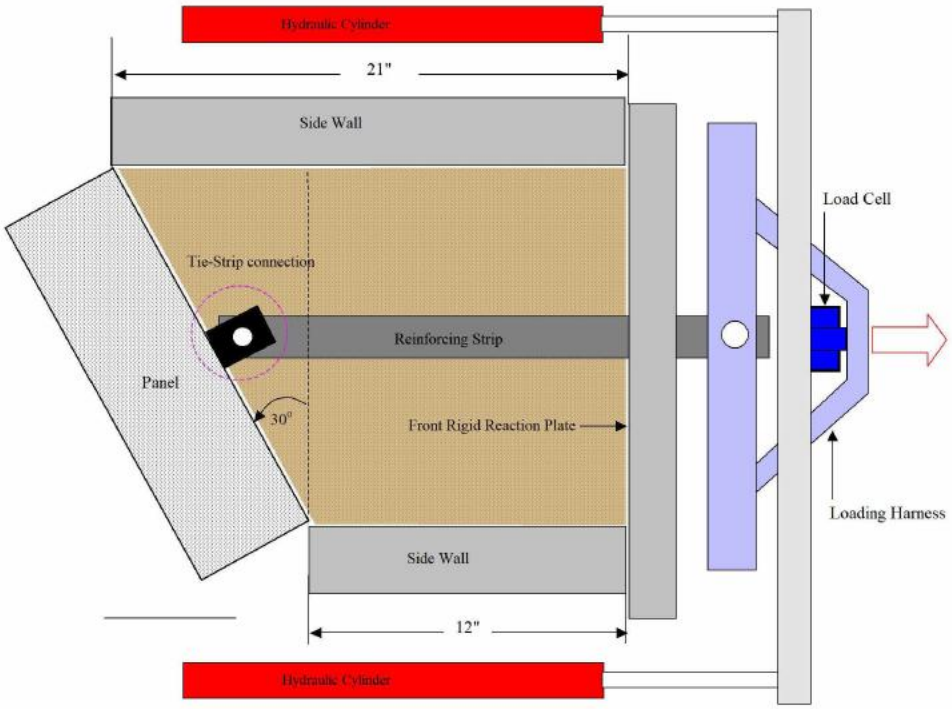


Figure 7. Schematic diagram of the confined connection test setup with the concrete panel at a skew angle α (30°).

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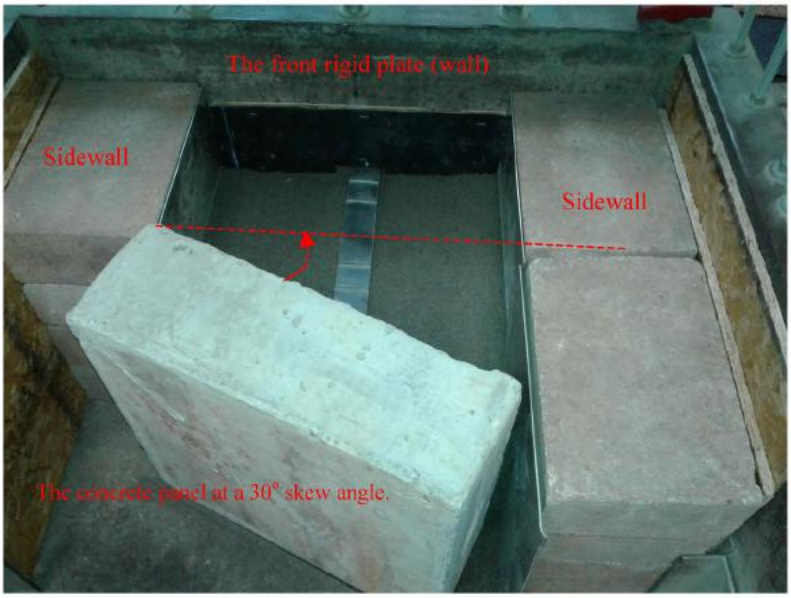


Figure 8. The confined connection test setup with the concrete panel at a 30° skew angle.

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Figure 9. The confined connection test setup with the concrete panel at a 15° skew angle.

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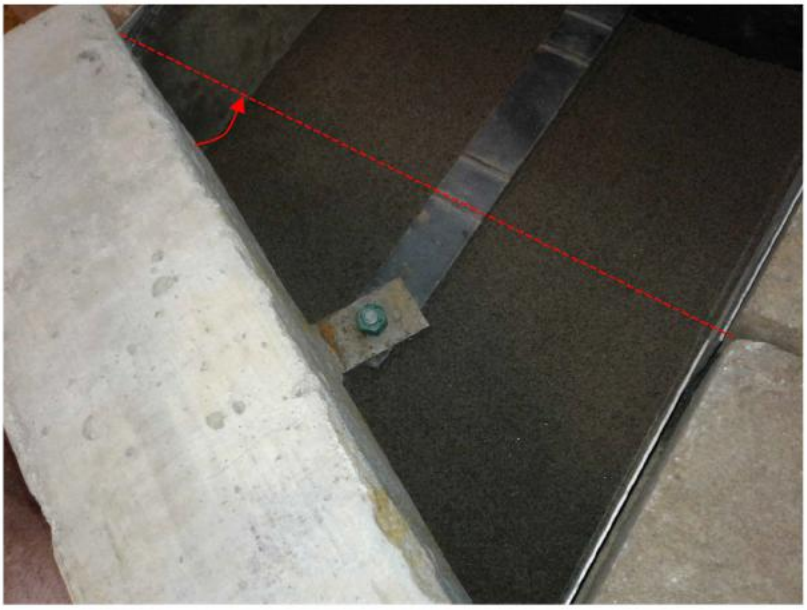


Figure 10. The confined connection test setup with the concrete panel at a 30° skew angle (ribbed steel strip connected to the regular Tie-Strip).

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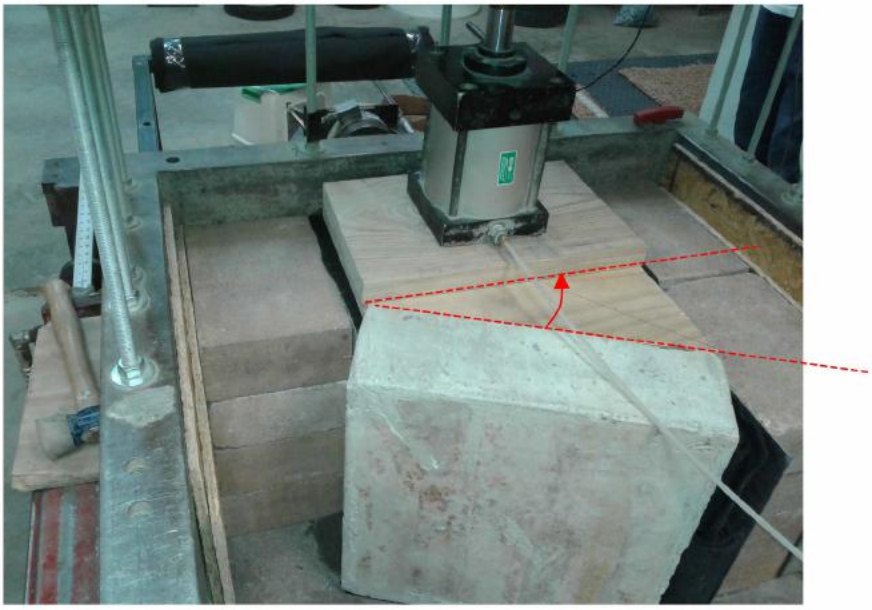


Figure 11. The confined connection test setup with the concrete panel at a 30° skew angle.

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Figure 12. Failure mode: panel cracking (Test Series #1: ribbed reinforcing strips connected to the regular Tie-Strip, straight pull).

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Figure 13. Failure mode: panel cracking (Test Series #3: ribbed reinforcing strips connected to the regular Tie-Strip, pull at a 15° skew angle).

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Figure 14. Failure mode: cracking of concrete panel at the completion of Test Series #4 (ribbed reinforcing strips connected to the regular Tie-Strip, straight pull, no soil confinement).

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Figure 15. Failure mode: rupture of the ground Tie-Strip tabs at the completion of test #5 of Test Series #5 (ribbed reinforcing strip connected to the ground Tie-Strip confined in concrete sand).

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Figure 16. Failure mode: rupture of the ground Tie-Strip tabs at the completion of Test Series #6 (ribbed reinforcing strip connected to the ground Tie-Strip at a 30° skew angle confined in concrete sand).

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Figure 17. Test setup for measuring the panel displacement in the horizontal direction. The confining soil is compressed in the horizontal direction as the reinforcing strip connected to the panel is pulled. As the result, the panel moves in the horizontal direction.

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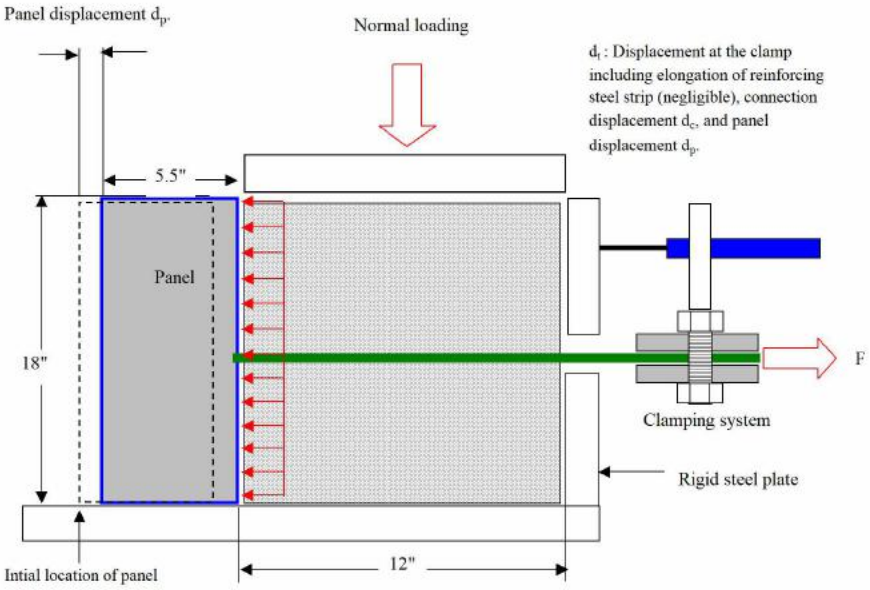


Figure 18. Connection displacement = Clamp displacement – Panel displacement.
$d_c = d_t - d_p$

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APPENDIX A

SUMMARY OF SOIL TEST RESULTS

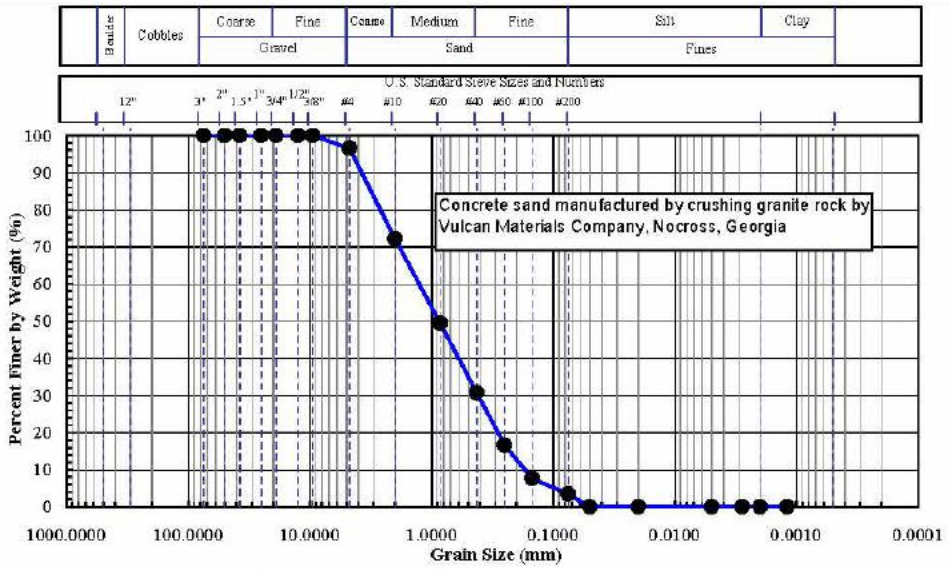
SGI Testing Services, LLC
4405 International Blvd., Suite B-117, Norcross, GA 30093
Ph: (770) 931 8222 Fax: (770) 931 8240

Project Name:	Pullout
Project No:	SGI2020
Client Sample ID:	Concrete Sand
Lab Sample No:	SGISP

ASTM D 2216, D 1140, D 422, C 136, D 4318, D 1557

SOIL INDEX PROPERTIES

Moisture Content, Grain Size, Atterberg Limits, Classification

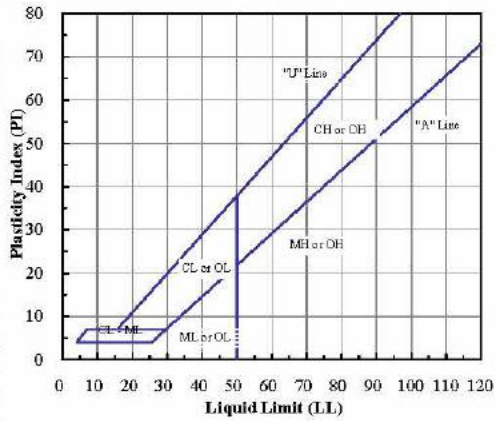


Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
1/2"	12.5	100.0
3/8"	9.5	100.0
#4	4.75	96.6
#10	2.00	72.1
#20	0.850	49.5
#40	0.425	30.7
#60	0.250	16.6
#100	0.150	7.6
#200	0.075	3.4

Hydrometer Particle Diameter (mm)	% Finer
0.0500	
0.0200	
0.0050	
0.0020	
0.0012	

Gravel (%):	3.4
Sand (%):	93.2
Fines (%):	3.4
Silt (%):	
Clay (%):	

Coeff. Unif. (Cu):	7.4
Coeff. Curv. (Cc):	0.8



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	-	3.4				SP (Poorly-Graded Sand)

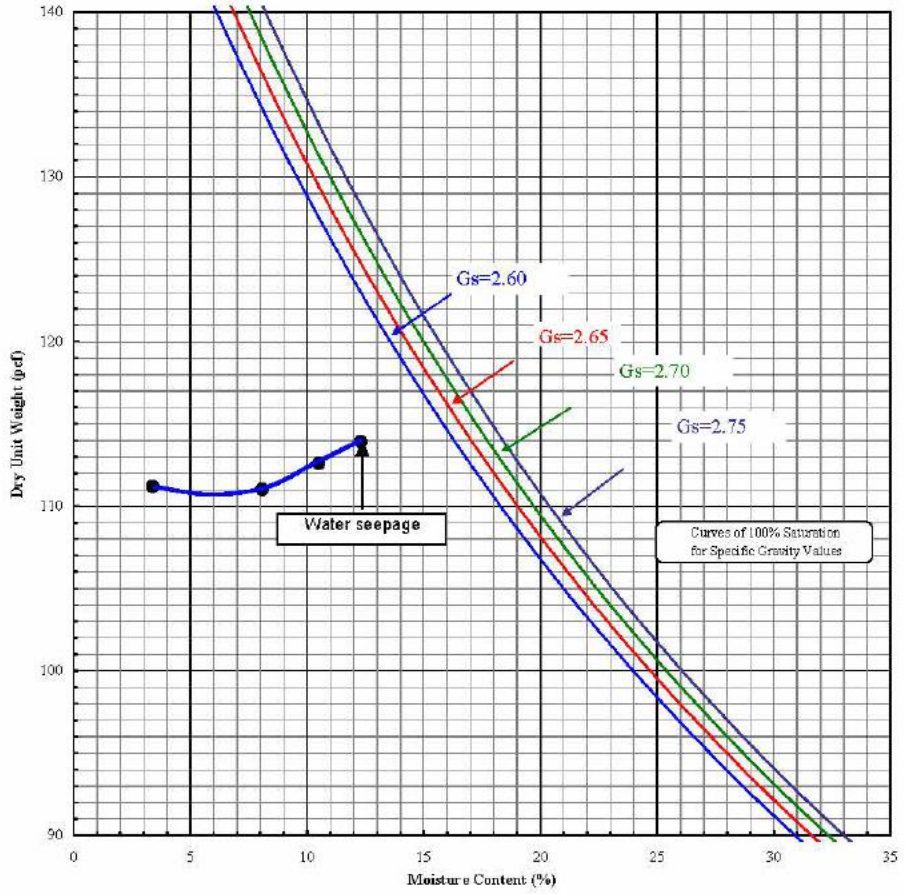
Note(s)

Concretesand soil indexxls

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Project Name:	Pullout
Project No:	SGI2020
Client Sample ID:	Concrete Sand
Lab Sample No:	SGISP

COMPACTION MOISTURE-DENSITY RELATIONSHIP (ASTM D 698 - Method A)



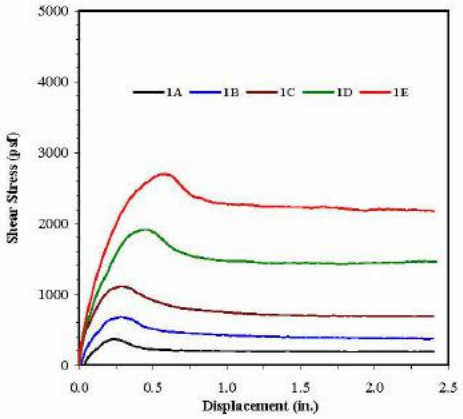
Client Sample ID.	SGI Sample ID NO.	Compaction Data		Maximum Dry Unit Weight and Optimum Moisture Content			
				Measured		Corrected	
		ω_i (%)	γ_d (pcf)	ω_{om} (%)	γ_{dmax} (pcf)	ω_{om} (%)	γ_{dmax} (pcf)
Concrete Sand	SGISP	9.4	111.2	11.0	112.0		
		8.1	111.0				
		10.5	112.6				
		12.3	113.9				

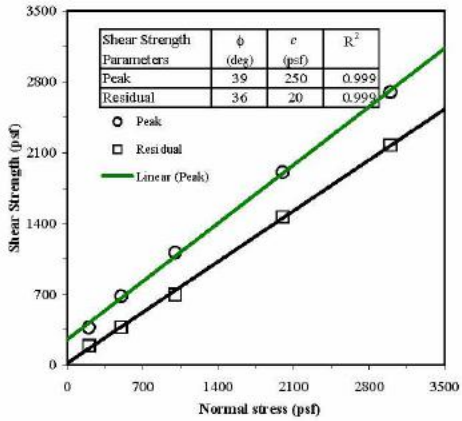
% Retained on #4 Sieve: 4.1
% Retained on #3/8" Sieve:
% Retained on 3/4" Sieve:
% Retained on 2.0" Sieve:

ConcreteSandProctor.xls

SGI TESTING SERVICES, LLC
DIRECT SHEAR TESTING (ASTM D 3080)

Internal shear strength of sand compacted to approximately 95% of maximum standard Proctor dry density at approximately 9% moisture content
(γ_{dmax} = 114 pcf, OMC = 12%)





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)	Initial γ_d (pcf)	Initial ω_i (%)	Initial ω_f (%)	After γ_d (pcf)	After ω_i (%)	After ω_f (%)	Shear Strength τ_P (psf)	Shear Strength τ_R (psf)	Failure Mode
1A	12 x 12	200	0.04	-	-	200	1.0	108.4	8.9					370	191	(1)
1B	12 x 12	500	0.04	-	-	500	1.0	107.9	9.4					680	375	(1)
1C	12 x 12	1000	0.04	-	-	1000	1.0	108.1	9.2					1112	695	(1)
1D	12 x 12	2000	0.04	-	-	2000	1.0	108.4	8.9					1908	1465	(1)
1E	12 x 12	3000	0.04	-	-	3000	1.0	108.6	8.7					2698	2175	(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: 11/29/2020

SGI TESTING SERVICES, LLC

FIGURE NO.	A-3
PROJECT NO.	SGI20031
DOCUMENT NO.	
FILE NO.	

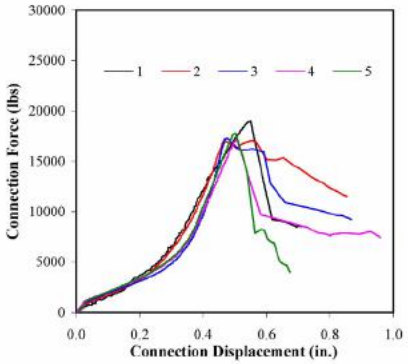
SGI SP Shear-2020.xls

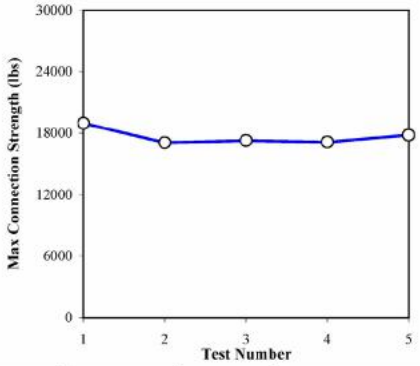
APPENDIX B

SUMMARY OF CONNECTION TEST RESULTS

THE REINFORCED EARTH COMPANY
CONNECTION STRENGTH TESTING (ASTM D 6638 MODIFIED)

TEST SERIES NO. 1: RECO ribbed reinforcing strip connected to 18" L x 18" H x 5.5" thick precast panel with standard rebars via the regular Tie-Strip confined in concrete sand compacted to approximately 90% of maximum standard Proctor dry unit weight (straight pull)





Test No.	Panel No.	Strip Dimensions Width x Thickness	Regular Tie-Strip Width	Regular Tie-Strip Thickness	Connection Hole Diameter	Normal Stress	Strength at Select Displacements Measured at the Front Clamp 0.2-in.	0.5-in.	0.75-in.	Max Connection Force F_{max}	Displacement at Max Force	Failure Mode
	(-)	(mm x mm)	(mm)	(mm)	(mm)	(psf)	(lbs)	(lbs)	(lbs)	(lbs)	(inch)	
1	TS-12	50 x 6.0	50.0	3.4	14.0	1000				18951	0.55	Rupture of concrete panel
2	TS-11	50 x 6.0	50.0	3.4	14.0	1000				17055	0.46	Rupture of concrete panel
3	TS-26	50 x 6.0	50.0	3.4	14.0	1000				17270	0.48	Rupture of concrete panel
4	TS-28	50 x 6.0	50.0	3.4	14.0	1000				17120	0.51	Rupture of concrete panel
5	TS-25	50 x 6.0	50.0	3.4	14.0	1000				17805	0.50	Rupture of concrete panel
AVERAGE										**17640**	**0.50**	

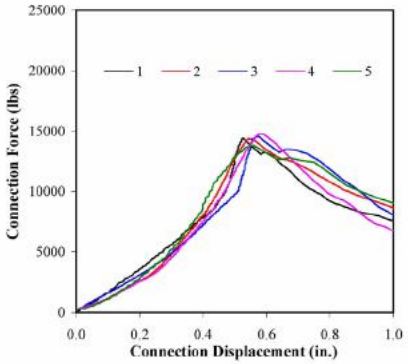
SGI TESTING SERVICES, LLC

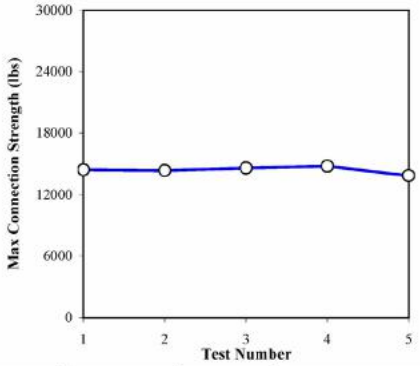
DATE REPORTED:	3/15/2021
FIGURE NO.	B-1
PROJECT NO.	SGI21015
DOCUMENT NO.	
FILE NO.	

C20015-21RR.CN.xls

THE REINFORCED EARTH COMPANY
CONNECTION STRENGTH TESTING (ASTM D 6638 MODIFIED)

TEST SERIES NO. 2: RECO ribbed reinforcing strip connected to 18" L x 18" H x 5.5" thick precast panel with standard rebars and a 30° skew angle via the regular Tie-Strip confined in concrete sand compacted to approximately 90% of maximum standard Proctor dry unit weight





Test No.	Panel No.	Strip Dimensions Width x Thickness	Regular Tie-Strip Width	Regular Tie-Strip Thickness	Connection Hole Diameter	Normal Stress	Strength at Select Displacements Measured at the Front Clamp 0.2-in.	0.5-in.	0.75-in.	Max Connection Force F_{max}	Displacement at Max Force	Failure Mode
	(-)	(mm x mm)	(mm)	(mm)	(mm)	(psf)	(lbs)	(lbs)	(lbs)	(lbs)	(inch)	
1	TS-8	50 x 6.0	50.0	3.4	14.0	1000				14420	0.53	Rupture of concrete panel
2	TS-5	50 x 6.0	50.0	3.4	14.0	1000				14347	0.54	Rupture of concrete panel
3	TS-7	50 x 6.0	50.0	3.4	14.0	1000				14587	0.57	Rupture of concrete panel
4	TS-6	50 x 6.0	50.0	3.4	14.0	1000				14777	0.57	Rupture of concrete panel
5	TS-2	50 x 6.0	50.0	3.4	14.0	1000				13840	0.56	Rupture of concrete panel
AVERAGE										**14394**	**0.55**	

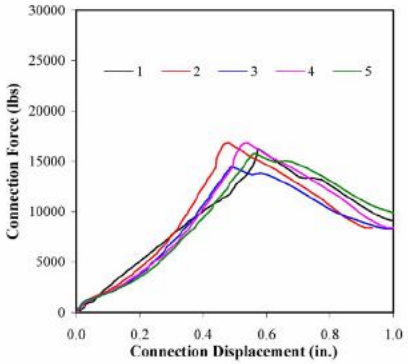
SGI TESTING SERVICES, LLC

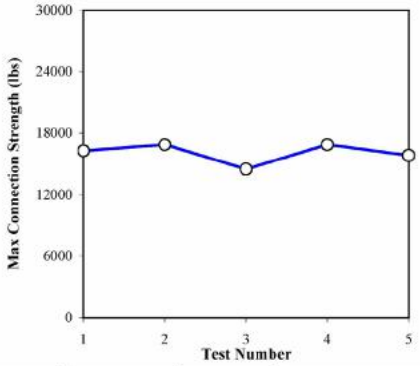
DATE REPORTED:	3/15/2021
FIGURE NO.	B-2
PROJECT NO.	SGI21015
DOCUMENT NO.	
FILE NO.	

C20015-22RR.CN.xls

THE REINFORCED EARTH COMPANY
CONNECTION STRENGTH TESTING (ASTM D 6638 MODIFIED)

TEST SERIES NO. 3: RECO ribbed reinforcing strip connected to 18" L x 18" H x 5.5" thick precast panel with standard rebars and a 15° skew angle via the regular Tie-Strip confined in concrete sand compacted to approximately 90% of maximum standard Proctor dry unit weight





Test No.	Panel No.	Strip Dimensions Width x Thickness	Regular Tie-Strip Width	Regular Tie-Strip Thickness	Connection Hole Diameter	Normal Stress	Strength at Select Displacements Measured at the Front Clamp 0.2-in.	0.5-in.	0.75-in.	Max Connection Force F_{max}	Displacement at Max Force	Failure Mode
	(-)	(mm x mm)	(mm)	(mm)	(mm)	(psf)	(lbs)	(lbs)	(lbs)	(lbs)	(inch)	
1	TS-4	50 x 6.0	50.0	3.4	14.0	1000				16243	0.57	Rupture of concrete panel
2	TS-3	50 x 6.0	50.0	3.4	14.0	1000				16864	0.48	Rupture of concrete panel
3	TS-18	50 x 6.0	50.0	3.4	14.0	1000				14493	0.50	Rupture of concrete panel
4	TS-1	50 x 6.0	50.0	3.4	14.0	1000				16864	0.54	Rupture of concrete panel
5	TS-19	50 x 6.0	50.0	3.4	14.0	1000				15798	0.57	Rupture of concrete panel
AVERAGE										**16052**	**0.53**	

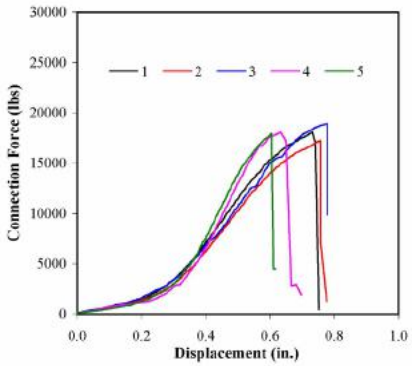
SGI TESTING SERVICES, LLC

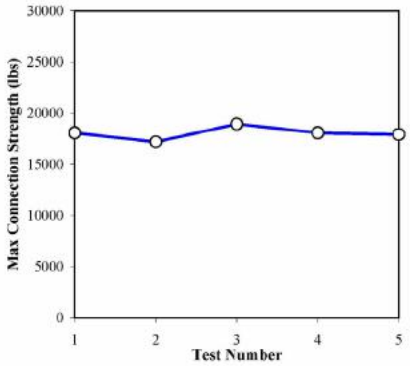
DATE REPORTED:	3/15/2021
FIGURE NO.	B-3
PROJECT NO.	SGI21015
DOCUMENT NO.	
FILE NO.	

C20015-23RR.CN.xls

THE REINFORCED EARTH COMPANY
CONNECTION STRENGTH TESTING (ASTM D 6638 MODIFIED)

TEST SERIES NO. 4: RECO ribbed reinforcing strip connected to 18" L x 18" H x 5.5" thick precast panel #T2 with standard rebars via the regular Tie-Strip with no soil confinement





Test No.	Reinforcing Strip Style	Reinforcing Strip Width x Thickness	Regular Tie-Strip Width	Regular Tie-Strip Thickness	Connection Hole Diameter	Normal Stress	Strength at Select Displacements Measured at the Front Clamp 0.2-in.	0.5-in.	0.75-in.	Max Connection Force F_{max}	Displacement at Max Force	Failure Mode
	(-)	(mm x mm)	(mm)	(mm)	(mm)	(psf)	(lbs)	(lbs)	(lbs)	(lbs)	(inch)	
1	Ribbed	50 x 6.0	50.0	3.4	14.0	No soil confinement				18074	0.73	Cracking of concrete panels
2	Ribbed	50 x 6.0	50.0	3.4	14.0					17213	0.76	
3	Ribbed	50 x 6.0	50.0	3.4	14.0					18926	0.78	
4	Ribbed	50 x 6.0	50.0	3.4	14.0					18080	0.63	
5	Ribbed	50 x 6.0	50.0	3.4	14.0					17940	0.60	
AVERAGE										**18047**	**0.70**	

NOTE (1): connection strength at 3/4" is considered the same as max connection strength occurring at displacement < 0.75".

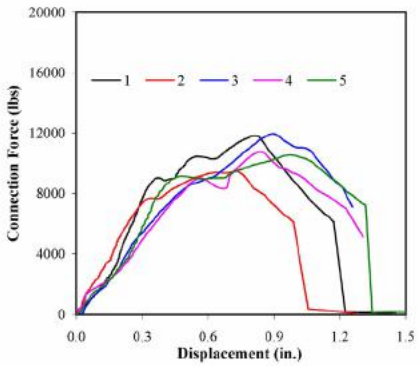
SGI TESTING SERVICES, LLC

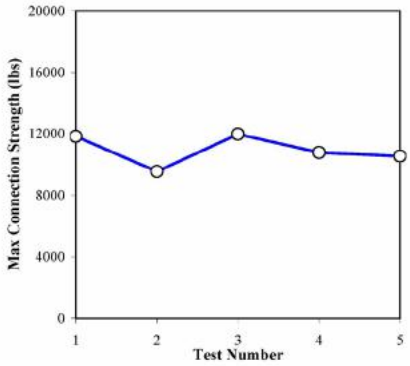
DATE REPORTED:	11/28/2020
FIGURE NO.	B-4
PROJECT NO.	SGI20015
DOCUMENT NO.	
FILE NO.	

C20015-14.T2.CN.xls

THE REINFORCED EARTH COMPANY
CONNECTION STRENGTH TESTING (ASTM D 6638 MODIFIED)

TEST SERIES NO. 5: RECO flat and ribbed reinforcing strip connected to 18" L x 18" H x 5.5" thick precast panel #T6 & 7 with standard rebars via the ground Tie-Strip confined in concrete sand compacted to approximately 90% of maximum standard Proctor dry unit weight





Test No.	Reinforcing Strip Style	Reinforcing Strip Width x Thickness	Ground Tie-Strip Width	Ground Tie-Strip Thickness	Connection Hole Diameter	Normal Stress	Strength at Select Displacements Measured at the Front Clamp 0.2-in.	0.5-in.	0.75-in.	Max Connection Force F_{max}	Displacement at Max Force	Failure Mode
	(-)	(mm x mm)	(mm)	(mm)	(mm)	(psf)	(lbs)	(lbs)	(lbs)	(lbs)	(inch)	
1	Flat	50 x 4.0	50.0	1.8	14.0	1000				11837	0.81	Rupture of two ground Tie-Strip tabs
2	Flat	50 x 4.0	50.0	1.8	14.0	1000				9550	0.73	
3	Flat	50 x 4.0	50.0	1.8	14.0	1000				11975	0.90	
4	Flat	50 x 4.0	50.0	1.8	14.0	1000				10785	0.84	
5	Ribbed	50 x 6.0	50.0	1.8	14.0	1000				10554	0.97	
AVERAGE										**10940**	**0.85**	

DATE REPORTED: 11/27/2021

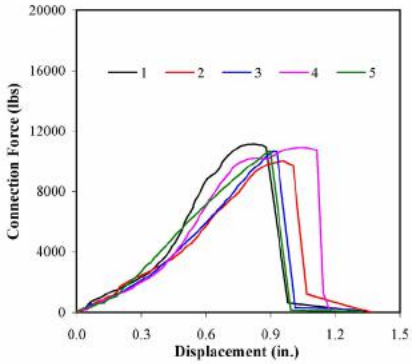
SGI TESTING SERVICES, LLC

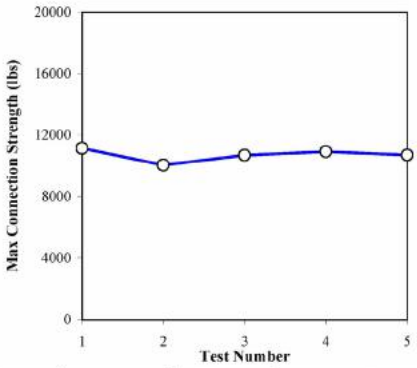
FIGURE NO.	B-5
PROJECT NO.	SGI20015
DOCUMENT NO.	
FILE NO.	

C20015-11RR.T6&7.CN.xls

THE REINFORCED EARTH COMPANY
CONNECTION STRENGTH TESTING (ASTM D 6638 MODIFIED)

TEST SERIES NO. 6: RECO ribbed reinforcing strip connected to 18" L x 18" H x 5.5" thick precast panel #T6 & 7 with standard rebars and a 30 ° skew angle via the ground Tie-Strip confined in concrete sand compacted to approximately 90% of maximum standard Proctor dry unit weight





Test No.	Reinforcing Strip Style	Reinforcing Strip Width x Thickness	Ground Tie-Strip Width	Ground Tie-Strip Thickness	Connection Hole Diameter	Normal Stress	Strength at Select Displacements Measured at the Front Clamp 0.2-in.	0.5-in.	0.75-in.	Max Connection Force F_{max}	Displacement at Max Force	Failure Mode
	(-)	(mm x mm)	(mm)	(mm)	(mm)	(psf)	(lbs)	(lbs)	(lbs)	(lbs)	(inch)	
1	Ribbed	50 x 6.0	50.0	1.8	14.0	1000				11146	0.83	Rupture of two ground Tie-Strip tabs
2	Ribbed	50 x 6.0	50.0	1.8	14.0	1000				10040	0.96	
3	Ribbed	50 x 6.0	50.0	1.8	14.0	1000				10689	0.91	
4	Ribbed	50 x 6.0	50.0	1.8	14.0	1000				10919	1.04	
5	Ribbed	50 x 6.0	50.0	1.8	14.0	1000				10693	0.90	
AVERAGE										**10698**	**0.93**	

SGI TESTING SERVICES, LLC

DATE REPORTED:	11/27/2021
FIGURE NO.	B-6
PROJECT NO.	SGI20015
DOCUMENT NO.	
FILE NO.	

C20015-12RR.T6&7.CN.xls

RECo Tie Strip Testing
AEL Laboratories

0	04-22-2022	0	KPB		KPB
Rev	Date	Modification	Drafted by	Checked by	Approved by

CORPORATE
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Sterling, VA 20166
www.reinforcedearth.com

A SUBSIDIARY OF SOLETANCHE FREYSSINET

Tie Strip Connection Pullout Tests

A series of pullout tests on tie-strip connections was conducted in 1990 on standard 50 mm wide tie strips that are used in standard Reinforcement Earth works throughout the United States. The results of these pullout tests are considered conservative since the soil pressure inherent in a Reinforced Earth structure that acts on the rear surface of the panel and tie strip shear cone was not present.

The following summarizes the results of the tie strip connection pullout tests:

Minimum Pullout Resistance of Standard Tie Strip
15,000 lbs./tie strip

Average Pullout Resistance of Standard Tie Strip
17,624 lbs./tie strip

Discussion of Results
Since the maximum allowable reinforcement tension is 7.2 kips (design tensile strength is 9.76 kips) per reinforcing strip, these test results lead to the following factors of safety and capacity demand ratio with respect to tie strip pullout:

Minimum Factor of Safety with Respect to Pullout (ASD)
FS =
15,000 lbs.
7,200 lbs.
= 2.08 > 2.0

Average Factor of Safety with Respect to Pullout (ASD)
FS =
17,624 lbs.
7,200 lbs.
= 2.45 > 2.0

Minimum Capacity Demand Ratio with Respect to Pullout (LRFD)
CDR =
0.75×15,000 lbs.
9,760 lbs.
= 1.2 > 1.0

Average Capacity Demand Ratio with Respect to Pullout (LRFD)
CDR =
0.75×17,624 lbs.
9,760 lbs.
= 1.4 > 1.0

The above factors of safety are greater than 2.0 (capacity demand ratio great than 1.0) based on full scale pullout tests. In addition, these test results are considered conservative since the beneficial effects of the earth pressure acting on the rear surface of the facing panel (and shear cone) was not present in the test.

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American Engineering Laboratories, Inc.
7940 Arjons Drive, Suite A
San Diego, CA 92126
Telephone (619) 695-3730
FAX (619) 695-6712

San Diego • Corona • Modesto • Yucca Valley • Apple Valley
Palm Desert • New York • New Jersey

November 21, 1990

THE REINFORCED EARTH COMPANY
2010 Corporate Ridge, Suite 1000
McLean, Virginia 22102

Re: TEST PROCEDURE AND RESULTS FOR THE PULL-OUT TESTS FOR THE STRIPS OF THE PRECAST CONCRETE PANELS

Gentlemen:

American Engineering Laboratories, Inc. (AEL) conducted pull-out tests for the strip anchors on October 22, 1990 as requested by The Reinforced Earth Company (RECO). Charles Levergood of RECO, Eric Iscan of AEL, Eddie Bolandi of AEL, and George Folk of AEL were present during testing.

Following are the product description test procedures and results:

PRODUCT DESCRIPTION

1. **PANELS:** Manufactured by RECO used for earth retaining created by the frictional bonding of metal reinforcements (strips) and granular soil. Strips are connected to the tie strips which are anchored into the panels.

2. **TIE STRIPS:** ASTM A-570, Grade 50, 50 mm wide and 4 mm thick. Embedded 4" into the panel. Lower part is triangular shape with 4.0" base, 3.125" high. Upper part straight 4.375" long - total height is 7.5".

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TEST PROCEDURE

1. **CASTING OF PANELS:** Los Angeles City Licensed Inspector was to be present during casting of panels to conform placing of embed, per plans and specifications, and also control the placing of concrete as defined in the plan. Slump tests were to be made and cylinders were to be taken per compression tests (please see attached compressive strength reports).

2. **ASSEMBLY OF FRAME:**

   a) Level base for the tubes was provided by rubber mats (1).

   b) Two structural tubes (TS 6 X 6 X 1/4) were placed 11" apart and another pair (TS 6 X 6 X 1/4) were placed 9" apart in the lateral direction to form a 9" X 11" unobstructed shear cone area. (2)

      TS 4 X 4 X 1/4 structural tubes were placed on top of the 6 X 6 tubes to provide the height necessary for the tie strip connection to fit in and move freely.

   c) 2' - 0" X 2' - 0" steel plate of 1" thickness with a 1 1/4" diameter hole at the center as placed at top of the 4 X 4 tubes.

   d) The ram of the hydraulic jack was then placed on 2' X 2' plate. (5). Nuts and washers were used to hold the bolt (1" diameter) against the tensile force of the jack. (6)

   e) 3/4" PL was used to provide transition between tie strip and the 1" diameter bolt. (7)

3. **TEST:** The ram was then raised by force of the jack applied per requirement of the specification ASTM E-488-88, and then the gauge pressure was read. From calibration curve, ultimate pull-out strength values were obtained. An initial load of 5% of the estimated maximum load capacity was applied in 30 seconds followed by a continuous increase in load until failure was reached.

4. **EQUIPMENT:** Hydraulic jack and ram were calibrated. The whole operation of tensile pull was video recorded for permanent record.

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TEST RESULTS

TEST NO.	TIME	GAUGE READING	LBS.	COMMENTS
NO. 1	9:05	-	-	Start
Panel #10-2	9:05:30	500	3,125	Initial force
(cast 10/2)	9:07	1700	11,090	Yield
	9:08	2300	15,000	Failure
VISUAL EXAMINATION: Cracks along the shear cone.				
NO. 2	9:40	-	-	Start
	9:40:30	500	3,125	Initial force
Panel #10-2	9:44	2,200	14,370	Yield
(cast 10/2)	9:45	3,000	19,530	Failure
VISUAL EXAMINATION: Shear cone shape obvious.				
NO. 3	10:14	-	-	Start
	10:14:30	500	3,125	Initial force
Panel #10-3	10:16	2,200	14,370	Yield
(cast 10/3)	10:17	3,000	19,530	Failure
VISUAL EXAMINATION: Shear cone shape obvious with 45 degree shear planes.				
NO. 4	10:47	-	-	Start
	10:47:30	500	3,125	Initial force
Panel #10-3	10:48:45	2,200	14,370	Yield
(cast 10/3)	10:49	2,450	15,780	Failure
VISUAL EXAMINATION: Cracks extending from shear cone to the edge of panels.				

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TEST NO.	TIME	GAUGE READING	LBS.	COMMENTS
NO. 5				
Panel #10-3	12:18	-	-	Start
(cast 10/3)	12:18:30	500	3,125	Initial Force
	12:20	2,100	13,750	Yield
	12:24	2,800	18,281	Failure
VISUAL EXAMINATION: Typical shear-cone. One crack lead to edge. Top dimension of cone 12" X 10".				

$$P\ (\text{Average}) = \frac{15{,}000+19{,}530+19{,}530+15{,}780+18{,}281}{5} = 17{,}624 \text{ lbs}$$

CONCLUSION: One of the first three tests had more than 15% variation from the average. Because of this 2 more tests were conducted and all results were averaged. The average value given above should be used as Ultimate Pull-Out strength of the strips.

The test method and loading were performed according to ASTM E-488: "Strength of Anchors in Concrete and Masonry Elements." This procedure does not include any pressure applied directly to the concrete element (panel) in the shear cone area. This varies from the actual in-service condition in which a uniform soil pressure acts upon the entire concrete panel, including the shear cone area. The magnitude of the soil pressure on the panel in-service obviously varies with the soil depth.

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Please contact the undersigned if you have any questions. If more detailed information is required, please contact the following individuals:

Test Procedures: Eddie Bolandi
AEL - Corona office (800) 266-4230

Concrete Casting Procedures: Saeed Hefazi
AEL - Corona office

Sincerely,

AMERICAN ENGINEERING LABORATORIES, INC.

Eric Iscan, RCE 26243
Director of Engineering

EI:klc

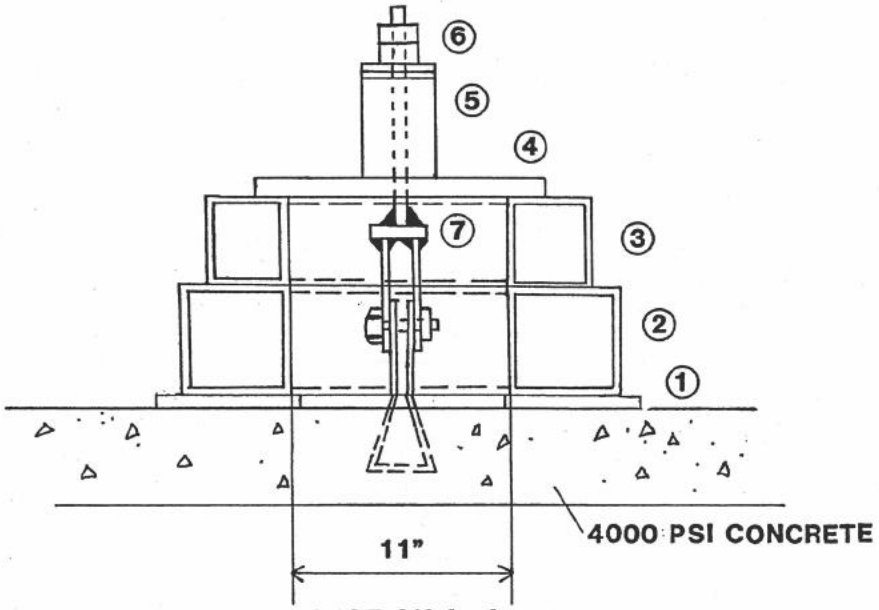


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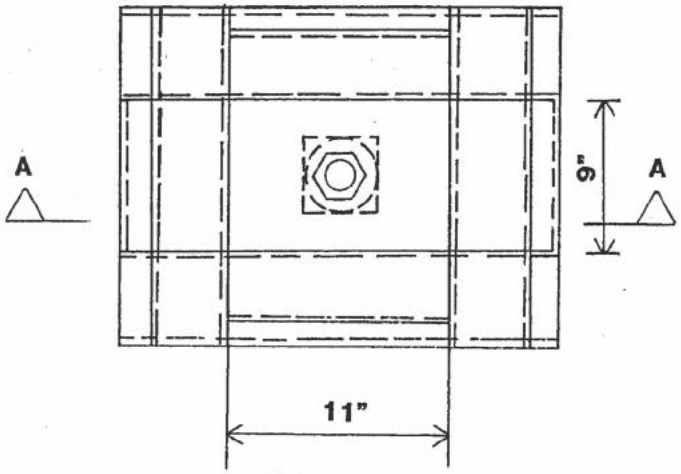
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TEST FRAME FOR THE PULLOUT TEST OF THE TIE STRIPS

(SEE ALSO ASSEMBLY OF THE FRAME IN THE REPORT)



SECTION A–A



PLAN VIEW

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Bakersfield • Palm Desert • New York • New Jersey

COMPRESSIVE STRENGTH TEST RESULTS

CUSTOMER: Reinforced Earth Co.
2010 Corporate Ridge
Suite 1000
McLean, VA

PROJECT: Corona Test Panels

JOB NO.: 1280
LAB. NO.: 1434

DATE: 31-Oct-90

SPECIMEN: Concrete Cylinders

LOCATION IN STRUCTURE: Precast Panels A6 R4 LT.3

REF. NO.: C-100

CONCRETE SUPPLIER: S.C.P., Inc.
MIX NO.: 1598 ADMIX
SLUMP, INCHES: 3 1/2"
DATE CAST: 02-Oct-90

NO. OF SPECIMENS: 5

LABORATORY NO.: 1434					
CYLINDER NO.:	A	B	C	D	E
DATE TESTED:	09-Oct	16-Oct	16-Oct	30-Oct	30-Oct
AGE AT TEST, DAYS:	7	14	14	28	28
DIMENSION, INCHES:	6x12	6x12	6x12	6x12	6x12
AREA, SQ. INCHES:	28.27	28.27	28.27	28.27	28.27
ULTIMATE LOAD, LBS.:	115000	120000	118000	140000	144000
COMPRESSIVE STENGTH, P.S.I.:	4070	4240	4170	4950	5090

SPECIFIED STRENGTH @28 DAYS, P.S.I.: 4000

REMARKS:

MEETS REQUIREMENTS @28 DAYS	FAILS TO MEET SPECIFICATIONS @28 DAYS	NO STRENGTH REQUIREMENTS GIVEN	CAST BY OUR INSPECTOR(S) @ JOBSITE	CAST BY OTHERS
X	-----	-----	X	-----

CAST BY: Saeed Hatari

Respectfully submitted,

American Engineering Laboratories, Inc.
dja

30

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COMPRESSIVE STRENGTH TEST RESULTS

CUSTOMER: Reinforced Earth Co.
2010 Corporate Ridge
Suite 1000
McLean, VA

PROJECT: Corona Test Panels

JOB NO.: 1280
LAB. NO.: 1433
DATE: 01-Nov-90

SPECIMEN: Concrete Cylinders

LOCATION IN STRUCTURE: Precast Panel Type A5 - A9 LtL

REF. NO.: C-101

CONCRETE SUPPLIER: S.C.P., Inc.
MIX NO.: 1598
SLUMP, INCHES: 3 1/2"
DATE CAST: 03-Oct-90

NO. OF SPECIMENS: 5

LABORATORY NO.: 1433					
CYLINDER NO.:	A	B	C	D	E
DATE TESTED:	10-Oct	17-Oct	17-Oct	31-Oct	31-Oct
AGE AT TEST, DAYS:	7	14	14	28	28
DIMENSION, INCHES:	6x12	6x12	6x12	6x12	6x12
AREA, SQ. INCHES:	28.27	28.27	28.27	28.27	28.27
ULTIMATE LOAD, LBS.:	103000	110000	108500	145000	150000
COMPRESSIVE STENGTH, P.S.I.:	3640	3890	3840	5130	5310

SPECIFIED STRENGTH @28 DAYS, P.S.I.: 4000

REMARKS:

MEETS REQUIREMENTS @28 DAYS	FAILS TO MEET SPECIFICATIONS @28 DAYS	NO STRENGTH REQUIREMENTS GIVEN	CAST BY OUR INSPECTOR(S) @ JOBSITE	CAST BY OTHERS
X	-----	-----	X	-----

CAST BY: Saeed Hefazi

Respectfully submitted,

American Engineering Laboratories, Inc.
djs

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A1.2.16 Soil Reinforcement Pullout Test Results

See **Appendix A1.2.1** (Innovations) for soil reinforcement pullout test results.

A1.2.17 Soil Reinforcement Interface Shear Test Results

This section is intentionally left blank.

A1.3 Other Components

This section is intentionally left blank.

A1.3.1 Innovations

The following documentation is included in this section:

- "Low-Density Cellular Concrete in MSE Structures with Steel Strip Reinforcements – Design and Construction Considerations and Case Histories," Deni and Gladstone, 2018 (14 pages)
- "Pullout Testing of Reinforced Earth® High Adherence Steel Strips in Low Density Cellular Concrete," Brabant, 2018 (102 pages)

Low-Density Cellular Concrete in MSE Structures with Steel Strip Reinforcements – Design and Construction Considerations and Case Histories

Nicholas Deni, P.E., M. ASCE[1] and Robert A. Gladstone, P.E., M. ASCE[2]

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ABSTRACT

A Mechanically Stabilized Earth (MSE) structure reinforced with steel soil reinforcements is a flexible coherent gravity mass, sized to resist external loads applied by the retained soil and any surcharge. The structure's high internal strength and high load-carrying ability derive from the permanent, predictable bond between reinforcements and granular backfill (the reinforced fill), assuring internal stability. The strength and flexibility of an MSE structure make it well-suited to sites where significant settlement is expected. Such sites may require foundation improvement, a costly but effective solution; a less costly solution is to reduce the load applied to the foundation soil by using a lightweight aggregate or low-density cellular concrete (LDCC) as the MSE reinforced fill. The selection of LDCC as the reinforced fill is typically made by the owner or the owner's engineer, not by the MSE supplier. Design and construction of the MSE structure must consider the properties of the selected LDCC product.

INTRODUCTION

Mechanically Stabilized Earth (MSE) structures with inextensible steel reinforcements were introduced in the United States in 1971. This new composite material was quickly successful structurally and economically, giving rise to competing systems, expanding usage, and research and innovation. With experience and greater understanding of the flexible nature of MSE structures, engineers attempted new ways to use the technology, including on sites characterized by lower bearing capacities and a higher expectation of settlement. The use of (frequently manmade) lightweight fill materials, in lieu of either natural soils/aggregates or ground improvement methods, led to successful MSE construction on some of these poor-foundation sites. Low density cellular concrete, a flowable material, has proven to be an effective lightweight fill for MSE structures, although some design adjustments are necessary and controlled handling and installation procedures must be followed to achieve predictable performance of the resulting MSE/LDCC composite.

To provide for safe performance over the life of an MSE structure constructed with LDCC, design and construction procedures must address the unique properties of LDCC and its behavior within MSE structures. The MSE wall designer must consider its low unit weight, its angle of internal friction and pullout resistance, the resulting required soil reinforcement length and the

possibility of buoyant behavior of the fill material. Contractors and installers must modify standard MSE installation practices to include proper handling and placement of LDCC during construction and the need for fluid containment.

This case histories paper is based on experience with MSE structures using ribbed steel reinforcing strips and backfilled with low-density cellular concrete (LDCC). Two of the case histories illustrate the use of LDCC to reduce structure settlement, while a third case demonstrates the advantages of LDCC as a flowable fill. The choice to use LDCC is always made by owners or their engineers, not by the MSE suppliers who design the actual MSE structures. It is incumbent on the MSE designers, however, to set forth requirements that will assure the desired composite behavior between the MSE system components and the LDCC. This compatibility can assure predictable results throughout the required structure life.

MSE WALL DESIGN AND CONSTRUCTION – GENERAL CONSIDERATIONS

An MSE wall using steel reinforcements is an embankment with a vertical face, comprising alternating layers of compacted granular backfill and discrete, inextensible galvanized steel soil reinforcements (strips or grids), and typically incorporating a precast concrete or wire mesh facing system (Fig. 1). The strength and stability of MSE walls derives from the permanent, predictable frictional interaction (bond) between the granular material and the reinforcements, combining the compressive and shear strengths of compacted granular fill with the tensile strength of horizontal, inextensible reinforcements. The result is a unique composite construction material having high internal strength and load-carrying ability.

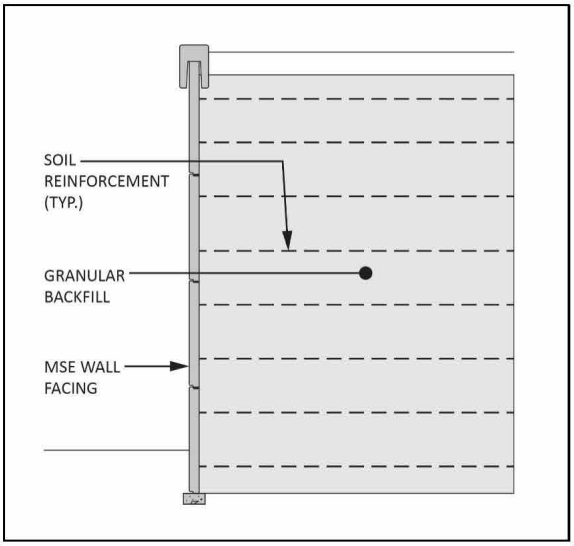


Figure 1. Typical MSE Wall Section

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Design is performed by assuming the MSE structure behaves as a rigid body, sized to resist external loads applied by the retained soil and any surcharge, while internal stability of the reinforced soil is verified by checking reinforcement tensile rupture and pullout. This design method, derived from basic soil mechanics, is known as the Coherent Gravity Method (Anderson, P.L. et al. 2010, 2012). MSE walls typically support fill retained behind them, creating eccentric loading which produces a foundation bearing pressure that is approximately 135% of the dead weight of the mass, plus any surcharge. On sites where an unacceptable magnitude of settlement is expected, foundation improvement, or substituting a lightweight backfill or low-density cellular concrete for the granular backfill, may be a solution. No matter the backfill used, however, all MSE design principles, discussed below, must be followed to assure the finished structure is truly Mechanically Stabilized Earth.

To provide for external (gravity) stability of MSE walls with inextensible steel reinforcements, as well as to assure adequate resistance to reinforcement pullout, the assumed initial reinforcement length, L, for a wall of height H is taken as $L = 0.7H$ (with $L_{min} = 8$ ft.). As required by the AASHTO code (AASHTO, 2017), the assumed angle of internal friction (ϕ) of the MSE backfill is 34° (a typical value for granular materials), absent project-specific test data supporting a higher value. Calculation of internal horizontal and vertical forces is performed using ϕ and the soil unit weight (γ) but neglecting cohesion. The total horizontal force on a facing unit divided by the number of reinforcements connected to that facing unit is the reinforcement tensile load (Section 11.10.6.2.1, AASHTO 2017). Since the specific metal loss rates associated with a given MSE backfill or low-density cellular concrete are not known at the time of design, conservative metal loss rates as specified by AASHTO should be assumed in design. The AASHTO metal loss model for MSE walls (AASHTO 2017, Sect. 11.10.6.4.2a) is used to calculate the galvanized steel soil reinforcement cross-sectional area expected at the end of the service life, allowing determination of reinforcement tensile capacity.

Pullout resistance is a function of the tensile load in the reinforcement, the vertical stress on the reinforcement (calculated over both top and bottom surfaces), and the apparent coefficient of friction between backfill and reinforcements, F* (unitless). Typical values of F* have been found through extensive laboratory testing and confirmed empirically. No matter the selected backfill material, the use of both an appropriate F* and an appropriate ϕ is critical to the safe design of any MSE structure.

Since the reinforcements provide the structure tensile strength, the MSE design must provide a sufficient number and cross section of reinforcements to achieve both the expected structural performance and the required design life. When using LDCC as the MSE backfill, the conservative design approach must consider possible LDCC degradation, from a solid mass to a fractured mass having granular backfill-like properties. A conservative maximum value of the design friction angle for LDCC is 40°, which is the maximum value of this parameter specified by AASHTO Section 11.10.6.2 (AASHTO 2017) for soil backfill. Testing of the selected LDCC product may render a higher value.

Construction of an MSE wall is a repetitive process once required excavation and pouring of an unreinforced concrete leveling pad are complete. Panels are placed in an offset-alternate pattern

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(Fig. 2), generally raising the wall by 30 in. per row of panels and with layers of reinforcements typically also spaced 30 in. vertically. Reinforcements are connected to panels using a positive mechanical connection such as a high strength bolt through the reinforcing strip and the embedded tie strips (pictured) or interlocking loops and connecting pin (grids). Fill is compacted on top of reinforcements until reaching the next level of panels or reinforcements, then the process repeats. Geotextile glued over panel joints prevents backfill outflow through the joints. If the backfill is low-density cellular concrete, modification of both joint materials and construction procedures is necessary.



Figure 2. Offset-Alternate Panel Arrangement with Panel-to-Panel Bracing

LOW-DENSITY CELLULAR CONCRETE MSE DESIGN CONSIDERATIONS

Low-density cellular concrete is defined by the American Concrete Institute (ACI) as a concrete made with hydraulic cement, water and preformed foam to produce a hardened material with an oven dry density of 50 lbs./per cu. ft. or less (ACI 2006). During mixing and placement, LDCC is a moderately viscous fluid which can be pumped to its intended location through hoses, allowing for relatively rapid delivery. When used for MSE wall backfill, structure design must address fluid containment during construction as well as fluid pressure and leakage prior to curing. It is also necessary to limit the distance to which the LDCC flows freely from the hose, to avoid segregation or breakdown of the material (including rupturing of air cells).

First used in MSE walls along Philadelphia's Schuylkill Expressway in the mid-1980s, LDCC for MSE backfill is typically supplied at a unit weight of 30-40 pcf and at a compressive strength of 140-330 psi (Aerix Industries). In an MSE application, the low unit weight of LDCC reduces the bearing pressure beneath the wall proportionate to the ratio of the unit weight of LDCC to that of granular fill, so using 40 pcf low-density cellular concrete in place of 120 pcf granular fill produces an approximately 67% reduction in applied bearing pressure. If LDCC is also placed in the retained fill zone (Fig. 3), the effect of eccentricity on bearing pressure is also reduced.

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By adjusting the LDCC foam's chemical composition, the material can be produced in either an open-cell (air cells interconnect) or closed-cell form, meaning the LDCC can be designed to be permeable or impermeable, respectively. Most MSE projects have used closed-cell LDCC technology but, as will be discussed in Case History 1, an open cell structure can reduce buoyancy of the lighter-than-water mass. As with standard concrete, cellular concrete material properties and use are the subject of ACI guidance (ACI 2006), which includes cross-references to multiple ASTM standards.

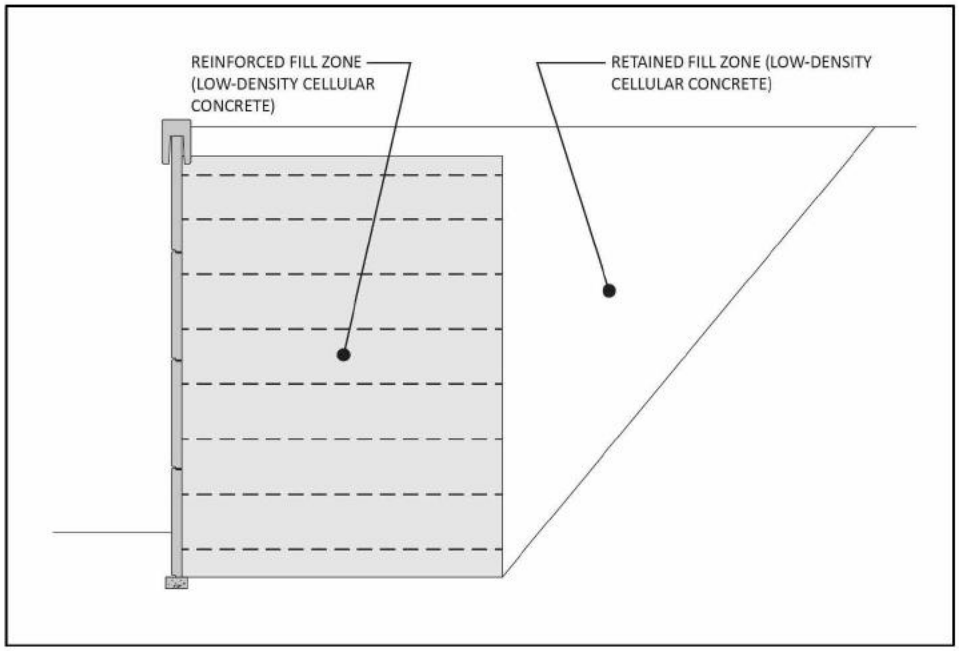


Figure 3. MSE Wall Fill Zones

When designing an MSE wall with LDCC fill, the design should follow normal MSE design procedures, including consideration of external and internal stability, tensile rupture, and pullout, to assure good performance in case of cracking or fracturing of the LDCC into a granular backfill-like mass. Shear strength testing of cured LDCC reports values exceeding typical shear strengths of granular backfills, but MSE experience and the conservative approach to MSE design, as well as the AASHTO specifications (AASHTO 2017), indicate that the design friction angle when using LDCC should be limited to $\phi = 40^\circ$ in the absence of higher values from testing.

The apparent coefficient of friction, F*, should be determined by testing of each reinforcement type, ideally at the time of project design and using the project backfill. Testing has confirmed F* values for ribbed steel strips in granular backfills are typically between 1.5 and 4.0 at the ground surface. This value decreases linearly to tan ϕ at a depth of 20 ft. and deeper (Sect.

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11.10.6.3.2, AASHTO 2017). Similarly, the F* value for low density cellular concrete must be determined from product-specific pullout tests in the selected LDCC product. Current practice, following the same provision in AASHTO, considers F* to decrease with depth from the pullout-determined value at the top to tan ϕ at 20 ft. Current practice also limits F* to 5.0 at the top to avoid over-estimating the pullout resistance and, by so doing, potentially using too few reinforcements to maintain serviceability in case of LDCC degradation.

As in the construction of any MSE wall, panels being backfilled with low-density cellular concrete must be braced to the ground (typically first row only) and to each other, or to the cured LDCC (subsequent rows, Fig. 4), to resist the fluid pressure until the LDCC has set sufficiently that bond strength and pullout resistance take over. In addition, panel joints must be covered with a separation geotextile with a small apparent opening size (AOS) to prevent material leakage, and bulkheads must be used to establish lift boundaries. Specifications regarding how far LDCC may be allowed to flow from the end of the hose to its placement position must be carefully followed to avoid mix decomposition that could result in a weaker-than-required cured layer. Since cellular concrete is a self-leveling material, each day the top lift should be finished with a slight slope away from the wall face to prevent ponding and the possibility of absorption into even closed-cell LDCC. Since each lift of LDCC must cure before installation of the next lift, a series of horizontal cold joints will be present, generally with the same vertical spacing (30 in) as the reinforcements.

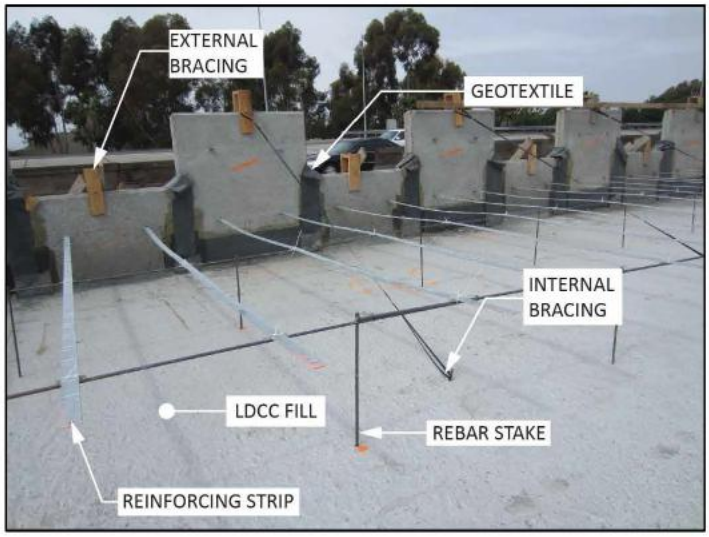


Figure 4. Support of Reinforcing Strips in Low-Density Cellular Concrete

Reinforcements should not rest atop cold joints to avoid possible reduction in pullout resistance. To control strip elevation within the cellular concrete lift, they must be supported in a relatively level position throughout their length. One method of reinforcement elevation control is shown in Figure 4 and Figure 5, where the contractor drove rows of vertical rebar stakes (which easily

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penetrate the LDCC) along lines perpendicular to the wall face and at spacing close enough to minimize reinforcement sagging. Horizontal crossbars, parallel to the wall face, are tied to the stakes to support the strips just as if they were lying on top of the backfill, while keeping them at least 6 in. above the previous day's cold joint.

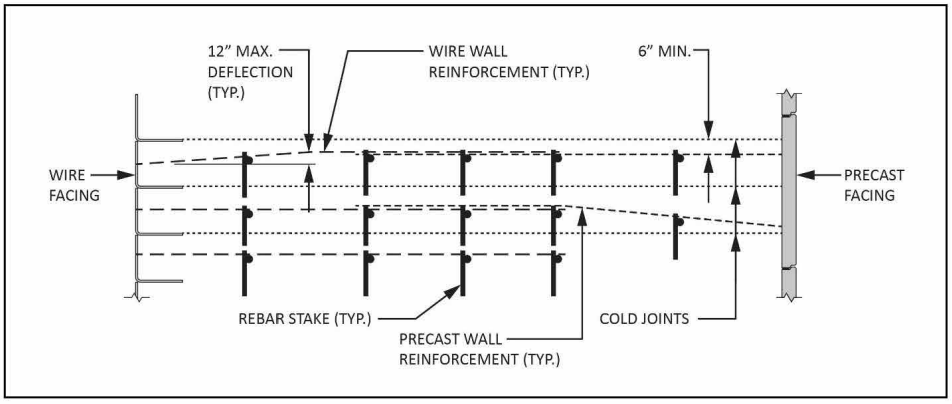


Figure 5. Reinforcing Strip Adjustments to Avoid Cold Joints/Prevent Sagging

Stake material should be chosen to avoid placing dissimilar metals in contact with each other (reinforcements are typically galvanized); epoxy-coated rebar satisfies this requirement. In addition, panel plumbness must always be maintained. When using LDCC backfill, plumbness may be achieved by a combination of external braces for bottom panels, with panel-to-panel clamping combined with internal ties anchored in the LDCC, for panels too high in the wall for bracing to the ground. Figure 4 illustrates these bracing options.

It is important to establish communication among the wall installer, the LDCC supplier, and the MSE wall designer/supplier to make sure that the wall installation process follows the wall installation guidelines. LDCC placement must never dictate wall construction procedures, i.e., it is never acceptable to stack panels two high, or to install multiple layers of reinforcements prior to a more-than-30-in.-deep LDCC pour. Only careful attention to proper placement of all components, including the low-density cellular concrete backfill, can assure safe MSE structure performance.

CASE HISTORIES

Case History 1: Illinois DOT, I-64, St. Clair County

The widening of I-64 east of St. Louis required constructing an MSE wall at the toe of an existing slope situated in the 100-year floodplain of the Mississippi River, meaning the lower portion of the wall would be periodically submerged. The Illinois DOT (IDOT) contract drawings required a permanent sheet pile wall to support the excavated embankment toe, but

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placement of the sheet piling left less than 70% of MSE wall height (L < 0.7H) available for several bottom layers of reinforcements (Fig. 6). This configuration is called a Shored MSE wall (SMSE) (FHWA, 2006), a composite system comprising a shoring wall supporting the backslope and an MSE wall constructed against and above the supported backslope. SMSEs have a design methodology that permits reinforcement lengths in the shored zone as short as 0.4-0.5H, appropriate to the limitations of this project.

The foundation beneath the proposed wall consists of fine sands and clays, with blow counts averaging less than 10 blows per foot (geotechnical investigation and design performed by IDOT). At 31 ft. tall, the structure would impose a dead load exceeding 3.7 ksf (based on 120 pcf backfill) – clearly exceeding the bearing capacity of the low blow count soil and causing unacceptable structure settlement. To address this bearing capacity deficiency, IDOT issued the contract with a requirement to use low-density cellular concrete for the MSE backfill, specifying Illinois Class IV Lightweight Cellular Concrete Fill (maximum density 37 pcf, minimum 28-day compressive strength 35 psi, resulting structure dead load of 1.1 ksf). The reduced strip lengths in the shored zone were offset by the virtually zero horizontal earth pressure due to the sheet pile wall, leaving adequate sliding resistance at the base of the wall and sufficient pullout capacity for the strips embedded in the LDCC.

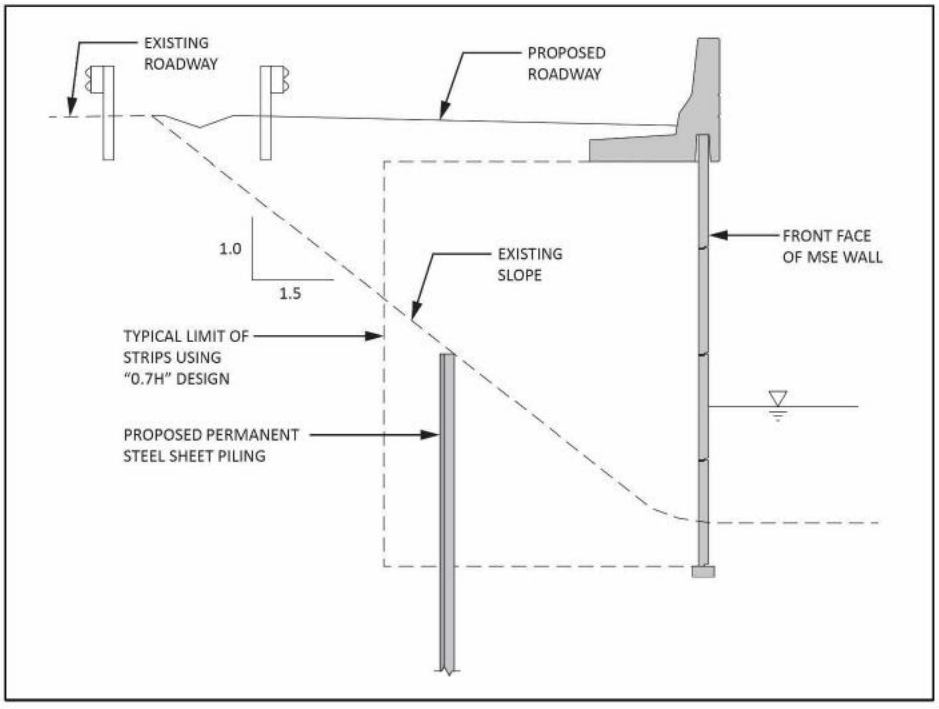


Figure 6. Existing and Proposed Roadways

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In addition to the foundation considerations, design of this MSE structure had to address the possible flooding and the potential for structure buoyancy. An MSE structure with inextensible soil reinforcements and LDCC backfill behaves as a coherent gravity mass, so the effect of buoyant conditions depends on the cell structure of the LDCC. Cell structure can be open or closed; in a closed cell LDCC mass the cells do not interconnect, making the mass essentially impermeable. Due to its light weight – in an MSE application, likely less than that of water – buoyancy is a risk to structure global stability. If open-cell LDCC is used, the cells interconnect, making the mass permeable but reducing buoyancy. The type of low-density cellular concrete to be used in an MSE structure – open or closed cell – must be determined at the time of design, and the minimum required unit weight of the material must also be specified.

At the time of this writing, there is no widely accepted standard for testing permeability of open-cell cellular concrete, but individual cellular concrete suppliers may provide guidance for testing the permeability of their products. For MSE structure design, open-cell LDCC material is assumed to be free-draining, like open-graded crushed stone. Further information on permeability in cellular concrete is available in ACI 523.1R-06 (ACI 2006).

Case History 2: I-55; Cook County, Illinois

Located at the interchange between I-55 and Rte. 41 (S. Lake Shore Drive), this Illinois DOT project required construction of multiple roadway-supporting bridge approach walls, along with construction phasing to facilitate maintenance of traffic. As in Case History 1, the owner's decision to use LDCC on this project was based on weak substrata. Soil borings at the wall locations showed fill consisting of fine sands and clays with blow counts averaging less than 10 blows per foot beginning only a few feet below ground surface. (Fig. 7), suggesting the new walls would experience significant settlement and require lengthy consolidation periods if constructed of standard 120 pcf select backfill. Using LDCC would avoid this problem, so Illinois DOT specified Class II Lightweight Cellular Concrete Fill having a density range of 24-30 pcf and a 28-day minimum compressive strength of 40 psi.

The permanent MSE walls were built using precast panels, but the temporary phasing walls used a wire mesh facing because they would eventually be buried behind fill for a later phase of the project. When backfilled with soil, wire facings are typically backed by a layer of geotextile to prevent loss of backfill. By comparison, an MSE wall using LDCC in the reinforced zone requires a geotextile with the lowest possible AOS to contain the fluid LDCC until it cures. Careful overlapping and taping of seams, along with avoiding or repairing cuts, is necessary to contain the thick liquid LDCC.

As discussed earlier in this paper, reinforcements should not be placed directly on cold joints to avoid potential reduction of pullout resistance. Best practice calls for 6 in. of cover over reinforcements, meaning LDCC lifts should follow the typical 30 in. vertical reinforcement spacing. On this project, however, the wire-faced phasing walls had an 18 in. reinforcement vertical spacing so, in preplanning the sequencing of LDCC pours, the contractor had to compare the anticipated top elevation of an LDCC lift with the elevations of the strips from both the permanent and the temporary walls. Some reinforcements had to be deflected vertically to avoid them being in the plane of a cold joint (Fig. 5).

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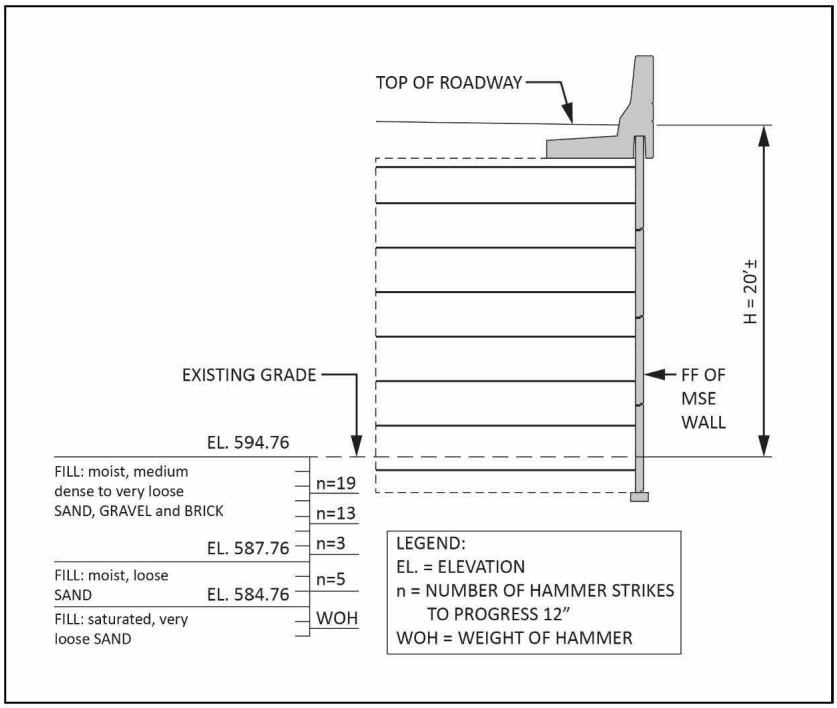


Figure 7. New Wall atop Weak Substrata

As occurs on this project, many MSE walls support a roadway on top of the wall. When using granular material, the top layer of backfill can be finished on a slope to parallel the roadway grade, but this cannot be accomplished with the fluid backfill that is LDCC. Instead, bulkheads are required to accomplish elevation changes that approximate the roadway grade, producing a stair-stepped top to the backfill material (Fig. 8). This stair-stepped condition leaves some MSE reinforcements exposed or with insufficient LDCC cover, so the top backfill layer should be a granular fill meeting all AASHTO MSE specifications (AASHTO 2017, Sect. 11.10.6.4.2a). A secondary benefit of this granular backfill layer is the ability to grade the top of this fill to follow roadway grade, simplifying placement of subbase, base course and pavement.

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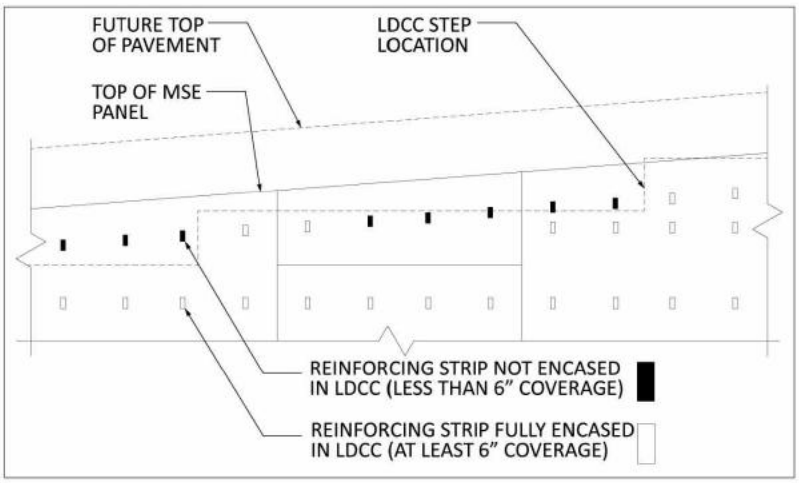


Figure 8. Stair-Step Effect at Top of LDCC

Case History 3: Merchants Bridge Replacement over Mississippi River, St. Louis, MO

Merchants Bridge, constructed in 1890, is a major railroad artery across the Mississippi River, averaging 32 freight and passenger train crossings daily. Due to its age and deteriorated structural condition, the bridge has restrictions on speed, clearance and weight. Replacement spans for the river crossing will be floated into place, while onshore approach trestle downtime will be minimized by leaving the trestle in place and enclosing it in a retained fill comprising back-to-back MSE walls located just outside the existing trestle legs (Fig. 9). With trains still running, the MSE walls will be backfilled with (flowable) low-density cellular concrete to an elevation just below the top of the trestle, then trains will be stopped for removal of the track and upper portion of the trestle. L-walls, sub-ballast, ballast and new railroad tracks will complete the rehabilitation and put the reconstructed former trestle back in service, minimizing interruption of train traffic until the river spans are replaced.

With maximizing trestle availability being a major factor in design, the installation speed, small footprint and low foundation loading of Mechanically Stabilized Earth technologies made an MSE structure a clear choice for the trestle-replacement wall system. Traditional MSE construction, backfilled with granular material, would require a slow, labor-intensive process due to the limited space around the existing trestle supports, while achieving specified compaction would require use of hand-operated compaction equipment and backfill placement in thin lifts. The use of low-density cellular concrete as the MSE backfill will not only minimize the surcharge applied to the existing trestle foundations, it will also further reduce reconstruction time since the LDCC backfill will flow into every space and requires no compactive effort, only curing time.

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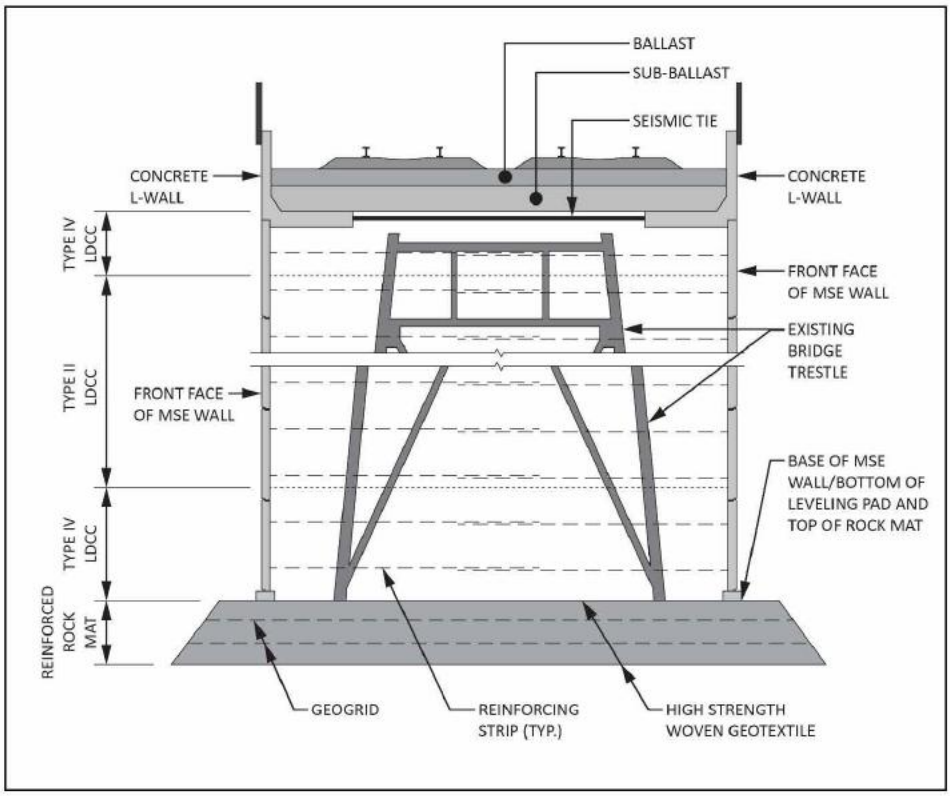


Figure 9. Section through LDCC MSE Walls

Designers chose to use two different classes of low-density cellular concrete in this MSE structure. Missouri DOT Type IV (42 pcf, 220 psi) was used for the bottom of the MSE fill (providing greater strength and density over the rock base) and for the top of the MSE fill just below the L-walls and ballast section (directly beneath the applied loads). The bulk of the structure, requiring less strength, was Missouri Type II, at 30 pcf and 120 psi. Design loading is Cooper E-80, roughly equivalent to a 15.5 ft high soil surcharge.

RECOMMENDATIONS WHEN CONSIDERING USE OF LDCC IN MSE WALLS

Every MSE structure is custom-designed to meet the unique needs of a specific project. For those requiring low-density cellular concrete backfill, similar customization must go into specifying or selecting the LDCC product. Some owners, such as the DOTs of Illinois and Missouri cited above, have their own LDCC specifications and pre-select the LDCC class to be used in each structure. For first-time users or others who do not maintain a pre-established specification, a

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performance-based specification can be a good choice. This type of specification puts responsibility for detailed design, material selection and correct installation on the contractor and the LDCC supplier, relieving the owner of the need to research and modify specifications for every project.

Regardless of the specification type used, the owner retains overall responsibility for the project. Therefore, the owner (or owner's engineer) must understand and consider:

- Design Parameters. Specify the parameters for design of MSE structures.
- Material Properties. Select properties appropriate to project needs, such as LDCC compressive strength when planning for heavy loads such as Cooper E-80, and geotextile AOS for joints and wire facing.
- Construction Loads. Though temporary, construction loading may exceed design loads and must be considered in the design.
- Buoyancy. Evaluate buoyancy of flood-susceptible structures.
- Cracking. Evaluate cracking potential of the LDCC MSE mass due to site or loading conditions.
- Inspection. Owners must always be (or hire someone who is) knowledgeable regarding material properties and installation methods for LDCC, to be sure they receive the finished project they expect.
- Communication. The owner, the engineer, the MSE supplier, the LDCC supplier and the contractor must communicate their respective plans and requirements, to assure practicality of the design and realistic expectations regarding LDCC daily production, installation procedures and sequencing, and lift thickness relative to MSE dimensions.

CONCLUSION

MSE walls are routinely constructed over soft foundation soils, typically using granular backfill. Where additional bearing pressure reduction is required to reduce settlement and improve structure performance, low-density cellular concrete may be used as MSE backfill in place of granular material. LDCC product selection must consider the material's unit weight and compressive strength, while normal MSE design standards must be followed. Modifications to typical MSE installation procedures address the fluid nature of LDCC by sealing joints, use of bulkheads, and supporting reinforcements to make sure they do not rest on cold joints. Attentive inspection by owners can assure correct construction practices that will lead to satisfactory long-term performance. Suppliers and owners must work together to develop more test data, to document experiences, and to build a body of knowledge about LDCC backfill in MSE walls.

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REFERENCES

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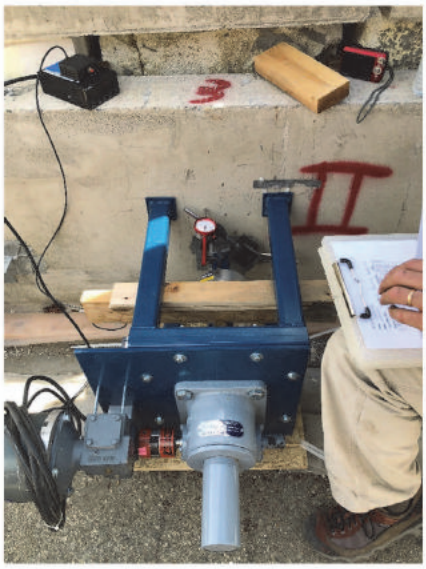
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Pullout Testing of Reinforced Earth® High Adherence Steel Strips in Low Density Cellular Concrete





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RECo HA Strips in LDCC Pullout Testing Program

Executive Summary

Low Density Cellular Concrete (LDCC) is a viable alternative backfill for Mechanically Stabilized Earth (MSE) walls. LDCC may be used for MSE backfill due to inadequate foundation bearing capacity, or to mitigate foundation settlements by providing a lighter weight compared to conventional fill. Since the 1980's, many successful projects have been built with Reinforced Earth® walls backfilled with LDCC.

LDCC has physical properties that fall between conventional fills (sands, gravels, etc.) and concrete. While certain parameters are strictly a function of the LDCC physical properties (Unit Weight, Compressive Strength, Permeability, Shear Strength), other design parameters (soil reinforcement interaction) are dependent on soil reinforcement type.

This memo provides a summary of the results of a program of tests prepared by Cell Crete and The Reinforced Earth Company. Supervision of casting, preparation of the samples and testing was performed by TRI Environmental.

The tests followed the procedure outlined in ASTM D6706 in individually prepared boxes with a minimum coverage of 12". The program tested the Reinforced Earth Company (RECo) High Adherence (HA) Steel Strip embedded in LDCC Class II and IV (Provided by Cell-Crete). Samples were tested at 24 hours after pouring and 28 days after pouring with the samples embedded in the LCC monolithic block and with the HA strip placed at a cold joint.

The test results from this program were in good agreement with the 1988 Tests performed by Law Engineering for The Reinforced Earth Company (report is included herein).

Apparent Coefficient of Friction f^* between Ribbed Strips and LDCC

The results are summarized in the following graphs with recommended values for the apparent coefficient of friction, f^*, between the ribbed reinforcing strips and LDCC.

Based on the data obtained in the test program, a conservative design value of f^* (for service I, strength I design) may be taken as 4.0 from the ground surface to $\tan\phi$ at depth of 20 ft (500 psf+/- for Class II and 600 psf+/- for Class IV).

For construction loading design case or Extreme Event design case, f^* may be taken as 8.0 from the ground surface to $\tan\phi$ at depth of 20 ft (500 psf +/- for Class II and 600 psf +/- for Class IV).

The design bilinear envelope, which is typical for granular fills and ribbed steel strips, transitions from f^* at ground surface to tan Φ at a depth of 20 ft. However, the 20 ft depth for granular fills equates to an overburden pressure of about 2400 psf and is generally regarded as a function of dilation of granular fill. Since LDCC does not dilate, and the unit weight of the material is 25% to 40% of conventional fills, the lower bound for the apparent coefficient of friction (f^*) could be much deeper than 20 ft. Additional testing is necessary to determine if a value greater than tan Φ should be used for overburdens greater than 600 psf.

Shear Strength of LDCC for MSE Design

The recommended friction angle for design (using conventional MSE design methods) is 40 degrees, but may be less depending on the applied overburden pressure. Cohesion is present in LDCC, but is typically ignored for design. Based on the data provided by Tiwari [1], a Shear Strength of 40º is applicable for most MSE applications:

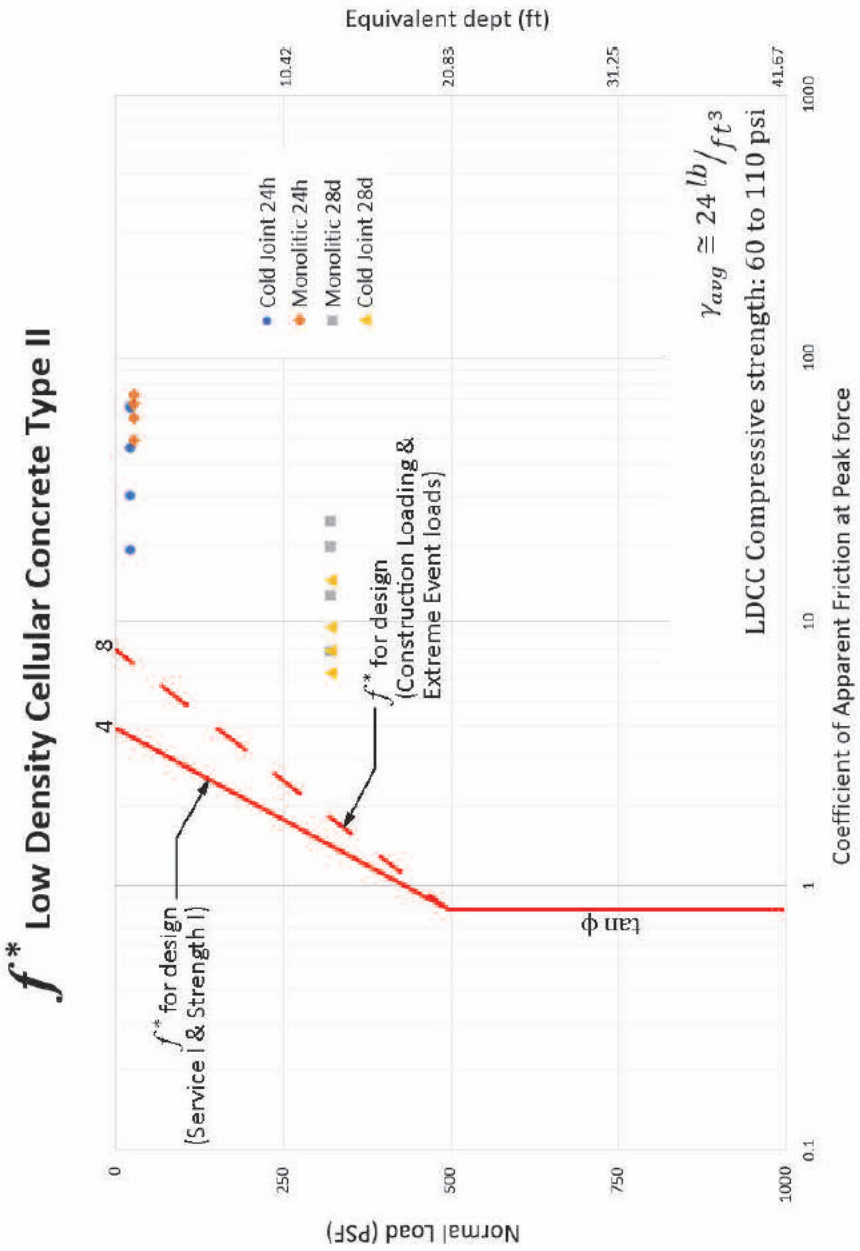
LDCC Class	Normal Stress (psf)	Shear Strength for Design (Φ)
II	<8300	40°
IV	<8300	40°
II	>8300	35°
IV	>8300	35°

Note: 8300 psf equates to approximately 276 ft of LDCC Class IV. Conditions that might result in higher normal stress include: Heavy Rail Loading and Abutment Loads.

In place unit weight of LDCC

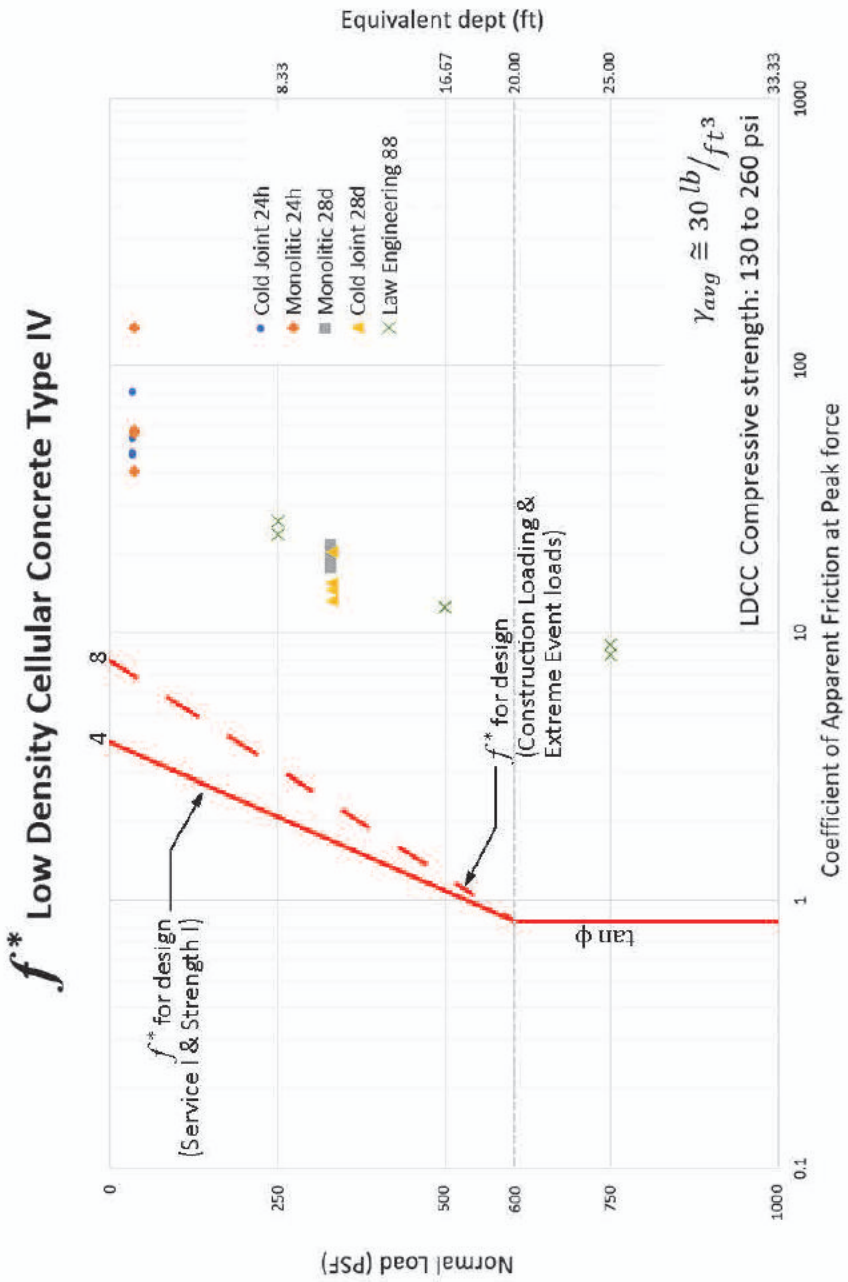
The unit weight of in-place LDCC varies depending on mix proportions, air content, etc. A typical variance of +/- 5 pcf has been observed. As such, design calculations for critical conditions such as foundation sliding and buoyancy effects should consider the effects of heavier and/or lighter unit weights than the design value.

LDCC Class	Design Unit Weight (pcf)	Typical Unit Weight Range (pcf)	28 day Compressive Strength Range (psi)
II	24	19 to 29	60 to 110
IV	30	27 to 33	130 to 260



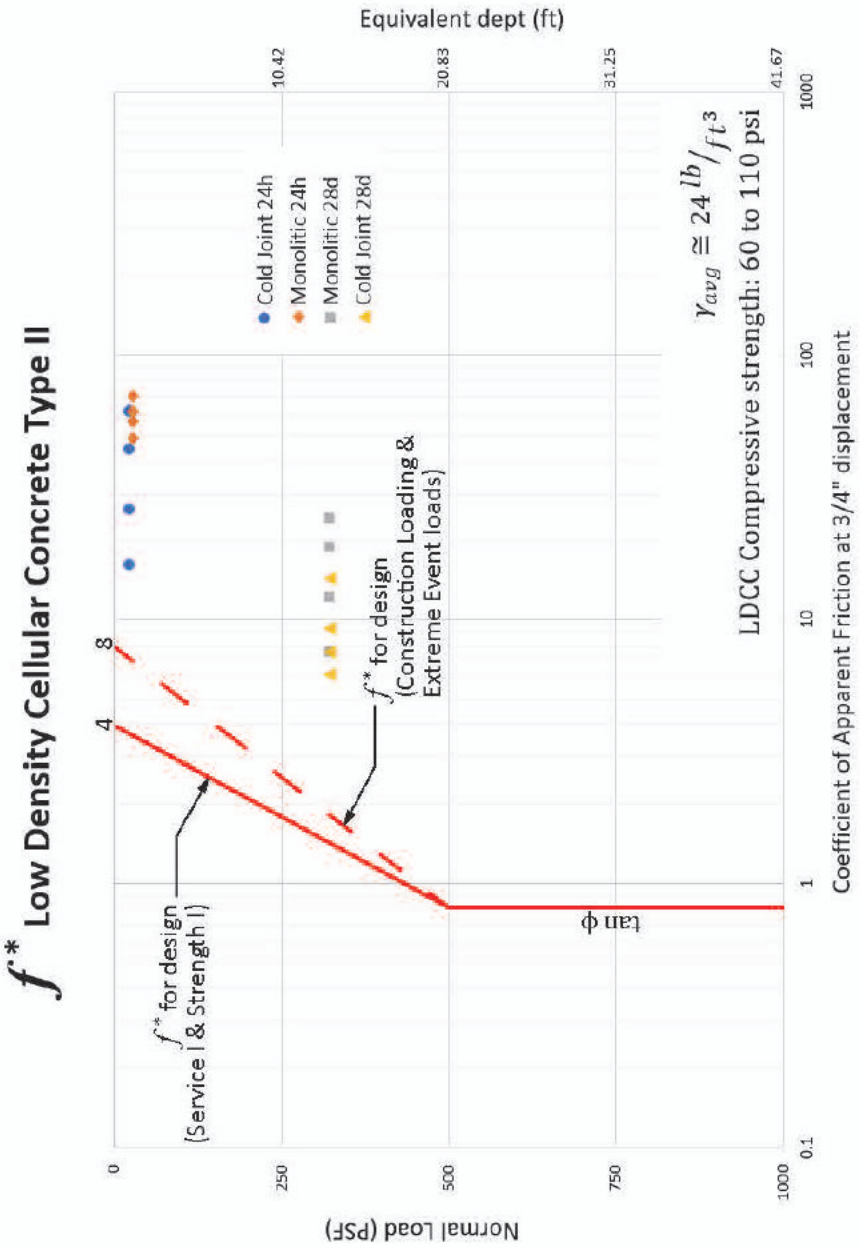
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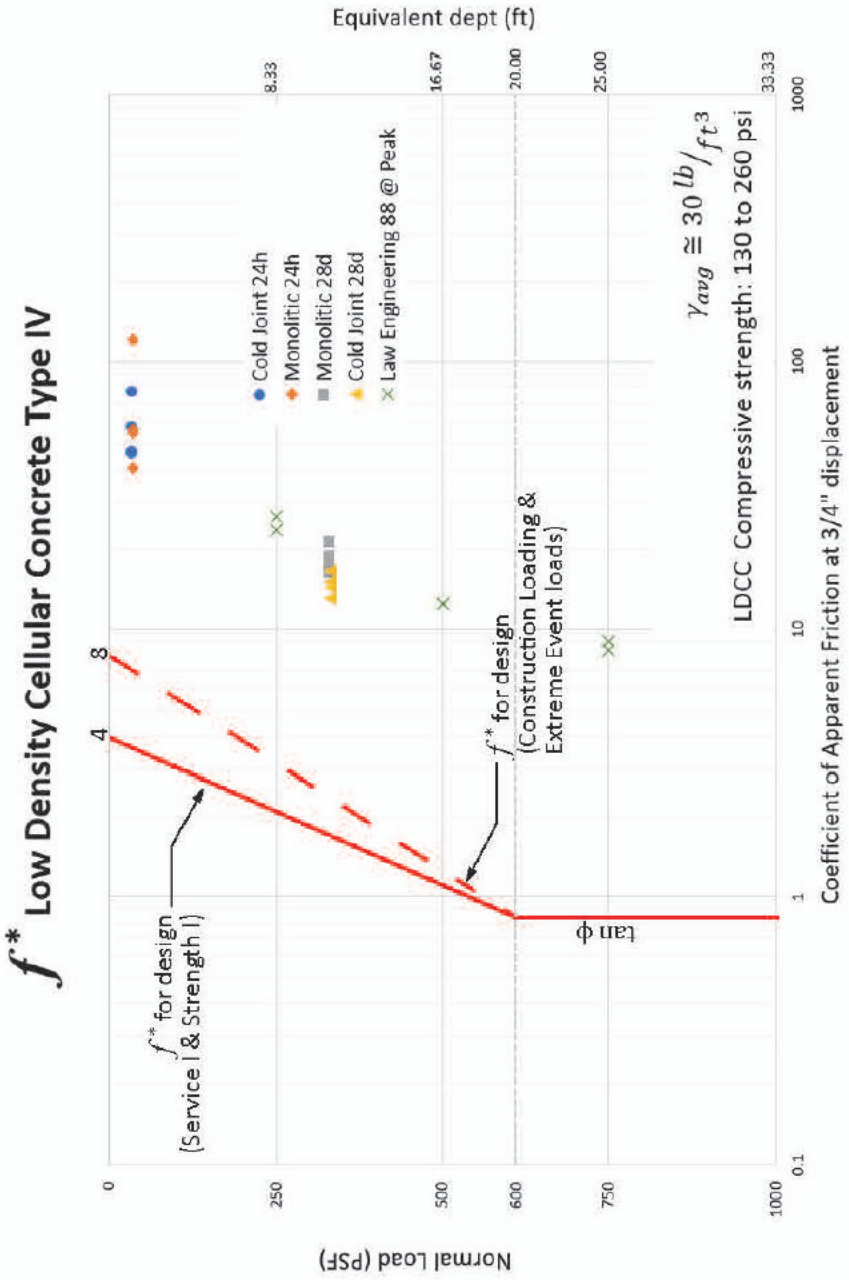
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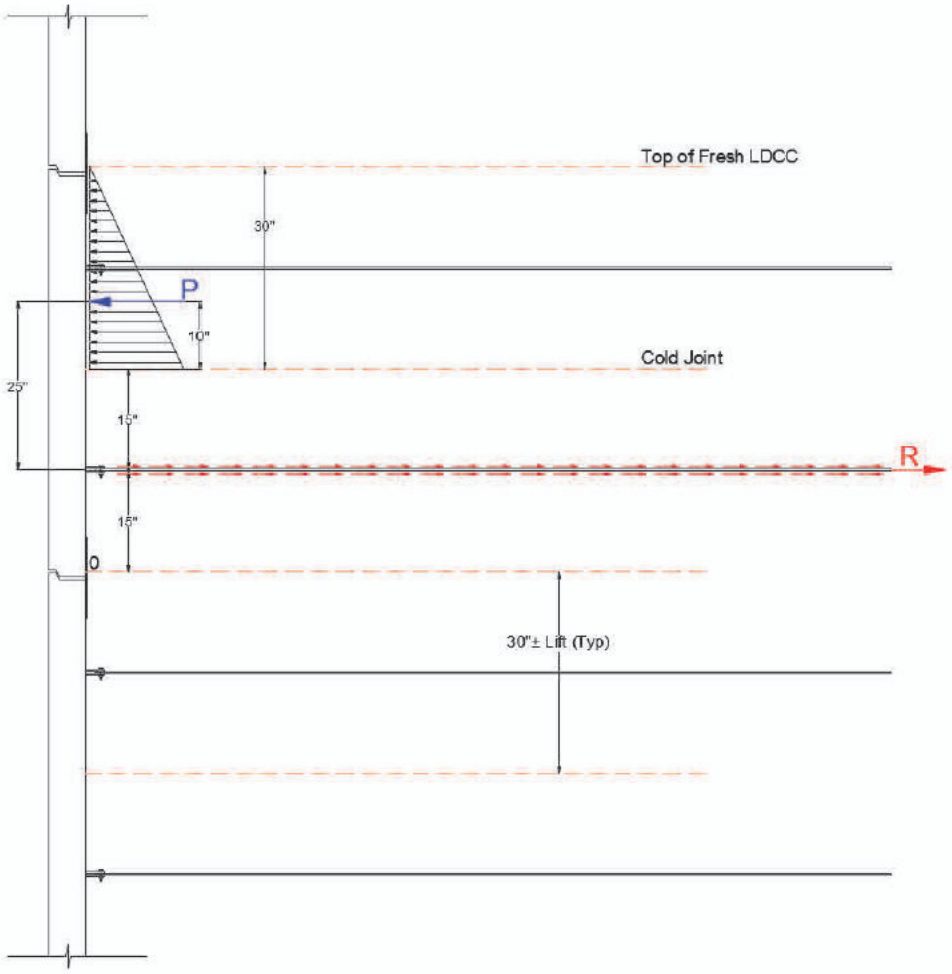
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f* Low Density Cellular Concrete Type IV
Equivalent dept (ft)
8.33
16.67
20.00
25.00
33.33
Cold Joint 24h
Monolitic 24h
Monolitic 28d
Cold Joint 28d
Law Engineering 88 @ Peak
f'* for design
(Service I & Strength I)
f'* for design
(Construction Loading &
Extreme Event loads)
tan ϕ
$\gamma_{avg} \cong 30\ ^{lb}/_{ft^3}$
LDCC Compressive strength: 130 to 260 psi
Coefficient of Apparent Friction at 3/4" displacement
Normal Load (PSF)
0
250
500
600
750
1000
8
4
1
0.1
10
100
1000

Construction Loading

***Note**: Design should consider short term loading condition of 30" fluid head acting on panel with a semi-cured lift below:*



Applied Loading and Resistance by HA Strips in LDCC

$$P = \frac{1}{2}\gamma Z^2$$

$$P = \frac{1}{2}\ x\ (24PCF) x\ 2.5^2$$

$$P = 75\ ^{lb}/_{ft}$$

$$M_0 = P \times 3.33' - R \times 1.25' = 0$$

$$R = \frac{P \times 3.33'}{1.25'} = \frac{75 \times 3.33'}{1.25'} = 200\ ^{lb}/_{ft}$$

Resistance of HA Strip in fresh LDCC:

f of semi cured lift = 10 for analysis purposes – See summary charts above, and test reports attached for more information.*

For one REW/SQR (4.92'x4.92' nominal) A-4 panel

$$R_{min} = 200\ ^{lb}/_{ft} \times 4.92' = 984lb$$

Check 8 ft reinforcement length

$$R_8 = 2b\gamma Z f^* l_e$$

$$R_8 = 2\ x\ 0.166'\ x\ 24\ ^{lb}/_{ft^3}\ x\ 1.25'\ x\ 10\ x\ 8ft = 637\ \text{lb}$$

$$R_8 = 637lb\ \text{x}\ 2\ \text{RS over}\ 4.92' = 1275\ \text{lb resistance}$$

$$SF_{(pullout)} = 1275\ ^{lb}/_{984\ lb} = 1.30\ \text{OK for temporary condition}$$

For one RLA (4.92'x9.84' nominal) A-6 panel

$$R_{min} = 200\ ^{lb}/_{ft} \times 9.84' = 1968\ lb$$

Check 12 ft reinforcement length

$$R_{12} = 2b\gamma Z f^* l_e$$

$$R_{12} = 2\ x\ 0.166'\ x\ 24\ ^{lb}/_{ft^3}\ x\ 1.25'\ x\ 10\ x\ 12ft = 1195\ \text{lb}$$

$R_{12} = 1195\ lb$ x 3 RS over 9.84′ = 3585 lb resistance

$SF_{(pullout)} = {}^{3585\ lb}/_{1968\ lb} = 1.80$ OK for temporary condition

For one RLA (4.92'x9.84' nominal) A-4 panel

$R_{min} = 200\ {}^{lb}/_{ft} \times 9.84' = 1968\ lb$

Check 16 ft reinforcement length

$R_{16} = 2b\gamma Z f^* l_e$

$R_{16} = 2\ x\ 0.166'\ x\ 24\ {}^{lb}/_{ft^3}\ x\ 1.25'\ x\ 10\ x\ 16ft = 1594$ lb

$R_{16} = 1594\ lb$ x 2 RS over 9.84′ = 3187 lb resistance

$SF_{(pullout)} = {}^{3187\ lb}/_{1968\ lb} = 1.60$ OK for temporary condition

RS Length	N_{min} (RLA)	REW/SQR
8'	A-8	A-4
10'	A-6	A-4
16'	A-4	A-3

Note, a similar check was performed with a unit weight of 40 pcf, and the above reinforcing strip length and density (N) recommendations were validated.

Enclosures:

Appendix I:

TRI Report: MAPP-TR-1807-1 "Pullout of RECo High-Adherence Reinforcing Strips from Light-Weight Cellular Concrete Fill" Issued July 3, 2018 (90 pages)

References:

(1) Tiwari, Almera, Maw, Cole, Villegas, Palmerson (2017). Technical Note: *"Mechanical Properties of Lightweight Cellular Concrete for Geotechnical Applications"*. Journal of Materials in Civil Engineering, 2017, 29(7):06017007

(2) American Concrete Institute (2006). *Guide for Cast-in-Place Low-Density Cellular Concrete*. ACI 523.1R-06

APPENDIX I

TRI Report: MAPP-TR-1807-1 "Pullout of RECo High-Adherence Reinforcing Strips from Light-Weight Cellular Concrete Fill" Issued July 3, 2018 (90 pages)

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TRI-MAPP Report

MAPP-TR 1807-1
Pullout of RECo High-Adherence Reinforcing Strips from Light-Weight Cellular Concrete Fill

Issued July 2018

Final Report

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July 3, 2018

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Coefficients of Apparent Friction for Pullout Tests Conducted at 28 Days

Test Number	Concrete Type	Casting Method	$\sigma_n^{\dagger}$ (psi)	$P_{0.75"}$ (lb.)	P_{max} (lb.)	$f^*_{0.75"}$	f^*_{max}
1	II	Monolithic	2.23	8,100	8,300	18.9	19.4
2	II	Monolithic	2.23	10,406	10,407	24.3	24.3
3	II	Monolithic	2.23	3,270	3,340	7.6	7.8
4	II	Monolithic	2.23	5,220	5,404	12.2	12.6
5	IV	Monolithic	2.29	9,408	9,429	21.4	21.5
6	IV	Monolithic	2.29	8,334	8,439	19.0	19.2
7	IV	Monolithic	2.29	7,806	7,896	17.8	18.0
8	IV	Monolithic	2.29	7,199	7,725	16.4	17.6
9	II	Cold Joint	2.23	3,218	3,348	7.5	7.8
10	II	Cold Joint	2.23	2,644	2,744	6.2	6.4
11	II	Cold Joint	2.23	3,922	4,043	9.2	9.5
12	II	Cold Joint	2.23	6,099	6,109	14.3	14.3
13	IV	Cold Joint	2.30	7,291	8,907	16.5	20.2
14	IV	Cold Joint	2.30	6,642	6,810	15.1	15.4
15	IV	Cold Joint	2.30	6,305	6,460	14.3	14.6
16	IV	Cold Joint	2.30	5,800	5,835	13.1	13.2

† determined using unit weight of material above the reinforcing strip at 28 days, plus 300 psf added confinement

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Coefficients of Apparent Friction for Pullout Tests Conducted at 24 Hours

Test Number	Concrete Type	Casting Method	σ_n† (psi)	$P_{0.75"}$ (lb.)	P_{max} (lb.)	$f^*_{0.75"}$	f^*_{max}
17	II	Monolithic	0.20	2,307	2,507	61.4	66.7
18	II	Monolithic	0.20	1,829	1,838	48.6	48.9
19	II	Monolithic	0.20	2,116	2,240	56.3	59.6
20	II	Monolithic	0.20	2,630	2,734	69.9	72.7
21	IV	Monolithic	0.23	3,450	3,589	77.2	80.4
22	IV	Monolithic	0.23	2,563	2,418	57.4	54.1
23	IV	Monolithic	0.23	2,081	2,103	46.6	47.1
24	IV	Monolithic	0.23	2,044	2,124	45.8	47.6
25	II	Cold Joint	0.16	1,876	1,982	62.3	65.8
26	II	Cold Joint	0.16	1,340	1,398	44.5	46.4
27	II	Cold Joint	0.16	489	566	16.2	18.8
28	II	Cold Joint	0.16	792	920	26.3	30.5
29	IV	Cold Joint	0.25	5,802	6,661	120.5	138.4
30	IV	Cold Joint	0.25	1,946	1,950	40.4	40.5
31	IV	Cold Joint	0.25	2,622	2,714	54.5	56.4
32	IV	Cold Joint	0.25	2,714	2,780	56.4	57.8

† determined using unit weight of material above the reinforcing strip at 28 days

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Introduction

TRI Environmental, Inc. conducted field pullout tests on 2-in. wide, 48 in. long, galvanized steel reinforcing strips manufactured by The Reinforced Earth Company (RECo) embedded in Cell-Crete light-weight concrete. Testing conditions and protocols were specified by RECo, as outlined in Appendix A.

Test Setup and Materials

Eight separate concrete panels, each having four repeat test boxes, were constructed by Cell-Crete, Inc. to accommodate pullout tests of RECo reinforcing strips (see Figure 1). Each test box was built to accommodate a single reinforcing strip embedded in a volume of Cell-Crete approximately 2 ft. wide by 2 ft. tall by 5 ft. long (detailed drawings of setup are provided in Appendix A).

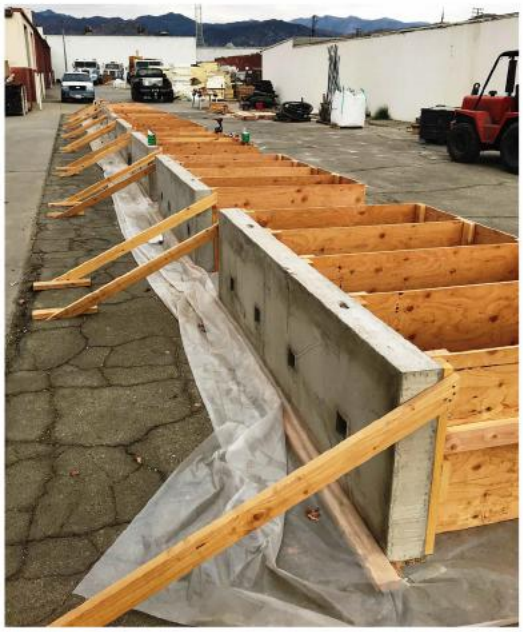


Figure 1: Layout of test boxes prior to filling with Cell-Crete.

Three test parameters were varied to determine their effect on the pullout strength of the RECo reinforcing strips: 1) unit weight/strength of the Cell-Crete lightweight fill (Type II or Type IV), 2) reinforcing strip placement method (monolithic or cold-joint), and 3) age of Cell-Crete at the day of testing (28 days or 24 hours). A summary of the test program is outlined in Table 1. Test boxes associated with Panels 1, 2, 5 and 6 were cast monolithically, meaning that the entire reinforcing strip was embedded in the Cell-Crete during a single pour. Test boxes associated with Panels 3, 4, 7 and 8 were cast as a cold-

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joint where the Cell-Crete beneath the reinforcing strip was cast first, and the Cell-Crete above the reinforcing strip was cast 24 hours later.

Table 1: Summary of Test Program

Test Panel	Test Strips	Cell-Crete Class	Casting Method	2018 Casting Dates	Age at Testing
1	1-4	II	Monolithic	3/13	28 days
2	5-8	IV	Monolithic	3/13	28 days
3	9-12	II	Cold-Joint	3/13-3/14†	28 days
4	13-16	IV	Cold-Joint	3/13-3/14†	28 days
5	17-20	II	Monolithic	4/10	24 hours
6	21-24	IV	Monolithic	4/10	24 hours
7	25-28	II	Cold-Joint	4/10-4/11†	24 hours
8	29-32	IV	Cold-Joint	4/10-4/11†	24 hours

† bottom half of box filled the first day, top half was filled the second day, testing was done 24 hours after second pour

Each test box accommodated a single RECo reinforcing anchor strips 2 in. wide and 48 inches long cast into Cell-Crete (see Figure 2). The embedment lengths of the reinforcing strips were measured from the trailing end of the reinforcing strip to the HDPE flaps that formed the sleeve at the front of the box (sleeve is evident in Figure 2 near the top of the photo). Strips were held in vertical position by resting on Styrofoam blockouts at the front of the box and a piece of rebar near the end of the strip (rebar is evident in Figure 2 near the bottom of the photo). A small, thin piece of HDPE was placed between the RECo test strip and the rebar to keep it from snagging on the rebar during pullout.

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Figure 2: Photo of RECo reinforcing strips prior to casting them in Cell-Crete.

The test boxes were filled with Cell-Crete having two different unit weights (referred to as Type II and Type IV), where Type II has a lower unit weight/strength than Type IV. The unit weights and average compressive strengths of the Cell-Crete at 24 hours, 7 days, and 28 days after casting are summarized in Table 2 for the various test setups and pour dates. A report documenting the casting process in March and the unit weight and compressive strength data is included in Appendix B, as reported by California Testing and Inspections, Inc. Multiple samples of Cell-Crete were taken during casting. Samples of Cell-Crete were taken directly from the batching equipment.

Cell-Crete was mixed on site using a mobile batch plant. Cell-Crete was transported from the mobile batch plant to the test boxes using a hose (see Figure 3). The monolithic samples boxes were filled until they were 2 ft. deep (RECo reinforcing strip was located at 1 ft. depth). Cold joints were cast by first filling the test boxes to 1 ft. deep (just beneath the RECo reinforcing strip) and then filling the remaining 1 ft. depth above the reinforcing strip 24 hours later. During casting of the first set of samples in March, some of the strips were left in place with the Cell-Crete just touching the bottom side of the reinforcing strip (see Figure 4). During casting in April, however, the strips rested on the cast material while the remaining Cell-Crete was cast above it. Sometimes there was a small gap beneath the reinforcing strip, as shown in Figure 5.

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Table 2: Summary of Cell-Crete Unit Weights and Compressive Strengths

Location†*	Cell-Crete Class	Cure Time	Unit Weight‡ (lb/ft³)	Compressive Strength* (psi)	COV (%)
Panel 1M	II	24 hours	23.6	36.0	3.9
Panel 1M	II	7 days	22.8	55.0	6.4
Panel 1M	II	28 days	20.9	104.7	15.6
Panel 2M	IV	24 hours	35.9	103.8	5.9
Panel 2M	IV	7 days	32.7	164.1	10.0
Panel 2M	IV	28 days	29.6	273.2	7.4
Panel 3B	II	24 hours	33.2	97.2	1.8
Panel 3B	II	7 days	27.6	141.2	3.8
Panel 3B	II	28 days	29.3	222.4	10.5
Panel 3T	II	24 hours	24.0	27.8	17.1
Panel 3T	II	7 days	20.0	55.0	21.3
Panel 3T	II	28 days	20.4	90.9	14.1
Panel 4B	IV	24 hours	34.1	120.5	3.2
Panel 4B	IV	7 days	31.5	179.0	8.3
Panel 4B	IV	28 days	30.4	227.7	14.9
Panel 4T	IV	24 hours	34.6	107.2	8.4
Panel 4T	IV	7 days	29.9	176.9	20.4
Panel 4T	IV	28 days	30.8	264.6	5.3
Panel 5M	II	24 hours	28.2	22.4	38.9
Panel 5M	II	7 days	23.8	113.5	12.5
Panel 5M	II	28 days	24.2	117.2	11.3
Panel 6M	IV	24 hours	33.5	49.6	6.2
Panel 6M	IV	7 days	26.7	129.8	9.8
Panel 6M	IV	28 days	27.2	131.7	3.5
Panel 7B	II	24 hours	25.6	28.0	24.3
Panel 7B	II	7 days	19.7	89.2	12.6
Panel 7B	II	28 days	21.9	89.8	17.4
Panel 7T	II	24 hours	22.6	14.5	30.3
Panel 7T	II	7 days	18.2	37.2	14.8
Panel 7T	II	28 days	18.8	60.7	21.1
Panel 8B	IV	24 hours	37.3	57.6	3.1
Panel 8B	IV	7 days	29.0	131.2	6.8
Panel 8B	IV	28 days	31.4	200.9	7.1
Panel 8T	IV	24 hours	36.1	113.3	11.3
Panel 8T	IV	7 days	32.1	171.9	8.2
Panel 8T	IV	28 days	33.0	205.2	17.5

†M indicates monolithic pour of the concrete cast around the reinforcing strips
B indicates bottom half of the concrete cast for reinforcing strips embedded at a cold-joint
T indicates top half of the concrete cast for reinforcing strips embedded at a cold-joint
‡ based on one or two measurements from each sample lot
* based on 5 or 6 individual compression tests from each sample lot

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Figure 3: Casting of Cell-Crete concrete.



Figure 4: Bottom half of cold joint with RECo reinforcing strip slightly embedded in surface.

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Figure 5: Bottom half of cold joint in place with RECo reinforcing strip slightly above surface.

Test Equipment

The pullout equipment consisted of a motor driven screw actuator, a load cell with digital readout, a reaction frame, and a dial gauge. The load frame rested against the face of the concrete fascia panel and reacted against it during testing. The load cell was connected to the load actuator through a threaded yoke. Likewise another threaded yoke on the opposite side of the load cell was attached to the protruding end of the RECo anchor strip. A variable-speed motor was used to precisely control the speed of the pullout. The system was calibrated to apply a constant displacement of 0.04 in./min. An analog dial gauge was used to measure displacement of the reinforcing strip during pullout. A photo of the pullout equipment is shown in Figure 6.

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Figure 6: Pullout device.

Test Procedure

The pullout tests were conducted in general accordance with the procedure outlined in ASTM D6706 "Standard Test Method for Measuring Geosynthetic Pullout Resistance in Soil." The steps employed to conduct the pullout tests on the embedded reinforcing strips are listed below, along with any modifications. Testing began with Panel #1 and proceeded through Panel #8.

1. A vertical confinement of 300 psf was placed atop test samples tested 28 days after casting. This normal load was applied using concrete panels that rested on blocks of Styrofoam (see Figure 7). The normal load was placed at least 15 minutes prior to pullout testing.
2. The pullout apparatus was mounted to a wheeled cart that allowed it to be rolled into place and positioned directly horizontal to the protruding end of the reinforcing strip.
3. The pulling yoke was connected to the reinforcing strip using a $^{9}/_{16}$ in. bolt and the motor was turned on to draw the feet of the reaction frame toward the concrete panel face.
4. Just prior to the feet of the reaction frame touching, the pullout device was turned off, and final leveling and positioning of the pullout apparatus was made to ensure that the reinforcing strip was pulled from the Cell-Crete without placing any off-axis forces on it. Initial loads on the strip were in the range of 25 to 75 lbf.

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5. The dial gauge was attached and zeroed. Displacement of the reinforcing strip was measured from the front of the exposed tab. It was assumed that the steel strip was, for all practical purposes, inextensible, so that displacements measured at the embedded end would be the same as those measured at the exposed end
6. Pullout was initiated at 0.04 in. per minute. The reinforcing strip was pulled until it reached a maximum load. Pullout was discontinued when displacement of the reinforcing strip reached 1.0 to 2.0 in.
7. Load and displacement were recorded at various intervals – more frequently at the beginning of pullout.

During testing of the first reinforcing strip (Test Strip 1) it was discovered that the pullout apparatus caused the protruding metal tab to rotate under load. This was caused by the screw mechanism associated with the pullout device, and likely had a negligible effect on the results. Nevertheless, this issue was remedied for the remaining tests by shimming between the loading yoke and the test frame to prevent rotation. The induced torque caused the end of the tab to be permanently distorted, as shown in Figure 8.



Figure 7: Test setup showing normal confinement.

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Figure 8: Torsional distortion of Test Strip 1 after pullout.

Test Results

Pullout tests were run on test specimens embedded in Cell-Crete cured for 28 days and 24 hours, the results of which are summarized in Table 3 and Table 4, respectively. Graphs of pullout resistance versus displacement are provided in Appendix C.

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Table 3: Summary of Results for Pullout Tests Conducted at 28 Days

Test Number	Concrete Type	Casting Method	Initial Peak		Residual Peak	
			Load (lb.)	Disp. (in.)	Load (lb.)	Disp. (in.)
1	II	Monolithic	NR	NR	8,300	0.848
2	II	Monolithic	6,450	0.090	10,407	0.700
3	II	Monolithic	3,279	0.080	3,340	0.425
4	II	Monolithic	4,290	0.050	5,404	0.900
5	IV	Monolithic	5,964	0.175	9,429	0.950
6	IV	Monolithic	7,468	0.100	8,439	0.850
7	IV	Monolithic	7,325	0.200	7,896	1.000
8	IV	Monolithic	5,790	0.125	7,725	0.800
9	II	Cold Joint	2,012	0.025	3,348	0.950
10	II	Cold Joint	1,876	0.020	2,744	1.000
11	II	Cold Joint	3,823	0.030	4,043	0.500
12	II	Cold Joint	4,290	0.035	6,109	0.800
13	IV	Cold Joint	---	---	8,907	0.400
14	IV	Cold Joint	---	---	6,810	0.350
15	IV	Cold Joint	---	---	6,460	1.000
16	IV	Cold Joint	---	---	5,835	0.400

--- no initial peak
NR = not recorded

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Table 4: Summary of Results for Pullout Tests Conducted at 24 Hours

Test Number	Concrete Type	Casting Method	Initial Peak		Residual Peak	
			Load (lb.)	Disp. (in.)	Load (lb.)	Disp. (in.)
17	II	Monolithic	1,416	0.020	2,507	0.400
18	II	Monolithic	1,605	0.02	1,838	0.800
19	II	Monolithic	1,787	0.022	2,240	0.375
20	II	Monolithic	2,502	0.032	2,734	0.500
21	IV	Monolithic	2,116	0.023	3,589	0.500
22	IV	Monolithic	2,701	0.030	2,418	0.400
23	IV	Monolithic	2,359	0.027	2,103	0.375
24	IV	Monolithic	1,587	0.022	2,124	0.450
25	II	Cold Joint	592	0.007	1,982	0.650
26	II	Cold Joint	769	0.011	1,398	0.550
27	II	Cold Joint	363	0.006	566	0.375
28	II	Cold Joint	678	0.017	920	0.400
29	IV	Cold Joint	---	---	6,661	0.325
30	IV	Cold Joint	---	---	1,950	0.400
31	IV	Cold Joint	---	---	2,714	0.750
32	IV	Cold Joint	---	---	2,780	0.650

--- no initial peak

The coefficient of apparent friction (f^*) was calculated using the pullout resistance at 0.75 in. of displacement (Equation 1), and the maximum pullout resistance (Equation 2):

$$f^*_{0.75"} = \frac{P_{0.75"}}{2L_e W \sigma_n} \qquad \text{Equation 1}$$

$$f^*_{max} = \frac{P_{max}}{2L_e W \sigma_n} \qquad \text{Equation 2}$$

where,

$f^*_{0.75"}$ = coefficient of apparent friction at a displacement of 0.75 in.

$P_{0.75"}$ = pullout resistance at a displacement of 0.75 in.

f^*_{max} = coefficient of apparent friction at a maximum pullout load

P_{max} = maximum pullout resistance

L_e = initial embedded length of the steel reinforcing strip

W = width of the embedded steel reinforcing strip

σ_n = normal confinement stress above the steel reinforcing strip

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Values of $f^*_{0.75"}$ and f^*_{max} are summarized in Table 5 and Table 6 for pullout tests run on samples cured for 28 days and 24 hours, respectively.

Table 5: Coefficients of Apparent Friction for Pullout Tests Conducted at 28 Days

Test Number	Concrete Type	Casting Method	σ_n † (psi)	$P_{0.75"}$ (lb.)	P_{max} (lb.)	$f^*_{0.75"}$	f^*_{max}
1	II	Monolithic	2.23	8,100	8,300	18.9	19.4
2	II	Monolithic	2.23	10,406	10,407	24.3	24.3
3	II	Monolithic	2.23	3,270	3,340	7.6	7.8
4	II	Monolithic	2.23	5,220	5,404	12.2	12.6
5	IV	Monolithic	2.29	9,408	9,429	21.4	21.5
6	IV	Monolithic	2.29	8,334	8,439	19.0	19.2
7	IV	Monolithic	2.29	7,806	7,896	17.8	18.0
8	IV	Monolithic	2.29	7,199	7,725	16.4	17.6
9	II	Cold Joint	2.23	3,218	3,348	7.5	7.8
10	II	Cold Joint	2.23	2,644	2,744	6.2	6.4
11	II	Cold Joint	2.23	3,922	4,043	9.2	9.5
12	II	Cold Joint	2.23	6,099	6,109	14.3	14.3
13	IV	Cold Joint	2.30	7,291	8,907	16.5	20.2
14	IV	Cold Joint	2.30	6,642	6,810	15.1	15.4
15	IV	Cold Joint	2.30	6,305	6,460	14.3	14.6
16	IV	Cold Joint	2.30	5,800	5,835	13.1	13.2

† determined using unit weight of material above the reinforcing strip at 28 days (refer to Table 2) plus 300 psf added confinement

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Table 6: Coefficients of Apparent Friction for Pullout Tests Conducted at 24 Hours

Test Number	Concrete Type	Casting Method	σ_n † (psi)	$P_{0.75"}$ (lb.)	P_{max} (lb.)	$f^*_{0.75"}$	f^*_{max}
17	II	Monolithic	0.20	2,307	2,507	61.4	66.7
18	II	Monolithic	0.20	1,829	1,838	48.6	48.9
19	II	Monolithic	0.20	2,116	2,240	56.3	59.6
20	II	Monolithic	0.20	2,630	2,734	69.9	72.7
21	IV	Monolithic	0.23	3,450	3,589	77.2	80.4
22	IV	Monolithic	0.23	2,563	2,418	57.4	54.1
23	IV	Monolithic	0.23	2,081	2,103	46.6	47.1
24	IV	Monolithic	0.23	2,044	2,124	45.8	47.6
25	II	Cold Joint	0.16	1,876	1,982	62.3	65.8
26	II	Cold Joint	0.16	1,340	1,398	44.5	46.4
27	II	Cold Joint	0.16	489	566	16.2	18.8
28	II	Cold Joint	0.16	792	920	26.3	30.5
29	IV	Cold Joint	0.25	5,802	6,661	120.5	138.4
30	IV	Cold Joint	0.25	1,946	1,950	40.4	40.5
31	IV	Cold Joint	0.25	2,622	2,714	54.5	56.4
32	IV	Cold Joint	0.25	2,714	2,780	56.4	57.8

† determined using unit weight of material above the reinforcing strip at 28 days (refer to Table 2)

Post-Test Forensics

Test box 1 was dismantled and dissected using a concrete saw to expose the RECo test strip after testing. The test sample was cut apart so that it separated along the horizontal plane intersecting the RECo test strip. A photo of the top half of the Cell-Crete and the reinforcing strip are shown in Figure 9. Overall, the exposed reinforcing strip showed that the Cell-Crete sheared from the face of the strip except for small areas near the raised bumps. This indicates that the shear force measured during pullout is a combination of friction between the Cell-Crete and the flat face of the RECo reinforcing strip, and the passive pressure from the leading edge of the raised bump reacting against the Cell-Crete.

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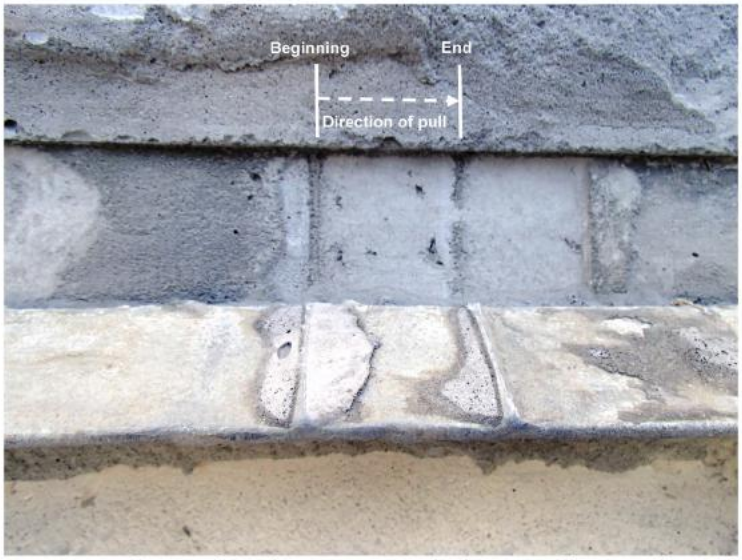


Figure 9: Exposed view of reinforcing strip and adjacent Cell-Crete after pullout.

Closing Remarks

Pullout tests were successfully done on test specimens prepared by RECo and Cell-Crete staff. Data collected related to the unit weight and strength of the Cell-Crete revealed that Type II was less dense and weaker than Type IV; however, coefficients of variation (COV) indicated that there was significant variability in the concrete unit weights and compressive strength (refer to Table 2). Likewise, pullout strengths seemed to vary considerably for repeat samples (refer to Table 3 and Table 4). Because the RECo reinforcing strips are relatively consistent, it is believed that significant differences in the pullout strength are likely due to variations in the cellular concrete fill. It is therefore suggested that if these tests are to be repeated in the future, thought should be given to the Cell-Crete mixing and filling method to ensure that it is consistent.

A plot of the unit weight versus peak pullout strength for the 28-day and 24-hour tests is shown in Figure 10. R-squared values associated with the linear regressions are low, although slightly greater for the 24-hour tests. If there is a direct correlation between the strength of the Cell-Crete and pullout strength, then the low R-squared values may indicate that the actual unit weights of the Cell-Crete differed from what were measured. Samples for unit weight and strength were taken directly from the filler hose and may not represent exactly what was used to cast the individual test boxes. For future testing, it is therefore suggested that unit weight and strength samples be collected directly from the boxes after they are filled. This may be done by collecting wet samples prior to curing, or coring samples later. (Note – it may be difficult to collect 24-hour samples by coring.)

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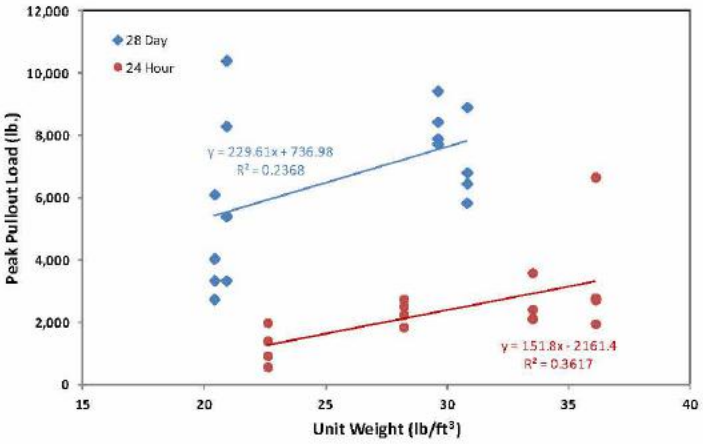


Figure 10: Unit weight of Cell-Crete versus peak pullout load.

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Appendix A

Construction Drawings, Pullout Test Procedure, and Historic Data from The Reinforced Earth Company

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Pullout Test Procedure for High Adherence (HA) Reinforcing Strips in Lightweight Cellular Concrete Fill

Test Location:
Cell-Crete Corporation
135 E Railroad Ave
Monrovia, CA 91016

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Rev.	Date	Modification	Design by	Checked by	Approved by
F					
E	01/02/2018	FINAL ISSUE prior to testing	RL	KBP	KBP
D	15/12/2017	Change on sheath 1 of 5 drawing	RL	KPB	KPB
C	28/08/2017	R1 per comments	RL		
B	14/09/2016	Issue	RL	KPB	KPB
A	01/07/2016	Reviewed	RL	KPB	KPB
Orig	20/01/2016	Original	RL	TM	

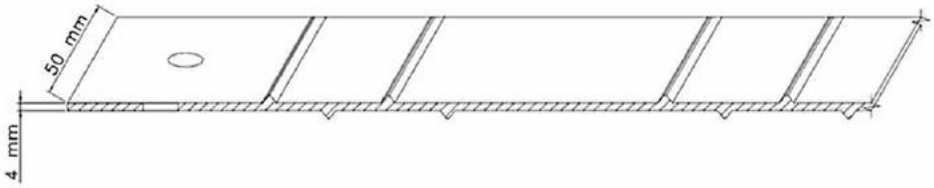
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Preparation and Testing Equipment 7
Testing Procedure 8
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Introduction

For many years now, Lightweight Cellular Concrete (LCC) has been used on certain projects as select backfill for Mechanically Stabilized Earth (MSE) walls. The choice of LCC for MSE backfill is usually due to inadequate foundation bearing capacity, or to mitigate foundation settlements by providing a lighter weight embankment compared to an embankment composed of conventional fill. Since the 1980's, many successful projects have been built with Reinforced Earth® walls backfilled with LCC.

Since Lightweight Cellular Concrete has physical properties that fall between conventional fills (sands, gravels, etc.) and concrete, the design methods and parameters have varied. While certain parameters are strictly a function of the LCC physical properties (Unit Weight, Compressive Strength, Permeability, Shear Strength), other design parameters are dependent on soil reinforcement type (such as frictional resistance). A critical design issue is the pullout resistance of the reinforcing elements in LLC at various stages of curing. This document will provide the test procedure for determining the pullout resistance of high adherence (HA) galvanized steel reinforcing strips in two classes (Classes II and IV) of Lightweight Cellular Concrete (LCC).



Reinforced Earth Company High Adherence (HA) Strip: 50mm x 4mm

There are distinct types and densities of LCC that can be used. They are generally categorized into six classes as per the following table:

Closed cell LCC classes and parameters

Class	Unit weight PCF (Cast density)	Minimum Compressive strength PSI (at 28 days)
I	18 – 24	10
II	24 – 30	40
III	30 – 36	80
IV	36 – 42	120
V	50	160
VI	80	300

Ref: Cell-Crete Corporation, Engineered fill technical literature downloaded 6/21/16 from www.cell-crete.com

Previous pullout testing performed by The Reinforced Earth Company in 1988 (attached for reference) utilized HA Reinforcing Strips embedded in Elastizell LCC with parameters in the Class IV / Class V range. This test program will investigate HA Strip pullout in Class II and IV LCC.

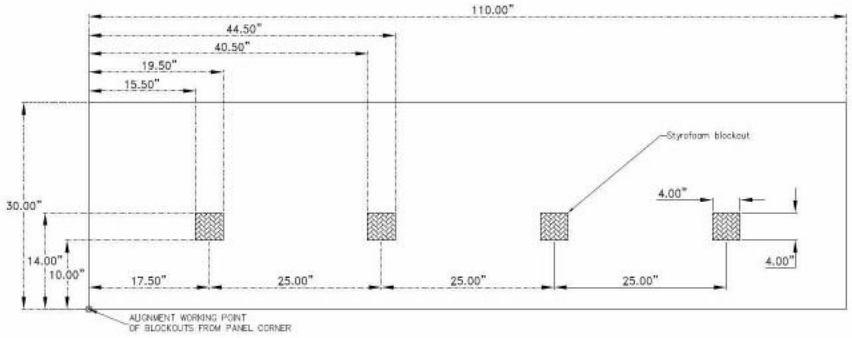
Placement of the HA Strip will be tested both in the center of the pour and at a cold joint in the LCC since placement at a cold joint interface has a potential reduction of bond.

The pullout testing will be performed by TRI with coordination from RECo/Cellcrete and documented by a certified testing lab, and photographed during each step of the process. The results will be documented in a formal report prepared by the lab. We will also have a representative from The Reinforced Earth Company present during the tests. Caltrans is invited to witness any and all tests.

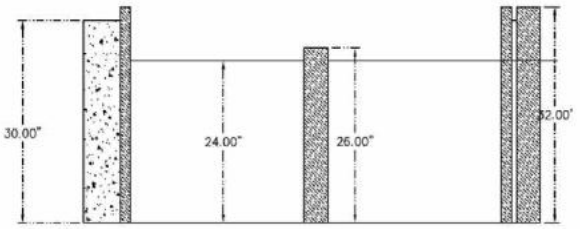
Testing Boxes

To test the pullout resistance of the HA strips soil reinforcements in LCC, several testing pullout boxes will be constructed. These will consist of wooden boxes with partitions to contain the LLC mixture and a precast panel at one end. The precast panels will have openings in them to facilitate the pullout of the HA strip and serve as a jacking surface for the testing equipment to apply the pullout force against. Eight (8) pullout boxes will be constructed for testing.

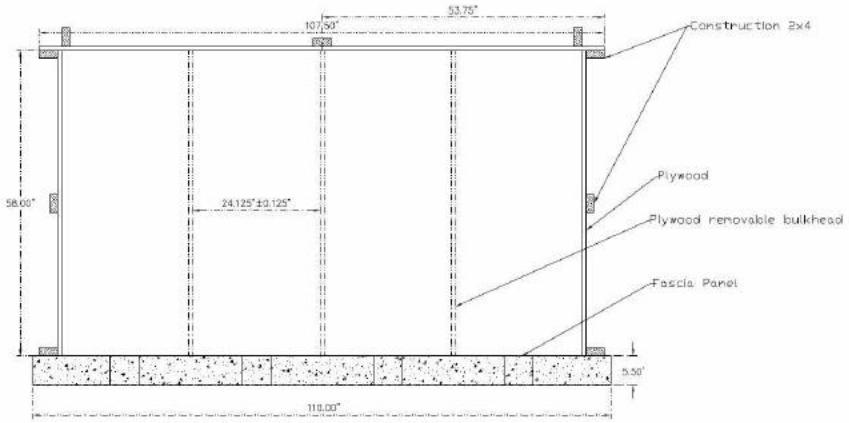
Each precast panel will have 4 blocked out openings in them. The concrete panel will be cast using a minimum 4,000 psi concrete mix. Attached to the back of this panel will be a wooden box with 4 separate isolated partitions for the LCC. Any joints in the pullout box and at the precast panel interface with the box are to be sealed. The full details of the concrete fascia panel and the assembled box are shown below and in Appendix A.



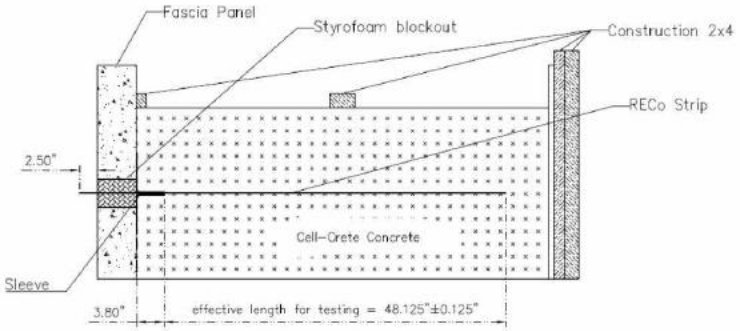
FASCIA PANEL FRONT VIEW



PULLOUT BOX EXTERNAL SIDE VIEW



PULLOUT BOX TOP VIEW
GENERAL DIMENSIONS



SECTION AA SIDE VIEW
SHOWING BOX FILLED

Preparation and Testing Equipment

The HA Strips will be supplied by The Reinforced Earth Company for the testing, and these will be taken from our normal inventory. These HA Strips need to be cut to a length of 60" +/- 1". This will allow the required effective length of 48" +/- ½" to be encapsulated in the LCC. There are 4 of these HA Strips needed per pullout box.

Also supplied is HDPE that is to be cut into pieces (10" length x 9.5" width) preparing the two flaps of the sleeve. Place it over one end of the HA Strip and position it so that the required effective length of 48" +/- ½" is encapsulated in the LCC. Number each HA Strip for testing reference and identification.

This HA Strip assembly shall then be placed in one partition of the pullout box with at least 2.50" protruding past the precast panel surface to allow for attachment of the testing equipment. This should leave an approximately 6" gap between the end of the HA Strip and the back wall of the pullout box. The HA strip can be supported from above or on a support chair to get the correct elevation to be centered in the opening of the precast panel (approximately 12" off the bottom).

Place some Styrofoam or similar material around the HA Strip and in the opening of the precast panel, along with the HDPE flaps, to prevent the LCC from leaking out of the pullout box. Continue this procedure until all remaining HA Strips samples have been placed in each section of each pullout box (32 HA Strips total for 8 pullout boxes with additional 16 HA strips using recycled boxes for a total of 48 test in total).

Proceed with placing the LCC in each partition of each box as required per the specified manufacture's requirements. Mark each section of pour with the type of LCC used along with the date and time.

Leave each test box undisturbed for the required amount of cure time.

Testing Procedure

Since there is no standard ASTM, AASTHO or FHWA pullout testing procedure for inextensible (steel) soil reinforcements, the pullout tests will be performed in general accordance with the ASTM D 6706, "Measuring Geosynthetic Pullout Resistance in Soil" with modifications based on FHWA guidelines. These documents are in Appendix B for reference.

The testing firm will supply and utilize hydraulic jacks to run the pullout tests. Details of jacks and measuring devices to be provided separately. Connect the exposed end of the HA Strip to the jack or pullout clamp by using a 0.5-inch diameter steel bolt (A 325 Minimum) going thru the connection hole of the steel strip.

There will be a total of 48 strips tested. These will be tested at the following curing times:

Test No.	Box No.	LCC Class	No. of Strips	Strip Location	Cure Time
1-4	1	II	4	Middle of Pour	28 Days [2]
5-8	2	IV	4	Middle of Pour	28 Days [2]
9-12	3	II	4	At Cold Joint [1]	28 Days [2]
13-16	4	IV	4	At Cold Joint [1]	28 Days [2]
17-20	5	II	4	Middle of Pour	24 hours
21-24	6	IV	4	Middle of Pour	24 hours
25-28	7	II	4	At Cold Joint [1]	24 hours
29-32	8	IV	4	At Cold Joint [1]	24 hours

Notes: 1 Cellular Concrete at Cold Joints shall be cured for 12 Hour Minimum.
2 Apply overburden Load (300 psf) for 28 days only

For the 28 days tests, a normal load will be applied to the test sample. The normal load will consist of four (4) six inch (6") thick concrete slabs. To accomplish this, first remove the plywood sides serving as formwork from the pullout boxes. Then carefully lift and place 4 slabs on top of each of the LCC samples. The slabs should rest directly on the LCC cellular concrete for a minimum of 15 minutes prior to application of pullout force. This equates to 300 psf of overburden, or roughly 10 ft of LCC fill.

Using the procedure outlined in ASTM D6706, start the test by applying a pullout force to the sample using the jack at a constant displacement rate of 0.04 inch/minute as measured at the attachment point. The load corresponding to displacement shall be recorded at ¼" intervals. Measure the displacement of the HA Strip by direct observation (ruler or a dial gage) or by using a Linear Variable Displacement Transducer (LVDT). Each HA Strip should be continually pulled until it reaches its maximum pullout load.

Documents

As stated previously, the pullout testing will be completed and documented by TRI. The following information will be provided from Cell-Crete for the LCC used for the pullout tests.

Class	Cure Time	Unit Weight (pcf)	Compressive Strength (psi)	Shear Strength ($\phi + c$)	Air Content %
II	28 Days	X	X	X	X
IV	28 Days	X	X	X	X
II	24 Hours	X	X		X
IV	24 Hours	X	X		X

Detailed photographic record of the entire process shall be taken, including the retrieved HA strips and the final state of the LCC after the facing panel is removed from the test box.

A formal report will be prepared by the testing lab and will be made available once completed.

Attachments/Appendices

Appendix A: Drawings and Details of
Pullout Box, Facing Panel, and overburden panels
HA Strip Reinforcement

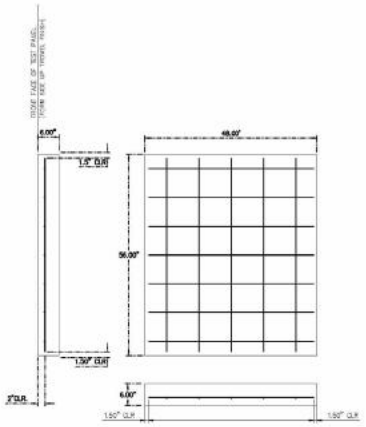
Appendix B: ASTM D6706
FHWA Recommendations for Soil Reinforcement Pullout Testing

Appendix C: Historic Test Data for Comparison Purposes

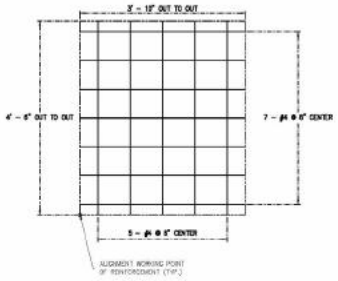
Appendix A

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PRELOAD PANEL FRONT AND SIDE VIEWS
WITH REINFORCEMENT SHOWN



PANEL REINFORCEMENT DETAILS

GENERAL NOTES:

- PLACE AND SECURE REINFORCEMENT, AS WELL AS SUFFICIENT STRAPPING LOOPS OR LIFTING INSERTS.
- ALL REINFORCEMENT SHOWN IS #4 REBAR UNLESS NOTED.
- A MINIMUM OF 4 ONE TON LIFTING INSERTS PER PANEL SHALL BE PROVIDED BY THE PRECASTER.
- REQUIRED MINIMUM CONCRETE DESIGN MIX AND STRENGTH PER STANDARD MIX USED FOR CALTRANS PROJECTS.
- THE LIFTING, HANDLING AND LOADING PROCEDURES SHALL INCLUDE AN ANALYSIS AND ASSESSMENT OF THE SITE SPECIFIC CONDITIONS, THE CONDITION OF ANY EQUIPMENT, A STANDARD APPROVED MEANS AND METHODS UTILIZED BY FABRICATOR, TRANSPORT COMPANY, CONTRACTOR. ALL ENTITIES INVOLVED IN THE HANDLING, LOADING, UNLOADING, TRANSPORTING OR ERECTION OF PRODUCTS SUPPLIED BY REDA. ALL INVOLVED SHALL MAKE AN INDEPENDENT REVIEW AND ASSESSMENT OF ALL CONDITIONS AND PROCEDURES, AND SHALL NOT SOLELY RELY ON ANY GENERAL RECOMMENDATIONS SUPPLIED BY REDA.
- ALL PARTIES SHALL DEVISE AND EXECUTE A SITE SPECIFIC PROCEDURE FOR COMPLYING WITH ALL FEDERAL, STATE AND LOCAL SAFETY LAWS, REGULATIONS AND REQUIREMENTS.
- ALL PARTIES USING THE ENCLOSED DETAILS AGREE TO INDEMNIFY, DEFEND AND HOLD HARMLESS REDA FROM LOSS, COST, DAMAGE OR CLAIM ARISING FROM THEIR HANDLING, LOADING LIFTING AND ERECTION PROCEDURES.

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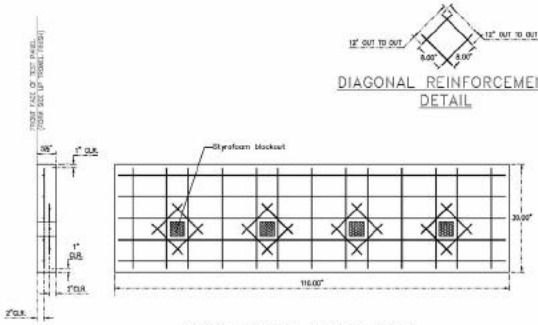
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ENG. MANAGER: KB

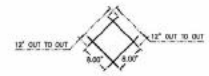
CASTING DETAILS
PRECAST TEST PANEL
TESTING PROGRAM FOR FOAMED CONCRETE PULLOUT
REINFORCED CONCRETE PANELS
FOR PRELOADING

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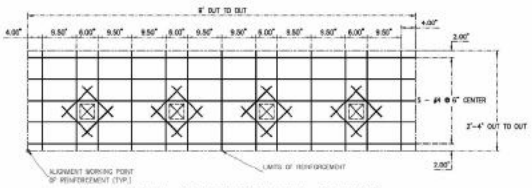
SHEET 1 OF 5
TEST PANEL
Page 19 1" = 1"



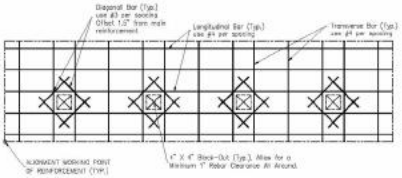
FASCIA PANEL FRONT VIEW
WITH REINFORCEMENT SHOWN



DIAGONAL REINFORCEMENT
DETAIL



PANEL REINFORCEMENT DETAILS



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- ALL REINFORCEMENT SHOWN IS #4 REBAR UNLESS NOTED.
- A MINIMUM OF 4 ONE TON LIFTING INSERTS PER PANEL SHALL BE PROVIDED BY THE PRECASTER.
- 8 BLOCK-OUT LOCATIONS (4" x 4") GLUED AND SCREWED TO BOTTOM SIDE OF BRIDGE BOARD.

  PRIOR TO SETTING AND SCREWING THE BRIDGE BOARD IS PLACE, COAT (SPRAY) THE BLOCK-OUTS WITH FORM RELEASE OIL, ALLOWING FOR THE CASE OF STRIPPING.

  WHEN STRIPPING LOOSEN EACH END, WITH A PERSON CONTROLLING EACH END LIFT VERTICALLY EVENLY UNTIL THE BLOCK-OUTS ARE CLEAR OF SURFACE SET THE UNIT ASIDE.
- REQUIRED MINIMUM CONCRETE DESIGN MIX AND STRENGTH PER STANDARD MIX USED FOR CALTRANS PROJECTS.
- THE LIFTING, HANDLING AND LOADING PROCEDURES SHALL INCLUDE AN ANALYSIS AND ASSESSMENT OF THE SITE SPECIFIC CONDITIONS, THE CONDITION OF ANY EQUIPMENT, A STANDARD APPROVED MEANS AND METHODS UTILIZED BY FABRICATOR, TRANSPORT COMPANY, CONTRACTOR. ALL ENTITIES INVOLVED IN THE HANDLING, LOADING, UNLOADING, TRANSPORTING OR ERECTION OF PRODUCTS SUPPLIED BY REDA. ALL INVOLVED SHALL MAKE AN INDEPENDENT REVIEW AND ASSESSMENT OF ALL CONDITIONS AND PROCEDURES, AND SHALL NOT SOLELY RELY ON ANY GENERAL RECOMMENDATIONS SUPPLIED BY REDA.
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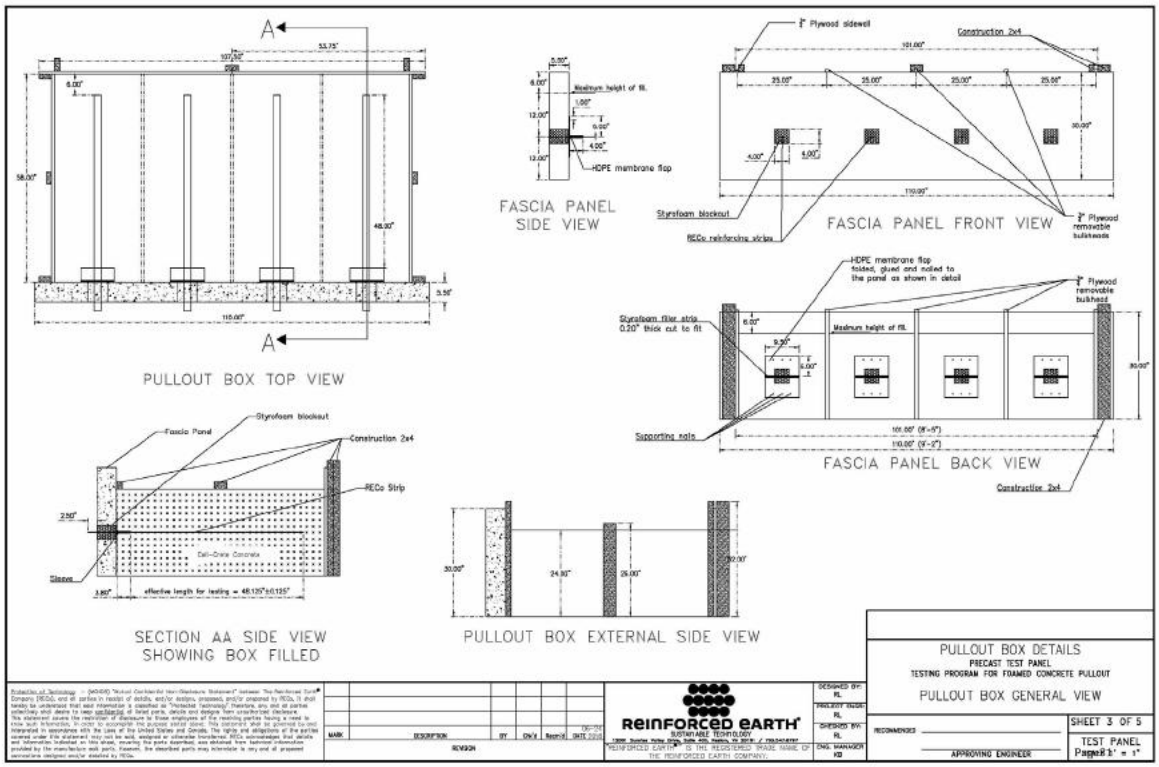
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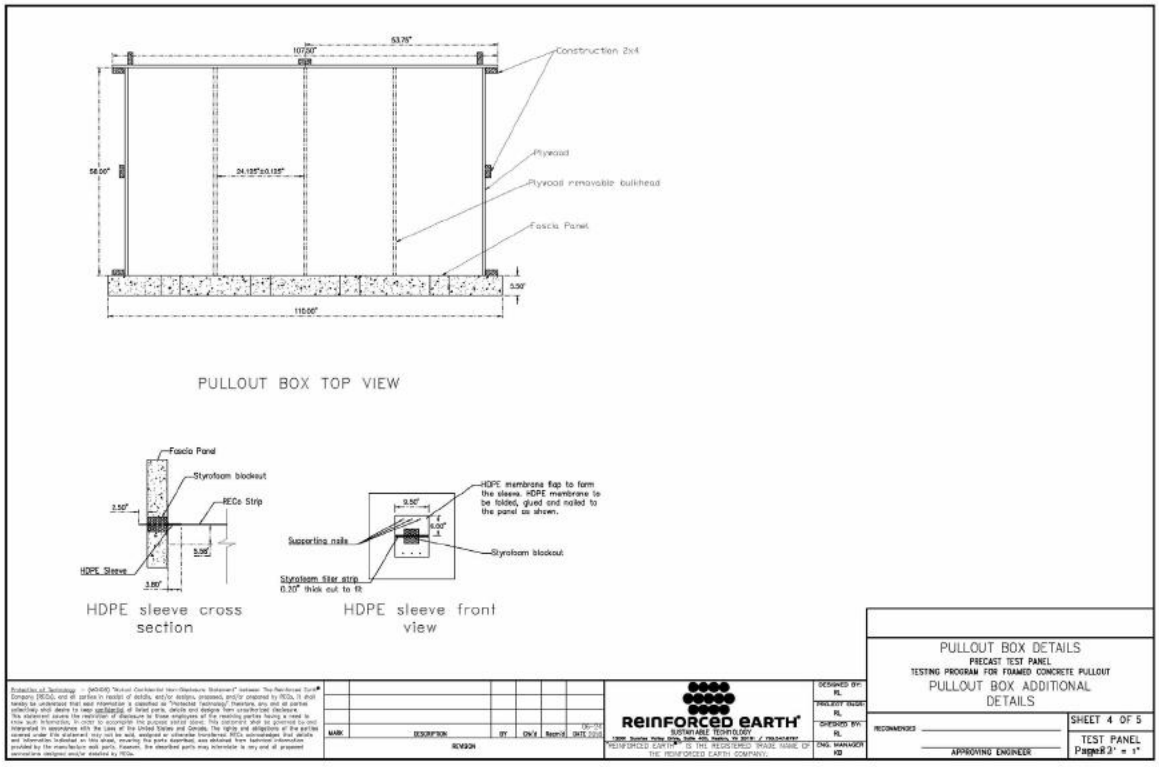
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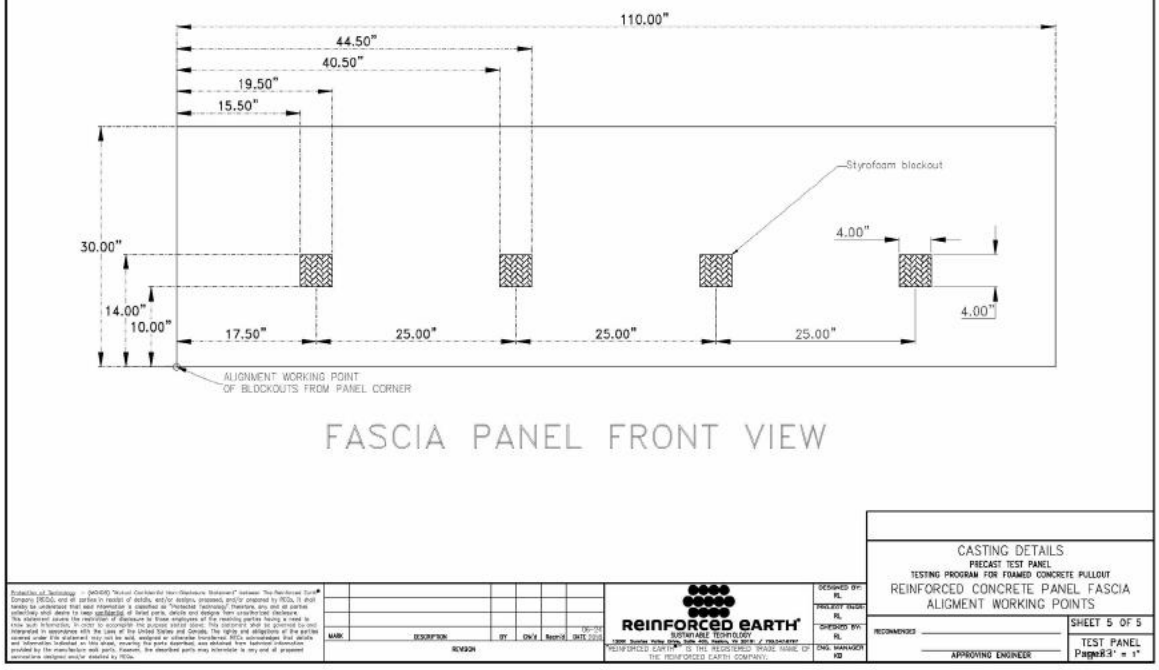
CASTING DETAILS
PRECAST TEST PANEL
TESTING PROGRAM FOR FOAMED CONCRETE PULLOUT
REINFORCED CONCRETE PANELS
FOR FASCIA

RECOMMENDED _____
APPROVING ENGINEER _____

SHEET 2 OF 5
TEST PANEL
Page 20 1" = 1"


PULLOUT BOX TOP VIEW
FASCIA PANEL SIDE VIEW
FASCIA PANEL FRONT VIEW
FASCIA PANEL BACK VIEW
SECTION AA SIDE VIEW SHOWING BOX FILLED
PULLOUT BOX EXTERNAL SIDE VIEW
PULLOUT BOX DETAILS
PRECAST TEST PANEL
TESTING PROGRAM FOR FOAMED CONCRETE PULLOUT
PULLOUT BOX GENERAL VIEW
SHEET 3 OF 5
TEST PANEL
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PULLOUT BOX TOP VIEW
HDPE sleeve cross section
HDPE sleeve front view
PULLOUT BOX DETAILS
PRECAST TEST PANEL
TESTING PROGRAM FOR FOAMED CONCRETE PULLOUT
PULLOUT BOX ADDITIONAL DETAILS
SHEET 4 OF 5
TEST PANEL
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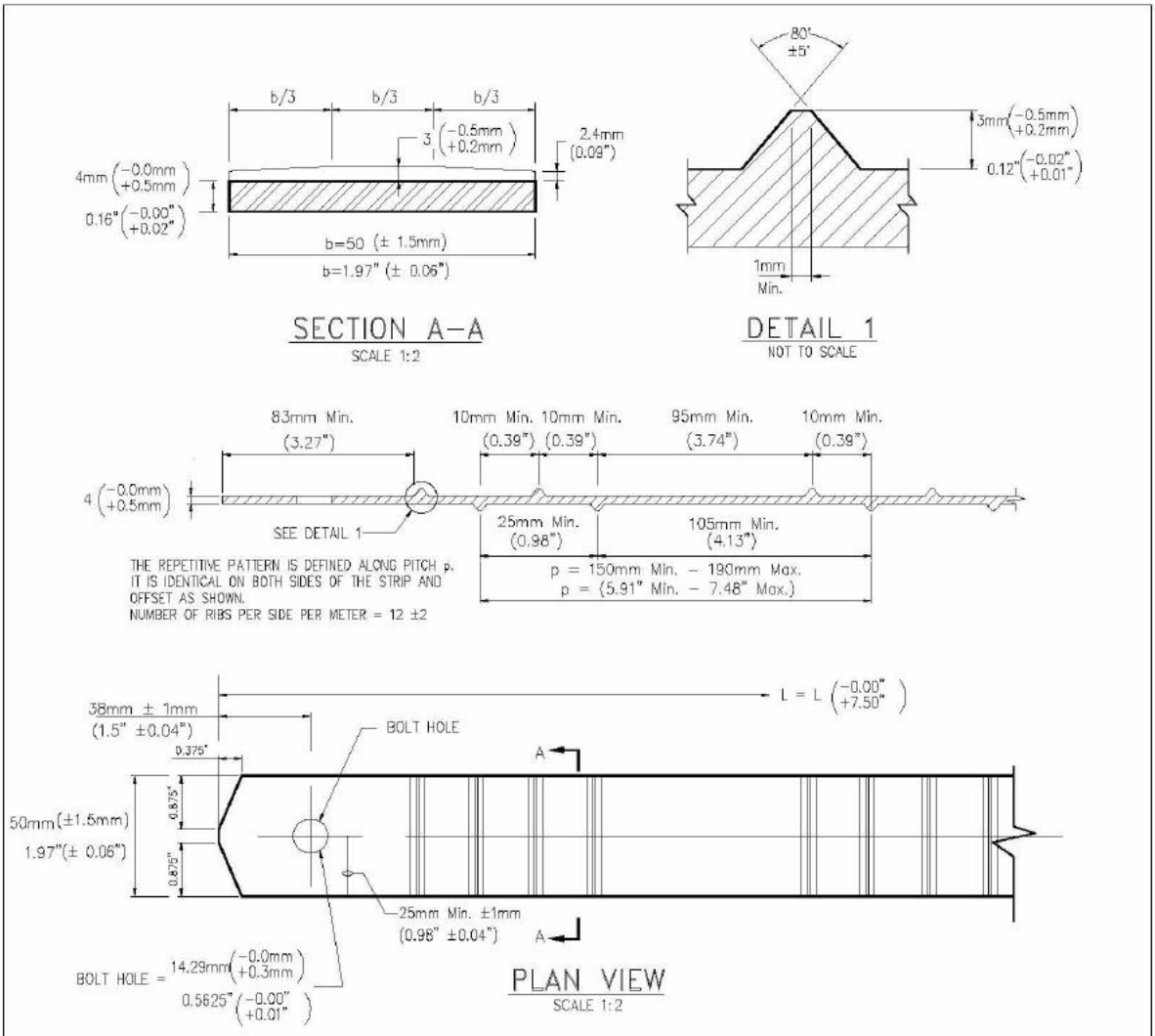
FASCIA PANEL FRONT VIEW

CASTING DETAILS
PRECAST TEST PANEL
TESTING PROGRAM FOR FOAMED CONCRETE PULLOUT
REINFORCED CONCRETE PANEL FASCIA
ALIGMENT WORKING POINTS

SHEET 5 OF 5

TEST PANEL

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GENERAL NOTES:

1. ALL STEEL SHALL COMPLY WITH THE FOLLOWING SPECIFICATION AND REQUIREMENTS:
   ASTM A 572/A–07, GRADE 65

2. GALVANIZING SHALL BE APPLIED AFTER THE STRIPS ARE FABRICATED AND SHALL CONFORM TO THE MINIMUM REQUIREMENTS OF ASTM A–123. (2 oz./SQ FT– ALL SURFACES) (3.4 mils THK.).

ALL MATERIAL SHALL BE DOMESTIC STEEL COMPLYING WITH GUIDELINES SET FOURTH WITHIN THE UNITED STATES GOVERNMENT "BUY AMERICAN ACT", INCLUDING ALL MODIFYING PROVISIONS.

GENERAL PART INFO.	DATE 4–9–2015
REINFORCING STRIP–HA 50x4 mm	REV NO. 0

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Appendix B

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Designation: D6706 – 01 (Reapproved 2013)

Standard Test Method for Measuring Geosynthetic Pullout Resistance in Soil[1]

This standard is issued under the fixed designation D6706; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 Resistance of a geosynthetic to pullout from soil is determined using a laboratory pullout box.

1.2 The test method is intended to be a performance test conducted as closely as possible to replicate design or as-built conditions. It can also be used to compare different geosynthetics, soil types, etc., and thereby be used as a research and development test procedure.

1.3 The values stated in SI units are to be regarded as standard. The values stated in parentheses are provided for information only.

1.4 *This standard may involve hazardous materials, and equipment. This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*[2]

D123 Terminology Relating to Textiles
D653 Terminology Relating to Soil, Rock, and Contained Fluids
D3080 Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions
D4354 Practice for Sampling of Geosynthetics and Rolled Erosion Control Products(RECPs) for Testing
D4439 Terminology for Geosynthetics

3. Terminology

3.1 *Definitions of Terms Specific to This Standard:*

3.1.1 *apertures, n*—the open spaces in geogrids which enable soil interlocking to occur.

3.1.2 *atmosphere for testing geosynthetics, n*—air maintained at a relative humidity of 60 ± 10 % and a temperature of 21 ± 2°C (70 ± 4°F).

3.1.3 *cross-machine direction, n*—the direction in the plane of the geosynthetic perpendicular to the direction of manufacture.

3.1.4 *failure, n*—a defined point at which a material ceases to be functionally capable of its intended use.

3.1.5 *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. **(D4439)**

3.1.6 *junction, n*—the point where geogrid ribs are interconnected in order to provide structure and dimensional stability.

3.1.7 *machine direction, n*—the direction in the plane of the geosynthetic parallel to the direction of manufacture.

3.1.8 *pullout, n*—the movement of a geosynthetic over its entire embedded length, with initial pullout occurring when the back of the specimen moves, and ultimate pullout occurring when the movement is uniform over the entire embedded length.

3.1.9 *pullout force,* (kN), *n*—force required to pull a geosynthetic out of the soil during a pullout test.

3.1.10 *pullout resistance,* (kN/m), *n*—the pullout force per width of geosynthetic measured at a specified condition of displacement.

3.1.11 *rib, n*—the continuous elements of a geogrid which are either in the machine or cross-machine direction as manufactured.

3.1.12 *ultimate pullout resistance,* (kN/m), *n*—the maximum pullout resistance measured during a pullout test.

3.1.13 *wire gage, n*—a displacement gage consisting of a non extensible wire attached to the geosynthetic and monitored by connection to a dial extensometer, or electronic displacement transducer.

3.2 For definitions of other terms used in this test method refer to Terminology D123, D653, and D4439.

[1] This test method is under the jurisdiction of ASTM Committee D35 on Geosynthetics and is the direct responsibility of Subcommittee D35.01 on Mechanical Properties.

Current edition approved Jan. 1, 2013. Published January 2013. Originally approved in 2001. Last previous edition approved in 2007 as D6706–01(2007). DOI: 10.1520/D6706-01R13.

[2] For referenced ASTM standards, visit the ASTM website, www.astm.org, or contact ASTM Customer Service at service@astm.org. For *Annual Book of ASTM Standards* volume information, refer to the standard's Document Summary page on the ASTM website.

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4. Summary of Test Method

4.1 In this method, a geosynthetic is embedded between two layers of soil, horizontal force is applied to the geosynthetic and the force required to pull the geosynthetic out of the soil is recorded.

4.2 Pullout resistance is obtained by dividing the maximum load by the test specimen width.

4.3 The test is performed while subjected to normal compressive stresses which are applied to the top soil layer.

4.4 A plot of maximum pullout resistance versus applied normal stress is obtained by conducting a series of such tests.

5. Significance and Use

5.1 The pullout test method is intended as a performance test to provide the user with a set of design values for the test conditions examined.

5.1.1 The test method is applicable to all geosynthetics and all soils.

5.1.2 This test method produces test data, which can be used in the design of geosynthetic-reinforced retaining walls, slopes, and embankments, or in other applications where resistance of a geosynthetic to pullout under simulated field conditions is important.

5.1.3 The test results may also provide information related to the in-soil stress-strain response of a geosynthetic under confined loading conditions.

5.2 The pullout resistance versus normal stress plot obtained from this test is a function of soil gradation, plasticity, as-placed dry unit weight, moisture content, length and surface characteristics of the geosynthetic and other test parameters. Therefore, results are expressed in terms of the actual test conditions. The test measures the net effect of a combination of pullout mechanisms, which may vary depending on type of geosynthetic specimen, embedment length, relative opening size, soil type, displacement rate, normal stress, and other factors.

5.3 Information between laboratories on precision is incomplete. In cases of dispute, comparative tests to determine if there is a statistical bias between laboratories may be advisable.

6. Apparatus

6.1 *Pullout Box*—An open rigid box consisting of two smooth parallel sides, a back wall, a horizontal split removable door, a bottom plate, and a load transfer sleeve. The door is at the front as defined by the direction of applied pullout force. A typical box is shown in Fig. 1.

6.1.1 The box should be square or rectangular with minimum dimensions 610 mm (24 in.) long by 460 mm (18 in.) wide by 305 mm (12 in.) deep, if sidewall friction is minimized, otherwise the minimum width should be 760 mm (30 in.). The dimensions should be increased, if necessary, so that minimum width is the greater of 20 times the D85 of the soil or 6 times the maximum soil particle size, and the minimum length greater than 5 times the maximum geosynthetic aperture size. The box shall allow for a minimum depth of 150 mm (6 in.) above and below the geosynthetic. The depth of the soil in the box above or below the geosynthetic shall be a minimum of 6 times the D85 of the soil or 3 times the maximum particle size of the soil, whichever is greater. The box must allow for at least 610 mm (24 in.) embedment length beyond the load transfer sleeve and a minimum specimen length to width ratio of 2.0. It should be understood that when testing large aperture geosynthetics the actual pullout box may have to be larger than the stated minimum dimensions.

NOTE 1—To remove side wall friction as much as possible a high density polyethylene (HDPE) geomembrane should be bonded to the inside surfaces of the pullout box. The sidewalls may also be covered with a layer of silk fabric, which has been shown to eliminate adhesion and has a very low friction value. Alternatively, a lubricant can be spread on the sidewalls of the box and thin sheets of polyethylene film used to minimize the side wall friction. It should be also noted that the effect of sidewall friction on the soil-geosynthetic interface can also be eliminated if a minimum distance is kept between the specimen and the side wall. This

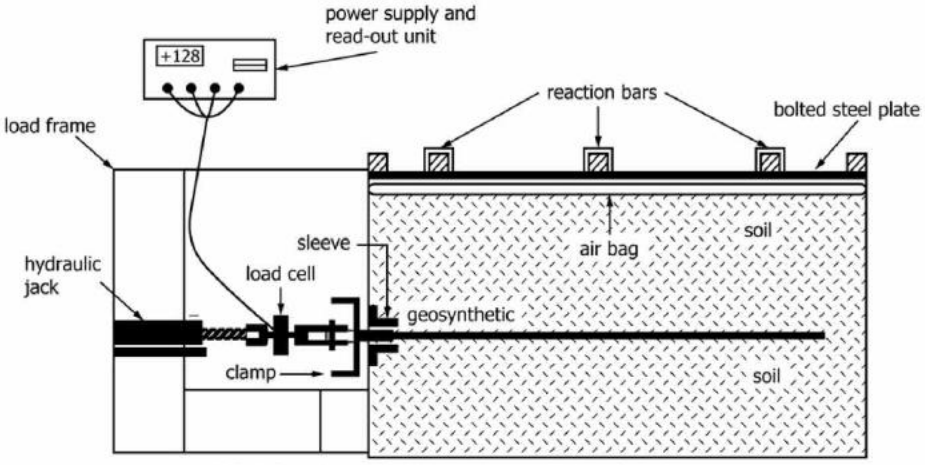


FIG. 1 Experimental Set-Up for Geosynthetic Pullout Testing

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minimum distance is recommended to be 150 mm (6 in.).

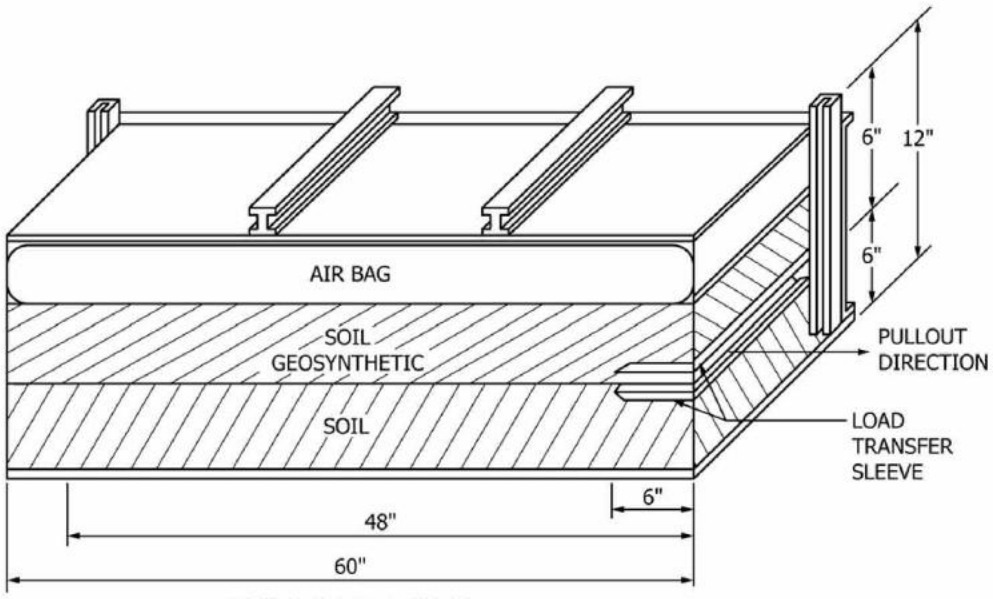
6.1.2 The box shall be fitted with a metal sleeve at the entrance of the box to transfer the force into the soil to a sufficient horizontal distance so as to significantly reduce the stress on the door of the box. The sleeve, as shown in Fig. 2, shall consist of two thin plates (no more than 13 mm (0.5 in.) thick) extending the full width of the pullout box and into the pullout box a minimum distance of 150 mm (6 in.) but it is recommended that this distance equal the total soil depth above or below the geosynthetic. The plates shall be tapered as shown in Fig. 2, such that at the point of load application in the soil, the plates forming the sleeve are no more then 3 mm (0.12 in.) thick. The plates shall be rigidly separated at the sides with spacers and be sufficiently stiff such that normal stress is not transferred to the geosynthetic between the plates.

6.2 *Normal Stress Loading Device*—Normal stress applied to the upper layer of soil above the geosynthetic must be constant and uniform for the duration of the test. To maintain a uniform normal stress, a flexible pneumatic or hydraulic diaphragm-loading device which is continuous over the entire pullout box area should be used and capable of maintaining the applied normal stress within ±2 % of the required normal stress. Normal stresses utilized will depend on testing requirements, however, stresses up to 250 kPa (35 psi) should be anticipated. A recommended normal stress-loading device is an air bag is shown in Fig. 2.

6.3 *Pullout Force Loading Device*—Pullout force must be supplied by a device with the ability to pull the geosynthetic horizontally out of the pullout box. The force must be at the same level with the specimen. The pullout system must be able to apply the pullout force at a constant rate of displacement, slow enough to dissipate soil pore pressures as outlined in Test Method D3080. If excess pore pressures are not anticipated and in the absence of a material specification, apply the pullout force at a rate of 1 mm/min ±10 percent, and the pullout rate should be monitored during testing, see Note 2. Also, a device to measure the pullout force such as a load cell or proving ring must be incorporated into the system and shall be accurate within ±0.5 % of its full-scale range.

NOTE 2—Pullout tests may also be conducted using a constant stress loading approach. This approach can be achieved using one of the three methods described: (*1*) Controlled Stress Rate Method (short-term loading condition) where the pullout force is applied to the geosynthetic under a uniform rate of load application, not exceeding 2 kN/m/min until pullout or failure of the geosynthetic is achieved; (*2*) Incremental Stress Method (short-term loading condition) where the pullout force is applied in uniform or doubling increments and held for a specific time before proceeding to the next increment, as agreed to by the parties involved until pullout or failure of the geosynthetic is achieved; and (*3*) Constant Stress (Creep) Method (long-term loading condition) where the pullout force is applied using one of the first two methods mentioned above to achieve the required constant stress for the test. The constant stress is maintained and the test specimen is monitored over time for the duration of time agreed to by the parties involved (i.e., typically 100 to 10,000 h depending on application). It should be noted that the constant stress procedures described above, have not been widely researched and comparisons with the constant strain method have not been determined.

6.4 *Displacement Indicators*—Horizontal displacement of the geosynthetic is measured at the entrance of the pullout box



CROSS SECTION VIEW

FIG. 2 Cross-Sectional Detail View for Geosynthetic Pullout Setup

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and at several locations on the embedded portion of the specimen. Measurements outside the door at the pullout box entrance are made by a dial extensometer or electronic displacement transducers (e.g. liner variable differential transformers (LVDT's) can be used) mounted to the box frame to read against a plate attached to the specimen near the door.

6.4.1 Determine the displacement of the geosynthetic at a minimum of three equally spaced distances from the clamping plates. Displacement measurements within the box may employ any of several methods, which place sensors or gauge connectors directly on the geosynthetic and monitor their change in location remotely. One such device utilizes wire gages, which are protected from normal stress by a surrounding tube, which runs from a location mounted on the specimen to the outside of the box where displacements are measured by a dial indicator or electronic displacement transducer. A typical instrumentation setup is shown in Fig. 3.

6.4.2 All dial gauges or electronic measurement devices must be accurate to ± 0.10 mm. Locations of the devices must be accurately determined and recorded. Minimum extension capabilities of 50 mm (2 in.) are recommended.

6.5 *Geosynthetic Clamping Devices*— Clamps which connect the specimen to the pullout force system without slipping, causing clamp breaks or weakening the material may be used, see Note 3. The clamps shall be swiveled to allow the pulling forces to be distributed evenly through out the width of the sample. The clamps must allow the specimen to remain horizontal during loading and not interfere with the pullout/shear surface. Gluing, bonding, or otherwise molding of a geosynthetic within the clamp area is acceptable and recommended whenever slippage might occur.

NOTE 3—A suggested device is shown in Fig. 4 and includes a simple clamp consisting of two, 100 mm (4 in.) wide metal angle pieces with a series of bolts and nuts holding the material between them. One possible modification is the addition of a metal rod behind the back flange which allows looping of the material around the rod and back into the clamp. The use of epoxy bonding within the clamp is generally recommended when accurate measurement of the geosynthetic displacements within the soil are required.

6.6 *Soil Preparation Equipment*—Use equipment as necessary for the placement of soils at desired conditions. This may include compaction devices such as vibratory or "jumping-jack" type compaction, or hand compaction hammers. Soil container or hopper, leveling tools and soil placement/removal tools may be required.

6.7 *Miscellaneous Equipment*—Measurement and trimming equipment as necessary for geosynthetic preparation, a timing device and soil property testing equipment if desired.

7. Geosynthetic Sampling

7.1 *Lot Sample*—Divide the product into lots and for any lot to be tested, take the lot samples as directed in Practice D4354, see Note 4.

NOTE 4—Lots of geosynthetics are usually designated by the producer during manufacture. While this test method does not attempt to establish a frequency of testing for determination of design oriented data, the lot number of the laboratory sample should be identified. The lot number should be unique to the raw material and manufacturing process for a specific number of units (for example, rolls, panel, etc.) designated by the producer.

7.2 *Laboratory Sample*—Consider the units in the lot sample as the units in the laboratory sample for the lot to be tested. Take for a laboratory sample, a sample extending the full width of the geosynthetic production unit, of sufficient length along the selvage or edge from each sample roll so that the requirement of 7.3 can be met. Take a sample that will exclude material from the outer wrap unless the sample is taken at the production site, at which point inner and outer wrap material may be used.

7.3 *Test Specimens*—For each unit in the laboratory sample, remove the required number of specimens. In the absence of a material specification, select a minimum of 3 specimens if pullout force versus normal stress relations are to be established.

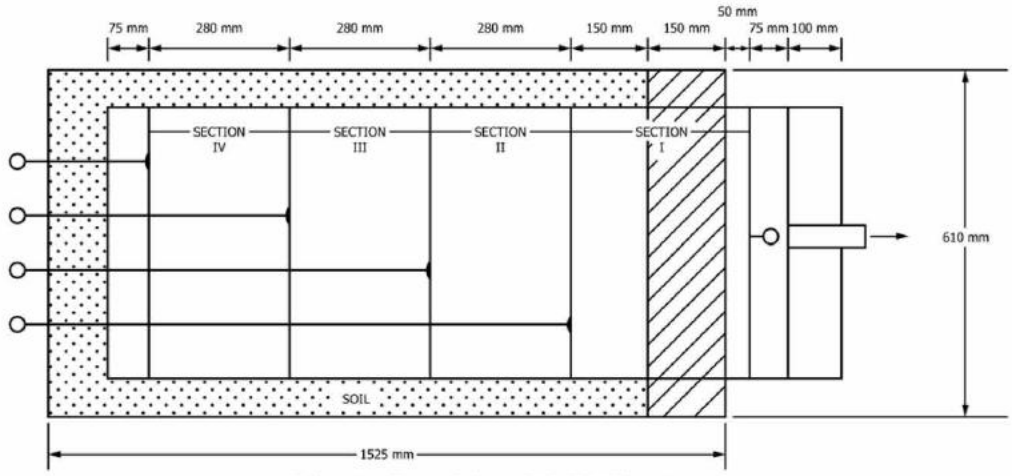


FIG. 3 Plan View and Typical "Tell-Tail" Gage Layout

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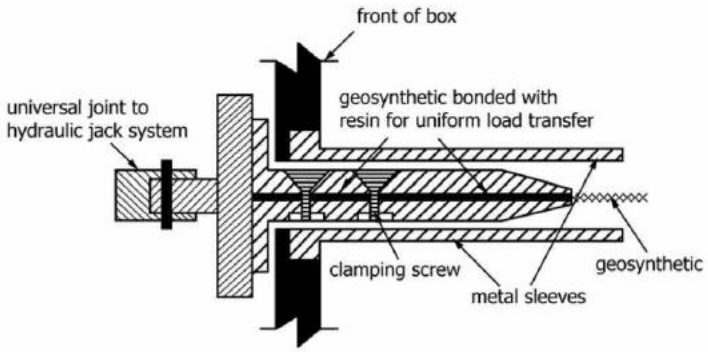


FIG. 4 Geosynthetic Pullout Clamping Detail

7.3.1 Remove the minimum of specimens for pullout testing in a required direction, see Note 5. The minimum embedded test specimen shall have dimensions of 610 mm (24 in.) long by 305 mm (12 in.) wide. The actual size of the test specimen must allow for a minimum of 75 mm (3 in.) clearance on each side of the test specimen from the side walls of the pullout box if the side wall friction is minimized (see Note 1), otherwise the minimum clearance should be 150 mm (6 in.) on each side. The length of the test specimen shall be of sufficient size to facilitate clamping and maintain the minimum length to width ratio of 2. The minimum width of the test specimen shall be 305 mm (12 in.) and should include a minimum of five tensile elements (i.e., ribs/strands). All specimens should be free of surface defects, etc, not typical of the laboratory sample. Take no specimens nearer the selvage edge of the geosynthetic production unit than 1/10 the width of the unit.

NOTE 5—The pullout characteristics of some geosynthetics may depend on the direction tested. In some applications, it may be necessary to perform pullout tests in both the machine and the cross-machine directions. In all cases, the direction of pullout of the geosynthetic specimen(s) should be clearly noted.

8. Conditioning

8.1 When soil is included in the test specimen, the method of conditioning is selected by the user or mutually agreed upon by the user and testing agency. In the absence of specified conditioning criteria, the test should be performed in the atmosphere for testing geotextiles defined in 3.1.2.

8.2 When the geosynthetic is to be tested in the wet condition, saturate the specimen in water for a minimum of 24 h prior to testing, see Note 6.

NOTE 6—Geosynthetics which do not absorb measurable quantities of water, such as some geomembranes, geogrids, and geonets, may not require a full 24 h saturation period for the purpose of this test.

9. Procedure

9.1 *Prepare Pullout Box*—Assemble pullout box with only the bottom half of the door in place. Determine the amount of soil necessary to achieve the desired dry unit weight of the soil when placed in the lower half of the pullout box. The bottom layer of soil should be slightly above the bottom half of the door (approximately 10 mm (0.4 in.)) to avoid dragging of the geosynthetic on the door. The calculated amount of soil is placed in the bottom section of the box and compacted as required. The required number of lifts and amount of compactive effort to be used is a function of the soil type and moisture content, and should be noted. The soil placement procedure that is used should allow for a uniform soil dry unit weight along the pullout box. Level the soil surface. The front section of soil should be excavated such that the bottom part of the metal sleeve can be placed with the upper surface horizontal and level with the soil.

9.2 *Place Geosynthetic*—Obtain a test specimen as described in Section 7. Trim the specimen to fit loosely inside the pullout box with a minimum of 75 mm (3 in.) between the edge of the specimen and the edge of the pullout box parallel to the direction of pull, if sidewall friction is minimized (see Note 1), otherwise there should be a minimum of 150 mm (6 in.) between the edge of the specimen and the side of the pullout box. Clamp the sample outside the entrance of the box, gluing or preparing area in clamp as necessary for uniform pressure, without damaging the sample. Connect sample and clamp to pullout force device. Place the geosynthetic within the metal sleeve at the box entrance. Attach the dial gauge or electronic displacement transducer to pullout clamp outside of the pullout box.

9.2.1 Next, the in-soil displacement devices are installed. Connect the gauges to the specimen and measure locations of gauges relative to the door. The gauges should be evenly spaced along the length of the specimen on a diagonal across the width of the specimen, see Fig. 3. Wire gauges can be attached by hooking wire to a glued-on tab or in the case of geogrids, tying directly on to the specimen. Care must be taken to assure any slack in the wire is eliminated.

9.3 *Embed the Geosynthetic*—Install top metal sleeve and top half of door positioned above the test specimen at the entrance of the pullout box. Place the desired amount of soil on top of geosynthetic to the required level (see 6.1). Use the same placement method as used for the bottom soil layer, see 9.1.

9.4 *Apply Normal Compressive Stress*— Normal stress can be provided by way of a hydraulic or pneumatic diaphragm method as previously described in 6.2. Hydraulic and/or pneumatic devices must be calibrated and any change in

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pressure during testing noted. Normal stresses must be applied before test is started. If consolidation of the soil in the pullout box is required to eliminate excess soil pore pressure or to model field conditions, the required time for consolidation should be calculated as outlined in Test Method D3080.

9.5 *Testing*—Insure complete connection of the pullout system by applying a slight seating load with the pullout force device, then take initial gauge readings. Load the specimen by pulling at a constant rate of displacement. The rate is to be determined according to displacement of dial gauge or electronic displacement transducer outside the pullout box. Take readings of load and displacement, including those inside the box, periodically.

9.5.1 Continue loading until the geosynthetic fails or until pullout occurs, or to a predetermined displacement is reached. Pullout occurs when deformation of all gauged locations becomes equal to the displacement rate while the load is constant or decreasing. For most design work, a minimum displacement of 75 mm (3 in.) may be used to terminate the test. Record maximum load and mode of failure.

9.6 *After Test*—Remove normal stress and disassemble the device. Identify and inspect the soil-geosynthetic interface. Check for uniform geosynthetic deformation.

9.7 Repeat the procedure as required under additional normal compressive stresses.

10. Calculations

10.1 Determine unit weight of soil above and below the geosynthetic and water content of soil if appropriate.

10.2 The total normal stress applied to the test specimen is determined by adding the applied normal stress to the normal stress due to soil above the geosynthetic according to Eq 1 as follows:

$$\sigma_N = \sigma_s + \sigma_a \quad (1)$$

where:

σ_N = total normal stress applied to test specimen, kPa,
σ_s = normal stress due to soil above geosynthetic, kPa, and
σ_a = normal stress due to the applied normal stress, kPa.

10.3 Calculate pullout resistance, P_r, applied by the geosynthetic as follows: (*1*) for geotextiles, geomembranes, and reinforcing strips, use Eq 2; and (*2*) for geogrids and other grid like structures, use Eq 3.

$$P_r = \frac{F_p}{W_g} \quad (2)$$

$$P_r = \frac{F_p \times n_g}{N_g} \quad (3)$$

where:

P_r = pullout resistance, kN/m,
F_p = pullout force, kN,
W_g = width of geosynthetic, m,
n_g = number of ribs per unit width of geogrid in the direction of the pullout force, and
N_g = number of ribs of geogrid test specimen in the direction of the pullout force.

10.4 Plot the test data as a graph of maximum pullout resistance verses normal stress. Plot the pullout resistance verses displacement for each section of specimen within the pullout box. An example of both plots is provided in Figs. 5 and 6.

11. Report

11.1 The report shall include the following:
11.1.1 Description of test apparatus.
11.1.2 Test conditions.
11.1.3 Any departures from standard procedure.
11.1.4 Identification and description of geosynthetic sample(s).
11.1.5 Dimensions of geosynthetic specimen within the pullout box.
11.1.6 Identification of and descriptions of soil including soil classification, water content, unit weight, grain size, and other identifying information if available.
11.1.7 All basic data including normal stresses, displacement measurements, and corresponding resistance values.
11.1.8 Plot(s) of pullout resistance versus displacement and maximum pullout resistance versus normal stress.
11.1.9 Description of the geosynthetic specimen conditions before and after testing.

12. Precision and Bias

12.1 *Precision*—The precision of the procedure in this test method is being established.

12.2 *Bias*—No justifiable statement can be made on the bias of the procedure in this test method since the true value cannot be established by accepted referee methods.

13. Keywords

13.1 geosynthetic; performance test; pullout resistance; soil; soil-geosynthetic interface

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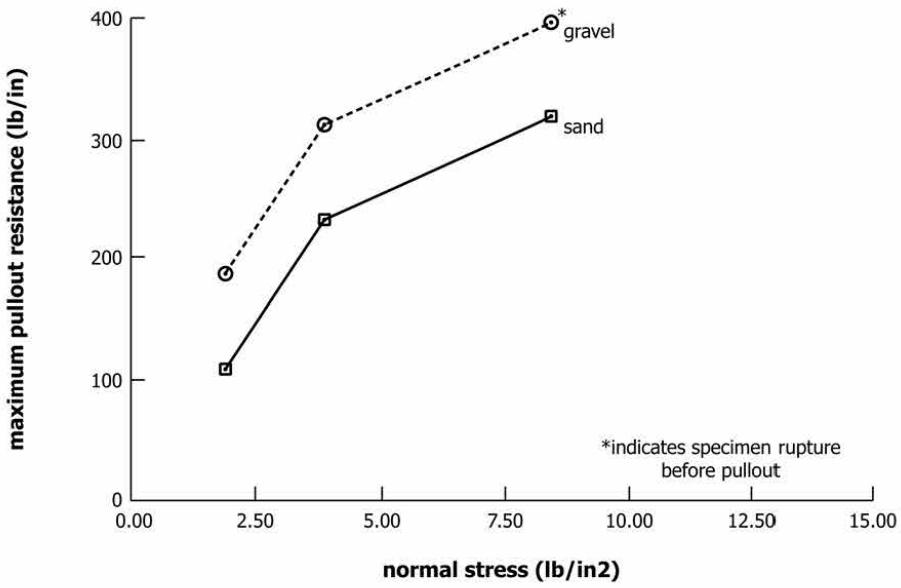


FIG. 5 Typical Maximum Pullout Resistance Versus Normal Stress Plot

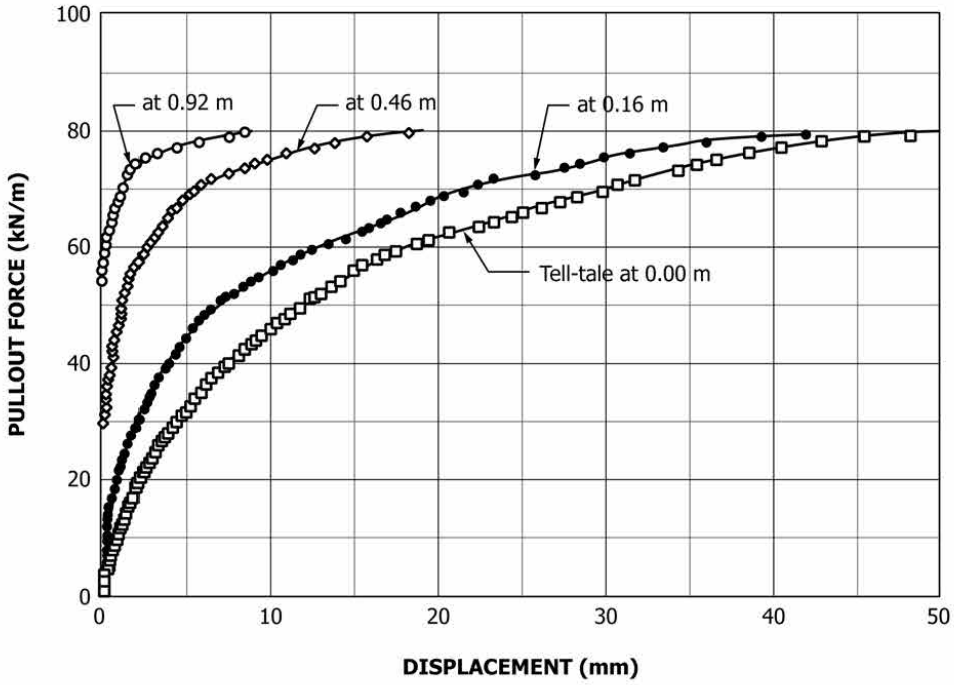


FIG. 6 Typical Relationship Between Pullout Load and Displacement at Front of Pullout Box and "Tell-Tail" Wire Displacements

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DETERMINATION OF PULLOUT RESISTANCE FACTORS and MBW UNIT – GEOSYNTHETIC CONNECTION STRENGTH

Pullout resistance of soil reinforcement is defined by the ultimate pullout resistance required to cause outward sliding of the reinforcement through the soil. Reinforcement specific data has been developed and is presented in Chapter 3. The empirical data uses different interaction parameters, and it is therefore difficult to compare the pullout performance of different reinforcements.

The method for determining reinforcement pullout presented herein, consists of the normalized approach recommended in the FHWA manual FHWA-RD-89-043 (Christopher et al., 1990). The pullout resistance, F* is a function of both frictional and passive resistance, depending on the specific reinforcement type. The scale effect correction factor, α, is a function of the nonlinearity in the pullout load - mobilized reinforcement length relationship observed in pullout tests. Inextensible reinforcements usually have little, if any nonlinearity in this relationship, resulting in α equal to 1.0, whereas extensible reinforcements can exhibit substantial nonlinearity due to a decreasing shear displacement over the length of the reinforcement, resulting in an α of less than 1.0.

Both F* and α must be determined through product specific tests, or empirically/theoretically using the procedures provided herein and in Section 3.3, in particular Table 5. It should be noted that the empirical procedures provided in this appendix for the determination of F* reduce, for the most part, to the equations currently provided in 2007 AASHTO for pullout design.

The pullout resistance of partial/full friction facing/reinforcement connections is defined as the load required to cause sliding of the reinforcement relative to the facing blocks or reinforcement rupture at the facing connection, whichever occurs first.

B.1 EMPIRICAL PROCEDURES TO DETERMINE F^* AND α

Pullout resistance can be estimated empirically/theoretically using the method provided in Chapter 3. F^* using this method, is calculated as follows:

F^* = Frictional Resistance + Passive Resistance
= $\text{Tan}\ \rho + F_q\ \alpha_\beta$

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where Tan ρ is an apparent friction coefficient for the specific reinforcement, ρ is the soil-reinforcement interface friction angle, F_q is the embedment (or surcharge) bearing capacity factor, and α_β is a structural geometric factor for passive resistance. The determination of each of these parameters is provided in Table 5, Chapter 3, with α estimated analytically using direct shear test data and the "t-z" method used in the design of friction piles. However, since some test data is required and the analytical method is complex, it is better to obtain α directly from pullout test data or use conservative default values for α. If pullout test data is not available, a default value of 1.0 can be used for α for inextensible reinforcements and a default value of 0.6 to 0.8 can be used for extensible reinforcements.

B.2 EXPERIMENTAL PROCEDURES TO DETERMINE F* AND α

Two types of tests are used to obtain pullout resistance parameters: the direct shear test, and the pullout test. The direct shear test is useful for obtaining the peak or residual interface friction angle between the soil and the reinforcement material. ASTM D-5321 should be used for this purpose. In this case, F^* would be equal to tan ρ_{peak}. F^* can be obtained directly from this test for sheet and strip type reinforcements. However, the value for α must be assumed or analytically derived, as α cannot be determined directly from direct shear tests. A pullout test can also be used to obtain pullout parameters for these types of soil reinforcement. A pullout test must be used to obtain pullout parameters for bar mat and grid type reinforcements, and to obtain values for α for all types of reinforcements. In general, the pullout test is preferred over the direct shear test for obtaining pullout parameters for all soil reinforcement types. It is recommended that ASTM D6706 test procedure using the controlled strain rate method be used. For long-term interaction coefficients, the constant stress (creep) method can be used. For extensible reinforcements, it is recommended that specimen deformation be measured at several locations along the length of the specimen (e.g., three to four points) in addition to the deformation at the front of the specimen. For all reinforcement materials, it is recommended that the specimen tested for pullout have a minimum embedded length of 24 inches (600 mm). Additional guidance is provided herein regarding interpretation of pullout test results.

For geogrids, the grid joint, or junction strength, must be adequate to allow the passive resistance on the transverse ribs to develop without failure of the grid joint throughout the design life of the structure. To account for this, F^* for geogrids should be determined using one of the following approaches:

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- Using quick effective stress pullout tests (i.e., constant rate of displacement method for short-term loading condition per ASTM D6706) and creep testing (per ASTM D5262) of the geogrid with clamping and loading through-the-junction.
- Using quick effective stress pullout tests (i.e., constant rate of displacement method for short-term loading condition per ASTM D6706), but with the geogrid transverse ribs severed.
- Using quick effective stress pullout tests (i.e., constant rate of displacement method for short-term loading condition per ASTM D6706) if the summation of the shear strengths of the joints occurring in a 1 ft. (300 mm) length of grid sample is equal to or greater than the ultimate strength of the grid element to which they are attached. If this joint strength criteria is used, grid joint shear strength should be measured in accordance with GRI:GG2 (GRI, 1988).
- Conduct long-term effective stress pullout tests of the entire geogrid structure in accordance with the constant stress (creep) method of ASTM D6706.

For pullout tests, a normalized pullout versus mobilized reinforcement length curve should be established as shown in Figure B-1. Different mobilized lengths can be obtained by instrumenting the reinforcement specimen. Strain or deformation measuring devices such as wire extensometers attached to the reinforcement surface at various points back from the grips should be used for this purpose. A section of the reinforcement is considered to be mobilized when the deformation measuring device indicates movement at its end. Note that the displacement versus mobilized length plot (uppermost plot in figure) represents a single confining pressure. Tests must be run at several confining pressures to develop the P_r versus $\sigma_v L_p$ plot (middle plot in figure). The value of P_r selected at each confining pressure to be plotted versus $\sigma_v L_p$ is the lessor of either the maximum value of P_r (i.e., maximum sustainable load), the load which causes rupture of the specimen, or the value of P_r obtained at a predefined maximum deflection measured at either the front or the back of the specimen. Note that P_r is measured in terms of load per unit reinforcement width.

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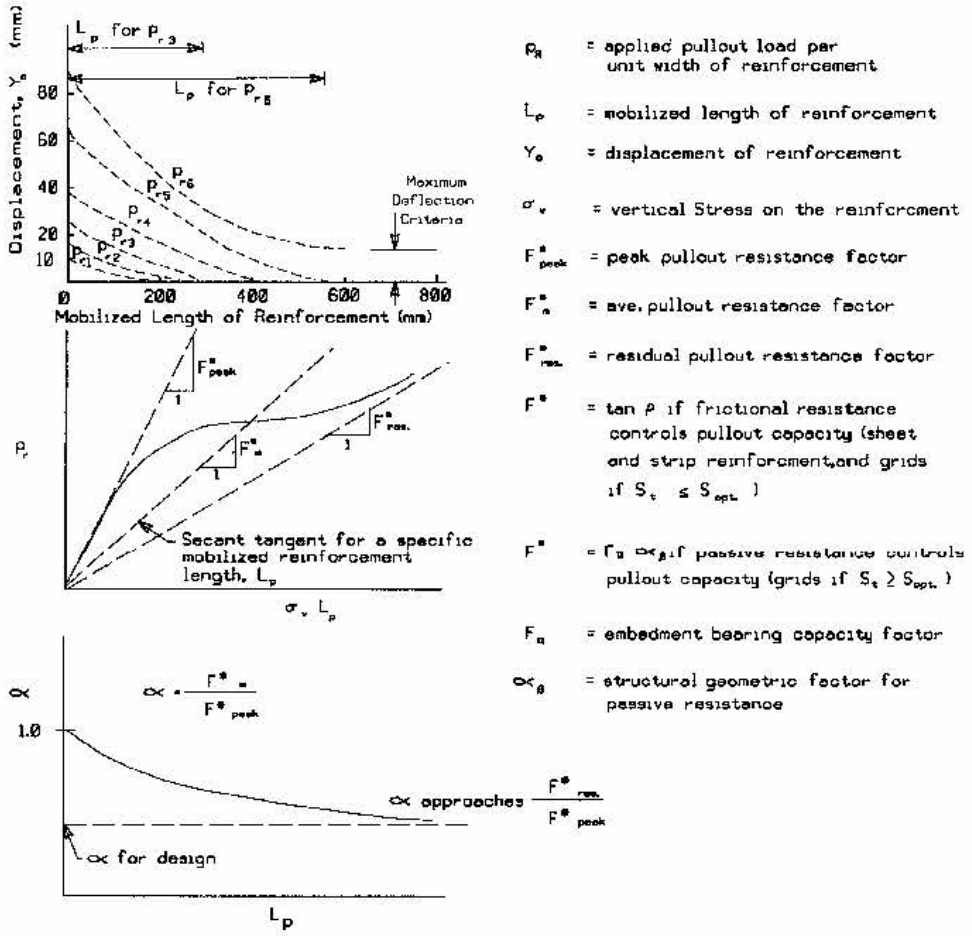


Figure B-1 Experimental procedure to determine F* and α for soil reinforcement using pullout test.

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It is recommended that for inextensible reinforcements, a maximum deflection of ¾-inch (20 mm) measured at the front of the specimen be used to select P_r if the maximum value for P_r or rupture of the specimen does not occur first. For extensible reinforcements, it is recommended that a maximum deflection of 15 mm (5/8-inch) measured at the back of the specimen be used to select P_r if the maximum value for P_r or rupture of the specimen does not occur first. Note that it is acceptable, as an alternative, to define P_r for inextensible reinforcements based on a maximum deflection of 5/8-inch (15 mm) measured at the back of the specimen as is recommended for extensible reinforcements.

F^*_{peak} and F^*_m are determined from the pullout data as shown in Figure B-1. The method provided in this figure is known as the corrected area method (Bonczkiewicz et. al., 1988). The determination of α is also illustrated in Figure B-1. Typical values of F^* and α for various types of reinforcements are provided by Christopher (1993).

Note that the conceptualized curves provided in Figure B-1 represent a relatively extensible material. For inextensible materials, the deflection at the front of the specimen will be nearly equal to the deflection at the back of the specimen, making the curves in the uppermost plot in the figure nearly horizontal. Therefore, whether the deflection criteria to determine P_r for inextensible reinforcements is applied at the front of the specimen or at the back of the specimen makes little difference. For extensible materials, the deflection at the front of the specimen can be considerably greater than the deflection at the back of the specimen. The goal of the deflection criteria is to establish when pullout occurs, not to establish some arbitrary serviceability criteria. For extensible materials, the pullout test does not model well the reinforcement deflections which occur in full scale structures. Therefore, just because relatively large deflections occur at the front of an extensible reinforcement material in a pullout test when applying the deflection criteria to the back of the specimen does not mean that unacceptable deflections will occur in the full scale structure.

B.3 CONNECTION RESISTANCE AND STRENGTH OF PARTIAL AND FULL FRICTION SEGMENTAL BLOCK/REINFORCEMENT FACING CONNECTIONS

For reinforcement connected to the facing through embedment between facing elements using a partial or full friction connection (e.g., segmental concrete block faced walls), the connection strength can be determined directly through long-term testing of the connection to failure. The test set up should be in general accordance with ASTM D6638 with the modifications as described in the interim Long-Term Connection Strength Testing Protocol described below. Extrapolation of test data should be conducted in general accordance with

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Appendix C

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Law Engineering Test Report

3/02/88

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LAW ENGINEERING
GEOTECHNICAL, ENVIRONMENTAL
& CONSTRUCTION MATERIALS
CONSULTANTS

The Reinforced Earth Company
Rosslyn Center
1400 North Moore Street
Arlington, Virginia 22209-1960

3-2-88

Attention: Mr. Peter Anderson

Subject: Elastizell Testing
Law Engineering Project Number W7-5674

Gentlemen:

Law Engineering has completed the laboratory testing program you requested for the project shown above. Our scope of services included determining the pull-out capacity of Reinforced Earth strips embedded in blocks of Elastizell. This report includes a description of the test method used, the results of the eight pull-out tests, and pertinent photographs.

Our laboratory received nine blocks of Elastizell on February 17, 1988. Each block measured approximately 12"x12"x36", with a Reinforced Earth strip embedded longitudinally in each block. Tests indicated the density of these Elastizell blocks to be approximately 40 pcf. The test program requested by Mr. Anderson called for pulling two strips at each of four confining (normal) stresses: 0, .25, .50, and .75 ksf. Strips were pulled using a calibrated 30-ton center hole hydraulic jack. Although stress-deformation readings were taken, all of the strips reached a maximum stress level just as they broke free of the Elastizell. Failures were generally characterized by the Elastizell Block completely splitting through the plane of the strip (see photographs, Appendix B).

Test stresses and failure stresses are as shown below in table Number 1:

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CHANTILLY, VIRGINIA 22021

TABLE NO. 1

Strip #	τ (ksf)	σ_y (ksf)	F* (at failure)
1	4.83	0	-----
2	5.18	0	-----
3	6.59	0.250	26.36
7	5.89	0.250	23.56
4	6.24	0.500	12.48
8	6.24	0.500	12.48
5	6.77	0.750	9.03
6	6.24	0.750	8.32

As shown in the table, the effects of increasing the stress in the plane normal to the strip were minimal. Plots of shear stress versus normal stress and F* versus normal stress are included in Appendix A. Since the configuration of the blocks prevented them from actually being placed in the large shear box, normal stresses were applied by inserting the blocks into a large compression machine as shown in the photographs.

If you have any questions regarding this report or require additional information, please contact us at your convenience. We plan to dispose of the Elastizell blocks within the next fourteen days unless we hear differently from you. As always, we appreciate the opportunity to work on another interesting project with your firm.

Sincerely,

Law Engineering

Stephen Cook
Operations Manager
Soils Laboratory

Greg Hamadock
Manager
Laboratory Services

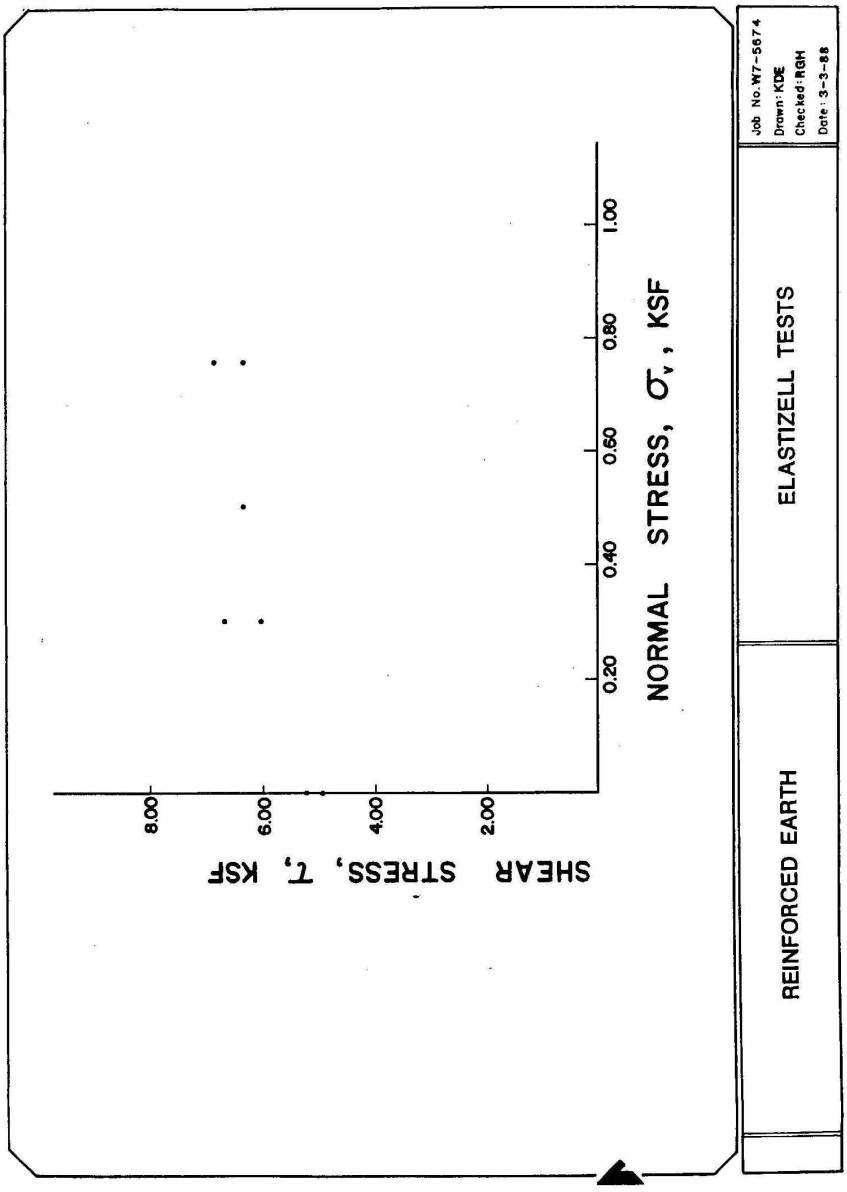
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APPENDIX A
TEST RESULTS

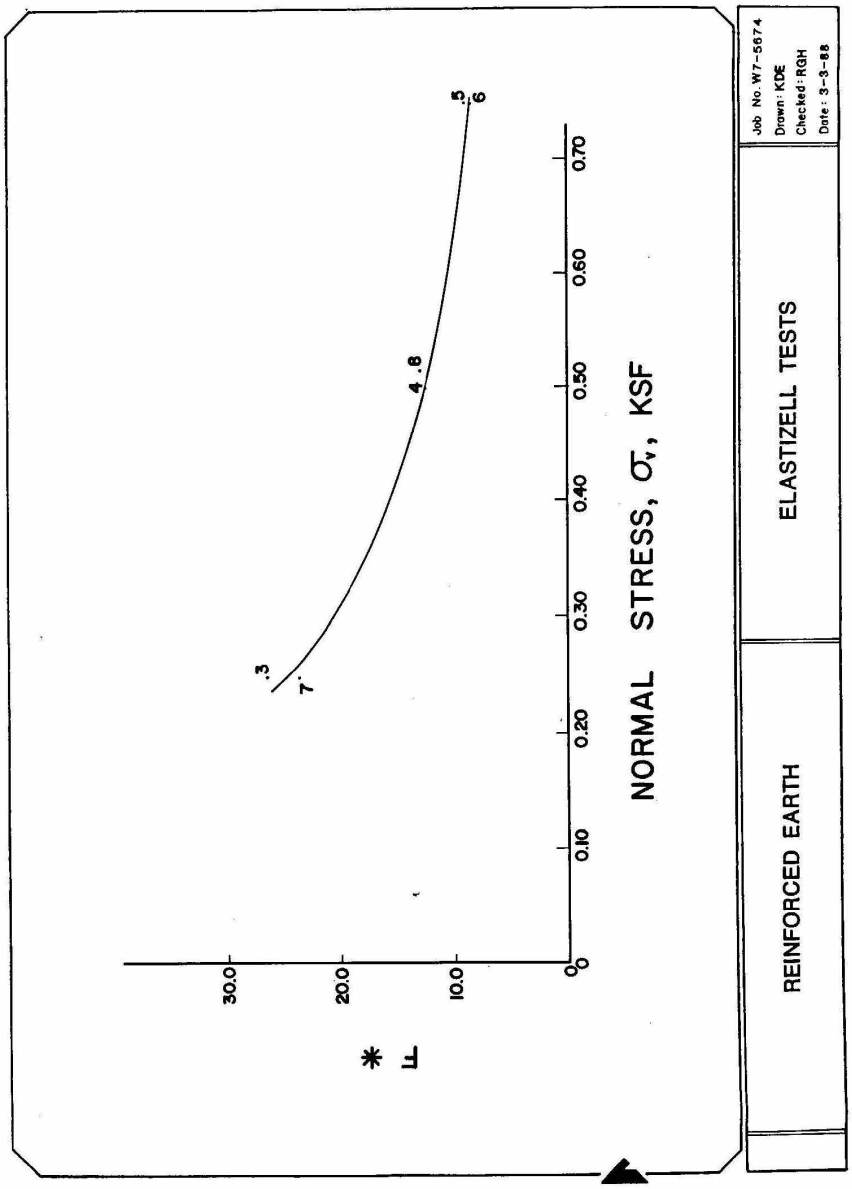
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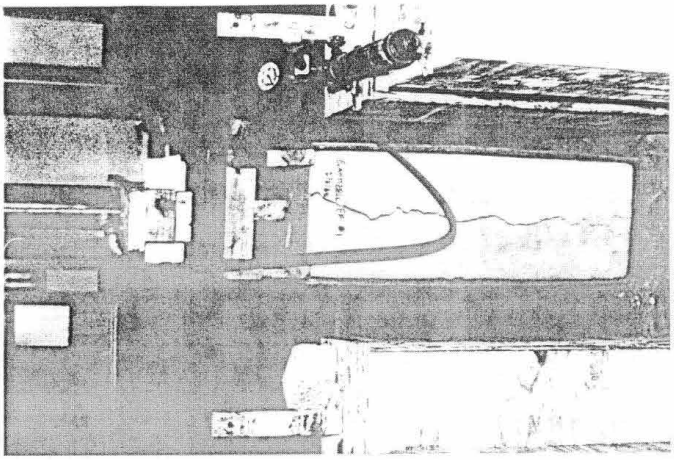
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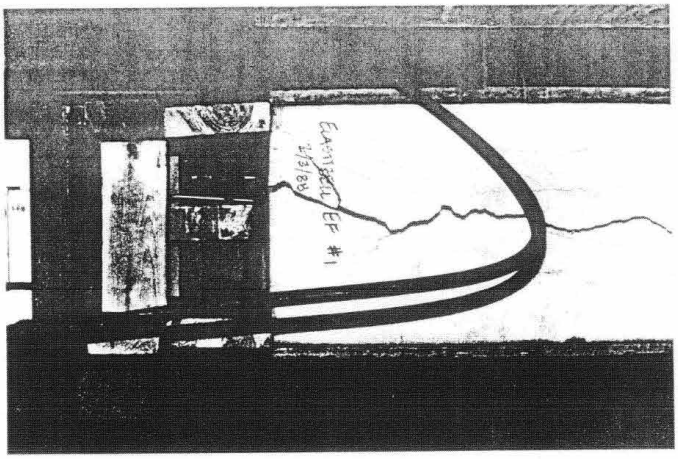
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APPENDIX B
PHOTOGRAPHS

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Elastizell technical data from website (www.elastizell.com) 6/25/07

Technical Data

Density

The specific mix designs may be customized for cast densities ranging from 20 pcf to 120 pcf.

Water Absorption

The discrete cell structure of Elastizell results in lower water absorption than concretes with interconnecting cells or concretes made with expanded aggregates. Elastizell concrete requires only about one-fourth the mix water of expanded aggregate concretes. Its water/cement ratio is similar to that of standard concrete.

ACI formulas for lightweight concretes apply for all ranges of Elastizell Concrete with appropriate substitution for compressive strength (f'_c) and density (w).

Elastizell may be customized through mix designs to include fly ash, lightweight aggregates, fibers and other additives for specialized performance of the standard mixes.

Basic Physical Properties

Air Dry Density*	Normal Compressive Strength Range**	Thermal Conductivity	56 Day Shrinkage
(pcf)	(f'_c)(psi)	K $(BTU \cdot in) / (hr \cdot ft^2 \cdot {}^\circ F)$	(in/in)
110	1500 - 2400	5.4	.0011
100	1000 - 1600	4.4	.0012
90	800 - 1100	3.6	.0014
60	320 - 500	1.8	.0023
40	180 - 350	1.0	.0028
30	140 - 220	0.7	.0036

* Air dry density is about 5 pcf less than wet density.
** These strengths can be increased by the use of additives.

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Appendix B

Materials Testing and Inspection Report for Cell-Crete Concrete

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April 11, 2018

Mr. Eli Cuelho, Senior Engineer
TRI-Environmental, Inc.
Via e-mail: ecuelho@tri-env.com

RE: Report for Inspection and Testing Services | Lightweight Concrete

Dear Mr. Cuelho,

Per your authorization, California Testing & Inspections (CT&I) had a field representative at the Cell-Crete facilities located in the City of Irwindale, California, on March 13th, 2018 to witness and record the casting of lightweight concrete samples for the mock ups prepared by the subject company.

The lightweight concrete samples were retrieved to our laboratory facility in the City of Paramount, on March 14th, 2018 to perform compressive strength testing in accordance to ASTMC-39 at 24-hrs, 7 and 28 days, as directed on your email dated 2/28/18.

A summary of the field observations and laboratory test results is included herein for your review, along with a digital file containing pictures taken during the concrete sampling and laboratory testing.

We appreciate the opportunity to provide you assistance on this project and look forward to continue a business and professional relationship with TRI-Environmental team.

Respectfully,

REGISTERED PROFESSIONAL ENGINEER
FABIOLA VERONICA JAQUE
76471
CIVIL
STATE OF CALIFORNIA

Fabiola Jaque-Diaz, P.E., President
California Testing & Inspections

Encl. Field observations summary, Field Daily Report 3/13/18, Laboratory Testing Summary, Pictures.

FIELD OBSERVATIONS SUMMARY

Project: TRI-Environmental
Project #: Q180305
Client: TRI-Environmental, Inc.
Attention to: Mr. Eli Cuelho., Sr. Engineer
Field Visit Date: March 13, 1018
Field Representative: Reyes Gonzalez

CT&I field representative observed the mockups set-up, the unit weight procedure and sampling procedures for the light weight concrete placed by Cell-Crete at their facility located in the City of Irwindale, California.

General Set-up View



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Mock ups - Inserts Preparation





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Unit Weight Equipment



Sampling Procedures



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Placement of Light Weight concrete



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Six sets of samples were retrieved to CT&I laboratory; four sets casted on 3/13/18 and two sets casted on 3/12/18. Each set consisted of four Styrofoam boxes containing four samples each of one with nominal dimension of 3-in on diameter and 6-in height.

All sampling, identification of the samples, and field testing was performed by a Cell-Crete representative. The list of the samples received at our laboratory is:

Samples Description/Marking:

Sample 1: Class II, 27 pcf, Full Lift, marked as box 1 to 4
Sample 2: Class IV, 36 pcf, Full Lift, marked as box 5 to 8
Sample 3: Class II, 27 pcf, 2nd lift, marked as box 9-12
Sample 4: Class IV, 36 pcf, 2nd lift, marked as box 13-16
Sample 5: Class II, 1st lift, 27 pcf, marked as box 9-12
Sample 6: Class IV, 1st lift, 36 pcf, marked as box 13-16





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LABORATORY TESTING SUMMARY

Project: TRI-Environmental
Project #: Q180305
Client: TRI-Environmental, Inc.
Attention to: Mr. Eli Cuelho., Sr. Engineer
Sample dates: 3/12/18 & 3/13/18

Compressive Strength (ASTM C-39)
Laboratory Sample #8532 – Class II, Sample date 3/13/18 (Field Sample #1)

Test #	Net Area (sq.in)	Compressive Strength (lbs) Test date 3/14/18 Age 24-hr	Compressive Strength (lbs) Test date 3/20/18 Age 7-days	Compressive Strength (lbs) Test date 4/10/18 Age 28-days
1		240.00	425.15	808.90
2		247.74	349.99	823.40
3		258.63	385.63	859.50
4		269.23	402.56	557.80
5		258.01	381.36	651.40
Ave.	7.07	254.72	388.94	740.20
Ave. Compressive Strength (psi)		**40**	**60**	**100**
Wt. of sample #1 before test:		**262.20 g**	**253.50 g**	**232.60 g**

Compressive Strength (ASTM C-39)
Laboratory Sample #8533 – Class IV, Sample date 3/13/18 (Field Sample #2)

Test #	Net Area (sq.in)	Compressive Strength (lbs) Test date 3/14/18 Age 24-hr	Compressive Strength (lbs) Test date 3/20/18 Age 7-days	Compressive Strength (lbs) Test date 4/10/18 Age 28-days
1		659.81	1,101.63	1,944.70
2		732.28	1,033.30	1,922.20
3		795.78	1,344.97	1,935.50
4		747.16	1,242.64	1,702.50
5		732.88	1,078.80	2,150.70
Ave.	7.07	733.58	1,160.27	1,931.12
Ave. Compressive Strength (psi)		**100**	**160**	**270**
Wt. of sample #1 before test:		**399.22 g**	**364.20 g**	**329.30 g**

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Compressive Strength (ASTM C-39)
Laboratory Sample #8534 – Class IV, Sample date 3/13/18 (Field Sample #4)

Test #	Net Area (sq.in)	Compressive Strength (lbs) Test date 3/14/18 Age 24-hr	Compressive Strength (lbs) Test date 3/20/18 Age 7-days	Compressive Strength (lbs) Test date 4/10/18 Age 28-days
1		646.48	1,409.74	1,847.60
2		741.41	1,427.11	1,753.60
3		797.21	1,412.88	1,785.70
4		833.15	757.86	2,020.90
5		768.99	1,244.30	1,943.80
Ave.	7.07	757.45	1,250.38	1,870.32
Ave. Compressive Strength (psi)		**110**	**180**	**260**
Wt. of sample #1 before test:		**385.50 g**	**333.10 g**	**343.00 g**

Compressive Strength (ASTM C-39)
Laboratory Sample #8535 – Class IV, Sample date 3/12/18 (Field Sample #6)

Test #	Net Area (sq.in)	Compressive Strength (lbs) Test date 3/14/18 Age 48-hr	Compressive Strength (lbs) Test date 3/20/18 Age 8-days	Compressive Strength (lbs) Test date 4/10/18 Age 28-days
1		822.13	1,171.44	1,169.80
2		831.03	1,462.55	1,651.40
3		884.52	1,186.68	1,893.10
4		835.71	1,276.68	1,616.50
5		885.34	1,228.47	1,718.50
Ave.	7.07	851.75	1,265.16	1,609.80
Ave. Compressive Strength (psi)		**120**	**180**	**230**
Wt. of sample #1 before test:		**379.70 g**	**350.70 g**	**338.40 g**

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Compressive Strength (ASTM C-39)
Laboratory Sample #8536 – Class II, Sample date 3/13/18 (Field Sample #3)

Test #	Net Area (sq.in)	Compressive Strength (lbs) Test date 3/14/18 Age 24-hr	Compressive Strength (lbs) Test date 3/20/18 Age 7-days	Compressive Strength (lbs) Test date 4/10/18 Age 28-days
1		189.54	328.10	714.20
2		213.34	345.32	783.30
3		178.82	308.79	589.80
4		150.31	532.69	549.40
5		249.78	430.59	574.50
Ave.	7.07	196.36	389.10	642.24
Ave. Compressive Strength (psi)		**30**	**60**	**90**
Wt. of sample #1 before test:		**267.30 g**	**223.00 g**	**226.70 g**

Compressive Strength (ASTM C-39)
Laboratory Sample #8537 – Class II, Sample date 3/12/18 (Field Sample #5)

Test #	Net Area (sq.in)	Compressive Strength (lbs) Test date 3/14/18 Age 48-hr	Compressive Strength (lbs) Test date 3/20/18 Age 8-days	Compressive Strength (lbs) Test date 4/10/18 Age 28-days
1		668.64	993.04	1,488.80
2		686.78	984.26	1,710.50
3		704.18	1,014.19	1,690.90
4		696.85	940.31	1,683.70
5		678.74	1,057.36	1,285.30
Ave.	7.07	687.04	997.83	1,571.84
Ave. Compressive Strength (psi)		**100**	**140**	**220**
Wt. of sample #1 before test:		**369.90 g**	**307.10 g**	**326.40 g**

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CT&I Laboratory #8532 | Field Sample 1



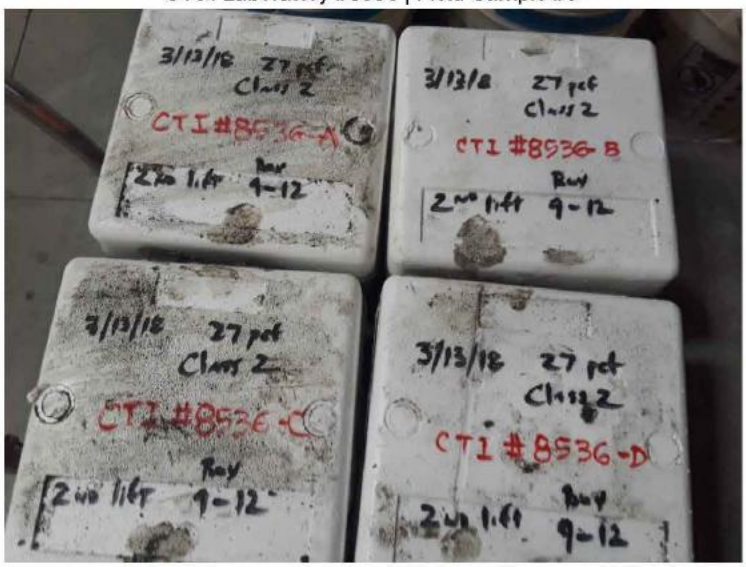
CT&I Laboratory #8533 | Field Sample #2



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CT&I Laboratory #8536 | Field Sample #3



CT&I Laboratory # 8534 | Field Sample #4



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CT&I Laboratory #8537 | Field Sample #5



CT&I Laboratory #8535 | Field Sample #6



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Laboratory sample de-molding process





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SPECIAL INSPECTION REPORT

Project Name: Tri-Environmental
Client Project #: Q180305 (CT&I)
Project Location: Cell-Crete Plant, Irwindale, CA
Contractor: Cell-Crete Corporation

Date: 3/13/18
Time Arrived: 8:00 am
Time Departed: 10:00 am
Travel Time: N/A

Type of Inspection

- [x] Engineered Fill
- [ ] Foundation
- [ ] Batch Plant
- [ ] Prestressed Concrete
- [ ] Shotcrete
- [ ] Masonry
- [ ] High Strength Bolting
- [ ] Welding
- [ ] Fireproofing
- [ ] _____
- [ ] _____

Revised Documents: _____

Work Inspected:

I arrived on site as scheduled at 8:00 am to witness the placement of Cell-Crete in "Mock up cells". Randy of Cell-Crete did not take temperature or rod the samples at the time of casting samples. Each batch was weighed in a small container for unit weight; sample number 2 was class 4 and was 3.8 lbs weight, placed in full lift. Sample number 1 was class 2 and was 2.7 lbs weight, placed in full lift. Sample number 3 was class 2 and the unit weight was 2.7 lbs, taken at the second lift. Sample number 4 was class 4 and the weight was 3.6 lbs, placed on full lift. There is a fifth sample that was already half way full from 3/12/18 and the molds were filled up this day (cold joint pour).
The inserts on the mock ups were inserted a minimum of 48 inches leaving at least 3 inches on the outer part of the mold.

Use Additional Sheets if Needed

Reyes Gonzalez
Signature of Field Inspector

Reyes R. Gonzalez (Senior field technician)
Print Name / Title

cc: Project Manager
Project File
Field Inspector

Report and time approved by: _____

May 30, 2018

Mr. Eli Cuelho, Senior Engineer
TRI-Environmental, Inc.
Via e-mail: ecuelho@tri-env.com

RE: Testing Services | Lightweight Concrete

Dear Mr. Cuelho,

The lightweight concrete samples were delivered to our laboratory facility in the City of Paramount, on April 11th, 2018 and April 12th to perform compressive strength testing in accordance to ASTM C-39 at 24-hrs, 7 and 28 days, as directed on your email.

A summary of the laboratory test results is included herein for your review.

We appreciate the opportunity to provide you assistance on this project and look forward to continue a business and professional relationship with TRI-Environmental team.

Respectfully,

Fabiola Jaque-Diaz, P.E., President
California Testing & Inspections

Enclosed: Laboratory Testing Summary.

Six sets of samples were delivered to CT&I laboratory; four sets casted on 4/10/18 and two sets casted on 4/11/18. Each set casted on 4/10/18 consisted of four Styrofoam boxes containing four samples each of one with nominal dimension of 3-in on diameter and 6-in height. Samples casted on 4/11/18 consisted of one set of 4 Styrofoam boxes and another set with five Styrofoam boxes.

All sampling, identification of the samples, and field testing was performed by a Cell-Crete representative. The list of the samples received at our laboratory is:

Samples Description/Marking:

Sample 1: Class IV, 36.5 pcf, Panel #6 Full 24" Lift, 9:35 am 4/10/18
Sample 2: Class IV, 36.8 pcf, Panel #8 Half 12" Lift, 9:48 am 4/10/18
Sample 3: Class II, 27.8 pcf, Panel #5 Full 24" Lift, 10:05 am 4/10/18
Sample 4: Class II, 27.6 pcf, Panel #7 Half 12" Lift, 10:21 am 4/10/18
Sample 5: Class II, N/P pcf, Panel #7 2nd Lift, 9:00 am 4/11/18
Sample 6: Class IV, N/P pcf, Panel #8 2nd Lift, 9:26 am 4/11/18

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LABORATORY TESTING SUMMARY

Project: TRI-Environmental
Project #: Q180305
Client: TRI-Environmental, Inc.
Attention to: Mr. Eli Cuelho., Sr. Engineer
Sample dates: 4/10/18 & 4/11/18

Compressive Strength (ASTM C-39)
Laboratory Sample #8873 – Class IV, 36.5 pcf, Panel #6 Full 24" Lift, 9:35 am 4/10/18

Test #	Net Area (sq.in)	Compressive Strength (lbs) Test date 4/11/18 Age 24-hr	Compressive Strength (lbs) Test date 4/17/18 Age 7-days	Compressive Strength (lbs) Test date 5/8/18 Age 28-days
1		350.75	1048.00	926.66
2		325.37	924.29	900.61
3		345.77	907.37	911.01
4		345.86	793.91	910.34
5		384.93	912.69	971.06
Ave.	7.07	350.54	917.25	931.14
Ave. Compressive Strength (psi)		**50**	**130**	**130**
Wt. of sample #1 before test:		**374.60 g**	**300.40 g**	**302.30 g**

Compressive Strength (ASTM C-39)
Laboratory Sample #8874 – Class IV, 36.8 pcf, Panel #8 Half 12" Lift, 9:48 am 4/10/18

Test #	Net Area (sq.in)	Compressive Strength (lbs) Test date 4/11/18 Age 24-hr	Compressive Strength (lbs) Test date 4/17/18 Age 7-days	Compressive Strength (lbs) Test date 5/8/18 Age 28-days
1		420.21	893.86	1391.32
2		390.00	854.42	1254.93
3		416.80	1024.16	1490.93
4		398.25	939.11	1470.02
5		411.15	924.05	1493.24
Ave.	7.07	407.28	927.12	1420.09
Ave. Compressive Strength (psi)		**60**	**130**	**200**
Wt. of sample #1 before test:		**415.40 g**	**323.50 g**	**334.20 g**

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Compressive Strength (ASTM C-39)
Laboratory Sample #8875 – Class II, 27.8 pcf, Panel #5 Full 24" Lift, 10:05 am 4/10/18

Test #	Net Area (sq.in)	Compressive Strength (lbs) Test date 4/11/18 Age 24-hr	Compressive Strength (lbs) Test date 4/17/18 Age 7-days	Compressive Strength (lbs) Test date 5/8/18 Age 28-days
1		265.73	669.13	872.68
2		118.80	798.48	775.88
3		120.31	750.14	973.80
4		135.20	868.56	764.83
5		150.51	926.78	756.24
Ave.	7.07	158.11	802.62	828.69
Ave. Compressive Strength (psi)		**20**	**110**	**120**
Wt. of sample #1 before test:		**357.80 g**	**263.40 g**	**291.50 g**

Compressive Strength (ASTM C-39)
Laboratory Sample #8876 – Class II, 27.6 pcf, Panel #7 Half 12" Lift, 10:21 am 4/10/18

Test #	Net Area (sq.in)	Compressive Strength (lbs) Test date 4/11/18 Age 24-hr	Compressive Strength (lbs) Test date 4/17/18 Age 7-days	Compressive Strength (lbs) Test date 5/8/18 Age 28-days
1		173.79	583.71	757.14
2		279.96	713.44	563.18
3		197.78	562.87	754.23
4		159.32	572.80	549.20
5		177.00	721.33	550.96
Ave.	7.07	197.57	630.83	634.94
Ave. Compressive Strength (psi)		**30**	**90**	**90**
Wt. of sample #1 before test:		**280.30 g**	**224.10 g**	**256.20 g**

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Compressive Strength (ASTM C-39)
Laboratory Sample #8886 – Class II, N/P pcf, Panel #7 2nd Lift, 9:00 am 4/11/18

Test #	Net Area (sq.in)	Compressive Strength (lbs) Test date 4/12/18 Age 24-hr	Compressive Strength (lbs) Test date 4/18/18 Age 7-days	Compressive Strength (lbs) Test date 5/9/18 Age 28-days
1		91.45	236.94	567.73
2		156.47	239.44	392.79
3		85.22	264.39	345.18
4		100.01	243.84	469.85
5		79.56	329.85	370.03
Ave.	7.07	102.54	262.89	429.12
Ave. Compressive Strength (psi)		**10**	**40**	**60**
Wt. of sample #1 before test:		**251.90 g**	**202.10 g**	**214.10 g**

Compressive Strength (ASTM C-39)
Laboratory Sample #8887 – Class IV, N/P pcf, Panel #8 2nd Lift, 9:26 am 4/11/18

Test #	Net Area (sq.in)	Compressive Strength (lbs) Test date 4/12/18 Age 24-hr	Compressive Strength (lbs) Test date 4/18/18 Age 7-days	Compressive Strength (lbs) Test date 5/9/18 Age 28-days
1		944.97	1296.40	1745.04
2		745.57	1056.32	1324.71
3		818.59	1202.03	1203.06
4		710.39	1305.31	1702.77
5		784.88	1213.76	1277.76
Ave.	7.07	800.88	1214.76	1450.67
Ave. Compressive Strength (psi)		**110**	**170**	**210**
Wt. of sample #1 before test:		**401.30 g**	**357.50 g**	**372.60 g**

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Appendix C

Pullout Test Results

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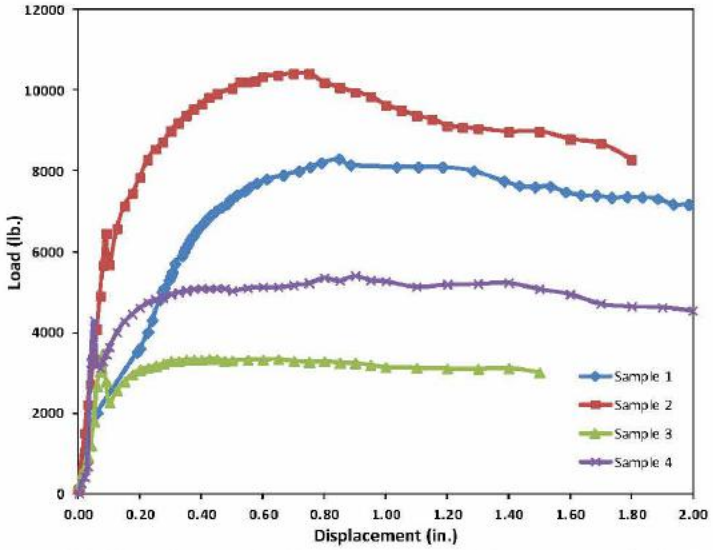


Figure 11: Full pullout results for Type II monolithic pour tested at 28 days.

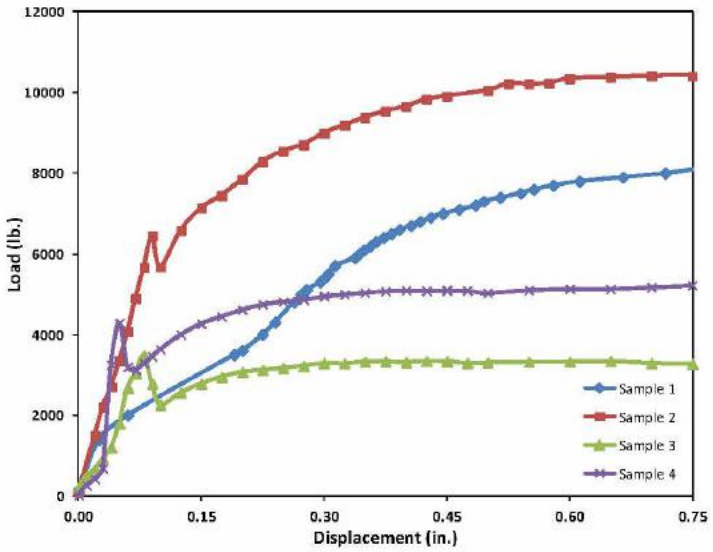


Figure 12: Initial pullout results for Type II monolithic pour tested at 28 days.

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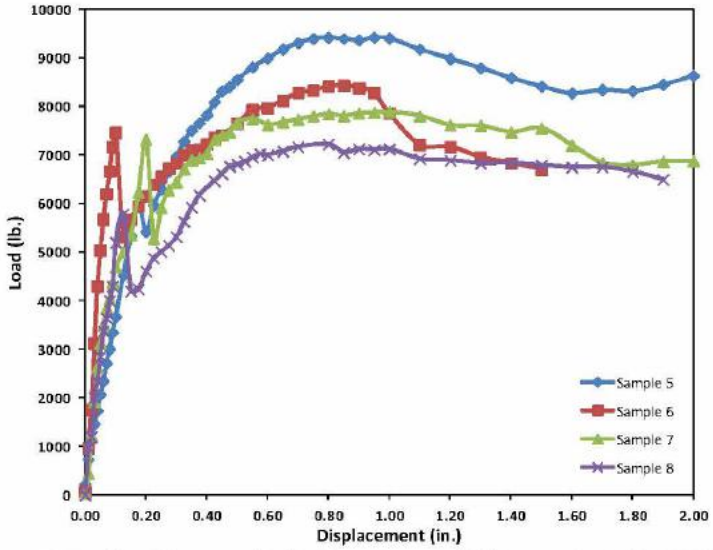


Figure 13: Full pullout results for Type IV monolithic pour tested at 28 days.

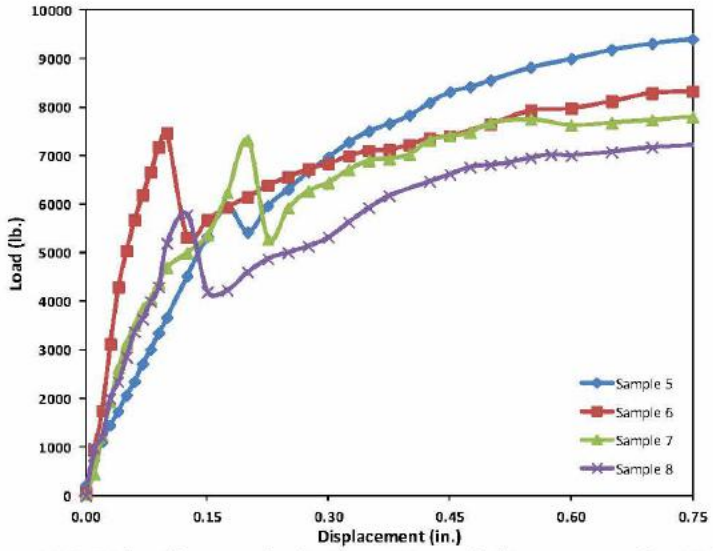


Figure 14: Initial pullout results for Type IV monolithic pour tested at 28 days.

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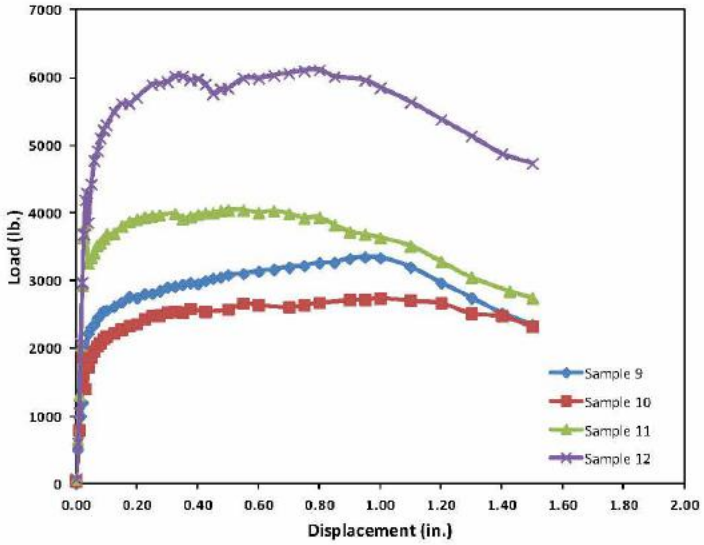


Figure 15: Full pullout results for Type II cold-joint pour tested at 28 days.

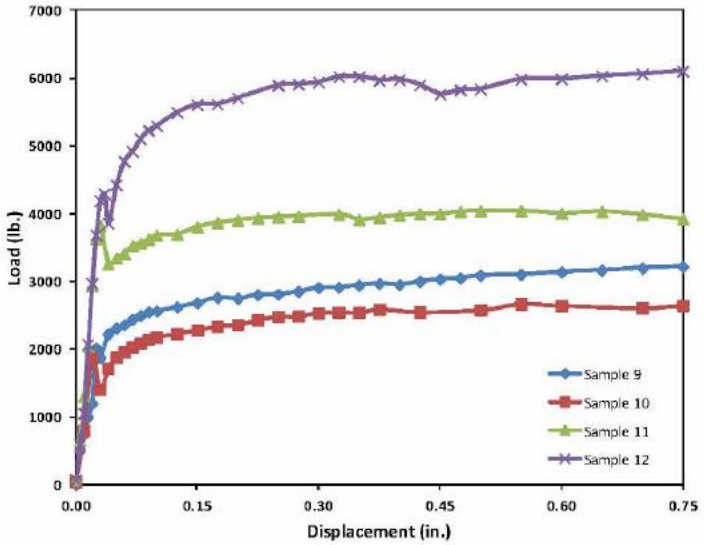


Figure 16: Initial pullout results for Type II cold-joint pour tested at 28 days.

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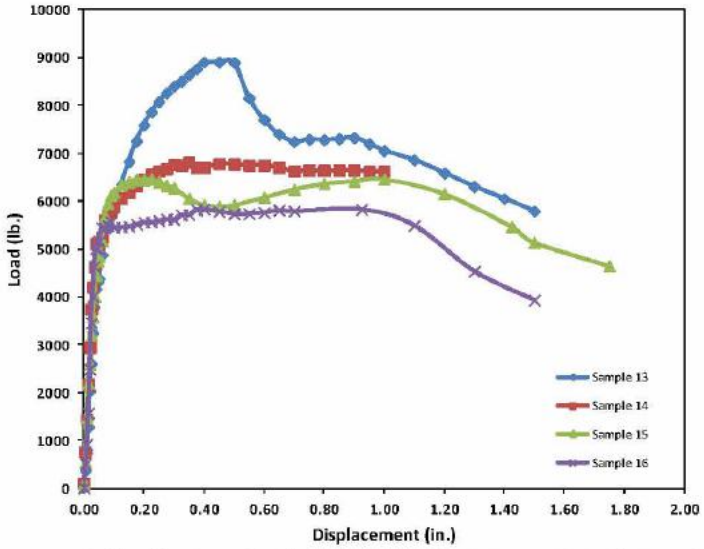


Figure 17: Full pullout results for Type IV cold-joint pour tested at 28 days.

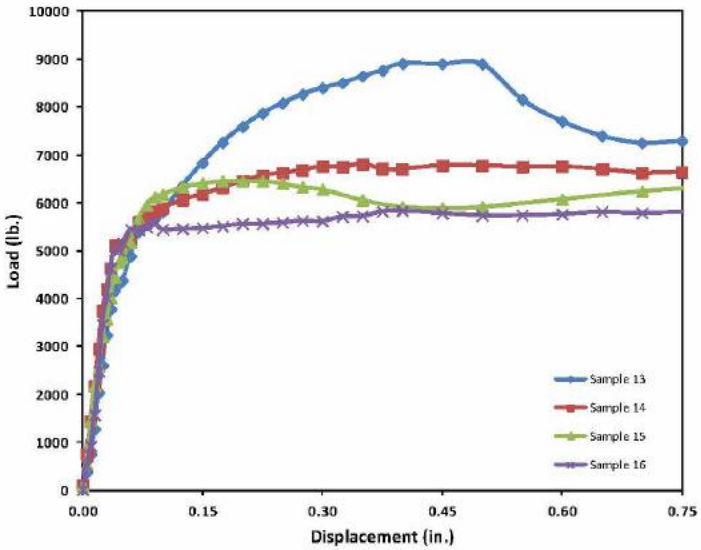


Figure 18: Initial pullout results for Type IV cold-joint pour tested at 28 days.

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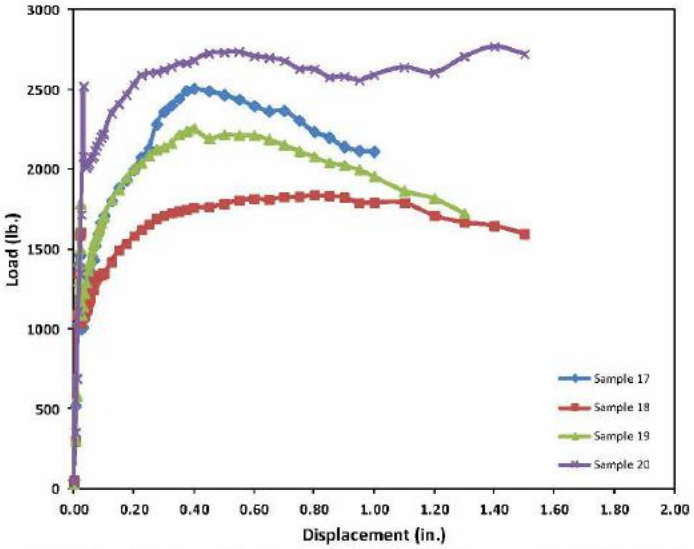


Figure 19: Full pullout results for Type II monolithic pour tested at 24 hours.

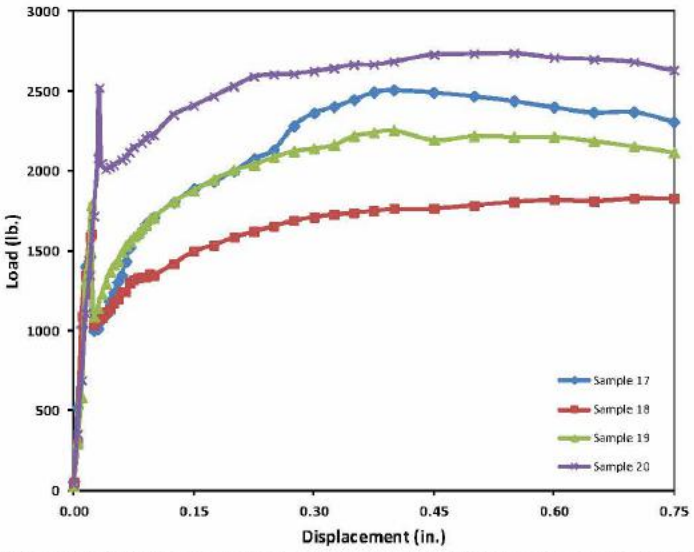


Figure 20: Initial pullout results for Type II monolithic pour tested at 24 hours.

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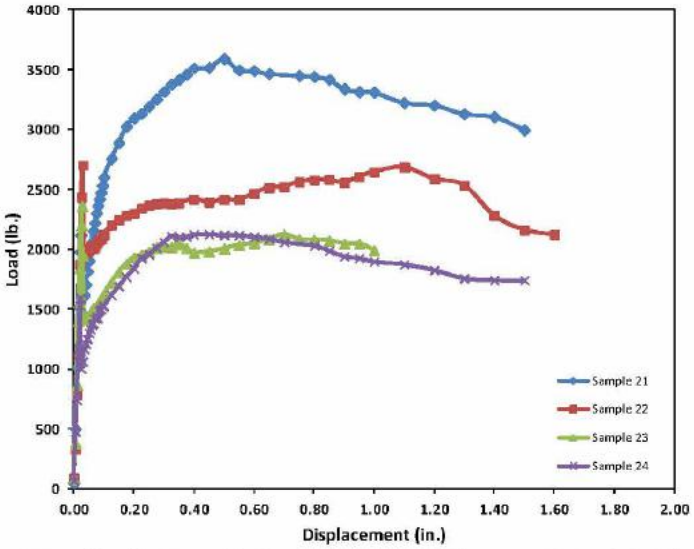


Figure 21: Full pullout results for Type IV monolithic pour tested at 24 hours.

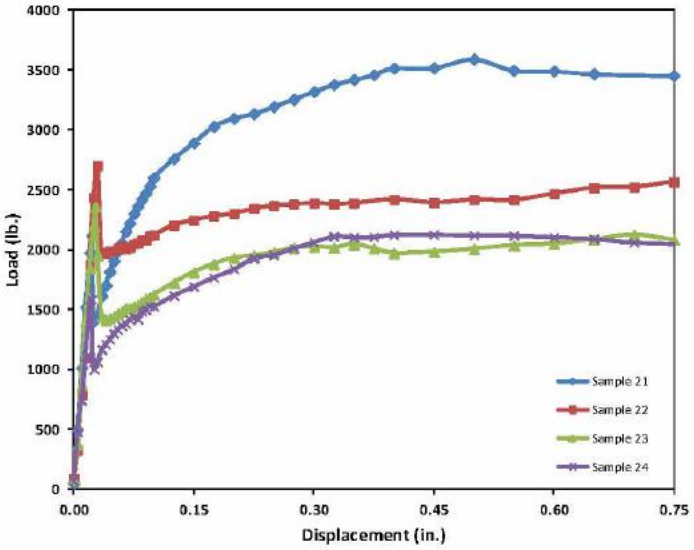


Figure 22: Initial pullout results for Type IV monolithic pour tested at 24 hours.

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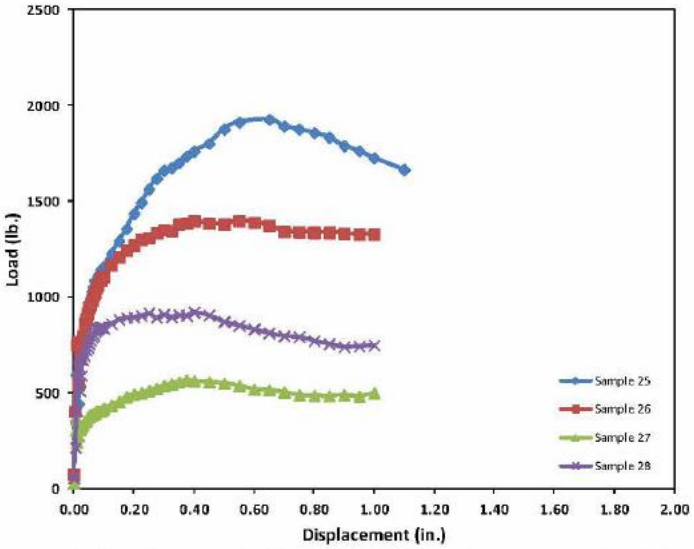


Figure 23: Full pullout results for Type II cold-joint pour tested at 24 hours.

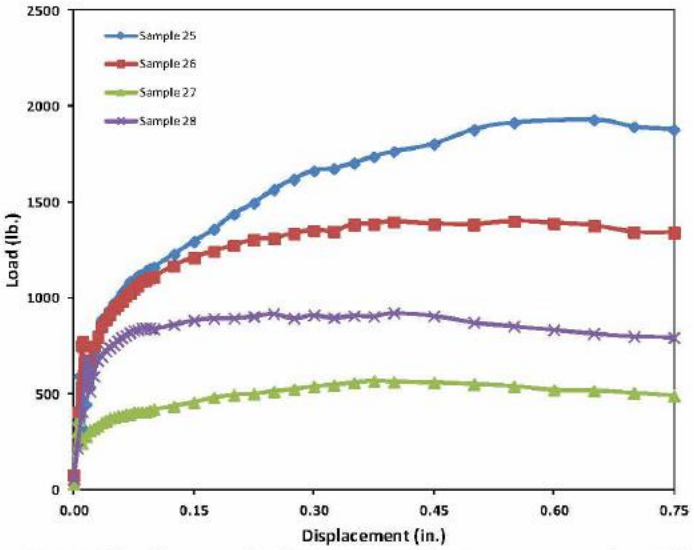


Figure 24: Initial pullout results for Type II cold-joint pour tested at 24 hours.

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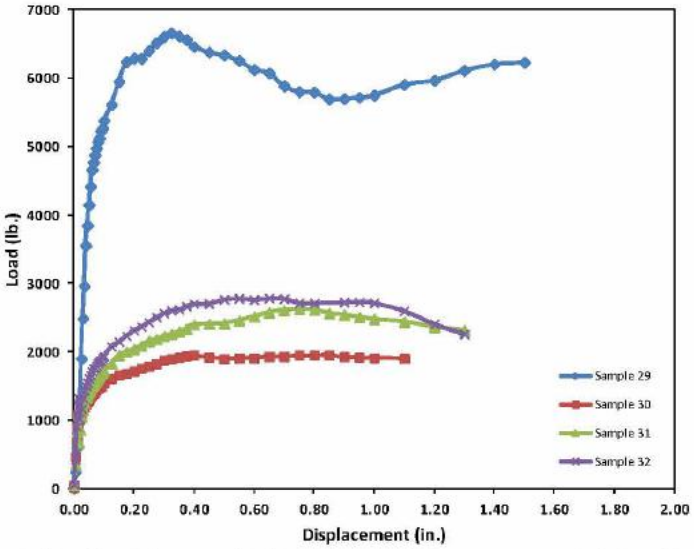


Figure 25: Full pullout results for Type IV cold-joint pour tested at 24 hours.

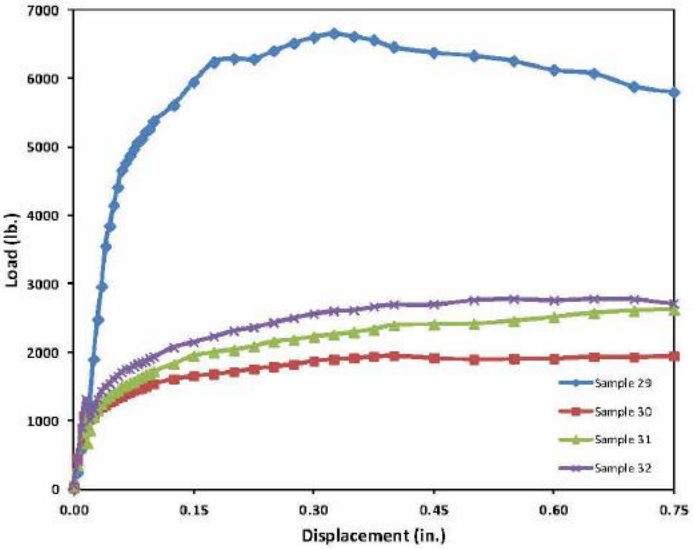


Figure 26: Initial pullout results for Type IV cold-joint pour tested at 24 hours.

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A1.3.2 Reinforced Soil Properties

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A1.3.3 Drainage Measures

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A1.3.4 Coping Top-out

Included in this section are standard engineering details for precast and cast-in-place coping.

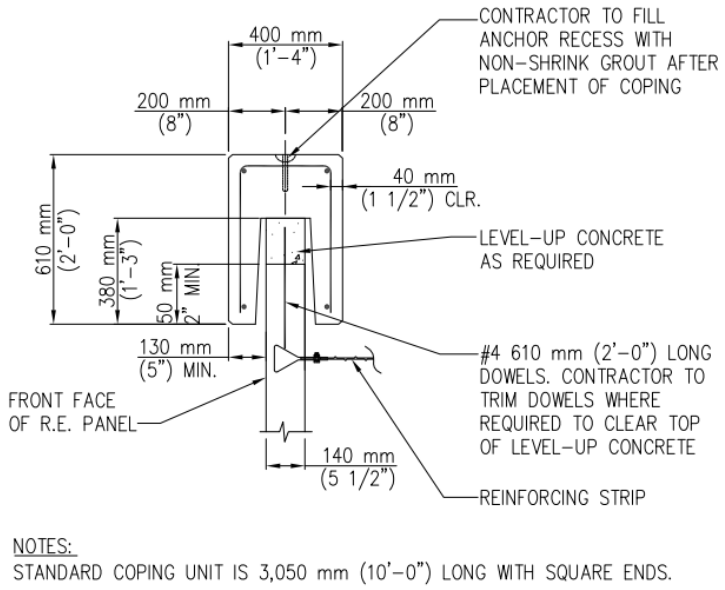


Figure A1.3.4-1: Standard precast coping section

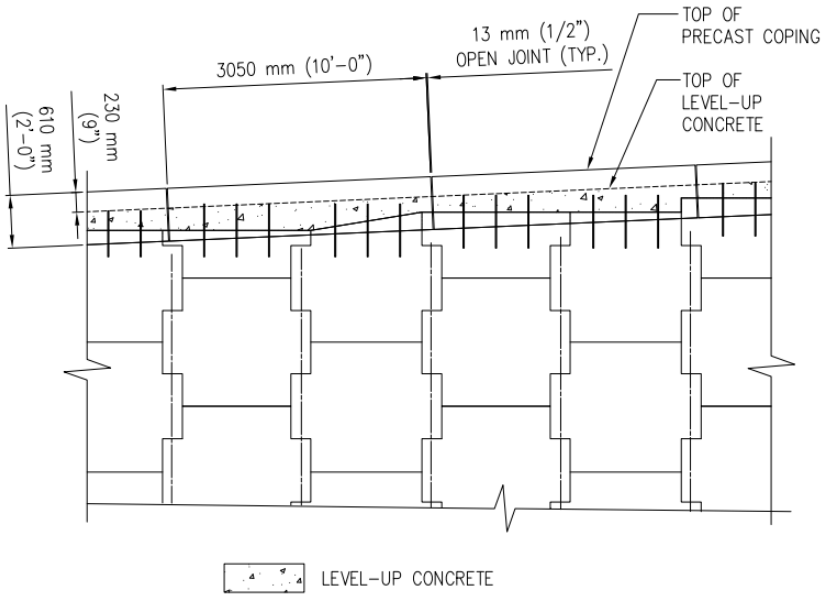
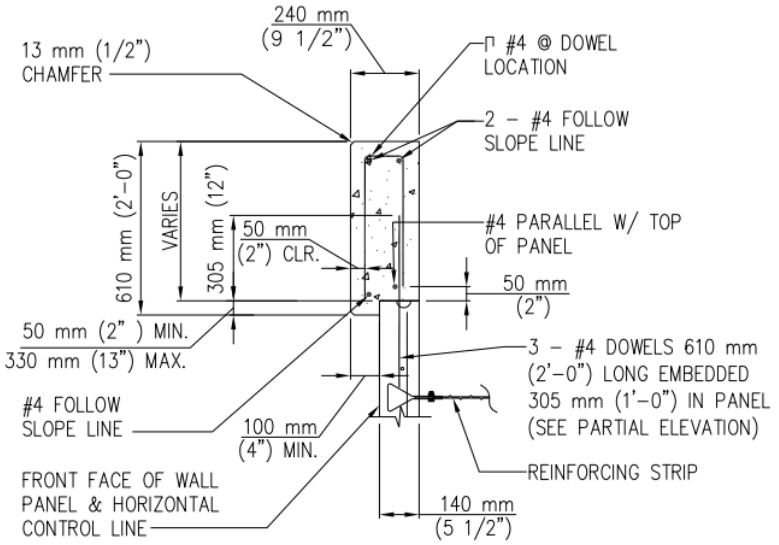
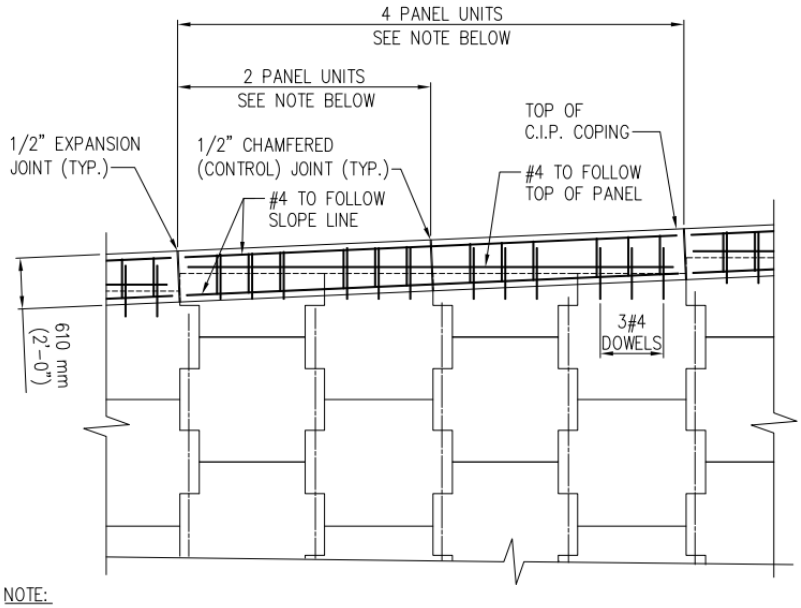


Figure A1.3.4-2: Standard precast coping elevation



NOTES:
1. CONCRETE = 3,500 psi. MIN.
2. STEEL = GRADE 60
3. ALL LONGITUDINAL BARS ARE #4
4. EXPANSION JOINTS 13mm (1/2") SHALL BE EVERY 20'± AND ALIGN WITH FACING PANEL VERTICAL JOINTS

Figure A1.3.4-3: Standard cast-in-place coping section

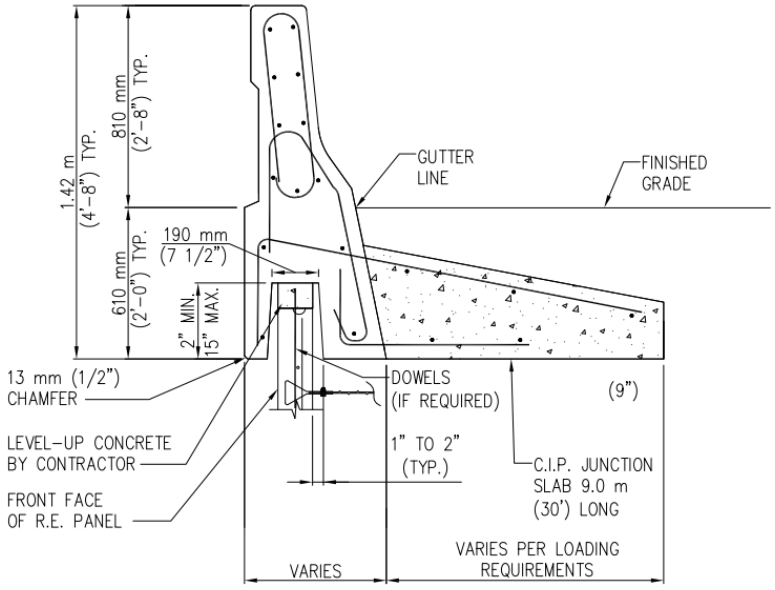


NOTE:
ONE–HALF INCH CHAMFERED (CONTROL) JOINTS SHALL BE PLACED AT EVERY TWO–PANEL INTERVAL COINCIDING WITH EVERY OTHER PANEL JOINT. ONE–HALF INCH EXPANSION JOINTS SHALL BE PLACED AT EVERY FOUR–PANEL INTERVAL WHEREBY ALL LONGITUDINAL REINFORCEMENT SHALL BE FIELD CUT TWO INCHES (2") SHORT OF EACH SIDE OF THE EXPANSION JOINTS.

Figure A1.3.4-4: Standard cast-in-place coping elevation

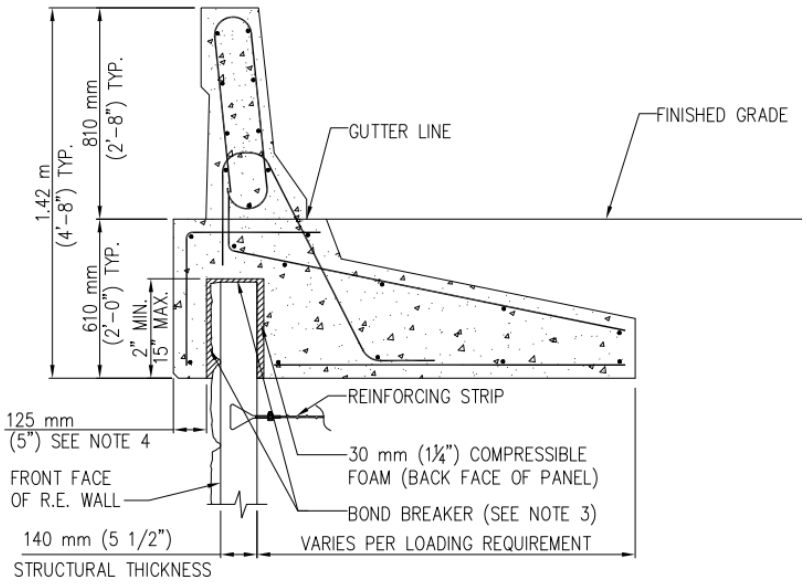
A1.3.5 Traffic Barrier Top-out

Included in this section are standard engineering details for precast and cast-in-place traffic barrier.



NOTES:
1. DIMENSIONS AND BARRIER SHAPE ARE TYPICAL, BUT CAN VARY TO MEET OWNER SPECIFICATIONS
2. BARRIER SHOWN IS FOR USE WITH ASPHALT PAVEMENT. CONTACT RECo FOR A SEPARATE DETAIL FOR USE WITH CONCRETE PAVEMENT.
3. 13 mm (1/2") CHAMFER ALL AROUND EACH END (EXPOSED SURFACES)
4. REBAR SIZE AND SPACING T.B.D. BASED ON OWNER'S SPECIFICATIONS

Figure A1.3.5-1: Standard precast traffic barrier with CIP moment slab section



NOTES:
1. DIMENSIONS AND BARRIER SHAPE ARE TYPICAL, BUT CAN VARY TO MEET OWNER SPECIFICATIONS
2. BARRIER SHOWN IS FOR USE WITH ASPHALT PAVEMENT. CONTACT RECo FOR A SEPARATE DETAIL FOR USE WITH CONCRETE PAVEMENT.
3. IF PANELS REQUIRE ARCHITECTURAL FINISH, BOND BREAKER SHOULD BE PLACED WITH 1/2" EXP. JOINT MATERIAL.
4. IF PANELS REQUIRE ARCH. FINISH, ADD THICKNESS OF FINISH TO OFFSET DIMENSION FROM STRUCTURAL FACE OF MSE WALL
5. REBAR SIZE AND SPACING T.B.D. BASED ON OWNER'S SPECIFICATIONS

Figure A1.3.5-2: Standard CIP traffic barrier section

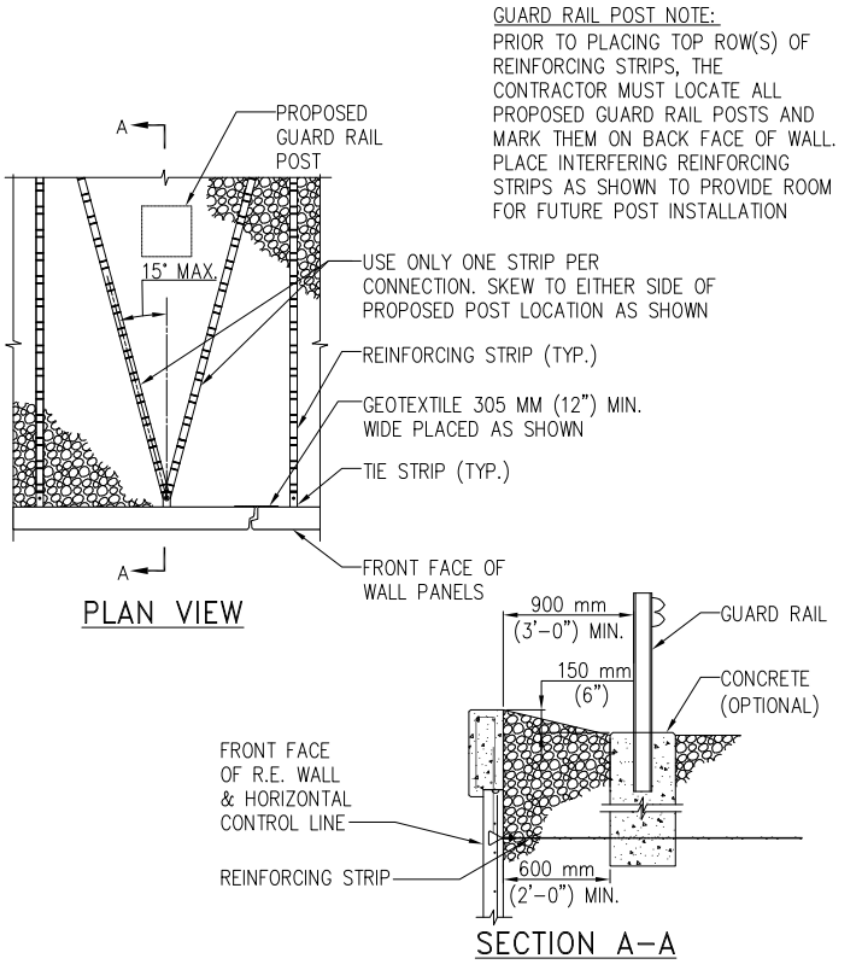


Figure A1.3.5-3: Standard details for post and beam barrier atop RE wall.

A1.3.6 Slip Joints

Included in this section are standard engineering details for slip joints.

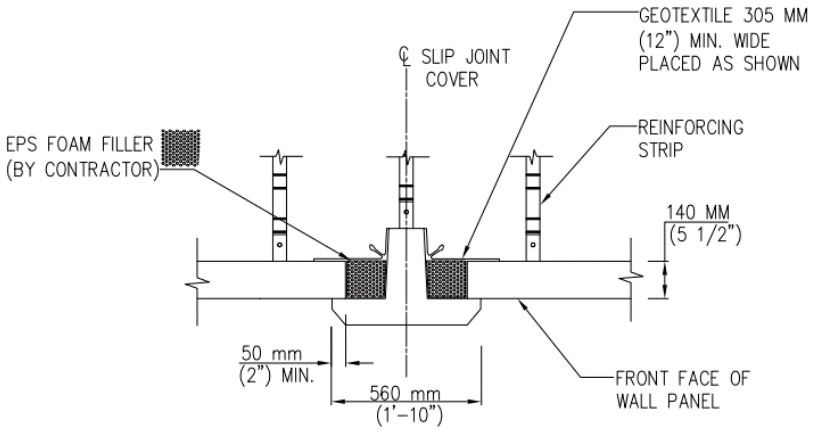


Figure A1.3.6-1: Standard slip joint cover detail

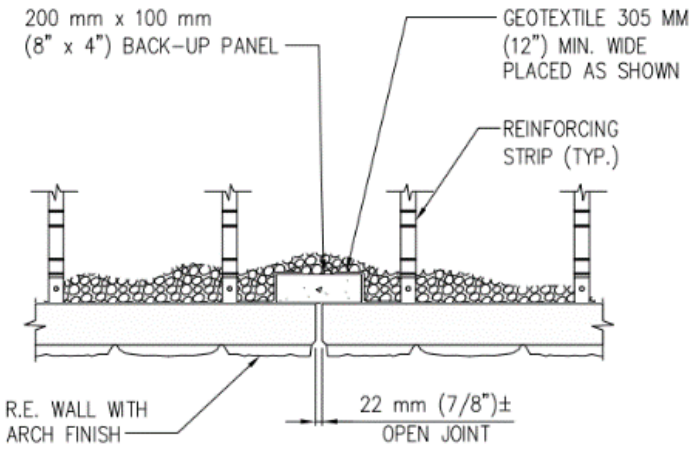


Figure A1.3.6-2: Standard internal slip joint detail

A2.0 ERS Design

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A2.1 Design Methodology

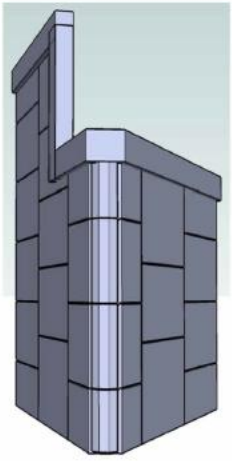
Included in this section are standard engineering details for precast and cast-in-place traffic barrier.

- RECo Technical Bulletin 2021-01, “Use of MSE walls for Acute Corners Less than 60 Degrees,” December 17, 2021 (24 pages)
- RECo Technical Memo No. 2020-01, “MSE Walls on Soft Soils,” February 28, 2020 (11 pages)
- “Unique Design Considerations for Mechanically Stabilized Earth Walls in Transportation Widening Applications”, IABSE Symposium, 2017 (8 pages)

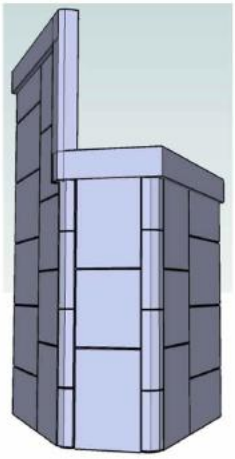
Use of MSE Walls for Acute Corners Less than 60 degrees

Tech Bulletin 2021-01

This Technical Bulletin Supersedes RECo Tech Bulletin MSE-4 (Use of MSE Walls for Acute Corners less than 70 degrees), May 1, 1992







1	6/27/2022		MRT	KPB	KPB
0	12/17/2021		KPB	KT	KPB
Rev.	Date	Modification	Design by	Checked by	Approved by

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Acute Corner- Tech Bulletin

I. Introduction

MSE structures offer significant economic benefits, especially in tight urban areas where right-of-ways are severely restricted. Skewed bridges and acute corners, in particular, benefit from MSE construction. The design and construction of Reinforced Earth structures with acute corners considerably less than 70° has been standard practice since 1986 and they have performed exceptionally well. The use of MSE structures in these applications requires a proper design procedure such as the one described below.

II. Background

A design method was developed in 1986 based on experience gained on the first acute corner project, on I-277 at Mint and Morehead Streets in Charlotte, North Carolina. In plan, the structure is a triangular closed cell, with one side being an ordinary retaining wall and the other two sides being "true" (no piles) bridge abutments. The bridge loads are distributed through a beam seat that rests directly on the reinforced soil.



Figure 1

At one end of one of the beam seats, the abutment and the retaining wall form an acute angle of 26°. The site is underlain by compressible foundation soils, resulting in approximately 5 inches of settlement of the overall structure. However, the acute corner, which is relatively light in comparison to the mass of the entire structure, only settled 1-1/2 inches. The resulting differential

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settlement of nearly 4 inches occurred over a distance of approximately 15 feet, and the panel movements caused some panels to bear against adjacent panels along their edges and corners.

The panel contact resulted in cracking of panel ears and opening of a vertical joint approximately 1 ½ inches wide from top to bottom of the wall. All the panel damage and the open joints were in one location on the abutment side of the structure at the transition from the tied-across acute corner to the normally reinforced structure. Concerns were raised by the FHWA regional office in Atlanta with respect to the performance of this new use of Reinforced Earth technology and, therefore, it was agreed to load test the structure to 115% of its maximum design load. A large loading frame and hydraulic jack, restrained by vertical rock anchors drilled and grouted into bedrock, was used to load the affected end of the abutment. The wall was carefully monitored for vertical and lateral wall movements during this loading process. No movements were detected. The structure was subsequently completed and placed in service in 1987 and has performed as intended.

In 1992, The Reinforced Earth Company (RECo) issued Technical Bulletin MSE-4, Use of MSE Walls for Acute Corners less than 70 degrees. Since that time, hundreds of Reinforced Earth structures with acute corners between 25° and 70° have been constructed and are performing as intended. A representative list of completed projects can be obtained from RECo Engineering upon request.

Based on 30+ years of experience in design acute corners, RECo has learned the following:

- For wall flexibility and constructability, the tied-across nose section should be as small as possible.
- When feasible, the nose should be truncated (see Appendix A)
- Inclusion of deep foundation elements in the acute corner area creates additional obstructions, but discrete strips may be splayed to clear the deep foundation elements. It is recommended that deep foundation elements in the acute corner area be less than 2 ft diameter, and a minimum of 18" clear from back face of MSE facing elements.
- Acute corners (less than 70°) at temporary phase line locations should be constructed with precast concrete facing panels and not wire facing units (typically the first 5 to 15 ft of phasing wall)

III. Design Method and FLAC Models

Between 1986 and 2020, the standard design approach was to detail corners less than 70 ° as a tied-across bin structure. The 70-degree angle limit was determined by RECo, and then later adopted by AASHTO, based on a simple geometric approach to limit the splay angle of reinforcements to less than 15 degrees. We have subsequently researched the effects of splaying reinforcements (both in the pullout resistance of the reinforcement and the tie strips, as well as the stress on the tie strips), and confirmed that individual reinforcements may be splayed up to 30 degrees with only minor reduction in ultimate pullout resistance. This reduction in pullout resistance is negligible and does not affect the design pullout resistance (see Tech Bulletin 2020-1 for more information).

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Based on the research of splayed reinforcements, the unique bin wall design required for acute corners has been relaxed to angles less than 60°. The nose portion of the structure is designed as an internally tied bin structure with earth pressures estimated by Formula 1 (see below). The reinforcing strips are designed as tension members only and are generally at low stresses due to the bin effect (soil arching). The tied bin section is separated from the adjacent normally reinforced portion of the structure by slip joints. The slip joints allow the relatively light bin section to act independently of the rest of the structure. In addition, reinforcing strips are extended back from the nose portion of the structure to tie this relatively light section back into the major portion of the mass.

20 and 30 foot tall wall models were created using FLAC3D to better understand the behavior and interaction of the soil reinforcement within confined spaces.

Summary of Findings in the FLAC Models

- For acute corners 60 degrees or greater, the reinforcements may be splayed. The stresses in the reinforcements are low (see plots in Appendix B) and may be estimated with the Bin Equation (Formula 1). However, for simplicity, RECo will routinely use the coherent gravity method for estimating the reinforcing strip tensions. This is a conservative approach as shown on the plots in Appendix B.

- For tied across acute corners less than 60 degrees, the FLAC models indicate that Extended Full Length (EFL) Strips have little tension on them in the static case (See Appendix C). The FLAC models also indicate no "active" zone behind the acute corner nose. However, they are provided to satisfy conventional external stability calculations for factors of safety and capacity/demand ratios. The EFL strips provide redundancy in case of a seismic event or other extreme event.

- FLAC models indicate stress in tied reinforcements on the order of 30% to 60% of what is predicted using bin pressure equations. Horizontal earth pressure (σh) in the tied across bin may be conservatively estimated by the following equation based in part on Janssen's original arching theory (as defined by Handy, with the addition of the surcharge term "q"):

$$\sigma_h = \frac{\gamma W}{2 \tan \Phi}\left(1 - e^{-2K_o \tan \Phi \frac{Z}{W}}\right) + qK_o\left(e^{-2K_o \tan \Phi \frac{Z}{W}}\right)$$

Formula 1

Where,

- W is the width of the section in question
- Z is the depth of the point of interest below the top of the wall
- ∅ is the interface friction angle between the soil
- K_o is the at rest earth pressure coefficient
- γ is the unit weight of the backfill.
- q is the surcharge load (typically 250 psf)

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- The bin wall theory used in the design spreadsheet has been verified by FLAC3D and now bases the earth pressure on the acute corner on ϕ=40°, which is a conservative equivalent angle of the reinforced select fill.

DESIGN APPROACH

The design approach shall include the following:

1. Define the critical nose section of the acute corner.
2. Design the nose section as a bin structure capable of tolerating differential settlements.
3. Design the nose restraint reinforcements, extending from within the nose section back into the fill, beyond all potential failure planes.
4. Check the structural capacity of soil reinforcements and connection devices.

DEFINING THE NOSE SECTION

The nose section of the acute corner shall be defined as the smallest block extending back from the point(s) of intersection of the wall faces with sufficient mass to be stable (CDR >1.0, at Service I Limit State) against sliding along its base without considering the restraining effect of soil reinforcements extending beyond the potential failure plane behind the nose section, see Figure 2. The calculation to determine the dimensions of the nose section shall conform to the following:

a. Determine the limits of the acute corner geometrically by locating where the reinforcing strips may be splayed up to an angle of 30 degrees.

b. Locate a slip joint at the panel edge nearest the splayed reinforcing strip.

c. Assume a Rankine failure plane intersecting the base of the wall at a joint between facing units and at an angle of 45 + Ø/2, where Ø is the backfill angle of internal friction as established for wall design on the project. (This assumption is conservative based on FLAC model results and may be modified on a project by project basis with additional numerical modeling)

d. Assume equal dimensions along the two wall faces extending back from the point of intersection to the limit of the nose section.

e. Calculate the driving forces, P_A and P_S, behind the nose section due to earth pressure and surcharges

f. Calculate the Factored Sliding stability of the nose section mass along its base, using the effective length of reinforcement past the driving wedge to provide additional resistance.

g. Soil reinforcements shall be designed to restrain the nose section by connecting directly to each of the facing elements in the nose section and running back into the backfill to a point at least six (6) feet, (2m) beyond the Rankine failure surface defined above.

DESIGNING THE NOSE SECTION AS A BIN

Structural connections of soil reinforcements between the wall elements within the nose section shall be designed (using Formula 1 to estimate internal earth pressures) to create a stable bin structure without eliminating the flexibility of the facing to tolerate differential settlements. The design for connections to facing elements and allowable stresses in the connecting members shall conform to the requirements for

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the wall design in general. The nose section shall be designed with a slip joint to allow it to settle differentially from the remainder of the structure.

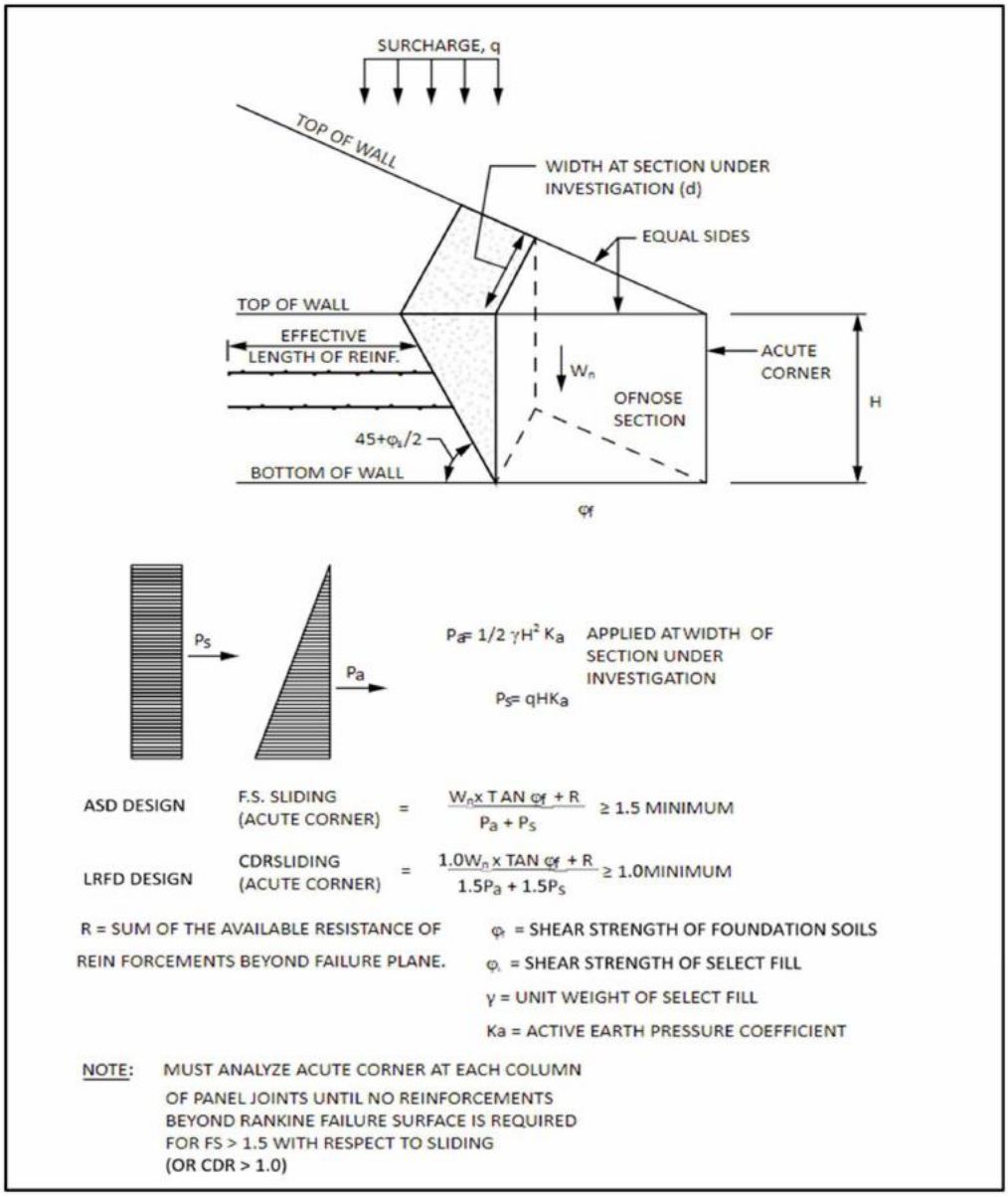


Figure 2

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Design Details

The performance of this system has been remarkably good due to its high strength and flexibility. In addition, the individual reinforcing strips and numerous connection locations on the back of the panels make the system very versatile and constructible in the presence of obstructions such as piles.

Although internal stability suggests that no EFL strips are necessary, the acute corner design should include a minimum of 3 per level spaced a maximum of 5 ft vertically.

Phasing, abutment piles, and other construction uncertainties have created additional complexities within the acute corner mass. Tied-across smooth strips provides a solution for reducing the congestion of crisscrossing and excessively skewed reinforcing strips. RECo has also investigated the effect of high splay angles on the tensile and pullout capacity of embedded tie strips. The results of the testing indicate that splay angles up to 30 degrees result in a minor reduction in pullout capacity of the tie strip, but the steel strength in tension (at end of service life) is the controlling factor. The testing shows that bending moments are not developed in the tie strip over the 2" moment arm due to the confining stress provided by the surrounding backfill.

Acute corners constructed with any MSE system will perform adequately if designed and detailed to take into account the three-dimensional stresses and geometrical constraints. However, the design and details cannot be addressed in the field merely by cutting reinforcement members to clear obstructions or to fit in limited spaces. Complete and comprehensive calculations and reinforcement layout details must be a part of the final design submittal and must be reviewed and approved prior to construction.

Details of three different types of acute corners are enclosed in Appendix A: a) a 30-degree acute corner with a truncated nose (Figure A-1a), b) a 45-degree acute corner (Figure A-1b) and c) a 60-degree acute corner (not tied across) Figure A-1c.

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Summary and Recommendations

Design

- When two intersecting Reinforced Earth® walls form an enclosed angle of less than 60°, the structure should be designed in accordance with the recommendations included herein.
- Discrete Reinforcing Strips may be splayed up to 30 degrees when attached to independent facing panels
- Extended Full Length (EFL) reinforcing strips may be skewed more than 30 degrees when attached to tied-across panels (recall that the tension in EFL reinforcing strips is very low).
- Nose sections should be truncated where feasible for constructability and aesthetics
- Inclusion of deep foundation elements in the acute corner area creates additional obstructions, but discrete strips may be splayed to clear the deep foundation elements. It is recommended that deep foundation elements in the acute corner area be less than 2 ft diameter and placed a minimum of 18" clear of back face of MSE wall facing elements.
- Acute corners (less than 70 degrees) at temporary phase line locations should be constructed with precast concrete facing panels and not wire facing units (typically the first 5 ft to 15 ft of phasing wall)
- In special circumstances, discrete reinforcing strips may be splayed in excess of 30 degrees with proper analysis.

Construction

- Backfill in tied-across acute corner sections should have low fines (<5%). Course aggregate (such as #57 stone) is preferred, but sands are feasible.
- Thin Lifts (~ 8") are typically necessary to achieve compaction with small walk behind equipment.
- Compaction should be a minimum 95% AASHTO T-99 (Standard Proctor). Heavier compaction (modified proctor) is not recommended due to confined space of acute corner.

References:

Handy R.L., 1985. The Arch in Soil Arching. Journal of Geotechnical Engineering, ASCE 111, No. 3, 302-318

Jayawickrama P., Ghebrab T., Lawson W. "A Comprehensive Field Guide for Reconfiguration of MSE Reinforcement Around Obstructions". Texas Tech University Multidisciplinary Research in Transportation Research Report 5-6493-01-P1. Dec. 12, 2017.

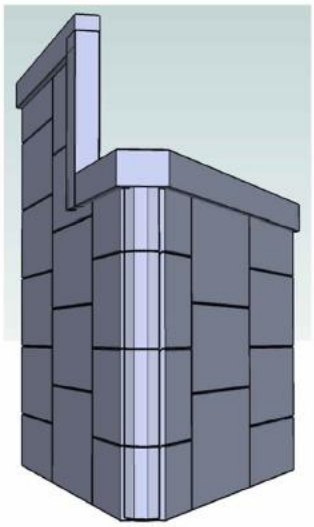
Kniss, Wright, Zornberg, Yan (2007) Design Considerations for MSE Retaining Walls Constructed in Confined Spaces. Center for Transportation Research, The University of Texas at Austin.

FHWA-NHI-10-024. 2009. Acute Angle Corners, pg 5-42.

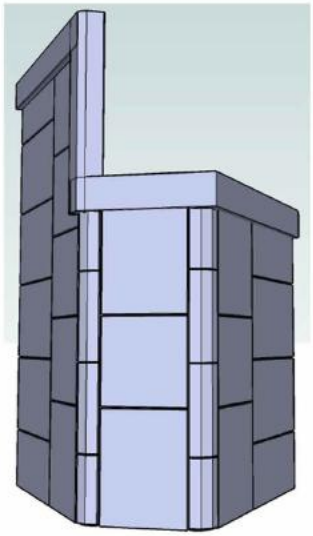
AASHTO LRFD Bridge Design Specifications, 9th ed. 2020, Chapter 11, Section 11.10.1.

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Typical Acute Corner



Acute Corner with Truncated Nose

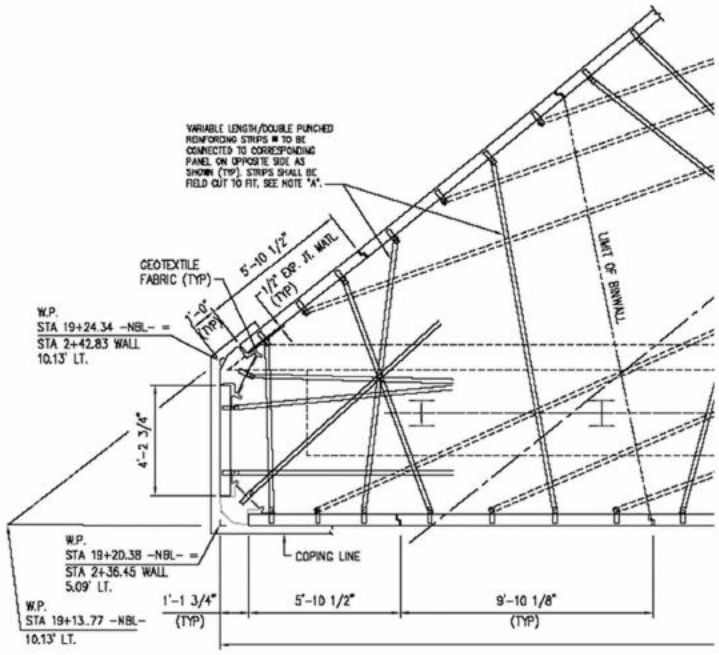


Figure A-1a- 30° A.C. w/ Truncated Acute Corner – Plan View

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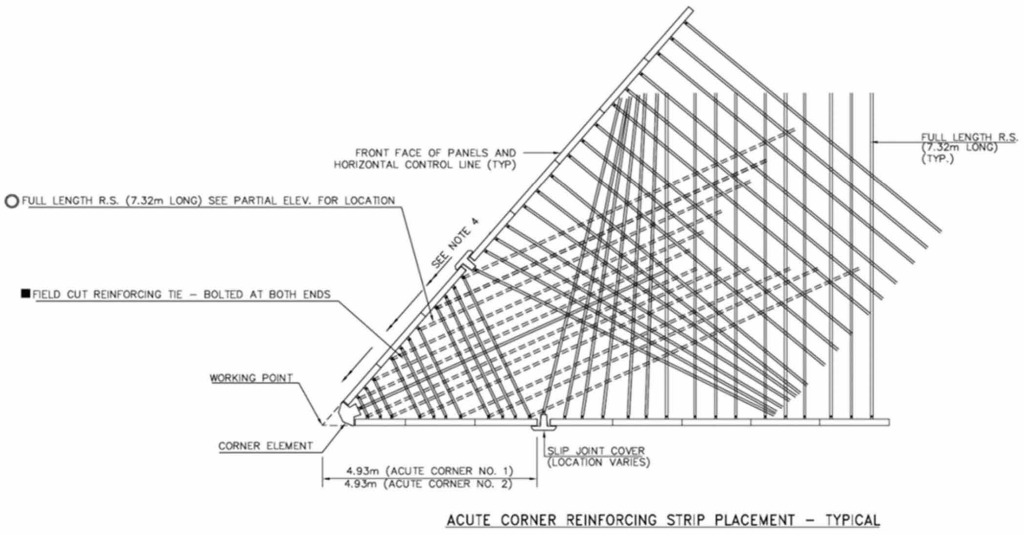


Figure A-1b - 45° Typical Acute Corner – Plan View

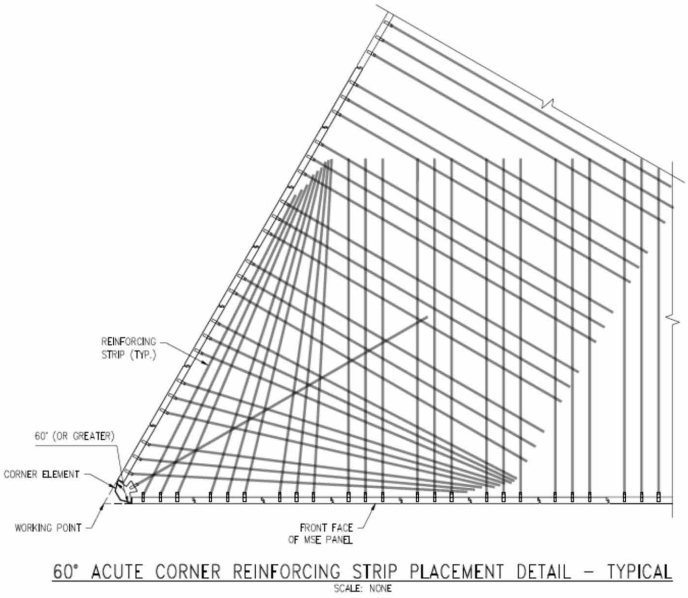


Figure A-1c – 60° Typical Acute Corner – Plan View

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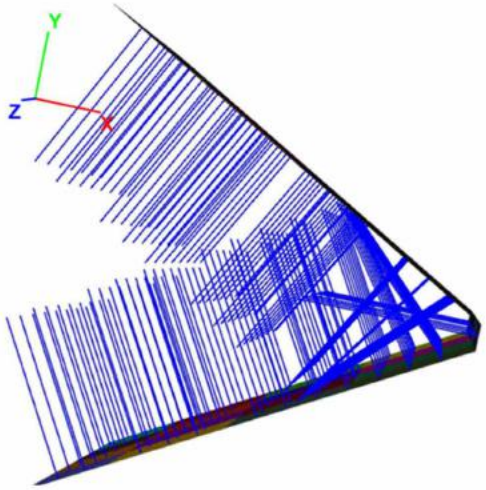
Appendix B

Numerical Modeling of Acute Corner (FLAC3D)

Wall height: 20 ft; Reinforcing Strip Length: 14 ft; Acute Angle = 60 degrees

Reinforcing Strip Type: 50mm x 4mm Ribbed (A572, Grade 65). Reinforcements NOT tied accross

Select Backfill:Ø= 34°; γ = 120 pcf



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Figure B-1: 60 degree Acute Corner Plan View

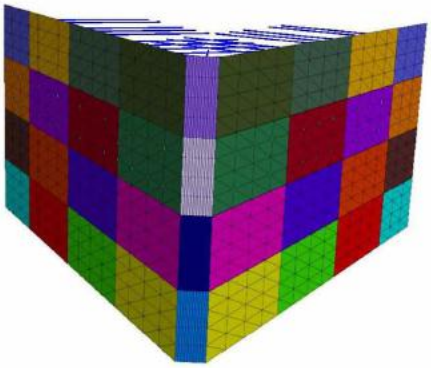


Figure B-2: 60 degree Acute Corner Elevation View

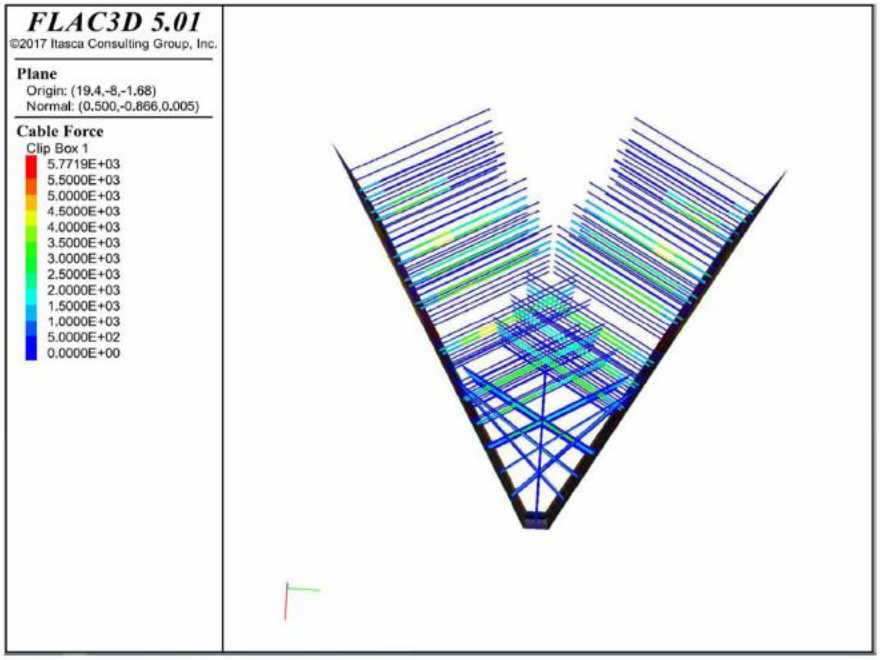


Figure B-3a

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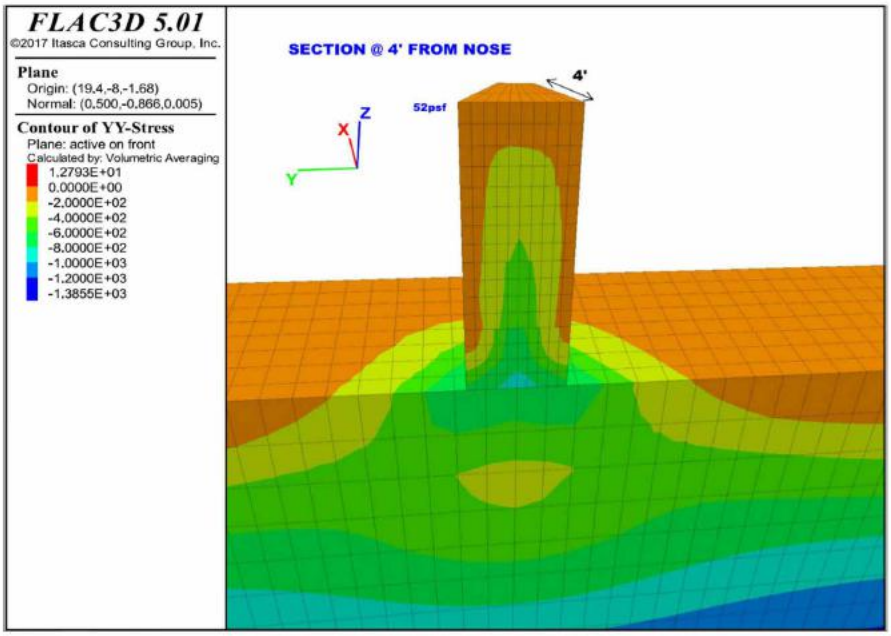


Figure B-3b

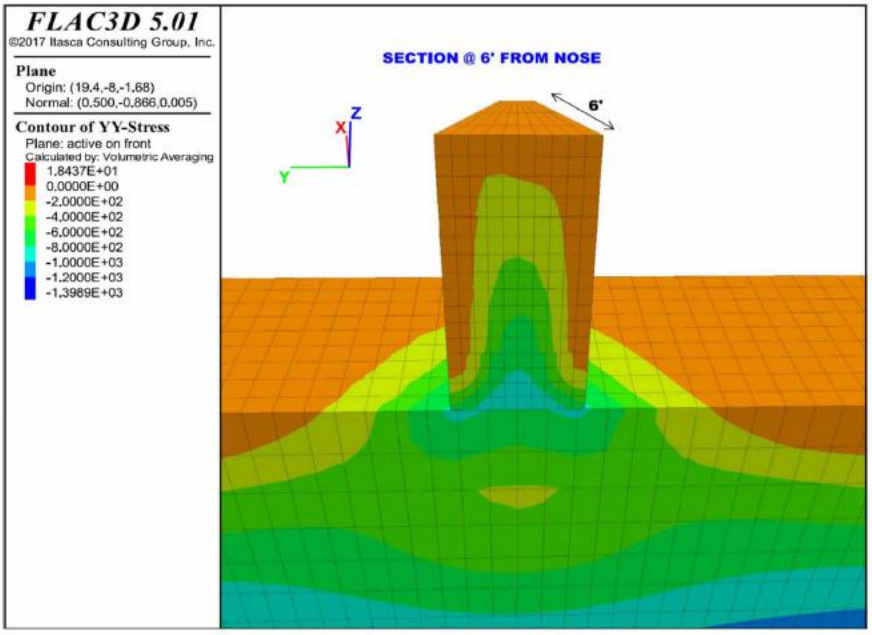


Figure B-3c

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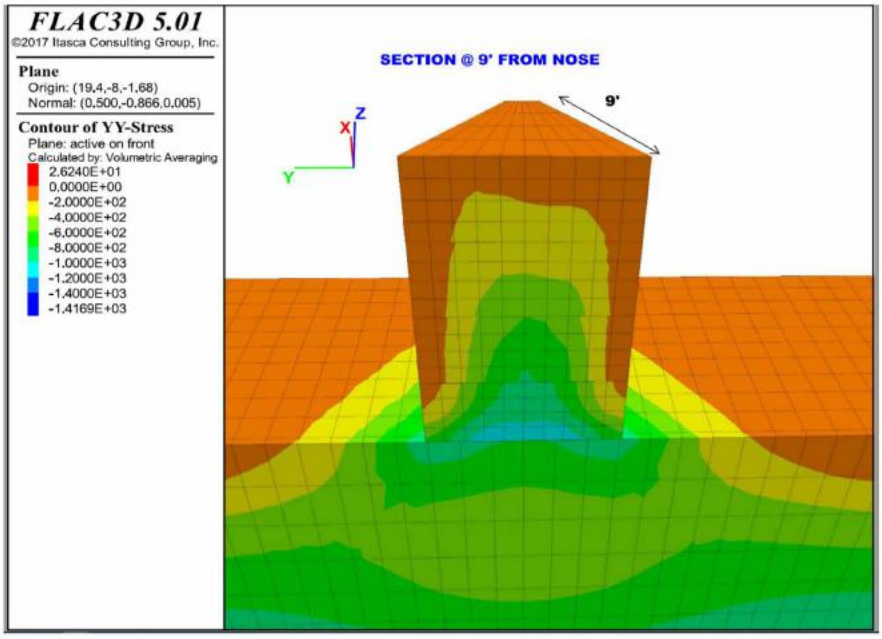


Figure B-3d

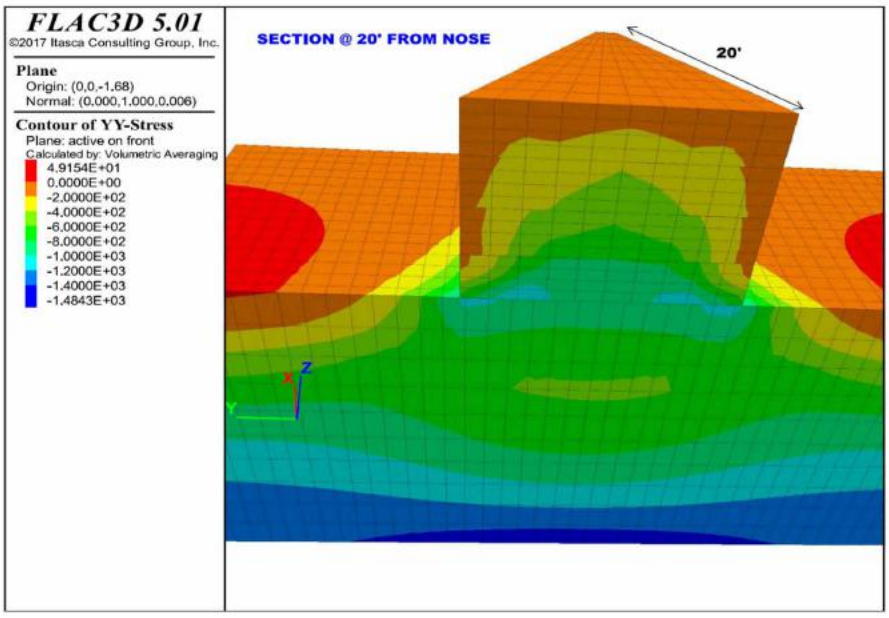


Figure B-3e

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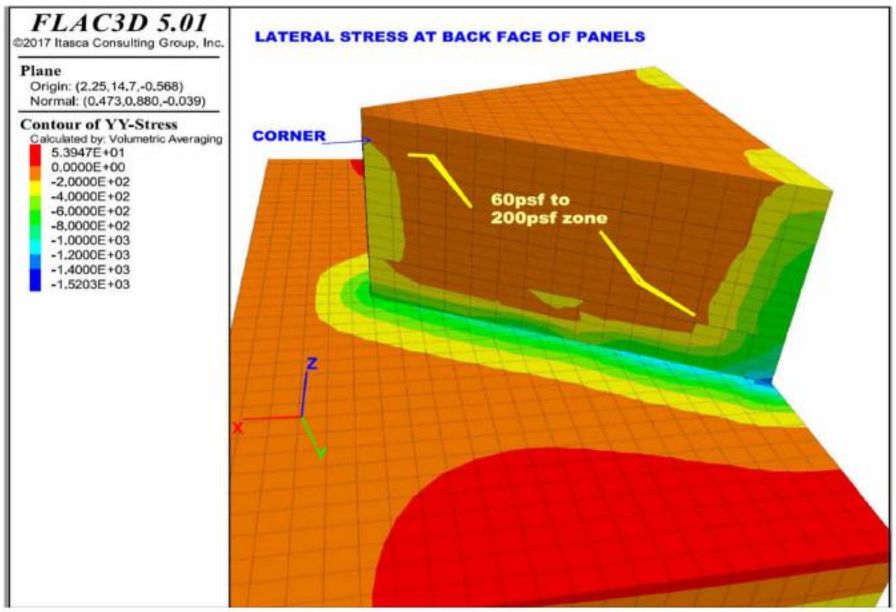


Figure B-3f

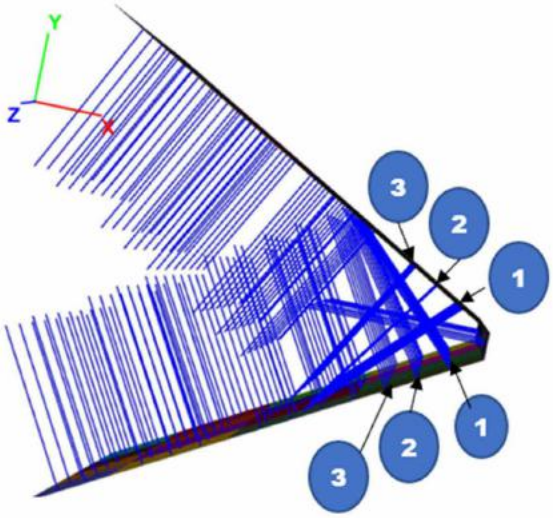


Figure B-4

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TABLE OF REINFORCING STRIP TENSIONS FROM FLAC ANALYSIS

BIN WIDTH

DEPTH	height	LEVEL #	1 STRIP SKEWED 4.26ft	2 STRIP SKEWED 6.12ft	3 BIN WIDTH- STRIP O.R.W SKEWED LEFT SIDE 9.00ft	BIN WIDTH- STRIP O.R.W SKEWED RIGHT SIDE 9.00ft	13.23ft	15.31ft	17.21ft	22.24ft	26.00ft	
1.25	18.75	8	635lbs	1029lbs	1101lbs	2076lbs	2064lbs	1709lbs	1947lbs	2529lbs	1547lbs	FLAC RESULTS IN LBS
3.75	16.25	7	960lbs	1041lbs	1206lbs	2503lbs	2115lbs	1615lbs	2420lbs	2589lbs	1738lbs	
6.25	13.75	6	1354lbs	2357lbs	2574lbs	2929lbs	2941lbs	2370lbs	3160lbs	3016lbs	2293lbs	
8.75	11.25	5	1484lbs	2779lbs	2743lbs	3376lbs	2525lbs	3389lbs	3466lbs	2936lbs	4367lbs	
11.25	8.75	4	2060lbs	3300lbs	3457lbs	3822lbs	3804lbs	3304lbs	4259lbs	4440lbs	4335lbs	
13.75	6.25	3	2900lbs	3383lbs	3542lbs	3764lbs	3772lbs	3546lbs	4418lbs	4863lbs	5263lbs	
16.25	3.75	2	1419lbs	3406lbs	3623lbs	3623lbs						
18.75	1.25	1	1155lbs	1178lbs	1295lbs	1295lbs						

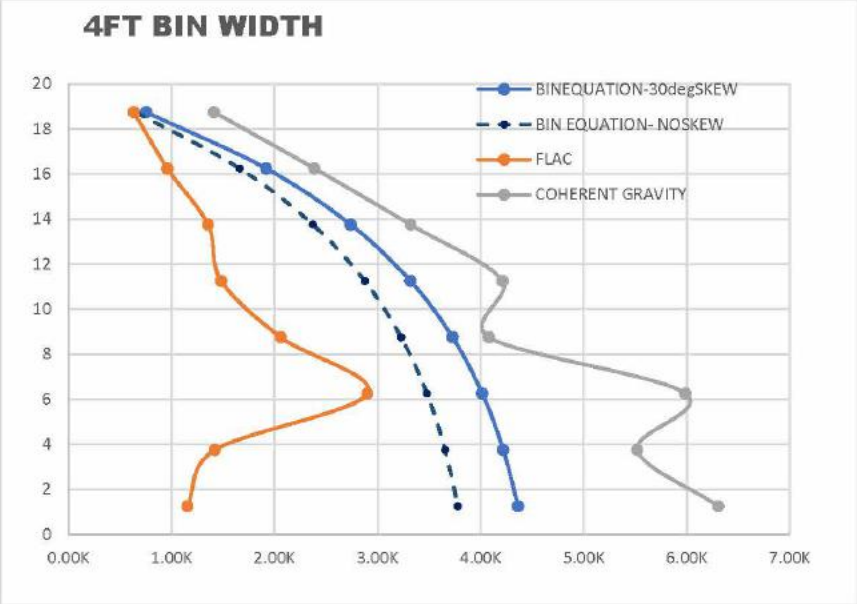


Figure B-5a

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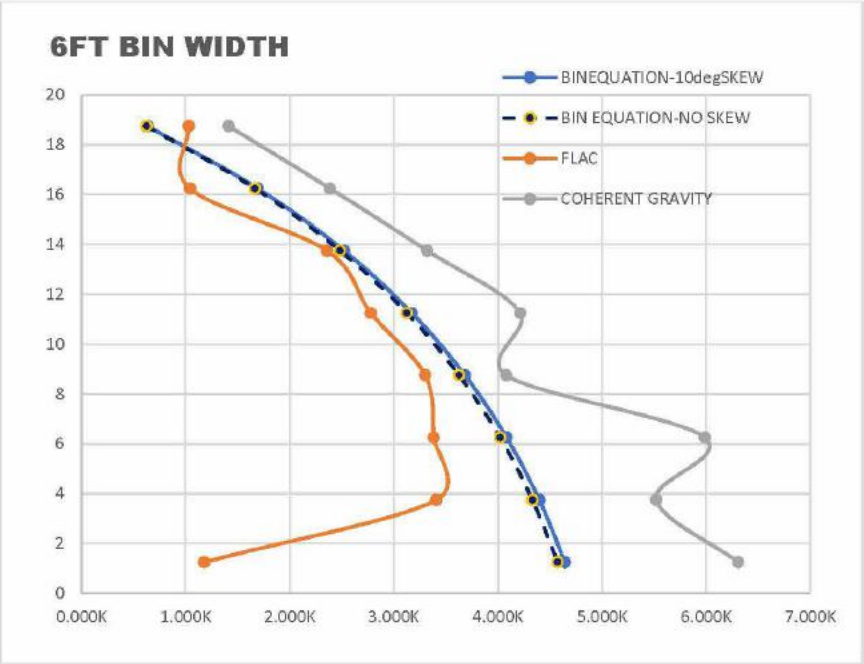
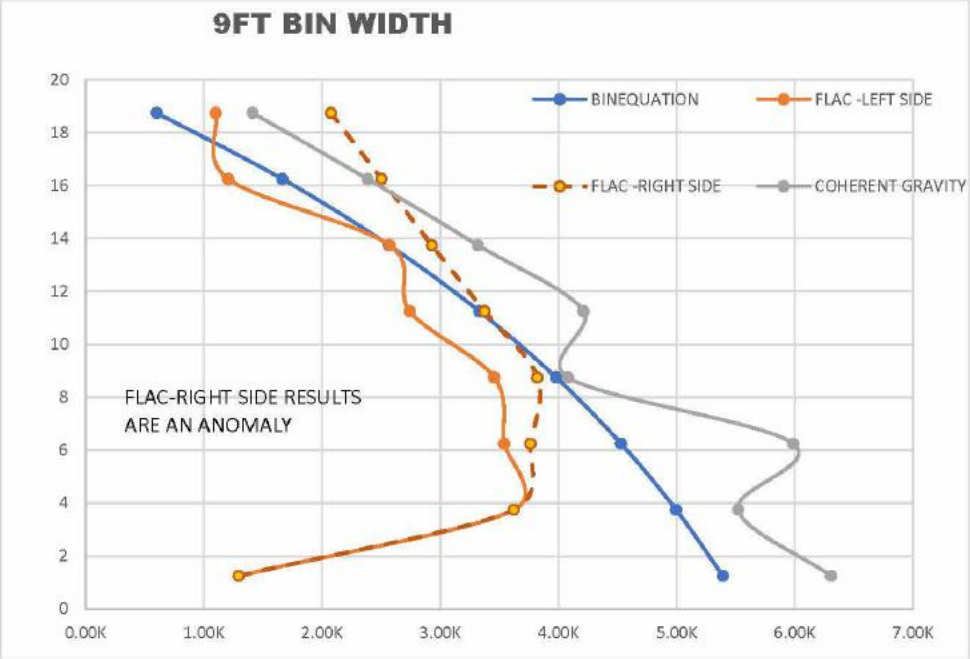


Figure B-5b



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Figure B-5c

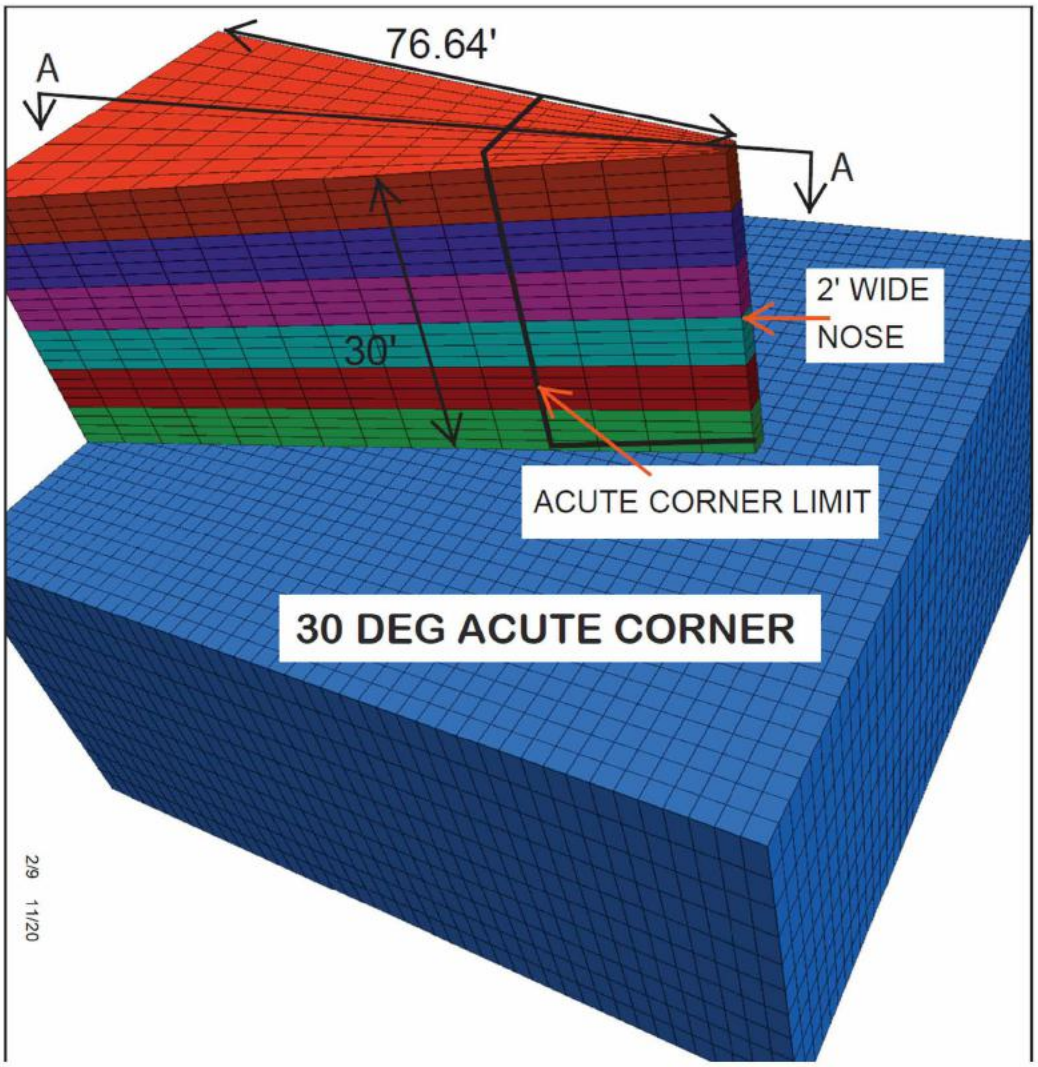
Appendix C

Numerical Modeling of Acute Corner (FLAC3D)

Wall height: 23 ft; Reinforcing Strip Length: 21 ft; Acute Angle = 30 degrees

Reinforcing Strip Type: 50mm x 4mm Ribbed (A572, Grade 65). Reinforcements NOT tied accross

Select Backfill:Ø= 34°; γ = 120 pcf



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Figure C-1

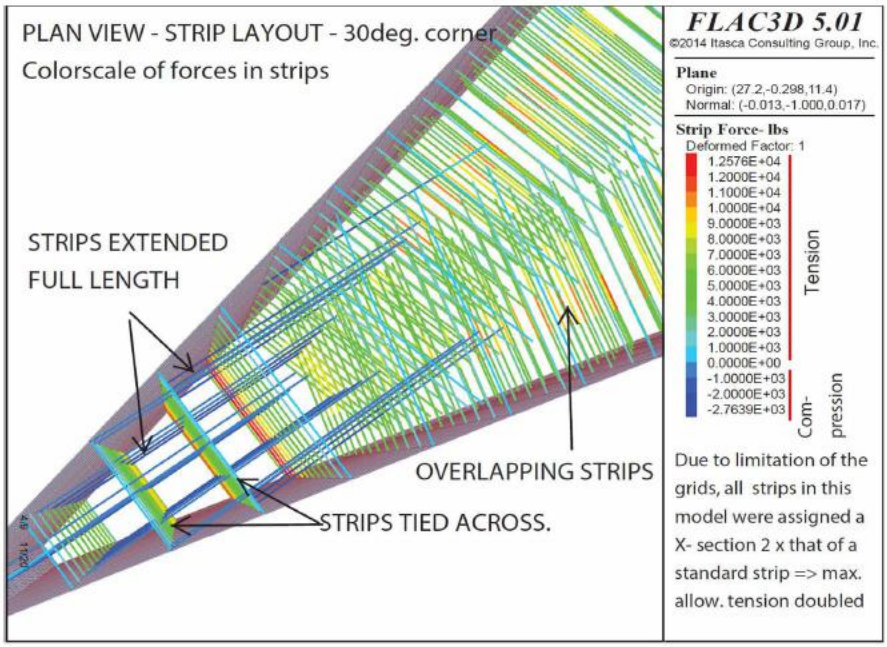
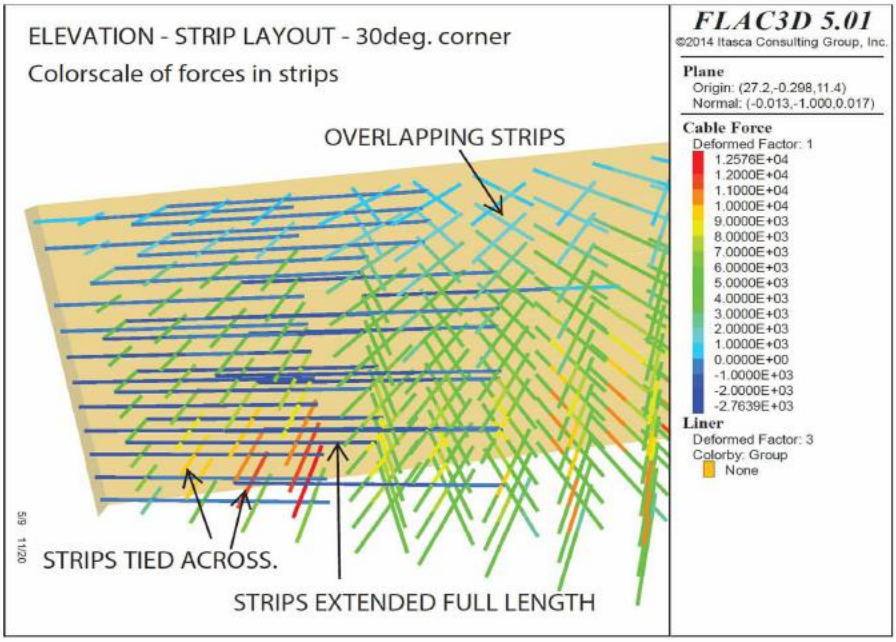


Figure C-2



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Figure C-3

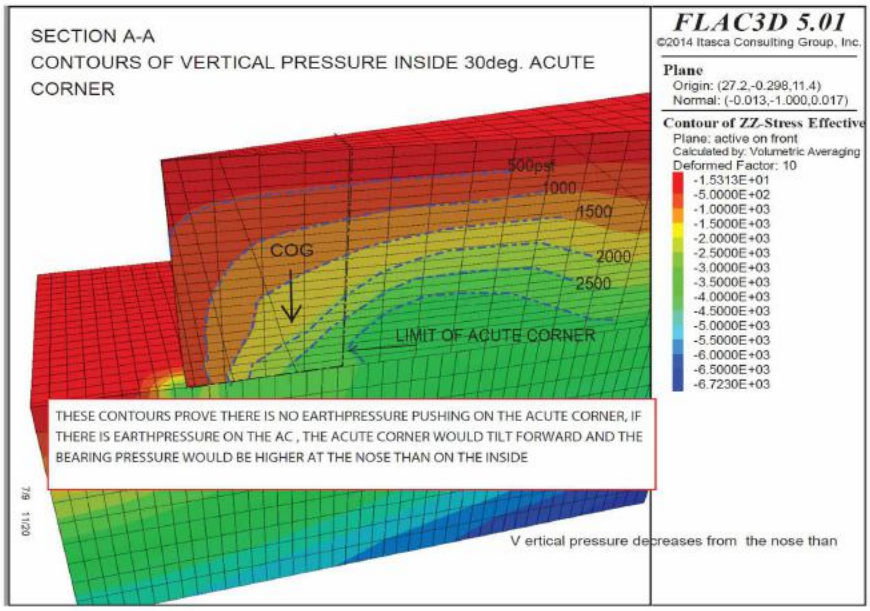
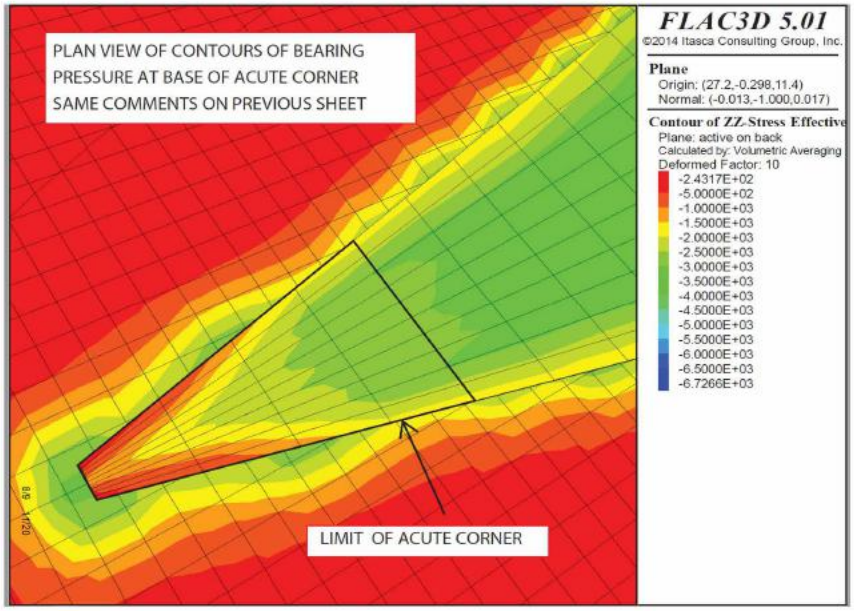


Figure C-4



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Figure C-5

TABLE OF REINFORCING STRIP TENSIONS FROM FLAC ANALYSIS

FLAC RESULTS IN KIPS

BIN WIDTH

			STRIP TIED ACROSS			
DEPTH	height		5.44ft	7.10ft	9.67ft	12.20ft
1.25	28.75	1	0.32	0.41	0.45	0.48
3.75	26.25	2	0.87	1.01	0.98	0.97
6.25	23.75	3	1.47	1.74	1.71	1.71
8.75	21.25	4	2	2.34	2.31	2.23
11.25	18.75	5	2.47	2.97	3	2.99
13.75	16.25	6	2.77	3.35	3.47	3.51
16.25	13.75	7	3.09	3.81	4.09	4.31
18.75	11.25	8	3.21	4.15	4.6	4.9
21.25	8.75	9	3.61	4.9	5.52	5.81
23.75	6.25	10	4.13	5.56	6.09	6.27
26.25	3.75	11	3.87	5.2	5.95	6.01
28.75	1.25	12	0.83	1.19	2.35	2.16

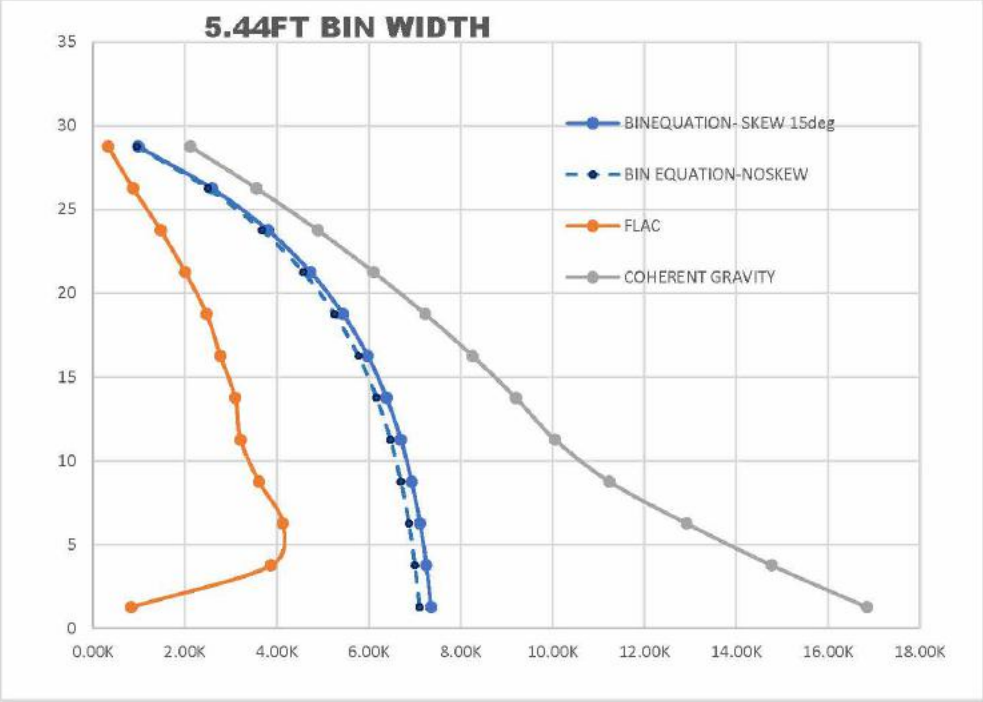


Figure C-6

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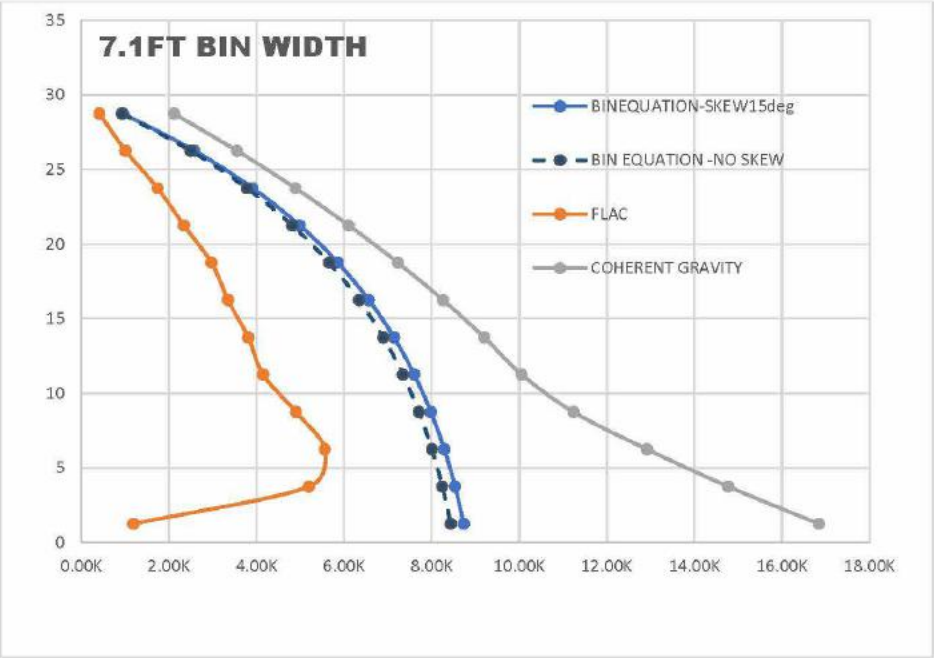


Figure C-7

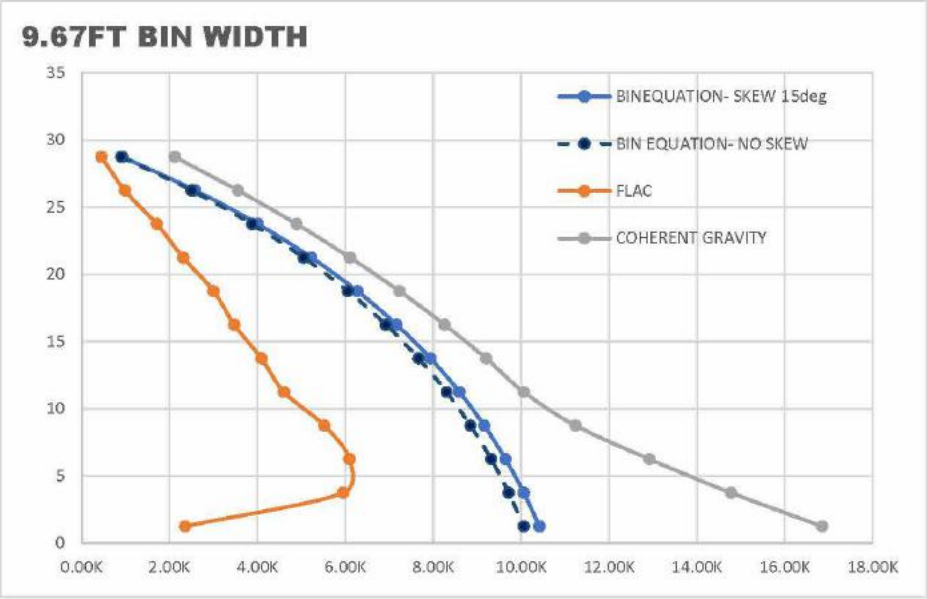


Figure C-8

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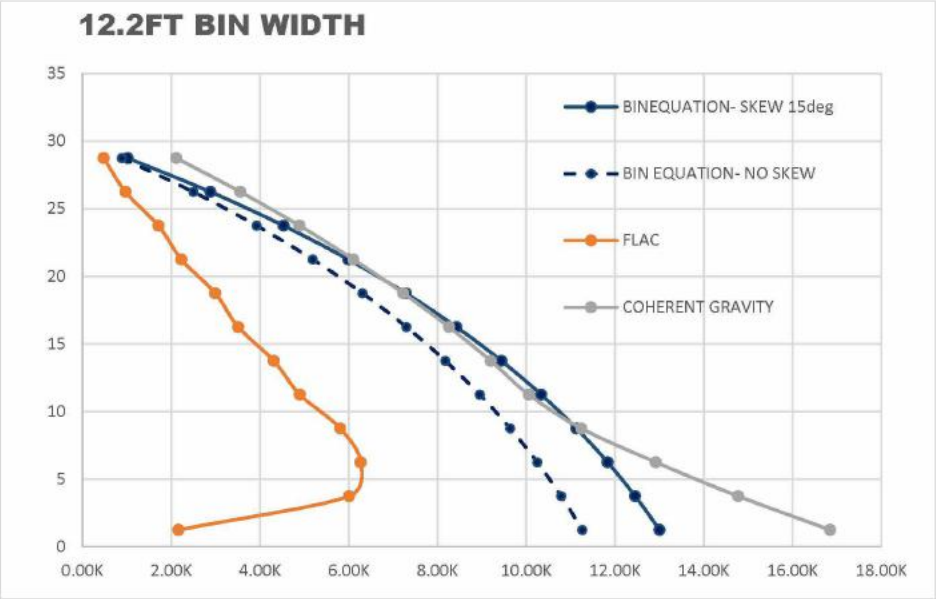


Figure C-9

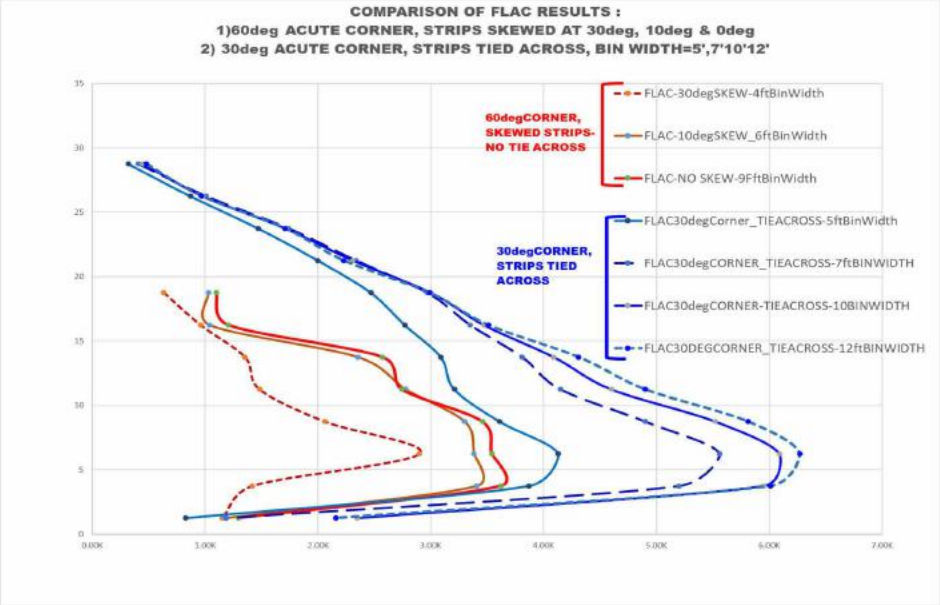


Figure C-10

FLAC 3D Material Parameters

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SELECT FILL				FOUNDATION		
TYPE	MOHR-COULOMB			TYPE	MOHR-COULOMB	
Density	120	pcf		Density	120	pcf
Elastic Modulus	1.E+06	lbf/ft^2		Elastic Modulus	8.E+05	lbf/ft^2
Poisson's Ratio	0.3			Poisson's Ratio	0.3	
Friction Angle	34	degrees		Friction Angle	30	degrees
Dilation Angle	4	degrees		Dilation Angle	0	degrees

STRIPS MODELLED AS CABLES IN FLAC3D			
Calculation Width	10.00	ft	
Strip Width	0.16	ft	
Strip Thickness	0.013	ft	
E	2.90E+04	Ksi	
Tensile Force	7200	lbf	
Compressive Force	7200	lbf	
Tensile Strain	0.12		
Fiction Angle	34	degrees	VALUES RECOMMENDED BY ITASCA
Shear Stiffness	5.00E+07	psf	
Cohesion	0	lbs/ft	

PANELS MODELLED AS LINERS STRUCTURAL ELEMENTS		
E	3.60E+06	PSI
Poisson's Ratio	0.2	
Panel Thickness	5.5 "	
Normal Coupling Spring Stiffness	2.10E+07	psf
Shear Coupling Spring Stiffness	2.10E+07	pcf
Normal Coupling Spring Tensile Strength	4.00E+05	psf
Shear Coupling Spring Cohesion	4.00E+05	psf
Shear Coupling Spring Residual Cohesion	2.00E+05	psf
Shear Coupling Spring Friction Angle	23 deg	

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TECHNICAL MEMO NO. (2020-01)

To: Engineering, Bus. Dev., Regional and Project Managers **Date: February 28, 2020**
cc: Melissa Berkebile, Pablo Siwak
Re: MSE Walls on Soft Soils
Phased Construction, Lightweight Fills, Ground Improvement and Two Stage Options
(Pages: 1 of 11)

Prepared by: Keith Brabant
Reviewed by: Kim Truong, Lauren Yip, Daniel Engidashet, Robert Lozano & Pete Anderson

The inherent flexibility of Reinforced Earth® walls (RE Walls) allow for construction of structures on soft soils with significant anticipated settlement. There are documented cases of RE Walls with total settlements in excess of 3 ft (1m), and differential settlements (parallel to wall face) up to 2%. Hundreds of Reinforced Earth walls have been built on sites that have settled in excess of six inches. However, in the past 20 years, whenever foundation settlement is expected, there is a trend to use Two-Stage MSE wall construction (where a first stage wire faced MSE wall is allowed to settle, and then the wall is clad with precast facing panels in the second stage). Use of Two-Stage MSE wall construction has become more frequent due to lack of understanding of the performance of conventional MSE walls on soft soils. This document will provide general guidance on the options for design of MSE walls on soft soils including the use of phased construction, use of lightweight fills, use of ground improvement and for when all other options are not feasible: use of Two-Stage Construction.

Definitions:

	Negligible	Moderate	Significant	Extreme
Total Settlement (Δ)	$\Delta < 3"$ (75mm)	$3" < \Delta < 12"$	$12" < \Delta < 30"$	$\Delta > 30"$
Differential Settlement: 5'x5' panels (Parallel to wall face) δ_p	$\delta_p < 0.5\%$	$0.5\% < \delta_p < 1\%$	$1\% < \delta_p < 2\%$	$\delta_p > 2\%$
Differential Settlement – 5'x10' panels (Parallel to wall face) δ_p	$\delta_p < 0.25\%$	$0.25\% < \delta_p < 0.5\%$	$0.5\% < \delta_p < 1\%$	$\delta_p > 1\%$
Differential Settlement (Perpendicular to wall face) $\delta_\perp$	$\delta_\perp <$ 1.0% (or 4")[1]	1.0% (or 4") $\delta_\perp <$ 2.0% (or 8")[1]	2.0% (or 8") $\delta_\perp <$ 3.0% (or 12")[1]	$\delta_\perp >$ 3.0% (or 12")[1]

Table 1

Notes: 1. Limitations on differential settlement parallel to the wall face are based on ¾" horizontal and ¾" vertical panel joint openings between panels.
2. Limitations on Differential Settlement Perpendicular to the wall face require setting the reinforcing strips higher than the connections by at least one-half the anticipated differential settlement.
3. Full Height Panels have little tolerance for total and differential settlement (1" and 1/500, respectively) and should not be considered for use on sites with soft foundation soils.

When moderate to extreme foundation settlements are expected, the following questions need to be answered by the Geotechnical Engineer of Record:

Settlement Profile

a. What is the estimated total settlement?
b. What is the estimated post construction settlement? (Note post construction settlement limitations due to pavements, slabs, utilities, drainage structures)

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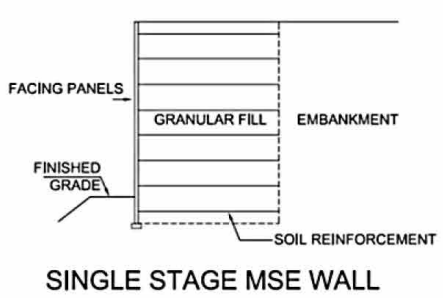
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c. What is the estimated settlement at the face (at end of construction, and post construction)?
d. What is the estimated settlement at the center of reinforced zone (at end of construction, and post construction)?
e. What is the differential settlement along the face? If less than 0.5%, then 5x10 panels OK. If less than 1%, then 5x5 panels OK. If more than 1%, then consider adding slip joints every 30'. If more than 1.5%, then investigate alternatives such as ground improvement or use of lightweight fills to reduce settlement.

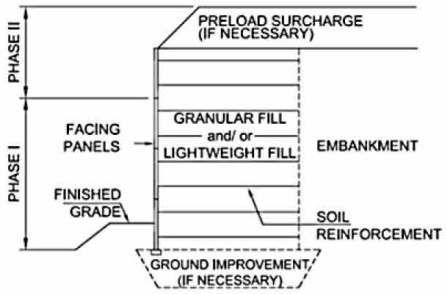
In addition to the settlement estimates, the Geotechnical Engineer of Record is responsible for providing design parameters for foundation soils and embankment (retained) soils; determining the bearing capacity of the foundation soils; the global stability of the MSE wall/foundation/embankment; the minimum embedment of the MSE wall and the minimum lengths of soil reinforcement for global stability.

Based on the information provided by the Geotechnical Engineer of Record, if the expected foundation settlement ranges from moderate to significant, then conventional Single Stage MSE wall construction built in phases should be implemented.

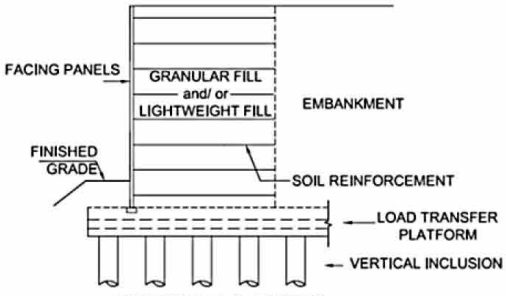
MSE Wall Design/Construction Options for Soft Foundation Soils



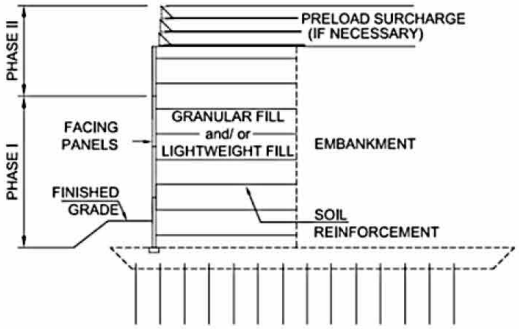
SINGLE STAGE MSE WALL



MSE WALL
BUILT IN PHASES



MSE WALL WITH
GROUND IMPROVEMENT



MSE WALL
BUILT IN PHASES
ATOP GROUND IMPROVEMENT

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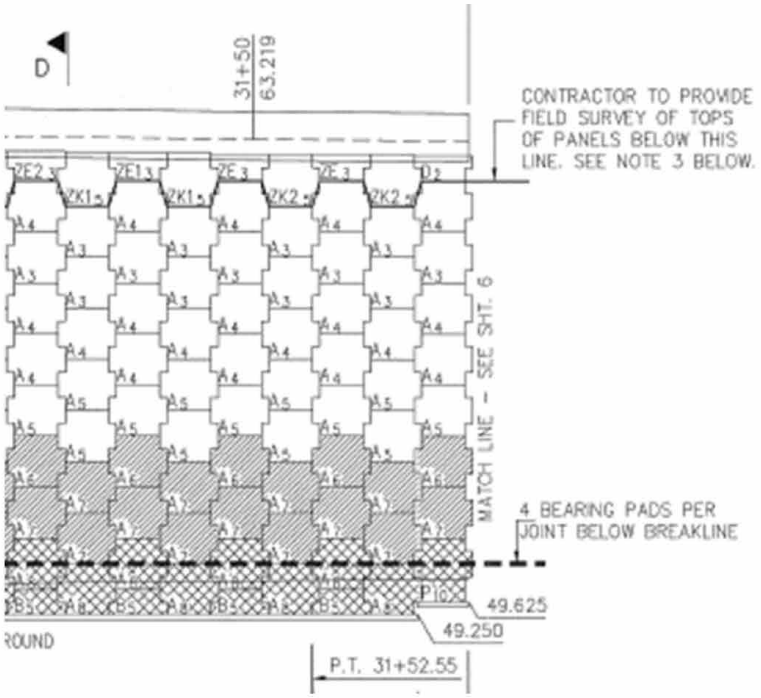
Single Stage MSE wall built in phases

1. If estimated settlement at face of wall is <6", then no special details or construction sequencing is required. Simply account for potential 0" to 6" difference in panel height inside of coping (i.e. use a 6" +2" minimum coping embedment). Note – it is not recommended to set the leveling pad higher than the design elevation to account for settlement, as settlement estimates are often conservative and the maximum settlement may not occur, leading to geometric conflicts at the top of wall.

2. If estimated settlement at face of wall is greater than 6", then leave off top panels and install after field survey confirms settlement is substantially complete.
    a. A temporary surcharge wall may be required to expedite settlement.
    b. When total estimated settlement is greater than 12", or when foundation bearing capacity is inadequate, there may be a need to discontinue construction of the wall for a period to allow the foundation soils to compress (settle) before continuing construction.

3. Depending on the time required to achieve primary settlement, the above solutions may also require ground improvement (wick drains, rigid inclusions) or lightweight fill (lightweight aggregates (foamed glass; expanded clay/shale; pumice), low density cellular concrete).

An example of a wall elevation with phased construction is shown below:

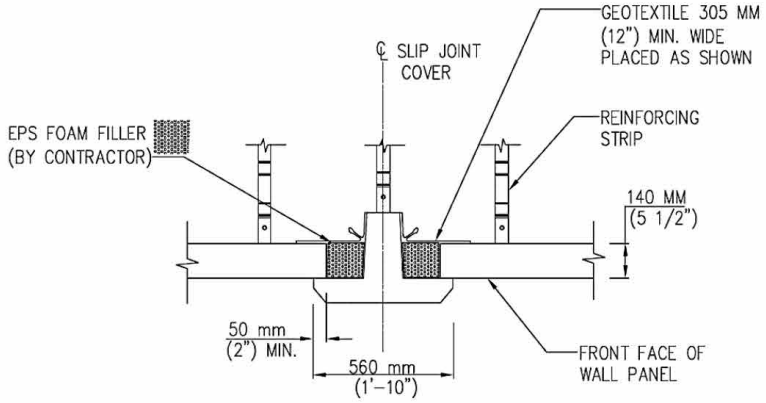


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Accommodating Differential Settlement Parallel to Wall Facing

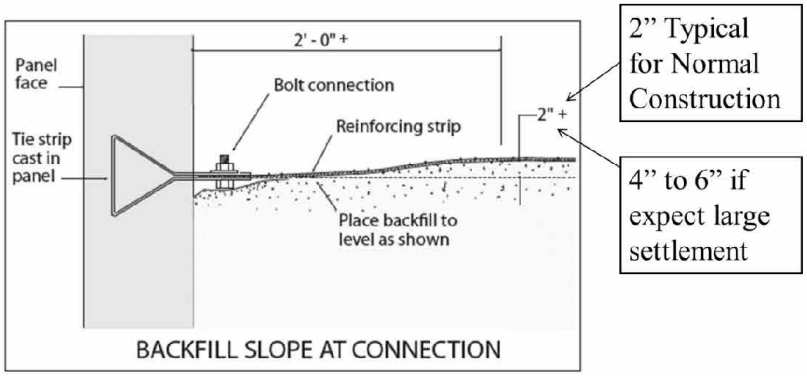
As noted in Table 1, MSE walls with precast facing panels can tolerate moderate differential settlement. When differential settlement parallel to the wall facing is significant or extreme, then the use of slip joints placed in intervals of 30' to 50' can be considered to provide some points of relief. Following is a typical detail of an external slip joint:



NOTE:
PROPER INSTALLATION OF FOAM FILLER AND GEOTEXTILE AT SLIP JOINTS IS IMPERATIVE IN PREVENTING LOSS OF BACKFILL MATERIAL.

PARTIAL PLAN – SLIP JOINT DETAIL

Accommodating Differential Settlement Perpendicular to the Wall Face



BACKFILL SLOPE AT CONNECTION

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Ground Improvement and Lightweight Fills

Many Reinforced Earth walls have been constructed with lightweight aggregates (expanded clay, shale & slate; foamed glass, pumice), and low-density cellular concrete. Design guidance is available for each of these lightweight materials. Use of Lightweight fills may mitigate foundation settlement, and/or reduce the time for settlement.

The following table from the FHWA manual on Ground Modification Methods indicates several options for ground improvements. All of the techniques noted below have been used successfully in conjunction with MSE walls. Specific lightweight fills noted in the below table that are not compatible with MSE walls are as follows (*Geofoam (EPS), wood fiber, blast furnace slag, fly ash, boiler slag and tire shreds*).

Table 1-2. General Applicability of Technologies

Category	Technologies	Applicability
Vertical Drains and Accelerated Consolidation	PVDs, with and without fill preloading	Compressible clays, saturated low strength clays
Lightweight Fills	Compressive Strength Fills: Geofoam; Foamed Concrete	Broad applicability; no geologic or geometric limitations
Lightweight Fills	Granular Fills: Wood Fiber; Blast Furnace Slag; Fly Ash; Boiler Slag; Expanded Shale, Clay, and Slate; Tire Shreds	Broad applicability; no geologic or geometric limitations
Deep Compaction	Deep Dynamic Compaction	Loose pervious and semi-pervious soils with fines contents less than 15%, materials containing large voids, spoils and waste areas
Deep Compaction	Vibro-Compaction	Cohesionless soils, clean sands with less than 15% silts and/or less than 2% clay
Aggregate Columns	Stone Columns	Clays, silts, loose silty sands, and uncompacted fill
Aggregate Columns	Rammed Aggregate Piers	Clays, silts, loose silty sands, uncompacted fill
Column Supported Embankments	Column Supported Embankments	Soft compressible clay, peats, and organic soils where settlement and global stability are concerns
Column Supported Embankments	Reinforced Soil Load Transfer Platform	Soft compressible clay, peats, and organic soils where settlement and global stability are concerns
Column Supported Embankments	Columns: Non-compressible	All soil types, in particular weak soils that cannot support surface loads
Column Supported Embankments	Columns: Compressible	All soil types except very soft soils low undrained shear strength
Soil Mixing	Deep Mixing	Suitable in large range of soils, ones that can be stabilized with cement, lime, slag, or other binders
Soil Mixing	Mass Mixing	Peat, soft clay, dredged soil, soft silt, sludges, contaminated soils
Grouting	Chemical (Permeation) Grouting	Wide range of soil types including weakly cemented rock-fill materials

REF: FHWA-NHI-16-027 Ground Modification Methods

Load Transfer Platforms for MSE Walls atop Ground Improvement

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The design of the Load Transfer Platform (LTP) is a critical element to be coordinated between the ground improvement/LTP designer and the MSE wall designer. Based on full scale structure instrumentation, RECo has developed proprietary LTP design methodology with our sister company Menard for use when Reinforced Earth Walls are placed atop Menard Controlled Modulus Columns (CMC's). When other forms of ground improvement are used, a project specific design of the LTP is required, which should consider the following:

- Top elevation of ground improvement elements should be at least 12" below the bottom of leveling pad. The actual depth will depend on the spacing and stiffness of the ground improvement elements and the thickness of the LTP
- The LTP may not extend above the top of leveling pad
- The LTP should extend beyond the end of MSE reinforcements by at least 2 ft and as far as necessary as determined by the limits of the ground improvement
- The LTP should extend beyond the MSE facing elements by at least (3') three feet.
- The design of the LTP thickness and reinforcement should result in a uniform distribution of loads at the base of MSE (i.e. top of leveling pad) and not rely on the MSE reinforcement for strength or stability.

Two Stage Walls

As noted in the introduction, Two Stage walls have been widely over-used due to a lack of understanding of the performance of conventional MSE walls on soft soils. After 20 years of experience in designing and supplying two stage walls (when specified), we have learned a few lessons. The lessons learned are as follows:

1. Two Stage Walls are overly complex to design and build.

2. Two Stage Walls are less tolerant of secondary settlement (settlement after stage 2 facing is installed) than Single Stage walls.

3. Two Stage walls require longer periods of time for primary settlement to be completed because of the consequences of #2 above.

4. Two Stage Walls are more expensive to construct than conventional MSE walls built atop ground improvement.

Additional lessons learned are discussed in AASHTO LRFD Bridge Design Specifications section C11.10.7.4 where it is stated: *"Past experience with second stage precast incremental facing panels indicates that performance problems can occur if the connections between the panels and the first stage wall can rotate or otherwise have some looseness, especially if wall settlement is not complete. Therefore, incremental second stage facia panels should be avoided for walls located in seismically active areas. Full height second stage precast or cast-in-place concrete panels have performed more consistently, provided the panels are installed after wall settlement is essentially complete."*

If all other settlement mitigation techniques discussed previously in this document are not feasible, then a two stage MSE wall can be constructed with one of the following facings: a cast in place facing; segmental panels with a concrete closure pour; or full height panels. The post construction settlement tolerance of these techniques will depend on the weight of the final facing, the use of expansion joints, the width of leveling pad (footing) and of course, the compressibility of the foundation soils. Cast in place facings for wire walls should be designed by the Engineer of Record. Full Height Precast Facing Panels require special footing designs and special – project specific top of wall connection designs, that are beyond the scope of this document.

Recommendations for use of segmental concrete facing panels with a concrete closure pour

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The facing panels for a Two Stage MSE wall with concrete closure pour are effectively the formwork for the cast in place facing that is poured between the wire faced wall and the precast panels. The details shown here-in for cast in place facing are general in nature and may need to be modified based on project specific requirements. In any case, it is imperative that the Geotechnical Engineer of Record recognize the limitations of this technique to tolerate secondary settlement. RECo recommends placement of expansion joints every 20 ft +/- (at panel joint locations) to accommodate thermal expansion and contraction of the cast in place closure pour and to offer some flexibility of the facing to tolerate secondary settlement (1" maximum).

Design of the underdrain system, as well as the drainage composite to be placed between the wire facing and cast in place closure pour, should be shown in the contract documents.

NOTES:

1. PRECAST FACING PANELS AND C.I.P. CLOSURE POUR SHALL NOT BE PLACED UNTIL THE GEOTECHNICAL ENGINEER OF RECORD HAS REVIEWED SETTLEMENT MONITORING INFORMATION AND DETERMINED THAT ALL PRIMARY SETTLEMENT HAS OCCURRED. THE ALLOWABLE SECONDARY (LONG-TERM) SETTLEMENT OF THE TWO-STAGE WALL AFTER PLACEMENT OF FACING PANELS AND CONCRETE CLOSURE POUR IS 1 INCH.

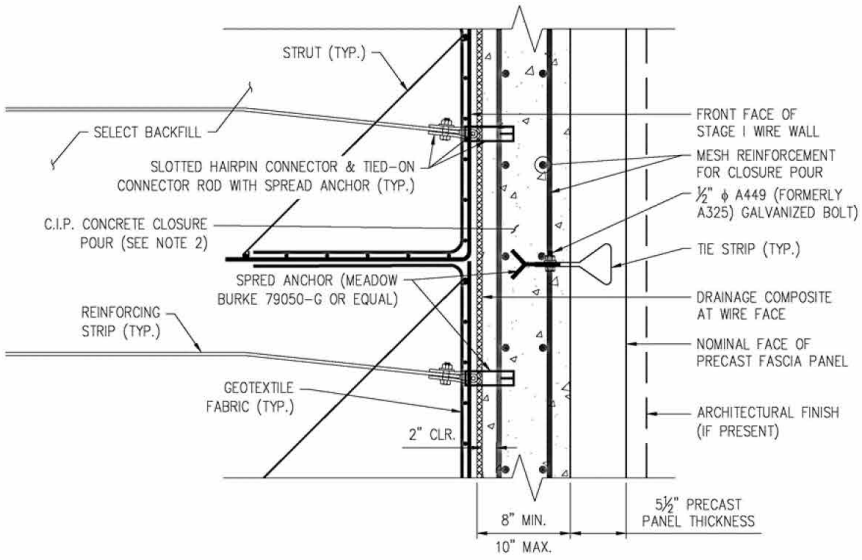
2. CLOSURE POUR SHALL BE LIMITED TO 30" LIFTS. PANELS SHALL NOT BE STACKED UNTIL PRIOR CLOSURE POUR IS COMPLETED AND CURED FOR A MINIMUM OF 24 HOURS, OR 12 HOURS IF HIGH EARLY STRENGTH CONCRETE IS UTILIZED. IN PLACE CONCRETE SHALL HAVE A MINIMUM 3000 PSI STRENGTH PRIOR TO PLACING NEXT LIFT OF CONCRETE.

3. CONTRACTOR SHALL PROVIDE ADEQUATE EXTERNAL BRACING TO RESIST FLUID HEAD OF CONCRETE. CONCRETE PLACEMENT/CLOSURE POUR METHOD AND PANEL BRACING DURING POURING IS THE RESPONSIBILITY OF THE CONTRACTOR.

4. EXPANSION JOINTS SHALL BE INSTALLED AT VERTICAL PANEL JOINTS, SPACED 20' MAX., DURING CONSTRUCTION OF THE SECOND STAGE CLOSURE POUR.

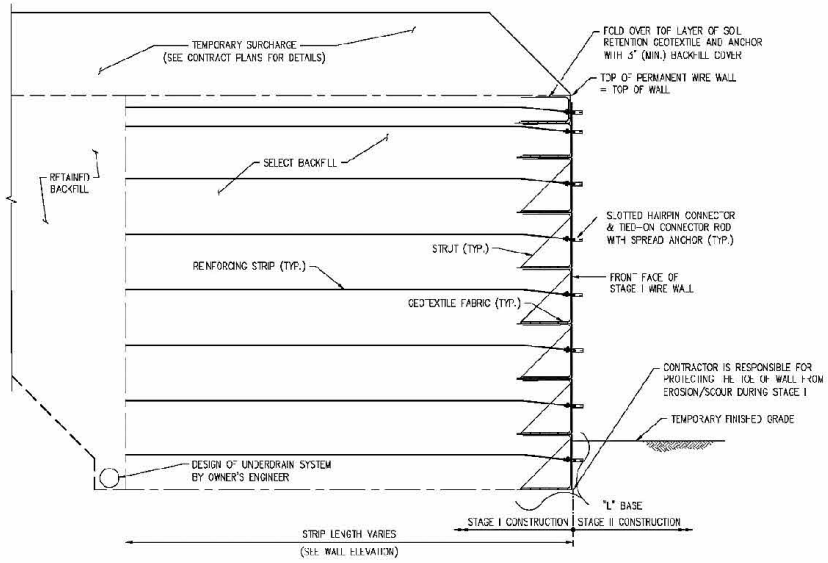
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VERTICAL SECTION – 2-STAGE CLOSURE POUR

SCALE: 1½" = 1'-0"

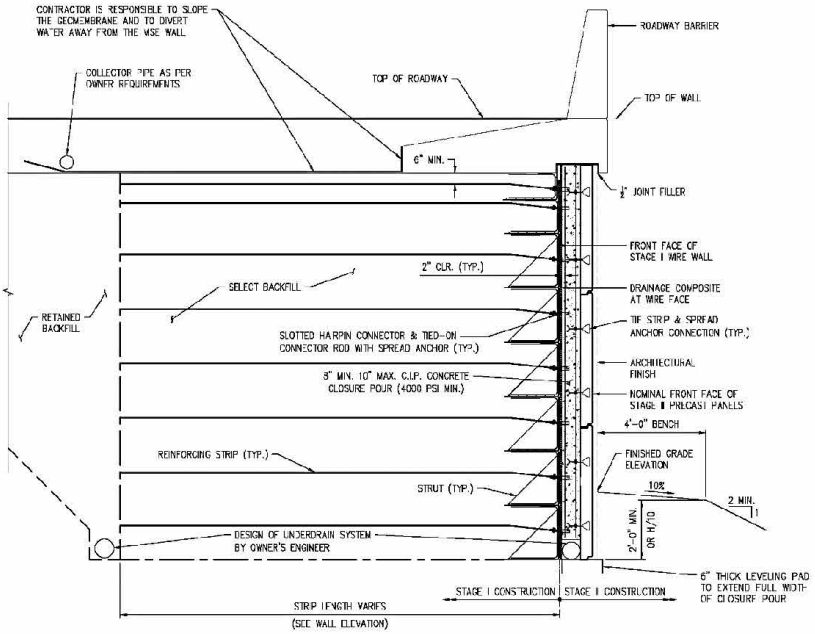


STAGE I AT ROADWAY – TYPICAL SECTION

SCALE: ½" = 1'-0"

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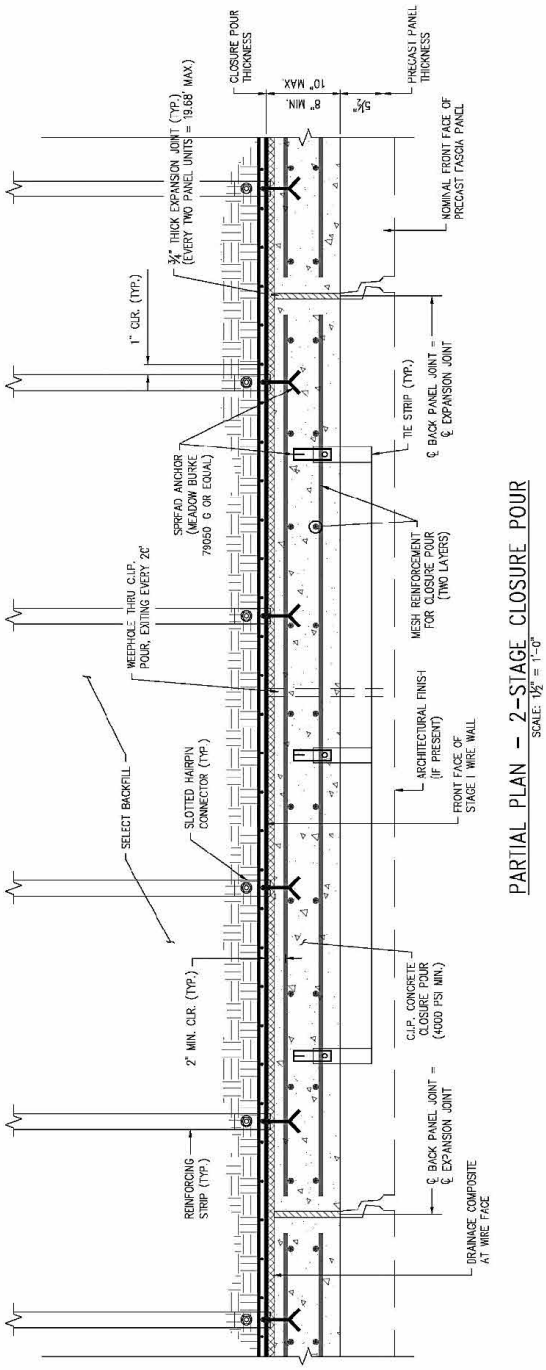
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STAGE II AT ROADWAY TYPICAL SECTION
SEGMENTAL PANELS
SCALE: ⅜" = 1'-0"

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PARTIAL PLAN – 2-STAGE CLOSURE POUR

SCALE: 1½" = 1'-0"

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Conclusion

This document provides state of the practice guidance for the use of MSE walls constructed atop compressible foundation soils. It is the opinion of The Reinforced Earth Company that single stage MSE walls (combined with phasing, lightweight fills and/or ground improvement) are technically superior to two stage MSE walls. For applications where two stage MSE walls are the only feasible option, the second stage facing should be cast in place or consist of precast panels with a cast-in-place closure pour. Full height facing panels may also be considered.

The treatment of pavement, slabs, and utilities are outside the scope of this document. Settlement tolerances for pavement, slabs, and utilities should be considered by the Owner's project team.

References:

Sankey J.E, Brabant K, Masse F. Stand Alone and Combined Technologies for MSE Walls: State of Practice for Compressible Soils. Geo-Frontiers 2011© ASCE 2011

AASHTO LRFD Bridge Design Specifications, Section 11

FHWA – NHI 10-024: Design and Construction of Mechanically Stabilized Earth Walls, November 2009

FHWA – NHI-16-027 Ground Modification Methods, April 2017

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September 21-23 2017, Vancouver, Canada

Unique Design Considerations for Mechanically Stabilized Earth Walls in Transportation Widening Applications

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Abstract

As many transportation corridors have exceeded design capacity, additional lanes are being constructed adjacent to existing traffic. The construction of new lanes often requires support of excavation or relies on existing retaining structures for temporary and permanent load relief on new MSE walls. Although various types of hybrid MSE structures using combinations of existing walls or multiple wall types have been successfully constructed for decades, minimal formal design guidance existed until 2006 when the US Federal Highway Administration issued a manual for Shored MSE walls (SMSE).

This paper will present case studies and best practice suggestions for the use of MSE walls in highway widening applications, including where new MSE walls are placed in close proximity to existing MSE walls or other retaining structures.

Keywords: MSE, Shored Mechanically Stabilized Earth, soil nail, reinforced soil

1 Introduction

Mechanically Stabilized Earth (MSE) walls have been in use in North American highway and rail applications for over 45 years. Due to the economy, flexibility and load bearing resistance of properly designed and constructed MSE walls, these structures have become the preferred method for earth retention in fill situations. With transportation facilities expanding due to need for increased capacity, right of way is often at a premium and innovative solutions are needed to provide safe, economical MSE walls in areas where conventional MSE solutions require too much space to fit the typical length of soil reinforcements.

MSE Walls consist of three major elements (Soil Reinforcement, Backfill and Facing Elements). The length of soil reinforcement required is based on providing a coherent gravity structure, capable of resisting externally applied forces (from retained fill, earth quakes, etc) with its own mass. Fundamental to the concept of a coherent gravity mass, the structure must be internally stable to behave as a semi-rigid block. The internal stability of a MSE structure relies on the interaction of reinforcement (spaced at regular intervals) and frictional backfill.

The length of the soil reinforcement is typically on the order of 70% of the height of the wall [1] (See Figure 1). This "aspect ratio" is frequently referred to as L/H. While the typical length of reinforcement suits construction of MSE retaining structures in fill situations, it often precludes the use of MSE walls in cuts or in areas with limited fill space (i.e. where there is less than 70% of the wall height available for placement of soil reinforcements).

While the 70% ratio has become gospel to many MSE wall designers, there is in fact no theoretical reason to limit the length to this ratio. It should be noted that the 70% ratio is based on typical conditions, and if field conditions permit and analysis shows that the MSE wall is internally, externally and globally stable, reinforcement lengths shorter than 70% of the wall height may be used, provided that deformations of the wall face are controlled.

In the field of reinforced soil structures, the pioneering work of Terre Armee, starting in the 1970's, led to additional research throughout the

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world on use of various reinforcement types, special applications and various soil conditions. Full scale test walls constructed by Terre Armee in the 1980's (Bastick et al. [2]) demonstrated that MSE walls constructed with ribbed steel strip reinforcement and precast panels could be constructed to aspect ratios as low as L/H = 0.5 and retain embankment fills, while confirming that the coherent gravity design method accurately predicts reinforcement tensions. As conventional MSE walls were demonstrated to be feasible with aspect ratios as narrow 0.5, the concept of MSE walls with aspect ratios less than 0.5 became a consideration in situations where a stable structure (be it a wall, or in-situ rock face) behind the MSE volume relieved the externally applied earth pressure.

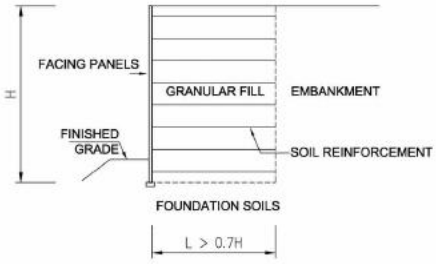


Figure 1. Conventional MSE Wall

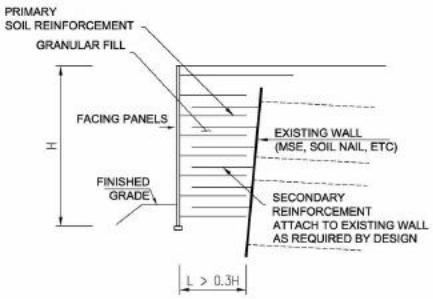


Figure 2. Shored MSE Wall

In locations where support of excavation (S.O.E.) is placed (such as adjacent to an existing roadway embankment), where there is an existing retaining structure (MSE, Cast-in-Place, Cantilever, etc) or a stable rock feature behind a new MSE wall, the applied external earth pressure on the new MSE wall is eliminated. As such, there is no longer a need to provide a large mass to resist the applied earth pressure. Rather, if the S.O.E. wall or existing retaining structure can be expected to be stable for the design life of the new MSE structure, the new MSE structure may be designed for soil reinforcement lengths less than 70% of the wall height, and as low as 30% of the wall height, depending on the conditions of the project. This type of structure is now commonly referred to as a Shored Mechanically Stabilized Earth Wall (SMSE) – see Figure 2.

This paper will discuss the conditions affecting the design length of soil reinforcements for SMSE walls, provide case studies, and summarize the findings of numerical modeling by The Reinforced Earth Company (RECo) as well as lessons learned from other research provided in the literature.

2 Design Considerations

SMSE walls, like conventional MSE walls, need to be internally, externally and globally stable. Bearing Capacity, Settlement and Global Stability of the proposed system should be investigated at the feasibility stage. If the foundation soils do not have adequate bearing capacity, then soil improvements or lightweight backfill may be required. If significant settlement is expected, then designs requiring a direct connection of the new MSE wall to the inner wall may not be feasible.

2.1 External Stability

In accordance with FHWA recommendations, it is not necessary to check overturning and sliding [3] due to the presence of a permanent shoring wall behind the MSE mass. However, even for narrow base widths (L/H < 0.7), it is prudent to check sliding with a consideration of the effect of the MSE fill "pushing off" the shored wall component.

Although the FHWA SMSE manual does not provide guidance on sliding and overturning effects, research by Yang, et al. [4] provides a detailed study of the forces acting on narrow MSE walls. This study correlates both centrifuge testing and finite element studies, with respect to the aspect ratio (L/H) and shows that as the aspect ratio is reduced, the applied earth pressure on the MSE mass is reduced. This phenomena is logical, and

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agrees in concept with arching behaviour described by Spangler and Handy [5]. However, for SMSE walls without secondary reinforcements attached to the inner wall, the reduction in earth pressure is minor (less than 10%) for aspect ratios (L/H) greater than 0.3.

For SMSE walls constructed of Soil Nail, Cantilever or Tie Back walls that are to be built just prior to the construction of the outer MSE wall, the designers of each structure must coordinate design efforts with regard to offsets, construction benches, etc. The (buried) inner wall should be designed as a permanent wall, and incorporate corrosion protection on nails, tie-backs, etc. Drainage composites should be placed at the face of the inner wall and provisions for drainage outlets under or through the outer MSE wall are required.

For SMSE walls constructed in front of existing MSE or other structures, the owner's engineer should provide an investigation of the existing structure to determine the following:

- Anticipated remaining service life of MSE, Soil Nail, etc reinforcing elements.
- Structural conditions of the facing (as the SMSE wall may need to have anchors attached to the existing wall facing).
- Provisions for drainage structure modifications or relocations.
- Provisions for passing upper row(s) of primary soil reinforcement over the top of existing wall (if required by design)
- Effects of the proposed structure on Bearing capacity, settlement and global stability.
- Need for shoring at the base of the existing structure if the bottom of the proposed structure is down-slope of the existing structure.

Figure 3 shows the potential failure modes, based on slip-circle analysis methods that shall be analysed during the design phase.

2.2 Internal Stability

The internal stability of an MSE wall is a function of reinforcement type / strength / length / spacing, loading conditions, wall geometry and select backfill weight and strength parameters.

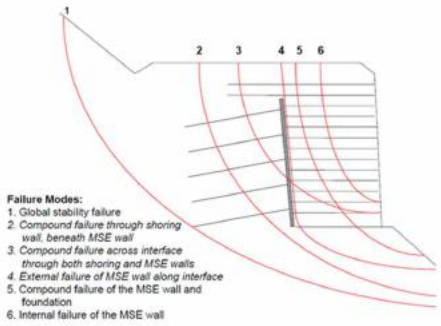


Figure 3 - SMSE Potential Failure Modes [3]

2.2.1 MSE Select Backfill Recommendations

For aspect ratios (L/H) of 0.5 to 0.7, the select backfill should meet the requirements of AASHTO LRFD Bridge Construction Specifications Section 7.3.6.3, with an additional restriction of 10 % maximum passing the #200 sieve, per FHWA SMSE Manual section 3.3 [3].

Due to space limitations, potential deformations, and to provide ease in placement and compaction, it is recommended to use open graded course angular aggregate backfill (such as #57 stone) in narrow width applications (i.e. aspect ratio L/H less than 0.5). An added benefit of increased internal drainage is provided when open graded aggregates are used for MSE fill.

2.2.2 Effects of Reinforcement Type

It is important to note the differences in internal failure planes for MSE structures reinforced with extensible (geosynthetic) or inextensible (steel) soil reinforcements as shown in Figures 4 and 5. The simple explanation for this is:

Extensible (Geosynthetic) reinforcements do not modify the behavior of the soil, and therefore a linear Rankine failure surface develops and near peak shear strength of the soil is realized within the structure. Only the length of extensible reinforcement needed to resist pullout is mobilized beyond the failure plane resulting in an MSE mass that behaves like a tied-back structure. For inextensible (steel) reinforcements, soil deformation is minimal, a bilinear line of maximum

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tension develops, and a mobilized soil shear strength of about 34 degrees is realized within the structure. Nearly the full length of the inextensible reinforcements are under tension resisting pullout, creating a composite material that behaves like a coherent gravity structure.

In depth discussion on the differences in design of inextensible and extensible reinforcements may be found in Anderson, et al [6].

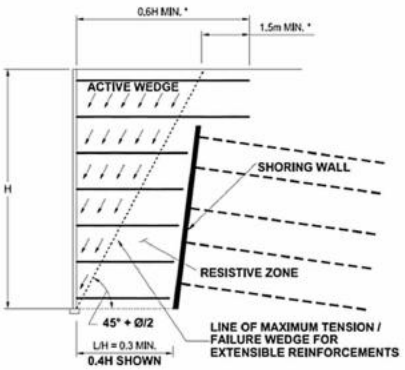


Figure 4 – SMSE with Extensible Reinf.

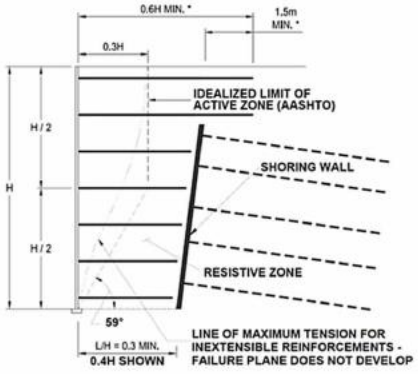


Figure 5 – SMSE with Inextensible Reinf.

** FHWA SMSE Manual recommends that upper rows of reinforcement be extended over top of shoring wall.*

2.2.3 Geometric Constraints

2.2.3.1 Aspect Ratio L/H 0.5 to 0.7

For aspect ratios (L/H) of 0.5 to 0.7, a conventional MSE wall may be constructed. The typical minimum width between the inner and outer wall is 2m, and may be wider when the inner wall is being constructed as a soil nail or tie back wall and the working bench for equipment requires more than 2m.

The design process is the same as for conventional MSE walls. For compound and global stability analysis, the two walls should be modelled in one software and potential failure modes checked. Internal stability of the MSE portion may be analysed using the AASHTO Simplified Method for inextensible or extensible soil reinforcements.

2.2.3.2 Aspect Ratio L/H 0.3 to 0.5

Centrifuge Modelling by Yang, et al.[4], Kniss, et al [7] and Lee [8] show that aspect ratios (L/H) as narrow as 0.3 may be achieved with an independent MSE wall in front of a stable inner wall. Due to minimal effective length of extensible reinforcement at the top of narrow structures, the FHWA SMSE manual recommends extending the upper reinforcement layers (typically within the top 1m to 2m +/-) an additional 1.5m beyond the face of the interior shoring wall (or 0.6H total length) [3]. In addition to providing additional pullout resistance for the upper level(s) of reinforcement, the extended reinforcements also provide resistance to the formation of tension cracks in the soil/pavement above the shoring wall location.

In order to minimize deformations of the outer wall, provide reinforcements against compound stability, and to restrain the mass from pushing away from the inner wall, The Reinforced Earth Company recommends the use of secondary reinforcements – attached to the inner wall – for aspect ratios less than 0.5. These secondary reinforcements provide a "sandwich" effect and have been successfully used on multiple SMSE projects by The Reinforced Earth Company / Terre Armee throughout the world.

Research by Tavakolian, et al [9], [10] and Freitag et. al. [11] of RECo / Terre Armee shows that the

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sandwich connection technique not only provides the above noted benefits, but also reduces the horizontal stresses within the MSE volume, and thus results in lower tensile forces in the soil reinforcing elements.

The primary and secondary reinforcements modelled in FLAC studies and used in construction of Reinforced Earth SMSE Walls have been galvanized steel High Adherence (HA) ribbed strips, galvanized steel HA ladders and HA geosynthetic strips (GeoStraps®) [12].

Where space cannot be provided for extending the upper rows of reinforcements above an existing wall, it has been demonstrated that using secondary "sandwich" reinforcements at the top of the structure also serve to provide the necessary pullout resistance and preclude tension cracking at the ground surface above the shoring wall interface. [12] Only in specific circumstances should a direct connection between the outer and inner wall be contemplated, as discussed below.

For practical consideration, deformations are a possibility for aspect ratios less than 0.7, for either reinforcement type, if compaction equipment is not properly sized and backfill lifts adjusted for the project constraints. Note that in order to accommodate deformation concerns and compaction equipment size limitations, some narrow width structures have been successfully constructed with cement stabilized backfill [4] and lightweight cellular foam concrete fill.

2.2.3.3 Direct Connection to Inner Wall(s)

When the distance between the inner and outer wall provides less than 0.3H or 2m, it may be necessary to provide a direct connection between the outer MSE panels and the inner wall. When a direct connection is to be used, the effects of settlement of the foundation soils become a paramount concern as settlement will induce strain on the connections that may exceed the connection strength.

An example of a direct connection between an existing MSE wall and a new MSE wall is shown below (figure 6). Note that the connections have been located and rotated 90° to accommodate minor foundation settlement.

In the case of a tied (direct connection) structure, the reinforcements should be designed for at rest earth pressure. Special consideration should be made for accommodating the transfer of impact loading from traffic barriers and guardrails.

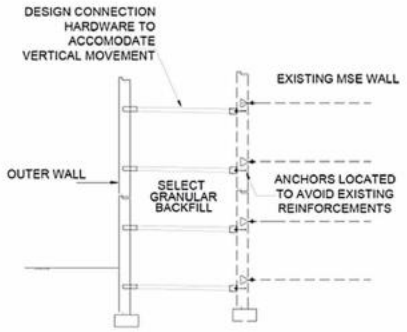


Figure 6 – Tied Bin Wall

2.2.3.4 Effects on existing wall / soil nails, etc.

The effects of attaching primary, or secondary reinforcements to the face of existing MSE wall, soil nail wall facing or directly to the heads of soil nails needs to be analysed. Numerical modelling by The Reinforced Earth Company indicates that there is a potential for increased loads on the inner wall soil reinforcements when secondary reinforcements are attached to the facing. The magnitude of loading will depend on geometry, backfill parameters, foundation settlement and loading conditions.

3 Case Studies

3.1 I-895 – Richmond, VA

In 2010, a portion of I-895 in Richmond, VA was widened to provide additional capacity for a major route to Richmond Airport. At one existing bridge to be widened by 3.6m, there were existing MSE walls retaining the approach fills. The new wall approached 9m in height. As such, the minimum aspect ratio of the new structure (L/H) 3.6m/9m = 0.4. The design team considered independent MSE

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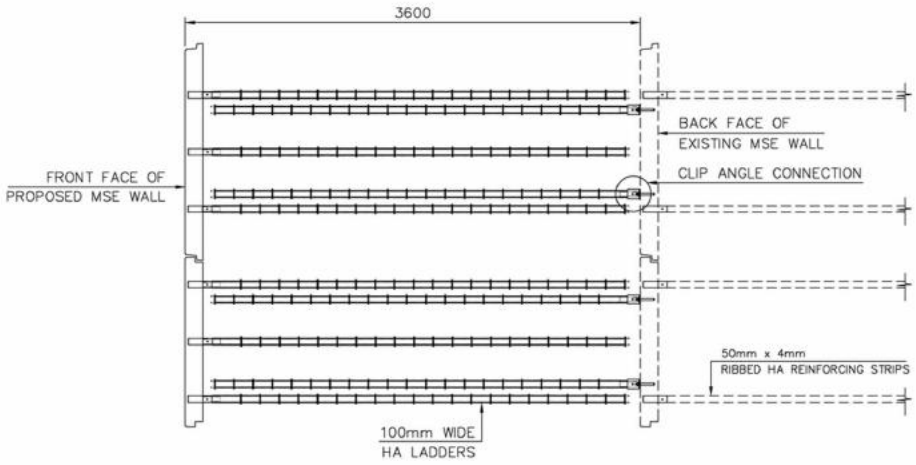


Figure 7 - Sandwich Reinforcements

reinforcements, direct connection to the existing MSE wall face and "sandwich" connections. Where the L/H ratio was less than 0.5, The Reinforced Earth Company proposed the use of sandwich connections, which provide independent reinforcements attached to the outer face of the existing wall and inner face of the new wall.

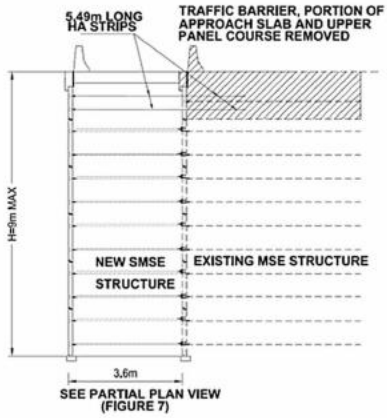


Figure 8 MSE Wall Widening – I-895, VA

A double angle connection consisting of galvanized steel clip angles were attached to the face of the existing wall with adhesive anchors. Galvanized steel high adherence (HA) ladder reinforcements were attached to the clip angle assemblies and laid side by side with the galvanized HA ladders which were attached to the new MSE wall.

The project provided confirmation of the sandwich reinforcement technique, which has been further refined with various soil reinforcement types and further studied with FLAC. As a result of FLAC models, one modification to the technique used on I-895 is to vertically stagger the primary and secondary reinforcing elements to reduce internal loads on the soil reinforcements and provide more resistance to lateral deformations.

3.2 Banner Elk, NC

In 2013, SMSE walls were constructed in mountainous terrain for widening of state route NC 194. Several retaining walls were required, with a maximum height of 14m. Due to the narrow existing roadway, and steep slopes at the base of wall, Shored MSE was the optimal solution.

The shoring component was achieved with permanent soil nail walls, and the MSE component was achieved with precast facing panels, galvanized ribbed steel strip reinforcements and course aggregate (AASHTO #57) backfill.

The coordination between the MSE designer and the soil nail designer resulted in minimum clear distance between the soil nail wall and MSE wall of 0.5H, allowing for the MSE wall to be built independent of the soil nail wall with no need for special connection hardware. The 0.5H offset also

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allowed for the soil nail contractor to have an adequate bench to perform the soil nail wall construction.

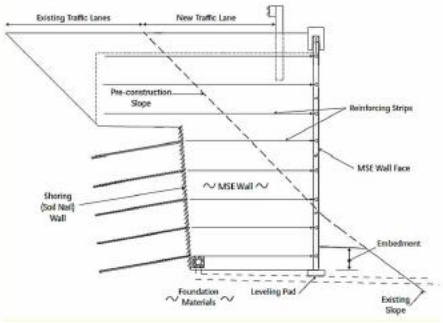


Figure 9 – SMSE Wall – Banner Elk, NC

The success of this project has led to other projects in the area being designed and constructed with SMSE techniques.

3.3 I-215 / I-15 – Murray, Utah

In 2011, an on-ramp from I-215 to Southbound I-15 in Murray, Utah was widened. The existing MSE wall consisted of precast facing panels and galvanized steel bar mat reinforcements. The maximum height of the new MSE wall was approximately 9.1 m, and the available space between the existing MSE wall and the new MSE wall was 3.5m, with a resulting minimum L/H ratio of 0.38. The Reinforced Earth Company designed and supplied a SMSE wall with sandwich reinforcements consisting of HA Ladder reinforcements. Galvanized Steel Clip Angles were installed on the face of the existing wall.

Due to the high seismic design load (A=0.4g), the design was checked using FLAC numerical analysis. The sandwich connection with (HA) ladders provided more than adequate pullout resistance, and maintained the desired seismic flexibility of the new MSE wall.



Figure 10 – I-215/I-15 SMSE Wall under Construction

Conclusions

MSE Walls can be safely constructed with aspect ratios (L/H) less than the conventional 0.7.

SMSE Walls with aspect ratios (L/H) between 0.5 and 0.7 may be designed as independent structures without special connections to inner walls.

SMSE Walls with aspect ratios between 0.3 and 0.5 should be designed with considerations for deformations and secondary reinforcements are recommended to minimize deformation, provide additional resistance to compound failure planes and added seismic resistance.

SMSE walls with base widths less than 0.3H or 2m require special considerations for direct connections, and may require alternative fill materials such as cement stabilized backfill or lightweight cellular concrete.

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A2.1.1 Innovations

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A2.1.2 Design Methodology Description

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A2.1.3 Obstructions in the Reinforced Soil Zone

Vertical Obstructions:

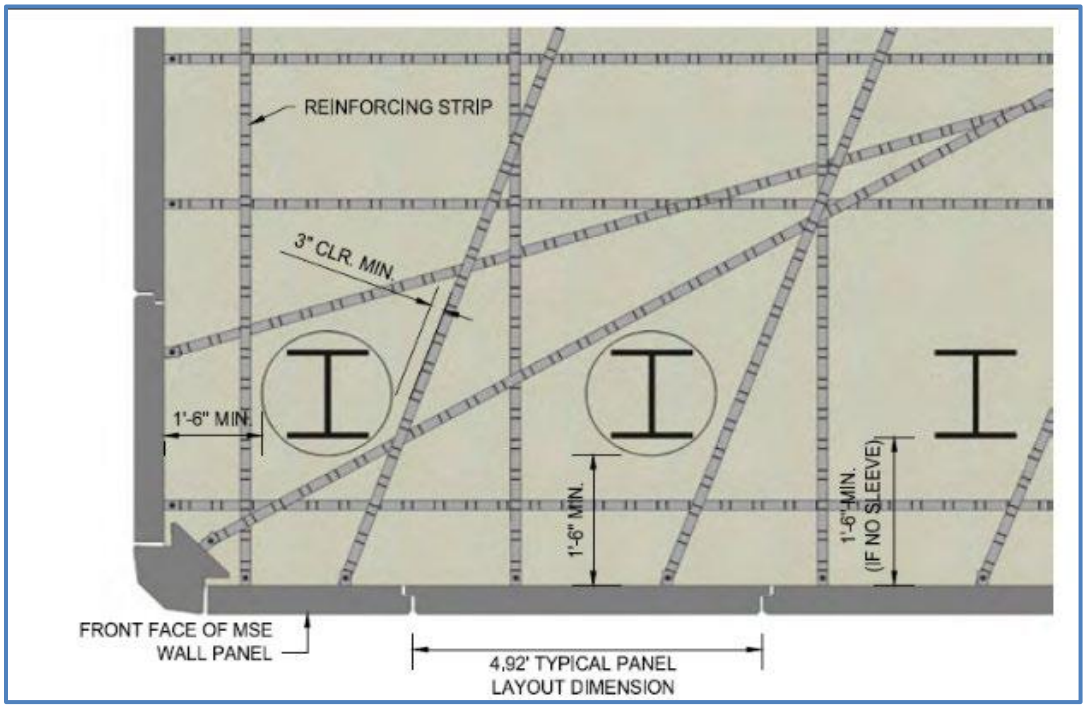


Figure A2.1.3.-3: Reinforcing Strip Splay at Abutment Piles/Casings

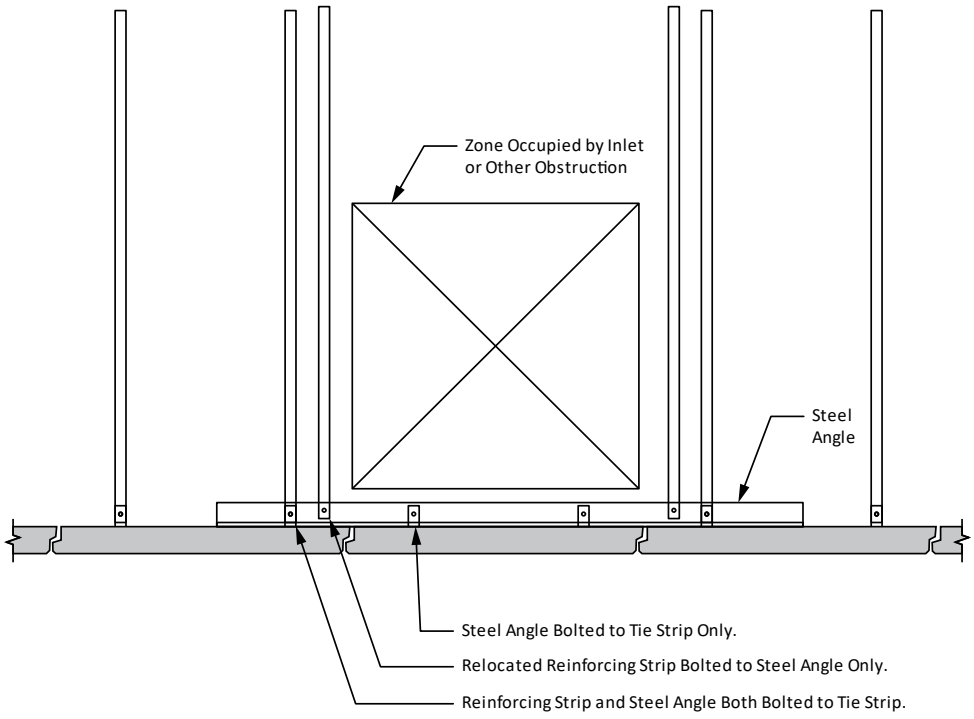


Figure A2.1.3-4: Partial Plan - Inlet Obstruction

Horizontal Obstructions:

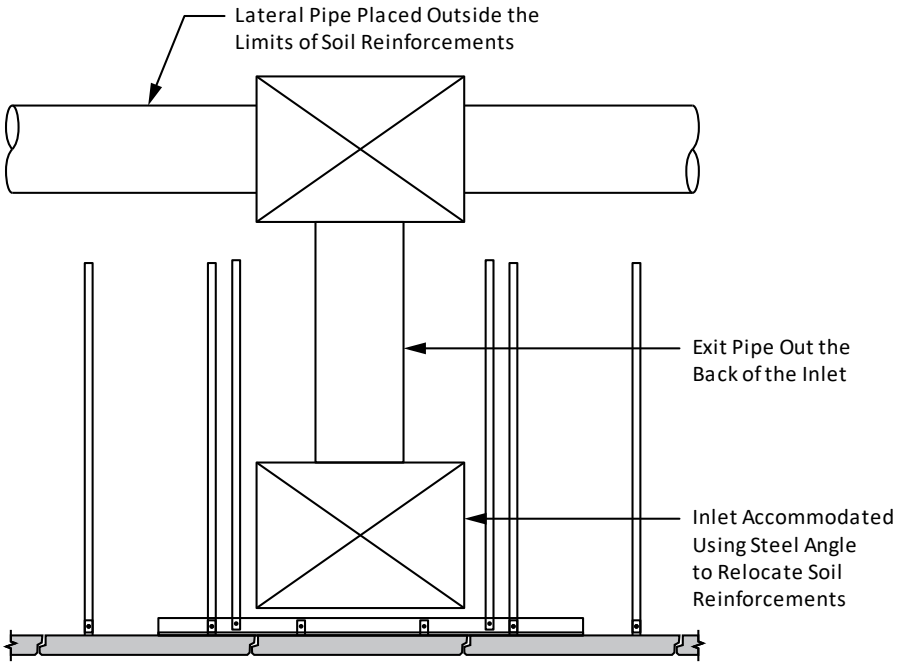


Figure A2.1.3-1: Partial Plan - Inlet and Pipe Obstruction

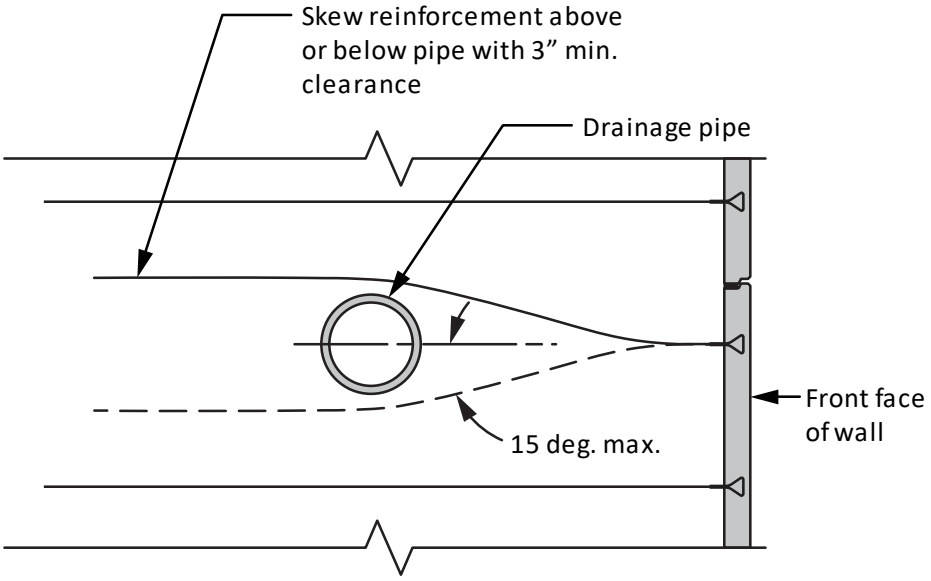


Figure A2.1.3-2: Pipe Obstruction

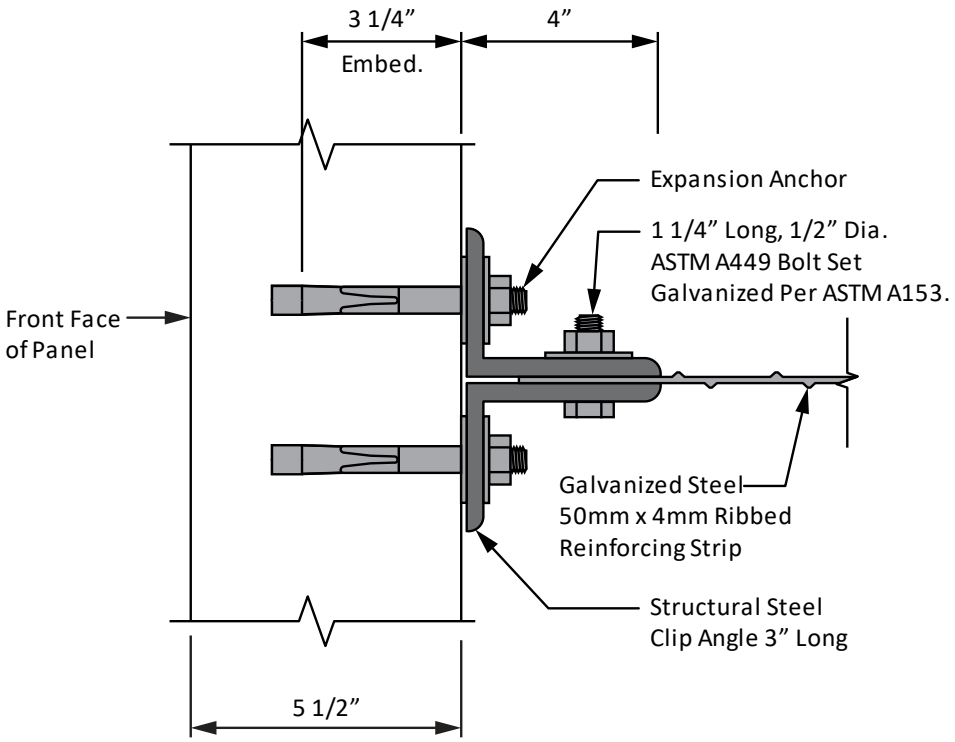


Figure A2.1.3-3: Clip Angle Connection

A2.2 Design Example

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A2.2.1 Problems 1 & 2

The following documentation is included in this section:

- Calculations for Problem 1 (14 pages)
- Calculations for Problem 2 (14 pages)

AASHTO 2017-2020
IDEA
MSEW+: Update # 2021.20

PROJECT IDENTIFICATION

Title: IDEA
Project Number: LEVEL SURCHARGE
Client:
Designer: DE
Station Number:

Description:

Company's information:

Name:
Street:

Telephone #:
Fax #:
E-Mail:

File path and name: C:\Users\daniel.engidashet\OneDrive - VINCI Constructio.....
.....DEA\MSEW1-Level.BENp

Original date and time of creating this file: Thu May 12 14:19:19 2022

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

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SOIL DATA

REINFORCED SOIL

Unit weight,	γ	135.0 lb/ft^3
Design value of internal angle of friction,	ϕ	34.0 °

RETAINED SOIL

Unit weight,	γ	120.0 lb/ft^3
Design value of internal angle of friction,	ϕ	30.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight,	$\gamma_{equiv.}$	125.0 lb/ft^3
Equivalent internal angle of friction,	$\phi_{equiv.}$	30.0 °
Equivalent cohesion,	$c_{equiv.}$	0.0 lb/ft^2

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Ka (external stability) = 0.2973 (eq. 17 is utilized to calculate Ka for all batters)
(For external stability user specified $\delta = 20.00°$)

BEARING CAPACITY

Bearing capacity is controlled by general shear.
Bearing capacity factors (calculated by MSEW): Nc = 30.14 N γ= 22.40

SEISMICITY

Not Applicable

FOR EXTERNAL STABILITY

Ka = 0.2973 In Coulomb equation for Ka, Omega was taken as ZERO and backslope inclined at angle I.

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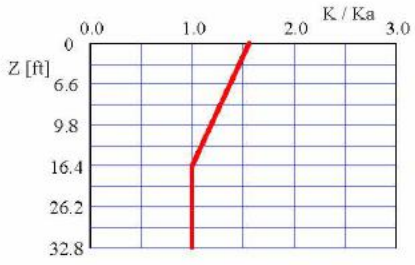
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INPUT DATA: Metal strips (Analysis)

D A T A	Metal strip type #1	Metal strip type #2	Metal strip type #3	Metal strip type #4	Metal strip type #5
Yield strength of steel, Fy [kips/in ²]	65.0	N/A	N/A	N/A	N/A
Gross width of strip, b [in]	2.0	N/A	N/A	N/A	N/A
Vertical spacing, Sv [ft]	Varies	N/A	N/A	N/A	N/A
Design cross section area, Ac [in ²]	0.20	N/A	N/A	N/A	N/A
Ribbed steel strips.					
Uniformity Coefficient of reinforced soil, Cu = D60/D10 = 4.0					
Friction angle along reinforcement-soil interface, ρ					
@ the top	70.97	N/A	N/A	N/A	N/A
@ 19.7 ft or below	34.00	N/A	N/A	N/A	N/A
Pullout resistance factor, F*					
@ the top	2.90	N/A	N/A	N/A	N/A
@ 19.7 ft or below	0.67	N/A	N/A	N/A	N/A
Scale-effect correction factor, α	1.00	N/A	N/A	N/A	N/A

Variation of Lateral Earth Pressure Coefficient With Depth (Coherent Mass)

Z	K / Ka
0 ft	1.56
3.3 ft	1.47
6.6 ft	1.37
9.8 ft	1.28
13.1 ft	1.19
16.4 ft	1.09
19.7 ft	1.00



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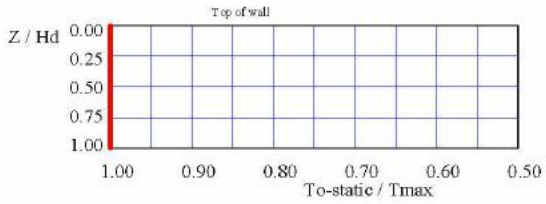
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INPUT DATA: Facia and Connection (Analysis)

FACIA type: Segmental precast concrete panels.
Depth of panel is 0.50 ft. Horizontal distance to Center of Gravity of panel is 0.25 ft.
Average unit weight of panel is γ_f = 150.00 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	HA50X4	N/A	N/A	N/A	N/A
Strength reduction at the connection, CRu = Fyc / Fy	1.00	N/A	N/A	N/A	N/A

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

Design height, Hd 30.00 [ft] { Embedded depth is E = 3.00 ft, and height above top of finished bottom grade is H = 27.00 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)

UNIFORM SURCHARGE

Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 250.0 [lb/ft ²]

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 2 4 6 8 10[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
Metal Strips:		0.75	1.00

Resistance factor for reinforcement tension in connectors	ϕ	Static	Combined static/seismic
Metal Strips:		0.75	1.00

Resistance factor for Metal Strips 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}):	$(\gamma_{p\text{-}EH})_{EQ}$	1.50
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	0.90

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ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, CDR = 2.28, factored bearing load = 7302 lb/ft².
Foundation Interface: Direct sliding, CDR = 2.077, Eccentricity, e/L = 0.0910, CDR-overturning = 3.89

	METAL STRIP			CONNECTION						
#	Elevation [ft]	Length [ft]	Type #	CDR [connection break]	CDR Strength	Metal strip strength CDR	Pullout resistance CDR	Direct sliding CDR	Eccentricity e/L	Product name
1	1.23	21.00	1	1.11	1.11	1.106	1.944	2.507	0.0828	HA50X4
2	3.69	21.00	1	1.23	1.23	1.232	1.848	2.685	0.0674	HA50X4
3	6.15	21.00	1	1.10	1.10	1.103	1.395	2.894	0.0533	HA50X4
4	8.61	21.00	1	1.24	1.24	1.241	1.301	3.143	0.0406	HA50X4
5	11.07	21.00	1	1.37	1.37	1.372	1.318	3.442	0.0292	HA50X4
6	13.53	21.00	1	1.11	1.11	1.112	1.162	3.811	0.0193	HA50X4
7	15.99	21.00	1	1.22	1.22	1.216	1.306	4.275	0.0109	HA50X4
8	18.45	21.00	1	1.36	1.36	1.362	1.507	4.880	0.0039	HA50X4
9	20.91	21.00	1	1.58	1.58	1.576	1.688	5.697	-0.0015	HA50X4
10	23.37	21.00	1	1.91	1.91	1.912	1.851	6.864	-0.0054	HA50X4
11	25.83	21.00	1	2.64	2.64	2.637	2.087	8.666	-0.0076	HA50X4
12	28.00	21.00	1	3.18	3.18	3.183	2.206	11.330	-0.0083	HA50X4

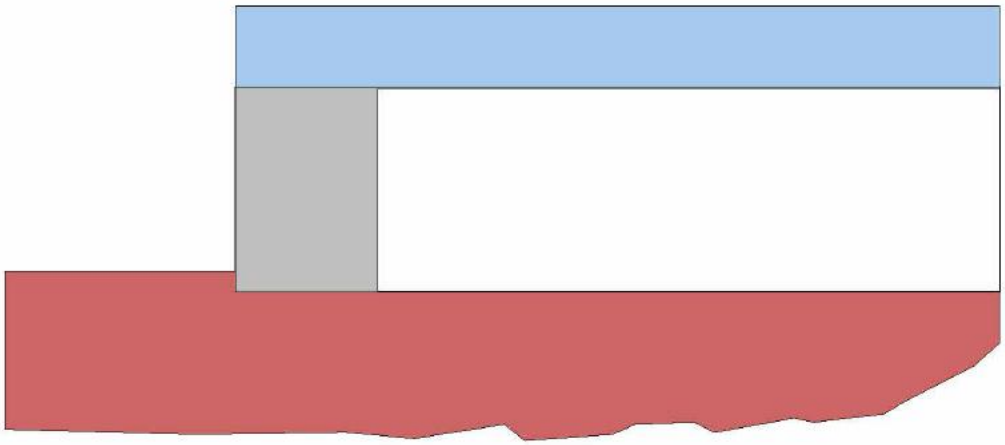
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BEARING CAPACITY for GIVEN LAYOUT – Using AASHTO 2017-2020 method

	STATIC	SEISMIC	UNITS
(Water table does not affect bearing capacity)			
Factored bearing resistance, q-n	16648	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	2.28	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 2 4 6 8 10[ft]

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DIRECT SLIDING for GIVEN LAYOUT (for METAL STRIPS reinforcements)

Along reinforced and foundation soils interface: CDR-static = 2.077

#	Metal strip Elevation [ft]	Metal strip Length [ft]	CDR Static	CDR Seismic	Metal strip Type #	Product name
1	1.23	21.00	2.507	N/A	1	HA50X4
2	3.69	21.00	2.685	N/A	1	HA50X4
3	6.15	21.00	2.894	N/A	1	HA50X4
4	8.61	21.00	3.143	N/A	1	HA50X4
5	11.07	21.00	3.442	N/A	1	HA50X4
6	13.53	21.00	3.811	N/A	1	HA50X4
7	15.99	21.00	4.275	N/A	1	HA50X4
8	18.45	21.00	4.880	N/A	1	HA50X4
9	20.91	21.00	5.697	N/A	1	HA50X4
10	23.37	21.00	6.864	N/A	1	HA50X4
11	25.83	21.00	8.666	N/A	1	HA50X4
12	28.00	21.00	11.330	N/A	1	HA50X4

ECCENTRICITY for GIVEN LAYOUT (for Coherent Gravity Mass Method)

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

#	Metal strip Elevation [ft]	Metal strip Length [ft]	e / L Static	e / L Seismic	Metal strip Type #	Product name
1	1.23	21.00	0.0828	N/A	1	HA50X4
2	3.69	21.00	0.0674	N/A	1	HA50X4
3	6.15	21.00	0.0533	N/A	1	HA50X4
4	8.61	21.00	0.0406	N/A	1	HA50X4
5	11.07	21.00	0.0292	N/A	1	HA50X4
6	13.53	21.00	0.0193	N/A	1	HA50X4
7	15.99	21.00	0.0109	N/A	1	HA50X4
8	18.45	21.00	0.0039	N/A	1	HA50X4
9	20.91	21.00	-0.0015	N/A	1	HA50X4
10	23.37	21.00	-0.0054	N/A	1	HA50X4
11	25.83	21.00	-0.0076	N/A	1	HA50X4
12	28.00	21.00	-0.0083	N/A	1	HA50X4

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RESULTS for STRENGTH **[Note: Actual CDR = (Yield stress) / (Actual stress)]** For Coherent Mass Method, Option A
Live Load included in calculating Tmax

#	Metal strip Elevation [ft]	Coverage ratio, Rc=b/Sh	Horizontal spacing, Sh [ft]	LTDS = Fy·Ac·Rc/b [lb/ft]	Tmax [lb/ft]	Tmd [lb/ft]	Specified minimum CDR Static	Actual calculated CDR Static	Specified minimum CDR seismic	Actual calculated CDR seismic
1	1.23	0.085	1.970	4949.2	4473.5	N/A	N/A	1.106	N/A	N/A
2	3.69	0.085	1.970	4949.2	4017.7	N/A	N/A	1.232	N/A	N/A
3	6.15	0.068	2.460	3963.4	3592.3	N/A	N/A	1.103	N/A	N/A
4	8.61	0.068	2.460	3963.4	3192.6	N/A	N/A	1.241	N/A	N/A
5	11.07	0.068	2.460	3963.4	2889.3	N/A	N/A	1.372	N/A	N/A
6	13.53	0.051	3.280	2972.6	2673.2	N/A	N/A	1.112	N/A	N/A
7	15.99	0.051	3.280	2972.6	2444.3	N/A	N/A	1.216	N/A	N/A
8	18.45	0.051	3.280	2972.6	2182.2	N/A	N/A	1.362	N/A	N/A
9	20.91	0.051	3.280	2972.6	1886.0	N/A	N/A	1.576	N/A	N/A
10	23.37	0.051	3.280	2972.6	1554.5	N/A	N/A	1.912	N/A	N/A
11	25.83	0.051	3.280	2972.6	1127.2	N/A	N/A	2.637	N/A	N/A
12	28.00	0.051	3.280	2972.6	933.8	N/A	N/A	3.183	N/A	N/A

RESULTS for PULLOUT

Live Load NOT included in calculating Tmax
NOTE: Live load is not included in calculating the overburden pressure used to assess pullout resistance.

#	Metal strip Elevation [ft]	Coverage Ratio Rc=b/Sh	Factored: Stat./Seis. Tmax [lb/ft]	Tmd [lb/ft]	Le (see NOTE) [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	1.23	0.085	4129.0	N/A	20.26	0.74	8028.4	N/A	1.944	N/A	N/A	N/A
2	3.69	0.085	3681.7	N/A	18.79	2.21	6805.1	N/A	1.848	N/A	N/A	N/A
3	6.15	0.068	3263.8	N/A	17.31	3.69	4552.9	N/A	1.395	N/A	N/A	N/A
4	8.61	0.068	2870.4	N/A	15.83	5.17	3733.8	N/A	1.301	N/A	N/A	N/A
5	11.07	0.068	2563.7	N/A	14.36	6.64	3379.6	N/A	1.318	N/A	N/A	N/A
6	13.53	0.051	2332.5	N/A	12.88	8.12	2709.2	N/A	1.162	N/A	N/A	N/A
7	15.99	0.051	2086.0	N/A	12.00	9.00	2723.3	N/A	1.306	N/A	N/A	N/A
8	18.45	0.051	1805.7	N/A	12.00	9.00	2721.6	N/A	1.507	N/A	N/A	N/A
9	20.91	0.051	1490.8	N/A	12.00	9.00	2517.0	N/A	1.688	N/A	N/A	N/A
10	23.37	0.051	1139.8	N/A	12.00	9.00	2109.4	N/A	1.851	N/A	N/A	N/A
11	25.83	0.051	718.3	N/A	12.00	9.00	1498.8	N/A	2.087	N/A	N/A	N/A
12	28.00	0.051	358.8	N/A	12.00	9.00	791.6	N/A	2.206	N/A	N/A	N/A

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RESULTS for CONNECTION (static conditions)

Live Load included in calculating Tmax

#	Metal strip Elevation [ft]	Coverage ratio Rc=b/Sh	Horizontal spacing, Sh [ft]	Connection force, To [lb/ft]	Reduction factor for connection break, CRu	Long-term connection strength,Tac (break criterion) [lb/ft]	Metal strip long-term strength, [lb/ft]	CDR connection break		CDR Metal strip strength		Product name
								Specified	Actual	Specified	Actual	
1	1.23	0.085	1.970	4474	1.00	4949	4949	N/A	1.11	N/A	1.11	HA50X4
2	3.69	0.085	1.970	4018	1.00	4949	4949	N/A	1.23	N/A	1.23	HA50X4
3	6.15	0.068	2.460	3592	1.00	3963	3963	N/A	1.10	N/A	1.10	HA50X4
4	8.61	0.068	2.460	3193	1.00	3963	3963	N/A	1.24	N/A	1.24	HA50X4
5	11.07	0.068	2.460	2889	1.00	3963	3963	N/A	1.37	N/A	1.37	HA50X4
6	13.53	0.051	3.280	2673	1.00	2973	2973	N/A	1.11	N/A	1.11	HA50X4
7	15.99	0.051	3.280	2444	1.00	2973	2973	N/A	1.22	N/A	1.22	HA50X4
8	18.45	0.051	3.280	2182	1.00	2973	2973	N/A	1.36	N/A	1.36	HA50X4
9	20.91	0.051	3.280	1886	1.00	2973	2973	N/A	1.58	N/A	1.58	HA50X4
10	23.37	0.051	3.280	1554	1.00	2973	2973	N/A	1.91	N/A	1.91	HA50X4
11	25.83	0.051	3.280	1127	1.00	2973	2973	N/A	2.64	N/A	2.64	HA50X4
12	28.00	0.051	3.280	934	1.00	2973	2973	N/A	3.18	N/A	3.18	HA50X4

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REINFORCED EARTH

JOB NUMBER :0 06/14/22
Job Name : IDEA RECo MSE Submittal
Designed By : KPB

Reinforced Earth Design Calculations
LRFD METHOD-AASHTO, 2020
Material Properties

REINFORCING STRIPS and/or LADDER REINFORCING STRIPS Fy = 65Ksi

Wall Geometry

HEIGHT OF WALL = H ft SOIL REINFORCEMENT LENGTH = B ft

Soils Properties

	Unit Weight - Kcf	Φ(deg)	Cohesion -Ksf	Ka	Ko	f*max@ground surface , SteelStrips
Select Material	0.135	34.00	-	0.28	0.44	2.900
Random fill	0.120	30.00	0.00	0.30	-	
Foundation	-	30.00	0.00			

f* = Apparent Coefficient Of Friction (Bond)=2.900 @ top, f*min at 20ft depth = 0.675

Load and Resistance Factors	Strength1 (Static)	Extreme Event I (Seismic)
BEARING PRESSURE &REINFORCEMENT TENSION & PULL-OUT		
Vertical Dead Load (MSE Backfil-) -Tension & Pull-Out force in Internal Stability	1.35	1.35
Vertical Dead Load, Pull-Out Resisting force.	1.00	1.00
Horizontal Earthpressure due to Random Backfill	1.50	1.00
Horizontal Earthpressure due to Live Load surcharge	1.75	0.50
Live Load Weight	1.75	0.50
Dynamic Earthpressure and RE mass Dynamic Inertia Force (Extreme Event I)	-----	1.00
Tensile Resistance of soil reinforcements	0.75	1.00
Resistance Factor for Pull Out of Strips (Internal Stability).	0.90	1.20
SLIDING & OVERTURNING		
Vertical Dead Load	1.00	1.00
Horizontal Earthpressure due to Random Backfill	1.50	1.00
Horizontal Earthpressure due to Live Load surcharge	1.75	0.50
Dynamic Earthpressure and RE mass Dynamic Inertia Force (Extreme Event I)	-----	1.00
Resistance Factor for Base Sliding (Mass Stability)	1.00	1.00

Wind on Structures (Strength III), Load Factor = 1.00, AASHTO 2016 and thereafter
Vehicular Collision Force (Extreme Event II) Load Factor = 1.00
Equivalent Height For Traffic Surcharge (Where Applicable)=2.08ft or 0.25Ksf
Coefficient Of Friction At Foundation Level (Sliding) = 0.58
Tributary Panel Width = 9.84ft, Vertical spacing of reinforcement =2.46ft
Horizontal Stress @ Facing= 100% Of Maximum Stress @ Line Of Active Zone..
Factored Maximum Reinforcement Tension = 9.76Kips / strip

These calculations are furnished exclusively for the use in connection with this project. These calculations contain information proprietary to the Reinforced Earth Company. Except as specifically authorized in writing by the Reinforced Earth Co., possession of these calculations does not authorize use of proprietary information contained therein, nor transmission to any other organization of the calculations or of information contained therein.

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REINFORCED EARTH

06/14/22

JOB NUMBER :0 DESIGNED : KPB COMMENTS: Example Wall

MASS STABILITY AND MAXIMUM BEARING PRESSURE - LRFD STATIC , COULOMB'S THEORY

H = 30.00ft, H'(active zone) = 30.00ft, B = 27.00ft

AASHTO - LRFD 2020- DESIGN TYPE : LEVEL SURCHARGE

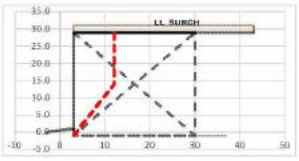
Equiv. height Traffic Live Load Surcharge= 2.08ft or 0.25Ksf
Coeff. Active Earthpressure= Ka =0.2973, Angle δ w/horiz. = 20.00°
Foundation,Coeff. of friction@ bott/wall=0.58, Cohesion=0.00Ksf, Tributary width= 9.84ft

f*= Coefficient of apparent friction @finished grade = 2.90

Select Fill γ=0.135Kcf, Φ=34.00°, Rand. Fill γ=0.120Kcf, Φ=30.00°, Rand.Cohes=0.00Ksf -

STATIC MASS STABILITY

FACTORED VERTICAL LOADS (Kips)		Arm (ft)	Moment (Kip-ft)	
109.35		13.50	1476.2	
0.00	109.35	27.00	0.0	1476.23
0.00		27.00	0.0	
0.00		0.00	0.0	
0.00		0.00	0.0	
0.00		0.00	0.0	
8.24	VertComp.Pa	27.00	222.4	
1.33	VertComp.Ps	27.00	36.0	
0.00	VertComp.Ppsv	27.00	0.00	
118.92			1734.65	



FACTORED HORIZONTAL LOADS, COULOMB'S THEORY				
22.63	HorizCompontPa	10.00	226.30	Soil
3.67	HorizCompontPs	15.00	55.00	Traffic LiveLoad
0.00		15.00	0.00	
26.30			281.30	

Eccentricity =1.28ft, OK, RESULT MIDDLE 2/3 OF REINFORCED MASS [4.50ft ≤ 12.22ft ≤ 22.50ft]

CAPACITY/DEMAND RATIO (**CDR**)

1) OVERTURNING	6.17	>=1.0, OK
2) SLIDING	2.61	>=1.0, OK

Bearing Pressure

STATIC MASS STABILITY			Arm (ft)	MOMENT (Kip-ft)		
1.35DL	1.00DL	1DL +0.0LL		1.35DL	1.00DL	1DL +0.0LL
147.62	109.35	109.35	13.50	1992.9	1476.2	1476.2
0.00	0.00	0.00	0.00	0.0	0.0	0.0
9.57	9.57	9.57	27.00	258.4	258.4	258.4
11.81	11.81	0.00	13.50	159.5	159.5	0.0
169.01	130.73	118.92		2410.8	1894.1	1734.7

HORIZONTAL LOADS (SAME AS FOR MASS STABILITY, static case)
1.35DL factor, Eccent. = 0.90ft, Factored Bearing Pressure = 6.71Ksf
1.00DL, Eccent. = 1.16ft , Factored BP = 5.30Ksf
1.00DL+0.0 LL, Eccent. = 1.28ft, Factored BP = 4.87Ksf
Unfactored Bearing Pressure @ toe of wall = 4.82Ksf

LRFD - REINFORCED EARTH INTERNAL STABILITY SUMMARY - Coherent Gravity Method LargeRectangPanel

Steel Strips

LEVEL	Factored Horiz. Stress σ_h @ Fac'g -Ksf	Density Strips 50x4mm	Factored Tension <= 9.76Kips	Factored Horiz. Stress σ_h Bond -Ksf	Effective Length CDR >=1.0	Reinf. Length & Minimum Density >=1.0
2.000	0.34	3	2.77	0.16	3.03	27ft (0.99)
4.170	0.49	3	3.98	0.32	2.87	27ft (1.22)
6.630	0.65	3	5.24	0.48	2.66	27ft (1.61)
9.090	0.79	3	6.36	0.63	2.43	27ft (1.96)
11.550	0.91	3	7.35	0.76	2.19	27ft (2.26)
14.010	1.02	3	8.22	0.87	1.91	27ft (2.52)
16.470	1.11	3	8.98	0.98	1.68	27ft (2.76)
18.930	1.19	3	9.62	1.06	1.42	27ft (2.96)
21.390	1.31	4	7.91	1.18	1.75	27ft (3.24)
23.850	1.46	4	8.85	1.34	1.84	27ft (3.63)
26.310	1.62	5	7.87	1.50	2.41	27ft (4.03)
28.770	1.79	5	8.68	1.67	2.51	27ft (4.45)

REWall_Design_2021 FWHA IDEA

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REINFORCED EARTH

STANDARD MSE WALL DESIGN SUMMARY - LRFD - Coherent Gravity Method

INTERNAL STABILITY

JOB NUMBER :0

STRUCTURE No.: Example Wall

Designed by : KPB

Date: 6/14/2022

WALL HEIGHT = 30.00 ft

REINFORCING STRIP LENGTH = 27 ft

DESIGN TYPE : LEVEL SURCHARGE

Equiv. height Traffic Live Load Surcharge= 2.08 ft or 0.25Ksf

DESIGN LIFE = 75yrs

Factored Maximum Reinforcement Tension = 9.76Kips

Reinf. Layer No.	Depth LEVEL	Earth Pressure Coefficient K	Coefficient Apparent Friction f*	Effective Strip Length Le ft	Number of Strips N	Factored max. vert STRESS σ_v Ksf	Factored max. horiz. STRESS σ_h Ksf	Factored max. horiz. @ Fac'g σ_h Kips	Factored Strip Tension Kips	Factored Connection Tension Kips	Factored max. vert (Bond) Ksf	Factored max. horiz. (Bond) Ksf	Strip Tension (bond) T Kips	Pull Out Resistance per strip R Kips	EFFECT. LENGTH CDR >1.0
1	2.000	0.425	2.677	18.0	3	0.807	0.343	0.343	2.77	2.77	0.369	0.157	8.30	3.84	3.03
2	4.170	0.408	2.436	18.0	3	1.210	0.494	0.494	3.98	3.98	0.773	0.315	11.95	7.29	2.87
3	6.630	0.388	2.162	18.0	3	1.672	0.649	0.649	5.24	5.24	1.234	0.479	15.72	10.28	2.66
4	9.090	0.369	1.889	18.0	3	2.137	0.789	0.789	6.36	6.36	1.700	0.627	19.09	12.31	2.43
5	11.550	0.350	1.615	18.0	3	2.607	0.911	0.911	7.35	7.35	2.169	0.758	22.05	13.38	2.19
6	14.010	0.330	1.341	18.0	3	3.085	1.018	1.018	8.22	8.22	2.648	0.874	24.65	13.48	1.91
7	16.470	0.311	1.067	18.9	3	3.583	1.113	1.113	8.98	8.98	3.145	0.977	26.94	13.23	1.68
8	18.930	0.291	0.794	20.4	3	4.096	1.193	1.193	9.62	9.62	3.657	1.065	28.87	12.19	1.42
9	21.390	0.283	0.675	21.8	4	4.626	1.308	1.308	7.91	7.91	4.186	1.184	31.66	12.55	1.75
10	23.850	0.283	0.675	23.3	4	5.176	1.463	1.463	8.85	8.85	4.735	1.339	35.42	14.94	1.84
11	26.310	0.283	0.675	24.8	5	5.748	1.625	1.625	7.87	7.87	5.307	1.500	39.33	17.53	2.41
12	28.770	0.283	0.675	26.3	5	6.345	1.794	1.794	8.68	8.68	5.903	1.669	43.42	20.31	2.51

REWall_Design_2021 FAYHA DEA

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AASHTO 2017-2020
IDEA

MSEW+: Update # 2021.20

PROJECT IDENTIFICATION

Title: IDEA
Project Number: SLOPE SURCHARGE
Client:
Designer: DE
Station Number:

Description:

Company's information:

Name:
Street:

Telephone #:
Fax #:
E-Mail:

File path and name: C:\Users\daniel.engidashet\OneDrive - VINCI Constructio.....
.....DEA\MSEW1-Slope.BENp

Original date and time of creating this file: Thu May 12 14:19:19 2022

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

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SOIL DATA

REINFORCED SOIL

Unit weight,	γ	135.0 lb/ft^3
Design value of internal angle of friction,	ϕ	34.0 °

RETAINED SOIL

Unit weight,	γ	120.0 lb/ft^3
Design value of internal angle of friction,	ϕ	30.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight,	$\gamma_{equiv.}$	125.0 lb/ft^3
Equivalent internal angle of friction,	$\phi_{equiv.}$	30.0 °
Equivalent cohesion,	$c_{equiv.}$	0.0 lb/ft^2

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Ka (external stability) = 0.5254 (eq. 17 is utilized to calculate Ka for all batters)
(For external stability user specified $\delta = 20.00°$)

BEARING CAPACITY

Bearing capacity is controlled by general shear.
Bearing capacity factors (calculated by MSEW): Nc = 30.14 Nγ= 22.40

SEISMICITY

Not Applicable

FOR EXTERNAL STABILITY

Ka = 0.5254 In Coulomb equation for Ka, Omega was taken as ZERO and backslope inclined at angle I.

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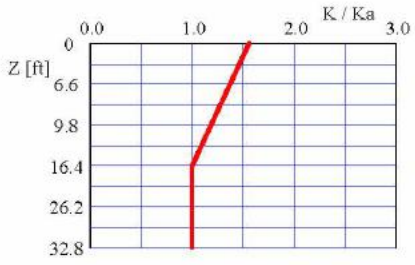
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INPUT DATA: Metal strips (Analysis)

D A T A	Metal strip type #1	Metal strip type #2	Metal strip type #3	Metal strip type #4	Metal strip type #5
Yield strength of steel, Fy [kips/in ²]	65.0	N/A	N/A	N/A	N/A
Gross width of strip, b [in]	2.0	N/A	N/A	N/A	N/A
Vertical spacing, Sv [ft]	Varies	N/A	N/A	N/A	N/A
Design cross section area, Ac [in ²]	0.20	N/A	N/A	N/A	N/A
Ribbed steel strips.					
Uniformity Coefficient of reinforced soil, Cu = D60/D10 = 4.0					
Friction angle along reinforcement-soil interface, ρ					
@ the top	70.97	N/A	N/A	N/A	N/A
@ 19.7 ft or below	34.00	N/A	N/A	N/A	N/A
Pullout resistance factor, F*					
@ the top	2.90	N/A	N/A	N/A	N/A
@ 19.7 ft or below	0.67	N/A	N/A	N/A	N/A
Scale-effect correction factor, α	1.00	N/A	N/A	N/A	N/A

Variation of Lateral Earth Pressure Coefficient With Depth (Coherent Mass)

Z	K / Ka
0 ft	1.56
3.3 ft	1.47
6.6 ft	1.37
9.8 ft	1.28
13.1 ft	1.19
16.4 ft	1.09
19.7 ft	1.00



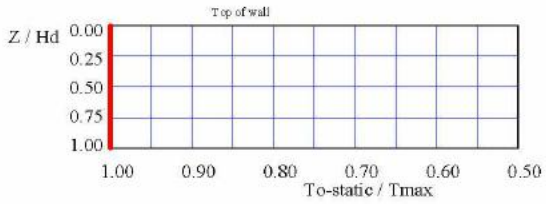
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INPUT DATA: Facia and Connection (Analysis)

FACIA type: Segmental precast concrete panels.
Depth of panel is 0.50 ft. Horizontal distance to Center of Gravity of panel is 0.25 ft.
Average unit weight of panel is γ_f = 150.00 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	HA50X4	N/A	N/A	N/A	N/A
Strength reduction at the connection, CRu = Fyc / Fy	1.00	N/A	N/A	N/A	N/A

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

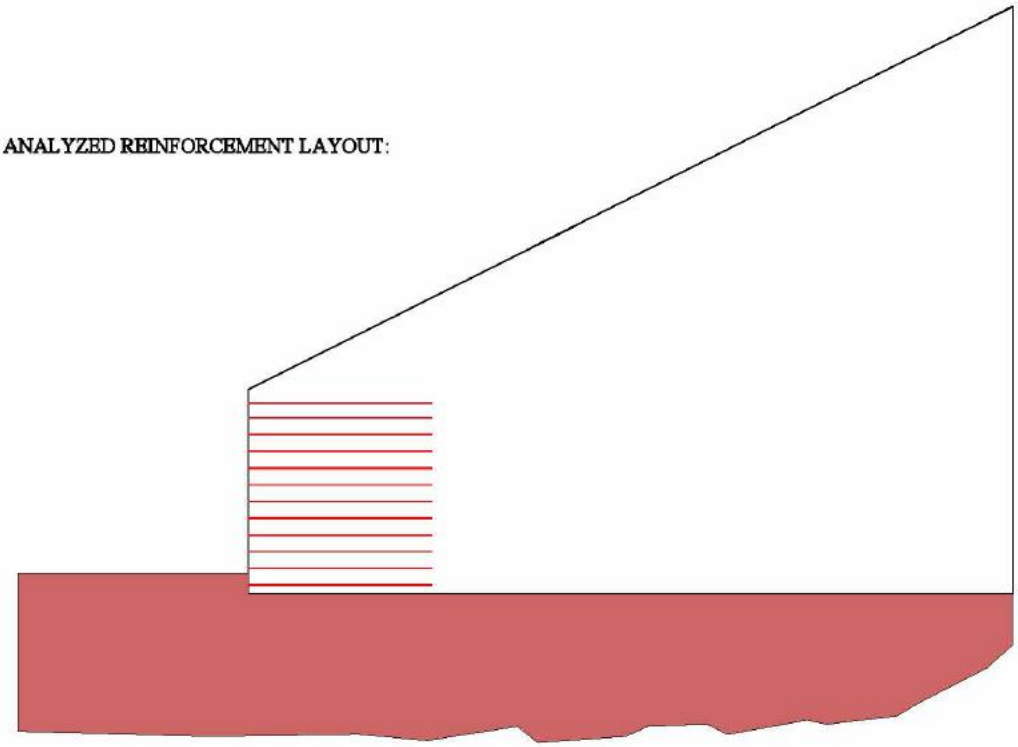
Design height, Hd 30.00 [ft] { Embedded depth is E = 3.00 ft, and height above top of finished bottom grade is H = 27.00 ft }

Soil in front of wall is Horizontal.

Batter, ω 0.0 [deg]
Backslope, β 26.6 [deg]
Backslope rise 100.0 [ft] Broken back equivalent angle, I = 26.60° (see Fig. 25 in DEMO 82)

UNIFORM SURCHARGE
Uniformly distributed dead load is 0.0 [lb/ft²]

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 2 4 6 8 10[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
Metal Strips:		0.75	1.00

Resistance factor for reinforcement tension in connectors	ϕ	Static	Combined static/seismic
Metal Strips:		0.75	1.00

Resistance factor for Metal Strips 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}):	$(\gamma_{p\text{-}EH})_{EQ}$	1.50
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	0.90

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ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, CDR = 1.86, factored bearing load = 10178 lb/ft².
Foundation Interface: Direct sliding, CDR = 1.129, Eccentricity, e/L = 0.1567, CDR-overturning = 2.25

	METAL STRIP			CONNECTION						
#	Elevation [ft]	Length [ft]	Type #	CDR [connection break]	CDR Strength	Metal strip strength CDR	Pullout resistance CDR	Direct sliding CDR	Eccentricity e/L	Product name
1	1.23	27.00	1	1.27	1.27	1.265	3.302	1.345	0.1447	HA50X4
2	3.69	27.00	1	1.23	1.23	1.229	2.849	1.399	0.1213	HA50X4
3	6.15	27.00	1	1.39	1.39	1.392	2.847	1.458	0.0986	HA50X4
4	8.61	27.00	1	1.32	1.32	1.318	2.357	1.524	0.0767	HA50X4
5	11.07	27.00	1	1.21	1.21	1.208	2.111	1.595	0.0553	HA50X4
6	13.53	27.00	1	1.39	1.39	1.395	2.858	1.675	0.0344	HA50X4
7	15.99	27.00	1	1.20	1.20	1.196	2.631	1.762	0.0138	HA50X4
8	18.45	27.00	1	1.35	1.35	1.346	3.059	1.857	-0.0071	HA50X4
9	20.91	27.00	1	1.53	1.53	1.535	3.619	1.961	-0.0288	HA50X4
10	23.37	27.00	1	1.82	1.82	1.816	4.266	2.070	-0.0524	HA50X4
11	25.83	27.00	1	2.39	2.39	2.393	5.376	2.178	-0.0802	HA50X4
12	28.00	27.00	1	2.50	2.50	2.500	5.197	2.258	-0.1118	HA50X4

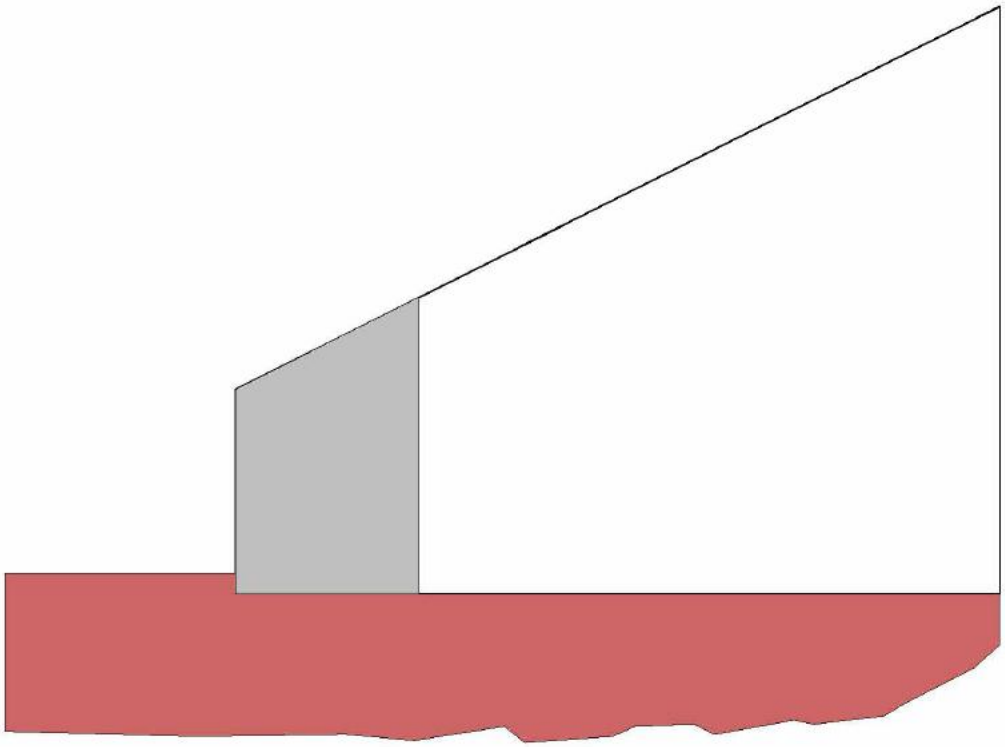
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BEARING CAPACITY for GIVEN LAYOUT – Using AASHTO 2017-2020 method

	STATIC	SEISMIC	UNITS
(Water table does not affect bearing capacity)			
Factored bearing resistance, q-n	18913	N/A	[lb/ft ²]
Factored bearing load, σ_V	10178.2	N/A	[lb/ft ²]
Eccentricity, e	3.11	N/A	[ft]
Eccentricity, e/L	0.115	N/A	
CDR calculated	1.86	N/A	
Base length	27.00	N/A	[ft]

Unfactored applied bearing pressure = (Unfactored R) / [L - 2 * (Unfactored e)] =
Unfactored R = 154412.00 [lb/ft], L = 27.00, Unfactored e = 2.77 [ft], and Sigma = 7193.74 [lb/ft ²]



SCALE:

0 2 4 6 8 10[ft]

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DIRECT SLIDING for GIVEN LAYOUT (for METAL STRIPS reinforcements)

Along reinforced and foundation soils interface: CDR-static = 1.129

#	Metal strip Elevation [ft]	Metal strip Length [ft]	CDR Static	CDR Seismic	Metal strip Type #	Product name
1	1.23	27.00	1.345	N/A	1	HA50X4
2	3.69	27.00	1.399	N/A	1	HA50X4
3	6.15	27.00	1.458	N/A	1	HA50X4
4	8.61	27.00	1.524	N/A	1	HA50X4
5	11.07	27.00	1.595	N/A	1	HA50X4
6	13.53	27.00	1.675	N/A	1	HA50X4
7	15.99	27.00	1.762	N/A	1	HA50X4
8	18.45	27.00	1.857	N/A	1	HA50X4
9	20.91	27.00	1.961	N/A	1	HA50X4
10	23.37	27.00	2.070	N/A	1	HA50X4
11	25.83	27.00	2.178	N/A	1	HA50X4
12	28.00	27.00	2.258	N/A	1	HA50X4

ECCENTRICITY for GIVEN LAYOUT (for Coherent Gravity Mass Method)

At interface with foundation: e/L static = 0.1567; Overturning: CDR-static = 2.25

#	Metal strip Elevation [ft]	Metal strip Length [ft]	e / L Static	e / L Seismic	Metal strip Type #	Product name
1	1.23	27.00	0.1447	N/A	1	HA50X4
2	3.69	27.00	0.1213	N/A	1	HA50X4
3	6.15	27.00	0.0986	N/A	1	HA50X4
4	8.61	27.00	0.0767	N/A	1	HA50X4
5	11.07	27.00	0.0553	N/A	1	HA50X4
6	13.53	27.00	0.0344	N/A	1	HA50X4
7	15.99	27.00	0.0138	N/A	1	HA50X4
8	18.45	27.00	-0.0071	N/A	1	HA50X4
9	20.91	27.00	-0.0288	N/A	1	HA50X4
10	23.37	27.00	-0.0524	N/A	1	HA50X4
11	25.83	27.00	-0.0802	N/A	1	HA50X4
12	28.00	27.00	-0.1118	N/A	1	HA50X4

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RESULTS for STRENGTH **[Note: Actual CDR = (Yield stress) / (Actual stress)]** For Coherent Mass Method, Option A
Live Load included in calculating Tmax

#	Metal strip Elevation [ft]	Coverage ratio, Rc=b/Sh	Horizontal spacing, Sh [ft]	LTDS = Fy·Ac·Rc/b [lb/ft]	Tmax [lb/ft]	Tmd [lb/ft]	Specified minimum CDR Static	Actual calculated CDR Static	Specified minimum CDR seismic	Actual calculated CDR seismic
1	1.23	0.118	1.410	6914.9	5465.7	N/A	N/A	1.265	N/A	N/A
2	3.69	0.102	1.640	5945.1	4838.0	N/A	N/A	1.229	N/A	N/A
3	6.15	0.102	1.640	5945.1	4271.0	N/A	N/A	1.392	N/A	N/A
4	8.61	0.085	1.970	4949.2	3754.5	N/A	N/A	1.318	N/A	N/A
5	11.07	0.068	2.460	3963.4	3280.2	N/A	N/A	1.208	N/A	N/A
6	13.53	0.068	2.460	3963.4	2841.4	N/A	N/A	1.395	N/A	N/A
7	15.99	0.051	3.280	2972.6	2484.4	N/A	N/A	1.196	N/A	N/A
8	18.45	0.051	3.280	2972.6	2209.0	N/A	N/A	1.346	N/A	N/A
9	20.91	0.051	3.280	2972.6	1936.7	N/A	N/A	1.535	N/A	N/A
10	23.37	0.051	3.280	2972.6	1637.1	N/A	N/A	1.816	N/A	N/A
11	25.83	0.051	3.280	2972.6	1242.3	N/A	N/A	2.393	N/A	N/A
12	28.00	0.051	3.280	2972.6	1189.1	N/A	N/A	2.500	N/A	N/A

RESULTS for PULLOUT
Live Load NOT included in calculating Tmax

#	Metal strip Elevation [ft]	Coverage Ratio Rc=b/Sh	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	1.23	0.118	5465.7	N/A	26.26	0.74	18048.4	N/A	3.302	N/A	N/A	N/A
2	3.69	0.102	4838.0	N/A	24.79	2.21	13784.4	N/A	2.849	N/A	N/A	N/A
3	6.15	0.102	4271.0	N/A	23.31	3.69	12159.8	N/A	2.847	N/A	N/A	N/A
4	8.61	0.085	3754.5	N/A	21.83	5.17	8850.7	N/A	2.357	N/A	N/A	N/A
5	11.07	0.068	3280.2	N/A	20.36	6.64	6924.6	N/A	2.111	N/A	N/A	N/A
6	13.53	0.068	2841.4	N/A	18.88	8.12	8121.6	N/A	2.858	N/A	N/A	N/A
7	15.99	0.051	2484.4	N/A	17.41	9.59	6535.9	N/A	2.631	N/A	N/A	N/A
8	18.45	0.051	2209.0	N/A	16.41	10.59	6758.3	N/A	3.059	N/A	N/A	N/A
9	20.91	0.051	1936.7	N/A	16.41	10.59	7009.6	N/A	3.619	N/A	N/A	N/A
10	23.37	0.051	1637.1	N/A	16.41	10.59	6983.2	N/A	4.266	N/A	N/A	N/A
11	25.83	0.051	1242.3	N/A	16.41	10.59	6679.1	N/A	5.376	N/A	N/A	N/A
12	28.00	0.051	1189.1	N/A	16.41	10.59	6180.2	N/A	5.197	N/A	N/A	N/A

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RESULTS for CONNECTION (static conditions)

Live Load included in calculating Tmax

#	Metal strip Elevation [ft]	Coverage ratio Rc=b/Sh	Horizontal spacing, Sh [ft]	Connection force, To [lb/ft]	Reduction factor for connection break, CRu	Long-term connection strength,Tac (break criterion) [lb/ft]	Metal strip long-term strength, [lb/ft]	CDR connection break		CDR Metal strip strength		Product name
								Specified	Actual	Specified	Actual	
1	1.23	0.118	1.410	5466	1.00	6915	6915	N/A	1.27	N/A	1.27	HA50X4
2	3.69	0.102	1.640	4838	1.00	5945	5945	N/A	1.23	N/A	1.23	HA50X4
3	6.15	0.102	1.640	4271	1.00	5945	5945	N/A	1.39	N/A	1.39	HA50X4
4	8.61	0.085	1.970	3754	1.00	4949	4949	N/A	1.32	N/A	1.32	HA50X4
5	11.07	0.068	2.460	3280	1.00	3963	3963	N/A	1.21	N/A	1.21	HA50X4
6	13.53	0.068	2.460	2841	1.00	3963	3963	N/A	1.39	N/A	1.39	HA50X4
7	15.99	0.051	3.280	2484	1.00	2973	2973	N/A	1.20	N/A	1.20	HA50X4
8	18.45	0.051	3.280	2209	1.00	2973	2973	N/A	1.35	N/A	1.35	HA50X4
9	20.91	0.051	3.280	1937	1.00	2973	2973	N/A	1.53	N/A	1.53	HA50X4
10	23.37	0.051	3.280	1637	1.00	2973	2973	N/A	1.82	N/A	1.82	HA50X4
11	25.83	0.051	3.280	1242	1.00	2973	2973	N/A	2.39	N/A	2.39	HA50X4
12	28.00	0.051	3.280	1189	1.00	2973	2973	N/A	2.50	N/A	2.50	HA50X4

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REINFORCED EARTH

JOB NUMBER :0 06/14/22
Job Name : IDEA RECo MSE Submittal
Designed By : KPB

Reinforced Earth Design Calculations
LRFD METHOD-AASHTO, 2020
Material Properties
REINFORCING STRIPS and/or LADDER REINFORCING STRIPS Fy = 65Ksi

Wall Geometry
HEIGHT OF WALL = H ft SOIL REINFORCEMENT LENGTH = B ft

Soils Properties

	Unit Weight - Kcf	Φ(deg)	Cohesion -Ksf	Ka	Ko	f*max@ground surface , SteelStrips
Select Material	0.135	34.00	-	0.28	0.44	2.900
Random fill	0.120	30.00	0.00	0.52	-	
Foundation	-	30.00	0.00			

f* = Apparent Coefficient Of Friction (Bond)=2.900 @ top, f*min at 20ft depth = 0.675

Load and Resistance Factors	StrengthI (Static)	Extreme Event I (Seismic)
BEARING PRESSURE &REINFORCEMENT TENSION & PULL-OUT		
Vertical Dead Load (MSE Backfil-) -Tension & Pull-Out force in Internal Stability	1.35	1.35
Vertical Dead Load, Pull-Out Resisting force.	1.00	1.00
Horizontal Earthpressure due to Random Backfill	1.50	1.00
Horizontal Earthpressure due to Live Load surcharge	1.75	0.50
Live Load Weight	1.75	0.50
Dynamic Earthpressure and RE mass Dynamic Inertia Force (Extreme Event I)	-----	1.00
Tensile Resistance of soil reinforcements	0.75	1.00
Resistance Factor for Pull Out of Strips (Internal Stability).	0.90	1.20
SLIDING & OVERTURNING		
Vertical Dead Load	1.00	1.00
Horizontal Earthpressure due to Random Backfill	1.50	1.00
Horizontal Earthpressure due to Live Load surcharge	1.75	0.50
Dynamic Earthpressure and RE mass Dynamic Inertia Force (Extreme Event I)	-----	1.00
Resistance Factor for Base Sliding (Mass Stability)	1.00	1.00

Wind on Structures (Strength III), Load Factor = 1.00, AASHTO 2016 and thereafter
Vehicular Collision Force (Extreme Event II) Load Factor = 1.00
Equivalent Height For Traffic Surcharge (Where Applicable)=2.08ft or 0.25Ksf
Coefficient Of Friction At Foundation Level (Sliding) = 0.58
Tributary Panel Width = 9.84ft, Vertical spacing of reinforcement =2.46ft
Horizontal Stress @ Facing= 100% Of Maximum Stress @ Line Of Active Zone..
Factored Maximum Reinforcement Tension = 9.76Kips / strip

These calculations are furnished exclusively for the use in connection with this project. These calculations contain information proprietary to the Reinforced Earth Company. Except as specifically authorized in writing by the Reinforced Earth Co., possession of these calculations does not authorize use of proprietary information contained therein, nor transmission to any other organization of the calculations or of information contained therein.

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REINFORCED EARTH

06/14/22

JOB NUMBER :0 DESIGNED : KPB COMMENTS: Example Wall

MASS STABILITY AND MAXIMUM BEARING PRESSURE - LRFD STATIC , COULOMB'S THEORY

H = 30.00ft, H'(active zone) = 35.29ft, B = 27.00ft

AASHTO - LRFD 2020- DESIGN TYPE : 2.00 :1 INFINITE SLOPE

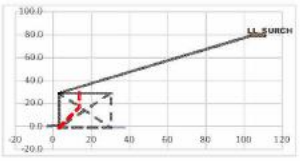
Equiv. height Traffic Live Load Surcharge= NEGLIGIBLE
Coeff. Active Earthpressure= Ka =0.5244, Angle δ w/horiz. = 20.00°
Foundation,Coeff. of friction@ bott/wall=0.58, Cohesion=0.00Ksf, Tributary width= 9.84ft

f*= Coefficient of apparent friction @finished grade = 2.90

Select Fill γ=0.135Kcf, Φ=34.00°, Rand. Fill γ=0.120Kcf, Φ=30.00°, Rand.Cohes=0.00Ksf -

STATIC MASS STABILITY

FACTORED VERTICAL LOADS (Kips)		Arm (ft)	Moment (Kip-ft)	
109.35		13.50	1476.2	
0.00	131.22	27.00	0.0	1869.89
0.00		27.00	0.0	
21.87		18.00	393.7	
0.00		0.00	0.0	
0.00		0.00	0.0	
30.54	VertComp.Pa	27.00	824.7	
0.00	VertComp.Ps	27.00	0.0	
0.00	VertComp.Ppsv	27.00	0.00	
161.76			2694.60	



FACTORED HORIZONTAL LOADS, COULOMB'S THEORY

83.92	HorizCompontPa	14.50	1216.86	Soil
0.00	HorizCompontPs	0.00	0.00	Traffic LiveLoad
0.00		0.00	0.00	
83.92			1216.86	

Eccentricity =4.36ft, OK, RESULT MIDDLE 2/3 OF REINFORCED MASS [4.50ft ≤ 9.14ft ≤ 22.50ft]

CAPACITY/DEMAND RATIO (**CDR**)

1) OVERTURNING	2.21	>=1.0, OK
2) SLIDING	1.11	>=1.0, OK

Bearing Pressure

STATIC MASS STABILITY			Arm (ft)	MOMENT (Kip-ft)		
1.35DL	1.00DL	1DL +0.0LL		1.35DL	1.00DL	1DL +0.0LL
177.15	131.22	131.22	14.25	2524.3	1869.9	1869.9
0.00	0.00	0.00	0.00	0.0	0.0	0.0
30.54	30.54	30.54	27.00	824.7	824.7	824.7
0.00	0.00	0.00	0.00	0.0	0.0	0.0
207.69	161.76	161.76		3349.1	2694.6	2694.6

HORIZONTAL LOADS (SAME AS FOR MASS STABILITY, static case)
1.35DL factor, Eccent. = 3.23ft, Factored Bearing Pressure = 10.12Ksf
1.00DL, Eccent. = 4.36ft , Factored BP = 8.85Ksf
1.00DL+0.0 LL, Eccent. = 4.36ft, Factored BP = 8.85Ksf
Unfactored Bearing Pressure @ toe of wall = 7.14Ksf

LRFD - REINFORCED EARTH INTERNAL STABILITY SUMMARY - Coherent Gravity Method LargeRectangPanel

Steel Strips

LEVEL	Factored Horiz. Stress σ_h @ Fac'g -Ksf	Density Strips 50x4mm	Factored Tension <= 9.76Kips	Factored Horiz. Stress σ_h Bond -Ksf	Effective Length CDR >=1.0	Reinf. Length & Minimum Density >=1.0
2.000	0.61	3	4.95	0.61	2.46	27ft (1.52)
4.170	0.75	3	6.02	0.75	2.05	27ft (1.85)
6.630	0.88	3	7.11	0.88	1.65	27ft (2.19)
9.090	1.00	3	8.06	1.00	1.26	27ft (2.48)
11.550	1.10	3	8.87	1.10	1.04	27ft (2.88)
14.010	1.20	3	9.66	1.20	1.12	27ft (2.97)
16.470	1.35	4	8.17	1.35	1.57	27ft (3.35)
18.930	1.56	4	9.43	1.56	1.59	27ft (3.86)
21.390	1.78	5	8.64	1.78	2.00	27ft (4.43)
23.850	2.03	6	8.20	2.03	2.41	27ft (5.04)
26.310	2.30	6	9.29	2.30	2.41	27ft (5.71)
28.770	2.60	7	9.01	2.60	2.80	27ft (6.46)

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REINFORCED EARTH

STANDARD MSE WALL DESIGN SUMMARY - LRFD - Coherent Gravity Method

INTERNAL STABILITY

JOB NUMBER :0

STRUCTURE No.: Example Wall

Designed by : KPB

Date: 6/14/2022

WALL HEIGHT = 30.00 ft

REINFORCING STRIP LENGTH = 27 ft

DESIGN TYPE : 2.00 :1 INFINITE SLOPE

Equiv. height Traffic Live Load Surcharge= NEGLIGIBLE

DESIGN LIFE = 75yrs

Factored Maximum Reinforcement Tension = 9.76Kips

Reinf. Layer No.	Depth LEVEL	Earth Pressure Coefficient K	Coefficient Apparent Friction f*	Effective Strip Length Le ft	Number of Strips N	Factored max. vert STRESS σ_v Ksf	Factored max. horiz. STRESS σ_h Ksf	Factored max. horiz. @ Fac'g σ_h Ksf	Factored Strip Tension Kips	Factored Connection Tension Kips	Factored max. vert (Bond) Ksf	Factored max. horiz. (Bond) Ksf	Strip Tension (bond) T Kips	Full Out Resistance per strip R Kips	EFFECT. LENGTH C.D.R. >1.0
1	2.000	0.383	1.632	16.4	3	1.602	0.614	0.614	4.95	4.95	1.602	0.614	14.85	12.16	2.46
2	4.170	0.366	1.390	16.4	3	2.040	0.747	0.747	6.02	6.02	2.040	0.747	18.07	12.34	2.05
3	6.630	0.347	1.117	16.4	3	2.544	0.882	0.882	7.11	7.11	2.544	0.882	21.34	11.70	1.65
4	9.090	0.327	0.843	16.4	3	3.055	0.999	0.999	8.06	8.06	3.055	0.999	24.19	10.19	1.26
5	11.550	0.308	0.675	16.4	3	3.574	1.099	1.099	8.87	8.87	3.574	1.099	26.61	9.24	1.04
6	14.010	0.292	0.675	17.4	3	4.099	1.198	1.198	9.66	9.66	4.099	1.198	28.99	10.84	1.12
7	16.470	0.283	0.675	18.9	4	4.773	1.349	1.349	8.17	8.17	4.773	1.349	32.66	12.82	1.57
8	18.930	0.283	0.675	20.4	4	5.512	1.558	1.558	9.43	9.43	5.512	1.558	37.72	14.96	1.59
9	21.390	0.283	0.675	21.8	5	6.313	1.785	1.785	8.64	8.64	6.313	1.785	43.20	17.27	2.00
10	23.850	0.283	0.675	23.3	6	7.188	2.032	2.032	8.20	8.20	7.188	2.032	49.19	19.75	2.41
11	26.310	0.283	0.675	24.8	6	8.148	2.304	2.304	9.29	9.29	8.148	2.304	55.76	22.40	2.41
12	28.770	0.283	0.675	26.3	7	9.211	2.604	2.604	9.01	9.01	9.211	2.604	63.04	25.21	2.80

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A3.0 Construction

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A3.1 Construction Procedures

The following document is a marketing flyer that describes the construction procedure in basic terms.

Reinforced Earth®

Construction Procedure

REINFORCED EARTH®

This is an abbreviated guide to constructing a Reinforced Earth retaining wall with precast concrete facing panels. Refer to our construction manual for complete procedure, handling of materials and recommended safety precautions.





Preparing the bottom course of panels:

1. Form and pour leveling pad.
2. Set bottom panels.
   - 2a. Batter the panels as required.
   - 2b. Drive wedges to hold batter.
   - 2c. Brace the bottom panels.
3. Apply adhesive and filter fabric.









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Bottom row of soil reinforcements:

- Spread and compact select backfill (in lifts) to 2" above the first level of tie strips on the back face of the panels.
- Install the soil reinforcements.

Subsequent rows of soil reinforcements (repeat until top course of panels are installed and backfilled:

- Spread and compact select backfill (in lifts) to 2" above the next level of tie strips on the back face of the panels.
- Install the soil reinforcements.
- When needed, install the next course of panels.
  - Batter the panels as required.
  - Drive wedges to hold the batter.
  - Shoulder clamp new panels to the adjacent secured panels.
- Apply adhesive and filter fabric to the new panels.
- Install fall protection as required.

Prepare the top grading and install the wall top out as required:



Soil reinforcements:



Spread and compact backfill:



Shoulder clamp:



Compaction Equipment:



Flat plate tamper:
Within 3 feet of wall face



Large roller:
Keep 3 feet away from wall face

Fall Protection:



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A3.1.1 Innovations

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A3.1.2 Construction Manual

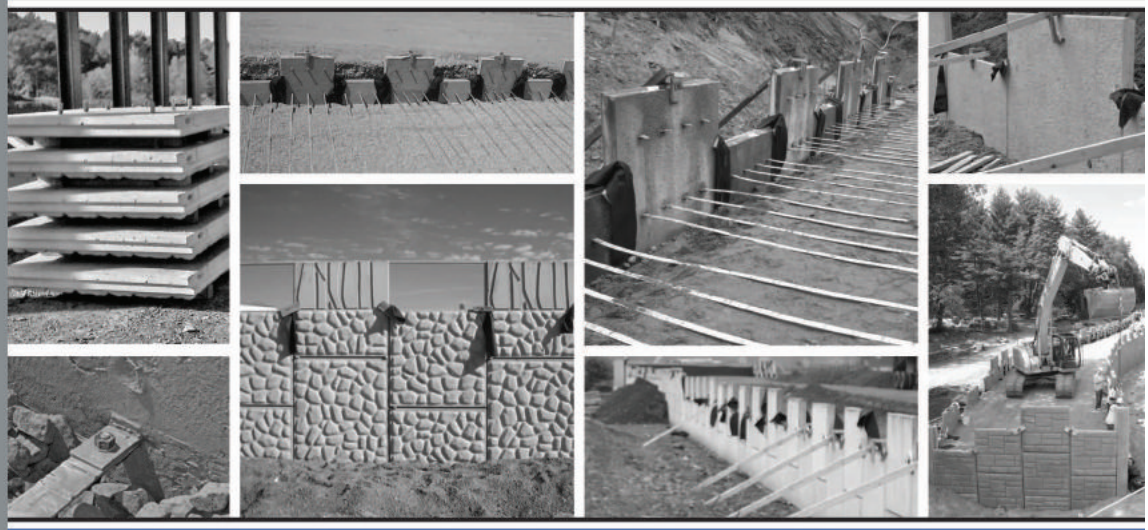
The following 29 pages are from RECo's square panel construction manual. Some blank or unnecessary pages from the beginning and end have been omitted.

See RECo website for additional construction manuals, including Cruciform Panels and Rectangular panels: https://reinforcedearth.com/resources/construction-manuals/

Reinforced Earth Construction Manual: **Square Panels**

REINFORCED EARTH®
SUSTAINABLE TECHNOLOGY

Construction and Quality Control Manual



Reinforced Earth Square Panels

Version v2020.1

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REINFORCED EARTH®

SAFETY BULLETIN

February 25, 2014

Re: Safety Reminder for Reinforced Earth® Panel Installation

Please review the Reinforced Earth® Construction Manual carefully and practice safety in all aspects of handling and installing Reinforced Earth wall materials. The following provides important safety reminders when handling and installing wall panels:

1. Panels at the levelling pad course should be externally braced as shown in the construction manual upon installation and before additional panels are installed.

2. Panels must remain attached to lifting equipment until such time as external bracing, wedges and shoulder clamps are in place and secured.

3. All subsequent panels must remain attached to the lifting equipment until alignment adjustments have been made and wedges and clamps have been installed and secured.

4. Cut panels adjacent to a cast-in-place structure require special attention. Bracing, wedging and clamping is required before releasing the panel from the lifting equipment. Timber or steel angle can be anchored to the vertical surface of the adjacent cast-in-place structure and then the cut panel can be wedged and clamped to the secured timber or angle.

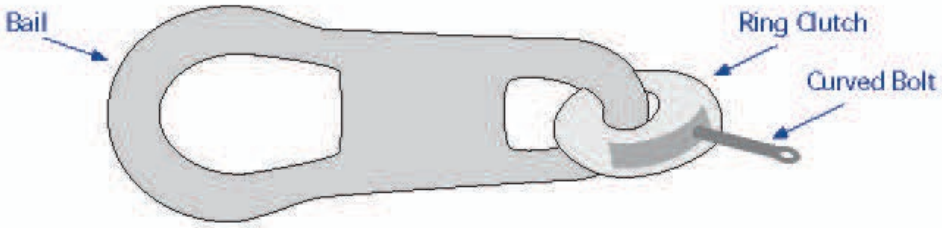
5. The last panel at the free end of wall should be externally braced on the free end as well as secured with wedges and a clamp to the adjacent panel.

Please ensure your crew is aware of the above safety reminders.

Version v2020.1

REINFORCED EARTH®

Proper Positioning of Ring Clutch for lifting Precast Panels



To ensure that the ring clutch properly engages the anchor.
Position the ring clutch so that the bail is **ABOVE** the clutch.
When lifting **DO NOT** pull towards the panel.



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REINFORCED EARTH®

Safety Reminder for Reinforced Earth® Panel Installation

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5 The last panel at the free end of wall should be externally braced and secured with wedges and a clamp to the adjacent panel.

Please ensure your crew is aware of the above safety reminders.

1 2 3 4 5



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CONSTRUCTION AND QUALITY CONTROL MANUAL

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I. Preface

This Construction and Quality Control Procedures Manual has been prepared as a guide in building Reinforced Earth structures.

Its contents should be thoroughly reviewed by the Contractor, the superintendent and the foreman responsible for construction prior to the delivery of Reinforced Earth materials to the job site.

The Reinforced Earth Company will provide construction advisors to assist the Contractor in the implementation of correct construction procedures. However, in the event of any conflict between the Plans, Specifications or Contract Documents and this Manual, the former will prevail. If there is any doubt with regard to any aspect of the Reinforced Earth construction, contact The Reinforced Earth Company before commencing or continuing work.

The Reinforced Earth Company supplies precast concrete facing panels and accessories to be used in conjunction with other materials in the construction of Reinforced Earth retaining walls detailed from the construction drawings. The construction and quality control procedures manual furnished by The Reinforced Earth Company is intended to provide a general explanation of the system. It is the Contractor's obligation to devise and execute a project specific erection sequence, panel unloading, handling and bracing system, and fall protection system. The bracing system shown in the construction and quality control procedures manual is general in nature and does not account for project specific criteria. Compliance with the guidelines in this manual does not relieve the contractor of its responsibility to adhere to the project plans, specifications and contract documents or compliance with all fall protection, safety laws, standards and procedures at the job site. Contractors should take special precautions to prevent the panels from shifting or falling during the erection process.

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II. Non-Disclosure Notice and Disclaimer

1. This Document is the property of The Reinforced Earth Company, with headquarters at 45610 Woodland Road, Suite 200, Sterling, Virginia 20166, and contains information, which is proprietary to The Reinforced Earth Company. This Document is being made available solely for use in connection with your interest or participation in the proposed Reinforced Earth project, and for no other purpose.

2. Except as specified in the paragraph above you must not, without the prior express written consent of The Reinforced Earth Company, use any of the information contained in this Document whether to construct, design, manufacture, fabricate, assemble, produce or install, or otherwise utilize - or cause the same or any of them to be done - any elements of Reinforced Earth (whether or not patented), or special forms of equipment unique to the production, manufacture, fabrication, assembly, installation, or utilization of Reinforced Earth, or for any other purpose.

3. This Document and the information contained herein must not be copied or disclosed in any manner or form, in whole or in part, to any third party without the prior express written consent of The Reinforced Earth Company.

4. Receipt of Document gives no entitlement to any property right in the Document or in the information contained therein by virtue of the temporary supply of the Document in accordance herewith.

5. Reinforced Earth structures designed by The Reinforced Earth Company comprise or are based solely upon:

   (a) The internal stability of the Reinforced Earth structure based upon the design assumptions noted on all drawings provided by the The Reinforced Earth Company relating to the structure and the external loads, surcharges and site geometries supplied by or on behalf of the Owner.

   (b) The layout and geometry of the structure based upon survey details, plans and drawings supplied by or on behalf of the Owner; and

   (c) The Job Specifications.

   The design does not include a check of the overall stability of the foundation soils below or behind the structure, nor a check of any potential failure planes external to the structure, nor a check of the stability of any permanent or temporary slopes above or below the wall or temporary excavations. Based on the completeness and accuracy of the above information used or relied upon in designing the structure, The Reinforced Earth Company warrants the internal stability of the structure only.

6. Upon demand, the Document and all copies thereof must be immediately surrendered and returned to The Reinforced Earth Company.

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III. Introduction

Reinforced Earth® is a composite material formed by the interaction between a frictional soil and reinforcing strips. In concept, it is like reinforced concrete; that is, Reinforced Earth is an economical means of improving the mechanical properties of basic material, earth, by reinforcing it with another, steel.

The reinforcing strips resist stresses produced within the soil mass; stresses are transferred to the strips via friction.

A Reinforced Earth structure constructed using this technique is shown as the "reinforced volume" in Figure 1. Precast Concrete panels are used at the face of the reinforced volume to prevent erosion of the backfill and to provide an attractive, finished appearance.

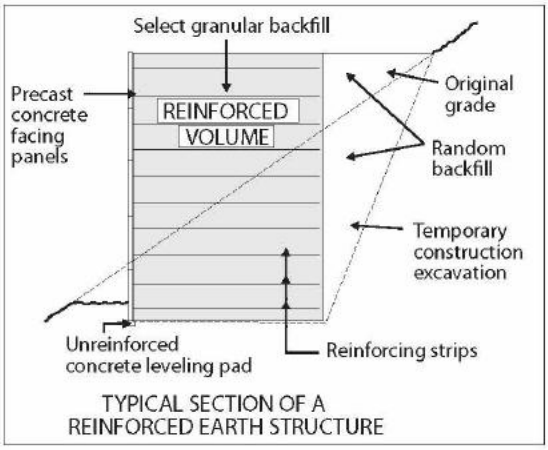


Figure 1

A. Purpose

This document is intended to provide the Owner, Contractor, Engineer and the inspection staff, those who are responsible for overall quality control and inspection during construction with the criteria necessary to monitor the erection of Reinforced Earth structures for compliance with the Plans, Specifications, and Contract Documents.

B. Responsibilities

It is the Contractor's responsibility to complete construction in strict accordance with the Plans, Specifications, and Contract Documents. To assist the Contractor in this regard, The Reinforced Earth Company provides recommended erection procedures in this manual. Nothing in this document is intended to relieve the Contractor of the responsibility of complying with all safety standards and construction procedures, including fall protection, at the job site.

The Contractor and Owner should verify that the Contractor's on-site erection personnel are in possession of and are familiar with the recommendations of this Procedures Manual.

Technical Advisors from the Reinforced Earth Company are available on site during initial construction and thereafter on a request basis. They may assist the Contractor with material scheduling and coordination and provide advice on recommended construction procedures for Reinforced Earth structures as set out in this manual.

Technical Advisors are not available on-site on a full-time basis, and are not provided with the intent of replacing the Owner's and Contractor's designated quality control and/or inspection staff.

Only the Engineer can Enforce the requirements of the Plan, Specifications, and Contract Documents.

C. Plans, Specifications, and Layout

- Prior to commencing any site work, the contractor should verify that the latest issue of the Plans, Specifications, and Contract Documents-approved for construction-are being used to build the retaining wall.
- The Contractor should also confirm that the retaining wall is being constructed at the proper location by verifying elevation, line, grade, offset, and all other location criteria.

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D. Components

Reinforced Earth structures consist of the following:

Concrete Leveling Pad – A cast-in-place or precast unreinforced concrete leveling pad serves as a smooth, level surface for placing panels. Generally this pad is 6-in. thick and 12-in. wide. Refer to the Plans and Specifications for dimensions and requirements.

Precast Concrete Facing Panels

- Full-size – or A_n – panels are used for the majority of the structure. The subscript "n" in panel designations indicates the number of reinforcing strip connections or tie strips on each panel.
- Alternating full and half height panels of varying heights are typically required in the initial course.
- Top-course panels may have a flat top and vary in height to provide a stepped transition at the top of the structure.
- Specially cut, bent, or sloping panels as required by the geometry of the structure.

Facing Panel Joint Materials

- Rubber bearing pads are placed in the horizontal joints throughout the structure to prevent concrete to concrete contact. Rubber shims are used as needed to adjust for minor variations in panel height.
- Filter cloth is applied with adhesive to the backfill side of the panels to cover all the horizontal and vertical panel joints.

Reinforcing Strips

- Ribbed strips are supplied in a 50-mm (2-in.) width and varying lengths as required by the design of the structure.
- High Adherence (HA) ladder strips are supplied in 100-mm (4-in.) width and varying length as required by the design of the structure.

Both types of reinforcing strips are either galvanized for permanent applications or black steel for temporary applications.

Fasteners – During construction, reinforcing strips are fastened to tie strip connections embedded in the back of each facing panel using ½-in. diameter bolts, washers and nuts made of galvanized structural steel.

Select Granular Backfill – Backfill conforming to Contract Specifications **must be used within the reinforced volume.**

E. Materials and Services Provided by The Reinforced Earth Company

- Engineering and design of the Reinforced Earth structure
- Precast concrete facing panels
- Horizontal and vertical joint material and adhesive
- Reinforcing strips
- Bearing pads and shims
- Structural bolts, washer and nuts
- One set of panel lifting devices
- Delivery of The Reinforced Earth Company furnished materials to the site (F.O.B.) with two hours of time allowed for unloading
- Initial on-site technical assistance

Reinforcing strips, bolt sets, filter fabric, adhesive, bearing pads, and other special items provided by The Reinforced Earth Company are bundled and packed to minimize damage in unloading and handling.

Materials should be thoroughly inspected upon delivery to the job site. Any damaged items should be set aside and The Reinforced Earth Company notified immediately. Materials should be handled and stored to prevent damage or theft. Filter fabric must be stored in a sheltered location and protected from sunlight. Adhesive must be stored in dry location and protected from the elements.

Certificates of compliance with project specifications for all materials are furnished by The Reinforced Earth Company. However, it is the Contractor's responsibility to verify that all materials received at the job site are in accordance with shipping documents and project requirements. Any discrepancies should be reported to The Reinforced Earth Company immediately.

To prevent construction delays, the Contractor should continuously monitor the quantity of materials on hand to ensure an adequate supply consistent with the Plans, Specifications, and Contract Documents.

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F. Equipment, Materials, Tools and Work Supplied by Contractor

Materials and Equipment supplied by the Contractor:

- **Panel lifting** – A hydraulic crane, boom truck or similar equipment is required. A standard 5½-in. thick A_n facing panels weighs 1,700 lbs; the heaviest plain finish panel weighs 2,700 lbs. Panels with architectural finish may be heavier.
- **Backfilling** – Dump trucks, scrapers, dozers, graders, front-end loaders, water trucks, etc, are used for hauling dumping and spreading backfill. (Specific equipment selection will depend on backfill, lift thickness, compaction specifications, etc).
- **Compaction** – Large smooth-drum vibratory rollers are used for mass compaction of most backfills. Fine uniform sands are compacted using a smooth-drum static roller.
- Small walk-behind vibrating rollers or flat-plate compactors are needed for compaction within 3 ft. of facing panels.
- ¾-in. Wooden Spacers (Figure 2).
- Wooden wedges in a quantity at least sufficient to provide 4 to 6 wedges per vertical joint for the length of structure under construction (Figure 2).
- Clamps, one per vertical joint for the length of structure under construction. Additional clamps will be needed to brace the initial course of panels (Figure 2).
- Nylon slings for unloading panel stacks.
- Lumber for the initial course of panels (Figures 1)
- 24-in. to 30-in. Crowbars (pinch bars)
- A 4 ft. carpenter level
- Wrenches or socket sets (⅞-in.)
- Claw hammers and 16 penny duplex nails
- A Sledge Hammer
- Chalk line
- Brooms or brushes
- A plumb bob
- Equal length cables with shackles to connect to panel lifting devices
- Large Size Caulking Gun for (2) lb tubes
- Survey Equipment

Summary of Work Performed by Contractor

- Site preparation including shoring, excavation and installation of drainage systems as required.
- Forming and pouring concrete leveling pad(s).
- Mark wall layout line on leveling pad(s).
- Construction of the Reinforced Earth structure consisting of the erection and positioning of facing panels, installation of joint materials, connection of reinforcing strips, placement and compaction of Select Granular Backfill.
- Placement of any concrete coping, traffic barrier, or other C.I.P. Concrete as required.

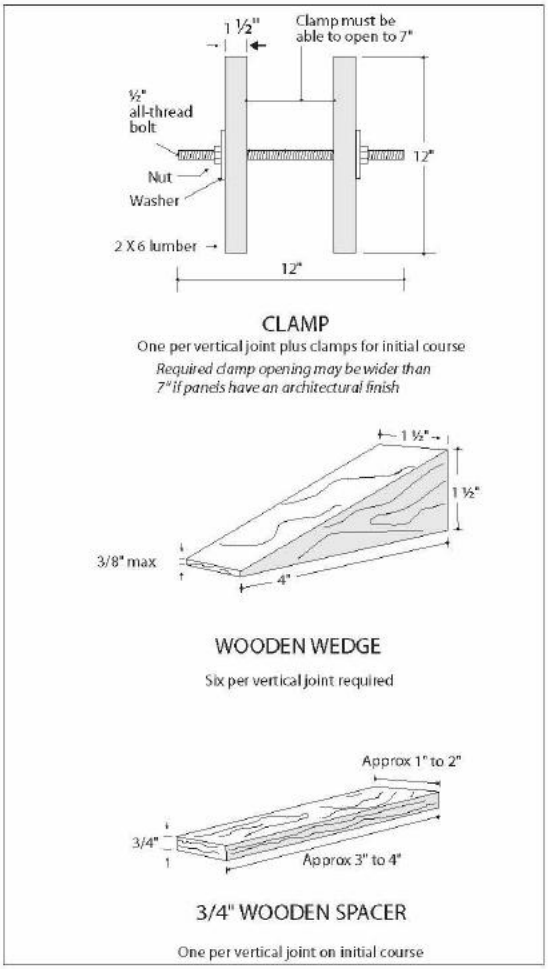


Figure 2

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IV. Handling Reinforced Earth Materials

A. Concrete Facing Panels

Panel Delivery - Prior to the start of construction, the Contractor should establish a panel delivery schedule that will allow The Reinforced Earth Company's precaster to match their panel production and delivery to the Contractor's construction schedule.

Panels are usually delivered on flatbed trailers in stacks of four or five panels high. The delivery point is made as close to the retaining walls as a truck can be driven under its own power.

Panels should be free of any surface defects that may occur in transportation, unloading, or storage at the construction site, including:

- Chipped or broken front corners.
- Permanent stains on exposed face.
- A crack in panel's exposed face.

These panels may be repaired before they are used in the structure.

Any repairs to panels must be completed to the satisfaction of the Engineer.

Unloading Panels - Under normal conditions, a two-hour unloading period is allowed per delivery to unload panels. In this time, panels may be placed directly into the structure being constructed or temporarily stacked following either of these methods:

1. by using lifting devices to lift and handle individual panels (Figure 3A and 3B) OR
2. by using nylon slings to lift and handle individual or stacks of panels (Figure 4)

Care must be taken to protect facing panels from damage during handling and storage.

Panels can be stored at the job site by re-stacking. Select a location with firm, level ground for both stability and to protect panels from staining. Carefully lift and place each panel face down on the nylon pads of the dunnage. Stacks should be no more than five panels high with dunnage used between each panel as illustrated in Figure 5.

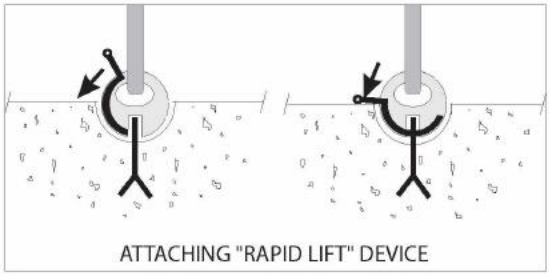


Figure 3A



Figure 3B

Never re-stack panels without dunnage. Never place panels face down directly on the ground. Never stand panels up on end.



Figure 4

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Panel Storage - Panels should be securely set and blocked on firm, level ground to prevent damage and staining during storage.

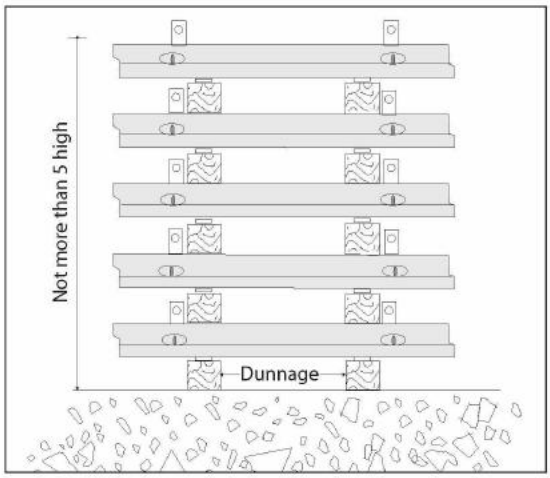


Figure 5

Note: All dunnage and pallets remain the property of The Reinforced Earth Company. They should be stacked and made available for pick-up as soon as they are no longer needed for panel storage.

B. Reinforcing Strips, Fasteners, and Joint Materials

Reinforcing strips - Strips may be up to 32 ft. long and are delivered to the site in bundles of 25 each. Each bundle weighs approximately 30 lbs. per linear ft. Storage in the open is acceptable but bundles should not be placed directly on the ground (Figure 6A).

High Adherence (HA) Ladder Reinforcing Strips - alternate reinforcement type (Figure 6B).



Figure 6A



Figure 6B

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Bearing Pads - Rubber bearing pads are packed in cartons each weighing 50 to 75 lbs (Figure 7).

Fasteners - Sets of 500 bolts, washers and nuts are packed in containers, each weighing 125 lbs (Figure 7A).

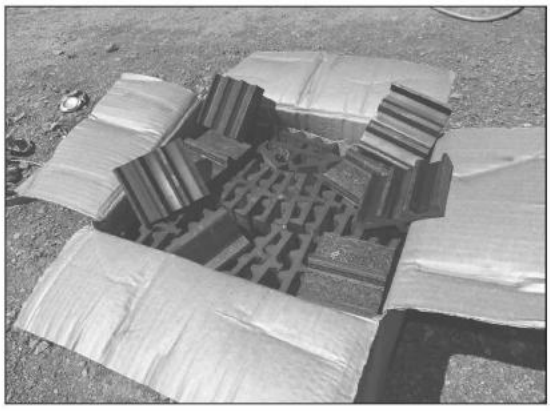


Figure 7



Figure 7A

Filter cloth - Filter cloth is supplied in 12-in or 18-in. wide rolls. Foam strips (if required) are supplied in plastic bags, each containing up to 100 of the 2-in^2, 7 to 9 ft. strips. In addition to normal security, filter cloth and foam must be stored in a sheltered location, protected from the elements (Figure 8).



Figure 8

Adhesive - Adhesive for filter cloth is supplied in appropriate containers. Adhesive tubes must be stored in a dry location and protected from the elements (Figure 9).



Figure 9

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V. Construction Procedures

A. Overview

The basic erection sequence for a Reinforced Earth structure can be summarized in these steps:

- Prepare the site including excavation and installation of drainage systems if required.
- Form and pour leveling pad (see Step B-3 for tolerances).
- Set and brace the initial course of facing panels, which consists of alternating half- and full-height panels.
- Use wood wedges and clamps to hold panels in position.
- Attach filter cloth with adhesive.
- Spread and compact backfill in lifts up to 1 to 2 inches above the lowest level of panel tie strips
- Connect reinforcing strips to panel tie strips.
- Spread and compact backfill in lifts to within 3 to 5 inches of the top of the half panels.
- Place bearing pads and set the second course of full panels
- Repeat cycle of backfilling and compacting in lifts, connecting strips, placing filter cloth and bearing pads and setting panels until design height is reached.
- As each course is completed, remove the wooden wedges from the panels in the course three levels below.
- Set top panels; connect strips, complete backfilling and compaction.
- Remove all wedges and clamps.
- Install concrete coping, traffic barriers, or any C.I.P. concrete as required.

The finished appearance of a Reinforced Earth structure depends to a large extent on the care taken in erecting and positioning facing panels. For this reason, particular attention must be paid to the initial course of facing panels and to backfill placement.

Close attention to detail and accuracy at this point will help ensure trouble free and rapid construction of the remainder of the structure.

The following position, alignment and procedures should be reviewed prior to the start of construction.

<u>Lifting, Placing and Spacing Panels</u> - Panels can be lifted from the horizontal, or stacked position, directly to a vertical position by attaching a lifting device to each of the two cast-in-place lifting inserts at the top edge of the panel (Figure 3). Use dunnage as blocking to prevent damage as each panel rotates from horizontal to vertical (Figure 10).

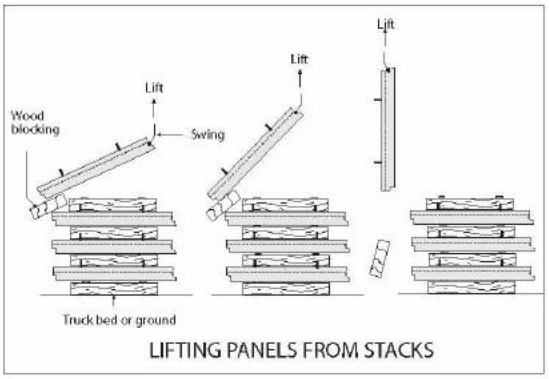


Figure 10

When placing the initial course of panels, insert a spacer to create a joint ¾" wide between each panel at the base of the front face of the wall (Figure 11B). Note: A minimum of one spacer should remain in place until the next panel joint (in the direction of panel placement) has been established by a spacer. Subsequent courses of panels will be placed by centering the panel in its' respective slot (Figure 26).

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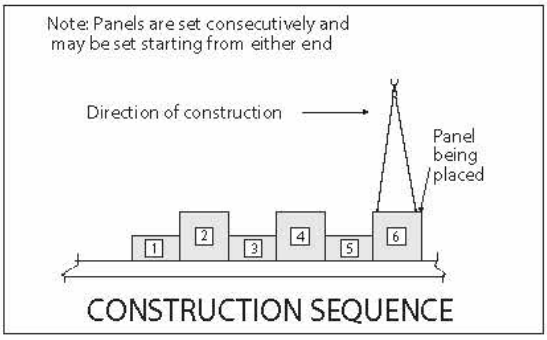


Figure 11A



Figure 11B

Care should be taken to assure that the panel remains centered and properly located until it has been aligned, battered, and clamped.

Check alignment - Visually check the alignment of each panel in relation to either the control line on the leveling pad for the initial course of panels or to the panel below in subsequent courses. Make adjustments with a crowbar on the fill sides of the panel (Figure 12) so that the surfaces of successive panel courses are aligned. Do not attempt to adjust the panel by using the crowbar on the front side – this can result in unacceptable chipping or spalling.

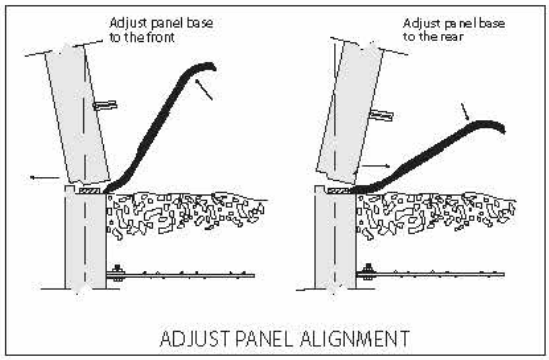


Figure 12

Check horizontal level. The horizontal level of each panel should be checked and adjusted in order to assure a uniform appearance and even joints throughout the structure. Small rubber shims are provided to aid in leveling the panels.

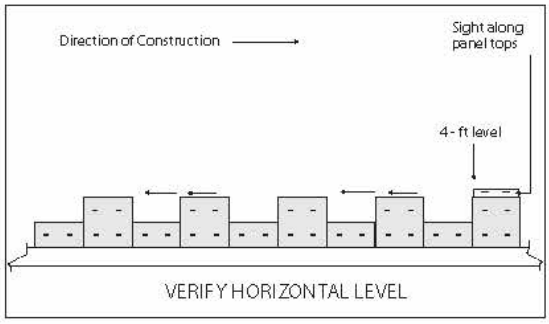


Figure 13

As shown in Figure 13, use a 4 ft. level to verify that the new panel is level. Then sight back along the tops of the panels to ensure that the new panel is at the elevation of the others in its course.

Correct any variations by lifting the panel slightly and inserting rubber shims in the horizontal joint at the panel base. Recheck the horizontal spacing and alignment after any such adjustment.

Set Batter - Panels must be given a slight batter, or tilt, toward the backfill in order to compensate for a subsequent outward movement, which occurs during backfill placement and compaction. This movement will tend to push the panel to a true vertical position. Measure batter using a 4 ft. level (Figure 14).

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Figure 14

Set the batter by adjusting the panel using a crowbar from the fill side. Batter of the initial course can be set with turn buckles. To maintain the batter, drive one wooden wedge from the face of the structure into each of the short horizontal panel joints at the shoulder of the panel and/or the vertical joint and clamp the panel to the adjacent panels (Figures 16,17 and 18).

The horizontal panel joint wooden wedges should remain in place during the erection of three subsequent courses but must be promptly removed. If wedges are left in place for more than three courses, removal will be difficult and spalling can occur. Vertical joint wooden wedges must be removed prior to placement of the next panel course.

The amount of batter varies and depends on the type and moisture content of the backfill, required compaction, type of compaction equipment, and length of the reinforcing strips. A batter of ½-in. in 4 ft. is generally used as a starting point. Coarse backfill, such as crushed stone, may require less batter, while fine backfill, such as sand, may require more.

Monitor the actual movement of panels during the placement and compaction of each lift of backfill; adjust the amount of batter according to field conditions.

Vertical Alignment Check - During construction, check the overall verticality of the structure using a plumb bob. This should be done on panels completely backfilled. Make any changes in batter necessary to assure that final verticality is within tolerances (see specifications), in subsequent lifts of panels.

Erection Tolerances

- The overall vertical alignment tolerance, or plumbness, from top to bottom of the structure, shall not exceed ¾-in., per 10 ft. of height.

Vertical and horizontal alignment tolerance, and plumbness, shall not exceed ¾-in. when measured with a 10 ft. straight edge on a selected wall section. Vertical and horizontal alignment should be checked at every course throughout the erection process.

- The maximum allowable offset between any two panels shall not exceed one inch.
- Horizontal and vertical joints should be uniform in appearance.

Corrective action should be taken immediately when any of the specified tolerances are exceeded.

B. Foundation Preparation

Step B-1: Excavation - Excavate the site to the depth and width specified on the Plans for the length of the section to be built. Remove all unsuitable material and replace it, as necessary, with compacted fill as directed by the Engineer.

If required by the Specifications and as directed by the Engineer, proof-roll the foundation to a density suitable for the bearing pressure shown on the Plans. In the event of an over-excavation of the sub-grade, the Engineer must approve the gradation, placement, and compaction of replacement material.

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Evaluation and approval of foundation suitability is the responsibility of the Engineer. Any foundation soils found to be unsuitable shall be removed and replaced with material approved by the Engineer. The material shall then be compacted, as directed by the Engineer, to a density suitable for the bearing pressure as shown on the Plans, Specifications, and Contract Documents.

Foundation evaluation and control are critical; the behavior and performance of a Reinforced Earth structure is largely dependent upon the foundation on which the reinforced volume is placed.

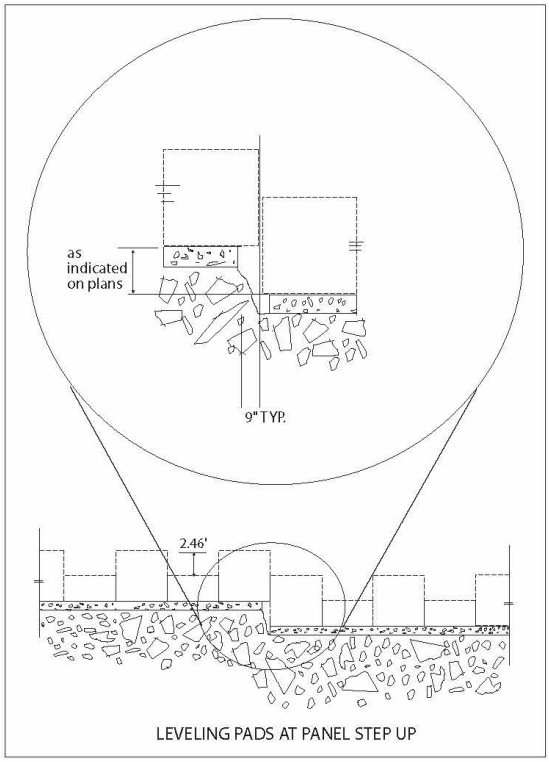
Step B-2: Drainage System(s) - Install any drainage system(s) as required by the Plans and Specifications or as directed by the Engineer.

Step B-3: Leveling Pad - An unreinforced smooth finish concrete leveling pad is formed and cast at each foundation elevation. Leveling pads have nominal dimensions of 6-in. thick by 1 ft. wide and are cast using a minimum 2,000-psi, 28-day compressive strength concrete. Leveling pads should cure for a minimum of 12 hours before the setting of panels.

Leveling pads must be cast to the design elevations as shown on the Plans. The allowable elevation tolerances are +0.01 ft (⅛-in) and –0.02 ft. (¼-in.) at design elevation.

An improperly placed leveling pad can result in subsequent panel misalignment and decreased wall construction productivity.

If the Plans call for the structure to have a step-up in elevation, pour the higher leveling pad as indicated on the Plans. Leave a 12-in. maximum (9-in typical) gap between the higher pad and the centerline of the joint of the last panel of the lower course (Figure 15).



LEVELING PADS AT PANEL STEP UP

Figure 15

Step B-4: Wall Line - On the leveling pad establish a layout line for the front face of the structure. This may be different than the wall's LOL given in the contract documents.

C. Erecting the Initial Panel Course

Step C-1: Panel Placement - Panel layout usually begins at the lowest leveling pad or a fixed point such as a corner or existing structure (Figure 16). Place the first panel [1], this may be a full or half height panel, on the leveling pad. Check the horizontal level and shim as needed. Align the face of the panel along the layout line; and using bracing in addition to wooden wedges at the base of the panel, set the batter for the panel.

Brace the initial course of panels by securing an adequate lumber brace to each full height bottom panel in excess of 3 ft. in height and two adequate lumber braces to each panel in excess of 6 ½ ft. in height (Figures 18 and 19). Bracing is not required on subsequent courses.

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Throughout construction it is of utmost importance that the panel type and number of tie strips of each panel match the requirements as shown on the approved Plans.

Bearing pads are not used under the first course of panels between the leveling pad, unless specifically shown on the Plans or separately authorized in writing by The Reinforced Earth Company. If needed, only rubber shims may be used to shim between the leveling pad and first course of panels. Permanent wood shims are not permitted at any point in the structure.

Panels must be braced as shown in Figures 18 and 19 prior to releasing the crane from the panel. Bracing must remain in place until all braced panels have had reinforcing strips attached and have been backfilled and compacted up to the top of the braced panels.

Step C-2: Second Panel Placement - Place the second panel [2] on the leveling pad and place a spacer to create a joint ¾-in. wide between it and panel [1] (Figure 16). Spacers must be used during panel erection. Spacers are supplied by the Contractor and may be fabricated from any available ¾-in. material.

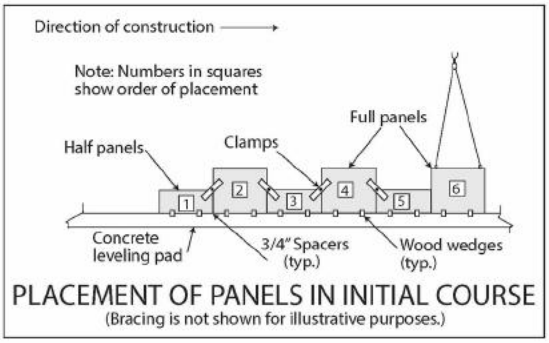


Figure 16

Step C-3: Setting Panel Batter - Set the panel's batter as before. Clamp the panel to panel [1] as shown in Figure 17. Tighten clamps sufficiently to hold the panel in position without movement.

Wooden wedges may be used at the base of the panels to temporarily set batter in the initial course of panels. These wedges must be removed during subsequent construction.



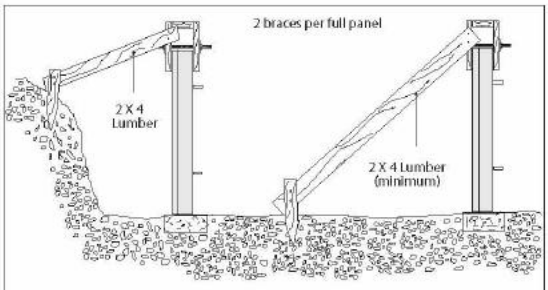
Figure 17



Figure 18

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BRACED INITIAL COURSE OF PANELS

Figure 19

Step C-4: Place a third panel [3], aligning the panel with the control line and use a ¾-in. spacer to ensure spacing. Check the horizontal level of the panel and shim as necessary. Set the panel's batter.

Step C-5: Continue setting the panels in this manner and site back along the tops of the full height panels to assure that each new panel is at the elevation of the others in that course (Figure 13). After ten panels have been set, recheck the wall's alignment by sighting along the wall face. Adjust panels if needed to obtain a true line.

D. Joint Materials

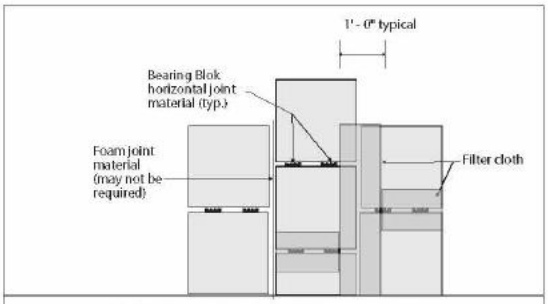
Step D-1: Joint materials are installed from the backfill side of the structure only. Filter cloth prevents the loss of fine backfill particles while allowing the structure to be free draining. Bearing pads prevent concrete-to-concrete contact between facing elements vertically.

- Filter Cloth is affixed to the backfill side of both the vertical and the horizontal panel joints (Figure 20) using several dabs of a contact adhesive.

The adhesive provides a temporary attachment for the filter cloth and should be used sparingly. It is not recommended that the filter cloth be glued solid to the back of the panel. Filter cloth is not generally required at the base of the wall where the panel rests on the leveling pad.

- Bearing pads are placed on top of each panel. Thickness and/or quantity at various levels within the structure may differ and must be in strict accordance with the Plans. Bearing pads should be placed 1/4" clear of back panel lip.

Bearing pads are designed to compress during the construction process. The initial joint created by the bearing pads may decrease in size when the wall is completely constructed.



(Not to scale - joint spacing expanded for illustration)

JOINT MATERIALS

Figure 20

E. Backfilling

The constructibility and performance of a Reinforced Earth structure directly relates to the quality of the Select Granular Backfill and to the manner in which it is installed.

Prior to placing the Select Granular Backfill, the Contractor shall certify to the Engineer that the material conforms to the requirements stated in the Plans, Specifications, and Contract Documents for Reinforced Earth structures.

Select Granular Backfill material to be used in the reinforced volume must be tested and shown to strictly conform to the Specifications. Material, which does not conform, cannot be used as Select Granular Backfill.

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Step E-1: Place and compact initial lifts of Select Granular Backfill up to bottom row of panel tie strips (Figure 21). Note that the uniform loose thickness placement of each lift of backfill material must not exceed 1 ft. The level of the compacted backfill should be 2+ inches above the tie strips as shown in Figure 22. In order to avoid pushing the braced panels out of alignment, initial lifts of backfill are neither placed nor compacted against the back of the panels

Only after the first layer of reinforcing strips has been connected to the panel tie strips as detailed in Section F and a lift of backfill placed and compacted over the strips can backfill then be placed and compacted against the back of the panels illustrated in Figure 23.

Compact each backfill lift using a large smooth-drum vibratory roller except within a 3 ft. zone directly behind the panels where a small hand-operated vibratory compactor must be used to avoid undue panel movement.

After compaction has taken place, check wall alignment visually and with a level adjust panels as necessary.

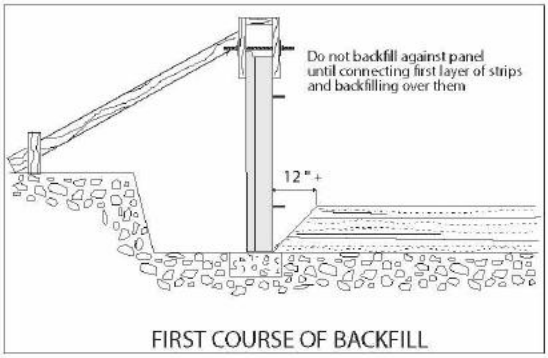


Figure 21

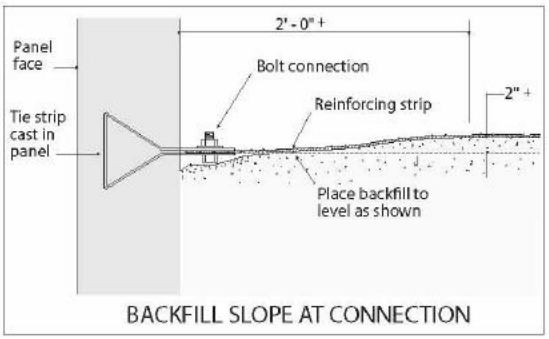


Figure 22

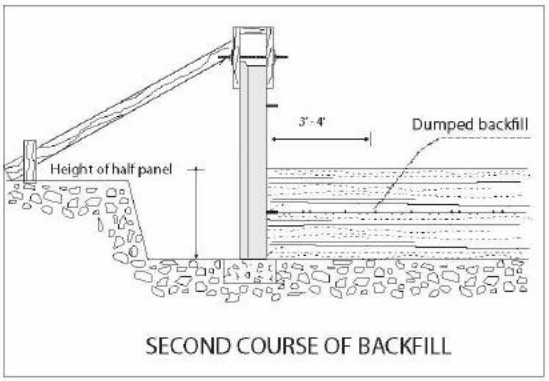


Figure 23

The gradation of the Select Granular Backfill should be tested periodically during construction to assure compliance with the Specifications. This gradation testing should be performed for every 2,000 cubic yards of material placed and/or whenever the appearance or behavior of the material noticeably changes.

Immediate gradation and moisture testing is required if either excessive panel movement or backfill pumping occurs during construction.

Step E-2: Compaction - Large smooth-drum vibratory rollers are used to accomplish mass compaction of Select Granular Backfill materials, except for fine uniform sands.

Sheep foot Rollers are never to be used for compaction of Select Granular Backfill.

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Fine uniform sands, which contain more than 60 percent passing a No. 40 sieve used for Select Granular Backfill, must be compacted using a smooth-drum static roller.

Vibratory compaction equipment should not be used to compact fine uniform sands.

Compaction within 3 feet of the back face of the panels shall be achieved by at least three passes using a lightweight mechanical tamper, roller or vibratory compactor. Compaction testing should not be performed in this 3 foot zone.

At a minimum, Select Granular Backfill material must be compacted to 95% of maximum density, per AASHTO T-99, methods C or D (with oversize correction as outlined in Note 7).

If 30% of the Select granular Backfill material is greater than ¾-in. in size, AASHTO T-99 is not applicable. For such material, the acceptance criterion for compacting is either a minimum of 70% of the Relative Density of the material as determined by ASTM D-4253 and D-4254, or a Method Specification based on a test compaction section which defines the type of equipment, lift thickness, number of passes of the specified equipment and placement moisture content.

Moisture content of Select Granular Backfill material during placement should be approximately 1% to 2% less than its optimum moisture content.

The minimum frequency of compaction testing shall be one test per lift of Select Granular Backfill material placed. Test locations are determined by the Engineer.

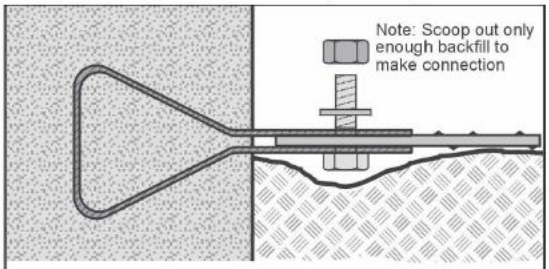
Step E-3: Grading - At the end of each day's work, backfill must be graded to slope away from the back of the panels in order to divert water runoff from the structure area.

Failure to properly grade the backfill can result in excessive water in the Select Granular Backfill and cause subsequent movement of the panels beyond alignment tolerances.

F. Reinforcing Strips

Step F-1: Place reinforcing strips on the compacted backfill. Position strips perpendicular to the facing panels, unless otherwise shown on the Plans. Reinforcing strips are supplied in lengths as shown on plans, with a tolerance in length from 0-in. to + 8-in.

Connect each reinforcing strip to the embedded panel tie strip by inserting the end of the reinforcing strip into the gap between the two exposed ends of the tie strip. Match the three holes and push a bolt through the holes from below, placing a washer on top, threading on a nut (Figure 24), and tightening. Bolts must fit up through both tie strip flanges, perpendicular to the steel surfaces, and have full bearing of the bolt head and washer/nut against the tie strip flanges. Use a crescent or socket-head ratchet to securely hand tighten the nut.



REINFORCING STRIP CONNECTION TO FACING

Figure 24

Reinforcing strips should be placed perpendicular to the back of facing panels. However, in specific, limited situations, it may be necessary to skew a reinforcing strip from its design location (perpendicular to the facing panel) in either the horizontal or vertical plane.

Design of the Reinforced Earth structure is based on the perpendicular placement and connection of reinforcing strips, unless otherwise detailed on the Plans, and on installation of the correct density and length of strips to each panel.

The placement of any reinforcing strip in a skewed manner, unless shown on the Plans, Specifications, and Contract Documents, must be authorized in writing by The Reinforced Earth Company prior to placement in the field.

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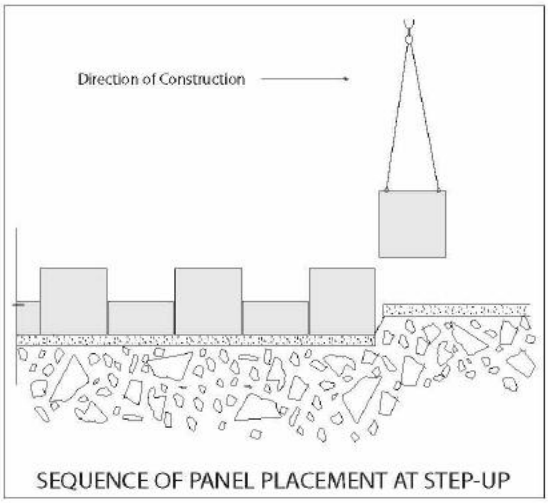
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Step F-2: Dump backfill onto the reinforcing strips so that the toe of the backfill pile is 3-4 ft. from the panels. Spread the backfill by pushing the pile parallel to the panels and wind rowing it toward the panels and toward the free end strips. If strips are long, a second load may be required to backfill to the ends of the strips. If so, dump and spread this load only after spreading the first. Continue to backfill to the full height of the half panels (Figure 23).

Metal tracks of earthmoving equipment must never come in contact with the reinforcing strips. Rubber-tired vehicles, however, can operate directly on the exposed strips if backfill conditions permit and care is exercised.

Step F-3: Step-up: If required to make a step-up in elevation of the wall, use the following procedure.

Mark a wall layout line on the upper level pad to establish a wall-face control line. Place the next required panel along the control line, space ¾-in. set its batter and brace if necessary. Then continue construction of the upper course using the procedure used on the lower level.



SEQUENCE OF PANEL PLACEMENT AT STEP-UP

Figure 25

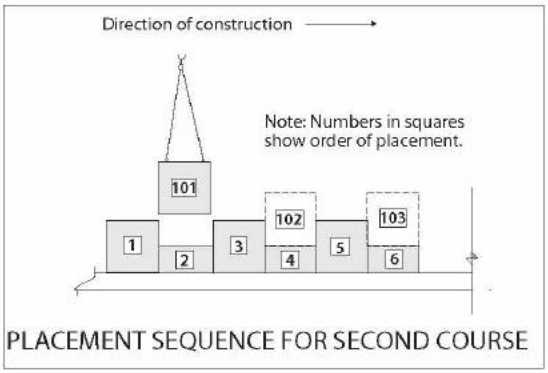
G. Constructing Second and Subsequent Courses

Step G-1: Only after backfill has reached the top of the half panels can construction of the second course begin. Throughout construction, panels should only be set after backfilling and compaction to grade has been completed.

Removing the clamps from a panel or placing any panel atop a panel, which has not been completely backfilled can create an unstable situation, and lead to misalignment of panels, and is strictly prohibited.

Begin the second and subsequent courses of panels at the end of the wall where construction began (Figure 26).

Step G-2: Remove the clamp holding the full height panels [1] and [3] to the half height panel [2]. (As each course proceeds, remove only two clamps at a time to allow for setting of each new panel. Clamps should not be removed prior to complete backfilling of the affected panels.) To prevent concrete-to-concrete contact at horizontal joints, set two bearing pads onto the top edge of panel [2]. Allow for 1/4" clearance between bearing pad and back of panel lip.



PLACEMENT SEQUENCE FOR SECOND COURSE

Figure 26

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Step G-3: Set the full height panel [101] onto the half height panel [2] (Figure 26), centering the panel to ensure equal vertical joints and matching the panel's front face to that of panel [2]. Set the batter of the panel as for the panels of the first course and clamp the full height panel [101] to the initial course panels [1] and [3]. Don't over-tighten the clamps as they may remove the batter set in the panel.

Step G-4: Remove the next pair of clamps; place horizontal joint material onto the half panel [4]; and set the full height panel [102] onto it. Center the panel to assure equal vertical joints. Match the face of the panel just placed to that of the panel below, and set its' batter. Clamp the panel [102] to adjacent panels [3] and [5].

Continue to set full height panels in the same sequence. As work proceeds, check the wall's alignment frequently. Install joint material as in Step D-1.

Step G-5: When the course of panels is complete and after vertical and horizontal filter cloth has been installed, place backfill in lifts according to the Plans, Specification, and Contract Documents up to 2-in. above the tie strip level. Figure 27 illustrates the sequence for backfilling the second and subsequent courses.

Step G-6: After backfilling, recheck the batter and alignment of the wall, then place and connect the next layer of reinforcing strips as in step F-1.

Step G-7: Backfill up to the top of the full height panels of the initial course and the bracing can now be removed. If after backfilling to the top of the full height panels, panels of the initial course did not become almost vertical, or if panels have gone beyond the vertical, adjust the amount of batter in the second course so that the third row will become vertical after backfilling.

Placement and compaction of part or all of a berm or an embedment at the lower front surface of the structure can begin (Figure 28). Removal of the spacers at the base of the wall must be complete prior to placing this material.

Reinforced Earth structures with a leveling pad at or above existing grade requires placement of an earth berm as toe protection along the face. Structures with a leveling pad below existing grade require the placement of an embedment in front of the wall. The height of the berm or the depth of the embedment is shown on the Plans.

The berm or embedment must be placed immediately to prevent erosion before the wall reaches 50% of its height or 20 ft., whichever is less.

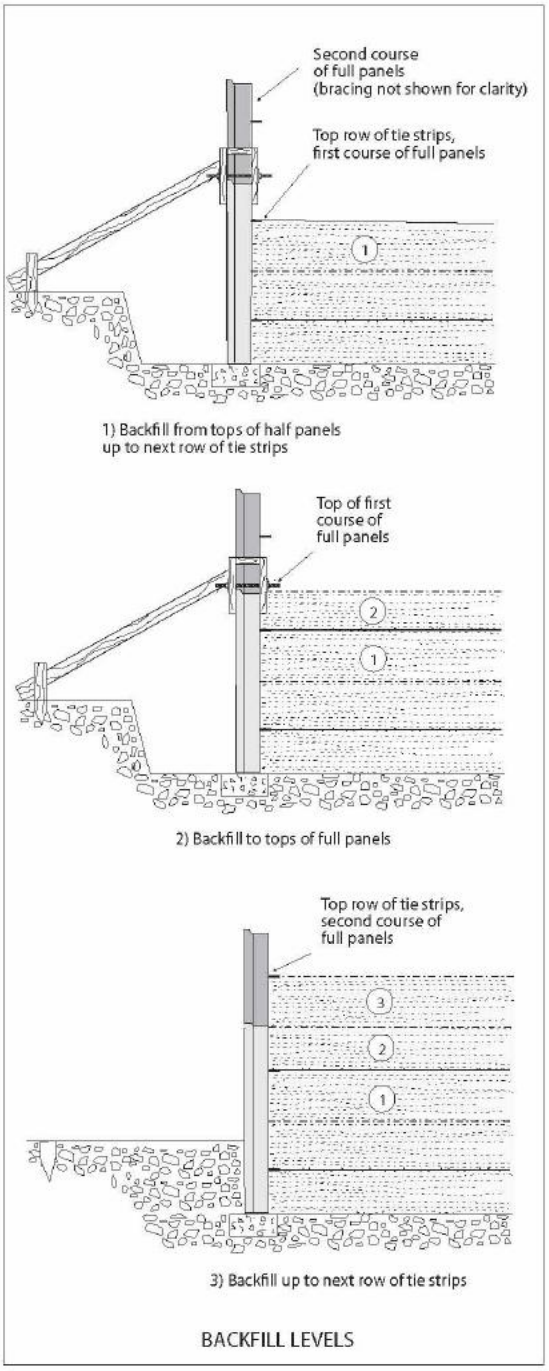


Figure 27

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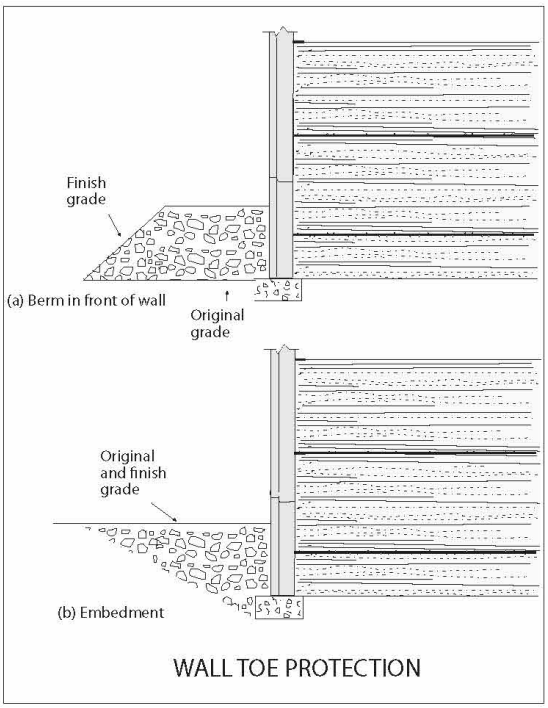


Figure 28

Step G-8: When backfill reaches the top of the initial course of full panels (which is halfway up the second course), begin placing the next course of panels.

Repeat steps G-1 through G-5 for panel installation, backfill placement and compaction, reinforcing strip installation, and placement of horizontal/vertical joint materials. Follow these same procedures for the second and subsequent panel courses until the structure is ready to be topped off. Bracing is not required on subsequent courses.

After the erection of each course of panels, the wooden wedges of that course and the two courses below it should be checked to ensure that they are securely seated.

Panel batter should be checked after backfilling each course with necessary adjustments made in subsequent courses to ensure plumbness.

Quality control requirements for Select Granular Backfilling, including density and placement moisture, are the same for the second and subsequent courses as for the first course, unless otherwise indicated in the Plans or Specifications.

At least two, but no more than three rows of panel wedges should remain in place at all times during erection. When construction is complete, all wedges must be removed.

H. Completion of the Wall

Step H-1: In placing top course panels, the construction sequence continues as previously outlined. However, top course panels have either a flat or a sloping edge and may be supplied in varying heights to meet finished-elevation requirements. Refer to the Plans for the location of specific top panels.

Step H-2: After backfilling is complete, remove all clamps and wooden wedges from the structure.

Step H-3: Install top wall treatment. If required. Several types are commonly used:

- **Cast-in-Place** - If required, rebar for connection will protrude from top panels. All necessary attachment details for a barrier, coping, parapet, or paved ditch will be shown in the plans.
- **Precast Coping** - If approved, precast coping will be supplied. Attachment details will be shown in the Plans.
- **Plain** - The top of panels will remain exposed with no further treatment necessary. Sometimes called stepped top.

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VI. Condition-Cause Relationships

Reinforced Earth structures are to be erected in strict compliance with the structural and aesthetic requirements of the Plans, Specifications, and Contract Documents. The desired results can be achieved through the use of quality materials, correct construction procedures, and proper inspection. However, considering the nature of construction work, there may be occasions when dimensional tolerances and/or aesthetic limits are exceeded. Corrective measures must be taken immediately to return the structure to acceptable tolerances.

CONDITION	CAUSE
1. Distortion in Wall (a) Differential settlement or low spot in wall (b) Overall wall leaning (c) Panel contact, resulting in spalling/chipping	Foundation (subgrade) material too weak or wet for proper bearing. If fill material, poor quality or improper compaction.
2. (a) First course difficult to set and/or maintain level. (b) Panel-to panel contact resulting in spalling and/or chipping.	Leveling pad not within tolerance.
3. Wall Leaning Out	a) Panels not battered sufficiently. b) Large backfill placing and/or compaction equipment working within 3-ft. zone of back of wall. c) Backfill material placed wet of optimum moisture content. d) Backfill contains excessive fine materials (beyond the Specifications for percent of materials passing a No.200 sieve). e) Backfill material pushed against back of wall before being compacted on strips. f) Excessive or vibratory compaction on uniform fine sand (more than 60 percent passing a No.40 sieve). g) Backfill material dumped close to free end of reinforcing strips, then spread towards back of wall, causing bulge in strips and pushing panel out. h) Wedges not seated securely. i) Clamps not tight. j) Excessive compactive effort. k) Excessive lift thickness. l) Plasticity index of backfill material in excess of specification limits.
4. Wall Leaning In	a) Excessive batter set in panels for Select Granular Backfill material being used. b) Inadequate compaction of backfill.
5. Wall Out of Horizontal Alignment Tolerance, or Bulging	(a) See causes 3C-G, and 3l. (b) Backfill saturated (heavy rain or improper grading of backfill after each day's operation).
6. Panels Do Not Fit Properly in Intended Locations Resulting In Subsequent Panels Spalling or Chipping	(a) Panels are not level. (b) Differential settlement (see Condition 1). (c) Failure to use spacers between panels. (d) Leveling pad incorrect.

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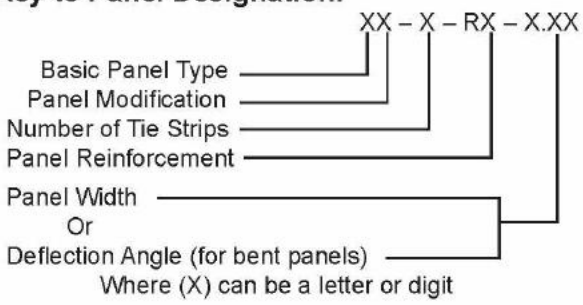
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VII. Appendix

A. Facing Panel Types and Nomenclature

A. Panel Designations: All design and/or shop drawings, and precast panels, are designed as follows:

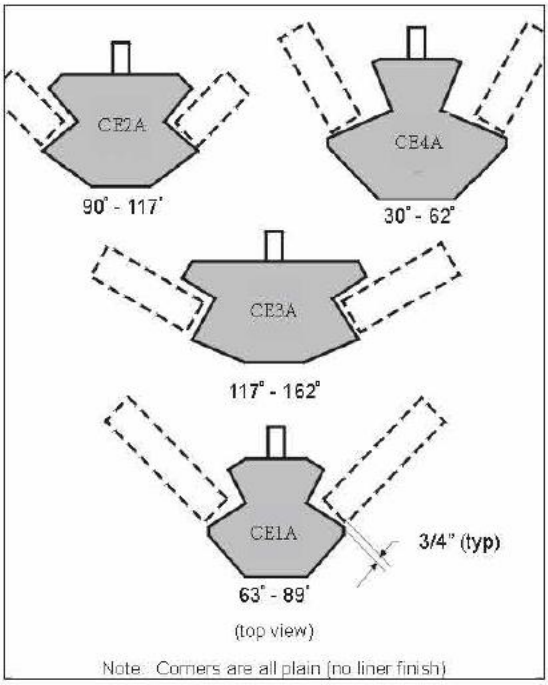
Key to Panel Designation:



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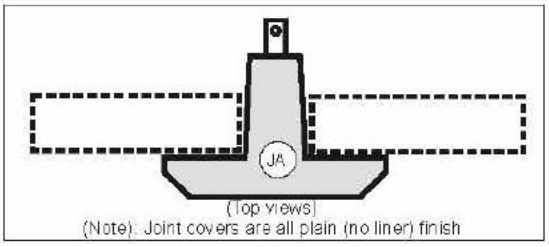
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B. Corner Elements: Corner elements are designed to accept vertical cut panels and cover a range of bends. The corner element designations are CE1A, CE2A, CE3A, and CE4A. The letter following indicates corner element height. Corner element details and shape may vary. Refer to the Plans and Specifications.



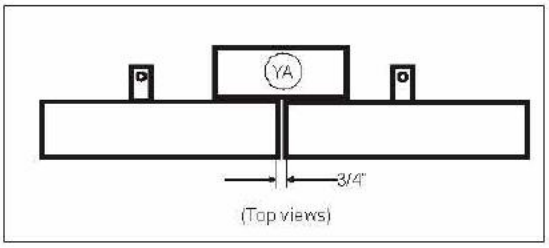
Note: Corners are all plain (no liner finish)

C. Slip Joint Cover: Slip joints are designed to accept vertical cut panels and provide system flexibility and rapid changes in elevation. The cover designation is the letter J. The following letter indicates the joint height.



(Note): Joint covers are all plain (no liner) finish

D. Blind Slip Joint: Again, the slip joint is designed to accept vertical cut panels and provide system flexibility for varying foundation conditions. The cut panels are assembled with a ¾-in. butt joint. The letter Y designates the back-up panel and the following letter indicates the panel height.



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B. Glossary of Terms

Agency	The person(s), firm, or corporation acting as Agent for the Owner.
Contract Documents	The Owner-Contract agreement, including the conditions of the Contract (general, supplementary, and other conditions), the drawings, Specifications and the provisions of the agreement between the Contractor and The Reinforced Earth Company; and also including all addenda issued prior to execution of the Contract, all modifications thereto and any other items specifically stipulated as being included in the Contract Documents.
Contractor	The individual, firm or corporation undertaking the execution of the Work under the terms of the Contract, and acting directly through its Agents or employees.
Engineer	The person(s) designated by the Owner, as having authoritative charge over certain specific engineering operations and duties.
Inspector	The authorized representative assigned to make a detailed inspection of any or all portions of the Work or materials thereof in the Owner's behalf.
Owner	The Owner of a project. The agency, person, firm or corporation with which a Contract has been made for the payment of the Work performed under the Contract.
Plans	The official approved plans, profiles, typical cross sections, working drawings and supplemental drawings, or exact reproductions thereof, which show the locations, character, dimensions and details of the Work to be performed.
Specifications	A description, for contract purposes, of the materials and workmanship required in a structure(s), as also shown on the related working drawings. The written material containing the standard provisions and special provisions, as may be necessary, pertaining to the quantities and qualities of materials to be furnished under the Contract.
Technical Advisor	Representative of The Reinforced Earth Company who may be available to assist the Contractor with material scheduling and coordination, and give advice on the recommended construction procedures applicable to Reinforced Earth structures as set out in this manual.
Work	The entire scope of the Work to be performed at the site of the construction project including labor, materials, equipment, transportation and such other facilities as are necessary to fulfill all obligations under the Contract.

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C. Safety Tips for Unloading Reinforced Earth Products

1. Upon arrival of truck, examine the load for any shifting or unstable conditions prior to removing tie downs.

2. The truck should be on level ground when unloading. Unloading on unlevel ground could result in shifting of panels or possibly panels falling from trailer.

3. Lifting equipment (straps, cables, ring clutches, etc.) should be checked for excessive wear or cracking prior to unloading truck.

4. Do not move the tractor while product is not tied down.

5. If drivers are required to remove chains or binders next to lane of moving traffic, cones and flagman should be used to direct traffic away from the trailer and driver.

6. Drivers are not trained as riggers or swampers and should stay in cab or clear away from unloading operations. The drivers are acting in a delivery capacity only.

7. Personal protective equipment required by the general contractor on site should also be required of delivery drivers.

8. Personnel should not be allowed under a suspended load.

9. Once removed from the trailer, panels not placed directly in the wall should be stacked or secured on flat ground to prevent tipping or falling.

10. Areas between the truck and crane should be restricted to personnel required to unload trailer.

11. If any unsafe situations exist while loading or unloading RECo products, contact The Reinforced Earth Company immediately to eliminate any hazards or exposure to illness or injury.

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A3.1.3 Construction Limitations for Curves and Corners

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A3.1.4 Construction Procedures for Curves and Corners

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A3.1.5 Vertical and Horizontal Alignment

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A3.1.6 Reinforced Soil Installation

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A3.1.7 Erosion Prevention During Construction

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A3.1.8 Required Experience and Qualifications for Wall Installer

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A4.0 Quality Control

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A4.1 Manufacturing

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A4.1.1 Facing Unit Quality Control

The following 27 pages are from RECo's Precast Concrete MSE Wall Panels and Coping Units Example Quality Control Manual.

Precast Concrete MSE Wall Panels and Coping Units

EXAMPLE QUALITY CONTROL MANUAL

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Preface

This Precast MSE Wall Panel Manufacturing and Quality Control Manual has been prepared as a guide in fabricating Reinforced Earth precast MSE (Mechanically Stabilized Earth) panels. Each manufacturing facility shall have its own Quality Control / Quality Assurance Plan in accordance with local standards. RECo recommends that precast plants be certified by NPCA in addition to state DOT requirements.

Its contents should be thoroughly reviewed by the Precaster prior to commencing the fabrication of Reinforced Earth precast panels.

The Reinforced Earth Company will provide fabrication advisors to assist the Precaster in the implementation of correct fabrication procedures. However, in the event of any conflict between the Plans, Specifications, or Contract Documents and this Manual, the former will prevail. If there is any doubt with regards to any aspect of the Reinforced Earth panel fabrication, contact The Reinforced Earth Company before commencing or continuing work.

The Reinforced Earth Company supplies precast forms, form liners (if applicable) and tie-strips to be used in conjunction with other materials in the fabrication of Reinforced Earth precast panels detailed within the precast construction drawings.

This Precast MSE Wall Panel Manufacturing and Quality Control Manual, furnished by The Reinforced Earth Company, is intended to provide general direction as to the fabrication of precast MSE wall panels. It is the Precaster's obligation to devise and execute a safe and responsible (project approved as applicable) quality control plan. Compliance with the guidelines in this manual does not relieve the Precaster of its responsibility to adhere to the project plans, specifications and contract documents or compliance with all fall protection, safety laws, standards and procedures at the job site. Precasters shall take all necessary special precautions to prevent improper fabricating processes while ensuring safe handling of the precast panels during the fabrication, storage and loading processes.

This manual incorporates proven standards of practice that have been shown to achieve the desired level of quality. The requirements of this manual are not intended to be applied in a manner that is restrictive to the development of progressive techniques or innovation. As new materials and processes are developed, their application should be considered within the scope and intent of these standards.

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1.1 PLANT QUALITY CONTROL PROCEDURES AND MANAGEMENT POLICIES

1.1.1 This Precast MSE Wall Panel and Coping Unit Manufacturing and Quality Control Manual, furnished by The Reinforced Earth Company, is intended to provide general direction as to the fabrication of precast MSE wall panels. It is the Precaster's obligation to devise and execute a safe and responsible (project approved as applicable) quality control plan. Compliance with the guidelines in this manual does not relieve the Precaster of its responsibility to adhere to the project plans, specifications and contract documents or compliance with all fall protection, safety laws, standards and procedures at the job site. Precasters shall take all necessary special precautions to prevent improper fabricating processes while ensuring safe handling of the precast panels during the fabrication and loading processes.

A quality control program is tasked with preventing defects rather than only the detection and correction of quality issues, thereby assuring the customer and Owner of a high quality product is being manufactured and supplied at the best possible price. Through this system, The Reinforced Earth will continue to maintain its reputation for engineering excellence, architectural creativity, and an unyielding focus on quality, safety and customer support.

1.1.2 Plant Requirements:

1. Maintain a current copy of the NPCA manual and a copy of the applicable Departments of Transportation Standard Specifications that is readily accessible to inspectors and plant personnel.
2. Develop and maintain a current Quality Control Plan with a formal written policy manual.
3. Maintain current copies of applicable ASTM Test Methods and Specifications.
4. Maintain files of project specifications and requirements.
5. Maintain in company files, records of employee responsibilities, certifications and training.
6. Maintain deficiency logs of all quality assurance activities. These are to be tracked and discussed on a regular basis as to their cause and remedy to serve as a learning tool for future plant activities.
7. Designate and train a plant QC inspector for each work shift. The QC inspector shall report to plant management and not directly to production personnel. A designated QC inspector shall be present any time the plant is producing products.

Plant QC inspectors shall at a minimum hold a certificate from the NPCA Production and Quality School Level 1 certification course and the ACI Concrete Field-Testing Technician Grade I certification.

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8. Management or a designated representative shall hold QC meetings with QC and plant personnel at a minimum of once every 6 months. A record of the minutes of these meetings and a list of attendees shall be kept in plant files.

1.2 PLANT SAFETY

1.2.1 The plant shall have an active safety program. The program shall include requirements of local, state, and federal laws, as well as the Occupational Safety and Health Administration (OSHA).

1.2.2 Maintain a plant safety manual and documented safety program.

1. Management or designated personnel shall hold safety meetings with plant personnel at a minimum of once per week. A record of the minutes of these meetings and a list of attendees shall be kept in the plant files.
2. A list of all plant equipment and equipment inspections shall be kept in the plant files. (Exhibit 9)

CHAPTER 2- MATERIALS

2.1 CONCRETE

The concrete mix design shall be pre-approved through the appropriate State Materials Division and where applicable any project channels. The cement shall conform to the requirements defined in AASHTO M85. Air entrainment, admixtures and other agents shall be clearly referenced in the approved mix design. The minimum 28-day strength shall be in accordance with the State and project specifications and shall not be less than 4,000 psi.

2.2 REINFORCEMENT

2.2.1 Reinforcing bars

1. Steel reinforcing bars shall conform to the specifications required per project specific design.
2. Reinforcing steel use for production purposes shall conform to ASTM A615 & SI, ASTM A 996, and ASTM 796. Other bars may be used for specific purposes if permitted by the design. Reinforcing bar suppliers shall be chosen from approved DOT Material Producers Lists and are checked for conformance by that agency. Reinforcing bars are tagged with heat numbers and ASTM. Mill certifications will be accepted and kept on file. All steel shall conform to Buy American standards.
3. Epoxy coated steel
    a. Handling – Epoxy coated reinforcement steel will be handled in a manner that will not damage or remove the epoxy coating. Straps will be used when handling

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epoxy coated reinforcement. Chains or cables will not be allowed. Epoxy coated steel will be stored on pallets and covered with a tarp.

b. Epoxy coated reinforcement will be placed into forms in a manner to prevent damage during placement. The epoxy coated reinforcement will not be dragged against form edges.
c. Damaged (cuts, cracks, and abrasions) epoxy coated reinforcement will be repaired using a two-part epoxy material in accordance with ASTM D 3963. Manufacturer's directions will be followed.

2.2.2 Bar mats and welded wire fabric

1. Steel bar mats and welded wire fabric shall conform to the specifications required in the design:
    a. Deformed steel bar mats must meet ASTM A184.
    b. Plain welded wire fabric must meet ASTM A185.
    c. Deformed welded wire fabric must meet ASTM A497.
    d. Suppliers of bar mats and welded wire fabric shall furnish mill certificates with each shipment and will be kept on file in the plant. No welded reinforcing steel mats shall be fabricated by RECo or its employees.

2.2.3 Plant Requirements

1. Mill certificates and certificates of conformance shall be maintained current for all reinforcement including reinforcing bars, reinforcing wire, bar mats, welded wire fabric, and epoxy coated reinforcement.

2. The QC Manager will verify that certificates are on file for all reinforcing heat numbers being used or stored. These records will be kept for a minimum of 3 years.

2.2.4 Reinforcing steel Storage

1. Reinforcing steel will be stored in accordance with all relevant DOT specifications and NPCA industry standards. Reinforcing steel will be placed on wood dunnage or concrete panels and not in direct contact with the ground surface.

2.3 MISCELLANEOUS MATERIALS

2.3.1 Lifting devices and lifting apparatus

Lifting devices used in precast concrete products shall be verified for capacity and have an adequate factor of safety for lifting and handling products. Various forces acting on the device

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including form release suction, impact, and positions of the product during handling should be considered.

2.3.2 Embedded steel shapes and plates

Steel shapes and plates that are to be embedded in precast concrete shall at a minimum conform to the requirements of ASTM A36. Other types of steel shapes and plates may be used if the requirements are specified in the design. All metallic embeds requiring corrosion protection shall be galvanized in accordance with ASTM A123 - Standard Specification for Zinc (Hot-Dipped Galvanized) Coatings on Iron and Steel Products.

2.3.3 Manufacturing accessories

Spacers for reinforcement, inserts, form ties, and similar accessories incidental to the manufacturing for precast concrete products shall be adequate for their intended purposes and shall result in minimum marring to the concrete surfaces. These are made of plastic or other non-corrosive material.

2.3.4 Plant Requirements

1. Commercial lifting devices shall be certified and posted for maximum capacity all lifting apparatuses shall be inspected annually and records kept in plant files.
2. Non-commercial lifting devices shall be load tested to determine ultimate load and a safety factor of 5 shall be used. Maintain test records for a minimum of 3 years.

CHAPTER 3- CONCRETE

3.1 CONCRETE MIXES

The concrete mix design shall be pre-approved through the appropriate State Materials Division and where applicable any project channels. The cement shall conform to the requirements defined in AASHTO M85. Air entrainment, admixtures and other agents shall be clearly referenced in the approved mix design. The minimum 28-day strength shall be in accordance with the State and project specifications and shall not be less than 4,000 psi.

AASHTO Minimum Class A(AE) Mix requirements are shown below for reference:

Per AASHTO LRFD Bridge Construction Specifications, Table 8.2.2-1:

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Class of Concrete	Minimum Cement Content	Maximum Water/Cementitious Material Ratio	Air Content Range	Size of Course Aggregate Per AASHTO M43	Size Number per AASHTO M43	Specified Compressive Strength at 28 days
	lb /yd^3	lb per lb	%	Nominal Size		Psi
A	611	0.49	-	1.0 in. to No. 4	57	4,000
A(AE)	611	0.45	6 +/- 1.5	1.0 in. to No. 4	57	4,000

Table 3.1: AASHTO Mix Design Requirements

Additional recommendations for Facing Unit Mix Design can be found at the National Precast Concrete Association (NPCA): https://precast.org/wp-content/uploads/2014/09/Retaining-Wall-Best-Practices-Manual.pdf

3.1.1 Ready-mixed concrete

Ready-mixed concrete shall be purchased from a NRMCA (National Ready Mix Concrete Association) certified plant and meet all State and project specifications respectively. The requirements for the use of truck-mixers include:

1. Materials for approved sources and mix designs will be proportioned at the ready-mix plant and deposited into the transit truck-mixer.
2. The number of revolutions of the drum or blades at mixing speed shall be no less than 70 nor more than 100, and not less than that recommended by the mixer manufacturer.
3. The truck-mixer shall be equipped with means by which the number of revolutions of the drum may be readily verified. The counters shall be actuated at the time of starting the mix at mixing speed.
4. Mixing operations shall begin within 30 minutes after the cement has been added to the aggregates.
5. Wash water shall not be used as a portion of the mixing water for succeeding batches.

The truck-mixer should not mix at the mixing speed beyond the required numbers of revolutions. Extended mixing at this speed will result in concrete strength loss, temperature rise, excessive loss of entrained air, and accelerated slump loss. Truck delivery receipts shall be

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received with each load and reviewed for proportioning compliance. Water added at the plant to the ready-mixed concrete deliveries shall be recorded along with any admixtures added by plant personnel.

3.1.2 Concrete testing

1. All concrete trucks will stop at the quality control testing area for inspection by an ACI concrete field certified inspector before moving to the production lines.
2. Delivery tickets will be reviewed to verify mix design proportions and water/cement ratio compliance.
3. A slump test will be performed to determine initial slump on the first truck of the day verify compliance of a 2-3-inch slump. The second truck will be checked to determine consistency. Tests for initial slump with be checked throughout the day as determined by ACI field testing technician certified QC personnel or whenever there is a question of compliance or accuracy from production supervisors or crew leader.
4. Water reducing admixtures listed on the approved mix design will then be added to the concrete truck to achieve the desired slump for casting operations. A slump test will be performed on the mix again to verify slump requirements.
5. A slump test will be performed to determine initial slump compliance of a 2-3-inches and for slump requirements after water reducing admixtures are added each set of cylinders made. Slump tests will be performed in accordance with ASTM C143, "Test Method for Slump of Hydraulic Cement Concrete".
6. The temperature of fresh concrete shall be measured at the beginning of each pour to determine compliance, when slump or air content tests are made and when compressive test cylinders are made. A thermometer accurate to +/- 1-degree F shall be inserted into the fresh concrete for at least two minutes or until an accurate reading can be obtained and shall be recorded with other test data. Place concrete only when its temperature at the time of placement is between 50 and 95 degrees F. Refer to sections 4.4.6 and 4.4.7 for hot and cold weather precautions.
7. Tests for air content shall be made at the start of each day's production and for each set of cylinders made. Air content will be determined by the pressure method and in accordance with ASTM C231. Air content will be reported to the Reinforced Earth's QC manager and to the Ready-mix plant if readings do not meet specifications. Tests for air content will be run each subsequent load of concrete until compliance is verified.
8. Compressive strength cylinders will be made in accordance with ASTM C31 using 4X8 or 6x8 inch cylinders, and cured in a properly constructed wooden box with plastic covering over the cylinders to prevent moisture loss, or in the cure tank between 60-80 degrees F. A minimum of 8 cylinders will be made for each mix design per day for testing purposes. All tests are to be in accordance with ASTM C39.

3.1.3 Plant requirements

1. Concrete shall be supplied through a DOT pre-approved source either through self-batching or appropriate Ready-Mix organization.

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Other, back-up ready-mix suppliers should be identified for contingency purposes.

2. Concrete should only be accepted for DOT casting operations if a qualified Quality Control inspector is present to perform required concrete tests.
3. Test for initial slump, air content, and concrete temperature will be performed on the first load of concrete, whenever concrete strength specimens are made, and whenever there is a question of quality or slump requirements. Signs displaying individual DOT concrete testing requirements shall be posted at the site of testing.
4. All equipment for weighing or proportioning admixture must have current calibration stickers posted and visible for inspection.
5. Calibrated amounts of water-reducing admixtures will be added to the mixer truck and mixed for the minimum required amount of revolutions.
6. Tests for slump requirements and air content will be performed again to verify concrete mixture requirements.
7. Batch tickets will be collected from the ready-mix truck driver and reviewed to ensure that the water/cement ratio is correct and accurate.

CHAPTER 4 – PRODUCTION PRACTICES

4.1 GENERAL

4.1.1 Plant Layout

The physical layout of the plant shall be such that production, handling, storage, and shipment of products is done in an efficient, safe manner and with minimal product damage.

The plant layout incorporates the following general guidelines:

1. Minimize transport distances of fresh concrete.
2. Adequate workspace to minimize safety and tripping hazards.
3. Avoid stripping or lifting products over personnel or equipment.
4. Avoid having work personnel in elevated position while minimizing excessive reach or bending positions.
5. Prevent marking or splash on other products during casting operation.
6. Adequate storage space for materials.
7. Adequate spaces to strip products and perform post pour inspections and repairs.

4.1.2 Housekeeping

The plant shall have an active housekeeping program. The purpose of the program shall be to provide clean and safe environment to that quality precast concrete products can be manufactured efficiently. QC inspectors shall spot check housekeeping practices daily.

4.1.3 Forms

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Forms for manufacturing precast products shall be of a quality that prevents product damage due to forces and vibrations subjected to the forms. Forms shall be measured prior to initial use and not less than annually for dimensional conformance. A WC inspector will check all forms prior to each day's production to ensure that all forms conform to the approved drawings.

Forms that are found to need repair are taken from production. Repairs will be made in a timely manner before returning to production.

4.1.4 Handling Equipment

Equipment such as hoists, overhead cranes, gantries, mobile cranes, fork-lift trucks, shall be used to lift and handle products which weigh less than the rated capacity of the equipment.

Inspection and maintenance records for all handling equipment must be maintained and documented in accordance with applicable requirements.

4.1.5 Precast concrete Panels with Architectural / Form Liner Finish

The level of quality in terms of appearance is of the greatest importance. Final product shall match previously approved samples with a consistent aesthetic appeal and or established industry standards stated in job specifications.

As per ACI301-10 Specifications for Structural Concrete, 6.3.13 "Final acceptance of architectural concrete – Upon completion of architectural concrete, final acceptance is based upon matching the architectural cast-in-place concrete with the accepted field mockup when viewed at 20 ft in daylight. Defective Work, including repair areas not accepted, shall be removed and replaced".

4.1.6 Plant requirements

1. Maintain an active housekeeping plan, Efforts will be made by all production personnel to maintain a clean work area.
2. Maintain records of form dimensional checks for each new form and each form annually.
3. Maintain inspection records of all handling equipment in accordance with applicable requirements.
4. Appearance of architectural precast concrete shall consistently match approved samples and meet industry standards.
5. Institute a documented safety plan included the use of relevant personal protection equipment (i.e. hardhats, safety vest, eye protection, safety footwear, etc.)

4.2 FABRICATION OF REINFORCEMENT AND BLOCKOUTS

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4.2.1 Fabrication of reinforcement

1. Reinforcing steel shall be fabricated in conformance with the plan design and concrete product tolerances.
2. Cages of reinforcement shall be fabricated either by tying or clipping the bars, wires or welded wire fabric into rigid assemblies.
3. Reinforcement steel will be stored in a manner to protect it from dirt, oil, or other contamination. The reinforcing steel will be stored on wood pallets or concrete slabs.
4. Welded wire fabric will be spliced using the overlap of a minimum of two cross wires plus two inches on each sheet.

4.2.2 Fabrication and positioning of block outs

Block outs may be made of any rigid, non-absorptive material that will not harm the concrete and that can be held in place during the casting and curing of concrete. Any required block outs shall be formed to the dimensions as shown in the approved fabrication drawings and to specified tolerances

4.2.3 Plant Requirements

1. Reinforcement shall be fabricated within plan design tolerances and supported rigidly. Dimensions shown for reinforcing steel are to the center of the bar. Reinforcement shall be placed as near as possible to the position shown on the plans. Bars shall not vary from plan placement by more than ¼ inch. Concrete coverage to the nearest surface of steel shall never be less than one inch.
2. Block outs shall be non-absorptive and held in place with non-corrosive support.

4.3 PRE-POUR OPERATIONS

4.3.1 Cleaning of forms

Forms shall be cleaned after each use. Concrete, tape, polystyrene, and other materials adhering to the forms shall be removed. Forms are cleaned with steel wool, scrapers, chipping hammers. Air will be used to remove debris from forms.

4.3.2 Application of from release agent

Form release agent shall be applied using a sprayer after the forms is cleaned and seams sealed. Reinforcement and other items to be embedded in concrete shall be free of form release agent. Care shall be taken to avoid putting too much form release agent in the form to avoid puddling. The form release agent is applied as fine mist from an industrial sprayer. If too much form release agent is applied, it is to be wiped out with cloth rags; this is done before placing rebar in form. This is checked by a QC inspector as part of the pre-pour inspection.

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4.3.3 Positioning of reinforcement

Reinforcing steel shall be positioned as specified by the design and the concrete cover must conform to requirements as outlined in the approved fabrication plans. Positive means shall be taken to assure that their reinforcement does not shift during the casting operations. Cages shall be supported away from all form surfaces. The panel reinforcement mat or welded wire fabric shall be supported through the use of hangers, supported by the embed holders or otherwise supported, in a manner to minimize movement, assure proper concrete cover and shall not be in contact with the galvanized soil reinforcement embeds (tie-strip, clevis or other).

4.3.4 Positioning of Headers and Embeds

This includes the placement and securing of headers to create partial panels, positioning of lifting embeds (galvanized – positioned at top edge of product / form) along with other proprietary embeds (galvanized tie-strips, clevis or other). The specifics as to the quantity, spacing and other relevant information provided in the approved fabrication drawings and panel details. It is essential the specific panel details are readily accessible to facilitate pre and post pour inspections.

Place and secure the defined number of soil reinforcement connection embeds (galvanized tie-strips, clevis or other) within the embed holder secured via pin or bolt while being certain all galvanized connection embeds shall not be in direct contact with black steel.

4.3.5 Plant requirements

Pre-pour inspections shall be performed prior to casting each form. Form dimensions, form tightness, form cleanliness, form release agent application, positioning and securing of reinforcing, embedded items, and block outs shall be verified and documented.

Pre-pour inspections will be provided by QC personnel qualified to perform the duties of inspectors. A QC inspector, whom has a valid NPCA Precast Quality School level 1 certification, will inspect all product prior to casting according. Personnel responsible for fresh concrete tests, casting of cylinders, documentation of fresh concrete data, and fresh concrete conformance shall hold a current certification for ACI Field testing technician and or the relevant DOT equivalent. Personnel responsible for strength tests shall hold a current certification for ACI Strength testing technician and/or DOT specific certification.

Documentation of the pre-pour inspections shall be noted the daily production schedule and must be stamped with certifying statements of conformance by the pre-pour QC inspector.

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Pre-Pour Inspection Checklist - (Exhibit 1)
Post-Pour Inspection Checklist - (Exhibit 2)

4.4 CASTING CONCRETE

4.4.1 Transporting concrete

Concrete will be delivered by an approved Ready-mix supplier in transit truck mixers. Transit truck mixer drivers should have knowledge of concrete mixing requirements. The first load of the day for each mix shall also include a copy of the approved concrete mix design identifying the mix.

DOT specifications typically require that concrete be mixed at mixing speed between 70 and 100 revolutions of the drum. All other mixing is done at mixing speed and concrete should be discharged before a total of 300 revolutions of the drum. The counters should be set at zero after all materials have been batched and entered the truck mixer.

4.4.2 Depositing concrete into forms

Tests for slump, air, and unit weight will shall be performed on the first load of the day by a certified ACI field testing technician prior to casting product. Quality control will monitor and approve all subsequent loads of concrete during casting operations

Concrete will be deposited into forms free of puddled water, oil, dirt, or debris. Forms will be cleaned by hand or with air before casting.

4.4.3 Consolidating concrete

Non SCC (Self Consolidating Concrete) Concrete shall be consolidated in such a manner to minimize segregations of the concrete. Vibrators used to consolidate concrete shall have frequencies and amplitudes sufficient to produce well-consolidated concrete.

Internal vibrators shall be lowered vertically into the concrete under their own weight and withdrawn at the same pace. The tip of the vibrator should reach the bottom of the form or penetrate the previously form layer. Vibrate the concrete until air bubbles stop coming to the surface. Repeat the vibration process until all the concrete has been consolidated. Do not allow prolonged contact of vibrators with forms so that vibrator marks on concrete are minimal.

MSE wall panels may be vibrated using vibrating table or form mounted external vibration. Coping units and associated products may use a combination of internal and external vibrators. All vibrations practices will be monitored for spacing, frequency, amplitude and duration of vibration according to concrete mix and size of member to produce uniform consolidation of concrete.

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4.4.4 Finishing unformed surfaces

Unformed surfaces of precast concrete products shall be finished as specified in the plan design or finished using a strike off trowel to level the concrete with the top of the form.

Unformed surfaces must not have distortions greater than ¼ inch. MSE wall panels will be finished with a wood or magnesium float to bring enough mortar to the surface to cover the aggregate and provide a uniform rough wood float finish. Coping units and associated products will have a smooth metal trowel finish.

Upon concluding the floating/troweling operation, and after an appropriate wait period for initial concrete set, each panel shall be scribed / etched with the product name, project number, lot number and cast date to ensure proper tracking of the product and in accordance with project specifications.

Following the panel marking, each precast panel form and all exposed concrete surfaces shall be covered with a cure-blankets or any other curing measures as required by specification to facilitate appropriate cure environment.

4.4.5 Secondary pours

For products that require secondary pours, procedures shall be in place to assure that concrete cast during the secondary pour bonds to the precast concrete products and becomes an integral part of the product.

4.4.6 Hot-weather precautions

The temperature of the concrete during hot weather shall not exceed 95 degrees F. Special precautions will be taken to control the concrete placement temperature. Hot weather casting precautions include but are not limited to:

1. Using an approved retarding agent from an approved source and approved mix design.
2. Covering the product with a membrane cure on unformed surfaces or wet cotton mats as soon casting is complete.
3. Place concrete only when the concrete temperature at time of placement is between 50 degrees F and 95 degrees F. The items listed above will be used achieve satisfactory concrete temperatures for casting operations.
4. Steel forms or reinforcing steel with a temperature greater than 120 degrees F will be cooled by applying a fog spray (fine mist) of water before concrete placement. Water droplets left on form surfaces must not adversely affect surface finish.

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5. The covering of forms prior to and immediately following concrete placement may also be used when pouring during the heat of the day in direct sunshine to further assist with minimizing the form temperatures.

4.4.7 Cold-weather precautions

The temperature of concrete in cold weather at the time of placement shall not be below 50 degrees F. Cold weather casting precautions may include but are not limited to:

1. Heat forms prior to and after casting.
2. Monitor concrete temperature during curing. The use of digital temperature recorders or internal probes at a minimum of 1 per every 100-cubic yard of concrete or fraction thereof to monitor low concrete temperature regions.
3. If accelerated curing is used, one digital temperature monitor or internal temperature probe per 100-cubic yards of concrete or fraction thereof shall be used to monitor the temperature of the curing enclosure at the concrete surfaces.
4. One digital temperature recorder or internal probe will be placed on the panel production line to represent the first 100-cubic yards of concrete. If more than 200 cubic
5. yards is cast, a sufficient amount of recording devices shall be placed to ensure proper monitoring of concrete temperatures can be achieved.

4.4.8 Plant requirements

1. Plant personnel shall document and monitor the concrete placement practices, consolidation, and finishing of product to ensure that all concrete slump and temperature requirements are met, as well as all other requirements outlined in this text.
2. Workers shall be trained in the use of internal and external vibrators.
3. The plant shall maintain procedures for casting during hot and cold weather conditions.

4.5 CURING CONCRETE

4.5.1 General

Curing for MSE wall panels and associated products should begin as soon as casting is complete, and before the formation of shrinkage cracks.

Curing will be accomplished by radiant heat, portable heater, tarping over plastic sheeting and wet mats (curing compounds may be used if desired). These may be used individually or in combination with one another depending on temperature, rainfall, or windy conditions.

Products will remain in cure continuously until the desired strength has been attained.

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Cold weather curing- use DOT approved curing procedures as needed on all productions lines to maintain concrete temperature above 50 degrees F during the curing process. When the forecasted temperature is below 50 degrees the use of wet mats, plastic sheeting, insulated tarps, and blankets will be used to protect the concrete and maintain the concrete's internal temperature above 50 degrees during the curing period.

4.5.2 Curing by Moisture Retention

Products will be covered with wet cotton mats and tarps placed in close contact with concrete surfaces to keep them continuously wet. Barriers, coping, half connectors will be cured in a similar manner.

4.5.3 Curing with Heat and Moisture

1. If products are concrete heat-cured, the temperature of the air around the concrete shall be monitored during the curing period. Temperature records shall be maintained in the plant records.
2. Products cast outdoors shall be protected from moisture loss by applications of a curing compound, moist curing or impervious sheeting.

4.5.4 Plant Requirements

The QC inspector shall inspect curing of products and exposed surfaces of stripped products for evidence of plastic cracking. Damage shall be documented. Repairs will be made to these panels with the use of an epoxy sealant, painted or poured into the crack.

4.6 STRIPPING PRODUCTS FROM FORMS

4.6.1 Minimum strength requirement

All units shall remain in the form a minimum of 12 hours unless stated otherwise and have a stripping strength to ensure the integrity of the precast units without damage to the units during the stripping process. Minimum stripping strength is 1,800 psi.

4.6.2 Product Stripping

The panels shall be lifted from the form using 4 lift points equally spaced locations utilizing rigging of the same length. The panel shall be lifted level from the form to minimize any focused stress on the panel. Note, on deep relief liner finishes, additional care may be required due to the interaction between the product and liner may develop a higher level of friction/suction thereby more difficult to release from the form.

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The stripping process will proceed with the removal of the previous day's products have been removed from their forms and stacked in a safe and expeditious manner. Quality Control shall verify the panel quality, panel name and the relevant panel information (panel name, production date and wall name (if applicable)) transcribed via paint marker on a designated side edge of each product. Once completed, the panel stacks will be relocated to an appropriate short-term staging / curing area. The Panel Details along with a completed Pre & Post Pour Checklist shall be filed within the project file.

4.6.3 Product damage during stripping

Products damaged during stripping shall be evaluated by qualified plant personnel to determine if the repairs are necessary, and what repair is required before shipping.

4.6.4 Formed surfaces shall be considered satisfactory if they are relatively free of air voids and honeycombed areas unless the surfaces are required by the design to have a certain finish.

4.6.5 Post-pour inspection

After products are stripped from the forms they shall be inspected for conformance with design. Items to be repaired shall be classified as MAJOR or MINOR defects or as honey combed areas.

4.6.6 Plant requirements

1. A post-pour inspection shall be made of each product. The inspection shall document any damage, excessive bug holes or honeycombing, poor dimensional tolerances or other problems. A decision will be made whether it is acceptable, requires repairs, or if it has been rejected.

4.7 REPAIRING CONCRETE

5.7.1 Repairing minor or major defects repairs made on products will be made in accordance with procedures outlined in the products respective DOT repair manuals or according to individually approved procedures on file in the back of the Precast Concrete Repair Manual and according to manufacturer's recommendations.

4.7.2 Plant requirements

1. The plant shall have repair procedures for physical damage, honeycomb areas, excessive air voids, and minor and major defects. A list of approved repair materials should be located in the QC office, and maintained in a Precast Concrete repair manual binder.
2. After repairs are completed and inspected, the QC inspector will mark the product with a green dot to indicate that the product meets or exceeds specifications. The QC

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inspectors shall also affix an approved certification stamp on the product. Products that are rejected will be marked with a red dot, inscribed with the word "REJECT", listed on the QC reject log, and initialed and dated by the inspector.

3. The QC inspectors will perform spot checks of all repairs to verify that all patching procedures are being performed correctly and within specification.

4.8 STORAGE AND SHIPMENT OF PRODUCTS

4.8.1. Storage areas

Areas used for storage of products shall be firm enough and level enough to avoid causing damage to stored products.

4.8.2 Storage of products

Products shall be stored in a manner that will minimize damage caused by uneven bearing, improperly located dunnage blocks, stacking products too high, or difficulty in handling.

The storage area for MSE wall panels and associated products will be clean, well-drained, stable surface that will prevent differential settling of product. Dunnage (4" or 6" dimensional lumber or equivalent) will be placed between the ground and the finished product, as well as between each successive layer of product within the stack.

- Precast panels shall be stored in a manner to insure a safe and stable stack. This will be accomplished by using either 2 full length pieces of 4 x 4, 4 x 6 or 6 x 6 dunnage between the bottom panel and the ground.
- Each panel positioned above the bottom panel closest to the ground will require 2 continuous full-length pieces of 4 x 4, 4 x 6 or 6 x 6 dunnage (or dunnage blocks positioned at the approximate quarter points) to facilitate the product's stability.
- Each piece of dunnage should have either preco pads, styro-foam or some other covering attached to each piece of dunnage that shall be in contact with the exposed front precast face to prevent or minimize any panel discoloration, staining or face scaring.

MSE wall panels should be stacked no higher than five panels high unless a plant specific stacking procedure is developed to address applicable risks. Dunnage will be arranged in vertical planes and aligned with the lifting anchors.

4.8.3 Shipment of products

Trucks and other conveyances used to transport precast concrete products from the plant to the job site shall be equipped and maintained to deliver product without damaging them.
Shipping procedures shall be followed as outlined in the company shipping procedures outline.

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4.8.4 Final inspection

Prior to shipment, products shall be inspected to assure design conformance and proper identification, and repairs have been completed.

4.8.5 Plant Requirements

1. Storage areas shall be maintained firm and level.
2. Product shall be stored to minimize damage.
3. A final inspection of the product shall be made prior to shipment. The inspection shall include verification that the product conforms to project requirements, was properly inspected and that repairs have been made where needed and inspected. A QC inspector shall make a final inspection of the products as loaded on a trailer for damaged or cracked products that may have occurred during loading. The QC inspector shall also make a safety inspection to ensure that loads are secured properly by checking the loading and tie-down procedures.

5.0 MARKING AND LABELING OF PRODUCTS

5.1 Plant requirements

1. All products will be labeled with the product name, project number, lot number and cast date. Panels will have the above information scribed into the back of the panels immediately after the concrete has been floated. Coping and associated products will be labeled with the above information on the side of the product after being stripped from the forms.

6.0 PRODUCT DEFICIENCY CLASSIFICATION AND REPAIR METHODS

6.1 Classification Definitions

1. Bug Hole: "A void caused by air that is trapped against the form and that has an area up to 3.0 sq. in. and a depth up to 1.5 inches."
2. Honeycombing: "Is voids in the concrete, loss of fines or other material from between the aggregate particles, the inclusion of air pockets between aggregate particles, or larger volumes of lost material. "
    i. Two types: Minor & Major. "Minor Honeycombing is a void no deeper than 1.5 inches to sound concrete and no larger than 1.0 sq. ft. in area that results after the removal of unsound material."
3. "Spall: "A depression resulting when fragment is detached from a larger mass by impact, action of weather, by pressure or by expansion within the larger mass."

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i. Three types: Cosmetic, Minor & Major. "A cosmetic spall is a circular or oval depression not greater than 1.0 inch in depth no greater than 3.0 sq. inches in area. A minor spall is no larger than 1.0 sq. ft. and no deeper than 1.5 inches."

In an effort not to supply any structurally deficient products we have foregone addressing "major" deficiencies in each classification. A major deficiency can be defined as damaged or deficiency exceeding that as defined as cosmetic or minor and some DOT specifications permits engineering evaluation of a specific product / repair proposals for these types of findings. In general, Major Defects are subject to rejection. Repairs of Major Defects will be addressed on a case-by-case basis.

6.2 Deficiencies and Repairs

The Quality Control Manager will examine all deficiencies and will determine the specific nature of the repairs and decide the most appropriate course of action of remedy. The means of remedy could range from minor cleaning of connections/embeds up to and including minor concrete deficiencies. All minor deficiencies will be listed and described on a "Repair Record"

sheet. Furthermore, concrete deficiencies will be classified under two categories; Major or Minor (Cosmetic):

- Major / Non-repairable: In instances where a product is located containing deficiencies exceeding that as defined above as cosmetic or minor, and the product is deemed un-repairable, the product will be physically marked by red paint or crayon
- along all 4 sides and relocated to the rejected / culled area of the manufacturing facility until the rejected product can be relocated off-site to the disposal facility.
- Out of Tolerance Rejection: In the rare instances where a product may be poured despite the form passing the quality control review process (i.e. checked during the morning setup for squareness and face deficiencies, etc.) the product shall be designated as rejection if the product is not in conformance with the following allowable tolerances.
    - Precast component dimensions: lateral position of soil reinforcement attachment devices shall be within 1 inch (25 mm). All other dimensions - within 3/16 inch (5 mm).
    - Precast panel squareness: Squareness, as determined by the difference between the two diagonals, shall not exceed ½ inch (12 mm).
    - Precast Units shall not have an angular distortion exceeding 0.2 inches (5 mm) in 5 feet (1524 mm) or 0.4 inches (10 mm) over 10 feet (3048 mm).
    - Precast panel surface finish shall not have surface deficiency on smooth formed surfaces over a length of 5 feet (1524 mm) must not exceed more than 0.1 inches (2.5 mm). Textured surfaces shall not have surface deficiency greater than 5/16 inch (8 mm) when measured over a length of 5 feet (1524 mm).

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- Other Miscellaneous deficiencies subject to rejection.
  - Defects indicating honeycombed or open texture concrete.
  - Signs of aggregate segregation
  - Broken or cracked corners
  - Soil reinforcement attachment devices improperly installed/damaged
  - Lifting inserts not useable
  - Exposed reinforcing steel
  - Insufficient cover over reinforcing steel
  - Cracks beyond temperature cracks which extend to the edge of product and or wrap the edge of the product.
  - 28 day concrete compressive strength failure
  - Precast component thickness in excess of plus ½ inch (12mm) or minus 3/16 inch (5 mm)
  - Stained front face due to excess form oil, dirt, debris or other.
- Cosmetic / Minor: The product will be designated to be repaired by either pointing the product with 1 part sand / 1 part cement, which would typically be accomplished while the product is or about to be placed into the cure area, or at the discretion of the Q.C. Manager with a specifically approved patching product contained on the Qualified Product List (QPL).
- The approved repair products to be considered for use may include but not limited to the following products; AL Patterson Preblend Blendcrete; Lambert Epiweld 560 / 580 epoxy bonding agent, Euclid Euco-Speed MP, Lambert Vibropruf #11, SikaQuick 1000, Bonsal Fast Set Cement and/or with 1 part sand, 1 part cement paste and as necessary white Portland cement (to facilitate the matching of product color & insure proper blending). All products being considered for use shall be approved or part of the respective state qualified product list ((QPL – if applicable) and shall be prepared and applied in accordance with the manufacture recommendations. The actual concrete repair procedure shall include but not limited to proper surface preparation, the application of an epoxy bonding compound to the affected area, proper patch material preparation, application of patching material and final shaping / texturing and grinding to insure proper product blending.

All repairs will be conducted under direct supervision of the Q.C. Manager in a manner to best facilitate proper blending of the repair product with the precast element and to insure appropriate strength and quality to the satisfaction of the engineer. All repairs conducted outside the use of 1 cementitious mix of part sand / 1 part cement shall be tracked via use of "Repair Record" to record deficiencies and repairs of significant nature.

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7.0 Supporting documents

1. Pre-Pour Inspection Checklist - (Exhibit 1)
2. Post-Pour Inspection Checklist - (Exhibit 2)

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REINFORCED EARTH

Exhibit - 1
Pre-Pour Inspection Checklist

Pre-Pour Inspection Checklist - MSE/Precast

Date:
Lot #:
Job #:

Reinforced Earth - XXXX Plant Location

					Forms						Rebar / Imbeds				Concrete					
Piece #	Wall #	Panel	Liner	Comments	Form Bolted / Clamped Correctly	Form Level	Proper Dimensions	Cleaned & Oiled Properly	Proper Lifting Anchors & Locations	Block-Outs Proper & Secured	Tie Strips Positioned Properly	Proper Rebar Clearance	Proper Rebar Size	Other Embeds Properly Installed	Good Quality Concrete at Proper Slump	Concrete Test Completed	Vibrated Properly	Proper Finish	Curing Compound Applied	Covered and Curing Started

Pre Pour Inspection 1-7-2022 Page 1 of 1

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REINFORCED EARTH

Exhibit - 2
Post-Pour Inspection Checklist

Post-Pour Inspection Checklist - MSE/Precast

Reinforced Earth - XXXX Plant Location

Date:
Lot #:
Job #:

					Stripping						Label				Yarding					
Piece #	Wall #	Panel	Liner	Comments	Stripping P.S.I.	Dimensions Match Drawings	Inspect the Face of Panel	Cracking on Back of Panel	Any Chips or Damage	Clean All Edges	Proper Labeling	Label Match Product Type	Proper Job Number	Proper Piece Number	Stacked With Same Size Panels	Dunnage in the Proper Place	Panel Damaged During Handling	Moved to the Yard	Moved to the Rapair Area	Damaged Panel Logged on the Non-Conformance

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A4.1.2 Soil Reinforcement Quality Control

The following 76 pages are from RECo's Reinforced Earth Materials Quality Control Process.

Reinforced Earth Materials

QUALITY CONTROL PROCESS

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Introduction:

The Reinforced Earth Company is a design/supply firm recognized as the inventor and market leader of the Mechanically Stabilized Earth (MSE) retaining wall industry. The company provides retaining wall systems, reflective and absorptive sound walls, precast arches and slope stabilization systems for use in civil engineering applications including transportation, bridges, railways, industrial facilities and marine structures. The company's products are constructed with an innovative civil engineering material and sold under the trademark "Reinforced Earth ®."

The Reinforced Earth Company has more than 50 years of experience designing structures and supplying materials to contractors and owners. In that time, more than 40,000 structures have been built using the company's products. With each project, we deliver the elements for success - a firm price, a reliable schedule, predictable construction and a commitment to excellence in Engineering design and service.

The Reinforced Earth Company manages a distribution center located in Houston, TX where material is received, inspected, tagged, warehoused and ultimately packaged for shipment, as required, to each individual project.

The goal for the central distribution center (warehouse) is to maintain an adequate level of inventory and supply all the necessary materials for a project when required and in a timely manner and concurrently ensure that the material received meets or exceeds the required specifications.

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Sterling, VA 20166
www.reinforcedearth.com

A SUBSIDIARY OF SOLETANCHE FREYSSINET

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CMC Steel-Magnolia, AR	ISO - 9001	certificate	Reinforcing Strips
Nucor Fastener Division	ISO - 9001	certificate	Fasteners
	ISO - 16949	certificate	
Dyna Mix/Rex-Hide	ISO - 9001	certificate	Bearing Pads
Carthage Mills	Quality Control Program		Filter Cloth
Nucor Steel Indiana	ISO - 9001	certificate	Tie Strips
Metalplate Galvanizing	Quality Control Program		Galvanizing
Accu-Test Labs	Accreditations /certifications		Steel Strip Testing
V.Louison & CIE	Certifications		Geomega
TRI Environmental	Accreditations		Geosynthetic Testing
Surebuilt Manufacturing	Certifications/ Testing		Wire Ladders

7. Sample Certificate of Compliance and Sample RECo certification package
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1. QUALITY POLICY

This manual details the Quality Control Process utilized by The Reinforced Earth Company – Steel Division and its suppliers and reaffirms our commitment to continue to supply and manufacture where applicable, the highest possible quality Mechanically Stabilized Earth products.

To achieve this goal we have built our own processing system which includes specific standards which must be met by our material suppliers as well as a continuous internal verification process which when put together, ensures that the required standards are met.

We are constantly improving and developing new procedures based on a continuous process review and develop new guidelines to accommodate any changes in the industry, internal and external.

Our employees receive training techniques that are used in the monitoring of key processes such as material quality, manufacturing technology, business, and personal development.

Our suppliers continuously provide RECo with updated information on their latest technologies, Quality Control processes or certifications as they become available.

It is our commitment that the achievement of quality starts and ends with constant involvement by dedicated people at all levels of our company and working together with all our material supplier partners.

Our goal is not just to meet, but also to exceed specifications and meet our customer's and project specific requirements.

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2. SCOPE

RECo maintains a Quality Control system through which all products shipped from its manufacturing facility meet the requirements as specified by the customer. The Quality Control system is established to evaluate incoming material, monitor manufacturing processes, audit finished goods, inspect outgoing shipments, and implement corrective action as required.

Quality Control activities are aimed primarily at preventing defects rather than only detection and correction, thereby assuring the customer of the highest quality products which will meet all the necessary requirements and specifications.

All material certifications provided by our suppliers are entered in RECo's automated certification system for ease of retrieval at later times as required. The retention of these records follow the standard procedures in the industry for the retention of electronic data.

Through this system, RECo will continue to maintain its leadership position in the marketplace and ensure that all material meets or exceed the specifications requirements.

RECo's Organization:

Eric Hunter
National Procurement Manager (Sterling, VA)
ehunter@reinforcedearth.com

William Crutcher
Plant Manager (Houston, TX)
bcrutcher@reinforcedearth.com

Paul Taylor
Quality Control Supervisor (Houston, TX)
ptaylor@reinforcedearth.com

2.1 ROLES AND RESPONSIBILITIES OF COMPLIANCE CONTORL PERSONNEL

National Procurement Manager
Responsible for evaluating suppliers and negotiating contracts to achieve the lowest price for high-quality goods. Duties and responsibilities include:

- Planning and overseeing the work of Buyers and Purchasing Agents
- Establishing and managing procurement rules and regulations
- Buying goods and services for the company
- Discussing contracts
- Evaluating quality, price, reliability, technical support and availability of goods and services

Plant Manager
Ultimately responsible for establishing, implementing and maintaining the quality system. Specific responsibilities include formulating the quality policy, defining the organizational structure, assigning authorities and responsibilities, periodically reviewing the quality system and making available the resources and personnel necessary to maintain the system.

Quality Control Supervisor
This person has the authority and responsibility to ensure that the quality management system is maintained, and its efficiency is continuously improved.

- Oversee the quality management system
- Performs or arranges for internal audits
- Prepares quality plans conforming to applicable laws and standards
- Manages non-conformance issues
- Coordinates document & data control activities
- Submits plans to appropriate design review entity for approval.
- Assist with daily inspections

3. SUPPLIER RELATIONS

3.1 ORGANIZATION

The quality of materials is ensured by partnering with suppliers that are prepared and qualified to adhere by strict standards imposed by RECo. RECo monitors and makes periodic visits to the partner's sites to ensure that these agreed standards and quality control procedures as submitted are adhered to.

3.2 SUPPLIER RELATIONS

RECo exercises oversight over its suppliers to ensure that services and supplies conform to specified requirements. Purchase orders are issued and controlled to ensure incorporation of pertinent technical and quality requirements, including authorized changes. Adequate records of inspections and tests performed on purchased material are maintained by the supplier and provided to RECo for verification before the material arrives at the warehouse.

3.2 SUPPLIER SELECTION AND INFORMATION

All RECo suppliers are carefully vetted and become accepted RECo suppliers based on their ability to demonstrate that they do have an internal Quality Control program in place or alternatively, provide a certificate of compliance from an independent authority stating that these standards are met.

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4. MATERIAL INSPECTION AND ACCEPTANCE

4.1 REINFORCING STRIPS

4.1.1 Supplier Reporting Requirement

The reinforcing strip supplier is required to provide mechanical and chemical characteristic data immediately after fabricating the reinforcing strips certifying their material to the specifications as called for on the RECo purchase order.

A Heat number consists of a series of billets totaling approximately 40-60 tons. The chemistry for these Heats is tested and verified prior to casting the billets which are then marked with the unique Heat number. Billets are then rolled and reinforcing strips are manufactured. At the start of the rolling for every Heat number, the reinforcing strip is tested to ensure it meets all required mechanical properties per the specifications.

The data is submitted to RECo in electronic format certifying that the product was melted and fabricated in the USA, conforming to the requirements of the "Buy American Act". These certifications include laboratory analysis of chemical and mechanical properties for each Heat number used to produce the reinforcing strips. RECo references a group of Heat numbers used to manufacture reinforcing strips as a "rolling date".

The steel mills utilize their own internal laboratories and they have all been certified ISO-9001 compliant.

All received data is entered in RECo's verification system to ensure that the corresponding values for mechanical properties and chemical composition of the required elements meets the latest ASTM requirements.

Once the RECo's system verifies that all the requirements are met then informs the supplier of its acceptance and that the shipment of such material can commence.

In addition, RECo randomly performs testing by independent laboratories to confirm the validity of material certifications supplied by the manufacturer. These tests are performed at least once a month or whenever a new "rolling date" is reported. The laboratories used are Accu-Test Labs, of Houston, TX and Element Materials Technology, of Houston, TX. Laboratory Accreditations are provided at the end of this report.

4.1.2 Receiving Procedure

All incoming reinforcing strip material is staged in the dock area until it is confirmed that it is the material which has been ordered and accepted based on the required criteria and previously submitted certifications. The supplier's shipping documents are verified for specification's compliance. The material is then checked for dimensional compliance, and for any visible damage. The inspection results are recorded into an electronic incoming inspection file along with the corresponding heat number and rolling date and entered in the RECo automated certification system and is given an unique identifying number for later retrieval when the material is released to a project. Accepted material is then brought into the warehouse for storage and later sent to the production area for final preparation before shipping to the project.

4.1.3 Defective Material Reporting Procedure

In the event that defective material or not previously approved material is received, these shipments are given a red reject tag and placed in the quarantine area. A defective material report is then issued to the vendor and appropriate RECo departments. After further investigation, should the material still be found to be non-conforming to specifications, it is rejected and the supplier instructs RECo of the method for final disposal to be used.

4.1.4 Processing of Reinforcing Strips

The reinforcing strips required for each project and structure vary in length according to the design requirement and height of the structure. The Reinforced Earth Company utilizes an automated process to cut and punch the reinforcing strips to the required lengths and sends the product to the galvanizer for galvanizing.

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4.1.5 Galvanizing

The reinforcing strips are galvanized within the same grounds by Metalplate Galvanizing, L.P. which operates independently from all of RECo's activities. This galvanizing process is done in a controlled environment providing consistent and predictable results. The galvanizer monitors its process daily and verifies with electronic or magnetic gages that the finished product has been galvanized according to the required specifications.

The galvanizer provides RECo with certifications that the material meets or exceeds the required specifications as noted in the drawings.

In addition, RECo personnel is also present at delivery time of the galvanized product and inspects for quality and specification compliance prior to moving the finished product into the inventory of finished goods. The inspection results are recorded in a finish goods inspection logbook. Random spot checks are also done prior to shipment to the jobsite.

4.1.6 Defective Galvanizing Material Reporting Procedure

In the event that non-compliant material is received, the reinforcing strips are red tagged, quarantined and a report is prepared and sent to the galvanizer. The galvanizer inspects the reinforcing strips and makes the necessary repairs or chooses to re-processes the material as necessary to meet specifications.

An additional inspection is carried out on a routine basis once the material is loaded for shipping using an acceptance sampling procedure verifying the length of the reinforcing strips and the quality prior to leaving the site.

4.1.7 Metalplate's Organization

The Organization structure and quality control program for Metalplate Galvanizing, L.P. is included with this document.
For this location, Phil Sloop (psloop@metalplate.com) is the Quality Manager for Metalplate.

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4.2 ACCESSSORIES

4.2.1 Supplier Reporting Requirement

All suppliers of accessories are required to submit certifications for the materials certifying their material to the specifications as called for on the RECo purchase order.

The data is submitted in electronic format and reviewed by RECo personnel to ensure that the corresponding values meet the necessary specification requirements.

Once RECo personnel verifies that all the requirements are met then the supplier is informed of its acceptance and that the shipment of such material can commence.

In addition, RECo randomly performs testing by independent laboratories to confirm the validity of material certifications supplied by the manufacturer.

4.2.2 Receiving Procedure

All incoming material is staged in the dock area and inspected for compliance and verification that the material received is the material previously certified.
The material is also checked for dimensional compliance, and for any visible damage. The inspection results are recorded in an incoming inspection logbook and entered in the RECo automated certification system and is given a unique identifying number. The received containers are tagged with this unique identifier number, later to be used for retrieval of the appropriate certifications once the material is ready for shipment to the project.
Accepted material is then brought into the warehouse for storage.

4.2.3 Defective Material Reporting Procedure

In the event that defective material or not previously approved material is received, these shipments are given a red reject tag and placed in the quarantine area. A defective material report is then issued to the vendor and appropriate RECo departments.

After further investigation, should the material still be found to be non-conforming to specifications, it is rejected and the supplier instructs RECo of the method for final disposal to be used.

5. PACKAGING AND SHIPPING

5.1 Ordering Process and Gathering Materials

A detailed material order is submitted to the distribution center with specific instructions for parts, quantity and delivery date approximately three weeks prior to shipping. RECo personnel prepares a packaging slip and the material begins to be assembled in a common area for each individual order. As the material is being assembled and removed from the inventory location, the unique identifier found in each container is recorded in the packaging slip.

5.2 Packaging and Securing the Load

Once all the material is assembled in one location, personnel begins to secure the contents for this order as appropriate (shrink wrapped, boxed, palletized, etc) to ensure the safe arrival and without damage to any of its contents.

5.3 Recording and Shipping

Before departure of every truck, the content for the particular load is verified for quantity accuracy and expected quality. A Bill Of Lading (BOL) is then prepared, identifying all of the material being shipped with its unique identifier number as previously recorded in the Packaging Slip. Certifications are later retrieved via RECo's automated system utilizing the cross references of unique identifier numbers recorded. In addition, a photographic record of the materials loaded on the truck is made and stored in the system for later retrieval should it be necessary.

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6. RECo's major suppliers Quality Control Certificates and processes

○ Nucor Steel Jackson, MS	ISO - 9001 ISO - 14001	certificate certificate	Reinforcing Strips
○ CMC Steel Magnolia, AR	ISO - 9001	certificate	Reinforcing Strips
○ Nucor Fastener Division	ISO - 9001 ISO - 16949	certificate certificate	Fasteners
○ Dyna Mix/Rex-Hide	ISO - 9001	certificate	Bearing Pads
○ Carthage Mills	Quality Control Program		Filter Cloth
○ Nucor Steel Indiana	ISO - 9001	certificate	Tie Strips
○ Metalplate Galvanizing	Quality Control Program		Galvanizing
○ Accu-Test Labs	Accreditations /certificates		Steel Strip Testing
○ Element Materials Technology	Accreditations /certificates		Steel Strip Testing
○ V.Louison & CIE	Certifications		Geomega
○ TRI Environmental	Accreditations		Geosynthetic Testing
○ Surebuilt Manufacturing	Certifications/ Testing		Wire Ladders

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7. Sample RECo certification package

- Certificate of Compliance
- Cover letter
- Bill of Lading
- Reinforcing Strips Steel Certification (CMC Steel)
- Reinforcing Strips Steel Certification (Nucor)
- Galvanization (Metalplate)
- Filter fabric (Carthage Mills)
- Bearing pads (Dyna-Mix/Rex-Hide)
- Bolts (Nucor)

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CERTIFICATE OF REGISTRATION

SRI Quality System Registrar

ANAB ACCREDITED ISO/IEC 17021 MANAGEMENT SYSTEMS CERTIFICATION BODY

Having been audited in accordance with requirements of

ISO 9001:2015 – ANSI/ISO/ASQ Q9001-2015

SRI Quality System Registrar, 300 Northpointe Circle, Seven Fields, Pennsylvania, 16046, USA, hereby grants to:

Nucor Steel Jackson, Inc.

Registration of the management system at its locations:

Nucor Steel Jackson, Inc.
3630 Fourth Street, Flowood, Mississippi, 39232, USA
General Recycling LLC
2050 Flowood Drive, Flowood, Mississippi, 39232, USA

The conditions for maintaining this certificate of registration are set forth in the SRI registration agreements R20.3 and R20.4. Further clarifications regarding the scope of this certificate and the applicability of ISO 9001:2015 requirements may be obtained by consulting the organization.

Scope of ISO 9001:2015 registration: "Manufacture and supply of merchant bar, reinforcing bar products, billets, and scrap recycling of ferrous and non ferrous material."

Initial SRI Registration date: May 4, 2009

Current registration period: May 3, 2018 through May 2, 2021

Signed for SRI: _____
Christopher H. Lake, President & COO

Release Date: May 3, 2018
Certificate Number: 018276
Registration Number: 0985-22

ISO 9001:2015 SRI CERTIFIED

CERTIFICATE OF REGISTRATION

SRI Quality System Registrar

ANAB ACCREDITED ISO/IEC 17021 MANAGEMENT SYSTEMS CERTIFICATION BODY

Having been audited in accordance with requirements of

ISO 14001:2004

SRI Quality System Registrar, 300 Northpointe Circle, Seven Fields, Pennsylvania, 16046, USA, hereby grants to:

Nucor Steel Jackson, Inc.

Registration of the management system at its locations:

Nucor Steel Jackson, Inc.
3630 Fourth Street, Flowood, Mississippi, 39232, USA

General Recycling LLC
2050 Flowood Drive, Flowood, Mississippi, 39232, USA

Nucor Jackson Warehouse
2000 Underwood Drive, Flowood, Mississippi, 39232, USA

The conditions for maintaining this certificate of registration are set forth in the SRI registration agreements R20.3 and R20.4.

Scope of ISO 14001:2004 registration:	"Environmental activities related to metallic scrap and the manufacture of steel bars."
Initial SRI registration dates:	January 31, 2007 at Fourth Street and Flowood Drive December 7, 2015 at Underwood Drive
Current registration period:	December 7, 2015 through September 14, 2018

Signed for SRI: _____
Christopher H. Lake, President & COO

Release Date: December 7, 2015
Certificate Number: 016050
Registration Number: 0985-22

ISO 14001 SRI CERTIFIED

CERTIFICATE OF REGISTRATION

SRI Quality System Registrar

ANAB ACCREDITED ISO/IEC 17021 MANAGEMENT SYSTEMS CERTIFICATION BODY

Having been audited in accordance with requirements of

ISO 9001:2008 – ANSI/ISO/ASQ Q9001-2008

SRI Quality System Registrar, 300 Northpointe Circle, Seven Fields, Pennsylvania, 16046, USA, hereby grants to:

CMC Steel Arkansas, Inc.

Registration of the management system at its location:

100 Columbia 7-B
Magnolia, Arkansas, 71753, USA

The conditions for maintaining this certificate of registration are set forth in the SRI registration agreements R20.3 and R20.4. Further clarifications regarding the scope of this certificate and the applicability of ISO 9001:2008 requirements may be obtained by consulting the organization.

Scope of ISO 9001:2008 registration:	"Manufacture and supply of rebar, flats, angles, and specialty rail section products."
Exclusions:	Design and Development; Service Provision; Validation of Processes for Production and Service Provision; Customer Property
Initial SRI Registration date:	June 28, 2004
Current registration period:	June 26, 2016 through September 14, 2018

Signed for SRI:

Christopher H. Lake, President & COO

Release Date: June 26, 2016
Certificate Number: 016471
Registration Number: 2903-01

ISO 9001 SRI CERTIFIED

CERTIFICATE OF REGISTRATION

SRI Quality System Registrar

ANAB ACCREDITED ISO/IEC 17021 MANAGEMENT SYSTEMS CERTIFICATION BODY

MGMT. SYS. RvA C 074

Having been audited in accordance with requirements of

ISO 9001:2008 – ANSI/ISO/ASQ Q9001-2008

SRI Quality System Registrar, 300 Northpointe Circle, Seven Fields, Pennsylvania, 16046, USA, hereby grants to:

Nucor Fastener

Registration of the management system at its location:

6730 County Road 60
St. Joe, Indiana, 46785, USA

The conditions for maintaining this certificate of registration are set forth in the SRI registration agreements R20.3 and R20.4. Further clarifications regarding the scope of this certificate and the applicability of ISO 9001:2008 requirements may be obtained by consulting the organization.

Scope of ISO 9001:2008 registration:	"Manufacture of fasteners."
Exclusions:	Design and Development
Initial SRI Registration date:	December 13, 2004
Current registration period:	July 14, 2016 through September 14, 2018

Signed for SRI:

Christopher H. Lake, President & COO

Release Date: July 14, 2016
Certificate Number: 016576
Registration Number: 1439-01

ISO 9001 SRI CERTIFIED

CERTIFICATE OF REGISTRATION

Having been audited in accordance with requirements of

ISO/TS 16949:2009

SRI Quality System Registrar, 300 Northpointe Circle, Seven Fields, Pennsylvania, 16046, USA, hereby grants to:

Nucor Fastener

Registration of the management system at its location:

6730 County Road 60
St. Joe, Indiana, 46785, USA

The conditions for maintaining this certificate of registration are set forth in the SRI registration agreements R20.3 and R20.4. Further clarification regarding the validity of this certificate can be obtained on SRI's website with this code 016577.

Scope of ISO/TS 16949:2009 registration: "Manufacture of fasteners."
Exclusions: Product Design and Development
Current registration period: July 14, 2016 through September 14, 2018

Signed for SRI:

Christopher H. Lake, President & COO

Release Date: July 14, 2016
IATF Certificate Number: 0243178
Certificate Number: 1439-01

CERTIFICATE OF REGISTRATION

This is to certify that

DYNA-Mix, Inc.

705 South Lyons Ave, Tyler, Texas 75702-6857 USA

operates a

Quality Management System

which complies with the requirements of

ISO 9001:2015

for the following scope of certification

Design, manufacture and distribution of custom mixed rubber materials.

Certificate No.:	CERT-0122702	Original Certification Date:	January 2, 2003
File No.:	013398	Certification Effective Date:	August 21, 2018
Issue Date:	August 21, 2018	Certificate Expiry Date:	November 18, 2020

Kevin Goodwin
General Manager Technical Services
SAI Global Assurance

Certified System ISO 9001

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Quality Control Program Outline

Definition of Lot:

A planned production quantity satisfying all of the following:

- Manufactured under the same material specification, including yarn and weave specifications.
- Identified as the same style (fabric designation).
- When tested, having physical characteristics consistent with published values.

Quality Control Sampling of Each Lot:

As a minimum, a number of production units shall be selected at random from each lot in accordance with Table 1.

TABLE 1 Number of Units to be Selected as Lot Sample Specification Conformance

Number of Units in Lot			Number of Units Selected
1	to	2	1
3	to	8	2
9	to	27	3
28	to	64	4
65	to	125	5
126	to	216	6
217	to	343	7
344	to	512	8
513	to	729	9
730	to	1000	10
1001	to	more	11

Note: A production unit is considered to be a shipment roll.

Typically, the first shipment roll from each loom roll will be sampled. It will be necessary to consider the minimum planned production quantity to determine if more frequent sampling and testing is required.

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Quality Control Testing of Each Sample:

Each quality control sample shall be sent to the quality control lab before the end of the shift during which the sample was taken. Full identification of the sampled shipment roll and corresponding loom roll shall be provided with the sample.

The following tests shall be run on every sample received in the Q.C. lab:

Test Property	Test Method
Weight	ASTM D5261
Thickness	ASTM D5261
Grab Tensile	ASTM D4632
Wide Width Tensile	ASTM D4595
Mullen Burst	ASTM D3786
Puncture	ASTM D4833
Trapezoid Tear	ASTM D4533
A.O.S.	ASTM D4751
Permittivity	ASTM D4491
Percent Open Area	C.O.E. 02215

Note: It is not necessary to always run any of these tests which are not listed on the published literature of the material being tested.

Quality Control Test Results:

All quality control test results shall be maintained by the Quality Control Manager with the corresponding shipment roll identification.

The Quality Control Manager will make lot testing summaries available, upon request, detailing the individual test results and aggregate mean, minimum and standard deviations of each test property for the shipment rolls under consideration.

Failing Test Results:

Individual test results falling below published minimums do not of the themselves define failing production. First, duplicate test should be run on the material in question to determine if the problem is in the test procedure. If the retest shows similarly failing results, then the Quality Control Manager shall require an additional sample from an associated loom roll (i.e. common beam and/or fill yarn box depending on the property in question.). If this additional testing shows similarly failing results, then all material from all associated loom rolls will be given a different, unique style number (i.e. downgraded).

If the additional testing produces satisfactory test results, then the additional test results will be averaged with the initial test results and used as representative of the material produced. If the averaged test results fall below published limits, then all material from all associated loom rolls will be downgraded.

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Since 1958: America's *First* Geotextile Company

Publishing Physical Property Values:

Every manufacturing process has inherent variability. Not only do actual properties of the finished product vary randomly around "target" production values, the "trend" or "drift" of these properties varies with time around the target. Therefore, characteristic properties of material produced during any given time will likely be different than properties of material produced at a later time.

As a result, it is not possible to provide values months in advance of production which will accurately reflect actual material properties at all times. At best, historical results can be reported with hopes that production enhancements can be made to reduce manufacturing availability and improve reported physical properties. Realistically, since so many properties are being "balanced", it is most clear and honest to report manufacturing "target" or "typical" values and "minimum" values below which the material is considered to have physical characteristics excessively different from those of other material in the same lot (i.e. greater than 2.5 to 3 standard deviations below the "target" values).

Minimum Average Roll Values (MARV):

Industry has coined the term MARV to reflect a 95% confidence level. To the Manufacturer this means that 97.5% of all material produced should exceed a given MARV. Statistically, this is equivalent to the mean, $\bar{x}$, less two standard deviations, 2σ.

Since $\bar{x}$ and σ vary over time and are unique to any given period of production the MARV for a given month of production may be significantly higher or lower than a MARV calculated (i.e. averaged) over an entire year.

As a result, it is only possible to confidently certify a MARV specification after referring to the actual test results for the shipment rolls to be provided (and others produced in the same time frame).

Packaging and Shipments:

All geotextile rolls shall be furnished with suitable wrapping for protection against moisture and extended ultraviolet exposure prior to placement. Each roll shall be labeled or tagged to provide product identification sufficient for inventory and quality control purposes. Rolls shall be stored in a manner which protects them from the elements. If stored outdoors, they shall be elevated and protected with a waterproof cover.

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Since 1958: America's *First* Geotextile Company

CERTIFICATE OF REGISTRATION

SRI Quality System Registrar

ANAB ACCREDITED ISO/IEC 17021 MANAGEMENT SYSTEMS CERTIFICATION BODY

Having been audited in accordance with requirements of

ISO 14001:2015

SRI Quality System Registrar, 300 Northpointe Circle, Seven Fields, Pennsylvania, 16046, USA, hereby grants to:

Nucor Corporation

Registration of the management system at its locations:

Indiana Division
4537 South Nucor Road, Crawfordsville, Indiana, 47933, USA
3560 South Nucor Road, Crawfordsville, Indiana, 47933, USA

The conditions for maintaining this certificate of registration are set forth in the SRI registration agreements R20.3 and R20.4.

Scope of ISO 14001:2015 registration:	"Environmental activities related to the manufacture of Hot Rolled, Thin Strip Casted, Cold Rolled, and Galvanized Products."
Initial SRI registration date:	May 27, 2003 at 4537 South Nucor Road May 12, 2018 at 3560 South Nucor Road
Current registration period:	May 12, 2018 through May 11, 2021

Signed for SRI: _____
Christopher H. Lake, President & COO

Release Date: May 12, 2018
Certificate Number: 018375
Registration Number: 0985-02/64

ISO 14001:2015 SRI CERTIFIED

HOT DIP GALVANIZING PROCEDURE

Metalplate Galvanizing, L.P.
Birmingham • Atlanta • Jacksonville • Houston

1. Purpose

It is the purpose of this document to describe to our customers the normal operating practices, specifications and standards used by Metalplate Galvanizing, L.P. to produce and deliver cost effective, high quality coatings of hot dip galvanized products.

2. Scope

This Hot Dip Galvanizing Procedure applies to the hot dip galvanizing of iron and steel products at all Metalplate facilities. It encompasses the process steps for material preparation and zinc coating using the hot dip galvanizing method as well as material finishing and cleaning procedures. **Detailed process parameters and practices are documented in Standard Operating Procedures for each Metalplate facility.**

3. References

Any special customer requirements or needs, as well as the use of any other additional references and standards relating to hot dip galvanizing not listed below, will be discussed, clearly defined and mutually agreed upon in writing by the customer and Metalplate prior to the beginning of work.

3.1. ASTM Standards relating to hot dip galvanized materials:

3.1.1. ASTM A123 - *Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products*

3.1.2. ASTM A143 - *Standard Practice for Safeguarding Against Embrittlement of Hot-Dip Galvanized Structural Steel Products and Procedure for Detecting Embrittlement*

3.1.3. ASTM A153 - *Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware*

3.1.4. ASTM A384 - *Standard Practice for Safeguarding Against Warpage and Distortion During Hot-Dip Galvanizing of Steel Assemblies*

3.1.5. ASTM A385 - *Standard Practice for Providing High-Quality Zinc Coatings (Hot-Dip)*

3.1.6. ASTM A767/A767M - *Standard Specification for Zinc Coated (Galvanized) Steel Bars for Concrete Reinforcement*

3.1.7. ASTM A780 - *Standard Practice for Repair of Damaged and Uncoated Areas of Hot-Dip Galvanized Coatings*

3.1.8. ASTM B6 - *Standard Specification for Zinc*

3.1.9. ASTM E376 - *Standard Practice for Measuring Coating Thickness by Magnetic-Field or Eddy-Current (Electromagnetic) Test Methods*

3.2. American Galvanizers Association (AGA) publications:

3.2.1. *The Design of Products to be Hot Dip Galvanized After Fabrication*

3.2.2. *Recommended Details for Galvanized Structures*

3.2.3. *Inspection of Products Hot Dip Galvanized After Fabrication*

3.2.4. *Suggested Specification for Hot Dip Galvanizing*

3.3. Federal Specifications:

3.3.1. DOD-P-21035 - *Paint High Dust Content, Galvanizing Repair*

3.3.2. MIL-P-26915 - *Primer Coating, Zinc Dust Pigmented*

3.4. Standard Operating Procedures: Metalplate Galvanizing, L.P.

4. Responsibility and Authority

The plant manager of each Metalplate facility is ultimately responsible for the overall implementation of this procedure. He does so by defining the responsibility, authority and interrelation of personnel who manage, perform and verify work affecting production and quality, particularly for personnel who need the organizational freedom and authority to:

- Initiate action to prevent nonconformities relating to the process and final product.
- Identify and record any problems related to the process and final product.
- Initiate, recommend or provide solutions through designated channels.
- Control further processing and/or delivery of nonconforming product until the deficiency or unsatisfactory condition has been corrected.

5. Scope of Work to be Performed

5.1. Contract Review Prior to Commencement of Work

5.1.1. In the event of a large project or job, Metalplate shall offer or be amenable to meet with the Engineer, Contractor, and Fabricator prior to the beginning of galvanizing but preferably to the

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start of fabrication. At that time, agreements should be finalized on the application of current standards, practices, conditions, and/or allowable variations. Because of its potential influence on galvanized coating development, the selection of steel chemistry should be discussed. Other discussion items should include: inspection criteria, allowable amounts of repair and touch-up techniques, design and fabrication techniques that promote quality coatings, maximum sizes and weights which can be accommodated, potential distortion/warpage following galvanizing, and the amount of pregalvanizing fabrication to be done.

5.1.2. In lieu of any written agreement between Metalplate and the contracting party, all iron and steel products will be galvanized according to the appropriate standards and specifications set forth in Section 3 of this document.

5.2. Preparation of Material for Galvanizing

5.2.1. The engineering firm and/or fabricator are responsible for the selection of iron or steel material that is chemically suitable for galvanizing as described in ASTM A385 Section 3.

5.2.2. The engineering firm and/or fabricator are responsible for the design of the product to be galvanized and fabrication requirements should be in accordance with Class [I] [II] [III] guidelines as described in AGA's publication *Recommended Details for Galvanized Structures* as well as the applicable portions of ASTM A143, A384, and A385.

5.2.3. It is recommended that the engineering firm and/or fabricator consult with Metalplate regarding potential problems during the galvanizing process which may require modification of design before fabrication proceeds.

5.2.4. All material shipped to Metalplate should meet the following conditions:

- All surface contaminants and coatings, such as welding slag, splatter, anti-splatter compounds, burrs, and marking paints which would not be removable by the normal chemical cleaning process, have been removed by blast cleaning or other methods prior to delivery.
- Holes and/or lifting lugs have been provided to facilitate handling through the galvanizing process.
- Whenever possible, slip joints have been used to minimize field welding of material.

5.3. Processing of Material to be Galvanized

5.3.1. Materials received at each Metalplate facility will be unloaded and verified against their packing list. Customers will be notified promptly if discrepancies are found.

5.3.2. Metalplate will not be responsible in the event of material warpage due to the release of stresses during the galvanizing process. The engineering firm and/or fabricator are responsible for insuring that the material to be galvanized is prepared properly according to the applicable portions of ASTM Standards A143, A384, and A385. However, Metalplate does have the ability to straighten some types of material and will do so at the customer's request.

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5.4. Finishing and Shipping of Galvanized Material

5.4.1. Upon request, Metalplate will transfer to the finished product the individual piece mark and/or job number using a permanent type marker. When the job uses the bar code system, the appropriate label will be placed on the material at a location specified by the customer. (This section generally does not apply to Spinner work due to the large numbers of pieces which also tend to be small in size. These two factors make it difficult to individually identify parts. Identification is instead maintained on a lot basis.)

5.4.2. All finishing procedures performed on the galvanized coating will be performed to one of the following six Metalplate finishing codes: A, B, C, D, E, or S (see Addendum 1 for code descriptions). The selection of the finishing code will be clearly defined and mutually agreed upon in writing by the customer and Metalplate prior to the beginning of the work and prior to shipment of the material. (The Metalplate finishing codes as a whole do not apply to Spinner work. This is due to the large numbers of small pieces which would be impossible to work with and finish individually. Instead, the finishing operations associated with Spinner work are understood to be roughly equivalent to finishing code designation C.)

5.4.3. Each truck load of material will be processed as a unit and reloaded as such. No mixing of loads will be performed by Metalplate unless requested by the customer in writing.

5.4.4. All finished material will be reloaded with dunnage to prevent damage during shipment to the jobsite.

6. Galvanizing Process

Hot Dip Galvanizing at Metalplate facilities is accomplished using the following sequence:

6.1. *Degreasing* - Material may be immersed in a hot caustic solution to remove grease, oil, water-based paints and any other stubborn surface contaminants. Other surface contaminants and coatings which can not be removed by normal chemical cleaning procedures shall be removed by the customer. These include but are not limited to: welding slag, splatter, anti-splatter, lacquer and oil-based paints, etc. The caustic solution will be tested at least once per week or more often if necessary and additions made as needed.

6.2. *Inspection* - A visual inspection is performed following the withdrawal of the work from the caustic solution to determine if all surface contaminants and coatings have been removed from the material's surface. Step 6.1 will be repeated as needed.

6.3. *Rinsing* - A water rinse may be performed following degreasing to remove any caustic solution carried out on the work from the caustic bath.

6.4. *Acid Pickling* - Material is immersed in dilute Sulfuric or Hydrochloric acid solution to remove all rust, mill scale and any other surface contaminants. The type of acid solution used is dependent upon the plant location. The acid solution will be tested at least twice per week or more often if necessary and additions made as needed.

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6.5. *Inspection* - A visual inspection of the work is performed to determine if all mill scale and rust have been removed from the material's surface. Step 6.4 will be repeated as needed if the inspection of the work is found to be unsatisfactory.

6.6. *Rinsing* - A water rinse is performed following acid pickling to remove any dissolved iron salts carried out on the work from the acid bath.

6.7. *Fluxing* - During this step, the material is immersed in a heated aqueous zinc-ammonium chloride solution. This procedure is performed to remove any remaining impurities, moisture, or oxide film from the steel and to ensure that the steel surface is chemically clean and ready for galvanizing. Upon removal from the flux solution, the work is air-dried before entering the molten zinc. The fluxing solution will be tested at least once per week by the plant and a detailed analysis will be performed every other week by Metalplate's Control Lab and additions made as needed.

6.8. *Inspection* - Kettle crew members will again inspect the material prior to immersion in the molten zinc to ensure that the material has been properly cleaned from all impurities and that it is free from air pockets which can result in ungalvanized areas.

6.9. *Galvanizing* - The galvanizing kettles at all Metalplate Galvanizing facilities are operated in accordance with the galvanizing industry standard for No-Lead requirements. The term "No-Lead" generally refers to a Lead concentration of 0.06% or less by weight in the working volume of the molten zinc metal contained in the galvanizing bath. The No-Lead requirement is met by specifying the molten zinc feedstock to be High Grade Zinc or better as defined in ASTM B6. During galvanizing, the work is immersed in molten zinc where it will react to form a series of zinc-iron alloy layers. Immersion time varies according to the thickness and weight of the material being galvanized. The material will be withdrawn from the zinc kettle when the coating thickness meets or exceeds the appropriate minimum coating thickness as specified in the relevant ASTM standard. A detailed analysis of the zinc will be performed on a regularly scheduled basis by an independent laboratory.

6.10. *Quenching* - Unless otherwise instructed by the customer, the work will be immersed in a quenching solution following galvanizing. Quenching is performed to minimize the formation of dark, matte gray coatings by stopping the galvanizing reaction. In addition, the freshly galvanized surface is very chemically active and the quenching solution will passivate the surface making it less susceptible to the formation of Wet Storage Stain (White Rust). The quench solution will be tested at least once per week or more often if necessary and additions made as needed.

7. Material Finishing and Cleaning

7.1 The general appearance of the galvanized product should be relatively smooth, continuous and free from gross surface imperfections. The visual examination and surface quality check of all galvanized material includes inspecting for the conditions described in Addenda 1 & 2.

7.2 As mentioned in section 5.4.2 of this document, all finishing of galvanized material will be performed according to an internal system of finishing codes. In the event that a customer has not specified a finishing category or has no preference for a particular finish, Metalplate reserves the right to assign a finishing category that will fit that customer's needs for finishing quality to the best of our knowledge. (As stated in section 5.4.2, all Spinner work is finished to the equivalent of finishing code C.)

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8. Quality Assurance

8.1. Metalplate maintains an in-house system of quality assurance. Prior to shipment, all material will be inspected to ensure that it conforms to applicable requirements and standards. If material fails to meet any specified condition, it will be rejected, documented and handled according to Metalplate's Standard Operating Procedure for nonconforming material.

8.2. Metalplate maintains control programs for process solution and zinc testing as well as calibration, adjustment and maintenance of measuring and test equipment to demonstrate the conformance of product to specific standards. At the customer's request, documentation of the control programs will be made available for inspection.

8.3. At the request of the customer in their purchase order, Metalplate will perform inspections and provide to the customer certified reports which indicate that their material is in compliance with all applicable specifications.

9. Audits

Metalplate's Technical Department personnel perform internal quality system audits to ensure that processes and product quality are in compliance with our Standard Operating Procedures. The audits are performed on a regularly scheduled basis at each Metalplate facility. Following an audit, action plans are developed to eliminate observed discrepancies and to further our quest for continuing improvements.



Adam T. Brown
Vice President
Technical & Environmental Affairs
Metalplate Galvanizing, L.P.

Procedure ID: Hot Dip Galvanizing Procedure
Issued Date: February 26, 1996
Revision Date/Number: April 5, 2012/8
Revised by: Adam T. Brown
Approved by: E. Marty Cain

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HOT DIP GALVANIZING PROCEDURE

ADDENDUM 1

FINISHING CODES

	Work Requirements	Repair Methods	Cleaning Methods
A	• Remove all excessive zinc • Remove and repair all weld slag • Check for straightness • Inspect all welds for cracking • Remove all inclusions (flux, skims, acid) and touch up uncoated areas that are within repairable limits • Inspect all copes for cracking • Check and clean all bolt holes and/or mating surfaces • Remove all sharp points • Properly identify material • Visually inspect galvanizing finish • Prepare material for delivery: stack and/or package • Compliance with ASTM Coating Thickness Standards	• Zinc-Rich Paint • Zinc-Based Alloy • Sprayed Zinc	• Files • Sandpaper
B	• Check for straightness • Inspect all welds for cracking • Remove all inclusions (flux, skims, acid) and touch up uncoated areas that are within repairable limits • Inspect all copes for cracking • Check and clean all bolt holes and/or mating surfaces • Remove all sharp points • Properly identify material • Visually inspect galvanizing finish • Prepare material for delivery: stack and/or package • Compliance with ASTM Coating Thickness Standards	• Zinc-Rich Paint • Zinc-Based Alloy • Sprayed Zinc	• Files • Grinders
C	• Inspect all copes for cracking • Check and clean all bolt holes and/or mating surfaces • Remove all sharp points • Properly identify material • Visually inspect galvanizing finish • Prepare material for delivery: stack and/or package • Compliance with ASTM Coating Thickness Standards	• Zinc-Rich Paint	• Files • Grinders
D	• Remove all sharp points • Properly identify material • Visually inspect galvanizing finish • Prepare material for delivery: stack and/or package • Compliance with ASTM Coating Thickness Standards	• Zinc-Rich Paint	• Files • Grinders
E	• Properly identify material • Visually inspect galvanizing finish • Prepare material for delivery: stack and/or package • Compliance with ASTM Coating Thickness Standards	• Zinc-Rich Paint	
S	• Special Requirements	Category "S" meets specific requirements a customer may have regarding a particular type of product or material.	

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HOT DIP GALVANIZING PROCEDURE
ADDENDUM 2
INSPECTION AND REPAIR GUIDELINES

CONDITION	POSSIBLE CAUSES	ACTION REQUIRED
Sharp Edges/Icicles	• Rapid freezing of run-off zinc. • Rough material contour, and/or inadequate drainage from design or fabrication.	• File or grind smooth.
Bare Spots	• Scale or rust residues. • Residual welding slag. • Overdrying of preflux. • Embedded sand in castings. • Paint, grease or oil residues. • Rolling defects in base steel. • Excess aluminum content in the zinc. • Articles in contact during galvanizing.	• If these areas are within repairable limits as set forth in ASTM A123, repair per ASTM A780. • If these areas are not within repairable limits, the affected workpiece(s) should be handled in accord with the appropriate SOP Finishing Section.
General Roughness	• Overpickling. • Uneven cold working. • Entrapped dross particles. • Original surface condition of steel. • High galvanizing temperature and/or long immersion time.	• None, unless this condition interferes with the intended use of the material in question. If so, file or grind to acceptable smoothness.
Blisters	• Absorbed hydrogen. • Surface defects in steel.	• None.
Lumpiness and Runs	• Molten zinc is "Cold". • Withdrawal speed too high. • Articles in contact during withdrawal. • Run-off from seams, joints, bolt holes,etc.	• None, unless this condition interferes with the intended use of the material in question. If so, remove excess zinc.
Flux Inclusions	• Surface residues on steel. • Stale flux burnt on during dipping. • Flux picked up from surface of molten zinc.	• If these areas are within repairable limits as set forth in ASTM A123, repair per ASTM A780. • If these areas are not within repairable limits, the affected workpiece(s) should be handled in accord with the appropriate SOP Finishing Section.
Skims Inclusions	• Skims burnt on during dipping. • Skims picked up from surface of molten zinc.	• If these areas are within repairable limits as set forth in ASTM A123, repair per ASTM A780. • If these areas are not within repairable limits, the affected workpiece(s) should be handled in accord with the appropriate SOP Finishing Section.
Dull Gray Coating or Mottled Appearance	• Slow cooling after galvanizing. • Steel composition or severe cold work.	• None.
Rust Stains	• "Weeping" of acid, etc., from seams & folds. • Storage on or near rusty material.	• None. • Clean off surface stain.
Wet Storage Stain	• Confinement of close-packed articles under damp conditions.	• Depends upon severity - contact foreman.

**** Any or all of these requirements may be modified by prior written agreement between Metalplate and the customer in the form of a selected finishing code.***

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WORLD CERTIFICATION SERVICES

CERTIFICATE OF REGISTRATION

Certificate Number

7685

This Certificate has been awarded to:

Accu-Test Labs LTD

7821 Pinemont Drive
Houston, TX 77040
USA

Date First Registered

3 May 1999

In Recognition of the Organization's Management System which complies with:

ISO 9001:2015

Date Certificate Issued

3 March 2017

Date Certificate Expires

3 April 2020

For the Scope of Activities described below:

Metallurgical and Non – Destructive Testing.

This Certificate has been awarded by

WORLD CERTIFICATION SERVICES LTD.

World Certification Services Ltd.
Station Court, Ormskirk Road
Aintree, Liverpool, L9 5AA
United Kingdom

Issued By: W.E. Holcomb

Revision Date: **3 March 2017**
Revision 0

MEMBER OF MULTILATERAL RECOGNITION ARRANGEMENT IAF

The use of the accreditation mark indicates accreditation in respect of those activities covered by UKAS accreditation certificate number 60 and / or ANAB accreditation certificate number WCS - 99024139

SCOPE OF ACCREDITATION TO ISO/IEC 17025:2017[1]

ELEMENT MATERIALS TECHNOLOGY HOUSTON LLC
14805 Yorktown Plaza Drive
Houston, TX 77040
Daisy Gaona Phone: 713 692 9151

MECHANICAL

Valid To: December 31, 2020 Certificate Number: 1480.05

In recognition of the successful completion of the A2LA evaluation process, accreditation is granted to this laboratory to perform the following tests on fasteners, metals, alloys, and non-metallics:

Test:	Test Method(s):
Chemical	
Combustion Analysis (H, C, N, O, S)	ASTM E1019, ASTM E1409, ASTM E1447; SOP MET-30.03
Optical Emission Spectroscopy (Al, B, Be, Bi, C, Cd, Co, Cr, Cu, Fe, Mn, Mo, N, Nb, Ni, P, Pb, S, Sb, Si, Ta, Ti, V, W, Zn, Zr)	ASTM E415, ASTM E1086, ASTM E1251, ASTM E1999, ASTM E3047; SOP MET-30.01 ASTM A751[1] ASTM E322[1], ASTM E522[1], ASTM E1245[1]
Corrosion	
Intergranular Attack	ASTM A262, ASTM G28; ASTM A923[1], ASTM G48[1]
Pitting	ASTM G48
Crevice	ASTM G48
Hydrogen-Induced Cracking	NACE TM0284
Sulfide Stress Cracking	NACE TM0177, TM0316
Slow Strain Rate	NACE TM0198
Salt Spray[1]	ASTM B117[1]; BS EN ISO 9227[1]
Failure Analysis	ASM Handbook, Volume 11
Hardness	
Brinell (1500[2], 3000 lbs)	ASTM A370, ASTM E10, ISO 6506[2]
Rockwell (A[2], B, C and 15N)	ASTM A370, E18; AWS B4.0 (Clause 8); BS EN ISO 6508[2]
	ISO 15156-1, 898-1
Knoop (100 g, 500 g)	ASTM E92, ASTM E384
Vickers (100 g, 500 g, 5 kg, 10 kg)	ASTM E92, ASTM E384; ISO 15614-1, 15156-1, ISO 6507[1]
Magnetic Permeability	ASTM A342, A342M; MIL-I-17214
Metallographic Examination	
Preparation of Metallographic Specimens	ASTM E3; BS EN 3114-001[2]
Alpha Case	SOP MET-40.24; GE P3TF19, P3TF32
Carburization/Decarburization (Hardness, Optical)	ASTM E1077, ASTM F2328, ASTM F2328M; ISO 898-1; SAE J121
Coating Thickness (Cross Sectioning)	ASTM B487
Coating Weight	ASTM A90/A90M

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Test:	Test Method(s):
Metallographic Examination (Continued) Macroetch Microetch Grain Size Banding / Microstructure Inclusion Content Volume Fraction Microstructure of Graphite in Iron Casting	 ASME IX; ASTM A604, ASTM E340, ASTM E381; AWS D1.2/D1.2M (Clause 3), D1.5M/D1.5 (Clause 5), D1.6/D1.6M (Clause 4), D14.1/D14.1M (Clause 9); ISO 15614-1, AWS D1.1/D1.1M[2], API 1104[2], EN 287[2], 288[2], 1321[2] ASTM E407, ASTM E3, BS EN 3114-001[2] ASTM E112, ASTM E930, ASTM E1181 ASTM E1268; ASM Metals Handbook, Vol. 9 ASTM E45, Methods A & D ASTM E562 ASTM A247
Detrimental Intermetallic Phase Determination in Duplex Stainless Steel	ASTM A923
SEM Electron Microscopy	ASM Handbook, Vol. II
Energy Dispersive Spectroscopy (Semi-quantitative)	ASTM E1508
Bend	API 1104; ASME IX; ASTM A370; ISO 15614-1; AWS B4.0 (Clause 6), D1.1 (Clause 4), D1.2 (Clause 3), D1.5 (Clause 5), D1.6 (Clause 4), ISO 5173[1], ABS Rules[2], DNV Rules[2]
Fillet Weld Break Test[2]	ASME IX[2], AWS D1.1/D1.1/M[2], ISO 9606-1[2]
Compression	ASTM E9
Cyclic Straining	DNV-OS-F101
Drop Weight Testing	ASTM E208
Flattening	ASTM A370
Fracture Toughness	API 1104; ASTM E399, ASTM E1290, ASTM E1820; BS 7448 Pts 1, 2, & 4, BS 8571; DNV-OS-F101 (SENB Geometry Jcritical), DNV-RP-F108 (SENT Geometry Jcritical & JRcurve - multi-specimen); ISO 12135, ISO 12737, ISO 15653
Nick-Break	API 1104
Ring Flattening[2]	ASTM A53/A53M[2], ASTM A530/A530M[2], ASTM A999/A999M[2], API 5L[2], ASME SA530[2]
Impact (Charpy) – Metals (R.T to -320°F)	API 1104; ASME VIII, IX, B31.3; ASTM A370, E23; AWS B4.0 (Clause 7), D1.1 (Clause 4), D1.5 (Clause 5); ISO 148-1
Proof Load (Internal/External Threads)	ASTM A370, F606/F606M; ISO 898-1
Tensile Machined (Round & Flat) Fastener (Wedge / Axial)	 API 1104; ASME IX; ASTM A370, B557, E8/E8M, E21 (Up to 400°F), F606/F606M; AWS B4.0 (Clause 5), D1.1, D1.2 (Clause 3), D1.5 (Clause 5), D1.6 (Clause 4); ISO 15614-1, 898-1; ASTM A770/A770M[2], ABS Rules[2], BS 4515[2], DNV Rules[2], EN ISO 6892-1[2] ASTM A370, F606/F606M; ISO 898-1

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Test:	Test Method(s):
Non-Metallics	
Tensile Rubber Plastic Fiberglass (Room Temperature)	 ASTM D412 ASTM D638 ASTM D3039/D3039M
Flexural Properties of Unreinforced and Reinforced Plastics and Electrical Insulating Materials	ASTM D790
Izod Pendulum Impact Resistance of Plastics	ASTM D256
Hardness Shore A Barcol	 ASTM D2240 ASTM D2583

[1,2] his laboratory is also accredited to perform this test at the following Element Houston field laboratory:

Element Materials Technology Houston - Regal Row
9925 Regal Row
Houston, TX 77040
A2LA Certificate No. 1283.01

[1]This laboratory also meets the requirements of ISO/IEC 17025:2005.

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Accredited Laboratory

A2LA has accredited

ELEMENT MATERIALS TECHNOLOGY HOUSTON LLC

Houston, TX

for technical competence in the field of

Mechanical Testing

This laboratory is accredited in accordance with the recognized International Standard ISO/IEC 17025:2017 *General requirements for the competence of testing and calibration laboratories*. This accreditation demonstrates technical competence for a defined scope and the operation of a laboratory quality management system (*refer to joint ISO-ILAC-IAF Communiqué dated April 2017*).

Presented this 22nd day of January 2019.

Senior Director, Accreditation Services
For the Accreditation Council
Certificate Number 1480.05
Valid to December 31, 2020

For the tests to which this accreditation applies, please refer to the laboratory's Mechanical Scope of Accreditation.

SCOPE OF ACCREDITATION TO ISO/IEC 17025:2017[3]

ELEMENT MATERIALS TECHNOLOGY HOUSTON LLC
14805 Yorktown Plaza Drive
Houston, TX 77040
Daisy Gaona Phone: 713 692 9151

NONDESTRUCTIVE

Valid To: December 31, 2020 Certificate Number: 1480.06

In recognition of the successful completion of the A2LA evaluation process, accreditation is granted to this laboratory at the location listed above, ***as well as the field laboratory location listed below***, to perform the following tests under the ASTM recommended practice E543:

Test:	Test Method(s):
Liquid Penetrant[1] Visible Fluorescent	ASTM E165, E1417; ANSI/ASME B31.1, B31.3; API 1104, 6A; ASME Section V, Article 6; AWS D1.1 (Clause 6 Part C, Clause 9 Part F)
Magnetic Particle[1] Yoke – Visible and/or Fluorescent (Wet and/or Dry) Bench – Visible and/or Fluorescent	ASTM E709, E1444; ANSI/ASME B31.1, B31.3; API 1104, 6A; ASME Section V, Article 7; MIL STD 1949; AWS D1.1 (Clause 6 Part C, Clause 9 Part F)
Radiographic[1, 2] X-ray Gamma	ASTM E94; ANSI/ASME B31.1, B31.3; API 1104, 6A; ASME Section V, Article 2; AWS D1.1 (Clause 6 Part C & E, Clause 9 Part F)
Ultrasonic[1] Contact Straight Beam Contact Angled Beam Phased Array	ASTM A388, A578, E114, E164, E213, E273, E428, E587, E797; ASME Section V, Article 4, 5, 23; AWS D1.1, (Clause 6 Part C & F, Clause 9 Part F), D1.5 (Clause 6, Inspection)
Visual Inspection[1]	ASME Section V, Article 6; AWS D1.1 (Clause 6, Inspection)
X-Ray Fluorescence (PMI)[1]	NDE-60.18

[1] This laboratory meets A2LA *R104 – General Requirements: Accreditation of Field Testing Laboratories* for these tests.

[2] This laboratory is also accredited to perform this test at the following Element Houston field laboratory:

Enerflex
10815 Telge Road
Houston, TX 77095

[3]This laboratory also meets the requirements of ISO/IEC 17025:2005.

(A2LA Cert. No. 1480.06) 02/05/2019 Page 1 of 1

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Sterling, VA 20166
www.reinforcedearth.com

839

A SUBSIDIARY OF SOLETANCHE FREYSSINET



Accredited Laboratory

A2LA has accredited

ELEMENT MATERIALS TECHNOLOGY HOUSTON LLC

Houston, TX

for technical competence in the field of

Nondestructive Testing

This laboratory is accredited in accordance with the recognized International Standard ISO/IEC 17025:2017 *General requirements for the competence of testing and calibration laboratories*. This accreditation demonstrates technical competence for a defined scope and the operation of a laboratory quality management system (*refer to joint ISO-ILAC-IAF Communiqué dated April 2017*).

Presented this 22nd day of January 2019.

Senior Director, Accreditation Services
For the Accreditation Council
Certificate Number 1480.06
Valid to December 31, 2020

For the tests to which this accreditation applies, please refer to the laboratory's Nondestructive Scope of Accreditation.

V.LOUISON & CIE ZI du Coin BP 87 42402 ST CHAMOND Cedex FRANCE	**CERTIFICAT DE CONFORMITE** CERTIFICATE OF COMPLIANCE **AUX STIPULATIONS D'UNE COMMANDE ETABLIE PAR L'INDUSTRIEL** With the order received established by the industrial **FOURNISSEUR. CONFORMEMENT A LA NORME :** supplier conformed to standards : **NF L 00-015**	N° : 043/013 Page n° : 1/1

Raison sociale du client: THE REINFORCED EARTH CY
(Client's company name) 12001 SUNRISE VALLEY DRIVE – SUITE 400 RESTON – VA 20191 - USA

Numéro de la commande : 41840
(Order number)
Renseignements particuliers aux fournitures livrées,
(Particular information concerning the delivred supplies)

Désignation: Bande extrudée type GEOSTRAP 5 / 65 kN
(Désignation : Extruded Polymèric strip GEOSTRAP 5/ 65 kN)

Référence : 00433
(Ref)

Numéro de lot: 40196 – 40217 – 40218 – 40219
(Run number)
Remarques : bobines de 1 à 150 (40196) 1 à 290 – 311 à 330 – 371 à 430 – 471 à 490 (40217) 78 à 137 –
(Remarks) 158 à 500 (40218) 1 à 217 (40219)

Critères *(criteria)*	Méthode essai *(Test Method)*	Spécifications	Moyenne *(mean)*	Ecart type *(standard deviation)*
Largeur / *Width* (mm)	Internal method	$49.5^{+/-0.5}$	49.46	0.229
Poids/*Weight* PEBD (grs)	Internal method	87	91.74	2.359
Résistance/*Strenght* (kN)	EN10319 / ASTM D6637	65	68.80	0.647
Allongement rupture/elongation at break (%)	EN10319 / ASTM D6637	$12^{+/4}$	10.48	0.641
Carboxyl (mmol/Kg)	GRI-GG7	< 30	18.3	0.3
Molecular Weigth (Mn)	GRI-GG8 or équivalent	>25.000	42.335	-

Numéro du Bon de Livraison : 57 5211 du 26/11/2013
(Delivery slip number)

Quantité: 118 000 mètres (1180 bobines /*coils*)
(Quantity)

Nous certifions que sauf exceptions et dérogations énumérées ci-dessus, la fourniture citée a été fabriquée, contrôlée,
(We certify that except the exceptions and derogation quoted above,the quoted supply has been built and controlled)
conformément aux spécifications techniques du marché, commande ou sous commande du client et que toutes les opérations et
(according to the technical specifications of the contrat,the client's order or under order and that the operation)
essais de contrôle ont été effectués. Elle répond sous tous ses aspects aux qualifications particulières du CDC version G du 25/05/2012
(control have been done.This order considering all its aspects, corresponds to the particular qualification of the CDC version G dated the 25/05/2012)

CONTRÔLE FOURNISSEUR:
(Supplier Control)

Nom (Name) : H.GAILLARD Service : QUALITE (Quality)

Date : 3 janvier, 2014 Visa:


Accreditation #:
GAI-LAP - 01 - 1995
TEL (610) 522-8440
Geosynthetic Institute
475 Kedron, Ave.
Folsom, PA 19033
TRI Environmental, Inc.
is granted accreditation
for designated geosynthetic test methods in accordance with the
Geosynthetic Accreditation Institute - Laboratory Accreditation Program
(GAI-LAP), as published in its annual directory.
This accreditation is valid until June 30, 2019.
Robert M. Koerner, Ph.D., P.E.
Director Emeritus
GRI
GEI
GII
GSI
GAI
GCI
George R. Koerner, Ph.D., P.E. & CQA
Auditor

Geosynthetic Institute
475 Kedron Avenue
Folsom, PA 19033-1208 USA
TEL (610) 522-8440
FAX (610) 522-8441



June 14, 2018

Mr. Jarrett Nelson
TRI/ Environmental Inc.
9063 Bee Caves Road
Austin, TX 78733-6201

Re: GAI-LAP Accreditation

Dear Mr. Nelson,

The Geosynthetic Institute (GSI) is pleased to acknowledge TRI/ Environmental Inc. on its repertoire of Geosynthetic Accreditation Institute's-Laboratory Accreditation Program (GAI-LAP) accredited tests. This letter should serve as notification that TRI/Environmental Inc. located in Austin, TX is currently accredited for the following one hundred and sixty-one test methods until June 30, 2019.

1. ASTM D413 Test Methods for Rubber Property-Adhesion to Flexible Substrate
2. ASTM D570 Test Method for Water Absorption of Plastics
3. ASTM D638 Test Method for Tensile Properties of Plastics
4. ASTM D696 Test Method for Coefficient of Linear Thermal Expansion of Plastics
5. ASTM D737 Test Method for Air Permeability of Textiles
6. ASTM D746 Test Method for Brittleness Temperature of Plastics and Elastomers by Impact
7. ASTM D751 Test Methods for Coated Fabrics (thickness), (mass/unit area), (tongue tear), (grab), (hydrostatic resistance) and/or (bonded seam strength)
8. ASTM D790 Test Method for Flexural Properties of Unreinforced and Reinforced Plastics and Electrical Insulating Materials
9. ASTM D792 Test Method for Specific Gravity (Relative Density) and Density of Plastics by Displacement
10. ASTMD882 Test Methods for Tensile Properties of Thin Plastic Sheeting (strip tensile)
11. ASTM D1004 Test Method for Initial Tear Resistance of Plastic Film and Sheeting
12. ASTM D1203 Test Method for Volatile Loss from Plastics Using Activated Carbon Methods
13. ASTM D1204 Test Method for Linear Dimensional Changes of Nonrigid Thermoplastic Sheeting or Film at Elevated Temperature

14. ASTM D1238 Test Method for Flow Rates of Thermoplastics by Extrusion Plastometer
15. ASTM D1388 Test Method for Stiffness of Fabrics
16. ASTM D1505 Test Method for Density of Plastics by the Density-Gradient Technique
17. ASTM D1593 Specification for Nonrigid Vinyl Chloride Plastic Sheeting (thickness)
18. ASTM D1603 Test Method for Carbon Black in Olefin Plastics
19. ASTM D1621 Test Method for Compressive Properties of Rigid Cellular Plastics
20. ASTM D1693Test Method for Environmental Stress-Cracking of Ethylene Plastics
21. ASTM D1709 Standard Test Methods for Impact Resistance of Plastic Film by the Free-Falling Dart Method
22. ASTM D1777 Test Method for Measuring Thickness of Textile Materials
23. ASTM D 1790 Brittleness Temperature by Impact (PVC)
24. ASTM D1822 Standard Test Method for Tensile-Impact Energy to Break Plastics and Electrical Insulating Materials
25. ASTM D1922 Standard Test Method for Propagation Tear Resistance of Plastic Film and Thin Sheeting by Pendulum Method
26. ASTM D1987 Test Method for Biological Clogging of Geotextile or Soil Geotextile Filters
27. ASTM D2122 Test Method for Determining Dimensions of Thermoplastic Pipe and Fittings
28. ASTM D2136 Test Method for Coated Fabrics-Low Temperature Bend Test
29. ASTM D2240 Test Method for Measuring Durometer Hardness
30. ASTM D2256 Standard Test Method for Tensile Properties of Yarns by the Single-Strand Method
31. ASTM D2412 Test Method for Determination of the External Loading Characteristics of Plastic Pipe by Parallel-Plate Loading
32. ASTM D2444 Test Method for Determination of the Impact Resistance of Thermoplastic Pipe Fittings by Means of a Tup (Falling Weight)
33. ASTM D3015 Standard Practice for Microscopic Examination of Pigment Dispersion in Plastic Compounds
34. ASTM D3030 Test Method for Volatile Matter (Including Water) of Vinyl Chloride Resins
35. ASTM D3083 Standard Specification for Flexible Poly (Vinyl Chloride) Plastic Sheeting for Pond, Canal, and Reservoir Lining, (soil burial), (water extraction) and/or (bonded seam strength)
36. ASTM D3350 Specification for Polyethylene Plastic Pipe and Fittings Materials
37. ASTM D3776 Test Method for Mass Per Unit Area (Weight) or Woven Fabric
38. ASTM D3786 Test Method for Hydraulic Burst Strength of Knitted Goods and Nonwoven Fabrics (Diaphragm Bursting Strength Tester Method)
39. ASTM D3787 Test Method for Bursting Strength of Textiles-Constant-Rate-of-Traverse (CRT) Ball Burst Test
40. ASTM D3895 Test Methods for Oxidative-Induction Time of Polyolefins by Differential Scanning Calorimetry

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41. ASTM D4218 Test Method for Carbon Black Content in Polyethylene Compounds by the Muffle-Furnace Technique
42. ASTM D4355 Test Method for Determination of Geotextiles from Exposure to Ultraviolet Light and Water (Xenon-Arc Type Apparatus)
43. ASTM D4491 Test Methods for Water Permeability of Geotextiles by Permittivity
44. ASTM D4533 Test Method for Index Trapezoidal Tearing Strength of Geotextiles
45. ASTM D4595 Test Method for Tensile Properties of Geotextiles by the Wide-Width Strip Method
46. ASTM D4603 Test Method for Determining Inherent Viscosity of Poly (Ethylene Terephthalate) (PET)
47. ASTM D4632 Test Method for Grab Breaking Load and Elongation of Geotextiles
48. ASTM D4703 Standard Practice for Compression Molding Thermoplastic Materials into Test Specimens, Plaques, or Sheets
49. ASTM D4716 Test Method for Determining the (In-Plane) Flow Rate per Unit Width and Hydraulic Transmissivity of a Geosynthetic Using a Constant Head
50. ASTM D4751 Test Method for Determining Apparent Opening Size of a Geotextile
51. ASTM D4833 Test Method for Index Puncture Resistance of Geotextiles, Geomembranes and Related Products
52. ASTM D4884 Test Method for Seam Strength of Sewn Geotextiles
53. ASTM D4885 Test Method for Determining Performance Tensile Strength of Geomembranes Using Wide Strip Testing
54. ASTM D4886 Test Method for Abrasion Resistance of Geotextiles (Sand Paper/Sliding Block Method)
55. ASTM D5035 Test Method for Breaking Strength and Elongation of Textile Fabrics (Strip Method)
56. ASTM D5101 Test Method for Measuring the Soil-Geotextile System Clogging Potential by the Gradient Ratio
57. ASTM D5199 Test Method for Measuring Nominal Thickness of Geotextiles and Geomembranes
58. ASTM D5261 Test Method for Measuring Mass per Unit Area of Geotextiles
59. ASTM D5262 Test Method for Evaluating the Unconfined Tension Creep Behavior of Geosynthetics
60. ASTM D5321 Test Methods for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method,
61. ASTM D5322 Standard Practice for Immersion Procedures for Evaluating the Chemical Resistance of Geosynthetics to Liquids
62. ASTM D5323 Determination of 2% Secant Modulus for Polyethylene Geomembranes
63. ASTM D5397 Test Method for Evaluation of Stress Crack Resistance of Polyolefin Geomembranes using Notched Constant Tension Load Test
64. ASTM D5493 Test Method for the Permittivity of Geotextiles Under Load
65. ASTM D5494 Test Methods for the Determination of Pyramidal Puncture Resistance of Unprotected and Protected Geomembranes
66. ASTM D5514 Test Method for Large Scale Hydrostatic Puncture Testing of Geosynthetics

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67. ASTM D5596 Test Methods for Microscopic Evaluation of the Dispersion of Carbon Black in Polyolefin Geosynthetics
68. ASTM D5617 Test Methods for Multi-Axial Tension Test for Geosynthetics
69. ASTM D5721 Standard Practice for Air-Oven Aging of Polyolefin Geomembranes
70. ASTM D5747 Standard Practice for Tests to Evaluate the Chemical Resistance of Geomembranes to Liquids
71. ASTM D5818 Standard Practice for Obtaining Samples of Geosynthetics from a Test Section for Assessment of Installation Damage
72. ASTM D5884 Test Method for the Tearing Strength of Internally Reinforced Geomembranes
73. ASTM D5885 Standard Test Method for Oxidative Induction Time of Polyolefin Geosynthetics by High-Pressure Differential Scanning Calorimetry
74. ASTM D5887 Standard Test Method for Measurement of Index Flux Through Saturated Geosynthetic Clay Liners Specimens Using a Flexible Wall Permeameter
75. ASTM D5890 Standard Test Method for Swell Index of Clay Mineral Component of Geosynthetic Clay Liners
76. ASTM D5891 Standard Test Method for Fluid Loss of Clay Component of Geosynthetic Clay Liners
77. ASTM D5993 Test Method for Measuring the Mass Per Unit Area of GCL
78. ASTM D5994 Test Method for Measuring the Core Thickness of Textured Geomembranes
79. ASTM D6140 Test Method for Determine of Asphalt Retention of Paving Fabrics Used in Asphalt Paving for Full Width Applications
80. ASTM D6213 Standard Practice for Tests to Evaluate the Chemical Resistance of Geogrids to Liquids
81. ASTM D6214 Test Method for Determining the Integrity of Field Seams Used in Joining Geomembranes by Chemical Fusion Methods
82. ASTM D6241 Test Method for the Static Puncture Strength of Geotextiles and Geotextile Related Products Using a 50-mm Probe
83. ASTM D6243 Test Method for Determine the Internal and Interface Shear Resistance of Geosynthetic Clay Liners by the Direct Shear Method
84. ASTM D6364 Test Method for Determining the Short-Term Compression Behavior of Geosynthetics
85. ASTM D6389 Standard Practice for Tests to Evaluate the Chemical Resistance of Geotextiles to Liquids
86. ASTM D6392 Standard Test Method for Determining the Integrity of Nonreinforced Geomembrane Seams Produced Using Thermo-Fusion Methods
87. ASTM D6454 Standard Test Method for Determining the Short-Term Compression Behavior of Turf Reinforcement Mats (TRMs)
88. ASTM D6475 Test Method for Measuring Mass Per Unit Area of Erosion Control Blankets
89. ASTM D6496 Test Method for Determining the Average Bonding Peel Strength Between Top and Bottom Layers of Needle-Punched Geosynthetic Clay Liners
90. ASTM D6524 Test Method for Measuring Resiliency of Turf Reinforcement Mats (TRMs)

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91. ASTM D6525 Test Method for Measuring Nominal Thickness of Permanent Erosion Control Products
92. ASTM D6566 Test Method for Measuring Mass per Unit Area of Turf Reinforcement Mats (TRMs)
93. ASTM D6567 Test Method for Measuring Light Penetration of Turf Reinforcement Mat (TRM)
94. ASTM D6574 Test Method for Determining the In-plane Hydraulic Transmissivity of a Geosynthetic by Radial Flow
95. ASTM D6575 Test Method for Determining Stiffness of Geosynthetics used as Turf Reinforcement Mats
96. ASTM D6636 Test Method for Determination of Ply Adhesion Strength of Reinforced Geomembranes
97. ASTM D6637 Test Method for Determining Tensile Properties of Geogrids by the Single or Multi-Rib Tensile Method
98. ASTM D6638 Test Method for Determining Connection Strength Between Geosynthetic Reinforcement and Segmental Concrete Units (Modular Concrete Blocks)
99. ASTM D6693 Test Method for Determining Tensile Properties of Nonreinforced Polyethylene and Nonreiforced Flexible Polypropylene Geomembranes
100. ASTM D6706 Test Method for Determining Pull-Out Resistance of Geosynthetics
101. ASTM D6766 Test Method for Evaluation of Hydraulic Properties of Geosynthetic Clay Liners Permeated with Potentially Incompatible Liquids
102. ASTM D6767 Test Method for Pore Size Characteristics of Geotextiles by Capillary Flow Test
103. ASTM D6768 Test Method for Tensile Strength of Geosynthetic Clay Liners
104. ASTM D6818 Test Method for Ultimate Tensile Properties of Turf Reinforcement Mats
105. ASTM D 6916 Geogrid Pull-out from Segmental Concrete walls.
106. ASTM D6918 Standard Test Method for Testing Vertical Strip Drains in the Crimped Condition
107. ASTM D6992 Test Method for Time-Temperature Superposition Using Stepped Isothermal Method
108. ASTM D7003 Test Method for Strip Tensile Properties of Reinforced Geomembranes
109. ASTM D7004 Test Method for Grab Tensile Properties of Reinforced Geomembranes
110. ASTM D7005 Test Method for Determining the Bond Strength (Ply Adhesion) of Geocomposites
111. ASTM D7056 Test Method for Determining the Tensile Shear Strength of Prefabricated Bituminous Geomembrane Seams
112. ASTM D7101 Test Method for RECP slope
113. ASTM D7179 Test Method for Determining Geonet Breaking Force
114. ASTM D7207 Test Method for RECP shear
115. ASTM D7238 Standard Test Method for Effect of Exposure of Unreinforced Polyolefin Geomembrane Using Fluorescent UV Condensation Apparatus

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116. ASTM D7272 Test Method for Determining the Integrity of Seams Used in Joining Geomembranes by Pre-manufactured Taped Methods
117. ASTM D7275 Test Method for Tensile Properties of Bituminous Geomembranes (BGM)
118. ASTM D7322 Test Method for RECP germination
119. ASTM D7361 Test Method for Compressive SIM
120. ASTM D7406 Test Method for Time-Dependent (Creep) Deformation Under Constant Pressure for Geosynthetic Drainage Products
121. ASTM D7409 Test Method for CEG of PET
122. ASTM D 7447 RGM-Seams Strip Method
123. ASTM D 7448 Flexural Rigidity of Geogrid
124. ASTM D 7449 RGM-Seams - Grab Method
125. ASTM D7466 Test Method for Asperity Height
126. ASTM D 7737 Junction Strength Geogrid
127. ASTM E96 Test Method for Water Vapor Transmission of Materials
128. ASTM F904 Test Method for Comparison of Bond Strength or Ply Adhesion of Similar Laminates Made from Flexible Materials
129. ASTM F1249 Standard Test Method for Water Vapor Transmission Rate Through Plastic Film and Sheeting Using a Modulated Infrared Sensor
130. ASTM F1473 Test Method for Notched Tensile Test to Measure the Resistance to Slow Crack Growth of Polyethylene Pipes and Resins
131. ASTM F2136 Test Method for Notched, Constant-Ligament-Stress (NCLS) Test to Determine Slow Crack-Growth Resistance of HDPE Resins and HDPE Corrugated Pipe
132. FTM STD. No. 101c(method 2065-82), Puncture Resistance and Elongation Test (1/8 in. radius probe)
133. GRI GG-1 Geogrid Rib Tensile Strength
134. GRI GG-2 Geogrid Junction Strength
135. GRI GG-7 Carboxyl End Group Content of PET Yarns
136. GRI GG-8 Determination of the Number Average Molecular Weight of PET Yarns Based on Relative Viscosity Value
137. GRI GM-11 Accelerated Weathering of Geomembranes Using a Fluorescent UVA Device
138. GRI GS-7 Determining the Index Friction Properties of Geosynthetics
139. ISO 527 Plastics – Determination of Tensile properties – Part 3: Test conditions for films and sheets
140. ISO 9863 Thickness
141. ISO 9864 Geotextiles - Determination of mass per unit area
142. ISO 10319 Geotextiles - Wide width tensile test
143. ISO 10321 Wide Width Seams
144. ISO 10722 Geotextiles and geotextiles - related products – Procedure for simulating damage during installation Part 1: Installation in granular materials
145. ISO 11058 Geotextiles and geotextile-related products -- Determination of water permeability characteristics normal to the plane, without load
146. ISO 12225 Geotextiles and geotextiles - related products – Screening test method for determining the resistance to microbiological degradation

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147. ISO 12236 Geotextiles and geotextiles - related products – Static puncture test (CBR test)
148. ISO 12447 Geotextiles and geotextiles - related products – Screening test method for determining the resistance to hydrolysis
149. ISO 12956 Geotextiles and geotextiles - related products – Determination of the characteristic opening size
150. ISO 12957 Geosynthetics – Determination of friction characteristics Part 1: Direct shear test, Part 2: Inclined plane test
151. ISO 12958 Geotextiles and geotextiles - related products – Determination of water flow capacity in their plane
152. ISO 12960 Geotextiles and geotextiles - related products – Screening test method for determining the resistance to liquids
153. ISO 13426 Geotextiles and geotextiles - related products – Strength of internal structural junctions – Part 1: Geocells Part 2: Geocomposites
154. ISO 13431 Geotextiles and geotextiles – related products - Determination of the tensile creep and creep rupture behavior
155. ISO 13433 Geosynthetics -- Dynamic perforation test (cone drop test)
156. ISO 13438 Geotextiles and geotextiles - related products – Screening test method for determining the resistance to oxidation
157. ISO 13934 Textiles -- Tensile properties of fabrics -- Part 1: Determination of maximum force and elongation at maximum force using the strip -- Part 2: Determination of maximum force using the grab method
158. EN 12224 Geotextiles and geotextile-related products –. Determination of the resistance to weathering.
159. EN 14030 Geotextiles and geotextiles-related products – Screening test method for determining the resistance to liquids
160. Germany: Federwal Waterway Engineering and Research Institute (BAW) RPG - - Guidelines for Testing Geotextiles for Navigable Waterways: Section 3.11 Abrasion resitance
161. CAN/CGSB-148.1–10 Filtration Opening Size of Geotextiles by Hydrodynamic Sieving

Any questions regarding your accreditation should be directed to George or Robert Koerner at (610) 522-8440. Once again congratulation and thank you for participating in the GAI-LAP.

Best Regards,

George R. Koerner, Ph.D., P.E. & CQA
Director

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SureBuilT Manufacturing
Serving the Construction Industry!
840 South 25th Ave, Bellwood IL 60104 • 708-493-9569 • Fax 708-493-9601

CERTIFICATE OF COMPLIANCE

Product Type - Ladders

TO:
The Reinforced Earth Company
12001 Sunrise Valley Drive, Suite 400
Reston, VA 20191

Date: 7/30/2013

With this letter SureBuilt Manufacturing certifies that the LADDERS supplied to the Reinforced Earth Company has been manufactured in the United States in conformity to the "Buy America Act".

This letter also certifies that all materials such as iron and steel have been melted and produced in the United States and in conformance of the American Recovery and Reinvestment Act of 2009 (ARRA) or "Buy American"

SureBuilt Manufacturing certifies all material complies with:

WIRE ROD: ASTM A-82 cold-drawn steel wire, Minimum Yield Strength = 65,000 psi (450 Mpa), Minimum Tensile Strength = 75,000 psi (515 Mpa), Minimum Weld Shear Strength = 35,000 psi and Galvanized to ASTM A123 as applicable.

I hereby certify that the statements provided in this Certificate of Compliance are true and correct to the best of my knowledge.

Quality Manager
SureBuilt Manufacturing

7/31/2013
Date

King Steel Corporation
King Steel
5225 East Cook Rd.
Grand Blanc, MI 48439
Tel 810-953-7637
Fax 810-953-1718

Material Certification

Customer: Surebuilt (Midwest Concrete & Masonry)
Ship To: Midwest Concrete & Masonry
Customer PO No: 178230
King Steel Order No: 29568
Order Line: 1
King Steel Part No: 0.406 1018 IQ HR RD ROD
Customer Part No: SBWIREROD0.406
Shipped Qty: 46745
Heat: NF13100207
Grade: 1018
Note:

Material Specification Type	Material Specification	Actual
Chemical	Carbon (C)	.17 %
	Manganese (Mn)	.81 %
	Phosphorus (P)	.016 %
	Sulfur (S)	.025 %
	Silicon (Si)	.28 %
	Nickel (Ni)	.07 %
	Chromium (Cr)	.10 %
	Molybdenum (Mo)	.02 %
	Aluminum (Al)	.001 %
	Copper (Cu)	.23 %
	Titanium (Ti)	.001 %
	Vanadium (V)	.003 %
	Columbium/Niobium (Nb)	.002 %
	Tin (Sn)	.010 %
Physical	Tensile Strength (PSI)	76616 psi
	Yield Strength (PSI)	58075 psi
	% Elongation	24 %
	Reduction Ratio:	338:1
	Melted & Manufactured in:	USA

We hereby certify that chemical analysis and/or physical characteristics shown are a true copy of orignal test reports on file with us from the producing source covering the heat or lot from which this material was taken.

Plex Online 7/9/13 11:54 AM nebler Page 1

Mill Certification Details

NUCOR

NUCOR CORPORATION
NUCOR STEEL NEBRASKA

Mill Certification Details - 7/9/2013 11:56 AM

Purchase Order #: 013614-ET-R
Customer: KING STEEL CORP - NORFOLK
Bill of Lading #: 257215
Chief Metallurgist : Jim Hill
Lot # : NF1310020711
Product : RDC
Grade: 1018
Customer Part # :
Comments :

Length: 0'0"
Date : 1/14/2013
Tag # : NF1311084110
Size : 13/32" (.4063) Wire Rod
Division : Norfolk, NE
Heat #: NF13100207

Chemical Properties -Wt.%

C	Mn	Si	S	P	Cu	Cr	Ni	Mo	Al	V	Nb	Pb	Sn	Ti
0.17	0.81	0.28	0.025	0.016	0.23	0.10	0.07	0.02	0.001	0.003	0.002	0.000	0.010	0.001

Carbon Equiv. = 0.35

Physical Properties

	Imperial -psi
Tensile:	76616
Yield:	58075
Elongation (in 8 inches):	
Elongation (in 2 inches):	

The testing was conducted in accordance with the requirements of this specification. All melting and manufacturing processes were performed in the United States of America.

Jim Hill
Division Metallurgist

KIEH
METALLURGICAL SERVICES
(708) 544-8811 544-8820 FAX

We accept no responsibility nor liability for results derived from misinformation, nor samples not representative of the corresponding material, nor a limited sampling plan nor insufficient testing. The information provided is for the private use of our client and may not be published without our expressed consent.

Laboratory:
837 MANNHEIM RD.
BELLWOOD, IL 60104

Date 30-Oct-2013
received ~ 30-Oct-2013
Report 113 44024 b of c
pages
Account 1477

P.O. 181096
revised (113 44024 b of c)

Sure Built Manufacturing
840 S. 25th Ave.
Bellwood, IL 60104

Email: Ed Klodzinski

our 34th year est. 1979

Test report / certificate

Sample identity	Y.S. lbs/in^2	T.S. lbs/in^2	%RA
0.375 Ht.NF13100207 -2	85,000	100,000	55.0
	95,000	101,000	62.0

Mechanical:ASTM E8/A370 Y.S.0.2%offset ||trans. *broke out of g.l. |1"g.l.

SCOPE OF ACCREDITATION TO ISO/IEC 17025:2005

KIEH Corporation
837 Mannheim Road
Bellwood IL 60104
Jamie Iatropulos Phone: 708 544 8811

MECHANICAL

Valid To: June 30, 2020 Certificate Number: 1299.01

In recognition of the successful completion of the A2LA evaluation process, accreditation is granted to this laboratory to perform the following tests on metals and alloys:

Test	Test Method(s)
Hardness	
Rockwell (A, B, C, F, 15N, 30N, 45N, 15T, 30T, 45T)	ASTM A370, E18
Microindentation (Knoop/Vickers: 100gf, 500gf, 1000gf)	ASTM E384
Tensile (Yield, Tensile, %Elongation, %RA)	ASTM A370, E8/E8M
Impact (V-notch)	ASTM A370, E23
Weight of Coating	ASTM A90/A90M
Plastic Strain Ratio (R)	ASTM E517
Strain Hardening Exponent (n)	ASTM E646
Tensile for Externally Threaded Fasteners (Axial, Wedge, Proof)	ASTM F606/F606M

5202 Presidents Court, Suite 220 | Frederick, MD 21703-8398 | Phone: 301 644 3248 | Fax: 240 454 9449 | www.A2LA.org

CORPORATE
45610 Woodland Road, Suite 200
Sterling, VA 20166
www.reinforcedearth.com

854

A SUBSIDIARY OF SOLETANCHE FREYSSINET

Accredited Laboratory

A2LA has accredited

KIEH CO.

Bellwood, IL

for technical competence in the field of

Mechanical Testing

This laboratory is accredited in accordance with the recognized International Standard ISO/IEC 17025:2005 *General requirements for the competence of testing and calibration laboratories*. This accreditation demonstrates technical competence for a defined scope and the operation of a laboratory quality management system (*refer to joint ISO-ILAC-IAF Communiqué dated April 2017*).

Presented this 10th day of July 2018.

President & CEO
For the Accreditation Council
Certificate Number 1299.01
Valid to June 30, 2020

For the tests to which this accreditation applies, please refer to the laboratory's Mechanical Scope of Accreditation.

SCOPE OF ACCREDITATION TO ISO/IEC 17025:2005

KIEH Corporation
837 Mannheim Road
Bellwood IL 60104
Jamie Iatropulos Phone: 708 544 8811

CHEMICAL

Valid To: June 30, 2020 Certificate Number: 1299.02

In recognition of the successful completion of the A2LA evaluation process, accreditation is granted to this laboratory to perform the following tests on carbon and low alloy steel:

Test	Test Method
Spectroscopy	
Spark Emission Analysis of Carbon and Low Alloy Steel (C, Mn, Si)	ASTM E415

(A2LA Cert. No. 1299.02) 07/10/2018 Page 1 of 1

Accredited Laboratory

A2LA has accredited

KIEH CO.

Bellwood, IL

for technical competence in the field of

Chemical Testing

This laboratory is accredited in accordance with the recognized International Standard ISO/IEC 17025:2005 *General requirements for the competence of testing and calibration laboratories*. This accreditation demonstrates technical competence for a defined scope and the operation of a laboratory quality management system (*refer to joint ISO-ILAC-IAF Communiqué dated April 2017*).

Presented this 10th day of July 2018.

President & CEO
For the Accreditation Council
Certificate Number 1299.02
Valid to June 30, 2020

For the tests to which this accreditation applies, please refer to the laboratory's Chemical Scope of Accreditation.

7. Sample RECo certification package

- Certificate of Compliance
- Cover letter
- Bill of Lading
- Reinforcing Strips Steel Certification (CMC Steel)
- Reinforcing Strips Steel Certification (Nucor)
- Galvanization (Metalplate)
- Filter fabric (Carthage Mills)
- Bearing pads (Dyna-Mix/Rex-Hide)
- Bolts (Nucor)

CORPORATE
45610 Woodland Road, Suite 200
Sterling, VA 20166
www.reinforcedearth.com

A SUBSIDIARY OF SOLETANCHE FREYSSINET

REINFORCED EARTH

September 19, 2017

CUSTOMER NAME
ADDRESS

Subject: MSE Wall Materials - Certificates of Compliance

To Whom It May Concern:

(1) The attached certificates of compliance provided by The Reinforced Earth Company apply to the MSE materials supplied by The Reinforced Earth Company and all steel complies with the **BUY AMERICA** requirements for the following project:

PROJECT NAME AND LOCATION
PROJECT NUMBER

(2) The attached certificates of compliance apply to the materials listed on the following bills of lading:

RECO PROJECT # AND MATERIAL ORDER #

(3) The attached certificates of compliance apply to the following materials

Part Description	Manufacturer	Galvanizer
45HS Non-Woven filter-cloth 4.5 oz/sf - 1.5' X 360' = 54	CARTHAGE MILLS	
Adhesive 28oz. Tube-Consumer Commodity*	DAP, INC.	
Bearing Pad 3/4" Thick	REX-HIDE INDUSTRIES, INC.	
Bolt Set, 1/2" - 13 x 1 1/4" A-449 (Formerly a-325) Modif	NUCOR FASTENER DIVISION	
Galv. 50 X 4mm Restrips single punch	CMC STEEL ARKANSAS (V)	METALPLATE GALVANIZING
Ring Clutch 2 Ton Manual #79-001 (Burke Lift)~FL001	CONAC CONCRETE ACCERIES, INC.	
Shims 3/16 X 2 X 3-3/8" - EPDM Shims, 80 DURO, RHP	REX-HIDE INDUSTRIES, INC.	

Sincerely,
The Reinforced Earth Company

Chris Sistrunk

Chris Sistrunk
Quality Assurance Manager

STRAIGHT BILL OF LADING-SHORT FORM-ORIGINAL-NOT NEGOTIABLE

RECEIVED, subject to the classification and tariffs in effect on the date of the issue of this Bill of Lading the property described below, in apparent good order, except as noted (contents and conditions of contents of package unknown), marked consigned and destined as indicated below, which said carrier (the word carrier being understood throughout this contract as meaning any person or corporation in possession of the property under contract) agrees to carry to its usually place of delivery of said destination. If on its route, otherwise to deliver to another carrier on the route to said destination, it is mutually agreed, as to eachcarrier is all or any of said property over all or any portion of said route to destination, and as to each party at any timeinterested in all or said property, that every service to be performed hereunder shall be subject to all the terms and conditions of the Uniform Domestic Bill of Lading set forth (1) in Official, Southern, Western and Illinois Freight Classification in effect on the date hereof, if this is a rail or a railwater shipment, or (2) if the applicable motor carrier classification or tariff in this is a motor carrier shipment.

Shipper hereby certifies that he is familiar with all the forms and conditions of the said bill of lading, including those on the back thereof, set forth in the classification or tariff which governs the transportation of this shipment, and the said terms and conditionsare hereby agreed to by the shipper and accepted for himself and his assigns.

Project Mgr:
Carrier
Consigned to
Address

Date: 01 NOV 2011
Estimated Delivery
02 NOV 2011
Painted:
PURPLE

Carrier: call fax:

IMPORTANT: ALL FLAT BED DRIVERS MUST CALL 24 HOURS PRIOR TO DELIVERY FAILURE TO DO SO MAY RESULT IN OFF-LOADING DELAYS

[X] BILL TO: THE REINFORCED EARTH COMPANY, 8614 WESTWOOD CENTER DRIVE, SUITE 1100, VIENNA VA 22182

SHIPPED BY: The Reinforced Earth Company 5101 A Bird Creek Ave, Port of Catoosa, OK 74015

TRACKING NUMBER
14577-REW-1-001
Shipper's No.
14577-REW-1-001

Subject to Section 7 of conditions of applicable bill of lading, if this shipment is to be deliverted to the consignee without recourse on the consignor, the consignor shall sign the following statements.
The carrier shall not make delivery of this shipment without payment of freight and other lawful charges.

If charges are to be prepaid, check HERE: [X]

Received: $ _____
to apply in prepayment of the charges on the property described hereon.

Agent or Cashier
Per:_____
(Signature acknowledges on ly the amount prepaid)
Charges advanced: $_____

HM (X)	Total Quantity	Part Number	Item Description	Item/ Classification	Weight (Subj to correction)
	5	45HS	45HS Non-Woven filter-cloth 4.5 oz/sf	70	120
X	52	ADHT	Adhesive 28oz. Tube-Consumer Commodit	60	130
	ADHESIVE, CLASS 3(ORM-D), UN1133. PG GROUP III, EMERGENCY #800-543-3840/800-879-4444, 49 CFR-172.101				
	924	BL14	Bolt Set, 1/2" - 13 x 1 1/4" A-449 (F	50	203
	282	BP34	Bearing Pad 3/4" Thick	60	113
	2	LFT2	Ring Clutch 2 Ton Manual #79-001 (Bur	50	10
	64	S504G08	8' (2.44 M) Galv. 50 X 4mm Restrips s	50	616
	72	S504G10	10' (3.05 M) Galv. 50 X 4mm Restrips	50	866
	54	S504G12	12' (3.66 M) Galv. 50 X 4mm Restrips	50	780
	197	S504G14	14' (4.27 M) Galv. 50 X 4mm Restrips	50	3,317
	144	S504G16	16' (4.88 M) Galv. 50 X 4mm Restrips	50	2,772
	216	S504G18	18' (5.49 M) Galv. 50 X 4mm Restrips	50	4,676
	87	S504G20	20' (6.10 M) Galv. 50 X 4mm Restrips	50	2,093
	92	S504G22	22' (6.71 M) Galv. 50 X 4mm Restrips	50	2,435
	100	SHIM	Shims 3/16 X 2 X 3-3/8" - EPDM Shims,	60	7

Dunnage and weight adjustments: 1580.0346
One Pallet # 2 nd STOP TRAILER # 12474A
Wall Summary:14577(REW)

Rolling Dates: 08/10/2011:NUCOR STEEL JACKSON, INC/RECOPURCH1, 09/20/2011:CMC STEEL ARKANSAS (V)/CMC

TOTAL 19,719

(Shipper's imprint in lieu of stamp; not a part of bill of lading approved by the Interstate Commerce Commission.)

NOTE - Where the rate is dependent on value shippers are required to state specifically in writing the agreed or declared value of the property. The agreed or declared value of the property is hereby specifically stated by the shipper to be not exceeding: $_____ per _____

The fiber boxes used for this shipment conform to the specifications set forth in the box maker's certificate thereon, and all other requirements of Rule 41 of the Consolidated Freight Classification.

"The shipment is tendered and received subject to the terms and conditions of the carrier's bill of lading shown in the Tariff and Classification on file with the Interstate Commerce Commission naming and covering the rates or rates lawfully applicable to this shipment." THE REINFORCED EARTH COMPANY

Shipper, Per _____ Agent, Per _____

Receiver

Receiver, Per _____ Printed Name _____ Date _____

CERTIFIED MILL TEST REPORT

For additional copies call
870-234-8703

We hereby certify that the test results presented here are accurate and conform to the reported grade specification

Quality Assurance Manager

CMC STEEL ARKANSAS
100 COLUMBIA 7-6
MAGNOLIA AR 71753-9238

SECTION:	EARTHBAR NR
GRADE:	A572-65T2
HEAT NO:	
MELT DATE:	
ROLL DATE:	8/8/2011

Heat Number	Yield Point KSI	Tensile Strength KSI	% Elong 8 in.	Lbs./ Feet	Thick-ness	Width Inches	Carbon % C	Silicon % SI	Sulfur % S	Phos. % P	Mang. % Mn	Van. % V	Columb % CB
1013194	76.7	102.7	13%	1.124	0.157	1.969	0.23%	0.17%	0.031%	0.015%	1.14%	0.067%	0.001%
1013843	76.0	103.3	12%	1.124	0.157	1.969	0.21%	0.21%	0.040%	0.019%	1.14%	0.066%	0.001%
1013848	79.2	102.7	13%	1.124	0.157	1.969	0.21%	0.20%	0.035%	0.019%	1.15%	0.065%	0.001%
1013849	79.8	105.0	12%	1.124	0.157	1.969	0.21%	0.19%	0.037%	0.017%	1.13%	0.067%	0.001%
1013850	79.8	103.3	13%	1.124	0.157	1.969	0.22%	0.19%	0.029%	0.012%	1.14%	0.064%	0.001%
1013852	78.5	100.3	12%	1.124	0.157	1.969	0.22%	0.18%	0.032%	0.010%	1.16%	0.064%	0.001%
1013853	77.0	98.4	12%	1.124	0.157	1.969	0.21%	0.20%	0.030%	0.011%	1.16%	0.068%	0.001%
1013854	74.0	95.0	14%	1.124	0.157	1.969	0.21%	0.20%	0.033%	0.013%	1.14%	0.064%	0.001%
1014234	77.0	97.0	13%	1.124	0.157	1.969	0.21%	0.19%	0.038%	0.012%	1.14%	0.059%	0.001%
1014236	76.0	100.0	13%	1.124	0.157	1.969	0.23%	0.17%	0.033%	0.011%	1.13%	0.065%	0.001%

THIS MATERIAL IS FULLY KILLED, 100% MELTED AND MANUFACTURED IN THE USA, WITH NO WELD REPAIR OR MERCURY CONTAMINATION IN THE PROCESS.

QC/INVENTORY COORDINATOR

Printed:8/10/2011

SOLD TO:

NUCOR
NUCOR STEEL JACKSON, INC.

CERTIFIED MILL TEST REPORT

Page: 1

Roll Data 4-10-2011

Ship from:
Nucor Steel Jackson, Inc.
3630 Fourth Street
Flowood, MS 39232
800-723-1623

SHIP TO:

Date: 12-Sep-2011 - Sent Today
B.L. Number:
Load Number:

Material Safety Data Sheets are available at www.nucorbar.com or by contacting your inside sales representative. NBMG-68 March 9, 2011

HEAT NUM. *	DESCRIPTION	PHYSICAL TESTS					CHEMICAL TESTS							
		YIELD P.S.I.	TENSILE P.S.I.	ELONG % IN 8"	BEND	WT% DEF	C / Ni	Mn / Cr	P / Mo	S / V	Si / Cb	Cu / Sn	C.E.
JK1110372001	Nucor Steel - Jackson Inc	80,202	102,890	16.3%			.24	1.05	.017	.034	.18	.34	
	50x4mm HA Reinforcing Str	553MPa	709MPa				.10	.10	.028	.043	.003		
	49'06" A572 Gr65 TY2	80,892	102,980	12.5%			MN/C						
	ASTM A572/A572M-07 GR 65 TY2 (RE CO)	558MPa	710MPa				04.38						
JK1110372101	Nucor Steel - Jackson Inc	80,954	101,710	17.5%			.23	1.06	.016	.034	.17	.30	
	50x4mm HA Reinforcing Str	558MPa	701MPa				.10	.10	.026	.049	.003		
	49'06" A572 Gr65 TY2	80,127	101,620	16.3%			MN/C						
	ASTM A572/A572M-07 GR 65 TY2 (RE CO)	552MPa	701MPa				04.61						
JK1110372201	Nucor Steel - Jackson Inc	83,937	107,330	16.3%			.25	1.09	.016	.031	.16	.29	
	50x4mm HA Reinforcing Str	579MPa	740MPa				.10	.11	.025	.046	.004		
	49'06" A572 Gr65 TY2	82,508	106,030	17.5%			MN/C						
	ASTM A572/A572M-07 GR 65 TY2 (RE CO)	569MPa	731MPa				04.36						
JK1110372301	Nucor Steel - Jackson Inc	81,382	103,370	16.3%			.26	1.06	.015	.031	.17	.31	
	50x4mm HA Reinforcing Str	561MPa	713MPa				.10	.11	.025	.045	.004		
	49'06" A572 Gr65 TY2	80,480	103,300	16.3%			MN/C						
	ASTM A572/A572M-07 GR 65 TY2 (RE CO)	555MPa	712MPa				04.08						
JK1110372401	Nucor Steel - Jackson Inc	76,889	96,130	15.0%			.24	1.04	.015	.034	.17	.32	
	50x4mm HA Reinforcing Str	530MPa	663MPa				.09	.10	.025	.049	.004		
	49'06" A572 Gr65 TY2	76,416	96,920	16.3%			MN/C						
	ASTM A572/A572M-07 GR 65 TY2 (RE CO)	527MPa	668MPa				04.33						
JK1110372501	Nucor Steel - Jackson Inc	74,727	99,020	15.0%			.23	1.09	.019	.031	.17	.32	
	50x4mm HA Reinforcing Str	515MPa	683MPa				.10	.11	.025	.045	.004		
	49'06" A572 Gr65 TY2	76,609	97,940	15.0%			MN/C						
	ASTM A572/A572M-07 GR 65 TY2 (RE CO)	528MPa	675MPa				04.74						

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed above and that it satisfies those requirements.
1.) Weld repair was not performed on this material.
2.) Melted and Manufactured in the United States.
3.) Mercury, Radium, or Alpha source materials in any form have not been used in the production of this material.

QUALITY ASSURANCE: Christopher Smith

Christopher Smith

SOLD TO:

NUCOR
NUCOR STEEL JACKSON, INC.

CERTIFIED MILL TEST REPORT

8-10-11

SHIP TO:

Ship from:
Nucor Steel Jackson, Inc.
3630 Fourth Street
Flowood, MS 39232
800-723-1623

Date
B.L. Number
Load Number

Material Safety Data Sheets are available at www.nucorbar.com or by contacting your inside sales representative. NSMG-08 March 9, 2011

HEAT NUM. *	DESCRIPTION	PHYSICAL TESTS					CHEMICAL TESTS							
		YIELD P.S.I.	TENSILE P.S.I.	ELONG % IN 8"	BEND	WT% DEF	C / Ni	Mn / Cr	P / Mo	S / V	Si / Cb	Cu / Sn	C.E.
JK1110372601	Nucor Steel - Jackson Inc	83,111	103,210	16.3%			.24	1.12	.019	.034	.16	.35	
	50x4mm HA Reinforcing Str	573MPa	712MPa				.10	.12	.023	.046	.003		
	49'06" A572 Gr65 TY2	78,635	100,350	16.3%			MN/C						
	ASTM A572/A572M-07 GR 65 TY2 (RE CO)	542MPa	692MPa				04.67						
JK1110372701	Nucor Steel - Jackson Inc	79,426	98,700	15.0%			.25	1.07	.021	.036	.16	.30	
	50x4mm HA Reinforcing Str	548MPa	681MPa				.09	.12	.022	.050	.003		
	49'06" A572 Gr65 TY2	78,984	98,830	16.3%			MN/C						
	ASTM A572/A572M-07 GR 65 TY2 (RE CO)	545MPa	681MPa				04.28						
JK1110372801	Nucor Steel - Jackson Inc	83,397	102,600	15.0%			.23	1.09	.020	.029	.17	.27	
	50x4mm HA Reinforcing Str	575MPa	707MPa				.10	.12	.023	.051	.003		
	49'06" A572 Gr65 TY2	81,757	103,240	15.0%			MN/C						
	ASTM A572/A572M-07 GR 65 TY2 (RE CO)	564MPa	712MPa				04.74						
JK1110372901	Nucor Steel - Jackson Inc	77,238	102,160	16.3%			.24	1.12	.021	.031	.16	.37	
	50x4mm HA Reinforcing Str	533MPa	704MPa				.13	.18	.027	.043	.003		
	49'06" A572 Gr65 TY2	79,176	102,290	16.3%			MN/C						
	ASTM A572/A572M-07 GR 65 TY2 (RE CO)	546MPa	705MPa				04.67						

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed above and that it satisfies those requirements.
3.1 Weld repair was not performed on this material.
2.1 Melted and Manufactured in the United States.
3.1 Mercury, Radium, or Alpha source materials in any form have not been used in the production of this material.

QUALITY ASSURANCE: Christopher Smith

BIRMINGHAM | ATLANTA | JACKSONVILLE | HOUSTON

Metalplate Galvanizing, L.P.

September 23, 2013

The Reinforced Earth Company
12001 Sunrise Valley Drive
Suite 400
Reston, VA 20191

All material supplied to The Reinforced Earth Company in 2013.

This Certificate of Compliance includes all material processed from 09-23-2013 through 09-29-2013.

We certify the material (4x50mm reinforcing strip) on your above order was galvanized in accordance with specifications set forth in ASTM Standard Specification Designation A-123-09. This material was galvanized in the United States of America.

METALPLATE GALVANIZING, L.P.

Phil Sloop
Plant Superintendant

I certify the above to be correct.

Heather Kunkle
Heather Kunkle, Notary Public
My commission expires February 20, 2016

HEATHER KUNKLE
Notary Public
STATE OF TEXAS
My Comm. Exp. February 20, 2016

Coating Grade	Mils	Oz/ft	um	g/m
.85	3.3	2.0	85	600

Corporate Office
P.O. Box 966
1120 39th Street North
Birmingham, AL 35201
Phone (205) 595-4700
Fax (205) 595-7800

Plant 1
757 44th Street North
Birmingham, AL 35212
Phone (205) 595-1106
Fax (205) 591-4659

Plant 2
1120 39th Street North
Birmingham, AL 35234
Phone (205) 595-7103
Fax (205) 595-2965

Atlanta Plant
500 Selig Drive, S.W.
Atlanta, GA 30336
Phone (404) 691-0600
Fax (404) 699-2270

Jacksonville Plant
7123 Moncrief Road, West
Jacksonville, FL 32219
Phone (904) 768-6330
Fax (904) 764-3948

Houston East
10635 Needham Street
Houston, TX 77013
Phone (713) 672-9480
Fax (713) 672-9892

Houston West
10625 Needham Street
Houston, TX 77013
Phone (713) 671-2454
Fax (713) 671-2957

Carthage Mills | **Certification of Compliance**

Reinforced Earth Co

Shipment Date 6/18/10
Order #105493
PO# 31925

■ FX®–45HS

Carthage Mills' FX®–45HS is a multipurpose nonwoven geotextile of 100% polypropylene staple fibers which are formed into a random network, needlepunched and heatset for dimensional stability. Carthage Mills' FX®–45HS is inert to biological degradation and resistant to naturally encountered chemicals, alkalis, and acids and meets or exceeds the following values.

PROPERTY	TEST METHOD	DATA	
		METRIC	ENGLISH
☐ **Mechanical** Grab Tensile Strength	ASTM D 4632	0.53 kN	120 lbs
Grab Tensile Elongation		50%	
Mullen Burst	ASTM D 3786	1586 kPa	230 psi
Trapezoidal Tear	ASTM D 4533	0.22 kN	50 lbs
Puncture	ASTM D 4833	0.29 kN	65 lbs
CBR Puncture	ASTM D 6241	1.49 kN	335 lbs
☐ **Endurance** UV Resistance	ASTM D 4355	70% @ 500 hrs	
☐ **Hydraulics / Filtration** Permittivity (1)	ASTM D 4491	1.50 sec^{-1}	
Water Flow Rate (1)		4889 lpm/m^2	120 gpm/ft^2
Apparent Opening Size (AOS) (1)	ASTM D 4751	0.212 mm	70 US Std. Sieve

* Unless otherwise stated, all values stated here are Minimum Average Roll Values (MARV), are calculated as the Typical minus two standard deviations and are based on a 97.7% confidence level.

On behalf of Carthage Mills, I hereby certify the above to be true and correct.

Jim Paulsen
Vice President

Subscribed and sworn before me this 24th day of June 2010 at Cincinnati, Hamilton County.
Seal:

NOTARIAL SEAL * STATE OF OHIO *

Toni M. Haines
Notary Public, State of Ohio
My commission expires September 8, 2013.

Carthage Mills makes no warranty, express or implied, including but not limited to warranties of fitness for a particular purpose or merchantability concerning the product furnished hereunder other than at the time of delivery it shall be of the quality and specification stated herein. If, at the time of delivery, the product does not meet Carthage Mills current published specifications and written notice of the deficiency is given to Carthage Mills prior to installation of the product, Carthage Mills will replace the product with materials meeting the quality and specification stated herein at no additional charge or refund the purchase price of the deficient material.

4243 Hunt Road
Cincinnati, OH 45242
www.carthagemills.com
www.gxgeogrids.com

513-794-1600 TELEPHONE
800-643-4430 TOLL FREE
513-794-3434 FACSIMILE
info@carthagemills.com

Since 1958: America's *First* Geotextile Company

REX-HIDE INDUSTRIES, INC.

P.O. Box 4726
Tyler, Texas 75712-4726
Phone: 903-593-7387
Fax: 903-592-0122

Custom Rubber Extrusion Division

Product Certification

Customer: **REINFORCED EARTH CO.**
Part Number: **RHTBP1A [1" Bearing Pad]**
Rex-Hide Part No.: 73163024
Order No.: 58979
Compound: 148
Quantity: 100,000 lb
Date Cured: 2/21/18
Date Shipped: 3/14/18

Material Specification: **ASTM D2000: 4AA812, A13, B33, C12, F17**

Requirements	Limits	Results
Originals:		
Tensile (PSI) min.:	1,200	1356
Elongation (%) min.:	150	379
Durometer:	75 – 85	85
Specific Gravity:		1.22
B33: Compression Set (ASTM D395)		
22 hrs @ 70 °C, solid		
(%) max:	35	20
A13: Heat Age (70 hrs @ 70 °C)		
Tensile change (%) max:	- 25	+ 7
Elongation Change (%) max:	- 25	- 7
Durometer Change:	+ 10	+ 3
C12: Ozone Resistance (ASTM D1171)		
Quality retention rating, Min. %:	85	85
F17: Low Temp Brittleness: (ASTM D2137) Method A, 9.3.2 non-brittle after 3 min @ -40 °C	Pass	Pass

% Elastomer in Rubber Compound: 24.4 %
% of Elastomer that is E.P.D.M. Polymer: 100 %

David Minick, Chemist Gaye Ray, Quality Assurance Gaye Ray

NUCOR
FASTENER DIVISION

LOT NO.
297562C

Post Office Box 6100
Saint Joe, Indiana 46785
Telephone 260/337-1600

NUCOR ORDER#: 772138
SERIAL#: FB379615 CUST PO: 36166 CUST #/NAME: 9151 THE REINFORCED EARTH CO.
ISSUE DATE: 12/19/11 DATE SHIPPED: 12/19/11

********** CERTIFIED MATERIAL TEST REPORT **********

PART NO.	LOT NO.	DESCRIPTION
919579	297562C	1/2-13 X 1 1/4 A449 H.D.G. STRUC H.D.G. W/NUT & WASHER

CUST PART #: QUANTITY: 2500

********** BOLT **********

PART NO.	LOT NO.	DESCRIPTION:	MANUFACTURE DATE
919577	297483B	1/2-13 X 1 1/4 A449 H.D.G. STRUC SCREW H.D.G.	11/15/11

--MECHANICAL PROPERTIES IN ACCORDANCE WITH ASTM A449-07b
AVERAGE VALUES OF 5 TESTS
CORE HARDNESS: 29.1 (RC)
TOO SHORT TO TEST



--DIMENSIONS PER ASME B18.2.6-2006
--CHEMISTRY MATERIAL GRADE -1035MR

MATERIAL NUMBER	HEAT NUMBER	**CHEMISTRY COMPOSITION (WT% HEAT ANALYSIS) BY MATERIAL SUPPLIER C	MN	P	S	SI	NUCOR STEEL - NEBRASKA
RM026867	NF11103295	.33	.84	.013	.016	.25	
	MIN	.28	.60				
	MAX	.55		.040	.050		

--COATING - HOT DIP GALVANIZED TO ASTM F2329-05 AVERAGE THICKNESS FROM 15 TESTS: .00345

********** NUT **********

PART NO.	LOT NO.	DESCRIPTION	MANUFACTURE DATE
919567	297929B	1/2-13 GR DH HV HX NUT H.D.G. HEX NUT H.D.G.	11/10/11

--MECHANICAL PROPERTIES IN ACCORDANCE WITH ASTM A563-07a
CORE HARDNESS: 28.6 (RC) PROOF LOAD: 21,300(LBS) AVERAGE VALUES OF 5 TESTS



--DIMENSIONS PER ASME B18.2.6-2006
--CHEMISTRY MATERIAL GRADE -1026L

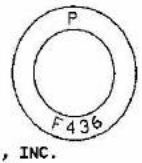
MATERIAL NUMBER	HEAT NUMBER	**CHEMISTRY COMPOSITION (WT% HEAT ANALYSIS) BY MATERIAL SUPPLIER C	MN	P	S	SI	NUCOR STEEL - NEBRASKA
RM026874	NF11103167	.23	.72	.010	.015	.21	
	MIN	.20	.60				
	MAX	.55		.040	.050		

--COATING - HOT DIP GALVANIZED TO ASTM F2329-05 AVERAGE THICKNESS FROM 15 TESTS: .00322

********** WASHER **********

PART NO.	LOT NO.	DESCRIPTION	MANUFACTURE DATE
919557	300675A	1/2 STRUCTURAL WASHER H.D.G. ASTM F436 H.D.G.	

--MECHANICAL PROPERTIES IN ACCORDANCE WITH ASTM F436
CORE HARDNESS: 41.6 (RC)
--CHEMISTRY MATERIAL GRADE -CF436



HEAT NUMBER	**CHEMISTRY COMPOSITION (WT% HEAT ANALYSIS) BY WASHER SUPPLIER C	MN	P	S	SI	PRESTIGE STAMPING, INC. 23513 GROESBECK HWY WARREN, MI 48089
A27786	.52	.84	.011	.001	.24	

--COATING - HOT DIP GALVANIZED TO ASTM F2329-05 AVERAGE THICKNESS FROM 15 TESTS: .00549

****

ALL TESTS ARE IN ACCORDANCE WITH THE LATEST REVISIONS OF THE METHODS PRESCRIBED IN THE APPLICABLE SAE AND ASTM SPECIFICATIONS. THE SAMPLES TESTED CONFORM TO THE SPECIFICATIONS AS DESCRIBED/LISTED ABOVE AND WERE MANUFACTURED FREE OF MERCURY CONTAMINATION. NO INTENTIONAL ADDITIONS OF BISMUTH, SELENIUM, TELLURIUM, OR LEAD WERE USED IN THE STEEL USED TO PRODUCE THIS PRODUCT.
THE STEEL WAS MELTED AND MANUFACTURED IN THE U.S.A. AND THE PRODUCT WAS MANUFACTURED AND TESTED IN THE U.S.A. PRODUCT COMPLIES WITH DFARS 252.225-7014. WE CERTIFY THAT THIS DATA IS A TRUE REPRESENTATION OF INFORMATION PROVIDED BY THE MATERIAL SUPPLIER AND OUR TESTING LABORATORY. THIS CERTIFIED MATERIAL TEST REPORT RELATES ONLY TO THE ITEMS LISTED ON THIS DOCUMENT AND MAY NOT BE REPRODUCED EXCEPT IN FULL.

A2LA ACCREDITED
MECHANICAL FASTENER
Certificate No. A2LA 0139.01
Expiration Date 12/31/11

NUCOR FASTENER
A DIVISION OF NUCOR CORPORATION

BOB HAYWOOD
QUALITY ASSURANCE SUPERVISOR

AZZ GALVANIZING-HAMILTON
7825 S. HOMESTEAD DRIVE
HAMILTON, IN 46742
TEL: (260) 488-4477
FAX: (260) 488-4499

NUCOR FASTENER
6730 CR 60
ST JOE, IN 46785

12-9-2011

MATERIAL CERTIFICATION

DESCRIPTION:

Part # Lot #

5 BINS (160247) (298254A) 5/8-11 x 2 1/2
1 BIN (160237) (298252B) 5/8-11 x 2 1/4
1 BIN (164097) (298611A) 1 ¼-7 x 2 3/4
10 BINS (919577) (297483B) ½-13 x 1 1/4
1 BIN (160067) (298109A) ½-13 x 1 3/4
3 BINS (160577) (298356B) ¾-10 X 2 1/4

P. O. #: 128560

GALVANIZIED WEIGHT#: 40,489

AZZ JOB #: 43505194

CERTIFICATION: AZZ GALVANIZING-HAMILTON CERTIFIES THAT THE ABOVE REFERENCED MATERIALS HAVE BEEN GALVANIZIED IN ACCORDANCE WITH THE ASTM-123 AND OR ASTM-153/F2329 SPECIFICATION STANDARD AND MEET THE CRITERIA THEREIN.

AVERAGE MIL THICKNESS: 3.43

Maurice Cox
ISO Mgmt. Rep.

AZZ GALVANIZING-HAMILTON
7825 S. HOMESTEAD DRIVE
HAMILTON, IN 46742
TEL: (260) 488-4477
FAX: (260) 488-4499

NUCOR FASTENER
6730 CR 60
ST JOE, IN 46785

12-1-2011

MATERIAL CERTIFICATION

DESCRIPTION: Part # Lot #

2 BINS (910497) (299867A) ¾-10 GR C MULTI PLATE
4 BINS (164227) (298583A) 1 ¼-7 x 6
1 BIN (164267) (298581A) 1 ¼-7 x 7
2 BINS (919567) (297929B) ½-13 GR DH HV HX NUT
7 BINS (4190087) (299141B) ½-13 X 1 1/2
4 BINS (4190087) (299141A) ½-13 X 1 1/2

P. O. #: 128402

GALVANIZIED WEIGHT#: 37,248

AZZ JOB #: 43505056

CERTIFICATION: AZZ GALVANIZING-HAMILTON CERTIFIES THAT THE ABOVE REFERENCED MATERIALS HAVE BEEN GALVANIZIED IN ACCORDANCE WITH THE ASTM-123 AND OR ASTM-153/F2329 SPECIFICATION STANDARD AND MEET THE CRITERIA THEREIN.

AVERAGE MIL THICKNESS: 3.54

Maurice Cox
ISO Mgmt. Rep.

8. Precast Concrete; Procedures and Controls

The Reinforced Earth Company partners with precasters that are approved by the required agency of every State Department of Transportation to manufacture the precast concrete products as required on a project by project basis. In addition, RECo encourages all precasters to obtain the National Precast Concrete Association (NPCA) certification as an additional qualification that ensures all products will be produced according to specifications, utilizing best practices in the industry while maintaining a safe working environment.

8.1 Materials supplied by RECo.

RECo supplies the precaster with a complement of molds and liners as required for every specific project. The quantity of molds and liners supplied varies and it is determined by the size and schedule of the project. The molds are periodically reconditioned and are sent to the precaster ready to be used, further confirming RECo's commitment to produce and achieve the highest possible quality on every product. Aesthetics, architectural details, and liners will be detailed, manufactured, and supplied per the project specifications and approvals by the owner

RECo supplies the Tie Strips, Clevis Loops and GeoMega sleeve to be cast into the panels as required by the project.

RECo employees regularly visit the precasters to ensure that all molds are properly cared for, rebar clearance and Tie Strip, Clevis Loops or GeoMega sleeve positioning is correct, safety procedures are in place and followed and ensure that a good quality of the final product is maintained. Visits are frequent during startup and less frequent as the job progresses but not less than once every three months.

8.2 Precaster's supplied Material

The precaster is responsible for supplying the labor, concrete, steel reinforcement and lifting devices as required. The precaster is also responsible for maintaining a log of all concrete being used, referencing a lot number and properly entering this information in the system for future reference. RECo recommends to all Precasters to place and secure each mold at an ergonomic working height and that it be leveled and secured to its mounting above the floor. Any block, beams or other methods to raise the form above the floor level are provided by the Precaster.

8.3 Quality Control and Production Inventory

RECo provides the precaster access to our proprietary Precast Web system to view the Bill of Materials for the project, report produced product as the job progresses (Fig 1), and maintaining a clear record and the inventory of product (Fig 2). Precasters are responsible for assigning a unique identifier number for the lots to be used at the start of every day. No production can be entered in the system unless the unique lot# exists. Precasters also prepare concrete cylinders for testing as required by the specifications of the project.

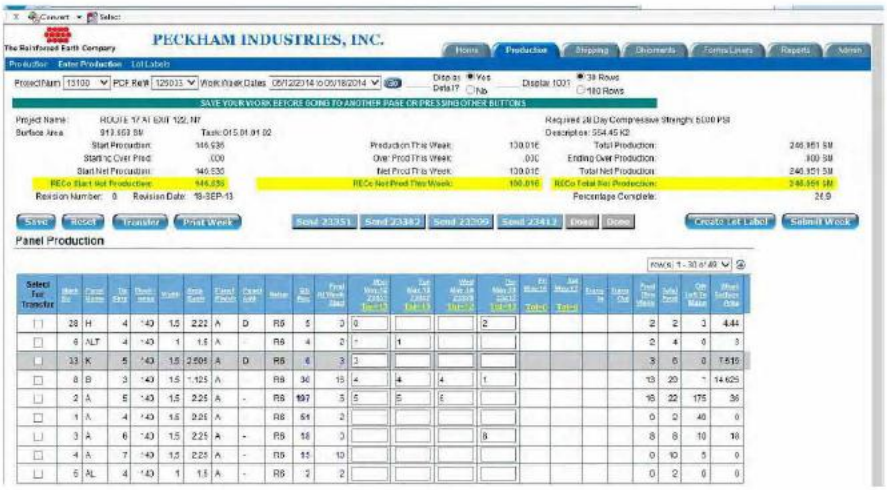


Figure 1

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Convert Select

PCF Inventory for Job Num 16420 and PCF Ref# 126045

1 - 21

Mark #	Panel Name	Tie Strips	Thick-ness	Width	Area Each	Panel Finish	Panel Add	Rebar	Qty Shipped	Scheduled To Ship	Inven-tory	Weight Per Piece	Select	Req
3	B	2	5.50	4.92	12.10	FF	-	R4	0	16	16	847	☐	16
4	C	2	5.50	4.92	8.80	FF	D	R4	0	1	1	616	☐	1
10	DJ	2	5.50	4.92	10.60	FF	D	R4	0	4	4	742	☐	4
11	DJJ	2	5.50	4.92	9.10	FF	D	R4	0	1	1	637	☐	1
12	E	3	5.50	4.92	14.90	FF	D	R4	0	3	3	1043	☐	3
18	FJJ	3	5.50	4.92	14.90	FF	D	R4	0	2	2	1043	☐	2
20	GJ	4	5.50	4.92	19.40	FF	D	R4	0	2	2	1358	☐	2
21	GJJ	4	5.50	4.92	17.90	FF	D	R4	0	3	3	1253	☐	3
22	H	4	5.50	4.92	24.20	FF	D	R4	0	2	2	1694	☐	2
23	HJ	4	5.50	4.92	22.70	FF	D	R4	0	2	2	1589	☐	2
24	HJJ	4	5.50	4.92	21.20	FF	D	R4	0	1	1	1484	☐	1

Figure 2

8.4 Panel Handling, Storage, and Shipping

Panels are stripped from the mold based on results from cylinder breaks, which typically is the following day, and stacked on adequate height dunnage to prevent damage to the front face of the panel by the Tie Strips or Clevis Loops. The product is marked with the proper labels and the fabrication date to have traceability to the Lot# of the concrete.

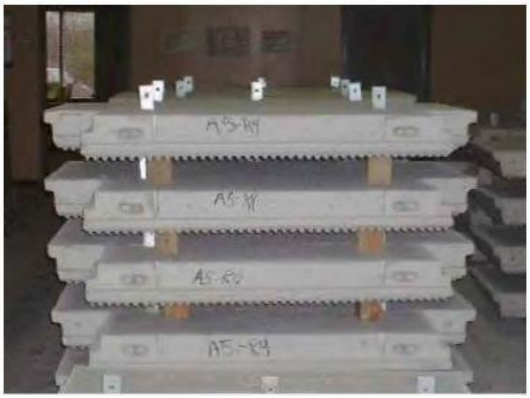


Figure 3

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The Reinforced Earth Company - Quality Control Manual June 2019
The stacks of panels are then transported to the outside storage area, which has been graded and stabilized (Fig 4). Placement of the stacks of panels are made so that any one Mark # can be accessed without having to move additional stacks. Rows of panels are spaced so that forklifts can easily access all panels. Stacking heights will be controlled by the quality of the ground stabilization and OSHA regulations.





Figure 4

If required, an inspection and approval by the Owner's representative at the Precast Plant can be made prior to loading the product onto flatbed trailers for transport to the jobsite. (Fig 5)



Figure 5

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Once the panels arrive at the jobsite they are additionally inspected by the on-site Owner's representative and the Contractor and any damage that may have occurred during transport is noted on the shipping documents. The panels are then unloaded by the Contractor directly off the truck in full stacks and placed, with the same dunnage supplied on the trailer, near the wall site, where they will then be individually lifted and erected into the MSE wall.



Figure 6

RECo also supplies the precasters with an instructional CD which contains additional detailed procedures to assemble the molds and liners, proper handling and care of the molds and liners, and best practices to achieve optimum quality of the product and within specifications and emphasize the need for a safe working environment.

A preconstruction meeting is arranged between the contractor/wall installer and RECo personnel to establish schedules and discuss installation procedures. The contractor is given construction manuals to distribute amongst the responsible personnel erecting the walls.

Precasters work closely with the onsite contractor to schedule the delivery of the product to the site as needed. RECo's staff stays in close communication and monitors the progress at the precaster site as well as the job site.

8.5 Contractor Responsibilities

Contractor is responsible for inspecting all materials upon delivery. Contractor must also ensure materials are not damaged during storage and handling. RECo to be notified of any damaged material.

9. Quality Validation of major components for a Reinforced Earth Wall.

9.1 Precast Products

In addition to the reporting requirements by the precaster to RECo's system (see Section 8), RECo employees will regularly visit the precasters to confirm that all molds are properly cared for, rebar clearance and Tie Strip, Clevis Loops or GeoMega sleeve positioning is correct, safety procedures are in place and followed and ensure that a good quality of the final product is maintained. For verification purposes, visits are frequent during startup and less frequent as the job progresses but not less than once every three months.
All records for the concrete used and tested are kept on file at the plant.

9.2 Reinforcing Strips

The Steel Mill provides RECo with a certificate of compliance and test data for every Heat Number used to manufacture the reinforcing strips, which includes chemical composition and physical testing. Each Heat# consist of approximately 60 tons of reinforcing strips.
RECo does random testing of this material at an independent laboratory to validate the certification sent. The frequency of the testing is approximately once every 600-800 tons and depending on the amount of material being processed, this could be one to two times a month. Tests are performed to validate chemical composition, physical characteristics and geometry.
Records for these random tests are kept on file at the plant.

9.3 Galvanizing

Every Reinforcing Strip load that is galvanized is tested for compliance. This is done in a daily basis. RECo personnel verifies the coating thickness in the same manner and in a daily basis. Records are kept at the plant.

9.4 Welded Wire Material

The manufacturer receives raw material with all required mill certifications. Once the wire is drawn to the required diameter, three random samples from each heat# are sent to the lab for testing to ensure that the required strength has been reached, prior to manufacturing a product. Depending on the amount of wire processed, the frequency of testing may be from one per month to three months.

9.5 Tie Strips

The manufacturer receives the coils with all required mill certifications. A sample from each truck load (44,000lbs) or heat number is obtained and sent to an independent testing facility to validate that the proper Grade material was received, prior to manufacturing the Tie Strips. Depending on the amount of material processed in a month, the frequency of testing may be one per month to three months.
The manufacturer manages and keeps records at his site.

10. Product Changes

Product changes are a 5 step process:

1. Identify issue - An issue is any reported problem or suggested improvement to a product design and is typically captured in a change request or change notice. The issue is analyzed, discussed, and validated before it is moved to the next step.

2. Request change - Change requests start the change process and capture all the relevant data and analysis, including the technical and business justification to execute the change.

3. Plan change - When a change is accepted for implementation, the work to document and implement the change will be planned. The plan will include separate tasks that are associated to the product data to be changed. A schedule shall be proposed and recommendations made for the disposition of existing product inventory and possible in-house testing or independent laboratory testing.

4. Change implementation - Change tasks are distributed to personnel responsible for updating product documentation and capturing new product configurations. New product configurations are released to manufacturing when the physical implementation begins.

5. Physical implementation - Changes are incorporated in production and/or purchase orders, and the new and modified parts are inspected to make sure they comply with design requirements. Variances from product specifications shall follow our non-conformance process.

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11. REVISION LOG

REVISION #	DATE	DESCRIPTION OF REVISION	PAGE #
Rev.10	4/23/2021	Change in Introduction from 40 yrs to 50 yrs of experience	2
Rev.10	4/23/2021	Updated RECo Org chart in Scope section	5
Rev.10	4/23/2021	Product changes section added	20
Rev.10	4/23/2021	Revision log incorporated	21
Rev.11	4/28/2021	Added section 2.1	3
Rev.11	4/28/2021	Added section 2.1 - Roles and Responsibilities of Compliance Control Personnel	6
Rev.12	9/27/21	Changed Rev. number in revision log to correspond with change.	22
Rev. 12	11/2/21	Updated Org chart	5

Specific Document information

The Reinforced Earth Company Procurement Department is responsible to update this manual upon any changes.

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A4.1.3 Other Components Quality Control

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A4.2 Construction

This section is intentionally left blank.

A4.2.1 Quality Control Measures During Construction

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A5.0 Performance

This section is intentionally left blank.

A5.1 Performance History

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A5.1.1 Development and Usage History

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A5.1.2 Oldest Three Structures

- Oldest structures:
  - RECo project numbers 1, 7, 14 and 16
- Oldest structures using current ribbed reinforcing strips:
  - RECo project numbers 478, 482 and 1584
- Oldest structures using HA ladder soil reinforcements:
  - RECo project numbers 7673 and 7856

RECo project number 1 (34.346523, -117.848929):

- Constructed in 1971
- Highway 39, Crystal Lake Road, Angeles National Forest, CA (Caltrans)
- Steel facing and flat (3mm thick) steel reinforcing strips

More information can be found here: https://reinforcedearth.com/project/highway-39-crystal-lake-road/

A documentary video by Caltrans can be found here:
https://www.youtube.com/watch?v=AeAW6D89aWk&t=171s



Figure A5.1.2-1: Reinforced Earth wall under construction in 1972



Figure A5.1.2-2: Reinforced Earth wall in 2021.

RECo project number 7 (40.158252, -118.490550):

- Constructed in 1973
- Big Meadow Ranch Interchange, Lovelock, NV (Nevada DOT)
- Cruciform-shape concrete facing and flat (60mm and 80mm x 3mm) steel reinforcing strips.
- The first true abutment in the United States, meaning the bridge load is bearing directly on the RE volume rather than piles.

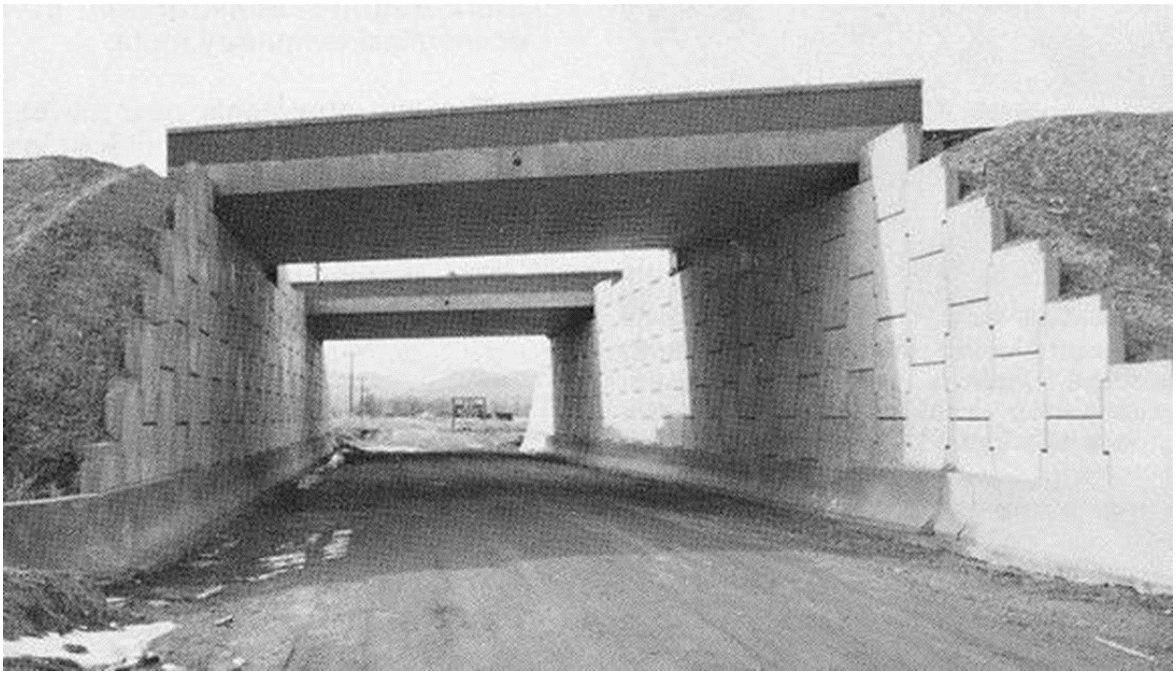


Figure A5.1.2-3: The first Reinforced Earth true abutment wall in 1973.



Figure A5.1.2-4: The first Reinforced Earth true abutment wall present day (via Google Maps)

RECo project number 14 (41.232903,-122.273308):

- Constructed in 1974
- I-5, Dunsmuir, CA (Caltrans)
- Cruciform-shape concrete facing and flat (3mm thick) steel reinforcing strips.



Figure A5.1.2-5: I-5 in Dunsmuir, CA shortly after construction

RECo project number 16 (42.957159, -115.356163):

- Constructed in 1977
- I-84, Glenns Ferry, ID (Idaho DOT)
- Cruciform-shape concrete facing and flat (3mm thick) steel reinforcing strips
- Upper rows were excavated in 2020 during re-paving. Although the AASHTO Metal Loss model considers galvanization (zinc) to be fully consumed in 16 years, there was still zinc present on many samples after 43 years of service. Full report available upon request.



Figure A5.1.2-6: I-84 in Glenns Ferry, ID shortly after construction

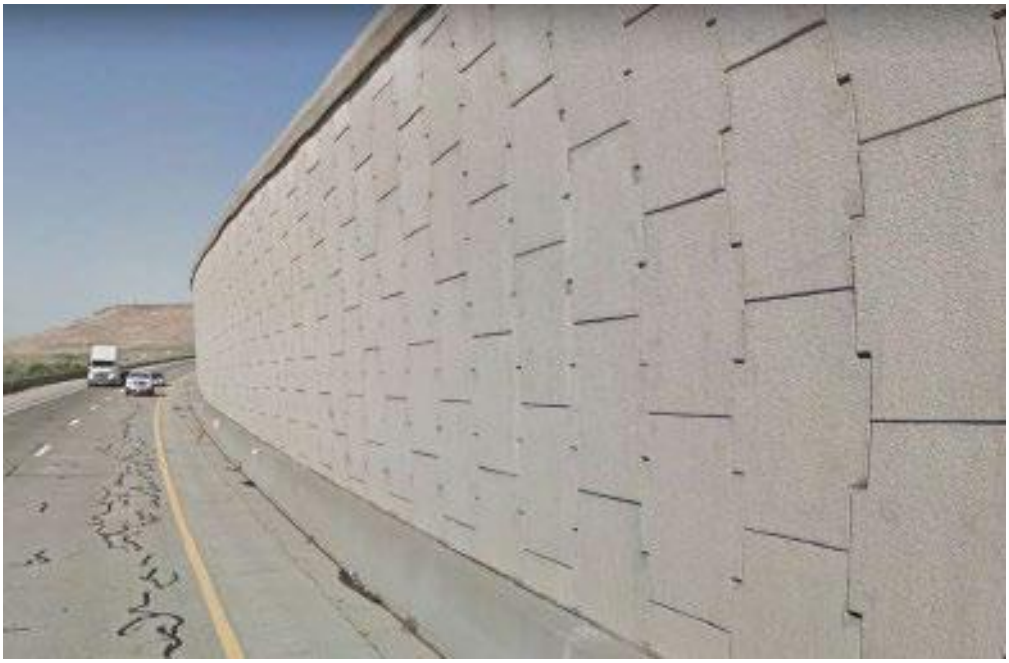


Figure A5.1.2-7: I-84 in Glenns Ferry, ID present day (via Google Maps)

RECo project number 478 (38.886452, -77.117789)

- Constructed in 1978
- I-66 at Glebe Road, Arlington, VA
- Cruciform-shape concrete facing and ribbed (40mm x 5mm and 60mm x 5mm) steel reinforcing strips



Figure A5.1.2-8: I-66 at Glebe Road shortly after construction



Figure A5.1.2-9: I-66 at Glebe Road (via Google Maps)

RECo project number 482 (61.137422, -149.925116)

- Constructed in 1978
- Campbell Creek Bridge, Anchorage, AK
- Dimond Blvd. over Campbell Creek
- Cruciform-shape concrete facing and ribbed (40mm x 5mm and 60mm x 5mm) steel reinforcing strips
- True Abutment



Figure A5.1.2-10



Figure A5.1.2-11



Figure A5.1.2-12



Figure A5.1.2-13: Walls shown supporting Dimond Blvd. over Campbell Creek (via Google Maps)

RECo project number 1584 (40.632078, -75.278255)

- Constructed in 1984
- I-78 over Reddington Road. Northhampton County, PA. (PennDot)
- Cruciform-shape concrete facing and ribbed (50mm x 4mm) steel reinforcing strips
- True Abutment

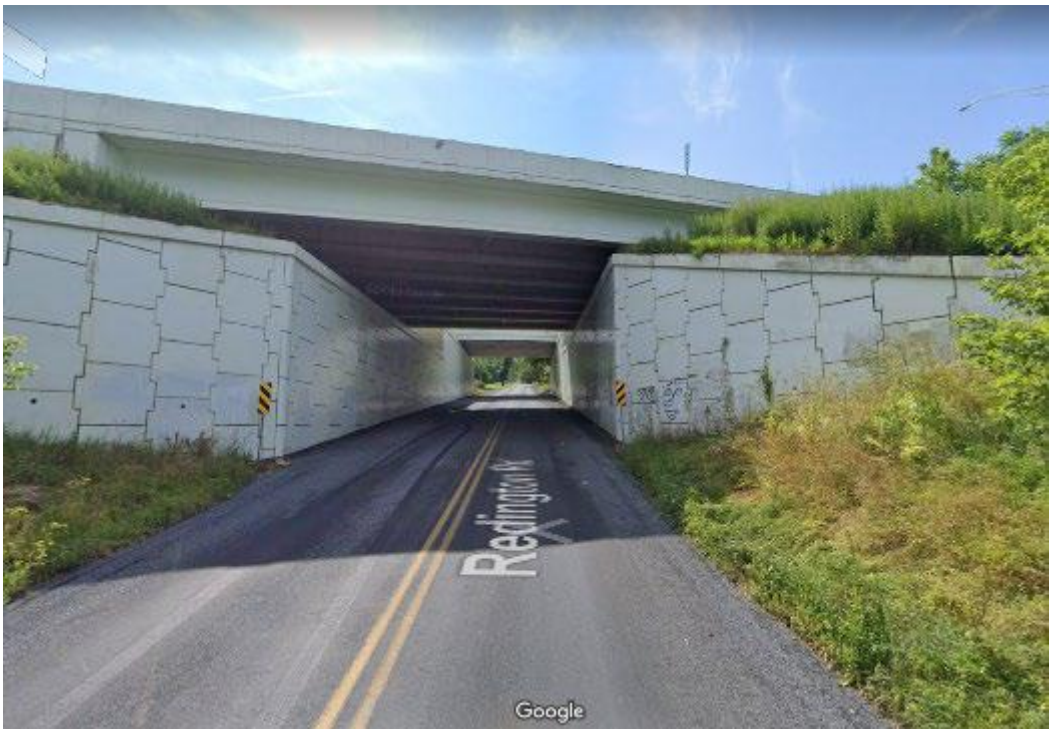


Figure A5.1.2-14: I-78 over Reddington Road present day (via Google Maps)

RECo project number 7673 (30.115478, -93.820959)

- Constructed in 1999
- I-10, Orange, TX
- Rectangular-shape panels with HA ladder soil reinforcements



Figure A5.1.2-15: I-10 RE wall shortly after construction



Figure A5.1.2-16: I-10 RE wall present day (via Google Maps)

RECo project number 7856 (38.306771, -85.752535)

- Constructed in 2000
- I-65, Jeffersonville, IN
- Cruciform-shape facing panels with HA ladder soil reinforcements



Figure A5.1.2-17: I-65 at Eastern, Blvd. present day (via Google Maps)



Figure A5.1.2-18: I-65 at E Lewis and Clark Parkway present day (via Google Maps)



Figure A5.1.2-19: I-65 at E Lewis and Clark Parkway present day (via Google Maps)

A5.1.3 Tallest Three Structures

Three tallest structures in the United States:

- Tiered:
    - RECo project number 8079
    - RECo project number 4130
    - RECo project number 6253
- Vertical
    - RECo project number 537
    - RECo project number 18408
    - RECo project number 7818

RECo project number 8079 (47.45634, -122.31921)

- Seattle-Tacoma International Airport Third Runway
- Seattle, WA
- Constructed in 2005
- 134.5 feet tall (41 m)
- Cruciform-shape facing panels with ribbed strip soil reinforcements



Figure A5.1.3-1: A section of tiered wall at Seattle-Tacoma International Airport

RECo project number 4130 (35.957415, -82.560188)

- Highway 181
- Unicoi, TN
- Constructed in 1994
- 105 feet tall (32 m)
- Cruciform-shape facing panels with ribbed strip soil reinforcements



Figure A5.1.3-2: Aerial view of the tiered wall in Unicoi, TN, shortly after construction.

RECo project number 6253 (33.88313, -84.45201)

- Kennedy Parkway Interchange
- Atlanta, GA
- Constructed in 1996
- 98.5 feet tall (30 m)
- Cruciform-shape facing panels with ribbed strip soil reinforcements
- Present day hidden behind vegetation



Figure A5.1.3-3: Kennedy Parkway Interchange tiered wall shortly after construction.

RECo project number 537

- Climax Mine
- Leadville, CO
- Constructed in 1980
- 110 feet tall (33.5 m)
- Cruciform-shape facing panels with ribbed strip soil reinforcements
- The wall was for a mining operation and may not be existing present day.



Figure A5.1.3-4: The tallest section of wall is in the upper area (center), not fully visible.

RECo project number 18408 (33.95227, -118.33956)

- SoFi Stadium
- Inglewood, CA
- Constructed in 2018
- 110 feet tall (33.5 m)
- Cruciform-shape facing panels with ribbed strip soil reinforcements.
- The wall forms a bowl around the stadium's excavated area and is obscured from view by the stadium structure.



Figure A5.1.3-5: SoFi Stadium Reinforced Earth wall at about mid construction height

RECo project number 7818 (37.672044, -77.650724)

- Route 288 over I-64
- Richmond, VA
- Constructed in 2002
- 79 feet tall (24 m)
- Cruciform-shape facing panels with ribbed strip soil reinforcements.



Figure A5.1.3-6: Aerial view of the RE wall (via Google Maps)

A5.1.4 Private and Public Approvals

This section is intentionally left blank.

A6.0 Other Information

This section is intentionally left blank.

A6.1 Any other Helpful Information

This section is intentionally left blank.