# REPORT OF REVIEW OF INITIAL TECHNICAL EVALUATION OF THE EARTHTEC™ REINFORCED SOIL WALL SYSTEM

**APRIL 2021**

**HIGHWAY INNOVATIONS, DEVELOPMENTS, ENHANCEMENTS AND ADVANCEMENTS (IDEA)**

## EARTHTEC REINFORCED SOIL WALL SYSTEM

The EarthTec™ Reinforced Soil Wall System (RSW) has been evaluated in accordance with the latest, applicable 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 the attached final submittal. Design parameters defined within the submittal are summarized in the two tables located at the end of this IDEA report.

### Applicant Information

Ground Improvement Systems
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### Review Summary

Following its initial review of the EarthTec Reinforced Soil Wall System submittal, the review team provided the applicant with four series of comments, requests for clarification, and questions. The applicant has been thorough in their responses and the review team finds that there are no outstanding issues. The IDEA Evaluation finds the EarthTec Reinforced Soil Wall system to be a technically viable MSE retaining wall system. The final version of the EarthTec submittal is dated March 2021 and it is attached to this report of review.

### Submittal Checklist

The checklist used from the IDEA protocol for this evaluation is C4 – Evaluation Checklist for Precast Concrete Panel Paired with Inextensible Reinforcement, dated November 2020 (available at https://www.geoinstitute.org/special-projects/idea).

### Confidential Information

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

**System Description**

The EarthTec Reinforced Soil Wall (RSW) System submittal presented to The Innovations, Developments, Enhancement, and Advancements (IDEA) Program for the technical evaluation is in conformance with IDEA Protocol C4. This Mechanically Stabilized Earth (MSE) retaining wall system consists of inextensible soil reinforcing paired with segmental concrete panel (SCP) facing units. The soil reinforcing consists of discrete steel ribbed strips connected to the SCP. 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 ribbed soil reinforcement strips. The SCP is a precast concrete unit that has a minimum thickness of 5 ½ inches. The standard panel is 5-feet tall by 10-feet long. There are two alternative of steel soil reinforcing elements used in the EarthTec RSW, EarthTrac™ Type SS1 and EarthTrac Type SS3. Both types are 2-inch wide discrete, inextensible steel strips with transverse ribs formed in the top and bottom surface. The two vary by the fabrication process and rib configuration. Only one type of soil reinforcement (e.g., SS1 or SS3), will be used on a given project.

**System History**

The EarthTrac system has been used in the United States since 2007. Since its inception, over 44 projects have been completed. Several projects have multiple structures. The first EarthTec retaining wall system project was constructed in July 2007 on Route 193 in Greenbelt, Maryland and was 1587 square feet in area. It has been used on both private and public projects, and to-date has been approved for use by three DOTs. The tallest EarthTec project was constructed on the Jordon Bridge in Chesapeake, Virginia and was a multitier structure that is a total of 55 feet in height and 16,682 square foot in area. Additionally, both the 11th Street Bridge in Washington D.C. and the I-95 and I-24 project in Baltimore, Maryland had several bridge abutments that were approximately 40 feet in height.

**System Innovations**

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

- The unique configuration, spacing, and offset (top surface to bottom surface) of the major and minor ribs of the EarthTrac SS1 hot rolled soil reinforcement is an innovation. This unique configuration has been purposely designed to provide enhanced pullout resistance from different soils.
- The unique shape and configuration of the transverse ribs of the EarthTrac SS3 cold-formed soil reinforcing element is an innovation. The spacing, offset, the peak, and the

valley of the top surface rib and the bottom surface rib were purposely designed to provide enhanced pullout resistance.

- The EarthTec system is designed using a proprietary MSE design software program called RSW-Calc™. This program is a design and analysis program that interfaces with AutoCAD to draw and detail the structure. Through the RSW-Calc interface, each column of panels in the AutoCAD drawing has a unique calculation performed. The software determines the required density and length of soil reinforcing. The necessary length and density are determined based on the defined material parameters and the prescribed calculation method. The program RSW-Calc design calculations can be verified using the software program MSEW.

**System Properties**

The following properties are reported by the applicant for the EarthTec reinforced soil wall system.

Soil Reinforcements

The EarthTrac Type SS1 soil reinforcements are discrete, inextensible steel strips with transverse ribs formed in the top and bottom surface. The standard size is 2 inches wide and 3/16-inch thick. Type SS1 is manufactured with low-carbon steel in conformance with ASTM A572, and the ribs are hot rolled into the strip. 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 Type SS1 soil reinforcement has tensile strengths of $F_u$ = 65 ksi and $F_y$ = 50 ksi.

The EarthTrac Type SS3 soil reinforcements are discrete, inextensible steel strips with transverse ribs formed in the top and bottom surface. The standard size is 2 inches wide and 5/32-inch thick. Type SS3 is manufactured with low-carbon steel in conformance with ASTM A1011, and with cold-formed ribs in the strip. 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 Type SS3 soil reinforcement has tensile strengths of $F_u$ = 65 ksi and $F_y$ = 50 ksi.

Soil Reinforcement-Facing Panel Connection Capacity

A common anchor is used for both types of EarthTrac soil reinforcements, for connection to the precast concrete panel facing. This panel anchor consists of a 2-inch wide, 0.135-inch thick fabricated steel strip fabricated with a 4-inch long anchorage width embedded 4 inches into the concrete panel. The connection ends form parallel steel plates, that are offset by 1/8-inch, extending out the back of the panel. Each plate has a centralized 9/16" diameter punched bolt hole. This connector is protected from corrosion through the application of a zinc coating, applied by hot-dip galvanizing in conformance with ASTM A123.

The anchor to soil reinforcement is connected in the field with a ½-inch diameter, galvanized Grade A325 bolt set.

Soil Reinforcement Design Tensile Strengths

The design tensile strength of the EarthTrac Type SS1 soil reinforcement is controlled by the strip at the bolt hole connection section. This assumes that the strip is exposed to corrosion at this location, due to the hot rolled ribs within the panel connector plates preventing a skintight assembly between the surfaces of the parallel plates and the reinforcement strip. The resistance of the strip in the connection includes both a check of a) yielding of the gross section and b) fracture of the net section surrounding the bolt hole. The yielding of the gross section factored tensile resistance is computed with the yield strength, $F_y$, and a resistance factor for yielding of tension member, $\phi_y$, equal to 0.75 per AASHTO 6.8.2.1-1. The fracture of the net section factored tensile resistance is computed with the ultimate strength, $F_u$, a resistance factor for fracture of tension member, $\phi_u$, equal to 0.80 per AASHTO 6.5.4.2, and a reduction factor for punched full size holes equal to 0.90 per AASHTO 6.8.2.2. The fracture of the net section controls the factored tensile resistance.

The design tensile strength EarthTrac Type SS3 soil reinforcement is controlled by the strip outside of the connection section. This strip does not have ribs within the connection and a skintight assembly between the faying surfaces of the parallel plates and the reinforcement strip limits corrosion. The controlling factored tensile resistance is computed with the yield strength, $F_y$, and a resistance factor for yielding of tension member, $\phi_y$, equal to 0.75 per AASHTO 6.8.2.1-1.

Pullout Design Parameters

The submittal includes two detailed pullout test reports (under appended TAB 1.2.16), one for the EarthTrac SS1 reinforcement and the other for the EarthTrac SS3 reinforcement. The test reports show, for both types of reinforcement, lower bound friction factor, F*, values equal to 3.0 at the top of the grade, decreasing linearly to 1.0 at a depth of 20 feet, and then a constant value of 1.0 below a depth of 20 feet from the top of the grade. EarthTec recommends, the use of a lower bound friction factor, F*, values equal to 3.0 at the top of the grade, decreasing linearly to 1.0 at a depth of 20 feet, and then a constant value of 1.0 below a depth of 20 feet from the top of the grade, for both types of reinforcements. However, EarthTec used the more conservative value of tan $\phi$ at a depth of 20 feet and a constant value of tan $\phi$ below a depth of 20 feet in the submitted example calculations. Furthermore, the submittal states that these friction factor values are valid for all soil types that are commonly used in MSE structures for transportation projects (i.e., material with less than 15% fines, material with 3-inch maximum). This submittal statement is based upon the tests utilizing a poorly-graded sand (SP) material (from Florida), that is considered a lower bound material by EarthTec; and that any other granular MSE wall fill material will provide better friction factors.

Retaining Wall Design

The submittal notes that this wall system is designed using a proprietary software, that interfaces with AutoCAD (for production of shop drawings). It is noted that integration of design and shop drawings is advantageous for quality control purposes. The scope of this review did not provide for a comprehensive evaluation of this computer program. However, the applicant states that the design of EarthTec structures 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 tables attached to this review report.

**Conclusions**

Following its initial review of the EarthTec submittal, the review team provided the applicant with more than 200 comments and requests for clarification. The applicant has been thorough in its responses, which have been fully incorporated into the attached, final submittal. The IDEA Evaluation finds the EarthTec Reinforced Soil Wall system to be a technically viable MSE retaining wall system.

**Closing**

An update technical evaluation should be performed for the EarthTec system in five years (i.e., March 2026) 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 1. MSEW Input Parameters for EarthTec Wall System<br>EarthTrac Inextensible Steel Strip Soil Reinforcements | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Reinforcement Type | $F_y$ (ksi) | b (in) | $A_c$ (in$^2$) | $R_c$ | | F* | | α | Thickness (in) | Width (in) |
| | | | | 5' *x* 10' panel | 10' *x* 10' panel | @ top | @ 20' depth | | | |
| SS1 | 50 | 2.0 | 0.237 | 0.050 to 0.167 | 0.067 to 0.200 | 3.0 | tan ϕ | 1.00 | 0.174 | 2.00 |
| SS3 | 50 | 2.0 | 0.202 | 0.050 to 0.167 | 0.067 to 0.20 | 3.0 | tan ϕ | 1.00 | 0.156 | 2.00 |

| Summary Table 2. MSEW Input Parameters for EarthTec Wall System<br>Segmental Concrete Panels | | | | |
|---|---|---|---|---|
| Panel Thickness | $W_u$ (ft) | γ (lb/ft$^3$) | $G_u$ (ft) | $CR_U$ |
| 5½-inches | 0.50 | 150 | 0.25 | 1.00 |
| 6-inches | 0.46 | 150 | 0.23 | 1.00 |

EARTHTEC™

# IDEA

## HIGHWAY INNOVATIONS, DEVELOPMENTS, ENHANCEMENTS AND ADVANCEMENTS

## REINFORCED SOIL WALL SYSTEM

# MARCH 2021

GIS Support Through Innovation

**GROUND IMPROVEMENT SYSTEMS LLC**
114 South Collins Street
Arlington, TX 76011

# EARTHTEC™

# IDEA

## HIGHWAY INNOVATIONS, DEVELOPMENTS, ENHANCEMENTS AND ADVANCEMENTS

## MECHANICALLY STABILIZED EARTH
## EARTHTRAC™ SOIL REINFORCING

**114 South Collins Street**

**Arlington, Texas 76011**

**817-223-0969**

## PREFACE

This submittal contains information for the EarthTec™ Reinforced Soil Wall™. The system in this submittal consists of a Segmental Concrete Panel and discrete steel ribbed strip reinforcing. The steel ribbed strip soil reinforcing tradename is the EarthTrac™. The EarthTrac™ soil reinforcing can be fabricated from different metallic materials. This submittal covers soil reinforcing that is manufactured using low-carbon steel Type ASTM A572 for the EarthTrac SS1 and ASTM A1011 for the EarthTrac SS3.

All values and specifications that are defined in this submittal should be considered the minimum acceptable values. These values may need to be adjusted to meet the specific Department of Transportation specifications and the project specifications. Also, in the submittal body, reference is made to the American Society for Testing and Materials (ASTM) specifications. The ASTM specification may have an American Association of State Highway and Transportation Official (AASHTO) equivalent specification. It is assumed that the ASTM specification is the most recent.

The MSE calculations that are included in this submittal use the Coherent Gravity method of analysis. Other methods can be used to design the EarthTec™ Reinforced Soil Wall™. An acceptable alternate method consists of the Simplified method. The calculations are intended to guide the reviewer on the methodology used to design the EarthTec™ Reinforced Soil Wall™. All projects must be designed using the appropriate project specification, including the design method, soil strength parameters, and externally applied loads. The calculations will be adjusted to meet project requirements. Although only two design cases are described in this submittal, other structure configurations can be used with the EarthTec RSW system, such as abutments supported by deep foundations and abutments that support the substructure spread-footing, tiered structures, etc.

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## INTRODUCTION

This submittal is presented to The Innovations, Developments, Enhancement, and Advancements (IDEA) Program for the technical evaluation of the EarthTec Reinforced Soil Wall (RSW) system in conformance with Protocol C4. The system presented consists of a Mechanically Stabilized Earth (MSE) retaining wall utilizing inextensible soil reinforcing paired with segmental concrete panel (SCP) facing units. The soil reinforcing consists of discrete steel ribbed strips connected to the SCP at an embedded panel anchor. The SCP is a precast concrete unit that has a minimum thickness of 5 ½ inches. The SCP standard panel is 5-feet tall by 10-feet long. The connection includes dual steel plates cast into the SCP, and that extends from the back face. The soil reinforcing is attached to the panel anchor with a bolt set so it is snug-tight.

## 1.0 ERS COMPONENTS

The components for the EarthTec™ Reinforced Soil Wall consist of a facing unit and inextensible soil reinforcing. The facing unit consists of a segmental concrete panel. The inextensible soil reinforcing consists of a discrete steel ribbed strip called EarthTrac™. For this submittal, two steel soil reinforcements are being reviewed and include the SS1 and SS3. The SS1 and SS3 vary by geometry and rib manufacturing process. Only one soil reinforcing type, i.e., SS1 or SS3, will be used on a given project. Only one standard size of reinforcing will be used on a project with varying horizontal spacings to meet design requirements. Additionally, there are incidental components used with the RWS system and include but are not limited to compacted select backfill, nuts and bolts, coping and barriers, bearing pads, joint geotextile, joint adhesive, and drainage systems. Each of these items will be discussed in this section.

### 1.1 Facing Units

The EarthTec™ system uses precast concrete facing units that are called segmental concrete panels (SCP). The SCP units can be manufactured to any functional dimension. The standard nominal SCP height is 5'-0". There are two standard nominal SCP lengths and include 5'-0" and 10'-0". The thickness of the panel varies based on the design requirements. For traditional concrete mix designs, the minimum SCP thickness is 5 ½". The SCP is reinforced with steel welded wire mesh or mild steel reinforcing bars. The standard height SCP, i.e., 5'-0", has two rows of panel anchors. The number of rows of panel anchors varies and is based on the height of the SCP but typically ranges from one row to three rows. Panels to a height of 2'-6" have one row of soil reinforcing. Panels heights greater than 2'-6" but less than 5'-6" have two rows of soil reinforcing. Panels with heights greater than 5'-6" to the maximum 7'-6" panel height have three panel anchors rows. The SCP is cast face down in steel forms. Form liners can be used to provide an architectural finish. Depending on the type of form liner finish, the SCP thickness will increase.

### 1.1.1 Facing Unit Innovations

There are no facing unit innovations claimed.

### 1.1.2 Facing Unit Types

The EarthTec™ facing units include the following standard units:

1. Standard square and rectangular panel
2. Slip joint panel
3. Adjustable corner panel

### 1.1.3 Facing Unit Specification

The generic SCP Specification is provided on pages 1 through 6 of the Procedures and Quality Control Guidelines, Precast Segmental Concrete Panel document appended in Appendix in TAB 1.1.3. The mix design and the minimum compressive strength varies from project to project.

### 1.1.4 Facing Unit Dimensions

The nominal panel dimension width is 5’-0” or 10’-0”. The standard nominal panel height is 5’-0” (Type-G). The cast dimension is ½” larger than the nominal dimension. The standard panel joints for both horizontal and vertical joints is ¾”. The standard nominal panel heights range from 2’-0” to 7’-6” in 6” increments as describe in Table 1. There are two additional standard panel types identified as Type-Y and Type-Z to accommodate leveling course steps in increments of 15-inches (1.25 feet). The panel reinforcing is ASTM A1064 Grade 65 welded wire mesh or ASTM A615 Grade-60 reinforcing steel. The standard facing unit dimensions, joint dimensions, reinforcing dimensions and grades, and general notes are shown in Appendix TAB 1.1.4. The standard precast specification and quality control manual is located in Appendix TAB 1.1.3 and provides information on casting tolerances and reinforcing type and grade.

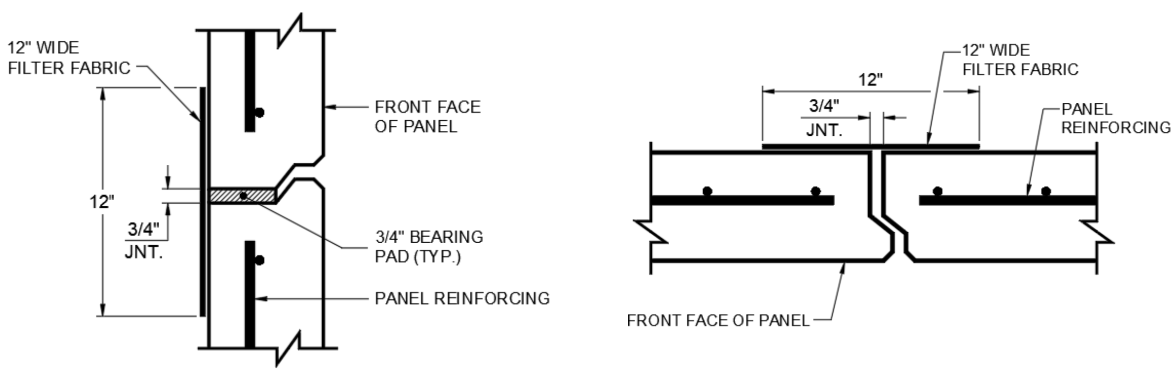
**Table 1 Standard Nominal Panel Sizes**

| Panel Designation | Nominal Height |
|---|---|
| A | 2’-0” |
| B | 2’-6” |
| C | 3’-0” |
| D | 3’-6” |
| E | 4’-0” |

**Table 1 Standard Nominal Panel Sizes**

| Panel Designation | Nominal Height |
|---|---|
| F | 4’-6” |
| G | 5’-0” |
| H | 5’-6” |
| J | 6’-0” |
| K | 6’-6” |
| L | 7’-0” |
| M | 7’-6” |
| Y | 3’-9” |
| Z | 6’-3” |

The panel joints for both the vertical and horizontal joint are shiplap. The standard joint spacing for both the vertical and horizontal joint is ¾” as shown in Figure 1-1. The horizontal joint is maintained using a ¾” bearing pad. The vertical joint is maintained by the means and methods of the installer during placement of the panel in the wall.




Horizontal Joint Vertical Joint

**Figure 1-1 SCP Panel Joint**

### 1.1.5 Facing Unit Compressive Strength

The minimum compressive strength required for the SCP, using standard concrete mix design, is 4000 psi. Other higher compressive strengths can be used. The concrete producer must select a mix design that will achieve the specified minimum compressive strength 28 days after fabrication. It should be noted that 4000 psi is the minimum strength after 28 days and is not representative of the strength that can be attained before 28 days. A target compressive strength of 1500 psi should be reached before stripping, handling, and shipping the panel. All concrete mix design shall be following project specifications. Determine maximum aggregate size in accordance with AASHTO LRFD standards and the governing specification.

### 1.1.6 Facing Unit Percent Air Entrainment

Air entrainment is required where the facing unit is exposed to freezing and thawing environments. The required air entrainment should follow project specifications. In the absence of air entrainment specifications, EarthTec™ guides the precaster and relies on the recommendations of the Portland Cement Associations (PCA) *Manual on Control of Air Content in Concrete (PCA EB116*). Air entraining is incorporated into the facing unit using admixtures or agents. EarthTec™ requires the admixtures that are used to produce a stable system of entrained air and meet the project specifications.

The entrained air generates discrete air voids in the concrete. The intent of the air-entrainment, i.e., air voids or empty spaces, within the concrete, is to provide a reservoir for the freezing water to expand unobstructed. The reservoirs act to relieve pressure preventing damage to the concrete. The air content is a function of the aggregate size and environmental exposure. The PCA recommends the air content for different exposures be as shown in Table 2.

**Table 2 Recommended Total Air Content for Concrete (PCA)**

| Nominal Maximum Aggregate Size (in) | Air Content (Percent) | | |
|---|---|---|---|
| | Severe Exposure[1] | Moderate Exposure[2] | Mild Exposure[3] |
| 3/8 | 7.5 | 6 | 4.5 |
| 1/2 | 7.0 | 5.5 | 4.0 |
| 3/4 | 6.0 | 5.0 | 3.5 |
| 1 | 6.0 | 4.5 | 3.0 |

1. Severe Exposure - Concrete exposed to wet-freeze-thaw conditions, deicers, or other aggressive agents.
2. Moderate Exposure - Concrete exposed to freezing but not continually moist, and not in contact with deicers or aggressive chemicals.
3. Mild Exposure - Concrete not exposed to freezing conditions, deicers, or aggressive agents.

### 1.1.7 Facing Unit Mix Designs

The Owner typically specifies the SCP mix design. The EarthTec™ system relies on the Precaster to develop the mix design to meet the project specifications. This typically requires the Precaster to develop the fresh concrete properties, gradation, required mechanical properties of hardened concrete (strength and durability), and the inclusion, exclusion, or limits on specific ingredients. The concrete mix design shall have acceptable workability, durability, strength, and uniform appearance of the hardened concrete that meets the project requirements. EarthTec requires a minimum 4000 psi compressive strength (28-day) and suggests that the panels not be stripped from the form until a minimum 1500 psi compressive strength has been reached. The maximum aggregate size shall be in accordance with AASHTO LRFD standards and the governing specification but shall be no larger than ¾".

### 1.1.8 Facing Unit Alignment Pin and Bearing Pad

An alignment pin is not required to be used with this system.

The bearing pads are placed on the top of the facing panel on the shoulder. The top of the facing unit is cast with a shiplap configuration. The top panel shiplap joint consists of a cheek and a shoulder. The cheek is at the unit's front face, and the shoulder is at the unit's back face. The cheek hides the shoulder and the bearing pad. The shoulder is the surface that the bearing pad is placed (Figure 1-2). The flat surface at the bottom of the panel is placed on the bearing pad.

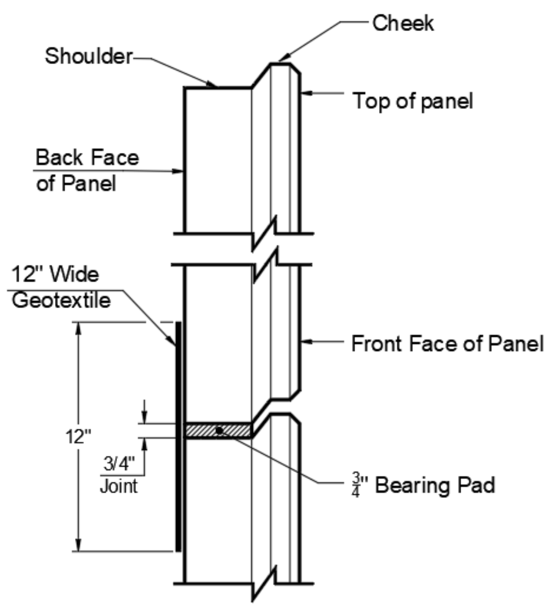



**Figure 1-2 Side-View of Shiplap Panel Joint**

The bearing pads shall consist of 3" x 6" x ¾" SBR rubber with a 60 Duro Shore-A. The bearing pad is supplied by EarthTec™ and shall conform to ASTM D2240 and ASTM D2000. The horizontal panel joint is ¾" as shown in Figure 1-3. Bearing pad test for the ¾" SRB bearing pad is contained in Appendix TAB 1.1.8.

Bearing pads are placed on the shoulder behind the cheek of the shiplap joint. Two bearing pads are required for every five feet of panel. A minimum of two bearing pads shall be used on the standard

5'-0" panel, and a minimum of four bearing pads are required to be used with the standard 10'-0" panel. At panel heights that exceed 40 feet, a minimum of four bearing pads shall be used on the standard 5'-0" panel, and a minimum of eight bearing pads shall be used with the standard 10'-0" panel.

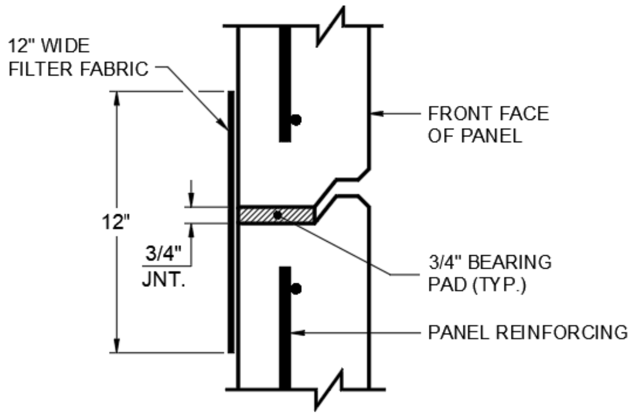



**Figure 1-3 Bearing Pad Placement at the Top of Panel**

### 1.1.9 Facing Unit Joint Filter

A 12-inch wide nonwoven, needle punched geotextile is used at the interface of all facing unit joints. The geotextile is placed on the back face of the panel using an adhesive compound. EarthTec supplies the geotextile, and the wall contractor supplies the adhesive. The adhesive shall be placed in an amount where the geotextile makes intimate contact with the panel and remains so during fill placement and compaction. This is especially critical with sand backfill. Once the backfill is placed, the geotextile cannot move. The adhesive usually consists of a subfloor adhesive that can be applied to concrete.

**Figure 1-4 Application of Geotextile Adhesive**

The geotextile shall be permeable and provide good flow to remove water from the MSE backfill by passing it through the facing joints. The geotextile ability to allow water to flow through it is referred to as hydraulic conductivity by permittivity. The geotextile is designed and specified to allow water flow in the normal direction while inhibiting soil particles' movement from the MSE backfill. The apparent opening size, or the AOS, is a common physical property tested on geosynthetic fabrics used for filtering. Project specifications for geotextile can vary between projects and jurisdictions. The minimum values required by EarthTec™ are shown in Table 3. Reference Appendix TAB 1.1.9. The AASHTO M288-17 can be used as a guide. AASHTO M288-17 addresses the following applications: Subsurface Drainage, Stabilization, Separation, Permanent Erosion Control, Sediment Control, and Paving Fabrics. The user of the specification must be cautioned that AASHTO M288-17 is not a design guideline.

**Table 3 Sample Minimum Geotextile Criteria**

| Permittivity ($Sec^{-1}$) | AOS (Sieve #) | Minimum Tensile Strength (lbf) | Minimum Puncture (lbf) | Minimum Trapezoidal Tear (lbf) | UV Resistance (Min. Allowed) | |
|---|---|---|---|---|---|---|
| | | | | | % | (Hours) |
| ASTM D 4491 | ASTM D 4751 | ASTM D 4632 | ASTM D 6241 | ASTM D 4533 | ASTM D 4355 | |
| 0.50 | 40 | 300 | 100 | 125 | 50 | 500 |

The geotextile requirements may be project-specific. The EarthTec supplier and end-user should verify the required geotextile properties.

### 1.1.10 Facing Unit Aesthetic Options

The EarthTec™ facing unit is cast face down in steel forms. The forms are designed to allow for the use of form liners to provide an architectural appearance. Typical facing options are smooth grey, fractured fin, ashlar stone, random rake, among others. The form liner should not decrease the structural thickness of the segmental concrete panel. The color photos showing texture options are presented in Appendix TAB 1.1.10.

### 1.1.11 Facing Unit Alignment Requirements for Curves and Corners

Panels can be placed on curves of varying radius by placing them in a series of cords. The size of the radius will dictate the cord length, i.e., the length of the panel and the required panel joint configuration. The tighter the radius, the smaller the panel length will be. Further, a tight radius may require that the panel joints be cast with unique edge treatments, e.g., beveled or cropped. Typically, for the 10’ panel a radius less than 160 feet requires that the standard joint be modified and for the 5’ panel a radius less than 80 feet requires that the standard joint be modified.

Corner panels are used for both inside and outside corners. The EarthTec™ corner panel is adjustable. If the corner angle is less than 70-degrees, the MSE structure may be required to be designed using bin pressure theory. When small corner angles are used, the panels at the adjacent sides of the corner element may be required to be attached to one another using special steel panel-to-panel connectors.

### 1.2 Inextensible Reinforcements

The soil reinforcing that is used in the EarthTec™ RSW uses the trade name EarthTrac™. There are two EarthTrac™ steel soil reinforcing elements reviewed in this submittal and include type SS1 and type SS3. The EarthTrac™ SS1 and SS3 soil reinforcing is a 2” wide discrete, inextensible steel strip with transverse ribs. The lead end of the EarthTrac™ SS1 and SS3 has a centralized 9/16” hole punched into it. The centralized hole is used to attach it to the SCP at the location of the panel anchor. Each of the type SS1 and type SS3 has transverse ribs formed in the top and bottom surface. The type SS1 and type SS3 are described in Section 1.2.2. Only one type of soil reinforcement (e.g., SS1 or SS3), as selected by EarthTec, will be used on a project.

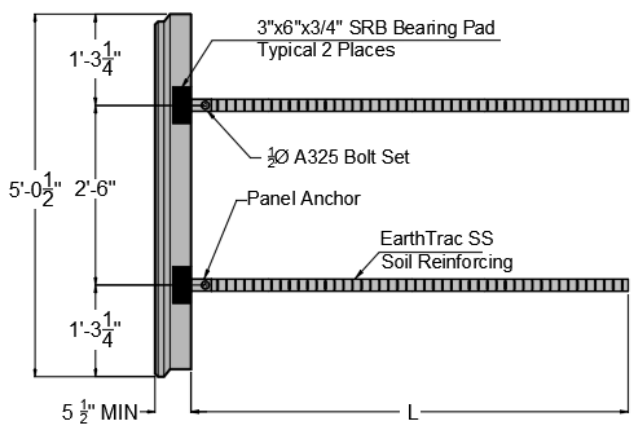



**Figure 1-5 Plan View EarthTrac™ Soil Reinforcing**

#### 1.2.1 Soil Reinforcing Innovations

The EarthTrac™ SS1 hot rolled soil reinforcing innovation is the unique configuration, spacing, and offset (top surface to bottom surface) of the major and minor ribs. This unique configuration has been purposely designed so it provides superior pullout resistance from different soils. The soil reinforcing element shown in Figure 1-6 is manufactured using A572 Grade-50 steel. The complete geometry of the SS1 is shown in Appendix TAB 1.2.2.

The EarthTrac™ SS3 is a cold-formed soil reinforcing element. The SS3 innovation is the unique shape and configuration of the transverse ribs. The spacing, offset, the peak, and the valley of the top surface rib and the bottom surface rib were purposely designed to provide superior pullout resistance. The soil reinforcing element shown in Figure 1-7 manufactured from A1011 Grade-50 steel is being reviewed in this submittal. The complete geometry of the SS3 is shown in Appendix TAB 1.2.2.

### 1.2.2 Soil Reinforcing Types

#### 2.2.1.1 EarthTrac™ Type SS1

The EarthTrac™ SS1 consists of a discrete steel strip. The EarthTrac™ SS1 soil reinforcing is manufactured at a thickness equal to 3/16”. The EarthTrac™ SS1 soil reinforcing is manufactured using low-carbon steel in conformance with ASTM A572. The ribs are hot rolled into the strip. The standard size Type SS1 reinforcement is 2 inches wide, 3/16-inch thick, and with major and minor rib configuration, as shown in Figure 1-6. The geometric configuration of the SS1 can be found in Appendix TAB 1.2.7.

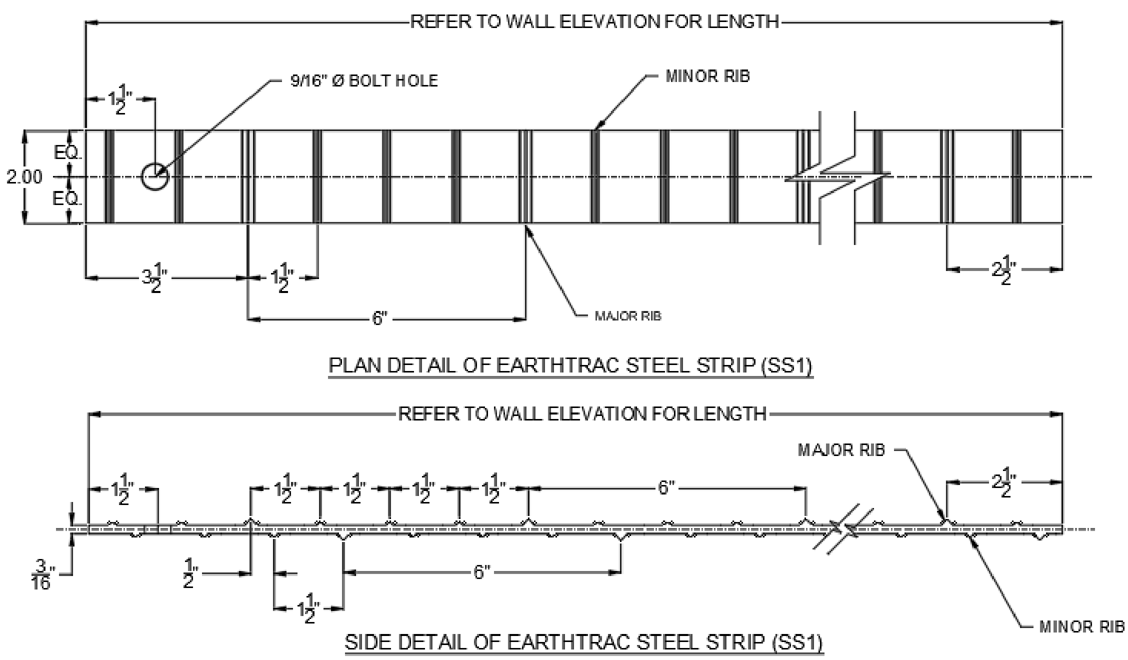



**Figure 1-6 Standard EarthTrac™ SS1 Soil Reinforcing**

#### 2.2.1.2 EarthTrac™ Type SS3

The EarthTrac™ SS3 consists of a discrete steel strip with cold-formed ribs. The EarthTrac™ SS3 soil reinforcing is manufactured with a thickness equal to 5/32. The EarthTrac™ SS3 soil reinforcing is manufactured using low-carbon steel in conformance with ASTM A1011. The standard size Type SS3 reinforcement is 2 inches wide, 5/32-inch thick, and with major and minor rib configuration, as shown in Figure 1-7. The geometric configuration of the SS3 can be found in Appendix TAB 1.2.7.

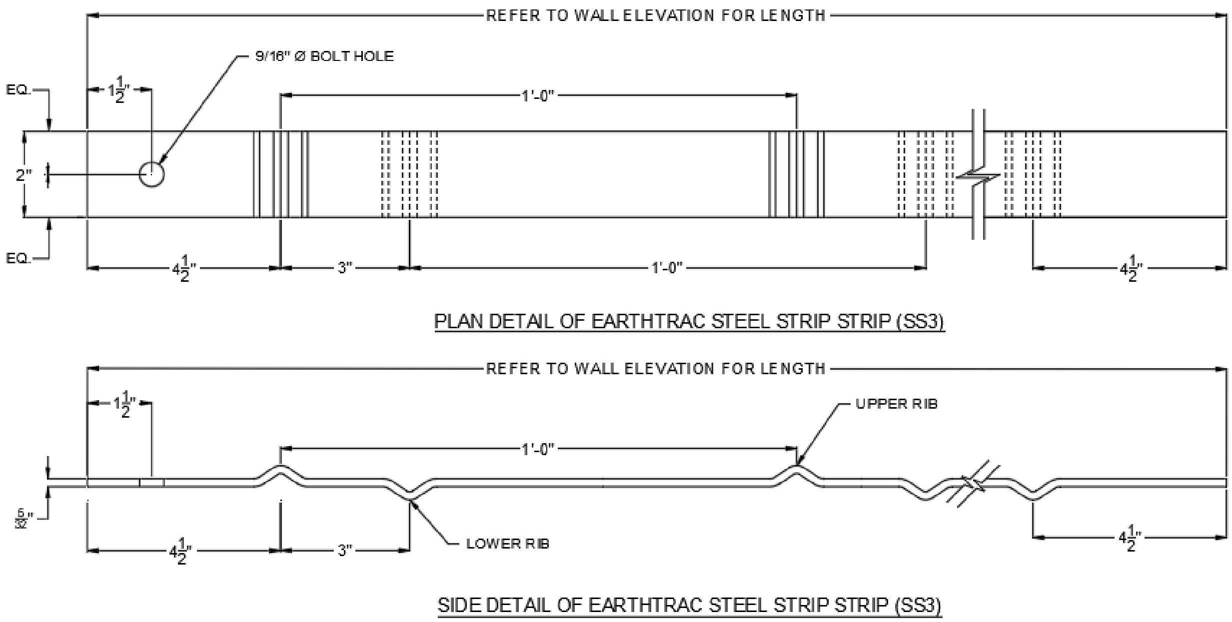



**Figure 1-7 Standard EarthTrac™ SS3 Soil Reinforcing**

### 1.2.3 Soil Reinforcing Properties

The EarthTrac™ SS1 soil reinforcing is manufactured from steel, meeting the requirements of ASTM A572 Grade-50 (yield strength equal to 50 ksi). The EarthTrac™ SS3 soil reinforcing is manufactured from steel, meeting the requirements of ASTM A1011 Grade-50 (yield strength equal to 50 ksi).

### 1.2.4 Soil Reinforcing Corrosion Protection

The EarthTrac™ SS1 and SS3 soil reinforcing is protected from corrosion through the application of a zinc coating. The zinc coating is applied by the method of hot-dip galvanizing in conformance with ASTM A123. The EarthTrac™ is galvanized then shipped to the project site. All galvanizing is performed after the complete fabrication of the EarthTrac™ soil reinforcing.

### 1.2.5 Soil Reinforcing Sacrificial Steel as a Function of Service Life

The steel area of the EarthTrac™ SS1 and SS3 soil reinforcing is corrected for the loss of corrosion in structures buried in soil. The sacrificial steel loss is determined using Equation 1 as specified in AASHTO Article 11.10.6.4.2a-1. The sacrificial steel loss is a function of the anticipated service life.

$$E_c = E_n - E_s \qquad \text{Equation 1}$$

where:

| | | | |
|---|---|---|---|
| where: | $E_c$ | = | thickness of metal reinforcement at the end of service life as shown in AASHTO Figure 11.10.6.4.1-1 (mil) |
| | $E_n$ | = | nominal thickness of steel reinforcement at construction (mil) |
| | $E_s$ | = | sacrificial thickness of metal expected to be lost by uniform corrosion during the service life of the structure (mil) |

The service life for permanent MSE structures is typically 75 years. For critical structures, i.e., bridge abutments, the service life is sometimes increased by 25 years for a total service life equal to 100 years. Therefore, $E_c$ will vary for each service life. For structural design, corrosion loss, $E_{c,}$ is deducted from the thickness of the strip. The following metal loss rates are used in the determination of the sacrificial thickness.

- Loss of galvanizing equal to 0.58 mil/yr for the first two years and 0.16 mil/yr for each subsequent year.
- Loss of carbon steel equal to 0.47 mil/yr after the depletion of the zinc.

The zinc coating, also known as galvanizing, is applied by the hot-dip method in conformance with AASHTO A123. The galvanized coating is required to be applied at a minimum of 2 oz./ft$^2$ or 3.4 mils in thickness. The sacrificial thickness of metal calculated to be lost, per Equation 1, at the end of the 75-year service life is equal to 0.056 inches. At the end of 100-year service life the sacrificial thickness of metal calculated to be lost, per Equation 1, is equal to 0.079 inches.

### 1.2.6 Soil Reinforcing Corrosion Testing

The EarthTrac™ utilizes the corrosion models specified in the AASHTO LRFD Bridge Specification. Therefore, no independent corrosion testing has been performed.

### 1.2.7 Soil Reinforcing Dimensional Tolerances

The dimensional tolerances are shown in Appendix TAB 1.2.7.

### 1.2.8 Soil Reinforcing Connection

The EarthTrac™ soil reinforcement is connected to the panel anchor by inserting the proximal end between the gap formed by the TSG10 panel anchor's parallel plates, and a galvanized F3125 Grade A325 bolt set is passed through the aligned bolt holes as shown in Figure 1-8. A nut is placed on the end of the A325 bolt and is then tightened to snug-tight. The connection is a single point connection that allows the EarthTrac™ soil reinforcing to be rotated in the horizontal plane. The ability of the reinforcing to rotate allows it to avoid obstructions. The soil reinforcement should be rotated before tightening the nut.

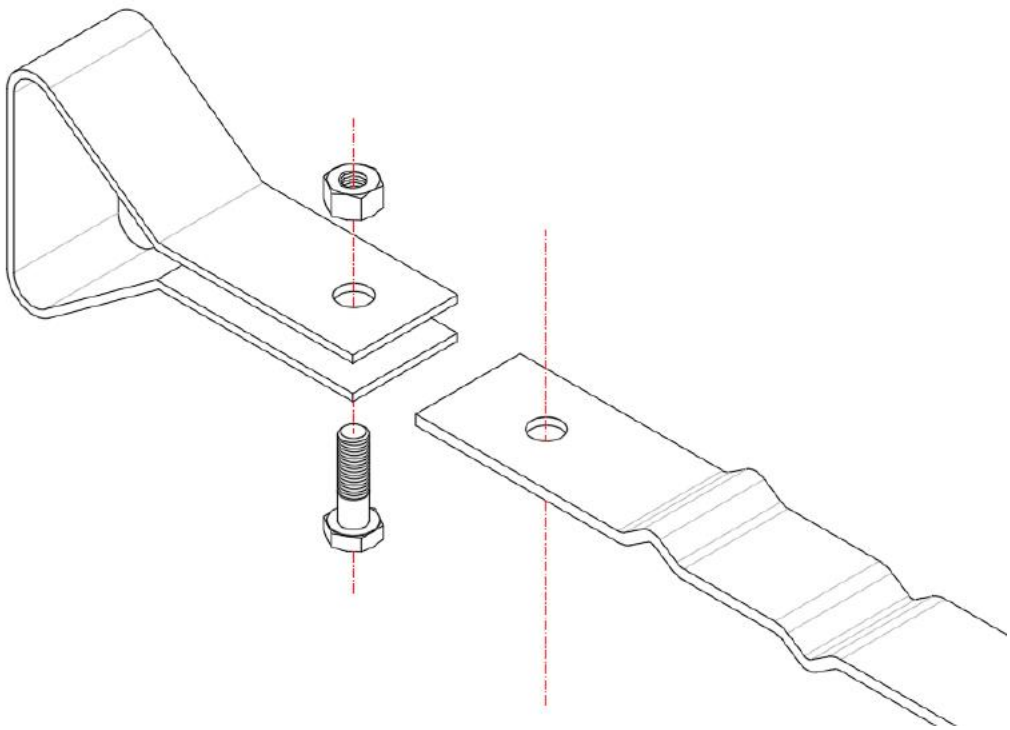


**Figure 1-8 Isometric of EarthTrac™ SS3 Soil Reinforcing Connection**

### 1.2.9 Soil Reinforcing Connection Components

The SCP panel anchor that is cast into the panel and used to attach the EarthTrac™ soil reinforcing consists of a special shaped steel plate shown in Figure 1-9. The panel anchor is cast into the SCP, so it extends from the back face. The panel anchor that is used by EarthTec™ is classified as the TSG10. The minimum ebedment of the anchor is 4”.

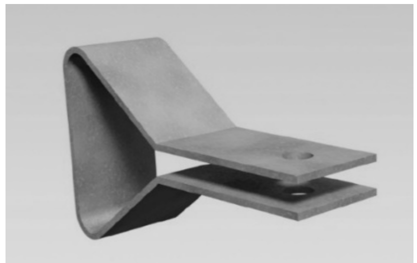


**Figure 1-9 TSG10 Panel Anchor**

The TSG10 panel anchor consists of a fabricated steel plate that is 2” in width, and that is 0.135” in thickness. The anchor is fabricated with the connection ends forming parallel steel plates that are offset from one another by 1/8”. Each plate has a centralized $^{9}/_{16}$” diameter punched bolt hole. The panel anchor is embedded into the panel a minimum of 4”. The end of the anchor cast into the panel is fabricated into a triangular configuration and positioned behind the panel reinforcing. This anchoring system is consistent with the anchoring system used by several commercially available MSE retaining wall systems and has been successfully used for over 40 years with no problems. Calculations of the anchoring system are provided in Appendix TAB 1.2.10.

The maximum number of panel anchor locations that can be cast into one row of the 5 x 5 panel is equal to 15 and in one row of the 5 x 10 panel is equal to 31 shown in Figure 1-10. These optional anchorage arrangements allow for the unique placement of the soil reinforcement strips to avoid obstructions and accommodate different load cases and design conditions. The total number of anchors in each row of the panel depends on the number of soil reinforcing elements required to be attached to the panel. Typically no more than 5 anchors are cast in a row for the 5x5 panel, and eight anchors are cast in a row for the 5x10 panel. The panel anchors should be spaced a minimum of 7.5” apart.

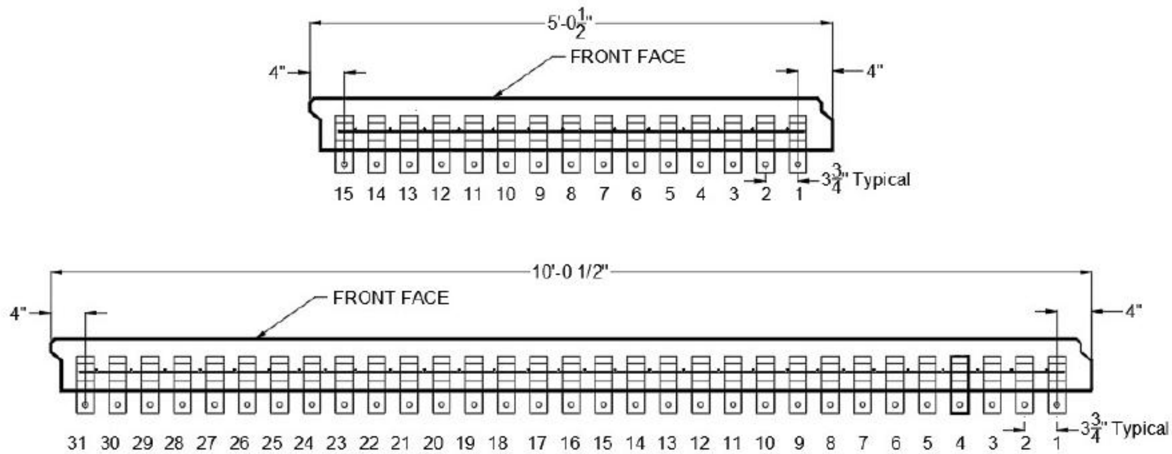



**Figure 1-10 Plan View Facing Unit Panel Anchor Layout**

### 1.2.10 Soil Reinforcing Connection Properties

The soil reinforcing connection is designed so that the limiting component is the EarthTrac™ soil reinforcing. The SCP TSG10 anchor is fabricated in conformance with ASTM A572 or A1011 Grade-50 requirements. The minimum tensile strength is 65 ksi and the minimum yield strength is 50 ksi. The ½" diameter bolt is fabricated in conformance with ASTM F3125 Grade A325. The minimum tensile strength is 120 ksi and the minimum yield strength is 92 ksi at 14% elongation and 14% reduction in area. The shear capacity of the F3125 Grade A325 bolt is 72 ksi.

### 1.2.11 Soil Reinforcing Connection Corrosion Protection

The EarthTrac™ connection components are protected from corrosion through the application of a zinc coating. The zinc coating is applied by the method of hot-dip galvanizing in conformance with ASTM A123. For small component parts, the zinc coating is applied by the method of spin-dip in conformance with ASTM A153. The EarthTrac™ components are galvanized in their finished state. In other words, once the components are galvanized, no further fabrication is performed on the element.

### 1.2.12 Soil Reinforcing Connection Sacrificial Steel as a Function of Service Life

The steel area of the EarthTrac™ soil reinforcing is corrected for the loss of corrosion. The sacrificial steel loss is determined using Equation 1 as given in AASHTO Article 11.10.6.4.2a-1 and as shown again in Equation 2.

$$E_c = E_n - E_s \qquad \text{Equation 2}$$

where: $E_c$ = thickness of metal reinforcement at the end of service life as shown in AASHTO Figure 11.10.6.4.1-1 (mil)

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

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

The service life for permanent MSE structures is typically 75 years or 100 years and is dependent on the critical nature of the structure. Therefore, $E_c$ will vary for each service life. The following metal loss rates are used in the determination of the sacrificial thickness.

- Loss of galvanizing equal to 0.58 mil/yr for the first two years and 0.16 mil/yr for each subsequent year.
- Loss of carbon steel equal to 0.47 mil/yr after the depletion of the zinc.

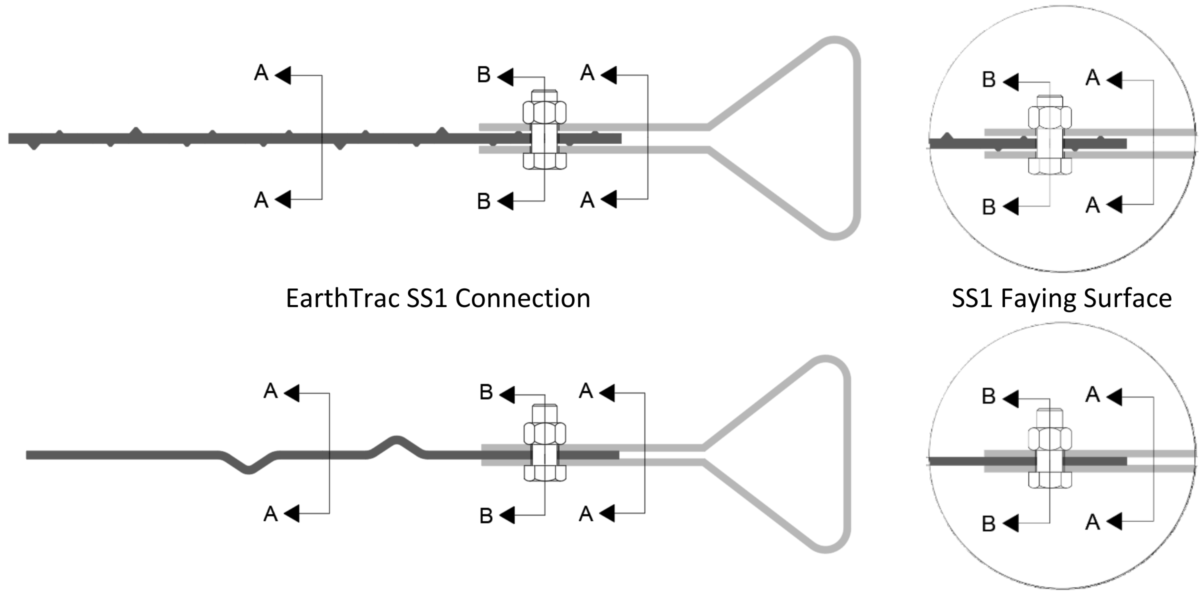
The zinc coating, also known as galvanizing, can be applied by the spin-dip method in conformance with AASHTO A153 or hot-dip in conformance with ASTM A123. The galvanized coating is required to be applied at a minimum of 2 oz./ft$^2$ or 3.4 mils in thickness. The sacrificial thickness of metal expected to be lost at the end of the 75-year service life is equal to 0.056 inches. At the end of the 100-year service life, the sacrificial thickness of metal expected to be lost is 0.079 inches.

The SS1 and SS3 soil reinforcement are connected to the panel anchor with an F3125-Grade A325 bolt set that is tightened to snug-tight with a wrench. At snug-tight the SS1 soil reinforcing will not provide full faying surface contact because of the minor rib configuration that is in the plane of the connection (Figure 1-11). As such corrosion is considered at the surface of the SS1. The SS3 soil reinforcement has no rib in the faying surface and will make full contact when the connection is snug-tight (Figure 1-11). As such, corrosion is not considered at the surface. The results of the controlling maximum allowable tension for the SS1 and SS3 soil reinforcing is shown in Table 4 and the calculation cross-section in Figure 1-11.

**Table 4 EarthTrac SS1 and SS3 Connection Calculation at 75 Year Design Life**

| Element | Strip | | Connection | | Anchor | Equivalent | |
|---|---|---|---|---|---|---|---|
| Location | A-A | B-B | A-A | B-B | Pullout | Area$_c$ | Thickness$_c$ |
| SS1 | 9.90 kip | 8.88 kip | 10.72 kip | 9.62 kip | 12.14 kip | 0.237 in$^2$ | 0.118 in |
| SS3 | 7.56 kip | 10.51 kip | 10.72 kip | 9.62 kip | 12.14 kip | 0.202 in$^2$ | 0.101 in |

- c denotes that corrosion has been considered




EarthTrac SS1 Connection    SS1 Faying Surface

EarthTrac SS3 Connection    SS3 Faying Surface

**Figure 1-11 Connection Calculation Cross Section EarthTrac**

The controlling Article in AASHTO is 6.8.2.1 equation 6.8.2.1-2 for the SS1 soil reinforcing and equation 6.8.2.1-1 for the SS3.

| | | |
|---|---|---|
| $P_r = \phi_y \cdot F_y \cdot A_g$ | AASHTO 6.8.2.1-1 | Equation 3 |
| $P_r = \phi_u \cdot F_u \cdot A_n \cdot R_p \cdot U$ | AASHTO 6.8.2.1-2 | Equation 4 |

| | | | |
|---|---|---|---|
| Where: | $P_r$ | = | Factored Tensile Resistance (kip) |
| | $\phi_y$ | = | Resistance factor for yield strength Article 11.5.7 and Table 11.5.7-1 |
| | $F_y$ | = | specified minimum yield strength (ksi) |
| | $A_g$ | = | gross cross-sectional area of member (in$^2$) |
| | $\phi_u$ | = | resistance factor for tensile strength |
| | $F_u$ | = | specified tensile strength (ksi) |
| | $A_n$ | = | net area of the member as specified in Article 6.8.3 (in$^2$) |
| | $R_p$ | = | reduction factor for holes taken as 0.90 for bolt holes punched full size |
| | $U$ | = | reduction factor to account for shear lag 1.0 for components where the force effects are transmitted to all elements – Article 6.8.2.2 |

### 1.2.13 Soil Reinforcing Connection Corrosion Testing

The EarthTrac™ utilizes the corrosion models specified in the AASHTO LRFD Bridge Specification. Therefore, no independent corrosion testing has been performed or is required to be performed.

### 1.2.14 Soil Reinforcing Connection Dimensional Tolerances

The dimensional tolerances for each of the components are shown in the Appendix TAB 1.2.14

### 1.2.15 Soil Reinforcing Connection Strength and Testing

The connection of the EarthTrac™ to the panel anchor is similar to connections used in the MSE industry for over 40 years. The connection is proven, and no connection testing is provided. The connection calculations are provided in Appendix TAB 1.2.10.

### 1.2.16 Soil Reinforcing Pullout Testing and Results

The pullout tests for the EarthTrac™ have been performed to establish the friction factor used in designing and analyzing the Reinforced Soil Wall™ system. The testing was performed at the GIS Civil Products Laboratory located in Arlington, Texas, under the supervision of The Collin Group, LLC. The testing was performed using state-of-practice procedures in general conformance with ASTM D 6706 "Standard Test Method for Measuring Geosynthetic Pullout Resistance in Soil."

Both the SS1 and SS3 were tested in a poorly-graded sand (SP) material. The poorly graded sand used in this experimental program was obtained from Florid and is considered a lower bound material. Any other granular material, especially well-graded material, will provide better friction factors. Rone Engineering of Dallas, Texas, determined the soil strength properties. According to the Unified Soil Classification System, the material was determined to be non-plastic (ASTM D4318) and classifies as SP-SM (ASTM D2487). The coefficient of uniformity, Cu, of this sandy backfill is 4.0, which is the default Cu value recommended by AASHTO when a specific Cu is not known for the wall backfill at the time of the wall design. The backfill has a maximum dry unit weight of 98.70 pcf and optimum water content equal to 17.1% as established through testing conducted in accordance with ASTM. The sand internal friction angle was determined by direct shear at optimum moisture content equal to 17.1% and the dry of optimum water content equal to 6.7% percent. Based on the direct shear test, the internal friction angle was equivalent to 34 degrees and 35 degrees, respectively.

Both the EarthTrac™ SS1 and EarthTrac™ SS3 use lower bound friction factor values. The friction factor is equal to 3.0 at the top of the grade, decreasing linearly to 1.0 at a depth of 20 feet and below the top of the grade. These values are higher than the default values reported in the AASHTO Article 11.10.6.3

and Figure 11.10.6.3.2-2 for other ribbed strip systems. The values in AASHTO are for the Reinforced Earth High-Adherence soil reinforcing system. The default values in AASHTO are known to be conservative. The higher values for the SS1 and SS3 should be expected because the shape, height, and density of the ribs vary from the strip reported in AASHTO. The pullout values are justified based on the age of the AASHTO Figure 11.10.6.3.2-2 and pullout tests and testing performed by other researchers such as the Texas Department of Transportation in conjunction with Texas Tech University (Lawson et al., 2012). The SS1 pullout resistance was activated before 3/4-inch of displacement and slowly decreased. The SS3 soil reinforcing continues to gain pullout resistance after ¾-inch of displacement. The pullout test reports are provided in the Appendix in TAB 1.2.16.

#### 1.2.17 Soil Reinforcing Interface Shear Connection Strength

Not applicable.

### 1.3 OTHER COMPONENTS

#### 1.3.1 Other Component Innovations

No innovations in other components are claimed.

#### 1.3.2 Reinforced Soil Properties

Backfill material used in the reinforced zone shall meet the gradation, unit weight, and shear strength criteria specified in AASHTO LRFD Bridge Construction Specification Section 7.3.6.3.

The AASHTO gradations are listed in Table 5. It is understood that these are not the only backfill material that may be used with the EarthTec™ system. The use of the proposed backfill shall be evaluated on a project-by-project basis.

Table 5 Standard Backfill Gradation

| U.S. Sieve Size | Percent Passing |
|---|---|
| 4” | 100 |
| No. 40 | 0-60 |
| No. 200 | 0-15 |

The Plasticity Index (PI), as determined by AASHTO T 90 shall not exceed 6. Also, the backfill shall meet the following electrochemical requirements:

- pH of 5 to 10
- resistivity not less than 3000 Ω·cm

- chlorides not greater than 100 ppm
- sulfates not greater than 200 ppm
- Organics not greater than 1%

The minimum effective friction angle for the reinforced soil volume shall be 30 degrees.

### 1.3.3 ERS Drainage

The joint configuration of the EarthTec™ SCP allows water that infiltrates the MSE backfill to drain out of the MSE structure. Geotextile covering the panel joints is used to prevent the migration of fine material from the soil directly behind the SCP. The geotextile allows water to pass through and, at the same time, prevents fine material from passing through. This combination allows for the drainage of water from the MSE backfill. Reference Appendix TAB 1.3.3.

The drainage requirements for the MSE structure is a function of the application. Proper drainage is critical to the performance of the MSE structure. The drainage design is required to consider both external and internal water sources. External drainage diverts water that flows externally over and around the structure. The external drainage requirements will depend on the location of the MSE structure. External drainage is typically designed to divert all water flow away from the MSE structure. The internal drainage of the MSE structure accounts for infiltration by surface or subgrade water. The internal drainage of the MSE structure is a function of the reinforced soil mass's backfill characteristics. The surface water or subsurface water must be collected and diverted away from the MSE structure. The backfill must consist of material that will allow unimpeded flow while preventing the backfill material's degradation and movement.

When required, drainage is considered at the retained backfill and reinforced backfill interface, the foundation's interface and reinforce backfill, and at the interface of the facing unit and the reinforced backfill. Drainage from these zones can use back-drains (chimney-drain), blanket-drains, and face drains, respectively. All three of the zones or any combination of the zones may require drainage systems. The long-term function of the drain must be considered in the drainage design. Reference Appendix TAB 1.3.3 for examples of these drainage systems.

EarthTec™ requires the Owner to specify the drainage requirements for each project. When necessary EarthTec™ makes recommendations for drainage systems. If drainage is required, the details shown in Appendix TAB 1.3.3 may be implemented. The implementation of the drainage concepts shown in Appendix TAB 1.3.3 requires analysis by the drainage engineer and coordination with the EarthTec engineer.

### 1.3.4 ERS Coping

A coping unit typically covers the top panel. Coping units are shown in Photograph 1-1. The standard U-shaped coping unit is shown in Figure 1-12. EarthTec™ can supply coping units that are used for applications on standard structures that can accommodate traffic barriers, pedestrian barriers, drainage swells, approach slabs, and standard grade changes. Coping unit details are shown in Appendix TAB 1.3.4.

**Photograph 1-1 Coping Unit at Top Facing Unit**

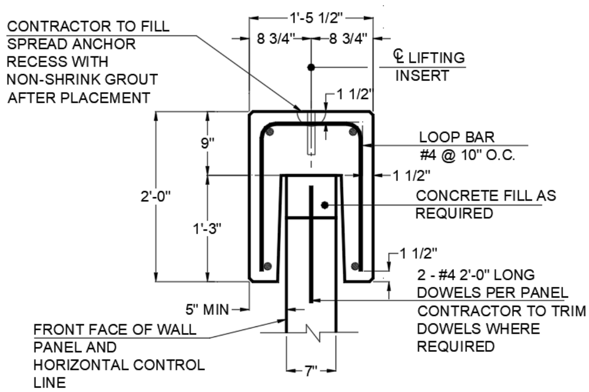



**Figure 1-12 Standard U-Shaped Coping Unit**

### 1.3.5 ERS Traffic Barrier

EarthTec™ coping units may be supplied to allow for the attachment of a traffic barrier to the top surface. The coping unit and traffic barrier can be designed to accommodate all TL load cases. The coping unit's details with traffic barriers are shown in Appendix TAB 0 and in Figure 1-13. Post and beam barriers may be used with the EarthTec™ system. Post and beam systems are placed in the reinforced soil volume behind the face of the MSE structure. Care should be exercised when locating the post with the soil reinforcing.

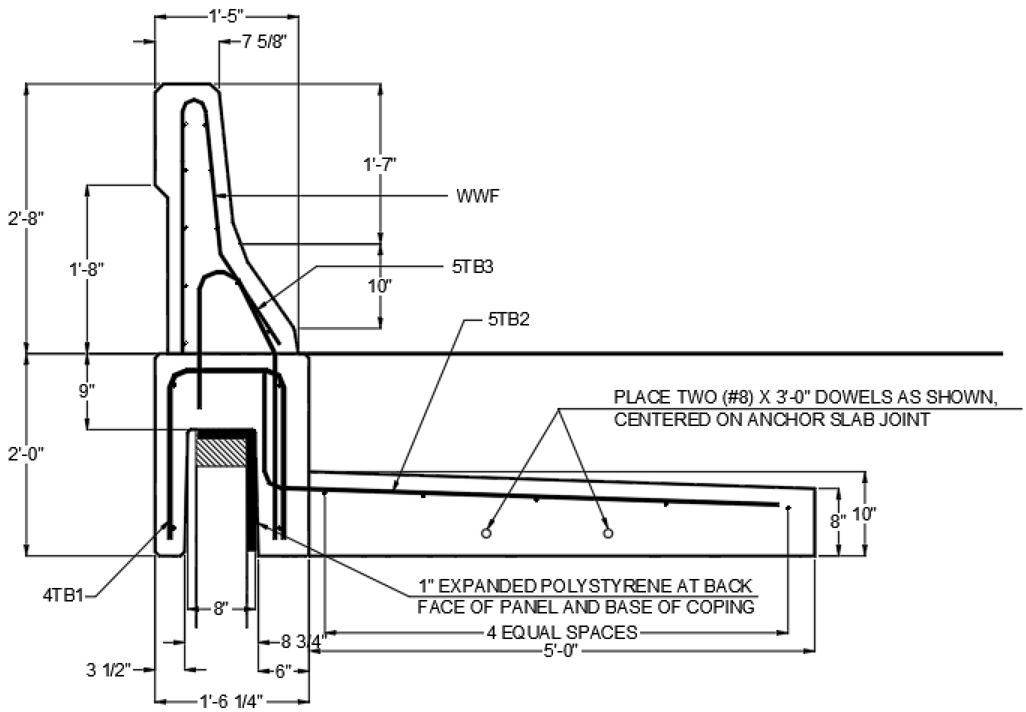



**Figure 1-13 Coping Unit with Traffic Barrier (for reference only)**

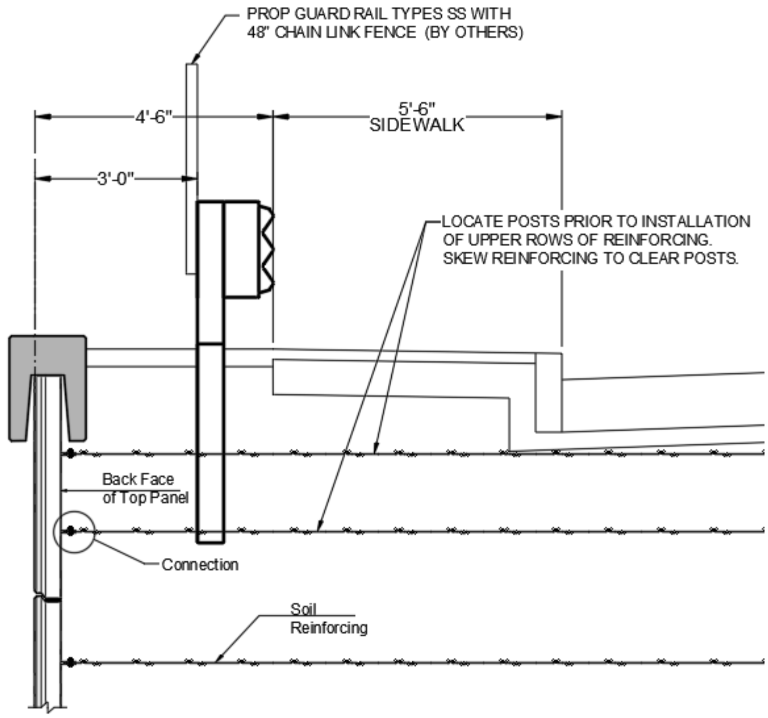



**Figure 1-14 Post and Barrier (for reference only)**

### 1.3.6 ERS Slip-Joints

Slip-joints are used at locations where differential settlement is anticipated. The slip-joint unit is T-shaped. The stem of the T-shape is the location where the soil reinforcing is attached. The soil reinforcing provides stability to the unit. Geotextile (filter fabric) is placed at each side of the slip joint to prevent fine material from migrating out of the reinforced backfill. Details for slip joints are shown in Appendix TAB 1.3.6 and in Figure 1-15.

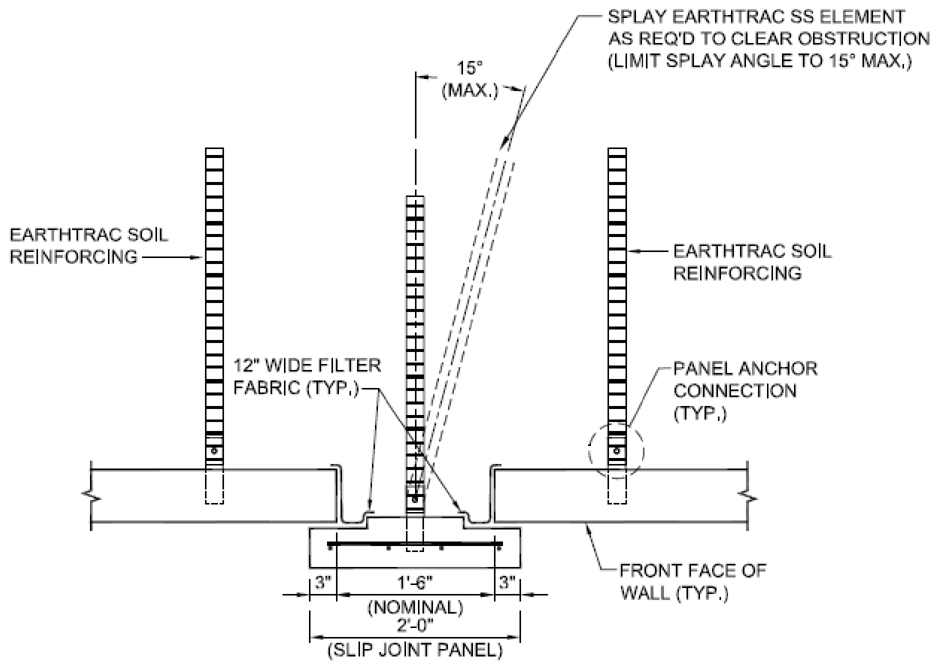



**Figure 1-15 Slip-Joint Detail**

## 2.0 ERS DESIGN

The EarthTec™ Reinforced Soil Wall™ can be designed using different design methods. The system is designed per the project specifications. The methods that are in this submittal are examples only.

## 2.1 DESIGN METHODOLOGY

### 2.1.1 ERS Design Innovations

The EarthTec™ system is designed using a proprietary MSE design software program called RSW-Calc™. This program is a design and analysis program that interfaces with AutoCAD to draw and detail the structure. Through the RSW-Calc™ interface, each column of panels in the AutoCAD drawing has a unique calculation performed. The software determines the required density and length of soil reinforcing. The necessary length and density are determined based on the defined material parameters and the prescribed calculation method. For columns with a slope at the top of the wall, the average wall height, represented by the average of the right and left the column's height, is used in the calculations. The program RSW-Calc can be verified using the software program MSEW.

### 2.1.2 ERS AASHTO LRFD Design Methodology

The EarthTec™ RSW design methodology shall conform to the required edition and year of the AASHTO LRFD Bridge Specification. Complete descriptions of the design methods conforms with the 2020

8th Edition and interim are provided in Appendix TAB 2.2.1 and TAB 2.2.2. The design that is submitted in this report follows the Coherent Gravity design method. Other methods, such as the simplified method, can be used with the EarthTec™ systems.

### 2.1.3 Vertical and Horizontal Obstruction Requirements

In conditions where vertical obstructions interfere with the standard connection, the adjacent facing units may be required to be connected using a panel-to-panel connector. The size of the panel-to-panel connector will be dependent on the force that is needed to be resisted. Typically the panel-to-panel connection consists of a 3” x 3” x 3/8” steel angle that is hot-dip galvanized in conformance to ASTM A123. The steel angle can be field attached to the panels using galvanized adhesive anchors. One adhesive anchor is required at each panel. The adhesive anchor shall be a minimum 5/8” diameter. EarthTec designs the panel-to-panel detail on a project-by-project basis. The force required to be resisted is a function of the vertical obstruction's size and the tributary area of the facing unit.

A horizontal obstruction that is five feet behind the facing unit is by-passed by gradually deflecting the soil reinforcing up and over, or down and under, the obstruction. The soil reinforcing should not be deflected within 12” of the connection to the facing unit. In other words, the soil reinforcing shall be horizontal for a minimum distance of 12” from the back of the facing unit before the soil reinforcing is deflected. Because of the possibility of differential settlement of the backfill located at and near the obstruction, it is better to by-pass the soil reinforcing by deflecting the soil reinforcing up, at a maximum angle of 15 degrees, and over the obstruction, as shown in Figure 2-1. However, it may be necessary to deflect the soil reinforcing down and under the horizontal obstruction in some instances. In this case, the soil reinforcing is placed before the placement of the obstruction.

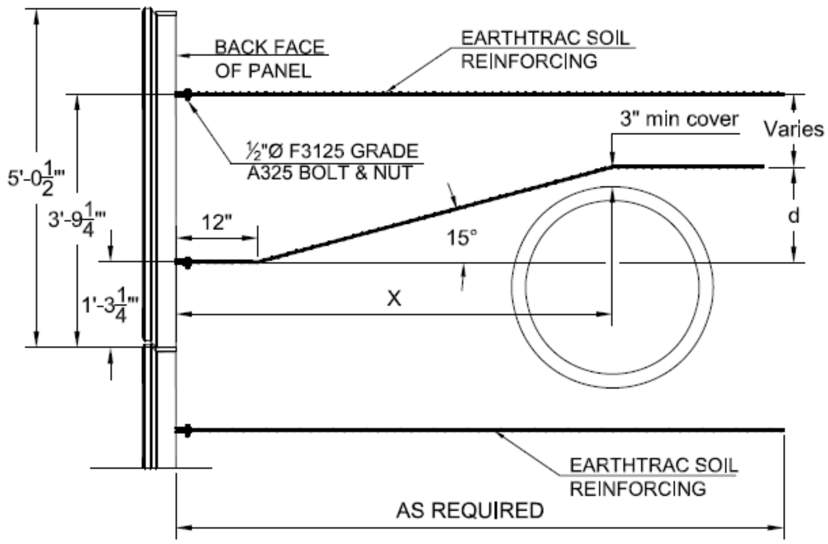



**Figure 2-1 Horizontal Obstruction Detail Deflected Soil Reinforcing**

If the horizontal obstruction cross-section is greater than 36", it may be necessary to use a flowable fill material in the area that is in front of the obstruction. When this is implemented, a "back-up" panel behind the obstruction is used as shown in Figure 2-2. The "back-up" panel is a standard facing unit with anchors cast into it so soil reinforcing can be attached to the back-up panel.

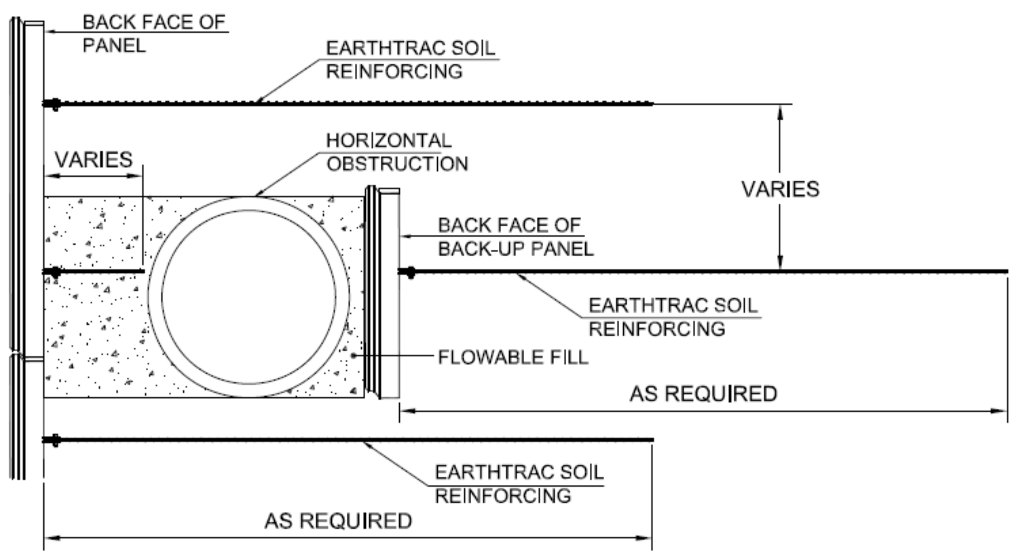



**Figure 2-2 Horizontal Obstruction Detail With Back-Up Panel**

Vertical obstructions can be by-passed by skewing the soil, reinforcing a maximum of 15 degrees from perpendicular to the panel's back face as shown in Figure 2-3. The soil reinforcing skew and panel anchor location are detailed in the EarthTec shop drawings.

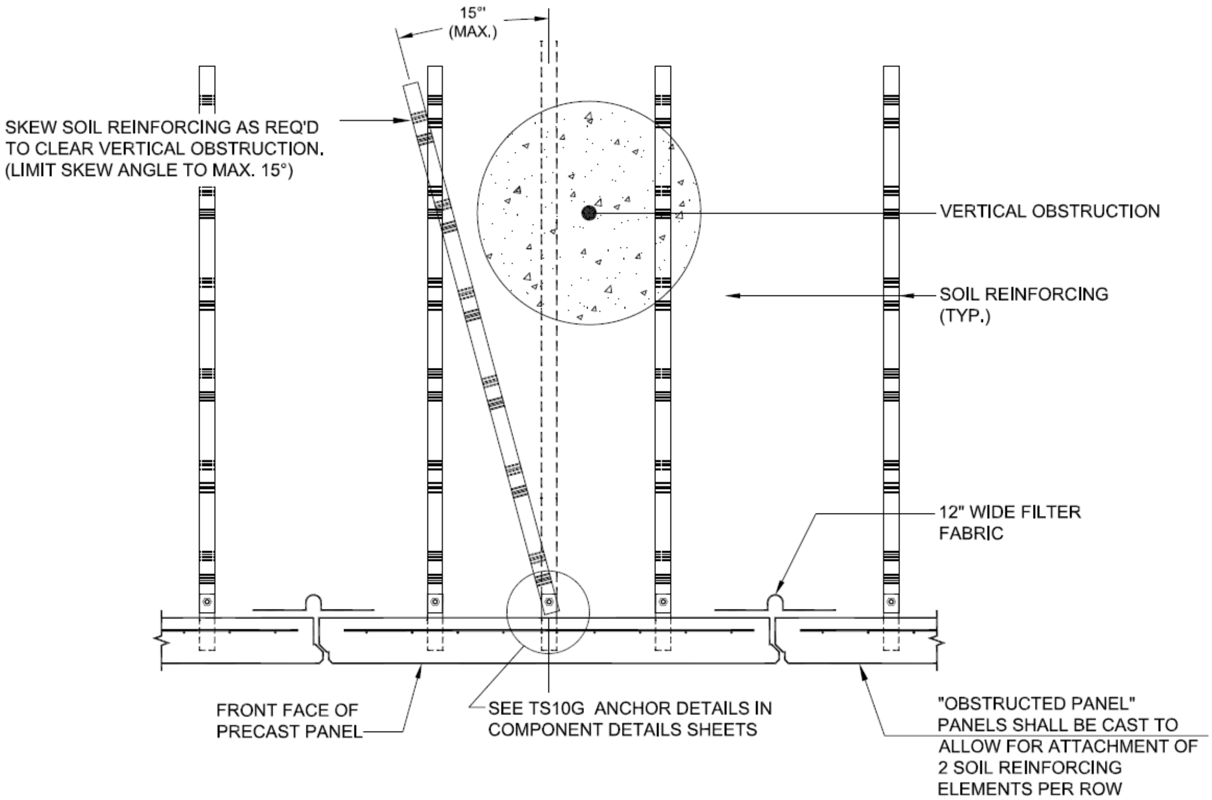



**Figure 2-3 Vertical Obstruction Detail**

Typical obstruction details are shown in Appendix TAB 2.1.3.

## 2.2 DESIGN EXAMPLES

Each design example in the Appendix includes a detailed description of the AASHTO Coherent Gravity design procedure. At the end of each description, the calculations are validated using the GIS proprietary design software RSW-Calc™ and the commercially available software program MSEW.

### 2.2.1 Level Backslope with Traffic Live Load Design Example

Reference Appendix TAB 2.2.1 for a sample Level Backslope with Traffic Live Load Calculation.

### 2.2.2 2:1 Backslope Design Example

Reference Appendix TAB 2.2.2 for a sample 2:1 Infinite Back-Slope Calculation.

# 3.0 CONSTRUCTION

## 3.1 CONSTRUCTION PROCEDURES

### 3.1.1 ERS Construction Innovations

EarthTec™ does not claim any construction innovation.

### 3.1.2 ERS Construction Manual

The EarthTec™ Reinforced Soil Wall Quality Control Installation Guidelines is located in Appendix TAB 3.1.2. The installation manual includes recommended means and methods to construct the EarthTec RSW, including step-by-step sequencing, quality control construction process, troubleshooting information, and an MSE wall construction checklist.

### 3.1.3 ERS Facing Installation Requirements

EarthTec™ segmental concrete panels (SCP) are available in three nominal sizes: 5’ x 5’ and 5’ x 10’. Each panel is labeled and detailed on the panel schedule or the standard panel detail sheet. The panel label provides the panel type and the number of panel anchors per row. Note that you can quickly identify each panel by the information scribed on the back of the panel by the Precaster.

To create a staggered joint arrangement, the initial course of panels alternate between a standard height panel, Type-G (5’-0”), and a half-height panel, Type-B (2’-6”). The first row of panels is placed on the leveling course. If the leveling course is not positioned correctly, the panel will need to be adjusted to ensure that the horizontal and vertical alignment is achieved and maintained during the installation process. At no time should a bearing pad be placed on the leveling course for the panel to bear on. The panel should be set directly on the leveling course. It is essential to maintain the required ¾” vertical joint spacing in the first row of panels. Proper spacing and alignment in the first row will make subsequent rows line up and make installation easier.

Some differences in vertical tolerance can be corrected by using hard-wood or plastic shims. However, shims should be minimized and used only for minor corrections in joint/panel alignment. The use of wedges along the panels' front face may be used to facilitate proper panel batter. However, their use should be monitored to prevent front face panel spalling. The wedges should be removed after the panel row above has been placed and backfilled.

To provide proper placement of the first row of panels and to set the correct panel batter, it is recommended, but not required, to position two wood wedges on the bottom panel's front face and back face. When the wedges are lodged under the panel, they will aid in the stabilization of the panel. The wedges will provide for easy adjustment of the batter and aid in leveling the panel to the proper elevation before the backfill placement. It is also recommended that the first row (or rows) of panel use an adjustable bracing system to adjust the panel alignment. All wood wedges at the back face of the wall shall be removed before placement of backfill.

The backfill is placed and compacted slightly above (1” to 2”) the elevation of the panel anchor. Once the backfill is compacted and is above the panel anchor, the soil reinforcing is installed. The next row of panels is placed when the backfill is brought to the alternating panel's top surface. The procedure of placing panels, backfill, compacting, and soil reinforcing placement continues until the top of the structure is reached.

Structures that are placed on curves are approximated by placing the panels in a series of chords. The width of the facing unit along the curve is a function of the size of the radius. When panels are required to be placed on a curve the shop drawings prepared by EarthTec™ will detail the curve layout, including the required panel joint configuration. Depending on the curve's radius, the joints may consist of the standard shiplap configuration or a variation thereof. The panel vertical joint will typically increase in width (open up) or decrease in width (close up) from the standard ¾” gap. In the case of small radiused curves, the panel joint may be required to be beveled or cropped. If this is needed, the panels will be detailed in the shop drawings. In some cases, the leveling course width may need to be increased to ensure that the panel base is placed on a complete concrete surface. At no time should the panel span over the backfill or foundation. Inside curves and outside curves are set using the same chord procedure. The difference will be in the joint detail when the curve radius is small.

Facing units at corners are cast using an adjustable corner panel shown in Figure 3-1 and Figure 3-2. The corner panel form is designed to accommodate angles from 15 degrees to 180 degrees. Corners that have an angle less than 70 degrees may require special design considerations using bin theory.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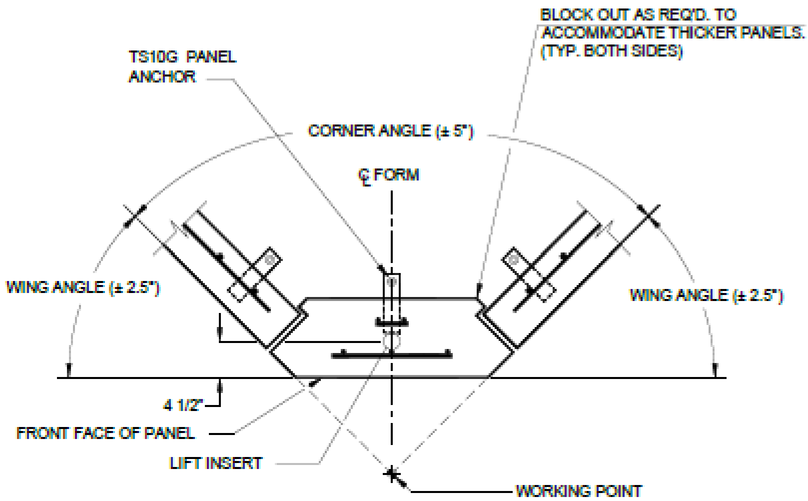


**Figure 3-1 EarthTrac™ Outside Corner Detail**

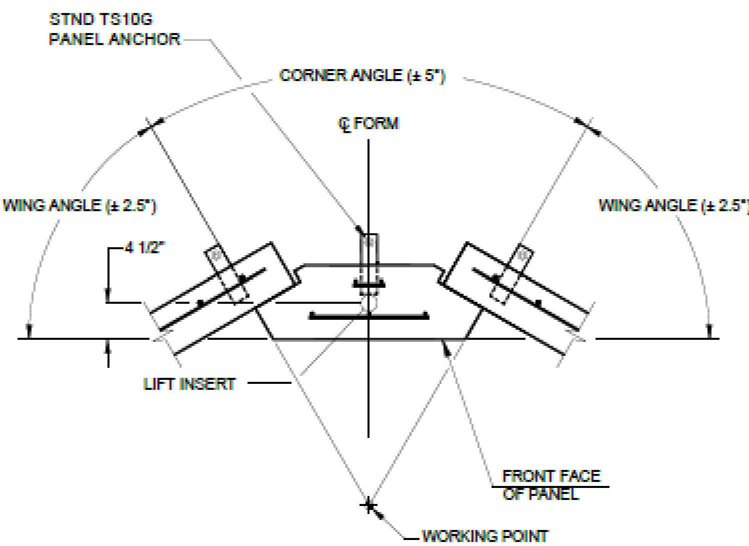


**Figure 3-2 EarthTrac™ Inside Corner Detail**

### 3.1.4 ERS Soil Reinforcing Installation Requirements

The EarthTrac™ soil reinforcing is placed on the compacted select backfill and the connection is made to the panel anchor. The EarthTrac™ soil reinforcing is connected to the panel by securing the proximal end of the soil reinforcing strip between the slot formed by the two plates of the TSG10 panel anchor using a bolt and nut. The nut is attached to the bolt as a snug-tight connection. The nut should be tightened so all faying surfaces make contact, and the nut can't be removed without a wrench. *Never connect an EarthTrac™ soil reinforcing with the backfill lower than the elevation of the panel anchor*. The backfill should be slightly higher than the panel anchor. To place the bolt and nut, it may be necessary to remove some backfill from under the panel anchor as shown in Figure 3-3. The length of the EarthTrac™ soil reinforcing and the number of required EarthTrac™ soil reinforcing to be placed on each panel is detailed in the approved EarthTec™ shop drawings.

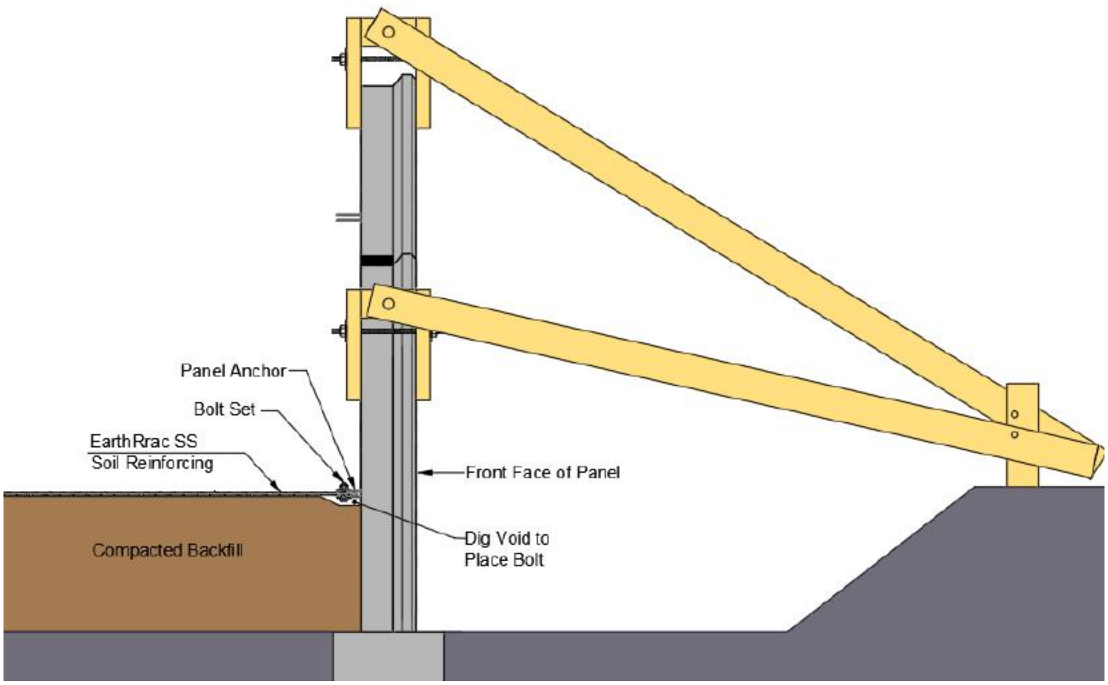


**Figure 3-3 Placement of EarthTrac™ on Frist Row of Panels**

The soil reinforcing should be placed at corners and curves as near to perpendicular to the panel's back face as possible. In most cases, the soil reinforcing will be required to be skewed and will overlap soil reinforcing from the opposing side of the corner and, in the case of outside curves, the adjacent panel. Because the soil reinforcing does not have an extensive coverage ratio, i.e., it is a 2-inch wide discrete element, the interface area between two overlapping elements is small. It is unnecessary to separate the element with soil. If the project requires overlapping soil reinforcing to not come in contact, it is recommended that 3-inches of soil be placed between the overlapping interface. A corner detail is shown in Figure 3-4

Manually operated compaction equipment is required to be used in areas where there is limited space to use large compaction equipment. The backfill shall be compacted to 95% of the required proctor. The lift thickness of the backfill shall be reduced to achieve compaction. Compaction tests and gradation tests should be taken and recorded in accordance with the contract specifications.

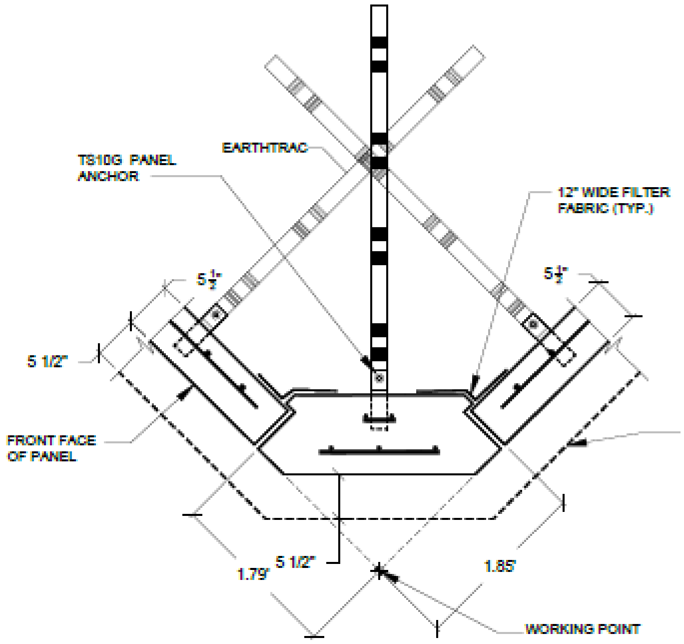


**Figure 3-4 EarthTrac™ Overlapping Soil Reinforcing at Corner Element**

### 3.1.5 ERS Facing Alignment Requirements

The vertical and horizontal alignment of the wall face shall maintain a ½ inch tolerance at any point when measured along a 10-foot straight edge placed against the wall. No facing panel shall be placed with more than ½ inch out of vertical or horizontal alignment from the adjacent panels. The wall's final overall vertical tolerance (also sometimes referenced as the plumbness from top to bottom) shall be less than ½ inch per 10 feet of wall height. During construction, it is recommended that a plumb-bob be used to check the overall vertical tolerances for every panel at the third row of panels. The Installer should continuously monitor the facing batter, facing alignment, and facing tolerances and make necessary installation adjustments as required. The panel alignment is highly dependent on the means and methods used to compact the soil, the soil's moisture content, the elevation of the backfill in relationship to the panel anchor, and the methods used to set and clamp the panels the experience of the crew. The ultimate acceptance of the wall alignment shall be the responsibility of the Owner. When alignment adjustments are required, they shall be made with the use of Hardwood Shims. The Hardwood shims should be removed after the adjustment has been made.

### 3.1.6 ERS Reinforced Backfill Placement Requirements

The backfill placement should begin parallel to the wall face at a distance greater than or equal to 3-feet from the back face of the panel. The backfill should be placed in 6-inch to 12-inch compacted lifts. The backfill can be placed in larger lifts if approved by the Owner or Owner's representative and if the Wall Installer can demonstrate that the required backfill density can be achieved. *The backfill shall be leveled by equipment moving parallel to the wall face. The backfill shall be spread so it is fanned toward the terminal end of the soil reinforcing.* The backfill placement from the panel's back face to the terminal end of the soil reinforcement will keep the soil reinforcement tensioned.

Compaction of the backfill at a distance greater than 3-feet from the panel's back face shall be performed with a 8-ton to 10-ton roller. A smooth wheel or rubber tire roller is acceptable. No compaction equipment that employs grid type rollers shall be used. Grid type rollers can dislodge the soil reinforcing from its proper orientation. *Compaction must be parallel to the wall face working toward the terminal end of the soil reinforcement.* The correct moisture content of the backfill material should be maintained uniformly within each lift. Care should be used in adding water to the backfill material. After all rain events, it is advised that compaction of the backfill or placement of additional backfill not commence until the in place backfill material and the new backfill has dried out to a moisture content at the dry side of optimum.

The 3-foot zone of fill located at the back of the panel is placed with an end loader and spread manually. The material is then compacted using a manually operated vibratory roller or plate compactor with a weight equal to a maximum of 2000-pounds and 1000-pound respectively. Care should be exercised when compacting the 3-foot zone not to disturb the panel's alignment. Fine-grain soils, such as poorly graded sand, should be compacted with care. Compaction in the three-foot zone is shown in Photograph 3-1.


**Photograph 3-1 Compaction of 3-Foot Zone**

Compaction tests and gradation tests should be taken and recorded following the contract specifications. The compaction test reports shall be made part of the Wall Installers log. *Proper compaction will alleviate possible problems with the future performance of the structure.* Improper compaction can cause outward movement of the panels.

### 3.1.7 ERS Erosion Prevention Requirements

Erosion of the soil from the reinforced soil volume, or at the face in front of the structure, shall be prevented during construction. This can be accomplished by sloping the backfill away from the wall face at the top of the structure and at the face, thereby preventing high-velocity water flow. The placement of the fill material for the finish grade in front of the wall shall occur as soon as possible, and before the wall height exceeds 20 feet. Ideally, the finish grade should be placed as soon as possible to prevent undercutting of the leveling course and possible foundation saturation.

### 3.1.8 ERS Installer Requirements

The following are minimum requirements that should be used to qualify the Installer of the EarthTec™ RSW system. These requirements are minimum and are only suggestions. Actual requirements shall follow the governing specification and are to be determined at the Owner or Contractor's discretion.

**Project Job-Site MSE Wall Installer**

The Job-Site MSE Wall Foreman shall have experience in the construction of at least five transportation-related MSE wall projects within the last three years. Transportation-related MSE wall projects are defined as walls that carry or are adjacent to vehicular traffic and are constructed with MSE reinforcement in the reinforced backfill zone. The Foreman must have prior experience or adequate training on the installation of the EarthTec™ RSW system. The Foreman shall be at the project site when the work is being performed on the structure.

**Alternate Prequalification Criterion (in the absence of above experience)**

If the Project Foreman does not have prior experience in the installation of MSE retaining walls than a wall test segment shall be constructed. The following qualification criteria shall be used for the implementation of the test wall procedure.

The Project Foreman shall direct the construction of the wall test segment. The wall test segment shall be constructed in the presence of an EarthTec™ Technical Representative and the Project Engineer. The minimum length of the wall test segment shall be 40 feet or the full length of the wall if less than 40 feet. The Contractor shall arrange for a Technical Representative of EarthTec™ to be present during each wall test segment's construction. The EarthTec™ Technical Representative shall be present for the construction of the wall test segment. The wall test segment shall include the construction of each of the five elements listed below.

1. Placement of a minimum of the first four layers of primary soil reinforcement and backfill
2. If obstructions (i.e., steel piles, concrete piers/abutments, concrete boxes, pipes, etc.) exist, placement of primary soil reinforcement and backfill at obstructions
3. Placement of a minimum of the first two rows of panels or a minimum of a four-foot wall height
4. If a vertical slip joint is required, construction of the vertical slip joint in a minimum of a two-row portion of panels or a minimum of a four-foot wall height
5. If corners are required, the construction of a corner representative of the corners in the project. Construction shall be at a minimum of a two-row portion of panels or a minimum of a four-foot wall height.

Before the construction of the wall test segment, the EarthTec™ Technical Representative will provide the Contractor, Project Foreman, and the Project Engineer the following:

- Technical instructions in the construction methods for the EarthTec™ EarthTrac™ RSW system.
- Product-specific specifications in the placement of the soil reinforcement and backfill in accordance with the project requirements
- Guidelines in placing the facing units and attaching them to the soil reinforcement in accordance with the system requirements

At the completion of the wall test segment, the EarthTec™ Technical Representative will provide the following documentation for final approval by the Project Engineer:

- Documentation that the wall test segment was constructed following the product-specific specifications. This documentation shall include a location description (starting and ending stations and elevations) of the wall test segment
- Documentation that the job site wall Foreman is familiar with the wall products used to construct the walls on the project

#### 3.1.9 ERS Retained Backfill Placement Requirements

The retained backfill shall be placed concurrently with the placement of the reinforced backfill. When required, the retained backfill interface with the reinforced backfill shall have a chimney drain constructed. The chimney drain shall be constructed concurrently with the placement of the retained backfill and reinforced backfill. The chimney drain shall be constructed in accordance with the project specification, including all geotextile drainage material. The chimney drain shall have adequate exit drains to remove the water from the chimney drain and away from the structure. The lift thickness, compaction, and density, of the retained backfill, shall be in accordance with the project specifications.

## 4.0 QUALITY CONTROL

### 4.1 MANUFACTURING

#### 4.1.1 Facing Unit QA/QC

Reference Appendix TAB 4.1.1 for the Facing Unit QA/QC requirements.

### 4.1.2 Soil Reinforcing QA/QC

Reference Appendix TAB 4.1.2 for Soil Reinforcing QA/QC requirements.

### 4.1.3 Miscellaneous Component QA/QC

Reference Appendix TAB 4.1.3 for Miscellaneous Component QA/QC requirements.

## 4.2 CONSTRUCTION

### 4.2.1 Construction QA/QC

The Process Control, Trouble Shooting, and MSE Checklist sections in the Quality Control Guidelines (attached under TAB 3.1.2) address QA/QC of construction.

# 5.0 PERFORMANCE

## 5.1 PERFORMANCE HISTORY

### 5.1.1 ERS Performance History

The EarthTec™ system has been used in the United States since 2007. Since its inception, over 44 projects have been completed. Several projects have multiple structures. EarthTec™ is not aware of major system performance problems. Challenges that have occurred are considered normal to MSE construction. These include joint alignment, panel alignment, and backfill compaction. These problems can typically be attributed to means and methods of construction and are not indicative of system performance problems. A list of projects is included in Appendix TAB 5.1.4. The EarthTec RSW system can be used with abutments supported by deep foundations. The EarthTrac™ soil reinforcing articulates and can be adjusted to by-pass obstructions such as piles. The EarthTec RSW system can be used with abutments where the MSE supports the bridge substructure on a spread-footing. The external vertical load and the horizontal load can be resisted by adding more EarthTrac™ soil reinforcing to the columns of panels. The EarthTec RSW system can be used with complex structures such as tiered structures by performing compound an global stability analysis.

### 5.1.2 ERS Oldest Structures

The first EarthTec™ RSW project was constructed on Route 193 in Greenbelt, Maryland (07/03/2007) and was 1587 square feet. The second oldest project was ICC in Montgomery, Maryland (9/11/2007) and was 89,845 square feet. The third oldest project was Fox Run Boulevard in Calvert City, Maryland (2/5/2008) and was 2, 192 square feet.

### 5.1.3 ERS Tallest Structures

The tallest EarthTec™ RSW project was constructed on the Jordon Bridge in Chesapeake, Virginia and was a multitier structure that was 55 feet tall with the tallest tier being 40 feet and had 16,682 square feet of MSE. The 11th Street Bridge in Washington D.C. and the I95 and I24 project in Baltimore, Maryland, had several bridge abutments that were approximately 40 feet in height.

### 5.1.4 ERS Private and Public Users

Table 6 lists the State Department of Transportation's where the EarthTrac™ system has been reviewed and approved. A spreadsheet with a list of 40 projects where the system has been used is in the Appendix.

Table 6 EarthTec System State Approvals

| Agency | Date | Contact | Email |
|---|---|---|---|
| Florida DOT | 11/2020 | Rodrigo Herrera, P.E. | Rodrigo.Herrera@dot.state.fl.us |
| Virginia DOT | 12/2020 | Kevin Lee, P.E. | Kevin.Lee@VDOT.Virginia.gov |
| Illinois DOT | 12/2020 | Mark Shaffer, S.E. | Mark.Shaffer@illinois.gov |
| | | | |
| | | | |
| | | | |
| | | | |

## 6.0 OTHER

### 6.1 Other Information

No other information is warranted at this time.

# Appendix

# IDEA

## HIGHWAY INNOVATIONS, DEVELOPMENTS, ENHANCEMENTS AND ADVANCEMENTS

## EARTHTEC™ EARTHTRAC™

## SYSTEM DETAILS



GROUND IMPROVEMENT SYSTEMS LLC
114 South Collins Street
Arlington, TX 76011

# Appendix

# IDEA

## HIGHWAY INNOVATIONS, DEVELOPMENTS, ENHANCEMENTS AND ADVANCEMENTS

## EARTHTEC™ EARTHTRAC™

## SYSTEM DETAILS

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# 1.0
# ERS COMPONENTS

EARTHTEC

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# 1.1
# FACING UNITS

EARTHTEC

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EARTHTEC

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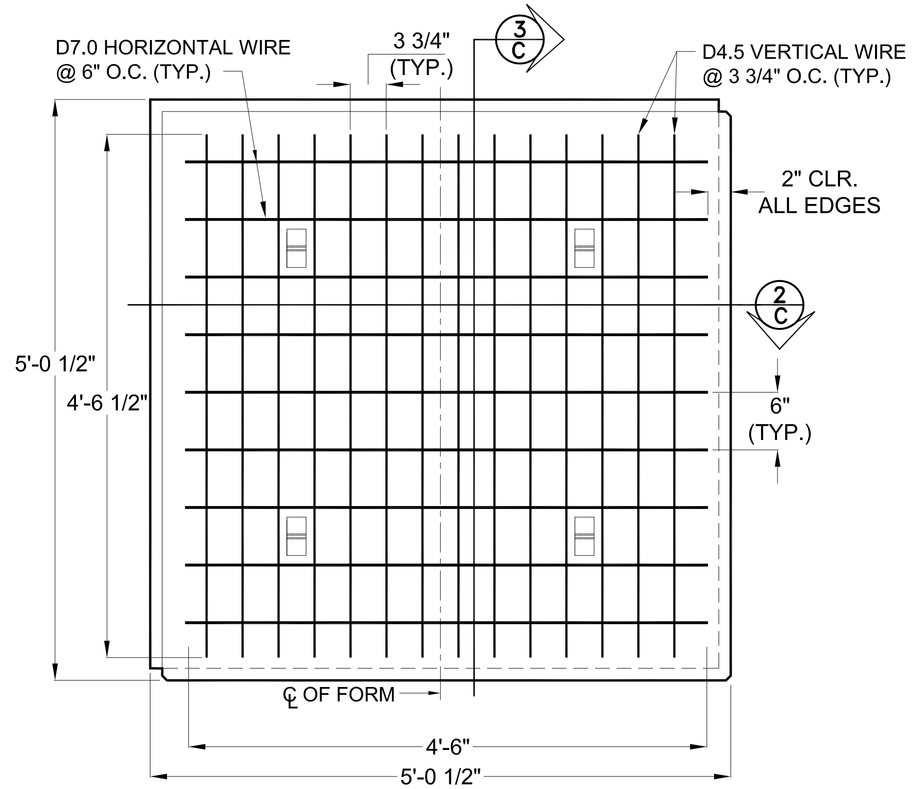


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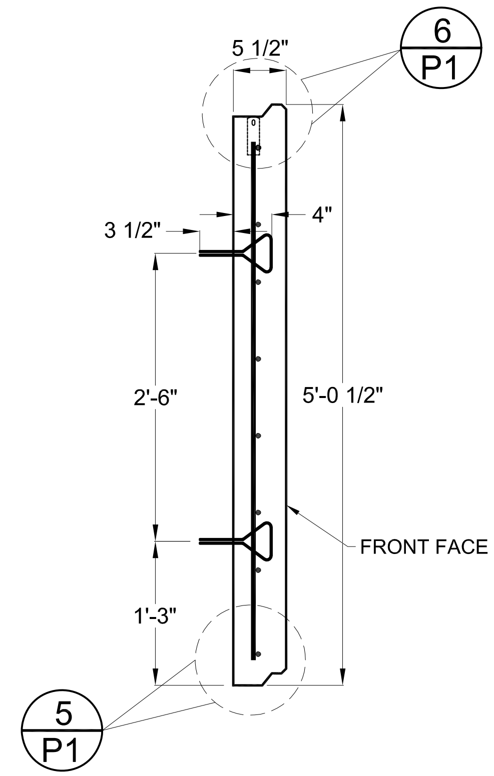
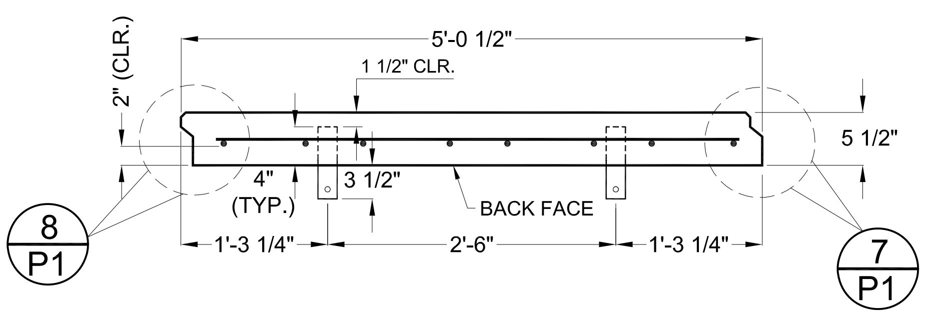
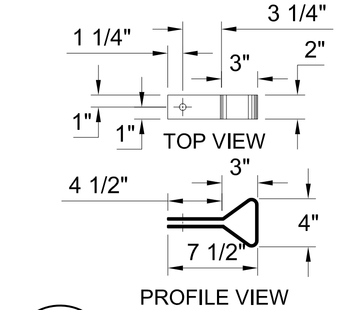
BACK FACE



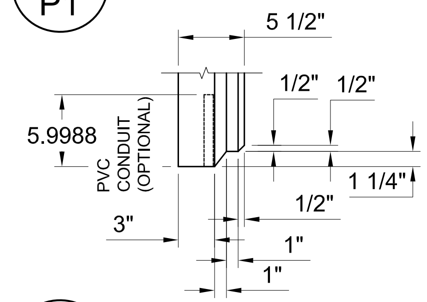

3 / P1 STANDARD SQUARE PANEL
SECTION VIEW



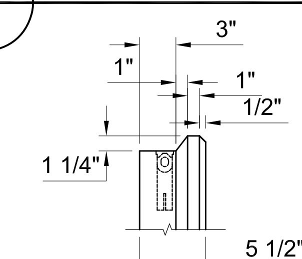

2 / P1 STANDARD SQUARE PANEL
TOP VIEW



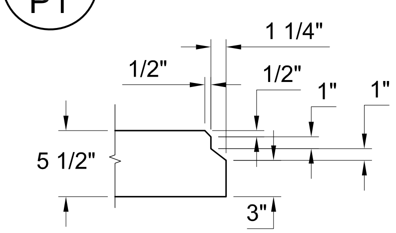

4 / P1 PANEL ANCHOR



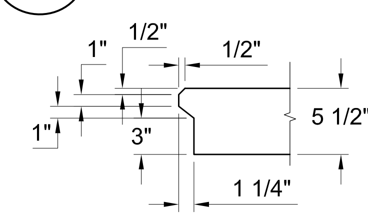

5 / P1 PANEL BOTTOM




6 / P1 PANEL TOP




7 / P1 PANEL RIGHT SIDE




8 / P1 PANEL LEFT SIDE

GENERAL NOTES

1. ALL DIMENSIONS ARE IN FEET AND INCHES (IMPERIAL UNITS) UNLESS NOTED OTHERWISE.
2. ALL PANELS ARE SHOWN BACK FACE.
3. ALL DIMENSIONS ARE SHOWN MEASURED FROM BACK FACE / BACK EDGE OF PRECAST PANEL FORM, TYPICAL.
4. ALL BLACK STEEL REINFORCEMENT AND EMBEDS SHALL HAVE A MINIMUM 1 $\frac{1}{2}$" CONCRETE COVER
5. CONCRETE SHALL HAVE A MINIMUM 28-DAY COMPRESSIVE STRENGTH EQUAL TO 4000 PSI (UNLESS NOTED OTHERWISE) AND SHALL BE CAST IN ACCORDANCE WITH THE PROJECT SPECIFICATIONS.
6. CONCRETE SAMPLING, TESTING, CERTIFICATIONS AND PANEL PRODUCTION (INCLUDING BUT NOT LIMITED TO CASTING, CURING, LABELING, FINISHING, HANDLING, STORAGE AND REPORTING) SHALL BE CONDUCTED IN ACCORDANCE WITH THE GOVERNING PROJECT SPECIFICATION
7. EACH PANEL SHALL HAVE THE PANEL NAME, DATE OF MANUFACTURE AND LOT NUMBER SCRIBE IN THE BACK FACE OF PANEL.
8. THE FRONT FACE EDGE OF ALL PANELS SHALL HAVE A ½" X ½" CHAMFER (UNLESS NOTED OTHERWISE).
9. PANEL REINFORCING SHALL BE WELDED WIRE FABRIC CONFORMING TO ASTM A1064 STANDARD SPECIFICATION FOR STEEL WIRE AND WELDED WIRE REINFORCEMENT, PLAIN AND DEFORMED, FOR CONCRETE WITH A MINIMUM YIELD STRENGTH EQUAL TO 65 KSI.
10. THE GALVANIZED ANCHOR EMBEDS SHALL NOT BE IN CONTACT WITH THE BLACK PANEL REINFORCEMENT.
11. PLACE PANEL ANCHORS AS DETAILED AND SO THEY ARE 90° TO THE BACK FACE OF THE PANEL..
12. PANEL LIFT ANCHOR SHALL BE DAYTON SUPERIOR P-93 FLEET-LIFT L-ANCHOR (PRODUCT CODE FL050, HOT-DIPPED GALVANIZED) OR EQUIVALENT.
13. ALL PANELS ARE TO BE MANUFACTURED TO THE DIMENSION SHOWN IN THE SHOP DRAWINGS UNLESS NOTED OTHERWISE. PANEL SQUARENESS AND TOLERANCES OUTLINED IN THE EARTHTEC PRECAST PANEL QUALITY CONTROL SPECIFICATION AND WITHIN THE GOVERNING SPECIFICATIONS REQUIREMENTS.
14. PANELS SHALL NOT BE REMOVE FROM FORM UNTIL A MINIMUM 1500 PSI STRENGTH BEEN ACHIEVED.
15. PANELS SHALL BE REMOVED FROM THE FORM USING A MINIMUM OF 4 POINTS THAT ARE EVENLY AND SYMMETRICALLY DISTRIBUTED ON THE BACK FACE OF THE PANEL.

THE DESIGN CONTAINED HEREIN IS BASED ON INFORMATION SUPPLIED BY OTHERS.GIS CERTIFYING INTERNAL STABILITY OF THE STRUCTURE ONLY. EXTERNAL AND GLOBAL STABILITY REQUIREMENTS ARE THE RESPONSIBILITY OF OTHERS.

| REV | DESCRIPTION | BY | DATE |
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EARTHTEC
GIS Support Through Innovation

PRECAST PANEL DETAIL
5' WIDE PANEL


GROUND IMPROVEMENT SYSTEMS LLC
114 SOUTH COLLINS STREET
ARLINGTON, TX 76011
(817) 223-0969
www.groundimprovementsystems.com


GIS-2020
DESIGNED BY TPT
DRAWN BY TPT
CHECKED BY XXX

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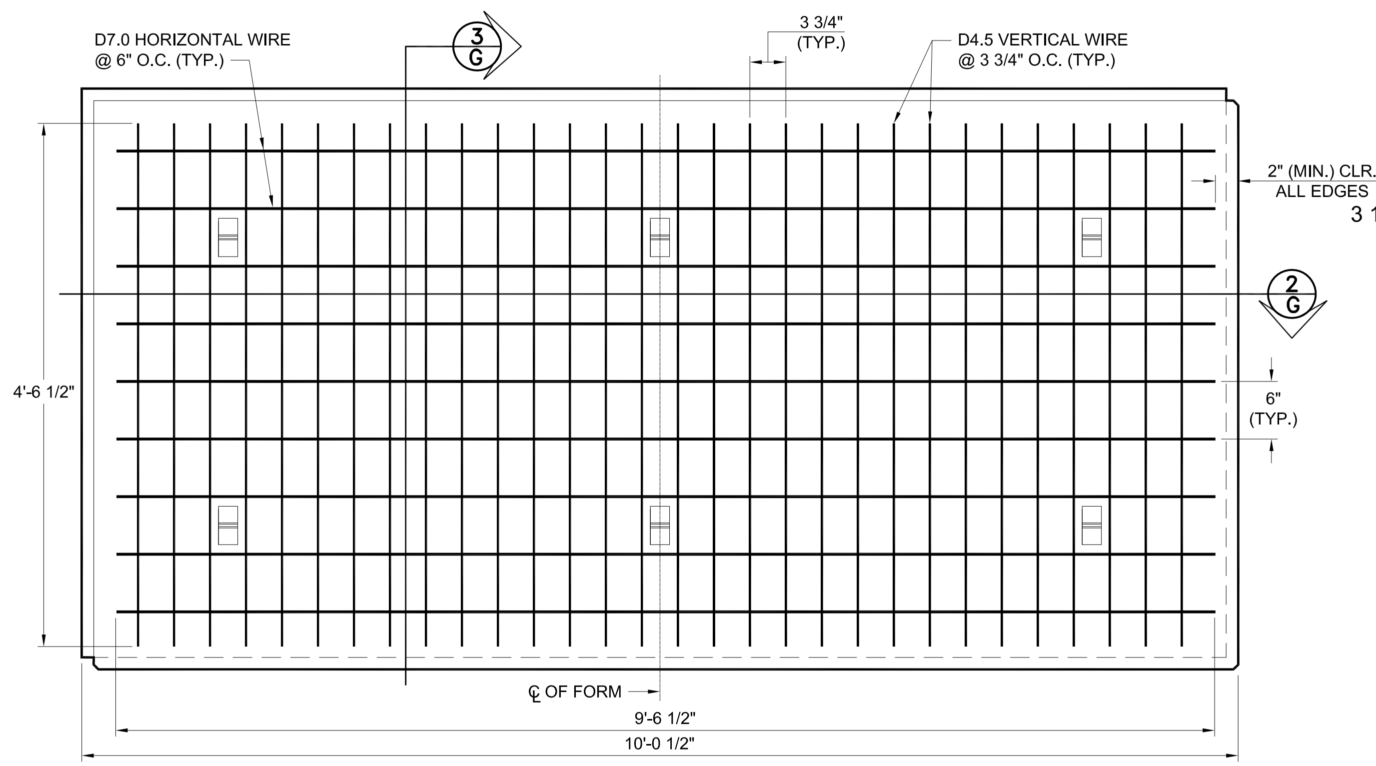


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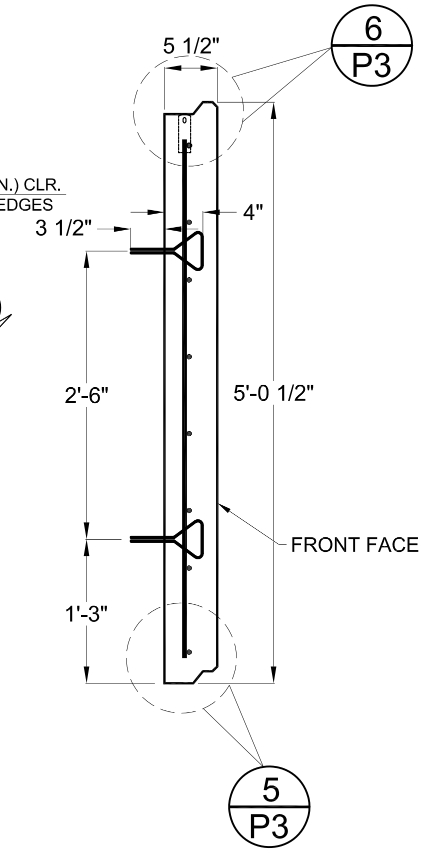
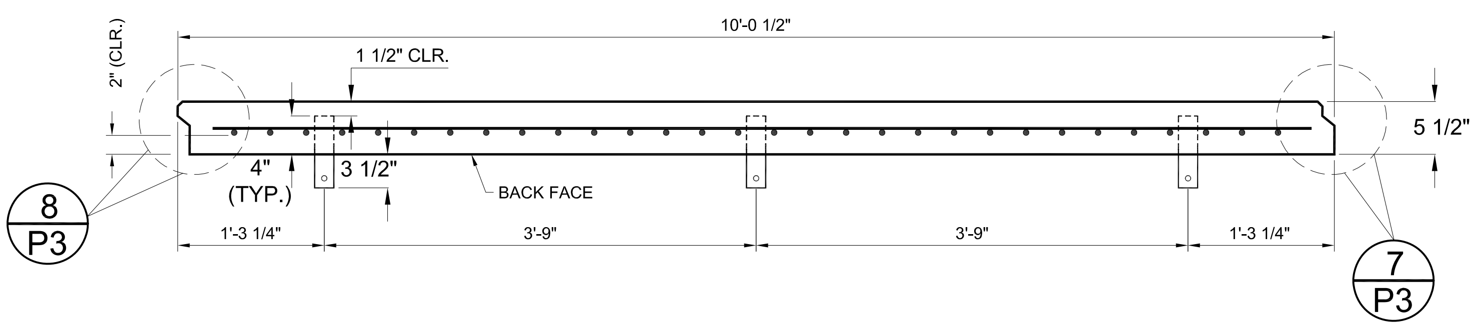
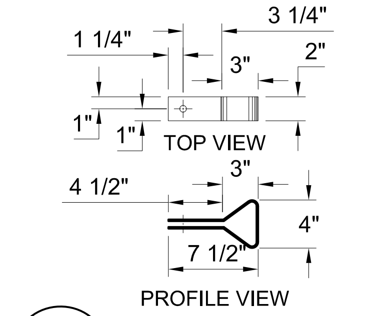


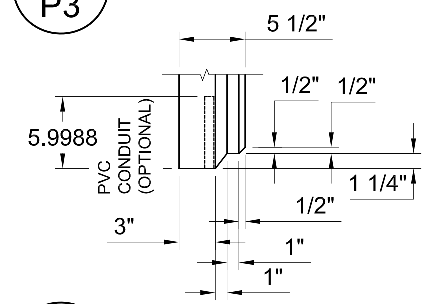

3 / P3 STANDARD RECTANGULAR PANEL
SECTION VIEW



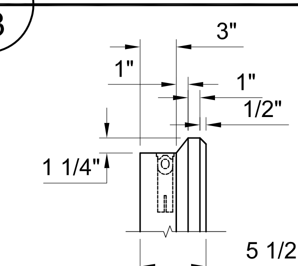

2 / P3 STANDARD RECTANGULAR PANEL
TOP VIEW



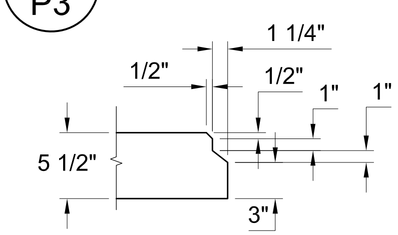

4 / P3 PANEL ANCHOR



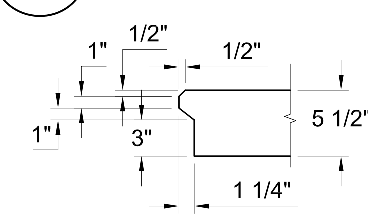

5 / P3 PANEL BOTTOM




6 / P3 PANEL TOP




7 / P3 PANEL RIGHT SIDE




8 / P3 PANEL LEFT SIDE

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10. THE GALVANIZED ANCHOR EMBEDS SHALL NOT BE IN CONTACT WITH THE BLACK PANEL REINFORCEMENT.
11. PLACE PANEL ANCHORS AS DETAILED AND SO THEY ARE 90° TO THE BACK FACE OF THE PANEL..
12. PANEL LIFT ANCHOR SHALL BE DAYTON SUPERIOR P-93 FLEET-LIFT L-ANCHOR (PRODUCT CODE FL050, HOT-DIPPED GALVANIZED) OR EQUIVALENT.
13. ALL PANELS ARE TO BE MANUFACTURED TO THE DIMENSION SHOWN IN THE SHOP DRAWINGS UNLESS NOTED OTHERWISE. PANEL SQUARENESS AND TOLERANCES OUTLINED IN THE EARTHTEC PRECAST PANEL QUALITY CONTROL SPECIFICATION AND WITHIN THE GOVERNING SPECIFICATIONS REQUIREMENTS.
14. PANELS SHALL NOT BE REMOVE FROM FORM UNTIL A MINIMUM 1500 PSI STRENGTH BEEN ACHIEVED.
15. PANELS SHALL BE REMOVED FROM THE FORM USING A MINIMUM OF 4 POINTS THAT ARE EVENLY AND SYMMETRICALLY DISTRIBUTED ON THE BACK FACE OF THE PANEL.

| REV | DESCRIPTION | BY | DATE |
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EARTHTEC
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PRECAST PANEL DETAIL
10' WIDE PANEL

GROUND IMPROVEMENT SYSTEMS LLC
114 SOUTH COLLINS STREET
ARLINGTON, TX 76011
(817) 223-0969
www.groundimprovementsystems.com

GIS-2020
DESIGNED BY TPT
DRAWN BY TPT
CHECKED BY XXX

SHT: P3

## 1.1.1
## Facing Unit Innovations

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114 SOUTH COLLINS STREET
ARLINGTON, TX 76011

## 1.1.2
## Facing Unit Types

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114 SOUTH COLLINS STREET
ARLINGTON, TX 76011

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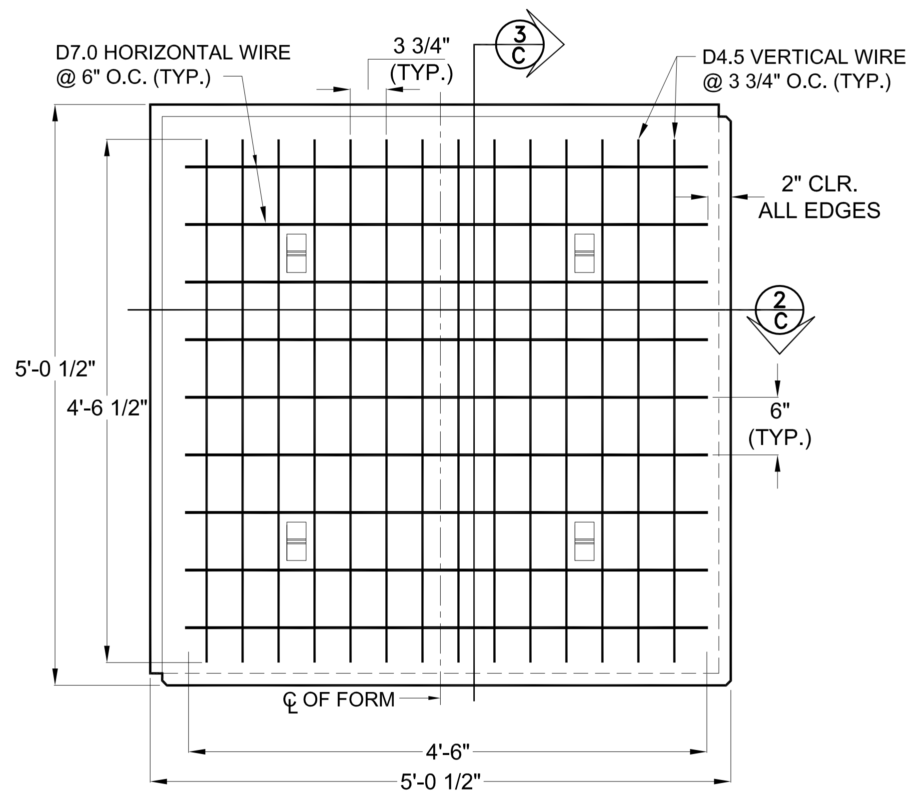
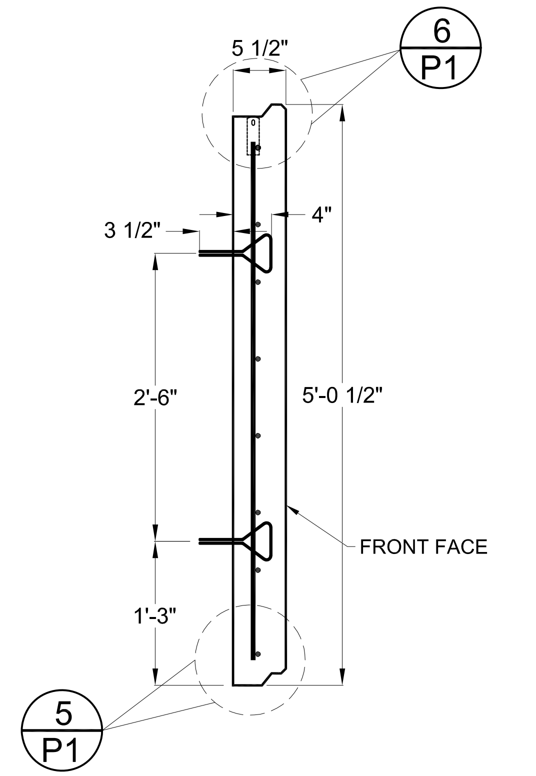
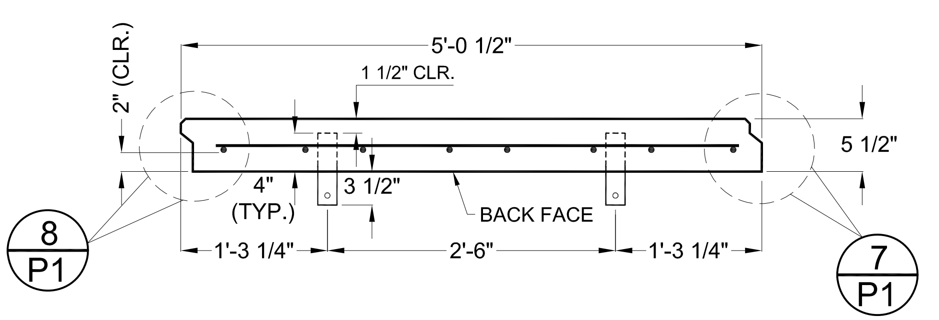


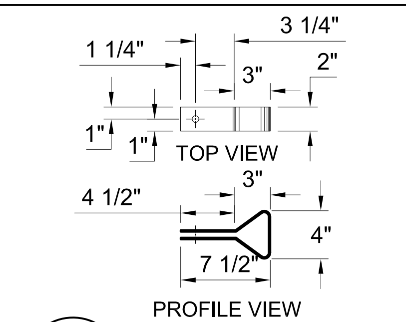

1 P1 STANDARD SQUARE PANEL
BACK FACE



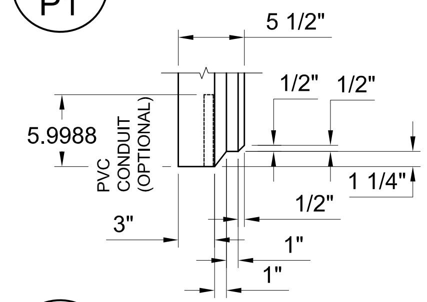

3 P1 STANDARD SQUARE PANEL
SECTION VIEW



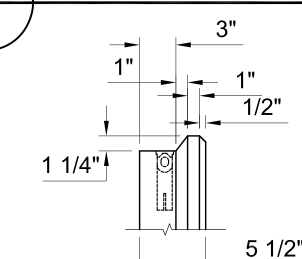

2 P1 STANDARD SQUARE PANEL
TOP VIEW



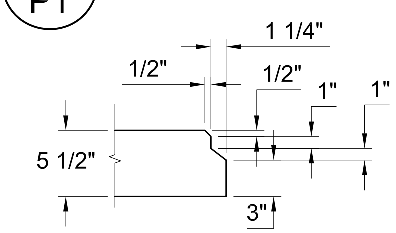

4 P1 PANEL ANCHOR



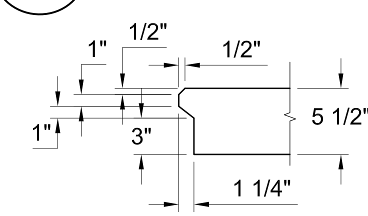

5 P1 PANEL BOTTOM




6 P1 PANEL TOP




7 P1 PANEL RIGHT SIDE




8 P1 PANEL LEFT SIDE

GENERAL NOTES

1. ALL DIMENSIONS ARE IN FEET AND INCHES (IMPERIAL UNITS) UNLESS NOTED OTHERWISE.
2. ALL PANELS ARE SHOWN BACK FACE.
3. ALL DIMENSIONS ARE SHOWN MEASURED FROM BACK FACE / BACK EDGE OF PRECAST PANEL FORM, TYPICAL.
4. ALL BLACK STEEL REINFORCEMENT AND EMBEDS SHALL HAVE A MINIMUM 1 ½" CONCRETE COVER
5. CONCRETE SHALL HAVE A MINIMUM 28-DAY COMPRESSIVE STRENGTH EQUAL TO 4000 PSI (UNLESS NOTED OTHERWISE) AND SHALL BE CAST IN ACCORDANCE WITH THE PROJECT SPECIFICATIONS.
6. CONCRETE SAMPLING, TESTING, CERTIFICATIONS AND PANEL PRODUCTION (INCLUDING BUT NOT LIMITED TO CASTING, CURING, LABELING, FINISHING, HANDLING, STORAGE AND REPORTING) SHALL BE CONDUCTED IN ACCORDANCE WITH THE GOVERNING PROJECT SPECIFICATION
7. EACH PANEL SHALL HAVE THE PANEL NAME, DATE OF MANUFACTURE AND LOT NUMBER SCRIBE IN THE BACK FACE OF PANEL.
8. THE FRONT FACE EDGE OF ALL PANELS SHALL HAVE A ½" X ½" CHAMFER (UNLESS NOTED OTHERWISE).
9. PANEL REINFORCING SHALL BE WELDED WIRE FABRIC CONFORMING TO ASTM A1064 STANDARD SPECIFICATION FOR STEEL WIRE AND WELDED WIRE REINFORCEMENT, PLAIN AND DEFORMED, FOR CONCRETE WITH A MINIMUM YIELD STRENGTH EQUAL TO 65 KSI.
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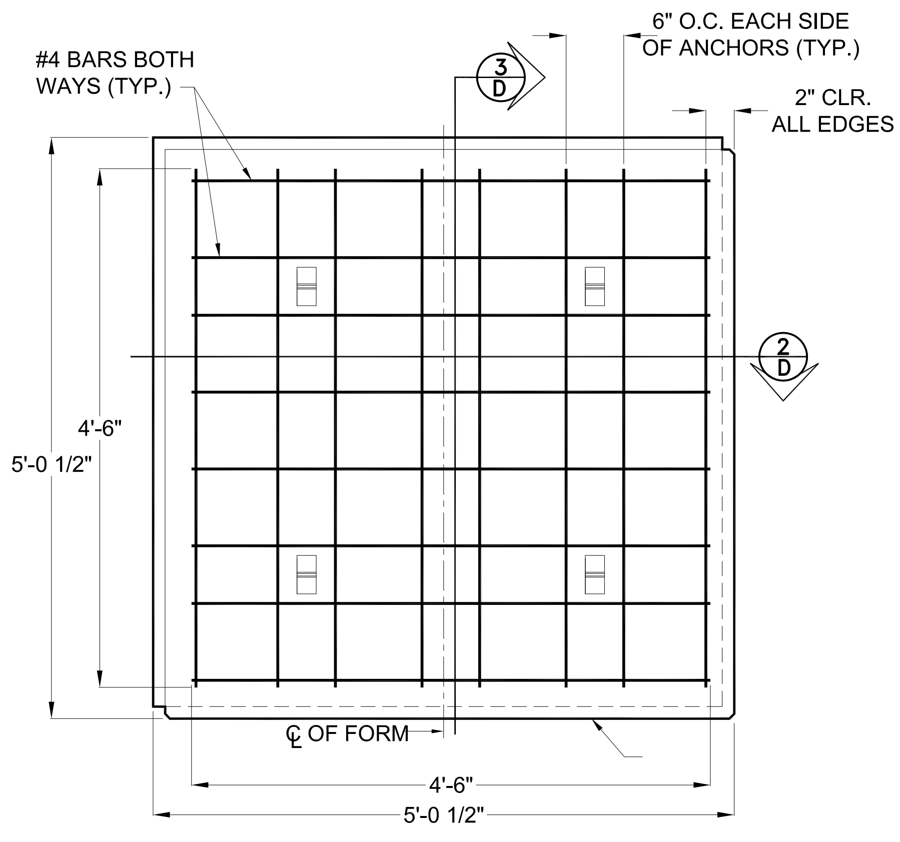
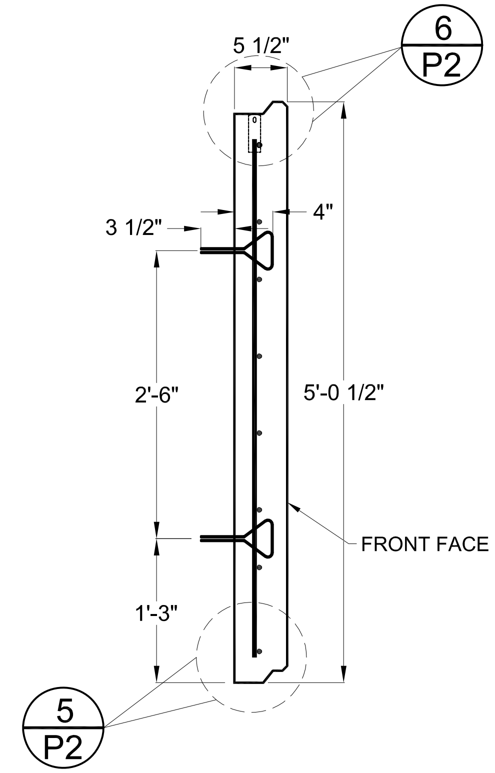
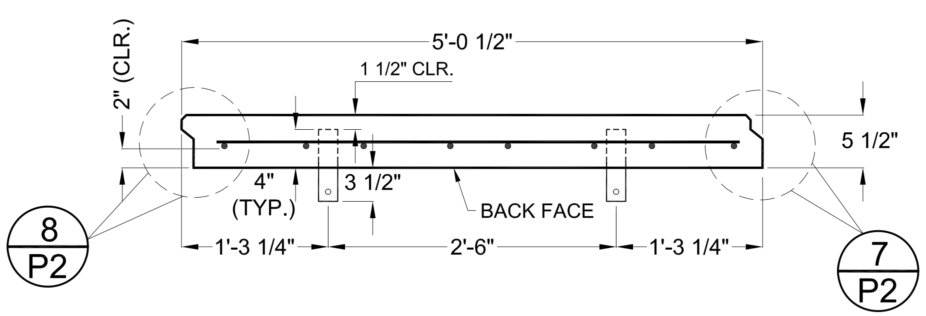


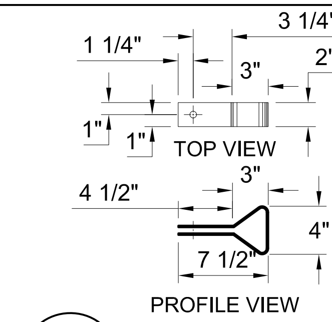

1 P2 STANDARD SQUARE PANEL
BACK FACE



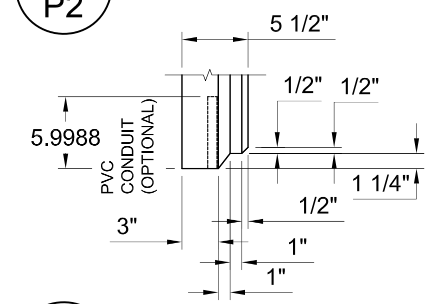

3 P2 STANDARD SQUARE PANEL
SECTION VIEW



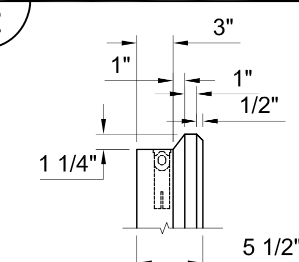

2 P2 STANDARD SQUARE PANEL
TOP VIEW



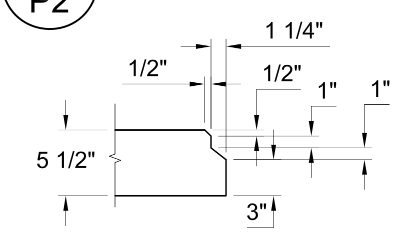

4 P2 PANEL ANCHOR



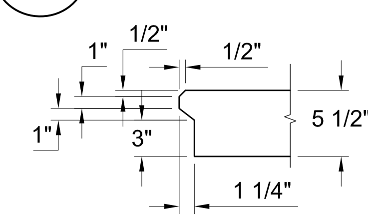

5 P2 PANEL BOTTOM




6 P2 PANEL TOP




7 P2 PANEL RIGHT SIDE




8 P2 PANEL LEFT SIDE

GENERAL NOTES

1. ALL DIMENSIONS ARE IN FEET AND INCHES (IMPERIAL UNITS) UNLESS NOTED OTHERWISE.
2. ALL PANELS ARE SHOWN BACK FACE.
3. ALL DIMENSIONS ARE SHOWN MEASURED FROM BACK FACE / BACK EDGE OF PRECAST PANEL FORM, TYPICAL.
4. ALL BLACK STEEL REINFORCEMENT AND EMBEDS SHALL HAVE A MINIMUM OF 1 ½" CONCRETE COVER
5. CONCRETE SHALL HAVE A MINIMUM 28-DAY COMPRESSIVE STRENGTH EQUAL TO 4,000 PSI (UNLESS NOTED OTHERWISE) AND SHALL BE IN CAST ACCORDANCE WITH THE PROJECT SPECIFICATIONS.
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7. ALL PANELS SHALL HAVE THE PANEL NAME, DATE OF MANUFACTURE AND LOT NUMBER SCRIBE IN THE BACK FACE OF PANEL.
8. THE FRONT FACE EDGE OF ALL PANELS SHALL HAVE A ½" X ½" CHAMFER (UNLESS NOTED OTHERWISE).
9. PANEL REINFORCING SHALL BE A MINIMUM #4 BAR (BOTH WAYS) CONFORMING TO ASTM A615 MILD REINFORCING (REBAR) GRADE 60 OR EQUIVALENT.
10. THE GALVANIZED ANCHOR EMBEDS SHALL NOT BE IN CONTACT WITH THE BLACK PANEL REINFORCEMENT.
11. ALL ANCHORS ARE TO BE POSITIONED AS DETAILED AND POSITION PERPENDICULAR (90 DEGREES) WITH RESPECT TO THE BACK FACE OF PANEL.
12. PANEL LIFT ANCHOR SHALL BE DAYTON SUPERIOR P-93 FLEET-LIFT L-ANCHOR (PRODUCT CODE FL050, HOT-DIPPED GALVANIZED) OR EQUIVALENT.
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PRECAST PANEL DETAIL
5' WIDE PANEL

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www.groundimprovementsystems.com

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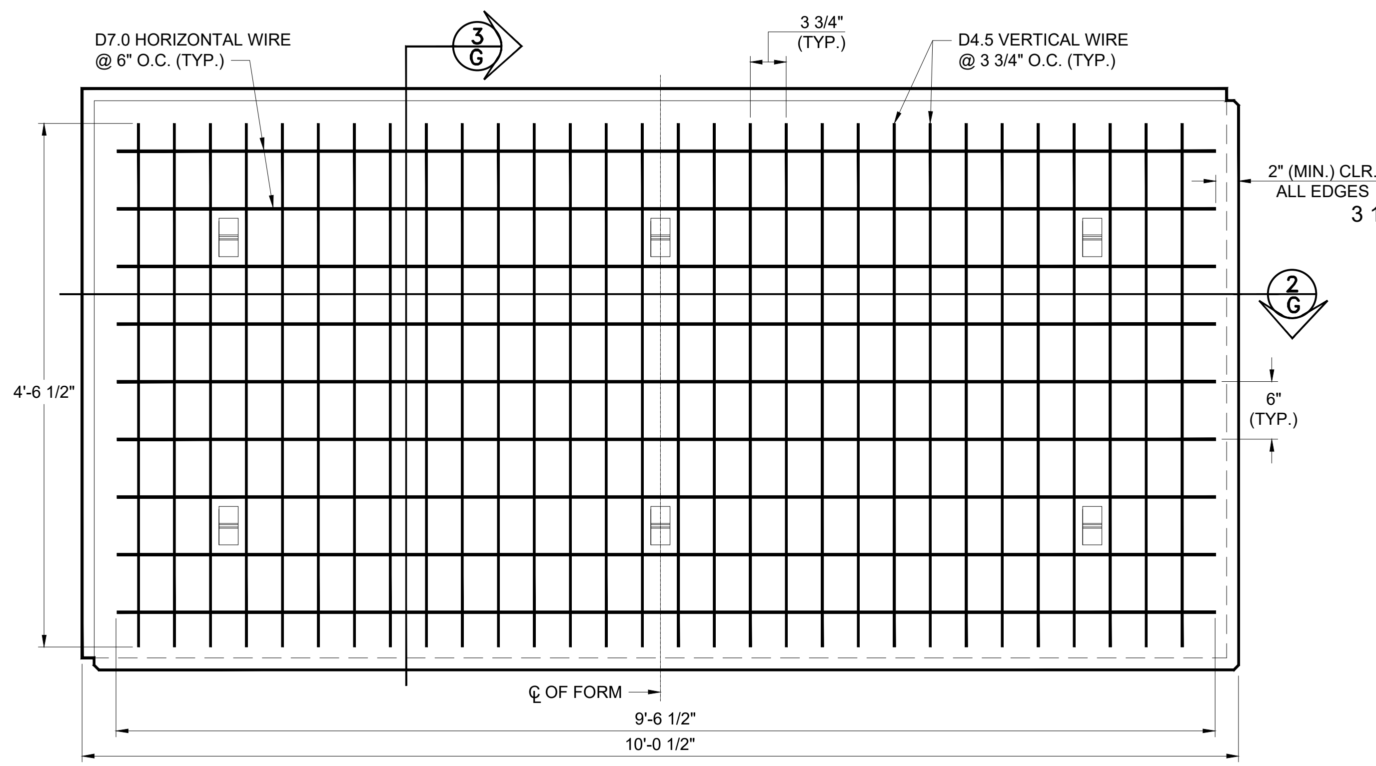
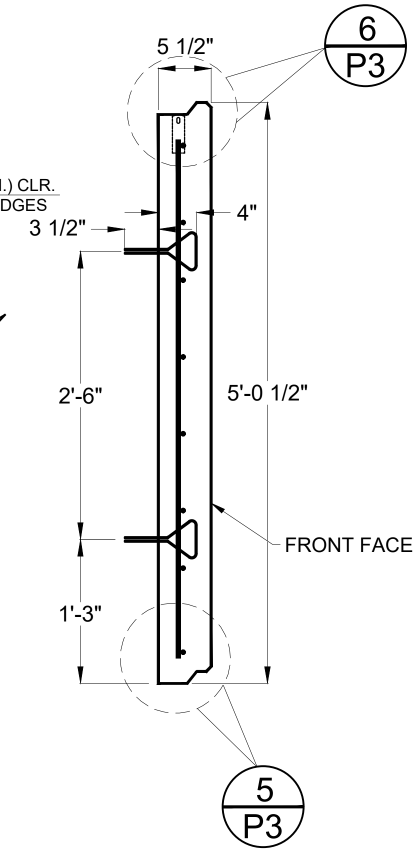
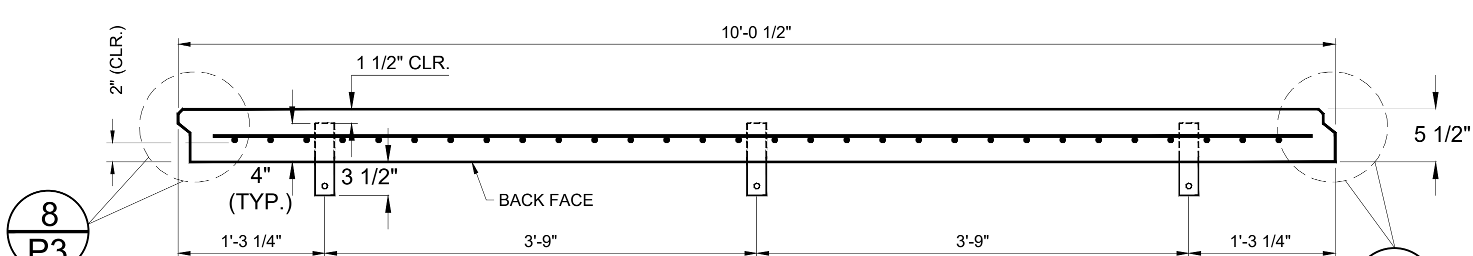


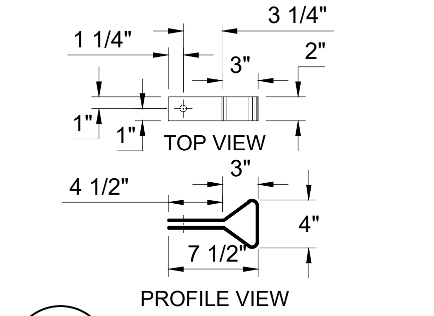




1 P3 STANDARD RECTANGULAR PANEL
BACK FACE

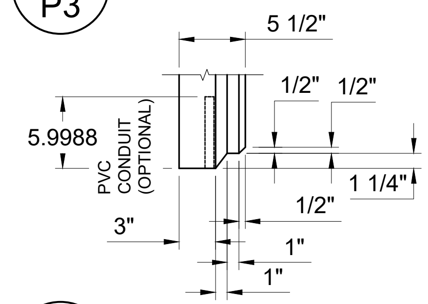
3 P3 STANDARD RECTANGULAR PANEL
SECTION VIEW



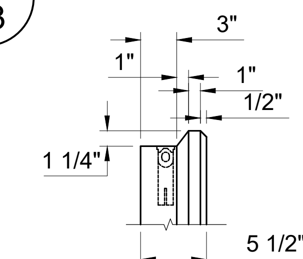

2 P3 STANDARD RECTANGULAR PANEL
TOP VIEW



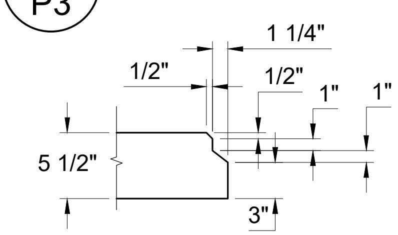

4 P3 PANEL ANCHOR



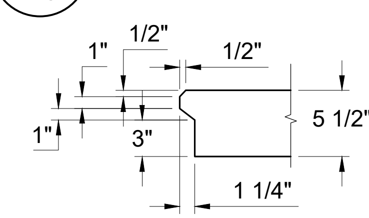

5 P3 PANEL BOTTOM




6 P3 PANEL TOP




7 P3 PANEL RIGHT SIDE




8 P3 PANEL LEFT SIDE

GENERAL NOTES

1. ALL DIMENSIONS ARE IN FEET AND INCHES (IMPERIAL UNITS) UNLESS NOTED OTHERWISE.
2. ALL PANELS ARE SHOWN BACK FACE.
3. ALL DIMENSIONS ARE SHOWN MEASURED FROM BACK FACE / BACK EDGE OF PRECAST PANEL FORM, TYPICAL.
4. ALL BLACK STEEL REINFORCEMENT AND EMBEDS SHALL HAVE A MINIMUM 1 ½" CONCRETE COVER
5. CONCRETE SHALL HAVE A MINIMUM 28-DAY COMPRESSIVE STRENGTH EQUAL TO 4000 PSI (UNLESS NOTED OTHERWISE) AND SHALL BE CAST IN ACCORDANCE WITH THE PROJECT SPECIFICATIONS.
6. CONCRETE SAMPLING, TESTING, CERTIFICATIONS AND PANEL PRODUCTION (INCLUDING BUT NOT LIMITED TO CASTING, CURING, LABELING, FINISHING, HANDLING, STORAGE AND REPORTING) SHALL BE CONDUCTED IN ACCORDANCE WITH THE GOVERNING PROJECT SPECIFICATION
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9. PANEL REINFORCING SHALL BE WELDED WIRE FABRIC CONFORMING TO ASTM A1064 STANDARD SPECIFICATION FOR STEEL WIRE AND WELDED WIRE REINFORCEMENT, PLAIN AND DEFORMED, FOR CONCRETE WITH A MINIMUM YIELD STRENGTH EQUAL TO 65 KSI.
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11. PLACE PANEL ANCHORS AS DETAILED AND SO THEY ARE 90° TO THE BACK FACE OF THE PANEL..
12. PANEL LIFT ANCHOR SHALL BE DAYTON SUPERIOR P-93 FLEET-LIFT L-ANCHOR (PRODUCT CODE FL050, HOT-DIPPED GALVANIZED) OR EQUIVALENT.
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14. PANELS SHALL NOT BE REMOVE FROM FORM UNTIL A MINIMUM 1500 PSI STRENGTH BEEN ACHIEVED.
15. PANELS SHALL BE REMOVED FROM THE FORM USING A MINIMUM OF 4 POINTS THAT ARE EVENLY AND SYMMETRICALLY DISTRIBUTED ON THE BACK FACE OF THE PANEL.

THE DESIGN CONTAINED HEREIN IS BASED ON INFORMATION SUPPLIED BY OTHERS.GIS CERTIFYING INTERNAL STABILITY OF THE STRUCTURE ONLY. EXTERNAL AND GLOBAL STABILITY REQUIREMENTS ARE THE RESPONSIBILITY OF OTHERS.

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10' WIDE PANEL


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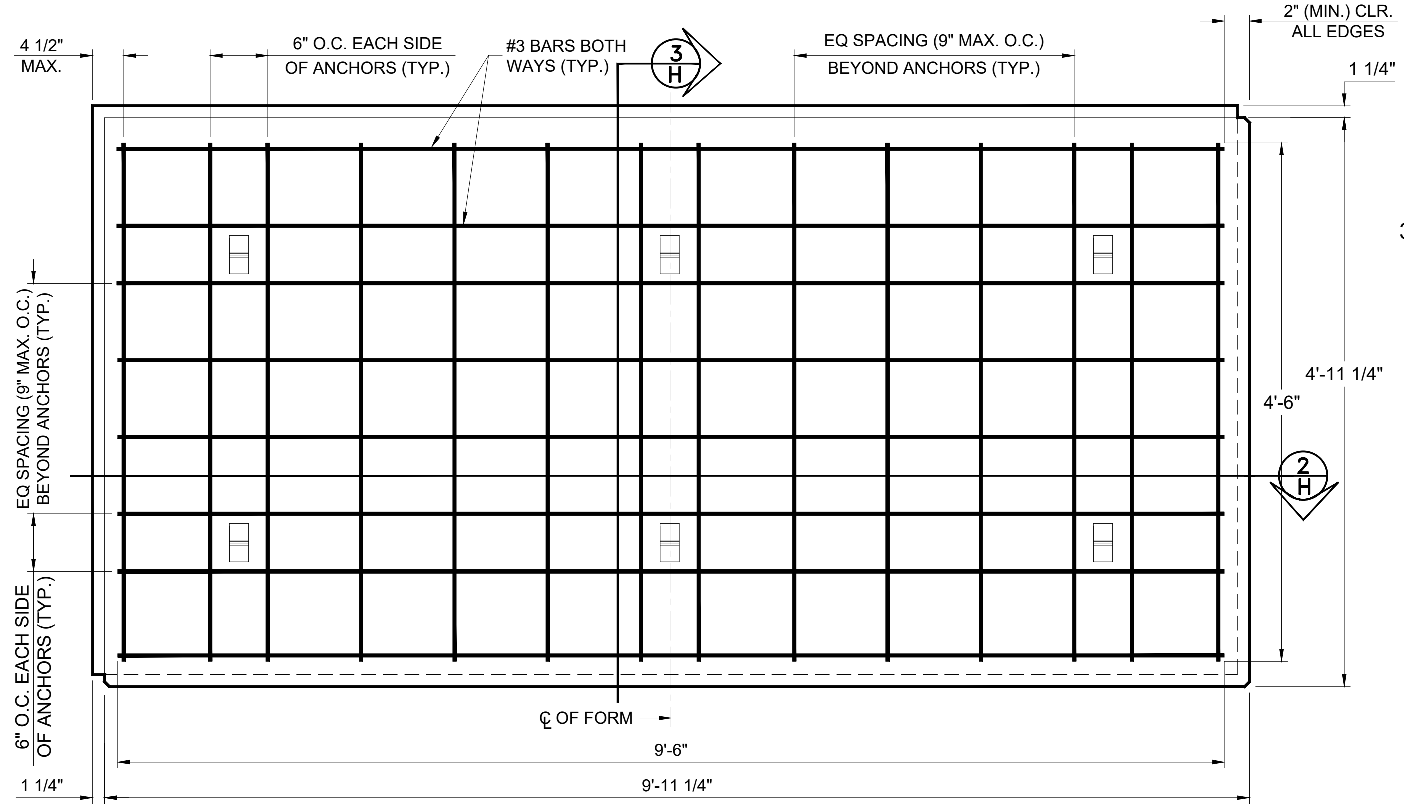
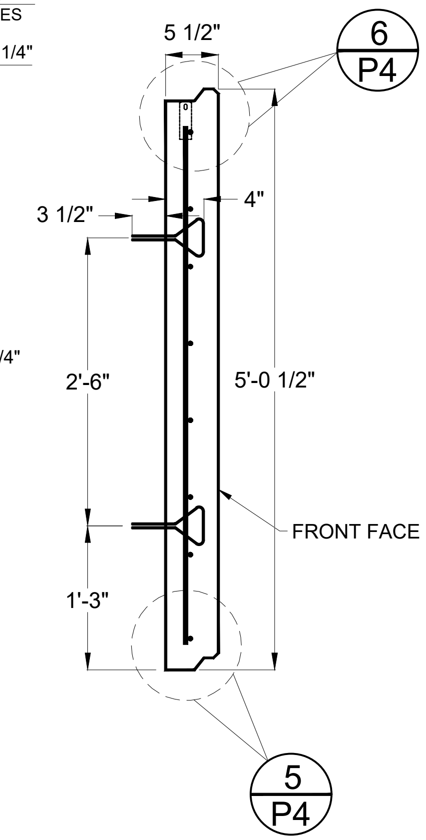
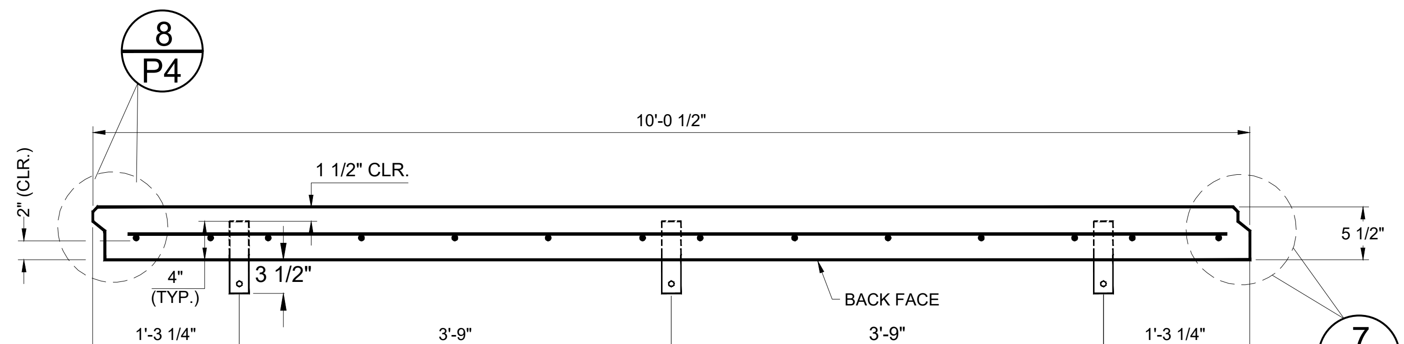


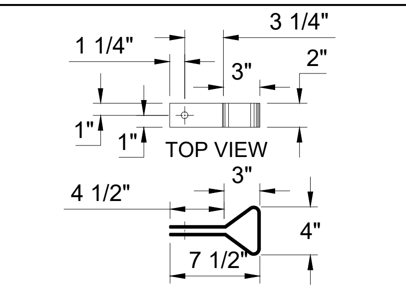

1/P4 STANDARD RECTANGULAR PANEL
BACK FACE



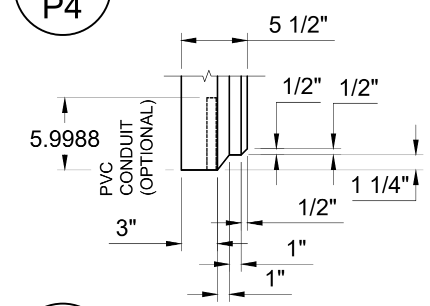

3/P4 STANDARD RECTANGULAR PANEL
SECTION VIEW



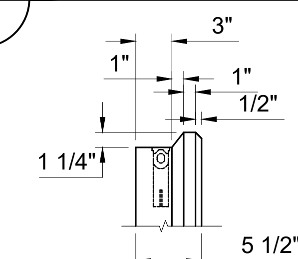

2/P4 STANDARD RECTANGULAR PANEL
TOP VIEW



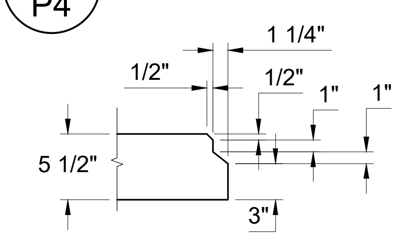

4/P4 PANEL ANCHOR



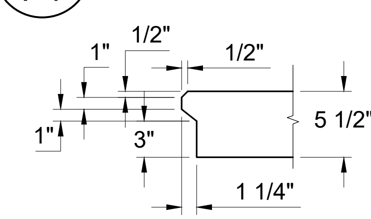

5/P4 PANEL BOTTOM




6/P4 PANEL TOP




7/P4 PANEL RIGHT SIDE




8/P4 PANEL LEFT SIDE

GENERAL NOTES

1. ALL DIMENSIONS ARE IN FEET AND INCHES (IMPERIAL UNITS) UNLESS NOTED OTHERWISE.
2. ALL PANELS ARE SHOWN BACK FACE.
3. ALL DIMENSIONS ARE SHOWN MEASURED FROM BACK FACE / BACK EDGE OF PRECAST PANEL FORM, TYPICAL.
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10. THE GALVANIZED ANCHOR EMBEDS SHALL NOT BE IN CONTACT WITH THE BLACK PANEL REINFORCEMENT.
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CHECKED BY XXX

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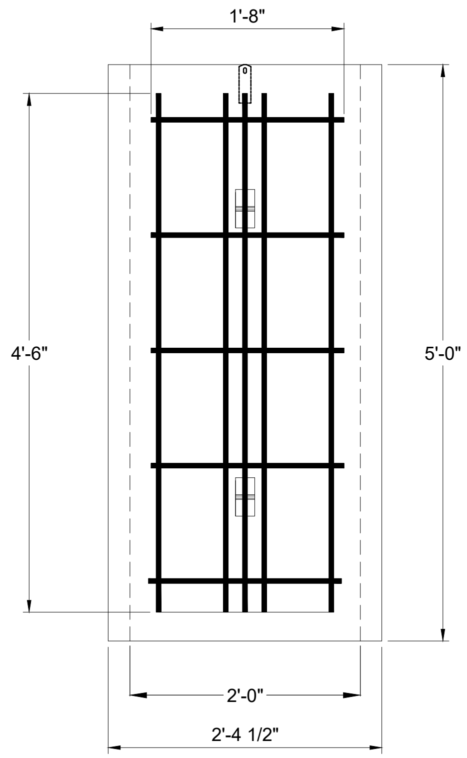
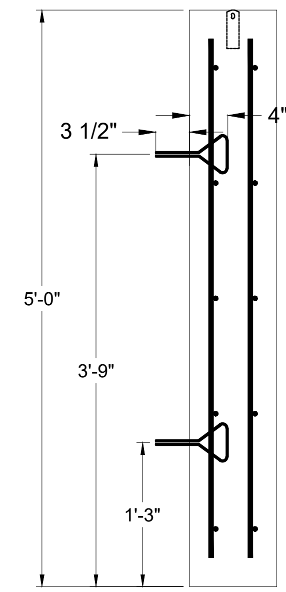
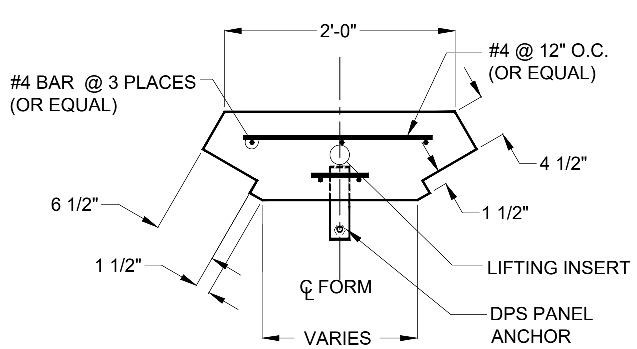


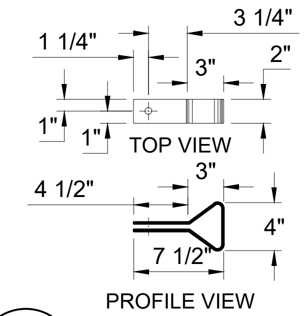

D P5 VARIABLE CORNER PANEL (TYPE-X)
BACK FACE



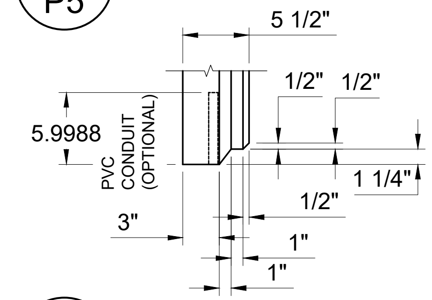

D P5 VARIABLE CORNER PANEL (TYPE-X)
SECTION VIEW



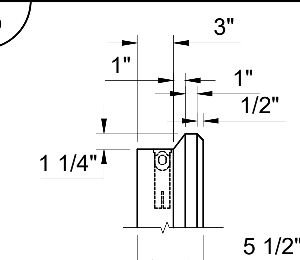

D P5 VARIABLE CORNER PANEL (TYPE-X)
PLAN VIEW



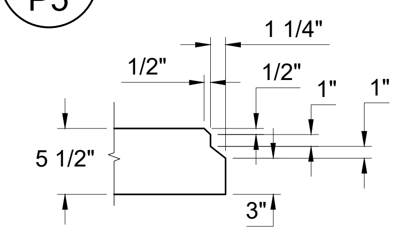

4 P5 PANEL ANCHOR



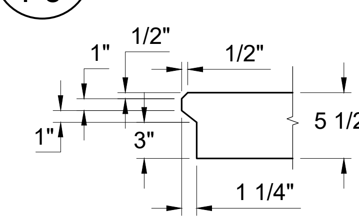

5 P5 PANEL BOTTOM




6 P5 PANEL TOP




7 P5 PANEL RIGHT SIDE




8 P5 PANEL LEFT SIDE

GENERAL NOTES

1. ALL DIMENSIONS ARE IN FEET AND INCHES (IMPERIAL UNITS) UNLESS NOTED OTHERWISE.
2. ALL PANELS ARE SHOWN BACK FACE.
3. ALL DIMENSIONS ARE SHOWN MEASURED FROM BACK FACE / BACK EDGE OF PRECAST PANEL FORM, TYPICAL.
4. ALL BLACK STEEL REINFORCEMENT AND EMBEDS SHALL HAVE A MINIMUM 1 ½" CONCRETE COVER
5. CONCRETE SHALL HAVE A MINIMUM 28-DAY COMPRESSIVE STRENGTH EQUAL TO 4000 PSI (UNLESS NOTED OTHERWISE) AND SHALL BE CAST IN ACCORDANCE WITH THE PROJECT SPECIFICATIONS.
6. CONCRETE SAMPLING, TESTING, CERTIFICATIONS AND PANEL PRODUCTION (INCLUDING BUT NOT LIMITED TO CASTING, CURING, LABELING, FINISHING, HANDLING, STORAGE AND REPORTING) SHALL BE CONDUCTED IN ACCORDANCE WITH THE GOVERNING PROJECT SPECIFICATION
7. EACH PANEL SHALL HAVE THE PANEL NAME, DATE OF MANUFACTURE AND LOT NUMBER SCRIBE IN THE BACK FACE OF PANEL.
8. THE FRONT FACE EDGE OF ALL PANELS SHALL HAVE A ½" X ½" CHAMFER (UNLESS NOTED OTHERWISE).
9. PANEL REINFORCING SHALL BE WELDED WIRE FABRIC CONFORMING TO ASTM A1064 STANDARD SPECIFICATION FOR STEEL WIRE AND WELDED WIRE REINFORCEMENT, PLAIN AND DEFORMED, FOR CONCRETE WITH A MINIMUM YIELD STRENGTH EQUAL TO 65 KSI.
10. THE GALVANIZED ANCHOR EMBEDS SHALL NOT BE IN CONTACT WITH THE BLACK PANEL REINFORCEMENT.
11. PLACE PANEL ANCHORS AS DETAILED AND SO THEY ARE 90° TO THE BACK FACE OF THE PANEL..
12. PANEL LIFT ANCHOR SHALL BE DAYTON SUPERIOR P-93 FLEET-LIFT L-ANCHOR (PRODUCT CODE FL050, HOT-DIPPED GALVANIZED) OR EQUIVALENT.
13. ALL PANELS ARE TO BE MANUFACTURED TO THE DIMENSION SHOWN IN THE SHOP DRAWINGS UNLESS NOTED OTHERWISE. PANEL SQUARENESS AND TOLERANCES OUTLINED IN THE EARTHTEC PRECAST PANEL QUALITY CONTROL SPECIFICATION AND WITHIN THE GOVERNING SPECIFICATIONS REQUIREMENTS.
14. PANELS SHALL NOT BE REMOVE FROM FORM UNTIL A MINIMUM 1500 PSI STRENGTH BEEN ACHIEVED.
15. PANELS SHALL BE REMOVED FROM THE FORM USING A MINIMUM OF 4 POINTS THAT ARE EVENLY AND SYMMETRICALLY DISTRIBUTED ON THE BACK FACE OF THE PANEL.

THE DESIGN CONTAINED HEREIN IS BASED ON INFORMATION SUPPLIED BY OTHERS.GIS CERTIFYING INTERNAL STABILITY OF THE STRUCTURE ONLY. EXTERNAL AND GLOBAL STABILITY REQUIREMENTS ARE THE RESPONSIBILITY OF OTHERS.

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PRECAST PANEL DETAIL
STANDARD CORNER PANEL

GROUND IMPROVEMENT SYSTEMS LLC
114 SOUTH COLLINS STREET
ARLINGTON, TX 76011
(817) 223-0969
www.groundimprovementsystems.com

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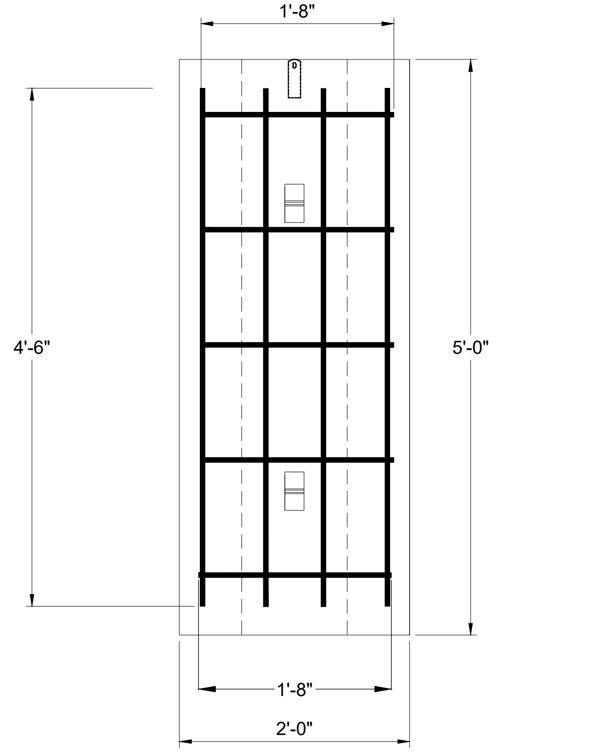
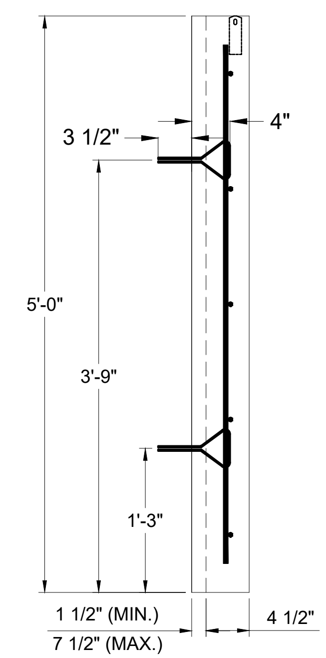
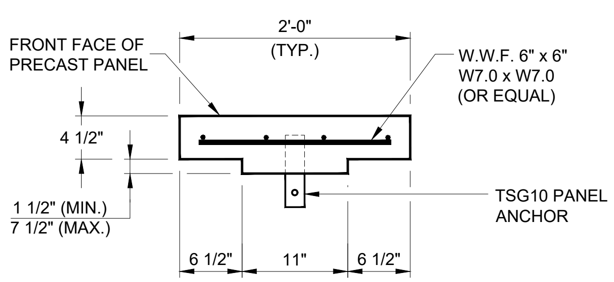


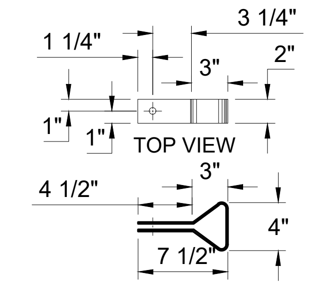

E / P6 SLIP JOINT PANEL DETAIL
BACK FACE




E / P6 SLIP JOINT PANEL DETAIL
SECTION VIEW




E / P6 SLIP JOINT PANEL DETAIL
PLAN VIEW

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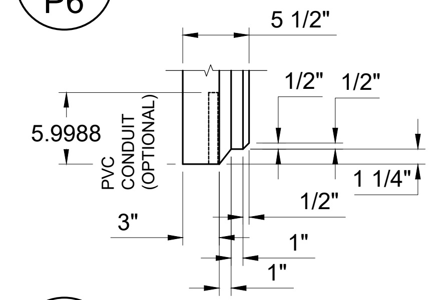


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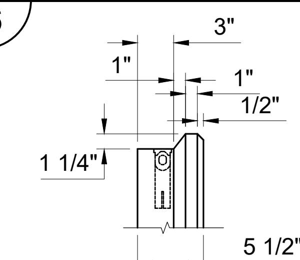


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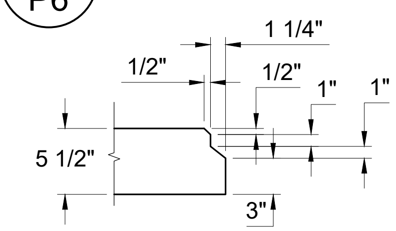
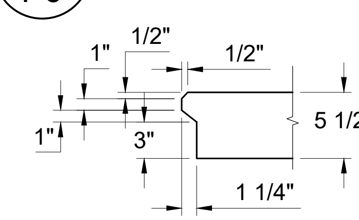



7 / P6 PANEL RIGHT SIDE




8 / P6 PANEL LEFT SIDE

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## 1.1.3
## Facing Unit Specification

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# EARTHTEC™ REINFORCED SOIL WALL

# PROCEDURES AND QUALITY CONTROL GUIDELINES

# PRECAST SEGMENTAL CONCRETE PANEL


**GROUND IMPROVEMENT SYSTEMS LLC**

114 South Collins Street

Arlington, TX 76011

817-223-0969 Office

www.groundimprovementsystems.com


Support Through Innovation

# MECHANICALLY STABILIZED EARTH STRUCTURES

## REINFORCED SOIL WALL (RWS) SYSTEM

## PROCEDURES AND QUALITY CONTROL GUIDELINES

## PRECAST SEGMENTAL CONCRETE PANEL


**GROUND IMPROVEMENT SYSTEMS LLC**

114 South Collins Street

Arlington, TX 76011

817-223-0969

# TABLE OF CONTENTS

# SEGMENTAL CONCRETE PANEL SPECIFICATION

## 1 GENERAL

This specification pertains to the casting of the Segmental Concrete panel Reinforced SoilWall system. Panels shall be cast according to this specification and in reasonably close conformity with the dimensions shown on the plans or established by the Engineer.

## 2 REFERENCES

**2.1 American Society for Testing and Materials (ASTM)**

**2.1.1** A36 - Standard Specification for Carbon Structural Steel

**2.1.2** A123 - Standard Specifications for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products

**2.1.3** A153 - Standard Specifications for Zinc Coating (Hot-Dip) on Iron and Steel Hardware

**2.1.4** A325 - Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength

**2.1.5** A496 - Standard Specifications for Steel Wire Reinforcement, Deformed, for Concrete

**2.1.6** A497 - Standard Specifications for Welded Wire Reinforcement, Deformed, for Concrete

**2.1.7** A525 - Specification for General Requirements for Steel Sheet, Zinc-Coated (Galvanized) by the Hot-Dip Process

**2.1.8** A510 - Standard Specification for General Requirements for Wire Rods and Coarse Round Wire, Carbon Steel

**2.1.9** A615 - Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement

**2.1.10** A780 - Standard Specification for the Repair of Damaged and Uncoated Areas of Hot-Dip Galvanized Coatings

**2.1.11** A884 - Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Reinforcement

**2.1.12** A1064 -Standard Specification for Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete.

**2.1.13** C150 – Standard Specification for Portland Cement

**2.2 American Association of State Highway and Transportation Officials**

**2.2.1** M85 – Standard Specification for Portland Cement

**2.2.2** T22 - Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimens

**2.2.3** T23 - Standard Method of Test for Making and Curing Concrete Test Specimens in the Field

**2.2.4** T141 - Standard Method of Test for Sampling Freshly Mixed Concrete

**2.3 CRSI - MANUAL of Standard Practice**

**2.4 Wire Reinforcement Institute -**

MANUAL of Standard Practice

**2.5 ACI 318-08 - Building codes for Structural Concrete**

## 3 CEMENT

Cement shall be Types I, II and III with 3% to 6% air entrainment and shall conform to the requirements of AASHTO M85. Concrete shall have a compressive strength at twenty-eight (28) days in accordance with Section 8, Concrete Compressive Strength. Air entraining, retarding, accelerating agents, or any additives containing chloride shall not be used without the Owner's approval. Determine the maximum aggregate size in accordance with AASHTO LRFD standards and the governing specification.

## 4 TESTING AND INSPECTION

Acceptability of all panels shall be on the basis of compressibility tests and visual inspection. Precast units shall be considered acceptable regardless of curing time when compressive strength meets or exceeds the 28-day compressive strength. Contractor shall be responsible for all testing and shall provide a facility to perform tests. Units using Type-I or Type-II cement shall be deemed acceptable to be placed in the retaining wall when the seven (7) day compressive strength exceeds 85% of the 28 day compressive strength requirements. Units utilizing Type-III cement will be deemed acceptable for placement in the retaining wall when the compressive strength meets or exceeds the 28 day compressive strength requirements. Production lots will be recorded and tested for conformance. Any lot not meeting this specification shall be rejected.

## 5 CASTING

All panels shall be cast face down in smooth, flat, steel forms. Panel anchors and inserts shall be placed in a template at the back of the panel. Galvanized anchors and galvanized inserts shall not be allowed to contact black steel panel reinforcing. If contact is to occur they shall be separated by a non-conductive isolator. Concrete shall be placed without interruption. Concrete shall be vibrated using a form vibrator or hand vibrator. Clear form oil shall be used.

## 6 CURING

When the temperature of the air is between zero (0°) F and 30° F, the minimum concrete temperature should be 65° F at placement.

When the air temperature is above 30° F, the temperature of the concrete should be 60° F.

### 6.1 Hot Weather or Indoor

The panel shall be cured in the steel form for a sufficient length of time that allows the panel to be stripped without causing undue stress or damage to the panel. The panel shall be kept sufficiently wet and protected in order to prevent the temperature of the concrete from dropping below 80° F.

### 6.2 Cold Weather

The panel shall be cured in the steel forms placed a minimum of 6” off the ground. The concrete slump shall be kept less than four (4) inches. No extra water shall be sprinkled on the concrete surface. Newly placed concrete shall be kept from freezing by maintaining 55° F for 72 hours and maintain temperatures above 40° F for an additional four (4) days. Monitor temperature on corners and edges. Use approved curing compounds to reduce drying.

## 7 Lifting Devices

All lifting devices as specified by EarthTec, or an approved equivalent, shall be used to strip panels from the form. No panel shall be placed in the MSE structure until it meets the requirements of this specification

## 8 Concrete Finish

Unless otherwise noted on the plans or elsewhere in the project specifications, the exposed concrete surface shall be smooth gray. The rear of each panel shall be hand screed smooth to eliminate open pockets of aggregate and surface distortions in excess of ¼” (6 mm).

## 9 Tolerances

### 9.1 Panel dimensions

Panel dimensions shall be within 3/16” (5 mm) of dimensions as noted on the plans.

### 9.2 Panel Squareness

The panel shall be considered square when the differences of two parallel verticals measured at the side of the panel do not exceed ½” (13 mm) or the difference between tow diagonals do not exceed ½”

### 9.3 Panel Smoothness

Smooth panel surface finish shall be free of defects that exceed 1/8” (2.5 mm) as measured on a length of 60 inches (1525 mm). Textured panel surface finish shall be free of defects that exceed ¼” (6 mm) as measured on a length of (1525 mm)

## 10 CONCRETE COMPRESSIVE STRENGTH

### 10.1 Acceptance

The acceptance of concrete units with respect to compressive strength will be determined based on production lots. A production lot is represented as a single compressive strength sample and will not be more than 80 panels or one day's production whichever is less. The compressive strength shall be no less than 4000 psi.

### 10.2 Sampling

Concrete will be sampled for each production lot in accordance with AASHTO T-141. A minimum of four cylinders will be randomly selected for each production lot.

**10.2.1** Frequency

Cylinders shall be taken in accordance with AASHTO T-23 on 6" (150 mm) x 12" (300 mm) specimens. For every compressive strength sample, a minimum of two (2) cylinders will be cured in the same manner as the panels are and tested at approximately seven (7) days. The average compressive strength of these two (2) cylinders when tested in accordance with AASHTO T-22 will provide a test result, which will determine the initial strength of concrete. In addition, two (2) cylinders will be cured in accordance with AASHTO T-23 and tested at approximately twenty-eight (28) days. The average compressive strength of these two (2) cylinders when tested in accordance with AASHTO T-22 will provide a compressive strength test result, which will determine the compressive strength of the production lot.

**10.2.2** Initial Test Results

For the initial strength test results if the compressive strength is in excess of 4000 psi then these test results will be utilized as the compressive strength test results for that production lot, and the 28 day requirement will be waived for the lot in question.

**10.2.3** Compressive Strength Acceptance

Acceptance of a production lot will be made if the compressive strength test result is greater than or equal to 4000 psi. If the compressive strength is less than 4000 psi the acceptance of the production lot will be based on its meeting the following acceptance criteria in its entirety:

**10.2.3.1** Ninety Percent Rule

If 90% of the compressive strength test results for the overall production exceed 4000 psi.

**10.2.3.2** Average Six Rule

If the average of any six (6) consecutive compressive strength tests results exceed 4000 psi.

**10.2.4** Compressive Strength Rejection

Production lots will be rejected for failure to meet specified compressive strength requirements. In order to get the production lot accepted the manufacture, at his or her own expense, may obtain and submit evidence the strength and quality of concrete placed within the panels of the production lot is acceptable. All core samples shall be obtained and tested in accordance with AASHTO T-24.

## 11 REJECTION

Units shall be subject to rejection for failure to meet any requirements specified above. In addition, any or all of the following defects may be sufficient cause for rejection.

### 11.1 Molding

Any defects that would indicate the imperfect molding of the panel.

### 11.2 Texture

Defects indicating honeycombed or open texture in the concrete.

### 11.3 Physical Characteristics

Defects in physical characteristics of the concrete, such as broken or chipped concrete.

### 11.4 Repair

It shall be the responsibility of the Owner to determine whether the spalled, honeycombed, chipped or otherwise imperfect concrete shall be repaired or be cause for rejection. The panel shall be repaired in such a manner that is acceptable to and approved by the Owner.

### 11.5 Marking

The date of production, the production lot number and the piece mark shall be clearly scribed on the rear face of the panel.

## 12 PANEL ACCESSORIES

Panel anchors, clips, and inserts shall be set in place to the dimensions and tolerances as shown on the plans.

### 12.1 Panel Anchors

All panel anchors shall be in accordance with ASTM A510 - Standard Specification for General Requirements for Wire Rods and Coarse Round wire, Carbon Steel.

### 12.2 Structural Members

All structural members shall be in accordance with ASTM A36/36M - standard Specification for Structural Steel

### 12.3 Fasteners

All fasteners and inserts shall be in accordance with ASTM A325 - Standard Specification for High-Strength Bolts for Structural Steel

### 12.4 Panel Reinforcement

All panel reinforcing shall have a minimum concrete cover of 1 ½" and shall be placed no closer than 2" to the edge of the concrete. Reinforcement shall be placed so the panel anchor is not in contact.

**12.4.1** Welded Wire Reinforcing

All welded wire panel reinforcement shall be fabricated in accordance with ASTM 1064 - Standard Specification for Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete.

**12.4.2** Reinforcing Bars

All bar reinforcement shall be in accordance with ASTM A615 - Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement and shall be Grade 60.

### 12.5 Galvanizing

All metallic accessories that require corrosion protection shall be galvanized in accordance with ASTM A123 - Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products or ASTM A153 Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware.

### 12.6 Epoxy Coating

All accessories that require epoxy coating shall be in accordance with ASTM A884 - Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Fabric for Reinforcement

## APPENDIX A – QUALITY CONTROL PROGRAM FOR PRECAST PANELS

### 1. LIMITATIONS

Information that is contained in this Appendix and all EarthTec, documents are not to be used to design, fabricate, manufacture, assemble, construct, produce, or install or otherwise use any elements, forms, or other special equipment (whether patented or not) that is exclusive to the EarthTec Reinforced Soil Wall (RSW) system, or for any other purpose other than this project without the express written consent of EarthTec. The information contained herein shall not be copied, disclosed, or distributed in any manner, in whole or in part, to any third party without the prior express written consent of EarthTec. The Quality Control procedures outlined in this manual have been developed to aid in designing, manufacturing, supplying materials, and installing the EarthTec RSW system. It is not intended to replace any of the Owner's requirements and is used as a supplement thereto.

### 2. PRE-POUR FORM PREPARATION

The forms supplied by EarthTec are unique to the RSW system. They have been designed to aid the Pre-Caster in rapid set up and stripping of the RSW product. The following procedure should be used prior to the placement of concrete and in conjunction with each subsequent pour. The key to making an acceptable and error free panel is to keep the forms clean.

#### 1.1. SET-UP

Set each forming pallet up to the required plant specific height and in their designated locations. Each form should arrive to the site with a pallet, side rails, top and bottom rails, panel anchor holders and all necessary hardware to attach each item.

#### 1.2. CLEAN-UP

After the forming pallet is set up thoroughly clean each steel element and assure that they are free from dirt, grease, oil, and debris. This is especially true at areas of contact points between interfaces.

### 1.3. Release Agent

Using a hand pumped or airless sprayer, coat the interior form surfaces of forming elements that will be in contact with the placed concrete with an evenly distributed and uniform coat of release agent. The release agent shall be applied in such a manner that minimizes the formation of release agent puddles on the form face and at the interface of rail and bed elements. The release agent when applied properly will help insure a defect free concrete surface and maintain the working condition of the forms. Note that puddles of the release agent will create a halo stain on the concrete face.

### 1.4. Form Liner Placement

If a form liner panel finish is required, place and attach the project specific elastomeric or urethane form liner on the steel form using the required mounting hardware. Insure that the liner is placed in the form so it is flush to the surface. If required use a caulking compound to seal the form liner at the interface of form face and the side rails, bottom rails, and top rails to prevent concrete bleed from occurring. As prescribed by the form liner manufacturer apply a release agent on the concrete surface of the liner.

## 3. Individual Form Set-Up Procedure

All required embedded items, panel reinforcing, attachment devices, panel anchors, etc., shall be in accordance with the approved shop drawings. Any special panel requirements shall be clearly illustrated on a production shop drawing and shall match requirements as specified within the approved shop drawings. All production shop drawings shall be used by the person who is charged with the set-up of the form, or forms. Production shop drawings shall be located in close proximity to the form until the concrete placement has concluded.

The general steps to setting up each individual form are:

STEP 1. Place and secure any headers, side rails, and, or, block outs that are required to create special panels.

STEP 2. Place the rebar or equivalent welded wire panel reinforcing to assure the proper depth of embedment is achieved.

STEP 3. Position the lifting inserts in the top edge of the panel or panel header.

STEP 4. Place and secure the required number of panel anchor holders in the proper location as specified on the panel production drawing.

STEP 5. Place the required number of individual panel anchors at the proper location in each panel anchor holder and secure in place with the required form mounting hardware.

STEP 6. Perform a final form setup inspection by the Quality Control representative to assure the form size, anchor placement, and special requirements have been included as illustrated in the panel production drawing.

## 4. Concrete Placement

### 4.1. Concrete Truck

Prior to the batching of the first load of concrete, the Owner Certified mixer truck(s) to be used for the delivery and dispensing of that days concrete shall be thoroughly checked (using a standardize checklist) to insure the proper handling and mixing of the concrete. Further, the mixer truck(s) are to be subject to regular washing and rinsing throughout each production day to prevent any buildup of cement deposits or deleterious materials from occurring.

### 4.2. Concrete Class

The class of concrete as required by contract documents and the approved mix design shall be batched and supplied by the Precaster from the point of placement/destination and in accordance with the Owner's concrete specification.

### 4.3. Concrete Transportation and Handling

All concrete shall be transported by a certified mixer truck from the Precast batch facility to the precast panel form location and handled in accordance with the Owner's concrete specification and the Owner's materials manual. The concrete will be deposited into the panel forms directly from the chute or via concrete hopper that is filled in close proximity to panel forms and mixer truck. As required, and to fill remaining voids within each respective form, the concrete shall be placed via use of hand dispensed spade shovel to insure proper concrete volume in each panel form.

### 4.4. Concrete Consolidation Method

Once a form is filled to the appropriate volume, the concrete will be consolidated via use of hand held electric internal vibrators and/or pneumatic external vibrators in accordance with the Owner's concrete specification. Vibration shall be continued in overlapping fields of action until proper consolidation has occurred. Once all air bubbles from the overlapping field of action has ceased appearing at the surface, the internal vibration shall be discontinued immediately to prevent possible segregation of aggregates. Once proper consolidation has occurred, strike-off the exposed surface with a straight-edge to insure the concrete extends to the top edge of the panel form and to remove any excessive concrete remaining in the form.

## 5. Protection Methods During Inclement Weather

### 5.1. Hot Weather Precautions –

In the summer months and late afternoon pours, necessary steps shall be taken to minimize the heating of the steel forms due to the direct sunlight. A procedure using water on the exterior surfaces and/or the temporary covering of the forms to shield the steel forms from the direct sunlight shall be instituted.

### 5.2. Pre-Inclement Weather Precautions

If foreseeable inclement weather is approaching prior to the placement of concrete, the Quality Control Manager will consult with the relevant parties to decide on whether to proceed with a schedule pour or reschedule as required.

### 5.3. Rain Events

If rain begins to develop during a placement of concrete, the charging mouth of the mixer/agitator truck will be covered immediately to prevent additional water from entering the truck and concrete. In addition, all cast panels will be immediately covered with plastic or a cure blanket to prevent deformation or the introduction of additional water into the exposed face.

### 5.4. Post-Inclement Weather Precautions

As a result of the inclement weather, if water has accumulated on the flat exposed form surface, the water shall be removed using portable air to blow the excessive water from the forming surface prior to the placement of concrete. This shall be done in each affected form.

## 6. Concrete Finishing and Curing Method

Once the form has been struck-off to the appropriate elevation, the precast panel's exposed back surface will be floated to remove any remaining high or low surfaces. Before concluding the final surface preparations, a final review of the location, alignment and condition of the attachment devices shall occur. As required, minor adjustments may be necessary to ensure the proper attachment/embed alignment is maintained.

Upon concluding the floating/troweling operation, each panel shall be etched on the exposed concrete surface with the panel name, date, batch/lot number and job number to ensure proper tracking of the product.

Following the panel marking, each precast panel form and all exposed concrete surfaces shall either be covered with a moist cure blanket within an appropriate amount of time or the exposed surface can be treated with an approved membrane curing compound.

The cast products will remain in the precast form for a minimum of 12 hours. After such time, the form will be disassembled and the precast product will be lifted from the form and inspected for any voids or defects. If voids or imperfections are found, the product will be designated for immediate repair and relocated to an appropriate area within the short-term cure area to receive the necessary attention. Before relocating the precast product, if a membrane curing compound is to be used, the product shall be immediately sprayed with the approved membrane curing compound following the manufacturer's recommendations and then relocated to a more permit storage location.

Product requiring minimal repairs/patching will be pointed with 1-part sand, 1-part cement paste and as necessary to match product color some portion of white portland cement and following the Owner's concrete specification. After the panel satisfies the quality inspection, including name verification, tolerance check, and quality verification, the product will either have the recently repaired area retreated with a membrane curing compound or the product will be transported to a temporary storage location to undergo further controlled curing.

If a membrane curing compound has not been used, the final control curing process will commence in a short-term storage location, once the previous day's products have been removed from their forms and stacked in a safe and expeditious manner. Once the days production has been re-inventoried, all of the recently cast products will be re-covered in a continuously moist environment for the remaining 72 hours.

During the 72 hours cure process, the panels/products are sacked/rubbed as necessary and re-checked to confirm all products meet or exceed the established quality standards.

At the conclusion of the 72 hours, the stacked panels and/or products will be transported to long-term storage where they will remain until the products are shipped to their respective job location.

## 7. Test Methods and Procedures

STEP 7. It is the sole intent of EarthTec to utilize on-site/in-house testing to conduct all required methods specified the Owner's concrete specification which includes oversight, sampling, field and lab testing and all necessary reporting. In situations where the in-house resources are unavailable to perform the specified requirements, an outside Owner approved testing laboratory and personnel shall conduct and oversee all required tests and methods and will generate the required reporting.

STEP 8. Prior to placement of an initial day's concrete a plastic properties test shall be conducted to determine the slump, air content, and temperature and will be executed at or near as possible to the point of placement to assure the concrete meets the Owner's concrete specifications.

STEP 9. All sampling will be obtained at the discharge destination point or at the end of the chute.

STEP 10. The sampling and test methods shall be in accordance with the methods as outlined in the Owner's concrete specification.

STEP 11. Compressive strength cylinders shall be 4" x 8" or 6" x 12" with the frequency of sampling shall not exceed 100 cubic yard batch increments and shall constitute a LOT. Each LOT shall include a minimum of 8 cylinders with an anticipated break frequency as follows: 2 at 7 days, 3 at 28 days and a minimum of 2 cylinders remaining in reserve.

STEP 12. All documentation relating to the test results, sampling and quality control data will be maintained and available for review at the manufacturing location with the required test results being forwarded directly to Owner's designated representative for approval.

STEP 13. If the approved mix design proves to provide a product that is outside of the allowable tolerances outlined in the contract documents and the Owner's concrete specification, the production shall be suspended and corrective actions shall be initiated. These actions shall include and be limited to a review and/or change in mix design. Prior to recommencing with material production, the Owner's designated representative shall approve all revisions relevant to any procedural changes and/or concrete mix designs.

## 8. Steel Sampling and Storage

All non-galvanized steel (black steel) items that are used as embeds and/or product reinforcement will be stored of the ground. A representative sample for every 80 tons of mild reinforcement received at the manufacturing facility shall be made available for independent testing to confirm the validity of the mill certifications provided with each steel material shipment.

## 9. Quality Control

As stated in the above procedures, a precise quality control process shall be used to insure the greatest possible consistency of quality that meets or exceeds that as required by the contract documents, specifications, and shop drawings. Listed below is a step-by-step process that shall constitute the minimum quality control program. This process shall be verified and/or monitored at key points during each day's manufacturing process.

### 9.1. Manufacturing Procedure / Production Protocol

STEP 1. Each day's operation will start with the removal of all curing blankets to prepare for the stripping of product from the forms.

STEP 2. Remove the panel anchor holders and all form recesses.

STEP 3. Unbolt all side rails from the pallet to allow the precast product to be unconfined and readied for removal.

STEP 4. Remove product form using appropriate lifting device.

STEP 5. After each product is removed from the forms, and before they are relocated to the 72 hour cure area, the designated quality control representative shall inspect each product to verify the they were manufactured in accordance with the project specifications and are free from chips, spalls, cracks, honeycomb, or any other defects that would be cause for rejection or repairs.

STEP 6. If the quality of the product is acceptable and in compliance with the referenced detail, the individual product will be marked with green paint, or other suitable marking, along the right side edge of the precast panel and on the end for all top of wall treatments that signifies acceptance. If the product is of acceptable quality but requires minor repairs (i.e. minor patching, cleaning of paste from embeds, etc.) the product will be marked with yellow paint, or other suitable marking, in the same designated locations. In addition the product shall be tagged describing the specific repair that is required.

STEP 7. Both green and yellow marked products will be relocated to the 72 hour curing area. All yellow marked products shall be immediately repaired as required.

STEP 8. The yellow marked products will be repaired under the QC Manager's direct guidance or the QC Control representative. Once the repairs have been completed the products shall be inspected to insure that the product meets or exceeds the quality requirements. After the product has been repaired and the repair is accepted, it shall be marked with green paint adjacent to the yellow paint to signify acceptable quality. The paper tag describing the required repair will be removed and filed.

STEP 9. If the quality of the product is unacceptable or it is deemed to be not repairable, the product is to be immediately designated with a red mark along all 4 sides of the precast product. This rejected product will be removed from the 72 hour curing area and located to the Culled Panel area.

STEP 10. Once each of the products have been removed from their respective forms, all loose debris and foreign substance shall be removed and cleaned from the forming surfaces to facilitate the reconstruction of each form in preparation for the day's production.

STEP 11. Once the form(s) have been re-assembled, the forming surface shall be treated with form release agent with the use of hand sprayer or equivalent method.

STEP 12. Prior to the form setup for that day's production, each form will have within its immediate proximity a form detail/drawing that will include a duplication of a specific panel detail as included within the approved shop drawings. This detail will serve as a representation as to the specific panel that is to be manufactured in the respective form. In addition to the detail, a sheet containing a checklist of items to insure quality control shall be placed.

STEP 13. Each form shall be prepared in accordance with the specific detail with special attention addressed to the product dimensions, header locations, panel anchor locations, number of embeds, the embed locations, and the embed orientation.

STEP 14. Once the form preparation has been successfully completed, the quality control representative shall walk the form line and individually inspect each form while comparing the form setup with the specific detail that is still located with that individual form. The QC representative shall verify that the checklist items contained on the form setup drawing and as specified as Pre-Pour Quality Checklist has been properly addressed.

STEP 15. If the specific form being reviewed is acceptable, the form will be labeled with a green acceptance flag, or other suitable marking, to signify that the form is approved for pouring. If during the review the panel form setup is found unacceptable, the form shall be immediately flagged with a red flag, or other suitable marking, and the unacceptable area noted on the form drawing. The production foreman shall be notified immediately, and the form setup shall be immediately corrected, or it shall be removed from that day's production. Once it is corrected the red flag shall be replaced with a green flag.

STEP 16. In final preparation prior to placing concrete in the forms, a final cursory walk through will be conducted to insure all forms have been flagged green. If water and/or foreign debris has managed to accumulate on the flat exposed form surface, the form shall be blown free of foreign matters via use of portable air.

STEP 17. After concrete has been placed in each panel form, it has been screed and finish floated and the panel has achieved the initial set a final panel review shall be made to confirm that all quality issues have been addressed (i.e. proper back face finish, clevis embed alignment, etc.). Any minor imperfection shall be immediately fixed. If the panel is deemed of acceptable quality it shall have the panel name, date, batch/lot number and job number etched in the back face.

STEP 18. Immediately following the panel etching, a post pour review shall be conducted by the Quality Control representative to approve the panel for final acceptance. Once the product has been accepted the form shall be covered with continuous overlapping moist cure blankets for a minimum period of 12 hours.

STEP 19. The production area shall be cleaned.

STEP 20. Each production day will then recommence with the same repeated process as stated above.

### 9.2. Pre-Pour / Post Pour Inspection Summary:

STEP 1. Prior to the actual placement of concrete, a final form setup inspection shall be initiated by the quality control representative to assure the form size, setup, and all special requirements are as illustrated in the production drawing.

STEP 2. After the final surface preparations, each panel shall be etched on the exposed concrete surface with the panel name, date, batch/lot number and job number.

STEP 3. Immediately following the panel etching a post-pour inspection shall be conducted by the Quality Control representative to insure proper alignment and condition of the attachment devices, the panel etching, header location, and the product dimensions corresponds with the illustrated production on the detail sheet. If a problem is encounter, and can be remedied, immediate steps will be initiated to correct the problem. All corrections shall be performed under direct supervision of the quality control representative. If the problem can't be corrected, the product in question will be immediately rejected and marked in with red paint.

## 10. Manufactured Product Storage:

### 10.1. General Precast Panel Requirements

All precast panels shall be stored in a safe and accessible manner.

#### 10.1.1. Bottom Panel Storage

Under no circumstance shall any precast panel be stored directly in contact with the ground. Depending upon the panel type, 4 x 4 timber dunnage, or a pallet shall be used between the ground and any precast product. The surface of the dunnage or the pallet shall be coated with plastic to prevent staining of the panel face.

#### 10.1.2. Maximum Stack Height

No single stack of panels shall exceed 10 panels in height. Precast panels shall be stored in a manner to insure a safe and stable stack.

#### 10.1.3. Dunnage

A minimum of two (2) pieces of 4 x 4 dunnage shall be used between the bottom panel and the ground. Each piece of dunnage shall have either preco pads or styrofoam attached to each piece of dunnage on the side that will be in contact with the exposed precast face to prevent or minimize any panel deformations or face scaring.

#### 10.1.4. Intermediate Panel Storage

All intermediate panels positioned above the bottom panel require either 2 continuous pieces of 4 x 4 dunnage or 4 pieces of 4 x 4 blocking placed at quarter points to insure the panel's stability. The dunnage shall be positioned in a way to assist the stack stability. Placement of the dunnage shall insure that they are tall enough so the panel does not come in contact panel embeds of other items extending from the panel face. Further, steps shall be made to prevent scaring or staining at the end of the dunnage that is in contact with the panel's front face.

#### 10.1.5. Stack Configuration

Place the largest precast panels on the lower portion of each respective panel stack with the smaller partial pieces being positioned higher in the stack. The exception to this procedure would be the strategic combining of panels in order to create a full size panel made up of smaller precast panels. Larger pieces of 4 x 4 dunnage or additional dunnage may be required to insure the stack's stability.

### 10.2. Other Precast Element Storage

When storing precast top of wall treatments (i.e. precast traffic barrier, precast coping and precast parapet), 2 pieces of continuous 4 x 4 shall be positioned (1 at each end) across the product length at the approximate quarter point from each end of the product.

## 11. Product Repair Classification

### 11.1. 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.

### 11.2. Honeycombing

A series of voids in the concrete that may be caused by the loss of fines or other material between the aggregate particles, the inclusion of air pockets between aggregate particles, or larger volumes of lost material.

### 11.3. Spall

A depression in the panel that is a result of a fragment of concrete being detached from the larger mass of concrete and can be caused by impact, the action of weather, uneven pressure, or uncontrolled expansion.

#### 11.3.1. Cosmetic

A circular or oval depression not greater than 1.0 inch in depth no greater than 3.0 square inches in area

#### 11.3.2. Minor

A spall no larger than 1.0 square foot and no deeper than 1.5 inches.

### 11.4. Chip

The local breaking of corners or edges of the concrete that results in a void containing angular surfaces.

#### 11.4.1. Cosmetic

Cosmetic chips are chips where the sum of the two lateral dimensions perpendicular to the length does not exceed 2.0 inches.

#### 11.4.2. Minor

Chips are where the sum of the two lateral dimensions perpendicular to the length exceeds two inches, but does not exceed four inches, and with a length of no more than 12 inches.

### 11.5. Major Concrete Deficiencies

In an effort not to supply any product of a compromising structural nature 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 shall deem that the product is rejected and not eligible to be repaired.

## 12. Product Repair Methods

The Quality Control Manager will examine all deficiencies and will determine the specific nature of the repairs and the most appropriate course of action required to correct the deficiency. The correction can range from minor cleaning of connections/embeds up to, and including, minor concrete deficiencies. All minor deficiencies shall be listed and described on a "Minor Repair Record" sheet. Furthermore, all concrete deficiencies shall be classified as, non-repairable (major), or repairable (cosmetic or minor).

All repairs will be conducted under direct supervision of the Quality Control manager in a manner to insure appropriate strength and quality. All repairs shall be made in a manner that is acceptable to the engineer.

### 12.1. Major / Non-repairable

All product containing deficiencies exceeding cosmetic or minor definitions described above, shall be deemed un-repairable and shall be physically marked by red paint or grease pencil along all 4 sides and shall be relocated to the rejected/culled area of the manufacturing facility until the rejected product can be relocated off-site to the disposal facility.

### 12.2. Cosmetic / Minor

All minor cosmetic repairs shall be repaired by either pointing the product with 1 part sand and 1 part cement, which would typically be accomplished while the product is or about to be placed into the 72 hour cure area, or at the discretion of the Q.C. manager with a specifically approved patching product contained on the Owner's Qualified Product List (QPL). The approved repair products to be considered for use include the following products: Lambert Epiweld 560 / 580 epoxy bonding agent, Euclid Euco-Speed MP, Lambert Vibropruf #11, SikaQuick 1000, Bonsal Fast Set Cement, 1 part sand and 1 part cement paste and as necessary white Portland cement. All repairs shall match the product color and shall insure proper blending. The product shall be prepared and applied in accordance with the manufacture recommendations. The actual concrete repair procedure shall include; proper surface preparation, the application of an epoxy bonding compound to the affected area, proper preparation, and use of patching material and final shaping/texturing and grinding to insure the proper product blending.

## 13. Handling of Failed or Rejected Products:

### 13.1. Rejected Product

All manufactured precast products that have been deemed rejected during the manufacturing, curing, or storage process shall be immediately physically marked by red paint or grease pencil and immediately relocated to the rejected/culled area of the manufacturing facility until the rejected product can be relocated to a disposal facility off-site.

### 13.2. Rejected Component

Prior to the acceptance of any raw matter or material used in the manufacturing process, each and every product shall be physically checked, tested and/or mill certifications confirmed as acceptable for use. If a specific item is found unacceptable, or is found not to meet the project requirements, the item in question will be immediately refused for unloading and be sent back to the supplier for disposal.

### 13.3. Rejected Product at Job-Site

If product that has been unloaded and that is determined to be unacceptable for use, it shall be immediately marked rejected with red paint and relocated to the rejected/ cull area of the facility until the material can be permanently removed from the manufacturing site.

### 13.4. Rejected at Cure site

During the product curing process, if the products are found to have insufficient material strength, all the materials produced during that time and containing the relevant LOT number will be immediately marked rejected and relocated to the facility's rejected/culled area until permanent removal can occur.

## 1.1.4
## Facing Unit Dimensions

EARTHTEC



GROUND IMPROVEMENT SYSTEMS LLC
114 SOUTH COLLINS STREET
ARLINGTON, TX 76011

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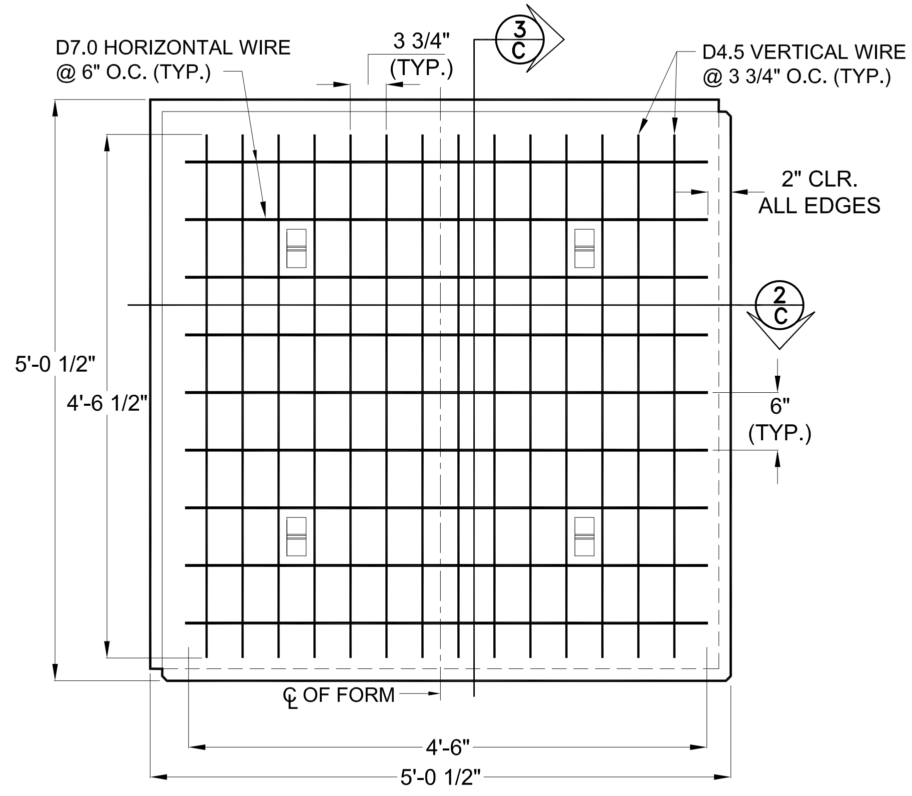
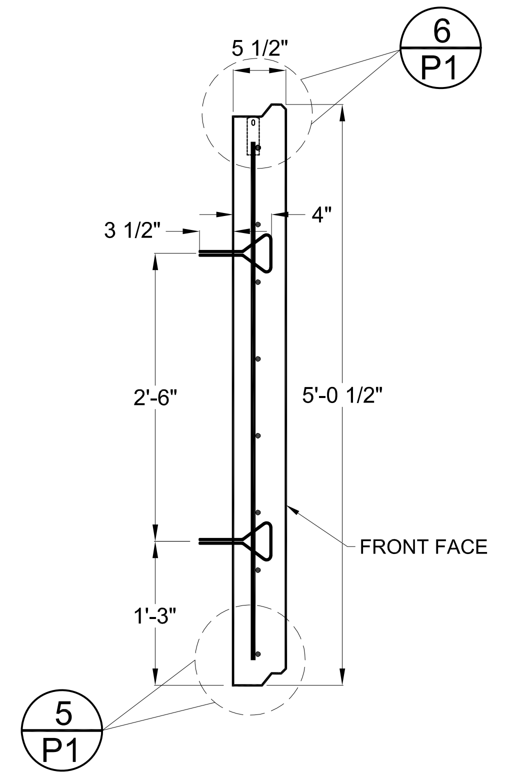
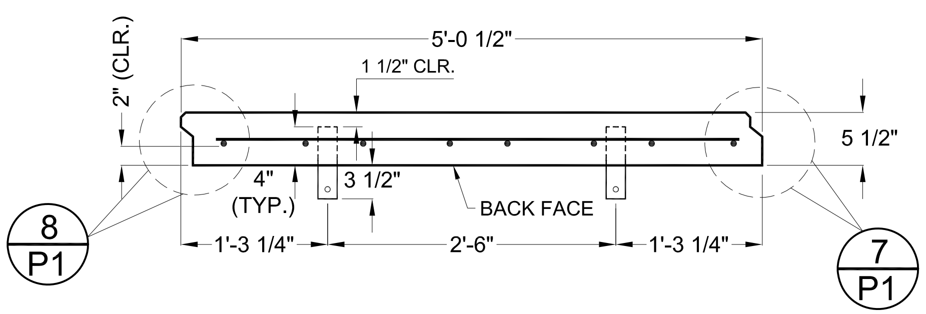


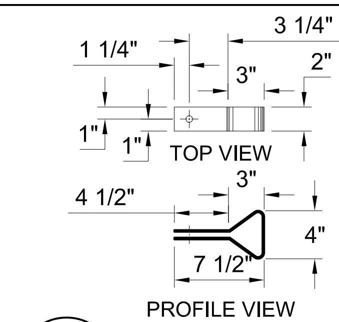

1 P1 STANDARD SQUARE PANEL
BACK FACE



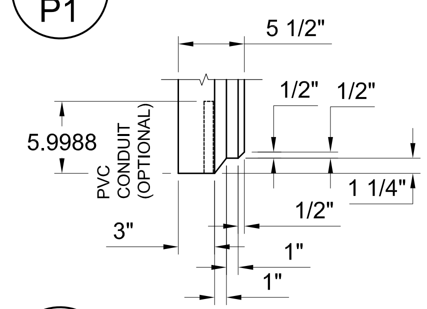

3 P1 STANDARD SQUARE PANEL
SECTION VIEW



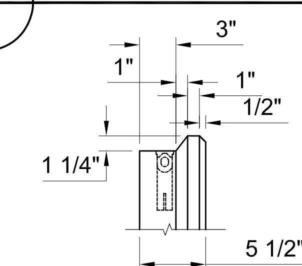

2 P1 STANDARD SQUARE PANEL
TOP VIEW



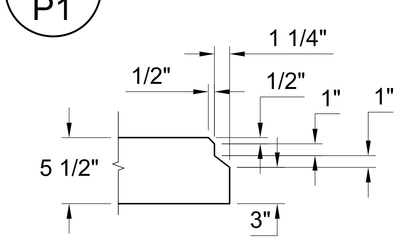

4 P1 PANEL ANCHOR



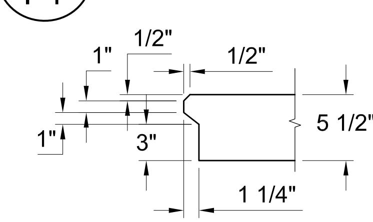

5 P1 PANEL BOTTOM




6 P1 PANEL TOP




7 P1 PANEL RIGHT SIDE




8 P1 PANEL LEFT SIDE

GENERAL NOTES

1. ALL DIMENSIONS ARE IN FEET AND INCHES (IMPERIAL UNITS) UNLESS NOTED OTHERWISE.
2. ALL PANELS ARE SHOWN BACK FACE.
3. ALL DIMENSIONS ARE SHOWN MEASURED FROM BACK FACE / BACK EDGE OF PRECAST PANEL FORM, TYPICAL.
4. ALL BLACK STEEL REINFORCEMENT AND EMBEDS SHALL HAVE A MINIMUM 1 ½" CONCRETE COVER
5. CONCRETE SHALL HAVE A MINIMUM 28-DAY COMPRESSIVE STRENGTH EQUAL TO 4000 PSI (UNLESS NOTED OTHERWISE) AND SHALL BE CAST IN ACCORDANCE WITH THE PROJECT SPECIFICATIONS.
6. CONCRETE SAMPLING, TESTING, CERTIFICATIONS AND PANEL PRODUCTION (INCLUDING BUT NOT LIMITED TO CASTING, CURING, LABELING, FINISHING, HANDLING, STORAGE AND REPORTING) SHALL BE CONDUCTED IN ACCORDANCE WITH THE GOVERNING PROJECT SPECIFICATION
7. EACH PANEL SHALL HAVE THE PANEL NAME, DATE OF MANUFACTURE AND LOT NUMBER SCRIBE IN THE BACK FACE OF PANEL.
8. THE FRONT FACE EDGE OF ALL PANELS SHALL HAVE A ½" X ½" CHAMFER (UNLESS NOTED OTHERWISE).
9. PANEL REINFORCING SHALL BE WELDED WIRE FABRIC CONFORMING TO ASTM A1064 STANDARD SPECIFICATION FOR STEEL WIRE AND WELDED WIRE REINFORCEMENT, PLAIN AND DEFORMED, FOR CONCRETE WITH A MINIMUM YIELD STRENGTH EQUAL TO 65 KSI.
10. THE GALVANIZED ANCHOR EMBEDS SHALL NOT BE IN CONTACT WITH THE BLACK PANEL REINFORCEMENT.
11. PLACE PANEL ANCHORS AS DETAILED AND SO THEY ARE 90° TO THE BACK FACE OF THE PANEL..
12. PANEL LIFT ANCHOR SHALL BE DAYTON SUPERIOR P-93 FLEET-LIFT L-ANCHOR (PRODUCT CODE FL050, HOT-DIPPED GALVANIZED) OR EQUIVALENT.
13. ALL PANELS ARE TO BE MANUFACTURED TO THE DIMENSION SHOWN IN THE SHOP DRAWINGS UNLESS NOTED OTHERWISE. PANEL SQUARENESS AND TOLERANCES OUTLINED IN THE EARTHTEC PRECAST PANEL QUALITY CONTROL SPECIFICATION AND WITHIN THE GOVERNING SPECIFICATIONS REQUIREMENTS.
14. PANELS SHALL NOT BE REMOVE FROM FORM UNTIL A MINIMUM 1500 PSI STRENGTH BEEN ACHIEVED.
15. PANELS SHALL BE REMOVED FROM THE FORM USING A MINIMUM OF 4 POINTS THAT ARE EVENLY AND SYMMETRICALLY DISTRIBUTED ON THE BACK FACE OF THE PANEL.

| REV | DESCRIPTION | BY | DATE |
|---|---|---|---|

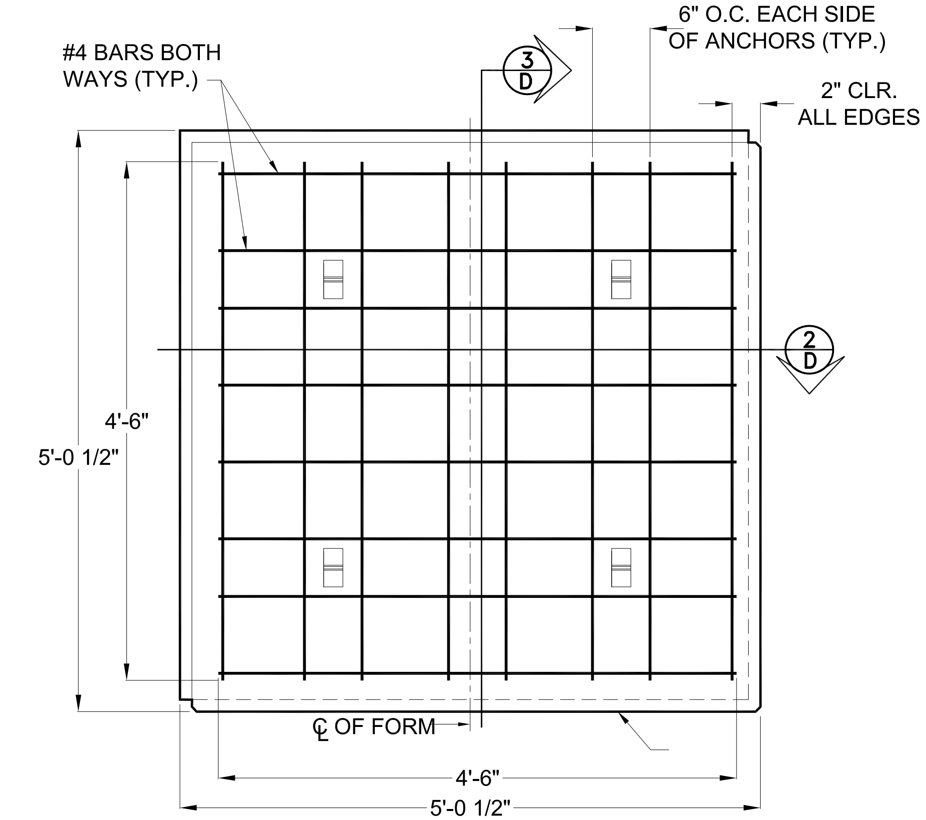
EARTHTEC
GIS Support Through Innovation

PRECAST PANEL DETAIL
5' WIDE PANEL

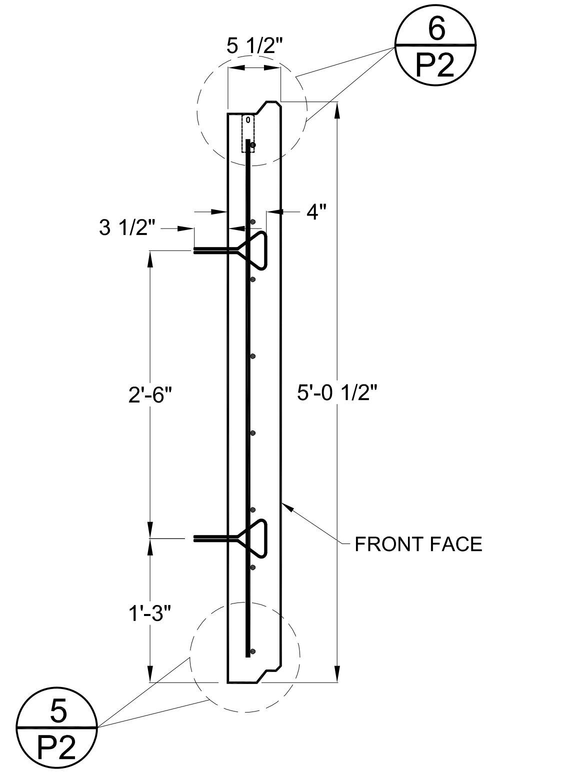
GROUND IMPROVEMENT SYSTEMS LLC
114 SOUTH COLLINS STREET
ARLINGTON, TX 76011
(817) 223-0969
www.groundimprovementsystems.com

GIS-2020
DESIGNED BY TPT
DRAWN BY TPT
CHECKED BY XXX

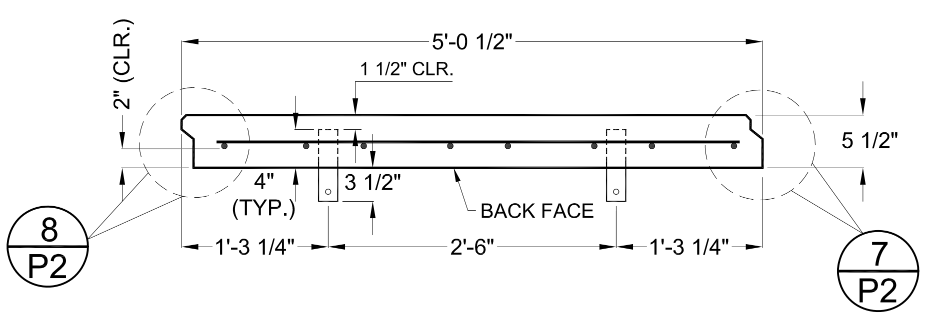
SHT: P1



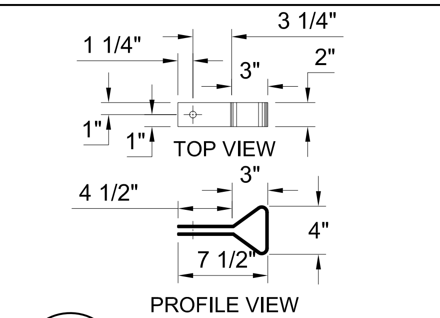

1 P2 STANDARD SQUARE PANEL
BACK FACE



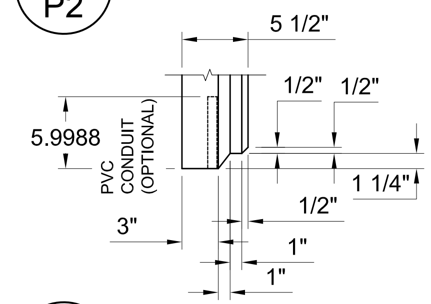

3 P2 STANDARD SQUARE PANEL
SECTION VIEW



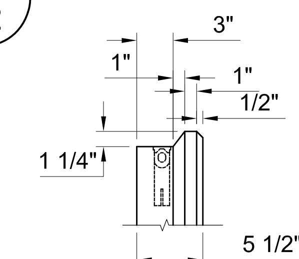

2 P2 STANDARD SQUARE PANEL
TOP VIEW



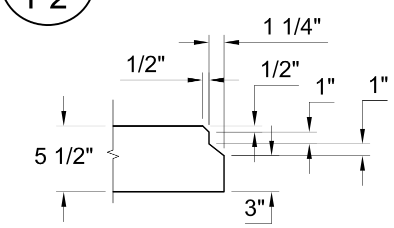

4 P2 PANEL ANCHOR



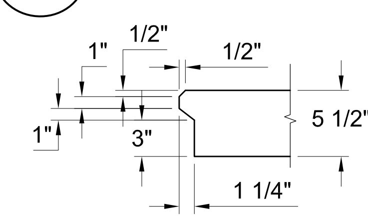

5 P2 PANEL BOTTOM




6 P2 PANEL TOP




7 P2 PANEL RIGHT SIDE




8 P2 PANEL LEFT SIDE

GENERAL NOTES

1. ALL DIMENSIONS ARE IN FEET AND INCHES (IMPERIAL UNITS) UNLESS NOTED OTHERWISE.
2. ALL PANELS ARE SHOWN BACK FACE.
3. ALL DIMENSIONS ARE SHOWN MEASURED FROM BACK FACE / BACK EDGE OF PRECAST PANEL FORM, TYPICAL.
4. ALL BLACK STEEL REINFORCEMENT AND EMBEDS SHALL HAVE A MINIMUM OF 1 ½" CONCRETE COVER
5. CONCRETE SHALL HAVE A MINIMUM 28-DAY COMPRESSIVE STRENGTH EQUAL TO 4,000 PSI (UNLESS NOTED OTHERWISE) AND SHALL BE IN CAST ACCORDANCE WITH THE PROJECT SPECIFICATIONS.
6. CONCRETE SAMPLING, TESTING, CERTIFICATIONS AND PANEL PRODUCTION (INCLUDING BUT NOT LIMITED TO CASTING, CURING, LABELING, FINISHING, HANDLING, STORAGE AND REPORTING) SHALL BE CONDUCTED IN ACCORDANCE WITH THE GOVERNING PROJECT SPECIFICATION.
7. ALL PANELS SHALL HAVE THE PANEL NAME, DATE OF MANUFACTURE AND LOT NUMBER SCRIBE IN THE BACK FACE OF PANEL.
8. THE FRONT FACE EDGE OF ALL PANELS SHALL HAVE A ½" X ½" CHAMFER (UNLESS NOTED OTHERWISE).
9. PANEL REINFORCING SHALL BE A MINIMUM #4 BAR (BOTH WAYS) CONFORMING TO ASTM A615 MILD REINFORCING (REBAR) GRADE 60 OR EQUIVALENT.
10. THE GALVANIZED ANCHOR EMBEDS SHALL NOT BE IN CONTACT WITH THE BLACK PANEL REINFORCEMENT.
11. ALL ANCHORS ARE TO BE POSITIONED AS DETAILED AND POSITION PERPENDICULAR (90 DEGREES) WITH RESPECT TO THE BACK FACE OF PANEL.
12. PANEL LIFT ANCHOR SHALL BE DAYTON SUPERIOR P-93 FLEET-LIFT L-ANCHOR (PRODUCT CODE FL050, HOT-DIPPED GALVANIZED) OR EQUIVALENT.
13. ALL PANELS ARE TO BE MANUFACTURED TO THE DIMENSION SHOWN IN THE SHOP DRAWINGS UNLESS NOTED OTHERWISE. PANEL SQUARENESS AND TOLERANCES OUTLINED IN THE EARTHTEC PRECAST PANEL QUALITY CONTROL SPECIFICATION AND WITHIN THE GOVERNING SPECIFICATIONS REQUIREMENTS.
14. PANELS SHALL NOT BE REMOVED FROM THE FORM UNTIL A MINIMUM 1500 PSI STRENGTH HAS BEEN ACHIEVED.
15. PANELS SHALL BE REMOVED FROM THE FORM USING A MINIMUM OF 4 POINTS THAT ARE EVENLY AND SYMMETRICALLY DISTRIBUTED ON THE BACK FACE OF THE PANEL.



| REV | DESCRIPTION | BY | DATE |
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EARTHTEC
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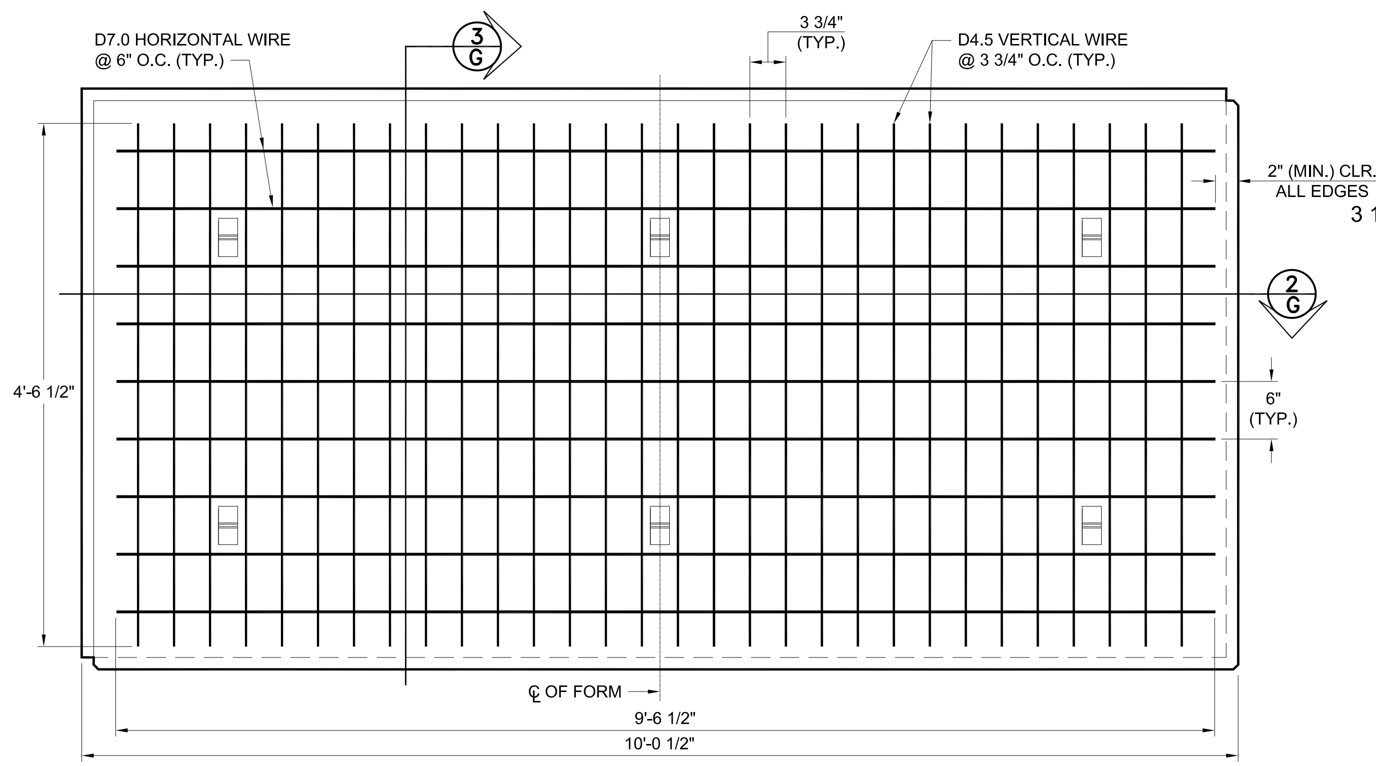
PRECAST PANEL DETAIL
5' WIDE PANEL

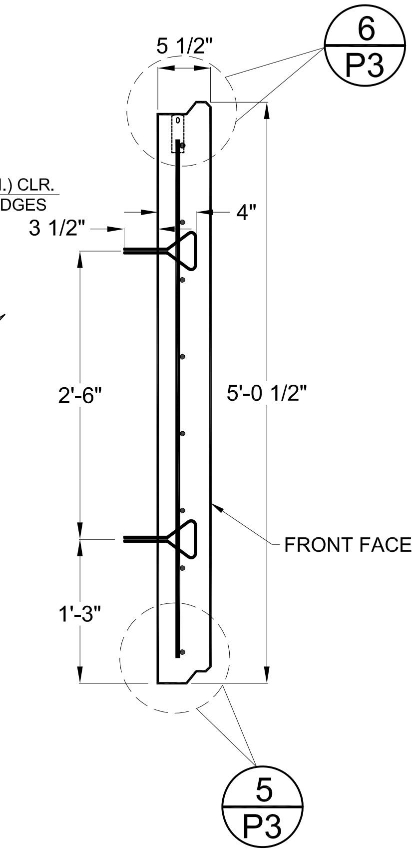
GROUND IMPROVEMENT SYSTEMS LLC
114 SOUTH COLLINS STREET
ARLINGTON, TX 76011
(817) 223-0969
www.groundimprovementsystems.com

GIS-2020
DESIGNED BY TPT
DRAWN BY TPT
CHECKED BY XXX

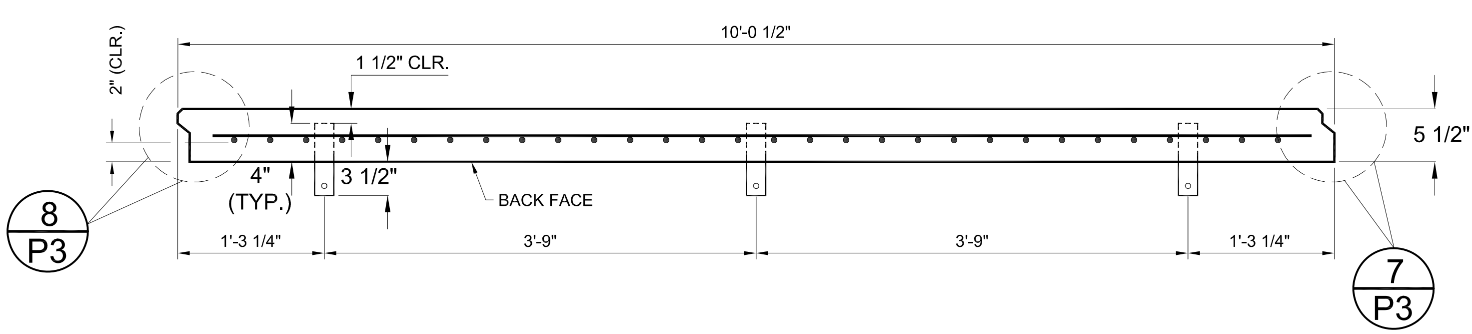
SHT: P2



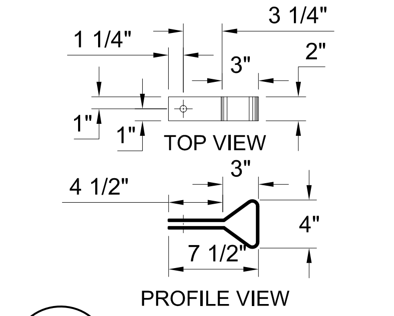

1 P3 STANDARD RECTANGULAR PANEL
BACK FACE



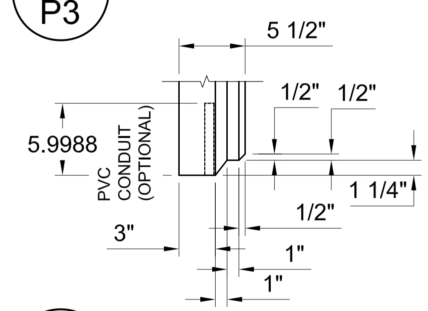

3 P3 STANDARD RECTANGULAR PANEL
SECTION VIEW



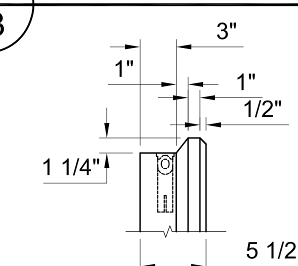

2 P3 STANDARD RECTANGULAR PANEL
TOP VIEW



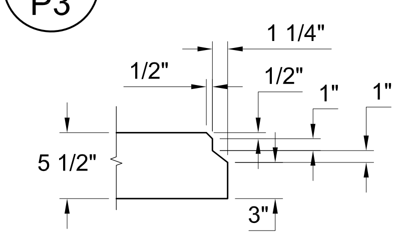

4 P3 PANEL ANCHOR



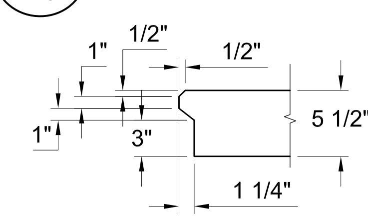

5 P3 PANEL BOTTOM




6 P3 PANEL TOP




7 P3 PANEL RIGHT SIDE




8 P3 PANEL LEFT SIDE

GENERAL NOTES

1. ALL DIMENSIONS ARE IN FEET AND INCHES (IMPERIAL UNITS) UNLESS NOTED OTHERWISE.
2. ALL PANELS ARE SHOWN BACK FACE.
3. ALL DIMENSIONS ARE SHOWN MEASURED FROM BACK FACE / BACK EDGE OF PRECAST PANEL FORM, TYPICAL.
4. ALL BLACK STEEL REINFORCEMENT AND EMBEDS SHALL HAVE A MINIMUM 1 ½" CONCRETE COVER
5. CONCRETE SHALL HAVE A MINIMUM 28-DAY COMPRESSIVE STRENGTH EQUAL TO 4000 PSI (UNLESS NOTED OTHERWISE) AND SHALL BE CAST IN ACCORDANCE WITH THE PROJECT SPECIFICATIONS.
6. CONCRETE SAMPLING, TESTING, CERTIFICATIONS AND PANEL PRODUCTION (INCLUDING BUT NOT LIMITED TO CASTING, CURING, LABELING, FINISHING, HANDLING, STORAGE AND REPORTING) SHALL BE CONDUCTED IN ACCORDANCE WITH THE GOVERNING PROJECT SPECIFICATION
7. EACH PANEL SHALL HAVE THE PANEL NAME, DATE OF MANUFACTURE AND LOT NUMBER SCRIBE IN THE BACK FACE OF PANEL.
8. THE FRONT FACE EDGE OF ALL PANELS SHALL HAVE A ½" X ½" CHAMFER (UNLESS NOTED OTHERWISE).
9. PANEL REINFORCING SHALL BE WELDED WIRE FABRIC CONFORMING TO ASTM A1064 STANDARD SPECIFICATION FOR STEEL WIRE AND WELDED WIRE REINFORCEMENT, PLAIN AND DEFORMED, FOR CONCRETE WITH A MINIMUM YIELD STRENGTH EQUAL TO 65 KSI.
10. THE GALVANIZED ANCHOR EMBEDS SHALL NOT BE IN CONTACT WITH THE BLACK PANEL REINFORCEMENT.
11. PLACE PANEL ANCHORS AS DETAILED AND SO THEY ARE 90° TO THE BACK FACE OF THE PANEL..
12. PANEL LIFT ANCHOR SHALL BE DAYTON SUPERIOR P-93 FLEET-LIFT L-ANCHOR (PRODUCT CODE FL050, HOT-DIPPED GALVANIZED) OR EQUIVALENT.
13. ALL PANELS ARE TO BE MANUFACTURED TO THE DIMENSION SHOWN IN THE SHOP DRAWINGS UNLESS NOTED OTHERWISE. PANEL SQUARENESS AND TOLERANCES OUTLINED IN THE EARTHTEC PRECAST PANEL QUALITY CONTROL SPECIFICATION AND WITHIN THE GOVERNING SPECIFICATIONS REQUIREMENTS.
14. PANELS SHALL NOT BE REMOVE FROM FORM UNTIL A MINIMUM 1500 PSI STRENGTH BEEN ACHIEVED.
15. PANELS SHALL BE REMOVED FROM THE FORM USING A MINIMUM OF 4 POINTS THAT ARE EVENLY AND SYMMETRICALLY DISTRIBUTED ON THE BACK FACE OF THE PANEL.



| REV | DESCRIPTION | BY | DATE |
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# PRECAST PANEL DETAIL
10' WIDE PANEL

GROUND IMPROVEMENT SYSTEMS LLC
114 SOUTH COLLINS STREET
ARLINGTON, TX 76011
(817) 223-0969
www.groundimprovementsystems.com

GIS-2020
DESIGNED BY TPT
DRAWN BY TPT
CHECKED BY XXX

SHT: P3

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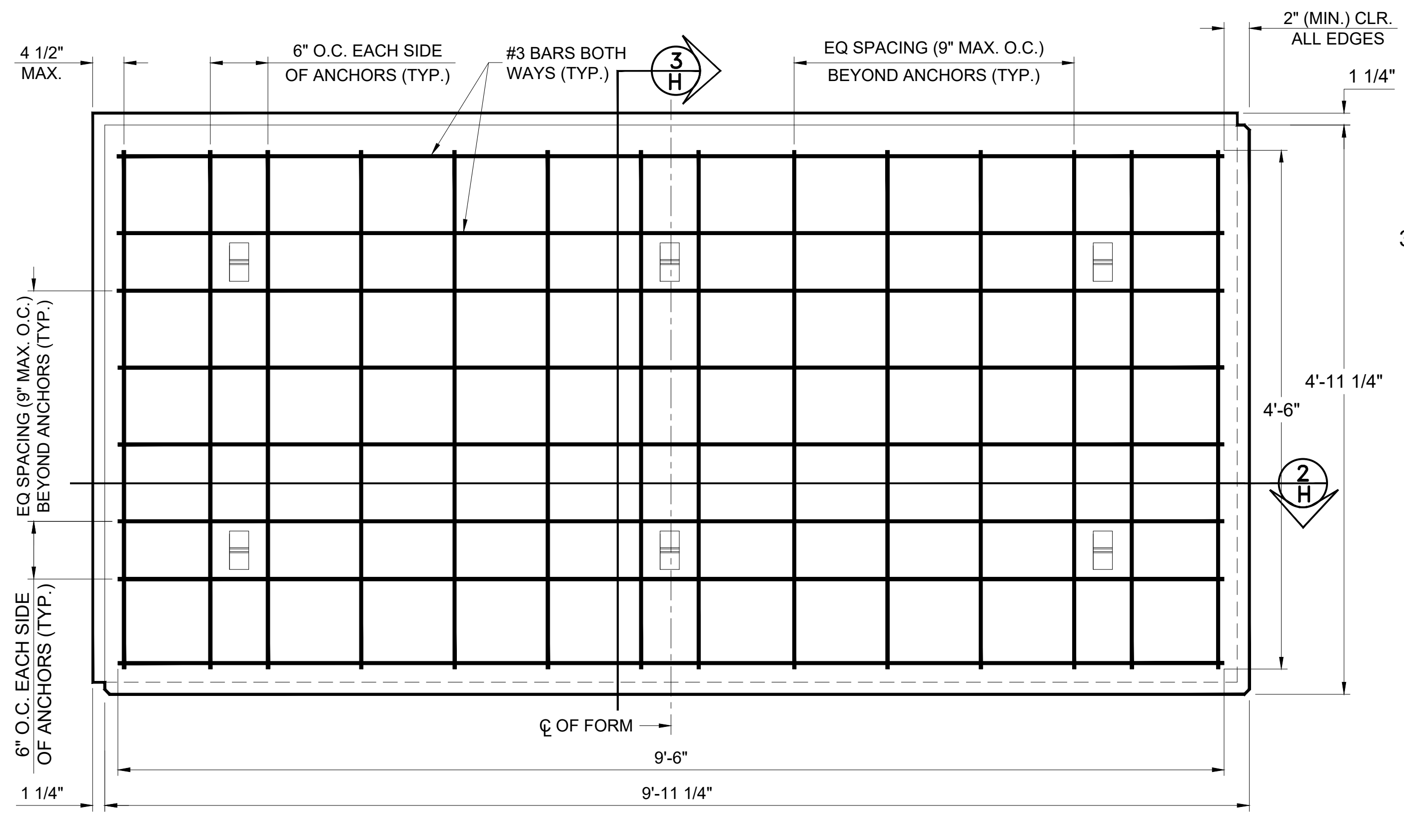
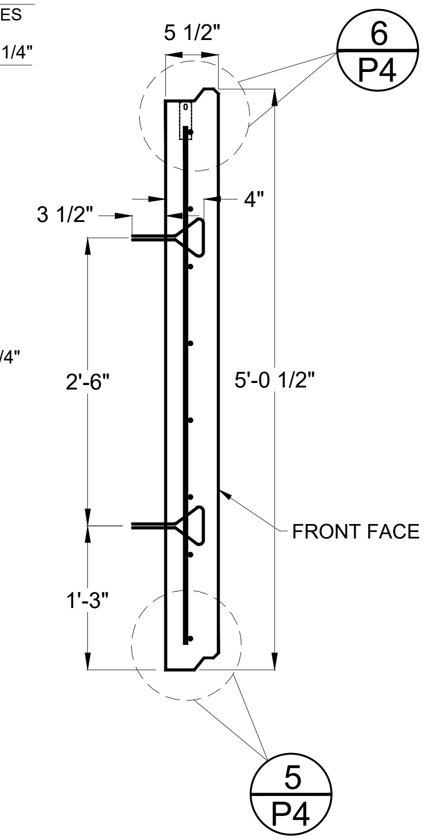
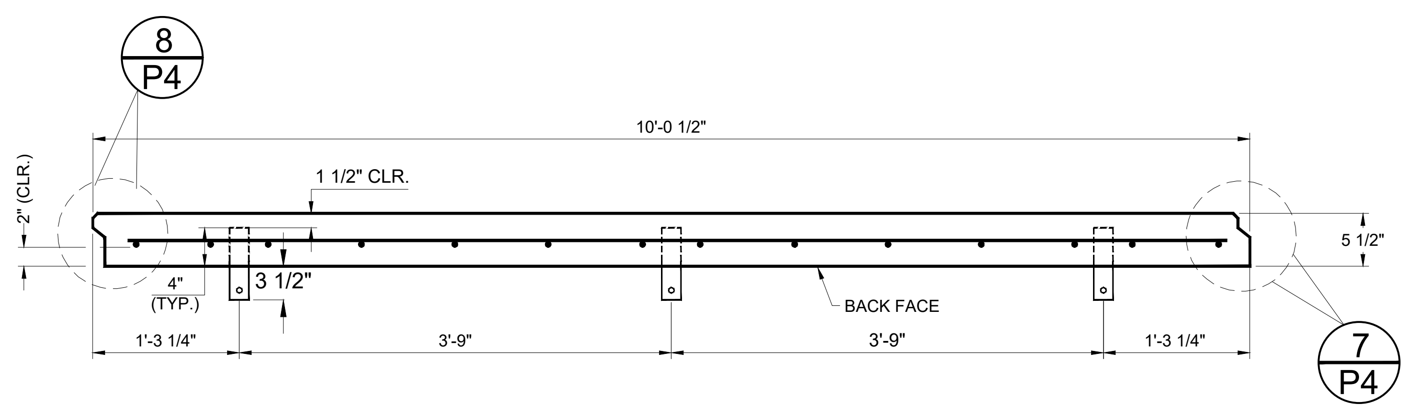


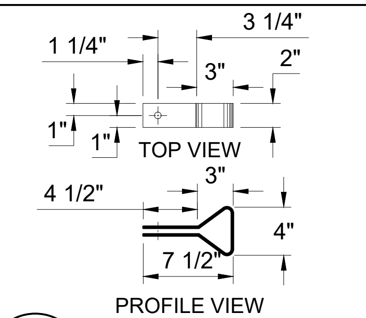

1 / P4 STANDARD RECTANGULAR PANEL
BACK FACE



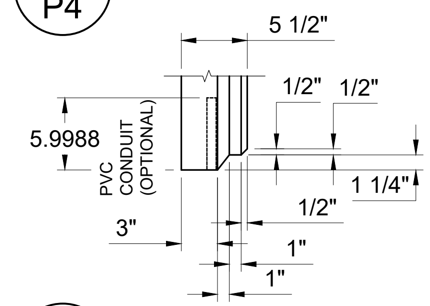

3 / P4 STANDARD RECTANGULAR PANEL
SECTION VIEW



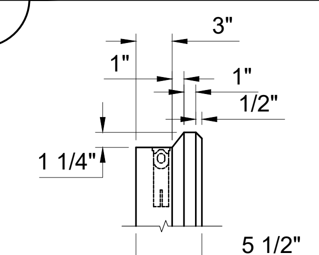

2 / P4 STANDARD RECTANGULAR PANEL
TOP VIEW



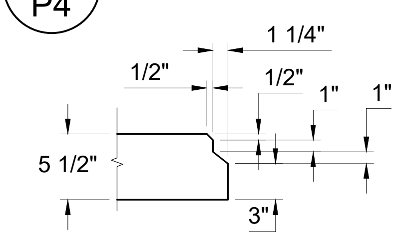

4 / P4 PANEL ANCHOR



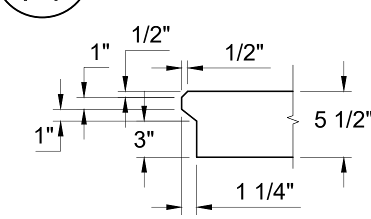

5 / P4 PANEL BOTTOM




6 / P4 PANEL TOP




7 / P4 PANEL RIGHT SIDE




8 / P4 PANEL LEFT SIDE

GENERAL NOTES

1. ALL DIMENSIONS ARE IN FEET AND INCHES (IMPERIAL UNITS) UNLESS NOTED OTHERWISE.
2. ALL PANELS ARE SHOWN BACK FACE.
3. ALL DIMENSIONS ARE SHOWN MEASURED FROM BACK FACE / BACK EDGE OF PRECAST PANEL FORM, TYPICAL.
4. ALL BLACK STEEL REINFORCEMENT AND EMBEDS SHALL HAVE A MINIMUM OF 1 ½" CONCRETE COVER
5. CONCRETE SHALL HAVE A MINIMUM 28-DAY COMPRESSIVE STRENGTH EQUAL TO 4,000 PSI (UNLESS NOTED OTHERWISE) AND SHALL BE IN CAST ACCORDANCE WITH THE PROJECT SPECIFICATIONS.
6. CONCRETE SAMPLING, TESTING, CERTIFICATIONS AND PANEL PRODUCTION (INCLUDING BUT NOT LIMITED TO CASTING, CURING, LABELING, FINISHING, HANDLING, STORAGE AND REPORTING) SHALL BE CONDUCTED IN ACCORDANCE WITH THE GOVERNING PROJECT SPECIFICATION.
7. ALL PANELS SHALL HAVE THE PANEL NAME, DATE OF MANUFACTURE AND LOT NUMBER SCRIBE IN THE BACK FACE OF PANEL.
8. THE FRONT FACE EDGE OF ALL PANELS SHALL HAVE A ½" X ½" CHAMFER (UNLESS NOTED OTHERWISE).
9. PANEL REINFORCING SHALL BE A MINIMUM #4 BAR (BOTH WAYS) CONFORMING TO ASTM A615 A615 MILD REINFORCING (REBAR) GRADE 60 OR EQUIVALENT.
10. THE GALVANIZED ANCHOR EMBEDS SHALL NOT BE IN CONTACT WITH THE BLACK PANEL REINFORCEMENT.
11. ALL ANCHORS ARE TO BE POSITIONED AS DETAILED AND POSITION PERPENDICULAR (90 DEGREES) WITH RESPECT TO THE BACK FACE OF PANEL.
12. PANEL LIFT ANCHOR SHALL BE DAYTON SUPERIOR P-93 FLEET-LIFT L-ANCHOR (PRODUCT CODE FL050, HOT-DIPPED GALVANIZED) OR EQUIVALENT.
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14. PANELS SHALL NOT BE REMOVED FROM THE FORM UNTIL A MINIMUM 1500 PSI STRENGTH HAS BEEN ACHIEVED.
15. PANELS SHALL BE REMOVED FROM THE FORM USING A MINIMUM OF 4 POINTS THAT ARE EVENLY AND SYMMETRICALLY DISTRIBUTED ON THE BACK FACE OF THE PANEL.

*THE DESIGN CONTAINED HEREIN IS BASED ON INFORMATION SUPPLIED BY OTHERS. GIS CERTIFYING INTERNAL STABILITY OF THE STRUCTURE ONLY. EXTERNAL AND GLOBAL STABILITY REQUIREMENTS ARE THE RESPONSIBILITY OF OTHERS.*

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EARTHTEC
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PRECAST PANEL DETAIL
10' WIDE PANEL


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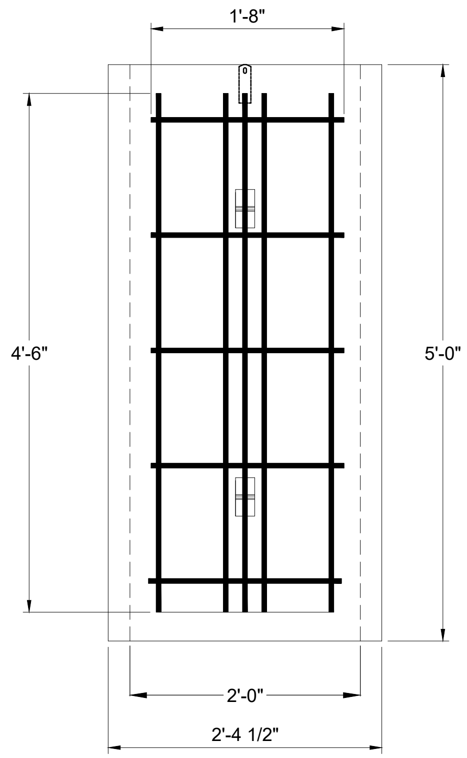
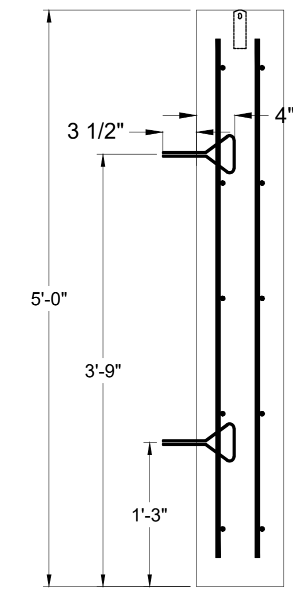
GIS-2020
DESIGNED BY TPT
DRAWN BY TPT
CHECKED BY XXX

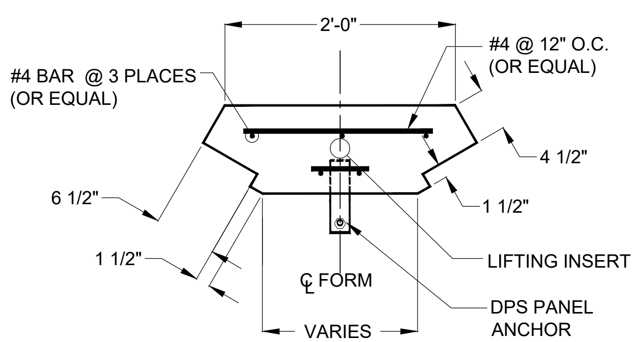
SHT: P4



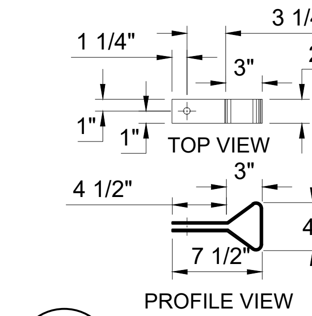

D / P5 VARIABLE CORNER PANEL (TYPE-X)
BACK FACE



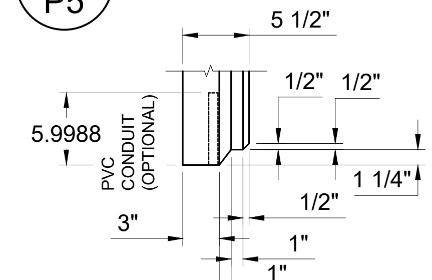

D / P5 VARIABLE CORNER PANEL (TYPE-X)
SECTION VIEW



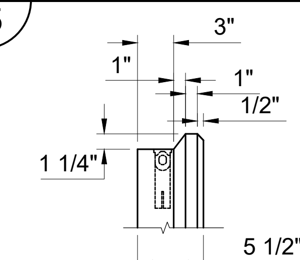

D / P5 VARIABLE CORNER PANEL (TYPE-X)
PLAN VIEW



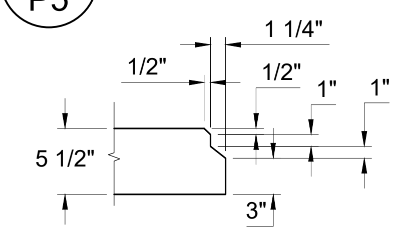

4 / P5 PANEL ANCHOR



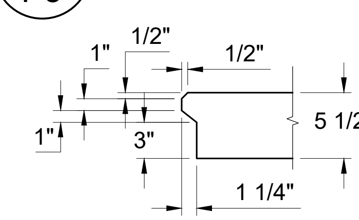

5 / P5 PANEL BOTTOM




6 / P5 PANEL TOP




7 / P5 PANEL RIGHT SIDE




8 / P5 PANEL LEFT SIDE

GENERAL NOTES

1. ALL DIMENSIONS ARE IN FEET AND INCHES (IMPERIAL UNITS) UNLESS NOTED OTHERWISE.
2. ALL PANELS ARE SHOWN BACK FACE.
3. ALL DIMENSIONS ARE SHOWN MEASURED FROM BACK FACE / BACK EDGE OF PRECAST PANEL FORM, TYPICAL.
4. ALL BLACK STEEL REINFORCEMENT AND EMBEDS SHALL HAVE A MINIMUM 1 ½" CONCRETE COVER
5. CONCRETE SHALL HAVE A MINIMUM 28-DAY COMPRESSIVE STRENGTH EQUAL TO 4000 PSI (UNLESS NOTED OTHERWISE) AND SHALL BE CAST IN ACCORDANCE WITH THE PROJECT SPECIFICATIONS.
6. CONCRETE SAMPLING, TESTING, CERTIFICATIONS AND PANEL PRODUCTION (INCLUDING BUT NOT LIMITED TO CASTING, CURING, LABELING, FINISHING, HANDLING, STORAGE AND REPORTING) SHALL BE CONDUCTED IN ACCORDANCE WITH THE GOVERNING PROJECT SPECIFICATION
7. EACH PANEL SHALL HAVE THE PANEL NAME, DATE OF MANUFACTURE AND LOT NUMBER SCRIBE IN THE BACK FACE OF PANEL.
8. THE FRONT FACE EDGE OF ALL PANELS SHALL HAVE A ½" X ½" CHAMFER (UNLESS NOTED OTHERWISE).
9. PANEL REINFORCING SHALL BE WELDED WIRE FABRIC CONFORMING TO ASTM A1064 STANDARD SPECIFICATION FOR STEEL WIRE AND WELDED WIRE REINFORCEMENT, PLAIN AND DEFORMED, FOR CONCRETE WITH A MINIMUM YIELD STRENGTH EQUAL TO 65 KSI.
10. THE GALVANIZED ANCHOR EMBEDS SHALL NOT BE IN CONTACT WITH THE BLACK PANEL REINFORCEMENT.
11. PLACE PANEL ANCHORS AS DETAILED AND SO THEY ARE 90° TO THE BACK FACE OF THE PANEL..
12. PANEL LIFT ANCHOR SHALL BE DAYTON SUPERIOR P-93 FLEET-LIFT L-ANCHOR (PRODUCT CODE FL050, HOT-DIPPED GALVANIZED) OR EQUIVALENT.
13. ALL PANELS ARE TO BE MANUFACTURED TO THE DIMENSION SHOWN IN THE SHOP DRAWINGS UNLESS NOTED OTHERWISE. PANEL SQUARENESS AND TOLERANCES OUTLINED IN THE EARTHTEC PRECAST PANEL QUALITY CONTROL SPECIFICATION AND WITHIN THE GOVERNING SPECIFICATIONS REQUIREMENTS.
14. PANELS SHALL NOT BE REMOVE FROM FORM UNTIL A MINIMUM 1500 PSI STRENGTH BEEN ACHIEVED.
15. PANELS SHALL BE REMOVED FROM THE FORM USING A MINIMUM OF 4 POINTS THAT ARE EVENLY AND SYMMETRICALLY DISTRIBUTED ON THE BACK FACE OF THE PANEL.

THE DESIGN CONTAINED HEREIN IS BASED ON INFORMATION SUPPLIED BY OTHERS.GIS CERTIFYING INTERNAL STABILITY OF THE STRUCTURE ONLY. EXTERNAL AND GLOBAL STABILITY REQUIREMENTS ARE THE RESPONSIBILITY OF OTHERS.

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EARTHTEC
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PRECAST PANEL DETAIL
STANDARD CORNER PANEL

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SHT: P5



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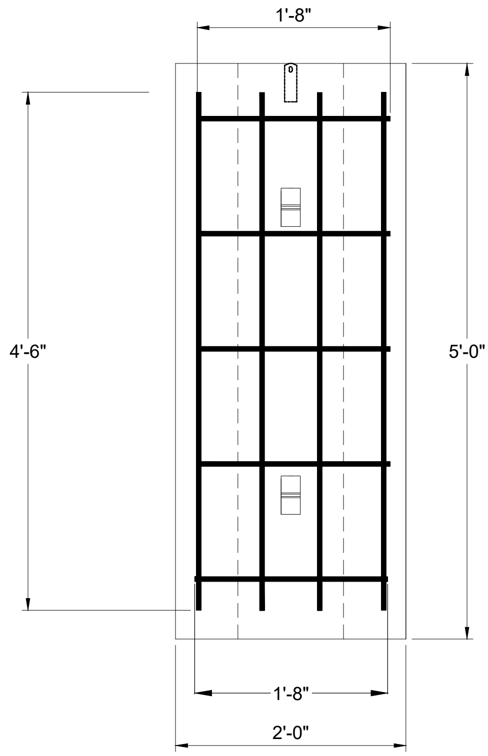
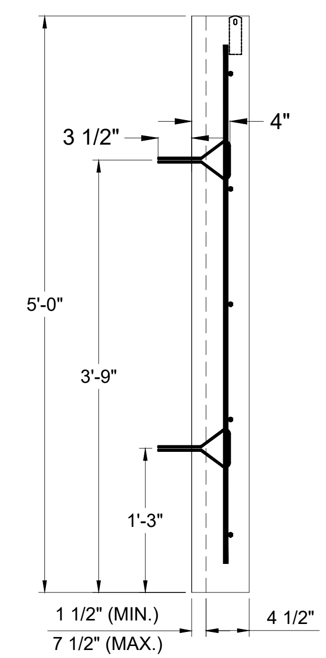


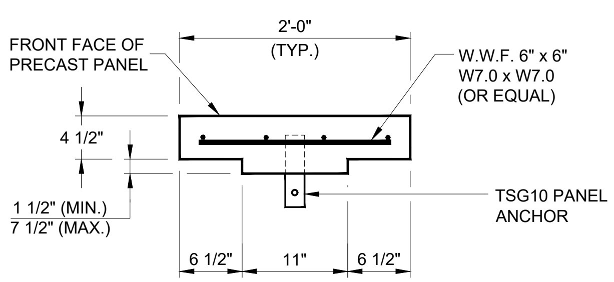

E / P6 SLIP JOINT PANEL DETAIL

BACK FACE



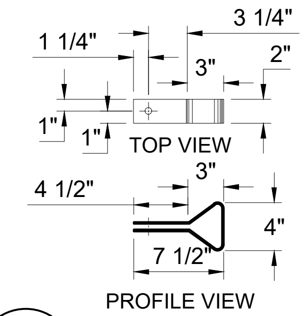

E / P6 SLIP JOINT PANEL DETAIL

SECTION VIEW

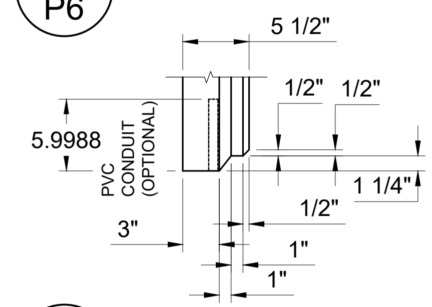



E / P6 SLIP JOINT PANEL DETAIL

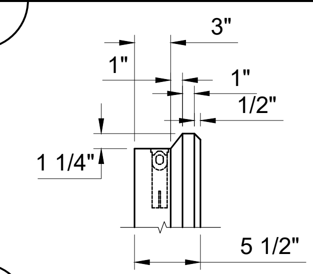
PLAN VIEW



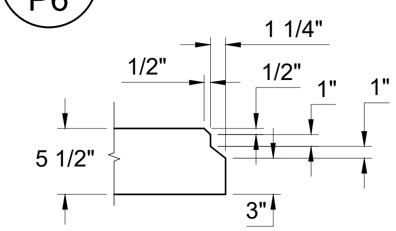

4 / P6 PANEL ANCHOR



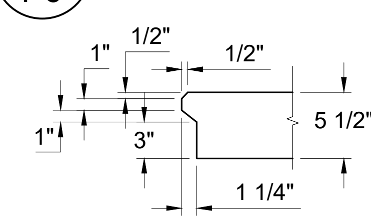

5 / P6 PANEL BOTTOM




6 / P6 PANEL TOP




7 / P6 PANEL RIGHT SIDE




8 / P6 PANEL LEFT SIDE

GENERAL NOTES

1. ALL DIMENSIONS ARE IN FEET AND INCHES (IMPERIAL UNITS) UNLESS NOTED OTHERWISE.
2. ALL PANELS ARE SHOWN BACK FACE.
3. ALL DIMENSIONS ARE SHOWN MEASURED FROM BACK FACE / BACK EDGE OF PRECAST PANEL FORM, TYPICAL.
4. ALL BLACK STEEL REINFORCEMENT AND EMBEDS SHALL HAVE A MINIMUM 1 ½" CONCRETE COVER
5. CONCRETE SHALL HAVE A MINIMUM 28-DAY COMPRESSIVE STRENGTH EQUAL TO 4000 PSI (UNLESS NOTED OTHERWISE) AND SHALL BE CAST IN ACCORDANCE WITH THE PROJECT SPECIFICATIONS.
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8. THE FRONT FACE EDGE OF ALL PANELS SHALL HAVE A ½" X ½" CHAMFER (UNLESS NOTED OTHERWISE).
9. PANEL REINFORCING SHALL BE WELDED WIRE FABRIC CONFORMING TO ASTM A1064 STANDARD SPECIFICATION FOR STEEL WIRE AND WELDED WIRE REINFORCEMENT, PLAIN AND DEFORMED, FOR CONCRETE WITH A MINIMUM YIELD STRENGTH EQUAL TO 65 KSI.
10. THE GALVANIZED ANCHOR EMBEDS SHALL NOT BE IN CONTACT WITH THE BLACK PANEL REINFORCEMENT.
11. PLACE PANEL ANCHORS AS DETAILED AND SO THEY ARE 90° TO THE BACK FACE OF THE PANEL..
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EARTHTEC
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PRECAST PANEL DETAIL
STANDARD SLIP JOINT PANEL

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GIS-2020

DESIGNED BY TPT
DRAWN BY TPT
CHECKED BY XXX

SHT: P6

**1.1.5**
**Facing Unit Compressive Strength**

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**1.1.6**
**Facing Unit Percent Air Entrainment**

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GROUND IMPROVEMENT SYSTEMS LLC
114 SOUTH COLLINS STREET
ARLINGTON, TX 76011

## 1.1.7
## Facing Unit Mix Designs

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**1.1.8**
**Facing Unit Alignment Pin and Bearing Pad**

EARTHTEC

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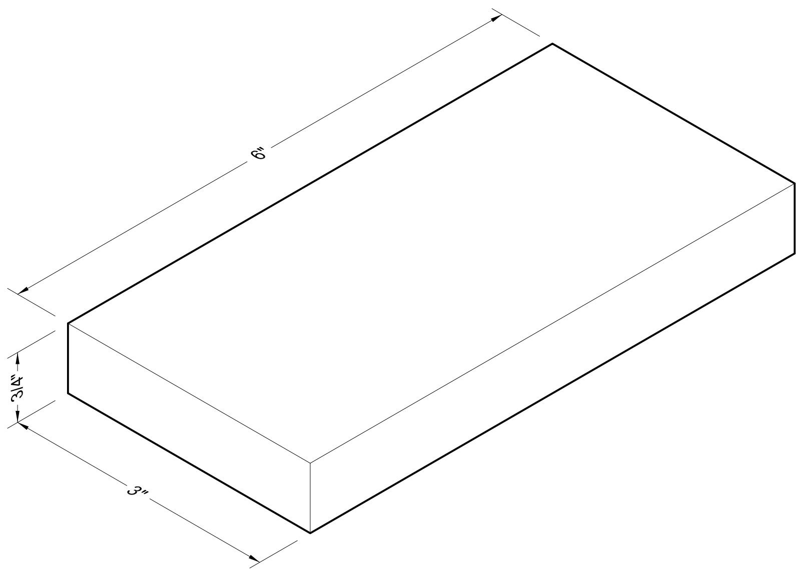


BEARING PAD
6" x 3" x 3/4"
60 Durometer

| | | | | | |
|---|---|---|---|---|---|
| GIS PN | Project Engineer | | **Ground Improvement Systems LLC**<br>114 South Collins Street<br>Arlington, TX. 76011<br>(817) 222-0969<br>www.groundimprovementsystems.com | Bearing Pads 6" x 3" x 3/4"<br>60 Durometer<br>Unit Wt. = 0.70 lbs | GS1 |
| SCALE<br>N.T.S. | Drawn By<br>tpt | Date Drawn | | Fabrication Drawing | BP |
| | Checked By: | Date Checked | | | |
| | Approved By: | Date Approved | | | |

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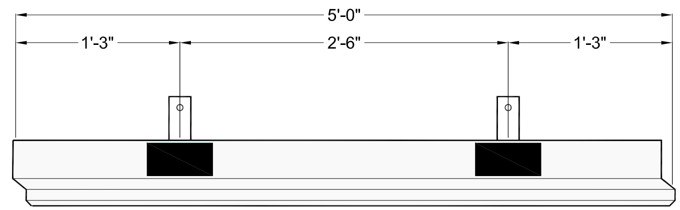
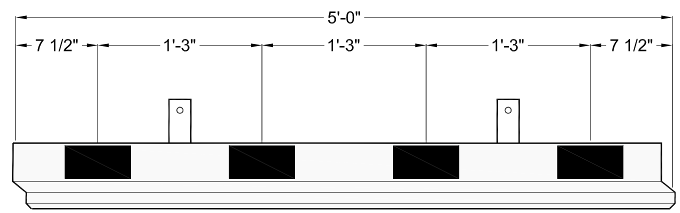



BEARING PAD PLACEMENT ON 5-FOOT PANEL
2 BEARING PADS




BEARING PAD PLACEMENT ON 5-FOOT PANEL
4 BEARING PADS

NOTE:

1. FOR WALL HEIGHTS THAT EXCEED 40 FEET USE 4 BEARING PADS PER PANEL AND PLACE EQUALLY ALONG TOP EDGE. ONLY THE WALL BELOW 40 FEET REQUIRE 4 BEARING PADS.
2. WALL HEIGHT IS MEASURED FROM TOP OF LEVELING PAD TO TOP OF PANEL.
3. BEARING PAD IS 60 DUROMETER SRB AT $\frac{3}{4}$" X 3" X 6"

| | | | | | | |
|---|---|---|---|---|---|---|
| GIS PN | Project Engineer | | GROUND IMPROVEMENT SYSTEMS LLC<br>114 South Collins Street<br>Arlington, TX. 76011<br>(817) 222-0969<br>www.groundimprovementsystems.com | BEARING PAD PLACEMENT<br>5-FOOT PANEL | GS1 | BP-5 |
| SCALE<br>N.T.S. | Drawn By<br>tpt | Date Drawn | | | | |
| | Checked By: | Date Checked | | Fabrication Drawing | | |
| | Approved By: | Date Approved | | | | |

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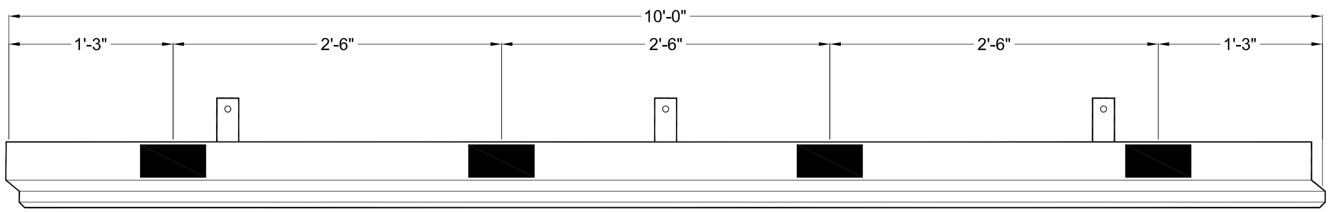
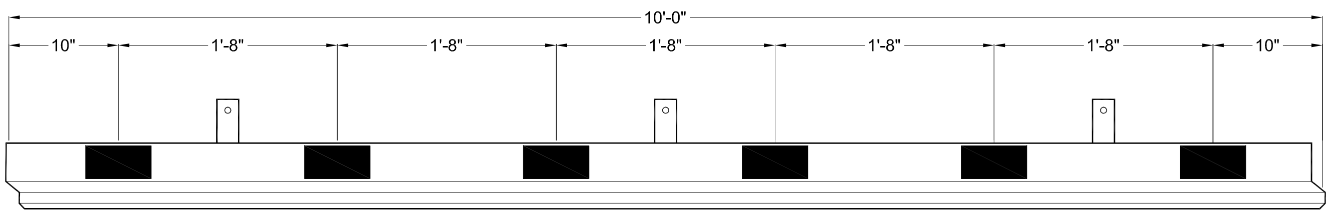



BEARING PAD PLACEMENT ON 10-FOOT PANEL
4 BEARING PADS




BEARING PAD PLACEMENT ON 10-FOOT PANEL
8 BEARING PADS

NOTE:

1. FOR WALL HEIGHTS THAT EXCEED 40 FEET USE 8 BEARING PADS PER PANEL AND PLACE EQUALLY ALONG TOP EDGE. ONLY THE WALL BELOW 40 FEET REQUIRE 4 BEARING PADS.
2. WALL HEIGHT IS MEASURED FROM TOP OF LEVELING PAD TO TOP OF PANEL.
3. BEARING PAD IS 60 DUROMETER SRB AT $\frac{3}{4}$" X 3" X 6"

| GIS PN | Project Engineer | | GROUND IMPROVEMENT SYSTEMS LLC | BEARING PAD PLACEMENT 10-FOOT PANEL | GS1 | BP-10 |
|---|---|---|---|---|---|---|
| SCALE N.T.S. | Drawn By tpt | Date Drawn | 114 South Collins Street | | | |
| | Checked By: | Date Checked | Arlington, TX. 76011 | | | |
| | Approved By: | Date Approved | (817) 222-0969 www.groundimprovementsystems.com | Fabrication Drawing | | |

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EARTHTEC

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# LABORATORY TEST REPORT
## BEARING PAD

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



| CLIENT | CLIENT PROJECT REF. NO. | CLIENT ORDER NO. |
|---|---|---|
| Tom Taylor | 3/4" Sample | Tom Taylor |

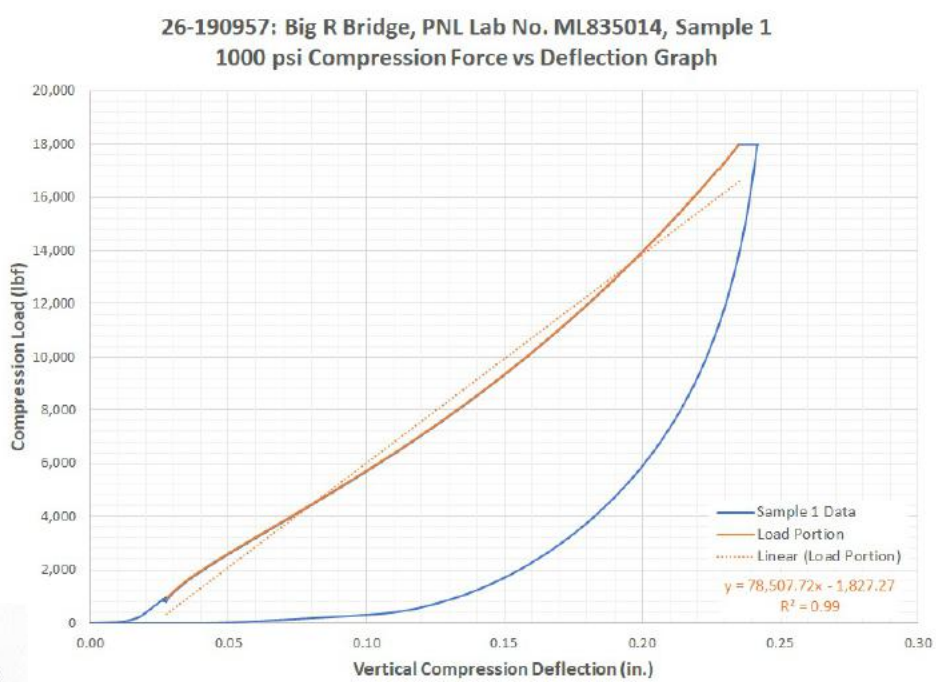
| STATE | STATE PROJECT NO. | PROJECT DESCRIPTION |
|---|---|---|
| NC | Unknown | Compression Deflection Testing of SBR Bearing for MSE Wall |

| MATERIAL SUBMITTED BY | DATE SUBMITTED | PNL PROJECT NO. | S.O. NO. | PNL LAB NO. | REPORT DATE |
|---|---|---|---|---|---|
| Client | 12/23/2019 | 26-190957 | 001 | ML835014 | 01/10/2020 |

| SPECIFICATION | DUROMETER | GRADE | ELASTOMER | LOT NO. | SAMPLE SIZE |
|---|---|---|---|---|---|
| ASTM D575 | 60 | NA | SBR | 3/4 in. | 3/4" x 3" x 6" |

**TEST RESULTS**

| STANDARD | DESCRIPTION | REQUIREMENT | COMP. DEFLECTION (%) | PAD THICKNESS AT 100% LOAD (in.) |
|---|---|---|---|---|
| **ASTM D575** | **COMPRESSION DEFLECTION TEST: 250 psi (4500 lbf)** | Record Results | 0.085 | 0.686 |
| **ASTM D575** | **COMPRESSION DEFLECTION TEST: 500 psi (9000 lbf)** | Record Results | 0.167 | 0.625 |
| **ASTM D575** | **COMPRESSION DEFLECTION TEST: 750 psi (13,500 lbf)** | Record Results | 0.223 | 0.582 |
| **ASTM D575** | **COMPRESSION DEFLECTION TEST: 1000 psi (18,000 lbf)** | Record Results | 0.278 | 0.542 |





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

REVIEWED BY________________

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

# LABORATORY TEST REPORT
## BEARING PAD

Registered Professional Engineer (Mechanical) CERTIFICATE NO. 53776 KYLE J. FLEEGE DATE SIGNED 01-14-2020 ARIZONA U.S.A.
Expires 09/30/2021



| CLIENT | STATE PROJECT NO. | CLIENT ORDER NO. |
|---|---|---|
| Tom Taylor | Unknown | Tom Taylor |
| PROJECT DESCRIPTION | | PNL LAB NO. |
| Compression Deflection Testing of SBR Bearing | | ML835014 |
| LOT NO. | PAD SIZE | REPORT DATE |
| 3/4 in. | 3/4" x 3" x 6" | 01/10/2020 |

## SPECIFIC TEST DATA

| TEST DATA & EQUIPMENT INFORMATION | | | |
|---|---|---|---|
| **TEMPERATURE:** | 73.4 ± 3.6F | **HUMIDITY:** | 50 ± 5% |
| **EQUIPMENT MODEL:** | Instron 5985 | **SERIAL NUMBER:** | U1246 |
| **LOAD CELL:** | 250 kN (56,202 lbf) | **SERIAL NUMBER:** | 203548 |
| **TEST SPEED:** | 0.5 in./min. | **TEST SAMPLE SIZE:** | 0.75 in. x 3 in. x 6 in. |

**SAMPLE 1: COMPRESSION DEFLECTION (STRAIN) - ASTM D575, METHOD B**

| TEST LOAD | | 5% TL (MINOR FORCE) DEFLECTION, A (in.) | 100% TL (MAJOR FORCE) DEFLECTION, B (in.) | TOTAL COMPRESSIVE DEFLECTION, B-A (in.) | ERT (in.) | STRAIN (B-A)/ERT | PAD THICKNESS AT TEST LOAD (in.) |
|---|---|---|---|---|---|---|---|
| psi | lbf | | | | | | |
| 250 | 4,500 | 0.017 | 0.081 | 0.064 | 0.75 | 0.085 | 0.686 |
| 500 | 9,000 | 0.020 | 0.146 | 0.125 | 0.75 | 0.167 | 0.625 |
| 750 | 13,500 | 0.023 | 0.195 | 0.172 | 0.75 | 0.229 | 0.578 |
| 1000 | 18,000 | 0.028 | 0.242 | 0.214 | 0.75 | 0.285 | 0.536 |

**SAMPLE 2: COMPRESSION DEFLECTION (STRAIN) - ASTM D575, METHOD B**

| TEST LOAD, TL | | 5% TL (MINOR FORCE) DEFLECTION, A (in.) | 100% TL (MAJOR FORCE) DEFLECTION, B (in.) | TOTAL COMPRESSIVE DEFLECTION, B-A (in.) | ERT (in.) | STRAIN (B-A)/ERT | PAD THICKNESS AT TEST LOAD (in.) |
|---|---|---|---|---|---|---|---|
| psi | lbf | | | | | | |
| 250 | 4,500 | 0.013 | 0.084 | 0.071 | 0.75 | 0.094 | 0.679 |
| 500 | 9,000 | 0.019 | 0.145 | 0.125 | 0.75 | 0.167 | 0.625 |
| 750 | 13,500 | 0.025 | 0.192 | 0.167 | 0.75 | 0.223 | 0.583 |
| 1000 | 18,000 | 0.031 | 0.236 | 0.206 | 0.75 | 0.274 | 0.544 |

**SAMPLE 3: COMPRESSION DEFLECTION (STRAIN) - ASTM D575, METHOD B**

| TEST LOAD, TL | | 5% TL (MINOR FORCE) DEFLECTION, A (in.) | 100% TL (MAJOR FORCE) DEFLECTION, B (in.) | TOTAL COMPRESSIVE DEFLECTION, B-A (in.) | ERT (in.) | STRAIN (B-A)/ERT | PAD THICKNESS AT TEST LOAD (in.) |
|---|---|---|---|---|---|---|---|
| psi | lbf | | | | | | |
| 250 | 4,500 | 0.015 | 0.077 | 0.062 | 0.75 | 0.083 | 0.688 |
| 500 | 9,000 | 0.019 | 0.139 | 0.121 | 0.75 | 0.161 | 0.629 |
| 750 | 13,500 | 0.022 | 0.189 | 0.168 | 0.75 | 0.223 | 0.582 |
| 1000 | 18,000 | 0.027 | 0.235 | 0.208 | 0.75 | 0.278 | 0.542 |

Notes: Due to natural curvature of test sample surfaces, an initial 50 lbf force was applied to the test specimen to bring load plates into full contact with SBR surface and the deflection was zeroed. Deflection was recorded after 3 s for the 1000 psi Test Load Minor Force and Major Force load points. Deflection at additional points was recorded instantaneously.

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

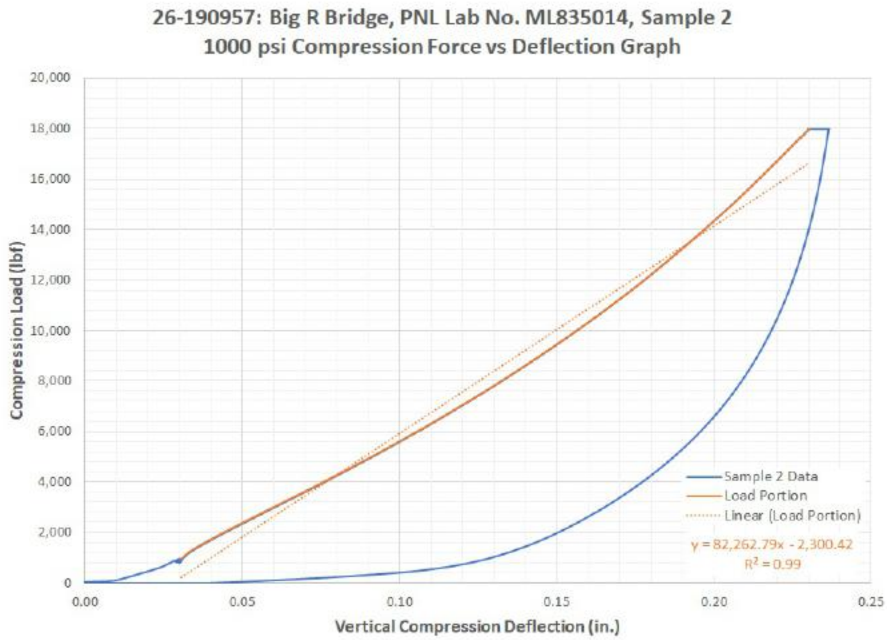
# LABORATORY TEST REPORT
## BEARING PAD

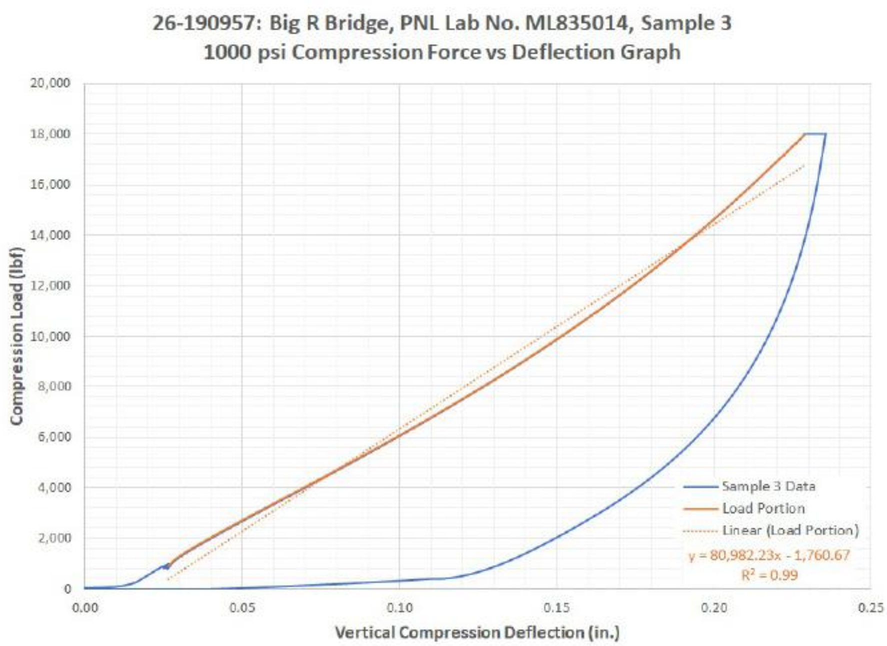
Registered Professional Engineer (Mechanical) Certificate No. 53776 KYLE J. FLEEGE
Date Signed 01-14-2020 Arizona U.S.A.
Expires 09/30/2021



| CLIENT | STATE PROJECT NO. | CLIENT ORDER NO. |
|---|---|---|
| Tom Taylor | Unknown | Tom Taylor |
| PROJECT DESCRIPTION | | PNL LAB NO. |
| Compression Deflection Testing of SBR Bearing | | ML835014 |
| LOT NO. | PAD SIZE | REPORT DATE |
| 3/4 in. | 3/4" x 3" x 6" | 01/10/2020 |

## SPECIFIC TEST DATA



# 1.1.9
# Facing Unit Joint Filter

EARTHTEC

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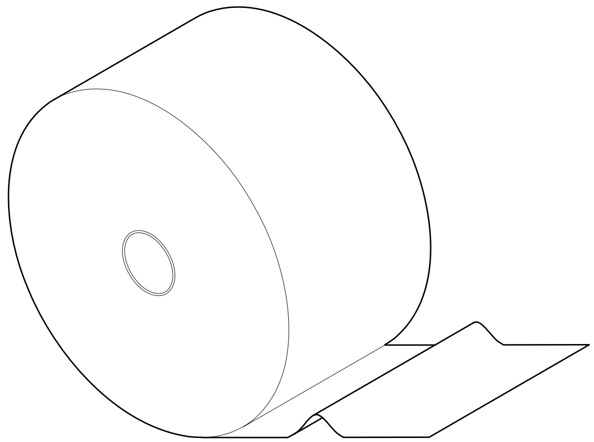

NON-WOVEN FILTER FABRIC
12" WIDE
1 Roll = 360 LF


SUBFLOOR ADHESIVE
6 Tubes per Roll

1. Specifications
   1.1. ASTM D4632 - Standard Test Method for Grab Breaking Load and Elongation of Geotextiles
   1.2. ASTM D4833 - Standard Test Method for Index Puncture Resistance of Geomembranes and Related Products
   1.3. ASTM D3786 - Standard Test Method for Bursting Strength of Textile Fabrics--Diaphragm Bursting Strength Tester Method
   1.4. ASTM D3787 - Standard Test Method for Bursting Strength of Textiles Constant-Rate-Traverse (CRT) Burst Test
   1.5. ASTM D4533 - Standard Test Method for Trapezoid Tearing Strength of Geotextiles
   1.6. ASTM D4751 - Standard Test Method for Determining Apparent Opening Size of a Geotextile
   1.7. ASTM D4491 - Standard Test Methods for Water Permeability of Geotextiles by Permittivity
   1.8. ASTM D4355 - Standard Test Method for Deterioration of Geotextiles by Exposure to Light, Moisture and Heat in a Xenon Arc Type Apparatus
2. Acceptance

   All material will be accepted on the basis of the required certification and testing required by the engineer.
3. Material

   The filter fabric shall consist of a needle punched geotextile that is resistance to ultraviolet degradation and to biological and chemical attack normally found in soils.
4. Property Criteria

   The following properties will be tested and adhered to the minimum values provided in the table.
   4.1. Grab Tensile Strength D4632 100 lbs
   4.2. Grab Elongation D4632 50%
   4.3. Puncture Strength D4833 65 lbs
   4.4. Mullen burst D37872 25 psi
   4.5. Trapezoidal Tear D4533 45 lbs
   4.6. Apparent Opening Size (AOS)D4751 70 US Standard Sieve
   4.7. Permittivity D4491 2.00 $sec^{-1}$
   4.8. Permability D4491 0.22 cm/sec
   4.9. Water Flow Rate D44911 40 $gpm/ft^2$
   4.10. UV Resistance (% Retained after 500 hours) D4355 70%
5. Certification

   The manufacturer shall furnish certification of all material. The certification shall indicate that the components are in accordance with this specification and shall include typical test results representative of the material.

| | | | | |
|---|---|---|---|---|
| GIS PN | Project Engineer | | | |
| SCALE N.T.S. | Drawn By tpt | Date Drawn | | |
| | Checked By: | Date Checked | | |
| | Approved By: | Date Approved | | |

GIS Support Through Innovation
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GROUND IMPROVEMENT SYSTEMS LLC
114 South Collins Street
Arlington, TX. 76011
(817) 222-0969
www.groundimprovementsystems.com

Non-Woven Filter Fabric
12" Wide 1 Roll = 360 LF /
6 Tubes Adhesive
Fabrication Drawing

GS1

FF

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# 1.1.10
# Facing Unit Aesthetic Options

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## AESTHETIC CHOICES

EarthTec offers a wide variety of architectural finishes, including form liners and concrete color. Form liners can be manufactured to suit almost any designer's needs. Color can be applied to the surface of the panel or as a pigment to the cement. Anti-Graffiti coatings can be applied to the bare concrete surface or to the painted concrete surface. The following are standard form liners used by EarthTec.

**Ashlar Stone Form Liner**

The Ashlar Stone form liner provides the look of a stacked random rock or stone look. The form linear can be supplied in several different block sizes, grout widths and depths, and block finish.

**Reverse Rope Form Liner**

The Reverse Rope form liner provides the look vertically placed rope. The form linear can be supplied in several different rope sizes, braids, and depths.

**Fractured Fin**

The Fractured Fin form liner provides a striated, rough, raised relief. This form liner is used extensively with architectural art work. The striations can be produced in many different widths, depths and shapes.

**Raised Vertical Relief**

The Raised Vertical Relief provides the look of three-dimensional columns. The vertical relief can be supplied in varying widths. The number of columns per form can be varied.

**Custom Art Work**

The custom form liners can be prepared to provide murals on the wall faces. The wall face can be painted. Stained, or left natural.

**Brick Form Liner**

The Brick form liner provides a masonry brick look. The brick size, pattern (running bond or stack bond) and mortar joint width and depth can be varied as required. The bricks can be stained or painted to provide a more three-dimensional look.

**Vertical Grave Stake**

The Vertical Grave Stake form liner provides a rough three-dimensional look. The pattern can be modified in several different thicknesses and relief patterns.

# 1.1.11
# Facing Unit Alignment Requirements for Curves and Corners

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114 SOUTH COLLINS STREET
ARLINGTON, TX 76011

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## 1.2
## INEXTENSIBLE REINFORCEMENTS

EARTHTEC



GROUND IMPROVEMENT SYSTEMS LLC
114 SOUTH COLLINS STREET
ARLINGTON, TX 76011

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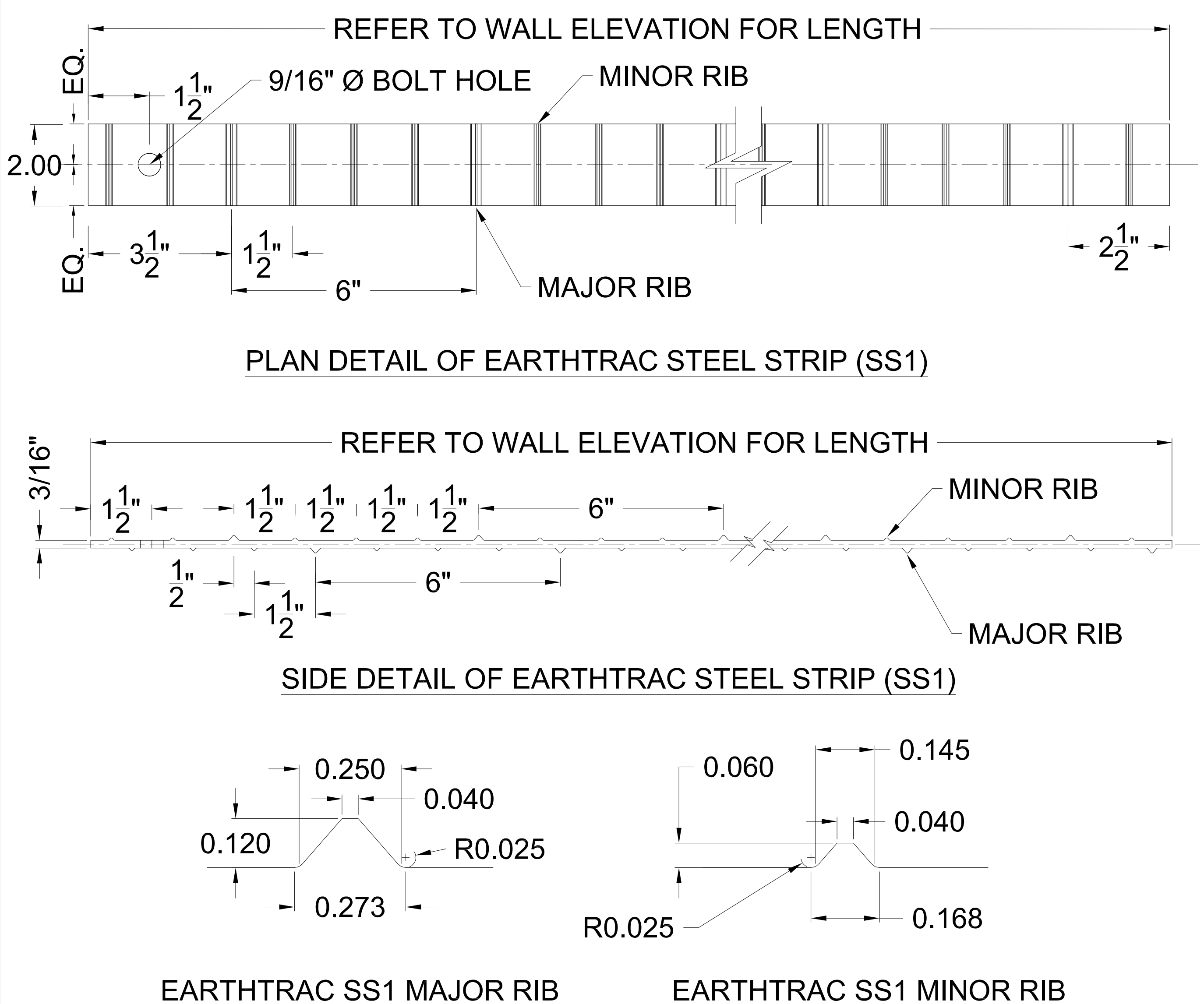


1. Specifications
   1.1. ASTM A572 - Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel.
2. Acceptance
   All material will be accepted on the basis of the required certification and testing required by the ASTM specifications.
3. Material
   The soil reinforcing shall be Grade 50 steel and have a minimum tensile capacity of 65 ksi and a minimum yield capacity of 50 ksi.
4. Tolerances
   The permissible variation from the dimensions and configuration shown shall be as follows:
   4.1. Overall Plate Vertical Dimensions$\pm \frac{1}{8}$
   4.2. Overall Plate Horizontal Dimensions$\pm \frac{1}{8}$
   4.3. Overall Hole Dimension$\pm \frac{1}{64}$
   4.4. Overall Edge Distance$\pm \frac{1}{32}$
5. Certification
   The manufacturer shall furnish certification of all material. The certification shall indicate that the components are in accordance with this specification and shall include typical test results representative of the material. The certification shall be notarized.

| | | | | |
|---|---|---|---|---|
| GIS PN | Project Engineer | **GROUND IMPROVEMENT SYSTEMS LLC**<br>114 South Collins Street<br>Arlington, TX. 76011<br>(817) 222-0969<br>www.groundimprovementsystems.com | EARTHTRAC SS1 | GS1 |
| SCALE N.T.S. | Drawn By tpt / Date Drawn | | | SS1 |
| | Checked By: / Date Checked | | Fabrication Drawing | |
| | Approved By: / Date Approved | | | |

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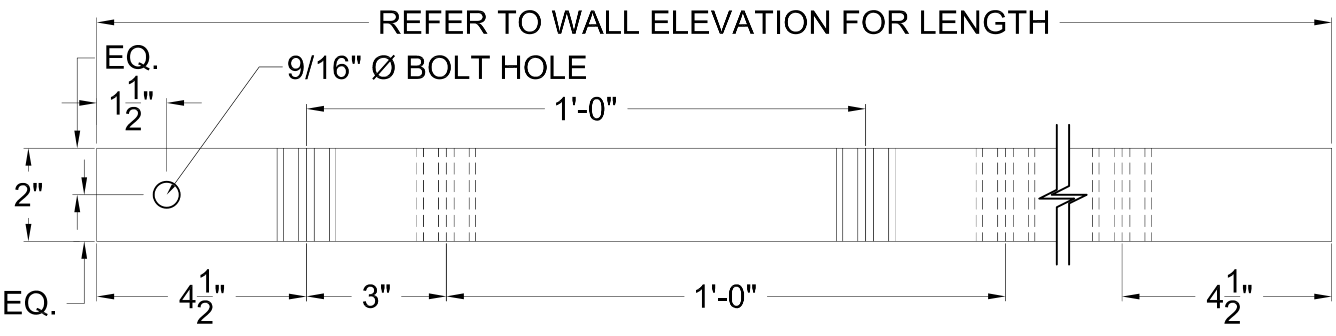
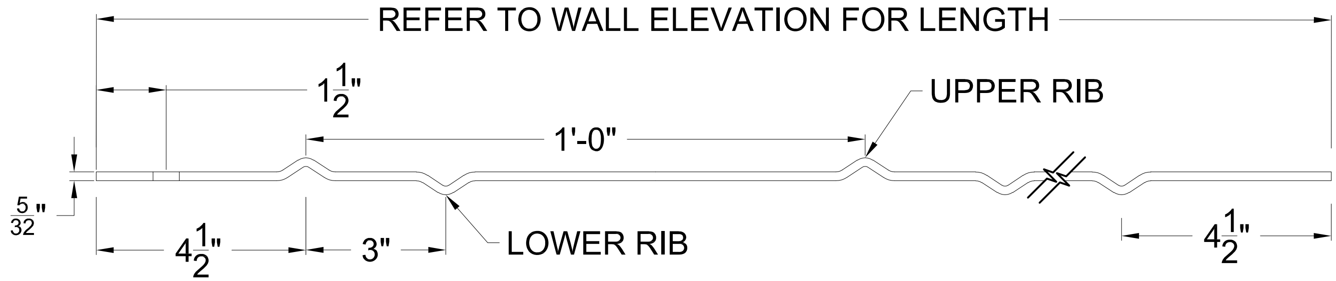
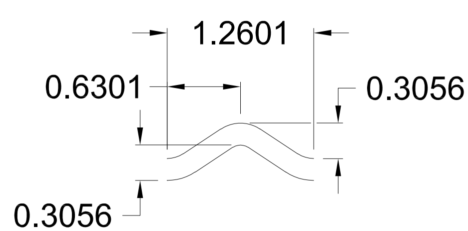



PLAN DETAIL OF EARTHTRAC STEEL STRIP STRIP (SS3)




SIDE DETAIL OF EARTHTRAC STEEL STRIP STRIP (SS3)




EARTHTRAC SS3 RIB

1. Specifications
   1.1. ASTM A1011 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.
2. Acceptance
   All material will be accepted on the basis of the required certification and testing required by the ASTM specifications.
3. Material
   The soil reinforcing shall be Grade 50 steel and have a minimum tensile capacity of 65 ksi and a minimum yield capacity of 50 ksi.
4. Tolerances
   The permissible variation from the dimensions and configuration shown shall be as follows:
   4.1. Overall Plate Vertical Dimensions$\pm \frac{1}{8}$
   4.2. Overall Plate Horizontal Dimensions$\pm \frac{1}{8}$
   4.3. Overall Hole Dimension$\pm \frac{1}{64}$
   4.4. Overall Edge Distance$\pm \frac{1}{32}$
5. Certification
   The manufacturer shall furnish certification of all material. The certification shall indicate that the components are in accordance with this specification and shall include typical test results representative of the material. The certification shall be notarized.

| GIS PN | Project Engineer | | GROUND IMPROVEMENT SYSTEMS LLC | EARTHTRAC SS3 | GS1 | SS3 |
|---|---|---|---|---|---|---|
| SCALE N.T.S. | Drawn By tpt | Date Drawn | 114 South Collins Street | | | |
| | Checked By: | Date Checked | Arlington, TX. 76011 | | | |
| | Approved By: | Date Approved | (817) 222-0969 | Fabrication Drawing | | |
| | | | www.groundimprovementsystems.com | | | |

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**1.2.1**
**Soil Reinforcing Innovations**

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## 1.2.2
## Soil Reinforcing Types

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GROUND IMPROVEMENT SYSTEMS LLC
114 SOUTH COLLINS STREET
ARLINGTON, TX 76011

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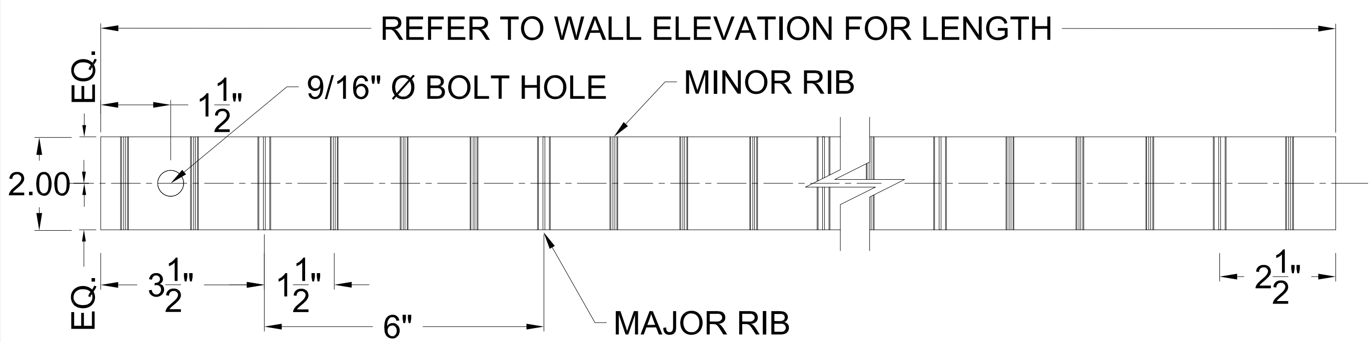
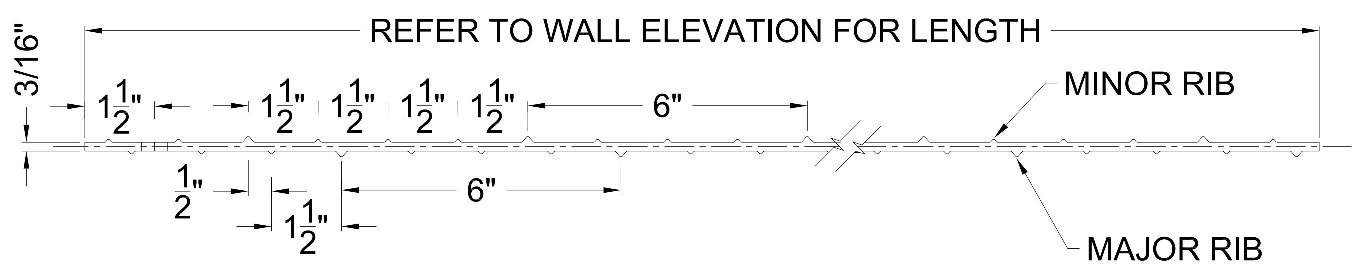
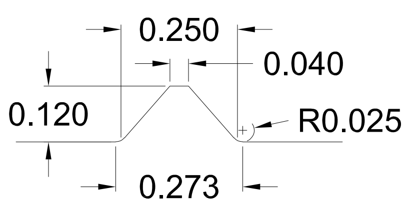


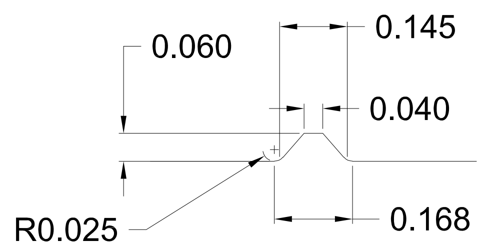

PLAN DETAIL OF EARTHTRAC STEEL STRIP (SS1)




SIDE DETAIL OF EARTHTRAC STEEL STRIP (SS1)




EARTHTRAC SS1 MAJOR RIB




EARTHTRAC SS1 MINOR RIB

1. Specifications
   1.1. ASTM A572 - Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel.
2. Acceptance
   All material will be accepted on the basis of the required certification and testing required by the ASTM specifications.
3. Material
   The soil reinforcing shall be Grade 50 steel and have a minimum tensile capacity of 65 ksi and a minimum yield capacity of 50 ksi.
4. Tolerances
   The permissible variation from the dimensions and configuration shown shall be as follows:
   4.1. Overall Plate Vertical Dimensions$\pm \frac{1}{8}$
   4.2. Overall Plate Horizontal Dimensions$\pm \frac{1}{8}$
   4.3. Overall Hole Dimension$\pm \frac{1}{64}$
   4.4. Overall Edge Distance$\pm \frac{1}{32}$
5. Certification
   The manufacturer shall furnish certification of all material. The certification shall indicate that the components are in accordance with this specification and shall include typical test results representative of the material. The certification shall be notarized.

| | | | |
|---|---|---|---|
| GIS PN | Project Engineer | | |
| SCALE N.T.S. | Drawn By tpt | Date Drawn | |
| | Checked By: | Date Checked | |
| | Approved By: | Date Approved | |

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**GROUND IMPROVEMENT SYSTEMS LLC**
114 South Collins Street
Arlington, TX. 76011
(817) 222-0969
www.groundimprovementsystems.com

EARTHTRAC SS1

Fabrication Drawing

GS1

**SS1**

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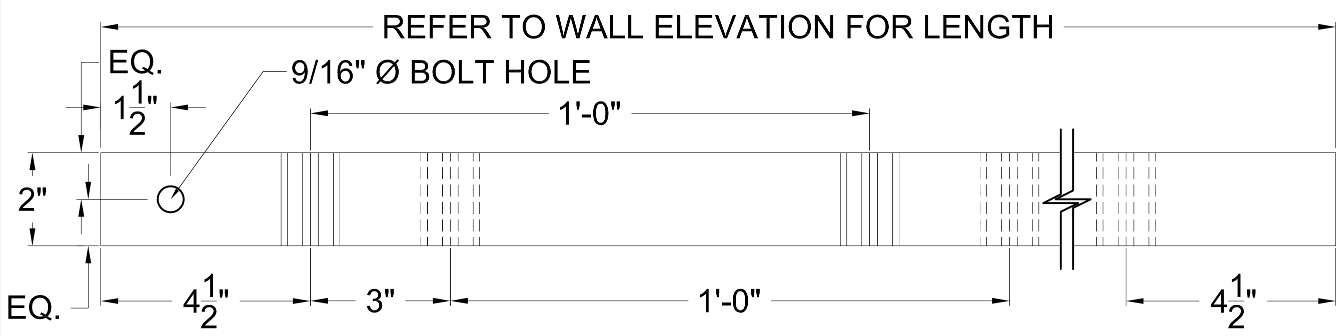
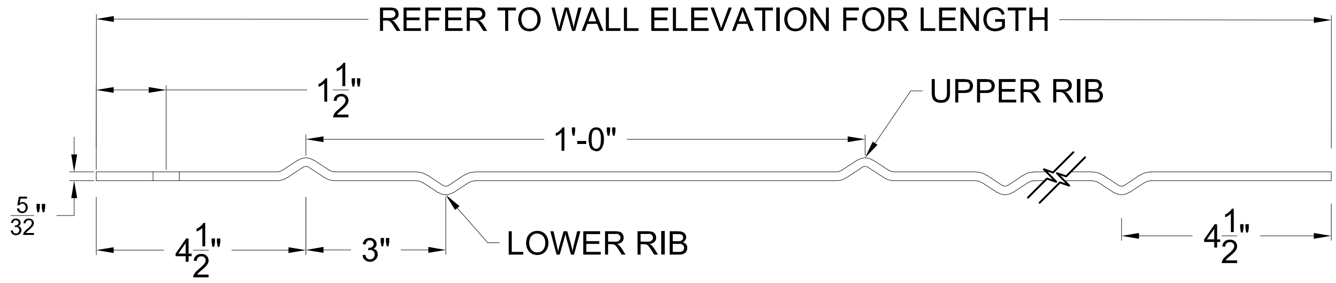
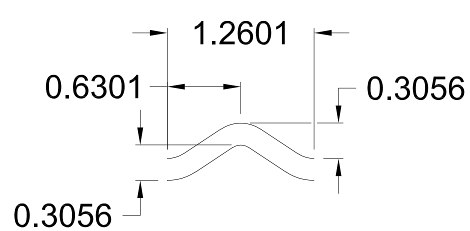



PLAN DETAIL OF EARTHTRAC STEEL STRIP STRIP (SS3)




SIDE DETAIL OF EARTHTRAC STEEL STRIP STRIP (SS3)




EARTHTRAC SS3 RIB

1. Specifications
   1.1. ASTM A1011 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.
2. Acceptance

   All material will be accepted on the basis of the required certification and testing required by the ASTM specifications.
3. Material

   The soil reinforcing shall be Grade 50 steel and have a minimum tensile capacity of 65 ksi and a minimum yield capacity of 50 ksi.
4. Tolerances

   The permissible variation from the dimensions and configuration shown shall be as follows:
   4.1. Overall Plate Vertical Dimensions$\pm \frac{1}{8}$
   4.2. Overall Plate Horizontal Dimensions$\pm \frac{1}{8}$
   4.3. Overall Hole Dimension$\pm \frac{1}{64}$
   4.4. Overall Edge Distance$\pm \frac{1}{32}$
5. Certification

   The manufacturer shall furnish certification of all material. The certification shall indicate that the components are in accordance with this specification and shall include typical test results representative of the material. The certification shall be notarized.

| GIS PN | Project Engineer | | GROUND IMPROVEMENT SYSTEMS LLC<br>114 South Collins Street<br>Arlington, TX. 76011<br>(817) 222-0969<br>www.groundimprovementsystems.com | EARTHTRAC SS3 | GS1 | SS3 |
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| SCALE<br>N.T.S. | Drawn By<br>tpt | Date Drawn | | Fabrication Drawing | | |
| | Checked By: | Date Checked | | | | |
| | Approved By: | Date Approved | | | | |

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**1.2.3**
**Soil Reinforcing Properties**

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**1.2.4**
**Soil Reinforcing Corrosion Protection**

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**1.2.5**
**Soil Reinforcing Sacrificial Steel**
**as a Function of Service Life**

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# MSE CALCULATION PROCEDURE

## REINFORCED SOIL WALL

## SEGMENTAL CONCRETE PANEL

## SOIL REINFORCING DURABILITY

EARTHTEC

**PREPARED FOR:**

## IDEA SUBMITTAL


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**Arlington, TX 76011**

The following report describes the EarthTec Reinforced Soil Wall design methodology for Mechanically Stabilized Earth retaining structures to determine the durability of the EarthTrac soil reinforcing. The method is consistent with the 9$^{th}$ Edition of the American Association of State Highway and Transportation Officials *AASHTO LRFD Bridge Design Specification 2020 and all interims*. Reference is made to NCHRP Report 675 *LRFD Metal loss and Service-Life Strength Reduction Factors for Metal-Reinforced Systems.*



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## Durability

The durability of galvanized steel is dependent on the electro-chemical properties of the steel and the backfill. The AASHTO service life model for Mechanically Stabilized Earth reinforcement is based on extensive studies of galvanized steel in various environments. A conservative metal loss rate has been developed based on 60 years of research.

The galvanization of steel consists of the application of a coating of zinc. AASHTO 11.10.6.4.2 Design Life Considerations require the zinc coating to be a minimum of 2 oz/ft2 or thickness equal to 3.4 mils. AASHTO requires the zinc coating conform to ASTM A 123 *Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products*. The specification requires applying the zinc after fabrication of the soil reinforcing, including welding, forming, and bending.

The AASHTO metal loss models assume that the 3.4 mils thick coating is the minimum thickness applied to the bare metal. The amount of degradation is a function of the soil electro-chemical composition, steel protective coating, and the length of time the element is buried in the soil. Based on this model, AASHTO quantifies the rate of degradation of the zinc coating to be equal to:

Zinc loss rate (first two years) = $0.58 \frac{\text{mil}}{\text{year}}$

Zinc loss rate (remaining years) = $0.16 \frac{\text{mil}}{\text{year}}$

Assuming that the zinc is applied following ASTM A 123, the sacrificial life, in years, of the zinc coating can be calculated as shown in Equation 1

$$Y_G = \frac{3.4\text{mil} - 2 \cdot 0.58 \frac{\text{mil}}{\text{year}}}{0.16 \frac{\text{mil}}{\text{year}}} + 2 \cdot \text{years} = 16\text{years} \qquad \text{Equation 1}$$

Once the zinc coating is sacrificed, the steel is then sacrificed. AASHTO metal loss models have quantified the sacrificial loss rate, per year, of steel as:

Steel loss rate = $0.47\dfrac{mil}{year}$

The total steel thickness loss computed for75-year design life and 100-year design life is shown in Equation 2 and Equation 3, respectively. A factor of 2 is included to consider metal losses for the top and bottom surfaces of the strip reinforcements.

$$E_{S75} = (75\cdot years - 16\cdot years)\cdot 0.47\frac{mil}{year}\cdot 2 = 55.46mil = 0.056\cdot in \qquad \text{Equation 2}$$

$$E_{S100} = (100\cdot years - 16\cdot years)\cdot 0.47\frac{mil}{year}\cdot 2 = 78.96mil = 0.079\cdot in \qquad \text{Equation 3}$$

The sacrificial amount of steel is applied to the thickness of the strip to determine the end of deign life thickness. The end of the design life thickness is used to determine the design area of steel.

It is worth noting that the zinc uptake during the galvanization process varies with the micro-metallurgy of the metal being galvanized. The amount of zinc uptake to the metal is dependent on the time it is submerged in the zinc bath. Micro-metallurgy is not controllable. Because of this, most Galvanizers will increase the time that the metal is submerged in the zinc bath. By increasing the length of time that the metal is submerged in the zinc bath increases the amount of zinc uptake. Because of this increased submergence, the AASHTO metal loss model is conservative.

Backfill electrochemical properties will also affect the service life of buried galvanized steel. These electrochemical properties include the levels of dissolved sulfate and chloride ions, the ph, and the soil resistivity at 100% saturation. These parameters are interdependent, and the combination influences the soil corrosiveness. The independent range of measurement of these parameters is defined in AASHTO section 11.10.6.4.2a and is given as:

- Chlorides $<$ 100 ppm
- Sulfates $<$ 200 ppm.
- pH 5 - 10
- Resistivity $>$ 3000 ohm-cm

It is generally understood that soil resistivity is the most accurate indicator of corrosion potential. The NCHRP-50 report, *Durability/Corrosion of Drainage Pipe,* defines the aggressiveness of corrosion based on the range of resistivity and is defined by the following ranges of resistivity.

| Aggressiveness | Resistivity |
|---|---|
| Very Corrosive | $<$ 700 ohm-cm |
| Corrosive | 700-2000 ohm-cm |
| Moderately Corrosive | 2000-5000 ohm-cm |
| Mildly Corrosive | 5000-10000 ohm-cm |
| Non-corrosive | $>$ 10000 ohm-cm |

Another essential characteristic is the gradation of the select granular backfill. The select granular backfill material used in the mechanically stabilized earth structure must be reasonably free from organic and otherwise deleterious materials and typically conform to the following gradation limits as determined by AASHTO T-27. Gap grading is not allowed.

| Sieve Size | Percent Passing |
|---|---|
| 3” | 100 |
| ¾” | 70-100 |
| No. 4 | 30-100 |
| No. 40 | 15-100 |
| No. 100 | 5-65 |
| No. 200 | 0-15 |

In addition, the backfill shall have a Plasticity Index (P.I.), less than 6, as determined by AASHTO T-90. The fraction of material finer than the Number-200 sieve (15-micron) size shall not exceed 15 percent. The soundness of the material shall be substantially free of shale or other soft, poor durability particles. The materials shall have a magnesium sulfate soundness loss of less than 20 percent after five (5) cycles, as determined by AASHTO T-104.

## Resistance Factor

The EarthTrac soil reinforcing is attached to prefabricated segmental concrete panels that are of a fixed size. Because the size of the panel is known, the EarthTrac soil reinforcement is placed in a fixed and consistent spacing in both the horizontal and vertical directions. The spacing of the EarthTrac soil reinforcing is selected based on the location in the structure and is dependent on the soil design parameters, steel design parameters, design method, and design procedure. The required density of steel is determined based on the yield limit state. The EarthTrac tension cannot exceed the yield limit multiplied by a resistance factor. The yield resistance factor is given in AASHTO and is based on the soil reinforcing type. The resistance factor for strip reinforcement is defined as 0.75.



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**CLIENT:** DOT SUBMITTAL DURABILITY
**PROJECT:** GIS EarthTrac Reinforced Soil Wall System

**PROJECT No:** **GIS P. N.**

**CALC BY:** TPT **DATE:** **CHKD BY:** **DATE:**

**SUBJECT:** Durability of Soil Reinforcing

**SPECIFICATION:** AASHTO LRFD Bridge Design Specification Article 11.10.6.4.2

---

**STEEL STRIP ASTM GRADE-50 MATERIAL PARAMETERS**

Yield Stress of Steel.................................................................................... $f_y := 50.00 \cdot ksi$

Yield Coefficient........................................................................................ $\phi_y := 0.75$

**MATERIAL SPECIFICATIONS**

ASTM A1064 - Standard Specification for Steel Welded Wire Fabric, Plain and Deformed , for Concrete Reinforcement

ASTM A123 - Standard Specification for Hot Dipped Galvanizing

**STEEL DEGRADATION RATES**

75 year design life ......................................................................................... $Y_{75} := 75 \cdot yr$

100 year design life ......................................................................................... $Y_{100} := 100 \cdot yr$

Thickness of galvanized coating ......................................................................... $t_{zinc} := 3.4 \cdot mil$

Zinc loss rate (first two years) ............................................................................ $t_{zinc\_2} := 0.591 \cdot \frac{mil}{yr}$

Zinc loss rate (remaining years) .......................................................................... $t_{zinc\_R} := 0.157 \cdot \frac{mil}{yr}$

Steel loss rate (Coarse) ...................................................................................... $t_S := 0.47 \cdot \frac{mil}{yr}$

Design Life of Galvanized Coating

$$Y_G := \frac{t_{zinc} - 2 \cdot yr \cdot t_{zinc\_2}}{t_{zinc\_R}} + 2 \cdot yr = 16.127\ yr$$

Sacrificial Steel at 75 years

$$E_{75} := \textbf{if}\left(Y_{75} \le Y_G, 0, \left(\left(Y_{75} - Y_G\right) \cdot t_S\right) \cdot 2\right) = 0.0553\ in$$

Sacrificial Steel at 100 years

$$E_{100} := \textbf{if}\left(Y_{100} \le Y_G, 0, \left(\left(Y_{100} - Y_G\right) \cdot t_S\right) \cdot 2\right) = 0.0788\ in$$

**EarthTrac SS1 Soil Reinforcing (2" x 3/16") - Parameters**

Thickness of Strip ........................................................................................ $t_{SS1} := \left(\frac{3}{16}\right) \cdot in$

Width of strip ............................................................................................. $w_{SS1} := 2 \cdot in$

Area of strip no corrosion

$$A_{SS1} := t_{SS1} \cdot w_{SS1} = 0.375\ in^2$$

Maximum horizontal force no corrosion

$$T_{max\_SS1} := \phi_y \cdot f_y \cdot A_{SS1} = 14.063\ kip$$

**EarthTrac SS1 Soil Reinforcing (2" x 3/16") - 75 years**

Thickness with effects of corrosion at 75 years

$$t_{SS1\_75} := \left(t_{SS1} - E_{75}\right) = 0.132\ in$$

Area of strip with effects of corrosion at 75 years

$$A_{SS1\_75} := t_{SS1\_75} \cdot w_{SS1} = 0.264\ in^2$$

Maximum horizontal force at 75 years

$$T_{max\_SS1\_75} := \phi_y \cdot f_y \cdot A_{SS1\_75} = 9.912\ kip$$

**EarthTrac SS1 Soil Reinforcing (2" x 3/16") - 100 years**

Thickness with effects of corrosion at 100 years

$$t_{SS1\_100} := \left(t_{SS1} - E_{100}\right) = 0.109\ in$$

Area of strip with effects of corrosion at 100 years

$$A_{SS1\_100} := t_{SS1\_100} \cdot w_{SS1} = 0.217\ in^2$$

Maximum horizontal force at 100 years

$$T_{max\_SS1\_100} := \phi_y \cdot f_y \cdot A_{SS1\_100} = 8.149\ kip$$

**EarthTrac SS3 Soil Reinforcing (2" x 5/32") - Parameters**

Thickness of Strip ........................................................................................ $t_{SS3} := \left(\frac{5}{32}\right) \cdot in$

Width of strip ........................................................................................ $w_{SS3} := 2 \cdot in$

Area of strip no corrosion

$$A_{SS3} := t_{SS3} \cdot w_{SS3} = 0.313\ in^2$$

Maximum horizontal force no corrosion

$$T_{max\_SS3} := \phi_y \cdot f_y \cdot A_{SS3} = 11.719\ kip$$

**EarthTrac SS3 Soil Reinforcing (2" x 5/32") - 75 years**

Thickness with effects of corrosion at 75 years

$$t_{SS3\_75} := \left(t_{SS3} - E_{75}\right) = 0.101\ in$$

Area of strip with effects of corrosion at 75 years

$$A_{SS3\_75} := t_{SS3\_75} \cdot w_{SS3} = 0.202\ in^2$$

Maximum horizontal force at 75 years

$$T_{max\_SS3\_75} := \phi_y \cdot f_y \cdot A_{SS3\_75} = 7.568\ kip$$

**EarthTrac Soil SS3 Reinforcing (2" x 5/32") - 100 years**

Thickness with effects of corrosion at 100 years

$$t_{SS3\_100} := \left(t_{SS3} - E_{100}\right) = 0.077\ in$$

Area of strip with effects of corrosion at 100 years

$$A_{SS3\_100} := t_{SS3\_100} \cdot w_{SS1} = 0.155\ in^2$$

Maximum horizontal force at 100 years

$$T_{max\_SS3\_100} := \phi_y \cdot f_y \cdot A_{SS3\_100} = 5.806\ kip$$

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# 1.2.6
# Soil Reinforcing Corrosion Testing

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**1.2.7**
**Soil Reinforcing Dimensional Tolerances**

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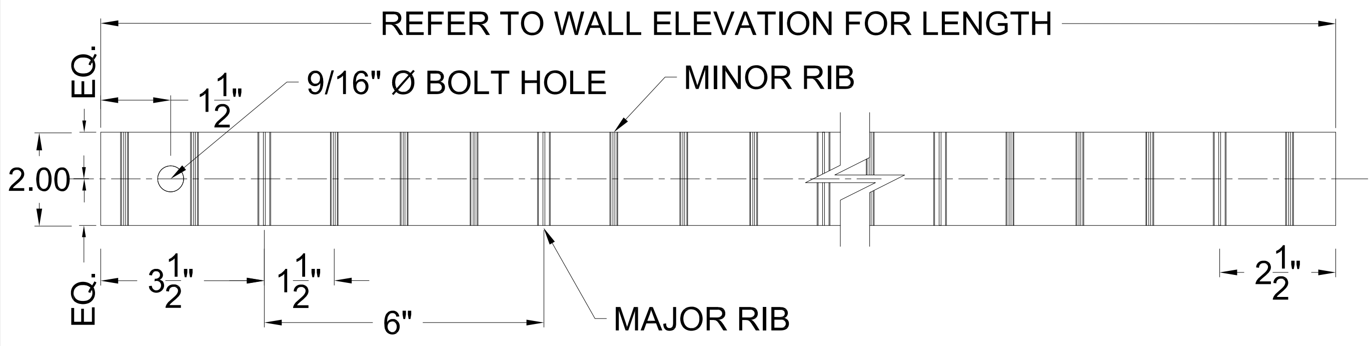
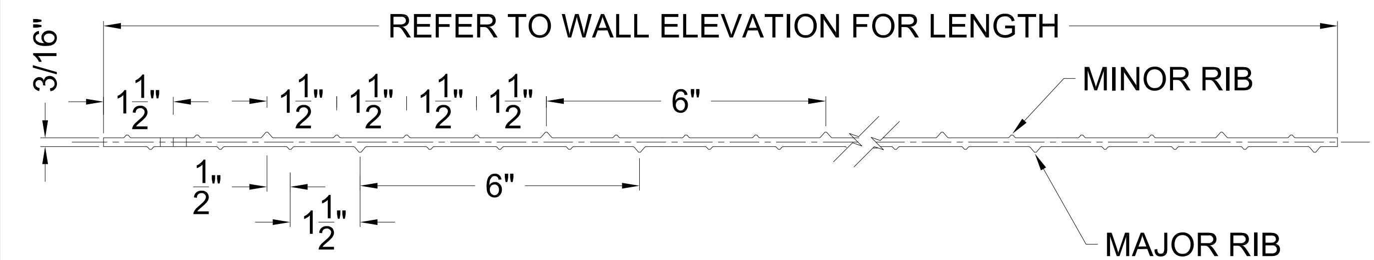
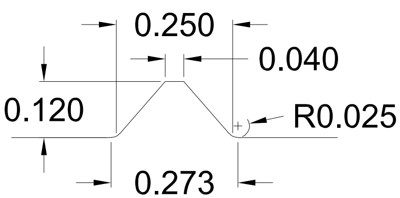


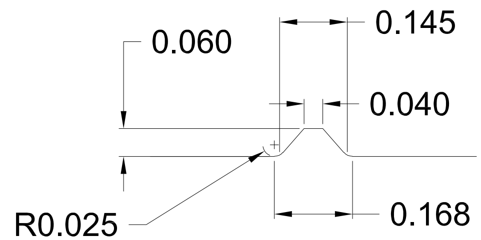

PLAN DETAIL OF EARTHTRAC STEEL STRIP (SS1)




SIDE DETAIL OF EARTHTRAC STEEL STRIP (SS1)




EARTHTRAC SS1 MAJOR RIB




EARTHTRAC SS1 MINOR RIB

1. Specifications
   1.1. ASTM A572 - Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel.
2. Acceptance

   All material will be accepted on the basis of the required certification and testing required by the ASTM specifications.
3. Material

   The soil reinforcing shall be Grade 50 steel and have a minimum tensile capacity of 65 ksi and a minimum yield capacity of 50 ksi.
4. Tolerances

   The permissible variation from the dimensions and configuration shown shall be as follows:
   4.1. Overall Plate Vertical Dimensions$\pm \frac{1}{8}$
   4.2. Overall Plate Horizontal Dimensions$\pm \frac{1}{8}$
   4.3. Overall Hole Dimension$\pm \frac{1}{64}$
   4.4. Overall Edge Distance$\pm \frac{1}{32}$
5. Certification

   The manufacturer shall furnish certification of all material. The certification shall indicate that the components are in accordance with this specification and shall include typical test results representative of the material. The certification shall be notarized.

| GIS PN | Project Engineer | | |
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| | Checked By: | Date Checked | |
| | Approved By: | Date Approved | |

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EARTHTRAC SS1

Fabrication Drawing

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SS1

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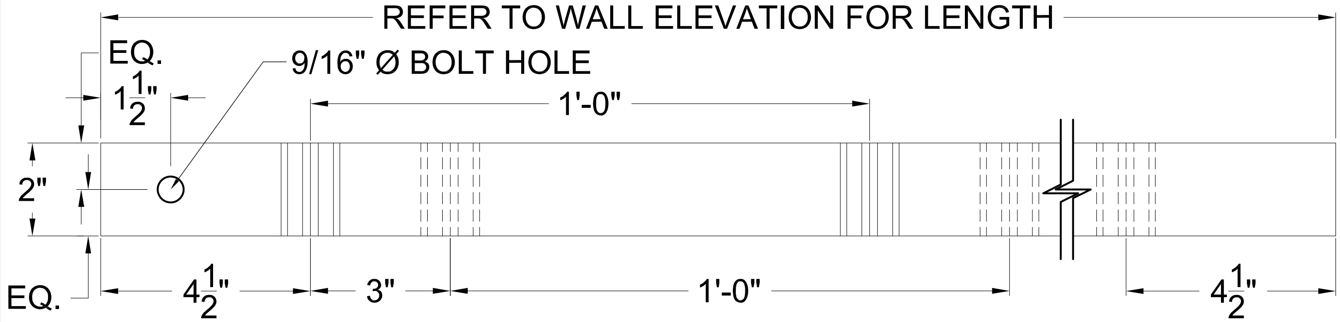
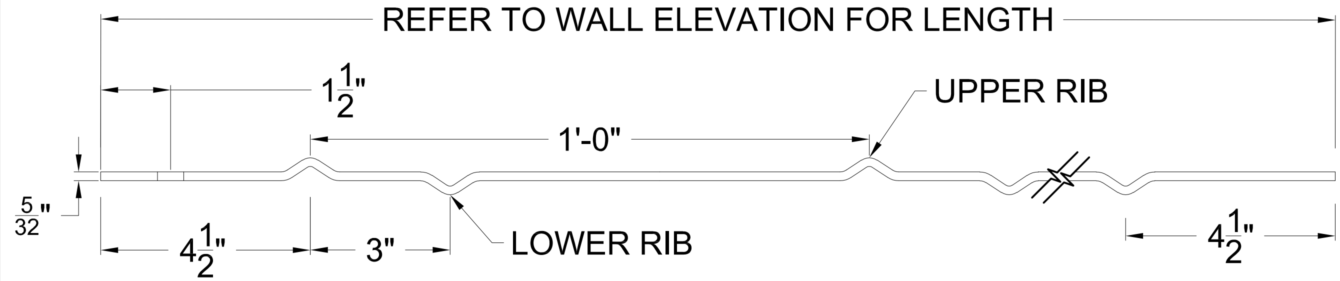
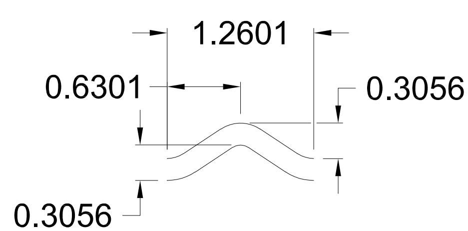



PLAN DETAIL OF EARTHTRAC STEEL STRIP STRIP (SS3)




SIDE DETAIL OF EARTHTRAC STEEL STRIP STRIP (SS3)




EARTHTRAC SS3 RIB

1. Specifications
   1.1. ASTM A1011 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.
2. Acceptance

   All material will be accepted on the basis of the required certification and testing required by the ASTM specifications.
3. Material

   The soil reinforcing shall be Grade 50 steel and have a minimum tensile capacity of 65 ksi and a minimum yield capacity of 50 ksi.
4. Tolerances

   The permissible variation from the dimensions and configuration shown shall be as follows:
   4.1. Overall Plate Vertical Dimensions$\pm\frac{1}{8}$
   4.2. Overall Plate Horizontal Dimensions$\pm\frac{1}{8}$
   4.3. Overall Hole Dimension$\pm\frac{1}{64}$
   4.4. Overall Edge Distance$\pm\frac{1}{32}$
5. Certification

   The manufacturer shall furnish certification of all material. The certification shall indicate that the components are in accordance with this specification and shall include typical test results representative of the material. The certification shall be notarized.

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| SCALE N.T.S. | |
| Project Engineer | |
| Drawn By tpt | Date Drawn |
| Checked By: | Date Checked |
| Approved By: | Date Approved |

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EARTHTRAC SS3

Fabrication Drawing

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## 1.2.8
## Soil Reinforcing Connection

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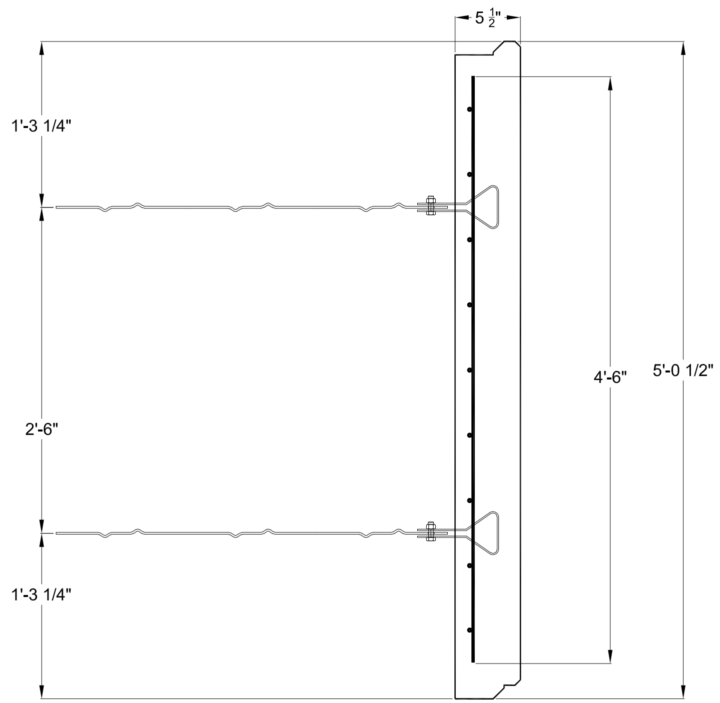
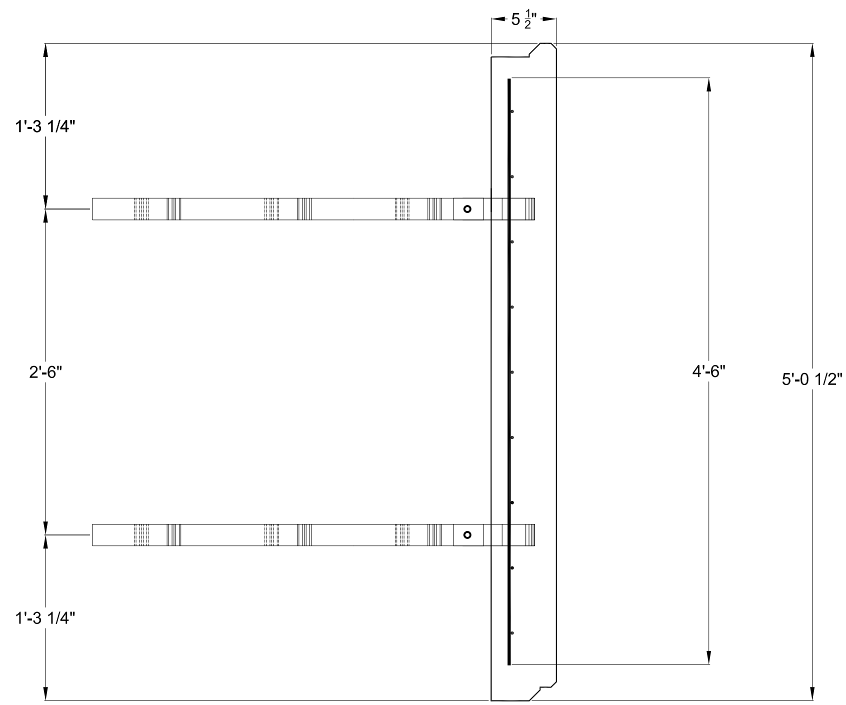
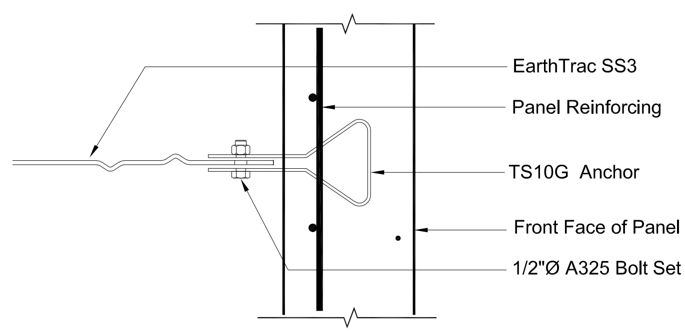


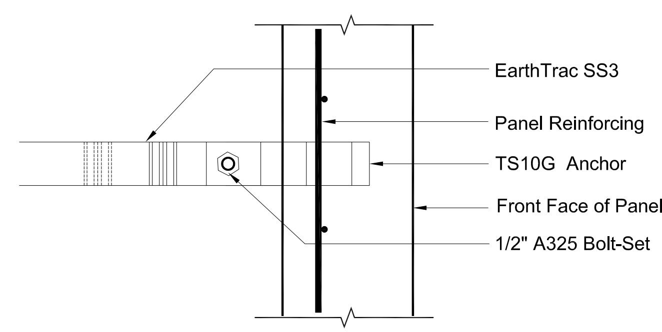

STANDARD PANEL - G
SECTION DETAIL




STANDARD PANEL - G
PLAN SECTION




EARTHTRAC CONNECTION AT TS10G ANCHOR
CROSS SECTION




EARTHTRAC CONNECTION AT TS10G ANCHOR
PLAN VIEW

| GIS PN | Project Engineer | |
|---|---|---|
| SCALE N.T.S. | Drawn By tpt | Date Drawn |
| | Checked By: | Date Checked |
| | Approved By: | Date Approved |

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EARTHTRAC SS CONNECTION AT TS10G PANEL ANCHOR

Connection Detail Drawing

GSI

SS

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**1.2.9**
**Soil Reinforcing Connection Components**

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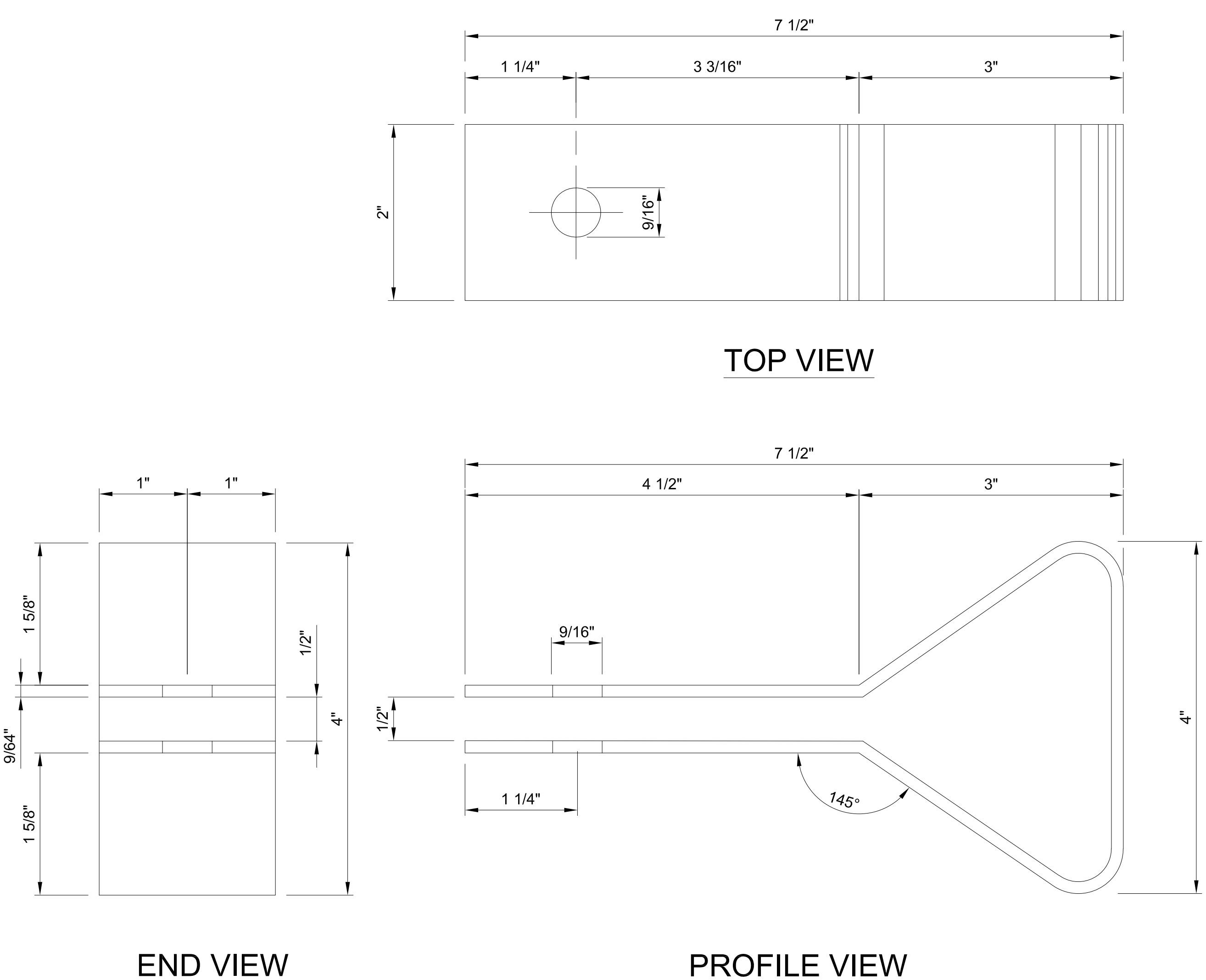


1. Specifications
   1.1. ASTM A1011 Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, and Ultra-High Strength.
2. Acceptance

   All material will be accepted on the basis of the required certification and testing required by the ASTM specifications.
3. Material

   The TS10G Anchor shall be Grade 50 steel and have a minimum tensile capacity of 65 ksi and a minimum yield capacity of 50 ksi.
4. Tolerances

   The permissible variation from the dimensions and configuration shown shall be as follows:

   4.1. Overall Plate Vertical Dimensions$\pm\frac{1}{8}$

   4.2. Overall Plate Horizontal Dimensions$\pm\frac{1}{8}$

   4.3. Overall Hole Dimension$\pm\frac{1}{64}$

   4.4. Overall Edge Distance$\pm\frac{1}{32}$
5. Certification

   The manufacturer shall furnish certification of all material. The certification shall indicate that the components are in accordance with this specification and shall include typical test results representative of the material. The certification shall be notarized.

| GIS PN | Project Engineer | |
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| SCALE N.T.S. | Drawn By tpt | Date Drawn |
| | Checked By: | Date Checked |
| | Approved By: | Date Approved |

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Panel Anchor
Unit Wt. = 1.41 lbs

Fabrication Drawing

GSI

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# 1.2.10
# Soil Reinforcing Connection Properties



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**1.2.11**
**Soil Reinforcing Connection Corrosion Protection**

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# 1.2.12
# Soil Reinforcing Connection Sacrificial Steel as a Function of Service Life

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**1.2.13**
**Soil Reinforcing Connection Corrosion Testing**

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# 1.2.14
# Soil Reinforcing Connection Dimensional Tolerances



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ARLINGTON, TX 76011

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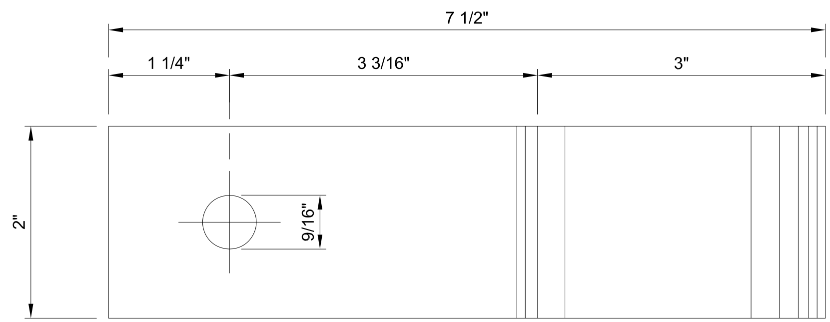
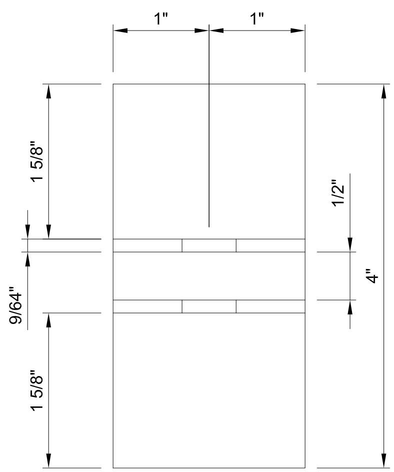
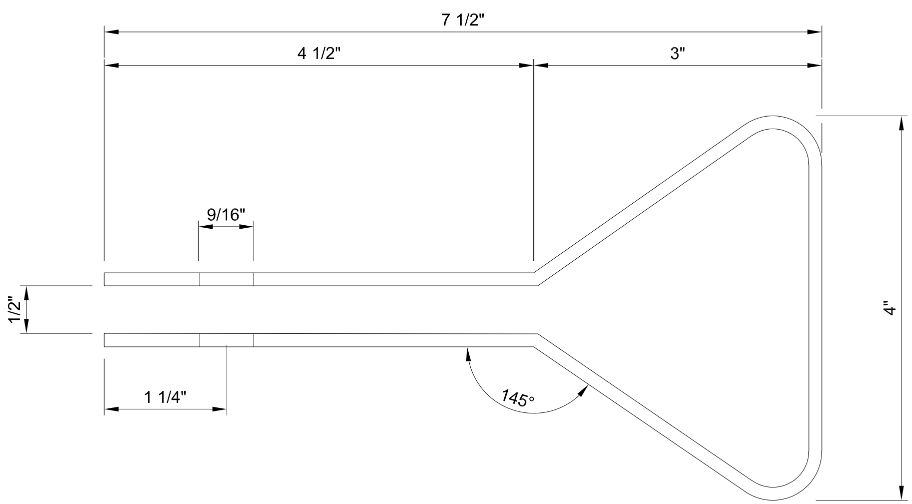



TOP VIEW




END VIEW




PROFILE VIEW

1. Specifications
   1.1. ASTM A1011 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.
2. Acceptance

   All material will be accepted on the basis of the required certification and testing required by the ASTM specifications.
3. Material

   The TS10G Anchor shall be Grade 50 steel and have a minimum tensile capacity of 65 ksi and a minimum yield capacity of 50 ksi.
4. Tolerances

   The permissible variation from the dimensions and configuration shown shall be as follows:

   4.1. Overall Plate Vertical Dimensions$\pm \frac{1}{8}$

   4.2. Overall Plate Horizontal Dimensions$\pm \frac{1}{8}$

   4.3. Overall Hole Dimension$\pm \frac{1}{64}$

   4.4. Overall Edge Distance$\pm \frac{1}{32}$
5. Certification

   The manufacturer shall furnish certification of all material. The certification shall indicate that the components are in accordance with this specification and shall include typical test results representative of the material. The certification shall be notarized.

| | | |
|---|---|---|
| GIS PN | Project Engineer | |
| SCALE N.T.S. | Drawn By tpt | Date Drawn |
| | Checked By: | Date Checked |
| | Approved By: | Date Approved |

GIS Support Through Innovation

EARTHTEC

**GROUND IMPROVEMENT SYSTEMS LLC**
114 South Collins Street
Arlington, TX. 76011
(817) 222-0969
www.groundimprovementsystems.com

Panel Anchor
Unit Wt. = 1.41 lbs

Fabrication Drawing

GSI

PA

H:\TOMS COMPUTER\AB MY DATA\DOT_AB LRFD BINDER\1. EARTHTRAC\2. SYSTEM DETAILS\EARTHTRAC_FABRICATION DRAWINGS 20200808.DWG

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**1.2.15**
**Soil Reinforcing Connection Strength and Testing**

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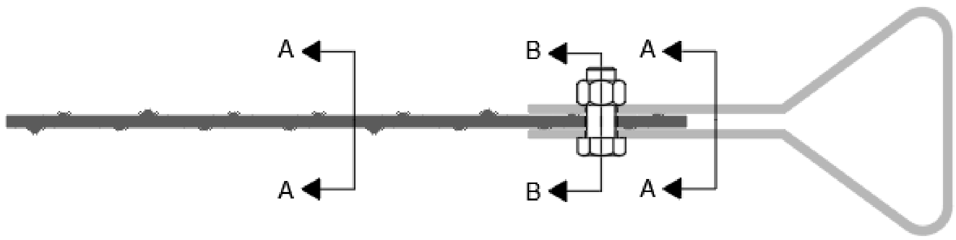
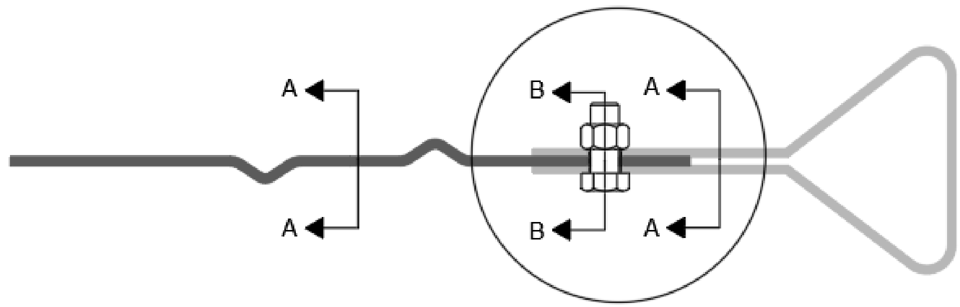
# DESIGN CONNECTION RESULTS FOR SS1 AND SS3




**EarthTrac SS1 - Inextensible Steel Strip**




**EarthTrac SS3 - Inextensible Steel Strip**

**EarthTrac Connection Calculation and Equivalent Properties**

| Element | Strip | | Connection | | Anchor | Equivalent | |
|---|---|---|---|---|---|---|---|
| Location | A-A | B-B | Plate B-B | Bolt | Pullout | Area$_C$ | Thickness$_C$ |
| SS1 | 9.90 kip | **8.88** kip | 10.72 kip | 9.62 kip | 12.14 kip | 0.237 in2 | 0.118 in |
| SS3 | **7.56** kip | 10.51 kip | 10.72 kip | 9.62 kip | 12.14 kip | 0.202 in2 | 0.101 in |

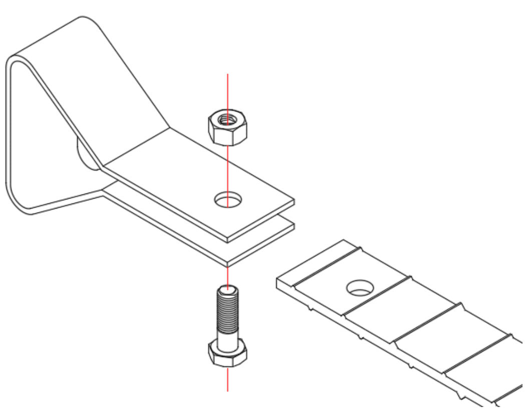
- c denotes that corrosion has been considered

The equivalent area for a steel strip is the area of the limiting allowable tension based on the calculations of the soil reinforcing and the connection plate. The area and thickness that will be used in calculations for MSEW and for the design of the EarthTec EarthTrac System. The limiting factor is the bolt hole at the connection in conformance with AASHTO 2020 Article 6.8.2 and equation 6.8.2.1-2 using a reduction for punched holes equal to 0.90. The yield strength of the steel strip and connection plate is 50 ksi. Values shown above are for a 75 year design life.

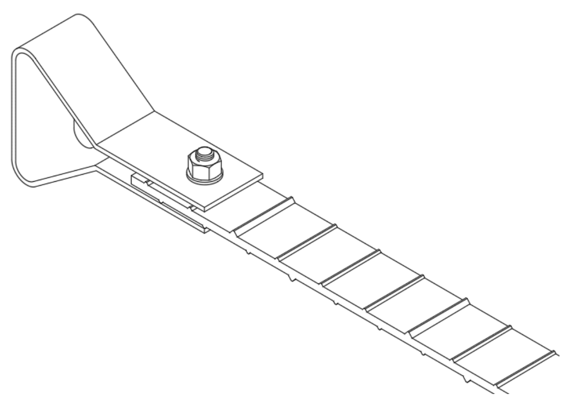
# EARTHTRAC SS1 CONNECTION CALCULATION

| | | | |
|---|---|---|---|
| Project Name: | EARTHTRAC SS1 CONNECTION CALCULATION | | |
| Project Location: | Grade 50 Steel with Panel Anchor | | |
| Project Number: | | | |
| Design Code: | AASHTO | Design Method: | LRFD |
| DSGN By: TPT | Date: 20210116 | CHCK By: | Date: |

This worksheet contains the calculation method for the connection of the EarthTrac SS1 to the standard Panel anchor. The EarthTrac SS1 soil reinforcing consists of a hot-rolled steel strip with major and minor ribs. The steel strip has a 9/16" hole punched in the proximal end so it can be bolted to a panel anchor. The panel anchor consists of a special fabricated steel plate. The plate is fabricated to include two extending parallel plates each with a 9/16" through bore that will accept a bolt. The bolt-set contains an F3125-Grade A325 ½" diameter bolt, and nut. The bolt is placed so the nut is on the top side of the panel anchor. The bolt is tightened with a wrench to snug-tight.



**Explode Connection**



**Connection**

## Degradation Design Rates

AASHTO requires that all components buried in soil be designed assuming a degrade steel area. The degradation rates are specified in AASHTO in section 11.10.6.4.2. A 75 year design life will be used here since the design load will be at maximum.

Thickness of galvanized coating (ASTM A123 2 oz/ft$^2$)............................................... $g_T := 3.4 \cdot mil$

Structure design life........................................................................................ $Y_t := 75 \cdot yr$

Zinc loss rate (first two years): $g_2 := 0.58 \cdot \frac{mil}{yr}$

Zinc loss rate (remaining years): $g_R := 0.16 \cdot \frac{mil}{yr}$

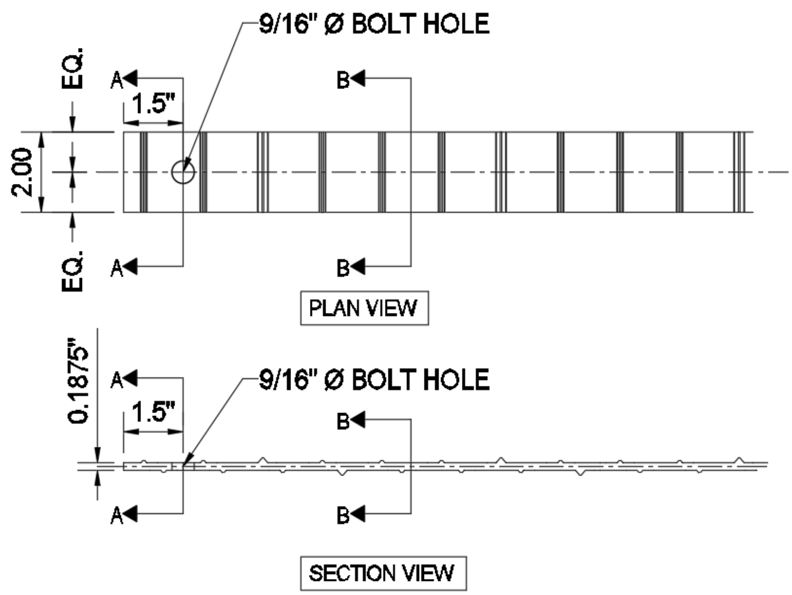
Steel loss rate (0-75 years) : $E_{S75} := 0.47 \cdot \frac{mil}{yr}$

Design Life of Galvanized Coating : $Y_G := \frac{g_T - 2 \cdot yr \cdot g_2}{g_R} + 2 \cdot yr = 16 \cdot yr$

Sacrificial Steel: $E_c := \text{if}\left[Y_t \le Y_G, 0, \left[\left(Y_t - Y_G\right) \cdot E_{S75}\right] \cdot 2\right] = 0.055 \cdot in$

**Design Model for EarthTrac Soil Reinforcing**

The bolt that is used to tie the plate to the panel anchor is an F3125 Grade A325 ½ diameter bolt and nut.




**AASHTO Tensile Design 6.8.2.1**

The factored tensile resistance, $P_r$, shall be taken as the lesser of the values as given by equations 6.8.2.1-1 and 6.8.2.1-2.

$$P_r = \phi_y \cdot F_y \cdot A_g \quad \text{AASHTO 6.8.2.1-1}$$

$$P_r = \phi_u \cdot F_u \cdot A_n \cdot R_p \cdot U \quad \text{AASHTO 6.8.2.1-2}$$

Where:

$P_r$ = factored tensile resistance

$\phi_y$ = resistance factor for yielding of tension member (0.75) - Article 11.5.7

$\phi_u$ = resistance factor for fracture of tension member (0.80) - Article 6.5.4.2

$F_y$ = specified minimum yield strength (ksi)

$F_u$ = specified tensile strength (ksi)

$A_g$ = gross cross-sectional area of member ($in^2$)

$A_n$ = net area of the member ($in^2$) – Article 6.8.3

$R_p$ = reduction factor for punched full size holes – Article 6.8.2.2

$U$ = reduction factor to account for shear lag

**CALCULATIONS**

Yield coefficient for soil reinforcing (AASHTO Table 11.5.7-1 ) ............................... $\phi_y := 0.75$

Specified yield strength of soil reinforcing .......................................................... $F_y := 50 \cdot ksi$

Specified ultimate tensile strength of ASTM A572 Steel .................................... $F_u := 65 \cdot ksi$

Width of EarthTrac SS1 ..................................................................................... $w_{SS1} := 2 \cdot in$

Thickness of EarthTrac SS1 ............................................................................... $t_{SS1} := 0.1875 \cdot in$

**1. Rupture of EarthTrac SS1 at Section B-B**

Design Area

$$A_{SS1\_c} := w_{SS1} \cdot \left(t_{SS1} - E_c\right) = 0.264 \cdot in^2$$

Maximum tensile resistance of EarthTrac SS1

$$T_{max} := \phi_y \cdot F_y \cdot A_{SS1\_c} = 9.903 \cdot kip$$

**Material Properties**

Resistance factor for tension fracture in net section (AASHTO 6.5.4.2)...................... $\phi_u := 0.80$

Resistance factor for yielding in gross section (AASHTO 6.5.4.2)............................ $\phi_y := 0.75$

Reduction factor for shear lag (AASHTO 6.8.2.1) ................................................ $U := 1.00$

Reduction factor for punched holes (AASHTO 6.8.2.1) ....................................... $R_p := 0.90$

A325 bolt diameter ........................................................................................ $d_b := 0.500 \cdot in$

**Connector Section**

Net section of Strip (A-A)

$$A_n := \left(w_{SS1} - d_b - \frac{1}{16} in\right) \cdot \left(t_{SS1} - E_c\right) = 0.19 \cdot in^2$$ (Assumes faying surface is not in contact)

Gross section of Strip (B-B)

$$A_g := \left(w_{SS1}\right) \cdot \left(t_{SS1} - E_c\right) = 0.264 \cdot in^2$$ (Assumes faying surface is not in contact)

Factored Tensile Resistance

Net area

$$P_{Pr\_n} := \phi_u \cdot F_u \cdot A_n \cdot R_p \cdot U = 8.883 \cdot kip$$

Gross area

$$P_{Pr\_g} := \phi_y \cdot F_y \cdot A_g = 9.903 \cdot kip$$

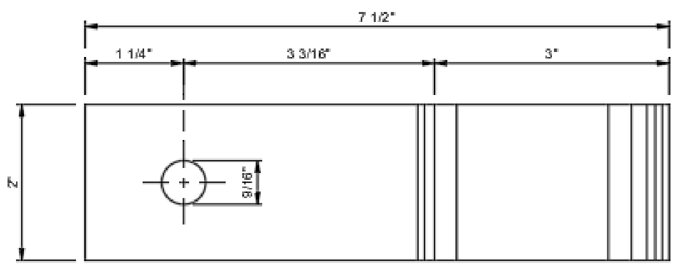
Maximum factored tensile resistance of Plate connector

$$P_P := \text{if}\left(P_{Pr\_g} > P_{Pr\_n}, P_{Pr\_n}, P_{Pr\_g}\right) = 8.883 \cdot kip$$ **Controls**

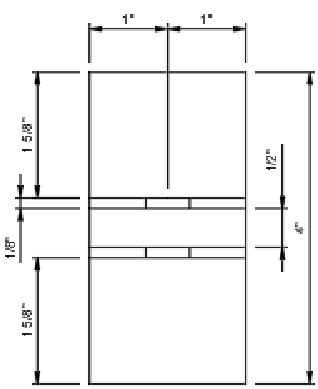
Equivalent area is equal to 0.238 $in^2$

## Design Model for Tie Strip

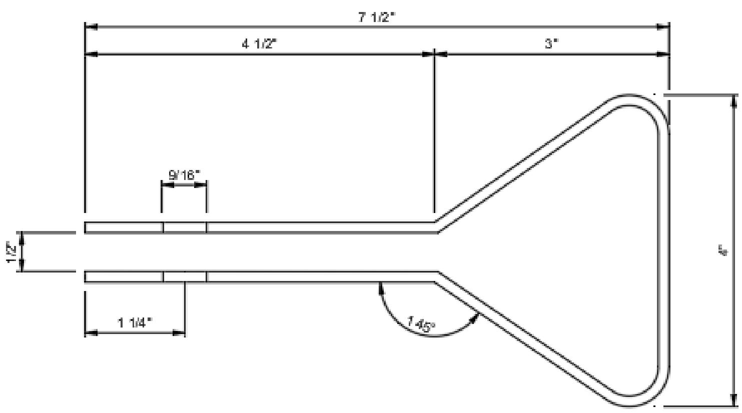
The bolt that is used to connect the plate to the panel anchor is an F3125-Grade A325 ½ diameter bolt.




TOP VIEW




END VIEW




PROFILE VIEW

**200/135 DPTS Anchor Properties (ASTM A1101 Grade 50 - 10 gauge)**

Width of Panel Anchor ........................................................................... $W_A := 2.00 \cdot in$

Thickness of Panel Anchor ........................................................................ $t_A := 0.135 \cdot in$

Number of plates in anchors ...................................................................... $n_A := 2$

Specified ultimate Tensile Strength of Grade 50 Steel ........................................ $F_{u\_A} := 65 \cdot ksi$

Specified yield Tensile Strength of Grade 50 Steel ............................................. $F_{y\_A} := 50 \cdot ksi$

Resistance factor for tension fracture in net section (AASHTO 6.5.4.2) ................ $\phi_u := 0.80$

Resistance factor for yielding in gross section (AASHTO 6.5.4.2) ....................... $\phi_y := 0.75$

Effective Cross Sectional Area of Anchor:

$$A_{se} := W_A \cdot t_A = 0.27 \cdot in^2$$

**Panel Anchor Shear**

Gross section of plate

$$A_{A\_g} := \left(W_A\right) \cdot \left(t_A - E_c\right) = 0.159 \cdot in^2$$

Net section of one plate at bolt hole

$$A_{A\_n} := R_p \cdot \left(W_A - d_b - E_c\right) \cdot \left(t_A - E_c\right) = 0.103 \cdot in^2$$

Maximum tension on gross section of one plate

$$P_{A\_g} := n_A \cdot \phi_y \cdot F_{y\_A} \cdot A_{A\_g} = 11.931 \cdot kip$$

Maximum tension on net section of one plate

$$P_{A\_n} := n_A \cdot \phi_u \cdot F_{u\_A} \cdot A_{A\_n} = 10.755 \cdot kip$$

Maximum factored tensile resistance of panel anchor (Upper and Lower Plate)

$$P_{r\_A} := \text{if}\left(P_{A\_g} > P_{A\_n}, P_{A\_n}, P_{A\_g}\right) = 10.755 \cdot kip$$

## AASHTO Bolt Shear Resistance Design 6.13.2.7

$$R_n = 0.45 \cdot A_b \cdot F_{ub} \cdot N_s \quad \text{AASHTO 6.13.2.7-2}$$

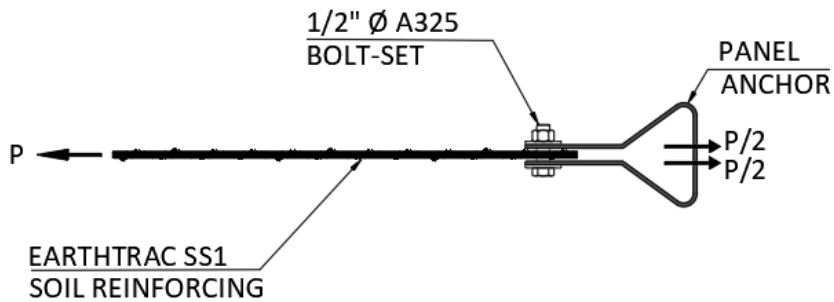
Where:

$R_n$ = nominal shear resistance

$A_b$ = area of bolt corresponding to the nominal diameter ($in^2$)

$F_{ub}$ = specified minimum tensile resistance of the bolt (ksi) – Article 6.4.3

$N_s$ = number of shear planes per bolt




The bolt that is used to connect the plate to the panel anchor is an F3125-Grade A325 ½ diameter bolt and nut. The bolt is in double shear

Tensile fracture of F3125 Grade A325 bolt ........................................................ $F_{u\_A325} := 120 \cdot ksi$

Tensile yield of F3125 Grade A325 bolt ............................................................ $F_{y\_A325} := 90 \cdot ksi$

Number of bolts for bolt shear.......................................................................... $n_b := 1$

Number of shear planes .................................................................................. $m_b := 2$

Distance from edge of hole to end of plate....................................................... $L_c := 0.969 \cdot in$

Net area of bolt with corrosion $A_b := \dfrac{\pi \cdot (d_b - E_c)^2}{4} = 0.155 \cdot in^2$

Shear resistance of bolt in double shear $R_n := 0.45 A_b \cdot F_{u\_A325} \cdot n_b \cdot m_b = 16.762 \cdot kip$

Bearing resistance of bolt hole (2-Plates) $R_b := 2 \cdot 1.2 \cdot L_c \cdot (t_A - E_c) \cdot F_{u\_A} = 12.024 \cdot kip$

Distance from center of bolt hole to edge of plate .................................................. $L_e := 1.25 \cdot in$

Thickness of plate ........................................................................................ $t_p := 0.135 \cdot in$

Nominal diameter of bolt hole ............................................................................ $d_{bh} := 0.50 \cdot in$

Tensile strength of steel ...................................................................................... $F_u := 65 \cdot ksi$

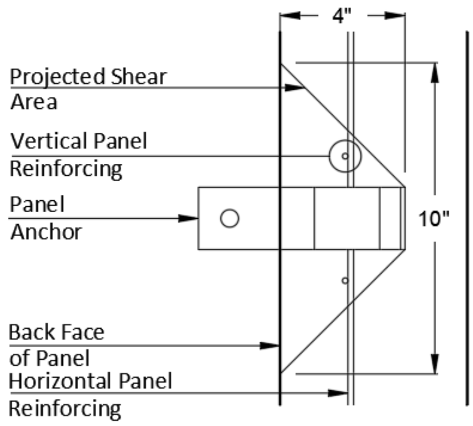
Resistance factor for bearing .............................................................................. $\phi_{bb} := 0.80$

$$R_n := 2 \cdot \phi_{bb} \cdot 1.2 \cdot \left( L_e - \frac{d_{bh} + E_c}{2} \right) \cdot \left( t_p - E_c \right) \cdot F_u = 9.651 \cdot kip$$

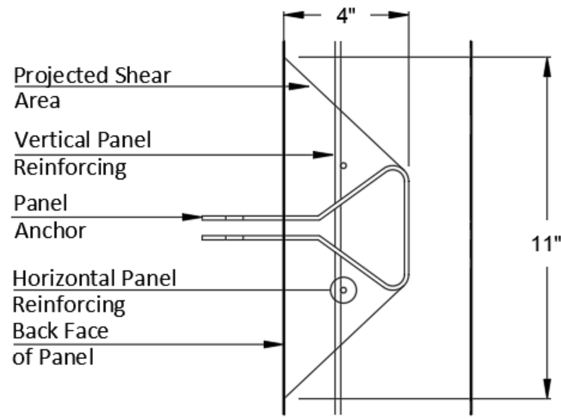
Distance from center of bolt hole to edge of strip .................................................. $L_{eS} := 1.50 \cdot in$

## Design Model for Anchor Pullout in Concrete

The pullout of the anchor from the concrete will be calculated in conformance with ACI Appendix D




Plan View




Section View

**Segmental Concrete Panel Material Properties**

Compressive strength of concrete........................................................................ $f_c := 4000 \cdot psi$

Yield strength of reinforcing................................................................................. $F_{ys} := 65 \cdot ksi$

Number of anchors ............................................................................................ $n := 1$

Depth of embedment of anchor........................................................................... $h_{ef} := 4.00 \cdot in$

Prismatic Area of concrete Shear $A_{brg} := 11 \cdot in \cdot 10 \cdot in = 110 \cdot in^2$

**D.4** General requirements for strength of anchors

**D4.4** Strength Reduction Factor:

a) Anchor governed by strength of a ductile steel..................................... $\phi_s := 0.75$

b) Anchor governed by concrete breakout....................................... $\phi_c := 0.75$

**D.5** Design requirements for tensile loading

**D.5.1** Steel Strength of Anchor in Tension

**D.5.1.2** Nominal Strength of Group of Anchors

$$\phi N_s := \phi_s \cdot n \cdot A_{se} \cdot F_{u\_A} = 13.163 \cdot kip \qquad \textbf{\textit{(D - 3)}}$$

**D.5.2** Concrete breakout strength of anchor in tension

**D.5.2.4** Modification factor for eccentrically loaded anchor

$\psi_1 := 1.0$ no eccentricity

**D.5.2.5** Modification factor for edge effects

$\psi_2 := 1.0$ no edge effects

**D.5.2.6** Modification factor with no cracking

$\psi_3 := 1.25$ cast-in-anchors

**D 5.2.2** Basic concrete breakout strength

$k := 24$ for cast-in anchors

$$N_b := k \cdot \sqrt{f_c \cdot psi} \cdot h_{ef}^{1.5} \cdot \left(\frac{in^2}{in^{1.5}}\right) = 12.143 \cdot kip$$ ***(D - 7)***

**D.5.2.1** Nominal concrete breakout for single anchor

Projected area of pullout (1) (Assumes prismatic projected area)

$$A_{NO} := \left[2 \cdot (1.5) \cdot h_{ef}\right] \cdot \left[2 \cdot (1.5) \cdot h_{ef}\right] = 144 \cdot in^2$$

Projected area of pullout (2) (Effects of edge distance)

$c_1 := 7.5 \cdot in$

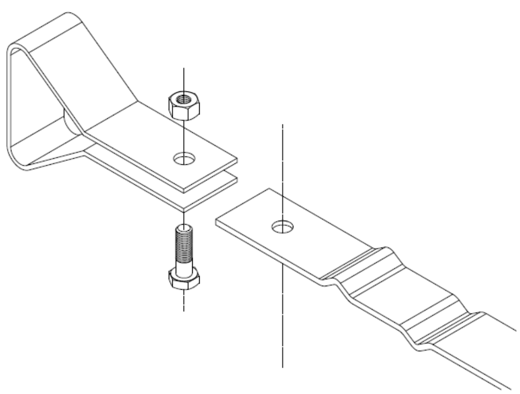
$$A_N := \left(c_1 + 1.5 \cdot h_{ef}\right) \cdot \left[2 \cdot (1.5) \cdot h_{ef}\right] = 162 \cdot in^2$$

$$\phi N_{cb} := \phi_s \cdot \frac{A_N}{A_{NO}} \cdot \psi_2 \cdot \psi_3 \cdot N_b = 12.807 \cdot kip$$ ***(D - 4)***

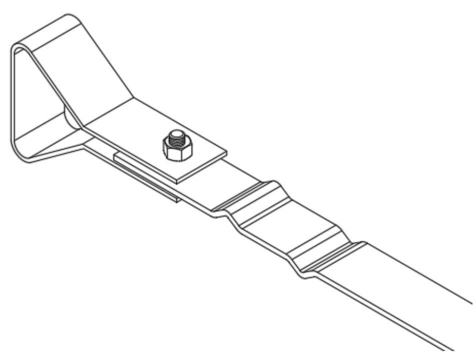
# EARTHTRAC SS3 CONNECTION CALCULATION

| | | | |
|---|---|---|---|
| Project Name: | EARTHTRAC SS3 CONNECTION CALCULATION | | |
| Project Location: | A1011 Grade 50 Steel SS3 and Panel Anchor | | |
| Project Number: | | | |
| Design Code: | AASHTO | Design Method: | LRFD |
| DSGN By: TPT | Date: 20210308 | CHCK By: | Date: |

This worksheet contains the calculation method for the connection of the EarthTrac SS3 to the standard Panel anchor. The EarthTrac SS3 soil reinforcing consists of a hot-rolled steel strip with upper and lower ribs. The steel strip has a 9/16" hole punched in the proximal end so it can be bolted to a panel anchor. The panel anchor consists of a special fabricated steel plate. The plate is fabricated to include two extending parallel plates each with a 9/16" through bore that will accept a bolt. The bolt-set contains an F3125 Grade A325 ½" dimeter bolt and nut. The bolt is placed so the nut is on the top side of the panel anchor. The bolt is tightened with a wrench to snug-tight. In a snug-tight connection the faying surfaces are brought into contact.



**Exploded Connection**



**Connection**

## Degradation Design Rates

AASHTO requires that all components buried in soil be designed assuming a decreased steel area. The degradation rates are specified in AASHTO in section 11.10.6.4.2. A 75 year design life will be used here since the design load will be at maximum.

Thickness of galvanized coating (ASTM A123 2 oz/ft$^2$)................................................ $g_T := 3.4 \cdot mil$

Structure design life............................................................................................ $Y_t := 75 \cdot yr$

Zinc loss rate (first two years): $g_2 := 0.58 \cdot \frac{mil}{yr}$

Zinc loss rate (remaining years): $g_R := 0.16 \cdot \frac{mil}{yr}$

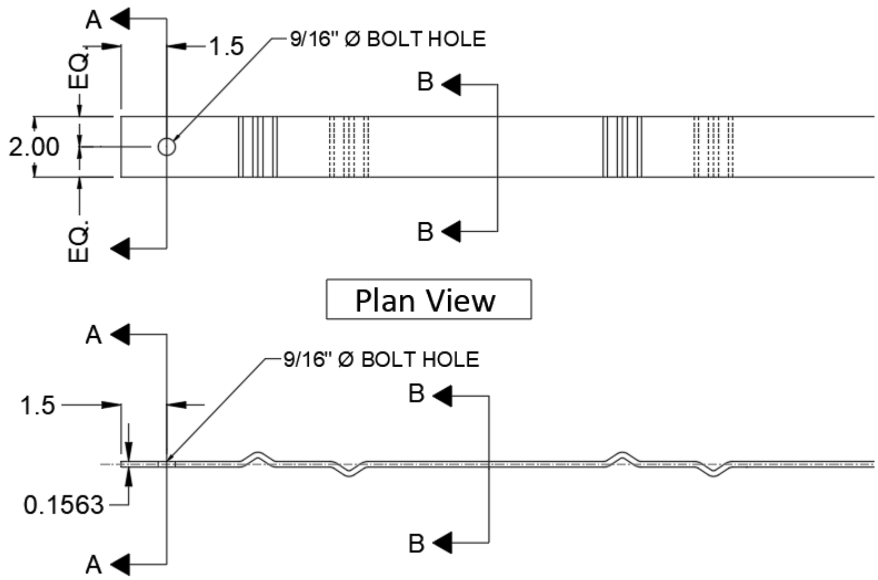
Steel loss rate (0-75 years) : $E_{S75} := 0.47 \cdot \frac{mil}{yr}$

Design Life of Galvanized Coating : $Y_G := \frac{g_T - 2 \cdot yr \cdot g_2}{g_R} + 2 \cdot yr = 16 \cdot yr$

Sacrificial Steel: $E_S := \text{if}\left[Y_t \le Y_G, 0, \left[\left(Y_t - Y_G\right) \cdot E_{S75}\right] \cdot 2\right] = 0.055 \cdot in$

**Design Model for EarthTrac Soil Reinforcing**

The bolt that is used to attach the plate to the panel anchor is an F3125 Grade A325 ½ diameter bolt. The hole in the plate is assumed to be punched.




**AASHTO Tensile Design 6.8.2.1**

The factored tensile resistance, $P_r$, shall be taken as the lesser of the values as given by equations 6.8.2.1-1 and 6.8.2.1-2.

$$P_r = \phi_y \cdot F_y \cdot A_g \quad \text{AASHTO 6.8.2.1-1}$$

$$P_r = \phi_u \cdot F_u \cdot A_n \cdot R_p \cdot U \quad \text{AASHTO 6.8.2.1-2}$$

Where:

$P_r$ = factored tensile resistance

$\phi_y$ = resistance factor for yielding of tension member (0.75) – Article 11.5.7

$\phi_u$ = resistance factor for fracture of tension member (0.80) - Article 6.5.4.2

$F_y$ = specified minimum yield strength (ksi)

$F_u$ = specified tensile strength (ksi)

$A_g$ = gross cross-sectional area of member ($in^2$)

$A_n$ = net area of the member ($in^2$) – Article 6.8.3

$R_p$ = reduction factor for punched full size holes – Article 6.8.2.1

$U$ = reduction factor to account for shear lag – Article 6.8.2.2

**CALCULATIONS**

Yield coefficient for soil reinforcing (AASHTO Table 11.5.7-1 ) ................................ $\phi_y := 0.75$

Specified yield strength of ASTM A1011-G50 soil reinforcing ................................ $F_y := 50 \cdot ksi$

Specified ultimate tensile strength of ASTM A1011-G50 .................................... $F_u := 65 \cdot ksi$

Width of EarthTrac SS3 .................................................................................. $w_{SS3} := 2 \cdot in$

Thickness of EarthTrac SS3 (9G) ........................................................................ $t_{SS3} := \frac{5}{32} \cdot in$

**1. Yield of strip in conformance with Section 11.10**

Design Area

$$A_{SS3G\_c} := w_{SS3} \cdot \left(t_{SS3} - E_S\right) = 0.202 \cdot in^2$$

Maximum tensile resistance of EarthTrac SS3

$$T_{max} := \phi_y \cdot F_y \cdot A_{SS3G\_c} = 7.559 \cdot kip$$

**Material Properties**

Resistance factor for tension fracture in net section (AASHTO 6.5.4.2)...................... $\phi_u := 0.80$

Resistance factor for yielding in gross section (AASHTO 6.5.4.2)............................ $\phi_y := 0.75$

Reduction factor for shear lag (AASHTO 6.8.2.2) ................................................ $U := 1.00$

Reduction factor for punched holes (AASHTO 6.8.2.2) ........................................ $R_p := 0.90$

A325 bolt diameter ......................................................................................... $d_b := 0.500 \cdot in$

**Strip at bolt hole**

Gross section of Strip (B-B)

$$A_g := \left(w_{SS3}\right) \cdot \left(t_{SS3}\right) = 0.313 \cdot in^2$$

(snug tight connection - faying surface in contact)

Net section of Strip (A-A)

$$A_n := \left(w_{SS3} - d_b - \frac{1}{16} \cdot in\right) \cdot \left(t_{SS3}\right) = 0.225 \cdot in^2$$

(snug tight connection - faying surface in contact)

Factored Tensile Resistance

net area of strip (A-A)

$$P_{Pr\_n} := \phi_u \cdot F_u \cdot A_n \cdot R_p \cdot U = 10.512 \cdot kip$$ AASHTO 6.8.2.1-2

gross area of strip (B-B)

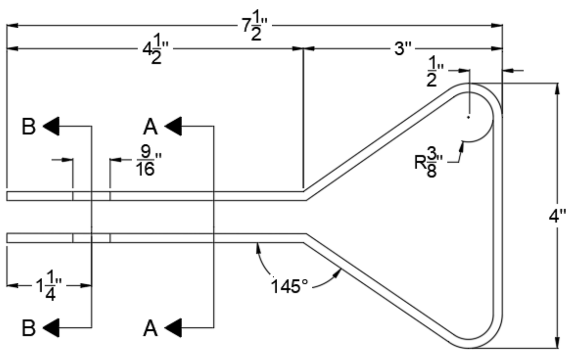
$$P_{Pr\_g} := \phi_y \cdot F_y \cdot A_g = 11.719 \cdot kip$$ AASHTO 6.8.2.1-1

Maximum factored tensile resistance of strip at connection

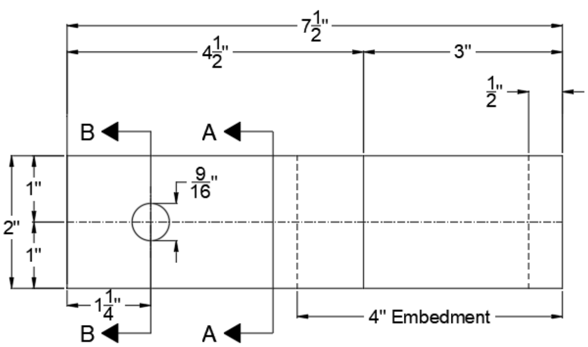
$$P_P := \text{if}\left(P_{Pr\_g} > P_{Pr\_n}, P_{Pr\_n}, P_{Pr\_g}\right) = 10.512 \cdot kip$$

## Design Model for Tie Strip

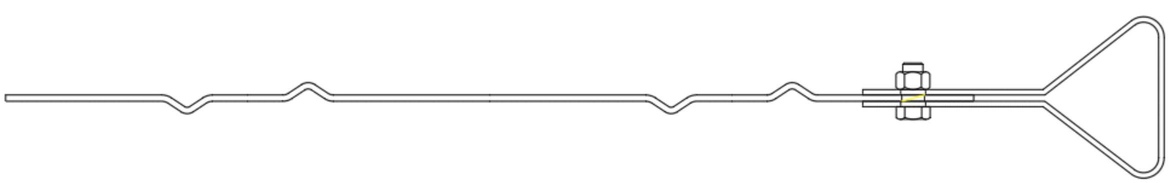
The bolt that is used to connect the plate to the panel anchor is a ½ diameter bolt. The hole in the plate is assumed to be punched.




Cross Section




Plan View



Connection

**TS10G Anchor Properties (ASTM A1101 Grade 50 - 10 gauge)**

Width of Panel Anchor ........................................................................... $W_A := 2.00 \cdot in$

Thickness of Panel Anchor ..................................................................... $t_A := 0.135 \cdot in$

Number of plates in anchors .................................................................. $n_A := 2$

Specified ultimate Tensile Strength of Grade 50 Steel ........................................ $F_{u\_A} := 65 \cdot ksi$

Specified yield Tensile Strength of Grade 50 Steel ............................................ $F_{y\_A} := 50 \cdot ksi$

Resistance factor for tension fracture in net section (AASHTO 6.5.4.2) ................. $\phi_u := 0.80$

Resistance factor for yielding in gross section (AASHTO 11.5.7) ........................... $\phi_y := 0.75$

Effective Cross Sectional Area of Anchor:

$$A_{se} := W_A \cdot t_A = 0.27 \cdot in^2$$

**Panel Anchor Shear Section at Connection**

Gross section of plate

$$A_{A\_g} := \left(W_A\right) \cdot \left(t_A - E_S\right) = 0.159 \cdot in^2$$

Net section of one plate

$$A_{A\_n} := \left(W_A - d_b - \frac{1}{16} \cdot in\right) \cdot \left(t_A - E_S\right) = 0.114 \cdot in^2$$

Maximum tension on gross section of one plate

$$P_{A\_g} := n_A \cdot \phi_y \cdot F_{y\_A} \cdot A_{A\_g} = 11.931 \cdot kip$$

Maximum tension on net section of one plate

$$P_{A\_n} := n_A \cdot \phi_u \cdot F_{u\_A} \cdot A_{A\_n} \cdot U \cdot R_p = 10.702 \cdot kip$$

Maximum factored tensile resistance of panel anchor (Upper and Lower Plate)

$$P_{r\_A} := \text{if}\left(P_{A\_g} > P_{A\_n}, P_{A\_n}, P_{A\_g}\right) = 10.702 \cdot kip$$

## AASHTO Bolt Shear Resistance Design 6.13.2.7

$$R_n = 0.45 \cdot A_b \cdot F_{ub} \cdot N_s$$ AASHTO 6.13.2.7-2

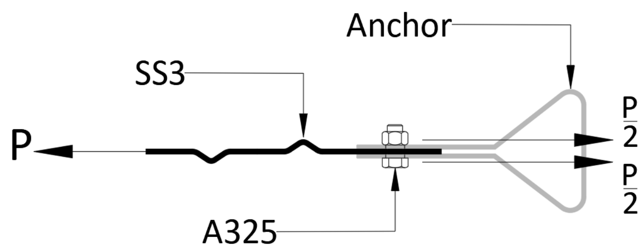
Where:

$R_n$ = nominal shear resistance

$A_b$ = area of bolt corresponding to the nominal diameter ($in^2$)

$F_{ub}$ = specified minimum tensile resistance of the bolt (ksi) – Article 6.4.3

$N_s$ = number of shear planes per bolt




The bolt that is used to connect the plate to the panel anchor is a ½ diameter bolt. The hole in the plate is assumed to be punched. The bolt is in double shear

Tensile fracture of F3125 Grade A325 bolt ........................................................ $F_{u\_A325} := 120 \cdot ksi$

Tensile yield of F3125 Grade A325 bolt ............................................................ $F_{y\_A325} := 90 \cdot ksi$

Number of bolts for bolt shear........................................................................ $n_b := 1$

Number of shear planes ................................................................................ $m_b := 2$

Distance from edge of hole to end of plate....................................................... $L_c := 0.969 \cdot in$

Net area of bolt with corrosion $A_b := \dfrac{\pi \cdot (d_b - E_S)^2}{4} = 0.155 \cdot in^2$

Shear resistance of bolt in double shear $R_n := 0.45 A_b \cdot F_{u\_A325} \cdot n_b \cdot m_b = 16.762 \cdot kip$

Bearing resistance of bolt hole (2-Plates) $R_b := 2 \cdot 1.2 \cdot L_c \cdot (t_A - E_S) \cdot F_{u\_A} = 12.024 \cdot kip$

Distance from center of bolt hole to edge of plate ................................................. $L_e := 1.25 \cdot in$

Thickness of plate ............................................................................................ $t_p := 0.135 \cdot in$

Nominal diameter of bolt hole ............................................................................ $d_{bh} := 0.562 \cdot in$

Tensile strength of steel ...................................................................................... $F_u := 65 \cdot ksi$

Resistance factor for bearing (AASHTO 6.5.4.2) ..................................................... $\phi_{bb} := 0.80$

$$R_n := 2 \cdot \phi_{bb} \cdot 1.2 \cdot \left(L_e - \frac{d_{bh}}{2}\right) \cdot \left(t_p - E_S\right) \cdot F_u = 9.619 \cdot kip$$ (AASHTO 6.13.2.9-2)

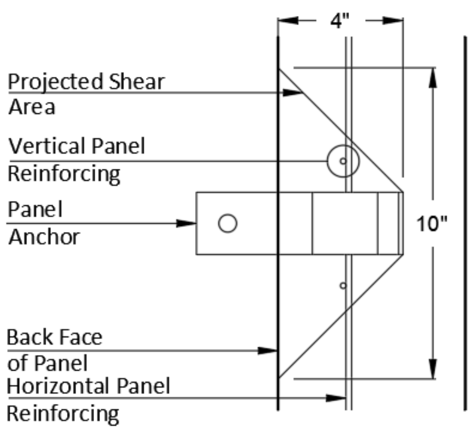
Distance from center of bolt hole to edge of strip ................................................. $L_{eS} := 1.50 \cdot in$

Bearing on Soil Reinforcing Strip

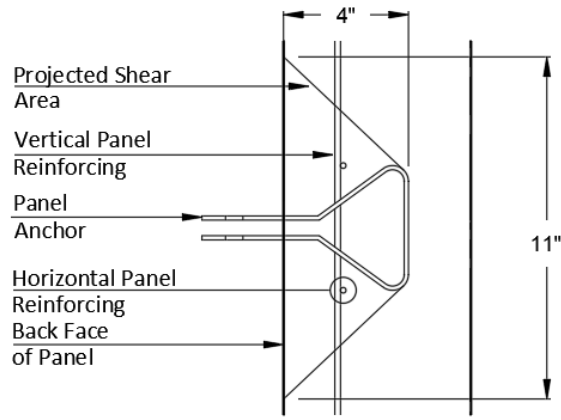
$$R_{nS} := \phi_{bb} \cdot 2.4 \cdot \left(d_{bh}\right) \cdot \left(t_{SS3}\right) \cdot F_u = 10.959 \cdot kip$$

## Design Model for Anchor Pullout in Concrete

The pullout of the anchor from the concrete will be calculated in conformance with ACI Appendix D




Plan View




Section View

**Segmental Concrete Panel Material Properties**

Compressive strength of concrete........................................................................ $f_c := 4000 \cdot psi$

Yield strength of reinforcing................................................................................ $F_{ys} := 65 \cdot ksi$

Number of anchors ........................................................................................ $n := 1$

Depth of embedment of anchor......................................................................... $h_{ef} := 4.00 \cdot in$

Prismatic Area of concrete Shear $A_{brg} := 11 \cdot in \cdot 10 \cdot in = 110 \cdot in^2$

**D.4** General requirements for strength of anchors

**D4.4** Strength Reduction Factor:

a) Anchor governed by strength of a ductile steel..................................... $\phi_s := 0.75$

b) Anchor governed by concrete breakout......................................... $\phi_c := 0.75$

**D.5** Design requirements for tensile loading

**D.5.1** Steel Strength of Anchor in Tension

**D.5.1.2** Nominal Strength of Group of Anchors

$$\phi N_s := \phi_s \cdot n \cdot A_{se} \cdot F_{u\_A} = 13.163 \cdot kip \qquad \textit{(D - 3)}$$

**D.5.2** Concrete breakout strength of anchor in tension

**D.5.2.4** Modification factor for eccentrically loaded anchor

$\psi_1 := 1.0$ no eccentricity

**D.5.2.5** Modification factor for edge effects

$\psi_2 := 1.0$ no edge effects

**D.5.2.6** Modification factor with no cracking

$\psi_3 := 1.25$ cast-in-anchors

**D 5.2.2** Basic concrete breakout strength

$k := 24$ for cast-in anchors

$$N_b := k \cdot \sqrt{f_c \cdot psi} \cdot h_{ef}^{1.5} \cdot \left( \frac{in^2}{in^{1.5}} \right) = 12.143 \cdot kip$$ ***(D - 7)***

**D.5.2.1** Nominal concrete breakout for single anchor

Projected area of pullout (1) (Assumes prismatic projected area)

$$A_{NO} := \left[2 \cdot (1.5) \cdot h_{ef}\right] \cdot \left[2 \cdot (1.5) \cdot h_{ef}\right] = 144 \cdot in^2$$

Projected area of pullout (2) (Effects of edge distance)

$c_1 := 7.5 \cdot in$

$$A_N := \left(c_1 + 1.5 \cdot h_{ef}\right) \cdot \left[2 \cdot (1.5) \cdot h_{ef}\right] = 162 \cdot in^2$$

$$\phi N_{cb} := \phi_s \cdot \frac{A_N}{A_{NO}} \cdot \psi_2 \cdot \psi_3 \cdot N_b = 12.807 \cdot kip$$ ***(D - 4)***

Projected area of pullout (3) (Effects of 2 Anchors Spaced at $S_1$)

$S_1 := 7.5 \cdot in$

$$A_{Ng} := \left[2 \cdot (1.5) \cdot h_{ef}\right] \cdot \left[S_1 + 2 \cdot (1.5) \cdot h_{ef}\right] = 234 \cdot in^2$$

$$\phi N_{cbg} := \phi_s \cdot \frac{A_{Ng}}{A_{NO}} \cdot \psi_2 \cdot \psi_3 \cdot N_b = 18.499 \cdot kip$$ ***(D - 4)***

# 1.2.16
# Soil Reinforcing Pullout Testing and Results

EARTHTEC

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# EARTHTRAC SS1
# PULLOUT REPORT



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# THE COLLIN GROUP

# EARTHTRAC SS1 PULLOUT TESTING REPORT

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## Table of Contents

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## 1.0 INTRODUCTION

The Collin Group has supervised pullout tests on the EarthTrac SS1™ soil reinforcing element. The tests were performed in a poorly graded sand (SP). The test results from poorly graded sand will give lower bound pullout values when compared to material that is classified as well-grade (GW) granular material. The tests were performed to establish friction factors to be used in the design and analysis of the Ground Improvement Systems, LLC., (GIS) Reinforced Soil Wall (RSW) system. The tests were performed using state-of-practice testing procedures at the Civil Products laboratory located at in Arlington, Texas. The following sections discuss the testing program and test procedures and present the test results.

## 2.0 TESTING PROGRAM

The pullout tests were performed in accordance with the requirements outlined in FHWA NHI-10-024 “Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines” (2009) and the procedures in ASTM D 6706 “Standard Test Method for Measuring Geosynthetic Pullout Resistance in Soil”.

### 2.1 SOIL REINFORCING

The reinforcing material evaluated in the pullout tests consisted of the EarthTec EarthTrac SS1 metallic steel strip. The metallic EarthTrac SS1 was specifically produced to meet the design strength requirements of the GIS and in conformance with AASHTO and FHWA. The EarthTrac SS1 consist of a galvanized high strength steel strip. The steel strip is fabricated with a series of raised ribs. The steel strip is 2” wide and 3/16” thick. There are two ribs, classified as a major rib and a minor rib. The ribs are on each surface of the steel strip. The major rib is spaced at 6” on center. The minor rib is spaced between major ribs and are 1.5 inches on center. Details of the EarthTrac SS1 are shown in Figure 2-1 and Figure 2-2.

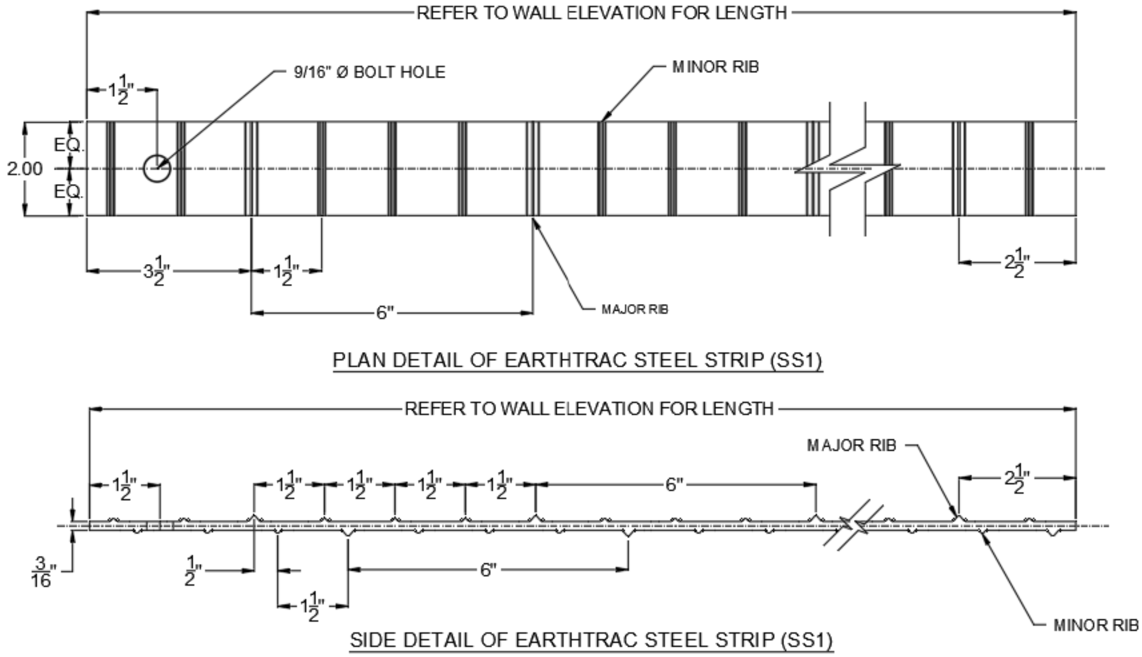



**Figure 2-1 Plan and Section View of EarthTrac SS1 Soil Reinforcing**

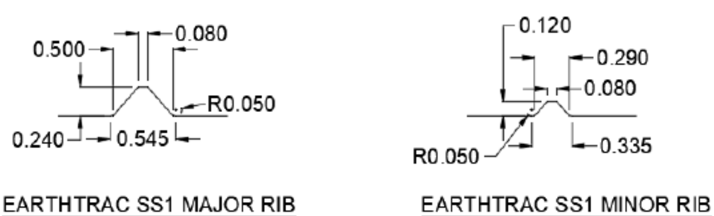



**Figure 2-2 Major and Minor Rib Configuration for the EarthTrac SS1 Soil Reinforcing**

All pullout test specimens had an approximate embedment length in the pullout box equal to 48 in. (1200 mm).

## 2.2 Soil

The pullout tests were performed using a poorly graded sand (SP) as backfill. The natural sand used in these tests was obtained from Florida and was supplied by GIS. The sand is classified as SP and the gradation (ASTM D422 and ASTM D1140) is shown in the Appendix. Standard Proctor compaction (ASTM D 698) test results performed on the sand found a maximum dry density of 98.7 pcf (15.5 kN/m$^3$) at an optimal moisture content of 17.1%.

### 2.3 TEST APPARATUS

The tests were performed using the Civil Products Laboratory pullout test box. This pullout box is modeled after the ASTM test apparatus and is shown schematically in Figure 2-3. The box is fabricated from structural steel components.

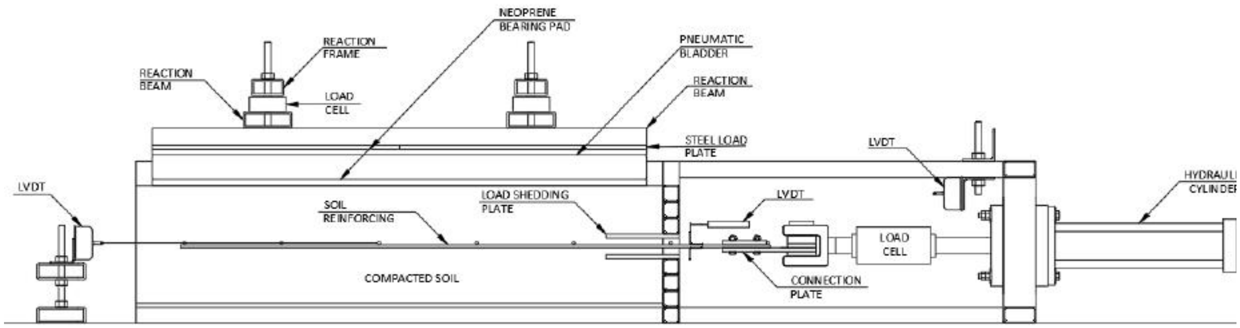



**Figure 2-3 Cross Section Pullout Box**

The pullout box has an inside dimension of 18” (450 mm) wide by 60” (1500 mm) long and is 15” (375 mm) deep. The soil reinforcing exits the back of the box through a gate. The exit gate consists of a series of 2” x 2” (50 mm x 50 mm) structural steel tubes. A hole is formed by two ½” x 7” (12mm x 350 mm) steel plates that are welded to two of the 2” x 2” (50 mm x 50 mm) structural steel tubes. The steel plates protrude into the soil. The steel plates reduce any boundary condition that may occur at the back of the box.

**Photograph 2-1 Pullout Box**

The vertical overburden pressure is applied using a pneumatic diaphragm (air bladder). The pneumatic diaphragm is inflated and pushes against a set of reaction beams as shown in Photograph 2-2. The load is controlled using a pneumatic regulator and is measured using two Omega load cells. The pneumatic diaphragm is sandwiched between two ¾” (19 mm) neoprene bearing pads. On the top neoprene bearing pad a ½” thick steel plate is placed. Two 2” x 6” (50 mm x 150 mm) structural steel tubes are placed on the steel plate. Two 2” x 6” (50 mm x 150 mm) structural steel tubes span between each 2” x 6” (50 mm x 150 mm) structural steel tube. The load cells are placed on top of the 2” x 6” (50 mm x 150 mm) structural steel tubes. Two 2” x 6” (50 mm x 150 mm) structural steel tube reaction beams are then attached to 1” diameter all-thread. The pneumatic diaphragm and steel components are isolated from the side of the pullout box using a plastic sheet. The plastic sheet reduces any down drag that may occur do to the inflation of the pneumatic diaphragm.

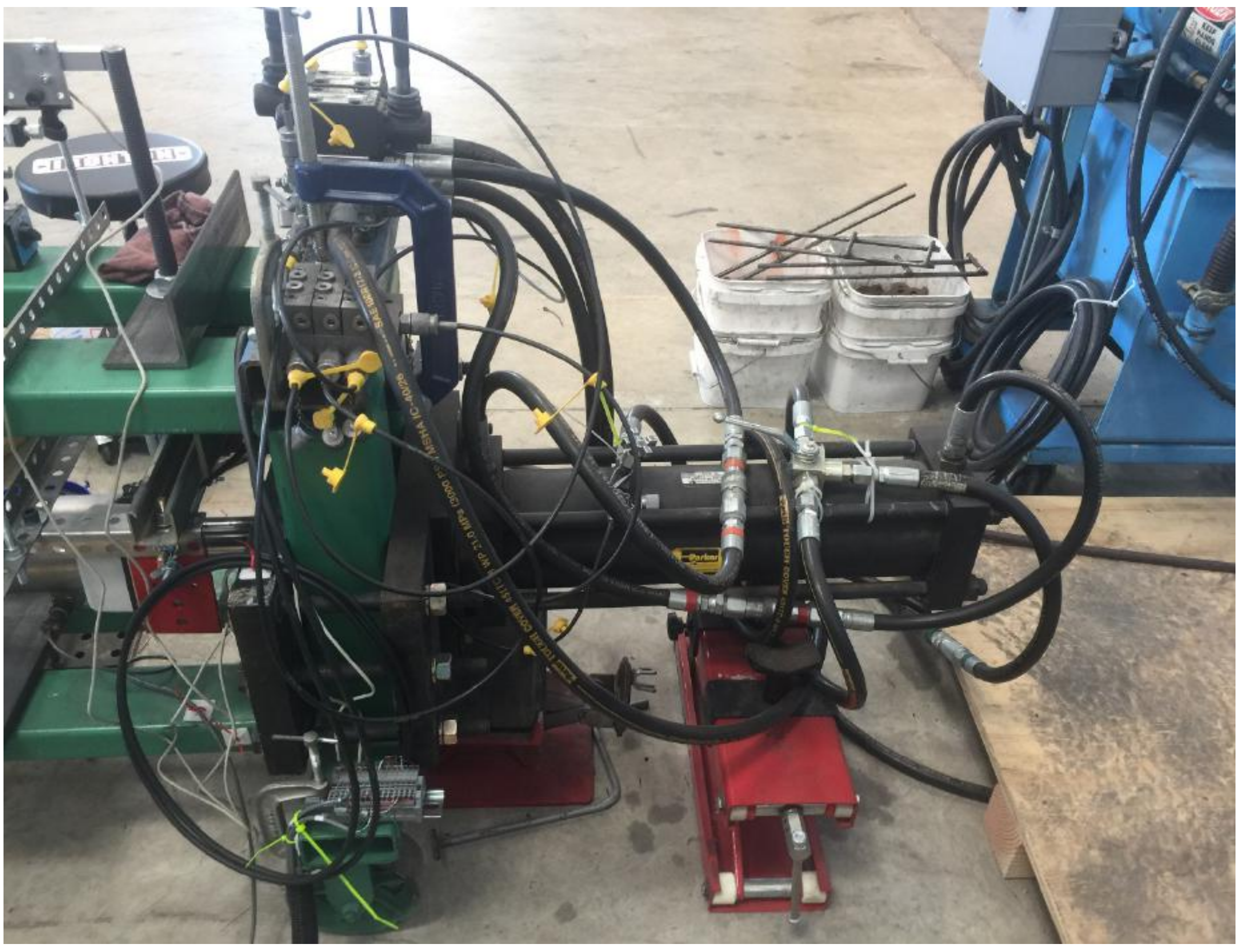
**Photograph 2-2 Vertical Overburden Apparatus**

At the back of the pullout box the soil reinforcing is sandwiched between two specialized connection plate as shown in Photograph 2-3. The steel plates are ½" (12 mm) thick. The soil reinforcing is secured between the two plates using bolts. To reduce slippage at the interface of the soil reinforcing and the face of the plate, a series of diagonal grooves were machined into the plate and the plate was heat treated. The grooves resemble the pattern on a steel file. The bottom steel plate is wider than the top steel plate to allow for attachment to the horizontal loading mechanism. The connection plate is attached to the horizontal load apparatus at a single point using a clevis. The single point of connection allows the plate to rotate so the load is applied uniformly to the soil reinforcing element during testing.

**Photograph 2-3 Connection Plate Apparatus**

The horizontal load is applied using a Parker Heavy Duty hydraulic cylinder (Photograph 2-4). The cylinder bore is 5" (125 mm) and has an 18" (450 mm) stroke. The maximum tension force for the cylinder is equal to 50-kip (220 kN). The load is applied at an approximate rate of 2 mm to 3 mm per minute. The load application varies based on system conditions, atmospheric conditions, and load conditions. The load application is controlled using a metering valve that is attached to the hydraulic system. Attached to the hydraulic cylinder rod end is an Omega Canister load cell with a maximum 50-kip capacity (LC1001-50k). The load cells are calibrated prior to testing using an MTS.

Linear Variable Displacement Transducers (LVDT) were used to record the position of the soil reinforcing during application of the horizontal load. Three LVDT's were used. One LVDT is attached to the soil reinforcing inside the soil. This LVDT is positioned at the front of the box and to the end of the soil reinforcing as shown in Figure 2-4. At the back of the box and on each side of the soil reinforcing connection plate, two LVDT's are positioned.



**Photograph 2-4 Horizontal Load Apparatus**

**Figure 2-4 LVDT Location Attached to Soil Reinforcing Inside Soil Box**

A Campbell Scientific Data Acquisition system is used to record the horizontal load and the position of the LVDT's. The Campbell Scientific PC400 software was used to record the data to the central computer. The data is extracted in a text file and imported into Excel.

### 2.4 TEST SET-UP

The test set-up is repetitive and does not vary from test to test. The following steps are used in each test

1. Determine weight of soil at 2% below optimum moisture content
2. Determine total weight of soil for test box
3. Determine weight of soil below elevation of soil reinforcing element
4. Determine weight of soil above elevation of soil reinforcing element
5. Determine weight of reaction frame components
6. Determine system pressure applied to soil reinforcing
7. Place front gate in pullout box
8. Place soil in desired lift thickness and compact to required density
9. Repeat until soil is placed to elevation of soil reinforcing
10. Place soil reinforcing element

11. Place LVDT wire guides on soil reinforcing
12. Place remaining front gate in pullout box
13. Place soil in desired lift thickness and compact to required density
14. Level soil at top of the box.
15. Place neoprene pad on top of soil
16. Place plastic sheet on top of neoprene pad
17. Place expanded pneumatic diaphragm on top of reaction plate.
18. Place neoprene pad on top of pneumatic diaphragm
19. Place reaction plate on top of pneumatic diaphragm
20. Place reaction beams on top of reaction plate
21. Place reaction cross beams on all-thread columns and secure with nut
22. Deflate pneumatic diaphragm. Check position of all reaction frame components in relationship to sides of box.
23. Place load cell reaction cross beams on reaction beams
24. Place load cell and level reaction beam until the bottom of the reaction beam meets the load button on the load cell.
25. Verify that the all-thread columns are secure by tightening the nut on bottom interface of the 2x2 HSST.
26. Move connection plate into proper position. Connect soil reinforcing to connection load plate. Verify that connection plate is level. Secure bolts with air wrench.
27. Place LVDT fixture and LVDT's at front of the pullout box. Level and straighten each LVDT.
28. Connect LVDT at back of pullout box
29. Turn Campbell Scientific on and perform the following:
    a. Zero load cells.
    b. Record the beginning positions of LVDTs
    c. Calcuate the position of the LVDT at ¾" deformation and record the value
    d. Expand air bag until required load is applied to the load cells. Verify that loads are equal in the load cell.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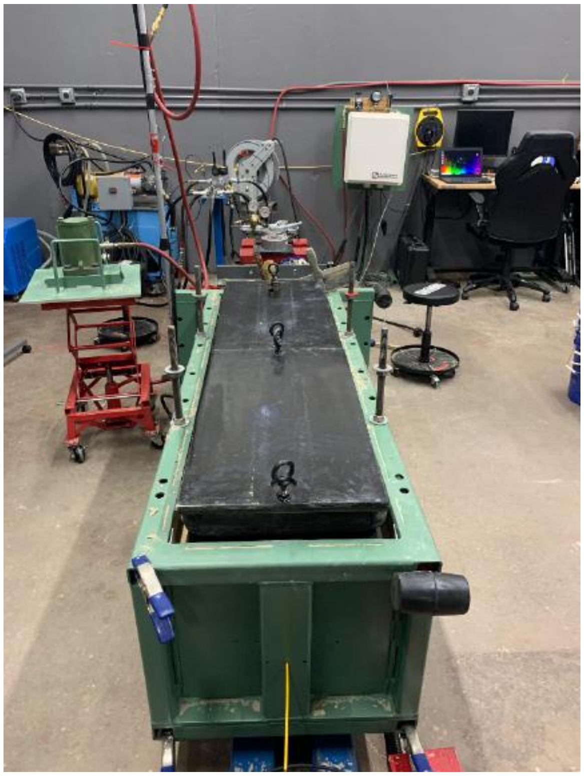
30. Turn on hydraulic pump. Verify that the ball valve is in the position required to perform test and is not in “set-up” position.
31. Activate flow control to perform test
32. Stop test 1 ½” total displacement at exit gate
33. Release pressure in hydraulic cylinder
34. Save data from Campbell scientific PC400 software
35. Remove reaction frame
36. Remove pneumatic diaphragm
37. Remove soil to top of soil reinforcing
38. Examine soil reinforcing in place in soil
39. Remove soil reinforcing and examine for deflection and damage

The EarthTrac SS1 soil reinforcing was embedded horizontally between two 6” (152 mm) layers of soil. Then the first six inches (152 mm) of soil was placed and compacted in the bottom of the box at near optimum moisture content to approximately 95% of the maximum dry density. The metallic EarthTrac SS1 soil reinforcing was then placed on the compacted soil and centered in the box. The metallic strip was extended through the 0.5 inch (12.7 mm) slot at the back (i.e., door) of the box. The opening consists of a “sleeve” that extends into and over the soil. The sleeve consists of two, 6 inch (152 mm) wide horizontal metal plates that are separated by spacers. The sleeve prevents normal stress from transferring to the lead end of the soil reinforcing reducing progressive increased loading at the opening. Then the second layer of 6 in. (152 mm) of sand is placed over the soil reinforcing and sleeve and compacted to the top of the box.

A ¾” rubber bearing pad is placed on the top of the compacted soil. The pneumatic diaphragm is placed on the ¾” bearing pad that is followed by another ¾” bearing pad. A ridged cover plate is placed on the top of the ¾” rubber bearing pad.



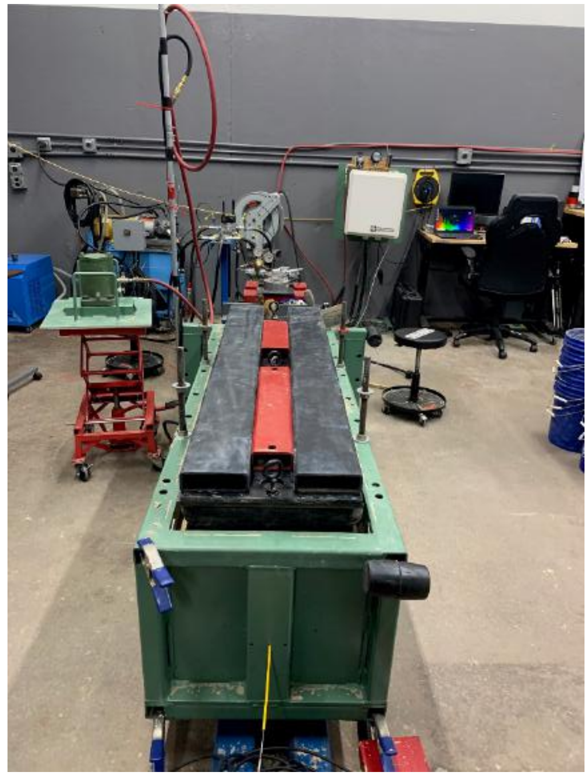


**Figure 2-5 Cover Plate at Top of Pullout Box**

The reaction frame consists of structural steel cross bracing that is secured to the top of the box before application of the normal stress. For the lowest stress level 6” of soil and steel components is used to apply the normal stress. For the remaining, higher normal stress levels, additional stress (i.e., to the weight of the soil) was applied to the sample by pressurizing the pneumatic diaphragm.

**Figure 2-6 Normal Load Application**

A variable speed hydraulic ram applies the pullout force. The force is measured by a calibrated load cell attached to the end of the rod. The metallic soil reinforcing specimens are gripped in a clamp consisting of two metal plates and a series of bolts as shown in Figure 2-4. The metallic soil reinforcing is sandwiched between the plates. The plates are tightly bolted together providing a clamping force. The clamping system is attached to the rod end using a swivel connection so the clamp can adjust for elements that were not exactly straight and for uneven pullout scenarios.

The horizontal displacement of the front of the soil reinforcing is monitored outside of the box by two Linear Variable Differential Transducers (LVDTs). The movement is measured by attaching the LVDTs (one on each side) to a small steel plate that is attached to the pullout connection plate. The plunger end of the LVTDs are positioned against the front of the plate. To monitor movements of the embedded portion of the soil reinforcing a wire rope extensometer was used. The lead end of the extensometer is embedded in the soil. The lead end of the extensometer consisted of a wire contained inside a hollow tube housing. The terminal end of the extensometer consisted of an LVDT that was mounted outside the front of the box on a steel frame. The lead end of the wire extensometers is attached to the middle of the soil reinforcing. The LVDTs and load cell are connected to a computer data acquisition system to acquire the data during the tests.

The metallic ribbed strip soil specimens were pulled out at rate of 0.4 in. per minute (1 mm/min) in accordance with the ASTM D 6706 test procedure. Tests were performed until the specimen either pulled out, as defined by at least 3/4 inch (20 mm) of movement at the front end of the specimen (as required by FHWA guidelines), or rupture, if it occurred prior to pullout. Rupture did not occur in any of the specimens. Because of this, and in most cases, pullout was extended up to 1.5 inches (75 mm) to identify the load to achieve a maximum peak pullout resistance and obtain a complete pullout deformation curve. The specimens were examined after the test to observe noticeable specimen distortion and/or deformation.

## 3.0 PULLOUT RESISTANCE TEST RESULTS

The pullout test measures the pullout resistance ($P_r$) at ¾" of deformation and at the peak deformation response. The friction factor ($F^*$) is then back calculated using the following equation:

$$F^* = \frac{P_r}{C \cdot \alpha \cdot \sigma_v \cdot w \cdot L_e}$$

| | | | |
|---|---|---|---|
| Where: | $F^*$ | = | friction factor (dim) |
| | $P_r$ | = | pullout resistance (lbf) |
| | $C$ | = | reinforcement effective unit perimeter (C = 2 for strips) (dim) |
| | $\alpha$ | = | a scale effect correction factor (1.0 for inextensible soil reinforcement) (dim) |
| | $\sigma_v$ | = | the effective vertical stress at the soil-reinforcement interfaces (psf) |
| | $w$ | = | width of soil reinforcing (ft) |
| | $L_e$ | = | embedment length in soil box (ft) |

Table 1 provides a summary of the measured $P_r$ values for both the peak and ¾ inch failure condition. For inextensible type reinforcements the FHWA 10-024 manual recommends that a maximum deflection equal to ¾ inch (20 mm) as measured at the front of the specimen be used to select $P_r$, if the peak value for $P_r$, or rupture of the specimen does not occur before ¾ inch (20 mm) deflection. The friction factor is back calculated and is plotted in Figure 3-1. A load-deflection plot from the test is shown in the Figure 3-2. In these figures, the LVDT displacements at the back of the box are plotted versus the corresponding load measured from the load cell.

The F* value determined for each normal stress is shown in Table 1 for both the peak and 3/4 inch (20 mm) failure condition. The $\alpha$ factor was determined to be one (1) based on the consistent and concurrent movement of the front and back LVDT's during the test.

Table 1. Measure Pullout Resistance and Calculated Friction Factor

| **Test** | **Name** | **Surcharge psf** | **$P_{r\_3/4}$** lbf | **$F^*_{3/4"}$** | **$P_{r\_peak}$** lbf | **$F^*_{peak}$** |
|---|---|---|---|---|---|---|
| 1 | 125-T1 | 125 | 1091 | 6.54 | 1091 | 6.54 |
| 2 | 125-T2 | 125 | 1044 | 6.26 | 1547 | 9.28 |
| 3 | 125-T3 | 125 | 1037 | 6.22 | 1046 | 6.27 |
| 4 | 625-T1 | 714 | 2647 | 2.78 | 2647 | 2.78 |
| 5 | 1250-T1 | 1237 | 4108 | 2.49 | 4108 | 2.49 |
| 6 | 1230-T2 | 1241 | 4446 | 2.69 | 4446 | 2.69 |
| 7 | 1849-T1 | 1844 | 5024 | 2.04 | 5024 | 2.04 |
| 8 | 2500-T1 | 2453 | 6112 | 1.87 | 6112 | 1.87 |
| 9 | 625-T2 | 601 | 2790 | 3.48 | 2790 | 3.48 |
| 10 | 1866-T2 | 1837 | 4626 | 1.89 | 4626 | 1.89 |
| 11 | 3125-T1 | 3056 | 6956 | 1.71 | 6956 | 1.71 |

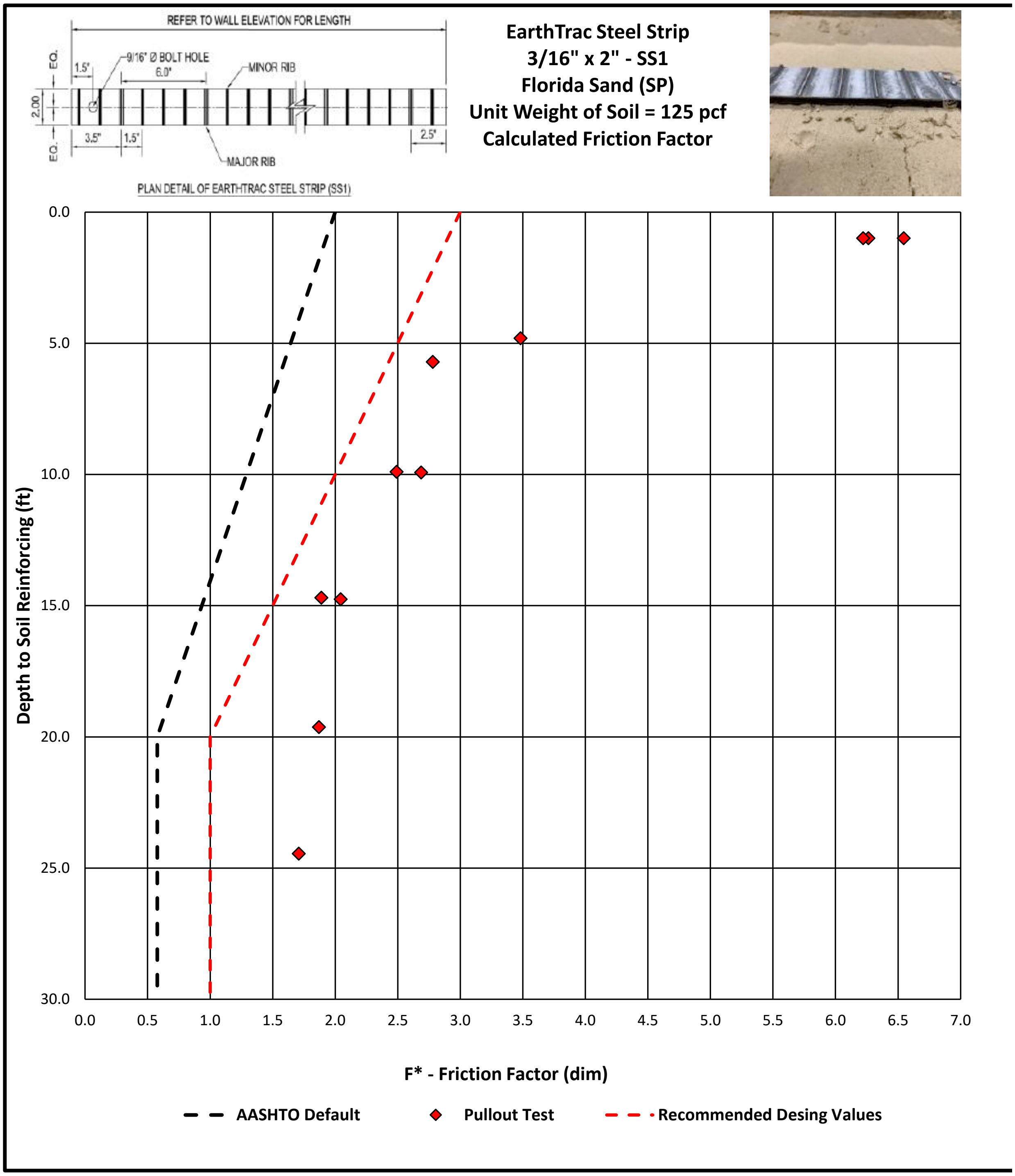



Figure 3-1. Friction Factor (F*)

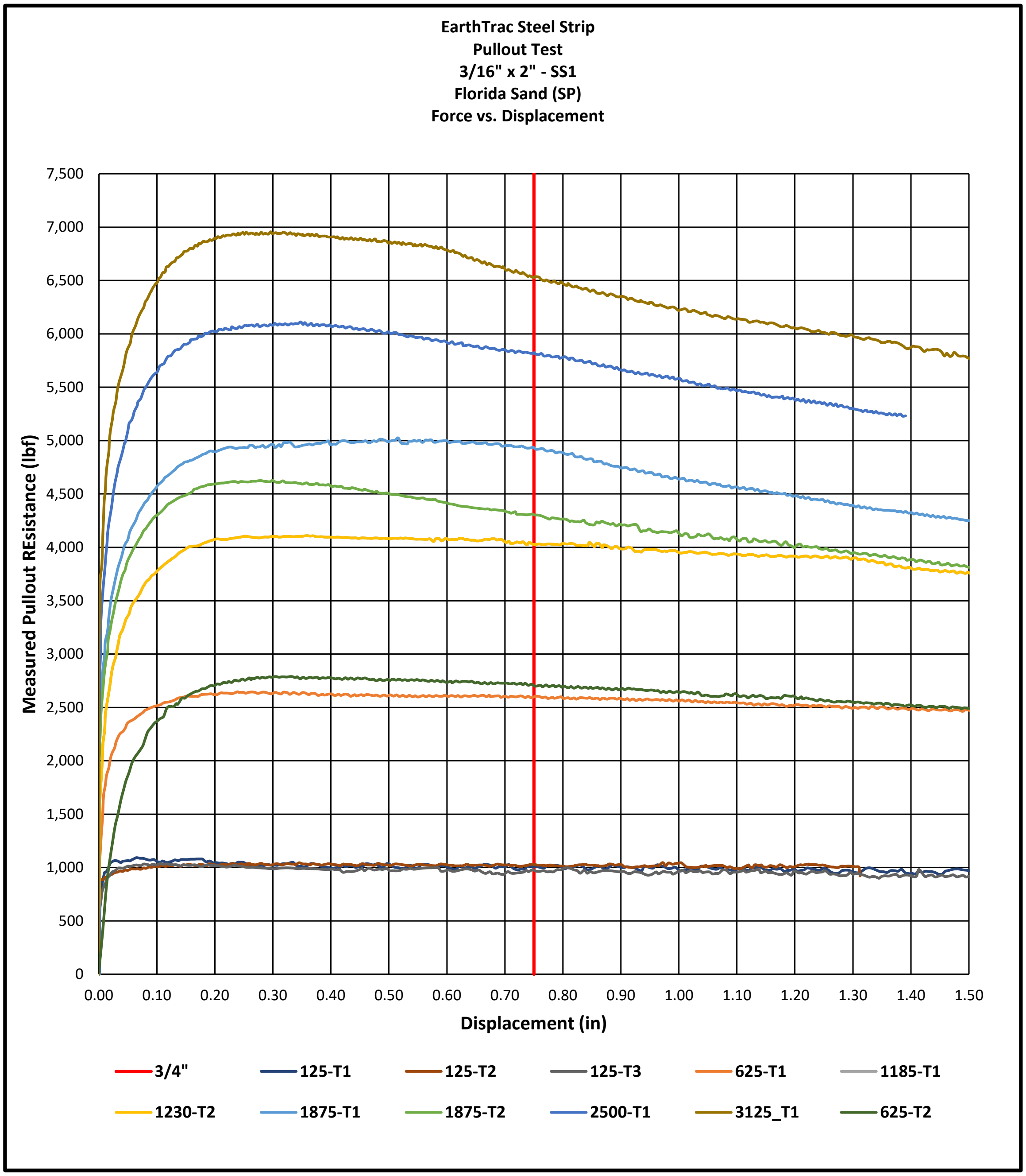



**Figure 3-2. Measured Force VS Displacement**

## 4.0 SUMMARY AND RECOMMENDATIONS

Based on the pullout testing the friction factor for the EarthTrac SS1 soil reinforcing can conservatively use a linearly decreasing value equal to 3.0 at the top of the structure decreasing to 1.0 at a depth of 20 feet and below. This friction factor is valid for all soils that are commonly used in MSE structures. Material with greater than 15% fines are not covered in this pullout report.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# APPENDIX A
# SOIL PROPERTIES


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8908 Ambassador Row, Dallas, TX 75247
Texas Engineering Firm Registration, No. F-1572
Corporate Phone: (214) 630-9745

# Report of Sieve Analysis

**Client:** Ground Improvement Systems, LLC
114 South Collins Street
Arlington, TX 76011

**Report No.:** 14284877
**Project No.:** 2024753
**Date of Service:** 11/16/2020
**Report Date:** 11/17/2020

**Project:** Miscellaneous Laboratory Testing
**Services:** Sieve Analysis Testing

Description: Light Brown Poorly-Graded Sand
Sample Weight: 446.0
Source: Client Delivered

| Sieve Size | Cumulative Weight Retained | Percent Passing |
|---|---|---|
| 3/8 | 0.0 | 100 |
| #4 | 1.6 | 99.6 |
| #8 | 4.3 | 99.0 |
| #16 | 8.4 | 98.1 |
| #30 | 15.6 | 96.5 |
| #50 | 28.9 | 93.5 |
| #100 | 341.7 | 23.4 |
| #200 | 443.6 | 0.5 |
| PAN | 446.0 | - |

**Test Methods:** ASTM C-136

**Technician:** Shane Tandy
**Report Distribution:**
Orig: Ground Improvement Systems, LLC (Arlington, TX)
Attn: Mr. Thomas Taylor (1-ec copy)
1-ec Rone Engineering Project Managers
Attn: Mr. Scott Watson, CET

**Rone Engineering**

K. Scott Watson, CET
Project Manager

8908 Ambassador Row, Dallas, TX 75247
Texas Engineering Firm Registration, No. F-1572
Corporate Phone: (214) 630-9745

# Report of Moisture Density Relationship

**Client:** Ground Improvement Systems, LLC
114 South Collins Street
Arlington, TX 76011

**Report No.:** 14284001
**Project No.:** 2024753
**Date of Service:** 11/06/2020
**Report Date:** 11/10/2020

**Project:** Miscellaneous Laboratory Testing

**Services:** Obtain a sample of material from the jobsite, bring the sample back to the laboratory and perform a moisture density relationship test in accordance with ASTM Standards.

## PROJECT DATA

**Contractor:**
**Test For:** Pull-out Sand
**Material:** Light Brown Poorly-Graded Sand
**Classification:** Pull-out Sand
**Test Method:** ASTM D-4318 Method-B
ASTM D-698 Method-B
ASTM D-1140

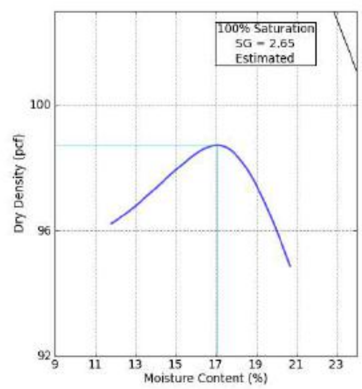
**Date Sampled:** 11/06/2020
**Material Preparation:** Dry
**Rammer Type:** Mechanical
**Sampled By:** Client
**Tested By:** Willie Wagner
**Sample Location:**
Client Delivered

## REPORT OF TEST

Maximum Dry Density, pcf: 98.70 pcf
Optimum Moisture Content, %: 17.1 %

**Liquid Limit:** NP
**Plastic Limit:** NP
**Plasticity Index:** NP

**% Passing #200:** 3




**Technician:** Shane Tandy
**Report Distribution:**
Orig: Ground Improvement Systems, LLC (Arlington, TX)
Attn: Mr. Thomas Taylor (1-ec copy)
1-ec Rone Engineering Project Managers
Attn: Mr. Scott Watson, CET

**Rone Engineering**

K. Scott Watson, CET
Project Manager

LIMITATIONS: The test results presented herein were prepared based upon the specific samples provided for testing. We assume no responsibility for variation in quality (composition, appearance, etc.) or any other feature of similar subject matter provided by persons or conditions over which we have no control. Our letters and reports are for the exclusive use of the clients to whom they are addressed and shall not be reproduced except in full without the written approval of Rone Engineering Services, Ltd. (SW/SW)

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# EARTHTRAC SS3
# PULLOUT REPORT

STATE OF MARYLAND
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# EARTHTRAC SS3
# PULLOUT REPORT


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# THE COLLIN GROUP

# EARTHTRAC SS3 PULLOUT REPORT

**7445 Arlington Road**

**Bethesda, MD 20184**

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## Table of Contents

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## 1.0 INTRODUCTION

The Collin Group has supervised pullout tests on the EarthTrac SS3™ soil reinforcing element. The tests were performed in a poorly graded sand (SP). The test results from poorly graded sand will give lower bound pullout values when compared to material that is classified as well-grade (GW) granular material. The tests were performed to establish the pullout friction factors to be used in the design and analysis of the EarthTec Reinforced Soil Wall (RSW) system using the EarthTrac SS3 soil reinforcement. The tests were performed using state-of-practice testing procedures at the Civil Products laboratory located at in Arlington, Texas. The following sections discuss the testing program and test procedures and present the test results.

## 2.0 TESTING PROGRAM

The pullout tests were performed in accordance with the requirements outlined in FHWA NHI-10-024 "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines" (2009) and the procedures in ASTM D 6706 "Standard Test Method for Measuring Geosynthetic Pullout Resistance in Soil".

### 2.1 SOIL REINFORCING

The reinforcing material evaluated in the pullout tests consisted of the EarthTrac SS3 steel strip. The EarthTrac SS3 steel strip was specifically produced to meet the design strength requirements of the EarthTec and in conformance with AASHTO and FHWA. The EarthTrac SS3 steel strip consist of galvanized high strength steel that is fabricated with a series of raised ribs. The steel strip is 2" wide and 5/32" thick. There are two ribs one on the top and one on the bottom and are mirror images. The opposing ribs are spaced 3" apart and are placed every twelve inches along the surface of the strip. Details of the EarthTrac SS3 are shown in Figure 2-1 and Figure 2-2.

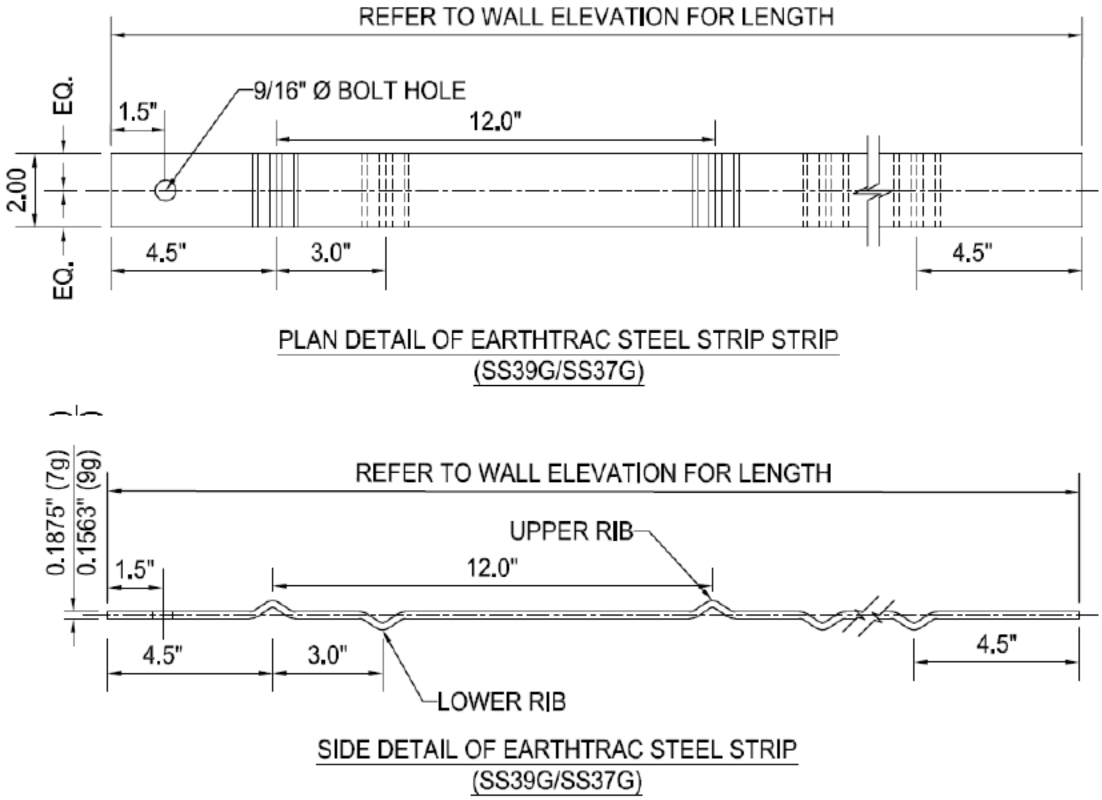



**Figure 2-1 Plan and Section View of EarthTrac SS3 Soil Reinforcing**

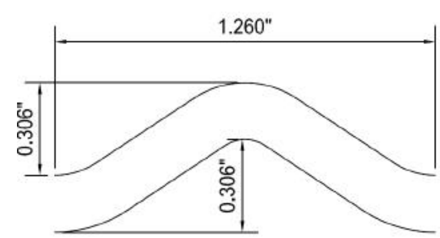



**Figure 2-2 Upper and Lower Rib Configuration for the EarthTrac SS3 Soil Reinforcing**

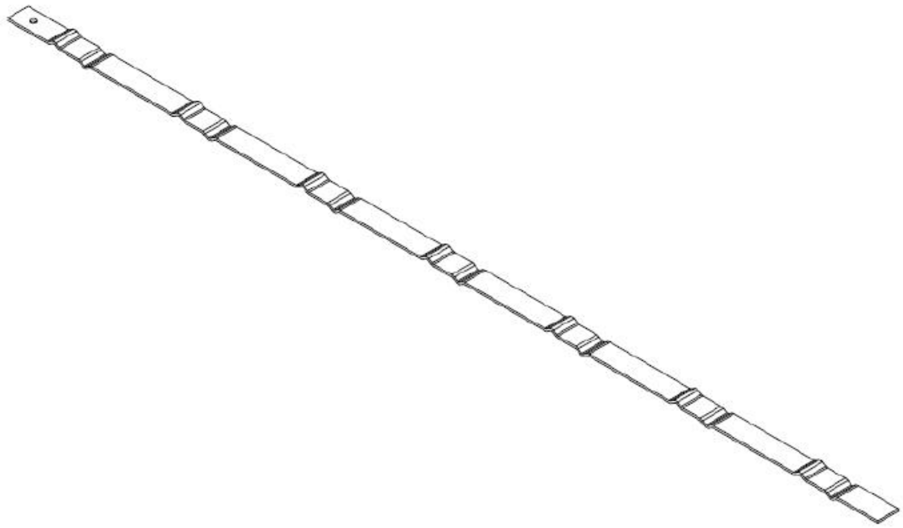


**Figure 2-3 Isometric EarthTrac SS3 Soil Reinforcing**

The SS3 Steel strip was placed in the pullout box. All pullout test specimens had an approximate embedment length in the pullout box equal to 48 in. (1200 mm).

## 2.2 SOIL

The pullout tests were performed using a poorly graded sand (SP) as backfill. The natural sand used in these tests was obtained from Florida and was supplied by EarthTec. The sand is classified as SP and the gradation (ASTM D422 and ASTM D1140) is shown in the Appendix. Standard Proctor compaction (ASTM D 698) test results determined that the maximum dry density is 98.7 pcf (15.5 kN/m$^3$) at an optimal moisture content of 17.1%.

## 2.3 TEST APPARATUS

The tests were performed using the Civil Products Laboratory pullout test box. This pullout box is modeled after the ASTM test apparatus and is shown schematically in Figure 2-4. The box is fabricated from structural steel components.

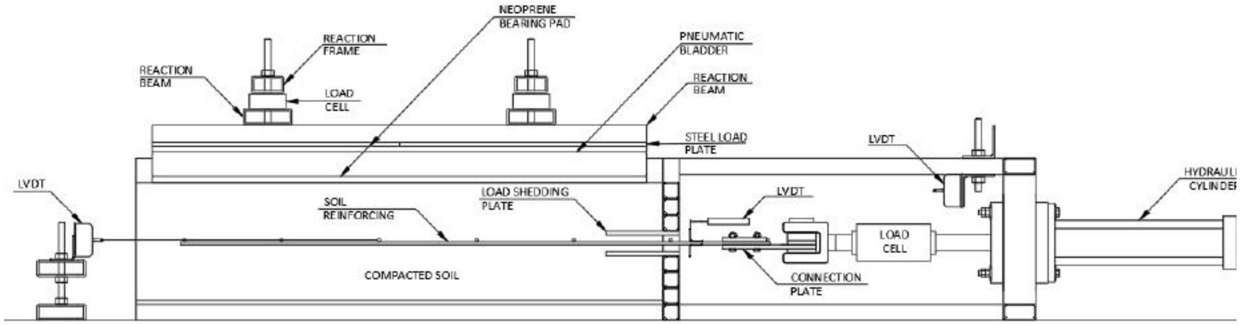



**Figure 2-4 Cross Section Pullout Box**

The pullout box has an inside dimension of 18” (450 mm) wide by 60” (1500 mm) long and is 15” (375 mm) deep. The soil reinforcing exits the back of the box through a gate. The exit gate consists of a series of 2” x 2” (50 mm x 50 mm) structural steel tubes. A slot is formed by two ½” x 7” (12mm x 350 mm) steel plates that are welded to two of the 2” x 2” (50 mm x 50 mm) structural steel tubes. The steel plates extend into the soil. The steel plates reduce the boundary conditions that may occur at the back of the box.

**Photograph 2-1 Pullout Box**

The vertical overburden pressure is applied using a pneumatic diaphragm (air bladder). The pneumatic diaphragm is inflated and pushes against a set of reaction beams as shown in Photograph 2-2. The overburden load is controlled using a pneumatic regulator and is measured using two Omega load cells. The pneumatic diaphragm is sandwiched between two ¾" (19 mm) neoprene bearing pads. On the top neoprene bearing pad a ½" thick steel plate is placed. Two 2" x 6" (50 mm x 150 mm) structural steel tubes are placed on the steel plate. Spanning between each of the 2" x 6" (50 mm x 150 mm) structural steel tube are 2" x 6" (50 mm x 150 mm) structural steel tubes. The load cells are placed on top of the 2" x 6" (50 mm x 150 mm) structural steel tubes. Two 2" x 6" (50 mm x 150 mm) structural steel tube reaction beams are then attached to 1" diameter all-thread posts. The pneumatic diaphragm and steel components are isolated from the side of the pullout box using a plastic sheet. The plastic sheet reduces any down drag that may occur due to the inflation of the pneumatic diaphragm.

**Photograph 2-2 Vertical Overburden Apparatus with Structural Steel Reaction Frame**

At the back of the pullout box the soil reinforcing is sandwiched between two specialized heat treated connection plates as shown in Photograph 2-3. The steel plates are ½" (12 mm) thick. The soil reinforcing is secured between the two plates using bolts. To reduce slippage at the interface of the soil reinforcing and the face of the plate, a series of diagonal grooves were machined into the plate. The grooves resemble the pattern on a steel file. The bottom steel plate is wider than the top steel plate to allow for attachment to the horizontal loading mechanism. The connection plate is attached to the horizontal load apparatus at a single point using a clevis. The single point of connection allows the plate to rotate so the load is applied uniformly to the soil reinforcing element during testing.

**Photograph 2-3 Connection Plate Apparatus**

The horizontal load is applied using a hydraulic cylinder (Photograph 2-4). The cylinder bore is 5” (125 mm) and has an 18” (450 mm) stroke. The maximum tension force that the cylinder can generate is equal to 50-kip (220 kN) at 3000 psi line pressure. The horizontal load is applied at an approximate rate of 2 mm to 3 mm per minute. The load application speed varies based on system conditions, atmospheric conditions, and load conditions. The load application is controlled using a metering valve that is attached to the hydraulic system. Attached to the hydraulic cylinder rod end is an Omega Canister load cell with a maximum 50-kip capacity (LC1001-50k). The load cells are calibrated prior to testing using a Material Testing System (MTS) load frame.

Linear Variable Displacement Transducers (LVDT) were used to record the position of the soil reinforcing during application of the horizontal load. Three LVDT’s were used. One LVDT is attached to the terminal end of the soil reinforcing inside the box. The other two LVDTs are positioned at the front of the box on each side of the soil reinforcing connection plate as shown in Photograph 2-3 and Photograph 2-5.

**Photograph 2-4 Horizontal Load Apparatus**

**Photograph 2-5 LVDT Location Attached to Soil Reinforcing Inside Soil Box**

A Campbell Scientific Data Acquisition system is used to record the horizontal load and the position of the LVDT's. The Campbell Scientific PC400 software was used to record the data to the central computer. The data is extracted in a text file and imported into an Excel spreadsheet for processing.

### 2.4 TEST SET-UP

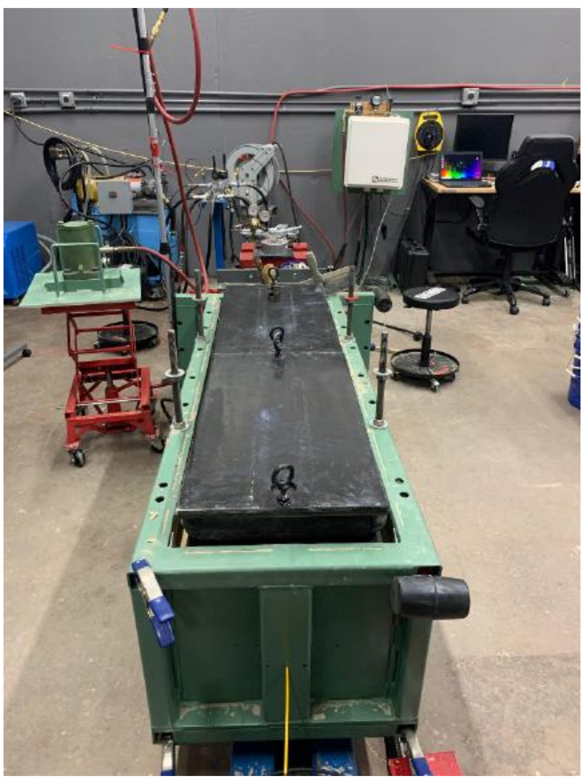
The test set-up is repetitive and does not vary from test to test. The following steps are used in each test

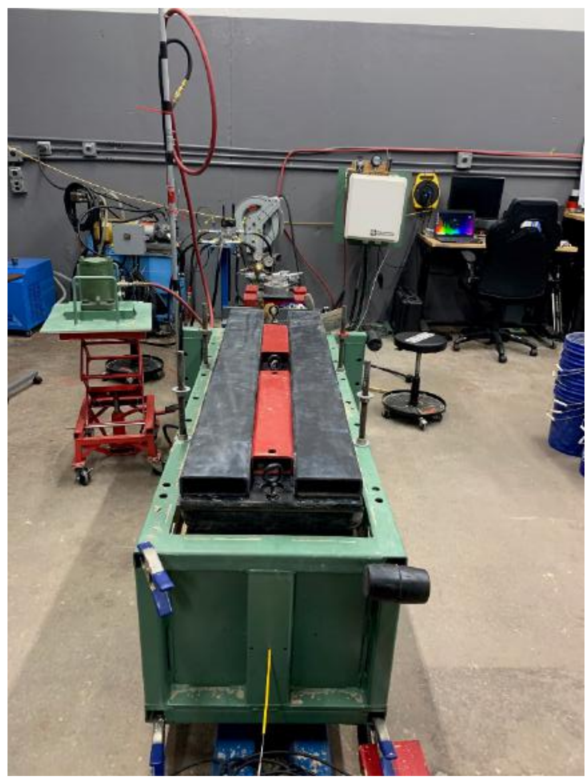
1. Determine weight of soil at 2% below optimum moisture content
2. Determine total weight of soil for test box
3. Determine weight of soil below elevation of soil reinforcing element
4. Determine weight of soil above elevation of soil reinforcing element
5. Determine weight of reaction frame components
6. Determine system pressure applied to soil reinforcing
7. Place front gate in pullout box
8. Place soil in desired lift thickness and compact to required density
9. Repeat until soil is placed to elevation of soil reinforcing
10. Place soil reinforcing element
11. Place LVDT wire guides on soil reinforcing
12. Place remaining front gate in pullout box
13. Place soil in desired lift thickness and compact to required density
14. Level soil at top of the box.
15. Place neoprene pad on top of soil
16. Place plastic sheet on top of neoprene pad
17. Place expanded pneumatic diaphragm on top of reaction plate.
18. Place neoprene pad on top of pneumatic diaphragm
19. Place reaction plate on top of pneumatic diaphragm
20. Place reaction beams on top of reaction plate
21. Place reaction cross beams on all-thread columns and secure with nut

22. Deflate pneumatic diaphragm. Check position of all reaction frame components in relationship to sides of box.
23. Place load cell reaction cross beams on reaction beams
24. Place load cell and level reaction beam until the bottom of the reaction beam meets the load button on the load cell.
25. Verify that the all-thread columns are secure by tightening the nut on bottom interface of the 2x2 HSST.
26. Move connection plate into proper position. Connect soil reinforcing to connection load plate. Verify that connection plate is level. Secure bolts with air wrench.
27. Place LVDT fixture and LVDT's at front of the pullout box. Level and straighten each LVDT.
28. Connect LVDT at back of pullout box
29. Turn Campbell Scientific on and perform the following:
    a. Zero load cells.
    b. Record the beginning positions of LVDTs
    c. Calculate the position of the LVDT at ¾" deformation and record the value
    d. Expand air bag until required load is applied to the load cells. Verify that loads are equal in the load cell.
30. Turn on hydraulic pump. Verify that the ball valve is in the position required to perform test and is not in "set-up" position.
31. Activate flow control to perform test
32. Stop test 1 ½" total displacement at exit gate
33. Release pressure in hydraulic cylinder
34. Save data from Campbell scientific PC400 software
35. Remove reaction frame
36. Remove pneumatic diaphragm
37. Remove soil to top of soil reinforcing
38. Examine soil reinforcing in place in soil
39. Remove soil reinforcing and examine for deflection and damage

The EarthTrac SS3 soil reinforcing was embedded horizontally between two 6” (152 mm) layers of soil. Then the first six inches (152 mm) of soil was placed and compacted in the bottom of the box at near optimum moisture content to approximately 95% of the maximum dry density. The metallic EarthTrac SS3 soil reinforcing was then placed on the compacted soil and centered in the box. The metallic strip was extended through the 0.5 inch (12.7 mm) slot (i.e., door) at the back box. The slot consists of a “sleeve” that extends into and over the soil. The sleeve consists of two, 6 inch (152 mm) wide horizontal metal plates that are separated by spacers. The sleeve prevents normal stress from transferring to the lead end of the soil reinforcing reducing progressive increased loading at the opening. Then the second layer of 6 in. (152 mm) of sand is placed over the soil reinforcing and sleeve and compacted to the top of the box.

A ¾” rubber bearing pad is placed on the top of the compacted soil. The pneumatic diaphragm is placed on the ¾” bearing pad that is followed by another ¾” bearing pad. A ridged cover plate is placed on the top of the ¾” rubber bearing pad (Photograph 2-6 and Photograph 2-7).





**Photograph 2-6 Cover Plate at Top of Pullout Box**

The reaction frame consists of structural steel cross bracing that is secured to the top of the box before application of the normal stress. For the lowest stress level 6" of soil and steel components is used to apply the normal stress. For the remaining, higher normal stress levels, additional stress (i.e., to the weight of the soil) was applied to the sample by pressurizing the pneumatic diaphragm.

**Photograph 2-7 Normal Load Application**

A variable speed hydraulic ram applies the pullout force. The force is measured by a calibrated load cell attached to the end of the rod. The metallic soil reinforcing specimens are gripped in a clamp consisting of two metal plates and a series of bolts as shown in Figure 2-4. The metallic soil reinforcing is sandwiched between the plates. The plates are tightly bolted together providing a clamping force. The clamping system is attached to the rod end using a swivel connection so the clamp can adjust for elements that were not exactly straight and for uneven pullout scenarios.

The horizontal displacement of the front of the soil reinforcing is monitored outside of the box by two Linear Variable Differential Transducers (LVDTs). The movement is measured by attaching the LVDTs (one on each side) to a small steel plate that is attached to the pullout connection plate. The plunger end of the LVTDs are positioned against the front of the plate. To monitor movements of the embedded portion of the soil reinforcing a wire rope extensometer is used. The lead end of the extensometer is embedded in the soil. The lead end of the extensometer consisted of a wire contained inside a hollow tube housing. The lead end of the wire extensometers is attached to the middle of the soil reinforcing. The LVDTs and load cell are connected to a computer data acquisition system to acquire the data during the tests.

The EarthTrac SS3 soil reinforcing specimens were pulled out at rate of 0.04 in. per minute (1 mm/min) in accordance with the ASTM D 6706 test procedure. Tests were performed until the specimen either pulled out, as defined by at least 3/4 inch (20 mm) of movement at the front end of the specimen (as required by FHWA guidelines), or rupture, if it occurred prior to pullout. Rupture did not occur in any of the specimens. Because of this, and in most cases, pullout tests were performed up to 1.5 inches (75 mm) of movement to identify the load to achieve a maximum peak pullout resistance and obtain a complete pullout deformation curve. The specimens were examined after the test to observe noticeable specimen distortion and/or deformation. No deformation or distortion was noticed on any pullout specimens.

## 3.0 PULLOUT RESISTANCE TEST RESULTS

The pullout test measures the pullout resistance ($P_r$) at ¾" of deformation and at the peak deformation response. The friction factor ($F^*$) is then back calculated using the following equation:

$$F^* = \frac{P_r}{C \cdot \alpha \cdot \sigma_v \cdot w \cdot L_e}$$

| | | | |
|---|---|---|---|
| Where: | $F^*$ | = | friction factor (dim) |
| | $P_r$ | = | pullout resistance (lbf) |
| | $C$ | = | reinforcement effective unit perimeter (C = 2 for strips) (dim) |
| | $\alpha$ | = | a scale effect correction factor (1.0 for inextensible soil reinforcement) (dim) |
| | $\sigma_v$ | = | the effective vertical stress at the soil-reinforcement interfaces (psf) |
| | $w$ | = | width of soil reinforcing (ft) |

$L_e$ = embedment length in soil box (ft)

Table 1 provides a summary of the measured pullout resistance ($P_r$) values for both the peak and ¾ inch failure condition. For inextensible type reinforcements the FHWA 10-024 manual recommends that a maximum movement equal to ¾ inch (20 mm) as measured at the front of the specimen be used to select $P_r$, if the peak value for $P_r$, or rupture of the specimen does not occur before ¾ inch (20 mm) deflection. The friction factor is back calculated and is plotted in Figure 3-1. A load-deflection plot from the testing is shown in the Figure 3-2. In these figures, the LVDT displacements at the back of the box are plotted versus the corresponding load measured from the load cell.

The F* value determined for each normal stress is shown in Table 1 for both the peak and 3/4 inch (20 mm) failure condition. The $\alpha$ factor was determined to be equal to one (1) based on the consistent and concurrent movement of the front and back LVDT's during the test.

Table 1. Measure Pullout Resistance and Calculated Friction Factor

| **Test** | **Name** | **Surcharge psf** | $P_{r\_3/4}$ lbf | $F^*_{3/4"}$ | $P_{r\_peak}$ lbf | $F^*_{peak}$ |
|---|---|---|---|---|---|---|
| 1 | 125-T1 | 125 | 1085 | 6.51 | 1091 | 6.54 |
| 2 | 125-T2 | 125 | 1074 | 6.44 | 1093 | 6.56 |
| 3 | 125-T3 | 125 | 1057 | 6.34 | 1074 | 6.44 |
| 4 | 468-T1 | 486 | 2079 | 3.21 | 2406 | 3.71 |
| 5 | 625-T1 | 635 | 2940 | 3.47 | 3265 | 3.86 |
| 6 | 625-T2 | 630 | 2684 | 3.19 | 2901 | 3.45 |
| 7 | 1250-T1 | 1237 | 3914 | 2.37 | 4461 | 2.70 |
| 8 | 1250-T2 | 1244 | 4080 | 2.46 | 4560 | 2.75 |
| 9 | 1875-T1 | 1853 | 5379 | 2.18 | 5902 | 2.39 |
| 10 | 1875-T2 | 1860 | 4881 | 1.97 | 5646 | 2.28 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 11 | 2500-T1 | 2496 | 5396 | 1.62 | 6245 | 1.88 |
| 12 | 3125-T1 | 3175 | 6172 | 1.46 | 7069 | 1.67 |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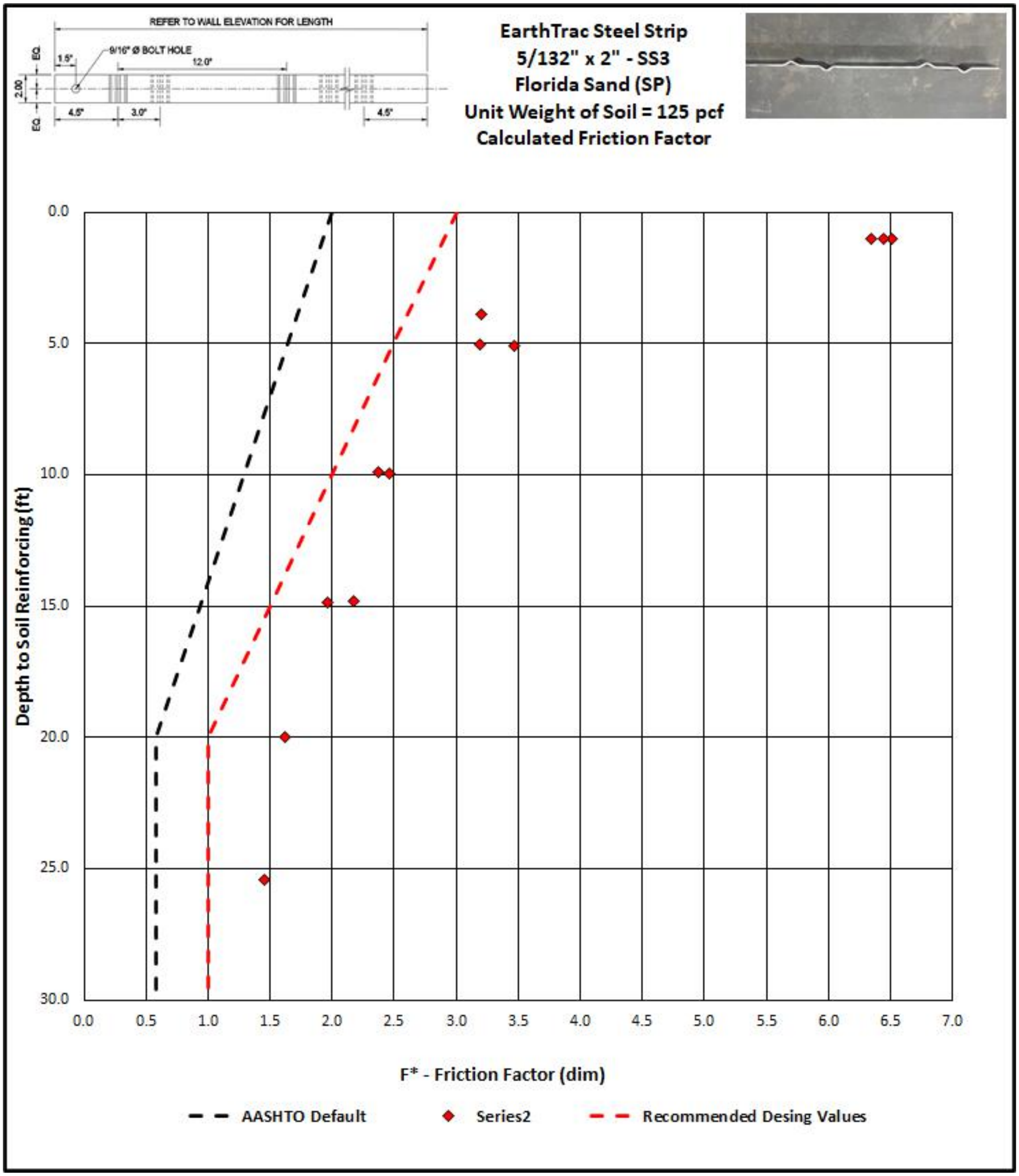


**Figure 3-1.    Friction Factor (F*)**

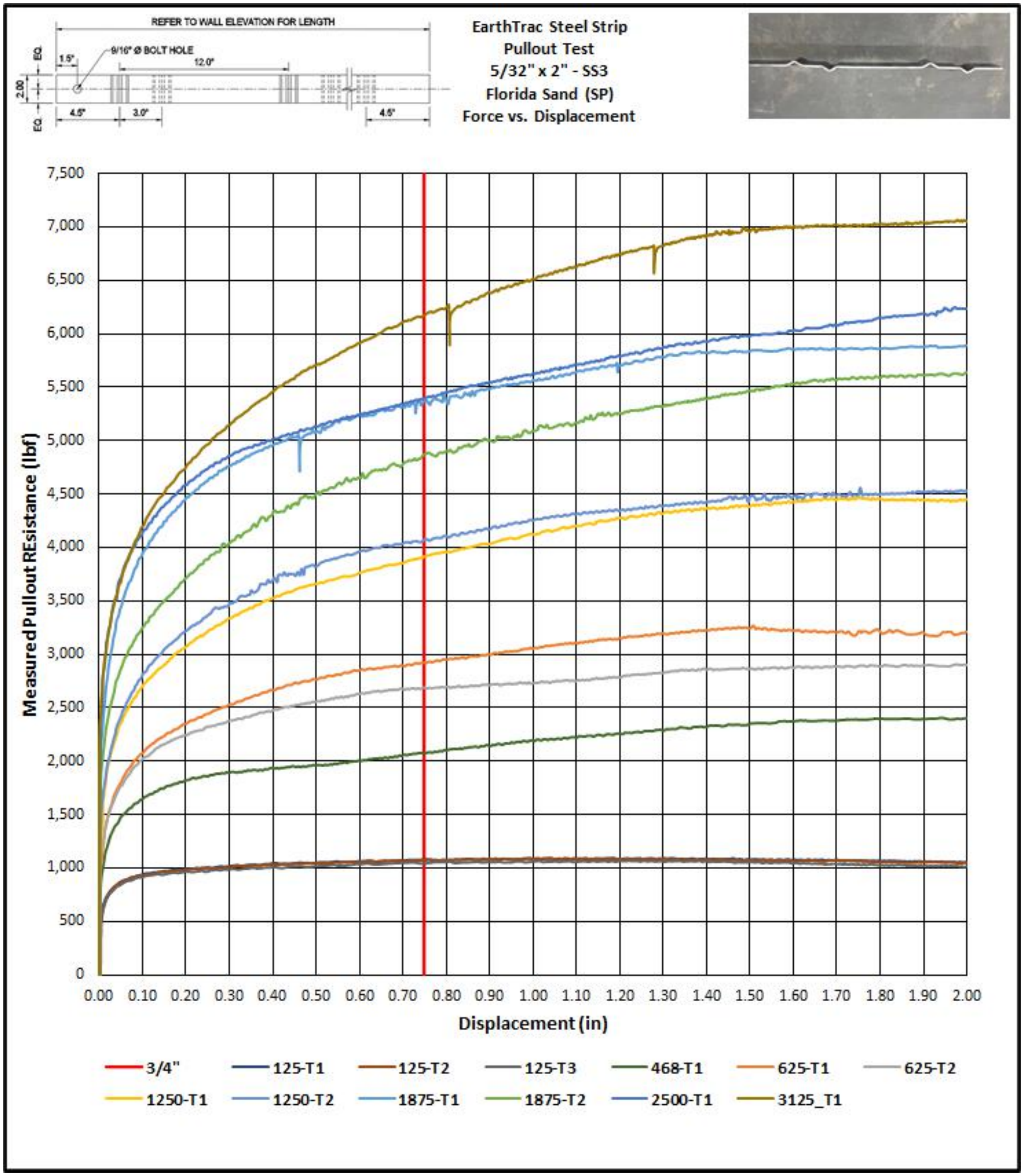
REFER TO WALL ELEVATION FOR LENGTH
9/16" Ø BOLT HOLE
1.5"
12.0"
4.5"
3.0"
4.5"
2.00
EQ
EQ
EarthTrac Steel Strip
Pullout Test
5/32" x 2" - SS3
Florida Sand (SP)
Force vs. Displacement
Measured Pullout REsistance (lbf)
0
500
1,000
1,500
2,000
2,500
3,000
3,500
4,000
4,500
5,000
5,500
6,000
6,500
7,000
7,500
0.00 0.10 0.20 0.30 0.40 0.50 0.60 0.70 0.80 0.90 1.00 1.10 1.20 1.30 1.40 1.50 1.60 1.70 1.80 1.90 2.00
Displacement (in)
3/4"
125-T1
125-T2
125-T3
468-T1
625-T1
625-T2
1250-T1
1250-T2
1875-T1
1875-T2
2500-T1
3125_T1

**Figure 3-2. Measured Force VS Displacement**

## 4.0 SUMMARY AND RECOMMENDATIONS

Based on the pullout testing the friction factor for the EarthTrac SS3 soil reinforcing can conservatively be defined by a linearly decreasing value equal to 3.0 at the top of the structure decreasing to 1.0 at a depth of 20 feet and below. This friction factor is valid for all soil types that are commonly used in MSE structures. Material with greater than 15% fines are not covered in this pullout report.

# APPENDIX A
# POORLY GRADED SAND (SP) TEST DATA

RONE ENGINEERING

8908 Ambassador Row, Dallas, TX 75247
Texas Engineering Firm Registration, No. F-1572
Corporate Phone: (214) 630-9745

## Report of Sieve Analysis

**Client:** Ground Improvement Systems, LLC
114 South Collins Street
Arlington, TX 76011

**Report No.:** 14284877
**Project No.:** 2024753
**Date of Service:** 11/16/2020
**Report Date:** 11/17/2020

**Project:** Miscellaneous Laboratory Testing
**Services:** Sieve Analysis Testing

Description: Light Brown Poorly-Graded Sand
Sample Weight: 446.0
Source: Client Delivered

| Sieve Size | Cumulative Weight Retained | Percent Passing |
|---|---|---|
| 3/8 | 0.0 | 100 |
| #4 | 1.6 | 99.6 |
| #8 | 4.3 | 99.0 |
| #16 | 8.4 | 98.1 |
| #30 | 15.6 | 96.5 |
| #50 | 28.9 | 93.5 |
| #100 | 341.7 | 23.4 |
| #200 | 443.6 | 0.5 |
| PAN | 446.0 | - |

**Test Methods:** ASTM C-136

**Technician:** Shane Tandy
**Report Distribution:**
Orig: Ground Improvement Systems, LLC (Arlington, TX)
Attn: Mr. Thomas Taylor (1-ec copy)
1-ec Rone Engineering Project Managers
Attn: Mr. Scott Watson, CET

**Rone Engineering**

K. Scott Watson, CET
Project Manager

8908 Ambassador Row, Dallas, TX 75247
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Corporate Phone: (214) 630-9745

# Report of Moisture Density Relationship

**Client:** Ground Improvement Systems, LLC
114 South Collins Street
Arlington, TX 76011

**Report No.:** 14284001
**Project No.:** 2024753
**Date of Service:** 11/06/2020
**Report Date:** 11/10/2020

**Project:** Miscellaneous Laboratory Testing

**Services:** Obtain a sample of material from the jobsite, bring the sample back to the laboratory and perform a moisture density relationship test in accordance with ASTM Standards.

## PROJECT DATA

**Contractor:**
**Test For:** Pull-out Sand
**Material:** Light Brown Poorly-Graded Sand
**Classification:** Pull-out Sand
**Test Method:** ASTM D-4318 Method-B
ASTM D-698 Method-B
ASTM D-1140

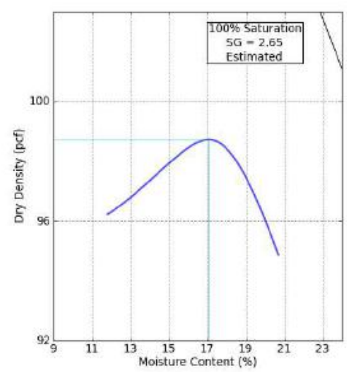
**Date Sampled:** 11/06/2020
**Material Preparation:** Dry
**Rammer Type:** Mechanical
**Sampled By:** Client
**Tested By:** Willie Wagner
**Sample Location:**
Client Delivered

## REPORT OF TEST

Maximum Dry Density, pcf: 98.70 pcf
Optimum Moisture Content, %: 17.1 %

**Liquid Limit:** NP
**Plastic Limit:** NP
**Plasticity Index:** NP

**% Passing #200:** 3




**Technician:** Shane Tandy
**Report Distribution:**
Orig: Ground Improvement Systems, LLC (Arlington, TX)
Attn: Mr. Thomas Taylor (1-ec copy)
1-ec Rone Engineering Project Managers
Attn: Mr. Scott Watson, CET

**Rone Engineering**

K. Scott Watson, CET
Project Manager

LIMITATIONS: The test results presented herein were prepared based upon the specific samples provided for testing. We assume no responsibility for variation in quality (composition, appearance, etc.) or any other feature of similar subject matter provided by persons or conditions over which we have no control. Our letters and reports are for the exclusive use of the clients to whom they are addressed and shall not be reproduced except in full without the written approval of Rone Engineering Services, Ltd. (SW/SW)

Page 1 of 1

**1.2.17**
**Soil Reinforcing Interface Shear Connection Strength**

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# 1.3
# OTHER COMPONENTS



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## 1.3.1
## Other Component Innovations

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v

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**1.3.2**
**Reinforced Soil Properties**

EARTHTEC

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# EARTHTEC REINFORCED SOIL WALL
# TECHNICAL SPECIFICATION
# BACKFILL

GROUND IMPROVEMENT SYSTEMS LLC

114 South Collins Street

Arlington, TX 76011

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www.groundimprovementsystems.com

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# MECHANICALLY STABILIZED EARTH STRUCTURES

## REINFORCED SOIL WALL (RWS) SYSTEM

## TECHNICAL SPECIFICATION

## BACKFILL

GROUND IMPROVEMENT SYSTEMS LLC

114 South Collins Street

Arlington, TX 76011

817-223-0969





**CONTENTS**

## MSE Backfill Technical Specification

### 1 General

This specification pertains to the selection of the backfill for use in the Mechanically Stabilized Earth retaining wall. Backfill shall be tested and placed according to this specification in reasonably close conformity to the dimensions shown on the plans or established by the Engineer.

### 2 Referenced Specifications

#### 2.1 American society for Testing and Materials (ASTM)

**2.1.1** C117 - Material finer than No. 200 sieve in Mineral Aggregate by Washing

**2.1.2** C136 - Sieve Analysis of Fine and Corse Aggregate

**2.1.3** D1248 - Standard Specification for Polyethylene Plastics Extrusion Materials

**2.1.4** D1556 - Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method

**2.1.5** D2922 - Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth)

**2.1.6** D3034 - Standard Specification for Type PSM Poly(Vinyl Chloride) (PVC) Sewer Pipe and Fittings

**2.1.7** D3080 - Direct Shear Test of Soils Consolidated Drained Conditions

#### 2.2 American Association of State Highway and Transportation Officials

**2.2.1** M208 - Standard Specification for Geotextile Specification for Highway Applications

**2.2.2** T-27 - Sieve Analysis of Fine and Corse Aggregate

**2.2.3** T11 - Materials Finer than No. 200 Sieve in Mineral Aggregate by Washing

**2.2.4** T90 - Determining the Plastic Limit and Plasticity Index in Soils

**2.2.5** T99 - Moisture-Density Relations of Soils Using a 5.5 lb Rammer and 12 Inch Drop

**2.2.6** T104 - Standard Method of Test for Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate

**2.2.7** T180 - Moisture-Density Relations of Soils Using a 10 lb Rammer and 18 Inch Drop

**2.2.8** T191 - Density of Soil in Place by the Sand Cone Method

**2.2.9 T236 - DIRECT SHEAR TEST OF SOILS CONSOLIDATED DRAINED CONDITIONS**

**2.2.10 T238 - DENSITY OF SOIL AND SOIL-AGGREGATE IN-PLACE BY NUCLEAR METHODS (SHALLOW DEPTH)**

**2.2.11 T267 - DETERMINATION OF ORGANIC CONTENT IN SOILS BY LOSS ON IGNITION**

**2.2.12 T288 - DETERMINING MINIMUM LABORATORY SOIL RESISTIVITY**

**2.2.13 T289 - DETERMINING PH OF SOIL IN USE IN CORROSION TESTING**

**2.2.14 T290 - DETERMINING WATER SOLUBLE SULFATE ION CONTENT IN SOIL**

**2.2.15 T291 - DETERMINING WATER SOLUBLE CHLORIDE ION CONTENT IN SOIL**

# 3 SELECT GRANULAR BACKFILL

## 3.1 GRADATION

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 minimum gradation limits as determined by AASHTO T-27. Alternatively material shall be well graded in conformance with Unified Soil Classification in ASTM D2487. Further the backfill shall not be gap graded.

**Backfill Gradation**

| U.S. Sieve Size | Percent Passing |
|---|---|
| 3" | 100 |
| No. 40 | 0-60 |
| No. 200 | 0-15 |

## 3.2 ADDITIONAL REQUIREMENTS

### 3.2.1 PLASTICITY INDEX

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

### 3.2.2 FINE MATERIAL

Fraction finer than 15-micron size shall not exceed 15 percent

### 3.2.3 INTERNAL FRICTION ANGLE

The material shall exhibit an angle of internal friction of not less than 30 degrees, as determined by the standard Direct Shear Test, AASHTO T-236, on the portion finer than the #10 sieve, utilizing a sample of the material compacted to 95 percent of AASHTO T-99, 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 ¾ inch.

### 3.2.4 SOUNDNESS

The material shall be substantially free of shale or other soft, poor durability particles. The materials shall have a magnesium sulfate soundness loss of less than 20 percent after five (5) cycles, as determined by AASHTO T-104.

#### 3.2.5 Electrochemical Requirements

For systems using steel reinforcement, the material shall conform to the following electrochemical requirements:

**Electrochemical Requirements**

| Property | Requirement | Test Method |
|---|---|---|
| Resistivity | >3000 ohm-cm | AASHTO T-288-91 |
| pH | 5-10 | AASHTO T-289-91 |
| Chlorides | <100 ppm | AASHTO T-291-91 |
| Sulfates | <200 ppm | AASHTO T-290-91 |
| Organic Content | <1% | AASHTO T-267-86 |

#### 3.2.6 Certification

The Contractor shall furnish to the Engineer a Certificate of Compliance certifying that the select granular backfill material complies with this section of the specifications. A copy of all test results performed by the Contractor, which are necessary to assure compliance with the specifications, shall also be furnished to the Engineer.

#### 3.2.7 Rejection

Backfill not conforming to this specification shall not be used without the written consent of both the Engineer and the wall supplier.

#### 3.2.8 Sampling

The frequency of sampling of select granular backfill material, necessary to assure gradation control throughout the construction, shall be directed by the Owner.

## 4 Drainage Aggregate

### 4.1 Gradation

Drainage aggregate shall be a clean; washed; 1-inch minus stone or granular fill meeting the following gradation:

**Drainage Aggregate Gradation**

| Sieve Size | Percent Passing |
|---|---|
| 1 1/2” | 100 |
| 1” | 75-100 |
| 3/4” | 50-75 |
| No. 4 | 0-60 |
| No. 40 | 0-50 |
| No. 200 | 0-5 |

### 4.2 Drainage Pipe

Drainage pipe shall be manufactured in accordance with ASTM D3034 and/or ASTM D1248.

### 4.3 Filter Fabric

Drainage pipe shall be encased in gravel filled trench. The gravel shall be wrapped in a filter fabric as approved by the Wall Design Engineer and in conformance with AASHTO M288.

# 5 CONSTRUCTION REQUIREMENTS

## 5.1 TECHNICAL ASSISTANCE

A qualified Technical Assistant as specified by T&B Structural Systems shall be provided during construction at a minimum as specified in the contract documents. Contractor and T&B Structural Systems shall negotiate duration of technical assistance.

## 5.2 SITE PREPARATION

All portions of the area to be filled shall be stripped of vegetation, roots, topsoil or other organic soil, peat, trash, or other materials that are determined to be deleterious.

## 5.3 FOUNDATION PREPARATION

The foundation for the structure shall be graded level for a width equal to or exceeding the length of the soil reinforcement as shown on the plans. Foundation shall be compacted as directed by the Owner. The contractor shall remove and replace with compacted fill any unsuitable sub-grade material as determined by the Owner's Engineer.

## 5.4 LEVELING COURSE

The leveling course shall be excavated and formed to the depths and dimensions as shown in the construction documents. The leveling course is classified as non-structural concrete and shall have a minimum compressive strength of 3000 psi

## 5.5 WALL INSTALLATION

### 5.5.1 PANEL PLACEMENT

Precast panels shall be placed vertically and horizontally by the aid of a crane. Panels shall be placed in and handled by the lifting devices provided by the wall supplier. Welded wire panels shall be placed by hand.

Vertical and horizontal alignment shall be maintained at all times. Hard wood shims may be used in horizontal joints to maintain proper alignment for concrete panels. Hard wood shims shall be removed at completion of the wall erection. Horizontal tolerance shall be ½" (13 mm) per panel. Vertical tolerance shall be 1" (25 mm) per 10 feet (3000 mm) of vertical height inward. No initial outward deformation will be allowed.

### 5.5.2 SOIL REINFORCEMENT PLACEMENT

Soil reinforcement shall be placed and attached to the panels as detailed in the shop drawings and in close proximity to the lines and grades as dictated in the wall elevation drawings. Prior to placement of soil reinforcement backfill shall be compacted in accordance with Section 3.5.3 Backfill Placement.

### 5.5.3 BACKFILL PLACEMENT

#### 5.5.3.1 LIFT THICKNESS

The reinforced select fill shall be placed as shown in construction plans in maximum 12 inch loose lifts and compacted to a minimum 95% of maximum density as determined by AASHTO T-99 Method C or D. Select fill shall be placed and compacted in such a manner that eliminates the development of movement of the soil reinforcement and wall face. Fill shall be spread in a direction moving from the back face of the panel to the tail of the soil reinforcing.

#### 5.5.3.2 MOISTURE CONTENT

The moisture content of the backfill shall be maintained evenly throughout the backfill at all times. Backfill shall have a moisture content of 3% less than or equal to the optimum moisture content as determined by AASHTO T99 (Standard Proctor) or T180 (Modified Proctor). Any backfill material exceeding the optimum moisture content will be removed, reworked until moisture content is acceptable throughout the entire lift.

#### 5.5.3.3 COMPACTION REPORTS

Compaction tests and gradation tests should be taken and recorded in accordance with the contract plans. At a minimum at least one test per 2000 ft$^2$ per 30 inches of fill thickness shall be performed. Each density test shall record the station number, elevation and distance behind the wall face in the testing log. These reports shall be made part of the Wall Installers log.

#### 5.5.3.4 COMPACTION OF 3-FOOT ZONE

The area directly behind the concrete panel and extending 3 feet into the reinforced backfill zone shall be compacted using a lightweight mechanical tamper or roller system. The backfill in this zone shall be compacted to a minimum 95% of maximum density as determined by AASHTO T99 (Standard Proctor) or T180 (Modified Proctor). When applicable the 3-foot zone may be required to be placed at 90% of maximum dry density.

#### 5.5.3.5 UNSUITABLE BACKFILL

The contractor shall remove and replace, at his own expense, any fill that is deemed unacceptable by the Owner or Wall Supplier.

#### 5.5.3.6 CONSTRUCTION EQUIPMENT

Tracked construction equipment shall not be operated directly on the soil reinforcement. A minimum backfill thickness of 6 inches is required prior to operation of tracked vehicles over the reinforcement. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and damaging the reinforcement. Rubber-tired equipment may pass over the reinforcement, if in accordance with the manufacturer's recommendations, at slow speeds less than 10 mph. Sudden braking and sharp turning should be avoided.

#### 5.5.3.7 End of Day Precautions

The backfill fill surface shall be sloped to rapidly direct run-off away from the face of the wall and to prevent ponding of surface water. During periods of anticipated inclement weather, the surface of the fill shall be sealed with an impervious membrane. If ponding of surface water does occur, the water shall be removed and the backfill replaced or allowed to dry to project requirements. All drainage elements such as catch basins and inlets shall be properly sealed to prevent surface runoff from entering the construction site and to prevent the backfill from washing out of the reinforced volume of soil.

## 1.3.3
## ERS Drainage

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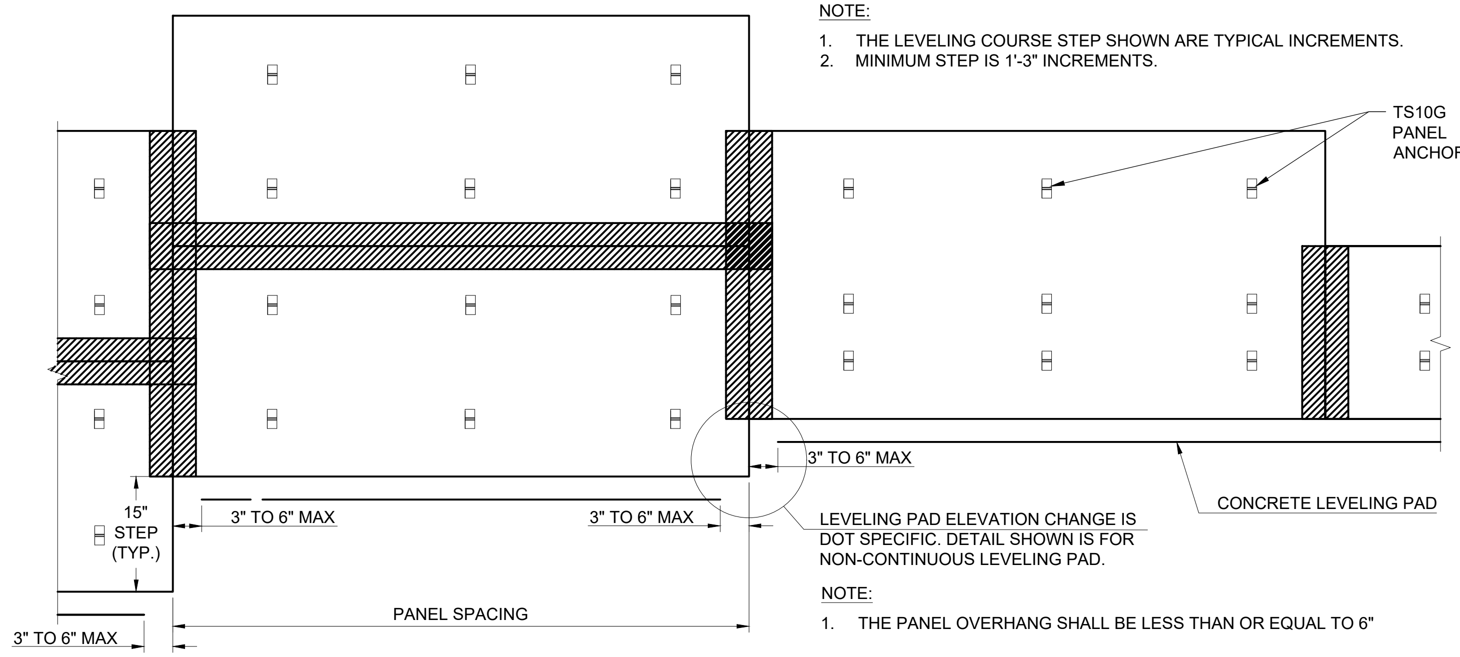



NOTE:

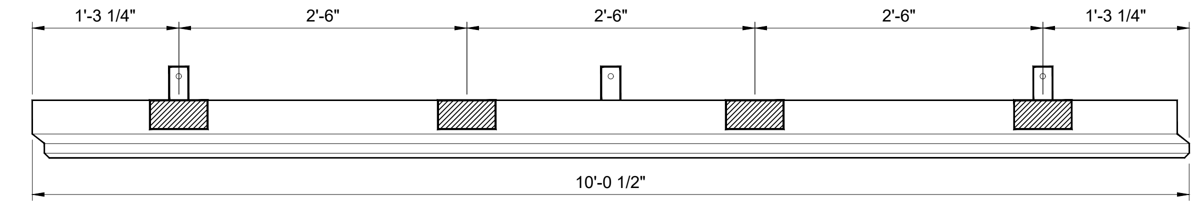
1. THE LEVELING COURSE STEP SHOWN ARE TYPICAL INCREMENTS.
2. MINIMUM STEP IS 1'-3" INCREMENTS.

NOTE:

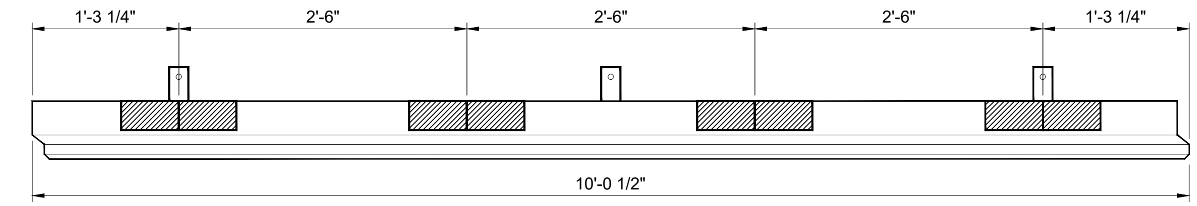
1. THE PANEL OVERHANG SHALL BE LESS THAN OR EQUAL TO 6"

A / D2 **LEVELING PAD AND PANEL PLACEMENT**

BACK FACE ELEVATION




4-BEARING PADS



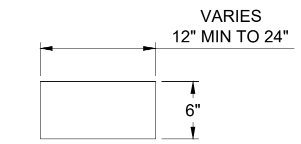

8-BEARING PADS

NOTE:

1. FOR WALL HEIGHTS THAT EXCEED 40 FEET USE 8 BEARING PADS PER PANEL AND PLACE SIDE BY SIDE.
2. WALL HEIGHT IS MEASURED FROM TOP OF LEVELING PAD TO TOP OF PANEL.
3. BEARING PAD IS 60 DUROMETER 3/4" x 3" x 6"

D / D2 **BEARING PAD PLACEMENT**

PLAN VIEW AT TOP OF PANEL

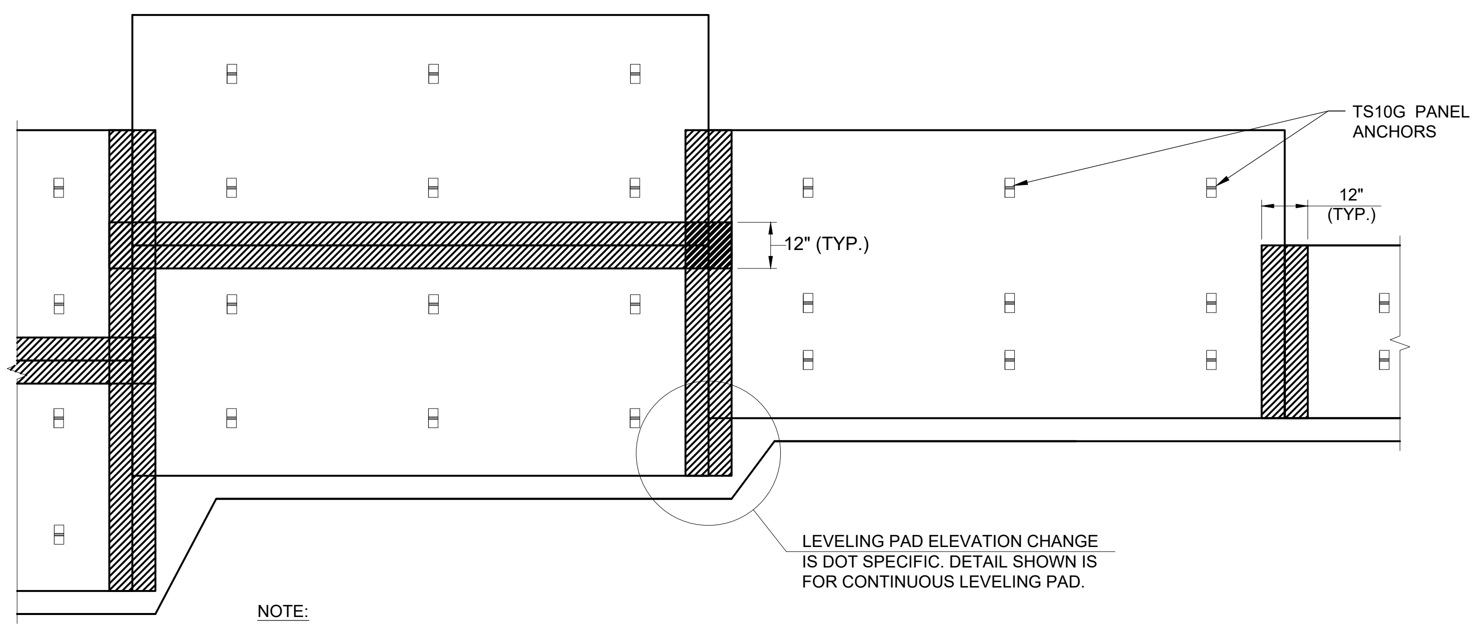



NOTE:

1. MINIMUM COMPRESSIVE STRENGTH SHALL BE 3000 PSI
2. CURE TIME SHALL BE A MINIMUM OF 12 HOURS
3. DIMENSIONS SHOWN ARE MINIMUM
4. REINFORCING IS NOT REQUIRED
5. NO PRECAST LEVELING PADS ARE ALLOWED

E / D2 **LEVELING PAD DETAIL**

CROSS SECTION

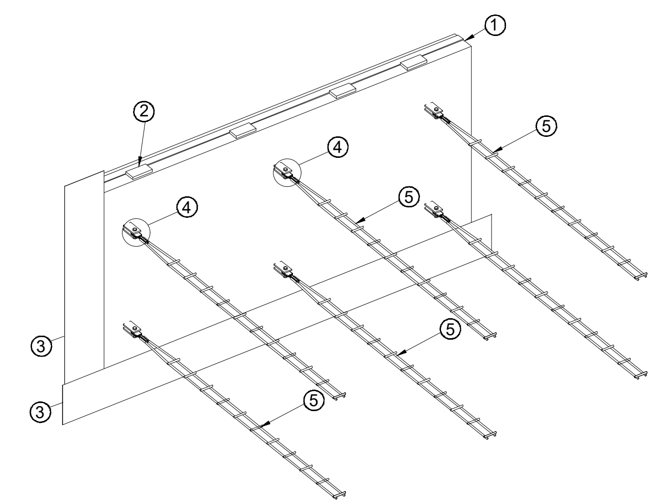



NOTE:

1. STRIPS OF FILTER FABRIC CLOTH SHALL BE PLACED OVER ALL VERTICAL AND HORIZONTAL JOINTS.
2. FILTER FABRIC SHALL BE ADHERED TO BACK FACE OF PANELS USING AN ADHESIVE COMPOUND.
3. ADHESIVE SHALL BE APPLIED TO PANEL THEN FILTER FABRIC PLACED ON BACK OF PANEL.
4. ADHESIVE BEAD IS NOT REQUIRED TO BE CONTINUOUS.
5. THERE SHALL BE A 12" OVERLAP BETWEEN SPLICED FILTER FABRIC.
6. HORIZONTAL JOINT FABRIC SHALL BE EXTENDED 6" PASSED VERTICAL JOINT.

B / D2 **FILTER FABRIC PLACEMENT**

BACK FACE ELEVATION




NOTE:

1. TYPICAL PRE-CAST PANEL WITH CAST-IN-PLACE ANCHORS
2. 60 DUROMETER 3" x 6" x 3/4" BEARING PADS - 2 PER PANEL (UNLESS NOTED OTHERWISE)
3. 12" WIDE FILTER FABRIC CLOTH ADHERE WITH ADHESIVE
4. 1/2" Ø A325 BOLT SET - 1 PER EARTHTRAC
5. EARTHTRAC ELEMENTS (SIZE AS REQUIRED)
6. CONSTRUCTION ADHESIVE (NOT SHOWN) TO ATTACH FILTER FABRIC TO BACK FACE OF PANEL
7. HARD WOOD SHIMS (USE IF NECESSARY BY OTHERS - NOT SHOWN)

C / D2 **TYPICAL MATERIAL ISOMETRIC**

STANDARD TYPE G PANEL

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SPECIAL DETAILS
SHEET 2 OF 4

REINFORCED SOIL WALL
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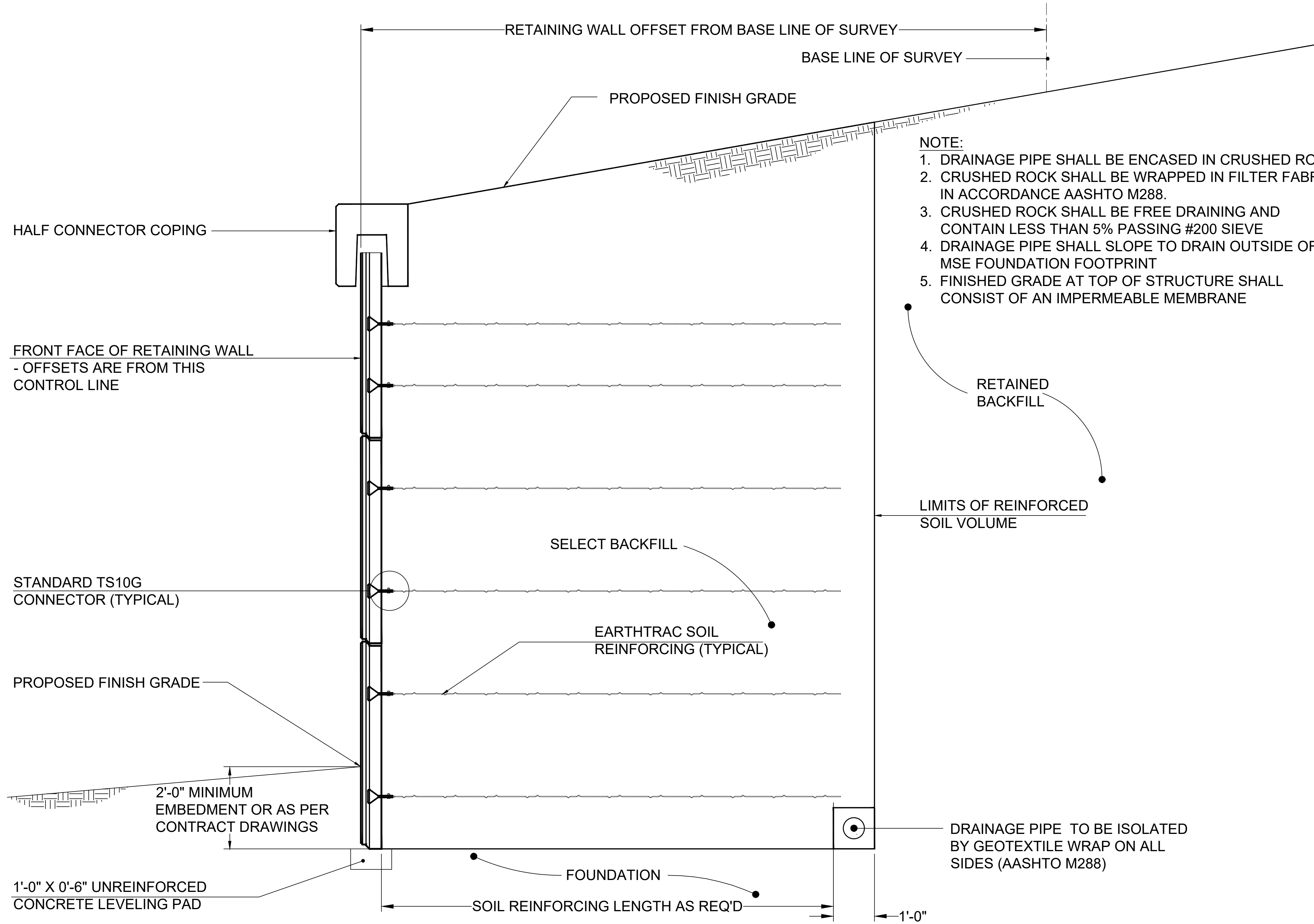


NOTE:
1. DRAINAGE PIPE SHALL BE ENCASED IN CRUSHED ROCK
2. CRUSHED ROCK SHALL BE WRAPPED IN FILTER FABRIC IN ACCORDANCE AASHTO M288.
3. CRUSHED ROCK SHALL BE FREE DRAINING AND CONTAIN LESS THAN 5% PASSING #200 SIEVE
4. DRAINAGE PIPE SHALL SLOPE TO DRAIN OUTSIDE OF MSE FOUNDATION FOOTPRINT
5. FINISHED GRADE AT TOP OF STRUCTURE SHALL CONSIST OF AN IMPERMEABLE MEMBRANE

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## DRAINAGE OF SELECT BACKFILL

TYPICAL CROSS SECTION

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MSE DRAINAGE DETAILS
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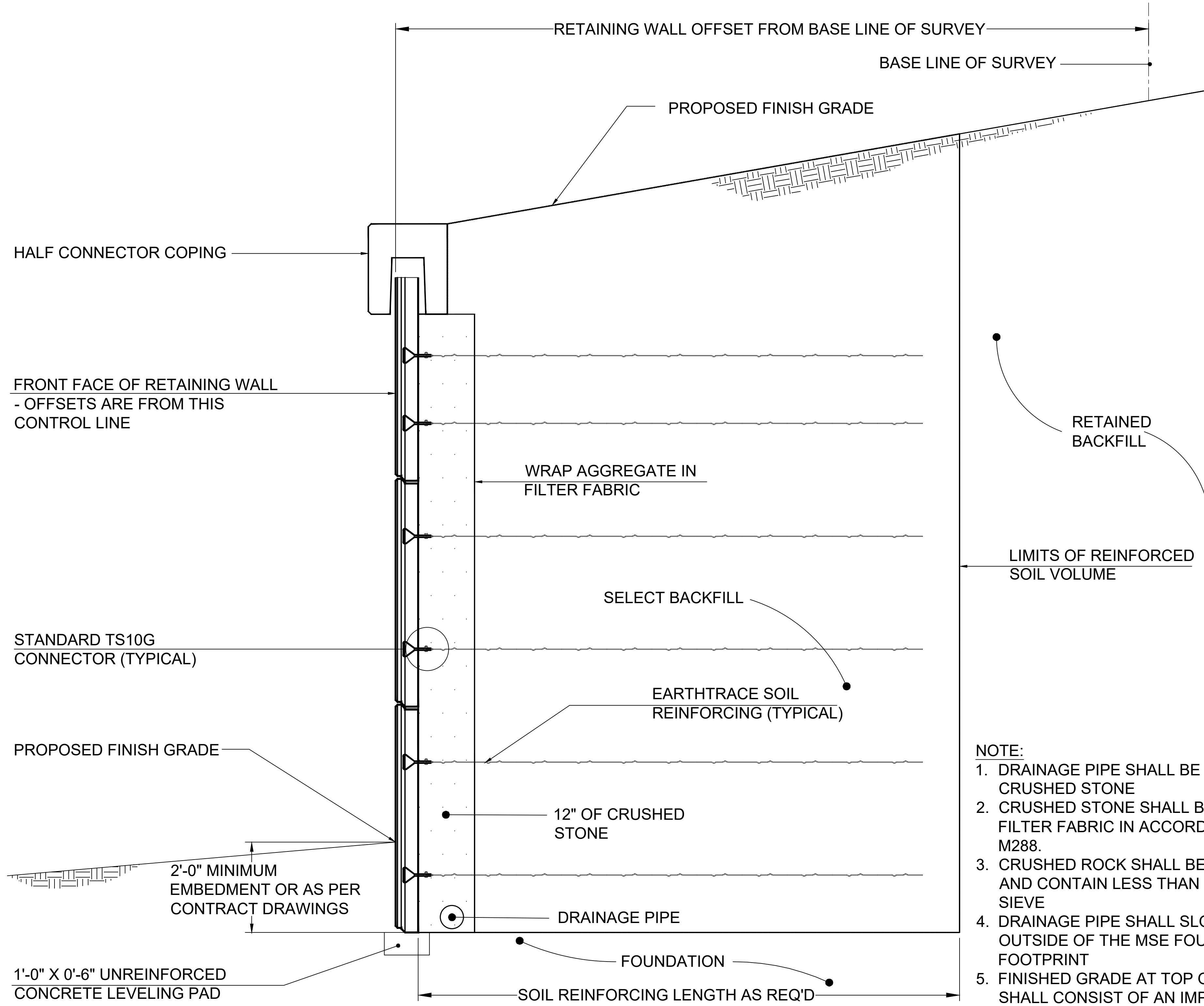


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5. FINISHED GRADE AT TOP OF STRUCTURE SHALL CONSIST OF AN IMPERMEABLE MEMBRANE

A / DR2 DRAINAGE AT FACE OF MSE
TYPICAL CROSS SECTION

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MSE DRAINAGE DETAILS
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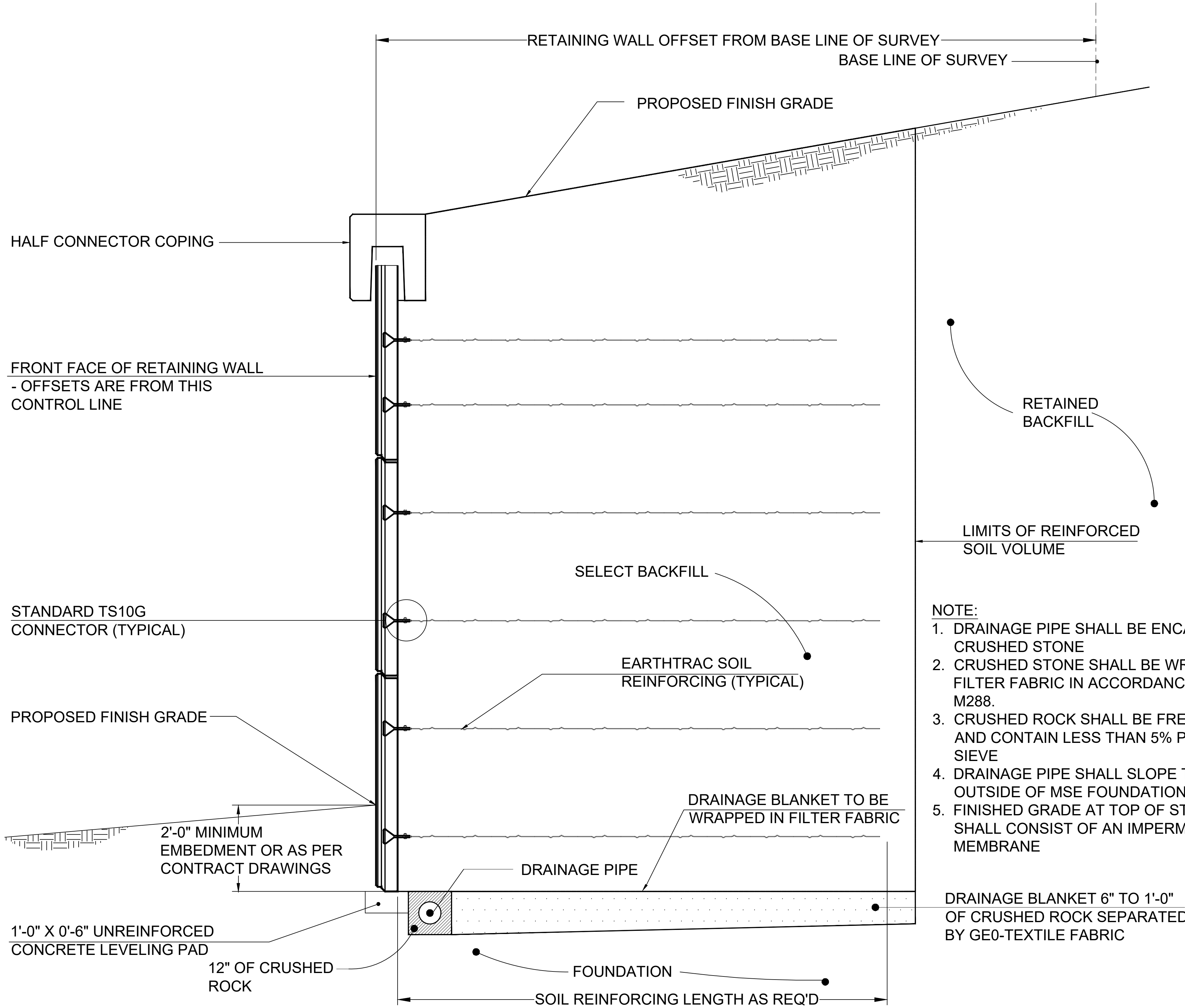


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5. FINISHED GRADE AT TOP OF STRUCTURE SHALL CONSIST OF AN IMPERMEABLE MEMBRANE

A / DR3

## DRAINAGE AT BASE OF MSE

TYPICAL CROSS SECTION

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MSE DRAINAGE DETAILS
SHEET 3 OF 5

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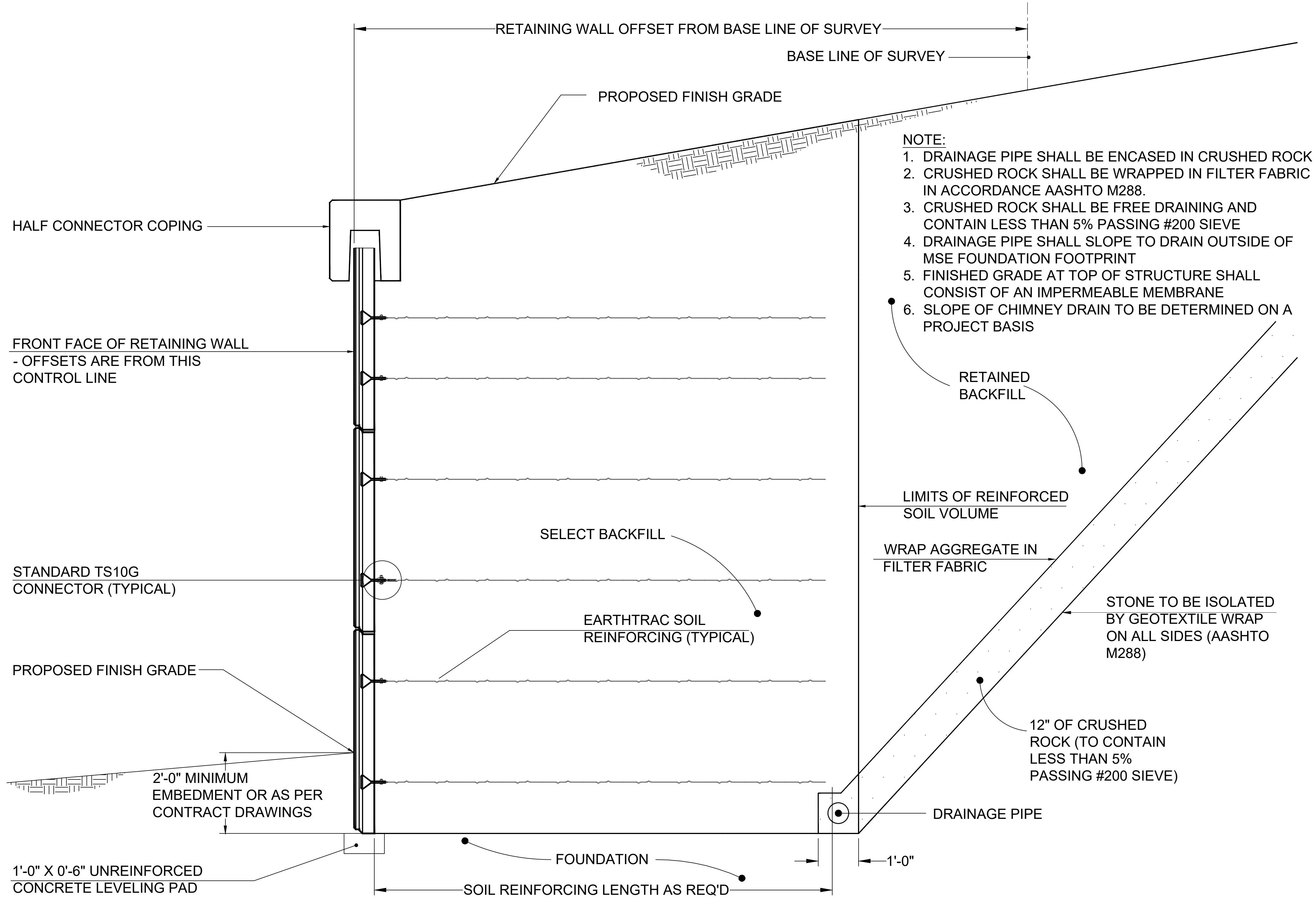


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3. CRUSHED ROCK SHALL BE FREE DRAINING AND CONTAIN LESS THAN 5% PASSING #200 SIEVE
4. DRAINAGE PIPE SHALL SLOPE TO DRAIN OUTSIDE OF MSE FOUNDATION FOOTPRINT
5. FINISHED GRADE AT TOP OF STRUCTURE SHALL CONSIST OF AN IMPERMEABLE MEMBRANE
6. SLOPE OF CHIMNEY DRAIN TO BE DETERMINED ON A PROJECT BASIS

A
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## DRAINAGE OF RETAINED FILL AT BACK OF MSE

TYPICAL CROSS SECTION

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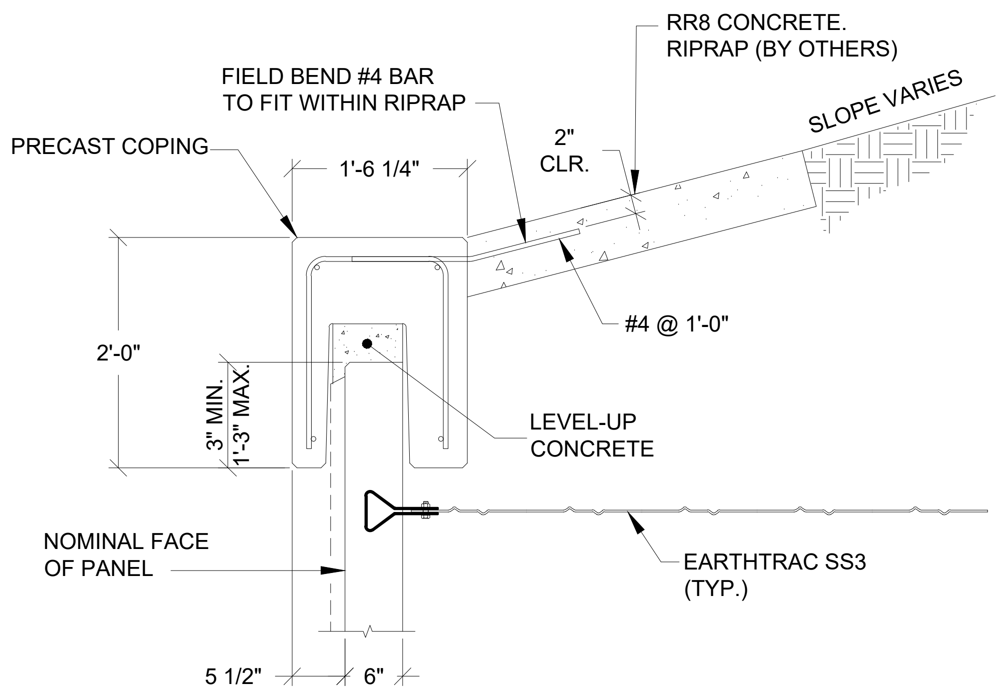


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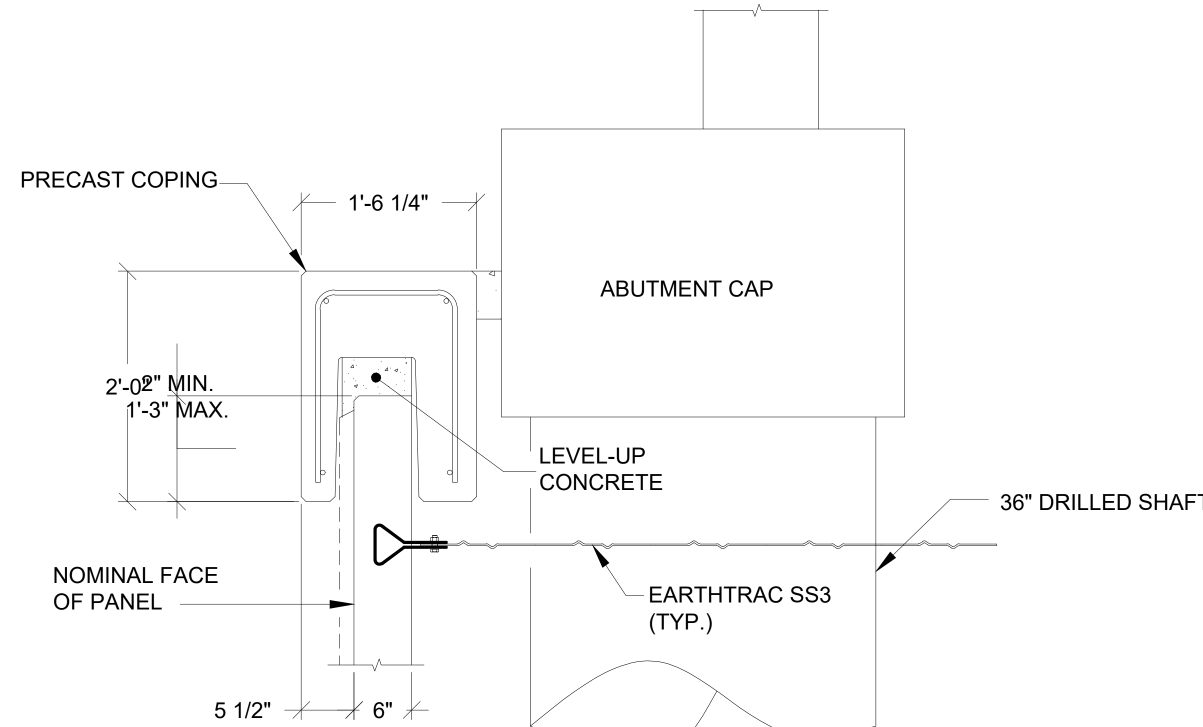


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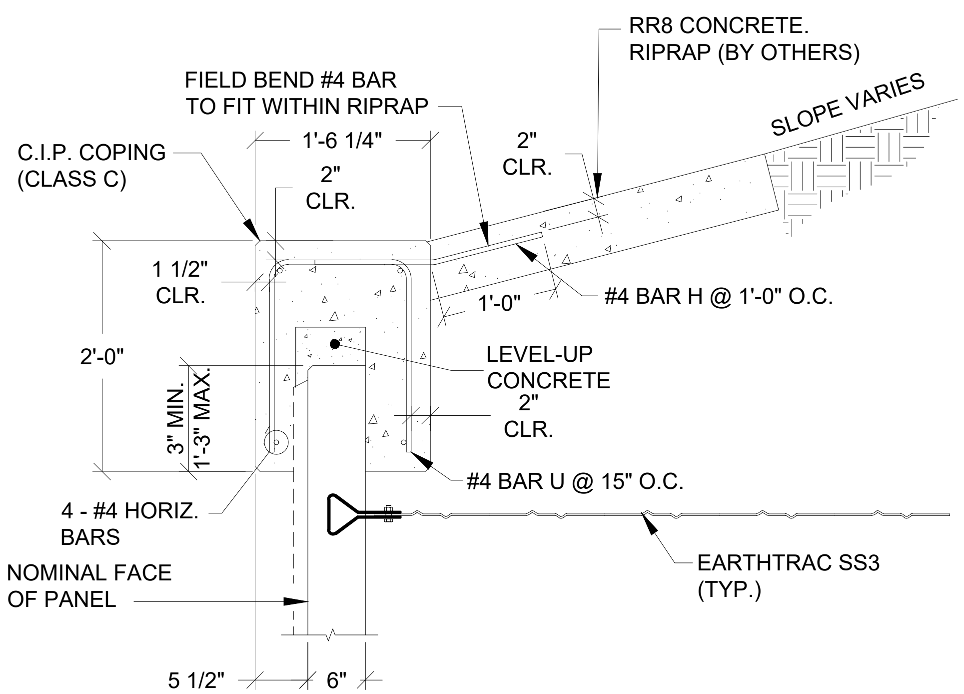


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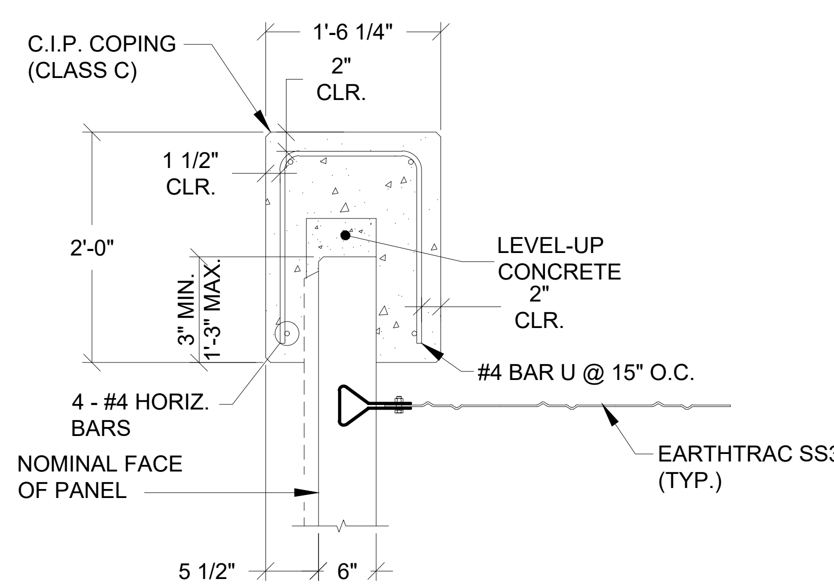


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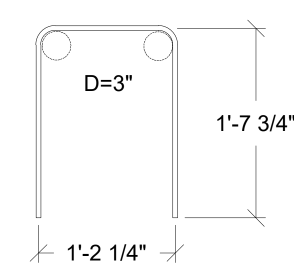


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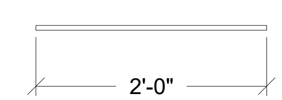


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SCP WITH EARTHTRAC SOIL REINFORCING

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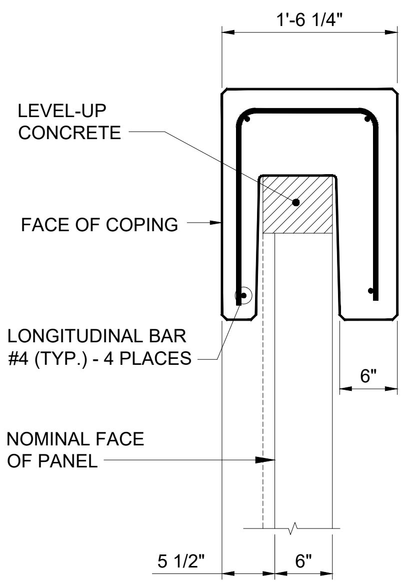
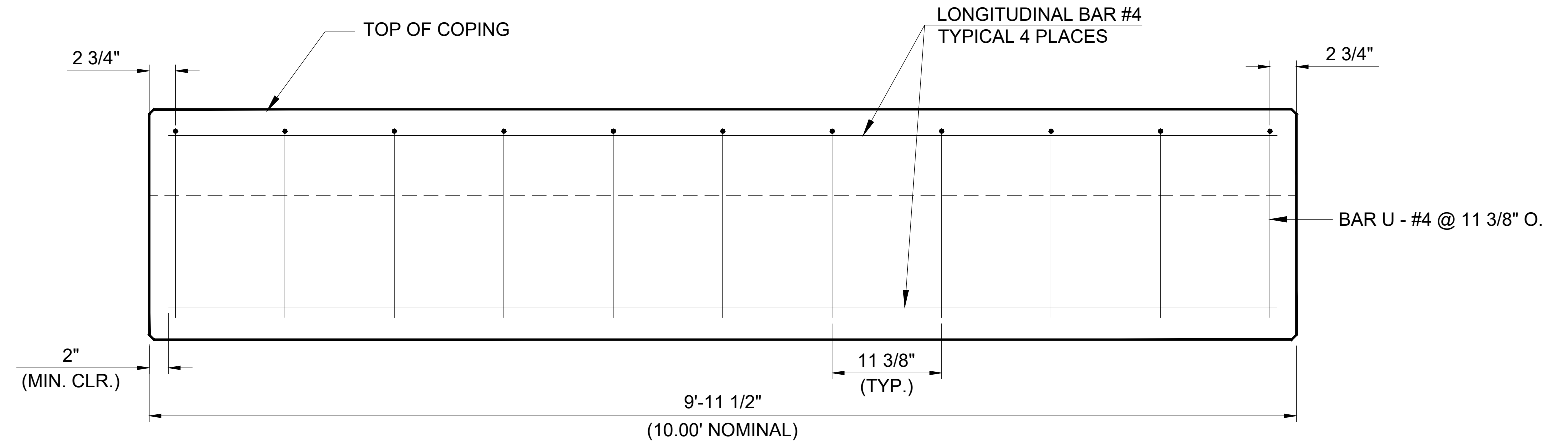
NOTES:

1. ALL LONGITUDINAL BARS ARE #4 PLACED AS SHOWN WITH EQUAL SPACING.
2. ALL DIMENSIONS ARE TO CENTER OF BAR UNLESS OTHERWISE NOTED.
3. MINIMUM COMPRESSIVE STRENGTH OF CONCRETE SHALL BE 4000 PSI FOR PRECAST AND 3600 PSI FOR CAST IN PLACE.



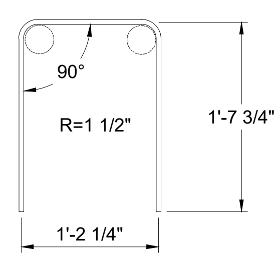

A / C2 PRECAST COPING @ ABUTMENT

PARTIAL SECTION

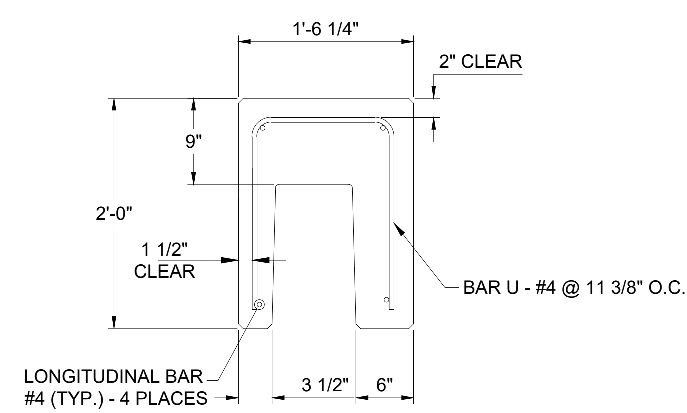



C / C2 PRECAST COPING @ ABUTMENT

FRONT FACE ELEVATION




#4 BAR U




B / C2 PRECAST COPING @ ABUTMENT - REINFORCING PLACEMENT

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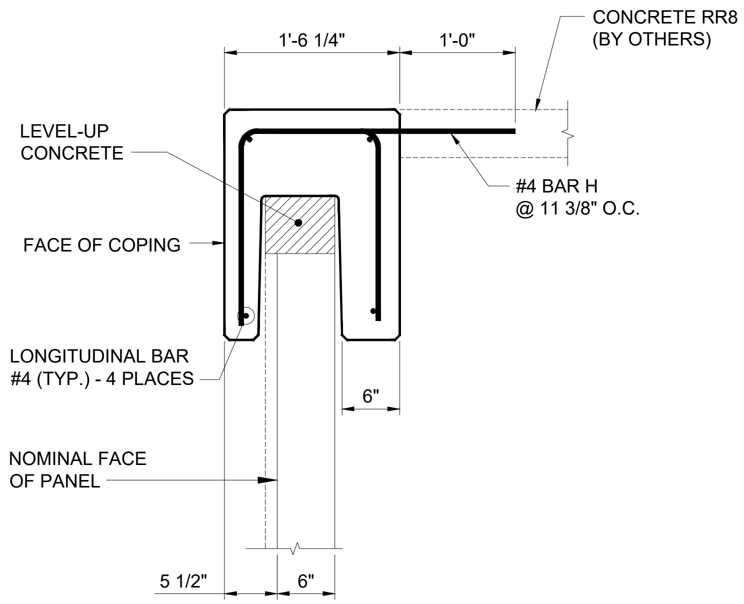
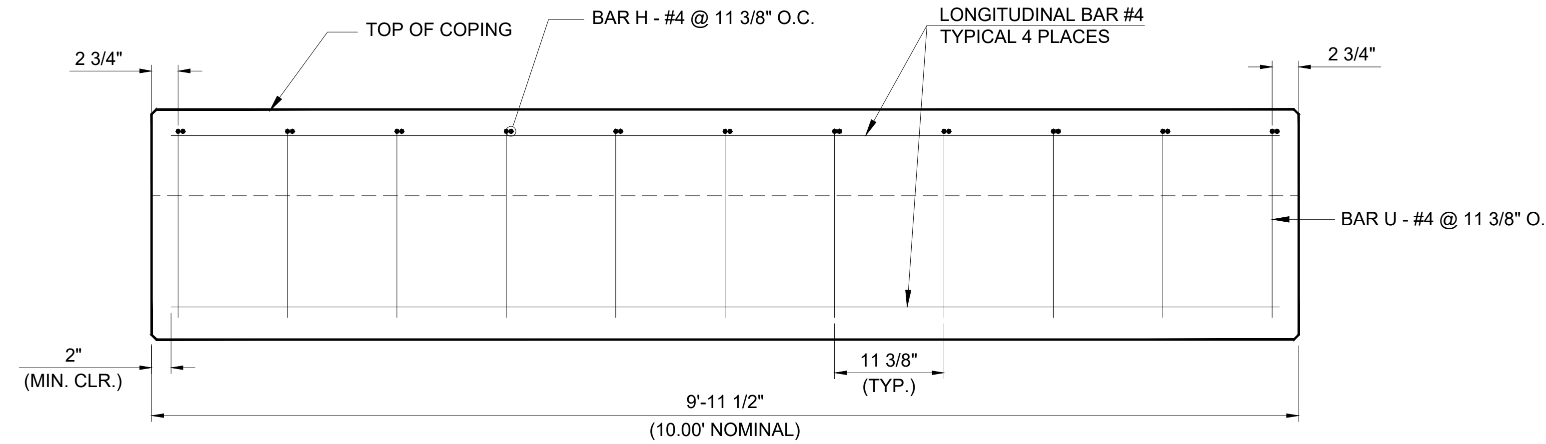


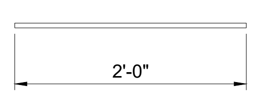

A / C3 PRECAST COPING WITH CONCRETE RR8 RIPRAP

PARTIAL SECTION

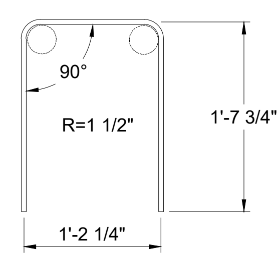



C / C3 PRECAST COPING WITH CONCRETE RR8 RIPRAP

FRONT FACE ELEVATION




#4 BAR H

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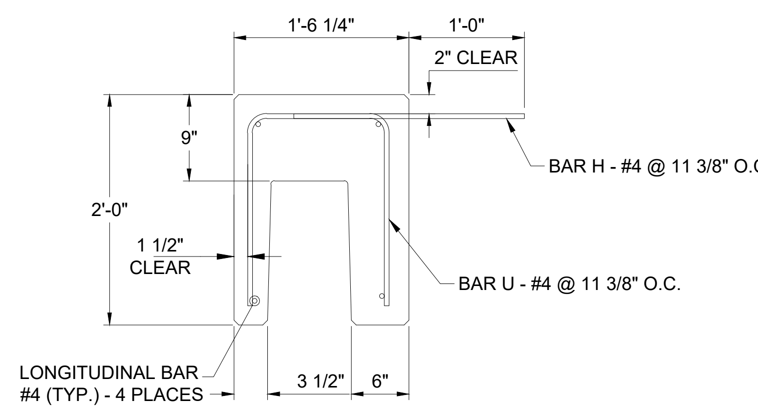


B / C3 PRECAST COPING W/CONCRETE RR8 RIPRAP

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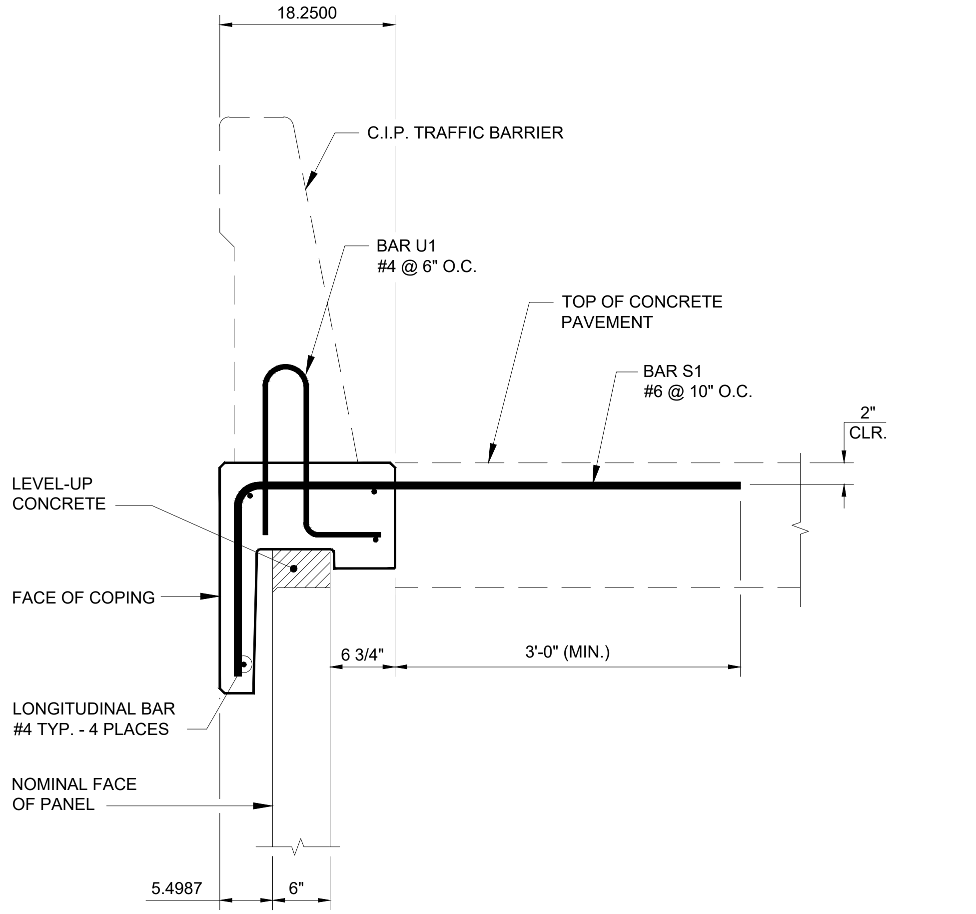
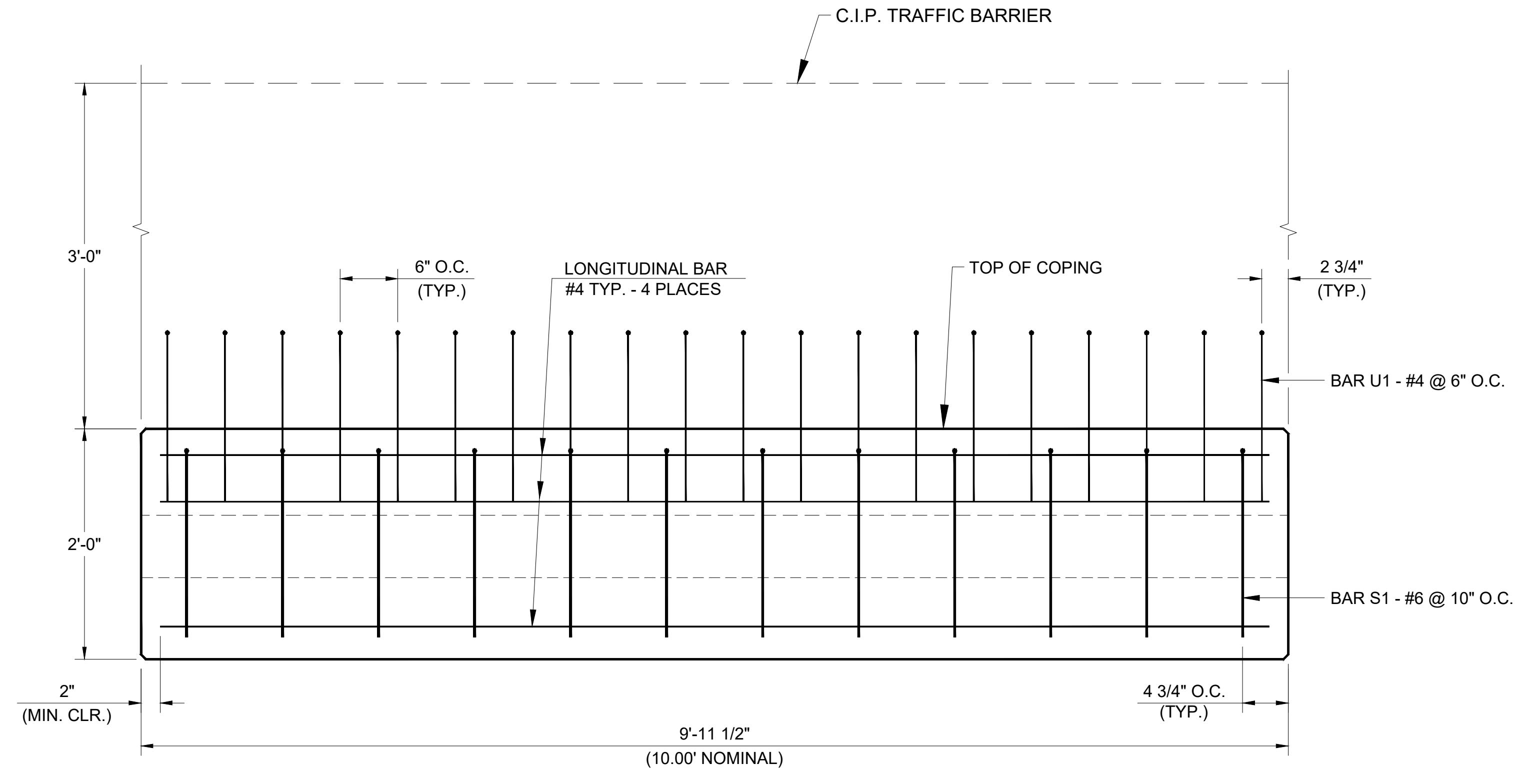
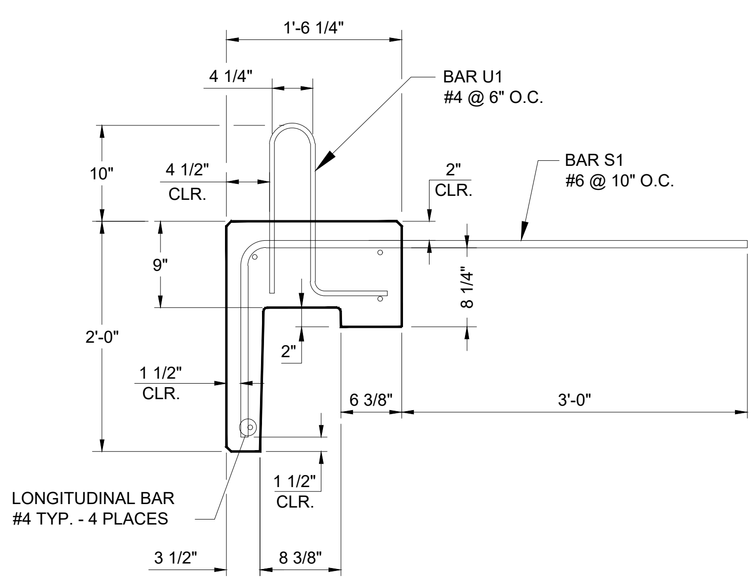



A
C4
PRECAST COPING W/C.I.P. TRAFFIC BARRIER
PARTIAL SECTION




B
C4
PRECAST COPING W/C.I.P. TRAFFIC BARRIER - REINFORCING PLACEMENT
FRONT FACE ELEVATION




NOTES:

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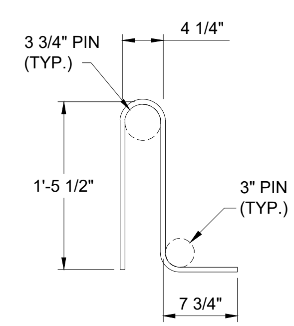
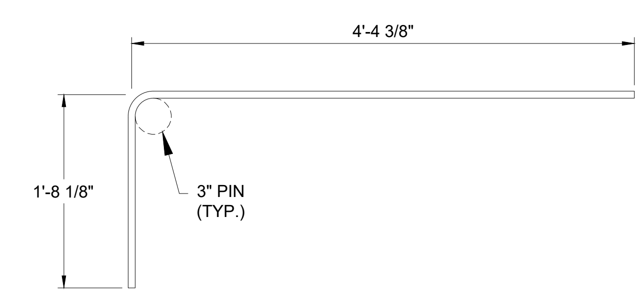



BAR U1 - #4 @ 6" O.C.




BAR S1 - #6 @ 10" O.C.

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SHEET 4 OF 4

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## 1.3.5
## ERS Traffic Barrier

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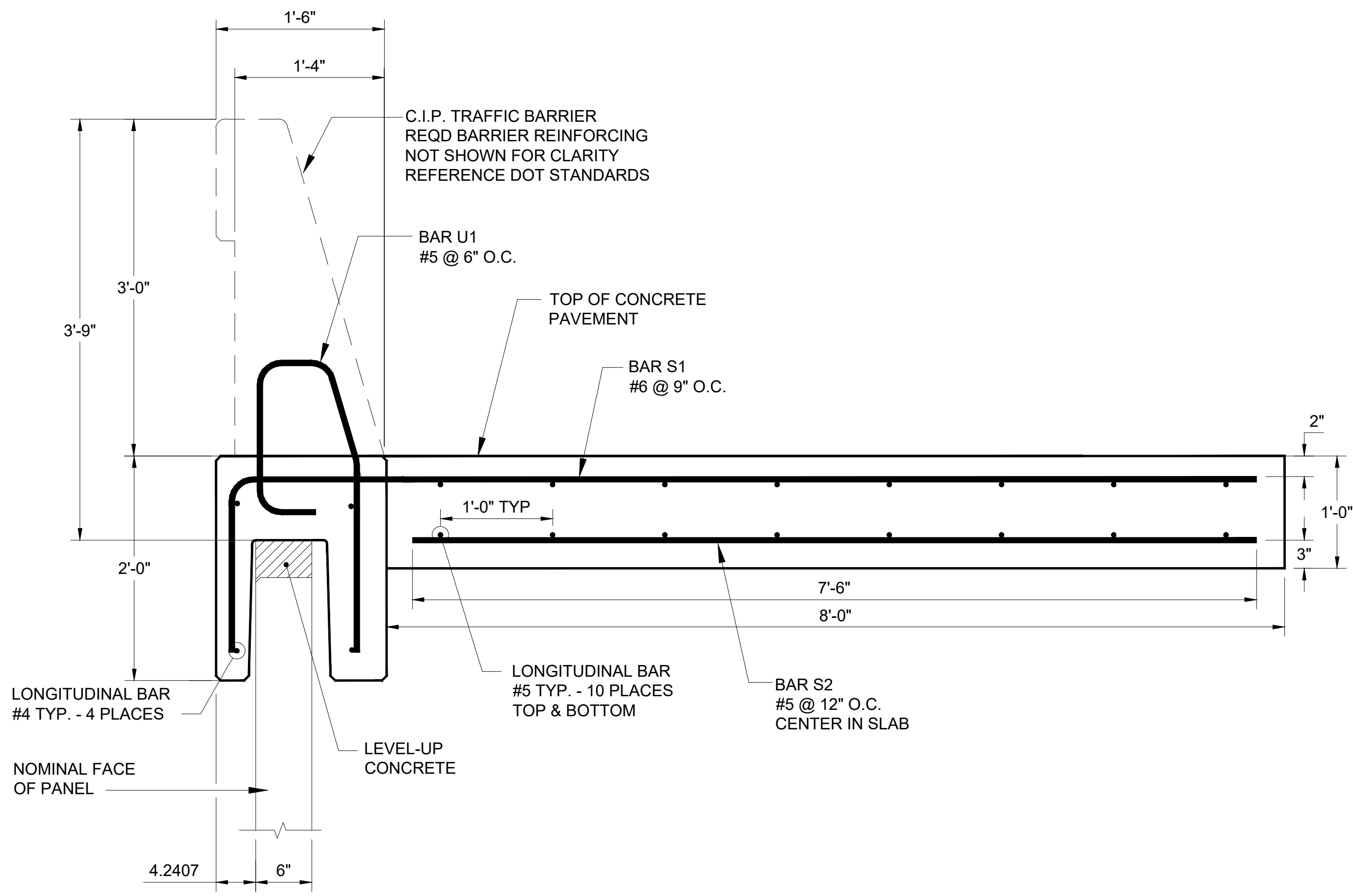
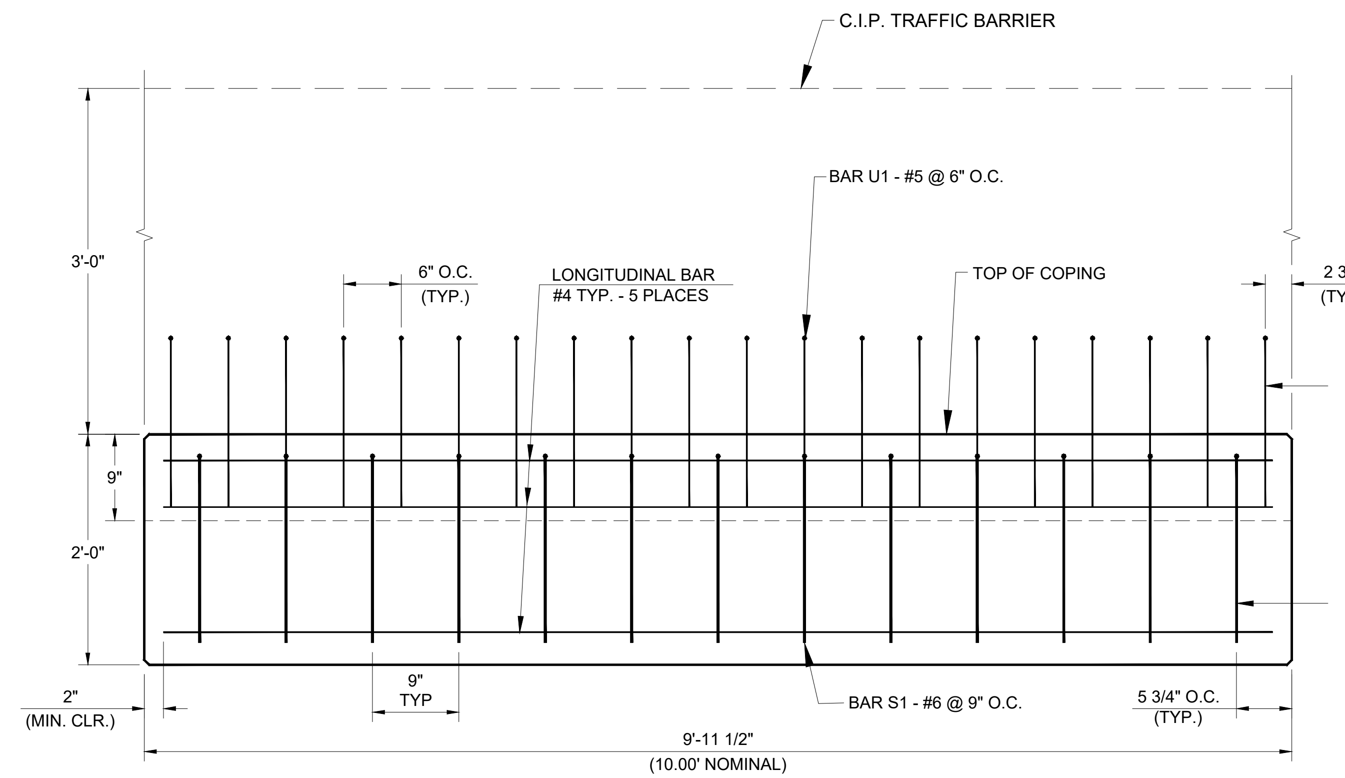
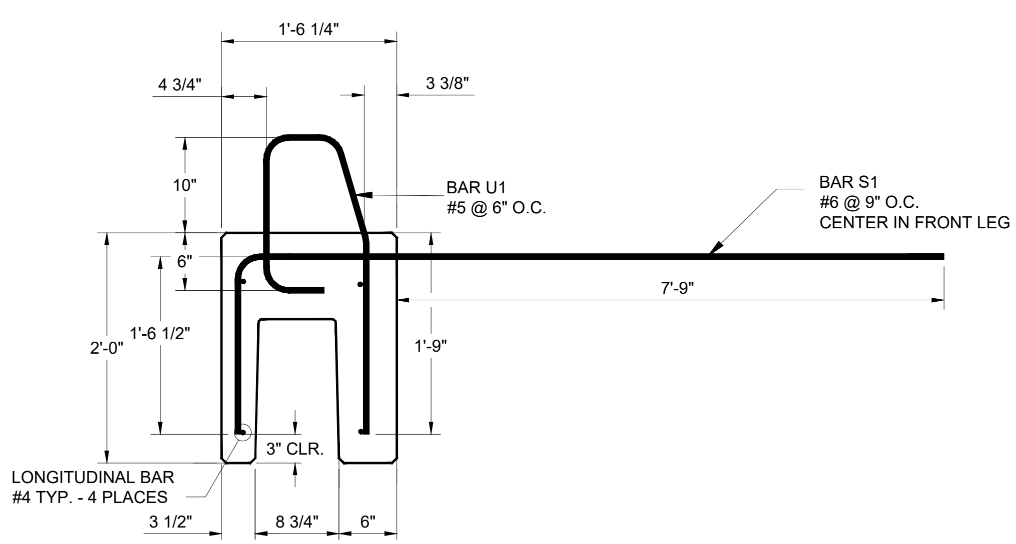
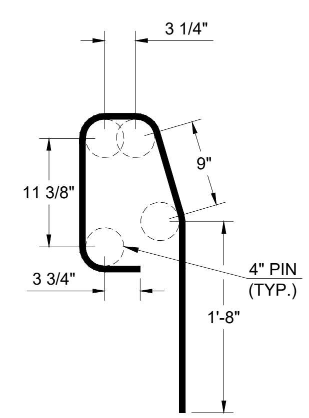



A
TB1
PRECAST COPING W/C.I.P. TRAFFIC BARRIER
PARTIAL SECTION



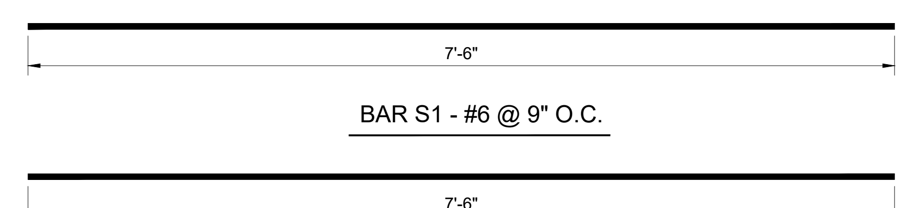

B
TB1
PRECAST COPING W/C.I.P. TRAFFIC BARRIER - REINFORCING PLACEMENT
FRONT FACE ELEVATION




Total bar length = 4'-10" (58")

BAR U1 - #5 @ 6" O.C.




BAR S1 - #6 @ 9" O.C.

BAR S2 - #5 @ 12" O.C.

NOTES:

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2. ALL DIMENSIONS ARE TO CENTER OF BAR UNLESS OTHERWISE NOTED.
3. MINIMUM COMPRESSIVE STRENGTH OF CONCRETE SHALL BE 5000 PSI

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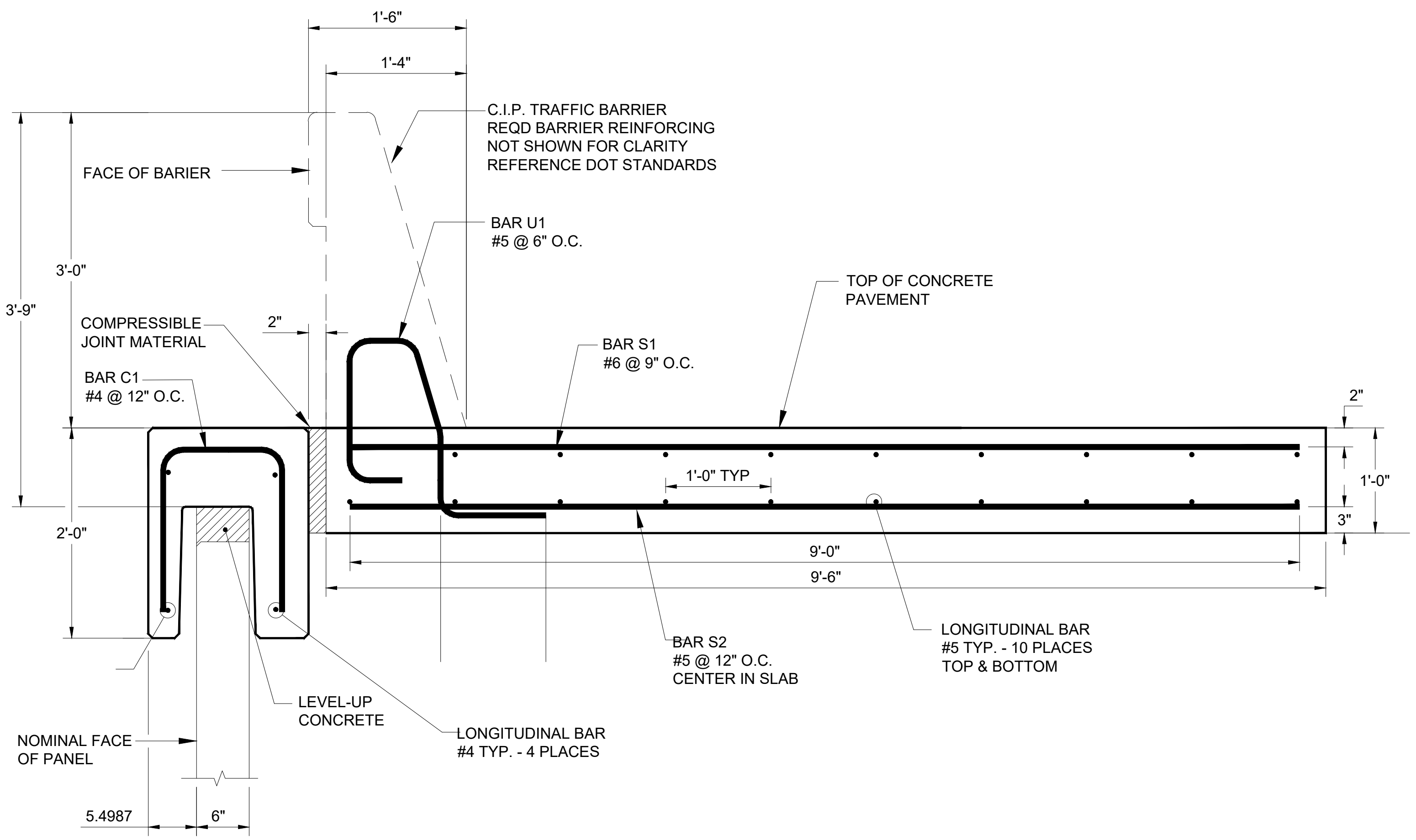
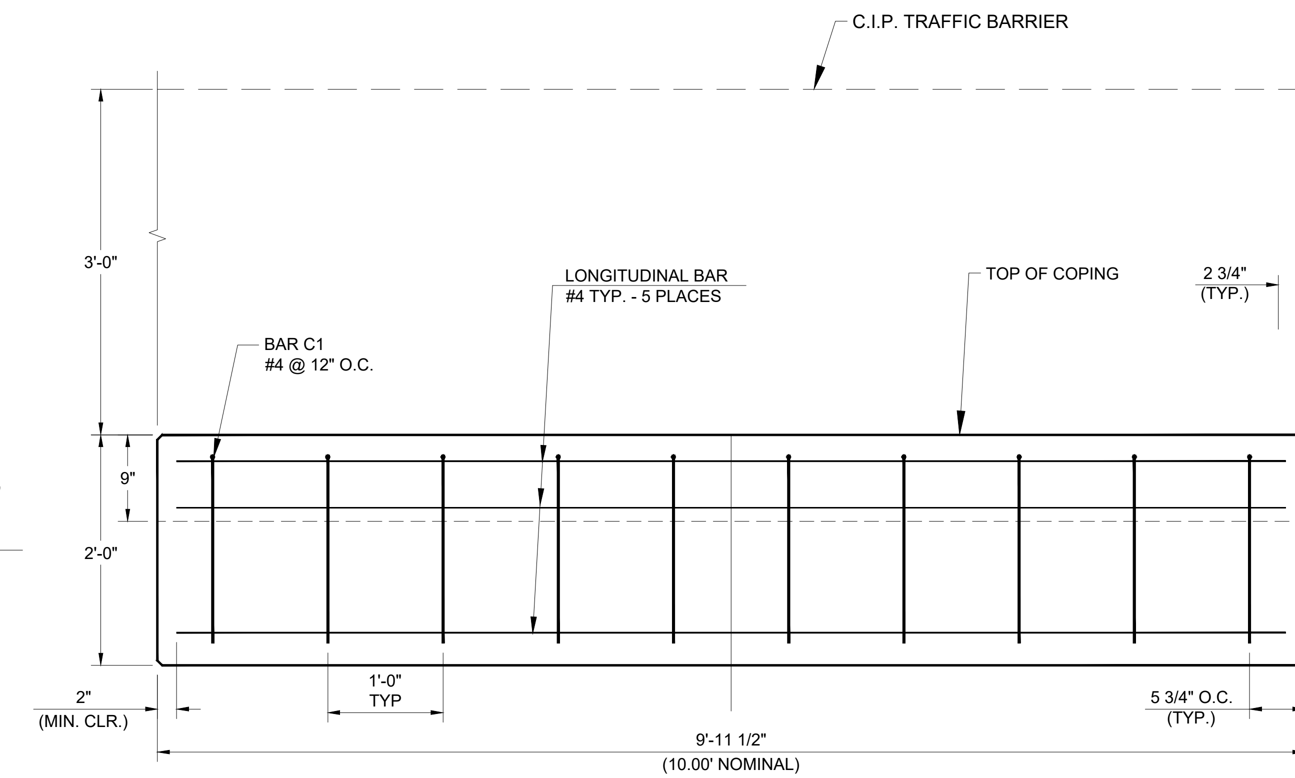
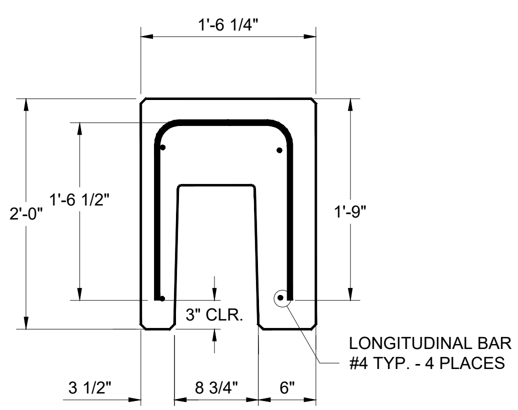
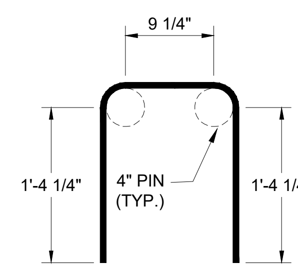



A
TB2
## PRECAST COPING W/C.I.P. TRAFFIC BARRIER
PARTIAL SECTION

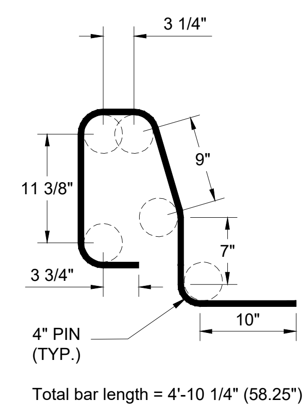



B
TB2
## PRECAST COPING - REINFORCING PLACEMENT
FRONT FACE ELEVATION

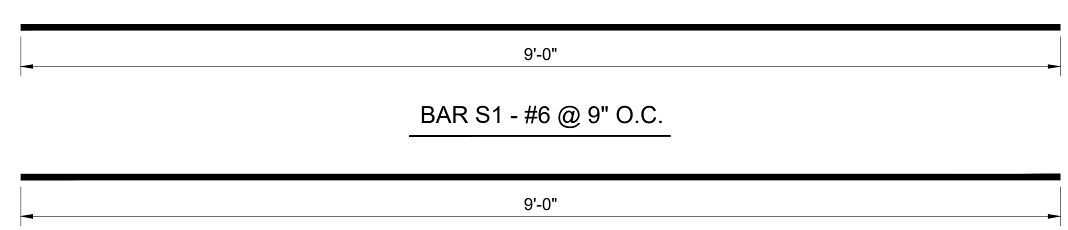



BAR C1 - #4 @ 12" O.C.




BAR U1 - #5 @ 6" O.C.




BAR S1 - #6 @ 9" O.C.

BAR S2 - #5 @ 12" O.C.

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**1.3.6**
**ERS Slip Joints**

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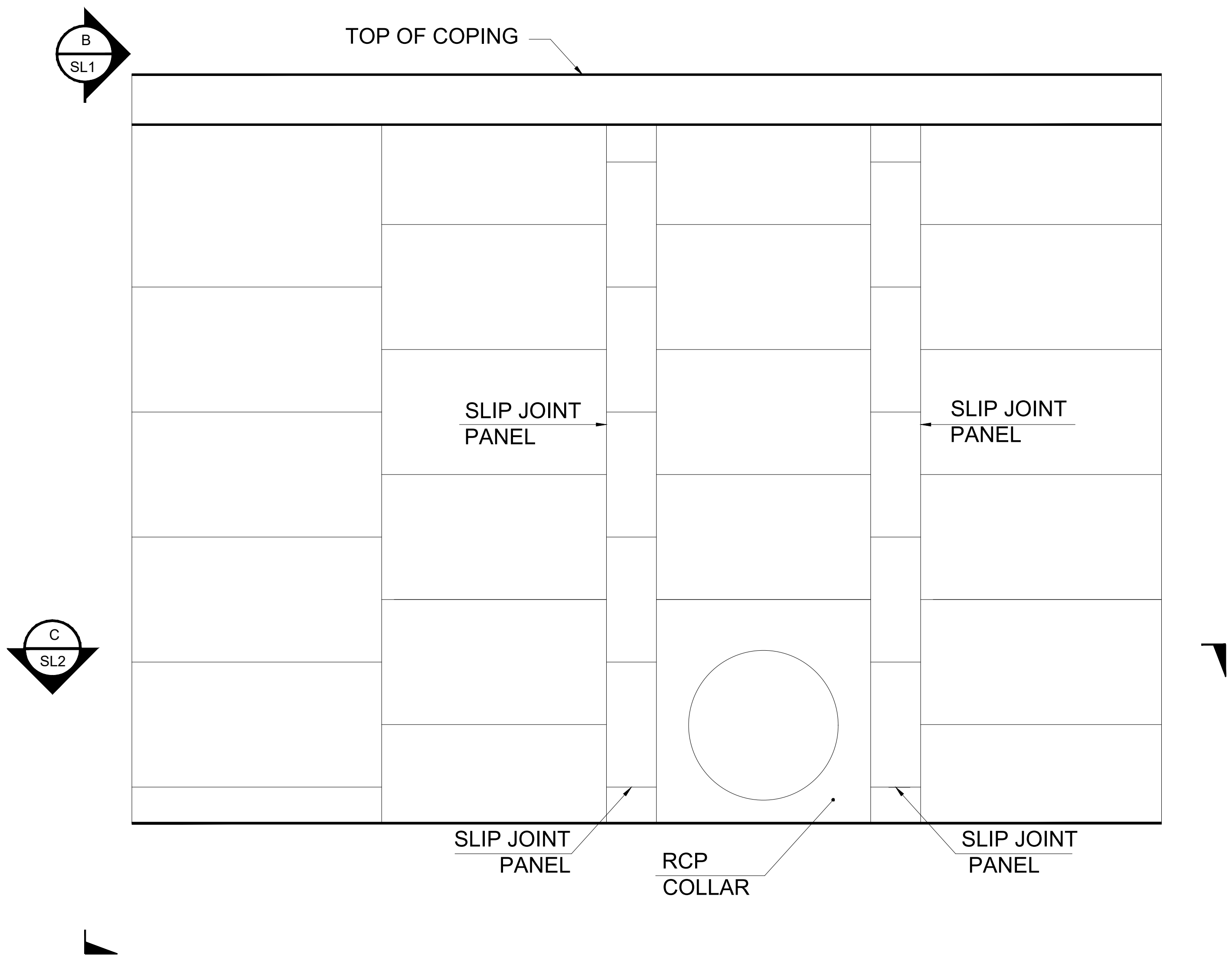
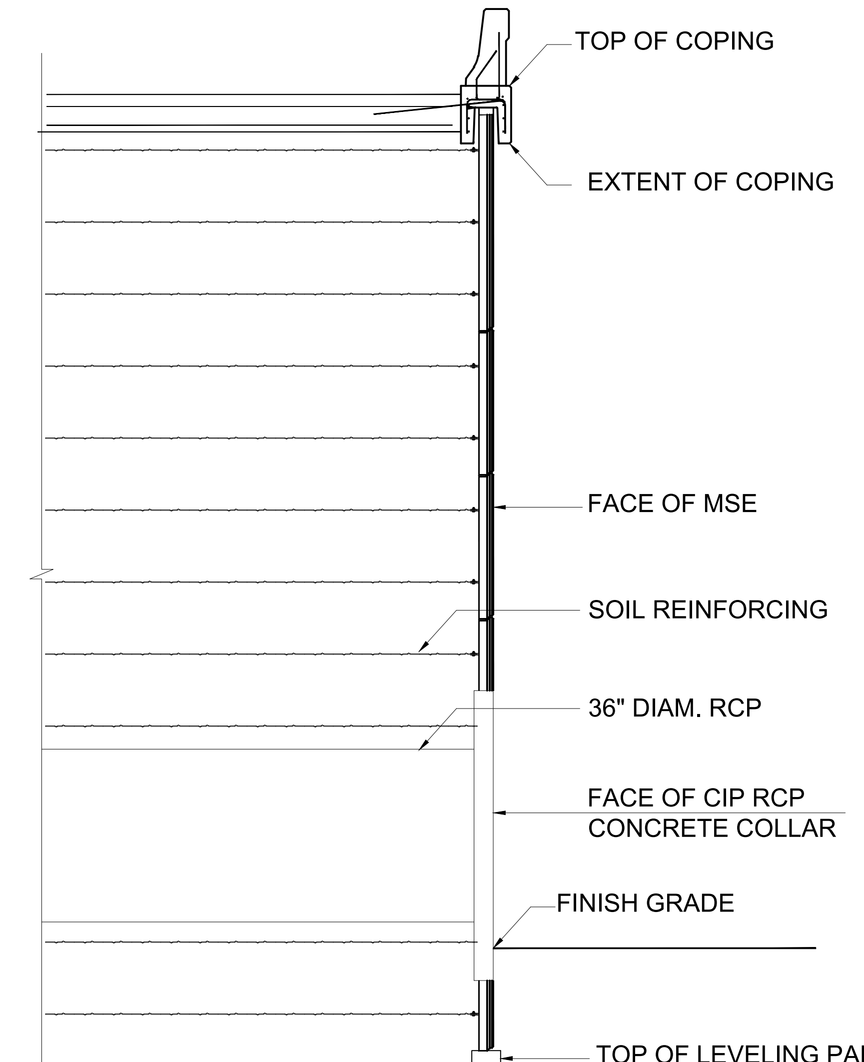



A SL1 ABUTMENT WING WALL WITH SLIP JOINT

ELEVATION




B SL1 ABUTMENT WING WALL WITH SLIP JOINT

CROSS SECTION AT MSE

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SLIP JOINT DETAIL
SHEET 1 OF 2

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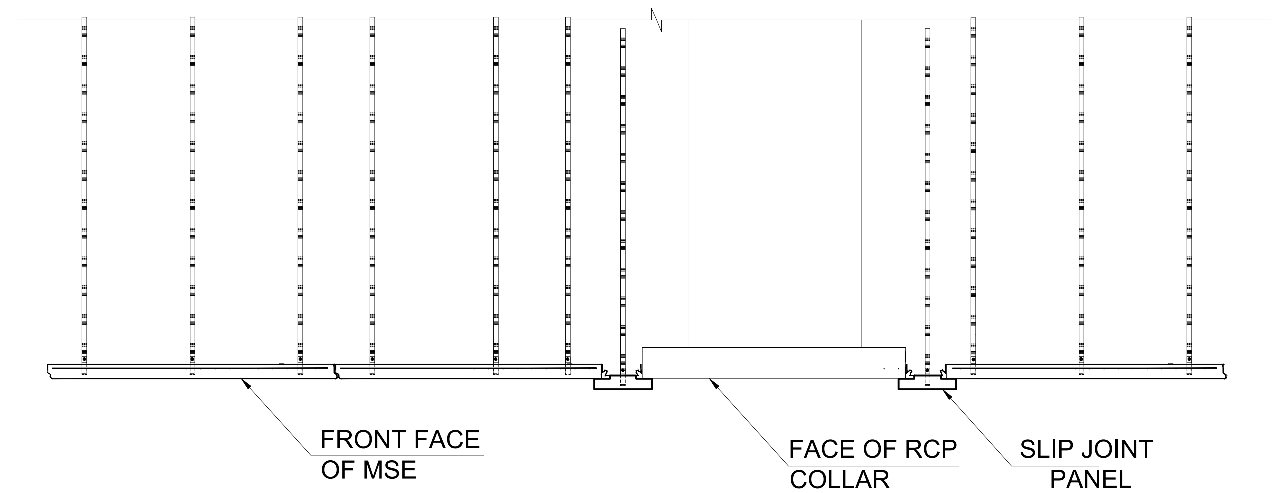
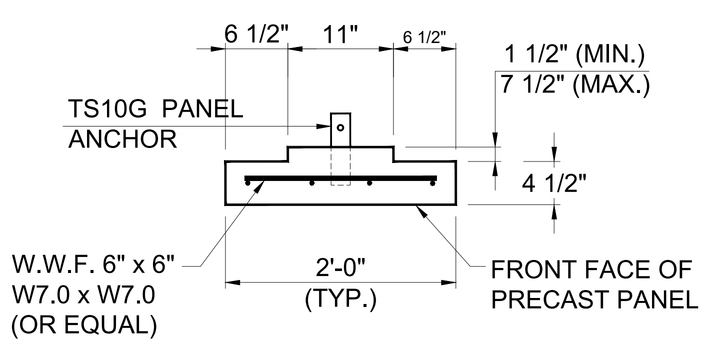
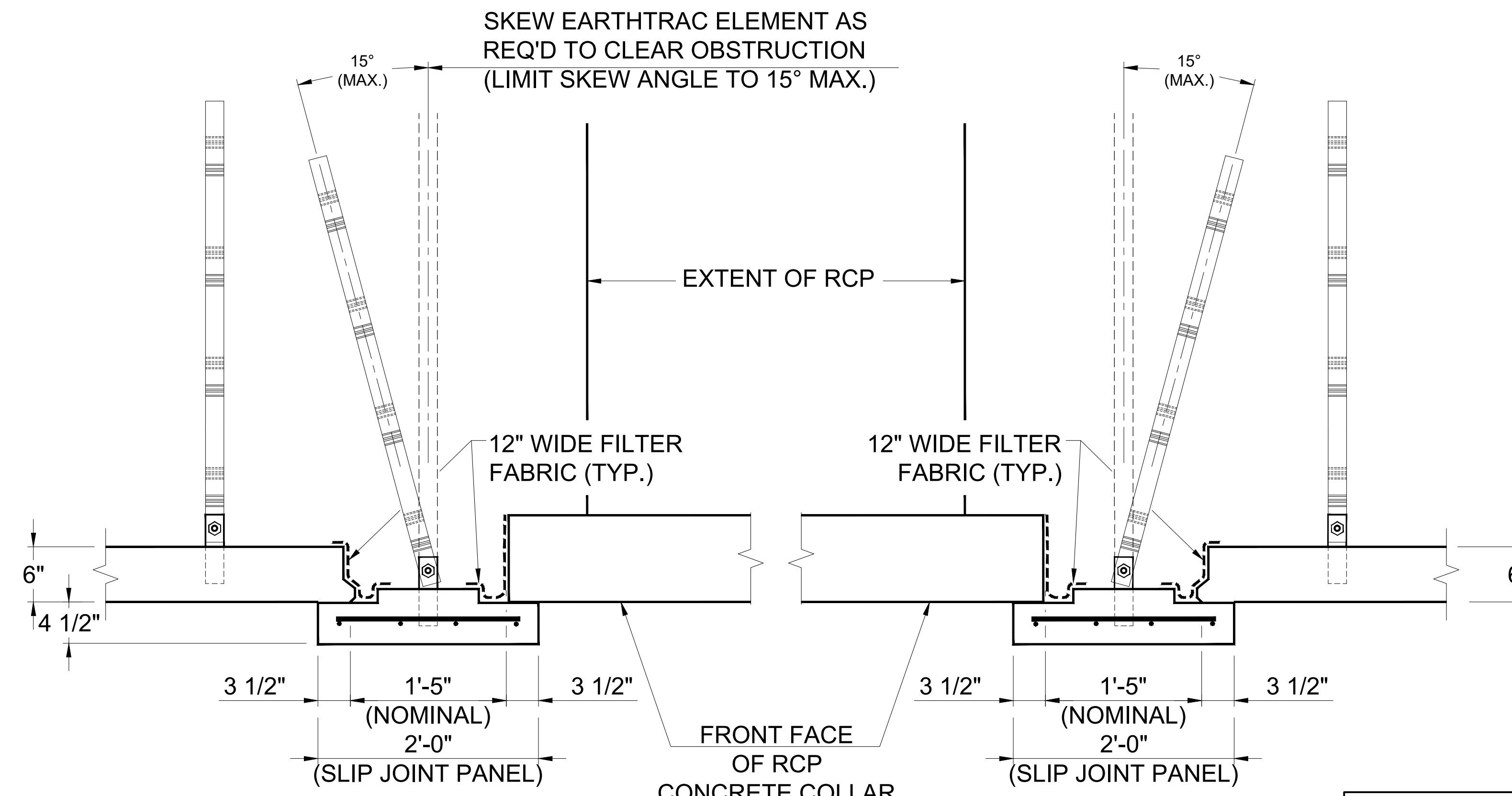



C SL2 ABUTMENT WING WALL WITH SLIP JOINT
PLAN VIEW




D SL2 SLIP JOINT PANEL DETAIL
PLAN VIEW




E SL2 SLIP JOINT DETAIL
PLAN VIEW

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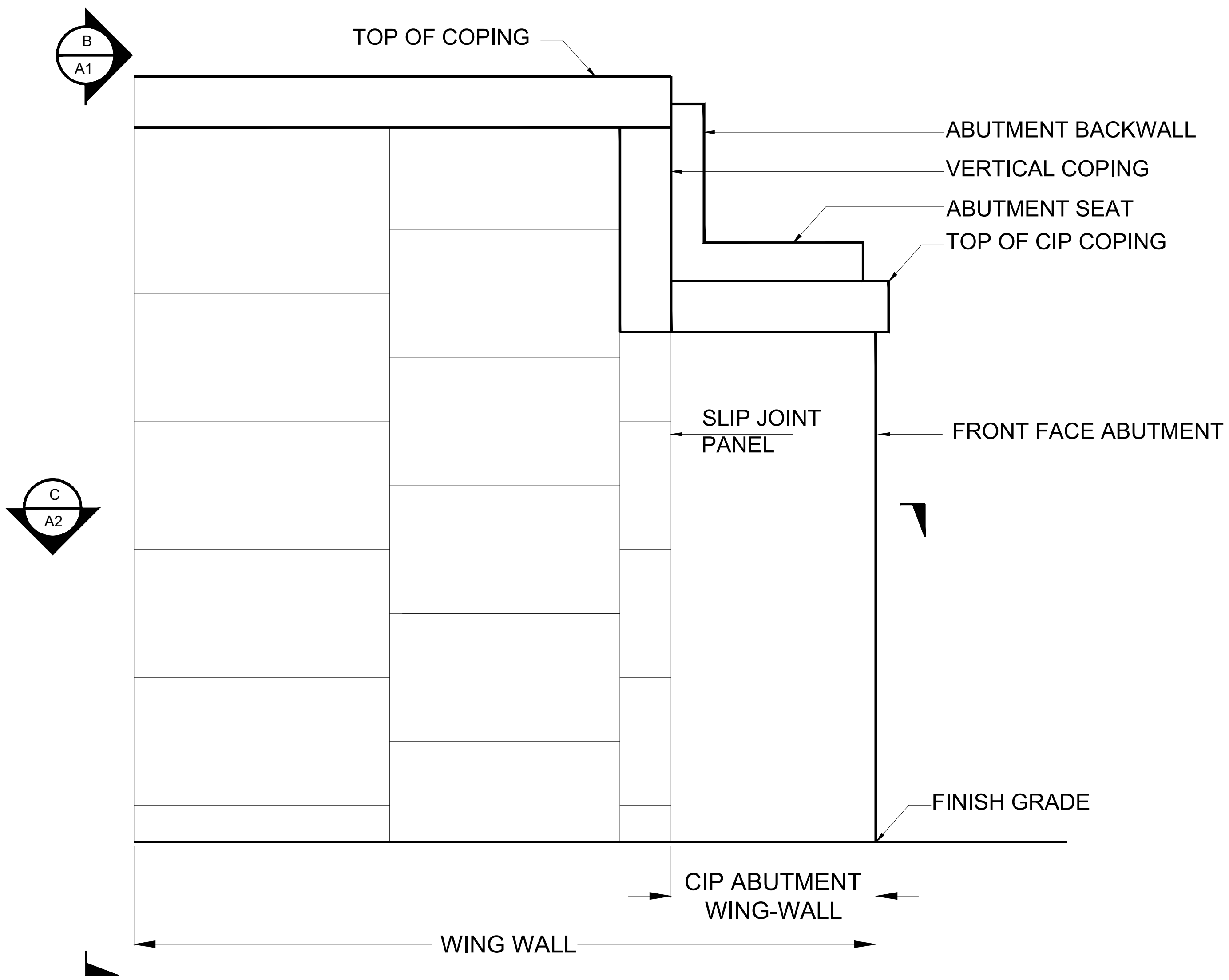


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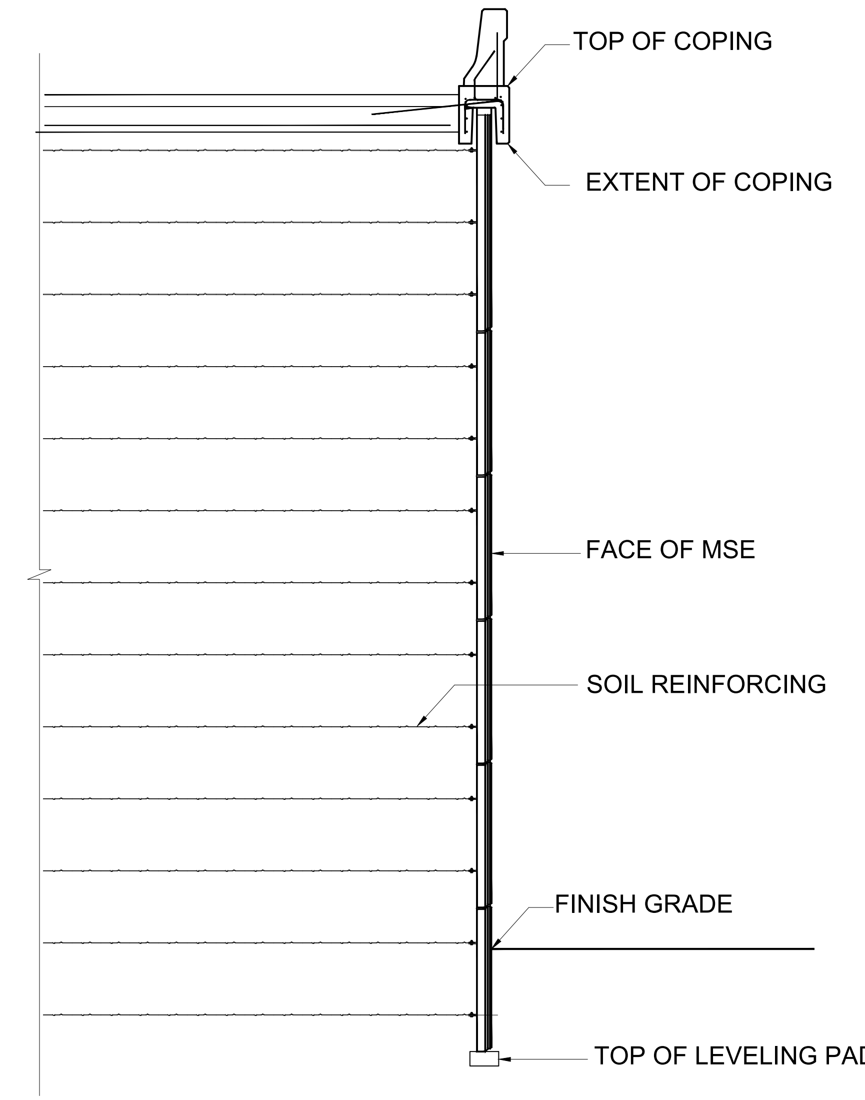
## ABUTMENT WING WALL WITH SLIP JOINT
ELEVATION




B
A1
## ABUTMENT WING WALL WITH SLIP JOINT
CROSS SECTION AT MSE

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SLIP JOINT DETAIL AT ABUTMENT INTERFACE
SHEET 1 OF 2

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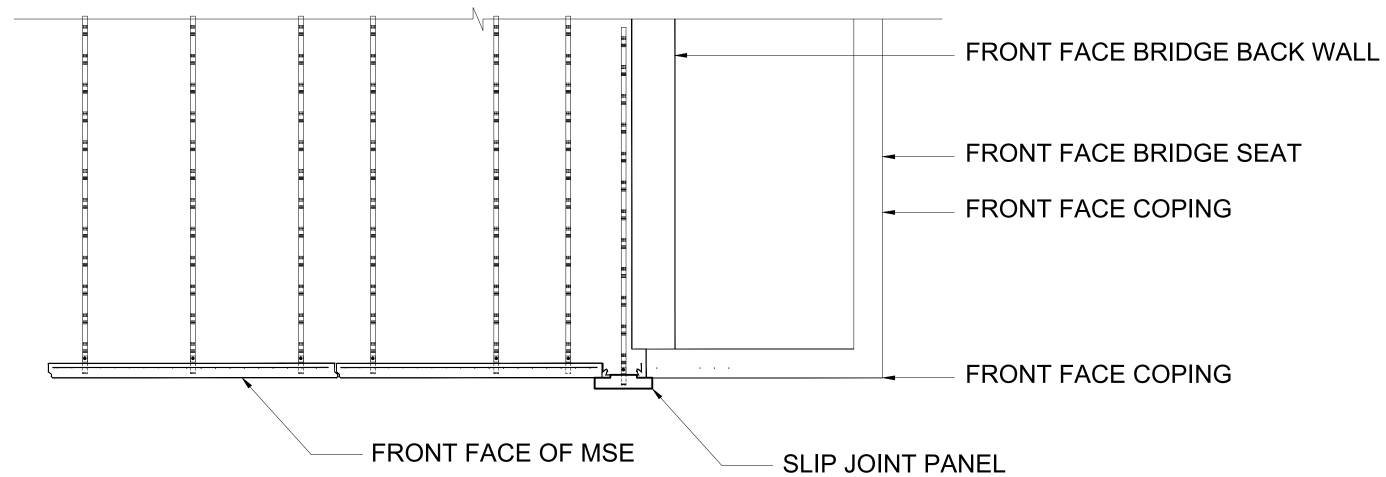
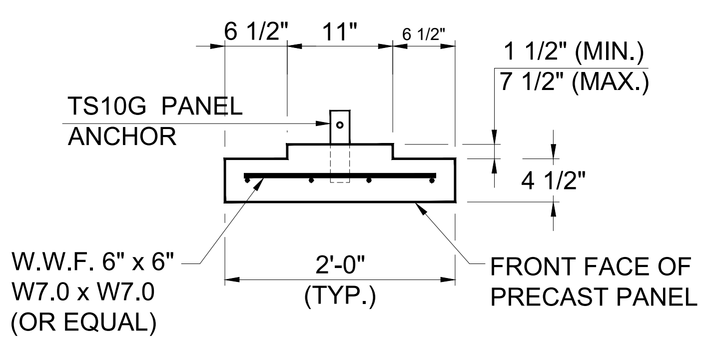


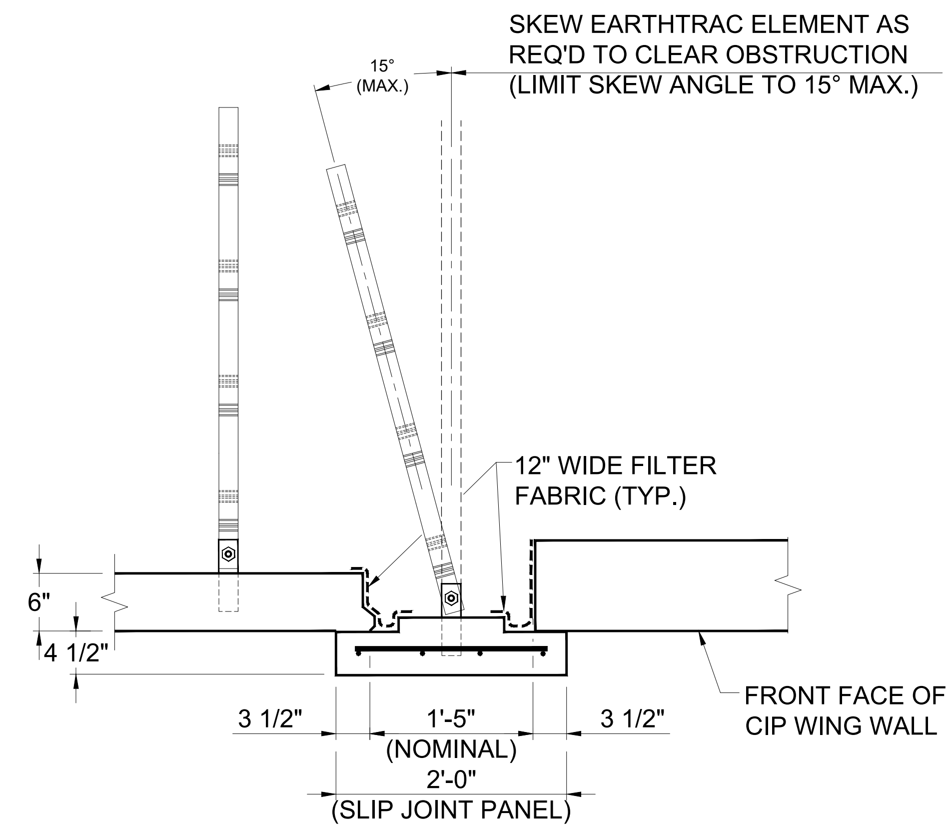

C / A2 — ABUTMENT WING WALL WITH SLIP JOINT

PLAN VIEW




D / A2 — SLIP JOINT PANEL DETAIL

PLAN VIEW




E / A2 — SLIP JOINT DETAIL

PLAN VIEW

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SLIP JOINT DETAIL AT ABUTMENT INTERFACE
SHEET 2 OF 2

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# 2.0
# ERS DESIGN

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## 2.1
## DESIGN METHODOLOGY

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## 2.1.1
## ERS Design Innovations

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**2.1.2**
**ERS AASHTO LRFD Design Methodology**

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**2.1.3**
**ERS Proprietary Design Methodologies**

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**2.1.4**
**ERS Vertical and Horizontal Obstruction Requirements**

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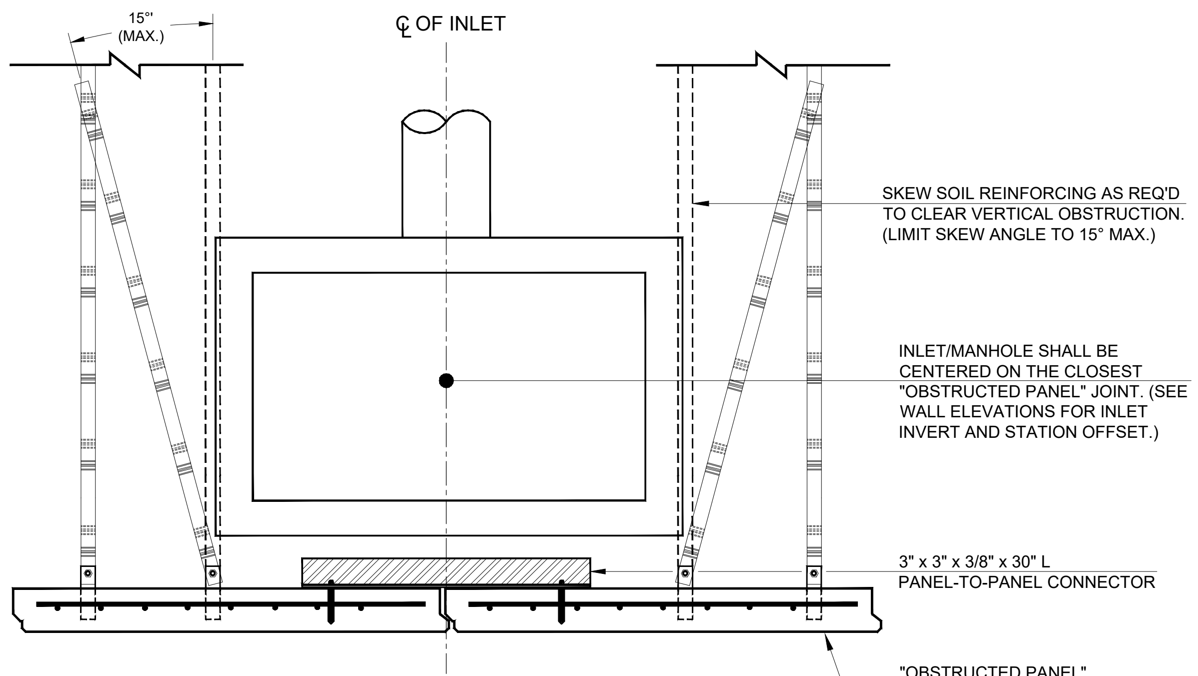
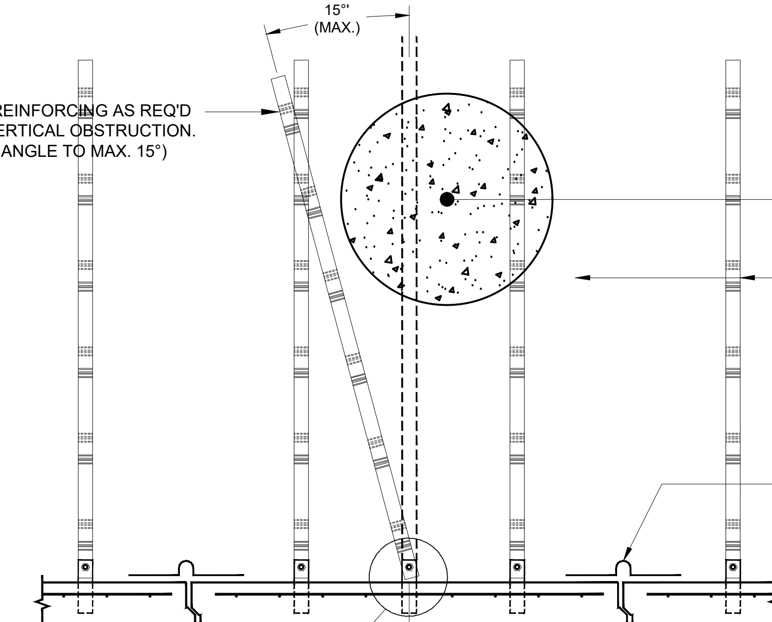


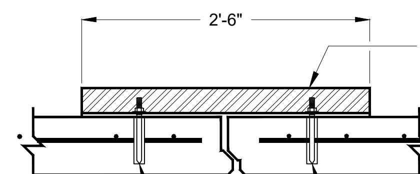

A / D3 VERTICAL INLET OBSTRUCTION

PLAN VIEW




B / D3 VERTICAL OBSTRUCTION DETAIL

PLAN VIEW

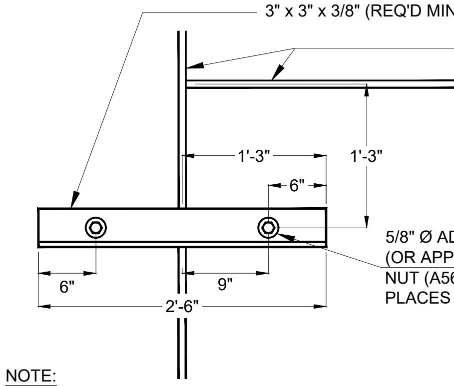



NOTE:

1. ALL STEEL TO BE HOT DIP GALVANIZED (UNLESS OTHERWISE NOTED)
2. ALL MATERIALS TO BE SUPPLIED BY OTHERS.

D / D3 PANEL-TO-PANEL CONNECTOR

PLAN VIEW




NOTE:

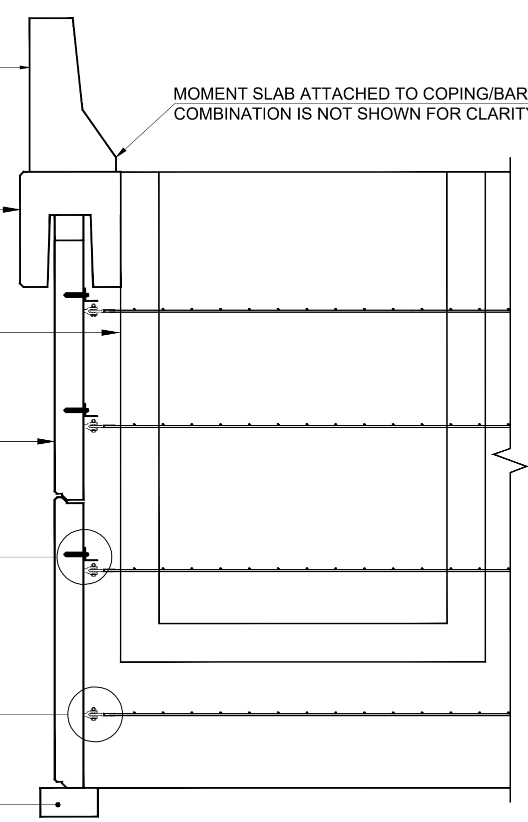
1. ALL STEEL TO BE HOT DIP GALVANIZED (UNLESS OTHERWISE NOTED)
2. ALL MATERIALS TO BE SUPPLIED BY OTHERS.

F / D3 PANEL-TO-PANEL CONNECTOR

BACK FACE ELEVATION

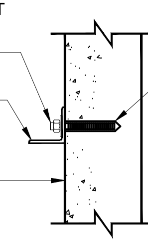
NOTES:

1. THREADED ROD TO BE 5/8" DIAMETER STAINLESS STEEL HILTI HAS ROD (Fy=65 KSI & Fu=100 KSI) MEETING REQUIREMENTS OF ASTM F 593 (AISI 304 OR AISI 316) WITH 4" EMBED USING HILTI HIT-HY 150 MAX ADHESIVE ANCHORING SYSTEM OR APPROVED EQUAL.
2. STAINLESS STEEL NUTS TO MEET REQUIREMENTS OF ASTM F 594 (AISI 304 OR AISI 316). STAINLESS STEEL WASHERS TO MEET REQUIREMENTS OF ASTM A 240 (AISI 304 OR AISI 316).
3. HOLES TO BE HAMMER DRILLED WITH A 3/4" CARBIDE BIT.
4. MANUFACTURER'S PREPARATION AND INSTALLATION PROCEDURES SHALL BE FOLLOWED.
5. ANCHOR TO BE INSTALLED NO CLOSER THAN 6" WITHIN PANEL EDGE.
6. ANGLE TO BE GALVANIZED.
7. ALL MATERIAL TO BE PROVIDED BY OTHERS.




B / D3 TYPICAL SECTION @ WALL INLET

PARTIAL SECTION




NOTE:

1. ALL STEEL TO BE HOT DIP GALVANIZED (UNLESS OTHERWISE NOTED)
2. ALL MATERIALS TO BE SUPPLIED BY OTHERS.

E / D3 PANEL-TO-PANEL CONNECTOR

PARTIAL SECTION

THE DESIGN CONTAINED HEREIN IS BASED ON INFORMATION SUPPLIED BY OTHERS GIS CERTIFYING INTERNAL SILITY OF THE STRUCTURE ONLY. EXTERNAL AND GLOBAL SILITY REQUIREMENTS ARE THE RESPONSIBILITY OF OTHERS.

| REV | DESCRIPTION | BY | DATE |
|---|---|---|---|
| | | | |

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SPECIAL DETAILS
SHEET 3 OF 4

REINFORCED SOIL WALL
SCP WITH EARTHTRAC SOIL REINFORCING


GROUND IMPROVEMENT SYSTEMS LLC
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(817) 223-0969
www.groundimprovementsystems.com


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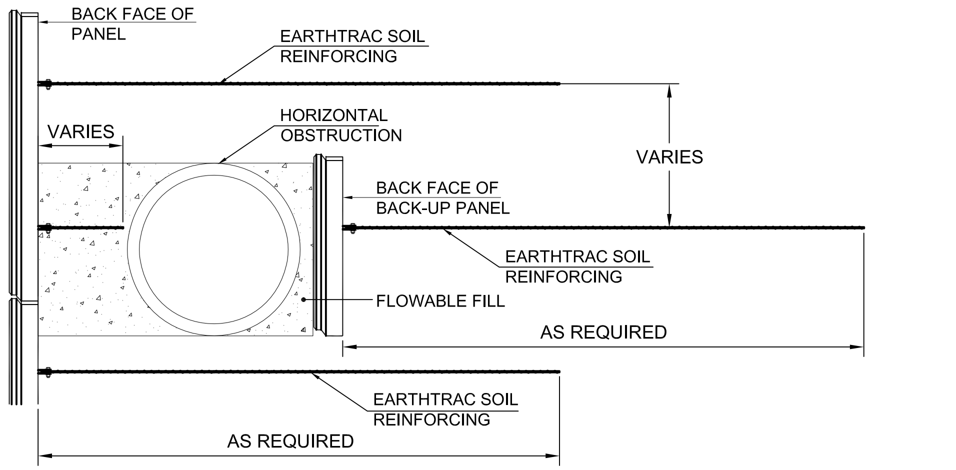
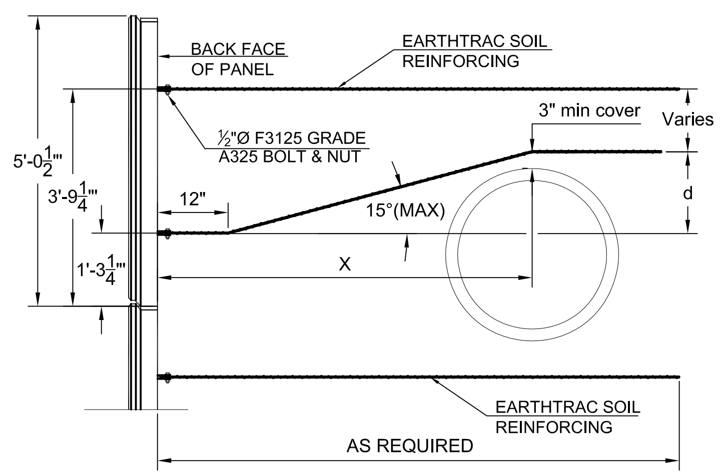
CHECKED BY XXX

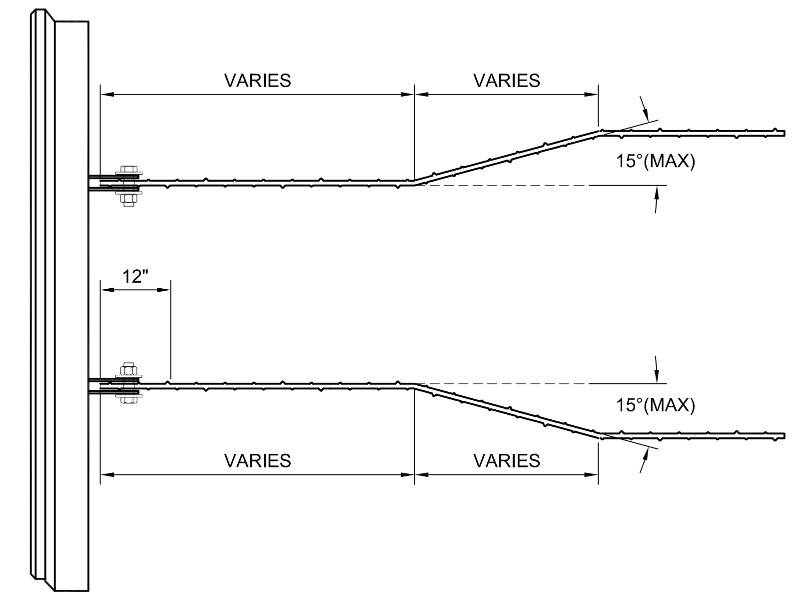
SHT: D3




## A / D5 HORIZONTAL OBSTRUCION WITH BACK-UP PANEL
PARTIAL SECTION




## B / D5 HORIZONTAL OBSTRUCTION WITH DEFLECTED SOIL RIENFORCING
PARTIAL SECTION




NOTES:
1. THE MINIMUM START OF DEFLECTION SHALL BE 12" FROM THE BACK FACE OF THE PANEL
2. THE DEFLECTIONS ANGLE SHALL BE 15°
3. 3" OF BACKFILL SHALL BE PLACED OVER ALL OBSTRUCTIONS THE SOIL REINFORCING PASSES OVER OR UNDER

## C / D5 SOIL REINFORCING VERTICAL DEFLECTION DETAIL
PARTIAL SECTION

| REV | DESCRIPTION | BY | DATE |
|---|---|---|---|

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SPECIAL OBSTRUCTION DETAILS
SHEET 5 OF 5

REINFORCED SOIL WALL
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## 2.2
## DESIGN EXAMPLES

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## 2.2.1
## Level Backslope with Traffic Live Load Design Example

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# MSE DESIGN

## SIMPLIFIED METHOD

### LEVEL BACK-SLOPE WITH TRAFFIC LIVE LOAD

## ENGINEERING REPORT


**GROUND IMPROVEMENT SYSTEMS LLC**

114 South Collins Street

Arlington, Texas 76011

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Appendix

## Introduction

This example problem demonstrates an MSE wall analysis with a horizontal back-slope that supports a live load surcharge for a 75 year design life. The MSE structure will include a segmental concrete panel face utilizing the EarthTrac soil reinforcing. The MSE structure configuration that will be analyzed is shown in Figure 1. The analysis is based on principles discussed in the FHWA-NHI-10-024 *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slope*s and conforms to the 2020-8$^{th}$ Edition AASHTO LRFD Bridge Design Specification. The following is a summary of the design steps that are used in the analysis. Each of the steps and any sub-steps is sequential. Based on the design steps' sequential nature, if the design is revised at any step or sub-step, all the previous calculations are required to be revised accordingly. Each of the steps and the sub-steps is explained in detail in this document.

## Design Steps

1 Establish Project Requirements

2 Define Project Parameters

2.1 Define Structure Parameters

2.2 Define External Live Load Parameters

2.3 Define Facing Parameters

2.4 Define Soil Reinforcing Parameters

2.5 Define Backfill Parameters

3 Evaluate Applicable Load and Resistance Factors

Table 3-1 Load Factors (AASHTO 2014 Tables 3.4.1-1 and 3.4.1-2)

Table 3-2 Resistance Factors (AASHTO 2014 Table 11.5.6-1)

4 Define Steel Corrosion Rates

5 Define Internal Stability Factors

Calculate Unfactored Loads for External Stability

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7.4 Calculate Maximum Tension at Elevation of Each Soil Reinforcing Element

7.5 Determine Required Area of Steel at Elevation of Each Soil Reinforcing Element

7.6 Calculate Factored Pullout Resistance

7.7 Summary of Internal Stability

Table 1-1 through Table 5-1 defines parameters that are required input. If the table's value contains a light-colored box around it, it is a required input parameter. If there is no light-colored box, the value will be calculated and is a function of the variables defined in the appropriate equation.

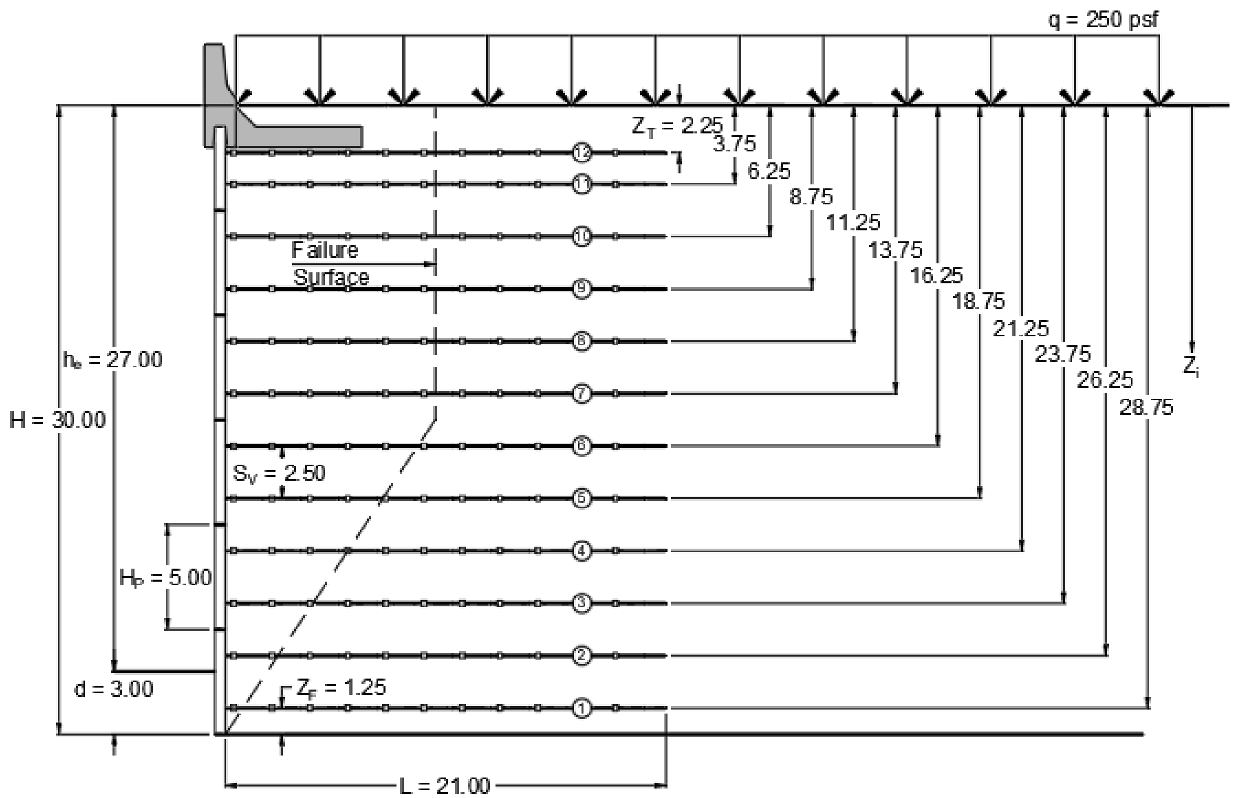



**Figure 1 Example Problem Cross Section**

## 1 Establish Project Requirements

The project requirements are established by the Engineer of Record (EOR). A general configuration of the MSE structure that is being designed is shown in Figure 1-1. The exposed wall height ($h_e$) is the distance from the finish grade to the top of the structure. This should not be confused with the design wall height (H) that is used in the calculations. The design wall height is equal to the exposed wall height plus the wall embedment distance (d). Therefore, the design wall height is equal to the distance from the top of the leveling pad to the top of the structure. The wall embedment depth is a function of the project requirements and shall conform to AASHTO Article 11.10.2.2. Any passive resistance that is provided by the embedment at the front face of the structure is not included in the calculations.

The soil reinforcing length aspect ratio is the ratio of the minimum length of soil reinforcing (L) to the structure's design height (H). The minimum length of soil reinforcing or aspect ratio (L:H) is typically provided by the EOR. The minimum length of soil reinforcing shall conform to AASHTO Article 11.10.2.1.

For this example, the minimum length of soil reinforcing will be set equal to 70% of the structure design height. The Project Requirements are defined in Table 1-1 for the structure analyzed in this document.

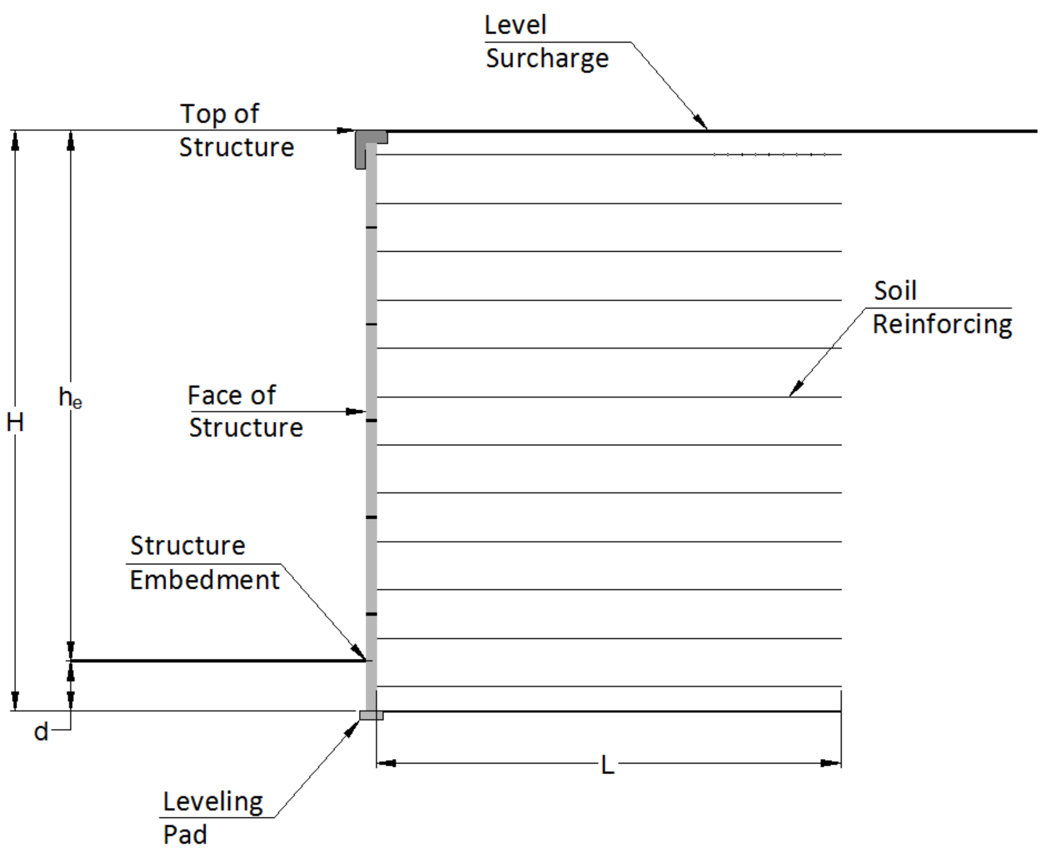



**Figure 1-1 Structure Project Requirements**

Table 1-1 Project Requirements

| Location | Variable | Value | Unit |
|---|---|---|---|
| • Exposed wall height | $h_e$ | 27.00 | ft |
| • Wall embedment distance | d | 3.00 | ft |
| • Soil reinforcing length aspect ratio | ratio | 0.70 | dim |
| • Length of wall | $L_w$ | 1000.00 | ft |
| • Design life | Life | 75.00 | yrs |
| • Top of wall slope condition | ◉ Horizontal ○ Broken ○ Infinite | | |
| • Facing | Precast Panel– 10 ft wide x 5 ft tall by 0.5 ft thick | | |

Table 1-1 Project Requirements

| Location | Variable | Value | Unit |
|---|---|---|---|
| • Type of Soil Reinforcement | EarthTrac SS<br>Grade 50 steel with area equal to 0.375 in$^2$<br>Galvanized – 3.4 mil zinc coating | | |
| • Is seismic to be considered | No | | |

## 2 Define Project Parameters

The project requirements provide a general framework of the constraints for the MSE structure analyzed in this calculation. A comprehensive set of parameters is provided in Table 2-1 and Table 2-2 and are detailed in Figure 2-1. For the MSE structure evaluated in this calculation the back-slope at the top of the structure is horizontal and supports an externally applied live load surcharge consisting of standard highway traffic.

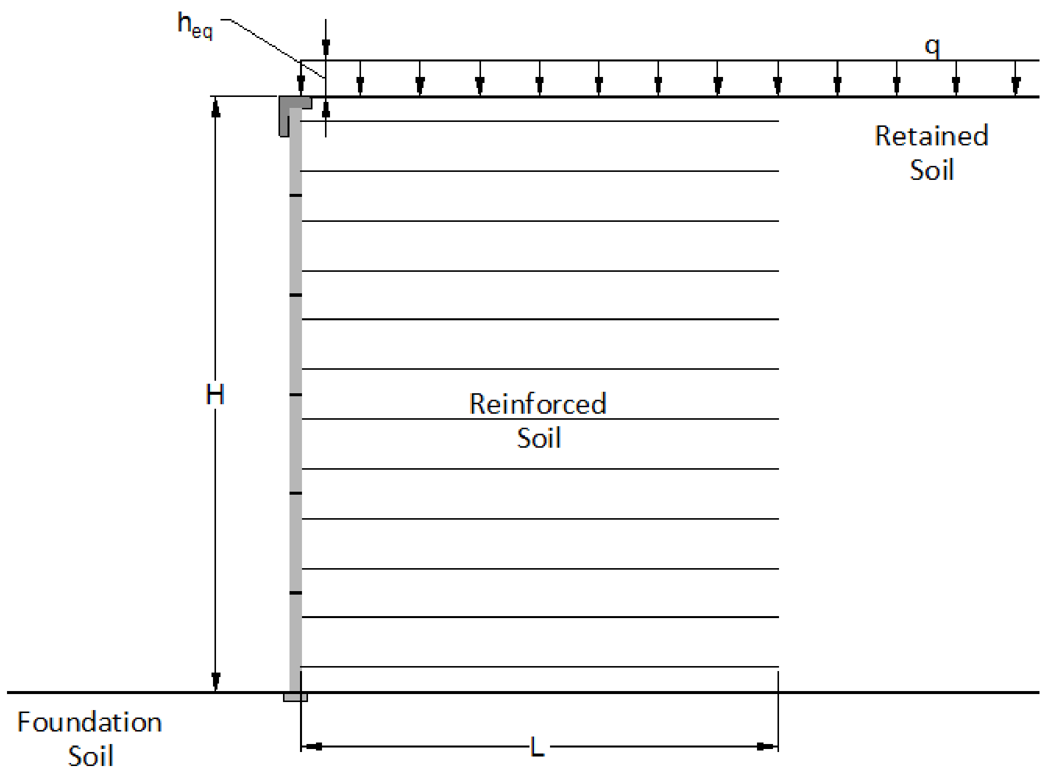



**Figure 2-1 Structure Project Requirements**

## 2.1 Define Structure Parameters

The back-slope at the top of the MSE structure is horizontal therefore there is no slope angle ($\beta$) and therefore there is no soil surcharge (S). Since the slope is horizontal the distance to the crest of the soil surcharge $X_s$ is also zero. The design height (H) is calculated using Equation 2-1.

$$H = h_e + d$$ Equation 2-1

Table 2-1 Structure Parameters

| Location | Variable | Value | Unit |
|---|---|---|---|
| Wall Design Height | H | 30.00 | ft |
| Soil Reinforcing Length | L | 21.00 | ft |
| Surcharge Height | S | 0.00 | ft |
| Distance from face to Crest | $X_S$ | 0.00 | ft |
| Slope of Surcharge | $\beta$ | 0.00 | deg |
| Top of Structure Back-Slope Condition | | Horizontal | |

## 2.2 Define External Live Load Parameters

The live load surcharge (LS) is based on the requirements in AASHTO Table 3.11.6.4-2. The equivalent soil height ($h_{eq}$) is equal to 2 feet and the unit weight ($\gamma_q$) used to calculate the live load pressure (q) is set equal to 125 pcf (Table 2-2). The pressure applied at the top of the structure from the live load surcharge is calculated using Equation 2-2. The live load is assumed to be applied over the entire horizontal earth surcharge. AASHTO does not require the live load surcharge to be used in the internal stability pullout analysis for the calculation of $T_{max}$ (AASHTO 11.10.6.2.1).

$$q = \gamma_q \cdot h_{eq}$$ Equation 2-2

Table 2-2 External Live Load Parameters

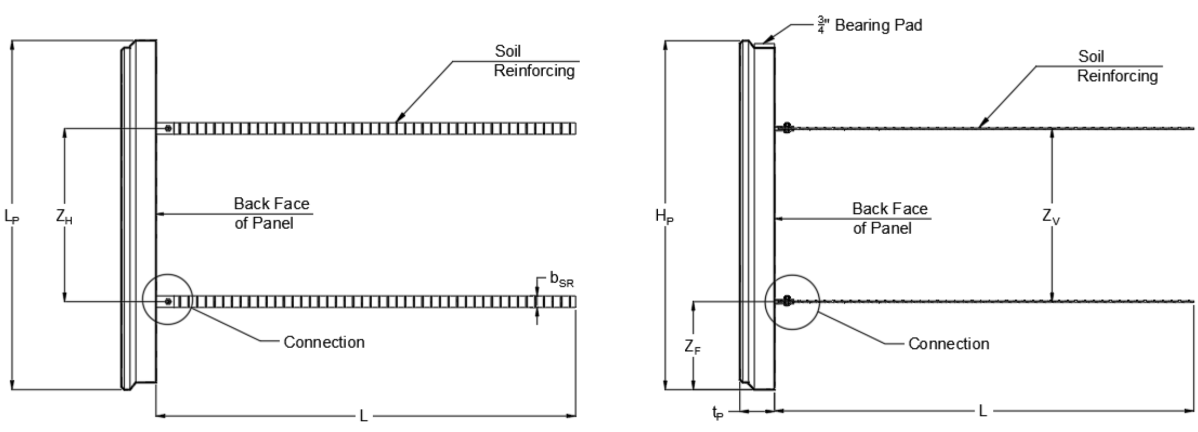
| Location | Variable | Value | Unit |
|---|---|---|---|
| Equivalent height of soil | $h_q$ | 2.00 | ft |
| Unit weight of surcharge | $\gamma_q$ | 125.00 | ft |

Table 2-2 External Live Load Parameters

| Location | Variable | Value | Unit |
|---|---|---|---|
| Live load pressure | q | 250.00 | psf |
| Include live load in calculating pullout | | ☐ Inlcude Live Load | |

### 2.3 Define Facing Parameters

The facing is specified to be a segmental concrete panel (SCP) with a height ($H_P$) equal to 5.00’ and a length ($L_P$) equal to 10.00’ (Figure 2-2 and Table 2-3). The thickness ($t_P$) of the panel is assumed to be 6”. The vertical spacing ($S_V$) of the soil reinforcing is typically a constant value and is equal to 2.50’. The spacing at the top of the structure may vary and is dependent on the coping treatment. The top of wall is assumed to have a traffic barrier that is cast integral to a coping element. Cast integral to the coping/barrier element is a drop moment slab. Therefore, in order to clear the drop moment slab the minimum distance from the top of the pavement to the top soil reinforcing element ($Z_T$) is equal to a value of 2.25’. The distance from the leveling pad to the first soil reinforcing element ($Z_F$) for the bottom panel is equal to 1.25’. The minimum distance between any two soil reinforcing elements ($Z_{min}$) is equal to a value of 0.50’. The input values, $S_V$, $Z_T$, $Z_F$, and $Z_{min}$ are required in order to set the depth to each soil reinforcing element. The horizontal spacing ($S_H$) of the soil reinforcing is a maximum value of 2.50’. The horizontal spacing of the soil reinforcing is a function of the type of the soil reinforcing being used in addition to the forces that are required to be resisted. (Reference Figure 8-1 for additional details).




Plan View Cross Section

**Figure 2-2 Facing Parameters Detail**

Table 2-3 Facing Parameters

| Location | Variable | Value | Unit |
|---|---|---|---|
| Panel Height | $H_P$ | 5.00 | ft |
| Panel Length | $L_P$ | 10.00 | ft |
| Vertical Spacing of Soil Reinforcing | $S_V$ | 2.50 | ft |
| Horizontal Spacing of Soil Reinforcing | $S_H$ | 2.50 | ft |
| Minimum Depth of Soil Reinforcing from Top of Panel | $Z_T$ | 2.25 | ft |
| Minimum soil reinforcing spacing | $S_{min}$ | 0.50 | ft |
| Distance from leveling pad to first soil reinforcing | $Z_F$ | 1.25 | ft |

## 2.4 Define Soil Reinforcing Parameters

The soil reinforcing parameters are a function of the type that is being used. For this example, the inextensible steel EarthTrac SS3 soil reinforcing will be used (Table 2-4 and Figure 2-3). The EarthTrac SS3 is a discrete steel strip with transverse ribs and a single point connector located at the proximal end. The EarthTrac SS3 has a width equal to 2-inches and a nominal thickness of 5/32-inches. Therefore, the EarthTrac SS3 initial steel area for one element is equal to 0.313 square inches. The EarthTrac SS3 is fabricated from Grade-50 steel with a yield strength equal to a minimum of 50 ksi. Based on the 5’ x 10’ SCP use, coupled with the minimum horizontal spacing, the minimum number of EarthTrac SS3 soil reinforcing elements (NSR) per row is set equal to 3. The actual number of EarthTrac SS3 for each panel is a function of the loading.

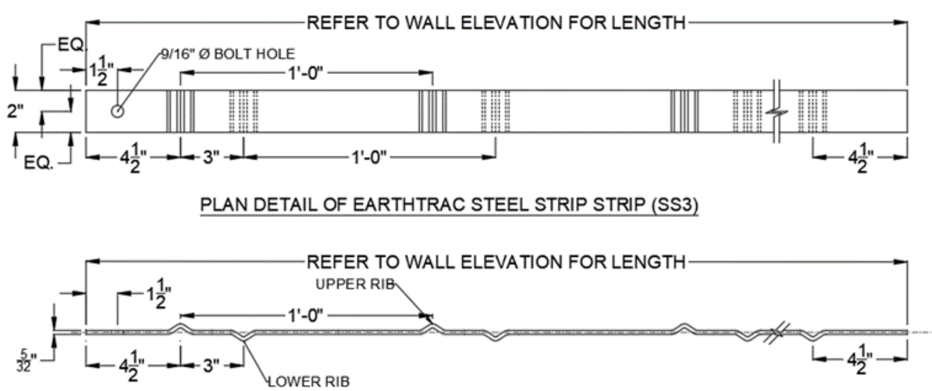



**Figure 2-3 EarthTrac SS3 Soil Reinforcing Parameters Detail**

Table 2-4 Soil Reinforcing Parameters

| Location | Variable | Value | Unit |
|---|---|---|---|
| Minimum number of soil reinforcing | $N_{SR}$ | 3 | dim |
| Soil reinforcing width | $b_w$ | 2.00 | in |
| Soil reinforcing thickness | $t_{ss}$ | 0.1563 | |
| Soil reinforcing steel area | $A_S$ | 0.3126 | $in^2$ |
| Yield strength of steel | $F_y$ | 50.00 | ksi |

## 2.5 Define Backfill Parameters

The backfill is project specific (Table 2-5), can vary significantly, is dependent on the geographical region, and the project specifications. The reinforced backfill is defined as the soil mass located at the back face of the facing element extending horizontally to the terminal end of the soil reinforcing and located from the top of the leveling pad extending vertically to the top of the coping element. The retained backfill is defined as the soil mass located directly behind the reinforced soil mass at the terminal end of the soil reinforcing and from the top of the leveling pad extending vertically to the top of the soil surcharge or pavement. The foundation soil is what the MSE structure bears on is defined as the volume of soil below the reinforced mass of soil and the retained mass of soil and (Figure 2-4).

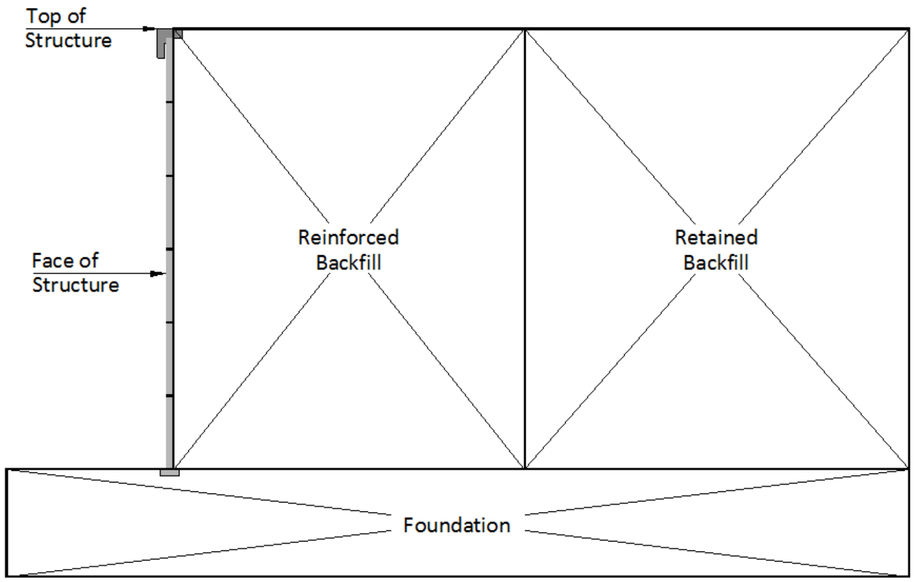



**Figure 2-4 Backfill Parameters**

Table 2-5 Backfill Parameters

| Location | Variable | Value | Unit |
|---|---|---|---|
| Unit Weight of Reinforced Soil | $\gamma_r$ | 125.00 | psf |
| Friction Angle of Reinforced Soil | $\phi_r$ | 34.00 | deg |
| Unit Weight of Retained Soil | $\gamma_f$ | 125.00 | psf |
| Friction Angle of Retained Soil | $\phi_f$ | 34.00 | deg |
| Unit Weight of Foundation Soil | $\gamma_{fd}$ | 125.00 | psf |
| Friction Angle of Foundation Soil | $\phi_{fd}$ | 30.00 | deg |

The internal active earth pressure coefficients for the reinforced soil mass and the retained soil mass are calculated using Equation 2-3 and Equation 2-5. The equivalent slope angle is calculated using the AREMA Manual method of analysis and is given in Equation 2-4. If there is no slope angle than Equation 2-3 reduces to Equation 2-5.

$$K_{af} = \cos(\beta_i) \cdot \left[ \frac{\cos(\beta_i) - \sqrt{\cos(\beta_i)^2 - \cos(\phi_f)^2}}{os(\beta_i) + \sqrt{\cos(\beta_i)^2 - \cos(\phi_f)^2}} \right]$$ 0.283 (dim) Equation 2-3

$$\beta_i = \operatorname{atan}\left(\frac{S}{2 \cdot H}\right)$$ 0.00 (deg) Equation 2-4

$$K_{ai} = \tan^2\left(45^\circ - \frac{\phi_r}{2}\right)$$ 0.283 (dim) Equation 2-5

Where: $K_{af}$ = active earth pressure coefficient for external stability (dim)

$\beta_i$ = the slope of the surcharge (deg)

S = height of surcharge (ft)

H = height of structure (ft)

$K_{ai}$ = active earth pressure coefficient for internal stability (dim)

## 3 Evaluate Applicable Load and Resistance Factors

The load and resistance factors conform to AASHTO Article 3-4, Table 3.4.1-1, Table 3.4.1-2 and Article 11.5, Table 11.5.7-1 and are as defined in Table 3-1 and Table 3-2.

Table 3-1 Load Factors (AASHTO 2014 Tables 3.4.1-1 and 3.4.1-2)

| Load Combination | Load Factors | | |
|---|---|---|---|
| | EV | EH | LS |
| Strength I (Max) | 1.35 | 1.50 | 1.75 |
| Strength I (Min) | 1.00 | 0.90 | 1.75 |
| Service I | 1.00 | 1.00 | 1.00 |

Table 3-2 Resistance Factors (AASHTO 2014 Table 11.5.6-1)

| Resistance Factor | Variable | Value | Unit |
|---|---|---|---|
| Sliding of MSE on foundation Soil | $\phi_s$ | 1.00 | dim |
| Bearing Resistance | $\phi_b$ | 0.65 | dim |
| Tensile Resistance (single point connector) | $\phi_t$ | 0.75 | dim |
| Pullout Resistance | $\phi_p$ | 0.90 | dim |

## 3.1 Extreme Force Effect

The load combination that creates the most extreme force effect is required to be checked. For external stability three combinations of load factors are checked. The first combination, Strength I-Maximum, uses the maximum load factors for both the vertical loads and the horizontal loads. The second combination, Strength I-Minimum, uses the minimum load factors for both the vertical loads and the horizontal loads. The third combination, Strength I-Critical, checks the minimum load factors against the maximum load factors using both the vertical and horizontal load combinations, e.g. minimum load factors for vertical loads checked against maximum load factors for horizontal loads. For the sliding analysis the most extreme force effect typically occurs using the minimum vertical force and maximum horizontal force, e.g. Strength-I Maximum controls. For the limiting eccentricity analysis, the most extreme force effect typically occurs using the minimum resisting moment and maximum overturning moment. For the bearing resistance analysis, the most extreme force effect typically occurs in the Strength I Maximum load factor combination. However, a critical value check is made using the minimum vertical force with the minimum resisting moment and the maximum overturning moment. The most severe force effect is the minimum CDR for sliding analysis, the largest eccentricity in the Limiting Eccentricity analysis, and the maximum bearing stress in the Bearing Resistance analysis. The most severe force effect may not be the value displayed in critical value check.

When an earth surcharge is applied at the top of the structure the horizontal force at the back of the MSE mass shall use consistent load factors in the critical check, e.g. if the horizontal force at the back of the retained mass of soil uses maximum load factors the complement vertical force will use maximum load factors as well. This will maintain consistency between the loads.

# 4 Define Steel Corrosion Rates

Steel that is buried in soil degrades over time. The amount of degradation is a function of the soil electro-chemical composition, steel protective coating and the length of time it is buried in soil. The reinforced backfill will be assumed to meet the electrochemical properties specified in AASHTO Article 11.01.6.4.2a. The protective coating for the steel will be zinc and it will be applied by the method of hot-dip galvanizing in conformance with ASTM A123. The thickness of the galvanized coating shall be a minimum of 3.4 mils. The corrosion rates are in conformance with AASHTO Article 11.10.6.4.2a as defined in Table 4-1 and the degradation thickness is calculated using Equation 4-1. The design life is typically specified by the Owner. When not specified a minimum design life of 75 years will be used.

$$E_s = 2 \cdot \left( \text{Life} - \left( \frac{t_{galv} - t_{zinc1}}{t_{zinc2}} + 2 \cdot yr \right) \right) \cdot t_{steel}$$ Equation 4-1

Table 4-1 Corrosion Parameters

| Location | Variable | Value | Unit |
|---|---|---|---|
| Design Life | Life | 75.00 | yrs |
| Galvanizing thickness | $t_{Galv}$ | 3.40 | mil |
| Zinc Loss first Two years | $t_{zinc1}$ | 0.58 | mil |
| Zinc Loss remaining years | $t_{zinc2}$ | 0.16 | mil/yr |
| Steel Loss Rate | $t_{steel}$ | 0.47 | mil/yr |
| Steel Loss for given design life | $E_s$ | 0.055 | in |

## 5 Define Internal Stability Factors

The internal stability factors for the lateral stress ratio ($K_r$) and the pullout friction factor (F*) shall be equal to the default values for metal strips and are consistent with AASHTO Article 11.10. The lateral stress ratio factor is taken from Table 11.10.6.2.1-3 and the pullout friction factor, F*, is derived from pullout testing. Both sets of factors decrease linearly to a depth of 20 feet at which time they remain constant with depth. Reference Section 8.2 for the calculation methodology used to determine the factors at any given depth.

Table 5-1 Internal Stability Parameters

| Location | Variable | Value | Unit |
|---|---|---|---|
| Earth Pressure Ratio at depth equal to 0.00’ | $K_{r0}$ | 1.70 | dim |
| Earth Pressure Ratio at depth equal to 20.00’ | $K_{r20}$ | 1.20 | dim |
| Pullout friction factor at depth equal to 0.00’ | $F^*_0$ | 3.00 | dim |
| Pullout friction factor at depth equal to 20.00’ | $F^*_{20}$ | 0.67 | dim |

## 6 Calculate Unfactored Loads for External Stability

The external stability of the MSE wall is a function of the various forces and moments as shown in Figure 6-1. In the LRFD context, the forces and moments need to be categorized into various load types. The primary load types for this example problem are the soil loads (EV, EH) and the live load (LS).

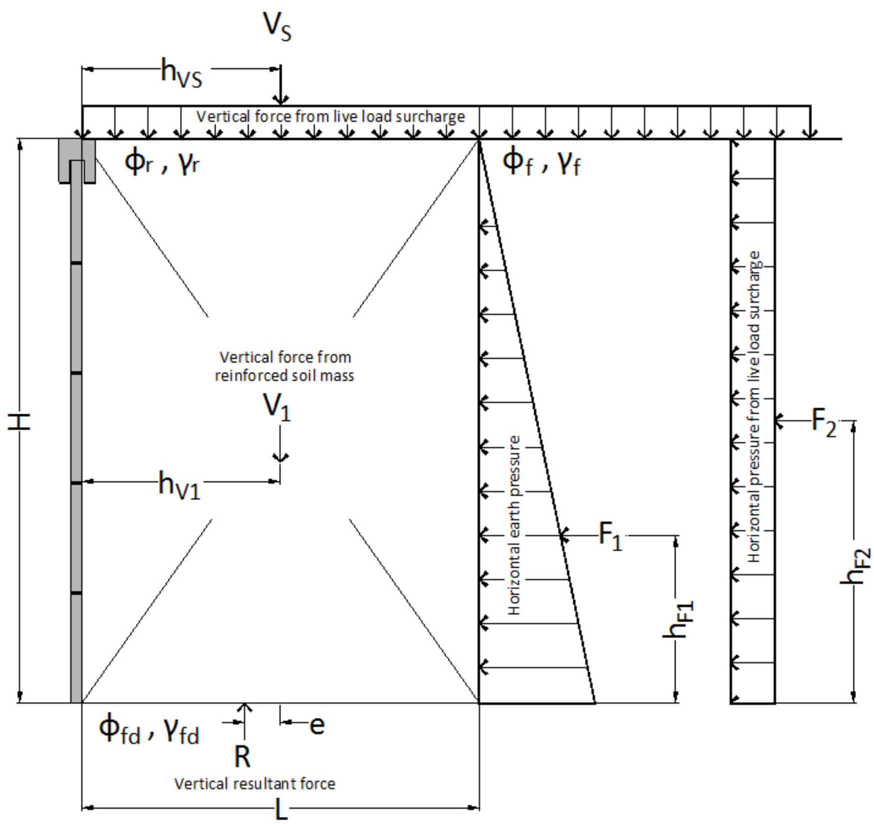



**Figure 6-1 Unfactored Load Diagram for External Stability**

Table 6-1 Unfactored Vertical Forces

| Vertical Force (Force/Length) | | Moment Arm (Length) | | Load Type |
|---|---|---|---|---|
| Equation | Value | Equation | Value | |
| $V_1 = \gamma_r \cdot H \cdot L$ | 78.75 k/ft | $h_{V1} = \frac{L}{2}$ | 10.50 ft | EV |
| $V_s = q \cdot L$ | 5.25 k/ft | $h_{VS} = \frac{L}{2}$ | 10.50 ft | LS |

Table 6-2 Unfactored Vertical Moments

| Moment (Force-Length/Length) | | Load Type |
|---|---|---|
| Equation | Value | |
| $M_{V1} = V_1 \cdot h_{V1}$ | 826.88 k-ft/ft | EV |
| $M_{VS} = V_S \cdot h_{VS}$ | 55.13 k-ft/ft | LS |

Table 6-3 Unfactored Horizontal Forces

| Vertical Force (Force/Length) | | Moment Arm (Length) | | Load Type |
|---|---|---|---|---|
| Equation | Value | Equation | Value | |
| $F_1 = \frac{1}{2} \cdot K_{af} \cdot \gamma_f \cdot H^2$ | 15.90 k/ft | $h_{F1} = \frac{H}{3}$ | 10.00 ft | EH |
| $F_2 = K_{af} \cdot q \cdot H$ | 2.12 k/ft | $h_{F2} = \frac{H}{2}$ | 15.00 ft | LS |

Table 6-4 Unfactored Horizontal Moments

| Moment (Force-Length/Length) | | Load Type |
|---|---|---|
| Equation | Value | |
| $M_{V1} = V_1 \cdot h_{V1}$ | 159.03 k-ft/ft | EH |
| $M_{VS} = V_S \cdot h_{VS}$ | 31.81 k-ft/ft | LS |

# 7 Evaluate External Stability

## 7.1 Calculate Sliding Resistance at Base of MSE Structure

The following calculations are used to evaluate the sliding resistance at the base of the MSE wall. For the purpose of the sliding calculation the beneficial contribution of the live load to resisting forces and moments is neglected. The calculations for sliding resistance at the base of the MSE wall for the three required load conditions (maximum, minimum and critical check) are illustrated in Table 7-1 through Table 7-3. The critical check uses the most severe force effect and is a combination of maximum and minimum values. For sliding the most severe force effect occurs when the resisting force is a minimum value and the driving force is a maximum value. Sliding resistance is a strength limit state check and therefore service limit state calculations are not considered. The minimum friction angle of the foundation soil, $\phi_{fd}$, and the reinforced soil, $\phi_r$, will be used for the sliding analysis.

Table 7-1 Evaluation of Sliding at Base of MSE Wall – Strength I Maximum

| Item | Equation | Value |
|---|---|---|
| Lateral Loads on the MSE wall | $F = \gamma_{EHmax} \cdot F_1 + \gamma_{LSmax} \cdot F_2$ | 27.56 k/ft |
| Vertical Load at Base of Wall without LS | $V = \gamma_{EVmax} \cdot V_1$ | 106.31 k/ft |
| Nominal sliding resistance at base of MSE wall | $V_{Nm} = V \cdot \tan(\phi_{fd})$ | 61.38 k/ft |
| Sliding resistance at base of MSE wall | $V_{Fm} = \phi_s \cdot V_{Nm}$ | 61.38 k/ft |
| Is $V_{Fm} > F_m$ | | Yes |
| Capacity Demand Ratio (CDR) for sliding | $CDR_s = \frac{V_{Fm}}{F}$ | 2.23 |

Table 7-2 Evaluation of Sliding at Base of MSE Wall – Strength I Minimum

| Item | Equation | Value |
|---|---|---|
| Lateral Loads on the MSE wall | $F = \gamma_{EHmin} \cdot F_1 + \gamma_{LSmin} \cdot F_2$ | 18.02 k/ft |
| Vertical Load at Base of Wall without LS | $V = \gamma_{EVmin} \cdot V_1$ | 78.75 k/ft |
| Nominal sliding resistance at base of MSE wall | $V_{Nm} = V \cdot \tan(\phi_{fd})$ | 45.47 k/ft |
| Sliding resistance at base of MSE wall | $V_{Fm} = \phi_s \cdot V_{Nm}$ | 45.47 k/ft |
| Is $V_{Fm} > F_m$ | | Yes |

Table 7-2 Evaluation of Sliding at Base of MSE Wall – Strength I Minimum

| Item | Equation | Value |
|---|---|---|
| Capacity Demand Ratio (CDR) | $CDR_s = \frac{V_{Fm}}{F}$ | 2.52 |

Table 7-3 Evaluation of Sliding at Base of MSE Wall – Strength I Critical Value Check

| Item | Equation | Value |
|---|---|---|
| Lateral Loads on the MSE wall | $F = \gamma_{EHmax} \cdot F_1 + \gamma_{LSmax} \cdot F_2$ | 27.56 k/ft |
| Vertical Load at Base of Wall without LS | $V = \gamma_{EVmin} \cdot V_1$ | 78.75 k/ft |
| Nominal sliding resistance at base of MSE wall | $V_{Nm} = V \cdot \tan(\phi_{fd})$ | 45.47 k/ft |
| Sliding resistance at base of MSE wall | $V_{Fm} = \phi_s \cdot V_{Nm}$ | 45.47 k/ft |
| Is $V_{Fm} > F_m$ | | Yes |
| Capacity Demand Ratio (CDR) | $CDR_s = \frac{V_{Fm}}{F}$ | 1.65 |

The critical value check results in the smallest CDR for this example and therefore is the most severe load effect and governs the sliding mode of failure (Table 7-3).

## 7.2 Calculate Limiting Eccentricity at Base of MSE Structure

The following calculations are used to evaluate the limiting eccentricity at the base of the MSE structure. For the purpose of this calculation the beneficial contribution of live load to resisting forces and moments is neglected. The calculations for limiting eccentricity at the base of the MSE structure for the three required load conditions are illustrated in Table 7-4 through Table 7-6. The critical check uses the most severe force effect and is a combination of maximum and minimum values. For limiting eccentricity the most severe force effect occurs when the resisting moment is a minimum value and the overturning moment is a maximum value. Note that limiting eccentricity is a strength limit state check and therefore service limit state calculations are not performed. Included in the calculation is the Capacity Demand Ratio for overturning. The MSE structure is founded on soil and therefore the eccentricity shall be located within the middle 2/3 of the base of the wall in conformance with AASHTO 11.6.3.3.

Table 7-4 Evaluation of Limiting Eccentricity for MSE Wall – Strength I Maximum

| Item | Equation | Value |
|---|---|---|
| Lateral Loads on the MSE wall | $F = \gamma_{EHmax} \cdot F_1 + \gamma_{LSmax} \cdot F_2$ | 27.56 k/ft |
| Overturning Moment about point A | $M_{OA} = \gamma_{EHmax} \cdot M_{F1} + \gamma_{LSmax} \cdot M_{F2}$ | 294.20 (k-ft)/ft |
| Vertical Load at Base of Wall without LS | $V_A = \gamma_{EVmax} \cdot V_1$ | 106.31 k/ft |
| Resisting moment about point A without LS | $M_{RA} = \gamma_{EVmax} \cdot M_{V1}$ | 1116.28 (k-ft)/ft |
| Net moment about Point A | $M_A = M_{RA} - M_{OA}$ | 822.08 (k-ft)/ft |
| Location of resultant force on base of wall from point A | $a = \frac{M_A}{V_A}$ | 7.73 ft |
| Eccentricity at base of MSE wall | $e_L = \frac{L}{2} - a$ | 2.77 ft |
| Limiting eccentricity | $e_{limit} = \frac{L}{3}$ | 7.00 ft |
| Is the resultant within limiting value of e | | Yes |
| Calculated e/L | $\frac{e_L}{L}$ | 0.13 dim |
| Capacity Demand Ratio (CDR) for Overturning | $CDR_O = \frac{M_{RA}}{M_{OA}}$ | 3.79 |

Table 7-5 Evaluation of Limiting Eccentricity for MSE Wall – Strength I Minimum

| Item | Equation | Value |
|---|---|---|
| Lateral Loads on the MSE wall | $F = \gamma_{EHmin} \cdot F_1 + \gamma_{LSmin} \cdot F_2$ | 18.02 k/ft |
| Overturning Moment about point A | $M_{OA} = \gamma_{EHmin} \cdot M_{F1} + \gamma_{LSmin} \cdot M_{F2}$ | 198.78 (k-ft)/ft |
| Vertical Load at Base of Wall without LS | $V_A = \gamma_{EVmin} \cdot V_1$ | 78.75 k/ft |
| Resisting moment about point A without LS | $M_{RA} = \gamma_{EVmin} \cdot M_{V1}$ | 826.88 (k-ft)/ft |
| Net moment about Point A | $M_A = M_{RA} - M_{OA}$ | 628.09 (k-ft)/ft |

Table 7-5 Evaluation of Limiting Eccentricity for MSE Wall – Strength I Minimum

| Item | Equation | Value |
|---|---|---|
| Location of resultant force on base of wall from point A | $a = \frac{M_A}{V_A}$ | 7.98 ft |
| Eccentricity at base of MSE wall | $e_L = \frac{L}{2} - a$ | 2.52 ft |
| Limiting eccentricity | $e_{limit} = \frac{L}{3}$ | 7.00 ft |
| Is the resultant within limiting value of e | | Yes |
| Calculated e/L | $\frac{e_L}{L}$ | 0.12 dim |
| Capacity Demand Ratio (CDR) for Overturning | $CDR_O = \frac{M_{RA}}{M_{OA}}$ | 4.16 |

Table 7-6 Evaluation of Limiting Eccentricity for MSE Wall – Strength I Critical Value Check

| Item | Equation | Value |
|---|---|---|
| Lateral Loads on the MSE wall | $F = \gamma_{EHmax} \cdot F_1 + \gamma_{LSmax} \cdot F_2$ | 27.56 k/ft |
| Overturning Moment about point A | $M_{OA} = \gamma_{EHmax} \cdot M_{F1} + \gamma_{LSmax} \cdot M_{F2}$ | 294.20 (k-ft)/ft |
| Vertical Load at Base of Wall without LS | $V_A = \gamma_{EVmin} \cdot V_1$ | 78.75 k/ft |
| Resisting moment about point A without LS | $M_{RA} = \gamma_{EVmin} \cdot M_{V1}$ | 826.88 (k-ft)/ft |
| Net moment about Point A | $M_A = M_{RA} - M_{OA}$ | 532.67 (k-ft)/ft |
| Location of resultant force on base of wall from point A | $a = \frac{M_A}{V_A}$ | 6.76 ft |
| Eccentricity at base of MSE wall | $e_L = \frac{L}{2} - a$ | 3.74 ft |
| Limiting eccentricity | $e_{limit} = \frac{L}{3}$ | 7.00 ft |
| Is the resultant within limiting value of e | | Yes |

Table 7-6 Evaluation of Limiting Eccentricity for MSE Wall – Strength I Critical Value Check

| Item | Equation | Value |
|---|---|---|
| Effective width at base of MSE wall | $B' = L - 2 \cdot e_L$ | 13.53 ft |
| Calculated e/L | $\frac{e_L}{L}$ | 0.18 dim |
| Capacity Demand Ratio (CDR) for Overturning | $CDR_O = \frac{M_{RA}}{M_{OA}}$ | 2.81 |

The critical value check results in the largest eccentricity for this example and therefore is the most sever load effect and governs the limiting eccentricity (Table 7-6).

### 7.3 Calculate Bearing Resistance at Base of MSE Structure

For bearing resistance calculations, the effect of live load over the soil reinforcing is included since it is the most extreme force effect and therefore creates the largest bearing stress. The bearing stress at the base of the MSE wall is calculated as shown in Equation 7-1.

$$\sigma_V = \frac{\Sigma V}{L - 2 \cdot e_L} \qquad \text{Equation 7-1}$$

Where the value $\Sigma V = R = V_1 + V_S$ is the resultant of vertical forces and the load eccentricity $e_L$ is calculated by principles of statics using appropriate loads and moments with the applicable load factors.

In the LRFD, $\sigma_V$ is compared with the factored bearing resistance when computed for strength limit state and for settlement analysis it is comparted to the service limit state bearing resistance. The various calculations for evaluation of bearing resistance are presented in Table 7-7 through Table 7-10. The Service I load combination is evaluated to compute the bearing stress for any settlement analysis and the results are presented in Table 7-10.

Table 7-7 Evaluation of Bearing Resistance for MSE Wall – Strength I Maximum

| Item | Equation | Value |
|---|---|---|
| Lateral Loads on the MSE wall | $F = \gamma_{EHmax} \cdot F_1 + \gamma_{LSmax} \cdot F_2$ | 27.56 k/ft |
| Overturning Moment about point A | $M_{OA} = \gamma_{EHmax} \cdot M_{F1} + \gamma_{LSmax} \cdot M_{F2}$ | 294.20 (k-ft)/ft |

Table 7-7 Evaluation of Bearing Resistance for MSE Wall – Strength I Maximum

| Item | Equation | Value |
|---|---|---|
| Vertical Load at Base of Wall with LS | $V_A = \gamma_{EVmax} \cdot V_1 + \gamma_{LSmax} \cdot V_S$ | 115.50 k/ft |
| Resisting moment about point A with LS | $M_{RA} = \gamma_{EVmax} \cdot M_1 + \gamma_{LSmax} \cdot M_S$ | 1212.75 (k-ft)/ft |
| Net moment about Point A | $M_A = M_{RA} - M_{OA}$ | 918.55 (k-ft)/ft |
| Location of resultant force on base of wall from point A | $a = \frac{M_A}{V_A}$ | 7.95 ft |
| Eccentricity at base of MSE wall from center | $e_L = \frac{L}{2} - a$ | 2.55 ft |
| Limiting eccentricity | $e_{limit} = \frac{L}{3}$ | 7.00 ft |
| Is the resultant within limiting value of e | | Yes |
| Effective width at base of MSE wall | $B' = L - 2 \cdot e_L$ | 15.91 ft |
| Bearing stress due to MSE Wall | $\sigma_V = \frac{\gamma_{EVmax} \cdot V_1 + \gamma_{LSmax} \cdot V_S}{L - 2 \cdot e_L}$ | 7.26 ksf |

Table 7-8 Evaluation of Bearing Resistance for MSE Wall – Strength I Minimum

| Item | Equation | Value |
|---|---|---|
| Lateral Loads on the MSE wall | $F = \gamma_{EHmin} \cdot F_1 + \gamma_{LSmin} \cdot F_2$ | 18.02 k/ft |
| Overturning Moment about point A | $M_{OA} = \gamma_{EHmin} \cdot M_{F1} + \gamma_{LSmin} \cdot M_{F2}$ | 198.78 (k-ft)/ft |
| Vertical Load at Base of Wall with LS | $V_A = \gamma_{EVmin} \cdot V_1 + \gamma_{LSmin} \cdot V_S$ | 87.94 k/ft |
| Resisting moment about point A with LS | $M_{RA} = \gamma_{EVmin} \cdot M_1 + \gamma_{LSmin} \cdot M_S$ | 923.34 (k-ft)/ft |
| Net moment about Point A | $M_A = M_{RA} - M_{OA}$ | 724.56 (k-ft)/ft |
| Location of resultant force on base of wall from point A | $a = \frac{M_A}{V_A}$ | 8.24 ft |
| Eccentricity at base of MSE wall from center | $e_L = \frac{L}{2} - a$ | 2.26 ft |

Table 7-8 Evaluation of Bearing Resistance for MSE Wall – Strength I Minimum

| Item | Equation | Value |
|---|---|---|
| Limiting eccentricity | $e_{limit} = \frac{L}{3}$ | 7.00 ft |
| Is the resultant within limiting value of e | | Yes |
| Effective width at base of MSE wall | $B' = L - 2 \cdot e_L$ | 16.48 ft |
| Bearing stress due to MSE Wall | $\sigma_V = \frac{\gamma_{EVmin} \cdot V_1 + \gamma_{LSmin} \cdot V_S}{L - 2 \cdot e_L}$ | 5.34 ksf |

Table 7-9 Evaluation of Bearing Resistance for MSE Wall – Strength I Critical Value Check

| Item | Equation | Value |
|---|---|---|
| Lateral Loads on the MSE wall | $H_m = \gamma_{EHmax} \cdot F_1 + \gamma_{LSmax} \cdot F_2$ | 27.56 k/ft |
| Overturning Moment about point A | $M_{OA} = \gamma_{EHmax} \cdot M_{F1} + \gamma_{LSmax} \cdot M_{F2}$ | 294.20 (k-ft)/ft |
| Vertical Load at Base of Wall with LS | $V_A = \gamma_{EVmin} \cdot V_1 + \gamma_{LSmin} \cdot V_S$ | 87.94 k/ft |
| Resisting moment about point A with LS | $M_{RA} = \gamma_{EVmin} \cdot M_{V1} + \gamma_{LSmin} \cdot M_{VS}$ | 923.34 (k-ft)/ft |
| Net moment about Point A | $M_A = M_{RA} - M_{OA}$ | 629.14 (k-ft)/ft |
| Location of resultant force on base of wall from point A | $a = \frac{M_A}{V_A}$ | 7.15 ft |
| Eccentricity at base of MSE wall from center | $e_L = \frac{L}{2} - a$ | 3.35 ft |
| Limiting eccentricity | $e_{limit} = \frac{L}{3}$ | 7.00 ft |
| Is the resultant within limiting value of e | | Yes |
| Effective width at base of MSE wall | $B' = L - 2 \cdot e_L$ | 14.31 ft |
| Bearing stress due to MSE Wall | $\sigma_V = \frac{\gamma_{EVmin} \cdot V_1 + \gamma_{LSmin} \cdot V_S}{L - 2 \cdot e_L}$ | 6.15 ksf |

Table 7-10 Evaluation of Bearing Resistance for MSE Wall – Strength I Service

| Item | Equation | Value |
|---|---|---|
| Lateral Loads n the MSE wall | $F = F_1 + F_2$ | 18.02 k/ft |
| Overturning Moment about point A | $M_{OA} = F_1 \cdot h_{F1} + F_2 \cdot h_{F2}$ | 190.83 (k-ft)/ft |
| Vertical Load at Base of Wall without LS | $V_A = V_1 + V_S$ | 84.00 k/ft |
| Resisting moment about point A without LS | $M_{RA} = V_1 \cdot h_{V1} + V_S \cdot h_{VS}$ | 882.00 (k-ft)/ft |
| Net moment about Point A | $M_A = M_{RA} - M_{OA}$ | 691.17 (k-ft)/ft |
| Location of resultant force on base of wall from point A | $a = \frac{M_A}{V_A}$ | 8.23 ft |
| Eccentricity at base of MSE wall from center | $e_L = \frac{L}{2} - a$ | 2.27 ft |
| Limiting eccentricity | $e_{limit} = \frac{L}{6}$ | 3.50 ft |
| Is the resultant within limiting value of e | | Yes |
| Effective width at base of MSE wall | $B' = L - 2 \cdot e_L$ | 16.46 ft |
| Bearing stress due to MSE Wall | $\sigma_V = \frac{V_1 + V_S}{L - 2 \cdot e_L}$ | 5.10 ksf |

The Strength I Maximum load combination result in the extreme force effect in terms of maximum bearing stress and therefore governs the bearing resistance mode of failure (Table 7-7)

## 7.4 Summary of External Stability

Table 7-11 provides a summary of the external stability results. These values can be compared to the results of the proprietary software program RSW-Calc and the Adama Engineering software program RSW-Calc. The capacity demand ratio must be greater or equal to 1.00. All comparable values are within 2%.

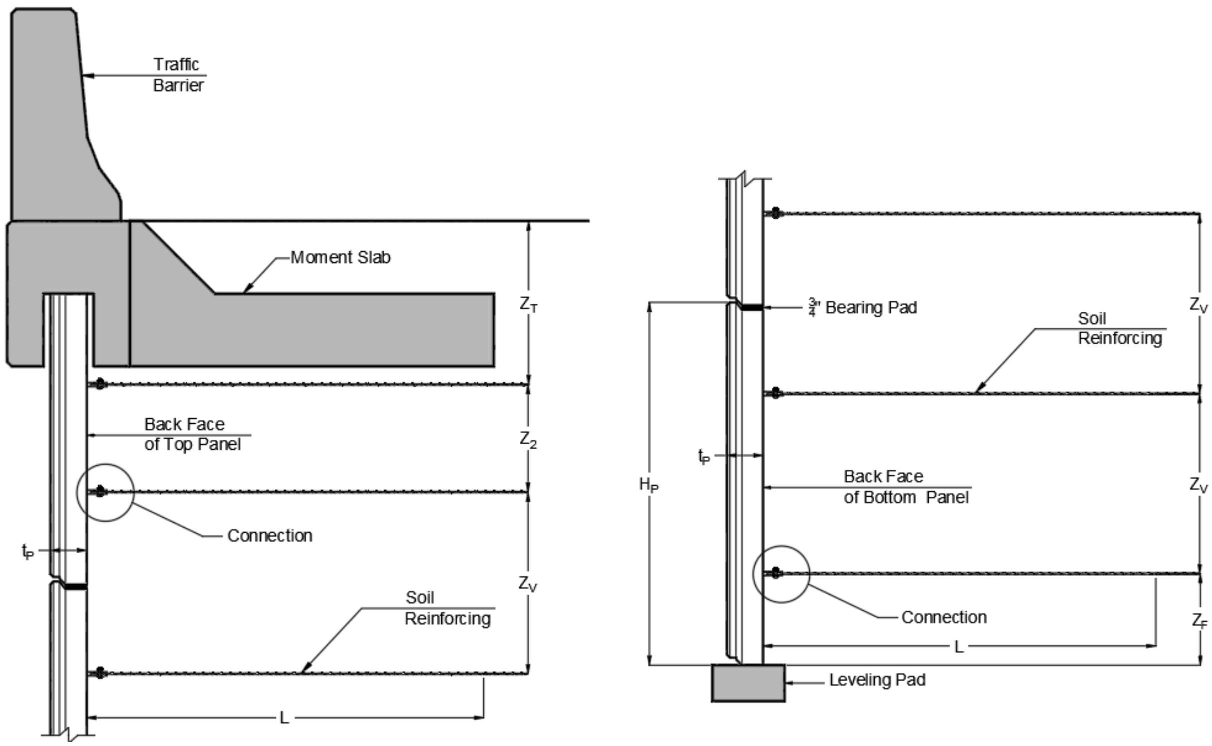
Table 7-11 Summary of External Stability

| Load Combination | CDR Sliding (dim) | Limiting Eccentricity/Maximum (ft) | CDR Overturning (dim) | Bearing Stress (ksf) | Effective Length (ft) |
|---|---|---|---|---|---|
| Strength I Maximum | 2.23 | 2.77/7.00 | 3.79 | 7.26 | 15.91 |
| Strength I Minimum | 2.52 | 2.52/7.00 | 4.16 | 5.34 | 16.48 |
| Strength I Critical Value Check | 1.65 | 3.74/7.00 | 2.81 | 6.15 | 14.31 |
| Service I | NA | NA | NA | 5.10 | 16.46 |

# 8 Evaluate Internal Stability

## 8.1 Determine Soil Reinforcing Depths and Tributary Area for Internal Stability

The depth to the soil reinforcing is a function of the facing panel and the vertical spacing of the anchors that are used to attach the soil reinforcing to the facing panel. The depth for the top soil reinforcing is a function of the top of wall treatment, e.g. coping element that is placed at the top of the structure. In the tables that follow the bottom most soil reinforcing element is assumed to be element 1.




Top of Structure        Bottom of Structure

**Figure 8-1 Soil Reinforcing Depth Parameters**

Table 8-1 Soil Reinforcing Layout

| Soil Element | Distance Above Leveling Pad ft | Depth Below Top of Wall ($Z_i$) ft | Vertical Spacing ($S_v$) ft |
|---|---|---|---|
| 12 | 27.75 | 2.25 | 3.00 |
| 11 | 26.25 | 3.75 | 2.00 |
| 10 | 23.75 | 6.25 | 2.50 |
| 9 | 21.25 | 8.75 | 2.50 |
| 8 | 18.75 | 11.25 | 2.50 |
| 7 | 16.25 | 13.75 | 2.50 |
| 6 | 13.75 | 16.25 | 2.50 |
| 5 | 11.25 | 18.75 | 2.50 |
| 4 | 8.75 | 21.25 | 2.50 |
| 3 | 6.25 | 23.75 | 2.50 |
| 2 | 3.75 | 26.25 | 2.50 |
| 1 | 1.25 | 28.75 | 2.50 |

## 8.2 Determine Variation of $K_r$, F*and Length of Embedment for Internal Stability

The failure surface for inextensible soil reinforcing is bilinear as shown in Figure 8-2. The failure surface is a function of the surcharge back-slope at the top of the structure and intersects the ground surface at the location of the mechanical height, $H_m$. The mechanical height is calculated using Equation 8-1. The length of embedment ($L_e$) is the length of the soil reinforcing that is contained behind the failure surface.

$$H_m = \mathrm{If}\left\{X_s > 0.3\cdot(H+S), H+\frac{\tan(\beta_s)\cdot 0.3\cdot H}{1-0.3\cdot\tan(\beta_s)}, H+S\right\}$$ Equation 8-1

For a horizontal back-slope with no earth surcharge the mechanical height is equal to the design height of the structure, H.

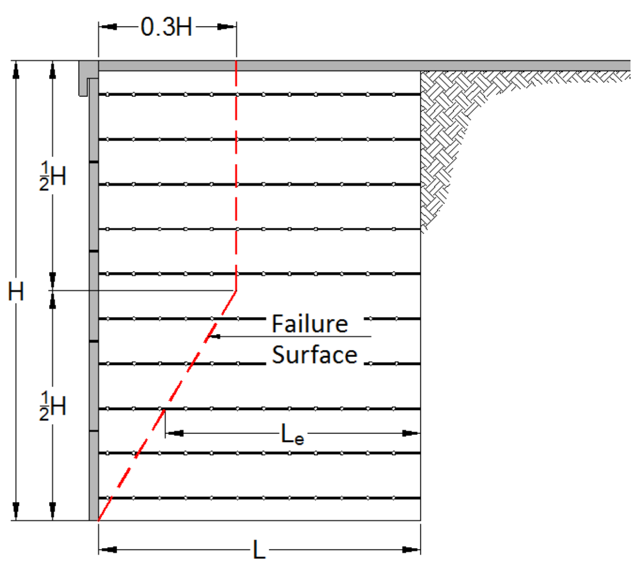



**Figure 8-2 Failure Surface Diagram**

The internal earth pressure coefficient and the pullout friction factor are calculated from the top of the structure H to the depth of the soil reinforcing $z_i$.

$$K_i = \left[1.7 - \frac{1.7-1.2}{20(ft)}\cdot z_i\right]\cdot K_a \rightarrow z_i < 20(ft)$$ Equation 8-2

$$K_i = 1.2\cdot(K_a) \rightarrow z_i \geq 20(ft)$$ Equation 8-3

$$F^* = 3.00 - \frac{3.00 - 0.67}{20(\text{ft})} \cdot z_i \rightarrow z_i < 20(\text{ft})$$ Equation 8-4

$$F^* = 0.67 \rightarrow z_i \geq 20(\text{ft})$$ Equation 8-5

Table 8-2 Internal Stability Factors

| Soil Element | Depth Below Top of Wall $Z_i$ ft | Earth Pressure Ratio $K_i$ dim | Pullout Friction Factor F* dim | Length of Embedment $L_e$ ft |
|---|---|---|---|---|
| 12 | 2.25 | 1.64 | 2.74 | 12.00 |
| 11 | 3.75 | 1.61 | 2.56 | 12.00 |
| 10 | 6.25 | 1.54 | 2.27 | 12.00 |
| 9 | 8.75 | 1.48 | 1.98 | 12.00 |
| 8 | 11.25 | 1.42 | 1.69 | 12.00 |
| 7 | 13.75 | 1.36 | 1.40 | 12.00 |
| 6 | 16.25 | 1.29 | 1.11 | 12.75 |
| 5 | 18.75 | 1.23 | 0.82 | 14.25 |
| 4 | 21.25 | 1.20 | 0.67 | 15.75 |
| 3 | 23.75 | 1.20 | 0.67 | 17.25 |
| 2 | 26.25 | 1.20 | 0.67 | 18.75 |
| 1 | 28.75 | 1.20 | 0.67 | 20.25 |

### 8.3 Calculate Horizontal Stress at Elevation of Each Soil Reinforcing Element

The horizontal stress at each soil elevation is a function of the vertical stress, $\sigma_V$, and coefficient of lateral earth pressure, $K_i$. The vertical stress at each soil reinforcing depth is calculated using Equation 8-6 and the horizontal stress is calculated using Equation 8-7. The lateral earth pressure coefficient is calculated using either Equation 8-2 or Equation 8-3.

$$\sigma_V = \gamma_{EV} \cdot (\gamma_r \cdot z_i + q)$$ Equation 8-6

$$\sigma_H = K_i \cdot \sigma_V$$ Equation 8-7

Table 8-3 Stress Calculation

| Soil Element | Depth Below Top of Wall $z_i$ ft | Vertical Stress $\sigma_V$ sf | Earth Pressure Coefficient $K_i$ dim | Horizontal Stress $\sigma_H$ ksf |
|---|---|---|---|---|
| 12 | 2.25 | 0.72 | 0.46 | 0.33 |
| 11 | 3.75 | 0.97 | 0.46 | 0.44 |
| 10 | 6.25 | 1.39 | 0.44 | 0.61 |

Table 8-3 Stress Calculation

| Soil Element | Depth Below Top of Wall $z_i$ ft | Vertical Stress $\sigma_V$ sf | Earth Pressure Coefficient $K_i$ dim | Horizontal Stress $\sigma_H$ ksf |
|---|---|---|---|---|
| 9 | 8.75 | 1.81 | 0.42 | 0.76 |
| 8 | 11.25 | 2.24 | 0.40 | 0.90 |
| 7 | 13.75 | 2.66 | 0.38 | 1.02 |
| 6 | 16.25 | 3.08 | 0.36 | 1.12 |
| 5 | 18.75 | 3.50 | 0.35 | 1.22 |
| 4 | 21.25 | 3.92 | 0.34 | 1.33 |
| 3 | 23.75 | 4.35 | 0.34 | 1.47 |
| 2 | 26.25 | 4.77 | 0.34 | 1.62 |
| 1 | 28.75 | 5.19 | 0.34 | 1.76 |

## 8.4 Calculate Maximum Tension at Elevation of Each Soil Reinforcing Element

The maximum tension at each soil reinforcing elevation is the product of the horizontal stress and the tributary area that the row of soil reinforcing is required to resist. The tributary spacing ($S_{TA}$) for the soil reinforcing depth being considered is the function of the vertical spacing of the soil reinforcing and is calculated as the distance between the soil reinforcing depth at the midpoint of the soil reinforcing above and the distance between the soil reinforcing depth at the midpoint of the soil reinforcing below (Equation 8-8). The tributary spacing is multiplied by the length of the panel to calculate the total tributary area, $A_T$ (Equation 8-9).

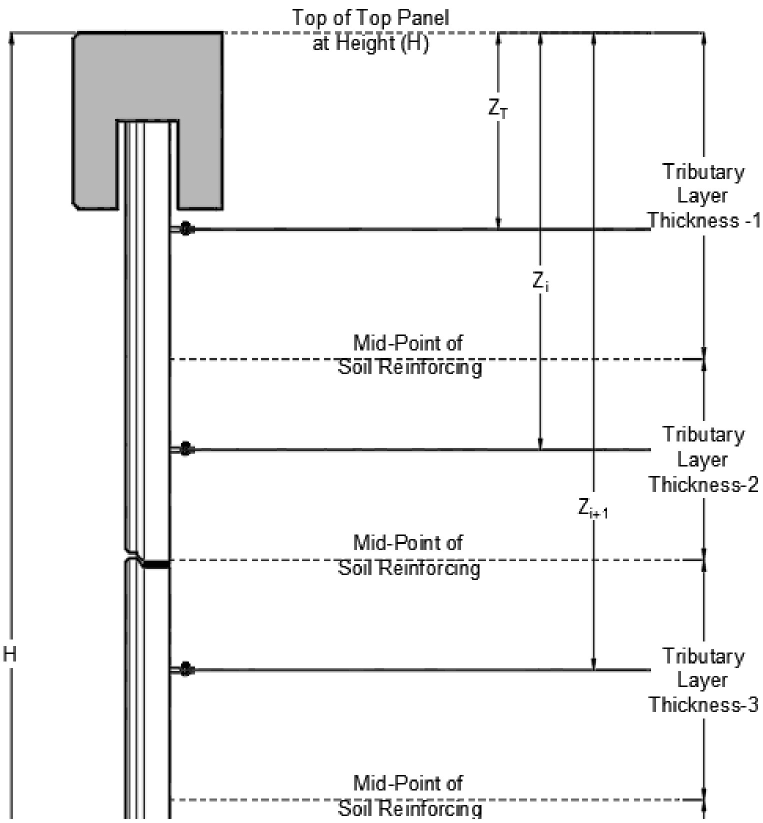



**Figure 8-3 Tributary Area**

$$S_{TA} = \frac{Z_i - Z_{top}}{2} + \frac{Z_{bot} - Z_i}{2} \qquad \text{Equation 8-8}$$

$$A_T = S_{TA} \cdot L_P \qquad \text{Equation 8-9}$$

The maximum tension required to be resisted by the row of soil reinforcing is the product of the horizontal stress times the tributary area (Equation 8-10).

$$T_{max} = \sigma_H \cdot A_T \qquad \text{Equation 8-10}$$

Table 8-4 Maximum Tension At Soil Reinforcing

| Soil Element | Depth Below Top of Wall $Z_i$ ft | Horizontal Stress $\sigma_H$ ksf | Tributary Area $A_T$ sf | Maximum Tension $T_{max}$ kip |
|---|---|---|---|---|
| 12 | 2.25 | 0.33 | 30.00 | 9.90 |
| 11 | 3.75 | 0.44 | 20.00 | 8.80 |
| 10 | 6.25 | 0.61 | 25.00 | 15.25 |

Table 8-4 Maximum Tension At Soil Reinforcing

| Soil Element | Depth Below Top of Wall $Z_i$ ft | Horizontal Stress $\sigma_H$ ksf | Tributary Area $A_T$ sf | Maximum Tension $T_{max}$ kip |
|---|---|---|---|---|
| 9 | 8.75 | 0.76 | 25.00 | 19.00 |
| 8 | 11.25 | 0.90 | 25.00 | 22.50 |
| 7 | 13.75 | 1.02 | 25.00 | 25.50 |
| 6 | 16.25 | 1.12 | 25.00 | 28.00 |
| 5 | 18.75 | 1.22 | 25.00 | 30.50 |
| 4 | 21.25 | 1.33 | 25.00 | 33.25 |
| 3 | 23.75 | 1.47 | 25.00 | 36.75 |
| 2 | 26.25 | 1.62 | 25.00 | 40.50 |
| 1 | 28.75 | 1.76 | 25.00 | 44.00 |

## 8.5 Determine Required Area of Steel at Elevation of Each Soil Reinforcing Element

After the maximum tensile force to be resisted by each row of soil reinforcing is calculated the area of steel required at the end of the design life can be determined (Equation 8-11). Once the required end of design life steel area is determined the required total steel area can be calculated (Equation 8-12). Based on the required total steel area the number of soil reinforcing elements for each row can be determined (Equation 8-13).

$$A_c = \frac{T_{max}}{\phi_t \cdot F_y}$$ Equation 8-11

$$A_{SS1} = (t_{SS1} - E_c) \cdot b$$ Equation 8-12

$$N_{SR} = \mathrm{int}\left[\frac{A_{req}}{A_{SS1}}\right] + 1$$ Equation 8-13

Where: $N_{SR}$ = Number of EarthTrac SS

$A_{SS}$ = Area of one EarthTrac SS

The value $N_{SR}$ is the integer value of the quotient plus 1 .

Table 8-5 Area of Steel Required for Rupture

| Soil Element | Depth Below Top of Wall $Z_i$ ft | Maximum Tension $T_{max}$ kip | Area of Steel Required at End of Design Life $A_c$ $In^2$ | Area of Steel Provided $A_{pro}$ $In^2$ | Number Soil Reinforcing Elements $N_{SR}$ dim | CDR Rupture dim |
|---|---|---|---|---|---|---|
| 12 | 2.25 | 9.90 | 0.26 | 0.61 | 3 | 2.29 |
| 11 | 3.75 | 8.80 | 0.23 | 0.61 | 3 | 2.58 |
| 10 | 6.25 | 15.25 | 0.41 | 0.61 | 3 | 1.49 |
| 9 | 8.75 | 19.00 | 0.51 | 0.61 | 3 | 1.19 |
| 8 | 11.25 | 22.50 | 0.60 | 0.61 | 3 | 1.01 |
| 7 | 13.75 | 25.50 | 0.68 | 0.81 | 4 | 1.19 |
| 6 | 16.25 | 28.00 | 0.75 | 0.81 | 4 | 1.08 |
| 5 | 18.75 | 30.50 | 0.81 | 1.01 | 5 | 1.24 |
| 4 | 21.25 | 33.25 | 0.89 | 1.01 | 5 | 1.14 |
| 3 | 23.75 | 36.75 | 0.98 | 1.01 | 5 | 1.03 |
| 2 | 26.25 | 40.50 | 1.08 | 1.21 | 6 | 1.12 |
| 1 | 28.75 | 44.00 | 1.17 | 1.21 | 6 | 1.03 |

## 8.6 Calculate Factored Pullout Resistance at Elevation of Each Soil Reinforcing Element

The nominal pullout of the soil reinforcing (Equation 8-15) is a function of the length of embedment ($L_e$), pullout friction factor (F*), depth of overburden ($\sigma_{VO}$), scale correction factor ($\alpha$), and the geometric factor (C). The factored pullout resistance ($P_\pi$) is the product of the nominal pullout resistance ($P_r$) multiplied by the pullout resistance factor, $\phi_{po}$ (Equation 8-16).

$$\sigma_{VO} = \gamma_r \cdot Z_i$$ Equation 8-14

$$P_r = \alpha \cdot C \cdot F^* \cdot \sigma_{VO} \cdot L_e$$ Equation 8-15

$$P_\pi = \phi_{po} \cdot P_r$$ Equation 8-16

Where: C = 2 (dim)

$\alpha$ = 1 (dim)

The number of EarthTrac soil reinforcing elements that are required to resist pullout for each row is a function of the maximum force that is to be resisted. The maximum force $T_{max}$ is calculated using Equation 8-17 and is not the same value that was calculated in Section 8.4 and Equation 8-18. AASHTO Article 11.10.6.2.1 states that the live load surcharge (q) used in Equation 8-6 is not required to be used in the calculation of $T_{max}$ for pullout. In this document the live load surcharge is not used in the calculation of $T_{max}$.

$$T_{max} = \gamma_{EV} \cdot \left(K_i \cdot \gamma_r \cdot z_i\right) \cdot A_T$$ Equation 8-17

$$n_{GS} = \frac{T_{max}}{P_\pi}$$ Equation 8-18

Table 8-6 Pullout Resistance for EarthTrac SS Element

| SR Element | Depth Below Top of Wall $Z_i$ ft | Length Of Embed. Le Ft | F* dim | Nominal Pullout Resistance for 1-SR $P_r$ kip | Factored Pullout Resistance $P_\pi$ kip | Max Tension $T_{max}$ kip | Number SR Elements $N_{SR}$ dim | CDR Pullout dim |
|---|---|---|---|---|---|---|---|---|
| 12 | 2.25 | 12.00 | 2.74 | 3.08 | 2.77 | 5.28 | 3 | 1.58 |
| 11 | 3.75 | 12.00 | 2.56 | 4.80 | 4.32 | 5.76 | 3 | 2.25 |
| 10 | 6.25 | 12.00 | 2.27 | 7.09 | 6.38 | 11.48 | 3 | 1.67 |
| 9 | 8.75 | 12.00 | 1.98 | 8.66 | 7.80 | 15.45 | 3 | 1.51 |
| 8 | 11.25 | 12.00 | 1.69 | 9.51 | 8.56 | 19.05 | 3 | 1.35 |
| 7 | 13.75 | 12.00 | 1.40 | 9.63 | 8.66 | 22.30 | 3 | 1.17 |
| 6 | 16.25 | 12.75 | 1.11 | 9.58 | 8.62 | 25.00 | 3 | 1.03 |
| 5 | 18.75 | 14.25 | 0.82 | 9.13 | 8.22 | 27.51 | 4 | 1.19 |
| 4 | 21.25 | 15.75 | 0.67 | 9.34 | 8.41 | 30.41 | 4 | 1.11 |
| 3 | 23.75 | 17.25 | 0.67 | 11.44 | 10.29 | 33.99 | 4 | 1.21 |
| 2 | 26.25 | 18.75 | 0.67 | 13.74 | 12.37 | 37.57 | 4 | 1.32 |
| 1 | 28.75 | 20.25 | 0.67 | 16.25 | 14.63 | 41.15 | 3 | 1.07 |

## 8.7 Summary of Internal Stability at Elevation of Each Soil Reinforcing Element

The number of EarthTrac soil reinforcing that are required is equal to the larger number of soil reinforcing calculated for rupture in Section 8.5 and the number of soil reinforcing calculated for pullout in Section 8.6. The capacity demand ratios calculated and summarized in Table 8-7 are based on the minimum number of EarthTrac SS that are required to satisfy both rupture and pullout.

Table 8-7 Summary of Internal Stability for EarthTrac SS Element

| SR | $Z_i$ ft | $N_{SR}$ dim | $\sigma_V$ ksf | $K_r$ dim | $\sigma_H$ ksf | $T_{max}$ kip | CDR Rupture dim | $L_e$ Ft | F* dim | CDR Pullout dim |
|---|---|---|---|---|---|---|---|---|---|---|
| 12 | 2.25 | 3 | 0.72 | 0.46 | 0.33 | 9.90 | 2.29 | 12.00 | 2.74 | 1.57 |
| 11 | 3.75 | 3 | 0.97 | 0.46 | 0.44 | 8.80 | 2.58 | 12.00 | 2.56 | 2.25 |
| 10 | 6.25 | 3 | 1.39 | 0.44 | 0.61 | 15.25 | 1.49 | 12.00 | 2.27 | 1.67 |

Table 8-7 Summary of Internal Stability for EarthTrac SS Element

| SR | $Z_i$ ft | $N_{SR}$ dim | $\sigma_V$ ksf | $K_r$ dim | $\sigma_H$ ksf | $T_{max}$ kip | CDR Rupture dim | $L_e$ Ft | F* dim | CDR Pullout dim |
|---|---|---|---|---|---|---|---|---|---|---|
| 9 | 8.75 | 3 | 1.81 | 0.42 | 0.76 | 19.00 | 1.19 | 12.00 | 1.98 | 1.51 |
| 8 | 11.25 | 3 | 2.24 | 0.40 | 0.90 | 22.50 | 1.01 | 12.00 | 1.69 | 1.35 |
| 7 | 13.75 | 4 | 2.66 | 0.38 | 1.02 | 25.50 | 1.19 | 12.00 | 1.40 | 1.55 |
| 6 | 16.25 | 4 | 3.08 | 0.36 | 1.12 | 28.00 | 1.08 | 12.75 | 1.11 | 1.38 |
| 5 | 18.75 | 5 | 3.50 | 0.35 | 1.22 | 30.50 | 1.24 | 14.25 | 0.82 | 1.49 |
| 4 | 21.25 | 5 | 3.92 | 0.34 | 1.33 | 33.25 | 1.14 | 15.75 | 0.67 | 1.38 |
| 3 | 23.75 | 5 | 4.35 | 0.34 | 1.47 | 36.75 | 1.03 | 17.25 | 0.67 | 1.51 |
| 2 | 26.25 | 6 | 4.77 | 0.34 | 1.62 | 40.50 | 1.12 | 18.75 | 0.67 | 1.98 |
| 1 | 28.75 | 6 | 5.19 | 0.34 | 1.76 | 44.00 | 1.03 | 20.25 | 0.67 | 2.13 |

## 9 MSEW Internal Stability Discussion

In order for MSEW to be compared to RSW-Calc and AASHTO requirements the internal stability analysis and external stability analysis must be run separately. In other words, when using MSEW, you must perform an external stability analysis as well as an internal stability analysis. MSEW does not allow for the use of different load factors for external stability and internal stability. AASHTO uses a load factor of 1.35 (EV) instead of 1.75 (LS) for traffic live load surcharge. This is discussed in the commentary in section 11.10.6.2-1 Maximum Reinforcement Loads.

MSEW may underestimate or overestimate the CDR for pullout and rupture for the top 2 rows of soil reinforcing. This occurs because MSEW uses an average horizontal pressure at each level of soil reinforcing that is based on the program's definition, and therefore, calculation of the Tributary Range. The Tributary Range is defined in MSEW by the variables Z-bottom and Z-top. For the top soil reinforcing element the Z-top elevation is defined as H and the Z-bottom elevation is defined as the average of the Metal Strip elevation for the top soil reinforcing layer and the second soil reinforcing layer. For the first soil reinforcing layer in this example Z-top is equal to 30.00 feet and the top soil reinforcing layer is defined at elevation 27.75 (a depth of 2.25 feet from the top of wall). The second soil reinforcing layer from the top is defined at elevation 26.25 (a depth of 3.75 feet from the top of wall). Therefore, the Z-bottom elevation is the average of 26.25(ft) and 27.75(ft) or 27.00(ft). The tributary range is the difference between Z-top and Z-bottom or 3.00(ft). In MSEW the horizontal stress for any level of soil reinforcing is equal to the average of the horizontal stress calculated in the Tributary Range. In other words the horizontal stress is the average of the horizontal stress calculated at Z-top and the horizontal stress calculated at Z-bottom. The maximum tension force per foot of wall is equal to the average horizontal stress times the tributary range. In the program MSEW the calculated CDR for the top soil reinforcing element and the second soil reinforcing element is 2.764 and 2.489 respectively. In the program RSW-Calc the calculated CDR for the top soil reinforcing element and the second soil reinforcing element is 2.30 and 2.58 respectively. MSEW over predicts the CDR in the top row. The same holds true for the pullout calculations. When the averaging method is used, and the soil reinforcing is below the midpoint of the Tributary Range MSEW underestimates the maximum tension force that is required to be resisted.

RSW-Calc does not use the average horizontal pressure to calculate the maximum tensile force. RSW-Calc uses the actual location of the soil reinforcing and actual the tributary area. The method used in RSW-Calc to determine the tributary area was defined in Section 8.1. The tributary area that each soil reinforcing element has to resist is defined as the mid-point distance between each soil reinforcing.

It is important to recognize how MSEW calculates the tension forces. It can underestimate or overestimate the CDR in soil reinforcing where the soil reinforcing spacing is not uniform. This becomes clearer when traffic impact or when large horizontal loads are applied in MSEW.

# Appendix A

## RSW-Calc Output

# MSE CALCULATIONS

Segmental Concrete Panel System

EarthTrac Steel Strip Soil Reinforcing

## RSW-CALC

EarthTec EarthTrac Verification Calculation

IDEA Submittal

2021

**LEVEL BACKSLOPE WITH LIVE LOAD SURCHARGE**

SUBMITTAL CALCULATIONS

LRFD PROCEDURE

SIMPLIFIED METHOD

Support Through Innovation

114 South Collins Street
Arlington, TX, 76034
817-223-0969

**Design Options**

- LRFD Procedure
- Simplified Method
- Live load is not applied to Tmax in pullout calculation - AASHTO Figure 11.10.6.2.1-1
- The Vertical Earth (EV) load factor is used for all internal loads - AASHTO 11.10.6.2.1-1
- The K- Ratio for pullout is set to 1.7 to 1.2 and not 1.7 to 1.2
- The K-Ratios are calculated from the top of the structure or top of coping

| Load Factors | Maximum | Minimum |
|---|---|---|
| Load factor for vertical earth pressure (EV) | 1.35 | 1.00 |
| Load factor for earth surcharge (ES) | 1.50 | 0.75 |
| Load factor for traffic live load (LS) | 1.75 | 1.75 |
| Load factor for horizontal earth pressure (EH) | 1.50 | 0.90 |
| Load factor for seismic (EQ) | 1.00 | |
| Load factor for structural components (DC) | 1.25 | 0.90 |
| Load factor for vehicular impact (CT) | 1.00 | 1.00 |

| Resistance Factor | Static | Seismic |
|---|---|---|
| Bearing resistance factor | 0.65 | |
| Sliding resistance factor | 1.00 | |
| Tensile resistance factor | 0.75 | 0.75 |
| Pullout resistance factor | 0.90 | 1.20 |
| Extreme event resistance factor | 1.00 | |

| Internal Earth Pressure Ratio | | |
|---|---|---|
| **Depth (Z)** | **Rupture** | **Pullout** |
| 0.00 (ft) | 1.70 | 1.70 |
| 20.00 (ft) | 1.20 | 1.20 |

| Pullout Friction Factors | |
|---|---|
| **Depth (Z)** | **F*** |
| 0.00 (ft) | 3.00 |
| 20.00 (ft) | 0.67 |

**General Notes**

1. AASHTO LRFD Bridge Design Specification 8th Edition/2017
2. FHWA NHI-10-024
3. IDEA
4. LEVEL BACK SLOPE WITH TRAFFIC SURCHARGE
5. EARTHTRAC SOIL REINFORCING

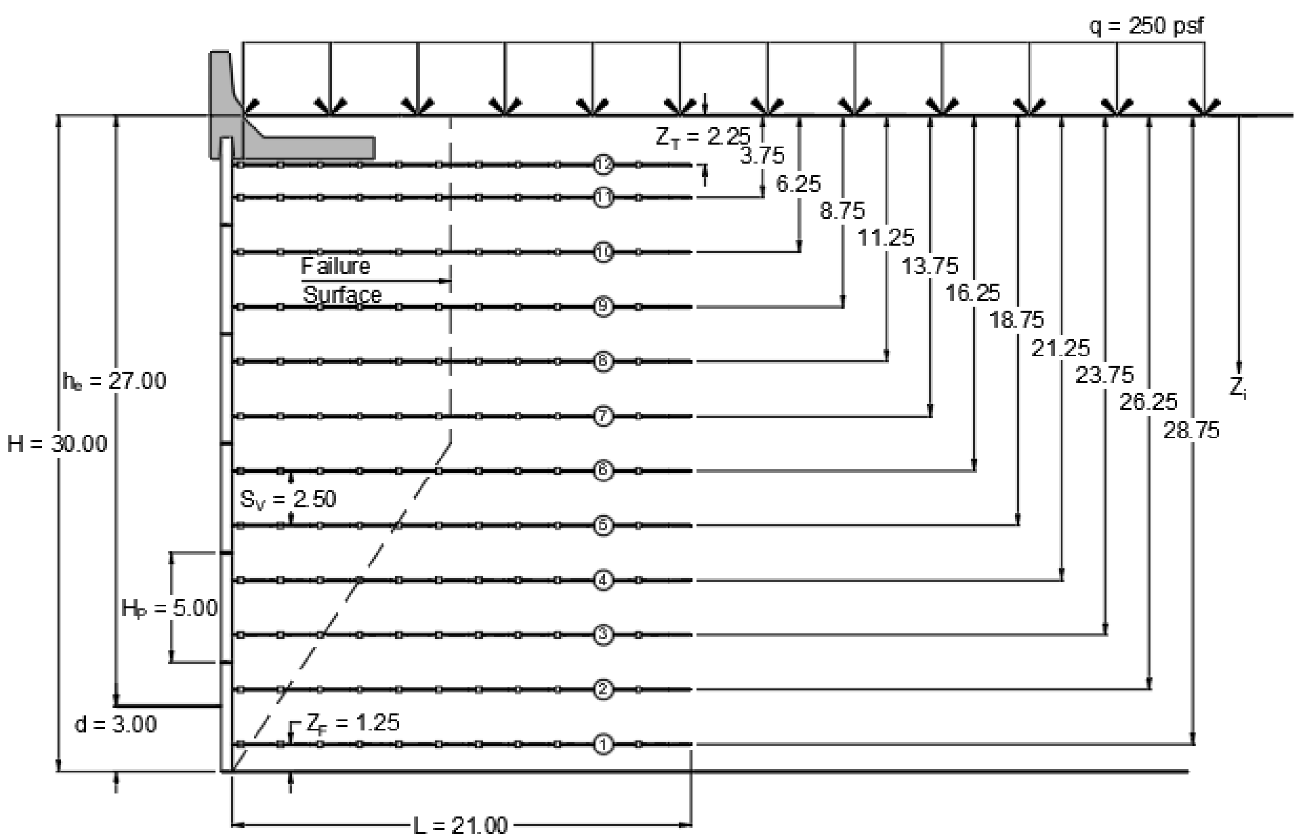
**System Design Parameters**

Steel Yield Stress: 50.000 (ksi)
Yield Coefficient: 0.750
Design Life: 75.0 years

| **Soil Reinforcing Schedule – EarthTrac SS3 Steel Strip Soil Reinforcing** | | | | |
|---|---|---|---|---|
| **Type** | **Min Sr Per Row** | **Thickness (in)** | **Width (in)** | **Design Steel Area** |
| **1** | **3** | **0.156** | **2.000** | **0.202** |

| **Soil Parameters** | | |
|---|---|---|
| **Soil** | **Unit Weight (kcf)** | **Friction Angle (deg)** |
| MSE Backfill | 0.125 | 34 |
| Retained Backfill | 0.125 | 34 |
| Foundation Backfill | 0.125 | 30 |

| **Rec. #1 Structure Parameters** | | | | | | |
|---|---|---|---|---|---|---|
| **H max (ft)** | **B min (ft)** | **B Ext. (ft)** | **S (ft)** | **Xs (ft)** | **β (deg)** | **βi (deg)** |
| 30.000 | 21.000 | 21.000 | 0.00 | 0.00 | 0.00 | 0.00 |




**Structure Parameters**

**External And Internal Stability Calculation Summary**

| Rec. #1 External Stability CDR Summary (Effective Stress) | | | | | |
|---|---|---|---|---|---|
| | CDR Sliding | Limiting Eccentricity (ft) | CDR Overturning | Bearing Pressure (ksf) | Eccentricity (ft) |
| Strength 1 - Min | 2.52 | 2.52 | 4.16 | 5.34 | 2.26 |
| Strength 1 - Max | 2.23 | 2.77 | 3.79 | 7.26 | 2.55 |
| Strength 1 - Critical | 1.65 | 3.74 | 2.81 | 6.15 | 3.35 |
| Service | 2.52 | 2.42 | 4.33 | 5.10 | 2.27 |

| Rec. #1 Internal Stability CDR Summary | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Depth Below Top of Wall (ft) | B min (ft) | Type | Strips | Strip Thickness (in) | Internal Earth Coeff. | Rupture CDR | Pullout CDR | F* | Le (ft) | H/B Ratio |
| 2.25 | 21.00 | Type-1 | 3 | 0.156 | 0.46 | 2.27 | 1.57 | 2.74 | 12.00 | 0.70 |
| 3.75 | 21.00 | Type-1 | 3 | 0.156 | 0.45 | 2.57 | 2.26 | 2.56 | 12.00 | 0.70 |
| 6.25 | 21.00 | Type-1 | 3 | 0.156 | 0.44 | 1.49 | 1.67 | 2.27 | 12.00 | 0.70 |
| 8.75 | 21.00 | Type-1 | 3 | 0.156 | 0.42 | 1.19 | 1.51 | 1.98 | 12.00 | 0.70 |
| 11.25 | 21.00 | Type-1 | 3 | 0.156 | 0.40 | 1.01 | 1.35 | 1.69 | 12.00 | 0.70 |
| 13.75 | 21.00 | Type-1 | 4 | 0.156 | 0.38 | 1.19 | 1.56 | 1.40 | 12.00 | 0.70 |
| 16.25 | 21.00 | Type-1 | 4 | 0.156 | 0.37 | 1.07 | 1.37 | 1.11 | 12.75 | 0.70 |
| 18.75 | 21.00 | Type-1 | 5 | 0.156 | 0.35 | 1.24 | 1.48 | 0.82 | 14.25 | 0.70 |
| 21.25 | 21.00 | Type-1 | 5 | 0.156 | 0.34 | 1.13 | 1.38 | 0.67 | 15.75 | 0.70 |
| 23.75 | 21.00 | Type-1 | 5 | 0.156 | 0.34 | 1.02 | 1.51 | 0.67 | 17.25 | 0.70 |
| 26.25 | 21.00 | Type-1 | 6 | 0.156 | 0.34 | 1.12 | 1.97 | 0.67 | 18.75 | 0.70 |
| 28.75 | 21.00 | Type-1 | 6 | 0.156 | 0.34 | 1.03 | 2.13 | 0.67 | 20.25 | 0.70 |

114 South Collins Street
Arlington, TX, 76034
817-223-0969

# Appendix B

## MSEW Output

# AASHTO 2007-2010 (LRFD)
# Design Case - Horizontal Back-Slope

MSEW(3.0): Update # 14.981

## PROJECT IDENTIFICATION

| | |
|---|---|
| Title: | Design Case - Horizontal Back-Slope |
| Project Number: | EarthTrac-SS3 |
| Client: | LRFD DOT Binder - Horizontal Back-Slope |
| Designer: | Thomas P Taylor, PhD, PE, DGE |
| Station Number: | 1 |

### Description:

LRFD Verification External Stability LL=1.75 / Internal Stability LL=EV=1.35

### Company's information:

Name: Ground Improvement Systems LLC
Street: 114 South Collins Street

Arlington , TX 76011
Telephone #: 817.223.0969
Fax #:
E-Mail: tptaylor.pe@gmail.com

**Original file path and name:** H:\Toms Computer\AB My Data\DOT_AB LRFD Binder\1. Eart..... .....imple - Standard.BEN

**Original date and time of creating this file:** 20210313

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

# AASHTO 2007-2010 (LRFD)
# Design Case - Horizontal Back-Slope

MSEW(3.0): Update # 14.981

## PROJECT IDENTIFICATION

| | |
|---|---|
| Title: | Design Case - Horizontal Back-Slope |
| Project Number: | EarthTrac-SS3 |
| Client: | LRFD DOT Binder - Horizontal Back-Slope |
| Designer: | Thomas P Taylor, PhD, PE, DGE |
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Name: Ground Improvement Systems LLC
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Arlington , TX 76011
Telephone #: 817.223.0969
Fax #:
E-Mail: tptaylor.pe@gmail.com

**Original file path and name:** H:\Toms Computer\AB My Data\DOT_AB LRFD Binder\1. Eart.....
.....imple - Standard.BEN

**Original date and time of creating this file:** 20210313

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

## SOIL DATA

REINFORCED SOIL

| | |
|---|---|
| Unit weight, $\gamma$ | 125.0 lb/ft $^3$ |
| Design value of internal angle of friction, $\phi$ | 34.0 ° |

RETAINED SOIL

| | |
|---|---|
| Unit weight, $\gamma$ | 125.0 lb/ft $^3$ |
| Design value of internal angle of friction, $\phi$ | 34.0 ° |

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) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

## BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 30.14 $N\gamma$= 22.40

## SEISMICITY

Not Applicable

## INPUT DATA: Metal strips
## (Analysis)

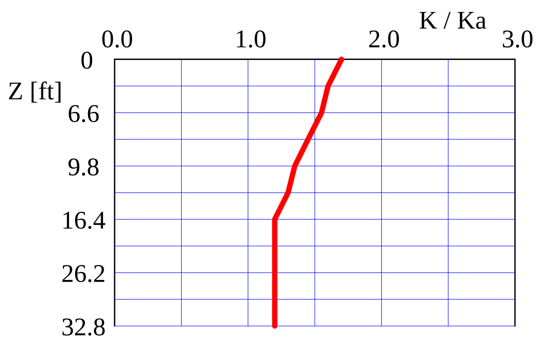
| D A T A | Metal strip type #1 | Metal strip type #2 | Metal strip type #3 | Metal strip type #4 | Metal strip type #5 |
|---|---|---|---|---|---|
| Yield strength of steel, Fy [kips/in ²] | 50.0 | N/A | N/A | N/A | N/A |
| Gross width of strip, b [in] | 2.0 | N/A | N/A | N/A | N/A |
| Vertical spacing, 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 | 60.97 | 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 | 3.00 | N/A | N/A | N/A | N/A |
| @ 19.7 ft or below | 1.00 | 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

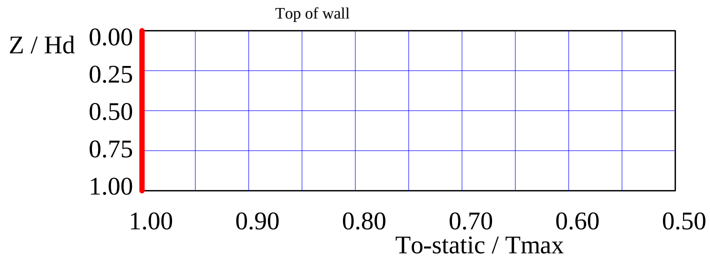
| Z | K / Ka |
|---|---|
| 0 ft | 1.70 |
| 3.3 ft | 1.60 |
| 6.6 ft | 1.55 |
| 9.8 ft | 1.45 |
| 13.1 ft | 1.35 |
| 16.4 ft | 1.30 |
| 19.7 ft | 1.20 |



## 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 $\gamma_f$ = 152.78 lb/ft ³

| Z / Hd | To-static / Tmax |
|---|---|
| 0.00 | 1.00 |
| 0.25 | 1.00 |
| 0.50 | 1.00 |
| 0.75 | 1.00 |
| 1.00 | 1.00 |




| D A T A (for connection only) | Type #1 | Type #2 | Type #3 | Type #4 | Type #5 |
|---|---|---|---|---|---|
| Product Name | EarthTrac.. | 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 |

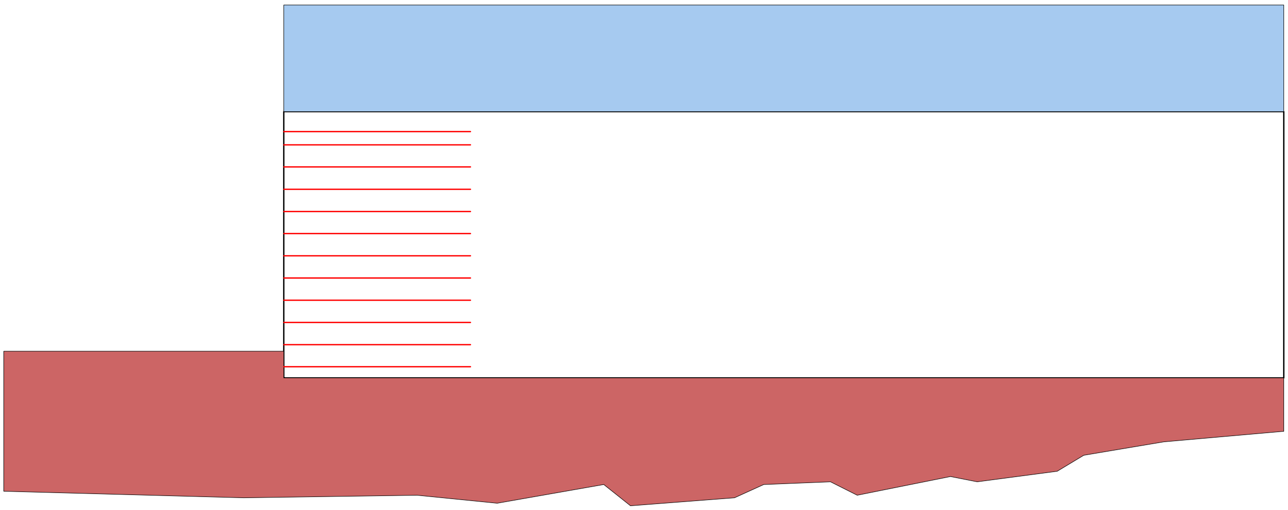
## INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

| | | | |
|---|---|---|---|
| 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 } |
| Batter, $\omega$ | 0.0 | [deg] | |
| Backslope, $\beta$ | 0.0 | [deg] | |
| Backslope rise | 0.0 | [ft] | Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82) |

U N I F O R M   S U R C H A R G E

Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 250.0 [lb/ft ²]

## ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 2 4 6 8 10[ft]

## AASHTO 2007-2010 (LRFD) Input Data

### INTERNAL STABILITY

| | | | |
|---|---|---|---|
| Load factor for vertical earth pressure, EV, from Table 3.4.1-2: | $\gamma_{p\text{-}EV}$ | 1.35 | |
| Load factor for earthquake loads, EQ, from Table 3.4.1-1: | $\gamma_{p\text{-}EQ}$ | 1.00 | |
| Load factor for live load surchrge, LS, from Figure C11.5.5-3(b): (Same as in External Stability). | $\gamma_{p\text{-}LS}$ | 1.75 | |
| Load factor for dead load surchrge, ES: (Same as in External Stability). | $\gamma_{p\text{-}ES}$ | 1.50 | |
| Resistance factor for reinforcement tension from Table 11.5.6-1: | $\phi$ | Static | Combined static/seismic |
| Metal Strips: | | 0.75 | 1.00 |
| Resistance factor for reinforcement tension in connectors from Table 11.5.6-1: | $\phi$ | Static | Combined static/seismic |
| Metal Strips: | | 0.75 | 1.00 |
| Resistance factor for reinforcement pullout from Table 11.5.6-1: | $\phi$ | 0.90 | 1.20 |

### EXTERNAL STABILITY

| | | | | |
|---|---|---|---|---|
| Load factor for vertical earth pressure, EV, from Table 3.4.1-2 and Figure C11.5.5-2: | | 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 |

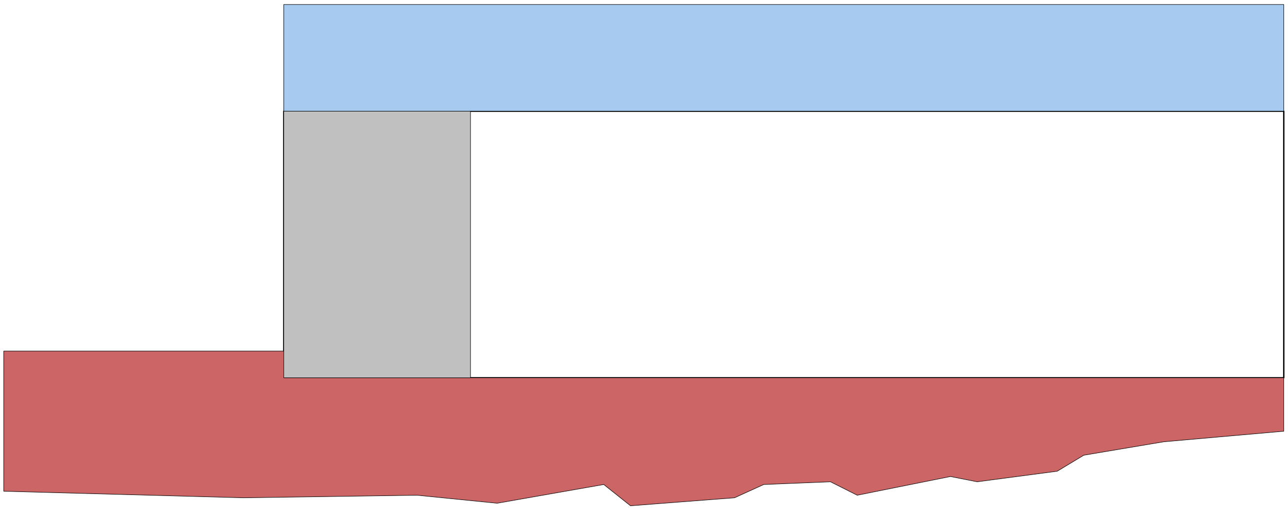
Load factor of active lateral earth pressure, EH, from Table 3.4.1-2 and Figure C11.5.5-2: $\gamma_{p\text{-}EH}$ 1.50
Load factor of active lateral earth pressure during earthquake (does not multiply $P_{AE}$ and $P_{IR}$ ): $(\gamma_{p\text{-}EH})_{EQ}$ 1.50
Load factor for earthquake loads, EQ, from Table 3.4.1-1 (multiplies $P_{AE}$ and $P_{IR}$ ): $\gamma_{p\text{-}EQ}$ 1.00

| | | | |
|---|---|---|---|
| Resistance factor for shear resistance along common interfaces from Table 11.5.6-1: | | Static | Combined Static/Seismic |
| Reinforced Soil and Foundation | $\phi_\tau$ | 1.00 | 1.00 |
| Reinforced Soil and Reinforcement | $\phi_\tau$ | 1.00 | 1.00 |
| Resistance factor for bearing capacity of shallow foundation from Table 11.5.6-1: | | Static | Combined Static/Seismic |
| | $\phi_b$ | 0.65 | 0.65 |

## BEARING CAPACITY for GIVEN LAYOUT

| | STATIC | SEISMIC | UNITS |
|---|---|---|---|
| (Water table does not affect bearing capacity) | | | |
| Factored bearing resistance. q-n | 14476 | N/A | [lb/ft ²] |
| Factored bearing load, $\sigma_V$ | 7261.6 | N/A | [lb/ft ²] |
| Eccentricity, e | 2.55 | N/A | [ft] |
| Eccentricity, e/L | 0.121 | N/A | |
| CDR calculated | 1.99 | N/A | |
| Base length | 21.00 | N/A | [ft] |

Unfactored applied bearing pressure = (Unfactored R) / [ L - 2 * (Unfactored e) ] =
Unfactored R = 83999.95 [lb/ft], L = 21.00, Unfactored e = 2.27 [ft], and Sigma = 5104.41 [lb/ft ²]



SCALE:

0 2 4 6 8 10[ft]

## DIRECT SLIDING for GIVEN LAYOUT (for METAL STRIPS reinforcements)

Along reinforced and foundation soils interface: CDR-static = 1.649

| # | Metal strip Elevation [ft] | Metal strip Length [ft] | CDR Static | CDR Seismic | Metal strip Type # | Product name |
|---|---|---|---|---|---|---|
| 1 | 1.25 | 21.00 | 1.999 | N/A | 1 | EarthTrac SS3 |
| 2 | 3.75 | 21.00 | 2.161 | N/A | 1 | EarthTrac SS3 |
| 3 | 6.25 | 21.00 | 2.351 | N/A | 1 | EarthTrac SS3 |
| 4 | 8.75 | 21.00 | 2.578 | N/A | 1 | EarthTrac SS3 |
| 5 | 11.25 | 21.00 | 2.853 | N/A | 1 | EarthTrac SS3 |
| 6 | 13.75 | 21.00 | 3.194 | N/A | 1 | EarthTrac SS3 |
| 7 | 16.25 | 21.00 | 3.627 | N/A | 1 | EarthTrac SS3 |
| 8 | 18.75 | 21.00 | 4.197 | N/A | 1 | EarthTrac SS3 |
| 9 | 21.25 | 21.00 | 4.979 | N/A | 1 | EarthTrac SS3 |
| 10 | 23.75 | 21.00 | 6.119 | N/A | 1 | EarthTrac SS3 |
| 11 | 26.25 | 21.00 | 7.937 | N/A | 1 | EarthTrac SS3 |
| 12 | 27.75 | 21.00 | 9.658 | N/A | 1 | EarthTrac SS3 |

## ECCENTRICITY for GIVEN LAYOUT (for Simplified Method)

At interface with foundation: e/L static = 0.1779; Overturning: CDR-static = 2.81

| # | Metal strip Elevation [ft] | Metal strip Length [ft] | e / L Static | e / L Seismic | Metal strip Type # | Product name |
|---|---|---|---|---|---|---|
| 1 | 1.25 | 21.00 | 0.1647 | N/A | 1 | EarthTrac SS3 |
| 2 | 3.75 | 21.00 | 0.1399 | N/A | 1 | EarthTrac SS3 |
| 3 | 6.25 | 21.00 | 0.1170 | N/A | 1 | EarthTrac SS3 |
| 4 | 8.75 | 21.00 | 0.0962 | N/A | 1 | EarthTrac SS3 |
| 5 | 11.25 | 21.00 | 0.0774 | N/A | 1 | EarthTrac SS3 |
| 6 | 13.75 | 21.00 | 0.0606 | N/A | 1 | EarthTrac SS3 |
| 7 | 16.25 | 21.00 | 0.0457 | N/A | 1 | EarthTrac SS3 |
| 8 | 18.75 | 21.00 | 0.0329 | N/A | 1 | EarthTrac SS3 |
| 9 | 21.25 | 21.00 | 0.0221 | N/A | 1 | EarthTrac SS3 |
| 10 | 23.75 | 21.00 | 0.0133 | N/A | 1 | EarthTrac SS3 |
| 11 | 26.25 | 21.00 | 0.0065 | N/A | 1 | EarthTrac SS3 |
| 12 | 27.75 | 21.00 | 0.0033 | N/A | 1 | EarthTrac SS3 |

# AASHTO 2007-2010 (LRFD)
# Design Case - Horizontal Back-Slope

MSEW(3.0): Update # 14.981

## PROJECT IDENTIFICATION

| | |
|---|---|
| Title: | Design Case - Horizontal Back-Slope |
| Project Number: | EarthTrac-SS3 |
| Client: | LRFD DOT Binder - Horizontal Back-Slope |
| Designer: | Thomas P Taylor, PhD, PE, DGE |
| Station Number: | 1 |

### Description:

LRFD Verification External Stability LL=1.75 / Internal Stability LL=EV=1.35

### Company's information:

Name: Ground Improvement Systems LLC
Street: 114 South Collins Street

Arlington , TX 76011
Telephone #: 817.223.0969
Fax #:
E-Mail: tptaylor.pe@gmail.com

**Original file path and name:** H:\Toms Computer\AB My Data\DOT_AB LRFD Binder\1. Eart..... .....imple - Standard.BEN

**Original date and time of creating this file:** 20210313

**PROGRAM MODE:** ANALYSIS of a SIMPLE STRUCTURE using METAL STRIPS as reinforcing material.

## SOIL DATA

REINFORCED SOIL

| | |
|---|---|
| Unit weight, $\gamma$ | 125.0 lb/ft $^3$ |
| Design value of internal angle of friction, $\phi$ | 34.0 ° |

RETAINED SOIL

| | |
|---|---|
| Unit weight, $\gamma$ | 125.0 lb/ft $^3$ |
| Design value of internal angle of friction, $\phi$ | 34.0 ° |

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) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

## BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 30.14 N $\gamma$= 22.40

## SEISMICITY

Not Applicable

## INPUT DATA: Metal strips
## (Analysis)

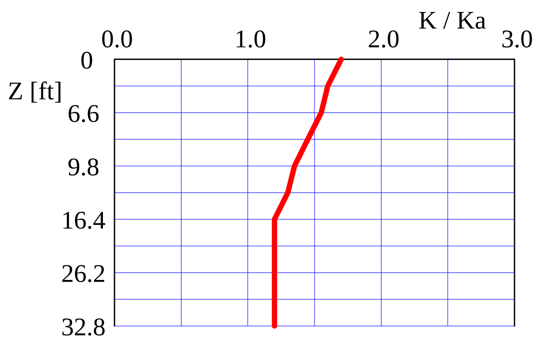
| D A T A | Metal strip type #1 | Metal strip type #2 | Metal strip type #3 | Metal strip type #4 | Metal strip type #5 |
|---|---|---|---|---|---|
| Yield strength of steel, Fy [kips/in ²] | 50.0 | N/A | N/A | N/A | N/A |
| Gross width of strip, b [in] | 2.0 | N/A | N/A | N/A | N/A |
| Vertical spacing, 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 | 60.97 | 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 | 3.00 | N/A | N/A | N/A | N/A |
| @ 19.7 ft or below | 1.00 | 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

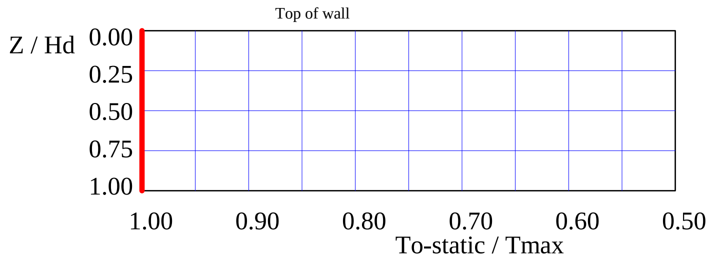
| Z | K / Ka |
|---|---|
| 0 ft | 1.70 |
| 3.3 ft | 1.60 |
| 6.6 ft | 1.55 |
| 9.8 ft | 1.45 |
| 13.1 ft | 1.35 |
| 16.4 ft | 1.30 |
| 19.7 ft | 1.20 |



## 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 $\gamma_f$ = 152.78 lb/ft ³

| Z / Hd | To-static / Tmax |
|---|---|
| 0.00 | 1.00 |
| 0.25 | 1.00 |
| 0.50 | 1.00 |
| 0.75 | 1.00 |
| 1.00 | 1.00 |




| D A T A (for connection only) | Type #1 | Type #2 | Type #3 | Type #4 | Type #5 |
|---|---|---|---|---|---|
| Product Name | EarthTrac.. | 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 |

## 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 } |
| Batter, ω | 0.0 | [deg] | |
| Backslope, β | 0.0 | [deg] | |
| Backslope rise | 0.0 | [ft] | Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82) |

U N I F O R M　S U R C H A R G E

Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 250.0 [lb/ft ²]

## ANALYZED REINFORCEMENT LAYOUT:

SCALE:

0 2 4 6 8 10[ft]

## AASHTO 2007-2010 (LRFD) Input Data

### INTERNAL STABILITY

| | | | |
|---|---|---|---|
| Load factor for vertical earth pressure, EV, from Table 3.4.1-2: | $\gamma_{p\text{-}EV}$ | 1.35 | |
| Load factor for earthquake loads, EQ, from Table 3.4.1-1: | $\gamma_{p\text{-}EQ}$ | 1.00 | |
| Load factor for live load surchrge, LS, from Figure C11.5.5-3(b): (Same as in External Stability). | $\gamma_{p\text{-}LS}$ | 1.35 | |
| Load factor for dead load surchrge, ES: (Same as in External Stability). | $\gamma_{p\text{-}ES}$ | 1.35 | |
| Resistance factor for reinforcement tension from Table 11.5.6-1: | $\phi$ | Static | Combined static/seismic |
| Metal Strips: | | 0.75 | 1.00 |
| Resistance factor for reinforcement tension in connectors from Table 11.5.6-1: | $\phi$ | Static | Combined static/seismic |
| Metal Strips: | | 0.75 | 1.00 |
| Resistance factor for reinforcement pullout from Table 11.5.6-1: | $\phi$ | 0.90 | 1.20 |

### EXTERNAL STABILITY

| | | | | |
|---|---|---|---|---|
| Load factor for vertical earth pressure, EV, from Table 3.4.1-2 and Figure C11.5.5-2: | | 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, from Table 3.4.1-2 and Figure C11.5.5-2: | $\gamma_{p\text{-}EH}$ | 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, from Table 3.4.1-1 (multiplies $P_{AE}$ and $P_{IR}$ ): | $\gamma_{p\text{-}EQ}$ | 1.00 |

| | | | |
|---|---|---|---|
| Resistance factor for shear resistance along common interfaces from Table 11.5.6-1: | | Static | Combined Static/Seismic |
| Reinforced Soil and Foundation | $\phi_{\tau}$ | 1.00 | 1.00 |
| Reinforced Soil and Reinforcement | $\phi_{\tau}$ | 1.00 | 1.00 |
| Resistance factor for bearing capacity of shallow foundation from Table 11.5.6-1: | | Static | Combined Static/Seismic |
| | $\phi_b$ | 0.65 | 0.65 |

**RESULTS for STRENGTH** [ Note: Actual CDR = (Yield stress) / (Actual stress) ] For Simplified Method
Live Load included in calculating Tmax

| # | Metal strip Elevation [ft] | Coverage ratio, Rc=b/Sh | Horizontal spacing, Sh [ft] | Long-term strength 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 | 1.25 | 0.100 | 1.670 | 4513 | 4401.07 | N/A | N/A | 1.026 | N/A | N/A |
| 2 | 3.75 | 0.100 | 1.670 | 4513 | 4043.26 | N/A | N/A | 1.116 | N/A | N/A |
| 3 | 6.25 | 0.083 | 2.000 | 3769 | 3685.45 | N/A | N/A | 1.023 | N/A | N/A |
| 4 | 8.75 | 0.083 | 2.000 | 3769 | 3327.64 | N/A | N/A | 1.133 | N/A | N/A |
| 5 | 11.25 | 0.083 | 2.000 | 3769 | 3047.28 | N/A | N/A | 1.237 | N/A | N/A |
| 6 | 13.75 | 0.067 | 2.500 | 3015 | 2812.54 | N/A | N/A | 1.072 | N/A | N/A |
| 7 | 16.25 | 0.067 | 2.500 | 3015 | 2523.42 | N/A | N/A | 1.195 | N/A | N/A |
| 8 | 18.75 | 0.050 | 3.330 | 2264 | 2218.00 | N/A | N/A | 1.021 | N/A | N/A |
| 9 | 21.25 | 0.050 | 3.330 | 2264 | 1896.14 | N/A | N/A | 1.194 | N/A | N/A |
| 10 | 23.75 | 0.050 | 3.330 | 2264 | 1518.90 | N/A | N/A | 1.490 | N/A | N/A |
| 11 | 26.25 | 0.050 | 3.330 | 2264 | 909.29 | N/A | N/A | 2.489 | N/A | N/A |
| 12 | 27.75 | 0.050 | 3.330 | 2264 | 818.87 | N/A | N/A | 2.764 | 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 | Tmax [lb/ft] | Tmd [lb/ft] | Le [ft] (see NOTE) | La [ft] (see NOTE) | 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 | 1.25 | 0.100 | 4114.8 | N/A | 20.25 | 0.75 | 13073.1 | N/A | 3.177 | N/A | N/A | N/A |
| 2 | 3.75 | 0.100 | 3757.0 | N/A | 18.75 | 2.25 | 11052.1 | N/A | 2.942 | N/A | N/A | N/A |
| 3 | 6.25 | 0.083 | 3399.2 | N/A | 17.25 | 3.75 | 7681.7 | N/A | 2.260 | N/A | N/A | N/A |
| 4 | 8.75 | 0.083 | 3041.4 | N/A | 15.75 | 5.25 | 6275.4 | N/A | 2.063 | N/A | N/A | N/A |
| 5 | 11.25 | 0.083 | 2753.1 | N/A | 14.25 | 6.75 | 5485.7 | N/A | 1.993 | N/A | N/A | N/A |
| 6 | 13.75 | 0.067 | 2503.9 | N/A | 12.75 | 8.25 | 4189.4 | N/A | 1.673 | N/A | N/A | N/A |
| 7 | 16.25 | 0.067 | 2202.5 | N/A | 12.00 | 9.00 | 3964.3 | N/A | 1.800 | N/A | N/A | N/A |
| 8 | 18.75 | 0.050 | 1882.4 | N/A | 12.00 | 9.00 | 2820.9 | N/A | 1.499 | N/A | N/A | N/A |
| 9 | 21.25 | 0.050 | 1542.3 | N/A | 12.00 | 9.00 | 2494.2 | N/A | 1.617 | N/A | N/A | N/A |
| 10 | 23.75 | 0.050 | 1149.7 | N/A | 12.00 | 9.00 | 1995.9 | N/A | 1.736 | N/A | N/A | N/A |
| 11 | 26.25 | 0.050 | 605.6 | N/A | 12.00 | 9.00 | 1326.2 | N/A | 2.190 | N/A | N/A | N/A |
| 12 | 27.75 | 0.050 | 345.3 | N/A | 12.00 | 9.00 | 842.0 | N/A | 2.438 | N/A | N/A | N/A |

## 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 Specified | CDR connection break Actual | CDR Metal strip strength Specified | CDR Metal strip strength Actual | Product name |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 1.25 | 0.100 | 1.670 | 4401 | 1.00 | 4513 | 4513 | N/A | 1.03 | N/A | 1.03 | EarthTrac .. |
| 2 | 3.75 | 0.100 | 1.670 | 4043 | 1.00 | 4513 | 4513 | N/A | 1.12 | N/A | 1.12 | EarthTrac .. |
| 3 | 6.25 | 0.083 | 2.000 | 3685 | 1.00 | 3769 | 3769 | N/A | 1.02 | N/A | 1.02 | EarthTrac .. |
| 4 | 8.75 | 0.083 | 2.000 | 3328 | 1.00 | 3769 | 3769 | N/A | 1.13 | N/A | 1.13 | EarthTrac .. |
| 5 | 11.25 | 0.083 | 2.000 | 3047 | 1.00 | 3769 | 3769 | N/A | 1.24 | N/A | 1.24 | EarthTrac .. |
| 6 | 13.75 | 0.067 | 2.500 | 2813 | 1.00 | 3015 | 3015 | N/A | 1.07 | N/A | 1.07 | EarthTrac .. |
| 7 | 16.25 | 0.067 | 2.500 | 2523 | 1.00 | 3015 | 3015 | N/A | 1.19 | N/A | 1.19 | EarthTrac .. |
| 8 | 18.75 | 0.050 | 3.330 | 2218 | 1.00 | 2264 | 2264 | N/A | 1.02 | N/A | 1.02 | EarthTrac .. |
| 9 | 21.25 | 0.050 | 3.330 | 1896 | 1.00 | 2264 | 2264 | N/A | 1.19 | N/A | 1.19 | EarthTrac .. |
| 10 | 23.75 | 0.050 | 3.330 | 1519 | 1.00 | 2264 | 2264 | N/A | 1.49 | N/A | 1.49 | EarthTrac .. |
| 11 | 26.25 | 0.050 | 3.330 | 909 | 1.00 | 2264 | 2264 | N/A | 2.49 | N/A | 2.49 | EarthTrac .. |
| 12 | 27.75 | 0.050 | 3.330 | 819 | 1.00 | 2264 | 2264 | N/A | 2.76 | N/A | 2.76 | EarthTrac .. |

**2.2.2**

**2:1 Backslope Design Example**

GROUND IMPROVEMENT SYSTEMS LLC
114 SOUTH COLLINS STREET
ARLINGTON, TX 76011

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# MSE DESIGN

## SIMPLIFIED METHOD

### INFINITE BACK-SLOPE

## ENGINEERING REPORT

GROUND IMPROVEMENT SYSTEMS LLC
114 South Collins Street
Arlington, Texas 76011
www.groundimprovementsystems.com

## Table of Contents

## Table of Figures



## Tables

**Appendix**

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**Introduction**

This example demonstrates the analysis of an MSE structure with an infinite back-slope with no live load surcharge for a 75 year design life. The MSE structure is assumed to include a segmental concrete panel face utilizing the EarthTrac soil reinforcing. The configuration that will be analyzed is shown in Figure 1. The analysis is based on principles that are discussed in the FHWA-NHI-10-043 *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slope*s and conforms to the AASHTO LRFD Bridge Design Specification 7th Edition 2014.

The following is a summary of the design steps that are used in the analysis. Each of the steps and any sub-steps are sequential. Based on the sequential nature of the design steps if the design is revised at any step or sub-step then all the previous calculations will need to be revised accordingly. Each of the steps and the sub-steps are explained in detail in this document.

**Design Steps**

1 Establish Project Requirements

2 Define Project Parameters

   2.1 Define Structure Parameters

   2.2 Define External Live Load parameters

   2.3 Define Facing Parameters

   2.4 Define Soil Reinforcing Parameters

   2.5 Define Backfill Parameters

3 Evaluate Load and Resistance Parameters

   3.1 Load Factors

   3.2 Resistance Factors

4 Define Steel Corrosion Rates

5 Define Internal Stability Factors

5.1 Define Earth Pressure Factor

5.2 Define Pullout Friction Factor

6 Calculate Unfactored Loads for External Stability

7 Evaluate External Stability

7.1 Calculate Sliding Resistance at Base of MSE Structure

7.2 Calculate Limiting Eccentricity at Base of MSE Structure

7.3 Calculate Bearing Resistance at Base of MSE Structure

7.4 Summarize External Stability CDR's and Bearing Resistance

8 Evaluate Internal Stability

8.1 Determine Soil Reinforcing Depths and Tributary Area for Internal Stability

8.2 Determine Variation of $K_r$, F*and Length of Embedment for Internal Stability

8.3 Calculate Horizontal Stress at Elevation of Each Soil Reinforcing Element

8.4 Calculate Maximum Tension at Elevation of Each Soil Reinforcing Element

8.5 Determine Required Area of Steel at Elevation of Each Soil Reinforcing Element

8.6 Calculate Factored Pullout Resistance

8.7 Summarize Structure Configuration and CDR's for Pullout and Rupture

Table 1-1 through Table 5-1 will define the parameters that are required as input. If the value in the table contains a light colored box around it, it is a required input parameter. If there is no light colored box, the value will be calculated and is a function of the variables defined in the appropriate equation.

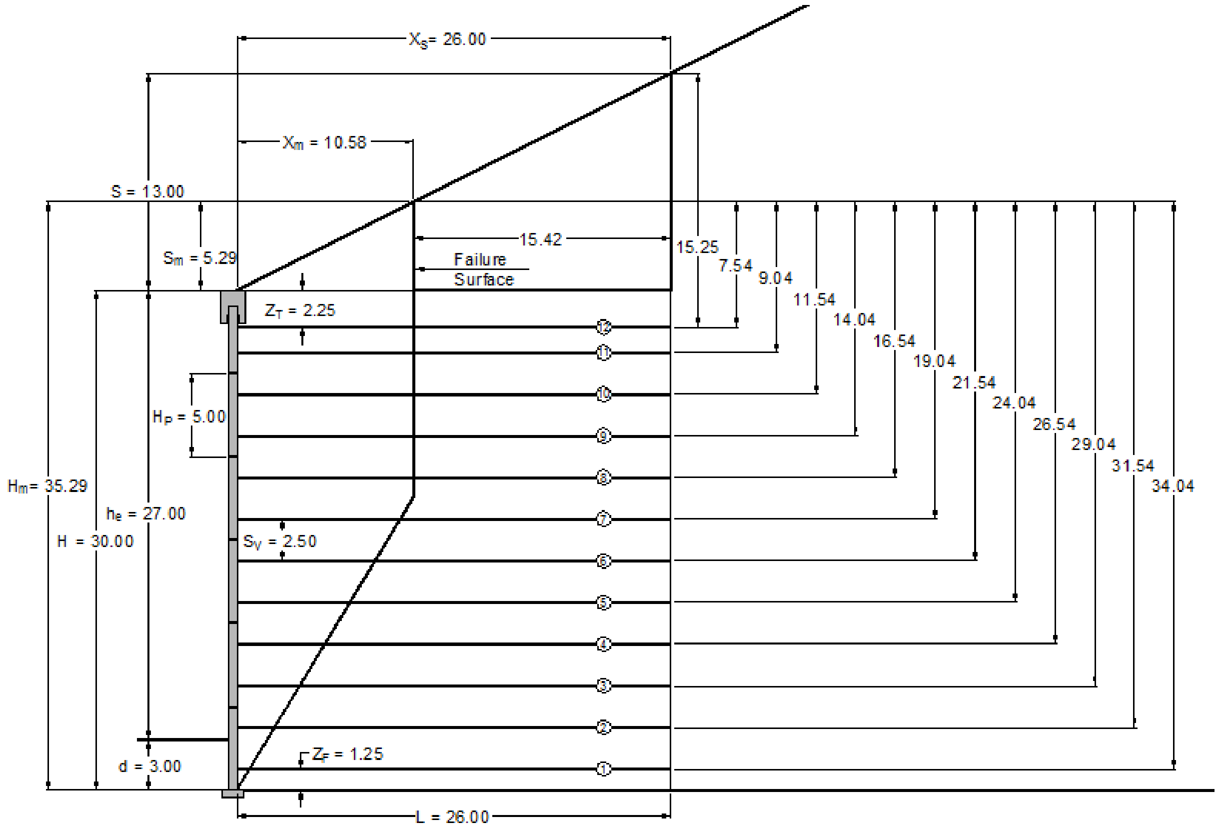



**Figure 1 Example Problem Cross Section**

## 1 Establish Project Requirements

The project requirements are typically established by the Engineer of Record (EOR). A general configuration of the MSE structure that is being designed in this document is shown in Figure 1-1. The exposed structure height ($h_e$) is the distance from the finish grade to the top of the structure. This should not be confused with the design structure height (H) that is used in the calculations. The design structure height is equal to the exposed structure height plus the structure embedment distance (d). Therefore, the design structure height is equal to the distance from the top of the leveling pad to the top of the structure. For this example the top of the structure is the top of the coping element. The structure embedment depth is a function of the project requirements and conforms to AASHTO Article 11.10.2.2. Any passive resistance that is provided by the embedment at the front face of the structure is not included in the calculations.

The soil reinforcing length aspect ratio is the ratio of the minimum length of soil reinforcing (L) to the design height of the structure (H). The minimum length of soil reinforcing or aspect ratio (L:H) is typically provided by the EOR as a result of geotechnical investigation, analysis and recommendations. The minimum length of soil reinforcing shall conform to AASHTO Article 11.10.2.1. For this example the minimum length of soil reinforcing is required to be a minimum of 70% of the structures design height to begin the calculation process. It is increased to a length that satisfies external and internal stability requirements. The Project Requirements are defined in Table 1-1 for the structure analyzed in this document.

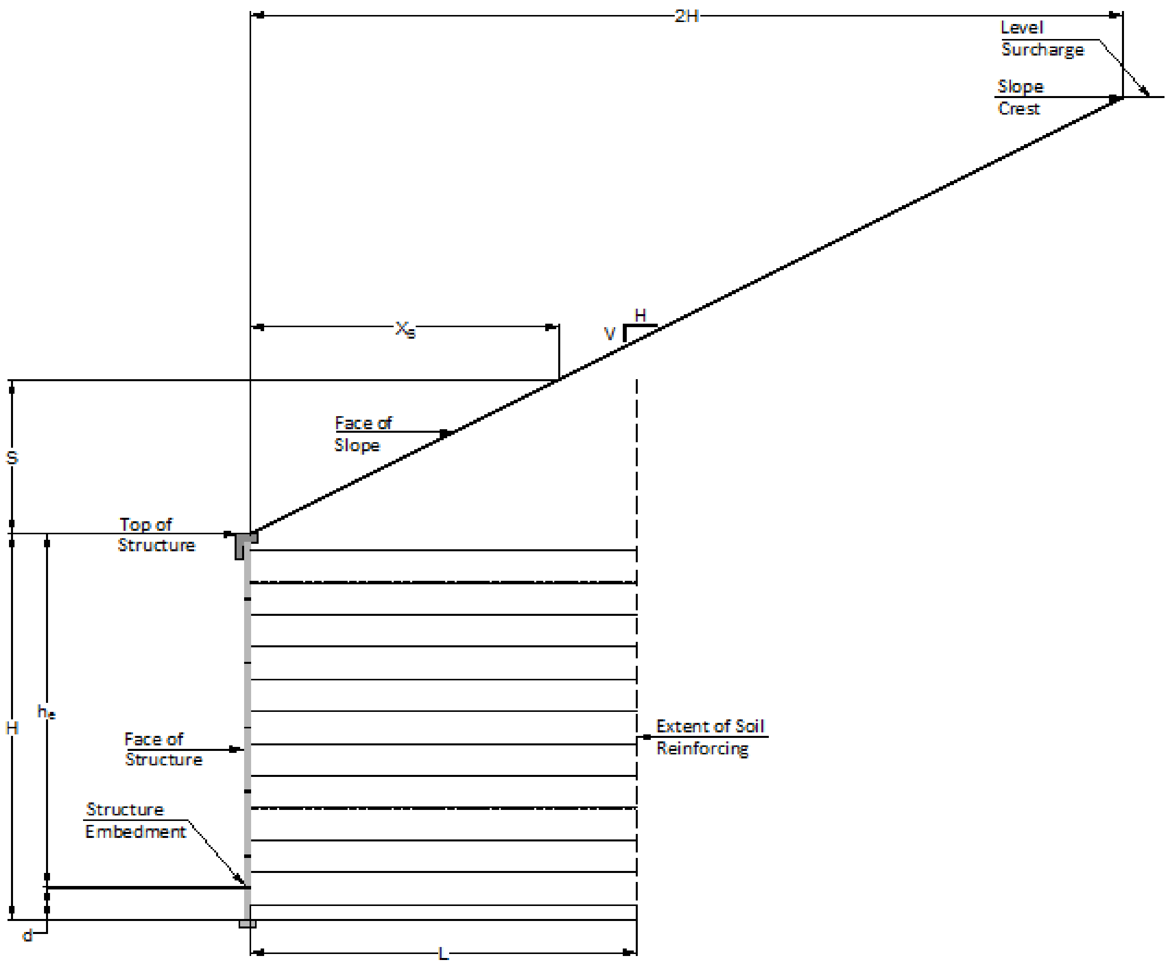



**Figure 1-1 Structure Project Requirements**

Table 1-1 Project Requirements

| Location | Variable | Value | Unit |
|---|---|---|---|
| • Exposed structure height | $h_e$ | 27.00 | ft |
| • Structure embedment distance | d | 3.00 | ft |
| • Soil reinforcing length aspect ratio | ratio | 0.86 | dim |
| • Length of structure | $L_w$ | 1000.00 | ft |
| • Design life | Life | 75.00 | yrs |
| • Top of structure slope condition | ○ Horizontal ○ Broken ◉ Infinite | | |
| • Facing | Precast Panel– 5 ft wide x 5 ft tall by 0.5 ft thick | | |
| • Type of Soil Reinforcement | ▪ EarthTrac<br>▪ Grade 50 steel with area equal to 0.3125 $in^2$<br>▪ Galvanized – 3.4 mil zinc coating | | |
| • Is seismic to be considered | No | | |

## 2 Define Project Parameters

The project requirements provide a general framework of the constraints for the MSE structure analyzed in this calculation. A comprehensive set of parameters is provided in Table 2-1 and Table 2-2 and are detailed in Figure 2-1. For the MSE structure evaluated in this calculation the back-slope at the top of the structure is infinite and the crest is located at a minimum distance (e.g. typically calculated as being equal to twice the structure height) where the externally applied live load surcharge consisting of standard highway traffic does not influence the external or internal stability of the structure. If the traffic live load surcharge falls within the Rankine failure surface at the back of the MSE structure than the designer should evaluate the effect that the load has on the stability of the structure.

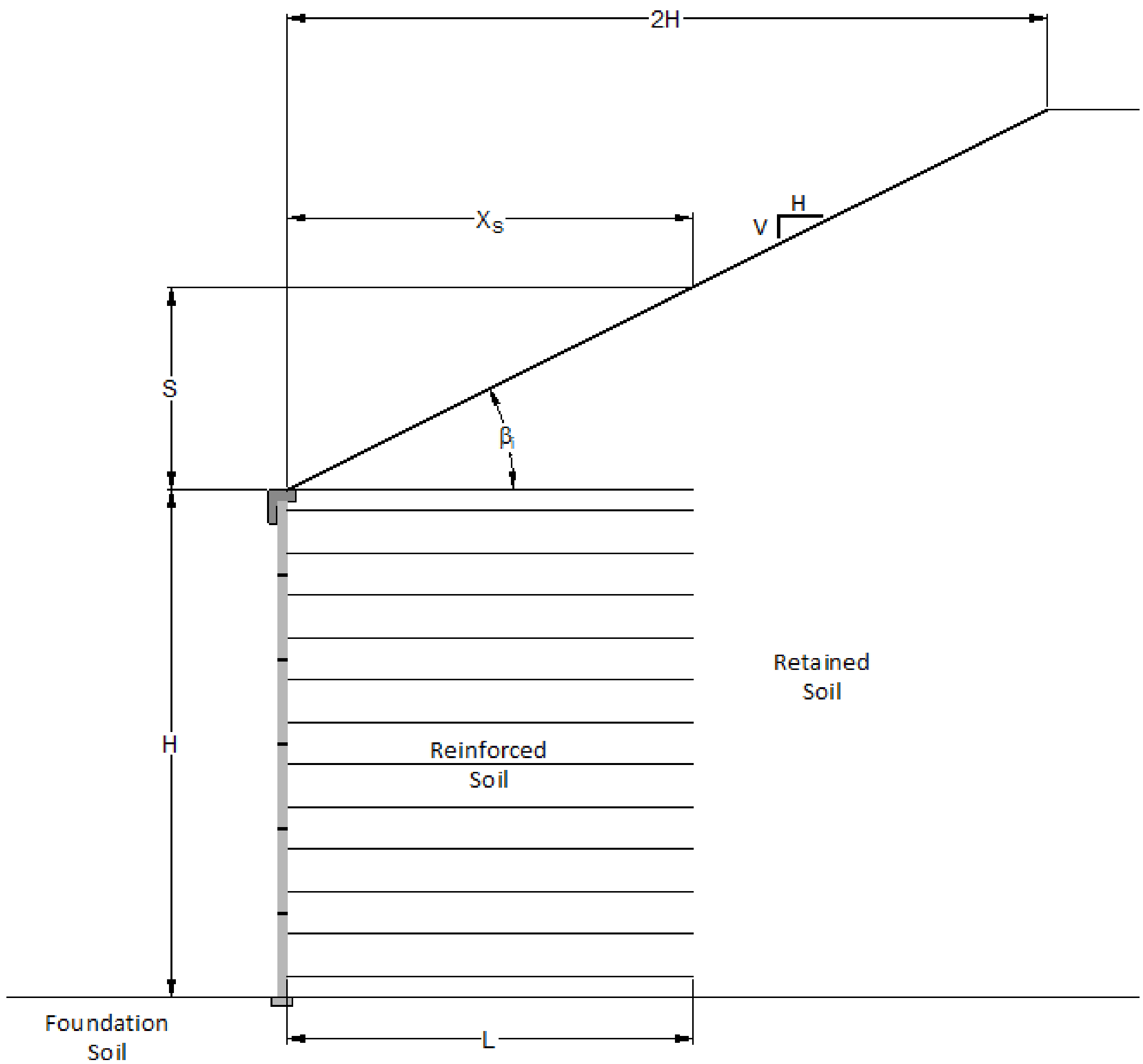



**Figure 2-1 Structure Project Requirements**

## 2.1 Define Structure Parameters

The back-slope at the top of the MSE structure is infinite with a slope angle (β) and a soil surcharge height (S) with the distance to the crest of the soil surcharge equal to the value 2H. The design height is calculated using Equation 2-1.

$H = h_e + d$ Equation 2-1

Table 2-1 Structure Parameters

| Location | Variable | Value | Unit |
|---|---|---|---|
| Structure Design Height | H | 30.00 | ft |
| Soil Reinforcing Length | L | 26.00 | ft |
| Surcharge Height | S | 30.00 | ft |
| Distance from face to Crest | $X_S$ | 60.00 | ft |
| Slope of Surcharge | $\beta$ | 26.565 | deg |
| Top of Structure Back-Slope Condition | | Infinite | |

## 2.2 Define External Live Load Parameters

When the live load surcharge is included it is based on the requirements in AASHTO Table 3.11.6.4-2. An infinite slope assumes the crest of the slope is a minimum distance equal to twice the structure design height (H) and therefore the live load surcharge will not influence the external stability of the structure because it is outside the Rankine failure surface. If the crest of the slope is within the distance of 2H, then the broken back-slope design method is used and the live load surcharge will have an influence on the external stability of the structure. In the broken back-slope analysis the equivalent soil height ($h_{eq}$) is equal to 2 feet and the unit weight ( $\gamma_q$ ) used to calculate the live load pressure (q) is set equal to 125 pcf (Table 2-2). When required the pressure applied at the top of the structure from the live load surcharge is calculated using Equation 2-2. AASHTO does not require the live load surcharge to be used in the internal stability pullout analysis for the calculation of $T_{max}$. When applied the live load is assumed to be applied over the entire earth surcharge (AASHTO 11.10.6.2.1).

$$q = \gamma_q \cdot h_{eq} \qquad \text{Equation 2-2}$$

Table 2-2 External Live Load Parameters

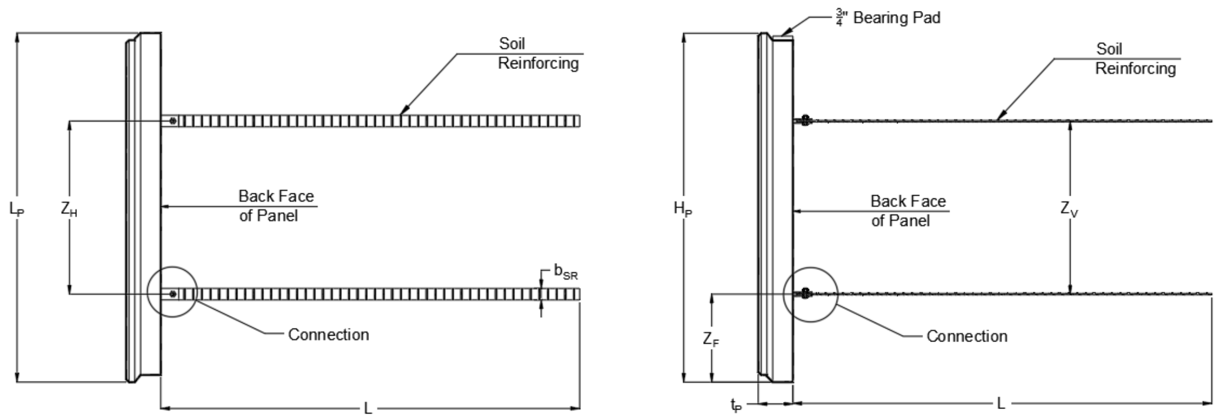
| Location | Variable | Value | Unit |
|---|---|---|---|
| Equivalent height of soil | $h_q$ | 0.00 | ft |
| Unit weight of surcharge | $\gamma_q$ | 0.00 | ft |
| Live load pressure | q | 0.00 | psf |

Table 2-2 External Live Load Parameters

| Location | Variable | Value | Unit |
|---|---|---|---|
| Include live load in calculating pullout | | ☐ Inlcude Live Load | |

## 2.3 Define Facing Parameters

For this calculation example the facing shall consist of a segmental concrete panel (SCP) with a height ($H_P$) equal to 5.00' and a length ($L_P$) equal to 5.00' (Figure 2-2 and Table 2-3). The thickness ($t_P$) of the panel is equal to a minimum 6". The vertical spacing ($S_V$) of the soil reinforcing is a consistent distance equal to 2.50' unless stipulated otherwise.




Plan View Cross Section

**Figure 2-2 Facing Parameters Detail**

The spacing at the top of the structure may vary and is dependent on the coping treatment. In this example the top of structure is assumed to have only a coping element. The minimum distance from the top of the coping to the top soil reinforcing element ($Z_T$) is equal to a value of 2.25'. The distance from the leveling pad to the first soil reinforcing element ($Z_F$) for the bottom panel is equal to 1.25'. When required the minimum distance between any two soil reinforcing elements ($Z_{min}$) shall be greater than or equal to 0.50'. The values, $S_V$, $Z_T$, $Z_F$, and $Z_{min}$ are required in order to set the depth to each soil reinforcing element. The maximum horizontal spacing ($S_H$) of the soil reinforcing shall be 2.50'. The horizontal spacing of the soil reinforcing is a function of the facing type and the soil reinforcing type that is being. It is also a function of the forces that are required to be resisted. The reader is advised to reference Figure 8-1 for details of the soil reinforcing configuration and coping configuration.

Table 2-3 Facing Parameters

| Location | Variable | Value | Unit |
|---|---|---|---|
| Panel Height | $H_P$ | 5.00 | ft |
| Panel Length | $L_P$ | 5.00 | ft |
| Vertical Spacing of Soil Reinforcing | $S_V$ | 2.50 | ft |
| Horizontal Spacing of Soil Reinforcing | $S_H$ | 2.50 | ft |
| Minimum Depth of Soil Reinforcing from Top of Structure | $Z_T$ | 2.25 | ft |
| Minimum soil reinforcing spacing | $S_{min}$ | 0.50 | ft |
| Distance from leveling pad to first soil reinforcing | $Z_F$ | 1.25 | ft |

## 2.4 Define Soil Reinforcing Parameters

The soil reinforcing parameters are a function of the type that is being used. For this example, inextensible steel EarthTrac SS3 soil reinforcing will be used ( Table 2-4 and Figure 2-3). The EarthTrac SS3 is a discrete steel strip with transverse ribs and a single point connector located at the proximal end. The EarthTrac SS3 has a width equal to 2-inches and a nominal thickness of 5/32-inches. Therefore, the EarthTrac SS3 initial steel area for one element is equal to 0.313 square inches. The EarthTrac SS3 is fabricated from Grade-50 steel with a yield strength equal to a minimum of 50 ksi. Based on the 5' x 5' SCP use, coupled with the minimum horizontal spacing, the minimum number of EarthTrac SS3 soil reinforcing elements (NSR) per row is set equal to 2. The actual number of EarthTrac SS3 for each panel is a function of the loading.

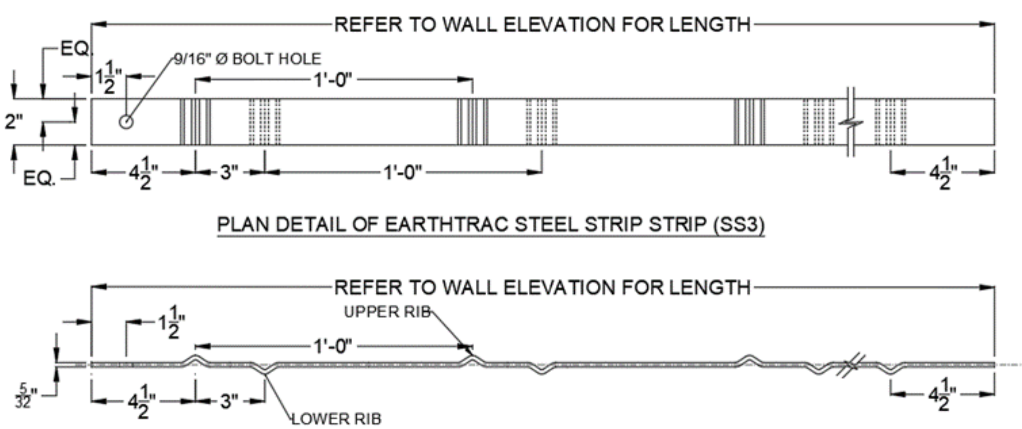


**Figure 2-3 EarthTrac SS1 Soil Reinforcing Parameters Detail**

Table 2-4 Soil Reinforcing Parameters

| Location | Variable | Value | Unit |
|---|---|---|---|
| Minimum number of soil reinforcing | $N_{SR}$ | 2 | dim |
| Soil reinforcing width | $b_w$ | 2.00 | in |
| Soil reinforcing thickness | $t_{ss}$ | 0.1563 | in |
| Soil reinforcing steel area | $A_S$ | 0.3126 | $in^2$ |
| Yield strength of steel | $F_y$ | 50.00 | ksi |

## 2.5 Define Backfill Parameters

The backfill is project specific (Table 2-5) and can vary significantly and is dependent on the geographical region and the project specifications. The reinforced backfill is defined as the volume of soil located at the back face of the facing element extending to the terminal end of the soil reinforcing and located from the top of the leveling pad extending to the top of the coping element. The retained backfill is defined as the soil mass located directly behind the reinforced soil mass at the terminal end of the soil reinforcing and from the top of the leveling pad extending to the top of the soil surcharge or pavement. The foundation soil is what the MSE structure bears on is defined as the volume of soil below the reinforced mass of soil and the retained mass of soil and (Figure 2-4). The foundation soil typically consists of in-situ material but may consist of material that has been replaced by the method of excavation. The frictional interface at the foundation is the minimum friction angle of the retained backfill and the foundation material. The earth surcharge is the volume of soil that is on top of the reinforced mass of soil and the retained mass of soil. The unit weight of the earth surcharge is required in the analysis. It can be equal to the retained backfill unit weight, the reinforced backfill unit weight, or a unit weight selected by the user. It is important that the friction angle of the material be greater than the slope angle to prevent local surcharge slope failures.

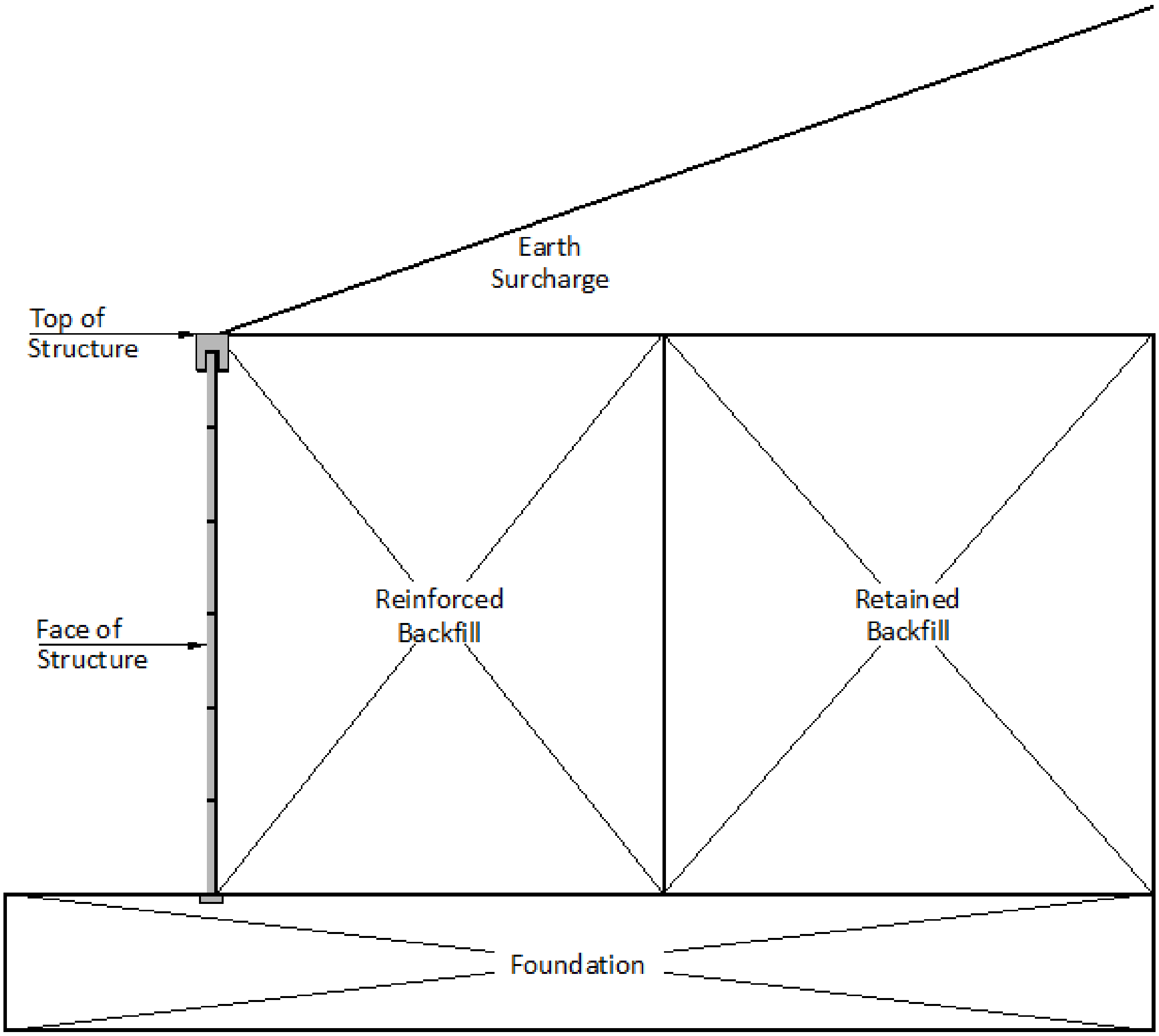



**Figure 2-4 Backfill Parameters**

Table 2-5 Backfill Parameters

| Location | Variable | Value | Unit |
|---|---|---|---|
| Unit weight of Reinforced Soil | $\gamma_r$ | 125.00 | psf |
| Friction Angle of Reinforced Soil | $\phi_r$ | 34.00 | deg |
| Unit weight of Retained Soil | $\gamma_f$ | 125.00 | psf |
| Friction Angle of Retained Soil | $\phi_f$ | 34.00 | deg |
| Unit weight of Foundation Soil | $\gamma_{fd}$ | 125.00 | psf |

Table 2-5 Backfill Parameters

| Location | | | Variable | Value | Unit |
|---|---|---|---|---|---|
| Friction Angle of Foundation Soil | | | $\phi_{fd}$ | 30.00 | deg |
| Unit weight of earth surcharge | | | $\gamma_{es}$ | 125.00 | pcf |
| ○ Retained Soil | ● Reinforced Soil | ○ User | | | |

The internal active earth pressure coefficients for the reinforced soil mass and the retained soil mass are calculated using Equation 2-3 and Equation 2-5. The equivalent slope angle is based on the AREMA Manual method of analysis and is calculated using Equation 2-4. For an infinite slope the slope angle is equal to the angle of the earth surcharge slope ( $\beta_i = \beta$ ).

$$K_{af} = \cos(\beta_i) \cdot \left[ \frac{\cos(\beta_i) - \sqrt{\cos(\beta_i)^2 - \cos(\phi_f)^2}}{\text{os}(\beta_i) + \sqrt{\cos(\beta_i)^2 - \cos(\phi_f)^2}} \right]$$ 0.406 (dim) Equation 2-3

$$\beta_i = a\tan\left(\frac{S}{2 \cdot H}\right)$$ 26.57 (deg) Equation 2-4

$$K_{ai} = \tan^2\left(45^\circ - \frac{\phi_r}{2}\right)$$ 0.283 (dim) Equation 2-5

$$K_{oi} = 1 - \sin(\phi_r)$$ 0.441 (dim) Equation 2-6

Where:

| | | | |
|---|---|---|---|
| Where: | $K_{af}$ | = | active earth pressure coefficient for external stability (dim) |
| | $\beta_i$ | = | the slope of the surcharge (deg) |
| | S | = | height of surcharge at distance 2H (ft) |
| | H | = | height of structure (ft) |
| | $K_{ai}$ | = | active earth pressure coefficient for internal stability (dim) |
| | $K_{oi}$ | = | at-rest earth pressure coefficient (dim) |

## 3 Evaluate Applicable Load and Resistance Factors

The load and resistance factors for a static condition conform to AASHTO Article 3-4, Table 3.4.1-1, Table 3.4.1-2 and Article 11.5, Table 11.5.7-1 and are as defined in Table 3-1 and Table 3-2.

Table 3-1 Load Factors (AASHTO 2014 Tables 3.4.1-1 and 3.4.1-2)

| Load Combination | Load Factors | | |
|---|---|---|---|
| | EV | EH | LS |
| Strength I (Max) | 1.35 | 1.50 | 1.75 |
| Strength I (Min) | 1.00 | 0.90 | 1.75 |
| Service I | 1.00 | 1.00 | 1.00 |

Table 3-2 Resistance Factors (AASHTO 2014 Table 11.5.6-1)

| Resistance Factor | Variable | Value | Unit |
|---|---|---|---|
| Sliding of MSE on foundation Soil | $\phi_s$ | 1.00 | dim |
| Bearing Resistance | $\phi_{bp}$ | 0.65 | dim |
| Tensile Resistance (single point connector) | $\phi_r$ | 0.75 | dim |
| Pullout Resistance | $\phi_{po}$ | 0.90 | dim |

### 3.1 Extreme Force Effect

The load combination that creates the most extreme force effect is required to be checked. For both external and internal stability three combinations of load factors are checked. The first combination, Strength I-Maximum, uses all the maximum load factors for both the vertical loads and the horizontal loads. The second combination, Strength I-Minimum, uses the minimum load factors for both the vertical loads and the horizontal loads. The third combination, Strength I-Critical, will utilize the critical values that are calculated in the Strength I Maximum and/or the Strength I Minimum load combinations. The critical values that produce the most extreme force effect are analyzed. For the sliding analysis the most extreme force effect typically occurs using the minimum vertical force and maximum horizontal force. For the limiting eccentricity analysis the most extreme force effect typically occurs using the minimum resisting moment and maximum overturning moment. For the bearing resistance analysis the most extreme force effect typically occurs in the Strength I Maximum load factor combination. However, a critical value check is made using the minimum vertical force with the minimum resisting moment and the maximum overturning moment. The most severe force effect is the minimum CDR for sliding analysis, the largest eccentricity in the Limiting Eccentricity analysis, and the maximum bearing stress in the Bearing Resistance analysis. It should be noted that the most severe force effect may not be the value displayed in “critical” check.

When an earth surcharge is applied at the top of the structure the horizontal force at the back of the MSE mass shall use consistent load factors in the critical check, e.g. if the horizontal force at the back of the retained mass of soil uses a maximum load factor then the complement vertical force will also use a maximum load factor. This will maintain consistency between the loads.

## 4 Define Steel Corrosion Rates

Steel that is buried in soil degrades over time. The amount of degradation is a function of the soil electro-chemical composition, steel protective coating and the length of time it is buried in soil. It will be assumed that the reinforced backfill meets the electrochemical properties specified in AASHTO Article 11.10.6.4.2a.

The protective coating for the steel will be zinc and shall be applied by the method of hot-dip galvanizing in conformance with ASTM A123. The thickness of the galvanized coating shall be a minimum of 3.4 mils. The corrosion rates are in conformance with AASHTO Article 11.10.6.4.2a as defined in Table 4-1 and the degraded thickness is calculated using Equation 4-1.

$$E_s = 2 \cdot \left( Life - \left( \frac{t_{galv} - t_{zinc1}}{t_{zinc2}} + 2 \cdot yr \right) \right) \cdot t_{steel} \qquad \text{Equation 4-1}$$

Table 4-1 AASHTO Corrosion Parameters

| Location | Variable | Value | Unit |
|---|---|---|---|
| Design Life | Life | 75.00 | yrs |
| Galvanizing thickness | $t_{Galv}$ | 3.40 | mil |
| Zinc Loss first Two years | $t_{zinc1}$ | 0.58 | mil |
| Zinc Loss remaining years | $t_{zinc2}$ | 0.16 | mil/yr |
| Steel Loss Rate | $t_{steel}$ | 0.47 | mil/yr |
| Steel Loss for given design life | $E_s$ | 0.055 | in |

## 5 Define Internal Stability Factors

The internal stability factors for the lateral stress ratio ($K_r$) and the pullout friction factor (F*) shall be set equal to the default values for metal strips and are consistent with AASHTO Article 11.10. The lateral stress ratio factor is taken from Table 11.10.6.2.1-3 and the pullout friction factor, F*, is based on pullout testing. Both sets of factors decrease linearly to a depth of 20 feet at which time they remain constant with depth. Reference Section 8.2 for the calculation methodology used to determine the factors at any given depth. The depth used in the calculation of the earth pressure and pullout coefficient shall be from the top of the coping and not the intersection of the failure surface at the earth surcharge. The Earth pressure coefficient will use the $K_r$ multiplier as specified in AASHTO section 11.10.6.2.1. The earth pressure coefficient is the maximum at the top of the structure and linearly decreases to 1.2 times the active earth pressure coefficient at depths of 20 feet and beyond. This is consistent with the development of the stiffness design method.

Table 5-1 Internal Stability Parameters

| Location | Variable | Value | Unit |
|---|---|---|---|
| Earth Pressure Coefficient at depth equal to 0.00’ | $K_{r0}$ | 1.70 | dim |
| Earth Pressure Ratio at depth equal to 20.00’ | $K_{r20}$ | 1.20 | dim |
| Pullout friction factor at depth equal to 0.00’ | $F^*_0$ | 3.00 | dim |
| Pullout friction factor at depth equal to 20.00’ | $F^*_{20}$ | 0.67 | dim |

## 6 Calculate Unfactored Loads for External Stability

The external stability of the MSE structure is a function of the various forces and moments as shown in Figure 6-1. In the LRFD context the forces and moments need to be categorized into various load types. The primary load types for this example problem are the soil loads (EV, EH).

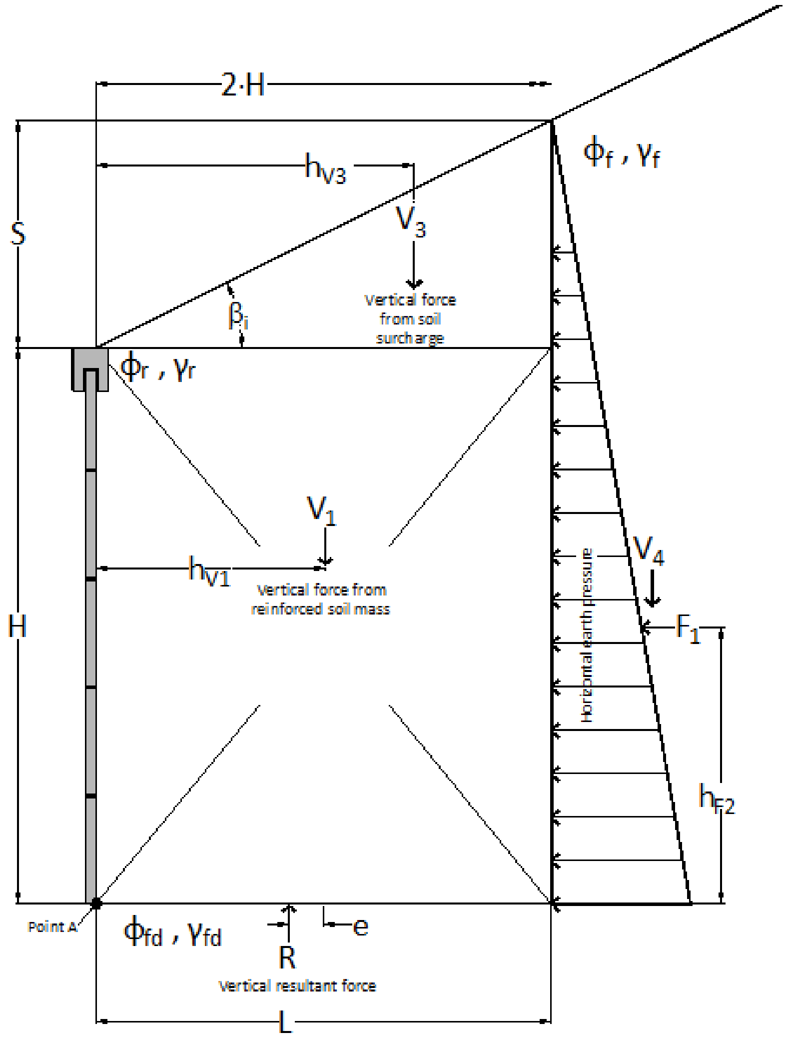



Figure 6-1 Unfactored Load Diagram for External Stability

Table 6-1 Unfactored Vertical Forces

| Vertical Force (Force/Length) | | Moment Arm (Length) | | Load Type |
|---|---|---|---|---|
| Equation | Value | Equation | Value | |
| $V_1 = \gamma_r \cdot H \cdot L$ | 97.50 k/ft | $h_{V1} = \frac{L}{2}$ | 13.00 ft | EV |
| $V_3 = \gamma_{ES} \cdot S \cdot \left[\frac{1}{2} \cdot X_s + (L - X_s)\right]$ | 21.12 k/ft | $h_{V3} = \frac{0.5 \cdot L^2 - \frac{X_s^2}{6}}{L - 0.5 \cdot X_s}$ | 17.33 ft | EV |

Table 6-1 Unfactored Vertical Forces

| Vertical Force (Force/Length) | | Moment Arm (Length) | | Load Type |
|---|---|---|---|---|
| Equation | Value | Equation | Value | |
| $V_4 = \frac{1}{2} \cdot K_{af} \cdot \gamma_r \cdot (H+S)^2 \cdot \sin(\beta_i)$ | 21.00 k/ft | $h_{V4} = L$ | 26.00 ft | EH |

Table 6-2 Unfactored Vertical Moments

| Moment (Force-Length/Length) | | Load Type |
|---|---|---|
| Equation | Value | |
| $M_{V1} = V_1 \cdot h_{V1}$ | 1267.50 k-ft/ft | EV |
| $M_{V3} = V_3 \cdot h_{V3}$ | 366.17 k-ft/ft | EV |
| $M_{V4} = V_4 \cdot h_{V4}$ | 545.88 k-ft/ft | EH |

Table 6-3 Unfactored Horizontal Forces

| Horizontal Force (Force/Length) | | Moment Arm (Length) | | Load Type |
|---|---|---|---|---|
| Equation | Value | Equation | Value | |
| $F_1 = \frac{1}{2} \cdot K_{af} \cdot \gamma_f \cdot (H+S)^2 \cdot \cos(\beta_i)$ | 41.99 k/ft | $h_{F1} = \frac{H+S}{3}$ | 14.33 ft | EH |

Table 6-4 Unfactored Horizontal Moments

| Moment (Force-Length/Length) | | Load Type |
|---|---|---|
| Equation | Value | |

Table 6-4 Unfactored Horizontal Moments

| Moment (Force-Length/Length) | | Load Type |
|---|---|---|
| Equation | Value | |
| $M_{F1} = F_1 \cdot h_{F1}$ | 601.86 k-ft/ft | EH |

# 7 Evaluate External Stability

## 7.1 Calculate Sliding Resistance at Base of MSE Structure

The following calculations are used to evaluate the sliding resistance at the base of the MSE structure. The calculations for sliding resistance at the base of the MSE structure for the three required load conditions (maximum, minimum and critical check) are illustrated in Table 7-1 through Table 7-3. The critical check uses the most severe force effect and is a combination of maximum and minimum values. For sliding the most severe force effect occurs when the resisting force is a minimum value and the driving force is a maximum value. Sliding resistance is a strength limit state check and therefore service limit state calculations are not considered. Since the friction angle of foundation soil, $\phi_{fd}$, is less than the friction angle of the reinforced soil, $\phi_r$, the sliding check will be performed using $\phi_{fd}$.

Table 7-1 Evaluation of Sliding at Base of MSE Structure – Strength I Maximum

| Item | Equation | Value |
|---|---|---|
| Lateral Loads on the MSE structure | $F = \gamma_{EH\max} \cdot F_1$ | 62.99 k/ft |
| Vertical Load at Base of Structure | $V = \gamma_{EV\max} \cdot (V_1 + V_3) + \gamma_{EH\max} \cdot V_4$ | 191.64 k/ft |
| Nominal sliding resistance at base of MSE structure | $V_{Nm} = V \cdot \tan(\phi_{fd})$ | 110.64 k/ft |
| Sliding resistance at base of MSE structure | $V_{Fm} = \phi_s \cdot V_{Nm}$ | 110.64 k/ft |
| Is $V_{Fm} > F_m$ | | Yes |
| Capacity Demand Ratio (CDR) for sliding | $CDR_s = \dfrac{V_{Fm}}{F}$ | 1.76 |

Table 7-2 Evaluation of Sliding at Base of MSE Structure – Strength I Minimum

| Item | Equation | Value |
|---|---|---|
| Lateral Loads on the MSE structure | $F = \gamma_{EH\min} \cdot F_1$ | 37.79 k/ft |
| Vertical Load at Base of Structure | $V = \gamma_{EV\min} \cdot (V_1 + V_3) + \gamma_{EH\min} \cdot V_4$ | 137.52 k/ft |
| Nominal sliding resistance at base of MSE structure | $V_{Nm} = V \cdot \tan(\phi_{fd})$ | 79.40 k/ft |
| Sliding resistance at base of MSE structure | $V_{Fm} = \phi_s \cdot V_{Nm}$ | 79.40 k/ft |
| Is $V_{Fm} > F_m$ | | Yes |
| Capacity Demand Ratio (CDR) | $CDR_s = \frac{V_{Fm}}{F}$ | 2.10 |

Table 7-3 Evaluation of Sliding at Base of MSE Structure – Strength I Critical Value Check

| Item | Equation | Value |
|---|---|---|
| Lateral Loads n the MSE structure | $F = \gamma_{EH\max} \cdot F_1$ | 62.99 k/ft |
| Vertical Load at Base of Structure | $V = \gamma_{EV\min} \cdot (V_1 + V_3) + \gamma_{EH\max} \cdot V_4$ | 150.12 k/ft |
| Nominal sliding resistance at base of MSE structure | $V_{Nm} = V \cdot \tan(\phi_{fd})$ | 86.67 k/ft |
| Sliding resistance at base of MSE structure | $V_{Fm} = \phi_s \cdot V_{Nm}$ | 86.67 k/ft |
| Is $V_{Fm} > F_m$ | | Yes |
| Capacity Demand Ratio (CDR) | $CDR_s = \frac{V_{Fm}}{F}$ | 1.38 |

The critical value check results in the smallest CDR for this example and therefore governs the sliding mode of failure (Table 7-3).

## 7.2 Calculate Limiting Eccentricity at Base of MSE Structure

The following calculations are used to evaluate the limiting eccentricity at the base of the MSE structure. For the purpose of this calculation the beneficial contribution of any specified live load to resisting forces and moments is neglected. The calculations for limiting eccentricity at the base of the MSE structure for the three required load conditions are illustrated in Table 7-4 through Table 7-6. The critical check uses the most severe force effect and is a combination of maximum and minimum values. For limiting eccentricity the most severe force effect occurs when the resisting moment is a minimum value and the overturning moment is a maximum value. Note that limiting eccentricity is a strength limit state check and therefore service limit state calculations are not performed. Included in the calculation is the Capacity Demand Ratio for overturning. The MSE structure is founded on soil and therefore the eccentricity shall be located within the middle 2/3 of the base of the structure in conformance with AASHTO 11.6.3.3.

Table 7-4 Evaluation of Limiting Eccentricity for MSE Structure – Strength I Maximum

| Item | Equation | Value |
|---|---|---|
| Lateral Loads on the MSE structure | $F = \gamma_{EH\max} \cdot F_1$ | 62.99 k/ft |
| Overturning Moment about point A | $M_{OA} = \gamma_{EH\max} \cdot M_{F1}$ | 902.80 (k-ft)/ft |
| Vertical Load at Base of Structure | $V_A = \gamma_{EV\max} \cdot (V_1 + V_3) + \gamma_{EH\max} \cdot V_4$ | 191.64 k/ft |
| Resisting moment about point A | $M_{RA} = \gamma_{EV\max} \cdot (M_{V1} + M_{V3}) + \gamma_{EH\max} \cdot M_{V4}$ | 3024.26 (k-ft)/ft |
| Net moment about Point A | $M_A = M_{RA} - M_{OA}$ | 2121.47 (k-ft)/ft |
| Location of resultant force on base of structure from point A | $a = \frac{M_A}{V_A}$ | 11.07 ft |
| Eccentricity at base of MSE structure | $e_L = \frac{L}{2} - a$ | 1.93 ft |
| Limiting eccentricity | $e_{limit} = \frac{L}{3}$ | 8.67 ft |
| Is the resultant within limiting value of e | | Yes |
| Calculated e/L | $\frac{e_L}{L}$ | 0.07 dim |
| Capacity Demand Ratio (CDR) for Overturning | $CDR_O = \frac{M_{RA}}{M_{OA}}$ | 3.35 |

Table 7-5 Evaluation of Limiting Eccentricity for MSE Structure – Strength I Minimum

| Item | Equation | Value |
|---|---|---|
| Lateral Loads n the MSE structure | $F = \gamma_{EH\min} \cdot F_1$ | 37.79 k/ft |

Table 7-5 Evaluation of Limiting Eccentricity for MSE Structure – Strength I Minimum

| Item | Equation | Value |
|---|---|---|
| Overturning Moment about point A | $M_{OA} = \gamma_{EH\min} \cdot M_{F1}$ | 541.68 (k-ft)/ft |
| Vertical Load at Base of Structure | $V_A = \gamma_{EV\min} \cdot (V_1 + V_3) + \gamma_{EH\min} \cdot V_4$ | 137.52 k/ft |
| Resisting moment about point A | $M_{RA} = \gamma_{EV\min} \cdot (M_{V1} + M_{V3}) + \gamma_{EH\min} \cdot M_{V4}$ | 2124.95 (k-ft)/ft |
| Net moment about Point A | $M_A = M_{RA} - M_{OA}$ | 1583.28 (k-ft)/ft |
| Location of resultant force on base of structure from point A | $a = \frac{M_A}{V_A}$ | 11.51 ft |
| Eccentricity at base of MSE structure | $e_L = \frac{L}{2} - a$ | 1.49 ft |
| Limiting eccentricity | $e_{limit} = \frac{L}{3}$ | 8.67 ft |
| Is the resultant within limiting value of e | | Yes |
| Calculated e/L | $\frac{e_L}{L}$ | 0.06 dim |
| Capacity Demand Ratio (CDR) for Overturning | $CDR_O = \frac{M_{RA}}{M_{OA}}$ | 3.92 |

Table 7-6 Evaluation of Limiting Eccentricity for MSE Structure – Strength I Critical Value Check

| Item | Equation | Value |
|---|---|---|
| Lateral Loads on the MSE structure | $F = \gamma_{EH\max} \cdot F_1$ | 62.99 k/ft |
| Overturning Moment about point A | $M_{OA} = \gamma_{EH\max} \cdot M_{F1}$ | 902.80 (k-ft)/ft |
| Vertical Load at Base of Structure without LS | $V_A = \gamma_{EV\min} \cdot (V_1 + V_3) + \gamma_{EH\min} \cdot V_4$ | 150.12 k/ft |
| Resisting moment about point A without LS | $M_{RA} = \gamma_{EV\min} \cdot (M_{V1} + M_{V3}) + \gamma_{EH\min} \cdot M_{V4}$ | 2452.48 (k-ft)/ft |
| Net moment about Point A | $M_A = M_{RA} - M_{OA}$ | 1549.68 (k-ft)/ft |
| Location of resultant force on base of structure from point A | $a = \frac{M_A}{V_A}$ | 10.32 ft |
| Eccentricity at base of MSE structure | $e_L = \frac{L}{2} - a$ | 2.68 ft |

Table 7-6 Evaluation of Limiting Eccentricity for MSE Structure – Strength I Critical Value Check

| Item | Equation | Value |
|---|---|---|
| Limiting eccentricity | $e_{\lim it} = \frac{L}{3}$ | 8.67 ft |
| Is the resultant within limiting value of e | | Yes |
| Effective width at base of MSE structure | $B' = L - 2 \cdot e_L$ | 20.65 ft |
| Calculated e/L | $\frac{e_L}{L}$ | 0.10 dim |
| Capacity Demand Ratio (CDR) for Overturning | $CDR_O = \frac{M_{RA}}{M_{OA}}$ | 2.72 |

The critical value check results in the largest eccentricity for this example and therefore governs the limiting eccentricity (Table 7-6).

## 7.3 Calculate Bearing Resistance at Base of MSE Structure

The vertical stress at the base of the MSE structure is calculated as shown in Equation 7-1.

$$\sigma_V = \frac{\Sigma V}{L - 2 \cdot e_L} \qquad \text{Equation 7-1}$$

Where the value $\Sigma V = R = V_1 + V_3 + V_4$ is the resultant of vertical forces and the load eccentricity $e_L$ is calculated by principles of statics using appropriate loads and moments with the applicable load factors.

In the LRFD method, $\sigma_v$ is compared with the factored bearing resistance when computed for strength limit state and for settlement analysis the service limit state bearing resistance is used. The various calculations for evaluation of bearing resistance are presented in Table 7-7 through Table 7-10. The Service I load combination is evaluated to compute the vertical stress for any settlement analysis and the results are presented in Table 7-10.

Table 7-7 Evaluation of Bearing Resistance for MSE Structure – Strength I Maximum

| Item | Equation | Value |
|---|---|---|

Table 7-7 Evaluation of Bearing Resistance for MSE Structure – Strength I Maximum

| Item | Equation | Value |
|---|---|---|
| Lateral Loads on the MSE structure | $F = \gamma_{EH\max} \cdot F_1$ | 62.99 k/ft |
| Overturning Moment about point A | $M_{OA} = \gamma_{EH\max} \cdot M_{F1}$ | 902.80 (k-ft)/ft |
| Vertical Load at Base of Structure | $V_A = \gamma_{EV\max} \cdot V_1 + \gamma_{EV\max} \cdot V_3 + \gamma_{EH\max} \cdot V_4$ | 191.64 k/ft |
| Resisting moment about point A | $M_{RA} = \gamma_{EV\max} \cdot M_1 + \gamma_{EV\max} \cdot M_3 + \gamma_{EH\max} \cdot M_4$ | 3024.26 (k-ft)/ft |
| Net moment about Point A | $M_A = M_{RA} - M_{OA}$ | 2121.47 (k-ft)/ft |
| Location of resultant force on base of structure from point A | $a = \frac{M_A}{V_A}$ | 11.07 ft |
| Eccentricity at base of MSE structure from center | $e_L = \frac{L}{2} - a$ | 1.93 ft |
| Limiting eccentricity | $e_{limit} = \frac{L}{3}$ | 8.67 ft |
| Is the resultant within limiting value of e | | Yes |
| Effective width at base of MSE structure | $B' = L - 2 \cdot e_L$ | 22.14 ft |
| Bearing stress due to MSE Structure | $\sigma_V = \frac{V_A}{L - 2 \cdot e_L}$ | 8.66 ksf |

Table 7-8 Evaluation of Bearing Resistance for MSE Structure – Strength I Minimum

| Item | Equation | Value |
|---|---|---|
| Lateral Loads n the MSE structure | $F = \gamma_{EH\min} \cdot F_1$ | 37.79 k/ft |
| Overturning Moment about point A | $M_{OA} = \gamma_{EH\min} \cdot M_{F1}$ | 541.68 (k-ft)/ft |
| Vertical Load at Base of Structure | $V_A = \gamma_{EV\min} \cdot V_1 + \gamma_{EV\min} \cdot V_3 + \gamma_{EH\min} \cdot V_4$ | 137.52 k/ft |
| Resisting moment about point A | $M_{RA} = \gamma_{EV\min} \cdot M_1 + \gamma_{EV\min} \cdot M_3 + \gamma_{EH\min} \cdot M_4$ | 2124.95 (k-ft)/ft |
| Net moment about Point A | $M_A = M_{RA} - M_{OA}$ | 1583.28 (k-ft)/ft |
| Location of resultant force on base of structure from point A | $a = \frac{M_A}{V_A}$ | 11.51 ft |

Table 7-8 Evaluation of Bearing Resistance for MSE Structure – Strength I Minimum

| Item | Equation | Value |
|---|---|---|
| Eccentricity at base of MSE structure from center | $e_L = \frac{L}{2} - a$ | 1.49 ft |
| Limiting eccentricity | $e_{\lim it} = \frac{L}{3}$ | 8.67 ft |
| Is the resultant within limiting value of e | | Yes |
| Effective width at base of MSE structure | $B' = L - 2 \cdot e_L$ | 23.03 ft |
| Bearing stress due to MSE Structure | $\sigma_V = \frac{V_A}{L - 2 \cdot e_L}$ | 5.97 ksf |

Table 7-9 Evaluation of Bearing Resistance for MSE Structure – Strength I Critical Value Check

| Item | Equation | Value |
|---|---|---|
| Lateral Loads n the MSE structure | $H_m = \gamma_{EH\max} \cdot F_1$ | 62.99 k/ft |
| Overturning Moment about point A | $M_{OA} = \gamma_{EH\max} \cdot M_{F1}$ | 902.80 (k-ft)/ft |
| Vertical Load at Base of Structure without LS | $V_A = \gamma_{EV\min} \cdot V_1 + \gamma_{EV\min} \cdot V_3 + \gamma_{VH\max} \cdot V_4$ | 150.12 k/ft |
| Resisting moment about point A without LS | $M_{RA} = \gamma_{EV\min} \cdot M_{V1} + \gamma_{EV\min} \cdot M_{V3} + \gamma_{EV\max} \cdot M_{V4}$ | 2452.48 (k-ft)/ft |
| Net moment about Point A | $M_A = M_{RA} - M_{OA}$ | 1549.68 (k-ft)/ft |
| Location of resultant force on base of structure from point A | $a = \frac{M_A}{V_A}$ | 10.32 ft |
| Eccentricity at base of MSE structure from center | $e_L = \frac{L}{2} - a$ | 2.68 ft |
| Limiting eccentricity | $e_{\lim it} = \frac{L}{3}$ | 8.67 ft |
| Is the resultant within limiting value of e | | Yes |
| Effective width at base of MSE structure | $B' = L - 2 \cdot e_L$ | 20.65 ft |

Table 7-9 Evaluation of Bearing Resistance for MSE Structure – Strength I Critical Value Check

| Item | Equation | Value |
|---|---|---|
| Bearing stress due to MSE Structure | $\sigma_V = \dfrac{V_A}{L - 2 \cdot e_L}$ | 7.27 ksf |

Table 7-10 Evaluation of Bearing Resistance for MSE Structure – Strength I Service

| Item | Equation | Value |
|---|---|---|
| Lateral Loads on the MSE structure | $F = F_1$ | 41.99 k/ft |
| Overturning Moment about point A | $M_{OA} = F_1 \cdot h_{F1}$ | 601.86 (k-ft)/ft |
| Vertical Load at Base of Structure | $V_A = V_1 + V_3 + V_4$ | 139.62 k/ft |
| Resisting moment about point A | $M_{RA} = M_{V1} + M_{V3} + M_{V4}$ | 2179.54 (k-ft)/ft |
| Net moment about Point A | $M_A = M_{RA} - M_{OA}$ | 1577.68 (k-ft)/ft |
| Location of resultant force on base of structure from point A | $a = \dfrac{M_A}{V_A}$ | 11.30 ft |
| Eccentricity at base of MSE structure from center | $e_L = \dfrac{L}{2} - a$ | 1.70 ft |
| Limiting eccentricity | $e_{\lim it} = \dfrac{L}{6}$ | 4.33 ft |
| Is the resultant within limiting value of e | | Yes |
| Effective width at base of MSE structure | $B' = L - 2 \cdot e_L$ | 22.60 ft |
| Bearing stress due to MSE Structure | $\sigma_V = \dfrac{V_A}{L - 2 \cdot e_L}$ | 6.18 ksf |

The Strength I Maximum load combination result in the extreme force effect in terms of maximum vertical stress and therefore governs the bearing resistance mode of failure (Table 7-7).

## 7.4 Summary of External Stability

Table 7-11 provides a summary of the external stability results. These values can be compared to the results of the proprietary software program MSE-Pro and the Adama Engineering software program MSEW. The capacity demand ratio (CDR) must be greater or equal to 1.00. All comparable values are within a margin of 2%.

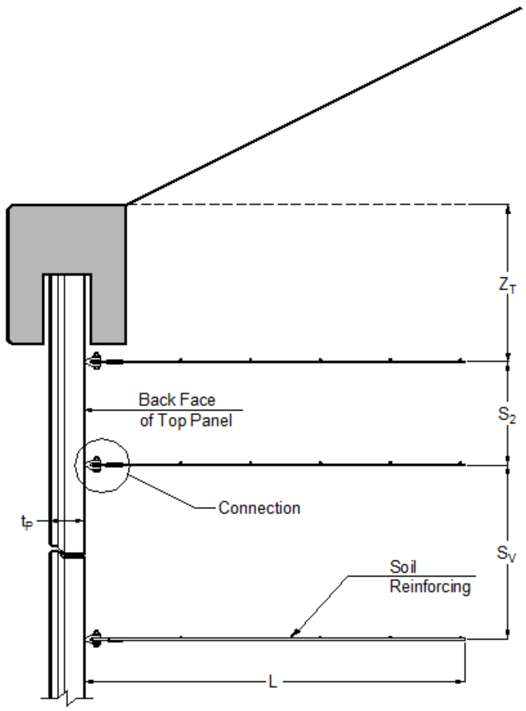
Table 7-11 Summary of External Stability

| Load Combination | CDR Sliding (dim) | Limiting Eccentricity/Maximum (ft) | CDR Overturning (dim) | Bearing Stress (ksf) | Effective Length (ft) |
|---|---|---|---|---|---|
| Strength I Maximum | 1.76 | 1.93/8.67 | 3.35 | 8.66 | 22.14 |
| Strength I Minimum | 2.10 | 1.49/8.67 | 3.92 | 5.97 | 23.03 |
| Strength I Critical Value Check | 1.38 | 2.68/8.67 | 2.72 | 7.27 | 20.65 |
| Service I | NA | NA | NA | 6.18 | 22.60 |

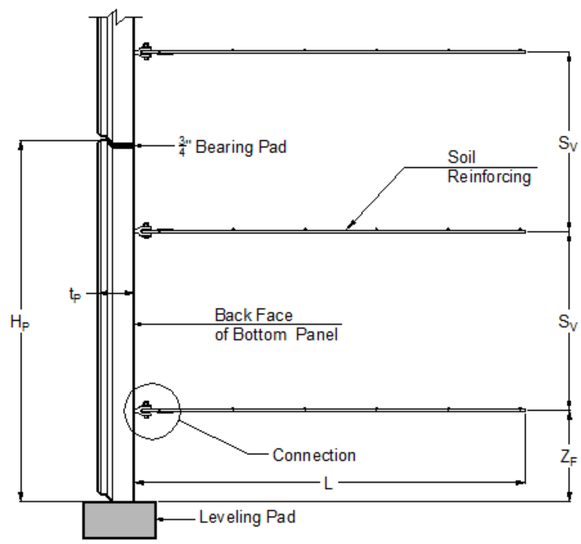
# 8 Evaluate Internal Stability

## 8.1 Determine Soil Reinforcing Depths and Tributary Area for Internal Stability

The depth to the soil reinforcing is a function of the facing panel and the vertical spacing of the anchors that are used to attach the soil reinforcing to the facing panel. The depth for the top soil reinforcing is a function of the top of structure treatment, e.g. coping element, that is placed at the top of the structure. In the tables that follow the bottom most soil reinforcing element is assumed to be element 1.




Top of Structure




Bottom of Structure

Figure 8-1 Soil Reinforcing Depth Parameters

Table 8-1 Soil Reinforcing Layout

| Soil Element | Distance Above Leveling Pad ft | Depth Below Top of Structure $Z_i$ ft | Depth Below Intersection Of Failure Surface ($d_m$) ft | Vertical Spacing ($S_v$) ft |
|---|---|---|---|---|
| 12 | 27.75 | 2.25 | 7.54 | 3.00 |
| 11 | 26.25 | 3.75 | 9.04 | 2.00 |
| 10 | 23.75 | 6.25 | 11.54 | 2.50 |
| 9 | 21.25 | 8.75 | 14.04 | 2.50 |
| 8 | 18.75 | 11.25 | 16.54 | 2.50 |
| 7 | 16.25 | 13.75 | 19.04 | 2.50 |
| 6 | 13.75 | 16.25 | 21.54 | 2.50 |
| 5 | 11.25 | 18.75 | 24.04 | 2.50 |
| 4 | 8.75 | 21.25 | 26.54 | 2.50 |
| 3 | 6.25 | 23.75 | 29.04 | 2.50 |
| 2 | 3.75 | 26.25 | 31.54 | 2.50 |
| 1 | 1.25 | 28.75 | 34.04 | 2.50 |

## 8.2 Determine Variation of $K_r$, F*and Length of Embedment for Internal Stability

The failure surface for inextensible soil reinforcing is bilinear as shown in Figure 8-2. The failure surface is a function of the surcharge back-slope at the top of the structure and intersects the ground surface at the location of the mechanical height, $H_m$. The mechanical height is calculated using Equation 8-1. The length of embedment ($L_e$) is the length of the soil reinforcing that is contained behind the failure surface.

$$H_m = If\left\{X_s > 0.3\cdot(H+S), H + \frac{\tan(\beta_s)\cdot 0.3\cdot H}{1-0.3\cdot\tan(\beta_s)}, H+S\right\}$$ Equation 8-1

For a horizontal back-slope with no earth surcharge the mechanical height is equal to the design height of the structure.

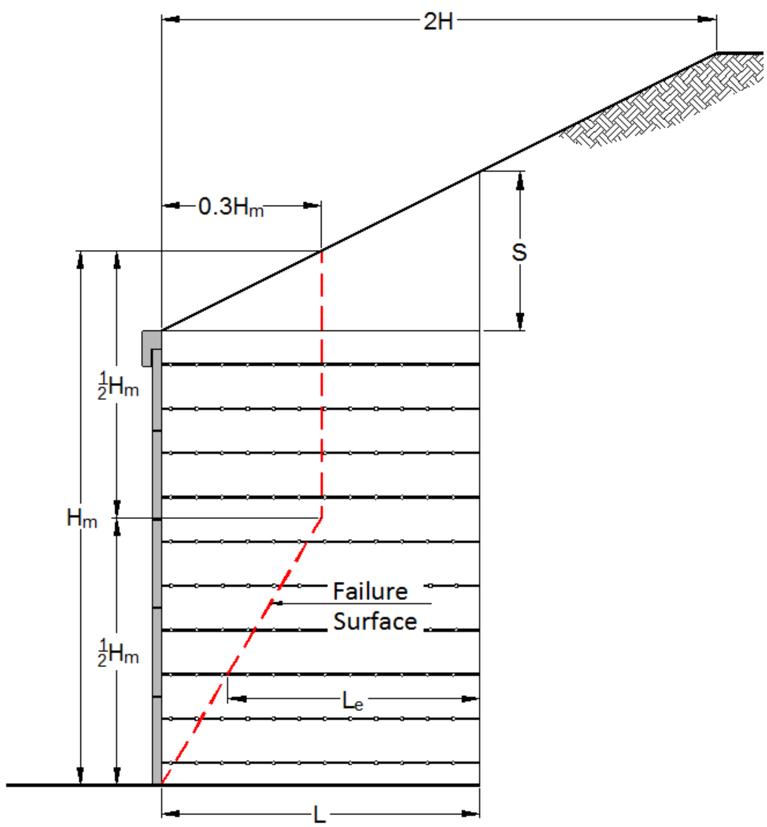



Figure 8-2 Failure Surface Diagram

For the simplified method the internal earth pressure coefficient and the pullout friction factor are calculated from the top of the structure H. The pullout friction factor F* is determined based on the reinforced backfill having a coefficient of uniformity equal to 4. These values are extremely conservative values for the EarthTrac soil reinforcing.

$$K_i = \left[1.7 - \frac{1.7-1.2}{20(ft)} \cdot z_i\right] \cdot K_{ai} \rightarrow z_i < 20(ft)$$ Equation 8-2

$$K_i = 1.2 \cdot K_a \rightarrow z_i \geq 20(ft)$$ Equation 8-3

$$F^* = 3.00 - \frac{3.00-0.67}{20(ft)} \cdot z_i \rightarrow z_i < 20(ft)$$ Equation 8-4

$$F^* = 0.67 \rightarrow z_i \geq 20(ft)$$ Equation 8-5

Table 8-2 Internal Stability Factors

| Soil Element | Depth Below Top of Structure $Z_i$ ft | Earth Pressure Ratio $K_i$ dim | Pullout Friction Factor F* dim | Length of Embedment $L_e$ ft |
|---|---|---|---|---|
| 12 | 2.25 | 0.465 | 2.738 | 15.41 |
| 11 | 3.75 | 0.454 | 2.563 | 15.41 |
| 10 | 6.25 | 0.436 | 2.272 | 15.41 |
| 9 | 8.75 | 0.419 | 1.981 | 15.41 |
| 8 | 11.25 | 0.401 | 1.689 | 15.41 |
| 7 | 13.75 | 0.383 | 1.398 | 16.25 |
| 6 | 16.25 | 0.366 | 1.107 | 17.75 |
| 5 | 18.75 | 0.348 | 0.816 | 19.25 |
| 4 | 21.25 | 0.339 | 0.670 | 20.75 |
| 3 | 23.75 | 0.339 | 0.670 | 22.25 |
| 2 | 26.25 | 0.339 | 0.670 | 23.75 |
| 1 | 28.75 | 0.339 | 0.670 | 25.25 |

### 8.3 Calculate Horizontal Stress at Elevation of Each Soil Reinforcing Element

The horizontal stress at each soil reinforcing location is a function of the vertical stress, $\sigma_V$, and coefficient of lateral earth pressure, $K_i$. For the earth surcharge an equivalent soil height, $S_{eq}$, is computed based upon the slope geometry in conformance with AASHTO Figure 11.10.6.2.1-2. The value of $S_{eq}$ shall not exceed the slope height for broken back-slope fills. A reinforcement length of 0.7H is used to compute the sloping backfill stress, $\Delta\sigma_V$, on the soil reinforcement, as a greater length would only have minimal effect on the reinforcement. The vertical stress is equal to the product of the equivalent soil height and the reinforced fill unit weight, and is uniformly applied across the top of the MSE zone.

$$\sigma_2 = \frac{1}{2} \cdot \gamma_r \cdot (0.7 \cdot H) \cdot \tan(\beta) \quad \text{Equation 8-6}$$

The vertical stress at each soil reinforcing location is calculated using Equation 8-7 and the horizontal stress is calculated using Equation 8-8. The lateral earth pressure coefficient is calculated using either Equation 8-2 or Equation 8-3.

$$\sigma_V = \gamma_{EV} \cdot (\gamma_r \cdot z_i + \sigma_2) \quad \text{Equation 8-7}$$

$$\sigma_H = K_i \cdot \sigma_V \quad \text{Equation 8-8}$$

Table 8-3 Stress Calculation

| Soil Element | Depth Below Top of Structure $z_i$ ft | Vertical Stress $\sigma_V$ ksf | Earth Pressure Coefficient $K_i$ dim | Horizontal Stress $\sigma_H$ ksf |
|---|---|---|---|---|
| 12 | 2.25 | 1.27 | 0.465 | 0.59 |
| 11 | 3.75 | 1.52 | 0.454 | 0.69 |
| 10 | 6.25 | 1.94 | 0.436 | 0.85 |
| 9 | 8.75 | 2.36 | 0.419 | 0.99 |
| 8 | 11.25 | 2.78 | 0.401 | 1.12 |
| 7 | 13.75 | 3.21 | 0.383 | 1.23 |
| 6 | 16.25 | 3.63 | 0.366 | 1.33 |
| 5 | 18.75 | 4.05 | 0.348 | 1.41 |
| 4 | 21.25 | 4.47 | 0.339 | 1.52 |
| 3 | 23.75 | 4.89 | 0.339 | 1.66 |
| 2 | 26.25 | 5.32 | 0.339 | 1.80 |
| 1 | 28.75 | 5.74 | 0.339 | 1.95 |

## 8.4 Calculate Maximum Tension at Elevation of Each Soil Reinforcing Element

The maximum tension at each soil reinforcing elevation is the product of the horizontal stress and the tributary area that the row of soil reinforcing is required to resist. The tributary spacing ($S_{TA}$) for the soil reinforcing depth being considered is a function of the vertical spacing of the soil reinforcing and is calculated as the distance between the soil reinforcing depth at the midpoint of the soil reinforcing above and the distance between the soil reinforcing depth at the midpoint of the soil reinforcing below (Equation 8-9). The tributary spacing is multiplied by the length of the panel to calculate the tributary area, $A_T$ (Equation 8-10). Table 8-4 list the tributary area that each soil reinforcing element is to resist and the corresponding tensile force. Please reference Section 0 for further discussion.

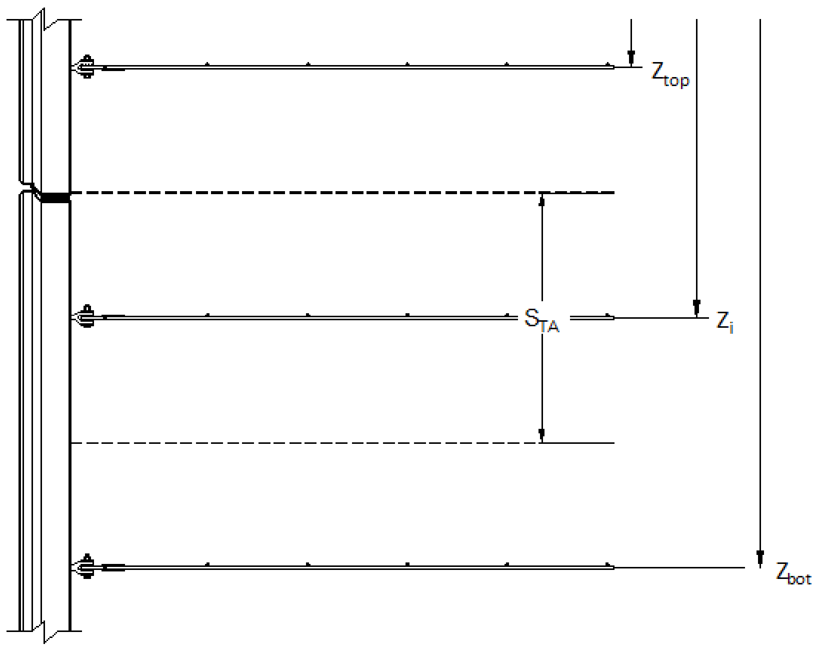



Figure 8-3 Tributary Area

$$S_{TA} = \frac{Z_i - Z_{top}}{2} + \frac{Z_{bot} - Z_i}{2} \qquad \text{Equation 8-9}$$

$$A_T = S_{TA} \cdot L_P \qquad \text{Equation 8-10}$$

The maximum tension required to be resisted by the row of soil reinforcing is the product of the horizontal stress times the tributary area (Equation 8-11).

$$T_{\max} = \sigma_H \cdot A_T \qquad \text{Equation 8-11}$$

Table 8-4 Maximum Tension At Soil Reinforcing

| Soil Element | Depth Below Top of Structure $Z_i$ ft | Horizontal Stress $\sigma_H$ ksf | Tributary Area $A_T$ sf | Maximum Tension $T_{max}$ kip |
|---|---|---|---|---|
| 12 | 2.25 | 0.5900 | 15.00 | 8.850 |

Table 8-4 Maximum Tension At Soil Reinforcing

| Soil Element | Depth Below Top of Structure $Z_i$ ft | Horizontal Stress $\sigma_H$ ksf | Tributary Area $A_T$ sf | Maximum Tension $T_{max}$ kip |
|---|---|---|---|---|
| 11 | 3.75 | 0.6900 | 10.00 | 6.900 |
| 10 | 6.25 | 0.8500 | 12.50 | 10.625 |
| 9 | 8.75 | 0.9900 | 12.50 | 12.375 |
| 8 | 11.25 | 1.1200 | 12.50 | 14.000 |
| 7 | 13.75 | 1.2300 | 12.50 | 15.375 |
| 6 | 16.25 | 1.3300 | 12.50 | 16.625 |
| 5 | 18.75 | 1.4100 | 12.50 | 17.625 |
| 4 | 21.25 | 1.5200 | 12.50 | 19.000 |
| 3 | 23.75 | 1.6600 | 12.50 | 20.750 |
| 2 | 26.25 | 1.8000 | 12.50 | 22.500 |
| 1 | 28.75 | 1.9500 | 12.50 | 24.375 |

## 8.5 Determine Required Area of Steel at Elevation of Each Soil Reinforcing Element

After the maximum tensile force to be resisted by each row of soil reinforcing is calculated the area of steel required at the end of the design life can be determined (Equation 8-12). Once the required end of design life steel area is determined the required steel area that includes the effects of degradation can be calculated. Based on the required steel area the number of soil reinforcing elements for each row can be determined (Equation 8-14).

$$A_c = \frac{T_{\max}}{\phi_t \cdot F_y} \qquad \text{Equation 8-12}$$

$$A_{SS1} = \left(t_{SS1} - E_s\right) \cdot b \qquad \text{Equation 8-13}$$

$$N_{SR} = \text{int}\left[\frac{A_{req}}{A_{GS}}\right] + 1 \qquad \text{Equation 8-14}$$

*The value $N_{SR}$ is the integer value of the quotient plus 1.

Where: $N_{SR}$ = Number of EarthTrac SS

$A_{SS}$ = Area of one EarthTrac SS

The tensile capacity demand ratio is the ratio of the supplied steel area to the required steel area and is calculated using Equation 8-15 and is required to be greater than or equal to 1.0.

$$CDR_{rup} = \frac{N_{SR} \cdot A_{GS}}{A_{req}}$$ Equation 8-15

Table 8-5 Area of Steel Required for Rupture

| Soil Element | Depth Below Top of Structure $Z_i$ ft | Maximum Tension $T_{max}$ kip | Area of Steel Required at end of Design Life $A_{req}$ In$^2$ | Area of Steel Provided at end of Design Life $A_c$ In$^2$ | Number Soil Reinforcing Elements $N_{SR}$ dim | CDR Rupture dim |
|---|---|---|---|---|---|---|
| 12 | 2.25 | 8.850 | 0.24 | 0.40 | 2 | 1.71 |
| 11 | 3.75 | 6.900 | 0.18 | 0.40 | 2 | 2.19 |
| 10 | 6.25 | 10.625 | 0.28 | 0.40 | 2 | 1.42 |
| 9 | 8.75 | 12.375 | 0.33 | 0.40 | 2 | 1.22 |
| 8 | 11.25 | 14.000 | 0.37 | 0.40 | 2 | 1.08 |
| 7 | 13.75 | 15.375 | 0.41 | 0.61 | 3 | 1.48 |
| 6 | 16.25 | 16.625 | 0.44 | 0.61 | 3 | 1.36 |
| 5 | 18.75 | 17.625 | 0.47 | 0.61 | 3 | 1.29 |
| 4 | 21.25 | 19.000 | 0.51 | 0.61 | 3 | 1.19 |
| 3 | 23.75 | 20.750 | 0.55 | 0.61 | 3 | 1.09 |
| 2 | 26.25 | 22.500 | 0.60 | 0.61 | 3 | 1.01 |
| 1 | 28.75 | 24.375 | 0.65 | 0.81 | 4 | 1.24 |

## 8.6 Calculate Factored Pullout Resistance at Elevation of Each Soil Reinforcing Element

The nominal pullout resistance of the soil reinforcing (Equation 8-18) is a function of the length of embedment ($L_e$), pullout friction factor (F*), depth of overburden over the soil reinforcing ($\sigma_{VO}$), scale correction factor ($\alpha$), and the geometric factor (C). The factored pullout resistance ($\sigma_{\pi}$) is the product of the nominal pullout resistance ($P_r$) multiplied by the pullout resistance factor, $\phi_{po}$ (Equation 8-19).

The overburden that is used in the pullout calculation will also include the earth surcharge that is over the length of embedment. The equivalent height of the surcharge load for a sloped back-slope is calculated as shown in AASHTO Figure 11.10.6.3.2-1. The equivalent surcharge height is a function of the location of the failure surface. For an infinite back-slope condition the equivalent surcharge height is a function of the failure surface and the location of the crest of the slope at the terminal end of the soil reinforcing (Figure 8-4) and varies with the depth Z. The values $X_m$, $X_1$, $L_e$, $Y_1$ and $Y_2$ vary and are a function of the location of the failure surface and crest of slope.

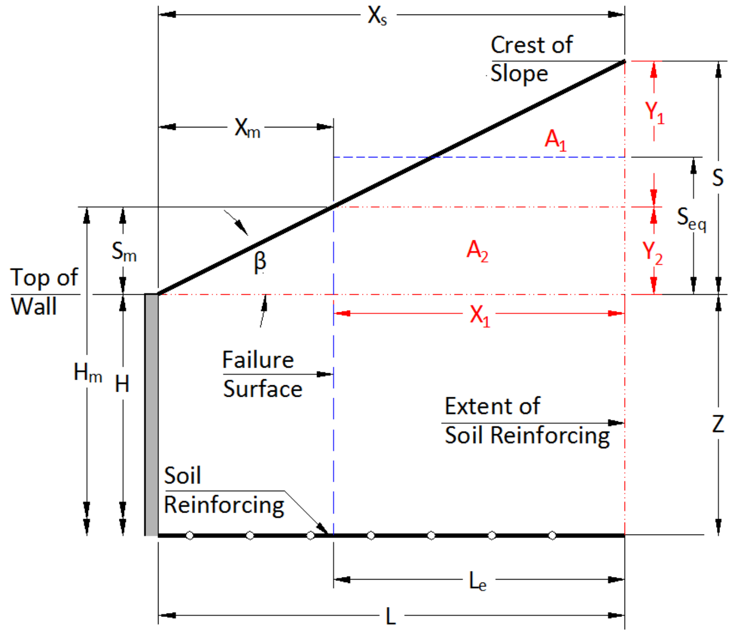



**Figure 8-4 Broken Back-Slope Equivalent Surcharge**

$$S_{eq} = \frac{A_1 + A_2}{X_1}$$ Equation 8-16

$$\sigma_{VO} = \gamma_r \cdot (Z + S_{eq})$$ Equation 8-17

$$P_r = \alpha \cdot C \cdot F^* \cdot \sigma_{VO} \cdot L_e$$ Equation 8-18

$$P_\pi = \phi_{po} \cdot P_r$$ Equation 8-19

The number of EarthTrac soil reinforcing elements that are required to resist pullout for each row is a function of the maximum force that is to be resisted. The maximum force $T_{max}$ is calculated using Equation 8-11. The maximum tension force to be resisted is equal to the force calculated for tension capacity because the structure contains an infinite back-slope where there is no live load surcharge and therefor the equations are identical. The Capacity Demand Ratio (Equation 8-21) is a function of the factored resistance of the EarthTrac and the total number grid strips required (Equation 8-20).

$$N_{GS} = \text{int}\left[\frac{T_{max}}{P_\pi}\right] + 1$$ Equation 8-20

$$CDR_{PO} = \frac{T_{max}}{N_{GS} \cdot P_{\pi}}$$ Equation 8-21

Table 8-6 Pullout Resistance for EarthTrac Element

| SR Element | Depth Below Top of Structure $Z_i$ ft | Length Of Embed. Le Ft | F* dim | Nominal Pullout Resistance $P_r$ Kip/Strip | Factored Pullout Resistance $P_{\pi}$ Kip/Strip | Max Tension $T_{max}$ kip | Required Number SR Elements $N_{SR}$ dim | CDR Pullout dim |
|---|---|---|---|---|---|---|---|---|
| 12 | 2.25 | 15.41 | 2.74 | 20.05 | 18.05 | 8.850 | 2 | 4.08 |
| 11 | 3.75 | 15.41 | 2.56 | 21.20 | 19.08 | 6.900 | 2 | 5.53 |
| 10 | 6.25 | 15.41 | 2.27 | 22.44 | 20.20 | 10.625 | 2 | 3.80 |
| 9 | 8.75 | 15.41 | 1.98 | 22.75 | 20.48 | 12.375 | 2 | 3.31 |
| 8 | 11.25 | 15.41 | 1.69 | 22.13 | 19.92 | 14.000 | 2 | 2.85 |
| 7 | 13.75 | 16.25 | 1.40 | 21.51 | 19.36 | 15.375 | 2 | 2.52 |
| 6 | 16.25 | 17.75 | 1.11 | 20.37 | 18.33 | 16.625 | 2 | 2.21 |
| 5 | 18.75 | 19.25 | 0.82 | 17.72 | 15.95 | 17.625 | 2 | 1.81 |
| 4 | 21.25 | 20.75 | 0.67 | 16.84 | 15.15 | 19.000 | 2 | 1.59 |
| 3 | 23.75 | 22.25 | 0.67 | 19.37 | 17.43 | 20.750 | 2 | 1.68 |
| 2 | 26.25 | 23.75 | 0.67 | 22.09 | 19.88 | 22.500 | 2 | 1.77 |
| 1 | 28.75 | 25.25 | 0.67 | 24.98 | 22.48 | 24.375 | 2 | 1.84 |

## 8.7 Summary of Internal Stability at Elevation of Each Soil Reinforcing Element

The number of EarthTrac that are required for each row is equal to the larger number of EarthTrac required to satisfy tensile capacity calculated in Section 8.5 and the number of EarthTrac required to satisfy resistance to pullout calculated in Section 8.6. The capacity demand ratios are recalculated calculated and summarized in Table 8-7. The CDR is based on the required number of EarthTrac that satisfy both rupture and pullout.

Table 8-7 Summary of Internal Stability for EarthTrac Element

| SR | $Z_i$ ft | $N_{SR}$ dim | $\sigma_v$ ksf | $K_r$ dim | $\sigma_H$ ksf | $T_{max}$ kip | CDR Rupture dim | $L_e$ Ft | F* dim | CDR Pullout dim |
|---|---|---|---|---|---|---|---|---|---|---|
| 12 | 2.25 | 2 | 1.270 | 0.465 | 0.590 | 8.850 | 1.71 | 15.41 | 2.740 | 4.08 |
| 11 | 3.75 | 2 | 1.520 | 0.454 | 0.690 | 6.900 | 2.19 | 15.41 | 2.560 | 5.53 |
| 10 | 6.25 | 2 | 1.940 | 0.436 | 0.850 | 10.625 | 1.42 | 15.41 | 2.270 | 3.80 |
| 9 | 8.75 | 2 | 2.360 | 0.419 | 0.990 | 12.375 | 1.22 | 15.41 | 1.980 | 3.31 |
| 8 | 11.25 | 2 | 2.780 | 0.401 | 1.120 | 14.000 | 1.08 | 15.41 | 1.690 | 2.85 |
| 7 | 13.75 | 3 | 3.210 | 0.383 | 1.230 | 15.375 | 1.48 | 16.25 | 1.400 | 3.78 |
| 6 | 16.25 | 3 | 3.630 | 0.366 | 1.330 | 16.625 | 1.36 | 17.75 | 1.110 | 3.31 |
| 5 | 18.75 | 3 | 4.050 | 0.348 | 1.410 | 17.625 | 1.29 | 19.25 | 0.820 | 2.71 |

Table 8-7 Summary of Internal Stability for EarthTrac Element

| SR | $Z_i$ ft | $N_{SR}$ dim | $\sigma_v$ ksf | $K_r$ dim | $\sigma_H$ ksf | $T_{max}$ kip | CDR Rupture dim | $L_e$ Ft | F* dim | CDR Pullout dim |
|---|---|---|---|---|---|---|---|---|---|---|
| 4 | 21.25 | 3 | 4.470 | 0.339 | 1.520 | 19.000 | 1.19 | 20.75 | 0.670 | 2.39 |
| 3 | 23.75 | 3 | 4.890 | 0.339 | 1.660 | 20.750 | 1.09 | 22.25 | 0.670 | 2.52 |
| 2 | 26.25 | 3 | 5.320 | 0.339 | 1.800 | 22.500 | 1.01 | 23.75 | 0.670 | 2.65 |
| 1 | 28.75 | 4 | 5.740 | 0.339 | 1.950 | 24.375 | 1.24 | 25.25 | 0.670 | 3.69 |

## 9 MSEW Internal Stability Discussion

MSEW may underestimate or overestimate the CDR for pullout and rupture for the top 2 rows of soil reinforcing. This occurs because MSEW uses an average horizontal pressure at each level of soil reinforcing and is based on the programs definitions and the calculation of the Tributary Range. The Tributary Range is defined in MSEW by the variables Z-bottom and Z-top. (This definition and the corresponding values can be found by activating the MSEW Results and Analysis screen and then selecting Strength). For the top soil reinforcing element the Z-top elevation is defined as H and the Z-bottom elevation is defined as the average of the Metal Strip elevation for the top soil reinforcing layer and the second soil reinforcing layer (mid-point between each soil reinforcing element). For the first soil reinforcing layer in this example Z-top is equal to 30.00 feet and the top soil reinforcing layer is defined at elevation 27.75 (a depth of 2.25 feet from the top of wall). The second soil reinforcing layer from the top is defined at elevation 26.25 (a depth of 3.75 feet from the top of wall). Therefore, the Z-bottom elevation is the average of 26.25(ft) and 27.75(ft) or 27.00(ft). The tributary range is the difference between Z-top and Z-bottom or 3.00(ft).

In MSEW the horizontal stress for any level of soil reinforcing is equal to the average horizontal stress calculated in the Tributary Range. In other words, the horizontal stress is the average of the horizontal stress calculated at Z-top and the horizontal stress calculated at Z-bottom. The maximum tension force per foot of wall is equal to the average horizontal stress times the tributary range. In the program MSEW the calculated CDR for the top soil reinforcing element and the second soil reinforcing element is 1.899 and 2.150 respectively. In the program MSE-Pro the calculated CDR for the top soil reinforcing element and the second soil reinforcing element is 1.71 and 2.19 respectively. MSEW over estimates the CDR in the top row and underestimates the CDR in the second row. The same holds true for the pullout calculations.

MSE-Pro does not use the average horizontal pressure to calculate the maximum tensile force. MSE-Pro uses the procedure defined in AASHTO, which uses the actual location of the soil reinforcing and actual tributary area. The method used in MSE-Pro to determine the tributary area was defined in Section 8.1. The tributary area that each soil reinforcing element has to resist is defined as the mid-point distance between each soil reinforcing.

It is important to recognize how MSEW calculates the tension forces. It can underestimate or overestimate the CDR in soil reinforcing where the soil reinforcing spacing is not uniform. This discrepancy becomes clearer when traffic impact or when large horizontal loads are applied in MSEW.

# Appendix A
## RSW-Calc Output

# MSE CALCULATIONS

Segmental Concrete Panel System
EarthTrac Steel Strip Soil Reinforcing

# RSW-CALC

EarthTrac Verification Calculation
IDEA Submittal
2021

**INFINITE BACKSLOPE**

Submittal Calculations
LRFD Procedure
Simplified Method

Support Through Innovation

114 South Collins Street
Arlington, TX, 76034
817-223-0969

114 South Collins Street
Arlington, TX, 76034
817-223-0969

**Design Options**

- LRFD Procedure
- Simplified Method
- The Vertical Earth (EV) load factor is used for all internal loads - AASHTO 11.10.6.2.1-1
- The K- Ratio for is set to 1.7 to 1.2 - AASHTO 11.10.6.2.1
- The K-Ratios are calculated from the top of the structure or top of coping

| Load Factors | Maximum | Minimum |
|---|---|---|
| Load factor for vertical earth pressure (EV) | 1.35 | 1.00 |
| Load factor for earth surcharge (ES) | 1.50 | 0.75 |
| Load factor for traffic live load (LS) | 1.75 | 1.75 |
| Load factor for horizontal earth pressure (EH) | 1.50 | 0.90 |
| Load factor for seismic (EQ) | 1.00 | |
| Load factor for structural components (DC) | 1.25 | 0.90 |
| Load factor for vehicular impact (CT) | 1.00 | 1.00 |

| Resistance Factor | Static | Seismic |
|---|---|---|
| Bearing resistance factor | 1.00 | |
| Sliding resistance factor | 1.00 | |
| Tensile resistance factor | 0.75 | 0.75 |
| Pullout resistance factor | 0.90 | 1.20 |
| Extreme event resistance factor | 1.00 | |

| Internal Earth Pressure Ratio | | |
|---|---|---|
| **Depth (Z)** | **Rupture** | **Pullout** |
| 0.00 (ft) | 1.70 | 1.70 |
| 20.00 (ft) | 1.20 | 1.20 |

| Pullout Friction Factors | |
|---|---|
| **Depth (Z)** | **F*** |
| 0.00 (ft) | 3.00 |
| 20.00 (ft) | 0.67 |

**General Notes**

1. AASHTO LRFD Bridge Design Specification 7th Edition/2014
2. Course Aggregate
3. MSE Structure with Infinite Back-Slope
4. EarthTrac Soil Reinforcing
5. F*= 3.00 to 0.67

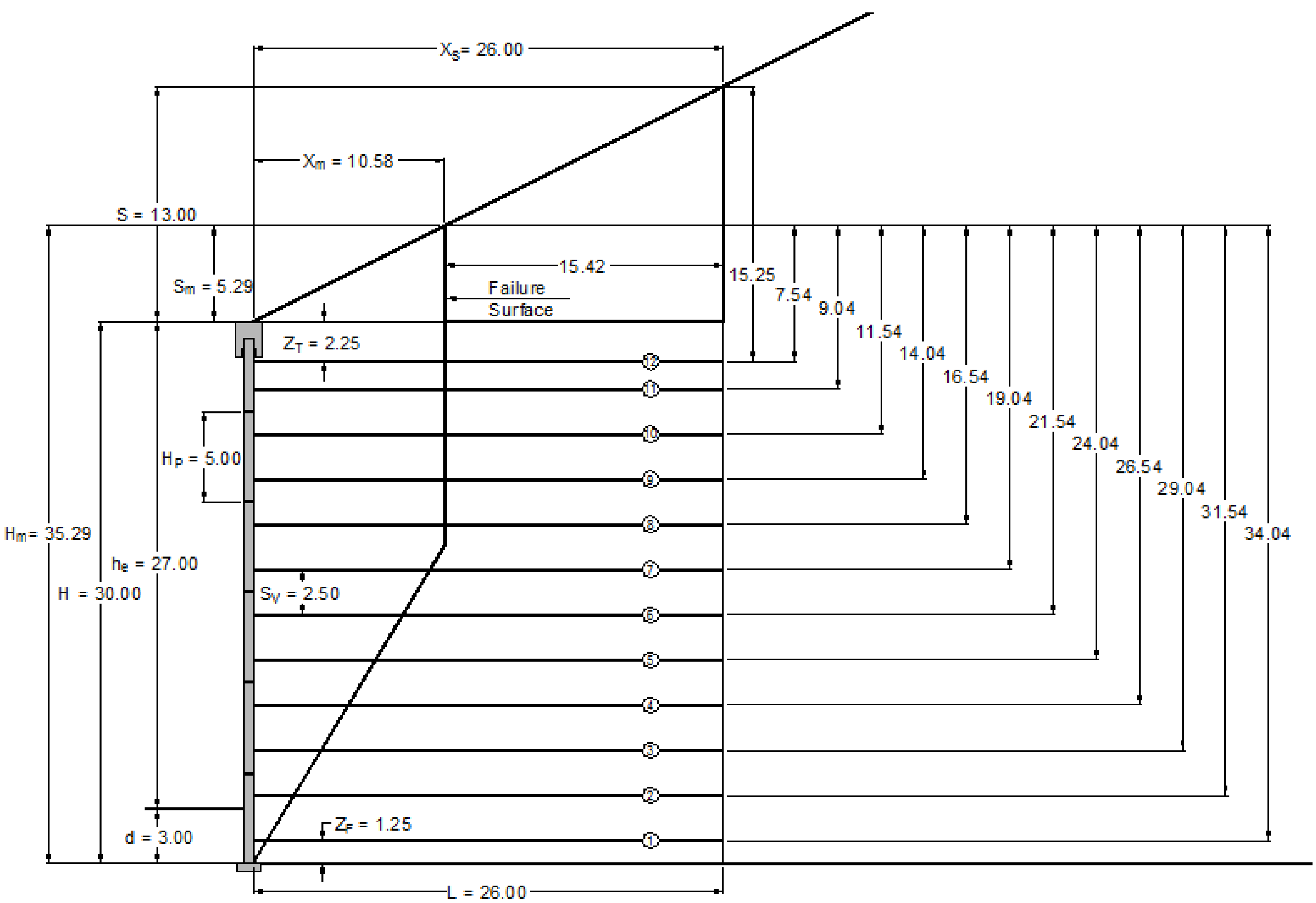
**System Design Parameters**

Steel Yield Stress: 50.000 (ksi)
Yield Coefficient: 0.750
Design Life: 75.0 years

| Soil Reinforcing Schedule - EarthTrac Steel Strip Soil Reinforcing | | | | |
|---|---|---|---|---|
| **Type** | **Minimum Per Row** | **Thickness (in)** | **Width (in)** | **Design Area (in$^2$)** |
| 1 | 2 | 0.156 | 2.000 | 0.202 |

| Soil Parameters | | |
|---|---|---|
| **Soil Region** | **Unit Weight (kcf)** | **Friction Angle (deg)** |
| MSE Backfill | 0.125 | 34 |
| Retained Backfill | 0.125 | 34 |
| Foundation Backfill | 0.125 | 30 |
| Earth Surcharge | 0.125 | 34 |

| Rec. #1 Structure Parameters | | | | | | |
|---|---|---|---|---|---|---|
| **H max (ft)** | **B min (ft)** | **B Ext. (ft)** | **S (ft)** | **Xs (ft)** | **β (deg)** | **βi (deg)** |
| 30.000 | 26.000 | 26.000 | 13.00 | 26.00 | 26.57 | 26.57 |




**Cross Section**

**External and Internal Stability Calculation Summary**

| Rec. #1 External Stability CDR Summary (Effective Stress) | | | | | | |
|---|---|---|---|---|---|---|
| | CDR Sliding | Limiting Eccentricity (ft) | CDR Overturning | Bearing Pressure (ksf) | Eccentricity (ft) | Cw |
| Strength 1 - Min | 2.10 | 1.49 | 3.92 | 5.97 | 1.49 | 0.500 |
| Strength 1 - Max | 1.76 | 1.93 | 3.35 | 8.66 | 1.93 | 0.500 |
| Strength 1 - Critical | 1.38 | 2.68 | 2.72 | 7.27 | 2.68 | 0.500 |
| Service | 1.92 | 1.70 | 3.62 | 6.18 | 1.70 | 0.500 |

| Rec. #1 Internal Stability CDR Summary | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Depth Below Top of Wall (ft) | B min (ft) | Type | Strips | Strip Thickness (in) | Internal Earth Coeff. | Rupture CDR | Pullout CDR | F* | Le (ft) | H/B Ratio |
| 15.25 | 26.00 | Type-1 | 2 | 0.156 | 0.46 | 1.71 | 4.09 | 2.74 | 15.41 | 0.87 |
| 16.75 | 26.00 | Type-1 | 2 | 0.156 | 0.45 | 2.19 | 5.54 | 2.56 | 15.41 | 0.87 |
| 19.25 | 26.00 | Type-1 | 2 | 0.156 | 0.44 | 1.43 | 3.82 | 2.27 | 15.41 | 0.87 |
| 21.75 | 26.00 | Type-1 | 2 | 0.156 | 0.42 | 1.22 | 3.31 | 1.98 | 15.41 | 0.87 |
| 24.25 | 26.00 | Type-1 | 2 | 0.156 | 0.40 | 1.08 | 2.85 | 1.69 | 15.41 | 0.87 |
| 26.75 | 26.00 | Type-1 | 3 | 0.156 | 0.38 | 1.48 | 3.77 | 1.40 | 16.25 | 0.87 |
| 29.25 | 26.00 | Type-1 | 3 | 0.156 | 0.37 | 1.37 | 3.31 | 1.11 | 17.75 | 0.87 |
| 31.75 | 26.00 | Type-1 | 3 | 0.156 | 0.35 | 1.29 | 2.70 | 0.82 | 19.25 | 0.87 |
| 34.25 | 26.00 | Type-1 | 3 | 0.156 | 0.34 | 1.20 | 2.40 | 0.67 | 20.75 | 0.87 |
| 36.75 | 26.00 | Type-1 | 3 | 0.156 | 0.34 | 1.09 | 2.52 | 0.67 | 22.25 | 0.87 |
| 39.25 | 26.00 | Type-1 | 3 | 0.156 | 0.34 | 1.01 | 2.65 | 0.67 | 23.75 | 0.87 |
| 41.75 | 26.00 | Type-1 | 4 | 0.156 | 0.34 | 1.24 | 3.70 | 0.67 | 25.25 | 0.87 |

# Appendix B
## MSEW Output

# AASHTO 2007-2010 (LRFD)
# Design Case - Infinite Back-Slope

MSEW(3.0): Update # 14.981

## PROJECT IDENTIFICATION

| | |
|---|---|
| Title: | Design Case - Infinite Back-Slope |
| Project Number: | EarthTrac |
| Client: | LRFD Binder - Infinite Back-Slope |
| Designer: | tpt |
| Station Number: | 1 |

### Description:

LRFD

### Company's information:

Name: Ground Improvement Systems LLC
Street: 114 South Collins Street

Arlington, TX 76011
Telephone #: 817-223-0969
Fax #:
E-Mail: tptaylor.pe@gmail.com

**Original file path and name:** H:\Toms Computer\AB My Data\DOT_AB LRFD Binder\1. Eart..... .....RFD-DOT Infinite.BEN

**Original date and time of creating this file:** 03/13/2021

**PROGRAM MODE:** ANALYSIS
of a SIMPLE STRUCTURE
using METAL STRIPS as reinforcing material.

## SOIL DATA

REINFORCED SOIL

| | |
|---|---|
| Unit weight, $\gamma$ | 125.0 lb/ft ³ |
| Design value of internal angle of friction, $\phi$ | 34.0 ° |

RETAINED SOIL

| | |
|---|---|
| Unit weight, $\gamma$ | 125.0 lb/ft ³ |
| Design value of internal angle of friction, $\phi$ | 34.0 ° |

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) = 0.4062 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

## BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 30.14 N $\gamma$= 22.40

## SEISMICITY

Not Applicable

## INPUT DATA: Metal strips (Analysis)

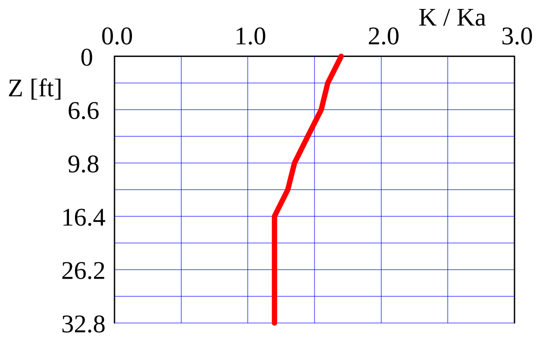
| D A T A | Metal strip type #1 | Metal strip type #2 | Metal strip type #3 | Metal strip type #4 | Metal strip type #5 |
|---|---|---|---|---|---|
| Yield strength of steel, Fy [kips/in ²] | 50.0 | N/A | N/A | N/A | N/A |
| Gross width of strip, b [in] | 2.0 | N/A | N/A | N/A | N/A |
| Vertical spacing, 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 | 60.97 | 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 | 3.00 | 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

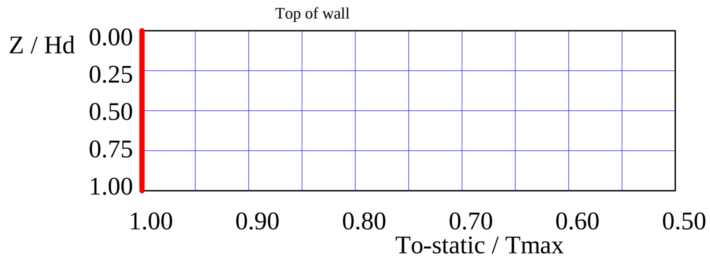
| Z | K / Ka |
|---|---|
| 0 ft | 1.70 |
| 3.3 ft | 1.60 |
| 6.6 ft | 1.55 |
| 9.8 ft | 1.45 |
| 13.1 ft | 1.35 |
| 16.4 ft | 1.30 |
| 19.7 ft | 1.20 |



## 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 $\gamma_f$ = 152.78 lb/ft $^3$

| Z / Hd | To-static / Tmax |
|---|---|
| 0.00 | 1.00 |
| 0.25 | 1.00 |
| 0.50 | 1.00 |
| 0.75 | 1.00 |
| 1.00 | 1.00 |



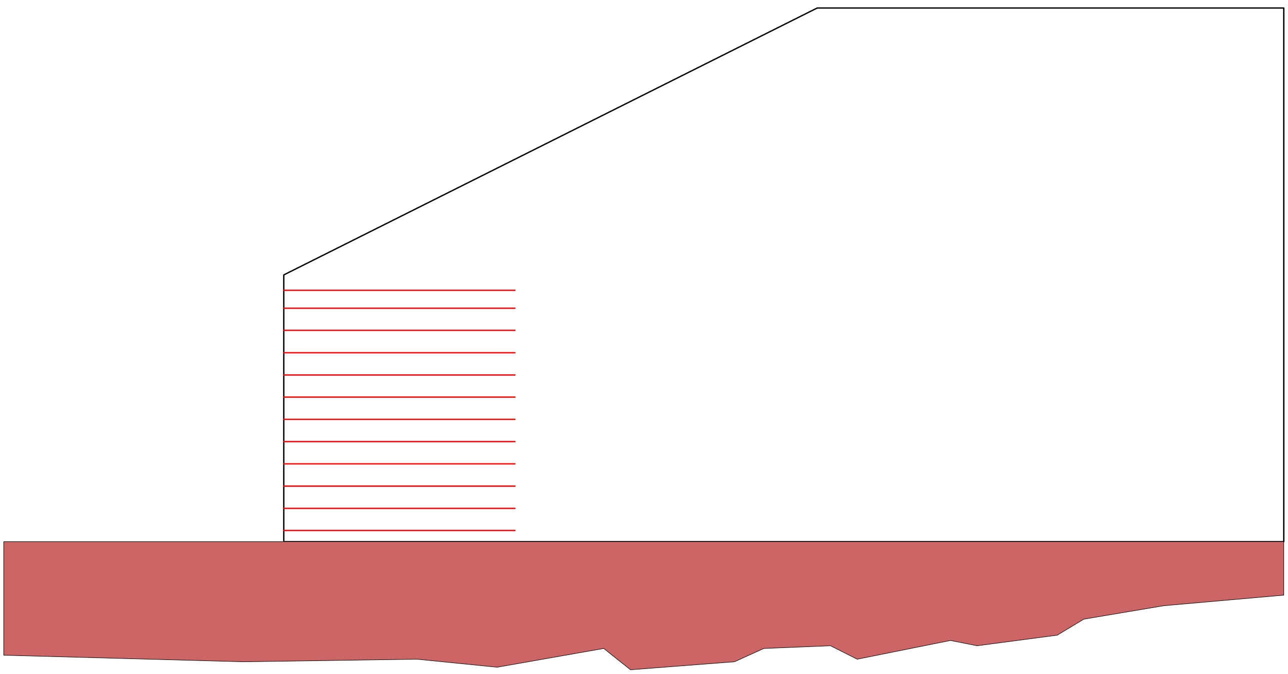

| D A T A (for connection only) | Type #1 | Type #2 | Type #3 | Type #4 | Type #5 |
|---|---|---|---|---|---|
| Product Name | SS3 | N/A | N/A | N/A | N/A |
| Strength reduction at the connection, CRu = Fyc / Fy | 1.00 | N/A | N/A | N/A | N/A |

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

| | | | |
|---|---|---|---|
| Design height, Hd | 30.00 | [ft] | { Embedded depth is E = 0.00 ft, and height above top of finished bottom grade is H = 30.00 ft } |
| Batter, ω | 0.0 | [deg] | |
| Backslope, β | 26.6 | [deg] | |
| Backslope rise | 30.0 | [ft] | Broken back equivalent angle, I = 26.56° (see Fig. 25 in DEMO 82) |

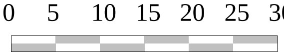
U N I F O R M   S U R C H A R G E
Uniformly distributed dead load is 0.0 [lb/ft ²]

## ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 5 10 15 20 25 30 [ft]



## AASHTO 2007-2010 (LRFD) Input Data

### INTERNAL STABILITY

| | | | |
|---|---|---|---|
| Load factor for vertical earth pressure, EV, from Table 3.4.1-2: | $\gamma_{p\text{-}EV}$ | 1.35 | |
| Load factor for earthquake loads, EQ, from Table 3.4.1-1: | $\gamma_{p\text{-}EQ}$ | 1.00 | |
| Load factor for live load surchrge, LS, from Figure C11.5.5-3(b): (Same as in External Stability). | $\gamma_{p\text{-}LS}$ | 1.75 | |
| Load factor for dead load surchrge, ES: (Same as in External Stability). | $\gamma_{p\text{-}ES}$ | 1.35 | |
| Resistance factor for reinforcement tension from Table 11.5.6-1: | $\phi$ | Static | Combined static/seismic |
| Metal Strips: | | 0.75 | 1.00 |
| Resistance factor for reinforcement tension in connectors from Table 11.5.6-1: | $\phi$ | Static | Combined static/seismic |
| Metal Strips: | | 0.75 | 1.00 |
| Resistance factor for reinforcement pullout from Table 11.5.6-1: | $\phi$ | 0.90 | 1.20 |

### EXTERNAL STABILITY

| | | | | |
|---|---|---|---|---|
| Load factor for vertical earth pressure, EV, from Table 3.4.1-2 and Figure C11.5.5-2: | | 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, from Table 3.4.1-2 and Figure C11.5.5-2: | $\gamma_{p\text{-}EH}$ | 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, from Table 3.4.1-1 (multiplies $P_{AE}$ and $P_{IR}$ ): | $\gamma_{p\text{-}EQ}$ | 1.00 |

| | | | |
|---|---|---|---|
| Resistance factor for shear resistance along common interfaces from Table 11.5.6-1: | | Static | Combined Static/Seismic |
| Reinforced Soil and Foundation | $\phi_\tau$ | 1.00 | 1.00 |
| Reinforced Soil and Reinforcement | $\phi_\tau$ | 1.00 | 1.00 |
| Resistance factor for bearing capacity of shallow foundation from Table 11.5.6-1: | | Static | Combined Static/Seismic |
| | $\phi_b$ | 0.65 | 0.65 |

## ANALYSIS: CALCULATED FACTORS (Static conditions)

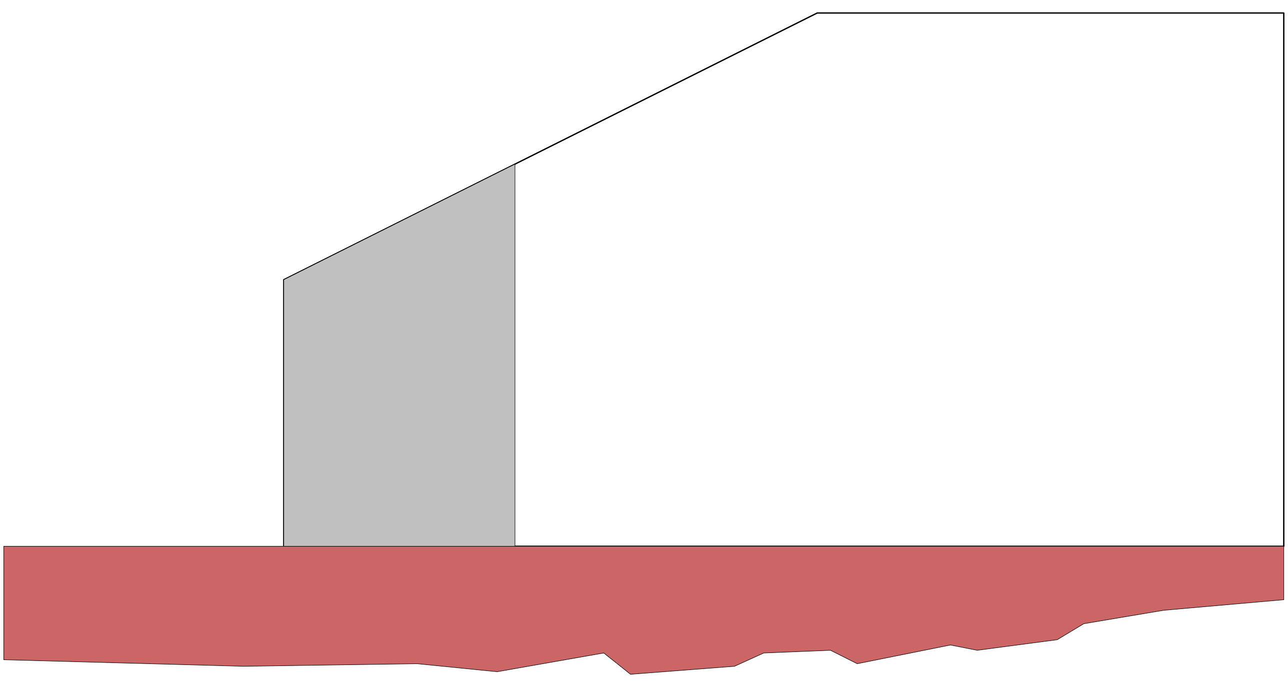
Bearing capacity, CDR = 2.33, factored bearing load = 8654 lb/ft².
Foundation Interface: Direct sliding, CDR = 1.376, Eccentricity, e/L = 0.1029, CDR-overturning = 2.72

| METAL STRIP | | | | CONNECTION | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| # | Elevation [ft] | Length [ft] | Type # | CDR [pullout resistance] | CDR [connection break] | CDR [metal strip strength] | Metal strip strength CDR | Pullout resistance CDR | Direct sliding CDR | Eccentricity e/L | Product name |
| 1 | 1.25 | 26.00 | 1 | N/A | 1.24 | 1.24 | 1.239 | 3.695 | 1.639 | 0.0930 | SS3 |
| 2 | 3.75 | 26.00 | 1 | N/A | 1.00 | 1.00 | 1.001 | 2.640 | 1.706 | 0.0736 | SS3 |
| 3 | 6.25 | 26.00 | 1 | N/A | 1.09 | 1.09 | 1.087 | 2.515 | 1.779 | 0.0548 | SS3 |
| 4 | 8.75 | 26.00 | 1 | N/A | 1.19 | 1.19 | 1.190 | 2.392 | 1.860 | 0.0366 | SS3 |
| 5 | 11.25 | 26.00 | 1 | N/A | 1.28 | 1.28 | 1.280 | 2.578 | 1.950 | 0.0189 | SS3 |
| 6 | 13.75 | 26.00 | 1 | N/A | 1.36 | 1.36 | 1.362 | 3.212 | 2.049 | 0.0014 | SS3 |
| 7 | 16.25 | 26.00 | 1 | N/A | 1.48 | 1.48 | 1.482 | 3.731 | 2.159 | -0.0161 | SS3 |
| 8 | 18.75 | 26.00 | 1 | N/A | 1.09 | 1.09 | 1.091 | 2.845 | 2.280 | -0.0340 | SS3 |
| 9 | 21.25 | 26.00 | 1 | N/A | 1.22 | 1.22 | 1.220 | 3.286 | 2.412 | -0.0530 | SS3 |
| 10 | 23.75 | 26.00 | 1 | N/A | 1.42 | 1.42 | 1.423 | 3.794 | 2.552 | -0.0746 | SS3 |
| 11 | 26.25 | 26.00 | 1 | N/A | 1.93 | 1.93 | 1.933 | 4.882 | 2.689 | -0.1013 | SS3 |
| 12 | 28.25 | 26.00 | 1 | N/A | 2.10 | 2.10 | 2.103 | 4.904 | 2.778 | -0.1302 | SS3 |

## BEARING CAPACITY for GIVEN LAYOUT

| | STATIC | SEISMIC | UNITS |
|---|---|---|---|
| (Water table does not affect bearing capacity) | | | |
| Factored bearing resistance. q-n | 20150 | N/A | [lb/ft ²] |
| Factored bearing load, $\sigma_V$ | 8654.3 | N/A | [lb/ft ²] |
| Eccentricity, e | 1.93 | N/A | [ft] |
| Eccentricity, e/L | 0.074 | N/A | |
| CDR calculated | 2.33 | N/A | |
| Base length | 26.00 | N/A | [ft] |

Unfactored applied bearing pressure = (Unfactored R) / [ L - 2 * (Unfactored e) ] =
Unfactored R = 139605.10 [lb/ft], L = 26.00, Unfactored e = 1.70 [ft], and Sigma = 6177.24 [lb/ft ²]



SCALE:

0 5 10 15 20 25 30 [ft]

### DIRECT SLIDING for GIVEN LAYOUT (for METAL STRIPS reinforcements)

Along reinforced and foundation soils interface: CDR-static = 1.376

| # | Metal strip Elevation [ft] | Metal strip Length [ft] | CDR Static | CDR Seismic | Metal strip Type # | Product name |
|---|---|---|---|---|---|---|
| 1 | 1.25 | 26.00 | 1.639 | N/A | 1 | SS3 |
| 2 | 3.75 | 26.00 | 1.706 | N/A | 1 | SS3 |
| 3 | 6.25 | 26.00 | 1.779 | N/A | 1 | SS3 |
| 4 | 8.75 | 26.00 | 1.860 | N/A | 1 | SS3 |
| 5 | 11.25 | 26.00 | 1.950 | N/A | 1 | SS3 |
| 6 | 13.75 | 26.00 | 2.049 | N/A | 1 | SS3 |
| 7 | 16.25 | 26.00 | 2.159 | N/A | 1 | SS3 |
| 8 | 18.75 | 26.00 | 2.280 | N/A | 1 | SS3 |
| 9 | 21.25 | 26.00 | 2.412 | N/A | 1 | SS3 |
| 10 | 23.75 | 26.00 | 2.552 | N/A | 1 | SS3 |
| 11 | 26.25 | 26.00 | 2.689 | N/A | 1 | SS3 |
| 12 | 28.25 | 26.00 | 2.778 | N/A | 1 | SS3 |

### ECCENTRICITY for GIVEN LAYOUT (for Simplified Method)

At interface with foundation: e/L static = 0.1029; Overturning: CDR-static = 2.72

| # | Metal strip Elevation [ft] | Metal strip Length [ft] | e / L Static | e / L Seismic | Metal strip Type # | Product name |
|---|---|---|---|---|---|---|
| 1 | 1.25 | 26.00 | 0.0930 | N/A | 1 | SS3 |
| 2 | 3.75 | 26.00 | 0.0736 | N/A | 1 | SS3 |
| 3 | 6.25 | 26.00 | 0.0548 | N/A | 1 | SS3 |
| 4 | 8.75 | 26.00 | 0.0366 | N/A | 1 | SS3 |
| 5 | 11.25 | 26.00 | 0.0189 | N/A | 1 | SS3 |
| 6 | 13.75 | 26.00 | 0.0014 | N/A | 1 | SS3 |
| 7 | 16.25 | 26.00 | -0.0161 | N/A | 1 | SS3 |
| 8 | 18.75 | 26.00 | -0.0340 | N/A | 1 | SS3 |
| 9 | 21.25 | 26.00 | -0.0530 | N/A | 1 | SS3 |
| 10 | 23.75 | 26.00 | -0.0746 | N/A | 1 | SS3 |
| 11 | 26.25 | 26.00 | -0.1013 | N/A | 1 | SS3 |
| 12 | 28.25 | 26.00 | -0.1302 | N/A | 1 | SS3 |

RESULTS for STRENGTH    [ Note: Actual CDR = (Yield stress) / (Actual stress) ]    For Simplified Method
Live Load included in calculating Tmax

| # | Metal strip Elevation [ft] | Coverage ratio, Rc=b/Sh | Horizontal spacing, Sh [ft] | Long-term strength 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 | 1.25 | 0.133 | 1.250 | 6030 | 4866.06 | N/A | N/A | 1.239 | N/A | N/A |
| 2 | 3.75 | 0.100 | 1.670 | 4513 | 4508.25 | N/A | N/A | 1.001 | N/A | N/A |
| 3 | 6.25 | 0.100 | 1.670 | 4513 | 4150.44 | N/A | N/A | 1.087 | N/A | N/A |
| 4 | 8.75 | 0.100 | 1.670 | 4513 | 3792.63 | N/A | N/A | 1.190 | N/A | N/A |
| 5 | 11.25 | 0.100 | 1.670 | 4513 | 3525.17 | N/A | N/A | 1.280 | N/A | N/A |
| 6 | 13.75 | 0.100 | 1.670 | 4513 | 3313.96 | N/A | N/A | 1.362 | N/A | N/A |
| 7 | 16.25 | 0.100 | 1.670 | 4513 | 3044.67 | N/A | N/A | 1.482 | N/A | N/A |
| 8 | 18.75 | 0.067 | 2.500 | 3015 | 2763.24 | N/A | N/A | 1.091 | N/A | N/A |
| 9 | 21.25 | 0.067 | 2.500 | 3015 | 2470.91 | N/A | N/A | 1.220 | N/A | N/A |
| 10 | 23.75 | 0.067 | 2.500 | 3015 | 2118.58 | N/A | N/A | 1.423 | N/A | N/A |
| 11 | 26.25 | 0.067 | 2.500 | 3015 | 1559.55 | N/A | N/A | 1.933 | N/A | N/A |
| 12 | 28.25 | 0.067 | 2.500 | 3015 | 1433.37 | N/A | N/A | 2.103 | N/A | N/A |

RESULTS for PULLOUT    Live Load NOT included in calculating Tmax

| # | Metal strip Elevation [ft] | Coverage Ratio Rc=b/Sh | 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 | 1.25 | 0.133 | 4866.1 | N/A | 25.25 | 0.75 | 17982.1 | N/A | 3.695 | N/A | N/A | N/A |
| 2 | 3.75 | 0.100 | 4508.3 | N/A | 23.75 | 2.25 | 11900.7 | N/A | 2.640 | N/A | N/A | N/A |
| 3 | 6.25 | 0.100 | 4150.4 | N/A | 22.25 | 3.75 | 10437.7 | N/A | 2.515 | N/A | N/A | N/A |
| 4 | 8.75 | 0.100 | 3792.6 | N/A | 20.75 | 5.25 | 9070.7 | N/A | 2.392 | N/A | N/A | N/A |
| 5 | 11.25 | 0.100 | 3525.2 | N/A | 19.25 | 6.75 | 9087.9 | N/A | 2.578 | N/A | N/A | N/A |
| 6 | 13.75 | 0.100 | 3314.0 | N/A | 17.75 | 8.25 | 10644.1 | N/A | 3.212 | N/A | N/A | N/A |
| 7 | 16.25 | 0.100 | 3044.7 | N/A | 16.25 | 9.75 | 11358.7 | N/A | 3.731 | N/A | N/A | N/A |
| 8 | 18.75 | 0.067 | 2763.2 | N/A | 15.41 | 10.59 | 7861.0 | N/A | 2.845 | N/A | N/A | N/A |
| 9 | 21.25 | 0.067 | 2470.9 | N/A | 15.41 | 10.59 | 8120.6 | N/A | 3.286 | N/A | N/A | N/A |
| 10 | 23.75 | 0.067 | 2118.6 | N/A | 15.41 | 10.59 | 8038.3 | N/A | 3.794 | N/A | N/A | N/A |
| 11 | 26.25 | 0.067 | 1559.5 | N/A | 15.41 | 10.59 | 7614.3 | N/A | 4.882 | N/A | N/A | N/A |
| 12 | 28.25 | 0.067 | 1433.4 | N/A | 15.41 | 10.59 | 7028.9 | N/A | 4.904 | N/A | N/A | N/A |

# 3.0
# CONSTRUCTION

EARTHTEC



GROUND IMPROVEMENT SYSTEMS LLC
114 SOUTH COLLINS STREET
ARLINGTON, TX 76011

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## 3.1
## CONSTRUCTION PROCEDURES

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**3.1.1**
**ERS Construction Innovations**

EARTHTEC

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# 3.1.2
# Construction Manual



GROUND IMPROVEMENT SYSTEMS LLC
114 SOUTH COLLINS STREET
ARLINGTON, TX 76011

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# EARTHTEC REINFORCED SOIL WALL

# QUALITY CONTROL

# INSTALLATION GUIDELINES


GROUND IMPROVEMENT SYSTEMS LLC
114 South Collins Street
Arlington, TX 76011
www.groundimprovementsystems.com


Support Through Innovation

## Table of Contents

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## 1 INTRODUCTION

The following manual is intended as a general guide for the construction of EarthTec Reinforced Soil Wall system using the EarthTrac SS™ soil reinforcing. The system incorporates precast panel facings and ribbed steel soil reinforcing strips in compacted select backfill. Before commencement of construction, the Contractor shall review the contents of this manual, submitted project drawings, contract specifications, and associate contract plans.

This document is not intended to replace the specification for any project. When there is a conflict between the contract specification and this document, the contract specification shall take precedence. Further, the Contractor or installer is completely responsible to develop their own means and methods of installation.

GIS will provide on-site technical assistance at the start of construction and construction advice throughout the period of construction as required by the contract documents.

If any further information is required, please contact GIS.

### 1.1 Associated Documents

The following documents should be referenced:

a) EarthTec Reinforced Soil Wall Specification.
b) EarthTec project drawings in their latest approved revisions.
c) Approved project specifications.
d) Contract drawings in their latest approved revisions.

### 1.2 Design Management

The project drawings will contain notes and construction details that are provided to ensure that the wall contractor is aware of the assumptions made in design. The Contractor shall be responsible for constructing the structure in accordance with the design requirements as well as coordination with other trades and activities within and associated with the MSE wall construction area.

The Contractor is solely responsible for adherence to all applicable safety requirements and quality control procedures.

Where appropriate, notes may also be provided to assist the Contractor in installing the structures safely and in accordance with relevant safety laws and best practice. All notes given on relevant drawings shall be read and understood before any construction work commences.

### 1.3 Disclaimer

The contents of this document shall be read as general in nature. This document does not contain project specific advice, and its contents should not be a substitute for professional or technical advice. It is the responsibility of the Contractor to construct the MSE structures in accordance with the specification and contract documents. Reliance on any information contained in this manual is solely at the readers risk and Ground Improvement Systems, LLC makes no warranty either expressed or implied as to the suitability of the methods described herein to any specific project, or generally. All means and methods used to install the MSE structure is by others.

## 2 CONSTRUCTION GUIDELINES

This section provides general guidelines for the construction of the EarthTec Reinforced Soil Wall using the EarthTrac SS™ soil reinforcing.

### 2.1 Skills Required

Effective construction of the EarthTec Reinforced Soil Wall using the EarthTrac SS™ soil reinforcing requires the following elements:

- Proper timing of material deliveries and appropriate construction staging.
- Sourcing of appropriate fill materials.
- Selection of appropriate concrete plant and effective concrete plant utilization for precast panels.
- Selection of appropriately skilled labor.
- Documented quality control procedures.
- Trained and qualified site labor and supervision.
- Coordination of installation scheduling with other trades such as utilities, piles, and site work within the MSE wall construction area.

### 2.2 Supply of Equipment, Labor and Materials

The Contractor needs to be aware of what is provided by ErthTec Vendor for the Reinforced Soil Wall using the EarthTrac SS™ soil reinforcing and what materials are provided by the Contractor.

#### 2.2.1 Materials and Components supplied by EarthTec Vendor

a) Precast concrete facing panels.
b) Hard SRB bearing pads for horizontal panel joints (used from the second course of panels upwards).
c) 12" Geotextile strip in rolls, for placing against panel joints.
d) EarthTrac SS™ ribbed steel soil reinforcing strips, supplied cut to length, and galvanized.
e) ½" nuts and bolts complying with ASTM F3125 Grade A325.
f) Any other item referred to as being supplied on contract drawings.
g) Site assistance and training.

Training by the EarthTec representative is advisory in nature. EarthTec assumes no responsibility for the quality of the Contractor's means and methods or installation practices.

#### 2.2.2 Materials and Components supplied by the Contractor

a) All labor and supervision.
b) All safety equipment.
c) Project specific safety plan.
d) All tools, construction, earthmoving, lifting and compaction equipment as outlined in the General Notes section of the design.
e) Backfill following the Project Specification.
f) Materials to install all required drainage systems.
g) Concrete and forming materials to install the leveling pad.
h) Material to install parapet or copings.
i) Timber braces for first row of panels.
j) Large and small crowbar for adjusting panels.
k) Timber clamps for temporarily clamping each row of panels until backfilled (1 per vertical joint).
l) Hardwood wedges for maintaining panel batter during setting of panels prior to clamping
m) ¾" spacers (2 per panel)
n) Rubber or asphalt shims.
o) 4' and 8' Spirit level.
p) Site level and rod.
q) Plumb bob.
r) Adhesive for securing geotextile at panel joints.
s) Corner or slip joint backing blocks.

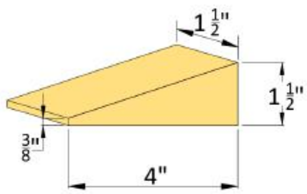



Figure 2-1. Hardwood Wedges

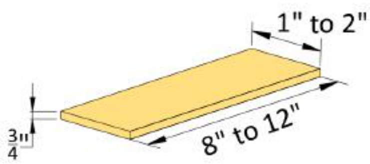



Figure 2-2. Hardwood Shims or Plastic

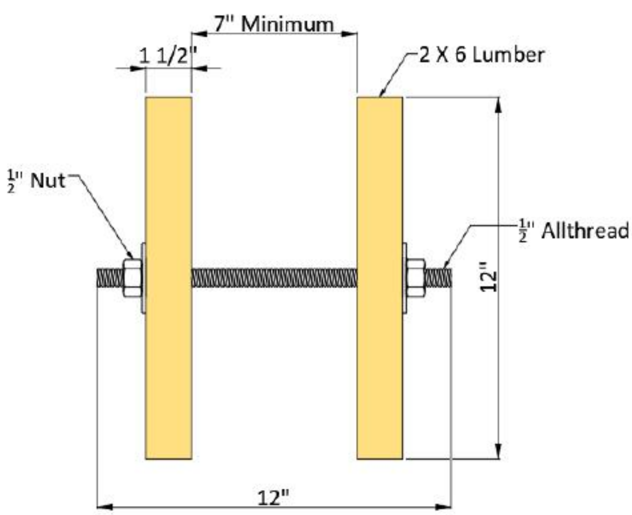



Figure 2-3. Wedges and Clamp

## 2.3 Site Preparation

### 2.3.1 Laying Out Wall (Staking)

Lay-out information is given on the construction drawings. Lay-out information may include coordinates or may be given in terms of stations and offsets from contract information.

It is the Contractor's responsibility to ensure proper alignment and adherence to the lines and grades shown on the contract drawings.

Based on lay-out information, string lines or pegs should be established to define wall lines and the width of the leveling pad. If the walls interface with an existing structure, then a plumb line should also be dropped from the top of the existing structure to define the vertical limit of the MSE panels. Pegs or profiles shall be used to establish excavation levels.

It should be noted that the elevation given as top of leveling pad on the construction drawings coincides with the founding elevation of the MSE structure. Thus, the elevation of the bottom of the leveling pad must be calculated from the elevations and dimensions provided on the drawings.

### 2.3.2 Preparation of Foundation Soils

The foundation material shall be consistent with the site investigation reports and any specific requirements stated on the drawings. Any unsuitable material that is within the foundation area (plan area of the wall) shall be removed (undercut) and replaced with compacted granular fill. The undercut and replacement (if required) shall be performed as directed by the Engineer.

The foundation area (measured from the back of the panel to the terminal end of the soil reinforcement) shall be proof rolled under the supervision of the Engineer or otherwise as provided in the contract quality plan.

Figure 2-4. Foundation Preparation

Approval of the foundation and permission to commence the wall installation and backfilling may be subject to approval by the Engineer.

## 2.4 General Wall Installation Procedure

An inspection should be performed before beginning the installation process to ensure all materials meet the specifications. Any damaged or defective materials are repaired or replaced before being placed in the wall.

It is recommended that a preconstruction meeting attended by the Contractor, Engineer, and EarthTec representative be held prior to work commencement.

The following general guidelines are outlined for the installation of the EarthTec Reinforced Soil Wall system:

- Construct leveling pads
- Establish the wall line
- Establish the extent of soil reinforcing
- Install the first panels
- Apply geotextile
- First stage backfilling
- Second stage backfilling
- Placement of Intermediate panels
- Placement of top panels
- Placement of leveling concrete on top panel
- Placement of coping unit

### 2.4.1 Construct Leveling Pads

Leveling pads may be constructed using unreinforced concrete. Side forms shall be used to place the concrete in an excavated trench. The top of the leveling pad shall be trowelled smooth to ensure that a consistent level surface is provided for the first course of panels. The top of the leveling pad shall be constructed to within $\pm$1/8" (3mm) of the specified elevation.

Figure 2-5. Leveling Pad at Step

It is critical that the levelling pad be cast or placed within the specified tolerances to establish proper vertical and horizontal alignment of subsequent panels. Widder leveling pads may be required and used at corners and curved sections to assure that the panel rests completely on the concrete leveling pad.

Figure 2-6. Leveling Pad at Corner

### 2.4.2 Establishing the Wall Line

The front face of the wall shall be established with a chalk line drawn on the top surface of the leveling pad. This line represents the front face of the panels, including any special finish. Additional points should be located at all bends, points of curvature or tangency of curves, changes in soil reinforcing lengths, etc.

### 2.4.3 Installing First Panels

The first panels will be installed along the bottom row directly on top of the leveling pad. Bearing pads are not installed below the first row of panels. If the wall interfaces with an existing structure, it is recommended to begin the panel installation from the existing structure.

Rectangular panels may be set out sequentially in alternating full-height and half-height pattern. A 3/4" spacer guide shall be used to set the required ¾" vertical joint space between panels.

The panels should be checked for alignment and battered back, with wedges. Typically, a minimum of ¼" (5mm) batter over the standard G (5' tall) panel allows for the effects of compaction. The actual batter required is a matter of trial and error based on the specific fill material used and various compaction regimes' effects. The ¼" defined above is only a recommendation. In general, finer fill materials require greater initial panel batter. The vertical joints should be checked for consistency and are ¾". The horizontal panel surface should be checked for levelness to ensure that subsequent panels can be positioned to correct line and level.

Figure 2-7. Leveling Pad Step

The bottom course must be supported vertical using 2" x 4" wood bracing after the appropriate batter is achieved. Plastic or asphalt shims should be installed, if necessary, to level the first row of panels. *Wooden shims may not be used to level the panel.*

Figure 2-8. Bottom Panel Bracing

The panel sequence for the first row of panels typically consists of a half-panel (Type-B) followed by a full panel (Type-G), or a full-panel (Type-G) flowed by a one and one-half panel (Type-M). Once the panel is securely held in position with proper clamping and front-support, the lifting mechanisms can be removed.

Figure 2-9. Setting and Clamping Panel

Figure 2-10. Typical Wood Panel Clamp

Figure 2-11. Bottom Panels Braced and Clamped

### 2.4.4 Applying Geotextile

Apply the 12" strips of geotextile over all the vertical and horizontal panel joints on the panel's back face and secure with an adhesive.

It is important to apply the adhesive to the panel and not to the geotextile filter fabric. The geotextile must remain porous and allow water to pass through the joint. It is not necessary to place adhesive entirely on the panel joints.

Apply enough adhesive that holds the geotextile in position to prevent it from becoming wrinkled, dislodged, or folded during the backfill placement.

The geotextile should be placed continuously. If the geotextile is not continuously placed, the geotextile shall be overlapped (shingled) a minimum of six inches between geotextile pieces.

The adhesive shall be a type that can be used on concrete surfaces and the geotextile. Subfloor adhesives such as products by Franklin and Dap are recommended.

### 2.4.5 First Stage Backfilling

The granular MSE fill should be placed, leveled, and compacted following the project specifications. This includes the required lift thickness. This first backfill stage shall be placed to the level of the first row of panel anchors, i.e., the elevation of the first soil reinforcing. The backfill shall be level with the panel anchor. The backfill shall be leveled so that all soil reinforcing layers are fixed at the required elevations and placed on compacted fill.

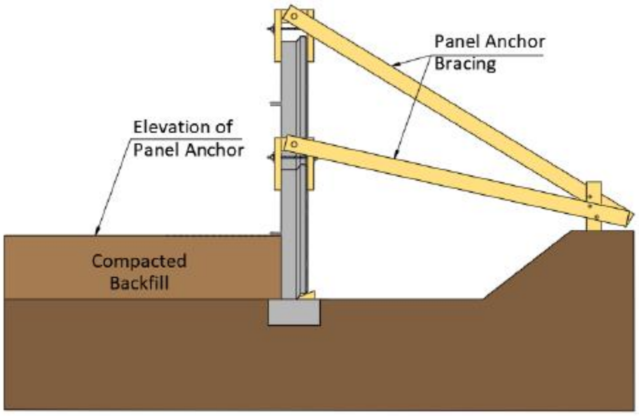



Figure 2-12. First Stage Backfill

Backfilling shall be carried out in a direction parallel to the facing panels and shall be carried out in stages to alternate with the placing and attachment of the soil reinforcing to the anchor on the facing panels.

Care should be exercised when compacting the backfill directly behind the panel until the first soil reinforcing element has been secured to the panel.

Care shall be taken to ensure that the soil reinforcing and facing panels are not damaged or displaced during backfilling. The backfilling operation shall be arranged so that no tracked equipment operates on the reinforcing strips.

All vehicles and earthmoving equipment weighing more than 1.5 tons shall be kept at least 3 feet away from the facing panels. The fill away from the three-foot zone shall be compacted using an 8-10 vibratory roller. The fill within 3 feet of the face shall be compacted using a vibrating roller weighing no more than 1-ton, or a vibrating plate compactor weighing no more than 0.5-tons.


Figure 2-13. Compaction Equipment

Thinner lift thickness may be required in the three-foot zone to achieve the specified compaction density when using light-weight compaction equipment.

Figure 2-14. Hand Operated Equipment in 3-Foot Zone

### 2.4.6 Soil Reinforcing Installation

The first layer of soil reinforcing should be laid out perpendicular to the wall face and placed on top of the compacted backfill. The connecting bolts should then be inserted from the bottom of the anchor so that the nut is installed on top of the anchor. It is necessary to hand-dig a small void at the panel anchor to insert the bolt from the bottom of the anchor. The nut shall be placed snug-tight so the bolt can't be removed without the aid of a wrench.

Figure 2-15. Connection Photo

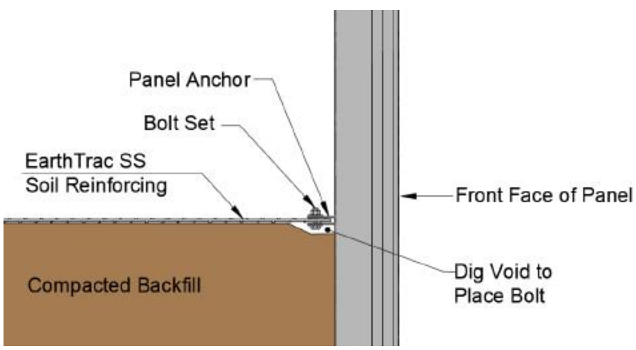



Figure 2-16. Connection

### 2.4.7 Second Stage Backfilling

The second stage backfilling shall be placed up to the top of the alternating half-panel or at abutment wing walls or walls where the foundation elevation steps to the subsequent soil reinforcing elevation.

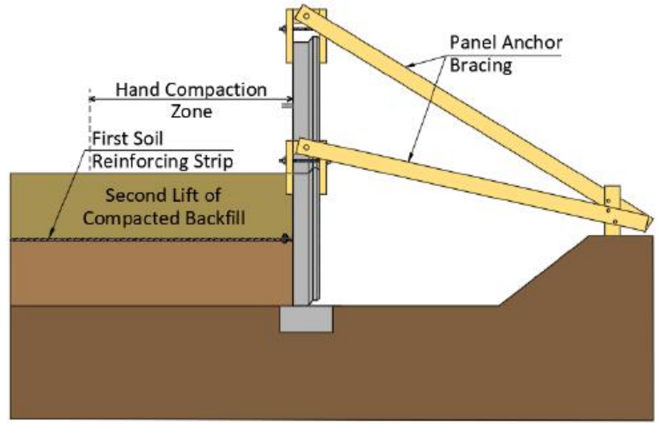



Figure 2-17. Second Stage Backfilling

It is important to understand that as backfilling and compaction is carried out, the wall panels will tend to creep forward. To limit this, compaction of fill outside of the first 3 feet (three-foot zone) should be performed to secure the soil reinforcing prior to compaction of fill in the three-foot zone.

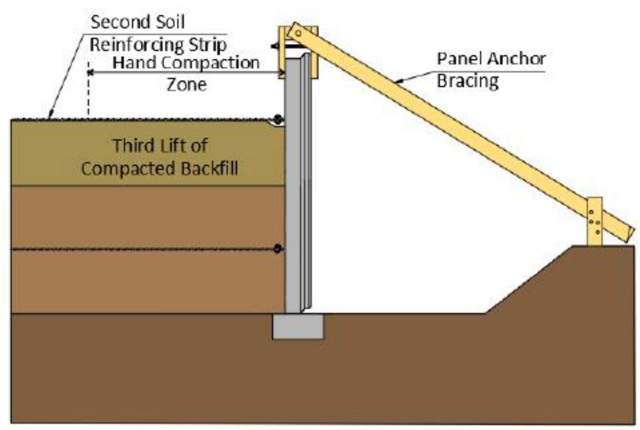



Figure 2-18. Third Stage Backfilling

Additional panels should not be placed until backfill is compacted to the top of the lower panel.

Upon completion of this fill stage, the panels' horizontal and vertical alignment should be verified and adjusted, as necessary. Additionally, safety regulations shall be referred to for the provision of temporary hand railing.

Figure 2-19. Spreading Fill on Compacted Lift

### 2.4.8 Intermediate Panels

Once the panel alignment, vertical joint spacing, and levelness of the top panel surface have been checked, the next row of panels may be added. The panels' top should be free of dirt or debris that will interfere with installing the intermediate panels and bearing pads.

For 5-foot panels, a minimum of two bearing pads shall be placed on top of each of the lower panels before installing the next panel row. For 10-foot panels, a minimum of four bearing pads shall be placed on top of each of the lower panels prior to installation of the next row. If the wall height exceeds 40 feet, the panels below the 40-foot height shall have the number of bearing pads doubled, i.e., four bearing pads for the 5-foot panel and eight bearing pads for the 10-foot panel.

The intermediate row of panels should be placed on top of the bearing pads and adjusted for batter and alignment. Hardwood wedges should be placed in the horizontal joints at the panel corners to maintain batter during backfilling. As in previous steps, the corners of adjacent panels should be clamped to control movement.

Figure 2-20. Setting Intermediate Panel

Hardwood wedges on the lower row of panels should be removed as soon as the next layer of panels is positioned. Repeat the panel placement, geotextile placement, and filling operations, ensuring that the various stages described previously are followed.

### 2.4.9 Top Panels

The procedure for the installation and backfilling for top panels is the same as previously described. It may be required that leveling concrete (mortar bedding) be added to the top of the panels to provide a smooth line for the placement of the coping unit.

Figure 2-21. Mortar bedding on top of panels

Care should be taken to ensure that the top row of soil reinforcing does not interfere with any parapet or anchor slab installation.

The upper layer of soil reinforcing may be required to be deflected down to not interfere with the casting of any anchor slabs (if required). The maximum deflection permissible is 15 degrees from horizontal. The deflection should be gradual.

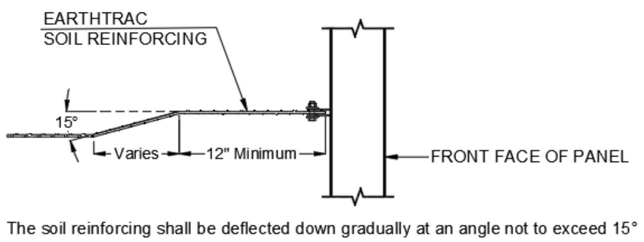



Figure 2-22. Horizontal Deflection of Soil Reinforcing

After the completion of all backfilling operations including the finish grade at the face of the wall, all hardwood wedges and clamps should be removed. Installation of coping or parapets should be performed with care so as not to damage the upper layer of soil reinforcing. Excavations within 2' of the reinforced zone should be made using only hand equipment.

Excavations should not be made within the reinforced zone of the MSE wall after installation of the soil reinforcing unless approved by the Engineer. Damage to the soil reinforcement may result in costly repairs and delays to the project.

# 3 REINFORCED SOIL WALL SYSTEM

## 3.1 System Description

The EarthTrac SS™ wall system utilizes discrete precast concrete facing panels bolted to galvanized ribbed steel reinforcing strips embedded in select granular backfill. A wall cross-section consisting of the various elements of construction is illustrated below.

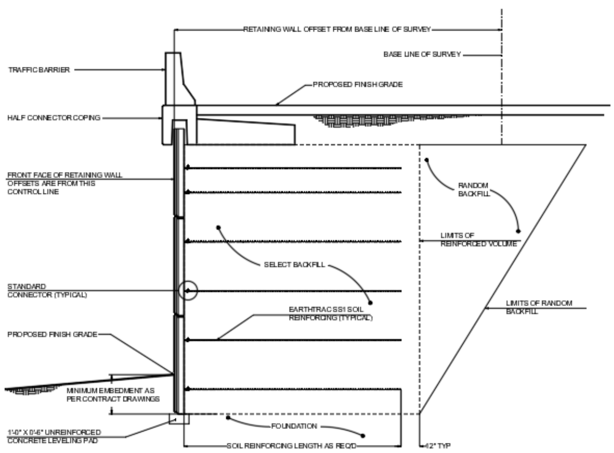



Figure 3-1. Typcial MSE Section

## 3.2 System Components

### 3.2.1 Leveling Pad

The leveling pad shall be constructed to the elevation, dimensions, and specifications given on the construction drawings using unreinforced cast-in-place (CIP) concrete. The dimensions given on the drawings are minimum dimensions and larger leveling pads may be required to suit various construction details such as curves, corners, steps, etc.

The leveling pad's primary function is to provide a firm, level surface from which erection of the facing panels can commence. The leveling pad is not a structural concrete element and shall not be considered as one.

### 3.2.2 Precast Concrete Facing Panels

Precast concrete facing panels shall be manufactured following the procedures outlined in the EarthTec precast manual.

Panels are supplied with either a plain or architectural finish, depending on specific project requirements. All panels incorporate an edge lip such that when erected, the lip forms a concrete backdrop for the 3/4" nominal joint around each panel. The panels incorporate lifting anchors in the top edge to facilitate easy installation.

Figure 3-2. Facing Panel Lifting

Galvanized steel panel anchors are cast into the back face of the precast facing panel. The panel anchors provide a means to bolt the steel reinforcing strip to the precast concrete panel.

Panels are designated with an alpha-numeric coding to identify specific height and width dimensions. The standard panel designation for a wall elevation is as follows:

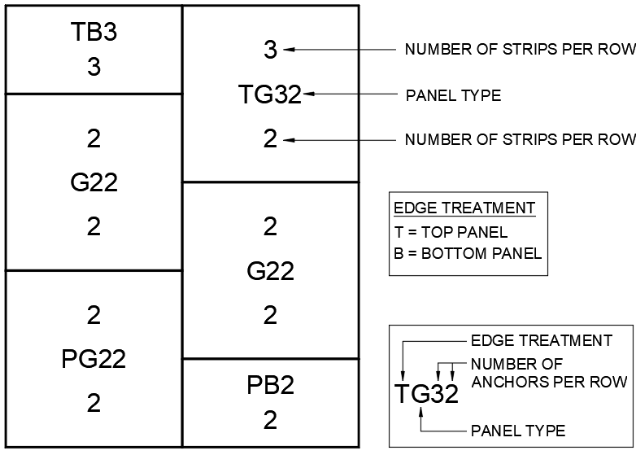



Figure 3-3. Panel Designation

Panels are delivered to the project site stacked flat with the front face down. Each panel is separated from one another by timber blocks to avoid damage. The timber blocking must have a hard-plastic faceplate to prevent staining from the blocking. The panels may be unloaded using slings in stacks or individually using lifting hooks attached to the lifting anchors.

Figure 3-4. Panel Lifting with Dunnage

To avoid damage to the panels timber blocks must be placed at the anchor positions to support each panel during transportation and on-site storage.

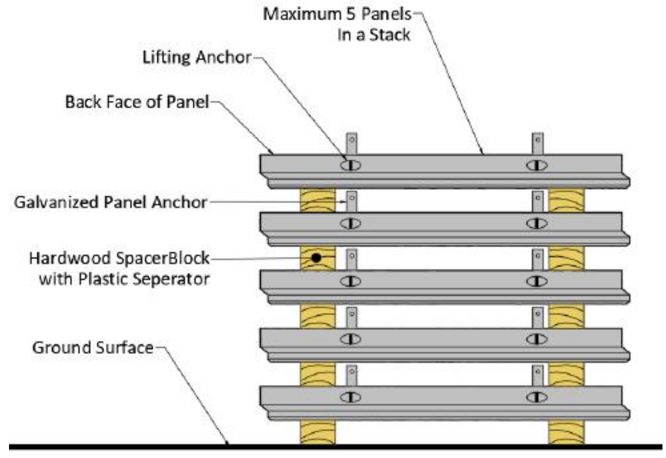



Figure 3-5. Panel Stacking

On-site, panels shall be stored on firm level ground and stacked no more than five high using timber spacer blocks of consistent height. A plastic separator shall be inserted between the timber block and the concrete panel's face to avoid staining.

### 3.2.3 Soil Reinforcing Straps

The EarthTrac SS™ System uses a specially designed ribbed steel strip that is 2" wide and 3/16" (SS1) or 5/32" (SS3) thick.

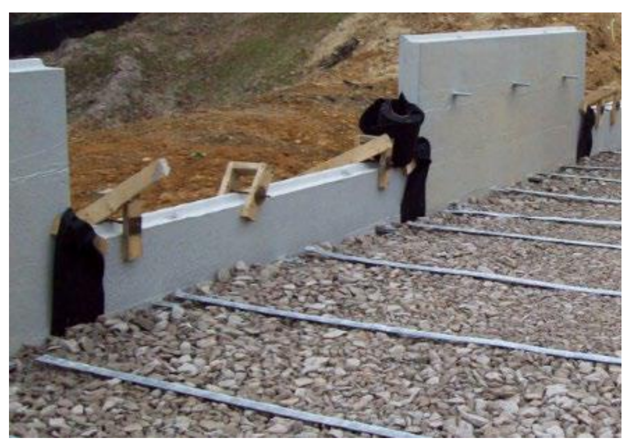


Figure 3-6. EarthTrac SS™ Soil Reinforcing

### 3.2.4 Bolts

Bolt sets shall be a minimum ½" diameter galvanized high strength ASTM F3125 Grade A325. If not supplied by EarthTec it is essential that the correct bolt grade is used to ensure connection strength.

### 3.2.5 Geotextile

A 12" wide geotextile is placed at the back face of the precast panels over all exposed panel joints and adhered with adhesive. If the geotextile is not continuous adjoining geotextile strips shall be overlapped a minimum of six-inches (shingled).

Figure 3-7. Filter Fabric Placement

### 3.2.6 Panel Bearing Pads

The panel bearing pads supplied by EarthTec are ¾" thick x 6" long x 3" wide, SRB rubber, with a minimum shore hardness of 60. Four bearing pads shall be placed on the top of each 10' panel and two bearing pads shall be placed on the top of each 5' panel. If the wall eight is greater than 40 feet, the panels below the 40-foot height shall have the number of bearing pads doubled. The bearing pads shall be equally distributed across the panel's top, as shown in the shop drawings.

The bearing pad controls the ¾" horizontal joint. The ¾" joint spacing mitigates against panel damage during installation and during the structure's service life.

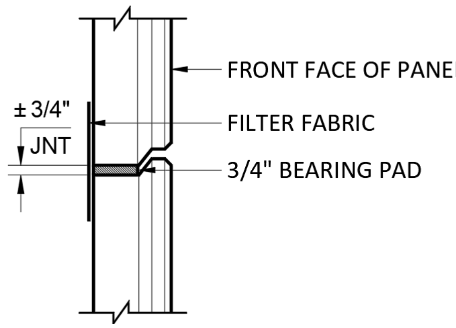



Figure 3-8. Horizontal Bearing Pad

### 3.2.7 MSE Backfill

The backfill shall be in accordance with the project specifications. The project specification provides the characteristics for the MSE backfill, including but not limited to gradation, PI, friction angle, and electrochemical properties.

### 3.2.8 CIP Concrete Anchor Slab

Where necessary, to support parapet units and traffic railings a cast-in-place reinforced concrete anchor slab (a.k.a. moment slab) is provided.

The size of the slab is dependent on the magnitude of the horizontal impact force that the parapet and railing has been designed to. Full construction details of the anchor slab (a.k.a. moment slab) are given in the specific construction drawings.

### 3.2.9 Barrier and Parapet Coping Units

Precast concrete barrier or coping units are provided as shown on the construction drawings. These units are designed for containment and/or for aesthetic purposes at the top of the wall.

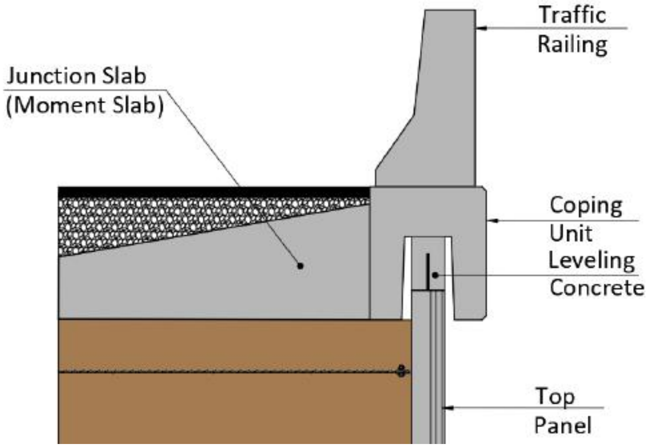



Figure 3-9. Top of Wall with Traffic Barrier

### 3.2.10 Slip Joints

Vertical slip joints are provided where differential movements are anticipated or when standard panel details cannot accommodate steps in the wall foundation.

Figure 3-10. Slip Joint Detail

Details of the slip joint and the location are in the project shop drawings.

## 4 PROCESS CONTROL

This section describes procedures and tools that can help ensure the successful construction and installation of the structure.

### 4.1 Facing Tolerance Overview

Facing movements may result from a combination of internal and external settlements such as the foundation. Facing panel control is a function of the consistency of the installer's methods and the control of surface water run-off.

Internal settlements of the MSE backfill are typically small and occur as the structure is being constructed. During erection and backfilling, the vertical movements of the backfill are accommodated by a gradual reduction in the horizontal joint dimension as well as a small outward rotation in the facing panels. Surface water run-off shall be diverted away from the MSE wall face.

### 4.2 Construction Control

The EarthTec Reinforced Soil Wall system requires that the panels achieve a broadly even joint distribution/appearance when installed. The panel bearing pads help establish the ¾" spacing of the horizontal joints. The ¾" construction spacers used during the panel's placement at the vertical joints helps assure a uniform joint appearance.

It is not uncommon for joints to vary by $\pm$ 50% of their nominal dimension because of a combination of settlement and compaction of the backfill and foundation. For most structures, the movement capacity of the system will be within excess of that required.

The generally accepted tolerances for the alignment of facing panels of MSE structures are:

### 4.3 Fill Placement and Compaction

Most sands, gravels and crushed rock are suitable fill materials for MSE structures. Typically the backfill materials have a maximum particle size of 4" with no more than 15% passing the No. 200 sieve and with a uniformity coefficient $D_{60}/D_{10}$ of not less than 4. Finer materials can sometimes be used, providing they are non-plastic and meet other design requirements. Project-specific backfill materials are specified in the MSE wall design and noted on the drawings.

Some fills may comply with the specification yet may not be suitable for use with MSE structures when construction occurs under adverse weather conditions. This generally applies to a material with fines close to but less than 15% passing No. 200 sieve with a marginally acceptable uniformity coefficient. Such material is particularly susceptible to the effects of moisture and is unsuitable for use in all but the most controlled site conditions.

When backfills containing fine materials with greater than 15% passing the number 200 sieve, construction personnel must be aware of the potential problems during construction. Some steps that can be implemented to mitigate poor service include:

- Ensure material delivered complies with accepted gradation.
- Ensure placing and compaction is carried out in strict accordance with Section 4.0 of these guidelines.
- Ensure temporary panel batter is sufficient.
- Ensure that the reinforced backfill is sloped away from the wall face at the surface to prevent ponding of rainwater and provide positive run-off away from the wall face at the

end of each day's work. The minimum slope necessary is 1:20.

- Provide temporary cover to completed work to prevent inundation and ensure that run-off is drained effectively away from the reinforced soil volume.
- Consider using compaction methods that do not introduce large vibrations.

Moisture and density control are imperative for the proper construction of the MSE system. If proper compaction control is not exercised, problems are almost certain to occur, mainly when using fine material. The MSE fill should be placed and compacted at or within 2 percent dry of the optimum moisture content. The MSE fill for the retaining walls should be compacted to a density that meets the project specifications. It should be noted that other densities may be required on specific projects.

When compacting uniform medium to fine sands (greater than 60 percent passing the No. 40 sieve), consider using smooth drum static rollers or lightweight hand-operated vibratory rollers instead of large vibratory compaction.

Compaction methods should be demonstrated and documented before construction commences.

### 4.4 Inspection and Testing Procedures

At a minimum, the following items should form the basis of the Contractor's Inspection Plan program.

Wall Alignment Checks:

- Use a plumb-bob to check the overall vertical tolerances for every panel at the third row of panels

Compaction Checks:

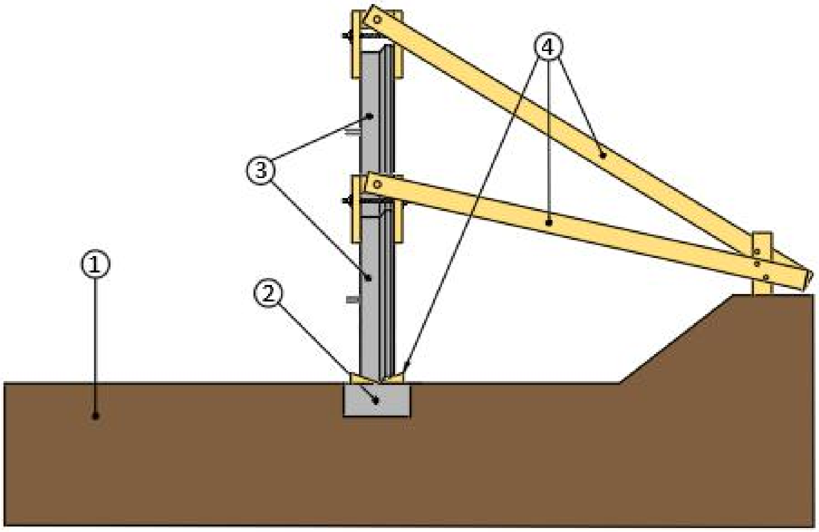
- In place density checks at a minimum of every 650 cubic yards.
- Check moisture content (MC) and Optimum Moisture Content (OMC) with the above.

Backfill Checks:

- Grading and Uniformity Coefficient (UC) every 4,000 cubic yards, minimum.
- Direct Shear testing every 8,000 cubic yards.
- Each of the above requirements should be carried out more frequently if fill materials are marginally acceptable.
- The above testing frequency may be decreased for exceptionally large projects providing that a minimum of 10 tests provides a 95% confidence level of compliance with the specifications.

## 5 INSTALLATION SEQUENCE

### 5.1 STEP ONE



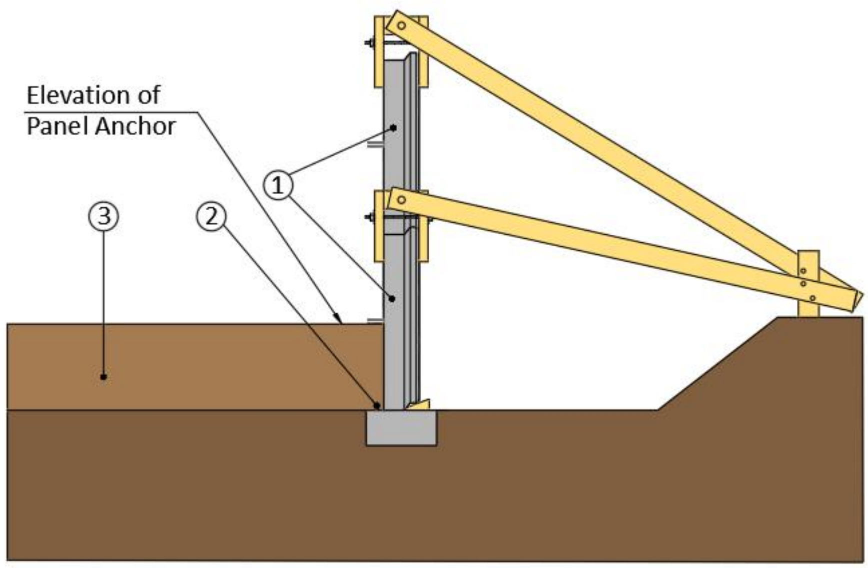

**STEP 1.** Excavate and prepare foundation

**STEP 2.** Form and place leveling pad to grades, lines and widths as shown on the project shop drawings.

**STEP 3.** Place first row of panels on leveling pad.

**STEP 4.** Plumb panels with wedges and support with brace as required.

## 5.2 STEP TWO

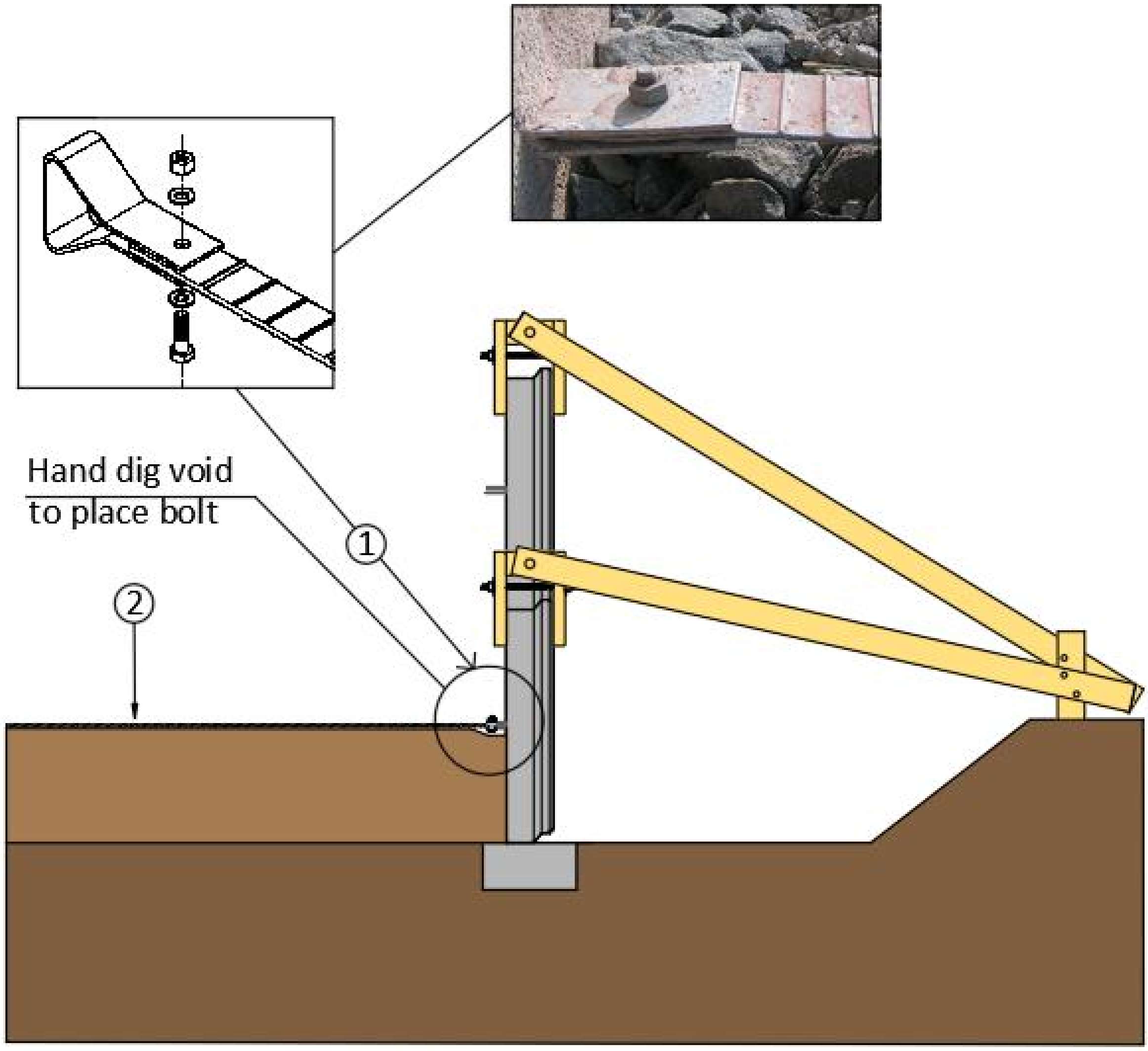



**STEP 1.** Place geotextile on back face of panel at all vertical and horizontal joints and secure to panel with adhesive.

**STEP 2.** Remove wood wedge from back of panel at leveling pad.

**STEP 3.** Place and compact backfill on prepared foundation to the level of the first row of panel anchors.

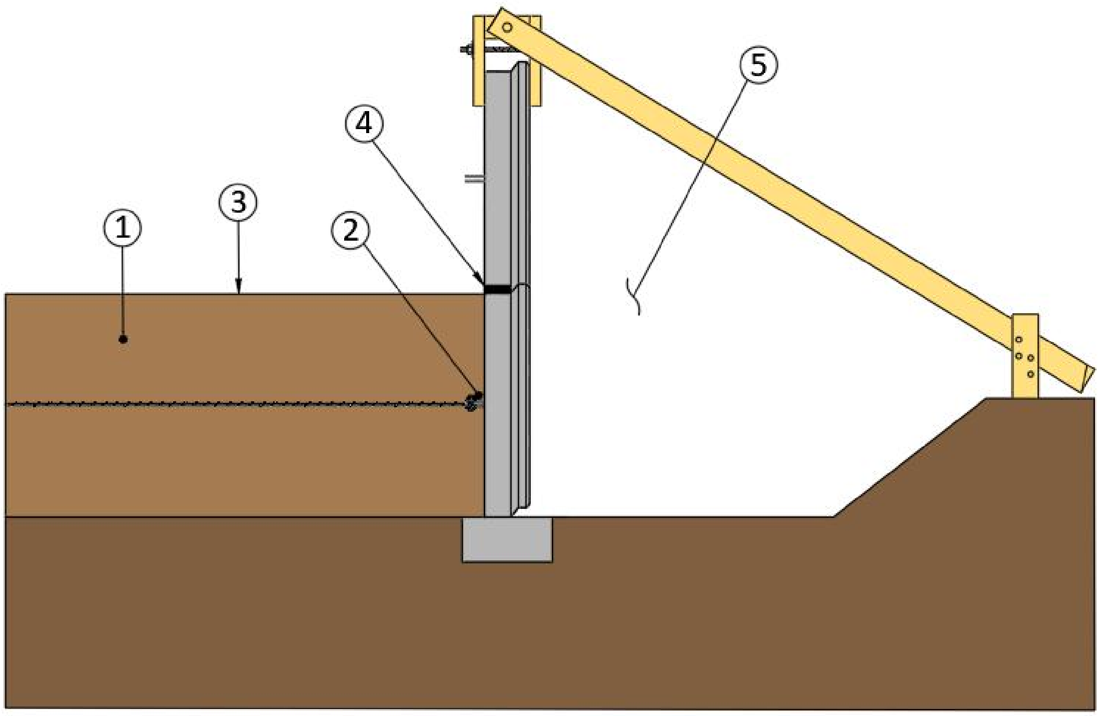
### 5.3 STEP THREE




**STEP 1.** Place EarthTrac SS™ soil reinforcing on compacted backfill making sure that it is the correct length. Hand dig a small void at the anchor to allow for attachment of bolt-set.

**STEP 2.** Connect the EarthTrac SS™ to the panel anchor with the bolt set. The nut should always be facing up. Finger tighten the nut, then snug-tight the nut using a wrench to a tightness where the nut cannot be removed by hand.

### 5.4 STEP FOUR




**STEP 1.** Place soil on the terminal end of soil reinforcing to prevent the soil reinforcing from moving

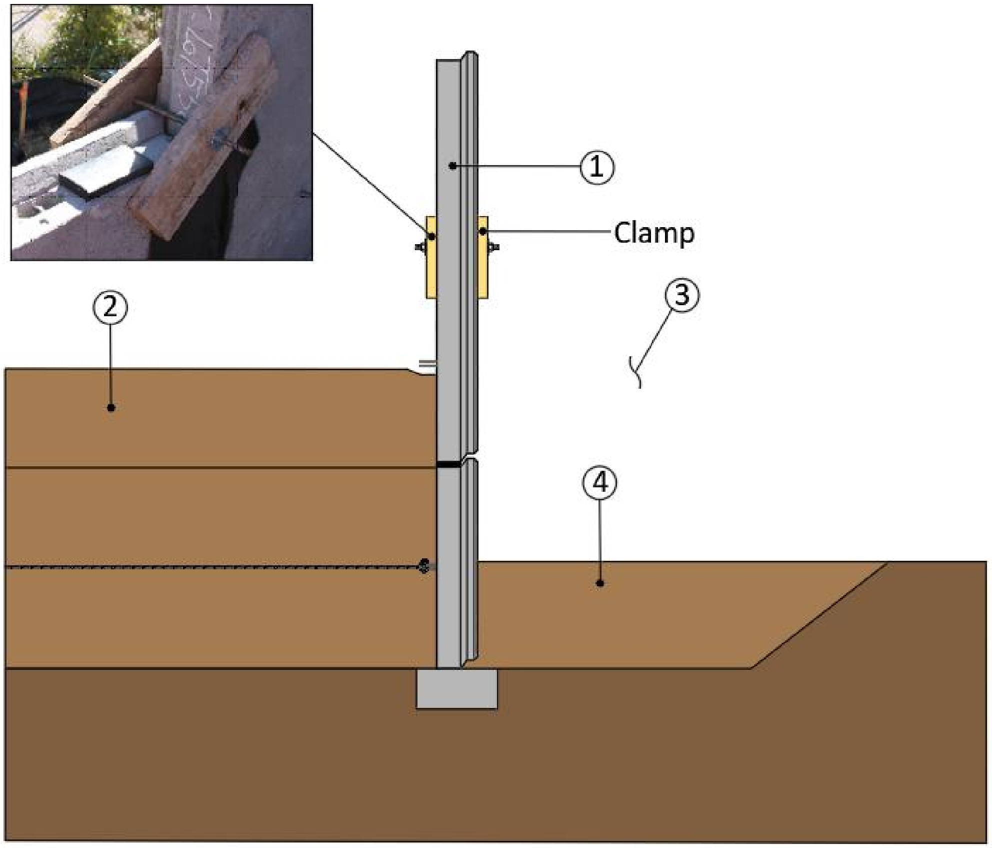
**STEP 2.** Place selected backfill in the hand-dug void at the panel's back face at the connection and compact.

**STEP 3.** Place select backfill to the level of the top of first row of half-panel and compact.

**STEP 4.** Place bearing pad on top of panel. Make sure top of panel is clean and free from backfill.

**STEP 5.** Remove clamps and blocking from first row of half panels. Remove all wedges.

## 5.5 STEP FIVE




**STEP 1.** Place the next row of alternating panels and clamp to the adjacent panel to prevent movement.

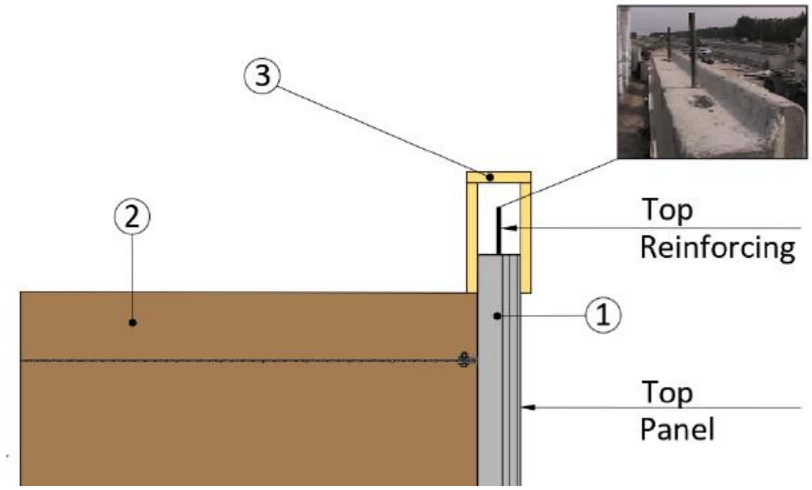
**STEP 2.** Place next layer of selected backfill and compact to the elevation of panel anchor. Hand dig small void at anchor for the placement of bolt-set.

**STEP 3.** Remove external bracing.

**STEP 4.** Place and compact the finish grade at the face of the wall.

**STEP 5.** Repeat installation Steps Two through Step 5 until top row of panels are placed.

### 5.6 STEP SIX

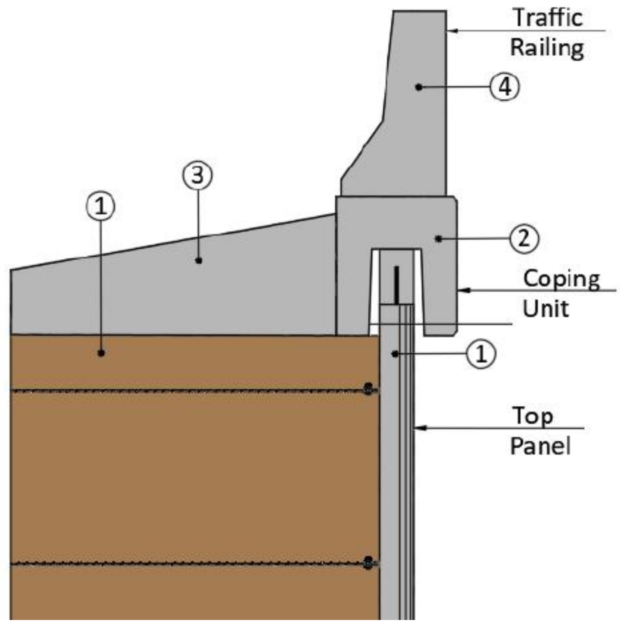



**STEP 1.** Place top row of panels.

**STEP 2.** Place layer of selected backfill compact.

**STEP 3.** Set wood forms for the leveling concrete. Place the leveling concrete to bring the top of the panel to the required elevation and grade.

## 5.7 STEP SEVEN




**STEP 1.** Place backfill to the level of bottom of coping or bottom of the moment slab.

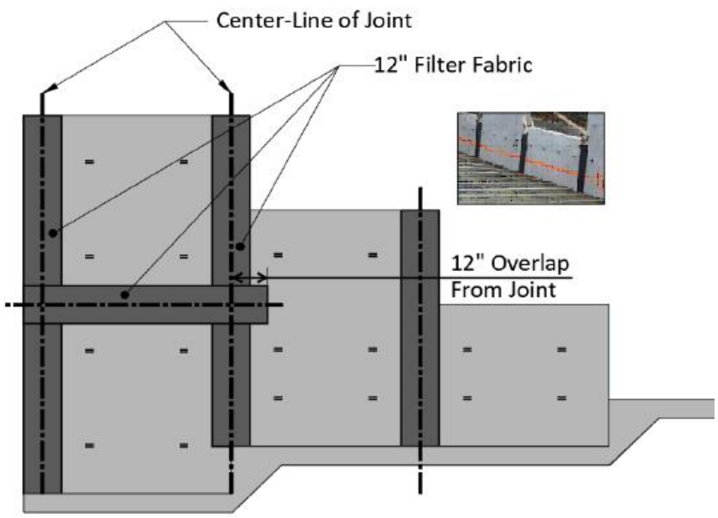
**STEP 2.** Form and place the coping unit if cast-in-place or set the precast coping unit.

**STEP 3.** Form and place moment slab.

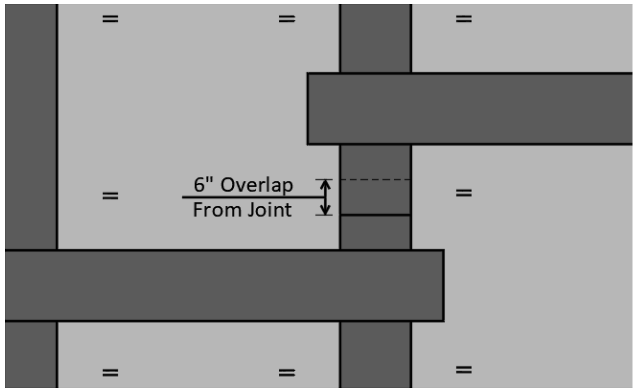
**STEP 4.** Form and place traffic barrier. This step may be performed after the wall has been completed.

**STEP 5.** Note: If a one-piece coping unit and traffic barrier are being used omit step 4

## 5.8 FILTER FABRIC PLACEMENT




Standard Placement




Vertical Overlap

**STEP 1.** Place 12" wide geotextile centreline of all horizontal and vertical joints.

**STEP 2.** Adhere geotextile with adhesive, so it makes contact with the panel.

**STEP 3.** There shall be a 12" overlap at staggered joints and between discontinuous joints.

**Note:** Filter fabric may be required to be placed at the bottom panel interface and leveling pad. Reference the Owners specification for guidance on this application.

If the geotextile is not continuous, the geotextile should be overlapped (shingled) 6".

## 5.9 PANEL LIFTING


Lift panels with approved lifting device pursuant to Burke Spread Anchor

One-Ton and Two-Ton specifications.

## 6 RISK ASSESSMENTS

This section describes some common risks and methods for risk control. This list is provided for guidance only and the ultimate responsibility for site safety lies solely with the Contractor.

| OPERATION | RISK/HAZARD | CONTROL |
|---|---|---|
| Unloading panels from delivery trucks | Falling Panels | Ensure truck is parked on firm level ground in a safe area protected from traffic. Inspect load before proceeding. Attach lifting devices to loose panels prior to releasing tie down connections. All panels should be properly secured before moving equipment. Only authorized personnel are allowed in the area during unloading. Do not stand or walk under suspended loads. All personnel should wear PPE as required by the General Contractor. Panels should be stored as specified in section 3.2.2 of this manual. |
| | Lifting device failure | Inspect all lifting devices (i.e. straps, chains, cables, etc.) for damage prior to use. Discard and replace damaged items. Use only devices with adequate capacity for the load. |
| | Lift Equipment Failure | Equipment should be inspected for excessive wear or damage prior to use. Maintenance records should be current and reviewed prior to the operation of equipment. Operators should be fully trained in the specific equipment being used. |
| Positioning of Panels in wall | Panel Falling Over | Prior to release from slings, panels shall be properly clamped in position. Release tension from slings slowly. |
| | Bracing | Each bottom row of panels shall be securely braced. |
| | Fall from height | Fall protection devices as required by the General Contractor should be installed or utilized for all leading edge workers. |

| OPERATION | RISK/HAZARD | CONTROL |
|---|---|---|
| | Fall of object from height | Personnel shall be restricted from working below wall.<br>All tools and material should be kept away from the face of the wall. |
| | Injury due to manual handling | Manual handling assessments shall be undertaken for any lifts indicating a possibility of injury. |
| Placing of Fill Materials | Reversing of Vehicles | All vehicles should have backup alarms. |
| | Earth moving equipment striking workers | Personnel to be kept a minimum of 600mm (2 ft.) from Earth moving equipment. Spotters should be utilized when equipment is operated near workers. |
| Excavation | Overhead/Underground Utilities | Permit to work shall be issued prior to commencement identifying hazards. All utilities should be marked. |

## 7 TROUBLESHOOTING

Although quality materials, strict compliance to the specifications and proper construction methods are undertaken, some out of tolerance conditions can still occur during construction. It is important that these deficiencies be recognized and corrected to return the structure to within acceptable tolerances. The following items are examples of out of tolerance conditions and possible causes for the condition:

| Out of Tolerance condition | Possible Cause |
|---|---|
| Difficulty setting or leveling first panel | Leveling pad out of tolerance |
| | |
| Differential Settlement causing a dip in the vertical alignment or panel to panel contact | Inadequate bearing capacity of foundation soils due to wet or poorly compacted fill.<br><br>Inadequate shear strength of residual soils.<br><br>Poorly compacted utility trench below wall. |
| Negative batter (wall leaning out) | Insufficient batter prior to fill placement<br><br>Backfill material not within specifications:<br>• More than 2% over optimum moisture content<br>• Excessive fines (% passing 200 sieve)<br>• Plasticity Index beyond specified limits<br><br>Heavy equipment operated within three feet of panels<br><br>Backfill material placed and compacted perpendicular to the wall.<br><br>Lift thickness beyond specifications<br><br>Excessive compaction of backfill<br><br>Clamps and wedges not properly secured |
| Positive batter (wall leaning in) | Excessive batter of panels prior to backfill placement<br><br>Insufficient compaction of backfill |

| Out of Tolerance condition | Possible Cause |
|---|---|
| Difficulty installing upper panels | Leveling pad out of tolerance<br><br>Panels not level<br><br>Incorrect horizontal spacing<br><br>Differential settlement causing panel to panel contact and spalling |
| Bulging or horizontal alignment out of tolerance | Backfill material not within specifications:<br>• More than 2% over optimum moisture content<br>• Excessive fines (% passing 200 sieve)<br>• Plasticity Index beyond specified limits<br><br>Heavy equipment operated within 5 feet of panels<br><br>Backfill material placed and compacted perpendicular to the wall.<br><br>Lift thickness beyond specifications<br><br>Excessive compaction of backfill<br><br>Saturation of backfill due to rain |

Inspection of the wall should be made continuously as the wall is erected. Out of tolerance conditions, if caught early, can usually be rectified with minimal downtime and effort. If the condition is not recognized as soon as possible, difficult and costly repairs could result.

## 8 MSE CHECKLIST

| | | | |
|---|---|---|---|
| 1. | Yes | No | Do you have an approved copy of shop drawings? |
| 2. | Yes | No | Do you have backfill certifications? |
| 3. | Yes | No | Do you have panel certifications? |
| 4. | Yes | No | Do you have Grid-Strip™ soil reinforcing certifications? |
| 5. | Yes | No | Is all required material on site? |
| 6. | Yes | No | Is the material stored properly to prevent on site damage? |
| 7. | Yes | No | Has damaged material been recorded and a copy of rejected material given to suppliers? |
| 8. | Yes | No | Is the foundation excavated and proof rolled per the specifications and to the required width and elevation? |
| 9. | Yes | No | Has unsuitable foundation material been compacted or removed and replaced? |
| 10. | Yes | No | Is the first row of Grid-Strip™ soil reinforcing properly placed, aligned, and spaced. |
| 11. | Yes | No | Are the proper face panels being installed? |
| 12. | Yes | No | Are the required number of Grid-Strip™ soil reinforcing elements and the correct length being used? |
| 13. | Yes | No | Are the correct bolt sets used and the nut snug-tight? |

| | | | |
|---|---|---|---|
| 14. | Yes | No | Is the geotextile being properly placed and adhered to the back face of the panel? |
| 15 | Yes | No | Is the backfill being properly placed? Is it being placed in proper lift thickness? |
| 16. | Yes | No | Is the backfill material being spread from the back face of panel to terminal end of soil reinforcing? |
| 17. | Yes | No | Is the equipment being kept off of the soil reinforcing until 6" of backfill material is placed? |
| 18. | Yes | No | Is proper compaction being achieved - 95% of maximum density? |
| 19. | Yes | No | Are the Grid-Strip™ soil reinforcing elements being properly aligned? |
| 20. | Yes | No | Is the vertical and horizontal alignment of the structure being checked periodically? |
| 21. | Yes | No | At the end of each days operation, is the reinforced volume being protected from run-off and saturation? |

## 3.1.3
## ERS Facing Installation Requirements

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# 3.1.4
# ERS Soil Reinforcing Installation Requirements

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**3.1.5**
**ERS Facing Alignment Requirements**

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**3.1.6**
**ERS Reinforced Backfill Placement Requirements**

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**3.1.7**

**ERS Erosion Prevention Requirements**

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**3.1.8**
**ERS Installer Requirements**



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**3.1.9**

**ERS Retained Backfill Placement Requirements**

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# 4.0
# QUALITY CONTROL

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# 4.1
# MANUFACTURING

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**4.1.1**
**Facing Unit QA/QC**

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# EARTHTEC™ REINFORCED SOIL WALL

# PROCEDURES AND QUALITY CONTROL GUIDELINES

# PRECAST SEGMENTAL CONCRETE PANEL

**GROUND IMPROVEMENT SYSTEMS LLC**
114 South Collins Street
Arlington, TX 76011
817-223-0969 Office
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Support Through Innovation

# MECHANICALLY STABILIZED EARTH STRUCTURES

## REINFORCED SOIL WALL (RWS) SYSTEM

## PROCEDURES AND QUALITY CONTROL GUIDELINES

## PRECAST SEGMENTAL CONCRETE PANEL


GROUND IMPROVEMENT SYSTEMS LLC

114 South Collins Street

Arlington, TX 76011

817-223-0969

# TABLE OF CONTENTS

# SEGMENTAL CONCRETE PANEL SPECIFICATION

## 1 GENERAL

This specification pertains to the casting of the Segmental Concrete panel Reinforced SoilWall system. Panels shall be cast according to this specification and in reasonably close conformity with the dimensions shown on the plans or established by the Engineer.

## 2 REFERENCES

**2.1 American Society for Testing and Materials (ASTM)**

**2.1.1** A36 - Standard Specification for Carbon Structural Steel

**2.1.2** A123 - Standard Specifications for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products

**2.1.3** A153 - Standard Specifications for Zinc Coating (Hot-Dip) on Iron and Steel Hardware

**2.1.4** A325 - Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength

**2.1.5** A496 - Standard Specifications for Steel Wire Reinforcement, Deformed, for Concrete

**2.1.6** A497 - Standard Specifications for Welded Wire Reinforcement, Deformed, for Concrete

**2.1.7** A525 - Specification for General Requirements for Steel Sheet, Zinc-Coated (Galvanized) by the Hot-Dip Process

**2.1.8** A510 - Standard Specification for General Requirements for Wire Rods and Coarse Round Wire, Carbon Steel

**2.1.9** A615 - Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement

**2.1.10** A780 - Standard Specification for the Repair of Damaged and Uncoated Areas of Hot-Dip Galvanized Coatings

**2.1.11** A884 - Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Reinforcement

**2.1.12** A1064 -Standard Specification for Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete.

**2.1.13** C150 – Standard Specification for Portland Cement

**2.2 American Association of State Highway and Transportation Officials**

**2.2.1** M85 – Standard Specification for Portland Cement

**2.2.2** T22 - Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimens

**2.2.3** T23 - Standard Method of Test for Making and Curing Concrete Test Specimens in the Field

**2.2.4** T141 - Standard Method of Test for Sampling Freshly Mixed Concrete

**2.3 CRSI - MANUAL of Standard Practice**

**2.4 Wire Reinforcement Institute -**

MANUAL of Standard Practice

2.5 **ACI 318-08 - Building codes for Structural Concrete**

## 3 CEMENT

Cement shall be Types I, II and III with 3% to 6% air entrainment and shall conform to the requirements of AASHTO M85. Concrete shall have a compressive strength at twenty-eight (28) days in accordance with Section 8, Concrete Compressive Strength. Air entraining, retarding, accelerating agents, or any additives containing chloride shall not be used without the Owner's approval. Determine the maximum aggregate size in accordance with AASHTO LRFD standards and the governing specification.

## 4 TESTING AND INSPECTION

Acceptability of all panels shall be on the basis of compressibility tests and visual inspection. Precast units shall be considered acceptable regardless of curing time when compressive strength meets or exceeds the 28-day compressive strength. Contractor shall be responsible for all testing and shall provide a facility to perform tests. Units using Type-I or Type-II cement shall be deemed acceptable to be placed in the retaining wall when the seven (7) day compressive strength exceeds 85% of the 28 day compressive strength requirements. Units utilizing Type-III cement will be deemed acceptable for placement in the retaining wall when the compressive strength meets or exceeds the 28 day compressive strength requirements. Production lots will be recorded and tested for conformance. Any lot not meeting this specification shall be rejected.

## 5 CASTING

All panels shall be cast face down in smooth, flat, steel forms. Panel anchors and inserts shall be placed in a template at the back of the panel. Galvanized anchors and galvanized inserts shall not be allowed to contact black steel panel reinforcing. If contact is to occur they shall be separated by a non-conductive isolator. Concrete shall be placed without interruption. Concrete shall be vibrated using a form vibrator or hand vibrator. Clear form oil shall be used.

## 6 CURING

When the temperature of the air is between zero (0°) F and 30° F, the minimum concrete temperature should be 65° F at placement.

When the air temperature is above 30° F, the temperature of the concrete should be 60° F.

### 6.1 Hot Weather or Indoor

The panel shall be cured in the steel form for a sufficient length of time that allows the panel to be stripped without causing undue stress or damage to the panel. The panel shall be kept sufficiently wet and protected in order to prevent the temperature of the concrete from dropping below 80° F.

### 6.2 Cold Weather

The panel shall be cured in the steel forms placed a minimum of 6” off the ground. The concrete slump shall be kept less than four (4) inches. No extra water shall be sprinkled on the concrete surface. Newly placed concrete shall be kept from freezing by maintaining 55° F for 72 hours and maintain temperatures above 40° F for an additional four (4) days. Monitor temperature on corners and edges. Use approved curing compounds to reduce drying.

## 7 LIFTING DEVICES

All lifting devices as specified by EarthTec, or an approved equivalent, shall be used to strip panels from the form. No panel shall be placed in the MSE structure until it meets the requirements of this specification

## 8 CONCRETE FINISH

Unless otherwise noted on the plans or elsewhere in the project specifications, the exposed concrete surface shall be smooth gray. The rear of each panel shall be hand screed smooth to eliminate open pockets of aggregate and surface distortions in excess of ¼” (6 mm).

## 9 TOLERANCES

### 9.1 Panel dimensions

Panel dimensions shall be within 3/16” (5 mm) of dimensions as noted on the plans.

### 9.2 Panel Squareness

The panel shall be considered square when the differences of two parallel verticals measured at the side of the panel do not exceed ½” (13 mm) or the difference between tow diagonals do not exceed ½”

### 9.3 Panel Smoothness

Smooth panel surface finish shall be free of defects that exceed 1/8” (2.5 mm) as measured on a length of 60 inches (1525 mm). Textured panel surface finish shall be free of defects that exceed ¼” (6 mm) as measured on a length of (1525 mm)

## 10 CONCRETE COMPRESSIVE STRENGTH

### 10.1 Acceptance

The acceptance of concrete units with respect to compressive strength will be determined based on production lots. A production lot is represented as a single compressive strength sample and will not be more than 80 panels or one day's production whichever is less. The compressive strength shall be no less than 4000 psi.

### 10.2 Sampling

Concrete will be sampled for each production lot in accordance with AASHTO T-141. A minimum of four cylinders will be randomly selected for each production lot.

**10.2.1** Frequency

Cylinders shall be taken in accordance with AASHTO T-23 on 6" (150 mm) x 12" (300 mm) specimens. For every compressive strength sample, a minimum of two (2) cylinders will be cured in the same manner as the panels are and tested at approximately seven (7) days. The average compressive strength of these two (2) cylinders when tested in accordance with AASHTO T-22 will provide a test result, which will determine the initial strength of concrete. In addition, two (2) cylinders will be cured in accordance with AASHTO T-23 and tested at approximately twenty-eight (28) days. The average compressive strength of these two (2) cylinders when tested in accordance with AASHTO T-22 will provide a compressive strength test result, which will determine the compressive strength of the production lot.

**10.2.2** Initial Test Results

For the initial strength test results if the compressive strength is in excess of 4000 psi then these test results will be utilized as the compressive strength test results for that production lot, and the 28 day requirement will be waived for the lot in question.

**10.2.3** Compressive Strength Acceptance

Acceptance of a production lot will be made if the compressive strength test result is greater than or equal to 4000 psi. If the compressive strength is less than 4000 psi the acceptance of the production lot will be based on its meeting the following acceptance criteria in its entirety:

**10.2.3.1** Ninety Percent Rule

If 90% of the compressive strength test results for the overall production exceed 4000 psi.

**10.2.3.2** Average Six Rule

If the average of any six (6) consecutive compressive strength tests results exceed 4000 psi.

**10.2.4** Compressive Strength Rejection

Production lots will be rejected for failure to meet specified compressive strength requirements. In order to get the production lot accepted the manufacture, at his or her own expense, may obtain and submit evidence the strength and quality of concrete placed within the panels of the production lot is acceptable. All core samples shall be obtained and tested in accordance with AASHTO T-24.

## 11 Rejection

Units shall be subject to rejection for failure to meet any requirements specified above. In addition, any or all of the following defects may be sufficient cause for rejection.

### 11.1 Molding

Any defects that would indicate the imperfect molding of the panel.

### 11.2 Texture

Defects indicating honeycombed or open texture in the concrete.

### 11.3 Physical Characteristics

Defects in physical characteristics of the concrete, such as broken or chipped concrete.

### 11.4 Repair

It shall be the responsibility of the Owner to determine whether the spalled, honeycombed, chipped or otherwise imperfect concrete shall be repaired or be cause for rejection. The panel shall be repaired in such a manner that is acceptable to and approved by the Owner.

### 11.5 Marking

The date of production, the production lot number and the piece mark shall be clearly scribed on the rear face of the panel.

## 12 Panel Accessories

Panel anchors, clips, and inserts shall be set in place to the dimensions and tolerances as shown on the plans.

### 12.1 Panel Anchors

All panel anchors shall be in accordance with ASTM A510 - Standard Specification for General Requirements for Wire Rods and Coarse Round wire, Carbon Steel.

### 12.2 Structural Members

All structural members shall be in accordance with ASTM A36/36M - standard Specification for Structural Steel

### 12.3 Fasteners

All fasteners and inserts shall be in accordance with ASTM A325 - Standard Specification for High-Strength Bolts for Structural Steel

### 12.4 Panel Reinforcement

All panel reinforcing shall have a minimum concrete cover of 1 ½" and shall be placed no closer than 2" to the edge of the concrete. Reinforcement shall be placed so the panel anchor is not in contact.

**12.4.1** Welded Wire Reinforcing

All welded wire panel reinforcement shall be fabricated in accordance with ASTM 1064 - Standard Specification for Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete.

**12.4.2** Reinforcing Bars

All bar reinforcement shall be in accordance with ASTM A615 - Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement and shall be Grade 60.

### 12.5 Galvanizing

All metallic accessories that require corrosion protection shall be galvanized in accordance with ASTM A123 - Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products or ASTM A153 Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware.

### 12.6 Epoxy Coating

All accessories that require epoxy coating shall be in accordance with ASTM A884 - Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Fabric for Reinforcement

## APPENDIX A – QUALITY CONTROL PROGRAM FOR PRECAST PANELS

### 1. LIMITATIONS

Information that is contained in this Appendix and all EarthTec, documents are not to be used to design, fabricate, manufacture, assemble, construct, produce, or install or otherwise use any elements, forms, or other special equipment (whether patented or not) that is exclusive to the EarthTec Reinforced Soil Wall (RSW) system, or for any other purpose other than this project without the express written consent of EarthTec. The information contained herein shall not be copied, disclosed, or distributed in any manner, in whole or in part, to any third party without the prior express written consent of EarthTec. The Quality Control procedures outlined in this manual have been developed to aid in designing, manufacturing, supplying materials, and installing the EarthTec RSW system. It is not intended to replace any of the Owner's requirements and is used as a supplement thereto.

### 2. PRE-POUR FORM PREPARATION

The forms supplied by EarthTec are unique to the RSW system. They have been designed to aid the Pre-Caster in rapid set up and stripping of the RSW product. The following procedure should be used prior to the placement of concrete and in conjunction with each subsequent pour. The key to making an acceptable and error free panel is to keep the forms clean.

#### 1.1. SET-UP

Set each forming pallet up to the required plant specific height and in their designated locations. Each form should arrive to the site with a pallet, side rails, top and bottom rails, panel anchor holders and all necessary hardware to attach each item.

#### 1.2. CLEAN-UP

After the forming pallet is set up thoroughly clean each steel element and assure that they are free from dirt, grease, oil, and debris. This is especially true at areas of contact points between interfaces.

### 1.3. Release Agent

Using a hand pumped or airless sprayer, coat the interior form surfaces of forming elements that will be in contact with the placed concrete with an evenly distributed and uniform coat of release agent. The release agent shall be applied in such a manner that minimizes the formation of release agent puddles on the form face and at the interface of rail and bed elements. The release agent when applied properly will help insure a defect free concrete surface and maintain the working condition of the forms. Note that puddles of the release agent will create a halo stain on the concrete face.

### 1.4. Form Liner Placement

If a form liner panel finish is required, place and attach the project specific elastomeric or urethane form liner on the steel form using the required mounting hardware. Insure that the liner is placed in the form so it is flush to the surface. If required use a caulking compound to seal the form liner at the interface of form face and the side rails, bottom rails, and top rails to prevent concrete bleed from occurring. As prescribed by the form liner manufacturer apply a release agent on the concrete surface of the liner.

## 3. Individual Form Set-Up Procedure

All required embedded items, panel reinforcing, attachment devices, panel anchors, etc., shall be in accordance with the approved shop drawings. Any special panel requirements shall be clearly illustrated on a production shop drawing and shall match requirements as specified within the approved shop drawings. All production shop drawings shall be used by the person who is charged with the set-up of the form, or forms. Production shop drawings shall be located in close proximity to the form until the concrete placement has concluded.

The general steps to setting up each individual form are:

STEP 1. Place and secure any headers, side rails, and, or, block outs that are required to create special panels.

STEP 2. Place the rebar or equivalent welded wire panel reinforcing to assure the proper depth of embedment is achieved.

STEP 3. Position the lifting inserts in the top edge of the panel or panel header.

STEP 4. Place and secure the required number of panel anchor holders in the proper location as specified on the panel production drawing.

STEP 5. Place the required number of individual panel anchors at the proper location in each panel anchor holder and secure in place with the required form mounting hardware.

STEP 6. Perform a final form setup inspection by the Quality Control representative to assure the form size, anchor placement, and special requirements have been included as illustrated in the panel production drawing.

## 4. CONCRETE PLACEMENT

### 4.1. CONCRETE TRUCK

Prior to the batching of the first load of concrete, the Owner Certified mixer truck(s) to be used for the delivery and dispensing of that days concrete shall be thoroughly checked (using a standardize checklist) to insure the proper handling and mixing of the concrete. Further, the mixer truck(s) are to be subject to regular washing and rinsing throughout each production day to prevent any buildup of cement deposits or deleterious materials from occurring.

### 4.2. CONCRETE CLASS

The class of concrete as required by contract documents and the approved mix design shall be batched and supplied by the Precaster from the point of placement/destination and in accordance with the Owner's concrete specification.

### 4.3. CONCRETE TRANSPORTATION AND HANDLING

All concrete shall be transported by a certified mixer truck from the Precast batch facility to the precast panel form location and handled in accordance with the Owner's concrete specification and the Owner's materials manual. The concrete will be deposited into the panel forms directly from the chute or via concrete hopper that is filled in close proximity to panel forms and mixer truck. As required, and to fill remaining voids within each respective form, the concrete shall be placed via use of hand dispensed spade shovel to insure proper concrete volume in each panel form.

### 4.4. Concrete Consolidation Method

Once a form is filled to the appropriate volume, the concrete will be consolidated via use of hand held electric internal vibrators and/or pneumatic external vibrators in accordance with the Owner's concrete specification. Vibration shall be continued in overlapping fields of action until proper consolidation has occurred. Once all air bubbles from the overlapping field of action has ceased appearing at the surface, the internal vibration shall be discontinued immediately to prevent possible segregation of aggregates. Once proper consolidation has occurred, strike-off the exposed surface with a straight-edge to insure the concrete extends to the top edge of the panel form and to remove any excessive concrete remaining in the form.

## 5. Protection Methods During Inclement Weather

### 5.1. Hot Weather Precautions –

In the summer months and late afternoon pours, necessary steps shall be taken to minimize the heating of the steel forms due to the direct sunlight. A procedure using water on the exterior surfaces and/or the temporary covering of the forms to shield the steel forms from the direct sunlight shall be instituted.

### 5.2. Pre-Inclement Weather Precautions

If foreseeable inclement weather is approaching prior to the placement of concrete, the Quality Control Manager will consult with the relevant parties to decide on whether to proceed with a schedule pour or reschedule as required.

### 5.3. Rain Events

If rain begins to develop during a placement of concrete, the charging mouth of the mixer/agitator truck will be covered immediately to prevent additional water from entering the truck and concrete. In addition, all cast panels will be immediately covered with plastic or a cure blanket to prevent deformation or the introduction of additional water into the exposed face.

### 5.4. Post-Inclement Weather Precautions

As a result of the inclement weather, if water has accumulated on the flat exposed form surface, the water shall be removed using portable air to blow the excessive water from the forming surface prior to the placement of concrete. This shall be done in each affected form.

## 6. Concrete Finishing and Curing Method

Once the form has been struck-off to the appropriate elevation, the precast panel's exposed back surface will be floated to remove any remaining high or low surfaces. Before concluding the final surface preparations, a final review of the location, alignment and condition of the attachment devices shall occur. As required, minor adjustments may be necessary to ensure the proper attachment/embed alignment is maintained.

Upon concluding the floating/troweling operation, each panel shall be etched on the exposed concrete surface with the panel name, date, batch/lot number and job number to ensure proper tracking of the product.

Following the panel marking, each precast panel form and all exposed concrete surfaces shall either be covered with a moist cure blanket within an appropriate amount of time or the exposed surface can be treated with an approved membrane curing compound.

The cast products will remain in the precast form for a minimum of 12 hours. After such time, the form will be disassembled and the precast product will be lifted from the form and inspected for any voids or defects. If voids or imperfections are found, the product will be designated for immediate repair and relocated to an appropriate area within the short-term cure area to receive the necessary attention. Before relocating the precast product, if a membrane curing compound is to be used, the product shall be immediately sprayed with the approved membrane curing compound following the manufacturer's recommendations and then relocated to a more permit storage location.

Product requiring minimal repairs/patching will be pointed with 1-part sand, 1-part cement paste and as necessary to match product color some portion of white portland cement and following the Owner's concrete specification. After the panel satisfies the quality inspection, including name verification, tolerance check, and quality verification, the product will either have the recently repaired area retreated with a membrane curing compound or the product will be transported to a temporary storage location to undergo further controlled curing.

If a membrane curing compound has not been used, the final control curing process will commence in a short-term storage location, once the previous day's products have been removed from their forms and stacked in a safe and expeditious manner. Once the days production has been re-inventoried, all of the recently cast products will be re-covered in a continuously moist environment for the remaining 72 hours.

During the 72 hours cure process, the panels/products are sacked/rubbed as necessary and re-checked to confirm all products meet or exceed the established quality standards.

At the conclusion of the 72 hours, the stacked panels and/or products will be transported to long-term storage where they will remain until the products are shipped to their respective job location.

## 7. TEST METHODS AND PROCEDURES

STEP 7. It is the sole intent of EarthTec to utilize on-site/in-house testing to conduct all required methods specified the Owner's concrete specification which includes oversight, sampling, field and lab testing and all necessary reporting. In situations where the in-house resources are unavailable to perform the specified requirements, an outside Owner approved testing laboratory and personnel shall conduct and oversee all required tests and methods and will generate the required reporting.

STEP 8. Prior to placement of an initial day's concrete a plastic properties test shall be conducted to determine the slump, air content, and temperature and will be executed at or near as possible to the point of placement to assure the concrete meets the Owner's concrete specifications.

STEP 9. All sampling will be obtained at the discharge destination point or at the end of the chute.

STEP 10. The sampling and test methods shall be in accordance with the methods as outlined in the Owner's concrete specification.

STEP 11. Compressive strength cylinders shall be 4" x 8" or 6" x 12" with the frequency of sampling shall not exceed 100 cubic yard batch increments and shall constitute a LOT. Each LOT shall include a minimum of 8 cylinders with an anticipated break frequency as follows: 2 at 7 days, 3 at 28 days and a minimum of 2 cylinders remaining in reserve.

STEP 12. All documentation relating to the test results, sampling and quality control data will be maintained and available for review at the manufacturing location with the required test results being forwarded directly to Owner's designated representative for approval.

STEP 13. If the approved mix design proves to provide a product that is outside of the allowable tolerances outlined in the contract documents and the Owner's concrete specification, the production shall be suspended and corrective actions shall be initiated. These actions shall include and be limited to a review and/or change in mix design. Prior to recommencing with material production, the Owner's designated representative shall approve all revisions relevant to any procedural changes and/or concrete mix designs.

## 8. Steel Sampling and Storage

All non-galvanized steel (black steel) items that are used as embeds and/or product reinforcement will be stored of the ground. A representative sample for every 80 tons of mild reinforcement received at the manufacturing facility shall be made available for independent testing to confirm the validity of the mill certifications provided with each steel material shipment.

## 9. Quality Control

As stated in the above procedures, a precise quality control process shall be used to insure the greatest possible consistency of quality that meets or exceeds that as required by the contract documents, specifications, and shop drawings. Listed below is a step-by-step process that shall constitute the minimum quality control program. This process shall be verified and/or monitored at key points during each day's manufacturing process.

### 9.1. Manufacturing Procedure / Production Protocol

STEP 1. Each day's operation will start with the removal of all curing blankets to prepare for the stripping of product from the forms.

STEP 2. Remove the panel anchor holders and all form recesses.

STEP 3. Unbolt all side rails from the pallet to allow the precast product to be unconfined and readied for removal.

STEP 4. Remove product form using appropriate lifting device.

STEP 5. After each product is removed from the forms, and before they are relocated to the 72 hour cure area, the designated quality control representative shall inspect each product to verify the they were manufactured in accordance with the project specifications and are free from chips, spalls, cracks, honeycomb, or any other defects that would be cause for rejection or repairs.

STEP 6. If the quality of the product is acceptable and in compliance with the referenced detail, the individual product will be marked with green paint, or other suitable marking, along the right side edge of the precast panel and on the end for all top of wall treatments that signifies acceptance. If the product is of acceptable quality but requires minor repairs (i.e. minor patching, cleaning of paste from embeds, etc.) the product will be marked with yellow paint, or other suitable marking, in the same designated locations. In addition the product shall be tagged describing the specific repair that is required.

STEP 7. Both green and yellow marked products will be relocated to the 72 hour curing area. All yellow marked products shall be immediately repaired as required.

STEP 8. The yellow marked products will be repaired under the QC Manager's direct guidance or the QC Control representative. Once the repairs have been completed the products shall be inspected to insure that the product meets or exceeds the quality requirements. After the product has been repaired and the repair is accepted, it shall be marked with green paint adjacent to the yellow paint to signify acceptable quality. The paper tag describing the required repair will be removed and filed.

STEP 9. If the quality of the product is unacceptable or it is deemed to be not repairable, the product is to be immediately designated with a red mark along all 4 sides of the precast product. This rejected product will be removed from the 72 hour curing area and located to the Culled Panel area.

STEP 10. Once each of the products have been removed from their respective forms, all loose debris and foreign substance shall be removed and cleaned from the forming surfaces to facilitate the reconstruction of each form in preparation for the day's production.

STEP 11. Once the form(s) have been re-assembled, the forming surface shall be treated with form release agent with the use of hand sprayer or equivalent method.

STEP 12. Prior to the form setup for that day's production, each form will have within its immediate proximity a form detail/drawing that will include a duplication of a specific panel detail as included within the approved shop drawings. This detail will serve as a representation as to the specific panel that is to be manufactured in the respective form. In addition to the detail, a sheet containing a checklist of items to insure quality control shall be placed.

STEP 13. Each form shall be prepared in accordance with the specific detail with special attention addressed to the product dimensions, header locations, panel anchor locations, number of embeds, the embed locations, and the embed orientation.

STEP 14. Once the form preparation has been successfully completed, the quality control representative shall walk the form line and individually inspect each form while comparing the form setup with the specific detail that is still located with that individual form. The QC representative shall verify that the checklist items contained on the form setup drawing and as specified as Pre-Pour Quality Checklist has been properly addressed.

STEP 15. If the specific form being reviewed is acceptable, the form will be labeled with a green acceptance flag, or other suitable marking, to signify that the form is approved for pouring. If during the review the panel form setup is found unacceptable, the form shall be immediately flagged with a red flag, or other suitable marking, and the unacceptable area noted on the form drawing. The production foreman shall be notified immediately, and the form setup shall be immediately corrected, or it shall be removed from that day's production. Once it is corrected the red flag shall be replaced with a green flag.

STEP 16. In final preparation prior to placing concrete in the forms, a final cursory walk through will be conducted to insure all forms have been flagged green. If water and/or foreign debris has managed to accumulate on the flat exposed form surface, the form shall be blown free of foreign matters via use of portable air.

STEP 17. After concrete has been placed in each panel form, it has been screed and finish floated and the panel has achieved the initial set a final panel review shall be made to confirm that all quality issues have been addressed (i.e. proper back face finish, clevis embed alignment, etc.). Any minor imperfection shall be immediately fixed. If the panel is deemed of acceptable quality it shall have the panel name, date, batch/lot number and job number etched in the back face.

STEP 18. Immediately following the panel etching, a post pour review shall be conducted by the Quality Control representative to approve the panel for final acceptance. Once the product has been accepted the form shall be covered with continuous overlapping moist cure blankets for a minimum period of 12 hours.

STEP 19. The production area shall be cleaned.

STEP 20. Each production day will then recommence with the same repeated process as stated above.

### 9.2. PRE-POUR / POST POUR INSPECTION SUMMARY:

STEP 1. Prior to the actual placement of concrete, a final form setup inspection shall be initiated by the quality control representative to assure the form size, setup, and all special requirements are as illustrated in the production drawing.

STEP 2. After the final surface preparations, each panel shall be etched on the exposed concrete surface with the panel name, date, batch/lot number and job number.

STEP 3. Immediately following the panel etching a post-pour inspection shall be conducted by the Quality Control representative to insure proper alignment and condition of the attachment devices, the panel etching, header location, and the product dimensions corresponds with the illustrated production on the detail sheet. If a problem is encounter, and can be remedied, immediate steps will be initiated to correct the problem. All corrections shall be performed under direct supervision of the quality control representative. If the problem can't be corrected, the product in question will be immediately rejected and marked in with red paint.

## 10. MANUFACTURED PRODUCT STORAGE:

### 10.1. GENERAL PRECAST PANEL REQUIREMENTS

All precast panels shall be stored in a safe and accessible manner.

#### 10.1.1. BOTTOM PANEL STORAGE

Under no circumstance shall any precast panel be stored directly in contact with the ground. Depending upon the panel type, 4 x 4 timber dunnage, or a pallet shall be used between the ground and any precast product. The surface of the dunnage or the pallet shall be coated with plastic to prevent staining of the panel face.

#### 10.1.2. Maximum Stack Height

No single stack of panels shall exceed 10 panels in height. Precast panels shall be stored in a manner to insure a safe and stable stack.

#### 10.1.3. Dunnage

A minimum of two (2) pieces of 4 x 4 dunnage shall be used between the bottom panel and the ground. Each piece of dunnage shall have either preco pads or styrofoam attached to each piece of dunnage on the side that will be in contact with the exposed precast face to prevent or minimize any panel deformations or face scaring.

#### 10.1.4. Intermediate Panel Storage

All intermediate panels positioned above the bottom panel require either 2 continuous pieces of 4 x 4 dunnage or 4 pieces of 4 x 4 blocking placed at quarter points to insure the panel's stability. The dunnage shall be positioned in a way to assist the stack stability. Placement of the dunnage shall insure that they are tall enough so the panel does not come in contact panel embeds of other items extending from the panel face. Further, steps shall be made to prevent scaring or staining at the end of the dunnage that is in contact with the panel's front face.

#### 10.1.5. Stack Configuration

Place the largest precast panels on the lower portion of each respective panel stack with the smaller partial pieces being positioned higher in the stack. The exception to this procedure would be the strategic combining of panels in order to create a full size panel made up of smaller precast panels. Larger pieces of 4 x 4 dunnage or additional dunnage may be required to insure the stack's stability.

### 10.2. Other Precast Element Storage

When storing precast top of wall treatments (i.e. precast traffic barrier, precast coping and precast parapet), 2 pieces of continuous 4 x 4 shall be positioned (1 at each end) across the product length at the approximate quarter point from each end of the product.

## 11. Product Repair Classification

### 11.1. 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.

### 11.2. Honeycombing

A series of voids in the concrete that may be caused by the loss of fines or other material between the aggregate particles, the inclusion of air pockets between aggregate particles, or larger volumes of lost material.

### 11.3. Spall

A depression in the panel that is a result of a fragment of concrete being detached from the larger mass of concrete and can be caused by impact, the action of weather, uneven pressure, or uncontrolled expansion.

#### 11.3.1. Cosmetic

A circular or oval depression not greater than 1.0 inch in depth no greater than 3.0 square inches in area

#### 11.3.2. Minor

A spall no larger than 1.0 square foot and no deeper than 1.5 inches.

### 11.4. Chip

The local breaking of corners or edges of the concrete that results in a void containing angular surfaces.

#### 11.4.1. Cosmetic

Cosmetic chips are chips where the sum of the two lateral dimensions perpendicular to the length does not exceed 2.0 inches.

#### 11.4.2. Minor

Chips are where the sum of the two lateral dimensions perpendicular to the length exceeds two inches, but does not exceed four inches, and with a length of no more than 12 inches.

### 11.5. Major Concrete Deficiencies

In an effort not to supply any product of a compromising structural nature 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 shall deem that the product is rejected and not eligible to be repaired.

## 12. Product Repair Methods

The Quality Control Manager will examine all deficiencies and will determine the specific nature of the repairs and the most appropriate course of action required to correct the deficiency. The correction can range from minor cleaning of connections/embeds up to, and including, minor concrete deficiencies. All minor deficiencies shall be listed and described on a "Minor Repair Record" sheet. Furthermore, all concrete deficiencies shall be classified as, non-repairable (major), or repairable (cosmetic or minor).

All repairs will be conducted under direct supervision of the Quality Control manager in a manner to insure appropriate strength and quality. All repairs shall be made in a manner that is acceptable to the engineer.

### 12.1. Major / Non-repairable

All product containing deficiencies exceeding cosmetic or minor definitions described above, shall be deemed un-repairable and shall be physically marked by red paint or grease pencil along all 4 sides and shall be relocated to the rejected/culled area of the manufacturing facility until the rejected product can be relocated off-site to the disposal facility.

### 12.2. Cosmetic / Minor

All minor cosmetic repairs shall be repaired by either pointing the product with 1 part sand and 1 part cement, which would typically be accomplished while the product is or about to be placed into the 72 hour cure area, or at the discretion of the Q.C. manager with a specifically approved patching product contained on the Owner's Qualified Product List (QPL). The approved repair products to be considered for use include the following products: Lambert Epiweld 560 / 580 epoxy bonding agent, Euclid Euco-Speed MP, Lambert Vibropruf #11, SikaQuick 1000, Bonsal Fast Set Cement, 1 part sand and 1 part cement paste and as necessary white Portland cement. All repairs shall match the product color and shall insure proper blending. The product shall be prepared and applied in accordance with the manufacture recommendations. The actual concrete repair procedure shall include; proper surface preparation, the application of an epoxy bonding compound to the affected area, proper preparation, and use of patching material and final shaping/texturing and grinding to insure the proper product blending.

## 13. Handling of Failed or Rejected Products:

### 13.1. Rejected Product

All manufactured precast products that have been deemed rejected during the manufacturing, curing, or storage process shall be immediately physically marked by red paint or grease pencil and immediately relocated to the rejected/culled area of the manufacturing facility until the rejected product can be relocated to a disposal facility off-site.

### 13.2. Rejected Component

Prior to the acceptance of any raw matter or material used in the manufacturing process, each and every product shall be physically checked, tested and/or mill certifications confirmed as acceptable for use. If a specific item is found unacceptable, or is found not to meet the project requirements, the item in question will be immediately refused for unloading and be sent back to the supplier for disposal.

### 13.3. Rejected Product at Job-Site

If product that has been unloaded and that is determined to be unacceptable for use, it shall be immediately marked rejected with red paint and relocated to the rejected/ cull area of the facility until the material can be permanently removed from the manufacturing site.

### 13.4. Rejected at Cure site

During the product curing process, if the products are found to have insufficient material strength, all the materials produced during that time and containing the relevant LOT number will be immediately marked rejected and relocated to the facility's rejected/culled area until permanent removal can occur.

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# 4.1.2
# Soil Reinforcing QA/QC

EARTHTEC

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EARTHTEC



GROUND IMPROVEMENT SYSTEMS LLC
114 SOUTH COLLINS STREET
ARLINGTON, TX 76011

# REINFORCED SOIL WALL
# FABRICATION QUALITY CONTROL GUIDELINES
# SOIL REINFORCING AND PANEL ANCHOR

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EARTHTEC™

**CONTENTS**

## 1 GENERAL

Work includes the furnishing of EarthTrac™ soil reinforcement and panel anchors.

## 2 REFERENCES

### 2.1 AMERICAN SOCIETY FOR TESTING AND MATERIALS (ASTM)

#### 2.1.1 A36 – STANDARD SPECIFICATION FOR CARBON STRUCTURAL STEEL

#### 2.1.2 A123 - STANDARD SPECIFICATIONS FOR ZINC (HOT-DIP GALVANIZED) COATINGS ON IRON AND STEEL PRODUCTS

#### 2.1.3 A153 - STANDARD SPECIFICATION FOR ZINC COATING (HOT-DIP) ON IORN AND STEEL HARDWARE

#### 2.1.4 ZINC COATING (HOT-DIP) ON IRON AND STEEL HARDWARE

#### 2.1.5 A572 – STANDARD SPECIFICATION FOR HIGH-STRENGTH LOW-ALLOY COLUMBIUM-VANADIUM STRUCTURAL STEEL

#### 2.1.6 A615 - STANDARD SPECIFICATION FOR DEFORMED AND PLAIN CARBON-STEEL BARS FOR CONCRETE REINFORCEMENT

#### 2.1.7 A780 - STANDARD SPECIFICATION FOR THE REPAIR OF DAMAGED AND UNCOATED AREAS OF HOT-DIP GALVANIZED COATINGS

#### 2.1.8 A1011 - 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

#### 2.1.9 A1059 - STANDARD SPECIFICATION FOR ZINC ALLOY THERMO-DIFFUSION COATINGS (TDC) ON STEEL FASTENERS, HARDWARE, AND OTHER PRODUCTS

## 3 DEFINITIONS

### 3.1 EARTHTRAC™ REINFORCEMENT

EarthTrac™ soil reinforcement is a material composed of hot-rolled steel or cold-worked steel plate that is fabricated into discrete strips.

The X following the SS is the type of strip. Type SS1 is a hot-rolled steel strip and type SS3 is a A1011 cold worked steel strip. Both strips have ribs that extend across the width of the plate.

### 3.2 EARTHTRAC™ THICKNESS

The EarthTrac SS1 is fabricated from material with a nominal thickness equal to or 3/16” (0.1875). The EarthTrac SS3 soil reinforcing is typically fabricated from material with a nominal thickness equal to 5/32 (0.1563). However, other thicknesses can be fabricated when required.

### 3.3 EARTHTRAC™ WIDTH

The EarthTrac soil reinforcing is typically fabricated from material with a nominal width equal to 2” (2.00). However, other widths can be fabricated when required.

### 3.4 EARTHTRAC™ RIB

The EarthTrac soil reinforcing steel strip is fabricated with a series of ribs that are parallel to width of the strip. The ribs add a three-dimensional affect to the strip and provide passive resistance to pullout.

#### 3.4.1 EARTHTRAC SS1

The SS1 rib is hot rolled into the steel strip.

#### 3.4.2 EARTHTRAC SS3

The SS3 rib is cold formed into the steel strip.

### 3.5 EARTH-TRAC™ LENGTH

The EarthTrac™ soil reinforcing length shall be measured from the proximal end of the strip to the terminal end of the strip.

## 4 QUALITY ASSURANCE

All Earth-Trac™ soil reinforcing shall be manufactured and fabricated to conform to the appropriate ASTM specifications and EarthTec performance specifications.

### 4.1 EARTHTRAC™ REINFORCEMENT TOLERANCES

#### 4.1.1 EARTHTRAC™ WIDTH

The permissible variation shall not exceed ± 1/16 inch.

#### 4.1.2 EARTHTRAC™ LENGTH

The overall length may vary by ±1 inch or 1% whichever is greater.

### 4.2 REQUIRED TESTING

All tests shall be performed in conformance with ASTM including modifications specified herein.

#### 4.2.1 TENSILE STRENGTH

Tensile strength shall be in conformance with ASTM and on each coil of plate. A minimum of 2 samples selected from each end of the coil and 1 sample selected at random for a total of a minimum of 5 tests shall be tested. The manufacture shall have sufficient traceability that it is possible to determine what coil was used in the fabrication process.

#### 4.2.2 OTHER TESTS

The Owner may conduct additional tests on the soil reinforcement to assure compliance with these specifications in a manner specified herein.

### 4.3 NONCOMPLIANCE

Any noncompliance demonstrated by any of these tests shall be cause for rejection of the material represented by the test samples.

## 5 STORAGE, HANDLING, AND DELIVERY

### 5.1 STORAGE

All EarthTrac™ soil reinforcement shall be stacked and stored based on the length. Bundles shall be in quantities of 25 or 50, except where remainder quantities are required to complete a specific length for an order. Bundles shall be stacked on their sides. Each bundle shall be banded in no less than two locations. Each bundle shall be tagged and marked with at least two redundant tags in accordance with the EarthTec requirements. Bundles of 5 stacks shall be banded together.

### 5.2 DELIVERY

Deliver EarthTrac™ soil reinforcement to project in undamaged condition. Bundles shall be separated by dunnage.

### 5.3 LIFTING OF BUNDLES WITH CRANE

Lift the EarthTrac soil reinforcement using a lifting bar and sling that is placed over the bundles. A minimum of two slings shall be used per bundle.

Lifting slings shall extend around the EarthTrac and no closer than 12” to the ends. A crane shall be used to grasp the lifting slings.

### 5.4 LIFTING WITH FORKLIFT

Loading and unloading material with a forklift requires the EarthTrac™ soil reinforcing to be supported by dunnage. Make sure that forks do not damage the individual steel strips and that the forks are passed under the bundle near a location where dunnage has raised the material off the ground. Long soil reinforcing when lifted with narrow spaced forks may sag down near the terminal and proximal ends. Operators must use extreme care when placing the soil reinforcing on the ground to prevent the terminal and proximal ends from becoming snagged, bent, or damaged.

### 5.5 STORAGE

Store EarthTrac™ soil reinforcing in a protected area to limit the potential for surface deterioration caused by prolonged exposure to conditions that accelerate the oxidation of steel or the galvanized coating. Store all material in an area that prevents the accumulation of dirt and oil. Protect reinforcement, panel anchors, and metal accessories from permanent distortion. Store the material off the ground using appropriate dunnage.

## 6 EARTHTRAC™ SOIL REINFORCING

### 6.1 REINFORCEMENT

All EarthTrac SS1 soil reinforcement shall be fabricated in conformance with ASTM A572. All EarthTrac SS3 soil reinforcement shall be fabricated in conformance with ASTM A1011.

### 6.2 YIELD STRENGTH

EarthTrac SS1 and EarthTrac SS3 soil reinforcement shall have a minimum yield strength of 50,000 psi.

### 6.3 Domestic Manufacturing

When specified to do so, all soil reinforcement shall be manufactured from domestic steel. No foreign steel or foreign billets used in manufacturing processes will be permitted.

### 6.4 Foreign Manufacturing

When allowed by project requirements, all soil reinforcement that is manufactured from foreign steel shall conform to ASTM A572 or ASTM A1011, no exceptions.

### 6.5 Finished EarthTrac™

EarthTrac™ soil reinforcement shall be furnished in flat strips.

## 7 Panel Anchors

The panel anchor shall consist of a Tie-Strip and shall be fabricated from ASTM A1011 Grade 50 steel in accordance with EarthTec dimensions as shown on the material fabrication sheet.

## 8 Steel Coatings

### 8.1 Galvanizing

#### 8.1.1 EarthTrac™

All galvanizing for EarthTrac™ soil reinforcement shall be in accordance with ASTM A153 or ASTM A123.

#### 8.1.2 Connection Accessories.

All galvanizing for connection shall be in accordance with ASTM A153, ASTM A123 or ASTM A1059. Coating shall be a minimum of 2 oz/sf or a thickness equal to 3.4 mils.

## 9 Galvanized Coating Repair

### 9.1 Damaged Material

All visible damage (i.e., scratches, nicks, cracks) to the galvanized coating of the soil reinforcing, caused during shipment, storage or placement shall be repaired by the Contractor at the job site in accordance with ASTM A780.

### 9.2 Cut Material

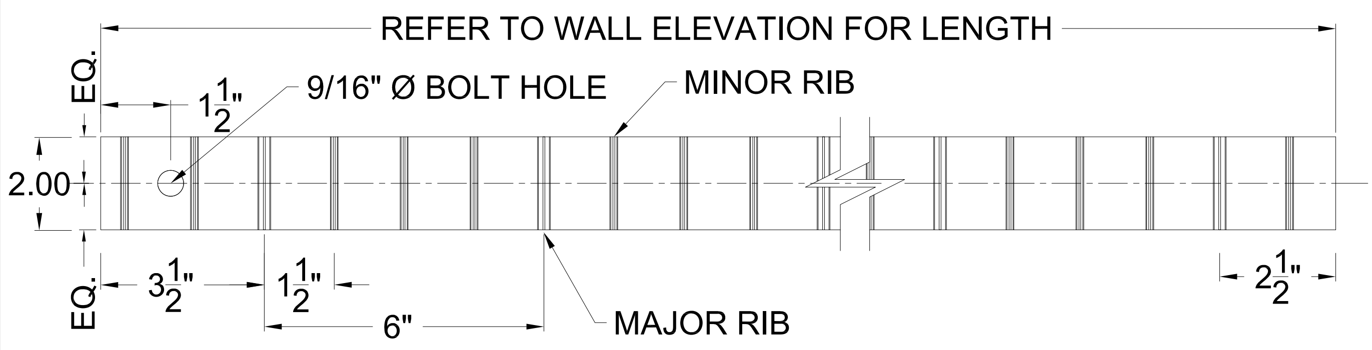
All ends of reinforcement that have been sheared, sawed, or cut by other means shall be coated with appropriate zinc-rich materials.

### 9.3 Zinc coating

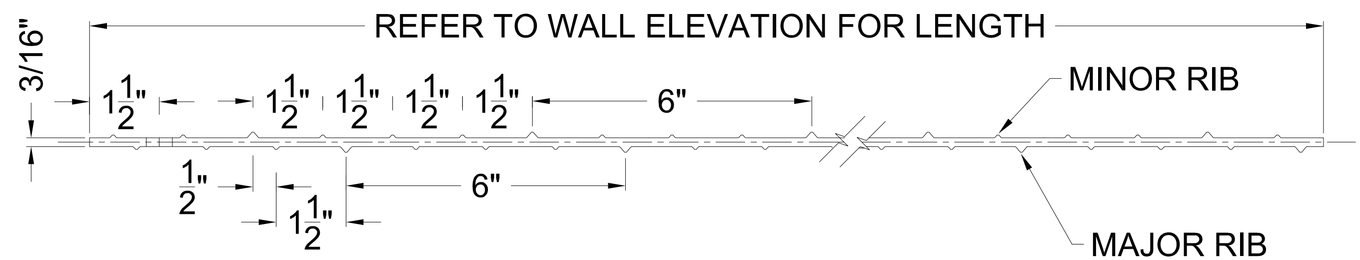
The applied zinc coating shall conform to ASTM A780 and shall be applied to achieve a dry film equal to or exceeding that designated in the contract documents. All touchup shall be cured fully prior to placement.

Appendix A

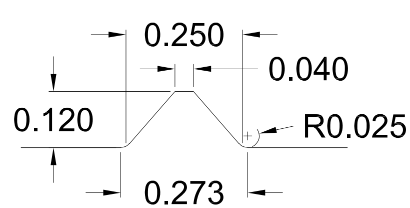
EarthTrac Soil Reinforcing



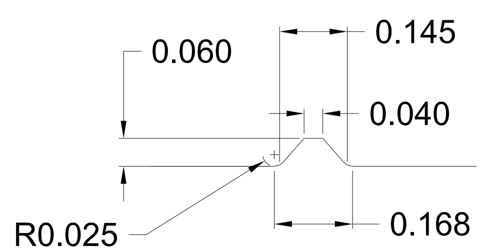

PLAN DETAIL OF EARTHTRAC STEEL STRIP (SS1)




SIDE DETAIL OF EARTHTRAC STEEL STRIP (SS1)




EARTHTRAC SS1 MAJOR RIB




EARTHTRAC SS1 MINOR RIB

1. Specifications
   1.1. ASTM A572 - Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel.
2. Acceptance

   All material will be accepted on the basis of the required certification and testing required by the ASTM specifications.
3. Material

   The soil reinforcing shall be Grade 50 steel and have a minimum tensile capacity of 65 ksi and a minimum yield capacity of 50 ksi.
4. Tolerances

   The permissible variation from the dimensions and configuration shown shall be as follows:
   4.1. Overall Plate Vertical Dimensions$\pm \frac{1}{8}$
   4.2. Overall Plate Horizontal Dimensions$\pm \frac{1}{8}$
   4.3. Overall Hole Dimension$\pm \frac{1}{64}$
   4.4. Overall Edge Distance$\pm \frac{1}{32}$
5. Certification

   The manufacturer shall furnish certification of all material. The certification shall indicate that the components are in accordance with this specification and shall include typical test results representative of the material. The certification shall be notarized.

| | | | |
|---|---|---|---|
| GIS PN | Project Engineer | | |
| SCALE N.T.S. | Drawn By tpt | Date Drawn | |
| | Checked By: | Date Checked | |
| | Approved By: | Date Approved | |

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EARTHTRAC SS1

Fabrication Drawing

GS1

**SS1**

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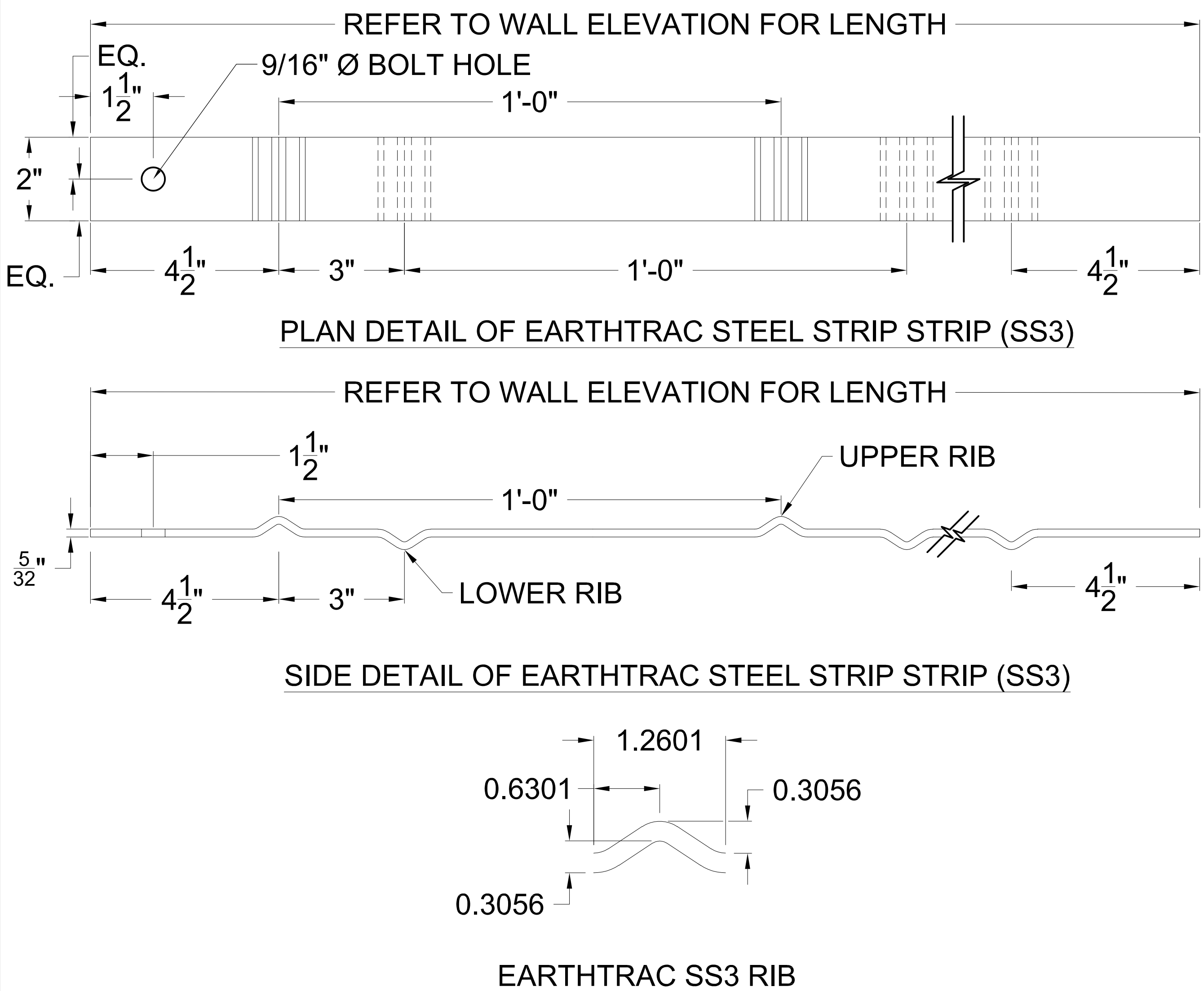


1. Specifications
   1.1. ASTM A1011 Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, and Ultra-High Strength.
2. Acceptance

   All material will be accepted on the basis of the required certification and testing required by the ASTM specifications.
3. Material

   The soil reinforcing shall be Grade 50 steel and have a minimum tensile capacity of 65 ksi and a minimum yield capacity of 50 ksi.
4. Tolerances

   The permissible variation from the dimensions and configuration shown shall be as follows:
   4.1. Overall Plate Vertical Dimensions$\pm \frac{1}{8}$
   4.2. Overall Plate Horizontal Dimensions$\pm \frac{1}{8}$
   4.3. Overall Hole Dimension$\pm \frac{1}{64}$
   4.4. Overall Edge Distance$\pm \frac{1}{32}$
5. Certification

   The manufacturer shall furnish certification of all material. The certification shall indicate that the components are in accordance with this specification and shall include typical test results representative of the material. The certification shall be notarized.

| GIS PN | Project Engineer | | |
|---|---|---|---|
| SCALE N.T.S. | Drawn By tpt | Date Drawn | |
| | Checked By: | Date Checked | |
| | Approved By: | Date Approved | |

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EARTHTRAC SS3

Fabrication Drawing

GS1

SS3

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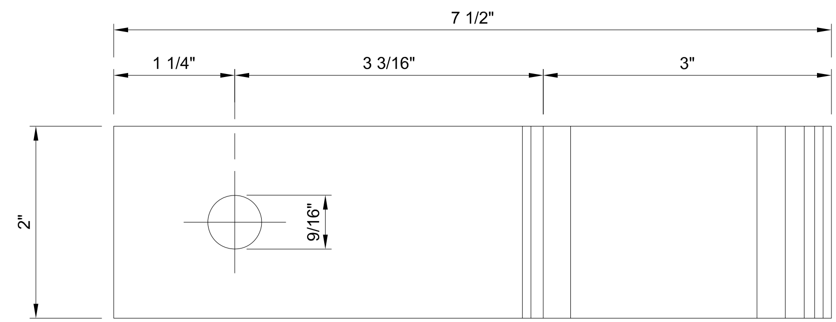
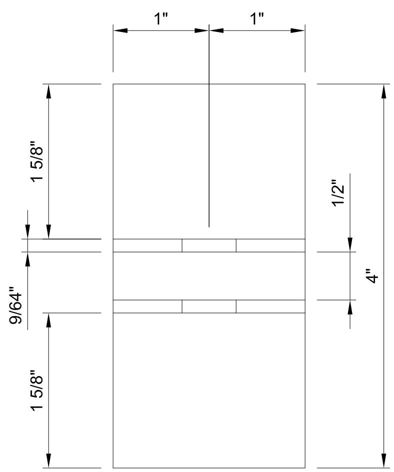
# Appendix B

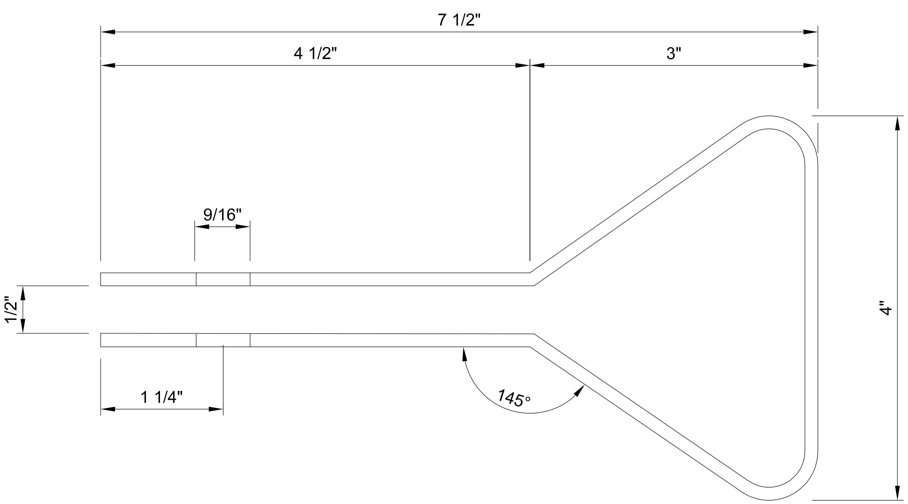
# EarthTrac Panel Anchor




TOP VIEW




END VIEW




PROFILE VIEW

1. Specifications
   1.1. ASTM A1011 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.
2. Acceptance

   All material will be accepted on the basis of the required certification and testing required by the ASTM specifications.
3. Material

   The TS10G Anchor shall be Grade 50 steel and have a minimum tensile capacity of 65 ksi and a minimum yield capacity of 50 ksi.
4. Tolerances

   The permissible variation from the dimensions and configuration shown shall be as follows:
   4.1. Overall Plate Vertical Dimensions$\pm\frac{1}{8}$
   4.2. Overall Plate Horizontal Dimensions$\pm\frac{1}{8}$
   4.3. Overall Hole Dimension$\pm\frac{1}{64}$
   4.4. Overall Edge Distance$\pm\frac{1}{32}$
5. Certification

   The manufacturer shall furnish certification of all material. The certification shall indicate that the components are in accordance with this specification and shall include typical test results representative of the material. The certification shall be notarized.

| | | |
|---|---|---|
| GIS PN | Project Engineer | |
| SCALE N.T.S. | Drawn By tpt | Date Drawn |
| | Checked By: | Date Checked |
| | Approved By: | Date Approved |

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Panel Anchor
Unit Wt. = 1.41 lbs

Fabrication Drawing

GSI

PA

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# 4.1.3
# Miscellaneous Component QA/QC

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# EARTHTEC™ REINFORCED SOIL WALL
# QUALITY CONTROL GUIDELINES
# INCIDENTAL COMPONENTS


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Support Through Innovation

# MECHANICALLY STABILIZED EARTH STRUCTURES

## REINFORCED SOIL WALL (RWS) SYSTEM

## QUALITY CONTROL GUIDELINES

## INCIDENTAL COMPONENTS

**GROUND IMPROVEMENT SYSTEMS LLC**

114 South Collins Street

Arlington, TX 76011

817-223-0969

# TABLE OF CONTENTS

## 1 GENERAL

The following material parameter specifications cover the incidental items required to construct the Ground Improvement Systems Reinforced Soil Wall system.

## 2 BOLT SET

This section covers the bolt set that is used to join the soil reinforcing to the panel anchor. The bolt set shall have a minimum diameter of ½” and conform to ASTM A325. The bolt set consist of a bolt and nut.

### 2.1 Manufacturer Certification of Fastener Components

Manufacturer certifications documenting conformance to the applicable specifications required in Sections 2.3 through 2.6 for all fastener components used in the fastener assemblies shall be available to the Retaining Wall Engineer of Record and inspector prior to assembly or erection of structural steel.

### 2.2 Storage of Fastener Components

Fastener components shall be protected from dirt and moisture in closed containers at the site of installation. Only as many fastener components as are anticipated to be installed during the work shift shall be taken from protected storage. Fastener components that are not incorporated into the work shall be returned to protected storage at the end of the work shift. Fastener components shall not be cleaned or modified from the as-delivered condition. Fastener components that accumulate rust or dirt shall not be incorporated into the work unless they are prequalified.

### 2.3 Heavy-Hex Structural Bolts

#### 2.3.1 Specifications

Heavy-hex structural bolts shall meet the requirements of ASTM A325 or ASTM A490. The Engineer of Record shall specify the ASTM designation and type of bolt to be used.

#### 2.3.2 Geometry

Heavy-hex structural bolt dimensions shall meet the requirements of ANSI/ASME B18.2.6. The bolt length used shall be such that the end of the bolt extends beyond or is at least flush with the outer face of the nut when properly installed.

### 2.4 Heavy-Hex Nuts

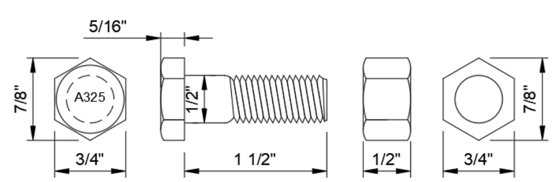
#### 2.4.1 Specifications

Heavy-hex nuts shall meet the requirements of ASTM F3125 and shall be A325 Type-1.

#### 2.4.2 Geometry

Heavy-hex nut dimensions shall meet the requirements of ANSI/ASME B18.2.6.

### 2.5 Bolt Dimensions




The thread dimensions shown are 1”. The threads may be ½” if special bolts are ordered.

### 2.6 Alternative-Design Fasteners

When approved by the Retaining Wall Engineer of Record, the use of alternative-design fasteners is permitted if they:

(1) Meet the materials, manufacturing and chemical composition requirements of ASTM A325 or other designated specification.

(2) Meet the mechanical property requirements of ASTM A325 in full-size tests or other designated specification

(3) Have a body diameter and bearing area under the bolt head and nut that is equal to or greater than those provided by a bolt and nut of the same nominal dimensions specified;

(4) Are supplied and used in the work as a fastener assembly.

## 3 PANEL ANCHOR

This section covers the Tie-Strip Panel Anchor that is joined with the proximal end of the soil reinforcing.

### 3.1 Specifications

**3.1.1 ASTM A572 - Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel.**

**3.1.2 ASTM A1011 - 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**

### 3.2 Acceptance

All material will be accepted on the basis of the required certification and testing required by the ASTM specifications.

### 3.3 Material

The panel anchor shall be Grade 50 steel and have a minimum tensile capacity of 65 ksi and a minimum yield capacity of 50 ksi.

### 3.4 Tolerances

The permissible variation from the dimensions and configuration shown on the Tie-Strip Panel Anchor Cut Sheet prepared by GIS shall be as follows:

| | |
|---|---|
| Overall Vertical Dimensions | ± 1/8 |
| Overall Horizontal Dimensions | ± 1/8 |
| Overall Hole Dimension | ± 1/64 |
| Overall Edge Distance | ± 1/32 |
| Overall Shaft Diameter | ± 1/32 |
| Overall Disk Diameter | ± 1/16 |

### 3.5 Certification

The manufacturer shall furnish certification of all material. The certification shall indicate that the components are in accordance with this specification and shall include typical test results representative of the material. The certification shall be notarized.

## 4 BEARING PADS

This section covers the bearing pads that are placed between panels at the top of edge to prevent concrete to concrete contact. The bearing pads shall be Styrene-butadiene rubber (SBR).

### 4.1 Specifications

**4.1.1 ASTM D2240 - Standard Test Method for Rubber Property—Durometer Hardness**

**4.1.2 ASTM D412 - Standard Test Methods for Vulcanized Rubber and Thermoplastic Elastomers—Tension**

**4.1.3 ASTM D573 - Standard Test Method for Rubber—Deterioration in an Air Oven**

**4.1.4 ASTM D395 - Standard Test Methods for Rubber Property—Compression Set**

**4.1.5 ASTM D1149 - Standard Test Methods for Rubber Deterioration-Cracking in an Ozone Controlled Environment**

**4.1.6 ASTM D429 - Standard Test Methods for Rubber Property—Adhesion to Rigid Substrates**

**4.1.7 ASTM D746 - Standard Test Method for Brittleness Temperature of Plastics and Elastomers by Impact**

### 4.2 Acceptance

All material will be accepted on the basis of the required certification and testing required by the engineer.

### 4.3 Material

The bearing pad shall be 100 percent virgin Styrene Butadiene Rubber (SBR, Buna-S) compound meeting the requirements shown below. The pads shall be of the Durometer Grade specified on the plans. If test specimens are cut from the finished product, a 10 percent variation in "Physical Properties" will be allowed.

### 4.4 Property Criteria

The following properties will be tested and adhere to the minimum values provided in the table.

| **Bearing Pad Property Criteria** | | |
|---|---|---|
| Durometer Grade | | 60 |
| **ASTM** | **Property** | **Value** |
| D2240 | Hardness | 60 ± 5 |
| D412 | Tensile Strength (psi) | 2500 |
| D412 | Ultimate elongation (% min) | 350 |
| D573 | Change in Durometer Hardness | +15 |
| | Change in Tensile Strength (% max) | -15 |
| | Change in Ultimate Elongation (% max) | -40 |
| D395 | Compressive Set 22 hars at 212 F (% max) | 35 |
| D1149 | Ozone 100 pphm ozone in air by volume | No Cracks |
| D429 | Adhesion Bond made during vulcanization | 40 |
| D746 | Lowe Temperature Test for Brittleness | No Failure |

### 4.5 Tolerances

For both plain and laminated bearings, the permissible variation from the dimensions and configuration shown on the plans shall be as follows.

| Bearing Pad Tolerance Table | |
|---|---|
| Overall Vertical Dimensions Average total thickness 1 1/4 inches (32 mm) or less | -0, +1/8 |
| Overall Horizontal Dimensions 36 inches (914 mm) and less | -0, +1/4 |
| Variation from a Plane Parallel to the Theoretical Surface (as determined by measurements at the edges of bearings) | |
| Top | 1/8 |
| Sides | 1/4 |
| Individual non-elastic laminates | 1/8 |

### 4.6 Certification

The manufacturer shall furnish certification of all material. The certification shall indicate that the components are in accordance with this specification and shall include typical test results representative of the material.

## 5 GEOTEXTILE

This section covers the geotextile that is used to cover the joints at the back of the segmental concrete panel. The geotextile consists of needle punched nonwoven fabric.

### 5.1 Specifications

**5.1.1 ASTM D4632 - Standard Test Method for Grab Breaking Load and Elongation of Geotextiles**

**5.1.2 ASTM D6241 - Standard Test Method for Static Puncture Strength of Geotextiles and Geotextile-Related Products Using a 50-mm Probe**

**5.1.3 ASTM D4533 - Standard Test Method for Trapezoid Tearing Strength of Geotextiles**

**5.1.4 ASTM D4751 - Standard Test Method for Determining Apparent Opening Size of a Geotextile**

**5.1.5 ASTM D4491 - Standard Test Methods for Water Permeability of Geotextiles by Permittivity**

**5.1.6 ASTM D4355 - Standard Test Method for Deterioration of Geotextiles by Exposure to Light, Moisture and Heat in a Xenon Arc Type Apparatus**

### 5.2 Acceptance

All material will be accepted on the basis of the required certification and testing required by the engineer.

### 5.3 Material

The shall consist of a needle punched fabric that is resistance to ultraviolet degradation and to biological and chemical attack normally found in soils.

### 5.4 Property Criteria

The following properties will be tested and adhered to the values provided for in AASHTO M288 as given in the table. Other product not meeting these requirements shall be reviewed on a project by project basis.

| Mechanical | ASTM | MARV |
|---|---|---|
| Grab Tensile Strength | D4632 | 300 lbs |
| Puncture Strength <50% Elongation | D6241 | 100 lbs |
| Trapezoidal Tear | D4533 | 125 lbs |

| Hydraulic | ASTM | Property |
|---|---|---|
| Maximum Apparent Opening Size (AOS) | D4751 | 40 US Standard Sieve |
| Permittivity | D4491 | 0.50 $sec^{-1}$ |

| Endurance | ASTM | Property |
|---|---|---|
| UV Resistance (% Retained after 500 hours) | D4355 | 50% |

### 5.5 Certification

The manufacturer shall furnish certification of all material. The certification shall indicate that the components are in accordance with this specification and shall include typical test results representative of the material.

## 6 ADHESIVE

This section covers the adhesive that is used to attach the filter fabric that is used to cover the joints at the back of the segmental concrete panel. The adhesive is a used to adhere the geotextile to the panel until the backfill has been place. The adhesive is supplied by the installer.

### 6.1 Specifications

No specifications cover the adhesive. The adhesive is supplied by the installer. Typical adhesive that is used consists of subfloor adhesive that is used with concrete applications. The adhesive shall be applied so the geotextile fabric makes full contact with the back of the MSE panel.

### 6.2 Material

The adhesive shall consist of any material that can be used to attach a material to a concrete surface. The material shall be resistance to sunlight and durable under moisture and temperature conditions.

### 6.3 Certification

No certification is required.

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# 4.2
# CONSTRUCTION

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EARTHTEC

**4.2.1**
**Construction QA/QC**

EARTHTEC

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GROUND IMPROVEMENT SYSTEMS LLC
114 SOUTH COLLINS STREET
ARLINGTON, TX 76011

# EARTHTEC™ REINFORCED SOIL WALL
# TECHNICAL SPECIFICATION
## PRECAST SEGMENTAL CONCRETE PANEL
## WITH EARHTRAC™ SOIL REINFORCING


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EARTHTEC™

# MECHANICALLY STABILIZED EARTH STRUCTURES

## REINFORCED SOIL WALL (RWS) SYSTEM

## TECHNICAL SPECIFICATION
## PRECAST SEGMENTAL CONCRETE PANEL
## WITH
## EARTHTRAC™ SOIL REINFORCING

**GROUND IMPROVEMENT SYSTEMS LLC**
114 South Collins Street
Arlington, TX 76011
817-223-0969



The information outlined in this document, including but not limited to all technical and engineering data, figures, tables, designs, drawings, details, procedures, specifications, and means and methods, is intended for this project. Every effort has been made to ensure design accuracy. This information should not be used or relied on for any application other than the intended project. Anyone making use of this information other than for this project does so at their risk and assumes all liability resulting from such use.

**CONTENTS**

## 1 DESCRIPTION

This work shall consist of designing, fabricating, and constructing the EarthTec™ Reinforced Soil Wall (RSW) with, soil reinforcing. The soil reinforcing is attached to a segmental concrete panel (SCP). All EarthTrac™ soil reinforcing, SCP, and accessories shall be fabricated and supplied in accordance with the project specification. The RSW shall be constructed in accordance with the lines, grades, dimensions, tolerances, and design shown on the approved shop drawings.

Design of the RSW shall include all loading from standard highway traffic, pavement and wearing surfaces, seismic loading (when applicable), vertical load from coping and traffic barrier, traffic impact (when applicable), precast segmental concrete panel, leveling pad, finish grade, required foundation preparation and treatment.

The Engineer of Record (EOR), Owner and or Geotechnical Engineer shall ensure that the in-situ foundation soil's bearing capacity is capable of supporting the applied uniform distribution of the stresses imparted by the Mechanically Stabilized Earth (MSE) structure. The in-situ foundation soil, the retained soil, the reinforced soil, and soil reinforcing shall be of a quality that prevents the formation of any failure surface that adversely affects the external sliding, limited eccentricity, compound stability, and global stability of the structure. The Contractor is responsible for verifying the select backfill, retained backfill, and insitu foundation soil meet the project specifications and design requirements before the execution of the work. Ground Improvement Systems (GIS) is responsible for the internal stability of the RSW only.

## 2 MATERIALS

### 2.1 General

The RSW with EarthTrac™ soil reinforcement is a proprietary product. The Contractor shall make all arrangements to purchase the wall materials (precast panels, EarthTrac™ soil reinforcing, accessories, and design) from an approved EarthTec supplier.

### 2.2 Precast Segmental Concrete Panel

The precast SCP shall be of a rectangular or a square shape, and the nominal dimensions shall conform to the details shown on the final construction drawings.

#### 2.2.1 Compressive Strength

Concrete shall have a compressive strength not less than 4000 psi. All add mixtures, and air entrainment shall conform to the project specifications. The maximum aggregate size shall be in conformance with project specifications and AASHTO.

### 2.2.2 Panel Reinforcing

Panel reinforcing shall consist of welded wire mesh conforming to ASTM A1064 and have a minimum yield strength equal to 65 ksi. Reinforcing bars shall conform to ASTM A615 Grade-60 and have a minimum yield strength equal to 60 ksi. All SCP reinforcement shall be placed as shown on the approved shop drawings and shall conform to the project specifications. All panel reinforcing shall be black steel unless the project specification requires a coating. Care must be exercised to ensure the black steel panel reinforcement does come into direct contact with any galvanized panel anchor or galvanized embedded component. The minimum concrete cover and edge distance shall be 1.5-inches.

### 2.2.3 Casting

The SCP shall be cast face down on flat, level surface in steel forms. Panel anchors and lifting inserts shall be cast in place to the dimensions and tolerances shown on the approved shop drawing. The panel anchors shall be placed in the form before the concrete placement using a panel anchor holder that prevents movement. The concrete placed in each form shall be performed without interruption and shall be compacted by using an approved vibration method such as pneumatic vibrators to ensure that the concrete flows into all corners of the forms, under and between all reinforcement, and into all pockets in the panel anchors. The vibration method that is utilized shall prevent the formation of air pockets, aggregate separation, and cleavage planes. Clear form oil shall be used in the forms. The form oil shall be of a type that prevents staining, sun halos, and color variation from appearing on the panel's front finished face. If a form liner is used, it shall be placed in the form flat with no bumps or depressions.

### 2.2.4 Curing

The precast SCP shall be cured for a sufficient length of time as approved by the Engineer so that the concrete develops the required compressive strength or develops enough strength that the SCP can be removed from the form without damage. Only fresh potable water shall be used for curing.

### 2.2.5 Removal of SCP

The SCP shall remain fully supported within the form until they can be removed, handled, and stacked without damage. This commonly occurs after the product remains in the precast panel form for a minimum of 12 hours under optimum curing conditions or after the product has reached a minimum stripping strength of 1,250 psi.

### 2.2.6 Identifying

The date of manufacture, SCP panel identifying name, and RSW project name, and project number shall be scribed on the rear face of each panel.

### 2.2.7 Concrete Finish

The front (exposed) face of the elements shall have the finish specified in the project specification and approved by the Engineer. The rear face shall have a rough screed finish. No open pockets of aggregates shall be allowed. Care should be exercised when screeding around the panel anchors.

### 2.2.8 Tolerances

All panels shall be manufactured within the following tolerances:

- All dimensions are within 3/16”.
- The evenness of the front face is within ± 1/8” over 5 feet.
- Surface defects on textured finished surfaces measured on a length of 5 feet shall not exceed 5/16 inch.
- Difference between lengths of two diagonals within ½” or the difference between two parallel vertical dimensions are within ½”
- Thickness shall be as specified in the project specifications and approved shop drawings but shall be a minimum 5 ½”, not including form liner finish.
- The lateral position of panel anchors shall be within 1 inch.
- Panel anchors shall not be skewed more than 2 degrees.

### 2.2.9 Handling, Storage and Transporting

All SCP shall be handled, stored, and transported in such a manner as to eliminate the danger of chipping, cracks, fractures, and excessive bending stresses. Panels, when stored, shall be supported on firm blocking located adjacent to the panel anchor. The blocking shall be a height that prevents bending and damage to the panel anchor. All blocking and dunnage are the property of the precaster and shall be returned to the precaster periodically throughout the delivery process. Handling shall be performed with a forklift or with the lifting inserts cast in the SCP. At no time is the panel handled by attaching any lifting device, sling, hook, chain, etc., to the panel anchor.

### 2.2.10 Compressive Strength Acceptability

The acceptance of concrete units for compressive strength will be determined based on production lots. A production lot is represented as a single compressive strength sample and will not be more than 80 panels or one day’s production, whichever is less.

### 2.2.11 Concrete Sampling

Concrete will be sampled for each production lot in accordance with AASHTO T-141. A minimum of four cylinders will be randomly selected for each production lot. Cylinders shall be taken in accordance with AASHTO T-23 on 6” (150 mm) x 12” (300 mm) specimens. For every compressive strength sample, a minimum of two (2) cylinders will be cured in the same manner as the panels are cast and shall be tested at approximately seven (7) days. The average compressive strength of these two (2) cylinders when tested in accordance with AASHTO T-22 will provide a test result, which will determine the initial strength of concrete. Also, two (2) cylinders will be cured in accordance with AASHTO T-23 and tested at approximately twenty-eight (28) days. The average compressive strength of these two (2) cylinders, when tested in accordance with AASHTO T-22 will provide a compressive strength test result, which will determine the compressive strength of the production lot.

The production lot is acceptable if the compressive strength test result is greater than or equal to a minimum of 4000 psi or the required project compressive strength. If the compressive strength is less than 4000 psi or the required project compressive strength, the acceptance of the production lot will be based on meeting the following additional acceptance criteria in its entirety.

#### 2.2.11.1 Ninety Percent Rule

If 90% of the compressive strength test results for the overall production exceed 4000 psi or the project specified compressive strength, the lot is acceptable.

#### 2.2.11.2 Average Six Rule

If the average of any six (6) consecutive compressive strength test results exceed 4000 psi or the project specified compressive strength, the lot is acceptable.

#### 2.2.11.3 Compressive Strength Rejection

Production lots will be rejected for failure to meet specified compressive strength requirements. To get the production lot accepted, the manufacturer may obtain and submit evidence that the strength and quality of concrete placed within the panels of the production lot are acceptable at their own expense. All core samples shall be obtained and tested in accordance with AASHTO T-24.

#### 2.2.11.4 Rejection

Panels shall be subject to rejection for failure to meet any of the requirements specified above. Also, defects, which indicate imperfect molding, or defects indicating honeycombed or open textured concrete, or the improper placement of panel anchors, shall be sufficient cause for rejection.

### 2.3 Leveling Pad

A cast in place leveling pad shall be required for the first row of panels to bear on. The cast in place leveling pad shall be level with a 1/8” in 10 feet tolerance. The minimum width of the leveling pad shall be 12”. The minimum thickness of the leveling pad shall be 6”. Concrete shall have a minimum of 3000 psi compressive strength. The leveling pad shall cure a minimum of 12 hours before placement of the panel. The leveling pad steps shall be fabricated in conformance with the project specifications. No precast leveling pads or aggregate leveling pads are allowed. Forming boards shall be used to cast the leveling pad.

## 2.4 EarthTrac™ Soil Reinforcing

The EarthTrac™ soil reinforcing SS1 shall be hot-rolled and the SS3 shall be cold-formed, steel ribbed strips in conformance with ASTM A572 or ASTM A1011, respectively. The EarthTrac™ SS1 and SS3 soil reinforcing shall be hot-dip galvanized after fabrication in conformance with ASTM A123.

### 2.4.1 Acceptance

All material will be accepted based on the required certification and testing required by the ASTM specifications.

### 2.4.2 Material

The EarthTrac™ soil reinforcing shall have a minimum tensile strength equal to 65 ksi and a minimum yield strength equal to 50 ksi for ASTM A572 material and a minimum tensile strength equal to 65 ksi and a minimum yield strength equal to 50 ksi for ASTM A1011 material.

### 2.4.3 Fabrication Tolerances

The permissible variation from the dimensions and configuration detailed shall be as follows:

- Overall strip Width ± 1/16"
- Overall strip Length ± 1"
- Overall Hole Dimension ± 1/64
- Overall Edge Distance ± 1/32

### 2.4.4 Certification

The manufacturer of the EarthTrac™ soil reinforcing shall furnish certification of all material. The certificate shall indicate that the components are in accordance with this specification and the EarthTec™ QA/QC requirements. The certificates shall include typical test results representative of the material. The certification shall be notarized.

## 2.5 Panel Anchors

The panel anchors are manufactured from hot-rolled steel strip in conformance with ASTM A1011.

### 2.5.1 Specifications

The EarthTrac™ soil reinforcing shall meet ASTM A1011 - Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, and Ultra-High Strength, Grade 50. The panel anchors shall be fabricated in accordance with the EarthTec™ QA/QC requirements and conformance with the panel anchor detail and conformance with ASTM A1011.

### 2.5.2 Acceptance

All material will be accepted based on the required certification and testing required by the ASTM specifications.

### 2.5.3 Material

The panel anchor shall be Grade 50 steel and have a minimum tensile capacity of 65 ksi and a minimum yield capacity of 50 ksi.

### 2.5.4 Fabrication Tolerances

The permissible variation from the dimensions and configuration shown shall be as follows:

- Overall Plate Vertical Dimensions ± 1/8
- Overall Plate Horizontal Dimensions ± 1/8
- Overall Hole Dimension ± 1/64
- Overall Edge Distance ± 1/32

### 2.5.5 Certification

The manufacturer shall furnish certification of the panel anchor. The certificate shall indicate that the panel anchors are in accordance with this specification and the Ground Improvement Systems EarthTec™ QA/QC requirements. The certifications shall include typical test results representative of the material. The certificate shall be notarized.

### 2.6 Galvanizing

All EarthTrac™ soil reinforcing shall be galvanized after fabrication by hot-dip method in conformance with ASTM A123-Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products with a minimum of 2 ounces per square foot of zinc applied. All panel anchors shall be galvanized after fabrication by the hot-dip method in conformance with ASTM A153-Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware and shall have a minimum zinc coating thickness equal to 3.4 mills. Small incidentals may also be coated with zinc in conformance with ASTM A1059-Standard Specification for Zinc Alloy Thermo-Diffusion Coating (TDC) on Steel Fasteners, Hardware, and Other Products. They shall require an equivalent coating thickness specified in the EarthTec™ QA/QC requirements.

### 2.7 Connection Bolt Sets

Bolt sets shall be in accordance with ASTM F3125 - Standard Specification for High Strength Structural Bolts and Assemblies, Steel and Alloy Steel, Heat Treated, Inch Dimensions 120 ksi and 150 ksi Minimum Tensile Strength, and Metric Dimensions 830 MPa and 1040 MPa Minimum Tensile Strength. Bolts and nuts shall be hexagonal in shape Type-I. The bolt shall be a ½" in diameter a minimum of 1 ½" in length. Galvanizing shall be in accordance ASTM A153.

### 2.8 Bearing Pad

The bearing pads shall be a 100 percent virgin Styrene Butadiene Rubber (SBR, Buna-S) compound meeting the requirements shown below. The pads shall be of the Durometer Grade minimum 60. If test specimens are cut from the finished product, a 10 percent variation in Physical Properties will be allowed.

**Bearing Pad Property Criteria**

| Durometer Grade | 60 | |
|---|---|---|
| **ASTM** | **Property** | **Value** |
| D2240 | Hardness | 60 ± 5 |
| D412 | Tensile Strength (psi) | 2500 |
| D412 | Ultimate elongation (% min) | 350 |
| D573 | Change in Durometer Hardness | +15 |
| D573 | Change in Tensile Strength (% max) | -15 |
| D573 | Change in Ultimate Elongation (% max) | -40 |
| D395 | Compressive Set 22 hars at 212 F (% max) | 35 |
| D1149 | Ozone 100 pphm ozone in air by volume | No Cracks |
| D429 | Adhesion Bond made during vulcanization | 40 |
| D746 | Low-Temperature Test for Brittleness | No Failure |

### 2.9 Filter Fabric

The filter fabric shall consist of a needle punched geotextile resistant to ultraviolet degradation and to biological and chemical attack normally found in soils. The following properties will be tested and adhered to the values provided for in AASHTO M288 as given in the table. Other products not meeting these requirements shall be reviewed on a project by project basis.

**Filter Fabric Specification**

| Mechanical | ASTM | MARV |
|---|---|---|
| Grab Tensile Strength | D4632 | 300 lbs |
| Puncture Strength <50% Elongation | D6241 | 100 lbs |
| Trapezoidal Tear | D4533 | 125 lbs |
| **Hydraulic** | **ASTM** | **Property** |
| Maximum Apparent Opening Size (AOS) | D4751 | 40 US Standard Sieve |
| Water Flow Rate (g/m/sf) | D4491 | 100 |
| Permittivity | D4491 | 0.50 sec-1 |
| **Endurance** | **ASTM** | **Property** |
| UV Resistance (% Retained after 500 hours) | D4355 | 50% |

## 2.10 Adhesive

The adhesive shall consist of any material used to attach a fabric material to a concrete surface. The material shall be resistant to sunlight and durable under moisture and temperature conditions. The adhesive applied to the SCP panel to attach the filter fabric shall be a quantity that allows for adherence until such time that the backfill is place. It is not a requirement to use a continuous bead of adhesive.

## 2.11 Drainage

All drainage requirements are by others and shall be strictly followed as specified in the project specification and as shown on the approved shop drawings or in the contract drawings. The select backfill that is used in the reinforced soil volume shall have less than 15% passing the #200 sieve. A maximum of 15% is required to allow for gravity drainage of the backfill.

The saturation of the reinforced soil volume can increase the pore pressure, resulting in a decrease in the soil's resistance capacity and the reduction of the backfill's strength, reducing the stability of the structure. If saturation conditions are expected, drainage shall be considered. Drainage requirements may include face drains, base drains (blanket drains) and back drains.

# 3 Backfill Requirements

The select granular backfill material used in the RSW shall be reasonably free from organic and otherwise deleterious materials. It shall conform to the following minimum gradation limits as determined by AASHTO T-27. Alternatively, the material shall be classified as GW, SW, GP, or SP in conformance with Unified Soil Classification in ASTM D2487. The backfill shall not be gap graded and shall be angular.

## 3.1 Mechanical requirements

### 3.1.1 Gradation

**Backfill Soil Gradation**

| U.S. Sieve Size | Percent Passing |
|---|---|
| 4” | 100 |
| No. 40 | 0-60 |
| No. 200 | 0-15 |

### 3.1.2 Plasticity Index

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

### 3.1.3 Fine Material

Fraction finer than the #200 sieve shall not exceed 15 percent (15%).

### 3.1.4 Internal Friction Angle

The material shall exhibit an angle of internal friction of not less than 30 degrees, as determined by the standard Direct Shear Test, AASHTO T-236, on the portion finer than the #10 sieve, utilizing a sample of the material compacted to 95 percent of AASHTO T-99, Methods C or D (with oversized correction as outlined in Note 7) at optimum moisture content.

### 3.1.5 Soundness

The material shall be substantially free of shale or other soft, low durability particles. The materials shall have a magnesium sulfate soundness loss of less than 20 percent after five (5) cycles, as determined by AASHTO T-104.

### 3.1.6 Electro-Chemical Requirements

The material shall conform to the following electro-chemical requirements:

**Electro-Chemical Requirements**

| Property | Requirement | Test Method |
|---|---|---|
| Resistivity | >3000 ohm-cm | AASHTO T-288-91 |
| pH | 5-10 | AASHTO T-289-91 |
| Chlorides | <100 ppm | AASHTO T-291-91 |
| Sulfates | <200 ppm | AASHTO T-290-91 |
| Organic Content | <1% | AASHTO T-267-86 |

### 3.1.7 Quality Control

The quality control of the backfill shall be by others. The Contractor or the Owners engineer shall approve the backfill before installation of the RSW begins. If the backfill source changes, the new source shall be checked and approved based on it meeting the project requirements. At a minimum, the gradation, friction angle, electro-chemical properties, and soundness shall be tested. All testing is to be performed by an approved testing agency. The Engineer shall approve all backfill. EarthTec will not certify or approve backfill.

### 3.1.8 Water for Compaction

Water that is to be used for compaction shall have a resistivity exceeding 700 Ω-cm.

### 3.1.9 Materials Not Conforming to The Above

Materials not conforming to the above requirements may not be used without the project engineer's written consent and only after the material has been tested. At a minimum, the non-conforming material shall be tested to assure that the functional properties are consistent with the design calculations' parameters.

## 4 CONSTRUCTION REQUIREMENTS

The construction of the EarthTec™ RSW is a repetitive process that requires successive layers of compacted soil, panels, and soil reinforcing. Each constructed layer typically requires the same installation steps and the same standard material. The elevation drawing (typically shown front face) of each RSW is shown in the Shop Drawings. Below each elevation, there are column numbers. The column numbers can be used to identify the required material and location in the RSW quickly.

The RSW area is routinely measured from the top of the leveling pad to the top of the panel. The project Wall Area is routinely measured from the top of the leveling pad to the top of the coping.

The finish grade at the face of the RSW shall conform to the project specifications and as shown in the approved shop drawings.

The plan foundation area of the RSW shall be excavated and prepared so the base is level and that there is not pumping or rutting.

The leveling pad shall be placed on a prepared base. No precast or aggregate leveling pads are allowed.

### 4.1 EarthTrac™ Soil Reinforcement Placement

The EarthTrac™ soil reinforcing is placed on the compacted select backfill and the connection is made to the panel anchor. The EarthTrac™ soil reinforcing is connected to the panel by securing the proximal end between the two plates of the panel anchor. The bolt set is a snug-tight connection. The nut is placed finger tight then a wrench is used to tighten it so it cannot be removed by hand. One thread minimum shall protrude from the top of the nut. The bolt shall always be placed so the nut is on top of the anchor. The EarthTrac™ soil reinforcing shall be placed at right angles to the face of the wall.

Preventive measures shall be exercised to prevent ponding and stormwater from collecting at the face of the structure.

At the top of wall, the backfill that the EarthTrac™ soil reinforcing is shall be placed on a grade that slopes 2.5% to 5% away from the SCP towards the terminal end of the soil reinforcing to prevent potential conflict with the base or mixing course of the roadway base material.

Never connect the EarthTrac™ soil reinforcing to a panel anchor with the backfill lower than the panel anchor's elevation. The backfill should ideally be slightly higher than the panel anchor. To place the bolt-set, it may be necessary to remove some backfill from under the panel anchor to place the bolt and to attach the nut. The nut shall always be on top of the panel anchor, never on the panel anchor's bottom surface.

The Wall Installer should verify that all nuts have been installed with each of the bolts to snug-tight. Any SCP that has anchors missing or severely bent or damaged shall not be used but shall be set aside and either repaired or rejected.

### 4.2 Facing Batter

The concrete panels are typically initially set with a slight inward batter toward the backfill. The amount of facing batter is a function of the gradation of the select backfill, the length of soil reinforcing, compaction equipment, and installer experience, among other things. The inward batter is removed from the SCP during the compaction of the backfill. The required amount of batter is to be established by the Wall installer in their means and methods. The facing shall be placed in a final alignment that meets the project specifications.

### 4.3 Drainage

Drainage shall be strictly followed in conformance with the project specifications and as shown on the approved shop drawings or in the contract documents. The select backfill that is used in the reinforced soil volume shall have less than 15% passing the #200 sieve to be considered a gravity drainage material. The saturation of the reinforced soil volume can increase the pore pressure, resulting in a decrease in the soil's resistance capacity and a reduction in the strength of the soil that may affect the stability of the structure.

### 4.4 Foundation Preparation

The foundation area shall be graded level for at least the width equal to the soil reinforcement length plus a minimum of six (6) inches. All foundation material that is suspected of being of poor quality shall be removed and replaced. The foundation preparation is a critical part of the wall construction. Taking time to prepare the foundation properly will significantly improve the wall's overall constructability and performance while decreasing the chances of problems occurring during or after construction. Soil reinforcement shall not be placed until the foundation has been prepared to support all anticipated loading.

### 4.5 Erection

Precast SCP shall be placed with the aid of a light crane, track hoe, loader, or some other lifting means. The panels are always to be handled utilizing the lifting devices cast into the panels' upper edge. Panels shall be placed in successive horizontal lifts in the sequence shown on the approved shop drawing. At all times, during backfill placement, the panels shall be maintained in a slightly battered to vertical position using panel clamps and temporary wooden wedges placed in the joint at the interface of the two adjacent panels. Plastic shims may also be used between the precast panels and panel clamps to facilitate proper panel batter. Continue to check vertical alignment as construction continues vertically (up-the-wall). It may be necessary to adjust the panel batter to ensure vertical plumbness.

Important Safety Note: *Never remove panel rigging until the precast panels have been adequately secured with wooden panel clamps and bracing (on the bottom row). The first row of panels is placed on the leveling pad. If the leveling pad is not set correctly, the panel will need to be adjusted to ensure proper horizontal and vertical alignment is achieved and maintained during the erection process. Alignment can be achieved with hard-wood shims and wedges. It is not necessary to place a bearing pad on the leveling pad. The precast panels are set directly on the leveling course.*

Panel clamps and external bracing that extends from the panel's front face away from the wall face is required for the first row of panels. No exceptions are allowed.

It is important to visually check the panel joints' horizontal lines (via looking down the wall) to ensure the horizontal joint's proper alignment. This is especially important for partial height panels along the bottom row.

### 4.6 Backfill Placement and Compaction

The backfill placement should begin parallel to the wall face at a distance greater than or equal to three (3) feet from the panel's back face. The backfill should be placed in 6”-10” compacted lifts. The backfill can be placed in larger lifts if approved by the Owner or Owner’s representative and if the Wall Installer can demonstrate that the proper compaction can be achieved. The backfill shall be leveled by equipment moving parallel and away from the wall face. The material shall be spread so it is fanned toward the terminal end of the soil reinforcing. The backfill placement from the front of the soil reinforcement to the terminal end of the soil reinforcement will keep the soil reinforcement fixed and tensioned.

Compaction of the backfill a distance of three (3) feet from the panel's back face shall be performed with a 8-ton to 10-ton smooth drum roller. A smooth wheel or rubber tire roller is also acceptable. No compactors that employ grid type rollers shall be used. Grid type rollers can dislodge the soil reinforcing from its proper orientation. Compaction must be parallel to the wall face working toward the terminal end of the soil reinforcement. The proper moisture content of the backfill material shall be maintained uniformly within each lift. The material shall be placed on the dry side of the optimum moisture content. Care should be used in adding water to the backfill material.

The 3-foot zone of fill located at the back of the panel is placed with an end loader and spread manually. The material is then compacted using a small hand-operated maximum 2000 pound vibratory roller or a maximum 1000 pound plate compactor. Care should be exercised when compacting this area so as not to disturb the alignment of the panel. Fine-grain soils should be compacted with care. Compaction should take place from the back face of the panel to the terminal end of the soil reinforcing.

Backfill placement shall follow the erection of each lift of panels closely. At each reinforcing element elevation, backfill should be roughly leveled before placing and connecting the soil reinforcing elements. Soil reinforcing elements shall be placed normal to the wall's face or as shown on the drawings.

At the end of each day's operation, the Contractor shall shape the last lift of backfill to permit positive runoff of stormwater away from the wall face. Care should be exercised in the placement of the top layer of soil reinforcement to prevent damage during the roadway base course placement or if the roadway is in a super-elevation. The top layer of soil reinforcing may be sloped downward as it moves away from the wall to prevent damage during the roadway base course placement.

Backfill shall be compacted following the project specifications to a 95% standard or modified proctor density over the embankment's entire width. Density shall be determined by the standard test methods in conformance with the project specification. The backfill's moisture content shall be plus or minus 2% of the optimum Moisture Content as determined by the standard or modified proctor test. Backfill compaction and testing shall be accomplished without disturbance of the soil reinforcing and the panel alignment.

## 5 Technical Assistance

A technical representative from the approved EarthTec supplier shall be present at the beginning of construction. If required, the technical representative shall be present periodically on-site during the casting and installation phases to ensure that the quality of the Contractor's works is following this specification. EarthTec will not certify construction. The means and methods of all construction are the responsibility of the installer and the general Contractor.

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# 5.0
# PERFORMANCE

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**PERFORMANCE HISTORY**

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**5.1.1**
**ERS Performance History**

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**5.1.2**
**ERS Oldest Structures**

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**5.1.3**
**ERS Tallest Structures**

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## 5.1.4
## ERS Private and Public Users

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ARLINGTON, TX 76011

Project Log

| EarthTec Reinforced Soil Wall Project List | | | | | | |
|---|---|---|---|---|---|---|
| Products | Project | Location | State | Area [SF] | Contract Date | Contract Value |
| MSE | 9th Street Bridge | Washington | DC | 8,472 | 7/21/2008 | $153,045 |
| MSE | 11th Street Bridges | Washington | DC | 153,012 | 3/31/2009 | $2,381,040 |
| MSE | Anacostia | Washington | DC | 10,038 | 1/10/2011 | $128,988 |
| MSE | St. Annes Church Road | Middlestown | DE | 9,500 | 3/11/2009 | $159,357 |
| MSE | Vogtle Elec. Generating Plant Units 3&4 | Waynesboro | GA | 20000 | 1/8/2009 | $862,656 |
| MSE | MTA MD I-95 & I-895 | Baltimore City | MD | 36,319 | 8/12/2008 | $537,521 |
| MSE | MD193 Greenbelt Metro | Greenbelt | MD | 1,320 | 9/15/2008 | $41,500 |
| MSE | I 695 & Charles Street | Baltimore | MD | 26699 | 1/13/2009 | $670,071 |
| MSE | Cherry Hill Road Extension Bridge | Baltimore Co | MD | 5201 | 10/20/2009 | $77,235 |
| MSE | I-695 Bridge over Liberty Rd MD 26 | Baltimore Co. | MD | 5000 | 12/10/2009 | $61,133 |
| MSE | Greenbelt Metro park MD193 | Greenbelt | MD | 1,546 | 6/27/2011 | $45,298 |
| MSE | MD 193 | Greenbelt | MD | 1587 | 7/3/2007 | $41,350 |
| MSE | ICC Contract C | Montgomery | MD | 89,845 | 9/11/2007 | $1,544,979 |
| MSE | Fox Run Blvd. | Calvert Cty. | MD | 2,192 | 2/5/2008 | $39,127 |
| MSE | East Access Road | Silver Spring | MD | 23,285 | 4/22/2008 | $381,095 |
| MSE | ICC Contract B | MD97 & US29 | MD | 65,000 | 5/2/2008 | $1,094,550 |
| MSE | I 95 & 24 | Baltimore | MD | 40,184 | 2/27/2009 | $764,244 |
| MSE | Rt 1 Section 6v | Brunswick | NJ | 12,019 | 4/1/2009 | $389,290 |
| MSE | Rt 80/287 Safety Improvements | Parsippany | NJ | 6,581 | 4/10/2009 | $145,342 |
| MSE | Garden State Parkway 63 to 80 | Ocean City | NJ | 20,505 | 4/28/2009 | $435,981 |
| MSE | NJTPK- Interchange 6-9 Section 5 T869.120.502 | Mercer Co | NJ | 27136 | 2/23/2010 | $338,198 |
| MSE | NJTA-Interchange 6-9 Sec 8-8A Ramp T802 | Middlesex Co. | NJ | 31682 | 5/6/2010 | $795,754 |
| MSE | Route 206 Bypass | Hillsborough | NJ | 54033 | 5/13/2010 | $1,096,278 |
| MSE | Route I 80 Truck Weigh Station | Warren County | NJ | 14,693 | 10/14/2010 | $399,154 |
| MSE | North Shore Connector | Pittsburgh | PA | 2,980 | 1/26/2009 | $58,497 |
| MSE | Telegraph Road Interchange | Alexandria | VA | 110,322 | 4/23/2008 | $1,341,074 |
| MSE | HOT Lanes Section 7 | Nova | VA | 3500 | 8/28/2008 | $28,300 |
| MSE | Fairfax County Pkwy @ Rolling Rd | Fairfax County | VA | 28,965 | 8/29/2008 | $499,213 |
| MSE | NGA Ft Belvoir | Ft Belvoir | VA | 52,872 | 9/29/2008 | $684,692 |
| MSE | Hot Lanes Section 8 | Fairfax Co. | VA | 25,532 | 3/19/2009 | $339,065 |
| MSE | Bridge 610 I 495 | Fairfax County | VA | 3,200 | 4/15/2009 | $28,320 |
| MSE | North Gayton Road | Henrico County | VA | 25,970 | 5/22/2009 | $478,669 |
| MSE | Potomac Yard | Alexandria | VA | 12,690 | 8/26/2009 | $217,344 |
| MSE | Fairfax County Pkwy Phase III | Fairfax County | VA | 6,610 | 9/16/2009 | $97,828 |
| MSE | 9B1 RTE 28 (Nokesville Road)/RTE 674 | Manassas | VA | 47,545 | 6/23/2010 | $610,188 |
| MSE | D28 Route I-95 over Lombardy St | Richmond | VA | 9,460 | 6/23/2010 | $273,924 |
| MSE | VDOT 2A8 - Rte 221 | Roanoke | VA | 10,286 | 6/23/2010 | $228,016 |
| MSE | Jordan Bridge | Chesapeake | VA | 16,682 | 12/15/2010 | $281,921 |
| MSE | Jefferson Park Bridge | charlottesville | VA | 1,793 | 2/7/2011 | $32,897 |
| MSE | Corridor H.W. WV 93 to E. Co 1 Bridge #1 | Grant Co | WV | 5700 | 9/15/2009 | $130,245 |
| | Total | | | **1,029,956** | | ########### |

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# 6.0
# OTHER

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**6.1**

**OTHER INFORMATION**

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