INTRODUCTION
In 1994 the Highway Innovative Technology Evaluation Center (HITEC) was established as a collaborative effort by the Federal Highway Administration (FHWA), American Association of State Highway and Transportation Officials (AASHTO) and the Transportation Research Board to accelerate the process by which technological advances are adopted for use in highway infrastructure (HITEC 1998). Until that time, many state transportation agencies had no formal process to evaluate earth retention systems and often lacked the technical resources that such evaluations require. The HITEC earth retaining system program was created to evaluate the performance of proprietary earth retention technologies. Over a period of about 20 years many mechanically stabilized earth (MSE) retention systems were evaluated using design guidelines and specifications published by the FHWA and AASHTO. Since that time, MSE technology has become a widely accepted component of transportation infrastructure.

In 2016 the FHWA published a protocol to further advance innovations in earth retention technology and encourage their use by public transportation agencies (Johnson and Valentine 2016). Under this protocol, earth retention systems are evaluated by a review team. The members of the team are selected based on their experience. One member is designated as the lead consultant and is responsible for the detailed review of the proposed system. The other members lend their experience to the process and assess the recommendations of the lead consultant. Overall administration of the review program is performed by the Geo-Institute of the American Society of Civil Engineers.

Technical evaluations for the IDEA program are based upon information provided by the system applicant, as well as existing guidelines and specifications such as those published by the FHWA (Berg et al. 2009) and AASHTO (AASHTO 2015). These references serve as a baseline to assess a system’s conformance with current engineering practices. However, a fundamental objective of the IDEA program is to encourage advancements in earth retention technology. Such advancements may not be contemplated by current design references. Thus, the IDEA program relies upon earth retention experts to evaluate potential innovations.

IDEA REVIEW PROCESS
The IDEA program provides for two types of reviews: an initial technical evaluation and an update technical evaluation. An initial technical evaluation is performed to assess an earth retention system which has not been previously reviewed using the IDEA protocol. An update evaluation is performed every five years after completion of the initial evaluation or in response to a notification of a change in an element of the system.

An initial technical evaluation of an earth retention system is performed using the IDEA protocol. In accordance with the protocol, an earth retention system is defined as a unit that comprises the following elements:
• Specific components and the materials used for their manufacture.
• Design methodologies.
• Construction procedures.
• Quality control measures.

The details of each of the above elements is considered in accordance with a checklist that is selected based on the characteristics of the system. The applicant prepares a submittal that is organized in accordance with the checklist in a digital portable document format (PDF). The extensive use of links to connect checklist topics with the table of contents and supporting sections in appendices are encouraged to facilitate an efficient review and use by transportation agencies.

An initial technical evaluation is performed in four phases.

• Pre-submittal Review Phase – It is initiated when an applicant provides a request for an initial technical evaluation. A checklist is selected or designed based on the characteristics of the proposed earth retention system. An agreement for the review is executed.

• Submittal Check Phase – It is initiated by the applicant. The submittal is checked for completeness and conformance to the evaluation checklist. A report of review is provided to the applicant that describes the review team’s findings and recommendations.

• Initial Submittal Review Phase – The submittal is rigorously evaluated for its technical content with emphasis given to any innovations proposed by the applicant. A report of review is provided to the applicant that describes the review team’s findings and recommendations.

• Final Submittal Review Phase – The applicant’s response to the previous review comments are considered and final review comments are discussed with the applicant. A final report of review is completed and attached to the applicant’s final submittal. The report and submittal are provided for use by transportation agencies.

VISTAWALL STABILIZED EARTH WALL SYSTEM
The Vistawall Stabilized Earth Wall (Vistawall) has been evaluated in accordance with the IDEA protocol. Key information regarding this system is presented in this section of this final report of review. Important details of the system’s components, design, construction and quality control measures are presented the attached submittal.

Applicant Information
Big R Bridge
PO Box 1290
Greeley, CO 80632-1290
Ph.: (970) 356-9600
www.bigrbridge.com

Date of Submittal
November 8, 2018
Submittal Checklist
The checklist used from the IDEA protocol for this evaluation is C4 – Initial Evaluation Checklist for Precast Concrete Panel Paired with Inextensible Reinforcement.

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 Vistawall no information has been designated by the applicant as confidential.

System Description
Vistawall comprises a precast concrete facing panel and inextensible Grid-Strip™ soil reinforcement. Other components include coping, barriers, bearing pads, alignment pins and geotextile filters. The standard facing panel height is 5 ft and is cast in standard widths of 5 ft and 10 ft. It is attached to elements of Grid-Strip™ reinforcement by a steel anchor embedded in the panel using one of two different types of connectors. In its standard configuration Grid-Strip™ comprises two longitudinal W11.0 wires that attach to transverse W11.0 wires that are typically spaced at intervals of 1 ft. Grid-Strip™ reinforcement and the facing panel connection components are zinc-coated (i.e. galvanized) to enhance protection against corrosion.

System History
Vistawall was first used with Grid-Strip™ for a state highway project in Texas in 2011. Since that time 4 million square feet of wall face has been installed. It has been approved for use by 13 U.S. state departments of transportation as well as by seven Canadian provincial departments of transportation. The tallest Vistawall structure was constructed in Florida and is 45 ft at its maximum height.

System Properties
The following properties are reported by the applicant for Vistawall.

Soil Reinforcement Tensile Strength
The ultimate tensile strength for Grid-Strip™ reinforcement is reduced to account for potential corrosion in accordance with AASHTO/FHW procedures. For reinforcement fabricated using W11.0 the design tensile strength is 7.8 kip for a 75-year service period and 6.7 kip for a 100-year service period. These design strengths are based on AASHTO/FHWA corrosion rates for soils with a resistivity greater than 3,000 ohm-cm, sulfates less than 200 ppm and chlorides less than 100 ppm.

Soil Reinforcement-Facing Panel Connection Capacity
The connection capacity of the Grid-Strip™ reinforcement has been evaluated by connection strength testing. The results of this testing indicates that the connection strength is limited by the Grid-Strip™ reinforcement design strength.

Soil Reinforcement Tensile Resistance Factors
A tensile resistance factor of 0.75 is recommended by the applicant for the Grid-Strip™ reinforcement. For metallic reinforcement designated as “strip” and “grid”, AASHTO specifies tensile resistance factors of 0.75 and 0.65, respectively. The applicant’s recommendation that 0.75 be used as the resistance factor is based on the history of the development of the AASHTO specification as well as the structural
and geometric properties of the Grid-Strip™ reinforcement. This review finds that the recommendation is well-reasoned (see section 2.1.5 of the attached submittal for further discussion of this issue).

Retaining Wall Design
Vistawall structures are designed by Big R Bridge using the proprietary computer program MSE Pro. The scope of this review did not provide for a comprehensive evaluation of this computer program. However, the applicant states that the design of Vistawall structures is consistent with current AASHTO specifications. This assertion is supported by the results of the analyses of two example problems in the Vistawall submittal. The submittal also includes the results of the two example problems as determined using the computer program MSEW (see Appendix 2.3 of the submittal).

The reinforcement strength properties used in MSEW are shown in Table 1.

Table 1. Reinforcement strength properties for use in MSEW.

<table>
<thead>
<tr>
<th>Reinforcement Type</th>
<th>Yield Strength, $F_y$</th>
<th>Gross Width of $b$</th>
<th>Cross-section Area Corrected for Corrosion loss, $A_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>GS 11 (W11.0 wire)</td>
<td>65 kips/in$^2$</td>
<td>2 in</td>
<td>0.16 in$^2$</td>
</tr>
</tbody>
</table>

System Innovations
Vistawall was submitted for review with no claim of an innovation. However, this review has noted two. First, the pullout testing of Grid-Strip™ reinforcement indicates a distinct improvement of the friction factor, $F^*$, compared to the default values provided for steel ribbed reinforcement in FHWA Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Volume I (Berg et al. 2009). There, the default value of $F^*$ is:

\[
F^* = \tan \rho = 1.2 \log C_u \text{ at the top of the structure} = 2.0 \text{ maximum} \tag{1}
\]

\[
F^* = \tan \phi \text{ at a depth of 20 ft and below} \tag{2}
\]

In Equation 1, $C_u$ is the coefficient of uniformity of the backfill soil. For fill soil that satisfies AASHTO/FHWA requirements for MSE walls, it may be assumed that $F^*$ is equal to 1.8 at the top of the retaining wall. In contrast, Grid-Strip™ reinforcement may be designed using $F^*$ equal to a maximum value of 3.0, as shown below in Figure 1.
The second innovation is that the Grid-Strip™ reinforcement comprises two longitudinal wires that intersect at a single-point connection with a rigid facing panel. As a result of this configuration, the applicant recommends the use of resistance factor of 0.75 that is normally associated with steel strip reinforcements, rather than the resistance factor of 0.65 that is normally associated with grid reinforcements that are attached to a rigid facing panel.

Conclusions
Following its initial review of the Vistawall submittal, the review team provided the applicant with more than 50 comments and requests for clarification. The applicant has been thorough in its responses and the review team finds that there are no outstanding issues that should be brought to the attention of the transportation agencies. Rather, the agencies are encouraged to rely upon the Vistawall submittal for projects where the Vistawall system is proposed.

The final version of the Vistawall submittal is dated November 8, 2018 and it is attached to this report of review. An update technical evaluation should be performed for the Vistawall system in 5 years (i.e. November, 2023) 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, see *Highway Innovations, Developments, Enhancements and Advancements (IDEA) – Protocol for Technical Evaluation of Earth Retention Systems* (Johnson and Valentine 2016).
References


IDEA
HIGHWAY INNOVATIONS, DEVELOPMENTS, ENHANCEMENTS AND ADVANCEMENTS

MECHANICALLY STABILIZED EARTH
GRID-STRIP™ SOIL REINFORCING

AUGUST 2018
The drawings, plans, models, designs, specifications, reports, photographs, computer software, surveys, calculations, and other data, including computer print-outs, contained in this report are the property of Vistawall. The documents included in this report are made available for your review for informational purposes only in relation to this report. The documents may not be copied, reproduced, or distributed in any way or for any purpose whatsoever. The report has been produced using the known State-Of-Practice at the time it was completed.
Table of Contents

Forward ............................................................................................................................... 1
1.0 ERS COMPONENTS .................................................................................................... 1

1.1 Facing Units .............................................................................................................. 1
  1.1.1 Facing Unit Innovations ......................................................................................... 2
  1.1.2 Facing Unit Types .................................................................................................. 2
  1.1.3 Facing Unit Fabrication ........................................................................................ 2
  1.1.4 Facing Unit Specification ..................................................................................... 2
  1.1.5 Facing Unit Dimensions ...................................................................................... 2
  1.1.6 Facing Unit Compressive Strength ...................................................................... 2
  1.1.7 Facing Unit Percent Air Entrainment .................................................................... 3
  1.1.8 Facing Unit Mix Designs ...................................................................................... 3
  1.1.9 Facing Unit Interface Shear Capacity .................................................................... 3
  1.1.10 Facing Unit Alignment Pin and Bearing Pad ..................................................... 4
  1.1.11 Facing Unit Joint Filter ....................................................................................... 5
  1.1.12 Facing Unit Aesthetic Options .......................................................................... 6
  1.1.13 Facing Unit Alignment Requirements for Curves and Corners ....................... 6

1.2 Inextensible Reinforcements .................................................................................... 7
  1.2.1 Soil Reinforcing Innovations ................................................................................ 7
  1.2.2 Soil Reinforcing Manufacturers .......................................................................... 8
  1.2.3 Soil Reinforcing Types ......................................................................................... 8
  1.2.4 Soil Reinforcing Properties ................................................................................ 8
  1.2.5 Soil Reinforcing Corrosion Protection .................................................................. 9
  1.2.6 Soil Reinforcing Sacrificial Steel as a Function of Service Life ....................... 9
  1.2.7 Soil Reinforcing Corrosion Testing .................................................................... 10
  1.2.8 Soil Reinforcing Dimensional Tolerances ......................................................... 10
  1.2.9 Soil Reinforcing Connection .............................................................................. 10
  1.2.10 Soil Reinforcing Connection Components ..................................................... 11
  1.2.11 Soil Reinforcing Connection Manufacturers ................................................... 13
  1.2.12 Soil Reinforcing Connection Properties ......................................................... 13
  1.2.13 Soil Reinforcing Connection Corrosion Protection ......................................... 14
  1.2.14 Soil Reinforcing Connection Sacrificial Steel as a Function of Service Life .......... 14
  1.2.15 Soil Reinforcing Connection Corrosion Testing .............................................. 15
  1.2.16 Soil Reinforcing Connection Dimensional Tolerances ..................................... 15
  1.2.17 Soil Reinforcing Connection Strength and Testing ......................................... 15
  1.2.18 Soil Reinforcing Pullout Testing and Results ................................................... 16

1.3 Other Components .................................................................................................... 17
1.3.1 Other component Innovations ................................................................. 17
1.3.2 Reinforced Soil Properties .................................................................. 17
1.3.3 ERS Drainage .................................................................................... 19
1.3.4 ERS Coping ...................................................................................... 19
1.3.5 ERS Traffic Barrier .......................................................................... 21
1.3.6 ERS Abutments ................................................................................ 21
1.3.7 ERS Slip-Joints ................................................................................ 22

2.0 ERS DESIGN .......................................................................................... 23

2.1 Design Methodology ............................................................................... 23
  2.1.1 ERS Design Innovations ................................................................. 23
  2.1.2 ERS AASHTO LRFD Design Methodology ................................... 23
  2.1.3 ERS Proprietary Design Methodologies ......................................... 23
  2.1.4 ERS Facing Design Requirements ............................................... 23
  2.1.5 ERS Splayed Soil Reinforcing Design Requirements .................. 24
  2.1.6 ERS Vertical Obstruction Design Requirements ......................... 27
  2.1.7 ERS Horizontal Obstruction Design Requirements .................... 27

2.2 Design Drawings ................................................................................... 29
2.3 Design Examples .................................................................................. 29
  2.3.1 Level Backslope with Traffic Live Load Design Example ........... 29
  2.3.2 2:1 Backslope Design Example .................................................... 31

3.0 CONSTRUCTION .................................................................................... 32

3.1 Construction Procedures ....................................................................... 32
  3.1.1 ERS Construction Innovations ...................................................... 32
  3.1.2 ERS Foundation and Leveling Course Preparation .................... 32
  3.1.3 ERS Construction Tool Requirements ......................................... 34
  3.1.4 ERS Facing Installation Requirements ......................................... 35
  3.1.5 ERS Soil Reinforcing Installation Requirements ......................... 36
  3.1.6 ERS Facing Alignment Requirements ......................................... 37
  3.1.7 ERS Reinforced Backfill Placement Requirements .................... 37
  3.1.8 ERS Erosion Prevention Requirements ....................................... 39
  3.1.9 ERS Installer Requirements ......................................................... 40
  3.1.10 ERS Retained Backfill Placement Requirements ...................... 42
  3.1.11 ERS Construction Manual ......................................................... 42

4.0 QUALITY CONTROL ............................................................................. 42

4.1 Manufacturing ...................................................................................... 42
  4.1.1 Facing Unit QA/QC ................................................................. 42
  4.1.2 Soil Reinforcing QA/QC ........................................................... 42
4.1.3 Miscellaneous Component QA/QC .......................................................... 42

4.2 Construction .................................................................................................. 42

4.2.1 Construction QA/QC ........................................................................... 42

5.0 PERFORMANCE .................................................................................................. 42

5.1 Warranties and Disclaimers .......................................................................... 42

5.2 Testing ............................................................................................................ 43

5.2.1 Pullout Testing ..................................................................................... 43

5.2.2 Connection Testing ............................................................................ 43

5.3 Performance History .................................................................................... 44

5.3.1 ERS Performance History .................................................................... 44

5.3.2 ERS Oldest Structures .......................................................................... 44

5.3.3 ERS Tallest Structures .......................................................................... 44

5.3.4 ERS Horizontal Displacement ............................................................... 45

5.3.5 ERS Differential Vertical Displacement ............................................... 45

5.3.6 ERS Surcharge Loading ........................................................................ 45

5.3.7 ERS Private and Public Users ............................................................... 45

6.0 OTHER ............................................................................................................ 46

6.1 Other Information .......................................................................................... 46
FORWARD

The submittal that follows has been organized to match the IDEA Checklist C4 – “Precast Panel Paired with Inextensible Reinforcement”. The submittal is organized into two sections:

- IDEA ERS Submittal Narrative
- IDEA ERS Submittal Appendix

The submittal narrative discusses each of the required items contained in the Checklist C4. If an item from Checklist C4 is not applicable to the submittal it is noted as “not applicable” and not information is given. If additional supporting documents are required, they are included in the Appendix using the same checklist numbering protocol.

The Appendix contains all of the sections in the Checklist C4. Each section, also referenced as a TAB, contains a cover page, followed by a blank page (this was done to facilitate the requirements for printing as a double-sided document), and then information for that section follows, if it is required. If the checklist did not explicitly require any information, or if information was not noted in the narrative as being included in the Appendix, the section will not contain any additional information.

1.0 ERS COMPONENTS

The components for the Big-R Bridge, Vistawall Stabilized Earth Wall (SEW) consist of a facing unit and inextensible soil reinforcing. The facing unit consists of a segmental concrete panel (SCP). The inextensible soil reinforcing consists of a discrete steel Grid-Strip™. There are incidental components that are used with the SEW system and include, but are not limited to, compacted select backfill, coping and barriers, bearing pads, alignment pins, joint filter fabric and drainage systems. Each of these will be presented in this section.

1.1 FACING UNITS

The Vistawall system uses segmental concrete panel facing units. The facing units can be manufactured to any practical dimension. The standard panel height is 5’-0”. There are two standard panel lengths and include 5’-0” and 10’-0”. The thickness of the panel varies based on the design requirements. For standard concrete mix designs the minimum panel thickness is 5 ½”. The panel is reinforced with steel welded wire mesh or steel reinforcing bars. The standard panel has two rows of panel anchors. The number of rows of panel anchors varies and is based on the height of the panel but
typically range from one row to three rows. The panels are cast face down in steel forms. Form liners can be used to provide specialized finish.

1.1.1 Facing Unit Innovations
No innovation in the facing element is claimed.

1.1.2 Facing Unit Types
The Vistawall facing units include the following standard shapes:

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

1.1.3 Facing Unit Fabrication
The segmental concrete panels (SCP) are fabricated near the project site. Precast concrete companies who meet the requirements of Vistawall QA/QC and the project specifications are used to cast the panels. A detailed Quality Control and Quality Assurance Manual is provided in the Appendix in Section 4.1.1. This manual describes the fabrication process including the casting tolerances for the SCP.

1.1.4 Facing Unit Specification
The generic facing unit specification is described in the Appendix TAB 4.1.1. The mix design and the minimum compressive strength varies from project to project.

1.1.5 Facing Unit Dimensions
Facing unit dimensions are shown in TAB Section 1.1.5.

1.1.6 Facing Unit Compressive Strength
The target minimum compressive strength using standard concrete mix design is 4000 psi. Other compressive strengths can be used. Calculations supporting other compressive strengths are provided as considered to be necessary. The concrete producer is required to select a mix design that will attain 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. Vistawall requires the concrete to have a compressive strength that is strong enough to prevent cracking during stripping, handling, and transport. From experience, it has been demonstrated that a target compressive strength of 1500 psi should be reached before stripping the panel from the form.
1.1.7 Facing Unit Percent Air Entrainment

Air entrainment is required where the facing unit is exposed to freezing and thawing environments. Vistawall provides guidance to 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. Vistawall requires the admixtures that are used to produce a stable system of entrained air. 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 the environmental exposure. The PCA recommends the air content for different exposures be as shown in Table 1.

<table>
<thead>
<tr>
<th>Nominal Maximum Aggregate Size (in)</th>
<th>Air Content (Percent)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Severe Exposure¹</td>
<td>Moderate Exposure²</td>
</tr>
<tr>
<td>3/8</td>
<td>7.5</td>
<td>6</td>
</tr>
<tr>
<td>1/2</td>
<td>7.0</td>
<td>5.5</td>
</tr>
<tr>
<td>3/4</td>
<td>6.0</td>
<td>5.0</td>
</tr>
<tr>
<td>1</td>
<td>6.0</td>
<td>4.5</td>
</tr>
</tbody>
</table>

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.

1.1.8 Facing Unit Mix Designs

The Owner typically specifies the mix design. Vistawall relies on the Precaster to develop the mix design to meet the project specified characteristics for the final concrete mixture. This typically requires that the Precaster develop the fresh concrete properties, required mechanical properties of hardened concrete (strength and durability) and the inclusion, exclusion, or limits, on specific ingredients. The concrete mix design shall have an acceptable workability, durability, strength, and uniform appearance of the hardened concrete.

1.1.9 Facing Unit Interface Shear Capacity

The interface shear capacity for segmental concrete panels that utilize a mechanical connector to attach the soil reinforcing is not applicable. This requirement typically relates to systems that employ...
a small-block facing unit and is not a requirement for systems that employ the large-block segmental concrete facing unit.

1.1.10 Facing Unit Alignment Pin and Bearing Pad

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 front face of the unit and the shoulder is toward the back face of the unit. The cheek hides the shoulder. The shoulder is where the alignment pin and bearing pad are placed (Figure 1-1).

![Figure 1-1 Side-View of Shiplap Panel Joint](image)

Two alignment pins are used for each facing unit and are located at the quarter-points of the panel. The alignment pin is a construction aid as such it is classified as a non-structural element. It is used to help the installer maintain consistent joint spacings. It can be omitted if the installer chooses to do so. This does not relieve the installer of placing the facing unit in the proper location. The alignment pin is fabricated from steel or plastic. The alignment pin has a diameter of 5/8” ±1/16” and has a minimum height equal to 10.5” and a maximum height equal to 11.5”. The alignment pin as placed at the quarter points of the panel and are placed in the precast hole on the top edge of the panel.

Bearing pads are placed on the shoulder behind the cheek of the shiplap joint. Two bearing pads are required for every 5 feet of panel. In other words, for a 5’-0” panel two bearing pads are required and for a 10’-0” panel four bearing pads are required. The bearing pads are 3” x 6” x ¾” SBR rubber with a 60 Duro shore-A. Placement of bearing pads shall as detailed in Figure 1-3 and Figure 1-4. The bearing pad conforms to ASTM D2240 and ASTM D2000. For additional specifications reference appendix 4.1.3.
1.1.11 Facing Unit Joint Filter

A nonwoven, needle punched geotextile filter fabric is used at the interface of all facing unit joints and is placed on the back face of the panel using an adhesive compound. The filter fabric shall conform to AASHTO M288 Table-1, Class 2. The adhesive is not required to be placed in a continuous bead. Enough adhesive that holds the fabric in place until the backfill is placed is all that is required. This is usually accomplished by using dabs of adhesive in a “stich” arrangement (reference Installation Manual in TAB 3.1.11).
The filter fabric is permeable and provides good flow to remove water out from the MSE backfill passing it through the facing joints. The ability of the geotextile to allow water to flow through it is referred to as hydraulic conductivity by permittivity. The geotextile filter fabric is designed and specified to allow water flow in the normal direction while inhibiting the movement of soil particles from the MSE backfill. The apparent opening size, or the AOS, is a common physical property that is tested on geosynthetic fabrics used for filtering. Project specifications for filter fabric can vary between projects and jurisdictions. The minimum values required by Vistawall are shown in Table 2. Reference Appendix TAB 1.1.11.

<table>
<thead>
<tr>
<th>Minimum Permittivity (Sec^{-1})</th>
<th>Maximum AOS (Sieve #)</th>
<th>Minimum Tensile Strength (lbf)</th>
<th>Minimum Puncture (lbf)</th>
<th>Minimum Trapezoidal Tear (lbf)</th>
<th>UV Resistance (Min. Allowed)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM D 4491</td>
<td>ASTM D 4751</td>
<td>ASTM D 4632</td>
<td>ASTM D 6241</td>
<td>ASTM D 4533</td>
<td>ASTM D 4355</td>
</tr>
<tr>
<td>1.50</td>
<td>70</td>
<td>180</td>
<td>310</td>
<td>75</td>
<td>70</td>
</tr>
</tbody>
</table>

### 1.1.12 Facing Unit Aesthetic Options

The Vistawall facing unit is cast face down in steel forms. The forms are designed to allow for the use of form liners to provide for an aesthetic facing. Typical facing options are smooth grey, fractured fin, ashlar stone, random rake, among others. The form liner does not decrease the structural thickness of the segmental concrete panel. Reference Appendix Tab 1.1.12.

### 1.1.13 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 also dictate 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 special edge treatments, e.g., beveled.

Corner panels are used for both inside and outside corners. The Vistawall corner panel is adjustable. If the corner angle is less than 70-degrees, the MSE structure may be required to be designed as a bin wall as specified in FHWA-NHI-10-024 for the extent of the wall where the full length of the reinforcement cannot be installed without encountering the opposite wall face. In addition, when small corner angels are used the panels from adjacent sides may be required to be attached to one another using a special steel panel-to-panel connector.
1.2 INEXTENSIBLE REINFORCEMENTS

The soil reinforcing that is used in the Vistawall SEW uses the trade name Grid-Strip™. The Grid-Strip™ is a 2” wide discrete strip of steel welded wire mesh. The Grid-Strip™ contains two parallel longitudinal bars that are spaced 2” on center. Resistance welded to the longitudinal bars are 3” long, steel, transverse bars. The transverse bars are positioned perpendicular to the longitudinal bars and are spaced at 12” on center. The transverse bar overhangs the longitudinal bar by ½”. The longitudinal bars and transverse bars are fabricated from W11 wire in conformance with ASTM A 1064. A W11 bar has a nominal black steel diameter equal to 0.375” and an area equal to 0.11 in².

![Figure 1-5 Plan View Grid-Strip™ 2-Wire Soil Reinforcing](image)

1.2.1 Soil Reinforcing Innovations

The Grid-Strip™ innovation is in the ability to attach many different end connectors to the proximal end of the longitudinal wires. The end-connector welder was designed and fabricated specifically for this application. Two end connectors consisting of the TAB and BAR connector can be used. Each of the end connectors are single point connectors that allow the soil reinforcing to be rotated when attached to the facing unit panel anchor. The unique single point connector applies equal force to the 2-longitudinal wires unlike wide mesh system that use a single point connector. Because of this there are only two longitudinal wires attached to the single connection point the force in each wire is equal.
1.2.2 Soil Reinforcing Manufacturers

The Grid-Strip™ soil reinforcing is manufactured by Big-R Bridge in Mansfield, Texas. The Big-R Bridge manufacturing facility uses specialized equipment that was specifically developed for the Vistawall product line. This includes specialized resistance end welders, specialized 2-wire mesh welder, specialized panel anchor machine, etc. The Grid-Strip™ is manufactured to meet the Vistawall specifications and QA/QC. It should be noted, that in addition to the plant in Mansfield, Texas, that the Grid-Strip™ can be manufactured by most welded wire mesh manufactures.

1.2.3 Soil Reinforcing Types

The Grid-Strip™ consists of a discrete steel strip of welded wire. The Grid-Strip™ typically utilizes a W11 wire. The W11 wire has a black steel diameter equal to 0.374”. The area of a W11 wire area is equal to 0.11 square inches. Other size wires can be used if deemed necessary and is a function of the project requirements. All soil reinforcing is manufactured using state-of-practice resistance welders. Reference Appendix Tab 1.2.3.

![Figure 1-6 Plan View Grid-Strip™ 2-Wire Soil Reinforcing](image)

1.2.4 Soil Reinforcing Properties

The Grid-Strip™ is manufactured from cold-drawn steel green rod meeting the requirements of ASTM A1064 (Formally ASTM A82). The green rod is drawn and straightened to the specified required diameters and lengths. The Grid-Strip™ is then welded into its final configuration meeting the requirements of ASTM A1064 (Formally ASTM A185). The minimum yield strength of the steel wire is 65 ksi and the minimum tensile strength shall be equal to 75 ksi pursuant to Table 6 of ASTM A1064.

The end component, e.g, BAR or TAB, is resistance welded to the longitudinal wires using QA/QC procedures and resistance welders specified by Vistawall. The resistance weld shall have a minimum bond length equal to 1”. The weld strength shall be equal to the governing strength of the longitudinal wires.
1.2.5 Soil Reinforcing Corrosion Protection

The Grid-Strip™ 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 Grid-Strip™ is galvanized then shipped to the project site. All galvanizing is performed after complete fabrication of the Grid-Strip™.

1.2.6 Soil Reinforcing Sacrificial Steel as a Function of Service Life

The steel area of the Grid-Strip™ soil reinforcing is corrected for the loss of corrosion that occurs 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.

\[ E_c = E_n - E_s \]  

where: 
- \( E_c \) = thickness of metal reinforcement at 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 service life of structure (mil)

The service life for permanent MSE structures is typically 75 years. For critical structures, i.e., bridge abutments, the service life is 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, the sacrificial thicknesses are determined for all exposed surfaces. Structural design, and therefore corrosion, is applied to the longitudinal wires only. The following metal loss rates are used in the determination of the sacrificial thickness and are consistent with AASHTO.

- 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 method of hot-dip in conformance with ASTM A123. The galvanized coating is required to be applied at a minimum of 2 oz./ft² 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 and at the end of 100-year service life the sacrificial thickness of metal expected to be lost is equal to 0.079 inches.

Based on the AASHTO requirements and removing the sacrificial thickness of the metal from the W11 wires, at end of the service life, for the 2-wire Grid-Strip™, the allowable design strength is equal to 7.8 kips for a 75-year service life and 6.7 kips for a 100-year service life. Reference Appendix Tab 1.2.6.

1.2.7 Soil Reinforcing Corrosion Testing

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

1.2.8 Soil Reinforcing Dimensional Tolerances

The dimensional tolerances are shown in the Appendix Tab 1.2.8.

1.2.9 Soil Reinforcing Connection

The Grid-Strip™ is connected to the panel anchor by passing the end connector into the gap formed by the parallel plates of the panel anchor and then by passing a galvanized A325 bolt set through the aligned bolt holes. The connection is a single point connection that allows the Grid-Strip™ soil reinforcing to be rotated in the horizontal plane. The ability of the reinforcing to rotate allows it to avoid obstructions that are contained within the MSE mass without any need to cut the soil reinforcing. The system components, e.g., end connectors and panel anchors, have been designed so they are interchangeable. Reference Appendix Tab 1.2.9.
1.2.10 Soil Reinforcing Connection Components

The facing unit panel anchor that is used to attach the Grid-Strip™ soil reinforcing consists of a steel dual plate configuration. The panel anchor is cast into the concrete facing unit so it extends from the back face and consists of the DPTS anchor.

![Figure 1-7 DPTS Panel Anchor](image)

The DPTS panel anchor consists of fabricated parallel steel plates that are 2” in width and that are 0.135” thick and that are offset from one another by 1/2”. Each plate has a 9/16” diameter centralized bolt hole. The parallel plates are fabricated into a triangular configuration. The panel anchor is embedded into the panel a minimum of 4”. The panel anchor terminal end is positioned behind the panel reinforcing. The thickness of the panel anchor plates allows for deflection and bending in the vertical plane if unanticipated settlement of the backfill is encountered. Skewing of the soil reinforcing to the recommended maximum 15 degrees does not compromise the anchor. Calculations of the anchor in bending can be found in Appendix Section 2.1.5. The 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 reported problems.

The maximum number of anchors 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. This allows for unique placement of the anchors to avoid obstructions and to accommodate many different load cases and design conditions. The number of anchors in each row of the panel is dependent on the number of soil reinforcing elements that are required to be attached to the panel and is dependent on the project specific requirements.
The lead end of the Grid-Strip™ is resistance welded to a special end connector. There are two end connectors that can be used with the Grid-Strip™ soil reinforcing, the TAB connector, and the flat BAR connector.

The TAB connection consists of a flat 1 ½" x 3/8" steel plate with a 9/16" diameter bolt hole. The TAB plate converges to a 5/8" x 2 ½" circular shaft. Located on the circular shaft of the TAB are a series of ridges that are forged in the length. The lead ends of the longitudinal wires of the Grid-Strip™ are resistance welded to the TAB shaft at the location of the ridges. The resistance weld length shall be a...
minimum of 1”, however, 2” are used in the welding process. The additional 1” can be considered an integral factor of safety.

The BAR connector is fabricated from steel plate and is 1.5” wide x 5” long and 3/8” thick. The terminal end of the connector is stamped with serrated edges. The serrations are used to seat the longitudinal wires of the Grid-Strip™ soil reinforcing at the location of the resistance weld. The lead ends of the longitudinal wires of the Grid-Strip™ are resistance welded to the terminal end of the BAR at the location of the serrations. The resistance weld length shall be a minimum of 1”, however, 2” are used in the welding process. The additional 1” can be considered an integral factor of safety.

The Grid-Strip™ end connector is connected to the facing unit panel anchor by passing the end connector into the gap formed by the parallel plates of the anchor and then by passing a galvanized A325 bolt set through the adjacent bolt holes. The connection is a single point connection that allows the Grid-Strip™ soil reinforcing to be rotated allowing it to avoid obstructions that are within the MSE mass. The system components have been designed so they are interchangeable. Reference Appendix Tab 1.2.9.

1.2.11 Soil Reinforcing Connection Manufacturers

The connection components are manufactured in-house as well as by outside Vendors to the required Vistawall specifications. Vistawall provides manufacturing guidelines and also reviews each Vendors QA/QC program. The following list are manufactures that have been used to manufacture connection components.

- DPTS Anchor-Precision-Hayes International - Sugar Land, Texas
- TAB Connector-Trenton Forging – Trenton, Michigan
- DPTS Anchor-Big-R Bridge – Mansfield, Texas
- BAR Connector-Big-R Bridge – Greeley, Colorado.

Each component is required to meet the QA/QC specification set forth by the project or by Vistawall with the more stringent taking precedence. All Vendors are required to certify that their product meets the requirements set forth by the project and Vistawall.

1.2.12 Soil Reinforcing Connection Properties

The connection is designed so that the limiting component is the Grid-Strip™ soil reinforcing longitudinal wire elements. The facing unit DPTS anchor is fabricated from a minimum Grade-50 steel in
conformance with ASTM A572. The minimum tensile strength is 65 ksi and the minimum yield strength is 50 ksi. The facing unit DPTS anchor is fabricated from a minimum Grade-36 steel in conformance with ASTM A572. The minimum tensile strength is 50 ksi and the minimum yield strength is 36 ksi. The ½” diameter bolt is fabricated in conformance with ASTM 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 A325 bolt is 72 ksi. The longitudinal wires are resistance welded to the terminal end of the end connector. The end connector terminal end is fabricated with special shaped serrations that provide strength to the welding process. The minimum weld length is 1”. The actual weld length is 2”. The resistance weld is applied using a special machine that was developed to manufacture this product. The weld is tested using the Vistawall Quality Control procedure. The weld test is a destructive test.

1.2.13 Soil Reinforcing Connection Corrosion Protection

The Grid-Strip™ 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 Grid-Strip™ components are galvanized in their finished state. In other words, once the components are galvanized no further fabrication is completed. The minimum coating thickness for a 75-year design life is 2 oz./ft² or 3.4 mils in thickness.

1.2.14 Soil Reinforcing Connection Sacrificial Steel as a Function of Service Life

The steel area of the Grid-Strip™ soil reinforcing and all steel components 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.

\[ E_c = E_n - E_s \]  

Equation 2

where:

- \( E_c \) = thickness of metal reinforcement at 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 service life of 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. For structural design,
the sacrificial thicknesses are determined for all exposed surfaces. Structural design, corrosion is applied to the plate and to the bolt. The following metal loss rates are used in the determination of the sacrificial thickness and are consistent with AASHTO.

- 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, for components other the soil-reinforcing is applied by the method of spin-dip in conformance with ASTM A153. The galvanized coating is required to be applied at a minimum of 2 oz./ft² or 3.4 mils in thickness. The sacrificial thickness of metal expected to be lost at the end of a 75-year service life is equal to 0.056 inches and at the end of 100-year service life the sacrificial thickness of metal expected to be lost is 0.079 inches.

1.2.15 Soil Reinforcing Connection Corrosion Testing

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

1.2.16 Soil Reinforcing Connection Dimensional Tolerances

The dimensional tolerances for each of the components is shown in the Appendix Tab 1.2.16

1.2.17 Soil Reinforcing Connection Strength and Testing

Connection strength testing has been completed on the Grid-Strip™ system connection components and for varying thickness of facing units. Full results are presented in Appendix Tab 1.2.17. The results of the connection pullout testing confirm that the limiting component is the Grid-Strip™ soil reinforcing. The following connection strength for a 100 year design life and 100 year design life are equal to the allowable strength of the Grid-Strip soil reinforcing for a W11 wire size using the AASHTO design methodology and the considerations of corrosion.

- 75-year design – 7.74 kips
- 100-year design – 6.66 kips
1.2.18 Soil Reinforcing Pullout Testing and Results

The pullout tests for the Grid-Strip™ have been performed to establish the friction factor to be used in the design and analysis of the Vistawall Mechanically Stabilized Earth (MSE) wall system utilizing Grid-Strip™ soil reinforcing. The testing was performed at the Civil Products Laboratory located at Big-R Bridge in Mansfield, Texas, using state-of-practice procedures in general conformance with ASTM D 6706 “Standard Test Method for Measuring Geosynthetic Pullout Resistance in Soil”. An outside consulting group supervised the testing and prepared the pullout test report. The pullout test report is provided in the Appendix in Tab 1.2.18. All type W11 cross wires were tested and are used with all Grid-Strip products. A W15 Grid-Strip consists of W15 longitudinal wires and W11 transverse wires; therefore, the pullout values that were tested for the Grid-Strip with W11 longitudinal wire and W11 cross wire are applicable and conservative.

The F* values used in the test are conservative representation of backfill material that will be used with the system. For fine-grained material F* values are based on 1-pieces linear regression (straight-line relationship) and are equal to 3.00 at the top of the structure decreasing to 1.25 at a depth of 20 feet and below. For granular material F* values are based on a 2-piece linear regression (Bi-linear relationship) equal to 6.00 at the top of the structure decreasing to 1.50 at a depth of 20 feet and below. The test demonstrated that the alpha value is equal to 1.0 for the grid-strip. Vistawall recommends using the fine-grain material pullout resistance in design with and F* value equal to 3.00 decreasing linearly to 1.00 at a depth of 20 feet and below as shown in Figure 1-11.
1.3 Other Components

1.3.1 Other component Innovations

No innovations in other components is claimed.

1.3.2 Reinforced Soil Properties

The FHWA-NHI-10-024 Design of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes (FHWA GEC 011) requires that MSE structures be constructed with “high quality fill for durability, good drainage, constructability, and good soil reinforcement interaction”. The Vistawall MSE Grid-Strip™ system pullout resistance depends on friction between the longitudinal wire and the soil and passive resistance that occurs at the transverse wire and the soil. The backfill is required to be a granular material.
with high friction characteristics and quality. The internal friction angle of the backfill shall not be less than 30 degrees. Experience has demonstrated that the performance of an MSE structure that is constructed with backfill that meets these performance requirements are easier to construct and have little post construction problems.

The reinforced soil typically consists of a select granular backfill material. The select granular backfill is required to be reasonably free from organic and otherwise deleterious materials. The gradation limits typically conform to the gradation limits as specified in FHWA-NHI-10-024 as shown in Table 3. The select granular backfill shall conform to the Unified Soil Classification (USC) in ASTM D2487. The backfill shall not be gap graded. The plasticity index (PI) shall not exceed 6. The use of the proposed backfill shall be evaluated on a project-by-project basis. Reference Appendix Tab 1.3.2. The electrochemical requirements shall conform to the requirements of FHWA-NHI-10-024 as shown in Table 4.

<table>
<thead>
<tr>
<th>U.S. Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>4”</td>
<td>100</td>
</tr>
<tr>
<td>No. 40</td>
<td>0-60</td>
</tr>
<tr>
<td>No. 200</td>
<td>0-15</td>
</tr>
<tr>
<td>Plasticity Index (PI)</td>
<td>PI &lt; 6</td>
</tr>
</tbody>
</table>

Table 3  FHWA GEC 11 Backfill Requirements (Table 3.2)

<table>
<thead>
<tr>
<th>Property</th>
<th>Criteria</th>
<th>Test Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resistivity</td>
<td>&gt; 3000 ohm-cm</td>
<td>AASHTO T-288</td>
</tr>
<tr>
<td>pH</td>
<td>&gt; 5 and &lt; 10</td>
<td>AASHTO T-289</td>
</tr>
<tr>
<td>Chlorides</td>
<td>&lt; 100 PPM</td>
<td>ASTM D4327</td>
</tr>
<tr>
<td>Sulfates</td>
<td>&lt; 200 PPM</td>
<td>ASTM D4327</td>
</tr>
<tr>
<td>Organic Content</td>
<td>1% max</td>
<td>AASHTO T-267</td>
</tr>
</tbody>
</table>

Table 4  Recommended Limits of Electrochemical Properties for Reinforced Fills with Steel Reinforcement.

The above backfill is consistent with FHWA requirements. It should be noted, that the backfill for each project is evaluated, and if determined to be acceptable, the material strength parameters defined through testing of the backfill are used in the design of the ERS.
1.3.3 ERS Drainage

Vistawall will require the Owner to design and specify the drainage requirements for each project. When necessary, Vistawall will make recommendations for drainage. If drainage is required, the details shown in Tab 1.3.1 will be implemented. At a minimum, Vistawall requires a base drain located at the back of the MSE as shown in Tab 1.31.

The drainage requirements for the MSE structure are a function of the application. It is well known that drainage is critical to the performance of an MSE wall. Drainage must be considered for both external and internal water sources. External drainage diverts water that flows externally over and/or around the structure. The external drainage depends on the location of the MSE structure. External drainage typically will divert any water flow away from the MSE structure. 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 characteristics of the backfill used in the reinforced soil mass. The surface water or subsurface water must be collected and diverted away from the MSE structure or the backfill must consist of material that will allow for unimpeded flow while preventing degradation of the backfill material.

The joint configuration of the Vistawall Segmental concrete panel allows water that infiltrates the MSE backfill to drain out of the MSE structure. To prevent the migration of fine material from the soil directly behind the SCP, all of the panel joints are covered with a filter fabric. The filter fabric allows water to pass through and at the same time prevents fine material from passing through. This combination allows for the drainage of water. Reference Appendix Tab 1.3.3.

When required, drainage must be considered at the interface of the retained backfill and reinforced backfill, the interface of the foundation and reinforce backfill, and at the interface of the facing unit and the reinforced backfill. This can be accomplished with back-drains (chimney-drain), blanket-drains, and face drains, respectively. All three of these drainage systems may be required, or any combination of these drainage systems may be required. The long-term function of the drain is an important consideration.

1.3.4 ERS Coping

A coping unit typically covers the top panel. Coping units are shown in Photograph 1-3. The standard U-shaped coping unit is shown in Figure 1-12. Vistawall has coping units for applications on
standard structures that can accommodate traffic barriers, drainage swells, approach slabs, and standard grade changes. Coping unit details are shown in Appendix Tab 1.3.4.
1.3.5 **ERS Traffic Barrier**

Vistawall coping units are designed to attach a traffic barrier to it. The coping unit and traffic barrier can be designed to accommodate all TL load cases. Details of the coping unit with traffic barrier are shown in Appendix Tab 1.3.5.

![Figure 1-13 Coping Unit with Traffic Barrier](image)

1.3.6 **ERS Abutments**

The Vistawall structure can be placed at the interface of abutments that consists of a cast-in-place structure that utilizes a stem and a footing (Figure 1-14). The interface can be isolated with the use of a slip-joint unit. The isolation using the slip joint unit allows for the MSE structure and the Abutment to act independently of one another. To provide stability to the slip-joint unit a Grid-Strip™ soil reinforcing element is attached to the slip joint unit. Details of the slip joint are shown in Appendix Tab 1.3.6.

In other situations, the interface can consist of a vertically on obstructed joint that runs the full height of the interface. The segmental concrete panel facing units can be cast without the shiplap joint. When this is performed a filter-fabric is applied to the back face of the panel at the interface. Details of this configuration are shown in Appendix Tab 1.3.6.
1.3.7 ERS Slip-Joints

Slip-joints are used at locations where differential settlement is anticipated. The slip-joint unit is T-shaped. The leg of the T-shape is where the soil reinforcing is attached. The soil reinforcing provides stability to the unit. To prevent fine material from migrating out of the reinforced backfill, filter fabric is applied on each side of the slip joint. Details for slip joints are shown in Appendix Tab 1.3.7 and in Figure 1-15.
2.0 ERS DESIGN

2.1 DESIGN METHODOLOGY

2.1.1 ERS Design Innovations
No innovations in design are claimed.

2.1.2 ERS AASHTO LRFD Design Methodology
The VistaWall design methodology conforms to the AASHTO LRFD Bridge Specification. The current specification does not specifically address two-wire grid-strips in the stiffness design methodology. Based on the derivation of the stiffness design methodology, the reinforcing ratio of the VistaWall grid-strip is classified as a linear steel strip. Therefore, the VistaWall grid-strip is modeled as a linear steel strip, with the two wires sustaining equal transfer of force and therefore each wire is stressed equally. The pullout friction factors indicated by tests for different VistaWall Grid-Strip configurations are provided at Appendix Tab 1.2.18.

2.1.3 ERS Proprietary Design Methodologies
No proprietary design methodologies are claimed.

2.1.4 ERS Facing Design Requirements
The facing panels are designed in conformance with Vistawall design methodology for Mechanically Stabilized Earth (MSE) retaining structures and for the analysis of the Stabilized Earth Wall™ utilizing a Segmental Concrete Panel. The outlined method is consistent with the 7th Edition of the American Association of State Highway and Transportation Officials LRFD Bridge Design Specifications, 2014, and all interim specifications.

The following methodology is proprietary to VAWS. It is consistent with the Simplified Coherent Gravity design procedure utilizing the Load and Resistance Factored Design. The panel force analysis is determined using the software program RISA-3D and VAWS design methodology. The facing panel analysis is consistent with the AASHTO, LRFD. Section 5 - Concrete Structures. The following design methodology is used for the facing analysis.

**Design Methodology Outline:**

1. Determine maximum applied force at each anchor
2. Perform finite element analysis to determine maximum moment
3. Determine concrete panel parameters
4. Determine panel reinforcing parameters

5. Vertical concrete reinforcing
   a. Calculate maximum moment
   b. Verify flexural requirements

6. Horizontal concrete reinforcing
   a. Calculate maximum moment
   b. Verify flexural requirements

7. Detail panel reinforcing

A facing design example for the 5 x 5 panel and for a 5 x 10 panel are included in the Appendix in Tab 2.1.4. these should be considered examples that defines the methodology. Panel analysis is performed on a project by project basis.

2.1.5 ERS Splayed Soil Reinforcing Design Requirements

The Grid-Strip soil-reinforcing is a linear steel strip with a single point connector. The Grid-Strip end connector consists of a plate with two, 2” spaced longitudinal wires welded to each side. The connection of the soil-reinforcing to the segmental concrete panel is made by passing a bolt between the panel anchor and the plate of the Grid-Strip. The Grid-Strip single point connector has the capability to swivel in the soil allowing for splaying of the soil-reinforcing by obstructions. The force applied to the Grid-Strip soil-reinforcing is equally shared between each of the longitudinal wires. The Grid-Strip connection is designed to be in conformance with AASHTO Section 10.32.

The history of the development of tensile resistance factor and why there is a difference between two steel system needs to be understood. The history starts with the NCHRP Report 290, then to Task force 27, then to FHWA 89-043, then to ReCO review comments to 1991 interim specification, then to the AASHTO Bridge specification 1994, then to the AASHTO 17th Edition 2002, etc.

Systems that use grid type reinforcing members connected to the segmental concrete panels using steel wire loop anchors and a pin has a resistance factor that is less than systems that utilize a single bolted connector. The allowable stress was reduced in recognition of the possibility that there would be uneven stresses measured in longitudinal members at the connection for grid type systems that had more than 2 wires. In other words, wide mesh systems. The connection of wide mesh soil reinforcing systems
is made at each of the longitudinal member to a single anchor point behind the first transverse wire. As such, it was assumed that the transverse bar during loading would act in a manner analogous to that of a continuous multiple span beam that was uniformly loaded and simply supported. This assumption completely ignored the fact that the beam is encased in compacted selected backfill and therefore is more analogous to a beam on an elastic foundation. Because early welded wire mesh systems commonly used grids consisting of members with four, five, or six longitudinal elements, the interior reactions were assumed to carry a load that ranged between 110 percent and 117 percent of the product of the span length and load. The simplest manner to introduce this non-uniform loading of longitudinal members was to reduce the allowable stress accordingly or to \((1/1.15 \times 0.55) \times 0.48 \text{ Fy} \) (now 0.65). Because the Grid-Strip is a two member element that is connected to a single-point bolted connector, coupled with the fact that the span length is 2” when compared to 6”, 8” and 9” spacing of wide mesh systems, the longitudinal wires will carry equal load. Based on this the Grid-Strip is designed to be in conformance with AASHTO Section 10.32 and use 0.75 as the resistance factor.

The splaying of the soil reinforcing decreases the allowable load that can be carried by the element. The reduction in load is based on the splay angle. As the splay angle increases the load reduction increases and the allowable load decreases. The reduction factor is based on the cosine of the splay angle. AASHTO limits the splay angle to 15 degrees. Splaying soil reinforcing greater than 15 degrees can be used if approved by the Retaining Wall Engineer of Record and the forces adjusted accordingly. Reference FHWA-NHI-10-02, Section 5.4.2, Vertical Obstructions in Reinforced Soil Mass, for more guidance on this concept.

\[
\phi_{\text{skew}} = \cos(\alpha)
\]  

Equation 3

Where:  
\(\phi_{\text{skew}}\) = splay reduction factor (dim)  
\(\alpha\) = splay angle (deg)
### Table 5  Soil Reinforcing Splay Angle Reduction Factor

<table>
<thead>
<tr>
<th>Force Diagram</th>
<th>Angle ($\alpha$) (deg)</th>
<th>Reduction Factor $\phi_{skew}$</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image" alt="Diagram" /></td>
<td>0</td>
<td>1.000</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>0.996</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>0.985</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>0.966</td>
</tr>
</tbody>
</table>

For a splayed soil reinforcing element, the appropriate reduction in the allowable capacity of the soil reinforcing in both tension and pullout is incorporated into the design calculations. The reduced allowable tensile design strength for a splayed soil reinforcing element is calculated as shown in Equation 4.

$$T_{max} = \phi_{skew} \cdot \phi_t \cdot A_c \cdot F_y$$  \hspace{1cm} Equation 4

Where:
- $T_{max}$ = Reduced maximum allowable tensile force (kip)
- $\phi_{skew}$ = Splay reduction factor (dim)
- $\phi_t$ = Tensile resistance factor (0.75 AASHTO Table 11.5.6-1) (dim)
- $A_c$ = Design area of steel bar (included effects of corrosion) (in\(^2\))
- $F_y$ = Steel Yield Strength (65 ksi – ASTM A1064) (ksi)

The reduced allowable pullout capacity of the splayed soil reinforcing element is calculated as shown in Equation 5.

$$T_{po} = \phi_{skew} \cdot \phi_p \cdot F^* \cdot \alpha \cdot \gamma \cdot z_i \cdot C \cdot L_e$$  \hspace{1cm} Equation 5

Where:
- $T_{po}$ = Calculated soil reinforcing pullout resistance
- $\phi_{skew}$ = Splay reduction factor
- $\phi_p$ = Pullout resistance factor
- $F^*$ = Pullout resistance factor
- $\alpha$ = Scale correction factor
\[ \gamma = \text{Unit weight of backfill} \]
\[ z_i = \text{Depth to soil reinforcing} \]
\[ C = \text{Unit perimeter factor} \]
\[ L_e = \text{Length of embedment} \]

When splaying soil reinforcing by obstructions care should be taken to balance forces in the segmental concrete panel facing to assure that the SCP panels do not rotate outward. This can be accomplished by adding more soil reinforcing anchorage points and soil reinforcing elements or by not having the soil reinforcing all splayed to the same side, or to the same angle. Further, the unbalanced loading can be resisted by connection of the panel that has splayed soil reinforcing to the adjacent segmental concrete panel using a panel-to-panel connection. The DPTS anchor used in the Vistawall system does not exceed the plastic moment when the soil reinforcing is skewed to 15 degrees.

2.1.6 **ERS Vertical Obstruction Design Requirements**

In conditions were vertical obstructions interfere with the standard connection of soil reinforcing the Vistawall utilizes a panel-to-panel connector to adjacent facing units together. The size of the panel-to-panel connector will be dependent on the force that is required to be resisted. The force that is required to be resisted is a function of the size of the vertical obstruction and the tributary area of the facing units that are resisting the force.

![Photograph 2-1 Vertical Obstruction (concept photo - for information only)](image)

2.1.7 **ERS Horizontal Obstruction Design Requirements**

A horizontal obstruction that is three feet behind the facing unit can be 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 18” of the connection. The soil reinforcing shall be horizontal for a minimum distance of 18” from the back of the facing unit before deflection of the soil reinforcing. In no instance shall the maximum deflection of the soil reinforcing be greater than 15 degrees. The degree of deflection is a function of the size of the horizontal obstruction including an additional 3” of backfill that is required to be placed on the obstruction as shown in Figure 2-1. The Vistawall system is designed in conformance with FHWA requirements.

![Figure 2-1 Horizontal Obstruction Detail](image)

Because of the possibility of differential settlement of the backfill located at, and near, the obstruction, it is better to bypass the soil reinforcing by deflecting the soil reinforcing up, and over the obstruction. However, in some instances it may be necessary to deflect the soil reinforcing down, and under the horizontal obstruction. In this case the soil reinforcing is placed before placement of the obstruction. Table 6 provides guidance on the recommended depth (d) and distance (x) for horizontal obstructions and is in conformance with FHWA-NHI-10-024. The maximum horizontal skew angle shall be limited to 15 degrees.

<table>
<thead>
<tr>
<th>Additional depth (d) required, in.</th>
<th>Required minimum distance (X)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>27</td>
</tr>
<tr>
<td>6</td>
<td>39</td>
</tr>
<tr>
<td>9</td>
<td>48</td>
</tr>
</tbody>
</table>
Table 6  Recommended Depth and Distance for Horizontal Obstruction

<table>
<thead>
<tr>
<th>Additional depth (d) required, in.</th>
<th>Required minimum distance (X)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>60</td>
</tr>
<tr>
<td>15</td>
<td>72</td>
</tr>
</tbody>
</table>

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. 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

The deflection of the anchor may cause the plates to bend downward. The soil reinforcing, and panel anchor are flexible and is encased in compacted backfill. If the load is great enough, or if compression of the backfill occurs, the anchor may exceed the elastic limit and permeant deformation may occur. Bending of the anchor is not a structural issue. It should be noted that the DPTS anchor is identical to MSE panel anchors that have been used in MSE for over 30 years without any undue problems.

2.2  Design Drawings
Reference Appendix Tab 0 for a sample set of project plans.

2.3  Design Examples
2.3.1  Level Backslope with Traffic Live Load Design Example
Reference Appendix Tab 2.3.1 for a sample Level Backslope with Traffic Live Load Calculation. The following is an excerpt from Section 8.8 of the Calculation that is provided in Tab 2.3.1 in the Appendix.
For MSEW to be compared to MSE-Pro and AASHTO requirements the external stability analysis and internal stability analysis must be performed 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 consistent load factor of 1.35 (EV) for all internal loads, in other words, it does not use 1.75 (LS) for traffic live load surcharge. The use of 1.35 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. Based on MSEW definitions, the program uses an average horizontal pressure at each level of soil reinforcing, and therefore, in the 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 in the appendix Tab 2.3.1, Z-top is equal to 30.00 feet and the top soil reinforcing element is defined at elevation 27.75 (a depth of 2.25 feet from the top of wall). The second soil reinforcing element 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 3.686 and 3.258 respectively. In the program MSE-Pro the calculated CDR for the top soil reinforcing element and the second soil reinforcing element is 2.96 and 3.38 respectively. MSEW over predicts the CDR in the top row by 25%. 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 and MSEW underestimates the maximum tension force that is required to be resisted.

MSE-Pro does not use the average horizontal pressure to calculate the maximum tensile force. MSE-Pro 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 in the calculations in Tab 2.3.1. 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.

### 2.3.2 2:1 Backslope Design Example

Reference Appendix Tab 2.3.2 for a sample 2:1 Backslope Calculation. The following is an excerpt from Section 8.8 of the Calculation that is provided in Tab 2.3.2 in the Appendix.

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 program’s 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 the example in the Appendix Tab 2.3.2 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 1.748 and 1.980 respectively. In the program MSE-Pro the calculated CDR for the top soil reinforcing element and the second soil reinforcing element is 1.80 and 1.42 respectively. MSEW under
predicts the CDR in the top row and over predicts 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 the appendix Tab 2.3.2 and 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.

3.0 CONSTRUCTION

3.1 CONSTRUCTION PROCEDURES

3.1.1 ERS Construction Innovations

Vistawall does not claim any construction innovation.

3.1.2 ERS Foundation and Leveling Course Preparation

The foundation must be able to support the MSE embankment including all externally applied loads. For foundations where the insitu strength cannot be quantified, or verified, the following proof-rolling procedure shall be used. Proof-rolling shall be performed to evaluate the stability and uniformity of the subgrades on which the MSE structure will be constructed. Before proof-rolling the foundation that will support the Vistawall MSE structure it shall be cleaned and grubbed in accordance with the project specifications.

Proof rolling shall be performed with a pneumatic-tired, tandem-axle roller with a minimum of 3 wheels for each axle. The roller shall have a gross weight of 25 tons (50 kips) with a minimum rolling width of 75 inches. The minimum tire pressure shall be 75 psi. Acceptable rollers include a Caterpillar PS-300B (or PF-300B), Ingersoll-Rand PT-240R, BOMAG BW24R, Dynapac CP271. Other roller equipment with equivalent capabilities may be used for proof-rolling.
Proof rolling equipment shall be operated at a speed between 1.5 to 3.0 miles per hour, or slower, as required to permit measurements of rutting and/or pumping deformations. Proof-rolling shall be carried out in two directions at right angles to each other. There shall be no more than 24 inches between tire tracks of adjacent passes. In cases where proof-rolling perpendicular to the wall alignment is not possible due to space constraints, proof-rolling shall be performed parallel to the wall with a minimum of 6 inches of overlap between the tracks of adjacent passes. The Contractor shall operate the proof roller in a pattern that readily allows for complete coverage of the subgrade and the recording of deformation data. The following actions shall be taken based on the results of the proof-rolling activity:

1. Rutting less than ¼ inches – The subgrade is acceptable.
2. Rutting greater than ¼ inches and less than 1½ inches – The subgrade needs to be scarified and re-compacted.
3. Rutting greater than 1½ inches – Removal and reconstruction of the compacted area.
4. Pumping greater than 1 inch – RemEDIATE as directed by the Engineer. Pumping shall be defined as deformation which rebounds or where materials are squeezed out of wheel’s path.

The segmental concrete panels shall be placed on an unreinforced concrete leveling course. It should be noted that the leveling course is sometimes referred to as a leveling pad by some jurisdictions. The leveling course shall be placed in accordance with the steps and grades shown in the Vistawall approved shop drawings. The foundation area that the leveling course base will bear on shall be compacted after excavation using a pneumatic tamper or walk-behind compactor. The leveling course
shall be placed to the required elevation to a tolerance of ±1/8”. Forms shall be used to contain and level the concrete. The use of a forming system provides for assurance that a level surface can be attained.

3.1.3 ERS Construction Tool Requirements

The construction of Vistawall Grid-Strip™ system is a relatively straightforward and repetitive process that does not require specialized labor or equipment. The following list includes suggestions for tools, equipment, and construction aids that can make the installation of the SEW easier. The following list are not all inclusive and are only suggestions. The contractor’s qualifications and experiences will dictate the tools that are required to construct the SEW.

**Hand Tools**

- 2-foot and 4-foot long carpenter levels.
- Claw hammers.
- Rubber mallet
- Chalk line and chalk.
- Caulking gun for 29-ounce tubes of adhesive.
- Wrenches for clamps (2 ea.)
- 30-inch or 36-inch crow bars (2 ea.)
- Sledgehammer.
- Hand-operated or power-operated saws.

**Lifting and Unloading Equipment**

- Panel lifting ring clutch
- Spacing tools
- Non-staining dunnage for storage of precast concrete panels.
- Two 22-foot long web slings for unloading panels.
- Lifting chains

**Construction Aids**

- Wood clamps with coil rods, coil nuts and washers
• Wood braces
• Wood or steel stakes
• Hard wood shims

Heavy Equipment

• Hydraulic crane or boom truck to lift and place precast concrete facing panels. A standard 5x5 panel weight is 2000 pounds. A standard 5x10 panel weight is 4000 pounds (assumes nominal 6” thickness)
• Dump trucks, front-end loaders, scrapers, bulldozers, or graders to place backfill.
• Water truck
• Smooth-drum vibratory roller or pneumatic-tired, tandem-axle roller with a minimum of 3 wheels for each axle
• Walk-behind vibratory roller or plate compactor

3.1.4 ERS Facing Installation Requirements

Vistawall segmental concrete panels are available in three basic sizes: 5’ x 5’, 5’ x 10’ and full height. Each panel is labeled and detailed on the panel schedule or on the standard panel detail sheet. The panel label provides the panel type and number of panel anchors per row. Note that you can easily identify each panel by the information that is scribed on the back of each 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 placed properly, the panel will need to be adjusted to ensure that the horizontal and vertical alignment is achieved and maintained during the erection process. It is not necessary to place a bearing pad on the leveling course for the panel to bear on. The panel should be set directly on the leveling course. It is important 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 will be easier to install.

Some difference in vertical tolerance can be corrected by using hard-wood or plastic shims. However, the use of shims should be minimized and used only for minor corrections in joint/panel alignment. It should be noted that the use of wedges along the front face of the panels may be used to
facilitate proper panel batter however, their use should be closely 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 batter, it is
recommended, but not required, to position 2 wooded wedges on the front and back side of the bottom
panel. The wedges when lodged under the panel will aid in stabilization of the panel, will provide for easy
adjustment of the batter, and will aid in leveling the panel to the proper elevation before placement of
the backfill. It is also recommended that the first row (or rows) of panel use a bracing system that can
adjust the panel alignment.

The backfill is placed and compacted slightly above (1” to 2”) the level of the panel anchor. Once
the level is compacted and is above the panel anchor, the soil reinforcing is then installed. The next row
of panels is placed when the backfill is brought to the top of the panel in the alternating panel
arrangement. The procedure of placing panels, backfill, compacting, and placement of soil reinforcing,
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 function of the size of the radius. When it is required
that panels be placed on a curve the shop drawings prepared by Vistawall will detail the curve layout and
the required joint configuration for each panel. Depending on the radius of the curve, the joints may
consist of the standard ship-lap configuration. The 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. If this is required, the panels will be detailed in the shop drawings.
It should be noted that in some cases the leveling course width may need to be increased to assure that
the panel base is able to be placed on a complete concrete surface.

3.1.5 ERS Soil Reinforcing Installation Requirements

The Grid-Strip™ is placed on the compacted select backfill and the connection is made to the
DPTS panel anchor. The Grid-Strip™ is connected to the panel by securing the TAB and/or BAR connector
between the two plates of the DPTS panel anchor using a bolt and nut. The bolt only needs to be finger
tight. It does not require that a wrench be used. The threads should just start to protrude from the nut. A
washer is not required. The washer is not required because this is not a structural joint. Never connect a
Grid-Strip™ 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 DPTS anchor. The length of the Grid-Strip™ and the number of required Grid-Strips to be placed on each panel is detailed in the approved Vistawall shop drawings.

![Figure 3-1 Placement of Grid-Strip™ on Frist Row of Panels](image)

If adjacent soil reinforcing is required to be overlapped, they shall be separated by 3” of compacted backfill. Overlapping of the soil reinforcement occurs at corners, tight radius, and back-to-back structures.

### 3.1.6 ERS Facing Alignment Requirements

The vertical and horizontal alignment of the wall face shall maintain a ½ inch tolerance at any point along a 10-foot straight edge that is 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 final overall vertical tolerance of the wall (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 at the third row of panels a plumb-bob is used to check the overall vertical tolerances for every panel. 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 methods used to compact the soil, the moisture content of the soil, the elevation of the backfill in relationship to the panel anchor, the methods used to set and clamp the panels, and the experience of the crew.

### 3.1.7 ERS Reinforced Backfill Placement Requirements

The placement of the backfill 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”-12” 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 backfill density can be achieved. *The fill shall be leveled by equipment moving parallel to the wall face. The material shall be spread so it is fanned toward the terminal end of the soil reinforcing.* The placement of the backfill from the front of the soil reinforcement 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 back face of the panel shall be performed with an 8-ton to 10-ton roller. A smooth wheel or rubber tire roller is 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 end of the reinforcement.* Proper moisture content of the backfill material should be maintained uniformly within each lift. Experience has established that the backfill should be placed on the dry side of the optimum moisture content. Care should be used in adding water to the backfill material. Further, after any rain event it is advised that compaction of the backfill or placement of additional backfill not commence until the material 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 with the use of a vibratory roller or plate compactor with a centrifugal force equal to a minimum of 1000-pound to a maximum of 4000-pounds. Care should be exercised when compacting the 3-foot zone so as to not disturb the alignment of the panel. Fine grain soils should be compacted with care.
Photograph 3-2 Compaction of 3-Foot Zone

Compaction tests and gradation tests should be taken and recorded in accordance with the contract specifications. At a minimum at least one test per 2000 ft² per 30 inches of fill thickness shall be performed. Compaction test should not be performed in the area directly adjacent to the back face of the panel. 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.

3.1.8 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 of the structure, in a manner that prevents high velocity water flow. Soil berms can also be used to control ground water. If the berm is constructed using a readily erodible material such as sand, then encase the soil in a geotextile. 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.
3.1.9  **ERS Installer Requirements**

The following are minimum requirements that should be used and verified to properly qualify the installer of the Vistawall MSE system.

**Project Job-Site MSE Wall Installer**

The Job-Site MSE Wall Foreman shall have experience in 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 Vistawall MSE system. The resume and credentials of the Foreman shall be submitted to the Vistawall Project Engineer for approval prior to the pre-construction meeting. The Foreman shall be at the project site during all times that the work is being performed.

**Alternate Prequalification Criterion (in 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 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 a Vistawall 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 Vistawall to be present during the construction of each wall test segment. The Vistawall Technical Representative shall be present for construction of the wall test segment. The wall test segment shall include construction of each of the 5 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, construction of a corner representative of the corners in the wall in the project in a minimum of a two-row portion of panels or a minimum of a four-foot wall height

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

- Technical instructions as required in the construction of the Vistawall Grid-Strip™ 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 Vistawall Technical Representative will provide the following documentation for final approval by the Project Engineer:

- Documentation that the wall test segment was constructed in accordance with 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

3.1.10 ERS Retained Backfill Placement Requirements

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

3.1.11 ERS Construction Manual

Reference Appendix Tab 3.1.11 for the Vistawall Grid-Strip™ Installation Manual.

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
Reference Appendix Tab 4.2.1 for Construction QA/QC requirements.

5.0 PERFORMANCE

5.1 WARRANTIES AND DISCLAIMERS

The Vistawall warrants the items of work that are in their direct control. All fabricators warrant the material by certifying that the material conforms to the project specifications and Vistawall Quality Control Specifications.
The standard disclaimer includes a clause dictating that Vistawall is responsible for the internal stability of the structure only and that all external stability (sliding, limiting eccentricity, bearing and settlement) and global stability (deep seated rotation) are the responsibility of others. Vistawall only has control on specifying and quantifying the material that is used as backfill in the reinforced soil volume.

Vistawall will not provide any warranty that the wall was built to specification unless full-time site assistance was provided. Vistawall can, and will, only warrant construction that was witnessed by the Vistawall Technical Representative.

Licensed Professional Engineers (PE) employed by big-R Bridge design the MSE structures. The PE is licensed in the state where the MSE wall is being constructed. The following are the limits of professional liability for Big-R Bridge.

Professional Coverage-total is $7M as follows:
Each Claim: $2M
Annual Aggregate: $2M
Excess Policy
Each Claim: $5M
Annual Aggregate: $5M

5.2 Testing
5.2.1 Pullout Testing

The pullout tests for the Grid-Strip™ have been performed to establish the friction factor to be used in the design and analysis of the Vistawall Mechanically Stabilized Earth (MSE) wall system utilizing Grid-Strip™ soil reinforcing. The testing was performed at the Civil Products Laboratory located at Big-R Bridge in Mansfield, Texas, using state-of-practice procedures in general conformance with ASTM D 6706 “Standard Test Method for Measuring Geosynthetic Pullout Resistance in Soil”. An outside consulting group supervised the testing and prepared the pullout test report. The pullout test report is provided in the Appendix in Tab 1.2.18.

5.2.2 Connection Testing

Connection strength testing has been completed on the Grid-Strip™ system connection components and for varying thickness of facing units. Full results are presented in Appendix Tab 1.2.17. The results of the connection pullout testing confirm that the limiting component is the Grid-Strip™ soil reinforcing.
5.3 PERFORMANCE HISTORY

5.3.1 ERS Performance History

The Grid-Strip™ system has been used since November of 2011. Since its inception over 4 million square foot of wall has been installed. VAWS is not aware of major system performance problems. Problems that have occurred are considered typical to MSE construction. These include joint alignment problems, panel alignment problems, and backfill compaction problems. These problems can typically be attributed as construction problems.

5.3.2 ERS Oldest Structures

The first Grid-Strip™ project was constructed on US Highway 281 for the Texas Department of Transportation (TxDOT) in Jack County on November 14, 2011. It was installed by the Earth Builders, LP. The maximum wall height was 21.5 feet and the total wall area equal to 5,904 square feet. The project was a bridge abutment supported on drilled shafts with approach ramps.

The second oldest structure was constructed for Nova Scotia Transportation and Infrastructure in Springhill, Nova Scotia, by Dexter Construction in the fall of 2011. The maximum wall height was 34.4 feet and the total wall area equal to 5,328 square feet. The project consisted of two bridge abutments supported on piles.

The third oldest structures were constructed for the Calgary Airport Authority in Calgary, Alberta by Wilco Contractors, and Wouthwest Inc., in the fall of 2012. The maximum wall height was 27.9 feet with a total wall area equal to 35,521 feet. The project consisted of grade separation structures.

5.3.3 ERS Tallest Structures

The tallest Grid-Strip™ project was constructed on Interstate 10 and US Highway 301 Interchange for the Florida Department of Transportation (FDOT) in Duval County, Baldwin on June 6, 2016. It was installed by RWH for Superior Construction. The maximum wall height was 45 feet and the total wall area was equal to 147,012 square feet. The project consisted of a series of approach ramps and grade changes.

The second tallest structure was constructed on Interstate Highway 35 (IHDPS-35) for the Texas Department of Transportation (TxDOT) in Bell County, on May 31, 2013. The maximum wall height was 39.4 feet with a total wall area equal to 396,967 square feet. The project consisted of grade separation structures, approach ramps and bridge abutments supported on piles.
The third tallest structure incorporated the Grid-Strip™ with a flexible wire facing element on the Bascom Norris Drive project for Columbia County in Lake City, Florida, in October of 2013. The maximum wall height was 45 feet and the total wall area was equal to 10,000 square feet. The project consisted of a bridge abutment.

5.3.4 **ERS Horizontal Displacement**

Vistawall has not had any experience with large horizontal displacement that required monitoring on any transportation related project. Displacements that have occurred have been within acceptable specified project limits.

5.3.5 **ERS Differential Vertical Displacement**

Vistawall has not had any experience with differential vertical displacement that required monitoring on any transportation related project. Displacements that have occurred have been within acceptable specified project limits.

5.3.6 **ERS Surcharge Loading**

VAWS has used the Grid-Strip™ with several true-abutment applications. The surcharge bearing pressure for the stub-abutment footing ranged from 6 ksf to 8 ksf.

In addition, VAWS has used the Grid-Strip™ soil reinforcing with a flexible wire facing in several mining applications. In one mining application, the surcharge consisted of a P&H 4100XPC Electric Mining Shovel with dual tracks with a gross load of 3,388,000 pounds. Each track consisted of an unfactored bearing equal to 63-psi. In a similar mining application, the surcharge consisted of a Komatsu 930E-4SE Electric Drive Truck with a total gross load of 1,115,000 pounds. The rear axle load was spread over two sets of dual-tires, each with an 87-psi rating.

5.3.7 **ERS Private and Public Users**

Table 7 list the State Department of Transportation’s where the Grid-Strip system has been reviewed and approved.

<table>
<thead>
<tr>
<th>Agency</th>
<th>Date</th>
<th>Contact</th>
<th>Number</th>
<th>Email</th>
</tr>
</thead>
<tbody>
<tr>
<td>Colorado DOT</td>
<td>2013</td>
<td>Trever Wang</td>
<td>303-512-4072</td>
<td><a href="mailto:Trever.Wang@state.co.us">Trever.Wang@state.co.us</a></td>
</tr>
<tr>
<td>Connecticut DOT</td>
<td>8-5-16</td>
<td>Andrew J Mroczkowski</td>
<td>860-594-3296</td>
<td><a href="mailto:andrew.Mroczkowski@ct.gov">andrew.Mroczkowski@ct.gov</a></td>
</tr>
<tr>
<td>Kentucky DOT</td>
<td>5-18-16</td>
<td>Scott Phelps</td>
<td>502-564-3160</td>
<td><a href="mailto:Scott.Phelps@ky.gov">Scott.Phelps@ky.gov</a></td>
</tr>
<tr>
<td>Maryland DOT</td>
<td>2-6-15</td>
<td>Randall Knieriem</td>
<td>410-545-8360</td>
<td><a href="mailto:rknieriem@sha.state.md.us">rknieriem@sha.state.md.us</a></td>
</tr>
</tbody>
</table>
Table 7  Vistawall System Grid-Strip™ State Approvals

<table>
<thead>
<tr>
<th>Agency</th>
<th>Date</th>
<th>Contact</th>
<th>Number</th>
<th>Email</th>
</tr>
</thead>
<tbody>
<tr>
<td>Massachusetts DOT</td>
<td>2013</td>
<td>Tanya M. Barros</td>
<td>857-368-9537</td>
<td><a href="mailto:tanya.barros@state.ma.us">tanya.barros@state.ma.us</a></td>
</tr>
<tr>
<td>New Jersey DOT</td>
<td>2015</td>
<td>Fred Lovett</td>
<td>609-530-5148</td>
<td>fred <a href="mailto:lovett@dot.state.nj.us">lovett@dot.state.nj.us</a></td>
</tr>
<tr>
<td>New York DOT</td>
<td>2016</td>
<td>Christopher C. Nebral</td>
<td>518-457-4717</td>
<td><a href="mailto:chris.nebral@dot.ny.gov">chris.nebral@dot.ny.gov</a></td>
</tr>
<tr>
<td>North Carolina DOT</td>
<td>2012</td>
<td>Scott Hidden</td>
<td>919-250-4088</td>
<td><a href="mailto:shidden@ncdot.gov">shidden@ncdot.gov</a></td>
</tr>
<tr>
<td>Puerto Rico DOT</td>
<td>2013</td>
<td>Carlos Contreras Aponte</td>
<td>787-721-8787</td>
<td><a href="mailto:ccontreras@dtop.pr.gov">ccontreras@dtop.pr.gov</a></td>
</tr>
<tr>
<td>South Carolina</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tennessee DOT</td>
<td>2014</td>
<td>Houston Walker</td>
<td>615-741-5335</td>
<td><a href="mailto:houston.walker@tn.gov">houston.walker@tn.gov</a></td>
</tr>
<tr>
<td>Texas DOT</td>
<td>2011</td>
<td>John Delphia</td>
<td>512-416-2359</td>
<td><a href="mailto:john.delphia@txdot.gov">john.delphia@txdot.gov</a></td>
</tr>
<tr>
<td>Virginia DOT</td>
<td>2013</td>
<td>Kevin Lee</td>
<td>804-371-9862</td>
<td><a href="mailto:kevin.lee@VDOT.Virginia.gov">kevin.lee@VDOT.Virginia.gov</a></td>
</tr>
<tr>
<td>Alberta MOT</td>
<td>2013</td>
<td>John Alexander</td>
<td>780-415-1024</td>
<td><a href="mailto:john.alexander@gov.ab.ca">john.alexander@gov.ab.ca</a></td>
</tr>
<tr>
<td>British Columbia MOT</td>
<td>2013</td>
<td>Crystal Lacher</td>
<td>250-387-7702</td>
<td><a href="mailto:crystal.lacher@gov.bc.ca">crystal.lacher@gov.bc.ca</a></td>
</tr>
<tr>
<td>New Brunswick MOT</td>
<td>2014</td>
<td>Serge Gagnon</td>
<td>506-470-7621</td>
<td><a href="mailto:serge.gagnon@gnb.ca">serge.gagnon@gnb.ca</a></td>
</tr>
<tr>
<td>Newfoundland and Labrador DOT</td>
<td>2014</td>
<td>John Morrissey</td>
<td>709-729-5493</td>
<td><a href="mailto:morrseyj@gov.nl.ca">morrseyj@gov.nl.ca</a></td>
</tr>
<tr>
<td>Nova Scotia MOT</td>
<td>2014</td>
<td>Peter Hackett</td>
<td>902-424-5687</td>
<td><a href="mailto:peter.hackett@novascotia.ca">peter.hackett@novascotia.ca</a></td>
</tr>
<tr>
<td>Ontario MOT</td>
<td>2014</td>
<td>David Staseff</td>
<td>416-235-4073</td>
<td><a href="mailto:david.staseff@ontario.ca">david.staseff@ontario.ca</a></td>
</tr>
<tr>
<td>Prince Edward Island MOT</td>
<td>2014</td>
<td>Darrell Evans</td>
<td>902-569-0578</td>
<td><a href="mailto:djevaans@gove.pe.ca">djevaans@gove.pe.ca</a></td>
</tr>
</tbody>
</table>

Table 8 lists the State Department of Transportation’s where the Grid-Strip system has been submitted and is pending approval.

Table 8  Vistawall System Grid-Strip™ State Approvals (Pending)

<table>
<thead>
<tr>
<th>Agency</th>
<th>Date</th>
<th>Contact</th>
<th>Number</th>
<th>Email</th>
</tr>
</thead>
<tbody>
<tr>
<td>California DOT</td>
<td>2015</td>
<td>Kathryn Griswell</td>
<td>916-227-7330</td>
<td></td>
</tr>
<tr>
<td>Florida DOT</td>
<td>2015</td>
<td>Karen Byram</td>
<td>850-414-4353</td>
<td><a href="mailto:karen.byram@dot.state.fl.us">karen.byram@dot.state.fl.us</a></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Rodrigo Herrera</td>
<td>850-414-4377</td>
<td><a href="mailto:rodrigo.herrera@dot.state.fl.us">rodrigo.herrera@dot.state.fl.us</a></td>
</tr>
<tr>
<td>Indiana DOT</td>
<td>2014</td>
<td>Aamir Turk</td>
<td>317-610-7251</td>
<td><a href="mailto:aturk@indot.IN.gov">aturk@indot.IN.gov</a></td>
</tr>
<tr>
<td>New Hampshire DOT</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ohio DOT</td>
<td>2014</td>
<td>Alexander Dettloff</td>
<td>614-275-1308</td>
<td><a href="mailto:alexander.dettloff@dot.state.oh.us">alexander.dettloff@dot.state.oh.us</a></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Michael McGonagle</td>
<td>717-783-8368</td>
<td><a href="mailto:mmcgonagle@pa.gov">mmcgonagle@pa.gov</a></td>
</tr>
<tr>
<td>Washington DOT</td>
<td>2016</td>
<td>Monique Pawelka</td>
<td>360-705-7754</td>
<td><a href="mailto:pawelkm@wsdot.wa.gov">pawelkm@wsdot.wa.gov</a></td>
</tr>
</tbody>
</table>

6.0  OTHER

6.1  OTHER INFORMATION

No other information is warranted at this time.
Appendix

IDEA
HIGHWAY INNOVATIONS, DEVELOPMENTS, ENHANCEMENTS AND ADVANCEMENTS

GRID-STRIP™
SYSTEM DETAILS
# Appendix Table of Contents

## 1.0 ERS COMPONENTS

### 1.1 FACING UNITS

- 1.1.1 Facing Unit Innovations
- 1.1.2 Facing Unit Types
- 1.1.3 Facing Unit Fabrication
- 1.1.4 Facing Unit Specification
- 1.1.5 Facing Unit Dimensions
- 1.1.6 Facing Unit Compressive Strength
- 1.1.7 Facing Unit Percent Air Entrainment
- 1.1.8 Facing Unit Mix Designs
- 1.1.9 Facing Unit Interface Shear Capacity
- 1.1.10 Facing Unit Alignment Pin and Bearing Pad
- 1.1.11 Facing Unit Joint Filter
- 1.1.12 Facing Unit Aesthetic Options
- 1.1.13 Facing Unit Alignment Requirements for Curves and Corners

### 1.2 Inextensible Reinforcements

- 1.2.1 Soil Reinforcing Innovations
- 1.2.2 Soil Reinforcing Manufacturers
- 1.2.3 Soil Reinforcing Types
- 1.2.4 Soil Reinforcing Properties
- 1.2.5 Soil Reinforcing Corrosion Protection
- 1.2.6 Soil Reinforcing Sacrificial Steel as a Function of Service Life
- 1.2.7 Soil Reinforcing Corrosion Testing
- 1.2.8 Soil Reinforcing Dimensional Tolerances
- 1.2.9 Soil Reinforcing Connection
- 1.2.10 Soil Reinforcing Connection Components
- 1.2.11 Soil Reinforcing Connection Manufacturers
- 1.2.12 Soil Reinforcing Connection Properties
- 1.2.13 Soil Reinforcing Connection Corrosion Protection
- 1.2.14 Soil Reinforcing Connection Sacrificial Steel as a Function of Service Life
- 1.2.15 Soil Reinforcing Connection Corrosion Testing
- 1.2.16 Soil Reinforcing Connection Dimensional Tolerances
- 1.2.17 Soil Reinforcing Connection Strength and Testing
- 1.2.18 Soil Reinforcing Pullout Testing and Results
- 1.2.19 Soil Reinforcing Interface Shear

### 1.3 Other Components
1.3.1 Other component Innovations
1.3.2 Reinforced Soil Properties
1.3.3 ERS Drainage
1.3.4 ERS Coping
1.3.5 ERS Traffic Barrier
1.3.6 ERS Abutments
1.3.7 ERS Slip Joints

2.0 ERS DESIGN

2.1 Design Methodology
2.1.1 ERS Design Innovations
2.1.2 ERS AASHTO LRFD Design Methodology
2.1.3 ERS Proprietary Design Methodologies
2.1.4 ERS Facing Design Requirements
2.1.5 ERS Skewed Soil Reinforcing Design Requirements
2.1.6 ERS Vertical Obstruction Design Requirements
2.1.7 ERS Horizontal Obstruction Design Requirements

2.2 Design Drawings

2.3 Design Examples
2.3.1 Level Backslope with Traffic Live Load Design Example
2.3.2 2:1 Backslope Design Example

3.0 CONSTRUCTION

3.1 Construction Procedures
3.1.1 ERS Construction Innovations
3.1.2 ERS Foundation and Leveling Course Preparation
3.1.3 ERS Construction Tool Requirements
3.1.4 ERS Facing Installation Requirements
3.1.5 ERS Soil Reinforcing Installation Requirements
3.1.6 ERS Facing Alignment Requirements
3.1.7 ERS Reinforced Backfill Placement Requirements
3.1.8 ERS Erosion Prevention Requirements
3.1.9 ERS Installer Requirements
3.1.10 ERS Retained Backfill Placement Requirements
3.1.11 ERS Construction Manual

4.0 QUALITY CONTROL

4.1 Manufacturing
4.1.1 Facing Unit QA/QC
4.1.2 Soil Reinforcing QA/QC
4.1.3 Miscellaneous Component QA/QC

4.2 Construction

4.2.1 Construction QA/QC

5.0 PERFORMANCE

5.1 Warranties and Disclaimers

5.2 Testing

5.3 Performance History

5.3.1 ERS Performance History

5.3.2 ERS Oldest Structures

5.3.3 ERS Tallest Structures

5.3.4 ERS Horizontal Displacement

5.3.5 ERS Differential Vertical Displacement

5.3.6 ERS Surcharge Loading

5.3.7 ERS Private and Public Users

6.0 OTHER

6.1.1 Other Information
1.0
ERS COMPONENTS
1.1 FACING UNITS
1.1.1
Facing Unit Innovations
1.1.2
Facing Unit Types
1.1.3
Facing Unit Fabrication
1.1.4
Facing Unit Specification
The information set forth in this design methodology, including but not limited to all technical and engineering data, figures, tables, designs, drawings, details, procedures and specifications are intended for information pertaining to this project. Every effort has been made to ensure the 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 any and all liability resulting from such use.
# TABLE OF CONTENTS

1. **GENERAL** ................................................................................................................................................ 1

2. **REFERENCES** ........................................................................................................................................... 1
   2.1 **AMERICAN SOCIETY FOR TESTING AND MATERIALS (ASTM)**.............................................................. 1
   2.2 **AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS** .............................................. 1
   2.3 **CRSI - MANUAL OF STANDARD PRACTICE** .................................................................................................... 1
   2.4 **WIRE REINFORCEMENT INSTITUTE - MANUAL OF STANDARD PRACTICE** ....................................................... 1
   2.5 **ACI 318-08 - BUILDING CODES FOR STRUCTURAL CONCRETE** ................................................................. 1

3. **CEMENT** ................................................................................................................................................. 1

4. **TESTING AND INSPECTION** ...................................................................................................................... 2

5. **CASTING** ................................................................................................................................................. 2

6. **CURING** .................................................................................................................................................. 2
   6.1 **HOT WEATHER OR INDOOR** ......................................................................................................................... 2
   6.2 **COLD WEATHER** ........................................................................................................................................ 2

7. **LIFTING DEVICES** ..................................................................................................................................... 2

8. **CONCRETE FINISH** ................................................................................................................................... 2

9. **TOLERANCES** ........................................................................................................................................... 3
   9.1 **PANEL DIMENSIONS** ................................................................................................................................... 3
   9.2 **PANEL SQUARENESS** .................................................................................................................................. 3
   9.3 **PANEL SMOOTHNESS** .................................................................................................................................. 3

10. **CONCRETE COMPRESSIVE STRENGTH** ...................................................................................................... 3
   10.1 **ACCEPTANCE** ........................................................................................................................................ 3
   10.2 **SAMPLING** ............................................................................................................................................ 3

11. **REJECTION** .............................................................................................................................................. 4
   11.1 **MOLDING** .............................................................................................................................................. 4
Idea Submittal
Mechanically Stabilized Earth
Grid-Strip Soil Reinforcing

<table>
<thead>
<tr>
<th>Section</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.2</td>
<td>TEXTURE</td>
<td>4</td>
</tr>
<tr>
<td>11.3</td>
<td>PHYSICAL CHARACTERISTICS</td>
<td>4</td>
</tr>
<tr>
<td>11.4</td>
<td>REPAIR</td>
<td>4</td>
</tr>
<tr>
<td>11.5</td>
<td>MARKING</td>
<td>4</td>
</tr>
<tr>
<td>12</td>
<td>PANEL ACCESSORIES</td>
<td>4</td>
</tr>
<tr>
<td>12.1</td>
<td>PANEL ANCHORS</td>
<td>4</td>
</tr>
<tr>
<td>12.2</td>
<td>STRUCTURAL MEMBERS</td>
<td>4</td>
</tr>
<tr>
<td>12.3</td>
<td>FASTENERS</td>
<td>4</td>
</tr>
<tr>
<td>12.4</td>
<td>PANEL REINFORCEMENT</td>
<td>4</td>
</tr>
<tr>
<td>12.5</td>
<td>GALVANIZING</td>
<td>4</td>
</tr>
<tr>
<td>12.6</td>
<td>EPOXY COATING</td>
<td>5</td>
</tr>
</tbody>
</table>

**APPENDIX A**
QUALITY CONTROL PROGRAM FOR PRECAST PANELS

**VISTAWALL SYSTEMS**

650 JUSTICE LANE
Mansfield, TX 76063

**BIG R BRIDGE**
SEGMENTAL CONCRETE PANEL SPECIFICATION

1 GENERAL

This specification pertains to the casting of the VAWS Segmental Concrete panel Stabilized Earth Wall 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 A82 - Standard Specifications for Steel Wire, Plain, for Concrete Reinforcement
   2.1.3 A123 - Standard Specifications for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products
   2.1.4 A185 - Standard Specifications for Steel Welded Wire Reinforcement, Plain, for Concrete
   2.1.5 A325 - Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength
   2.1.6 A496 - Standard Specifications for Steel Wire Reinforcement, Deformed, for Concrete
   2.1.7 A497 - Standard Specifications for Welded Wire Reinforcement, Deformed, for Concrete
   2.1.8 A525 - Specification for General Requirements for Steel Sheet, Zinc-Coated (Galvanized) by the Hot-Dip
   2.1.9 A510 - Standard Specification for General Requirements for Wire Rods and Coarse Round Wire, Carbon Steel
   2.1.10 A615 - Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement
   2.1.11 A780 - Standard Specification for the Repair of Damaged and Uncoated Areas of Hot-Dip Galvanized Coatings
   2.1.12 A884 - Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Reinforcement

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 that contain chloride shall not be used without approval of the Owner.

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 that are placed a minimum of 6” off of 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 VAWS, 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 in 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 verticals do not exceed ½” (13 mm).

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 days production whichever is less.

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

12.4.1 Welded Wire Reinforcing

All welded wire mesh panel reinforcement shall be in accordance with ASTM A82 - Standard Specification for Steel Wire, Plain, for Concrete Reinforcement and ASTM A185 - Standard Specification for Steel Welded Wire Fabric, Plain, for Concrete Reinforcement

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

12.6 Epoxy Coating

All accessories that require epoxy coating shall be in accordance with ASTM A884 -
APPENDIX A – QUALITY CONTROL PROGRAM FOR PRECAST PANELS

1. LIMITATIONS

Information that is contained in this Appendix and all Vistawall Systems, LLC., 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 VAWS Stabilized Earth Wall (SEW) system, or for any other purpose other than this project without the express written consent of VAWS. 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 VAWS. The Quality Control procedures outlined in this manual have been developed to aid in the design, manufacture, supply of materials, and installation of the VAWS SEW 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 VAWS are unique to the SEW system. They have been designed to aid the Pre-Caster in rapid set up and stripping of the SEW 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 fabric mesh panel reinforcing in a manner 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 to 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 day’s concrete shall be thoroughly checked (using a standardized checklist) to ensure 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**

The concrete will be transported by either Vistawall Systems or by the Owner’s certified mixer trucks 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 ensure 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 have 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 exposed back surface of the precast panel will be floated to remove any remaining high or low surfaces. Prior to concluding the final surface preparations, a final review of the location, alignment and condition of the attachment devices shall occur and as required minor adjustments may be necessary to assure 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 insure 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 that is located within the short-term cure area, so the product can receive the necessary attention. Prior to 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 in accordance with 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 in accordance with the Owner’s concrete specification. After the panel satisfies the quality inspection, which includes 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 a 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 1.** It is the sole intent of Vistawall Systems 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 2.** 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 3.** All sampling will be obtained at the discharge destination point or at the end of the chute.

   **STEP 4.** The sampling and test methods shall be in accordance with the methods as outlined in the Owner’s concrete specification.

   **STEP 5.** 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 6. 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 7. 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 direct guidance of the QC Manager or the QC Control representative. Once the repairs have been completed the products shall be inspected to ensure that the product meets or exceeds the quality requirements. After the product has been repaired and the repair accepted, it shall be marked with green paint adjacent to the yellow paint to signify it is of 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 encountered, 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 scarring or staining of the end of the dunnage that is in contact with the front face of the panel.
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**

Is the local breaking of corners or edges of the concrete with the resulting 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.
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 rejected/culled area of the facility until permanent removal can occur.
1.1.5
Facing Unit Dimensions
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 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 SCRIBED IN THE BACK FACE OF PANEL.

8. THE FRONT FACE EDGE OF ALL PANELS SHALL HAVE A 1/2" X 1/2" CHAMFER (UNLESS NOTED OTHERWISE).

9. PANEL REINFORCING SHALL BE WELDED WIRE FABRIC CONFORMING TO ASTM A480 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 LEFT 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. THE PANEL SQUARENESS AND TOLERANCES OUTLINED IN THE VISTAWALL 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.

PRORIETARY AND CONFIDENTIAL

THIS DRAWING IS SOLE PROPERTY OF BIG R MANUFACTURING, LLC AND CONTAINS PROPRIETARY INFORMATION FOR USE WITH THIS PROJECT ONLY. ANY REPRODUCTION IN PART OR AS A WHOLE WITHOUT WRITTEN PERMISSION IS STRICTLY PROHIBITED.

THE DESIGN CONTAINED HEREIN IS BASED ON INFORMATION SUPPLIED BY OTHERS. BIG R BRIDGE IS CERTIFYING INTERNAL STABILITY OF THE STRUCTURE ONLY. EXTERNAL AND GLOBAL STABILITY REQUIREMENTS ARE THE RESPONSIBILITY OF OTHERS.
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 1/2" 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, curving, 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 scribed in the back face of panel.
8. The front face edge of all panels shall have a 1/2" x 1/2" 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-Archer (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.
14. Panels shall not be removed from the form until a minimum 1200 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.
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 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 1/2” X 1/2” 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 65ksi.
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 LEFT 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. THE PANEL SQUARENESS AND TOLERANCES OUTLINED IN THE VISTAWALL PRECAST PANEL QUALITY CONTROL SPECIFICATION AND WITHIN THE GOVERNING SPECIFICATIONS REQUIREMENTS.
14. PANELS SHALL NOT BE REMOVED 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.

FOR USE WITH THIS PROJECT ONLY. ANY REPRODUCTION IN PART OR AS A WHOLE WITHOUT WRITTEN PERMISSION IS STRICTLY PROHIBITED.

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

DESIGNED BY
CHECKED BY
DRAWN BY
SPN

P.O. Box 1290
Greeley, Colorado 80632-1290
(970) 356-9600
www.bigrbridge.com
6" O.C. EACH SIDE OF ANCHORS (TYP.)

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX.

6" O.C. EACH SIDE OF ANCHORS (TYP.)

4'-6"

ALIGNMENT PIN VOID (TYP.)

5'-0 1/2"

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX.

6" O.C. EACH SIDE OF ANCHORS (TYP.)

4'-6"

ALIGNMENT PIN VOID (TYP.)

5'-0 1/2"

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX.

6" O.C. EACH SIDE OF ANCHORS (TYP.)

4'-6"

ALIGNMENT PIN VOID (TYP.)

5'-0 1/2"

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX.

6" O.C. EACH SIDE OF ANCHORS (TYP.)

4'-6"

ALIGNMENT PIN VOID (TYP.)

5'-0 1/2"

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX.

6" O.C. EACH SIDE OF ANCHORS (TYP.)

4'-6"

ALIGNMENT PIN VOID (TYP.)

5'-0 1/2"

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX.

6" O.C. EACH SIDE OF ANCHORS (TYP.)

4'-6"

ALIGNMENT PIN VOID (TYP.)

5'-0 1/2"

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX.

6" O.C. EACH SIDE OF ANCHORS (TYP.)

4'-6"

ALIGNMENT PIN VOID (TYP.)

5'-0 1/2"

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX.

6" O.C. EACH SIDE OF ANCHORS (TYP.)

4'-6"

ALIGNMENT PIN VOID (TYP.)

5'-0 1/2"

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX.

6" O.C. EACH SIDE OF ANCHORS (TYP.)

4'-6"

ALIGNMENT PIN VOID (TYP.)

5'-0 1/2"

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX.

6" O.C. EACH SIDE OF ANCHORS (TYP.)

4'-6"

ALIGNMENT PIN VOID (TYP.)

5'-0 1/2"

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX.

6" O.C. EACH SIDE OF ANCHORS (TYP.)

4'-6"

ALIGNMENT PIN VOID (TYP.)

5'-0 1/2"

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX.

6" O.C. EACH SIDE OF ANCHORS (TYP.)

4'-6"

ALIGNMENT PIN VOID (TYP.)

5'-0 1/2"

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX.

6" O.C. EACH SIDE OF ANCHORS (TYP.)

4'-6"

ALIGNMENT PIN VOID (TYP.)

5'-0 1/2"

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX.

6" O.C. EACH SIDE OF ANCHORS (TYP.)

4'-6"

ALIGNMENT PIN VOID (TYP.)

5'-0 1/2"

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX.

6" O.C. EACH SIDE OF ANCHORS (TYP.)

4'-6"

ALIGNMENT PIN VOID (TYP.)

5'-0 1/2"

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX.

6" O.C. EACH SIDE OF ANCHORS (TYP.)

4'-6"

ALIGNMENT PIN VOID (TYP.)

5'-0 1/2"

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX.

6" O.C. EACH SIDE OF ANCHORS (TYP.)

4'-6"

ALIGNMENT PIN VOID (TYP.)

5'-0 1/2"

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX.

6" O.C. EACH SIDE OF ANCHORS (TYP.)

4'-6"

ALIGNMENT PIN VOID (TYP.)

5'-0 1/2"

5'-0 1/2"

4'-6"

#4 BARS BOTH WAYS (TYP.)

4 1/2"

MAX. 
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 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 1/2" X 1/2" 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 LEFT ANCHOR SHALL BE DAYTON SUPERIOR P-93 (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 VISTAWALL PRECAST PANEL QUALITY CONTROL SPECIFICATION AND WITHIN THE GOVERNING SPECIFICATIONS REQUIREMENTS.
14. PANELS SHALL NOT BE REMOVED 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.

NOTE: BRIDGE SQUARENESS AND TOLERANCES OUTLINED IN THE VISTAWALL PRECAST PANEL QUALITY CONTROL SPECIFICATION AND WITHIN THE GOVERNING SPECIFICATIONS REQUIREMENTS.
Back Face Standard Rectangular Panel

Section View Rectangular Panel Standard

Top View Standard Rectangular Panel

Profile View

Panel Anchor

Panel Bottom

Panel Top

Panel Right Side

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 1-1/2" 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, curving, 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 scribed in the back face of panel.
8. The front face edge of all panels shall have a 1/2" x 1/2" 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 anchor shall be Dayton Superior P-93 Fleet-Lift 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 VIBRAWALL PRECAST PANEL QUALITY CONTROL SPECIFICATION and within the governing specifications requirements.
14. Panels shall not be removed from the form until a minimum 1,200 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.
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 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 1/2" X 1/2" 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 LEFT 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 VISTAWALL 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.
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 1/2" 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, curving, labeling, finishing, handling, storage and reporting) shall be conducted in accordance with the governing project specifications.
7. All panels shall have the panel name, date of manufacture and lot number scribed in the back face of panel.
8. The front face edge of all panels shall have a 1/2" x 1/2" chamfer (unless noted otherwise).
9. Panel reinforcing shall be a minimum #4 bar (both ways) conforming to ASTM A615 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 smoothness and tolerances outlined in the Vistawall precast panel quality control specification and within the governing specifications requirements.
14. Panels shall not be removed from the form until a minimum 1200 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.

P.O. Box 1290 • Greeley, Colorado 80632-1290 (970) 356-9600 • www.bigrbridge.com
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 1/2" CONCRETE COVER.

5. CONCRETE SHALL HAVE A MINIMUM 28-DAY COMpressive STRENGTH EQUAL TO A 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 CASTERING, 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 1/2" X 1/2" 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 VISTAWALL PRECAST PANEL QUALITY CONTROL SPECIFICATION AND WITHIN THE GOVERNING SPECIFICATIONS REQUIREMENTS.

14. PANELS SHALL NOT BE REMOVED FROM THE FORM UNTIL A MINIMUM 1000 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.
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 1/2" 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 TO BE CONDUCTED IN ACCORDANCE WITH THE GOVERNING PROJECT SPECIFICATION.
7. ALL PANELS SHALL HAVE THE PANEL NAME, DATE OF MANUFACTURE AND LOT NUMBER SCRIBE ON THE BACK FACE OF PANEL.
8. THE FRONT FACE EDGE OF ALL PANELS SHALL HAVE A 1/2" X 1/2" 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 DETAIL AND 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. PANELS SHALL NOT BE REMOVED FROM THE FORM UNTIL A MINIMUM 1500 PSI STRENGTH HAS BEEN ACHIEVED.
14. 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.
1.1.6
Facing Unit Compressive Strength
1.1.7

Facing Unit Percent Air Entrainment
1.1.8
Facing Unit Mix Designs
IDEA Submittal
Mechanically Stabilized Earth
Grid-Strip Soil Reinforcing

[This Page Is Intentionally left Blank]
1.1.9
Facing Unit Interface Shear Capacity
1.1.10
Facing Unit Alignment Pin and Bearing Pad
Alignment Pin
11 1/2" (White)
Unit Wt. = 0.16 lbs

Alignment Pin
10" (Blue)
Unit Wt. = 0.14 lbs
Alignment Pin
11 1/2" (5/8"±1/16" Diameter)
Unit Wt. = 1.02 lbs

Alignment Pin
10" (5/8"±1/16" Diameter)
Unit Wt. = 0.87 lbs

SECTION A-A
1. WHERE THE WALL HEIGHT TRANSITION TO A HEIGHT THAT EXCEEDS 40 FEET 4 BEARING PADS PER PANEL SHALL BE USED AND SHALL BE PLACED 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"

BEARING PAD PLACEMENT ON 5'X5' PANEL
4-BEARING PADS

8-BEARING PADS

NOTE:
1. WHERE THE WALL HEIGHT TRANSITION TO A HEIGHT THAT EXCEEDS 40 FEET 8 BEARING PADS PER PANEL SHALL BE USED AND SHALL BE PLACED SIDE BY SIDE.
2. WALL HEIGHT IS MEASURED FROM TOP OF LEVELING PAD TO TOP OF PANEL.
3. BEARING PAD IS SBR-60 DUROMETER 3/4" x 3" x 6"

BEARING PAD PLACEMENT ON 5'X10' PANEL
1.1.11
Facing Unit Joint Filter
1. Specifications
   1.1. ASTM D4632 - Standard Test Method for Grab Breaking Load and Elongation of Geotextiles
   1.2. ASTM D6241 - Standard Test Method for Static Puncture Strength of Geotextiles and Geotextile-Related Products Using a 50-mm Probe
   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
   1.7. 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. AASHTO M288 Class 2

4. Property Criteria
   The following properties will be tested and adhered to the minimum values provided in the table unless noted otherwise
   4.1. Grab Tensile Strength D4632 250 lbs
   4.2. Grab Elongation D4632 50%
   4.3. Puncture Strength D4833 500 lbs
   4.4. Mullen burst D37872 25 psi
   4.5. Trapezoidal Tear D4533 45 lbs
   4.6. Apparent Opening Size (AOS) (Max) 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.
The filter fabric is attached to back face of the segmental concrete panel using a construction adhesive. The adhesive is used to temporarily hold the fabric in place until the compacted backfill is placed at the back of the panel. The adhesive is a method to temporarily adhere the filter fabric to the back face of the panel and to keep the outside edges from curling, or bunching up. The adhesive is not applied in order to form a water tight seal and therefore does not have to be a continuous bead. Once the backfill is placed and compacted, the horizontal soil pressure will hold the fabric in place.

The adhesive supplied by VAWS is supplied in tubes that contain 29 fluid ounces. The adhesive is applied using a caulking gun. The adhesive should be placed using a 2” x 12” stitch pattern on the back of the panel or the filter fabric. The tip of the adhesive tube should be cut to produce a ½” diameter bead. A bead that is ½” x 2” has a volume of 0.4 cubic inches. Using basic conversion factors, it can be calculated that there are 1.8 cubic inches in one fluid ounce of product. Therefore, a 29 fluid ounce tube of adhesive can supply approximately 130 - ½” x 2” beads of adhesive.

<table>
<thead>
<tr>
<th>Element</th>
<th>Number of ½” x 2” Beads</th>
<th>Number of Tubes</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 x 5 Panel</td>
<td>15</td>
<td>0.12</td>
</tr>
<tr>
<td>5 x 10 Panel</td>
<td>30</td>
<td>0.24</td>
</tr>
<tr>
<td>*Roll filter fabric</td>
<td>360</td>
<td>3.0</td>
</tr>
</tbody>
</table>

*Assumes 360 linear feet of filter fabric per roll and two rows of 1/2” x 2” beads
It is the Wall Installer’s responsibility to apply the adhesive in a manner to not exceed the recommended application guidelines set forth in this document. The Wall Installer shall be responsible for purchasing additional adhesive than the amount supplied by VAWS based on these guidelines.
1.1.12
Facing Unit Aesthetic Options
**Aesthetic Choices**

Vistawall 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 just a sample of what is offered by Vistawall.

<table>
<thead>
<tr>
<th>Ashlar Stone Form Liner</th>
</tr>
</thead>
<tbody>
<tr>
<td>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.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Reverse Rope Form Liner</th>
</tr>
</thead>
<tbody>
<tr>
<td>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.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Fractured Fin</th>
</tr>
</thead>
<tbody>
<tr>
<td>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.</td>
</tr>
<tr>
<td><strong>Raised Vertical Relief</strong></td>
</tr>
<tr>
<td>---------------------------</td>
</tr>
<tr>
<td>The Raised Vertical Relief provides the look of three dimensional columns. The vertical relief can be supplied in varying widths and number of columns per form can be varied.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>Custom Art Work</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>The custom from liners can be prepared to provide murals on wall faces. The wall face can then be painted or stained.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>Brick Form Liner</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>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.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>Vertical Grave Stake</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>The Vertical Grave Stake form liner provides a rough three-dimensional look. The pattern can be modified in several different thickness and relief patterns.</td>
</tr>
<tr>
<td>Smooth Panel Finish</td>
</tr>
<tr>
<td>---------------------</td>
</tr>
<tr>
<td>Smooth panel finishes can be provided. These are cast without any form liner in the steel panel forming system.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Custom Motif</th>
</tr>
</thead>
<tbody>
<tr>
<td>Custom form liners can be fabricated at the designer’s request. The options of custom motifs and logos are endless.</td>
</tr>
</tbody>
</table>
1.1.13
Facing Unit Alignment Requirements
for Curves and Corners
1.2
INEXTENSIBLE REINFORCEMENTS
1.2.1
Soil Reinforcing Innovations
1.2.2
Soil Reinforcing Manufacturers
1.2.3

Soil Reinforcing Types
### Sample Soil Reinforcing Schedule

<table>
<thead>
<tr>
<th>Type</th>
<th>Longitudinal Bar Size</th>
<th>Transverse Bar Size</th>
<th>Longitudinal Bar Spacing</th>
<th>Transverse Bar Spacing</th>
<th>Transverse Bar Width</th>
<th>Number of Longitudinal Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>GS11</td>
<td>W11.0</td>
<td>W11.0</td>
<td>0'-2&quot;</td>
<td>1'-0&quot;</td>
<td>0'-3&quot;</td>
<td>2</td>
</tr>
<tr>
<td>GS15</td>
<td>W15.0</td>
<td>W11.0</td>
<td>0'-2&quot;</td>
<td>1'-0&quot;</td>
<td>0'-3&quot;</td>
<td>2</td>
</tr>
</tbody>
</table>

### Specifications


### Acceptance

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

### Material

The Grid-Strip soil reinforcing shall have a minimum tensile strength equal to 75 ksi and a minimum yield strength equal to 65 ksi in conformance with ASTM A1064.

### Tolerances

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

1. Overall Sheet Width ± 1/2"  
2. Overall Sheet Length ± 1"  
3. Overall Side Overhang ± 1/4"  
4. Overall Transverse Spacing ± 1/2"  
5. Overall Longitudinal Wire Spacing ± 1/2"  
6. Overall Transverse Wire Width ± 1/2"

### BAR Weld Length

The resistance weld shall be a minimum length equal to 1".

### 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.
1. Specifications

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

3. Material
   The Grid-Strip soil reinforcing shall have a minimum tensile strength equal to 75 ksi and a minimum yield strength equal to 65 ksi in conformance with ASTM A1064.

4. Tolerances
   The permissible variation from the dimensions and configuration shown shall be as follows:
   4.1. Overall Sheet Width ± ½”
   4.2. Overall Sheet Length ± 1”
   4.3. Overall Side Overhang ± ¼”
   4.4. Overall Transverse Spacing ± ½”
   4.5. Overall Longitudinal Wire Spacing ± ½”
   4.6. Overall Transverse Wire Width ± ½”

5. BAR Weld Length
   The resistance weld shall be a minimum length equal to 1”.

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. The certification shall be notarized.
1.2.4
Soil Reinforcing Properties
1.2.5
Soil Reinforcing Corrosion Protection
1.2.6
Soil Reinforcing Sacrificial Steel as a Function of Service Life
MSE CALCULATION PROCEDURE

STABILIZED EARTH WALL
SEGMENTAL CONCRETE PANEL

SOIL REINFORCING DURABILITY
GRID-STRIP

PREPARED FOR:
LRFD BINDER GRID-STRIP SUBMITTAL
The following report presents Vistawall System’s design methodology for Mechanically Stabilized Earth retaining structures to determine the durability of the soil reinforcing. The method is consistent with the 7th Edition of the American Association of State Highway and Transportation Officials AASHTO LRFD Bridge Design Specification 2014 and all interims. This methodology is proprietary to VAWS. Reference is made to NCHRP Report 675 LRFD Metal Loss and Service-Life Strength Reduction Factors for Metal-Reinforced Systems.
Durability

The durability of galvanized steel is dependent on the electrochemical 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 a variety of environments. These studies encompass over 60 years of research that have been used to develop a very conservative metal loss rate.

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 be a minimum of 2 oz/ft² or a thickness of 3.4 mils. AASHTO requires that the application of the zinc coating conform to ASTM A 123 Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products. Application of the coating is performed after fabrication of the soil reinforcing including all resistance welding and bending.

The AASHTO metal loss models assume that the 3.4 mil 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 it is buried in soil. Based on this model AASHTO quantifies the rate of degradation of the zinc coating to be equal to:

\[
\text{Zinc loss rate (first two years)} = 0.58 \text{ mil/year}
\]
\[
\text{Zinc loss rate (remaining years)} = 0.16 \text{ mil/year}
\]

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

\[
Y_0 = \frac{3.4 \text{ mil} - 2 \cdot 0.58 \text{ mil/year}}{0.16 \text{ mil/year}} + 2 \cdot \text{years} = 16 \text{years}
\]

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 for 75-year design life and 100-year design life are shown in Equation 2 and Equation 3 respectively.

\[
E_{C75} = (75 \cdot \text{years} - 16 \cdot \text{years}) \cdot 0.47 \frac{\text{mil}}{\text{year}} \cdot 2 = 55.46 \text{mil} = 0.056 \text{in}
\]

Equation 2

\[
E_{C100} = (100 \cdot \text{years} - 16 \cdot \text{years}) \cdot 0.47 \frac{\text{mil}}{\text{year}} \cdot 2 = 78.96 \text{mil} = 0.079 \text{in}
\]

Equation 3

The sacrificial amount of steel is applied to the diameter of the longitudinal soil reinforcing bar to determine the end of design life bar diameter. The end of design life bar diameter 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 will increase the amount of zinc uptake by the metal. 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 measure of these parameters is defined in AASHTO section 11.10.6.4.2a and is given as:

- Chlorides < 100 ppm
- Sulfates < 200 ppm.
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.

<table>
<thead>
<tr>
<th>Aggressiveness</th>
<th>Resistivity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Corrosive</td>
<td>&lt; 700 ohm-cm</td>
</tr>
<tr>
<td>Corrosive</td>
<td>700-2000 ohm-cm</td>
</tr>
<tr>
<td>Moderately Corrosive</td>
<td>2000-5000 ohm-cm</td>
</tr>
<tr>
<td>Mildly Corrosive</td>
<td>5000-10000 ohm-cm</td>
</tr>
<tr>
<td>Non-corrosive</td>
<td>&gt; 10000 ohm-cm</td>
</tr>
</tbody>
</table>

Another important characteristic is the gradation of the select backfill. The select granular backfill material used in the mechanically stabilized earth structure are to 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.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>3”</td>
<td>100</td>
</tr>
<tr>
<td>¾”</td>
<td>70-100</td>
</tr>
<tr>
<td>No. 4</td>
<td>30-100</td>
</tr>
<tr>
<td>No. 40</td>
<td>15-100</td>
</tr>
<tr>
<td>No. 100</td>
<td>5-65</td>
</tr>
<tr>
<td>No. 200</td>
<td>0-15</td>
</tr>
</tbody>
</table>

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 a 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.
The Grid-Strip soil reinforcing is attached to prefabricated segmental concrete panels that are of a fixed size. Because the size of the panel is known the Grid-Strip soil reinforcement is placed in a fixed and consistent spacing in both the horizontal and vertical direction. The spacing of the Grid-Strip 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 Grid-Strip 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.
CLIENT: VAWS-SCP Wall System @ 75 Year Design Life

PROJECT: Mechanically Stabilized Earth Product Submittal - SCP

PROJECT No: VAWS 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

WELDED WIRE MESH STEEL MATERIAL PARAMETERS

Yield Stress of Steel: \( f_y := 65.00 \text{ ksi} \)

Yield Coefficient: \( \theta_y := 0.75 \)

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

Thickness of galvanized coating: \( g_T := 3.4 \text{ mil} \)

Structure design life: \( Y_t := 75 \text{ yr} \)

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

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

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

Design Life of Galvanized Coating:

\[
Y_G := \frac{g_T - 2 \cdot \text{yr} \cdot g_2}{g_R} + 2 \cdot \text{yr} = 16.000 \text{ yr}
\]

Sacrificial Steel:

\[
E_c := \text{if} \left[ Y_t \leq Y_G , 0 , \left( Y_t - Y_G \right) \cdot E_{S75} \cdot 2 \right] = 0.0555 \text{ in}
\]
GS11 Grid-Strip Soil Reinforcing (W11.0 - 2 Wire or D11.0 - 2 Wire)

Wire Diameter (D11/W11) is 0.374 in

\[
d_{11} := \sqrt{\frac{11.0 \text{ in}^2}{100 \pi} \cdot 4 - E_c} = 0.319 \text{ in}
\]

Diameter of W11.0 with effects of corrosion

\[
A_{GS11} := \frac{\pi \cdot d_{11}^2}{4} \cdot 2 = 0.160 \text{ in}^2
\]

Total area of 2-Wire W11.0

Maximum horizontal force of 2-Wire W11.0

\[
T_{max_{GS11}} := \theta_y \cdot f_y \cdot A_{GS11} = 7.782 \text{ kip}
\]

GS15 Grid-Strip Soil Reinforcing (W15.0 - 2 Wire or D15.0 - 2 Wire)

Wire Diameter (D15/W15) is 0.437 in

\[
d_{15} := \sqrt{\frac{15.0 \text{ in}^2}{100 \pi} \cdot 4 - E_c} = 0.382 \text{ in}
\]

Diameter of W15.0 with effects of corrosion

\[
A_{GS15} := \frac{\pi \cdot d_{15}^2}{4} \cdot 2 = 0.229 \text{ in}^2
\]

Total area of 2-Wire W15.0

Maximum horizontal force of 2-Wire W15.0

\[
T_{max_{GS15}} := \theta_y \cdot f_y \cdot A_{GS15} = 11.149 \text{ kip}
\]

GS20 Grid-Strip Soil Reinforcing (W20.0 - 2 Wire or D20.0 - 2 Wire)

Wire Diameter (D20/W20) is 0.504 in

\[
d_{20} := \sqrt{\frac{20.0 \text{ in}^2}{100 \pi} \cdot 4 - E_c} = 0.449 \text{ in}
\]

Diameter of W20.0 with effects of corrosion

\[
A_{GS20} := \frac{\pi \cdot d_{20}^2}{4} \cdot 2 = 0.317 \text{ in}^2
\]

Total area of 2-Wire W20.0

Maximum horizontal force of 2-Wire W20.0

\[
T_{max_{GS20}} := \theta_y \cdot f_y \cdot A_{GS20} = 15.449 \text{ kip}
\]
CLIENT: VAWS-SCP Wall System @ 100 Year Design Life
PROJECT: Mechanically Stabilized Earth Product Submittal - SCP
PROJECT No: VAWS P. N.
CALC BY: TPT DATE: CHECKED BY: DATE:
SUBJECT: Durability of Soil Reinforcing
SPECIFICATION: AASHTO LRFD Bridge Design Specification Article 11.10.6.4.2

WELDED WIRE MESH STEEL MATERIAL PARAMETERS

Yield Stress of Steel................................. $f_y := 65.00 \text{ ksi}$
Yield Coefficient............................... $\theta_y := 0.75$

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

Thickness of galvanized coating ........................................... $g_T := 3.4 \text{ mil}$
Structure design life .................................................... $Y_t := 100 \text{ yr}$
Zinc loss rate (first two years) ........................................... $g_2 := 0.58 \text{ mil/yr}$
Zinc loss rate (remaining years) .......................................... $g_R := 0.16 \text{ mil/yr}$
Steel loss rate (0-75 years) ............................................... $E_{575} := 0.47 \text{ mil/yr}$
Design Life of Galvanized Coating $Y_G := \frac{g_T - 2 \cdot \text{ yr} \cdot g_2}{g_R} + 2 \cdot \text{ yr} = 16.000 \text{ yr}$
Sacrificial Steel: $E_c := \text{ if } Y_t \leq Y_G, 0, \left[ (Y_t - Y_G) \cdot E_{575} \right] \cdot 2 = 0.079 \text{ in}$
GS11 Grid-Strip Soil Reinforcing (W11.0 - 2 Wire or D11.0 - 2 Wire)

Wire Diameter (D11/W11) is 0.374 in

Diameter of W11.0 with effects of corrosion

\[ d_{11} := \sqrt{\frac{11.0 \text{ in}^2}{100 \pi \cdot 4 - E_c}} = 0.295 \text{ in} \]

Total area of 2-Wire W11.0

\[ A_{GS11} := \frac{\pi \cdot d_{11}^2}{4} \cdot 2 = 0.137 \text{ in}^2 \]

Maximum horizontal force of 2-Wire W11.0

\[ T_{\text{max}_{GS11}} := \theta_y \cdot f_y \cdot A_{GS11} = 6.677 \text{ kip} \]

GS15 Grid-Strip Soil Reinforcing (W15.0 - 2 Wire or D15.0 - 2 Wire)

Wire Diameter (D15/W15) is 0.437 in

Diameter of W15.0 with effects of corrosion

\[ d_{15} := \sqrt{\frac{15.0 \text{ in}^2}{100 \pi \cdot 4 - E_c}} = 0.358 \text{ in} \]

Total area of 2-Wire W15.0

\[ A_{GS15} := \frac{\pi \cdot d_{15}^2}{4} \cdot 2 = 0.201 \text{ in}^2 \]

Maximum horizontal force of 2-Wire W15.0

\[ T_{\text{max}_{GS15}} := \theta_y \cdot f_y \cdot A_{GS15} = 9.818 \text{ kip} \]

GS20 Grid-Strip Soil Reinforcing (W20.0 - 2 Wire or D20.0 - 2 Wire)

Wire Diameter (D20/W20) is 0.504 in

Diameter of W20.0 with effects of corrosion

\[ d_{20} := \sqrt{\frac{20.0 \text{ in}^2}{100 \pi \cdot 4 - E_c}} = 0.426 \text{ in} \]

Total area of 2-Wire W20.0

\[ A_{GS20} := \frac{\pi \cdot d_{20}^2}{4} \cdot 2 = 0.285 \text{ in}^2 \]

Maximum horizontal force of 2-Wire W20.0

\[ T_{\text{max}_{GS20}} := \theta_y \cdot f_y \cdot A_{GS20} = 13.875 \text{ kip} \]
1.2.7
Soil Reinforcing Corrosion Testing
1.2.8
Soil Reinforcing Dimensional Tolerances
### SAMPLE SOIL REINFORCING SCHEDULE

<table>
<thead>
<tr>
<th>TYPE</th>
<th>LONGITUDINAL BAR SIZE</th>
<th>TRANSVERSE BAR SIZE</th>
<th>LONGITUDINAL BAR SPACING</th>
<th>TRANSVERSE BAR SPACING</th>
<th>TRANSVERSE BAR WIDTH</th>
<th>NUMBER OF LONGITUDINAL BARS</th>
</tr>
</thead>
<tbody>
<tr>
<td>GS11</td>
<td>W11.0</td>
<td>W11.0</td>
<td>0'-2&quot;</td>
<td>1'-0&quot;</td>
<td>0'-3&quot;</td>
<td>2</td>
</tr>
<tr>
<td>GS15</td>
<td>W15.0</td>
<td>W15.0</td>
<td>0'-2&quot;</td>
<td>1'-0&quot;</td>
<td>0'-3&quot;</td>
<td>2</td>
</tr>
</tbody>
</table>

### Specifications

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

### Acceptance

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

### Material

The Grid-Strip soil reinforcing shall have a minimum tensile strength equal to 75 ksi and a minimum yield strength equal to 65 ksi.

### Tolerances

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

1. **Overall Sheet Width ± ½"**
2. **Overall sheet Length ± 1"**
3. **Overall Side Overhang ± ¼"**
4. **Overall Transverse Spacing ± ½"**
5. **Overall Longitudinal Wire Spacing ± ½"**
6. **Overall Transverse Wire Width ± ½"**

### BAR Weld Length

The resistance weld shall be a minimum length equal to 1".

### 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.
1. **Specifications**

2. **Acceptance**
   All material will be accepted on the basis of the required certification and testing required by the ASTM specifications.

3. **Material**
   The Grid-Strip soil reinforcing shall have a minimum tensile strength equal to 75 ksi and a minimum yield strength equal to 65 ksi.

4. **Tolerances**
   The permissible variation from the dimensions and configuration shown shall be as follows:
   4.1. Overall Sheet Width ± ½ "
   4.2. Overall Sheet Length ± 1"
   4.3. Overall Side Overhang ± ¼"
   4.4. Overall Transverse Spacing ± ½"
   4.5. Overall Longitudinal Wire Spacing ± ½"
   4.6. Overall Transverse Wire Width ± ½"

5. **BAR Weld Length**
   The resistance weld shall be a minimum length equal to 1".

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. The certification shall be notarized.

---

**SAMPLE SOIL REINFORCING SCHEDULE**

<table>
<thead>
<tr>
<th>TYPE</th>
<th>LONGITUDINAL BAR SIZE</th>
<th>TRANSVERSE BAR SIZE</th>
<th>LONGITUDINAL BAR SPACING</th>
<th>TRANSVERSE BAR SPACING</th>
<th>TRANSVERSE BAR WIDTH</th>
<th>NUMBER OF LONGITUDINAL BARS</th>
</tr>
</thead>
<tbody>
<tr>
<td>GS11</td>
<td>W11.0</td>
<td>W11.0</td>
<td>0'-2&quot;</td>
<td>1'-0&quot;</td>
<td>0'-3&quot;</td>
<td>2</td>
</tr>
<tr>
<td>GS15</td>
<td>W15.0</td>
<td>W15.0</td>
<td>0'-2&quot;</td>
<td>1'-0&quot;</td>
<td>0'-3&quot;</td>
<td>2</td>
</tr>
</tbody>
</table>

---

**PLAN VIEW**

**SIDE VIEW**
1.2.9
Soil Reinforcing Connection
IDEA Submittal
Mechanically Stabilized Earth
Grid-Strip Soil Reinforcing

[This Page Is Intentionally left Blank]
1.2.10

Soil Reinforcing Connection Components
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 DPTS 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 ± \( \frac{3}{8} \)
   4.2. Overall Plate Horizontal Dimensions ± \( \frac{3}{16} \)
   4.3. Overall Hole Dimension ± \( \frac{1}{64} \)
   4.4. Overall Edge Distance ± \( \frac{3}{8} \)

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.
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 BAR connector 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 $\pm \frac{1}{32}$
   4.4. Overall Edge Distances $\pm \frac{1}{32}$

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.
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 BAR connector 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 ± \( \frac{1}{8} \)
   4.2. Overall Plate Horizontal Dimensions ± \( \frac{1}{8} \)
   4.3. Overall Hole Dimension ± \( \frac{1}{64} \)
   4.4. Overall Edge Distances ± \( \frac{1}{32} \)

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.
1. Specifications
   1.1. ASTM A29 - Standard Specification for Steel Bars, Carbon and Alloy, Hot-Wrought, General Requirements for
   1.2. ASTM A1040 - Standard Guide for Specifying Harmonized Standard Grade Compositions for Wrought Carbon, Low-Alloy, and Alloy Steels

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

3. Material
   The TAB Connector shall be A1040 steel and have a minimum tensile capacity of 65 ksi.

4. Tolerances
   4.1. Overall Plate Vertical Dimensions ± 1/8
   4.2. Overall Plate Horizontal Dimensions ± 1/8
   4.3. Overall Hole Dimension ± 1/64
   4.4. Overall Edge Distance ± 1/32
   4.5. Overall Shaft Diameter ± 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.
1.2.11
Soil Reinforcing Connection Manufacturers
1.2.12
Soil Reinforcing Connection Properties
IDEA Submittal
Mechanically Stabilized Earth
Grid-Strip Soil Reinforcing

[This Page Is Intentionally left Blank]
1.2.13
Soil Reinforcing Connection Corrosion Protection
1.2.14
Soil Reinforcing Connection Sacrificial Steel
as a Function of Service Life
1.2.15
Soil Reinforcing Connection Corrosion Testing
1.2.16

Soil Reinforcing Connection Dimensional Tolerances
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 DPTS 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 ± 1/8
   4.2. Overall Plate Horizontal Dimensions ± 1/8
   4.3. Overall Hole Dimension ± 1/16
   4.4. Overall Edge Distance ± 1/8

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.
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 BAR connector 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 ± $\frac{1}{8}$
   4.2. Overall Plate Horizontal Dimensions ± $\frac{1}{8}$
   4.3. Overall Hole Dimension ± $\frac{1}{8}$
   4.4. Overall Edge Distance ± $\frac{1}{8}$

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.
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 BAR connector 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 ± 1/8
   4.2. Overall Plate Horizontal Dimensions ± 1/8
   4.3. Overall Hole Dimension ± 1/64
   4.4. Overall Edge Distances ± 1/64

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.
1. Specifications
   1.1. ASTM A29 - Standard Specification for Steel Bars, Carbon and Alloy, Hot-Wrought, General Requirements for
   1.2. ASTM A1040 - Standard Guide for Specifying Harmonized Standard Grade Compositions for Wrought Carbon, Low-Alloy, and Alloy Steels

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

3. Material
   The TAB Connector shall be A1040 steel and have a minimum tensile capacity of 65 ksi.

4. Tolerances
   4.1. Overall Plate Vertical Dimensions ± 1/8
   4.2. Overall Plate Horizontal Dimensions ± 1/8
   4.3. Overall Hole Diameter ± 1/64
   4.4. Overall Edge Distance ± 1/32
   4.5. Overall Shaft Diameter ± 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.

Tab End Connector
Unit Wt. = 0.49 lbs

Fabrication Drawing
1.2.17
Soil Reinforcing Connection Strength and Testing
CONNECTION CALCULATION

VIST-A-WALL SYSTEM

Segmental Concrete Panel with Grid-Strip Soil Reinforcing PennDOT Requirements
MECHANICALLY STABILIZED EARTH STRUCTURES

Grid-Strip™

Connection Design Methodology

Stabilized Earth Wall System

Vistawall Systems
650 Justice Lane
Mansfield, Texas 76063
Phone 817.507.0200 • Fax 817.507.0197

Copyright ©VAWS
ALL RIGHTS RESERVED 2017
PROPRIETARY INFORMATION VISTAWALL SYSTEMS
NO DISTRIBUTION IS ALLOWED WITHOUT WRITTEN PERMISSION FROM VISTAWALL SYSTEMS

The information set forth in this design methodology, including but not limited to all technical and engineering data, figures, tables, designs, drawings, details, procedures and specifications are intended for information pertaining to this project. Every effort has been made to ensure the 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 any and all liability resulting from such use.
# Table of Contents

1. System Description ........................................................................................................................................... 1
   1.1. SEGMENTAL CONCRETE PANEL DESCRIPTION .................................................................................. 1
   1.2. GRID-STRIP™ CONNECTION DESCRIPTION .................................................................................... 4
   1.3. RESISTANCE FACTOR DISCUSSION .................................................................................................... 4

2. Design Life Considerations ............................................................................................................................. 6

3. Grid-Strip™ Soil Reinforcing Description ...................................................................................................... 6

4. Soil Reinforcing Tensile Capacity (A-A) ......................................................................................................... 8

5. Grid-Strip™ Proximal End Connection Descriptions .................................................................................... 9
   5.1. TAB CONNECTOR ........................................................................................................................................ 9
     5.1.1. TAB Shaft Tensile Capacity (B-B) ............................................................................................... 10
     5.1.2. TAB Plate at Bolt Hole (C-C) ........................................................................................................ 12
   5.2. BAR CONNECTOR DESCRIPTION ....................................................................................................... 13
     5.2.1. BAR Terminal End (B-B) ............................................................................................................. 13
     5.2.2. BAR Plate at Bolt Hole (C-C) ........................................................................................................ 15

6. Panel Anchor ................................................................................................................................................... 17
   6.1. DUAL PLATE SHAFT (DPS) ANCHOR .................................................................................................. 17
     6.1.1. DPS Plate At Bolt Hole (D-D) ........................................................................................................ 17
     6.1.2. DPS Shaft At Terminal End (E-E) .................................................................................................. 19
   6.2. DUAL PLATE TIE STRIP (DPTS) ANCHOR .......................................................................................... 20
     6.2.1. DPTS Plate At Bolt Hole (D-D) .................................................................................................... 21
     6.2.2. DPTS Plate At Terminal End (E-E) ............................................................................................... 23

7. Bolt Shear ....................................................................................................................................................... 24

8. Bolt Bearing ..................................................................................................................................................... 25

9. Summary of Tensile Capacity of Connection ............................................................................................... 29

10. Anchor Pullout from Concrete .................................................................................................................... 29
    10.1. BREAKOUT AREA FOR DPS ANCHOR ............................................................................................ 30
      10.1.1. No Edge Limit ............................................................................................................................. 30
      10.1.2. Limited by Edge .......................................................................................................................... 30
    10.2. BREAKOUT AREA FOR DPTS ANCHOR ...................................................................................... 31
      10.2.1. No Edge Limit ............................................................................................................................. 31
      10.2.2. Limited by Edge .......................................................................................................................... 32
    10.3. GENERAL REQUIREMENTS FOR STRENGTH OF ANCHORS .................................................... 33
    10.4. NOMINAL STRENGTH OF DPS ANCHOR ....................................................................................... 33
    10.5. NOMINAL STRENGTH OF DPTS ANCHOR .................................................................................... 34
    10.6. BREAKOUT STRENGTH OF DPS ANCHOR .................................................................................... 34
    10.7. BREAKOUT STRENGTH OF DPTS ANCHOR .................................................................................. 35
    10.8. PULLOUT STRENGTH OF ANCHOR IN TENSION WITH HEAD .................................................. 36
10.9. ANCHOR PULLOUT SUMMARY ................................................................. 37
  10.9.1. DPS Anchor ...................................................................................... 37
  10.9.2. DPTS Anchor .................................................................................... 37

Table of Figures

Figure 1-1  DPS/TAB Grid-Strip™ Connection ............................................................ 1
Figure 1-2  Standard 5 x 5 SCP with 4-DPT Panel Anchors ........................................ 2
Figure 1-3  Standard SCP 5 x 5 Panel Reinforcing with 4-DPTS Panel Anchors ........ 3
Figure 1-4  Standard 5 x 5 with 4 Panel Anchors ...................................................... 4
Figure 1-5  TAB/DPS Calculation Locations ............................................................. 5
Figure 1-6  BAR/DPTS Calculation Locations ......................................................... 5
Figure 3-1  Grid-Strip™ with TAB Anchor - Plan View ............................................ 7
Figure 3-2  Grid-Strip™ with TAB Connector - Side View ....................................... 7
Figure 3-3  Grid-Strip™ with BAR Anchor - Plan View .......................................... 8
Figure 3-4  Grid-Strip™ with BAR Connector - Side View ..................................... 8
Figure 5-1  TAB Connector Dimensions ............................................................... 10
Figure 5-2  TAB Stem Limiting Area ..................................................................... 11
Figure 5-3  BAR Connector Dimensions ............................................................... 14
Figure 6-1  Dual Plate-Shaft Panel Anchor Dimensions ........................................ 17
Figure 6-2  TAB Connector Dimensions ............................................................... 21
Figure 7-1  Bolt Shear ......................................................................................... 24
Figure 9-1  Connection Analysis Locations ........................................................... 29
Figure 10-1  Concrete Breakout DPS ..................................................................... 30
Figure 10-2  Concrete Breakout at Edge for DPTS ................................................ 31
Figure 10-3  Concrete Breakout DPTS ................................................................. 32
Figure 10-4  Concrete Breakout at Edge for DPTS ................................................ 33
Figure 10-5  DPS Anchor ....................................................................................... 36

Table of Tables

Table 1  Corrosion Parameters ............................................................................. 6
Table 2  Grid-Strip Parameters .......................................................................... 7
Table 3  Tensile Summary .................................................................................. 29
Table 4  DPS Anchor Pullout Summary ............................................................... 37
Table 5  DPTS Anchor Pullout Summary .............................................................. 37
1. System Description

The Vist-A-Wall Systems (VAWS) Stabilized Earth Wall (SEW) system utilizes a reinforced segmental concrete panel (SCP) and discrete Grid-Strip™ soil reinforcing element that is mechanically attached to the panel with a bolt set. The SCP is fabricated from concrete and reinforced with steel bars. Soil reinforcement connector embeds are cast into the SCP and extend from the back face. The connector embeds consists of a steel dual plate with a central shaft (DPS) or flat steel dual plates (DPTS). The proximal end of the Grid-Strip™ soil reinforcing consists of a TAB connector or BAR connector each of which are welded to the Grid-Strip™ soil reinforcing and then attached to the panel anchor using a 1/2” diameter A325 bolt set.

![Figure 1-1 DPS/TAB Grid-Strip™ Connection](image)

1.1. Segmental Concrete Panel Description

In the Grid-Strip™ standard MSE design applications there are typically 2 columns of anchors in 2 rows per panel for the 5’ panel and 3 columns of anchors in 2 rows per panel for the 10’ panel. The system has been designed so a maximum of 15 panel anchors and 30 panel anchors can be placed in a single row for the 5’ panel and the 10’ panel, respectively. The number of panel anchors is dependent on the design loading and site obstruction conditions.
The precast panel shall be cast using a minimum 4000 psi compressive strength concrete. However, the project special provisions may require a higher strength. For the analysis contained herein the minimum 4000 psi shall be used and shall be considered a conservative analysis. The panel thickness can vary and is dependent on the application. Typically, the panel thickness is 6”. Nevertheless, it is not uncommon for slightly thinner panels to be designed and used for specific project applications. When a form liner is used the panel thickness will be dependent upon the actual liner type that is used. In all cases the minimum structural thickness of the panel shall be maintained. A thickness of 6” shall be used in the calculations herein.

**Figure 1-2  Standard 5 x 5 SCP with 4-DPT Panel Anchors**

The panel reinforcing consists of vertical and horizontal bars. The bars can be smooth or deformed wire rods or bars. The panel reinforcing is configured so the locations of the panel anchors are contained in a 3.75” x 3.75” mesh opening. The concrete cover over the reinforcing bars is a minimum
of 1-½” or 2” depending on the Owner’s specification for minimum concrete cover. The vertical bars are placed so they are closest to the back face of the panel. The soil reinforcing embedded anchors extend from the back face a distance of approximately 3” and are embedded in the concrete panel a distance of approximately 4”. Specially designed panel anchor holders uniformly position the soil embed anchors.

The segmental concrete panel reinforcing is fabricated to meet one or all of the following ASTM requirements:

- **ASTM A1064** Standard Specification for Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for concrete
- **ASTM A496** Deformed Steel Wire for Concrete Reinforcing.
- **ASTM A497** Welded Deformed Steel Wire Mesh for Concrete Reinforcement.

![Panel Back Face Elevation](image1)

![Panel Cross Section](image2)

![Panel Top Section View](image3)

**Figure 1-3** Standard SCP 5 x 5 Panel Reinforcing with 4-DPTS Panel Anchors
1.2. **Grid-Strip™ Connection Description**

The Grid-Strip™ soil reinforcing is attached to the back of the panel at the location of the DPS or DPTS Anchors by passing the TAB or BAR into the opening defined by the parallel dual plates of the DPS or DPTS anchor. The Grid-Strip™ soil reinforcing is secured from removal by passing a ½” diameter A325 bolt through the central openings. The bolt is secured with the aid of an oversized nut. The bolt shall be placed head side down, in other words, with the nut up.

![TAB Connector](image1.png) ![BAR Connector](image2.png)

*Figure 1-4 Standard 5 x 5 with 4 Panel Anchors*

MSE systems should be certified to demonstrate that they can develop 100% connection strength. To satisfy this requirement the allowable tension of the soil reinforcing must be less than the allowable loads in the connection components. The load path is from the soil reinforcing, to the connection, to the panel anchor, to the SCP. In addition, to confirming the connection strength through calculations, all connections are certified through full scale testing. The soil reinforcing, and anchoring system is checked at five locations as shown in Figure 1-5 and Figure 1-6.

1.3. **Resistance Factor Discussion**

The Grid-Strip™ soil reinforcing end connectors and panel anchor will be designed using the resistance factor that is defined in AASHTO Table 11.5.7-1 and is equal to 0.75. The connection components that include the end component (TAB and BAR) and the panel anchors (DPS and DPTS) could be designed using the resistance factor for tension yielding on the gross section as defined in AASHTO 6.5.4.2 equal to 0.95. Therefore, there is an inherent applied safety factor in the design analysis is contained in this document.
Figure 1-5  TAB/DPS Calculation Locations

Figure 1-6  BAR/DPTS Calculation Locations
2. Design Life Considerations

Steel 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 the steel is buried in the soil. For the design herein, the reinforced backfill shall be assumed to meet the electrochemical properties specified in AASHTO Article 11.10.6.4.2a. The protective coating for the steel will be zinc and 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 1, and the steel loss for the given design life is calculated using Equation 2-1. The value Ec is removed from the diameter of the soil reinforcing bar and the thickness of the plates to calculate the design steel area. Note a 100-year design life will be used in this document and will be considered to be conservative.

\[
E_c = 2 \cdot \left( \text{Life} - \left( \frac{t_{\text{galv}} - 2 \cdot t_{\text{zinc1}}}{t_{\text{zinc2}}} + 2 \right) \right) \cdot t_{\text{steel}}
\]

Equation 2-1

<table>
<thead>
<tr>
<th>Table 1 Corrosion Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Location</strong></td>
</tr>
<tr>
<td>Design Life</td>
</tr>
<tr>
<td>Galvanizing thickness</td>
</tr>
<tr>
<td>Zinc Loss First Two years</td>
</tr>
<tr>
<td>Zinc Loss remaining years</td>
</tr>
<tr>
<td>Steel Loss Rate</td>
</tr>
<tr>
<td>Steel Loss for given design life (per diameter)</td>
</tr>
</tbody>
</table>

Corrosion Calculation:

\[
E_c = 2 \cdot \left[ 100 \text{yr} - \left( \frac{3.4 - 2 \cdot 0.58}{0.16} + 2 \right) \text{yr} \right] \cdot 0.47 = 0.079 \text{ in}
\]

3. Grid-Strip™ Soil Reinforcing Description

The Grid-Strip™ soil reinforcing consists of a discrete strip of welded wire. The strip consists of 2 parallel longitudinal wires that are spaced 2” apart. The longitudinal wires are joined to a series of perpendicular, 3” wide, 12” spaced, cross wires by the method of resistance welding at their intersection. The Grid-Strip™ is fabricated in accordance with ASTM A1064. The Longitudinal wires and
the cross wires are steel wire rods meeting the requirements of ASTM A1064. The wire rods are W11 and have a diameter of 3/8” (0.374) as shown in Table 2. Please note that ASTM A1064 is a combined specification of the former specifications ASTM A185 and ASTM A82. All welds are verified by ASTM A1064 and the Vistawall QA/QC procedure manual.

<table>
<thead>
<tr>
<th>Location</th>
<th>Variable</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wire diameter for W11</td>
<td>$d_{11}$</td>
<td>0.374</td>
<td>in</td>
</tr>
<tr>
<td>Number of wires</td>
<td>$n_w$</td>
<td>2</td>
<td>dim</td>
</tr>
</tbody>
</table>

Each Grid-Strip™ has a proximal end and terminal end. The proximal end of the Grid-Strip™ is defined by a TAB connector or BAR connector that is resistance welded to the longitudinal wires. The terminal end of the Grid-Strip™ consists of a single crosswire. The length of each Grid-Strip™ is dependent on the project design requirements. The Grid-Strip™ is hot-dip galvanized as specified in Section 1.1.
4. **Soil Reinforcing Tensile Capacity (A-A)**

The steel area of the Grid-Strip™ is a function of the soil reinforcing wire diameter and the steel loss calculated in Section 1.1. There are 2 longitudinal soil reinforcing wires per Grid-Strip™. The gross area is calculated using Equation 4-1. At this location the net area and gross area are equal.

\[
A_g = \frac{\pi (d_{11} - E_c)^2}{4} \cdot n_w
\]

Equation 4-1

Where:

- \(d_{11}\) = diameter of W11 wire (in) \[0.375\]
- \(n_w\) = number of longitudinal wires for Grid-Strip (dim) \[2\]
- \(E_c\) = sacrificial steel (in) \[0.079\]
- \(A_g\) = gross cross-sectional area of member (in²)
- \(A_n\) = net area of the member (in²) – Article 6.8.3
Area Calculations at Design Life Equal to 100 Years:

\[ Ag = \left[ 3.142 \times (0.374 - 0.079)^2 / 4 \right] \times 2 = 0.137\text{in}^2 \]

\[ An = 0.137\text{in}^2 \]

The soil reinforcing tensile capacity is checked for both yield and fracture. The factored tensile resistance, \( P_r \), shall be equal to the smallest magnitude calculated using Equation 4-2 and Equation 4-3.

\[ P_r = \phi_y \cdot F_y \cdot A_g \quad \text{AASHTO 6.8.2.1-1} \]
\[ P_r = \phi_u \cdot F_u \cdot A_n \cdot R_p \cdot U \quad \text{AASHTO 6.8.2.1-2} \]

Where:

- \( \phi_y \) = resistance factor yielding of tension member AASHTO Table 11.5.7-1 (dim) \( 0.75 \)
- \( \phi_u \) = resistance factor for fracture of tension member AASHTO 6.5.4.2 (dim) \( 0.80 \)
- \( F_y \) = specified minimum yield strength (ksi) \( 65.00 \)
- \( F_u \) = specified tensile strength (ksi) \( 80.00 \)
- \( R_p \) = reduction factor for punched full size holes – Article 6.8.2.1 \( 1.00 \)
- \( U \) = reduction factor to account for shear lag – Article 6.8.2.1 \( 1.00 \)
- \( P_{ry} \) = factored tensile resistance for yield (kip)
- \( P_{ru} \) = factored tensile resistance for fracture (kip)

Note the gross area (\( A_g \)) and the net area (\( A_n \)) are equal.

Tension Calculations:

\[ P_{ry} = 0.75 \times 65.00\text{ksi} \times 0.1367\text{in}^2 = 6.666 \text{kips} \]
\[ P_{ru} = 0.80 \times 80.00\text{ksi} \times 0.1367\text{in}^2 \times 1.00 \times 1.00 = 8.751 \text{kips} \]

5. Grid-Strip™ Proximal End Connection Descriptions

5.1. TAB Connector

The TAB connector is forged from steel meeting the requirements of ASTM 1040. Each TAB anchor is forged with a 3/8” thick x 1 ½” wide plate that converges to a 5/8” diameter circular shaft. The shaft has a series of serrations forged into each side that are used to seat, align, and fuse the proximal
end of the Grid-Strip™ longitudinal wires. The longitudinal wires of the Grid-Strip™ are resistance welded to the shaft using a specialized welder that was exclusively designed by VAWS for this application. The weld strength is verified using destructive testing as specified in the VAWS QA/QC manual. The resistance weld is approximately 2” long.

The limiting design area for the TAB connector occurs when the longitudinal wires do not nest completely in the serrations at location B-B shown in Figure 5-1. The TAB is considered to be part of the soil reinforcing system. Therefore, cross section B-B and C-C will use the yield and fracture resistance factors for the soil reinforcing as shown in AASHTO Table 11.5.7-1 and AASHTO 6.5.4.2 respectively.

![Figure 5-1 TAB Connector Dimensions](image)

### 5.1.1. TAB Shaft Tensile Capacity (B-B)

The section of the stem at B-B is required to satisfy both yield and fracture tensile requirements as shown in Equation 5-1 and Equation 5-2. The area of the stem is determined using the 3D model that was created in the software program Pro-E and converted to AutoCAD 3D as shown in Figure 5-2. The gross cross-sectional area of the stem at the controlling section at a 100-year design life duration, including the effects of corrosion, is equal to 0.1664 square inches.
Figure 5-2  TAB Stem Limiting Area

\[ P_r = \phi_y \cdot F_y \cdot A_g \]  
\[ P_r = \phi_u \cdot F_u \cdot A_n \cdot R_p \cdot U \]

AASHTO 6.8.2.1-1  
AASHTO 6.8.2.1-2  

Equation 5-1  
Equation 5-2

Where:

- \( \phi_y \) = resistance factor for yielding of tension member AASHTO 11.5.7-1 (dim)  
  - 0.75
- \( \phi_u \) = resistance factor for fracture of tension member AASHTO 6.5.4.2 (dim)  
  - 0.80
- \( F_y \) = specified minimum yield strength ASTM 1040 steel (ksi)  
  - 60.00
- \( F_u \) = specified tensile strength ASTM 1040 steel (ksi)  
  - 90.00
- \( A_g \) = gross cross-sectional area of member [user input] (in²)  
  - 0.1664
- \( A_n \) = net area of the member [Equal to \( A_g \)] (in²) – Article 6.8.3  
  - 0.1664
- \( R_p \) = reduction factor for punched full size holes – Article 6.8.2.2  
  - 1.00
- \( U \) = reduction factor to account for shear lag  
  - 1.00
- \( P_{ry} \) = factored tensile resistance for yield (kip)
- \( P_{ru} \) = factored tensile resistance for fracture (kip)

Note:

1. The gross area (\( A_g \)) and the net area (\( A_n \)) are equal.
2. Resistance factor for yielding in a AASHTO 6.5.4.2 is equal to 0.95

Tension Calculations:

\[ P_{ry} = 0.75 \times 60.00 \text{ksi} \times 0.1664 \text{in}^2 = 7.488 \text{ kips} \]

\[ P_{ru} = 0.80 \times 90.00 \text{ksi} \times 0.1664 \text{in}^2 \times 1.00 \times 1.00 = 11.981 \text{ kips} \]
5.1.2. TAB Plate at Bolt Hole (C-C)

The TAB lead end consists of the forged steel plate and drilled central bolt hole. The lead end is attached to the panel anchor that protrudes from the back of the SCP with a 1/2”-diameter A325 bolt set. The lead end plate is 3/8” thick and has a width equal to 1 1/2”. The bolt hole is drilled 1/16” larger making the hole diameter size equal to 9/16”. The section C-C will use the yield and fracture resistance factors for the connection as shown in AASHTO Table 11.5.7-1 and AASHTO 6.5.4.2 respectively.

\[ A_g = W_p \cdot (t_p - E_c) \]  \hspace{1cm} \text{Equation 5-3}

\[ A_n = \left[ W_p - d_b - \left( \frac{1}{16} \cdot \text{in} \right) \right] \cdot (t_p - E_c) \]  \hspace{1cm} \text{Equation 5-4}

Where:

- \( W_p \) = width of plate (in)
- \( t_p \) = thickness of plate (in)
- \( d_b \) = nominal Diameter of bolt (in)
- \( E_c \) = sacrificial steel (in)
- \( A_g \) = gross cross-sectional area of member (in\(^2\))
- \( A_n \) = net area of the member (in\(^2\)) – Article 6.8.3

Area Calculations:

\[ Ag = 1.500\text{in} \times (0.375\text{in} - 0.079\text{in}) = 0.4441 \text{in}^2 \]

\[ An = (1.50\text{in} - 0.5000\text{in} - (1/16)\text{in}) \times (0.3750\text{in} - 0.079\text{in}) = 0.2775 \text{in}^2 \]

\[ P_r = \phi_y \cdot F_y \cdot A_g \]  \hspace{1cm} \text{AASHTO 6.8.2.1-1}  \hspace{1cm} \text{Equation 5-5}

\[ P_r = \phi_u \cdot F_u \cdot A_n \cdot R_p \cdot U \]  \hspace{1cm} \text{AASHTO 6.8.2.1-2}  \hspace{1cm} \text{Equation 5-6}

Where:

- \( \phi_y \) = resistance factor yielding of tension member AASHTO Table 11.5.7-1 (dim)
- \( \phi_u \) = resistance factor for fracture of tension member AASHTO 6.5.4.2 (dim)
F_y = specified minimum yield strength ASTM 1040 steel (ksi)  
F_u = specified tensile strength ASTM 1040 steel (ksi)  
R_p = reduction factor for drilled full size holes – Article 6.8.2.1  
U = reduction factor to account for shear lag – Article 6.8.2.1  
P_{ry} = factored tensile resistance for yield (kip)  
P_{ru} = factored tensile resistance for fracture (kip)  

Note:  
1. The gross area (Ag) and the net area (An) are equal.  
2. Resistance factor for yielding in a AASHTO 6.5.4.2 is equal to 0.95

**Tension Calculations:**

\[ P_{ry} = 0.75 \times 60.00 \text{ksi} \times 0.4441 \text{in}^2 = 19.983 \text{ kips} \]
\[ P_{ru} = 0.80 \times 90.00 \text{ksi} \times 0.2775 \text{in}^2 \times 1.00 \times 1.00 = 19.983 \text{ kips} \]

5.2. **BAR Connector Description**

The BAR connector is a hot rolled steel plate meeting the requirements of ASTM A572 Grade 50. Each BAR anchor is fabricated from plate material with a minimum 0.250” thickness and a width equal to 1-1/2”. The lead end of the plate has a drilled ½” diameter hole to accommodate a bolt set. The terminal end of the plate has a series of serrations that are machined into each side. The serrations are used to seat, align, and fuse to the proximal end of the Grid-Strip™ longitudinal wires during the welding process. The longitudinal wires of the Grid-Strip™ are resistance welded to the terminal end using a specialized welder that was exclusively designed by VAWS for this application. The weld strength is verified using destructive testing as specified in the VAWS QA/QC manual. The resistance weld is approximately 2” long.

5.2.1. **BAR Terminal End (B-B)**

The limiting design area for the BAR connector is when the longitudinal wires do not nest completely in the serrations at location B-B shown in Figure 5-3 and has a width equal to 1-3/8” (1.375”). The BAR is part of the soil reinforcing system. The BAR plate end at section C-C that attaches to the DPS anchor or the DPTS anchor (Cross Section D-D). The anchors are considered part of the connection and the BAR part of the soil reinforcing. Therefore, section B-B and section C-C will use the
yield and fracture resistance factors for the soil reinforcing as shown in AASHTO Table 11.5.7-1 and AASHTO 6.5.4.2 respectively.

\[ A_g = W_b \cdot (t_b - E_c) \]  
\text{Equation 5-7}

Where:

- \( W_b \) = width of bar at limiting section (in)  
  \[ 1.375 \]

- \( t_b \) = thickness of bar (in)  
  \[ 0.250 \]

- \( E_c \) = sacrificial steel (in)  
  \[ 0.079 \]

- \( A_g \) = gross cross-sectional area of member [user input] (in²)

- \( A_n \) = net area of the member [Equal to \( A_g \)] (in²) – Article 6.8.3

\[ A_g = 1.375 \text{in} \times (0.250 \text{in} - 0.079 \text{in}) = 0.2352 \text{in}^2 \]

\[ A_n = 0.2352 \text{in}^2 \]

\[ P_r = \phi_y \cdot F_y \cdot A_g \]  
\text{AASHTO 6.8.2.1-1}  
\text{Equation 5-8}
Connection Design Methodology

tpt (10/22/17)

\[
P_r = \phi_y \cdot F_y \cdot A_n \cdot R_p \cdot U \quad \text{AASHTO 6.8.2.1-2} \quad \text{Equation 5-9}
\]

Where:

\[
\phi_y = \text{resistance factor yielding of tension member AASHTO Table 11.5.7-1 (dim)} \quad 0.75
\]

\[
\phi_u = \text{resistance factor for fracture of tension member AASHTO 6.5.4.2 (dim)} \quad 0.80
\]

\[
F_y = \text{specified minimum yield strength ASTM A572 Grade 50 steel (ksi)} \quad 50.00
\]

\[
F_u = \text{specified tensile strength ASTM A572 Grade 50 steel (ksi)} \quad 65.00
\]

\[
R_p = \text{reduction factor for punched full size holes – Article 6.8.2.2} \quad 1.00
\]

\[
U = \text{reduction factor to account for shear lag} \quad 1.00
\]

\[
P_{r_y} = \text{factored tensile resistance for yield (kip)}
\]

\[
P_{r_u} = \text{factored tensile resistance for fracture (kip)}
\]

Note:

1. The gross area \((A_g)\) and the net area \((A_n)\) are equal.
2. Resistance factor for yielding in a AASHTO 6.5.4.2 is equal to 0.95. The 0.75 resistance factor should be considered conservative.

**Tension Calculation:**

\[
P_{r_y} = 0.75 \times 50.00\text{ksi} \times 0.2352\text{in}^2 = 8.819 \text{kips}
\]

\[
P_{r_u} = 0.80 \times 65.00\text{ksi} \times 0.2352\text{in}^2 \times 1.00 \times 1.00 = 12.229 \text{kips}
\]

**5.2.2. BAR Plate at Bolt Hole (C-C)**

The BAR connector lead end consists of the steel plate with a drilled bolt hole. The lead end is attached to the panel anchor that protrudes from the back of the SCP with a 1/2”-diameter A325 bolt set. The lead end plate is 1/4” thick x 1 1/2” wide. The bolt hole is drilled 1/16” larger making the hole diameter sized to 9/16”. The section C-C will use the yield and fracture resistance factors for the soil reinforcing as shown in AASHTO Table 11.5.7-1 and AASHTO 6.5.4.2 respectively.

\[
A_g = W_p \cdot (t_p - E_c) \quad \text{Equation 5-10}
\]

\[
A_n = \left[ W_p - d_b - \left( \frac{1}{16} \text{in} \right) \right] \cdot (t_p - E_c) \quad \text{Equation 5-11}
\]
Where:

\( W_p \) = width of plate (in) 
\( t_p \) = thickness of plate (in) 
\( d_b \) = nominal diameter of bolt (in) 
\( E_c \) = sacrificial steel (in) 
\( A_g \) = gross cross-sectional area of member (in\(^2\)) 
\( A_n \) = net area of the member (in\(^2\)) – Article 6.8.3

**Area Calculations:**

\[
A_g = 1.500\text{in} \times (0.250\text{in} - 0.079\text{in}) = 0.2566 \text{in}^2
\]

\[
A_n = (1.50\text{in} - 0.5000\text{in} - (1/16)\text{in}) \times (0.2500\text{in} - 0.079\text{in}) = 0.1604 \text{in}^2
\]

\[
P_y = \phi_y \cdot F_y \cdot A_g
\]

\[
P_y = \phi_y \cdot F_y \cdot A_n \cdot R_p \cdot U
\]

Where:

\( \phi_y \) = resistance factor yielding of tension member AASHTO Table 11.5.7-1 (dim) 
\( \phi_y \) = resistance factor for fracture of tension member AASHTO 6.5.4.2 (dim) 
\( F_y \) = specified minimum yield strength ASTM A572 steel Grade 50 (ksi) 
\( F_u \) = specified tensile strength ASTM A572 steel Grade 50 (ksi) 
\( R_p \) = reduction factor for drilled full size holes – Article 6.8.2.2 
\( U \) = reduction factor to account for shear lag 
\( P_{ry} \) = factored tensile resistance for yield (kip) 
\( P_{ru} \) = factored tensile resistance for fracture (kip)

Note:

1. The gross area (\( A_g \)) and the net area (\( A_n \)) are equal.
2. Resistance factor for yielding in a AASHTO 6.5.4.2 is equal to 0.95. The 0.75 resistance factor should be considered conservative.

**Tension Calculations:**

\[
P_{ry} = 0.75 \times 50.00\text{ksi} \times 0.2566\text{in}^2 = 9.621 \text{kips}
\]

\[
P_{ru} = 0.80 \times 65.00\text{ksi} \times 0.1604\text{in}^2 \times 1.00 \times 1.00 = 8.338 \text{kips}
\]

6. **Panel Anchor**

6.1. **Dual Plate Shaft (DPS) Anchor**

The Dual Plate-Shaft (DPS) anchor is forged from steel meeting the requirements of ASTM 1040. The DPS anchor consists of parallel dual plates that converge into a circular shaft (Figure 6-1). The shaft terminates into a 1-3/4” circular disk. The overall length of the DPS anchor is 6-7/8”. Each individual dual plate has a diameter of 1-1/2” with a 9/16” centrally located drilled bolt hole. The dual plates are parallel and offset from one another by a distance of 13/16”. Each plate has a thickness of 5/16”. The circular shaft is 5/8” in diameter and contains a series of serrations along its length that are embedded into the SCP. The shaft terminates into a circular disk with a diameter of 1-3/4” that is 1/4” thick.

[Diagram of Dual Plate Shaft Panel Anchor Dimensions]

6.1.1. **DPS Plate at Bolt Hole (D-D)**

The DPS lead end consists of dual plates each with a drilled bolt hole. The lead end is attached to the soil reinforcing TAB with a 1/2”-diameter A325 bolt set. The plates are 5/16” thick x 1-1/2” wide.
Bolt hole is drilled 1/16” larger making the hole diameter sized to 9/16”. The section D-D will use the yield and fracture resistance factors for the connection as shown in AASHTO Table 11.5.7-1 and AASHTO 6.5.4.2 respectively.

\[ A_g = W_p \cdot (t_p - E_c) \]  
\[ A_n = \left[ W_p - d_b - \left(\frac{1}{16}\right)\cdot\text{in}\right] \cdot (t_p - E_c) \]

Where:

\[ W_p = \text{width of plate (in)} \]  
\[ t_p = \text{thickness of plate (in)} \]  
\[ d_b = \text{nominal diameter of bolt (in)} \]  
\[ E_c = \text{sacrificial steel (in)} \]  
\[ A_g = \text{gross cross-sectional area of member [user input] (in}^2\text{)} \]  
\[ A_n = \text{net area of the member [Equal to } A_g\text{] (in}^2\text{)} - \text{Article 6.8.3} \]

**Area Calculations:**

\[ Ag = 1.500\text{in} \times (0.313\text{in} - 0.079\text{in}) = 0.3503 \text{in}^2 \]

\[ An = (1.50\text{in} - 0.5000\text{in} - (1/16)\text{in}) \times (0.3125\text{in} - 0.079\text{in}) = 0.2189 \text{in}^2 \]

\[ P_r = \phi_v \cdot F_y \cdot A_g \quad \text{AASHTO 6.8.2.1-1} \]
\[ P_f = \phi_u \cdot F_u \cdot A_n \cdot R_p \cdot U \quad \text{AASHTO 6.8.2.1-2} \]

Where:

\[ \phi_v = \text{resistance factor yielding of tension member AASHTO Table 11.5.7-1 (dim)} \]  
\[ \phi_u = \text{resistance factor for fracture of tension member AASHTO 6.5.4.2 (dim)} \]  
\[ F_y = \text{specified minimum yield strength ASTM 1040 steel (ksi)} \]  
\[ F_u = \text{specified tensile strength ASTM 1040 steel (ksi)} \]
\[ \frac{1}{2} \pi (d_s - E_c)^2 \]

Where:

- \( d_s \) = diameter of DPS shaft (in)
- \( E_c \) = sacrificial steel (in)
- \( A_g \) = gross cross-sectional area of the shaft (in^2)
- \( A_n \) = net area of the shaft (equal to gross area) (in^2)

Area Calculations:

\[ A_g = [3.142 \times (0.625 - 0.079)^2 / 4] = 0.234 \text{in}^2 \]
An = 0.234in^2

\[ P_y = \phi_y \cdot F_y \cdot A_g \]
AASHTO 6.8.2.1-1  
Equation 6-6

\[ P_y = \phi_y \cdot F_u \cdot A_n \cdot R_p \cdot U \]
AASHTO 6.8.2.1-2  
Equation 6-7

Where:

\[ \phi_y = \text{resistance factor yielding of tension member AASHTO Table 11.5.7-1 (dim)} \]

\[ \phi_u = \text{resistance factor for fracture of tension member AASHTO 6.5.4.2 (dim)} \]

\[ F_y = \text{specified minimum yield strength ASTM 1040 steel (ksi)} \]

\[ F_u = \text{specified tensile strength ASTM 1040 steel (ksi)} \]

\[ R_p = \text{reduction factor for punched full size holes – Article 6.8.2.2} \]

\[ U = \text{reduction factor to account for shear lag} \]

\[ P_{ry} = \text{factored tensile resistance for yield (kip)} \]

\[ P_{ru} = \text{factored tensile resistance for fracture (kip)} \]

Note:

1. The gross area \((A_g)\) and the net area \((A_n)\) are equal.
2. Resistance factor for yielding in a AASHTO 6.5.4.2 is equal to 0.95. The 0.75 resistance factor should be considered conservative.

**Tension Calculations:**

\[ P_{ry} = 0.75 \times 60.00\text{ksi} \times 0.2342\text{in}^2 \times 10.538 \text{kips} \]

\[ P_{ru} = 0.80 \times 90.00\text{ksi} \times 0.2342\text{in}^2 \times 1.00 \times 1.00 = 16.861 \text{kips} \]

### 6.2. Dual Plate Tie Strip (DPTS) Anchor

The Dual Plate Tie Strip (DPTS) anchor is fabricated from Grade-50 steel meeting the requirements of ASTM 1011. The DPTS anchor consists of parallel dual plates that converge into a triangular section. The overall length of the DPS anchor is 6-3/8”. Each individual dual plate has a length of 2” and a thickness of 0.135” with 9/16” centrally located bolt holes. The dual plates are parallel and offset from one another by a distance of 1/2”. This is a common panel anchor shape used by several MSE wall suppliers and has been successfully used since 1980.
6.2.1. DPTS Plate at Bolt Hole (D-D)

The DPTS lead end consists of dual plates each with a drilled bolt hole. The lead end is attached to the soil reinforcing TAB or BAR with a 1/2"-diameter A325 bolt set. The plates are 0.135" thick x 2" wide. The bolt hole is punched 1/16" larger making the hole diameter sized to 9/16". The section D-D will use the yield and fracture resistance factors for the connection as shown in AASHTO Table 11.5.7-1 and AASHTO 6.5.4.2 respectively.

\[
A_g = W_p \cdot (t_p - E_c) \quad \text{Equation 6-8}
\]

\[
A_n = \left[ W_p - d_b - \left( \frac{1}{16} \cdot \text{in} \right) \right] \cdot (t_p - E_c) \quad \text{Equation 6-9}
\]

Where:

\[
W_p = \text{width of plate (in)}
\]

2.000
Connection Design Methodology

tpt (10/22/17)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>t_p</td>
<td>thickness of plate (in)</td>
<td>0.135</td>
</tr>
<tr>
<td>d_b</td>
<td>nominal diameter of bolt (in)</td>
<td>0.500</td>
</tr>
<tr>
<td>E_c</td>
<td>sacrificial steel (in)</td>
<td>0.079</td>
</tr>
</tbody>
</table>

A_g = gross cross-sectional area of member [user input] (in²)

A_n = net area of the member [Equal to A_g] (in²) — Article 6.8.3

**Area Calculations:**

\[
A_g = 2.000\text{in} \times (0.135\text{in} - 0.079\text{in}) = 0.1121\text{in}^2
\]

\[
A_n = (2.00\text{in} - 0.500\text{in} - (1/16)\text{in}) \times (0.1350\text{in} - 0.079\text{in}) = 0.0806\text{in}^2
\]

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

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

Where:

\[
\phi_y = \text{resistance factor yielding of tension member AASHTO Table 11.5.7-1 (dim)} \quad 0.75
\]

\[
\phi_u = \text{resistance factor for fracture of tension member AASHTO 6.5.4.2 (dim)} \quad 0.80
\]

\[
F_y = \text{specified minimum yield strength ASTM 1011 Grade-50 steel (ksi)} \quad 50.00
\]

\[
F_u = \text{specified tensile strength ASTM 1011 Grade-50 steel (ksi)} \quad 65.00
\]

\[
R_p = \text{reduction factor for punched full size holes – Article 6.8.2.2} \quad 0.90
\]

\[
U = \text{reduction factor to account for shear lag} \quad 1.00
\]

\[
P_{ry} = \text{factored tensile resistance for yield (kip)}
\]

\[
P_{ru} = \text{factored tensile resistance for fracture (kip)}
\]

**Note:**

1. The gross area (Ag) and the net area (An) are equal.
2. Resistance factor for yielding in a AASHTO 6.5.4.2 is equal to 0.95. The 0.75 resistance factor should be considered conservative.
3. There are two plates in the DPS therefore the equation is multiplied by 2
Connection Design Methodology

Tension Calculations:

\[
P_{\text{Ry}} = 0.75 \times 50.00 \text{ksi} \times 0.1121 \text{in}^2 \times 2 = 8.406 \text{kips}
\]

\[
P_{\text{Ru}} = 0.80 \times 65.00 \text{ksi} \times 0.0806 \text{in}^2 \times 0.90 \times 1.00 \times 2 = 7.540 \text{kips}
\]

6.2.2. DPTS Plate at Terminal End (E-E)

The DPTS terminal plate end consists of a flat plate that is 2” wide and 0.135” thick. The DPTS is embed into the SCP. The section E-E will use the yield and fracture resistance factors for the connection as shown in AASHTO Table 11.5.7-1 and AASHTO 6.5.4.2 respectively.

\[A_g = W_p \cdot (t_p - E_c)\]  
\text{Equation 6-12}

Where:

\(W_p\) = width of the plate (in)  
\(t_p\) = thickness of the plate (in)  
\(E_c\) = sacrificial steel (in)  
\(A_g\) = gross cross-sectional area of the shaft (in\(^2\))  
\(A_n\) = net area of the shaft (equal to gross area) (in\(^2\))

Area Calculations:

\[A_g = 2.000 \text{in} \times (0.135 \text{in} - 0.079 \text{in}) = 0.1121 \text{in}^2\]

\[A_n = 0.1121 \text{in}^2\]

\[P_r = \phi_y \cdot F_y \cdot A_g\]  
\text{AASHTO 6.8.2.1-1}  
\text{Equation 6-13}

\[P_r = \phi_u \cdot F_u \cdot A_n \cdot R_y \cdot U\]  
\text{AASHTO 6.8.2.1-2}  
\text{Equation 6-14}

Where:

\(\phi_y\) = resistance factor yielding of tension member AASHTO Table 11.5.7-1 (dim)  
\(\phi_u\) = resistance factor for fracture of tension member AASHTO 6.5.4.2 (dim)  
\(F_y\) = specified minimum yield strength ASTM 1011 steel (ksi)

\(\phi_y = 0.75\)

\(\phi_u = 0.80\)

\(F_y = 50.00 \text{ksi}\)
F_u = specified tensile strength ASTM 1011 steel (ksi)  
\[ F_u = 65.00 \]

R_p = reduction factor for drilled full size holes – Article 6.8.2.1 (No Hole)  
\[ R_p = 1.00 \]

U = reduction factor to account for shear lag  
\[ U = 1.00 \]

P_{ry} = factored tensile resistance for yield (kip)  

P_{ru} = factored tensile resistance for fracture (kip)  

Note:
1. The gross area (A_g) and the net area (A_n) are equal.
2. Resistance factor for yielding in a AASHTO 6.5.4.2 is equal to 0.95
3. There are two plates in the DPS therefore the equation is multiplied by 2

Tension Calculations:
\[ P_{ry} = 0.75 * 50.00 \text{ksi} * 0.1121 \text{in}^2 * 2 = 8.406 \text{ kips} \]
\[ P_{ru} = 0.80 * 65.00 \text{ksi} * 0.1121 \text{in}^2 * 1.00 * 1.00 * 2 = 11.656 \text{ kips} \]

7. Bolt Shear

The bolt that is used to connect the plate to the panel anchor is an A325 \( \frac{1}{2} \) diameter bolt. The hole in the plate is assumed to be drilled oversized by \( 1/16'' \). The configuration of the connection places the bolt in double shear.

\[ R_n = 0.38 \cdot A_b \cdot F_{ub} \cdot N_i \quad \text{AASHTO 6.13.2.7-1} \]  
Equation 7-1

\[ A_b = \pi \cdot \left( \frac{d_b - E_c}{2} \right)^2 \quad \text{Equation 7-2} \]

\[ R_{allow} = \phi_s \cdot R_n \quad \text{Equation 7-3} \]
Where:

\[ \begin{align*}
R_n & = \text{nominal shear resistance (kip)} \\
A_b & = \text{area of the bolt with effects of corrosion (in}^2) \\
\phi_s & = \text{resistance factor for bolt shear AASHTO 6.5.} \\
F_u & = \text{specified tensile strength of A325 bolt (ksi)} \\
N_s & = \text{number of shear planes (dim)} \\
d_b & = \text{diameter of bolt (in)} \\
E_c & = \text{sacrificial steel (in)}
\end{align*} \]

\[ \begin{align*}
0.80 \\
120 \\
2 \\
0.500 \\
0.079
\end{align*} \]

**Area Calculations:**

\[ Ab = \frac{3.142 \times (0.500 \text{in} - 0.079 \text{in})^2}{4} = 0.139 \text{ in}^2 \]

**Shear Calculations:**

\[ \begin{align*}
R_n & = 0.38 \times 0.139 \text{in}^2 \times 120.00 \text{ksi} \times 2 = 12.698 \text{ kips} \\
R_{allow} & = 0.80 \times 12.70 \text{ kips} = 10.16 \text{ kips}
\end{align*} \]

8. **Bolt Bearing**

The resistance factor for bolt bearing is given in AASHTO 6.5.4.2.

\[ \phi_{bb} = \text{bolts bearing on material AASHTO 6.5.4.2} \]

8.1. **TAB Connector**

The bolt hole in the TAB connector is located a distance equal to 0.4688 inches from the leading edge of the TAB. The bolt bearing may control design and is checked. The nominal bearing resistance, \( R_n \), of the soil reinforcing lead end shall be checked in conformance with AASHTO Article 6.13. The effective bearing area of the bolt shall be taken as its edge distance \( L_c \) multiplied by the thickness of the connected part it bears on including the effects of corrosion.

\[ \begin{align*}
R_n & = 1.2 \times L_c \times (t - E_c) \times F_u \\
R_{allow} & = \phi_{bb} \times R_n
\end{align*} \]

**AASHTO 6.13.2.9-2 Equation 8-1**

**Equation 8-2**
Where:

- **t** = thickness of TAB plate (in)  
  - 0.375

- **Ec** = steel loss (in)  
  - 0.079

- **Lc** = distance from edge of bolt to edge of tab (in)  
  - 0.4688

- **Fu** = tensile strength of ASTM 1040 steel (ksi)  
  - 90.00

- **Rn** = nominal resistance (kip)

- **φbb** = bolts bearing on material AASHTO 6.5.4.2

- **Rallow** = allowable bolt bearing (kip)

### Bolt Bearing Calculation:

\[
R_n = 1.20 \times 0.4688\text{in} \times (0.3750\text{in} - 0.0790\text{in}) \times 90.0\text{ksi} = 14.99 \text{kips}
\]

\[
R_{allow} = 0.80 \times 14.99 \text{kips} = 11.99 \text{kips}
\]

#### 8.2. BAR Connector

The bolt hole in the BAR is located at distance equal to 0.7188 inches from the leading edge of the BAR. The bolt bearing may control design and is checked. The nominal bearing resistance, \( R_n \), of the soil reinforcing lead end shall be checked in conformance with AASHTO Article 6.13. The effective bearing area of the bolt shall be taken as its edge distance (\( L_c \)) multiplied by the thickness of the connected part it bears on including the effects of corrosion.

\[
R_n = 1.2 \cdot L_c \cdot (t - E_c) \cdot F_u \quad \text{AASHTO 6.13.2.9-2 Equation 8-3}
\]

\[
R_{allow} = \phi_{bb} \cdot R_n \quad \text{Equation 8-4}
\]

Where:

- **t** = thickness of BAR plate (in)  
  - 0.250

- **Ec** = steel loss (in)  
  - 0.079

- **Lc** = distance from edge of bolt to edge of tab (in)  
  - 0.7188
F_u = tensile strength of ASTM A572 Grade 50 steel (ksi)  
\[ 65.00 \]

R_n = nominal resistance (kip)  

\[ \phi_{bb} \] = bolts bearing on material AASHTO 6.5.4.2  

R_{allow} = allowable bolt bearing (kip)  

**Bolt Bearing Calculation:**  
\[ R_n = 1.20 \times 0.7188 \text{in} \times (0.2500 \text{in} - 0.0790 \text{in}) \times 65.0 \text{ksi} = 9.59 \text{kips} \]

\[ R_{allow} = 0.80 \times 9.59 \text{kips} = 7.67 \text{kips} \]

### 8.3. DPS Connector

The bolt hole in the DPS is located a distance equal to 0.4688 inches from the leading edge of the plate. The bolt bearing may control design and is checked. The nominal bearing resistance, R_n, of the DPS connection plate will be checked in conformance with AASHTO Article 6.13. The effective bearing area of the bolt shall be taken as its edge distance (L_c) multiplied by the thickness of the connected part it bears on including the effects of corrosion. The value is doubled since there are two plates.

\[ R_n = 1.2 \times L_c \times (t - E_c) \times F_u \]

\[ R_{allow} = \phi_{bb} \times R_n \]

Where:

\[ t = \text{thickness of DPS plate (in)} \quad 0.375 \]

\[ E_c = \text{steel loss (in)} \quad 0.079 \]

\[ L_c = \text{distance from edge of bolt hole to edge of plate (in)} \quad 0.4688 \]

\[ F_u = \text{tensile strength of ASTM 1040 steel (ksi)} \quad 90.00 \]

\[ R_n = \text{nominal resistance (kip)} \]

\[ \phi_{bb} = \text{bolts bearing on material AASHTO 6.5.4.2} \]

\[ R_{allow} = \text{allowable bolt bearing (kip)} \]
Bolt Bearing Calculation:

\[ R_n = 2 \times 1.20 \times 0.4688\text{in} \times (0.3750\text{in} - 0.0790\text{in}) \times 90.0\text{ksi} = 29.98 \text{ kips} \]

\[ R_{allow} = 0.80 \times 29.98 \text{ kips} = 23.98 \text{ kips} \]

Note:
1. There are two plates in the DPS therefore the equation is multiplied by 2

8.4. DPTS Connector

The bolt hole in the DPTS is located a distance equal to 0.7188 inches from the leading edge of the plate. The bolt bearing may control design and is checked. The nominal bearing resistance, \( R_n \), of the DPTS connection plate will be checked in conformance with AASHTO Article 6.13. The effective bearing area of the bolt shall be taken as its edge distance \( L_c \) multiplied by the thickness of the connected part it bears on including the effects of corrosion. The value is doubled since there are two plates.

\[ R_n = 1.2 \cdot L_c \cdot (t - E_c) \cdot F_u \quad \text{AASHTO 6.13.2.9-2} \quad \text{Equation 8-7} \]

\[ R_{allow} = \phi_{bb} \cdot R_n \quad \text{Equation 8-8} \]

Where:

- \( t \) = thickness of DPS plate (in)  
  - 0.3125
- \( E_c \) = steel loss (in)  
  - 0.079
- \( L_c \) = distance from edge of bolt hole to edge of plate (in)  
  - 0.7188
- \( F_u \) = specified tensile strength ASTM 1011 steel Grade 50 (ksi)  
  - 50.00
- \( R_n \) = nominal resistance (kip)
- \( \phi_{bb} \) = bolts bearing on material AASHTO 6.5.4.2
- \( R_{allow} \) = allowable bolt bearing (kip)

Bolt Bearing Calculation:

\[ R_n = 2 \times 1.20 \times 0.7188\text{in} \times (0.3125\text{in} - 0.0790\text{in}) \times 50.0\text{ksi} = 20.14 \text{ kips} \]

\[ R_{allow} = 0.80 \times 20.14 \text{ kips} = 16.12 \text{ kips} \]

Note:
1. There are two plates in the DPS therefore the equation is multiplied by 2
9. Summary of Tensile Capacity of Connection

<table>
<thead>
<tr>
<th>Component</th>
<th>Yield</th>
<th>Fracture</th>
<th>Bearing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Reinforcing (A-A)</td>
<td>6.666 kips</td>
<td>8.751 kips</td>
<td>N.A.</td>
</tr>
<tr>
<td>TAB Section (B-B)</td>
<td>7.488 kips</td>
<td>11.981 kips</td>
<td>N.A.</td>
</tr>
<tr>
<td>TAB Section (C-C)</td>
<td>19.983 kips</td>
<td>19.983 kips</td>
<td>11.99 kips</td>
</tr>
<tr>
<td>BAR Section (B-B)</td>
<td>8.819 kips</td>
<td>12.229 kips</td>
<td>N.A.</td>
</tr>
<tr>
<td>BAR Section (C-C)</td>
<td>9.621 kips</td>
<td>8.338 kips</td>
<td>7.672 kips</td>
</tr>
<tr>
<td>DPS Section (D-D)</td>
<td>31.528 kips</td>
<td>31.528 kips</td>
<td>23.982 kips</td>
</tr>
<tr>
<td>DPS Section (E-E)</td>
<td>10.538 kips</td>
<td>16.861 kips</td>
<td>N.A.</td>
</tr>
<tr>
<td>DPTS Section (D-D)</td>
<td>8.406 kips</td>
<td>7.540 kips</td>
<td>16.115 kips</td>
</tr>
<tr>
<td>DPTS Section (E-E)</td>
<td>8.406 kips</td>
<td>11.656 kips</td>
<td>N.A.</td>
</tr>
</tbody>
</table>

10. Anchor Pullout from Concrete

The panel anchor will be checked for pullout in conformance with the American Concrete Institute Building Code Requirements for Structural Concrete (ACI-318-11) Appendix D – Anchoring to Concrete. The compressive strength of the concrete will be conservatively assumed to be equal to 4000 psi and the yield strength of the reinforcing will be assumed to be equal to 60 ksi.

\[ f'_c = \text{compressive strength of concrete (psi)} \]
\[ f_y = \text{yield strength of steel (ksi)} \]
10.1. Breakout Area for DPS Anchor

10.1.1. No Edge Limit

The DPS anchor will break out of the concrete through a prismatic volume as shown in Figure 10-1 and is in conformance with ACI Figure RD.5.2.1a. The area of the prism is calculated using Equation 10-1.

\[
A_{Nco} = (2 \times 1.5 \cdot h_t) \cdot (2 \times 1.5 \cdot h_t) = 9 \cdot h_{ef}^2
\]

ACI Fig RD.5.2.1a

Equation 10-1

Calculated Breakout Area for DPS:

\[
A_{Nco} = 9 \times (3.625\text{in})^2 = 118.266 \text{ in}^2
\]

10.1.2. Limited by Edge

The minimum edge distance for the Grid-Strip panel anchor is 3.75”. The breakout area is shown in Figure 10-4 and the area is calculated using Equation 10-4. Note that the edge distance is greater than 1.5\( h_{ef} \) and will not control.
Where:

\[ C_{a1} = \text{minimum edge distance for anchor (in)} \]

\[ A_{nc} = (C_{a1} + 1.5 \cdot h_{ef}) \cdot (2 \cdot 1.5 \cdot h_{ef}) \quad \text{ACI Fig RD.5.2.1b} \]

Equation 10-2

**Calculated Breakout Area for DPS:**

\[ A_{nc} = (7.750 \text{in} + 1.5 \cdot 3.625 \text{in}) \cdot (2 \cdot 1.5 \cdot 3.625 \text{in}) = 143.414 \text{ in}^2 \]

10.2. **Breakout Area for DPTS Anchor**

10.2.1. **No Edge Limit**

The DPTS anchor will break out of the concrete through a prismatic volume as shown in Figure 10-3 and is in conformance with ACI Figure RD.5.2.1a. The area of the prism is calculated using Equation 10-3.
Where:

\[ h_{ef} = \text{effective embedment depth of anchor (in)} \]

\[ A_{Nco} = (2 \cdot 1.5 \cdot h_{ef}) \cdot (2 \cdot 1.5 \cdot h_{ef}) = 9 \cdot h_{ef}^2 \]

ACI Fig RD.5.2.1a Equation 10-3

Calculated Breakout Area for DPS:

\[ A_{Nco} = 9 \cdot (4.000\text{in})^2 = 144.000 \text{ in}^2 \]

10.2.2. Limited by Edge

The minimum edge distance for the Grid-Strip panel anchor is 7.75\". The breakout area is shown in Figure 10-4 and the area is calculated using Equation 10-4. Note that the edge distance is greater than 1.5\(h_{ef}\) and will not control.
Where:

\[ C_{a1} = \text{minimum edge distance for anchor (in)} \]

\[ A_{nc} = (C_{a1} + 1.5 \cdot h_{ef}) \cdot (2 \cdot 1.5 \cdot h_{ef}) \quad \text{ACI Fig RD.5.2.1b} \]

**Calculated Breakout Area for DPS:**

\[ \text{Anc} = (7.500\text{in} + 1.5 \cdot 4.000\text{in}) \cdot (2 \cdot 1.5 \cdot 4.000\text{in}) = 162.000 \text{in}^2 \]

### 10.3. General Requirements for Strength of Anchors

Strength Reduction Factors are in conformance with ACI section D4.4a is governed by the strength of a ductile steel element.

\[ \phi_t = \text{Tension loads for ductile steel element (dim)} \]

\[ \phi_s = \text{Shear loads for ductile steel element (dim)} \]

### 10.4. Nominal Strength of DPS Anchor

The nominal concrete strength for a single anchor in shear is equal to

\[ N_{sa} = A_{se,N} \cdot f_{uta} \quad \text{ACI D-3} \]

\[ f_{uta} = 1.9 \cdot f_{su} \leq 125 \text{ ksi} \quad \text{ACI D-3} \]
Where:

\( A_{es,N} \) = cross sectional area of the anchor in tension (in\(^2\))

\( f_{uta} \) = ultimate strength of the anchor (ksi)

\( f_{ya} \) = yield strength of the anchor (ksi)

\( N_{sa} \) = nominal strength of the anchor (ksi)

**Calculated Nominal Strength of DPS Anchor:**

\[ f_{uta} = 1.9 \times 60.00 \text{ksi} = 114.00 \text{ ksi} \]

\[ N_{sa} = 0.234 \text{in}^2 \times 114.00 \text{ksi} = 26.676 \text{ kips} \]

**10.5. Nominal Strength of DPTS Anchor**

The nominal concrete strength for a single anchor in shear is equal to

\[ N_{sa} = A_{se,N} \cdot f_{uta} \]  
**ACI D-3 Equation 10-7**

\[ f_{uta} = 1.9 \cdot f_{ya} \leq 125 \text{ ksi} \]  
**ACI D-3 Equation 10-8**

Where:

\( A_{es,N} \) = cross sectional area of the anchor in tension (in\(^2\))

\( f_{uta} \) = ultimate strength of the anchor (ksi)

\( f_{ya} \) = yield strength of the anchor (ksi)

\( N_{sa} \) = nominal strength of the anchor (ksi)

**Calculated Nominal Strength of DPTS Anchor:**

\[ f_{uta} = 1.9 \times 50.00 \text{ksi} = 95.00 \text{ ksi} \]

\[ N_{sa} = 2 \times 0.112 \text{in}^2 \times 95.00 \text{ksi} = 21.280 \text{ kips} \]

Note: there are 2 plates that comprise anchor

**10.6. Breakout Strength of DPS Anchor**

The nominal concrete breakout strength for a single anchor in tension is equal to

\[ N_b = k_c \cdot f_{c'} \cdot \sqrt{f_{c'}^* \cdot h_{ef}^{1.5}} \]  
**ACI D-6 Equation 10-9**
Connection Design Methodology

\[ N_{cb} = \phi_s \cdot \frac{N_c}{A_{Nco}} \cdot \psi_{ed,N} \cdot \psi_{c,N} \cdot \psi_{cp,N} \cdot N_b \]  
ACI D-3 \hspace{1cm} \text{Equation 10-10}

Where:

- \( k_c \) = breakout factor for cast-in anchors (dim)  
  \( k_c = 24 \)

- \( \lambda_a \) = modification factor for lightweight concrete (dim)  
  \( \lambda_a = 1.00 \)

- \( N_b \) = basic concrete breakout strength in tension (kip)  
  \( N_b = 10.476 \text{ kip} \)

- \( A_{Nc} \) = projected concrete failure area of single anchor (in\(^2\))  
  \( A_{Nc} = 143.414 \)

- \( A_{Nco} \) = projected concrete failure area of single anchor (in\(^2\))  
  \( A_{Nco} = 118.266 \)

- \( \psi_{ed,N} \) = edge factor (dim)  
  \( \psi_{ed,N} = 1.00 \)

- \( \psi_{c,N} \) = crack factor (dim)  
  \( \psi_{c,N} = 1.00 \)

- \( \psi_{cp,N} \) = post installed anchor factor (dim)  
  \( \psi_{cp,N} = 1.25 \)

**Calculated Breakout Strength of DPS Anchor:**

\[ N_b = 24.00 \cdot 1.00 \cdot \sqrt{4000 \text{psi}} \cdot (3.625 \text{in})^{1.5} = 10.476 \text{ kip} \]

\[ N_{cb} = 0.80 \cdot 1.00 \cdot 1.00 \cdot 1.25 \cdot 10.476 \text{kip} = 10.476 \text{ kip} \]

### 10.7. Breakout Strength of DPTS Anchor

The nominal concrete breakout strength for a single anchor in tension is equal to

\[ N_b = k_c \cdot \lambda_a \cdot \sqrt{f'_{c} \cdot h_{ef}^{1.5}} \]  
ACI D-6 \hspace{1cm} \text{Equation 10-11}

\[ N_{cb} = \phi_s \cdot \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ed,N} \cdot \psi_{c,N} \cdot \psi_{cp,N} \cdot N_b \]  
ACI D-3 \hspace{1cm} \text{Equation 10-12}

Where:

- \( k_c \) = breakout factor for cast-in anchors (dim)  
  \( k_c = 24 \)

- \( \lambda_a \) = modification factor for lightweight concrete (dim)  
  \( \lambda_a = 1.00 \)

- \( N_b \) = basic concrete breakout strength in tension (kip)  
  \( N_b = 143.414 \)

- \( A_{Nc} \) = projected concrete failure area of single anchor (in\(^2\))  
  \( A_{Nc} = 143.414 \)
\[ A_{Nco} = \text{projected concrete failure area of single anchor (in}^2) \]

\( \psi_{\text{ed},N} = \text{edge factor (dim)} \)

\( \psi_{c,N} = \text{crack factor (dim)} \)

\( \psi_{\text{cp},N} = \text{post installed anchor factor (dim)} \)

**Calculated Breakout Strength of DPTS Anchor:**

\[ Nb = 24.00 \times 1.00 \times \sqrt{4000\text{psi}} \times (4.000\text{in})^{1.5} = 12.143 \text{ kip} \]

\[ N_{cb} = 0.80 \times 1.00 \times 1.00 \times 1.25 \times 12.143\text{kip} = 12.143 \text{ kip} \]

**10.8. Pullout Strength of Anchor in Tension with Head**

The pullout strength in tension of a single headed stud or bolt shall not exceed Equation 10-13

\[ N_p = 8 \times A_{\text{brg}} \times f_c \]

ACI D-14

Equation 10-13

The DPS anchor shaft terminates into a circular head. The shaft head has a diameter equal to 1.750" as shown in Figure 10-5. The head area is equal to 2.405 square inches.

Where:

\[ d_{\text{head}} = \text{diameter of DPS shaft head (in)} \]

\[ A_{\text{brg}} = \text{area of DPS shaft head (in}^2) \]

\[ N_p = \text{Nominal pullout strength of anchor (kip)} \]

**Calculated Pullout Strength of Headed DPS Anchor:**

\[ N_p = 8 \times 2.405\text{in}^2 \times 4.0\text{ksi} = 76.969 \text{ kip} \]

\[ \phi_{N_p} = 0.80 \times 76.969\text{kip} = 61.575 \text{ kip} \]
10.9. Anchor Pullout Summary

10.9.1. DPS Anchor

Table 4 DPS Anchor Pullout Summary

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal Strength</td>
<td>26.676 kips</td>
</tr>
<tr>
<td>Breakout Strength</td>
<td>10.476 kip</td>
</tr>
<tr>
<td>Pullout Strength</td>
<td>61.575 kip</td>
</tr>
</tbody>
</table>

10.9.2. DPTS Anchor

Table 5 DPTS Anchor Pullout Summary

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal Strength</td>
<td>21.280 kips</td>
</tr>
<tr>
<td>Breakout Strength</td>
<td>12.143 kip</td>
</tr>
</tbody>
</table>
May 12, 2011

Mr. Thomas P. Taylor, P.E.
T&B Structural Systems, Inc.
6800 Manhattan Blvd. Suite 304
Ft. Worth, TX  76120

Subject: Summary of TAB Connector Pullout Test Results
T&B Structural Systems
GTX Project: 1578

Dear Mr. Taylor:

GeoTesting Express, Inc. (GTX) has completed a series of six connection tests on T&B Structural’s Stabilized Earth Wall (SEW) System composed of the dual plate shaft (DPS) anchor and TAB connector. The SEW system was confined in gravel and with application of three different overburden pressures to determine the effect of confinement pressure on the system load capacity in accordance with the test program requested by T&B Structural Systems, Inc. This letter provides the details of the testing program and a summary of the test results.

The SEW system evaluated in this series of tests was a DPS/TAB system having a narrow 2-wire soil reinforcing (SR) element called a Grid-Strip™. The Grid-Strip™ element is fabricated such that the longitudinal wires are spaced at 2 in. and with 3 in. wide transverse wires spaced 6 in. apart and resistance welded at their intersection with the longitudinal wires. The lead end longitudinal wires of the Grid-Strip™ are resistance welded to the forged steel TAB connector. The TAB connector consists of a 1 ½ in by 3/8 in. plate section that terminates into a 5/8 in. shaft. The shaft of the TAB connector is forged with raised indentations in order to increase the weld shear of the resistance weld. The plate section has a 9/16 in. diameter bolt hole fabricated into it. The segmental concrete panel is cast with at least one DPS anchor protruding from the back face. The DPS anchor is a forged steel shape that has parallel flat plates extending 1 ½ in. by 3/8 in. and which intersect at a 5/8 in. diameter shaft that terminates into a 3/8 in. x 1 ¾ in. diameter disc. The dual plates of the DPS anchor each have a 9/16 in. bolt hole that are aligned. The Grid-Strip™ TAB connector is attached to the DPS anchor by passing it between the parallel plates so the bolt holes line up and secure with a ½ in. diameter A325 bolt. The DPS/TAB SEW system is designed to be able to rotate in the horizontal plane and deflect in the vertical plane.

Confined (i.e., in soil) static tensile connection tests were conducted in the GTX stabilization box as shown in Figure 1. The connection test involved slowly applying an axial tension pullout force to the anchor at the rear of the SR Grid-Strip™ which extended out of the box. The section of reinforcement with transverse wires is isolated from the soil using a sleeve to prevent interaction with the soil, thus eliminating friction on the reinforcement strip. The sleeve is spaced to allow
the reinforcement to freely move within the sleeve. Displacement measurements were made using LVDTs to provide continuous readings of displacement in the direction of the applied load during tensile testing of the connection.

Three overburden pressures were applied to the confined connection and referred to as low, mid, and high for the purpose of discussion. Low overburden pressure corresponds to an average of 140 psf (21.9 kN/m²) and attained by placement of a 1-ft-thick (300 mm) compacted layer of gravel onto the connection, sleeve, and SR element between the sleeve and the connection. Mid overburden pressure of 775 psf (122 kN/m²) constituted placement of the 1-ft thick (300 mm) layer of compacted gravel followed by stacking dead weight on top of the gravel layer. High overburden pressure corresponding to 1,525 psf (240 kN/m²) was attained by stacking additional dead weight on top of the gravel layer.

Two 20-ton (180 kN) pneumatic/air-over-oil jacks with a steel reaction beam are used to apply the pullout force. The jacks are interconnected to provide simultaneous loading. The SR Grid-Strip™ specimens are gripped in a steel clamp that is attached to the pullout piston using swivel connections to allow for an evenly distributed load in the case of grid elements that are not exactly straight. The tensile force is measured by a calibrated load cell attached to the pullout piston. A photo of the clamp and tensile loading system is shown in Figure 2. The test setup with applied overburden pressure is shown in Figure 3. The horizontal displacement at the rear of the metallic grid is monitored outside the box by two LVDTs (one on each side) mounted to the longitudinal bars. Wire extensometers are used to monitor movements of the connection and the embedded portion of the TAB and Grid-Strip™ elements. These gages consist of wire in a metal tube to prevent friction on the wire and are connected to an LVDT on one end and to an embedded point on the connection or on the grid on the other end. Locations of the extensometers are shown in Figures 4 and 5. In addition, LVDTs are positioned on the outside of the front face of the segmental concrete panel to monitor possible movement of the panel in the direction of the applied tensile load.

The aggregate used in the tests was poorly graded gravel with sand (GP) and classified as A-1-a based on gradation analysis and according to the AASHTO system. Modified Proctor compaction tests (ASTM D1557) performed on the gravel found a maximum density of 135 pcf (21.3 kN/m³) at an optimal moisture content of 6.4% and with application of an oversize correction factor the maximum dry density was 148 pcf (23.2 kN/m³) at 3.9% optimal moisture. For the connection tests, the gravel was compacted to approximately 95% of the oversize corrected maximum dry density value with moisture at optimum or slightly greater.
Figure 1. Schematic diagram of the big-box connection test set-up.
Figure 2. Photos of DPS anchor and TAB connection system.
Figure 3. Schematic diagram of instrumented TAB and SR Grid-Strip™ element.
Figure 4. Photos of DPS/TAB connections and Grid-Strip™ elements.
Figure 5. Photos of clamp and instrumentation (a) plan view and (b) profile view prior to load cell connection.
Figure 6. Photos of connection tests with high overburden pressure sequentially conducted on a panel (a) left hand side and (b) right hand side.
The results of the SEW DPS anchor / TAB connector system with three different overburden pressures used to confine the connection are shown in the Table 1. The peak load shown is the maximum axial tension load applied to the connection system. Movement corresponds to displacement measured at the rear of the grid-strip. The peak tensile load values measured during testing range from 18.5 to 21.2 kips and tend to increase somewhat with confining stress. Also shown in Table 1 is the load at 19 mm of movement but every test, with one exception, did not exhibit this amount of deformation. According to AASHTO, the limiting deformation target value is 19 mm for pullout tests however this is a serviceability limit state and not a strength limit state as required for evaluating connection capacity. Design load evaluation at the connector should be evaluated based on at least a 35% reduction in the Service I load combination limit state, i.e., a 35% reduction in the allowable load at the connector.

For comparative purposes, T&B Structural provided the following calculations for the maximum design tension load for a 75-year design life for the 2 wire metallic Grid-Strip™ using LRFD and a resistance factor 0.75.

\[
T_{\text{max}_{75\text{ year}}} = n \cdot A_c \cdot F_y \cdot \theta_y \\
= n \cdot \pi \cdot \frac{d^2}{4} \cdot F_y \cdot \theta_y \\
= 2 \cdot \frac{\pi \cdot [(0.375 - 0.56)(in)]^2}{4} \cdot 65(ksi) \cdot 0.75 \\
= 7.789(kip)
\]

The ultimate tensile strength of all the DPS anchor / TAB connection system scenarios tested exceeds the LRFD values provided by T&B Structural. For a 35% reduction in the allowable load at the connector, loads range from about 12.0 kips to 13.8 kips for the confining stresses used with the system and exceed the calculated maximum design tension load of 7.8 kips by a factor of 2.3 to 2.8. Several photos of the exhumed connectors after testing are shown in Figure 7.

If you have any questions, or if we can be of further service, please do not hesitate to contact us.

Sincerely yours,

GeoTesting Express, Inc.

Lois G. Schwarz, Ph.D.    Marty Molino
Geotechnical Engineer    Laboratory Manager
### Table 1. DPS Anchor / TAB Connection Test Results

<table>
<thead>
<tr>
<th>Reinforcement / Connector Element Failure</th>
<th>Load at 19 mm Movement (lb)</th>
<th>Peak Tensile Load (lb)</th>
<th>Movement -Rear Longitudinal Wires at Peak Load (mm)</th>
<th>Movement -TAB at Peak Load (mm)</th>
<th>Movement -Longitudinal Wire 1” behind Tab at Peak Load (mm)</th>
<th>Applied Vertical Total Loading (lb)</th>
<th>Overburden (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GS11 Type 2 Type C Failure(^{a})</td>
<td>na</td>
<td>18,484</td>
<td>5.8</td>
<td>2.5</td>
<td>4.0</td>
<td>840</td>
<td>140</td>
</tr>
<tr>
<td>GS11 Type 2 Type B Failure</td>
<td>na</td>
<td>19,980</td>
<td>5.8</td>
<td>3.0</td>
<td>3.9</td>
<td>840</td>
<td>140</td>
</tr>
<tr>
<td>GS11 Type 2 Type B Failure</td>
<td>na</td>
<td>21,192</td>
<td>*</td>
<td>2.3</td>
<td>6.4</td>
<td>4,650</td>
<td>775</td>
</tr>
<tr>
<td>GS11 Type 2 Type B Failure</td>
<td>na</td>
<td>21,219</td>
<td>11.5</td>
<td>3.8</td>
<td>5.5</td>
<td>4,650</td>
<td>775</td>
</tr>
<tr>
<td>GS11 Type 2 Type C Failure</td>
<td>na</td>
<td>21,012</td>
<td>10.5</td>
<td>4.6</td>
<td>6.7</td>
<td>9,150</td>
<td>1,525</td>
</tr>
<tr>
<td>GS11 Type 2 Type C Failure(^{b})</td>
<td>19,920</td>
<td>20,852*</td>
<td>12.8</td>
<td>2.8</td>
<td>4.1</td>
<td>9,150</td>
<td>1,525</td>
</tr>
<tr>
<td></td>
<td></td>
<td>21,159</td>
<td>22.9</td>
<td>3.1</td>
<td>5.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Type of Element Failure:  
A: **Longitudinal wire(s) sheared**  
B: **Panel Failed**  
C: **No Failure – Test terminated**

Notes:  
* Instrumentation malfunction.  
\(^{a}\) Possible clamp slip.  
\(^{b}\) Subsequent to attaining load of 20,852 lbs, load varied between 20,400 to 20,900 lbs while movement of grid longitudinal wire nearly doubled until maximum load of 21,159 was attained.

na- not available
Figure 7. Photos of failure after exhuming connection and metallic grid—panel failure.
Segmental Concrete Panel Mechanically Stabilized Earth Connection Strength Grid-Strip Test Results

Prepared for: Atlantic Industrial Limited

Prepared By: Christopher Moignard
John Newhook, PhD., P.Eng

Centre for Innovation in Infrastructure
Dalhousie University

Date: June 2015
Disclaimer:

This material is presented for the information of the reader as a description of testing conducted and results obtained. While the material is believed to be technically correct and accurate, neither the author nor Dalhousie University warrant its suitability for any general or particular use other than to present information of testing. It is intended that the information be used by professionals competent to evaluate the significance and limitations of its contents and who accept responsibility for the application of this information.
# Table of Contents

1. INTRODUCTION ......................................................................................................................... 1

2. SYSTEM DESCRIPTION .................................................................................................................. 2
   2.1. Grid-Strip Soil Reinforcing .................................................................................................. 3
   2.2. Panel Anchor ....................................................................................................................... 5
   2.3. Concrete Panel ..................................................................................................................... 7
   2.4. Soil Properties ...................................................................................................................... 7
   2.5. System Connection ................................................................................................................ 8

3. TEST PROGRAM .......................................................................................................................... 9
   3.1. Soil Test Box ......................................................................................................................... 9
   3.2. Test-Set Up .......................................................................................................................... 11
   3.3. Deformation Measurement .................................................................................................... 13

4. TEST PROCEDURE ...................................................................................................................... 16

5. TESTING RESULTS ..................................................................................................................... 16
   5.1. DPS Testing Program .......................................................................................................... 16
      5.1.1. DPS Results 1 ............................................................................................................. 16
      5.1.2. DPS Results 2 ............................................................................................................. 18
      5.1.3. DPS Results 3 ............................................................................................................. 19
      5.1.4. DPS Results 4 ............................................................................................................. 20
   5.2. DPTS Testing Program ......................................................................................................... 21
      5.2.1. DPTS Results 1 ............................................................................................................ 21
      5.2.2. DPTS Results 2 ............................................................................................................ 22
      5.2.3. DPTS Results 3 ............................................................................................................ 23
   5.3. Summary of Results ............................................................................................................. 24

6. CONCLUSION .............................................................................................................................. 25

7. REFERENCES ............................................................................................................................... 26
# Table of Figures

Figure 1 - Mechanically Stabilized Earth Wall.......................................................................................... 2
Figure 2 - Mechanically Stabilized Earth Wall Components ................................................................. 2
Figure 3 – Connection Components ...................................................................................................... 3
Figure 4 – Grid-Strip Soil Reinforcing .................................................................................................... 4
Figure 5 – Grid-Strip Connected to Concrete Panel .............................................................................. 4
Figure 6 – Grid-Strip Panel Anchor .................................................................................................... 5
Figure 7 – DPS Anchor ......................................................................................................................... 6
Figure 8 – DPTS Anchor ....................................................................................................................... 6
Figure 9 – AIL Vistawall Panel Systems ............................................................................................... 7
Figure 10 – Grid-Strip Connection ....................................................................................................... 8
Figure 11 – Soil Box Isometric ............................................................................................................. 10
Figure 12 – Soil Box Front View .......................................................................................................... 10
Figure 13 – Soil Box Plan View ............................................................................................................. 11
Figure 14 – Soil Box Side View ............................................................................................................ 11
Figure 15 – Overview of Test-Set-Up .................................................................................................... 12
Figure 16 – Test-Set-Up At Back of Soil Box ........................................................................................ 13
Figure 17 – Welded Flat Plate on Clamping Mechanism ................................................................... 14
Figure 18 – Location of LVDTs on Clamping Mechanism ................................................................... 15
Figure 19 – Location of LVDTs On Panel Face .................................................................................. 15
Figure 20 – Test-1 DPS Soil Failure .................................................................................................... 17
Figure 21 – DPS Test-1 Results ........................................................................................................... 18
Figure 22 – Test-2 DPS Grid-Strip Failure Mechanism .................................................................... 18
Figure 23 – DPS Test-2 Results ........................................................................................................... 18
Figure 24 – Test-3 DPS Grid-Strip Failure Mechanism .................................................................... 19
Figure 25 – DPS Test-3 Results ........................................................................................................... 19
Figure 26 – Test-4 DPS Grid-Strip Failure Mechanism .................................................................... 20
Figure 27 – Test-4 DPS Results ........................................................................................................... 20
Figure 28 – Test-1 DPTS Grid-Strip Failure Mechanism .................................................................. 21
Figure 29 – Test-1 DPTS Results ........................................................................................................ 21
Figure 30 – Test-2 DPTS Grid-Strip Failure Mechanism .................................................................. 22
Figure 31 – Test-2 DPTS Results ........................................................................................................ 22
Figure 32 – Test-3 DPTS Grid-Strip Failure Mechanism .................................................................. 23
Figure 33 – Test-3 DPTS Results ........................................................................................................ 23
Table of Tables

Table 1 Test Result DPS Anchor ................................................................. 24
Table 2 Test Result DPTS Anchor ............................................................. 24
1. INTRODUCTION

Testing was performed on full-scale Mechanically Stabilized Earth (MSE) concrete panels with steel anchors that were attached to steel soil reinforcement embedded in poorly graded sand. The objective of the test program was to analyze the connection capacity of the system.

The proprietary trade name for the system being tested is the Vist-A-Wall Grid-Strip. The system components include a reinforced concrete panel, panel anchor, Grid-Strip soil reinforcement and steel connection bolt set.

The standard nominal concrete panel dimensions are 1.524 m tall by 1.524 m wide. Each standard panel contains a minimum of two rows, of two columns, of Grid-Strip soil reinforcing. The number of rows and columns of Grid-Strip soil reinforcing is project design dependent. The concrete panel is reinforced with steel welded wire mesh or conventional mild steel reinforcing bars. The size and configuration of the panel reinforcing is dependent on the loading and varies by design. The minimum panel thickness is a function of the concrete cover that is required to be placed over the panel reinforcing and design specifications. For this testing program a panel thickness of 140 mm was used.

The load from the soil reinforcing is transferred to the connection and to the panel anchor. The resistance to pullout of the panel anchor is a function of the concrete properties, reinforcement placement and panel geometric properties.

The panel was tested in a rectangular soil box. The soil box is a U shaped, open front box. The open front of the box is where the panel is placed. The three remaining sides consist of structural steel plates and beams. Interior to the soil box compacted well graded sand was placed. An overburden load was applied to the top of the box through a load platform. The soil reinforcing was attached to hydraulic loading frame where a controlled and increasing axial force was applied. Information collected with data acquisition software measured continuous displacement at the soil reinforcing, the axial load that was applied until failure and deformation at the front face of the panel.
2. **SYSTEM DESCRIPTION**

The Atlantic Industries (AIL) Vist-A-Wall system is classified as a Mechanically Stabilized Earth (MSE) retaining wall system. The AIL MSE consists of precast concrete panels and discrete steel Grid-Strip soil reinforcing. The system components include a reinforced concrete panel, panel anchor, Grid-Strip soil reinforcement, steel connection bolt set and compacted granular backfill.

![Figure 1 - Mechanically Stabilized Earth Wall](image1)

![Figure 2 - Mechanically Stabilized Earth Wall Components](image2)
2.1. Grid-Strip Soil Reinforcing

The Grid-Strip is a discrete steel soil reinforcing element that is manufactured from cold drawn wire. The lead end of the Grid-Strip is resistance welded to a special shaped connector. There are two end connectors that can be used with the Grid-Strip soil reinforcing, the TAB connector, and the flat BAR connector.

![TAB Connector](image1.png) ![BAR Connector](image2.png)

**Figure 3 – Connection Components**

The TAB connection consists of a flat 37 mm by 9 mm steel plate with a 14 mm diameter bolt hole. The TAB plate converges to a 16 mm x 62 mm circular shaft. The TAB shaft sides have a series of ridges forged in its length. The lead ends of the longitudinal wires of the Grid-Strip are resistance welded to the TAB shaft at the location of the ridges.

The BAR connector is fabricated from 9.5 mm thick steel plate that is 38.1 mm wide x 127 mm long. The lead end is fabricated with a 14 mm diameter bolt hole. The terminal end is fabricated with serrated edges. The lead ends of the longitudinal wires of the Grid-Strip are resistance welded to the terminal end of the BAR at the location of the serrations. The serrated edges are used to seat the longitudinal wires of the Grid-Strip soil reinforcing during application of the resistance weld.

The Grid-Strip soil reinforcing contains two parallel longitudinal bars that are spaced 50 mm on center. Steel transverse bars that are 75 mm long are resistance welded to the tops of the
longitudinal bars at their intersection. The bars are positioned perpendicular to the longitudinal bars and are spaced at 305 mm on center. The longitudinal bars are fabricated from W11 or W7.0 wire and cross bars are fabricated from W11 or W7.0 bars. All fabrication is in conformance with ASTM A1064.

Figure 4 – Grid-Strip Soil Reinforcing

Figure 5 – Grid-Strip Connected to Concrete Panel
2.2. **Panel Anchor**

The panel anchor consists of a steel dual plate that is cast into the concrete panel so it extends from the panel back face. Two different steel dual plate anchors can be used. These are the DPS and the DPTS anchor.

**Figure 6 – Grid-Strip Panel Anchor**

The DPS panel anchor consists of a forged steel plate and shaft system. The proximal end of the DPS anchor consists of two parallel plates that are 9 mm thick and that are offset to one another by a distance equal to 19 mm. Each plate has a 14 mm diameter centralized bolt hole. The parallel plates converge into a 16 mm diameter, 125 mm long, circular shaft. The shaft then terminates into a 44 mm disk. The panel anchor is embed into the panel a minimum of 90 mm and is placed behind the panel reinforcing.

The DPTS panel anchor consists of fabricated parallel steel plates that are 50 mm in width and that are 4 mm thick and that are offset to one another by a distance of 12 mm. Each plate has a 14 mm diameter centralized bolt hole. The parallel plates are fabricated into a triangular configuration. The panel anchor is embed into the panel a minimum of 85 mm. The anchor base is positioned behind the panel reinforcing. This anchoring system is consistent with the anchoring system used by several MSE retaining wall suppliers.
Figure 7 – DPS Anchor

Figure 8 – DPTS Anchor
2.3. Concrete Panel

The concrete panel is segmental. The panel shape is typically rectangular. The panel thickness varies. However, for this testing program the panel thickness was equal to 140 mm. This is the minimum panel thickness for this system. The panel was reinforced with steel reinforcing bars. The size and pattern of the reinforcing bars varies by design. Typically the reinforcing bars are placed in a rectangular grid pattern. The vertical bars are near the back face and are positioned to maintain a minimum 55 mm concrete cover. The concrete compressive strength varies by jurisdiction. For this testing program a minimum compressive strength equal to 30 kPa was used.

Figure 9 – AIL Vist-A-Wall Panel Systems

2.4. Soil Properties

The testing box includes a large cavity that is filled with compacted soil. The compacted soil is a critical component of the MSE system. The compacted soil helps to confine the panel to prevent bending of the panel during testing. The soil also confines and supports the panel anchor. The soil used in the testing was classified as poorly graded sand, SP (ASTM Standard D 422/D1140). MSE structures are constructed with an in place soil density equal to 95% of modified
proctor. This proctor density was used in this test program. Supplemental testing of the soil determined the bulk unit density to be equal to 1818 kg/m$^3$ at a moisture content of 4.2% (ASTM D1557). A table of soil testing results is provided in the Appendix.

2.5. System Connection

The Grid-Strip end connector is connected to the panel anchor by passing the connector into the gap formed by the parallel plates of the anchor and then by passing a galvanized A325 bolt set through the bolt holes. The connection is a single point connection that allows the Grid-Strip soil reinforcing to be rotated allowing it to avoid obstructions that are contained within the MSE mass. The Grid-Strip system components have been designed so they are interchangeable.

Figure 10 – Grid-Strip Connection
3. TEST PROGRAM

The design of Mechanically Stabilized Earth Walls (MSEW) is governed by several specifications such as the American Association of State Highway and Transportation Officials (AASHTO) *LRFD Bridge Design Specifications* (AASHTO 2014), AASHTO *LRFD Bridge Construction Specification* (AASHTO 2010b) and the Federal Highway Administration (FHWA) *Guidelines for the Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes* (FHWA 2009). Based on these design specifications the limiting component of the composite system should be the soil reinforcing element. In other words, the connection must be able to develop 100% of the limiting force of the soil reinforcing. This is a strength limit state requirement. Further, the AASHTO specifications limit the deformation at the connection to 19 mm. Deformation is considered a serviceability limit state and not a strength limit state. The testing program follows state of practice for connection strength of MSE walls. The testing program is a modification of the procedure outlined in ASTM D6638, *Standard Test Methods for Determining Connection Strength Between Geosynthetic Reinforcement and Segmental Concrete Units*. This modified method has been successfully used in other testing programs (Schwartz 2012).

The testing program was developed to evaluate the Dual Plate Shaft (DPS) and the Dual Plate Tie Strip (DPTS) anchor capacity. The testing was performed on panel coupons that have dimensions equal to 762 mm tall by 1524 mm wide. Two panel anchors were cast into each panel coupon. Attached to each panel anchor was a standard Grid-Strip soil reinforcing element. The panel thickness was 140 mm. The panels were reinforced with standard steel reinforcing bars for a “B” Panel consisting of 6–10 M bars in the vertical direction and 4–10 M bars in the horizontal direction.

3.1. Soil Test Box

To perform the study, a soil test box was designed and constructed at Dalhousie University. The box was U shaped and consisted of three walls, two side walls and one back wall. The segmental concrete panel coupon was placed at the opening of the side walls that formed the rectangular soil box. The back wall was fabricated with an opening. The opening was...
used to place the Grid-Strips through so they could be attached to a horizontal loading mechanism. Figure 11 through Figure 14 are various views of the soil box.

Figure 11 – Soil Box Isometric

Figure 12 – Soil Box Front View
3.2. Test-Set Up

The concrete panel was placed at the front of the box and temporally braced to maintain a horizontal and vertical position. The bracing was removed before the test was performed. Once the panel was placed a 305 mm thick lift of soil was placed, spread, leveled and compacted in the central cavity. Additional soil was placed and compacted to an elevation equal to the panel anchor. The Grid-Strip was then attached to the anchoring system using a bolt and nut set. After placement of the Grid-Strip a 305 mm lift of soil was placed, spread, leveled and compacted in the central cavity. The Grid-Strip soil reinforcing was then attached to the load application mechanism. Figure 15 is an isometric detail showing the concrete panel in place with the Grid-Strips attached to the panel and to the horizontal clamping system. For clarity the central cavity between the concrete panel and the side walls is shown without compacted soil.
As previously stated the soil was placed and compacted in 305 mm lift thicknesses to 95% of modified proctor. The compaction was verified using the sand cone test (ASTM D1556). The density was compared to the previously performed modified proctor tests results. If the soil was not at the require density it was re-compacted and then tested again until it passed.

The wires of the Grid-Strips were isolated from the soil by encasing them in plywood housings. The isolation of the Grid-Strip was done to prevent interaction between the soil and soil reinforcing. The isolation of the Grid-Strip prevented the reduction in load at the connection.

After all the soil lifts were compacted a vertical load system was applied to top of the soil. The load system was applied over soil area in the central cavity. The vertical load system consisted of a steel plate with HSS welded to it. Spanning across the HSS were three wide-flange spreader beams, two parallel to the panel face and one placed perpendicular to the panel face and on top of the two HSS. The combination of the steel plate and structural steel shapes applied a uniform pressure to the compacted soil within the central cavity. The load was applied to the system using a calibrated load controlled vertical hydraulic actuator. The load system was capable of applying varying overburden pressures. The variation in the overburden simulates the panel being located at different depths within an MSE structure.

Three different magnitudes of vertical loads were applied to the backfill. The loads simulated overburden pressures equal to heights of 0.0 m, 4.572 m and 9.144 m. The 0.0 m
overburden tests were slightly increased to prevent the loss of confinement of the soil inside the box. A nominal load of 15 kN was used in place of the 0.0 m overburden pressure. The 15 kN load correlates to approximately 150 mm of backfill. The load applied for the 4.572 m overburden was equal to 127.4 kN and for the 9.144 m case the load was equal to 255 kN.

Figure 16 shows the placement of the two horizontally positioned hydraulic jacks at the back edge of the soil box. These horizontal hydraulic jacks were used to apply a horizontal force on the free end of the Grid-Strips. The clamping system was designed so the Grid-Strips could freely rotate in the horizontal plane. Rotation of the clamping system prevented bending forces in the Grid-Strip. In addition, the free-end clamping system was able to articulate horizontally. This combination allowed the load for each Grid-Strips to be applied without interference.

![Figure 16 – Test-Set-Up At Back of Soil Box](image)

3.3. **Deformation Measurement**

LVDT’s were used to monitor the deformation of the system. In order to prevent slippage along the Grid-Strip longitudinal wires at the interface of the clamping mechanism a flat
bar was welded to the Grid-Strips. The flat bar was positioned directly at, and in contact with, the back of the clamp. To verify that slipping at the clamping mechanism was not occurring LVDTs were attached to each clamp and monitored. Figure 17 shows the location of the welded flat plate on the Grid-Strips. Figure 18 is a side view showing the final placement of the LVDTs at the front of the Grid-Strips clamping system. Figure 19 shows the LDVTs positioned at the front face of the panel. The LVTDs at the panel face were used to measure movement of the panel during application of the vertical load and the application of the horizontal load.

![Figure 17 – Welded Flat Plate on Clamping Mechanism](image_url)
Figure 18 – Location of LVDTs on Clamping Mechanism

Figure 19 – Location of LVDTs On Panel Face
4. TEST PROCEDURE

The first step of the test was to apply the overburden load. The load was applied and monitored through the internal data acquisition system of the vertical hydraulic actuator. Once the overburden load was applied it was allowed to come to equilibrium. Once it was verified the vertical load was in equilibrium the data-logger was started. Fluid was then pumped into the horizontal jacks. The rate at which the fluid was pumped was increased to maintain an increasing load. This was continued until failure occurred. Failure was defined as the peak tension load.

Failure of the system could occur in the following locations.

1. Longitudinal wires can fail in pure tension.
2. Weld located between the wires and the tab of the Grid-Strips can fail.
3. Failure of the bolt that attaches the Grid-Strips to the anchor.
4. The anchor could fail in tension
5. The anchor could fail in pullout from the concrete.

Once the test was stopped, the overburden pressure, the loading system, and the soil were carefully removed to inspect the system and to determine which failure mechanism had occurred.

5. TESTING RESULTS

5.1. DPS Testing Program

The first set of testing was performed on the DPS anchoring system while the second set of testing was performed on the DPTS anchoring system. The ultimate design strength of the Grid-Strip is 63.61 kN. Therefore, any value that was above this value signified a successful test.

5.1.1. DPS Results 1

The first test was performed with no overburden pressure. During this test it was determined that the panel was being pulled inward, toward the back of the soil box. The panel movement was attributed to a lack of confinement. The soil at the top of the soil box was being squeezed up and out of the box. In other words the horizontal force was large enough, and the
strength of the components was strong enough to cause the soil to bulge at the top of the soil box. Figure 20 shows the bulging of the soil mass during this test. Because there was not enough confinement at the top, the load never reached the peak load. The maximum load applied to Grid-Strip was 38.27 kN and the max movement was 10.18 mm. It should be noted that for this type of overburden the maximum horizontal load will be less than 5 kN per Grid-Strip. Figure 21 shows the graph of the average movement and applied load of the Grid-Strips. The test set up was modified from the observed behavior of the test box to better confine the backfill material. For that reason this result was not presented in the final values as shown in Table 1.
5.1.2. DPS Results 2

The second test was performed at an overburden pressure of 127 kN (4.572 m overburden). The test was performed until failure occurred in the longitudinal wires of the Grid-Strips as shown in Figure 22. The peak tension load was 75.6 kN and the peak movement was 18.4 mm. Figure 23 shows the graph of the average movement and applied load of the Grid-Strips.

![Figure 22 – Test-2 DPS Grid-Strip Failure Mechanism](image)

![Figure 23 – DPS Test-2 Results](image)
5.1.3. DPS Results 3

This test was performed at an overburden pressure of 155 kN this was below the 255 kN load. The difference in load was based on human error. However, the test is still an indication of the performance of the connection and the results were used. Figure 24 shows the location of the failure of the Grid-Strip during this test. Figure 25 shows the graph of the average movement and applied load of the Grid-Strips.

Figure 24 – Test-3 DPS Grid-Strip Failure Mechanism

![Figure 24 – Test-3 DPS Grid-Strip Failure Mechanism](image)

Figure 25 – DPS Test-3 Results

![Figure 25 – DPS Test-3 Results](image)
5.1.4. DPS Results 4

Test four was conducted at a simulated 9.144 m of overburden pressure. The maximum load was 86 kN and at a movement equal to 15 mm. Figure 26 shows the location of the failure of the Grid-Strip during this test. Figure 27 shows the graph of the average movement and applied load of the Grid-Strips.

Figure 26 – Test-4 DPS Grid-Strip Failure Mechanism

Figure 27 – Test-4 DPS Results
5.2. **DPTS Testing Program**

The second set of testing was performed on the DPTS anchoring system. The ultimate design strength of the Grid-Strip is 63.61 kN. Any value above this signifies a successful test.

**5.2.1. DPTS Results 1**

For the first DPTS test a nominal load of 15 kN was applied to the system to assure that failure occurred in the MSE wall system and not in the soil. The peak tensile load and maximum movement was 75.8 kN and 10.1 mm. Figure 28 shows the location of the failure of the Grid-Strip during this test. Figure 29 shows the graph of the average movement and applied load of the Grid-Strips.

![Figure 28 – Test-1 DPTS Grid-Strip Failure Mechanism](image)

![Figure 29 – Test-1 DPTS Results](image)
5.2.2. DPTS Results 2

For the second test of the DPTS the overburden pressure was 4.572 m equal to an applied force of 127 kN. The failure location was in the same location as DPTS Test-1, therefore the peak tensile load was nearly equal. For this test the failure load and peak movement were equal to 77.4 kN and 15.4 mm. Figure 30 shows the location of the failure of the Grid-Strip during this test. Figure 31 shows the graph of the average movement and applied load of the Grid-Strips.

![Test-2 DPTS Grid-Strip Failure Mechanism](image)

**Figure 30 – Test-2 DPTS Grid-Strip Failure Mechanism**

![Graph of average movement and applied load of Grid-Strips](image)

**Figure 31 – Test-2 DPTS Results**
5.2.3. DPTS Results 3

For the third test of the DPTS anchor the overburden pressure was 9.144 m equal to an applied force equal to 255 kN. The failure location was in the same location as test-1 and test-2, therefore the peak tensile load was nearly equal. Figure 32 shows the location of the failure of the Grid-Strip during this test. Figure 33 shows the graph of the average movement and applied load of the Grid-Strips.

Figure 32 – Test-3 DPTS Grid-Strip Failure Mechanism

Figure 33 – Test-3 DPTS Results
5.3. Summary of Results

The testing program showed that the limiting factor of the Grid-Strip system is the soil reinforcing. In the entire test program there was no sign of panel anchor pullout or concrete distress. Error! Reference source not found. lists the test results for the DPS anchor and Table 1 lists the results for the DPS anchor.

Table 1 lists the results for the DPS anchor.

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Reinforcement / Connector, Element Failure</th>
<th>Load at 19 mm Movement, kN</th>
<th>Peak Tensile Load, kN</th>
<th>Movement at Peak Load, mm</th>
<th>Applied Vertical Load, kN</th>
<th>Overburden Pressure, kN/m²</th>
<th>Respective Height of Backfill mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>DPS-2</td>
<td>GS11 strip / DPS-TAB Grid Strip</td>
<td>NA</td>
<td>75.6</td>
<td>18.4</td>
<td>127</td>
<td>91.4</td>
<td>4572</td>
</tr>
<tr>
<td>DPS-3</td>
<td>GS11 strip / DPS-TAB Grid Strip</td>
<td>NA</td>
<td>81.6</td>
<td>8.4</td>
<td>155</td>
<td>111.0</td>
<td>5486</td>
</tr>
<tr>
<td>DPS-4</td>
<td>GS11 strip / DPS-TAB Grid Strip</td>
<td>NA</td>
<td>86.0</td>
<td>15.0</td>
<td>255</td>
<td>183.0</td>
<td>9144</td>
</tr>
</tbody>
</table>

Table 1 Test Result DPTS Anchor.

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Reinforcement / Connector, Element Failure</th>
<th>Load at 19 mm Movement, kN</th>
<th>Peak Tensile Load, kN</th>
<th>Movement at Peak Load, mm</th>
<th>Applied Vertical Load, kN</th>
<th>Overburden Pressure, kN/m²</th>
<th>Respective Height of Backfill mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>DPTS-1</td>
<td>GS11 strip / DPTS Grid Strip</td>
<td>NA</td>
<td>75.8</td>
<td>10.1</td>
<td>15.0</td>
<td>10.8</td>
<td>150</td>
</tr>
<tr>
<td>DPTS-2</td>
<td>GS11 strip / DPTS Grid Strip</td>
<td>NA</td>
<td>77.4</td>
<td>15.4</td>
<td>127.0</td>
<td>91.4</td>
<td>4572</td>
</tr>
<tr>
<td>DPTS-3</td>
<td>GS11 strip / DPTS Grid Strip</td>
<td>NA</td>
<td>78.9</td>
<td>17.5</td>
<td>255.0</td>
<td>183.0</td>
<td>9144</td>
</tr>
</tbody>
</table>

Table 1. DPS and DPTS Results

State of Practice design requires that the Grid-Strip be the limiting design factor. Based on the testing all mechanical failures were localized to the longitudinal wires at the TAB. Further no panel anchor pullout or concrete distressed was observed.
6. CONCLUSION

The ultimate design resistance of the Grid-Strip is equal to 63.6 kN. The ultimate resistance for the Grid-Strip based on full scale testing on the DPS and DPTS MSE wall system is 79.2 kN. The Standard deviation is 4 kN. The serviceability limit state requirement that no more than 19 mm of deformation was met. The testing proved that the Grid-Strip fails before 19 mm of deflection is achieved. The overburden pressure proved not to add any additional capacity.
7. REFERENCES


1.2.18
Soil Reinforcing Pullout Testing and Results
GRID-STRIP™ SOIL REINFORCING

PULLOUT TEST REPORT

MAY 2017

Updated August 2018
The drawings, plans, models, designs, specifications, reports, photographs, computer software, surveys, calculations, and other data, including computer print-outs, contained in this report are the property of The Collin Group. The documents included in this report are made available for your review for informational purposes only in relation to this report. The documents may not be copied, reproduced, or distributed in any way or for any purpose whatsoever. The report has been produced using the known State-Of-Practice at the time it was completed.
# Table of Contents

1.0 INTRODUCTION ........................................................................................................ 1
2.0 TESTING PROGRAM .................................................................................................. 1
3.0 TEST MATERIAL ........................................................................................................ 1
   3.1 Soil Reinforcing ................................................................................................... 1
   3.2 Soil ....................................................................................................................... 2
4.0 TEST APPARATUS ..................................................................................................... 3
5.0 TEST PROGRAM ........................................................................................................ 7
   5.1 Test Set-Up .......................................................................................................... 7
   5.2 Test Procedure .................................................................................................... 9
   5.3 Test Methodology ............................................................................................... 9
6.0 AASHTO DESIGN EQUATIONS FOR INEXTENSIBLE SOIL REINFORCING .......... 9
   6.1 AASHTO Rupture Equation .............................................................................. 10
   6.2 AASHTO Pullout Equation ................................................................................ 11
   6.3 Pullout Test Equation ....................................................................................... 12
7.0 PULLOUT TESTING PROGRAM RESULTS ......................................................... 13
   7.1 Pullout Test Results for SP Material ................................................................. 13
   7.2 Pullout Test Results for GW Material ............................................................... 15
   7.3 Pullout Test Results for SP Material - W7.0 x W7.0 2” x 12” ........................... 17
8.0 CONCLUSION............................................................................................................ 19
1.0 INTRODUCTION

The Collin Group (TCG) has reviewed the pullout testing facility, pullout testing, and the pullout test results for the Grid-Strip™ soil reinforcing system. The pullout tests were performed to establish the friction factor to be used in the design and analysis of the Vistawall Mechanically Stabilized Earth (MSE) wall system utilizing Grid-Strip soil reinforcing. The testing was performed at the Civil Products Laboratory located at Big-R Bridge in Mansfield, Texas, using state-of-practice procedures. The following sections discuss the testing program, test procedures, and the test results.

2.0 TESTING PROGRAM

Two sets of pullout tests using two different backfill material were performed. 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 procedure specified in ASTM D 6706 “Standard Test Method for Measuring Geosynthetic Pullout Resistance in Soil”.

3.0 TEST MATERIAL

3.1 SOIL REINFORCING

The soil reinforcing material evaluated in the pullout tests consisted of the Vistawall metallic Grid-Strip™. The metallic Grid-Strip were specifically produced to meet the design strength requirements of the Vistawall system. The metallic Grid-Strip consists of discrete strip of galvanized high strength welded wire. It consists of two, parallel longitudinal wires that are spanned by perpendicular transverse wires as shown in Figure 3-1 and Figure 3-2. Pullout resistance is a function of both frictional resistance and passive resistance created by the apertures created by the longitudinal and transverse wires.

![Figure 3-1 Plan View Grid-Strip™ 2-Wire Soil Reinforcing](image-url)
The Grid-Strip longitudinal and transverse wire diameter that was used for these tests was equal. Testing was performed on Grid-Strips with wires equal to 0.375 inches (9.50 mm) and 0.299 inches (7.50 mm). The Wire Reinforcement Institute (WRI) classifies wires with these diameters as a W11 and W7.0, respectively. The center-to-center spacing of the longitudinal wires was equal to 2.0 inches (50 mm). The transverse wires for the Grid-Strip had a center-to-center spacing equal to 12.0 inches (305 mm). The transverse wires had a total width equal to 3.0 inches (75 mm). The transverse wire overhangs the longitudinal wire by 0.5 inches (12 mm). The average galvanized wire diameter for both the transverse and longitudinal wires was measured to be 0.381 inches (9.68 mm) for the W11 and 0.303 inches (7.56 mm) for the W7.0 and is based on a nominal galvanization thickness of 0.0035 inch (0.085 mm). All pullout test specimens had an effective embedment length equal to 48 inches (1200 mm). The end connector shown in Figure 3-1 and Figure 3-2 is one type of end connector used by Vitawall for the Grid-Strip™ system. The end connector is not relevant to this testing program and is shown for information purposes only.

3.2 Soil

The pullout tests were performed using a poorly graded sand (SP) and a poorly graded gravel (GP) as backfill. The sand used in the tests was obtained from Florida and was supplied by Vistawall. The sand is classified as SP and the gradation (ASTM D422 and ASTM D1140) is shown in the Appendix. The Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (ASTM D698) test results performed on the sand determined a maximum dry density equal to 96 pcf (16.3 kN/m3) at an optimal moisture content of 6.9%. The friction angle of the sand is equal to 31 degrees as determined using ASTM 3080. The gravel used in the pullout tests consisted of a poorly graded gravel (GP) found in Georgia. Relative density was determined using ASTM D 4253. The test results determined a maximum
dry density of 112.5 pcf (17.7 kN/m³) and a minimum dry density equal to 88.1 pcf (13.8). The friction angle of the gravel is equal to 42 degrees as determined using ASTM D3080 Modified. The grain size curves and proctor test results for both soils are included in Appendix A.

4.0 TEST APPARATUS

The pullout tests were performed at the Big-R Civil Products Laboratory using a state-of-practice pullout apparatus. The apparatus was fabricated in conformance with the recommendations of the ASTM D 6706. The apparatus is shown schematically in Figure 4-1. The pullout apparatus, including the reaction frame, were fabricated from structural steel components.

![Figure 4-1 Cross Section Pullout Box](image)

The pullout box has an inside dimension of 18” (450 mm) wide by 60” (1500 mm) long and is 18” (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. The height of the opening in the exit gate is 1” (25 mm). At the exit gate two ½” x 7” (12mm x 350 mm) steel plates are welded to the top and bottom 2” x 2” (50 mm x 50 mm) structural steel tubing. The steel plates protrude into the soil. The steel plates reduce any boundary condition that may occur at the back of the box from the applied vertical overburden pressure.
The vertical overburden pressure is applied using a pneumatic diaphragm (air bladder). The pneumatic diaphragm is inflated and pushes against the compacted soil and a reaction frame. The reaction frame is shown in Photograph 4-2. The vertical overburden pressure is controlled using a pneumatic regulator and is measured using two Omega low profile load cells.

Photograph 4-2 Vertical Overburden Apparatus

The pneumatic diaphragm is sandwiched between two, ¾” x 16” x 58” (19 mm x 400 mm x 1450 mm) neoprene bearing pads. Placed on the top neoprene bearing pad is a ½” thick steel plate. Placed on top of the steel plate are two, 2” x 6” (50 mm x 150 mm) structural steel tubes. Spanning between, and on top of the two, 2” x 6” (50 mm x 150 mm) structural steel tubes, are 2” x 6” x 14” (50 mm x 150 mm x 350 mm) structural steel tubes. The load cells are placed on top of the 2” x 6” x 14” (50 mm x 150 mm) structural steel tubes. The reaction frame is placed on top of the load cells. The reaction frame consists of two, 2” x 6” x 20” (50 mm x 150 mm x 500 mm) structural steel tubes that are attached to 1” diameter, grade-8, all-thread rods using a washer and nut. The pneumatic diaphragm and steel components are
isolated from the side of the pullout box using a sheet of 4 mm plastic. The plastic reduces the possible effects of down drag that may occur during the inflation of the pneumatic diaphragm.

At the back of the pullout box the soil reinforcing is attached to a specialized connection clamp as shown in Photograph 4-3. The soil reinforcing is clamped between two specialized steel plates. The steel plates are ½” (12 mm) thick. The bottom plate is fitted with a series protruding threaded rods. The top plate is fabricated with a series of holes that accept the threaded rods protruding from the bottom plate. The soil reinforcing is secured between the two plates by attaching nuts to the threaded rods. To prevent sliding of the soil reinforcing longitudinal wires at the interface of the plate faces, a series of diagonal, pointed, teeth are machined into the plate. The teeth resemble the pattern that is on a steel file and will “bite” into the longitudinal wire when the nuts are tightened against the plate. To harden the peak of the teeth the connection plates are heat treated. The bottom steel plate is fabricated to include a device that allows it to be attached to the horizontal loading device that consists of a hydraulic cylinder. The connection plate is attached to the hydraulic cylinder rod end using a clevis. The clevis consists of two parallel steel plates with a central through bore. When the connection plate is attached to the clevis it forms a single point connector. The combination of the clevis and the single point connector allows the connection plate to rotate in a horizontal and vertical plane. The rotation allows the load to be applied uniformly to the soil reinforcing element.

Photograph 4-3 Connection Plate Apparatus

The horizontal load is applied the soil reinforcing using a Parker Heavy Duty hydraulic cylinder (Photograph 4-4). The cylinder bore is 5” (125 mm) and has an 18” (450 mm) stroke. The maximum tensile
force for the cylinder is equal to 50-kip (220 kN). The horizontal load is applied so the rate of travel is between 2 mm to 5 mm per minute. The rate of travel is variable due to system conditions, such as, atmospheric conditions, load conditions, fluid conditions, etc. The rate of load application is controlled using a metering valve that is attached to the hydraulic fluid system. The horizontal load is monitored using an Omega Canister load cell with a maximum 25-kip capacity (LC1001-50k). The load cell is attached to the hydraulic rod end and the connection plate is attached to the load cell. All load cells used in the test were calibrated prior to testing using an MTS system.

![Photograph 4-4 Horizontal Load Apparatus](image)

Linear Variable Displacement Transducers (LVDT) are used to record the displacement of the soil reinforcing and connection plate during application of the horizontal load. Four LVDT’s were used. Three LVDT’s are placed on the soil reinforcing element inside the soil. The first LVDT is positioned at the end of the soil reinforcing and at the midpoint of the transverse wire. The second and third LVDT’s are positioned at the intersection of the transverse wire on the second and third wire respectively as shown in Figure 4-2. The fourth LVDT is positioned at the location of the exit gate near the connection plate. The first through third LVDT are used to measure the displacement of the soil reinforcing inside the soil and is
used to determine the alpha factor. The fourth LVDT measures the displacement of the soil reinforcing outside the soil box and is used to determine the friction factor.

![Figure 4-2 Plan View of LVDT Locations](image)

Figure 4-2 Plan View of LVDT Locations

A Campbell Scientific Data Acquisition system was used to record the horizontal load and the LVDT’s. The Campbell Scientific PC400 software was used to capture the data as shown in Photograph 4-5. The data is extracted in a text file and imported into Excel.

![Photograph 4-5 Data Acquisition System](image)

Photograph 4-5 Data Acquisition System

5.0 TEST PROGRAM

The test program is a repetitive process and does not vary from test to test. The following set-up, procedures and methodology are used for each test.

5.1 TEST SET-UP

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 system components
6. Determine system pressure required to be 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. Verify that the all-thread columns are secure by tightening the nut on bottom interface of the 2x2 HSST.
25. Move connection plate into proper position. Connect soil reinforcing to connection load plate. Verify that connection plate is level. Secure bolts with air wrench.
26. Place LVDT fixture and LVDT at front of the pullout box. Level and straighten the LVDT.
27. Connect LVDT’s at back of pullout box
5.2 **Test Procedure**

1. Turn Campbell Scientific on and perform the following:
   a. Zero load cells.
   b. Record the beginning positions of LVDTs
   c. Determine the position of the LVDT at ¾” deformation and record the value

2. Expand pneumatic diaphragm until required load is applied to the load cells. Verify that loads are equal in the load cell.

3. Turn on hydraulic pump. Verify that the ball valve is in the position required to perform test and is not in “set-up” position.

4. Activate flow control to perform test

5. Stop test at 1 ½” total displacement at exit gate

6. Release pressure in hydraulic cylinder

7. Save data from Campbell scientific PC400 software

8. Remove reaction frame

9. Remove pneumatic diaphragm

10. Remove soil to top of soil reinforcing

11. Examine soil reinforcing in place in soil

12. Remove soil reinforcing and examine for deflection and damage

5.3 **Test Methodology**

The Grid-Strip™ soil reinforcing was pulled at a rate of 1 mm to 4 mm per minute in accordance with the ASTM D 6706 test procedure. Tests were performed until the specimen either pulled out of the soil or failure of the soil reinforcing occurred prior to reaching the required pullout displacement. The defining displacement that is used to calculate the friction factor, as required by FHWA guidelines, is equal to 3/4 inch (20 mm). As stated previously this is measured at the outside of the box, at the exit gate. The pullout tests were performed to a displacement equal to 1.5-inch (38 mm). For this test program 1.5-inch (38 mm) displacement was used to attain the maximum peak pullout resistance and to create a complete pullout deformation curve. After the completion of each test the soil reinforcing was exhumed and examined to observe and record any distortion and/or deformation of the element.

6.0 **AASHTO Design Equations for Inextensible Soil Reinforcing**
The AASHTO specification for internal stability requires soil reinforcing to be designed for both rupture and pullout. It is important to know the maximum capacity of the soil reinforcing as it will determine which of the two requirements, rupture, or pullout, controls at each elevation of soil reinforcing. By knowing this, it is possible to determine at what depth rupture will control and at what depth pullout will control for various soil and loading conditions. In typical applications, pullout only controls in the upper 8 feet of the structure.

6.1 AASHTO RUPTURE EQUATION

AASHTO Article 11.10.6.4, “Reinforcement Strength”, requires the allowable strength of the soil reinforcing to be determined as specified in Article 11.10.6.4.3a using relationship shown in Equation 1.

\[ T_{al} = \frac{A_c \cdot F_y}{b} \]  

Equation 1

where:
- \( T_{al} \) = nominal long-term reinforcement design strength (kip/ft)
- \( A_c \) = area of reinforcement corrected for corrosion loss (in²)
- \( F_y \) = minimum yield strength of steel (ksi)
- \( b \) = unit width of reinforcement (ft)

Equation 1 gives the nominal strength in terms of a one-foot width of the structure. Removing the unit width of the reinforcement, \( b \), from the equation gives the nominal strength of a single soil reinforcing element. The nominal strength is reduced by the resistance factor given in AASHTO Table 11.5.7-1 equal to 0.75 to determine the allowable strength.

For steel soil reinforcing the area of the reinforcement is corrected for the loss of corrosion. The sacrificial steel loss is determined using Equation 2 as given in AASHTO Article 11.10.6.4.2a-1.

\[ E_c = E_n - E_s \]  

Equation 2

where:
- \( E_c \) = thickness of metal reinforcement at 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 service life of structure (mil)
The design 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. For structural design, the sacrificial thicknesses are determined for all exposed surfaces. Structural design, and therefore corrosion, is applied to the longitudinal wires only. 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 method of hot-dip in conformance with AASHTO M 111M/M 111 (ASTM A123/A123M). 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 a 75-year service life is equal to 0.056 inches and at the end of a 100-year service life the sacrificial thickness of metal expected to be lost is 0.079 inches.

Based on the AASHTO requirements, and removing the sacrificial thickness of the metal reinforcement at end of the service life to the original soil reinforcing wire area, the allowable design strength for the Grid-Strip soil reinforcing used in this testing program is 7.8 kips and 6.7 kips for a 75-year and 100-year service life respectively. As a result, any force determined in the pullout test that is above these calculated strength values will result in rupture of the soil reinforcing controlling the design. In other words, rupture of the soil reinforcing will occur before the available pullout resistance is reached.

### 6.2 AASHTO Pullout Equation

The equation for pullout resistance as defined in AASHTO Article 11.10.6.3.2-1, in relationship to the required length of embedment, is shown in Equation 3.

\[
L_e = \frac{T_{\text{max}}}{\phi \cdot F \cdot \alpha \cdot \sigma_v \cdot C \cdot R_c}
\]

**Equation 3**

where:
- \( L_e \) = required length of embedment behind failure surface (ft)
- \( T_{\text{max}} \) = applied factored load in the reinforcement from Eq. 11.10.6.2.1-2 (kips/ft)
\( \phi \) = resistance factor for reinforcement pullout from Table 11.5.7-1 (dim.)

\( F^* \) = pullout friction factor (dim.)

\( \alpha \) = scale Correction Factor (dim)

\( \sigma_v \) = unfactored vertical stress at the reinforcement level in the resistant zone (ksf)

\( C \) = effective unit perimeter (dim)

\( R_c \) = reinforcement coverage ratio from Article 11.10.6.4.1 (dim)

For a soil reinforcing element of a known width, Equation 3 can be rearranged to relate the ultimate pullout resistance as shown in Equation 4.

\[
P_r = \phi \cdot \alpha \cdot C \cdot F^* \cdot L_v \cdot \sigma_v \cdot w
\]

Equation 4

where:

\( P_r \) = pullout resistance (kip)

\( w \) = width of soil reinforcing (ft)

The effective unit perimeter, \( C \), is defined as being equal to 2. For inextensible soil reinforcing the scale correction factor, \( \alpha \), is defined as being equal to 1.00. The resistance factor, \( \phi \), is used in the equation during design and can be removed. Based on these conditions the equation can be reduced to Equation 5.

\[
P_r = 2 \cdot F^* \cdot L_v \cdot \sigma_v \cdot w
\]

Equation 5

The equation that would be used in design to check the capacity demand ratio (CDR) for pullout of the Grid-Strip soil reinforcing at a known embedment depth over a known tributary area is shown in Equation 6.

\[
CDR_{PD} = \frac{P_r}{T_{max}} = \frac{\phi \cdot 2 \cdot F^* \cdot L_v \cdot \sigma_v \cdot w}{T_{max}} \geq 1.00
\]

Equation 6

### 6.3 PULLOUT TEST EQUATION

The pullout test is used to determine the friction factor, \( F^* \). For inextensible soil reinforcing, FHWA 10-024 recommends that a maximum displacement equal to \( \frac{3}{4} \) inch (20 mm) as measured at the front of the soil reinforcing pullout box be used to select \( P_r \). This criterion is based on the condition that
the peak value for $P_r$, or rupture of the soil reinforcing, does not occur before ¾ inch (20 mm) displacement is reached. If the peak value, or rupture of the soil reinforcing occurs before the ¼ inch displacement is reached those values are used to determine $F^*$. Using the pullout test procedure defined in Section 2.0, and Equation 5, $F^*$ is the only variable that is unknown during the pullout testing. Therefore, the equation that is used to back calculate $F^*$ from the pullout test results is as shown in Equation 7.

$$F^* = \frac{P_{r\text{ test}}} {2 \cdot L_c \cdot \sigma_{v\text{ test}} \cdot W}$$

Equation 7

where: $P_{r\text{ test}}$ = Pullout force at ¾” deformation (kip)
$
\sigma_{v\text{ test}}$ = Applied vertical overburden (ksf)

A load-deflection plot from each test is shown in the Appendix, A. In these figures, the LVDT displacements are plotted versus the corresponding load measured at the load cell. The $F^*$ value determined for each normal stress is shown in Table 1 and Table 2 for ¾” (20 mm) displacement criteria and the peak value measured at 1 ½” (40 mm) displacement.

### 7.0 PULLOUT TESTING PROGRAM RESULTS

#### 7.1 Pullout Test Results for SP Material

Twenty pullout tests were performed on the SP soil. The applied overburden pressure ranged from 125 psf to 2500 psf. This equates to depths of soil from one foot of soil to twenty feet of soil. Based on the pullout test the $\alpha$ factor was determined to be equal to one (1). The test program showed that consistent and concurrent movements of the front and back transverse wires occurred during the pullout tests.

<table>
<thead>
<tr>
<th>Test</th>
<th>Name</th>
<th>Surcharge psf</th>
<th>$P_{r\text{ 3/4 lbf}}$</th>
<th>$F^*_{\text{3/4}}$</th>
<th>$P_{r\text{ peak lbf}}$</th>
<th>$F^*_{\text{peak}}$</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>125 psf Test-1</td>
<td>125</td>
<td>1147</td>
<td>6.87</td>
<td>1147</td>
<td>6.87</td>
<td>Reference note 1</td>
</tr>
<tr>
<td>2</td>
<td>1100 psf Test-1</td>
<td>1100</td>
<td>5406</td>
<td>3.68</td>
<td>5776</td>
<td>3.93</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1100 psf Test-2</td>
<td>1100</td>
<td>5676</td>
<td>3.86</td>
<td>6130</td>
<td>4.17</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1100 psf Test-3</td>
<td>1100</td>
<td>5473</td>
<td>3.72</td>
<td>5941</td>
<td>4.04</td>
<td></td>
</tr>
<tr>
<td>Test</td>
<td>Name</td>
<td>Surcharge psf</td>
<td>$P_{r, 3/4}$ lbf</td>
<td>$F^*_{3/4}$</td>
<td>$P_{r, peak}$ lbf</td>
<td>$F^*_{peak}$</td>
<td>Notes</td>
</tr>
<tr>
<td>------</td>
<td>---------------</td>
<td>---------------</td>
<td>------------------</td>
<td>-------------</td>
<td>-------------------</td>
<td>--------------</td>
<td>----------------------------</td>
</tr>
<tr>
<td>5</td>
<td>125 psf Test-2</td>
<td>125</td>
<td>919</td>
<td>5.50</td>
<td>941</td>
<td>5.63</td>
<td>Reference note 1</td>
</tr>
<tr>
<td>6</td>
<td>1675 psf Test-1</td>
<td>1650</td>
<td>6641</td>
<td>3.01</td>
<td>6999</td>
<td>3.18</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>550 psf Test-1</td>
<td>550</td>
<td>3400</td>
<td>0.00</td>
<td>0</td>
<td>0.00</td>
<td>Reference note 2</td>
</tr>
<tr>
<td>8</td>
<td>2200 psf Test-1</td>
<td>2200</td>
<td>6800</td>
<td>2.31</td>
<td>7250</td>
<td>2.47</td>
<td>Reference note 3</td>
</tr>
<tr>
<td>9</td>
<td>2200 psf Test-1</td>
<td>2200</td>
<td>6950</td>
<td>2.36</td>
<td>7194</td>
<td>2.45</td>
<td>Reference note 3</td>
</tr>
<tr>
<td>10</td>
<td>125 psf Test-1</td>
<td>125</td>
<td>977.3</td>
<td>5.85</td>
<td>977.3</td>
<td>5.85</td>
<td>Reference note 1</td>
</tr>
<tr>
<td>11</td>
<td>2200 psf Test-1</td>
<td>2200</td>
<td>7655</td>
<td>2.60</td>
<td>8245</td>
<td>2.81</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>550 psf Test-1</td>
<td>550</td>
<td>3216</td>
<td>4.38</td>
<td>3299</td>
<td>4.49</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>550 psf Test-1</td>
<td>550</td>
<td>2718</td>
<td>3.70</td>
<td>3523</td>
<td>4.79</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>125 psf Test-2</td>
<td>125</td>
<td>1006</td>
<td>6.02</td>
<td>1068</td>
<td>6.40</td>
<td>Reference note 1</td>
</tr>
<tr>
<td>15</td>
<td>1650 psf Test-2</td>
<td>1650</td>
<td>7035</td>
<td>3.19</td>
<td>7814</td>
<td>3.54</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>550 psf Test-4</td>
<td>550</td>
<td>4130</td>
<td>5.62</td>
<td>4459</td>
<td>6.07</td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>2500 psf Test-1</td>
<td>2500</td>
<td>8810</td>
<td>2.64</td>
<td>9758</td>
<td>2.92</td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>625 psf Test-1</td>
<td>725</td>
<td>4627</td>
<td>4.78</td>
<td>5274</td>
<td>5.44</td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>2050 psf Test-1</td>
<td>2175</td>
<td>7620</td>
<td>2.62</td>
<td>8595</td>
<td>2.96</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>2050 psf Test-1</td>
<td>315</td>
<td>2160</td>
<td>5.13</td>
<td>2363</td>
<td>5.61</td>
<td></td>
</tr>
</tbody>
</table>

1. Peak was achieved prior to 3/4”
2. Pullout data file corrupt. 3/4” recorded during test
3. Data set to 2-bytes in program (maximum value 6999). Data recorded during test
4. Effective embedment length equal to 48 inch (1200 mm)
5. All Grid-Strips had a transverse wire width equal to 3 inches (75 mm)
6. Longitudinal wire center-to-center spacing was equal to 2.0 in. (50 mm)
7. Transverse bar spacing of 12 inch (305 mm)
8. Soil and System components equate to 125 psf load
Figure 7-1 Friction Factor for SP Material

Based on the results of the pullout test for SP material the friction factor, F*, that can be used in design is equal to 3.00 at the top of the structure (0-feet) decreasing to 1.00 at a depth of 20-feet and below.

7.2 Pullout Test Results for GW Material

Nine pullout tests were performed on the gravel (GP) soil. The applied overburden pressure ranged from 125 psf to 2500 psf. This equates to depths of soil from one foot of soil to twenty feet of soil. Based on the pullout test the α factor was determined to be equal to one (1). The test program showed
that consistent and concurrent movements of the front and back transverse wires occurred during the pullout tests.

<table>
<thead>
<tr>
<th>Test</th>
<th>Name</th>
<th>Surcharge psf</th>
<th>$P_{r,3/4}$ lbf</th>
<th>$F^*_{3/4}$</th>
<th>$P_{r,peak}$ lbf</th>
<th>$F^*_{peak}$</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>125 psf Test-1</td>
<td>125</td>
<td>2079</td>
<td>12.45</td>
<td>2295</td>
<td>13.74</td>
<td>Reference note 1</td>
</tr>
<tr>
<td>2</td>
<td>125 psf Test-2</td>
<td>125</td>
<td>2060</td>
<td>12.34</td>
<td>941</td>
<td>5.63</td>
<td>Reference note 1</td>
</tr>
<tr>
<td>3</td>
<td>125 psf Test-3</td>
<td>125</td>
<td>1864</td>
<td>11.16</td>
<td>1940</td>
<td>11.62</td>
<td>Reference note 1</td>
</tr>
<tr>
<td>4</td>
<td>625 psf Test-1</td>
<td>625</td>
<td>5898</td>
<td>7.06</td>
<td>7017</td>
<td>8.40</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>1250 psf Test-1</td>
<td>1250</td>
<td>7212</td>
<td>4.32</td>
<td>9095</td>
<td>5.45</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>1875 psf Test-1</td>
<td>1875</td>
<td>7616</td>
<td>3.04</td>
<td>10516</td>
<td>4.20</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>2500 paf Test-1</td>
<td>2500</td>
<td>10058</td>
<td>3.01</td>
<td>12753</td>
<td>3.82</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>625 psf Test-2</td>
<td>625</td>
<td>5672</td>
<td>6.79</td>
<td>7047</td>
<td>8.44</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>1250 psf Test-2</td>
<td>1250</td>
<td>8478</td>
<td>5.08</td>
<td>9379</td>
<td>5.62</td>
<td></td>
</tr>
</tbody>
</table>

1. Peak was achieved prior to $\frac{3}{4}”$
Based on the results of the pullout test for GP material the friction factor $F^*$ that can be used in design is equal to 6.00 at the top of the structure, decreasing to 3.00 at a depth of 10 feet and then decreasing to 1.50 at a depth of 20 feet and below.

### 7.3 Pullout Test Results for SP Material - W7.0 x W7.0 2” x 12”

Eleven pullout tests were performed on the SP soil using a W7.0 x W7.0 – 2” x 12” Grid-Strip as shown in Table 3. The applied overburden pressure ranged from 125 psf to 2500 psf. This equates to depths of soil from one foot of soil to twenty feet of soil. Based on the pullout test the $\alpha$ factor was determined to be equal to one (1). The test program showed that consistent and concurrent movements of the front and back transverse wires occurred during the pullout tests.
### Table 3  Pullout Test Program - SP Fill Material - W7.0 x W7.0 – 2” x 12”

<table>
<thead>
<tr>
<th>Test</th>
<th>Name</th>
<th>Surcharge psf</th>
<th>$P_{r_{3/4}}$ lbf</th>
<th>$F^*_{3/4}$</th>
<th>$P_{r_{peak}}$ lbf</th>
<th>$F^*_{peak}$</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>125 psf Test-1</td>
<td>125</td>
<td>1000</td>
<td>5.99</td>
<td>1000</td>
<td>5.99</td>
<td>Reference note 1</td>
</tr>
<tr>
<td>2</td>
<td>125 psf Test-2</td>
<td>125</td>
<td>1055</td>
<td>6.32</td>
<td>1055</td>
<td>6.32</td>
<td>Reference note 1</td>
</tr>
<tr>
<td>3</td>
<td>675 psf Test-1</td>
<td>625</td>
<td>2910</td>
<td>3.49</td>
<td>3115</td>
<td>3.73</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1250 psf Test-1</td>
<td>1250</td>
<td>3752</td>
<td>2.25</td>
<td>5185</td>
<td>3.10</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>1875 psf Test-1</td>
<td>1875</td>
<td>5035</td>
<td>2.01</td>
<td>5225</td>
<td>2.09</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>2500 psf Test-1</td>
<td>2500</td>
<td>5780</td>
<td>1.73</td>
<td>6492</td>
<td>1.94</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>312 psf Test-1</td>
<td>312</td>
<td>1708</td>
<td>4.10</td>
<td>1860</td>
<td>4.46</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>937 psf Test-1</td>
<td>937</td>
<td>3248</td>
<td>2.59</td>
<td>3588</td>
<td>2.87</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>125 psf Test-3</td>
<td>125</td>
<td>703</td>
<td>4.21</td>
<td>767</td>
<td>4.59</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>125 psf Test-4</td>
<td>125</td>
<td>712</td>
<td>4.26</td>
<td>744</td>
<td>4.46</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>125 psf Test-5</td>
<td>125</td>
<td>726</td>
<td>4.35</td>
<td>748</td>
<td>4.48</td>
<td></td>
</tr>
</tbody>
</table>

1. Peak was achieved prior to ¾”
Based on the results of the pullout test for the W7.0 x W7.0 – 2” x 12” Grid-Strip in SP material the friction factor, F*, that can be used in design, is equal to 4.00 at the top of the structure decreasing to 2.00 at a depth of 10 decreasing to 1.50 at a depth of 20 feet and below as shown in Figure 7-3. The SP material would be a lower bound material, and therefore, the friction factor defined for this material is conservative.

8.0 CONCLUSION

The Grid-Strip soil reinforcing pullout friction factors were determined using AASHTO D6706 “Standard Test Method for Measuring Geosynthetic Pullout Resistance in Soil” and is known to be the
state-of-practice at the time of this report. Twenty tests in sand and nine tests in gravel were performed. The Grid-Strip soil reinforcing element consists of two longitudinal wires that have a series of transverse wires welded to them. This forms a soil reinforcing element that resembles a ladder with apparent openings between transverse elements. The longitudinal wires are spaced at 2 inches (50 mm) and the transvers wires are spaced at 12 inches (305 mm).

Based on this testing program, and the Grid-Strip described in this report, the friction factor varies between each soil type. The configuration of the Grid-Strip provides both frictional and passive resistance during application of a horizontal force. Passive resistance is the major contributor to the pullout resistance. For the Grid-Strip to be pulled out of the soil, the soil particles that are in contact with, and that are surrounding the transverse wire, must be moved out of the way. This is analogous to soil dilation in a direct shear test. At low confining stress, the ability of the soil to dilate increases as the transverse wire moves in the soil. The effect of this dilatancy is an increase in the pullout resistance and therefore an increase in the friction factor. It is well known that the magnitude of dilatancy is a function of the particle shape and size. Material with a smaller particle size and a rounder profile will dilate less than a material with a larger particle size and with an angular profile. The particle shape and particle size have been related to the friction angle of the soil. At equivalent overburden pressure, small particle soil that has rounded particle shape, is easier to move out of the way as the transverse wire pulls through the soil than a large, angular, particle soil. The small, round, particle soil will have a lower dilatancy. The dilatancy effect can clearly be seen in the difference in the friction factor at low overburden for each of the two materials used in the testing program.

The lower bound friction factor occurs in the sand material while the higher bound friction factor occurs in the gravel. This is a function of the friction angle of the material equal to 31 degrees for the sand and 42 degrees for the gravel. A lower bound friction factor that is equal to 3.00 at the top of the structure decreasing to 1.25 at a depth of 20 feet and below can be used for any soil type. This would be a conservative assumption for both sand and granular materials. For structures that are well graded the upper bound friction factor equal to 6.00 at the top of the structure, decreasing to 3.00 at a depth of 10 feet and decreasing to 1.50 at a depth of 20 feet and below.
Appendix – A

Soil Testing Results
<table>
<thead>
<tr>
<th>Sieve No.</th>
<th>Cumulative Wt. Retained (g)</th>
<th>% Retained</th>
<th>% Passing</th>
<th>Sieve Specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8&quot;</td>
<td>0.0</td>
<td>0.00</td>
<td>100.0</td>
<td></td>
</tr>
<tr>
<td>No. 4</td>
<td>0.20</td>
<td>0.00</td>
<td>100.0</td>
<td></td>
</tr>
<tr>
<td>No. 8</td>
<td>2.05</td>
<td>0.40</td>
<td>99.6</td>
<td></td>
</tr>
<tr>
<td>No. 16</td>
<td>5.39</td>
<td>1.20</td>
<td>98.8</td>
<td></td>
</tr>
<tr>
<td>No. 30</td>
<td>12.37</td>
<td>2.70</td>
<td>97.3</td>
<td></td>
</tr>
<tr>
<td>No. 50</td>
<td>22.52</td>
<td>5.60</td>
<td>94.4</td>
<td></td>
</tr>
<tr>
<td>No. 100</td>
<td>394.04</td>
<td>86.20</td>
<td>13.8</td>
<td></td>
</tr>
<tr>
<td>No. 200</td>
<td>450.68</td>
<td>98.60</td>
<td>1.40</td>
<td></td>
</tr>
</tbody>
</table>

PAN 451.26

Decant

Dry Wt. + Tare = 568.52
Washed Wt. + Tare = 563.25
Tare Wt. = 111.38
Decant % = 1.20

LIMITATIONS: The results reported herein were prepared based upon the specific samples provided for testing. We assume no responsibility for variation in quality (composition, appearance, performance, 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 client to whom they are addressed and shall not be reproduced except in full without our written approval. Rone Engineering Services, Ltd.
### Q/A Sieve Analysis Results

<table>
<thead>
<tr>
<th>Sieve Size (US)</th>
<th>Sieve Size (mm)</th>
<th>Sample 1 FL-Sand</th>
<th>Sample 2 Boland</th>
<th>% Passing by Weight</th>
<th>Average</th>
<th>Min. Limit</th>
<th>Max. Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>25.4</td>
<td>100.0</td>
<td></td>
<td></td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>0.8</td>
<td>16.0</td>
<td>100.0</td>
<td></td>
<td></td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>1/2</td>
<td>12.7</td>
<td>100.0</td>
<td></td>
<td></td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>3/8</td>
<td>9.5</td>
<td>100.0</td>
<td></td>
<td></td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>4.00</td>
<td>4.76</td>
<td>100.0</td>
<td></td>
<td></td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>8.00</td>
<td>2.38</td>
<td>99.6</td>
<td></td>
<td></td>
<td>99.6</td>
<td>99.6</td>
<td>99.6</td>
</tr>
<tr>
<td>16.00</td>
<td>1.19</td>
<td>98.8</td>
<td></td>
<td></td>
<td>98.8</td>
<td>98.8</td>
<td>98.8</td>
</tr>
<tr>
<td>30.000</td>
<td>0.595</td>
<td>97.3</td>
<td></td>
<td></td>
<td>97.3</td>
<td>97.3</td>
<td>97.3</td>
</tr>
<tr>
<td>50.000</td>
<td>0.297</td>
<td>94.4</td>
<td></td>
<td></td>
<td>94.4</td>
<td>94.4</td>
<td>94.4</td>
</tr>
<tr>
<td>100.000</td>
<td>0.149</td>
<td>13.4</td>
<td></td>
<td></td>
<td>13.4</td>
<td>13.4</td>
<td>13.4</td>
</tr>
<tr>
<td>200.000</td>
<td>0.074</td>
<td>1.4</td>
<td></td>
<td></td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
</tr>
</tbody>
</table>

**UNIFIED SOIL CLASSIFICATION SYSTEM**

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests:

- **Coarse Grained Soils** More than 50% retained on No. 200 sieve
  - Gravels: More than 50% of coarse fraction retained on No. 4 sieve
  - Clean Gravels: Less than 5% fines
  - Gravels with Fines: More than 12% fines

- **Sands** 50% or more of coarse fraction passes No. 4 sieve
  - Clean Sands: Less than 5% fines
  - Sands with Fines: More than 12% fines

- **Fine-Grained Soils** 50% or more passes the No. 200 sieve
  - Silts and Clays: Liquid limit less than 50
    - Inorganic: Pi > 7 and plots on or above “A” line
      - Organic: Liquid limit - oven dried
        - Liquid limit - not dried

- Silts and Clays: Liquid limit 50 or more
  - Inorganic: Pi plots on or above “A” line
    - Organic: Liquid limit - oven dried
      - Liquid limit - not dried

- **Soil Classification**
  - CL Lean clay
  - ML Silty clay
  - OL Organic clay
  - CH Fat clay
  - MH Elasic Silt
  - PT Peat

---

**Notes:**
- CL: Lean clay, primarily inorganic, non-plastic
- ML: Silty clay, organic, plastic
- OL: Organic clay, predominantly organic, non-plastic
- CH: Fat clay, organic, plastic
- MH: Elastic Silt, organic
- PT: Peat

---

**Calculations:**
- Cu = D10/D60
- Cc = D60/D10

---

**References:**
- Based on soil passing the 3-in. (75-mm) sieve
- If field sample contains cobbles or boulders, add “with cobbles or boulders, or both” to group name.
- If Gravels with 6 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.
- If Sands with 6 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-GC poorly graded sand with clay.

---

**Symbols:**
- D10: Diameter of 10% passing
- D30: Diameter of 30% passing
- D60: Diameter of 60% passing
- Cu: Coefficient of uniformity
- Cc: Coefficient of curvature
DIRECT SHEAR STRENGTH TEST RESULTS

<table>
<thead>
<tr>
<th>Strength Parameter</th>
<th>Peak</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesion (psf)</td>
<td>180</td>
</tr>
<tr>
<td>Friction Angle (deg.)</td>
<td>31</td>
</tr>
</tbody>
</table>

Trial No. | 1   | 2   | 3   |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water Content (%)</td>
<td>14.2</td>
<td>14.5</td>
<td>14.5</td>
</tr>
<tr>
<td>Dry Density (pcf)</td>
<td>95.5</td>
<td>95.3</td>
<td>95.8</td>
</tr>
<tr>
<td>Diameter (inch)</td>
<td>2.500</td>
<td>2.500</td>
<td>2.500</td>
</tr>
<tr>
<td>Height (inch)</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>At Test</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water Content (%)</td>
<td>26.7</td>
<td>25.8</td>
<td>24.9</td>
</tr>
<tr>
<td>Height (inch)</td>
<td>1.006</td>
<td>0.992</td>
<td>1.006</td>
</tr>
</tbody>
</table>

Normal Stress (psf) | 1,003 | 1,994 | 4,009 |
Peak Failure Stress (psf) | 849 | 1,297 | 2,640 |
Deformation at Peak Failure Stress (Inch) | 0.13 | 0.14 | 0.13 |

Notes: Test results from Geotac equipment. Horizontal movement measured with LVDT.
Revised: 8/21/2018
Report of Moisture Density Relationship

Client: Rone Engineering Services, Ltd.
8908 Ambassador Row
Dallas, TX 75247

Project:
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.

<table>
<thead>
<tr>
<th>Contractor:</th>
<th>Test For:</th>
<th>Material:</th>
<th>Classification:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cushion Sand</td>
<td>Tan Sand</td>
<td>Cushion Sand</td>
</tr>
<tr>
<td>Test Method:</td>
<td>ASTM D4318 Method-B</td>
<td>ASTM D698 Method-A</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Date Sampled:</th>
<th>Material Preparation:</th>
<th>Rammer Type:</th>
<th>Sampled By:</th>
<th>Sample Location:</th>
</tr>
</thead>
<tbody>
<tr>
<td>12/09/2016</td>
<td>Moist</td>
<td>Mechanical</td>
<td>Albrighton, Kregg</td>
<td>Delivered to Rone Lab by Client</td>
</tr>
</tbody>
</table>

**PROJECT DATA**

<table>
<thead>
<tr>
<th>Maximum Dry Density,pcf</th>
<th>Optimum Moisture Content, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>97.70 pcf</td>
<td>16.8 %</td>
</tr>
</tbody>
</table>

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

% Passing #200: 1

**REPORT OF TEST**

100% saturation line for SG = 2.6 Estimated

**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. (KA)
### ASTM # / TXDOT#:
NA

### Client:
Big R Bridge

### Project Name:
Aggregate Testing

### Date:
June 2, 2017

### Soil Sample Weight:
11289.4

### Source:
Client Supplied

#### Sieve No. | Cumulative Wt. Retained (g) | % Retained | % Passing | Sieve Specifications
---|---|---|---|---
2" | 0.0 | | |
1-3/4" | 0.0 | | |
1-1/2" | 83.8 | 0.01 | 99.99 |
1-1/4" | 1331.8 | 11.8 | 88.2 |
1" | 3125.1 | 27.7 | 72.3 |
3/4" | 5900.3 | 52.3 | 47.7 |
1/2" | 8826.0 | 78.2 | 21.8 |
3/8" | 9450.8 | 83.7 | 16.3 |
PAN | 10838.7 | | |

#### Decant

#### Dry Wt.+Tare

#### Washed Wt. +Tare

#### Tare Wt.

#### Decant %

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, performance, etc.) or any other feature or 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 client to whom they are addressed and shall not be reproduced except in full without the written approval of Rone Engineering Services, Ltd.
# Q/A Sieve Analysis Results

<table>
<thead>
<tr>
<th>Sieve Size (US)</th>
<th>Sieve Size (mm)</th>
<th>Sample 1 Gravel</th>
<th>Average</th>
<th>Min. Limit</th>
<th>Max. Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>38.1</td>
<td>100.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>31.8</td>
<td>88.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.3</td>
<td>25.4</td>
<td>72.3</td>
<td>72.3</td>
<td>72.3</td>
<td>72.3</td>
</tr>
<tr>
<td>1.0</td>
<td>19.0</td>
<td>60.4</td>
<td>47.7</td>
<td>47.7</td>
<td>47.7</td>
</tr>
<tr>
<td>0.8</td>
<td>16.0</td>
<td>47.7</td>
<td>21.8</td>
<td>21.8</td>
<td>21.8</td>
</tr>
<tr>
<td>1/2</td>
<td>9.5</td>
<td>16.3</td>
<td>16.3</td>
<td>16.3</td>
<td>16.3</td>
</tr>
<tr>
<td>0.8</td>
<td>6.3</td>
<td>10.0</td>
<td>10.0</td>
<td>10.0</td>
<td>10.0</td>
</tr>
<tr>
<td>0.4</td>
<td>3.2</td>
<td>2.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.2</td>
<td>1.6</td>
<td>1.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td>0.8</td>
<td>0.8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.05</td>
<td>0.3</td>
<td>0.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.02</td>
<td>0.1</td>
<td>0.1</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests:

<table>
<thead>
<tr>
<th>Coarse Grained Soils</th>
<th>Group Symbol</th>
<th>Group Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>More than 50% retained on No. 200 sieve</td>
<td>GW</td>
<td>Well-graded gravel</td>
</tr>
<tr>
<td>Gravels</td>
<td>More than 50% of coarse fraction retained on No. 4 sieve</td>
<td>GP</td>
</tr>
<tr>
<td>Clean Gravels</td>
<td>Less than 5% fines</td>
<td>GM</td>
</tr>
<tr>
<td>Gravels with Fines more than 12% fines</td>
<td>GC</td>
<td>Clayey gravel</td>
</tr>
<tr>
<td>Sands</td>
<td>50% or more of coarse fraction passes No. 4 sieve</td>
<td>SW</td>
</tr>
<tr>
<td>Clean Sands</td>
<td>Less than 5% fines</td>
<td>SP</td>
</tr>
<tr>
<td>Sands with Fines</td>
<td>More than 12% fines</td>
<td>SM</td>
</tr>
<tr>
<td>Fine-Grained Soils</td>
<td>SC</td>
<td>Clayey sand</td>
</tr>
<tr>
<td>Liquid limit less than 50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silts and Clays</td>
<td>Inorganic</td>
<td>Pi &gt; 7 and plots on or above “A” line</td>
</tr>
<tr>
<td>Organic</td>
<td>Liquid limit - oven dried</td>
<td>CH</td>
</tr>
<tr>
<td>Liquid limit - not dried</td>
<td>OL</td>
<td>Organic silt</td>
</tr>
<tr>
<td>&lt; 0.75</td>
<td>Organic silt</td>
<td></td>
</tr>
<tr>
<td>Organic</td>
<td>Liquid limit - oven dried</td>
<td>OH</td>
</tr>
<tr>
<td>Liquid limit - not dried</td>
<td>Organic silt</td>
<td></td>
</tr>
</tbody>
</table>

Highly organic soils | Primarily organic matter, dark in color, and organic odor | P | Peat |

---

*Based on the material passing the 3-in. (75-mm) sieve

*If field sample contained cobbles or boulders, or both, add “with cobbles or boulders, or both” to group name.

*Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

*Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SC poorly graded sand with silt, SP-GC poorly graded sand with clay.

*Cu = 3e/Dw 

*If fines are organic, add “with organic fines” to group name.

*If soil contains > 15% gravel, add “with gravel” to group name.

*If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

*If soil contains 15 to 20% plus No. 200, add “with sand” or “with gravel,” whichever is predominant.

*If soil contains > 30% plus No. 200 predominantly sand, add “sandy” to group name.

*If soil contains > 30% plus No. 200, predominantly gravel, add “gravely” to group name.

*Pi = 4 and plots on or above “A” line

*Pi plots below “A” line

*Pi plots below “A” line

*Pi plots below “K” line
BACKFILL GRADATION

Percent Finer Than vs. Diameter (mm)

- GRAVEL
- SAND
- FINES

Legend:
- Red: Boundry
- Green: Boundry
- Purple: Gravel
**Large Scale Direct Shear Test (ASTM D3080 Modified)**

Client: Big R Bridge  
Project: Large Scale Direct Shear Testing

**Gravel**

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>-</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>psf</td>
<td>625</td>
<td>1,250</td>
<td>2,500</td>
<td>5,000</td>
</tr>
<tr>
<td>Box Edge Dimension</td>
<td>in</td>
<td>12</td>
<td>12</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td>Bearing Slide Resistance</td>
<td>lbs</td>
<td>14</td>
<td>20</td>
<td>32</td>
<td>56</td>
</tr>
</tbody>
</table>

**Peak**

<table>
<thead>
<tr>
<th></th>
<th>Normal Stress</th>
<th>Shear Stress</th>
<th>Secant Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test</td>
<td>psf</td>
<td>lbs</td>
<td>deg.</td>
</tr>
<tr>
<td>Test - Peak</td>
<td>715</td>
<td>1,438</td>
<td>63.6</td>
</tr>
<tr>
<td>Test - Large Displacement</td>
<td>1,442</td>
<td>2,684</td>
<td>61.8</td>
</tr>
</tbody>
</table>

**Large Displacement**

<table>
<thead>
<tr>
<th></th>
<th>Normal Stress</th>
<th>Shear Stress</th>
<th>Secant Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test</td>
<td>psf</td>
<td>lbs</td>
<td>deg.</td>
</tr>
<tr>
<td>Test - Peak</td>
<td>833</td>
<td>1,667</td>
<td>55.5</td>
</tr>
<tr>
<td>Test - Large Displacement</td>
<td>1,212</td>
<td>2,375</td>
<td>54.9</td>
</tr>
</tbody>
</table>

**Test Conditions**

- **Upper:** Gravel, Tamp in Place
- **Lower:** Maximum Particle Size - 1.5 inch minus
- **Conditioning:** Wet - Loading applied and Interface flooded for a minimum of 2 hours prior to shear.
- **Shearing Rate:** inches/minute 0.04

**Test Notes**

1) Modification - Testing was conducted in a 12" x 12" x 7" interface friction box. For the provided and unmodified/unscaled 1.5 inch minus material, the ASTM criteria of a minimum height of six times the maximum particle size was satisfied. Please also note that the ASTM criteria for the linear dimension of the box plan area is ten times the maximum particle size or a maximum particle size of 1.2 inches for this device. Accordingly, the tested 1.5 inch material would constitute a modified procedure.

2) Presented Shear Strength Parameters - Please note that the provided friction angle and cohesion are based on a linear regression across all stresses. Please note that the linear regression parameters do not necessarily represent the measure shear stress under all levels of normal stresses. Accordingly, please note the presented per-stress secant angles for peak and large displacement as well as the presentation of minimum secant values for both peak and large displacement.

**Note** - Large Displacement Values Reported for 3.0 inches of Displacement

---

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested. TRI neither accepts responsibility for nor makes claims as to the final use and purpose of the material. TRI observes and maintains client confidentiality. TRI limits reproduction of this report, except in full, without prior approval of TRI.
**Relative Density Testing per ASTM D 4253 and D 4254**

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Dry Density (pcf)</td>
<td>112.5</td>
</tr>
<tr>
<td>Minimum Dry Density (pcf)</td>
<td>88.1</td>
</tr>
</tbody>
</table>

Shawn Hutcherson, P.E.  2/28/2017

Quality Review/Date

Tested by: Kahlil Hart
Appendix – B

Pullout Testing Results
Figure 8-1 Displacement vs Force for Sand
Figure 8-2 Displacement vs Force for Gravel
Soil Reinforcing Interface Shear
1.3 OTHER COMPONENTS
IDEA Submittal
Mechanically Stabilized Earth
Grid-Strip Soil Reinforcing

[This Page Is Intentionally left Blank]
1.3.1

Other component Innovations
1.3.2

Reinforced Soil Properties
BACKFILL

QUALITY CONTROL SPECIFICATION

Stabilized Earth Wall System
The information set forth in this Manual, including but not limited to all technical and engineering data, figures, tables, designs, drawings, details, procedures, and specifications are intended for information pertaining to this project. Every effort has been made to ensure the Manual 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 any and all liability resulting from such use.
CONTENTS

1 General ........................................................................................................................................................ 1

2 Referenced Specifications ............................................................................................................................ 1
   2.1 American society for Testing and Materials (ASTM) ............................................................................. 1
      2.1.1 C117 - Material finer than No. 200 sieve in Mineral Aggregate by Washing ........................... 1
      2.1.2 C136 - Sieve Analysis of Fine and Coarse Aggregate .................................................................... 1
      2.1.3 D1248 - Standard Specification for Polyethylene Plastics Extrusion Materials ........................ 1
      2.1.4 D1556 - Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone
                      Method ................................................................................................................................. 1
      2.1.5 D2922 - Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth) ..... 1
      2.1.6 D3034 - Standard Specification for Type PSM Poly(Vinyl Chloride) (PVC) Sewer Pipe and
                      Fittings ...................................................................................................................................... 1
      2.1.7 D3080 - Direct Shear Test of Soils Consolidated Drained Conditions ........................................... 1
   2.2 American Association of State Highway and Transportation Officials .................................................. 1
      2.2.1 M208 - Standard Specification for Geotextile Specification for Highway Applications ............ 1
      2.2.2 T-27 - Sieve Analysis of Fine and Coarse Aggregate .................................................................. 1
      2.2.3 T11 - Materials Finer than No. 200 Sieve in Mineral Aggregate by Washing ........................... 1
      2.2.4 T90 - Determining the Plastic Limit and Plasticity Index in Soils .............................................. 1
      2.2.5 T99 - Moisture-Density Relations of Soils Using a 5.5 lb Rammer and 12 Inch Drop ............... 1
      2.2.6 T104 - Standard Method of Test for Soundness of Aggregate by Use of Sodium Sulfate or
                      Magnesium Sulfate ................................................................................................................ 1
      2.2.7 T180 - Moisture-Density Relations of Soils Using a 10 lb Rammer and 18 Inch Drop ............... 1
      2.2.8 T191 - Density of Soil in Place by the Sand Cone Method ........................................................... 1
      2.2.9 T236 - Direct Shear Test of Soils Consolidated Drained Conditions .......................................... 1
      2.2.10 T238 - Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth) ....... 1
      2.2.11 T267 - Determination of Organic Content in Soils by Loss on Ignition ..................................... 2
      2.2.12 T288 - Determining Minimum Laboratory Soil Resistivity ....................................................... 2
      2.2.13 T289 - Determining pH of Soil in Use in Corrosion Testing ...................................................... 2
      2.2.14 T290 - Determining Water Soluble Sulfate Ion Content in Soil ................................................. 2
      2.2.15 T291 - Determining Water Soluble Chloride Ion Content in Soil ............................................. 2

3 SELECT GRANULAR BACKFILL ....................................................................................................................... 2
   3.1 Gradation ............................................................................................................................................ 2
   3.2 Additional Requirements ..................................................................................................................... 2
      3.2.1 Plasticity Index .............................................................................................................................. 2
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.2.2</td>
<td>Fine Material</td>
<td>2</td>
</tr>
<tr>
<td>3.2.3</td>
<td>Internal Friction Angle</td>
<td>2</td>
</tr>
<tr>
<td>3.2.4</td>
<td>Soundness</td>
<td>2</td>
</tr>
<tr>
<td>3.2.5</td>
<td>Electrochemical Requirements</td>
<td>2</td>
</tr>
<tr>
<td>3.2.6</td>
<td>Certification</td>
<td>3</td>
</tr>
<tr>
<td>3.2.7</td>
<td>Rejection</td>
<td>3</td>
</tr>
<tr>
<td>3.2.8</td>
<td>Sampling</td>
<td>3</td>
</tr>
<tr>
<td>4</td>
<td>Drainage Aggregate</td>
<td>3</td>
</tr>
<tr>
<td>4.1</td>
<td>Gradation</td>
<td>3</td>
</tr>
<tr>
<td>4.2</td>
<td>Drainage Pipe</td>
<td>3</td>
</tr>
<tr>
<td>4.3</td>
<td>Filter Fabric</td>
<td>3</td>
</tr>
<tr>
<td>5</td>
<td>Construction Requirements</td>
<td>3</td>
</tr>
<tr>
<td>5.1</td>
<td>Technical Assistance</td>
<td>3</td>
</tr>
<tr>
<td>5.2</td>
<td>Site Preparation</td>
<td>4</td>
</tr>
<tr>
<td>5.3</td>
<td>Foundation Preparation</td>
<td>4</td>
</tr>
<tr>
<td>5.4</td>
<td>Leveling Course</td>
<td>4</td>
</tr>
<tr>
<td>5.5</td>
<td>Wall Installation</td>
<td>4</td>
</tr>
<tr>
<td>5.5.1</td>
<td>Panel Placement</td>
<td>4</td>
</tr>
<tr>
<td>5.5.2</td>
<td>Soil Reinforcement Placement</td>
<td>4</td>
</tr>
<tr>
<td>5.5.3</td>
<td>Backfill Placement</td>
<td>4</td>
</tr>
<tr>
<td>5.5.3.1</td>
<td>Lift Thickness</td>
<td>4</td>
</tr>
<tr>
<td>5.5.3.2</td>
<td>Moisture Content</td>
<td>5</td>
</tr>
<tr>
<td>5.5.3.3</td>
<td>Compaction Reports</td>
<td>5</td>
</tr>
<tr>
<td>5.5.3.4</td>
<td>Compaction of 3-Foot Zone</td>
<td>5</td>
</tr>
<tr>
<td>5.5.3.5</td>
<td>Unsuitable Backfill</td>
<td>5</td>
</tr>
<tr>
<td>5.5.3.6</td>
<td>Construction Equipment</td>
<td>5</td>
</tr>
<tr>
<td>5.5.3.7</td>
<td>End of Day Precautions</td>
<td>5</td>
</tr>
</tbody>
</table>
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

2.2 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**

<table>
<thead>
<tr>
<th>U.S. Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>3&quot;</td>
<td>100</td>
</tr>
<tr>
<td>No. 40</td>
<td>0-60</td>
</tr>
<tr>
<td>No. 200</td>
<td>0-15</td>
</tr>
</tbody>
</table>

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**

<table>
<thead>
<tr>
<th>Property</th>
<th>Requirement</th>
<th>Test Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resistivity</td>
<td>&gt;3000 ohm-cm</td>
<td>AASHTO T-288-91</td>
</tr>
<tr>
<td>pH</td>
<td>5-10</td>
<td>AASHTO T-289-91</td>
</tr>
<tr>
<td>Chlorides</td>
<td>&lt;100 ppm</td>
<td>AASHTO T-291-91</td>
</tr>
<tr>
<td>Sulfates</td>
<td>&lt;200 ppm</td>
<td>AASHTO T-290-91</td>
</tr>
<tr>
<td>Organic Content</td>
<td>&lt;1%</td>
<td>AASHTO T-267-86</td>
</tr>
</tbody>
</table>

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**

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 1/2&quot;</td>
<td>100</td>
</tr>
<tr>
<td>1&quot;</td>
<td>75-100</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>50-75</td>
</tr>
<tr>
<td>No. 4</td>
<td>0-60</td>
</tr>
<tr>
<td>No. 40</td>
<td>0-50</td>
</tr>
<tr>
<td>No. 200</td>
<td>0-5</td>
</tr>
</tbody>
</table>

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 ft² 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
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 THE BACK OF THE PANEL.
4. THERE SHALL BE A 12" OVERLAP BETWEEN SPLICED FILTER FABRIC.
5. HORIZONTAL JOINT FABRIC SHALL BE EXTENDED 6" PASSED VERTICAL JOINT.
6. IT IS RECOMMENDED THAT OVERLAPPED FILTER FABRIC IS SHINGLED IN A DOWN DIRECTION.

BACK FACE ELEVATION
FILTER FABRIC PLACEMENT
DRAINAGE OF SELECT BACKFILL
TYPICAL CROSS SECTION
DRAINAGE OF SELECT BACKFILL
AT FACE OF MSE MASS

NOTE:
1. DRAINAGE PIPE SHALL BE ENCASED IN CRUSHED STONE
2. CRUSHED STONE SHALL BE WRAPPED IN FILTER FABRIC IN ACCORDANCE AASHTO M294.
3. CRUSHED ROCK SHALL BE FREE DRAINING AND CONTAIN LESS THAN 5% PASSING #200 SIEVE
4. DRAINAGE PIPE SHALL SLOPE TO DRAIN OUTSIDE OF THE MSE FOUNDATION FOOTPRINT
5. FINISHED GRADE AT TOP OF STRUCTURE SHALL CONSIST OF AN IMPERVIOUS MEMBRANE
TYPICAL CROSS SECTION
DRAINAGE OF SELECT BACKFILL
AT BASE OF MSE MASS

RETAINING WALL OFFSET FROM BASE LINE OF SURVEY
BASE LINE OF SURVEY

PROPOSED FINISH GRADE

HALF CONNECTOR COPING

FRONT FACE OF RETAINING WALL
- OFFSETS ARE FROM THIS CONTROL LINE

STANDARD TAB OR CONNECTOR (TYPICAL)

PROPOSED FINISH GRADE

GRID-STRIP SOIL REINFORCING (TYPICAL)

DRAINAGE PIPE

2'-0" MINIMUM EMBEDMENT OR AS PER CONTRACT DRAWINGS

1'-0" X 0'-0" UNREINFORCED CONCRETE LEVELING PAD

12" OF CRUSHED ROCK

FOUNDATION

SOIL REINFORCING LENGTH AS REC'D

LIMITS OF REINFORCED VOLUME

RANDOM BACKFILL

NOTE:
1. DRAINAGE PIPE SHALL BE ENCASED IN CRUSHED STONE
2. CRUSHED STONE SHALL BE WRAPPED IN FILTER FABRIC IN ACCORDANCE AASHTO M298.
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

DRAINAGE BLANKET 8" TO 12" OF CRUSHED ROCK SEPARATED BY GRS-TEXTILE FABRIC

DRAINAGE DETAILS
BLANKET (BASE) DRAIN

Special Detail Drawing
NOTE:
1. DRAINAGE PIPE SHALL BE ENCASED IN CRUSHED ROCK
2. CRUSHED ROCK SHALL BE WRAPPED IN FILTER FABRIC IN ACCORDANCE AASHTO M286
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 IMPERVIOUS MEMBRANE
6. SLOPE OF CHIMNEY DRAIN TO BE DETERMINED ON A PROJECT BASIS

TYPICAL CROSS SECTION
DRAINAGE OF RANDOM BACKFILL
AT BACK OF MSE MASS
1.3.4
ERS Coping
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.
**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.

**FACE OF COPING**

**CONCRETE RR8**

(BY OTHERS)

**LONGITUDINAL BAR #4**

(TYP.) - 4 PLACES

**NOMINAL FACE OF PANEL**

**#4 BARS H**

@ 11 3/8" O.C.

**TOP OF COPING**

**LONGITUDINAL BAR #4**

(TYPICAL PROCEDURE)

**LEVEL-UP CONCRETE**

**BAR U - #4 @ 11 3/8" O.C.**

**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.
PRECAST COPING W/C.I.P. TRAFFIC BARRIER

PARTIAL SECTION

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.
1.3.5
ERS Traffic Barrier
1.3.6
ERS Abutments
1.3.7
ERS Slip Joints
2.0
ERS DESIGN
2.1 DESIGN METHODOLOGY
2.1.1
ERS Design Innovations
2.1.2
ERS AASHTO LRFD Design Methodology
2.1.3
ERS Proprietary Design Methodologies
2.1.4
ERS Facing Design Requirements
Subject: Facing Units
Segmental Concrete Panel
Welded Wire Mesh Panel Reinforcing

STABILIZED EARTH WALL

REINFORCED 5X5 CONCRETE PANEL DESIGN

FEM METHOD
The following report presents Vistawall System’s (VAWS) design methodology for Mechanically Stabilized Earth retaining structures and for the analysis of the Stabilized Earth™ Wall - Segmental Concrete Panel. The outlined method is consistent with the 7th Edition of the American Association of State Highway and Transportation Officials LRFD Bridge Design Specifications, 2014 with Interim Revisions.

The following methodology is proprietary to VAWS. It is consistent with the Simplified Coherent Gravity design procedure utilizing the Load and Resistance Factored Design. The panel force analysis was completed using the software program RISA-3D. The panel loads were determined using VAWS design methodology. The panel analysis is consistent with the AASHTO, LRFD. Section 5 - Concrete Structures.

Design Methodology Outline

1. Determine Maximum Applied Pressure on Panel
2. Perform Finite Element Analysis to Determine Maximum Moment
3. Determine Concrete Panel Parameters
4. Determine Panel Reinforcing Parameters
5. Analyze Vertical Concrete Reinforcing
   a. Calculate Maximum Moment
   b. Verify Flexural Requirements
6. Analyze Horizontal Concrete Reinforcing
   a. Calculate Maximum Moment
   b. Verify Flexural Requirements
7. Detail Panel Reinforcing
**Project Name:** MSE Submittal – Panel Analysis  
**Project Location:** LRFD - Binder  
**Project Number:** SCP with Grid-Strip  
**Date:** 06/24/2016  
**Design Code:** AASHTO LRFD  
**Design Method:** Simplified CG

### Maximum Applied Pressure on Panel

The maximum applied pressure on the segmental concrete panel is determined using standard Mechanically Stabilized Earth design procedures. The earth pressure is distributed to the back of the panel using the maximum applied pressure as determined at a maximum depth.

\[
H = 40.00 \\
\phi_{lv} = 1.35 \text{ (AASHTO Table 3.4.1-2.2)} \\
\gamma_m = 0.125 \text{ kcf} \\
\phi_m = 34 \text{ degrees} \\
K_a = 0.283 \text{ (AASHTO C11.10.6.2.1-1)} \\
\gamma_q = 0.125 \text{ kcf} \\
q = 2.00' \\
K_a = \tan^2\left(45^\circ - \frac{\phi_m}{2}\right) \rightarrow \tan \left(45^\circ - \frac{30^\circ}{2}\right) = 0.333 \\
\sigma_n = 1.2K_a \cdot \phi_{lv} \cdot (\gamma \cdot H + \gamma_q \cdot q) \rightarrow (1.2 \cdot 0.283) \cdot 1.35 \cdot \left\{ (0.125 \cdot \text{kcf}) \cdot (40.00 \text{ft}) + (0.125 \cdot \text{kcf}) \cdot (2.00 \text{ft}) \right\} = 2.41 \cdot \text{ksf} \\
\text{AASHTO 11.10.6.2.1-1} \\
\text{Note: The 1.2 Factor is the K-ratio value at a depth of 20 feet and below.}

### Finite Element Analysis

The finite element model for the Vistawall Stabilized Earth Wall Segmental Concrete Panel is proportioned into a series of 400 plates. The plates in the vertical (Y) direction and the horizontal (X) direction are portioned in uniform dimensions equal to 3 inches for an overall panel height and width equal to 60 inches.
<table>
<thead>
<tr>
<th>Project Name:</th>
<th>MSE Submittal – Panel Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Location:</td>
<td>LRFD - Binder</td>
</tr>
<tr>
<td>Project Number:</td>
<td>SCP with Grid-Strip</td>
</tr>
<tr>
<td>Date:</td>
<td>06/24/2016</td>
</tr>
<tr>
<td>Design Code:</td>
<td>AASHTO LRFD</td>
</tr>
<tr>
<td>Design Method:</td>
<td>Simplified CG</td>
</tr>
</tbody>
</table>

The SEW panel is supported on two compressible bearing pads located at the quarter points. The reactions at this location are modeled as two-way springs and are restrained in the Y-direction. The spring constant is set equal to 10 kip-inch. To assure that the structure is stable, the top left corner of the panel is restrained from translation in the X direction and from rotation about the Z-axis with two way springs. The spring constant for both of these conditions is set equal to 0.010 kip-inch and 10 kip-ft/rad respectively.

The SEW panels have panel anchors extending from their back face that are used to connect the soil reinforcing to the panel. The panel anchors are always placed so they are symmetric about the centerline of the panel. For this model a group of 8 panel anchors are assumed in two rows of 4 anchors each. The rows of panel anchors are located at 7.5 inches from each side edge and spaced 15” on center. The rows are located at 15 inches and 45 inches from the bottom of the panel. The anchors are modeled as fixed reactions restrained from translation in Z- the direction.

![Plan View](image1.png) ![Section View](image2.png)

**Typical Grid-Strip Anchor Spacing**

A surface load is applied to the face of each plate. This load is based on the earth pressure load as determined in the calculations. This distributed load is applied as a uniform pressure and has the units of pounds per square inch (psi).
Once the structure is solved, the moments about the X and Y axis are determined. Based on the solved structure the maximum horizontal and vertical moments for the respective row and column are determined. These moments are then used to determine the design moment in the vertical and horizontal directions. (Reference the RISA-3D Plate Forces spreadsheet summary and Design Matrix for Stabilized Earth Panel).

**Concrete Panel Parameters**

The concrete panel parameters are based on typical Department of Transportation’s concrete requirements for MSE segmental concrete panels in addition to the VAWS SEW systems details.

<table>
<thead>
<tr>
<th>Concrete Panel Parameters</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength of Concrete</td>
<td>$f'_c$</td>
</tr>
<tr>
<td>Unit Weight of Concrete</td>
<td>$\gamma_c$</td>
</tr>
<tr>
<td>Resistance Factor for Flexure</td>
<td>$\phi$</td>
</tr>
<tr>
<td>Panel Thickness</td>
<td>$t$</td>
</tr>
<tr>
<td>Panel Width</td>
<td>$b$</td>
</tr>
<tr>
<td>Panel Anchors</td>
<td>4/row</td>
</tr>
</tbody>
</table>

**Panel Reinforcing Parameters**

The panel reinforcing parameters are based on typical Department of Transportation reinforcing requirements for MSE segmental concrete panels in addition to the VAWS SEW systems details.
**Panel Reinforcing Parameters**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Strength of Steel (ASTM A82)</td>
<td>$F_y$</td>
<td>65000 psi</td>
</tr>
<tr>
<td>Modulus of Elasticity</td>
<td>$E_s$</td>
<td>28000 ksi</td>
</tr>
<tr>
<td>Minimum Clear cover</td>
<td>$d_{\text{clear}}$</td>
<td>2.0 inches</td>
</tr>
<tr>
<td>Minimum Edge Distance</td>
<td>$S_{\text{edge}}$</td>
<td>3.0 inches</td>
</tr>
</tbody>
</table>

**Panel Reinforcing Calculations**

The panel reinforcing calculations are consistent with AASHTO 7th Edition LRFD Bridge Design specification Section 5, Concrete Structures. The panel reinforcing used by the VAWS SEW system consists of Welded Wire Fabric (WWF) size W4.5 x W7.0. There are 14 bars in the vertical direction and 9 bars in the horizontal direction. The panel reinforcing configuration is designed so bars are 3.75”
<table>
<thead>
<tr>
<th>Project Name:</th>
<th>MSE Submittal – Panel Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Location:</td>
<td>LRFD - Binder</td>
</tr>
<tr>
<td>Project Number:</td>
<td>SCP with Grid-Strip</td>
</tr>
<tr>
<td>Date:</td>
<td>06/24/2016</td>
</tr>
<tr>
<td>Design Code:</td>
<td>AASHTO LRFD</td>
</tr>
<tr>
<td>Design Method:</td>
<td>Simplified CG</td>
</tr>
</tbody>
</table>

apart at each possible panel anchor location as detailed above. The reinforcing is placed in the panel so the vertical bars are closest to the back face of the panel.

**Design Process**

- The required design area of steel is calculated and compared to the actual area of steel.
- The maximum reinforcing is calculated and compared to the actual reinforcing.
- The distribution of flexural reinforcing is verified against the actual distribution of reinforcing.
- The minimum reinforcing is calculated and compared to the actual reinforcing.
<table>
<thead>
<tr>
<th><strong>Project Name:</strong></th>
<th>MSE Submittal – Panel Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Project Location:</strong></td>
<td>LRFD - Binder</td>
</tr>
<tr>
<td><strong>Project Number:</strong></td>
<td>SCP with Grid-Strip</td>
</tr>
<tr>
<td><strong>Date:</strong></td>
<td>06/24/2016</td>
</tr>
<tr>
<td><strong>Design Code:</strong></td>
<td>AASHTO LRFD</td>
</tr>
<tr>
<td><strong>Design Method:</strong></td>
<td>Simplified CG</td>
</tr>
</tbody>
</table>

Panel Detail

![Structural Model (Isometric Back Face View)](image)

650 Justice Lane
Mansfield, TX 76120
888-280-9858
7/1/2016
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**RISA-3D - Structural Model (Back Face View)**

1. 16.7 psi surface load on back of panel
2. Anchor location modeled as support restrained in Z direction
3. Bearing Pad support is 2 way spring
4. Upper right corner restrained for stability
Project Name: MSE Submittal – Panel Analysis
Project Location: LRFD - Binder
Project Number: SCP with Grid-Strip
Date: 06/24/2016
Design Code: AASHTO LRFD
Design Method: Simplified CG

RISA3D MOMENTS FOR PANEL MODEL

*Note the axes shown above are local axis and not global axis
### RISA-3D PLATE FORCE RESULTS

<table>
<thead>
<tr>
<th>Plate</th>
<th>Mx</th>
<th>Width</th>
<th>3 in</th>
<th>lb-in</th>
</tr>
</thead>
<tbody>
<tr>
<td>P6</td>
<td>919.400 (lb-in)/in</td>
<td>3 in</td>
<td>2758.200 lb-in</td>
<td></td>
</tr>
<tr>
<td>P26</td>
<td>1032.768 (lb-in)/in</td>
<td>3 in</td>
<td>3098.304 lb-in</td>
<td></td>
</tr>
<tr>
<td>P46</td>
<td>1320.626 (lb-in)/in</td>
<td>3 in</td>
<td>3961.878 lb-in</td>
<td></td>
</tr>
<tr>
<td>P66</td>
<td>1802.449 (lb-in)/in</td>
<td>3 in</td>
<td>5407.347 lb-in</td>
<td></td>
</tr>
<tr>
<td>P86</td>
<td>2613.372 (lb-in)/in</td>
<td>3 in</td>
<td>7840.116 lb-in</td>
<td></td>
</tr>
<tr>
<td>P106</td>
<td>2612.802 (lb-in)/in</td>
<td>3 in</td>
<td>7838.406 lb-in</td>
<td></td>
</tr>
<tr>
<td>P126</td>
<td>1801.936 (lb-in)/in</td>
<td>3 in</td>
<td>5405.808 lb-in</td>
<td></td>
</tr>
<tr>
<td>P146</td>
<td>1324.362 (lb-in)/in</td>
<td>3 in</td>
<td>3973.086 lb-in</td>
<td></td>
</tr>
<tr>
<td>P166</td>
<td>1049.570 (lb-in)/in</td>
<td>3 in</td>
<td>3148.710 lb-in</td>
<td></td>
</tr>
<tr>
<td>P186</td>
<td>925.957 (lb-in)/in</td>
<td>3 in</td>
<td>2777.871 lb-in</td>
<td></td>
</tr>
<tr>
<td>P206</td>
<td>925.957 (lb-in)/in</td>
<td>3 in</td>
<td>2777.871 lb-in</td>
<td></td>
</tr>
<tr>
<td>P226</td>
<td>1049.570 (lb-in)/in</td>
<td>3 in</td>
<td>3148.710 lb-in</td>
<td></td>
</tr>
<tr>
<td>P246</td>
<td>1324.362 (lb-in)/in</td>
<td>3 in</td>
<td>3973.086 lb-in</td>
<td></td>
</tr>
<tr>
<td>P266</td>
<td>1801.935 (lb-in)/in</td>
<td>3 in</td>
<td>5405.805 lb-in</td>
<td></td>
</tr>
<tr>
<td>P286</td>
<td>2612.802 (lb-in)/in</td>
<td>3 in</td>
<td>7838.406 lb-in</td>
<td></td>
</tr>
<tr>
<td>P306</td>
<td>2613.372 (lb-in)/in</td>
<td>3 in</td>
<td>7840.116 lb-in</td>
<td></td>
</tr>
<tr>
<td>P326</td>
<td>1802.449 (lb-in)/in</td>
<td>3 in</td>
<td>5407.347 lb-in</td>
<td></td>
</tr>
<tr>
<td>P346</td>
<td>1320.626 (lb-in)/in</td>
<td>3 in</td>
<td>3961.878 lb-in</td>
<td></td>
</tr>
<tr>
<td>P366</td>
<td>1035.278 (lb-in)/in</td>
<td>3 in</td>
<td>3105.834 lb-in</td>
<td></td>
</tr>
<tr>
<td>P386</td>
<td>914.400 (lb-in)/in</td>
<td>3 in</td>
<td>2743.200 lb-in</td>
<td></td>
</tr>
<tr>
<td></td>
<td>30804 (lb-in)/in</td>
<td></td>
<td>92412 lb-in</td>
<td>7.70 ft-kip</td>
</tr>
</tbody>
</table>

### MOMENTS FOR ANCHORS AT MID-POINT OF ANCHORS (Mₓ)

---

VISTAWALL SYSTEMS

650 Justice Lane
Mansfield, TX 76120

888-280-9858
7/1/2016
**Project Name:** MSE Submittal – Panel Analysis  
**Project Location:** LRFD - Binder  
**Project Number:** SCP with Grid-Strip  
**Date:** 06/24/2016  
**Design Code:** AASHTO LRFD  
**Design Method:** Simplified CG

### RISA-3D PLATE FORCE RESULTS

<table>
<thead>
<tr>
<th>Plate</th>
<th>My</th>
<th>Width</th>
<th>Width</th>
</tr>
</thead>
<tbody>
<tr>
<td>P101</td>
<td>921.494 (lb-in)/in</td>
<td>3 in</td>
<td>2764.482 lb-in</td>
</tr>
<tr>
<td>P102</td>
<td>1037.322 (lb-in)/in</td>
<td>3 in</td>
<td>3111.966 lb-in</td>
</tr>
<tr>
<td>P103</td>
<td>1323.077 (lb-in)/in</td>
<td>3 in</td>
<td>3969.231 lb-in</td>
</tr>
<tr>
<td>P104</td>
<td>1802.987 (lb-in)/in</td>
<td>3 in</td>
<td>5408.961 lb-in</td>
</tr>
<tr>
<td>P105</td>
<td>2606.014 (lb-in)/in</td>
<td>3 in</td>
<td>7818.042 lb-in</td>
</tr>
<tr>
<td>P106</td>
<td>2605.441 (lb-in)/in</td>
<td>3 in</td>
<td>7816.323 lb-in</td>
</tr>
<tr>
<td>P107</td>
<td>1802.501 (lb-in)/in</td>
<td>3 in</td>
<td>5407.503 lb-in</td>
</tr>
<tr>
<td>P108</td>
<td>1326.951 (lb-in)/in</td>
<td>3 in</td>
<td>3980.853 lb-in</td>
</tr>
<tr>
<td>P109</td>
<td>1051.900 (lb-in)/in</td>
<td>3 in</td>
<td>3155.700 lb-in</td>
</tr>
<tr>
<td>P110</td>
<td>928.059 (lb-in)/in</td>
<td>3 in</td>
<td>2784.177 lb-in</td>
</tr>
<tr>
<td>P111</td>
<td>928.059 (lb-in)/in</td>
<td>3 in</td>
<td>2784.177 lb-in</td>
</tr>
<tr>
<td>P112</td>
<td>1051.900 (lb-in)/in</td>
<td>3 in</td>
<td>3155.700 lb-in</td>
</tr>
<tr>
<td>P113</td>
<td>1326.951 (lb-in)/in</td>
<td>3 in</td>
<td>3980.853 lb-in</td>
</tr>
<tr>
<td>P114</td>
<td>1802.501 (lb-in)/in</td>
<td>3 in</td>
<td>5407.503 lb-in</td>
</tr>
<tr>
<td>P115</td>
<td>2605.444 (lb-in)/in</td>
<td>3 in</td>
<td>7816.323 lb-in</td>
</tr>
<tr>
<td>P116</td>
<td>2606.014 (lb-in)/in</td>
<td>3 in</td>
<td>7818.042 lb-in</td>
</tr>
<tr>
<td>P117</td>
<td>1802.987 (lb-in)/in</td>
<td>3 in</td>
<td>5408.961 lb-in</td>
</tr>
<tr>
<td>P118</td>
<td>1323.077 (lb-in)/in</td>
<td>3 in</td>
<td>3969.231 lb-in</td>
</tr>
<tr>
<td>P119</td>
<td>1037.322 (lb-in)/in</td>
<td>3 in</td>
<td>3111.966 lb-in</td>
</tr>
<tr>
<td>P120</td>
<td>921.494 (lb-in)/in</td>
<td>3 in</td>
<td>2764.482 lb-in</td>
</tr>
<tr>
<td></td>
<td>30811 (lb-in)/in</td>
<td>92434 lb-in</td>
<td>7.70 ft-kip</td>
</tr>
</tbody>
</table>

**MOMENTS FOR ANCHORS AT BOTTOM OF PANEL (M_y)**
**Project Name:** MSE Submittal – Panel Analysis

**Project Location:** LRFD - Binder

**Project Number:** SCP with Grid-Strip

**Design Code:** AASHTO LRFD

**Design Method:** Simplified CG

**Date:** 06/24/2016

### Design Assumptions:

1. \( f'c = 4000 \text{ psi} \)
2. Unit weight of concrete 150 pcf
3. The maximum anchor load for a GS11 at 75 year design life is used. The design area of GS11 is 0.160 in\(^2\). The maximum allowable force is equal to 7.8 kips per GS11. (Note 100 year design life will produce a smaller design moment)
4. Load Factor for design force is equal to 1.35
5. W4.5 x W7.0 – 3.75” x 6.00” panel reinforcing with vertical bar 2” from back face of panel
6. Edge cover is a minimum of 3” (all around)
7. Anchor is embed a minimum of 4” into panel.
8. Horizontal panel reinforcing based on sum of moments for plates P101 to P120 (incremented by 1)
9. Vertical panel reinforcing based on sum of moments for plates P1 to P386 (incremented by 20)
<table>
<thead>
<tr>
<th><strong>Project Name:</strong></th>
<th>MSE Submittal – Panel Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Project Location:</strong></td>
<td>LRFD - Binder</td>
</tr>
<tr>
<td><strong>Project Number:</strong></td>
<td>SCP with Grid-Strip</td>
</tr>
<tr>
<td><strong>Design Code:</strong></td>
<td>AASHTO LRFD</td>
</tr>
</tbody>
</table>

**APPENDIX A**

**PANEL CALCULATIONS**
<table>
<thead>
<tr>
<th>Project Name:</th>
<th>MSE Submittal – Panel Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Location:</td>
<td>LRFD - Binder</td>
</tr>
<tr>
<td>Project Number:</td>
<td>SCP with Grid-Strip</td>
</tr>
<tr>
<td>Design Code:</td>
<td>AASHTO LRFD</td>
</tr>
</tbody>
</table>

650 Justice Lane
Mansfield, TX 76120
888-280-9858
7/1/2016
**VAWS SEW 5X5 PANEL - DESIGN ANALYSIS**

<table>
<thead>
<tr>
<th>Project Name:</th>
<th>Ohio Department of Transportation – Panel Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Location:</td>
<td>Ohio Product Submittal</td>
</tr>
<tr>
<td>Project Number:</td>
<td>DSGN By: TPT</td>
</tr>
<tr>
<td>Design Code:</td>
<td>CHK By: CMS</td>
</tr>
<tr>
<td>Design Method:</td>
<td>Date: 02-06-15</td>
</tr>
</tbody>
</table>

**Define Concrete Panel Parameters:**

- Panel Thickness: \( h := 6.0 \cdot \text{in} \)
- Panel Height: \( Z_{\text{pan}} := 5 \cdot \text{ft} \)
- Panel Length: \( b := 60 \cdot \text{in} \)
- Panel Anchor Vertical Spacing: \( S_{v,\text{anch}} := 2.50 \cdot \text{ft} \)
- Concrete compressive strength: \( f_c := 4000 \cdot \text{psi} \)
- Strength reduction factor (flexure): \( \phi := 0.90 \)
- Strength reduction factor (shear): \( \phi_s := 0.85 \)
- Unit weight of concrete: \( \gamma_c := 150 \cdot \text{pcf} \)

**Define Panel Reinforcing Parameters (W4.5 x W7.0 - 3.75" x 6")**

- Concrete Cover: \( C_{\text{cov}} := 2.00 \cdot \text{in} \)
- Steel Yield Stress: \( f_y := 65 \cdot \text{ksi} \)
- Vertical Reinforcing Bar Area (W4.5): \( A_{rv} := 0.045 \cdot \text{in}^2 \)

Vertical Reinforcing Bar Diameter (W4.5)

\[
d_{vb} := \sqrt{\frac{4 \cdot A_{rv}}{\pi}} = 0.239 \cdot \text{in}
\]

Vertical Reinforcing Spacing: \( S_{vb} := 3.75 \cdot \text{in} \)
Number of Vertical Reinforcing Bars (Reference attached detail)

\[ n_{bv} := \text{floor} \left( \frac{b - C_{cov}}{S_{vb}} \right) - 1 = 14 \]

Horizontal Reinforcing Bar Area (W7.0)

\[ A_{rh} := 0.070 \cdot \text{in}^2 \]

Horizontal Reinforcing Bar Diameter (W7.0)

\[ d_{hb} := \sqrt{\frac{4 \cdot A_{rh}}{\pi}} = 0.299 \cdot \text{in} \]

Horizontal Reinforcing Spacing

\[ S_{hb} := 6 \cdot \text{in} \]

Number of Horizontal Reinforcing Bars (Reference attached detail)

\[ n_{bh} := \text{floor} \left( \frac{Z_{pan} - C_{cov}}{S_{hb}} \right) = 9 \]

**VERTICAL REINFORCING STEEL DESIGN ANALYSIS**

Moment Results as Calculated by RISA3D

\[ M_{rv,\text{actual}} := 7.70 \cdot \text{ft} \cdot \text{kip} \]

Effective depth (Vertical)

\[ d_{ev} := h - C_{cov} - \frac{d_{vb}}{2} = 3.88 \cdot \text{in} \]

Vertical reinforcing area of steel per panel

\[ A_{sv} := \frac{\pi \cdot d_{vb}^2}{4} \cdot n_{bv} = 0.63 \cdot \text{in}^2 \]

Moment Capacity Provided by Vertical Steel

\[ M_{rv,\text{prov}} := \phi \cdot A_{sv} \cdot f_y \cdot \left[ d_{ev} - \frac{1}{2} \cdot \left( \frac{A_{sv} \cdot f_y}{0.85 \cdot f_c \cdot b} \right) \right] = 11.609 \cdot \text{ft} \cdot \text{kip} \]

\[ \text{MOMENT}_{\text{provided,v}} := \begin{cases} 
"OK" & \text{if } M_{rv,\text{prov}} \geq M_{rv,\text{actual}} = "OK" \\
"N.G." & \text{otherwise}
\end{cases} \]

Distance of Extreme Fiber

\[ d_{cv} := h - d_{ev} = 2.12 \cdot \text{in} \]
Depth of Rectangular Stress Distribution
\[ a := \frac{A_{SV} \cdot f_y}{0.85 \cdot f_c \cdot b} = 0.201 \cdot \text{in} \]

Moment arm for ideally reinforced section
\[ d_{ev} - \frac{a}{2} \]
\[ j_v := \frac{d_{ev}}{d_{ev}} = 0.974 \]

Unfactored Moment
\[ M_v := \frac{M_{rv,\text{actual}}}{1.5} = 5.133 \cdot \text{ft} \cdot \text{kip} \]

Tensile stress of steel reinforcing in panel
\[ f_{ss,v} := \frac{M_v}{j_v \cdot d_{ev} \cdot A_{SV}} = 25.867 \cdot \text{ksi} \]
\[ 0.6 \cdot f_y = 39 \cdot \text{ksi} \]

\[ \text{Tensile stress}_v := \begin{cases} 
\text{"OK"} & \text{if } f_{ss,v} \leq 0.6 \cdot f_y = \text{"OK"} \\
\text{"N.G."} & \text{otherwise} 
\end{cases} \]

5.7.3.4 Control of Cracking by Distribution of Reinforcement

Exposure factor (Limits Crack Width)
\[ \gamma_e := 1 \]

Flexural Strain Ratio (LRFD 5.7.3.4-1)
\[ \beta_{sv} := 1 + \frac{d_{cv}}{0.7 \cdot (h - d_{cv})} = 1.78 \]

Maximum spacing to satisfy LRFD 5.7.3.4-1 (Only required for layer closest to tension face)
\[ \text{spacing} := \frac{700 \cdot \frac{\text{kip}}{\text{ft}} \cdot \gamma_e \cdot 12 \cdot \beta_{sv} \cdot f_{ss,v}}{\beta_{sv} \cdot f_{ss,v}} - 2 \cdot d_{cv} = 10.96 \cdot \text{in} \]

\[ \text{LRFD5.7.3.4.1_v} := \begin{cases} 
\text{"SATISFIED"} & \text{if } \text{spacing} \geq S_{vb} = \text{"SATISFIED"} \\
\text{"N.G."} & \text{otherwise} 
\end{cases} \]
Maximum spacing to satisfy LRFD 5.10.3.2

\[
S_{\text{max}} := 1.5 \cdot h = 9 \cdot \text{in}
\]

LRFD_{5.10.3.2_v} := \begin{cases} 
"\text{SATISFIED}" & \text{if } S_{vb} \leq 1.5 \cdot h = "\text{SATISFIED}" \\
"\text{N.G.}" & \text{otherwise} 
\end{cases}

5.7.3.3.2 Minimum Reinforcement

Modulus of Rupture LRFD 5.4.2.6

\[
f_r := 0.37 \cdot \sqrt{f_c \cdot \text{ksi}} = 0.74 \cdot \text{ksi}
\]

Section Modulus for extreme fiber

\[
S_c := \frac{1}{6} \cdot b \cdot h^2 = 0.208 \cdot \text{ft}^3
\]

Cracking moment check to satisfy LRFD 5.7.3.4.2

\[
M_{cr} := f_r \cdot S_c = 22.2 \cdot \text{kip} \cdot \text{ft}
\]

1.2 Times the Cracking Moment

\[
1.2 \cdot M_{cr} = 26.64 \cdot \text{kip} \cdot \text{ft}
\]

Factored flexural resistance

\[
M_{rv,\text{actual}} = 7.7 \cdot \text{kip} \cdot \text{ft}
\]

LRFD_{5.7.3.3.2} := \begin{cases} 
"\text{SATISFIED}" & \text{if } 1.2 \cdot M_{cr} \geq M_{rv,\text{actual}} = "\text{SATISFIED}" \\
"\text{N.G.}" & \text{otherwise} 
\end{cases}

HORIZONTAL REINFORCING STEEL DESIGN ANALYSIS (Layer furthest from Tension Face)

Moment Results as Calculated by RISA3D................................................... \( M_{rh,\text{actual}} := 7.70 \cdot \text{ft} \cdot \text{kip} \)

Effective depth (Horizontal)

\[
d_{eh} := h - C_{cov} - d_{vb} - \frac{d_{hb}}{2} = 3.611 \cdot \text{in}
\]

Horizontal reinforcing area of steel per panel

\[
A_{sh} := \frac{\pi \cdot d_{hb}^2}{4} \cdot n_{bh} = 0.63 \cdot \text{in}^2
\]
Moment Capacity Provided by Horizontal Steel

\[ M_{rh,prov} := \phi \cdot A_{sh} \cdot f_y \cdot \left[ d_{eh} - \frac{1}{2} \cdot \left( \frac{A_{sh} \cdot f_y}{0.85 \cdot f_c \cdot b} \right) \right] = 10.783 \cdot \text{ft} \cdot \text{kip} \]

\[ \text{MOMENT}_{\text{provided}_h} := \begin{cases} \text{"OK"} & \text{if } M_{rh,prov} \geq M_{rh,actual} = \text{"OK"} \\ \text{"N.G."} & \text{otherwise} \end{cases} \]

Distance of Extreme Fiber

\[ d_{ch} := h - d_{eh} = 2.389 \cdot \text{in} \]

Depth of Rectangular Stress Distribution

\[ a_h := \frac{A_{sh} \cdot f_y}{0.85 \cdot f_c \cdot b} = 0.201 \cdot \text{in} \]

Moment arm for ideally reinforced section

\[ d_{eh} - \frac{a_h}{2} \]

\[ j_h := \frac{d_{eh} - \frac{a_h}{2}}{d_{eh}} = 0.972 \]

Unfactored Moment

\[ M_h := \frac{M_{rh,actual}}{1.5} = 5.133 \cdot \text{ft} \cdot \text{kip} \]

Tensile stress of steel reinforcing in panel

\[ f_{ss,h} := \frac{M_h}{j_h \cdot d_{eh} \cdot A_{sh}} = 27.849 \cdot \text{ksi} \]

\[ 0.6 \cdot f_y = 39 \cdot \text{ksi} \]

\[ \text{Tensile stress}_{h} := \begin{cases} \text{"OK"} & \text{if } f_{ss,h} \leq 0.6 \cdot f_y = \text{"OK"} \\ \text{"N.G."} & \text{otherwise} \end{cases} \]

5.7.3.4 Control of Cracking by Distribution of Reinforcement

Flexural Strain Ratio (LRFD 5.7.3.4-1)

\[ \beta_{sh} := 1 + \frac{d_{ch}}{0.7 \cdot (h - d_{ch})} = 1.945 \]
Maximum spacing to satisfy LRFD 5.10.3.2

\[
\text{LRFD}_{5.10.3.2} := \begin{cases} 
\text{"SATISFIED" if } S_{hb} \leq 1.5 \cdot h \text{ = "SATISFIED"} \\
\text{"N.G." otherwise} 
\end{cases}
\]

5.7.3.3.2 Minimum Reinforcement

1.2 Times the Cracking Moment

\[
1.2 \cdot M_{cr} = 26.64 \text{ kip} \cdot \text{ft}
\]

Factored flexural resistance

\[
M_{rh,\text{actual}} = 7.7 \text{ kip} \cdot \text{ft}
\]

\[
\text{LRFD}_{5.7.3.3.2} := \begin{cases} 
\text{"SATISFIED" if } 1.2 \cdot M_{cr} \geq M_{rh,\text{actual}} \text{ = "SATISFIED"} \\
\text{"N.G." otherwise} 
\end{cases}
\]
<table>
<thead>
<tr>
<th>Project Name:</th>
<th>MSE Submittal – Panel Analysis (5x10)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Location:</td>
<td>LRFD - Binder</td>
</tr>
<tr>
<td>Project Number:</td>
<td>SCP with Grid-Strip</td>
</tr>
<tr>
<td>Date:</td>
<td>06/29/2016</td>
</tr>
<tr>
<td>Design Code:</td>
<td>AASHTO LRFD</td>
</tr>
<tr>
<td>Design Method:</td>
<td>Simplified CG</td>
</tr>
</tbody>
</table>

Subject:  
Facing Units  
Segmental Concrete Panel - 5 x 10  
Welded Wire Mesh Panel Reinforcing

![Diagram of STABILIZED EARTH WALL]

**REINFORCED 5X10 CONCRETE PANEL DESIGN**

**FEM METHOD**

---

650 Justice Lane  
Mansfield, TX 76120  
888-280-9858  
7/1/2016
The following report presents Vistawall System’s (VAWS) design methodology for Mechanically Stabilized Earth retaining structures and for the analysis of the Stabilized Earth™ Segmental Concrete Panel. The outlined method is consistent with the 7th Edition of the American Association of State Highway and Transportation Officials LRFD Bridge Design Specifications, 2014, and interim specifications.

The following methodology is proprietary to VAWS. It is consistent with the Simplified Coherent Gravity design procedure utilizing the Load and Resistance Factored Design. The panel force analysis was completed using the software program RISA-3D. The panel loads were determined using VAWS design methodology. The panel analysis is consistent with the AASHTO, LRFD. Section 5 - Concrete Structures.

**Design Methodology Outline:**

1. Determine Maximum Applied Force at Each Anchor
2. Perform Finite Element Analysis to Determine Maximum Moment
3. Determine Concrete Panel Parameters
4. Determine Panel Reinforcing Parameters
5. Vertical Concrete Reinforcing
   a. Calculate Maximum Moment
   b. Verify Flexural Requirements
6. Horizontal Concrete Reinforcing
   a. Calculate Maximum Moment
   b. Verify Flexural Requirements
7. Detail Panel Reinforcing

**Maximum Applied Pressure on Panel**

The maximum applied force at each anchor is based on standard Mechanically Stabilized Earth design procedures and AASHTO corrosion models. The maximum force at each anchor location is a function of
the retaining structure height, soil parameters and the soil reinforcing configuration. The standard Grid-Strip, GS11 consist of 2, 2-inch space longitudinal wires and a series of 3” to 4” wide transvers wires that are spaced at 6” or 12” on center. The longitudinal and transvers wires consist of 65 ksi steel. The ends of the longitudinal wires are mechanically attached to the TAB or BAR connector by resistance welding. The steel yield resistance factor is 0.75.

75 year Design Life Surficial Steel $E_s = 0.056 \cdot \text{in}$  
(100 year design life is less conservative)

Area of Grid-Strip $A_3 = 2 \cdot \frac{\pi \cdot (d - E_s)}{4} = \frac{\pi \cdot (0.374(\text{in}) - 0.056(\text{in}))}{4} = 0.16 \cdot \text{in}^2$

Maximum Design Tension per GS11 $T_{\text{max}} = \phi_r \cdot F_y \cdot A_3 \rightarrow 0.75 \cdot 65(\text{ksi}) \cdot 0.16(\text{in}^2) = 7.75 \cdot \text{kip}$

**Maximum Tensile Capacity for Standard Grid Strip**

The SEW panels have panel anchors extending from their back face that are used to connect the Grid-Strip soil reinforcing to the panel. For standard panels, the panel anchors are always placed so they are symmetric about the center-line of the panel. For this model a group of 8 panel anchors are assumed in two rows. The rows of panel anchors are located at 15 inches and 45 inches from the bottom of the panel and are spaced 15” on center and 7.5” from each edge in each row. The anchors are modeled as fixed reactions restrained from translation in Z- the direction.
The load that is applied to the concrete panel is modeled as a surface load. The surface load is calculated based on standard MSE design principles in accordance with Section 11.10.6.2. The pressure is a function of the structure height, soil unit weight and friction angle, and all externally applied loads.

\[
\sigma_h = 1.2 \cdot K_a \cdot \gamma_{EV} \cdot (\gamma_m \cdot H + \gamma_q \cdot q)
\]

Where:
- \( \sigma_h \) = horizontal pressure (ksf)
- \( K_a \) = active earth pressure coefficient (dim)
- \( \gamma_{EV} \) = load factor for vertical earth pressure (dim)
- \( \gamma_m \) = soil unit weight (kcf)
- \( H \) = structure height (ft)
- \( \gamma_q \) = live load unit weight (kcf)
- \( q \) = equivalent height of soil for live load (ft)

Using a structure height of 40.00 feet, a soil unit weight equal to 0.125 kcf, soil internal friction angle of 34 degrees, and a live load equal to 0.250 ksf, the horizontal pressure applied to the back of the panel is equal to 2.41 ksf (16.7 psi.)
**Calculation of Horizontal Earth Pressure**

Finite Element Analysis

The finite element model for the VAWS Stabilized Earth Wall Segmental Concrete Panel is proportioned into a series of 512 plates. The plates in the vertical (Y) direction and the horizontal (X) direction are portioned in uniform dimensions equal to 3.75 inches for an overall panel height equal to 60" and a panel width equal to 120 inches. (Reference the RISA-3D joint coordinate spreadsheet for the location of the joints and plates).

The SEW panel is supported on four compressible bearing pads located at the quarter points. The reactions at this location are modeled as two way springs. The reactions are restrained in the Y-
direction. The spring constant is set equal to 10 kip-inch. To assure that the structure is stable, the top left corner of the panel is restrained from translation in the X direction and from rotation about the Z-axis with two way springs. The spring constant for both of these conditions is set equal to 0.010 kip-inch and 10 kip-ft/rad respectively.

Typical Grid-Strip Anchor Spacing

The panel structure is solved and the moments about the X and Y axis are determined. Based on the solved panel structure the maximum horizontal and vertical moments for the respective row and column are determined. The maximum moments are summed for each row and column. These moments are then used to determine the design moment in the vertical and horizontal directions. (Reference the RISA-3D Plate Forces spreadsheet summary and Design Matrix for Stabilized Earth Panel).

Concrete Panel Parameters

The concrete panel parameters are based on typical Department of Transportation’s concrete requirements for MSE segmental concrete panels in addition to the VAWS SEW systems details.
**Project Name:** MSE Submittal – Panel Analysis (5x10)  
**Project Location:** LRFD - Binder  
**Project Number:** SCP with Grid-Strip  
**Date:** 06/29/2016  
**Design Code:** AASHTO LRFD  
**Design Method:** Simplified CG

### Concrete Panel Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength of Concrete</td>
<td>( f'_c )</td>
<td>4000 psi</td>
</tr>
<tr>
<td>Unit Weight of Concrete</td>
<td>( \gamma_c )</td>
<td>150 pcf</td>
</tr>
<tr>
<td>Resistance Factor for Flexure</td>
<td>( \phi )</td>
<td>0.90</td>
</tr>
<tr>
<td>Panel Thickness</td>
<td>( t )</td>
<td>6.0 inches</td>
</tr>
<tr>
<td>Panel Width</td>
<td>( b )</td>
<td>120.0 inches</td>
</tr>
<tr>
<td>Panel Anchors</td>
<td>6</td>
<td>6.0 inches on center</td>
</tr>
</tbody>
</table>

### Panel Reinforcing Parameters

The panel reinforcing parameters are based on typical Department of Transportation reinforcing requirements for MSE segmental concrete panels in addition to the VAWS SEW systems details.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Strength of Steel (ASTM A82)</td>
<td>( F_y )</td>
<td>65000 psi</td>
</tr>
<tr>
<td>Modulus of Elasticity</td>
<td>( E_s )</td>
<td>28000 ksi</td>
</tr>
<tr>
<td>Minimum Clear cover</td>
<td>( d_{\text{clear}} )</td>
<td>2.0 inches</td>
</tr>
<tr>
<td>Minimum Edge Distance</td>
<td>( S_{\text{edge}} )</td>
<td>3.0 inches</td>
</tr>
</tbody>
</table>
The panel reinforcing calculations are consistent with AASHTO 7th Edition LRFD Bride Design specification Section 5, Concrete Structures. The panel reinforcing used by the VAWS SEW system consists of Welded Wire Fabric (WWF) W4.5V x W7.0H. There are 32 bars in the vertical and 10 bars in horizontal direction. The reinforcing is placed in the panel so the vertical bars are closest to the back face of the panel.

**Design Steps**

- The required design area of steel is calculated and compared to the actual area of steel.
- The maximum reinforcing is calculated and compared to the actual reinforcing.
- The distribution of flexural reinforcing is verified against the actual distribution of reinforcing
- The minimum reinforcing is calculated and compared to the actual reinforcing.
Project Name: MSE Submittal – Panel Analysis (5x10)
Project Location: LRFD - Binder
Project Number: SCP with Grid-Strip  Date: 06/29/2016
Design Code: AASHTO LRFD  Design Method: Simplified CG

Panel Detail

---

Structural Model

---

Isometric

VISTA WALL SYSTEMS

650 Justice Lane  888-280-9858
Mansfield, TX 76120  7/1/2016
1. 16.7 psi surface load on all plates
2. Reactions located at the panel anchors and are in the “Z” direction
3. Bearing Pad support is modeled as a 2 way spring at 15”, 30”, 60” and 90”
4. Upper right corner restrained for stability in X-direction and rotationally in Z-direction
<table>
<thead>
<tr>
<th><strong>Project Name:</strong></th>
<th>MSE Submittal – Panel Analysis (5x10)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Project Location:</strong></td>
<td>LRFD - Binder</td>
</tr>
<tr>
<td><strong>Project Number:</strong></td>
<td>SCP with Grid-Strip</td>
</tr>
<tr>
<td><strong>Date:</strong></td>
<td>06/29/2016</td>
</tr>
<tr>
<td><strong>Design Code:</strong></td>
<td>AASHTO LRFD</td>
</tr>
<tr>
<td><strong>Design Method:</strong></td>
<td>Simplified CG</td>
</tr>
</tbody>
</table>

![Diagram of Vertical Reinforcing](image1)

**Vertical Reinforcing**

![Diagram of Horizontal Reinforcing](image2)

**Horizontal Reinforcing**
VERTICAL PANEL REINFORCING

<table>
<thead>
<tr>
<th>Plate</th>
<th>My</th>
</tr>
</thead>
<tbody>
<tr>
<td>P129</td>
<td>1282.883</td>
</tr>
<tr>
<td>P130</td>
<td>1562.851</td>
</tr>
<tr>
<td>P131</td>
<td>1569.877</td>
</tr>
<tr>
<td>P132</td>
<td>1277.077</td>
</tr>
<tr>
<td>P133</td>
<td>1265.515</td>
</tr>
<tr>
<td>P134</td>
<td>1534.525</td>
</tr>
<tr>
<td>P135</td>
<td>1531.183</td>
</tr>
<tr>
<td>P136</td>
<td>1254.922</td>
</tr>
<tr>
<td>P137</td>
<td>1257.871</td>
</tr>
<tr>
<td>P138</td>
<td>1540.992</td>
</tr>
<tr>
<td>P139</td>
<td>1542.215</td>
</tr>
<tr>
<td>P140</td>
<td>1261.924</td>
</tr>
<tr>
<td>P141</td>
<td>1262.917</td>
</tr>
<tr>
<td>P142</td>
<td>1545.451</td>
</tr>
<tr>
<td>P143</td>
<td>1545.79</td>
</tr>
<tr>
<td>P144</td>
<td>1264.01</td>
</tr>
<tr>
<td>P145</td>
<td>1264.01</td>
</tr>
<tr>
<td>P146</td>
<td>1545.79</td>
</tr>
<tr>
<td>P147</td>
<td>1545.451</td>
</tr>
<tr>
<td>P148</td>
<td>1262.917</td>
</tr>
<tr>
<td>P149</td>
<td>1261.924</td>
</tr>
<tr>
<td>P150</td>
<td>1542.215</td>
</tr>
<tr>
<td>P151</td>
<td>1540.992</td>
</tr>
<tr>
<td>P152</td>
<td>1257.871</td>
</tr>
<tr>
<td>P153</td>
<td>1254.922</td>
</tr>
<tr>
<td>P154</td>
<td>1531.183</td>
</tr>
<tr>
<td>P155</td>
<td>1534.525</td>
</tr>
<tr>
<td>P156</td>
<td>1265.515</td>
</tr>
<tr>
<td>P157</td>
<td>1277.077</td>
</tr>
<tr>
<td>P158</td>
<td>1569.877</td>
</tr>
<tr>
<td>P159</td>
<td>1562.851</td>
</tr>
<tr>
<td>P160</td>
<td>1282.883</td>
</tr>
<tr>
<td>Sum</td>
<td>45000.006</td>
</tr>
</tbody>
</table>

PLATE MOMENTS (M_y)

MOMENTS AT MID-POINT OF ANCHORS (M_y)
HORIZONTAL PANEL REINFORCING

RISA3D MOMENTS FOR PANEL MODEL

PLATE MOMENTS (Mx)

Note: The moment for this load condition is low due to the unsupported length between the anchors being so close.

MOMENTS AT MID-POINT OF ANCHORS (Mx)

<table>
<thead>
<tr>
<th>Plate</th>
<th>Mx</th>
</tr>
</thead>
<tbody>
<tr>
<td>P3</td>
<td>58.271</td>
</tr>
<tr>
<td>P35</td>
<td>121.397</td>
</tr>
<tr>
<td>P67</td>
<td>282.302</td>
</tr>
<tr>
<td>P99</td>
<td>625.183</td>
</tr>
<tr>
<td>P131</td>
<td>625.129</td>
</tr>
<tr>
<td>P163</td>
<td>282.181</td>
</tr>
<tr>
<td>P195</td>
<td>121.544</td>
</tr>
<tr>
<td>P227</td>
<td>58.038</td>
</tr>
<tr>
<td>P259</td>
<td>58.038</td>
</tr>
<tr>
<td>P291</td>
<td>121.544</td>
</tr>
<tr>
<td>P323</td>
<td>282.181</td>
</tr>
<tr>
<td>P355</td>
<td>625.129</td>
</tr>
<tr>
<td>P387</td>
<td>625.183</td>
</tr>
<tr>
<td>P419</td>
<td>282.302</td>
</tr>
<tr>
<td>P451</td>
<td>121.397</td>
</tr>
<tr>
<td>P483</td>
<td>58.271</td>
</tr>
<tr>
<td>Sum</td>
<td>4348.09</td>
</tr>
<tr>
<td></td>
<td>1.36</td>
</tr>
</tbody>
</table>
Design Assumptions:

1. \( f'c = 4000 \text{ psi} \)
2. Unit weight of concrete 150 pcf
3. The maximum anchor load for a GS11 at 75 year design life is used. The design area of GS11 is 0.160 in\(^2\). The maximum allowable force is equal to 7.8 kips per GS11. (Note 100 year design life will produce a smaller design moment)
4. Load Factor for design force is equal to 1.35
5. W4.5 \( \times \) W7.0 – 3.75” \( \times \) 6.00” panel reinforcing with vertical bar 2” from back face of panel
6. Edge cover is a minimum of 3” (all around)
7. Anchor is embed a minimum of 4” into panel.
8. Horizontal panel reinforcing based on sum of moments for plates P129 to P160 (incremented by 1)
9. Vertical panel reinforcing based on sum of moments for plates P3 to P483 (incremented by 32)
<table>
<thead>
<tr>
<th>Project Name:</th>
<th>MSE Submittal – Panel Analysis (5x10)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Location:</td>
<td>LRFD - Binder</td>
</tr>
<tr>
<td>Project Number:</td>
<td>SCP with Grid-Strip</td>
</tr>
<tr>
<td>Date:</td>
<td>01/10/2014</td>
</tr>
<tr>
<td>Design Code:</td>
<td>AASHTO LRFD</td>
</tr>
<tr>
<td>Design Method:</td>
<td>Simplified CG</td>
</tr>
</tbody>
</table>

**APPENDIX A**

**PANEL CALCULATIONS**
<table>
<thead>
<tr>
<th><strong>Project Name:</strong></th>
<th>MSE Submittal – Panel Analysis (5x10)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Project Location:</strong></td>
<td>LRFD - Binder</td>
</tr>
<tr>
<td><strong>Project Number:</strong></td>
<td>SCP with Grid-Strip</td>
</tr>
<tr>
<td><strong>Date:</strong></td>
<td>01/10/2014</td>
</tr>
<tr>
<td><strong>Design Code:</strong></td>
<td>AASHTO LRFD</td>
</tr>
<tr>
<td><strong>Design Method:</strong></td>
<td>Simplified CG</td>
</tr>
</tbody>
</table>
### VAWS SEW 5X10 PANEL - DESIGN ANALYSIS

**Project Name:** Ohio Department of Transportation – LRFD Infinite Backslope  
**Project Location:** Ohio Product Submittal  
**Project Number:**  
**Design Code:** AASHTO  
**Design Method:** LRFD  
**DSGN By:** TPT  
**Date:** 02-06-15  
**CHECK By:** CMS  
**Date:** 02-06-15

#### Define Concrete Panel Parameters:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness</td>
<td>h := 6.0 \cdot \text{in}</td>
</tr>
<tr>
<td>Panel Height</td>
<td>Z_{\text{pan}} := 5 \cdot \text{ft}</td>
</tr>
<tr>
<td>Panel Length</td>
<td>b := 120 \cdot \text{in}</td>
</tr>
<tr>
<td>Panel Anchor Vertical Spacing</td>
<td>S_{v,\text{anch}} := 2.50 \cdot \text{ft}</td>
</tr>
<tr>
<td>Concrete compressive strength</td>
<td>f_c := 4000 \cdot \text{psi}</td>
</tr>
<tr>
<td>Strength reduction factor (flexure)</td>
<td>\phi := 0.90</td>
</tr>
<tr>
<td>Strength reduction factor (shear)</td>
<td>\phi_s := 0.85</td>
</tr>
<tr>
<td>Unit weight of concrete</td>
<td>\gamma_c := 150 \cdot \text{pcf}</td>
</tr>
</tbody>
</table>

#### Define Panel Reinforcing Parameters (W4.5 x W7.0 - 3.75'' x 6'')

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Cover</td>
<td>C_{\text{cov}} := 2.00 \cdot \text{in}</td>
</tr>
<tr>
<td>Steel Yield Stress</td>
<td>f_y := 65 \cdot \text{ksi}</td>
</tr>
<tr>
<td>Vertical Reinforcing Bar Area</td>
<td>A_{rV} := 0.050 \cdot \text{in}^2</td>
</tr>
</tbody>
</table>

#### Vertical Reinforcing Bar Diameter (W5.0)

\[
d_{vb} := \sqrt{\frac{4 \cdot A_{rV}}{\pi}} = 0.252 \cdot \text{in}
\]

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Reinforcing Spacing</td>
<td>S_{vb} := 3.75 \cdot \text{in}</td>
</tr>
</tbody>
</table>
Number of Vertical Reinforcing Bars (Reference attached detail)

\[ n_{bv} := \text{floor} \left[ \frac{(b - C_{cov})}{S_{vb}} \right] - 1 = 30 \]

Horizontal Reinforcing Bar Area (W7.0) .................................................................

\[ A_{rh} := 0.070 \cdot \text{in}^2 \]

Horizontal Reinforcing Bar Diameter (W7.0)

\[ d_{hb} := \sqrt{\frac{4 \cdot A_{rh}}{\pi}} = 0.299 \cdot \text{in} \]

Horizontal Reinforcing Spacing .................................................................

\[ S_{hb} := 6 \cdot \text{in} \]

Number of Horizontal Reinforcing Bars (Reference attached detail)

\[ n_{bh} := \text{floor} \left[ \frac{(Z_{pan} - C_{cov})}{S_{hb}} \right] = 9 \]

**VERTICAL REINFORCING STEEL DESIGN ANALYSIS**

Moment Results as Calculated by RISA3D .............................................................

\[ M_{rv.actual} := 14.1 \cdot \text{ft} \cdot \text{kip} \]

Effective depth (Vertical)

\[ d_{ev} := h - C_{cov} - \frac{d_{vb}}{2} = 3.874 \cdot \text{in} \]

Vertical reinforcing area of steel per panel

\[ A_{sv} := \frac{\pi \cdot d_{vb}^2}{4} \cdot n_{bv} = 1.5 \cdot \text{in}^2 \]

Moment Capacity Provided by Vertical Steel

\[ M_{rv.prov} := \phi \cdot A_{sv} \cdot f_y \cdot \left[ d_{ev} - \frac{1}{2} \cdot \left( \frac{A_{sv} \cdot f_y}{0.85 \cdot f_c \cdot b} \right) \right] = 27.454 \cdot \text{ft} \cdot \text{kip} \]

\[ \text{MOMENT}_{provided,v} := \begin{cases} "OK" & \text{if } M_{rv.prov} \geq M_{rv.actual} \Rightarrow "OK" \\ "N.G." & \text{otherwise} \end{cases} \]

Distance of Extreme Fiber

\[ d_{cv} := h - d_{ev} = 2.126 \cdot \text{in} \]
Depth of Rectangular Stress Distribution
\[ a := \frac{A_{SV} \cdot f_y}{0.85 \cdot f_c \cdot b} = 0.239 \cdot \text{in} \]

Moment arm for ideally reinforced section
\[ d_{ev} - \frac{a}{2} \]
\[ j_v := \frac{d_{ev}}{d_{ev}} = 0.969 \]

Unfactored Moment
\[ M_v := \frac{M_{rv,\text{actual}}}{1.35} = 10.444 \cdot \text{ft} \cdot \text{kip} \]

Tensile stress of steel reinforcing in panel
\[ f_{SS, v} := \frac{M_v}{j_v \cdot d_{ev} \cdot A_{SV}} = 22.256 \cdot \text{ksi} \]
\[ 0.6 \cdot f_y = 39 \cdot \text{ksi} \]

Tensile stress of steel reinforcing in panel
\[ \text{Tensile stress of steel reinforcing in panel} := \begin{cases} "OK" & \text{if } f_{SS, v} \leq 0.6 \cdot f_y = "OK" \\ "N.G." & \text{otherwise} \end{cases} \]

5.7.3.4 Control of Cracking by Distribution of Reinforcement

Exposure factor (Limits Crack Width)
\[ \gamma_e := 1 \]

Flexural Strain Ratio (LRFD 5.7.3.4-1)
\[ \beta_{SV} := 1 + \frac{d_{cv}}{0.7 \cdot (h - d_{cv})} = 1.784 \]

Maximum spacing to satisfy LRFD 5.7.3.4-1 (Only required for layer closest to tension face)
\[ \text{spacing} := \frac{700 \cdot \frac{\text{kip}}{\text{ft}} \cdot \gamma_e \cdot 12}{\beta_{SV} \cdot f_{SS, v} - 2 \cdot d_{cv}} = 13.377 \cdot \text{in} \]

LRFD5.7.3.4.1_v := \begin{cases} "SATISFIED" & \text{if spacing} \geq S_{vb} = "SATISFIED" \\ "N.G." & \text{otherwise} \end{cases}
Maximum spacing to satisfy LRFD 5.10.3.2

\[ S_{\text{max}} := 1.5 \cdot h = 9 \cdot \text{in} \]

LRFD5.10.3.2_v := 
\begin{align*}
"\text{SATISFIED}" & \quad \text{if} \quad S_{vb} \leq 1.5 \cdot h \quad \Rightarrow "\text{SATISFIED}" \\
"\text{N.G.}" & \quad \text{otherwise}
\end{align*}

5.7.3.3.2 Minimum Reinforcement

Modulus of Rupture LRFD 5.4.2.6

\[ f_r := 0.37 \cdot \sqrt{f_c \cdot \text{ksi}} = 0.74 \cdot \text{ksi} \]

Section Modulus for extreme fiber

\[ S_c := \frac{1}{6} \cdot b \cdot h^2 = 0.417 \cdot \text{ft}^3 \]

Cracking moment check to satisfy LRFD 5.7.3.4.2

\[ M_{cr} := f_r \cdot S_c = 44.4 \cdot \text{kip} \cdot \text{ft} \]

1.2 Times the Cracking Moment

\[ 1.2 \cdot M_{cr} = 53.28 \cdot \text{kip} \cdot \text{ft} \]

Factored flexural resistance

\[ M_{rv,\text{actual}} = 14.1 \cdot \text{kip} \cdot \text{ft} \]

LRFD5.7.3.3.2 := 
\begin{align*}
"\text{SATISFIED}" & \quad \text{if} \quad 1.2 \cdot M_{cr} \geq M_{rv,\text{actual}} \quad \Rightarrow "\text{SATISFIED}" \\
"\text{N.G.}" & \quad \text{otherwise}
\end{align*}

HORIZONTAL REINFORCING STEEL DESIGN ANALYSIS (Layer furthest from Tension Face)

Moment Results as Calculated by RISA3D....................................................... \[ M_{rh,\text{actual}} := 1.35 \cdot \text{ft} \cdot \text{kip} \]

Effective depth (Horizontal)

\[ d_{eh} := h - C_{cov} - d_{vb} - \frac{d_{hb}}{2} = 3.598 \cdot \text{in} \]

Horizontal reinforcing area of steel per panel

\[ A_{sh} := \frac{\pi \cdot d_{hb}^2}{4} \cdot n_{bh} = 0.63 \cdot \text{in}^2 \]
### Moment Capacity Provided by Horizontal Steel

\[
M_{rh,\text{prov}} := \phi \cdot A_{sh} \cdot f_y \cdot \left[ d_{eh} - \frac{1}{2} \left( \frac{A_{sh} \cdot f_y}{0.85 \cdot f_c \cdot b} \right) \right] = 10.898 \cdot \text{ft} \cdot \text{kip}
\]

\[
\text{MOMENT}_{\text{provided}_h} := \begin{cases} 
"OK" & \text{if } M_{rh,\text{prov}} \geq M_{rh,\text{actual}} \\
"N.G." & \text{otherwise}
\end{cases}
\]

### Distance of Extreme Fiber

\[
d_{ch} := h - d_{eh} = 2.402 \cdot \text{in}
\]

### Depth of Rectangular Stress Distribution

\[
a_h := \frac{A_{sh} \cdot f_y}{0.85 \cdot f_c \cdot b} = 0.1 \cdot \text{in}
\]

### Moment arm for ideally reinforced section

\[
j_h := \frac{d_{eh} - a_h}{2} = 0.986
\]

### Unfactored Moment

\[
M_h := \frac{M_{rh,\text{actual}}}{1.35} = 1 \cdot \text{ft} \cdot \text{kip}
\]

### Tensile stress of steel reinforcing in panel

\[
f_{ss_h} := \frac{M_h}{j_h \cdot d_{eh} \cdot A_{sh}} = 5.368 \cdot \text{ksi}
\]

\[0.6 \cdot f_y = 39 \cdot \text{ksi}\]

\[
\text{Tensile stress}_h := \begin{cases} 
"OK" & \text{if } f_{ss_h} \leq 0.6 \cdot f_y \\
"N.G." & \text{otherwise}
\end{cases}
\]

### 5.7.3.4 Control of Cracking by Distribution of Reinforcement

Flexural Strain Ratio (LRFD 5.7.3.4-1)

\[
\beta_{sh} := 1 + \frac{d_{ch}}{0.7 \cdot (h - d_{ch})} = 1.953
\]
Maximum spacing to satisfy LRFD 5.10.3.2

\[
\text{LRFD}_{5.10.3.2_{h}} := \begin{cases} 
\text{"Satisfied"} & \text{if } S_{h} \leq 1.5 \cdot h \\
\text{"N.G."} & \text{otherwise}
\end{cases}
\]

## 5.7.3.3.2 Minimum Reinforcement

1.2 Times the Cracking Moment

\[
1.2 \cdot M_{cr} = 53.28 \cdot \text{kip} \cdot \text{ft}
\]

Factored flexural resistance

\[
M_{rh,\text{actual}} = 1.35 \cdot \text{kip} \cdot \text{ft}
\]

\[
\text{LRFD}_{5.7.3.3.2_{h}} := \begin{cases} 
\text{"Satisfied"} & \text{if } 1.2 \cdot M_{cr} \geq M_{rh,\text{actual}} \\
\text{"N.G."} & \text{otherwise}
\end{cases}
\]
2.1.5
ERS Splayed Soil Reinforcing Design Requirements
**MSE STRUCTURE – DPTS ANCHOR BENDING CALCULATION**

Skewed Soil Reinforcing

<table>
<thead>
<tr>
<th>Project Name:</th>
<th>DOT Submittals</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Location:</td>
<td>DSGN By: TPT</td>
</tr>
<tr>
<td>Project Number:</td>
<td>Date: 08-20-18</td>
</tr>
<tr>
<td>Design Code:</td>
<td>CHCK By: CDA</td>
</tr>
<tr>
<td>Design Method:</td>
<td>Date: 08-20-18</td>
</tr>
</tbody>
</table>

- **Modulus of elasticity for steel** ................................................................. \( E_s := 29000 \cdot \text{ksi} \)
- **Yield strength of steel TS anchor** ............................................................ \( \phi_s := 0.95 \)
- **Resistance factor of bending for steel** ....................................................... \( F_y := 50 \cdot \text{ksi} \)
- **Maximum tensile load for grid-strip soil reinforcing** ................................... \( T := 7.72 \cdot \text{kip} \)
- **Skew angle of soil reinforcing** ................................................................. \( \alpha := 15 \cdot \text{deg} \)
- **Tension load from skewed soil reinforcing** ................................................ \( T_x := T \cdot \cos(\alpha) = 7.46 \cdot \text{kip} \)
- **Bending force from skewed soil reinforcing** ............................................... \( T_y := T \cdot \sin(\alpha) = 2.00 \cdot \text{kip} \)

---

**6800 MANHATTAN BLVD.**
**FT WORTH, TEXAS 76120**

---

**888-280-9858**
**8/30/2018**
Thickness of TS Anchor ................................................................. \( b := 0.135 \text{ in} \)

Width of TS Anchor ................................................................. \( h := 2 \text{ in} \)

Length of TS Anchor ................................................................. \( L := 3 \text{ in} \)

Location of application of bending force ........................................ \( x := 2 \text{ in} \)

**Material Degradation Parameters**

Service Life ........................................................................................................ \( Y_t := 75 \text{ yr} \)

Thickness of Galvanized Coating ............................................................... \( g_T := 3.40 \text{ mil} \)

Galvanization Degradation First Two Years ........................................ \( g_2 := 0.59 \frac{\text{mil}}{\text{yr}} \)

Galvanization Degradation Remaining Years ......................................... \( g_R := 0.16 \frac{\text{mil}}{\text{yr}} \)

Steel Degradation ......................................................................................... \( E_{S75} := 0.48 \frac{\text{mil}}{\text{yr}} \)

Time to Complete Galvanization Loss

\[
Y_G := \frac{g_T - 2 \cdot \text{yr} \cdot g_2}{g_R} + 2 \cdot \text{yr} = 15.88 \text{ yr}
\]

Sacrificial Steel Design Life

\[
Y_S := \text{if} (Y_G < 0, Y_t, Y_t - Y_G) = 59.13 \text{ yr}
\]

Applied Sacrificial Steel Thickness

\[
E_c := \left[ (Y_t - Y_G) \cdot E_{S75} - \frac{1}{2} \right] \cdot 2 = 0.06 \text{ in}
\]

Moment of inertia of TS Anchor for bending

\[
l_b := \frac{(b - E_c) \cdot h^3}{12} = 0.05216 \text{ in}^4
\]

Plastic section modulus

\[
Z_b := \frac{(b - E_c) \cdot h^2}{4} = 0.07824 \text{ in}^3
\]
Plastic moment for TS anchor (2 plates)

\[ M_p := 2\phi_s \cdot F_y \cdot Z_b = 0.62 \cdot \text{kip} \cdot \text{ft} \]

Constants to determine bending moment

\[ k := \left( \frac{T_x}{E_s \cdot I_b} \right)^{0.5} = 0.84 \text{ ft} - 1.00 \]

\[ F_1 := \cosh(k \cdot x) = 1.01 \]

\[ F_2 := \sinh(k \cdot x) = 0.14 \]

Applied bending moment

\[ M := \frac{T_y}{k} \cdot F_2 = 0.33 \cdot \text{kip} \cdot \text{ft} \]

Capacity Demand Ratio for Bending

\[ \frac{M_p}{M} = 1.85 \]
2.1.6
ERS Vertical Obstruction Design Requirements
2.1.7
ERS Horizontal Obstruction Design Requirements
IDEA Submittal
Mechanically Stabilized Earth
Grid-Strip Soil Reinforcing

[This Page Is Intentionally left Blank]
2.2
DESIGN DRAWINGS
DEAR MR. MASSA,

The attached submittal is prepared in conformance with the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications, 7th Edition (2014), FHWA NHI-10-024/025 Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes (NHI-10-024), and Vista-A-Wall Systems and MassDOT design methodology. Please be advised of the following.

The design contained herein is based on information supplied to Big R Bridge by others. The scope of our services did not include verification, analysis, or design of the foundation, native material, or backfill material. Furthermore, Big R Bridge did not visit the site, nor did Big R Bridge determine the required elevations and heights of the incorporated MSE structures. It is the responsibility of others to verify that the elevations and stations that are given in the shop drawings are true and accurate before beginning erection of the wall system. A sliding resistance factor of 1.0 was used in accordance with the geotechnical report prepared by Nobis Engineering, Inc. (Nobis) dated February 27, 2014.

This design submittal includes standard requirements for MassDOT MSE retaining wall submittals including reference documents, design methodology, design parameters, design software verification calculations and project specific calculations. Please note that design variables in the software verification calculations may vary from project specific values. The project specific calculations which include project specific parameters are included in the last section of this submittal.

Global stability analyses is included in the geotechnical report prepared by Nobis as referenced above. Big R Bridge has not independently verified these results.

Per the soil borings included within the geotechnical report, groundwater elevations are located below the bottom of proposed retaining wall footing. Should groundwater be encountered within the limits of the reinforced fill for the MSE walls, Big R Bridge and Nobis is to be notified and construction halted until appropriate design guidance is provided.

Sincerely,

David Massa
Project Manager
Lawrence-Lynch Corp.
PO Box 913
Falmouth, MA 02541

Big R Bridge
PO Box 1290
Greeley, Colorado 80632-1290
T 1 970 356 9600
F 1 970 356 9821
bigbridge.com
A minimum 6-inch thick crushed stone layer is required below the concrete leveling pads as noted in the geotechnical report. The report also recommends that drainage stone and foundation drains be installed so that hydrostatic pressures do not build behind the walls. Because most wall sections include back-to-back walls, development of hydrostatic pressure is unlikely; however, details of foundation stone and foundation drains are not included within the contract plans. We recommend the geotechnical engineer be consulted for specific requirements to be utilized. Shop drawings will be amended to include details once provided.

Sections of walls meeting the requirements of back-to-back walls were designed in accordance with section 6.4 of NHI-10-024. Maximum reinforcement lengths within the bridge approach sections of walls were reduced to a maximum length of 16.0’ so that the terminal end of reinforcing elements fit within the proposed section. Longer lengths of reinforcement for abutment walls provide minimum lengths equivalent to 0.7 of the height of structure measure from top of backwall to top of leveling pad.

Site adjusted peak acceleration for this project, A<sub>s</sub>, is 0.101g while PGA is 0.063g. In our experience, seismic design does not control design until PGA values approach 0.20g. At 0.063g PGA is considerably less than controlling limits. Per AASHTO Section 11.5.4.2, paragraph 1, seismic design is not considered mandatory for walls located where A<sub>s</sub> is less than 0.4g. Per the memorandum dated November 18, 2015 from Griffin Ryder with VHB, abutments are to be considered floating bridge abutments with longitudinal loads acting in the direction of the abutments, or into the retained volume. As such, seismic forces will act to reduce abutment wall loading such that true bridge abutment design methodology incorporating bridge loading excluding seismic loads controls the design.

Corrosion loss rates are based on the backfill material meeting or exceeding the electrochemical requirement specified in AASHTO Article 11.10.6.4.2 – Design Life Considerations. The soil reinforcing shall be fabricated in accordance with ASTM A1064 and shall be hot dip galvanized in accordance with ASTM A123 if required.

The following backfill material strength parameters were used in the design and analysis:

<table>
<thead>
<tr>
<th>Location</th>
<th>Unit Weight (pcf)</th>
<th>Friction Angle (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced Fill</td>
<td>135</td>
<td>34</td>
</tr>
<tr>
<td>Retained Fill</td>
<td>135</td>
<td>34</td>
</tr>
<tr>
<td>Foundation</td>
<td>120</td>
<td>30</td>
</tr>
</tbody>
</table>

The backfill and foundation shall be placed and compacted in accordance with the contract documents.
Big R Bridge has made general assumptions concerning the strength parameters of the foundation; select backfill material; and retained fill material as defined in the contract documents. The Contractor must verify that these design parameters are appropriate before building the MSE structure. Based on the above parameters, Big R Bridge is certifying the internal stability of the structure only. This certification is contingent on the fill material strength parameters meeting, or exceeding, the design assumptions. Furthermore, certification is contingent on the contractor meeting or exceeding the recommended installation procedures as set forth by the contract documents, and instructions as provided by Big R Bridge.

The shop drawings that are provided are extremely important. Big R Bridge requires that these documents be read by the Contractor and approving agency. These general guidelines will greatly aid the reviewer and Contractor in the approval and installation of this structure. The coping shown in our submittal is for information only. The installer shall refer to the contract documents for the steel layout and the coping dimensions.

Please review the submittal as required. Big R Bridge requires that an approved stamped calculations and shop drawings be received before fabrication of the MSE wall components or installation of the MSE wall begins.

If you require further information, clarification, or assistance, please contact this office.

Sincerely,

Christopher M. Staud, PE
Senior Engineer
Licensed PE (CO, FL, GA, KS, NH, OH, TX, VA)

cc:file VW14-02388
Experience

Founder and President of Vistawall Systems, LLC (formerly T&B Structural Systems, LLC) now a division of Big-R Bridge. Presently the Vice President and Director of Research and Development at Big-R Bridge.

- Geotechnical engineering experience in the design and analysis of reinforced soil structures, geosynthetic applications, embankments over soft soils, shallow foundations, retaining walls, traffic support systems, mining support structures, soil nail structures, tie-back structures, precast panels, traffic barriers, and 2-stage retaining walls.
- Responsible for research and development programs for pullout of soil reinforcing, pullout of panel anchors in concrete and instrumentation of full height MSE structures.
- Developed pullout testing apparatus for the National Concrete Masonry Association (NCMA) and the Texas Research Institute (TRI).
- Inventor of 25 United States and Foreign patents pertaining to earth structures.
- Author and software programmer of MSE-Pro, a proprietary design, analysis, and drafting program for Mechanically Stabilized Earth walls. Includes integration with AutoCAD and Visual Manufacturing.
- Past Technical Director of the Association of Metallically Stabilized Earth (AMSE).
- Responsible for technical consultation and overview on major engineering projects including contract negotiation and supervision of construction operations.
- Engineer of record for over 300 geotechnical projects that involved the production of construction drawings, contract specifications and calculations for heavy highway transportation, mining, and private sector projects.
- Certified NHI Instructor No-0923

Employment Experience

Big-R Bridge, 650 Justice Lane, Mansfield, TX 76063
  Vice President – Director of Research and Development
  March 2013 to Present

Vistawall Systems, L.L.C., 6800 Manhattan Blvd., Suite 304, Fort Worth, TX 76120
  Founder - Vice President
  March 2007 through February of 2013

T&B Structural Systems, L.L.C., 6800 Manhattan Blvd., Suite 304, Fort Worth, TX 76120
  Founder - President
  February 1995 through February 2007

Hilfiker Systems, LLC, 637 West Hurst Boulevard, Hurst, Texas 76024
  Engineering Manager
  November 1993 through February 1995
Robertson Roa & Associates, 106 East Church Street, Weatherford, Texas 76085
  Project Engineer
  February 1992 through November 1993
Hilfiker Texas 637 West Hurst Boulevard, Hurst, Texas 76024
  Design Engineer
  February 1991 through February 1992
SMI Joist Company PO Box 2000 Hope, Arkansas 71802
  Design Engineer
  July 1990 to February 1991

Achievement of Body of Knowledge Criteria – Slope Stability/Retaining Structures

Dr. Taylor holds a Bachelor of Science Degree in Education and a Bachelor of Science Degree in Civil Engineering from the University of Wyoming, a Master of Science Degree in Civil Engineering and a Doctor of Philosophy in Civil Engineering from the University of Texas at Arlington. He possesses a wide range of geotechnical engineering experience involving earth retaining structures, geosynthetics, embankments over soft soils, shallow foundations, MSE retaining structures, traffic support systems, mining support structures, soil nailing structures, precast concrete panel systems, traffic barriers, 2-stage MSE retaining walls, and pavement structures. Dr. Taylor has been the engineer of record for over 500 projects that involved the production of construction drawings, contract specifications and calculations. He has over 30 United States and Foreign patents covering earth retaining structures. He has written and implemented a proprietary design, analysis, and drafting program for Mechanically Stabilized Earth walls. He has been responsible for technical consultation and overview on major engineering projects in both the US and abroad and has been involved in contract negotiation and supervision of construction operations. Dr. Taylor has developed the testing equipment for, and has performed testing on, the pullout of soil reinforcing from various soils, connection testing of soil reinforcing from segmental retaining wall units, pullout of anchors from concrete facing panels, resilient modulus testing of pavements through the application of cyclic loading, and impact loading applied to flexible post traffic barrier systems. Lead author for the Soil Nail Walls Parametric Study presented to the NHI-FHWA. He is a certified NHI instructor for the FHWA.

United States Patents for MSE Structures

D599630    Soil Reinforcing Retaining Wall Anchor
5,484,235   Retaining Wall System
5,702,208   Grid-Locked Block Panel System
5,733,072  Wirewall With Stiffened High Wire Density Face
5,749,680  Wire Mat Connector
5,820,305  T-Block Wall System
5,975,809  Apparatus And Method For Securing Soil Reinforcing Elements To Earthen
5,975,810  Geo-Grid Anchor
6,024,516  Improved System For Securing A Face Panel To An Earthen Formation
6,517,293  Anchor Grid Connection Element
7,722,296  Retaining Wall Soil Reinforcing Connector And Method
7,891,912  Two-Stage Mechanically Stabilized Earth Wall System
7,972,086  Earthen Retaining Wall With Pinless Soil Reinforcing Elements
7,980,790  Compressible Mechanically Stabilized Earth Retaining Wall System And Method For Installation Thereof
8,177,458  Mechanically Stabilized Earth Connection Apparatus And Method
8,393,829  Wave Anchor Soil Reinforcing Connector And Method
8,496,411  Two-Stage Mechanically Stabilized Earth Wall System
8,632,277  Retaining Wall Soil Reinforcing Connector And Method
8,632,278  Mechanically Stabilized Earth Welded Wire Facing Connection System And Method
8,632,279  Splice For A Soil Reinforcing Element Or Connector
8,632,280  Welded Wire Wall With Grid-Strip T-Tab Connector
8,632,281  Mechanically Stabilized Earth System And Method
8,632,282  Mechanically Stabilized Earth System And Method
8,734,059  Soil Reinforcing Element For A Mechanically Stabilized Earth Structure

Others Pending

Publications for MSE


Essery, Derek, Taylor, Thomas P., El-Sharnouby, Meckkey (2017), “Comparison between AASHTO and CHBDC Design Methods for MSE Retaining Wall for and its Implications on Transportation Agencies”, Conference of the Transportation Association of Canada, St. John’s, NL

Taylor, Thomas P., (2015) “Predicting Horizontal Pressure in 2-Stage MSE”. In partial fulfillment for Masters of Science in Civil Engineering, University of Texas at Arlington.


**Recent MSE Projects**

**11th Street Corridor**
District of Columbia Department of Transportation
Washington D.C.
59,112 SF MSE retaining wall for approach ramps
Hardscapes LLC

**IH-35**
Temple, Texas
Texas Department of Transportation
390,750 SF MSE retaining walls for approach ramps and bridge abutments
James Construction

**SH-121 Section 2**
Tarrant County Texas
North Texas Toll Authority
167,423 SF MSE retaining walls for approach ramps and bridge abutments
Webber Construction

**SH-121 Section 5**
Tarrant County Texas
North Texas Toll Authority
20,331 SF MSE retaining walls for approach ramps and bridge abutments
Texas Sterling Construction

**US-36**
Tarrant County Texas
Colorado Department of Transportation
141,440 SF MSE retaining walls for approach ramps and bridge abutments
Ames-Granite
SH-121
The Colony, Texas
Texas Department of Transportation
49,023 SF MSE retaining walls for approach ramps and bridge abutments
Austin Bridge

US-6 DB
Denver, Colorado
Colorado Department of Transportation
120,000 SF MSE retaining walls for approach ramps and bridge abutments
Ed Kramer Construction

Post Graduate Classes

Structural Masonry
Masonry unit type and grades of mortar types, reinforcing and connectors. Design of beams, columns, pilasters, and walls. Structural behavior and construction practices. Includes plain and reinforced masonry. Building codes, Masonry Standard Joint Committee (MSJC) specifications, material specifications, test methods, and recommended practice documents.

Advanced Theory of Structures
Advanced analysis of indeterminate beams, frames, trusses, arches, and cables.

Theoretical Soil Mechanics
Theory of consolidation, magnitude, time rate, pore pressure dissipation with variable construction rate and layered soils. Secondary compression, preconsolidation, and preloading. Shear strength of soil. Critical state soil mechanics, dilation and strain-softening in drained shear, pore pressure response in undrained shear, including static liquefaction. Prerequisite: CE 3343 or consent of instructor.

Geosynthetics
Geosynthetics properties and testing, design of geotextiles, geogrids, geonets, and geomembranes for applications in separation, pavement, embankment and retaining wall reinforcement, soil stabilization, filtration, drainage and liquid barrier, construction guidelines and case histories.

In-Situ Testing
Site characterization, in-situ testing procedures, and soil property interpretation methods for standard penetration tests, cone penetration tests utilizing friction cone, piezocene, and seismic cone, dilatometer, vane shear, pressure meter, and bore hole shear tests, non-destructive tests for pavement subgrade characterization
Ground Improvement

Introduction and types of ground improvement for different problem soils including soft and expansive soils, shallow and deep soil densification, sand drains and wick drains, chemical modification, chemical binders and mechanisms of ground improvement, different types of grouting, deep mixing, stone columns, soil nailing, ground anchors, geosynthetics, MSE walls, reinforced slopes.

Experimental Soil Mechanics

Fundamentals of experimental studies of soil behavior, soil properties and their laboratory test methods which include consolidation, direct shear, static triaxial, cyclic triaxial, resonant column, bender elements and other advanced geotechnical laboratory tests, instrumentation and measurement techniques.

Foundation Analysis and Design

The design, construction, and performance of footings, rafts, and piles founded on or in sands, clays, silts, stratified soils, and weak rock. Includes the influence of various geologic terrain on selecting foundation type and constructability, in-situ investigations to determine material design parameters, bearing capacity, and settlement of foundations.

Advanced Foundation Design

The design, construction, and performance of footings, rafts, and piles founded on or in sands, clays, silts, stratified soils, and weak rock. Includes the influence of various geologic terrain on selecting foundation type and constructability, in-situ investigations to determine material design parameters, bearing capacity, and settlement of foundations.

Design of Earth Structures

Study of the states of stress and analysis techniques associated with cuts, fills, and retaining structures. Includes slope stability, embankment reinforcement, conventional and reinforced earth retaining walls, excavation bracing, and sheet pile wharf structures.

Unsaturated Soil Mechanics

Fundamental aspects of the mechanical behavior of unsaturated soils, including stress and volumetric state variables, matrix suction measurements and soil-water characteristic curves, shear-strain-strength and volume change responses, suction-controlled laboratory testing techniques and constitutive modeling.

Constitutive Modeling of soils

Fundamental aspects of elasto-plastic behavior of soils along axisymmetric stress paths, shear strength of soils in light of critical state soil mechanics, and constitutive models to
predict soil response under saturated conditions, including Cam Clay and modified Cam Clay models.

**Licenses and Registrations**


NCEES Record Certificate No. 20154
NHI Instructor Number 0923

**Education**

Doctor of Philosophy - Civil Engineering
University of Texas at Arlington

Masters of Science Geotechnical Engineering
University of Texas at Arlington

Bachelor of Science Civil Engineering
University of Wyoming

Bachelor of Science Education
University of Wyoming

**References**

**James G. Collin, Ph.D., P.E., D.GE**
The Collin Group, Ltd.
7445 Arlington Road
Bethesda, MD 20814
301-907-9501

**Anand J. Puppala, Ph.D., P.E., D.GE.**
Distinguished Teaching Professor
Associate Dean for Research, College of Engineering
University of Texas at Arlington
817-272-5821

**Barry Christopher, Ph.D., P.E., D.GE**
Geotechnical Engineering Consultant
210 Boxelder Lane
Roswell, GA 30076
770-641-8696
DESIGN CALCULATIONS

CAPE COD RAIL TRAIL EXTENSION – PHASE 1

FEDERAL AID PROJECT NO. CM/TAP/TI-002S(758)X

PROJECT FILE NO. 604488

BARNSTABLE COUNTY, MA

PREPARED FOR:

LAWRENCE-LYNCH CORP.

[Signature]

THOMAS P. TAYLOR
CIVIL
No. 45571
REGISTERED PROFESSIONAL ENGINEER

12-24-15

BIG R BRIDGE PROJECT NO. VW14-02388
VERIFICATION CALCULATIONS

CAPE COD RAIL TRAIL EXTENSION – PHASE 1

FEDERAL AID PROJECT NO. CM/TAP/TI-002S(758)X

PROJECT FILE NO. 604488

BARNSTABLE COUNTY, MA

PREPARED FOR:

LAWRENCE-LYNCH CORP.

BIG R BRIDGE PROJECT NO. VW14-02388
BROKEN BACK-SLOPE MSE STRUCTURE - DESIGN ANALYSIS

**Project Name:** CAPE COD RAIL TRAIL EXTENSION – PHASE 1  
**Project Location:** Town of Dennis & Yarmouth, Barnstable County, MA  
**DSGN By:** CMS  
**Date:** 12-24-15  
**Project Number:** N/A  
**Design Code:** AASHTO  
**CHCK By:** TPT  
**Date:** 12-24-15  
**Design Method:** LRFD - simplified

**Introduction**

This worksheet will demonstrate the design methodology for internal stability of an MSE structure. The method is based on the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specification, 2012, Sixth Edition, and all inclusive interim revisions through 2013. Section-11, Walls, Abutments, and Piers is followed in this worksheet. This methodology is for a simple structure, with, or without, a broken back-slope. Further, in this worksheet the user has the option to apply a uniform traffic surcharge.

The worksheet is set up to be used with the Grid-Strip™ soil reinforcing system. However, it can be modified as necessary to be used with any reinforcing system.

Any input that is required by the user is located at the far right of the page and is preceded by a series of dots (.............) and a definition. All calculated values are below the calculation definition, at the far left of the page and are slightly indented.

The worksheet is a verification calculation that can be used with the software programs, MSE-Pro (Vistawall Research and Development), and MSEW (Adama Engineering). Each of these software programs outputs are at the end of the report.
**Structure Definition**

A simple MSE structure consists of reinforced soil that includes compacted backfill, soil reinforcing and a facing element. The reinforced soil bears on the foundation. Directly behind the reinforced soil is the retained soil. The structure may also include an earth surcharge and/or a live load surcharge.

![Diagram of MSE Structure](image)

**Figure 1 – Typical MSE Cross section**

**Live Load Surcharge**

The live load surcharge is contained on the level portion of the earth surcharge to the right of the crest. If the live load is located in the external active wedge, defined as being equal to a distance equal to the following:

\[ X_q = B + (H + S) \cdot \tan \left( 45^\circ + \frac{\phi}{2} \right) \]

The above equation is based on the internal friction angle of the retained backfill. As the friction angle decreases the slope of the failure surface decreases. For internal stability analysis if the traffic live load is contained over the active zone than it shall be used in the analysis. It is always conservative to assume that there is a traffic live load. Traffic is not included over the sloped portion of the surcharge.
Structure Parameters

Figure 2 – Structure Definition

where:

\[ H = \text{MSE structure height} \]
\[ L = \text{Soil reinforcing length} \]
\[ S = \text{Earth surcharge height} \]
\[ X_s = \text{earth surcharge offset to full height} \]
\[ \beta = \text{slope of surcharge} \]

Earth Surcharge

The earth surcharge is the volume of soil that is above the top of the reinforced volume of soil. It usually contains a sloped section and a level section. The level section can be contained over the reinforced volume of soil or located over the retained soil. The distance that the level portion is from the face of the wall defines the earth surcharge condition. If the level portion of the earth surcharge is located a distance back from the MSE wall face equal to twice the structure height, it is considered an infinite slope. Otherwise it is considered a broken back-slope condition. For a broken back-slope condition the
angle of the slope is reduced to an equivalent slope angle. The height of the structure is defined as the distance from the top of the in-situ foundation to the top of the wall coping.

![Figure 3 – Equivalent Slope Angle (AREA)](image)

**Broken Back-Slope**

The broken back-slope uses the AREA method. In this method the direction of the resultant pressure at the interface of the retained soil and reinforced soil is parallel to a line that propagates from the back face edge of the top of the wall facing at a distance of twice the structure height to the intersection at the top of the level surface of the surcharge. This imaginary slope angle forms an angle \( \beta_i \). This method is established from the trial-wedge method. A discussion on this subject can be found in “Earth Pressure and Retaining Walls” by W. C. Huntington. Based on this the equivalent slope angle is equal to:

\[
\beta_i = \tan^{-1}\left( \frac{S}{2 \cdot H} \right)
\]

Where:

\[ \beta_i = \text{Equivalent slope angle (deg)} \]
\[
S = \text{Height of slope (ft)}
\]
\[
H = \text{Structure height (ft)}
\]

**Failure Surface**

The failure surface is defined as the interface between the active zone and the passive zone. Within the reinforced volume of soil it is the location of maximum stress and is a function of the mechanical height of the structure. The mechanical height of the structure is a function of the earth surcharge configuration.

![Failure Surface Diagram]

**Figure 4 – Failure Surface Definition**

The active zone is the wedge of soil closest to the wall facing. The mechanical height of the structure is calculated by the following equation:

\[
H_m = H + \frac{0.30 \cdot H \cdot \tan(\beta)}{1 - 0.30 \cdot \tan(\beta)}
\]

The area that is behind the failure surface is defined as the passive zone and is the area of soil that provides resistance to pullout.
INTERNAL DESIGN - SIMPLIFIED METHOD

Design Against Structural Failure

Internally each level of reinforcing is analyzed for tensile capacity and pullout capacity. Maximum reinforcement loads are calculated using the Simplified Method of analysis. The load in the reinforcements is determined by multiplying the vertical earth pressure \( (\sigma_v) \) at each reinforcement elevation by the lateral earth pressure coefficient.

**Design Against Tensile Failure**

The internal loads for the vertical pressures at the level of investigation include the traffic live load and use the following relationship:

\[
\sigma_v = \gamma_r \cdot Z + \sigma_q + \Delta \sigma_v \quad [\text{AASHTO 11.10.6.2.1-1}]
\]

Where:

- \( \sigma_v \) = Vertical stress (ksf)
- \( \gamma_r \) = unit weight of the back fill (kcf)
- \( Z \) = depth to the soil reinforcing measured form the top of the structure (ft)
- \( \sigma_q \) = live load surcharge (ksf)
- \( \Delta \sigma_v \) = Vertical stress due to surcharge loads and is calculated from AASHTO equation 11.10.10.1-1 (ksf)

The horizontal pressure is calculated based on the vertical pressure and the internal earth pressure coefficient and uses the following relationship:

\[
\sigma_h = \gamma_p \left( \phi_v \cdot K_r + \Delta \sigma_h \right) \quad [\text{AASHTO 11.10.6.2.1-1}]
\]

Where:

- \( \sigma_h \) = Factored horizontal stress (ksf)
- \( K_r \) = Horizontal pressure coefficient (dim)
- \( \gamma_p \) = Load factor for vertical earth pressure (EV) (AASHTO 11.10.6.2.1-1 reference commentary for discussion)
- \( \Delta \sigma_h \) = Horizontal stress at reinforcement level resulting from any applicable concentrated horizontal surcharge load as specified in AASHTO 11.10.10.1 (ksf)
The maximum tension in the reinforcement is determined by applying the resulting lateral pressure to the tributary area that the reinforcement occupies.

\[ T_{\text{max}} = \sigma_h \cdot S_v \cdot L_p \]  

[**AASHTO 11.10.6.2.1-2**]

Where:

- \( T_{\text{max}} \): Maximum tension at depth of soil reinforcing (kip)
- \( \sigma_h \): Factored horizontal stress (ksf)
- \( S_v \): Vertical spacing of the soil reinforcing (ft)
- \( L_p \): Length of face panel (ft)

The required area of steel is determined by dividing the maximum tension by the allowable stress.

\[ A_{\text{req}} = \frac{T_{\text{max}}}{\phi_y \cdot F_y} \]

Where:

- \( A_{\text{req}} \): Required area of soil reinforcing (in\(^2\))
- \( \phi_y \): Tensile Resistance Factor (0.75) [**AASHTO 11.5.6-1**]
- \( T_{\text{max}} \): Maximum tension (Kip)
- \( F_y \): Minimum yield strength of steel (ksi)

The required number of soil reinforcing elements is determined by dividing the required area of steel by the end of design life area for a single soil reinforcing element.

\[ n_{\text{req}} = \frac{A_{\text{req}}}{A_e} \]

Where:

- \( n_{\text{req}} \): Number of required soil reinforcing elements
- \( A_e \): Design area of soil reinforcing (in\(^2\))

The number of soil reinforcing is rounded up to the nearest whole number.

The capacity demand ratio for rupture is then determined using the following relationship:

\[ \text{CDR}_{\text{rupture}} = \frac{n_{\text{req}} \cdot A_e}{A_{\text{req}}} \geq 1.0 \]
Design Against Pullout Failure

The ultimate resistance to pullout for discrete steel strips is calculated using the following relationship:

\[ P_{ig} = n \cdot \phi \cdot F^* \cdot \alpha \cdot \sigma_v \cdot C \cdot L_e \quad \text{[AASHTO 11.10.6.3.2-1]} \]

Where:
- \( P_{ig} \) = Available resistance to pullout (kip)
- \( n \) = Number of soil reinforcing elements
- \( \phi \) = Pullout Tensile Resistance Factor from 11.5.6-1
- \( F^* \) = Pullout friction factor
- \( \alpha \) = Scale effect correction factor
- \( \sigma_v \) = Unfactored vertical stress at reinforcement level (ksf)
- \( C \) = Overall reinforcement surface area geometry factor based on the gross perimeter of the reinforcement and is equal to 2 for discrete steel strips
- \( L_e \) = Length of reinforcement in resisting zone (ft.)
The length of embedment is based on the failure surface. The length of soil reinforcing \( L_e \) that is behind the active zone is the distance that resists pullout. The unfactored vertical stress that resists pullout is the area of soil that bears on the soil reinforcing element.

The maximum tension in the reinforcement for pullout is determined by applying the resulting lateral pressure to the tributary area of the reinforcement. Note that the \( T_{\text{MAX}} \) does not contain any vertical live loads pursuant to AASHTO Figure 11.10.10.1-1 as discussed in the commentary.

\[
\sigma_v = \gamma_r \cdot Z + \Delta \sigma_v
\]

Where:

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sigma_v )</td>
<td>Vertical stress (ksf)</td>
</tr>
<tr>
<td>( \gamma_r )</td>
<td>unit weight of the backfill (kcf)</td>
</tr>
<tr>
<td>( Z )</td>
<td>depth to the soil reinforcing measured from the top of the structure (ft)</td>
</tr>
<tr>
<td>( \Delta \sigma_v )</td>
<td>Vertical stress due to surcharge loads and is calculated from AASHTO equation 11.10.10.1-1 (ksf)</td>
</tr>
</tbody>
</table>

\[
\sigma_h = \gamma_p \left( \phi_v \cdot K_r + \Delta \sigma_h \right) \quad [\text{AASHTO 11.10.6.2.1-1}]
\]

Where:

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sigma_h )</td>
<td>Factored horizontal stress (ksf)</td>
</tr>
<tr>
<td>( K_r )</td>
<td>Horizontal pressure coefficient (dim)</td>
</tr>
<tr>
<td>( \gamma_p )</td>
<td>Load factor for vertical earth pressure (EV) (AASHTO 11.10.6.2.1-1 reference commentary for discussion)</td>
</tr>
<tr>
<td>( \Delta \sigma_h )</td>
<td>Horizontal stress at reinforcement level resulting from any applicable concentrated horizontal surcharge load as specified in AASHTO 11.10.10.1 (ksf)</td>
</tr>
</tbody>
</table>

\[
T_{\text{MAX \_ PO}} = \sigma_h \cdot S_v \cdot L_p
\]

Where:

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( T_{\text{MAX _ PO}} )</td>
<td>Maximum tension at depth of soil reinforcing (kip)</td>
</tr>
<tr>
<td>( \sigma_h )</td>
<td>Factored horizontal stress (ksf)</td>
</tr>
<tr>
<td>( S_v )</td>
<td>Vertical spacing of the soil reinforcing (ft)</td>
</tr>
<tr>
<td>( L_p )</td>
<td>Length of face panel (ft)</td>
</tr>
</tbody>
</table>

The Capacity Demand Ratio (CDR) for pullout is the ratio of the available pullout to the applied tension at the level under investigation. The CDR is calculated using the following relationship.

\[
\text{CDR}_{\text{PO}} = \frac{P_{\text{fr}}}{T_{\text{MAX \_ PO}}}
\]
### SAMPLE CALCULATION

#### Structure Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structure design height</td>
<td>30.00·ft</td>
</tr>
<tr>
<td>Length of soil reinforcing</td>
<td>21.00·ft</td>
</tr>
<tr>
<td>Surcharge height form top of structure to extent of soil reinforcing</td>
<td>0.00·ft</td>
</tr>
<tr>
<td>Distance from face of structure to level portion of surcharge</td>
<td>0.0001·ft</td>
</tr>
<tr>
<td>Vertical Spacing of soil reinforcing</td>
<td>2.50·ft</td>
</tr>
<tr>
<td>Panel width</td>
<td>10.00·ft</td>
</tr>
<tr>
<td>Depth from top of wall to first soil reinforcing element</td>
<td>1.25·ft</td>
</tr>
</tbody>
</table>

#### Slope of Surcharge

\[
\beta_s := \arctan\left(\frac{S_{sf}}{X_{sf}}\right) = 0.000\cdot\text{deg}
\]

#### Distance of sloping surcharge over soil reinforcing

\[
X_s := \text{if}(X_{sf} \leq L, X_{sf}, L) = 0.000\ \text{ft}
\]

#### Depth of surcharge over soil reinforcing

\[
S := \text{if}(X_{sf} = 0, S_{sf}, X_s \cdot \tan(\beta_s)) = 0.000\ \text{ft}
\]

#### Distance of level portion over soil reinforcing

\[
B_s := \text{if}(X_s < L, L - X_s, 0) = 21.000\ \text{ft}
\]

#### Adjusted surcharge angle

\[
\beta_i := \text{if}(X_{sf} = 0, \beta_s, \arctan\left(\frac{S_{sf}}{2 \cdot H}\right)) = 0.000\cdot\text{deg}
\]
Mechanical Height

\[ H_m := \begin{cases} 
X > 0.3 \cdot (H + S), & H + \frac{\tan(\beta_s) \cdot 0.30 \cdot H}{1 - 0.30 \cdot \tan(\beta_s)}, H + S 
\end{cases} \]

\[ = 30.000 \text{ ft} \]

Distance From Mechanical Height to Top of Structure

\[ S_m := H_m - H = 0.000 \text{ ft} \]

Number of Rows of Reinforcement

\[ Z_n := \text{floor} \left[ \frac{(H) - Z_1}{S_v} + 1 \right] = 12 \]

Depth to Lowest Soil Reinforcing Element

\[ Z_m := Z_1 + (Z_n - 1) \cdot S_v = 28.750 \text{ ft} \]

Depth to Each Element

\[ d_i := Z_1 \cdot (Z_1 + S_v) \cdot Z_m \]

Depth From Top of Structure To Reinforcement Under Investigation

\[ d_m(d_i) := d_i \]

Depth From mechanical Height To Reinforcement Under Investigation

\[ d_{fp}(d_i) := d_m(d_i) + S_m \]

Depth to upper most soil reinforcing element from top of surcharge at failure plane

\[ Z_t := S_m + Z_1 = 1.250 \text{ ft} \]

Depth to bottom soil reinforcing element from top of surcharge at failure plane

\[ Z_b := Z_t + (Z_n - 1) \cdot S_v = 28.750 \text{ ft} \]

Depth from top of MSE retaining wall to each soil reinforcing element

\[ d_{sr}(d_i) := d_i \]
Function to determine the vertical spacing of each soil reinforcing element

\[ S_V(d_i) := \begin{cases} 
S_v \left( Z_1 + \frac{S_v}{2} \right) & \text{if } d_i = Z_1 \\
S_v \left( \frac{S_v}{2} \right) & \text{if } d_i = H \\
S_v & \text{otherwise}
\end{cases} \]

**Select Backfill Material Properties**

Unit weight of select backfill: \( \gamma_m := 130 \text{· pcf} \)

Internal friction angle of select backfill: \( \phi_m := 34 \text{· deg} \)

Internal Active Earth Pressure Coefficient:

\[ K_{ai} := \tan \left( 45 \text{· deg} - \frac{\phi_m}{2} \right) = 0.283 \]

Internal Active Earth Pressure Ratio (taken from top of coping):

\[ K_r(d_i) := K_{ai} \cdot \left[ d_i \geq 20 \text{· ft}, 1.2 \cdot 1.7 - \left( \frac{1.7 - 1.2}{20 \text{· ft}} \right) \cdot d_i \right] \]

**Retained Backfill Material Properties**

Unit weight: \( \gamma_r := 120 \text{· pcf} \)

Internal friction angle: \( \phi_r := 30 \text{· deg} \)

External Active Earth Pressure Coefficient

\[ K_a := \cos(\beta_i) \cdot \frac{\cos(\beta_i) - \sqrt{\cos(\beta_i)^2 - \cos(\phi_r)^2}}{\cos(\beta_i) + \sqrt{\cos(\beta_i)^2 - \cos(\phi_r)^2}} \]

\[ K_a = 0.333 \]

**In-situ Foundation Material Properties**

Unit weight: \( \gamma_f := 125 \text{· pcf} \)

Internal friction angle: \( \phi_f := 34 \text{· deg} \)
### External Live Load

Traffic live load pressure: \[ \sigma_q = 250 \text{-psf} \]

Do you want to include the live load in the pullout calculation (Yes or No): \[ \text{LLinclude} = "No" \]

### Load Factors (AASHTO 2004 Table 3.4.1-1 and Table 3.4.1-2)

Vertical earth pressure: \[ \gamma_{EVmax} = 1.35 \]

\[ \gamma_{EVmin} = 1.00 \]

Surcharge surface: \[ \gamma_{ESmax} = 1.50 \]

\[ \gamma_{ESmin} = 0.75 \]

Horizontal earth pressure: \[ \gamma_{EHmax} = 1.50 \]

\[ \gamma_{EHmin} = 0.90 \]

Live load: \[ \gamma_{LLmax} = 1.75 \]

\[ \gamma_{LLmin} = 1.75 \]

Sliding resistance factor: \[ \phi_{sliding} = 1.00 \]

Pullout resistance factor: \[ \phi_{po} = 0.90 \]

### Material Degradation Parameters

Service life: \[ Y_t = 75 \text{-yr} \]

Thickness of galvanized coating: \[ g_T = 3.40 \text{-mil} \]

Galvanization degradation first two years: \[ g_2 = 0.60 \frac{\text{mil}}{\text{yr}} \]

Galvanization degradation remaining years: \[ g_R = 0.16 \frac{\text{mil}}{\text{yr}} \]

Steel degradation: \[ E_{S75} = 0.47 \frac{\text{mil}}{\text{yr}} \]
Time to complete galvanization loss.

\[ Y_G := \frac{g_T - 2 \cdot \gamma_r g_2}{g_R} + 2 \cdot \gamma_r = 15.750 \cdot \gamma_r \]

Sacrificial steel design life

\[ Y_S := \text{if}(Y_G < 0, Y_t, Y_t - Y_G) = 59.250 \cdot \gamma_r \]

Applied sacrificial steel thickness

\[ E_c := \left[ (Y_t - Y_G) \cdot E_{S75} \right] \cdot 2 = 0.056 \text{ in} \]

**Soil Reinforcing Properties**

Yield Stress of Steel................................................................................................. \[ F_Y := 65 \text{ ksi} \]

Resistance factor for rupture.................................................................................... \[ \phi_R := 0.75 \]

Diameter of Longitudinal Wire.................................................................................. \[ d_L := 0.375 \text{ in} \]

Design area of Grid-Strip Soil Reinforcing

\[ A_{SR} := 2 \cdot \pi \cdot \frac{(d_L - E_c)^2}{4} = 0.160 \text{ in}^2 \]

Maximum Tension Capacity of Grid-Strip

\[ T_{SR} := A_{SR} \cdot \phi_R \cdot F_Y = 7.807 \text{ kip} \]

Width of soil reinforcement....................................................................................... \[ b_{sr} := 2 \text{ in} \]

Width of transverse soil reinforcing element.......................................................... \[ W_{GS} := 2 \text{ in} \]

Spacing of longitudinal soil reinforcing element..................................................... \[ L_z := 2 \text{ in} \]

Scale correction factor for soil reinforcement pullout (AASHTO 11.10.6.3.2)............ \[ \alpha := 1.00 \]

Unit perimeter factor for soil reinforcement pullout (AASHTO 11.10.6.3.2)............... \[ C := 2.00 \]
Resistance factor for pullout: \( \phi_p := 0.90 \)

Pullout Friction Factor at \( d_i = 0 \): \( f_0 := 2.50 \)

Pullout Friction Factor at \( d_i = 20 \): \( f_{20} := 1.25 \)

Length of Embedment of Soil Reinforcing Element In To Passive Zone

\[
L_e(d_i) := \begin{cases} 
    d_{fp}(d_i) & \text{if } d_{fp}(d_i) \leq \frac{H_m}{2}, L - 0.3H_m, L - \frac{H - (d_i)}{0.50 \cdot H} \\
    0 & \text{if } d_{fp}(d_i) > \frac{H_m}{2}
\end{cases}
\]

Local Pullout Resistance Factor (\( F_g \))

\[
F_{star}(d_i) := \begin{cases} 
    d_i & \text{if } d_i \geq 20 \cdot \text{ft}, f_{20}, f_0 - \frac{f_0 - f_{20}}{20 \cdot \text{ft}} \cdot (d_i)
\end{cases}
\]
External Stability Calculation

Broken Backslope MSE Structure – Load Diagram

Unfactored Vertical Force from Reinforced Mass

Vertical force of reinforced mass load

\[ V_1 := \gamma_m \cdot H \cdot L = \frac{81,900 {\text{kip}}}{\text{ft}} \]

Moment arm of reinforced mass

\[ h_{V1} := 0.5 \cdot L = 10.500 \, \text{ft} \]

Moment of reinforced mass

\[ M_{V1} := V_1 \cdot h_{V1} = \frac{859,950 \, \text{ft} \cdot \text{kip}}{\text{ft}} \]
**Unfactored Vertical Force from Traffic Surcharge**

Vertical live load surcharge

\[ V_2 := \sigma_q \left( L - X_s \right) = 5.250 \cdot \frac{\text{kip}}{\text{ft}} \]

Moment arm of live load surcharge

\[ h_{V2} := X_s + \frac{1}{2} \left( L - X_s \right) = 10.500 \text{ ft} \]

Moment of live load surcharge

\[ M_{V2} := V_2 \cdot h_{V2} = 55.125 \cdot \frac{\text{ft} \cdot \text{kip}}{\text{ft}} \]

**Unfactored Horizontal Active Earth Force**

Vertical earth surcharge

\[ V_3 := \gamma_m \cdot S \left[ \frac{1}{2} X_s + \left( L - X_s \right) \right] = 0.000 \cdot \frac{\text{kip}}{\text{ft}} \]

Moment arm of earth surcharge

\[ h_{V3} := \frac{0.5 \cdot L^2 - \frac{X_s^2}{6}}{L - \frac{X_s}{2}} = 10.500 \text{ ft} \]

Moment of earth surcharge

\[ M_{V3} := V_3 \cdot h_{V3} = 0.000 \cdot \frac{\text{ft} \cdot \text{kip}}{\text{ft}} \]

**Unfactored Vertical Active Earth Force**

Vertical earth pressure on back of MSE mass

\[ V_4 := \frac{1}{2} \cdot K_a \cdot \gamma_r \cdot (H + S)^2 \cdot \sin(\beta_i) = 0.000 \cdot \frac{\text{kip}}{\text{ft}} \]

Moment arm of vertical earth pressure

\[ h_{V4} := L = 21.000 \text{ ft} \]

Moment of vertical earth pressure

\[ M_{V4} := V_4 \cdot h_{V4} = 0.000 \cdot \frac{\text{ft} \cdot \text{kip}}{\text{ft}} \]
**Unfactored Vertical Traffic Live Load Surcharge**

Vertical live load from traffic surcharge on back of MSE mass

\[ V_5 := (H + S) \cdot K_a \cdot \sigma_q \cdot \sin(\beta_i) = 0.000 \cdot \frac{\text{kip}}{\text{ft}} \]

Moment arm of vertical live load

\[ h_{V5} := L = 21.000 \text{ ft} \]

Moment of vertical live load

\[ M_{V5} := V_5 \cdot h_{V5} = 0.000 \cdot \frac{\text{ft} \cdot \text{kip}}{\text{ft}} \]

**Unfactored Horizontal Active Earth Force**

Horizontal earth pressure on back of MSE mass

\[ F_1 := \frac{1}{2} \cdot K_a \cdot \gamma_r \cdot (H + S)^2 \cdot \cos(\beta_i) = 18.000 \cdot \frac{\text{kip}}{\text{ft}} \]

Moment arm of horizontal earth pressure

\[ h_{F1} := \frac{H + S}{3} = 10.000 \text{ ft} \]

Moment of horizontal earth pressure

\[ M_{F1} := F_1 \cdot h_{F1} = 180.000 \cdot \frac{\text{ft} \cdot \text{kip}}{\text{ft}} \]

**Unfactored Horizontal Traffic Live Load Surcharge**

Horizontal live load from traffic surcharge on back of MSE mass

\[ F_2 := (H + S) \cdot K_a \cdot \sigma_q \cdot \cos(\beta_i) = 2.500 \cdot \frac{\text{kip}}{\text{ft}} \]

Moment arm of horizontal live load

\[ h_{F2} := \frac{H + S}{2} = 15.000 \text{ ft} \]

Moment of horizontal live load

\[ M_{F2} := F_2 \cdot h_{F2} = 37.500 \cdot \frac{\text{ft} \cdot \text{kip}}{\text{ft}} \]
**Strength I Values - Limiting Eccentricity (Maximum)**

Total Vertical Load without live load

\[ V_{A\_e\_max} := \gamma_{EV} V_1 + \gamma_{EV} V_3 + \gamma_{EH} V_4 + \gamma_{LL} V_5 = 110.565 \text{kip/ft} \]

Resisting Moment without live load

\[ M_{A\_e\_max} := \gamma_{EV} M_1 + \gamma_{EV} M_3 + \gamma_{EH} M_4 + \gamma_{LL} M_5 = 1160.933 \text{kip-ft} \]

Total Horizontal Load

\[ F_{TH\_max} := \gamma_{EH} F_1 + \gamma_{LL} F_2 = 31.375 \text{kip/ft} \]

Overturning Moment

\[ M_{O\_e\_max} := \gamma_{EH} M_F_1 + \gamma_{LL} M_F_2 = 335.625 \text{kip-ft} \]

Net Moment

\[ M_{A\_e\_net\_max} := M_{A\_e\_max} - M_{O\_e\_max} = 825.307 \text{kip-ft} \]

Location of Resultant Force

\[ a_{e\_max} := \frac{M_{A\_e\_net\_max}}{V_{A\_e\_max}} = 7.464 \text{ft} \]

Eccentricity at base of wall

\[ e_{L\_e\_max} := 0.5 \cdot L - a_{e\_max} = 3.036 \text{ft} \]

Eccentricity Soil Reinforcing Ratio

\[ \frac{e_{L\_e\_max}}{L} = 0.145 \]

CDR for Overturning at base of wall

\[ \frac{M_{A\_e\_max}}{M_{O\_e\_max}} = 3.459 \]

Is resultant within limiting eccentricity value

\[ \text{if} \left( \frac{e_{L\_e\_max}}{L} > \frac{L}{4}, \text{"No"}, \text{"Yes"} \right) = \text{"Yes"} \]

**Capacity Demand Ratio - Overturning**

\[ \text{CDR}_{ot\_max} := \frac{M_{A\_e\_max}}{\gamma_{EH} M_F_1 + \gamma_{LL} M_F_2} = 3.459 \]
Strength I Values - Limiting Eccentricity (Minimum)

Total Vertical Load without live load

\[ V_{A_e\_min} := \gamma_{Evmin} \cdot V_1 + \gamma_{Evmin} \cdot V_3 + \gamma_{EHmin} \cdot V_4 + \gamma_{LLmin} \cdot V_5 = 81.900 \cdot \frac{\text{kip}}{\text{ft}} \]

Resisting Moment without live load

\[ M_{A_e\_min} := \gamma_{Evmin} \cdot M_{V1} + \gamma_{Evmin} \cdot M_{V3} + \gamma_{EHmin} \cdot M_{V4} + \gamma_{LLmin} \cdot M_{V5} = 859.950 \cdot \frac{\text{kip}\cdot\text{ft}}{\text{ft}} \]

Overturning Moment

\[ M_{O_e\_min} := \gamma_{EHmin} \cdot M_{F1} + \gamma_{LLmin} \cdot M_{F2} = 227.625 \cdot \frac{\text{kip}\cdot\text{ft}}{\text{ft}} \]

Net Moment

\[ M_{A_e\_net\_min} := M_{A_e\_min} - M_{O_e\_min} = 632.325 \cdot \frac{\text{kip}\cdot\text{ft}}{\text{ft}} \]

Location of Resultant Force

\[ a_{e\_min} := \frac{M_{A_e\_net\_min}}{V_{A_e\_min}} = 7.721 \text{ ft} \]

Eccentricity at base of wall

\[ e_{L_e\_min} := 0.5 \cdot L - a_{e\_min} = 2.779 \text{ ft} \]

Eccentricity Soil Reinforcing Ratio

\[ \frac{e_{L_e\_min}}{L} = 0.132 \]

CDR for Overturning at base of wall

\[ \frac{M_{A_e\_min}}{M_{O_e\_min}} = 3.778 \]

Is resultant within limiting eccentricity value

\[ \text{if} \left( e_{L_e\_min} > \frac{L}{4} \right. \left. \text{"No"}, \text{"Yes"} \right) = \text{"Yes"} \]

Capacity Demand Ratio - Overturning

\[ \text{CDR}_{ot\_min} := \frac{M_{A_e\_min}}{\gamma_{EHmin} \cdot M_{F1} + \gamma_{LLmin} \cdot M_{F2}} = 3.778 \]
**Strength 1 Values for Eccentricity Check (Critical)**

Total Vertical Load

\[ V_{A\_e\_critical} := \gamma_{EV_{min}} \cdot V_1 + \gamma_{EV_{min}} \cdot V_3 + \gamma_{EH_{max}} \cdot V_4 + \gamma_{LL_{max}} \cdot V_5 = 81.900 \frac{\text{kip}}{\text{ft}} \]

Overturning Moment without Live Load

\[ M_{O\_e\_critical} := \begin{cases} M_{O\_e\_max} & \text{if } M_{O\_e\_max} \geq M_{O\_e\_min} \\ M_{O\_e\_min} & \text{otherwise} \end{cases} = 335.625 \frac{\text{kip} \cdot \text{ft}}{\text{ft}} \]

Resisting Moment without Live Load

\[ M_{A\_e\_critical} := \gamma_{EV_{min}} \cdot M_V1 + \gamma_{EV_{min}} \cdot M_V3 + \gamma_{EH_{max}} \cdot M_V4 + \gamma_{LL_{max}} \cdot M_V5 = 859.950 \frac{\text{kip} \cdot \text{ft}}{\text{ft}} \]

Net Moment

\[ M_{A\_e\_net\_critical} := M_{A\_e\_critical} - M_{O\_e\_critical} = 524.325 \frac{\text{kip} \cdot \text{ft}}{\text{ft}} \]

Location of Resultant Force

\[ a_{e\_critical} := \frac{M_{A\_e\_net\_critical}}{V_{A\_e\_critical}} = 6.402 \text{ ft} \]

Eccentricity at base of wall

\[ e_{L\_e\_critical} := 0.5 \cdot L - a_{e\_critical} = 4.098 \text{ ft} \]

Eccentricity Soil Reinforcing Ratio

\[ \frac{e_{L\_e\_critical}}{L} = 0.195 \]

Is resultant within limiting eccentricity value

\[ \text{if } \left( e_{L\_e\_critical} > \frac{L}{4} \right) \text{ "Yes", "No"} = \text{"Yes"} \]

**CDR for Overturning at base of wall**

\[ CDR_{ot\_critical} := \frac{M_{A\_e\_critical}}{M_{O\_e\_critical}} = 2.56 \text{ MSEW 2.56} \]
**Strength I Values - Limiting Eccentricity (Service)**

Total Vertical Load without live load

\[ V_{A_e_{\text{ser}}} := V_1 + V_3 + V_4 = 81.900 \text{ kip/ft} \]

Resisting Moment without live load

\[ M_{A_e_{\text{ser}}} := M_{V1} + M_{V3} + M_{V4} = 859.950 \text{ kip-ft/ft} \]

Overturning Moment

\[ M_{O_e_{\text{ser}}} := M_{F1} + M_{F2} = 217.500 \text{ kip-ft/ft} \]

Net Moment

\[ M_{A_e_{\text{net_ser}}} := M_{A_e_{\text{ser}}} - M_{O_e_{\text{ser}}} = 642.450 \text{ kip-ft/ft} \]

Location of Resultant Force

\[ a_{e_{\text{ser}}} := \frac{M_{A_e_{\text{net_ser}}}}{V_{A_e_{\text{ser}}}} = 7.844 \text{ ft} \]

Eccentricity at base of wall

\[ e_{L_{e_{\text{ser}}}} := 0.5 \cdot L - a_{e_{\text{ser}}} = 2.656 \text{ ft} \]

Eccentricity Soil Reinforcing Ratio

\[ \frac{e_{L_{e_{\text{ser}}}}}{L} = 0.126 \]

CDR for Overturning at base of wall

\[ \frac{M_{A_e_{\text{ser}}}}{M_{O_e_{\text{ser}}}} = 3.954 \]

Is resultant within limiting eccentricity value

\[ \text{if } \left( e_{L_{e_{\text{ser}}}} > \frac{L}{4}, "No"; "Yes" \right) = "Yes" \]

**Capacity Demand Ratio - Overturning**

\[ CDR_{ot_{\text{ser}}} := \frac{M_{A_e_{\text{ser}}}}{M_{F1} + M_{F2}} = 3.954 \]
**Strength I Values for Bearing Capacity Check (Maximum)**

**Total Vertical Load**

\[
V_{A\_br\_max} := \gamma_{EV\max} \cdot V_{1} + \gamma_{LL\max} \cdot V_{2} + \gamma_{EV\max} \cdot V_{3} + \gamma_{EH\max} \cdot V_{4} + \gamma_{LL\max} \cdot V_{5} = 119.752 \text{ kip} \cdot \text{ft}
\]

**Resisting Moment with Live Load**

\[
M_{A\_br\_max} := \gamma_{EV\max} \cdot M_{V1} + \gamma_{LL\max} \cdot M_{V2} + \gamma_{EV\max} \cdot M_{V3} + \gamma_{EH\max} \cdot M_{V4} + \gamma_{LL\max} \cdot M_{V5} = 1257.401 \text{ kip} \cdot \text{ft}
\]

**Overturning Moment**

\[
M_{O\_br\_max} := \gamma_{EH\max} \cdot M_{F1} + \gamma_{LL\max} \cdot M_{F2} = 335.625 \text{ kip} \cdot \text{ft}
\]

**Net Moment**

\[
M_{br\_net\_max} := M_{A\_br\_max} - M_{O\_br\_max} = 921.776 \text{ kip} \cdot \text{ft}
\]

**Location of Resultant Force**

\[
a_{br\_max} := \frac{M_{br\_net\_max}}{V_{A\_br\_max}} = 7.697 \text{ ft}
\]

**Eccentricity at base of wall**

\[
e_{L\_br\_max} := 0.5 \cdot L - a_{br\_max} = 2.803 \text{ ft}
\]

Is resultant within limiting eccentricity value

\[
\text{if} \left( e_{L\_br\_max} > \frac{L}{4} \right) \Rightarrow \text{"No"}, \text{"Yes"} = \text{"Yes"}
\]

**Effective width at base of wall**

\[
B_{e\_max} := L - 2 \cdot e_{L\_br\_max} = 15.395 \text{ ft}
\]

**Maximum Applied Bearing Pressure at Base of Wall**

\[
\sigma_{br\_max} := \frac{V_{A\_br\_max}}{B_{e\_max}} = 7.779 \text{ ksf} \quad \text{MSEW 7.779 ksf}
\]
**Strength I Values for Bearing Capacity Check (Minimum)**

**Total Vertical Load with Live Load**

\[
V_{A\_br\_min} := \gamma_{EV\_min} \cdot V_1 + \gamma_{LL\_min} \cdot V_2 + \gamma_{EV\_min} \cdot V_3 + \gamma_{EH\_min} \cdot V_4 + \gamma_{LL\_min} \cdot V_5 = 91.087 \frac{\text{kip}}{\text{ft}}
\]

Resisting Moment with Live Load

\[
M_{A\_br\_min} := \gamma_{EV\_min} \cdot M_{V1} + \gamma_{LL\_min} \cdot M_{V2} + \gamma_{EV\_min} \cdot M_{V3} + \gamma_{EH\_min} \cdot M_{V4} + \gamma_{LL\_min} \cdot M_{V5} = 956.419 \frac{\text{kip\cdot ft}}{\text{ft}}
\]

**Overturning Moment**

\[
M_{O\_br\_min} := \gamma_{EH\_min} \cdot M_{F1} + \gamma_{LL\_min} \cdot M_{F2} = 227.625 \frac{\text{kip\cdot ft}}{\text{ft}}
\]

**Net Moment**

\[
M_{br\_net\_min} := M_{A\_br\_min} - M_{O\_br\_min} = 728.794 \frac{\text{kip\cdot ft}}{\text{ft}}
\]

**Location of Resultant Force**

\[
a_{br\_min} := \frac{M_{br\_net\_min}}{V_{A\_br\_min}} = 8.001 \text{ ft}
\]

**Eccentricity at base of wall**

\[
e_{L\_br\_min} := 0.5 \cdot L - a_{br\_min} = 2.499 \text{ ft}
\]

Is resultant within limiting eccentricity value

\[
\text{if} \left( e_{L\_br\_min} > \frac{L}{4}, "No", "Yes" \right) = "Yes"
\]

**Effective width at base of wall**

\[
B_{e\_min} := L - 2 \cdot e_{L\_br\_min} = 16.002 \text{ ft}
\]

**Maximum Applied Bearing Resistance at Base of Wall**

\[
\sigma_{br\_min} := \frac{V_{A\_br\_min}}{B_{e\_min}} = 5.692 \cdot \text{ksf}
\]
### Service I Values for Bearing Capacity Check

**Total Vertical Load with Live Load**

\[ V_{br\_service} := V_1 + V_2 + V_3 + V_4 + V_5 = 87.150 \text{kip} \]

**Resisting Moment with Live Load**

\[ M_{br\_service} := M_{V1} + M_{V2} + M_{V3} + M_{V4} + M_{V5} = 915.075 \text{kip}\cdot\text{ft} \]

**Overturning Moment**

\[ M_{O\_br\_service} := M_{F1} + M_{F2} = 217.500 \text{kip}\cdot\text{ft} \]

**Net Moment**

\[ M_{br\_net\_service} := M_{br\_service} - M_{O\_br\_service} = 697.575 \text{kip}\cdot\text{ft} \]

**Location of Resultant Force**

\[ a_{br\_service} := \frac{M_{br\_net\_service}}{V_{br\_service}} = 8.004 \text{ft} \]

**Eccentricity at base of wall**

\[ e_{L\_br\_service} := 0.5\cdot L - a_{br\_service} = 2.496 \text{ft} \]

**Is resultant within limiting eccentricity value**

\[ \text{if}\left( e_{L\_br\_service} > \frac{L}{4}, "No", "Yes" \right) = "Yes" \]

**Effective width at base of wall**

\[ B_{e\_service} := L - 2\cdot e_{L\_br\_service} = 16.009 \text{ft} \]

### Service Applied Bearing Resistance at Base of Wall

\[ \sigma_{br\_service} := \frac{V_{br\_service}}{L - 2\cdot e_{L\_br\_service}} = 5.444 \text{ksf} \]
Strength 1 Values for Bearing Resistance Check (Critical)

Resisting Moment

\[ M_{A_{\text{br\_critical}}} := \gamma_{\text{EVmin}} \cdot M_{V1} + \gamma_{\text{LLmin}} \cdot M_{V2} + \gamma_{\text{EVmin}} \cdot M_{V3} + \gamma_{\text{EHmax}} \cdot M_{V4} + \gamma_{\text{LLmax}} \cdot M_{V5} = 956.419 \cdot \frac{\text{kip} \cdot \text{ft}}{} \]

Overturning Moment

\[ M_{O_{\text{br\_critical}}} := \begin{cases} M_{O_{\text{br\_max}}} & \text{if } M_{O_{\text{br\_max}}} \geq M_{O_{\text{br\_min}}} \\ M_{O_{\text{br\_min}}} & \text{otherwise} \end{cases} \Rightarrow \left| \frac{335.625 \cdot \text{kip} \cdot \text{ft}}{} \right| \]

Net Moment

\[ M_{A_{\text{br\_net\_critical}}} := M_{A_{\text{br\_critical}}} - M_{O_{\text{br\_critical}}} = 620.794 \cdot \frac{\text{kip} \cdot \text{ft}}{} \]

Total Vertical Load

\[ V_{A_{\text{br\_critical}}} := \gamma_{\text{EVmin}} \cdot V_1 + \gamma_{\text{LLmin}} \cdot V_2 + \gamma_{\text{EVmin}} \cdot V_3 + \gamma_{\text{EHmax}} \cdot V_4 + \gamma_{\text{LLmax}} \cdot V_5 = 91.087 \cdot \frac{\text{kip}}{} \]

Location of Resultant Force

\[ a_{\text{br\_critical}} := \frac{M_{A_{\text{br\_net\_critical}}}}{V_{A_{\text{br\_critical}}}} = 6.815 \text{ ft} \]

Eccentricity at base of wall

\[ e_{L_{\text{br\_critical}}} := 0.5 \cdot L - a_{\text{br\_critical}} = 3.685 \text{ ft} \]

Is resultant within limiting eccentricity value

\[ \text{if} \left( e_{L_{\text{br\_critical}}} > \frac{L}{4} \right., "\text{No}" \ ; "\text{Yes}" \right) = "\text{Yes}" \]

Effective Width of Loaded at Base of Wall

\[ B_{\text{br\_critical}} := L - 2 \cdot e_{L_{\text{br\_critical}}} = 13.631 \text{ ft} \]

Maximum Applied Bearing Resistance at Base of Wall

\[ \sigma_{\text{br\_critical}} := \frac{V_{A_{\text{br\_critical}}}}{B_{\text{br\_critical}}} = 6.683 \cdot \text{ksf} \]
**Summary of Capacity Demand Ratios for External Stability**

**Capacity Demand Ratio - Sliding**

Minimum

\[
CDR_{sliding\_min} := \frac{V_1 + V_3 + V_4 + V_5}{\gamma_{EH\text{min}} F_1 + \gamma_{LL\text{min}} F_2} \cdot \tan(\phi_f) \cdot \phi_{sliding} = 2.68
\]

Maximum

\[
CDR_{sliding\_max} := \frac{\gamma_{EV\text{max}} V_1 + \gamma_{EV\text{max}} V_3 + \gamma_{EH\text{max}} V_4 + \gamma_{LL\text{max}} V_5}{\gamma_{EH\text{max}} F_1 + \gamma_{LL\text{max}} F_2} \cdot \tan(\phi_f) \cdot \phi_{sliding} = 2.38
\]

Critical

\[
CDR_{sliding\_critical} := \frac{V_1 + V_3 + V_4 + V_5}{\gamma_{EH\text{max}} F_1 + \gamma_{LL\text{max}} F_2} \cdot \tan(\phi_f) \cdot \phi_{sliding} = 1.76 \quad \text{MSEW} = 1.76
\]

Service

\[
CDR_{sliding\_service} := \frac{V_1 + V_3 + V_4 + V_5}{F_1 + F_2} \cdot \tan(\phi_f) = 2.69
\]

**Eccentricity Limit Check Summary**

Minimum Eccentricity Limit

\[
e_{L\text{e\_min}} = 2.779 \text{ ft} \quad \frac{e_{L\text{e\_min}}}{L} = 0.132 \quad \frac{M_{A\text{e\_min}}}{M_{O\text{e\_min}}} = 3.78
\]

Maximum Eccentricity Limit

\[
e_{L\text{e\_max}} = 3.036 \text{ ft} \quad \frac{e_{L\text{e\_max}}}{L} = 0.145 \quad \frac{M_{A\text{e\_max}}}{M_{O\text{e\_max}}} = 3.46
\]

Critical Eccentricity Limit

\[
e_{L\text{e\_critical}} = 4.098 \text{ ft} \quad \frac{e_{L\text{e\_critical}}}{L} = 0.195 \quad \frac{M_{A\text{e\_critical}}}{M_{O\text{e\_critical}}} = 2.56
\]

Service Eccentricity Limit

\[
e_{L\text{e\_ser}} = 2.656 \text{ ft} \quad \frac{e_{L\text{e\_ser}}}{L} = 0.126 \quad \frac{M_{A\text{e\_ser}}}{M_{O\text{e\_ser}}} = 3.95
\]

**Bearing Pressure and Eccentricity Summary**

Minimum Bearing Resistance

\[
\sigma_{br\_min} = 5.69 \text{ ksf} \quad e_{L\text{br\_min}} = 2.50 \text{ ft}
\]

Maximum Bearing Resistance

\[
\sigma_{br\_max} = 7.78 \text{ ksf} \quad e_{L\text{br\_max}} = 2.80 \text{ ft}
\]

Critical Bearing Resistance

\[
\sigma_{br\_critical} = 6.68 \text{ ksf} \quad e_{L\text{br\_critical}} = 3.68 \text{ ft}
\]

Service Bearing Resistance

\[
\sigma_{br\_service} = 5.44 \text{ ksf} \quad e_{L\text{br\_service}} = 2.50 \text{ ft}
\]
The vertical earth pressure is the force from the mass of soil over the effective length of the soil reinforcing. It is measured from the top of the structure to the depth of the soil reinforcing. The user can define which method to use.

**MSE-Pro Method** – this method uses the actual mass of soil that is contained at the interface of the failure surface to the extent of the reinforced volume of soil, and from the top of the structure to the top of the surcharge.

**MSEW Method** – this method uses an average surcharge height and applies it uniformly over each element. The average height is based on design height of the wall, H. It uses 70% of this value to set the average.

**Average Method** – this method uses an average surcharge height and applies it uniformly over each element. The average height is based on design height of the wall, H. It uses 70% of this value to set the average.
Please define the design method you are trying to match ............................................  
Slope := "MSE-Pro"

\[
\sigma_s(d_i) := \begin{cases} 
\sigma_s & \text{if Slope} = "\text{MSE-Pro}" \\
\frac{1}{2} \cdot \gamma_m \cdot L \cdot \tan(\beta_s) & \text{if } X_s \geq L \\
\frac{1}{2} \cdot \gamma_m \cdot S \cdot X_s + \gamma_m \cdot S \cdot (L - X_s) & \text{otherwise} \\
\end{cases}
\]

\[
\text{return } \sigma_s
\]

\[
\sigma_s \leftarrow \frac{1}{2} \cdot \gamma_m \cdot 0.7 \cdot H \cdot \tan(\beta_s) & \text{if } X_s \geq L \\
\frac{1}{2} \cdot \gamma_m \cdot S \cdot X_s + \gamma_m \cdot S \cdot (0.7 \cdot H_m - X_s) & \text{otherwise} \\
\sigma_s \leftarrow \frac{1}{0.7 \cdot H_m} & \text{return } \sigma_s
\]

\[
\text{if Slope} = "\text{MSEW}" \\
\sigma_s \leftarrow \frac{1}{2} \cdot \gamma_m \cdot L \cdot \tan(\beta_s) & \text{if } X_s \geq L \\
\frac{1}{2} \cdot \gamma_m \cdot S \cdot X_s + \gamma_m \cdot S \cdot (L - X_s) & \text{otherwise} \\
\sigma_s \leftarrow \frac{1}{0.7 \cdot H_m} & \text{return } \sigma_s
\]

\[
\text{if Slope} = "\text{Average}" \\
X_m \leftarrow L - L_e(d_i) \\
X_1 \leftarrow 0 & \text{if } X_s - X_m \leq 0 \\
X_s - X_m & \text{otherwise} \\
Y_1 \leftarrow X_1 \cdot \frac{S}{X_s} \\
Y_2 \leftarrow 0 & \text{if } X_s - X_m \leq 0 \\
S - Y_1 & \text{otherwise} \\
\text{Area}_s \leftarrow \frac{1}{2} X_1 \cdot Y_1 + X_1 \cdot Y_2 \cdot (L - X_s) \cdot S \\
S_{AVG} \leftarrow \frac{X_m \cdot L - L_e(d_i)}{S} & \text{if } X_s - X_m \leq 0 \\
\frac{\text{Area}_s}{L_e(d_i)} & \text{otherwise} \\
\text{return } \gamma_m \cdot S_{AVG}
\]

\text{return } "\text{No Solution}" & \text{otherwise}
HORIZONTAL FORCES FOR PULLOUT

There is conflicting requirements for what horizontal forces are to be included in the calculation of the maximum horizontal force required to be resisted. AASHTO Article 11.10.6.3.2 – Reinforcement Pullout Design states that traffic loads are neglected in pullout calculations and references figure 11.10.6.2.1-1 for information. FHWA and the software program MSEW use the traffic. The designer needs to be cautious in omitting the traffic load.

REINFORCEMENT PULLOUT CAPACITY

Pullout Friction Factor for soil reinforcing at depth \( d \), (Taken from the top of the structure)

The resistance to pullout is a function of the soil mass that is over the soil reinforcing length of embedment.
The resistance to pullout is a function of the soil reinforcing system. Steel systems derived pullout for both passive resistance and friction. Resistance to pullout is a function of the ultimate tensile load that causes the outward sliding of the reinforcement through the backfill.

**HEIGHT OF SOIL IN SURCHARGE WEDGE OVER SOIL REINFORCING**

\[
\text{Area}_S(d) := \begin{cases} 
X_m & \leftarrow L - L_e(d) \\
X_1 & \leftarrow 0 \text{ if } X_s - X_m \leq 0 \\
& \text{otherwise} \\
Y_1 & \leftarrow X_1 \cdot \frac{S}{X_s} \\
Y_2 & \leftarrow 0 \text{ if } X_s - X_m \leq 0 \\
& \text{otherwise} \\
\text{Area}_S & \leftarrow \frac{1}{2}X_1 \cdot Y_1 + X_1 \cdot Y_2 + (L - X_s) \cdot S
\end{cases}
\]

\[
S_{\text{Average}}(d) := \begin{cases} 
X_m & \leftarrow L - L_e(d) \\
S & \text{if } X_s - X_m \leq 0 \\
\text{Area}_S(d) & \leftarrow \frac{\text{Area}_S(d)}{L_e(d)} \text{ otherwise}
\end{cases}
\]
**INTERNAL STABILITY**

**VERTICAL PRESSURE FROM SELF-WEIGHT OF SOIL**

\[ d_i = \gamma_m (d_i) = \]

<table>
<thead>
<tr>
<th>( d_i ) (ft)</th>
<th>( \gamma_m (d_i) ) (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.250</td>
<td>0.163</td>
</tr>
<tr>
<td>3.750</td>
<td>0.488</td>
</tr>
<tr>
<td>6.250</td>
<td>0.813</td>
</tr>
<tr>
<td>8.750</td>
<td>1.137</td>
</tr>
<tr>
<td>11.250</td>
<td>1.463</td>
</tr>
<tr>
<td>13.750</td>
<td>1.788</td>
</tr>
<tr>
<td>16.250</td>
<td>2.113</td>
</tr>
<tr>
<td>18.750</td>
<td>2.438</td>
</tr>
<tr>
<td>21.250</td>
<td>2.762</td>
</tr>
<tr>
<td>23.750</td>
<td>3.088</td>
</tr>
<tr>
<td>26.250</td>
<td>3.412</td>
</tr>
<tr>
<td>28.750</td>
<td>3.737</td>
</tr>
</tbody>
</table>
Function To Determine Required Area of Steel At Each Depth

\[ A_{\text{req}}(d_i) := \left. \begin{align*}
\sigma_v & \leftarrow \gamma_m(d_i) + \sigma_q + \sigma_s(d_i) \\
S_{vi} & \leftarrow S_{vi}(d_i) \\
K_i & \leftarrow K_i(d_i) \\
T_i & \leftarrow \left( \sigma_v \cdot K_i \cdot \gamma_{EV_{\text{max}}} \right) \cdot \left( S_{vi} \cdot L_p \right) \\
A_{\text{req}} & \leftarrow \frac{T_i}{\phi_F Y} \\
A_{\text{req}} & \end{align*} \right. \]

nGS(d_i) := 
\[ A_{\text{req}} \leftarrow A_{\text{req}}(d_i) \]
\[ n_{di} \leftarrow 3 \]
\[ A_{\text{GS}} \leftarrow n_{di} \cdot A_{SR} \]
while \[ A_{\text{req}} > A_{\text{GS}} \]
\[ n_{di} \leftarrow n_{di} + 1 \]
\[ A_{\text{GS}} \leftarrow n_{di} \cdot A_{SR} \]
\[ n_{di} \]
# TENSION AND STRESS AT EACH LEVEL

Local Tension

\[ T_{ig}(d_i) := \left[ \gamma_m(d_i) + \sigma_q + \sigma_s(d_i) \cdot K_r(d_i) \cdot \gamma_{EVmax} \right] \cdot (S_{vr}(d_i) \cdot L_p) \]

Local Steel Stress

\[ \sigma_{SR}(d_i) := \frac{T_{ig}(d_i)}{n_{GS}(d_i) \cdot A_{SR}} \]

Local Capacity Demand Ratio

\[ CDR_r(d_i) := \frac{\phi_R \cdot F_Y}{\sigma_{SR}(d_i)} \]

<table>
<thead>
<tr>
<th>( H - (d_i) )</th>
<th>( T_{ig}(d_i) )</th>
<th>( n_{GS}(d_i) )</th>
<th>( \sigma_{SR}(d_i) )</th>
<th>( CDR_r(d_i) )</th>
<th>( S_{vr}(d_i) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>28.75 ft</td>
<td>6.568 kip</td>
<td>3</td>
<td>13.671 ksi</td>
<td>3.57</td>
<td>2.500 ft</td>
</tr>
<tr>
<td>26.25</td>
<td>11.303</td>
<td>3</td>
<td>23.526</td>
<td>2.07</td>
<td>2.500</td>
</tr>
<tr>
<td>23.75</td>
<td>15.651</td>
<td>3</td>
<td>32.574</td>
<td>1.50</td>
<td>2.500</td>
</tr>
<tr>
<td>21.25</td>
<td>19.610</td>
<td>3</td>
<td>40.816</td>
<td>1.19</td>
<td>2.500</td>
</tr>
<tr>
<td>18.75</td>
<td>23.182</td>
<td>3</td>
<td>48.251</td>
<td>1.01</td>
<td>2.500</td>
</tr>
<tr>
<td>16.25</td>
<td>26.367</td>
<td>4</td>
<td>41.159</td>
<td>1.18</td>
<td>2.500</td>
</tr>
<tr>
<td>13.75</td>
<td>29.164</td>
<td>4</td>
<td>45.525</td>
<td>1.07</td>
<td>2.500</td>
</tr>
<tr>
<td>11.25</td>
<td>31.573</td>
<td>5</td>
<td>39.429</td>
<td>1.24</td>
<td>2.500</td>
</tr>
<tr>
<td>8.75</td>
<td>34.493</td>
<td>5</td>
<td>43.075</td>
<td>1.13</td>
<td>2.500</td>
</tr>
<tr>
<td>6.25</td>
<td>38.214</td>
<td>5</td>
<td>47.723</td>
<td>1.02</td>
<td>2.500</td>
</tr>
<tr>
<td>3.75</td>
<td>41.935</td>
<td>6</td>
<td>43.641</td>
<td>1.12</td>
<td>2.500</td>
</tr>
<tr>
<td>1.25</td>
<td>45.657</td>
<td>6</td>
<td>47.514</td>
<td>1.03</td>
<td>2.500</td>
</tr>
</tbody>
</table>
PULLOUT FORMULATION AT EACH LEVEL

\[
\sigma_{s,p0}(d_i) := \begin{cases} 
\text{if Slope} = "\text{MSE-Pro}" \quad & \\
\sigma_S \leftarrow \frac{1}{2} \gamma_m \cdot L \cdot \tan(\beta_s) \quad \text{if} \quad X_S \geq L \\
\sigma_S \leftarrow \frac{1}{2} \gamma_m \cdot S \cdot X_S + \gamma_m \cdot S \cdot (L - X_S) \quad \text{otherwise} \\
\text{return} \quad \sigma_S \\
\text{if Slope} = "\text{MSEW}" \quad & \\
\sigma_S \leftarrow \frac{1}{2} \gamma_m \cdot 0.7 \cdot H \cdot \tan(\beta_s) \quad \text{if} \quad X_S \geq L \\
\sigma_S \leftarrow \frac{1}{2} \gamma_m \cdot S \cdot X_S + \gamma_m \cdot S \cdot (0.7 \cdot H_m - X_S) \quad \text{otherwise} \\
\text{return} \quad \sigma_S \\
\text{if Slope} = "\text{Average}" \quad & \\
X_m \leftarrow L - L_e(d_i) \\
X_1 \leftarrow \begin{cases} 
0 \quad \text{if} \quad X_S - X_m \leq 0 \\
X_S - X_m \quad \text{otherwise} \\
Y_1 \leftarrow X_1 \cdot \frac{S}{X_S} \\
Y_2 \leftarrow \begin{cases} 
0 \quad \text{if} \quad X_S - X_m \leq 0 \\
S - Y_1 \quad \text{otherwise} \\
\text{Area}_S \leftarrow \frac{1}{2} X_1 \cdot Y_1 + X_1 \cdot Y_2 + (L - X_S) \cdot S \\
S_{AVG} \leftarrow \begin{cases} 
X_m \leftarrow L - L_e(d_i) \\
S \quad \text{if} \quad X_S - X_m \leq 0 \\
\text{Area}_S \quad \text{otherwise} \\
\text{return} \quad \gamma_m \cdot S_{AVG} \\
\text{return} \quad "\text{No Solution}" \quad \text{otherwise} 
\end{cases}
\end{cases}
\end{cases}
\]
Resisting Pullout Force

\[ P_r(d_i) := \alpha \cdot C \cdot \phi \cdot F_{\text{star}}(d_i) \cdot L_e(d_i) \cdot n_{\text{GS}}(d_i) \cdot W_{\text{GS}} \cdot [\gamma_m \cdot (d_i) + \sigma_{s \text{-po}}(d_i)] \]

Maximum tension at each reinforcement elevation

\[ T_{\text{max-po}}(d_i) := [\gamma_m \cdot (d_i) + \sigma_s(d_i) + \text{if } \text{LLinclude} = "\text{Yes}" \cdot \gamma_{\text{EVmax}} \cdot (S_{\text{VR}}(d_i) \cdot L_p) \]

Pullout Capacity Demand Ratio

\[ \text{CDR}_{\text{po}}(d_i) := \frac{P_r(d_i)}{T_{\text{max-po}}(d_i)} \]

Pullout Summary

<table>
<thead>
<tr>
<th>(d_i)</th>
<th>(L_e(d_i))</th>
<th>(P_r(d_i))</th>
<th>(T_{\text{max-po}}(d_i))</th>
<th>(n_{\text{GS}}(d_i))</th>
<th>(F_{\text{star}}(d_i))</th>
<th>(\text{CDR}_{\text{po}}(d_i))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.250 ft</td>
<td>12.000 ft</td>
<td>4.250 kip</td>
<td>2.587 kip</td>
<td>3</td>
<td>2.422</td>
<td>1.64</td>
</tr>
<tr>
<td>3.750</td>
<td>12.000</td>
<td>11.929</td>
<td>7.472</td>
<td>3</td>
<td>2.266</td>
<td>1.60</td>
</tr>
<tr>
<td>6.250</td>
<td>12.000</td>
<td>18.510</td>
<td>11.968</td>
<td>3</td>
<td>2.109</td>
<td>1.55</td>
</tr>
<tr>
<td>8.750</td>
<td>12.000</td>
<td>23.994</td>
<td>16.077</td>
<td>3</td>
<td>1.953</td>
<td>1.49</td>
</tr>
<tr>
<td>11.250</td>
<td>12.000</td>
<td>28.382</td>
<td>19.798</td>
<td>3</td>
<td>1.797</td>
<td>1.43</td>
</tr>
<tr>
<td>13.750</td>
<td>12.000</td>
<td>42.230</td>
<td>23.132</td>
<td>4</td>
<td>1.641</td>
<td>1.83</td>
</tr>
<tr>
<td>16.250</td>
<td>12.750</td>
<td>47.977</td>
<td>26.078</td>
<td>4</td>
<td>1.484</td>
<td>1.84</td>
</tr>
<tr>
<td>18.750</td>
<td>14.250</td>
<td>69.197</td>
<td>28.636</td>
<td>5</td>
<td>1.328</td>
<td>2.42</td>
</tr>
<tr>
<td>21.250</td>
<td>15.750</td>
<td>81.580</td>
<td>31.630</td>
<td>5</td>
<td>1.250</td>
<td>2.58</td>
</tr>
<tr>
<td>23.750</td>
<td>17.250</td>
<td>99.861</td>
<td>35.352</td>
<td>5</td>
<td>1.250</td>
<td>2.82</td>
</tr>
<tr>
<td>26.250</td>
<td>18.750</td>
<td>143.965</td>
<td>39.073</td>
<td>6</td>
<td>1.250</td>
<td>3.68</td>
</tr>
<tr>
<td>28.750</td>
<td>20.250</td>
<td>170.290</td>
<td>42.794</td>
<td>6</td>
<td>1.250</td>
<td>3.98</td>
</tr>
</tbody>
</table>
## CALCULATION SUMMARY

**CLIENT:** Lawrence-Lynch Corp.  
**VAWS P.N.** 2014-02388  
**PROJECT:** Cape Cod Rail Trail Extension - Phase 1  
**CLIENT P.N.**  
**CALC BY:** TPT  
**DATE:** Dec 24, 2015  
**CHKD BY:** CMS  
**DATE:** Dec 24, 2015  
**SUBJECT:** LRFD SIMPLIFIED METHOD  
**SPECIFICATION:** AASHTO LRFD Bridge Design Specification 2014 - 7th Edition

### Structure Parameters

- **Wall Design Height:** $H = 30.00 \text{ ft}$  
  - **Surcharge Height:** $S = 0.000 \text{ ft}$
- **Soil Reinforcing Length:** $L = 21.00 \text{ ft}$  
  - **Distance to Full Height:** $X_S = 0.000 \text{ ft}$
- **Live Load Pressure:** $\sigma_q = 250.000 \text{ psf}$
- **Design Life:** $Y_t = 75.000 \text{ yr}$

### Soil Strength Parameters

- **Reinforced Soil mass**
  - Unit Weight: $\gamma_m = 130 \text{pcf}$  
  - Internal Friction Angle: $\phi_m = 34 \text{ deg}$
- **Retained Soil Mass (Random)**
  - Unit Weight: $\gamma_r = 120 \text{pcf}$  
  - Internal Friction Angle: $\phi_r = 30 \text{ deg}$
- **Foundation**
  - Unit Weight: $\gamma_f = 125 \text{pcf}$  
  - Internal Friction Angle: $\phi_f = 34 \text{ deg}$

### Static External Stability Capacity Demand Ratios

#### CDR SLIDING

- $\text{CDR}_{\text{sliding\_max}} = 2.377$  
  - $\text{CDR}_{\text{sliding\_min}} = 2.685$  
  - $\text{CDR}_{\text{sliding\_critical}} = 1.761$  
  - $\text{CDR}_{\text{sliding\_service}} = 2.695$

#### LIMITING ECCENTRICITY

- $\frac{e_{L\_e\_max}}{L} = 0.145$  
  - $\frac{e_{L\_e\_min}}{L} = 0.132$  
  - $\frac{e_{L\_e\_critical}}{L} = 0.195$  
  - $\frac{e_{L\_e\_ser}}{L} = 0.126$

- $\text{CDR}_{\text{ot\_max}} = 3.459$  
  - $\text{CDR}_{\text{ot\_min}} = 3.778$  
  - $\text{CDR}_{\text{ot\_critical}} = 2.562$  
  - $\text{CDR}_{\text{ot\_ser}} = 3.954$

#### BEARING PRESSURE

- $\sigma_{br\_max} = 7.78 \text{ ksf}$  
  - $\sigma_{br\_min} = 5.69 \text{ ksf}$  
  - $\sigma_{br\_critical} = 6.68 \text{ ksf}$  
  - $\sigma_{br\_service} = 5.444 \text{ ksf}$
### Internal Stability Capacity Demand Ratios

<table>
<thead>
<tr>
<th>Depth From Top of Wall To Layer (ft)</th>
<th>Number of Soil Reinforcing Per Row</th>
<th>CDR for Rupture</th>
<th>CDR for Pullout</th>
<th>Internal Earth Coefficient</th>
<th>Depth From Top of Failure Plane To Layer (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.25</td>
<td>3</td>
<td>3.57</td>
<td>1.64</td>
<td>0.47</td>
<td>1.250</td>
</tr>
<tr>
<td>3.75</td>
<td>3</td>
<td>2.07</td>
<td>1.60</td>
<td>0.45</td>
<td>3.750</td>
</tr>
<tr>
<td>6.25</td>
<td>3</td>
<td>1.50</td>
<td>1.55</td>
<td>0.44</td>
<td>6.250</td>
</tr>
<tr>
<td>8.75</td>
<td>3</td>
<td>1.19</td>
<td>1.49</td>
<td>0.42</td>
<td>8.750</td>
</tr>
<tr>
<td>11.25</td>
<td>3</td>
<td>1.01</td>
<td>1.43</td>
<td>0.40</td>
<td>11.250</td>
</tr>
<tr>
<td>13.75</td>
<td>4</td>
<td>1.18</td>
<td>1.83</td>
<td>0.38</td>
<td>13.750</td>
</tr>
<tr>
<td>16.25</td>
<td>4</td>
<td>1.07</td>
<td>1.84</td>
<td>0.37</td>
<td>16.250</td>
</tr>
<tr>
<td>18.75</td>
<td>5</td>
<td>1.24</td>
<td>2.42</td>
<td>0.35</td>
<td>18.750</td>
</tr>
<tr>
<td>21.25</td>
<td>5</td>
<td>1.13</td>
<td>2.58</td>
<td>0.34</td>
<td>21.250</td>
</tr>
<tr>
<td>23.75</td>
<td>5</td>
<td>1.02</td>
<td>2.82</td>
<td>0.34</td>
<td>23.750</td>
</tr>
<tr>
<td>26.25</td>
<td>6</td>
<td>1.12</td>
<td>3.68</td>
<td>0.34</td>
<td>26.250</td>
</tr>
<tr>
<td>28.75</td>
<td>6</td>
<td>1.03</td>
<td>3.98</td>
<td>0.34</td>
<td>28.750</td>
</tr>
</tbody>
</table>

**Note:** MSEW CDR’s for pullout and rupture are within 1% of the values in MSE-Pro

**Grid-Strip Soil Reinforcing Configuration**

GS-11 = W11.0 x W11.0 - 2” x 12”
MSE-PRO CALCULATIONS

Level Surcharge with Traffic Live Load
**SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP**

### Project Information
- **VAWS #:** VW14-02388
- **Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1
- **Location:** Barnstable County, MA
- **Project #:** CM/TAP/TI-002S(758)X
- **Design Engineer:** cms / chkd by tpt

### Soil Reinforcing Schedule - Grid Strip

<table>
<thead>
<tr>
<th>Mat-Type</th>
<th>Long Size (W)</th>
<th>Tran Size (W)</th>
<th>Long-Space (ft)</th>
<th>Tran-Space (ft)</th>
<th>Mat Width (ft)</th>
<th>Number Of Mats/Row</th>
<th>Max Stiff. (in²)</th>
<th>Ac (in²)</th>
<th>F* - 0</th>
<th>F* - 20</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11.0</td>
<td>11.0</td>
<td>0.167</td>
<td>1.000</td>
<td>0.167</td>
<td>3</td>
<td>6160.000</td>
<td>0.478</td>
<td>2.500</td>
<td>1.250</td>
</tr>
</tbody>
</table>

### Soil Parameters

- **Unit Weight:** 0.130 (kcf)
- **Friction Angle:** 34 (deg)

### Live Load Parameters
- **Equiv. Height of Soil:** 2.000 (ft)
- **Equiv. Unit Weight of Soil:** 0.125 (kcf)

### Internal Stability Parameters
- **Steel Yield Stress:** 65.000 (ksi)
- **Yield Coefficient:** 0.750
- **Design Life:** 75.0 years

### Design Options
- **LRFD Procedure**
  - Simplified Method
  - Live load is applied to T-max and pullout calculations - AASHTO Figure 11.10.6.2.1
  - Vertical Earth (EV) load is not used for all internal loads - AASHTO 11.10.6.2.1
  - K-Ratio is calculated from the top of the structure or top of coping

### General Notes

1. AASHTO LRFD Bridge Design
2. Simplified CG
3. Grid-S
4. F* - 2.5
5. Level Surcharge
SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

Project Information
- VAWS #: VW14-02388
- Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1
- Location: Barnstable County, MA
- Project #: CM/TAP/TI-002S(758)X
- Design Engineer: cms / chkd by tpt
- Wall Name: Verification Calculation

Load and Resistance Factor Design Input Data

<table>
<thead>
<tr>
<th>Load Factor</th>
<th>Maximum</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load factor for vertical earth pressure (EV)</td>
<td>1.35</td>
<td>1.00</td>
</tr>
<tr>
<td>Load factor for earth surcharge (ES)</td>
<td>1.50</td>
<td>0.75</td>
</tr>
<tr>
<td>Load factor for traffic live load (LS)</td>
<td>1.75</td>
<td>1.75</td>
</tr>
<tr>
<td>Load factor for horizontal earth pressure (EH)</td>
<td>1.50</td>
<td>0.90</td>
</tr>
<tr>
<td>Load factor for seismic (EQ)</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Load factor for structural components (DC)</td>
<td>1.25</td>
<td>0.90</td>
</tr>
<tr>
<td>Load factor for water (WA)</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Load factor for live load extreme event (LL)</td>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>Load factor for vehicular impact (CT)</td>
<td>1.50</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Resistance Factor

<table>
<thead>
<tr>
<th></th>
<th>Static</th>
<th>Seismic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing resistance factor</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Sliding resistance factor</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Tensile resistance factor</td>
<td>0.75</td>
<td>0.85</td>
</tr>
<tr>
<td>Pullout resistance factor</td>
<td>0.90</td>
<td>1.20</td>
</tr>
<tr>
<td>Extreme event resistance factor</td>
<td>1.00</td>
<td></td>
</tr>
</tbody>
</table>

Variation of Lateral Earth Pressure Coefficient with Depth (K/Ka)

<table>
<thead>
<tr>
<th>Depth (Z)</th>
<th>Rupture</th>
<th>Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00 (ft)</td>
<td>1.70</td>
<td>1.70</td>
</tr>
<tr>
<td>20.00 (ft)</td>
<td>1.20</td>
<td>1.20</td>
</tr>
</tbody>
</table>
**SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP**

### Project Information

- **VAWS #:** VW14-02388
- **Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1
- **Location:** Barnstable County, MA
- **Project #:** CM/TAP/TI-002S(758)X
- **Design Engineer:** cms / chkd by tpt
- **Wall Name:** Verification Calculation

### External And Internal Stability Calculation Summary

#### Structure Parameters

<table>
<thead>
<tr>
<th>H max (ft)</th>
<th>B min (ft)</th>
<th>B Ext. (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>β (deg)</th>
<th>β i (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>30.000</td>
<td>21.000</td>
<td>21.000</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

#### External Active Earth Pressure Coefficient: 0.333

<table>
<thead>
<tr>
<th>CDR Sliding</th>
<th>Limiting Eccentricity (ft)</th>
<th>CDR Overturning</th>
<th>Bearing Pressure (ksf)</th>
<th>Eccentricity CDR</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.68</td>
<td>2.78</td>
<td>3.78</td>
<td>5.69</td>
<td>2.50</td>
<td>0.500</td>
</tr>
<tr>
<td>2.38</td>
<td>3.04</td>
<td>3.46</td>
<td>7.78</td>
<td>2.80</td>
<td>0.500</td>
</tr>
<tr>
<td>1.76</td>
<td>4.10</td>
<td>2.56</td>
<td>6.68</td>
<td>3.68</td>
<td>0.500</td>
</tr>
</tbody>
</table>

#### Internal Active Earth Pressure Coefficient: 0.283

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Internal Earth Coeff.</th>
<th>Rupture CDR</th>
<th>Pullout CDR</th>
<th>F*</th>
<th>Le (ft)</th>
<th>H/B Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.25</td>
<td>21.00</td>
<td>3</td>
<td>0.47</td>
<td>3.55</td>
<td>1.65</td>
<td>2.42</td>
<td>12.00</td>
<td>0.70</td>
</tr>
<tr>
<td>3.75</td>
<td>21.00</td>
<td>3</td>
<td>0.45</td>
<td>2.06</td>
<td>1.60</td>
<td>2.27</td>
<td>12.00</td>
<td>0.70</td>
</tr>
<tr>
<td>6.25</td>
<td>21.00</td>
<td>3</td>
<td>0.44</td>
<td>1.49</td>
<td>1.55</td>
<td>2.11</td>
<td>12.00</td>
<td>0.70</td>
</tr>
<tr>
<td>8.75</td>
<td>21.00</td>
<td>3</td>
<td>0.42</td>
<td>1.19</td>
<td>1.50</td>
<td>1.95</td>
<td>12.00</td>
<td>0.70</td>
</tr>
<tr>
<td>11.25</td>
<td>21.00</td>
<td>3</td>
<td>0.40</td>
<td>1.01</td>
<td>1.44</td>
<td>1.80</td>
<td>12.00</td>
<td>0.70</td>
</tr>
<tr>
<td>13.75</td>
<td>21.00</td>
<td>4</td>
<td>0.38</td>
<td>1.18</td>
<td>1.83</td>
<td>1.64</td>
<td>12.00</td>
<td>0.70</td>
</tr>
<tr>
<td>16.25</td>
<td>21.00</td>
<td>4</td>
<td>0.37</td>
<td>1.07</td>
<td>1.84</td>
<td>1.48</td>
<td>12.75</td>
<td>0.70</td>
</tr>
<tr>
<td>18.75</td>
<td>21.00</td>
<td>5</td>
<td>0.35</td>
<td>1.23</td>
<td>2.42</td>
<td>1.33</td>
<td>14.25</td>
<td>0.70</td>
</tr>
<tr>
<td>21.25</td>
<td>21.00</td>
<td>5</td>
<td>0.34</td>
<td>1.13</td>
<td>2.58</td>
<td>1.25</td>
<td>15.75</td>
<td>0.70</td>
</tr>
<tr>
<td>23.75</td>
<td>21.00</td>
<td>5</td>
<td>0.34</td>
<td>1.02</td>
<td>2.83</td>
<td>1.25</td>
<td>17.25</td>
<td>0.70</td>
</tr>
<tr>
<td>26.25</td>
<td>21.00</td>
<td>6</td>
<td>0.34</td>
<td>1.11</td>
<td>3.69</td>
<td>1.25</td>
<td>18.75</td>
<td>0.70</td>
</tr>
<tr>
<td>28.75</td>
<td>21.00</td>
<td>6</td>
<td>0.34</td>
<td>1.02</td>
<td>3.99</td>
<td>1.25</td>
<td>20.25</td>
<td>0.70</td>
</tr>
</tbody>
</table>
### Static Internal Stability - Rupture Force Results

Rec. #1 - H = 30.00 (ft) B = 21.00 (ft)

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Internal Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Static CDR Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.25</td>
<td>21.00</td>
<td>3</td>
<td>25.00</td>
<td>0.47</td>
<td>0.56</td>
<td>0.26</td>
<td>6.57</td>
<td>3.55</td>
</tr>
<tr>
<td>3.75</td>
<td>21.00</td>
<td>3</td>
<td>25.00</td>
<td>0.45</td>
<td>1.00</td>
<td>0.45</td>
<td>11.30</td>
<td>2.06</td>
</tr>
<tr>
<td>6.25</td>
<td>21.00</td>
<td>3</td>
<td>25.00</td>
<td>0.44</td>
<td>1.43</td>
<td>0.63</td>
<td>15.65</td>
<td>1.49</td>
</tr>
<tr>
<td>8.75</td>
<td>21.00</td>
<td>3</td>
<td>25.00</td>
<td>0.42</td>
<td>1.87</td>
<td>0.78</td>
<td>19.61</td>
<td>1.19</td>
</tr>
<tr>
<td>11.25</td>
<td>21.00</td>
<td>3</td>
<td>25.00</td>
<td>0.40</td>
<td>2.31</td>
<td>0.93</td>
<td>23.18</td>
<td>1.01</td>
</tr>
<tr>
<td>13.75</td>
<td>21.00</td>
<td>4</td>
<td>25.00</td>
<td>0.38</td>
<td>2.75</td>
<td>1.05</td>
<td>26.37</td>
<td>1.18</td>
</tr>
<tr>
<td>16.25</td>
<td>21.00</td>
<td>4</td>
<td>25.00</td>
<td>0.37</td>
<td>3.19</td>
<td>1.17</td>
<td>29.16</td>
<td>1.07</td>
</tr>
<tr>
<td>18.75</td>
<td>21.00</td>
<td>5</td>
<td>25.00</td>
<td>0.35</td>
<td>3.63</td>
<td>1.26</td>
<td>31.57</td>
<td>1.23</td>
</tr>
<tr>
<td>21.25</td>
<td>21.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>4.07</td>
<td>1.38</td>
<td>34.49</td>
<td>1.13</td>
</tr>
<tr>
<td>23.75</td>
<td>21.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>4.51</td>
<td>1.53</td>
<td>38.21</td>
<td>1.02</td>
</tr>
<tr>
<td>26.25</td>
<td>21.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>4.94</td>
<td>1.68</td>
<td>41.94</td>
<td>1.11</td>
</tr>
<tr>
<td>28.75</td>
<td>21.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>5.38</td>
<td>1.83</td>
<td>45.66</td>
<td>1.02</td>
</tr>
</tbody>
</table>

### Static Internal Stability - Pullout Force Results

Rec. #1 - H = 30.00 (ft) B = 21.00 (ft)

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horz. Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Le (ft)</th>
<th>F*</th>
<th>Pr (Kips)</th>
<th>Static CDR Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.25</td>
<td>21.00</td>
<td>3</td>
<td>25.00</td>
<td>0.47</td>
<td>0.16</td>
<td>0.08</td>
<td>2.59</td>
<td>12.00</td>
<td>2.42</td>
<td>4.26</td>
<td>2.42</td>
</tr>
<tr>
<td>3.75</td>
<td>21.00</td>
<td>3</td>
<td>25.00</td>
<td>0.45</td>
<td>0.49</td>
<td>0.22</td>
<td>7.47</td>
<td>12.00</td>
<td>2.27</td>
<td>11.95</td>
<td>1.60</td>
</tr>
<tr>
<td>6.25</td>
<td>21.00</td>
<td>3</td>
<td>25.00</td>
<td>0.44</td>
<td>0.81</td>
<td>0.35</td>
<td>11.97</td>
<td>12.00</td>
<td>2.11</td>
<td>18.55</td>
<td>1.55</td>
</tr>
<tr>
<td>8.75</td>
<td>21.00</td>
<td>3</td>
<td>25.00</td>
<td>0.42</td>
<td>1.14</td>
<td>0.48</td>
<td>16.08</td>
<td>12.00</td>
<td>1.95</td>
<td>24.04</td>
<td>1.50</td>
</tr>
<tr>
<td>11.25</td>
<td>21.00</td>
<td>3</td>
<td>25.00</td>
<td>0.40</td>
<td>1.46</td>
<td>0.59</td>
<td>19.80</td>
<td>12.00</td>
<td>1.80</td>
<td>28.44</td>
<td>1.44</td>
</tr>
<tr>
<td>13.75</td>
<td>21.00</td>
<td>4</td>
<td>25.00</td>
<td>0.38</td>
<td>1.79</td>
<td>0.69</td>
<td>23.13</td>
<td>12.00</td>
<td>1.64</td>
<td>42.31</td>
<td>1.83</td>
</tr>
<tr>
<td>16.25</td>
<td>21.00</td>
<td>4</td>
<td>25.00</td>
<td>0.37</td>
<td>2.11</td>
<td>0.77</td>
<td>26.08</td>
<td>12.75</td>
<td>1.48</td>
<td>48.07</td>
<td>1.84</td>
</tr>
<tr>
<td>18.75</td>
<td>21.00</td>
<td>5</td>
<td>25.00</td>
<td>0.35</td>
<td>2.44</td>
<td>0.85</td>
<td>28.64</td>
<td>14.25</td>
<td>1.33</td>
<td>69.34</td>
<td>2.42</td>
</tr>
<tr>
<td>21.25</td>
<td>21.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>2.76</td>
<td>0.94</td>
<td>31.63</td>
<td>15.75</td>
<td>1.25</td>
<td>81.74</td>
<td>2.58</td>
</tr>
<tr>
<td>23.75</td>
<td>21.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>3.09</td>
<td>1.05</td>
<td>35.35</td>
<td>17.25</td>
<td>1.25</td>
<td>100.06</td>
<td>2.83</td>
</tr>
<tr>
<td>26.25</td>
<td>21.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>3.41</td>
<td>1.16</td>
<td>39.07</td>
<td>18.75</td>
<td>1.25</td>
<td>144.25</td>
<td>3.69</td>
</tr>
<tr>
<td>28.75</td>
<td>21.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>3.74</td>
<td>1.27</td>
<td>42.79</td>
<td>20.25</td>
<td>1.25</td>
<td>170.63</td>
<td>3.99</td>
</tr>
</tbody>
</table>
# Project Information

<table>
<thead>
<tr>
<th>VAWS #:</th>
<th>VW14-02388</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name:</td>
<td>CAPE COD RAIL TRAIL EXTENSION - PHASE 1</td>
</tr>
<tr>
<td>Location:</td>
<td>Barnstable County, MA</td>
</tr>
<tr>
<td>Project #:</td>
<td>CM/TAP/TI-002S(758)X</td>
</tr>
<tr>
<td>Design Engineer:</td>
<td>cm / chkd by tpt</td>
</tr>
<tr>
<td>Wall Name:</td>
<td>Verification Calculation</td>
</tr>
</tbody>
</table>

## Legend

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>H max</td>
<td>Height of the wall</td>
<td></td>
</tr>
<tr>
<td>B min</td>
<td>Minimum length of soil reinforcing or defined wall</td>
<td></td>
</tr>
<tr>
<td>S</td>
<td>Maximum height of surcharge</td>
<td></td>
</tr>
<tr>
<td>X</td>
<td>Distance of surcharge</td>
<td></td>
</tr>
<tr>
<td>β</td>
<td>Slope of surcharge</td>
<td></td>
</tr>
<tr>
<td>β l</td>
<td>Adjusted angle</td>
<td></td>
</tr>
<tr>
<td>Le</td>
<td>Length of embedment</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>FS</td>
<td>Factor of safety</td>
<td></td>
</tr>
<tr>
<td>e</td>
<td>Eccentricity</td>
<td></td>
</tr>
<tr>
<td>BP</td>
<td>Bearing pressure</td>
<td></td>
</tr>
<tr>
<td>F*</td>
<td>Pullout friction factor</td>
<td></td>
</tr>
<tr>
<td>S</td>
<td>Sliding</td>
<td></td>
</tr>
<tr>
<td>o</td>
<td>Overturning</td>
<td></td>
</tr>
</tbody>
</table>
### Project Information
- **VAWS #:** VW14-02388
- **Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1
- **Location:** Barnstable County, MA
- **Project #:** CM/TAP/TI-002S(758)X
- **Design Engineer:** cms / chkd by tpt

### Structure Parameters

<table>
<thead>
<tr>
<th>Rec #</th>
<th>H max (ft)</th>
<th>SR Ratio</th>
<th>B min (ft)</th>
<th>Actual B (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>β (deg)</th>
<th>β′ (deg)</th>
<th>Live Load Surcharge (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>30.000</td>
<td>0.70</td>
<td>21.000</td>
<td>22.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.25</td>
</tr>
</tbody>
</table>

### Material Strength Parameters

<table>
<thead>
<tr>
<th></th>
<th>Unit Weight of Reinforced Soil Pullout (kcf)</th>
<th>Unit Weight of Reinforced Soil Rupture (kcf)</th>
<th>Internal Friction Angle of Reinforced Soil (deg)</th>
<th>Unit Weight of Retained Soil (kcf)</th>
<th>Internal Friction Angle of Retained Soil (deg)</th>
<th>Cohesion of Retained Soil (ksf)</th>
<th>Unit Weight of Foundation Soil (kcf)</th>
<th>Internal Friction Angle of Foundation Soil (deg)</th>
<th>Cohesion of Foundation Soil Sliding (ksf)</th>
<th>Cohesion of Foundation Soil Bearing (kcf)</th>
<th>Bearing Capacity Factor Surchage</th>
<th>Bearing Capacity Factor Cohesion</th>
<th>Bearing Capacity Factor Over Burden</th>
<th>Depth to Water Table (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective Stress</td>
<td>0.130</td>
<td>0.130</td>
<td>34.0</td>
<td>0.120</td>
<td>30.0</td>
<td>0.000</td>
<td>0.125</td>
<td>34.0</td>
<td>0.000</td>
<td>29.44</td>
<td>42.16</td>
<td>41.06</td>
<td>0.00</td>
<td></td>
</tr>
</tbody>
</table>

### Effective Stress Evaluation

<table>
<thead>
<tr>
<th>Rec #</th>
<th>Applied Bearing Pressure (ksf)</th>
<th>Ultimate Bearing Pressure (ksf)</th>
<th>Factor of Safety for Bearing Capacity</th>
<th>Factor of Safety for Sliding</th>
<th>Factor of Safety for Overturning</th>
<th>Active Earth Pressure Coefficient</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7.78</td>
<td>19.76</td>
<td>2.54</td>
<td>2.38</td>
<td>3.75</td>
<td>0.333</td>
<td>0.500</td>
</tr>
</tbody>
</table>
MSE-W CALCULATIONS

Level Surcharge with Traffic Live Load
AASHTO 2007-2010 (LRFD)
CAPE COD RAIL TRAIL EXTENSION - PHASE 1

MSEW(3.0): Update # 14.94

PROJECT IDENTIFICATION

Title: CAPE COD RAIL TRAIL EXTENSION - PHASE 1
Project Number: Grid-Strip
Client: Lawrence-Lynch Corp.
Designer: cms
Station Number: 1

Description:
LRFD Verification External Stability with LL=1.75

Company's information:
Name: VAWS
Street: 650 Justice Lane
Mansfield, TX 76063
Telephone #: 817-507-0200
Fax #: 817-507-0197
E-Mail: cstaud@bigbridge.com

Original file path and name: P:\AA_Project File\14 Projects\VW14-02388 Cape Cod Rail....
.....GS External LRFD.BEN

Original date and time of creating this file: 12-24-15

PROGRAM MODE:
ANALYSIS
of a SIMPLE STRUCTURE
using METAL STRIPS as reinforcing material.
SOIL DATA

REINFORCED SOIL
Unit weight, $\gamma$ 130.0 lb/ft$^3$
Design value of internal angle of friction, $\phi$ 34.0°

RETAINED SOIL
Unit weight, $\gamma$ 120.0 lb/ft$^3$
Design value of internal angle of friction, $\phi$ 30.0°

FOUNDATION SOIL (Considered as an equivalent uniform soil)
Equivalent unit weight, $\gamma_{\text{equiv.}}$ 125.0 lb/ft$^3$
Equivalent internal angle of friction, $\phi_{\text{equiv.}}$ 34.0°
Equivalent cohesion, $c_{\text{equiv.}}$ 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) = 0.3333  (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): $N_c = 42.16$  $N_{\gamma} = 41.06$

SEISMICITY
Not Applicable
### INPUT DATA: Metal strips (Analysis)

<table>
<thead>
<tr>
<th>DATA</th>
<th>Metal strip type #1</th>
<th>Metal strip type #2</th>
<th>Metal strip type #3</th>
<th>Metal strip type #4</th>
<th>Metal strip type #5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield strength of steel, $F_y$ [kips/in²]</td>
<td>65.0</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Gross width of strip, $b$ [in]</td>
<td>2.0</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Vertical spacing, $S_v$ [ft]</td>
<td>Varies</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Design cross section area, $A_c$ [in²]</td>
<td>0.16</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Ribbed steel strips.
Uniformity Coefficient of reinforced soil, $Cu = D_{60}/D_{10} = 4.0$

Friction angle along reinforcement-soil interface, $\rho$

- @ the top: 60.97
- @ 19.7 ft or below: 34.00

Pullout resistance factor, $F^*$

- @ the top: 2.50
- @ 19.7 ft or below: 1.25

Scale-effect correction factor, $\alpha$

1.00

### Variation of Lateral Earth Pressure Coefficient With Depth

<table>
<thead>
<tr>
<th>Z</th>
<th>$K / K_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 ft</td>
<td>1.70</td>
</tr>
<tr>
<td>3.3 ft</td>
<td>1.60</td>
</tr>
<tr>
<td>6.6 ft</td>
<td>1.55</td>
</tr>
<tr>
<td>9.8 ft</td>
<td>1.45</td>
</tr>
<tr>
<td>13.1 ft</td>
<td>1.35</td>
</tr>
<tr>
<td>16.4 ft</td>
<td>1.30</td>
</tr>
<tr>
<td>19.7 ft</td>
<td>1.20</td>
</tr>
</tbody>
</table>
**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 $\gamma_f = 152.78 \text{ lb/ft}^3$

<table>
<thead>
<tr>
<th>Z / Hd</th>
<th>To-static / Tmax</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>1.00</td>
</tr>
<tr>
<td>0.25</td>
<td>1.00</td>
</tr>
<tr>
<td>0.50</td>
<td>1.00</td>
</tr>
<tr>
<td>0.75</td>
<td>1.00</td>
</tr>
<tr>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

D A T A (for connection only) | Type #1 | Type #2 | Type #3 | Type #4 | Type #5

<table>
<thead>
<tr>
<th>Product Name</th>
<th>GS11</th>
<th>N/A</th>
<th>N/A</th>
<th>N/A</th>
<th>N/A</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength reduction at the connection, $CR_u = F_{yc} / F_y$</td>
<td>1.00</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>
INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

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

Batter, \(\omega\)  
0.0 [deg]

Backslope, \(\beta\)  
0.0 [deg]

Backslope rise  
0.0 [ft]  

Broken back equivalent angle, \(I = 0.00^\circ\) (see Fig. 25 in DEMO 82)

UNIFORM SURCHARGE

Uniformly distributed dead load is 0.0 [lb/ft²], and live load is 250.0 [lb/ft²]

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: $\gamma_{p-EV}$ 1.35
Load factor for earthquake loads, $EQ$, from Table 3.4.1-1: $\gamma_{p-EQ}$ 1.00
Load factor for live load surcharge, $LS$, from Figure C11.5.5-3(b): $\gamma_{p-LS}$ 1.75
(Same as in External Stability).
Load factor for dead load surcharge, $ES$: $\gamma_{p-ES}$ 1.50
(Same as in External Stability).

Resistance factor for reinforcement tension from Table 11.5.6-1:
- Metal Strips: $\phi$ Static Combined static/seismic
  - $0.75$ 1.00

Resistance factor for reinforcement tension in connectors from Table 11.5.6-1:
- Metal Strips: $\phi$ Static Combined static/seismic
  - $0.75$ 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:
- Sliding and Eccentricity $\gamma_{p-EV}$ 1.00 $\gamma_{p-EQ}$ 1.00
- Bearing Capacity $\gamma_{p-EV}$ 1.35 $\gamma_{p-EQ}$ 1.35

Load factor of active lateral earth pressure, $EH$, from Table 3.4.1-2 and Figure C11.5.5-2: $\gamma_{p-EH}$ 1.50
Load factor of active lateral earth pressure during earthquake (does not multiply $P_{AE}$ and $P_{BR}$): $\gamma_{p-EH, EQ}$ 1.50
Load factor for earthquake loads, $EQ$, from Table 3.4.1-1 (multiplies $P_{AE}$ and $P_{BR}$): $\gamma_{p-EQ}$ 1.00

Resistance factor for shear resistance along common interfaces from Table 11.5.6-1:
- Reinforced Soil and Foundation $\phi_{f}$ 1.00 1.00
- Reinforced Soil and Reinforcement $\phi_{f}$ 1.00 1.00

Resistance factor for bearing capacity of shallow foundation from Table 11.5.6-1:
- $\phi_{b}$ 0.65 0.65
**BEARING CAPACITY for GIVEN LAYOUT**

<table>
<thead>
<tr>
<th>STATIC</th>
<th>SEISMIC</th>
<th>UNITS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factored bearing resistance, ( q_n )</td>
<td>25682</td>
<td>N/A</td>
</tr>
<tr>
<td>Factored bearing load, ( \sigma_V )</td>
<td>7778.8</td>
<td>N/A</td>
</tr>
<tr>
<td>Eccentricity, ( e )</td>
<td>2.80</td>
<td>N/A</td>
</tr>
<tr>
<td>Eccentricity, ( e/L )</td>
<td>0.133</td>
<td>N/A</td>
</tr>
<tr>
<td>CDR calculated</td>
<td>3.30</td>
<td>N/A</td>
</tr>
<tr>
<td>Base length</td>
<td>21.00</td>
<td>N/A</td>
</tr>
</tbody>
</table>

(Water table does not affect bearing capacity)

Unfactored applied bearing pressure = \( \frac{(\text{Unfactored } R)}{[L - 2 \times (\text{Unfactored } e)]} \)

Unfactored \( R = 87149.95 \) [lb/ft], \( L = 21.00 \), Unfactored \( e = 2.50 \) [ft], and \( \text{Sigma} = 5443.95 \) [lb/ft²]
**DIRECT SLIDING for GIVEN LAYOUT**  (for METAL STRIPS reinforcements)

Along reinforced and foundation soils interface:  CDR-static = 1.761

<table>
<thead>
<tr>
<th>#</th>
<th>Metal strip Elevation [ft]</th>
<th>Metal strip Length [ft]</th>
<th>CDR Static</th>
<th>CDR Seismic</th>
<th>Metal strip Type #</th>
<th>Product name</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.25</td>
<td>21.00</td>
<td>1.826</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>2</td>
<td>3.75</td>
<td>21.00</td>
<td>1.973</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>3</td>
<td>6.25</td>
<td>21.00</td>
<td>2.145</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>4</td>
<td>8.75</td>
<td>21.00</td>
<td>2.351</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>5</td>
<td>11.25</td>
<td>21.00</td>
<td>2.600</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>6</td>
<td>13.75</td>
<td>21.00</td>
<td>2.907</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>7</td>
<td>16.25</td>
<td>21.00</td>
<td>3.298</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>8</td>
<td>18.75</td>
<td>21.00</td>
<td>3.810</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>9</td>
<td>21.25</td>
<td>21.00</td>
<td>4.510</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>10</td>
<td>23.75</td>
<td>21.00</td>
<td>5.524</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>11</td>
<td>26.25</td>
<td>21.00</td>
<td>7.128</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>12</td>
<td>28.75</td>
<td>21.00</td>
<td>10.044</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
</tbody>
</table>

**ECCENTRICITY for GIVEN LAYOUT**  (for Simplified Method)

At interface with foundation:  e/L static = 0.1951; Overturning: CDR-static = 2.56

<table>
<thead>
<tr>
<th>#</th>
<th>Metal strip Elevation [ft]</th>
<th>Metal strip Length [ft]</th>
<th>e / L Static</th>
<th>e / L Seismic</th>
<th>Metal strip Type #</th>
<th>Product name</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.25</td>
<td>21.00</td>
<td>0.1807</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>2</td>
<td>3.75</td>
<td>21.00</td>
<td>0.1536</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>3</td>
<td>6.25</td>
<td>21.00</td>
<td>0.1286</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>4</td>
<td>8.75</td>
<td>21.00</td>
<td>0.1058</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>5</td>
<td>11.25</td>
<td>21.00</td>
<td>0.0852</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>6</td>
<td>13.75</td>
<td>21.00</td>
<td>0.0667</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>7</td>
<td>16.25</td>
<td>21.00</td>
<td>0.0505</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>8</td>
<td>18.75</td>
<td>21.00</td>
<td>0.0364</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>9</td>
<td>21.25</td>
<td>21.00</td>
<td>0.0245</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>10</td>
<td>23.75</td>
<td>21.00</td>
<td>0.0148</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>11</td>
<td>26.25</td>
<td>21.00</td>
<td>0.0072</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>12</td>
<td>28.75</td>
<td>21.00</td>
<td>0.0019</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
</tbody>
</table>
AASHTO 2007-2010 (LRFD)  
CAPE COD RAIL TRAIL EXTENSION - PHASE 1 
MSEW(3.0): Update # 14.94

PROJECT IDENTIFICATION

Title: CAPE COD RAIL TRAIL EXTENSION - PHASE 1 
Project Number: Grid-Strip 
Client: Lawrence-Lynch Corp.
Designer: cms
Station Number: 1

Description: LRFD Verification Internal Stability with LL=1.35

Company's information:

Name: VAWS 
Street: 650 Justice Lane 
Mansfield, TX 76063
Telephone #: 817-507-0200
Fax #: 817-507-0197
E-Mail: cstaud@bigrbridge.com

Original file path and name: P:\AA_Project File\14 Projects\VW14-02388 Cape Cod Rail..... 
.....GS Internal LRFD.BEN
Original date and time of creating this file: 12-21-15

PROGRAM MODE: ANALYSIS 
of a SIMPLE STRUCTURE 
using METAL STRIPS as reinforcing material.
SOIL DATA

REINFORCED SOIL
Unit weight, $\gamma$ 130.0 lb/ft$^3$
Design value of internal angle of friction, $\phi$ 34.0°

RETIRED SOIL
Unit weight, $\gamma$ 120.0 lb/ft$^3$
Design value of internal angle of friction, $\phi$ 30.0°

FOUNDATION SOIL (Considered as an equivalent uniform soil)
Equivalent unit weight, $\gamma_{equiv}$ 120.0 lb/ft$^3$
Equivalent internal angle of friction, $\phi_{equiv}$ 34.0°
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) = 0.3333 (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): $N_c = 42.16$  \hspace{1cm} $N_\gamma = 41.06$

SEISMICITY
Not Applicable
**INPUT DATA: Metal strips**  
(Analysis)

<table>
<thead>
<tr>
<th>DATA</th>
<th>Metal strip type #1</th>
<th>Metal strip type #2</th>
<th>Metal strip type #3</th>
<th>Metal strip type #4</th>
<th>Metal strip type #5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield strength of steel, $F_y$ [kips/in²]</td>
<td>65.0</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Gross width of strip, $b$ [in]</td>
<td>2.0</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Vertical spacing, $S_v$ [ft]</td>
<td>Varies</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Design cross section area, $A_c$ [in²]</td>
<td>0.16</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Ribbed steel strips.  
Uniformity Coefficient of reinforced soil, $C_u = D_{60}/D_{10} = 4.0$

Friction angle along reinforcement-soil interface, $\rho$  
- @ the top: 60.97
- @ 19.7 ft or below: 34.00

Pullout resistance factor, $F^*$  
- @ the top: 2.50
- @ 19.7 ft or below: 1.25

Scale-effect correction factor, $\alpha$  
- 1.00

**Variation of Lateral Earth Pressure Coefficient With Depth**

<table>
<thead>
<tr>
<th>$Z$ [ft]</th>
<th>$K / K_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.70</td>
</tr>
<tr>
<td>3.3</td>
<td>1.60</td>
</tr>
<tr>
<td>6.6</td>
<td>1.55</td>
</tr>
<tr>
<td>9.8</td>
<td>1.45</td>
</tr>
<tr>
<td>13.1</td>
<td>1.35</td>
</tr>
<tr>
<td>16.4</td>
<td>1.30</td>
</tr>
<tr>
<td>19.7</td>
<td>1.20</td>
</tr>
</tbody>
</table>

Graph showing variation of $K / K_a$ with depth $Z$. The graph ranges from 0 to 19.7 ft on the $Z$ axis and from 1.0 to 3.0 on the $K / K_a$ axis.
INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

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

Batter, \( \omega \) 0.0 [deg]
Backslope, \( \beta \) 0.0 [deg]
Backslope rise 0.0 [ft]  
Broken back equivalent angle, \( I = 0.00^\circ \) (see Fig. 25 in DEMO 82)

UNIFORM SURCHARGE

Uniformly distributed dead load is 0.0 [lb/ft²], and live load is 250.0 [lb/ft²]

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: \( \gamma_{p-EV} \) = 1.35
Load factor for earthquake loads, EQ, from Table 3.4.1-1: \( \gamma_{p-EQ} \) = 1.00
Load factor for live load surcharge, LS, from Figure C11.5.5-3(b): \( \gamma_{p-LS} \) = 1.35
(Same as in External Stability).
Load factor for dead load surcharge, ES:
(Same as in External Stability).

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:
Sliding and Eccentricity \( \gamma_{p-EV} \) = 1.00 \( \gamma_{p-EQ} \) = 1.00
Bearing Capacity \( \gamma_{p-EV} \) = 1.35 \( \gamma_{p-EQ} \) = 1.35

Load factor of active lateral earth pressure, EH, from Table 3.4.1-2 and Figure C11.5.5-2:
\( \gamma_{p-EH} \) = 1.50

Load factor of active lateral earth pressure during earthquake (does not multiply \( P_{AE} \) and \( P_{ER} \) ):
\( \gamma_{p-EH_{EQ}} \) = 1.50

Load factor for earthquake loads, EQ, from Table 3.4.1-1 (multiplies \( P_{AE} \) and \( P_{ER} \) ):
\( \gamma_{p-EQ} \) = 1.00

Resistance factor for shear resistance along common interfaces from Table 11.5.6-1:
Reinforced Soil and Foundation \( \phi_{\tau} \) = 1.00
Reinforced Soil and Reinforcement \( \phi_{\tau} \) = 1.00

Resistance factor for bearing capacity of shallow foundation from Table 11.5.6-1:
\( \phi_{b} \) = 0.65
### RESULTS for STRENGTH

[Note: Actual CDR = (Yield stress) / (Actual stress)]

**For Simplified Method**

**Live Load included in calculating Tmax**

<table>
<thead>
<tr>
<th>#</th>
<th>Metal strip Elevation [ft]</th>
<th>Coverage ratio, Rc=b/Sh</th>
<th>Horizontal spacing, Sh [ft]</th>
<th>Long-term strength Fy/Ac·Rc/b [lb/ft]</th>
<th>Tmax [lb/ft]</th>
<th>Tmd [lb/ft]</th>
<th>Specified minimum CDR static</th>
<th>Actual calculated CDR static</th>
<th>Specified minimum CDR seismic</th>
<th>Actual calculated CDR seismic</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.25</td>
<td>0.100</td>
<td>1.670</td>
<td>4671</td>
<td>4565.67</td>
<td>N/A</td>
<td>N/A</td>
<td>1.023</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>2</td>
<td>3.75</td>
<td>0.100</td>
<td>1.670</td>
<td>4671</td>
<td>4193.54</td>
<td>N/A</td>
<td>N/A</td>
<td>1.114</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>3</td>
<td>6.25</td>
<td>0.083</td>
<td>2.000</td>
<td>3900</td>
<td>3821.42</td>
<td>N/A</td>
<td>N/A</td>
<td>1.021</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>4</td>
<td>8.75</td>
<td>0.083</td>
<td>2.000</td>
<td>3900</td>
<td>3449.30</td>
<td>N/A</td>
<td>N/A</td>
<td>1.131</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>5</td>
<td>11.25</td>
<td>0.067</td>
<td>2.500</td>
<td>3120</td>
<td>3157.40</td>
<td>N/A</td>
<td>N/A</td>
<td>0.988</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>6</td>
<td>13.75</td>
<td>0.067</td>
<td>2.500</td>
<td>3120</td>
<td>2912.70</td>
<td>N/A</td>
<td>N/A</td>
<td>1.071</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>7</td>
<td>16.25</td>
<td>0.067</td>
<td>2.500</td>
<td>3120</td>
<td>2611.52</td>
<td>N/A</td>
<td>N/A</td>
<td>1.195</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>8</td>
<td>18.75</td>
<td>0.050</td>
<td>3.330</td>
<td>2342</td>
<td>2293.30</td>
<td>N/A</td>
<td>N/A</td>
<td>1.021</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>9</td>
<td>21.25</td>
<td>0.050</td>
<td>3.330</td>
<td>2342</td>
<td>1957.83</td>
<td>N/A</td>
<td>N/A</td>
<td>1.196</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>10</td>
<td>23.75</td>
<td>0.050</td>
<td>3.330</td>
<td>2342</td>
<td>1564.89</td>
<td>N/A</td>
<td>N/A</td>
<td>1.497</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>11</td>
<td>26.25</td>
<td>0.050</td>
<td>3.330</td>
<td>2342</td>
<td>1121.19</td>
<td>N/A</td>
<td>N/A</td>
<td>2.089</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>12</td>
<td>28.75</td>
<td>0.050</td>
<td>3.330</td>
<td>2342</td>
<td>648.20</td>
<td>N/A</td>
<td>N/A</td>
<td>3.614</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

### 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.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.25</td>
<td>0.100</td>
<td>4279.4</td>
<td>N/A</td>
<td>20.25</td>
<td>0.75</td>
<td>16995.0</td>
<td>N/A</td>
<td>3.971</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>2</td>
<td>3.75</td>
<td>0.100</td>
<td>3907.3</td>
<td>N/A</td>
<td>18.75</td>
<td>2.25</td>
<td>14367.8</td>
<td>N/A</td>
<td>3.677</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>3</td>
<td>6.25</td>
<td>0.083</td>
<td>3535.2</td>
<td>N/A</td>
<td>17.25</td>
<td>3.75</td>
<td>9986.1</td>
<td>N/A</td>
<td>2.825</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>4</td>
<td>8.75</td>
<td>0.083</td>
<td>3163.0</td>
<td>N/A</td>
<td>15.75</td>
<td>5.25</td>
<td>8158.0</td>
<td>N/A</td>
<td>2.579</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>5</td>
<td>11.25</td>
<td>0.067</td>
<td>2863.2</td>
<td>N/A</td>
<td>14.25</td>
<td>6.75</td>
<td>5457.6</td>
<td>N/A</td>
<td>1.906</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>6</td>
<td>13.75</td>
<td>0.067</td>
<td>2604.0</td>
<td>N/A</td>
<td>12.75</td>
<td>8.25</td>
<td>4741.7</td>
<td>N/A</td>
<td>1.821</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>7</td>
<td>16.25</td>
<td>0.067</td>
<td>2290.6</td>
<td>N/A</td>
<td>12.00</td>
<td>9.00</td>
<td>4184.3</td>
<td>N/A</td>
<td>1.827</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>8</td>
<td>18.75</td>
<td>0.050</td>
<td>1957.6</td>
<td>N/A</td>
<td>12.00</td>
<td>9.00</td>
<td>2821.0</td>
<td>N/A</td>
<td>1.441</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>9</td>
<td>21.25</td>
<td>0.050</td>
<td>1604.0</td>
<td>N/A</td>
<td>12.00</td>
<td>9.00</td>
<td>2389.2</td>
<td>N/A</td>
<td>1.490</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>10</td>
<td>23.75</td>
<td>0.050</td>
<td>1195.7</td>
<td>N/A</td>
<td>12.00</td>
<td>9.00</td>
<td>1845.9</td>
<td>N/A</td>
<td>1.544</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>11</td>
<td>26.25</td>
<td>0.050</td>
<td>739.8</td>
<td>N/A</td>
<td>12.00</td>
<td>9.00</td>
<td>1191.1</td>
<td>N/A</td>
<td>1.610</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>12</td>
<td>28.75</td>
<td>0.050</td>
<td>251.8</td>
<td>N/A</td>
<td>12.00</td>
<td>9.00</td>
<td>424.9</td>
<td>N/A</td>
<td>1.688</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>
DURABILITY CALCULATION

Grid-Strip Soil Reinforcing
DURABILITY DESIGN ANALYSIS

Project Name: VAWS Mechanically Stabilized Earth Product Submittal - SCP
Project Location: LRFD Binder
Project Number:
Design Code: AASHTO Design Method: LRFD

Degradation Design Rates

AASHTO requires that all components buried in soil be designed assuming a degrade area. The degradation rates are specified in AASHTO in section 11.10.6.4.2a.

Soil Reinforcing Material Parameters

Yield strength of steel................................................................. $f_y := 65.00 \text{ ksi}$
Yield coefficient........................................................................... $\theta_y := 0.75$
Bar diameter ................................................................................ $d_b := 0.374 \text{ in}$

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

Thickness of galvanized coating ...................................................... $g_T := 3.4 \text{ mil}$
Structure design life ........................................................................... $Y_t := 75 \text{ yr}$
Zinc loss rate (first two years) ....................................................... $g_2 := 0.58 \frac{\text{mil}}{\text{yr}}$
Zinc loss rate (remaining years) ...................................................... $g_R := 0.16 \frac{\text{mil}}{\text{yr}}$
Steel loss rate (0-75 years) ......................................................... $E_{S75} := 0.47 \frac{\text{mil}}{\text{yr}}$
Design Life of Galvanized Coating

\[ Y_G := \frac{g_T - 2 \cdot \text{yr} \cdot g_2}{g_R} + 2 \cdot \text{yr} = 16.000 \cdot \text{yr} \]

Sacrificial Steel (To be taken from diameter of bar)

\[ E_c := \text{if} \left[ Y_t \leq Y_G, 0, \left( Y_t - Y_G \right) \cdot E_{S75} \right] \cdot 2 = 0.0555 \cdot \text{in} \]

**GS11 Grid-Strip Soil Reinforcing (W11.0 - 2 Bars)**

Area of Grid-Strip tension steel

\[ A_b := \frac{\pi \cdot db^2}{4} = 0.110 \cdot \text{in}^2 \]

Diameter of W11.0 with effects of corrosion

\[ d_{11} := \left( \sqrt{\frac{A_b}{\pi} \cdot 4 - E_c} \right) = 0.319 \cdot \text{in} \]

Total area of 2-Bar W11.0

\[ A_{GS11} := \frac{\pi \cdot d_{11}^2}{4} \cdot 2 = 0.159 \cdot \text{in}^2 \]

Maximum horizontal force of 2-Bar W11.0

\[ T_{\text{max}_{GS11}} := \gamma \cdot f_y \cdot A_{GS11} = 7.770 \cdot \text{kip} \]
PROJECT SPECIFIC
DESIGN PARAMETERS

LEVEL SURCHARGE / STANDARD TRAFFIC
LOADING / STANDARD BACKFILL

SEGMENTAL CONCRETE PANEL MSE RETAINING WALL

CAPE COD RAIL TRAIL EXTENSION – PHASE 1

FEDERAL AID PROJECT NO. CM/TAP/TI-002S(758)X

PROJECT FILE NO. 604488

BARNSTABLE COUNTY, MA

PREPARED FOR:

LAWRENCE-LYNCH CORP.

BIG R BRIDGE PROJECT NO. VW14-02388
MECHANICALLY STABILIZED EARTH STRUCTURES

MSE DESIGN CALCULATIONS

CAPE COD RAIL TRAIL EXTENSION – PHASE 1

Towns of Dennis and Yarmouth
Plymouth County, MA

Federal Aid Project No. CM/TAP/TI-002S(758)X
Project File No. 604488

Big R Project No. VW14-02388

Prepared For

LAWRENCE-LYNCH CORP.

PROPRIETARY INFORMATION OF VISTA WALL SYSTEMS LLC
NO DISTRIBUTION IS ALLOWED WITHOUT WRITTEN PERMISSION FROM
VISTA WALL SYSTEMS LLC

650 Justice Lane
Mansfield, TX 76063
817-507-0454
## DESIGN PARAMETERS

<table>
<thead>
<tr>
<th>Description</th>
<th>Limit State</th>
<th>Value</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Design Life</td>
<td>All limit states</td>
<td>75 Years</td>
<td>6</td>
</tr>
<tr>
<td>2. Length of Soil Reinforcing (B or L)</td>
<td>All Limit States</td>
<td>0.7H or 8 min</td>
<td>1</td>
</tr>
<tr>
<td>3. Limiting eccentricity</td>
<td>Strength (all)</td>
<td>B/4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Service I</td>
<td>B/6</td>
<td></td>
</tr>
<tr>
<td>4. Coefficient of Sliding</td>
<td>Strength (all)</td>
<td>$\phi_i = 30^\circ$</td>
<td>2</td>
</tr>
<tr>
<td>5. Resistance Factors</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. Sliding</td>
<td>Strength (all)</td>
<td>0.8</td>
<td>3</td>
</tr>
<tr>
<td>b. Bearing</td>
<td>Strength (all)</td>
<td>0.65</td>
<td>4</td>
</tr>
<tr>
<td>c. Overall</td>
<td>Strength (all)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>i. Deep Seated Stability</td>
<td>Service I</td>
<td>0.65</td>
<td></td>
</tr>
<tr>
<td>ii. Compound Stability</td>
<td>Service I</td>
<td>0.65</td>
<td></td>
</tr>
<tr>
<td>d. Pullout</td>
<td>Strength/Extreme</td>
<td>0.90/1.20</td>
<td>5, 6, 7, 8</td>
</tr>
<tr>
<td>e. Tensile</td>
<td>Strength/Extreme</td>
<td>0.75/1.00</td>
<td>5, 6, 7, 8</td>
</tr>
</tbody>
</table>

### Notes

1. H is the design height of the wall and is defined as the difference in elevation between the finished grade at top of wall and the top of leveling pad. The top of the leveling pad shall always be below the minimum embedment reference line as indicated on the plans for that location. The length of the soil reinforcement, B, is measured from the back face of the facing unit.

2. $\phi_i = \text{friction angle of foundation soil}$
   - All friction angles are effective (drained) friction angles.

3. Passive resistance shall not be considered in evaluation of sliding resistance.

4. For all limit states, the design loading for the MSE retaining wall system shall not exceed the factored general and local bearing resistances specified in the Geotechnical Report(s).

5. Live load due to vehicular traffic shall be included in the computations to determine the maximum tensile load in reinforcement for rupture, but shall be neglected in computations for maximum tensile load for pullout resistance. Intensity of live load shall be considered as a uniform surcharge using the equivalent height of soil in accordance with Section Article 3.11.6.4 of AASHTO (2010).

6. Apply to gross cross-section less sacrificial area. For sections with holes, reduce gross area in accordance with Article 6.8.3 of AASHTO (2010) and apply to net section less sacrificial area.

7. Applies to reinforcements connected with a single point connection. Unless otherwise specified, all resistance factors shall be taken as 1.0 when investigating an extreme event limit state.

8. Value is static/seismic
### SOIL BACKFILL

Section 4.2.1 Geotechnical Report

<table>
<thead>
<tr>
<th>Location</th>
<th>Unit Weight</th>
<th>Friction Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced Soil</td>
<td>135 pcf</td>
<td>34 degrees</td>
</tr>
<tr>
<td>Retained Backfill</td>
<td>135 pcf</td>
<td>34 degrees</td>
</tr>
<tr>
<td>Foundation</td>
<td>120 pcf</td>
<td>30 degrees</td>
</tr>
</tbody>
</table>

### ELECTROCHEMICAL REQUIREMENTS FOR METALLIC REINFORCEMENTS

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Requirement</th>
<th>Test Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>pH</td>
<td>5.0 to 10.0</td>
<td>AASHTO T-289</td>
</tr>
<tr>
<td>Resistivity, min.*</td>
<td>2,500 ohm-cm</td>
<td>AASHTO T-288</td>
</tr>
<tr>
<td>Chlorides, max.</td>
<td>100 ppm</td>
<td>AASHTO T-291</td>
</tr>
<tr>
<td>Sulfates, max.</td>
<td>200 ppm</td>
<td>AASHTO T-290</td>
</tr>
</tbody>
</table>

* Backfill material will be acceptable when the moving average of the last three tests for resistivity is at least 2,500 ohm-cm, and no single test is less than 2,400 ohm-cm. For resistivity values greater than 5,000 ohm-cm, the sulfate and chlorides tests are not required. Resistivity is to be reported at saturation levels and not super-saturation levels.
AASHTO Load Combinations and Load Factors for MSE Structures

<table>
<thead>
<tr>
<th>Load Combination Limit State</th>
<th>EH</th>
<th>ES</th>
<th>EV</th>
<th>LL</th>
<th>LS</th>
<th>Use One of These at a Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>STRENGTH I</td>
<td>$\gamma_p$</td>
<td>1.75</td>
<td></td>
<td></td>
<td></td>
<td>EQ</td>
</tr>
<tr>
<td>EXTREME EVENT I</td>
<td>$\gamma_p$</td>
<td>$\gamma_{EQ}$</td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SERVICE I</td>
<td>1.00</td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:

$\gamma_p$ = load factor for permanent loading. May subscript as $\gamma_{EV}$, $\gamma_{EH}$, etc.

$\gamma_{EQ}$ = load factor for live load applied simultaneously with seismic loads

AASHTO Permanent Load Factors for MSE Structures

<table>
<thead>
<tr>
<th>Type of Load</th>
<th>Load Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum</td>
</tr>
<tr>
<td>DC: Component and Attachments</td>
<td>1.25</td>
</tr>
<tr>
<td>EH: Horizontal Earth Pressure</td>
<td></td>
</tr>
<tr>
<td>• Active</td>
<td>1.50</td>
</tr>
<tr>
<td>EV: Vertical Earth Pressure</td>
<td></td>
</tr>
<tr>
<td>• Overall Stability</td>
<td>1.00</td>
</tr>
<tr>
<td>• Retaining Walls and Abutments</td>
<td>1.35</td>
</tr>
<tr>
<td>ES: Earth Surcharge</td>
<td>1.50</td>
</tr>
</tbody>
</table>

Notes

May subscript as $\gamma_{EV-MIN}$, $\gamma_{EV-MAX}$, $\gamma_{EH-MIN}$, $\gamma_{EH-MAX}$ etc.
Resistance Factors for Permanent MSE Retaining Walls

<table>
<thead>
<tr>
<th>AASHTO (2007) Table 11.5.6-1</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bearing resistance</strong></td>
</tr>
<tr>
<td><strong>Sliding</strong></td>
</tr>
<tr>
<td><strong>Tensile resistance of metallic reinforcement and connectors</strong></td>
</tr>
<tr>
<td>Strip reinforcements</td>
</tr>
<tr>
<td>- Static loading</td>
</tr>
<tr>
<td>- Combined static/earthquake loading</td>
</tr>
<tr>
<td>Grid reinforcements</td>
</tr>
<tr>
<td>- Static loading</td>
</tr>
<tr>
<td>- Combined static/earthquake loading</td>
</tr>
<tr>
<td><strong>Pullout resistance of tensile reinforcement</strong></td>
</tr>
<tr>
<td>- Static loading</td>
</tr>
<tr>
<td>- Combined static/earthquake loading</td>
</tr>
</tbody>
</table>
PROJECT SPECIFIC CALCULATIONS

LEVEL SURCHARGE / STANDARD TRAFFIC LOADING / STANDARD BACKFILL

SEGMENTAL CONCRETE PANEL MSE RETAINING WALL

CAPE COD RAIL TRAIL EXTENSION – PHASE 1

FEDERAL AID PROJECT NO. CM/TAP/TI-002S(758)X

PROJECT FILE NO. 604488

BARNSTABLE COUNTY, MA

PREPARED FOR:

LAWRENCE-LYNCH CORP.
SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

Project Information
VAWS #: VW14-02388
Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1
Location: Barnstable County, MA
Proce #: CM/TAP/TI-002S(758)X
Design Engineer: cm
Wall Name: B=0.7H Minimum

Soil Reinforcing Schedule - Grid Strip

<table>
<thead>
<tr>
<th>Mat-Type</th>
<th>Long Size (W)</th>
<th>Tran Size (W)</th>
<th>Long-Space (ft)</th>
<th>Tran-Space (ft)</th>
<th>Mat Width (ft)</th>
<th>Number Of Mats/Row</th>
<th>Max Stiff.</th>
<th>Ac (in²)</th>
<th>F* - 0</th>
<th>F* - 20</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11.0</td>
<td>11.0</td>
<td>0.167</td>
<td>1.000</td>
<td>0.167</td>
<td>3</td>
<td>6160.000</td>
<td>0.479</td>
<td>2.500</td>
<td>1.250</td>
</tr>
</tbody>
</table>

Soil Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>MSE Backfill</td>
<td>0.135 (kcf)</td>
</tr>
<tr>
<td>Retained Backfill</td>
<td>0.135 (kcf)</td>
</tr>
<tr>
<td>Foundation Backfill</td>
<td>0.120 (kcf)</td>
</tr>
<tr>
<td>Depth to Water Table</td>
<td>0.00 (ft)</td>
</tr>
</tbody>
</table>

Live Load Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Weight of Soil</td>
<td>0.125 (kcf)</td>
</tr>
<tr>
<td>Friction Angle</td>
<td>34 (deg)</td>
</tr>
</tbody>
</table>

Live Load Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equivalent Height of Soil</td>
<td>2.000 (ft)</td>
</tr>
<tr>
<td>Equivalent Unit Weight of Soil</td>
<td>0.125 (kcf)</td>
</tr>
</tbody>
</table>

Internal Stability Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Yield Stress</td>
<td>65.000 (ksi)</td>
</tr>
<tr>
<td>Yield Coefficient</td>
<td>0.750</td>
</tr>
<tr>
<td>Design Life</td>
<td>75.0 years</td>
</tr>
</tbody>
</table>

Design Options

<table>
<thead>
<tr>
<th>Option</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>LRFD Procedure</td>
<td>Stiffness Method</td>
</tr>
<tr>
<td></td>
<td>Live load is applied to Tmax in pullout calculation - AASHTO Figure 11.10.6.2.1-1</td>
</tr>
<tr>
<td></td>
<td>The Vertical Earth (EV) load is not used for all internal loads - AASHTO 11.10.6.2.1-1</td>
</tr>
<tr>
<td></td>
<td>The K-Ratio for pullout is set to 1.7 to 1.2 - AASHTO 11.10.6.2.1</td>
</tr>
<tr>
<td></td>
<td>The K-Ratios are calculated from the top of the structure or top of coping</td>
</tr>
</tbody>
</table>

General Notes

<table>
<thead>
<tr>
<th>Note</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Proce: Specialized Design</td>
<td>AASHTO LRFD Bridge Design Specialized Design 2012, Sixth Edition</td>
</tr>
<tr>
<td></td>
<td>FHWA Simplified</td>
</tr>
<tr>
<td></td>
<td>Modified Pullout: 80° o 40 (Pullout Test)</td>
</tr>
<tr>
<td></td>
<td>Tension = 0.75</td>
</tr>
<tr>
<td></td>
<td>Standard Traffic Loading</td>
</tr>
</tbody>
</table>
## Project Information

<table>
<thead>
<tr>
<th>VAWS #:</th>
<th>VW14-02388</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name:</td>
<td>CAPE COD RAIL TRAIL EXTENSION - PHASE 1</td>
</tr>
<tr>
<td>Location:</td>
<td>Barnstable County, MA</td>
</tr>
<tr>
<td>Project #:</td>
<td>CM/TAP/TI-002S(758)X</td>
</tr>
<tr>
<td>Design Engineer:</td>
<td>cmś</td>
</tr>
<tr>
<td>Wall Name:</td>
<td>B=0.7H Minimum</td>
</tr>
</tbody>
</table>

### Load and Resistance Factor Design Input Data

<table>
<thead>
<tr>
<th>Load Factor</th>
<th>Maximum</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load factor for vertical earth pressure (EV)</td>
<td>1.35</td>
<td>1.00</td>
</tr>
<tr>
<td>Load factor for earth surcharge (ES)</td>
<td>1.50</td>
<td>0.75</td>
</tr>
<tr>
<td>Load factor for traffic live load (LS)</td>
<td>1.75</td>
<td>1.75</td>
</tr>
<tr>
<td>Load factor for horizontal earth pressure (EH)</td>
<td>1.50</td>
<td>0.90</td>
</tr>
<tr>
<td>Load factor for seismic (EQ)</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Load factor for structural components (DC)</td>
<td>1.25</td>
<td>0.90</td>
</tr>
<tr>
<td>Load factor for water (WA)</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Load factor for live load extreme event (LL)</td>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>Load factor for vehicular impact (CT)</td>
<td>1.50</td>
<td>1.00</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Resistance Factor</th>
<th>Static</th>
<th>Seismic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing resistance acor</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Sliding resistance acor</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Tensile resistance acor</td>
<td>0.75</td>
<td>0.85</td>
</tr>
<tr>
<td>Pullout resistance acor</td>
<td>0.90</td>
<td>1.20</td>
</tr>
<tr>
<td>Extreme event resistance acor</td>
<td>1.00</td>
<td></td>
</tr>
</tbody>
</table>

### Variation of Lateral Earth Pressure Coefficient with Depth (K/Ka)

<table>
<thead>
<tr>
<th>Depth (Z)</th>
<th>Rupture</th>
<th>Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00 (ft)</td>
<td>1.70</td>
<td>1.70</td>
</tr>
<tr>
<td>20.00 (ft)</td>
<td>1.20</td>
<td>1.20</td>
</tr>
</tbody>
</table>
**MSE - Pro**  
Mechanically Stabilized Earth Retaining Structures

**SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP**

### Project Information
- **VAWS #:** VW14-02388  
- **Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1  
- **Location:** Barnstable County, MA  
- **Project #:** CM/TAP/TI-002S(758)X  
- **Design Engineer:** cms  
- **Wall Name:** B=0.7H Minimum

### External And Internal Stability Calculation Summary

#### Rec. #1

<table>
<thead>
<tr>
<th>Structure Parameters</th>
<th>H max (ft)</th>
<th>B min (ft)</th>
<th>B Ext. (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>$\beta$ (deg)</th>
<th>$\beta_t$ (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>31.429</td>
<td>22.000</td>
<td>22.000</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

### External Active Earth Pressure Coefficient: 0.283
#### External Stability CDR Summary (Effective Stress)

<table>
<thead>
<tr>
<th>CDR Sliding</th>
<th>Limiting Eccentricity (ft)</th>
<th>CDR Overturning Bearing Pressure (ksf)</th>
<th>Eccentricity (ft)</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>S re: g 1 - Mi</td>
<td>2.58</td>
<td>2.56</td>
<td>4.30</td>
<td>5.93</td>
</tr>
<tr>
<td>S re: g 1 - Max</td>
<td>2.26</td>
<td>2.84</td>
<td>3.88</td>
<td>8.11</td>
</tr>
<tr>
<td>S re: g 1 - Critical</td>
<td>1.68</td>
<td>3.83</td>
<td>2.87</td>
<td>6.84</td>
</tr>
<tr>
<td>Service</td>
<td>2.56</td>
<td>2.49</td>
<td>4.42</td>
<td>5.71</td>
</tr>
</tbody>
</table>

### Internal Active Earth Pressure Coefficient: 0.283
#### Internal Stability CDR Summary

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Internal Earth Coeff.</th>
<th>Rupture CDR</th>
<th>Pullout CDR</th>
<th>F*</th>
<th>Le (ft)</th>
<th>H/B Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.68</td>
<td>22.00</td>
<td>3</td>
<td>0.46</td>
<td>1.56</td>
<td>1.08</td>
<td>74.79</td>
<td>12.57</td>
<td>0.70</td>
</tr>
<tr>
<td>5.18</td>
<td>22.00</td>
<td>3</td>
<td>0.44</td>
<td>1.64</td>
<td>1.65</td>
<td>69.78</td>
<td>12.57</td>
<td>0.70</td>
</tr>
<tr>
<td>7.68</td>
<td>22.00</td>
<td>3</td>
<td>0.43</td>
<td>1.26</td>
<td>1.59</td>
<td>64.77</td>
<td>12.57</td>
<td>0.70</td>
</tr>
<tr>
<td>10.18</td>
<td>22.00</td>
<td>3</td>
<td>0.41</td>
<td>1.04</td>
<td>1.53</td>
<td>59.76</td>
<td>12.57</td>
<td>0.70</td>
</tr>
<tr>
<td>12.68</td>
<td>22.00</td>
<td>4</td>
<td>0.39</td>
<td>1.20</td>
<td>1.96</td>
<td>54.75</td>
<td>12.57</td>
<td>0.70</td>
</tr>
<tr>
<td>15.18</td>
<td>22.00</td>
<td>4</td>
<td>0.37</td>
<td>1.07</td>
<td>1.86</td>
<td>49.74</td>
<td>12.57</td>
<td>0.70</td>
</tr>
<tr>
<td>17.68</td>
<td>22.00</td>
<td>5</td>
<td>0.36</td>
<td>1.23</td>
<td>2.40</td>
<td>44.73</td>
<td>13.75</td>
<td>0.70</td>
</tr>
<tr>
<td>20.18</td>
<td>22.00</td>
<td>5</td>
<td>0.34</td>
<td>1.14</td>
<td>2.50</td>
<td>40.08</td>
<td>15.25</td>
<td>0.70</td>
</tr>
<tr>
<td>22.68</td>
<td>22.00</td>
<td>5</td>
<td>0.34</td>
<td>1.03</td>
<td>2.75</td>
<td>40.08</td>
<td>16.75</td>
<td>0.70</td>
</tr>
<tr>
<td>25.18</td>
<td>22.00</td>
<td>6</td>
<td>0.34</td>
<td>1.12</td>
<td>3.59</td>
<td>40.08</td>
<td>18.25</td>
<td>0.70</td>
</tr>
<tr>
<td>27.68</td>
<td>22.00</td>
<td>6</td>
<td>0.34</td>
<td>1.02</td>
<td>3.89</td>
<td>40.08</td>
<td>19.75</td>
<td>0.70</td>
</tr>
<tr>
<td>30.18</td>
<td>22.00</td>
<td>7</td>
<td>0.34</td>
<td>1.10</td>
<td>4.88</td>
<td>40.08</td>
<td>21.25</td>
<td>0.70</td>
</tr>
</tbody>
</table>
**Project Information**

VAWS #: VW14-02388  
Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1  
Location: Barnstable County, MA  
Project #: CM/TAP/TI-002S(758)X  
Design Engineer: cms  
Wall Name: B=0.7H Minimum

### Static Internal Stability - Rupture Force Results
Rec. #1 - H = 31.43 (ft) B = 22.00 (ft)

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Static CDR Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.68</td>
<td>22.00</td>
<td>3</td>
<td>39.29</td>
<td>0.46</td>
<td>0.83</td>
<td>0.38</td>
<td>14.98</td>
<td>1.56</td>
</tr>
<tr>
<td>5.18</td>
<td>22.00</td>
<td>3</td>
<td>25.00</td>
<td>0.44</td>
<td>1.28</td>
<td>0.57</td>
<td>14.22</td>
<td>1.64</td>
</tr>
<tr>
<td>7.68</td>
<td>22.00</td>
<td>3</td>
<td>25.00</td>
<td>0.43</td>
<td>1.74</td>
<td>0.74</td>
<td>18.51</td>
<td>1.26</td>
</tr>
<tr>
<td>10.18</td>
<td>22.00</td>
<td>3</td>
<td>25.00</td>
<td>0.41</td>
<td>2.19</td>
<td>0.90</td>
<td>22.40</td>
<td>1.04</td>
</tr>
<tr>
<td>12.68</td>
<td>22.00</td>
<td>4</td>
<td>25.00</td>
<td>0.39</td>
<td>2.65</td>
<td>1.04</td>
<td>25.89</td>
<td>1.20</td>
</tr>
<tr>
<td>15.18</td>
<td>22.00</td>
<td>4</td>
<td>25.00</td>
<td>0.37</td>
<td>3.10</td>
<td>1.16</td>
<td>28.97</td>
<td>1.07</td>
</tr>
<tr>
<td>17.68</td>
<td>22.00</td>
<td>5</td>
<td>25.00</td>
<td>0.36</td>
<td>3.56</td>
<td>1.27</td>
<td>31.65</td>
<td>1.23</td>
</tr>
<tr>
<td>20.18</td>
<td>22.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>4.02</td>
<td>1.36</td>
<td>34.05</td>
<td>1.14</td>
</tr>
<tr>
<td>22.68</td>
<td>22.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>4.47</td>
<td>1.52</td>
<td>37.92</td>
<td>1.03</td>
</tr>
<tr>
<td>25.18</td>
<td>22.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>4.93</td>
<td>1.67</td>
<td>41.78</td>
<td>1.12</td>
</tr>
<tr>
<td>27.68</td>
<td>22.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>5.38</td>
<td>1.83</td>
<td>45.65</td>
<td>1.02</td>
</tr>
<tr>
<td>30.18</td>
<td>22.00</td>
<td>7</td>
<td>25.00</td>
<td>0.34</td>
<td>5.84</td>
<td>1.98</td>
<td>49.51</td>
<td>1.10</td>
</tr>
</tbody>
</table>

### Static Internal Stability - Pullout Force Results
Rec. #1 - H = 31.43 (ft) B = 22.00 (ft)

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Le (ft)</th>
<th>F*</th>
<th>Pr (Kips)</th>
<th>Static CDR Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.68</td>
<td>22.00</td>
<td>3</td>
<td>39.29</td>
<td>0.46</td>
<td>0.36</td>
<td>0.17</td>
<td>8.85</td>
<td>12.57</td>
<td>74.79</td>
<td>9.56</td>
<td>1.08</td>
</tr>
<tr>
<td>5.18</td>
<td>22.00</td>
<td>3</td>
<td>25.00</td>
<td>0.44</td>
<td>0.70</td>
<td>0.31</td>
<td>10.48</td>
<td>12.57</td>
<td>69.78</td>
<td>17.25</td>
<td>1.65</td>
</tr>
<tr>
<td>7.68</td>
<td>22.00</td>
<td>3</td>
<td>25.00</td>
<td>0.43</td>
<td>1.04</td>
<td>0.44</td>
<td>14.92</td>
<td>12.57</td>
<td>64.77</td>
<td>23.74</td>
<td>1.59</td>
</tr>
<tr>
<td>10.18</td>
<td>22.00</td>
<td>3</td>
<td>25.00</td>
<td>0.41</td>
<td>1.37</td>
<td>0.56</td>
<td>18.95</td>
<td>12.57</td>
<td>59.76</td>
<td>29.03</td>
<td>1.53</td>
</tr>
<tr>
<td>12.68</td>
<td>22.00</td>
<td>4</td>
<td>25.00</td>
<td>0.39</td>
<td>1.71</td>
<td>0.67</td>
<td>22.59</td>
<td>12.57</td>
<td>54.75</td>
<td>44.18</td>
<td>1.96</td>
</tr>
<tr>
<td>15.18</td>
<td>22.00</td>
<td>4</td>
<td>25.00</td>
<td>0.37</td>
<td>2.05</td>
<td>0.77</td>
<td>25.82</td>
<td>12.57</td>
<td>49.74</td>
<td>48.05</td>
<td>1.86</td>
</tr>
<tr>
<td>17.68</td>
<td>22.00</td>
<td>5</td>
<td>25.00</td>
<td>0.36</td>
<td>2.39</td>
<td>0.85</td>
<td>28.65</td>
<td>13.75</td>
<td>44.73</td>
<td>68.80</td>
<td>2.40</td>
</tr>
<tr>
<td>20.18</td>
<td>22.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>2.72</td>
<td>0.92</td>
<td>31.19</td>
<td>15.25</td>
<td>40.08</td>
<td>78.04</td>
<td>2.50</td>
</tr>
<tr>
<td>22.68</td>
<td>22.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>3.06</td>
<td>1.04</td>
<td>35.06</td>
<td>16.75</td>
<td>40.08</td>
<td>96.33</td>
<td>2.75</td>
</tr>
<tr>
<td>25.18</td>
<td>22.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>3.40</td>
<td>1.15</td>
<td>38.92</td>
<td>18.25</td>
<td>40.08</td>
<td>139.83</td>
<td>3.59</td>
</tr>
<tr>
<td>27.68</td>
<td>22.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>3.74</td>
<td>1.27</td>
<td>42.78</td>
<td>19.75</td>
<td>40.08</td>
<td>166.35</td>
<td>3.89</td>
</tr>
<tr>
<td>30.18</td>
<td>22.00</td>
<td>7</td>
<td>25.00</td>
<td>0.34</td>
<td>4.07</td>
<td>1.38</td>
<td>46.65</td>
<td>21.25</td>
<td>40.08</td>
<td>227.68</td>
<td>4.88</td>
</tr>
</tbody>
</table>
MSE - Pro
Mechanically Stabilized Earth Retaining Structures

SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

Project Information

VAWS #: VW14-02388
Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1
Location: Barnstable County, MA
Project #: CM/TAP/TI-002S(758)X
Design Engineer: cms
Wall Name: B=0.7H Minimum

Rec. #2
Structure Parameters

<table>
<thead>
<tr>
<th>H max (ft)</th>
<th>B min (ft)</th>
<th>B Ext. (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>β (deg)</th>
<th>β λ (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>30.000</td>
<td>21.000</td>
<td>21.000</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

External Active Earth Pressure Coefficient: 0.283
External Stability CDR Summary (Effective Stress)

<table>
<thead>
<tr>
<th></th>
<th>CDR Sliding</th>
<th>Limiting Eccentricity (ft)</th>
<th>CDR Overturning</th>
<th>Bearing Pressure (ksf)</th>
<th>Eccentricity</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>Service Min</td>
<td>2.56</td>
<td>2.47</td>
<td>4.25</td>
<td>5.70</td>
<td>2.23</td>
<td>0.500</td>
</tr>
<tr>
<td>Service Max</td>
<td>2.25</td>
<td>2.73</td>
<td>3.85</td>
<td>7.78</td>
<td>2.53</td>
<td>0.500</td>
</tr>
<tr>
<td>Service Critical</td>
<td>1.67</td>
<td>3.68</td>
<td>2.85</td>
<td>6.57</td>
<td>3.32</td>
<td>0.500</td>
</tr>
</tbody>
</table>

Internal Active Earth Pressure Coefficient: 0.283
Internal Stability CDR Summary

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Internal Earth Coeff.</th>
<th>Rupture CDR</th>
<th>Pullout CDR</th>
<th>F*</th>
<th>Le (ft)</th>
<th>H/B Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>21.00</td>
<td>3</td>
<td>0.47</td>
<td>2.48</td>
<td>1.42</td>
<td>76.15</td>
<td>12.00</td>
<td>0.70</td>
</tr>
<tr>
<td>3.75</td>
<td>21.00</td>
<td>3</td>
<td>0.45</td>
<td>2.37</td>
<td>1.88</td>
<td>72.64</td>
<td>12.00</td>
<td>0.70</td>
</tr>
<tr>
<td>6.25</td>
<td>21.00</td>
<td>3</td>
<td>0.44</td>
<td>1.45</td>
<td>1.55</td>
<td>67.63</td>
<td>12.00</td>
<td>0.70</td>
</tr>
<tr>
<td>8.75</td>
<td>21.00</td>
<td>3</td>
<td>0.42</td>
<td>1.15</td>
<td>1.50</td>
<td>62.82</td>
<td>12.00</td>
<td>0.70</td>
</tr>
<tr>
<td>11.25</td>
<td>21.00</td>
<td>4</td>
<td>0.40</td>
<td>1.30</td>
<td>1.92</td>
<td>57.61</td>
<td>12.00</td>
<td>0.70</td>
</tr>
<tr>
<td>13.75</td>
<td>21.00</td>
<td>4</td>
<td>0.38</td>
<td>1.14</td>
<td>1.83</td>
<td>52.60</td>
<td>12.00</td>
<td>0.70</td>
</tr>
<tr>
<td>16.25</td>
<td>21.00</td>
<td>4</td>
<td>0.37</td>
<td>1.03</td>
<td>1.84</td>
<td>47.59</td>
<td>12.75</td>
<td>0.70</td>
</tr>
<tr>
<td>18.75</td>
<td>21.00</td>
<td>5</td>
<td>0.35</td>
<td>1.19</td>
<td>2.42</td>
<td>42.58</td>
<td>14.25</td>
<td>0.70</td>
</tr>
<tr>
<td>21.25</td>
<td>21.00</td>
<td>5</td>
<td>0.34</td>
<td>1.09</td>
<td>2.58</td>
<td>40.08</td>
<td>15.75</td>
<td>0.70</td>
</tr>
<tr>
<td>23.75</td>
<td>21.00</td>
<td>6</td>
<td>0.34</td>
<td>1.18</td>
<td>3.40</td>
<td>40.08</td>
<td>17.25</td>
<td>0.70</td>
</tr>
<tr>
<td>26.25</td>
<td>21.00</td>
<td>6</td>
<td>0.34</td>
<td>1.07</td>
<td>3.69</td>
<td>40.08</td>
<td>18.75</td>
<td>0.70</td>
</tr>
<tr>
<td>28.75</td>
<td>21.00</td>
<td>7</td>
<td>0.34</td>
<td>1.15</td>
<td>4.65</td>
<td>40.08</td>
<td>20.25</td>
<td>0.70</td>
</tr>
</tbody>
</table>
## SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

### Project Information

VAWS #: VW14-02388  
Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1  
Location: Barnstable County, MA  
Project #: CM/TAP/TI-002S(758)X  
Design Engineer: cm  
Wall Name: B=0.7H Minimum

### Static Internal Stability - Rupture Force Results

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Static CDR Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>21.00</td>
<td>3</td>
<td>28.75</td>
<td>0.47</td>
<td>0.70</td>
<td>0.33</td>
<td>9.41</td>
<td>2.48</td>
</tr>
<tr>
<td>3.75</td>
<td>21.00</td>
<td>3</td>
<td>21.25</td>
<td>0.45</td>
<td>1.02</td>
<td>0.46</td>
<td>9.85</td>
<td>2.37</td>
</tr>
<tr>
<td>6.25</td>
<td>21.00</td>
<td>3</td>
<td>25.00</td>
<td>0.44</td>
<td>1.48</td>
<td>0.64</td>
<td>16.11</td>
<td>1.45</td>
</tr>
<tr>
<td>8.75</td>
<td>21.00</td>
<td>3</td>
<td>25.00</td>
<td>0.42</td>
<td>1.93</td>
<td>0.81</td>
<td>20.23</td>
<td>1.15</td>
</tr>
<tr>
<td>11.25</td>
<td>21.00</td>
<td>4</td>
<td>25.00</td>
<td>0.40</td>
<td>2.39</td>
<td>0.96</td>
<td>23.94</td>
<td>1.30</td>
</tr>
<tr>
<td>13.75</td>
<td>21.00</td>
<td>4</td>
<td>25.00</td>
<td>0.38</td>
<td>2.84</td>
<td>1.09</td>
<td>27.26</td>
<td>1.14</td>
</tr>
<tr>
<td>16.25</td>
<td>21.00</td>
<td>4</td>
<td>25.00</td>
<td>0.37</td>
<td>3.30</td>
<td>1.21</td>
<td>30.17</td>
<td>1.03</td>
</tr>
<tr>
<td>18.75</td>
<td>21.00</td>
<td>5</td>
<td>25.00</td>
<td>0.35</td>
<td>3.75</td>
<td>1.31</td>
<td>32.67</td>
<td>1.19</td>
</tr>
<tr>
<td>21.25</td>
<td>21.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>4.21</td>
<td>1.43</td>
<td>35.71</td>
<td>1.09</td>
</tr>
<tr>
<td>23.75</td>
<td>21.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>4.67</td>
<td>1.58</td>
<td>39.57</td>
<td>1.18</td>
</tr>
<tr>
<td>26.25</td>
<td>21.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>5.12</td>
<td>1.74</td>
<td>43.44</td>
<td>1.07</td>
</tr>
<tr>
<td>28.75</td>
<td>21.00</td>
<td>7</td>
<td>25.00</td>
<td>0.34</td>
<td>5.58</td>
<td>1.89</td>
<td>47.30</td>
<td>1.15</td>
</tr>
</tbody>
</table>

### Static Internal Stability - Pullout Force Results

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Static CDR Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>21.00</td>
<td>3</td>
<td>28.75</td>
<td>0.47</td>
<td>0.27</td>
<td>0.13</td>
<td>4.89</td>
<td>12.00</td>
</tr>
<tr>
<td>3.75</td>
<td>21.00</td>
<td>3</td>
<td>21.25</td>
<td>0.45</td>
<td>0.51</td>
<td>0.23</td>
<td>6.60</td>
<td>12.00</td>
</tr>
<tr>
<td>6.25</td>
<td>21.00</td>
<td>3</td>
<td>25.00</td>
<td>0.44</td>
<td>0.84</td>
<td>0.37</td>
<td>12.43</td>
<td>12.00</td>
</tr>
<tr>
<td>8.75</td>
<td>21.00</td>
<td>3</td>
<td>25.00</td>
<td>0.42</td>
<td>1.18</td>
<td>0.49</td>
<td>16.70</td>
<td>12.00</td>
</tr>
<tr>
<td>11.25</td>
<td>21.00</td>
<td>4</td>
<td>25.00</td>
<td>0.40</td>
<td>1.52</td>
<td>0.61</td>
<td>20.56</td>
<td>12.00</td>
</tr>
<tr>
<td>13.75</td>
<td>21.00</td>
<td>4</td>
<td>25.00</td>
<td>0.38</td>
<td>1.86</td>
<td>0.71</td>
<td>24.02</td>
<td>12.00</td>
</tr>
<tr>
<td>16.25</td>
<td>21.00</td>
<td>4</td>
<td>25.00</td>
<td>0.37</td>
<td>2.19</td>
<td>0.80</td>
<td>27.08</td>
<td>12.75</td>
</tr>
<tr>
<td>18.75</td>
<td>21.00</td>
<td>5</td>
<td>25.00</td>
<td>0.35</td>
<td>2.53</td>
<td>0.88</td>
<td>29.74</td>
<td>14.25</td>
</tr>
<tr>
<td>21.25</td>
<td>21.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>2.87</td>
<td>0.97</td>
<td>32.85</td>
<td>15.75</td>
</tr>
<tr>
<td>23.75</td>
<td>21.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>3.21</td>
<td>1.09</td>
<td>36.71</td>
<td>17.25</td>
</tr>
<tr>
<td>26.25</td>
<td>21.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>3.54</td>
<td>1.20</td>
<td>40.58</td>
<td>18.75</td>
</tr>
<tr>
<td>28.75</td>
<td>21.00</td>
<td>7</td>
<td>25.00</td>
<td>0.34</td>
<td>3.88</td>
<td>1.32</td>
<td>44.44</td>
<td>20.25</td>
</tr>
</tbody>
</table>
SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

**Project Information**

VAWS #: VW14-02388  
Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1  
Location: Barnstable County, MA  
Project #: CM/TAP/TI-002S(758)X  
Design Engineer: cm:  
Wall Name: B=0.7H Minimum

<table>
<thead>
<tr>
<th>Rec. #3 Structure Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>H max (ft)</td>
</tr>
<tr>
<td>----------------</td>
</tr>
<tr>
<td>28.571</td>
</tr>
</tbody>
</table>

**External Active Earth Pressure Coefficient:** 0.283  
**External Stability CDR Summary (Effective Stress)**

<table>
<thead>
<tr>
<th></th>
<th>CDR Sliding</th>
<th>Limiting Eccentricity (ft)</th>
<th>CDR Overturning</th>
<th>Bearing Pressure (ksf)</th>
<th>Eccentricity</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>Service 1 - Min</td>
<td>2.54</td>
<td>2.39</td>
<td>4.19</td>
<td>5.47</td>
<td>2.14</td>
<td>0.500</td>
</tr>
<tr>
<td>Service 1 - Max</td>
<td>2.24</td>
<td>2.62</td>
<td>3.81</td>
<td>7.45</td>
<td>2.42</td>
<td>0.500</td>
</tr>
<tr>
<td>Service 1 - Critical</td>
<td>1.66</td>
<td>3.54</td>
<td>2.83</td>
<td>6.30</td>
<td>3.18</td>
<td>0.500</td>
</tr>
</tbody>
</table>

**Internal Active Earth Pressure Coefficient:** 0.283  
**Internal Stability CDR Summary**

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Internal Earth Coeff.</th>
<th>Rupture CDR</th>
<th>Pullout CDR</th>
<th>F*</th>
<th>Le (ft)</th>
<th>H/B Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.32</td>
<td>20.00</td>
<td>3</td>
<td>0.46</td>
<td>1.85</td>
<td>1.08</td>
<td>75.51</td>
<td>11.43</td>
<td>0.70</td>
</tr>
<tr>
<td>4.82</td>
<td>20.00</td>
<td>3</td>
<td>0.45</td>
<td>1.72</td>
<td>1.50</td>
<td>70.50</td>
<td>11.43</td>
<td>0.70</td>
</tr>
<tr>
<td>7.32</td>
<td>20.00</td>
<td>3</td>
<td>0.43</td>
<td>1.30</td>
<td>1.45</td>
<td>65.49</td>
<td>11.43</td>
<td>0.70</td>
</tr>
<tr>
<td>9.82</td>
<td>20.00</td>
<td>3</td>
<td>0.41</td>
<td>1.07</td>
<td>1.40</td>
<td>60.48</td>
<td>11.43</td>
<td>0.70</td>
</tr>
<tr>
<td>12.32</td>
<td>20.00</td>
<td>4</td>
<td>0.39</td>
<td>1.22</td>
<td>1.79</td>
<td>55.46</td>
<td>11.43</td>
<td>0.70</td>
</tr>
<tr>
<td>14.82</td>
<td>20.00</td>
<td>4</td>
<td>0.38</td>
<td>1.09</td>
<td>1.75</td>
<td>50.45</td>
<td>11.75</td>
<td>0.70</td>
</tr>
<tr>
<td>17.32</td>
<td>20.00</td>
<td>5</td>
<td>0.36</td>
<td>1.24</td>
<td>2.33</td>
<td>45.44</td>
<td>13.25</td>
<td>0.70</td>
</tr>
<tr>
<td>19.82</td>
<td>20.00</td>
<td>5</td>
<td>0.34</td>
<td>1.16</td>
<td>2.43</td>
<td>40.43</td>
<td>14.75</td>
<td>0.70</td>
</tr>
<tr>
<td>22.32</td>
<td>20.00</td>
<td>5</td>
<td>0.34</td>
<td>1.04</td>
<td>2.67</td>
<td>40.08</td>
<td>16.25</td>
<td>0.70</td>
</tr>
<tr>
<td>24.82</td>
<td>20.00</td>
<td>6</td>
<td>0.34</td>
<td>1.13</td>
<td>3.49</td>
<td>40.08</td>
<td>17.75</td>
<td>0.70</td>
</tr>
<tr>
<td>27.32</td>
<td>20.00</td>
<td>6</td>
<td>0.34</td>
<td>1.04</td>
<td>3.79</td>
<td>40.08</td>
<td>19.25</td>
<td>0.70</td>
</tr>
</tbody>
</table>
### Project Information

VAWS #: VW14-02388  
Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1  
Location: Barnstable County, MA  
Project #: CM/TAP/TI-002S(758)X  
Design Engineer: cms  
Wall Name: B=0.7H Minimum

### Static Internal Stability - Rupture Force Results

Rec. #3 - H = 28.57 (ft) B = 20.00 (ft)

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Static CDR Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.32</td>
<td>20.00</td>
<td>3</td>
<td>35.71</td>
<td>0.46</td>
<td>0.76</td>
<td>0.35</td>
<td>12.61</td>
<td>1.85</td>
</tr>
<tr>
<td>4.82</td>
<td>20.00</td>
<td>3</td>
<td>25.00</td>
<td>0.45</td>
<td>1.22</td>
<td>0.54</td>
<td>13.58</td>
<td>1.72</td>
</tr>
<tr>
<td>7.32</td>
<td>20.00</td>
<td>3</td>
<td>25.00</td>
<td>0.43</td>
<td>1.67</td>
<td>0.72</td>
<td>17.92</td>
<td>1.30</td>
</tr>
<tr>
<td>9.82</td>
<td>20.00</td>
<td>3</td>
<td>25.00</td>
<td>0.41</td>
<td>2.13</td>
<td>0.87</td>
<td>21.87</td>
<td>1.07</td>
</tr>
<tr>
<td>12.32</td>
<td>20.00</td>
<td>3</td>
<td>25.00</td>
<td>0.39</td>
<td>2.58</td>
<td>1.02</td>
<td>25.41</td>
<td>1.22</td>
</tr>
<tr>
<td>14.82</td>
<td>20.00</td>
<td>3</td>
<td>25.00</td>
<td>0.37</td>
<td>3.04</td>
<td>1.14</td>
<td>28.55</td>
<td>1.09</td>
</tr>
<tr>
<td>17.32</td>
<td>20.00</td>
<td>3</td>
<td>25.00</td>
<td>0.35</td>
<td>3.49</td>
<td>1.25</td>
<td>31.29</td>
<td>1.24</td>
</tr>
<tr>
<td>19.82</td>
<td>20.00</td>
<td>3</td>
<td>25.00</td>
<td>0.34</td>
<td>3.95</td>
<td>1.35</td>
<td>33.63</td>
<td>1.16</td>
</tr>
<tr>
<td>22.32</td>
<td>20.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>4.41</td>
<td>1.49</td>
<td>37.37</td>
<td>1.04</td>
</tr>
<tr>
<td>24.82</td>
<td>20.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>4.86</td>
<td>1.65</td>
<td>41.23</td>
<td>1.13</td>
</tr>
<tr>
<td>27.32</td>
<td>20.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>5.32</td>
<td>1.80</td>
<td>45.09</td>
<td>1.04</td>
</tr>
</tbody>
</table>

### Static Internal Stability - Pullout Force Results

Rec. #3 - H = 28.57 (ft) B = 20.00 (ft)

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Le (ft)</th>
<th>F*</th>
<th>Pr (Kips)</th>
<th>Static CDR Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.32</td>
<td>20.00</td>
<td>3</td>
<td>35.71</td>
<td>0.46</td>
<td>0.31</td>
<td>0.15</td>
<td>7.01</td>
<td>11.43</td>
<td>75.51</td>
<td>7.61</td>
<td>1.08</td>
</tr>
<tr>
<td>4.82</td>
<td>20.00</td>
<td>3</td>
<td>25.00</td>
<td>0.45</td>
<td>0.65</td>
<td>0.29</td>
<td>9.81</td>
<td>11.43</td>
<td>70.50</td>
<td>14.75</td>
<td>1.50</td>
</tr>
<tr>
<td>7.32</td>
<td>20.00</td>
<td>3</td>
<td>25.00</td>
<td>0.43</td>
<td>0.99</td>
<td>0.42</td>
<td>14.31</td>
<td>11.43</td>
<td>65.49</td>
<td>20.80</td>
<td>1.45</td>
</tr>
<tr>
<td>9.82</td>
<td>20.00</td>
<td>3</td>
<td>25.00</td>
<td>0.41</td>
<td>1.33</td>
<td>0.55</td>
<td>18.40</td>
<td>11.43</td>
<td>60.48</td>
<td>25.77</td>
<td>1.40</td>
</tr>
<tr>
<td>12.32</td>
<td>20.00</td>
<td>3</td>
<td>25.00</td>
<td>0.39</td>
<td>1.66</td>
<td>0.65</td>
<td>22.09</td>
<td>11.43</td>
<td>55.46</td>
<td>39.54</td>
<td>1.79</td>
</tr>
<tr>
<td>14.82</td>
<td>20.00</td>
<td>3</td>
<td>25.00</td>
<td>0.37</td>
<td>2.00</td>
<td>0.75</td>
<td>25.38</td>
<td>11.75</td>
<td>50.45</td>
<td>44.48</td>
<td>1.75</td>
</tr>
<tr>
<td>17.32</td>
<td>20.00</td>
<td>5</td>
<td>25.00</td>
<td>0.36</td>
<td>2.34</td>
<td>0.84</td>
<td>28.27</td>
<td>13.25</td>
<td>45.44</td>
<td>66.00</td>
<td>2.33</td>
</tr>
<tr>
<td>19.82</td>
<td>20.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>2.68</td>
<td>0.91</td>
<td>30.75</td>
<td>14.75</td>
<td>40.43</td>
<td>74.80</td>
<td>2.43</td>
</tr>
<tr>
<td>22.32</td>
<td>20.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>3.01</td>
<td>1.02</td>
<td>34.50</td>
<td>16.25</td>
<td>40.08</td>
<td>91.98</td>
<td>2.67</td>
</tr>
<tr>
<td>24.82</td>
<td>20.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>3.35</td>
<td>1.14</td>
<td>38.37</td>
<td>17.75</td>
<td>40.08</td>
<td>134.07</td>
<td>3.49</td>
</tr>
<tr>
<td>27.32</td>
<td>20.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>3.69</td>
<td>1.25</td>
<td>42.23</td>
<td>19.25</td>
<td>40.08</td>
<td>160.05</td>
<td>3.79</td>
</tr>
</tbody>
</table>
## Project Information

**VAWS #:** VW14-02388  
**Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1  
**Location:** Barnstable County, MA  
**Project #:** CM/TAP/TI-002S(758)X  
**Design Engineer:** cms  
**Wall Name:** B=0.7H Minimum

### Structure Parameters

<table>
<thead>
<tr>
<th>H max (ft)</th>
<th>B min (ft)</th>
<th>B Ext. (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>β (deg)</th>
<th>β′ (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>27.143</td>
<td>19.000</td>
<td>19.000</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

### External Active Earth Pressure Coefficient: 0.283

**External Stability CDR Summary (Effective Stress)**

<table>
<thead>
<tr>
<th>Sliding</th>
<th>Limiting Eccentricity (ft)</th>
<th>CDR Overturning</th>
<th>Bearing Pressure (ksf)</th>
<th>Eccentricity Overturning</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>S: re: g1 - Min</td>
<td>2.51</td>
<td>2.30</td>
<td>4.13</td>
<td>5.23</td>
<td>2.05</td>
</tr>
<tr>
<td>S: re: g1 - Max</td>
<td>2.22</td>
<td>2.51</td>
<td>3.78</td>
<td>7.11</td>
<td>2.31</td>
</tr>
<tr>
<td>S: re: g1 - Critical</td>
<td>1.64</td>
<td>3.40</td>
<td>2.80</td>
<td>6.03</td>
<td>3.03</td>
</tr>
<tr>
<td>Service</td>
<td>2.52</td>
<td>2.20</td>
<td>4.32</td>
<td>5.00</td>
<td>2.06</td>
</tr>
</tbody>
</table>

### Internal Active Earth Pressure Coefficient: 0.283

**Internal Stability CDR Summary**

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Internal Earth Coeff.</th>
<th>Rupture CDR</th>
<th>Pullout CDR</th>
<th>F*</th>
<th>Le (ft)</th>
<th>H/B Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>19.00</td>
<td>3</td>
<td>0.47</td>
<td>2.64</td>
<td>1.37</td>
<td>76.15</td>
<td>10.86</td>
<td>0.70</td>
</tr>
<tr>
<td>3.39</td>
<td>19.00</td>
<td>3</td>
<td>0.46</td>
<td>2.75</td>
<td>1.87</td>
<td>73.36</td>
<td>10.86</td>
<td>0.70</td>
</tr>
<tr>
<td>5.89</td>
<td>19.00</td>
<td>3</td>
<td>0.44</td>
<td>1.51</td>
<td>1.41</td>
<td>68.35</td>
<td>10.86</td>
<td>0.70</td>
</tr>
<tr>
<td>8.39</td>
<td>19.00</td>
<td>3</td>
<td>0.42</td>
<td>1.19</td>
<td>1.36</td>
<td>63.34</td>
<td>10.86</td>
<td>0.70</td>
</tr>
<tr>
<td>10.89</td>
<td>19.00</td>
<td>4</td>
<td>0.40</td>
<td>1.33</td>
<td>1.74</td>
<td>58.33</td>
<td>10.86</td>
<td>0.70</td>
</tr>
<tr>
<td>13.39</td>
<td>19.00</td>
<td>4</td>
<td>0.39</td>
<td>1.16</td>
<td>1.67</td>
<td>53.32</td>
<td>10.86</td>
<td>0.70</td>
</tr>
<tr>
<td>15.89</td>
<td>19.00</td>
<td>4</td>
<td>0.37</td>
<td>1.05</td>
<td>1.79</td>
<td>48.31</td>
<td>12.25</td>
<td>0.70</td>
</tr>
<tr>
<td>18.39</td>
<td>19.00</td>
<td>5</td>
<td>0.35</td>
<td>1.20</td>
<td>2.36</td>
<td>43.30</td>
<td>13.75</td>
<td>0.70</td>
</tr>
<tr>
<td>20.89</td>
<td>19.00</td>
<td>5</td>
<td>0.34</td>
<td>1.11</td>
<td>2.50</td>
<td>40.08</td>
<td>15.25</td>
<td>0.70</td>
</tr>
<tr>
<td>23.39</td>
<td>19.00</td>
<td>6</td>
<td>0.34</td>
<td>1.20</td>
<td>3.30</td>
<td>40.08</td>
<td>16.75</td>
<td>0.70</td>
</tr>
<tr>
<td>25.89</td>
<td>19.00</td>
<td>6</td>
<td>0.34</td>
<td>1.09</td>
<td>3.59</td>
<td>40.08</td>
<td>18.25</td>
<td>0.70</td>
</tr>
</tbody>
</table>
## Static Internal Stability - Rupture Force Results

**Rec. #4 - H = 27.14 (ft) B = 19.00 (ft)**

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Static CDR Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>19.00</td>
<td>3</td>
<td>26.96</td>
<td>0.47</td>
<td>0.70</td>
<td>0.33</td>
<td>8.83</td>
<td>2.64</td>
</tr>
<tr>
<td>3.39</td>
<td>19.00</td>
<td>3</td>
<td>19.46</td>
<td>0.46</td>
<td>0.96</td>
<td>0.44</td>
<td>8.50</td>
<td>2.75</td>
</tr>
<tr>
<td>5.89</td>
<td>19.00</td>
<td>3</td>
<td>25.00</td>
<td>0.44</td>
<td>1.41</td>
<td>0.62</td>
<td>15.49</td>
<td>1.51</td>
</tr>
<tr>
<td>8.39</td>
<td>19.00</td>
<td>3</td>
<td>25.00</td>
<td>0.42</td>
<td>1.87</td>
<td>0.79</td>
<td>19.67</td>
<td>1.19</td>
</tr>
<tr>
<td>10.89</td>
<td>19.00</td>
<td>4</td>
<td>25.00</td>
<td>0.40</td>
<td>2.32</td>
<td>0.94</td>
<td>23.44</td>
<td>1.33</td>
</tr>
<tr>
<td>13.39</td>
<td>19.00</td>
<td>4</td>
<td>25.00</td>
<td>0.39</td>
<td>2.78</td>
<td>1.07</td>
<td>26.81</td>
<td>1.16</td>
</tr>
<tr>
<td>15.89</td>
<td>19.00</td>
<td>4</td>
<td>25.00</td>
<td>0.37</td>
<td>3.23</td>
<td>1.19</td>
<td>29.78</td>
<td>1.05</td>
</tr>
<tr>
<td>18.39</td>
<td>19.00</td>
<td>5</td>
<td>25.00</td>
<td>0.35</td>
<td>3.69</td>
<td>1.29</td>
<td>32.34</td>
<td>1.20</td>
</tr>
<tr>
<td>20.89</td>
<td>19.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>4.15</td>
<td>1.41</td>
<td>35.16</td>
<td>1.11</td>
</tr>
<tr>
<td>23.39</td>
<td>19.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>4.60</td>
<td>1.56</td>
<td>39.02</td>
<td>1.20</td>
</tr>
<tr>
<td>25.89</td>
<td>19.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>5.06</td>
<td>1.72</td>
<td>42.89</td>
<td>1.09</td>
</tr>
</tbody>
</table>

## Static Internal Stability - Pullout Force Results

**Rec. #4 - H = 27.14 (ft) B = 19.00 (ft)**

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horiz. Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Le (ft)</th>
<th>F*</th>
<th>Pr (Kips)</th>
<th>Static CDR Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>19.00</td>
<td>3</td>
<td>26.96</td>
<td>0.47</td>
<td>0.27</td>
<td>0.13</td>
<td>4.58</td>
<td>10.86</td>
<td>76.15</td>
<td>6.28</td>
<td>1.37</td>
</tr>
<tr>
<td>3.39</td>
<td>19.00</td>
<td>3</td>
<td>19.46</td>
<td>0.46</td>
<td>0.46</td>
<td>0.21</td>
<td>5.50</td>
<td>10.86</td>
<td>73.36</td>
<td>10.26</td>
<td>1.87</td>
</tr>
<tr>
<td>5.89</td>
<td>19.00</td>
<td>3</td>
<td>25.00</td>
<td>0.44</td>
<td>0.80</td>
<td>0.35</td>
<td>11.79</td>
<td>10.86</td>
<td>68.35</td>
<td>16.60</td>
<td>1.41</td>
</tr>
<tr>
<td>8.39</td>
<td>19.00</td>
<td>3</td>
<td>25.00</td>
<td>0.42</td>
<td>1.13</td>
<td>0.48</td>
<td>16.11</td>
<td>10.86</td>
<td>63.34</td>
<td>21.91</td>
<td>1.36</td>
</tr>
<tr>
<td>10.89</td>
<td>19.00</td>
<td>4</td>
<td>25.00</td>
<td>0.40</td>
<td>1.47</td>
<td>0.59</td>
<td>20.03</td>
<td>10.86</td>
<td>58.33</td>
<td>34.92</td>
<td>1.74</td>
</tr>
<tr>
<td>13.39</td>
<td>19.00</td>
<td>4</td>
<td>25.00</td>
<td>0.39</td>
<td>1.81</td>
<td>0.70</td>
<td>23.55</td>
<td>10.86</td>
<td>53.32</td>
<td>39.25</td>
<td>1.67</td>
</tr>
<tr>
<td>15.89</td>
<td>19.00</td>
<td>4</td>
<td>25.00</td>
<td>0.37</td>
<td>2.15</td>
<td>0.79</td>
<td>26.67</td>
<td>12.25</td>
<td>48.31</td>
<td>47.61</td>
<td>1.79</td>
</tr>
<tr>
<td>18.39</td>
<td>19.00</td>
<td>5</td>
<td>25.00</td>
<td>0.35</td>
<td>2.48</td>
<td>0.87</td>
<td>29.38</td>
<td>13.75</td>
<td>43.30</td>
<td>69.29</td>
<td>2.36</td>
</tr>
<tr>
<td>20.89</td>
<td>19.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>2.82</td>
<td>0.96</td>
<td>32.30</td>
<td>15.25</td>
<td>40.08</td>
<td>80.80</td>
<td>2.50</td>
</tr>
<tr>
<td>23.39</td>
<td>19.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>3.16</td>
<td>1.07</td>
<td>36.16</td>
<td>16.75</td>
<td>40.08</td>
<td>119.24</td>
<td>3.30</td>
</tr>
<tr>
<td>25.89</td>
<td>19.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>3.50</td>
<td>1.19</td>
<td>40.02</td>
<td>18.25</td>
<td>40.08</td>
<td>143.80</td>
<td>3.59</td>
</tr>
</tbody>
</table>
**Project Information**

VAWS #: VW14-02388  
Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1  
Location: Barnstable County, MA  
Project #: CM/TAP/TI-002S(758)X  
Design Engineer: cms  
Wall Name: B=0.7H Minimum

### Structure Parameters

<table>
<thead>
<tr>
<th>H max (ft)</th>
<th>B min (ft)</th>
<th>B Ext. (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>β (deg)</th>
<th>β t (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>25.714</td>
<td>18.000</td>
<td>18.000</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

### External Active Earth Pressure Coefficient: 0.283

#### External Stability CDR Summary (Effective Stress)

<table>
<thead>
<tr>
<th>CDR Sliding</th>
<th>Limiting Eccentricity (ft)</th>
<th>CDR Overturning</th>
<th>Bearing Pressure (ksf)</th>
<th>Eccentricity (ft)</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>S: re: g 1 - Min</td>
<td>2.48</td>
<td>2.21</td>
<td>4.07</td>
<td>5.00</td>
<td>1.96</td>
</tr>
<tr>
<td>S: re: g 1 - Max</td>
<td>2.20</td>
<td>2.41</td>
<td>3.74</td>
<td>6.78</td>
<td>2.20</td>
</tr>
<tr>
<td>S: re: g 1 - Critical</td>
<td>1.63</td>
<td>3.25</td>
<td>2.77</td>
<td>5.75</td>
<td>2.89</td>
</tr>
<tr>
<td>Service</td>
<td>2.50</td>
<td>2.10</td>
<td>4.28</td>
<td>4.76</td>
<td>1.96</td>
</tr>
</tbody>
</table>

### Internal Active Earth Pressure Coefficient: 0.283

#### Internal Stability CDR Summary

<table>
<thead>
<tr>
<th>Depth Below Top Wall of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Internal Earth Coeff.</th>
<th>Rupture CDR</th>
<th>Pullout CDR</th>
<th>F*</th>
<th>Le (ft)</th>
<th>H/B Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>18.00</td>
<td>3</td>
<td>0.47</td>
<td>2.21</td>
<td>1.08</td>
<td>76.15</td>
<td>10.29</td>
<td>0.70</td>
</tr>
<tr>
<td>4.46</td>
<td>18.00</td>
<td>3</td>
<td>0.45</td>
<td>1.82</td>
<td>1.37</td>
<td>71.21</td>
<td>10.29</td>
<td>0.70</td>
</tr>
<tr>
<td>6.96</td>
<td>18.00</td>
<td>3</td>
<td>0.43</td>
<td>1.35</td>
<td>1.32</td>
<td>66.20</td>
<td>10.29</td>
<td>0.70</td>
</tr>
<tr>
<td>9.46</td>
<td>18.00</td>
<td>3</td>
<td>0.41</td>
<td>1.09</td>
<td>1.27</td>
<td>61.19</td>
<td>10.29</td>
<td>0.70</td>
</tr>
<tr>
<td>11.96</td>
<td>18.00</td>
<td>4</td>
<td>0.40</td>
<td>1.25</td>
<td>1.62</td>
<td>56.18</td>
<td>10.29</td>
<td>0.70</td>
</tr>
<tr>
<td>14.46</td>
<td>18.00</td>
<td>4</td>
<td>0.38</td>
<td>1.11</td>
<td>1.69</td>
<td>51.17</td>
<td>11.25</td>
<td>0.70</td>
</tr>
<tr>
<td>16.96</td>
<td>18.00</td>
<td>4</td>
<td>0.36</td>
<td>1.01</td>
<td>1.81</td>
<td>46.16</td>
<td>12.75</td>
<td>0.70</td>
</tr>
<tr>
<td>19.46</td>
<td>18.00</td>
<td>5</td>
<td>0.34</td>
<td>1.17</td>
<td>2.37</td>
<td>41.15</td>
<td>14.25</td>
<td>0.70</td>
</tr>
<tr>
<td>21.96</td>
<td>18.00</td>
<td>5</td>
<td>0.34</td>
<td>1.06</td>
<td>2.58</td>
<td>40.08</td>
<td>15.75</td>
<td>0.70</td>
</tr>
<tr>
<td>24.46</td>
<td>18.00</td>
<td>6</td>
<td>0.34</td>
<td>1.15</td>
<td>3.40</td>
<td>40.08</td>
<td>17.25</td>
<td>0.70</td>
</tr>
</tbody>
</table>
**MSE - Pro**

Mechanically Stabilized Earth Retaining Structures

**SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP**

### Project Information

VAWS #: VW14-02388  
Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1  
Location: Barnstable County, MA  
Project #: CM/TAP/TI-002S(758)X  
Design Engineer: cm**  
Wall Name: B=0.7H Minimum

### Static Internal Stability - Rupture Force Results

**Rec. #5 - H = 25.71 (ft) B = 18.00 (ft)**

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Static CDR Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>18.00</td>
<td>3</td>
<td>32.32</td>
<td>0.47</td>
<td>0.70</td>
<td>0.33</td>
<td>10.58</td>
<td>2.21</td>
</tr>
<tr>
<td>4.46</td>
<td>18.00</td>
<td>3</td>
<td>24.82</td>
<td>0.45</td>
<td>1.15</td>
<td>0.52</td>
<td>12.83</td>
<td>1.82</td>
</tr>
<tr>
<td>6.96</td>
<td>18.00</td>
<td>3</td>
<td>25.00</td>
<td>0.43</td>
<td>1.61</td>
<td>0.69</td>
<td>17.33</td>
<td>1.35</td>
</tr>
<tr>
<td>9.46</td>
<td>18.00</td>
<td>3</td>
<td>25.00</td>
<td>0.41</td>
<td>2.06</td>
<td>0.85</td>
<td>21.33</td>
<td>1.09</td>
</tr>
<tr>
<td>11.96</td>
<td>18.00</td>
<td>4</td>
<td>25.00</td>
<td>0.40</td>
<td>2.52</td>
<td>1.00</td>
<td>24.93</td>
<td>1.25</td>
</tr>
<tr>
<td>14.46</td>
<td>18.00</td>
<td>4</td>
<td>25.00</td>
<td>0.38</td>
<td>2.97</td>
<td>1.13</td>
<td>28.13</td>
<td>1.11</td>
</tr>
<tr>
<td>16.96</td>
<td>18.00</td>
<td>4</td>
<td>25.00</td>
<td>0.36</td>
<td>3.43</td>
<td>1.24</td>
<td>30.92</td>
<td>1.01</td>
</tr>
<tr>
<td>19.46</td>
<td>18.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>3.88</td>
<td>1.33</td>
<td>33.32</td>
<td>1.17</td>
</tr>
<tr>
<td>21.96</td>
<td>18.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>4.34</td>
<td>1.47</td>
<td>36.81</td>
<td>1.06</td>
</tr>
<tr>
<td>24.46</td>
<td>18.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>4.80</td>
<td>1.63</td>
<td>40.68</td>
<td>1.15</td>
</tr>
</tbody>
</table>

### Static Internal Stability - Pullout Force Results

**Rec. #5 - H = 25.71 (ft) B = 18.00 (ft)**

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Le (ft)</th>
<th>F*</th>
<th>Pr (Kips)</th>
<th>Static CDR Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>18.00</td>
<td>3</td>
<td>32.32</td>
<td>0.47</td>
<td>0.27</td>
<td>0.13</td>
<td>5.50</td>
<td>10.29</td>
<td>76.15</td>
<td>5.95</td>
<td>1.08</td>
</tr>
<tr>
<td>4.46</td>
<td>18.00</td>
<td>3</td>
<td>24.82</td>
<td>0.45</td>
<td>0.60</td>
<td>0.27</td>
<td>9.07</td>
<td>10.29</td>
<td>71.21</td>
<td>12.42</td>
<td>1.37</td>
</tr>
<tr>
<td>6.96</td>
<td>18.00</td>
<td>3</td>
<td>25.00</td>
<td>0.43</td>
<td>0.94</td>
<td>0.41</td>
<td>13.69</td>
<td>10.29</td>
<td>66.20</td>
<td>18.01</td>
<td>1.32</td>
</tr>
<tr>
<td>9.46</td>
<td>18.00</td>
<td>3</td>
<td>25.00</td>
<td>0.41</td>
<td>1.28</td>
<td>0.53</td>
<td>17.84</td>
<td>10.29</td>
<td>61.19</td>
<td>22.62</td>
<td>1.27</td>
</tr>
<tr>
<td>11.96</td>
<td>18.00</td>
<td>4</td>
<td>25.00</td>
<td>0.40</td>
<td>1.62</td>
<td>0.64</td>
<td>21.59</td>
<td>10.29</td>
<td>56.18</td>
<td>35.00</td>
<td>1.62</td>
</tr>
<tr>
<td>14.46</td>
<td>18.00</td>
<td>4</td>
<td>25.00</td>
<td>0.38</td>
<td>1.95</td>
<td>0.74</td>
<td>24.94</td>
<td>11.25</td>
<td>51.17</td>
<td>42.15</td>
<td>1.69</td>
</tr>
<tr>
<td>16.96</td>
<td>18.00</td>
<td>4</td>
<td>25.00</td>
<td>0.36</td>
<td>2.29</td>
<td>0.83</td>
<td>27.88</td>
<td>12.75</td>
<td>46.16</td>
<td>50.54</td>
<td>1.81</td>
</tr>
<tr>
<td>19.46</td>
<td>18.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>2.63</td>
<td>0.90</td>
<td>30.42</td>
<td>14.25</td>
<td>41.15</td>
<td>72.22</td>
<td>2.37</td>
</tr>
<tr>
<td>21.96</td>
<td>18.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>2.97</td>
<td>1.01</td>
<td>33.95</td>
<td>15.75</td>
<td>40.08</td>
<td>87.73</td>
<td>2.58</td>
</tr>
<tr>
<td>24.46</td>
<td>18.00</td>
<td>6</td>
<td>25.00</td>
<td>0.34</td>
<td>3.30</td>
<td>1.12</td>
<td>37.82</td>
<td>17.25</td>
<td>40.08</td>
<td>128.42</td>
<td>3.40</td>
</tr>
</tbody>
</table>
### SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

#### Project Information
- **VAWS #**: VW14-02388
- **Name**: CAPE COD RAIL TRAIL EXTENSION - PHASE 1
- **Location**: Barnstable County, MA
- **Project #**: CM/TAP/TI-002S(758)X
- **Design Engineer**: cm
- **Wall Name**: B=0.7H Minimum

#### Structure Parameters

<table>
<thead>
<tr>
<th>H max (ft)</th>
<th>B min (ft)</th>
<th>B Ext. (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>β (deg)</th>
<th>β t (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24.286</td>
<td>17.000</td>
<td>17.000</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

#### External Active Earth Pressure Coefficient: 0.283

#### External Stability CDR Summary (Effective Stress)

<table>
<thead>
<tr>
<th>Stress</th>
<th>CDR Sliding</th>
<th>Limiting Eccentricity (ft)</th>
<th>CDR Overturning</th>
<th>Bearing Pressure (ksf)</th>
<th>Eccentricity (ft)</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min</td>
<td>2.45</td>
<td>2.13</td>
<td>4.00</td>
<td>4.77</td>
<td>1.88</td>
<td>0.500</td>
</tr>
<tr>
<td>Max</td>
<td>2.18</td>
<td>2.30</td>
<td>3.69</td>
<td>6.45</td>
<td>2.09</td>
<td>0.500</td>
</tr>
<tr>
<td>Critical</td>
<td>1.62</td>
<td>3.11</td>
<td>2.74</td>
<td>5.48</td>
<td>2.74</td>
<td>0.500</td>
</tr>
<tr>
<td>Service</td>
<td>2.48</td>
<td>2.01</td>
<td>4.23</td>
<td>4.52</td>
<td>1.87</td>
<td>0.500</td>
</tr>
</tbody>
</table>

#### Internal Active Earth Pressure Coefficient: 0.283

#### Internal Stability CDR Summary

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Internal Earth Coeff.</th>
<th>Rupture CDR</th>
<th>Pullout CDR</th>
<th>F*</th>
<th>Le (ft)</th>
<th>H/B Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>17.00</td>
<td>3</td>
<td>0.47</td>
<td>2.83</td>
<td>1.31</td>
<td>76.15</td>
<td>9.71</td>
<td>0.70</td>
</tr>
<tr>
<td>3.04</td>
<td>17.00</td>
<td>3</td>
<td>0.46</td>
<td>3.23</td>
<td>1.85</td>
<td>74.08</td>
<td>9.71</td>
<td>0.70</td>
</tr>
<tr>
<td>5.54</td>
<td>17.00</td>
<td>3</td>
<td>0.44</td>
<td>1.57</td>
<td>1.27</td>
<td>69.07</td>
<td>9.71</td>
<td>0.70</td>
</tr>
<tr>
<td>8.04</td>
<td>17.00</td>
<td>3</td>
<td>0.42</td>
<td>1.22</td>
<td>1.22</td>
<td>64.05</td>
<td>9.71</td>
<td>0.70</td>
</tr>
<tr>
<td>10.54</td>
<td>17.00</td>
<td>3</td>
<td>0.41</td>
<td>1.02</td>
<td>1.18</td>
<td>59.04</td>
<td>9.71</td>
<td>0.70</td>
</tr>
<tr>
<td>13.04</td>
<td>17.00</td>
<td>4</td>
<td>0.39</td>
<td>1.18</td>
<td>1.58</td>
<td>54.03</td>
<td>10.25</td>
<td>0.70</td>
</tr>
<tr>
<td>15.54</td>
<td>17.00</td>
<td>4</td>
<td>0.37</td>
<td>1.06</td>
<td>1.73</td>
<td>49.02</td>
<td>11.75</td>
<td>0.70</td>
</tr>
<tr>
<td>18.04</td>
<td>17.00</td>
<td>5</td>
<td>0.35</td>
<td>1.22</td>
<td>2.29</td>
<td>44.01</td>
<td>13.25</td>
<td>0.70</td>
</tr>
<tr>
<td>20.54</td>
<td>17.00</td>
<td>5</td>
<td>0.34</td>
<td>1.12</td>
<td>2.42</td>
<td>40.08</td>
<td>14.75</td>
<td>0.70</td>
</tr>
<tr>
<td>23.04</td>
<td>17.00</td>
<td>5</td>
<td>0.34</td>
<td>1.01</td>
<td>2.67</td>
<td>40.08</td>
<td>16.25</td>
<td>0.70</td>
</tr>
</tbody>
</table>
### Project Information

**VAWS #:** VW14-02388  
**Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1  
**Location:** Barnstable County, MA  
**Project #:** CM/TAP/TI-002S(758)X  
**Design Engineer:** cms  
**Wall Name:** B=0.7H Minimum

### Static Internal Stability - Rupture Force Results

**Rec. #6 - H = 24.29 (ft) B = 17.00 (ft)**

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Static CDR Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>17.00</td>
<td>3</td>
<td>25.18</td>
<td>0.47</td>
<td>0.70</td>
<td>0.33</td>
<td>8.25</td>
<td>2.83</td>
</tr>
<tr>
<td>3.04</td>
<td>17.00</td>
<td>3</td>
<td>17.68</td>
<td>0.46</td>
<td>0.89</td>
<td>0.41</td>
<td>7.23</td>
<td>3.23</td>
</tr>
<tr>
<td>5.54</td>
<td>17.00</td>
<td>3</td>
<td>25.00</td>
<td>0.44</td>
<td>1.35</td>
<td>0.59</td>
<td>14.86</td>
<td>1.57</td>
</tr>
<tr>
<td>8.04</td>
<td>17.00</td>
<td>3</td>
<td>25.00</td>
<td>0.42</td>
<td>1.80</td>
<td>0.76</td>
<td>19.09</td>
<td>1.22</td>
</tr>
<tr>
<td>10.54</td>
<td>17.00</td>
<td>3</td>
<td>25.00</td>
<td>0.41</td>
<td>2.26</td>
<td>0.92</td>
<td>22.92</td>
<td>1.02</td>
</tr>
<tr>
<td>13.04</td>
<td>17.00</td>
<td>4</td>
<td>25.00</td>
<td>0.39</td>
<td>2.71</td>
<td>1.05</td>
<td>26.35</td>
<td>1.18</td>
</tr>
<tr>
<td>15.54</td>
<td>17.00</td>
<td>4</td>
<td>25.00</td>
<td>0.37</td>
<td>3.17</td>
<td>1.18</td>
<td>29.38</td>
<td>1.06</td>
</tr>
<tr>
<td>18.04</td>
<td>17.00</td>
<td>5</td>
<td>25.00</td>
<td>0.35</td>
<td>3.62</td>
<td>1.28</td>
<td>32.00</td>
<td>1.22</td>
</tr>
<tr>
<td>20.54</td>
<td>17.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>4.08</td>
<td>1.38</td>
<td>34.61</td>
<td>1.12</td>
</tr>
<tr>
<td>23.04</td>
<td>17.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>4.54</td>
<td>1.54</td>
<td>38.47</td>
<td>1.01</td>
</tr>
</tbody>
</table>

### Static Internal Stability - Pullout Force Results

**Rec. #6 - H = 24.29 (ft) B = 17.00 (ft)**

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horz. Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Le (ft)</th>
<th>F*</th>
<th>Pr (Kips)</th>
<th>Static CDR Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>17.00</td>
<td>3</td>
<td>25.18</td>
<td>0.47</td>
<td>0.27</td>
<td>0.13</td>
<td>4.28</td>
<td>9.71</td>
<td>76.15</td>
<td>5.62</td>
<td>1.31</td>
</tr>
<tr>
<td>3.04</td>
<td>17.00</td>
<td>3</td>
<td>17.68</td>
<td>0.46</td>
<td>0.41</td>
<td>0.19</td>
<td>4.49</td>
<td>9.71</td>
<td>74.08</td>
<td>8.29</td>
<td>1.85</td>
</tr>
<tr>
<td>5.54</td>
<td>17.00</td>
<td>3</td>
<td>25.00</td>
<td>0.44</td>
<td>0.75</td>
<td>0.33</td>
<td>11.14</td>
<td>9.71</td>
<td>69.07</td>
<td>14.10</td>
<td>1.27</td>
</tr>
<tr>
<td>8.04</td>
<td>17.00</td>
<td>3</td>
<td>25.00</td>
<td>0.42</td>
<td>1.08</td>
<td>0.46</td>
<td>15.52</td>
<td>9.71</td>
<td>64.05</td>
<td>18.98</td>
<td>1.22</td>
</tr>
<tr>
<td>10.54</td>
<td>17.00</td>
<td>3</td>
<td>25.00</td>
<td>0.41</td>
<td>1.42</td>
<td>0.58</td>
<td>19.50</td>
<td>9.71</td>
<td>59.04</td>
<td>22.94</td>
<td>1.18</td>
</tr>
<tr>
<td>13.04</td>
<td>17.00</td>
<td>4</td>
<td>25.00</td>
<td>0.39</td>
<td>1.76</td>
<td>0.66</td>
<td>23.07</td>
<td>10.25</td>
<td>54.03</td>
<td>36.55</td>
<td>1.58</td>
</tr>
<tr>
<td>15.54</td>
<td>17.00</td>
<td>4</td>
<td>25.00</td>
<td>0.37</td>
<td>2.10</td>
<td>0.78</td>
<td>26.25</td>
<td>11.75</td>
<td>49.02</td>
<td>45.30</td>
<td>1.73</td>
</tr>
<tr>
<td>18.04</td>
<td>17.00</td>
<td>5</td>
<td>25.00</td>
<td>0.35</td>
<td>2.43</td>
<td>0.86</td>
<td>29.02</td>
<td>13.25</td>
<td>44.01</td>
<td>66.56</td>
<td>2.29</td>
</tr>
<tr>
<td>20.54</td>
<td>17.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>2.77</td>
<td>0.94</td>
<td>31.74</td>
<td>14.75</td>
<td>40.08</td>
<td>76.81</td>
<td>2.42</td>
</tr>
<tr>
<td>23.04</td>
<td>17.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>3.11</td>
<td>1.06</td>
<td>35.61</td>
<td>16.25</td>
<td>40.08</td>
<td>94.93</td>
<td>2.67</td>
</tr>
</tbody>
</table>
SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

Project Information
VAWS #: VW14-02388
Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1
Location: Barnstable County, MA
Projec#: CM/TAP/TI-002S(758)X
Design Engineer: cms
Wall Name: B=0.7H Minimum

Rec. #7
Structure Parameters

<table>
<thead>
<tr>
<th>H max (ft)</th>
<th>B min (ft)</th>
<th>B Ext. (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>β (deg)</th>
<th>β ′ (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>22.857</td>
<td>16.000</td>
<td>16.000</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

External Active Earth Pressure Coefficient: 0.283
External Stability CDR Summary (Effective Stress)

<table>
<thead>
<tr>
<th></th>
<th>CDR Sliding</th>
<th>Limiting Eccentricity (ft)</th>
<th>CDR Overturning</th>
<th>Bearing Pressure (ksf)</th>
<th>Eccentricity (ft)</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>Se: ge - 1 - Min</td>
<td>2.42</td>
<td>2.04</td>
<td>3.92</td>
<td>4.54</td>
<td>1.79</td>
<td>0.500</td>
</tr>
<tr>
<td>Se: ge - 1 - Max</td>
<td>2.16</td>
<td>2.19</td>
<td>3.65</td>
<td>6.12</td>
<td>1.99</td>
<td>0.500</td>
</tr>
<tr>
<td>Se: ge - 1 - Critical</td>
<td>1.60</td>
<td>2.96</td>
<td>2.70</td>
<td>5.21</td>
<td>2.59</td>
<td>0.500</td>
</tr>
<tr>
<td>Service</td>
<td>2.46</td>
<td>1.91</td>
<td>4.18</td>
<td>4.28</td>
<td>1.77</td>
<td>0.500</td>
</tr>
</tbody>
</table>

Internal Active Earth Pressure Coefficient: 0.283
Internal Stability CDR Summary

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Internal Earth Coeff.</th>
<th>Rupture CDR</th>
<th>Pullout CDR</th>
<th>F*</th>
<th>Le (ft)</th>
<th>H/B Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>16.00</td>
<td>3</td>
<td>0.47</td>
<td>2.33</td>
<td>1.02</td>
<td>76.15</td>
<td>9.14</td>
<td>0.70</td>
</tr>
<tr>
<td>4.11</td>
<td>16.00</td>
<td>3</td>
<td>0.45</td>
<td>2.07</td>
<td>1.32</td>
<td>71.93</td>
<td>9.14</td>
<td>0.70</td>
</tr>
<tr>
<td>6.61</td>
<td>16.00</td>
<td>3</td>
<td>0.43</td>
<td>1.40</td>
<td>1.17</td>
<td>66.92</td>
<td>9.14</td>
<td>0.70</td>
</tr>
<tr>
<td>9.11</td>
<td>16.00</td>
<td>3</td>
<td>0.42</td>
<td>1.12</td>
<td>1.13</td>
<td>61.91</td>
<td>9.14</td>
<td>0.70</td>
</tr>
<tr>
<td>11.61</td>
<td>16.00</td>
<td>4</td>
<td>0.40</td>
<td>1.27</td>
<td>1.47</td>
<td>56.90</td>
<td>9.25</td>
<td>0.70</td>
</tr>
<tr>
<td>14.11</td>
<td>16.00</td>
<td>4</td>
<td>0.38</td>
<td>1.12</td>
<td>1.63</td>
<td>51.89</td>
<td>10.75</td>
<td>0.70</td>
</tr>
<tr>
<td>16.61</td>
<td>16.00</td>
<td>4</td>
<td>0.36</td>
<td>1.02</td>
<td>1.76</td>
<td>48.88</td>
<td>12.25</td>
<td>0.70</td>
</tr>
<tr>
<td>19.11</td>
<td>16.00</td>
<td>5</td>
<td>0.35</td>
<td>1.18</td>
<td>2.31</td>
<td>41.86</td>
<td>13.75</td>
<td>0.70</td>
</tr>
<tr>
<td>21.61</td>
<td>16.00</td>
<td>5</td>
<td>0.34</td>
<td>1.07</td>
<td>2.50</td>
<td>40.08</td>
<td>15.25</td>
<td>0.70</td>
</tr>
</tbody>
</table>
## Static Internal Stability - Rupture Force Results

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Static CDR Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>16.00</td>
<td>3</td>
<td>30.54</td>
<td>0.47</td>
<td>0.70</td>
<td>0.33</td>
<td>10.00</td>
<td>2.33</td>
</tr>
<tr>
<td>4.11</td>
<td>16.00</td>
<td>3</td>
<td>23.04</td>
<td>0.45</td>
<td>1.09</td>
<td>0.49</td>
<td>11.30</td>
<td>2.07</td>
</tr>
<tr>
<td>6.61</td>
<td>16.00</td>
<td>3</td>
<td>25.00</td>
<td>0.43</td>
<td>1.54</td>
<td>0.67</td>
<td>16.72</td>
<td>1.40</td>
</tr>
<tr>
<td>9.11</td>
<td>16.00</td>
<td>3</td>
<td>25.00</td>
<td>0.42</td>
<td>2.00</td>
<td>0.83</td>
<td>20.78</td>
<td>1.12</td>
</tr>
<tr>
<td>11.61</td>
<td>16.00</td>
<td>4</td>
<td>25.00</td>
<td>0.40</td>
<td>2.45</td>
<td>0.98</td>
<td>24.44</td>
<td>1.27</td>
</tr>
<tr>
<td>14.11</td>
<td>16.00</td>
<td>4</td>
<td>25.00</td>
<td>0.38</td>
<td>2.91</td>
<td>1.11</td>
<td>27.70</td>
<td>1.12</td>
</tr>
<tr>
<td>16.61</td>
<td>16.00</td>
<td>4</td>
<td>25.00</td>
<td>0.36</td>
<td>3.36</td>
<td>1.22</td>
<td>30.55</td>
<td>1.02</td>
</tr>
<tr>
<td>19.11</td>
<td>16.00</td>
<td>5</td>
<td>25.00</td>
<td>0.35</td>
<td>3.82</td>
<td>1.32</td>
<td>33.00</td>
<td>1.18</td>
</tr>
<tr>
<td>21.61</td>
<td>16.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>4.28</td>
<td>1.45</td>
<td>36.26</td>
<td>1.07</td>
</tr>
</tbody>
</table>

## Static Internal Stability - Pullout Force Results

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Le (ft)</th>
<th>F*</th>
<th>Pr (Kips)</th>
<th>Static CDR Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>16.00</td>
<td>3</td>
<td>30.54</td>
<td>0.47</td>
<td>0.27</td>
<td>0.13</td>
<td>5.19</td>
<td>9.14</td>
<td>76.15</td>
<td>5.29</td>
<td>1.02</td>
</tr>
<tr>
<td>4.11</td>
<td>16.00</td>
<td>3</td>
<td>23.04</td>
<td>0.45</td>
<td>0.25</td>
<td>0.25</td>
<td>7.79</td>
<td>9.14</td>
<td>71.93</td>
<td>10.25</td>
<td>1.32</td>
</tr>
<tr>
<td>6.61</td>
<td>16.00</td>
<td>3</td>
<td>25.00</td>
<td>0.43</td>
<td>0.39</td>
<td>0.39</td>
<td>13.06</td>
<td>9.14</td>
<td>66.92</td>
<td>15.35</td>
<td>1.17</td>
</tr>
<tr>
<td>9.11</td>
<td>16.00</td>
<td>3</td>
<td>25.00</td>
<td>0.42</td>
<td>1.23</td>
<td>0.51</td>
<td>17.27</td>
<td>9.14</td>
<td>61.91</td>
<td>19.57</td>
<td>1.13</td>
</tr>
<tr>
<td>11.61</td>
<td>16.00</td>
<td>4</td>
<td>25.00</td>
<td>0.40</td>
<td>1.57</td>
<td>0.62</td>
<td>21.08</td>
<td>9.25</td>
<td>56.90</td>
<td>30.92</td>
<td>1.47</td>
</tr>
<tr>
<td>14.11</td>
<td>16.00</td>
<td>4</td>
<td>25.00</td>
<td>0.38</td>
<td>1.90</td>
<td>0.73</td>
<td>24.48</td>
<td>10.75</td>
<td>51.89</td>
<td>39.83</td>
<td>1.63</td>
</tr>
<tr>
<td>16.61</td>
<td>16.00</td>
<td>4</td>
<td>25.00</td>
<td>0.36</td>
<td>2.24</td>
<td>0.81</td>
<td>27.48</td>
<td>12.25</td>
<td>46.88</td>
<td>48.28</td>
<td>1.76</td>
</tr>
<tr>
<td>19.11</td>
<td>16.00</td>
<td>5</td>
<td>25.00</td>
<td>0.35</td>
<td>2.58</td>
<td>0.89</td>
<td>30.08</td>
<td>13.75</td>
<td>41.86</td>
<td>69.60</td>
<td>2.31</td>
</tr>
<tr>
<td>21.61</td>
<td>16.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>2.92</td>
<td>0.99</td>
<td>33.40</td>
<td>15.25</td>
<td>40.08</td>
<td>83.56</td>
<td>2.50</td>
</tr>
</tbody>
</table>
SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

Project Information
VAWS #: VW14-02388
Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1
Location: Barnstable County, MA
Project #: CM/TAP/TI-002S(758)X
Design Engineer: cms
Wall Name: B=0.7H Minimum

Rec. #8
Structure Parameters

<table>
<thead>
<tr>
<th>H max</th>
<th>B min</th>
<th>B Ext.</th>
<th>S</th>
<th>Xs</th>
<th>β</th>
<th>β l</th>
</tr>
</thead>
<tbody>
<tr>
<td>ft</td>
<td>ft</td>
<td>ft</td>
<td>ft</td>
<td>ft</td>
<td>deg</td>
<td>deg</td>
</tr>
<tr>
<td>21.429</td>
<td>15.000</td>
<td>15.000</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

External Active Earth Pressure Coefficient: 0.283
External Stability CDR Summary (Effective Stress)

<table>
<thead>
<tr>
<th></th>
<th>CDR Sliding</th>
<th>Limiting Eccentricity</th>
<th>CDR Overturning</th>
<th>Bearing Pressure</th>
<th>Eccentricity</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ft</td>
<td>ft</td>
<td></td>
<td>ft</td>
<td>ft</td>
<td></td>
</tr>
<tr>
<td>S: re: g-1 - Min</td>
<td>2.38</td>
<td>1.95</td>
<td>3.84</td>
<td>4.30</td>
<td>1.70</td>
<td>0.500</td>
</tr>
<tr>
<td>S: re: g-1 - Max</td>
<td>2.14</td>
<td>2.09</td>
<td>3.59</td>
<td>5.79</td>
<td>1.88</td>
<td>0.500</td>
</tr>
<tr>
<td>S: re: g-1 - Critical</td>
<td>1.59</td>
<td>2.82</td>
<td>2.66</td>
<td>4.94</td>
<td>2.45</td>
<td>0.500</td>
</tr>
<tr>
<td>Service</td>
<td>2.44</td>
<td>1.82</td>
<td>4.13</td>
<td>4.04</td>
<td>1.67</td>
<td>0.500</td>
</tr>
</tbody>
</table>

Internal Active Earth Pressure Coefficient: 0.283
Internal Stability CDR Summary

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Internal Earth Coeff.</th>
<th>Rupture CDR</th>
<th>Pullout CDR</th>
<th>F*</th>
<th>Le (ft)</th>
<th>H/B Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.68</td>
<td>15.00</td>
<td>5</td>
<td>0.46</td>
<td>2.60</td>
<td>1.23</td>
<td>74.79</td>
<td>8.57</td>
<td>0.70</td>
</tr>
<tr>
<td>5.18</td>
<td>15.00</td>
<td>3</td>
<td>0.44</td>
<td>1.64</td>
<td>1.12</td>
<td>69.78</td>
<td>8.57</td>
<td>0.70</td>
</tr>
<tr>
<td>7.68</td>
<td>15.00</td>
<td>3</td>
<td>0.43</td>
<td>1.26</td>
<td>1.09</td>
<td>64.77</td>
<td>8.57</td>
<td>0.70</td>
</tr>
<tr>
<td>10.18</td>
<td>15.00</td>
<td>3</td>
<td>0.41</td>
<td>1.04</td>
<td>1.04</td>
<td>59.76</td>
<td>8.57</td>
<td>0.70</td>
</tr>
<tr>
<td>12.68</td>
<td>15.00</td>
<td>4</td>
<td>0.39</td>
<td>1.20</td>
<td>1.52</td>
<td>54.75</td>
<td>9.75</td>
<td>0.70</td>
</tr>
<tr>
<td>15.18</td>
<td>15.00</td>
<td>4</td>
<td>0.37</td>
<td>1.07</td>
<td>1.67</td>
<td>49.74</td>
<td>11.25</td>
<td>0.70</td>
</tr>
<tr>
<td>17.68</td>
<td>15.00</td>
<td>5</td>
<td>0.36</td>
<td>1.23</td>
<td>2.23</td>
<td>44.73</td>
<td>12.75</td>
<td>0.70</td>
</tr>
<tr>
<td>20.18</td>
<td>15.00</td>
<td>5</td>
<td>0.34</td>
<td>1.14</td>
<td>2.34</td>
<td>40.08</td>
<td>14.25</td>
<td>0.70</td>
</tr>
</tbody>
</table>
### Project Information

**VAWS #:** VW14-02388  
**Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1  
**Location:** Barnstable County, MA  
**Project #:** CM/TAP/TI-002S(758)X  
**Design Engineer:** cms  
**Wall Name:** B=0.7H Maximum

#### Static Internal Stability - Rupture Force Results

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Static CDR Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.68</td>
<td>15.00</td>
<td>5</td>
<td>39.29</td>
<td>0.46</td>
<td>0.83</td>
<td>0.38</td>
<td>14.98</td>
<td>2.60</td>
</tr>
<tr>
<td>5.18</td>
<td>15.00</td>
<td>3</td>
<td>25.00</td>
<td>0.44</td>
<td>1.28</td>
<td>0.57</td>
<td>14.22</td>
<td>1.64</td>
</tr>
<tr>
<td>7.68</td>
<td>15.00</td>
<td>3</td>
<td>25.00</td>
<td>0.43</td>
<td>1.74</td>
<td>0.74</td>
<td>18.51</td>
<td>1.26</td>
</tr>
<tr>
<td>10.18</td>
<td>15.00</td>
<td>3</td>
<td>25.00</td>
<td>0.41</td>
<td>2.19</td>
<td>0.90</td>
<td>22.40</td>
<td>1.04</td>
</tr>
<tr>
<td>12.68</td>
<td>15.00</td>
<td>4</td>
<td>25.00</td>
<td>0.39</td>
<td>2.65</td>
<td>1.04</td>
<td>25.89</td>
<td>1.20</td>
</tr>
<tr>
<td>15.18</td>
<td>15.00</td>
<td>4</td>
<td>25.00</td>
<td>0.37</td>
<td>3.10</td>
<td>1.16</td>
<td>28.97</td>
<td>1.07</td>
</tr>
<tr>
<td>17.68</td>
<td>15.00</td>
<td>5</td>
<td>25.00</td>
<td>0.36</td>
<td>3.56</td>
<td>1.27</td>
<td>31.65</td>
<td>1.23</td>
</tr>
<tr>
<td>20.18</td>
<td>15.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>4.02</td>
<td>1.36</td>
<td>34.05</td>
<td>1.14</td>
</tr>
</tbody>
</table>

#### Static Internal Stability - Pullout Force Results

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horz. Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Le (ft)</th>
<th>F*</th>
<th>Pr (Kips)</th>
<th>Static CDR Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.68</td>
<td>15.00</td>
<td>5</td>
<td>39.29</td>
<td>0.46</td>
<td>0.17</td>
<td>8.86</td>
<td>8.57</td>
<td>74.79</td>
<td>10.87</td>
<td>1.23</td>
<td></td>
</tr>
<tr>
<td>5.18</td>
<td>15.00</td>
<td>3</td>
<td>25.00</td>
<td>0.44</td>
<td>0.70</td>
<td>10.48</td>
<td>8.57</td>
<td>69.78</td>
<td>11.76</td>
<td>1.12</td>
<td></td>
</tr>
<tr>
<td>7.68</td>
<td>15.00</td>
<td>3</td>
<td>25.00</td>
<td>0.43</td>
<td>1.04</td>
<td>14.92</td>
<td>8.57</td>
<td>64.77</td>
<td>16.19</td>
<td>1.09</td>
<td></td>
</tr>
<tr>
<td>10.18</td>
<td>15.00</td>
<td>3</td>
<td>25.00</td>
<td>0.41</td>
<td>1.37</td>
<td>18.95</td>
<td>8.57</td>
<td>59.76</td>
<td>19.80</td>
<td>1.04</td>
<td></td>
</tr>
<tr>
<td>12.68</td>
<td>15.00</td>
<td>4</td>
<td>25.00</td>
<td>0.39</td>
<td>1.71</td>
<td>22.59</td>
<td>9.75</td>
<td>54.75</td>
<td>34.26</td>
<td>1.52</td>
<td></td>
</tr>
<tr>
<td>15.18</td>
<td>15.00</td>
<td>4</td>
<td>25.00</td>
<td>0.37</td>
<td>2.05</td>
<td>25.82</td>
<td>11.25</td>
<td>49.74</td>
<td>43.00</td>
<td>1.67</td>
<td></td>
</tr>
<tr>
<td>17.68</td>
<td>15.00</td>
<td>5</td>
<td>25.00</td>
<td>0.36</td>
<td>2.39</td>
<td>28.65</td>
<td>12.75</td>
<td>44.73</td>
<td>63.80</td>
<td>2.23</td>
<td></td>
</tr>
<tr>
<td>20.18</td>
<td>15.00</td>
<td>5</td>
<td>25.00</td>
<td>0.34</td>
<td>2.72</td>
<td>31.19</td>
<td>14.25</td>
<td>40.08</td>
<td>72.92</td>
<td>2.34</td>
<td></td>
</tr>
</tbody>
</table>
### Project Information

VAWS #: VW14-02388  
Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1  
Location: Barnstable County, MA  
Project #: CM/TAP/TI-002S(758)X  
Design Engineer: cms  
Wall Name: B=0.7H Minimum

### Rec. #9  
**Structure Parameters**

<table>
<thead>
<tr>
<th>H (ft)</th>
<th>B min (ft)</th>
<th>B Ext. (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>$\beta$ (deg)</th>
<th>$\beta_t$ (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>18.667</td>
<td>14.000</td>
<td>14.000</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

### External Active Earth Pressure Coefficient: 0.283

**External Stability CDR Summary (Effective Stress)**

<table>
<thead>
<tr>
<th>CDR Sliding</th>
<th>Limiting Eccentricity (ft)</th>
<th>CDR Overturning</th>
<th>Bearing Pressure (ksf)</th>
<th>Eccentricity</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.46</td>
<td>1.67</td>
<td>4.20</td>
<td>3.71</td>
<td>1.42</td>
<td>0.500</td>
</tr>
<tr>
<td>2.24</td>
<td>1.76</td>
<td>3.99</td>
<td>4.94</td>
<td>1.56</td>
<td>0.500</td>
</tr>
<tr>
<td>1.66</td>
<td>2.37</td>
<td>2.95</td>
<td>4.16</td>
<td>2.02</td>
<td>0.500</td>
</tr>
<tr>
<td>2.56</td>
<td>1.52</td>
<td>4.60</td>
<td>3.45</td>
<td>1.38</td>
<td>0.500</td>
</tr>
</tbody>
</table>

### Internal Active Earth Pressure Coefficient: 0.283

**Internal Stability CDR Summary**

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Internal Earth Coeff.</th>
<th>Rupture CDR</th>
<th>Pullout CDR</th>
<th>F*</th>
<th>Le (ft)</th>
<th>H/B Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.42</td>
<td>14.00</td>
<td>4</td>
<td>0.46</td>
<td>2.35</td>
<td>1.03</td>
<td>75.32</td>
<td>8.40</td>
<td>0.75</td>
</tr>
<tr>
<td>4.92</td>
<td>14.00</td>
<td>3</td>
<td>0.45</td>
<td>1.70</td>
<td>1.10</td>
<td>70.31</td>
<td>8.40</td>
<td>0.75</td>
</tr>
<tr>
<td>7.42</td>
<td>14.00</td>
<td>3</td>
<td>0.43</td>
<td>1.29</td>
<td>1.07</td>
<td>65.29</td>
<td>8.40</td>
<td>0.75</td>
</tr>
<tr>
<td>9.92</td>
<td>14.00</td>
<td>3</td>
<td>0.41</td>
<td>1.06</td>
<td>1.07</td>
<td>60.28</td>
<td>8.75</td>
<td>0.75</td>
</tr>
<tr>
<td>12.42</td>
<td>14.00</td>
<td>4</td>
<td>0.39</td>
<td>1.22</td>
<td>1.60</td>
<td>55.27</td>
<td>10.25</td>
<td>0.75</td>
</tr>
<tr>
<td>14.92</td>
<td>14.00</td>
<td>4</td>
<td>0.38</td>
<td>1.09</td>
<td>1.75</td>
<td>50.26</td>
<td>11.75</td>
<td>0.75</td>
</tr>
<tr>
<td>17.42</td>
<td>14.00</td>
<td>5</td>
<td>0.36</td>
<td>1.24</td>
<td>2.33</td>
<td>45.25</td>
<td>13.25</td>
<td>0.75</td>
</tr>
</tbody>
</table>
### MSE - Pro

Mechanically Stabilized Earth Retaining Structures

#### SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

**Project Information**

- **VAWS #:** VW14-02388
- **Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1
- **Location:** Barnstable County, MA
- **Project #:** CM/TAP/TI-002S(758)X
- **Design Engineer:** cms
- **Wall Name:** B=0.7H Minimum

**Static Internal Stability - Rupture Force Results**

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Static CDR Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.42</td>
<td>14.00</td>
<td>4</td>
<td>36.67</td>
<td>0.46</td>
<td>0.78</td>
<td>0.36</td>
<td>13.22</td>
<td>2.35</td>
</tr>
<tr>
<td>4.92</td>
<td>14.00</td>
<td>3</td>
<td>25.00</td>
<td>0.45</td>
<td>1.23</td>
<td>0.55</td>
<td>13.75</td>
<td>1.70</td>
</tr>
<tr>
<td>7.42</td>
<td>14.00</td>
<td>3</td>
<td>25.00</td>
<td>0.43</td>
<td>1.69</td>
<td>0.72</td>
<td>18.08</td>
<td>1.29</td>
</tr>
<tr>
<td>9.92</td>
<td>14.00</td>
<td>3</td>
<td>25.00</td>
<td>0.41</td>
<td>2.14</td>
<td>0.88</td>
<td>22.01</td>
<td>1.06</td>
</tr>
<tr>
<td>12.42</td>
<td>14.00</td>
<td>4</td>
<td>25.00</td>
<td>0.39</td>
<td>2.60</td>
<td>1.02</td>
<td>25.54</td>
<td>1.22</td>
</tr>
<tr>
<td>14.92</td>
<td>14.00</td>
<td>4</td>
<td>25.00</td>
<td>0.38</td>
<td>3.06</td>
<td>1.15</td>
<td>28.67</td>
<td>1.09</td>
</tr>
<tr>
<td>17.42</td>
<td>14.00</td>
<td>5</td>
<td>25.00</td>
<td>0.36</td>
<td>3.51</td>
<td>1.26</td>
<td>31.39</td>
<td>1.24</td>
</tr>
</tbody>
</table>

**Static Internal Stability - Pullout Force Results**

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horiz. Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Le (ft)</th>
<th>F*</th>
<th>Pr (Kips)</th>
<th>Static CDR Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.42</td>
<td>14.00</td>
<td>4</td>
<td>36.67</td>
<td>0.46</td>
<td>0.33</td>
<td>0.15</td>
<td>7.49</td>
<td>8.40</td>
<td>75.32</td>
<td>7.74</td>
<td>1.03</td>
</tr>
<tr>
<td>4.92</td>
<td>14.00</td>
<td>3</td>
<td>25.00</td>
<td>0.45</td>
<td>0.66</td>
<td>0.30</td>
<td>9.99</td>
<td>8.40</td>
<td>70.31</td>
<td>11.02</td>
<td>1.10</td>
</tr>
<tr>
<td>7.42</td>
<td>14.00</td>
<td>3</td>
<td>25.00</td>
<td>0.43</td>
<td>1.00</td>
<td>0.43</td>
<td>14.47</td>
<td>8.40</td>
<td>65.29</td>
<td>15.45</td>
<td>1.07</td>
</tr>
<tr>
<td>9.92</td>
<td>14.00</td>
<td>3</td>
<td>25.00</td>
<td>0.41</td>
<td>1.34</td>
<td>0.55</td>
<td>18.55</td>
<td>8.75</td>
<td>60.28</td>
<td>19.86</td>
<td>1.07</td>
</tr>
<tr>
<td>12.42</td>
<td>14.00</td>
<td>4</td>
<td>25.00</td>
<td>0.39</td>
<td>1.68</td>
<td>0.66</td>
<td>22.23</td>
<td>10.25</td>
<td>55.27</td>
<td>35.61</td>
<td>1.60</td>
</tr>
<tr>
<td>14.92</td>
<td>14.00</td>
<td>4</td>
<td>25.00</td>
<td>0.38</td>
<td>2.01</td>
<td>0.76</td>
<td>25.50</td>
<td>11.75</td>
<td>50.26</td>
<td>44.60</td>
<td>1.75</td>
</tr>
<tr>
<td>17.42</td>
<td>14.00</td>
<td>5</td>
<td>25.00</td>
<td>0.36</td>
<td>2.35</td>
<td>0.84</td>
<td>28.37</td>
<td>13.25</td>
<td>45.25</td>
<td>66.08</td>
<td>2.33</td>
</tr>
</tbody>
</table>
SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

Project Information
VAWS #: VW14-02388
Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1
Location: Barnstable County, MA
Project #: CM/TAP/TI-002S(758)X
Design Engineer: cms
Wall Name: B=0.7H Minimum

Rec. #10
Structure Parameters

<table>
<thead>
<tr>
<th>H max (ft)</th>
<th>B min (ft)</th>
<th>B Ext. (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>β (deg)</th>
<th>β I (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>16.250</td>
<td>13.000</td>
<td>13.000</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

External Active Earth Pressure Coefficient: 0.283
External Stability CDR Summary (Effective Stress)

<table>
<thead>
<tr>
<th></th>
<th>CDR Sliding</th>
<th>Limiting Eccentricity (ft)</th>
<th>CDR Overturning</th>
<th>Bearing Pressure (ksf)</th>
<th>Eccentricity (ft)</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>Service</td>
<td>2.66</td>
<td>1.28</td>
<td>5.06</td>
<td>2.97</td>
<td>1.15</td>
<td>0.500</td>
</tr>
</tbody>
</table>

Internal Active Earth Pressure Coefficient: 0.283
Internal Stability CDR Summary

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Internal Earth Coeff.</th>
<th>Rupture CDR</th>
<th>Pullout CDR</th>
<th>F*</th>
<th>Le (ft)</th>
<th>H/B Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.50</td>
<td>13.00</td>
<td>5</td>
<td>0.46</td>
<td>2.83</td>
<td>1.22</td>
<td>75.15</td>
<td>8.13</td>
<td>0.80</td>
</tr>
<tr>
<td>5.00</td>
<td>13.00</td>
<td>3</td>
<td>0.45</td>
<td>1.68</td>
<td>1.07</td>
<td>70.14</td>
<td>8.13</td>
<td>0.80</td>
</tr>
<tr>
<td>7.50</td>
<td>13.00</td>
<td>3</td>
<td>0.43</td>
<td>1.28</td>
<td>1.03</td>
<td>65.13</td>
<td>8.13</td>
<td>0.80</td>
</tr>
<tr>
<td>10.00</td>
<td>13.00</td>
<td>3</td>
<td>0.41</td>
<td>1.05</td>
<td>1.13</td>
<td>60.12</td>
<td>9.25</td>
<td>0.80</td>
</tr>
<tr>
<td>12.50</td>
<td>13.00</td>
<td>4</td>
<td>0.39</td>
<td>1.21</td>
<td>1.68</td>
<td>55.11</td>
<td>10.75</td>
<td>0.80</td>
</tr>
<tr>
<td>15.00</td>
<td>13.00</td>
<td>4</td>
<td>0.37</td>
<td>1.08</td>
<td>1.82</td>
<td>50.10</td>
<td>12.25</td>
<td>0.80</td>
</tr>
</tbody>
</table>
### MSE - Pro
Mechanically Stabilized Earth Retaining Structures

#### SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

#### Project Information
- **VAWS #:** VW14-02388
- **Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1
- **Location:** Barnstable County, MA
- **Project #:** CM/TAP/TI-002S(758)X
- **Design Engineer:** cm
- **Wall Name:** B=0.7H Minimum

#### Static Internal Stability - Rupture Force Results
**Rec. #10 - H = 16.25 (ft) B = 13.00 (ft)**

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Static CDR Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.50</td>
<td>13.00</td>
<td>5</td>
<td>37.50</td>
<td>0.46</td>
<td>0.79</td>
<td>0.37</td>
<td>13.77</td>
<td>2.83</td>
</tr>
<tr>
<td>5.00</td>
<td>13.00</td>
<td>3</td>
<td>25.00</td>
<td>0.45</td>
<td>1.25</td>
<td>0.56</td>
<td>13.90</td>
<td>1.68</td>
</tr>
<tr>
<td>7.50</td>
<td>13.00</td>
<td>3</td>
<td>25.00</td>
<td>0.43</td>
<td>1.70</td>
<td>0.73</td>
<td>18.22</td>
<td>1.28</td>
</tr>
<tr>
<td>10.00</td>
<td>13.00</td>
<td>3</td>
<td>25.00</td>
<td>0.41</td>
<td>2.16</td>
<td>0.89</td>
<td>22.14</td>
<td>1.05</td>
</tr>
<tr>
<td>12.50</td>
<td>13.00</td>
<td>4</td>
<td>25.00</td>
<td>0.39</td>
<td>2.62</td>
<td>1.03</td>
<td>25.65</td>
<td>1.21</td>
</tr>
<tr>
<td>15.00</td>
<td>13.00</td>
<td>4</td>
<td>25.00</td>
<td>0.37</td>
<td>3.07</td>
<td>1.15</td>
<td>28.76</td>
<td>1.08</td>
</tr>
</tbody>
</table>

#### Static Internal Stability - Pullout Force Results
**Rec. #10 - H = 16.25 (ft) B = 13.00 (ft)**

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horz. Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Le (ft)</th>
<th>F*</th>
<th>Pr (Kips)</th>
<th>Static CDR Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.50</td>
<td>13.00</td>
<td>5</td>
<td>37.50</td>
<td>0.46</td>
<td>0.34</td>
<td>0.16</td>
<td>7.91</td>
<td>8.13</td>
<td>75.15</td>
<td>9.66</td>
<td>1.22</td>
</tr>
<tr>
<td>5.00</td>
<td>13.00</td>
<td>3</td>
<td>25.00</td>
<td>0.45</td>
<td>0.45</td>
<td>0.30</td>
<td>10.14</td>
<td>8.13</td>
<td>70.14</td>
<td>10.82</td>
<td>1.07</td>
</tr>
<tr>
<td>7.50</td>
<td>13.00</td>
<td>3</td>
<td>25.00</td>
<td>0.43</td>
<td>1.01</td>
<td>0.43</td>
<td>14.61</td>
<td>8.13</td>
<td>65.13</td>
<td>15.07</td>
<td>1.03</td>
</tr>
<tr>
<td>10.00</td>
<td>13.00</td>
<td>3</td>
<td>25.00</td>
<td>0.41</td>
<td>1.35</td>
<td>0.55</td>
<td>18.68</td>
<td>9.25</td>
<td>60.12</td>
<td>21.11</td>
<td>1.13</td>
</tr>
<tr>
<td>12.50</td>
<td>13.00</td>
<td>4</td>
<td>25.00</td>
<td>0.39</td>
<td>1.69</td>
<td>0.66</td>
<td>22.34</td>
<td>10.75</td>
<td>55.11</td>
<td>37.49</td>
<td>1.68</td>
</tr>
<tr>
<td>15.00</td>
<td>13.00</td>
<td>4</td>
<td>25.00</td>
<td>0.37</td>
<td>2.03</td>
<td>0.76</td>
<td>25.60</td>
<td>12.25</td>
<td>50.10</td>
<td>46.60</td>
<td>1.82</td>
</tr>
</tbody>
</table>
### Project Information

**VAWS #:** VW14-02388  
**Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1  
**Location:** Barnstable County, MA  
**Project #:** CM/TAP/TI-002S(758)X  
**Design Engineer:** cms  
**Wall Name:** B=0.7H Minimum

### Structure Parameters

<table>
<thead>
<tr>
<th>H max (ft)</th>
<th>B min (ft)</th>
<th>B Ext. (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>β (deg)</th>
<th>β t (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>14.118</td>
<td>12.000</td>
<td>12.000</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

### External Active Earth Pressure Coefficient: 0.283

### External Stability CDR Summary (Effective Stress)

<table>
<thead>
<tr>
<th></th>
<th>CDR Sliding</th>
<th>Limiting Eccentricity (ft)</th>
<th>CDR Overturning</th>
<th>Bearing Pressure (ksf)</th>
<th>Eccentricity (ft)</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reg. - Min</td>
<td>2.55</td>
<td>1.24</td>
<td>4.83</td>
<td>2.82</td>
<td>1.01</td>
<td>0.500</td>
</tr>
<tr>
<td>Reg. - Max</td>
<td>2.39</td>
<td>1.27</td>
<td>4.73</td>
<td>3.67</td>
<td>1.08</td>
<td>0.500</td>
</tr>
<tr>
<td>Reg. - Critical</td>
<td>1.77</td>
<td>1.71</td>
<td>3.50</td>
<td>3.05</td>
<td>1.39</td>
<td>0.500</td>
</tr>
<tr>
<td>Service</td>
<td>2.75</td>
<td>1.09</td>
<td>5.50</td>
<td>2.57</td>
<td>0.96</td>
<td>0.500</td>
</tr>
</tbody>
</table>

### Internal Active Earth Pressure Coefficient: 0.283

### Internal Stability CDR Summary

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Internal Earth Coeff.</th>
<th>Rupture CDR</th>
<th>Pullout CDR</th>
<th>F*</th>
<th>Le (ft)</th>
<th>H/B Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.87</td>
<td>12.00</td>
<td>5</td>
<td>0.46</td>
<td>2.39</td>
<td>1.06</td>
<td>74.41</td>
<td>7.76</td>
<td>0.85</td>
</tr>
<tr>
<td>5.37</td>
<td>12.00</td>
<td>3</td>
<td>0.44</td>
<td>1.60</td>
<td>1.01</td>
<td>69.40</td>
<td>7.76</td>
<td>0.85</td>
</tr>
<tr>
<td>7.87</td>
<td>12.00</td>
<td>3</td>
<td>0.43</td>
<td>1.24</td>
<td>1.04</td>
<td>64.39</td>
<td>8.25</td>
<td>0.85</td>
</tr>
<tr>
<td>10.37</td>
<td>12.00</td>
<td>3</td>
<td>0.41</td>
<td>1.03</td>
<td>1.18</td>
<td>59.38</td>
<td>9.75</td>
<td>0.85</td>
</tr>
<tr>
<td>12.87</td>
<td>12.00</td>
<td>4</td>
<td>0.39</td>
<td>1.19</td>
<td>1.74</td>
<td>54.37</td>
<td>11.25</td>
<td>0.85</td>
</tr>
</tbody>
</table>
## Project Information

**VAWS #:** VW14-02388  
**Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1  
**Location:** Barnstable County, MA  
**Design Engineer:** cms  
**Wall Name:** B=0.7H Minimum

### Static Internal Stability - Rupture Force Results

Rec. #11 - H = 14.12 (ft) B = 12.00 (ft)

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Static CDR Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.87</td>
<td>12.00</td>
<td>5</td>
<td>41.18</td>
<td>0.46</td>
<td>0.86</td>
<td>0.40</td>
<td>16.31</td>
<td>2.39</td>
</tr>
<tr>
<td>5.37</td>
<td>12.00</td>
<td>3</td>
<td>25.00</td>
<td>0.44</td>
<td>1.32</td>
<td>0.58</td>
<td>14.56</td>
<td>1.60</td>
</tr>
<tr>
<td>7.87</td>
<td>12.00</td>
<td>3</td>
<td>25.00</td>
<td>0.43</td>
<td>1.77</td>
<td>0.75</td>
<td>18.82</td>
<td>1.24</td>
</tr>
<tr>
<td>10.37</td>
<td>12.00</td>
<td>3</td>
<td>25.00</td>
<td>0.41</td>
<td>2.23</td>
<td>0.91</td>
<td>22.68</td>
<td>1.03</td>
</tr>
<tr>
<td>12.87</td>
<td>12.00</td>
<td>4</td>
<td>25.00</td>
<td>0.39</td>
<td>2.69</td>
<td>1.05</td>
<td>26.13</td>
<td>1.19</td>
</tr>
</tbody>
</table>

### Static Internal Stability - Pullout Force Results

Rec. #11 - H = 14.12 (ft) B = 12.00 (ft)

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Le (ft)</th>
<th>F*</th>
<th>Pr (Kips)</th>
<th>Static CDR Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.87</td>
<td>12.00</td>
<td>5</td>
<td>41.18</td>
<td>0.46</td>
<td>0.39</td>
<td>0.18</td>
<td>9.91</td>
<td>7.76</td>
<td>74.41</td>
<td>10.49</td>
<td>1.06</td>
</tr>
<tr>
<td>5.37</td>
<td>12.00</td>
<td>3</td>
<td>25.00</td>
<td>0.44</td>
<td>0.72</td>
<td>0.32</td>
<td>10.83</td>
<td>7.76</td>
<td>69.40</td>
<td>10.98</td>
<td>1.01</td>
</tr>
<tr>
<td>7.87</td>
<td>12.00</td>
<td>3</td>
<td>25.00</td>
<td>0.43</td>
<td>1.06</td>
<td>0.45</td>
<td>15.24</td>
<td>8.25</td>
<td>64.39</td>
<td>15.87</td>
<td>1.04</td>
</tr>
<tr>
<td>10.37</td>
<td>12.00</td>
<td>3</td>
<td>25.00</td>
<td>0.41</td>
<td>1.40</td>
<td>0.57</td>
<td>19.24</td>
<td>9.75</td>
<td>59.38</td>
<td>22.79</td>
<td>1.18</td>
</tr>
<tr>
<td>12.87</td>
<td>12.00</td>
<td>4</td>
<td>25.00</td>
<td>0.39</td>
<td>1.74</td>
<td>0.68</td>
<td>22.85</td>
<td>11.25</td>
<td>54.37</td>
<td>39.84</td>
<td>1.74</td>
</tr>
</tbody>
</table>
Project Information

VAWS #: VW14-02388
Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1
Location: Barnstable County, MA
Project #: CM/TAP/T1-002S(758)X
Design Engineer: cm
Wall Name: B=0.7H Minimum

Rec. #12
Structure Parameters

<table>
<thead>
<tr>
<th>H max (ft)</th>
<th>B min (ft)</th>
<th>B Ext. (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>β (deg)</th>
<th>β l (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.000</td>
<td>11.000</td>
<td>11.000</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

External Active Earth Pressure Coefficient: 0.283
External Stability CDR Summary (Effective Stress)

<table>
<thead>
<tr>
<th></th>
<th>CDR Sliding</th>
<th>Limiting Eccentricity (ft)</th>
<th>CDR Overturning</th>
<th>Bearing Pressure (ksf)</th>
<th>Eccentricity (ft)</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>S: re: g: 1 - Min</td>
<td>2.74</td>
<td>0.92</td>
<td>5.95</td>
<td>2.21</td>
<td>0.71</td>
<td>0.500</td>
</tr>
<tr>
<td>S: re: g: 1 - Max</td>
<td>2.64</td>
<td>0.92</td>
<td>6.01</td>
<td>2.83</td>
<td>0.75</td>
<td>0.500</td>
</tr>
<tr>
<td>S: re: g: 1 - Critical</td>
<td>1.95</td>
<td>1.24</td>
<td>4.45</td>
<td>2.33</td>
<td>0.95</td>
<td>0.500</td>
</tr>
<tr>
<td>Service</td>
<td>3.06</td>
<td>0.78</td>
<td>7.05</td>
<td>1.97</td>
<td>0.67</td>
<td>0.500</td>
</tr>
</tbody>
</table>

Internal Active Earth Pressure Coefficient: 0.283
Internal Stability CDR Summary

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Internal Earth Coeff.</th>
<th>Rupture CDR</th>
<th>Pullout CDR</th>
<th>F*</th>
<th>Le (ft)</th>
<th>H/B Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.25</td>
<td>11.00</td>
<td>5</td>
<td>0.46</td>
<td>3.20</td>
<td>1.24</td>
<td>75.65</td>
<td>7.70</td>
<td>1.00</td>
</tr>
<tr>
<td>4.75</td>
<td>11.00</td>
<td>3</td>
<td>0.45</td>
<td>1.74</td>
<td>1.01</td>
<td>70.64</td>
<td>7.70</td>
<td>1.00</td>
</tr>
<tr>
<td>7.25</td>
<td>11.00</td>
<td>3</td>
<td>0.43</td>
<td>1.31</td>
<td>1.11</td>
<td>65.63</td>
<td>8.75</td>
<td>1.00</td>
</tr>
<tr>
<td>9.75</td>
<td>11.00</td>
<td>3</td>
<td>0.41</td>
<td>1.07</td>
<td>1.26</td>
<td>60.82</td>
<td>10.25</td>
<td>1.00</td>
</tr>
</tbody>
</table>
### Project Information

- **VAWS #:** VW14-02388
- **Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1
- **Location:** Barnstable County, MA
- **Project #:** CM/TAP/TI-002S(758)X
- **Design Engineer:** cms
- **Wall Name:** B=0.7H Minimum

### Static Internal Stability - Rupture Force Results

**Rec. #12 - H = 11.00 (ft) B = 11.00 (ft)**

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (kcf)</th>
<th>Static Tmax (Kips)</th>
<th>Static CDR Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.25</td>
<td>11.00</td>
<td>5</td>
<td>35.00</td>
<td>0.46</td>
<td>0.75</td>
<td>0.35</td>
<td>12.16</td>
<td>3.20</td>
</tr>
<tr>
<td>4.75</td>
<td>11.00</td>
<td>3</td>
<td>25.00</td>
<td>0.45</td>
<td>1.20</td>
<td>0.54</td>
<td>13.45</td>
<td>1.74</td>
</tr>
<tr>
<td>7.25</td>
<td>11.00</td>
<td>3</td>
<td>25.00</td>
<td>0.43</td>
<td>1.66</td>
<td>0.71</td>
<td>17.81</td>
<td>1.31</td>
</tr>
<tr>
<td>9.75</td>
<td>11.00</td>
<td>3</td>
<td>25.00</td>
<td>0.41</td>
<td>2.11</td>
<td>0.87</td>
<td>21.76</td>
<td>1.07</td>
</tr>
</tbody>
</table>

### Static Internal Stability - Pullout Force Results

**Rec. #12 - H = 11.00 (ft) B = 11.00 (ft)**

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (kcf)</th>
<th>Static Horizontal Stress (kcf)</th>
<th>Le (ft)</th>
<th>F*</th>
<th>Pr (Kips)</th>
<th>Static CDR Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.25</td>
<td>11.00</td>
<td>5</td>
<td>35.00</td>
<td>0.46</td>
<td>0.30</td>
<td>0.14</td>
<td>6.67</td>
<td>7.70</td>
<td>75.65</td>
<td>8.29</td>
</tr>
<tr>
<td>4.75</td>
<td>11.00</td>
<td>3</td>
<td>25.00</td>
<td>0.45</td>
<td>0.64</td>
<td>0.29</td>
<td>9.67</td>
<td>7.70</td>
<td>70.64</td>
<td>9.81</td>
</tr>
<tr>
<td>7.25</td>
<td>11.00</td>
<td>3</td>
<td>25.00</td>
<td>0.43</td>
<td>0.98</td>
<td>0.42</td>
<td>14.18</td>
<td>8.75</td>
<td>65.63</td>
<td>15.81</td>
</tr>
<tr>
<td>9.75</td>
<td>11.00</td>
<td>3</td>
<td>25.00</td>
<td>0.41</td>
<td>1.32</td>
<td>0.54</td>
<td>18.29</td>
<td>10.25</td>
<td>60.62</td>
<td>23.00</td>
</tr>
</tbody>
</table>
SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

Project Information
VAWS #: VW14-02388
Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1
Location: Barnstable County, MA
Project #: CM/TAP/TI-002S(758)X
Design Engineer: cms
Wall Name: B=0.7H Minimum

Rec. #13
Structure Parameters
<table>
<thead>
<tr>
<th>H max (ft)</th>
<th>B min (ft)</th>
<th>B Ext. (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>β (deg)</th>
<th>β I (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.000</td>
<td>10.000</td>
<td>10.000</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

External Active Earth Pressure Coefficient: 0.283
External Stability CDR Summary (Effective Stress)

<table>
<thead>
<tr>
<th>Se: g a 1 - Min</th>
<th>CDR Sliding</th>
<th>Limiting Eccentricity (ft)</th>
<th>CDR Overturning</th>
<th>Bearing Pressure (ksf)</th>
<th>Eccentricity (ft)</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.64</td>
<td>0.88</td>
<td>5.67</td>
<td>2.06</td>
<td>0.67</td>
<td>0.500</td>
<td></td>
</tr>
<tr>
<td>2.57</td>
<td>0.86</td>
<td>5.79</td>
<td>2.63</td>
<td>0.70</td>
<td>0.500</td>
<td></td>
</tr>
<tr>
<td>1.90</td>
<td>1.16</td>
<td>4.29</td>
<td>2.17</td>
<td>0.88</td>
<td>0.500</td>
<td></td>
</tr>
<tr>
<td>2.98</td>
<td>0.73</td>
<td>6.82</td>
<td>1.83</td>
<td>0.62</td>
<td>0.500</td>
<td></td>
</tr>
</tbody>
</table>

Internal Active Earth Pressure Coefficient: 0.283
Internal Stability CDR Summary

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Internal Earth Coeff.</th>
<th>Rupture CDR</th>
<th>Pullout CDR</th>
<th>F*</th>
<th>Le (ft)</th>
<th>H/B Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>10.00</td>
<td>4</td>
<td>0.47</td>
<td>3.31</td>
<td>1.10</td>
<td>76.15</td>
<td>7.00</td>
<td>1.00</td>
</tr>
<tr>
<td>3.75</td>
<td>10.00</td>
<td>3</td>
<td>0.45</td>
<td>2.37</td>
<td>1.10</td>
<td>72.64</td>
<td>7.00</td>
<td>1.00</td>
</tr>
<tr>
<td>6.25</td>
<td>10.00</td>
<td>3</td>
<td>0.44</td>
<td>1.45</td>
<td>1.00</td>
<td>67.63</td>
<td>7.75</td>
<td>1.00</td>
</tr>
<tr>
<td>8.75</td>
<td>10.00</td>
<td>3</td>
<td>0.42</td>
<td>1.15</td>
<td>1.15</td>
<td>62.82</td>
<td>9.25</td>
<td>1.00</td>
</tr>
</tbody>
</table>
### Static Internal Stability - Rupture Force Results

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Le (ft)</th>
<th>F*</th>
<th>Pr (Kips)</th>
<th>Static CDR Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>10.00</td>
<td>4</td>
<td>28.75</td>
<td>0.47</td>
<td>0.70</td>
<td>0.33</td>
<td>9.41</td>
<td>3.31</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.75</td>
<td>10.00</td>
<td>3</td>
<td>21.25</td>
<td>0.45</td>
<td>1.02</td>
<td>0.46</td>
<td>9.85</td>
<td>2.37</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.25</td>
<td>10.00</td>
<td>3</td>
<td>25.00</td>
<td>0.44</td>
<td>1.48</td>
<td>0.64</td>
<td>16.11</td>
<td>1.45</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8.75</td>
<td>10.00</td>
<td>3</td>
<td>25.00</td>
<td>0.42</td>
<td>1.93</td>
<td>0.81</td>
<td>20.23</td>
<td>1.15</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Static Internal Stability - Pullout Force Results

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horz. Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Le (ft)</th>
<th>F*</th>
<th>Pr (Kips)</th>
<th>Static CDR Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>10.00</td>
<td>4</td>
<td>28.75</td>
<td>0.47</td>
<td>0.27</td>
<td>0.13</td>
<td>4.89</td>
<td>7.00</td>
<td>76.15</td>
<td>5.40</td>
<td>1.10</td>
</tr>
<tr>
<td>3.75</td>
<td>10.00</td>
<td>3</td>
<td>21.25</td>
<td>0.45</td>
<td>0.51</td>
<td>0.23</td>
<td>6.60</td>
<td>7.00</td>
<td>72.64</td>
<td>7.24</td>
<td>1.10</td>
</tr>
<tr>
<td>6.25</td>
<td>10.00</td>
<td>3</td>
<td>25.00</td>
<td>0.44</td>
<td>0.84</td>
<td>0.37</td>
<td>12.43</td>
<td>7.75</td>
<td>67.63</td>
<td>12.44</td>
<td>1.00</td>
</tr>
<tr>
<td>8.75</td>
<td>10.00</td>
<td>3</td>
<td>25.00</td>
<td>0.42</td>
<td>1.18</td>
<td>0.49</td>
<td>16.70</td>
<td>9.25</td>
<td>62.62</td>
<td>19.24</td>
<td>1.15</td>
</tr>
</tbody>
</table>
SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

Project Information
VAWS #: VW14-02388
Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1
Location: Barnstable County, MA
Project #: CM/TAP/TI-002S(758)X
Design Engineer: cms
Wall Name: B=0.7H Minimum

Rec. #14
Structure Parameters

<table>
<thead>
<tr>
<th>H max</th>
<th>B min</th>
<th>B Ext.</th>
<th>S</th>
<th>Xs</th>
<th>β</th>
<th>β t</th>
</tr>
</thead>
<tbody>
<tr>
<td>(ft)</td>
<td>(ft)</td>
<td>(ft)</td>
<td>(ft)</td>
<td>(ft)</td>
<td>(deg)</td>
<td>(deg)</td>
</tr>
<tr>
<td>7.200</td>
<td>9.000</td>
<td>9.000</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

External Active Earth Pressure Coefficient: 0.283
External Stability CDR Summary (Effective Stress)

<table>
<thead>
<tr>
<th></th>
<th>CDR Sliding</th>
<th>Limiting Eccentricity</th>
<th>CDR Overturning</th>
<th>Bearing Pressure</th>
<th>Eccentricity</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(ft)</td>
<td></td>
<td>(ft)</td>
<td>(ksf)</td>
<td></td>
</tr>
<tr>
<td>Static - Min</td>
<td>2.84</td>
<td>0.61</td>
<td>7.37</td>
<td>1.56</td>
<td>0.42</td>
<td>0.500</td>
</tr>
<tr>
<td>Static - Max</td>
<td>2.87</td>
<td>0.57</td>
<td>7.85</td>
<td>1.93</td>
<td>0.43</td>
<td>0.500</td>
</tr>
<tr>
<td>Static - Critical</td>
<td>2.13</td>
<td>0.77</td>
<td>5.82</td>
<td>1.60</td>
<td>0.53</td>
<td>0.500</td>
</tr>
<tr>
<td>Service</td>
<td>3.37</td>
<td>0.48</td>
<td>9.36</td>
<td>1.34</td>
<td>0.38</td>
<td>0.500</td>
</tr>
</tbody>
</table>

Internal Active Earth Pressure Coefficient: 0.283
Internal Stability CDR Summary

<table>
<thead>
<tr>
<th>Depth Below Top of Wall</th>
<th>B min</th>
<th>SR/Row</th>
<th>Internal Earth Coeff.</th>
<th>Rupture CDR</th>
<th>Pullout CDR</th>
<th>F*</th>
<th>Le</th>
<th>H/B Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>(ft)</td>
<td>(ft)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.00</td>
<td>9.00</td>
<td>4</td>
<td>0.47</td>
<td>3.49</td>
<td>1.14</td>
<td>76.15</td>
<td>6.84</td>
<td>1.25</td>
</tr>
<tr>
<td>3.45</td>
<td>9.00</td>
<td>3</td>
<td>0.46</td>
<td>2.68</td>
<td>1.16</td>
<td>73.25</td>
<td>6.84</td>
<td>1.25</td>
</tr>
<tr>
<td>5.95</td>
<td>9.00</td>
<td>3</td>
<td>0.44</td>
<td>1.50</td>
<td>1.07</td>
<td>68.23</td>
<td>8.25</td>
<td>1.25</td>
</tr>
</tbody>
</table>
### MSE - Pro
Mechanically Stabilized Earth Retaining Structures

#### SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

**Project Information**

VAWS #: VW14-02388  
Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1  
Location: Barnstable County, MA  
Project #: CM/TAP/TI-002S(758)X  
Design Engineer: cms  
Wall Name: B=0.7H Minimum

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Static CDR Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>9.00</td>
<td>4</td>
<td>27.25</td>
<td>0.47</td>
<td>0.70</td>
<td>0.33</td>
<td>8.92</td>
<td>3.49</td>
</tr>
<tr>
<td>3.45</td>
<td>9.00</td>
<td>3</td>
<td>19.75</td>
<td>0.46</td>
<td>0.97</td>
<td>0.44</td>
<td>8.71</td>
<td>2.68</td>
</tr>
<tr>
<td>5.95</td>
<td>9.00</td>
<td>3</td>
<td>25.00</td>
<td>0.44</td>
<td>1.42</td>
<td>0.62</td>
<td>15.59</td>
<td>1.50</td>
</tr>
</tbody>
</table>

### Static Internal Stability - Pullout Force Results

Rec. #14 - H = 7.20 (ft) B = 9.00 (ft)

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Le (ft)</th>
<th>F*</th>
<th>Pr (Kips)</th>
<th>Static CDR Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>9.00</td>
<td>4</td>
<td>27.25</td>
<td>0.47</td>
<td>0.27</td>
<td>0.13</td>
<td>4.63</td>
<td>6.84</td>
<td>76.15</td>
<td>5.27</td>
<td>1.14</td>
</tr>
<tr>
<td>3.45</td>
<td>9.00</td>
<td>3</td>
<td>19.75</td>
<td>0.46</td>
<td>0.47</td>
<td>0.21</td>
<td>5.67</td>
<td>6.84</td>
<td>73.25</td>
<td>6.56</td>
<td>1.16</td>
</tr>
<tr>
<td>5.95</td>
<td>9.00</td>
<td>3</td>
<td>25.00</td>
<td>0.44</td>
<td>0.80</td>
<td>0.35</td>
<td>11.89</td>
<td>8.25</td>
<td>68.23</td>
<td>12.72</td>
<td>1.07</td>
</tr>
</tbody>
</table>
# SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

## Project Information

**VAWS #:** VW14-02388  
**Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1  
**Location:** Barnstable County, MA  
**Project #:** CM/TAP/TI-002S(758)X  
**Design Engineer:** cms  
**Wall Name:** B=0.7H Minimum

## Structure Parameters

<table>
<thead>
<tr>
<th>H max (ft)</th>
<th>B min (ft)</th>
<th>B Ext. (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>( \beta ) (deg)</th>
<th>( \beta_i ) (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.333</td>
<td>8.000</td>
<td>8.000</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

## External Active Earth Pressure Coefficient: 0.283

### External Stability CDR Summary (Effective Stress)

<table>
<thead>
<tr>
<th>S: re: g 1 - Min</th>
<th>CDR Sliding</th>
<th>Limiting Eccentricity (ft)</th>
<th>CDR Overturning</th>
<th>Bearing Pressure (ksf)</th>
<th>Eccentricity (ft)</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2.90</td>
<td>0.46</td>
<td>8.77</td>
<td>1.25</td>
<td>0.28</td>
<td>0.500</td>
</tr>
<tr>
<td>S: re: g 1 - Max</td>
<td>3.05</td>
<td>0.41</td>
<td>9.70</td>
<td>1.52</td>
<td>0.28</td>
<td>0.500</td>
</tr>
<tr>
<td>S: re: g 1 - Crit</td>
<td>2.26</td>
<td>0.56</td>
<td>7.19</td>
<td>1.27</td>
<td>0.35</td>
<td>0.500</td>
</tr>
<tr>
<td>Service</td>
<td>3.62</td>
<td>0.34</td>
<td>11.70</td>
<td>1.04</td>
<td>0.25</td>
<td>0.500</td>
</tr>
</tbody>
</table>

## Internal Active Earth Pressure Coefficient: 0.283

### Internal Stability CDR Summary

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Internal Earth Coeff.</th>
<th>Rupture CDR</th>
<th>Pullout CDR</th>
<th>F*</th>
<th>Le (ft)</th>
<th>H/B Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>8.00</td>
<td>5</td>
<td>0.47</td>
<td>3.91</td>
<td>1.19</td>
<td>76.15</td>
<td>6.40</td>
<td>1.50</td>
</tr>
<tr>
<td>4.08</td>
<td>8.00</td>
<td>3</td>
<td>0.45</td>
<td>2.08</td>
<td>1.05</td>
<td>71.98</td>
<td>7.25</td>
<td>1.50</td>
</tr>
</tbody>
</table>
### Project Information

**VAWS #:** VW14-02388  
**Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1  
**Location:** Barnstable County, MA  
**Project #:** CM/TAP/TI-002S(758)X  
**Design Engineer:** cm  
**Wall Name:** B=0.7H Minimum

### Static Internal Stability - Rupture Force Results

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Static CDR Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>8.00</td>
<td>5</td>
<td>30.42</td>
<td>0.47</td>
<td>0.70</td>
<td>0.33</td>
<td>9.96</td>
<td>3.91</td>
</tr>
<tr>
<td>4.08</td>
<td>8.00</td>
<td>3</td>
<td>22.92</td>
<td>0.45</td>
<td>1.08</td>
<td>0.49</td>
<td>11.20</td>
<td>2.08</td>
</tr>
</tbody>
</table>

### Static Internal Stability - Pullout Force Results

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Le (ft)</th>
<th>F*</th>
<th>Pr (Kips)</th>
<th>Static CDR Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>8.00</td>
<td>5</td>
<td>30.42</td>
<td>0.47</td>
<td>0.27</td>
<td>0.13</td>
<td>5.17</td>
<td>6.40</td>
<td>76.15</td>
<td>6.17</td>
<td>1.19</td>
</tr>
<tr>
<td>4.08</td>
<td>8.00</td>
<td>3</td>
<td>22.92</td>
<td>0.45</td>
<td>0.55</td>
<td>0.25</td>
<td>7.70</td>
<td>7.25</td>
<td>71.98</td>
<td>8.09</td>
<td>1.05</td>
</tr>
</tbody>
</table>

### Legend

- **H max**  
  Height of the wall  
- **B min**  
  Minimum length of soil reinforcing for defined wall height  
- **S**  
  Maximum height of surcharge  
- **X**  
  Distance of surcharge slope over soil reinforcement  
- **β**  
  Slope of surcharge  
- **βi**  
  Adjusted angle  
- **Le**  
  Length of embedment of soil reinforcing in passive zone  
- **FS**  
  Factor of safety  
- **BP**  
  Bearing pressure  
- **F**  
  Pullout friction factor  
- **Le**  
  Overturning
# SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

## Project Information
- **VAWS #:** VW14-02388
- **Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1
- **Location:** Barnstable County, MA
- **Project #:** CM/TAP/TI-002S(758)X
- **Design Engineer:** cm
- **Wall Name:** B=0.7H Minimum

## Structure Parameters

<table>
<thead>
<tr>
<th>Rec #</th>
<th>H max (ft)</th>
<th>SR Ratio</th>
<th>B min (ft)</th>
<th>Actual B (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>$\beta$ (deg)</th>
<th>$\beta_t$ (deg)</th>
<th>Live Load Surcharge (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>31.429</td>
<td>0.70</td>
<td>22.000</td>
<td>23.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>30.000</td>
<td>0.70</td>
<td>21.000</td>
<td>22.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>3</td>
<td>28.571</td>
<td>0.70</td>
<td>20.000</td>
<td>21.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>4</td>
<td>27.143</td>
<td>0.70</td>
<td>19.000</td>
<td>20.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>5</td>
<td>25.714</td>
<td>0.70</td>
<td>18.000</td>
<td>19.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>6</td>
<td>24.286</td>
<td>0.70</td>
<td>17.000</td>
<td>18.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>7</td>
<td>22.857</td>
<td>0.70</td>
<td>16.000</td>
<td>17.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>8</td>
<td>21.429</td>
<td>0.70</td>
<td>15.000</td>
<td>16.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>9</td>
<td>18.667</td>
<td>0.75</td>
<td>14.000</td>
<td>15.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>10</td>
<td>16.250</td>
<td>0.80</td>
<td>13.000</td>
<td>14.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>11</td>
<td>14.118</td>
<td>0.85</td>
<td>12.000</td>
<td>13.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>12</td>
<td>11.000</td>
<td>1.00</td>
<td>11.000</td>
<td>12.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>13</td>
<td>10.000</td>
<td>1.00</td>
<td>10.000</td>
<td>11.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>14</td>
<td>7.200</td>
<td>1.25</td>
<td>9.000</td>
<td>10.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>15</td>
<td>5.333</td>
<td>1.50</td>
<td>8.000</td>
<td>9.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>
---

### Project Information

- **VAWS #:** VW14-02388  
- **Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1  
- **Location:** Barnstable County, MA  
- **Design Engineer:** cm.w  
- **Wall Name:** B=0.7H Minimum

### Material Strength Parameters

| Soil  | Reinforced Soil Pullout (kcf) | Reinforced Soil Rupture (kcf) | Friction Angle of Reinforced Soil (deg) | Unit Weight of Retained Soil (kcf) | Friction Angle of Retained Soil (deg) | Cohesion of Retained Soil (kcf) | Unit Weight of Foundation Soil (kcf) | Friction Angle of Foundation Soil (deg) | Cohesion of Foundation Soil (ksf) | Cohesion of Foundation Soil Sliding (ksf) | Cohesion of Foundation Soil Bearing (ksf) | Bearing Capacity Factor Surcharge | Bearing Capacity Factor Cohesion | Bearing Capacity Factor Over Burden | Depth to Water Table (ft) |
|-------|-------------------------------|-------------------------------|----------------------------------------|-----------------------------------|----------------------------------------|---------------------------------|-------------------------------------|----------------------------------------|---------------------------------|----------------------------------------|-------------------------------------|-------------------------------|-----------------------------|-----------------------------|-----------------------------|-----------------------------|
| Soil  | Effective Stress 0.135        | Effective Stress 0.135        | Friction Angle of Retained Soil 34.0   | Unit Weight of Retained Soil 0.135 | Friction Angle of Retained Soil 34.0  | Cohesion of Retained Soil 0.000  | Unit Weight of Foundation Soil 0.120 | Friction Angle of Foundation Soil 30.0 | Cohesion of Foundation Soil 0.000 | Cohesion of Foundation Soil Sliding 0.000 | Cohesion of Foundation Soil Bearing 18.40 | Bearing Capacity Factor Surcharge 30.14 | Bearing Capacity Factor Cohesion 22.40 | Bearing Capacity Factor Over Burden 0.00 | Depth to Water Table 0.00 |

### Effective Stress Evaluation

<table>
<thead>
<tr>
<th>Rec #</th>
<th>Applied Bearing Pressure (ksf)</th>
<th>Ultimate Bearing Pressure (ksf)</th>
<th>Factor of Safety for Bearing Capacity</th>
<th>Factor of Safety for Sliding</th>
<th>Factor of Safety for Overturning</th>
<th>Active Earth Pressure Coefficient</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8.11</td>
<td>11.24</td>
<td>1.39</td>
<td>2.26</td>
<td>4.18</td>
<td>0.283</td>
<td>0.500</td>
</tr>
<tr>
<td>2</td>
<td>7.78</td>
<td>10.72</td>
<td>1.38</td>
<td>2.25</td>
<td>4.16</td>
<td>0.283</td>
<td>0.500</td>
</tr>
<tr>
<td>3</td>
<td>7.45</td>
<td>10.19</td>
<td>1.37</td>
<td>2.24</td>
<td>4.13</td>
<td>0.283</td>
<td>0.500</td>
</tr>
<tr>
<td>4</td>
<td>7.11</td>
<td>9.66</td>
<td>1.36</td>
<td>2.22</td>
<td>4.11</td>
<td>0.283</td>
<td>0.500</td>
</tr>
<tr>
<td>5</td>
<td>6.78</td>
<td>9.14</td>
<td>1.35</td>
<td>2.20</td>
<td>4.09</td>
<td>0.283</td>
<td>0.500</td>
</tr>
<tr>
<td>6</td>
<td>6.45</td>
<td>8.61</td>
<td>1.33</td>
<td>2.18</td>
<td>4.06</td>
<td>0.283</td>
<td>0.500</td>
</tr>
<tr>
<td>7</td>
<td>6.12</td>
<td>8.08</td>
<td>1.32</td>
<td>2.16</td>
<td>4.03</td>
<td>0.283</td>
<td>0.500</td>
</tr>
<tr>
<td>8</td>
<td>5.79</td>
<td>7.56</td>
<td>1.30</td>
<td>2.14</td>
<td>4.00</td>
<td>0.283</td>
<td>0.500</td>
</tr>
</tbody>
</table>
---
# SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

## Project Information

- **VAWS #:** VW14-02388
- **Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1
- **Location:** Barnstable County, MA
- **Project #:** CM/TAP/TI-002S(758)X
- **Design Engineer:** cms
- **Wall Name:** B=0.7H Minimum

## Effective Stress Evaluation

<table>
<thead>
<tr>
<th>Rec #</th>
<th>Applied Bearing Pressure (ksf)</th>
<th>Ultimate Bearing Pressure (ksf)</th>
<th>Factor of Safety for Bearing Capacity</th>
<th>Factor of Safety for Sliding</th>
<th>Factor of Safety for Overturning</th>
<th>Active Earth Pressure Coefficient</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>4.94</td>
<td>7.32</td>
<td>1.48</td>
<td>2.24</td>
<td>4.50</td>
<td>0.283</td>
<td>0.500</td>
</tr>
<tr>
<td>10</td>
<td>4.25</td>
<td>6.99</td>
<td>1.65</td>
<td>2.32</td>
<td>5.01</td>
<td>0.283</td>
<td>0.500</td>
</tr>
<tr>
<td>11</td>
<td>3.67</td>
<td>6.61</td>
<td>1.80</td>
<td>2.39</td>
<td>5.53</td>
<td>0.283</td>
<td>0.500</td>
</tr>
<tr>
<td>12</td>
<td>2.83</td>
<td>6.38</td>
<td>2.26</td>
<td>2.64</td>
<td>7.32</td>
<td>0.283</td>
<td>0.500</td>
</tr>
<tr>
<td>13</td>
<td>2.63</td>
<td>5.79</td>
<td>2.20</td>
<td>2.57</td>
<td>7.19</td>
<td>0.283</td>
<td>0.500</td>
</tr>
<tr>
<td>14</td>
<td>1.93</td>
<td>5.47</td>
<td>2.83</td>
<td>2.87</td>
<td>10.47</td>
<td>0.283</td>
<td>0.500</td>
</tr>
<tr>
<td>15</td>
<td>1.52</td>
<td>4.99</td>
<td>3.29</td>
<td>3.05</td>
<td>14.07</td>
<td>0.283</td>
<td>0.500</td>
</tr>
</tbody>
</table>
PROJECT SPECIFIC CALCULATIONS

BACK-TO-BACK WALL - SECTION REINFORCEMENT LENGTH REQUIREMENTS

SEGMENTAL CONCRETE PANEL MSE RETAINING WALL

CAPE COD RAIL TRAIL EXTENSION – PHASE 1

FEDERAL AID PROJECT NO. CM/TAP/TI-002S(758)X

PROJECT FILE NO. 604488

BARNSTABLE COUNTY, MA

PREPARED FOR:

LAWRENCE-LYNCH CORP.

BIG R BRIDGE PROJECT NO. VW14-02388
<table>
<thead>
<tr>
<th>COLUMN NO.</th>
<th>WALL HEIGHT (FT)</th>
<th>0.3H MIN OVERLAP (FT)</th>
<th>0.6H MIN REINFORCING LENGTH (FT)</th>
<th>BACK-TO-BACK REINFORCING LENGTH (FT)</th>
<th>COLUMN NO.</th>
<th>WALL HEIGHT (FT)</th>
<th>0.6H MIN REINFORCING LENGTH (FT)</th>
<th>BACK-TO-BACK REINFORCING LENGTH (FT)</th>
<th>REQUIRED TOTAL LENGTH INCLUDING OVERLAP (FT)</th>
<th>PROVIDED TOTAL LENGTH INCLUDING OVERLAP (FT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>13.25</td>
<td>4</td>
<td>8</td>
<td>11</td>
<td>80</td>
<td>13.50</td>
<td>9</td>
<td>11</td>
<td>21</td>
<td>22</td>
</tr>
<tr>
<td>15</td>
<td>13.70</td>
<td>5</td>
<td>9</td>
<td>11</td>
<td>79</td>
<td>13.95</td>
<td>9</td>
<td>11</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>16</td>
<td>14.15</td>
<td>5</td>
<td>9</td>
<td>11</td>
<td>78</td>
<td>14.40</td>
<td>9</td>
<td>11</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>17</td>
<td>14.60</td>
<td>5</td>
<td>9</td>
<td>11</td>
<td>77</td>
<td>14.85</td>
<td>9</td>
<td>11</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>18</td>
<td>15.05</td>
<td>5</td>
<td>10</td>
<td>11</td>
<td>76</td>
<td>15.30</td>
<td>10</td>
<td>11</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>19</td>
<td>15.50</td>
<td>5</td>
<td>10</td>
<td>11</td>
<td>75</td>
<td>15.75</td>
<td>10</td>
<td>11</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>20</td>
<td>15.94</td>
<td>5</td>
<td>10</td>
<td>11</td>
<td>74</td>
<td>16.20</td>
<td>10</td>
<td>11</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>21</td>
<td>16.39</td>
<td>5</td>
<td>10</td>
<td>11</td>
<td>73</td>
<td>16.65</td>
<td>10</td>
<td>11</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>22</td>
<td>16.84</td>
<td>6</td>
<td>11</td>
<td>12</td>
<td>72</td>
<td>17.10</td>
<td>11</td>
<td>11</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>23</td>
<td>17.29</td>
<td>6</td>
<td>11</td>
<td>12</td>
<td>71</td>
<td>17.55</td>
<td>11</td>
<td>11</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>24</td>
<td>17.74</td>
<td>6</td>
<td>11</td>
<td>12</td>
<td>70</td>
<td>18.00</td>
<td>11</td>
<td>11</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>25</td>
<td>18.19</td>
<td>6</td>
<td>11</td>
<td>12</td>
<td>69</td>
<td>18.45</td>
<td>12</td>
<td>11</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>26</td>
<td>18.63</td>
<td>6</td>
<td>12</td>
<td>12</td>
<td>68</td>
<td>18.90</td>
<td>12</td>
<td>12</td>
<td>23</td>
<td>24</td>
</tr>
<tr>
<td>27</td>
<td>19.08</td>
<td>6</td>
<td>12</td>
<td>12</td>
<td>67</td>
<td>19.35</td>
<td>12</td>
<td>12</td>
<td>23</td>
<td>24</td>
</tr>
<tr>
<td>28</td>
<td>19.53</td>
<td>6</td>
<td>12</td>
<td>12</td>
<td>66</td>
<td>19.80</td>
<td>12</td>
<td>12</td>
<td>23</td>
<td>24</td>
</tr>
<tr>
<td>29</td>
<td>19.98</td>
<td>6</td>
<td>12</td>
<td>12</td>
<td>65</td>
<td>20.25</td>
<td>13</td>
<td>13</td>
<td>23</td>
<td>25</td>
</tr>
<tr>
<td>30</td>
<td>20.43</td>
<td>7</td>
<td>13</td>
<td>13</td>
<td>64</td>
<td>20.70</td>
<td>13</td>
<td>13</td>
<td>24</td>
<td>26</td>
</tr>
<tr>
<td>31</td>
<td>20.87</td>
<td>7</td>
<td>13</td>
<td>13</td>
<td>63</td>
<td>21.15</td>
<td>13</td>
<td>13</td>
<td>24</td>
<td>26</td>
</tr>
<tr>
<td>32</td>
<td>21.32</td>
<td>7</td>
<td>13</td>
<td>13</td>
<td>62</td>
<td>21.60</td>
<td>13</td>
<td>13</td>
<td>24</td>
<td>26</td>
</tr>
<tr>
<td>33</td>
<td>21.77</td>
<td>7</td>
<td>14</td>
<td>14</td>
<td>61</td>
<td>22.05</td>
<td>14</td>
<td>14</td>
<td>24</td>
<td>28</td>
</tr>
<tr>
<td>34</td>
<td>22.22</td>
<td>7</td>
<td>14</td>
<td>14</td>
<td>60</td>
<td>22.50</td>
<td>14</td>
<td>14</td>
<td>24</td>
<td>28</td>
</tr>
<tr>
<td>35</td>
<td>22.67</td>
<td>7</td>
<td>14</td>
<td>14</td>
<td>59</td>
<td>22.95</td>
<td>14</td>
<td>14</td>
<td>24</td>
<td>28</td>
</tr>
<tr>
<td>36</td>
<td>23.11</td>
<td>7</td>
<td>14</td>
<td>14</td>
<td>58</td>
<td>23.40</td>
<td>15</td>
<td>15</td>
<td>24</td>
<td>29</td>
</tr>
<tr>
<td>37</td>
<td>23.56</td>
<td>8</td>
<td>15</td>
<td>15</td>
<td>57</td>
<td>23.85</td>
<td>15</td>
<td>15</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>38</td>
<td>24.01</td>
<td>8</td>
<td>15</td>
<td>15</td>
<td>56</td>
<td>24.30</td>
<td>15</td>
<td>15</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>39</td>
<td>24.46</td>
<td>8</td>
<td>15</td>
<td>15</td>
<td>55</td>
<td>24.75</td>
<td>15</td>
<td>15</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>40</td>
<td>24.85</td>
<td>8</td>
<td>15</td>
<td>15</td>
<td>54</td>
<td>25.13</td>
<td>16</td>
<td>16</td>
<td>25</td>
<td>31</td>
</tr>
<tr>
<td>41</td>
<td>25.22</td>
<td>8</td>
<td>16</td>
<td>16</td>
<td>53</td>
<td>25.65</td>
<td>16</td>
<td>16</td>
<td>25</td>
<td>32</td>
</tr>
<tr>
<td>42</td>
<td>25.59</td>
<td>8</td>
<td>16</td>
<td>16</td>
<td>53</td>
<td>25.65</td>
<td>16</td>
<td>16</td>
<td>25</td>
<td>32</td>
</tr>
<tr>
<td>COLUMN NO.</td>
<td>WALL HEIGHT (FT)</td>
<td>0.3H MIN OVERLAP (FT)</td>
<td>0.6H MIN REINFORCING LENGTH (FT)</td>
<td>BACK-TO-BACK REINFORCING LENGTH (FT)</td>
<td>COLUMN NO.</td>
<td>WALL HEIGHT (FT)</td>
<td>0.6H MIN REINFORCING LENGTH (FT)</td>
<td>BACK-TO-BACK REINFORCING LENGTH (FT)</td>
<td>REQUIRED TOTAL LENGTH INCLUDING OVERLAP (FT)</td>
<td>PROVIDED TOTAL LENGTH INCLUDING OVERLAP (FT)</td>
</tr>
<tr>
<td>-----------</td>
<td>-----------------</td>
<td>-----------------------</td>
<td>-------------------------------</td>
<td>-----------------------------------</td>
<td>-----------</td>
<td>-----------------</td>
<td>-------------------------------</td>
<td>-----------------------------------</td>
<td>-----------------------------------------------</td>
<td>-----------------------------------------------</td>
</tr>
<tr>
<td>82</td>
<td>9.84</td>
<td>3</td>
<td>6</td>
<td>8</td>
<td>28</td>
<td>20.02</td>
<td>13</td>
<td>15</td>
<td>20</td>
<td>23</td>
</tr>
<tr>
<td>81</td>
<td>10.24</td>
<td>4</td>
<td>7</td>
<td>8</td>
<td>29</td>
<td>20.46</td>
<td>13</td>
<td>15</td>
<td>20</td>
<td>23</td>
</tr>
<tr>
<td>80</td>
<td>10.71</td>
<td>4</td>
<td>7</td>
<td>8</td>
<td>30</td>
<td>20.89</td>
<td>13</td>
<td>15</td>
<td>21</td>
<td>23</td>
</tr>
<tr>
<td>79</td>
<td>11.18</td>
<td>4</td>
<td>7</td>
<td>8</td>
<td>31</td>
<td>21.32</td>
<td>13</td>
<td>15</td>
<td>21</td>
<td>23</td>
</tr>
<tr>
<td>78</td>
<td>16.66</td>
<td>5</td>
<td>10</td>
<td>11</td>
<td>32</td>
<td>21.75</td>
<td>14</td>
<td>15</td>
<td>21</td>
<td>26</td>
</tr>
<tr>
<td>77</td>
<td>17.13</td>
<td>6</td>
<td>11</td>
<td>11</td>
<td>33</td>
<td>22.18</td>
<td>14</td>
<td>15</td>
<td>22</td>
<td>26</td>
</tr>
<tr>
<td>76</td>
<td>17.60</td>
<td>6</td>
<td>11</td>
<td>11</td>
<td>34</td>
<td>22.61</td>
<td>14</td>
<td>15</td>
<td>23</td>
<td>26</td>
</tr>
<tr>
<td>75</td>
<td>23.06</td>
<td>7</td>
<td>14</td>
<td>15</td>
<td>35</td>
<td>23.04</td>
<td>14</td>
<td>15</td>
<td>23</td>
<td>30</td>
</tr>
<tr>
<td>74</td>
<td>23.52</td>
<td>8</td>
<td>15</td>
<td>15</td>
<td>36</td>
<td>23.49</td>
<td>15</td>
<td>15</td>
<td>24</td>
<td>30</td>
</tr>
<tr>
<td>73</td>
<td>23.96</td>
<td>8</td>
<td>15</td>
<td>15</td>
<td>37</td>
<td>23.94</td>
<td>15</td>
<td>15</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>72</td>
<td>24.41</td>
<td>8</td>
<td>15</td>
<td>15</td>
<td>38</td>
<td>24.39</td>
<td>15</td>
<td>15</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>71</td>
<td>23.52</td>
<td>8</td>
<td>15</td>
<td>15</td>
<td>39</td>
<td>24.84</td>
<td>15</td>
<td>15</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>70</td>
<td>25.31</td>
<td>8</td>
<td>16</td>
<td>16</td>
<td>40</td>
<td>25.29</td>
<td>16</td>
<td>16</td>
<td>25</td>
<td>32</td>
</tr>
<tr>
<td>69</td>
<td>25.76</td>
<td>8</td>
<td>16</td>
<td>16</td>
<td>41</td>
<td>25.74</td>
<td>16</td>
<td>16</td>
<td>25</td>
<td>32</td>
</tr>
<tr>
<td>68</td>
<td>26.21</td>
<td>8</td>
<td>16</td>
<td>16</td>
<td>42</td>
<td>26.19</td>
<td>16</td>
<td>16</td>
<td>25</td>
<td>32</td>
</tr>
<tr>
<td>67</td>
<td>26.65</td>
<td>8</td>
<td>16</td>
<td>16</td>
<td>43</td>
<td>26.64</td>
<td>16</td>
<td>16</td>
<td>25</td>
<td>32</td>
</tr>
<tr>
<td>66</td>
<td>27.08</td>
<td>9</td>
<td>17</td>
<td>17</td>
<td>44</td>
<td>27.11</td>
<td>17</td>
<td>17</td>
<td>25</td>
<td>34</td>
</tr>
<tr>
<td>65</td>
<td>27.42</td>
<td>9</td>
<td>17</td>
<td>17</td>
<td>45</td>
<td>27.52</td>
<td>17</td>
<td>17</td>
<td>26</td>
<td>34</td>
</tr>
<tr>
<td>64</td>
<td>27.74</td>
<td>9</td>
<td>17</td>
<td>17</td>
<td>46</td>
<td>27.83</td>
<td>17</td>
<td>17</td>
<td>26</td>
<td>34</td>
</tr>
<tr>
<td>63</td>
<td>28.07</td>
<td>9</td>
<td>17</td>
<td>17</td>
<td>47</td>
<td>28.14</td>
<td>17</td>
<td>17</td>
<td>26</td>
<td>34</td>
</tr>
<tr>
<td>62</td>
<td>28.39</td>
<td>9</td>
<td>18</td>
<td>18</td>
<td>48</td>
<td>28.45</td>
<td>18</td>
<td>18</td>
<td>26</td>
<td>36</td>
</tr>
<tr>
<td>61</td>
<td>28.71</td>
<td>9</td>
<td>18</td>
<td>18</td>
<td>49</td>
<td>28.77</td>
<td>18</td>
<td>18</td>
<td>26</td>
<td>36</td>
</tr>
<tr>
<td>COLUMN NO.</td>
<td>WALL HEIGHT (FT)</td>
<td>0.3H MIN OVERLAP (FT)</td>
<td>0.6H MIN REINFORCING LENGTH (FT)</td>
<td>BACK-TO-BACK REINFORCING LENGTH (FT)</td>
<td>COLUMN NO.</td>
<td>WALL HEIGHT (FT)</td>
<td>0.6H MIN REINFORCING LENGTH (FT)</td>
<td>BACK-TO-BACK REINFORCING LENGTH (FT)</td>
<td>REQUIRED TOTAL LENGTH INCLUDING OVERLAP (FT)</td>
<td>PROVIDED TOTAL LENGTH INCLUDING OVERLAP (FT)</td>
</tr>
<tr>
<td>-----------</td>
<td>----------------</td>
<td>-----------------------</td>
<td>---------------------------------</td>
<td>-------------------------------------</td>
<td>-----------</td>
<td>----------------</td>
<td>---------------------------------</td>
<td>-------------------------------------</td>
<td>-----------------------------------------------</td>
<td>-----------------------------------------------</td>
</tr>
<tr>
<td>3</td>
<td>10.41</td>
<td>4</td>
<td>7</td>
<td>9</td>
<td>76</td>
<td>10.43</td>
<td>7</td>
<td>12</td>
<td>21</td>
<td>21</td>
</tr>
<tr>
<td>4</td>
<td>10.86</td>
<td>4</td>
<td>7</td>
<td>9</td>
<td>75</td>
<td>10.88</td>
<td>7</td>
<td>12</td>
<td>21</td>
<td>21</td>
</tr>
<tr>
<td>5</td>
<td>11.31</td>
<td>4</td>
<td>7</td>
<td>9</td>
<td>74</td>
<td>11.33</td>
<td>7</td>
<td>12</td>
<td>21</td>
<td>21</td>
</tr>
<tr>
<td>6</td>
<td>11.76</td>
<td>4</td>
<td>8</td>
<td>9</td>
<td>73</td>
<td>11.78</td>
<td>8</td>
<td>12</td>
<td>21</td>
<td>21</td>
</tr>
<tr>
<td>7</td>
<td>12.21</td>
<td>4</td>
<td>8</td>
<td>9</td>
<td>72</td>
<td>12.23</td>
<td>8</td>
<td>12</td>
<td>21</td>
<td>21</td>
</tr>
<tr>
<td>8</td>
<td>12.66</td>
<td>4</td>
<td>8</td>
<td>9</td>
<td>71</td>
<td>12.68</td>
<td>8</td>
<td>12</td>
<td>21</td>
<td>21</td>
</tr>
<tr>
<td>9</td>
<td>13.11</td>
<td>4</td>
<td>8</td>
<td>9</td>
<td>70</td>
<td>13.13</td>
<td>8</td>
<td>12</td>
<td>21</td>
<td>21</td>
</tr>
<tr>
<td>10</td>
<td>13.56</td>
<td>5</td>
<td>9</td>
<td>10</td>
<td>69</td>
<td>13.58</td>
<td>9</td>
<td>12</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>11</td>
<td>14.01</td>
<td>5</td>
<td>9</td>
<td>10</td>
<td>68</td>
<td>14.03</td>
<td>9</td>
<td>12</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>12</td>
<td>14.46</td>
<td>5</td>
<td>9</td>
<td>10</td>
<td>67</td>
<td>14.48</td>
<td>9</td>
<td>12</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>13</td>
<td>14.91</td>
<td>5</td>
<td>9</td>
<td>10</td>
<td>66</td>
<td>14.93</td>
<td>9</td>
<td>12</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>14</td>
<td>15.11</td>
<td>5</td>
<td>10</td>
<td>10</td>
<td>65</td>
<td>15.13</td>
<td>10</td>
<td>12</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>15</td>
<td>15.81</td>
<td>5</td>
<td>10</td>
<td>10</td>
<td>64</td>
<td>15.83</td>
<td>10</td>
<td>12</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>16</td>
<td>16.26</td>
<td>5</td>
<td>10</td>
<td>10</td>
<td>63</td>
<td>16.28</td>
<td>10</td>
<td>12</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>17</td>
<td>16.71</td>
<td>6</td>
<td>11</td>
<td>11</td>
<td>62</td>
<td>16.73</td>
<td>11</td>
<td>12</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>18</td>
<td>17.16</td>
<td>6</td>
<td>11</td>
<td>11</td>
<td>61</td>
<td>17.18</td>
<td>11</td>
<td>12</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>19</td>
<td>17.61</td>
<td>6</td>
<td>11</td>
<td>11</td>
<td>60</td>
<td>17.63</td>
<td>11</td>
<td>12</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>20</td>
<td>18.06</td>
<td>6</td>
<td>11</td>
<td>11</td>
<td>59</td>
<td>18.08</td>
<td>11</td>
<td>12</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>21</td>
<td>18.51</td>
<td>6</td>
<td>12</td>
<td>12</td>
<td>58</td>
<td>18.53</td>
<td>12</td>
<td>12</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>22</td>
<td>18.96</td>
<td>6</td>
<td>12</td>
<td>12</td>
<td>57</td>
<td>18.98</td>
<td>12</td>
<td>12</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>23</td>
<td>19.41</td>
<td>6</td>
<td>12</td>
<td>12</td>
<td>56</td>
<td>19.43</td>
<td>12</td>
<td>12</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>24</td>
<td>19.86</td>
<td>6</td>
<td>12</td>
<td>12</td>
<td>55</td>
<td>19.88</td>
<td>12</td>
<td>12</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>25</td>
<td>20.31</td>
<td>7</td>
<td>13</td>
<td>13</td>
<td>54</td>
<td>20.33</td>
<td>13</td>
<td>13</td>
<td>24</td>
<td>26</td>
</tr>
<tr>
<td>26</td>
<td>20.76</td>
<td>7</td>
<td>13</td>
<td>13</td>
<td>53</td>
<td>20.78</td>
<td>13</td>
<td>13</td>
<td>24</td>
<td>26</td>
</tr>
<tr>
<td>27</td>
<td>21.21</td>
<td>7</td>
<td>13</td>
<td>13</td>
<td>52</td>
<td>21.23</td>
<td>13</td>
<td>13</td>
<td>24</td>
<td>26</td>
</tr>
<tr>
<td>28</td>
<td>21.66</td>
<td>7</td>
<td>13</td>
<td>13</td>
<td>51</td>
<td>21.68</td>
<td>14</td>
<td>14</td>
<td>24</td>
<td>27</td>
</tr>
<tr>
<td>29</td>
<td>22.11</td>
<td>7</td>
<td>14</td>
<td>14</td>
<td>50</td>
<td>22.13</td>
<td>14</td>
<td>14</td>
<td>24</td>
<td>28</td>
</tr>
<tr>
<td>30</td>
<td>22.56</td>
<td>7</td>
<td>14</td>
<td>14</td>
<td>49</td>
<td>22.58</td>
<td>14</td>
<td>14</td>
<td>24</td>
<td>28</td>
</tr>
<tr>
<td>31</td>
<td>22.92</td>
<td>7</td>
<td>14</td>
<td>14</td>
<td>48</td>
<td>23.03</td>
<td>14</td>
<td>14</td>
<td>24</td>
<td>28</td>
</tr>
<tr>
<td>32</td>
<td>23.26</td>
<td>7</td>
<td>14</td>
<td>14</td>
<td>47</td>
<td>23.36</td>
<td>15</td>
<td>15</td>
<td>24</td>
<td>29</td>
</tr>
<tr>
<td>33</td>
<td>23.59</td>
<td>8</td>
<td>15</td>
<td>15</td>
<td>46</td>
<td>23.70</td>
<td>15</td>
<td>15</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>34</td>
<td>23.93</td>
<td>8</td>
<td>15</td>
<td>15</td>
<td>45</td>
<td>24.04</td>
<td>15</td>
<td>15</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>35</td>
<td>24.27</td>
<td>8</td>
<td>15</td>
<td>15</td>
<td>44</td>
<td>24.40</td>
<td>15</td>
<td>15</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>COLUMN NO.</td>
<td>WALL HEIGHT (FT)</td>
<td>0.3H MIN OVERLAP (FT)</td>
<td>0.6H MIN REINFORCING LENGTH (FT)</td>
<td>BACK-TO-BACK REINFORCING LENGTH (FT)</td>
<td>WALL 4C REQUIRED TOTAL LENGTH INCLUDING OVERLAP (FT)</td>
<td>WALL 4D PROVIDED TOTAL LENGTH INCLUDING OVERLAP (FT)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-----------</td>
<td>-----------------</td>
<td>-----------------------</td>
<td>----------------------------------</td>
<td>--------------------------------------</td>
<td>-----------------------------------------------------</td>
<td>-----------------------------------------------------</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>34</td>
<td>18.30</td>
<td>6</td>
<td>11</td>
<td>11</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>45</td>
<td>22.86</td>
<td>14</td>
<td>14</td>
<td>23</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
PROJECT SPECIFIC CALCULATIONS

BACK-TO-BACK WALL - SECTION BEARING STRESS CALCULATIONS

SEGMENTAL CONCRETE PANEL MSE RETAINING WALL

CAPE COD RAIL TRAIL EXTENSION – PHASE 1

FEDERAL AID PROJECT NO. CM/TAP/TL-002S(758)X

PROJECT FILE NO. 604488

BARNSTABLE COUNTY, MA

PREPARED FOR:

LAWRENCE-LYNCH CORP.

BIG R BRIDGE PROJECT NO. VW14-02388
### External And Internal Stability Calculation Summary

**MSE - Pro**  
Mechanically Stabilized Earth Retaining Structures  
EXTERNAL AND STABILITY ANALYSIS SUMMARY

#### SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

**Project Information**
- **VAWS #:** VW14-02388  
- **Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1  
- **Location:** Barnstable County, MA  
- **Project #:** CM/TAP/TI-002S(758)X  
- **Design Engineer:** 
- **Wall Name:** Back-to-Back Bearing Pressure Calculation - Wall 1

<table>
<thead>
<tr>
<th>Rec #</th>
<th>H max (ft)</th>
<th>SR Ratio</th>
<th>B min (ft)</th>
<th>Actual B (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>( \beta ) (deg)</th>
<th>( \beta_1 ) (deg)</th>
<th>Live Load Surcharge (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>25.650</td>
<td>0.62</td>
<td>16.000</td>
<td>17.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>24.850</td>
<td>0.60</td>
<td>15.000</td>
<td>16.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>3</td>
<td>24.750</td>
<td>0.61</td>
<td>15.000</td>
<td>16.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>4</td>
<td>23.400</td>
<td>0.60</td>
<td>14.000</td>
<td>15.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>5</td>
<td>22.950</td>
<td>0.61</td>
<td>14.000</td>
<td>15.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>6</td>
<td>21.600</td>
<td>0.60</td>
<td>13.000</td>
<td>14.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>7</td>
<td>19.980</td>
<td>0.60</td>
<td>12.000</td>
<td>13.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>8</td>
<td>19.980</td>
<td>0.60</td>
<td>12.000</td>
<td>13.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>9</td>
<td>18.000</td>
<td>0.61</td>
<td>11.000</td>
<td>12.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>10</td>
<td>16.650</td>
<td>0.66</td>
<td>11.000</td>
<td>12.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

#### Material Strength Parameters

<table>
<thead>
<tr>
<th></th>
<th>Unit Weight of Reinforced Soil Pullout (kcf)</th>
<th>Unit Weight of Reinforced Soil Rupture (kcf)</th>
<th>Unit Weight of Retained Soil (kcf)</th>
<th>Unit Weight of Retained Soil (deg)</th>
<th>Cohesion of Retained Soil (kcf)</th>
<th>Cohesion of Retained Soil (deg)</th>
<th>Unit Weight of Foundation Soil (kcf)</th>
<th>Unit Weight of Foundation Soil (deg)</th>
<th>Cohesion of Foundation Soil Sliding (ksf)</th>
<th>Cohesion of Foundation Soil Bearing (ksf)</th>
<th>Bearing Capacity Factor Slab</th>
<th>Bearing Capacity Factor Cohesion</th>
<th>Bearing Capacity Factor Over Burden</th>
<th>Depth to Water Table (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective Stress</td>
<td>0.135</td>
<td>0.135</td>
<td>34.0</td>
<td>0.001</td>
<td>89.5</td>
<td>0.000</td>
<td>0.120</td>
<td>30.0</td>
<td>0.000</td>
<td>18.40</td>
<td>30.14</td>
<td>22.40</td>
<td>0.00</td>
<td></td>
</tr>
</tbody>
</table>

Project File Name: P:\AA_Project Files\14 Projects\VW14-02388 Cape Cod Rail Trail Extension - Dennis Yarmouth, MA\Engineering\CALCULATIONS\SUBMITTAL\VW14-02388 MSE-Pro Submittal Calc Back to Back BP_W1.mse  
Thu. December 24, 2015 @ 10:41 AM
**SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP**

**Project Information**
- VAWS #: VW14-02388
- Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1
- Location: Barnstable County, MA
- Project #: CM/TAP/TI-002S(758)X
- Design Engineer: cms
- Wall Name: Back-to-Back Bearing Pressure Calculation - Wall 1

**Effective Stress Evaluation**

<table>
<thead>
<tr>
<th>Rec #</th>
<th>Applied Bearing Pressure (ksf)</th>
<th>Ultimate Bearing Pressure (ksf)</th>
<th>Factor of Safety for Bearing Capacity</th>
<th>Factor of Safety for Sliding</th>
<th>Factor of Safety for Overturning</th>
<th>Active Earth Pressure Coefficient</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.11</td>
<td>10.75</td>
<td>2.10</td>
<td>193606.52</td>
<td>232010.60</td>
<td>0.000</td>
<td>0.500</td>
</tr>
<tr>
<td>2</td>
<td>4.97</td>
<td>10.08</td>
<td>2.03</td>
<td>181744.86</td>
<td>211247.78</td>
<td>0.000</td>
<td>0.500</td>
</tr>
<tr>
<td>3</td>
<td>4.95</td>
<td>10.08</td>
<td>2.04</td>
<td>181774.75</td>
<td>212200.38</td>
<td>0.000</td>
<td>0.500</td>
</tr>
<tr>
<td>4</td>
<td>4.70</td>
<td>9.41</td>
<td>2.00</td>
<td>170033.93</td>
<td>196807.14</td>
<td>0.000</td>
<td>0.500</td>
</tr>
<tr>
<td>5</td>
<td>4.62</td>
<td>9.41</td>
<td>2.04</td>
<td>170160.13</td>
<td>201132.93</td>
<td>0.000</td>
<td>0.500</td>
</tr>
<tr>
<td>6</td>
<td>4.37</td>
<td>8.74</td>
<td>2.00</td>
<td>158358.45</td>
<td>185634.60</td>
<td>0.000</td>
<td>0.500</td>
</tr>
<tr>
<td>7</td>
<td>4.08</td>
<td>8.06</td>
<td>1.98</td>
<td>146569.54</td>
<td>172697.56</td>
<td>0.000</td>
<td>0.500</td>
</tr>
<tr>
<td>8</td>
<td>4.08</td>
<td>8.06</td>
<td>1.98</td>
<td>146569.54</td>
<td>172697.56</td>
<td>0.000</td>
<td>0.500</td>
</tr>
<tr>
<td>9</td>
<td>3.72</td>
<td>7.39</td>
<td>1.99</td>
<td>134797.80</td>
<td>163338.30</td>
<td>0.000</td>
<td>0.500</td>
</tr>
<tr>
<td>10</td>
<td>3.47</td>
<td>7.39</td>
<td>2.13</td>
<td>135101.10</td>
<td>178536.60</td>
<td>0.000</td>
<td>0.500</td>
</tr>
</tbody>
</table>
## MSE - Pro
Mechanically Stabilized Earth Retaining Structures
EXTERNAL AND STABILITY ANALYSIS SUMMARY

### SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

### Project Information
- **VAWS #:** VW14-02388
- **Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1
- **Location:** Barnstable County, MA
- **Project #:** CM/TAP/TI-002S(758)X
- **Design Engineer:** cms
- **Wall Name:** Back-to-Back Bearing Pressure Calculation - Wall 2

### Structure Parameters

<table>
<thead>
<tr>
<th>Rec #</th>
<th>H max (ft)</th>
<th>SR Ratio</th>
<th>B min (ft)</th>
<th>Actual B (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>β (deg)</th>
<th>β† (deg)</th>
<th>Live Load Surcharge (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>28.770</td>
<td>0.56</td>
<td>16.000</td>
<td>17.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.25</td>
</tr>
<tr>
<td>2</td>
<td>24.840</td>
<td>0.60</td>
<td>15.000</td>
<td>16.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.25</td>
</tr>
</tbody>
</table>

### Material Strength Parameters

|                | Unit Weight of Reinforced Soil Pullout (kcf) | Unit Weight of Reinforced Soil Rupture (kcf) | Unit Weight of Retained Soil (kcf) | Internal Friction Angle of Retained Soil (deg) | Cohesion of Retained Soil (kcf) | Unit Weight of Foundation Soil (kcf) | Internal Friction Angle of Foundation Soil (deg) | Cohesion of Foundation Soil (kcf) | Cohesion of Foundation Soil Sliding (kcf) | Bearing Capacity Factor Surchage | Bearing Capacity Factor Cohesion | Bearing Capacity Factor Over Burden | Depth to Water Table (ft) |
|----------------|---------------------------------------------|---------------------------------------------|-----------------------------------|---------------------------------------------|-------------------------------|--------------------------------------|-----------------------------------------------|-----------------------------------|----------------------------------|---------------------------------|------------------------------|-------------------------------|-----------------------------|-----------------------------|
| Effective Stress | 0.135                                       | 0.135                                       | 34.0                              | 0.001                                       | 89.5                          | 0.000                                | 0.120                                         | 30.0                              | 0.000                           | 18.40                           | 30.14                        | 22.40                         | 0.00                         |

### Effective Stress Evaluation

<table>
<thead>
<tr>
<th>Rec #</th>
<th>Applied Bearing Pressure (ksf)</th>
<th>Ultimate Bearing Pressure (ksf)</th>
<th>Factor of Safety for Bearing Capacity</th>
<th>Factor of Safety for Sliding</th>
<th>Factor of Safety for Overturning</th>
<th>Active Earth Pressure Coefficient</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.68</td>
<td>10.75</td>
<td>1.89</td>
<td>192619.67</td>
<td>204222.72</td>
<td>0.000</td>
<td>0.500</td>
</tr>
<tr>
<td>2</td>
<td>4.96</td>
<td>10.08</td>
<td>2.03</td>
<td>181747.85</td>
<td>211342.67</td>
<td>0.000</td>
<td>0.500</td>
</tr>
</tbody>
</table>
# External And Internal Stability Calculation Summary

## MSE - Pro
Mechanically Stabilized Earth Retaining Structures
EXTERNAL AND STABILITY ANALYSIS SUMMARY

### SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

#### Project Information

- **VAWS #:** VW14-02388
- **Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1
- **Location:** Barnstable County, MA
- **Project #:** CM/TAP/TI-002S(758)X
- **Design Engineer:** cm
- **Wall Name:** Back-to-Back Bearing Pressure Calculation - Wall 3

#### Structure Parameters

<table>
<thead>
<tr>
<th>Rec #</th>
<th>H max (ft)</th>
<th>SR Ratio</th>
<th>B min (ft)</th>
<th>Actual B (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>( \beta ) (deg)</th>
<th>( \beta' ) (deg)</th>
<th>Live Load Surcharge (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>24.040</td>
<td>0.62</td>
<td>15.000</td>
<td>16.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>23.360</td>
<td>0.60</td>
<td>14.000</td>
<td>15.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>3</td>
<td>23.030</td>
<td>0.61</td>
<td>14.000</td>
<td>15.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>4</td>
<td>21.680</td>
<td>0.60</td>
<td>13.000</td>
<td>14.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>5</td>
<td>21.230</td>
<td>0.61</td>
<td>13.000</td>
<td>14.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>6</td>
<td>19.880</td>
<td>0.60</td>
<td>12.000</td>
<td>13.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>7</td>
<td>18.080</td>
<td>0.61</td>
<td>11.000</td>
<td>12.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>8</td>
<td>16.280</td>
<td>0.61</td>
<td>10.000</td>
<td>11.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>9</td>
<td>13.130</td>
<td>0.69</td>
<td>9.000</td>
<td>10.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

#### Material Strength Parameters

<table>
<thead>
<tr>
<th></th>
<th>Unit Weight of Reinforced Soil Pullout (kcf)</th>
<th>Unit Weight of Reinforced Soil Rupture (kcf)</th>
<th>Internal Friction Angle of Reinforced Soil (deg)</th>
<th>Unit Weight of Retained Soil</th>
<th>Cohesion of Retained Soil (ksf)</th>
<th>Unit Weight of Foundation Soil</th>
<th>Internal Friction Angle of Foundation Soil (deg)</th>
<th>Cohesion of Foundation Soil Sliding (ksf)</th>
<th>Cohesion of Foundation Soil Bearing (ksf)</th>
<th>Bearing Capacity Factor Surchage</th>
<th>Bearing Capacity Factor Cohesion</th>
<th>Bearing Capacity Factor Over Burden</th>
<th>Depth to Water Table (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective Stress</td>
<td>0.135</td>
<td>0.135</td>
<td>34.0</td>
<td>0.001</td>
<td>89.5</td>
<td>0.000</td>
<td>0.120</td>
<td>30.0</td>
<td>0.000</td>
<td>0.000</td>
<td>18.40</td>
<td>30.14</td>
<td>22.40</td>
</tr>
</tbody>
</table>
### SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

#### Project Information
- **VAWS #:** VW14-02388
- **Name:** CAPE COD RAIL TRAIL EXTENSION - PHASE 1
- **Location:** Barnstable County, MA
- **Project #:** CM/TAP/TI-002S(758)X
- **Design Engineer:** CMS
- **Wall Name:** Back-to-Back Bearing Pressure Calculation - Wall 3

#### Effective Stress Evaluation

<table>
<thead>
<tr>
<th>Rec #</th>
<th>Applied Bearing Pressure (ksf)</th>
<th>Ultimate Bearing Pressure (ksf)</th>
<th>Factor of Safety for Bearing Capacity</th>
<th>Factor of Safety for Sliding</th>
<th>Factor of Safety for Overturning</th>
<th>Active Earth Pressure Coefficient</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.82</td>
<td>10.08</td>
<td>2.09</td>
<td>181987.24</td>
<td>219210.99</td>
<td>0.000</td>
<td>0.500</td>
</tr>
<tr>
<td>2</td>
<td>4.69</td>
<td>9.41</td>
<td>2.00</td>
<td>170045.14</td>
<td>197184.32</td>
<td>0.000</td>
<td>0.500</td>
</tr>
<tr>
<td>3</td>
<td>4.63</td>
<td>9.41</td>
<td>2.03</td>
<td>170137.68</td>
<td>200350.47</td>
<td>0.000</td>
<td>0.500</td>
</tr>
<tr>
<td>4</td>
<td>4.39</td>
<td>8.74</td>
<td>1.99</td>
<td>158337.51</td>
<td>184864.85</td>
<td>0.000</td>
<td>0.500</td>
</tr>
<tr>
<td>5</td>
<td>4.31</td>
<td>8.74</td>
<td>2.03</td>
<td>158456.37</td>
<td>189277.21</td>
<td>0.000</td>
<td>0.500</td>
</tr>
<tr>
<td>6</td>
<td>4.06</td>
<td>8.06</td>
<td>1.99</td>
<td>146593.84</td>
<td>173679.32</td>
<td>0.000</td>
<td>0.500</td>
</tr>
<tr>
<td>7</td>
<td>3.73</td>
<td>7.39</td>
<td>1.98</td>
<td>134779.87</td>
<td>162516.34</td>
<td>0.000</td>
<td>0.500</td>
</tr>
<tr>
<td>8</td>
<td>3.40</td>
<td>6.72</td>
<td>1.97</td>
<td>122894.97</td>
<td>151399.33</td>
<td>0.000</td>
<td>0.500</td>
</tr>
<tr>
<td>9</td>
<td>2.83</td>
<td>6.05</td>
<td>2.14</td>
<td>111189.60</td>
<td>157297.97</td>
<td>0.000</td>
<td>0.500</td>
</tr>
</tbody>
</table>
MSE - Pro
Mechanically Stabilized Earth Retaining Structures
EXTERNAL AND STABILITY ANALYSIS SUMMARY

SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

Project Information
VAWS #: VW14-02388
Name: CAPE COD RAIL TRAIL EXTENSION - PHASE 1
Location: Barnstable County, MA
Project #: CM/TAP/TI-002S(758)X
Design Engineer: cms
Wall Name: Back-to-Back Bearing Pressure Calculation - Wall 4

Structure Parameters
<table>
<thead>
<tr>
<th>Rec #</th>
<th>H max (ft)</th>
<th>SR Ratio</th>
<th>B min (ft)</th>
<th>Actual B (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>β (deg)</th>
<th>β ′ (deg)</th>
<th>Live Load Surcharge (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>24.040</td>
<td>0.62</td>
<td>15.000</td>
<td>16.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.25</td>
</tr>
</tbody>
</table>

Material Strength Parameters

<table>
<thead>
<tr>
<th></th>
<th>Unit Weight of Reinforced Soil Pullout (kcf)</th>
<th>Unit Weight of Reinforced Soil Rupture (kcf)</th>
<th>Internal Friction Angle of Reinforced Soil (deg)</th>
<th>Unit Weight of Retained Soil (kcf)</th>
<th>Internal Friction Angle of Retained Soil (deg)</th>
<th>Cohesion of Retained Soil (kcf)</th>
<th>Unit Weight of Foundation Soil (kcf)</th>
<th>Internal Friction Angle of Foundation Soil (deg)</th>
<th>Cohesion of Foundation Soil Sliding (kcf)</th>
<th>Cohesion of Foundation Soil Bearing (kcf)</th>
<th>Bearing Capacity Factor Surcharge</th>
<th>Bearing Capacity Factor Cohesion</th>
<th>Bearing Capacity Factor Over Burden</th>
<th>Depth to Water Table (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective Stress</td>
<td>0.135</td>
<td>0.135</td>
<td>34.0</td>
<td>0.001</td>
<td>89.5</td>
<td>0.000</td>
<td>0.120</td>
<td>30.0</td>
<td>0.000</td>
<td>18.40</td>
<td>30.14</td>
<td>22.40</td>
<td>0.00</td>
<td></td>
</tr>
</tbody>
</table>

Effective Stress Evaluation

<table>
<thead>
<tr>
<th>Rec #</th>
<th>Applied Bearing Pressure (ksf)</th>
<th>Ultimate Bearing Pressure (ksf)</th>
<th>Factor of Safety for Bearing Capacity</th>
<th>Factor of Safety for Sliding</th>
<th>Factor of Safety for Overturning</th>
<th>Active Earth Pressure Coefficient</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.82</td>
<td>10.08</td>
<td>2.09</td>
<td>181987.24</td>
<td>219210.99</td>
<td>0.000</td>
<td>0.500</td>
</tr>
</tbody>
</table>

Project File Name: P:\AA_Project File\14 Projects\VW14-02388 Cape Cod Rail Trail Extension - Dennis Yarmouth, MA\Engineering\CALCULATIONS\SUBMITTAL\VW14-02388 MSE-Pro Submittal Calc Back to Back BP_W4.mse
Thu. December 24, 2015 @ 10:39 AM
PROJECT SPECIFIC
CALCULATIONS
TRUE BRIDGE ABUTMENT LOADING

SEGMENTAL CONCRETE PANEL MSE RETAINING WALL
CAPE COD RAIL TRAIL EXTENSION – PHASE 1
FEDERAL AID PROJECT NO. CM/TAP/TI-002S(758)X
PROJECT FILE NO. 604488
BARNSTABLE COUNTY, MA

PREPARED FOR:
LAWRENCE-LYNCH CORP.

BIG R BRIDGE PROJECT NO. VW14-02388
## CALCULATION SUMMARY

**Project Name:** Cape Cod Rail Trail – MSE Abutment RW1  
**Project Location:** Cape Cod, MA  
**Project Number:** 2014-02388  
**Design Code:** AASHTO 2012  
**Design Method:** LRFD 7th Edition  
**DSGN By:** TPT  
**Date:** 12-10-15  
**CHCK By:** CMS  
**Date:** 12-10-15

### Structure Parameters

- **Wall Design Height:** \( H = 19.51 \text{ ft} \)
- **Soil Reinforcing Length:** \( L = 19.00 \text{ ft} \)
- **Surcharge Height:** \( S = 8.42 \text{ ft} \)
- **Distance to Full Height:** \( X_s = 8.87 \text{ ft} \)

### Soil Strength Parameters

- **Reinforced Soil mass**  
  - Unit Weight: \( \gamma_r = 135 \text{pcf} \)  
  - Internal Friction Angle: \( \phi_r = 34 \text{deg} \)
- **Retained Soil Mass**  
  - Unit Weight: \( \gamma_f = 135 \text{pcf} \)  
  - Internal Friction Angle: \( \phi_f = 34 \text{deg} \)
- **Foundation**  
  - Internal Friction Angle: \( \phi_{fd} = 30 \text{deg} \)

### Static External Stability Capacity Demand Ratios

**CDR SLIDING**

- **CDRMSE_Sliding_Max = 2.63**  
- **CDRMSE_Sliding_Min = 2.92**  
- **CDRMSE_Sliding_Critical = 1.94**

**LIMITING ECCENTRICITY**

- \( e_{LBP_{\text{Min}}} = 0.75 \text{ ft} \)  
  - \( \frac{e_{LBP_{\text{Max}}}}{L} = 0.04 \)  
  - \( \frac{e_{LBP_{\text{Min}}}}{L} = 0.04 \)
- \( e_{LBP_{\text{Max}}} = 0.75 \text{ ft} \)  
  - \( \frac{e_{LBP_{\text{Critical}}}}{L} = 0.05 \)
- **CDROT_min :=**  
  - \( \frac{M_{RB_{\text{Min}}}}{M_{OB_{\text{Min}}}} = 4.81 \)
- **CDROT_max :=**  
  - \( \frac{M_{RB_{\text{Max}}}}{M_{OB_{\text{Max}}}} = 4.48 \)
- **CDROT_critical :=**  
  - \( \frac{M_{RB_{\text{Critical}}}}{M_{OB_{\text{Critical}}}} = 3.31 \)

### BEARING RESISTANCE

- \( \sigma_{BP_{\text{Max}}} = 7.02 \text{ksf} \)  
- \( \sigma_{BP_{\text{Min}}} = 5.21 \text{ksf} \)  
- \( \sigma_{V_{\text{Critical}}} = 5.84 \text{ksf} \)  
- \( \sigma_{BP} = 4.98 \text{ksf} \)
## Capacity Demand Ratio for Rupture and Pullout

<table>
<thead>
<tr>
<th>Depth From Top of Wall To Layer (d_i)</th>
<th>n_SR(d_i)</th>
<th>CDR_{rupture}(d_i)</th>
<th>CDR_{pullout}(d_i)</th>
<th>Internal Earth Coefficient (K_i(d_i))</th>
<th>d_FP(d_i)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 ft</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1 ft</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>2 ft</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>3 ft</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>4 ft</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>5 ft</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>6 ft</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>6</td>
</tr>
</tbody>
</table>

\[
(d_i) = n_{SR}(d_i) \quad \text{CDR}_{rupture}(d_i) = \text{CDR}_{pullout}(d_i) = (K_i(d_i)) = d_FP(d_i) = 0 \quad \text{ft}
\]

### Grid-Strip Configuration

**GS-11 = W11.0 x W11.0 - 2" x 12"**
CALCULATION SUMMARY

Project Name: Cape Cod Rail Trail – MSE Abutment RW2
Project Location: Cape Cod, MA
Project Number: 2014-02388
Design Code: AASHTO 2012
Design Method: LRFD 7th Edition

<table>
<thead>
<tr>
<th>Structure Parameters</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Design Height:</td>
<td>H = 22.75 ft</td>
</tr>
<tr>
<td>Soil Reinforcing Length:</td>
<td>L = 21.00 ft</td>
</tr>
<tr>
<td>Surcharge Height:</td>
<td>S = 8.70 ft</td>
</tr>
<tr>
<td>Distance to Full Height:</td>
<td>Xs = 8.87 ft</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Soil Strength Parameters</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced Soil mass</td>
<td>Unit Weight: γ_r = 135·pcf</td>
</tr>
<tr>
<td>Retained Soil Mass</td>
<td>Unit Weight: γ_f = 135·pcf</td>
</tr>
<tr>
<td>Foundation</td>
<td>Internal Friction Angle φ_f = 34·deg</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Static External Stability Capacity Demand Ratios</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>CDR SLIDING</td>
<td></td>
</tr>
<tr>
<td>CDRMSE_Sliding_Max = 2.55</td>
<td></td>
</tr>
<tr>
<td>CDRMSE_Sliding_Min = 2.86</td>
<td></td>
</tr>
<tr>
<td>CDRMSE_Sliding_Critical = 1.88</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LIMITING ECCENTRICITY</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>eL_BP_Min = 0.77 ft</td>
<td></td>
</tr>
<tr>
<td>eL_BP_Max = 0.76 ft</td>
<td></td>
</tr>
<tr>
<td>eL_BP_Critical = 0.88 ft</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>BEARING RESISTANCE</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>σ_BP_Max = 7.93·ksf</td>
<td></td>
</tr>
<tr>
<td>σ_BP_Min = 5.85·ksf</td>
<td></td>
</tr>
<tr>
<td>σ_V_Critical = 6.62·ksf</td>
<td></td>
</tr>
<tr>
<td>σ_BP = 5.63·ksf</td>
<td></td>
</tr>
</tbody>
</table>
Capacity Demand Ratio for Rupture and Pullout

<table>
<thead>
<tr>
<th>Depth From Top of Wall To Layer (ft)</th>
<th>Number of Soil Reinforcing Per Row</th>
<th>CDR for Rupture</th>
<th>CDR for Pullout</th>
<th>Internal Earth Coefficient</th>
<th>Depth From Top of Failure Plane To Layer (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0.20</td>
<td>0.9</td>
<td>1.09</td>
<td>0.135</td>
<td>0.4</td>
<td>0.170</td>
</tr>
<tr>
<td>0.45</td>
<td>0.7</td>
<td>1.13</td>
<td>1.59</td>
<td>0.39</td>
<td>0.37</td>
</tr>
<tr>
<td>0.70</td>
<td>0.7</td>
<td>1.17</td>
<td>1.85</td>
<td>0.37</td>
<td>0.37</td>
</tr>
<tr>
<td>0.90</td>
<td>0.6</td>
<td>1.03</td>
<td>1.90</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>1.20</td>
<td>0.6</td>
<td>1.05</td>
<td>2.23</td>
<td>0.34</td>
<td>0.34</td>
</tr>
<tr>
<td>1.45</td>
<td>0.6</td>
<td>1.02</td>
<td>2.65</td>
<td>0.34</td>
<td>0.34</td>
</tr>
<tr>
<td>1.70</td>
<td>0.7</td>
<td>1.11</td>
<td>3.47</td>
<td>0.34</td>
<td>0.34</td>
</tr>
<tr>
<td>1.95</td>
<td>0.7</td>
<td>1.03</td>
<td>3.85</td>
<td>0.34</td>
<td>0.34</td>
</tr>
</tbody>
</table>

**Grid-Strip Configuration**

GS-11 = W11.0 x W11.0 - 2” x 12”
# CALCULATION SUMMARY

<table>
<thead>
<tr>
<th>Project Name:</th>
<th>Cape Cod Rail Trail – MSE Abutment RW3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Location:</td>
<td>Cape Cod, MA</td>
</tr>
<tr>
<td>Project Number:</td>
<td>2014-02388</td>
</tr>
<tr>
<td>Design Code:</td>
<td>AASHTO 2012</td>
</tr>
<tr>
<td>Design Method:</td>
<td>LRFD 7th Edition</td>
</tr>
<tr>
<td>DSGN By:</td>
<td>TPT</td>
</tr>
<tr>
<td>Date:</td>
<td>12-10-15</td>
</tr>
<tr>
<td>CHCK By:</td>
<td>CMS</td>
</tr>
<tr>
<td>Date:</td>
<td>12-10-15</td>
</tr>
</tbody>
</table>

## Structure Parameters

- **Wall Design Height:** \( H = 18.16 \text{ ft} \)
- **Soil Reinforcing Length:** \( L = 18.00 \text{ ft} \)
- **Surcharge Height:** \( S = 8.55 \text{ ft} \)
- **Distance to Full Height:** \( X_s = 8.87 \text{ ft} \)

## Soil Strength Parameters

- **Reinforced Soil mass Unit Weight:** \( \gamma_r = 135 \cdot \text{pcf} \)
- **Internal Friction Angle:** \( \phi_r = 34 \cdot \text{deg} \)
- **Retained Soil Mass Unit Weight:** \( \gamma_f = 135 \cdot \text{pcf} \)
- **Internal Friction Angle:** \( \phi_f = 34 \cdot \text{deg} \)
- **Foundation Internal Friction Angle:** \( \phi_{fd} = 30 \cdot \text{deg} \)

## Static External Stability Capacity Demand Ratios

### CDR SLIDING

- **CDRMSE_Sliding_Max:** 2.64
- **CDRMSE_Sliding_Min:** 2.92
- **CDRMSE_Sliding_Critical:** 1.94

### LIMITING ECCENTRICITY

- **\( e \text{L}_{BP_{-Min}} = 0.79 \text{ ft} \)**
  - \( \frac{eL_{BP_{-Max}}}{L} = 0.04 \)
  - **CDROT_{min} := \frac{M_{RB_{-Min}}}{M_{oB_{-Min}}} = 4.73**
- **\( e \text{L}_{BP_{-Max}} = 0.78 \text{ ft} \)**
  - \( \frac{eL_{BP_{-Min}}}{L} = 0.04 \)
  - **CDROT_{max} := \frac{M_{RB_{-Max}}}{M_{oB_{-Max}}} = 4.44**
- **\( e \text{L}_{BP_{-Critical}} = 0.89 \text{ ft} \)**
  - \( \frac{eL_{BP_{-Critical}}}{L} = 0.05 \)
  - **CDROT_{critical} := \frac{M_{RB_{-Critical}}}{M_{oB_{-Critical}}} = 3.28**

### BEARING RESISTANCE

- **\( \sigma_{BP_{-Max}} = 6.76 \cdot \text{ksf} \)**
- **\( \sigma_{BP_{-Min}} = 5.03 \cdot \text{ksf} \)**
- **\( \sigma_{V_{-Critical}} = 5.64 \cdot \text{ksf} \)**
- **\( \sigma_{BP} = 4.80 \cdot \text{ksf} \)**

---

650 Justice Lane
Mansfield, TX 76063

888-280-9858

34 of 42

VISTAWALL SYSTEMS
## Capacity Demand Ratio for Rupture and Pullout

<table>
<thead>
<tr>
<th>d_i</th>
<th>nSR(d_i)</th>
<th>CDR(_{\text{rupture}})(d_i)</th>
<th>CDR(_{\text{po}})(d_i)</th>
<th>(K_i(d_i))</th>
<th>d_f(p)(d_i)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>(0, 0)</td>
</tr>
<tr>
<td>1</td>
<td>10</td>
<td>1.02</td>
<td>1.03</td>
<td>0.40</td>
<td>(0, 10.96)</td>
</tr>
<tr>
<td>2</td>
<td>7</td>
<td>1.09</td>
<td>1.43</td>
<td>0.39</td>
<td>(1, 13.46)</td>
</tr>
<tr>
<td>3</td>
<td>6</td>
<td>1.13</td>
<td>1.85</td>
<td>0.35</td>
<td>(2, 15.96)</td>
</tr>
<tr>
<td>4</td>
<td>6</td>
<td>1.02</td>
<td>2.19</td>
<td>0.34</td>
<td>(3, 18.46)</td>
</tr>
<tr>
<td>5</td>
<td>7</td>
<td>1.14</td>
<td>3.01</td>
<td>0.34</td>
<td>(4, 20.96)</td>
</tr>
</tbody>
</table>

### Grid-Strip Configuration

**GS-11 = W11.0 x W11.0 - 2" x 12"**
CALCULATION SUMMARY

Project Name: Cape Cod Rail Trail – MSE Abutment  
Project Location: Cape Cod, MA  
Project Number: 2014-02388  
Design Code: AASHTO 2012  
Design Method: LRFD 7th Edition

**Structure Parameters**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Design Height:</td>
<td>H = 17.40 ft</td>
</tr>
<tr>
<td>Soil Reinforcing Length:</td>
<td>L = 17.00 ft</td>
</tr>
<tr>
<td>Surcharge Height:</td>
<td>S = 8.64 ft</td>
</tr>
<tr>
<td>Distance to Full Height:</td>
<td>Xs = 8.87 ft</td>
</tr>
</tbody>
</table>

**Soil Strength Parameters**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced Soil mass Unit Weight:</td>
<td>γr = 135·pcf</td>
</tr>
<tr>
<td>Internal Friction Angle:</td>
<td>φr = 34·deg</td>
</tr>
<tr>
<td>Retained Soil Mass Unit Weight:</td>
<td>γf = 135·pcf</td>
</tr>
<tr>
<td>Internal Friction Angle:</td>
<td>φf = 34·deg</td>
</tr>
<tr>
<td>Foundation Internal Friction Angle:</td>
<td>φfd = 30·deg</td>
</tr>
</tbody>
</table>

**Static External Stability Capacity Demand Ratios**

**CDR SLIDING**

<table>
<thead>
<tr>
<th>Ratio</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>CDRMSE_Sliding_Max</td>
<td>2.57</td>
</tr>
<tr>
<td>CDRMSE_Sliding_Min</td>
<td>2.83</td>
</tr>
<tr>
<td>CDRMSE_Sliding_Critical</td>
<td>1.89</td>
</tr>
</tbody>
</table>

**LIMITING ECCENTRICITY**

<table>
<thead>
<tr>
<th>Ratio</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>eL_BP_Min</td>
<td>0.79 ft</td>
</tr>
<tr>
<td>eL_BP_Max/L</td>
<td>0.05</td>
</tr>
<tr>
<td>CDR_OT_min</td>
<td>M_RB_Min</td>
</tr>
<tr>
<td></td>
<td>M_oB_Min</td>
</tr>
<tr>
<td></td>
<td>4.46</td>
</tr>
<tr>
<td>eL_BP_Max</td>
<td>0.78 ft</td>
</tr>
<tr>
<td>eL_BP_Min/L</td>
<td>0.05</td>
</tr>
<tr>
<td>CDR_OT_max</td>
<td>M_RB_Max</td>
</tr>
<tr>
<td></td>
<td>M_oB_Max</td>
</tr>
<tr>
<td></td>
<td>4.20</td>
</tr>
<tr>
<td>eL_BP_Critical</td>
<td>0.89 ft</td>
</tr>
<tr>
<td>eL_BP_Min/L</td>
<td>0.05</td>
</tr>
<tr>
<td>CDR_OT_critical</td>
<td>M_RB_Critical</td>
</tr>
<tr>
<td></td>
<td>M_oB_Critical</td>
</tr>
<tr>
<td></td>
<td>3.10</td>
</tr>
</tbody>
</table>

**BEARING RESISTANCE**

<table>
<thead>
<tr>
<th>Ratio</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>σ_BP_Max</td>
<td>6.72·ksf</td>
</tr>
<tr>
<td>σ_BP_Min</td>
<td>4.99·ksf</td>
</tr>
<tr>
<td>σ_V_Critical</td>
<td>5.65·ksf</td>
</tr>
<tr>
<td>σ_BP</td>
<td>4.76·ksf</td>
</tr>
</tbody>
</table>
## Capacity Demand Ratio for Rupture and Pullout

<table>
<thead>
<tr>
<th>Depth From Top of Wall To Layer</th>
<th>Number of Soil Reinforcing Per Row</th>
<th>CDR for Rupture</th>
<th>CDR for Pullout</th>
<th>Internal Earth Coefficient</th>
<th>Depth From Top of Failure Plane To Layer</th>
</tr>
</thead>
<tbody>
<tr>
<td>( d_i ) = 0 ft</td>
<td>( n_{SR}(d_i) = 0 )</td>
<td>( CDR_{rupture}(d_i) = 0 )</td>
<td>( CDR_{po}(d_i) = 0 )</td>
<td>( K_i(d_i) = 0 )</td>
<td>( d_{fp}(d_i) = 0 ) ft</td>
</tr>
<tr>
<td>0</td>
<td>1.15</td>
<td>1.06</td>
<td>0.87</td>
<td>0.41</td>
<td>9.79</td>
</tr>
<tr>
<td>1</td>
<td>3.65</td>
<td>1.07</td>
<td>1.18</td>
<td>0.39</td>
<td>12.29</td>
</tr>
<tr>
<td>2</td>
<td>6.15</td>
<td>1.11</td>
<td>1.54</td>
<td>0.38</td>
<td>14.79</td>
</tr>
<tr>
<td>3</td>
<td>8.65</td>
<td>1.15</td>
<td>1.90</td>
<td>0.36</td>
<td>17.29</td>
</tr>
<tr>
<td>4</td>
<td>11.15</td>
<td>1.02</td>
<td>1.92</td>
<td>0.34</td>
<td>19.79</td>
</tr>
<tr>
<td>5</td>
<td>13.65</td>
<td>1.01</td>
<td>2.33</td>
<td>0.34</td>
<td>22.29</td>
</tr>
</tbody>
</table>

### Grid-Strip Configuration

**GS-11 = W11.0 x W11.0 - 2" x 12"**
CONSTRUCTION DRAWINGS

VIST-A-WALL SEGMENTAL CONCRETE PANEL MSE RETAINING WALL SYSTEM

VISTAWALL

SYSTEMS

CAPE COD RAIL TRAIL EXTENSION - PHASE 1
IN THE TOWNS OF YARMOUTH & DENNIS, BARNSTABLE COUNTY, MA.

MASSACHUSETTS DEPARTMENT OF TRANSPORTATION

FEDERAL AID PROJECT NO.: CM/TAP/11-002S(758)X
PROJECT FILE NO.: 604488

PREPARED FOR:

LAWRENCE-LYNCH CORP.
GENERAL MSE RETAINING WALL NOTES

DESIGN CRITERIA
1. The attached details are based on the assumptions that the material used in the reinforced volume, field fabrication of construction and quality of prefabricated components meet the owner specifications.
2. Minimum design requirements.
3. Reference wall construction drawings for soil characteristics of foundation and proposed backfill material, to be used in the wall design. The contractor shall verify that the select backfill material has material strength parameters in accordance with the design criteria set forth within the contract plans. This shall include the internal friction angle, unit weight and cohesion of the fill material. For design criteria of the fill material, refer to the appropriate specification for fill.
4. The owner's design engineer or equivalent for possible wall design.

INSTALLATION CRITERIA

WEAP SCALES AND LOADS

1. Internal stability
2. External stability
3. Sliding
4. Bearing pressure

LOAD & RESISTANCE FACTORS

1. In accordance with the ACI-530/SCI-530 design specifications and internal specifications.
2. The maximum allowed bearing pressure at the interface of the foundation and the select backfill material is displayed in the calculations. The bearing pressure calculated is the maximum for the length of the slope. It is the responsibility of the owner to determine the bearing pressure to be allowed for that particular job.
3. Any inadequate foundation material below the reinforced volume as determined by the engineer shall be excavated and replaced with suitable material as directed by the engineer.
4. The design engineer's calculations are based on the assumption that the owner has the basis of the foundation. The contractor is responsible for the internal stability of the structure. The internal stability design assuming foundation and slope stability is the responsibility of the owner.

WALL CONSTRUCTION

1. Walls founded on curvatures shall have their panel(s) dimensioned as a series of arcs (as dimensioned in shop drawings) in order to match the required radius. The design engineer's calculations are based on the assumption that the owner has the basis of the foundation. The contractor is responsible for the internal stability of the structure. The internal stability design assuming foundation and slope stability is the responsibility of the owner.
2. For location and alignment of the MSE structures refer to the contract plans.
3. If inadequate foundation material below the reinforced volume as determined by the engineer shall be excavated and replaced with suitable material as directed by the engineer.
4. The reference wall construction drawings for soil characteristics of foundation and proposed backfill material, to be used in the wall design. The contractor shall verify that the select backfill material has material strength parameters in accordance with the design criteria set forth within the contract plans. This shall include the internal friction angle, unit weight and cohesion of the fill material. For design criteria of the fill material, refer to the appropriate specification for fill.
5. The responsibility of the contractor to locate any fence posts prior to placing the top layer of soil reinforcement (shovel or blast) to allow clearance from the edge post. Refer to obstruction details for additional information when deflecting and shifting drain strips.
6. If existing or future structures are to be placed in the reinforced volume that interfere with the proper placement of the soil reinforcement, the contractor shall notify the engineer immediately for a course of action.
7. The design engineer's calculations are based on the assumption that the owner has the basis of the foundation. The contractor is responsible for the internal stability of the structure. The internal stability design assuming foundation and slope stability is the responsibility of the owner.

WALL CONSTRUCTION (CONTINUED)

1. External stability
2. Internal stability
3. Sliding
4. Bearing pressure

WALL CONSTRUCTION (CONTINUED)

1. The responsibility of the contractor to relocate any fence posts prior to placing the top layers of soil reinforcement (shovel or blast) to allow clearance from the edge post. Refer to obstruction details for additional information when deflecting and shifting drain strips.
2. If existing or future structures are to be placed in the reinforced volume that interfere with the proper placement of the soil reinforcement, the contractor shall notify the engineer immediately for a course of action.

WEAP SCALES AND LOADS

1. Internal stability
2. External stability
3. Sliding
4. Bearing pressure

LOAD & RESISTANCE FACTORS

1. In accordance with the ACI-530/SCI-530 design specifications and internal specifications.
2. The maximum allowed bearing pressure at the interface of the foundation and the select backfill material is displayed in the calculations. The bearing pressure calculated is the maximum for the length of the slope. It is the responsibility of the owner to determine the bearing pressure to be allowed for that particular job.
3. Any inadequate foundation material below the reinforced volume as determined by the engineer shall be excavated and replaced with suitable material as directed by the engineer.
4. The design engineer's calculations are based on the assumption that the owner has the basis of the foundation. The contractor is responsible for the internal stability of the structure. The internal stability design assuming foundation and slope stability is the responsibility of the owner.

WALL CONSTRUCTION

1. Walls founded on curvatures shall have their panel(s) dimensioned as a series of arcs (as dimensioned in shop drawings) in order to match the required radius. The design engineer's calculations are based on the assumption that the owner has the basis of the foundation. The contractor is responsible for the internal stability of the structure. The internal stability design assuming foundation and slope stability is the responsibility of the owner.
2. For location and alignment of the MSE structures refer to the contract plans.
3. If inadequate foundation material below the reinforced volume as determined by the engineer shall be excavated and replaced with suitable material as directed by the engineer.
4. The reference wall construction drawings for soil characteristics of foundation and proposed backfill material, to be used in the wall design. The contractor shall verify that the select backfill material has material strength parameters in accordance with the design criteria set forth within the contract plans. This shall include the internal friction angle, unit weight and cohesion of the fill material. For design criteria of the fill material, refer to the appropriate specification for fill.
5. The responsibility of the contractor to locate any fence posts prior to placing the top layer of soil reinforcement (shovel or blast) to allow clearance from the edge post. Refer to obstruction details for additional information when deflecting and shifting drain strips.
6. If existing or future structures are to be placed in the reinforced volume that interfere with the proper placement of the soil reinforcement, the contractor shall notify the engineer immediately for a course of action.

WALL CONSTRUCTION (CONTINUED)

1. External stability
2. Internal stability
3. Sliding
4. Bearing pressure

LOAD & RESISTANCE FACTORS

1. In accordance with the ACI-530/SCI-530 design specifications and internal specifications.
2. The maximum allowed bearing pressure at the interface of the foundation and the select backfill material is displayed in the calculations. The bearing pressure calculated is the maximum for the length of the slope. It is the responsibility of the owner to determine the bearing pressure to be allowed for that particular job.
3. Any inadequate foundation material below the reinforced volume as determined by the engineer shall be excavated and replaced with suitable material as directed by the engineer.
4. The design engineer's calculations are based on the assumption that the owner has the basis of the foundation. The contractor is responsible for the internal stability of the structure. The internal stability design assuming foundation and slope stability is the responsibility of the owner.
TIE STRIPS REQUIRED COLUMNS 45 & 47. SEE SHEET 19A FOR DETAILS.
ELEVATION RETAINING WALL NO. 2
FRONT FACE

TIE STRIPS REQUIRED COLUMNS 54 & 56. SEE SHEET 19A FOR DETAILS.

NOTES:
1. DIMENSIONS FOR WALLS ARE GIVEN ALONG FRONT OF PANEL.
2. FRONT END OF PANELS ARE ALONG THE FRONT FACE OF PANEL.
3. PANELS ARE TO THE TOP OF CONCRETE ON TOP OF LEVELING PAD.
4. INTERSECTIONS PANELS ARE BILLED TO PANEL.
5. ALL GRID STRIPS MUST BE DATUM TO PANEL.
6. GRID STRIPS MUST BE TO PANEL.
7. PANEL IS THE CONTRACTOR'S RESPONSIBILITY. THAT PANEL IS NOT REINFORCED TO BE PLACED IN CONCRETE.
8. PANELS OF REINFORCING ELEMENTS MUST BE REINFORCED ELEMENTS OF PANEL.
9. PANELS OF REINFORCING ELEMENTS TO PANELS OF CLEARANCE FROM CURTAIN.

INITIAL SUBMITTAL
CMG 1200/01
BY DATE

PROJECT AND CONFIDENTIAL
THIS DRAWING IS SOLE PROPERTY OF BIG R MANUFACTURING, LLC AND CONTAINS PROPRIETARY INFORMATION FOR USE WITH THIS PROJECT ONLY. ANY REPRODUCTION IN PART OR AS A WHOLE WITHOUT WRITTEN PERMISSION IS STRICTLY PROHIBITED.

Thomas P. Taylor
CIVIL No. 45571
VistaWall Systems
CAPE COD RAIL TRAIL EX - PHASE 1
TOWING OF YARMOUTH & DENNIS
BARNSTABLE COUNTY, MA
FED: AD PROJ: CMS407-002679-0BFX
PROJECT FILE NO. 804488
THE DESIGN CONTAINED HEREIN IS BASED ON INFORMATION SUPPLIED BY OTHERS. BIG R BRIDGE IS CERTIFYING INTERNAL STABILITY OF THE STRUCTURE ONLY. EXTERNAL AND GLOBAL STABILITY REQUIREMENTS ARE THE RESPONSIBILITY OF OTHERS.

ELEVATION RETAINING WALL NO. 2
SCC SIE retaining wall
GRID STRIP: SOIL GRADE REINFORCING
ELEVATION – RETAINING WALL NO. 3
FRONT FACE

NOTES:

1. DIMENSIONING FOR WALLS ARE SHOWN ALONG FRONT FACE OF PANEL.
2. ALL DIMENSIONS ARE SHOWN ALONG THE FRONT FACE OF PANEL.
3. DIMENSIONS ARE TO THE TOP OF CORNER OR TOP OF SOIL TRAVELING.
4. NUMBER OF PANELS AND PANEL DETAILS.
5. IT IS THE CONTRACTOR'S RESPONSIBILITY TO PROVIDE THE SOIL TRAVELING AND PANEL DETAILS.

SHEET MATERIAL SPECIFICATIONS
1. FABRICATION SPECIFICATIONS PER ASTM A490/B/A490M
2. LENGTH OF SOIL TRAVELING = 5'' (DARK TRAVELING DIRECTION LINE)

SOIL TRAVELING SCHEDULE

GRID STRIP

<table>
<thead>
<tr>
<th>PAY</th>
<th>PANEL</th>
<th>TOP OF CORNER PANEL</th>
<th>TYPE</th>
<th>PANEL</th>
<th>PANEL</th>
<th>PANEL</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

FINISH PANEL SCHEDULE

<table>
<thead>
<tr>
<th>PAY</th>
<th>PANEL</th>
<th>TOP OF CORNER PANEL</th>
<th>TYPE</th>
<th>PANEL</th>
<th>PANEL</th>
<th>PANEL</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

THOMAS P.
TAYLOR
CIVIL
NO. 45571
ABUTMENT PLAN DETAIL FOR WALLS 1 & 2
PLAN VIEW

56° BEND WALL  
STA 156+08.00 WALL 1  
OFFSET 10.50' LT.  
STA 157+62.49 WALL 2  
OFFSET 10.50' LT.

56° BEND WALL  
STA 156+08.00 WALL 1  
OFFSET 10.50' RT.  
STA 157+62.49 WALL 2  
OFFSET 10.50' LT.

1131° BEND WALL  
STA 156+70.31 WALL 1  
OFFSET 10.50' LT.  
STA 157+70.34 WALL 2  
OFFSET 10.50' RT.

WALL STEPY  
C.I.P. VERTICAL COPING  
STA 156+70.31 WALL 1  
OFFSET 10.50' RT.  
STA 157+70.34 WALL 2  
OFFSET 10.50' LT.

WALL STEPY  
C.I.P. VERTICAL COPING  
STA 156+70.31 WALL 1  
OFFSET 10.50' LT.  
STA 157+70.34 WALL 2  
OFFSET 10.50' RT.
ABUTMENT PLAN DETAIL FOR WALLS 3 & 4
PLAN VIEW
TYPICAL MSE WALL SECTION

SECTION VIEW

- Horizontal & Vertical Control Line of Wall
- Prop. 3' x 4" Wood Railing
- Bit. Conc. Surface
- 1.5% Slope
- Nominal Face of Precast Panel
- Finish Grade
- Grid-Strip Soil Reinforcing (Typ.)
- 1' x 4' Reinforced Leveling Pad (Class B)

Soil Reinforcing Length Varies (See Wall Elevations for S Lengths)

Limits of Reinforced Volume

Prop. 2" Compost - Topsoil and See Over 4' Later

B 24
C.L.C. CORNING (BY OTHERS)

C.P.I. CORNING (BY OTHERS)

1' x 4' REINFORCED LEVELING PAD (CLASS B)

VISTAWALL SYSTEMS
P.O. Box 1290 • Greely, Colorado 80431-1290
970-356-9600 • www.vistawall.com

THOMAS P. TAYLOR
CIVIL
No. 45571

CAPE COD RAIL TRAIL EXT. - PHASE 1
TOWING OF YARMOUTH & DENNIS
BARNSTABLE COUNTY, MA
FED AID PROJ. #91617-0025/158X
PROJECT FILE NO. 004488

TYPICAL SECTIONS

COMM. OF MASSACHUSETTS
REGISTERED PROFESSIONAL ENGINEER

REV: 3
DATE: 03/30/10
CHECKED BY CMS
DESIGNED BY CMS
DRAWN BY CMS

PROPRIETARY AND CONFIDENTIAL
THIS DRAWING IS THE PROPERTY OF BIG R BRIDGE MANUFACTURING, LLC AND CONTAINS PROPRIETARY INFORMATION FOR USE WITH THIS PROJECT ONLY. ANY REPRODUCTION IN WHOLE OR IN PART WITHOUT WRITTEN PERMISSION IS STRICTLY PROHIBITED.
A. C.I.P. COPING FOR APPROACH WALLS

SECTION VIEW

- BAR U1 (#4)
  - 3/4" following slope line
  - Top of panel
  - Nominal face of precast panel

B. C.I.P. COPING DETAIL FOR ABUTMENT WALLS

SECTION VIEW

- BAR U2 (#4)
  - 3/4" following slope line
  - Top of panel
  - Nominal face of precast panel

CAST-IN-PLACE COPING NOTES:
1. 1/2" Preformed Expansion Joint shall be provided each panel joint.
2. 1/8" Square Joint Sealer shall be applied to the back and top face at each joint.
3. 1/2" Vertical Bevel to be provided at face of coping above edge of panel.
4. Concrete shall have a minimum compressive strength of 4,000 psi with a maximum aggregate size of 3/4".
5. All reinforcing shall be Grade 60.

P.O. Box 1290 - Grayley, Colorado 80432-1290
(970) 356-9600 • www.bigbridge.com

THOMAS P. TAYLOR
No. 45571

PROPRIETARY AND CONFIDENTIAL
This drawing is sole property of BIG BRIDGE MANUFACTURING, LLC and contains proprietary information for use with this project only. Any reproduction in part or as a whole without written permission is strictly prohibited.

VISTAWALL SYSTEMS
F.O. Box 1290 • Grayley, Colorado 80432-1290
(970) 356-9600 • www.vistawall.com

CAPE COD RAIL TRAIL EXT. - PHASE 1
TOWNS OF YARMOUTH & DENNIS
BARNSTABLE COUNTY, MA
FED. AND PROJ. CNTR.: 01128/055558
PROJECT FILE NO. 684488
C.I.P. COPING DETAILS

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

VW16-02388
DESIGNED BY CMS
CHECKED BY CMS
SHEET NO. 24
NOTE:
1. MINIMUM 2" COVER AT ALL EDGES
2. CENTER WIRE IN FURNACE AT BONDS OF PANEL
3. TRIM AS REQUIRED
4. MINIMUM 1" DOCK BOTHWAYS
5. TIE REBAR TOGETHER AT INTERSECTION POINTS

OPTIONAL PANEL REINFORCEMENT

ANCHOR PLACEMENT
GRID-STRIP SOIL REINFORCING SCHEDULE

<table>
<thead>
<tr>
<th>TYPE</th>
<th>LONGITUDINAL BAR SIZE</th>
<th>TRANSVERSE BAR SIZE</th>
<th>LONGITUDINAL BAR SPACING</th>
<th>TRANSVERSE BAR SPACING</th>
<th>TRANVERSE BAR WIDTH</th>
<th>NUMBER OF LONG BARS</th>
</tr>
</thead>
<tbody>
<tr>
<td>GS11</td>
<td>W11.0</td>
<td>W11.0</td>
<td>10'-0&quot;</td>
<td>10'-0&quot;</td>
<td>2'-0&quot;</td>
<td>2</td>
</tr>
</tbody>
</table>

NOTE:
1. SOIL REINFORCEMENT SHALL BE MANUFACTURED IN ACCORDANCE WITH ASTM A490.
2. FOR THE DEFORMED WIRE, BAR SPACING MAY BE INCREASED FOR SMOOTH WIRE.
3. ALL WIRE SPECIFICATIONS AND CONNECTIONS SHALL BE IN ACCORDANCE WITH THE SPECIFIC PROJECT DESIGN REQUIREMENTS.
4. AS MANUFACTURED SPECIFICATIONS ARE THE RESPONSIBILITY OF THE CONTRACTOR.

A. GRID STRIP REINFORCING SCHEDULE

B. GRID STRIP DETAIL

C. GRID STRIP DETAIL

D. PANEL KEY

E. STANDARD ANCHOR PLACEMENT

NOTE:
1. WIRE COMPRESSION SHALL BE MANUFACTURED IN ACCORDANCE WITH ASTM A490.
2. FOR THE DEFORMED WIRE, BAR SPACING MAY BE INCREASED FOR SMOOTH WIRE.
3. ALL WIRE SPECIFICATIONS AND CONNECTIONS SHALL BE IN ACCORDANCE WITH THE SPECIFIC PROJECT DESIGN REQUIREMENTS.
4. AS MANUFACTURED SPECIFICATIONS ARE THE RESPONSIBILITY OF THE CONTRACTOR.

TAIWALL SYSTEMS
P.O. Box 1290 • Greely, Colorado 80432-1290
19703 356-9650 • www.vistaWall.com

THOMAS P. TAYLOR
CIVIL
No. 45571

COMMUNITY OF MASSACHUSETTS
PROFESSIONAL ENGINEER

CAPE COD RAIL TRAIL EXT. - PHASE 1
TOWNS OF YARMOUTH & DERRIS
BARNSTABLE COUNTY, MA
FED: AID Proj. CM#7710321761X
PROJECT FILE NO. 804488

THE DESIGN CONTAINED WITHIN THIS DRAWING IS BASED ON INFORMATION SUPPLIED BY OTHERS. BIG R BRIDGE IS QUALITY AND PERFORMANCE BASED ON THE DESIGN REQUIREMENTS. ALL REQUIREMENTS ARE THE RESPONSIBILITY OF THE CONTRACTOR.
NOTE:
1. THE LEVELING COURSE STEP SHOWN ARE TYPICAL INCREMENTS;
2. MINIMUM STEP IS 6 INCHES;

REFERENCE DETAIL E THIS SHEET FOR DETAIL AT LEVELING PAD STEP.

6' x 12' REINFORCED CONCRETE LEVELING PAD

1. LEVELING PAD AND PANEL PLACEMENT

- PANEL SPACING

2. FILTER FABRIC PLACEMENT

- FILTER FABRIC CLOTH SHALL BE PLACED OVER ALL VERTICAL AND HORIZONTAL JOINTS.
- FILTER FABRIC SHALL BE APPLIED TO BACK FACE OF PANELS USING ADHESIVE.
- ADHESIVE SHALL BE APPLIED TO PANEL THEN FILTER FABRIC PLACED ON BACK OF PANEL.
- THERE SHALL BE A 12" OVERLAP BETWEEN SPACED FILTER FABRIC.
- HORIZONTAL JOINT FABRIC SHALL BE EXTENDED 6 INCHES PAST VERTICAL Joints.

3. BEARING PAD AND PANEL PLACEMENT

- BEARING PADS

- 4-BEARING PADS

- 8-BEARING PADS

- 12" FILTER FABRIC (TYP.)

- 6" x 12" REINFORCED CONCRETE LEVELING PAD

- MEDIAN PANEL

- 3.4 M LONGITUDINAL BARS 90 @ 34" TRANSVERSE (CUT TO 9" IN FIELD)

8. LEVELING PAD STEP & REINFORCING DETAIL

- BACK FACE ELEVATION

NOTE:
1. TYPICAL PRE-CAST PANELS, CAST IN PLACE ANCHORS;
2. 8" DIAMETER 9" X 9" X 9" BEARING PAD - 4 PER PANEL;
3. 12" FILTER FABRIC;
4. CONSTRUCTION ADHESIVE NOT SHOWN TO ATTACH FILTER FABRIC TO BACK OF PANEL.

- FILTER FABRIC TO BACK OF PANEL
NOTE: TG AND PG PANELS ARE TO BE CAST AS STANDARD 'G' PANELS. TG & PG DETAILS ARE PROVIDED AS OPTIONS.
NOTE:

PANEL ANCHORS SHOWN ARE FOR REFERENCE ONLY. REFER TO WALL ELEVATIONS FOR THE NUMBER OF ANCHORS REQUIRED IN EACH ROW.
NOTE:
Panel anchors shown are for reference only. Refer to wall elevations for the number of anchors required in each row.
# 4 - 24" DOWELS EMBED 12"
NOTE:
ALL PANELS SHOWN ARE BACKFACE ELEVATIONS.
#4 - 24" DOWELS @ 1'-5 1/2" O.C.
EMBEDED 12" (TYP.)

NOTE
ALL PANELS SHOWN ARE BACKFACE ELEVATIONS

VISTAWALL
SYSTEMS
P.O. Box 1290 - Greely, Colorado 80632-1290
(871) 356-9690 • www.bigbridge.com

BARNSTABLE COUNTY, MA
PROJECT FILE NO. 89448
FEDERAL AID NO. DM207901/0697/034X
THE DESIGN CONTAINED HERIN IS BASED ON INFORMATION SUPPLIED BY OTHERS. BIG BRIDGE IS CERTIFYING INTERNAL STABILITY OF THE STRUCTURE ONLY. EXTERNAL AND GLOBAL STABILITY REQUIREMENTS ARE THE RESPONSIBILITY OF OTHERS.
#4 - 24" Dowels @ 1-1/8" O.C. Embedded 12" (TYP.)

NOTE:
ALL PANELS SHOWN ARE BACKFACE ELEVATIONS

Panel TJK543
Panel TJK555
Panel TJK540
Panel TJK533
Panel TJK644
# 4. 24" DOWELS @ 1'-5 1/8" O.C.
EMBEDDED 12" (TYP.)

NOTE:
ALL PANELS SHOWN ARE BACKFACING ELEVATIONS.
A VARIABLE CORNER PANEL (TYPE-X)

1 PLAN VIEW

- PANEL TXH2: 1 pc REQUIRED
- PANEL XG2: 2 pc REQUIRED
- PANEL PXB2: 1 pc REQUIRED
- PANEL TXH3: 1 pc REQUIRED
- PANEL XG3: 2 pc REQUIRED
- PANEL PXB3: 1 pc REQUIRED

Legend:
- Q OF FORM
- LIFTING INSERT
- ANCHOR
- VARIOUS
- DPT'S PANEL
- # BAR @ 12" O.C.
- # BAR EQ 8" (TYP.
- (3 PLACES))
- 2 1/2" CLR.
- 1 1/2"
- 4 1/2"
- 2 16"
- 2 0"
A VARIABLE CORNER PANEL (TYPE-X)

PLAN VIEW

Q OF FORM

DPTS PANEL ANCHOR

VARIES

LIFTING INSERT

#4 BAR SQ. SP.
(TYP. 3 PLACES)

1.1/2" CLR.

2.1/2" CLR

2.1/4" CLR

2.6" (TYP.)

11/2" CLR.

6 1/2"

4 1/2"
A VARIABLE CORNER PANEL (TYPE-X)

PLAN VIEW

Q OF FORM

DPS PANEL ANCHOR

VARIES

LIFTING INSERT

2 1/2" CLR.

#4 BAR SQ. SP.
(TYP. 3 PLACES)

2 1/2" CLR.

#4 BAR @ 12" O.C.

2 1/2" CLR.

4 1/2"

4 1/2"

2 1/8"

2 1/8"

2 1/8"

2 1/8"

1 1/2"

6 1/2"

VISTAWALL SYSTEMS
P.O. Box 1290 • Greeley, Colorado 80632-1290
(970) 356-9650 • www.bigbridge.com

CORNER ELEMENTS DETAILS
WALL NO. 3 SHEET 1 OF 7
SCP MSE RETAINING WALL
GRID-STRIP SOIL, REINFORCING

COMMUNITY OF MASSACHUSETTS

SP37

THOMAS R TAYLOR
CIVIL
No. 45671

CARE CO-HNL TRAIL EXTENSION - PHASE
IN THE TOWNS OF YARMOUTH & DENNIS,
BARNSTABLE COUNTY, MA.
PROJECT FILE NO. 06-A073
FEDERAL AID NO. C90737701773-A01
THE DESIGN CONTAINED HEREIN IS BASED ON INFORMATION SUPPLIED BY OTHERS AND IS TREAT
AS ONE OF ELIMINATING STABILITY OF THE STRUCTURE ONLY, INTERNAL AND EXTERNAL STABILITY
REQUIREMENTS ARE THE RESPONSIBILITY OF OTHERS

VW14-02388
DESIGNED BYorns
DRAWN BY ARG
CHECKED BY CAS
SHEET NO.
VARIABLE CORNER PANEL (TYPE-X)

PLAN VIEW

Q OF FORM

DPTS PANEL ANCHOR

VARIIES

LIFTING INSERT

#4 BAR EQ. SP.
(TYP. 3 PLACES)

2 1/2" CLRL

4 1/2"

2 1/2" CLRL

2 1/8" (TYP.)

#4 BAR @ 12" O.C.
BLOCKOUT CORNER DETAIL

NOTE:
ALL PANELS SHOWN ARE BACKFACE ELEVATIONS

SP40

PLAN VIEW

1 pc REQUIRED

PANEL TX4

1 pc REQUIRED

PANEL TXB4

3 pc REQUIRED

PANEL PXB4

14 pc REQUIRED

PANEL PX4
A BLOCKOUT CORNER DETAIL

NOTE:
ALL PANELS SHOWN ARE BACKFACE ELEVATIONS
INSTALLATION GUIDE

VIST-A-WALL SYSTEM

Segmental Concrete Panel with Grid-Strip Soil Reinforcing
The information set forth in this design methodology, including but not limited to all technical and engineering data, figures, tables, designs, drawings, details, procedures and specifications are intended for information pertaining to this project. Every effort has been made to ensure the 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 any and all liability resulting from such use.
# Table of Contents

1 LIMITATIONS .......................................................... 1

2 OVERVIEW ........................................................... 2

3 TERMINOLOGY .......................................................... 3

4 REQUIRED TOOLS, EQUIPMENT AND MATERIALS ..... 5
   4.1 HAND TOOLS ........................................................................................................... 5
   4.2 LIFTING AND UNLOADING EQUIPMENT .................................................................... 5
   4.3 CONSTRUCTION AIDS .............................................................................................. 6
   4.4 HEAVY EQUIPMENT ................................................................................................. 6

5 SCOPE OF WORK TO BE PERFORMED BY CONTRACTOR ........................................... 6
   5.1 SITE PREPARATION ...................................................................................................... 6
   5.2 WALL LAYOUT ............................................................................................................. 6
   5.3 LEVELING COURSE INSTALLATION .............................................................................. 6
   5.4 CONSTRUCTION OF VIST-A-WALL ............................................................................. 7
   5.5 TOP OF WALL TREATMENT ....................................................................................... 7

6 HANDLING VIST-A-WALL COMPONENTS ................................................................. 7
   6.1 DELIVERY OF PRECAST PANELS ............................................................................. 7
   6.2 UNLOADING OF PANELS .......................................................................................... 8
   6.3 GRID-STRIP™ SOIL REINFORCING ........................................................................... 9
   6.4 OTHER COMPONENTS ............................................................................................... 9
   6.5 VERIFICATION OF MATERIALS ................................................................................ 10

7 CONSTRUCTION ............................................................................................................. 10
   7.1 SITE PREPARATION .................................................................................................... 10
   7.2 PRECAST PANEL ....................................................................................................... 11
   7.3 SELECT BACKFILL REQUIREMENTS ........................................................................ 16
   7.4 BACKFILL PLACEMENT ............................................................................................ 17
   7.5 SOIL REINFORCEMENT PLACEMENT ...................................................................... 18
   7.6 JOINT MATERIALS .................................................................................................... 20
   7.7 OBSTRUCTIONS ......................................................................................................... 21
7.8 DRAINAGE ........................................................................................................................................... 22
7.9 FINISH GRADE PLACEMENT ................................................................................................................... 23
7.10 COPING PLACEMENT ............................................................................................................................... 24

8 TYPICAL WALL COMPONENTS ............................................................................................................. 27

9 INSTALLATION SEQUENCE ................................................................................................................ 28
9.1 STEP ONE ........................................................................................................................................... 28
9.2 STEP TWO .......................................................................................................................................... 29
9.3 STEP THREE ........................................................................................................................................ 30
9.4 STEP FOUR ........................................................................................................................................... 31
9.5 STEP FIVE ........................................................................................................................................... 32
9.6 STEP SIX ............................................................................................................................................. 33
9.7 STEP SEVEN ........................................................................................................................................ 34
9.8 FILTER FABRIC PLACEMENT ............................................................................................................... 35
9.9 PANEL LIFTING ................................................................................................................................... 36

10 MSE CHECK LIST ................................................................................................................................ 37
1 LIMITATIONS

Information in this Installation Guide and all documents are not to be used to design, fabricate, manufacture, assemble, construct, produce, install, or otherwise use any elements, forms, or other special equipment (whether patented or not) that is exclusive to Vistawall Systems, LLC., (VAWS) Vist-A-Wall system, for any other purpose other than this project, without the express written consent of VAWS.

The information contained herein shall not be copied, disclosed or distributed in any manner, in whole or in part, to any third party without prior written consent of VAWS.

The Mechanically Stabilized Earth (MSE) structures for this project are designed by VAWS and/or their consultants. The design is based on the following:

- The internal stability of the reinforced soil mass is based on the design assumptions and material properties noted on the drawings and calculations for the related structure, including all external loads, surcharges, and structure geometry that were provided by or on behalf of the Owner.
- The structure geometry including the layout is based on survey information, contract plans, contract drawings and other information provided by or on behalf of the Owner.
- VAWS is responsible only for the internal design of the MSE structures. The external design responsibilities and considerations, including all global slope stability, bearing capacity, settlement, sliding, overturning and drainage under both static and seismic loading, are the responsibility of the Owner. The external design material parameters are outside of the control of VAWS, it is for this reason that we cannot take responsibility for the design thereof.

This manual is part of the VAWS contract documents. It is designed to provide a set of general guidelines and specifications for the Owner, Contractor, and Installer of the VIST-A-WALL system for the Segmental Concrete Panel (SCP) retaining wall system using Grid-Strip™ soil reinforcing. This manual shall be read and followed in conjunction with the contract documents, shop drawings and project specifications. When conflicts between the guidelines contained in this document and the guidelines specified in the contract documents occur, the more stringent guidelines shall take precedence.
2 OVERVIEW

MSE structures are composite systems consisting of concrete, soil reinforcing, and soil. The inclusion of tensile resisting reinforcing in a mass of soil significantly improves the strength of the soil. This unique combination, when designed and installed properly, will create a cost effective integrated retaining structure.

VAWS shall make available a qualified Technical Adviser (TA) to assist the contractor in the beginning of construction and periodic monitoring during the construction process as stipulated in the contract. Both parties, before commencement of the installation must agree to additional technical advisement in writing.
The Contractor shall be responsible for assuring that all required material is on site and is properly stored as outlined in this manual. The Contractor is responsible for the proper installation of the structure and shall certify that the structure was constructed to the lines, offsets, and elevations as outlined in the contract documents. All compliance with OSHA and other safety organizations shall be the responsibility of the Installer or Contractor.

3 TERMINOLOGY

Alignment Pin - A non-structural dowel that is placed in successive layers of panels in order to maintain proper alignment during erection. This is an optional element.

BAR - The proximal end of the Grid-Strip™ consisting of a special shaped steel plate that is used to attach the Grid-Strip™ to the panel anchor.

Bearing Pad - A compressible generally rectangular element that is placed on the top edge of the concrete panel. The bearing pad prevents concrete-to-concrete contact and concrete spalling.

Bottom Panel (P) – A special height panel that is placed on the concrete leveling course and is the first panel in the wall structure.

Connection Bolt - A threaded fastener, with a head, designed to be used in conjunction with a nut and that joins the Grid-Strip™ to the panel anchor.

Connection Pin - A special shaped smooth cylindrical fastener that joins the soil reinforcement to a panel anchor. This connection pin can be used in lieu of a bolt if approved by the Engineer of Record (Wall EOR).

Coping - A concrete element that is used as the top of wall treatment. The coping can be either pre-cast or cast-in-place.

Coping Panel - A special height panel that is placed at the top of the wall prior to placement of the coping unit

DPS Anchor – A special shaped steel forged Dual Plate Shaft (DPS) anchor that is cast into the concrete panel so it protrudes form the back face and that is used to connect the Grid-Strip™ soil reinforcing to the panel.

DPTS Anchor – A special shaped steel Dual Plate Tie Strip (DPTS) anchor that is cast into the concrete panel so it protrudes form the back face and that is used to connect the Grid-Strip™ soil reinforcing to the panel.

Face Of Wall – The front face of the...
panel. This can differ from the control point of the structure.

**Facing Element** – The facing consists of a pre-cast concrete panel that is structurally connected to the soil reinforcement and prevents the raveling of soil between the layers of soil reinforcement.

**Filter Fabric (Cloth)** - A needle punched geo-textile fabric that is placed over the horizontal and vertical joints of each concrete panel to prevent the soil from eroding from the joints.

**Finish Grade** - The material that is placed in front of the MSE wall that protects the bottom of the wall from erosion and undercutting.

**Flip-Pin** - A special shaped smooth cylindrical fastener that joins the soil reinforcement to the panel anchor.

**Geocomposite** - A combination of geo-textile and plastic that forms a composite fabric that is used in lieu of the filter fabric for specific applications.


**Inclusion** - Any man made element that is inserted into the soil mass to improve the structural properties of the soil.

**Junction Slab** – Reinforced concrete slab that is structurally attached to the coping element. This prevents the coping element from being dislodge from the structure during impact to the coping element. This is sometimes referred to as a moment slab.

**Level-Up Concrete** - Non-reinforced concrete or grout that is placed on the top panel and leveled in order to give the coping a smooth surface to bear on. This concrete brings the top of wall to the correct elevation and orientation.

**Leveling Course** – A level non-structural, non-reinforced, concrete element that the first row of panels is placed on.

**Mechanically Stabilized Earth (MSE)** - Engineering term for the stabilization of earth structures through the use of soil inclusions.

**Moment Slab** - Reinforced concrete slab that is structurally attached to the coping element. This element prevents the coping element from being dislodge from the structure during impact to the coping element. This is sometimes referred to as a junction slab.

**Panel Anchor** - An element that is cast directly into the back face of the panel and that soil reinforcement is attached to.

**Prepared Foundation** - Excavated and proofed rolled area that the reinforced
mass of soil and leveling course bears on.

**Reinforced Soil** - A composite structure composed of alternating layers of soil and inclusions.

**Retained Fill** - The backfill material that is placed directly behind the reinforced soil mass. Usually consists of normal highway embankment material.

**Select Fill** - The mass of soil that is placed within the reinforced soil.

**Shim** - Wedge shaped or thin element that is used to correct panel orientation and to keep joints horizontal. Shim material can be wood or plastic.

**Splice** – The structural joining of two soil reinforcing elements in order to increase the length of the soil reinforcing.

**Soil Reinforcement** - A manufactured grid element that is placed in the select backfill and that is structurally attached to the back face of the facing element.

**TAB** – The proximal end of the Grid-Strip™ consisting of a special forged steel plate that is used to attach the Grid-Strip™ to the panel anchor.

**Traffic Barrier** - A structural element that retains traffic and directs impact in a desired direction. Typically the traffic barrier is structurally attached to the coping element.

**V-Plate** – slotted steel plate that is used to splice two Grid-Strip™ soil reinforcing elements together in the field.

**Wave Plate** – Composite element that is used to structurally splice two Grid-Strip™ soil reinforcing elements together in the field.

### 4 Required Tools, Equipment and Materials

The construction of VIST-A-WALL is a relatively straightforward and repetitive process that does not require specialized labor or equipment.

#### 4.1 Hand Tools

- 2-foot and 4-foot long carpenter levels.
- Claw hammers.
- Rubber mallet
- Chalk line and chalk.
- Caulking gun for 29-ounce tubes of adhesive.
- Wrenches for clamps (2 ea.)
- 30-inch or 36-inch crow bars (2 ea.)
- Sledgehammer.
- Hand-operated or power-operated saws.

#### 4.2 Lifting and Unloading Equipment

- Panel lifting ring clutch
- Spacing tools
• Non-staining dunnage for storage of precast concrete panels.
• Two 22-foot long web slings for unloading panels.
• Lifting chains

4.3 CONSTRUCTION AIDS
• Wood clamps with coil rods, coil nuts and washers
• Wood braces
• Wood or steel stakes
• Hard wood shims

4.4 HEAVY EQUIPMENT
• Hydraulic crane or boom truck to lift and place precast concrete facing panels. A standard 5x5 panel weight is 2000 pounds. A standard 5x10 panel weight is 4000 pounds (assumes nominal 6” thickness)
• Dump trucks, front-end loaders, scrapers, bulldozers or graders to place backfill.
• Water truck
• Smooth-drum vibratory roller
• Walk-behind vibratory rollers or plate compactor

5 SCOPE OF WORK TO BE PERFORMED BY CONTRACTOR

The following scope of work is a general guideline. If the contract warrants different responsibilities then they shall be followed accordingly.

5.1 SITE PREPARATION

The Contractor is responsible to prepare the site properly. This included all grubbing, excavation, dewatering, foundation stabilization and installation of all drainage systems. The site preparation should be coordinated with the installation of all utilities that encroach on the MSE structure.

5.2 WALL LAYOUT

The Contractor is responsible for the VIST-A-WALL layout. When laying out the wall use the Contract Plans and not the VIST-A-WALL approved shop drawings. The plan views that are provided in the SEW plans are for information only and may not contain all required layout information. Further, the plan view is representative of a unique elevation in the structure and may, or may not, contain all required information.

5.3 LEVELING COURSE INSTALLATION

The Contractor is responsible to install the unreinforced concrete leveling course in accordance with the steps and grades given in the VIST-A-WALL approved shop drawings. The foundation area that the
leveling course will bear on is required to be compacted. The leveling course shall be placed to the required elevation to a tolerance of ±1/8”.

5.4 CONSTRUCTION OF VIST-A-WALL

The Contractor is responsible for the construction of the Vist-A-Wall system. This includes, but is not limited to, the placement and installation of precast concrete facing panels, installation of joint materials including bearing pads and filter cloth, placement and connection of Grid-Strip™ soil reinforcing, and placement and compaction of the reinforced soil backfill.

5.5 TOP OF WALL TREATMENT

The Contractor is responsible for placement of the precast or cast-in-place concrete coping, traffic impact barrier, junction slab, or any other secondary concrete components that are required. In addition, the contractor is responsible to form and place the leveling concrete on the top panel to the lines and grades necessary to place the coping at the correct orientation and elevation.

6 HANDLING VIST-A-WALL COMPONENTS

The VIST-A-WALL components are specially designed and fabricated for a specific project and should not be used on any other project without the written consent of the VAWS Engineers. It is essential that the materials, especially precast concrete panels and soil reinforcing Grid-Strips™, are handled correctly. It is the responsibility of the Contractor to ensure that all VIST-A-WALL components are free from any damage that might render them unacceptable for use in the installation process. Because the components are manufactured for a specific project, replacement materials take time to fabricate and deliver. VAWS will not be responsible for any replacement costs, time delays, or lost production that is associated from damage to any components sustained following delivery and acceptance by the Contractor.

6.1 DELIVERY OF PRECAST PANELS

To ensure timely delivery, it is imperative that the Contractor, Installer, VAWS and the VAWS Precaster agree on a panel delivery schedule for the project. While many of the panels have the same overall geometry, there are slight differences with respect to the number of panel anchors, the panel reinforcement, and panel dimensions.

Panels are not available for shipping until they have achieved the minimum
concrete compressive strength. It is the responsibility of the Owners’ Inspectors, or the Precaster to determine when panels are ready for shipping.

Precast concrete panels are shipped to the site on flatbed trailers. Typical truckloads consist of four or five stacks of four or five panels. The average panel area for one truckload is 600 square feet. Panels are stacked finished face down on dunnage. Care must be taken during unloading and moving of panels to avoid damaging the panel finish.

6.2 **UNLOADING OF PANELS**

Precast panels are typically unloaded at a central location for the entire project and within close proximity to the actual wall site. If panels are unloaded at the wall site, the panels may either be placed directly into the wall or stored temporarily adjacent to the wall. Placing the panels directly into the wall may require additional unloading time and extra costs.

Typical unloading time for a truckload of panels is two hours. Panels are to be stored in stacks on a flat, firm surface using non-staining dunnage. Stacks of panels should be unloaded using either a crane with two-web slings or a forklift with padded forks to protect the panel finish. Any damaged panels on the flatbed trailer should be brought to the attention of the driver and recorded on the delivery ticket.
If the panels are to be placed directly into the wall, then ring clutches must be attached to the lifting embeds in the top edge of the panel to lift and handle individual panels. Both lifting embeds must be used in lifting and handling panels. All dunnage is the property of VAWS or the Precaster and should be made available for pick-up and reuse.

6.3 **GRID-STRIP™ SOIL REINFORCING**

The Grid-Strip™ soil reinforcing elements for the VIST-A-WALL system comprise discrete steel grids. Each wall will require several lengths of grid. The Grid-Strips™ are delivered in tagged bundles on flatbed trailers. Grid-Strip™ lengths range from 6 feet to 40 feet.

Grid-Strip™ bundles shall be unloaded using a forklift or slings. For Grid-Strips™ in excess of 20 feet it is recommended that a “spreader bar” be used to avoid damage by excessive deflection. Grid-Strips™ shall not be stored directly on the ground but shall be stored on dunnage.

The Contractor is responsible for the proper storage and handling of the Grid-Strips™ after delivery. The Contractor shall guarantee that each Grid-Strip™ is placed in the proper location in the wall as indicated on the approved shop drawings.

6.4 **OTHER COMPONENTS**

The bolt sets, flip-pins, filter fabric, adhesive, bearing pads, spacing tools,
lifting eyes and other secondary components are normally shipped with the first truckload of panels or Grid-Strips™. It is the responsibility of the Contractor to store these materials away from direct sunlight. Further, all material that is shipped to the site in cardboard boxes shall be kept in an area that keeps them dry to prevent degradation of the container.

6.5 Verification of Materials

All material quantities must be verified by the Contractor or Contractor’s agent at the time of delivery and any discrepancies reported to VAWS within 48 hours of delivery. VAWS shall not be responsible for any costs or delays associated with the failure to report discrepancies in material quantities within 48 hours of delivery.

7 Construction

The construction of a Vist-A-Wall structure is a repetitive process that requires successive layers of compacted soil, concrete panels, and Grid-Strips™. Each constructed layer normally requires the same installation steps that require the same standard material. The elevation of each wall is shown in the Shop Drawings. Below each wall are column numbers. The column numbers can be used to easily identify material.

In order to speed installation it is highly recommended that the Installer be familiar with the location of each of the material components that form the wall structure, and where each component is stored on site. Further, it is recommended that the Installer studies the contract documents fully and completely.

7.1 Site Preparation

The foundation area shall be graded level for a width equal to the length of the soil reinforcement plus six (6) inches. All foundation material that is suspected of being of poor quality shall be removed.
and replaced. The foundation preparation is the critical part of the wall construction. *Taking time to properly prepare the foundation will unquestionably decrease the chances of problems occurring during or after construction.* Soil reinforcement shall not be placed until the foundation has been prepared so it is capable of supporting all anticipated loading.

### Compaction Equipment

Once the foundation is properly prepared the non-reinforced leveling pad shall be formed and the concrete placed. Excavate and form the leveling course so that the top of the leveling course is at the desired elevation to within a tolerance of ±1/8”. The leveling course will have a minimum thickness of 6 inches and a minimum width of 12”. For panels placed on curves the width of the leveling course may need to be increased.

### Typical Leveling Course

Compact the excavated leveling course foundation area before placing the concrete. The leveling pad shall be allowed to cure a minimum of 24 hours before placement of the first row of panels. The leveling pad must be placed in the proper vertical and horizontal position as detailed on the plans. *Experience has shown that improper placement of the leveling pad increases the chance for an unsatisfactory finished product.*

### 7.2 Precast Panel

Vist-A-Wall concrete panels are available in three basic sizes: 5’ x 5’, 5’ x 10’ and full height. Each panel is labeled and detailed on the panel schedule or on the standard panel detail sheet. The panel label provides the panel type and number of panel anchors per row. Note that you can easily identify a panel by the scribed information that is placed by the Pre-Caster on the back of each panel.
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”).

**Staggered Bottom Panel Arrangement**

The top-course of panels is available in a range of heights. Varying height panels create a stepped transition at the top of the wall.

**Stepped Top Panel Arrangement**

Other specially cut, notched, slope-topped and special width panels are available to meet the specific requirements of the wall geometry.

**Notched Panel**

Panel designation and casting dates are scribed into the back face of each panel. If required, or stipulated, this information will be marked on one edge of the panel for further identification.

Standard type “G” panels are the most common panel. Typically, these panels will comprise more than 75 percent of the panels in a typical wall. A type “G” square panel has an actual dimension of 5’-0 ½” x 5’-0 ½”.

**Typical Panel Detail**
The panels are placed in the wall with a ¾” spacing on all sides. This arrangement provides a nominal panel dimension of 5’-0” x 5’-0”. The shop drawings provide a column number for each column of panels. All non-standard panel widths include the required joint spacing by design.

### ¾” Vertical Joint Spacing

Bottom-of-wall panels are designated with a “P” and the appropriate panel size indicator. The “P” allows for the precaster to use a special block out for the bottom of the panel. A “PB” denotes a bottom “P” standard “B” panel that has the nominal dimensions of 2’-6” x 5’-0”.

Top-of-wall panels are designated with a “T” and the appropriate panel size indicator. The “T” allows for the precaster to use a special block out for the top of the panel and also to insure that “top steel” is added if required. A “TG” denotes a top “T” standard “G” panel that has the nominal dimensions of 5’-0” x 5’-0”.

Sloped top panels are designated by a “T” and a double alpha character such as “TBG”. This is looking at the front face of the panel, with the first alpha following the “T” (top) being equal to the left height (B) and the subsequent alpha being the right height (G). A “TBG” is a top panel with a nominal left height of 2’-6” and a nominal right height of 5’-0”. The “T” is as designated previously.

Corner or vertical slip-joint panels are designated as “X” panels. The nomenclature also includes the equivalent standard panel alpha designation. Special panels are designated “S”, followed by a number.

### Typical Panel Types

<table>
<thead>
<tr>
<th>Panel Type</th>
<th>Nominal Height</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>2’-0”</td>
</tr>
<tr>
<td>B</td>
<td>2’-6”</td>
</tr>
<tr>
<td>C</td>
<td>3’-0”</td>
</tr>
<tr>
<td>D</td>
<td>3’-6”</td>
</tr>
<tr>
<td>E</td>
<td>4’-0”</td>
</tr>
<tr>
<td>F</td>
<td>4’-6”</td>
</tr>
<tr>
<td>G</td>
<td>5’-0”</td>
</tr>
<tr>
<td>H</td>
<td>5’-6”</td>
</tr>
<tr>
<td>J</td>
<td>6’-0”</td>
</tr>
<tr>
<td>K</td>
<td>6’-6”</td>
</tr>
<tr>
<td>L</td>
<td>7’-0”</td>
</tr>
<tr>
<td>M</td>
<td>7’-6”</td>
</tr>
</tbody>
</table>

Grid-Strip™ - Installation Guide
tpt (10/20/13)
Note: The alpha character i (I) has been omitted because of the similarity to the numeral one (1)

Cut or non-standard width panels with no edge tongue and groove are designated with an L or R. The letter L refers to a panel that is cut on the left side (viewed from the front face of the panel). Similarly, R denotes a panel cut on the right side (looking at the front face of the panel). These panels are used at corners and in instances where the wall structure intersects a concrete structure.

The Wall Installer shall give notice to the Pre-Caster as to which structure to begin casting. The Wall Installer shall provide a schedule of installation to the Pre-Caster. In addition, the Wall Installer shall provide a load list to the Pre-Caster. The Wall Installer shall notify the Pre-Caster a minimum one week before said installation shall take place. The Wall Installer shall keep track of all panel shipments and a log of what panels were installed, on which day, and in what structure.

Panels that are of poor quality and that do not meet tolerances shall not be used and shall be rejected and set aside. Inspection of the panel should be performed when the panels arrive to the site and as they are being removed from the truck. The Wall Installer should make note as to the panel number, lot number, date and reason for rejecting the panel. This list shall be given to the Owner and wall supplier.

The first row of panels is placed on the leveling course. If the leveling course is not placed properly, the panel will need to be adjusted to insure that the horizontal and vertical alignment is achieved and maintained during the erection process. This can be achieved with hard-wood shims. It is not necessary to place a bearing pad on the leveling course for the panel to bear on. The panel should be set directly on the leveling course.

 Placement of Bottom Row of Panels

It is important 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 will be easier to install.

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

In order to provide proper placement of the first row of panels and to set the correct batter, it is recommended, but not required, to position 2 wooded wedges on the front and back side of the bottom panel.

The wedges when lodged under the panel will aid in stabilization of the panel, will provide for easy adjustment of the batter, and will aid in leveling the panel to the proper elevation.

**Bottom Panels with Leveling Wedges**

Each panel is identified by an attribute. The number preceding the panel alpha indicates how many DPS or DPTS anchors are cast in the panel. A “G22” is a type a “G” panel with 2-DPS anchors in the top row and 2-DPS anchors in the bottom row.

**Typical Panel Attribute**

The overall installation speed of any MSE structure is completely a function of the delivery and compaction of the backfill material. It should not be assumed that the larger the precast concrete facing panel the faster the overall rate of wall construction will be.

MSE structures are very flexible. This
flexibility enables the entire structure to accommodate significant total and differential settlements without any damage to the facing itself. It should be understood that as the panel size increases the total amount of differential settlement the structure can tolerate decreases. Further, it should be understood that the ¾” joint that is innate to the system is extremely important and needs to be maintained. By diminishing the ¾” joint spacing the amount of differential settlement the MSE structure can tolerate decreases.

The panels that are delivered to the site are marked with the appropriate designation and must be checked against the approved shop drawings to ensure that panels are placed in the correct location in the field. It is the responsibility of the Owner’s inspection personnel to ensure that all panels meet the necessary quality control requirements with respect to concrete strength, tolerances on overall dimensions, damaged or missing connectors, cracked or broken concrete, etc., before being placed in the wall.

7.3 SELECT BACKFILL REQUIREMENTS

The backfill is the most critical component of the structure. It has been demonstrated that to assure maximum strength, minimum compressibility, and the ability to compact to a high density, that the backfill should consist of well-graded granular material. Well graded material typically possesses electro-chemical properties that meet the AASHTO corrosion model. A well graded material also has excellent drainage characteristics and the post-construction movements are rare.

MSE structures can be constructed using fine-grained soils. It should be noted that the installation process takes considerably more time and more observant control by the Wall Installer. Further, fine grain soils are normally poorly drained and the stress transfer between the soil and the soil reinforcement is not immediate. Because of this, the panel may require a slight batter when set. The batter will slowly be removed as the fine grained soils are compacted. There is not a rule on how much the panel should be battered; it must be done by trial and error. A good starting point would be so the panel will be within tolerance if the panel does not move. It should also be noted that fine-grained soils also exhibit a time-dependent comportment that can cause post construction movement.

A select backfill material should possess a
wide range of grain sizes with a limit of no more than 15 percent passing the No. 200 sieve. The select backfill material should not be gap-graded. Gap-graded material will make compaction more difficult. Gap-graded material will have voids that will allow for soil loss to occur when a hydraulic gradient or dynamic load is introduced into the backfill. This will cause the backfill to settle and will cause post-construction movements of the system.

The MSE backfill shall meet the project specifications. It is the Contractors responsibility to verify that the MSE backfill gradation, shear strength, permeability, and electrochemical properties, including resistivity, pH, organic, chloride, and sulfate contents are in accordance with the specifications. VAWS is not responsible for approving backfill sources or backfill materials. The final acceptance to all backfill material is the Owner’s responsibility.

7.4 BACKFILL PLACEMENT

The placement of the backfill 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”-12” compacted layers. The backfill can be placed in larger lifts if approved by the Owner or Owners representative and if the Wall Installer can demonstrate that the proper compaction is achieved. The fill shall be leveled by equipment moving parallel to the wall face. The material shall be spread so it is fanned toward the tail of the soil reinforcing. The placement of the backfill from the front of the soil reinforcement to the tail of the soil reinforcement will keep the soil reinforcement fixed.

Compaction of the backfill a distance of 3 feet from the back face of the panel shall be performed with an 8 ton to 10 ton 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 end of the reinforcement. Proper moisture content of the backfill material should be maintained uniformly within each layer. The material should 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 with the use of a 1000 pound vibratory roller or 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 tail of the soil reinforcing.*

**Compaction of 3-Foot Zone**

Compaction tests and gradation tests should be taken and recorded in accordance with the contract plans. These reports shall be made part of the Wall Installers log. *Proper compaction will alleviate possible problems in the future performance of the structure.* Improper compaction can cause outward movement of the panels. Compaction test should not be performed in the area directly adjacent to the back face of the panel.

### 7.5 Soil Reinforcement Placement

The Grid-Strip™ is attached to the DPS and/or DPTS panel anchor with a bolt set. A washer is not required. The washer is not required because this is not a structural joint. The bolt can be considered a pin, and the nut the method used to keep the pin in place. In addition to using a bolt, the Grid-Strip™ can be connected with a flip-pin.

**Grid-Strip™ to Panel Bolt Connection**
The number of anchors and the grouping of the anchors are dependent on the project calculations and the location in the wall structure. Typically, the panel anchors are cast symmetric about the panel.

**Grid-Strip™ to Panel Connection**

The shop drawings panel attribute defines the number of Grid-Strip™ soil reinforcement that is to be used in each row.

Typical Panel Attribute

The length of the soil reinforcing is given under the dimension line in the elevation drawing and is identified by the designator, “B = “ or “L = “. It is important that the Wall Installer place and connect the correct number and correct length of Grid-Strip™ on the correct panel and location in the wall.

**Grid-Strip™ to Panel Connection Detail**

The Grid-Strip™ is placed on the compacted select backfill and the connection is made to the DPS and/or DPTS panel anchor. The Grid-Strip™ is connected to the panel by securing the TAB and/or BAR connector between the two plates of the panel anchor. The bolt only needs to be finger tight. It does not require that a wrench be used. The threads should just start to protrude from the nut.
Never connect a Grid-Strip with the backfill lower than the elevation of the panel anchor. The backfill should ideally be slightly higher than the panel anchor. In order to place the bolt or flip pin, it may be necessary to remove some backfill from under the DPS anchor.

The flip-pin is designed to be placed from the bottom of the DPS anchor and then rotated toward the back of the panel so the pin “flips” over the shaft. This will prevent the pin from being accidently removed.

The Wall Installer should verify that all nuts have been installed with the bolt. Equally important, the Wall Installer shall verify that flip-pins are properly engaged with the DPS anchor shaft. Any panel that has anchors missing, or that are severely bent, or that are damaged, shall not be used, but shall be set aside and either repaired or rejected (Reference VAWS Maintenance and Repair Manual).

### 7.6 Joint Materials

The horizontal panel joints are maintained by placement of bearing pads. The bearing pads shall be placed adjacent to the alignment pins (where applicable) or at the approximate quarter span location of each panel. A minimum of two (2) bearing pads are required for each panel regardless of panel width.
Standard Joint Materials

The filter cloth is supplied in 12 inch wide rolls and is to be centered along each and every panel joint. It is not necessary, but may be a project specific requirement, to place filter fabric along the bottom edge of the bottom panel at the interface of the leveling course.

Filter Fabric Joint Material

An acceptable length of filter fabric shall be lapped at each joint intersection. The filter fabric shall be designed and selected in accordance with AASHTO M288.

The filter fabric is held into place by the supplied adhesive. The durability of the adhesive is not important since its only purpose is to temporarily hold the filter cloth in place during the backfilling operation. Once the backfill is placed and compacted, soil pressure will hold the fabric in place. The adhesive is not intended to provide a water tight seal. The purpose of the filter fabric is to prevent the migration of fine material from the joints during the life of the structure and at the same time allow water to escape. The amount of adhesive that is used is equal to only the amount to hold the fabric. It may be placed only at the corners. If the Wall Installer decides to use a continuous bead along the panel edges the Wall Installer shall supply additional adhesive. Please reference the Application of Adhesive for Filter Fabric document that is at the end of this document.

To recap, in certain instances the filter fabric may be required to be placed at the interface of the leveling pad and the first course of panels. Reference should be made to the governing specification for guidance.

7.7 Obstructions

During the design phase an attempt is made to detail special panels, and connections at the locations of vertical and horizontal obstructions.
Pile Obstructions in MSE Back Fill

In some instances it may be necessary to shift the Grid-Strip along the length of the panel so it is grouped near one end. This may be detailed on the panel elevation and in the plan view. The Grid-Strip™ is an articulating connection that can easily be rotated to by-pass most obstruction.

The cutting of the Grid-Strip™ is strictly prohibited. It may be necessary to rotate the Grid-Strip so it is skewed. The angle of rotation shall be no more than 15°. The use of angles greater than 15° can be used if approved by the Retaining Wall Engineer of Record.

Horizontal obstructions can be passed by gradually skewing the soil reinforcement above or below the obstruction. If the soil reinforcement is skewed horizontally, care should be taken so as not to kink the reinforcement. The deflected angle shall be less than 15°. The use of angles greater than 15° can be used if approved by the Retaining Wall Engineer of Record. Place a 4”-6” buffer of soil over the obstruction before the placement of the soil reinforcement.

RCP Obstruction

7.8 Drainage

It is extremely important not to allow the reinforced volume to become saturated at any time during construction. At the end of each day’s operation proper precautions shall be taken in order to assure that the MSE mass does not
become saturated. The Wall Installer shall slope the reinforced volume of soil away from the facing elements at the end of each days operation. Saturation of the MSE mass can result in destabilizing forces that cause the structure to fail. Heavy rainfall can cause erosion of the soil from within the layers of the MSE mass especially at the end of the constructed wall segment. If any erosion does occur, or if the backfill becomes saturated, it shall be replaced with non-saturated material that conforms to the backfill specifications. Care shall be taken during periods of heavy rain to assure proper drainage and to provide positive flow away from the facing.

**Consequences of Heavy Rain Washout**

After heavy rains and in saturated sandy soils, it is advisable to exercise great care in the placement and operation of construction equipment near the face of the wall. The use of construction equipment must be kept away from the back of the panel until such time the sandy soils have dried out.

7.9 **Finish Grade Placement**

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.

![Finish Grade at Face of MSE Structure](image)

Although the fill in front of the wall is not considered in the stability calculations it should be understood that any excavation in front of the wall at depths below the leveling course could greatly affect the structural stability and integrity of the reinforced volume. No excavation below the leveling course shall be allowed until written permission is received from the
Engineer of Record and the wall supplier.

7.10 COPING PLACEMENT

The placement of the coping requires that the top of wall be at the proper elevation and orientation as shown in the contract plans. Leveling concrete may be required when precast coping is specified. The leveling concrete is placed in the area between the top panel and the precast coping seat. The leveling concrete is placed in a wooden form that is attached to the panels. The required elevation is marked on the inside of the form and the leveling concrete is placed in the form and leveled to the marked line or top of the form boards.

**Leveling Concrete - Forming System**

Depending on the coping that is used the details may call for dowels to be placed in the top edge of the top panel. The dowels are used to tie the coping panel and leveling concrete together. Further it may be required that a cushion layer be placed on the top of the panel, or leveling concrete, and at the back face of the panel and coping leg. This cushion layer helps to lessen the stress transfer of the impact load into the top panel.

**Coping Unit on Leveling Concrete**

Because the coping is usually placed at a slope on the top panel it is very difficult to pre-cast any type of insert into the panel or coping that assures that they will line up in the field. Experience has shown that the best solution for placing dowels in a sloped coping panel is to field drill them after the panels are set and epoxy the dowels in place.

It is possible to cast inserts into coping that are to be placed parallel to the top
panel. The desired number of inserts is cast into the coping and into the panel and a dowel inserted in the field. If dowels are precast into a sloping panel then they may be required to be manually bent so they are perpendicular to the bottom of the coping.

It should be noted that the dowel is only helping prevent the coping from being pushed off of the wall by construction equipment and that should not be relied on to transfer impact forces.
8 TYPICAL WALL COMPONENTS

1 TYPICAL PRE-CAST PANEL WITH ANCHORS (2 ROWS AND 2 COLUMNS OF ANCHORS)
2 2-2" X 6" X 3/4" NEOPRENE BEARING PAD (2 PER PANEL @ ALIGNMENT PIN)
3 2-1/2" X 8" GALVANIZED STEEL ALIGNMENT PINS (2 PER PANEL – If applicable plastic pins may be substituted for galvanized. This element is optional)
4 12" WIDE FILTER FABRIC ADHERE WITH CONSTRUCTION ADHESIVE (ALL JOINTS)
6 CONNECTION BOLT OR FLIP-PIN (ONE PER GRID-STRIP™ SOIL REINFORCING)
7 GRID-STRIP™ SOIL REINFORCING (AS REQUIRED)
9 INSTALLATION SEQUENCE

9.1 STEP ONE

STEP 1. Excavate and prepare foundation

STEP 2. Form and place leveling course to grades, lines and widths as shown on the project shop drawings.

STEP 3. Place first row of panels on leveling course.

STEP 4. Plumb panels with wedges and brace as required.
9.2  STEP TWO

STEP 1.  Place filter fabric on back face of panel at all vertical and horizontal joints and secure to panel with adhesive.

STEP 2.  Place and compact backfill on prepared foundation to the level of the first row of panel anchors.
9.3 STEP THREE

STEP 1. Place Grid-Strip™ soil reinforcing on compacted backfill making sure that it is the correct length.

STEP 2. Connect the TAB of the Grid-Strip™ to the DPS panel anchor with the bolt set. The nut should be up. Finger tight nut so thread emerges from top.
9.4 STEP FOUR

**STEP 1.** Place soil on tail of soil reinforcing to prevent the soil reinforcing from moving.

**STEP 2.** Place selected backfill in the void at the back face of the panel and compact.

**STEP 3.** Place select backfill to level of top of first row of half panel and compact. Make sure to place alignment pin in top edge of panel before placing backfill to prevent alignment pin holes from becoming filled with backfill material.

**STEP 4.** Remove clamps and blocking from first row of half panels.
9.5 STEP FIVE

**STEP 1.** Place the next row of alternating panels over the alignment pins and clamp to adjacent panel to prevent movement.

**STEP 2.** Place next layer of selected backfill and compact.

**STEP 3.** Remove external bracing and place and compact finish grade at face of wall.

**STEP 4.** Repeat installation Steps Two through Step 5 until top row of panels are placed.
9.6  STEP SIX

STEP 1. Place top row of panels.

STEP 2. Place layer of selected backfill compact.

STEP 3. Set forms and place leveling concrete to bring top of panel to required elevation and grade.
9.7 STEP SEVEN

STEP 1. Place backfill to level of bottom of coping or bottom of moment slab.

STEP 2. Form and place coping unit.

STEP 3. Form and place moment slab.

STEP 4. Form and place traffic barrier.

STEP 5. Note: If a one piece coping unit and traffic barrier are being used omit step 4
9.8 FILTER FABRIC PLACEMENT

**STEP 1.** Place 12” wide filter fabric centerline of all horizontal and vertical joints.

**STEP 2.** Adhere filter fabric with adhesive.

**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 interface of the bottom panel and leveling pad. Reference the Owners specification for guidance on this matter.
9.9 PANEL LIFTING

Lift panels with approved lifting device pursuant to Burke Spread Anchor - One Ton and Two Ton specifications.
10 MSE CHECK LIST

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 and tightening of bolts and or placement of flip-pins being used?
14. Yes No  Is the filter fabric 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 tail 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?  A minimum 90% of maximum density for first three foot area and 95% of maximum density for the remaining area.

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 runoff and saturation?
**11 APPLICATION OF ADHESIVE TO FILTER FABRIC**

The filter fabric is attached to back face of the segmental concrete panel using a construction adhesive. The adhesive is used to *temporarily* hold the fabric in place until the compacted backfill is placed at the back of the panel. The adhesive is a method to *temporarily* adhere the filter fabric to the back face of the panel and to keep the outside edges from curling, or bunching up. The adhesive is not applied in order to form a water tight seal and therefore *does not have to be a continuous bead*. Once the backfill is placed and compacted, the horizontal soil pressure will hold the fabric in place.

The adhesive supplied by VAWS is supplied in tubes that contain 29 fluid ounces. The adhesive is applied using a caulking gun and putty knife. The adhesive should be placed using a 2” x 12” stitch pattern on the back of the panel or the filter fabric. The tip of the adhesive tube should be cut to produce a ½” diameter bead. A bead that is ½” x 2” has a volume of 0.4 cubic inches. Using basic conversion factors, it can be calculated that there are 1.8 cubic inches in one fluid ounce of product. Therefore, a 29 fluid ounce tube of adhesive can supply approximately 130 - ½” x 2” beads of adhesive. An alternate method of placement of the adhesive consisting of 2” diameter blobs applied with the putty knife maybe used and applied only at the corners.

<table>
<thead>
<tr>
<th>REQUIRED TUBES PER ELEMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Element</strong></td>
</tr>
<tr>
<td>5 x 5 Panel</td>
</tr>
<tr>
<td>5 x 10 Panel</td>
</tr>
<tr>
<td><em>Roll filter fabric</em></td>
</tr>
</tbody>
</table>

*Assumes 360 linear feet of filter fabric per roll and two rows of 1/2” x 2” beads*
It is the Wall Installer’s responsibility to apply the adhesive in a manner to not exceed the recommended application guidelines set forth in this document. The Wall Installer shall be responsible for purchasing additional adhesive than the amount supplied by VAWS based on these guidelines.
2.3 DESIGN EXAMPLES
IDEA Submittal
Mechanically Stabilized Earth
Grid-Strip Soil Reinforcing

[This Page Is Intentionally left Blank]
2.3.1
Level Backslope with Traffic Live Load Design Example
IDEA
Highway Innovations, Developments, Enhancements and Advancements
MSE Submittal

Horizontal Back-Slope with Live-Load Surcharge

Design Methodology
Static Analysis
<table>
<thead>
<tr>
<th>Project Name:</th>
<th>Verification Calculation for Horizontal Back-Slope</th>
<th>Designer: tpt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Location:</td>
<td>IDEA Submittal [H=30/B=21]</td>
<td>Checker: cms</td>
</tr>
<tr>
<td>Project Number:</td>
<td></td>
<td>Date: 06/07/16</td>
</tr>
<tr>
<td>Design Code:</td>
<td>AASHTO</td>
<td>Page:</td>
</tr>
<tr>
<td>Design Method:</td>
<td>LRFD</td>
<td></td>
</tr>
</tbody>
</table>

This Page Intentionally Left Blank
Introduction .................................................................................................................................................. 1
Design Steps .................................................................................................................................................. 1
1  Establish Project Requirements ....................................................................................................... 3
2  Define Project Parameters ............................................................................................................... 5
   2.1  Define Structure Parameters .............................................................................................. 6
   2.2  Define External Live Load Parameters ................................................................................ 6
   2.3  Define Facing Parameters ................................................................................................... 7
   2.4  Define Soil Reinforcing Parameters .................................................................................... 8
   2.5  Define Backfill Parameters .................................................................................................. 9
3  Evaluate Applicable Load and Resistance Factors ......................................................................... 11
   3.1  Extreme Force Effect ......................................................................................................... 12
4  Define Steel Corrosion Rates ......................................................................................................... 13
5  Define Internal Stability Factors .................................................................................................... 13
6  Calculate Unfactored Loads for External Stability ......................................................................... 14
7  Evaluate External Stability ............................................................................................................. 17
   7.1  Calculate Sliding Resistance at Base of MSE Structure ..................................................... 17
   7.2  Calculate Limiting Eccentricity at Base of MSE Structure ................................................. 18
   7.3  Calculate Bearing Resistance at Base of MSE Structure ................................................... 21
   7.4  Summary of External Stability ........................................................................................... 25
8  Evaluate Internal Stability .............................................................................................................. 25
   8.1  Determine Soil Reinforcing Depths and Tributary Area for Internal Stability ................. 25
   8.2  Determine Variation of Kr, F*and Length of Embedment for Internal Stability ............... 27
   8.3  Calculate Horizontal Stress at Elevation of Each Soil Reinforcing Element ..................... 28
   8.4  Calculate Maximum Tension at Elevation of Each Soil Reinforcing Element ................. 29
   8.5  Determine Required Area of Steel at Elevation of Each Soil Reinforcing Element .......... 31
   8.6  Calculate Factored Pullout Resistance at Elevation of Each Soil Reinforcing Element .... 32
   8.7  Summary of Internal Stability at Elevation of Each Soil Reinforcing Element ............... 33
   8.8  MSEW Internal Stability Discussion .................................................................................. 34
Table of Figures

Figure 1-1  Structure Project Requirements ................................................................. 4
Figure 2-1  Structure Project Requirements ................................................................. 5
Figure 2-2  Facing Parameters Detail ............................................................................. 8
Figure 2-3  Grid-Strip Parameters Detail ....................................................................... 9
Figure 2-4  Backfill Parameters .................................................................................... 10
Figure 6-1  Unfactored Load Diagram for External Stability ........................................ 15
Figure 8-1  Soil Reinforcing Depth Parameters ........................................................... 26
Figure 8-2  Failure Surface Diagram .......................................................................... 27
Figure 8-3  Tributary Area ............................................................................................ 30

Tables

Table 1  Project Requirements ....................................................................................... 4
Table 2  Structure Parameters ......................................................................................... 6
Table 3  External Live Load Parameters ........................................................................ 7
Table 4  Facing Parameters ........................................................................................... 8
Table 5  Soil Reinforcing Parameters ........................................................................... 9
Table 6  Backfill Parameters ......................................................................................... 10
Table 7  Load Factors (AASHTO 2014 Tables 3.4.1-1 and 3.4.1-2) ............................... 11
Table 8  Resistance Factors (AASHTO 2014 Table 11.5.6-1) ........................................ 12
Table 9  Corrosion Parameters ................................................................................. 13
Table 10  Internal Stability Parameters ....................................................................... 14
Table 11  Unfactored Vertical Forces .......................................................................... 15
Table 12  Unfactored Vertical Moments ..................................................................... 16
Table 13  Unfactored Horizontal Forces ...................................................................... 16
Table 14  Unfactored Horizontal Moments ................................................................ 16
Table 15  Evaluation of Sliding at Base of MSE Wall – Strength I Maximum ............... 17
Table 16  Evaluation of Sliding at Base of MSE Wall – Strength I Minimum ............... 17
Table 17  Evaluation of Sliding at Base of MSE Wall – Strength I Critical Value Check ... 18
Table 18  Evaluation of Limiting Eccentricity for MSE Wall – Strength I Maximum ...... 19
Table 19  Evaluation of Limiting Eccentricity for MSE Wall – Strength I Minimum ...... 20
Table 20  Evaluation of Limiting Eccentricity for MSE Wall – Strength I Critical Value Check .... 20
Table 21  Evaluation of Bearing Resistance for MSE Wall – Strength I Maximum .......... 22
Table 22  Evaluation of Bearing Resistance for MSE Wall – Strength I Minimum .......... 23
Table 23  Evaluation of Bearing Resistance for MSE Wall – Strength I Critical Value Check .... 23
Table 24  Evaluation of Bearing Resistance for MSE Wall – Strength I Service .......... 24
Table 25  Summary of External Stability ...................................................................... 25
Table 26  Soil Reinforcing Layout .............................................................................. 26
Table 27  Internal Stability Factors ............................................................................ 28
Table 28  Stress Calculation ....................................................................................... 29
Table 29  Maximum Tension At Soil Reinforcing .......................................................... 30
Table 30  Area of Steel Required for Rupture ............................................................... 32
Table 31  Pullout Resistance for Grid-Strip Element ................................................................. 33
Table 32  Summary of Internal Stability for Grid-Strip Element ................................................. 34

Appendix

Appendix A  MSE-Pro Output
Appendix B  MSEW Output
<table>
<thead>
<tr>
<th>Project Name:</th>
<th>Verification Calculation for Horizontal Back-Slope</th>
<th>Designer</th>
<th>tpt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Location:</td>
<td>IDEA Submittal [H=30/B=21]</td>
<td>Checker</td>
<td>cms</td>
</tr>
<tr>
<td>Project Number:</td>
<td></td>
<td>Date:</td>
<td>06/07/16</td>
</tr>
<tr>
<td>Design Code:</td>
<td>AASHTO</td>
<td>Page:</td>
<td>_____ of _____</td>
</tr>
<tr>
<td>Design Method:</td>
<td>LRFD</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Introduction

This example problem demonstrates the analysis of an MSE wall with a horizontal back-slope that supports a live load surcharge. The MSE structure will include a segmental concrete panel face utilizing the Grid-Strip soil reinforcing. The MSE structure 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 Slopes and conforms to the 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 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 are required 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
   1. Define Structure Parameters
   2. Define External Live Load Parameters
   3. Define Facing Parameters
   4. Define Soil Reinforcing Parameters
   5. Define Backfill Parameters
3. Evaluate Applicable Load and Resistance Factors
   1. Table 7 Load Factors (AASHTO 2014 Tables 3.4.1-1 and 3.4.1-2)
   2. Table 8 Resistance Factors (AASHTO 2014 Table 11.5.6-1)
4. Define Steel Corrosion Rates
5. Define Internal Stability Factors
6. Calculate Unfactored Loads for External Stability
7. Evaluate External Stability
   1. Calculate Sliding Resistance at Base of MSE Structure
   2. Calculate Limiting Eccentricity at Base of MSE Structure
3. Calculate Bearing Resistance at Base of MSE Structure

4. Summary of External Stability Evaluate Internal Stability

8. Evaluate Internal Stability

1. Determine Soil Reinforcing Depths and Tributary Area for Internal Stability

2. Determine Variation of Kr, F* and Length of Embedment for Internal Stability

3. Calculate Horizontal Stress at Elevation of Each Soil Reinforcing Element

4. Calculate Maximum Tension at Elevation of Each Soil Reinforcing Element

5. Determine Required Area of Steel at Elevation of Each Soil Reinforcing Element

6. Calculate Factored Pullout Resistance

7. Summary of Internal Stability

Table 1 through Table 10 defines parameters that are required 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.
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 design height of the structure (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 for the structure analyzed in this document.
Figure 1-1  Structure Project Requirements

Table 1 Project Requirements

<table>
<thead>
<tr>
<th>Location</th>
<th>Variable</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exposed wall height</td>
<td>h_e</td>
<td>28.00</td>
<td>ft</td>
</tr>
<tr>
<td>Wall embedment distance</td>
<td>d</td>
<td>2.00</td>
<td>ft</td>
</tr>
<tr>
<td>Soil reinforcing length aspect ratio</td>
<td>ratio</td>
<td>0.70</td>
<td>dim</td>
</tr>
<tr>
<td>Length of wall</td>
<td>L_w</td>
<td>1000.00</td>
<td>ft</td>
</tr>
<tr>
<td>Design life</td>
<td>Life</td>
<td>75.00</td>
<td>yrs</td>
</tr>
<tr>
<td>Top of wall slope condition</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Facing</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type of Soil Reinforcement</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 1 Project Requirements

<table>
<thead>
<tr>
<th>Location</th>
<th>Variable</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Galvanized – 3.4 mil zinc coating</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Is seismic to be considered</td>
<td>No</td>
<td></td>
</tr>
</tbody>
</table>

2 Define Project Parameters

The project requirements provide a general framework of the constraints for the MSE structure analyzed in this document. A comprehensive set of parameters is provided in Table 2 and Table 3 and are detailed in Figure 2-1. For the MSE structure evaluated in this document 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](image_url)
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>
<thead>
<tr>
<th>Location</th>
<th>Variable</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Design Height</td>
<td>$H$</td>
<td>30.00</td>
<td>ft</td>
</tr>
<tr>
<td>Soil Reinforcing Length</td>
<td>$L$</td>
<td>21.00</td>
<td>ft</td>
</tr>
<tr>
<td>Surcharge Height</td>
<td>$S$</td>
<td>0.00</td>
<td>ft</td>
</tr>
<tr>
<td>Distance from face to Crest</td>
<td>$X_s$</td>
<td>0.00</td>
<td>ft</td>
</tr>
<tr>
<td>Slope of Surcharge</td>
<td>$\beta$</td>
<td>0.00</td>
<td>deg</td>
</tr>
<tr>
<td>Top of Structure Back-Slope Condition</td>
<td>Horizontal</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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 3). 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
2.3 Define Facing Parameters

The facing is specified to be a segmental concrete panel (SCP) with a height (Hₚ) equal to 5.00' and a length (Lₚ) equal to 5.00' (Figure 2-2 and Table 4). The thickness (tₚ) of the panel is assumed to be 6". The vertical spacing (Sᵥ) 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 (Z₁) is equal to a value of 2.25'. The distance from the leveling pad to the first soil reinforcing element (Z₁) for the bottom panel is equal to 1.25'. The minimum distance between any two soil reinforcing elements (Zₘᵢₙ) is equal to a value of 0.50'. The input values, Sᵥ, Z₁, Zₜᵣ, and Zₘᵢₙ are required to set the depth to each soil reinforcing element. The horizontal spacing (Sₕ) 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).
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 Grid-Strip soil reinforcing will be used (Table 5 and Figure 2-2). The Grid-Strip is a discrete steel strip with a single point connector and consists of dual longitudinal steel bars. Each bar has a diameter of 0.374” (W11 bar) with an area equal to 0.11 in². Therefore, the Grid-Strip initial total steel area for one element is equal to 0.22 in². The center-to-center spacing of the steel bars is equal to 2”. Therefore, the width of the Grid-Strip (b) is also equal to 2”. The Grid-Strip is fabricated from Grade-
65 steel with specified minimum yield strength equal to 65 ksi. Based on the use of the 5 x 5 SCP, coupled with the minimum horizontal spacing, the minimum number of soil reinforcing elements \( (N_{SR}) \) per row is set equal to 2. Calculations will determine the actual number of elements.

Table 5 Soil Reinforcing Parameters

<table>
<thead>
<tr>
<th>Location</th>
<th>Variable</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum number of soil reinforcing</td>
<td>( N_{SR} )</td>
<td>2</td>
<td>dim</td>
</tr>
<tr>
<td>Soil reinforcing width</td>
<td>( b_w )</td>
<td>2.00</td>
<td>in</td>
</tr>
<tr>
<td>Soil reinforcing steel area</td>
<td>( A_S )</td>
<td>0.22</td>
<td>in(^2)</td>
</tr>
<tr>
<td>Yield strength of steel</td>
<td>( F_Y )</td>
<td>65.00</td>
<td>ksi</td>
</tr>
</tbody>
</table>

2.5 Define Backfill Parameters

The backfill is project specific (Table 6), 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). The backfill parameters used in this calculation are specified in the IDEA submittal requirements.
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 ($\beta_i$) 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.

\[
K_{af} = \cos(\beta_i) \left[ \frac{\cos(\beta_i) - \sqrt{\cos(\beta_i)^2 - \cos(\phi_i)^2}}{\cos(\beta_i) + \sqrt{\cos(\beta_i)^2 - \cos(\phi_i)^2}} \right] 
\]

\[
0.333 \quad \text{(dim)} \quad \text{Equation 2-3}
\]

\[
\beta_i = \arctan \left( \frac{S}{2 \cdot H} \right) 
\]

\[
0.00 \quad \text{(deg)} \quad \text{Equation 2-4}
\]

\[
K_{ai} = \tan^2 \left( 45^\circ - \frac{\phi_i}{2} \right) 
\]

\[
0.283 \quad \text{(dim)} \quad \text{Equation 2-5}
\]

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 7 and Table 8.

### Table 7 Load Factors (AASHTO 2014 Tables 3.4.1-1 and 3.4.1-2)

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Load Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>EV</td>
</tr>
<tr>
<td>Strength I (Max)</td>
<td>1.35</td>
</tr>
<tr>
<td>Strength I (Min)</td>
<td>1.00</td>
</tr>
<tr>
<td>Service I</td>
<td>1.00</td>
</tr>
</tbody>
</table>
Table 8 Resistance Factors (AASHTO 2014 Table 11.5.6-1)

<table>
<thead>
<tr>
<th>Resistance Factor</th>
<th>Variable</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sliding of MSE on foundation Soil</td>
<td>$\phi_s$</td>
<td>1.00</td>
<td>dim</td>
</tr>
<tr>
<td>Bearing Resistance</td>
<td>$\phi_b$</td>
<td>0.65</td>
<td>dim</td>
</tr>
<tr>
<td>Tensile Resistance (single point connector)</td>
<td>$\phi_t$</td>
<td>0.75</td>
<td>dim</td>
</tr>
<tr>
<td>Pullout Resistance</td>
<td>$\phi_p$</td>
<td>0.90</td>
<td>dim</td>
</tr>
</tbody>
</table>

### 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 9 and the degradation thickness is calculated using Equation 4-1. The Owner typically specifies the service life. When not specified a minimum service life of 75 years will be used.

\[ E_c = 2 \cdot \left( \text{Life} - \left( \frac{t_{\text{galv}} - t_{\text{zinc1}}}{t_{\text{zinc2}}} + 2 \right) \right) \cdot t_{\text{steel}} \]  

Equation 4-1

<table>
<thead>
<tr>
<th>Location</th>
<th>Variable</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Life</td>
<td>Life</td>
<td>75.00</td>
<td>yrs</td>
</tr>
<tr>
<td>Galvanizing thickness</td>
<td>( t_{\text{Galv}} )</td>
<td>3.40</td>
<td>mil</td>
</tr>
<tr>
<td>Zinc Loss first Two years</td>
<td>( t_{\text{zinc1}} )</td>
<td>0.58</td>
<td>mil</td>
</tr>
<tr>
<td>Zinc Loss remaining years</td>
<td>( t_{\text{zinc2}} )</td>
<td>0.16</td>
<td>mil</td>
</tr>
<tr>
<td>Steel Loss Rate</td>
<td>( t_{\text{steel}} )</td>
<td>0.47</td>
<td>mil</td>
</tr>
<tr>
<td>Steel Loss for given design life (per diameter)</td>
<td>( E_c )</td>
<td>0.055</td>
<td>in</td>
</tr>
</tbody>
</table>

5 Define Internal Stability Factors

The internal stability factors for the lateral stress ratio (\( K_r \)) will be equal to the default values for metal strips and is consistent with AASHTO Article 11.10. The lateral stress ratio factor is taken from Table 11.10.6.2.1-3. The pullout friction factor, \( F^* \), is derived from pullout testing information for the Grid-Strip soil reinforcing. 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 10 Internal Stability Parameters

<table>
<thead>
<tr>
<th>Location</th>
<th>Variable</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earth Pressure Ratio at depth equal to 0.00’</td>
<td>$K_{r0}$</td>
<td>1.70</td>
<td>dim</td>
</tr>
<tr>
<td>Earth Pressure Ratio at depth equal to 20.00’</td>
<td>$K_{r20}$</td>
<td>1.20</td>
<td>dim</td>
</tr>
<tr>
<td>Pullout friction factor at depth equal to 0.00’</td>
<td>$F^*_0$</td>
<td>3.00</td>
<td>dim</td>
</tr>
<tr>
<td>Pullout friction factor at depth equal to 20.00’</td>
<td>$F^*_20$</td>
<td>1.25</td>
<td>dim</td>
</tr>
</tbody>
</table>

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) and the live load (LS).
Table 11  Unfactored Vertical Forces

<table>
<thead>
<tr>
<th>Vertical Force (Force/Length)</th>
<th>Moment Arm (Length)</th>
<th>Load Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equation</td>
<td>Value</td>
<td>Equation</td>
</tr>
<tr>
<td>$V_1 = \gamma_r \cdot H \cdot L$</td>
<td>85.05 k/ft</td>
<td>$h_{v1} = \frac{L}{2}$</td>
</tr>
<tr>
<td>$V_s = q \cdot L$</td>
<td>5.25 k/ft</td>
<td>$h_{vs} = \frac{L}{2}$</td>
</tr>
</tbody>
</table>
### Table 12  Unfactored Vertical Moments

<table>
<thead>
<tr>
<th>Moment (Force-Length/Length)</th>
<th>Load Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{V_1} = V_1 \cdot h_{V_1} )</td>
<td>EV</td>
<td>893.03 k-ft/ft</td>
</tr>
<tr>
<td>( M_{V_S} = V_S \cdot h_{V_S} )</td>
<td>LS</td>
<td>55.13 k-ft/ft</td>
</tr>
</tbody>
</table>

### Table 13  Unfactored Horizontal Forces

<table>
<thead>
<tr>
<th>Vertical Force (Force/Length)</th>
<th>Moment Arm (Length)</th>
<th>Load Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>( F_1 = \frac{1}{2} K_{sf} \cdot \gamma_f \cdot H^2 )</td>
<td>( h_{f_1} = \frac{H}{3} )</td>
<td>EH</td>
</tr>
<tr>
<td>( F_2 = K_{sf} \cdot q \cdot H )</td>
<td>( h_{f_2} = \frac{H}{2} )</td>
<td>LS</td>
</tr>
</tbody>
</table>

### Table 14  Unfactored Horizontal Moments

<table>
<thead>
<tr>
<th>Moment (Force-Length/Length)</th>
<th>Load Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{V_1} = V_1 \cdot h_{V_1} )</td>
<td>EH</td>
</tr>
<tr>
<td>( M_{V_S} = V_s \cdot h_{V_S} )</td>
<td>LS</td>
</tr>
</tbody>
</table>
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 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 structure for the three required load conditions (maximum, minimum and critical check) are illustrated in Table 15 through Table 17. 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 between the foundation soil, $\phi_{fd}$, and the reinforced soil, $\phi_r$, will be used for the sliding analysis.

### Table 15 Evaluation of Sliding at Base of MSE Wall – Strength I Maximum

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads on the MSE wall</td>
<td>$F = \gamma_{EHmax} \cdot F_1 + \gamma_{LSmax} \cdot F_2$</td>
<td>31.37 k/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Wall without LS</td>
<td>$V = \gamma_{EVmax} \cdot V_1$</td>
<td>114.82 k/ft</td>
</tr>
<tr>
<td>Nominal sliding resistance at base of MSE wall</td>
<td>$V_{nem} = V \cdot \tan(\phi_{fd})$</td>
<td>66.29 k/ft</td>
</tr>
<tr>
<td>Sliding resistance at base of MSE wall</td>
<td>$V_{fm} = \phi_s \cdot V_{nem}$</td>
<td>66.29 k/ft</td>
</tr>
<tr>
<td>Is $V_{fm} &gt; F_m$</td>
<td></td>
<td>Yes</td>
</tr>
<tr>
<td>Capacity Demand Ratio (CDR) for sliding</td>
<td>$CDR_s = \frac{V_{nem}}{F}$</td>
<td>2.11</td>
</tr>
</tbody>
</table>

### Table 16 Evaluation of Sliding at Base of MSE Wall – Strength I Minimum

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads on the MSE wall</td>
<td>$F = \gamma_{EHmin} \cdot F_1 + \gamma_{LSmin} \cdot F_2$</td>
<td>20.57 k/ft</td>
</tr>
<tr>
<td>Item</td>
<td>Equation</td>
<td>Value</td>
</tr>
<tr>
<td>-------------------------------------------</td>
<td>--------------------------------------------------------------------------</td>
<td>----------------</td>
</tr>
<tr>
<td>Vertical Load at Base of Wall without LS</td>
<td>$V = \gamma_{EV\text{min}} \cdot V_1$</td>
<td>85.05 k/ft</td>
</tr>
<tr>
<td>Nominal sliding resistance at base of MSE wall</td>
<td>$V_{nm} = V \cdot \tan(\phi_{tu})$</td>
<td>49.10 k/ft</td>
</tr>
<tr>
<td>Sliding resistance at base of MSE wall</td>
<td>$V_Fm = \phi_s \cdot V_{nm}$</td>
<td>49.10 k/ft</td>
</tr>
<tr>
<td>Is $V_{Fm} &gt; F_m$</td>
<td></td>
<td>Yes</td>
</tr>
<tr>
<td>Capacity Demand Ratio (CDR)</td>
<td>$CDR_s = \frac{V_{Fm}}{F}$</td>
<td>2.39</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads on the MSE wall</td>
<td>$F = \gamma_{EHLmax} \cdot F_1 + \gamma_{LSmax} \cdot F_2$</td>
<td>31.37 k/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Wall without LS</td>
<td>$V = \gamma_{EV\text{min}} \cdot V_1$</td>
<td>85.05 k/ft</td>
</tr>
<tr>
<td>Nominal sliding resistance at base of MSE wall</td>
<td>$V_{nm} = V \cdot \tan(\phi_{tu})$</td>
<td>49.10 k/ft</td>
</tr>
<tr>
<td>Sliding resistance at base of MSE wall</td>
<td>$V_Fm = \phi_s \cdot V_{nm}$</td>
<td>49.10 k/ft</td>
</tr>
<tr>
<td>Is $V_{Fm} &gt; F_m$</td>
<td></td>
<td>Yes</td>
</tr>
<tr>
<td>Capacity Demand Ratio (CDR)</td>
<td>$CDR_s = \frac{V_{Fm}}{F}$</td>
<td>1.57</td>
</tr>
</tbody>
</table>

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 17).

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 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 18 through Table 20. 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>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads on the MSE wall</td>
<td>( F = \gamma_{EH_{\max}} \cdot F_1 + \gamma_{LS_{\max}} \cdot F_2 )</td>
<td>31.37 k/ft</td>
</tr>
<tr>
<td>Overturning Moment about point A</td>
<td>( M_{OA} = \gamma_{EH_{\max}} \cdot M_{1} + \gamma_{LS_{\max}} \cdot M_{2} )</td>
<td>335.62 (k-ft)/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Wall without LS</td>
<td>( V_A = \gamma_{EV_{\max}} \cdot V_1 )</td>
<td>114.82 k/ft</td>
</tr>
<tr>
<td>Resisting moment about point A without LS</td>
<td>( M_{RA} = \gamma_{EV_{\max}} \cdot M_{V1} )</td>
<td>1205.58 (k-ft)/ft</td>
</tr>
<tr>
<td>Net moment about Point A</td>
<td>( M_A = M_{RA} - M_{OA} )</td>
<td>869.96 (k-ft)/ft</td>
</tr>
<tr>
<td>Location of resultant force on base of wall from point A</td>
<td>( a = \frac{M_A}{V_A} )</td>
<td>7.58 ft</td>
</tr>
<tr>
<td>Eccentricity at base of MSE wall</td>
<td>( e_L = \frac{L}{2} - a )</td>
<td>2.92 ft</td>
</tr>
<tr>
<td>Limiting eccentricity</td>
<td>( e_{\text{limit}} = \frac{L}{3} )</td>
<td>7.00 ft</td>
</tr>
<tr>
<td>Is the resultant within limiting value of e</td>
<td></td>
<td>Yes</td>
</tr>
<tr>
<td>Calculated e/L</td>
<td>( \frac{e_L}{L} )</td>
<td>0.14 dim</td>
</tr>
<tr>
<td>Capacity Demand Ratio (CDR) for Overturning</td>
<td>( CDR_O = \frac{M_{RA}}{M_{OA}} )</td>
<td>3.59</td>
</tr>
</tbody>
</table>
Table 19  Evaluation of Limiting Eccentricity for MSE Wall – Strength I Minimum

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads on the MSE wall</td>
<td>$F = \gamma_{\text{EHmin}} \cdot F_1 + \gamma_{\text{LSmin}} \cdot F_2$</td>
<td>20.57 k/ft</td>
</tr>
<tr>
<td>Overturning Moment about point A</td>
<td>$M_{OA} = \gamma_{\text{EHmin}} \cdot M_{F1} + \gamma_{\text{LSmin}} \cdot M_{F2}$</td>
<td>227.62 (k-ft)/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Wall without LS</td>
<td>$V_A = \gamma_{\text{EVmin}} \cdot V_1$</td>
<td>85.05 k/ft</td>
</tr>
<tr>
<td>Resisting moment about point A without LS</td>
<td>$M_{RA} = \gamma_{\text{EVmin}} \cdot M_{V1}$</td>
<td>893.03 (k-ft)/ft</td>
</tr>
<tr>
<td>Net moment about Point A</td>
<td>$M_A = M_{RA} - M_{OA}$</td>
<td>665.40 (k-ft)/ft</td>
</tr>
<tr>
<td>Location of resultant force on base of wall from point A</td>
<td>$a = \frac{M_A}{V_A}$</td>
<td>7.82 ft</td>
</tr>
<tr>
<td>Eccentricity at base of MSE wall</td>
<td>$e_L = \frac{L}{2} - a$</td>
<td>2.68 ft</td>
</tr>
<tr>
<td>Limiting eccentricity</td>
<td>$e_{\text{limit}} = \frac{L}{3}$</td>
<td>7.00 ft</td>
</tr>
<tr>
<td>Is the resultant within limiting value of e</td>
<td></td>
<td>Yes</td>
</tr>
<tr>
<td>Calculated e/L</td>
<td>$\frac{e_L}{L}$</td>
<td>0.13 dim</td>
</tr>
<tr>
<td>Capacity Demand Ratio (CDR) for Overturning</td>
<td>$\text{CDR}<em>O = \frac{M</em>{RA}}{M_{OA}}$</td>
<td>3.92</td>
</tr>
</tbody>
</table>

Table 20  Evaluation of Limiting Eccentricity for MSE Wall – Strength I Critical Value Check

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads on the MSE wall</td>
<td>$F = \gamma_{\text{EHmax}} \cdot F_1 + \gamma_{\text{LSmax}} \cdot F_2$</td>
<td>31.37 k/ft</td>
</tr>
<tr>
<td>Overturning Moment about point A</td>
<td>$M_{OA} = \gamma_{\text{EHmax}} \cdot M_{F1} + \gamma_{\text{LSmax}} \cdot M_{F2}$</td>
<td>335.62 (k-ft)/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Wall without LS</td>
<td>$V_A = \gamma_{\text{EVmin}} \cdot V_1$</td>
<td>85.05 k/ft</td>
</tr>
</tbody>
</table>
### Table 20 Evaluation of Limiting Eccentricity for MSE Wall – Strength I Critical Value Check

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resisting moment about point A without LS</td>
<td>$M_{RA} = \gamma_{E_{Vmin}} \cdot M_{V1}$</td>
<td>893.03 (k-ft)/ft</td>
</tr>
<tr>
<td>Net moment about Point A</td>
<td>$M_A = M_{RA} - M_{OA}$</td>
<td>557.40 (k-ft)/ft</td>
</tr>
<tr>
<td>Location of resultant force on base of wall from point A</td>
<td>$a = \frac{M_A}{V_A}$</td>
<td>6.55 ft</td>
</tr>
<tr>
<td>Eccentricity at base of MSE wall</td>
<td>$e_L = \frac{L}{2} - a$</td>
<td>3.95 ft</td>
</tr>
<tr>
<td>Limiting eccentricity</td>
<td>$e_{limit} = \frac{L}{3}$</td>
<td>7.00 ft</td>
</tr>
<tr>
<td>Is the resultant within limiting value of e</td>
<td></td>
<td>Yes</td>
</tr>
<tr>
<td>Effective width at base of MSE wall</td>
<td>$B' = L - 2 \cdot e_L$</td>
<td>13.11 ft</td>
</tr>
<tr>
<td>Calculated e/L</td>
<td>$\frac{e_L}{L}$</td>
<td>0.19 dim</td>
</tr>
<tr>
<td>Capacity Demand Ratio (CDR) for Overturning</td>
<td>$CDR_O = \frac{M_{RA}}{M_{OA}}$</td>
<td>2.66</td>
</tr>
</tbody>
</table>

The critical value check results in the largest eccentricity for this example and therefore is the most severe load effect and governs the limiting eccentricity (Table 20).

### 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.

$$\sigma_v = \frac{\sum V}{L - 2 \cdot e_L}$$  

Equation 7-1
Where the value $\Sigma V = R = V_1 + V_5$ 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 compared to the service limit state bearing resistance. The various calculations for evaluation of bearing resistance are presented in Table 21 through Table 24. The Service I load combination is evaluated to compute the bearing stress for any settlement analysis and the results are presented in Table 24.

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads on the MSE wall</td>
<td>$F = \gamma_{EHmax} \cdot F_1 + \gamma_{LSmax} \cdot F_2$</td>
<td>31.37 k/ft</td>
</tr>
<tr>
<td>Overturning Moment about point A</td>
<td>$M_{OA} = \gamma_{EHmax} \cdot M_1 + \gamma_{LSmax} \cdot M_2$</td>
<td>335.62 (k-ft)/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Wall with LS</td>
<td>$V_A = \gamma_{EVmax} \cdot V_1 + \gamma_{LSmax} \cdot V_5$</td>
<td>124.01 k/ft</td>
</tr>
<tr>
<td>Resisting moment about point A with LS</td>
<td>$M_{RA} = \gamma_{EVmax} \cdot M_1 + \gamma_{LSmax} \cdot M_2$</td>
<td>1302.05 (k-ft)/ft</td>
</tr>
<tr>
<td>Net moment about Point A</td>
<td>$M_A = M_{RA} - M_{OA}$</td>
<td>966.43 (k-ft)/ft</td>
</tr>
<tr>
<td>Location of resultant force on base of wall from point A</td>
<td>$a = \frac{M_A}{V_A}$</td>
<td>7.79 ft</td>
</tr>
<tr>
<td>Eccentricity at base of MSE wall from center</td>
<td>$e_L = \frac{L}{2} - a$</td>
<td>2.71 ft</td>
</tr>
<tr>
<td>Limiting eccentricity</td>
<td>$e_{limit} = \frac{L}{3}$</td>
<td>7.00 ft</td>
</tr>
<tr>
<td>Is the resultant within limiting value of $e$</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>Effective width at base of MSE wall</td>
<td>$B' = L - 2 \cdot e_L$</td>
<td>15.59 ft</td>
</tr>
<tr>
<td>Bearing stress due to MSE Wall</td>
<td>$\sigma_v = \frac{\gamma_{EVmax} \cdot V_1 + \gamma_{LSmax} \cdot V_5}{L - 2 \cdot e_L}$</td>
<td>7.96 ksf</td>
</tr>
</tbody>
</table>
### Table 22  Evaluation of Bearing Resistance for MSE Wall – Strength I Minimum

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads on the MSE wall</td>
<td>( F = \gamma_{EH\text{min}} \cdot F_1 + \gamma_{LS\text{min}} \cdot F_2 )</td>
<td>20.57 k/ft</td>
</tr>
<tr>
<td>Overturning Moment about point A</td>
<td>( M_{OA} = \gamma_{EH\text{min}} \cdot M_{F1} + \gamma_{LS\text{min}} \cdot M_{F2} )</td>
<td>227.62 (k-ft)/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Wall with LS</td>
<td>( V_A = \gamma_{EV\text{min}} \cdot V_1 + \gamma_{LS\text{min}} \cdot V_S )</td>
<td>94.24 k/ft</td>
</tr>
<tr>
<td>Resisting moment about point A with LS</td>
<td>( M_{RA} = \gamma_{EV\text{min}} \cdot M_{V1} + \gamma_{LS\text{min}} \cdot M_{V5} )</td>
<td>989.49 (k-ft)/ft</td>
</tr>
<tr>
<td>Net moment about Point A</td>
<td>( M_A = M_{RA} - M_{OA} )</td>
<td>761.87 (k-ft)/ft</td>
</tr>
<tr>
<td>Location of resultant force on base of wall from point A</td>
<td>( a = \frac{M_A}{V_A} )</td>
<td>8.08 ft</td>
</tr>
<tr>
<td>Eccentricity at base of MSE wall from center</td>
<td>( e_L = \frac{L}{2} - a )</td>
<td>2.42 ft</td>
</tr>
<tr>
<td>Limiting eccentricity</td>
<td>( e_{\text{limit}} = \frac{L}{3} )</td>
<td>7.00 ft</td>
</tr>
<tr>
<td>Is the resultant within limiting value of e</td>
<td></td>
<td>Yes</td>
</tr>
<tr>
<td>Effective width at base of MSE wall</td>
<td>( B' = L - 2 \cdot e_L )</td>
<td>16.17 ft</td>
</tr>
<tr>
<td>Bearing stress due to MSE Wall</td>
<td>( \sigma_y = \frac{\gamma_{EV\text{min}} \cdot V_1 + \gamma_{LS\text{min}} \cdot V_S}{L - 2 \cdot e_L} )</td>
<td>5.83 ksf</td>
</tr>
</tbody>
</table>

### Table 23  Evaluation of Bearing Resistance for MSE Wall – Strength I Critical Value Check

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads on the MSE wall</td>
<td>( H_m = \gamma_{EH\text{max}} \cdot F_1 + \gamma_{LS\text{max}} \cdot F_2 )</td>
<td>31.37 k/ft</td>
</tr>
<tr>
<td>Overturning Moment about point A</td>
<td>( M_{OA} = \gamma_{EH\text{max}} \cdot M_{F1} + \gamma_{LS\text{max}} \cdot M_{F2} )</td>
<td>335.62 (k-ft)/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Wall with LS</td>
<td>( V_A = \gamma_{EV\text{min}} \cdot V_1 + \gamma_{LS\text{min}} \cdot V_S )</td>
<td>94.24 k/ft</td>
</tr>
<tr>
<td>Resisting moment about point A with LS</td>
<td>( M_{RA} = \gamma_{EV\text{min}} \cdot M_{V1} + \gamma_{LS\text{min}} \cdot M_{V5} )</td>
<td>989.49 (k-ft)/ft</td>
</tr>
<tr>
<td>Net moment about Point A</td>
<td>( M_A = M_{RA} - M_{OA} )</td>
<td>653.87 (k-ft)/ft</td>
</tr>
</tbody>
</table>
### Table 23 Evaluation of Bearing Resistance for MSE Wall – Strength I Critical Value Check

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location of resultant force on base of wall from point A</td>
<td>( a = \frac{M_A}{V_A} )</td>
<td>6.94 ft</td>
</tr>
<tr>
<td>Eccentricity at base of MSE wall from center</td>
<td>( e_L = \frac{L}{2} - a )</td>
<td>3.56 ft</td>
</tr>
<tr>
<td>Limiting eccentricity</td>
<td>( e_{\text{limit}} = \frac{L}{3} )</td>
<td>7.00 ft</td>
</tr>
<tr>
<td>Is the resultant within limiting value of e</td>
<td></td>
<td>Yes</td>
</tr>
<tr>
<td>Effective width at base of MSE wall</td>
<td>( B' = L - 2 \cdot e_L )</td>
<td>13.88 ft</td>
</tr>
<tr>
<td>Bearing stress due to MSE Wall</td>
<td>[ \sigma_v = \gamma_{EV\min} \cdot V_1 + \gamma_{LS\min} \cdot V_S ] ( L - 2 \cdot e_L )</td>
<td>6.79 ksf</td>
</tr>
</tbody>
</table>

### Table 24 Evaluation of Bearing Resistance for MSE Wall – Strength I Service

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads n the MSE wall</td>
<td>( F = F_1 + F_2 )</td>
<td>20.50 k/ft</td>
</tr>
<tr>
<td>Overturning Moment about point A</td>
<td>( M_{OA} = F_1 \cdot h_{f1} + F_2 \cdot h_{f2} )</td>
<td>217.50 (k-ft)/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Wall without LS</td>
<td>( V_A = V_1 + V_S )</td>
<td>90.30 k/ft</td>
</tr>
<tr>
<td>Resisting moment about point A without LS</td>
<td>( M_{RA} = V_1 \cdot h_{V1} + V_S \cdot h_{VS} )</td>
<td>948.15 (k-ft)/ft</td>
</tr>
<tr>
<td>Net moment about Point A</td>
<td>( M_A = M_{RA} - M_{OA} )</td>
<td>730.65 (k-ft)/ft</td>
</tr>
<tr>
<td>Location of resultant force on base of wall from point A</td>
<td>( a = \frac{M_A}{V_A} )</td>
<td>8.09 ft</td>
</tr>
<tr>
<td>Eccentricity at base of MSE wall from center</td>
<td>( e_L = \frac{L}{2} - a )</td>
<td>2.41 ft</td>
</tr>
<tr>
<td>Limiting eccentricity</td>
<td>( e_{\text{limit}} = \frac{L}{6} )</td>
<td>3.50 ft</td>
</tr>
<tr>
<td>Is the resultant within limiting value of e</td>
<td></td>
<td>Yes</td>
</tr>
</tbody>
</table>
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 21).

### 7.4 Summary of External Stability

Table 25 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 must be greater or equal to 1.00. All comparable values are within 2%.

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>CDR Sliding (dim)</th>
<th>Limiting Eccentricity/Maximum (ft)</th>
<th>CDR Overturning (dim)</th>
<th>Bearing Stress (ksf)</th>
<th>Effective Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength I Maximum</td>
<td>2.11</td>
<td>2.92/7.00</td>
<td>3.59</td>
<td>7.96</td>
<td>15.59</td>
</tr>
<tr>
<td>Strength I Minimum</td>
<td>2.39</td>
<td>2.68/7.00</td>
<td>3.92</td>
<td>5.83</td>
<td>16.17</td>
</tr>
<tr>
<td>Strength I Critical Value Check</td>
<td>1.57</td>
<td>3.95/7.00</td>
<td>2.66</td>
<td>6.79</td>
<td>13.88</td>
</tr>
<tr>
<td>Service I</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>5.58</td>
<td>16.18</td>
</tr>
</tbody>
</table>

### 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](image1)

![Bottom of Structure](image2)

**Figure 8-1** Soil Reinforcing Depth Parameters

<table>
<thead>
<tr>
<th>Soil Element</th>
<th>Distance Above Leveling Pad (ft)</th>
<th>Depth Below Top of Wall (Zi) (ft)</th>
<th>Vertical Spacing (Sv) (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>27.75</td>
<td>2.25</td>
<td>3.00</td>
</tr>
<tr>
<td>11</td>
<td>26.25</td>
<td>3.75</td>
<td>2.00</td>
</tr>
<tr>
<td>10</td>
<td>23.75</td>
<td>6.25</td>
<td>2.50</td>
</tr>
<tr>
<td>9</td>
<td>21.25</td>
<td>8.75</td>
<td>2.50</td>
</tr>
<tr>
<td>8</td>
<td>18.75</td>
<td>11.25</td>
<td>2.50</td>
</tr>
<tr>
<td>7</td>
<td>16.25</td>
<td>13.75</td>
<td>2.50</td>
</tr>
<tr>
<td>6</td>
<td>13.75</td>
<td>16.25</td>
<td>2.50</td>
</tr>
<tr>
<td>5</td>
<td>11.25</td>
<td>18.75</td>
<td>2.50</td>
</tr>
<tr>
<td>4</td>
<td>8.75</td>
<td>21.25</td>
<td>2.50</td>
</tr>
<tr>
<td>3</td>
<td>6.25</td>
<td>23.75</td>
<td>2.50</td>
</tr>
</tbody>
</table>
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.

\[
H_m = \frac{X_s > 0.3 \cdot (H + S)}{1 - 0.3 \cdot \tan(\beta_s) \cdot H + S + \tan(\beta_s) \cdot 0.3 \cdot H}
\]

Equation 8-1

For a horizontal back-slope with no earth surcharge the mechanical height is equal to the design height of the structure, \(H\).
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 $z_i$.

$$K_i = \left[ 1.7 - \frac{1.2}{20(\text{ft})} \right] \cdot K_s \rightarrow z_i < 20(\text{ft})$$  \hspace{1cm} \text{Equation 8-2}

$$K_i = 1.2 \cdot (K_s) \rightarrow z_i \geq 20(\text{ft})$$  \hspace{1cm} \text{Equation 8-3}

$$F^* = 3.00 - \frac{3.00 - 1.25}{20(\text{ft})} \cdot z_i \rightarrow z_i < 20(\text{ft})$$  \hspace{1cm} \text{Equation 8-4}

$$F^* = 1.25 \rightarrow z_i \geq 20(\text{ft})$$  \hspace{1cm} \text{Equation 8-5}

<table>
<thead>
<tr>
<th>Soil Element</th>
<th>Depth Below Top of Wall $Z_i$ (ft)</th>
<th>Earth Pressure Ratio $K_i$ (dim)</th>
<th>Pullout Friction Factor $F^*$ (dim)</th>
<th>Length of Embedment $L_e$ (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>2.25</td>
<td>1.64</td>
<td>2.80</td>
<td>12.00</td>
</tr>
<tr>
<td>11</td>
<td>3.75</td>
<td>1.61</td>
<td>2.67</td>
<td>12.00</td>
</tr>
<tr>
<td>10</td>
<td>6.25</td>
<td>1.54</td>
<td>2.45</td>
<td>12.00</td>
</tr>
<tr>
<td>9</td>
<td>8.75</td>
<td>1.48</td>
<td>2.23</td>
<td>12.00</td>
</tr>
<tr>
<td>8</td>
<td>11.25</td>
<td>1.42</td>
<td>2.02</td>
<td>12.00</td>
</tr>
<tr>
<td>7</td>
<td>13.75</td>
<td>1.36</td>
<td>1.80</td>
<td>12.00</td>
</tr>
<tr>
<td>6</td>
<td>16.25</td>
<td>1.29</td>
<td>1.58</td>
<td>12.75</td>
</tr>
<tr>
<td>5</td>
<td>18.75</td>
<td>1.23</td>
<td>1.36</td>
<td>14.25</td>
</tr>
<tr>
<td>4</td>
<td>21.25</td>
<td>1.20</td>
<td>1.25</td>
<td>15.75</td>
</tr>
<tr>
<td>3</td>
<td>23.75</td>
<td>1.20</td>
<td>1.25</td>
<td>17.25</td>
</tr>
<tr>
<td>2</td>
<td>26.25</td>
<td>1.20</td>
<td>1.25</td>
<td>18.75</td>
</tr>
<tr>
<td>1</td>
<td>28.75</td>
<td>1.20</td>
<td>1.25</td>
<td>20.25</td>
</tr>
</tbody>
</table>

8.3 Calculate Horizontal Stress at Elevation of Each Soil Reinforcing Element

The horizontal stress at each soil reinforcing 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.
\[ \sigma_v = \gamma_v \cdot (\gamma_e \cdot z_i + q) \]  

Equation 8-6

\[ \sigma_h = k_i \cdot \sigma_v \]  

Equation 8-7

### Table 28 Stress Calculation

<table>
<thead>
<tr>
<th>Soil Element</th>
<th>Depth Below Top of Wall (ft)</th>
<th>Vertical Stress ((\sigma_v)) sf</th>
<th>Earth Pressure Coefficient ((k_i)) dim</th>
<th>Horizontal Stress ((\sigma_h)) ksf</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>2.25</td>
<td>0.75</td>
<td>0.46</td>
<td>0.35</td>
</tr>
<tr>
<td>11</td>
<td>3.75</td>
<td>1.02</td>
<td>0.46</td>
<td>0.46</td>
</tr>
<tr>
<td>10</td>
<td>6.25</td>
<td>1.48</td>
<td>0.44</td>
<td>0.64</td>
</tr>
<tr>
<td>9</td>
<td>8.75</td>
<td>1.93</td>
<td>0.42</td>
<td>0.81</td>
</tr>
<tr>
<td>8</td>
<td>11.25</td>
<td>2.39</td>
<td>0.40</td>
<td>0.96</td>
</tr>
<tr>
<td>7</td>
<td>13.75</td>
<td>2.84</td>
<td>0.38</td>
<td>1.09</td>
</tr>
<tr>
<td>6</td>
<td>16.25</td>
<td>3.30</td>
<td>0.36</td>
<td>1.20</td>
</tr>
<tr>
<td>5</td>
<td>18.75</td>
<td>3.75</td>
<td>0.35</td>
<td>1.31</td>
</tr>
<tr>
<td>4</td>
<td>21.25</td>
<td>4.21</td>
<td>0.34</td>
<td>1.43</td>
</tr>
<tr>
<td>3</td>
<td>23.75</td>
<td>4.67</td>
<td>0.34</td>
<td>1.58</td>
</tr>
<tr>
<td>2</td>
<td>26.25</td>
<td>5.12</td>
<td>0.34</td>
<td>1.74</td>
</tr>
<tr>
<td>1</td>
<td>28.75</td>
<td>5.58</td>
<td>0.34</td>
<td>1.89</td>
</tr>
</tbody>
</table>

### 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).
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).

\[ T_{\text{max}} = \sigma_H \cdot A_T \]  

Equation 8-10

<table>
<thead>
<tr>
<th>Soil Element</th>
<th>Depth Below Top of Wall</th>
<th>Horizontal Stress $\sigma_H$ ksf</th>
<th>Tributary Area $A_T$ sf</th>
<th>Maximum Tension $T_{\text{max}}$ kip</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>2.25</td>
<td>0.35</td>
<td>15.00</td>
<td>5.25</td>
</tr>
<tr>
<td>11</td>
<td>3.75</td>
<td>0.46</td>
<td>10.00</td>
<td>4.60</td>
</tr>
<tr>
<td>10</td>
<td>6.25</td>
<td>0.64</td>
<td>12.50</td>
<td>8.00</td>
</tr>
<tr>
<td>9</td>
<td>8.75</td>
<td>0.81</td>
<td>12.50</td>
<td>10.13</td>
</tr>
<tr>
<td>8</td>
<td>11.25</td>
<td>0.96</td>
<td>12.50</td>
<td>12.00</td>
</tr>
</tbody>
</table>
### 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).

\[
A_c = \frac{T_{\text{max}}}{\phi \cdot F_y} \quad \text{Equation 8-11}
\]

\[
A_{\text{req}} = \frac{\pi}{4} \left( \frac{4 \cdot A_c}{\pi} + E_c \right)^2 \quad \text{Equation 8-12}
\]

\[
N_{SR} = \text{int} \left[ \frac{A_{\text{req}}}{A_{GS}} \right] + 1 \quad \text{Equation 8-13}
\]

Where:
- \( N_{SR} = \) Number of Grid-Strips
- \( A_{GS} = \) Area of one Grid-Strip

The value \( N_{SR} \) is the integer value of the quotient plus 1.
Table 30  Area of Steel Required for Rupture

<table>
<thead>
<tr>
<th>Soil Element</th>
<th>Depth Below Top of Wall (ft)</th>
<th>Maximum Tension (kips)</th>
<th>Area of Steel Required at End of Design Life (in²)</th>
<th>Area of Steel Provided (in²)</th>
<th>Number of Soil Reinforcing Elements (Nsr)</th>
<th>CDR (dim)</th>
<th>Rupture (dim)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>2.25</td>
<td>5.25</td>
<td>0.11</td>
<td>0.32</td>
<td>2</td>
<td></td>
<td>2.96</td>
</tr>
<tr>
<td>11</td>
<td>3.75</td>
<td>4.60</td>
<td>0.09</td>
<td>0.32</td>
<td>2</td>
<td></td>
<td>3.38</td>
</tr>
<tr>
<td>10</td>
<td>6.25</td>
<td>8.00</td>
<td>0.16</td>
<td>0.32</td>
<td>2</td>
<td></td>
<td>1.95</td>
</tr>
<tr>
<td>9</td>
<td>8.75</td>
<td>10.13</td>
<td>0.21</td>
<td>0.32</td>
<td>2</td>
<td></td>
<td>1.54</td>
</tr>
<tr>
<td>8</td>
<td>11.25</td>
<td>12.00</td>
<td>0.25</td>
<td>0.32</td>
<td>2</td>
<td></td>
<td>1.30</td>
</tr>
<tr>
<td>7</td>
<td>13.75</td>
<td>13.63</td>
<td>0.28</td>
<td>0.32</td>
<td>2</td>
<td></td>
<td>1.14</td>
</tr>
<tr>
<td>6</td>
<td>16.25</td>
<td>15.00</td>
<td>0.31</td>
<td>0.32</td>
<td>2</td>
<td></td>
<td>1.04</td>
</tr>
<tr>
<td>5</td>
<td>18.75</td>
<td>16.38</td>
<td>0.34</td>
<td>0.48</td>
<td>3</td>
<td></td>
<td>1.43</td>
</tr>
<tr>
<td>4</td>
<td>21.25</td>
<td>17.88</td>
<td>0.37</td>
<td>0.48</td>
<td>3</td>
<td></td>
<td>1.31</td>
</tr>
<tr>
<td>3</td>
<td>23.75</td>
<td>19.75</td>
<td>0.41</td>
<td>0.48</td>
<td>3</td>
<td></td>
<td>1.18</td>
</tr>
<tr>
<td>2</td>
<td>26.25</td>
<td>21.75</td>
<td>0.45</td>
<td>0.48</td>
<td>3</td>
<td></td>
<td>1.07</td>
</tr>
<tr>
<td>1</td>
<td>28.75</td>
<td>23.63</td>
<td>0.48</td>
<td>0.64</td>
<td>4</td>
<td></td>
<td>1.32</td>
</tr>
</tbody>
</table>

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 ($\sigma_v$), scale correction factor ($\alpha$), and the geometric factor (C). The factored pullout resistance ($P_r$) is the product of the nominal pullout resistance ($P_t$) multiplied by the pullout resistance factor, $\phi_{po}$ (Equation 8-16).

\[
\sigma_v = \gamma_r \cdot Z_i \quad \text{Equation 8-14}
\]

\[
P_r = \alpha \cdot C \cdot F^* \cdot \sigma_v \cdot L_e \quad \text{Equation 8-15}
\]

\[
P_o = \phi_{po} \cdot P_r \quad \text{Equation 8-16}
\]

Where:

\[
C = 2 \text{ (dim)}
\]

\[
\alpha = 1 \text{ (dim)}
\]
The number of Grid-Strip 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 $T_{\text{max}}$ 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 $T_{\text{max}}$ for pullout. In this document, the live load surcharge is not used in the calculation of $T_{\text{max}}$.

\[
T_{\text{max}} = \gamma_{\text{EV}} \cdot (K_i \cdot \gamma_r \cdot z_i) \cdot A_T
\]

Equation 8-17

\[
n_{GS} = \frac{T_{\text{max}}}{P_\pi}
\]

Equation 8-18

### Table 31
Pullout Resistance for Grid-Strip Element

<table>
<thead>
<tr>
<th>SR Element</th>
<th>Depth Below Top of Wall $Z_i$ ft</th>
<th>Length Of Embed. Le Ft</th>
<th>F* $\text{dim}$</th>
<th>Nominal Pullout Resistance for 1-SR $P_r$ kip</th>
<th>Factored Pullout Resistance $P_\pi$ kip</th>
<th>Max Tension $T_{\text{max}}$ kip</th>
<th>Number SR Elements $N_{SR}$</th>
<th>CDR Pullout $\text{dim}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>2.25</td>
<td>12.00</td>
<td>2.80</td>
<td>3.40</td>
<td>3.06</td>
<td>2.85</td>
<td>2</td>
<td>2.15</td>
</tr>
<tr>
<td>11</td>
<td>3.75</td>
<td>12.00</td>
<td>2.67</td>
<td>5.41</td>
<td>4.87</td>
<td>3.11</td>
<td>2</td>
<td>3.13</td>
</tr>
<tr>
<td>10</td>
<td>6.25</td>
<td>12.00</td>
<td>2.45</td>
<td>8.27</td>
<td>7.44</td>
<td>6.20</td>
<td>2</td>
<td>2.40</td>
</tr>
<tr>
<td>9</td>
<td>8.75</td>
<td>12.00</td>
<td>2.33</td>
<td>10.54</td>
<td>9.48</td>
<td>8.34</td>
<td>2</td>
<td>2.27</td>
</tr>
<tr>
<td>8</td>
<td>11.25</td>
<td>12.00</td>
<td>2.02</td>
<td>12.27</td>
<td>11.04</td>
<td>10.29</td>
<td>2</td>
<td>2.15</td>
</tr>
<tr>
<td>7</td>
<td>13.75</td>
<td>12.00</td>
<td>1.80</td>
<td>13.37</td>
<td>12.03</td>
<td>12.04</td>
<td>2</td>
<td>2.00</td>
</tr>
<tr>
<td>6</td>
<td>16.25</td>
<td>12.75</td>
<td>1.58</td>
<td>14.73</td>
<td>13.26</td>
<td>13.50</td>
<td>2</td>
<td>1.96</td>
</tr>
<tr>
<td>5</td>
<td>18.75</td>
<td>14.25</td>
<td>1.36</td>
<td>16.35</td>
<td>14.72</td>
<td>14.85</td>
<td>2</td>
<td>1.98</td>
</tr>
<tr>
<td>4</td>
<td>21.25</td>
<td>15.75</td>
<td>1.25</td>
<td>18.83</td>
<td>16.94</td>
<td>16.42</td>
<td>2</td>
<td>2.06</td>
</tr>
<tr>
<td>3</td>
<td>23.75</td>
<td>17.25</td>
<td>1.25</td>
<td>23.04</td>
<td>20.74</td>
<td>18.36</td>
<td>2</td>
<td>2.26</td>
</tr>
<tr>
<td>2</td>
<td>26.25</td>
<td>18.75</td>
<td>1.25</td>
<td>27.69</td>
<td>24.92</td>
<td>20.29</td>
<td>2</td>
<td>2.46</td>
</tr>
<tr>
<td>1</td>
<td>28.75</td>
<td>20.25</td>
<td>1.25</td>
<td>32.75</td>
<td>29.47</td>
<td>22.22</td>
<td>2</td>
<td>2.65</td>
</tr>
</tbody>
</table>

### 8.7 Summary of Internal Stability at Elevation of Each Soil Reinforcing Element

The number of Grid-Strips that are required at each soil reinforcing elevation is equal to the greater number between the Grid-Strips calculated for rupture in Section 8.5, and the number of Grid-Strips calculated for pullout in Section 8.6. The capacity demand ratios calculated and summarized in...
Table 32 are based on the minimum number of Grid-Strips that are required to satisfy both rupture and pullout.

### Table 32  
**Summary of Internal Stability for Grid-Strip Element**

<table>
<thead>
<tr>
<th>SR</th>
<th>$Z_i$ ft</th>
<th>$N_{SR}$ dim</th>
<th>$\sigma_v$ ksf</th>
<th>$K_r$ dim</th>
<th>$\sigma_H$ ksf</th>
<th>$T_{max}$ kip</th>
<th>CDR Rupture dim</th>
<th>$L_e$ Ft</th>
<th>$F^*$ dim</th>
<th>CDR Pullout dim</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>2.25</td>
<td>2</td>
<td>0.75</td>
<td>0.46</td>
<td>0.35</td>
<td>5.25</td>
<td>2.96</td>
<td>12.00</td>
<td>2.80</td>
<td>2.15</td>
</tr>
<tr>
<td>11</td>
<td>3.75</td>
<td>2</td>
<td>1.02</td>
<td>0.46</td>
<td>0.46</td>
<td>4.60</td>
<td>3.38</td>
<td>12.00</td>
<td>2.67</td>
<td>3.13</td>
</tr>
<tr>
<td>10</td>
<td>6.25</td>
<td>2</td>
<td>1.48</td>
<td>0.44</td>
<td>0.64</td>
<td>8.00</td>
<td>1.95</td>
<td>12.00</td>
<td>2.45</td>
<td>2.40</td>
</tr>
<tr>
<td>9</td>
<td>8.75</td>
<td>2</td>
<td>1.93</td>
<td>0.42</td>
<td>0.81</td>
<td>10.13</td>
<td>1.54</td>
<td>12.00</td>
<td>2.23</td>
<td>2.27</td>
</tr>
<tr>
<td>8</td>
<td>11.25</td>
<td>2</td>
<td>2.39</td>
<td>0.40</td>
<td>0.96</td>
<td>12.00</td>
<td>1.30</td>
<td>12.00</td>
<td>2.02</td>
<td>2.15</td>
</tr>
<tr>
<td>7</td>
<td>13.75</td>
<td>2</td>
<td>2.84</td>
<td>0.38</td>
<td>1.09</td>
<td>13.63</td>
<td>1.14</td>
<td>12.00</td>
<td>1.80</td>
<td>2.00</td>
</tr>
<tr>
<td>6</td>
<td>16.25</td>
<td>2</td>
<td>3.30</td>
<td>0.36</td>
<td>1.20</td>
<td>15.00</td>
<td>1.04</td>
<td>12.75</td>
<td>1.58</td>
<td>1.96</td>
</tr>
<tr>
<td>5</td>
<td>18.75</td>
<td>3</td>
<td>3.75</td>
<td>0.35</td>
<td>1.31</td>
<td>16.38</td>
<td>1.43</td>
<td>14.25</td>
<td>1.36</td>
<td>2.97</td>
</tr>
<tr>
<td>4</td>
<td>21.25</td>
<td>3</td>
<td>4.21</td>
<td>0.34</td>
<td>1.43</td>
<td>17.88</td>
<td>1.31</td>
<td>15.75</td>
<td>1.25</td>
<td>3.10</td>
</tr>
<tr>
<td>3</td>
<td>23.75</td>
<td>3</td>
<td>4.67</td>
<td>0.34</td>
<td>1.58</td>
<td>19.75</td>
<td>1.18</td>
<td>17.25</td>
<td>1.25</td>
<td>3.39</td>
</tr>
<tr>
<td>2</td>
<td>26.25</td>
<td>3</td>
<td>5.12</td>
<td>0.34</td>
<td>1.74</td>
<td>21.75</td>
<td>1.07</td>
<td>18.75</td>
<td>1.25</td>
<td>3.68</td>
</tr>
<tr>
<td>1</td>
<td>28.75</td>
<td>4</td>
<td>5.58</td>
<td>0.34</td>
<td>1.89</td>
<td>23.63</td>
<td>1.32</td>
<td>20.25</td>
<td>1.25</td>
<td>5.31</td>
</tr>
</tbody>
</table>

### 8.8 MSEW Internal Stability Discussion

For MSEW to be compared to MSE-Pro and AASHTO requirements the external stability analysis and internal stability analysis must be performed 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 consistent load factor of 1.35 (EV) for all internal loads, in other words, it does not use 1.75 (LS) for traffic live load surcharge. The use of 1.35 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. Based on MSEW definitions, the program uses an average horizontal pressure at each level of soil reinforcing, and therefore, in the 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 element is defined at elevation 27.75 (a depth of 2.25 feet from the top of wall). The second soil reinforcing element 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 3.686 and 3.258 respectively. In the program MSE-Pro the calculated CDR for the top soil reinforcing element and the second soil reinforcing element is 2.96 and 3.38 respectively. MSEW over predicts the CDR in the top row by 25%. 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 and MSEW underestimates the maximum tension force that is required to be resisted.

MSE-Pro does not use the average horizontal pressure to calculate the maximum tensile force. MSE-Pro 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 becomes clearer when traffic impact or when large horizontal loads are applied in MSEW.
Appendix A

MSE-Pro Output
<table>
<thead>
<tr>
<th>Project Name:</th>
<th>Verification Calculation for Horizontal Back-Slope</th>
<th>Designer</th>
<th>tpt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Location:</td>
<td>IDEA Submittal [H=30/B=21]</td>
<td>Checker</td>
<td>cms</td>
</tr>
<tr>
<td>Project Number:</td>
<td></td>
<td>Date:</td>
<td>06/07/16</td>
</tr>
<tr>
<td>Design Code:</td>
<td>AASHTO</td>
<td>Design Method:</td>
<td>LRFD</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Page:</td>
<td>of</td>
</tr>
</tbody>
</table>

This Page Intentionally Left Blank
MSE - Pro
Mechanically Stabilized Earth Retaining Structures

SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

Project Information
VAWS #: Grid-Strip MSE Product Submitt-IDEA
Name: Department of Transportation Method
Location: Project #:
Design Engineer: Wall Name: tpt SCP MSE Walls

Soil Reinforcing Schedule - Grid Strip

<table>
<thead>
<tr>
<th>Mat-Type</th>
<th>Long Size (W)</th>
<th>Tran Size (W)</th>
<th>Long-Space (ft)</th>
<th>Tran-Space (ft)</th>
<th>Mat Width (ft)</th>
<th>Number Of Mats/Row</th>
<th>Max Stiff.</th>
<th>Ac (in²)</th>
<th>F* - 0</th>
<th>F* - 20</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11.0</td>
<td>11.0</td>
<td>0.167</td>
<td>1.000</td>
<td>0.167</td>
<td>2</td>
<td>6160.000</td>
<td>0.319</td>
<td>3.000</td>
<td>1.250</td>
</tr>
</tbody>
</table>

Soil Parameters

<table>
<thead>
<tr>
<th>Unit Weight</th>
<th>Friction Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>MSE Backfill: 0.135 (kcf)</td>
<td>34 (deg)</td>
</tr>
<tr>
<td>Retained Backfill: 0.120 (kcf)</td>
<td>30 (deg)</td>
</tr>
<tr>
<td>Foundation Backfill: 0.120 (kcf)</td>
<td>30 (deg)</td>
</tr>
<tr>
<td>Earth Surcharge: 0.120 (kcf)</td>
<td>Depth to Water Table: 0.00 (ft)</td>
</tr>
</tbody>
</table>

Live Load Parameters

| Equiv. Height of Soil: 2.000 (ft) | Equiv. Unit Weight of Soil: 0.125 (kcf) |

Internal Stability Parameters

| Steel Yield Stress: 65.000 (ksi) | Yield Coefficient: 0.750 | Design Life: 75.0 years |

Design Options

LRFD Procedure
Stiffness 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 - AASHTO 11.10.6.2.1
The K-Ratios are calculated from the top of the structure or top of coping

General Notes

Project Specifications
2. Course Aggregate
3. MSE Structure with Horizontal Back-Slope
4. Grid-Strip Soil Reinforcing
5. F*= 3.00 to 1.25
## Project Information

**VAWS #:** Grid-Strip MSE Product Submitt-IDEA  
**Location:** Department of Transportation Method  
**Project #:**  
**Design Engineer:** tpt  
**Wall Name:** SCP MSE Walls

### Load and Resistance Factor Design Input Data

<table>
<thead>
<tr>
<th>Load Factor</th>
<th>Maximum</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load factor for vertical earth pressure (EV)</td>
<td>1.35</td>
<td>1.00</td>
</tr>
<tr>
<td>Load factor for earth surcharge (ES)</td>
<td>1.50</td>
<td>1.00</td>
</tr>
<tr>
<td>Load factor for traffic live load (LS)</td>
<td>1.75</td>
<td>1.75</td>
</tr>
<tr>
<td>Load factor for horizontal earth pressure (EH)</td>
<td>1.50</td>
<td>0.90</td>
</tr>
<tr>
<td>Load factor for seismic (EQ)</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Load factor for structural components (DC)</td>
<td>1.25</td>
<td>0.90</td>
</tr>
<tr>
<td>Load factor for water (WA)</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Load factor for live load extreme event (LL)</td>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>Load factor for vehicular impact (CT)</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Resistance Factor</th>
<th>Static</th>
<th>Seismic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing resistance factor</td>
<td>0.65</td>
<td></td>
</tr>
<tr>
<td>Sliding resistance factor</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Tensile resistance factor</td>
<td>0.75</td>
<td>1.00</td>
</tr>
<tr>
<td>Pullout resistance factor</td>
<td>0.90</td>
<td>1.20</td>
</tr>
<tr>
<td>Extreme event resistance factor</td>
<td>1.00</td>
<td></td>
</tr>
</tbody>
</table>

### Variation of Lateral Earth Pressure Coefficient with Depth (K/Ka)

<table>
<thead>
<tr>
<th>Depth (Z)</th>
<th>Rupture</th>
<th>Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00 (ft)</td>
<td>1.70</td>
<td>1.70</td>
</tr>
<tr>
<td>20.00 (ft)</td>
<td>1.20</td>
<td>1.20</td>
</tr>
</tbody>
</table>

### Variation of Friction Factor with Depth (F*)

<table>
<thead>
<tr>
<th>Depth (Z)</th>
<th>F*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00 (ft)</td>
<td>3.00</td>
</tr>
<tr>
<td>20.00 (ft)</td>
<td>1.25</td>
</tr>
</tbody>
</table>
## External And Internal Stability Calculation Summary

### Rec. #1
**Structure Parameters**

<table>
<thead>
<tr>
<th>H max (ft)</th>
<th>B min (ft)</th>
<th>B Ext. (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>( \beta ) (deg)</th>
<th>( \beta_i ) (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>30.000</td>
<td>21.000</td>
<td>21.000</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

### External Active Earth Pressure Coefficient: 0.333
**External Stability CDR Summary (Effective Stress)**

<table>
<thead>
<tr>
<th></th>
<th>CDR Sliding</th>
<th>Limiting Eccentricity (ft)</th>
<th>CDR Overturning</th>
<th>Bearing Pressure (ksf)</th>
<th>Eccentricity (ft)</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength 1 - Min</td>
<td>2.39</td>
<td>2.68</td>
<td>3.92</td>
<td>5.83</td>
<td>2.42</td>
<td>0.500</td>
</tr>
<tr>
<td>Strength 1 - Max</td>
<td>2.11</td>
<td>2.92</td>
<td>3.59</td>
<td>7.96</td>
<td>2.71</td>
<td>0.500</td>
</tr>
<tr>
<td>Strength 1 - Critical</td>
<td>1.57</td>
<td>3.95</td>
<td>2.66</td>
<td>6.79</td>
<td>3.56</td>
<td>0.500</td>
</tr>
<tr>
<td>Service</td>
<td>2.40</td>
<td>2.56</td>
<td>4.11</td>
<td>5.58</td>
<td>2.41</td>
<td>0.500</td>
</tr>
</tbody>
</table>

### Internal Active Earth Pressure Coefficient: 0.283
**Internal Stability CDR Summary**

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Internal Earth Coeff.</th>
<th>Rupture CDR</th>
<th>Pullout CDR</th>
<th>F*</th>
<th>Le (ft)</th>
<th>H/B Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.25</td>
<td>21.00</td>
<td>2</td>
<td>0.46</td>
<td>2.99</td>
<td>2.15</td>
<td>2.80</td>
<td>12.00</td>
<td>0.70</td>
</tr>
<tr>
<td>3.75</td>
<td>21.00</td>
<td>2</td>
<td>0.45</td>
<td>3.36</td>
<td>3.14</td>
<td>2.67</td>
<td>12.00</td>
<td>0.70</td>
</tr>
<tr>
<td>6.25</td>
<td>21.00</td>
<td>2</td>
<td>0.44</td>
<td>1.93</td>
<td>2.40</td>
<td>2.45</td>
<td>12.00</td>
<td>0.70</td>
</tr>
<tr>
<td>8.75</td>
<td>21.00</td>
<td>2</td>
<td>0.42</td>
<td>1.54</td>
<td>2.28</td>
<td>2.23</td>
<td>12.00</td>
<td>0.70</td>
</tr>
<tr>
<td>11.25</td>
<td>21.00</td>
<td>2</td>
<td>0.40</td>
<td>1.30</td>
<td>2.15</td>
<td>2.02</td>
<td>12.00</td>
<td>0.70</td>
</tr>
<tr>
<td>13.75</td>
<td>21.00</td>
<td>2</td>
<td>0.38</td>
<td>1.14</td>
<td>2.00</td>
<td>1.80</td>
<td>12.00</td>
<td>0.70</td>
</tr>
<tr>
<td>16.25</td>
<td>21.00</td>
<td>2</td>
<td>0.37</td>
<td>1.03</td>
<td>1.96</td>
<td>1.58</td>
<td>12.75</td>
<td>0.70</td>
</tr>
<tr>
<td>18.75</td>
<td>21.00</td>
<td>3</td>
<td>0.35</td>
<td>1.43</td>
<td>2.97</td>
<td>1.36</td>
<td>14.25</td>
<td>0.70</td>
</tr>
<tr>
<td>21.25</td>
<td>21.00</td>
<td>3</td>
<td>0.34</td>
<td>1.31</td>
<td>3.10</td>
<td>1.25</td>
<td>15.75</td>
<td>0.70</td>
</tr>
<tr>
<td>23.75</td>
<td>21.00</td>
<td>3</td>
<td>0.34</td>
<td>1.18</td>
<td>3.40</td>
<td>1.25</td>
<td>17.25</td>
<td>0.70</td>
</tr>
<tr>
<td>26.25</td>
<td>21.00</td>
<td>3</td>
<td>0.34</td>
<td>1.07</td>
<td>3.69</td>
<td>1.25</td>
<td>18.75</td>
<td>0.70</td>
</tr>
<tr>
<td>28.75</td>
<td>21.00</td>
<td>4</td>
<td>0.34</td>
<td>1.32</td>
<td>5.32</td>
<td>1.25</td>
<td>20.25</td>
<td>0.70</td>
</tr>
</tbody>
</table>
### MSE - Pro
Mechanically Stabilized Earth Retaining Structures

#### SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

### Project Information

VAWS #:  
Name: Grid-Strip MSE Product Submitt-IDEA  
Location: Department of Transportation Method  
Project #:  
Design Engineer: tpt  
Wall Name: SCP MSE Walls

### Static Internal Stability - Rupture Force Results

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Static CDR Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.25</td>
<td>21.00</td>
<td>2</td>
<td>15.00</td>
<td>0.46</td>
<td>0.75</td>
<td>0.35</td>
<td>5.21</td>
<td>2.99</td>
</tr>
<tr>
<td>3.75</td>
<td>21.00</td>
<td>2</td>
<td>10.00</td>
<td>0.45</td>
<td>1.02</td>
<td>0.46</td>
<td>4.64</td>
<td>3.36</td>
</tr>
<tr>
<td>6.25</td>
<td>21.00</td>
<td>2</td>
<td>12.50</td>
<td>0.44</td>
<td>1.48</td>
<td>0.64</td>
<td>8.06</td>
<td>1.93</td>
</tr>
<tr>
<td>8.75</td>
<td>21.00</td>
<td>2</td>
<td>12.50</td>
<td>0.42</td>
<td>1.93</td>
<td>0.81</td>
<td>10.11</td>
<td>1.54</td>
</tr>
<tr>
<td>11.25</td>
<td>21.00</td>
<td>2</td>
<td>12.50</td>
<td>0.40</td>
<td>2.39</td>
<td>0.96</td>
<td>11.97</td>
<td>1.30</td>
</tr>
<tr>
<td>13.75</td>
<td>21.00</td>
<td>2</td>
<td>12.50</td>
<td>0.38</td>
<td>2.84</td>
<td>1.09</td>
<td>13.63</td>
<td>1.14</td>
</tr>
<tr>
<td>16.25</td>
<td>21.00</td>
<td>2</td>
<td>12.50</td>
<td>0.37</td>
<td>3.30</td>
<td>1.21</td>
<td>15.08</td>
<td>1.03</td>
</tr>
<tr>
<td>18.75</td>
<td>21.00</td>
<td>3</td>
<td>12.50</td>
<td>0.35</td>
<td>3.75</td>
<td>1.31</td>
<td>16.34</td>
<td>1.43</td>
</tr>
<tr>
<td>21.25</td>
<td>21.00</td>
<td>3</td>
<td>12.50</td>
<td>0.34</td>
<td>4.21</td>
<td>1.43</td>
<td>17.85</td>
<td>1.31</td>
</tr>
<tr>
<td>23.75</td>
<td>21.00</td>
<td>3</td>
<td>12.50</td>
<td>0.34</td>
<td>4.67</td>
<td>1.58</td>
<td>19.79</td>
<td>1.18</td>
</tr>
<tr>
<td>26.25</td>
<td>21.00</td>
<td>3</td>
<td>12.50</td>
<td>0.34</td>
<td>5.12</td>
<td>1.74</td>
<td>21.72</td>
<td>1.07</td>
</tr>
<tr>
<td>28.75</td>
<td>21.00</td>
<td>4</td>
<td>12.50</td>
<td>0.34</td>
<td>5.58</td>
<td>1.89</td>
<td>23.65</td>
<td>1.32</td>
</tr>
</tbody>
</table>

### Static Internal Stability - Pullout Force Results

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horiz. Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Le (ft)</th>
<th>F*</th>
<th>Pr (Kips)</th>
<th>Static CDR Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.25</td>
<td>21.00</td>
<td>2</td>
<td>15.00</td>
<td>0.46</td>
<td>0.30</td>
<td>0.14</td>
<td>2.86</td>
<td>12.00</td>
<td>2.80</td>
<td>6.14</td>
<td>2.15</td>
</tr>
<tr>
<td>3.75</td>
<td>21.00</td>
<td>2</td>
<td>10.00</td>
<td>0.45</td>
<td>0.51</td>
<td>0.23</td>
<td>3.10</td>
<td>12.00</td>
<td>2.67</td>
<td>9.76</td>
<td>3.14</td>
</tr>
<tr>
<td>6.25</td>
<td>21.00</td>
<td>2</td>
<td>12.50</td>
<td>0.44</td>
<td>0.84</td>
<td>0.37</td>
<td>6.21</td>
<td>12.00</td>
<td>2.45</td>
<td>14.93</td>
<td>2.40</td>
</tr>
<tr>
<td>8.75</td>
<td>21.00</td>
<td>2</td>
<td>12.50</td>
<td>0.42</td>
<td>1.18</td>
<td>0.49</td>
<td>8.35</td>
<td>12.00</td>
<td>2.23</td>
<td>19.04</td>
<td>2.28</td>
</tr>
<tr>
<td>11.25</td>
<td>21.00</td>
<td>2</td>
<td>12.50</td>
<td>0.40</td>
<td>1.52</td>
<td>0.61</td>
<td>10.28</td>
<td>12.00</td>
<td>2.02</td>
<td>22.08</td>
<td>2.15</td>
</tr>
<tr>
<td>13.75</td>
<td>21.00</td>
<td>2</td>
<td>12.50</td>
<td>0.38</td>
<td>1.86</td>
<td>0.71</td>
<td>12.01</td>
<td>12.00</td>
<td>1.80</td>
<td>24.06</td>
<td>2.00</td>
</tr>
<tr>
<td>16.25</td>
<td>21.00</td>
<td>2</td>
<td>12.50</td>
<td>0.37</td>
<td>2.19</td>
<td>0.80</td>
<td>13.54</td>
<td>12.75</td>
<td>1.58</td>
<td>26.54</td>
<td>1.96</td>
</tr>
<tr>
<td>18.75</td>
<td>21.00</td>
<td>3</td>
<td>12.50</td>
<td>0.35</td>
<td>2.53</td>
<td>0.88</td>
<td>14.87</td>
<td>14.25</td>
<td>1.36</td>
<td>44.22</td>
<td>2.97</td>
</tr>
<tr>
<td>21.25</td>
<td>21.00</td>
<td>3</td>
<td>12.50</td>
<td>0.34</td>
<td>2.87</td>
<td>0.97</td>
<td>16.42</td>
<td>15.75</td>
<td>1.25</td>
<td>50.93</td>
<td>3.10</td>
</tr>
<tr>
<td>23.75</td>
<td>21.00</td>
<td>3</td>
<td>12.50</td>
<td>0.34</td>
<td>3.21</td>
<td>1.09</td>
<td>18.36</td>
<td>17.25</td>
<td>1.25</td>
<td>62.35</td>
<td>3.40</td>
</tr>
<tr>
<td>26.25</td>
<td>21.00</td>
<td>3</td>
<td>12.50</td>
<td>0.34</td>
<td>3.54</td>
<td>1.20</td>
<td>20.29</td>
<td>18.75</td>
<td>1.25</td>
<td>74.90</td>
<td>3.69</td>
</tr>
<tr>
<td>28.75</td>
<td>21.00</td>
<td>4</td>
<td>12.50</td>
<td>0.34</td>
<td>3.88</td>
<td>1.32</td>
<td>22.22</td>
<td>20.25</td>
<td>1.25</td>
<td>118.13</td>
<td>5.32</td>
</tr>
</tbody>
</table>
SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

Project Information
VAWS #:
Name: Grid-Strip MSE Product Submitt-IDEA
Location: Department of Transportation Method
Project #:
Design Engineer: tpt
Wall Name: SCP MSE Walls

Legend
H max  Height of the wall
B min  Minimum length of soil reinforcing for defined wall height
S     Maximum height of surcharge
Xs    Distance of surcharge slope over soil reinforcement
β     Slope of surcharge
β i   Adjusted angle
Le    Length of embedment of soil reinforcing in passive zone
FS    Factor of safety
e     Eccentricity
BP    Bearing pressure
F*    Pullout friction factor
s     Sliding
o     Overturning

Project File Name: I:\Toms Computer\AB My Data\DOT\AA-IDEA\Calculations\Level BackSlope with LiveLoad\MSE-Pro IDEA H30B21 Simple with LS - Grid-Strip.mse
Mon. June 5, 2017 @ 6:27 AM Page 5 of 5
<table>
<thead>
<tr>
<th>Project Name:</th>
<th>Verification Calculation for Horizontal Back-Slope</th>
<th>Designer</th>
<th>tpt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Location:</td>
<td>IDEA Submittal [H=30/B=21]</td>
<td>Checker</td>
<td>cms</td>
</tr>
<tr>
<td>Project Number:</td>
<td></td>
<td>Date:</td>
<td>06/07/16</td>
</tr>
<tr>
<td>Design Code:</td>
<td>AASHTO</td>
<td>Design Method:</td>
<td>LRFD</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Page:</td>
<td>of</td>
</tr>
</tbody>
</table>

**Appendix B**

**MSEW Output**
<table>
<thead>
<tr>
<th>Project Name:</th>
<th>Verification Calculation for Horizontal Back-Slope</th>
<th>Designer</th>
<th>tpt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Location:</td>
<td>IDEA Submittal [H=30/B=21]</td>
<td>Checker</td>
<td>cms</td>
</tr>
<tr>
<td>Project Number:</td>
<td></td>
<td>Date:</td>
<td>06/07/16</td>
</tr>
<tr>
<td>Design Code:</td>
<td>AASHTO</td>
<td>Design Method:</td>
<td>LRFD</td>
</tr>
<tr>
<td>Page:</td>
<td></td>
<td></td>
<td>of</td>
</tr>
</tbody>
</table>
AASHTO 2007-2010 (LRFD)
Design Case - Horizontal Back-Slope

PROJECT IDENTIFICATION
Title: Design Case - Horizontal Back-Slope
Project Number: Grid-Strip IDEA
Client: LRFD Binder - Level Back-Slope with LS
Designer: tpt
Station Number: 1

Description:
LRFD Verification External Stability LL=1.75 / Internal Stability LL=EV=1.35

Company's information:
Name: VAWS
Street: 650 Justice Lane
Mansfield, TX 76063
Telephone #: 817-507-0200
Fax #: 817-507-0197
E-Mail: ttaylor@bigrbridge.com

Original file path and name: I:\Toms Computer\AB My Data\DOT\AA-IDEA\Calculations\Level BackSlope with LiveLoad\Grid Strip External LRFD-IDEA Simple.BEN
Original date and time of creating this file: 06-04-2017

PROGRAM MODE:
ANALYSIS
of a SIMPLE STRUCTURE
using METAL STRIPS as reinforcing material.
SOIL DATA

REINFORCED SOIL
Unit weight, $\gamma$ 135.0 lb/ft³  
Design value of internal angle of friction, $\phi$ 34.0°

RETIRED SOIL
Unit weight, $\gamma$ 120.0 lb/ft³  
Design value of internal angle of friction, $\phi$ 30.0°

FOUNDATION SOIL (Considered as an equivalent uniform soil)
Equivalent unit weight, $\gamma_{equiv}$ 120.0 lb/ft³  
Equivalent internal angle of friction, $\phi_{equiv}$ 30.0°  
Equivalent cohesion, $c_{equiv}$ 0.0 lb/ft²

Water table is at wall base elevation

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) = 0.3333 (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):  
$N_c = 30.14$  
$N = 22.40$

SEISMICITY

Not Applicable
INPUT DATA: Metal strips
(Analysis)

<table>
<thead>
<tr>
<th>Data</th>
<th>Metal strip type #1</th>
<th>Metal strip type #2</th>
<th>Metal strip type #3</th>
<th>Metal strip type #4</th>
<th>Metal strip type #5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield strength of steel, Fy [kips/in²]</td>
<td>65.0</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Gross width of strip, b [in]</td>
<td>2.0</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Vertical spacing, Sv [ft]</td>
<td>Varies</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Design cross section area, Ac [in²]</td>
<td>0.16</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Ribbed steel strips.
Uniformity Coefficient of reinforced soil, Cu = D60/D10 = 4.0

Friction angle along reinforcement-soil interface,
\( \rho \)
- @ the top: 60.97
- @ 19.7 ft or below: 34.00

Pullout resistance factor, \( F^* \)
- @ the top: 3.00
- @ 19.7 ft or below: 1.25

Scale-effect correction factor, \( \alpha \)
- 1.00

Variation of Lateral Earth Pressure Coefficient With Depth

<table>
<thead>
<tr>
<th>Z [ft]</th>
<th>( K / K_a )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.70</td>
</tr>
<tr>
<td>3.3</td>
<td>1.60</td>
</tr>
<tr>
<td>6.6</td>
<td>1.55</td>
</tr>
<tr>
<td>9.8</td>
<td>1.45</td>
</tr>
<tr>
<td>13.1</td>
<td>1.35</td>
</tr>
<tr>
<td>16.4</td>
<td>1.30</td>
</tr>
<tr>
<td>19.7</td>
<td>1.20</td>
</tr>
</tbody>
</table>
**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 $\gamma_f = 152.78 \text{ lb/ft}^3$

<table>
<thead>
<tr>
<th>$Z / Hd$</th>
<th>To-static / $T_{max}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>1.00</td>
</tr>
<tr>
<td>0.25</td>
<td>1.00</td>
</tr>
<tr>
<td>0.50</td>
<td>1.00</td>
</tr>
<tr>
<td>0.75</td>
<td>1.00</td>
</tr>
<tr>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>D A T A (for connection only)</th>
<th>Type #1</th>
<th>Type #2</th>
<th>Type #3</th>
<th>Type #4</th>
<th>Type #5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Product Name</td>
<td>GS11</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Strength reduction at the connection, $CR_u = \frac{F_{yc}}{F_y}$</td>
<td>1.00</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>
INPUT DATA:  Geometry and Surchage loads (of a SIMPLE STRUCTURE)

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

Batter, \(\omega\) 0.0 [deg]
Backslope, \(\beta\) 0.0 [deg]
Backslope rise 0.0 [ft]  Broken back equivalent angle, \(I = 0.00^\circ\)  (see Fig. 25 in DEMO 82)

UNIFORM SURCHARGE  
Uniformly distributed dead load is 0.0 [lb/ft²], and live load is 250.0 [lb/ft²]

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: | \( \gamma_{p-EV} \) | 1.35 |
| Load factor for earthquake loads, EQ, from Table 3.4.1-1: | \( \gamma_{p-EQ} \) | 1.00 |
| Load factor for live load surcharge, LS, from Figure C11.5.5-3(b): | \( \gamma_{p-LS} \) | 1.75 |
| Load factor for dead load surcharge, ES: | \( \gamma_{p-ES} \) | 1.35 |

**Internal Stability**

Resistance factor for reinforcement tension from Table 11.5.6-1:

| Metal Strips: | \( \phi \) | Static | Combined static/seismic |
| | | | |
| | 0.75 | 1.00 |

Resistance factor for reinforcement tension in connectors from Table 11.5.6-1:

| Metal Strips: | \( \phi \) | Static | Combined static/seismic |
| | | | |
| | 0.75 | 1.00 |

Resistance factor for reinforcement pullout from Table 11.5.6-1:

| \( \phi \) | Static | Combined static/seismic |
| | 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:

| Sliding and Eccentricity | \( \gamma_{p-EV} \) | 1.00 | \( \gamma_{p-EQ} \) | 1.00 |
| Bearing Capacity | \( \gamma_{p-EV} \) | 1.35 | \( \gamma_{p-EQ} \) | 1.35 |

Load factor of active lateral earth pressure, EH, from Table 3.4.1-2 and Figure C11.5.5-2:

| \( \gamma_{p-EH} \) | 1.50 |

Load factor of active lateral earth pressure during earthquake (does not multiply \( P_{AE} \) and \( P_{R} \)):

| \( \gamma_{p-EH} \) | 1.50 |

Load factor for earthquake loads, EQ, from Table 3.4.1-1 (multiplies \( P_{AE} \) and \( P_{R} \)):

| \( \gamma_{p-EQ} \) | 1.00 |

Resistance factor for shear resistance along common interfaces from Table 11.5.6-1:

| Reinforced Soil and Foundation | \( \phi_{r} \) | 1.00 | 1.00 |
| Reinforced Soil and Reinforcement | \( \phi_{r} \) | 1.00 | 1.00 |

Resistance factor for bearing capacity of shallow foundation from Table 11.5.6-1:

| \( \phi_{b} \) | Static | Combined Static/Seismic |
| | 0.65 | 0.65 |
ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, CDR = 0.82, factored bearing load = 7956 lb/ft².

Foundation Interface: Direct sliding, CDR = 1.565, Eccentricity, e/L = 0.1879, CDR-overturning = 2.66

<table>
<thead>
<tr>
<th>Elevation [ft]</th>
<th>Length [ft]</th>
<th>Type</th>
<th>Metal strip strength CDR</th>
<th>Pullout resistance CDR</th>
<th>Direct sliding CDR</th>
<th>Eccentricity e/L</th>
<th>Product name</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.25</td>
<td>21.00</td>
<td>1</td>
<td>1.30</td>
<td>1.30</td>
<td>1.296</td>
<td>5.306</td>
<td>GS11</td>
</tr>
<tr>
<td>3.75</td>
<td>21.00</td>
<td>1</td>
<td>1.05</td>
<td>1.05</td>
<td>1.055</td>
<td>3.677</td>
<td>GS11</td>
</tr>
<tr>
<td>6.25</td>
<td>21.00</td>
<td>1</td>
<td>1.16</td>
<td>1.16</td>
<td>1.155</td>
<td>3.383</td>
<td>GS11</td>
</tr>
<tr>
<td>8.75</td>
<td>21.00</td>
<td>1</td>
<td>1.28</td>
<td>1.28</td>
<td>1.278</td>
<td>3.089</td>
<td>GS11</td>
</tr>
<tr>
<td>11.25</td>
<td>21.00</td>
<td>1</td>
<td>1.39</td>
<td>1.39</td>
<td>1.392</td>
<td>2.905</td>
<td>GS11</td>
</tr>
<tr>
<td>13.75</td>
<td>21.00</td>
<td>1</td>
<td>1.01</td>
<td>1.01</td>
<td>1.005</td>
<td>1.929</td>
<td>GS11</td>
</tr>
<tr>
<td>16.25</td>
<td>21.00</td>
<td>1</td>
<td>1.12</td>
<td>1.12</td>
<td>1.116</td>
<td>1.996</td>
<td>GS11</td>
</tr>
<tr>
<td>18.75</td>
<td>21.00</td>
<td>1</td>
<td>1.26</td>
<td>1.26</td>
<td>1.264</td>
<td>2.150</td>
<td>GS11</td>
</tr>
<tr>
<td>21.25</td>
<td>21.00</td>
<td>1</td>
<td>1.47</td>
<td>1.47</td>
<td>1.469</td>
<td>2.267</td>
<td>GS11</td>
</tr>
<tr>
<td>23.75</td>
<td>21.00</td>
<td>1</td>
<td>1.81</td>
<td>1.81</td>
<td>1.814</td>
<td>2.390</td>
<td>GS11</td>
</tr>
<tr>
<td>26.25</td>
<td>21.00</td>
<td>1</td>
<td>2.98</td>
<td>2.98</td>
<td>2.978</td>
<td>2.970</td>
<td>GS11</td>
</tr>
<tr>
<td>27.75</td>
<td>21.00</td>
<td>1</td>
<td>3.16</td>
<td>3.16</td>
<td>3.162</td>
<td>3.281</td>
<td>GS11</td>
</tr>
</tbody>
</table>
**BEARING CAPACITY for GIVEN LAYOUT**

<table>
<thead>
<tr>
<th>STATIC</th>
<th>SEISMIC</th>
<th>UNITS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factored bearing resistance, q-n</td>
<td>6531</td>
<td>N/A</td>
</tr>
<tr>
<td>Factored bearing load, σV</td>
<td>7955.7</td>
<td>N/A</td>
</tr>
<tr>
<td>Eccentricity, e</td>
<td>2.71</td>
<td>N/A</td>
</tr>
<tr>
<td>Eccentricity, e/L</td>
<td>0.129</td>
<td>N/A</td>
</tr>
<tr>
<td>CDR calculated</td>
<td>0.82</td>
<td>N/A</td>
</tr>
<tr>
<td>Base length</td>
<td>21.00</td>
<td>N/A</td>
</tr>
</tbody>
</table>

(Water table is at wall base elevation)

Unfactored applied bearing pressure = (Unfactored R) / [ L - 2 * (Unfactored e) ] =

Unfactored R = 90299.94 [lb/ft], L = 21.00, Unfactored e = 2.41 [ft], and Sigma = 5580.02 [lb/ft²]
**DIRECT SLIDING for GIVEN LAYOUT**  (for METAL STRIPS reinforcements)

Along reinforced and foundation soils interface:  CDR-static = 1.617

<table>
<thead>
<tr>
<th>#</th>
<th>Metal strip Elevation [ft]</th>
<th>Metal strip Length [ft]</th>
<th>CDR Static</th>
<th>CDR Seismic</th>
<th>Metal strip Type #</th>
<th>Product name</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.25</td>
<td>21.00</td>
<td>1.961</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>2</td>
<td>3.75</td>
<td>21.00</td>
<td>2.125</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>3</td>
<td>6.25</td>
<td>21.00</td>
<td>2.318</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>4</td>
<td>8.75</td>
<td>21.00</td>
<td>2.550</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>5</td>
<td>11.25</td>
<td>21.00</td>
<td>2.833</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>6</td>
<td>13.75</td>
<td>21.00</td>
<td>3.187</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>7</td>
<td>16.25</td>
<td>21.00</td>
<td>3.642</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>8</td>
<td>18.75</td>
<td>21.00</td>
<td>4.249</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>9</td>
<td>21.25</td>
<td>21.00</td>
<td>5.099</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>10</td>
<td>23.75</td>
<td>21.00</td>
<td>6.374</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>11</td>
<td>26.25</td>
<td>21.00</td>
<td>8.499</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>12</td>
<td>27.75</td>
<td>21.00</td>
<td>10.624</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
</tbody>
</table>

**ECCENTRICITY for GIVEN LAYOUT**  (for Simplified Method)

At interface with foundation:  e/L static = 0.1795; Overturning: CDR-static = 2.79

<table>
<thead>
<tr>
<th>#</th>
<th>Metal strip Elevation [ft]</th>
<th>Metal strip Length [ft]</th>
<th>e / L Static</th>
<th>e / L Seismic</th>
<th>Metal strip Type #</th>
<th>Product name</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.25</td>
<td>21.00</td>
<td>0.1660</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>2</td>
<td>3.75</td>
<td>21.00</td>
<td>0.1405</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>3</td>
<td>6.25</td>
<td>21.00</td>
<td>0.1172</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>4</td>
<td>8.75</td>
<td>21.00</td>
<td>0.0959</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>5</td>
<td>11.25</td>
<td>21.00</td>
<td>0.0768</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>6</td>
<td>13.75</td>
<td>21.00</td>
<td>0.0597</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>7</td>
<td>16.25</td>
<td>21.00</td>
<td>0.0447</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>8</td>
<td>18.75</td>
<td>21.00</td>
<td>0.0319</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>9</td>
<td>21.25</td>
<td>21.00</td>
<td>0.0211</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>10</td>
<td>23.75</td>
<td>21.00</td>
<td>0.0125</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>11</td>
<td>26.25</td>
<td>21.00</td>
<td>0.0059</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>12</td>
<td>27.75</td>
<td>21.00</td>
<td>0.0030</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
</tbody>
</table>
**RESULTS for STRENGTH**  
[Note: Actual CDR = (Yield stress) / (Actual stress)]  
For Simplified Method

<table>
<thead>
<tr>
<th>#</th>
<th>Metal strip Elevation [ft]</th>
<th>Coverage Ratio, Rc=b/Sh</th>
<th>Horizontal spacing, Sh [ft]</th>
<th>Long-term strength, Fy·Ac·Rc/b [lb/ft]</th>
<th>Tmax [lb/ft]</th>
<th>Tmd [lb/ft]</th>
<th>Specified minimum CDR static</th>
<th>Actual calculated CDR static</th>
<th>Specified minimum CDR seismic</th>
<th>Actual calculated CDR seismic</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.25</td>
<td>0.133</td>
<td>1.250</td>
<td>6240</td>
<td>4730.26</td>
<td>N/A</td>
<td>N/A</td>
<td>1.319</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>2</td>
<td>3.75</td>
<td>0.100</td>
<td>1.670</td>
<td>4671</td>
<td>4343.82</td>
<td>N/A</td>
<td>N/A</td>
<td>1.075</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>3</td>
<td>6.25</td>
<td>0.100</td>
<td>1.670</td>
<td>4671</td>
<td>3957.39</td>
<td>N/A</td>
<td>N/A</td>
<td>1.180</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>4</td>
<td>8.75</td>
<td>0.100</td>
<td>1.670</td>
<td>4671</td>
<td>3570.95</td>
<td>N/A</td>
<td>N/A</td>
<td>1.308</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>5</td>
<td>11.25</td>
<td>0.100</td>
<td>1.670</td>
<td>4671</td>
<td>3267.53</td>
<td>N/A</td>
<td>N/A</td>
<td>1.429</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>6</td>
<td>13.75</td>
<td>0.067</td>
<td>2.500</td>
<td>3120</td>
<td>3012.85</td>
<td>N/A</td>
<td>N/A</td>
<td>1.036</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>7</td>
<td>16.25</td>
<td>0.067</td>
<td>2.500</td>
<td>3120</td>
<td>2699.62</td>
<td>N/A</td>
<td>N/A</td>
<td>1.156</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>8</td>
<td>18.75</td>
<td>0.067</td>
<td>2.500</td>
<td>3120</td>
<td>2368.95</td>
<td>N/A</td>
<td>N/A</td>
<td>1.317</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>9</td>
<td>21.25</td>
<td>0.067</td>
<td>2.500</td>
<td>3120</td>
<td>2019.53</td>
<td>N/A</td>
<td>N/A</td>
<td>1.545</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>10</td>
<td>23.75</td>
<td>0.067</td>
<td>2.500</td>
<td>3120</td>
<td>1610.88</td>
<td>N/A</td>
<td>N/A</td>
<td>1.937</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>11</td>
<td>26.25</td>
<td>0.067</td>
<td>2.500</td>
<td>3120</td>
<td>957.74</td>
<td>N/A</td>
<td>N/A</td>
<td>3.258</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>12</td>
<td>27.75</td>
<td>0.067</td>
<td>2.500</td>
<td>3120</td>
<td>846.50</td>
<td>N/A</td>
<td>N/A</td>
<td>3.686</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

**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.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.25</td>
<td>0.133</td>
<td>4444.0</td>
<td>N/A</td>
<td>20.25</td>
<td>0.75</td>
<td>23578.6</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>2</td>
<td>3.75</td>
<td>0.100</td>
<td>4057.6</td>
<td>N/A</td>
<td>18.75</td>
<td>2.25</td>
<td>14920.4</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>3</td>
<td>6.25</td>
<td>0.100</td>
<td>3671.1</td>
<td>N/A</td>
<td>17.25</td>
<td>3.75</td>
<td>12419.4</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>4</td>
<td>8.75</td>
<td>0.100</td>
<td>3284.7</td>
<td>N/A</td>
<td>15.75</td>
<td>5.25</td>
<td>10145.9</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>5</td>
<td>11.25</td>
<td>0.100</td>
<td>2973.3</td>
<td>N/A</td>
<td>14.25</td>
<td>6.75</td>
<td>8638.3</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>6</td>
<td>13.75</td>
<td>0.067</td>
<td>2704.2</td>
<td>N/A</td>
<td>12.75</td>
<td>8.25</td>
<td>5216.7</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>7</td>
<td>16.25</td>
<td>0.067</td>
<td>2378.7</td>
<td>N/A</td>
<td>12.00</td>
<td>9.00</td>
<td>4747.9</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>8</td>
<td>18.75</td>
<td>0.067</td>
<td>2032.9</td>
<td>N/A</td>
<td>12.00</td>
<td>9.00</td>
<td>4370.3</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>9</td>
<td>21.25</td>
<td>0.067</td>
<td>1665.7</td>
<td>N/A</td>
<td>12.00</td>
<td>9.00</td>
<td>3776.9</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>10</td>
<td>23.75</td>
<td>0.067</td>
<td>1241.7</td>
<td>N/A</td>
<td>12.00</td>
<td>9.00</td>
<td>2967.6</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>11</td>
<td>26.25</td>
<td>0.067</td>
<td>654.1</td>
<td>N/A</td>
<td>12.00</td>
<td>9.00</td>
<td>1942.5</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>12</td>
<td>27.75</td>
<td>0.067</td>
<td>373.0</td>
<td>N/A</td>
<td>12.00</td>
<td>9.00</td>
<td>1223.8</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>
General Information

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Date</strong></td>
<td>tpt</td>
</tr>
<tr>
<td><strong>Designer</strong></td>
<td>tpt</td>
</tr>
<tr>
<td><strong>Title</strong></td>
<td>Design Case - Infinite Back-Slope</td>
</tr>
<tr>
<td><strong>Number</strong></td>
<td>Grid Strip - IDEA</td>
</tr>
<tr>
<td><strong>Client</strong></td>
<td>LRFD Binder - Infinite Back-Slope</td>
</tr>
</tbody>
</table>

Geometry & Surcharge

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Geometry</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Height</strong></td>
<td>27.00 ft</td>
</tr>
<tr>
<td><strong>Backslope</strong></td>
<td>26.56 deg</td>
</tr>
<tr>
<td><strong>Backslope rise</strong></td>
<td>50.00 ft</td>
</tr>
<tr>
<td><strong>Embedment</strong></td>
<td>3.00 ft</td>
</tr>
<tr>
<td><strong>Surcharge</strong></td>
<td>Uniformly Distributed</td>
</tr>
<tr>
<td><strong>Dead load</strong></td>
<td>0.00 psf</td>
</tr>
<tr>
<td><strong>Live load</strong></td>
<td>0.00 psf</td>
</tr>
</tbody>
</table>

Soil Properties

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Reinforced Soil</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Unit weight</strong></td>
<td>135 pcf</td>
</tr>
<tr>
<td><strong>Friction angle</strong></td>
<td>34 deg</td>
</tr>
<tr>
<td><strong>Cohesion</strong></td>
<td>0 psf</td>
</tr>
<tr>
<td><strong>Retained Soil</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Unit weight</strong></td>
<td>120 pcf</td>
</tr>
<tr>
<td><strong>Friction angle</strong></td>
<td>30 deg</td>
</tr>
<tr>
<td><strong>Cohesion</strong></td>
<td>0 psf</td>
</tr>
<tr>
<td><strong>Foundation Soil</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Unit weight</strong></td>
<td>120 pcf</td>
</tr>
<tr>
<td><strong>Friction angle</strong></td>
<td>30 deg</td>
</tr>
<tr>
<td><strong>Cohesion</strong></td>
<td>0 psf</td>
</tr>
</tbody>
</table>

Soil Reinforcement

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Discrete Strip</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Strength and Spacing</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Product Name</strong></td>
<td>Yield Strength (ksi)</td>
</tr>
<tr>
<td>GS11</td>
<td>65</td>
</tr>
</tbody>
</table>

650 Justice Lane
Mansfield, TX 76063

VISTAWALL SYSTEMS

650 Justice Lane
Mansfield, TX 76063
Layer | Height (ft) | Length (ft) | Horizontal Spacing (ft) | Type
--- | --- | --- | --- | ---
1 | 1.25 | 26.00 | 1.25 | 1
2 | 3.75 | 26.00 | 1.25 | 1
3 | 6.25 | 26.00 | 1.67 | 1
4 | 8.75 | 26.00 | 1.67 | 1
5 | 11.25 | 26.00 | 1.67 | 1
6 | 13.75 | 26.00 | 1.67 | 1
7 | 16.25 | 26.00 | 1.67 | 1
8 | 18.75 | 26.00 | 1.67 | 1
9 | 21.25 | 26.00 | 2.50 | 1
10 | 23.75 | 26.00 | 2.50 | 1
11 | 26.25 | 26.00 | 2.50 | 1
12 | 27.75 | 26.00 | 2.50 | 1

Pullout Parameters

<table>
<thead>
<tr>
<th>Type</th>
<th>( \rho ) (dim)</th>
<th>( F^* ) (dim)</th>
<th>( \alpha ) (dim)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top</td>
<td>19.7</td>
<td>Top</td>
<td>19.7</td>
</tr>
<tr>
<td>1</td>
<td>60.93</td>
<td>34</td>
<td>3.00</td>
</tr>
</tbody>
</table>

Variation of Lateral Earth Pressure Coefficient with Depth (K/Ka)

<table>
<thead>
<tr>
<th>Depth (Z)</th>
<th>Rupture</th>
<th>Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>1.7</td>
<td>1.7</td>
</tr>
<tr>
<td>20.00</td>
<td>1.2</td>
<td>1.2</td>
</tr>
</tbody>
</table>

Facing Panel

<table>
<thead>
<tr>
<th>Depth of Concrete (ft)</th>
<th>Unit Weight (pcf)</th>
<th>Distance to Center (ft)</th>
<th>Strength Reduction (dim)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.50</td>
<td>150</td>
<td>0.25</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Load and Resistance Factor Design Input Data

<table>
<thead>
<tr>
<th>External</th>
<th>Maximum</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Factor</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Load factor for vertical earth pressure (EV)</td>
<td>1.35</td>
<td>1.00</td>
</tr>
<tr>
<td>Load factor for earth surcharge (ES)</td>
<td>1.50</td>
<td>0.75</td>
</tr>
<tr>
<td>Load factor for traffic live load (LS)</td>
<td>1.75</td>
<td>1.75</td>
</tr>
<tr>
<td>Load factor for horizontal earth pressure (EH)</td>
<td>1.50</td>
<td>0.75</td>
</tr>
<tr>
<td>Load factor for seismic (EQ)</td>
<td>1.00</td>
<td></td>
</tr>
</tbody>
</table>
### External

<table>
<thead>
<tr>
<th>Load Factor</th>
<th>Maximum</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load factor for structural components (DC)</td>
<td>1.25</td>
<td>0.90</td>
</tr>
<tr>
<td>Load factors for water (WA)</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Load factor for live load extreme event (LL)</td>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>Load factor for vehicular impact (CT)</td>
<td>1.50</td>
<td>1.00</td>
</tr>
</tbody>
</table>

### Internal

<table>
<thead>
<tr>
<th>Load Factor</th>
<th>Maximum</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load factor for vertical earth pressure (EV)</td>
<td>1.35</td>
<td>1.00</td>
</tr>
<tr>
<td>Load factor for earth surcharge (ES)</td>
<td>1.50</td>
<td>0.75</td>
</tr>
<tr>
<td>Load factor for traffic live load (LS)</td>
<td>1.35</td>
<td>1.35</td>
</tr>
<tr>
<td>Load factor for horizontal earth pressure (EH)</td>
<td>1.50</td>
<td>0.75</td>
</tr>
<tr>
<td>Load factor for seismic (EQ)</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Load factor for structural components (DC)</td>
<td>1.25</td>
<td>0.90</td>
</tr>
<tr>
<td>Load factors for water (WA)</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Load factor for live load extreme event (LL)</td>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>Load factor for vehicular impact (CT)</td>
<td>1.50</td>
<td>1.00</td>
</tr>
</tbody>
</table>

### Resistance Factor

<table>
<thead>
<tr>
<th>Resistance Factor</th>
<th>Static</th>
<th>Seismic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing resistance factor</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Sliding resistance factor</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Tensile resistance factor</td>
<td>0.75</td>
<td>1.00</td>
</tr>
<tr>
<td>Pullout resistance factor</td>
<td>0.90</td>
<td>1.20</td>
</tr>
<tr>
<td>Extreme event resistance factor</td>
<td>1.00</td>
<td></td>
</tr>
</tbody>
</table>
2.3.2

2:1 Backslope Design Example
IDEA
Highway Innovations, Developments, Enhancements and Advancements
MSE Submittal
Infinite Back-Slope

Design Methodology
Static Analysis
Introduction .................................................................................................................................................. 1

Design Steps .................................................................................................................................................. 1

1 Establish Project Requirements ....................................................................................................... 3

2 Define Project Parameters ............................................................................................................... 5

2.1 Define Structure Parameters .............................................................................................. 6

2.2 Define External Live Load Parameters ................................................................................ 7

2.3 Define Facing Parameters ................................................................................................... 8

2.4 Define Soil Reinforcing Parameters .................................................................................... 9

2.5 Define Backfill Parameters ................................................................................................ 10

3 Evaluate Applicable Load and Resistance Factors ......................................................................... 13

3.1 Extreme Force Effect ......................................................................................................... 13

4 Define Steel Corrosion Rates ......................................................................................................... 14

5 Define Internal Stability Factors .................................................................................................... 15

6 Calculate Unfactored Loads for External Stability ......................................................................... 16

7 Evaluate External Stability ............................................................................................................. 18

7.1 Calculate Sliding Resistance at Base of MSE Structure ..................................................... 18

7.2 Calculate Limiting Eccentricity at Base of MSE Structure ................................................. 20

7.3 Calculate Bearing Resistance at Base of MSE Structure ................................................... 23

7.4 Summary of External Stability ........................................................................................... 27

8 Evaluate Internal Stability .............................................................................................................. 27

8.1 Determine Soil Reinforcing Depths and Tributary Area for Internal Stability .......... 27

8.2 Determine Variation of Kr, F*and Length of Embedment for Internal Stability .......... 29

8.3 Calculate Horizontal Stress at Elevation of Each Soil Reinforcing Element ............... 31

8.4 Calculate Maximum Tension at Elevation of Each Soil Reinforcing Element .......... 32

8.5 Determine Required Area of Steel at Elevation of Each Soil Reinforcing Element ...... 34

8.6 Calculate Factored Pullout Resistance at Elevation of Each Soil Reinforcing Element... 35

8.7 Summary of Internal Stability at Elevation of Each Soil Reinforcing Element .......... 37

8.8 MSEW Internal Stability Discussion .................................................................................. 38
Table 30  Area of Steel Required for Rupture ......................................................................................... 35
Table 31  Pullout Resistance for Grid-Strip Element ............................................................................. 37
Table 32  Summary of Internal Stability for Grid-Strip Element .......................................................... 38

Appendix

Appendix A  MSE-Pro Output
Appendix B  MSEW Output
Introduction

This example demonstrates the analysis of an MSE structure with an infinite back-slope with no live load surcharge. The MSE structure is assumed to include a segmental concrete panel face utilizing the Grid-Strip 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 Slopes 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 Kr, 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 through Table 10 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.
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 because of the geotechnical investigation, analysis and subsequent 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 for the structure analyzed in this document.

**Figure 1-1** Structure Project Requirements
Table 1 Project Requirements

<table>
<thead>
<tr>
<th>Location</th>
<th>Variable</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exposed structure height</td>
<td>$h_e$</td>
<td>27.00</td>
<td>ft</td>
</tr>
<tr>
<td>Structure embedment distance</td>
<td>$d$</td>
<td>3.00</td>
<td>ft</td>
</tr>
<tr>
<td>Soil reinforcing length aspect ratio</td>
<td>ratio</td>
<td>0.87</td>
<td>dim</td>
</tr>
<tr>
<td>Length of structure</td>
<td>$L_w$</td>
<td>1000.00</td>
<td>ft</td>
</tr>
<tr>
<td>Design life</td>
<td>Life</td>
<td>75.00</td>
<td>yrs</td>
</tr>
<tr>
<td>Top of structure slope condition</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Facing</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type of Soil Reinforcement</td>
<td>Grid-Strip</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Grade 65 steel with area equal to 0.22 in²</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Galvanized – 3.4 mil zinc coating</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Is seismic to be considered</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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 and Table 3 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) so that 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.
2.1 Define Structure Parameters

The back-slope at the top of the MSE structure is infinite with a slope angle ($\beta$) 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
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 falls 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 3). 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).

\[ q = \gamma_q \cdot h_{eq} \]  

\[ \text{Equation 2-2} \]

Table 3 External Live Load Parameters

<table>
<thead>
<tr>
<th>Location</th>
<th>Variable</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equivalent height of soil</td>
<td>h_q</td>
<td>0.00</td>
<td>ft</td>
</tr>
<tr>
<td>Unit weight of surcharge</td>
<td>\gamma_q</td>
<td>0.00</td>
<td>ft</td>
</tr>
</tbody>
</table>
Table 3 External Live Load Parameters

<table>
<thead>
<tr>
<th>Location</th>
<th>Variable</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Live load pressure</td>
<td>q</td>
<td>0.00</td>
<td>psf</td>
</tr>
<tr>
<td>Include live load in calculating pullout</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

2.3 Define Facing Parameters

For this calculation example the facing shall consist of a segmental concrete panel (SCP) with a height (H₀) equal to 5.00' and a length (L₀) equal to 5.00' (Figure 2-2 and Table 4). The thickness (t₀) of the panel is equal to a minimum 6". The vertical spacing (Sᵥ) of the soil reinforcing is a consistent distance equal to 2.50' unless stipulated otherwise.

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 (Z₁) is equal to a value of 2.25'. The distance from the leveling pad to the first soil reinforcing element (Z₁) for the bottom panel is equal to 1.25'. When required the minimum distance between any two soil reinforcing elements (Zmin) shall be greater than or equal to 0.50'. The values, Sᵥ, Z₁, Z₉, and Zmin are required to set the depth to each soil reinforcing element. The maximum horizontal spacing (Sᵥ) 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>
<thead>
<tr>
<th>Location</th>
<th>Variable</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Height</td>
<td>HP</td>
<td>5.00</td>
<td>ft</td>
</tr>
<tr>
<td>Panel Length</td>
<td>LP</td>
<td>5.00</td>
<td>ft</td>
</tr>
<tr>
<td>Vertical Spacing of Soil Reinforcing</td>
<td>SV</td>
<td>2.50</td>
<td>ft</td>
</tr>
<tr>
<td>Horizontal Spacing of Soil Reinforcing</td>
<td>SH</td>
<td>2.50</td>
<td>ft</td>
</tr>
<tr>
<td>Minimum Depth of Soil Reinforcing from Top of Structure</td>
<td>ZT</td>
<td>2.25</td>
<td>ft</td>
</tr>
<tr>
<td>Minimum soil reinforcing spacing</td>
<td>Smin</td>
<td>0.50</td>
<td>ft</td>
</tr>
<tr>
<td>Distance from leveling pad to first soil reinforcing</td>
<td>Zf</td>
<td>1.25</td>
<td>ft</td>
</tr>
</tbody>
</table>

### 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 Grid-Strip soil reinforcing will be used (Table 5 and Figure 2-2). The Grid-Strip is a discrete metal strip that consists of dual longitudinal steel bars with a single point connector. Each bar has a diameter of 0.374” (W11 bar) with an area equal to 0.11 in². Therefore, the Grid-Strip initial steel area for one element is equal to 0.22 square inches. The center-to-center spacing of the longitudinal steel bars is 2”. Therefore, the width of the Grid-Strip is 2”. Transverse W11 bars are resistance welded to the longitudinal bars at 12” centers. The Grid-Strip is fabricated from Grade-65 steel with a minimum yield strength equal to 65 ksi. Based on the use of the 5x5 SCP and coupled with the minimum horizontal spacing, the minimum number of soil reinforcing elements (N_{50}) per row shall be greater than or equal to 2.
2.5 Define Backfill Parameters

The backfill is project specific (Table 6) 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 and 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.
Table 6 Backfill Parameters

<table>
<thead>
<tr>
<th>Location</th>
<th>Variable</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction Angle of Reinforced Soil</td>
<td>(\phi_r)</td>
<td>34.00</td>
<td>deg</td>
</tr>
<tr>
<td>Unit weight of Retained Soil</td>
<td>(\gamma_f)</td>
<td>120</td>
<td>psf</td>
</tr>
<tr>
<td>Friction Angle of Retained Soil</td>
<td>(\phi_f)</td>
<td>30.00</td>
<td>deg</td>
</tr>
<tr>
<td>Unit weight of Foundation Soil</td>
<td>(\gamma_{fd})</td>
<td>120.00</td>
<td>psf</td>
</tr>
<tr>
<td>Friction Angle of Foundation Soil</td>
<td>(\phi_{fd})</td>
<td>30.00</td>
<td>deg</td>
</tr>
<tr>
<td>Unit weight of earth surcharge</td>
<td>(\gamma_{em})</td>
<td>135.00</td>
<td>pcf</td>
</tr>
</tbody>
</table>

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 \((\beta_i = \beta)\).

\[
K_{af} = \cos(\beta_i) \cdot \frac{\cos(\beta_i) - \left(1 - \cos(\phi_f)^2 - \cos(\phi_f)^2\right)}{\cos(\beta_i) + \sqrt{\cos(\beta_i)^2 - \cos(\phi_f)^2}} \\
\beta_i = \arctan\left(\frac{S}{2 \cdot H}\right) \\
K_{mi} = \tan^2\left(45^\circ - \frac{\phi_f}{2}\right) \\
K_{ui} = 1 - \sin(\phi_f)
\]

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)
### 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 7 and Table 8.

#### Table 7 Load Factors (AASHTO 2014 Tables 3.4.1-1 and 3.4.1-2)

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>EV</th>
<th>EH</th>
<th>LS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength I (Max)</td>
<td>1.35</td>
<td>1.50</td>
<td>1.75</td>
</tr>
<tr>
<td>Strength I (Min)</td>
<td>1.00</td>
<td>0.90</td>
<td>1.75</td>
</tr>
<tr>
<td>Service I</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

#### Table 8 Resistance Factors (AASHTO 2014 Table 11.5.6-1)

<table>
<thead>
<tr>
<th>Resistance Factor</th>
<th>Variable</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sliding of MSE on foundation Soil</td>
<td>$\phi_s$</td>
<td>1.00</td>
<td>dim</td>
</tr>
<tr>
<td>Bearing Resistance</td>
<td>$\phi_b$</td>
<td>0.65</td>
<td>dim</td>
</tr>
<tr>
<td>Tensile Resistance (single point connector)</td>
<td>$\phi_t$</td>
<td>0.75</td>
<td>dim</td>
</tr>
<tr>
<td>Pullout Resistance</td>
<td>$\phi_p$</td>
<td>0.90</td>
<td>dim</td>
</tr>
</tbody>
</table>

#### 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

It is well known that 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 9 and the degraded thickness is calculated using Equation 4-1.

\[
E_c = 2 \cdot \left( \text{Life} - \frac{t_{\text{galv}} - t_{\text{zinc1}}}{t_{\text{zinc2}}} + 2 \right) \cdot t_{\text{steel}} \quad \text{Equation 4-1}
\]
5 Define Internal Stability Factors

The internal stability factors for the lateral stress ratio (K_r) for metal strips and is consistent with AASHTO Article 11.10. The lateral stress ratio factor is taken from Table 11.10.6.2.1-3. The friction factor (F*) is based on pullout testing that was performed on the Grid-Strip soil reinforcing. The friction factor is equal to 3.00 at the top of the structure decreasing to 1.25 at a depth of 20 feet and below. 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.
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).

![Unfactored Load Diagram for External Stability](image)

Figure 6-1  Unfactored Load Diagram for External Stability
### Table 11  Unfactored Vertical Forces

<table>
<thead>
<tr>
<th>Vertical Force (Force/Length)</th>
<th>Equation</th>
<th>Value</th>
<th>Moment Arm (Length)</th>
<th>Equation</th>
<th>Value</th>
<th>Load Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$V_1 = \gamma_r \cdot H \cdot L$</td>
<td>105.30 k/ft</td>
<td></td>
<td>$h_{v_1} = \frac{L}{2}$</td>
<td>13.00 ft</td>
<td>EV</td>
</tr>
<tr>
<td></td>
<td>$V_3 = \gamma_s \cdot S \cdot \left[ \frac{1}{2} \cdot X_s + (L - X_s) \right]$</td>
<td>22.81 k/ft</td>
<td></td>
<td>$h_{v_3} = 0.5 \cdot L^2 - \frac{X_s^2}{6} \cdot \frac{L - 0.5 \cdot X_s}{L}$</td>
<td>17.33 ft</td>
<td>EV</td>
</tr>
<tr>
<td></td>
<td>$V_4 = \frac{1}{2} \cdot K_{af} \cdot \gamma_r \cdot (H + S)^2 \cdot \sin(\beta_i)$</td>
<td>26.63 k/ft</td>
<td></td>
<td>$h_{v_4} = L$</td>
<td>26.00 ft</td>
<td>EH</td>
</tr>
</tbody>
</table>

### Table 12  Unfactored Vertical Moments

<table>
<thead>
<tr>
<th>Moment (Force-Length/Length)</th>
<th>Equation</th>
<th>Value</th>
<th>Load Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{v_1} = V_1 \cdot h_{v_1}$</td>
<td>1368.90 k-ft/ft</td>
<td>EV</td>
<td></td>
</tr>
<tr>
<td>$M_{v_3} = V_3 \cdot h_{v_3}$</td>
<td>395.46 k-ft/ft</td>
<td>EV</td>
<td></td>
</tr>
<tr>
<td>$M_{v_4} = V_4 \cdot h_{v_4}$</td>
<td>692.26 k-ft/ft</td>
<td>EH</td>
<td></td>
</tr>
</tbody>
</table>
Table 13  Unfactored Horizontal Forces

<table>
<thead>
<tr>
<th>Horizontal Force</th>
<th>Moment Arm</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Force/Length)</td>
<td>(Length)</td>
<td>Type</td>
</tr>
<tr>
<td>Equation</td>
<td>Value</td>
<td>Equation</td>
</tr>
<tr>
<td>$F_1 = \frac{1}{2}K_a \cdot \gamma_i \cdot (H + S)^2 \cdot \cos(\beta_1)$</td>
<td>53.25 k/ft</td>
<td>$h_{r1} = \frac{H + S}{3}$</td>
</tr>
</tbody>
</table>

Table 14  Unfactored Horizontal Moments

<table>
<thead>
<tr>
<th>Moment</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Force-Length/Length)</td>
<td>Type</td>
</tr>
<tr>
<td>Equation</td>
<td>Value</td>
</tr>
<tr>
<td>$M_{r1} = F_1 \cdot h_{r1}$</td>
<td>763.26 k-ft/ft</td>
</tr>
</tbody>
</table>

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 15 through Table 17. 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 15  Evaluation of Sliding at Base of MSE Structure – Strength I Maximum

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads on the MSE structure</td>
<td>( F = \gamma_{EH}\max \cdot F_1 )</td>
<td>79.88 k/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Structure</td>
<td>( V = \gamma_{EV}\max \cdot \left( V_1 + V_2 \right) + \gamma_{EH}\max \cdot V_4 )</td>
<td>212.89 k/ft</td>
</tr>
<tr>
<td>Nominal sliding resistance at base of MSE structure</td>
<td>( V_{nm} = V \cdot \tan(\phi_{fd}) )</td>
<td>122.91 k/ft</td>
</tr>
<tr>
<td>Sliding resistance at base of MSE structure</td>
<td>( V_{fm} = \phi_s \cdot V_{nm} )</td>
<td>122.91 k/ft</td>
</tr>
<tr>
<td>Is ( V_{fm} &gt; F_m )</td>
<td></td>
<td>Yes</td>
</tr>
<tr>
<td>Capacity Demand Ratio (CDR) for sliding</td>
<td>( CDR_s = \frac{V_{fm}}{F} )</td>
<td>1.54</td>
</tr>
</tbody>
</table>

### Table 16  Evaluation of Sliding at Base of MSE Structure – Strength I Minimum

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads on the MSE structure</td>
<td>( F = \gamma_{EH}\min \cdot F_1 )</td>
<td>47.93 k/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Structure</td>
<td>( V = \gamma_{EV}\min \cdot \left( V_1 + V_2 \right) + \gamma_{EH}\min \cdot V_4 )</td>
<td>152.08 k/ft</td>
</tr>
<tr>
<td>Nominal sliding resistance at base of MSE structure</td>
<td>( V_{nm} = V \cdot \tan(\phi_{fd}) )</td>
<td>87.80 k/ft</td>
</tr>
<tr>
<td>Sliding resistance at base of MSE structure</td>
<td>( V_{fm} = \phi_s \cdot V_{nm} )</td>
<td>87.80 k/ft</td>
</tr>
<tr>
<td>Is ( V_{fm} &gt; F_m )</td>
<td></td>
<td>Yes</td>
</tr>
<tr>
<td>Capacity Demand Ratio (CDR)</td>
<td>( CDR_s = \frac{V_{fm}}{F} )</td>
<td>1.83</td>
</tr>
</tbody>
</table>

### Table 17  Evaluation of Sliding at Base of MSE Structure – Strength I Critical Value Check

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads on the MSE structure</td>
<td>( F = \gamma_{EH}\max \cdot F_1 )</td>
<td>79.88 k/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Structure</td>
<td>( V = \gamma_{EV}\min \cdot \left( V_1 + V_2 \right) + \gamma_{EH}\max \cdot V_4 )</td>
<td>168.05 k/ft</td>
</tr>
<tr>
<td>Nominal sliding resistance at base of MSE structure</td>
<td>( V_{nm} = V \cdot \tan(\phi_{fd}) )</td>
<td>97.03 k/ft</td>
</tr>
</tbody>
</table>

---

Project Name: Verification Calculation for Infinite Back-Slope  
Designer: tpt  
Checker: cda  
Date: 06/07/17
### Table 17 Evaluation of Sliding at Base of MSE Structure – Strength I Critical Value Check

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sliding resistance at base of MSE</td>
<td>$V_{fm} = \phi \cdot V_{nm}$</td>
<td>97.03 k/ft</td>
</tr>
<tr>
<td>structure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Is $V_{fm} &gt; F_{m}$</td>
<td></td>
<td>Yes</td>
</tr>
<tr>
<td>Capacity Demand Ratio (CDR)</td>
<td>$\text{CDR}<em>s = \frac{V</em>{fm}}{F_m}$</td>
<td>1.21</td>
</tr>
</tbody>
</table>

The critical value check results in the smallest CDR for this example and therefore governs the sliding mode of failure (Table 17).

#### 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 18 through Table 20. 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 18 Evaluation of Limiting Eccentricity for MSE Structure – Strength I Maximum

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads on the MSE structure</td>
<td>$F = \gamma_{Ehmax} \cdot F_1$</td>
<td>79.88 k/ft</td>
</tr>
<tr>
<td>Overturning Moment about point A</td>
<td>$M_{OA} = \gamma_{Ehmax} \cdot M_{F_1}$</td>
<td>1144.90 (k-ft)/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Structure</td>
<td>$V_A = \gamma_{Evmax} \cdot (V_1 + V_3) + \gamma_{Ehmax} \cdot V_4$</td>
<td>212.89 k/ft</td>
</tr>
</tbody>
</table>
### Table 18  Evaluation of Limiting Eccentricity for MSE Structure – Strength I Maximum

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resisting moment about point A</td>
<td>$M_{RA} = \gamma_{EV_{max}} \cdot (M_{V1} + M_{V3}) + \gamma_{EH_{max}} \cdot M_{V4}$</td>
<td>3420.28 (k-ft)/ft</td>
</tr>
<tr>
<td>Net moment about Point A</td>
<td>$M_A = M_{RA} - M_{OA}$</td>
<td>2275.38 (k-ft)/ft</td>
</tr>
<tr>
<td>Location of resultant force on base of structure from point A</td>
<td>$a = \frac{M_A}{V_A}$</td>
<td>10.69 ft</td>
</tr>
<tr>
<td>Eccentricity at base of MSE structure</td>
<td>$e_L = \frac{L}{2} - a$</td>
<td>2.31 ft</td>
</tr>
<tr>
<td>Limiting eccentricity</td>
<td>$e_{limit} = \frac{L}{3}$</td>
<td>8.67 ft</td>
</tr>
<tr>
<td>Is the resultant within limiting value of e</td>
<td></td>
<td>Yes</td>
</tr>
<tr>
<td>Calculated e/L</td>
<td>$\frac{e_L}{L}$</td>
<td>0.09 dim</td>
</tr>
<tr>
<td>Capacity Demand Ratio (CDR) for Overturning</td>
<td>$CDR_O = \frac{M_{RA}}{M_{OA}}$</td>
<td>2.99</td>
</tr>
</tbody>
</table>

### Table 19  Evaluation of Limiting Eccentricity for MSE Structure – Strength I Minimum

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads n the MSE structure</td>
<td>$F = \gamma_{EH_{min}} \cdot F_1$</td>
<td>47.93 k/ft</td>
</tr>
<tr>
<td>Overturning Moment about point A</td>
<td>$M_{OA} = \gamma_{EH_{min}} \cdot F_1$</td>
<td>686.94 (k-ft)/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Structure</td>
<td>$V_A = \gamma_{EV_{min}} \cdot (V_1 + V_3) + \gamma_{EH_{min}} \cdot V_4$</td>
<td>152.08 k/ft</td>
</tr>
<tr>
<td>Resisting moment about point A</td>
<td>$M_{RA} = \gamma_{EV_{min}} \cdot (M_{V1} + M_{V3}) + \gamma_{EH_{min}} \cdot M_{V4}$</td>
<td>2387.39 (k-ft)/ft</td>
</tr>
<tr>
<td>Net moment about Point A</td>
<td>$M_A = M_{RA} - M_{OA}$</td>
<td>1700.46 (k-ft)/ft</td>
</tr>
<tr>
<td>Location of resultant force on base of structure from point A</td>
<td>$a = \frac{M_A}{V_A}$</td>
<td>11.18 ft</td>
</tr>
<tr>
<td>Eccentricity at base of MSE structure</td>
<td>$e_L = \frac{L}{2} - a$</td>
<td>1.82 ft</td>
</tr>
</tbody>
</table>
### Table 19  Evaluation of Limiting Eccentricity for MSE Structure – Strength I Minimum

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limiting eccentricity</td>
<td>$e_{\text{limit}} = \frac{L}{3}$</td>
<td>8.67 ft</td>
</tr>
<tr>
<td>Is the resultant within limiting value of e</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>Calculated $e/L$</td>
<td>$\frac{e}{L}$</td>
<td>0.07 dim</td>
</tr>
<tr>
<td>Capacity Demand Ratio (CDR) for Overturning</td>
<td>$\text{CDR}<em>D = \frac{M</em>{\text{RA}}}{M_{\text{OA}}}$</td>
<td>3.48</td>
</tr>
</tbody>
</table>

### Table 20  Evaluation of Limiting Eccentricity for MSE Structure – Strength I Critical Value Check

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads on the MSE structure</td>
<td>$F = \gamma \cdot E_{\text{Hmax}} \cdot F_1$</td>
<td>79.88 k/ft</td>
</tr>
<tr>
<td>Overturning Moment about point A</td>
<td>$M_{\text{OA}} = \gamma \cdot E_{\text{Hmax}} \cdot M_{F1}$</td>
<td>1144.90 (k-ft)/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Structure without LS</td>
<td>$V_A = \gamma \cdot E_{\text{Vmin}} \cdot (V_1 + V_3) + \gamma \cdot E_{\text{Hmin}} \cdot V_4$</td>
<td>168.05 k/ft</td>
</tr>
<tr>
<td>Resisting moment about point A without LS</td>
<td>$M_{\text{RA}} = \gamma \cdot E_{\text{Vmin}} \cdot (V_1 + V_3) + \gamma \cdot E_{\text{Hmin}} \cdot M_{V4}$</td>
<td>2802.75 (k-ft)/ft</td>
</tr>
<tr>
<td>Net moment about Point A</td>
<td>$M_A = M_{\text{RA}} - M_{\text{OA}}$</td>
<td>1657.86 (k-ft)/ft</td>
</tr>
<tr>
<td>Location of resultant force on base of structure from point A</td>
<td>$a = \frac{M_A}{V_A}$</td>
<td>9.87 ft</td>
</tr>
<tr>
<td>Eccentricity at base of MSE structure</td>
<td>$e_L = \frac{L}{2} - a$</td>
<td>3.13 ft</td>
</tr>
<tr>
<td>Limiting eccentricity</td>
<td>$e_{\text{limit}} = \frac{L}{3}$</td>
<td>8.67 ft</td>
</tr>
<tr>
<td>Is the resultant within limiting value of e</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>Effective width at base of MSE structure</td>
<td>$B' = L - 2 \cdot e_L$</td>
<td>19.73 ft</td>
</tr>
</tbody>
</table>

650 Justice Lane  
Mansfield, Texas 76063  

VISTAWALL  
Systems
The critical value check results in the largest eccentricity for this example and therefore governs the limiting eccentricity (Table 20).

### 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.

\[
\sigma_v = \frac{\sum V}{L - 2 \cdot e_L}
\]

Equation 7-1

Where the value \( \sum V = R = V_1 + V_4 + V_s \) 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 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 21 through Table 24. The Service I load combination is evaluated to compute the vertical stress for any settlement analysis and the results are presented in Table 24.

### Table 21 Evaluation of Bearing Resistance for MSE Structure – Strength I Maximum

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads on the MSE structure</td>
<td>( F = \gamma_{Eh\text{max}} \cdot F_1 )</td>
<td>79.88 k/ft</td>
</tr>
</tbody>
</table>
### Table 21  Evaluation of Bearing Resistance for MSE Structure – Strength I Maximum

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overturning Moment about point A</td>
<td>$M_{OA} = \gamma \cdot \frac{F_1}{E_{Hmax}}$</td>
<td>1144.90 (k-ft)/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Structure</td>
<td>$V_A = \gamma \cdot \frac{V_1 + V_{E_{Vmax}} + V_{E_{Hmax}}}{E_{Hmax}}$</td>
<td>212.89 k/ft</td>
</tr>
<tr>
<td>Resisting moment about point A</td>
<td>$M_{RA} = \gamma \cdot \frac{M_1 + M_{E_{Vmax}} + M_{E_{Hmax}}}{E_{Hmax}}$</td>
<td>3420.28 (k-ft)/ft</td>
</tr>
<tr>
<td>Net moment about Point A</td>
<td>$M_A = M_{RA} - M_{OA}$</td>
<td>2275.38 (k-ft)/ft</td>
</tr>
<tr>
<td>Location of resultant force on base of structure from point A</td>
<td>$a = \frac{M_A}{V_A}$</td>
<td>10.69 ft</td>
</tr>
<tr>
<td>Eccentricity at base of MSE structure from center</td>
<td>$e_L = \frac{L}{2} - a$</td>
<td>2.31 ft</td>
</tr>
<tr>
<td>Limiting eccentricity</td>
<td>$e_{limit} = \frac{L}{3}$</td>
<td>8.67 ft</td>
</tr>
<tr>
<td>Is the resultant within limiting value of e</td>
<td></td>
<td>Yes</td>
</tr>
<tr>
<td>Effective width at base of MSE structure</td>
<td>$B' = L - 2 \cdot e_L$</td>
<td>21.38 ft</td>
</tr>
<tr>
<td>Bearing stress due to MSE Structure</td>
<td>$\sigma_v = \frac{V_A}{L - 2 \cdot e_L}$</td>
<td>9.96 ksf</td>
</tr>
</tbody>
</table>

### Table 22  Evaluation of Bearing Resistance for MSE Structure – Strength I Minimum

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads n the MSE structure</td>
<td>$F = \gamma \cdot \frac{F_1}{E_{Hmin}}$</td>
<td>47.93 k/ft</td>
</tr>
<tr>
<td>Overturning Moment about point A</td>
<td>$M_{OA} = \gamma \cdot \frac{F_1}{E_{Hmin}}$</td>
<td>686.94 (k-ft)/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Structure</td>
<td>$V_A = \gamma \cdot \frac{V_1 + V_{E_{Vmin}} + V_{E_{Hmin}}}{E_{Hmin}}$</td>
<td>152.08 k/ft</td>
</tr>
<tr>
<td>Resisting moment about point A</td>
<td>$M_{RA} = \gamma \cdot \frac{M_1 + M_{E_{Vmin}} + M_{E_{Hmin}}}{E_{Hmin}}$</td>
<td>2387.39 (k-ft)/ft</td>
</tr>
</tbody>
</table>
Table 22 Evaluation of Bearing Resistance for MSE Structure – Strength I Minimum

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Net moment about Point A</td>
<td>$M_A = M_{RA} - M_{OA}$</td>
<td>1700.46 (k-ft)/ft</td>
</tr>
<tr>
<td>Location of resultant force on base of structure from point A</td>
<td>$a = \frac{M_A}{V_A}$</td>
<td>11.18 ft</td>
</tr>
<tr>
<td>Eccentricity at base of MSE structure from center</td>
<td>$e_L = \frac{L}{2} - a$</td>
<td>1.82 ft</td>
</tr>
<tr>
<td>Limiting eccentricity</td>
<td>$e_{limit} = \frac{L}{3}$</td>
<td>8.67 ft</td>
</tr>
<tr>
<td>Is the resultant within limiting value of $e$</td>
<td></td>
<td>Yes</td>
</tr>
<tr>
<td>Effective width at base of MSE structure</td>
<td>$B' = L - 2 \cdot e_L$</td>
<td>22.36 ft</td>
</tr>
<tr>
<td>Bearing stress due to MSE Structure</td>
<td>$\sigma_v = \frac{V_A}{L - 2 \cdot e_L}$</td>
<td>6.80 ksf</td>
</tr>
</tbody>
</table>

Table 23 Evaluation of Bearing Resistance for MSE Structure – Strength I Critical Value Check

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads in the MSE structure</td>
<td>$H_m = \gamma_E \cdot H_{max} \cdot F_1$</td>
<td>79.88 k/ft</td>
</tr>
<tr>
<td>Overturning Moment about point A</td>
<td>$M_{OA} = \gamma_E \cdot M_{F1}$</td>
<td>1144.90 (k-ft)/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Structure without LS</td>
<td>$V_A = \gamma_{EVmin} \cdot V_1 + \gamma_{EVmin} \cdot V_3 + \gamma_{VHmax} \cdot V_4$</td>
<td>168.05 k/ft</td>
</tr>
<tr>
<td>Resisting moment about point A without LS</td>
<td>$M_{RA} = \gamma_{EVmin} \cdot M_{V1} + \gamma_{EVmin} \cdot M_{V3} + \gamma_{EVmax} \cdot M_{V4}$</td>
<td>2802.75 (k-ft)/ft</td>
</tr>
<tr>
<td>Net moment about Point A</td>
<td>$M_A = M_{RA} - M_{OA}$</td>
<td>1657.86 (k-ft)/ft</td>
</tr>
<tr>
<td>Location of resultant force on base of structure from point A</td>
<td>$a = \frac{M_A}{V_A}$</td>
<td>9.87 ft</td>
</tr>
<tr>
<td>Eccentricity at base of MSE structure from center</td>
<td>$e_L = \frac{L}{2} - a$</td>
<td>3.13 ft</td>
</tr>
</tbody>
</table>
### Table 23  Evaluation of Bearing Resistance for MSE Structure – Strength I Critical Value Check

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limiting eccentricity</td>
<td>$e_{\text{limit}} = \frac{L}{3}$</td>
<td>8.67 ft</td>
</tr>
<tr>
<td>Is the resultant within limiting value of $e$</td>
<td></td>
<td>Yes</td>
</tr>
<tr>
<td>Effective width at base of MSE structure</td>
<td>$B' = L - 2 \cdot e_L$</td>
<td>19.73 ft</td>
</tr>
<tr>
<td>Bearing stress due to MSE Structure</td>
<td>$\sigma_v = \frac{V_A}{L - 2 \cdot e_L}$</td>
<td>8.52 ksf</td>
</tr>
</tbody>
</table>

### Table 24  Evaluation of Bearing Resistance for MSE Structure – Strength I Service

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Loads on the MSE structure</td>
<td>$F = F_1$</td>
<td>53.25 k/ft</td>
</tr>
<tr>
<td>Overturning Moment about point A</td>
<td>$M_{OA} = F_1 \cdot h_{F1}$</td>
<td>763.26 (k-ft)/ft</td>
</tr>
<tr>
<td>Vertical Load at Base of Structure</td>
<td>$V_A = V_1 + V_3 + V_4$</td>
<td>154.74 k/ft</td>
</tr>
<tr>
<td>Resisting moment about point A</td>
<td>$M_{RA} = M_{V1} + M_{V3} + M_{V4}$</td>
<td>2456.62 (k-ft)/ft</td>
</tr>
<tr>
<td>Net moment about Point A</td>
<td>$M_A = M_{RA} - M_{OA}$</td>
<td>1693.36 (k-ft)/ft</td>
</tr>
<tr>
<td>Location of resultant force on base of structure from point A</td>
<td>$a = \frac{M_A}{V_A}$</td>
<td>10.94 ft</td>
</tr>
<tr>
<td>Eccentricity at base of MSE structure from center</td>
<td>$e_L = \frac{L}{2} - a$</td>
<td>2.06 ft</td>
</tr>
<tr>
<td>Limiting eccentricity</td>
<td>$e_{\text{limit}} = \frac{L}{6}$</td>
<td>4.33 ft</td>
</tr>
<tr>
<td>Is the resultant within limiting value of $e$</td>
<td></td>
<td>Yes</td>
</tr>
<tr>
<td>Effective width at base of MSE structure</td>
<td>$B' = L - 2 \cdot e_L$</td>
<td>21.89 ft</td>
</tr>
</tbody>
</table>
Table 24 Evaluation of Bearing Resistance for MSE Structure – Strength I Service

<table>
<thead>
<tr>
<th>Item</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing stress due to MSE</td>
<td>$\sigma_v = \frac{V_A}{L - 2 \cdot e_L}$</td>
<td>7.07 ksf</td>
</tr>
</tbody>
</table>

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 21).

7.4 Summary of External Stability

Table 25 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>
<thead>
<tr>
<th>Load Combination</th>
<th>CDR Sliding (dim)</th>
<th>Limiting Eccentricity/Maximum (ft)</th>
<th>CDR Overturning (dim)</th>
<th>Bearing Stress (ksf)</th>
<th>Effective Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength I Maximum</td>
<td>1.54</td>
<td>2.31/8.67</td>
<td>2.99</td>
<td>9.96</td>
<td>21.38</td>
</tr>
<tr>
<td>Strength I Minimum</td>
<td>1.83</td>
<td>1.82/8.67</td>
<td>3.48</td>
<td>6.80</td>
<td>22.36</td>
</tr>
<tr>
<td>Strength I Critical Value Check</td>
<td>1.21</td>
<td>3.13/8.67</td>
<td>2.45</td>
<td>8.52</td>
<td>19.73</td>
</tr>
<tr>
<td>Service I</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>7.07</td>
<td>21.89</td>
</tr>
</tbody>
</table>

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.

**Figure 8-1  Soil Reinforcing Depth Parameters**

**Table 26  Soil Reinforcing Layout**

<table>
<thead>
<tr>
<th>Soil Element</th>
<th>Distance Above Leveling Pad (ft)</th>
<th>Depth Below Top of Structure (Z₁ (ft))</th>
<th>Depth Below Intersection Of Failure Surface (dm (ft))</th>
<th>Vertical Spacing (Sᵥ (ft))</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>27.75</td>
<td>2.25</td>
<td>7.54</td>
<td>3.00</td>
</tr>
<tr>
<td>11</td>
<td>26.25</td>
<td>3.75</td>
<td>9.04</td>
<td>2.00</td>
</tr>
<tr>
<td>10</td>
<td>23.75</td>
<td>6.25</td>
<td>11.54</td>
<td>2.50</td>
</tr>
<tr>
<td>9</td>
<td>21.25</td>
<td>8.75</td>
<td>14.04</td>
<td>2.50</td>
</tr>
<tr>
<td>8</td>
<td>18.75</td>
<td>11.25</td>
<td>16.54</td>
<td>2.50</td>
</tr>
<tr>
<td>7</td>
<td>16.25</td>
<td>13.75</td>
<td>19.04</td>
<td>2.50</td>
</tr>
<tr>
<td>6</td>
<td>13.75</td>
<td>16.25</td>
<td>21.54</td>
<td>2.50</td>
</tr>
<tr>
<td>5</td>
<td>11.25</td>
<td>18.75</td>
<td>24.04</td>
<td>2.50</td>
</tr>
<tr>
<td>4</td>
<td>8.75</td>
<td>21.25</td>
<td>26.54</td>
<td>2.50</td>
</tr>
</tbody>
</table>
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.

$$H_m = \text{If } \left\{ X > 0.3 \cdot (H + S), H + \frac{\tan(\beta_s) \cdot 0.3 \cdot H}{1 - 0.3 \cdot \tan(\beta_s)} \right\}$$  

Equation 8-1

For a horizontal back-slope with no earth surcharge the mechanical height is equal to the design height of the structure.
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 Grid-Strip soil reinforcing.

\[
K_i = \left[1.7 - \frac{1.7 - 1.2}{20 \text{ (ft)}} \cdot z_i \right] \cdot K_{ai} \rightarrow z_i < 20 \text{ (ft)} \tag{Equation 8-2}
\]

\[
K_i = 1.2 \cdot K_a \rightarrow z_i \geq 20 \text{ (ft)} \tag{Equation 8-3}
\]

\[
F^* = 3.00 - \frac{3.00 - 1.25}{20 \text{ (ft)}} \cdot z_i \rightarrow z_i < 20 \text{ (ft)} \tag{Equation 8-4}
\]

\[
F^* = 1.25 \rightarrow z_i \geq 20 \text{ (ft)} \tag{Equation 8-5}
\]
### Table 27  Internal Stability Factors

<table>
<thead>
<tr>
<th>Soil Element</th>
<th>Depth Below Top of Structure (Z_i) ft</th>
<th>Earth Pressure Ratio (K_i) dim</th>
<th>Pullout Friction Factor (F^*) (\text{dim})</th>
<th>Length of Embedment (L_e) ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>2.25</td>
<td>0.465</td>
<td>2.803</td>
<td>15.41</td>
</tr>
<tr>
<td>11</td>
<td>3.75</td>
<td>0.454</td>
<td>2.672</td>
<td>15.41</td>
</tr>
<tr>
<td>10</td>
<td>6.25</td>
<td>0.436</td>
<td>2.453</td>
<td>15.41</td>
</tr>
<tr>
<td>9</td>
<td>8.75</td>
<td>0.419</td>
<td>2.234</td>
<td>15.41</td>
</tr>
<tr>
<td>8</td>
<td>11.25</td>
<td>0.401</td>
<td>2.016</td>
<td>15.41</td>
</tr>
<tr>
<td>7</td>
<td>13.75</td>
<td>0.383</td>
<td>1.797</td>
<td>16.25</td>
</tr>
<tr>
<td>6</td>
<td>16.25</td>
<td>0.366</td>
<td>1.578</td>
<td>17.75</td>
</tr>
<tr>
<td>5</td>
<td>18.75</td>
<td>0.348</td>
<td>1.359</td>
<td>19.25</td>
</tr>
<tr>
<td>4</td>
<td>21.25</td>
<td>0.339</td>
<td>1.250</td>
<td>20.75</td>
</tr>
<tr>
<td>3</td>
<td>23.75</td>
<td>0.339</td>
<td>1.250</td>
<td>22.25</td>
</tr>
<tr>
<td>2</td>
<td>26.25</td>
<td>0.339</td>
<td>1.250</td>
<td>23.75</td>
</tr>
<tr>
<td>1</td>
<td>28.75</td>
<td>0.339</td>
<td>1.250</td>
<td>25.25</td>
</tr>
</tbody>
</table>

#### 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, \(\sigma_y\), 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, \(\Delta\sigma_y\), 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.

\[
\sigma_y = \frac{1}{2} \cdot \gamma \cdot (0.7 \cdot H) \cdot \tan(\beta)
\]

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.
\[ \sigma_v = \gamma \cdot \sigma + \sigma_2 \]
\[ \sigma_h = K_i \cdot \sigma_v \]

**Equation 8-7**

**Equation 8-8**

<table>
<thead>
<tr>
<th>Soil Element</th>
<th>Depth Below Top of Structure ( z_i ) ft</th>
<th>Vertical Stress ( \sigma_v ) ksf</th>
<th>Earth Pressure Coefficient ( K_i ) dim</th>
<th>Horizontal Stress ( \sigma_h ) ksf</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>2.25</td>
<td>1.37</td>
<td>0.465</td>
<td>0.64</td>
</tr>
<tr>
<td>11</td>
<td>3.75</td>
<td>1.64</td>
<td>0.454</td>
<td>0.74</td>
</tr>
<tr>
<td>10</td>
<td>6.25</td>
<td>2.10</td>
<td>0.436</td>
<td>0.91</td>
</tr>
<tr>
<td>9</td>
<td>8.75</td>
<td>2.55</td>
<td>0.419</td>
<td>1.07</td>
</tr>
<tr>
<td>8</td>
<td>11.25</td>
<td>3.01</td>
<td>0.401</td>
<td>1.21</td>
</tr>
<tr>
<td>7</td>
<td>13.75</td>
<td>3.46</td>
<td>0.383</td>
<td>1.33</td>
</tr>
<tr>
<td>6</td>
<td>16.25</td>
<td>3.92</td>
<td>0.366</td>
<td>1.43</td>
</tr>
<tr>
<td>5</td>
<td>18.75</td>
<td>4.37</td>
<td>0.348</td>
<td>1.52</td>
</tr>
<tr>
<td>4</td>
<td>21.25</td>
<td>4.83</td>
<td>0.339</td>
<td>1.64</td>
</tr>
<tr>
<td>3</td>
<td>23.75</td>
<td>5.29</td>
<td>0.339</td>
<td>1.79</td>
</tr>
<tr>
<td>2</td>
<td>26.25</td>
<td>5.74</td>
<td>0.339</td>
<td>1.95</td>
</tr>
<tr>
<td>1</td>
<td>28.75</td>
<td>6.20</td>
<td>0.339</td>
<td>2.10</td>
</tr>
</tbody>
</table>

**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 \( S_T \) for the soil reinforcing elevation that is 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 29 list the tributary area that each soil reinforcing element is to resist and the corresponding tensile force. Please reference Section 8.8 for further discussion.
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).

\[ T_{\text{max}} = \sigma_H \cdot A_T \]  

**Table 29  Maximum Tension At Soil Reinforcing**

<table>
<thead>
<tr>
<th>Soil Element</th>
<th>Depth Below Top of Structure ( Z_i ) ft</th>
<th>Horizontal Stress ( \sigma_H ) ksf</th>
<th>Tributary Area ( A_T ) sf</th>
<th>Maximum Tension ( T_{\text{max}} ) kip</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>2.25</td>
<td>0.6400</td>
<td>15.00</td>
<td>9.600</td>
</tr>
<tr>
<td>11</td>
<td>3.75</td>
<td>0.7400</td>
<td>10.00</td>
<td>7.400</td>
</tr>
<tr>
<td>10</td>
<td>6.25</td>
<td>0.9100</td>
<td>12.50</td>
<td>11.375</td>
</tr>
<tr>
<td>9</td>
<td>8.75</td>
<td>1.0700</td>
<td>12.50</td>
<td>13.375</td>
</tr>
<tr>
<td>8</td>
<td>11.25</td>
<td>1.2100</td>
<td>12.50</td>
<td>15.125</td>
</tr>
</tbody>
</table>
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 service life can be determined (Equation 8-12). Once the required end of service life steel area is determined the required steel area that includes the effects of degradation can be calculated (Equation 8-13). Based on the required steel area the number of soil reinforcing elements for each row can be determined (Equation 8-15).

\[
A_c = \frac{T_{max}}{\phi \cdot F_y}
\]

\[
A_{req} = \frac{\pi}{4} \left( \frac{4 \cdot A_c}{\pi} + E_c \right)^2
\]

\[
A_{GS} = 2 \left( \frac{\pi}{4} \cdot d_b^2 \right)
\]

\[
N_{SR} = \text{int} \left[ \frac{A_{req}}{A_{GS}} \right] + 1
\]

*The value \(N_{SR}\) is the integer value of the quotient plus 1.

Where:
- \(A_c\) = Degraded area of steel required
- \(A_{req}\) = Area of steel required at beginning of design (in²)
- \(d_b\) = Degraded diameter of Grid-Strip bar (in)
The tensile capacity demand ratio is the ratio of the supplied steel area to the required steel area and is calculated using Equation 8-16 and is required to be greater than or equal to 1.0.

\[
\text{CDR}_{\text{rup}} = \frac{N_{\text{SR}} \cdot A_{\text{GS}}}{A_{\text{req}}}
\]

Equation 8-16

<table>
<thead>
<tr>
<th>Soil Element</th>
<th>Depth Below Top of Structure (Zi) ft</th>
<th>Maximum Tension (Tmax) kip</th>
<th>Area of Steel Required at end of Design Life (Areq) in²</th>
<th>Area of Steel Provided at end of Design Life (Ae) in²</th>
<th>Number Soil Reinforcing Elements (N_{SR})</th>
<th>CDR Rupture (dim)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>2.25</td>
<td>9.600</td>
<td>0.20</td>
<td>0.32</td>
<td>2</td>
<td>1.62</td>
</tr>
<tr>
<td>11</td>
<td>3.75</td>
<td>7.400</td>
<td>0.15</td>
<td>0.32</td>
<td>2</td>
<td>2.10</td>
</tr>
<tr>
<td>10</td>
<td>6.25</td>
<td>11.375</td>
<td>0.23</td>
<td>0.32</td>
<td>2</td>
<td>1.37</td>
</tr>
<tr>
<td>9</td>
<td>8.75</td>
<td>13.375</td>
<td>0.27</td>
<td>0.32</td>
<td>2</td>
<td>1.16</td>
</tr>
<tr>
<td>8</td>
<td>11.25</td>
<td>15.125</td>
<td>0.31</td>
<td>0.32</td>
<td>2</td>
<td>1.03</td>
</tr>
<tr>
<td>7</td>
<td>13.75</td>
<td>16.625</td>
<td>0.34</td>
<td>0.48</td>
<td>3</td>
<td>1.40</td>
</tr>
<tr>
<td>6</td>
<td>16.25</td>
<td>17.875</td>
<td>0.37</td>
<td>0.48</td>
<td>3</td>
<td>1.31</td>
</tr>
<tr>
<td>5</td>
<td>18.75</td>
<td>19.000</td>
<td>0.39</td>
<td>0.48</td>
<td>3</td>
<td>1.23</td>
</tr>
<tr>
<td>4</td>
<td>21.25</td>
<td>20.500</td>
<td>0.42</td>
<td>0.48</td>
<td>3</td>
<td>1.14</td>
</tr>
<tr>
<td>3</td>
<td>23.75</td>
<td>22.375</td>
<td>0.46</td>
<td>0.48</td>
<td>3</td>
<td>1.04</td>
</tr>
<tr>
<td>2</td>
<td>26.25</td>
<td>24.375</td>
<td>0.50</td>
<td>0.64</td>
<td>4</td>
<td>1.28</td>
</tr>
<tr>
<td>1</td>
<td>28.75</td>
<td>26.250</td>
<td>0.54</td>
<td>0.64</td>
<td>4</td>
<td>1.19</td>
</tr>
</tbody>
</table>

8.6 Calculate Factored Pullout Resistance at Elevation of Each Soil Reinforcing Element

The nominal pullout resistance of the soil reinforcing (Equation 8-19) is a function of the length of embedment (L_e), pullout friction factor (F^*), depth of overburden over the soil reinforcing (σ_{vo}), scale correction factor (α), and the geometric factor (C). The factored pullout resistance (σ_{n}) is the product of the nominal pullout resistance (P_r) multiplied by the pullout resistance factor, Φ_{po} (Equation 8-20).
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 $X_m$, $X_i$, $L_e$, $Y_1$ 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](image)

\[
S_{eq} = \frac{A_1 + A_2}{X_1} \quad \text{Equation 8-17}
\]

\[
\sigma_{V_0} = \gamma_r \cdot (Z + S_{eq}) \quad \text{Equation 8-18}
\]

\[
P_r = \alpha \cdot C \cdot F^* \cdot \sigma_{V_0} \cdot L_e \quad \text{Equation 8-19}
\]

\[
P_x = \phi_{po} \cdot P_r \quad \text{Equation 8-20}
\]

The number of Grid-Strip 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 therefore the equations are identical. The Capacity Demand Ratio (Equation 8-22) is a function of the factored resistance of the Grid-Strip and the total number grid strips required (Equation 8-21).

\[
N_{GS} = \text{int} \left( \frac{T_{\text{max}}}{P_\pi} \right) + 1
\]

Equation 8-21

\[
\text{CDR}_{P_O} = \frac{T_{\text{max}}}{N_{GS} \cdot P_\pi}
\]

Equation 8-22

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>2.25</td>
<td>15.41</td>
<td>2.80</td>
<td>22.13</td>
<td>19.92</td>
<td>9.600</td>
<td>2</td>
<td>4.15</td>
</tr>
<tr>
<td>11</td>
<td>3.75</td>
<td>15.41</td>
<td>2.67</td>
<td>23.88</td>
<td>21.49</td>
<td>7.400</td>
<td>2</td>
<td>5.81</td>
</tr>
<tr>
<td>10</td>
<td>6.25</td>
<td>15.41</td>
<td>2.45</td>
<td>26.16</td>
<td>23.54</td>
<td>11.375</td>
<td>2</td>
<td>4.14</td>
</tr>
<tr>
<td>9</td>
<td>8.75</td>
<td>15.41</td>
<td>2.23</td>
<td>27.68</td>
<td>24.91</td>
<td>13.375</td>
<td>2</td>
<td>3.72</td>
</tr>
<tr>
<td>8</td>
<td>11.25</td>
<td>15.41</td>
<td>2.02</td>
<td>28.57</td>
<td>25.71</td>
<td>15.125</td>
<td>2</td>
<td>3.40</td>
</tr>
<tr>
<td>7</td>
<td>13.75</td>
<td>16.25</td>
<td>1.80</td>
<td>29.86</td>
<td>26.88</td>
<td>16.625</td>
<td>2</td>
<td>3.23</td>
</tr>
<tr>
<td>6</td>
<td>16.25</td>
<td>17.75</td>
<td>1.58</td>
<td>31.31</td>
<td>28.18</td>
<td>17.875</td>
<td>2</td>
<td>3.15</td>
</tr>
<tr>
<td>5</td>
<td>18.75</td>
<td>19.25</td>
<td>1.36</td>
<td>31.74</td>
<td>28.56</td>
<td>19.000</td>
<td>2</td>
<td>3.01</td>
</tr>
<tr>
<td>4</td>
<td>21.25</td>
<td>20.75</td>
<td>1.25</td>
<td>33.92</td>
<td>30.53</td>
<td>20.500</td>
<td>2</td>
<td>2.98</td>
</tr>
<tr>
<td>3</td>
<td>23.75</td>
<td>22.25</td>
<td>1.25</td>
<td>39.03</td>
<td>35.13</td>
<td>22.375</td>
<td>2</td>
<td>3.14</td>
</tr>
<tr>
<td>2</td>
<td>26.25</td>
<td>23.75</td>
<td>1.25</td>
<td>44.50</td>
<td>40.05</td>
<td>24.375</td>
<td>2</td>
<td>3.29</td>
</tr>
<tr>
<td>1</td>
<td>28.75</td>
<td>25.25</td>
<td>1.25</td>
<td>50.33</td>
<td>45.30</td>
<td>26.250</td>
<td>2</td>
<td>3.45</td>
</tr>
</tbody>
</table>

8.7 Summary of Internal Stability at Elevation of Each Soil Reinforcing Element

The number of Grid-Strips that are required for each row is equal to the larger number of Grid-Strips required to satisfy tensile capacity calculated in Section 8.5 and the number of Grid-Strips required to satisfy resistance to pullout calculated in Section 8.6. The capacity demand ratios are
Table 32  Summary of Internal Stability for Grid-Strip Element

<table>
<thead>
<tr>
<th>SR</th>
<th>( Z_i ) ft</th>
<th>( N_{SR} ) dim</th>
<th>( \sigma_v ) ksf</th>
<th>( K_r ) dim</th>
<th>( \sigma_H ) ksf</th>
<th>( T_{max} ) kip</th>
<th>CDR Rupture dim</th>
<th>( L_e ) Ft</th>
<th>( F^* ) dim</th>
<th>CDR Pullout dim</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>2.25</td>
<td>2</td>
<td>1.370</td>
<td>0.465</td>
<td>0.640</td>
<td>9.600</td>
<td>1.62</td>
<td>15.41</td>
<td>2.800</td>
<td>4.15</td>
</tr>
<tr>
<td>11</td>
<td>3.75</td>
<td>2</td>
<td>1.640</td>
<td>0.454</td>
<td>0.740</td>
<td>7.400</td>
<td>2.10</td>
<td>15.41</td>
<td>2.670</td>
<td>5.81</td>
</tr>
<tr>
<td>10</td>
<td>6.25</td>
<td>2</td>
<td>2.100</td>
<td>0.436</td>
<td>0.910</td>
<td>11.375</td>
<td>1.37</td>
<td>15.41</td>
<td>2.450</td>
<td>4.14</td>
</tr>
<tr>
<td>9</td>
<td>8.75</td>
<td>2</td>
<td>2.550</td>
<td>0.419</td>
<td>1.070</td>
<td>13.375</td>
<td>1.16</td>
<td>15.41</td>
<td>2.230</td>
<td>3.72</td>
</tr>
<tr>
<td>8</td>
<td>11.25</td>
<td>2</td>
<td>3.010</td>
<td>0.401</td>
<td>1.210</td>
<td>15.125</td>
<td>1.03</td>
<td>15.41</td>
<td>2.020</td>
<td>3.40</td>
</tr>
<tr>
<td>7</td>
<td>13.75</td>
<td>3</td>
<td>3.460</td>
<td>0.383</td>
<td>1.330</td>
<td>16.625</td>
<td>1.40</td>
<td>16.25</td>
<td>1.800</td>
<td>4.85</td>
</tr>
<tr>
<td>6</td>
<td>16.25</td>
<td>3</td>
<td>3.920</td>
<td>0.366</td>
<td>1.430</td>
<td>17.875</td>
<td>1.31</td>
<td>17.75</td>
<td>1.580</td>
<td>4.73</td>
</tr>
<tr>
<td>5</td>
<td>18.75</td>
<td>3</td>
<td>4.370</td>
<td>0.348</td>
<td>1.520</td>
<td>19.000</td>
<td>1.23</td>
<td>19.25</td>
<td>1.360</td>
<td>4.51</td>
</tr>
<tr>
<td>4</td>
<td>21.25</td>
<td>3</td>
<td>4.830</td>
<td>0.339</td>
<td>1.640</td>
<td>20.500</td>
<td>1.14</td>
<td>20.75</td>
<td>1.250</td>
<td>4.47</td>
</tr>
<tr>
<td>3</td>
<td>23.75</td>
<td>3</td>
<td>5.290</td>
<td>0.339</td>
<td>1.790</td>
<td>22.375</td>
<td>1.04</td>
<td>22.25</td>
<td>1.250</td>
<td>4.71</td>
</tr>
<tr>
<td>2</td>
<td>26.25</td>
<td>4</td>
<td>5.740</td>
<td>0.339</td>
<td>1.950</td>
<td>24.375</td>
<td>1.28</td>
<td>23.75</td>
<td>1.250</td>
<td>6.57</td>
</tr>
<tr>
<td>1</td>
<td>28.75</td>
<td>4</td>
<td>6.200</td>
<td>0.339</td>
<td>2.100</td>
<td>26.250</td>
<td>1.19</td>
<td>25.25</td>
<td>1.250</td>
<td>6.90</td>
</tr>
</tbody>
</table>

8.8  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 1.748 and 1.980 respectively. In the program MSE-Pro the calculated CDR for the top soil reinforcing element and the second soil reinforcing element is 1.80 and 1.42 respectively. MSEW under predicts the CDR in the top row and over predicts 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

MSE-Pro Output
MSE - Pro
Mechanically Stabilized Earth Retaining Structures

SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

Project Information

VAWS #:
Name: VAWS Mechanically Stabilized Earth Product Submitt
Location: IDEA Submittal
Project #:
Design Engineer: tpt
Wall Name: SCP MSE Walls

Soil Reinforcing Schedule - Grid Strip

<table>
<thead>
<tr>
<th>Mat-Type</th>
<th>Long Size (W)</th>
<th>Tran Size (W)</th>
<th>Long-Space (ft)</th>
<th>Tran-Space (ft)</th>
<th>Mat Width (ft)</th>
<th>Number Of Mats/Row</th>
<th>Max Stiff.</th>
<th>Ac (in²)</th>
<th>F* - 0</th>
<th>F* - 20</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11.0</td>
<td>11.0</td>
<td>0.167</td>
<td>1.000</td>
<td>0.167</td>
<td>2</td>
<td>6160.000</td>
<td>0.319</td>
<td>3.000</td>
<td>1.250</td>
</tr>
</tbody>
</table>

Soil Parameters

<table>
<thead>
<tr>
<th></th>
<th>Unit Weight</th>
<th>Friction Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>MSE Backfill</td>
<td>0.135 (kcf)</td>
<td>34 (deg)</td>
</tr>
<tr>
<td>Retained Backfill</td>
<td>0.120 (kcf)</td>
<td>30 (deg)</td>
</tr>
<tr>
<td>Foundation Backfill</td>
<td>0.120 (kcf)</td>
<td>30 (deg)</td>
</tr>
<tr>
<td>Earth Surcharge</td>
<td>0.135 (kcf)</td>
<td>30 (deg)</td>
</tr>
<tr>
<td>Depth to Water Table</td>
<td>0.00 (ft)</td>
<td></td>
</tr>
</tbody>
</table>

Internal Stability Parameters

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Yield Stress</td>
<td>65.000 (ksi)</td>
</tr>
<tr>
<td>Yield Coefficient</td>
<td>0.750</td>
</tr>
<tr>
<td>Design Life</td>
<td>75.0 years</td>
</tr>
</tbody>
</table>

Design Options

LRFD Procedure
Stiffness 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 - AASHTO 11.10.6.2.1
The K-Ratios are calculated from the top of the structure or top of coping

General Notes

Project Specifications
2. Course Aggregate
3. Simple MSE Structure
4. Grid-Strip Soil Reinforcing
5. F*=3.00 to 1.25
# SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

## Project Information

| VAWS #: | VAWS Mechanically Stabilized Earth Product Submitt |
| Location: | IDEA Submittal |
| Design Engineer: | tpt |
| Wall Name: | SCP MSE Walls |

## Load and Resistance Factor Design Input Data

<table>
<thead>
<tr>
<th>Load Factor</th>
<th>Maximum</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load factor for vertical earth pressure (EV)</td>
<td>1.35</td>
<td>1.00</td>
</tr>
<tr>
<td>Load factor for earth surcharge (ES)</td>
<td>1.50</td>
<td>0.75</td>
</tr>
<tr>
<td>Load factor for traffic live load (LS)</td>
<td>1.75</td>
<td>1.75</td>
</tr>
<tr>
<td>Load factor for horizontal earth pressure (EH)</td>
<td>1.50</td>
<td>0.90</td>
</tr>
<tr>
<td>Load factor for seismic (EQ)</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Load factor for structural components (DC)</td>
<td>1.25</td>
<td>0.90</td>
</tr>
<tr>
<td>Load factor for water (WA)</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Load factor for live load extreme event (LL)</td>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>Load factor for vehicular impact (CT)</td>
<td>1.50</td>
<td>1.00</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Resistance Factor</th>
<th>Static</th>
<th>Seismic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing resistance factor</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Sliding resistance factor</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Tensile resistance factor</td>
<td>0.75</td>
<td>0.85</td>
</tr>
<tr>
<td>Pullout resistance factor</td>
<td>0.90</td>
<td>1.20</td>
</tr>
<tr>
<td>Extreme event resistance factor</td>
<td>1.00</td>
<td></td>
</tr>
</tbody>
</table>

## Variation of Lateral Earth Pressure Coefficient with Depth (K/Ka)

<table>
<thead>
<tr>
<th>Depth (Z)</th>
<th>Rupture</th>
<th>Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00 (ft)</td>
<td>1.70</td>
<td>1.70</td>
</tr>
<tr>
<td>20.00 (ft)</td>
<td>1.20</td>
<td>1.20</td>
</tr>
</tbody>
</table>

## Variation of Friction Factor with Depth (F*)

<table>
<thead>
<tr>
<th>Depth (Z)</th>
<th>F*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00 (ft)</td>
<td>3.00</td>
</tr>
<tr>
<td>20.00 (ft)</td>
<td>1.25</td>
</tr>
</tbody>
</table>
MSE - Pro
Mechanically Stabilized Earth Retaining Structures

SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

Project Information

VAWS #:
Name: VAWS Mechanically Stabilized Earth Product Submitt
Location: IDEA Submittal
Project #:
Design Engineer: tpt
Wall Name: SCP MSE Walls

External And Internal Stability Calculation Summary

Rec. #1
Structure Parameters

<table>
<thead>
<tr>
<th>H max (ft)</th>
<th>B min (ft)</th>
<th>B Ext. (ft)</th>
<th>S (ft)</th>
<th>Xs (ft)</th>
<th>β (deg)</th>
<th>β i (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>30.000</td>
<td>26.000</td>
<td>26.000</td>
<td>13.00</td>
<td>26.00</td>
<td>26.57</td>
<td>26.57</td>
</tr>
</tbody>
</table>

External Active Earth Pressure Coefficient: 0.537
External Stability CDR Summary (Effective Stress)

<table>
<thead>
<tr>
<th></th>
<th>CDR Sliding</th>
<th>Limiting Eccentricity (ft)</th>
<th>CDR Overturning</th>
<th>Bearing Pressure (ksf)</th>
<th>Eccentricity (ft)</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength 1 - Min</td>
<td>1.83</td>
<td>1.82</td>
<td>3.48</td>
<td>6.80</td>
<td>1.82</td>
<td>0.500</td>
</tr>
<tr>
<td>Strength 1 - Max</td>
<td>1.54</td>
<td>2.31</td>
<td>2.99</td>
<td>9.96</td>
<td>2.31</td>
<td>0.500</td>
</tr>
<tr>
<td>Strength 1 - Critical</td>
<td>1.21</td>
<td>3.14</td>
<td>2.45</td>
<td>8.52</td>
<td>3.14</td>
<td>0.500</td>
</tr>
<tr>
<td>Service</td>
<td>1.68</td>
<td>2.06</td>
<td>3.22</td>
<td>7.07</td>
<td>2.06</td>
<td>0.500</td>
</tr>
</tbody>
</table>

Internal Active Earth Pressure Coefficient: 0.283
Internal Stability CDR Summary

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Internal Earth Coeff.</th>
<th>Rupture CDR</th>
<th>Pullout CDR</th>
<th>F*</th>
<th>Le (ft)</th>
<th>H/B Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>14.25</td>
<td>26.00</td>
<td>2</td>
<td>0.47</td>
<td>2.23</td>
<td>5.38</td>
<td>2.89</td>
<td>15.41</td>
<td>0.87</td>
</tr>
<tr>
<td>16.75</td>
<td>26.00</td>
<td>2</td>
<td>0.45</td>
<td>1.67</td>
<td>4.63</td>
<td>2.67</td>
<td>15.41</td>
<td>0.87</td>
</tr>
<tr>
<td>19.25</td>
<td>26.00</td>
<td>2</td>
<td>0.44</td>
<td>1.36</td>
<td>4.13</td>
<td>2.45</td>
<td>15.41</td>
<td>0.87</td>
</tr>
<tr>
<td>21.75</td>
<td>26.00</td>
<td>2</td>
<td>0.42</td>
<td>1.17</td>
<td>3.75</td>
<td>2.23</td>
<td>15.41</td>
<td>0.87</td>
</tr>
<tr>
<td>24.25</td>
<td>26.00</td>
<td>2</td>
<td>0.40</td>
<td>1.03</td>
<td>3.41</td>
<td>2.02</td>
<td>15.41</td>
<td>0.87</td>
</tr>
<tr>
<td>26.75</td>
<td>26.00</td>
<td>3</td>
<td>0.38</td>
<td>1.41</td>
<td>4.86</td>
<td>1.80</td>
<td>16.25</td>
<td>0.87</td>
</tr>
<tr>
<td>29.25</td>
<td>26.00</td>
<td>3</td>
<td>0.37</td>
<td>1.30</td>
<td>4.72</td>
<td>1.58</td>
<td>17.75</td>
<td>0.87</td>
</tr>
<tr>
<td>31.75</td>
<td>26.00</td>
<td>3</td>
<td>0.35</td>
<td>1.23</td>
<td>4.51</td>
<td>1.36</td>
<td>19.25</td>
<td>0.87</td>
</tr>
<tr>
<td>34.25</td>
<td>26.00</td>
<td>3</td>
<td>0.34</td>
<td>1.14</td>
<td>4.48</td>
<td>1.25</td>
<td>20.75</td>
<td>0.87</td>
</tr>
<tr>
<td>36.75</td>
<td>26.00</td>
<td>3</td>
<td>0.34</td>
<td>1.04</td>
<td>4.71</td>
<td>1.25</td>
<td>22.25</td>
<td>0.87</td>
</tr>
<tr>
<td>39.25</td>
<td>26.00</td>
<td>4</td>
<td>0.34</td>
<td>1.28</td>
<td>6.59</td>
<td>1.25</td>
<td>23.75</td>
<td>0.87</td>
</tr>
<tr>
<td>41.75</td>
<td>26.00</td>
<td>4</td>
<td>0.34</td>
<td>1.18</td>
<td>6.91</td>
<td>1.25</td>
<td>25.25</td>
<td>0.87</td>
</tr>
</tbody>
</table>
### Static Internal Stability - Rupture Force Results
**Rec. #1 - H = 30.00 (ft) B = 26.00 (ft)**

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horizontal Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Static CDR Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>14.25</td>
<td>26.00</td>
<td>2</td>
<td>12.50</td>
<td>0.47</td>
<td>1.18</td>
<td>0.56</td>
<td>6.99</td>
<td>2.23</td>
</tr>
<tr>
<td>16.75</td>
<td>26.00</td>
<td>2</td>
<td>12.50</td>
<td>0.45</td>
<td>1.64</td>
<td>0.74</td>
<td>9.31</td>
<td>1.67</td>
</tr>
<tr>
<td>19.25</td>
<td>26.00</td>
<td>2</td>
<td>12.50</td>
<td>0.44</td>
<td>2.10</td>
<td>0.91</td>
<td>11.43</td>
<td>1.36</td>
</tr>
<tr>
<td>21.75</td>
<td>26.00</td>
<td>2</td>
<td>12.50</td>
<td>0.42</td>
<td>2.55</td>
<td>1.07</td>
<td>13.36</td>
<td>1.17</td>
</tr>
<tr>
<td>24.25</td>
<td>26.00</td>
<td>2</td>
<td>12.50</td>
<td>0.40</td>
<td>3.01</td>
<td>1.21</td>
<td>15.08</td>
<td>1.03</td>
</tr>
<tr>
<td>26.75</td>
<td>26.00</td>
<td>3</td>
<td>12.50</td>
<td>0.38</td>
<td>3.46</td>
<td>1.33</td>
<td>16.60</td>
<td>1.41</td>
</tr>
<tr>
<td>29.25</td>
<td>26.00</td>
<td>3</td>
<td>12.50</td>
<td>0.37</td>
<td>3.92</td>
<td>1.43</td>
<td>17.92</td>
<td>1.30</td>
</tr>
<tr>
<td>31.75</td>
<td>26.00</td>
<td>3</td>
<td>12.50</td>
<td>0.35</td>
<td>4.37</td>
<td>1.52</td>
<td>19.03</td>
<td>1.23</td>
</tr>
<tr>
<td>34.25</td>
<td>26.00</td>
<td>3</td>
<td>12.50</td>
<td>0.34</td>
<td>4.83</td>
<td>1.64</td>
<td>20.48</td>
<td>1.14</td>
</tr>
<tr>
<td>36.75</td>
<td>26.00</td>
<td>3</td>
<td>12.50</td>
<td>0.34</td>
<td>5.29</td>
<td>1.79</td>
<td>22.41</td>
<td>1.04</td>
</tr>
<tr>
<td>39.25</td>
<td>26.00</td>
<td>4</td>
<td>12.50</td>
<td>0.34</td>
<td>5.74</td>
<td>1.95</td>
<td>24.35</td>
<td>1.28</td>
</tr>
<tr>
<td>41.75</td>
<td>26.00</td>
<td>4</td>
<td>12.50</td>
<td>0.34</td>
<td>6.20</td>
<td>2.10</td>
<td>26.28</td>
<td>1.18</td>
</tr>
</tbody>
</table>

### Static Internal Stability - Pullout Force Results
**Rec. #1 - H = 30.00 (ft) B = 26.00 (ft)**

<table>
<thead>
<tr>
<th>Depth Below Top of Wall (ft)</th>
<th>B min (ft)</th>
<th>SR/Row</th>
<th>Tributary Area (sf)</th>
<th>Internal Earth Coeff. Rupture</th>
<th>Static Vertical Stress (ksf)</th>
<th>Static Horz. Stress (ksf)</th>
<th>Static Tmax (Kips)</th>
<th>Le (ft)</th>
<th>F*</th>
<th>Pr (Kips)</th>
<th>Static CDR Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>14.25</td>
<td>26.00</td>
<td>2</td>
<td>12.50</td>
<td>0.47</td>
<td>1.40</td>
<td>0.66</td>
<td>6.99</td>
<td>15.41</td>
<td>2.89</td>
<td>37.60</td>
<td>5.38</td>
</tr>
<tr>
<td>16.75</td>
<td>26.00</td>
<td>2</td>
<td>12.50</td>
<td>0.45</td>
<td>1.74</td>
<td>0.79</td>
<td>9.31</td>
<td>15.41</td>
<td>2.67</td>
<td>43.11</td>
<td>4.63</td>
</tr>
<tr>
<td>19.25</td>
<td>26.00</td>
<td>2</td>
<td>12.50</td>
<td>0.44</td>
<td>2.08</td>
<td>0.91</td>
<td>11.43</td>
<td>15.41</td>
<td>2.45</td>
<td>47.25</td>
<td>4.13</td>
</tr>
<tr>
<td>21.75</td>
<td>26.00</td>
<td>2</td>
<td>12.50</td>
<td>0.42</td>
<td>2.42</td>
<td>1.01</td>
<td>13.36</td>
<td>15.41</td>
<td>2.23</td>
<td>50.02</td>
<td>3.75</td>
</tr>
<tr>
<td>24.25</td>
<td>26.00</td>
<td>2</td>
<td>12.50</td>
<td>0.40</td>
<td>2.75</td>
<td>1.10</td>
<td>15.08</td>
<td>15.41</td>
<td>2.02</td>
<td>51.43</td>
<td>3.41</td>
</tr>
<tr>
<td>26.75</td>
<td>26.00</td>
<td>3</td>
<td>12.50</td>
<td>0.38</td>
<td>3.06</td>
<td>1.17</td>
<td>16.60</td>
<td>16.25</td>
<td>1.80</td>
<td>80.65</td>
<td>4.86</td>
</tr>
<tr>
<td>29.25</td>
<td>26.00</td>
<td>3</td>
<td>12.50</td>
<td>0.37</td>
<td>3.35</td>
<td>1.23</td>
<td>17.92</td>
<td>17.75</td>
<td>1.58</td>
<td>84.62</td>
<td>4.72</td>
</tr>
<tr>
<td>31.75</td>
<td>26.00</td>
<td>3</td>
<td>12.50</td>
<td>0.35</td>
<td>3.64</td>
<td>1.27</td>
<td>19.03</td>
<td>19.25</td>
<td>1.36</td>
<td>85.82</td>
<td>4.51</td>
</tr>
<tr>
<td>34.25</td>
<td>26.00</td>
<td>3</td>
<td>12.50</td>
<td>0.34</td>
<td>3.92</td>
<td>1.33</td>
<td>20.48</td>
<td>20.75</td>
<td>1.25</td>
<td>91.77</td>
<td>4.48</td>
</tr>
<tr>
<td>36.75</td>
<td>26.00</td>
<td>3</td>
<td>12.50</td>
<td>0.34</td>
<td>4.21</td>
<td>1.43</td>
<td>22.41</td>
<td>22.25</td>
<td>1.25</td>
<td>105.60</td>
<td>4.71</td>
</tr>
<tr>
<td>39.25</td>
<td>26.00</td>
<td>4</td>
<td>12.50</td>
<td>0.34</td>
<td>4.50</td>
<td>1.53</td>
<td>24.35</td>
<td>23.75</td>
<td>1.25</td>
<td>160.54</td>
<td>6.59</td>
</tr>
<tr>
<td>41.75</td>
<td>26.00</td>
<td>4</td>
<td>12.50</td>
<td>0.34</td>
<td>4.78</td>
<td>1.62</td>
<td>26.28</td>
<td>25.25</td>
<td>1.25</td>
<td>181.56</td>
<td>6.91</td>
</tr>
</tbody>
</table>
SEGMENTAL CONCRETE PANEL SYSTEM WITH GRID STRIP

**Project Information**

<table>
<thead>
<tr>
<th>VAWS #:</th>
<th>VAWS Mechanically Stabilized Earth Product Submitt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name:</td>
<td>IDEA Submittal</td>
</tr>
<tr>
<td>Location:</td>
<td></td>
</tr>
<tr>
<td>Project #:</td>
<td></td>
</tr>
<tr>
<td>Design Engineer:</td>
<td>tpt</td>
</tr>
<tr>
<td>Wall Name:</td>
<td>SCP MSE Walls</td>
</tr>
</tbody>
</table>

**Legend**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Abbreviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>H max</td>
<td>Height of the wall</td>
<td>FS</td>
</tr>
<tr>
<td>B min</td>
<td>Minimum length of soil reinforcing for defined wall height</td>
<td>e</td>
</tr>
<tr>
<td>S</td>
<td>Maximum height of surcharge</td>
<td>BP</td>
</tr>
<tr>
<td>Xs</td>
<td>Distance of surcharge slope over soil reinforcement</td>
<td>F*</td>
</tr>
<tr>
<td>β</td>
<td>Slope of surcharge</td>
<td>s</td>
</tr>
<tr>
<td>βi</td>
<td>Adjusted angle</td>
<td>o</td>
</tr>
<tr>
<td>Le</td>
<td>Length of embedment of soil reinforcing in passive zone</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Appendix B

MSEW Output
AASHTO 2007-2010 (LRFD)
Design Case - Infinite Back-Slope
MSEW(3.0): Update # 14.96

PROJECT IDENTIFICATION

Title: Design Case - Infinite Back-Slope
Project Number: Grid-Strip IDEA
Client: LRFD Binder - Infinite Back-Slope
Designer: tpt
Station Number: 1

Description:
LRFD Verification Infinite Back-Slope

Company's information:
Name: VAWS
Street: 650 Justice Lane
Mansfield, TX  76063
Telephone #: 817-507-0200
Fax #: 817-507-0197
E-Mail: ttaylor@bigrbridge.com

Original file path and name: I:\Toms Computer\AB My Data\DOT\AA-IDEA\Calculations\Infinite BackSlope\Grid-Strip IDEA Infinite.BEN

Original date and time of creating this file: 06-06-17

PROGRAM MODE:

ANALYSIS of a SIMPLE STRUCTURE using METAL STRIPS as reinforcing material.
**SOIL DATA**

**REINFORCED SOIL**

- Unit weight, $\gamma$: 135.0 lb/ft$^3$
- Design value of internal angle of friction, $\phi$: 34.0°

**RETAINED SOIL**

- Unit weight, $\gamma$: 120.0 lb/ft$^3$
- Design value of internal angle of friction, $\phi$: 30.0°

**FOUNDATION SOIL (Considered as an equivalent uniform soil)**

- Equivalent unit weight, $\gamma_{equiv}$: 120.0 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**

- $K_a$ (internal stability) = 0.2827 (if batter is less than 10°, $K_a$ is calculated from eq. 15. Otherwise, eq. 38 is utilized)
- $K_a$ (external stability) = 0.5365 (if batter is less than 10°, $K_a$ is calculated from eq. 16. Otherwise, eq. 17 is utilized)

**BEARING CAPACITY**

Bearing capacity coefficients (calculated by MSEW): $N_c = 30.14$  \(N \gamma = 22.40\)

**SEISMICITY**

Not Applicable
### INPUT DATA: Metal strips
(Analysis)

<table>
<thead>
<tr>
<th>DATA</th>
<th>Metal strip type #1</th>
<th>Metal strip type #2</th>
<th>Metal strip type #3</th>
<th>Metal strip type #4</th>
<th>Metal strip type #5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield strength of steel, Fy [kips/in²]</td>
<td>65.0</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Gross width of strip, b [in]</td>
<td>2.0</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Vertical spacing, Sv [ft]</td>
<td>Varies</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Design cross section area, Ac [in²]</td>
<td>0.16</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Ribbed steel strips.
Uniformity Coefficient of reinforced soil, Cu = D60/D10 = 4.0

Friction angle along reinforcement-soil interface, \( \rho \)
- @ the top: 60.97
- @ 19.7 ft or below: 34.00

Pullout resistance factor, \( F^* \)
- @ the top: 3.00
- @ 19.7 ft or below: 1.25

Scale-effect correction factor, \( \alpha \)
- 1.00

Variation of Lateral Earth Pressure Coefficient With Depth

<table>
<thead>
<tr>
<th>Z</th>
<th>( K / K_a )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 ft</td>
<td>1.70</td>
</tr>
<tr>
<td>3.3 ft</td>
<td>1.60</td>
</tr>
<tr>
<td>6.6 ft</td>
<td>1.55</td>
</tr>
<tr>
<td>9.8 ft</td>
<td>1.45</td>
</tr>
<tr>
<td>13.1 ft</td>
<td>1.35</td>
</tr>
<tr>
<td>16.4 ft</td>
<td>1.30</td>
</tr>
<tr>
<td>19.7 ft</td>
<td>1.20</td>
</tr>
</tbody>
</table>

Graph showing variation of \( K / K_a \) with depth (Z).
**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$^3$.

<table>
<thead>
<tr>
<th>$Z / H_d$</th>
<th>To-static / $T_{max}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>1.00</td>
</tr>
<tr>
<td>0.25</td>
<td>1.00</td>
</tr>
<tr>
<td>0.50</td>
<td>1.00</td>
</tr>
<tr>
<td>0.75</td>
<td>1.00</td>
</tr>
<tr>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Design</th>
<th>Type #1</th>
<th>Type #2</th>
<th>Type #3</th>
<th>Type #4</th>
<th>Type #5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Product Name</td>
<td>GS11</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Strength reduction at the connection, CRu = Fyc / Fy</td>
<td>1.00</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>
INPUT DATA: Geometry and Surchage loads (of a SIMPLE STRUCTURE)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design height, (H_d)</td>
<td>30.00</td>
<td>ft</td>
<td>Embedded depth is (E = 3.00) ft, and height above top of finished bottom grade is (H = 27.00) ft</td>
</tr>
<tr>
<td>Batter, (\omega)</td>
<td>0.0</td>
<td>deg</td>
<td></td>
</tr>
<tr>
<td>Backslope, (\beta)</td>
<td>26.6</td>
<td>deg</td>
<td></td>
</tr>
<tr>
<td>Backslope rise</td>
<td>50.0</td>
<td>ft</td>
<td>Broken back equivalent angle, (I = 26.56^\circ) (see Fig. 25 in DEMO 82)</td>
</tr>
</tbody>
</table>

UNIFORM SURCHARGE

Uniformly distributed dead load is 0.0 \([\text{lb/ft}^2]\)
## AASHTO 2007-2010 (LRFD) Input Data

### INTERNAL STABILITY

| Load factor for vertical earth pressure, EV, from Table 3.4.1-2: | $\gamma_{p-EV}$ | 1.35 |
| Load factor for earthquake loads, EQ, from Table 3.4.1-1: | $\gamma_{p-EQ}$ | 1.00 |
| Load factor for live load surcharge, LS, from Figure C11.5.5-3(b): (Same as in External Stability). | $\gamma_{p-LS}$ | 1.75 |
| Load factor for dead load surcharge, ES: (Same as in External Stability). | $\gamma_{p-ES}$ | 1.50 |

| Resistance factor for reinforcement tension from Table 11.5.6-1: Metal Strips: | $\phi$ | Static | Combined static/seismic |
| | | 0.75 | 1.00 |

| Resistance factor for reinforcement tension in connectors from Table 11.5.6-1: Metal Strips: | $\phi$ | Static | Combined static/seismic |
| | | 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: Sliding and Eccentricity | $\gamma_{p-EV}$ | 1.00 | $\gamma_{p-EQ}$ | 1.00 |
| Bearing Capacity | $\gamma_{p-EV}$ | 1.35 | $\gamma_{p-EQ}$ | 1.35 |

| Load factor of active lateral earth pressure, EH, from Table 3.4.1-2 and Figure C11.5.5-2: | $\gamma_{p-EH}$ | 1.50 |
| Load factor of active lateral earth pressure during earthquake (does not multiply $P_{AE}$ and $P_{IR}$): | $(\gamma_{p-EH})_{EQ}$ | 1.50 |
| Load factor for earthquake loads, EQ, from Table 3.4.1-1 (multiplies $P_{AE}$ and $P_{IR}$): | $\gamma_{p-EQ}$ | 1.00 |

| Resistance factor for shear resistance along common interfaces from Table 11.5.6-1: Reinforced Soil and Foundation | $\phi_{t}$ | 1.00 | 1.00 |
| Reinforced Soil and Reinforcement | $\phi_{t}$ | 1.00 | 1.00 |

| Resistance factor for bearing capacity of shallow foundation from Table 11.5.6-1: | $\phi_{b}$ | 0.65 | 0.65 |
**BEARING CAPACITY for GIVEN LAYOUT**

(Water table does not affect bearing capacity)

<table>
<thead>
<tr>
<th>STATIC</th>
<th>SEISMIC</th>
<th>UNITS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factored bearing resistance, q-n</td>
<td>18677</td>
<td>N/A</td>
</tr>
<tr>
<td>Factored bearing load, σv</td>
<td>9957.6</td>
<td>N/A</td>
</tr>
<tr>
<td>Eccentricity, e</td>
<td>2.31</td>
<td>N/A</td>
</tr>
<tr>
<td>Eccentricity, e/L</td>
<td>0.089</td>
<td>N/A</td>
</tr>
<tr>
<td>CDR calculated</td>
<td>1.88</td>
<td>N/A</td>
</tr>
<tr>
<td>Base length</td>
<td>26.00</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Unfactored applied bearing pressure = \( \frac{\text{Unfactored R}}{\left[ L - 2 \times \text{Unfactored e} \right]} \) =

Unfactored R = 154719.03 [lb/ft], L = 26.00, Unfactored e = 2.06 [ft], and Sigma = 7068.87 [lb/ft²]
## DIRECT SLIDING for GIVEN LAYOUT (for METAL STRIPS reinforcements)

Along reinforced and foundation soils interface: CDR-static = 1.215

<table>
<thead>
<tr>
<th>#</th>
<th>Metal strip Elevation [ft]</th>
<th>Metal strip Length [ft]</th>
<th>CDR Static</th>
<th>CDR Seismic</th>
<th>Metal strip Type #</th>
<th>Product name</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.25</td>
<td>26.00</td>
<td>1.446</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>2</td>
<td>3.75</td>
<td>26.00</td>
<td>1.503</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>3</td>
<td>6.25</td>
<td>26.00</td>
<td>1.565</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>4</td>
<td>8.75</td>
<td>26.00</td>
<td>1.634</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>5</td>
<td>11.25</td>
<td>26.00</td>
<td>1.710</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>6</td>
<td>13.75</td>
<td>26.00</td>
<td>1.795</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>7</td>
<td>16.25</td>
<td>26.00</td>
<td>1.889</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>8</td>
<td>18.75</td>
<td>26.00</td>
<td>1.992</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>9</td>
<td>21.25</td>
<td>26.00</td>
<td>2.105</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>10</td>
<td>23.75</td>
<td>26.00</td>
<td>2.224</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>11</td>
<td>26.25</td>
<td>26.00</td>
<td>2.340</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>12</td>
<td>27.75</td>
<td>26.00</td>
<td>2.400</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
</tbody>
</table>

## ECCENTRICITY for GIVEN LAYOUT (for Simplified Method)

At interface with foundation: e/L static = 0.1206; Overturning: CDR-static = 2.45

<table>
<thead>
<tr>
<th>#</th>
<th>Metal strip Elevation [ft]</th>
<th>Metal strip Length [ft]</th>
<th>e / L Static</th>
<th>e / L Seismic</th>
<th>Metal strip Type #</th>
<th>Product name</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.25</td>
<td>26.00</td>
<td>0.1095</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>2</td>
<td>3.75</td>
<td>26.00</td>
<td>0.0880</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>3</td>
<td>6.25</td>
<td>26.00</td>
<td>0.0672</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>4</td>
<td>8.75</td>
<td>26.00</td>
<td>0.0471</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>5</td>
<td>11.25</td>
<td>26.00</td>
<td>0.0275</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>6</td>
<td>13.75</td>
<td>26.00</td>
<td>0.0083</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>7</td>
<td>16.25</td>
<td>26.00</td>
<td>-0.0107</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>8</td>
<td>18.75</td>
<td>26.00</td>
<td>-0.0301</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>9</td>
<td>21.25</td>
<td>26.00</td>
<td>-0.0504</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>10</td>
<td>23.75</td>
<td>26.00</td>
<td>-0.0730</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>11</td>
<td>26.25</td>
<td>26.00</td>
<td>-0.1006</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
<tr>
<td>12</td>
<td>27.75</td>
<td>26.00</td>
<td>-0.1217</td>
<td>N/A</td>
<td>1</td>
<td>GS11</td>
</tr>
</tbody>
</table>
### RESULTS for STRENGTH

[Note: Actual CDR = (Yield stress) / (Actual stress)]

For Simplified Method

<table>
<thead>
<tr>
<th>Metal strip Elevation [ft]</th>
<th>Coverage ratio, Re=b/Sh</th>
<th>Horizontal spacing, Sh [ft]</th>
<th>Long-term strength Fy·Ac·Re/b [lb/ft]</th>
<th>Tmax [lb/ft]</th>
<th>Tmd [lb/ft]</th>
<th>Specified minimum CDR static</th>
<th>Actual calculated CDR static</th>
<th>Specified minimum CDR seismic</th>
<th>Actual calculated CDR seismic</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.25</td>
<td>0.133</td>
<td>1.25</td>
<td>6240</td>
<td>5255.35</td>
<td>N/A</td>
<td>N/A</td>
<td>1.187</td>
<td>N/A</td>
</tr>
<tr>
<td>2</td>
<td>3.75</td>
<td>0.133</td>
<td>1.25</td>
<td>6240</td>
<td>4868.91</td>
<td>N/A</td>
<td>N/A</td>
<td>1.282</td>
<td>N/A</td>
</tr>
<tr>
<td>3</td>
<td>6.25</td>
<td>0.100</td>
<td>1.67</td>
<td>4671</td>
<td>4482.48</td>
<td>N/A</td>
<td>N/A</td>
<td>1.042</td>
<td>N/A</td>
</tr>
<tr>
<td>4</td>
<td>8.75</td>
<td>0.100</td>
<td>1.67</td>
<td>4671</td>
<td>4096.04</td>
<td>N/A</td>
<td>N/A</td>
<td>1.140</td>
<td>N/A</td>
</tr>
<tr>
<td>5</td>
<td>11.25</td>
<td>0.100</td>
<td>1.67</td>
<td>4671</td>
<td>3807.18</td>
<td>N/A</td>
<td>N/A</td>
<td>1.227</td>
<td>N/A</td>
</tr>
<tr>
<td>6</td>
<td>13.75</td>
<td>0.100</td>
<td>1.67</td>
<td>4671</td>
<td>3579.07</td>
<td>N/A</td>
<td>N/A</td>
<td>1.305</td>
<td>N/A</td>
</tr>
<tr>
<td>7</td>
<td>16.25</td>
<td>0.100</td>
<td>1.67</td>
<td>4671</td>
<td>3288.25</td>
<td>N/A</td>
<td>N/A</td>
<td>1.420</td>
<td>N/A</td>
</tr>
<tr>
<td>8</td>
<td>18.75</td>
<td>0.100</td>
<td>1.67</td>
<td>4671</td>
<td>3094.30</td>
<td>N/A</td>
<td>N/A</td>
<td>1.565</td>
<td>N/A</td>
</tr>
<tr>
<td>9</td>
<td>21.25</td>
<td>0.067</td>
<td>2.500</td>
<td>3120</td>
<td>2668.58</td>
<td>N/A</td>
<td>N/A</td>
<td>1.169</td>
<td>N/A</td>
</tr>
<tr>
<td>10</td>
<td>23.75</td>
<td>0.067</td>
<td>2.500</td>
<td>3120</td>
<td>2288.07</td>
<td>N/A</td>
<td>N/A</td>
<td>1.364</td>
<td>N/A</td>
</tr>
<tr>
<td>11</td>
<td>26.25</td>
<td>0.067</td>
<td>2.500</td>
<td>3120</td>
<td>1514.75</td>
<td>N/A</td>
<td>N/A</td>
<td>2.060</td>
<td>N/A</td>
</tr>
<tr>
<td>12</td>
<td>27.75</td>
<td>0.067</td>
<td>2.500</td>
<td>3120</td>
<td>1715.14</td>
<td>N/A</td>
<td>N/A</td>
<td>1.819</td>
<td>N/A</td>
</tr>
</tbody>
</table>

### RESULTS for PULLOUT

Live Load NOT included in calculating Tmax

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.25</td>
<td>0.133</td>
<td>5255.3</td>
<td>N/A</td>
<td>25.25</td>
<td>0.75</td>
<td>36232.5</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>2</td>
<td>3.75</td>
<td>0.133</td>
<td>4868.9</td>
<td>N/A</td>
<td>23.75</td>
<td>2.25</td>
<td>32030.0</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>3</td>
<td>6.25</td>
<td>0.100</td>
<td>4482.5</td>
<td>N/A</td>
<td>22.25</td>
<td>3.75</td>
<td>21031.2</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>4</td>
<td>8.75</td>
<td>0.100</td>
<td>4096.0</td>
<td>N/A</td>
<td>20.75</td>
<td>5.25</td>
<td>18276.7</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>5</td>
<td>11.25</td>
<td>0.100</td>
<td>3807.2</td>
<td>N/A</td>
<td>19.25</td>
<td>6.75</td>
<td>16760.5</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>6</td>
<td>13.75</td>
<td>0.100</td>
<td>3579.1</td>
<td>N/A</td>
<td>17.75</td>
<td>8.25</td>
<td>16608.1</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>7</td>
<td>16.25</td>
<td>0.100</td>
<td>3288.2</td>
<td>N/A</td>
<td>16.25</td>
<td>9.75</td>
<td>15888.5</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>8</td>
<td>18.75</td>
<td>0.100</td>
<td>2984.3</td>
<td>N/A</td>
<td>15.41</td>
<td>10.59</td>
<td>15234.4</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>9</td>
<td>21.25</td>
<td>0.067</td>
<td>2668.6</td>
<td>N/A</td>
<td>15.41</td>
<td>10.59</td>
<td>9921.3</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>10</td>
<td>23.75</td>
<td>0.067</td>
<td>2288.1</td>
<td>N/A</td>
<td>15.41</td>
<td>10.59</td>
<td>9388.7</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>11</td>
<td>26.25</td>
<td>0.067</td>
<td>1514.7</td>
<td>N/A</td>
<td>15.41</td>
<td>10.59</td>
<td>8578.9</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>12</td>
<td>27.75</td>
<td>0.067</td>
<td>1715.1</td>
<td>N/A</td>
<td>15.41</td>
<td>10.59</td>
<td>7959.8</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>
General Information

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Date</td>
<td></td>
</tr>
<tr>
<td>Designer</td>
<td>tpt</td>
</tr>
<tr>
<td>Title</td>
<td>Design Case – Horizontal Back-Slope</td>
</tr>
<tr>
<td>Number</td>
<td>Grid Strip - IDEA</td>
</tr>
<tr>
<td>Client</td>
<td>LRFD Binder - Infinite Back-Slope</td>
</tr>
</tbody>
</table>

Geometry & Surcharge

<table>
<thead>
<tr>
<th>Geometry</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Height</td>
<td>ft 30.00</td>
</tr>
<tr>
<td>Backslope</td>
<td>deg 0.00</td>
</tr>
<tr>
<td>Backslope rise</td>
<td>ft 0.00</td>
</tr>
<tr>
<td>Embedment</td>
<td>ft 0.00</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Surcharge</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniformly Distributed</td>
<td></td>
</tr>
<tr>
<td>Dead load</td>
<td>psf 0.00</td>
</tr>
<tr>
<td>Live load</td>
<td>psf 0.00</td>
</tr>
</tbody>
</table>

Soil Properties

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced Soil</td>
<td></td>
</tr>
<tr>
<td>Unit weight</td>
<td>pcf 135</td>
</tr>
<tr>
<td>Friction angle</td>
<td>deg 34</td>
</tr>
<tr>
<td>Cohesion</td>
<td>psf 0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Retained Soil</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight</td>
<td>pcf 120</td>
</tr>
<tr>
<td>Friction angle</td>
<td>deg 30</td>
</tr>
<tr>
<td>Cohesion</td>
<td>psf 0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Foundation Soil</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight</td>
<td>pcf 120</td>
</tr>
<tr>
<td>Friction angle</td>
<td>deg 30</td>
</tr>
<tr>
<td>Cohesion</td>
<td>psf 0</td>
</tr>
</tbody>
</table>

Soil Reinforcement

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Discrete Strip</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Product Name</th>
<th>Yield Strength (ksi)</th>
<th>Gross Width (in)</th>
<th>Cross Sectional Area (in²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GS11</td>
<td>65</td>
<td>2</td>
<td>0.16</td>
</tr>
</tbody>
</table>
Layer | Height (ft) | Length (ft) | Horizontal Spacing (ft) | Type
--- | --- | --- | --- | ---
1 | 1.25 | 26.00 | 1.25 | 1
2 | 3.75 | 26.00 | 1.67 | 1
3 | 6.25 | 26.00 | 1.67 | 1
4 | 8.75 | 26.00 | 1.67 | 1
5 | 11.25 | 26.00 | 1.67 | 1
6 | 13.75 | 26.00 | 2.50 | 1
7 | 16.25 | 26.00 | 2.50 | 1
8 | 18.75 | 26.00 | 2.50 | 1
9 | 21.25 | 26.00 | 2.50 | 1
10 | 23.75 | 26.00 | 2.50 | 1
11 | 26.25 | 26.00 | 2.50 | 1
12 | 27.75 | 26.00 | 2.50 | 1

### Pullout Parameters

<table>
<thead>
<tr>
<th>Type</th>
<th>( \rho ) (dim)</th>
<th>( F^* ) (dim)</th>
<th>( \alpha ) (dim)</th>
<th>Top</th>
<th>19.7</th>
<th>Top</th>
<th>19.7</th>
<th>1.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>60.93</td>
<td>34</td>
<td>3.00</td>
<td></td>
<td>1.25</td>
<td></td>
<td>1.25</td>
<td>1.00</td>
</tr>
</tbody>
</table>

### Variation of Lateral Earth Pressure Coefficient with Depth (K/Ka)

<table>
<thead>
<tr>
<th>Depth (Z)</th>
<th>Rupture</th>
<th>Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>1.7</td>
<td>1.7</td>
</tr>
<tr>
<td>20.00</td>
<td>1.2</td>
<td>1.2</td>
</tr>
</tbody>
</table>

### Facing Panel

<table>
<thead>
<tr>
<th>Depth of Concrete (ft)</th>
<th>Unit Weight (pcf)</th>
<th>Distance to Center (ft)</th>
<th>Strength Reduction (dim)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.50</td>
<td>150</td>
<td>0.25</td>
<td>1.00</td>
</tr>
</tbody>
</table>

### Load and Resistance Factor Design Input Data

<table>
<thead>
<tr>
<th>Load Factor</th>
<th>Maximum</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load factor for vertical earth pressure (EV)</td>
<td>1.35</td>
<td>1.00</td>
</tr>
<tr>
<td>Load factor for earth surcharge (ES)</td>
<td>1.50</td>
<td>0.75</td>
</tr>
<tr>
<td>Load factor for traffic live load (LS)</td>
<td>1.75</td>
<td>1.75</td>
</tr>
<tr>
<td>Load factor for horizontal earth pressure (EH)</td>
<td>1.50</td>
<td>0.75</td>
</tr>
<tr>
<td>Load factor for seismic (EQ)</td>
<td>1.00</td>
<td></td>
</tr>
</tbody>
</table>
### External

<table>
<thead>
<tr>
<th>Load Factor</th>
<th>Maximum</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load factor for structural components (DC)</td>
<td>1.25</td>
<td>0.90</td>
</tr>
<tr>
<td>Load factors for water (WA)</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Load factor for live load extreme event (LL)</td>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>Load factor for vehicular impact (CT)</td>
<td>1.50</td>
<td>1.00</td>
</tr>
</tbody>
</table>

### Internal

<table>
<thead>
<tr>
<th>Load Factor</th>
<th>Maximum</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load factor for vertical earth pressure (EV)</td>
<td>1.35</td>
<td>1.00</td>
</tr>
<tr>
<td>Load factor for earth surcharge (ES)</td>
<td>1.50</td>
<td>0.75</td>
</tr>
<tr>
<td>Load factor for traffic live load (LS)</td>
<td>1.35</td>
<td>1.35</td>
</tr>
<tr>
<td>Load factor for horizontal earth pressure (EH)</td>
<td>1.50</td>
<td>0.75</td>
</tr>
<tr>
<td>Load factor for seismic (EQ)</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Load factor for structural components (DC)</td>
<td>1.25</td>
<td>0.90</td>
</tr>
<tr>
<td>Load factors for water (WA)</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Load factor for live load extreme event (LL)</td>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>Load factor for vehicular impact (CT)</td>
<td>1.50</td>
<td>1.00</td>
</tr>
</tbody>
</table>

### Resistance Factor

<table>
<thead>
<tr>
<th>Resistance Factor</th>
<th>Static</th>
<th>Seismic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing resistance factor</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Sliding resistance factor</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Tensile resistance factor</td>
<td>0.75</td>
<td>1.00</td>
</tr>
<tr>
<td>Pullout resistance factor</td>
<td>0.90</td>
<td>1.20</td>
</tr>
<tr>
<td>Extreme event resistance factor</td>
<td>1.00</td>
<td></td>
</tr>
</tbody>
</table>
3.0
CONSTRUCTION
3.1
CONSTRUCTION PROCEDURES
3.1.1
ERS Construction Innovations
3.1.2
ERS Foundation and Leveling Course Preparation
3.1.3
ERS Construction Tool Requirements
3.1.4
ERS Facing Installation Requirements
3.1.5
ERS Soil Reinforcing Installation Requirements
3.1.6
ERS Facing Alignment Requirements
3.1.7
ERS Reinforced Backfill Placement Requirements
3.1.8
ERS Erosion Prevention Requirements
3.1.9
ERS Installer Requirements
DEA Submittal
Mechanically Stabilized Earth
Grid-Strip Soil Reinforcing

[This Page Is Intentionally left Blank]
3.1.10
ERS Retained Backfill Placement Requirements
3.1.11
ERS Construction Manual
INSTALLATION GUIDE

VIST-A-WALL SYSTEM

Segmental Concrete Panel with Grid-Strip Soil Reinforcing
The information set forth in this design methodology, including but not limited to all technical and engineering data, figures, tables, designs, drawings, details, procedures and specifications are intended for information pertaining to this project. Every effort has been made to ensure the 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 any and all liability resulting from such use.
# Table of Contents

1. **LIMITATIONS** ................................................................. 1

2. **OVERVIEW** .................................................................. 2

3. **TERMINOLOGY** .......................................................... 3

4. **REQUIRED TOOLS, EQUIPMENT AND MATERIALS** ....... 5
   4.1 **HAND TOOLS** .......................................................... 5
   4.2 **LIFTING AND UNLOADING EQUIPMENT** ...................... 5
   4.3 **CONSTRUCTION AIDS** .............................................. 6
   4.4 **HEAVY EQUIPMENT** .................................................. 6

5. **SCOPE OF WORK TO BE PERFORMED BY CONTRACTOR** ... 6
   5.1 **SITE PREPARATION** .................................................. 6
   5.2 **WALL LAYOUT** ........................................................ 6
   5.3 **LEVELING COURSE INSTALLATION** ............................ 6
   5.4 **CONSTRUCTION OF VIST-A-WALL** ............................ 7
   5.5 **TOP OF WALL TREATMENT** ........................................ 7

6. **HANDLING VIST-A-WALL COMPONENTS** ..................... 7
   6.1 **DELIVERY OF PRECAST PANELS** .............................. 7
   6.2 **UNLOADING OF PANELS** ......................................... 8
   6.3 **GRID-STRIP™ SOIL REINFORCING** ............................ 9
   6.4 **OTHER COMPONENTS** .............................................. 9
   6.5 **VERIFICATION OF MATERIALS** .................................... 9

7. **CONSTRUCTION** ......................................................... 10
   7.1 **SITE PREPARATION** .................................................. 10
   7.2 **PRECAST PANEL** ................................................... 11
   7.3 **SELECT BACKFILL REQUIREMENTS** .......................... 16
   7.4 **BACKFILL PLACEMENT** ............................................. 17
   7.5 **SOIL REINFORCEMENT PLACEMENT** ......................... 18
   7.6 **JOINT MATERIALS** ................................................... 20
   7.7 **OBSTRUCTIONS** ........................................................ 21
1 LIMITATIONS

Information in this Installation Guide and all documents are not to be used to design, fabricate, manufacture, assemble, construct, produce, install, or otherwise use any elements, forms, or other special equipment (whether patented or not) that is exclusive to Vistawall Systems, LLC., (VAWS) Vist-A-Wall system, for any other purpose other than this project, without the express written consent of VAWS.

The information contained herein shall not be copied, disclosed or distributed in any manner, in whole or in part, to any third party without prior written consent of VAWS.

The Mechanically Stabilized Earth (MSE) structures for this project are designed by VAWS and/or their consultants. The design is based on the following:

• The internal stability of the reinforced soil mass is based on the design assumptions and material properties noted on the drawings and calculations for the related structure, including all external loads, surcharges, and structure geometry that were provided by or on behalf of the Owner.
• The structure geometry including the layout is based on survey information, contract plans, contract drawings and other information provided by or on behalf of the Owner.

• VAWS is responsible only for the internal design of the MSE structures. The external design responsibilities and considerations, including all global slope stability, bearing capacity, settlement, sliding, overturning and drainage under both static and seismic loading, are the responsibility of the Owner. The external design material parameters are outside of the control of VAWS, it is for this reason that we cannot take responsibility for the design thereof.

This manual is part of the VAWS contract documents. It is designed to provide a set of general guidelines and specifications for the Owner, Contractor, and Installer of the VIST-A-WALL system for the Segmental Concrete Panel (SCP) retaining wall system using Grid-Strip™ soil reinforcing. This manual shall be read and followed in conjunction with the contract documents, shop drawings and project specifications. When conflicts between the guidelines contained in this document and the guidelines specified in the contract documents occur, the more stringent guidelines shall take precedence.
2 OVERVIEW

MSE structures are composite systems consisting of concrete, soil reinforcing, and soil. The inclusion of tensile resisting reinforcing in a mass of soil significantly improves the strength of the soil. This unique combination, when designed and installed properly, will create a cost effective integrated retaining structure.

VAWS shall make available a qualified Technical Adviser (TA) to assist the contractor in the beginning of construction and periodic monitoring during the construction process as stipulated in the contract. Both parties, before commencement of the installation must agree to additional technical advisement in writing.
The Contractor shall be responsible for assuring that all required material is on site and is properly stored as outline in this manual. The Contractor is responsible for the proper installation of the structure and shall certify that the structure was constructed to the lines, offsets, and elevations as outlined in the contract documents. All compliance with OSHA and other safety organizations shall be the responsibility of the Installer or Contractor.

3 TERMINOLOGY

**Alignment Pin** - A non-structural dowel that is placed in successive layers of panels in order to maintain proper alignment during erection. This is an optional element.

**BAR** - The proximal end of the Grid-Strip™ consisting of a special shaped steel plate that is used to attach the Grid-Strip™ to the panel anchor.

**Bearing Pad** - A compressible generally rectangular element that is placed on the top edge of the concrete panel. The bearing pad prevents concrete-to-concrete contact and concrete spalling.

**Bottom Panel (P)** – A special height panel that is placed on the concrete leveling course and is the first panel in the wall structure.

**Connection Bolt** - A threaded fastener, with a head, designed to be used in conjunction with a nut and that joins the Grid-Strip™ to the panel anchor.

**Connection Pin** - A special shaped smooth cylindrical fastener that joins the soil reinforcement to a panel anchor. This connection pin can be used in lieu of a bolt if approved by the Engineer of Record (Wall EOR).

**Coping** - A concrete element that is used as the top of wall treatment. The coping can be either pre-cast or cast-in-place.

**Coping Panel** - A special height panel that is placed at the top of the wall prior to placement of the coping unit.

**DPS Anchor** – A special shaped steel forged Dual Plate Shaft (DPS) anchor that is cast into the concrete panel so it protrudes form the back face and that is used to connect the Grid-Strip™ soil reinforcing to the panel.

**DPTS Anchor** – A special shaped steel Dual Plate Tie Strip (DPTS) anchor that is cast into the concrete panel so it protrudes form the back face and that is used to connect the Grid-Strip™ soil reinforcing to the panel.
Face Of Wall – The front face of the panel. This can differ from the control point of the structure.

Facing Element – The facing consists of a pre-cast concrete panel that is structurally connected to the soil reinforcement and prevents the raveling of soil between the layers of soil reinforcement.

Filter Fabric (Cloth) - A needle punched geo-textile fabric that is placed over the horizontal and vertical joints of each concrete panel to prevent the soil from eroding from the joints.

Finish Grade - The material that is placed in front of the MSE wall that protects the bottom of the wall from erosion and undercutting.

Flip-Pin - A special shaped smooth cylindrical fastener that joins the soil reinforcement to the panel anchor.

Geocomposite - A combination of geo-textile and plastic that forms a composite fabric that is used in lieu of the filter fabric for specific applications.


Inclusion - Any man made element that is inserted into the soil mass to improve the structural properties of the soil.

Junction Slab – Reinforced concrete slab that is structurally attached to the coping element. This prevents the coping element from being dislodge from the structure during impact to the coping element. This is sometimes referred to as a moment slab.

Level-Up Concrete - Non-reinforced concrete or grout that is placed on the top panel and leveled in order to give the coping a smooth surface to bear on. This concrete brings the top of wall to the correct elevation and orientation.

Leveling Course – A level non-structural, non-reinforced, concrete element that the first row of panels is placed on.

Mechanically Stabilized Earth (MSE) - Engineering term for the stabilization of earth structures through the use of soil inclusions.

Moment Slab - Reinforced concrete slab that is structurally attached to the coping element. This element prevents the coping element from being dislodge from the structure during impact to the coping element. This is sometimes referred to as a junction slab.

Panel Anchor - An element that is cast directly into the back face of the panel and that soil reinforcement is attached to.
**Prepared Foundation** - Excavated and proofed rolled area that the reinforced mass of soil and leveling course bears on.

**Reinforced Soil** - A composite structure composed of alternating layers of soil and inclusions.

**Retained Fill** - The backfill material that is placed directly behind the reinforced soil mass. Usually consists of normal highway embankment material.

**Select Fill** - The mass of soil that is placed within the reinforced soil.

**Shim** - Wedge shaped or thin element that is used to correct panel orientation and to keep joints horizontal. Shim material can be wood or plastic.

**Splice** – The structural joining of two soil reinforcing elements in order to increase the length of the soil reinforcing.

**Soil Reinforcement** - A manufactured grid element that is placed in the select backfill and that is structurally attached to the back face of the facing element.

**TAB** – The proximal end of the Grid-Strip™ consisting of a special forged steel plate that is used to attach the Grid-Strip™ to the panel anchor.

**Traffic Barrier** - A structural element that retains traffic and directs impact in a desired direction. Typically the traffic barrier is structurally attached to the coping element.

**V-Plate** – slotted steel plate that is used to splice two Grid-Strip™ soil reinforcing elements together in the field.

**Wave Plate** – Composite element that is used to structurally splice two Grid-Strip™ soil reinforcing elements together in the field.

### 4 REQUIRED TOOLS, EQUIPMENT AND MATERIALS

The construction of VIST-A-WALL is a relatively straightforward and repetitive process that does not require specialized labor or equipment.

#### 4.1 HAND TOOLS

- 2-foot and 4-foot long carpenter levels.
- Claw hammers.
- Rubber mallet
- Chalk line and chalk.
- Caulking gun for 29-ounce tubes of adhesive.
- Wrenches for clamps (2 ea.)
- 30-inch or 36-inch crow bars (2 ea.)
- Sledgehammer.
- Hand-operated or power-operated saws.

#### 4.2 LIFTING AND UNLOADING EQUIPMENT

- Panel lifting ring clutch
• Spacing tools
• Non-staining dunnage for storage of precast concrete panels.
• Two 22-foot long web slings for unloading panels.
• Lifting chains

4.3 CONSTRUCTION AIDS
• Wood clamps with coil rods, coil nuts and washers
• Wood braces
• Wood or steel stakes
• Hard wood shims

4.4 HEAVY EQUIPMENT
• Hydraulic crane or boom truck to lift and place precast concrete facing panels. A standard 5x5 panel weight is 2000 pounds. A standard 5x10 panel weight is 4000 pounds (assumes nominal 6” thickness)
• Dump trucks, front-end loaders, scrapers, bulldozers or graders to place backfill.
• Water truck
• Smooth-drum vibratory roller
• Walk-behind vibratory rollers or plate compactor

5 SCOPE OF WORK TO BE PERFORMED BY CONTRACTOR
The following scope of work is a general guideline. If the contract warrants different responsibilities then they shall be followed accordingly.

5.1 SITE PREPARATION
The Contractor is responsible to prepare the site properly. This included all grubbing, excavation, dewatering, foundation stabilization and installation of all drainage systems. The site preparation should be coordinated with the installation of all utilities that encroach on the MSE structure.

5.2 WALL LAYOUT
The Contractor is responsible for the VIST-A-WALL layout. When laying out the wall use the Contract Plans and not the VIST-A-WALL approved shop drawings. The plan views that are provided in the SEW plans are for information only and may not contain all required layout information. Further, the plan view is representative of a unique elevation in the structure and may, or may not, contain all required information.

5.3 LEVELING COURSE INSTALLATION
The Contractor is responsible to install the unreinforced concrete leveling course in accordance with the steps and grades given in the VIST-A-WALL approved shop drawings. The foundation area that the leveling course will bear on is required to
be compacted. The leveling course shall be placed to the required elevation to a tolerance of ±1/8".

5.4 Construction of VIST-A-WALL
The Contractor is responsible for the construction of the Vist-A-Wall system. This includes, but is not limited to, the placement and installation of precast concrete facing panels, installation of joint materials including bearing pads and filter cloth, placement and connection of Grid-Strip™ soil reinforcing, and placement and compaction of the reinforced soil backfill.

5.5 Top of Wall Treatment
The Contractor is responsible for placement of the precast or cast-in-place concrete coping, traffic impact barrier, junction slab, or any other secondary concrete components that are required. In addition, the contractor is responsible to form and place the leveling concrete on the top panel to the lines and grades necessary to place the coping at the correct orientation and elevation.

6 Handling VIST-A-WALL Components
The VIST-A-WALL components are specially designed and fabricated for a specific project and should not be used on any other project without the written consent of the VAWS Engineers. It is essential that the materials, especially precast concrete panels and soil reinforcing Grid-Strips™, are handled correctly. It is the responsibility of the Contractor to ensure that all VIST-A-WALL components are free from any damage that might render them unacceptable for use in the installation process. Because the components are manufactured for a specific project, replacement materials take time to fabricate and deliver. VAWS will not be responsible for any replacement costs, time delays, or lost production that is associated from damage to any components sustained following delivery and acceptance by the Contractor.

6.1 Delivery of Precast Panels
To ensure timely delivery, it is imperative that the Contractor, Installer, VAWS and the VAWS Precaster agree on a panel delivery schedule for the project. While many of the panels have the same overall geometry, there are slight differences with respect to the number of panel anchors, the panel reinforcement, and panel dimensions.

Panels are not available for shipping until they have achieved the minimum concrete compressive strength. It is the responsibility of the Owners’ Inspectors,
or the Precaster to determine when panels are ready for shipping.

Precast concrete panels are shipped to the site on flatbed trailers. Typical truckloads consist of four or five stacks of four or five panels. The average panel area for one truckload is 600 square feet. Panels are stacked finished face down on dunnage. Care must be taken during unloading and moving of panels to avoid damaging the panel finish.

6.2 UNLOADING OF PANELS

Precast panels are typically unloaded at a central location for the entire project and within close proximity to the actual wall site. If panels are unloaded at the wall site, the panels may either be placed directly into the wall or stored temporarily adjacent to the wall. Placing the panels directly into the wall may require additional unloading time and extra costs.

Typical unloading time for a truckload of panels is two hours. Panels are to be stored in stacks on a flat, firm surface using non-staining dunnage. Stacks of panels should be unloaded using either a crane with two-web slings or a forklift with padded forks to protect the panel finish. Any damaged panels on the flatbed trailer should be brought to the attention of the driver and recorded on the delivery ticket.
If the panels are to be placed directly into the wall, then ring clutches must be attached to the lifting embeds in the top edge of the panel to lift and handle individual panels. Both lifting embeds must be used in lifting and handling panels. All dunnage is the property of VAWS or the Precaster and should be made available for pick-up and reuse.

6.3 GRID-STRIP™ SOIL REINFORCING

The Grid-Strip™ soil reinforcing elements for the VIST-A-WALL system comprise discrete steel grids. Each wall will require several lengths of grid. The Grid-Strips™ are delivered in tagged bundles on flatbed trailers. Grid-Strip™ lengths range from 6 feet to 40 feet.

The Grid-Strip™ bundles shall be unloaded using a forklift or slings. For Grid-Strips™ in excess of 20 feet it is recommended that a “spreader bar” be used to avoid damage by excessive deflection. Grid-Strips™ shall not be stored directly on the ground but shall be stored on dunnage.

The Contractor is responsible for the proper storage and handling of the Grid-Strips™ after delivery. The Contractor shall guarantee that each Grid-Strip™ is placed in the proper location in the wall as indicated on the approved shop drawings.

6.4 OTHER COMPONENTS

The bolt sets, flip-pins, filter fabric, adhesive, bearing pads, spacing tools, lifting eyes and other secondary components are normally shipped with the first truckload of panels or Grid-Strips™. It is the responsibility of the Contractor to store these materials away from direct sunlight. Further, all material that is shipped to the site in cardboard boxes shall be kept in an area that keeps them dry to prevent degradation of the container.

6.5 VERIFICATION OF MATERIALS

All material quantities must be verified by the Contractor or Contractor’s agent at the time of delivery and any discrepancies reported to VAWS within 48 hours of delivery. VAWS shall not be responsible
for any costs or delays associated with the failure to report discrepancies in material quantities within 48 hours of delivery.

7 CONSTRUCTION

The construction of a Vist-A-Wall structure is a repetitive process that requires successive layers of compacted soil, concrete panels, and Grid-Strips™. Each constructed layer normally requires the same installation steps that require the same standard material. The elevation of each wall is shown in the Shop Drawings. Below each wall are column numbers. The column numbers can be used to easily identify material.

In order to speed installation it is highly recommended that the Installer be familiar with the location of each of the material components that form the wall structure, and where each component is stored on site. Further, it is recommended that the Installer studies the contract documents fully and completely.

7.1 SITE PREPARATION

The foundation area shall be graded level for a width equal to the length of the soil reinforcement plus six (6) inches. All foundation material that is suspected of being of poor quality shall be removed and replaced. The foundation preparation is the critical part of the wall construction. Taking time to properly prepare the foundation will unquestionably decrease the chances of problems occurring during or after construction. Soil reinforcement shall not be placed until the foundation has been prepared so it is capable of supporting all anticipated loading.
Once the foundation is properly prepared the non-reinforced leveling pad shall be formed and the concrete placed. Excavate and form the leveling course so that the top of the leveling course is at the desired elevation to within a tolerance of ±1/8". The leveling course will have a minimum thickness of 6 inches and a minimum width of 12". For panels placed on curves the width of the leveling course may need to be increased.

**Typical Leveling Course**

Compact the excavated leveling course foundation area before placing the concrete. The leveling pad shall be allowed to cure a minimum of 24 hours before placement of the first row of panels. The leveling pad must be placed in the proper vertical and horizontal position as detailed on the plans. *Experience has shown that improper placement of the leveling pad increases the chance for an unsatisfactory finished product.*

7.2 **Precast Panel**

Vist-A-Wall concrete panels are available in three basic sizes: 5’ x 5’, 5’ x 10’ and full height. Each panel is labeled and detailed on the panel schedule or on the standard panel detail sheet. The panel label provides the panel type and number of panel anchors per row. Note that you can easily identify a panel by the scribed information that is placed by the Pre-Caster on the back of each panel.

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”).

**Staggered Bottom Panel Arrangement**

The top-course of panels is available in a range of heights. Varying height panels create a stepped transition at the top of the wall.
Stepped Top Panel Arrangement

Other specially cut, notched, slope-topped and special width panels are available to meet the specific requirements of the wall geometry.

Notched Panel

Panel designation and casting dates are scribed into the back face of each panel. If required, or stipulated, this information will be marked on one edge of the panel for further identification.

Standard type “G” panels are the most common panel. Typically, these panels will comprise more than 75 percent of the panels in a typical wall. A type “G” square panel has an actual dimension of 5'-0 ½" x 5'-0 ½".

Typical Panel Detail

The panels are placed in the wall with a ¾” spacing on all sides. This arrangement provides a nominal panel dimension of 5’-0” x 5’-0”. The shop drawings provide a column number for each column of panels. All non-standard panel widths include the required joint spacing by design.

⅜” Vertical Joint Spacing
Bottom-of-wall panels are designated with a “P” and the appropriate panel size indicator. The “P” allows for the precaster to use a special block out for the bottom of the panel. A “PB” denotes a bottom “P” standard “B” panel that has the nominal dimensions of 2’-6” x 5’-0”.

Top-of-wall panels are designated with a “T” and the appropriate panel size indicator. The “T” allows for the precaster to use a special block out for the top of the panel and also to insure that “top steel” is added if required. A “TG” denotes a top “T” standard “G” panel that has the nominal dimensions of 5’-0” x 5’-0”.

Sloped top panels are designated by a “T” and a double alpha character such as “TBG”. This is looking at the front face of the panel, with the first alpha following the “T” (top) being equal to the left height (B) and the subsequent alpha being the right height (G). A “TBG” is a top panel with a nominal left height of 2’-6” and a nominal right height of 5’-0”. The “T” is as designated previously.

Corner or vertical slip-joint panels are designated as “X” panels. The nomenclature also includes the equivalent standard panel alpha designation. Special panels are designated “S”, followed by a number.

### Typical Panel Types

<table>
<thead>
<tr>
<th>Panel Type</th>
<th>Nominal Height</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>2’-0”</td>
</tr>
<tr>
<td>B</td>
<td>2’-6”</td>
</tr>
<tr>
<td>C</td>
<td>3’-0”</td>
</tr>
<tr>
<td>D</td>
<td>3’-6”</td>
</tr>
<tr>
<td>E</td>
<td>4’-0”</td>
</tr>
<tr>
<td>F</td>
<td>4’-6”</td>
</tr>
<tr>
<td>G</td>
<td>5’-0”</td>
</tr>
<tr>
<td>H</td>
<td>5’-6”</td>
</tr>
<tr>
<td>J</td>
<td>6’-0”</td>
</tr>
<tr>
<td>K</td>
<td>6’-6”</td>
</tr>
<tr>
<td>L</td>
<td>7’-0”</td>
</tr>
<tr>
<td>M</td>
<td>7’-6”</td>
</tr>
</tbody>
</table>

Note: The alpha character i (i) has been omitted because of the similarity to the numeral one (1)

Cut or non-standard width panels with no edge tongue and groove are designated with an L or R. The letter L refers to a panel that is cut on the left side (viewed from the front face of the panel). Similarly, R denotes a panel cut on the right side (looking at the front face of the panel). These panels are used at corners and in instances where the wall structure intersects a concrete structure.

The Wall Installer shall give notice to the Pre-Caster as to which structure to begin casting. The Wall Installer shall provide a schedule of installation to the Pre-Caster. In addition, the Wall Installer shall provide a load list to the Pre-Caster. The Wall Installer shall notify the Pre-Caster a minimum one week before said
installation shall take place. The Wall Installer shall keep track of all panel shipments and a log of what panels were installed, on which day, and in what structure.

Panels that are of poor quality and that do not meet tolerances shall not be used and shall be rejected and set aside. Inspection of the panel should be performed when the panels arrive to the site and as they are being removed from the truck. The Wall Installer should make note as to the panel number, lot number, date and reason for rejecting the panel. This list shall be given to the Owner and wall supplier.

The first row of panels is placed on the leveling course. If the leveling course is not placed properly, the panel will need to be adjusted to insure that the horizontal and vertical alignment is achieved and maintained during the erection process. This can be achieved with hard-wood shims. It is not necessary to place a bearing pad on the leveling course for the panel to bear on. The panel should be set directly on the leveling course.

Placement of Bottom Row of Panels

It is important 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 will be easier to install.

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

In order to provide proper placement of the first row of panels and to set the correct batter, it is recommended, but not
required, to position 2 wooded wedges on the front and back side of the bottom panel.

The wedges when lodged under the panel will aid in stabilization of the panel, will provide for easy adjustment of the batter, and will aid in leveling the panel to the proper elevation.

**Bottom Panels with Leveling Wedges**

Each panel is identified by an attribute. The number preceding the panel alpha indicates how many DPS or DPTS anchors are cast in the panel. A “G22” is a type a “G” panel with 2-DPS anchors in the top row and 2-DPS anchors in the bottom row.

The overall installation speed of any MSE structure is completely a function of the delivery and compaction of the backfill material. It should not be assumed that the larger the precast concrete facing panel the faster the overall rate of wall construction will be.

MSE structures are very flexible. This flexibility enables the entire structure to accommodate significant total and differential settlements without any damage to the facing itself. It should be understood that as the panel size increases the total amount of differential settlement the structure can tolerate decreases. Further, it should be understood that the ¾” joint that is innate to the system is extremely important and needs to be maintained. By diminishing the ¾” joint spacing the amount of differential settlement the MSE structure can tolerate decreases.

The panels that are delivered to the site are marked with the appropriate designation and must be checked against the approved shop drawings to ensure that panels are placed in the correct location in the field. It is the responsibility of the Owner’s inspection personnel to ensure that all panels meet the necessary quality control requirements with respect
to concrete strength, tolerances on overall dimensions, damaged or missing connectors, cracked or broken concrete, etc., before being placed in the wall.

7.3 SELECT BACKFILL REQUIREMENTS

The backfill is the most critical component of the structure. It has been demonstrated that to assure maximum strength, minimum compressibility, and the ability to compact to a high density, that the backfill should consist of well-graded granular material. Well graded material typically possesses electrochemical properties that meet the AASHTO corrosion model. A well graded material also has excellent drainage characteristics and the post-construction movements are rare.

MSE structures can be constructed using fine-grained soils. It should be noted that the installation process takes considerably more time and more observant control by the Wall Installer. Further, fine grain soils are normally poorly drained and the stress transfer between the soil and the soil reinforcement is not immediate. Because of this, the panel may require a slight batter when set. The batter will slowly be removed as the fine grained soils are compacted. There is not a rule on how much the panel should be battered; it must be done by trial and error. A good starting point would be so the panel will be within tolerance if the panel does not move. It should also be noted that fine-grained soils also exhibit a time-dependent comportment that can cause post construction movement.

A select backfill material should possess a wide range of grain sizes with a limit of no more than 15 percent passing the No. 200 sieve. The select backfill material should not be gap-graded. Gap-graded material will make compaction more difficult. Gap-graded material will have voids that will allow for soil loss to occur when a hydraulic gradient or dynamic load is introduced into the backfill. This will cause the backfill to settle and will cause post-construction movements of the system.

The MSE backfill shall meet the project specifications. It is the Contractors responsibility to verify that the MSE backfill gradation, shear strength, permeability, and electrochemical properties, including resistivity, pH, organic, chloride, and sulfate contents are in accordance with the specifications. VAWS is not responsible for approving backfill sources or backfill materials. The final acceptance to all backfill material is
the Owner’s responsibility.

7.4 BACKFILL PLACEMENT

The placement of the backfill 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”-12” compacted layers. The backfill can be placed in larger lifts if approved by the Owner or Owners representative and if the Wall Installer can demonstrate that the proper compaction is achieved. *The fill shall be leveled by equipment moving parallel to the wall face. The material shall be spread so it is fanned toward the tail of the soil reinforcing.* The placement of the backfill from the front of the soil reinforcement to the tail of the soil reinforcement will keep the soil reinforcement fixed.

Compaction of the backfill shall be performed with an 8 ton to 10 ton 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 end of the reinforcement.* Proper moisture content of the backfill material should be maintained uniformly within each layer. The material should 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 with the use of a vibratory roller or plate compactor that can deliver a minimum of 1000 (lbf) centrifugal force and not to exceed 4000 (lbf) centrifugal force. 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 tail of the soil reinforcing.*

Compaction of 3-Foot Zone

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 ft² 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. Proper compaction will alleviate possible problems in the future performance of the structure. Improper compaction can cause outward movement of the panels. Compaction test should not be performed in the area directly adjacent to the back face of the panel.

Compaction Test

7.5 Soil Reinforcement Placement

The Grid-Strip™ is attached to the DPS and/or DPTS panel anchor with a bolt set. A washer is not required. The washer is not required because this is not a structural joint. The bolt can be considered a pin, and the nut the method used to keep the pin in place. In addition to using a bolt, the Grid-Strip™ can be connected with a flip-pin.

Grid-Strip™ to Panel Bolt Connection

The number of anchors and the grouping of the anchors are dependent on the project calculations and the location in the wall structure. Typically, the panel anchors are cast symmetric about the panel.

Grid-Strip™ to Panel Connection

The shop drawings panel attribute defines the number of Grid-Strip™ soil reinforcement that is to be used in each
Typical Panel Attribute

The length of the soil reinforcing is given under the dimension line in the elevation drawing and is identified by the designator, “B = “ or “L = “. It is important that the Wall Installer place and connect the correct number and correct length of Grid-Strip™ on the correct panel and location in the wall.

Grid-Strip™ to Panel Connection Detail

The Grid-Strip™ is placed on the compacted select backfill and the connection is made to the DPS and/or DPTS panel anchor. The Grid-Strip™ is connected to the panel by securing the TAB and/or BAR connector between the two plates of the panel anchor. The bolt only needs to be finger tight. It does not require that a wrench be used. The threads should just start to protrude from the nut.

Grid-Strip™ to Panel Connection

Never connect a Grid-Strip with the backfill lower than the elevation of the panel anchor. The backfill should ideally be slightly higher than the panel anchor. In order to place the bolt or flip pin, it may be necessary to remove some backfill from under the DPS anchor.

Optional Flip-Pin

The flip-pin is designed to be placed from the bottom of the DPS anchor and then rotated toward the back of the panel so the pin “flips” over the shaft. This will
prevent the pin from being accidently removed.

The Wall Installer should verify that all nuts have been installed with the bolt. Equally important, the Wall Installer shall verify that flip-pins are properly engaged with the DPS anchor shaft. Any panel that has anchors missing, or that are severely bent, or that are damaged, shall not be used, but shall be set aside and either repaired or rejected (Reference VAWS Maintenance and Repair Manual).

### 7.6 JOINT MATERIALS

The horizontal panel joints are maintained by placement of bearing pads. The bearing pads shall be placed adjacent to the alignment pins (where applicable) or at the approximate quarter span location of each panel. A minimum of two (2) bearing pads are required for each panel regardless of panel width.

![Standard Bearing Pad Placement](image1)

**Alignment Pin Placement**

The filter cloth is supplied in 12 inch wide rolls and is to be centered along each and every panel joint. It is not necessary, but may be a project specific requirement, to place filter fabric along the bottom edge of the bottom panel at the interface of the leveling course.

![Filter Fabric Joint Material](image2)

**Filter Fabric Joint Material**

An acceptable length of filter fabric shall be lapped at each joint intersection. The filter fabric shall be designed and selected in accordance with AASHTO M288.

The filter fabric is held into place by the supplied adhesive. The durability of the
adhesive is not important since its only purpose is to temporarily hold the filter cloth in place during the backfilling operation. Once the backfill is placed and compacted, soil pressure will hold the fabric in place. The adhesive is not intended to provide a water tight seal. The purpose of the filter fabric is to prevent the migration of fine material from the joints during the life of the structure and at the same time allow water to escape. The amount of adhesive that is used is equal to only the amount to hold the fabric. It may be placed only at the corners. If the Wall Installer decides to use a continuous bead along the panel edges the Wall Installer shall supply additional adhesive. Please reference the Application of Adhesive for Filter Fabric document that is at the end of this document.

To recap, in certain instances the filter fabric may be required to be placed at the interface of the leveling pad and the first course of panels. Reference should be made to the governing specification for guidance.

7.7 Obstructions

During the design phase an attempt is made to detail special panels, and connections at the locations of vertical and horizontal obstructions.

Pile Obstructions in MSE Back Fill

In some instances it may be necessary to shift the Grid-Strip along the length of the panel so it is grouped near one end. This may be detailed on the panel elevation and in the plan view. The Grid-Strip™ is an articulating connection that can easily be rotated to by-pass most obstruction.

The cutting of the Grid-Strip™ is strictly prohibited. It may be necessary to rotate the Grid-Strip so it is skewed. The angle of rotation shall be no more than 15°. The
use of angles greater than 15° can be used if approved by the Retaining Wall Engineer of Record.

Horizontal obstructions can be passed by gradually skewing the soil reinforcement above or below the obstruction. If the soil reinforcement is skewed horizontally, care should be taken so as not to kink the reinforcement. The deflected angle shall be less than 15°. The use of angles greater than 15° can be used if approved by the Retaining Wall Engineer of Record. Place a 4”-6” buffer of soil over the obstruction before the placement of the soil reinforcement.

RCP Obstruction

7.8 DRAINAGE

It is extremely important not to allow the reinforced volume to become saturated at any time during construction. At the end of each day’s operation proper precautions shall be taken in order to assure that the MSE mass does not become saturated. The Wall Installer shall slope the reinforced volume of soil away from the facing elements at the end of each days operation. Saturation of the MSE mass can result in destabilizing forces that cause the structure to fail. Heavy rainfall can cause erosion of the soil from within the layers of the MSE mass especially at the end of the constructed wall segment. If any erosion does occur, or if the backfill becomes saturated, it shall be replaced with non-saturated material that conforms to the backfill specifications. Care shall be taken during periods of heavy rain to assure proper drainage and to provide positive flow away from the facing.

Consequences of Heavy Rain Washout

After heavy rains and in saturated sandy soils, it is advisable to exercise great care in the placement and operation of construction equipment near the face of the wall. The use of construction
equipment must be kept away from the back of the panel until such time the sandy soils have dried out.

7.9 **FINISH GRADE PLACEMENT**

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.

**Finish Grade at Face of MSE Structure**

Although the fill in front of the wall is not considered in the stability calculations it should be understood that any excavation in front of the wall at depths below the leveling course could greatly affect the structural stability and integrity of the reinforced volume. No excavation below the leveling course shall be allowed until written permission is received from the Engineer of Record and the wall supplier.

7.10 **COPING PLACEMENT**

The placement of the coping requires that the top of wall be at the proper elevation and orientation as shown in the contract plans. Leveling concrete may be required when precast coping is specified. The leveling concrete is placed in the area between the top panel and the precast coping seat. The leveling concrete is placed in a wooden form that is attached to the panels. The required elevation is marked on the inside of the form and the leveling concrete is placed in the form and leveled to the marked line or top of the form boards.

**Leveling Concrete - Forming System**

Depending on the coping that is used the details may call for dowels to be placed in the top edge of the top panel. The dowels
are used to tie the coping panel and leveling concrete together. Further it may be required that a cushion layer be placed on the top of the panel, or leveling concrete, and at the back face of the panel and coping leg. This cushion layer helps to lessen the stress transfer of the impact load into the top panel.

Coping Unit on Leveling Concrete

Because the coping is usually placed at a slope on the top panel it is very difficult to pre-cast any type of insert into the panel or coping that assures that they will line up in the field. Experience has shown that the best solution for placing dowels in a sloped coping panel is to field drill them after the panels are set and epoxy the dowels in place.

It is possible to cast inserts into coping that are to be placed parallel to the top panel. The desired number of inserts is cast into the coping and into the panel and a dowel inserted in the field. If dowels are precast into a sloping panel then they may be required to be manually bent so they are perpendicular to the bottom of the coping.

It should be noted that the dowel is only helping prevent the coping from being pushed off of the wall by construction equipment and that should not be relied on to transfer impact forces.

Dowel at Top Panel
8 TYPICAL WALL COMPONENTS

1 TYPICAL PRE-CAST PANEL WITH ANCHORS (2 ROWS AND 2 COLUMNS OF ANCHORS)
2 2-2” X 6” X 3/4” NEOPRENE BEARING PAD (2 PER PANEL @ ALIGNMENT PIN)
3 2-1/2” X 8” GALVANIZED STEEL ALIGNMENT PINS (2 PER PANEL – If applicable plastic pins may be substituted for galvanized. This element is optional)
4 12” WIDE FILTER FABRIC ADHERE WITH CONSTRUCTION ADHESIVE (ALL JOINTS)
6 CONNECTION BOLT OR FLIP-PIN (ONE PER GRID-STRIP™ SOIL REINFORCING)
7 GRID-STRIP™ SOIL REINFORCING (AS REQUIRED)
9 INSTALLATION SEQUENCE

9.1 STEP ONE

**STEP 1.** Excavate and prepare foundation

**STEP 2.** Form and place leveling course to grades, lines and widths as shown on the project shop drawings.

**STEP 3.** Place first row of panels on leveling course.

**STEP 4.** Plumb panels with wedges and brace as required.
9.2 STEP TWO

STEP 1. Place filter fabric on back face of panel at all vertical and horizontal joints and secure to panel with adhesive.

STEP 2. Place and compact backfill on prepared foundation to the level of the first row of panel anchors.
9.3 STEP THREE

**STEP 1.** Place Grid-Strip™ soil reinforcing on compacted backfill making sure that it is the correct length.

**STEP 2.** Connect the TAB of the Grid-Strip™ to the DPS panel anchor with the bolt set. The nut should be up. Finger tight nut so thread emerges from top.
9.4 STEP FOUR

STEP 1. Place soil on tail of soil reinforcing to prevent the soil reinforcing from moving.

STEP 2. Place selected backfill in the void at the back face of the panel and compact.

STEP 3. Place select backfill to level of top of first row of half panel and compact. Make sure to place alignment pin in top edge of panel before placing backfill to prevent alignment pin holes form becoming filled with backfill material.

STEP 4. Remove clamps and blocking from first row of half panels.
STEP 1. Place the next row of alternating panels over the alignment pins and clamp to adjacent panel to prevent movement.

STEP 2. Place next layer of selected backfill and compact.

STEP 3. Remove external bracing and place and compact finish grade at face of wall.

STEP 4. Repeat installation Steps Two through Step 5 until top row of panels are placed.
9.6 STEP SIX

**STEP 1.** Place top row of panels.

**STEP 2.** Place layer of selected backfill compact.

**STEP 3.** Set forms and place leveling concrete to bring top of panel to required elevation and grade.
9.7 STEP SEVEN

STEP 1. Place backfill to level of bottom of coping or bottom of moment slab.

STEP 2. Form and place coping unit.

STEP 3. Form and place moment slab.

STEP 4. Form and place traffic barrier.

STEP 5. Note: If a one piece coping unit and traffic barrier are being used omit step 4.
9.8 FILTER FABRIC PLACEMENT

**STEP 1.** Place 12” wide filter fabric centerline of all horizontal and vertical joints.

**STEP 2.** Adhere filter fabric with adhesive.

**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 interface of the bottom panel and leveling pad. Reference the Owners specification for guidance on this matter.
9.9 PANEL LIFTING

Lift panels with approved lifting device pursuant to Burke Spread Anchor - One Ton and Two Ton specifications.
10 MSE CHECK LIST

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 and tightening of bolts and or placement of flip-pins being used?
14. Yes No Is the filter fabric 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 tail 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? When applicable a minimum 90% of maximum density for first three foot area?

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 runoff and saturation?
11 APPLICATION OF ADHESIVE TO FILTER FABRIC

The filter fabric is attached to back face of the segmental concrete panel using a construction adhesive. The adhesive is used to temporarily hold the fabric in place until the compacted backfill is placed at the back of the panel. The adhesive is a method to temporarily adhere the filter fabric to the back face of the panel and to keep the outside edges from curling or bunching up. The adhesive is not applied in order to form a watertight seal and therefore does not have to be a continuous bead. Once the backfill is placed and compacted, the horizontal soil pressure will hold the fabric in place.

The adhesive supplied by VAWS is supplied in tubes that contain 29 fluid ounces. The adhesive is applied using a caulking gun and putty knife. The adhesive should be placed using a 2” x 12” stitch pattern on the back of the panel or the filter fabric. The tip of the adhesive tube should be cut to produce a ½” diameter bead. A bead that is ½” x 2” has a volume of 0.4 cubic inches. Using basic conversion factors, it can be calculated that there are 1.8 cubic inches in one fluid ounce of product. Therefore, a 29 fluid ounce tube of adhesive can supply approximately 130 - ½” x 2” beads of adhesive. An alternate method of placement of the adhesive consisting of 2” diameter blobs applied with the putty knife maybe used and applied only at the corners.

<table>
<thead>
<tr>
<th>REQUIRED TUBES PER ELEMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Element</strong></td>
</tr>
<tr>
<td>----------------</td>
</tr>
<tr>
<td>5 x 5 Panel</td>
</tr>
<tr>
<td>5 x 10 Panel</td>
</tr>
<tr>
<td><em>Roll filter fabric</em></td>
</tr>
</tbody>
</table>

*Assumes 360 linear feet of filter fabric per roll and two rows of 1/2” x 2’” beads
It is the Wall Installer’s responsibility to apply the adhesive in a manner to not exceed the recommended application guidelines set forth in this document. The Wall Installer shall be responsible for purchasing additional adhesive than the amount supplied by VAWS based on these guidelines.
IDEA Submittal
Mechanically Stabilized Earth
Grid-Strip Soil Reinforcing

[This Page Is Intentionally left Blank]
4.0
QUALITY CONTROL
4.1 Manufacturing
4.1.1
Facing Unit QA/QC
The information set forth in this design methodology, including but not limited to all technical and engineering data, figures, tables, designs, drawings, details, procedures and specifications are intended for information pertaining to this project. Every effort has been made to ensure the 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 any and all liability resulting from such use.
# TABLE OF CONTENTS

1. **GENERAL** ................................................................................................................................................ 1

2. **REFERENCES** ........................................................................................................................................... 1
   2.1 **AMERICAN SOCIETY FOR TESTING AND MATERIALS (ASTM)** ................................................................. 1
   2.2 **AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICILAS** ........................................... 1
   2.3 **CRSI - MANUAL OF STANDARD PRACTICE** .................................................................................................... 1
   2.4 **WIRE REINFORCEMENT INSTITUTE - MANUAL OF STANDARD PRACTICE** .................................................... 1
   2.5 **ACI 318-08 - BUILDING CODES FOR STRUCTURAL CONCRETE** ........................................................................ 1

3. **CEMENT** ................................................................................................................................................. 1

4. **TESTING AND INSPECTION** ...................................................................................................................... 2

5. **CASTING** ................................................................................................................................................. 2

6. **CURING** .................................................................................................................................................. 2
   6.1 **HOT WEATHER OR INDOOR** ......................................................................................................................... 2
   6.2 **COLD WEATHER** ............................................................................................................................................ 2

7. **LIFTING DEVICES** ..................................................................................................................................... 2

8. **CONCRETE FINISH** ................................................................................................................................... 2

9. **TOLERANCES** ........................................................................................................................................... 3
   9.1 **PANEL DIMENSIONS** ................................................................................................................................... 3
   9.2 **PANEL SQUARENESS** .................................................................................................................................. 3
   9.3 **PANEL SMOOTHNESS** .................................................................................................................................. 3

10. **CONCRETE COMPRESSIVE STRENGTH** .................................................................................................... 3
   10.1 **ACCEPTANCE** ............................................................................................................................................ 3
   10.2 **SAMPLING** ................................................................................................................................................. 3

11. **REJECTION** ............................................................................................................................................. 4
   11.1 **MOLDING** .................................................................................................................................................. 4
# IDEA Submittal
Mechanically Stabilized Earth
Grid-Strip Soil Reinforcing

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.2</td>
<td>TEXTURE</td>
<td>4</td>
</tr>
<tr>
<td>11.3</td>
<td>PHYSICAL CHARACTERISTICS</td>
<td>4</td>
</tr>
<tr>
<td>11.4</td>
<td>REPAIR</td>
<td>4</td>
</tr>
<tr>
<td>11.5</td>
<td>MARKING</td>
<td>4</td>
</tr>
<tr>
<td>12</td>
<td>PANEL ACCESSORIES</td>
<td>4</td>
</tr>
<tr>
<td>12.1</td>
<td>PANEL ANCHORS</td>
<td>4</td>
</tr>
<tr>
<td>12.2</td>
<td>STRUCTURAL MEMBERS</td>
<td>4</td>
</tr>
<tr>
<td>12.3</td>
<td>FASTENERS</td>
<td>4</td>
</tr>
<tr>
<td>12.4</td>
<td>PANEL REINFORCEMENT</td>
<td>4</td>
</tr>
<tr>
<td>12.5</td>
<td>GALVANIZING</td>
<td>4</td>
</tr>
<tr>
<td>12.6</td>
<td>EPOXY COATING</td>
<td>5</td>
</tr>
</tbody>
</table>

## APPENDIX A
QUALITY CONTROL PROGRAM FOR PRECAST PANELS
SEGMENTAL CONCRETE PANEL SPECIFICATION

1 GENERAL

This specification pertains to the casting of the VAWS Segmental Concrete panel Stabilized Earth Wall 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 A82 - Standard Specifications for Steel Wire, Plain, for Concrete Reinforcement
2.1.3 A123 - Standard Specifications for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products
2.1.4 A185 - Standard Specifications for Steel Welded Wire Reinforcement, Plain, for Concrete
2.1.5 A325 - Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength
2.1.6 A496 - Standard Specifications for Steel Wire Reinforcement, Deformed, for Concrete
2.1.7 A497 - Standard Specifications for Welded Wire Reinforcement, Deformed, for Concrete
2.1.8 A525 - Specification for General Requirements for Steel Sheet, Zinc-Coated (Galvanized) by the Hot-Dip
2.1.9 A510 - Standard Specification for General Requirements for Wire Rods and Coarse Round Wire, Carbon Steel
2.1.10 A615 - Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement
2.1.11 A780 - Standard Specification for the Repair of Damaged and Uncoated Areas of Hot-Dip Galvanized Coatings
2.1.12 A884 - Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Reinforcement

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 that contain chloride shall not be used without approval of the Owner.

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 that are placed a minimum of 6” off of 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 VAWS, 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 in 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 verticals do not exceed ½” (13 mm).

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 days production whichever is less.

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

12.4.1 Welded Wire Reinforcing

All welded wire mesh panel reinforcement shall be in accordance with ASTM A82 - Standard Specification for Steel Wire, Plain, for Concrete Reinforcement and ASTM A185 - Standard Specification for Steel Welded Wire Fabric, Plain, for Concrete Reinforcement

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

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 Vistawall Systems, LLC., 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 VAWS Stabilized Earth Wall (SEW) system, or for any other purpose other than this project without the express written consent of VAWS. 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 VAWS. The Quality Control procedures outlined in this manual have been developed to aid in the design, manufacture, supply of materials, and installation of the VAWS SEW 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 VAWS are unique to the SEW system. They have been designed to aid the Pre-Caster in rapid set up and stripping of the SEW 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 fabric mesh panel reinforcing in a manner 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 to 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 day’s concrete shall be thoroughly checked (using a standardized checklist) to ensure 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**

The concrete will be transported by either Vistawall Systems or by the Owner’s certified mixer trucks 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 ensure 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 have 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 ensure 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 exposed back surface of the precast panel will be floated to remove any remaining high or low surfaces. Prior to concluding the final surface preparations, a final review of the location, alignment and condition of the attachment devices shall occur and as required minor adjustments may be necessary to assure 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 insure 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 that is located within the short-term cure area, so the product can receive the necessary attention. Prior to 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 in accordance with 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 in accordance with the Owner’s concrete specification. After the panel satisfies the quality inspection, which includes 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 a 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 1.** It is the sole intent of Vistawall Systems 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 2.** 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 3.** All sampling will be obtained at the discharge destination point or at the end of the chute.

**STEP 4.** The sampling and test methods shall be in accordance with the methods as outlined in the Owner’s concrete specification.

**STEP 5.** 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 6. 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 7. 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 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 direct guidance of the QC Manager or the QC Control representative. Once the repairs have been completed the products shall be inspected to ensure that the product meets or exceeds the quality requirements. After the product has been repaired and the repair accepted, it shall be marked with green paint adjacent to the yellow paint to signify it is of 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 encountered, 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 of the end of the dunnage that is in contact with the front face of the panel.
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**

Is the local breaking of corners or edges of the concrete with the resulting 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.
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 rejected/culled area of the facility until permanent removal can occur.
4.1.2
Soil Reinforcing QA/QC
MECHANICALLY STABILIZED EARTH STRUCTURES

Welded Wire Specification

Stabilized Earth Wall System

Big-R Bridge - Vistawall
650 Justice Lane
Mansfield, TX 76063
Phone 817.507.0200 • Fax 817.507.0197

Copyright © Big-R
ALL RIGHTS RESERVED 2017
PROPRIETARY INFORMATION OF BIG-R BRIDGE - VISTAWALL
NO DISTRIBUTION IS ALLOWED WITHOUT WRITTEN PERMISSION FROM
BIG-R BRIDGE

The information set forth in this Manual, including but not limited to all technical and engineering data, figures, tables, design drawings, details, procedures, and technical information are intended for information pertaining to this project. Every effort has been made to ensure the Manual accuracy. This information should not be used or relied upon 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 therefrom.
# Table of Contents

1. **GENERAL** ........................................................................................................................................................... 1

2. **REFERENCES** ...................................................................................................................................................... 1
   2.1 AMERICAN SOCIETY FOR TESTING AND MATERIALS (ASTM) ................................................................. 1
   2.2 CRSI - MANUAL OF STANDARD PRACTICE ................................................................................................. 1
   2.3 WIRE REINFORCEMENT INSTITUTE - MANUAL OF STANDARD PRACTICE ............................................... 1

3. **DEFINITIONS** ..................................................................................................................................................... 1
   3.1 WELDED WIRE REINFORCEMENT .................................................................................................................. 1
   3.2 WIRE SIZE............................................................................................................................................... 1
   3.3 WIRE SPACING......................................................................................................................................... 1
   3.4 SHEET WIDTH.......................................................................................................................................... 1
   3.5 SHEET LENGTH.......................................................................................................................................... 1
   3.6 SIDE OVERHANG ...................................................................................................................................... 2
   3.7 OVERALL WIDTH ...................................................................................................................................... 2
   3.8 END OVERHANGS ..................................................................................................................................... 2

4. **QUALITY ASSURANCE** ........................................................................................................................................ 2
   4.1 QUALITY ASSURANCE ................................................................................................................................ 2
   4.2 WELDED WIRE REINFORCEMENT TOLERANCES ........................................................................................... 2
   4.3 ADDITIONAL TEST METHODS ........................................................................................................................... 2
   4.4 NONCOMPLIANCE ..................................................................................................................................... 2

5. **STORAGE, HANDLING, AND DELIVERY** .......................................................................................................... 3
   5.1 STORAGE ................................................................................................................................................ 3
   5.2 DELIVERY ................................................................................................................................................ 3
   5.3 LIFTING OF BUNDLES WITH CRANE ............................................................................................................ 3
   5.4 LIFTING WITH FORKLIFT ............................................................................................................................. 3
   5.5 STORAGE ................................................................................................................................................ 3

6. **WELDED WIRE REINFORCING MATERIAL PARAMETERS** .............................................................................. 3
   6.1 PLAIN WELDED WIRE REINFORCEMENT ......................................................................................................... 3
   6.2 DEFORMED WELDED WIRE REINFORCEMENT .............................................................................................. 4

7. **OTHER REINFORCEMENT** .................................................................................................................................. 4
   7.1 PANEL ANCHORS ...................................................................................................................................... 4
   7.2 CONNECTION PINS .................................................................................................................................... 4
   7.3 STEEL REINFORCING BARS .......................................................................................................................... 4
   7.4 DOWEL BARS: ......................................................................................................................................... 4
## TIE WIRE

8 STEEL COATINGS

8.1 GALVANIZING

8.2 EPOXY

9 FIELD CUTTING OF WELDED WIRE

9.1 COATED WELDED WIRE REINFORCEMENT

9.2 CUTTING

9.3 FLAME CUTTING

9.4 REPAIR

10 GALVANIZED COATING REPAIR

10.1 DAMAGED MATERIAL

10.2 CUT MATERIAL

10.3 ZINC COATING

11 EPOXY COATING REPAIR

11.1 DAMAGED MATERIAL

11.2 CUT MATERIAL

11.3 DAMAGED SURFACE AREA

11.4 CURING
WELDED WIRE REINFORCING MATERIAL SPECIFICATION

1 General
Work includes the furnishing of welded wire reinforcement and miscellaneous welded wire reinforcement accessories.

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 A525 - Specification for General Requirements for Steel Sheet, Zinc-Coated (Galvanized) by the Hot-Dip Process
2.1.4 A615 - Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement
2.1.5 A641 - Standard Specification for Zinc-Coated (Galvanized) Carbon Steel Wire
2.1.6 A780 - Standard Specification for the Repair of Damaged and Uncoated Areas of Hot-Dip Galvanized Coatings
2.1.7 A884 - Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Reinforcement
2.1.8 A1064 - Standard Specification for Welded Wire Reinforcement, Plain and Deformed for Concrete

2.2 CRSI - Manual of Standard Practice
2.3 Wire Reinforcement Institute- Manual of Standard Practice

3 Definitions
3.1 Welded Wire Reinforcement
Welded Wire Reinforcement (WWR) designates a material composed of cold-worked steel wire, fabricated into sheets by the process of electric resistance welding.

3.2 Wire Size
The individual wire size designations are based on the cross-sectional area of a given wire. The "W" and "D" number represents the cross-sectional area of the wire multiplied by 100. The "W" represents a plain wire and the "D" represents a deformed wire. A D10 wire would indicate a deformed wire with a cross sectional area of 0.10 square inches.

3.3 Wire Spacing
The wire spacing shall be measured from the centerline-to-centerline distance between parallel wires. This shall apply to both the longitudinal wire and the transverse wire, also known as the cross wire.

3.4 Sheet Width
The sheet width shall be measured from the center-to-center distance between outside longitudinal wires. This dimension does not include side overhangs.

3.5 Sheet Length
The sheet length shall be measured from the tip-to-tip dimension of the longitudinal wires.
(the length dimension always includes end overhangs).

3.6 Side Overhang
The side overhang includes the extension of transverse wires beyond centerline of outside longitudinal wires (side overhangs are not included in the sheet width dimension). Sometimes referred to as "tangs"

3.7 Overall Width
The overall width shall be measured from the tip-to-tip dimension of transverse wires (cross wires). This dimension is the sheet width plus both side overhangs.

3.8 End Overhangs
The end overhang includes the extension of longitudinal wires beyond centerline of first and last traverse wires. The end overhangs are included in the sheet length dimension.

4 Quality Assurance

4.1 Quality Assurance
All steel and welded mesh shall conform to governing ASTM specifications.

4.2 Welded Wire Reinforcement Tolerances

4.2.1 Sheet Width
The permissible variation shall not exceed ± 1/2-inch, center-to-center distance between outside longitudinal wires.

4.2.2 Overall Width
The permissible variation shall not exceed ± 1 inch of the overall width (tip-to-tip length of transverse wire).

4.2.3 Sheet Length
The overall length may vary by ± 1 inch or 1% whichever is greater.

4.2.4 Side Overhang
The permissible variation shall not exceed ± 1/2 inch.

4.2.5 Deformed Wire Weight
The weight of any deformed wire shall be within is ± 6% of a comparable smooth wire weight of the same parameters.

4.2.6 Plain Wire Diameter
The allowable variation in diameter of plain wire is as follows:

<table>
<thead>
<tr>
<th>Wire Size</th>
<th>Diam. Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smaller than W5.0</td>
<td>± 0.003 inch</td>
</tr>
<tr>
<td>W5.0 to W12.0</td>
<td>± 0.004 inch</td>
</tr>
<tr>
<td>W12.1 to W20.0</td>
<td>± 0.006 inch</td>
</tr>
<tr>
<td>Over W20</td>
<td>± 0.008 inch</td>
</tr>
</tbody>
</table>

4.3 Additional Test methods
The Owner can conduct additional tests on the welded wire reinforcement to assure compliance with these specifications.

4.4 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 welded wire reinforcement shall be stored in bundles, tagged, and marked in accordance with the Vistawall Systems Shop Drawings.

5.2 Delivery
All welded wire shall be delivered to the project in an undamaged condition. All deliveries shall be coordinated with the VAWS Operations Department. Delivery of material shall be in truck load quantities unless stipulated by VAWS.

5.3 Lifting of Bundles with Crane
Lift welded wire reinforcement using the lifting eyes that are placed in the material. Lifting eyes are lengths of wire passing completely through to the underside of the bundle and brought back up to the top and twisted around 3 to 4 times to form an eye. Lifting eyes shall be in the bundle to limit deflecting and bending in the center of the bundle. Typically, there are four lifting eyes per bundle located along the outside edge of the sheet. A spreader bar maybe required when lifting sheets more than 25 feet.

5.4 Lifting with Forklift
Loading and unloading material with a forklift requires the welded wire reinforcement to be supported by dunnage. Make sure that forks do not damage the wire and that the forks are passed into the bundle at a location where proper duannage has raised the material.

5.5 Storage
Store welded wire reinforcement in a protected area to limit the potential for surface deterioration caused by prolonged exposure to conditions that accelerate the oxidation of steel. Store all material in area to prevent the accumulation of dirt, and soil. Protect reinforcement, ties, and metal accessories form permanent distortion and store them off the ground using appropriate dunnage.

6 Welded Wire Reinforcing Material Parameters

6.1 Plain Welded Wire Reinforcement
All plain, smooth, welded wire mesh shall be fabricated in conformance to ASTM A1064.

6.1.1 WWR Yield Strength
Provide minimum yield strength of 65,000 psi (515 MPa).

6.1.2 Domestic Manufacturing
When specified to do so all welded wire reinforcement shall be manufactured from domestic steel conforming to ASTM A1064. No foreign steel or foreign billets used in manufacturing process will be permitted.

6.1.3 Foreign Manufacturing
When allowed by project all welded wire reinforcement that is manufactured from foreign steel shall conform to ASTM A1064.

6.1.4 Wire Spacing and Size:
Provide wire spacing and size, as calculated to maintain the specified area of steel as indicated on the contract drawings.

6.1.5 Finished Mesh
Welded Wire Reinforcement shall be furnished in flat sheets or fabricated into bent sheets as indicated in the VAWS shop.
drawings.

6.2 Deformed Welded Wire Reinforcement
All deformed welded wire mesh shall be fabricated in conformance to ASTM A1064

6.1.6 WWR Yield Strength:
Provide minimum yield strength of 80,000 psi (550 MPa).

6.1.7 Domestic Manufacturing
When specified to do so all welded wire reinforcement shall be manufactured from domestic steel conforming to ASTM A1064. No foreign steel or foreign billets used in manufacturing process will be permitted.

6.1.8 Foreign Manufacturing
When allowed by project all welded wire reinforcement that is manufactured from foreign steel shall conform to ASTM A1064.

6.1.9 Wire Spacing and Size:
Provide wire spacing and size, as calculated to maintain the specified area of steel as indicated on the contract drawings.

6.1.10 Finished Mesh
Welded Wire Reinforcement shall be furnished in flat sheets or fabricated into bent sheets as indicated in the VAWS shop drawings.

7 Other Reinforcement

7.1 Panel Anchors
All panel anchors shall be fabricated from steel rod conforming to ASTM A1064. Panel anchors shall be bent into the final configuration using CNC bending apparatus. No panel anchor shall be bent after galvanizing.

7.2 Connection Pins
All connection pins shall be fabricated from steel rod conforming to ASTM A1064

7.3 Steel Reinforcing Bars
Reinforcing bars shall consist of deformed bars meeting the requirements of ASTM A615, Grade 60.

7.4 Dowel Bars:
Dowel bars shall be round steel bars meeting the requirements of ASTM A36.

7.5 Tie Wire
Shall be 16-gauge, black soft-annealed wire conforming to ASTM A641

8 Steel Coatings

8.1 Galvanizing
All galvanizing for welded wire reinforcement shall be in accordance with ASTM A525 or ASTM A123.

8.2 Epoxy
All epoxy coatings for welded wire reinforcement shall be in accordance with ASTM A884. Epoxy powders are electrostatically spray applied to a sandblasted near-white steel finish (fusion bonded epoxy resin). Ties, supports, and inserts used in conjunction with epoxy coated steel welded wire reinforcement shall be similarly coated.

9 Field Cutting of Welded Wire

9.1 Coated Welded Wire Reinforcement
Coated welded wire reinforcing shall not be field cut, unless permitted by the Engineer.
9.2 Cutting
All cutting of coated reinforcement shall be performed using hydraulic-powered or friction cutting tools to minimize coating damage.

9.3 Flame Cutting
At no time is the use of a flame cutter allowed to cut coated reinforcement.

9.4 Repair
Field cut coated reinforcement shall be repaired immediately with compatible patching material and suitable for repairs in the field.

10 Galvanized Coating Repair

10.1 Damaged Material
All visible damage (i.e., scratches, nicks, cracks) to the galvanized coating of the welded wire reinforcement, caused during shipment, storage or placement shall be repaired by the Contractor at the job site in accordance with appropriate ASTM specifications.

10.2 Cut Material
All ends of reinforcement that have been sheared, sawed, or cut by other means shall be coated.

10.3 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 placing concrete.

11 Epoxy Coating Repair

11.1 Damaged Material
All visible damage (i.e., scratches, nicks, cracks) to the epoxy coating of the welded wire reinforcement, caused during shipment, storage or placement shall be repaired by the Contractor at the job site with approved patching material.

11.2 Cut Material
All ends of reinforcement that have been sheared sawed, or cut by other means shall be coated with approved patching material.

11.3 Damaged Surface Area
The surface area required to be patched shall not exceed 10% of the total sheet surface area, unless otherwise approved by the engineer. Should this limit be exceeded the sheet shall be removed and replaced with an acceptable sheet.

11.4 Curing
The patching material shall be fully cured prior to placing concrete. The patching material shall be compatible with the epoxy coating, inert in concrete, and suitable for repairs in the field.
4.1.3
Miscellaneous Component QA/QC
Stabilized Earth Wall System
MECHANICALLY STABILIZED EARTH STRUCTURES

INCIDENTALS SPECIFICATION

Stabilized Earth Wall System

Grid-Strip Soil Reinforcing

Big-R Bridge - Vistawall
650 Justice Lane
Mansfield, TX 76063
Phone 817.507.0200 • Fax 817.507.0197

Copyright ©Big-R

ALL RIGHTS RESERVED 2018

PROPRIETARY INFORMATION OF BIG-R BRIDGE - VISTAWALL

NO DISTRIBUTION IS ALLOWED WITHOUT WRITTEN PERMISSION FROM

BIG-R BRIDGE

The information set forth in this Manual, including but not limited to all technical and engineering data, figures, tables, designs, drawings, details, procedures, and specifications are intended for information pertaining to this project. Every effort has been made to ensure the Manual 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 any and all liability resulting from such use.
TABLE OF CONTENTS

1 General ........................................................................................................................................ 1
2 Bolt Set ........................................................................................................................................ 1
  2.1 Manufacturer Certification of Fastener Components ........................................................... 1
  2.2 Storage of Fastener Components ........................................................................................... 1
  2.3 Heavy-Hex Structural Bolts .................................................................................................... 1
    2.3.1 Specifications .................................................................................................................. 1
    2.3.2 Geometry ....................................................................................................................... 1
  2.4 Heavy-Hex Nuts ...................................................................................................................... 1
    2.4.1 Specifications .................................................................................................................. 1
    2.4.2 Geometry ....................................................................................................................... 1
  2.5 Grid-Strip™ Bolt Dimensions ................................................................................................. 1
  2.6 Alternative-Design Fasteners ................................................................................................. 1
3 Flip-Pin ....................................................................................................................................... 2
  3.1 Manufacturer Certification of Flip-Pin Components ............................................................ 2
  3.2 Storage of Flip-Pin Components ............................................................................................ 2
  3.3 Structural Flip-Points ............................................................................................................. 2
    3.3.1 Specifications .................................................................................................................. 2
    3.3.2 Geometry ....................................................................................................................... 2
4 TAB Connector ................................................................................................................................ 2
  4.1 Specifications ......................................................................................................................... 2
    4.1.1 ASTM A29 - Standard Specification for Steel Bars, Carbon and Alloy, Hot-Wrought, General Requirements for ................................................................. 2
  4.2 Acceptance ............................................................................................................................ 2
  4.3 Material .................................................................................................................................. 2
  4.4 Tolerances ............................................................................................................................. 3
  4.5 Certification ........................................................................................................................... 3
5 BAR Connector ........................................................................................................................... 3
  5.1 Specifications ......................................................................................................................... 3
    5.1.1 ASTM A572 - Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel ................................................................. 3
  5.2 Acceptance ............................................................................................................................ 3
  5.3 Material .................................................................................................................................. 3
  5.4 Tolerances ............................................................................................................................. 3
  5.5 Certification ........................................................................................................................... 3
6 END CONNECTOR RESISTANCE WELD ........................................................................ 3
7 DPS Anchor ............................................................................................................................... 4

VISTAWALL SYSTEMS

650 JUSTICE LANE
Mansfield, TX 76063
7.1 Specification ........................................................................................................................ 4
  7.1.1 ASTM A29 - Standard Specification for Steel Bars, Carbon and Alloy, Hot-Wrought, General Requirements for ............................................................... 4
  7.1.2 ASTM A1040 - Standard Guide for Specifying Harmonized Standard Grade Compositions for Wrought Carbon, Low-Alloy, and Alloy Steels ............................................................... 4

7.2 Acceptance ......................................................................................................................... 4
7.3 Material ............................................................................................................................... 4
7.4 Tolerances .......................................................................................................................... 4
7.5 Certification ......................................................................................................................... 4

8 DPTS Anchor .......................................................................................................................... 4
  8.1 Specifications ...................................................................................................................... 4
  8.1.1 ASTM A572 - Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel ................................................................. 4

8.2 Acceptance ......................................................................................................................... 4
8.3 Material ............................................................................................................................... 4
8.4 Tolerances .......................................................................................................................... 4
8.5 Certification ......................................................................................................................... 5

9 Bearing Pads .......................................................................................................................... 5
  9.1 Specifications ...................................................................................................................... 5
  9.1.1 ASTM D2240 - Standard Test Method for Rubber Property—Durometer Hardness .......................................................................................................................... 5
  9.1.4 ASTM D395 - Standard Test Methods for Rubber Property—Compression Set ...................................................................................................................... 5
  9.1.5 ASTM D1149 - Standard Test Methods for Rubber Deterioration—Cracking in an Ozone Controlled Environment ................................................................. 5
  9.1.6 ASTM D429 - Standard Test Methods for Rubber Property—Adhesion to Rigid Substrates ........................................................................................................... 5
  9.1.7 ASTM D746 - Standard Test Method for Brittleness Temperature of Plastics and Elastomers by Impact ........................................................................... 5

9.2 Acceptance ......................................................................................................................... 5
9.3 Material ............................................................................................................................... 5
9.4 Property Criteria .................................................................................................................. 5
9.5 Tolerances .......................................................................................................................... 6
9.6 Certification ......................................................................................................................... 6

10 Alignment pin ......................................................................................................................... 6

11 Filter Fabric ............................................................................................................................ 6
  11.1 Specifications ..................................................................................................................... 6
11.1.1 ASTM D4632 - Standard Test Method for Grab Breaking Load and Elongation of Geotextiles

11.1.2 ASTM D6241 - Standard Test Method for Static Puncture Strength of Geotextiles and Geotextile-Related Products Using a 50-mm Probe

11.1.3 ASTM D3786 - Standard Test Method for Bursting Strength of Textile Fabrics—Diaphragm Bursting Strength Tester Method

11.1.4 ASTM D3787 – Standard Test Method for Bursting Strength of Textiles Constant-Rate-Traverse (CRT) Burst Test

11.1.5 ASTM D4533 - Standard Test Method for Trapezoid Tearing Strength of Geotextiles

11.1.6 ASTM D4751 - Standard Test Method for Determining Apparent Opening Size of a Geotextile

11.1.7 ASTM D4491 - Standard Test Methods for Water Permeability of Geotextiles by Permittivity

11.1.8 ASTM D4355 - Standard Test Method for Deterioration of Geotextiles by Exposure to Light, Moisture and Heat in a Xenon Arc Type Apparatus

11.2 Acceptance

11.3 Material

11.4 Property Criteria

11.5 Certification

12 Adhesive

12.1 Specifications

12.1.1 ASTM D3498 - Standard Specification for Adhesives for Field-Gluing Plywood to Lumber Framing for Floor Systems

12.1.2 ASTM D905 - Standard Test Method for Strength Properties of Adhesive Bonds in Shear by Compression Loading

12.2 Material

12.3 Certification
1 GENERAL
The following material parameter specifications cover the incidental items required to construct the VISTAWALL SEW retaining wall system.

2 BOLT SET
This section covers the bolt set that is used to join the Grid-Strip TAB/BAR connector to the DPS/DPTS 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 A563.

2.4.2 Geometry
Heavy-hex nut dimensions shall meet the requirements of ANSI/ASME B18.2.6.

2.5 Grid-Strip™ 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
(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 FLIP-PIN

This section covers the Flip-Pin that is used to join the Grid-Strip TAB/BAR connector to the DPS/DPTS Panel Anchor. The Flip-Pin shall be a minimum diameter of ½” and conform to ASTM A307.

3.1 Manufacturer Certification of Flip-Pin Components

Manufacturer certifications documenting conformance to the applicable specifications required in Sections 3.3 for all Flip-Pin components used in the fastener assemblies shall be available to the Engineer of Record and inspector prior to assembly or erection of structural steel.

3.2 Storage of Flip-Pin Components

Flip-Pin components shall be protected from dirt and moisture in closed containers at the site of installation. Only as many Flip-Pin components as are anticipated to be installed during the work shift shall be taken from protected storage. Flip-Pin components that are not incorporated into the work shall be returned to protected storage at the end of the work shift. Flip-Pin components shall not be cleaned or modified from the as-delivered condition. Flip-Pin components that accumulate rust or dirt shall not be incorporated into the work unless they are requalified.

3.3 Structural Flip-Pins

3.3.1 Specifications

Structural Flip-Pins shall meet the requirements of ASTM A307 Grade B or ASTM A1064, plain wire.

3.3.2 Geometry

Flip-Pin dimensions shall meet the requirements as specified by VISTAWALL. The Flip-Pin length used shall be such that the end of the pin extends beyond the DPS/DPTS anchor by ½” when properly installed.

4 TAB CONNECTOR

This section covers the TAB Connector that is attached to the proximal ends of the Grid-Strip. The TAB connector connects to the DPS/DPTS Panel Anchor.

4.1 Specifications

4.1.1 ASTM A29 - Standard Specification for Steel Bars, Carbon and Alloy, Hot-Wrought, General Requirements for


4.2 Acceptance

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

4.3 Material
The TAB Connector shall be A1040 steel and have a minimum tensile capacity of 65 ksi.

4.4 Tolerances
The permissible variation from the dimensions and configuration shown on the TAB Connector Cut Sheet prepared by Vistawall shall be as follows:

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Tolerance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall Vertical Dimensions</td>
<td>± 1/8</td>
</tr>
<tr>
<td>Overall Horizontal Dimensions</td>
<td>± 1/8</td>
</tr>
<tr>
<td>Overall Hole Dimension</td>
<td>± 1/64</td>
</tr>
<tr>
<td>Overall Edge Distance</td>
<td>± 1/32</td>
</tr>
<tr>
<td>Overall Shaft Diameter</td>
<td>± 1/32</td>
</tr>
</tbody>
</table>

4.5 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.

5 BAR CONNECTOR
This section covers the BAR Connector that is attached to the proximal ends of the Grid-Strip. The TAB connector connects to the DPS/DPTS Panel Anchor.

5.1 Specifications
5.1.1 ASTM A572 - Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel.

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

5.3 Material

The BAR Connector shall be Grade 50 steel and have a minimum tensile capacity of 65 ksi and a minimum yield capacity of 50 ksi.

5.4 Tolerances
The permissible variation from the dimensions and configuration shown on the BAR Connector Cut Sheet prepared by VISTAWALL shall be as follows:

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Tolerance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall Vertical Dimensions</td>
<td>± 1/8</td>
</tr>
<tr>
<td>Overall Horizontal Dimensions</td>
<td>± 1/8</td>
</tr>
<tr>
<td>Overall Hole Dimension</td>
<td>± 1/64</td>
</tr>
<tr>
<td>Overall Edge Distance</td>
<td>± 1/32</td>
</tr>
<tr>
<td>Overall Shaft Diameter</td>
<td>± 1/32</td>
</tr>
</tbody>
</table>

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. The certification shall be notarized.

6 END CONNECTOR RESISTANCE WELD
The TAB and BAR are resistance welded to the longitudinal wires using specialized resistance weld machines fabricated to the specification of Vistawall. The resistance weld shall be made with a minimum 1.5” weldment. The end connector tensile capacity shall be tested at a minimum of 2 samples selected at random 2 times a day from a day’s production run for a total of a minimum of 4 tests. The first 2 test shall be selected from the first production run welds. The end connector shall be placed in the testing machine and pulled until failure. A successful test is considered to have occurred when the longitudinal wire, or wires, yields.
When one longitudinal wire ruptures it is possible for the remaining longitudinal wire side to fail in weld shear. The technician shall inspect the wire and verify that both wires have started to yield. The force for a W11 Grid-Strip (GS11) is equal to the following:

\[ F = F_y \cdot (2 \cdot A) \]

Where:
- \( F \) = Force (lbf)
- \( F_y \) = Yield strength of steel (psi)
- \( A \) = Area of wire (in\(^2\))

### 7 DPS ANCHOR

This section covers the DPS Anchor that is attached to the proximal end of the Grid-Strip. The DPS Panel Anchor connects to the TAB/BAR Connector.

#### 7.1 Specification

7.1.1 ASTM A29 - Standard Specification for Steel Bars, Carbon and Alloy, Hot-Wrought, General Requirements for


#### 7.2 Acceptance

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

#### 7.3 Material

The DPS Anchor shall be A1040 steel and have a minimum tensile capacity of 65 ksi.

#### 7.4 Tolerances

The permissible variation from the dimensions and configuration shown on the DPS Anchor Cut Sheet prepared by VISTAWALL shall be as follows:

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Tolerance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall Vertical Dimensions</td>
<td>± 1/8</td>
</tr>
<tr>
<td>Overall Horizontal Dimensions</td>
<td>± 1/8</td>
</tr>
<tr>
<td>Overall Hole Dimension</td>
<td>± 1/64</td>
</tr>
<tr>
<td>Overall Edge Distance</td>
<td>± 1/32</td>
</tr>
<tr>
<td>Overall Shaft Diameter</td>
<td>± 1/32</td>
</tr>
<tr>
<td>Overall Disk Diameter</td>
<td>± 1/16</td>
</tr>
</tbody>
</table>

#### 7.5 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.

### 8 DPTS ANCHOR

This section covers the DPTS Anchor that is attached to the proximal end of the Grid-Strip. The DPTS Panel Anchor connects to the TAB/BAR Connector.

#### 8.1 Specifications


#### 8.2 Acceptance

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

#### 8.3 Material

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

#### 8.4 Tolerances

The permissible variation from the dimensions and configuration shown on the DPTS Anchor
Cut Sheet prepared by VISTAWALL shall be as follows:

<table>
<thead>
<tr>
<th>Specification</th>
<th>Tolerances</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall Vertical Dimensions</td>
<td>± 1/8</td>
</tr>
<tr>
<td>Overall Horizontal Dimensions</td>
<td>± 1/8</td>
</tr>
<tr>
<td>Overall Hole Dimension</td>
<td>± 1/64</td>
</tr>
<tr>
<td>Overall Edge Distance</td>
<td>± 1/32</td>
</tr>
<tr>
<td>Overall Shaft Diameter</td>
<td>± 1/32</td>
</tr>
<tr>
<td>Overall Disk Diameter</td>
<td>± 1/16</td>
</tr>
</tbody>
</table>

8.5 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.

9 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).

9.1 Specifications

9.1.1 ASTM D2240 - Standard Test Method for Rubber Property—Durometer Hardness

9.1.2 ASTM D412 - Standard Test Methods for Vulcanized Rubber and Thermoplastic Elastomers—Tension

9.1.3 ASTM D573 - Standard Test Method for Rubber—Deterioration in an Air Oven

9.1.4 ASTM D395 - Standard Test Methods for Rubber Property—Compression Set

9.1.5 ASTM D1149 - Standard Test Methods for Rubber Deterioration—Cracking in an Ozone Controlled Environment

9.1.6 ASTM D429 - Standard Test Methods for Rubber Property—Adhesion to Rigid Substrates

9.1.7 ASTM D746 - Standard Test Method for Britteness Temperature of Plastics and Elastomers by Impact

9.2 Acceptance

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

9.3 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.

9.4 Property Criteria

The following properties will be tested and adhere to the minimum values provided in the table.
### Bearing Pad Property Criteria

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Durometer Grade</td>
<td>60</td>
</tr>
<tr>
<td><strong>ASTM</strong></td>
<td>Property</td>
</tr>
<tr>
<td>D2240 Hardness</td>
<td>60 ± 5</td>
</tr>
<tr>
<td>D412 Tensile Strength (psi)</td>
<td>2500</td>
</tr>
<tr>
<td>D412 Ultimate elongation (%)</td>
<td>350</td>
</tr>
<tr>
<td>D573 Change in Durometer</td>
<td>15</td>
</tr>
<tr>
<td>Hardness</td>
<td></td>
</tr>
<tr>
<td>D573 Change in Tensile Strength</td>
<td>-15</td>
</tr>
<tr>
<td>(% max)</td>
<td></td>
</tr>
<tr>
<td>D573 Change in Ultimate Elongation</td>
<td>-40</td>
</tr>
<tr>
<td>(% max)</td>
<td></td>
</tr>
<tr>
<td>D395 Compressive Set 22 hars at 212 F (% max)</td>
<td>35</td>
</tr>
<tr>
<td>D1149 Ozone 100 ppm ozone in air by volume</td>
<td>No Cracks</td>
</tr>
<tr>
<td>D429 Adhesion Bond made during vulcanization</td>
<td>40</td>
</tr>
<tr>
<td>D746 Lowe Temperature Test for Brittleness</td>
<td>No Failure</td>
</tr>
</tbody>
</table>

### 9.5 Tolerances

For both plain and laminated bearings, the permissible variation from the dimensions and configuration shown on the plans shall be as follows.

<table>
<thead>
<tr>
<th>Bearing Pad Tolerance Table</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall Vertical Dimensions</td>
</tr>
<tr>
<td>Average total thickness 1 1/4 inches (32 mm) or less</td>
</tr>
<tr>
<td>Overall Horizontal Dimensions</td>
</tr>
<tr>
<td>36 inches (914 mm) and less</td>
</tr>
<tr>
<td>Variation from a Plane Parallel to the Theoretical Surface (as determined by measurements at the edges of bearings)</td>
</tr>
<tr>
<td>Top</td>
</tr>
<tr>
<td>Sides</td>
</tr>
<tr>
<td>Individual non-elastic laminates</td>
</tr>
</tbody>
</table>

### 9.6 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.

### 10 ALIGNMENT PIN

The alignment pin is a construction aid. As such is classified as a non-structural element. The pin can be fabricated from steel, fiberglass or plastic. The dimensional requirements shall be such that the pin have a maximum length equal to 11.5 inches and a minimum length equal to 10.0 inches. The diameter of the alignment pin shall be 5/8” ± 1/16”. If the alignment pin is fabricated from steel it shall be hot-dip galvanized.

### 11 FILTER FABRIC

This section covers the filter fabric that is used to cover the joints at the back of the segmental concrete panel. The filter fabric consists of polypropylene needle punched nonwoven geotextile.

#### 11.1 Specifications

11.1.1 **ASTM D4632 - Standard Test Method for Grab Breaking Load and Elongation of Geotextiles**

11.1.2 **ASTM D6241 - Standard Test Method for Static Puncture Strength of Geotextiles and Geotextile-Related Products Using a 50-mm Probe**

11.1.3 **ASTM D3786 - Standard Test Method for Bursting Strength of Textile Fabrics—Diaphragm Bursting Strength Tester Method**

11.1.4 **ASTM D3787 – Standard Test Method for Bursting Strength of Textiles**
Constant-Rate-Traverse (CRT) Burst Test

11.1.5 ASTM D4533 - Standard Test Method for Trapezoid Tearing Strength of Geotextiles

11.1.6 ASTM D4751 - Standard Test Method for Determining Apparent Opening Size of a Geotextile

11.1.7 ASTM D4491 - Standard Test Methods for Water Permeability of Geotextiles by Permittivity

11.1.8 ASTM D4355 - Standard Test Method for Deterioration of Geotextiles by Exposure to Light, Moisture and Heat in a Xenon Arc Type Apparatus

11.2 Acceptance

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

11.3 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.

11.4 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.

<table>
<thead>
<tr>
<th>Mechanical</th>
<th>ASTM</th>
<th>MARV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grab Tensile Strength</td>
<td>D4632</td>
<td>250 lbs</td>
</tr>
<tr>
<td>Puncture Strength &gt;50% Elongation</td>
<td>D6241</td>
<td>500 lbs</td>
</tr>
<tr>
<td>Trapezoidal Tear</td>
<td>D4533</td>
<td>90 lbs</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Hydraulic</th>
<th>ASTM</th>
<th>Property</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Apparent Opening Size (AOS)</td>
<td>D4751</td>
<td>70 US Standard Sieve</td>
</tr>
<tr>
<td>Permittivity</td>
<td>D4491</td>
<td>1.50 sec⁻¹</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Endurance</th>
<th>ASTM</th>
<th>Property</th>
</tr>
</thead>
<tbody>
<tr>
<td>UV Resistance (% Retained after 500 hours)</td>
<td>D4355</td>
<td>70%</td>
</tr>
</tbody>
</table>

11.5 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.

12 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 temporary means to adhere the geotextile filter fabric to the panel until the backfill has been place.

12.1 Specifications

12.1.1 ASTM D3498 - Standard Specification for Adhesives for Field-Gluing Plywood to Lumber Framing for Floor Systems

12.1.2 ASTM D905 - Standard Test Method for Strength Properties of Adhesive Bonds in Shear by Compression Loading

12.2 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 strong under moisture and temperature conditions.
12.3 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.
4.2 CONSTRUCTION
4.2.1
Construction QA/QC
CONSTRUCTION
QUALITY CONTROL SPECIFICATION

Stabilized Earth Wall System
MECHANICALLY STABILIZED EARTH STRUCTURES

Construction Specification

Stabilized Earth Wall System

Big-R Bridge - Vistawall
650 Justice Lane
Mansfield, TX 76063
Phone 817.507.0200 • Fax 817.507.0197

Copyright ©Big-R

ALL RIGHTS RESERVED 2017
PROPRIETARY INFORMATION OF BIG-R BRIDGE - VISTAWALL
NO DISTRIBUTION IS ALLOWED WITHOUT WRITTEN PERMISSION FROM
BIG-R BRIDGE

The information set forth in this Manual, including but not limited to all technical and engineering data, figures, tables, designs, drawings, details, procedures, and specifications are intended for information pertaining to this project. Every effort has been made to ensure the Manual 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.
# Table of Contents

1 **GENERAL** ...........................................................................................................................................1  
   1.1 **WIRE REINFORCEMENT INSTITUTE- MANUAL OF STANDARD PRACTICE** ........................................1  

2 **DEFINITIONS** ...................................................................................................................................1  

3 **REFERENCE DOCUMENTS** ..............................................................................................................3  
   3.1 **VISTAWALL INSTALLATION GUIDE** ...............................................................................................3  
   3.2 **VISTAWALL CONCRETE QA/QC** ....................................................................................................3  
   3.3 **VISTAWALL WELDED WIRE QA/QC** ...............................................................................................3  
   3.4 **VISTAWALL INCIDENTAL QA/QC** ....................................................................................................3  
   3.5 **VISTAWALL REINFORCED SOIL PROPERTIES** .............................................................................3  

4 **QUALITY ASSURANCE** .......................................................................................................................3  

Appendix A – Installation Checklist  
Appendix B – Field Report  
Appendix C – Contractor Prequalification
MSE CONSTRUCTION QA/QC PROGRAM

1 General

Work includes the construction of the Vistawall Stabilized Earth Wall system.

1.1 Wire Reinforcement Institute- Manual of Standard Practice

2 Definitions

Alignment Pin - A non-structural dowel that is placed in successive layers of panels in order to maintain proper alignment during erection. This is an optional element.

BAR - The proximal end of the Grid-Strip™ consisting of a special shaped steel plate that is used to attach the Grid-Strip™ to the panel anchor.

Bearing Pad - A compressible generally rectangular element that is placed on the top edge of the concrete panel. The bearing pad prevents concrete-to-concrete contact and concrete spalling.

Bottom Panel (P) – A special height panel that is placed on the concrete leveling course and is the first panel in the wall structure.

Connection Bolt - A threaded fastener, with a head, designed to be used in conjunction with a nut and that joins the Grid-Strip™ to the panel anchor.

Connection Pin - A special shaped smooth cylindrical fastener that joins the soil reinforcement to a panel anchor. This connection pin can be used in lieu of a bolt if approved by the Engineer of Record (Wall EOR).

Coping - A concrete element that is used as the top of wall treatment. The coping can be either pre-cast or cast-in-place.

Coping Panel - A special height panel that is placed at the top of the wall prior to placement of the coping unit.

DPS Anchor – A special shaped steel forged Dual Plate Shaft (DPS) anchor that is cast into the concrete panel, so it protrudes form the back face and that is used to connect the Grid-Strip™ soil reinforcing to the panel.

DPTS Anchor – A special shaped steel Dual Plate Tie Strip (DPTS) anchor that is cast into the concrete panel, so it protrudes form the back face and that is used to connect the Grid-Strip™ soil reinforcing to the panel.

Face of Wall – The front face of the panel. This can differ from the control point of the structure.

Facing Element – The facing consists of a pre-cast concrete panel that is structurally connected to the soil reinforcement and prevents the raveling of soil between the layers of soil reinforcement.

Filter Fabric (Cloth) - A needle punched geotextile fabric that is placed over the horizontal and vertical joints of each concrete panel to prevent the soil from eroding from the joints.

Finish Grade - The material that is placed in front of the MSE wall that protects the bottom of the wall from erosion and undercutting.
**Flip-Pin** - A special shaped smooth cylindrical fastener that joins the soil reinforcement to the panel anchor.

**Geocomposite** - A combination of geo-textile and plastic that forms a composite fabric that is used in lieu of the filter fabric for specific applications.


**Inclusion** - Any man-made element that is inserted into the soil mass to improve the structural properties of the soil.

**Junction Slab** – Reinforced concrete slab that is structurally attached to the coping element. This prevents the coping element from being dislodge from the structure during impact to the coping element. This is sometimes referred to as a moment slab.

**Level-Up Concrete** - Non-reinforced concrete or grout that is placed on the top panel and leveled in order to give the coping a smooth surface to bear on. This concrete brings the top of wall to the correct elevation and orientation.

**Leveling Course** – A level non-structural, non-reinforced, concrete element that the first row of panels is placed on.

**Mechanically Stabilized Earth (MSE)** - Engineering term for the stabilization of earth structures through the use of soil inclusions.

**Moment Slab** - Reinforced concrete slab that is structurally attached to the coping element. This element prevents the coping element from being dislodge from the structure during impact to the coping element. This is sometimes referred to as a junction slab.

**Panel Anchor** - An element that is cast directly into the back face of the panel and that soil reinforcement is attached to.

**Prepared Foundation** - Excavated and proofed rolled area that the reinforced mass of soil and leveling course bears on.

**Reinforced Soil** - A composite structure composed of alternating layers of soil and inclusions.

**Retained Fill** - The backfill material that is placed directly behind the reinforced soil mass. Usually consists of normal highway embankment material.

**Select Fill** - The mass of soil that is placed within the reinforced soil.

**Shim** - Wedge shaped or thin element that is used to correct panel orientation and to keep joints horizontal. Shim material can be wood or plastic.

**Splice** – The structural joining of two soil reinforcing elements in order to increase the length of the soil reinforcing.

**Soil Reinforcement** - A manufactured grid element that is placed in the select backfill and that is structurally attached to the back face of the facing element.

**TAB** – The proximal end of the Grid-Strip™ consisting of a special forged steel plate that is used to attach the Grid-Strip™ to the panel anchor.

**Traffic Barrier** - A structural element that retains traffic and directs impact in a desired direction.
In most cases the traffic barrier is structurally attached to the coping element.

**V-Plate** – slotted steel plate that is used to splice two Grid-Strip™ soil reinforcing elements together in the field.

**Wave Plate** – Composite element that is used to structurally splice two Grid-Strip™ soil reinforcing elements together in the field.

3 Reference Documents

3.1 Vistawall Installation Guide

3.2 Vistawall Concrete QA/QC

3.3 Vistawall Welded Wire QA/QC

3.4 Vistawall Incidental QA/QC

3.5 Vistawall Reinforced Soil Properties

4 Quality Assurance

The MSE retaining wall shall be installed in conformance with the governing project specifications, the requirements given in the Vistawall Installation Guide and information contained herein.
This Page Intentionally Left Blank
APPENDIX A – INSTALLATION CHECKLIST
This Page Intentionally Left Blank
NOTES FOR CHECKLIST

1. This checklist is intended to be completed, signed, and dated by the Technical Advisor providing Job Site Assistance.

2. All pages of the form are to have the Project Number and Date placed in the header.

3. Each “Item” must have a “Yes”, “No”, or “NA” box checked. Any comment or action required should be entered in the “Comment/Action” column. If the “No” or “NA” box is checked then an appropriate comment or action required must be entered. Use the “notes sheet” at the end of document to add additional comments.

4. Add any pertinent project specific questions to the checklist as necessary.

5. All material certifications should be obtained by the Technical Advisor prior to beginning construction.

6. The Technical Advisor should contact the Project Engineer in case of discrepancies between the submittals and field conditions.

7. All directions provided by the Project Construction Inspector shall be documented and shall include the following:
   a. Date
   b. Name of Inspector
   c. Direction
   d. People present during direction.
<table>
<thead>
<tr>
<th>No.</th>
<th>Yes</th>
<th>No</th>
<th>NA</th>
<th>Item</th>
<th>Comment/Action</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>Do you have an approved copy of shop drawings?</td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>Do you have backfill certifications?</td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>Do you have panel certifications?</td>
<td></td>
</tr>
<tr>
<td>4.</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>Do you have soil reinforcing certifications?</td>
<td></td>
</tr>
<tr>
<td>5.</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>Is all required material on site?</td>
<td></td>
</tr>
<tr>
<td>6.</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>Is the material stored properly to prevent on site damage?</td>
<td></td>
</tr>
<tr>
<td>7.</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>Has damaged material been recorded and a copy of rejected material given to suppliers?</td>
<td></td>
</tr>
<tr>
<td>8.</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>Is the foundation excavated and proof rolled per the specifications and to the required width and elevation?</td>
<td></td>
</tr>
<tr>
<td>9.</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>Has unsuitable foundation material been compacted or removed and replaced?</td>
<td></td>
</tr>
<tr>
<td>No.</td>
<td>Yes</td>
<td>No</td>
<td>NA</td>
<td>Item</td>
<td>Comment/Action</td>
</tr>
<tr>
<td>-----</td>
<td>-----</td>
<td>----</td>
<td>----</td>
<td>----------------------------------------------------------------------</td>
<td>----------------</td>
</tr>
<tr>
<td>10.</td>
<td></td>
<td></td>
<td></td>
<td>Is the first row of soil reinforcing properly placed, aligned, and spaced.</td>
<td></td>
</tr>
<tr>
<td>11.</td>
<td></td>
<td></td>
<td></td>
<td>Are the proper face panels being installed?</td>
<td></td>
</tr>
<tr>
<td>12.</td>
<td></td>
<td></td>
<td></td>
<td>Are the required number of soil reinforcing elements and the correct type being used?</td>
<td></td>
</tr>
<tr>
<td>13.</td>
<td></td>
<td></td>
<td></td>
<td>Are the correct connection pins being used and are the soil reinforcing making proper contact with the panel anchors?</td>
<td></td>
</tr>
<tr>
<td>14.</td>
<td></td>
<td></td>
<td></td>
<td>Is the filter fabric being properly placed and adhered to the back face of the panel?</td>
<td></td>
</tr>
<tr>
<td>15.</td>
<td></td>
<td></td>
<td></td>
<td>Is the backfill being properly placed? Is it being placed in 12-inch lifts?</td>
<td></td>
</tr>
<tr>
<td>16.</td>
<td></td>
<td></td>
<td></td>
<td>Is the backfill material being spread from the back face of panel to tail of soil reinforcing?</td>
<td></td>
</tr>
<tr>
<td>17.</td>
<td></td>
<td></td>
<td></td>
<td>Is the equipment being kept off of the soil reinforcing until 6&quot; of backfill material is placed?</td>
<td></td>
</tr>
<tr>
<td>18.</td>
<td></td>
<td></td>
<td></td>
<td>Is specified compaction being achieved? Compaction shall be no less that 95% when measured using standard proctor (AASHTO T99 or ASTM D698)</td>
<td></td>
</tr>
<tr>
<td>19.</td>
<td></td>
<td></td>
<td></td>
<td>Are the soil reinforcing elements being properly aligned?</td>
<td></td>
</tr>
<tr>
<td>No.</td>
<td>Yes</td>
<td>No</td>
<td>NA</td>
<td>Item</td>
<td>Comment/Action</td>
</tr>
<tr>
<td>-----</td>
<td>-----</td>
<td>----</td>
<td>----</td>
<td>-------------------------------------------------------------------------------------------</td>
<td>---------------------------</td>
</tr>
<tr>
<td>20.</td>
<td></td>
<td></td>
<td></td>
<td>Is the vertical and horizontal alignment of the structure being checked periodically?</td>
<td></td>
</tr>
<tr>
<td>21.</td>
<td></td>
<td></td>
<td></td>
<td>At the end of each day’s operation is the reinforced volume being protected from runoff and saturation?</td>
<td></td>
</tr>
</tbody>
</table>

### Attachments

<table>
<thead>
<tr>
<th>No.</th>
<th>Attachments</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td></td>
</tr>
</tbody>
</table>
APPENDIX B – FIELD REPORT
<table>
<thead>
<tr>
<th>Field</th>
<th>Field</th>
<th>Field</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date Of Visit:</td>
<td>Project Name:</td>
<td>Project Number:</td>
</tr>
<tr>
<td>Location:</td>
<td>Problems:</td>
<td></td>
</tr>
<tr>
<td>Observations:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Solutions:</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
# Field Inspection Report

<table>
<thead>
<tr>
<th></th>
<th>YES</th>
<th>NO</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td></td>
<td>Do you have an approved set of plans</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td>Do you have backfill certifications</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td>Do you have material certifications</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td>Does precaster have correct panels cast</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td>Is all necessary material on site</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
<td>Is material stored properly to prevent damage</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
<td>Has damaged material been inventoried and submitted to supplier</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td>Is the foundation proof rolled and at the required elevation</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td></td>
<td>Has any unsuitable material been removed and replaced</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td>Is the leveling course trenched and properly formed</td>
</tr>
<tr>
<td>11</td>
<td></td>
<td></td>
<td>Is the leveling course poured and properly cured</td>
</tr>
<tr>
<td>12</td>
<td></td>
<td></td>
<td>Is the leveling course at the required elevations and tolerances</td>
</tr>
<tr>
<td>13</td>
<td></td>
<td></td>
<td>Is the first row of facing panels properly placed, plumbed, and spaced</td>
</tr>
<tr>
<td>14</td>
<td></td>
<td></td>
<td>Are the proper panels being installed</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
<td>Are the required number of soil reinforcing elements being placed</td>
</tr>
<tr>
<td>16</td>
<td></td>
<td></td>
<td>Are the connection pins and soil reinforcing elements making proper contact at the anchor</td>
</tr>
<tr>
<td></td>
<td>YES</td>
<td>NO</td>
<td>Description</td>
</tr>
<tr>
<td>---</td>
<td>-----</td>
<td>----</td>
<td>-----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>17</td>
<td>YES</td>
<td>NO</td>
<td>Are alignment pins being placed before back fill</td>
</tr>
<tr>
<td>18</td>
<td>YES</td>
<td>NO</td>
<td>Is the filter fabric or screen cloth being properly placed and the proper securing agent being used</td>
</tr>
<tr>
<td>19</td>
<td>YES</td>
<td>NO</td>
<td>Is the backfill being placed in required lift thickness and spread toward the tail of the soil reinforcing element</td>
</tr>
<tr>
<td>20</td>
<td>YES</td>
<td>NO</td>
<td>Is equipment being kept off of the soil reinforcing element until six inches of fill is placed</td>
</tr>
<tr>
<td>21</td>
<td>YES</td>
<td>NO</td>
<td>Is the proper compaction density being achieved and recorded.</td>
</tr>
<tr>
<td>22</td>
<td>YES</td>
<td>NO</td>
<td>Is the fill being placed and compacted to a level equal to or greater than the anchor elevation</td>
</tr>
<tr>
<td>23</td>
<td>YES</td>
<td>NO</td>
<td>Are the mats being properly aligned and pulled tight.</td>
</tr>
<tr>
<td>24</td>
<td>YES</td>
<td>NO</td>
<td>Is the vertical and horizontal alignment being checked and maintained</td>
</tr>
<tr>
<td>25</td>
<td>YES</td>
<td>NO</td>
<td>Are the wood shims being removed from the panel every third row</td>
</tr>
<tr>
<td>26</td>
<td>YES</td>
<td>NO</td>
<td>At the end of each days operation is the fill being groomed to provide for positive drainage away from face of wall</td>
</tr>
<tr>
<td>27</td>
<td>YES</td>
<td>NO</td>
<td>Has the bury at the face of the wall been placed and compacted</td>
</tr>
<tr>
<td>28</td>
<td>YES</td>
<td>NO</td>
<td>Are dowels being proved for the coping</td>
</tr>
<tr>
<td>29</td>
<td>YES</td>
<td>NO</td>
<td>Is leveling concrete being placed to the proper elevation</td>
</tr>
</tbody>
</table>
APPENDIX C – CONTRACTOR PREQUALIFICATION
This Page Intentionally Left Blank
MSE WALL INSTALLER PREQUALIFICATION REQUIREMENTS

The following are minimum requirements that should be used and verified to properly qualify the installer of all Vistawall Stabilized Earth Wall (SEW) systems.

**Project Job-Site MSE Wall Installer**

The Job-Site MSE Wall Foreman shall have experience in construction of at least five transportation related MSE walls within the last three years. Transportation related MSE walls shall be defined as MSE walls that carry or are adjacent to vehicular traffic and are constructed with MSE reinforcement in the reinforced structure backfill zone. The foreman must have prior experience or adequate training on the installation of the Vistawall SEW wall system. The resume and credentials of the Foreman shall be submitted to the Project Engineer for approval prior to the pre-construction meeting. The Foreman shall be on the site for 100 percent of time during which the work is being done.

**Alternate Prequalification Criterion (in 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.

The Project Foreman shall construct a wall test segment. The wall test segment shall be constructed in the presence of a Vistawall Technical Representative and 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 Vistawall to be present during the construction of each wall test segment. The Technical Representative shall be present for construction of the wall test segment. The wall test segment shall include construction of each of the 5 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, and
5. If corners are required, construction of a corner representative of the corners in the wall in the project in a minimum of a two row portion of panels or a minimum of a four foot wall height
Before construction of the wall test segment the Vistawall Technical Representative will provide the Contractor, Project Foreman and the Engineer the following:

- Technical instructions as required in the construction of the Vistawall SEW 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 Vistawall Technical Representative will provide the following documentation for final approval by the Engineer:

- Documentation that the wall test segment was constructed in accordance with 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.
5.0 PERFORMANCE
5.1 Warranties and Disclaimers
5.2 TESTING
5.2.1
Bearing Pad Testing
June 1, 2011

Mr. Thomas P. Taylor  
T&B Structural Systems  
6800 Manhattan Blvd, Suite 304  
Fort Worth, TX  76120

Subject: Additional Numerical Model Analysis of T&B Structural Systems’ Stabilized Earth Wall (SEW) System  
Project GCC# 20290

Dear Tom,

The following provides documentation of Young’s modulus for the rubber bearing pads in compression loading that will be used in further development of a numerical model of T&B Structural’s new Stabilized Earth Wall (SEW) system using the W11 Grid Strip™ soil reinforcing elements. The local deflection characteristics of the bearing pad material used between the wall panels was determined by laboratory testing in general accordance with ASTM D575 “Rubber Properties in Compression”. Test results and selection of a modulus value is presented as follows.

Results from the laboratory compression tests in terms of deformation versus applied load are shown in Figure 1 and pad deformation of ¼ inch (strain value of 33%) corresponds to an applied load of about 17,000 lbs. Testing results are also shown as stress-strain curves in Figure 2 which include area correction and Figure 3 without area correction. Area corrections were calculated according to $A_o/(1-\epsilon)$, where $A_o$ is the original area of the bearing pad, $\epsilon$ is the unit strain defined as the ratio $\Delta H/H_o$, $\Delta H$ is the change in thickness of the pad, and $H_o$ is the original pad thickness. Based on the test results, modulus values range from about 1500 psi to 2000 psi for wall heights of 25 ft to 30 ft and correspond to nominal strains of about 15%. Accordingly, a modulus value of 1500 psi will be used for additional numerical model analysis of the SEW system.

If you have any questions, please do not hesitate to contact us.

Sincerely yours,

Geocomp Consulting, Inc.

Lois G. Schwarz, Ph.D.  
Geotechnical Engineer

Martin Hawkes, P.E.  
Project Engineer

Barry Christopher, Ph.D., P.E.  
Staff Consultant
Figure 1. Laboratory compression test results on rubber bearing pads.
Figure 2. Stress-strain relationships for rubber pads with corrected area.

Figure 3. Stress-strain relationships for rubber pads without area correction.
5.3

PERFORMANCE HISTORY

BIG R BRIDGE

650 Justice Lane
Mansfield, TX 76063
5.3.1
ERS Performance History
5.3.2
ERS Oldest Structures
5.3.3
ERS Tallest Structures
5.3.4
ERS Horizontal Displacement
5.3.5
ERS Differential Settlement
5.3.6
ERS Surcharge Loading
5.3.7
ERS Private and Public Users
6.0
OTHER
6.1.1
Other Information