REPORT OF REVIEW THE SSL, MSE Plus® MECHANICALLY STABILIZED EARTH WITH PRECAST CONCRETE FACING PANELS AND INEXTENSIBLE SOIL REINFORCEMENTS

October 2023

HIGHWAY INNOVATIONS, DEVELOPMENTS, ENHANCEMENTS AND ADVANCEMENTS (IDEA)

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

Applicant Information
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Review Summary
Following its initial review of the MSE Plus® Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System submittal, the IDEA review team provided the applicant with a series of comments and requests for clarification and the need for additional connection testing and analysis. The applicant has been thorough in its responses and the review team finds that there are no outstanding issues that should be brought to the attention of the transportation agencies. Rather, the agencies are encouraged to rely upon the MSE Plus® Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System submittal for projects where the MSE Plus® Retaining Wall System is proposed.

Submittal Checklist
The checklist used from the IDEA protocol for this evaluation is C4 – Initial Technical Evaluation Checklist for Concrete Panel Paired with Inextensible Reinforcement. This is the second evaluation of the MSE Plus® Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System by the IDEA evaluation program. This evaluation was performed at SSL’s request review new testing of the connection between the soil reinforcement and the concrete face panels. The system was originally evaluated by the HITEC evaluation program in 1999.

Confidential Information
The applicant has the option to omit information from the version of its submittal that is attached to the final report if it believes that such information is confidential. In such instances, the applicant will notify the review team. However, for the SSL MSE Plus® Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System no information has been designated by the applicant as confidential.
System Description

Components
The major components of the MSE Plus® Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System are precast concrete facing panels, inextensible steel soil reinforcements, connection system and select granular backfill. The facing system is a precast concrete panel. The inextensible soil reinforcing consists of a grid type welded wire mesh. The other components that are used with the MSE system include, and are not limited to a concrete leveling pad, coping or barriers, bearing pads and joint filter fabric for soil retention.

The MSE Plus system offers four distinct standard size precast facia wall panels. They are 5’ x 5’, 5’ x 6’, 5’ x 10’, and 5’ x 12’ system. Each of the facing panels are 6-inches in thickness with a standard concrete strength of 4,000 PSI. The SSL MSE Plus system does not incorporate the use shear pins or alignment devices for wall construction. The system does use two bearing pads at the top of each 5’ or 6’ wide standard size panel to control the horizontal panel joint width. Larger 10’ and 12’ width panels will use 3 or 4 bearing pads per panel joint. The SSL MSE Plus retaining wall system utilizes three longitudinal wire sizes, W11, W20, and W24 with W11 transverse wires spaced at 0.5, 1.0, 1.5 2.0 ft, 2.5, and 3.0 spacing.

The connection system is comprised of three components. The first is a pair of wire loops are cast into the panel to form a clevis, the second component is the one end of the soil reinforcement that is bent 90 degrees to form a 90-degree loop. The looped end of the soil reinforcement is inserted in between the two loops cast into the panel which forms the clevis. The third component is a pin that is inserted into the soil reinforcement loop and clevis to complete the connection of the soil reinforcement to the panel. The MSE Plus system uses only one type of connection device paired to up to 6 connectors per layer. The connection system is fabricated in accordance with ASTM A 1064. The embed loops are fabricated with material with a minimum yield strength of 75,000 PSI.

System History
SSL’s MSE Plus® system was developed in 1997 and SSL received its first approval in California by The California Department of Transportation. With the completion of our HITEC evaluation in 1999 SSL received approvals by most state departments of transportation agencies. The connection system evaluated in the 1999 HITEC reports was for an “internal” connection. In 2003 SSL revised their connection system to a more conventional external system using clevis embed connectors cast into the concrete panel by then joining the 90-degree loop in soil reinforcement to the clevis using a connection pin. Testing requirements were modified for LRFD designs. Testing was performed in December of 2012 and SSL received approval of their LRFD design shortly thereafter.

Three of the oldest structures using this new connection system are in California. The projects are the SR 22 Project in Orange County, SR 101 Ralston in San Mateo County and Route 116 /101 in San Ramon. All these projects were for Caltrans. Two of the tallest structures using the connection system are in California. The tallest structure was of the SR 91 Widening Project in Corona California. Wall 115C was 52 feet tall with 115B in similar height. The third tallest structure was a project for the Nevada Department of Transportation on the I 580 Realignment Project constructed south of Reno, Nevada. One of the MSE walls on this project was 65.0 feet tall.
System Properties
The following properties are reported by the applicant for MSE Plus® Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System.

Soil Reinforcement
Depending on the specific wall design parameters and loading requirements conditions the soil reinforcement mesh sheet can be 2 wires up to a 6-wire mesh sheet. MSE Plus retaining walls system utilizes weld wire reinforcement manufactured in conformance with ASTM A1064 utilizing wire with a minimum yield strength of 75,000 PSI. Longitudinal wire sizes vary from a diameter of W8 (0.08 in²) up to a maximum diameter wire size of W24 (0.24 in²). The reinforcing mesh sheets are fabricated using welds, which are then evaluated in accordance with ASTM A1064. The crossbar spacing of the mesh sheets varies from 6 inches up to 30 inches. The length of each sheet is based upon a minimum of 8 feet or 70% of the design height of the wall.

For the welded wire soil reinforcement, the corrosion measure is hot dipped galvanizing which applied in accordance with ASTM A123. The minimum galvanized coating thickness should be no less than 3.4 mills or 2-ounces per square foot of the reinforcement surface area. The zinc coating provides a sacrificial anode that corrodes while protecting the base metal. Galvanization also assists in preventing the formation of pits in the base metal during the first years of aggressive corrosion.

Soil Reinforcement-Facing Panel Connection Capacity
SSL conducted a supplemental connection test program in July 2023 and testing was witnessed by Twining Labs. This test focused on the strength of the panel connectors embedded into the precast panel with respect to tensile capacity. The test was conducted on a full-scale standard production “Y” panel designed for use with a 6 wire W24 mesh soil reinforcement sheet. Two loads were applied TL1 and TL2. The test loads were determined as follows: 1) One W24 wire = 0.24 in² x 75,000 psi = 18,000 lbs. An unfactored load for a 6-wire sheet would be 108,000 lbs. TL1 uses the AASHTO resistance factor of 0.65 for a nominal TL1 load of 70,200 lbs. and TL2 used a provisional loading factor of 0.70 for a test load 75,600 lbs.

The testing was successful with both applied loads with the panel and connectors showing no deformations or cracking. The test load applied to this panel would equate to a wall design height of approximately of 75 feet with a 5’ x 5’ panel.

AASHTO Section 11.10.6.2.2 Reinforcement Loads at the Connection to Wall Face states: “The nominal tensile load applied to the soil reinforcement connection at the wall face, T₀, shall be equal to the nominal reinforcement tension, Tₓ, for all wall systems regardless of facing and reinforcement type.” The appropriate resistance factor must be applied to the connection capacity per AASHTO Table 11.5.7.1 for inextensible grid systems (i.e., the SSL MSE Plus system) the resistance factor is 0.65 when using the coherent gravity method of analysis.

The current connection design by SSL is based on a laboratory connection testing program discussed above. This testing program was recommended by the IDEA review team during their preliminary evaluation of the MSE Plus retaining wall system. The IDEA review Team aided SSL regarding details of
connection laboratory testing program. The test program details and results are included as an appendix of the MSE Plus IDEA evaluation submittal.

Soil Reinforcement Design Tensile Strength
The design tensile strength of the reinforcement is controlled by the tensile strength of the reinforcement. MSE Plus retaining walls system utilizes weld wire reinforcement manufactured in conformance with ASTM A1064 utilizing wire with a minimum yield strength of 75,000 PSI. The connection between the reinforcement and facing panel is designed as discussed above.

Pullout Design Parameters
Based on the data available for AASHTO specified backfill gradations, SSL recommends the following relationship for determining $f^*$ for SSL welled wire mesh for granular backfills meeting AASHTO requirements for gradation, shear strength and durability:

$F^* = 20 \left( \frac{t}{St} \right)$ at the top of structure

$F^* = 10 \left( \frac{t}{St} \right)$ at a depth of 20'

Where: $t$ = the thickness of the transverse bar and $St$ = transverse bar spacing

Retaining Wall Design
The SSL MSE wall system is designed in accordance with AASHTO and National Highway Institute (NHI) documents and follows conventional design practices of inextensible reinforcement systems and does not include any design innovations.

The design methodology of a MSE Plus wall considers both the external and internal stability of the structure. The design uses a bi-lineal failure plane to design the soil reinforcement. Depending on the transportation agency SSL designs the MSE Plus structure using either the coherent gravity method or the simplified method to determine the vertical internal pressures at each reinforcement layer to determine the reinforcement stresses at each layer. In almost all design conditions SSL uses the industry standard software MSEW+. Information with respect to MSEW+ is available at https://geoprograms.com/msew/.

The internal and external stability design methodology also includes a seismic analysis, if required. SSL follows Section 11.10 of AASHTO.

The design examples in this submittal were performed using MSEW+. Agencies may check designs with a commercial program, with the design properties listed in the table attached to this review report.

System Innovations
The IDEA evaluation concurs with SSL that the MSE Plus® retaining wall system does not contain any innovations.
Reviewer Comments
Following its initial review of the SSL MSE Plus Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System submittal, the review team provided the applicant with several comments and requests for clarification. The applicant has been thorough in its responses and the review team finds that there are no outstanding issues that should be brought to the attention of the transportation agencies. Rather, the agencies are encouraged to rely upon the final SSL MSE Plus Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System submittal for projects where the MSE Plus System is proposed.

Closing
An update technical evaluation should be performed for the SSL MSE Plus Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System in five years (i.e., October 2028) or upon notice that a significant modification of the system has been made. For details regarding update technical evaluations and other guidance for the use of technical evaluations by transportation agencies, go to https://www.geoinstitute.org/special-projects/idea.
Summary Table MSEW+ Input Parameters for MSE Plus® Precast Concrete Facing and Inextensible Reinforcement Retaining Wall System

### Inextensible Soil Reinforcement:

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<th>Data/Type</th>
<th>3W11 x 0.5W11</th>
<th>4W11 x 1.0W11</th>
<th>5W11 x 1.5W11</th>
<th>6W11 x 2.0W11</th>
<th>4W20 x 2.5W11</th>
<th>5W20 x 3.0W11</th>
<th>6W20 x 3.0W11</th>
<th>6W24 x 3.0W11</th>
<th>7W24 x 3.0W11</th>
<th>8W24 x 3.0W11</th>
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<td>17,925</td>
<td>23,903</td>
<td>29,873</td>
<td>35,850</td>
<td>47,475</td>
<td>59,348</td>
<td>71,213</td>
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<td>101,865</td>
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<td>15,368</td>
<td>20,490</td>
<td>25,605</td>
<td>30,728</td>
<td>42,608</td>
<td>53,265</td>
<td>63,915</td>
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<td>Coverage Ratio (R&lt;sub&gt;c&lt;/sub&gt;)</td>
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<td>0.40</td>
<td>0.53</td>
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<td>32</td>
<td>40</td>
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<td>T&lt;sub&gt;long&lt;/sub&gt;-term per unit length of wall (lb/ft) [(F&lt;sub&gt;y&lt;/sub&gt;&lt;sup&gt;•&lt;/sup&gt;A&lt;sub&gt;c&lt;/sub&gt;)*R&lt;sub&gt;c&lt;/sub&gt;/b] (75 years)</td>
<td>3,585</td>
<td>4,781</td>
<td>5,975</td>
<td>7,170</td>
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<td>T&lt;sub&gt;long&lt;/sub&gt;-term per unit length of wall (lb/ft) [(F&lt;sub&gt;y&lt;/sub&gt;&lt;sup&gt;•&lt;/sup&gt;A&lt;sub&gt;c&lt;/sub&gt;)*R&lt;sub&gt;c&lt;/sub&gt;/b] (100 years)</td>
<td>3,074</td>
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<td>5,121</td>
<td>6,146</td>
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<td>15,843</td>
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<td>Pullout resistance factor, F&lt;sup&gt;*&lt;/sup&gt;</td>
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<tr>
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<td>0.35</td>
<td>0.27</td>
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<td>Friction Angle along reinforcement-soil interface, p</td>
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<td></td>
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<td></td>
</tr>
<tr>
<td>Top of Wall</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>Depth of 20 ft below top of wall</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
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<tr>
<td>Scale-effect correction factor, a</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td></td>
</tr>
</tbody>
</table>

| Connection Strengths: | a<sup>•</sup> (lb/ft²) | CRe<sup>a</sup> = F<sub>r</sub>/F<sub>σ</sub> | 3W11 x 0.5W11 | 4W11 x 1.0W11 | 5W11 x 1.5W11 | 6W11 x 2.0W11 | 4W20 x 2.5W11 | 5W20 x 3.0W11 | 6W20 x 3.0W11 | 6W24 x 3.0W11 | 7W24 x 3.0W11 | 8W24 x 3.0W11 |
|-----------------------|-------------------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|
|                       | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
|                       | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |

F<sub>r</sub> = Yield strength of reinforcement
F<sub>σ</sub> = Yield strength of connection between reinforcement and precast panel
A<sub>c</sub> = Area of reinforcement at the design service life (i.e., 75 or 100 years)

* Normal pressure (lb/ft²); † MSEW program term
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HIGHWAY INNOVATIONS, DEVELOPMENTS, ENHANCEMENTS AND ADVANCEMENTS

MECHANICALLY STABILIZED EARTH WALL - MSE Plus™

February 2022
MSE Plus™ Wall SYSTEM

IDEA
HIGHWAY INNOVATIONS, DEVELOPMENTS, ENHANCEMENTS AND ADVANCEMENTS

MECHANICALLY STABILIZED EARTHWALL SYSTEM

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Table of Contents

1.0 ERS COMPONENTS

1.1 Facing Units

1.1.1 Facing Unit Innovations .......................................................... 2
1.1.2 Facing Unit Types ..................................................................... 2
1.1.3 Facing Unit Specification ......................................................... 2
1.1.4 Facing Unit Dimensions ............................................................ 2
1.1.5 Facing Unit Compressive Strength ............................................ 2
1.1.6 Facing Unit Percent Air Entrainment ......................................... 2
1.1.7 Facing Unit Mix Designs ............................................................. 2
1.1.8 Facing Unit Alignment Pin and Bearing Pad ......................... 2
1.1.9 Facing Unit Joint Filter ............................................................. 2
1.1.10 Facing Unit Aesthetic Options ............................................... 2
1.1.11 Facing Unit Alignment Requirements for Curves and Corners 2

1.2 Inextensible Reinforcements

1.2.1 Soil Reinforcing Innovations .................................................. 2
1.2.2 Soil Reinforcing Types .............................................................. 2
1.2.3 Soil Reinforcing Properties ....................................................... 2
1.2.4 Soil Reinforcing Corrosion Protection ...................................... 3
1.2.5 Soil Reinforcing Sacrificial Steel for 75 and 100 Year Life .......... 3
1.2.6 Soil Reinforcing Corrosion Testing .......................................... 3
1.2.7 Soil Reinforcing Dimensional Tolerances ............................... 4
1.2.8 Soil Reinforcing Connection ..................................................... 4
1.2.9 Soil Reinforcing Connection Device Types .............................. 4
1.2.10 Soil Reinforcing Connection Material Specifications ............. 4
1.2.11 Soil Reinforcing Connection Properties ................................. 4
1.2.12 Soil Reinforcing Connection Sacrificial Steel Thickness .......... 4
1.2.13 Soil Reinforcing Connection Corrosion Test ......................... 5
1.2.14 Soil Reinforcing Connection Corrosion Testing .................... 5
1.2.15 Soil Reinforcing Connection Strength and Testing ............... 5
1.2.16 Soil Reinforcing Pullout Testing ............................................ 6
1.2.17 Soil Reinforcing Interface Shear Test .................................... 6

1.3 Other Components
1.3.1 Other component Innovations .................................................. 6
1.3.2 Reinforced Soil Properties ....................................................... 6
1.3.3 ERS Drainage .......................................................................... 6
1.3.4 ERS Coping ............................................................................. 6
1.3.5 ERS Traffic Barrier ................................................................. 7
1.3.6 ERS Slip-Joints ......................................................................... 7

2.0 ERS DESIGN

2.1 Design Methodology

2.1.1 ERS Design Innovations ........................................................ 7
2.1.2 ERS Design AASHTO Methodology ........................................... 7
2.1.3 ERS Design Obstruction Details ............................................... 7

2.2 Design Examples

2.2.1 Traffic Live Load, 2:1 Slope and Abutment Design Examples ....... 8

3.0 CONSTRUCTION

3.1 Construction Procedures

3.1.1 ERS Construction Innovations ................................................ 8
3.1.2 ERS Construction Manual ........................................................ 8
3.1.3 ERS Construction Limitations for Curved Wall Sections .......... 8
3.1.4 ERS Construction for Reinforcement for Curved Sections ...... 8
3.1.5 ERS Construction Measures for the Facing Alignment .......... 8
3.1.6 ERS Construction Procedures for MSE Fill Installation ......... 9
3.1.7 ERS Construction Measures to Control Erosion ................. 9
3.1.8 ERS Construction Experience Qualifications of Installers ....... 9
3.1.9 ERS Construction Procedures to Install Soil Reinforcement .... 9

4.0 QUALITY CONTROL

4.1 Manufacturing

4.1.1 Facing Unit QA/QC Manual ..................................................... 9
4.1.2 Soil Reinforcing QA/QC Manual .............................................. 9
4.1.3 Miscellaneous Component QA/QC ........................................... 9

4.2 Construction

4.2.1 Construction QA/QC ............................................................... 10

5.0 PERFORMANCE

5.1 Performance History

5.1.1 ERS Performance History ....................................................... 10
5.1.2 ERS Oldest Structures ................................................................. 10
5.1.3 ERS Tallest Structures ................................................................. 10
5.1.4 ERS Private and Public Users .................................................. 10

6.0 OTHER

6.1 Other Information ........................................................................... 10
INTRODUCTION

This evaluation document has been organized to follow the IDEA Checklist C4 – “Precast Panel Paired with Inextensible Reinforcement”. The submittal is organized into two sections.

IDEA ERS Submittal Checklist Response

IDEA ERS Section 1 through 5 Appendixes

The submittal package addresses each of the items contained in the Checklist C4. If an item from Checklist C4 is not applicable to the submittal it is noted with then a response will be provided as not applicable. If additional supporting documents are required for our response that information will be included in the Appendix using the same checklist numbering format. The Appendix contains all the sections in Checklist C4.

1.0 ERS COMPONENTS

The primary components for the SSL, MSE Plus™ retaining wall system consist of a facing unit, connection system and inextensible soil reinforcing. The facing unit is segmental concrete panel. The inextensible soil reinforcing consists of a grid type welded wire mesh. The other components that are used with the MSE system include, and are not limited to, compacted select backfill, coping or barriers, bearing pads and joint filter fabric for soil retention.

1.1 FACING UNITS

1.1.1 The MSE Plus retaining wall system does not claim any innovations to the facing system.

1.1.2 The MSE Plus system offers four distinct standard size facia wall panels. They are 5’ x 5’, 5’ x 6’, 5’ x 10’, and 5’ x 12’ system. The panel sizes vary to the requirements of each agency. The system also includes standard half size panels for the bottom of the wall. For the top of the wall extended sloped top panels are fabricated to match the roadway grade and can be up to 7.5’ in height.

1.1.3 Each of the facings listed in Section 1.1.2 are 6-inches in thickness with a standard concrete strength of 4,000 PSI without air entrainment. Of the four size types noted in Section 1.1.2 the panels are designed and fabricated in three structural types - Type A, X and Y. The determination on the panel type chosen for the MSE structure is a function of the wall design height, top of wall geometric configuration and applied loading.
1.1.4 The standard panel design, dimensions, tolerances, and typical steel reinforcement schedule for each type of panel discussed in Sections 1.1.2 and 1.1.3 are provided in Appendix 1.1.4.

1.1.5 The 28-day standard target strength for MSE facia wall panels is 4,000 PSI. The panels in some instances are fabricated to higher strengths depending on the DOT agency. The MSE wall facia panels are also fabricated using higher concrete strengths when exceptionally tall walls are designed or when the facing panels are subject to extreme impact loading.

1.1.6 The target percentage of air content for the concrete is specified by the DOT agency and the amount of air content can vary from state to state.

1.1.7 The mix design of the MSE fascia panels varies from state to state. Some agencies require a higher minimum 28-day concrete compressive strength, the use of fly ash, to include slag or air entrainment. SSL uses a different producer for each state and each local producer is familiar with their agency’s respective mix design requirements.

1.1.8 The SSL MSE Plus system does not incorporate the use shear pins or alignment devices for wall construction. The system does use two bearing pads at the top of each 5’ or 6’ wide standard size panel to control the horizontal panel joint width. Larger 10’ and 12’ width panels will use 3 or 4 bearing pads per panel joint. The bearing pads are fabricated using an injection molded polypropylene copolymer. See Appendix 1.1.8 for bearing pad details.

1.1.9 All panel joints are covered using a minimum 1-foot wide filter cloth to prevent the migration of the select fill material through the panel joints. SSL specifies a Mirafi FW 402 or equal type geotextile, which is a woven high tenacity monofilament yarn. See Appendix 1.1.9 for the filter cloth specifications.

1.1.10 Many aesthetic possibilities are available. From fractured fin, seashells, rock etc. Examples of the various finishes are provided in Appendix 1.1.10.

1.1.11 Facing units using a 5 and 6 foot standard width and a ¾” panel joints are limited to a 50 foot inside radius.

1.2 INEXTENSIBLE REINFORCEMENT

1.2.1 The MSE Plus system does not use soil reinforcement that would be considered innovative.

1.2.2 The SSL MSE Plus retaining wall system utilizes only one type of welded wire mesh soil
reinforcement. Depending on the specific wall design parameters and loading requirements conditions the soil reinforcement mesh sheet can be 2 wires up to a 6 wire mesh sheet. The crossbar spacing of the mesh sheets varies from 6 inches up to 30 inches. The length of each sheet is based upon a minimum of 8 feet or 70% of the design height of the wall as defined by AASHTO.

1.2.3 The SSL MSE Plus retaining walls system utilizes weld wire reinforcement manufactured in conformance with ASTM A1064 utilizing wire with a minimum yield strength of 75,000 PSI. Longitudinal wire sizes vary from a diameter of W8 (0.08 in²) up to a maximum diameter wire size of W24 (0.24 in²). The reinforcing mesh sheets are fabricated using welds, which are then evaluated in accordance with ASTM A1064. A copy of the ASTM specification is included in Appendix 1.2.3.

1.2.4 For the welded wire soil reinforcement as described in Section 1.2.3 the corrosion measure is hot dipped galvanizing which applied in accordance with ASTM A123. The minimum galvanized coating thickness should be no less than 3.4 mills or 2-ounces per square foot of the reinforcement surface area. The zinc coating provides a sacrificial anode that corrodes while protecting the base metal. Galvanization also assists in preventing the formation of pits in the base metal during the first years of aggressive corrosion.

1.2.5 The sacrificial steel thickness for the weld wire mesh reinforcement is determined as follows for W11 and W24 wire sizes.

**75 Year Design Life Corrosion Reduction Analysis**

Galvanizing coverage ASTM A123 = 2 ounces per square foot
Rate of corrosion for zinc = 15₂μm per year – first 2 years
4₂μm per year – thereafter
For residual carbon steel = 12₂μm per year
Galvanizing thickness = (2 oz x (1728 /16))/(440 x 144) = 0.00341 inches = 3.41 mils = 86.59₂μm
First 2 years = 2 x 15₂μm = 30₂μm per year
Number of years of galvanized coating = 2 + ((86.59 – 30)₂μm)/ 4₂μm = 16.15 years
Steel loss = (75 – 16.14) years x 12₂μm / year = 706.32₂μm = 0.0278 inches

W11 wire diameter = 0.374 inches
W11 effective wire diameter = (0.374 – (2 x 0.0278)) = 0.318 in²
W24 wire diameter = 0.553 inches
W24 effective wire diameter = (0.553 – (2 x 0.0278)) = 0.497 in²

100 Year Design Life Corrosion Reduction Analysis

Steel loss = (100 – 16.14) years x 12μm / year = 1,002.32μm = 0.0396 inches

W11 effective diameter = 0.374 – (2 x 0.0396) = 0.295 in²
W24 effective diameter = (0.553 – (2 x 0.0396)) = 0.474 in²

1.2.6 Corrosion testing has been performed by others which is well documented in past publications. SSL follows the requirements for galvanized coating and sacrificial steel rates provided and recommended in AASHTO and FHWA NHI 10-024.

1.2.7 The SSL MSE Plus system utilizes weld wire mesh fabricated in accordance with the tolerances outlined in ASTM A1064. Drawings for reinforcement types with tolerances are available in Appendix 1.1.4.

1.2.8 The connection system is comprised of three components. The first is a pair of wire loops are cast into the panel to form a clevis, the second is where one end of the soil reinforcement is bent 90 degrees to form a 90 degree loop. The looped end of the soil reinforcement is inserted in between the two loops cast into the panel which forms the clevis. The third is a pin is inserted into the soil reinforcement loop and clevis to complete the connection of the soil reinforcement to the panel. Complete details are shown in Appendix 1.2.8.

1.2.9 The MSE Plus system uses only one type of connection device paired to up to 6 connectors per layer. That connection system is described in Section 1.2.8.

1.2.10 The connection system is fabricated in accordance with ASTM A 1064. The embed loops are fabricated with material with a minimum yield strength of 75,000 PSI.

1.2.11 The connection system components are galvanized in accordance with NHI 10-024 with a minimum coating thickness of 3.4 mils. The coating is applied in accordance with ASTM A123.

1.2.12 The sacrificial steel thickness is determined as follows for both 75 and 100 years for the connection device is as follows.

75 Year Design Life Corrosion Reduction Analysis
Galvanizing coverage ASTM A123 = 2 ounces per square foot
Rate of corrosion for zinc = 15 $\mu$m per year – first 2 years
4 $\mu$m per year – thereafter
For residual carbon steel = 12 $\mu$m per year
Galvanizing thickness = \((2 \text{ oz} \times (1728 /16))/(440 \times 144) = 0.00341\) inches = 3.41 mils = 86.59 $\mu$m
First 2 years = \(2 \times 15\) $\mu$m = 30 $\mu$m per year
Number of years of galvanized coating = \(2 + ((86.59 - 30)/4)\) years = 16.15 years
Steel loss = \((75 - 16.14)\) years \(\times 12\) $\mu$m / year = 706.32 $\mu$m = 0.0278 inches

W11 wire diameter = 0.374 inches
Effective area for connection = \((0.374 - (2 \times 0.0278)) = ((0.318/2)^2 \times 3.14) = 0.080\) in² x 4 = 0.318 in²

W24 wire diameter = 0.553 inches
W24 effective area = \((0.553 - (2 \times 0.0278)) = (0.497/2)^2 \times 3.14) = 0.194\) in²
Connection area vs. wire Area = 0.318 in² / 0.194 in² = 1.64. Panel connector area greater than W24 wire.

100 Year Design Life Corrosion Reduction Analysis
Steel loss = \((100 - 16.14)\) years \(\times 12\) $\mu$m / year = 1,002.32 $\mu$m = 0.0396 inches

W11 effective area for connection = \((0.374 - (2 \times 0.0396)) = ((0.295/2)^2 \times 3.14) = 0.275\) in²
W24 effective area = \((0.553 - (2 \times 0.0396)) = (0.474/2)^2 \times 3.14) = 0.176\) in²
Connection area vs. wire area = 0.275 in² / 0.176 in² = 1.56. Panel connector area greater than W24 wire.

1.2.13 Corrosion tests have not been performed on this connection system by SSL. The performance of the system is based upon corrosion rates recommended in AASHTO and NHI 10 - 024.

1.2.14 See Appendix 1.2.8 and 1.2.14 for drawings with the dimensional tolerances for the soil reinforcement and panel connector, respectively.

1.2.15 SSL developed a panel connection system for MSE retaining walls, which uses welded wire soil reinforcement with precast concrete fascia panels. Connection testing was performed by SSL in November 2013, and this testing focused on the capacity of the connection bends in the individual wires to ensure loading of 100% of yield strength of the wire could be achieved.
SSL conducted a supplemental connection test program in July 2023, and testing was witnessed by Twining Labs. This test focused on the strength of the panel connectors embedded into the precast panel with respect to tensile capacity. The test was conducted on a full-scale standard production “Y” panel designed for use with a 6 wire W24 mesh soil reinforcement sheet. Two loads were applied TL1 and TL2. The test loads were determined as follows: 1) One W24 wire = 0.24 in² x 75,000 psi = 18,000 lbs. An unfactored load for a 6-wire sheet would be 108,000 lbs. TL1 uses the AASHTO resistance factor of 0.65 for a nominal TL1 load of 70,200 lbs. and TL2 used a provisional loading factor of 0.70 for a test load 75,600 lbs.

The testing was successful with both applied loads with the panel and connectors showing no deformations or cracking. The test load applied to this panel would equate to a wall design height of approximately of 75 feet with a 5’ x 5’ panel.

AASHTO Section 11.10.6.2.2 Reinforcement Loads at the Connection to Wall Face states: “The nominal tensile load applied to the soil reinforcement connection at the wall face, TO, shall be equal to the nominal reinforcement tension, Tmax, for all wall systems regardless of facing and reinforcement type.” The appropriate resistance factor must be applied to the connection capacity per AASHTO Table 11.5.7.1 for inextensible grid systems (i.e., the SSL MSE Plus system) the resistance factor is 0.65 when using the coherent gravity method of analysis.

1.2.16 SSL has not performed pullout tests and uses the default values provided in NHI-10-024.
1.2.17 This section only applies for geosynthetic reinforcement and inextensible systems.

1.3 **OTHER COMPONENTS**

1.3.1 The wall system does not contain an innovation with respect to any wall components.

1.3.2 This section does not apply to our MSE system.

1.3.3 Our MSE Plus system utilizes drainage methods such as underdrains or drainage structures at the roadway surface to collect and distribute runoff away from the MSE structure. In the majority of the MSE wall projects the transportation agency specifies their drainage system or systems designed by the agencies drainage engineers. Please see Appendix 1.3.3 for a standard drainage structure used at the roadway surface and a standard underdrain system.

1.3.4 Our system uses both precast and cast in place styles of coping. Our precast coping comes in 10’ and 12’ lengths and are usually 24 inches tall using a panel embedment of 12 inches. The cast in place coping is usually installed in continuous lengths with a crack control joints at 30’ intervals. The overall dimensions can be the same as precast coping. Both precast and cast in place copings can vary for each agency. Typical coping details can be found in Appendix 1.3.4.

1.3.5 Most transportation agencies integrate the cast in place traffic barrier with a moment slab for roadway conditions. Please see Appendix 1.3.5 for a typical standard example. Each agency will have a slight variation of what is presented in this Appendix.

1.3.6 The slip joint is a “T” shaped panel in cross section with wide flanges for the standard panels to slip in behind the flanges. The wide flanges allows standard panel to move freely within the joint in the event differential settlement exceeds 1/100. The height of the slip joint panels is in the same increments as the standard MSE wall panels. Please see Appendix 1.3.6 for additional details and dimensions.

2.1 **ERS DESIGN**

2.1.1 The SSL MSE wall system is designed in accordance with AASHTO and NHI documents and follows conventional design practices of inextensible reinforcement systems and does not include any design innovations.

2.1.2 The internal and external stability design methodology also includes a seismic analysis, if required, for the MSE structures. SSL follows Section 11.10 of AASHTO as well as Chapter 4
Section 4.4.7 of NHI-10-024 for the design of internal and external stability. Since SSL utilizes an inextensible metallic mesh style reinforcement, we recognize the design must incorporate a bi-lineal failure plane to design the soil reinforcement. Depending on the transportation agency we design the MSE structure using either the coherent gravity method or the simplified method to determine the vertical internal pressures at each reinforcement layer to determine the reinforcement stresses at each layer. In almost all design conditions SSL uses the industry standard software MSEW+. Information with respect to MSEW+ is available at https://geoprograms.com/msew/.

2.1.3 For vertical obstructions in the MSE reinforced zone SSL follows the recommendation provided in NHI-10-024 Section 5.4.2. by either shifting or offsetting the reinforcement to ends of the panel so a reinforcement sheet is on each side of the vertical obstruction as recommended in Alternative 1. In the event the detail shown in Alternative 1 and not feasible we follow the guidance recommended shown in Alternative 2 by using narrow reinforcements in conjunction with an angle frame. In both instances the obstruction is centered over a panel joint such as a drainage structure. For situations where the vertical obstruction is large such as a cast in place bridge column SSL incorporates a structural frame as recommended in Alternative 3. The photograph shown in NHI documents of Figure 5-19(a) is a SSL design detail utilizing a structural frame. For horizontal obstructions in the MSE reinforced zone SSL follows the recommendation in NHI-10-024 Section 5.4.3 whereby SSL deviates the reinforcement no more than 15 degrees either above or below the obstruction as shown in Figure 5-21. SSL will also recommend to the Owner to move the obstruction outside the reinforce zone, if possible, to ease construction and to facilitate future maintenance for the utilities if necessary. Plan details for vertical and horizontal obstructions are provided in Appendix 2.1.3.

2.2 DESIGN EXAMPLE

2.2.1 We have included three design examples which are for a MSE roadway MSE wall with 250 PSF live load, a MSE wall with a 2:1 infinite slope and a pile supported bridge MSE abutment structure. The MSEW+ calculations for all three design examples are included in Appendix 2.2.1.

3.1 CONSTRUCTION PROCEDURES

3.1.1 The SSL MSE Plus does contain an innovative construction procedure. Our MSE walls system
follows conventional construction procedures for inextensible structures.

3.1.2 The SSL MSE Plus Installation Manual is included in Appendix 3.1.2.

3.1.3 For inside curves in our wall system, the curves are developed by a series of panel chords, which are 5’ foot in width. Based upon a ¾” panel joint we recommend a 50 foot minimum radius. For outside curves in our wall system, we recommend a 50 foot minimum radius and special consideration to protect the filter cloth from UV exposure is a major consideration.

3.1.4 Since metallic systems designs do not consider coverage ratios for the design for internal stability overlapping of reinforcement is typically not a design issue. The SSL MSE Plus wall system utilizes a bar mat system whereby the pullout capacity of the mat is developed using the interaction between the soil and the bar mats crossbars to develop the passive resistance. Since bar mat systems do not rely on frictional resistance to develop pullout capacity, vertical separation between layers in overlapping situations should not be required.

3.1.5 The procedures to maintain the vertical and horizontal alignment of our system are described in detail in our Installation Manual in Appendix 3.1.2 beginning on page 17.

3.1.6 The procedures to install soil in the reinforced zone of our system are described in detail in our Installation Manual in Appendix 3.1.2 beginning on page 17.

3.1.7 During construction we recommend that after the end of each working day the grade of the backfill at the top of the wall be sloped downward away from the back of the wall panels to allow drainage of any water away from the wall to prevent erosion. For the front face of the wall, we recommend finished grade is sloped away from the front face to prevent erosion and the collection of water.

3.1.8 The experience and qualifications of the MSE wall contractor is a requirement usually specified by the transportation agency. Some agencies’ specifications require little or no experience. But most agencies do request that the MSE wall manufacturer provide jobsite assistance for varying lengths of time. Some states such as Arizona stipulate specific experience requirements from management as well as the jobsite personnel and labor staff.

3.1.9 The procedure to install the MSE select fill in the reinforced zone is described in our Installation Manual in Appendix 3.1.2 beginning on page 17.
4.1 MANUFACTURING

4.1.1 The precast concrete fascia panels are manufactured following the precast concrete manufacturer’s Quality Control Plan for each state agency. An example of a Precast Quality Control Plan also known as a PCQCP is presented in Appendix 4.1.1. This PCQCP is from Precast Solutions Incorporate for the INDOT and is prepared with respect to the requirements of the National Precast Concrete Association also known as NPCA. Other agencies require similar plans for approvals for each precaster by each agency, the NPCA or both.

4.1.2 The weld wire mesh soil reinforcements are manufactured following SSL’s Quality Control Manual. This manual is available in Appendix 4.1.2.

4.1.3 SSL MSE Plus system does not utilize shear or alignment devices. The bearing pads used at the horizontal panel joints are tested with the results evaluated for each batch of pads manufactured. Bearing pads are evaluated under an incremental compressive load up to 40,000 pounds. During the compressive loading, the vertical displacement is recorded to develop a load vs. displacement curve. Upon completion of the compressive testing the bearing pad is measured for dimensional changes and photographed. Please see Appendix 4.1.3 for a sample test report. The connection devices are manufactured in accordance with the Quality Control Manual in Appendix 4.1.2.

4.2 CONSTRUCTION

4.2.1 The quality control measures are not described in the construction manual with respect to assembling the MSE structure. On page 36 of the Construction Manual in Appendix 4.1.2 SSL provides a checklist of items that should be addressed during wall construction. The backfill and the preparation of the MSE structure foundation are performed in accordance with the transportation agency recommendation and their specifications. The contractor’s work is inspected by the transportation agency or the Engineer of Record for the project.

5.1 PERFORMANCE HISTORY

5.1.1 Our MSE Plus™ system was developed in 1997 and SSL received our first approval in California by The California Department of Transportation or Caltrans. With the completion of our HITEC evaluation in 1999 SSL received approvals by most state departments of transportation agencies. The connection system evaluated in the 1999 HITEC reports was for our internal
connection. In 2003 SSL revised the internal connection system to a more conventional external system using clevis embed connectors cast into the concrete panel by then joining the 90 degree loop in soil reinforcement to the clevis using a connection pin. This connection was approved by Caltrans in 2003 and once again in 2012 as their connection testing requirements were modified for their LRFD approval. Testing was performed for Caltrans in December 2012 SSL received LRFD approval shortly thereafter.

5.1.2 Three of the oldest structures using this new connection system are in California. The projects are the SR 22 Project in Orange County, SR 101 Ralston in San Mateo County and Route 116 / 101 in San Ramon. All these projects were for Caltrans.

5.1.3 Two of the tallest structures using the connection system are in California. The tallest structure was of the SR 91 Widening Project in Corona California. Wall 115C was 52 feet tall with 115B in similar height. The third tallest structure was a project for the Nevada Department of Transportation on the I 580 Realignment Project constructed south of Reno, Nevada. One of the MSE walls was on this project was 65.0 feet tall.

5.1.4 Provide a list of private and public sector users who have approved the use of the system. Also provide contact information for a person at the user agency who may be contacted regarding the wall system’s performance. Please see Appendix 5.1.4 for a list of users for the SSL MSE Plus retaining wall system.

6.0 OTHER INFORMATION –

No additional information is provided for Section 6.
Appendix

IDEA
HIGHWAY INNOVATIONS, DEVELOPMENTS, ENHANCEMENTS AND ADVANCEMENTS

MSE Plus™ Wall System
# Table of Contents

## 1 ERS COMPONENTS

1.1 Facing Units

   - 1.1.4 Facing Unit Dimensions
   - 1.1.5 Facing Unit Bearing Pad
   - 1.1.9 Facing Filter Cloth Specifications
   - 1.1.10 Facing Unit Aesthetic Options

1.2 Inextensible Reinforcements

   - 1.2.3 Soil Reinforcing Properties
   - 1.2.8 Soil Reinforcing Connection
   - 1.2.14 Soil Reinforcing Connection Dimensions
   - 1.2.15 Soil Reinforcing Connection Strength and Testing

1.3 Other Components

   - 1.3.3 ERS Drainage
   - 1.3.4 ERS Coping
   - 1.3.5 ERS Traffic Barrier
   - 1.3.6 Slip Joint Details

## 2 ERS DESIGN

2.1 Design Methodology

   - 2.1.3 ERS Obstruction Design Detail

2.2 Design Example

   - 2.2.1 Design Problems – 3 Examples

## 3 CONSTRUCTION

3.1 Construction Procedures

   - 3.1.2 ERS Construction Manual

## 4 QUALITY CONTROL

4.1 Manufacturing

   - 4.1.1 Facing Unit QA/QC
   - 4.1.2 Soil Reinforcing QA/QC
5 PERFORMANCE

5.1 Performance History

5.1.4 ERS System Approvals by User
Section 1: ERS Components
1.1

Facing Units
1.1.4 Facing Unit Dimensions
5’ x 5’ STANDARD PANELS
STANDARD "A" PANEL  
SHOWN FROM BACK FACE

SECTION A-A

NOTE:
TOTAL PANEL REINFORCEMENT SHOWN. ALTERNATIVE REINFORCEMENT MAY BE USED AS LONG AS MINIMUM CROSS SECTIONAL AREA ARE EQUIVALENT.
18" WAX SPACING BETWEEN BARS

THIS DRAWING CONTAINS INFORMATION PROPRIETARY TO DOL AND IS INTENDED FOR THE PROJECT SHOWN ONLY. MSRS. ANY INFORMATION MAY BE TRANSMITTED TO ANY PERSON FOR ANY PURPOSE WHATSOEVER WITHOUT CONSENT OF DOL.

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SPRING CONSTANT
Modulus of Sub-Grade Reaction and Spring Constant Check
From Principles of Foundation Engineering; Braja M. Das 8th Edition:

<table>
<thead>
<tr>
<th>Soil type</th>
<th>MN/m³</th>
<th>lb/in³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry or moist sand:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>8–25</td>
<td>30–90</td>
</tr>
<tr>
<td>Medium</td>
<td>25–125</td>
<td>90–450</td>
</tr>
<tr>
<td>Dense</td>
<td>125–375</td>
<td>450–1350</td>
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<td>Saturated sand:</td>
<td></td>
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<tr>
<td>Loose</td>
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<tr>
<td>Very stiff</td>
<td>25–50</td>
<td>90–185</td>
</tr>
<tr>
<td>Hard</td>
<td>&gt;50</td>
<td>&gt;185</td>
</tr>
</tbody>
</table>

\[
k_{0.3}(k_1) = 1350 \text{ lb/in}^3 \quad \text{(is selected)}
\]

( dense Sand, value is conservative considering no reinforcement is included )

Adjust to panel 5' x 10' (1.5m x 3m)

**Foundations on Sandy Soils**
For foundations on sandy soils,

\[
k = k_{0.3} \left( \frac{B + 0.3}{2B} \right)^2
\]

(8.45)

where \(k_{0.3}\) and \(k\) = coefficients of subgrade reaction of foundations measuring 0.3 m x 0.3 m and \(B\) (m) x \(B\) (m), respectively (unit is kN/m²).

In English units, Eq. (8.45) may be expressed as

\[
k = k_{1.5} \left( \frac{B + 1}{2B} \right)^2
\]

(8.46)

where \(k_{1.5}\) and \(k\) = coefficients of subgrade reaction of foundations measuring 1 ft x 1 ft and \(B\) (ft) x \(B\) (ft), respectively (unit is lb/in²),

\[
k_{1.5} = k_{0.3} \cdot \left( \frac{B + 0.3}{2B} \right)^2
\]

\[
k_{1.5} = 1350 \times \left( \frac{1.5 + 0.3}{2 \times 1.5} \right)^2
\]

\[
k_{1.5} = 486 \text{ lb/in}^3
\]

\[
k_{1.5 \times 3.0} = k_{1.5} \cdot \left( \frac{1 + 3.0}{1.5} \right)^2
\]

\[
k_{1.5 \times 3.0} = 839.755 \text{ kip/ft}^3
\]

\[
k_{1.5 \times 3.0} = 486 \text{ lb/in}^3
\]

\[
k_{1.5 \times 3.0} = 839.755 \text{ kip/ft}^3
\]
10-5 MODULUS OF SUBGRADE REACTION $k_s$
FOR MATS AND PLATES

All three discrete element methods given in this chapter for mats/plates use the modulus of subgrade reaction $k_s$ to support the plate. The modulus $k_s$ is used to compute node springs based on the contributing plan area of an element to any node as in Fig. 10-5. From the figure we see the following:

<table>
<thead>
<tr>
<th>Node</th>
<th>Contributing area</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (corner)</td>
<td>$\frac{1}{4}$ of rectangle abde</td>
</tr>
<tr>
<td>2 (side)</td>
<td>$\frac{1}{4}$ of abde + $\frac{1}{4}$ of bcef</td>
</tr>
<tr>
<td>3 (interior)</td>
<td>$\frac{1}{4}$ of each rectangle framing to a common node (as node 3)</td>
</tr>
</tbody>
</table>

For a triangle one should arbitrarily use one-third of the triangle area to any corner node. For these area contributions the fraction of $k_s$ node resistance from any element is

$$K_i = k_s, \text{ kN/m}^3, \times \text{Area, m}^2 = \text{units of kN/m (or kips/ft in Fps)}$$

Since this computation gives units of a "spring" it is common to call the effect a node spring.

In this form the springs are independent of each other, the system of springs supporting the plate is termed a "Winkler" foundation, and the springs are uncoupled. Uncoupling means that the deflection of any spring is not influenced by adjacent springs.

**Spring Constant**

Note Spacing: 1ft x 1ft

Tributary Area: 1' x 1' = 1SF

$Ka = k \times a$

$Ka = 839.755 \text{ kip/ft} \quad (\text{Input to Staad Pro})$
MAXIMUM BENDING MOMENT
AND SHEAR
**Typical Panel Type (mesh 3W11), Panel Thickness 6", 5'x5'**

**Point load Input Force in panel for Staad Pro input**

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
</tr>
<tr>
<td>P1</td>
<td>0</td>
</tr>
<tr>
<td>P2</td>
<td>-0.33</td>
</tr>
<tr>
<td>P3</td>
<td>0.33</td>
</tr>
<tr>
<td>P4</td>
<td>-0.33</td>
</tr>
<tr>
<td>P5</td>
<td>0.33</td>
</tr>
<tr>
<td>P6</td>
<td>0</td>
</tr>
</tbody>
</table>

**Load to panel based on Bar Mat Capacity per Bar Mat Point**

Bar Mat Used:
W11 - 75 Years

- \( A = 0.0797 \text{ in}^2 \)
- \( F_y = 75 \text{ ksi} \)
- \( P = 0.65 \times A \times F_y \)
- \( = 3.885 \text{ Kips (per wire point)} \)

**ATTACHMENT BY No. WIRES**
(Standard Panels Only)
SOX (local)
N/mm²

<table>
<thead>
<tr>
<th>Value</th>
<th>Color</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;= -0.119</td>
<td>Red</td>
</tr>
<tr>
<td>-0.104</td>
<td>Orange</td>
</tr>
<tr>
<td>-0.089</td>
<td>Yellow</td>
</tr>
<tr>
<td>-0.075</td>
<td>Green</td>
</tr>
<tr>
<td>-0.060</td>
<td>Blue</td>
</tr>
<tr>
<td>-0.045</td>
<td>Purple</td>
</tr>
<tr>
<td>-0.030</td>
<td>Pink</td>
</tr>
<tr>
<td>-0.015</td>
<td>White</td>
</tr>
<tr>
<td>0</td>
<td>Black</td>
</tr>
<tr>
<td>0.015</td>
<td>Red</td>
</tr>
<tr>
<td>0.030</td>
<td>Orange</td>
</tr>
<tr>
<td>0.045</td>
<td>Yellow</td>
</tr>
<tr>
<td>0.060</td>
<td>Green</td>
</tr>
<tr>
<td>0.075</td>
<td>Blue</td>
</tr>
<tr>
<td>0.089</td>
<td>Purple</td>
</tr>
<tr>
<td>0.104</td>
<td>Pink</td>
</tr>
<tr>
<td>&gt;= 0.119</td>
<td>White</td>
</tr>
</tbody>
</table>

Load 1

STAAD.Pro for Windows 20.07.04.12
SQY (local)
N/mm²

<= -0.118
-0.103
-0.088
-0.074
-0.059
-0.044
-0.029
-0.015
0
0.015
0.029
0.044
0.059
0.074
0.088
0.103
>= 0.118
1. STAAD SPACE

INPUT FILE: Panel Check - 5x5 type A (3N11).STD
2. START JOB INFORMATION
3. ENGINEER DATE 04-MAR-21
4. JOB CLIENT SUL
5. ENGINEER NAME CH
6. JOB REV PANEL CHECK
7. END JOB INFORMATION
8. INPUT WIDTH 79
9. UNIT FEET INCH
10. JOINT COORDINATES
  11. 1 0 0 0 2 1 0 0 3 2 0 0 4 3 0 0 5 4 0 0 6 5 0 0 7 6 0 0 8 7 0 0 9 8 0 0 10 9 0 0
  12. 11 0 0 -1 12 0 0 -1 13 0 0 -1 14 0 0 -1 15 0 0 -1 16 0 0 -1 17 0 0 -1 18 0 0 -1
  19. 20 0 0 -1 21 0 0 -1 22 0 0 -1 23 0 0 -1 24 0 0 -1 25 0 0 -1 26 0 0 -1 27 0 0 -1
  28. 29 0 0 -1 30 0 0 -1 31 0 0 -1 32 0 0 -1 33 0 0 -1 34 0 0 -1 35 0 0 -1 36 0 0 -1
  37. 38 0 0 -1 39 0 0 -1 40 0 0 -1 41 0 0 -1 42 0 0 -1 43 0 0 -1 44 0 0 -1 45 0 0 -1
  46. ELEMENT INCIDENCES SHELL
  47. 1 1 10 11 1 2 10 19 20 1 1 19 29 30 20 4 2 11 12 2 5 11 20 21 12 18 6 20 29 30 21 7 3 12 13 4 8 12 21 22 13 21 30 31 22 10 4 13 14 5
  48. 11 13 22 23 14 12 22 31 22 31 22 13 5 14 15 6 14 13 23 24 15 15 23 32 33 24 20 31 28 45 29 29 29 46 47 30 33 30 47 48 31 34 31 48 49 32 35 32 49 50 33
  49. 31 43 56 57 64 42 46 57 58 67 43 47 58 59 48 44 48 59 60 49 45 49 60 61 50
  50. 22. ELEMENT PROPERTY
  51. 23. 1 TO 15 30 TO 35 41 TO 45 THICKNESS 0.5
  52. 24. DEFINE MATERIAL
  53. 25. ISOTROPIC CONCRETE
  54. 26. 5. 614304
  55. 27. POISSON 0.17
  56. 28. DENSITY 2.4
  57. 29. ALPRA 12.075
  58. 30. DAMP 0.05
  59. 31. END DEFINE MATERIAL
  60. 32. CONSTANTS
  61. 33. MATERIAL CONCRETE ALL
  62. 34. SUPPORTS
  63. 35. 1 TO 6 10 TO 15 19 TO 24 28 28 33 45 TO 50 56 TO 60
  64. 36. 61 FIXED BUT KFX 839.76 KFY 839.76 KFX 839.76 KMY 1 KNY 1 KNE 1
  65. 37. LOAD 1 LOADS TYPE PUSH TITLE WIND STRENGTH
  38. ELEMENT LOAD
  39. 39. 33 PR GY -3.885 0.25 0
  40. 33 PR GY -3.885 0.25 -0.33
  41. 34. 33 PR GY -3.885 0.25 0.33
  42. 34. 33 PR GY -3.885 0.25 -0.33
  43. 35. 33 PR GY -3.885 0.25 0.33
  44. 35. 33 PR GY -3.885 0.25 0
  45. 36. PERFORM ANALYSIS
  46. PROBLEM STATISTICS

NUMBER OF JOINTS/MEMBER+ELEMENTS+SUPPORTS = 36/ 25/ 36

SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER
ORIgINAL/FINAL BAND-WIDTH = 7/ 7/ 48 DOF
TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 216
SIZE OF STIFFNESS MATRIX = 11 DOUBLE KILO-WORDS
REQUESTED/DISK SPACE = 12.2/ 263455.4 MB

46. FINISH
Typical Panel Type (mesh 4W11), Panel Thickness 6", 5’x5’

Point load input force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1’x1’</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>0  0.25</td>
</tr>
<tr>
<td>P2</td>
<td>-0.33  0.25</td>
</tr>
<tr>
<td>P3</td>
<td>0.33  0.25</td>
</tr>
<tr>
<td>P4</td>
<td>0  0.25</td>
</tr>
<tr>
<td>P5</td>
<td>0 -0.25</td>
</tr>
<tr>
<td>P6</td>
<td>-0.33 -0.25</td>
</tr>
<tr>
<td>P7</td>
<td>0.33 -0.25</td>
</tr>
<tr>
<td>P8</td>
<td>0 -0.25</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point
Bar Mat Used:
W11 - 75 Years

\[
A = 0.0797 \text{ in}^2 \\
F_y = 75 \text{ ksi} \\
P = 0.65 \times A \times F_y = 3.885 \text{ Kips (per wire point)}
\]

ATTACHMENT BY NO. WIRES
(standard panels only)
Panel Check - 5x5 type A (4W11).std

Load 1

5.00ft

5.00ft
MX (local) kNm/m

- <= 1.47
- 1.63
- 1.78
- 1.94
- 2.1
- 2.25
- 2.41
- 2.57
- 2.73
- 2.88
- 3.04
- 3.2
- 3.35
- 3.51
- 3.67
- 3.82
- >= 3.98
MY (local) kNm/m

- <= 1.48
- 1.88
- 2.27
- 2.67
- 3.06
- 3.45
- 3.85
- 4.24
- 4.64
- 5.03
- 5.43
- 5.82
- 6.21
- 6.61
- 7
- 7.4
- >= 7.79
SQX (local)
N/mm²

-0.145
-0.127
-0.109
-0.090
-0.072
-0.054
-0.036
-0.018
0
0.018
0.036
0.054
0.072
0.090
0.109
0.127
0.145

Load 1

5.00ft 5.00ft
Software licensed to Optimal Engineering Support

Job Title

Client

SSL

Job No

1

Part

Ref

Panel Check

By

CH

Date

04-Mar-21

Chd

06-Mar-21 06:54

File

Panel Check - 5x5 type A

Print Time/Date: 06/03/2021 06:59

Print Run 1 of 1

STAAD.Pro for Windows 20.07.04.12
STAAD SPACE

INPUT FILE: Panel Check - 5x5 type A (4W11).STD
2. START JOB INFORMATION
3. ENGINEER DATE 04-MAR-21
4. JOB CLIENT SEL
5. ENGINEER NAME CH
6. JOB REV PANEL CHECK
7. END JOB INFORMATION
8. INPUT WIDTH 79
9. UNIT PEET KIP
10. JOINT COORDINATES
11. 1 0 0 0; 2 1 0 0; 3 2 0 0; 4 3 0 0; 5 4 0 0; 6 5 0 0; 10 0 0 -1; 11 1 0 -1.
12. 12 2 0 -1; 13 3 0 -1; 14 4 0 -1; 15 5 0 -1; 19 0 0 -2; 20 1 0 -2; 21 2 0 -2.
13. 22 3 0 -2; 23 4 0 -2; 24 5 0 -2; 28 0 0 -3; 29 1 0 -3; 30 2 0 -3; 31 3 0 -3.
14. 32 4 0 -3; 33 5 0 -3; 45 0 0 -4; 46 1 0 -4; 47 2 0 -4; 48 3 0 -4; 49 4 0 -4.
15. 50 5 0 -4; 51 6 0 -4; 52 7 1 0 -5; 53 8 2 0 -5; 54 9 3 0 -5; 55 10 0 0 -5.
16. ELEMENT INCIDENCES SHELL
17. 1 1 10 11 2 10 19 20 11; 5 19 29 20 20; 4 2 11 12 9 5 11 20 21 12.
18. 6 20 29 30 21; 7 3 12 13 4; 8 12 21 22 13; 9 21 30 31 22; 10 4 13 14 5.
20. 31 28 45 46 29; 32 29 46 47 30; 33 30 47 48 31; 34 31 48 49 32; 35 32 49 50 33.
21. 41 40 51 52 46; 42 46 57 58 47; 43 47 58 59 48; 44 48 59 60 49; 45 49 60 51 50.
22. ELEMENT PROPERTY
23. 1 15 21 20 35 41 45 THICKNESS 0.5
24. DEFINE MATERIAL START
25. ISOTROPIC CONCRETE
26. 6.414304
27. POISSON 0.17
28. DENSITY 0.15
29. ALPHA 15-05
30. DAMP 0.05
31. END DEFINE MATERIAL
32. CONSTANTS
33. MATERIAL CONCRETE ALL
34. SUPPORTS
35. 1 2 6 10 14 19 TO 24 28 30 33 45 TO 50 56 TO 60 -
36. 61 FIXED BUT KFX 839.76 KFY 839.76 KFX 839.76 KMX 1 KNY 1 KMX 1
37. LOAD 1 LOADTYPE PUSH TITHE WINE STRENGTH
38. ELEMENT LOAD
39. 33 33 839.76 0.25 0
40. 33 33 839.76 0.25 0.3
Typical Panel Type (mesh 5W11), Panel Thickness 6", 5'x5'

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>X</th>
<th>Y</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>0.33</td>
<td>0.25</td>
</tr>
<tr>
<td>P2</td>
<td>0</td>
<td>0.25</td>
</tr>
<tr>
<td>P3</td>
<td>-0.33</td>
<td>0.25</td>
</tr>
<tr>
<td>P4</td>
<td>0.33</td>
<td>0.25</td>
</tr>
<tr>
<td>P5</td>
<td>0</td>
<td>0.25</td>
</tr>
<tr>
<td>P6</td>
<td>0</td>
<td>-0.25</td>
</tr>
<tr>
<td>P7</td>
<td>-0.33</td>
<td>-0.25</td>
</tr>
<tr>
<td>P8</td>
<td>0.33</td>
<td>-0.25</td>
</tr>
<tr>
<td>P9</td>
<td>0</td>
<td>-0.25</td>
</tr>
<tr>
<td>P10</td>
<td>-0.33</td>
<td>-0.25</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[
\begin{align*}
A &= 0.0797 \text{ in}^2 \\
F_y &= 75 \text{ ksi} \\
P &= 0.65 \times A \times F_y \\
    &= 3.885 \text{ Kips (per wire point)}
\end{align*}
\]

ATTACHMENT BY No. WIRES
(Standard Panels Only)
MX (local) kNm/m

- <= 1.33
- 1.55
- 1.77
- 1.99
- 2.21
- 2.43
- 2.65
- 2.87
- 3.09
- 3.3
- 3.52
- 3.74
- 3.96
- 4.18
- 4.4
- 4.62
- >= 4.84
MY (local) kNm/m
- <= 1.38
- 1.75
- 2.12
- 2.49
- 2.86
- 3.23
- 3.61
- 3.98
- 4.35
- 4.72
- 5.09
- 5.46
- 5.83
- 6.0
- 6.57
- 6.94
- 7.31

Load 1
SOX (local) N/mm²

<= -0.149
-0.130
-0.112
-0.093
-0.074
-0.056
-0.037
-0.019
0
0.019
0.037
0.056
0.074
0.093
0.112
0.130
>= 0.149

Load 1

5.00ft

5.00ft

Print Time/Date: 06/03/2021 12:03

STAAD.Pro for Windows 20.07.04.12

Print Run 1 of 1
SQY (local) N/mm²

- <= -0.120
- -0.105
- -0.090
- -0.075
- -0.060
- -0.045
- -0.030
- -0.015
- 0
- 0.015
- 0.030
- 0.045
- 0.060
- 0.075
- 0.090
- 0.105
- >= 0.120

Load 1

5.00ft

5.00ft

Print Time/Date: 06/03/2021 12:03
STAAD.Pro for Windows 20.07.04.12
Print Run 1 of 1
1. STAAD SPACE

INPUT FILE: Panel Check - 5x5 type A (SM11).STD
2. START JOB INFORMATION
3. ENGINEER DATE 04-MAR-21
4. JOB CLIENT SEL
5. ENGINEER NAME CH
6. JOB NAME PANEL CHECK
7. END JOB INFORMATION
8. INPUT WIDTH 79
9. UNIT FEET XIP
10. JOINT COORDINATES
   11. 1 0 0 0; 2 1 0 0; 3 2 0 0; 4 3 0 0; 5 4 0 0; 6 5 0 0; 7 6 0 0; 8 1 0 0; 9 1 1 0 -1.
   12. 12 2 0 -1; 13 3 0 -1; 14 4 0 -1; 15 5 0 -1; 16 6 0 -1; 17 7 0 -1; 18 8 1 0 -2; 19 9 1 0 -2;
   20. 21 2 0 -2; 22 3 0 -2; 23 4 0 -2; 24 5 0 -2; 25 6 0 -2; 26 7 0 -2; 27 8 1 0 -3; 28 9 1 0 -3;
   30. 31 3 0 -3; 32 4 0 -3; 33 5 0 -3; 45 4 0 0 -4; 46 5 0 -4; 47 6 0 -4; 48 7 0 -4; 49 8 0 -4;
   50. 5 0 -4; 36 0 0 -2; 37 1 0 -3; 38 2 0 -3; 59 3 0 -3; 60 4 0 -3; 61 5 0 -3.
   31. ELEMENT INCIDENCES SHELL
   32. 1 1 0 1 1 2 1 1 1 1 1 2 3 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2 1 1 1 2 1 1 1 1 1 1 1 2
Typical Panel Type (mesh 6W11), Panel Thickness 6'', 5'x5'

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
</thead>
<tbody>
<tr>
<td>X</td>
<td>Y</td>
</tr>
<tr>
<td>P1</td>
<td>0.33 0.25</td>
</tr>
<tr>
<td>P2</td>
<td>0 0.25</td>
</tr>
<tr>
<td>P3</td>
<td>-0.33 0.25</td>
</tr>
<tr>
<td>P4</td>
<td>0.33 0.25</td>
</tr>
<tr>
<td>P5</td>
<td>0 0.25</td>
</tr>
<tr>
<td>P6</td>
<td>-0.33 0.25</td>
</tr>
<tr>
<td>P7</td>
<td>0.33 -0.25</td>
</tr>
<tr>
<td>P8</td>
<td>0 -0.25</td>
</tr>
<tr>
<td>P9</td>
<td>-0.33 -0.25</td>
</tr>
<tr>
<td>P10</td>
<td>0.33 -0.25</td>
</tr>
<tr>
<td>P11</td>
<td>0 -0.25</td>
</tr>
<tr>
<td>P12</td>
<td>-0.33 -0.25</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[ A = 0.0797 \text{ in}^2 \]
\[ F_y = 75 \text{ ksi} \]

\[ P = 0.65 \times A \times F_y \]
\[ = 3.885 \text{ Kips} \text{ (per wire point)} \]
MX (local)
kNm/m

- <= 2.64
- 2.8
- 2.96
- 3.11
- 3.27
- 3.43
- 3.59
- 3.74
- 3.9
- 4.06
- 4.22
- 4.37
- 4.53
- 4.69
- 4.85
- 5
- >= 5.16

Load 1
MY (local) kN/m

- <= 2.08
- 2.38
- 2.68
- 2.97
- 3.27
- 3.57
- 3.87
- 4.16
- 4.46
- 4.76
- 5.05
- 5.35
- 5.65
- 5.95
- 6.24
- 6.54
- >= 6.84

Load 1
SQY (local)
N/mm²

<table>
<thead>
<tr>
<th>Value</th>
<th>Color</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;= -0.113</td>
<td>Dark Red</td>
</tr>
<tr>
<td>-0.099</td>
<td>Red</td>
</tr>
<tr>
<td>-0.085</td>
<td>Dark Orange</td>
</tr>
<tr>
<td>-0.071</td>
<td>Orange</td>
</tr>
<tr>
<td>-0.057</td>
<td>Light Orange</td>
</tr>
<tr>
<td>-0.042</td>
<td>Light Yellow</td>
</tr>
<tr>
<td>-0.028</td>
<td>Yellow</td>
</tr>
<tr>
<td>-0.014</td>
<td>Light Green</td>
</tr>
<tr>
<td>0</td>
<td>Green</td>
</tr>
<tr>
<td>0.014</td>
<td>Dark Green</td>
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<tr>
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</tr>
<tr>
<td>0.071</td>
<td>Yellow</td>
</tr>
<tr>
<td>0.085</td>
<td>Orange</td>
</tr>
<tr>
<td>0.099</td>
<td>Dark Orange</td>
</tr>
<tr>
<td>&gt;= 0.113</td>
<td>Red</td>
</tr>
</tbody>
</table>

Load 1

Print Time/Date: 06/03/2021 12:52

Print Run 1 of 1
1. STAAD SPACE

INPUT FILE: Panel Check - 3x5 type A (6M11).STD
2. START JOB INFORMATION
3. ENGINEER DATE (C-24-MAY-21)
4. JOB CLIENT SIC
5. ENGINEER NAME CE
6. JOB REF PANEL CHECK
7. END JOB INFORMATION
8. INPUT WIDTH 79
9. UNIT FEET XIP
10. JOINT COORDINATES
    11. 1 0 0 0; 2 1 0 0; 3 2 0 0; 4 3 0 0; 5 4 0 0; 6 5 0 0; 10 0 0 -1; 11 1 0 -1.
    12. 12 2 0 -1; 13 3 0 -1; 14 4 0 -1; 15 5 0 -1; 19 0 0 -2; 25 1 0 -2; 21 2 0 -2.
    13. 22 3 0 -2; 23 4 0 -2; 24 5 0 -2; 28 0 0 -3; 29 1 0 -3; 32 2 0 -3; 31 3 0 -3.
    14. 32 4 0 -3; 33 5 0 -3; 34 6 0 -4; 45 0 0 -4; 46 1 0 -4; 47 2 0 -4; 48 3 0 -4; 49 4 0 -4.
    15. 50 5 0 -4; 51 6 0 -5; 52 7 0 -5; 58 2 0 -5; 59 3 0 -5; 60 4 0 -5; 61 5 0 -5.
16. ELEMENT INCIDENCES SHELL
    17. 1 1 10 11 2 1 10 20 11 5 19 29 20 4 2 11 12 3 5 11 20 21 12.
    18. 6 20 29 30 21 7 3 12 13 6 8 12 21 22 13 9 21 30 31 22 10 4 3 14 5.
    20. 31 28 45 46 28 32 28 46 47 30 33 30 47 48 31 34 31 38 48 32 35 32 49 50 33.
    21. 41 45 56 57 46 42 46 57 58 67; 43 47 58 59 48; 44 48 59 60 49; 45 49 60 61 50.
22. ELEMENT PROPERTY
    23. 1 TO 15 21 TO 35 41 TO 45 THICKNESS 0.5
24. DEFINE MATERIAL START
25. ISOTROPIC CONCRETE
    26. E 616304
    27. POISON 0.17
28. DENSITY 2.5
29. ALPHA 15.005
30. DAMP 0.05
31. END DEFINE MATERIAL
32. CONSTANTS
33. MATERIAL CONCRETE ALL
34. SUPPORTS
    35. 31 TO 6 10 TO 15 19 TO 24 28 TO 33 45 TO 50 56 TO 60 -
    36. 1 FIXED BUT KFX 839.76 KFY 839.76 KFZ 839.76 KM1 1 KMN 1 KMB 1
37. LOAD 1 LOADTYPE PUSH TITLE WIRE STRENGTH
38. ELEMENT LOAD
    39. 31 PR Y 3.085 0.25 0.33
40. 32 PR Y 3.085 0.25 0.33

PROBLEM STATISTICS
---------------------------------------------------------------------
NUMBER OF JOINTS/MEMBER/ELEMENT/SUPPORTS = 36/ 25/ 36
SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER
ORIGINAL/FINAL BANDWIDTH = 7/ 7/ 48 DOF
TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 216
SIZE OF STIFFNESS MATRIX = 11 DOUBLE XILIO-WORDS
REQUIRED/AVAIL. DISK SPACE = 12.2/ 263446.3 MB
---------------------------------------------------------------------
52. FINISH
Typical Panel Type (mesh 4W20), Panel Thickness 6", 5'x5'

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th></th>
<th>1.5</th>
<th>0.67</th>
<th>0.67</th>
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<tr>
<td>P8</td>
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<td></td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W20 - 75 Years

\[
\begin{align*}
A &= 0.1583 \text{ in}^2 \\
F_y &= 75 \text{ ksi} \\
P &= 0.65 \times A \times F_y \\
&= 7.717 \text{ Kips (per wire point)}
\end{align*}
\]

ATTACHMENT BY NO. WIRES
(STANDARD PANELS ONLY)
MX (local) kNm/m

- <= 2.92
- 3.23
- 3.54
- 3.85
- 4.17
- 4.48
- 4.79
- 5.1
- 5.41
- 5.73
- 6.04
- 6.35
- 6.66
- 6.97
- 7.28
- 7.6
- >= 7.91

Load 1

5.00 ft

5.00 ft

STAAD.Pro for Windows 20.07.04.12
MY (local)
kNm/m

<= 2.94
3.73
4.51
5.29
5.08
5.86
7.64
8.43
9.21
9.99
10.8
11.6
12.3
13.1
13.9
14.7
>= 15.5
SQY (local)
N/mm²

<= -0.238
-0.208
-0.179
-0.149
-0.119
-0.089
-0.060
-0.030
0
0.030
0.060
0.089
0.119
0.149
0.179
0.208
>= 0.238
STAAD SPACE

PROBLEM STATISTICS

NUMBER OF JOINTS/MEMBER + ELEMENTS/SUPPORTS = 36/ 25/ 36

SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER

ORIGINAL/FINAL BAND-WIDTH = 7/ 7/ 48 DOF

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 216

SIZE OF StIFFNESS MATRIX = 11 DOUBLE KILO-WORDS

ME/RAND/AVAL. DISK SPACE = 12.2/ 263435.0 MB

44. FINISH

1. STAAD SPACE

INPUT FILE: Panel Check - 3x5 type A (4820).STD

2. START JOB INFORMATION

3. ENGINEER DATE 06-MAY-21

4. JOB CLIENT SSL

5. ENGINEER NAME CH

6. JOB REV PANEL CHECK

7. END JOB INFORMATION

8. INPUT WIDTH 79

9. UNIT FEET XIP

10. JOINT COORDINATES

11. 0 0 0; 2 1 0 0; 3 2 0 0; 4 3 0 0; 5 4 0 0; 6 5 0 0; 10 0 0 0; 11 0 0 1.

12. 12 0 0 2; 13 0 0 3; 14 0 0 4; 15 0 0 5; 16 0 0 6; 17 0 0 7; 18 0 0 8; 19 0 0 9.

13. 20 0 0 10; 21 0 0 11; 22 0 0 12; 23 0 0 13; 24 0 0 14; 25 0 0 15; 26 0 0 16; 27 0 0 17.

14. 28 0 0 18; 29 0 0 19; 30 0 0 20; 31 0 0 21; 32 0 0 22; 33 0 0 23; 34 0 0 24; 35 0 0 25.

15. 36 0 0 26; 37 0 0 27; 38 0 0 28; 39 0 0 29; 40 0 0 30; 41 0 0 31; 42 0 0 32; 43 0 0 33.

16. ELEMENT INCIDENCES SHELL

17. 1 1 10 11 2 10 19 20 11 5 19 29 20 9 22 12 12 5 11 20 21 12.

18. 12 29 30 31 11 7 12 13 11 8 11 21 22 13 9 21 10 31 22 10 4 13 14 5.


20. 31 28 45 46 29 32 46 47 30 33 30 47 38 31 34 31 48 39 32 35 32 49 50 33.

21. 14 14 46 57 46 42 46 57 58 47; 43 47 58 59 48; 44 48 59 60 49; 45 60 61 50.

22. ELEMENT PROPERTY

23. 1 21 15 21 25 41 40 45 THICKNESS 0.5

24. DEFINE MATERIAL START

25. ISOTROPIC CONCRETE

26. K 614340

27. POISSON 0.17

28. DENSITY 0.15

29. ALPHA 12-075

30. DAMP 0.05

31. END DEFINE MATERIAL

32. CONSTANTS

33. MATERIAL CONCRETE ALL

34. SUPPORTS

35. 1 TO 10 10 15 19 TO 24 28 70 33 45 TO 50 55 TO 60 -

36. 61 FIXED BUT KFX 839.76 KFY 839.76 KFX 839.76 KMY 1 KYX 1 KMF 1

37. LOAD 1 LOADTYPE PUSH TITLE WIRE STRENGTH

38. ELEMENT LOAD

39. 32 PR GR -7.71 0.25 0

40. 33 PR GR -7.71 0.25 -0.33
Typical Panel Type (mesh 5W20), Panel Thickness 6”, 5’x5’

Point load input force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1’x1’</th>
</tr>
</thead>
<tbody>
<tr>
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<tr>
<td>P9</td>
<td>0</td>
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<tr>
<td>P10</td>
<td>-0.33</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[
A = 0.1583 \text{ in}^2 \\
F_y = 75 \text{ ksi} \\
P = 0.65 \times A \times F_y = 7.717 \text{ Kips (per wire point)}
\]

ATTACHMENT BY No. WIRES
(STANDARD PANELS ONLY)
MX (local)
kNm/m

<table>
<thead>
<tr>
<th>&lt;= 2.64</th>
<th>3.08</th>
<th>3.51</th>
<th>3.95</th>
<th>4.39</th>
<th>4.82</th>
<th>5.26</th>
<th>5.69</th>
<th>6.13</th>
<th>6.56</th>
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<tbody>
<tr>
<td>3.56</td>
<td>7</td>
<td>7.44</td>
<td>7.87</td>
<td>8.31</td>
<td>8.74</td>
<td>9.18</td>
<td>&gt;= 9.62</td>
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<td></td>
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</table>

Load 1

Print Time/Date: 07/03/2021 01:51

STAAD.Pro for Windows 20.07.04.12

Print Run 1 of 1
MY (local) kNm/m

- <= 2.74
- 3.48
- 4.21
- 4.95
- 5.69
- 6.42
- 7.16
- 7.9
- 8.64
- 9.37
- 10.1
- 10.8
- 11.6
- 12.3
- 13.1
- 13.8
- >= 14.5
SOX (local) N/mm²

-0.295
-0.259
-0.222
-0.185
-0.148
-0.111
-0.074
-0.037
0
0.037
0.074
0.111
0.148
0.185
0.222
0.259
0.295

Load 1

STAAD.Pro for Windows 20.07.04.12
**SQY (local)**

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<tr>
<th>N/mm²</th>
<th>Value</th>
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<tbody>
<tr>
<td>&lt;= -0.239</td>
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<tr>
<td>-0.209</td>
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<tr>
<td>-0.179</td>
<td></td>
</tr>
<tr>
<td>-0.149</td>
<td></td>
</tr>
<tr>
<td>-0.119</td>
<td></td>
</tr>
<tr>
<td>-0.089</td>
<td></td>
</tr>
<tr>
<td>-0.060</td>
<td></td>
</tr>
<tr>
<td>-0.030</td>
<td></td>
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<tr>
<td>0</td>
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<td>0.209</td>
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<tr>
<td>&gt;= 0.239</td>
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**Load 1**

STAAD.Pro for Windows 20.07.04.12

Print Time/Date: 07/03/2021 01:52
<table>
<thead>
<tr>
<th>Problem Statistics</th>
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<tbody>
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<td>Number of joints/elements supports = 36/25/36</td>
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<tr>
<td>Solver used is the out-of-core basic solver</td>
</tr>
<tr>
<td>Original/Final band width = 7/7/45 dof</td>
</tr>
<tr>
<td>Total primary load cases = 1, total degrees of freedom = 216</td>
</tr>
<tr>
<td>Size of stiffness matrix = 11 double {{double}}</td>
</tr>
<tr>
<td>Energy/available disk space = 12.2/2634320 MB</td>
</tr>
</tbody>
</table>

50. Finish
Typical Panel Type (mesh 6W20), Panel Thickness 6", 5'x5'

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
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<td>P12</td>
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</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[
A = 0.1583 \text{ in}^2 \\
Fy = 75 \text{ ksi} \\
P = 0.65 \times A \times Fy = 7.717 \text{ Kips (per wire point)}
\]

ATTACHMENT BY No. WIRES
(Standard Panels Only)
MX (local)
kNm/m
- <= 5.25
- 5.56
- 5.87
- 6.19
- 6.5
- 6.81
- 7.12
- 7.44
- 7.75
- 8.06
- 8.38
- 8.69
- 9
- 9.31
- 9.63
- 9.94
- >= 10.3
MY (local) kNm/m

<= 4.14
4.73
5.32
5.91
6.5
7.09
7.68
8.27
8.86
9.45
10
10.6
11.2
11.8
12.4
13

>= 13.6
SOX (local) N/mm²

-0.303
-0.265
-0.228
-0.190
-0.152
-0.114
-0.076
-0.038
0
0.038
0.076
0.114
0.152
0.190
0.228
0.265
0.303

Load 1
### Software licensed to Optimal Engineering Support

<table>
<thead>
<tr>
<th>Job Title</th>
<th>Client</th>
<th>Job No</th>
<th>Sheet No</th>
<th>Rev</th>
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<table>
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<tr>
<th>Part</th>
<th>Ref</th>
<th>Panel Check</th>
<th>By</th>
<th>Date</th>
<th>Chd</th>
<th>File</th>
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<tr>
<td></td>
<td></td>
<td></td>
<td>CH</td>
<td>04-Mar-21</td>
<td></td>
<td>Panel Check - 5x5 type A</td>
</tr>
</tbody>
</table>

**Print Time/Date:** 07/03/2021 02:45  
**Print Run 1 of 1**  
**STAAD.Pro for Windows 20.07.04.12**

#### SQY (local) N/mm²
- <= -0.225
- -0.196
- -0.168
- -0.140
- -0.112
- -0.084
- -0.056
- -0.028
- 0
- 0.028
- 0.056
- 0.084
- 0.112
- 0.140
- 0.168
- 0.196
- >= 0.225

**5.00ft**  
**5.00ft**

**Load 1**
1. STAAD SPACE

INPUT FILE: Panel Check  - 5x5 type A (6M2O).STD

2. START JOB INFORMATION
   ENGINEER DATE 04-MAR-21
   JOB CLIENT SIS
   ENGINEER NAME CH
   JOB REF PANEL CHECK
   END JOB INFORMATION

3. INPUT WIDTH 79
   UNIT FACTOR 1

10. JOINT COORDINATES
   1  1.000 0.000 0.000 0.000 0.000 0.000 11.000 0.000
   12. 2.001 0.000 0.000 0.000 0.000 0.000 21.210 0.000
   13. 22.350 0.000 0.000 0.000 0.000 0.000 24.000 0.000
   14. 31.500 0.000 0.000 0.000 0.000 0.000 40.000 0.000
   15. 50.000 0.000 0.000 0.000 0.000 0.000 60.000 0.000

16. ELEMENT INCIDENCES SHELL
   1. 1 1 10 21 2 10 19 20 11 5 19 29 29 20 4 2 11 12 9 5 11 20 21 12
   18. 6 20 29 30 21 7 3 12 13 4 8 12 21 22 13 9 21 30 31 22 10 4 3 14 5
   19. 11 12 23 14 12 22 31 32 23 13 5 14 15 6 14 14 23 24 15 13 32 33 24
   20. 31 28 45 46 29 32 29 46 47 30 33 30 47 48 31 34 31 48 59 32 35 32 49 50 33
   21. 41 45 56 74 42 46 57 58 67 43 47 55 54 49 48 59 60 49 45 49 60 61 50

22. ELEMENT PROPERTY
   23. 1 TO 15 21 TO 35 41 TO 45 THICKNESS 0.5

24. DEFINE MATERIAL START
   25. E 6.14304
   26. K 6.14304
   27. POISSON 0.17
   28. DENSITY 0.15
   29. ALPH 0.075
   30. DAMP 0.05
   31. END DEFINE MATERIAL
   32. CONSTANTS
   33. MATERIAL CONCRETE ALL
   34. SUPPORTS
   35. 1 TO 6 10 TO 11 19 TO 24 28 TO 33 45 TO 50 56 TO 60 -
   36. 51 FIXED BUT KFX 839.76 KFY 839.76 KXZ 839.76 KYZ 1 KXZ 1
   37. LOAD 1 LOADTYPE PUSH TITLE WIRE STRENGTH
   38. ELEMENT LOAD
   39. 31 PR DY -7.713 0.25 0.33
   40. 32 PR DY -7.713 0.25 0.33

PROBLEM STATISTICS

NUMBER OF JOINTS/MEMBER/ELEMENTS/SUPPORTS = 36/ 25/ 36
SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER

ORIGINAL/FINAL BANDWIDTH = 77/ 77/ 48 DOF
TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 216
SIZE OF STIFFNESS MATRIX = 11 DOUBLE KILO-WORDS
REQD/AVAL. DISK SPACE = 12.2/ 263430.2 MB

52. FINISH
Typical Panel Type (mesh 5W24), Panel Thickness 6", 5'x5'

Point load Input Force in panel for Staad Pro input

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<thead>
<tr>
<th>Location</th>
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<th>Y</th>
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</thead>
<tbody>
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<td>P3</td>
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<td>P4</td>
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<tr>
<td>P6</td>
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<tr>
<td>P7</td>
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<tr>
<td>P10</td>
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<td>-0.25</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

A = 0.194 in²
Fy = 75 ksi

P = 0.65 x A x Fy
   = 9.458 Kips (per wire point)

3–WIRE CONNECTION PANEL
4–WIRE CONNECTION PANEL
5–WIRE CONNECTION PANEL

ATTACHMENT BY No. WIRES
(STANDARD PANELS ONLY)
MX (local)
kNm/m

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<tr>
<th>&lt;= 3.24</th>
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<th>4.84</th>
<th>5.37</th>
<th>5.91</th>
<th>6.44</th>
<th>6.98</th>
<th>7.51</th>
<th>8.05</th>
<th>8.58</th>
<th>9.11</th>
<th>9.65</th>
<th>10.2</th>
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Load 1
SOX (local) N/mm2

-0.362
-0.317
-0.272
-0.226
-0.181
-0.136
-0.091
-0.045
0
0.045
0.091
0.136
0.181
0.226
0.272
0.317
0.362

Load 1

 STAAD.Pro for Windows 20.07.04.12

Print Time/Date: 07/03/2021 13:22
SQY (local) N/mm²
≤ -0.292
-0.256
-0.219
-0.183
-0.146
-0.110
-0.073
-0.037
0
0.037
0.073
0.110
0.146
0.183
0.219
0.256
≥ 0.292
1. STAAD SPACE
   INPUT FILE: Panel Check - 5x5 type A (SM24).STD
   2. START JOB INFORMATION
   3. ENGINEER DATE 24-NOV-21
   4. JOB CLIENT NIL
   5. ENGINEER NAME CE
   6. JOB REV PANEL CHECK
   7. END JOB INFORMATION
   8. INPUT WIDTH 79
   9. UNIT FEET KIP
   10. JOINT COORDINATES
       11. 1 0 0 0 0; 2 1 0 0 0; 3 0 0 0 0; 4 3 0 0 0; 5 4 0 0 0; 6 5 0 0 0; 10 0 0 -1; 11 1 0 -1.
       12. 13 2 0 -1; 13 3 0 -1; 14 4 0 -1; 15 5 0 -1; 19 0 0 -2; 20 1 0 -2; 21 2 0 -2.
       13. 22 3 0 -2; 23 4 0 -2; 24 5 0 -2; 28 0 0 -3; 29 1 0 -3; 32 0 0 -3; 31 3 0 -3.
       14. 32 4 0 -3; 33 5 0 -3; 45 0 0 -4; 46 1 0 -4; 47 2 0 -4; 48 3 0 -4; 49 4 0 -4.
       15. 50 5 0 -4; 51 6 0 -4; 52 7 0 -4; 53 1 0 -5; 54 2 0 -5; 55 3 0 -5; 60 4 0 -5; 61 5 0 -5.
   16. ELEMENT INCIDENCES SHELL
       17. 1 10 11 2 10 19 20 11; 5 19 29 39 20; 4 2 11 12 3 11 20 21 12.
       18. 6 20 29 30 21; 7 3 12 13 4; 8 12 21 22 13; 9 21 30 31 22; 10 4 3 14 5.
       20. 31 28 45 46 29; 32 29 46 47 30; 33 30 47 48 31; 34 31 48 49 32; 35 32 49 50 33.
       21. 41 40 54 57 46; 42 46 57 58 47; 43 47 58 59 48; 44 48 59 60 49; 45 60 60 50 50.
   22. ELEMENT PROPERTY
       23. 1 10 15 21 20 35 45 45 45 45 45 45 45 45 45 45 45 45 45 45 45 45 45 45 45 45 45 45 45 45.
       24. DEFINE MATERIAL IDENT
       25. ALGEBRA IDENT
       26. ALPHA 10.035
       27. DAMP 0.05
       28. END DEFINE MATERIAL
       29. CONSTANTS
       30. MATERIAL CONCRETE ALL
       31. SUPPORTS
       32. 1 TO 6 10 TO 15 19 TO 24 28 TO 33 45 TO 50 56 TO 60 -
       33. 61 FIXED BUT KFX 839.76 KFX 839.76 KFX 839.76 KMX 1 KNY 1 KMY 1.
       34. LOAD 1 LOADTYPE PUSH TITLE WIND STRESSES
       35. ELEMENT LOAD
       36. 31 PR FY -9.458 0.25 -0.33.
       37. 32 PR FY -9.458 0.25 -0.33.
Typical Panel Type (mesh 6W24), Panel Thickness 6", 5'x5'

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
</tr>
<tr>
<td>P1</td>
<td>0.33</td>
</tr>
<tr>
<td>P2</td>
<td>0</td>
</tr>
<tr>
<td>P3</td>
<td>-0.33</td>
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<tr>
<td>P4</td>
<td>0.33</td>
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<tr>
<td>P11</td>
<td>0</td>
</tr>
<tr>
<td>P12</td>
<td>-0.33</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[
A = 0.194 \text{ in}^2 \\
Fy = 75 \text{ ksi} \\
\]

\[
P = 0.65 \times A \times Fy = 9.458 \text{ Kips (per wire point)}
\]

ATTACHMENT BY No. WIRES
(Stanard Panels Only)
MX (local) kNm/m
- <= 6.43
- 6.82
- 7.2
- 7.58
- 7.97
- 8.35
- 8.73
- 9.12
- 9.5
- 9.88
- 10.3
- 10.6
- 11
- 11.4
- 11.8
- 12.2
- >= 12.6

Load 1

STAAD.Pro for Windows 20.07.04.12
MY (local) kNm/m

- <= 5.07
- 5.8
- 5.52
- 7.24
- 7.97
- 8.69
- 9.41
- 10.1
- 10.9
- 11.6
- 12.3
- 13
- 13.8
- 14.5
- 15.2
- 15.9
- >= 16.6
SQY (local)
N/mm²

<= -0.275
-0.241
-0.206
-0.172
-0.138
-0.103
-0.069
-0.034
0
0.034
0.069
0.103
0.138
0.172
0.206
0.241
>= 0.275

Load 1
5.00ft
5.00ft

Print Time/Date: 07/03/2021 14:58
STAAD.Pro for Windows 20.07.04.12
Print Run 1 of 1
1. STAAD SPACE

**INPUT FILE: Panel Check - Sax type A (6M24).STD**

2. START JOB INFORMATION

3. ENGINEER DATE 01-MAR-21

4. JOB CLIENT SEL

5. ENGINEER NAME CH

6. JOB REV PANEL CHECK

7. END JOB INFORMATION

8. INPUT WIDTH 79

9. UNIT FIELD KIP

10. JOINT COORDINATES

| 11 | 100 0 0; 2 10 0; 3 20 0; 4 30 0; 5 40 0; 6 50 0; 10 0 0; 11 10 0 |
| 12 | 12 20 -2; 13 30 -1; 14 40 -1; 15 50 -1; 19 0 2; 20 1 2; 21 2 0 -2 |
| 13 | 22 30 -2; 23 40 -2; 24 50 -2; 28 0 3; 29 1 0 -3; 30 2 0 -3; 31 3 0 -3 |
| 14 | 32 40 -3; 33 50 -3; 45 0 0 -4; 46 1 0 -4; 47 2 0 -4; 48 3 0 -4; 49 4 0 -4 |
| 15 | 50 5 0 -4; 56 2 0 -3; 57 1 0 -3; 58 2 0 -3; 59 3 0 -3; 60 4 0 -3; 61 5 0 -3 |

16. ELEMENT INCIDENCES SHELL

| 17 | 1 1 10 11 2 10 19 20 11 5 19 29 20 4 21 12 12 5 11 20 21 12 |
| 18 | 6 20 29 30 21 7 32 13 4 8 12 21 22 13 9 21 30 31 22 10 4 33 14 5 |
| 19 | 11 33 22 14 12 22 31 32 23 13 5 14 15 6 14 14 23 24 15 15 33 12 24 |
| 20 | 31 28 45 46 29 32 29 46 47 30 33 30 47 48 31 34 31 48 99 32 35 32 49 50 33 |
| 21 | 41 43 56 57 46 42 66 57 58 67 43 47 58 59 48 44 48 59 60 49 45 49 60 61 50 |

22. ELEMENT PROPERTY

| 23 | 1 20 15 21 20 35 41 45 THICKNESS 0.5 |

24. EXFINE MATERIAL START

25. ISOTHROPIC CONCRETE

26. K 614304

27. POISON 0.17

28. DENSITY 0.15

29. ALPHA 12.075

30. DAMP 0.05

31. END DEFINE MATERIAL

32. CONSTANTS

33. MATERIAL CONCRETE ALL

34. SUPPORTS

35. 1 TO 6 TO 15 19 24 28 70 33 45 TO 50 56 TO 60 -

36. 61 FIXED BUT KFX 839.76 KFX 839.76 KFX 839.76 KMX 1 KNY 1 KME 1

37. LOAD 1 LOADTYPE PUSH TITLE WIRE STRENGTH

38. ELEMENT LOAD

39. 31 PR GR 9,458 0.25 0.33

40. 32 PR GR 9,458 0.25 0.33

41. 33 PR GR 9,458 0.25 -0.33

42. 33 PR GR 9,458 0.25 0.33

43. 34 PR GR 9,458 0.25 0

44. 35 PR GR 9,458 0.25 -0.33

45. 2 PR GR 9,458 0.25 0.33

46. 5 PR GR 9,458 0.25 -0.33

47. 8 PR GR 9,458 0.25 0.33

48. 8 PR GR 9,458 0.25 -0.33

49. 11 PR GR 9,458 0.25 0

50. 14 PR GR 9,458 0.25 -0.33

51. PERFORM ANALYSIS

---

**PROBLEM STATISTICS**

| MEMBER/JOINT/ELEMENT/SUPPORTS | 36/25/36 |

**SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER**

**ORIGINAL/FINAL BAND-WIDTH**

**TOTAL PRIMARY LOAD CASES**

**TOTAL DEGREES OF FREEDOM**

**SIZE OF STIFFNESS MATRIX**

**REQUIRED/AVAILABLE DISK SPACE**

---

**52. FINISH**
## Summary Result From Staad Pro Analysis Output

<table>
<thead>
<tr>
<th>Standard Panel Size</th>
<th>Bar Mat Type</th>
<th>Panel Type</th>
<th>Horz Rebar</th>
<th>Vert Rebar</th>
<th>Staad Pro Output</th>
<th>Data for AASHTO LRFD Design Check</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Mx (kNm/m)</td>
<td>My (kNm/m)</td>
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<tr>
<td>5'x5'</td>
<td>3W11</td>
<td>A</td>
<td>4 #4</td>
<td>4 #4</td>
<td>3.150</td>
<td>6.520</td>
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<tr>
<td></td>
<td>4W11</td>
<td>X</td>
<td>5 #4</td>
<td>5 #4</td>
<td>3.980</td>
<td>7.790</td>
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<td></td>
<td>5W11</td>
<td>Y</td>
<td>6 #4</td>
<td>5 #4</td>
<td>4.840</td>
<td>7.310</td>
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<td></td>
<td>6W11</td>
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<td></td>
<td></td>
<td>5.160</td>
<td>6.840</td>
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<td>7.910</td>
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<tr>
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<td>5W20</td>
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<td>10.300</td>
<td>13.600</td>
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<td>6W24</td>
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<td></td>
<td>12.600</td>
<td>16.600</td>
</tr>
</tbody>
</table>

* Maximum bending moment and shear for each panel type shown in red bold font
5' x 5' STANDARD
A PANEL
Typical Panel Type A, Panel Thickness 6", 5'x5', Horizontal Rebar

**Panel Strength Design**

Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This design is to present the information about the acceptable panel reinforcement sizes and spacing.

Panel Thickness \( t_{\text{panel}} \) = 6 in

Finishing Thickness \( C_{\text{panel}} \) = 1.5 in

Additional Load from Finishes = 0.0125 Kip/ft²

Concrete Cover \( C_{\text{panel}} \) = 1.5 in

Vertical Bar Size #4

Rebar Diameter \( d_b \) = 0.5 in \( d_{bh} = 0 \) in

Rebar Area \( A_b \) = 0.196 in² \( A_p = \frac{1}{4} \pi d_b^2 \)

Rebar spacing \( s \) = 18 in

Width of the design section \( b \) = 12 in

Effective depth of section \( d_s \) = 4.25 in \( d_s = t_{\text{panel}} - C_{\text{panel}} - d_{bh} - \frac{1}{2} d_b \)

Effective depth of section for Negative Moment \( d_s(M^-) \) = 1.75 in \( d_s(M^-) = t_{\text{panel}} - d_s \)

Correction factor for source aggregate \( K_s \) = 1 in AASHTO 5.4.2.4

Concrete Density \( W_c \) = 0.15 kcf

Concrete Strength \( f'c \) = 4.00 ksi (Concrete Class A compressive strength)

Reinforcement Strength \( f_y \) = 60 ksi (Minimum yield strength of grade 60 steel)

Modulus Elasticity of concrete \( E_c \) = 4266 ksi

Modulus Elasticity of reinforcement \( E_s \) = 29000 ksi

Modular Ratio \( n \) = 6.798 \( n = E_s/E_c \)

Area of Steel per Design Strip \( A_s \) = 0.131 in² \( A_s = b(\frac{A_p}{s}) \)

\[ +M_u = 1.75 \text{ kip-ft} \]

\[ -M_u = 0.00 \text{ kip-ft} \]

Resistance factor for tension-controlled section \( \phi_{\text{STR}} \) = 0.90 AASHTO 5.5.4.2

**Positive Moment Capacity**

Depth of equivalent stress block \( a = \frac{A_s f_y}{0.85 f'c b} = 0.192 \) in

Factored flexural resistance \[ +\varphi M_n = \varphi A_s f_y \left( d_s - \frac{a}{2} \right) = 2.447 \] kip-ft

Check \[ +\varphi M_n > +M_u \]

2.447 > 1.75 OK

**Negative Moment Capacity**

Depth of equivalent stress block \( a = \frac{A_s f_y}{0.85 f'c b} = 0.192 \) in

Factored flexural resistance \[ -\varphi M_n = \varphi A_s f_y \left( d_s(M^-) - \frac{a}{2} \right) = 0.974 \] kip-ft

Check \[ -\varphi M_n > -M_u \]

0.974 > 0.00 OK
**Minimum Reinforcement**  
AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate
to develop a factored flexural resistance, \( M_r = \varphi M_n \), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

\[
M_{cr} = \gamma_3 \left( \gamma_1 f_r + \gamma_2 R_{ce} \right) S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) 
\]

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

where:

- flexural cracking variability factor \( \gamma_1 = 1.60 \) (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength \( \gamma_3 = 0.67 \) (A615 steel)
- concrete density modification factor \( \lambda = 1.00 \) AASHTO 5.4.2.8

- modulus of rapture \( f_r = 0.24 \gamma \lambda \sqrt{\frac{f_{ck}}{6}} \)
- \( f_r = 0.48 \) ksi AASHTO 5.4.2.6

- section modulus of design section \( S_c = \frac{bh^2}{6} = \frac{b t_{panel}^2}{6} \)
- \( S_c = 72.00 \text{ in}^3 \)

Cracking moment

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

\( M_{cr} = 3.09 \) kip-ft

Check positive moment reinforcement:

- 1.33 x factored ultimate moment \( +\varphi M_n \) = 2.33 kip-ft
- Cracking moment \( M_{cr} \) = 3.09 kip-ft
- min from (1.33 x factored +\( M_u \) and \( M_{cr} \)) \( +\varphi M_n > \text{min}(+M_u, M_{cr}) \)
  \( 2.447 > 2.33 \text{ OK} \)

Check negative moment reinforcement:

- 1.33 x factored ultimate moment \( -\varphi M_n \) = 0.00 kip-ft
- Cracking moment \( M_{cr} \) = 3.09 kip-ft
- min from (1.33 x factored -\( M_u \) and \( M_{cr} \)) \( -\varphi M_n > \text{min}(-M_u, M_{cr}) \)
  \( 0.974 > 0.00 \text{ OK} \)
Typical Panel Type A, Panel Thickness 6", 5'x5', Horizontal Rebar

Shear on Panel

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness (t_{\text{panel}})</td>
<td>6 in</td>
</tr>
<tr>
<td>Concrete Cover (C_{\text{panel}})</td>
<td>1.5 in</td>
</tr>
<tr>
<td>Concrete Strength (f'_{c})</td>
<td>4.0 ksi</td>
</tr>
<tr>
<td>Reinforcement Strength (f_{y})</td>
<td>60 ksi</td>
</tr>
<tr>
<td>Bar Size #4</td>
<td></td>
</tr>
<tr>
<td>Rebar Diameter (d_{b})</td>
<td>0.500 in</td>
</tr>
<tr>
<td>Rebar Area (A_{b})</td>
<td>0.196 in²</td>
</tr>
<tr>
<td>Rebar spacing (s)</td>
<td>18 in</td>
</tr>
<tr>
<td>Width of the design section (b)</td>
<td>12 in</td>
</tr>
<tr>
<td>Area of Steel per Design Strip (A_{s})</td>
<td>0.131 in²</td>
</tr>
</tbody>
</table>

\[
c = \frac{A_{s} \cdot f_{y}}{\alpha_{1} f'_{c} \cdot \beta_{1} \cdot b}
\]

\[
d_{v} = t_{\text{panel}} - \frac{c}{2} - C_{\text{panel}} - \frac{d_{b}}{2}
\]

AASHTO 5.7.3.3

Vn shall be the lesser of:

(1) \(V_{n} = V_{c} + V_{s}\)

\[
V_{c} = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f'_{c} \cdot b_{v} \cdot d_{v}}
\]

\[
V_{s} = A_{v} \cdot f_{y} \cdot d_{v} \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha
\]

\[
(1) \quad V_{n} = 6.275 \quad \text{Kips}
\]

\[
(2) \quad V_{n} = 2.708 \quad \text{Kips}
\]

From (1) and (2) \(V_{n}\) should be: \(V_{n} = 8.982\) Kips

\[
\phi \cdot V_{n} = 0.9 \cdot V_{n}
\]

\[
= 8.084 \quad \text{Kips}
\]

Shear Force:

From Staad Pro Result \(V_{\text{max}}\) = \(0.12\) N/mm²

\[
= 17.40 \quad \text{lbs/ln}^2
\]

\[
\text{Shear Section Area} = b \times t_{\text{panel}}
\]

\[
= 12 \times 6
\]

\[
V_{\text{max}} = 1253.13 \quad \text{lbs}
\]

\[
= 1.253 \quad \text{Kips} \quad \text{OK, Vmax less than Vn}
\]
**Typical Panel Type A, Panel Thickness 6", 5'x5', Vertical Rebar**

**Panel Strength Design**
Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

Panel Thickness $t_{panel} = 6$ in
Finishing Thickness $t_{c} = 1$ in
Additional Load from Finishes $= 0.0125$ kip/ft
Concrete Cover $C_{c} = 1.5$ in

Vertical Bar Size #4
Rebar Diameter $d_{b} = 0.5$ in
Rebar Area $A_{b} = 0.196$ in$^2$
Rebar Diameter $d_{bh} = 0.5$ in
Rebar spacing $s = 18$ in
Width of the design section $b = 12$ in
Effective depth of section $d_{s} = 3.75$ in
Effective depth of section for Negative Moment $d_{s(M^-)} = 2.25$ in
Correction factor for source aggregate $K_{s} = 1$ in AASHTO 5.4.2.4
Concrete Density $W_{c} = 0.15$ kcf
Concrete Strength $f'_{c} = 4.00$ ksi (Concrete Class A compressive strength)
Reinforcement Strength $f_{y} = 60$ ksi (Minimum yield strength of grade 60 steel)
Modulus Elasticity of concrete $E_{c} = 4266$ ksi
Modulus Elasticity of reinforcement $E_{s} = 29000$ ksi
Modular Ratio $n = 6.798$ $n = E_{s}/E_{c}$
Area of Steel per Design Strip $A_{s} = 0.131$ in$^2$

$$A_{s} = b(A_{bh})$$

$$+M_{u} = 1.16$$ kip-ft $+M_{u, service} = 0.86$ kip-ft (FK = 1.35)

$$-M_{u} = 0.00$$ kip-ft $-M_{u, service} = 0.00$ kip-ft (FK = 1.35)

Resistance factor for tension-controlled section $\phi_{STR} = 0.90$ AASHTO 5.5.4.2

**Positive Moment Capacity**
Depth of equivalent stress block $a = \frac{A_{s}f_{y}}{0.85f'_{c}b} = 0.192$ in
Factored flexural resistance $+\varphi M_{n} = \varphi A_{s}f_{y}\left(d_{s} - \frac{a}{2}\right) = 2.152$ kip-ft
Check $+\varphi M_{n} > +M_{u}$ 2.152 > 1.16 OK

**Negative Moment Capacity**
Depth of equivalent stress block $a = \frac{A_{s}f_{y}}{0.85f'_{c}b} = 0.192$ in
Factored flexural resistance $-\varphi M_{n} = \varphi A_{s}f_{y}\left(d_{s(M^-)} - \frac{a}{2}\right) = 1.269$ kip-ft
Check $-\varphi M_{n} > -M_{u}$ 1.269 > 0.00 OK
**Minimum Reinforcement** AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r = \varphi M_n \), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment
\[
M_{cr} = \gamma_3 \left[ (\gamma_1 f_r + \gamma_2 \gamma_{pc}) S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \right]
\]
AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

where:
- flexural cracking variability factor \( \gamma_1 = 1.60 \) (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength \( \gamma_3 = 0.67 \) (A615 steel)
- concrete density modification factor \( \lambda = 1.00 \) AASHTO 5.4.2.8
- modulus of rupture \( f_r = 0.24\lambda\sqrt{f_c^i} \) \( f_r = 0.48 \) ksi AASHTO 5.4.2.6
- section modulus of design section \( S_c = \frac{b h^2}{6} = \frac{b t_{panel}^2}{6} \) \( S_c = 72.00 \) in\(^3\)

Cracking moment \( M_{cr} = \gamma_3 \gamma_1 f_r S_c \) \( M_{cr} = 3.09 \) kip-ft

check positive moment reinforcement:
- 1.33 x factored ultimate moment = 1.54 kip-ft
- Cracking moment = 3.09 kip-ft
- min from (1.33 x factored +\( M_u \) and \( M_{cr} \)) = 1.54 kip-ft

Check:
\[ +\varphi M_n > \min(+M_u , M_{cr}) \]
\[
2.152 > 1.54 \quad \text{OK}
\]

check negative moment reinforcement:
- 1.33 x factored ultimate moment = 0.00 kip-ft
- Cracking moment = 3.09 kip-ft
- min from (1.33 x factored -\( M_u \) and \( M_{cr} \)) = 0.00 kip-ft

Check:
\[ -\varphi M_n > \min(-M_u , M_{cr}) \]
\[
1.269 > 0.00 \quad \text{OK}
\]
Typical Panel Type A, Panel Thickness 6", 5'x5', Vertical Rebar

**Shear on Panel**

Panel Thickness
\[ t_{\text{panel}} = 6 \text{ in} \]

Concrete Cover
\[ c_{\text{panel}} = 2.5 \text{ in} \]

Concrete Strength
\[ f'_{c} = 4.0 \text{ ksi} \] (Concrete Class A compressive strength)

Reinforcement Strength
\[ f_{y} = 60 \text{ ksi} \] (Minimum yield strength of grade 60 steel)

Bar Size

**Rebar Diameter**
\[ d_{b} = 0.500 \text{ in} \]

Rebar Area
\[ A_{b} = 0.196 \text{ in}^2 \]

Rebar spacing
\[ s = 18 \text{ in} \]

Width of the design section
\[ b = 12 \text{ in} \]

Area of Steel per Design Strip
\[ A_{s} = 0.131 \text{ in}^2 \]

\[ c = \frac{A_{s} \cdot f_{s}}{\alpha_{1} \cdot f'_{c} \cdot A_{b} \cdot b} \]
\[ c = 0.226 \text{ in} \]

\[ d_{v} = t_{\text{panel}} - \frac{c}{2} - c_{\text{panel}} - \frac{d_{b}}{2} \]
\[ d_{v} = 3.137 \text{ in} \]

**AASHTO 5.7.3.3**

Vn shall be the lesser of:

1. \[ V_{n} = V_{c} + V_{s} \]

\[ V_{c} = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f'_{c} \cdot b_{v} \cdot d_{v}} \]
\[ V_{c} = 4.758 \text{ Kips} \]

\[ V_{s} = \frac{A_{v} \cdot f_{y} \cdot d_{v} \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha}{s} \]
\[ V_{s} = 2.053 \text{ Kips} \]

(1) \[ V_{n} = 6.811 \text{ Kips} \]

(2) \[ V_{n} = 0.25 \cdot f'_{c} \cdot b_{v} \cdot d_{v} \]

(2) \[ V_{n} = 37.641 \text{ Kips} \]

From (1) and (2) Vn should be:
\[ \phi \cdot V_{n} = 0.9 \cdot V_{n} \]
\[ \phi \cdot V_{n} = 6.130 \text{ Kips} \]

**Shear Force:**

From Staad Pro Result
\[ V_{\text{max}} = 0.153 \text{ N/mm}^2 \]
\[ V_{\text{max}} = 22.19 \text{ lbs/in}^2 \]

Shear Section Area
\[ A_{b} = b \times t_{\text{panel}} \]
\[ A_{b} = 12 \times 6 \]

\[ V_{\text{max}} = 1597.74 \text{ lbs} \]
\[ V_{\text{max}} = 1.598 \text{ Kips} \] OK, Vmax less than Vn
5’ x 5’ STANDARD X PANEL
**Typical Panel Type X, Panel Thickness 6", 5'x5', Horizontal Rebar**

**Panel Strength Design**
Design of panel reinforcement per
AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

### Panel Thickness
- \( t_{\text{panel}} = 6 \text{ in} \)

### Finishing Thickness
- \( 1 \text{ in} \)

### Additional Load from Finishes
- \( 0.0125 \text{ Kip/ft}^2 \)

### Concrete Cover
- \( C_{\text{panel}} = 1.5 \text{ in} \)

### Vertical Bar Size
- \#4

### Rebar Diameter
- \( d_b = 0.5 \text{ in} \)

### Rebar Area
- \( A_b = 0.196 \text{ in}^2 \)

### Rebar spacing
- \( s = 12 \text{ in} \)

### Width of the design section
- \( b = 12 \text{ in} \)

### Effective depth of section
- \( d_s = 4.25 \text{ in} \)

### Effective depth of section for Negative Moment
- \( d_{s(M^-)} = 1.75 \text{ in} \)

### Correction factor for source aggregate
- \( K_1 = 1 \text{ in} \)

### Concrete Density
- \( W_c = 0.15 \text{ kcf} \)

### Concrete Strength
- \( f' c = 4.00 \text{ ksi} \) (Concrete Class A compressive strength)

### Reinforcement Strength
- \( f_y = 60 \text{ ksi} \) (Minimum yield strength of grade 60 steel)

### Modulus Elasticity of concrete
- \( E_c = 4266 \text{ ksi} \)

### Modulus Elasticity of reinforcement
- \( E_s = 29000 \text{ ksi} \)

### Modular Ratio
- \( n = 6.798 \)

### Area of Steel per Design Strip
- \( A_s = 0.196 \text{ in}^2 \)

### Resistance factor for tension-controlled section
- \( \phi_{\text{STR}} = 0.90 \) AASHTO 5.5.4.2

### Positive Moment Capacity

#### Depth of equivalent stress block
- \( a = \frac{A_s f_y}{0.85 f' c b} = 0.289 \text{ in} \)

#### Factored flexural resistance
- \( +\phi M_n = \phi A_s f_y \left( d_s - \frac{a}{2} \right) = 3.628 \text{ kip-ft} \)

#### Check
- \( +\phi M_n > +M_u \quad 3.628 > 3.48 \text{ OK} \)

### Negative Moment Capacity

#### Depth of equivalent stress block
- \( a = \frac{A_s f_y}{0.85 f' c b} = 0.289 \text{ in} \)

#### Factored flexural resistance
- \( -\phi M_n = \phi A_s f_y \left( d_{s(M^-)} - \frac{a}{2} \right) = 1.419 \text{ kip-ft} \)

#### Check
- \( -\phi M_n > -M_u \quad 1.419 > 0.00 \text{ OK} \)
Minimum Reinforcement  AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r = \phi M_n \), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

\[
M_{cr} = \gamma_3 \left( \gamma_1 f_t + \gamma_2 \gamma_{cpe} S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \right)
\]

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_t S_c
\]

where:

- flexural cracking variability factor \( \gamma_1 = 1.60 \) (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength \( \gamma_3 = 0.67 \) (A615 steel)
- concrete density modification factor \( \lambda = 1.00 \) AASHTO 5.4.2.8

- modulus of rupture \( f_r = 0.24\lambda \sqrt{f'_c} \) \( f_r = 0.48 \) ksi AASHTO 5.4.2.6

- section modulus of design section \( S_c = \frac{b h^2}{6} = \frac{b t_{panel}^2}{6} \) \( S_c = 72.00 \) in³

Cracking moment

\[
M_{cr} = \gamma_3 \gamma_1 f_t S_c
\]

\( M_{cr} = 3.09 \) kip-ft

Check positive moment reinforcement:

- 1.33 x factored ultimate moment = 4.63 kip-ft
- Cracking moment = 3.09 kip-ft
- min from (1.33 x factored +Mₜ and Mₜ) = 3.09 kip-ft

Check:

\[ +\phi M_n > \min(+M_u, M_{cr}) \]

\[ 3.628 > 3.09 \quad \text{OK} \]

Check negative moment reinforcement:

- 1.33 x factored ultimate moment = 0.00 kip-ft
- Cracking moment = 3.09 kip-ft
- min from (1.33 x factored -Mₜ and Mₜ) = 0.00 kip-ft

Check:

\[ -\phi M_n > \min(-M_u, M_{cr}) \]

\[ 1.419 > 0.00 \quad \text{OK} \]
**Typical Panel Type X, Panel Thickness 6", 5'x5', Horizontal Rebar**

**Shear on Panel**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness</td>
<td>$t_{panel} = 6$ in</td>
</tr>
<tr>
<td>Concrete Cover</td>
<td>$c_{panel} = 1.5$ in</td>
</tr>
<tr>
<td>Concrete Strength</td>
<td>$f'c = 4.0$ ksi (Concrete Class A compressive strength)</td>
</tr>
<tr>
<td>Reinforcement Strength</td>
<td>$f_y = 60$ ksi (Minimum yield strength of grade 60 steel)</td>
</tr>
<tr>
<td>Bar Size #4</td>
<td></td>
</tr>
<tr>
<td>Rebar Diameter</td>
<td>$d_b = 0.500$ in</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>$A_b = 0.196$ in$^2$</td>
</tr>
<tr>
<td>Rebar spacing</td>
<td>$s = 12$ in</td>
</tr>
<tr>
<td>Width of the design section</td>
<td>$b = 12$ in</td>
</tr>
<tr>
<td>Area of Steel per Design Strip</td>
<td>$A_s = 0.196$ in$^2$</td>
</tr>
<tr>
<td></td>
<td>$c = 0.340$ in</td>
</tr>
<tr>
<td></td>
<td>$d_v = 4.080$ in</td>
</tr>
</tbody>
</table>

**AASHTO 5.7.3.3**

$V_n$ shall be the lesser of:

1. $V_n = V_c + V_s$

   \[
   V_c = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f'c \cdot b_v \cdot d_v} \quad V_c = 6.189 \text{ Kips}
   \]

   \[
   V_s = A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha \quad V_s = 4.006 \text{ Kips}
   \]

   (1) $V_n = 10.194 \text{ Kips}$

2. $V_n = 0.25 \cdot f'c \cdot b_v \cdot d_v$

   (2) $V_n = 48.962 \text{ Kips}$

From (1) and (2) $V_n$ should be: $V_n = 10.194 \text{ Kips}$

\[
\phi \cdot V_n = 0.9 \cdot V_n = 9.175 \text{ Kips}
\]

**Shear Force:**

From Staad Pro Result $V_{max} = 0.239 \text{ N/mm}^2$

\[
= 34.66 \text{ lbs/in}^2
\]

Shear Section Area $= b \times t_{panel}$

\[
= 12 \times 6
\]

$V_{max} = 2495.81 \text{ lbs}$

$OK, V_{max} \text{ less than } V_n$
**Typical Panel Type X, Panel Thickness 6'', 5'x5', Vertical Rebar**

**Panel Strength Design**
Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

<table>
<thead>
<tr>
<th>Panel Thickness</th>
<th>t_{panel} = 6 in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Finishing Thickness</td>
<td>C_{panel} = 1.5 in</td>
</tr>
<tr>
<td>Additional Load from Finishes</td>
<td>= 0.0125 Kip/ft²</td>
</tr>
<tr>
<td>Concrete Cover</td>
<td>= 1.5 in</td>
</tr>
<tr>
<td>Vertical Bar Size</td>
<td>#4</td>
</tr>
<tr>
<td>Rebar Diameter</td>
<td>d_b = 0.5 in</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>A_b = 0.196 in²</td>
</tr>
<tr>
<td>Rebar spacing</td>
<td>s = 12 in</td>
</tr>
<tr>
<td>Width of the design section</td>
<td>b = 12 in</td>
</tr>
<tr>
<td>Effective depth of section</td>
<td>d_s = 3.75 in</td>
</tr>
<tr>
<td>Effective depth of section for Negative Moment</td>
<td>d_{s(M^-)} = 2.25 in</td>
</tr>
<tr>
<td>Correction factor for source aggregate</td>
<td>K_1 = 1 in</td>
</tr>
<tr>
<td>Concrete Density</td>
<td>W_c = 0.15 kcf</td>
</tr>
<tr>
<td>Concrete Strength</td>
<td>f'_{c} = 4.00 ksi</td>
</tr>
<tr>
<td>Reinforcement Strength</td>
<td>f_y = 60 ksi</td>
</tr>
<tr>
<td>Modulus Elasticity of concrete</td>
<td>E_c = 4266 ksi</td>
</tr>
<tr>
<td>Modulus Elasticity of reinforcement</td>
<td>E_s = 29000 ksi</td>
</tr>
<tr>
<td>Modular Ratio</td>
<td>n = 6.798</td>
</tr>
<tr>
<td>Area of Steel per Design Strip</td>
<td>A_s = 0.196 in²</td>
</tr>
</tbody>
</table>

\[
A_0 = b \left( \frac{A_s}{s} \right)
\]

\[
\phi_{STR} = 0.90
\]

**Positive Moment Capacity**

\[
a = \frac{A_sf_y}{0.85f'_{c}b} = 0.289 \text{ in}
\]

Factored flexural resistance

\[
+\phi M_n = \phi A_s f_y \left( d_s - \frac{a}{2} \right) = 3.186 \text{ kip-ft}
\]

Check

\[
+\phi M_n > +M_u \quad 3.186 > 2.32 \quad \text{OK}
\]

**Negative Moment Capacity**

\[
a = \frac{A_sf_y}{0.85f'_{c}b} = 0.289 \text{ in}
\]

Factored flexural resistance

\[
-\phi M_n = \phi A_s f_y \left( d_{s(M^-)} - \frac{a}{2} \right) = 1.860 \text{ kip-ft}
\]

Check

\[
-\phi M_n > -M_u \quad 1.860 > 0.00 \quad \text{OK}
\]
Minimum Reinforcement  

AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r = \phi M_n \), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

\[
M_{cr} = \gamma_3 \left[ (\gamma_1 f_r + \gamma_2 \gamma_{cpe}) S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \right]
\]

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

where:

- flexural cracking variability factor \( \gamma_1 = 1.60 \) (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength \( \gamma_3 = 0.67 \) (A615 steel)
- concrete density modification factor \( \lambda = 1.00 \) AASHTO 5.4.2.8

- modulus of rapture \( f_r = 0.24 \lambda \sqrt{f'_c} \)
- section modulus of design section \( S_c = \frac{b h^2}{6} = \frac{b t_{panel^2}}{6} \)

Cracking moment \( M_{cr} = \gamma_3 \gamma_1 f_c S_c \)

\( M_{cr} = 3.09 \) kip-ft

Check positive moment reinforcement:

1.33 x factored ultimate moment \( = 3.08 \) kip-ft

Cracking moment \( = 3.09 \) kip-ft

min from (1.33 x factored +\( M_u \) and \( M_{cr} \)) \( = 3.08 \) kip-ft

Check: \( +\phi M_n > \min( +M_u , M_{cr} ) \)

3.186 > 3.08 OK

Check negative moment reinforcement:

1.33 x factored ultimate moment \( = 0.00 \) kip-ft

Cracking moment \( = 3.09 \) kip-ft

min from (1.33 x factored -\( M_u \) and \( M_{cr} \)) \( = 0.00 \) kip-ft

Check: \( -\phi M_n > \min( -M_u , M_{cr} ) \)

1.860 > 0.00 OK
# Typical Panel Type X, Panel Thickness 6", 5'x5', Vertical Rebar

## Shear on Panel

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness</td>
<td>6 in</td>
</tr>
<tr>
<td>Concrete Cover</td>
<td>2.5 in</td>
</tr>
<tr>
<td>Concrete Strength</td>
<td>4.0 ksi (Concrete Class A compressive strength)</td>
</tr>
<tr>
<td>Reinforcement Strength</td>
<td>60 ksi  (Minimum yield strength of grade 60 steel)</td>
</tr>
<tr>
<td>Bar Size</td>
<td>#4</td>
</tr>
<tr>
<td>Rebar Diameter</td>
<td>0.500 in</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>0.196 in$^2$</td>
</tr>
<tr>
<td>Rebar spacing</td>
<td>12 in</td>
</tr>
<tr>
<td>Width of the design section</td>
<td>12 in</td>
</tr>
<tr>
<td>Area of Steel per Design Strip</td>
<td>0.196 in$^2$</td>
</tr>
</tbody>
</table>

\[
c = \frac{A_s \cdot f_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b}
\]

\[
d_v = t_{\text{panel}} - \frac{c}{2} - C_{\text{panel}} - \frac{d_b}{2}
\]

\[c = 0.340 \text{ in}
\]

\[d_v = 3.080 \text{ in}
\]

## AASHTO 5.7.3.3

Vn shall be the lesser of:

(1) \( V_n = V_c + V_s \)

\[
V_c = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f'_c \cdot b_v \cdot d_v}
\]

\[V_c = 4.672 \text{ Kips}
\]

\[
V_s = \frac{A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha}{s}
\]

\[V_s = 3.024 \text{ Kips}
\]

(1) \( V_n = 7.696 \text{ Kips}
\]

(2) \( V_n = 0.25 \cdot f'_c \cdot b_v \cdot d_v \)

(2) \( V_n = 36.962 \text{ Kips}
\]

From (1) and (2) Vn should be:

\[\phi \cdot V_n = 0.9 \cdot V_n = 6.926 \text{ Kips}
\]

## Shear Force:

From Staad Pro Result Vmax = 0.303 N/mm$^2$

= 43.95 lbs/in$^2$

Shear Section Area = \( b \times t_{\text{panel}} \)

\[b = 12 \text{ in}, \quad t_{\text{panel}} = 6 \text{ in}
\]

\[V_{\text{max}} = 3164.15 \text{ lbs} = 3.164 \text{ Kips}
\]

OK, Vmax less than Vn
5’ x 5’ STANDARD Y PANEL
Typical Panel Type Y, Panel Thickness 6", 5'x5', Horizontal Rebar

**Panel Strength Design**

Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

<table>
<thead>
<tr>
<th>Panel Thickness</th>
<th>( t_{\text{panel}} ) = 6 in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Finishing Thickness</td>
<td>( C_{\text{panel}} ) = 1.5 in</td>
</tr>
<tr>
<td>Additional Load from Finishes</td>
<td>= 0.0125 Kip/ft²</td>
</tr>
</tbody>
</table>

Vertical Bar Size | #4 |

Rebar Diameter | \( d_b \) = 0.5 in |

Rebar Area | \( A_b \) = 0.196 in² |

Rebar spacing | \( s \) = 10 in |

Width of the design section | \( b \) = 12 in |

Effective depth of section | \( d_s \) = 4.25 in |

Effective depth of section for Negative Moment | \( d_{s(M^-)} \) = 1.75 in |

Correction factor for source aggregate | \( K_s \) = 1 in |

Concrete Cover | \( C_{\text{panel}} \) = 1.5 in |

Concrete Density | \( W_c \) = 0.15 kcf |

Concrete Strength | \( f'_{c} \) = 4.00 ksi |

Reinforcement Strength | \( f_y \) = 60 ksi |

Modulus Elasticity of concrete | \( E_c \) = 4266 ksi |

Modulus Elasticity of reinforcement | \( E_s \) = 29000 ksi |

Modular Ratio | \( n \) = 6.798 |

Area of Steel per Design Strip | \( A_s \) = 0.236 in² |

\[ +M_u = 4.00 \text{ kip-ft} \]

\[ -M_u = 0.00 \text{ kip-ft} \]

Resistance factor for tension-controlled section | \( \phi_{\text{STR}} \) = 0.90 |

**Positive Moment Capacity**

Depth of equivalent stress block | \( a = \frac{A_s f_y}{0.85 f'_{c} b} = 0.346 \text{ in} \)

Factored flexural resistance | \( +M_n = \phi A_s f_y \left( d_s - \frac{a}{2} \right) = 4.323 \text{ kip-ft} \)

Check | \(+\phi M_n > +M_u \) | 4.323 > 4.00 OK |

**Negative Moment Capacity**

Depth of equivalent stress block | \( a = \frac{A_s f_y}{0.85 f'_{c} b} = 0.346 \text{ in} \)

Factored flexural resistance | \( -M_n = \phi A_s f_y \left( d_{s(M^-)} - \frac{a}{2} \right) = 1.672 \text{ kip-ft} \)

Check | \(-\phi M_n > -M_u \) | 1.672 > 0.00 OK |
**Typical Panel Type Y, Panel Thickness 6”, 5’x5’, Horizontal Rebar**

**Shear on Panel**
- **Panel Thickness** \( t_{\text{panel}} = 6 \) in
- **Concrete Cover** \( C_{\text{panel}} = 1.5 \) in
- **Concrete Strength** \( f'_{c} = 4.0 \) ksi (Concrete Class A compressive strength)
- **Reinforcement Strength** \( f_{y} = 60 \) ksi (Minimum yield strength of grade 60 steel)
- **Bar Size** #4
- **Rebar Diameter** \( d_{b} = 0.500 \) in
- **Rebar Area** \( A_{b} = 0.196 \) in\(^2\)
- **Rebar spacing** \( s = 10 \) in
- **Width of the design section** \( b = 12 \) in
- **Area of Steel per Design Strip** \( A_{s} = 0.236 \) in\(^2\)

\[
c = \frac{A_{s} \cdot f_{s}}{\alpha_{1} \cdot f'_{c} \cdot \beta_{1} \cdot b}
\]

\[
d_{v} = t_{\text{panel}} - \frac{c}{2} - C_{\text{panel}} - \frac{d_{b}}{2}
\]

**AASHTO 5.7.3.3**

Vn shall be the lesser of:

1. \( V_{n} = V_{c} + V_{s} \)
\[
V_{c} = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f'_{c} \cdot b_{v} \cdot d_{v}}
\]
\[
V_{s} = A_{v} \cdot f_{y} \cdot d_{v} \cdot \left( \cot \theta + \cot \alpha \right) \cdot \sin \alpha
\]

\( (1) \ V_{n} = 10.904 \) Kips

2. \( V_{n} = 0.25 \cdot f'_{c} \cdot b_{v} \cdot d_{v} \)

\( (2) \ V_{n} = 48.554 \) Kips

From (1) and (2) Vn should be:

\( \phi \cdot V_{n} = 0.9 \cdot V_{n} = 9.814 \) Kips

**Shear Force:**

From Staad Pro Result \( V_{\text{max}} = 4.001 \) N/mm\(^2\) = 580.30 lbs/in\(^2\)

**Shear Section Area**

\[
b \times t_{\text{panel}} = 12 \times 6
\]

\( V_{\text{max}} = 41781.39 \) lbs

\( V_{\text{max}} = 41.781 \) Kips **not ok**
**Typical Panel Type Y, Panel Thickness 6”, 5’x5’, Vertical Rebar**

**Panel Strength Design**
Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness</td>
<td>t\textsubscript{panel} = 6 in</td>
<td></td>
</tr>
<tr>
<td>Finishing Thickness</td>
<td>C\textsubscript{panel} = 1.5 in</td>
<td></td>
</tr>
<tr>
<td>Additional Load from Finishes</td>
<td>= 0.0125 Kip/ft\textsuperscript{2}</td>
<td></td>
</tr>
<tr>
<td>Vertical Bar Size</td>
<td>#4</td>
<td></td>
</tr>
<tr>
<td>Rebar Diameter</td>
<td>d\textsubscript{b} = 0.5 in</td>
<td>d\textsubscript{bh} = 0.5 in</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>A\textsubscript{b} = 0.196 in\textsuperscript{2}</td>
<td>A\textsubscript{p} = \frac{1}{4} \pi d\textsubscript{b} \textsuperscript{2}</td>
</tr>
<tr>
<td>Rebar spacing</td>
<td>s = 10 in</td>
<td></td>
</tr>
<tr>
<td>Width of the design section</td>
<td>b = 12 in</td>
<td></td>
</tr>
<tr>
<td>Effective depth of section</td>
<td>d\textsubscript{s} = 3.75 in</td>
<td>d\textsubscript{s} = t\textsubscript{panel} - C\textsubscript{panel} - d\textsubscript{bh} - \frac{1}{2} d\textsubscript{b}</td>
</tr>
<tr>
<td>Effective depth of section for Negative Moment</td>
<td>d\textsubscript{s(M\textsuperscript{−})} = 2.25 in</td>
<td>d\textsubscript{s(M\textsuperscript{−})} = t\textsubscript{panel} - d\textsubscript{s}</td>
</tr>
<tr>
<td>Correction factor for source aggregate</td>
<td>K\textsubscript{s} = 1 in</td>
<td></td>
</tr>
<tr>
<td>Concrete Density</td>
<td>W\textsubscript{c} = 0.15 kcf</td>
<td></td>
</tr>
<tr>
<td>Concrete Strength</td>
<td>f\textsubscript{c}’ = 4.00 ksi</td>
<td>(Concrete Class A compressive strength)</td>
</tr>
<tr>
<td>Reinforcement Strength</td>
<td>f\textsubscript{y} = 60 ksi</td>
<td>(Minimum yield strength of grade 60 steel)</td>
</tr>
<tr>
<td>Modulus Elasticity of concrete</td>
<td>E\textsubscript{c} = 4266 ksi</td>
<td>E\textsubscript{c} = 120000k\textsubscript{t}W\textsubscript{c} \textsuperscript{2} f\textsubscript{c}’ \textsuperscript{0.33}</td>
</tr>
<tr>
<td>Modulus Elasticity of reinforcement</td>
<td>E\textsubscript{s} = 29000 ksi</td>
<td></td>
</tr>
<tr>
<td>Modular Ratio</td>
<td>n = 6.798</td>
<td>n = E\textsubscript{s}/E\textsubscript{c}</td>
</tr>
<tr>
<td>Area of Steel per Design Strip</td>
<td>A\textsubscript{s} = 0.236 in\textsuperscript{2}</td>
<td>A\textsubscript{s} = b\left(\frac{A\textsubscript{b}}{s}\right)</td>
</tr>
<tr>
<td>+M\textsubscript{u} = 3.73 kip-ft</td>
<td>+M\textsubscript{u,service} = 2.76 kip-ft (FK = 1.35)</td>
<td></td>
</tr>
<tr>
<td>-M\textsubscript{u} = 0.00 kip-ft</td>
<td>-M\textsubscript{u,service} = 0.00 kip-ft (FK = 1.35)</td>
<td></td>
</tr>
<tr>
<td>Resistance factor for tension-controlled section</td>
<td>(\phi_{STR} = 0.90)</td>
<td></td>
</tr>
</tbody>
</table>

**Positive Moment Capacity**
Depth of equivalent stress block
\[ a = \frac{A\textsubscript{s}f\textsubscript{y}}{0.85f\textsubscript{c}’b} = 0.346 \text{ in} \]
Factored flexural resistance
\[ +\varphi M\textsubscript{n} = \varphi A\textsubscript{s}f\textsubscript{y} \left( d\textsubscript{s} - \frac{a}{2} \right) = 3.792 \text{ kip-ft} \]
Check
\[ +\varphi M\textsubscript{n} > +M\textsubscript{u} \]
\[ 3.792 > 3.73 \text{ OK} \]

**Negative Moment Capacity**
Depth of equivalent stress block
\[ a = \frac{A\textsubscript{s}f\textsubscript{y}}{0.85f\textsubscript{c}’b} = 0.346 \text{ in} \]
Factored flexural resistance
\[ -\varphi M\textsubscript{n} = \varphi A\textsubscript{s}f\textsubscript{y} \left( d\textsubscript{s(M\textsuperscript{−})} - \frac{a}{2} \right) = 2.202 \text{ kip-ft} \]
Check
\[ -\varphi M\textsubscript{n} > -M\textsubscript{u} \]
\[ 2.202 > 0.00 \text{ OK} \]
Minimum Reinforcement

AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r = \phi M_n \), at least equal to the lesser of:

- 1.33 times the positive factored ultimate moment
- Cracking moment

Cracking moment

\[
M_{cr} = \gamma_3 \left[ (1.60 f_r + 0.67 \lambda \frac{S_c}{S_{nc}} - 1) \right]
\]

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 f_r S_c
\]

where:

- flexural cracking variability factor \( \gamma_1 = 1.60 \) (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength \( \gamma_3 = 0.67 \) (A615 steel)
- concrete density modification factor \( \lambda = 1.00 \) AASHTO 5.4.2.8

- modulus of rupture \( f_r = 0.24 \lambda \sqrt{f_c^t} \)
- section modulus of design section \( S_c = \frac{b h^2}{6} = \frac{b t_{panel}^2}{6} \)
- \( S_c = 72.00 \text{ in}^3 \)

Cracking moment

\( M_{cr} = \gamma_3 f_r S_c \)

\( M_{cr} = 3.09 \text{ kip-ft} \)

check positive moment reinforcement:

- 1.33 x factored ultimate moment \( = 4.96 \text{ kip-ft} \)
- Cracking moment \( = 3.09 \text{ kip-ft} \)
- min from (1.33 x factored +\( M_u \) and \( M_{cr} \)) \( = 3.09 \text{ kip-ft} \)

Check: \( +\phi M_n > \min(+M_u, M_{cr}) \)

\( 3.792 > 3.09 \text{ OK} \)

check negative moment reinforcement:

- 1.33 x factored ultimate moment \( = 0.00 \text{ kip-ft} \)
- Cracking moment \( = 3.09 \text{ kip-ft} \)
- min from (1.33 x factored -\( M_u \) and \( M_{cr} \)) \( = 0.00 \text{ kip-ft} \)

Check: \( -\phi M_n > \min(-M_u, M_{cr}) \)

\( 2.202 > 0.00 \text{ OK} \)
Minimum Reinforcement  AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, Mr = \( \phi M_n \), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

\[
M_{cr} = y_3 \left( \gamma_1 f_r + \gamma_2 y_{cpe} S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \right)
\]

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = y_3 y_1 f_r S_c
\]

where:

7. flexural cracking variability factor \( y_1 = 1.60 \) (non-segmental brg)
8. ratio of specified min. yield strength to ultimate tensile strength \( y_3 = 0.67 \) (A615 steel)
9. concrete density modification factor \( \lambda = 1.00 \) AASHTO 5.4.2.8

modulus of rapture \( f_r = 0.24 \lambda \sqrt{f_{c}'^3} \)

\( f_r = 0.48 \) ksi AASHTO 5.4.2.6

section modulus of design section \( S_c = \frac{b h^2}{6} = \frac{bt_{panel} l^2}{6} \)

\( S_c = 72.00 \) in\(^3\)

Cracking moment

\[
M_{cr} = y_3 y_1 f_r S_c
\]

\( M_{cr} = 3.09 \) kip-ft

check positive moment reinforcement:

1.33 x factored ultimate moment = 5.32 kip-ft

Cracking moment = 3.09 kip-ft

min from (1.33 x factored +\( M_u \) and \( M_{cr} \)) = 3.09 kip-ft

Check: \( +\phi M_n > \min( +M_u , M_{cr} ) \)

\( 4.323 > 3.09 \) OK

check negative moment reinforcement:

1.33 x factored ultimate moment = 0.00 kip-ft

Cracking moment = 3.09 kip-ft

min from (1.33 x factored -\( M_u \) and \( M_{cr} \)) = 0.00 kip-ft

Check: \( -\phi M_n > \min( -M_u , M_{cr} ) \)

\( 1.672 > 0.00 \) OK
Typical Panel Type Y, Panel Thickness 6", 5'x5', Vertical Rebar

Shear on Panel

Panel Thickness \( t_{\text{panel}} = 6 \) in
Concrete Cover \( c_{\text{panel}} = 2.5 \) in
Concrete Strength \( f'_{c} = 4.0 \) ksi (Concrete Class A compressive strength)
Reinforcement Strength \( f_{y} = 60 \) ksi (Minimum yield strength of grade 60 steel)
Bar Size #4
Rebar Diameter \( d_{b} = 0.500 \) in
Rebar Area \( A_{b} = 0.196 \) in\(^2\)
Rebar spacing \( s = 12 \) in
Width of the design section \( b = 12 \) in
Area of Steel per Design Strip \( A_{s} = 0.196 \) in\(^2\)

\[
c = \frac{A_{s} \cdot f_{s}}{\alpha_{s} \cdot f'_{c} \cdot \beta_{s} \cdot b}
\]
\[
d_{v} = t_{\text{panel}} - \frac{c}{2} - c_{\text{panel}} - \frac{d_{b}}{2}
\]

\[
c = 0.340 \text{ in}
\]
\[
d_{v} = 3.080 \text{ in}
\]

AASHTO 5.7.3.3

Vn shall be the lesser of:
(1) \( V_{n} = V_{c} + V_{s} \)

\[
V_{c} = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f'_{c} \cdot b_{v} \cdot d_{v}}
\]
\[
V_{c} = 4.672 \text{ Kips}
\]

\[
V_{s} = \frac{A_{v} \cdot f_{y} \cdot d_{v} \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha}{s}
\]
\[
V_{s} = 3.024 \text{ Kips}
\]

(1) \( V_{n} = 7.696 \text{ Kips} \)

(2) \( V_{n} = 0.25 \cdot f'_{c} \cdot b_{v} \cdot d_{v} \)

(2) \( V_{n} = 36.962 \text{ Kips} \)

From (1) and (2) Vn should be:

\[
\phi \cdot V_{n} = 0.9 \cdot V_{n}
\]
\[
\phi \cdot V_{n} = 6.926 \text{ Kips}
\]

Shear Force:

From Staad Pro Result \( V_{\text{max}} = 0.372 \text{ N/mm}^2 \)
\[
V_{\text{max}} = 53.95 \text{ lbs/in}^2
\]

Shear Section Area \( = b \times t_{\text{panel}} \)
\[
\text{Shear Section Area} = 12 \times 6
\]

\[
V_{\text{max}} = 3884.70 \text{ lbs}
\]
\[
V_{\text{max}} = 3.885 \text{ Kips} \quad \text{OK, Vmax less than Vn}\]
5’ x 6’ STANDARD PANELS
<table>
<thead>
<tr>
<th>Wire</th>
<th>Area sqin</th>
<th>Diameter in</th>
<th>Diameter after microns</th>
<th>Diameter after microns</th>
<th>% effective</th>
<th>Area after sqin</th>
</tr>
</thead>
<tbody>
<tr>
<td>W5</td>
<td>0.05</td>
<td>0.252</td>
<td>6408.757</td>
<td>4992.757</td>
<td>0.1966</td>
<td>0.0303</td>
</tr>
<tr>
<td>W8</td>
<td>0.08</td>
<td>0.319</td>
<td>8106.507</td>
<td>6690.507</td>
<td>0.2634</td>
<td>0.0545</td>
</tr>
<tr>
<td>W11</td>
<td>0.11</td>
<td>0.374</td>
<td>9505.722</td>
<td>8089.722</td>
<td>0.3185</td>
<td>0.0797</td>
</tr>
<tr>
<td>W15</td>
<td>0.15</td>
<td>0.437</td>
<td>11100.292</td>
<td>9684.292</td>
<td>0.3813</td>
<td>0.1142</td>
</tr>
<tr>
<td>W20</td>
<td>0.20</td>
<td>0.505</td>
<td>12817.513</td>
<td>11401.513</td>
<td>0.4489</td>
<td>0.1583</td>
</tr>
<tr>
<td>W24</td>
<td>0.24</td>
<td>0.553</td>
<td>14040.882</td>
<td>12624.882</td>
<td>0.4970</td>
<td>0.1940</td>
</tr>
<tr>
<td>W30</td>
<td>0.30</td>
<td>0.618</td>
<td>15698.184</td>
<td>14282.184</td>
<td>0.5623</td>
<td>0.2877</td>
</tr>
</tbody>
</table>
SPRING CONSTANT
Modulus of Sub-Grade Reaction and Spring Constant Check
From Principles of Foundation Engineering; Braja M. Das 8th Edition:

<table>
<thead>
<tr>
<th>Soil type</th>
<th>k₀,₃ (k₁)</th>
<th>MN/m³</th>
<th>lb/in³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry or moist sand:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>8–25</td>
<td>30–90</td>
<td></td>
</tr>
<tr>
<td>Medium</td>
<td>25–125</td>
<td>90–450</td>
<td></td>
</tr>
<tr>
<td>Dense</td>
<td>125–375</td>
<td>450–1350</td>
<td></td>
</tr>
<tr>
<td>Saturated sand:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>10–15</td>
<td>35–55</td>
<td></td>
</tr>
<tr>
<td>Medium</td>
<td>35–40</td>
<td>125–145</td>
<td></td>
</tr>
<tr>
<td>Dense</td>
<td>130–150</td>
<td>475–550</td>
<td></td>
</tr>
<tr>
<td>Clay:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stiff</td>
<td>10–25</td>
<td>40–90</td>
<td></td>
</tr>
<tr>
<td>Very stiff</td>
<td>25–50</td>
<td>90–185</td>
<td></td>
</tr>
<tr>
<td>Hard</td>
<td>&gt;50</td>
<td>&gt;185</td>
<td></td>
</tr>
</tbody>
</table>

\[ k₀,₃ (k₁) = \frac{1350}{\text{lb/in}^3} \quad \text{(is selected)} \]

(dense Sand, value is conservative considering no reinforcement is included)

Adjust to panel 5' x 10' (1.5m x 3m)

**Foundations on Sandy Soils**

For foundations on sandy soils,

\[ k = k₀,₃ \left( \frac{B + 0.3}{2B} \right)^2 \]  \hspace{1cm} (8.45)

where \( k₀,₃ \) and \( k \) = coefficients of subgrade reaction of foundations measuring 0.5 m x 0.3 m and \( B(m) \times B(m) \), respectively (unit is kN/m²).

In English units, Eq. (8.45) may be expressed as

\[ k = k₁ \left( \frac{B + 1}{2B} \right)^2 \]  \hspace{1cm} (8.46)

where \( k₁ \) and \( k \) = coefficients of subgrade reaction of foundations measuring 1 ft x 1 ft and \( B(ft) \times B(ft) \), respectively (unit is lb/ft²),

\[ k₁,₅ = k₀,₃ \cdot \left( \frac{B + 0.3}{2B} \right)^2 \]

\[ k₁,₅ = 1350 \times \left( \frac{1.5 + 0.3}{2 \times 1.5} \right)^2 \]

\[ k₁,₅ = 486 \quad \text{lb/in}^3 \]

\[ k₁,₅₂ = k₁,₅ \cdot \left( \frac{1 + \frac{1.5}{3.0}}{1.5} \right)^2 \]

\[ k₁,₅₂ = 486 \quad \text{lb/in}^3 \]

\[ k₁,₅₂ = 839.755 \quad \text{kip/ft}^2 \]
10.5 MODULUS OF SUBGRADE REACTION $k_z$ FOR MATS AND PLATES

All three discrete element methods given in this chapter for mats/plates use the modulus of subgrade reaction $k_z$ to support the plate. The modulus $k_z$ is used to compute node springs based on the contributing plan area of an element to any node as in Fig. 10-5. From the figure we see the following:

<table>
<thead>
<tr>
<th>Node</th>
<th>Contributing area</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (corner)</td>
<td>$\frac{1}{4}$ of rectangle abde</td>
</tr>
<tr>
<td>2 (side)</td>
<td>$\frac{1}{2}$ of abde + $\frac{1}{2}$ of bcef</td>
</tr>
<tr>
<td>3 (interior)</td>
<td>$\frac{1}{4}$ of each rectangle framing to a common node (as node 3)</td>
</tr>
</tbody>
</table>

For a triangle one should arbitrarily use one-third of the triangle area to any corner node. For these area contributions the fraction of $k_z$ node resistance from any element is

$$K_i = k_z \times \text{Area} \times m^2 = \text{units of kN/m (or kips/ft in Fps)}$$

Since this computation gives units of a “spring” it is common to call the effect a node spring.

In this form the springs are independent of each other, the system of springs supporting the plate is termed a “Winkler” foundation, and the springs are uncoupled. Uncoupling means that the deflection of any spring is not influenced by adjacent springs.

**Spring Constant**

Note Spacing: 1ft x 1ft

Tributary Area: 1’ x 1’ = 1SF

\[
K_a = k \times a
\]

\[
K_a = 839.755 \text{ kip/ft} \quad \text{(Input to Staad Pro)}
\]
MAXIMUM BENDING MOMENT
AND SHEAR
Typical Panel Type (mesh 3W11), Panel Thickness 6", 5'x6'

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
<th>X</th>
<th>Y</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>0.5</td>
<td>0.25</td>
<td></td>
</tr>
<tr>
<td>P2</td>
<td>0.1667</td>
<td>0.25</td>
<td></td>
</tr>
<tr>
<td>P3</td>
<td>-0.1667</td>
<td>0.25</td>
<td></td>
</tr>
<tr>
<td>P4</td>
<td>0.1667</td>
<td>-0.25</td>
<td></td>
</tr>
<tr>
<td>P5</td>
<td>-0.1667</td>
<td>-0.25</td>
<td></td>
</tr>
<tr>
<td>P6</td>
<td>0.5</td>
<td>-0.25</td>
<td></td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point
Bar Mat Used:
W11 - 75 Years

\[
A = 0.0797 \text{ in}^2 \\
F_y = 75 \text{ ksi} \\
P = 0.65 \times A \times F_y = 3.885 \text{ Kips (per wire point)}
\]

ATTACHMENT BY NO. WIRES
(standard panels only)
MX (local) kNm/m
- <= 0.394
- 0.568
- 0.742
- 0.915
- 1.09
- 1.26
- 1.44
- 1.61
- 1.78
- 1.96
- 2.13
- 2.31
- 2.48
- 2.65
- 2.83
- 3
- >= 3.18

Load 1

 STAAD.Pro for Windows 20.07.04.12
MY (local)
kNm/m

\[
\begin{array}{cccccccccccc}
\leq 0.336 & 0.757 & 1.18 & 1.6 & 2.02 & 2.44 & 2.86 & 3.28 & 3.71 & 4.13 & 4.55 & 4.97 & \geq 7.08 \\
\end{array}
\]
SOX (local)
N/mm²
-0.125
-0.110
-0.094
-0.078
-0.063
-0.047
-0.031
-0.016
0
0.016
0.031
0.047
0.063
0.078
0.094
0.110
>= 0.125

STAAD.Pro for Windows 20.07.04.12
1. STAAD SPACE

INPUT FILE: Panel Check - 5x6 type A (3W11).STD

2. START JOB INFORMATION

3. ENGINEER DATE 04-MAR-21

4. JOB CLIENT SUL

5. ENGINEER NAME CH

6. JOB KEY PANEL CHECK

7. END JOB INFORMATION

8. INPUT WIDTH 79

9. UNIT FEET KIP

10. JOINT COORDINATES

11. 1 0 0 0; 2 0 0 0; 3 2 0 0; 4 3 0 0; 5 4 0 0; 6 5 0 0; 7 6 0 0; 8 1 0 0 -1.

12. 11 1 -1; 12 1 -1; 13 3 0 -1; 14 4 0 -1; 15 5 0 -1; 16 6 0 -1; 17 9 0 0 -2.

13. 20 1 0 -2; 21 2 0 -2; 22 3 0 -2; 23 4 0 -2; 24 5 0 -2; 25 6 0 -2; 28 0 0 -3.

14. 29 1 0 -3; 30 2 0 -3; 31 3 0 -3; 32 4 0 -3; 33 5 0 -3; 34 6 0 -3; 45 0 0 -4.

15. 48 1 0 -4; 47 2 0 -4; 48 3 0 -4; 49 4 0 -4; 50 5 0 -4; 51 6 0 -4; 56 0 0 -5.

16. 57 3 0 -5; 58 2 0 -5; 59 3 0 -5; 60 4 0 -5; 61 5 0 -5; 62 6 0 -5.

17. ELEMENT TRUSSCES SHELL

18. 1 1 10 11 2 2 10 19 20 11; 3 19 28 29 30; 4 2 11 12 3 5 11 20 21 12.

19. 6 20 29 30 21; 7 3 12 13 4; 8 12 21 22 13; 9 21 30 21 22; 10 4 13 14 5.


21. 16 6 15 16 7; 17 15 24 25 14; 18 24 33 34 20; 21 28 45 46 29; 22 28 46 47 30.

22. 33 30 47 48 31; 34 31 48 49 32; 35 32 49 50 33; 36 33 50 51 34; 41 45 56 57 46.

23. 42 46 57 58 47; 43 47 58 59 48; 44 48 59 60 49; 45 49 60 61 50; 46 50 61 62 51.

24. ELEMENT PROPERTY

25. 1 TO 18 31 TO 36 41 TO 46 THICKNESS 0.5.

26. DEFINE MATERIAL START

27. ISOHOTROPIC CONCRETE


29. POISSON 0.17.

30. DENSITY 0.15.

31. ALPH 18-005.

32. EPSM 0.05.

33. END DEFINE MATERIAL

34. CONSTANTS

35. MATERIAL CONCRETE ALL

36. SUPPORTS

37. 1 TO 7 10 TO 16 19 TO 25 28 TO 34 45 TO 51 56 TO 61.

38. 43 FIXED NUT M10 819.76 KTY 819.76 KTY 819.76 KRY 1 KMY 1 KMY 1.

39. LOAD 1 LOADTYPE PUSH TITLE WIRE STRENGTHS

40. ELEMENT LOAD

STAAD SPACE

41. 32 FR GY -3.885 0.25 0.5.

42. 33 FR GY -3.885 0.25 0.1667.

43. 34 FR GY -3.885 0.25 -0.1667.

44. 8 FR GY -3.885 0.25 0.1667.

45. 11 FR GY -3.885 0.25 -0.1667.

46. 11 FR GY -3.885 -0.25 0.6.

47. PERFORM ANALYSIS

PROBLEM STATISTICS

NUMBER OF JOINTS/ MEMBER+ ELEMENTS/SUPPORTS = 42/ 30/ 42.

SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER.

ORIGINAL/FINAL BAND-WIDTH= 8/ 8/ 54 DOF.

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 252.

SIZE OF STIFFNESS MATRIX = 14 DOUBLE KILO-WORDS.

REQ/RD/AVAIL. DISK SPACE = 12.3/ 263443.7 MB.

48. FINISH.
Typical Panel Type (mesh 4W11), Panel Thickness 6", 5'x6'

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
</tr>
<tr>
<td>P1</td>
<td>0.5</td>
</tr>
<tr>
<td>P2</td>
<td>0.1667</td>
</tr>
<tr>
<td>P3</td>
<td>-0.1667</td>
</tr>
<tr>
<td>P4</td>
<td>0.5</td>
</tr>
<tr>
<td>P5</td>
<td>0.5</td>
</tr>
<tr>
<td>P6</td>
<td>0.1667</td>
</tr>
<tr>
<td>P7</td>
<td>-0.1667</td>
</tr>
<tr>
<td>P8</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[
A = 0.0797 \text{ in}^2 \\
F_y = 75 \text{ ksi} \\
P = \frac{0.65 \times A \times F_y}{\text{per wire point}} = 3.885 \text{ Kips}
\]
MX (local)
kNm/m

| <= 0.887 |
| 1.06    |
| 1.23    |
| 1.41    |
| 1.58    |
| 1.75    |
| 1.93    |
| 2.1     |
| 2.27    |
| 2.45    |
| 2.62    |
| 2.79    |
| 2.97    |
| 3.14    |
| 3.31    |
| 3.49    |
| >= 3.66 |

STAAD.Pro for Windows 20.07.04.12
MY (local)
\( kNm/m \)
\[ \leq 0.788 \]
\[ 1.24 \]
\[ 1.7 \]
\[ 2.16 \]
\[ 2.62 \]
\[ 3.07 \]
\[ 3.53 \]
\[ 3.99 \]
\[ 4.44 \]
\[ 4.9 \]
\[ 5.36 \]
\[ 5.81 \]
\[ 6.27 \]
\[ 6.73 \]
\[ 7.18 \]
\[ 7.64 \]
\[ \geq 8.1 \]
SQY (local)
N/mm²

-0.149
-0.130
-0.112
-0.093
-0.074
-0.056
-0.037
-0.019
0
0.019
0.037
0.056
0.074
0.093
0.112
0.130
0.149

5.00ft
6.00ft

Load 1
Problem Statistics

Number of Joints/Member/Elements/Supports = 42/30/42

Solver Used is the Out-of-Core Basic Solver

Original/Final Band Width = 8/8/54 DFS
Total Primary Load Cases = 1. Total Degrees of Freedom = 252
Size of Stiffness Matrix = 14 Double Kilograms

50. Finish
Typical Panel Type (mesh 5W11), Panel Thickness 6”, 5’x6’

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>X</th>
<th>Y</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>-0.1667</td>
<td>0.25</td>
</tr>
<tr>
<td>P2</td>
<td>0.5</td>
<td>0.25</td>
</tr>
<tr>
<td>P3</td>
<td>0.1667</td>
<td>0.25</td>
</tr>
<tr>
<td>P4</td>
<td>-0.1667</td>
<td>0.25</td>
</tr>
<tr>
<td>P5</td>
<td>0.5</td>
<td>0.25</td>
</tr>
<tr>
<td>P6</td>
<td>0.5</td>
<td>-0.25</td>
</tr>
<tr>
<td>P7</td>
<td>0.1667</td>
<td>-0.25</td>
</tr>
<tr>
<td>P8</td>
<td>-0.1667</td>
<td>-0.25</td>
</tr>
<tr>
<td>P9</td>
<td>0.5</td>
<td>-0.25</td>
</tr>
<tr>
<td>P10</td>
<td>0.1667</td>
<td>-0.25</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point
Bar Mat Used:
W11 - 75 Years

\[
A = 0.0797 \text{ in}^2 \\
F_y = 75 \text{ ksi} \\
P = 0.65 \times A \times F_y = 3.885 \text{ Kips (per wire point)}
\]

ATTACHMENT BY No. WIRES
(standard panels only)
MX (local)
kNm/m

\[
\begin{array}{cccccccccccc}
  \leq 0.78 & 1.01 & 1.25 & 1.48 & 1.72 & 1.95 & 2.18 & 2.42 & 2.65 & 2.89 & 3.12 & 3.35 & 3.59 & 3.82 & 4.05 & 4.29 & \geq 4.52 \\
\end{array}
\]

Print Time/Date: 06/03/2021 22:05
MY (local) kNm/m

- <= 0.709
- 1.2
- 1.69
- 2.18
- 2.67
- 3.15
- 3.64
- 4.13
- 4.62
- 5.11
- 5.6
- 6.09
- 6.58
- 7.07
- 7.56
- 8.05
- >= 8.54
SOX (local)
N/mm²

-0.147
-0.129
-0.110
-0.092
-0.074
-0.055
-0.037
-0.018
0
0.018
0.037
0.055
0.074
0.092
0.110
0.129
>= 0.147
SQY (local) N/mm²

- <= -0.149
- -0.131
- -0.112
- -0.093
- -0.075
- -0.056
- -0.037
- -0.019
- 0
- 0.019
- 0.037
- 0.056
- 0.075
- 0.093
- 0.112
- 0.131
- >= 0.149
1. STAAD SPACE

INPUT FILE: Panel Check - Sx6 type A (SM11).STD

2. START JOB INFORMATION

3. ENGINEER DATE 04-MAY-21

4. JOB CLIENT SUL

5. ENGINEER NAME CH

6. JOB REF PANEL CHECK

7. END JOB INFORMATION

8. INPUT WIDTH 79

9. UNIT PIES KIP

10. JOINT COORDINATES

11. 1 0 0 0; 2 1 0 0; 3 2 0 0; 4 3 0 0; 5 4 0 0; 6 5 0 0; 7 6 0 0; 10 0 0 -1

12. 11 1 0 -1; 12 2 0 -1; 13 3 0 -1; 14 4 0 -1; 15 5 0 -1; 16 6 0 -1; 19 0 0 -2

13. 20 1 0 -2; 21 2 0 -2; 22 3 0 -2; 23 4 0 -2; 24 5 0 -2; 25 6 0 -2; 28 0 0 -3

14. 29 1 0 -3; 30 2 0 -3; 31 3 0 -3; 32 4 0 -3; 33 5 0 -3; 34 6 0 -3; 35 6 0 -4

15. 48 1 0 -4; 47 2 0 -4; 43 3 0 -4; 43 4 0 -4; 50 5 0 -4; 51 6 0 -4; 56 0 0 -5

16. 57 3 0 -5; 58 2 0 -5; 59 3 0 -5; 60 4 0 -5; 61 5 0 -5; 62 6 0 -6

17. ELEMENT TRUSS/BEAM SHELL

18. 1 1 10 10 2 2 10 10 12 11 1 19 28 29 30; 4 2 11 12 3 5 11 20 21 12

19. 6 20 29 30 21; 7 3 12 13 4; 8 12 21 22 13; 9 21 30 21 22; 10 4 13 14 5

20. 11 13 22 23 14; 12 22 31 32 23; 13 5 34 15 6; 14 14 25 24 15; 15 23 32 33 24

21. 16 6 15 16 7; 17 15 24 25 16; 18 24 33 34 20; 31 28 40 46 29; 32 29 46 47 30

22. 33 30 47 48 31; 34 31 48 49 30; 35 32 49 50 33; 36 33 50 51 34; 41 45 56 57 46

23. 42 46 57 58 47; 43 47 58 59 48; 44 48 59 60 49; 45 49 60 61 50; 46 50 61 62 51

24. ELEMENT PROPERTY

25. 1 TO 18 31 TO 36 41 TO 46 YTHICKNESS 0.5

26. DEFINE MATERIAL START

27. ISOTROPIC CONCRETE

28. E 614304

29. POISSON 0.17

30. DENSITY 0.15

31. ALPHA 1E-005

32. EAM 0.03

33. END DEFINE MATERIAL

34. CONSTANTS

35. MATERIAL CONCRETE ALL

36. SUPPORTS

37. 1 TO 7 10 TO 16 19 TO 25 28 TO 34 45 TO 51 58 TO 61

38. 42 FIXED NUT M3K 839.76 KTY 839.76 KTY 839.76 KMK 1 KNY 1 KMK 1

39. LOAD 1 LOADTYPE PUSH TITLE WIRE STRENGTHS

40. ELEMENT LOAD

PENDING STATISTICS

---------------------------------------------------------------------

NUMBER OF JOINTS/MEMBER/ELEMENTS/SUPPORTS = 42/ 30/ 42

SOLVER USED IS THE GON-OFF-CORE BASIC SOLVER

ORIGINAL/FINAL BAND-WIDTH = 8/ 8/ 54 DOP

TOTAL PRIMARY LOAD CASES= 1, TOTAL DEGREES OF FREEDOM = 252

SIZE OF STIFFNESS MATRIX = 14 DOUBLE KILO-WORDS

REQD/AVAIL. DISK SPACE = 12.3/ 263438.8 MB

52. FINISH

41. 32 FY 3.885 0.25 -0.1667

42. 32 FY 3.885 0.25 0.5

43. 33 FY 3.885 0.25 0.25

44. 34 FY 3.885 0.25 -0.1667

45. 34 FY 3.885 0.25 0.5

46. 5 FY 3.885 -0.25 0.5

47. 8 FY 3.885 -0.25 0.1667

48. 11 FY 3.885 -0.25 -0.1667

49. 11 FY 3.885 -0.25 0.5

50. 14 FY 3.885 -0.25 0.1667

51. PERFORM ANALYSIS
Typical Panel Type (mesh 6W11), Panel Thickness 6", 5'x6'

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
</tr>
<tr>
<td>P1</td>
<td>-0.1667</td>
</tr>
<tr>
<td>P2</td>
<td>0.1667</td>
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<tr>
<td>P3</td>
<td>0.5</td>
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<tr>
<td>P4</td>
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<td>P5</td>
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<td>P6</td>
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<tr>
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<tr>
<td>P11</td>
<td>0.1667</td>
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<tr>
<td>P12</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[
A = 0.0797 \text{ in}^2 \\
F_y = 75 \text{ ksi} \\
P = 0.65 \times A \times F_y = 3.885 \text{ Kips (per wire point)}
\]

ATTACHMENT BY No. WIRES
(standard panels only)
MX (local) kNm/m

<table>
<thead>
<tr>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;= 1.76</td>
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<tr>
<td>1.95</td>
</tr>
<tr>
<td>2.14</td>
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<tr>
<td>2.33</td>
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<td>2.52</td>
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<td>2.71</td>
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<td>2.9</td>
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<td>4.43</td>
</tr>
<tr>
<td>4.62</td>
</tr>
<tr>
<td>&gt;= 4.81</td>
</tr>
</tbody>
</table>

Print Time/Date: 06/03/2021 22:42
Print Run 1 of 1
SOX (local)
N/mm²

|-0.151|
|-0.133|
|-0.114|
|-0.095|
|-0.076|
|-0.057|
|-0.038|
|-0.019|
|0     |
|0.019 |
|0.038 |
|0.057 |
|0.076 |
|0.095 |
|0.114 |
|0.133 |
|>= 0.151|

Load 1

STAAD.Pro for Windows 20.07.04.12
SQY (local) N/mm²

- <= -0.125
- -0.109
- -0.094
- -0.078
- -0.062
- -0.047
- -0.031
- -0.016
- 0
- 0.016
- 0.031
- 0.047
- 0.062
- 0.078
- 0.094
- 0.109
- >= 0.125

Print Time/Date: 06/03/2021 22:43

STAAD.Pro for Windows 20.07.04.12

Print Run 1 of 1
**Typical Panel Type (mesh 4W20), Panel Thickness 6", 5'x6'**

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
</tr>
<tr>
<td>P1</td>
<td>0.5</td>
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<tr>
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<tr>
<td>P3</td>
<td>-0.1667</td>
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<tr>
<td>P4</td>
<td>0.5</td>
</tr>
<tr>
<td>P5</td>
<td>0.5</td>
</tr>
<tr>
<td>P6</td>
<td>0.1667</td>
</tr>
<tr>
<td>P7</td>
<td>-0.1667</td>
</tr>
<tr>
<td>P8</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[
A = 0.1583 \text{ in}^2 \\
F_y = 75 \text{ ksi} \\
P = 0.65 \times A \times F_y \\
= 7.717 \text{ Kips (per wire point)}
\]

**ATTACHMENT BY No. WIRES**

(standard panels only)
### Panel Check - 5x6 type A

**MX (local)**

<table>
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<th>Value</th>
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<tbody>
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<td>2.45</td>
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<td>2.79</td>
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<td>3.14</td>
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<tr>
<td>3.48</td>
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</tr>
<tr>
<td>3.83</td>
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</tr>
<tr>
<td>4.17</td>
<td></td>
</tr>
<tr>
<td>4.52</td>
<td></td>
</tr>
<tr>
<td>4.86</td>
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</tr>
<tr>
<td>5.2</td>
<td></td>
</tr>
<tr>
<td>5.55</td>
<td></td>
</tr>
<tr>
<td>5.89</td>
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</tr>
<tr>
<td>6.24</td>
<td></td>
</tr>
<tr>
<td>6.58</td>
<td></td>
</tr>
<tr>
<td>6.93</td>
<td></td>
</tr>
<tr>
<td>7.27</td>
<td></td>
</tr>
</tbody>
</table>

**Load 1**

- Load 1
- Load 2
- Load 3
- Load 4
- Load 5
- Load 6
- Load 7
- Load 8
- Load 9
- Load 10
- Load 11
- Load 12
- Load 13
- Load 14
- Load 15
- Load 16
- Load 17
- Load 18
- Load 19
- Load 20

**Notes:**

- The calculations were performed using STAAD.Pro for Windows 20.07.04.12.
- The file was printed on 07/03/2021 at 22:38.
MY (local)
kNm/m

<table>
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<th>Description</th>
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</thead>
<tbody>
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<tr>
<td>3.38</td>
<td></td>
</tr>
<tr>
<td>4.29</td>
<td></td>
</tr>
<tr>
<td>5.2</td>
<td></td>
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<tr>
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<td>7.01</td>
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</tr>
<tr>
<td>7.92</td>
<td></td>
</tr>
<tr>
<td>8.83</td>
<td></td>
</tr>
<tr>
<td>9.73</td>
<td></td>
</tr>
<tr>
<td>10.6</td>
<td></td>
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<tr>
<td>11.5</td>
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<td>12.5</td>
<td></td>
</tr>
<tr>
<td>13.4</td>
<td></td>
</tr>
<tr>
<td>14.3</td>
<td></td>
</tr>
<tr>
<td>15.2</td>
<td></td>
</tr>
<tr>
<td>&gt;= 16.1</td>
<td></td>
</tr>
</tbody>
</table>
1. STAAD SPACE

INPUT FILE: Panel Check - S6t type A (4620).STD

2. START JOB INFORMATION

3. ENGINEER DATE 07-MAR-21

4. JOB CLIENT SUL

5. ENGINEER NAME CH

6. JOB REF PANEL CHECK

7. END JOB INFORMATION

8. INPUT WIDTH 79

9. UNIT FEET KIP

10. JOINT COORDINATES

11. 1 0 0 0; 2 1 0 0; 3 2 0 0; 4 3 0 0; 5 4 0 0; 6 5 0 0; 7 6 0 0; 10 0 0 -1

12. 11 1 0 -1; 12 2 0 -1; 13 3 0 -1; 14 4 0 -1; 15 5 0 -1; 16 6 0 -1; 19 0 0 -2

13. 20 1 0 -2; 21 2 0 -2; 22 3 0 -2; 23 4 0 -2; 24 5 0 -2; 25 6 0 -2; 28 0 0 -3

14. 29 1 0 -3; 30 2 0 -3; 31 3 0 -3; 32 4 0 -3; 33 5 0 -3; 34 6 0 -3; 45 0 0 -4

15. 48 1 0 -4; 47 2 0 -4; 48 3 0 -4; 49 4 0 -4; 50 5 0 -4; 51 6 0 -4; 56 0 0 -5

16. 57 3 0 -5; 58 2 0 -5; 59 3 0 -5; 60 4 0 -5; 61 5 0 -5; 62 6 0 -5

17. ELEMENT TRIDIAGONAL SHELL

18. 1 1 10 11; 2 10 19 20; 11 10 19 20; 13 19 28 29; 2 11 12 3; 11 10 20 21; 12

19. 13 20 29 30; 21 7 12 13; 8 12 21 22; 9 21 30 22; 10 4 13 14; 5

20. 11 13 22 23; 12 21 30 32; 23 13 5 14 15 6; 14 14 3 24 24 15; 15 13 32 33 24; 16

21. 16 6 15 16; 17 15 24 25 16; 18 24 33 34 20; 31 28 40 46 29; 32 29 46 47 30; 22

22. 33 30 47 48 31; 34 31 48 49 32; 35 32 49 50 33; 36 33 50 51 34; 41 45 56 57 46; 23

23. 42 46 57 58 47; 43 47 58 59 48; 44 48 59 60 49; 45 49 60 61 50; 46 50 61 62 51; 24

24. ELEMENT PROPERTY

25. 1 TO 18 31 TO 36 41 TO 46 THICKNESS 0.5

26. DEFINE MATERIAL STAGE

27. ISOTROPIC CONCRETE

28. E 61434

29. POISSON 0.17

30. DENSITY 0.15

31. ALUM 12-005

32. DAMP 0.05

33. END DEFINE MATERIAL

34. CONSTANTS

35. MATERIAL CONCRETE ALL

36. SUPPORTS

37. 1 TO 7 10 TO 16 19 TO 25 28 TO 34 45 TO 51 56 TO 61 -

38. 42 FIXED HUT KFX 89.74 KFY 89.74 KFZ 89.74 KMY 1 KMY 1 KMY 1

39. LOAD 1 LOADTYPE PUSH TITLE WIRE STRENGTHS

40. ELEMENT LOAD

Problem Statistics

NUMBER OF JOINTS/MEMBER ELEMENTS/SUPPORTS = 42/30/42

SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER

ORIGIN/FINAL BAND WIDTHS = 8/8/54 DOF

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEG OF FREEDOM = 252

SIZE OF STIFFNESS MATRIX = 14 DOUBLE XILO-WORDS

RGMAT/AVAIL. DISK SPACE = 13.5/263423.7 MB

50. FINISH
**Typical Panel Type (mesh 5W20), Panel Thickness 6'', 5'x6'**

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
</tr>
<tr>
<td>P1</td>
<td>-0.1667</td>
</tr>
<tr>
<td>P2</td>
<td>0.5</td>
</tr>
<tr>
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<tr>
<td>P6</td>
<td>0.5</td>
</tr>
<tr>
<td>P7</td>
<td>0.1667</td>
</tr>
<tr>
<td>P8</td>
<td>-0.1667</td>
</tr>
<tr>
<td>P9</td>
<td>0.5</td>
</tr>
<tr>
<td>P10</td>
<td>0.1667</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point
Bar Mat Used:
W11 - 75 Years

\[
A = 0.1583 \text{ in}^2
\]

\[
F_y = 75 \text{ ksi}
\]

\[
P = 0.65 \times A \times F_y = 7.717 \text{ Kips (per wire point)}
\]

3–WIRE CONNECTION PANEL
4–WIRE CONNECTION PANEL
5–WIRE CONNECTION PANEL

**ATTACHMENT BY No. WIRES**
(Standard Panels Only)
Panel Check - 5x6 type A

5.00ft

6.00ft

Load 1

Print Time/Date: 07/03/2021 22:58
Print Run 1 of 1

STAAD.Pro for Windows 20.07.04.12
MX (local) kNm/m

- <= 1.55
- 2.01
- 2.48
- 2.94
- 3.41
- 3.87
- 4.34
- 4.8
- 5.27
- 5.73
- 6.2
- 6.66
- 7.12
- 7.59
- 8.05
- 8.52
- >= 8.98
MY (local) kNm/m

<table>
<thead>
<tr>
<th>Value (kNm/m)</th>
</tr>
</thead>
<tbody>
<tr>
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<td>8.21</td>
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<td>9.18</td>
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<tr>
<td>10.2</td>
</tr>
<tr>
<td>11.1</td>
</tr>
<tr>
<td>12.1</td>
</tr>
<tr>
<td>13.1</td>
</tr>
<tr>
<td>14</td>
</tr>
<tr>
<td>15</td>
</tr>
<tr>
<td>16</td>
</tr>
<tr>
<td>&gt;= 17</td>
</tr>
</tbody>
</table>

Load 1
SQY (local)
N/mm²

<= -0.296
-0.259
-0.222
-0.185
-0.148
-0.111
-0.074
-0.037
0
0.037
0.074
0.111
0.148
0.185
0.222
0.259
>= 0.296
1. STAAD SPACE

INPUT FILE: Panel Check - 5x6 type A (5x62).STD

2. START JOB INFORMATION

3. ENGINEER DATE C-D-2021

4. JOB CLIENT SIS

5. ENGINEER NAME CH

6. JOB REF PANEL CHECK

7. END JOB INFORMATION

8. INPUT WIDTH 79

9. UNIT REAL WIP

10. JOINT COORDINATES

11. 1 0 0 0 0 2 1 0 0 0 0 3 2 0 0 0 4 3 0 0 0 5 4 0 0 0 6 5 0 0 0 7 6 0 0 0 10 0 0 0

12. 11 1 0 0 0 12 2 0 0 0 13 3 0 0 0 14 4 0 0 0 15 5 0 0 0 16 6 0 0 0 19 0 0 0

13. 20 1 0 0 2 11 2 0 0 2 12 3 0 0 2 13 4 0 0 2 14 5 0 0 2 15 6 0 0 2 18 0 0 0

14. 19 1 0 0 3 10 2 0 0 3 11 3 0 0 3 12 4 0 0 3 13 5 0 0 3 14 6 0 0 3 17 0 0 0

15. 12 1 0 0 4 9 2 0 0 4 10 3 0 0 4 11 4 0 0 4 12 5 0 0 4 13 6 0 0 4 16 0 0 0

16. 5 7 0 0 5 8 2 0 0 5 9 3 0 0 5 6 0 0 0 5 6 0 0 0 5 6 0 0 0 5 6 0 0 0

17. ELEMENT TRUSS ELEMENTS SHELL

18. 1 1 10 1 1 2 1 10 1 1 11 1 1 19 28 29 30 3 2 1 1 12 3 1 1 11 20 21 12

19. 6 20 20 30 21 3 7 12 13 4 8 12 21 22 13 9 21 30 22 10 11 13 4 5

20. 11 13 22 23 14 12 22 31 23 13 5 4 14 28 14 14 25 24 15 13 32 23 24

21. 14 3 15 16 7 17 15 24 25 14 18 24 33 34 20 31 28 45 46 29 32 46 47 30

22. 33 30 47 48 31 34 31 48 49 30 35 32 49 50 33 36 33 50 51 34 41 45 56 57 46

23. 42 47 37 58 47 42 47 38 59 48 44 48 59 60 49 45 49 60 61 50 46 50 61 62 51

24. ELEMENT PROPERTY

25. 1 TO 18 31 TO 36 41 TO 46 THICKNESS 0.5

26. DEFINE MATERIAL START

27. ISOTROPIC CONCRETE

28. E 614504

29. POISSON 0.17

30. DENSITY 0.15

31. ALPH A 0-005

32. DAMP 0.03

33. END DEFINE MATERIAL

34. CONSTANTS

35. MATERIAL CONCRETE ALL

36. SUPPORTS

37. 1 TO 7 10 TO 16 19 TO 25 28 TO 34 45 TO 51 56 TO 61 -

38. 46 FIXED HUT RTX 839.76 RTY 89.76 RFT 839.76 RKM 1 RMY 1 RMZ 1

39. LOAD 1 LOADTYPE PUSH TITLE WIRE STRENGTHS

40. ELEMENT LOAD

**PROBLEM STATISTICS**

---------------------------------------------------------------------

NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS = 42/ 30/ 42

SOLVER USED IS THE GNP-CODE BASIC SOLVER

ORIGINAL/FINAL BANDWIDTH= 8/ 8/ 54 DOF

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 252

SIZE OF STIFFNESS MATRIX = 14 DOUBLE Xilo-Words

REQUEST/AVAIl. DISK SPACE = 12.3/ 263422.0 MB

52. FINISH
Typical Panel Type (mesh 6W20), Panel Thickness 6", 5'x6'

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>X</th>
<th>Y</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>-0.1667</td>
<td>0.25</td>
</tr>
<tr>
<td>P2</td>
<td>0.5</td>
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<td>0.1667</td>
<td>0.25</td>
</tr>
<tr>
<td>P7</td>
<td>-0.1667</td>
<td>-0.25</td>
</tr>
<tr>
<td>P8</td>
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<td>-0.25</td>
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<tr>
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<td>0.1667</td>
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<td>P10</td>
<td>-0.1667</td>
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<tr>
<td>P12</td>
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<td>-0.25</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[
A = 0.1583 \text{ in}^2 \\
F_y = 75 \text{ ksi} \\
P = 0.65 \times A \times F_y \\
= 7.717 \text{ Kips (per wire point)}
\]

ATTACHMENT BY No. WIRES
(STANDARD PANELS ONLY)
MX (local) kNm/m

- <= 3.49
- 3.87
- 4.25
- 4.63
- 5
- 5.38
- 5.76
- 6.14
- 6.52
- 6.9
- 7.28
- 7.66
- 8.03
- 8.41
- 8.79
- 9.17
- 9.55

Load 1

5.00ft

6.00ft

Load 1
MY (local) kNm/m

- <= 3.13
- 3.98
- 4.83
- 5.69
- 6.54
- 7.39
- 8.25
- 9.1
- 9.95
- 10.8
- 11.7
- 12.5
- 13.4
- 14.2
- 15.1
- 15.9
- >= 16.8

Load 1
SSL

Panel Check

Panel Check - 5x6 type A

SQX (local)
N/mm²

-0.301
-0.263
-0.226
-0.188
-0.150
-0.113
-0.075
-0.038
0
0.038
0.075
0.113
0.150
0.188
0.226
0.263
0.301

STAAD.Pro for Windows 20.07.04.12

Print Time/Date: 07/03/2021 23:35
Print Run 1 of 1
1. STAAD SPACE
INPUT FILE: Panel Check - 3x6 type A (BM20).STD
2. START JOB INFORMATION
3. ENGINEER DATE C-AAA-02
4. JOB CLIENT SEL
5. ENGINEER NAME CHE
6. JOB REF PANEL CHECK
7. END JOB INFORMATION
8. INPUT WIDTH 79
9. UNIT PRET KIP
10. JOINT COORDINATES

11. 1 0 0 0; 2 1 0 0; 3 2 0 0; 4 3 0 0; 5 4 0 0; 6 5 0 0; 7 6 0 0; 10 0 0 -1
12. 11 1 0 -1; 12 2 0 -1; 13 3 0 -1; 14 4 0 -1; 15 5 0 -1; 16 6 0 -1; 19 0 0 -2
13. 20 1 0 -2; 21 2 0 -2; 22 3 0 -2; 23 4 0 -2; 24 5 0 -2; 25 6 0 -2; 28 0 0 -3
14. 29 1 0 -3; 30 2 0 -3; 31 3 0 -3; 32 4 0 -3; 33 5 0 -2; 34 6 0 -2; 35 0 0 -4
15. 48 1 0 -4; 49 2 0 -4; 50 3 0 -4; 51 4 0 -4; 52 5 0 -4; 53 6 0 -4; 56 0 0 -5
16. 57 3 0 -5; 58 2 0 -5; 59 3 0 -5; 60 4 0 -5; 61 5 0 -5; 62 6 0 -5
17. ELEMNT IDENTIFICATIONS SHELL
18. 1 3 10 31 2 1 2 10 19 20 11 3 19 28 29 30 4 2 11 12 3 5 11 20 21 12
19. 6 20 29 30 21 7 12 13 4 8 12 21 22 13 9 21 30 21 22 10 4 13 14 5
20. 11 13 22 23 14; 12 22 31 23 23 13 5 14 15 6; 14 14 23 24 15; 15 23 32 23 24
21. 16 6 15 16 7; 17 15 24 25 14; 18 24 33 34 20; 31 28 40 46 29; 32 28 40 46 30
22. 33 30 47 48 31; 34 31 48 49 30; 35 32 49 50 33; 36 33 50 51 34; 37 45 56 57 46
23. 42 46 57 58 47; 43 47 58 59 48; 44 48 59 60 49; 45 49 60 61 50; 46 50 61 62 51
24. ELEMNT PROPERTY
25. 1 TO 18 31 TO 36 41 TO 46 THICKNESS 0.5
26. DEFINE MATERIAL START
27. ISOTROPIC CONCRETE
28. E 61400
29. POISSON 0.17
30. DENSITY 0.15
31. ALPHA 1E-005
32. DAMP 0.03
33. END DEFINE MATERIAL
34. CONSTANTS
35. MATERIAL CONCRETE ALL
36. SUPPORTS
37. 1 TO 7 10 TO 16 19 TO 20 25 TO 34 45 TO 51 56 TO 61
38. 42 FIXED RUT RXY 839.76 RXY 839.76 RXY 839.76 KM 1 KM 1 KM 1
39. LOAD 1 LOADTYPE PUSH TITTLE WIRE STRENGTHS
40. ELEMENT LOAD

PROBLEM STATISTICS
-------------------------------------
NUMBER OF JOINTS/MEMBER/ELEMENTS/SUPPORTS = 42/ 30/ 42
SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER
ORIGINAL/FINAL BAND-WIDTH= 8/ 8/ 54 DOF
TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 252
SIZE OF STIFFNESS MATRIX = 14 DOUBLE KILO-WORDS
MEMLD/AVAIL. DISK SPACE = 12.3/ 263420.3 MB
54. FINISH
Typical Panel Type (mesh 5W24), Panel Thickness 6", 5'x6'

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
</tr>
<tr>
<td>P1</td>
<td>-0.1667</td>
</tr>
<tr>
<td>P2</td>
<td>0.5</td>
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<td>P3</td>
<td>0.1667</td>
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<tr>
<td>P4</td>
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<tr>
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<td>0.5</td>
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<tr>
<td>P6</td>
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<tr>
<td>P7</td>
<td>0.1667</td>
</tr>
<tr>
<td>P8</td>
<td>-0.1667</td>
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<tr>
<td>P9</td>
<td>0.5</td>
</tr>
<tr>
<td>P10</td>
<td>0.1667</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[
A = 0.194 \text{ in}^2 \\
F_y = 75 \text{ ksi}
\]

\[
P = 0.65 \times A \times F_y \\
= 9.458 \text{ Kips (per wire point)}
\]
MX (local) kNm/m

- <= 1.9
- 2.47
- 3.04
- 3.61
- 4.18
- 4.75
- 5.32
- 5.89
- 6.45
- 7.02
- 7.59
- 8.16
- 8.73
- 9.3
- 9.87
- 10.4
- >= 11

Load 1
MY (local) kNm/m
<= 1.73
2.92
4.11
5.3
5.49
7.68
8.87
10.1
11.3
12.4
13.6
14.8
16
17.2
18.4
19.6
>= 20.8

Load 1

5.00ft
6.00ft
SOX (local) N/mm²

-0.359
-0.314
-0.269
-0.224
-0.179
-0.134
-0.090
-0.045
0
0.045
0.090
0.134
0.179
0.224
0.269
0.314
>= 0.359
### SQY (local)
\[ \text{N/mm}^2 \]

- \( \leq -0.363 \)
- \(-0.318\)
- \(-0.273\)
- \(-0.227\)
- \(-0.182\)
- \(-0.136\)
- \(-0.091\)
- \(-0.045\)
- \(0\)
- \(0.045\)
- \(0.091\)
- \(0.136\)
- \(0.182\)
- \(0.227\)
- \(0.273\)
- \(0.318\)
- \(\geq 0.363\)
1. STAAD SPACE

INPUT FILE: Panel Check - 5x6 type A (SM24).STD

2. START JOB INFORMATION

3. ENGINEER DATE 01-MAR-21

4. JOB CLIENT SQL

5. ENGINEER NAME CH

6. JOB REF PANEL CHECK

7. END JOB INFORMATION

8. INPUT WIDTH 79

9. UNIT FEET XIP

10. JOINT COORDINATES

11. 1 0 0 0; 2 1 0 0; 3 2 0 0; 4 3 0 0; 5 4 0 0; 6 5 0 0; 7 6 0 0; 8 0 0 0

12. 11 1 0 -1; 12 0 -1; 13 3 0 -1; 14 4 0 -1; 15 5 0 -1; 16 6 0 -1; 17 9 0 0 -2

13. 20 1 0 -2; 21 2 0 -2; 22 3 0 -2; 23 4 0 -2; 24 5 0 -2; 25 6 0 -2; 26 8 0 0 -3

14. 29 1 0 -3; 30 2 0 -3; 31 3 0 -3; 32 4 0 -3; 33 5 0 -3; 34 6 0 -3; 35 8 0 0 -4

15. 38 1 0 -4; 37 2 0 -4; 38 3 0 -4; 39 4 0 -4; 40 5 0 -4; 41 6 0 -4; 42 8 0 0 -5

16. 55 3 0 -5; 56 2 0 -5; 59 3 0 -5; 60 4 0 -5; 61 5 0 -5; 62 6 0 -5

17. ELEMENT TRUSS/ELEMENTS SHELL

18. 1 1 10 3 2 2 10 19 20 11; 3 19 28 29 20; 4 2 1 12 3 5 11 20 21 12

19. 6 20 29 30 21; 7 12 13 4; 8 12 21 22 13; 9 21 30 22; 10 4 13 14 5

20. 11 13 22 23 14; 12 22 31 23 23; 13 5 14 15 6; 14 14 23 24 15; 15 23 32 23 24

21. 16 26 15 16 7; 17 15 24 23 14; 18 24 33 34 20; 21 28 40 41 29 22 29 46 47 30

22. 23 30 47 48 31; 34 31 48 49 30; 35 32 49 50 33; 36 33 50 51 34; 41 45 56 57 46

23. 42 46 57 58 47; 43 47 58 59 48; 44 48 59 60 49; 45 49 60 61 50; 46 50 61 62 51

24. ELEMENT PROPERTY

25. 1 TO 31 TO 36 41 TO 46 THICKNESS 0.5

26. DEFINE MATERIAL START

27. ISOTROPIC CONCRETE

28. E 614504

29. POISSON 0.17

30. DENSITY 0.15

31. ALPH 12-005

32. DANS 0.05

33. END DEFINE MATERIAL

34. CONSTANTS

35. MATERIAL CONCRETE ALL

36. SUPPORTS

37. 1 TO 7 10 TO 16 19 TO 25 28 TO 34 45 TO 51 58 TO 61

38. 42 FIXED BUT KFX 89.76 KFY 89.76 KFX 89.76 KFY 89.76 noise 1 KMY 1 KMW 1

39. LOAD 1 LOADTYPE PUSH TITTLE WIRE STRENGTHS

40. ELEMENT LOAD

---------- PROBLEM STATISTICS ----------

NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS = 42/ 30/ 42

SOLVER USED IS THE GLO-OOP-CORK BASIC SOLVER

ORIGINAL/FINAL BAND-WIDTH = 8/ 8/ 54 DOP

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 252

SIZE OF STIFFNESS MATRIX = 14 DOUBLE KILO-WORDS

REQ/RND/AVAL. DISK SPACE = 12.3/ 263418.5 MB
Typical Panel Type (mesh 6W20), Panel Thickness 6'', 5'x6'

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>X</th>
<th>Y</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>-0.1667</td>
<td>0.25</td>
</tr>
<tr>
<td>P2</td>
<td>0.5</td>
<td>0.25</td>
</tr>
<tr>
<td>P3</td>
<td>0.1667</td>
<td>0.25</td>
</tr>
<tr>
<td>P4</td>
<td>-0.1667</td>
<td>0.25</td>
</tr>
<tr>
<td>P5</td>
<td>0.5</td>
<td>0.25</td>
</tr>
<tr>
<td>P6</td>
<td>0.1667</td>
<td>0.25</td>
</tr>
<tr>
<td>P7</td>
<td>-0.1667</td>
<td>-0.25</td>
</tr>
<tr>
<td>P8</td>
<td>0.5</td>
<td>-0.25</td>
</tr>
<tr>
<td>P9</td>
<td>0.1667</td>
<td>-0.25</td>
</tr>
<tr>
<td>P10</td>
<td>-0.1667</td>
<td>-0.25</td>
</tr>
<tr>
<td>P11</td>
<td>0.5</td>
<td>-0.25</td>
</tr>
<tr>
<td>P12</td>
<td>0.1667</td>
<td>-0.25</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[
A = 0.194 \text{ in}^2 \\
F_y = 75 \text{ ksi} \\
P = 0.65 \times A \times F_y = 9.458 \text{ Kips (per wire point)}
\]

ATTACHMENT BY NO. WIRES
(standard panels only)
MX (local) kNm/m

- <= 4.28
- 4.74
- 5.21
- 5.67
- 6.13
- 6.6
- 7.06
- 7.53
- 7.99
- 8.45
- 8.92
- 9.38
- 9.85
- 10.3
- 10.8
- 11.2
- >= 11.7

Load 1
MY (local)
kNm/m

4.88
5.92
6.97
8.02
9.06
10.1
11.2
12.2
13.2
14.3
15.3
16.4
17.4
18.5
19.5
>= 20.6

Load 1

5.00ft
6.00ft
SQY (local)
N/mm²

<table>
<thead>
<tr>
<th>&lt;= -0.304</th>
<th>-0.266</th>
<th>-0.228</th>
<th>-0.190</th>
<th>-0.152</th>
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<th>0.114</th>
<th>0.152</th>
<th>0.190</th>
<th>0.228</th>
<th>0.266</th>
<th>&gt;= 0.304</th>
</tr>
</thead>
</table>

5.00ft 6.00ft
### PROBLEM STATISTICS

**NUMBER OF JOINTS/MEMBERS/ELEMENTS/SUPPORTS** = 42/ 30/ 42

**SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER**

**ORIGINAL/SURFACE-BAND-WIDTH** = 8/ 8/ 54 DOF

**TOTAL PRIMARY LOAD CASES** = 1, **TOTAL DEGREES OF FREEDOM** = 252

**SIZE OF STIFFNESS MATRIX** = 14 DOUBLE KILOBYTES

**REQD/AVAIL. DISK SPACE** = 12.3/ 263416.8 MB

**FINISH**

---

<table>
<thead>
<tr>
<th>STAAD SPACE</th>
<th>PAGE NO. 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>41. 32 FR GR -9.458 0.25 -0.1667</td>
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<tr>
<td>42. 32 FR GR -9.458 0.25 0.5</td>
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<tr>
<td>43. 32 FR GR -9.458 0.25 0.1667</td>
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<tr>
<td>44. 34 FR GR -9.458 0.25 -0.1667</td>
<td></td>
</tr>
<tr>
<td>45. 34 FR GR -9.458 0.25 0.5</td>
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<td>46. 35 FR GR -9.458 0.25 0.1667</td>
<td></td>
</tr>
<tr>
<td>47. 5 FR GR -9.458 0.25 -0.1667</td>
<td></td>
</tr>
<tr>
<td>48. 5 FR GR -9.458 0.25 0.5</td>
<td></td>
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<tr>
<td>49. 8 FR GR -9.458 0.25 0.1667</td>
<td></td>
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<tr>
<td>50. 11 FR GR -9.458 0.25 -0.1667</td>
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<td>51. 11 FR GR -9.458 0.25 0.5</td>
<td></td>
</tr>
<tr>
<td>52. 14 FR GR -9.458 0.25 0.1667</td>
<td></td>
</tr>
</tbody>
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**LOAD 1 IDA_TYPE PUSH TITLE WIRE STRAIN**

**ELEMENT LOAD**
### Summary Result From Staad Pro Analysis Output

<table>
<thead>
<tr>
<th>Standard Panel Size</th>
<th>Barmat Type</th>
<th>Panel Type</th>
<th>Horz Rebar</th>
<th>Vert Rebar</th>
<th>Staad Pro Output</th>
<th>Data for AASHTO LRFD Design Check</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Mx (kNm/m)</td>
<td>My (kNm/m)</td>
</tr>
<tr>
<td>5'x6'</td>
<td>3W11</td>
<td>A</td>
<td>4 #4</td>
<td>4 #4</td>
<td>3.180</td>
<td>7.080</td>
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<tr>
<td></td>
<td>4W11</td>
<td></td>
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<td></td>
<td>3.660</td>
<td>8.100</td>
</tr>
<tr>
<td></td>
<td>5W11</td>
<td></td>
<td></td>
<td></td>
<td>4.520</td>
<td>8.540</td>
</tr>
<tr>
<td></td>
<td>4W11</td>
<td></td>
<td></td>
<td></td>
<td>5.200</td>
<td>8.450</td>
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<td></td>
<td>5W11</td>
<td></td>
<td></td>
<td></td>
<td>6.770</td>
<td>10.100</td>
</tr>
<tr>
<td></td>
<td>6W11</td>
<td></td>
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<td>8.800</td>
<td>12.000</td>
</tr>
<tr>
<td></td>
<td>5W24</td>
<td>A</td>
<td>7 #4</td>
<td>7 #4</td>
<td>11.000</td>
<td>20.800</td>
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<td>11.700</td>
<td>20.600</td>
</tr>
</tbody>
</table>

*Maximum bending moment and shear for each standard panel size shown in **red bold** font*
5’ x 6’ STANDARD
A PANEL
Typical Panel Type A, Panel Thickness 6", 5'x6', Horizontal Rebar

Panel Strength Design
Design of panel reinforcement per
AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi.
This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

Panel Thickness \( t_{\text{panel}} = 6 \) in
Finishing Thickness \( = 1 \) in
Additional Load from Finishes \( = 0.0125 \) Kip/ft\(^2\)
Concrete Cover \( C_{\text{panel}} = 1.5 \) in

Vertical Bar Size #4
Rebar Diameter \( d_b = 0.5 \) in \( d_{bh} = 0 \) in
Rebar Area \( A_b = 0.196 \) in\(^2\) \( A_b = \frac{1}{4} \pi d_b^2 \)
Rebar spacing \( s = 15 \) in
Width of the design section \( b = 12 \) in
Effective depth of section \( d_s = 4.25 \) in \( d_s = t_{\text{panel}} - C_{\text{panel}} - d_{bh} - \frac{1}{2} d_b \)
Effective depth of section for Negative Moment \( d_{s(M^-)} = 1.75 \) in \( d_{s(M^-)} = t_{\text{panel}} - d_s \)
Correction factor for source aggregate \( k_1 = 1 \) in AASHTO 5.4.2.4
Concrete Density \( W_c = 0.15 \) kcf
Concrete Strength \( f'c = 4.00 \) ksi (Concrete Class A compressive strength)
Reinforcement Strength \( f_y = 60 \) ksi (Minimum yield strength of grade 60 steel)
Module Elasticity of concrete \( E_c = 4266 \) ksi
Module Elasticity of reinforcement \( E_s = 29000 \) ksi
Modular Ratio \( n = 6.798 \) \( n = E_s/E_c \)
Area of Steel per Design Strip \( A_s = 0.157 \) in\(^2\) \( A_s = b(A_b/A_s) \)

\[ +M_u = 1.92 \text{ kip-ft} \]
\[ -M_u = 0.00 \text{ kip-ft} \]

Resistance factor for tension-controlled section \( \psi_{\text{STR}} = 0.90 \) AASHTO 5.5.4.2

Positive Moment Capacity
Depth of equivalent stress block \[ a = \frac{A_s f_y}{0.85 f'c b} = 0.231 \text{ in} \]
Factored flexural resistance \[ +\psi M_n = \psi A_s f_y (d_s - \frac{a}{2}) = 2.923 \text{ kip-ft} \]
Check \[ +\psi M_n > +M_u \]
\[ 2.923 > 1.92 \text{ OK} \]

Negative Moment Capacity
Depth of equivalent stress block \[ a = \frac{A_s f_y}{0.85 f'c b} = 0.231 \text{ in} \]
Factored flexural resistance \[ -\psi M_n = \psi A_s f_y (d_{s(M^-)} - \frac{a}{2}) = 1.155 \text{ kip-ft} \]
Check \[ -\psi M_n > -M_u \]
\[ 1.155 > 0.00 \text{ OK} \]
Minimum Reinforcement  AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate
to develop a factored flexural resistance, $Mr = \varphi Mn$, at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

$$M_{cr} = \gamma_3 \left[ (\gamma_1 f_r + \gamma_2 y_{cp}) S_c - M_{dn} \left( \frac{S_c}{S_{nc}} - 1 \right) \right]$$  
AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section,
this equation becomes the following:

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c$$

where:

- flexural cracking variability factor $\gamma_1 = 1.60$ (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength $\gamma_3 = 0.67$ (A615 steel)
- concrete density modification factor $\lambda = 1.00$ AASHTO 5.4.2.8

modulus of rapture $f_r = 0.24 \lambda \sqrt{f_c}$

section modulus of design section $S_c = \frac{b h^2}{6} = \frac{bt_{panel}^2}{6}$

Cracking moment $M_{cr} = \gamma_3 \gamma_1 f_r S_c$

check positive moment reinforcement:

- 1.33 x factored ultimate moment $= 2.55$ kip-ft
- Cracking moment $= 3.09$ kip-ft
- min from $(1.33 \times$ factored $+ M_u \text{ and } M_{cr}) = 2.55$ kip-ft

Check: $+\varphi M_n > \text{min}(+M_u, M_{cr})$

2.923 > 2.55  OK

check negative moment reinforcement:

- 1.33 x factored ultimate moment $= 0.00$ kip-ft
- Cracking moment $= 3.09$ kip-ft
- min from $(1.33 \times$ factored $- M_u \text{ and } M_{cr}) = 0.00$ kip-ft

Check: $-\varphi M_n > \text{min}(-M_u, M_{cr})$

1.155 > 0.00  OK
**Typical Panel Type A, Panel Thickness 6'', 5'x6', Horizontal Rebar**

**Shear on Panel**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness</td>
<td>( t_{\text{panel}} = 6 \text{ in} )</td>
</tr>
<tr>
<td>Concrete Cover</td>
<td>( C_{\text{panel}} = 1.5 \text{ in} )</td>
</tr>
<tr>
<td>Concrete Strength</td>
<td>( f_c' = 4.0 \text{ ksi} ) (Concrete Class A compressive strength)</td>
</tr>
<tr>
<td>Reinforcement Strength</td>
<td>( f_y = 60 \text{ ksi} ) (Minimum yield strength of grade 60 steel)</td>
</tr>
<tr>
<td>Bar Size</td>
<td>#4</td>
</tr>
<tr>
<td>Rebar Diameter</td>
<td>( d_b = 0.500 \text{ in} )</td>
</tr>
</tbody>
</table>
| Rebar Area                 | \( A_b = 0.196 \text{ in}^2 \)  
  \( A_b = \frac{1}{4} \pi d_b^2 \) |
| Rebar spacing              | \( s = 15 \text{ in} \) |
| Width of the design section| \( b = 12 \text{ in} \) |
| Area of Steel per Design Strip | \( A_s = 0.157 \text{ in}^2 \)  
  \( A_s = b \left( \frac{A_b}{s} \right) \) |

\[
c = \frac{A_s \cdot f_s}{\alpha_1 \cdot f_c' \cdot \beta_1 \cdot b}
\]

\[
d_v = t_{\text{panel}} - \frac{c}{2} - C_{\text{panel}} - \frac{d_b}{2}
\]

**AASHTO 5.7.3.3**

\( V_n \) shall be the lesser of:

1. \( V_n = V_c + V_s \)
   
   \[
   V_c = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f_c' \cdot b_v \cdot d_v}
   \]
   \[
   V_s = A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha
   \]
   \[
   V_n = 6.240 \text{ Kips}
   \]
   \[
   V_s = 3.231 \text{ Kips}
   \]
   \[
   (1) \ V_n = 9.472 \text{ Kips}
   \]
   \[
   (2) \ V_n = 0.25 \cdot f_c' \cdot b_v \cdot d_v
   \]
   \[
   (2) \ V_n = 49.369 \text{ Kips}
   \]

From (1) and (2) \( V_n \) should be:

\[
\phi \cdot V_n = 9.472 \text{ Kips}
\]

\[
\phi \cdot V_n = 0.9 \cdot V_n
\]

\[
\phi \cdot V_n = 8.524 \text{ Kips}
\]

**Shear Force:**

From Staad Pro Result \( V_{\text{max}} = 21.61 \text{ lbs/in}^2 \)

\[
\text{Shear Section Area} = b \times t_{\text{panel}}
\]

\[
= 12 \times 6
\]

\[
V_{\text{max}} = 1555.97 \text{ lbs}
\]

\[
= 1.556 \text{ Kips} \quad \text{OK, Vmax less than Vn}
\]
Typical Panel Type A, Panel Thickness 6", 5'x6', Vertical Rebar

Panel Strength Design
Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design is to present the information about the acceptable panel reinforcement sizes and spacing.

Panel Thickness \( t_{\text{panel}} = 6 \text{ in} \)
Finishing Thickness = 1 in
Additional Load from Finishes = 0.0125 Kip/ft²
Concrete Cover \( c_{\text{panel}} = 1.5 \text{ in} \)
Vertical Bar Size \#4
Rebar Diameter \( d_b = 0.5 \text{ in} \) \( d_{bh} = 0.5 \text{ in} \)
Rebar Area \( A_b = 0.196 \text{ in}² \) \( A_b = \frac{1}{4} \pi d_b^2 \)
Rebar spacing \( s = 18 \text{ in} \)
Width of the design section \( b = 12 \text{ in} \)
Effective depth of section \( d_s = 3.75 \text{ in} \)
Effective depth of section for Negative Moment \( d_s(M^-) = 2.25 \text{ in} \)
Correction factor for source aggregate \( k_1 = 1 \text{ in} \)
Concrete Density \( W_c = 0.15 \text{ kcf} \)
Concrete Strength \( f'c = 4.00 \text{ ksi} \) (Concrete Class A compressive strength)
Reinforcement Strength \( f_y = 60 \text{ ksi} \) (Minimum yield strength of grade 60 steel)
Modulus Elasticity of concrete \( E_c = 4266 \text{ ksi} \)
Modulus Elasticity of reinforcement \( E_s = 29000 \text{ ksi} \)
Modular Ratio \( n = 6.798 \) \( n = \frac{E_s}{E_c} \)
Area of Steel per Design Strip \( A_s = 0.131 \text{ in}² \)

\[
A_s = \frac{A_{b}}{\phi} = 0.80 \text{ kip-ft} \quad (\text{FK} = 1.35) \\
A_s = \frac{A_{b}}{\phi} = 0.80 \text{ kip-ft} \quad (\text{FK} = 1.35)
\]

Resistance factor for tension-controlled section \( \phi_{\text{STR}} = 0.90 \) AASHTO 5.5.4.2

Positive Moment Capacity
Depth of equivalent stress block \( a = \frac{A_{b}f_y}{0.85f'c_b} = 0.192 \text{ in} \)
Factored flexural resistance \( +\phi M_n = \phi A_s f_y \left( d_s - \frac{a}{2} \right) = 2.152 \text{ kip-ft} \)
Check \(+\phi M_n > +M_u\) 2.152 > 1.08 OK

Negative Moment Capacity
Depth of equivalent stress block \( a = \frac{A_{b}f_y}{0.85f'c_b} = 0.192 \text{ in} \)
Factored flexural resistance \( -\phi M_n = \phi A_s f_y \left( d_s(M^-) - \frac{a}{2} \right) = 1.269 \text{ kip-ft} \)
Check \(-\phi M_n > -M_u\) 1.269 > 0.00 OK
Minimum Reinforcement

AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( Mr = \varphi Mn \), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

\[
M_{cr} = \gamma_3 \left( (\gamma_1 f_r + \gamma_2 y_{cpe}) S_c - M_{dyn} \left( \frac{S_c}{S_{nc}} - 1 \right) \right)
\]  

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

where:

- flexural cracking variability factor \( \gamma_1 = 1.60 \) (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength \( \gamma_3 = 0.67 \) (A615 steel)
- concrete density modification factor \( \lambda = 1.00 \) AASHTO 5.4.2.8

modulus of rapture

\[
f_r = 0.24\lambda\sqrt{f_c'}
\]

\( f_r = 0.48 \) ksi AASHTO 5.4.2.6

section modulus of design section

\[
S_c = \frac{b h^2}{6} = \frac{b t_{panel}^2}{6}
\]

\( S_c = 72.00 \) in\(^3\)

Cracking moment

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

\( M_{cr} = 3.09 \) kip-ft

check positive moment reinforcement:

1.33 x factored ultimate moment

Cracking moment

\( = 1.44 \) kip-ft

min from (1.33 x factored +M_u and M_{cr})

\( = 1.44 \) kip-ft

Check:

\[
+\varphi M_n > \min(+M_u, M_{cr})
\]

\( 2.152 > 1.44 \) OK

check negative moment reinforcement:

1.33 x factored ultimate moment

Cracking moment

\( = 0.00 \) kip-ft

min from (1.33 x factored -M_u and M_{cr})

\( = 0.00 \) kip-ft

Check:

\[
-\varphi M_n > \min(-M_u, M_{cr})
\]

\( 1.269 > 0.00 \) OK
**Typical Panel Type A, Panel Thickness 6", 5'x6', Vertical Rebar**

**Shear on Panel**

- **Panel Thickness** $t_{\text{panel}} = 6$ in
- **Concrete Cover** $C_{\text{panel}} = 2.5$ in
- **Concrete Strength** $f'_{c} = 4.0$ ksi (Concrete Class A compressive strength)
- **Reinforcement Strength** $f_{y} = 60$ ksi (Minimum yield strength of grade 60 steel)
- **Bar Size** #4
- **Rebar Diameter** $d_b = 0.500$ in
- **Rebar Area** $A_b = 0.196$ in$^2$
- **Rebar spacing** $s = 18$ in
- **Width of the design section** $b = 12$ in
- **Area of Steel per Design Strip** $A_s = 0.131$ in$^2$

$$d_v = t_{\text{panel}} - \frac{c}{2} - C_{\text{panel}} - \frac{d_b}{2}$$

$$c = \frac{A_s \cdot f_s}{\alpha_1 \cdot f'_{c} \cdot \beta_1 \cdot b}$$

$$\phi \cdot V_n = 0.9 \cdot V_n = 6.130 \text{ Kips}$$

**AASHTO 5.7.3.3**

$V_n$ shall be the lesser of:

1. $V_n = V_c + V_s$

$$V_c = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f'_{c}} \cdot b_v \cdot d_v$$

$$V_c = 4.758 \text{ Kips}$$

$$V_s = \frac{A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha}{s}$$

$$V_s = 2.053 \text{ Kips}$$

2. $V_n = 0.25 \cdot f'_{c} \cdot b_v \cdot d_v$

$$V_n = 37.641 \text{ Kips}$$

From (1) and (2) $V_n$ should be:

$$V_n = 6.811 \text{ Kips}$$

**Shear Force:**

- From Staad Pro Result $V_{\text{max}} = \boxed{21.90 \text{ lbs/in}^2}$
- From Staad Pro Result $V_{\text{max}} = \boxed{0.151 \text{ N/mm}^2}$

$$V_{\text{max}} = 1576.85 \text{ lbs} \quad \text{OK, } V_{\text{max}} \text{ less than } V_n$$

$$V_{\text{max}} = 1.577 \text{ Kips}$$

$$\text{Shear Section Area} = b \times t_{\text{panel}}$$

- $b = 12$ in
- $t_{\text{panel}} = 6$ in

$$V_{\text{max}} = 1576.85 \text{ lbs}$$

$$V_{\text{max}} = 1.577 \text{ Kips}$$

$$\text{OK, } V_{\text{max}} \text{ less than } V_n$$
5’ x 6’ STANDARD X PANEL
Typical Panel Type X, Panel Thickness 6", 5'x6', Horizontal Rebar

Panel Strength Design
Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

Panel Thickness \( t_{\text{panel}} = 6 \) in
Finishing Thickness \( C_{\text{panel}} = 1.5 \) in
Additional Load from Finishes \( = 0.0125 \) Kip/ft²
Concrete Cover \( t_{\text{panel}} = 6 \) in
Vertical Bar Size #4 Logitudinal Bar Size #4
Rebar Diameter \( d_b = 0.5 \) in \( \text{dbh} = 0 \) in
Rebar Area \( A_b = 0.196 \) in² \( A_b = \frac{1}{4} \pi d_b^2 \)
Rebar spacing \( s = 10 \) in
Width of the design section \( b = 12 \) in
Effective depth of section \( d_s = 4.25 \) in \( d_s(M^-) = 1.75 \) in \( d_s = t_{\text{panel}} - C_{\text{panel}} - d_{\text{bh}} - \frac{1}{2} d_b \)
Effective depth of section for Negative Moment \( d_s(M^-) = t_{\text{panel}} - d_s \)
Correction factor for source aggregate \( K_1 = 1 \) in AASHTO 5.4.2.4
Concrete Density \( W_c = 0.15 \) kcf
Concrete Strength \( f'c = 4.00 \) ksi (Concrete Class A compressive strength)
Reinforcement Strength \( f_y = 60 \) ksi (Minimum yield strength of grade 60 steel)
Modulus Elasticity of concrete \( E_c = 4266 \) ksi \( E_c = 120000k_s W_c^2 f'c^{0.33} \)
Modulus Elasticity of reinforcement \( E_s = 29000 \) ksi
Modular Ratio \( n = 6.798 \) \( n = E_s/E_c \)
Area of Steel per Design Strip \( A_s = 0.236 \) in² \( A_s = b\left(\frac{A_b}{s}\right) \)
+\( M_u \) = 3.82 kip-ft \( +M_{u,\text{service}} = 2.83 \) kip-ft (FK = 1.35)
-\( M_u \) = 0.00 kip-ft \( -M_{u,\text{service}} = 0.00 \) kip-ft (FK = 1.35)
Resistance factor for tension-controlled section \( \Phi_{\text{STR}} = 0.90 \) AASHTO 5.5.4.2

Positive Moment Capacity
Depth of equivalent stress block \( a = \frac{A_s f_y}{0.85 f'c b} = 0.346 \) in
Factored flexural resistance \( +\varphi M_n = \varphi A_s f_y \left( d_s - \frac{a}{2} \right) = 4.323 \) kip-ft
Check \( +\varphi M_n > +M_u \) 4.323 > 3.82 OK

Negative Moment Capacity
Depth of equivalent stress block \( a = \frac{A_s f_y}{0.85 f'c b} = 0.346 \) in
Factored flexural resistance \( -\varphi M_n = \varphi A_s f_y \left( d_s(M^-) - \frac{a}{2} \right) = 1.672 \) kip-ft
Check \( -\varphi M_n > -M_u \) 1.672 > 0.00 OK
**Minimum Reinforcement**

AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r = \varphi M_n \), at least equal to the lesser of:

- 1.33 times the positive factored ultimate moment

**Cracking moment**

\[
M_{cr} = \gamma_3 \left[ (\gamma_1 f_r + \gamma_2 \gamma_{cpe}) S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \right]
\]

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

where:

- flexural cracking variability factor \( \gamma_1 = 1.60 \) (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength \( \gamma_3 = 0.67 \) (A615 steel)
- concrete density modification factor \( \lambda = 1.00 \) AASHTO 5.4.2.8

- modulus of rapture \( f_r = 0.24 \lambda \sqrt{f_c^t} \)
  \( f_r = 0.48 \text{ ksi} \) AASHTO 5.4.2.6

- section modulus of design section \( S_c = \frac{bh^2}{6} = \frac{b t_{panel}^2}{6} \)
  \( S_c = 72.00 \text{ in}^3 \)

**Cracking moment**

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

\( M_{cr} = 3.09 \text{ kip-ft} \)

check positive moment reinforcement:

- 1.33 x factored ultimate moment
  \( = 5.08 \text{ kip-ft} \)
- Cracking moment
  \( = 3.09 \text{ kip-ft} \)
- min from \((1.33 \times \text{factored} + M_u \text{ and } M_{cr})\)
  \( = 3.09 \text{ kip-ft} \)

Check: \( +\varphi M_n > \min(+M_u, M_{cr}) \)

\[
4.323 > 3.09 \quad \text{OK}
\]

check negative moment reinforcement:

- 1.33 x factored ultimate moment
  \( = 0.00 \text{ kip-ft} \)
- Cracking moment
  \( = 3.09 \text{ kip-ft} \)
- min from \((1.33 \times \text{factored} - M_u \text{ and } M_{cr})\)
  \( = 0.00 \text{ kip-ft} \)

Check: \( -\varphi M_n > \min(-M_u, M_{cr}) \)

\[
1.672 > 0.00 \quad \text{OK}
\]
**Typical Panel Type X, Panel Thickness 6", 5'x6', Horizontal Rebar**

**Shear on Panel**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness ( t_{\text{panel}} )</td>
<td>6 in</td>
</tr>
<tr>
<td>Concrete Cover ( C_{\text{panel}} )</td>
<td>1.5 in</td>
</tr>
<tr>
<td>Concrete Strength ( f_c )</td>
<td>4.0 ksi</td>
</tr>
<tr>
<td>Reinforcement Strength ( f_y )</td>
<td>60 ksi</td>
</tr>
<tr>
<td>Bar Size #4</td>
<td></td>
</tr>
<tr>
<td>Rebar Diameter ( d_b )</td>
<td>0.500 in</td>
</tr>
<tr>
<td>Rebar Area ( A_b )</td>
<td>0.196 in²</td>
</tr>
<tr>
<td>Rebar spacing ( s )</td>
<td>10 in</td>
</tr>
<tr>
<td>Width of the design section ( b )</td>
<td>12 in</td>
</tr>
<tr>
<td>Area of Steel per Design Strip ( A_s )</td>
<td>0.236 in²</td>
</tr>
</tbody>
</table>

\[
c = \frac{A_s \cdot f_s}{\alpha_1 \cdot f_c \cdot \beta_1 \cdot b}
\]

\[
d_v = t_{\text{panel}} - \frac{c}{2} - C_{\text{panel}} - \frac{d_b}{2}
\]

\[
c = 0.408 \text{ in}
\]

\[
d_v = 4.046 \text{ in}
\]

**AASHTO 5.7.3.3**

Vn shall be the lesser of:

1. \( V_n = V_c + V_s \)

\[
V_c = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f_c' \cdot b_v \cdot d_v}
\]

\[
V_s = \frac{A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha}{s}
\]

<table>
<thead>
<tr>
<th>Calculation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( V_c )</td>
<td>6.137 Kips</td>
</tr>
<tr>
<td>( V_s )</td>
<td>4.767 Kips</td>
</tr>
</tbody>
</table>

(1) \( V_n = 10.904 \) Kips

(2) \( V_n = 0.25 \cdot f_c' \cdot b_v \cdot d_v \)

(2) \( V_n = 48.554 \) Kips

From (1) and (2) Vn should be:

\( \phi \cdot V_n = 0.9 \cdot V_n = 9.814 \) Kips

**Shear Force:**

\[
V_{\text{max}} = 0.296 \text{ N/mm}^2
\]

\[
V_{\text{max}} = 42.93 \text{ lbs/in}^2
\]

\[
\text{Shear Section Area} = b \times t_{\text{panel}} = 12 \times 6
\]

\[
V_{\text{max}} = 3091.05 \text{ lbs} = 3.091 \text{ Kips}
\]

OK, \( V_{\text{max}} \) less than Vn.
## Typical Panel Type X, Panel Thickness 6", 5'x6', Vertical Rebar

### Panel Strength Design
Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Design Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness t_{panel}</td>
<td>6 in</td>
</tr>
<tr>
<td>Finishing Thickness</td>
<td>1 in</td>
</tr>
<tr>
<td>Additional Load from Finishes</td>
<td>0.0125 kip/ft²</td>
</tr>
<tr>
<td>Concrete Cover C_{panel}</td>
<td>1.5 in</td>
</tr>
<tr>
<td>Vertical Bar Size #4</td>
<td></td>
</tr>
<tr>
<td>Rebar Diameter d_b</td>
<td>0.5 in</td>
</tr>
<tr>
<td>Rebar Area A_b</td>
<td>0.196 in²</td>
</tr>
<tr>
<td>Rebar spacing s</td>
<td>12 in</td>
</tr>
<tr>
<td>Width of the design section b</td>
<td>12 in</td>
</tr>
<tr>
<td>Effective depth of section d_s</td>
<td>3.75 in</td>
</tr>
<tr>
<td>Effective depth of section for Negative Moment d_{s(M^-)}</td>
<td>2.25 in</td>
</tr>
<tr>
<td>Correction factor for source aggregate k_1</td>
<td>1 in</td>
</tr>
<tr>
<td>Concrete Density W_c</td>
<td>0.15 kcf</td>
</tr>
<tr>
<td>Concrete Strength f'_c</td>
<td>4.00 ksi</td>
</tr>
<tr>
<td>Reinforcement Strength f_y</td>
<td>60 ksi</td>
</tr>
<tr>
<td>Modulus Elasticity of concrete E_c</td>
<td>4266 ksi</td>
</tr>
<tr>
<td>Modulus Elasticity of reinforcement E_s</td>
<td>29000 ksi</td>
</tr>
<tr>
<td>Modular Ratio n</td>
<td>6.798</td>
</tr>
<tr>
<td>Area of Steel per Design Strip A_s</td>
<td>0.196 in²</td>
</tr>
<tr>
<td>Resistance factor for tension-controlled section ( \varphi_{STR} )</td>
<td>0.90</td>
</tr>
</tbody>
</table>

### Positive Moment Capacity

\[
\begin{align*}
\Delta = & \frac{A_s f_y}{0.85 f'_c b} = 0.289 \text{ in} \\
\text{Factored flexural resistance} +\varphi M_n &= \varphi A_s f_y \left( d_s - \frac{a}{2} \right) = 3.186 \text{ kip-ft} \\
\text{Check} +\varphi M_n &> +M_u \\
3.186 &> 2.15 \text{ OK}
\end{align*}
\]

### Negative Moment Capacity

\[
\begin{align*}
\Delta = & \frac{A_s f_y}{0.85 f'_c b} = 0.289 \text{ in} \\
\text{Factored flexural resistance} -\varphi M_n &= \varphi A_s f_y \left( d_{s(M^-)} - \frac{a}{2} \right) = 1.860 \text{ kip-ft} \\
\text{Check} -\varphi M_n &> -M_u \\
1.860 &> 0.00 \text{ OK}
\end{align*}
\]
**Minimum Reinforcement**

AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r = \varphi M_n \), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

\[
M_{cr} = \gamma_3 \left[ (\gamma_1 f_r + \gamma_2 \gamma_{c,pe})S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \right]
\]

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

where:

- flexural cracking variability factor
  \( \gamma_1 = 1.60 \) (non-segmental brg)

- ratio of specified min. yield strength to ultimate tensile strength
  \( \gamma_3 = 0.67 \) (A615 steel)

- concrete density modification factor
  \( \lambda = 1.00 \) AASHTO 5.4.2.8

- modulus of rupture
  \[ f_r = 0.24 \lambda \sqrt{f_{ct}} \]
  \( f_r = 0.48 \text{ ksi} \) AASHTO 5.4.2.6

- section modulus of design section
  \[ S_c = \frac{b h^2}{6} = \frac{b t_{panel}^2}{6} \]
  \( S_c = 72.00 \text{ in}^3 \)

Cracking moment

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

\( M_{cr} = 3.09 \text{ kip-ft} \)

check positive moment reinforcement:

1.33 x factored ultimate moment

\( = 2.86 \text{ kip-ft} \)

Cracking moment

\( = 3.09 \text{ kip-ft} \)

min from (1.33 x factored +M\(_u\) and M\(_{cr}\))

\( = 2.86 \text{ kip-ft} \)

Check:

\( +\varphi M_n > \min(+M_u, M_{cr}) \)

\( 3.186 > 2.86 \text{ OK} \)

check negative moment reinforcement:

1.33 x factored ultimate moment

\( = 0.00 \text{ kip-ft} \)

Cracking moment

\( = 3.09 \text{ kip-ft} \)

min from (1.33 x factored -M\(_u\) and M\(_{cr}\))

\( = 0.00 \text{ kip-ft} \)

Check:

\( -\varphi M_n > \min(-M_u, M_{cr}) \)

\( 1.860 > 0.00 \text{ OK} \)
Typical Panel Type X, Panel Thickness 6", 5'x6', Vertical Rebar

**Shear on Panel**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness t&lt;sub&gt;panel&lt;/sub&gt;</td>
<td>6 in</td>
</tr>
<tr>
<td>Concrete Cover C&lt;sub&gt;panel&lt;/sub&gt;</td>
<td>2.5 in</td>
</tr>
<tr>
<td>Concrete Strength f&lt;sub&gt;c&lt;/sub&gt;</td>
<td>4.0 ksi (Concrete Class A compressive strength)</td>
</tr>
<tr>
<td>Reinforcement Strength f&lt;sub&gt;y&lt;/sub&gt;</td>
<td>60 ksi (Minimum yield strength of grade 60 steel)</td>
</tr>
<tr>
<td>Bar Size</td>
<td>#4</td>
</tr>
<tr>
<td>Rebar Diameter d&lt;sub&gt;b&lt;/sub&gt;</td>
<td>0.500 in</td>
</tr>
<tr>
<td>Rebar Area A&lt;sub&gt;b&lt;/sub&gt;</td>
<td>0.196 in&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>Rebar spacing s</td>
<td>12 in</td>
</tr>
<tr>
<td>Width of the design section b</td>
<td>12 in</td>
</tr>
<tr>
<td>Area of Steel per Design Strip A&lt;sub&gt;s&lt;/sub&gt;</td>
<td>0.196 in&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

\[
 c = \frac{A_b \cdot f_y}{\alpha_1 \cdot f_c \cdot \beta_1 \cdot b}
\]
\[
d_v = t_{panel} - \frac{c}{2} - C_{panel} - \frac{d_b}{2}
\]

\[
 A_{ASHTO} = A = \frac{1}{\pi d_b^2}
\]

**AASHTO 5.7.3.3**

\[
 V_n = \min(V_c + V_s, 2V_n)
\]

\[
 V_c = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f_c' \cdot b_v \cdot d_v}
\]
\[
 V_s = \frac{A_b \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha}{s}
\]

\[
 (1) \ V_n = 7.696 \ \text{Kips}
\]

\[
 (2) \ V_n = 36.962 \ \text{Kips}
\]

From (1) and (2) \( V_n \) should be:

\[
 \phi \cdot V_n = 0.9 \cdot V_n = 6.926 \ \text{Kips}
\]

**Shear Force:**

\[
 \text{From Staad Pro Result Vmax} = 0.301 \ \text{N/mm}^2 = 43.66 \ \text{lbs/in}^2
\]

Shear Section Area = \( b \times t_{panel} \)

\[
 b = 12 \times 6
\]

\[
 V_{max} = 3143.26 \ \text{lbs}, \ 3.143 \ \text{Kips} \quad \text{OK, Vmax less than Vn}
\]
5’ x 6’ STANDARD Y PANEL
**Typical Panel Type Y, Panel Thickness 6", 5'x6', Horizontal Rebar**

### Panel Strength Design
Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness</td>
<td>6 in</td>
</tr>
<tr>
<td>Finishing Thickness</td>
<td>1 in</td>
</tr>
<tr>
<td>Additional Load from Finishes</td>
<td>0.0125 kip/ft²</td>
</tr>
<tr>
<td>Concrete Cover</td>
<td>1.5 in</td>
</tr>
<tr>
<td>Vertical Bar Size</td>
<td>#4</td>
</tr>
<tr>
<td>Rebar Diameter</td>
<td>0.5 in</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>0.196 in²</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>0.196 in²</td>
</tr>
<tr>
<td>Rebar spacing</td>
<td>8.57 in</td>
</tr>
<tr>
<td>Width of the design section</td>
<td>12 in</td>
</tr>
<tr>
<td>Effective depth of section</td>
<td>4.25 in</td>
</tr>
<tr>
<td>Effective depth of section for Negative Moment</td>
<td>1.75 in</td>
</tr>
<tr>
<td>Correction factor for source aggregate</td>
<td>1 in</td>
</tr>
<tr>
<td>Concrete Density</td>
<td>0.15 kcf</td>
</tr>
<tr>
<td>Concrete Strength</td>
<td>4.00 ksi</td>
</tr>
<tr>
<td>Reinforcement Strength</td>
<td>60 ksi</td>
</tr>
<tr>
<td>Modulus Elasticity of concrete</td>
<td>4266 ksi</td>
</tr>
<tr>
<td>Modulus Elasticity of reinforcement</td>
<td>29000 ksi</td>
</tr>
<tr>
<td>Modular Ratio</td>
<td>6.798</td>
</tr>
<tr>
<td>Area of Steel per Design Strip</td>
<td>0.275 in²</td>
</tr>
<tr>
<td>+M_u</td>
<td>4.68 kip-ft</td>
</tr>
<tr>
<td>-M_u</td>
<td>0.00 kip-ft</td>
</tr>
<tr>
<td>+M_u_service</td>
<td>3.46 kip-ft</td>
</tr>
<tr>
<td>-M_u_service</td>
<td>0.00 kip-ft</td>
</tr>
<tr>
<td>Resistance factor for tension-controlled section</td>
<td>0.90</td>
</tr>
</tbody>
</table>

#### Positive Moment Capacity
Depth of equivalent stress block

\[ a = \frac{A_s f_y}{0.85 f'_c b} = 0.404 \text{ in} \]

Factored flexural resistance

\[ +\varphi M_u = \varphi A_s f_y \left(d_s - \frac{a}{2}\right) = 5.008 \text{ kip-ft} \]

Check \( +\varphi M_u > +M_u \)

\[ 5.008 > 4.68 \quad \text{OK} \]

#### Negative Moment Capacity
Depth of equivalent stress block

\[ a = \frac{A_s f_y}{0.85 f'_c b} = 0.404 \text{ in} \]

Factored flexural resistance

\[ -\varphi M_n = -\varphi A_s f_y \left(d_s(M^-) - \frac{a}{2}\right) = 1.915 \text{ kip-ft} \]

Check \(-\varphi M_n > -M_u \)

\[ 1.915 > 0.00 \quad \text{OK} \]
Minimum Reinforcement (AASHTO 5.7.3.3.2)

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r = \varphi M_n \), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

\[
M_{cr} = \gamma_3 \left( \gamma_1 f_r + \gamma_2 f_{te} \right) S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right)
\]

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

where:

- flexural cracking variability factor \( \gamma_1 = 1.60 \) (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength \( \gamma_3 = 0.67 \) (A615 steel)
- concrete density modification factor \( \lambda = 1.00 \) AASHTO 5.4.2.8

- modulus of rupture \( f_r = 0.24 \lambda \sqrt{f_c} \) \( f_r = 0.48 \) ksi AASHTO 5.4.2.6

- section modulus of design section \( S_c = \frac{bh^2}{6} = \frac{b t_{panel}^2}{6} \) \( S_c = 72.00 \) in\(^3\)

Cracking moment \( M_{cr} = \gamma_3 \gamma_1 f_r S_c \) \( M_{cr} = 3.09 \) kip-ft

check positive moment reinforcement:

1.33 x factored ultimate moment \( = 6.22 \) kip-ft
Cracking moment \( = 3.09 \) kip-ft
min from (1.33 x factored +\( M_u \) and \( M_{cr} \)) \( = 3.09 \) kip-ft

Check: \( +\varphi M_n > \min (+M_u, M_{cr}) \) 5.008 > 3.09 OK

check negative moment reinforcement:

1.33 x factored ultimate moment \( = 0.00 \) kip-ft
Cracking moment \( = 3.09 \) kip-ft
min from (1.33 x factored -\( M_u \) and \( M_{cr} \)) \( = 0.00 \) kip-ft

Check: \( -\varphi M_n > \min (-M_u, M_{cr}) \) 1.915 > 0.00 OK
### Typical Panel Type Y, Panel Thickness 6", 5'x6', Horizontal Rebar

#### Shear on Panel

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness</td>
<td>( t_{\text{panel}} = 6 ) in</td>
</tr>
<tr>
<td>Concrete Cover</td>
<td>( c_{\text{panel}} = 1.5 ) in</td>
</tr>
<tr>
<td>Concrete Strength ( f'_c )</td>
<td>( 4.0 ) ksi (Concrete Class A compressive strength)</td>
</tr>
<tr>
<td>Reinforcement Strength ( f_y )</td>
<td>( 60 ) ksi (Minimum yield strength of grade 60 steel)</td>
</tr>
<tr>
<td>Bar Size #4</td>
<td>#4</td>
</tr>
<tr>
<td>Rebar Diameter</td>
<td>( d_b = 0.500 ) in</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>( A_b = 0.196 ) in²</td>
</tr>
<tr>
<td>Rebar spacing</td>
<td>( s = 9 ) in</td>
</tr>
<tr>
<td>Width of the design section</td>
<td>( b = 12 ) in</td>
</tr>
<tr>
<td>Area of Steel per Design Strip</td>
<td>( A_s = 0.275 ) in²</td>
</tr>
</tbody>
</table>

\[
c = \frac{A_s \cdot f_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b}
\]

\[
d_v = t_{\text{panel}} - \frac{c}{2} - c_{\text{panel}} - \frac{d_b}{2}
\]

\[
c = 0.476 \text{ in}
\]

\[
d_v = 4.012 \text{ in}
\]

#### AASHTO 5.7.3.3

\( V_n \) shall be the lesser of:

1. \( V_n = V_c + V_s \)

\[
V_c = 0.0316 \cdot \beta \cdot \alpha \cdot \frac{f'_c \cdot b_v \cdot d_v}{\sqrt{f'_c \cdot b_v \cdot d_v}}
\]

\[
V_s = \frac{A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha}{s}
\]

\[
(1) \quad V_n = 11.601 \text{ Kips}
\]

\[
(2) \quad V_n = 48.146 \text{ Kips}
\]

From (1) and (2) \( V_n \) should be:

\[
\phi \cdot V_n = 0.9 \cdot V_n = 10.441 \text{ Kips}
\]

\[
\text{Shear Force:}
\]

From Staad Pro Result

\[
V_{\text{max}} = 3790.71 \text{ lbs}
\]

\[
V_{\text{max}} = 3.791 \text{ Kips} \quad \text{OK, Vmax less than Vn}
\]
**Typical Panel Type Y, Panel Thickness 6"", 5'x6', Vertical Rebar**

**Panel Strength Design**
Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness</td>
<td>6 in</td>
</tr>
<tr>
<td>Finishing Thickness</td>
<td>1 in</td>
</tr>
<tr>
<td>Additional Load from Finishes</td>
<td>0.0125 Kip/ft²</td>
</tr>
<tr>
<td>Concrete Cover</td>
<td>1.5 in</td>
</tr>
<tr>
<td>Vertical Bar Size</td>
<td>#4</td>
</tr>
<tr>
<td>Rebar Diameter</td>
<td>0.5 in</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>0.196 in²</td>
</tr>
<tr>
<td>Rebar spacing</td>
<td>10.28 in</td>
</tr>
<tr>
<td>Width of the design section</td>
<td>12 in</td>
</tr>
<tr>
<td>Effective depth of section</td>
<td>3.75 in</td>
</tr>
<tr>
<td>Effective depth of section for Negative Moment</td>
<td>2.25 in</td>
</tr>
<tr>
<td>Correction factor for source aggregate</td>
<td>1 in</td>
</tr>
<tr>
<td>Concrete Density</td>
<td>0.15 kcf</td>
</tr>
<tr>
<td>Concrete Strength</td>
<td>4.00 ksi</td>
</tr>
<tr>
<td>Reinforcement Strength</td>
<td>60 ksi</td>
</tr>
<tr>
<td>Modulus Elasticity of concrete</td>
<td>4266 ksi</td>
</tr>
<tr>
<td>Modulus Elasticity of reinforcement</td>
<td>29000 ksi</td>
</tr>
<tr>
<td>Modular Ratio</td>
<td>6.798</td>
</tr>
<tr>
<td>Area of Steel per Design Strip</td>
<td>0.229 in²</td>
</tr>
<tr>
<td>+Mu</td>
<td>2.63 kip-ft</td>
</tr>
<tr>
<td>-Mu</td>
<td>0.00 kip-ft</td>
</tr>
<tr>
<td>+Mu_service</td>
<td>1.95 kip-ft</td>
</tr>
<tr>
<td>-Mu_service</td>
<td>0.00 kip-ft</td>
</tr>
<tr>
<td>Resistance factor for tension-controlled section</td>
<td>0.90</td>
</tr>
</tbody>
</table>

**Positive Moment Capacity**
Depth of equivalent stress block
\[ a = \frac{A_s f_y}{0.85 f'_c b} = 0.337 \text{ in} \]

Factored flexural resistance
\[ +\phi M_n = \phi A_s f_y \left( d_e - \frac{a}{2} \right) = 3.694 \text{ kip-ft} \]
Check
\[ +\phi M_n > +M_u \]
\[ 3.694 > 2.63 \text{ OK} \]

**Negative Moment Capacity**
Depth of equivalent stress block
\[ a = \frac{A_s f_y}{0.85 f'_c b} = 0.337 \text{ in} \]

Factored flexural resistance
\[ -\phi M_n = \phi A_s f_y \left( d_e(M^-) - \frac{a}{2} \right) = 2.147 \text{ kip-ft} \]
Check
\[ -\phi M_n > -M_u \]
\[ 2.147 > 0.00 \text{ OK} \]
Minimum Reinforcement

AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r = \varphi M_n \), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

\[
M_{cr} = \gamma_3 \left[ (\gamma_1 f_r + \gamma_2 C_p) S_c - M_{dn} \left( \frac{S_c}{S_{nc}} - 1 \right) \right]
\]

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

where:

- flexural cracking variability factor
  - \( \gamma_1 = 1.60 \) (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength
  - \( \gamma_3 = 0.67 \) (A615 steel)
- concrete density modification factor
  - \( \lambda = 1.00 \) AASHTO 5.4.2.8

- modulus of rupture
  - \( f_r = 0.24\lambda \sqrt{f_{ct}} \)
  - \( f_r = 0.48 \) ksi AASHTO 5.4.2.6

- section modulus of design section
  - \( S_c = \frac{bh^2}{6} = \frac{b t_{panel}^2}{6} \)
  - \( S_c = 72.00 \) in

Cracking moment

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c \quad M_{cr} = 3.09 \text{ kip-ft}
\]

check positive moment reinforcement:

1.33 x factored ultimate moment

\[
\text{Cracking moment} = 3.09 \text{ kip-ft}
\]

min from (1.33 x factored +\( M_u \) and \( M_{cr} \))

\[
\text{min} = 3.09 \text{ kip-ft}
\]

Check:

\[
+\varphi M_n > \text{min}(+M_u, M_{cr}) \quad 3.694 > 3.09 \quad \text{OK}
\]

check negative moment reinforcement:

1.33 x factored ultimate moment

\[
\text{Cracking moment} = 3.09 \text{ kip-ft}
\]

min from (1.33 x factored -\( M_u \) and \( M_{cr} \))

\[
\text{min} = 0.00 \text{ kip-ft}
\]

Check:

\[
-\varphi M_n > \text{min}(-M_u, M_{cr}) \quad 2.147 > 0.00 \quad \text{OK}
\]
**Typical Panel Type Y, Panel Thickness 6", 5'x6', Vertical Rebar**

**Shear on Panel**

Panel Thickness \( t_{\text{panel}} = 6 \text{ in} \)

Concrete Cover \( c_{\text{panel}} = 2.5 \text{ in} \)

Concrete Strength \( f'_{c} = 4.0 \text{ ksi} \) (Concrete Class A compressive strength)

Reinforcement Strength \( f_{y} = 60 \text{ ksi} \) (Minimum yield strength of grade 60 steel)

Bar Size \( \#4 \)

Rebar Diameter \( d_{b} = 0.500 \text{ in} \)

Rebar Area \( A_{b} = 0.196 \text{ in}^{2} \) \( A_{b} = \frac{1}{4} \pi d_{b}^{2} \)

Rebar spacing \( s = 10 \text{ in} \)

Width of the design section \( b = 12 \text{ in} \)

Area of Steel per Design Strip \( A_{s} = 0.229 \text{ in}^{2} \)

\[
c = \frac{A_{s} \cdot f_{s}}{\alpha \cdot f'_{c} \cdot \beta \cdot b}
\]

\[
d_{v} = t_{\text{panel}} - \frac{c}{2} - c_{\text{panel}} - \frac{d_{b}}{2}
\]

**AASHTO 5.7.3.3**

Vn shall be the lesser of:

1. \( V_{n} = V_{c} + V_{s} \)

\[
V_{c} = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f'_{c}} \cdot b_{v} \cdot d_{v}
\]

\[
V_{c} = 4.629 \text{ Kips}
\]

\[
V_{s} = \frac{A_{b} \cdot f_{y} \cdot d_{v} \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha}{s}
\]

\[
V_{s} = 3.497 \text{ Kips}
\]

\[
(1) \ V_{n} = 8.126 \text{ Kips}
\]

\[
(2) \ V_{n} = 0.25 \cdot f'_{c} \cdot b_{v} \cdot d_{v}
\]

\[
(2) \ V_{n} = 36.621 \text{ Kips}
\]

From (1) and (2) \( V_{n} \) should be:

\[
\phi \cdot V_{n} = 0.9 \cdot V_{n} = 7.314 \text{ Kips}
\]

**Shear Force:**

From Staad Pro Result \( V_{\text{max}} = 0.369 \text{ N/mm}^{2} \)

\( = 53.52 \text{ lbs/in}^{2} \)

Shear Section Area \( = b \times t_{\text{panel}} \)

\( = 12 \times 6 \)

\[
V_{\text{max}} = 3853.37 \text{ lbs} \]

\[
= 3.853 \text{ Kips} \text{ OK, } V_{\text{max}} \text{ less than } V_{n}
\]
5’ x 10’ STANDARD PANELS
STANDARD "X" PANEL
SHOWN FROM BACK FACE

SECTION A-A

NOTE:
TYPICAL PANEL REINFORCEMENT SHOWN. ALTERNATIVE REINFORCEMENT MAY BE USED AS LONG AS MINIMUM CROSS SECTIONAL AREAS ARE EQUVALENT. 18" WAX SPACING BETWEEN BARS.

CERTIFIED ONLY WITH RESPECT TO INTEGRIAL STABILITY OF REINFORCED GUTTUR STRUCTURES

TYPE 'X' PANEL DETAILS

MIX PLUS WALL SYSTEM

REVISIONS:
- D/24/22
- D/24/22
- A/24/22
- D/24/22
- D/24/22
<table>
<thead>
<tr>
<th>Wire</th>
<th>Area sqin</th>
<th>Diameter in</th>
<th>Diameter after microns</th>
<th>Diameter after microns</th>
<th>Area after % effective</th>
<th>Area after sqin</th>
</tr>
</thead>
<tbody>
<tr>
<td>W5</td>
<td>0.05</td>
<td>0.252</td>
<td>6408.757</td>
<td>4992.757</td>
<td>0.1966</td>
<td>0.6069</td>
</tr>
<tr>
<td>W8</td>
<td>0.08</td>
<td>0.319</td>
<td>8106.507</td>
<td>6690.507</td>
<td>0.2634</td>
<td>0.6812</td>
</tr>
<tr>
<td>W11</td>
<td>0.11</td>
<td>0.374</td>
<td>9505.722</td>
<td>8089.722</td>
<td>0.3185</td>
<td>0.7243</td>
</tr>
<tr>
<td>W15</td>
<td>0.15</td>
<td>0.437</td>
<td>11100.292</td>
<td>9684.292</td>
<td>0.3813</td>
<td>0.7611</td>
</tr>
<tr>
<td>W20</td>
<td>0.20</td>
<td>0.505</td>
<td>12817.513</td>
<td>11401.513</td>
<td>0.4489</td>
<td>0.7913</td>
</tr>
<tr>
<td>W24</td>
<td>0.24</td>
<td>0.553</td>
<td>14040.882</td>
<td>12624.882</td>
<td>0.4970</td>
<td>0.8085</td>
</tr>
<tr>
<td>W30</td>
<td>0.30</td>
<td>0.618</td>
<td>15698.184</td>
<td>14282.184</td>
<td>0.5623</td>
<td>0.8277</td>
</tr>
</tbody>
</table>
SPRING CONSTANT
Modulus of Sub-Grade Reaction and Spring Constant Check
From Principles of Foundation Engineering; Braja M. Das 8th Edition:

Table 8.2 Typical Subgrade Reaction Values, \( k_{0.3}(k_3) \)

<table>
<thead>
<tr>
<th>Soil type</th>
<th>MN/m²</th>
<th>lb/in³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry or moist sand:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>8–25</td>
<td>30–90</td>
</tr>
<tr>
<td>Medium</td>
<td>25–125</td>
<td>90–450</td>
</tr>
<tr>
<td>Dense</td>
<td>125–375</td>
<td>450–1350</td>
</tr>
<tr>
<td>Saturated sand:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>10–15</td>
<td>35–55</td>
</tr>
<tr>
<td>Medium</td>
<td>35–40</td>
<td>125–145</td>
</tr>
<tr>
<td>Dense</td>
<td>130–150</td>
<td>475–550</td>
</tr>
<tr>
<td>Clay:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stiff</td>
<td>10–25</td>
<td>40–90</td>
</tr>
<tr>
<td>Very stiff</td>
<td>25–50</td>
<td>90–185</td>
</tr>
<tr>
<td>Hard</td>
<td>&gt;50</td>
<td>&gt;185</td>
</tr>
</tbody>
</table>

\[
k_{0.3}(k_3) = \frac{1350 \text{ lb/in}^3}{(\text{dense Sand, value is conservative considering no reinforcement is included})}
\]

Adjust to panel 5' x 10' (1.5m x 3m)

Foundations on Sandy Soils
For foundations on sandy soils,

\[
k = k_{0.3} \left( \frac{B + 0.3}{2B} \right)^2
\]

(8.45)

where \( k_{0.3} \) and \( k \) = coefficients of subgrade reaction of foundations measuring 0.5 m × 0.3 m and \( B \times B \) m, respectively (unit is kN/m²).

In English units, Eq. (8.45) may be expressed as

\[
k = k_{1.5} \left( \frac{B + 1}{2B} \right)^2
\]

(8.46)

where \( k_{1.5} \) and \( k \) = coefficients of subgrade reaction of foundations measuring 1 ft × 1 ft and \( B \times B \) ft, respectively (unit is lb/in²),

\[
k_{1.5} = k_{0.3} \cdot \left( \frac{B + 0.3}{2B} \right)^2
\]

\[
k_{1.5} = 1350 \times \left( \frac{1.5 + 0.3}{2 \times 1.5} \right)^2
\]

\[
k_{1.5} = 486 \text{ lb/in}^3
\]

\[
k_{1.5 \times 3.0} = k_{1.5} \cdot \left( \frac{1 + \frac{15}{3.0}}{1.5} \right)^2
\]

\[
k_{1.5 \times 3.0} = 486 \text{ lb/in}^3
\]

\[
k_{1.5 \times 3.0} = 839.755 \text{ kip/ft}^2
\]
10-5 MODULUS OF SUBGRADE REACTION $k_z$
FOR MATS AND PLATES

All three discrete element methods given in this chapter for mats/plates use the modulus of subgrade reaction $k_z$ to support the plate. The modulus $k_z$ is used to compute node springs based on the contributing plan area of an element to any node as in Fig. 10-5. From the figure we see the following:

<table>
<thead>
<tr>
<th>Node</th>
<th>Contributing area</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (corner)</td>
<td>$\frac{1}{4}$ of rectangle abde</td>
</tr>
<tr>
<td>2 (side)</td>
<td>$\frac{1}{4}$ of abde + $\frac{1}{4}$ of bcef</td>
</tr>
<tr>
<td>3 (interior)</td>
<td>$\frac{1}{4}$ of each rectangle framing</td>
</tr>
</tbody>
</table>

For a triangle one should arbitrarily use one-third of the triangle area to any corner node. For these area contributions the fraction of $k_z$ node resistance from any element is

$$K_l = k_z \times Area, \ m^2 = \text{units of kN/m (or kips/ft in Fps)}$$

Since this computation gives units of a “spring” it is common to call the effect a node spring.

In this form the springs are independent of each other, the system of springs supporting the plate is termed a “Winkler” foundation, and the springs are uncoupled. Uncoupling means that the deflection of any spring is not influenced by adjacent springs.

**Spring Constant**

Note Spacing: 1ft x 1ft
Tributary Area: 1' x 1' = 1SF
$K_a = k \times a$
$K_a = 839.755 \text{ kip/ft (Input to Staad Pro)}$
MAXIMUM BENDING MOMENT
AND SHEAR
**Typical Panel Type (mesh 3W11), Panel Thickness 6", 5'x10'**

**Point load Input Force in panel for Staad Pro input**

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
</tr>
<tr>
<td>P1</td>
<td>0</td>
</tr>
<tr>
<td>P2</td>
<td>-0.33</td>
</tr>
<tr>
<td>P3</td>
<td>0.33</td>
</tr>
<tr>
<td>P4</td>
<td>-0.33</td>
</tr>
<tr>
<td>P5</td>
<td>0.33</td>
</tr>
<tr>
<td>P6</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
</tr>
<tr>
<td>P7</td>
<td>-0.33</td>
</tr>
<tr>
<td>P8</td>
<td>0.33</td>
</tr>
<tr>
<td>P9</td>
<td>0</td>
</tr>
<tr>
<td>P10</td>
<td>0</td>
</tr>
<tr>
<td>P11</td>
<td>-0.33</td>
</tr>
<tr>
<td>P12</td>
<td>0.33</td>
</tr>
</tbody>
</table>

**Load to panel based on Bar Mat Capacity per Bar Mat Point**

Bar Mat Used:
W11 - 75 Years

\[
A = 0.0797 \text{ in}^2 \\
F_y = 75 \text{ ksi} \\
P = 0.65 \times A \times F_y \\
= 3.885 \text{ Kips (per wire point)}
\]

**ATTACHMENT BY No. WIRES**

(standard panels only)
MX (local)
kNm/m
- <= 0.562
- 0.722
- 0.883
- 1.04
- 1.2
- 1.36
- 1.52
- 1.69
- 1.85
- 2.01
- 2.17
- 2.33
- 2.49
- 2.65
- 2.81
- 2.97
- >= 3.13
MY (local) kNm/m

<table>
<thead>
<tr>
<th>Value</th>
<th>Color</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;= -2.32</td>
<td>Purple</td>
</tr>
<tr>
<td>-1.86</td>
<td>Red</td>
</tr>
<tr>
<td>-1.4</td>
<td>Orange</td>
</tr>
<tr>
<td>-0.945</td>
<td>Brown</td>
</tr>
<tr>
<td>-0.487</td>
<td>Green</td>
</tr>
<tr>
<td>-0.029</td>
<td>Blue</td>
</tr>
<tr>
<td>0.430</td>
<td>Teal</td>
</tr>
<tr>
<td>0.888</td>
<td>Cyan</td>
</tr>
<tr>
<td>1.35</td>
<td>Light Blue</td>
</tr>
<tr>
<td>1.8</td>
<td>Coral</td>
</tr>
<tr>
<td>2.26</td>
<td>Yellow</td>
</tr>
<tr>
<td>2.72</td>
<td>Gold</td>
</tr>
<tr>
<td>3.18</td>
<td>Pink</td>
</tr>
<tr>
<td>3.64</td>
<td>Magenta</td>
</tr>
<tr>
<td>4.1</td>
<td>Lime</td>
</tr>
<tr>
<td>4.55</td>
<td>Orange Red</td>
</tr>
<tr>
<td>&gt;= 5.01</td>
<td>Red</td>
</tr>
</tbody>
</table>

Load 1

5.00ft 10.00ft
SOX (local)
N/mm²

- <= -0.118
- -0.103
- -0.089
- -0.074
- -0.059
- -0.044
- -0.029
- -0.015
- 0
- 0.015
- 0.030
- 0.045
- 0.060
- 0.074
- 0.089
- 0.104
- >= 0.119
1. STAAD SPACE

INPUT FILE: Panel Check - 3x10 type A (3W11).STD

2. START JOB INFORMATION
3. ENGINEER DATE C4-MM-21
4. JOB CLIENT SUL
5. ENGINEER NAME CH
6. JOB KEY PANEL CHECK
7. END JOB INFORMATION
8. INPUT WIDTH 79
9. UNIT LENGTH IP

10. JOINT COORDINATES
11. 1 0 0 0; 2 1 0 0; 3 2 0 0; 4 3 0 0; 5 4 0 0; 6 5 0 0; 10 0 0 -1; 11 1 0 -1.
12. 13 2 0 -1; 13 3 0 -1; 14 4 0 -1; 15 5 0 -1; 19 0 0 -2; 25 1 0 -2; 21 2 0 -2.
13. 22 3 0 -2; 23 4 0 -2; 24 5 0 -2; 28 0 0 -3; 29 1 0 -3; 32 2 0 -3; 31 3 0 -3.
14. 32 4 0 -3; 33 5 0 -3; 34 6 0 -4; 45 0 0 -4; 46 1 0 -4; 47 2 0 -4; 48 3 0 -4; 49 4 0 -4.
15. 50 5 0 -4; 51 6 0 -5; 52 7 0 -5; 53 8 0 -5; 54 9 0 -5; 61 0 0 -5.
16. 62 6 0 -1; 63 7 0 -1; 64 8 0 -1; 65 9 0 -1; 66 7 0 -1; 67 8 0 -2; 68 9 0 -2.
17. 79 6 0 -1; 70 7 0 0 -1; 71 8 0 0 -1; 72 8 0 0 -2; 73 8 0 -3; 74 9 0 -1; 15 9 0 0.
18. 16 6 0 -1; 17 7 0 -2; 18 8 0 -2; 19 9 0 -2; 16 7 0 -1; 17 8 0 -2; 18 9 0 -2.
19. 20 6 0 -1; 21 7 0 -1; 22 8 0 -1; 23 8 0 0 -1; 24 9 0 0 -1; 25 8 0 -1; 26 9 0 0 -1.
20. 97 6 0 -5; 98 7 0 -5; 99 8 0 -5.

21. ELEMENT INCIDENCES SHELL
22. 1 1 1 2; 1 1 2 2; 2 1 2 2; 1 1 2 3; 1 1 2 4; 2 1 2 4.
23. 3 2 3 2; 3 2 4 2; 4 2 4 2; 3 2 4 3; 3 2 3 4; 3 2 3 4.
24. 6 2 6 2; 6 2 6 3; 6 2 6 3; 6 2 6 4; 6 2 6 4; 6 2 6 4.
25. 9 2 9 2; 9 2 9 3; 9 2 9 3; 9 2 9 4; 9 2 9 4; 9 2 9 4.
26. 15 2 1 5 2; 15 2 1 5 3; 15 2 1 5 4; 15 2 1 5 4; 15 2 1 5 4; 15 2 1 5 4.
27. 21 2 2 1 2 2; 21 2 2 1 2 2; 21 2 2 1 2 2; 21 2 2 1 2 2; 21 2 2 1 2 2; 21 2 2 1 2 2.
28. 24 2 4 2 2 4; 24 2 4 2 2 4; 24 2 4 2 2 4; 24 2 4 2 2 4; 24 2 4 2 2 4; 24 2 4 2 2 4.
29. 26 2 6 2 6 2; 26 2 6 2 6 2; 26 2 6 2 6 2; 26 2 6 2 6 2; 26 2 6 2 6 2; 26 2 6 2 6 2.
30. 28 2 8 2 8 2; 28 2 8 2 8 2; 28 2 8 2 8 2; 28 2 8 2 8 2; 28 2 8 2 8 2; 28 2 8 2 8 2.
31. 30 3 0 3 0 3; 30 3 0 3 0 3; 30 3 0 3 0 3; 30 3 0 3 0 3; 30 3 0 3 0 3; 30 3 0 3 0 3.
32. 34 1 4 3 4 3; 34 1 4 3 4 3; 34 1 4 3 4 3; 34 1 4 3 4 3; 34 1 4 3 4 3; 34 1 4 3 4 3.
33. 35 1 5 3 5 3; 35 1 5 3 5 3; 35 1 5 3 5 3; 35 1 5 3 5 3; 35 1 5 3 5 3; 35 1 5 3 5 3.
34. 40 2 0 2 0 2; 40 2 0 2 0 2; 40 2 0 2 0 2; 40 2 0 2 0 2; 40 2 0 2 0 2; 40 2 0 2 0 2.
**Typical Panel Type (mesh 4W11), Panel Thickness 6", 5'x10'**

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
<td>Y</td>
<td>X</td>
<td>Y</td>
<td>X</td>
<td>Y</td>
<td>X</td>
</tr>
<tr>
<td>P1</td>
<td>0</td>
<td>0.25</td>
<td>P7</td>
<td>0.33</td>
<td>0.25</td>
<td>P9</td>
<td>0</td>
</tr>
<tr>
<td>P2</td>
<td>-0.33</td>
<td>0.25</td>
<td>P8</td>
<td>0</td>
<td>0.25</td>
<td>P10</td>
<td>-0.33</td>
</tr>
<tr>
<td>P3</td>
<td>0.33</td>
<td>0.25</td>
<td>P11</td>
<td>0.33</td>
<td>-0.25</td>
<td>P11</td>
<td>0.33</td>
</tr>
<tr>
<td>P4</td>
<td>0</td>
<td>0.25</td>
<td>P12</td>
<td>0</td>
<td>-0.25</td>
<td>P12</td>
<td>0</td>
</tr>
<tr>
<td>P5</td>
<td>-0.33</td>
<td>0.25</td>
<td>P13</td>
<td>0</td>
<td>-0.25</td>
<td>P13</td>
<td>0</td>
</tr>
<tr>
<td>P6</td>
<td></td>
<td></td>
<td>P14</td>
<td>-0.33</td>
<td>-0.25</td>
<td>P14</td>
<td>-0.33</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[
A = 0.0797 \text{ in}^2 \\
F_y = 75 \text{ ksi} \\
P = 0.65 \times A \times F_y \\
= 3.885 \text{ Kips} \quad \text{(per wire point)}
\]

![3-wire connection panel](image1)

![4-wire connection panel](image2)

![5-wire connection panel](image3)

**ATTACHMENT BY No. WIRES**

(standard panels only)
MX (local) kNm/m
- <= 1.4
- 1.56
- 1.72
- 1.88
- 2.04
- 2.2
- 2.35
- 2.51
- 2.67
- 2.83
- 2.99
- 3.15
- 3.31
- 3.47
- 3.63
- 3.79
- >= 3.95
### Load 1

#### MY (local) kNm/m

- <= -1.58
- -1.11
- -0.643
- -0.176
- 0.292
- 0.759
- 1.23
- 1.69
- 2.16
- 2.63
- 3.1
- 3.56
- 4.03
- 4.5
- 4.96
- 5.43
- >= 5.9

---

**File:** Panel Check - 5x10 type A (4W11).std  
**Print Time/Date:** 10/03/2021 03:06

**STAAD.Pro for Windows 20.07.04.12**
SOX (local)
N/mm²

- <= -0.144
-0.126
-0.108
-0.090
-0.072
-0.054
-0.036
-0.018
0
0.018
0.036
0.054
0.072
0.090
0.108
0.126

>= 0.144
SQY (local) N/mm²

- <= -0.131
- -0.115
- -0.098
- -0.082
- -0.065
- -0.049
- -0.033
- -0.016
- 0
- 0.016
- 0.033
- 0.049
- 0.065
- 0.082
- 0.098
- 0.115
- >= 0.131

Load 1

5.00ft 10.00ft
PAGE NO. 1

STAAD SPACE

INPUT FILE: Panel Check - 5x10 type A (4W11).STD

1. STAAD SPACE

-- PAGE NO. 2

STAAD SPACE

41. END DEFINE MATERIAL

42. CONSTANTS

43. MATERIAL CONCRETE ALL

44. SUPPORTS

45. 1 TO 6 10 TO 15 19 TO 24 28 TO 33 43 TO 50 56 TO 90 =

46. 91. FIXED NOT NXP 839.76 MXX 839.76 MXY 839.76 MZ 1

47. LOAD 1 LOADTYPE PUSH TUTTLE WIRE STRENGTH

48. ELEMENT LOAD

49. 32 FR DY -3.885 0.25 0

50. 33 FR GY -3.885 0.25 -0.33

51. 33 FR QY -3.885 0.25 0.33

52. 34 FR GZ -3.885 0.25 0

53. 62 FR QZ -3.885 0.25 0

54. 63 FR ZQ -3.885 0.25 -0.33

55. 63 FR QY -3.885 0.25 0.33

56. 64 FR QZ -3.885 0.25 0

57. 5 FR GY -3.885 -0.25 0

58. 8 FR GY -3.885 -0.25 -0.33

59. 8 FR QY -3.885 -0.25 0.33

60. 11 FR GY -3.885 -0.25 0

61. 50 FR QY -3.885 -0.25 0

62. 53 FR QZ -3.885 -0.25 -0.33

63. 53 FR ZQ -3.885 -0.25 0.33

64. 54 FR QZ -3.885 -0.25 0

65. PERFORM ANALYSIS

--- PROBLEM STATISTICS ---

NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS = 66/ 50/ 66

SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER

ORIGINAL/FINAL BANDWIDTH = 33/ 10/ 66 DOF

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 396

SHAPE OF STIFFNESS MATRIX = REGULAR/AVAIL. DISK SPACE = 12.5/ 263352.2 MB

66. FINISH

3. DEFINE MATERIAL START

35. ISOTROPIC CONCRETE

36. E 616304

37. POISSON 0.17

38. ECONOMY 0.15

39. ALPHA IF 0.05

40. EAMM 0.05

D:\SSL\505 Checking Only\Panel Design\staad\Panel Check - 5x10 type A (4W11).sol Page 1 of 5

D:\SSL\505 Checking Only\Panel Design\staad\Panel Check - 5x10 type A (4W11).sol Page 2 of 5
Typical Panel Type (mesh 5W11), Panel Thickness 6", 5'x10'

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>X</th>
<th>Y</th>
<th>X</th>
<th>Y</th>
<th>X</th>
<th>Y</th>
<th>X</th>
<th>Y</th>
<th>X</th>
<th>Y</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.83</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
<td>1.5</td>
<td>1.5</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
</tr>
<tr>
<td>P1</td>
<td>P2</td>
<td>P3</td>
<td>P4</td>
<td>P5</td>
<td>P6</td>
<td>P7</td>
<td>P8</td>
<td>P9</td>
<td>P10</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[
A = 0.0797 \text{ in}^2
\]

\[
F_y = 75 \text{ ksi}
\]

\[
P = 0.65 \times A \times F_y
\]

\[
= 3.885 \text{ Kips (per wire point)}
\]

ATTACHMENT BY No. WIRES
(Standard Panels Only)
MY (local) kNm/m

-2.29
-1.81
-1.32
-0.837
-0.353
0.132
0.616
1.1
1.58
2.07
2.55
3.04
3.52
4.01
4.49
4.98
5.46

Load 1

STAAD.Pro for Windows 20.07.04.12
**SQX (local)**

N/mm²

- <= -0.148
- -0.129
- -0.111
- -0.092
- -0.074
- -0.055
- -0.037
- -0.018
- 0
- 0.019
- 0.038
- 0.056
- 0.075
- 0.093
- 0.112
- 0.130
- >= 0.149

---

**Load 1**

5.00 ft  10.00 ft
SQY (local) N/mm²

- <= -0.131
- -0.115
- -0.098
- -0.082
- -0.065
- -0.049
- -0.033
- -0.016
- 0
- 0.016
- 0.033
- 0.049
- 0.065
- 0.082
- 0.098
- 0.115
- >= 0.131

STAAD.Pro for Windows 20.07.04.12
Typical Panel Type (mesh 6W11), Panel Thickness 6", 5'x10'

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
</thead>
<tbody>
<tr>
<td>X</td>
<td>Y</td>
</tr>
<tr>
<td>P1</td>
<td>0.33</td>
</tr>
<tr>
<td>P2</td>
<td>0</td>
</tr>
<tr>
<td>P3</td>
<td>-0.33</td>
</tr>
<tr>
<td>P4</td>
<td>0.33</td>
</tr>
<tr>
<td>P5</td>
<td>0</td>
</tr>
<tr>
<td>P6</td>
<td>-0.33</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
</thead>
<tbody>
<tr>
<td>X</td>
<td>Y</td>
</tr>
<tr>
<td>P7</td>
<td>0.33</td>
</tr>
<tr>
<td>P8</td>
<td>0</td>
</tr>
<tr>
<td>P9</td>
<td>-0.33</td>
</tr>
<tr>
<td>P10</td>
<td>0.33</td>
</tr>
<tr>
<td>P11</td>
<td>0</td>
</tr>
<tr>
<td>P12</td>
<td>-0.33</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[
A = 0.0797 \text{ in}^2
\]

\[
F_y = 75 \text{ ksi}
\]

\[
P = 0.65 \times A \times F_y = 3.885 \text{ Kips (per wire point)}
\]

ATTACHMENT BY No. WIRES
(STANDARD PANELS ONLY)
MX (local)
kNm/m

<table>
<thead>
<tr>
<th>Value</th>
<th>Colour</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;= 2.57</td>
<td>Black</td>
</tr>
<tr>
<td>2.73</td>
<td>Red</td>
</tr>
<tr>
<td>2.89</td>
<td>Red</td>
</tr>
<tr>
<td>3.05</td>
<td>Red</td>
</tr>
<tr>
<td>3.22</td>
<td>Red</td>
</tr>
<tr>
<td>3.38</td>
<td>Red</td>
</tr>
<tr>
<td>3.54</td>
<td>Red</td>
</tr>
<tr>
<td>3.7</td>
<td>Red</td>
</tr>
<tr>
<td>3.86</td>
<td>Red</td>
</tr>
<tr>
<td>4.02</td>
<td>Red</td>
</tr>
<tr>
<td>4.18</td>
<td>Red</td>
</tr>
<tr>
<td>4.34</td>
<td>Red</td>
</tr>
<tr>
<td>4.5</td>
<td>Red</td>
</tr>
<tr>
<td>4.66</td>
<td>Red</td>
</tr>
<tr>
<td>4.83</td>
<td>Red</td>
</tr>
<tr>
<td>4.99</td>
<td>Red</td>
</tr>
<tr>
<td>&gt;= 5.15</td>
<td>Red</td>
</tr>
</tbody>
</table>

Load 1

5.00ft  10.00ft
MY (local)
kNm/m

-0.061
0.249
0.56
0.871
1.18
1.49
1.8
2.12
2.43
2.74
3.05
3.36
3.67
3.98
4.29
4.6
>= 4.91

Load 1
SOX (local)
N/mm²

-0.153
-0.134
-0.115
-0.095
-0.076
-0.057
-0.038
-0.019
0
0.019
0.038
0.057
0.076
0.095
0.115
0.134
≥ 0.153
SQY (local)
N/mm²

- <= -0.103
- -0.090
- -0.077
- -0.064
- -0.051
- -0.039
- -0.026
- -0.013
- 0
- 0.013
- 0.026
- 0.039
- 0.051
- 0.064
- 0.077
- 0.090
- >= 0.103

Load 1

5.00ft 10.00ft

Print Time/Date: 10/03/2021 21:26
Print Run 1 of 1

STAAD.Pro for Windows 20.07.04.12
1. END MATERAIL
2. CONSTANTS
3. MATERIAL CONCRETE ALL
4. SUPPORTS
45. 1 TO 6 10 TO 15 19 TO 24 28 TO 33 37 TO 40 44 TO 47 -
46. 8 FIXED NOT NYK 839.76 MYK 839.76 NYK 839.76 NYK 1 NYK 1 KNZ 1
47. LOAD 1 LOADTYPE PUSH 1 TUTLE WIRE STRENTH
48. ELEMENT LOAD
49. 31 PR YX -3.885 0.25 0.33
50. 32 PR YX -3.885 0.25 0.33
51. 33 PR YX -3.885 0.25 -0.33
52. 34 PR YX -3.885 0.25 0.33
53. 34 PR YX -3.885 0.25 0.33
54. 35 PR YX -3.885 0.25 -0.33
55. 41 PR YX -3.885 0.25 0.33
56. 62 PR YX -3.885 0.25 0.33
57. 63 PR YX -3.885 0.25 -0.33
58. 63 PR YX -3.885 0.25 0.33
59. 64 PR YX -3.885 0.25 0.33
60. 65 PR YX -3.885 0.25 -0.33
61. 2 PR YX -3.885 0.25 -0.33
62. 5 PR YX -3.885 0.25 -0.33
63. 8 PR YX -3.885 0.25 -0.33
64. 8 PR YX -3.885 0.25 -0.33
65. 47 PR YX -3.885 0.25 -0.33
66. 48 PR YX -3.885 0.25 -0.33
67. 49 PR YX -3.885 0.25 -0.33
68. 50 PR YX -3.885 0.25 -0.33
69. 53 PR YX -3.885 0.25 -0.33
70. 53 PR YX -3.885 0.25 -0.33
71. 56 PR YX -3.885 0.25 -0.33
72. 59 PR YX -3.885 0.25 -0.33
73. PERFORM ANALYSIS

PROBLEM STATISTICS

NUMBER OF JOINTS/MEMBER/ELEMENTS/SUPPORTS = 66/ 50/ 66

SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER

ORIGINAL FINAL BAND-BREADTH= 33/ 10/ 66 DOF
TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 396
SIZE OF RIGIDITY MATRIX = 27 DOUBLE KILO-WORDS
RIGIDITY/AVAIL. DISK SPACE = 12.5/ 26322.1 MB

74. FINISH
**Typical Panel Type (mesh 4W20), Panel Thickness 6”, 5’x10’**

**Point load Input Force in panel for Staad Pro input**

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1’x1’</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
</tr>
<tr>
<td>P1</td>
<td>0</td>
</tr>
<tr>
<td>P2</td>
<td>-0.33</td>
</tr>
<tr>
<td>P3</td>
<td>0.33</td>
</tr>
<tr>
<td>P4</td>
<td>0</td>
</tr>
<tr>
<td>P5</td>
<td>0</td>
</tr>
<tr>
<td>P6</td>
<td>-0.33</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1’x1’</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
</tr>
<tr>
<td>P7</td>
<td>0.33</td>
</tr>
<tr>
<td>P8</td>
<td>0</td>
</tr>
</tbody>
</table>

**Load to panel based on Bar Mat Capacity per Bar Mat Point**

Bar Mat Used:
W11 - 75 Years

\[ A = 0.1583 \text{ in}^2 \]
\[ F_y = 75 \text{ ksi} \]

\[ P = 0.65 \times A \times F_y \]
\[ = 7.717 \text{ Kips} \] (per wire point)

**ATTACHMENT BY No. WIRES**

(Standard Panels Only)
MX (local) kNm/m
- <= 2.78
- 3.1
- 3.41
- 3.73
- 4.05
- 4.36
- 4.68
- 4.99
- 5.31
- 5.62
- 5.94
- 6.26
- 6.57
- 6.89
- 7.2
- 7.52
- >= 7.84

Load 1
<table>
<thead>
<tr>
<th>Load 1</th>
<th>5.00ft</th>
<th>10.00ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>MY (local)</td>
<td>kNm/m</td>
<td>&lt;= -3.13</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-2.21</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-1.28</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-0.349</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.579</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.51</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.44</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.36</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.29</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5.22</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6.15</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7.08</td>
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<td></td>
<td></td>
<td>8.01</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8.93</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8.86</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&gt;= 11.7</td>
</tr>
</tbody>
</table>
SOX (local) N/mm²

- <= -0.286
- -0.250
- -0.214
- -0.179
- -0.143
- -0.107
- -0.071
- -0.036
- 0
- 0.036
- 0.071
- 0.107
- 0.143
- 0.179
- 0.214
- 0.250
- >= 0.286

Print Time/Date: 10/03/2021 05:38
Print Run 1 of 1

STAAD.Pro for Windows 20.07.04.12
SQY (local) N/mm²

-0.260
-0.227
-0.195
-0.162
-0.130
-0.097
-0.065
-0.032
0
0.032
0.065
0.097
0.130
0.162
0.195
0.227
0.260

Load 1

STAAD.Pro for Windows 20.07.04.12
STAAD SPACE

INPUT FILE: Panel Check - 3x10 type A (4W20).STD

2. START JOB INFORMATION
3. ENGINEER DATE 09-NOV-21
4. JOB CLIENT SUL
5. ENGINEER NAME CE
6. JOB REV SHEET CHECK
7. END JOB INFORMATION
8. INPUT WIDTH 79
9. UNIT FEET KIP
10. JOINT COORDINATES
11. 1 00 0; 2 10 0; 3 20 0; 4 30 0; 5 40 0; 6 50 0; 10 00 -1; 11 10 -1.
12. 13 20 -1; 13 30 -1; 14 40 -1; 15 50 -1; 19 00 -2; 25 10 -2; 21 20 -2.
13. 22 30 -2; 23 40 -2; 24 50 -2; 28 00 -3; 29 10 -3; 32 20 -3; 33 30 -3.
14. 34 40 -3; 35 50 -3; 36 60 -4; 45 00 -4; 46 10 -4; 47 20 -4; 48 30 -4; 49 40 -4.
15. 50 50 -4; 56 60 -5; 59 70 -5; 67 80 -5; 68 90 -5; 69 10 -5.
16. 70 60 -6; 64 60 -6; 65 70 -6; 66 80 -7; 67 90 -7; 68 10 -7.
17. 71 80 -8; 72 90 -8; 73 10 -8; 74 90 -1; 75 10 -5.
18. 76 90 -5; 77 90 -3; 78 10 -5; 79 10 -5; 80 10 -5; 81 10 -3; 82 90 -5.
19. 83 70 -4; 84 80 -4; 85 90 -4; 86 100 -4; 87 70 -5; 88 80 -5.
20. 90 70 -5; 91 10 0 -5.
21. ELEMENT INCIDENCE SHEET
22. 1 1 10 12 1 2 10 19 20 11; 3 19 29 20 2 4 2 11 12 2 3 1 11 20 12.
23. 6 20 29 30 21; 7 12 13 4; 1 8 12 22 12 13; 9 21 30 31; 22; 10 13 24 9.
25. 31 28 45 46 29; 32 29 46 47 30; 33 30 47 48 31; 34 31 48 49 32; 35 32 49 50 33.
26. 41 45 56 57 46; 42 46 57 58 47; 43 47 58 59 48; 44 48 59 60 49; 45 49 60 61 50.
27. 46 6 56 61 31; 47 15 24 64 62; 48 24 33 65 64; 49 63 62 66 67; 50 62 66 68 66.
28. 51 64 65 69 68; 52 67 66 70 71; 53 66 68 72 70; 54 68 69 73 72; 55 71 70 74 75.
29. 56 70 72 76 74; 57 72 73 77 76; 58 75 74 78 79; 59 74 76 80 78; 60 76 77 81 80.
30. 61 35 82 86 65; 62 85 82 83 69; 63 59 83 86 73; 66 73 86 85 77; 67 55 85 86 81.
31. 66 80 81 87 92; 67 82 87 88 83; 68 83 88 89 84; 69 84 85 90 85; 70 95 90 91 86.
32. ELEMENT PROPERTY
33. 1 0 15 31 70 35 41 70 7 THICKNESS 0.5
34. DEFINE MATERIAL
35. ISOHOTIC CONCRETE
36. E 614304
37. POISSON 0.17
38. ENSITY 0.35
39. ALPHR ID 005
40. EAMIP 0.05

--- PROBLEM STATISTICS ---

NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS = 66/ 50/ 66
SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER
ORIGINAL/FINAL BANDWIDTH = 33/10/66 DOF
TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 396
STIFFNESS MATRIX = 27 DOUBLE KILO-WOOD
MAX. ALLOWABLE DISK SPACE = 12.5/263347.2 MB
## Typical Panel Type (mesh 5W20), Panel Thickness 6", 5'x10'

**Point load Input Force in panel for Staad Pro input**

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
</tr>
<tr>
<td>P1</td>
<td>0.33</td>
</tr>
<tr>
<td>P2</td>
<td>0</td>
</tr>
<tr>
<td>P3</td>
<td>-0.33</td>
</tr>
<tr>
<td>P4</td>
<td>0.33</td>
</tr>
<tr>
<td>P5</td>
<td>0</td>
</tr>
<tr>
<td>P6</td>
<td>0</td>
</tr>
</tbody>
</table>

**Load to panel based on Bar Mat Capacity per Bar Mat Point**

Bar Mat Used:
W11 - 75 Years

\[
A = 0.1583 \text{ in}^2 \\
F_y = 75 \text{ ksi} \\
P = 0.65 \times A \times F_y \\
= 7.717 \text{ Kips} \quad \text{ (per wire point)}
\]

3–WIRE CONNECTION PANEL 4–WIRE CONNECTION PANEL 5–WIRE CONNECTION PANEL

**ATTACHMENT BY No. WIRES**

(Standards Panels Only)
Load 1
MX (local) kNm/m

- <= 2.51
- 2.96
- 3.4
- 3.84
- 4.28
- 4.72
- 5.16
- 5.6
- 6.05
- 6.49
- 6.93
- 7.37
- 7.81
- 8.25
- 8.7
- 9.14
- >= 9.58
MY (local)
kNm/m

-4.55
-3.59
-2.62
-1.66
-0.701
0.261
1.22
2.19
3.15
4.11
5.07
6.03
7
7.96
8.92
9.88
>= 10.8
SQX (local)
N/mm²

-0.294
-0.257
-0.220
-0.183
-0.146
-0.109
-0.073
-0.036
0.001
0.038
0.075
0.112
0.148
0.185
0.222
0.259
0.296

Load 1
1. STRAD SPACE

INPUT FILE: Panel Check - 5x10 type A (SM20).STD

2. START JOB INFORMATION

3. ENGINEER DATE 09-Mar-21

4. JOB CLIENT 85L

5. ENGINEER NAME CS

6. JOB SHEET NAME CHECK

7. END JOB INFORMATION

8. INPUT WIDTH 79

9. UNIT FEET INCH

10. JOINT COORDINATES

11. 1 0 0 0; 2 1 0 0; 3 2 0 0; 4 3 0 0; 5 4 0 0; 6 5 0 0; 10 0 0 -1; 11 1 0 -1.
12. 12 2 0 -1; 13 3 0 -1; 14 4 0 -1; 15 5 0 -1; 19 0 0 -2; 20 1 0 -2; 21 2 0 -2.
13. 22 0 0; 23 0 0; 24 0 0; 25 0 0; 26 0 0; 27 0 0; 28 0 0; 29 0 0; 30 0 0; 31 0 0.
14. 32 0 0; 33 0 0; 34 0 0; 35 0 0; 36 0 0; 37 0 0; 38 0 0; 39 0 0; 40 0 0; 41 0 0.
15. 42 0 0; 43 0 0; 44 0 0; 45 0 0; 46 0 0; 47 0 0; 48 0 0; 49 0 0; 50 0 0; 51 0 0.
16. 52 0 0; 53 0 0; 54 0 0; 55 0 0; 56 0 0; 57 0 0; 58 0 0; 59 0 0; 60 0 0; 61 0 0.
17. 62 0 0; 63 0 0; 64 0 0; 65 0 0; 66 0 0; 67 0 0; 68 0 0; 69 0 0; 70 0 0; 71 0 0.
18. 72 0 0; 73 0 0; 74 0 0; 75 0 0; 76 0 0; 77 0 0; 78 0 0; 79 0 0; 80 0 0; 81 0 0.
19. 82 0 0; 83 0 0; 84 0 0; 85 0 0; 86 0 0; 87 0 0; 88 0 0; 89 0 0; 90 0 0; 91 0 0.
20. 92 0 0; 93 0 0; 94 0 0; 95 0 0; 96 0 0; 97 0 0; 98 0 0; 99 0 0; 100 0 0.

21. ELEMENT OCCURRENCE SHELL

22. 1 1 10 11 2 10 19 20 11 3 15 29 29 20 4 2 11 12 3 2 11 20 21 12.
23. 6 20 29 30 21 7 3 12 13 4 8 12 21 22 12 9 21 30 21 12 14 13 24.
25. 31 28 45 46 29 32 29 46 47 30 33 30 48 49 32 35 32 49 50 32.
26. 41 43 58 57 46 42 46 57 58 47 43 47 58 59 48 44 48 59 60 49 45 49 60 63 50.
27. 46 56 62 61 47 45 54 64 62 48 24 33 65 64 49 63 62 66 67 50 62 64 68 64.
28. 51 64 65 65 68 52 52 67 66 70 71 53 66 68 72 72 54 68 69 73 72 55 71 70 74 75.
29. 58 70 72 76 74 57 12 73 77 74 58 75 74 78 79 59 74 76 80 78 60 76 77 81 80.
30. 61 33 50 82 61 62 65 82 63 69 63 59 83 86 73 66 73 86 85 77 65 77 85 86.
31. 69 50 81 87 82 61 87 82 87 88 88 83 60 83 88 89 84 69 64 89 90 85 70 85 90 91 86.
32. ELEMENT PROPERTY

33. 1 TO 15 31 TO 35 41 TO 75 THICKNESS 0.5
34. DEFINE MATERIAL START
35. ISOTROPIC CONCRETE
36. ; E 614304
37. POISSON 0.17
38. EQUITY 0.15
39. ALPHA 1E-05
40. EMPP 0.05

41. END DEFINE MATERIAL
42. CONSTANTS
43. MATERIAL CONCRETE ALL
44. SUPPORTS
45. 1 TO 10 TO 15 19 TO 24 28 TO 33 35 TO 50 56 TO 90
46. 81. FIXED BUT XYZ 839.76 MXT 839.76 MXT 839.76 MXT 839.76 MXT 840.15 KUX 840.15 KMZ 1
47. LOAD 1 LOADTYPE PUSH TITTLE WIRE STRENGTH
48. ELEMENT LOAD
49. 31 FR Y -7.717 -0.25 0.33
50. 32 FR Y -7.717 -0.25 0
51. 33 FR Y -7.717 -0.25 -0.33
52. 33 FR Y -7.717 -0.25 0.33
53. 34 FR Y -7.717 -0.25 0
54. 62 FR Y -7.717 -0.25 0
55. 62 FR Y -7.717 -0.25 -0.33
56. 62 FR Y -7.717 -0.25 0.33
57. 68 FR Y -7.717 -0.25 0
58. 68 FR Y -7.717 -0.25 -0.33
59. 5 FR Y -7.717 -0.25 -0.33
60. 8 FR Y -7.717 -0.25 -0.33
61. 8 FR Y -7.717 -0.25 -0.33
62. 11 FR Y -7.717 -0.25 0
63. 14 FR Y -7.717 -0.25 -0.33
64. 3 FR Y -7.717 -0.25 0
65. 50 FR Y -7.717 -0.25 0
66. 53 FR Y -7.717 -0.25 -0.33
67. 53 FR Y -7.717 -0.25 0.33
68. 70 FR Y -7.717 -0.25 0
69. 70 FR Y -7.717 -0.25 -0.33
70. PERFORM ANALYSIS

PROBLEM STATISTIC

NUMBER OF JOINTS/MEMBER/ELEMENTS/SUPPORTS = 66/ 50/ 66
SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER

ORIGINAL/FINAL BAND WIDTH = 33/ 10/ 66 DEF
TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 396
SIZE OF STIFFNESS MATRIX = 27 DOUBLE KILO-WORDS
MAXMEM/AVAIL. DISK SPACE = 12.5/ 263329.7 MB

70. FINISH
### Typical Panel Type (mesh 6W20), Panel Thickness 6”, 5’x10’

#### Point load input force in panel for Staad Pro input

<table>
<thead>
<tr>
<th></th>
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<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>0.83</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
<td>0.83</td>
<td>0.83</td>
<td>0.67</td>
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<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
<td>0.83</td>
<td>0.83</td>
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<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
<td>0.83</td>
</tr>
</tbody>
</table>

#### Load to panel based on Bar Mat Capacity per Bar Mat Point

**Bar Mat Used:**
- W11 - 75 Years

\[
A = 0.1583 \text{ in}^2 \\
F_y = 75 \text{ ksi} \\
P = 0.65 \times A \times F_y \\
= 7.717 \text{ Kips (per wire point)}
\]

#### 3-Wire Connection Panel

#### 4-Wire Connection Panel

#### 5-Wire Connection Panel

**Attachment by No. Wires**

*(Standard panels only)*
MX (local) 
kNm/m 

- <= 5.11
- 5.43
- 5.75
- 6.07
- 6.39
- 6.71
- 7.03
- 7.35
- 7.67
- 7.99
- 8.31
- 8.63
- 8.95
- 9.27
- 9.59
- 9.91
- >= 10.2

Load 1

5.00ft
10.00ft
MY (local) kNm/m
-0.122
0.496
1.11
1.73
2.35
2.97
3.58
4.2
4.82
5.44
6.05
6.67
7.29
7.91
8.52
9.14
9.76

Load 1

Print Time/Date: 10/03/2021 21:40
Print Run 1 of 1
SOX (local)
N/mm²

-0.303
-0.266
-0.228
-0.190
-0.152
-0.114
-0.076
-0.038
0
0.038
0.076
0.114
0.152
0.190
0.228
0.266
>= 0.303
SQY (local)
N/mm²

<table>
<thead>
<tr>
<th>Value</th>
<th>Color</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;= -0.204</td>
<td>dark red</td>
</tr>
<tr>
<td>-0.179</td>
<td>red</td>
</tr>
<tr>
<td>-0.153</td>
<td>brown</td>
</tr>
<tr>
<td>-0.128</td>
<td>purple</td>
</tr>
<tr>
<td>-0.102</td>
<td>blue</td>
</tr>
<tr>
<td>-0.077</td>
<td>dark blue</td>
</tr>
<tr>
<td>-0.051</td>
<td>blue</td>
</tr>
<tr>
<td>-0.026</td>
<td>green</td>
</tr>
<tr>
<td>0</td>
<td>yellow</td>
</tr>
<tr>
<td>0.026</td>
<td>light yellow</td>
</tr>
<tr>
<td>0.051</td>
<td>green</td>
</tr>
<tr>
<td>0.077</td>
<td>yellow</td>
</tr>
<tr>
<td>0.102</td>
<td>brown</td>
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<tr>
<td>0.128</td>
<td>purple</td>
</tr>
<tr>
<td>0.153</td>
<td>red</td>
</tr>
<tr>
<td>0.179</td>
<td>dark red</td>
</tr>
<tr>
<td>&gt;= 0.204</td>
<td>dark red</td>
</tr>
</tbody>
</table>

STAAD.Pro for Windows 20.07.04.12

Print Time/Date: 10/03/2021 21:40
Print Run 1 of 1
**Typical Panel Type (mesh 5W24), Panel Thickness 6”, 5’x10’**

**Point load Input Force in panel for Staad Pro input**

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1’x1’</th>
<th>Location</th>
<th>Coordinated From Center Panel 1’x1’</th>
<th>Location</th>
<th>Coordinated From Center Panel 1’x1’</th>
<th>Location</th>
<th>Coordinated From Center Panel 1’x1’</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
<td>Y</td>
<td></td>
<td>X</td>
<td>Y</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>P1</td>
<td>0.33</td>
<td>0.25</td>
<td>P7</td>
<td>-0.33</td>
<td>0.25</td>
<td>P11</td>
<td>0</td>
</tr>
<tr>
<td>P5</td>
<td>0</td>
<td>0.25</td>
<td>P15</td>
<td>-0.33</td>
<td>-0.25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P6</td>
<td>0</td>
<td>0.25</td>
<td>P16</td>
<td>0.33</td>
<td>-0.25</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Load to panel based on Bar Mat Capacity per Bar Mat Point**

Bar Mat Used:
W11 - 75 Years

\[
A = 0.194 \text{ in}^2 \\
F_y = 75 \text{ ksi} \\
P = 0.65 \times A \times F_y = 9.458 \text{ Kips} \quad \text{(per wire point)}
\]

**3–WIRE CONNECTION PANEL**

**4–WIRE CONNECTION PANEL**

**5–WIRE CONNECTION PANEL**

**ATTACHMENT BY No. WIRES**

*(STANDARD PANELS ONLY)*
MX (local)
kNm/m

- <= 3.08
- 3.62
- 4.16
- 4.7
- 5.25
- 5.79
- 6.33
- 6.87
- 7.41
- 7.95
- 8.49
- 9.03
- 9.58
- 10.1
- 10.7
- 11.2
- >= 11.7
MY (local)
kNm/m

-5.58
-4.4
-3.22
-2.04
-0.859
0.320
1.5
2.68
3.86
5.04
6.22
7.4
8.57
9.75
10.9
12.1
>= 13.3

Load 1
SOX (local)
N/mm²

-0.360
-0.315
-0.270
-0.224
-0.179
-0.134
-0.089
-0.044
0.001
0.046
0.092
0.137
0.182
0.227
0.272
0.317
>= 0.362
SQY (local)
N/mm²
<= -0.319
-0.279
-0.239
-0.199
-0.159
-0.119
-0.080
-0.040
0
0.040
0.080
0.119
0.159
0.199
0.239
0.279
>= 0.319
1. STAAD SPACE
INPUT FILE: Panel Check - 5x10 type A (SM24).STD
2. START JOB INFORMATION
3. ENGINEER DATE 03-MAR-21
4. JOB CLIENT ESL
5. ENGINEER NAME CH
6. JOB REF SHEET CHECK
7. END JOB INFORMATION
8. INPUT WIDTH 79
9. UNIT PRESET KIP
10. JOINT COORDINATES
11. 1 0 0 0; 2 1 0 0; 3 2 0 0; 4 3 0 0; 5 4 0 0; 6 5 0 0; 10 0 0 -1; 11 1 0 -1.
12. 12 2 0 -1; 13 3 0 -1; 14 4 0 -1; 15 5 0 -1; 19 0 0 -2; 25 1 0 -2; 21 2 0 -2
13. 22 3 0 -2; 23 4 0 -2; 24 5 0 -2; 28 0 0 -3; 29 1 0 -3; 32 2 0 -3; 33 3 0 -3
14. 34 4 0 -3; 35 5 0 -3; 36 6 0 -3; 37 7 0 -3; 38 8 0 -3; 39 9 0 -3
15. 50 6 0 -4; 56 6 0 -5; 57 1 0 -5; 58 2 0 -5; 59 3 0 -5; 60 4 0 -5; 61 5 0 -5
16. 62 6 0 -6; 63 6 0 -6; 64 6 0 -6; 65 6 0 -6; 67 7 0 -6; 67 7 0 -6; 68 7 0 -2
17. 69 7 0 -3; 70 8 0 -1; 71 9 0 0; 72 8 0 -2; 73 8 0 -3; 74 9 0 -1; 75 9 0 0
18. 76 9 0 -2; 77 9 0 -2; 78 10 0 -1; 79 10 0 0; 80 10 0 -2; 81 10 0 -3; 82 6 0 -4
19. 83 7 0 -4; 84 8 0 -4; 85 9 0 -4; 86 10 0 -4; 87 6 0 -5; 88 7 0 -5; 89 6 0 -5
20. 90 9 0 -5; 91 10 0 -5
21. ELEMENT DESCRIPTION SHELL
22. 1 111 2 1 10 19 20 11; 3 15 29 29 20 4 2 11 12 21 3 11 20 21 12
23. 6 20 29 30 21; 7 3 12 13 4; 8 12 21 22 13; 9 21 30 31 22; 10 4 13 24
24. 11 12 23 24 13; 12 24 31 32 23; 13 3 14 15 6; 14 15 16 24 15; 15 23 32 33 24
25. 31 28 46 46 29; 32 29 46 47 30; 33 30 48 48 31; 34 31 48 49 32; 35 32 49 50 33
26. 41 45 56 57 44; 42 44 57 58 47; 43 47 58 59 46; 44 48 59 60 49; 45 49 60 61 50
27. 46 6 15 6 31; 47 15 24 6 62; 48 24 33 56 64; 49 63 62 66 67; 50 62 64 68 66
28. 51 66 65 68 68; 52 67 66 70 71; 53 66 68 72 70; 54 68 69 73 72; 55 71 70 74 75
29. 56 70 72 76 74; 57 72 73 77 76; 58 75 74 78 73; 59 74 76 80 78; 60 76 77 81 80
30. 61 33 50 82 65; 62 65 82 83 69; 63 59 83 86 73; 66 73 85 85 77; 65 77 85 86 81
31. 66 50 61 87 82; 67 82 87 88 83; 68 83 88 89 84; 69 84 89 90 85; 70 85 90 91 86
32. ELEMENT PROPERTIES
33. 1 15 31 70 35 41 70 TD THICKNESS 0.5
34. DEFINE MATERIAL START
35. ISOTROPIC CONCRETE
36. E 614304
37. POISSON 0.17
38. DENSITY 2.45
39. ALPHER 1.05
40. EARTH 0.05

STAAD SPACE

41. END DEFINE MATERIAL
42. CONSTANTS
43. MATERIAL CONCRETE ALL
44. SUPPORTS
45. 1 TO 6 10 TO 15 19 TO 24 28 TO 33 45 TO 50 56 TO 90 -
46. 91 FIXED BUT RXF 839.76 MYF 839.76 KYF 839.76 KXY 1 KMY 1 KMZ 1
47. LOAD 1 LOADTYPE PUSH TITTLE WIRE STRENGTH
48. ELEMENT LOAD
49. 31 01 0.45 0.25 0.33
50. 32 FR QY 0.45 0.25 0.8
51. 33 FR QY 0.45 0.25 0.33
52. 35 FR QY 0.45 0.25 0.33
53. 34 FR QY 0.45 0.25 0.33
54. 32 FR QY 0.45 0.25 0.33
55. 63 FR QY 0.45 0.25 0.33
56. 63 FR QY 0.45 0.25 0.33
57. 64 FR QY 0.45 0.25 0.33
58. 65 FR QY 0.45 0.25 0.33
59. 5 FR QY 0.45 0.25 0.33
60. 8 FR QY 0.45 0.25 0.33
61. 14 FR QY 0.45 0.25 0.33
62. 15 FR QY 0.45 0.25 0.33
63. 14 FR QY 0.45 0.25 0.33
64. 47 FR QY 0.45 0.25 0.33
65. 50 FR QY 0.45 0.25 0.33
66. 53 FR QY 0.45 0.25 0.33
67. 53 FR QY 0.45 0.25 0.33
68. 56 FR QY 0.45 0.25 0.33
69. PERFORM ANALYSIS

PROBLEM STATISTICS

NUMBER OF JOINTS/MEMBER ELEMENT SUPPORTS = 66 / 50 / 66
SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER
ORIGINAL/FINAL BAND-WIDTH = 33 / 10 / 66 DDF
TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 396
SIZE OF STIFFNESS MATRIX = 27 DOUBLE KILO-WORDS
REQUIRED/AVAILABLE DISK SPACE = 12.5 / 263327.2 Mb

70. FINISH
Typical Panel Type (mesh 6W24), Panel Thickness 6", 5’x10’

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1’x1’ X</th>
<th>Y</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>0.33</td>
<td>0.25</td>
</tr>
<tr>
<td>P2</td>
<td>0</td>
<td>0.25</td>
</tr>
<tr>
<td>P3</td>
<td>-0.33</td>
<td>0.25</td>
</tr>
<tr>
<td>P4</td>
<td>0.33</td>
<td>0.25</td>
</tr>
<tr>
<td>P5</td>
<td>0</td>
<td>0.25</td>
</tr>
<tr>
<td>P6</td>
<td>-0.33</td>
<td>0.25</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1’x1’ X</th>
<th>Y</th>
</tr>
</thead>
<tbody>
<tr>
<td>P7</td>
<td>0.33</td>
<td>0.25</td>
</tr>
<tr>
<td>P8</td>
<td>0</td>
<td>0.25</td>
</tr>
<tr>
<td>P9</td>
<td>-0.33</td>
<td>0.25</td>
</tr>
<tr>
<td>P10</td>
<td>0.33</td>
<td>0.25</td>
</tr>
<tr>
<td>P11</td>
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<td>0.25</td>
</tr>
<tr>
<td>P12</td>
<td>-0.33</td>
<td>0.25</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[
A = 0.194 \text{ in}^2
\]
\[
Fy = 75 \text{ ksi}
\]
\[
P = 0.65 \times A \times Fy = 9.458 \text{ Kips (per wire point)}
\]

ATTACHMENT BY No. WIRES
(Standard panels only)
MX (local)  kNm/m
<= 6.26
6.65
7.04
7.43
7.83
8.22
8.61
9
9.4
9.79
10.2
10.6
11
11.4
11.7
12.1
>= 12.5

Load 1
MY (local) kNm/m
- <= -0.150
- 0.607
- 1.36
- 2.12
- 2.88
- 3.64
- 4.39
- 5.15
- 5.91
- 6.66
- 7.42
- 8.18
- 8.93
- 9.69
- 10.4
- 11.2
- >= 12
SOX (local) N/mm²

- <= -0.372
- -0.325
- -0.279
- -0.232
- -0.186
- -0.139
- -0.093
- -0.046
- 0
- 0.046
- 0.093
- 0.139
- 0.186
- 0.232
- 0.279
- 0.325
- >= 0.372

STAAD.Pro for Windows 20.07.04.12
SQY (local) N/mm²
-0.250
-0.219
-0.188
-0.156
-0.125
-0.094
-0.063
-0.031
0
0.031
0.063
0.094
0.125
0.156
0.188
0.219
0.250

5.00ft 10.00ft
1. STAAD SPACE

INPUT FILE: Panel Check - 3x10 type A (E024).STD

2. START JOB INFORMATION

3. ENGINEER DATE 9/21/93

4. JOB CLIENT CSL

5. ENGINEER NAME CH

6. JOB REF SHEET CHECK

7. END JOB INFORMATION

8. INPUT WIDTH 79

9. UNIT FEET KIP

10. JOINT COORDINATES

11. 1 0 0 0; 2 1 0 0; 3 2 0 0; 4 3 0 0; 5 4 0 0; 6 5 0 0; 10 0 0 -1; 11 1 0 -1.

12. 12 2 0 -1; 13 3 0 -1; 14 4 0 -1; 15 5 0 -1; 19 0 0 -2; 25 1 0 -2; 21 2 0 -2.

13. 22 3 0 -2; 23 4 0 -2; 24 5 0 -2; 28 0 0 -3; 29 1 0 -3; 32 2 0 -3; 33 3 0 -3.

14. 34 4 0 -3; 35 5 0 -3; 36 6 0 -3; 37 5 0 -3; 58 2 0 -3; 59 3 0 -3; 60 4 0 -3; 61 5 0 -3.

15. 62 6 0 -1; 63 6 0 -1; 64 6 0 -2; 65 6 0 -3; 66 7 0 -1; 67 7 0 -1; 68 7 0 -2.

16. 69 7 0 -3; 70 8 0 -1; 71 8 0 -1; 72 8 0 -2; 73 8 0 -3; 74 8 0 -1; 75 9 0 0.

17. 76 9 0 -2; 77 8 0 -3; 78 8 0 -4; 79 8 0 -3; 80 8 0 -2; 81 0 0 -1; 82 0 0 -1.

18. 83 0 0 -1; 84 8 0 -1; 85 9 0 -1; 86 10 0 -4; 87 6 0 -5; 88 7 0 -5; 89 6 0 -5.

19. 90 9 0 -5; 91 9 0 -5.

21. ELEMENT INCIDENCE SHELL

22. 1 1 10 12 2 10 19 20 11; 3 19 29 20 4 2 1 12 3 1 12 20 12.

23. 6 20 29 30 21; 7 2 22 3; 8 1 21 22 12; 9 21 30 22; 21 4 13 24.


25. 31 28 45 46 29; 32 29 46 47 30; 33 30 47 48 31; 34 31 48 49 32; 35 32 49 50 33.

26. 41 43 56 57 44; 42 44 57 58 47; 43 47 58 59 48; 44 48 59 60 49; 45 49 60 61 50.

27. 46 6 5 6 2 6; 47 15 24 64 62; 48 24 33 65 64; 49 63 62 66 67; 50 62 64 68 66.

28. 51 64 65 68 68; 52 67 66 70 71; 53 66 68 72 70; 54 68 69 73 72; 55 70 70 74 70.

29. 56 70 72 74 70; 57 72 73 77 74; 58 75 74 78 79; 59 74 76 80 78; 60 76 77 81 80.

30. 61 33 50 82 61; 62 65 82 83 69; 63 59 63 86 73; 66 73 86 85 77; 67 77 85 86 81.

31. 66 50 61 87 82; 67 82 87 88 83; 68 83 88 89 84; 69 84 90 90 85; 70 85 90 91 86.

32. ELEMENT PROPERTY

33. 1 70 15 31 10 35 41 70 THICKNESS 0.5

34. DEFINE MATERIAL LIST

35. ISOTROPIC CONCRETE

36. E 614304

37. POISON 0.17

38. DENSITY 0.15

39. ALPHE 0.05

40. DEMF 0.05

---

STAAD SPACE

41. END DEFINE MATERIAL

42. CONSTANTS

43. MATERIAL CONCRETE ALL

44. SUPPORTS

45. 1 TO 6 10 TO 72 13 TO 24 28 TO 33 43 TO 50 56 TO 90

46. 41 FIXED NOT NTK 839.76 NTK 839.76 NTK 839.76 NTK 839.76 NTK 839.76 NTK 839.76

47. LOAD 1 LOADTYPE PUSH TUTTLE WIRE STRENGTH

48. ELEMENT LOAD

49. 31 FR YG -9.458 0.25 0.33

50. 32 FR YG -9.458 0.25 0.33

51. 33 FR YG -9.458 0.25 0.33

52. 33 FR YG -9.458 0.25 0.33

53. 34 FR YG -9.458 0.25 0.33

54. 35 FR YG -9.458 0.25 0.33

55. 36 FR YG -9.458 0.25 0.33

56. 37 FR YG -9.458 0.25 0.33

57. 38 FR YG -9.458 0.25 0.33

58. 39 FR YG -9.458 0.25 0.33

59. 40 FR YG -9.458 0.25 0.33

60. 41 FR YG -9.458 0.25 0.33

61. 42 FR YG -9.458 0.25 0.33

62. 43 FR YG -9.458 0.25 0.33

63. 44 FR YG -9.458 0.25 0.33

64. 45 FR YG -9.458 0.25 0.33

65. 46 FR YG -9.458 0.25 0.33

66. 47 FR YG -9.458 0.25 0.33

67. 48 FR YG -9.458 0.25 0.33

68. 49 FR YG -9.458 0.25 0.33

69. 50 FR YG -9.458 0.25 0.33

70. 51 FR YG -9.458 0.25 0.33

71. 52 FR YG -9.458 0.25 0.33

72. 53 FR YG -9.458 0.25 0.33

73. 54 FR YG -9.458 0.25 0.33

74. FINISH

---

PROBLEM STATISTICS

NUMBER OF JOINTS/MEMBER/ELEMENTS/SUPPORTS = 66/ 50/ 66

SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER

ORIGINAL FINAL BAND-WIDTH = 33/ 10/ 66 DOF

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 396

SMALL SRGE/AVAIL. DISK SPACE = 27 DOUBLE KILO-WORDS

---

74. FINISH
## Summary Result From Staad Pro Analysis Output

<table>
<thead>
<tr>
<th>Standard Panel Size</th>
<th>Barmat Type</th>
<th>Panel Type</th>
<th>Horz Rebar</th>
<th>Vert Rebar</th>
<th>Staad Pro Output</th>
<th>Data for AASHTO LRFD Design Check</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Mx (kNm/m)</td>
<td>My (kNm/m)</td>
</tr>
<tr>
<td>5'x10'</td>
<td>3W11</td>
<td>A</td>
<td>4 #4</td>
<td>7 #4</td>
<td>3.130</td>
<td>5.010</td>
</tr>
<tr>
<td></td>
<td>4W11</td>
<td>A</td>
<td>4 #4</td>
<td></td>
<td>3.950</td>
<td>5.900</td>
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<tr>
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<td>5W11</td>
<td>A</td>
<td>4 #4</td>
<td></td>
<td>4.820</td>
<td>5.460</td>
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<tr>
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<td>6W11</td>
<td>A</td>
<td>4 #4</td>
<td></td>
<td>5.150</td>
<td>4.910</td>
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<tr>
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<td>4W20</td>
<td>X</td>
<td>5 #4</td>
<td>10 #4</td>
<td>7.840</td>
<td>11.700</td>
</tr>
<tr>
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<td>5W20</td>
<td>X</td>
<td>5 #4</td>
<td>10 #4</td>
<td>9.580</td>
<td>10.800</td>
</tr>
<tr>
<td></td>
<td>6W20</td>
<td>X</td>
<td>5 #4</td>
<td>10 #4</td>
<td>10.200</td>
<td>9.760</td>
</tr>
<tr>
<td></td>
<td>5W24</td>
<td>Y</td>
<td>6 #4</td>
<td>10 #4</td>
<td>11.700</td>
<td>13.300</td>
</tr>
<tr>
<td></td>
<td>6W24</td>
<td>Y</td>
<td>6 #4</td>
<td>10 #4</td>
<td>12.500</td>
<td>12.000</td>
</tr>
</tbody>
</table>

* Maximum bending moment and shear for each standard panel size shown in red bold font
5’ x 10’ STANDARD
A PANEL
**Typical Panel Type A, Panel Thickness 6", 5'x10', Horizontal Rebar**

**Panel Strength Design**
Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

- **Panel Thickness**: \( t_{\text{panel}} \) = 6 in
- **Finishing Thickness**: \( C_{\text{panel}} \) = 1.5 in
- **Additional Load from Finishes**: = 0.0125 Kip/ft²
- **Concrete Cover**: \( C_{\text{panel}} \) = 1.5 in
- **Vertical Bar Size**: #4
- **Logitudinal Bar Size**: #4
- **Rebar Diameter**: \( d_b \) = 0.5 in
- **Rebar Area**: \( A_b \) = 0.196 in²
- **Rebar spacing**: \( s \) = 18 in
- **Width of the design section**: \( b \) = 12 in
- **Effective depth of section**: \( d_s \) = 4.25 in
- **Effective depth of section for Negative Moment**: \( d_s(M^-) \) = 1.75 in
- **Correction factor for source aggregate**: \( K_s \) = 1 in
- **Concrete Density**: \( W_c \) = 0.15 kcf
- **Concrete Strength**: \( f' c \) = 4.00 ksi (Concrete Class A compressive strength)
- **Reinforcement Strength**: \( f_y \) = 60 ksi (Minimum yield strength of grade 60 steel)
- **Modulus Elasticity of concrete**: \( E_c \) = 4266 ksi
- **Modulus Elasticity of reinforcement**: \( E_s \) = 29000 ksi
- **Modular Ratio**: \( n \) = 6.798
- **Area of Steel per Design Strip**: \( A_s \) = 0.131 in²

\[
+M_u = 1.33 \text{ kip-ft} \quad +M_{u, \text{service}} = 0.98 \text{ kip-ft} \quad \text{(FK = 1.35)}
\]

\[
-M_u = 0.52 \text{ kip-ft} \quad -M_{u, \text{service}} = 0.39 \text{ kip-ft} \quad \text{(FK = 1.35)}
\]

**Resistance factor for tension-controlled section** \( \phi_{\text{STR}} \) = 0.90

**Positive Moment Capacity**
Depth of equivalent stress block \( a = \frac{A_s f_y}{0.85 f'_c b} \) = 0.192 in

Factored flexural resistance \(+\varphi M_n = \varphi A_s f_y \left( d_s - \frac{a}{2} \right) \) = 2.447 kip-ft

Check \(+\varphi M_n > +M_u \) 2.447 > 1.33 OK

**Negative Moment Capacity**
Depth of equivalent stress block \( a = \frac{A_s f_y}{0.85 f'_c b} \) = 0.192 in

Factored flexural resistance \(-\varphi M_n = \varphi A_s f_y \left( d_s(M^-) - \frac{a}{2} \right) \) = 0.974 kip-ft

Check \(-\varphi M_n > -M_u \) 0.974 > 0.52 OK
**Minimum Reinforcement**  
AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \(M_r = \varphi M_n\), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

\[
M_{cr} = \gamma_3 \left( \left( \gamma_1 f_r + \gamma_2 E_{cpe} \right) S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \right)
\]

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

where:

- flexural cracking variability factor \(\gamma_1 = 1.60\) (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength \(\gamma_3 = 0.67\) (A615 steel)
- concrete density modification factor \(\lambda = 1.00\) AASHTO 5.4.2.8

- modulus of rapture \(f_r = 0.24 \lambda \sqrt{f_c'}\)
- \(f_r = 0.48\) ksi AASHTO 5.4.2.6

- section modulus of design section \(S_c = \frac{bh^2}{6} = \frac{b t_{panel}^2}{6}\)
- \(S_c = 72.00\) in³

Cracking moment \(M_{cr} = \gamma_3 \gamma_1 f_r S_c\)

\(M_{cr} = 3.09\) kip-ft

check positive moment reinforcement:

\[
1.33 \times M_{u} + M_{cr} > \min(M_u, M_{cr})
\]

Check: \(+ \varphi M_n > \min(+M_u , M_{cr})\)

\(2.447 > 1.76\) OK

check negative moment reinforcement:

\[
1.33 \times M_{u} - M_{cr} > \min(-M_u, M_{cr})
\]

Check: \(- \varphi M_n > \min(-M_u , M_{cr})\)

\(0.974 > 0.69\) OK
Typical Panel Type A, Panel Thickness 6", 5'x10', Horizontal Rebar

Shear on Panel

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness</td>
<td>( t_{\text{panel}} = 6 ) in</td>
</tr>
<tr>
<td>Concrete Cover</td>
<td>( C_{\text{panel}} = 1.5 ) in</td>
</tr>
<tr>
<td>Concrete Strength</td>
<td>( f'_c = 4.0 ) ksi (Concrete Class A compressive strength)</td>
</tr>
<tr>
<td>Reinforcement Strength</td>
<td>( f_y = 60 ) ksi (Minimum yield strength of grade 60 steel)</td>
</tr>
<tr>
<td>Bar Size</td>
<td>#4</td>
</tr>
<tr>
<td>Rebar Diameter</td>
<td>( d_b = 0.500 ) in</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>( A_b = 0.196 ) in(^2) ( A_b = \frac{1}{4} \pi d_b^2 )</td>
</tr>
<tr>
<td>Rebar spacing</td>
<td>( s = 18 ) in</td>
</tr>
<tr>
<td>Width of the design section</td>
<td>( b = 12 ) in</td>
</tr>
<tr>
<td>Area of Steel per Design Strip</td>
<td>( A_s = 0.131 ) in(^2) ( A_s = b \left( \frac{A_b}{s} \right) )</td>
</tr>
<tr>
<td>( c )</td>
<td>( c = 0.226 ) in</td>
</tr>
<tr>
<td>( d_v )</td>
<td>( d_v = 4.137 ) in</td>
</tr>
</tbody>
</table>

AASHTO 5.7.3.3

\( V_n \) shall be the lesser of:

1. \( V_n = V_c + V_s \)
2. \( V_n = 0.25 \cdot f'_c \cdot b_v \cdot d_v \)

\( V_c = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f'_c \cdot b_v \cdot d_v} \)
\( V_c = 6.275 \) Kips

\( V_s = A_v \cdot f_y \cdot d_v \cdot \frac{(\cot \theta + \cot \alpha) \cdot \sin \alpha}{s} \)
\( V_s = 2.708 \) Kips

(1) \( V_n = 8.982 \) Kips
(2) \( V_n = 49.641 \) Kips

From (1) and (2) \( V_n \) should be:
\( \phi \cdot V_n = 0.9 \cdot V_n = 8.084 \) Kips

Shear Force:

From Staad Pro Result \( V_{\text{max}} = 0.131 \) N/mm\(^2\)
\( = 19.00 \) lbs/in\(^2\)

Shear Section Area = \( b \times t_{\text{panel}} \)
\( = 12 \times 6 \)

\( V_{\text{max}} = 1368.00 \) lbs
\( = 1.368 \) Kips \( \text{OK, } V_{\text{max}} \text{ less than } V_n \)
Typical Panel Type A, Panel Thickness 6", 5'x10', Vertical Rebar

Panel Strength Design
Design of panel reinforcement per
AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi.
This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

| Panel Thickness | t<sub>panel</sub> = 6 in |
| Finishing Thickness | C<sub>panel</sub> = 1.5 in |
| Additional Load from Finishes | = 0.0125 Kip/ft<sup>2</sup> |
| Concrete Cover | C<sub>b</sub> = 1.5 in |
| Vertical Bar Size | #4 |
| Rebar Diameter | d<sub>b</sub> = 0.5 in |
| Rebar Area | A<sub>b</sub> = 0.196 in<sup>2</sup> |
| Rebar spacing | s = 18 in |
| Width of the design section | b = 12 in |
| Effective depth of section | d<sub>s</sub> = 3.75 in |
| Effective depth of section for Negative Moment | d<sub>s(M)</sub> = 2.25 in |
| Correction factor for source aggregate | K<sub>s</sub> = 1 in |
| Concrete Density | W<sub>c</sub> = 0.15 kcf |
| Concrete Strength | f' <sub>c</sub> = 4.00 ksi |
| Reinforcement Strength | f<sub>y</sub> = 60 ksi |
| Modulus Elasticity of concrete | E<sub>c</sub> = 4266 ksi |
| Modulus Elasticity of reinforcement | E<sub>s</sub> = 29000 ksi |
| Modular Ratio | n = 6.798 |
| Area of Steel per Design Strip | A<sub>s</sub> = 0.131 in<sup>2</sup> |

\[ M_u = 1.32 \text{ kip-ft} \]
\[ M_{u,service} = 0.97 \text{ kip-ft (FK = 1.35)} \]
\[ M_u = 0.00 \text{ kip-ft} \]
\[ M_{u,service} = 0.00 \text{ kip-ft (FK = 1.35)} \]

Resistance factor for tension-controlled section \( \varphi_{STR} = 0.90 \) AASHTO 5.5.4.2

**Positive Moment Capacity**

Depth of equivalent stress block \( a = \frac{A_s f_y}{0.85 f' c b} = 0.192 \text{ in} \)

Factored flexural resistance \( +\varphi M_n = \varphi A_s f_y \left( d_s - \frac{a}{2} \right) = 2.152 \text{ kip-ft} \)

Check \( +\varphi M_n > +M_u \)

2.152 > 1.32 OK

**Negative Moment Capacity**

Depth of equivalent stress block \( a = \frac{A_s f_y}{0.85 f' c b} = 0.192 \text{ in} \)

Factored flexural resistance \( -\varphi M_n = \varphi A_s f_y \left( d_s(M) - \frac{a}{2} \right) = 1.269 \text{ kip-ft} \)

Check \( -\varphi M_n > -M_u \)

1.269 > 0.00 OK
**Minimum Reinforcement**

**AASHTO 5.7.3.3.2**

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r = \phi M_n \), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

\[
M_{cr} = \gamma_3 \left[ (\gamma_1 f_r + \gamma_2 f_{cpe}) S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \right]
\]

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

where:

- flexural cracking variability factor \( \gamma_1 = 1.60 \) (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength \( \gamma_3 = 0.67 \) (A615 steel)
- concrete density modification factor \( \lambda = 1.00 \) AASHTO 5.4.2.8

- modulus of rupture \( f_r = 0.24 \lambda \sqrt{f_c^2} \) \( f_r = 0.48 \) ksi AASHTO 5.4.2.6

- section modulus of design section \( S_c = \frac{b h^2}{6} = \frac{b t_{panel}}{6} \) \( S_c = 72.00 \) in^3

Cracking moment

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

\( M_{cr} = 3.09 \) kip-ft

Check positive moment reinforcement:

1.33 x factored ultimate moment \( = 1.75 \) kip-ft

Cracking moment \( = 3.09 \) kip-ft

min from (1.33 x factored +\( M_u \) and \( M_{cr} \)) \( = 1.75 \) kip-ft

Check:

\[
+\phi M_n > \min(+M_u, M_{cr})
\]

2.152 > 1.75 OK

Check negative moment reinforcement:

1.33 x factored ultimate moment \( = 0.00 \) kip-ft

Cracking moment \( = 3.09 \) kip-ft

min from (1.33 x factored -\( M_u \) and \( M_{cr} \)) \( = 0.00 \) kip-ft

Check:

\[
-\phi M_n > \min(-M_u, M_{cr})
\]

1.269 > 0.00 OK
**Typical Panel Type A, Panel Thickness 6", 5'x10', Vertical Rebar**

**Shear on Panel**

- **Panel Thickness**
  \[ t_{\text{panel}} = 6 \text{ in} \]
- **Concrete Cover**
  \[ c_{\text{panel}} = 2.5 \text{ in} \]
- **Concrete Strength**
  \[ f'_{c} = 4.0 \text{ ksi} \] (Concrete Class A compressive strength)
- **Reinforcement Strength**
  \[ f_{y} = 60 \text{ ksi} \] (Minimum yield strength of grade 60 steel)
- **Bar Size**
  \[ #4 \]
- **Rebar Diameter**
  \[ d_{b} = 0.500 \text{ in} \]
- **Rebar Area**
  \[ A_{b} = 0.196 \text{ in}^{2} \]
- **Rebar spacing**
  \[ s = 18 \text{ in} \]
- **Width of the design section**
  \[ b = 12 \text{ in} \]
- **Area of Steel per Design Strip**
  \[ A_{s} = 0.131 \text{ in}^{2} \]
  \[ A = \frac{A_{s}}{s} \]

\[
c = \frac{A_{s} \cdot f_{s}}{\alpha_{1} \cdot f'_{c} \cdot \beta_{1} \cdot b}
\]

\[
d_{v} = t_{\text{panel}} - c - c_{\text{panel}} - \frac{d_{b}}{2}
\]

**AASHTO 5.7.3.3**

Vn shall be the lesser of:

1. \[ V_{n} = V_{c} + V_{s} \]

\[
V_{c} = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f'_{c} \cdot b_{v} \cdot d_{v}}
\]

\[
V_{s} = \frac{A_{v} \cdot f_{y} \cdot d_{v} \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha}{s}
\]

(1) \[ V_{n} = 6.811 \text{ Kips} \]

(2) \[ V_{n} = 0.25 \cdot f'_{c} \cdot b_{v} \cdot d_{v} \]

(2) \[ V_{n} = 37.641 \text{ Kips} \]

From (1) and (2) Vn should be:

\[ \phi \cdot V_{n} = 0.9 \cdot V_{n} = 6.130 \text{ Kips} \]

**Shear Force:**

From Staad Pro Result Vmax

\[ V_{\text{max}} = 0.153 \text{ N/mm}^{2} = 22.19 \text{ lbs/in}^{2} \]

\[ \text{Shear Section Area} = b \times t_{\text{panel}} = 12 \times 6 \]

\[ V_{\text{max}} = 1597.74 \text{ lbs = 1.598 Kips} \] *OK, Vmax less than Vn*
5’ x 10’ STANDARD
X PANEL
**Typical Panel Type X, Panel Thickness 6", 5'x10', Horizontal Rebar**

**Panel Strength Design**
Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

<table>
<thead>
<tr>
<th>Panel Thickness</th>
<th>( t_{\text{panel}} )</th>
<th>6 in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Finishing Thickness</td>
<td>( C_{\text{panel}} )</td>
<td>1.5 in</td>
</tr>
<tr>
<td>Additional Load from Finishes</td>
<td>( A_{\text{fb}} )</td>
<td>0.0125 Kip/ft²</td>
</tr>
<tr>
<td>Concrete Cover</td>
<td>( C_{\text{panel}} )</td>
<td>1.5 in</td>
</tr>
<tr>
<td>Vertical Bar Size</td>
<td>#4</td>
<td></td>
</tr>
<tr>
<td>Rebar Diameter</td>
<td>( d_b )</td>
<td>0.5 in</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>( A_b )</td>
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</tr>
<tr>
<td>Rebar spacing</td>
<td>( s )</td>
<td>12 in</td>
</tr>
<tr>
<td>Width of the design section</td>
<td>( b )</td>
<td>12 in</td>
</tr>
<tr>
<td>Effective depth of section</td>
<td>( d_s )</td>
<td>4.25 in</td>
</tr>
<tr>
<td>Effective depth of section for Negative Moment</td>
<td>( d_{s(M^-)} )</td>
<td>1.75 in</td>
</tr>
<tr>
<td>Correction factor for source aggregate</td>
<td>( K_s )</td>
<td>1 in</td>
</tr>
<tr>
<td>Concrete Density</td>
<td>( W_{c} )</td>
<td>0.15 kcf</td>
</tr>
<tr>
<td>Concrete Strength</td>
<td>( f'_{c} )</td>
<td>4.0 ksi (Concrete Class A compressive strength)</td>
</tr>
<tr>
<td>Reinforcement Strength</td>
<td>( f_y )</td>
<td>60 ksi (Minimum yield strength of grade 60 steel)</td>
</tr>
<tr>
<td>Modulus Elasticity of concrete</td>
<td>( E_c )</td>
<td>4266 ksi</td>
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<td>Modulus Elasticity of reinforcement</td>
<td>( E_s )</td>
<td>29000 ksi</td>
</tr>
<tr>
<td>Modular Ratio</td>
<td>( n )</td>
<td>6.798</td>
</tr>
<tr>
<td>Area of Steel per Design Strip</td>
<td>( A_s )</td>
<td>0.196 in²</td>
</tr>
</tbody>
</table>

\[ +M_u = 2.63 \text{ kip-ft} \]  
\[ -M_u = 1.02 \text{ kip-ft} \]

Factored flexural resistance

- Positive Moment Capacity
  \[ +M_n = \phi A_s f_y \left( d_s - \frac{a}{2} \right) \]  
  \[ a = \frac{A_s f_y}{0.85 f'_{c} b} = 0.289 \text{ in} \]  
  \[ +\phi M_n > +M_u \text{ (OK)} \]

- Negative Moment Capacity
  \[ -M_n = \phi A_s f_y \left( d_{s(M^-)} - \frac{a}{2} \right) \]  
  \[ a = \frac{A_s f_y}{0.85 f'_{c} b} = 0.289 \text{ in} \]  
  \[ -\phi M_n > -M_u \text{ (OK)} \]

AASHTO 5.5.4.2
**Minimum Reinforcement**

* AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r = \phi M_n \), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

\[
M_{cr} = \gamma_3 \left( \gamma_1 f_r + \gamma_2 y_{cpe} S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \right)
\]

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

where:

- flexural cracking variability factor \( \gamma_1 = 1.60 \) (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength \( \gamma_3 = 0.67 \) (A615 steel)
- concrete density modification factor \( \lambda = 1.00 \) AASHTO 5.4.2.8
- modulus of rupture \( f_r = 0.24 \lambda \sqrt{f_c} \)
- section modulus of design section \( S_c = \frac{b h^2}{6} = \frac{b t_{panel}^2}{6} \)
  \( S_c = 72.00 \text{ in}^3 \)

Cracking moment

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

\( M_{cr} = 3.09 \text{ kip-ft} \)

Check positive moment reinforcement:

- 1.33 x factored ultimate moment = 3.50 kip-ft
- Cracking moment = 3.09 kip-ft
- min from (1.33 x factored +\( M_u \) and \( M_{cr} \)) = 3.09 kip-ft

Check: \( +\phi M_n > \min( +M_u , M_{cr} ) \)

\( 3.628 > 3.09 \text{ OK} \)

Check negative moment reinforcement:

- 1.33 x factored ultimate moment = 1.36 kip-ft
- Cracking moment = 3.09 kip-ft
- min from (1.33 x factored -\( M_u \) and \( M_{cr} \)) = 1.36 kip-ft

Check: \( -\phi M_n > \min( -M_u , M_{cr} ) \)

\( 1.419 > 1.36 \text{ OK} \)
Typical Panel Type X, Panel Thickness 6", 5'x10', Horizontal Rebar

### Shear on Panel

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
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<tbody>
<tr>
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<td>1.5 in</td>
</tr>
<tr>
<td>Concrete Strength</td>
<td>( f'_c )</td>
<td>4.0 ksi</td>
</tr>
<tr>
<td>Reinforcement Strength</td>
<td>( f_y )</td>
<td>60 ksi</td>
</tr>
<tr>
<td>Bar Size</td>
<td>#4</td>
<td></td>
</tr>
<tr>
<td>Rebar Diameter</td>
<td>( d_b )</td>
<td>0.500 in</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>( A_b )</td>
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</tr>
<tr>
<td>Rebar spacing</td>
<td>( s )</td>
<td>12 in</td>
</tr>
<tr>
<td>Width of the design section</td>
<td>( b )</td>
<td>12 in</td>
</tr>
<tr>
<td>Area of Steel per Design Strip</td>
<td>( A_s )</td>
<td>0.196 in²</td>
</tr>
</tbody>
</table>

\[
c = \frac{A_s \cdot f_y}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b}
\]
\[
d_v = t_{\text{panel}} - \frac{c}{2} - C_{\text{panel}} - \frac{d_b}{2}
\]

\[
c = 0.340 \text{ in}
\]
\[
d_v = 4.080 \text{ in}
\]

### AASHTO 5.7.3.3

\( V_n \) shall be the lesser of:

1. \( V_n = V_c + V_s \)

\[
V_c = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f'_c \cdot b_v \cdot d_v}
\]

\[
V_s = A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha
\]

\[
V_c = 6.189 \text{ Kips}
\]
\[
V_s = 4.006 \text{ Kips}
\]

\( V_n = 10.194 \text{ Kips} \)

2. \( V_n = 0.25 \cdot f'_c \cdot b_v \cdot d_v \)

\( V_n = 48.962 \text{ Kips} \)

From (1) and (2) \( V_n \) should be:

\[
\phi \cdot V_n = 9.175 \text{ Kips}
\]

\( \phi \cdot V_n = 0.9 \cdot V_n \)

**Shear Force:**

From Staad Pro Result \( V_{\text{max}} = 37.71 \text{ lbs/in}^2 \)

\[
V_{\text{max}} = 2715.11 \text{ lbs}
\]

\( V_{\text{max}} = 2.715 \text{ Kips} \)

OK, \( V_{\text{max}} \) less than \( V_n \)
**Typical Panel Type X, Panel Thickness 6", 5'x10', Vertical Rebar**

**Panel Strength Design**
Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

Panel Thickness \( t_{panel} = 6 \) in
Finishing Thickness \( C_{panel} = 1.5 \) in
Additional Load from Finishes \( = 0.0125 \) Kip/ft²

Vertical Bar Size \#4
Rebar Diameter \( d_b = 0.5 \) in \( d_{bh} = 0.5 \) in
Rebar Area \( A_b = 0.196 \) in² \( A_p = \frac{1}{4} \pi d_b^2 \)
Rebar spacing \( s = 12 \) in
Width of the design section \( b = 12 \) in
Effective depth of section \( d_s = 3.75 \) in \( d_s = t_{panel} - C_{panel} - d_{bh} - \frac{1}{2} d_b \)
Effective depth of section for Negative Moment \( d_{s(M^-)} = 2.25 \) in \( d_{s(M^-)} = t_{panel} - d_s \)
Correction factor for source aggregate \( K_s = 1 \) in AASHTO 5.4.2.4
Concrete Density \( W_c = 0.15 \) kcf
Concrete Strength \( f'c = 4.00 \) ksi (Concrete Class A compressive strength)
Reinforcement Strength \( f_y = 60 \) ksi (Minimum yield strength of grade 60 steel)
Modulus Elasticity of concrete \( E_c = 4266 \) ksi
Modulus Elasticity of reinforcement \( E_s = 29000 \) ksi
Modular Ratio \( n = 6.798 \) \( n = E_s/E_c \)
Area of Steel per Design Strip \( A_s = 0.196 \) in² \( A_s = b(\frac{A_b}{s}) \)

\[ +M_u = 2.29 \] kip-ft \[ +M_{u\text{,service}} = 1.70 \] kip-ft (FK = 1.35)
\[ -M_u = 0.00 \] kip-ft \[ -M_{u\text{,service}} = 0.00 \] kip-ft (FK = 1.35)

Resistance factor for tension-controlled section \( \phi_{STR} = 0.90 \) AASHTO 5.5.4.2

**Positive Moment Capacity**
Depth of equivalent stress block \( a = \frac{A_s f_y}{0.85 f'c b} = 0.289 \) in
Factored flexural resistance \[ +\varphi M_n = \varphi A_s f_y \left( d_s - \frac{a}{2} \right) \] = 3.186 kip-ft
Check \[ +\varphi M_n > +M_u \] \( 3.186 > 2.29 \) OK

**Negative Moment Capacity**
Depth of equivalent stress block \( a = \frac{A_s f_y}{0.85 f'c b} = 0.289 \) in
Factored flexural resistance \[ -\varphi M_n = \varphi A_s f_y \left( d_{s(M^-)} - \frac{a}{2} \right) \] = 1.860 kip-ft
Check \[ -\varphi M_n > -M_u \] \( 1.860 > 0.00 \) OK
**Minimum Reinforcement**

AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r = \varphi M_n \), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

\[
M_{cr} = \gamma_3 \left( \gamma_1 f_r + \gamma_2 \gamma_{cpe} S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \right)
\]

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

where:

- flexural cracking variability factor \( \gamma_1 = 1.60 \) (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength \( \gamma_3 = 0.67 \) (A615 steel)
- concrete density modification factor \( \lambda = 1.00 \) AASHTO 5.4.2.8

- modulus of rapture \( f_r = 0.24 \lambda \sqrt{f_c'} \) \( f_r = 0.48 \) ksi AASHTO 5.4.2.6

- section modulus of design section \( S_c = \frac{bh^2}{6} = \frac{bt_{panel}^2}{6} \) \( S_c = 72.00 \) in\(^3\)

Cracking moment

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

\( M_{cr} = 3.09 \) kip-ft

check positive moment reinforcement:

1.33 x factored ultimate moment

\( = 3.05 \) kip-ft

Cracking moment

\( = 3.09 \) kip-ft

min from (1.33 x factored +\( M_u \) and \( M_{cr} \))

\( = 3.05 \) kip-ft

Check: \( +\varphi M_n > \min( +M_u , M_{cr}) \)

\( 3.186 > 3.05 \) OK

check negative moment reinforcement:

1.33 x factored ultimate moment

\( = 0.00 \) kip-ft

Cracking moment

\( = 3.09 \) kip-ft

min from (1.33 x factored -\( M_u \) and \( M_{cr} \))

\( = 0.00 \) kip-ft

Check: \( -\varphi M_n > \min( -M_u , M_{cr}) \)

\( 1.860 > 0.00 \) OK
Typical Panel Type X, Panel Thickness 6", 5'x10', Vertical Rebar

**Shear on Panel**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness</td>
<td>$t_{\text{panel}} = 6 \text{ in}$</td>
</tr>
<tr>
<td>Concrete Cover</td>
<td>$C_{\text{panel}} = 2.5 \text{ in}$</td>
</tr>
<tr>
<td>Concrete Strength</td>
<td>$f'_{c} = 4.0 \text{ ksi}$ (Concrete Class A compressive strength)</td>
</tr>
<tr>
<td>Reinforcement Strength</td>
<td>$f_{y} = 60 \text{ ksi}$ (Minimum yield strength of grade 60 steel)</td>
</tr>
<tr>
<td>Bar Size</td>
<td>#4</td>
</tr>
<tr>
<td>Rebar Diameter</td>
<td>$d_{b} = 0.500 \text{ in}$</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>$A_{b} = 0.196 \text{ in}^2$</td>
</tr>
<tr>
<td>Rebar spacing</td>
<td>$s = 12 \text{ in}$</td>
</tr>
<tr>
<td>Width of the design section</td>
<td>$b = 12 \text{ in}$</td>
</tr>
<tr>
<td>Area of Steel per Design Strip</td>
<td>$A_{s} = 0.196 \text{ in}^2$</td>
</tr>
</tbody>
</table>

where $A_b = \frac{1}{4} \pi d_b^2$

- $c = \frac{A_s . f_s}{\alpha_1 . f'_c . \beta_1 . b}$
- $d_v = t_{\text{panel}} - \frac{c}{2} - C_{\text{panel}} - \frac{d_b}{2}$
- $d_v = 3.080 \text{ in}$

**AASHTO 5.7.3.3**

$V_n$ shall be the lesser of:

1. $V_n = V_c + V_s$

$$
V_c = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f'_c} \cdot b_v \cdot d_v \\
V_c = 4.672 \text{ Kips}
$$

$$
V_s = \frac{A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha}{s} \\
V_s = 3.024 \text{ Kips}
$$

(1) $V_n = 7.696 \text{ Kips}$

2. $V_n = 0.25 \cdot f'_c \cdot b_v \cdot d_v$

(2) $V_n = 36.962 \text{ Kips}$

From (1) and (2) $V_n$ should be:

$$
\phi \cdot V_n = 0.9 \cdot V_n \\
\phi \cdot V_n = 6.926 \text{ Kips}
$$

**Shear Force:**

From Staad Pro Result $V_{\text{max}} = 0.303 \text{ N/mm}^2$

$= 43.95 \text{ lbs/in}^2$

Shear Section Area

$$
= b \times t_{\text{panel}} \\
= 12 \times 6
$$

$V_{\text{max}} = 3164.15 \text{ lbs}$

$= 3.164 \text{ Kips}$

OK, $V_{\text{max}}$ less than $V_n$
5’ x 10’ STANDARD
Y PANEL
**Typical Panel Type Y, Panel Thickness 6”, 5’x10’, Horizontal Rebar**

**Panel Strength Design**

Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi.

This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

<table>
<thead>
<tr>
<th>Panel Thickness</th>
<th>$t_{\text{panel}}$</th>
<th>6 in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Finishing Thickness</td>
<td>$C_{\text{panel}}$</td>
<td>1.5 in</td>
</tr>
<tr>
<td>Additional Load from Finishes</td>
<td>$f_{\text{c}}$</td>
<td>0.0125 Kip/ft$^2$</td>
</tr>
<tr>
<td>Concrete Cover</td>
<td>$C_{\text{c}}$</td>
<td>1.5 in</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Vertical Bar Size</th>
<th>#4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rebar Diameter</td>
<td>$d_b$</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>$A_b$</td>
</tr>
<tr>
<td>Rebar spacing</td>
<td>$s$</td>
</tr>
<tr>
<td>Width of the design section</td>
<td>$b$</td>
</tr>
<tr>
<td>Effective depth of section</td>
<td>$d_s$</td>
</tr>
<tr>
<td>Effective depth of section for Negative Moment</td>
<td>$d_{s(M^-)}$</td>
</tr>
<tr>
<td>Correction factor for source aggregate</td>
<td>$K_1$</td>
</tr>
</tbody>
</table>

| Concrete Density | $W_c$ | 0.15 kcf |
| Reinforcement Strength | $f_y$ | 60 ksi |
| Modulus Elasticity of concrete | $E_c$ | 4266 ksi |
| Modulus Elasticity of reinforcement | $E_s$ | 29000 ksi |
| Modular Ratio | $n$ | 6.798 |
| Area of Steel per Design Strip | $A_s$ | 0.236 in$^2$ |

$$+M_u = 2.99 \text{ kip-ft}$$
$$-M_u = 1.25 \text{ kip-ft}$$

$$A = b \left( \frac{A_s}{n} \right)$$

**Resistance factor for tension-controlled section**

$$\varphi_{\text{TR}} = 0.90$$

**Positive Moment Capacity**

Depth of equivalent stress block

$$a = \frac{A_s f_y}{0.85 f_c^\prime b} \approx 0.346 \text{ in}$$

Factored flexural resistance

$$+\varphi M_n = \varphi A_s f_y \left( d_s - \frac{a}{2} \right) = 4.323 \text{ kip-ft}$$

Check

$$+\varphi M_n > +M_u \quad 4.323 > 2.99 \quad \text{OK}$$

**Negative Moment Capacity**

Depth of equivalent stress block

$$a = \frac{A_s f_y}{0.85 f_c^\prime b} \approx 0.346 \text{ in}$$

Factored flexural resistance

$$-\varphi M_n = \varphi A_s f_y \left( d_{s(M^-)} - \frac{a}{2} \right) = 1.672 \text{ kip-ft}$$

Check

$$-\varphi M_n > -M_u \quad 1.672 > 1.25 \quad \text{OK}$$
**Minimum Reinforcement**

AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r = \phi M_n \), at least equal to the lesser of:

- 1.33 times the positive factored ultimate moment
- Cracking moment

Cracking moment

\[
M_{cr} = \gamma_3 \left( \gamma_1 f_r + \gamma_2 f_{cpe} \right) S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right)
\]

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

where:

- flexural cracking variability factor \( \gamma_1 = 1.60 \) (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength \( \gamma_3 = 0.67 \) (A615 steel)
- concrete density modification factor \( \lambda = 1.00 \)

- modulus of rapture \( f_r = 0.24 \lambda \sqrt{f'_c} \)
- section modulus of design section \( S_c = \frac{b \ h^2}{6} = \frac{b \ t \ panel^2}{6} \)
  \( S_c = 72.00 \) in\(^3\)

Cracking moment

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

\( M_{cr} = 3.09 \) kip-ft

check positive moment reinforcement:

1.33 x factored ultimate moment  \( = 3.98 \) kip-ft
Cracking moment  \( = 3.09 \) kip-ft
min from (1.33 x factored +\( M_u \) and \( M_{cr} \))  \( = 3.09 \) kip-ft

Check:

\[ +\phi M_n > \min(+M_u, M_{cr}) \]

4.323 > 3.09 OK

check negative moment reinforcement:

1.33 x factored ultimate moment  \( = 1.67 \) kip-ft
Cracking moment  \( = 3.09 \) kip-ft
min from (1.33 x factored -\( M_u \) and \( M_{cr} \))  \( = 1.67 \) kip-ft

Check:

\[ -\phi M_n > \min(-M_u, M_{cr}) \]

1.672 > 1.67 OK
Typical Panel Type Y, Panel Thickness 6”, 5’x10’, Horizontal Rebar

**Shear on Panel**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness ( t_{\text{panel}} )</td>
<td>6 in</td>
</tr>
<tr>
<td>Concrete Cover ( C_{\text{panel}} )</td>
<td>1.5 in</td>
</tr>
<tr>
<td>Concrete Strength ( f'_c )</td>
<td>4.0 ksi</td>
</tr>
<tr>
<td>Reinforcement Strength ( f_y )</td>
<td>60 ksi</td>
</tr>
<tr>
<td>Bar Size #4</td>
<td></td>
</tr>
<tr>
<td>Rebar Diameter ( d_b )</td>
<td>0.500 in</td>
</tr>
<tr>
<td>Rebar Area ( A_b )</td>
<td>0.196 in²</td>
</tr>
<tr>
<td>Rebar spacing ( s )</td>
<td>10 in</td>
</tr>
<tr>
<td>Width of the design section</td>
<td>12 in</td>
</tr>
<tr>
<td>Area of Steel per Design Strip ( A_s )</td>
<td>0.236 in²</td>
</tr>
</tbody>
</table>

\[
c = \frac{A_s \cdot f_s}{\alpha f'_c \cdot \beta_1 \cdot b} \quad c = 0.408 \text{ in}
\]

\[
d_v = t_{\text{panel}} \cdot \frac{c}{2} - C_{\text{panel}} \cdot \frac{d_b}{2} \quad d_v = 4.046 \text{ in}
\]

**AASHTO 5.7.3.3**

\( V_n \) shall be the lesser of:

1. \( V_n = V_c + V_s \)
   \[
   V_c = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f'_c \cdot b_v \cdot d_v} \quad V_c = 6.137 \text{ Kips}
   \]
   \[
   V_s = A_v \cdot f_y \cdot d_v \cdot \left( \cot \theta + \cot \alpha \right) \cdot \sin \alpha \quad V_s = 4.767 \text{ Kips}
   \]

2. \( V_n = 0.25 \cdot f'_c \cdot b_v \cdot d_v \)
   \( V_n = 48.554 \text{ Kips} \)

From (1) and (2) \( V_n \) should be:

\( \phi \cdot V_n = 0.9 \cdot V_n = 9.814 \text{ Kips} \)

**Shear Force:**

\[
\text{From Staad Pro Result } V_{\text{max}} = 0.319 \text{ N/mm}^2 = 46.27 \text{ lbs/in}^2
\]

\[
\text{Shear Section Area } = b \times t_{\text{panel}} = 12 \times 6
\]

\[
V_{\text{max}} = 3331.23 \text{ lbs} = 3.331 \text{ Kips} \quad \text{OK, } V_{\text{max}} \text{ less than } V_n
Typical Panel Type Y, Panel Thickness 6”, 5'x10', Vertical Rebar

Panel Strength Design
Design of panel reinforcement per
AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

Panel Thickness \( t_{\text{panel}} = 6 \) in
Finishing Thickness \( c_{\text{panel}} = 1 \) in
Additional Load from Finishes \( = 0.0125 \text{ Kip/ft}^2 \)
Concrete Cover \( C_{\text{panel}} = 1.5 \) in

Vertical Bar Size \#4
Rebar Diameter \( d_b = 0.5 \) in \( d_{bh} = 0.5 \) in
Rebar Area \( A_b = 0.196 \text{ in}^2 \)
Rebar spacing \( s = 12 \) in
Width of the design section \( b = 12 \) in
Effective depth of section \( d_s = 3.75 \) in \( d_s(M) = t_{\text{panel}} - C_{\text{panel}} - d_{bh} - \frac{1}{2} d_b \)
Effective depth of section for Negative Moment \( d_{s(M^-)} = 2.25 \) in \( d_{s(M^-)} = t_{\text{panel}} - d_s \)
Correction factor for source aggregate \( K_1 = 1 \) in AASHTO 5.4.2.4
Concrete Density \( W_c = 0.15 \text{ kcf} \)
Concrete Strength \( f'_c = 4.00 \text{ ksi} \) (Concrete Class A compressive strength)
Reinforcement Strength \( f_y = 60 \text{ ksi} \) (Minimum yield strength of grade 60 steel)
Modulus Elasticity of concrete \( E_c = 4266 \text{ ksi} \)
Modulus Elasticity of reinforcement \( E_s = 29000 \text{ ksi} \)
Modular Ratio \( n = 6.798 \)
Area of Steel per Design Strip \( A_s = 0.196 \text{ in}^2 \) \( A_s = b \left( \frac{A_b}{s} \right) \)

\[ +M_u = 2.81 \text{ kip-ft} \]
\[ -M_u = 0.00 \text{ kip-ft} \]

\[ +M_{u_{\text{service}}} = 2.08 \text{ kip-ft} \quad \text{(FK = 1.35)} \]
\[ -M_{u_{\text{service}}} = 0.00 \text{ kip-ft} \quad \text{(FK = 1.35)} \]

Resistance factor for tension-controlled section \( \psi_{\text{STR}} = 0.90 \) AASHTO 5.5.4.2

Positive Moment Capacity
Depth of equivalent stress block \( a = \frac{A_s f_y}{0.85 f'_c b} = 0.289 \text{ in} \)
Factored flexural resistance \( +\varphi M_n = \varphi A_s f_y \left( d_s - \frac{a}{2} \right) = 3.186 \text{ kip-ft} \)
Check \( +\varphi M_n > +M_u \)

Negative Moment Capacity
Depth of equivalent stress block \( a = \frac{A_s f_y}{0.85 f'_c b} = 0.289 \text{ in} \)
Factored flexural resistance \( -\varphi M_n = \varphi A_s f_y \left( d_{s(M^-)} - \frac{a}{2} \right) = 1.860 \text{ kip-ft} \)
Check \( -\varphi M_n > -M_u \)
**Minimum Reinforcement**  
AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( \text{Mr} = \phi \text{Mn} \), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

\[
M_{cr} = \gamma_3 \left( \gamma_1 f_t + \gamma_2 y_{cpe} \right) S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right)
\]

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non-composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_t S_c
\]

where:

- flexural cracking variability factor \( \gamma_1 = 1.60 \) (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength \( \gamma_3 = 0.67 \) (A615 steel)
- concrete density modification factor \( \lambda = 1.00 \) AASHTO 5.4.2.8
- modulus of rupture \( f_r = 0.24 \lambda \sqrt{f_c^t} \)
- \( f_r = 0.48 \) ksi AASHTO 5.4.2.6
- section modulus of design section \( S_c = \frac{bh^2}{6} = \frac{b t_{panel}^2}{6} \)
- \( S_c = 72.00 \) in\(^3\)

Cracking moment

\[
M_{cr} = \gamma_3 \gamma_1 f_t S_c
\]

\( M_{cr} = 3.09 \) kip-ft

check positive moment reinforcement:

1.33 x factored ultimate moment

\( 1.33 \times \text{factored ultimate moment} = 3.74 \) kip-ft

Cracking moment

\( M_{cr} = 3.09 \) kip-ft

min from (1.33 x factored +M\(_u\) and M\(_{cr}\))

\( M_{cr} = 3.09 \) kip-ft

Check:

\[
+ \phi M_n > \min(+M_u, M_{cr})
\]

\( 3.186 > 3.09 \) OK

check negative moment reinforcement:

1.33 x factored ultimate moment

\( 1.33 \times \text{factored ultimate moment} = 0.00 \) kip-ft

Cracking moment

\( M_{cr} = 3.09 \) kip-ft

min from (1.33 x factored -M\(_u\) and M\(_{cr}\))

\( M_{cr} = 0.00 \) kip-ft

Check:

\[
- \phi M_n > \min(-M_u, M_{cr})
\]

\( 1.860 > 0.00 \) OK
Typical Panel Type Y, Panel Thickness 6”, 5’x10’, Vertical Rebar

**Shear on Panel**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness</td>
<td>$t_{panel}$ = 6 in</td>
</tr>
<tr>
<td>Concrete Cover</td>
<td>$C_{panel}$ = 2.5 in</td>
</tr>
<tr>
<td>Concrete Strength</td>
<td>$f'_c$ = 4.0 ksi (Concrete Class A compressive strength)</td>
</tr>
<tr>
<td>Reinforcement Strength</td>
<td>$f_y$ = 60 ksi (Minimum yield strength of grade 60 steel)</td>
</tr>
<tr>
<td>Bar Size</td>
<td>#4</td>
</tr>
<tr>
<td>Rebar Diameter</td>
<td>$d_b$ = 0.500 in</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>$A_b$ = 0.196 in$^2$</td>
</tr>
<tr>
<td>Rebar spacing</td>
<td>$s$ = 12 in</td>
</tr>
<tr>
<td>Width of the design section</td>
<td>$b$ = 12 in</td>
</tr>
<tr>
<td>Area of Steel per Design Strip</td>
<td>$A_s$ = 0.196 in$^2$</td>
</tr>
</tbody>
</table>

**AASHTO 5.7.3.3**

$V_n$ shall be the lesser of:

1. $V_n = V_c + V_s$

$$V_c = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f'_c \cdot b_v \cdot d_v}$$

$$V_c = 4.672 \text{ Kips}$$

$$V_s = \frac{A_s \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha}{s}$$

$$V_s = 3.024 \text{ Kips}$$

(1) $V_n = 7.696 \text{ Kips}$

2. $V_n = 0.25 \cdot f'_c \cdot b_v \cdot d_v$

$$V_n = 36.962 \text{ Kips}$$

From (1) and (2) $V_n$ should be:

$$\phi \cdot V_n = 0.9 \cdot V_n$$

$$\phi \cdot V_n = 6.926 \text{ Kips}$$

**Shear Force:**

From Staad Pro Result

$$V_{max} = 0.372 \text{ N/mm}^2$$

$$= 53.95 \text{ lbs/in}^2$$

Shear Section Area = $b \times t_{panel}$

= 12 x 6

$$V_{max} = 3884.70 \text{ lbs}$$

= 3.885 $\text{Kips}$ OK, Vmax less than $V_n$
5’ x 12’ STANDARD PANELS
STANDARD "A" PANEL
SHOWN FROM BACK FACE

SECTION A-A

NOTE:
TYPICAL PANEL REINFORCEMENT SHOWN. ALTERNATIVE
REINFORCEMENT MAY BE USED AS LONG AS MINIMUM
CROSS SECTIONAL AREAS ARE EQUIVALENT.
18" MAX SPACING BETWEEN BARS

CERTIFIED ONLY WITH RESPECT
TO INTERNAL STABILITY OF
REINFORCED CONCRETE STRUCTURES

TYPE "A" 5'X12' PANEL DETAILS

MISE PLUS WALL SYSTEM
BRIE F EVALUATION

PLAN BY KEVIN NAKATANI
ENGINEER

SOUTHERN CALIFORNIA RESOURCES
2355 W. CAMERON AVE. SUITE 204
IRVINE, CA 92604
PHONE: (949) 457-0817
FAX: (949) 457-0818

DATE: 7-24-03

NOTES:
1. PANELS MUST BE SHIPPED IN 4'X12' OR 5'X12' CUTS.
2. PANELS ARE TO BE SHIPPED WITH JOINTS ON SHORTER EDGE.
3. PANELS MUST BE SHIPPED WITH REINFORCEMENT IN PLACE.
4. PANELS MUST BE SHIPPED WITH JOINTS CUT TO MATCH.
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99. PANELS MUST BE SHIPPED WITH JOINTS CUT TO MATCH.
100. PANELS MUST BE SHIPPED WITH JOINTS CUT TO MATCH.
STANDARD "Y" PANEL
SHOWN FROM FACE FACE

SECTION A-A

NOTE:
TYPICAL PANEL REINFORCEMENT SHOWN. ALTERNATIVE
REINFORCEMENT MAY BE USED AS LONG AS MINIMUM
CROSS SECTIONAL AREAS ARE EQUIVALENT.
18" MAX SPACING BETWEEN BARS
<table>
<thead>
<tr>
<th>Wire</th>
<th>Area sqin</th>
<th>Diameter in</th>
<th>Diameter after microns</th>
<th>Diameter after microns</th>
<th>% effective</th>
<th>Area after sqin</th>
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<td>9684.292</td>
<td>0.3813</td>
<td>0.7611</td>
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<td>0.2283</td>
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<td>12817.513</td>
<td>11401.513</td>
<td>0.4489</td>
<td>0.7913</td>
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<td>0.4748</td>
<td>0.6330</td>
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<td></td>
<td></td>
<td>0.7913</td>
<td>0.9495</td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td>1.1078</td>
<td>1.2660</td>
</tr>
<tr>
<td>W24</td>
<td>0.24</td>
<td>0.553</td>
<td>14040.882</td>
<td>12624.882</td>
<td>0.4970</td>
<td>0.8085</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.1940</td>
<td>0.3881</td>
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<td></td>
<td>0.5821</td>
<td>0.7761</td>
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<tr>
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<td></td>
<td></td>
<td>0.9702</td>
<td>1.1642</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.3582</td>
<td>1.5523</td>
</tr>
<tr>
<td>W30</td>
<td>0.30</td>
<td>0.618</td>
<td>15698.184</td>
<td>14282.184</td>
<td>0.5623</td>
<td>0.8277</td>
</tr>
<tr>
<td></td>
<td></td>
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<td></td>
<td></td>
<td>0.2483</td>
<td></td>
</tr>
</tbody>
</table>
SPRING CONSTANT
**Modulus of Sub-Grade Reaction and Spring Constant Check**

From Principles of Foundation Engineering; Braja M. Das 8th Edition:

### Table 8.2 Typical Subgrade Reaction Values, $k_{0.3}(k_1)$

<table>
<thead>
<tr>
<th>Soil type</th>
<th>MN/m³</th>
<th>lb/in²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry or moist sand:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>8–25</td>
<td>30–90</td>
</tr>
<tr>
<td>Medium</td>
<td>25–125</td>
<td>90–450</td>
</tr>
<tr>
<td>Dense</td>
<td>125–375</td>
<td>450–1350</td>
</tr>
<tr>
<td>Saturated sand:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>10–15</td>
<td>35–55</td>
</tr>
<tr>
<td>Medium</td>
<td>35–40</td>
<td>125–145</td>
</tr>
<tr>
<td>Dense</td>
<td>130–150</td>
<td>475–550</td>
</tr>
<tr>
<td>Clay:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stiff</td>
<td>10–25</td>
<td>40–90</td>
</tr>
<tr>
<td>Very stiff</td>
<td>25–50</td>
<td>90–185</td>
</tr>
<tr>
<td>Hard</td>
<td>&gt;50</td>
<td>&gt;185</td>
</tr>
</tbody>
</table>

\[
K_{0.3}(k_1) = \frac{1350}{k_1} \text{ lb/in}^3 \quad (\text{is selected})
\]

(dense Sand, value is conservative considering no reinforcement is included)

Adjust to panel 5' x 10' (1.5m x 3m)

**Foundations on Sandy Soils**

For foundations on sandy soils,

\[
k = k_{0.3} \left( \frac{B + 0.3}{2B} \right)^2
\]

where $k_{0.3}$ and $k = \text{coefficients of subgrade reaction of foundations measuring } 0.5 \text{ m} \times 0.3 \text{ m} \text{ and } B \text{ (m) } \times B \text{ (m)}, \text{ respectively (unit is KN/m}^3\text{)}. \text{ In English units, Eq. (8.45) may be expressed as}

\[
k = \frac{k_{0.3}(B + 1)^2}{2B}
\]

where $k_{1.5}$ and $k = \text{coefficients of subgrade reaction of foundations measuring } 1 \text{ ft} \times 1 \text{ ft} \text{ and } B \text{ (ft) } \times B \text{ (ft)}, \text{ respectively (unit is lb/in}^3\text{)}$.

\[
k_{1.5} = k_{0.3} \cdot \left( \frac{B + 0.3}{2B} \right)^2
\]

\[
k_{1.5} = 1350 \times \left( \frac{1.5 + 0.3}{2 \times 1.5} \right)^2
\]

\[
k_{1.5} = 486 \text{ lb/in}^3
\]

\[
k_{1.5x3.0} = k_{1.5} \cdot \left( \frac{1 + 3.0}{1.5} \right)^2
\]

\[
k_{1.5x3.0} = 486 \text{ lb/in}^3
\]

\[
k_{1.5x3.0} = 839.755 \text{ kip/ft}^3
\]
10-5 MODULUS OF SUBGRADE REACTION $k_s$
FOR MATS AND PLATES

All three discrete element methods given in this chapter for mats/plates use the modulus of subgrade reaction $k_s$ to support the plate. The modulus $k_s$ is used to compute node springs based on the contributing plan area of an element to any node as in Fig. 10-5. From the figure we see the following:

<table>
<thead>
<tr>
<th>Node</th>
<th>Contributing area</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (corner)</td>
<td>$\frac{1}{4}$ of rectangle abde</td>
</tr>
<tr>
<td>2 (side)</td>
<td>$\frac{1}{2}$ of abde + $\frac{1}{2}$ of bcef</td>
</tr>
<tr>
<td>3 (interior)</td>
<td>$\frac{1}{4}$ of each rectangle framing to a common node (as node 3)</td>
</tr>
</tbody>
</table>

For a triangle one should arbitrarily use one-third of the triangle area to any corner node. For these area contributions the fraction of $k_s$ node resistance from any element is

$$K_i = k_s \times \text{Area, m}^2 = \text{units of kN/m (or kips/ft in Fps)}$$

Since this computation gives units of a “spring” it is common to call the effect a node spring.

In this form the springs are independent of each other, the system of springs supporting the plate is termed a “Winkler” foundation, and the springs are uncoupled. Uncoupling means that the deflection of any spring is not influenced by adjacent springs.

**Spring Constant**

Note Spacing: 1ft x 1ft
Tributary Area: 1' x 1' = 1SF

$$\text{Ka} = \text{k x a}$$

$$\text{Ka} = 839.755 \text{ kip/ft} \quad \text{ (Input to Staad Pro)}$$
MAXIMUM BENDING MOMENT
AND SHEAR
**Typical Panel Type (mesh 3W11), Panel Thickness 6”, 5’x12’**

**Point load Input Force in panel for Staad Pro input**

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1’x1’</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
</tr>
<tr>
<td>P1</td>
<td>0.5</td>
</tr>
<tr>
<td>P2</td>
<td>0.1667</td>
</tr>
<tr>
<td>P3</td>
<td>-0.167</td>
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<tr>
<td>P4</td>
<td>0.1667</td>
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<tr>
<td>P5</td>
<td>-0.167</td>
</tr>
<tr>
<td>P6</td>
<td>-0.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1’x1’</th>
</tr>
</thead>
<tbody>
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<td></td>
<td>X</td>
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<td>P7</td>
<td>0.1667</td>
</tr>
<tr>
<td>P8</td>
<td>-0.167</td>
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<td>P9</td>
<td>-0.5</td>
</tr>
<tr>
<td>P10</td>
<td>0.5</td>
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<tr>
<td>P11</td>
<td>0.1667</td>
</tr>
<tr>
<td>P12</td>
<td>-0.167</td>
</tr>
</tbody>
</table>

**Load to panel based on Bar Mat Capacity per Bar Mat Point**

Bar Mat Used:
W11 - 75 Years

\[
A = 0.0797 \text{ in}^2
\]

\[
F_y = 75 \text{ ksi}
\]

\[
P = 0.65 \times A \times F_y = 3.885 \text{ Kips per wire point}
\]

**ATTACHMENT BY No. WIRES**

(standard panels only)
MX (local)  
kNm/m  
<= 0.360  
0.534  
0.709  
0.883  
1.06  
1.23  
1.41  
1.58  
1.75  
1.93  
2.1  
2.28  
2.45  
2.63  
2.8  
2.98  
>= 3.15
MY (local)
kNm/m

<table>
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<tr>
<th>Value</th>
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</tr>
<tr>
<td>-2.71</td>
<td>Purple</td>
</tr>
<tr>
<td>-2.13</td>
<td>Brown</td>
</tr>
<tr>
<td>-1.54</td>
<td>Pink</td>
</tr>
<tr>
<td>-0.951</td>
<td>Blue</td>
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<tr>
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<tr>
<td>0.223</td>
<td>Yellow</td>
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<td>Light Green</td>
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<td>4.92</td>
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<td>5.51</td>
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<tr>
<td>&gt;= 6.09</td>
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Load 1
SOX (local)
N/mm²

-0.124
-0.108
-0.093
-0.077
-0.062
-0.046
-0.031
-0.015
0
0.016
0.031
0.047
0.062
0.078
0.093
0.109
0.124

Load 1

STAAD.Pro for Windows 20.07.04.12
1. STAAD SPACE
   2. START JOINT INFORMATION
   3. ENGINEER DATE 1-2-MAY-21
   4. JOG CLIENT SUL
   5. ENGINEER NAME CH
   6. JOG REF SHEET CHECK
   7. END JOB INFORMATION
   8. INPUT WIDTH 79
   9. UNIT FEET KIP
   10. JOINT COORDINATES
   11. 1 0 0 0; 2 1 0 0; 3 2 0 0; 4 3 0 0; 5 4 0 0; 6 5 0 0; 7 6 0 0; 10 0 0 -1.
   12. 11 1 0 -1; 12 2 0 -1; 13 3 0 -1; 14 4 0 -1; 15 5 0 -1; 16 6 0 -1; 19 0 0 -2.
   13. 20 1 0 -2; 21 2 0 -2; 22 3 0 -2; 23 4 0 -2; 24 5 0 -2; 25 6 0 -2; 28 0 0 -3.
   14. 29 1 0 -3; 30 2 0 -3; 31 3 0 -3; 32 4 0 -3; 33 5 0 -3; 34 6 0 -3; 35 0 0 -4.
   15. 48 1 0 -4; 47 2 0 -4; 48 3 0 -4; 49 4 0 -4; 50 5 0 -4; 51 6 0 -4; 56 0 0 -5.
   57. 1 0 -5; 58 2 0 -5; 59 3 0 -5; 60 4 0 -5; 61 5 0 -5; 62 6 0 -5; 63 7 0 -1.
   16. 84 7 0 0; 85 7 0 0; 86 7 0 0; 87 8 0 -1; 88 8 0 -1; 89 9 0 -1; 90 10 0 -4.
   91. 93 1 0 -4; 94 2 0 -4; 95 7 0 -5; 96 8 0 -5; 97 9 0 -5; 96 10 0 -5.
   23. ELEMENT INCLUDED SHELL
   24. 1 1 10 11 2; 2 10 19 20 11; 3 19 28 29 20; 4 2 11 12 3; 5 11 20 21 12.
   25. 20 29 30 21; 7 3 12 13 4; 8 12 21 22 13; 9 21 30 31 22; 10 14 13 14.
   26. 11 13 32 33 14; 12 22 33 34; 33 23; 13 5 14 15 6; 14 14 33 24 32; 15 23 32 33 24.
   16. 1 1 15 16 7; 17 15 24 25 16; 18 24 33 24 32; 19 28 46 29 22; 32 29 46 27 30.
   28. 33 30 47 48 31; 34 31 48 49 32; 35 32 49 50 33; 36 33 50 51 34; 37 41 56 57 46.
   29. 42 46 57 58 47; 43 47 58 59 48; 44 48 59 60 49; 45 49 64 65 50; 46 50 63 62 51.
   30. 77 1 6 3 6 6 6; 61 16 23 65 65; 62 53 55 56 55; 63 54 60 62 70; 64 60 65 69 67.
   31. 52 65 66 70 69; 53 68 67 71 72; 54 67 65 73 73; 55 69 70 74 72; 56 72 71 75 76.
   32. 57 71 73 77 77; 58 73 74 74 74; 59 76 75 79 80; 60 75 77 82 78; 61 77 78 82 81.
   33. 62 80 79 83 84; 63 79 81 85 83; 64 81 82 86 85; 65 84 81 87 66; 66 86 87 88 70.
   34. 87 70 88 85 74; 68 76 85 90 78; 69 78 95 91 82; 70 82 91 92 86; 71 51 62 53 87.
   35. 72 87 92 94 88; 73 88 94 95 89; 74 89 95 94 90; 75 95 96 97 92; 76 91 97 98 92.
   36. ELEMENT PROPERTY
   37. 1 7 18 31 76 41 76 THICKNESS 0.5.
   38. DEFINE MATERIAL DATA
   39. ISOORTHOPIC CONCRETE
   40. E 614304
**Typical Panel Type (mesh 4W11), Panel Thickness 6", 5'x12'**

**Point load Input Force in panel for Staad Pro input**

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
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<tr>
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<td>P6</td>
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<td>P16</td>
<td>-0.5</td>
</tr>
</tbody>
</table>

**Load to panel based on Bar Mat Capacity per Bar Mat Point**

Bar Mat Used:

W11 - 75 Years

\[
A = 0.0797 \text{ in}^2 \\
F_y = 75 \text{ ksi} \\
P = 0.65 \times A \times F_y = 3.885 \text{ Kips (per wire point)}
\]

**ATTACHMENT BY No. WIRES**

(Standard panels only)
MX (local)
kNm/m
<= 0.844
1.02
1.19
1.37
1.54
1.71
1.89
2.06
2.23
2.41
2.58
2.76
2.93
3.1
3.28
3.45
>= 3.62

Load 1

5.00ft
12.00ft
MY (local) kNm/m

-3.86
-3.19
-2.53
-1.86
-1.2
-0.532
0.132
0.797
1.46
2.13
2.79
3.45
4.12
4.78
5.45
6.11
>= 6.78
SOX (local)  
N/mm²  
| <= -0.128 | -0.112 | -0.096 | -0.080 | -0.064 | -0.048 | -0.032 | -0.016 | 0 | 0.016 | 0.032 | 0.048 | 0.064 | 0.080 | 0.096 | 0.112 | >= 0.128 |

Load 1

5.00ft 12.00ft

Print Time/Date: 11/03/2021 01:46

STAAD.Pro for Windows 20.07.04.12
SQY (local) N/mm²

- ≤ -0.169
- -0.148
- -0.127
- -0.106
- -0.085
- -0.063
- -0.042
- -0.021
- 0
- 0.021
- 0.042
- 0.063
- 0.085
- 0.106
- 0.127
- 0.148
- ≥ 0.169
## STAAD SPACE

**INPUT FILE:** Panel Check - Sxl2 type A (4W1).STD

**START JOB INFORMATION**

### ENGINEER NAME

### JOB REFERENCE CHECK

### END JOB INFORMATION

**INPUT WIDTH:** 79

**UNIT FEET XIP**

**JOINT COORDINATES**

1. **1.0 0.0; 2.0 0.0; 3.2 0.0; 4.3 0.0; 5.4 0.0; 6.5 0.0; 7.6 0.0; 10.0 0.0 - 1.
2. **11.1 0.0; 12.2 0.0; 13.3 0.0; 14.4 0.0; 15.5 0.0; 16.6 0.0; 19.0 0.0 - 2.
3. **20.1 0.0; 21.2 0.0; 22.3 0.0; 23.4 0.0; 24.5 0.0; 25.6 0.0; 28.0 0.0 - 3.
4. **29.1 0.0; 30.2 0.0; 31.3 0.0; 32.4 0.0; 33.5 0.0; 34.6 0.0; 38.0 0.0 - 4.
5. **48.1 0.0; 49.2 0.0; 50.3 0.0; 51.4 0.0; 52.5 0.0 - 5.
6. **57.1 0.0; 58.2 0.0; 59.3 0.0; 60.4 0.0; 61.5 0.0; 62.6 0.0 - 6.
7. **67.1 0.0; 68.2 0.0; 69.3 0.0; 70.4 0.0 - 7.
8. **78.1 0.0; 79.2 0.0; 80.3 0.0; 81.4 0.0; 82.5 0.0; 83.6 0.0 - 8.
9. **90.1 0.0; 91.2 0.0; 92.3 0.0; 93.4 0.0; 94.5 0.0; 95.6 0.0 - 9.
10. **97.1 0.0; 98.2 0.0 - 10.

### ELEMENT IDENTIFIED SHELL

1. **213.1 10.1 2.1 10.2 10.1; 3.1 28.29 29.2; 4.1 11.1 12.2 3.1 21.2 12.2.
4. **28.1 29.2 30.2 31.2 32.2 33.2 34.2 35.2 36.2 37.2 38.2 39.2 40.2 41.2 42.2 43.2 44.2 45.2 46.2 47.2 48.2 49.2 50.2 51.2 52.2 53.2 54.2 55.2 56.2 57.2 58.2 59.2 60.2 61.2 62.2 63.2 64.2 65.2 66.2 67.2 68.2 69.2 70.2 71.2 72.2 73.2 74.2 75.2 76.2 77.2 78.2 79.2 80.2 81.2 82.2 83.2 84.2 85.2 86.2 87.2 88.2 89.2 90.2 91.2 92.2 93.2 94.2 95.2 96.2 97.2 98.2 99.2 100.2.
5. **30.1 31.2 32.2 33.2 34.2 35.2 36.2 37.2 38.2 39.2 40.2 41.2 42.2 43.2 44.2 45.2 46.2 47.2 48.2 49.2 50.2 51.2 52.2 53.2 54.2 55.2 56.2 57.2 58.2 59.2 60.2 61.2 62.2 63.2 64.2 65.2 66.2 67.2 68.2 69.2 70.2 71.2 72.2 73.2 74.2 75.2 76.2 77.2 78.2 79.2 80.2 81.2 82.2 83.2 84.2 85.2 86.2 87.2 88.2 89.2 90.2 91.2 92.2 93.2 94.2 95.2 96.2 97.2 98.2 99.2 100.2.
6. **40.1 41.2 42.2 43.2 44.2 45.2 46.2 47.2 48.2 49.2 50.2 51.2 52.2 53.2 54.2 55.2 56.2 57.2 58.2 59.2 60.2 61.2 62.2 63.2 64.2 65.2 66.2 67.2 68.2 69.2 70.2 71.2 72.2 73.2 74.2 75.2 76.2 77.2 78.2 79.2 80.2 81.2 82.2 83.2 84.2 85.2 86.2 87.2 88.2 89.2 90.2 91.2 92.2 93.2 94.2 95.2 96.2 97.2 98.2 99.2 100.2.
**Typical Panel Type (mesh 5W11), Panel Thickness 6", 5’x12’**

Point load Input Force in panel for Staad Pro input

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</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[
A = 0.0797 \text{ in}^2
\]

\[
F_y = 75 \text{ ksi}
\]

\[
P = 0.65 \times A \times F_y
\]

\[
P = 3.885 \text{ Kips (per wire point)}
\]

**ATTACHMENT BY NO. WIRES**

(Standard panels only)
STAAD.Pro V8i (SELECTseries 6) 20.07.11.33

Load 1

Print Time/Date: 30/12/2021 10:18
Print Run 1 of 1
MX (local)
kNm/m

≤ 0.855
1.08
1.3
1.51
1.73
1.95
2.17
2.39
2.61
2.83
3.05
3.27
3.49
3.71
3.93
4.15
≥ 4.37

Load 1

STAAD.Pro V8i (SELECTseries 6) 20.07.11.33
MY (local)
kNm/m

<= -2.02
-1.49
-0.969
-0.444
0.080
0.605
1.13
1.65
2.18
2.7
3.23
3.75
4.28
4.8
5.33
5.85
>= 6.38
SOX (local)
N/mm²

-0.147
-0.128
-0.110
-0.092
-0.073
-0.055
-0.037
-0.018
0
0.018
0.037
0.055
0.073
0.092
0.110
0.128
>= 0.147
SQY (local) N/mm²

- <= -0.138
-0.121
-0.104
-0.087
-0.069
-0.052
-0.035
-0.017
0
0.017
0.035
0.052
0.069
0.087
0.104
0.121
>= 0.138
Typical Panel Type (mesh 6W11), Panel Thickness 6", 5'x12'

Point load Input Force in panel for Staad Pro input

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[
A = 0.0797 \text{ in}^2 \\
F_y = 75 \text{ ksi} \\
P = 0.65 \times A \times F_y = 3.885 \text{ Kips (per wire point)}
\]
MX (local) kNm/m
- <= 1.7
- 1.89
- 2.09
- 2.28
- 2.47
- 2.66
- 2.85
- 3.04
- 3.24
- 3.43
- 3.62
- 3.81
- 4
- 4.19
- 4.39
- 4.58
- >= 4.77
MY (local) kNm/m

-3.29
-2.66
-2.03
-1.4
-0.765
-0.135
0.495
1.13
1.76
2.39
3.02
3.65
4.28
4.91
5.54
6.17
6.8

Load 1

5.00 ft
12.00 ft
SOX (local)
N/mm²
- <= -0.150
- -0.131
- -0.113
- -0.094
- -0.075
- -0.056
- -0.038
- -0.019
- 0
- 0.019
- 0.038
- 0.056
- 0.075
- 0.094
- 0.113
- 0.131
- >= 0.150

STAAD.Pro for Windows 20.07.04.12
SQY (local)
N/mm²

-0.143 ≤
-0.125
-0.107
-0.089
-0.072
-0.054
-0.036
-0.018
0
0.018
0.036
0.054
0.072
0.089
0.107
0.125
≥ 0.143
**STAAD SPACE**

INPUT FILE: Panel Check - Sx12 type A (6W11).STD

| 41. | POISON 0.17 |
| 42. | DENSITY 0.15 |
| 43. | ALPHAS 12-005 |
| 44. | DAMP 0.05 |
| 45. | END DEFINE MATERIAL |
| 46. | CONSTANTS |
| 47. | MATERIAL CONCRETE ALL |
| 48. | SUPPORTS |
| 49. | 1 TO 7 10 TO 16 19 TO 25 28 29 34 45 TO 51 56 TO 70 |
| 50. | 98 FIXED BUT KFX 839.76 KFY 839.76 KFE 839.76 KMX 1 KNY 1 KMQ 1 |
| 51. | LOAD 1 LOADTYPES PUSH TITLE WIRE STRENGTH |
| 52. | ELEMENT LOAD |
| 53. | 32 PR GY -3.885 0.25 -0.1667 |
| 54. | 33 PR GY -3.885 0.25 0.5 |
| 55. | 35 PR GY -3.885 0.25 0.1667 |
| 56. | 34 PR GY -3.885 0.25 -0.1667 |
| 57. | 34 GY GY -3.885 0.25 0.5 |
| 58. | 35 GY GY -3.885 0.25 0.1667 |
| 59. | 66 PR GY -3.885 0.25 -0.1667 |
| 60. | 66 GY GY -3.885 0.25 0.5 |
| 61. | 67 PR GY -3.885 0.25 -0.1667 |
| 62. | 67 GY GY -3.885 0.25 0.5 |
| 63. | 68 PR GY -3.885 0.25 -0.1667 |
| 64. | 68 GY GY -3.885 0.25 0.5 |
| 65. | 8 PR GY -3.885 0.25 -0.1667 |
| 66. | 8 GY GY -3.885 0.25 0.5 |
| 67. | 91 11 10 11 2 2 10 19 20 11 3 19 28 29 20 4 2 11 12 3 5 11 20 21 12 |
| 68. | 8 5 12 21 12 19 21 30 31 22 12 10 4 13 14 5 |
| 69. | 11 13 22 33 14 12 22 33 33 23 33 13 5 14 15 6 14 14 33 24 15 15 33 33 24 |
| 70. | 16 16 16 7 17 15 24 25 16 18 24 33 34 32 31 28 45 46 29 32 29 47 30 |
| 71. | 33 30 47 31 31 34 48 39 32 35 49 50 33 36 50 51 34 34 45 56 57 46 |
| 72. | 42 48 57 58 61 63 47 58 59 48 48 59 60 49 45 49 60 62 50 46 50 62 52 51 |
| 73. | 47 76 63 64 68 16 25 65 65 49 23 34 66 65 50 64 63 67 68 81 63 65 69 67 |
| 74. | 52 65 66 67 68 71 72 54 67 69 73 71 59 69 70 74 72 56 72 71 75 76 |
| 75. | 57 77 73 77 75 58 73 74 78 77 59 76 75 79 80 60 75 77 82 81 67 77 78 82 81 |
| 76. | 62 80 79 83 84 63 79 81 85 83 66 81 82 86 85 55 34 51 87 66 87 88 70 |
| 77. | 64 70 88 85 74 68 76 85 90 78 63 78 95 91 82 70 82 91 92 86 71 51 62 53 87 |
| 78. | 72 87 93 84 88 73 88 95 95 89 76 89 95 94 50 72 95 97 97 76 91 97 88 92 |

---

** ELEMENT PROPERTIES **

1. TO 18 31 TO 36 11 TO 36 THICKNESS 0.5
2. DEFINE MATERIAL DATA
3. ISOTROPIC CONCRETE
4. E 614304
Typical Panel Type (mesh 4W20), Panel Thickness 6", 5'x12'

Point load Input Force in panel for Staad Pro input

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</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[
A = 0.1583 \text{ in}^2 \\
F_y = 75 \text{ ksi} \\
P = 0.65 \times A \times F_y = 7.717 \text{ Kips (per wire point)}
\]

ATTACHMENT BY NO. WIRES (STANDARD PANELS ONLY)
STAAD.Pro for Windows 20.07.04.12

Load 1

5.00ft
12.00ft

Print Time/Date: 11/03/2021 02:20
Print Run 1 of 1
MX (local)
kNm/m

- <= 1.68
- 2.02
- 2.37
- 2.71
- 3.06
- 3.4
- 3.75
- 4.09
- 4.44
- 4.78
- 5.13
- 5.47
- 5.82
- 6.16
- 6.51
- 6.85
- >= 7.2

STAAD.Pro for Windows 20.07.04.12
SOX (local)
N/mm²
<= -0.254
-0.222
-0.191
-0.159
-0.127
-0.095
-0.064
-0.032
0
0.032
0.064
0.095
0.127
0.159
0.191
0.222
>= 0.254
SQY (local) N/mm²

-0.336
-0.294
-0.252
-0.210
-0.168
-0.126
-0.084
-0.042
0
0.042
0.084
0.126
0.168
0.210
0.252
0.294
>= 0.336
1. **STAAD SPACE**

**INPUT FILE:** Panel Check - Sx12 type A (4820).STD

2. **START JOB INFORMATION**

3. **ENGINEER DATE** 2007-04-21

4. **JOB CLIENT** CEL

5. **ENGINEER NAME** CH

6. **JOB REF PANEL CHECK**

7. **END JOB INFORMATION**

8. **INPUT WIDTH** 79

9. **UNIT FEET XIP**

10. **JOINT COORDINATES**

   11. 0 0 0; 2 1 0 0; 3 2 0 0; 4 3 0 0; 5 4 0 0; 6 5 0 0; 7 6 0 0; 10 0 0 -1.

   12. 11 1 0 -1; 12 2 0 -1; 13 3 0 -1; 14 4 0 -1; 15 5 0 -1; 16 6 0 -1; 19 0 0 -2

   13. 20 1 0 -2; 21 2 0 -2; 22 3 0 -2; 23 4 0 -2; 24 5 0 -2; 25 6 0 -2; 28 0 0 -3

   14. 29 1 0 -3; 30 2 0 -3; 31 3 0 -3; 32 4 0 -3; 33 5 0 -3; 34 6 0 -3; 35 0 0 -4

   15. 48 1 0 -4; 47 2 0 -4; 46 3 0 -4; 45 4 0 -4; 44 5 0 -4; 43 6 0 -4; 46 0 -5

   16. 51 1 0 -5; 52 2 0 -5; 53 3 0 -5; 54 4 0 -5; 55 5 0 -5; 56 6 0 -5; 57 0 0 -1

   17. 68 7 0 0; 65 7 0 0; 66 7 0 -3; 67 8 0 -3; 68 8 0 -3; 69 8 0 -2; 70 8 0 -3

   18. 71 9 0 -1; 72 0 0 -2; 73 9 0 -1; 74 9 0 -3; 75 10 0 -2; 76 10 0 -3

   19. 77 10 0 -2; 78 10 0 -3; 79 11 0 -1; 80 11 0 -1; 81 11 0 -2; 82 11 0 -3; 83 12 0 -1

   20. 84 12 0 0; 85 12 0 -2; 86 12 0 -3; 87 7 0 -6; 88 8 0 -6; 89 8 0 -4; 90 10 0 -4

   21. 91 12 0 -4; 92 12 0 -4; 93 7 0 -5; 94 8 0 -5; 95 9 0 -5; 96 10 0 -5

   22. 97 11 0 -5; 98 12 0 -3

23. **ELEMENT INCLUSION SHELL**

   24. 1.1 1.1 1.2 2.1 1.9 20 11; 3 1.2 29 20 20; 4 2.1 20 11; 5 2.2 20 11;

   25. 6 2.3 29 20 21; 7 3 1 13 4; 8 12 21 21 13; 9 21 30 31; 22 10 13 14 10

   26. 11 13 23 33 14; 12 22 33; 13 23 33 14; 14 22 33 24 15; 15 23 33 22 24

   27. 16 15 16 7; 17 15 24 15; 16 15 24 15; 18 24 33 24; 21 24 45 24; 22 24 45 24;

   28. 23 30 47 31; 34 31 47 31; 35 31 47 31; 36 33 50 31; 37 33 50 31; 38 33 50 31;

   29. 39 48 58 31; 40 48 58 31; 41 48 58 31; 42 48 58 31; 43 48 58 31; 44 48 58 31;

   30. 47 7 16 6 6 1; 48 16 25 6 6 1; 49 25 38 6 6 1; 50 45 63 6 6 1; 51 63 65 6 6 1

   31. 52 66 67 6 6 1; 53 67 68 6 6 1; 54 67 68 6 6 1; 55 69 70 6 6 1; 56 70 71 7 1 6

   32. 57 71 72 7 1 6; 58 73 74 7 1 6; 59 75 76 7 1 6; 60 75 76 7 1 6; 61 77 78 8 2 8

   33. 62 80 93 8 8 4; 63 98 89 8 8 4; 64 81 82 8 8 5; 65 81 82 8 8 5; 66 81 82 8 8 5

   34. 67 70 88 8 8 3; 68 76 89 8 8 3; 69 78 90 8 8 3; 70 80 91 8 8 3; 71 91 92 8 8 3

   35. 72 82 93 8 8 3; 73 88 89 8 8 3; 74 86 89 8 8 3; 75 95 96 8 8 3; 76 96 97 8 8 3

   36. **ELEMENT PROPERTY**

   37. 1 to 18 31 to 22 36 41 to 76 THICKNESS 0.5

   38. **DEFINE MATERIAL**

   39. **ISOORTH CONCRETE**

   40. E 614304
Typical Panel Type (mesh 5W20), Panel Thickness 6"., 5’x12’

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1’x1’</th>
</tr>
</thead>
<tbody>
<tr>
<td>X</td>
<td>Y</td>
</tr>
<tr>
<td>P1</td>
<td>-0.167 0.25</td>
</tr>
<tr>
<td>P2</td>
<td>0.5 0.25</td>
</tr>
<tr>
<td>P3</td>
<td>0.1667 0.25</td>
</tr>
<tr>
<td>P4</td>
<td>-0.167 0.25</td>
</tr>
<tr>
<td>P5</td>
<td>0.5 0.25</td>
</tr>
<tr>
<td>P6</td>
<td>-0.5 0.25</td>
</tr>
</tbody>
</table>

Location Coordinated From Center Panel 1’x1’

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[ A = 0.1583 \text{ in}^2 \]
\[ F_y = 75 \text{ ksi} \]

\[ P = 0.65 \times A \times F_y = 7.717 \text{ Kips} \] (per wire point)

ATTACHMENT BY No. WIRES
(Stanard Panels Only)
### MX (local) kNm/m

<table>
<thead>
<tr>
<th>Value</th>
<th>Color</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;= 1.7</td>
<td></td>
</tr>
<tr>
<td>2.57</td>
<td>2.88</td>
</tr>
<tr>
<td>3.01</td>
<td>3.45</td>
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<tr>
<td>3.88</td>
<td>4.32</td>
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<td>4.76</td>
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<td>5.63</td>
<td>6.07</td>
</tr>
<tr>
<td>6.5</td>
<td>6.94</td>
</tr>
<tr>
<td>7.38</td>
<td>8.25</td>
</tr>
<tr>
<td>8.69</td>
<td>9.4</td>
</tr>
<tr>
<td>&gt;= 8.69</td>
<td></td>
</tr>
</tbody>
</table>

### XY Z

<table>
<thead>
<tr>
<th>Load 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.00ft</td>
</tr>
<tr>
<td>12.00ft</td>
</tr>
</tbody>
</table>

Software licensed to Optimal Engineering Support

Print Time/Date: 30/12/2021 05:33

Print Run 1 of 1

STAAD.Pro V8i (SELECTseries 6) 20.07.11.33
SOX (local)
N/mm²

<= -0.291
-0.255
-0.218
-0.182
-0.146
-0.109
-0.073
-0.036
0
0.036
0.073
0.109
0.146
0.182
0.218
0.255
>= 0.291

Load 1

STAAD.Pro V8i (SELECTseries 6) 20.07.11.33
**Typical Panel Type (mesh 6W20), Panel Thickness 6”, 5’x12’**

Point load Input Force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1’x1’</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>-0.167, 0.25</td>
</tr>
<tr>
<td>P2</td>
<td>0.5, 0.25</td>
</tr>
<tr>
<td>P3</td>
<td>0.1667, 0.25</td>
</tr>
<tr>
<td>P4</td>
<td>-0.167, 0.25</td>
</tr>
<tr>
<td>P5</td>
<td>0.5, 0.25</td>
</tr>
<tr>
<td>P6</td>
<td>0.1667, 0.25</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[ P = 0.65 \times A \times F_y \]

\[ P = 0.65 \times 0.1583 \text{ in}^2 \times 75 \text{ ksi} \]

\[ P = 7.717 \text{ Kips} \quad \text{(per wire point)} \]
<table>
<thead>
<tr>
<th>Job Title</th>
<th>Client</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SSL</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Ref</th>
<th>Panel Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>By</td>
<td>CH</td>
</tr>
<tr>
<td>Date</td>
<td>10-Mar-21</td>
</tr>
<tr>
<td>Chd</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>File</th>
<th>Panel Check - 5x12 type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date/Time</td>
<td>11-Mar-2021 03:55</td>
</tr>
</tbody>
</table>

**Diagram**

STAAD.Pro for Windows 20.07.04.12
MX (local) kNm/m

- <= 3.38
- 3.76
- 4.14
- 4.52
- 4.91
- 5.29
- 5.67
- 6.05
- 6.43
- 6.81
- 7.19
- 7.57
- 7.95
- 8.33
- 8.71
- 9.09
- >= 9.47
MY (local)
kN/m/m

-6.53
-5.28
-4.02
-2.77
-1.52
-0.268
0.984
2.24
3.49
4.74
5.99
7.24
8.5
9.75
11
12.3

>= 13.5
Typical Panel Type (mesh 5W24), Panel Thickness 6", 5'x12'

Point load input force in panel for Staad Pro input

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
</tr>
<tr>
<td>P1</td>
<td>-0.167</td>
</tr>
<tr>
<td>P2</td>
<td>0.5</td>
</tr>
<tr>
<td>P3</td>
<td>0.1667</td>
</tr>
<tr>
<td>P4</td>
<td>-0.167</td>
</tr>
<tr>
<td>P5</td>
<td>0.5</td>
</tr>
<tr>
<td>P6</td>
<td>-0.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
</tr>
<tr>
<td>P7</td>
<td>0.1667</td>
</tr>
<tr>
<td>P8</td>
<td>-0.167</td>
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<tr>
<td>P9</td>
<td>-0.5</td>
</tr>
<tr>
<td>P10</td>
<td>0.1667</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
</tr>
<tr>
<td>P11</td>
<td>0.5</td>
</tr>
<tr>
<td>P12</td>
<td>0.1667</td>
</tr>
<tr>
<td>P13</td>
<td>-0.167</td>
</tr>
<tr>
<td>P14</td>
<td>0.5</td>
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<tr>
<td>P15</td>
<td>0.1667</td>
</tr>
<tr>
<td>P16</td>
<td>-0.167</td>
</tr>
</tbody>
</table>

Load to panel based on Bar Mat Capacity per Bar Mat Point

Bar Mat Used:
W11 - 75 Years

\[
A = 0.194 \text{ in}^2
\]

\[
F_y = 75 \text{ ksi}
\]

\[
P = 0.65 \times A \times F_y
\]

\[
= 9.458 \text{ kips} \quad \text{(per wire point)}
\]

3-WIRE CONNECTION PANEL
4-WIRE CONNECTION PANEL
5-WIRE CONNECTION PANEL

ATTACHMENT BY No. WIRES
(Standard Panels Only)
SOX (local)
N/mm²
-0.357
-0.312
-0.268
-0.223
-0.178
-0.134
-0.089
-0.045
0
0.045
0.089
0.134
0.178
0.223
0.268
0.312
0.357

Load 1
1. STAAD SPACE

INPUT FILE:  Z:\\6SS\\00 मैसूर वॉली\\000 Checking Only\\Panel Design\\staad\\Panel Check - 5x12 type A (5x24)_R.._STD

2. START JOB INFORMATION
3. ENGINEER DATE 10-MAA-21
4. JOB CLIENT SSL
5. ENGINEER NAME CH
6. JOS NAME PANEL CHECK
7. END JOB INFORMATION
8. INPUT WIDTH 79
9. UNIT FEET XIP
10. JOINT COORDINATES
11. 1 0 0 0 2 1 0 2 3 2 0 0 4 3 0 0 5 4 0 0 6 5 0 0 7 6 0 0 8 7 0 0 9 0 0 0 -1
12. 11 1 0 -1 12 0 0 -1 13 0 0 -1 14 0 0 -1 15 0 0 -1 16 0 0 -1 19 0 0 2
13. 20 1 0 -2 21 0 0 -2 22 3 0 -2 23 4 0 -2 24 5 0 -2 25 6 0 -2 28 0 0 -3
14. 29 1 0 -3 30 2 0 -3 31 3 0 -3 32 4 0 -3 33 5 0 -3 34 6 0 -3 45 0 0 -4
15. 48 1 0 -4 49 2 0 -4 50 3 0 -4 51 4 0 -4 52 5 0 -4 53 6 0 -5 56 0 0 -5
16. 57 1 0 -5 58 2 0 -5 59 3 0 -5 60 4 0 -5 61 5 0 -5 62 6 0 -5 63 7 0 -1
17. 68 7 0 65 7 0 -2 66 7 0 -3 67 8 0 -3 68 8 0 -3 69 8 0 -2 70 8 0 -3
18. 71 8 0 -1 72 0 0 -2 73 9 0 -2 74 9 0 -3 75 10 0 -3 76 10 0 -2
19. 78 10 0 -3 79 11 0 -1 80 11 0 0 81 11 0 -2 82 11 0 -3 83 12 0 -1
20. 84 12 0 -5 85 12 0 -2 86 12 0 -3 87 7 0 -4 88 8 0 -4 89 9 0 -4 90 10 0 -4
21. 91 11 0 -4 92 10 0 -4 93 7 0 -5 94 8 0 -5 95 9 0 -5 96 10 0 -5
22. 97 11 0 -5 98 12 0 -3
23. ELEM ENCLOSED SHELL
24. 1 1 1 1 1 2 2 2 1 10 19 20 11 3 19 28 29 20 4 2 11 12 3 5 11 20 21 12
25. 6 20 20 30 21 7 12 13 4 8 12 21 22 13 9 21 30 31 22 10 14 13 14 15
26. 16 13 12 33 14 12 22 33 33 23 13 5 14 15 16 14 16 23 24 15 15 23 32 33 24
27. 16 15 16 7 17 15 24 25 16 18 24 33 24 34 25 31 28 45 46 29 32 29 46 47 30
28. 33 30 47 48 31 34 31 48 49 32 35 49 50 51 36 49 50 51 41 45 56 57 46
29. 42 46 57 58 47 43 67 58 59 48 44 48 59 60 49 45 49 60 61 50 46 61 62 51
30. 47 7 16 63 64 64 16 25 65 63 65 49 29 38 66 65 70 64 63 67 68 51 63 65 67 69
31. 52 65 66 70 69 53 68 67 71 72 54 67 68 73 71 55 69 70 74 72 56 72 71 75 76
32. 57 73 73 77 76 58 73 74 76 77 59 76 75 79 80 60 75 77 82 76 61 77 78 82 81
33. 62 80 79 83 84 63 19 81 85 83 64 81 82 86 85 65 34 81 87 66 66 87 88 70
34. 67 70 88 85 74 68 76 85 90 78 69 78 95 91 82 70 82 91 92 86 71 51 62 53 87
35. 72 87 93 94 88 73 88 94 95 89 76 89 95 94 50 72 95 94 97 52 76 91 97 98 92
36. ELEM PROPERTY
37. 1 16 31 31 36 41 17 76 THICKNESS 0.5
38. DEFINE MATERIAL DATA

PROBLEM STATISTICS

NUMBER OF JOINTS 78
NUMBER OF MEMBERS 0
NUMBER OF PLATES 60
NUMBER OF SOLIDS 0
NUMBER OF SURFACES 0
NUMBER OF SUPPORTS 78

SOLVER USED IS THE IN-CORE ADVANCED MATH SOLVER
TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 468
TOTAL LOAD COMBINATION CASES = 0 SO FAR.
## Typical Panel Type (mesh 6W24), Panel Thickness 6", 5'x12'

**Point load Input Force in panel for Staad Pro input**

<table>
<thead>
<tr>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
<th>Location</th>
<th>Coordinated From Center Panel 1'x1'</th>
</tr>
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<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td>Y</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P1</td>
<td>-0.167</td>
<td>0.25</td>
<td></td>
<td></td>
<td>P7</td>
<td>-0.167</td>
<td>0.25</td>
</tr>
<tr>
<td>P2</td>
<td>0.5</td>
<td>0.25</td>
<td></td>
<td></td>
<td>P8</td>
<td>0.5</td>
<td>0.25</td>
</tr>
<tr>
<td>P3</td>
<td>0.1667</td>
<td>0.25</td>
<td></td>
<td></td>
<td>P9</td>
<td>0.1667</td>
<td>0.25</td>
</tr>
<tr>
<td>P4</td>
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<td>0.25</td>
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<td>P10</td>
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<td>0.5</td>
<td>0.25</td>
<td></td>
<td></td>
<td>P11</td>
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<td>0.25</td>
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<tr>
<td>P6</td>
<td>0.1667</td>
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<td></td>
<td></td>
<td>P12</td>
<td>0.1667</td>
<td>0.25</td>
</tr>
</tbody>
</table>

**Load to panel based on Bar Mat Capacity per Bar Mat Point**

Bar Mat Used:
W11 - 75 Years

\[
A = 0.194 \text{ in}^2 \\
Fy = 75 \text{ ksi} \\
P = 0.65 \times A \times Fy \\
= 9.458 \text{ Kips} \text{ (per wire point)}
\]

---

**ATTACHMENT BY No. WIRES**

(Standard Panels Only)
MX (local) kNm/m

- <= 4.15
- 4.61
- 5.08
- 5.55
- 6.01
- 6.48
- 6.95
- 7.41
- 7.88
- 8.35
- 8.81
- 9.28
- 9.75
- 10.2
- 10.7
- 11.1
- >= 11.6

Load 1

STAAD.Pro for Windows 20.07.04.12
MY (local)
kNm/m

<= -8
-6.47
-4.93
-3.4
-1.86
-0.329
1.21
2.74
4.28
5.81
7.34
8.88
10.4
11.9
13.5
15

>= 16.6
SOX (local)
N/mm²

-0.365
-0.320
-0.274
-0.228
-0.183
-0.137
-0.091
-0.046
0
0.046
0.091
0.137
0.183
0.228
0.274
0.320
0.365

Load 1

STAAD.Pro for Windows 20.07.04.12
SQY (local) N/mm²

-0.349
-0.305
-0.261
-0.218
-0.174
-0.131
-0.087
-0.044
0
0.044
0.087
0.131
0.174
0.218
0.261
0.305
0.349

STAAD.Pro for Windows 20.07.04.12
### Summary Result From Staad Pro Analysis Output

<table>
<thead>
<tr>
<th>Standard Panel Size</th>
<th>Barmat Type</th>
<th>Panel Type</th>
<th>Horz Rebar</th>
<th>Vert Rebar</th>
<th>Staad Pro Output</th>
<th>Data for AASHTO LRFD Design Check</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Mx (kNm/m)</td>
<td>My (kNm/m)</td>
</tr>
<tr>
<td>5'x12'</td>
<td>3W11</td>
<td>A</td>
<td>4 #4</td>
<td>8 #4</td>
<td>3.150</td>
<td>6.090</td>
</tr>
<tr>
<td></td>
<td>4W11</td>
<td>X</td>
<td>8 #4</td>
<td>12 #4</td>
<td>7.200</td>
<td>13.500</td>
</tr>
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<td>5W11</td>
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<td>4.770</td>
<td>6.800</td>
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<td>4W20</td>
<td>X</td>
<td>8 #4</td>
<td>12 #4</td>
<td>8.690</td>
<td>12.700</td>
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<td>5W20</td>
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<td>9.470</td>
<td>13.500</td>
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<tr>
<td></td>
<td>6W24</td>
<td>Y</td>
<td>9 #4</td>
<td>12 #4</td>
<td>10.600</td>
<td>15.500</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>11.600</strong></td>
<td><strong>16.600</strong></td>
</tr>
</tbody>
</table>

* Maximum bending moment and shear for each standard panel size shown in red bold font
5’ x 12’ STANDARD
A PANEL
**Typical Panel Type A, Panel Thickness 6″, 5’x12’, Horizontal Rebar**

**Panel Strength Design**
Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness</td>
<td>t&lt;sub&gt;panel&lt;/sub&gt; = 6 in</td>
</tr>
<tr>
<td>Finishing Thickness</td>
<td>c&lt;sub&gt;panel&lt;/sub&gt; = 1 in</td>
</tr>
<tr>
<td>Additional Load from Finishes</td>
<td>= 0.0125 Kip/ft&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>Concrete Cover</td>
<td>= 1.5 in</td>
</tr>
<tr>
<td>Vertical Bar Size</td>
<td>= #4</td>
</tr>
<tr>
<td>Rebar Diameter</td>
<td>d&lt;sub&gt;b&lt;/sub&gt; = 0.5 in</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>A&lt;sub&gt;b&lt;/sub&gt; = 0.196 in&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>Rebar spacing</td>
<td>s = 15 in</td>
</tr>
<tr>
<td>Width of the design section</td>
<td>b = 12 in</td>
</tr>
<tr>
<td>Effective depth of section</td>
<td>d&lt;sub&gt;s&lt;/sub&gt; = 4.25 in</td>
</tr>
<tr>
<td>Effective depth of section for Negative Moment</td>
<td>d&lt;sub&gt;s(M-r)&lt;/sub&gt; = 1.75 in</td>
</tr>
<tr>
<td>Correction factor for source aggregate</td>
<td>k&lt;sub&gt;1&lt;/sub&gt; = 1 in</td>
</tr>
<tr>
<td>Concrete Density</td>
<td>W&lt;sub&gt;c&lt;/sub&gt; = 0.15 kcf</td>
</tr>
<tr>
<td>Reinforcement Strength</td>
<td>f&lt;sub&gt;y&lt;/sub&gt; = 60 ksi</td>
</tr>
<tr>
<td>Modulus Elasticity of concrete</td>
<td>E&lt;sub&gt;c&lt;/sub&gt; = 4266 ksi</td>
</tr>
<tr>
<td>Modulus Elasticity of reinforcement</td>
<td>E&lt;sub&gt;s&lt;/sub&gt; = 29000 ksi</td>
</tr>
<tr>
<td>Modular Ratio</td>
<td>n = 6.798</td>
</tr>
<tr>
<td>Area of Steel per Design Strip</td>
<td>A&lt;sub&gt;s&lt;/sub&gt; = 0.157 in&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Load</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>+M&lt;sub&gt;u&lt;/sub&gt;</td>
<td>= 1.53 kip-ft</td>
</tr>
<tr>
<td>-M&lt;sub&gt;u&lt;/sub&gt;</td>
<td>= 0.74 kip-ft</td>
</tr>
<tr>
<td>+M&lt;sub&gt;u&lt;/sub&gt;_service</td>
<td>= 1.13 kip-ft</td>
</tr>
<tr>
<td>-M&lt;sub&gt;u&lt;/sub&gt;_service</td>
<td>= 0.55 kip-ft</td>
</tr>
</tbody>
</table>

**Positive Moment Capacity**

Depth of equivalent stress block

\[
a = \frac{A_s f_y}{0.85 f_{c'} b} = 0.231 \text{ in}
\]

Factored flexural resistance

\[
+\varphi M_n = \varphi A_s f_y \left( d_s - \frac{a}{2} \right) = 2.923 \text{ kip-ft}
\]

Check

\[
+\varphi M_n > +M_u \quad 2.923 > 1.53 \quad \text{OK}
\]

**Negative Moment Capacity**

Depth of equivalent stress block

\[
a = \frac{A_s f_y}{0.85 f_{c'} b} = 0.231 \text{ in}
\]

Factored flexural resistance

\[
-\varphi M_n = \varphi A_s f_y \left( d_{s(M-r)} - \frac{a}{2} \right) = 1.155 \text{ kip-ft}
\]

Check

\[
-\varphi M_n > -M_u \quad 1.155 > 0.74 \quad \text{OK}
\]
Minimum Reinforcement AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, $M_r = \varphi M_n$, at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

$$M_{cr} = \gamma_3 \left[ (\gamma_1 f_r + \gamma_2 f_{pe}) S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \right]$$  

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c$$

where:

- flexural cracking variability factor $\gamma_1 = 1.20$ (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength $\gamma_3 = 0.67$ (A615 steel)
- concrete density modification factor $\lambda = 1.00$ AASHTO 5.4.2.8
- modulus of rupture $f_r = 0.24\lambda \sqrt{f_c}$, $f_r = 0.48$ ksi AASHTO 5.4.2.6

section modulus of design section

$$S_c = \frac{bh^2}{6} = \frac{bt_{panel}^2}{6}$$  

$S_c = 72.00$ in$^3$

Cracking moment

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c$$  

$M_{cr} = 2.32$ kip-ft

check positive moment reinforcement:

1.33 x factored ultimate moment

Cracking moment

min from (1.33 x factored +$M_u$ and $M_{cr}$)

Check:

$$+\varphi M_n > \min( +M_u , M_{cr} )$$  

2.923 > 2.03  OK

check negative moment reinforcement:

1.33 x factored ultimate moment

Cracking moment

min from (1.33 x factored -$M_u$ and $M_{cr}$)

Check:

$$-\varphi M_n > \min( -M_u , M_{cr} )$$  

1.155 > 0.98  OK
**Typical Panel Type A, Panel Thickness 6”, 5’x12’, Horizontal Rebar**

**Shear on Panel**

- **Panel Thickness** \( t_{\text{panel}} = 6 \text{ in} \)
- **Concrete Cover** \( c_{\text{panel}} = 1.5 \text{ in} \)
- **Concrete Strength** \( f'_{c} = 4.0 \text{ ksi} \) (Concrete Class A compressive strength)
- **Reinforcement Strength** \( f_{y} = 60 \text{ ksi} \) (Minimum yield strength of grade 60 steel)
- **Bar Size** #4
- **Rebar Diameter** \( d_{b} = 0.500 \text{ in} \)
- **Rebar Area** \( A_{b} = 0.196 \text{ in}^{2} \)
- **Rebar spacing** \( s = 15 \text{ in} \)
- **Width of the design section** \( b = 12 \text{ in} \)
- **Area of Steel per Design Strip** \( A_{s} = 0.157 \text{ in}^{2} \)

**AASHTO 5.7.3.3**

\( V_{n} \) shall be the lesser of:

1. \( V_{c} = 6.240 \text{ Kips} \)
2. \( V_{s} = 3.231 \text{ Kips} \)

(1) \( V_{n} = 9.472 \text{ Kips} \)

(2) \( V_{n} = 6.524 \text{ Kips} \)

From (1) and (2) \( V_{n} \) should be: \( V_{n} = 9.472 \text{ Kips} \)

**Shear Force**

- \( \phi \cdot V_{n} = 0.9 \cdot 9.472 = 8.524 \text{ Kips} \)

\( \frac{c}{d_{b}} = \frac{t_{\text{panel}} - c_{\text{panel}} - d_{b}}{2} \)

\( d_{v} = 4.114 \text{ in} \)

From Staad Pro Result \( V_{\text{max}} = 0.169 \text{ N/mm}^{2} \)

\( = 24.51 \text{ lbs/in}^{2} \)

**Shear Section Area**

\( b \times t_{\text{panel}} = 12 \times 6 \)

\( V_{\text{max}} = 1764.82 \text{ lbs} \)

\( = 1.765 \text{ Kips} \) OK, \( V_{\text{max}} \) less than \( V_{n} \)
Typical Panel Type A, Panel Thickness 6", 5'x12', Vertical Rebar

Panel Strength Design
Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

Panel Thickness

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>t&lt;sub&gt;panel&lt;/sub&gt;</td>
<td>6 in</td>
</tr>
<tr>
<td>Finishing Thickness</td>
<td>1 in</td>
</tr>
<tr>
<td>Additional Load from Finishes</td>
<td>0.0125 Kip/ft&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>Concrete Cover</td>
<td>C&lt;sub&gt;panel&lt;/sub&gt; = 1.5 in</td>
</tr>
</tbody>
</table>

Vertical Bar Size #4

Logitudinal Bar Size #4

Rebar Diameter

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>d&lt;sub&gt;b&lt;/sub&gt;</td>
<td>0.5 in</td>
</tr>
</tbody>
</table>

Rebar Area

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>A&lt;sub&gt;b&lt;/sub&gt;</td>
<td>0.196 in&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

Rebar spacing

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>s</td>
<td>18 in</td>
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</tbody>
</table>

Width of the design section

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>b</td>
<td>12 in</td>
</tr>
</tbody>
</table>

Effective depth of section

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>d&lt;sub&gt;s&lt;/sub&gt;</td>
<td>3.75 in</td>
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</table>

Effective depth of section for Negative Moment

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>d&lt;sub&gt;s(M)&lt;/sub&gt;</td>
<td>2.25 in</td>
</tr>
</tbody>
</table>

Correction factor for source aggregate

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>K&lt;sub&gt;s&lt;/sub&gt;</td>
<td>1 in</td>
</tr>
</tbody>
</table>

Concrete Density

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>W&lt;sub&gt;c&lt;/sub&gt;</td>
<td>0.15 kcf</td>
</tr>
</tbody>
</table>

Concrete Strength

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>f'&lt;sub&gt;c&lt;/sub&gt;</td>
<td>4.00 ksi (Concrete Class A compressive strength)</td>
</tr>
</tbody>
</table>

Reinforcement Strength

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>f&lt;sub&gt;y&lt;/sub&gt;</td>
<td>60 ksi (Minimum yield strength of grade 60 steel)</td>
</tr>
</tbody>
</table>

Modulus Elasticity of concrete

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>E&lt;sub&gt;c&lt;/sub&gt;</td>
<td>4266 ksi</td>
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Modulus Elasticity of reinforcement

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>E&lt;sub&gt;s&lt;/sub&gt;</td>
<td>29000 ksi</td>
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Modular Ratio

<table>
<thead>
<tr>
<th>Parameter</th>
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</tr>
</thead>
<tbody>
<tr>
<td>n</td>
<td>6.798</td>
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</tbody>
</table>

Area of Steel per Design Strip

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>A&lt;sub&gt;s&lt;/sub&gt;</td>
<td>0.131 in&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

Resistance factor for tension-controlled section

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>ϕ&lt;sub&gt;STR&lt;/sub&gt;</td>
<td>0.90</td>
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</tbody>
</table>

Positive Moment Capacity

Depth of equivalent stress block

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>0.192 in</td>
</tr>
</tbody>
</table>

Factored flexural resistance

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>+ϕM&lt;sub&gt;n&lt;/sub&gt;</td>
<td>2.152 kip-ft</td>
</tr>
</tbody>
</table>

Check

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>+ϕM&lt;sub&gt;n&lt;/sub&gt; &gt; +M&lt;sub&gt;u&lt;/sub&gt;</td>
<td>2.152 &gt; 1.07 OK</td>
</tr>
</tbody>
</table>

Negative Moment Capacity

Depth of equivalent stress block

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>0.192 in</td>
</tr>
</tbody>
</table>

Factored flexural resistance

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>-ϕM&lt;sub&gt;n&lt;/sub&gt;</td>
<td>1.269 kip-ft</td>
</tr>
</tbody>
</table>

Check

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>-ϕM&lt;sub&gt;n&lt;/sub&gt; &gt; -M&lt;sub&gt;u&lt;/sub&gt;</td>
<td>1.269 &gt; 0.00 OK</td>
</tr>
</tbody>
</table>
Minimum Reinforcement

AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r = \varphi M_n \), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

\[
M_{cr} = \gamma_3 \left( (\gamma_1 f_t + \gamma_2 y_{cp}) S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \right)
\]

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_t S_c
\]

where:

- flexural cracking variability factor \( \gamma_1 = 1.60 \) (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength \( \gamma_3 = 0.67 \) (A615 steel)
- concrete density modification factor \( \lambda = 1.00 \) AASHTO 5.4.2.8

- modulus of rapture \( f_r = 0.24\lambda \sqrt{f_c^3} \)
- \( f_r = 0.48 \) ksi AASHTO 5.4.2.6

- section modulus of design section \( S_c = \frac{b h^2}{6} = \frac{b t_{panel}^2}{6} \)
- \( S_c = 72.00 \) in³

Cracking moment

\[
M_{cr} = \gamma_3 \gamma_1 f_t S_c
\]

\( M_{cr} = 3.09 \) kip-ft

check positive moment reinforcement:

1.33 x factored ultimate moment \( = 1.43 \) kip-ft

Cracking moment \( = 3.09 \) kip-ft

min from (1.33 x factored +\( M_u \) and \( M_{cr} \)) \( = 1.43 \) kip-ft

Check:

\[
+\varphi M_n > \min(+M_u, M_{cr})
\]

2.152 > 1.43 OK

check negative moment reinforcement:

1.33 x factored ultimate moment \( = 0.00 \) kip-ft

Cracking moment \( = 3.09 \) kip-ft

min from (1.33 x factored -\( M_u \) and \( M_{cr} \)) \( = 0.00 \) kip-ft

Check:

\[
-\varphi M_n > \min(-M_u, M_{cr})
\]

1.269 > 0.00 OK
**Typical Panel Type A, Panel Thickness 6", 5'x12', Vertical Rebar**

**Shear on Panel**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness</td>
<td>(t_{\text{panel}} = 6) in</td>
</tr>
<tr>
<td>Concrete Cover</td>
<td>(C_{\text{panel}} = 2.5) in</td>
</tr>
<tr>
<td>Concrete Strength</td>
<td>(f'_c = 4.0) ksi</td>
</tr>
<tr>
<td>Reinforcement Strength</td>
<td>(f_y = 60) ksi</td>
</tr>
<tr>
<td>Bar Size</td>
<td>#4</td>
</tr>
<tr>
<td>Rebar Diameter</td>
<td>(d_b = 0.500) in</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>(A_b = 0.196) in(^2)</td>
</tr>
<tr>
<td>Rebar spacing</td>
<td>(s = 18) in</td>
</tr>
<tr>
<td>Width of the design section</td>
<td>(b = 12) in</td>
</tr>
<tr>
<td>Area of Steel per Design Strip</td>
<td>(A_s = 0.131) in(^2)</td>
</tr>
</tbody>
</table>

\[ c = \frac{A_s \cdot f_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b} \]
\[ d_v = t_{\text{panel}} - \frac{c}{2} - C_{\text{panel}} - \frac{d_b}{2} \]
\[ d_v = 3.137 \text{ in} \]

**AASHTO 5.7.3.3**

\(V_n\) shall be the lesser of:

1. \(V_n = V_c + V_s\)
2. \(V_n = 0.25 \cdot f'_c \cdot b_v \cdot d_v\)

\[ V_c = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f'_c \cdot b_v \cdot d_v} \quad V_c = 4.758 \text{ Kips} \]
\[ V_s = \frac{A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha}{s} \quad V_s = 2.053 \text{ Kips} \]

(1) \(V_n = 6.811 \text{ Kips}\)

(2) \(V_n = 37.641 \text{ Kips}\)

From (1) and (2) \(V_n\) should be:
\[ \phi \cdot V_n = 0.9 \cdot V_n = 6.130 \text{ Kips} \]

**Shear Force:**

From Staad Pro Result \(V_{\text{max}} = 0.15\) N/mm\(^2\)
\[ = 21.76 \text{ lbs/in}^2 \]

\(V_{\text{max}} = 1566.41 \text{ lbs} \quad \text{OK, } V_{\text{max}} \text{ less than } V_n\)

\(c = A_\theta \cdot f_\theta \cdot \alpha \cdot \beta_\theta \cdot b\)
\(d_v = t_{\text{panel}} - \frac{c}{2} - C_{\text{panel}} - \frac{d_b}{2}\)
\(d_v = 3.137 \text{ in}\)
STANDARD "A" PANEL

SECTION A-A

LOOP EMBED DETAILS

REINFORCING MESH CONNECTOR BAR DETAIL

NOTE:
Typical Panel Reinforcement Shown. Alternate Reinforcement May Be Used As Long As Minimum Cross Sectional Areas Are Equivalent. 18" Max Spacing Between Bars.
5’ x 12’ STANDARD X PANEL
Typical Panel Type X, Panel Thickness 6", 5'x12', Horizontal Rebar

Panel Strength Design
Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

Panel Thickness  \( t_{\text{panel}} = 6 \text{ in} \)
Finishing Thickness  \( = 1 \text{ in} \)
Additional Load from Finishes  \( = 0.0125 \text{ Kip/ft}^2 \)
Concrete Cover  \( c_{\text{panel}} = 1.5 \text{ in} \)
Vertical Bar Size  \#4
Rebar Diameter  \( d_b = 0.5 \text{ in} \)
Rebar Area  \( A_b = 0.196 \text{ in}^2 \)
Rebar spacing  \( s = 7.5 \text{ in} \)
Width of the design section  \( b = 12 \text{ in} \)
Effective depth of section  \( d_s = 4.25 \text{ in} \)
Effective depth of section for Negative Moment  \( d_{s(M^-)} = 1.75 \text{ in} \)
Correction factor for source aggregate  \( k_1 = 1 \text{ in} \)
Concrete Density  \( W_c = 0.15 \text{ kcf} \)
Concrete Strength  \( f'_c = 4.00 \text{ ksi} \)
Reinforcement Strength  \( f_y = 60 \text{ ksi} \)
Modulus Elasticity of concrete  \( E_c = 4266 \text{ ksi} \)
Modulus Elasticity of reinforcement  \( E_s = 29000 \text{ ksi} \)
Modular Ratio  \( n = 6.798 \)
Area of Steel per Design Strip  \( A_s = 0.314 \text{ in}^2 \)

\[ +M_u = 3.04 \text{ kip-ft} \]
\[ -M_u = 1.47 \text{ kip-ft} \]

Resistance factor for tension-controlled section  \( \varphi_{\text{STR}} = 0.90 \text{ AASHTO 5.5.4.2} \)

Positive Moment Capacity
Depth of equivalent stress block  \( a = \frac{A_s f_y}{0.85 f'_c b} = 0.462 \text{ in} \)
Factored flexural resistance  \( +\varphi M_n = \varphi A_s f_y \left( d_s - \frac{a}{2} \right) = 5.682 \text{ kip-ft} \)
Check  \( +\varphi M_n > +M_u \)

Negative Moment Capacity
Depth of equivalent stress block  \( a = \frac{A_s f_y}{0.85 f'_c b} = 0.462 \text{ in} \)
Factored flexural resistance  \( -\varphi M_n = \varphi A_s f_y \left( d_{s(M^-)} - \frac{a}{2} \right) = 2.147 \text{ kip-ft} \)
Check  \( -\varphi M_n > -M_u \)
Minimum Reinforcement

AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( Mr = \varphi Mn \), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

\[
M_{cr} = \gamma_3 \left( (\gamma_1 f_r + \gamma_2 f_{pe}) S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \right)
\]

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

where:

- flexural cracking variability factor \( \gamma_1 = 1.20 \) (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength \( \gamma_3 = 0.67 \) (A615 steel)
- concrete density modification factor \( \lambda = 1.00 \)

- modulus of rapture \( f_r = 0.24 \lambda \sqrt{f_c} \) \( f_r = 0.48 \) ksi AASHTO 5.4.2.6

- section modulus of design section \( S_c = \frac{bh^2}{6} = \frac{bt_{panel}^2}{6} \) \( S_c = 72.00 \) in³

Cracking moment

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

\[ M_{cr} = 2.32 \text{ kip-ft} \]

check positive moment reinforcement:

- 1.33 x factored ultimate moment = 4.04 kip-ft
- Cracking moment = 2.32 kip-ft
- min from (1.33 x factored +\( M_u \) and \( M_{cr} \)) = 2.32 kip-ft

Check: \[ +\varphi M_n > \min( +M_u , M_{cr} ) \] \[ 5.682 > 2.32 \text{ OK} \]

check negative moment reinforcement:

- 1.33 x factored ultimate moment = 1.95 kip-ft
- Cracking moment = 2.32 kip-ft
- min from (1.33 x factored -\( M_u \) and \( M_{cr} \)) = 1.95 kip-ft

Check: \[ -\varphi M_n > \min( -M_u , M_{cr} ) \] \[ 2.147 > 1.95 \text{ OK} \]
Typical Panel Type X, Panel Thickness 6", 5'x12', Horizontal Rebar

Shear on Panel

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness</td>
<td>t_{panel} = 6 in</td>
</tr>
<tr>
<td>Concrete Cover</td>
<td>C_{panel} = 1.5 in</td>
</tr>
<tr>
<td>Concrete Strength</td>
<td>f'_{c} = 4.0 ksi</td>
</tr>
<tr>
<td>Reinforcement Strength</td>
<td>f_{y} = 60 ksi</td>
</tr>
<tr>
<td>Bar Size</td>
<td>#4</td>
</tr>
<tr>
<td>Rebar Diameter</td>
<td>d_{b} = 0.500 in</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>A_{b} = 0.196 in^2</td>
</tr>
<tr>
<td>Rebar spacing</td>
<td>s = 8 in</td>
</tr>
<tr>
<td>Width of the design section</td>
<td>b = 12 in</td>
</tr>
<tr>
<td>Area of Steel per Design Strip</td>
<td>A_{s} = 0.314 in^2</td>
</tr>
</tbody>
</table>

\[
c = \frac{A_{s} \cdot f_{y}}{a_{1} \cdot f'_{c} \cdot \beta_{1} \cdot b}
\]

\[
d_{v} = t_{panel} - \frac{c}{2} - C_{panel} - \frac{d_{b}}{2}
\]

\[
V_{n} = \begin{cases} 
V_{c} = 6.034 \text{ Kips} \\
V_{s} = 6.249 \text{ Kips}
\end{cases}
\]

(1) Vn = 12.283 Kips

(2) Vn = 47.739 Kips

From (1) and (2) Vn should be: Vn = 12.283 Kips

\[
\phi \cdot Vn = 0.9 \cdot Vn = 11.055 \text{ Kips}
\]

Shear Force:

From Staad Pro Result Vmax = 0.336 N/mm^2
= 48.73 lbs/in^3

Shear Section Area = b x t_{panel}
= 12 x 6

Vmax = 3508.76 lbs
= 3.509 Kips  OK, Vmax less than Vn
Typical Panel Type X, Panel Thickness 6", 5'x12', Vertical Rebar

Panel Strength Design
Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

Panel Thickness \( t_{\text{panel}} = 6 \text{ in} \)
Finishing Thickness \( C_{\text{panel}} = 1.5 \text{ in} \)
Additional Load from Finishes \( = 0.0125 \text{ Kip/ft}^2 \)
Vertical Bar Size \( \#4 \)
Rebar Diameter \( d_b = 0.5 \text{ in} \)
Rebar Area \( A_b = 0.196 \text{ in}^2 \)
Rebar spacing \( s = 12 \text{ in} \)
Width of the design section \( b = 12 \text{ in} \)
Effective depth of section \( d_s = 3.75 \text{ in} \)
Effective depth of section for Negative Moment \( d_s(M^{-}) = 2.25 \text{ in} \)
Correction factor for source aggregate \( k_1 = 1 \text{ in} \)
Concrete Cover \( C = 1.5 \text{ in} \)
Concrete Density \( W_c = 0.15 \text{ kcf} \)
Concrete Strength \( f'_{\text{c}} = 4.00 \text{ ksi} \)
Reinforcement Strength \( f_y = 60 \text{ ksi} \)
Modulus Elasticity of concrete \( E_c = 4266 \text{ ksi} \)
Modulus Elasticity of reinforcement \( E_s = 29000 \text{ ksi} \)
Modular Ratio \( n = 6.798 \text{ (Concrete Class A compressive strength)} \)
Area of Steel per Design Strip \( A_s = 0.196 \text{ in}^2 \)

\[ +M_u = 2.13 \text{ kip-ft} \]
\[ -M_u = 0.00 \text{ kip-ft} \]

Resistance factor for tension-controlled section \( \phi_{\text{STR}} = 0.90 \text{ (FK = 1.35)} \)

Positive Moment Capacity
Depth of equivalent stress block \( a = \frac{A_s f_y}{0.85 f'_{\text{c}} b} = 0.289 \text{ in} \)
Factored flexural resistance \( +\phi M_n = \phi A_s f_y \left( d_s - \frac{a}{2} \right) = 3.186 \text{ kip-ft} \)
Check \( +\phi M_n > +M_u \quad 3.186 > 2.13 \text{ OK} \)

Negative Moment Capacity
Depth of equivalent stress block \( a = \frac{A_s f_y}{0.85 f'_{\text{c}} b} = 0.289 \text{ in} \)
Factored flexural resistance \( -\phi M_n = \phi A_s f_y \left( d_s(M^{-}) - \frac{a}{2} \right) = 1.860 \text{ kip-ft} \)
Check \( -\phi M_n > -M_u \quad 1.860 > 0.00 \text{ OK} \)
**Minimum Reinforcement**  
AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r = \phi M_n \), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

\[
M_{cr} = \gamma_3 \left[ (\gamma_1 f_c + \gamma_2 f_{cpe}) S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \right]
\]

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_c S_c
\]

where:

- flexural cracking variability factor
- ratio of specified min. yield strength to ultimate tensile strength
- concrete density modification factor

\[
\gamma_1 = 1.60 \quad \text{(non-segmental brg)}
\]

\[
\gamma_3 = 0.67 \quad \text{(A615 steel)}
\]

\[
\lambda = 1.00 \quad \text{AASHTO 5.4.2.8}
\]

- modulus of rupture
- section modulus of design section

\[
f_r = 0.24 \lambda \sqrt{f_c'}
\]

\[
S_c = \frac{bh^2}{6} = \frac{b t_{panel}^2}{6}
\]

\[
S_c = 72.00 \quad \text{in}^3
\]

Cracking moment

\[
M_{cr} = \gamma_3 \gamma_1 f_c S_c
\]

\[
M_{cr} = 3.09 \quad \text{kip-ft}
\]

check positive moment reinforcement:

1.33 x factored ultimate moment

\[
= 2.83 \quad \text{kip-ft}
\]

Cracking moment

\[
= 3.09 \quad \text{kip-ft}
\]

min from (1.33 x factored +\( M_u \) and \( M_{cr} \))

\[
= 2.83 \quad \text{kip-ft}
\]

Check:

\[
+\phi M_n > \min( +M_u , M_{cr})
\]

\[
3.186 > 2.83 \quad \text{OK}
\]

check negative moment reinforcement:

1.33 x factored ultimate moment

\[
= 0.00 \quad \text{kip-ft}
\]

Cracking moment

\[
= 3.09 \quad \text{kip-ft}
\]

min from (1.33 x factored -\( M_u \) and \( M_{cr} \))

\[
= 0.00 \quad \text{kip-ft}
\]

Check:

\[
-\phi M_n > \min ( -M_u , M_{cr})
\]

\[
1.860 > 0.00 \quad \text{OK}
\]
Typical Panel Type X, Panel Thickness 6", 5'x12', Vertical Rebar

Shear on Panel

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness t&lt;sub&gt;panel&lt;/sub&gt;</td>
<td>6 in</td>
</tr>
<tr>
<td>Concrete Cover C&lt;sub&gt;panel&lt;/sub&gt;</td>
<td>2.5 in</td>
</tr>
<tr>
<td>Concrete Strength f&lt;sub&gt;c&lt;/sub&gt;'</td>
<td>4.0 ksi (Concrete Class A compressive strength)</td>
</tr>
<tr>
<td>Reinforcement Strength f&lt;sub&gt;y&lt;/sub&gt;</td>
<td>60 ksi (Minimum yield strength of grade 60 steel)</td>
</tr>
<tr>
<td>Bar Size #4</td>
<td></td>
</tr>
<tr>
<td>Rebar Diameter d&lt;sub&gt;b&lt;/sub&gt;</td>
<td>0.500 in</td>
</tr>
<tr>
<td>Rebar Area A&lt;sub&gt;b&lt;/sub&gt;</td>
<td>0.196 in&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>Rebar spacing s</td>
<td>12 in</td>
</tr>
<tr>
<td>Width of the design section b</td>
<td>12 in</td>
</tr>
<tr>
<td>Area of Steel per Design Strip A&lt;sub&gt;s&lt;/sub&gt;</td>
<td>0.196 in&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

\[
c = \frac{A_s \cdot f_s}{\alpha_1 \cdot f_c' \cdot \beta_1 \cdot b}
\]

\[
d_v = t_{panel} - \frac{c}{2} - C_{panel} - \frac{d_b}{2}
\]

AASHTO 5.7.3.3

V<sub>n</sub> shall be the lesser of:

1. \[V_c = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f_c'} \cdot b_v \cdot d_v\]
   \[V_c = 4.672\text{ Kips}\]
2. \[V_s = \frac{A_s \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha}{s}\]
   \[V_s = 3.024\text{ Kips}\]

\[\text{From (1) and (2) Vn should be:}\]

\[\phi \cdot V_n = 0.9 \cdot V_n = 6.926\text{ Kips}\]

Shear Force:

\[\text{From Staad Pro Result Vmax} = 0.298\text{ N/mm}^2\]
\[= 43.22\text{ lbs/in}^2\]

\[\text{Shear Section Area} = b \times t_{panel}\]
\[= 12 \times 6\]
\[V_{max} = 3111.94\text{ lbs}\]
\[= 3.112\text{ Kips}\] OK, Vmax less than Vn
5’ x 12’ STANDARD Y PANEL
**Typical Panel Type Y, Panel Thickness 6", 5'x12', Horizontal Rebar**

**Panel Strength Design**
Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

Panel Thickness \( t_{\text{panel}} = 6 \) in  
Finishing Thickness \( = 1 \) in  
Additional Load from Finishes \( = 0.0125 \text{ Kip/ft}^2 \)  
Concrete Cover \( C_{\text{panel}} = 1.5 \) in  
Vertical Bar Size \( = #4 \)  
Rebar Diameter \( d_b = 0.5 \) in \( \text{ and } \) \( d_{bh} = 0 \) in  
Rebar Area \( A_b = 0.196 \text{ in}^2 \) \( A_b = \frac{1}{4} \pi d_b^2 \)  
Rebar spacing \( s = 6.66 \) in  
Width of the design section \( b = 12 \) in  
Effective depth of section \( d_s = 4.25 \) in  
Effective depth of section for Negative Moment \( d_s(M^-) = t_{\text{panel}} - c_{\text{panel}} - d_{bh} - \frac{1}{2} d_b \)  
Correction factor for source aggregate \( k_1 = 1 \) in AASHTO 5.4.2.4  
Concrete Density \( W_c = 0.15 \text{ kcf} \)  
Concrete Strength \( f'c = 4.00 \text{ ksi} \) (Concrete Class A compressive strength) \( f_y = 60 \text{ ksi} \) (Minimum yield strength of grade 60 steel) \( E_c = 120000 k_1 W_c^2 f'c^{0.33} \)  
Modulus Elasticity of concrete \( E_c = 4266 \text{ ksi} \) \( E_s = 29000 \text{ ksi} \)  
Modular Ratio \( n = 6.798 \)  
Area of Steel per Design Strip \( A_s = 0.354 \text{ in}^2 \) \( A_s = b \left( \frac{A_b}{s} \right) \)  

\[ a = \frac{A_s f_y}{0.85 f'c b} = 0.520 \text{ in} \]

Factored flexural resistance \( +\varphi M_n = \varphi A_s f_y \left( d_s - \frac{a}{2} \right) = 6.352 \text{ kip-ft} \)  
Check \( +\varphi M_n > +M_u \) \( 6.352 > 3.73 \text{ OK} \)

\[ a = \frac{A_s f_y}{0.85 f'c b} = 0.520 \text{ in} \]

Factored flexural resistance \( -\varphi M_n = \varphi A_s f_y \left( d_s(M^-) - \frac{a}{2} \right) = 2.372 \text{ kip-ft} \)  
Check \( -\varphi M_n > -M_u \) \( 2.372 > 1.80 \text{ OK} \)
Minimum Reinforcement  

AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r = \phi M_n \), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

\[
M_{cr} = \gamma_3 \left( (\gamma_1 f_r + \gamma_2 f_{pc}) s_c - M_{dnc} \left( \frac{s_c}{s_{nc}} - 1 \right) \right)
\]

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[ M_{cr} = \gamma_3 \gamma_1 f_r s_c \]

where:

- flexural cracking variability factor \( \gamma_1 \) = 1.20 (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength \( \gamma_3 \) = 0.67 (A615 steel)
- concrete density modification factor \( \lambda \) = 1.00

modulus of rupture \( f_r = 0.24 \lambda \sqrt{f_c^t} \)

section modulus of design section \( S_c = \frac{bh^2}{6} = \frac{b t_{panel}^2}{6} \)

Cracking moment

\[ M_{cr} = \gamma_3 \gamma_1 f_r s_c \]

\[ M_{cr} = 2.32 \text{ kip-ft} \]

check positive moment reinforcement:

- 1.33 x factored ultimate moment
  \[ = 4.96 \text{ kip-ft} \]
- Cracking moment
  \[ = 2.32 \text{ kip-ft} \]
- min from (1.33 x factored +\( M_u \) and \( M_{cr} \))
  \[ = 2.32 \text{ kip-ft} \]

Check: \[ +\phi M_n > \min(+M_u, M_{cr}) \]

\[ 6.352 > 2.32 \text{ OK} \]

check negative moment reinforcement:

- 1.33 x factored ultimate moment
  \[ = 2.39 \text{ kip-ft} \]
- Cracking moment
  \[ = 2.32 \text{ kip-ft} \]
- min from (1.33 x factored -\( M_u \) and \( M_{cr} \))
  \[ = 2.32 \text{ kip-ft} \]

Check: \[ -\phi M_n > \min(-M_u, M_{cr}) \]

\[ 2.372 > 2.32 \text{ OK} \]
**Typical Panel Type Y, Panel Thickness 6", 5'x12', Horizontal Rebar**

**Shear on Panel**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Thickness</td>
<td>( t_{\text{panel}} )</td>
<td>6 in</td>
</tr>
<tr>
<td>Concrete Cover</td>
<td>( C_{\text{panel}} )</td>
<td>1.5 in</td>
</tr>
<tr>
<td>Concrete Strength</td>
<td>( f'_c )</td>
<td>4.0 ksi</td>
</tr>
<tr>
<td>Reinforcement Strength</td>
<td>( f_y )</td>
<td>60 ksi</td>
</tr>
<tr>
<td>Bar Size</td>
<td>#4</td>
<td></td>
</tr>
<tr>
<td>Rebar Diameter</td>
<td>( d_b )</td>
<td>0.500 in</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>( A_{b} )</td>
<td>0.196 in²</td>
</tr>
<tr>
<td>Rebar spacing</td>
<td>( s )</td>
<td>7 in</td>
</tr>
<tr>
<td>Width of the design section</td>
<td>( b )</td>
<td>12 in</td>
</tr>
<tr>
<td>Area of Steel per Design Strip</td>
<td>( A_s )</td>
<td>0.354 in²</td>
</tr>
<tr>
<td></td>
<td>( c )</td>
<td>0.612 in</td>
</tr>
<tr>
<td></td>
<td>( d_{v} )</td>
<td>3.944 in</td>
</tr>
<tr>
<td></td>
<td>( V_{n} )</td>
<td>12.959 Kips</td>
</tr>
</tbody>
</table>

**AASHTO 5.7.3.3**

\( V_n \) shall be the lesser of:

1. \( V_n = V_c + V_s \)
2. \( V_n = 0.25 \cdot f'_c \cdot b_v \cdot d_v \)

\( V_c = 5.982 \) Kips
\( V_s = 6.977 \) Kips

(1) \( V_n = 12.959 \) Kips
(2) \( V_n = 47.328 \) Kips

From (1) and (2) \( V_n \) should be:

\( V_n = 12.959 \) Kips

\( \phi \cdot V_n = 0.9 \cdot V_n = 11.663 \) Kips

**Shear Force:**

From Staad Pro Result \( V_{\text{max}} = 0.349 \) N/mm²
\( = 50.62 \) lbs/in²

**Shear Section Area**

\( V_{\text{max}} = 3644.51 \) lbs
\( = 3.645 \) Kips

OK, \( V_{\text{max}} \) less than \( V_n \)
**Typical Panel Type Y, Panel Thickness 6", 5'x12', Vertical Rebar**

**Panel Strength Design**

Design of panel reinforcement per AASHTO LRFD Bridge Design Specification 5.7.3 (typical rectangular beam design)

The following design method can be used for normal weight concrete with specified compressive strength up to 15.0 ksi. This Design are to present the information about the acceptable panel reinforcement sizes and spacing.

<table>
<thead>
<tr>
<th>Panel Thickness</th>
<th>( t_{\text{panel}} ) = 6 in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Finishing Thickness</td>
<td>( t_{\text{finish}} ) = 1 in</td>
</tr>
<tr>
<td>Additional Load from Finishes</td>
<td>0.0125 Kip/ft²</td>
</tr>
<tr>
<td>Concrete Cover</td>
<td>( C_{\text{panel}} ) = 1.5 in</td>
</tr>
<tr>
<td>Vertical Bar Size</td>
<td>#4</td>
</tr>
<tr>
<td>Rebar Diameter</td>
<td>( d_b ) = 0.5 in</td>
</tr>
<tr>
<td>Rebar Area</td>
<td>( A_b ) = 0.196 in²</td>
</tr>
<tr>
<td>Rebar spacing</td>
<td>( s ) = 12 in</td>
</tr>
<tr>
<td>Width of the design section</td>
<td>( b ) = 12 in</td>
</tr>
<tr>
<td>Effective depth of section</td>
<td>( d_s ) = 3.75 in</td>
</tr>
<tr>
<td>Effective depth of section for Negative Moment</td>
<td>( d_{s(M^-)} ) = 2.25 in</td>
</tr>
<tr>
<td>Correction factor for source aggregate</td>
<td>( k_1 ) = 1 in</td>
</tr>
<tr>
<td>Concrete Density</td>
<td>( W_c ) = 0.15 kcf</td>
</tr>
<tr>
<td>Concrete Strength</td>
<td>( f'_c ) = 4.00 ksi</td>
</tr>
<tr>
<td>Reinforcement Strength</td>
<td>( f_y ) = 60 ksi</td>
</tr>
<tr>
<td>Modulus Elasticity of concrete</td>
<td>( E_c ) = 4266 ksi</td>
</tr>
<tr>
<td>Modulus Elasticity of reinforcement</td>
<td>( E_s ) = 29000 ksi</td>
</tr>
<tr>
<td>Modular Ratio</td>
<td>( n ) = 6.798</td>
</tr>
<tr>
<td>Area of Steel per Design Strip</td>
<td>( A_s ) = 0.196 in²</td>
</tr>
</tbody>
</table>

\[ a = \frac{A_s f_y}{0.85 f'_c b} \]

\[ 0.289 \text{ in} \]

\[ + \varphi M_n = \varphi A_s f_y \left( d_s - \frac{a}{2} \right) = 3.186 \text{ kip-ft} \]

\[ + \varphi M_n > + M_u \]

\[ 3.186 > 2.61 \text{ OK} \]

**Positive Moment Capacity**

\[ a = \frac{A_s f_y}{0.85 f'_c b} \]

\[ 0.289 \text{ in} \]

\[ - \varphi M_n = \varphi A_s f_y \left( d_{s(M^-)} - \frac{a}{2} \right) = 1.860 \text{ kip-ft} \]

\[ - \varphi M_n > - M_u \]

\[ 1.860 > 0.00 \text{ OK} \]

**Negative Moment Capacity**
Minimum Reinforcement AASHTO 5.7.3.3.2

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r = \phi M_n \), at least equal to the lesser of:

1.33 times the positive factored ultimate moment

Cracking moment

\[
M_{cr} = \gamma_3 \left[ \left( \gamma_1 f_r + \gamma_2 y_{cpe} \right) S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \right]
\]

AASHTO 5.7.3.3.2-1

When simplified by removing all values applicable to prestressed and non composite section, this equation becomes the following:

\[
M_{cr} = \gamma_3 \gamma_1 f_r S_c
\]

where:

- flexural cracking variability factor \( \gamma_1 = 1.60 \) (non-segmental brg)
- ratio of specified min. yield strength to ultimate tensile strength \( \gamma_3 = 0.67 \) (A615 steel)
- concrete density modification factor \( \lambda = 1.00 \) AASHTO 5.4.2.8

modulus of rapture

\[ f_r = 0.24 \lambda \sqrt{f_c} \]

\[ f_r = 0.48 \text{ ksi} \] AASHTO 5.4.2.6

section modulus of design section

\[ S_c = \frac{bh^2}{6} = \frac{bt_{panel}^2}{6} \]

\[ S_c = 72.00 \text{ in}^3 \]

Cracking moment

\[ M_{cr} = \gamma_3 \gamma_1 f_r S_c \]

\[ M_{cr} = 3.09 \text{ kip-ft} \]

check positive moment reinforcement:

\[ 1.33 \times \text{factored ultimate moment} = 3.47 \text{ kip-ft} \]

\[ \text{Cracking moment} = 3.09 \text{ kip-ft} \]

\[ \text{min from (1.33 x factored +} M_u \text{ and } M_{cr}) = 3.09 \text{ kip-ft} \]

Check:

\[ \phi M_n > \min(\phi M_u, M_{cr}) \]

\[ 3.186 > 3.09 \text{ OK} \]

check negative moment reinforcement:

\[ 1.33 \times \text{factored ultimate moment} = 0.00 \text{ kip-ft} \]

\[ \text{Cracking moment} = 3.09 \text{ kip-ft} \]

\[ \text{min from (1.33 x factored -} M_u \text{ and } M_{cr}) = 0.00 \text{ kip-ft} \]

Check:

\[ -\phi M_n > \min(-\phi M_u, M_{cr}) \]

\[ 1.860 > 0.00 \text{ OK} \]
Typical Panel Type Y, Panel Thickness 6", 5'x12', Vertical Rebar

Shear on Panel

- **Panel Thickness** \( t_{\text{panel}} = 6 \text{ in} \)
- **Concrete Cover** \( c_{\text{panel}} = 2.5 \text{ in} \)
- **Concrete Strength** \( f'_c = 4.0 \text{ ksi} \) (Concrete Class A compressive strength)
- **Reinforcement Strength** \( f_y = 60 \text{ ksi} \) (Minimum yield strength of grade 60 steel)
- **Bar Size** #4
- **Rebar Diameter** \( d_b = 0.500 \text{ in} \)
- **Rebar Area** \( A_b = 0.196 \text{ in}^2 \)
- **Rebar spacing** \( s = 12 \text{ in} \)
- **Width of the design section** \( b = 12 \text{ in} \)
- **Area of Steel per Design Strip** \( A_s = 0.196 \text{ in}^2 \)

\[
c = \frac{A_s \cdot f_s}{A_v \cdot f'_c \cdot \beta_1 \cdot b}
\]

\[
d_v = t_{\text{panel}} - \frac{c}{2} - c_{\text{panel}} - \frac{d_b}{2}
\]

**AASHTO 5.7.3.3**

\( V_n \) shall be the lesser of:

1. \( V_n = V_c + V_s \)

\[
V_c = 0.0316 \cdot \beta \cdot \alpha \cdot \sqrt{f'_c} \cdot b_v \cdot d_v \]

\[
V_c = 4.672 \text{ Kips}
\]

\[
V_s = \frac{A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha}{s}
\]

\[
V_s = 3.024 \text{ Kips}
\]

(1) \( V_n = 7.696 \text{ Kips} \)

(2) \( V_n = 0.25 \cdot f'_c \cdot b_v \cdot d_v \)

(2) \( V_n = 36.962 \text{ Kips} \)

From (1) and (2) \( V_n \) should be:

\[
\phi \cdot V_n = 0.9 \cdot V_n = 6.926 \text{ Kips}
\]

**Shear Force:**

From Staad Pro Result \( V_{\text{max}} = 0.365 \text{ N/mm}^2 \)

\( = 52.94 \text{ lbs/in}^2 \)

Shear Section Area \( = b \times t_{\text{panel}} \)

\( = 12 \times 6 \text{ in} \)

\( V_{\text{max}} = 3811.60 \text{ lbs} \)

\( = 3.812 \text{ Kips} \) **OK, Vmax less than Vn**
1.1.8 Face Unit Bearing Devices
1.1.9 Facing Unit Filter Cloth Specification
Mirafi® FW402 is composed of high-tenacity monofilament polypropylene yarns, which are woven into a stable network such that the yarns retain their relative position. Mirafi® FW402 geotextile is inert to biological degradation and resists naturally encountered chemicals, alkalis, and acids.

TenCate Geosynthetics Americas Laboratories are accredited by Geosynthetic Accreditation Institute – Laboratory Accreditation Program (GAI-LAP). NTPEP Listed

<table>
<thead>
<tr>
<th>MECHANICAL PROPERTIES</th>
<th>TEST METHOD</th>
<th>UNIT</th>
<th>MINIMUM AVERAGE ROLL VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>MD</td>
</tr>
<tr>
<td>Grab Tensile Strength</td>
<td>ASTM D4632</td>
<td>lbs (N)</td>
<td>365</td>
</tr>
<tr>
<td>Grab Tensile Elongation</td>
<td>ASTM D4632</td>
<td>%</td>
<td>24</td>
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<tr>
<td>Trapezoid Tear Strength</td>
<td>ASTM D4533</td>
<td>lbs (N)</td>
<td>115</td>
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<tr>
<td>CBR Puncture Strength</td>
<td>ASTM D6241</td>
<td>lbs (N)</td>
<td>675</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>MECHANICAL PROPERTIES</th>
<th>TEST METHOD</th>
<th>UNIT</th>
<th>MINIMUM ROLL VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percent Open Area</td>
<td>COE-02215</td>
<td>%</td>
<td>10</td>
</tr>
<tr>
<td>Permittivity</td>
<td>ASTM D4491</td>
<td>sec⁻¹</td>
<td>2.1</td>
</tr>
<tr>
<td>Flow Rate</td>
<td>ASTM D4491</td>
<td>gal/min/ft² (l/min/m²)</td>
<td>145 (5907)</td>
</tr>
<tr>
<td><strong>Maximum Opening Size</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Apparent Opening Size (AOS)</td>
<td>ASTM D4751</td>
<td>U.S. Sieve (mm)</td>
<td>40 (0.425)</td>
</tr>
<tr>
<td>UV Resistance (at 500 Hours)</td>
<td>ASTM D4355</td>
<td>% strength retained</td>
<td>Minimum Test Value 90</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>PHYSICAL PROPERTIES</th>
<th>UNIT</th>
<th>TYPICAL ROLL VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roll Dimensions (width x length)</td>
<td>ft (mm)</td>
<td>12.5 x 300 (3.8 x 91)</td>
</tr>
<tr>
<td>Roll Area</td>
<td>yd² (m²)</td>
<td>417 (348)</td>
</tr>
<tr>
<td>Roll Weight</td>
<td>lbs (kg)</td>
<td>178 (81)</td>
</tr>
</tbody>
</table>

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1.1.10 Facing Unit Aesthetic Options
Fracture Fin with Buttresses

Vertical Flute with Artwork
Seashells

Offset Vertical Flute
Integral Color Raised Relief with Exposed Aggregate

Fracture Fin with Pattern
1.2

Inextensible Reinforcements
1.2.3 Soil Reinforcement Properties
ASTM A1064
Standard Specification for Carbon-Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete

This standard is issued under the fixed designation A1064/A1064M; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This specification covers carbon-steel wire and welded wire reinforcement produced from hot-rolled rod to be used for the reinforcement of concrete. The steel wire is cold-worked, drawn or rolled, plain (non-deformed, as-drawn or galvanized), or deformed. Welded wire reinforcement is made from plain or deformed wire, or a combination of plain and deformed wire. Common wire sizes and dimensions are given in Table 1, Table 2, Table 3, and Table 4. Actual wire sizes are not restricted to those shown in the tables.

NOTE 1—Welded wire for concrete reinforcement has historically been described by various terms: welded wire fabric, WWF, fabric, and mesh. The wire reinforcement industry has adopted the term welded wire reinforcement (WWR) as being more representative of the applications of the products being manufactured. Therefore, the term welded wire fabric has been replaced with the term welded wire reinforcement in this specification and in related specifications.

1.2 The values stated in either inch-pound or SI units are to be regarded separately as standard. Within the text the SI units are shown in brackets (except in Table 2 and Table 4). The values stated in each system are not exact equivalents; therefore, each system must be used independently of the other. Combining values may result in nonconformance with the specification.

1.3 This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

1.4 This international standard was developed in accordance with internationally recognized principles on standardization established in the Decision on Principles for the Development of International Standards, Guides and Recommendations issued by the World Trade Organization Technical Barriers to Trade (TBT) Committee.

2. Referenced Documents

2.1 ASTM Standards:
A370 Test Methods and Definitions for Mechanical Testing of Steel Products
A641/A641M Specification for Zinc-Coated (Galvanized) Carbon Steel Wire
E83 Practice for Verification and Classification of Extensometer Systems
2.2 U.S. Military Standard:
MIL-STD-129 Marking for Shipment and Storage
2.3 U.S. Military Standard:
Fed. Std. No. 123 Marking for Shipments (Civil Agencies)
2.4 American Concrete Institute (ACI) Standard:
ACI 318 Building Code Requirements for Structural Concrete
2.5 Adjuncts:
Weld Tester Drawing

3. Terminology

3.1 Definitions of Terms Specific to This Specification:
3.1.1 convoluted wire—plain wire for welded wire reinforcement that is formed into a sinusoidal wave shape; deformed wire is not subject to convolution unless agreed upon by the purchaser and manufacturer.
3.1.1.1 Discussion—The wire is used in the manufacture of cages for certain applications of concrete pipe reinforcement.
3.1.2 deformed wire and deformed welded wire reinforcement—a material composed of cold-worked deformed steel wire as cold-drawn or cold-rolled from hot-rolled steel rod.

2 For referenced ASTM standards, visit the ASTM website, www.astm.org, or contact ASTM Customer Service at service@astm.org. For Annual Book of ASTM Standards volume information, refer to the standard’s Document Summary page on the ASTM website.
4 Available from American Concrete Institute (ACI), P.O. Box 9094, Farmington Hills, MI 48333-9094, http://www.concrete.org.

*A Summary of Changes section appears at the end of this standard
### Table 1 Dimensional Requirements for Plain Wire—Inch-Pound Units

<table>
<thead>
<tr>
<th>Size Number&lt;sup&gt;B, C, D&lt;/sup&gt;</th>
<th>Nominal Diameter in. [mm]&lt;sup&gt;E&lt;/sup&gt;</th>
<th>Nominal Area in.² [mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>W 0.5</td>
<td>0.080 [2.03]</td>
<td>0.005 [3.23]</td>
</tr>
<tr>
<td>W 1.2</td>
<td>0.124 [3.14]</td>
<td>0.012 [7.74]</td>
</tr>
<tr>
<td>W 1.4</td>
<td>0.124 [3.14]</td>
<td>0.014 [8.03]</td>
</tr>
<tr>
<td>W 2</td>
<td>0.160 [4.05]</td>
<td>0.020 [12.9]</td>
</tr>
<tr>
<td>W 2.5</td>
<td>0.178 [4.53]</td>
<td>0.025 [16.1]</td>
</tr>
<tr>
<td>W 2.9</td>
<td>0.192 [4.88]</td>
<td>0.029 [18.7]</td>
</tr>
<tr>
<td>W 3.5</td>
<td>0.211 [5.36]</td>
<td>0.035 [22.6]</td>
</tr>
<tr>
<td>W 4</td>
<td>0.226 [5.73]</td>
<td>0.040 [25.8]</td>
</tr>
<tr>
<td>W 4.5</td>
<td>0.239 [6.08]</td>
<td>0.045 [29.0]</td>
</tr>
<tr>
<td>W 5</td>
<td>0.252 [6.41]</td>
<td>0.050 [32.3]</td>
</tr>
<tr>
<td>W 5.5</td>
<td>0.265 [6.72]</td>
<td>0.055 [35.5]</td>
</tr>
<tr>
<td>W 6</td>
<td>0.276 [7.02]</td>
<td>0.060 [38.7]</td>
</tr>
<tr>
<td>W 8</td>
<td>0.319 [8.11]</td>
<td>0.080 [51.6]</td>
</tr>
<tr>
<td>W 10</td>
<td>0.357 [9.06]</td>
<td>0.100 [64.5]</td>
</tr>
<tr>
<td>W 11</td>
<td>0.374 [9.50]</td>
<td>0.110 [71.0]</td>
</tr>
<tr>
<td>W 12</td>
<td>0.391 [9.93]</td>
<td>0.120 [77.4]</td>
</tr>
<tr>
<td>W 14</td>
<td>0.422 [10.7]</td>
<td>0.140 [90.3]</td>
</tr>
<tr>
<td>W 16</td>
<td>0.451 [11.6]</td>
<td>0.150 [103]</td>
</tr>
<tr>
<td>W 18</td>
<td>0.479 [12.2]</td>
<td>0.160 [109]</td>
</tr>
<tr>
<td>W 20</td>
<td>0.505 [12.8]</td>
<td>0.180 [129]</td>
</tr>
<tr>
<td>W 22</td>
<td>0.529 [13.4]</td>
<td>0.200 [142]</td>
</tr>
<tr>
<td>W 24</td>
<td>0.553 [14.0]</td>
<td>0.220 [145]</td>
</tr>
<tr>
<td>W 26</td>
<td>0.575 [14.6]</td>
<td>0.240 [155]</td>
</tr>
<tr>
<td>W 28</td>
<td>0.597 [15.2]</td>
<td>0.260 [168]</td>
</tr>
<tr>
<td>W 30</td>
<td>0.618 [15.7]</td>
<td>0.300 [194]</td>
</tr>
<tr>
<td>W 31</td>
<td>0.628 [16.0]</td>
<td>0.310 [200]</td>
</tr>
<tr>
<td>W 45</td>
<td>0.757 [19.2]</td>
<td>0.450 [290]</td>
</tr>
</tbody>
</table>

<sup>A</sup> Table 1 should be used on projects that are designed using inch-pound units; Table 2 should be used on projects that are designed using SI units.

<sup>B</sup> The number following the prefix indicates the nominal cross-sectional area of the wire in square inches multiplied by 100.

<sup>C</sup> For sizes other than those shown above, the Size Number shall be the number of one hundredth of a square inch in the nominal area of the wire cross section, prefixed by the W.

<sup>D</sup> These sizes represent the most readily available sizes in the welded wire reinforcement industry. Other wire sizes are available and many manufacturers can produce them in 0.0015 in.² increments.

<sup>E</sup> The nominal diameter is based on the nominal area of the wire.

---

### Table 2 Dimensional Requirements for Plain Wire—SI Units

<table>
<thead>
<tr>
<th>Size Number&lt;sup&gt;B, C, D&lt;/sup&gt;</th>
<th>Nominal Diameter mm [in.]&lt;sup&gt;E&lt;/sup&gt;</th>
<th>Nominal Area mm² [in.²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>MW 5</td>
<td>2.52 [0.099]</td>
<td>5 [0.008]</td>
</tr>
<tr>
<td>MW 10</td>
<td>3.57 [0.140]</td>
<td>10 [0.016]</td>
</tr>
<tr>
<td>MW 15</td>
<td>4.37 [0.172]</td>
<td>15 [0.023]</td>
</tr>
<tr>
<td>MW 20</td>
<td>5.05 [0.199]</td>
<td>20 [0.031]</td>
</tr>
<tr>
<td>MW 25</td>
<td>5.64 [0.222]</td>
<td>25 [0.039]</td>
</tr>
<tr>
<td>MW 30</td>
<td>6.18 [0.243]</td>
<td>30 [0.047]</td>
</tr>
<tr>
<td>MW 35</td>
<td>6.68 [0.263]</td>
<td>35 [0.054]</td>
</tr>
<tr>
<td>MW 40</td>
<td>7.14 [0.281]</td>
<td>40 [0.062]</td>
</tr>
<tr>
<td>MW 45</td>
<td>7.57 [0.298]</td>
<td>45 [0.070]</td>
</tr>
<tr>
<td>MW 50</td>
<td>7.98 [0.314]</td>
<td>50 [0.078]</td>
</tr>
<tr>
<td>MW 55</td>
<td>8.37 [0.329]</td>
<td>55 [0.085]</td>
</tr>
<tr>
<td>MW 60</td>
<td>8.74 [0.344]</td>
<td>60 [0.093]</td>
</tr>
<tr>
<td>MW 65</td>
<td>9.10 [0.358]</td>
<td>65 [0.101]</td>
</tr>
<tr>
<td>MW 70</td>
<td>9.44 [0.372]</td>
<td>70 [0.109]</td>
</tr>
<tr>
<td>MW 80</td>
<td>10.1 [0.397]</td>
<td>80 [0.124]</td>
</tr>
<tr>
<td>MW 90</td>
<td>10.7 [0.421]</td>
<td>90 [0.140]</td>
</tr>
<tr>
<td>MW 100</td>
<td>11.3 [0.444]</td>
<td>100 [0.156]</td>
</tr>
<tr>
<td>MW 120</td>
<td>12.4 [0.487]</td>
<td>120 [0.186]</td>
</tr>
<tr>
<td>MW 130</td>
<td>12.9 [0.507]</td>
<td>130 [0.202]</td>
</tr>
<tr>
<td>MW 200</td>
<td>16.0 [0.628]</td>
<td>200 [0.310]</td>
</tr>
<tr>
<td>MW 290</td>
<td>19.2 [0.757]</td>
<td>290 [0.450]</td>
</tr>
</tbody>
</table>

<sup>A</sup> The wire sizes in Table 1 should be used on projects that are designed using inch-pound units; the wire sizes in Table 2 should be used on projects that are designed using SI units.

<sup>B</sup> The number following the prefix indicates the nominal cross-sectional area of the wire in square millimetres.

<sup>C</sup> For sizes other than those shown above, the Size Number shall be the number of square millimetres in the nominal area of the wire cross section, prefixed by the MW.

<sup>D</sup> These sizes represent the most readily available sizes in the welded wire reinforcement industry. Other wire sizes are available and many manufacturers can produce them in 1 mm² increments.

<sup>E</sup> The nominal diameter is based on the nominal area of the wire.

---

### Discussion

Deformations can be either indented or...
raised transverse ribs (protrusions). The deformations and the welded intersections provide bond strength and anchorage.

3.1.3 plain wire and plain welded wire reinforcement—a material composed of cold-worked plain steel wire, as cold-drawn or cold-rolled from hot-rolled steel rod.

3.1.3.1 Discussion—The welded intersections provide anchorage.

4. Ordering Information

4.1 Orders for wire or welded wire reinforcement under this specification shall contain the following information:

4.1.1 Quantity (weight [mass]) or square area for welded wire reinforcement,

4.1.2 Name of material (cold-drawn or rolled steel wire, or welded wire reinforcement, plain or deformed, for concrete),

4.1.3 Wire size number, wire spacing, and sheet or roll width and length for welded wire reinforcement,

4.1.4 Minimum yield strength or Grade,

4.1.5 Packaging (see Section 15), and

4.1.6 ASTM designation and year of issue.

4.2 The purchaser shall have the option to specify additional requirements, including but not limited to, the following:

4.2.1 Exclusion of over-steeling (see 10.4.2 and 10.5.1),

4.2.2 Report on tests performed on the steel wire or welded wire reinforcement being furnished (see 14.1), and

4.2.3 Special requirements (if desired).

5. Materials

5.1 The steel shall be made by any commercially accepted process.

5.2 Unless otherwise specified, the wire shall be supplied uncoated. When plain wire is specified as galvanized, it shall be galvanized at finish size as described in Specification A641/A641M.

5.3 Wire used in the manufacture of welded wire reinforcement shall conform to this specification either solely or in combination of plain or deformed wire, or both.
6. Manufacture

6.1 The wire shall be cold-worked, drawn or rolled, from rods that have been hot-rolled from billets.

6.2 For welded wire reinforcement, the wires shall be assembled by automatic machines or by other suitable mechanical means which will assure accurate spacing and alignment of all wires of the finished product. The finished welded wire reinforcement shall be furnished in flat or bent sheets or in rolls as specified by the purchaser.

6.3 Longitudinal and transverse wires shall be securely connected at every intersection by a process of electrical resistance welding which employs the principle of fusion combined with pressure.

6.4 Welded wire reinforcement of proper yield strength and quality when manufactured in the manner herein required shall result in a strong, serviceable mat-type product having substantially square or rectangular openings, and shall conform to this specification.

NOTE 2—A variation of manufacturing includes the application of one or more longitudinal convoluted wires at one edge of welded wire reinforcement for concrete pipe reinforcing cages. This shape allows the cage ends to be expanded to a larger diameter to accommodate the bell-shaped ends of concrete pipe.

7. Mechanical Property Requirements—Wire, Plain and Deformed

7.1 General Requirements for Plain Wire:

7.1.1 The relation between size number, diameter, and area requirements shown in Table 1 or Table 2 shall apply, whichever is applicable.

7.1.2 Specimens for mechanical properties testing shall be full wire sections and shall be obtained from ends of wire coils as drawn or rolled. The specimens shall be of sufficient length to perform testing described in Test Methods and Definitions A370.

7.1.3 If any test specimen exhibits obvious isolated imperfections not representative of the product, it shall be discarded and another specimen substituted.

7.1.4 Tension Test:

7.1.4.1 When tested as described in Test Methods and Definitions A370, the material, except as specified in 7.1.4.2, shall conform to the tensile property requirements in Table 5 or Table 6, whichever is applicable, based on the nominal area of the wire.

### TABLE 4 Dimensional Requirements for Deformed Wire—SI Units

<table>
<thead>
<tr>
<th>Deformed Wire Size</th>
<th>D [in.² × 100]</th>
<th>Unit Mass, kg/m</th>
<th>Diameter, mm</th>
<th>Cross-Sectional Area, mm²</th>
<th>Minimum Average Height of Deformations, mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>MD 25</td>
<td>0.196</td>
<td>6.64</td>
<td>25</td>
<td>0.252</td>
<td></td>
</tr>
<tr>
<td>MD 30</td>
<td>0.235</td>
<td>6.18</td>
<td>30</td>
<td>0.279</td>
<td></td>
</tr>
<tr>
<td>MD 35</td>
<td>0.275</td>
<td>6.68</td>
<td>35</td>
<td>0.302</td>
<td></td>
</tr>
<tr>
<td>MD 40</td>
<td>0.314</td>
<td>7.14</td>
<td>40</td>
<td>0.320</td>
<td></td>
</tr>
<tr>
<td>MD 45</td>
<td>0.353</td>
<td>7.57</td>
<td>45</td>
<td>0.342</td>
<td></td>
</tr>
<tr>
<td>MD 50</td>
<td>0.392</td>
<td>7.98</td>
<td>50</td>
<td>0.360</td>
<td></td>
</tr>
<tr>
<td>MD 55</td>
<td>0.432</td>
<td>8.37</td>
<td>55</td>
<td>0.378</td>
<td></td>
</tr>
<tr>
<td>MD 60</td>
<td>0.471</td>
<td>8.74</td>
<td>60</td>
<td>0.392</td>
<td></td>
</tr>
<tr>
<td>MD 65</td>
<td>0.510</td>
<td>9.10</td>
<td>65</td>
<td>0.415</td>
<td></td>
</tr>
<tr>
<td>MD 70</td>
<td>0.549</td>
<td>9.44</td>
<td>70</td>
<td>0.470</td>
<td></td>
</tr>
<tr>
<td>MD 80</td>
<td>0.628</td>
<td>10.1</td>
<td>80</td>
<td>0.505</td>
<td></td>
</tr>
<tr>
<td>MD 90</td>
<td>0.706</td>
<td>10.7</td>
<td>90</td>
<td>0.535</td>
<td></td>
</tr>
<tr>
<td>MD 100</td>
<td>0.785</td>
<td>11.3</td>
<td>100</td>
<td>0.565</td>
<td></td>
</tr>
<tr>
<td>MD 120</td>
<td>0.942</td>
<td>12.4</td>
<td>120</td>
<td>0.620</td>
<td></td>
</tr>
<tr>
<td>MD 130</td>
<td>1.02</td>
<td>12.9</td>
<td>130</td>
<td>0.645</td>
<td></td>
</tr>
<tr>
<td>MD 200</td>
<td>1.57</td>
<td>16.0</td>
<td>200</td>
<td>0.800</td>
<td></td>
</tr>
<tr>
<td>MD 290</td>
<td>2.28</td>
<td>19.2</td>
<td>290</td>
<td>0.961</td>
<td></td>
</tr>
</tbody>
</table>

### TABLE 5 Tension Test Requirements—Plain Wire

<table>
<thead>
<tr>
<th>Grade 70 [485]</th>
<th>Grade 72.5 [500]</th>
<th>Grade 75 [515]</th>
<th>Grade 77.5 [533]</th>
<th>Grade 80 [550]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile strength, min, psi [MPa]</td>
<td>80 000 [550]</td>
<td>85 000 [585]</td>
<td>87 500 [600]</td>
<td>90 000 [620]</td>
</tr>
<tr>
<td>Yield strength, min, psi [MPa]</td>
<td>70 000 [485]</td>
<td>75 000 [515]</td>
<td>77 500 [533]</td>
<td>80 000 [550]</td>
</tr>
<tr>
<td>Reduction of area, min, %</td>
<td>30^A</td>
<td>30A</td>
<td>30A</td>
<td>30A</td>
</tr>
</tbody>
</table>

^A For material testing over 100 000 psi [690 MPa] tensile strength, the reduction of area shall be not less than 25%.
When required by the purchaser, yield strength shall be determined as described using a Class B-I extensometer as described in Practice E83. The yield strength shall be determined as described in Test Methods and Definitions A370 at an extension under load of 0.5% of gage length or by the offset method (0.2%). It shall be permissible to remove the extensometer after the yield strength has been determined. The wire shall meet the requirements of Table 5 or Table 6, whichever is applicable.

For wire to be used in the manufacture of welded wire reinforcement, the tensile and yield strength properties shall conform to the requirements given in Table 6, based on the nominal area of the wire.

Bend Test—The bend test specimen shall be bent at room temperature through 180° without cracking on the outside of the bent portion visible to a person with normal or corrected vision, as prescribed in Table 8.

7.1.6 Reduction of Area Test—The reduction of area shall be determined as described in Test Methods and Definitions A370. The wire shall conform to the reduction of area requirements in Table 5 or Table 6, whichever is applicable.

Permissible Variation in Wire Diameter:

7.1.7.1 The permissible variation in wire diameter shall conform to the requirements in Table 7.

7.1.7.2 The difference between the maximum and minimum diameters, as measured on any given cross section of the wire, shall not exceed the tolerances listed in Table 7 for the given wire size.

Note 3—Cold-worked wire generally does not exhibit a definite yield point.
7.2 General Requirements for Deformed Wire:

7.2.1 The relation between size number, diameter, and area shown in Table 3 or Table 4 shall apply, whichever is applicable.

7.2.2 Specimens for mechanical properties testing shall be full wire sections and shall be obtained from ends of wire coils as rolled. The specimens shall be of sufficient length to perform testing described in Test Methods and Definitions A370.

7.2.3 If any test specimen exhibits obvious isolated imperfections not representative of the product, it shall be discarded and another specimen substituted.

7.2.4 Deformation Criteria:

7.2.4.1 Deformations shall be spaced along the wire at a substantially uniform distance and shall be symmetrically dispersed around the perimeter. The deformations on all longitudinal lines of the wire shall be similar in size and shape. A minimum of 25 % of the total surface area shall be deformed by measurable deformations.

7.2.4.2 Deformed wire shall have two or more lines of deformations.

7.2.4.3 The average longitudinal spacing of deformations shall be not less than 3.5 nor more than 5.5 deformations per inch [25 mm] in each line of deformations on the wire.

7.2.4.4 The minimum average height of the center of typical deformations based on the nominal wire diameters shown in Table 3 or Table 4 shall be as follows:

<table>
<thead>
<tr>
<th>Wire Sizes</th>
<th>Minimum Average Height of Deformations % of Nominal Wire Diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>D 3 (MD 20) and smaller</td>
<td>4%</td>
</tr>
<tr>
<td>Larger than D 3 (MD 20) through D 10 (MD 65)</td>
<td>4½%</td>
</tr>
<tr>
<td>Larger than D 10 (MD 65)</td>
<td>5</td>
</tr>
</tbody>
</table>

7.2.4.5 The deformations shall be placed with respect to the axis of the wire so that the included angle is not less than 45°; or if deformations are curvilinear, the angle formed by the transverse axis of the deformation and the wire axis shall be not less than 45°. Where the line of deformations forms an included angle with the axis of the wire from 45° to 70° inclusive, the deformations shall alternate, reverse in direction on each side, or those on one side shall be reversed in direction from those on the opposite side. Where the included angle is over 70°, a reversal in direction is not required.

7.2.4.6 The average spacing of deformations shall be determined by dividing a measured length (10 in. [250 mm] min) of the wire specimen by the number of individual deformations in any one row of deformations on any side of the wire specimen. A measured length of the wire specimen shall be considered the distance from a point on a deformation to a corresponding point on any other deformation in the same line of deformations on the wire.

7.2.4.7 The minimum average height of deformations shall be determined from measurements made on not less than two typical deformations from each line of deformations on the wire. Measurements shall be made at the center of indentations or raised ribs.

7.2.5 Tension Test:

7.2.5.1 When tested as described in Test Methods and Definitions A370, the material, except as specified in 7.2.5.2, shall conform to the tensile property requirements in Table 9, based on the nominal area of wire.

7.2.5.2 When required by the purchaser, the yield strength shall be determined as described in Test Methods and Definitions A370 at an extension of 0.5 % of gage length or by the offset method (0.2 %). For determining the yield strength, use a Class B-1 extensometer as described in Practice E83. It shall be permissible to remove the extensometer after the yield strength has been determined. The wire shall meet the requirements of Table 9 or Table 10, whichever is applicable.

7.2.5.3 For material to be used in the manufacture of welded wire reinforcement, the tensile and yield strength properties shall conform to the requirements given in Table 10, based on the nominal area of the wire.

7.2.6 Bend Test—The bend test specimen shall be bent at room temperature through 90° without cracking on the outside of the bent portion visible to a person with normal or corrected vision, as prescribed in Table 11.

7.2.7 Permissible Variation in Weight [Mass]:

7.2.7.1 The permissible variation in weight [mass] of any deformed wire is ±6 % of its nominal weight [mass]. The theoretical weight [mass] shown in Table 3 or Table 4, or similar calculations on unlisted sizes, shall be used to establish the variation.

7.3 Number of Tests—One tension and one bend test shall be made from each 10 tons [9000 kg] or less of each size of wire or fraction thereof in a lot, or a total of seven samples, whichever is less. A lot shall consist of all the coils of a single size offered for delivery at the same time.

7.4 Quality, Finish, and Appearance:

7.4.1 The wire shall be free of detrimental imperfections and shall meet the requirements of this specification.

7.4.2 Rust, surface seams, or surface irregularities shall not be a cause for rejection if provided the requirements of 7.4.3 are met, and the minimum dimensions and mechanical properties of a hand wire-brushed test specimen meet the requirements of this specification.

7.4.3 Wire intended for welded wire reinforcement shall be sufficiently free of rust and drawing lubricant so as not to interfere with electric resistance welding.

8. Mechanical Property Requirements—Welded Wire Reinforcement

8.1 Tension Test:

8.1.1 Wire for the production of welded wire reinforcement, plain and deformed, is described in Section 7. Tensile tests shall be made on wire cut from the welded wire reinforcement...
and tested either across or between the welds; no less than 50 % shall be across welds. Tensile tests across a weld shall have the welded intersection located approximately at the center of the wire being tested and the transverse wire forming the welded intersection shall extend approximately 1 in. [25 mm] beyond each side of the intersection.

8.1.2 When required by the purchaser, the yield strength shall be determined as described in Test Methods and Definitions A370 at an extension of 0.5 % of gage length or by the offset method (0.2 %). For determining the yield strength use a Class B-1 extensometer as described in Practice E83. It shall be permissible to remove the extensometer from the specimen after yield strength has been determined.

8.2 Bend Test—The wire shall withstand the bend test as described in 7.1.5 or 7.2.6, whichever is applicable, and shall be performed on a specimen taken from between the welds.

8.3 Weld Shear Strength:

8.3.1 The weld shear strength between longitudinal and transverse wires shall be tested as described in Section 9. The minimum average shear value in pounds-force shall not be less than 35 000 multiplied by the nominal area of the larger wire in square inches [in Newtons shall not be less than 241 multiplied by the nominal area of the larger wire in square millimetres], where the smaller wire has an area of 40 % or more of the area of the larger wire. For deformed welded wire reinforcement, the smaller wire shall not be less than D 4 [MD 26].

8.3.2 Deformed welded wire reinforcement having a relationship of larger and smaller wires other than that covered in 8.3.1 shall meet an average weld shear strength requirement of not less than 800 lbf [3.6 kN], provided that the smaller wire is not smaller than D 4 [MD 26]. Plain welded wire reinforcement having a relationship of larger and smaller wires other than those covered in 8.3.1 shall not be subject to the weld shear requirement.

8.3.3 Weld-shear tests for determination of conformance to the requirements of 8.3 shall be conducted using a weld tester as described in Section 9.

8.3.4 Four welds selected at random from the specimen described in 11.2 shall be tested for weld shear strength. The transverse wire of each test specimen shall extend approximately 1 in. [25 mm] on each side of the longitudinal wire. The longitudinal wire of each test specimen shall be of such length below the transverse wire so as to be adequately engaged by the grips of the testing machine. The longitudinal wire shall be of such length above the transverse wire that its end shall be above the center line of the upper bearing of the weld tester.

8.3.5 The material shall be deemed to conform to the requirements for weld shear strength if the average of the test results of the four specimens complies with the value stipulated in 8.3. If the average fails to meet the prescribed value, all the welds across the specimen shall then be tested. The welded wire reinforcement shall be deemed acceptable if the average of all weld shear test values across the specimen meets the prescribed minimum value.

8.4 Number of Tests:

8.4.1 One test for conformance to tensile strength and bend requirements shall be made for each 75 000 ft² [7 000 m²] of welded wire reinforcement or remaining fraction thereof. For testing prior to fabrication, one test for each 20 tons [18 metric tonnes] of wire shall be made.

8.4.2 One test for conformance to weld shear strength requirements shall be made for each 300 000 ft² [28 000 m²] or remaining fraction thereof.

9. Weld Shear Test Apparatus and Methods

9.1 As the welds in welded wire reinforcement contribute to the bond and anchorage value of the wires in concrete, the weld acceptance tests shall be made in a weld tester that stresses the weld in a manner similar to which it is stressed in concrete. In order to accomplish this, the vertical wire in the weld tester shall be stressed in an axis close to its center line. Also the horizontal wire shall be held closely to the vertical wire, and in the same relative position, so as to prevent rotation of the horizontal wire. When the welded wire reinforcement is manufactured with different wire sizes, the larger diameter wire shall be the “vertical wire” when tested (see Fig. 1).

9.2 The weld tester shown in Fig. 1 shall be hung in a ball and socket, or similar self aligning arrangement, at the center of the machine and used with an anvil sized such that it fully supports the horizontal wire and allows the vertical wire of the test specimen to move freely in the vertical direction. This, or a similarly effective fixture designed on the same principle, shall be acceptable.

9.3 Test specimens shall be inserted through the notch in the anvil using the smallest notch available in which the vertical wire fits loosely. The vertical wire shall be in contact with the surface of the free rotating rollers while the horizontal wire shall be supported by the anvil on each side of the slot. The bottom jaws of the testing machine shall grip the lower end of the vertical wire and the load shall be applied at a rate of stressing not to exceed 100 000 psi/min [689 MPa/min].

10. Dimensions and Permissible Variations for Welded Wire Reinforcement

10.1 Width—The width of welded wire reinforcement shall be considered to be the center-to-center distance between outside longitudinal wires. The permissible variation shall not exceed 0.5 in. [13 mm] greater or less than the specified width. In case the width of flat sheets or rolls is specified as the overall width (tip-to-tip length of transverse wires), the width shall not vary more than ±1 in. [±25 mm] from the specified width. When measurements involve a convoluted wire, the measurement shall be made to the approximate center of the sinusoidal wave shape.

10.2 Length—The overall length of flat sheets, measured on any wire, shall not vary more than ±1 in. [±25 mm], or 1 %, whichever is greater.

10.3 Overhang of the transverse wires shall not project beyond the centerline of each longitudinal edge wire more than a distance of 1 in. [25 mm], unless otherwise specified. When transverse wires are specified to project a specific length beyond the center line of a longitudinal edge wire, the permissible variation shall not exceed 0.5 in. [13 mm] greater or less than the specified length.
10.4 For plain welded wire reinforcement, the permissible variation in wire diameter of any wire in the finished product shall conform to the tolerances prescribed for the wire before welding, with the following exceptions:

10.4.1 Because of the mechanical characteristics of manufacturing welded wire reinforcement, the out-of-round requirements shall not apply.

10.4.2 Unless otherwise precluded by the purchaser in 4.2, the manufacturer shall be permitted to use over-sized plain wire. The size differential shall not exceed two “W” [ten “MW”] size increments on sizes W 8 [MW 52] and smaller, and four “W” [twenty “MW”] size increments on sizes larger than W 8 [MW 52]. A “W” [“MW”] size increment is a whole number increment, for example, W 5 to W 6 [MW 25 to MW 26], or W 5.4 to W 6.4 [MW 30 to MW 31]. In all cases where such over-steeling is practiced, the manufacturer shall identify the welded wire reinforcement with the style originally ordered. With the permission of the purchaser, the manufacturer shall be permitted to exceed the limits of this section.

10.5 For deformed welded wire reinforcement, the permissible variation in wire weight of any wire in the finished product shall conform to the tolerances prescribed for the wire before welding, with the following exceptions:

10.5.1 Unless otherwise precluded by the purchaser in 4.2, the manufacturer shall be permitted to apply over-sized deformed wire. The size differential shall not exceed two “D” [ten “MD”] size increments on sizes D 8 [MD 52] and smaller, and four “D” [twenty “MD”] size increments on sizes larger than D 8 [MD 52]. A “D” [“MD”] size increment is a whole number increment, for example, D 5 to D 6 [MD 25 to MD 26], or D 5.4 to D 6.4 [MD 30 to MD 31]. In all cases where such
11. Sampling

11.1 Test specimens for testing mechanical properties shall be obtained by cutting from the finished welded wire reinforcement, a full width section of sufficient length to perform testing described in 7.1, 7.2, and Section 8.

11.2 Test specimens for determining weld-shear properties shall be obtained by cutting from the finished welded wire reinforcement, a full width section of sufficient length to perform testing described in 8.3.4.

11.3 Measurements for conformance to dimensional characteristics shall be made on full sheets or rolls.

11.4 Any test specimen exhibiting obvious imperfections shall be discarded and another specimen substituted.

12. Inspection

12.1 Inspection of the wire or welded wire reinforcement shall be agreed upon between the purchaser and the manufacturer as part of the purchase order or contract.

12.2 All tests shall be made at the manufacturer’s facilities prior to shipment, unless otherwise specified, such as at other approved testing facilities. Such tests shall be so conducted as not to interfere unnecessarily with the operation of the manufacturer’s facilities.

13. Rejection and Retest

13.1 Unless otherwise specified, any rejection shall be reported to the manufacturer within five working days from the time of selection of test specimens.

13.2 In case a specimen fails to meet the tension or bend test, the material shall not be rejected until two additional specimens taken from other wires in the same sheet or roll have been tested. The material shall be considered as meeting the specification with respect to any prescribed tensile property, provided the average of the test results for the three specimens, including the specimen originally tested, is equal to or exceeds the required minimum for the particular property in question and provided further that none of the three specimens develops less than 80% of the required minimum for the tensile property in question. The material shall be considered as meeting this specification with respect to bend test requirements, provided both additional specimens pass the prescribed bend test.

13.3 Welded intersections shall withstand normal shipping and handling without becoming broken, but the presence of broken welds, regardless of cause, shall not constitute cause for rejection unless the number of broken welds per sheet exceeds 1% of the total number of intersections in a sheet. For material furnished in rolls, not more than 1% of the total number of intersections in 150 ft² [14 m²] of welded wire reinforcement shall be broken. Not more than one-half the permissible maximum number of broken welds shall be located on any one wire.

13.4 In the event of rejection because of failure to meet the weld shear requirements, four additional specimens shall be taken from four different sheets or rolls and tested in accordance with Section 9. If the average of all the weld shear tests performed does not meet the requirement, the material shall be rejected.

13.5 In the event of rejection because of failure to meet the requirements for dimensions, the amount of material rejected shall be limited to those individual sheets or rolls which fail to meet this specification.

13.6 Rust, surface seams, or surface irregularities shall not be cause for rejection provided the minimum welded wire reinforcement dimensions, cross-sectional area, tensile properties, and weld shear strength of a hand wire-brushed test specimen meet the requirements of this specification. The height of deformations above the minimum height requirements shall not be cause for rejection.

13.7 Rehearing—Rejected materials shall be preserved for a period of at least two weeks from the date of inspection, during which time the manufacturer shall be permitted to make claim for a rehearing and retesting.

14. Certification

14.1 If outside inspection is waived, a manufacturer’s certification that the material has been manufactured in accordance with and meets the requirements of this specification shall be the basis of acceptance of the material. The certification shall include the specification number, year-date of issue, and revision letter, if any.

14.2 This conformance is predicated upon testing and acceptance of wire prior to fabrication, coupled with random shear testing during production. The purchaser shall be furnished a manufacturer’s certification of conformance to this specification for each production date or production lot shipped. A production lot shall not exceed 300 000 ft² [28 000 m²].

14.3 Test results for yield strength, tensile strength, and bend tests shall be reported for plain wire above Grade 70 [485], plain welded wire reinforcement above Grade 65 [450], deformed wire above Grade 75 [515], and deformed welded wire reinforcement above Grade 70 [485].

14.4 A material test report, certificate of inspection, or similar document printed from or used in electronic form from an electronic data interchange (EDI) transmission shall be regarded as having the same validity as a counterpart printed in the certifier’s facility. The content of the EDI transmitted
document shall meet the requirements of the invoked ASTM standard(s) and conform to any existing EDI agreement between the purchaser and the manufacturer. Notwithstanding the absence of a signature, the organization submitting the EDI transmission is responsible for the content of the report.

NOTE 4—The industry definition as invoked here is: EDI is the computer-to-computer exchange of business information in a standard format such as ANSI ASC X12.

15. Packaging and Marking

15.1 For plain and deformed wire, the size of the wire, Specification A1064/A1064M, and name or mark of the manufacturer shall be marked on a tag securely attached to each coil of wire.

15.2 When welded wire reinforcement is furnished in flat sheets, it shall be assembled in bundles of convenient size containing not more than 150 sheets and securely fastened together.

15.3 When welded wire reinforcement is furnished in rolls, each roll shall be secured so as to prevent unwinding during shipping and handling.

15.4 Each bundle of flat sheets, bundle of rolls, and each roll weighing over 400 lb [180 kg] shall have attached thereto a minimum of one suitable tag bearing the name of the manufacturer, description of the material, ASTM designation A1064/A1064M, and such other information as may be specified by the purchaser.

15.5 Packaging, marking, and loading for shipment shall be agreed upon between the purchaser and manufacturer.

15.6 When specified in the contract or order, and for the direct procurement by or direct shipment to the U.S. government, marking for shipment, in addition to requirements specified in the contract or order, shall be in accordance with MIL-STD-129 for U.S. military agencies and in accordance with Fed. Std. No. 123 for U.S. government civil agencies.

16. Keywords

16.1 concrete reinforcement; deformations; deformed wire; reinforced concrete; reinforcing steels; steel wire; welded wire reinforcement

SUMMARY OF CHANGES

Committee A01 has identified the location of selected changes to this standard since the last issue (A1064/A1064M – 16b) that may impact the use of this standard. (Approved March 15, 2017.)

(1) Revised 3.1, 7.1.5, and 7.2.6.

Committee A01 has identified the location of selected changes to this standard since the last issue (A1064/A1064M – 16a) that may impact the use of this standard. (Approved Sept. 1, 2016.)

(1) Revised 12.2.  (2) Removed 12.3.

Committee A01 has identified the location of selected changes to this standard since the last issue (A1064/A1064M – 16) that may impact the use of this standard. (Approved May 1, 2016.)

(1) Revised 2.1 and 15.5.  (2) Revised the footnotes of Table 3 and Table 4.  (3) Added Note 3 and removed 7.1.4.4 and 7.2.5.4.

Committee A01 has identified the location of selected changes to this standard since the last issue (A1064/A1064M – 15a) that may impact the use of this standard. (Approved March 1, 2016.)

(1) Revised 12.1 and deleted 12.4.

Committee A01 has identified the location of selected changes to this standard since the last issue (A1064/A1064M – 15) that may impact the use of this standard. (Approved Nov. 1, 2015.)

(1) Revised Section 4.  (2) Revised Table 7 to include a table footnote.  (3) Revised 14.4.
1.2.8 Soil Reinforcing Connection
Panel Reinforcement Table

<table>
<thead>
<tr>
<th>Panel Type</th>
<th>Vertical Bore Num / Size</th>
<th>Horiz. Bore Num / Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>4 / #4</td>
<td>4 / #4</td>
</tr>
<tr>
<td>B</td>
<td>4 / #4</td>
<td>2 / #4</td>
</tr>
</tbody>
</table>

- **Bearing Pad Note:** Bearing pad placement shown is for typical "A" panel only. See standard details. 5 of 5 for "X" and "Y" panels.
- **Lifting Insert Detail:**
  - 18" max spacing between bars

**Typical Panel**
- Shown from front face
- **Plan View**
- **Elevation View**
- **Standard Type "A" Panel Details**
- **Section A-A**
  - Overall Dimensions:
    - Standard panel: ± 1/2" vertical, ± 1/2" horizontal
    - Top and special panels: ± 1/" vertical, ± 1/2" horizontal
  - Connection device locations:
    - Embeds: ± 1" vertical, ± 1" horizontal

**Panel Details:**
- Standard: 1 of 5
- Standard MSE Wall Details
- **Standard Details:**
TYPICAL PANEL
SHOWN FROM BACK FACE

SECTION A-A

LIFTING INSERT DETAIL

PLAN VIEW

ELEVATION VIEW

LOOP EMBED DETAILS

REBAR NOTE:
Panel types "X" and "X2" require the same amount of panel reinforcement.

BEARING PAD NOTE:
Bearing pad placement shown is for typical "A" panel only. See standard details: 5 of 5 for "X" and "X2" panels.

REBAR TYP.

CIRCULAR COVER

REBAR TYP.

CONCRETE COVER

PANEL TOLERANCES:
OVERALL DIMENSIONS:
- STANDARD PANEL
  ± 1/32 VERTICAL
  ± 1/32 HORIZONTAL
- TOP AND SPECIAL PANELS
  ± 1/16 VERTICAL
  ± 1/16 HORIZONTAL

CONNECTION DEVICE LOCATIONS:
- EMBERS
  ± 1/16 VERTICAL
  ± 1/16 HORIZONTAL

PANEL SQUARENESS:
- 90° Panel Corners
  ± 1/32 Using 2" Square
  (Measure 3 Panel Corners)

PANEL DIAGONAL:
- Panels with 90° Corners
  ± 1/32 Max. Difference Between Diagonals

SURFACE FINISH:
- Finish at Front Face
  ± 1/32 in 5'
HORIZONTAL PANEL JOINT DETAIL

ALL PANELS MUST HAVE A MINIMUM OF 1 1/2" COVERAGE WITH CONCRETE ON ALL SIDES.

FILTER FABRIC IS A NON-WOVEN GEOTEXTILE COMPOSED OF POLYPROPYLENE FIBERS, WHICH ARE FORMED INTO A STABLE NETWORK SUCH THAT THE FIBERS RETAIN THEIR RELATIVE POSITION. FILTER FABRIC IS INERT TO BIOLOGICAL DEGRADATION AND RESISTS NATURALLY ENCOUNTERED CHEMICALS, ALKALIS AND ACIDS.

THE MINIMUM INSIDE BEND DIAMETER FOR THE SOIL REINFORCEMENT SHALL BE NO LESS THAN TWICE THE NOMINAL DIAMETER OF THE WIRE SIZE AND IN NO INSTANCE BE LESS THAN 1 INCH.

CROSS WIRE, BEARING BARS WIRE AND LONITUAL WIRE ON SOIL REINFORCEMENT SHALL BE THE SAME SIZE.


MSE WALL INSTALLATION DETAILS

BEARING PAD CHART

VERTICAL PANEL JOINT DETAIL

MSE WALL INSTALLATION DETAILS

STANDARD DETAILS: 5 OF 5

STANDARD MSE WALL DETAILS

RW-05

FRANCISCO S. MARES
03/06/21

No. G0215
EXP. 05-06-22

CIVIL

3/4" JOINT

6" MIN.
6" MAX.

1. PANEL REINFORCEMENT BARS SHALL BE DEFORMED BILLET STEEL BARS FOR CONCRETE REINFORCEMENT CONFORMING TO THE SPECIFICATION OF ASTM DESIGNATION A 615, GRADE 60, INCLUDING SUPPLEMENTARY REQUIREMENT S1 OR LOW ALLOY Steel Deformed Bons Conforming to the Specifications of ASTM Designation A1038, Structural Welded Wire Reinforcement That Conforms to ASTM A1044 Specifications MAY NOT BE SUBSTITUTED FOR DESIGNATION A1038.

2. W11, W20 & W24 STEEL WIRE SHALL CONFORM TO THE ASTM DESIGNATION A1044. THE WELDED WIRE SOIL REINFORCEMENT AND LOOP EMBEDDING SHALL BE WELDED IN ACCORDANCE WITH ASTM DESIGNATION A1044. ALL SOIL REINFORCEMENT WIRE SHALL BE COMPOSED OF SMOOTH WIRE, DEFORMED WIRE SHALL NOT BE USED FOR SOIL REINFORCEMENT, LOOP EMBEDDING AND CONNECTION PIPI.

3. THE LOOP EMBEDDING, SOIL REINFORCEMENT AND CONNECTION PINS SHALL BE GUARANTEED IN ACCORDANCE WITH ASTM DESIGNATION A1223 AFTER BENDING.

4. CONCRETE PANELS TO HAVE A 28-DAY COMPRESSIVE STRENGTH OF 4000 PSI.

5. ALL PANEL REINFORCEMENT MUST HAVE A MINIMUM OF 1 1/2" COVERAGE WITH CONCRETE ON ALL SIDES.

6. FOR PANELS WITH W11 & W20 MESH USE "X" PANEL REINFORCEMENT, PANELS WITH 8 W24 MESH USE "Y" PANEL REINFORCEMENT, PANELS WITH 7 OR 8 W24 MESH USE "T" PANEL REINFORCEMENT, PANELS WITH 7 OR 8 W20 & 8 W24 USE "Z" PANEL REINFORCEMENT.

7. FILTER FABRIC IS A NON-WOVEN GEOTEXTILE COMPOSED OF POLYPROPYLENE FIBERS, WHICH ARE FORMED INTO A STABLE NETWORK SUCH THAT THE FIBERS RETAIN THEIR RELATIVE POSITION. FILTER FABRIC IS INERT TO BIOLOGICAL DEGRADATION AND RESISTS NATURALLY ENCOUNTERED CHEMICALS, ALKALIS AND ACIDS.

8. THE MINIMUM INSIDE BEND DIAMETER FOR THE SOIL REINFORCEMENT SHALL BE NO LESS THAN TWICE THE NOMINAL DIAMETER OF THE WIRE SIZE AND IN NO INSTANCE BE LESS THAN 1 INCH.

9. CROSS WIRE, BEARING BARS WIRE AND LONITUAL WIRE ON SOIL REINFORCEMENT SHALL BE THE SAME SIZE.

**Wire Mesh Loop Details**

(Cross wires, bearing bar wires and longitudinal wires on soil reinforcement shall be the same size)

**Bent Pin Detail**

Reference: 3/1-4/10
1.2.14 Connection Device Dimension
THE DESIGN OF ALL MSE PLUSE® WALLS IS BASED ON THE ASSUMPTION THAT ALL MATERIALS, INCLUDING THE BACKFILL AND METHODS OF CONSTRUCTION, CONFORM TO THE SPECIFICATIONS FOR MSE PLUSE® RETAINING WALLS AND THE PROJECT BID DOCUMENTS.

THESE DRAWINGS, AS WELL AS THE DESIGN, ARE CERTIFIED WITH RESPECT TO THE INTERNAL STABILITY OF THE MSE PLUS® STRUCTURES ONLY. EXTERNAL STABILITY INCLUDING, BUT NOT LIMITED TO, SLOPE AND FOUNDATION STABILITY IS THE RESPONSIBILITY OF THE OWNER AND THE ENGINEER FOR THE OWNER.

EMBED: ELEVATION VIEW (IN PANEL)

EMBED RACK (BEFORE BEND)

8" SPACING EMBED LOOP

INNOVATIVE CONSTRUCTION PRODUCTS
4780 SCOTTS VALLEY DRIVE, SUITE E
SCOTTS VALLEY, CA 95066
PHONE: (831) 430-9300  FAX: (831) 430-9340

THIS DRAWING CONTAINS INFORMATION PROPRIETARY TO SSL AND IS FURNISHED FOR THE PROJECT SHOWN ONLY. THIS INFORMATION SHALL NOT BE TRANSMITTED TO ANY OTHER PERSON OR AGENCY WITHOUT WRITTEN CONSENT OF SSL.
1.2.15 Soil Reinforcing Connection Strength and Testing
SSL Connection Test: MSEPlus™ Panel Wall System

December 6, 2012

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TABLE OF CONTENTS

1. Abstract
2. Introduction
3. Test Procedure
4. Summary & Conclusion
5. Test Data
6. Independent Verification Reports
7. Equipment Calibration Records - Technical Associated Services
8. Equipment and Material Details
9. Expected Elongation and Deflection Calculations
10. Material Compliance Certifications
1. ABSTRACT

Mechanically Stabilized Earth (MSE) retaining walls are structures consisting of multiple layers of soil reinforcement placed in select backfill and connected to a facing element. SSL has developed a soil reinforcement connection for MSE retaining walls which uses welded wire mesh as soil reinforcement and precast concrete panels as a facing element.

This report details a test program developed by SSL to examine the capacity of the connection system along with the interaction of the soil reinforcement and the concrete panel facing element. Caltrans requires that all elements of the soil reinforcement connection system can reach the yield strength of the steel. This test will show that the steel reinforcing wire and all elements of SSL’s connection system can reach the yield strength of the longitudinal wire element (65 ksi) before failure.

During the test, load is applied to soil reinforcement that is connected to a concrete panel. The soil reinforcement is gripped using a steel wire chuck that is fastened to the longitudinal wire and load is applied using a hollow-cylinder hydraulic ram that is supported by a steel frame placed on the concrete panel. A satisfactory or "passing" test run is one in which the yield strength of the steel or Target Load is reached without failure of the connection system. An unsatisfactory or "failing" test run is one in which any element of the connection system fails before Target Load is reached.

The test program included six steel soil reinforcement samples which consisted of single-wire W24 samples with W24 transverse bars. Each sample passed demonstrating that the SSL Panel Wall system can satisfy Caltrans requirements.
2. INTRODUCTION

The MSE Plus™ connection system has undergone the following changes to the welded wire mesh soil reinforcing elements:

- The bend radius of the longitudinal wires has been increased to 1.25” for W24 wire gauges.
- For Caltrans projects, the gauge of all transverse wires will equal to the gauge of the longitudinal wires.
- The use of deformed wire as soil reinforcement has been eliminated and only smooth wire will be used.

The Caltrans LRFD connection test requires the capacity of the MSE panel connection system to meet or exceed the yield strength of the reinforcing element.

The following report will detail the test program that was administered by SSL in order to ensure that the updated connection system, as a whole, can reach or exceed the yield strength of the longitudinal wire element.

2.1 BACKGROUND AND CALCULATIONS

SSL's system will utilize W11, W20 and W24 welded wire mesh as soil reinforcement for Caltrans projects. The samples for this test will be smooth-rolled wire in the W24 size, data for W11 and W20 wire sizes has been provided in the SSL Test Report dated March 20, 2012.

For this test, the Target Load (TL) is the yield strength of the longitudinal wire element of the soil reinforcing mesh (65,000 psi) multiplied by the cross sectional area of the wire. For W24 wire, the cross sectional area of the wire is 0.24 in². The TL requirement for W24 wire is shown below:

- W24: \( 65,000 \text{ psi} \times (0.24 \text{ in}^2) = 15,600 \text{ Lbs.} \)

The connection system will undergo elongation as load is applied. Different components of the connection system will contribute to elongation as the load increases. During a test run in which target load is met, the sequence of elongation is typically as follows:

- Phase 1: The 90° bend in the longitudinal wire opens until the free end comes in to contact with the back of the test panel (see Figure 1).
Phase 1A: The transverse bearing bar is pulled tight against the concrete panel's embed loops (see Figure 2). **NOTE:** Depending on the initial placement of the sample, Phase 1A of elongation may not occur.

Phase 2: The bend of the longitudinal wire will be pulled tightly against the W30 connector pin (see Figure 3).
• Phase 3: The longitudinal wire element of the soil reinforcement will elongate in the length between the connector pin and the grip point of the steel wire chuck. \textit{NOTE:} 0.0574 in of elongation is expected to occur in a wire sample with a free length of 26.5 in. (see calculation in Section 9).

• Phase 4: Loads in excess of the yield strength of the steel will cause plastic deformation or \textit{necking down} of the longitudinal wire followed by fracture (see Photo’s 1 and 2). \textit{NOTE:} Similar to W20, during test runs performed on W24 samples, the higher loads approaching and following yield strength will likely cause some deformation of the connector pin and embed loop elements of the system (the embed loops will deform by approximately 0.030 in as shown in Photo’s 3, 4 and 5).

\begin{tabular}{ll}
\textbf{PHOTO 1: W20 NECKED DOWN AND FRACTURED} & \textbf{PHOTO 2: W11 NECKED DOWN AND FRACTURED} \\
\textbf{PHOTO 3: EMBED LOOP PRIOR TO TESTING (0.933 in)} & \textbf{PHOTO 4: EMBED LOOP AFTER W20 TEST (0.963 in)}
\end{tabular}
PHOTO 5: DEFORMATION OF EMBED LOOP

The rate and length of elongation in all of the connection elements will vary due to any one of the following: wire size; bend angle; embed loop clearance and the initial placement of the wire sample (e.g., it is possible to connect the sample so that the transverse bearing bar is snug against the embed loops, eliminating Phase 1A of elongation).

Due to the arrangement of the steel test apparatus and the position of the hydraulic ram (See Figure 4) there is potential for deflection at the center of the beam that is supporting the ram. This deflection could interfere with the measurements being taken to record elongation. To minimize deflection, the subsequent supporting perpendicular beams are placed as close together as possible, typically around 10.5 in. apart. The expected maximum deflection caused by a point load of over 15,600 lbs on the center of a simply supported steel beam as exists in the test apparatus is .002 in. (See the calculations included in Section 9). Also, care should be taken to ensure that the base of the dial indicator is placed directly over the point at which the simply supported beam and the beam below intersect.
0.002 in of downward deflection when 15,600 lbs of load is applied.

FIGURE 4: DEFLECTION IN TEST APPARATUS
2.2 MATERIALS AND EQUIPMENT

Wire samples were manufactured in accordance with ASTM A82 and the mesh sheets were fabricated in accordance with ASTM A185 and then galvanized in accordance with ASTM A123 (See compliance certifications included in Section 10).

Four, hollow-cylinder ram and gauge combinations were calibrated prior to testing by Technical Associated Services (TAS). The calibration reports provided by TAS are included in Section 7 of this report. The ram/gauge combinations are identified as follows with identical labels for both the ram and gauge:

- Ram/Gauge #1 (12k-12-1)
- Ram/Gauge #2 (12k-12-2)
- Ram/Gauge #3 (12k-12-3)
- Ram/Gauge #4 (12k-12-4)

Based on the calibration reports, the following gauge readings coincide with the target load (TL) for each wire size:

- W24:
  - 12k-12-1: 6,555 psi = 15,600 lbs force
  - 12k-12-2: 5,690 psi = 15,600 lbs force
  - 12k-12-3: 5,650 psi = 15,600 lbs force
  - 12k-12-4: 5,710 psi = 15,600 lbs force

The gauge reading at the initial 500 lb data point and the subsequent data points at 1,000 lb intervals are taken directly from the calibration reports. The gauge reading at target load (TL) is taken directly from the calibration report and will be highlighted yellow and marked TL on the data sheet.

The test frame as detailed in Figure 4 and Section 8, consists of short pieces (between 3 ft. and 5 ft. long) of 0.25 in thick, 4 in. square steel tubing. The lowest supporting beams are placed parallel to the embed loops to allow access for the connector pin, the next course of beams are placed perpendicular to the lowest beams at a distance apart that allows access to the desired number of wire samples. Beams are stacked perpendicular to one another until the desired height of the test apparatus is achieved. The top beam that supports the hydraulic ram must be placed perpendicular to the bottom beams and it will have a 0.75 in hole drilled in the center of the beam. The wire sample will be threaded through the hole in the top beam. It may be necessary to have beams stacked on each other in parallel in order to ensure that the beam supporting the hydraulic ram(s) is perpendicular to the bottom beams.
2.3 TEST REQUIREMENTS

This test will confirm:

- The tensile capacity of the reinforcing wire will meet or exceed the yield strength of the longitudinal wire.
- The load capacity of all elements of the panel to soil-reinforcement connection.
- The rotational ductility of the “L” portion of the bend under load.
- Measurement of the total elongation in the connection while under load, and comparison with expected elongation of the longitudinal wire at yield.

Each test-run will be concluded when one of the following events occurs:

- The weld between the transverse bearing bar and the longitudinal wire is broken.
- The transverse bearing bar is sheared by the panel embed loops.
- The longitudinal wire breaks at any location.

Below are the Pass/Fail criteria:

- Pass: If the connection breaks at any location after reaching the Target Load.
- Fail: If the connection breaks at any location before reaching the Target Load.

A representative from a qualified independent testing laboratory shall be present during all testing, a report prepared by the lab will be included in the final testing report.

3. TEST PROCEDURE

3.1 TEST PANEL FABRICATION

1. Install connector and reinforcing steel.
2. Pour test panel.
3. Form concrete cylinders for testing. Testing may be performed upon 4,000 PSI concrete cylinders breaks.

3.2 WIRE SAMPLE PREPARATION

1. When single wire sample is being tested, ensure that the transverse bearing bar does not overlap or come into contact with adjacent embed loops. A length less than 12" for the transverse bearing bar is recommended.
2. When multiple wires of a sheet are being tested simultaneously, extreme care must be taken to ensure that there are no additional effects caused by the interaction between the transverse bearing bar and the adjacent embed loops. This can be done by:
   - Prior to testing, visually inspect embed loops to ensure that they are at a uniform height above the test panel
   - During testing, maintaining a consistent and even load to each wire that is being tested.

3. Prior to each test run, the wire sample must be thoroughly inspected with hand magnification device to look for cracks, condition of welds, condition of galvanizing, and any other structural or cosmetic defects. Record any observations.

4. Prior to each test run, the wire sample must be measured and recorded for length, angle of bend, distance between bend and transverse bearing bar, and the distance between the bend and the transverse cross bar (if present).

5. Prior to each test run, the wire diameter of all longitudinal and transverse wires must be confirmed and recorded. Measurements include thickness attributed to galvanizing.

3.3 PULL TEST

1. Attach wire mesh sheet with “L” connection to the test panel using W30 connector pin.

2. Install hollow cylinder ram onto the desired number of longitudinal wires and fasten with steel wire chuck. Care must be taken to ensure that all slack is out of the connection prior to installation of steel wire chuck.

3. For each wire being tested, attach measurement clip above steel chuck and align with magnetic base dial indicator. Care must be taken to place the base of the dial indicator above the point at which the top and subsequent steel tubes of the test apparatus cross this will minimize the effects of the maximum expected downward deflection of the test apparatus.

4. Apply load to connection(s), extreme care must be taken to ensure that equal load is being applied to each longitudinal wire. Unbalanced loads will be evident by differences in gauge readings. To balance the loads, make small increases in the trailing ram's pressure.

5. Alignment Load 1, 500 lbs (AL1): Record elongation reading from dial indicator beginning at Alignment Load 1 and continue applying load and recording at 1,000 lb intervals until Alignment Load 2 is reached.

6. Alignment Load 2 (AL2), varies by wire size: When the free end of the longitudinal wire reinforcing element is pulled against the test panel (Phase 1) rotational ductility has been exhibited and Alignment Load 2 has been reached, record elongation and identify on data sheet as "AL2". Continue applying load and recording at 1,000 lb intervals until Target Load is reached.

7. Alignment Load 3 (varies by wire size): When the transverse bearing bar is pulled tight against the embed loops (Phase 1A) Alignment Load 3 has been reached, record
elongation and identify on data sheet as "AL3". Continue applying load and recording at 1,000 lb intervals until Alignment Load 3 is reached. Depending on the initial attachment of the wire sample, Alignment Load 2 and Alignment Load 3 may be reached simultaneously. If this is the case only AL2 will be notated on the data sheet.

8. Due to a stroke limitations of 1", the Dial Indicator will likely need to be adjusted back to zero at some point during the test run, ideally this will be performed as close to TL or 1" of elongation as possible. The test administrator will make a notation on the data sheet indicating at which points the adjustment occurred. On the typed data sheet, the adjustment point will be indicated by a thick black line under the last elongation measurement taken before the adjustment.

9. Once Target Load is reached, record elongation and remove dial-indicator, continue applying load and recording elongation at 1,000 lb intervals using a tape measure until conclusion of test.

10. Record all aspects of the sample's performance including how/where breakage occurred, load at breakage or load at conclusion if no breakage occurred.

11. The wire sample must be thoroughly inspected after testing with hand magnification to look for cracks, weld condition and galvanizing condition. Record any observations.

12. The wire sample should be measured after testing and recorded for length, angle of bend, distance between bend and transverse bearing bar, and the distance between the bend and the transverse cross bar (if present). If the test run is concluded by a longitudinal wire fracture, no relevant measurements of the sample can be made other than the examination of weld condition.

13. Repeat test on remaining samples.

4. SUMMARY AND CONCLUSION

4.1 SUMMARY

For this test, Target Load (TL) was based upon the yield strength of the steel adjusted for the theoretical area of the samples. For W24 samples the TL is 15,600 lbs. In order to pass, each sample must reach TL before any type of failure in the connection.

Target load was reached in every test run, meaning that each sample passed testing. The W24 samples reached anywhere from 103% to 114% of TL.

Five out of six test runs were concluded by the longitudinal wire fracturing at or directly above the bend where the wire deformed around the connector pin as shown in Photos 1 and 2, one test run was concluded when the transverse bar weld sheared. All samples exhibited similar or identical fracture characteristics.
The results of the testing are presented in Tables 1:

### TABLE 1: DATA SUMMARY (W24 X W24)

<table>
<thead>
<tr>
<th>TEST</th>
<th>SAMPLE</th>
<th>PASS/FAIL</th>
<th>TARGET LOAD (lbs)</th>
<th>LOAD REACHED (lbs)</th>
<th>% of TARGET LOAD (65 ksi)</th>
<th>ELONGATION (in)</th>
<th>RESULT</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>W24 X W24</td>
<td>PASS</td>
<td>15,600</td>
<td>16,667</td>
<td>107%</td>
<td>Phase 1: 0.218</td>
<td>Weld Shear</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Phase 2: 1.030</td>
<td></td>
</tr>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td>Phase 3: 0.0574</td>
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<td></td>
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<td></td>
<td></td>
<td>Phase 4: 0.143</td>
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<td></td>
<td></td>
<td></td>
<td>Total: 1.448</td>
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</tr>
<tr>
<td>2</td>
<td>W24 X W24</td>
<td>PASS</td>
<td>15,600</td>
<td>17,750</td>
<td>114%</td>
<td>Phase 1: 0.256</td>
<td>Longitudinal Wire Fracture</td>
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<tr>
<td></td>
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<td></td>
<td>Phase 2: 1.069</td>
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<td></td>
<td></td>
<td></td>
<td>Phase 3: 0.0574</td>
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<td></td>
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<td>Phase 4: 0.278</td>
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<td></td>
<td>Total: 1.750</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>W24 X W24</td>
<td>PASS</td>
<td>15,600</td>
<td>16,000</td>
<td>103%</td>
<td>Phase 1: 0.206</td>
<td>Longitudinal Wire Fracture</td>
</tr>
<tr>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>Phase 2: 0.948</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>Phase 3: 0.0574</td>
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<td></td>
<td></td>
<td>Phase 4: -</td>
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<td></td>
<td>Total: 1.382</td>
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</tr>
<tr>
<td>4</td>
<td>W24 X W24</td>
<td>PASS</td>
<td>15,600</td>
<td>16,000</td>
<td>103%</td>
<td>Phase 1: 0.228</td>
<td>Longitudinal Wire Fracture</td>
</tr>
<tr>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>Phase 2: 1.150</td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>Phase 3: 0.0574</td>
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<td></td>
<td></td>
<td></td>
<td>Phase 4: 0.315</td>
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<td></td>
<td></td>
<td>Total: 1.211</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>W24 X W24</td>
<td>PASS</td>
<td>15,600</td>
<td>17,500</td>
<td>112%</td>
<td>Phase 1: 0.210</td>
<td>Longitudinal Wire Fracture</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Phase 2: 1.150</td>
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<td></td>
<td></td>
<td></td>
<td>Phase 3: 0.0574</td>
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<td></td>
<td></td>
<td></td>
<td>Phase 4: 0.333</td>
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<td></td>
<td></td>
<td></td>
<td>Total: 1.750</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>W24 X W24</td>
<td>PASS</td>
<td>15,600</td>
<td>17,750</td>
<td>114%</td>
<td>Phase 1: 0.210</td>
<td>Longitudinal Wire Fracture</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Phase 2: 1.150</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>Phase 3: 0.0574</td>
<td></td>
</tr>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>Phase 4: 0.333</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>Total: 1.750</td>
<td></td>
</tr>
</tbody>
</table>

For the six W24 samples, the total elongation ranged from 1.211 in to 1.750 in with an average of 1.549 in. Phase 1 of elongation ranged from 0.125 in to 0.256 in with an average of 0.207 in. Phase 2 of elongation ranged from 0.948 in to 1.290 in with an average of 1.106 in. Phase 3 measurements are based up the expected elongation calculation provided in Section 9. Phase 4 measurements were taken using a standard tape measure after the Target Load had been reached and the dial indicator had been removed. Five of six test-runs were concluded with a longitudinal wire fracture, meaning that an accurate measurement of the plastic deformation is largely impossible. The first test run was concluded with a shearing of the transverse bar weld, plastic deformation was 0.143 in. Phase 4 elongation ranged from 0 in to 0.333 in with an average of 0.178 in. (See Table 2 and Charts provided in Section 5).
The phases of elongation for both wire sizes are summarized in Table 2:

**TABLE 2: ELONGATION BY PHASE**

<table>
<thead>
<tr>
<th>PHASE 1</th>
<th>90 degree bend in longitudinal wire opens until free-end comes in to contact with test panel.</th>
</tr>
</thead>
<tbody>
<tr>
<td>W24</td>
<td></td>
</tr>
<tr>
<td>Average (in)</td>
<td>0.207</td>
</tr>
<tr>
<td>Range (in)</td>
<td>0.125 to 0.256</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>PHASE 2</th>
<th>The bend radius of the longitudinal wire closes around the connector pin.</th>
</tr>
</thead>
<tbody>
<tr>
<td>W24</td>
<td></td>
</tr>
<tr>
<td>Average (in)</td>
<td>1.106</td>
</tr>
<tr>
<td>Range (in)</td>
<td>0.948 to 1.290</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>PHASE 3</th>
<th>The yield strength of the longitudinal wire is reached causing elastic deformation.</th>
</tr>
</thead>
<tbody>
<tr>
<td>W24</td>
<td></td>
</tr>
<tr>
<td>Average (in)*</td>
<td>0.0574</td>
</tr>
<tr>
<td>Range (in)*</td>
<td>0.0574</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>PHASE 4</th>
<th>The yield strength of the longitudinal wire is exceeded causing plastic deformation or necking down, followed by fracture.</th>
</tr>
</thead>
<tbody>
<tr>
<td>W24</td>
<td></td>
</tr>
<tr>
<td>Average (in)</td>
<td>0.178</td>
</tr>
<tr>
<td>Range (in)</td>
<td>0 to .333</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TOTAL</th>
<th>The sum of all the phases of elongation.</th>
</tr>
</thead>
<tbody>
<tr>
<td>W24</td>
<td></td>
</tr>
<tr>
<td>Average (in)</td>
<td>1.549</td>
</tr>
<tr>
<td>Range (in)</td>
<td>1.211 to 1.750</td>
</tr>
</tbody>
</table>

*Phase 3 measurement is based upon expected elongation.

All samples tested were given a thorough visual inspection prior to testing, as noted on the Material Measurement sheets provided in Section 5, the condition of all samples was "OK".
4.2 CONCLUSION

The test results provided herein conclude that all elements of the panel to soil-reinforcement connection of the MSEPlus™ system meet or exceed the tensile capacity of the reinforcing wire for W24 sizes which satisfies Caltrans requirements.
5. TEST DATA
5.1 CHARTS DETAILING ELONGATION BY PHASE
Test 1 (W24 x W24): Elongation by Phase

Weld shear @ 16,667 lbs
Test 2 (W24 x W24): Elongation by Phase

Fracture at 17,750 lbs
Test 3 (W24 x W24): Elongation by Phase

Fracture at 16,000 lbs
Test 5 (W24 x W24): Elongation by Phase

Fracture @ 17,500 lbs
Test 6 (W24 x W24): Elongation by Phase

Fracture at 17,750 lbs
5.2 TEST DATA
**SSL Connection Test**

Test Date: 12/7/2012  
Test #: 1  
**RESULTS:** PASS, Weld shear at 16,667 lbs of force

Sample Identification: W24 x W24 (#1)  
Administered by: Nick Haven (SSL), Robert Ellefritz (SSL), witnessed by Jose Jimenez (Twining)

Ram/Gauge: 12k-12-3

<table>
<thead>
<tr>
<th>PSI</th>
<th>Elong</th>
<th>Load (lbs)</th>
<th>Elongation by Phase</th>
</tr>
</thead>
<tbody>
<tr>
<td>250/AL1</td>
<td>-</td>
<td>500</td>
<td>Phase 1 0.218 in</td>
</tr>
<tr>
<td>450</td>
<td>0.084</td>
<td>1,000</td>
<td>Phase 1A - in</td>
</tr>
<tr>
<td>775</td>
<td>0.168</td>
<td>2,000</td>
<td>Phase 2 1.030 in</td>
</tr>
<tr>
<td>1125/AL2</td>
<td>0.218</td>
<td>3,000</td>
<td>Phase 3* 0.0574 in</td>
</tr>
<tr>
<td>1,500</td>
<td>0.306</td>
<td>4,000</td>
<td>Phase 4 0.143 in</td>
</tr>
<tr>
<td>1,825</td>
<td>0.389</td>
<td>5,000</td>
<td>Total 1.448 in</td>
</tr>
<tr>
<td>2,225</td>
<td>0.505</td>
<td>6,000</td>
<td></td>
</tr>
<tr>
<td>2,550</td>
<td>0.572</td>
<td>7,000</td>
<td></td>
</tr>
<tr>
<td>2,925</td>
<td>0.670</td>
<td>8,000</td>
<td></td>
</tr>
<tr>
<td>3,300</td>
<td>0.754</td>
<td>9,000</td>
<td></td>
</tr>
<tr>
<td>3,650</td>
<td>0.826</td>
<td>10,000</td>
<td></td>
</tr>
<tr>
<td>4,000</td>
<td>0.900</td>
<td>11,000</td>
<td></td>
</tr>
<tr>
<td>4,350</td>
<td>0.985</td>
<td>12,000</td>
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SSL Connection Test

Test Date: 12/7/2012  
Test #: 2  
RESULTS: PASS, Longitudinal wire fracture at 17,750 lbs.

Sample Identification: W24 x W24 (#2)  
Administered by: Nick Haven (SSL), Robert Ellefritz (SSL), witnessed by Jose Jimenez (Twining)

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SSL Connection Test

Test Date: 12/7/2012
Test #: 3  RESULTS: PASS, Longitudinal wire fracture at 16,000 lbs.

Sample Identification: W24 x W24 (#3)
Administered by: Nick Haven (SSL), Robert Ellefritz (SSL), witnessed by Jose Jimenez (Twining)

Ram/Gauge 12k-12-3

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<th>Elongation by Phase</th>
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SSL Connection Test

Test Date: 12/7/2012  Test #: 4  RESULTS: PASS, Longitudinal wire fracture at 16,000 lbs.

Sample Identification: W24 x W24 (#4)  Administered by: Nick Haven (SSL), Robert Ellefritz (SSL), witnessed by Jose Jimenez (Twining)

Ram/Gauge  12k-12-3

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SSL Connection Test

Test Date: 12/7/2012  
Test #: 5  
RESULTS: PASS, Longitudinal wire fracture at 17,500 lbs.

Sample Identification: W24 x W24 (#5)  
Administered by: Nick Haven (SSL), Robert Ellefritz (SSL), witnessed by Jose Jimenez (Twining)

Ram/Gauge  12k-12-3

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Elongation by Phase

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</tr>
<tr>
<td>Phase 2</td>
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<td>in</td>
</tr>
<tr>
<td>Phase 3*</td>
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<td>in</td>
</tr>
<tr>
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**SSL Connection Test**

Test Date: 12/7/2012  
Test #: 6  
RESULTS: PASS, Longitudinal wire fracture at 17,750 lbs.

Sample Identification: W24 x W24 (#6)

Administered by: Nick Haven (SSL), Robert Ellefritz (SSL), witnessed by Jose Jimenez (Twining)

Ram/Gauge 12k-12-3

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**Elongation by Phase**

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<tbody>
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<tr>
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5.3 ORIGINAL TEST DATA
## SSL Connection Test

### Test Date:
12/7/12

### Sample Identification:
A4/24 #6

### Administered by:
[Signature]

### Test #: RESULTS:

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<th>Elong</th>
<th>Load (lbs)</th>
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*Break @ Bend 6400 PSI Tape Measured 1' - 15/16" @ 5650 PSI*
### SSL Connection Test

**Test Date:** 12-3-12

**Test #:** 

**Sample Identification:** 71121

**Administered by:**

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**Break @ Transverse Bar Weld**

**6000 PSI**

**Tape Measure: 1-1/4" @ 5650 PSI**
## SSL Connection Test

**Test Date:** 10-7/12  
**Sample Identification:**  
**Administered by:** 

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**Break @ Bond**  
6400 PSI  
Tape measured at 1'-13/8" @ 5650 PSI
### SSL Connection Test

**Test Date:** 12/7/12  
**Sample Identification:** 24/24  
**Administered by:** D.H.

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**Break @ Bend:** 5800 PSI  
**Tap Measured:** 1-1/4" @ 3650 PSI
## SSL Connection Test

**Test Date:** 12/17/10  
**Sample Identification:** 29/104 #1  
**Test #:**  
**Administered by:** KLS

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**Note:** Transverse bar broke at 5800 PSI. Tape measured 1'-13/8" @ 5650 PSI.
### SSL Connection Test

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*Break @ Bond 6300 psi*

*Tape Measured 1 1\(\frac{3}{8}\) inch @ 5650 psi*
5.4 MATERIAL MEASUREMENTS
### Specimen Number #1

**Before**

- **Weld OK**
- **Mesh Bend X-Section**
- **Mesh Plan View**

**After**

- **Weld shear @ 6,000 psi**
- **Mesh Bend X-Section**
- **Mesh Plan View**

**Date Tested:** 12/7

**Tester Initials:** NH

### Specimen Number #2

**Before**

- **Weld OK**
- **Mesh Bend X-Section**
- **Mesh Plan View**

**After**

- **Long wire fracture @ 6,400 psi**
- **Mesh Bend X-Section**
- **Mesh Plan View**

**Date Tested:** 12/7

**Tester Initials:** NH
Specimen Number: #3

**BEFORE**

Weld OK

**MESH BEND X-SECTION**

Bend Radius: 1.25"

**MESH PLAN VIEW**

---

Specimen Number: #4

**BEFORE**

Weld OK

**MESH BEND X-SECTION**

Bend Radius: 1.25"

**MESH PLAN VIEW**

---

Date Tested: 12/7

Tester Initials: NA

Long Wire Fracture @ 5,800 psi

---

Date Tested: 12/7

Tester Initials: NA

Long Wire Fracture @ 5,800 psi
**Specimen Number:** #5

**Before**

**Mesh Bend X-Section**

**After**

- Long wire fracture @ 6,300 psi

**Date Tested:** 12/7

**Tester Initials:** NK

**Specimen Number:** #6

**Before**

**Mesh Bend X-Section**

**After**

- Long frac @ 61400 psi

**Date Tested:**

**Tester Initials:**
6. INDEPENDENT VERIFICATION REPORTS
## SERVICE TICKET

### CLIENT
**SUBCAST WEST**

### ADDRESS
PO NO

### CITY
STATE
ZIP CODE

### TWINING NO.
120018-3

### PHONE
FAX

### PROJECT NO

#### JOB NAME
**SUBCAST WEST**

#### ADDRESS
**8203 ALABAMA ST**

#### CITY
**REDLANDS CA**

#### CONTRACTOR

#### CONTACT PERSON
Andrew Sezia

#### PHONE NO
909-335-6336

### SERVICE(S) PERFORMED

- [ ] PACHOMETER
- [x] PULL TEST
- [ ] TORQUE TEST
- [ ] MOISTURE TEST
- [ ] IN PLACE SHEAR TEST
- [ ] REBOUND HAMMER TEST
- [ ] CORE SAMPLING / REBAR SAMPLING
- [ ] MOBILE TESTING
- [ ] FIREPROOFING TEST
- [ ] OTHER(S)

### DESCRIPTION OF WORK

- Observed 6 pull tests to 13,600 kip on a 29' x 24' L connection with a 7.30 connector (con. flange results test #1: 1.248" 2.146" 3.182" 4.121" 1.397" 2.935") 1.56
- All passed pull out test.
- Observed 2 pull tests to 12,000 kip on a 20' x 11' L connector with a 7.30 connector
- Both tests were inconclusive.
- Observed 5 pull tests to 7,500 kip on a 14' x 11' L connection with a 7.30 connector (con. flange results test #1: 1.970" 2.960" 3.124" 4.165" 1.56") 1.20
- All passed pull out test.

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### MILEAGE
70

### REIMBURSEMENT

- [ ] YES
- [ ] NO

### EXPENSES

### PERFORMED BY

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### VERIFIED BY

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All reports remain the property of Twining, Inc. Authorization for publications of our reports, conclusions, or extracts from or regarding them is reserved pending our written approval as a mutual protection to clients, the public and ourselves.
7. EQUIPMENT CALIBRATION RECORDS
CERTIFICATE OF CALIBRATION

Event Information

Calibration Date: 2-27-2012
Calibration Interval: 12 months
Due Date: 2-27-2013
Location of Equip. Calibrated: Lab

Test Number: HB-212111
Temp./Humidity: 17°C 61% Rh
Procedure: Mfg

Equipment Information

Description: Holl-O-Ram
Manufacturer: Enerpac
Model: RCH-123
Serial Number: 12K-12-3
Range: 12 ton
Gauge S/N: 12K-12-3

Calibration Results

ENERPAC HYD RAM
S/N 12K-12-3

7,150 lbf = 2,625 PSI
13,000 lbf = 4,725 PSI

Remarks:
1.) Test conforms to ANSI/Z540 and ISO/IEC 17025.
2.) Calibration Standards used are traceable to NIST unless otherwise specified.
3.) Calibration data results relate only to the specified serial number stated in the equipment information section in this certificate.
4.) Equipment condition statements are the opinions of Technical Associated Services and are based on, but not limited to, data from measurements made, professional experience, and procedures utilized. It is the responsibility of the customer/user of this equipment to determine if the results identified meet the specific requirements of accuracy for its intended use.
5.) This report shall not be reproduced, except in full, without the written approval of Technical Associated Services.
CERTIFICATE OF CALIBRATION

Event Information

Calibration Date: 2-27-2012
Calibration Interval: 12 months
Due Date: 2-27-2013
Location of Equip. Calibrated: Lab

Test Number: HB-212111
Temp./Humidity: 17°C 61% Rh
Procedure: Mfg

Equipment Information

Model: RCH-123
Range: 12 ton
Gauge S/N: 12K-12-4

Description: Holl-O-Ram
Manufacturer: Enerpac

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Calibration Standards

9392  7/26/2012  Control
9205  4/7/2012  Strainsense

Remarks:
1.) Test conforms to ANSI/Z540 and ISO/EC 17025.
2.) Calibration Standards used are traceable to NIST unless otherwise specified.
3.) Calibration date results relate only to the specified serial number stated in the equipment information section of this certificate.
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CERTIFICATE OF CALIBRATION

Event Information
Calibration Date: 2-27-2012
Calibration Interval: 12 months
Due Date: 2-27-2013

Location of Equip. Calibrated: Lab

Test Number: HB-212111
Temp./Humidity: 17°C 61% Rh
Procedure: Mfg

Model: RCH-123
Serial Number: 12K-12-1
Range: 12 ton
Gauge S/N: 12K-12-1

Equipment Information

Description: Holl-O-Ram
Manufacturer: Enerpac

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Calibration Results

ENERPAC HYD RAM
S/N 12K-12-1

7,150 lbf = 3,025 PSI
13,000 lbf = 5,475 PSI

Approval: Terry Summers

Remarks:
1.) Test conforms to ANSI/Z540 and ISO/IEC 17025.
2.) Calibration Standards used are traceable to NIST unless otherwise specified.
3.) Calibration data results relate only to the specified serial number stated in the equipment information section in this certificate.
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CERTIFICATE OF CALIBRATION

Event Information

Calibration Date: 2-27-2012
Calibration Interval: 12 months
Due Date: 2-27-2013

Test Number: HB-212111
Temp./Humidity: 17°C 61% Rh
Procedure: Mfg

Location of Equip. Calibrated: Lab

Equipment Information

Model: RCH-123
Manufacturer: Enerpac
Range: 12 ton
Serial Number: 12K-12-2
Gauge S/N: 12K-12-2

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7,150 lbf = 2,690 PSI
13,000 lbf = 4,750 PSI

Remarks:
1.) Test conforms to ANSI/Z540 and ISO/IEC 17025.
2.) Calibration Standards used are traceable to NIST unless otherwise specified.
3.) Calibration data results relate only to the specified serial number stated in the equipment information section in this certificate.
4.) Equipment condition statements are the opinions of Technical Associated Services and are based on, but not limited to, data from measurements made, professional experience, and procedures utilized. It is the responsibility of the customer/user of this equipment to determine if the results identified meet the specific requirements of accuracy for its intended use.
5.) This report shall not be reproduced, except in full, without the written approval of Technical Associated Services.
8. EQUIPMENT AND MATERIAL DETAILS
"L" SHEET OR WELDED WIRE LOOPS

STEEL BLOCKING

SUPREME CHUCK

JACK

CONCRETE HALF PANEL

LOOP EMBED

NOTE: 4 RAMS SHOWN FOR INFORMATIONAL PURPOSES ONLY. TEST MAY BE RUN WITH 1-4 JACKS AT A TIME.
SSL W24 "L" LOOP
MESH WIRE END DETAIL
FIGURE 3B

THE USE OF THESE PLANS, DRAWINGS, SPECIFICATIONS IS RESTRICTED. ANY AND ALL REPRODUCTIONS, REUSE AND/OR DISCLOSURES IN WHOLE OR IN PART BY ANY METHOD IS PROHIBITED. THESE DRAWINGS CONTAIN PROPRIETARY INFORMATION AND AS SUCH THE CONTENT REMAINS PROPERTY OF SSL.
9. **CALCULATIONS**
The expected elongation of the 26.5 in longitudinal wire sample is calculated using Young's Modulus:

\[ E = \frac{Tensile \, Strength}{Tensile \, Strain} = \frac{\sigma}{\varepsilon} = \frac{F/A_0}{\Delta/L_0} = \frac{FL_0}{A_0\Delta L} \]

Where:

- \( E \) is Young's Modulus of Elasticity
- \( F \) is the force exerted on an object under tension
- \( A_0 \) is the original cross-sectional area through which force is applied
- \( \Delta L \) is the amount by which the object changes
- \( L_0 \) is the original length of the object

Therefore:

\[ \Delta L = \frac{65,000 \, \text{psi} \, (26.5 \, \text{in})}{30 \, (10^6)} = 0.0574 \, \text{in} \]
Steel Beam Deflection Calculation

Material Properties
- Analysis Method: Load Resistance Factor Design
- Beam Bracing: Completely Unbraced
- Bending Axis: Major Axis Bending
- Load Combination: 2006 IBC & ASCE 7-05

Service loads entered. Load Factors will be applied for calculations.

Applied Loads
- Load(s) for Span Number 1
  - Point Load: D = 15.60 k @ 0.4323 ft (Jack)

DESIGN SUMMARY
- Maximum Bending Stress Ratio = 0.268 : 1
- Maximum Shear Stress Ratio = 0.263 : 1
- Section used for this span: HSS4X4X1/4
- Location of maximum on span: 0.432 ft
- Span # where maximum occurs: Span # 1

Maximum Deflection
- Max Downward L+Lr+S Deflection: 0.000 in
- Max Upward L+Lr+S Deflection: 0.000 in
- Max Downward Total Deflection: 0.002 in
- Max Upward Total Deflection: 0.000 in

Maximum Forces & Stresses for Load Combinations

Overall Maximum Deflections - Unfactored Loads

Vertical Reactions - Unfactored
- Support notation: Far left is #1 Values in KIPS

Caltrans 5x6 6" Panel Calculations

ENERCALC, INC. 1983-2011, Build:6.12.01.12, Ver:6.11.5.3
Licensee : SSL
10. MATERIAL COMPLIANCE CERTIFICATIONS
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</thead>
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<td>82.1 K</td>
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<tr>
<td>Tensile (ksi)</td>
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<td>Tensile (LBS)</td>
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<td>Wire Size</td>
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The mechanical properties of the material ordered were tested for compliance with A.S.T.M. A-496 & A-497. The results are as follows:

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<td>20002-3 BXV W5X20X1.5 TAC TEST SHEETS - M2 6 SHEETS.</td>
</tr>
<tr>
<td>20002-1</td>
<td>BXV W6X24X1.5 TAC TEST SHEETS - M3 9 SHEETS.</td>
<td>20002-1 BXV W6X24X1.5 TAC TEST SHEETS - M3 9 SHEETS.</td>
</tr>
</tbody>
</table>

Testing Date: 8/20/12
Customer: SST

Concrete Reinforcements, Inc.
Tensile Test

Support, AZ 85379
13450 West Peoria Avenue
Concrete Reinforcements Inc.
Tensile Test

Surprise, AZ 85379
13450 West Peoria Avenue
Concrete Reinforcements Inc.
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<tr>
<td>PHYSICAL TESTS</td>
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<tr>
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**Specification Details**

- **Lot #**: 009902
- **Heat #**: KG1284501063
- **PO#:** 11061
- **Plant**: Nucor Steel Kingman, LLC
- **Location**: 300 West Old Highway 66, Kingman, AZ 86413
- **Contact**: (928) 716-7199
- **Owner**: Surprise, AZ 85379

**Certified Mill Test Report**

**Date**: 12-Jul-2012

**Load Number**: 111397

**Material Safety Data Sheets available at www.nucor.com or by contacting your inside sales representative.**
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<td>Elongation</td>
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<td>Reduction in Area</td>
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**Physical Tests**

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<td>Compressive Strength</td>
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<tr>
<td>Copper</td>
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<tr>
<td>Nickel</td>
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**Lot #**

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<th>Description</th>
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<tbody>
<tr>
<td>100755</td>
<td>928.7</td>
<td>KINGMANN A2 66</td>
</tr>
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</table>
SSL Panel Connection Test
MSEPlus™ Wall System

Steven Ruel, P.E.
SSL, LLC
4740 Scotts Valley Drive, Suite E
Scotts Valley, CA 95066
Phone: 831 430 9300
Email: Steve@MSEPlus.com

Date Performed: July 20, 2023

Test Location: Surecast West Redlands, CA
8203 Alabama Street
Redlands, CA 92373
# TABLE OF CONTENTS

1. Introduction  
2. Test Procedure Overview  
3. Test Panel Setup and Procedure  
4. Test Data  
5. Independent Verification Report  
6. Technical Information and Data  
7. Test Panel Drawing for W24 Wire  
8. Test Photos
1. INTRODUCTION

SSL developed a panel connection system for MSE retaining walls which uses welded wire soil reinforcement with precast concrete fascia panels. A dimensional description of the connection and test setup is provided in the attached figures. This test is a supplementary test to be used in conjunction with previous connecting testing performed by SSL in November 2013. The previous test was more focused on the capacity of the connection bends in the individual wires to ensure loading of 100% of yield strength of the wire could be achieved.

This test will be administered by SSL and will be witnessed by an independent testing agency to ensure that the precast panel connectors can meet or exceed the factored tensile capacity as required by AASHTO.

1.1 BACKGROUND AND CALCULATIONS

SSL's system utilizes W11, W20 and W24 welded wire sizes as the primary soil reinforcement sizes for MSE wall projects.

For this test, the Target Load (TL), is determined using the yield strength of the longitudinal wire element of the soil reinforcing mesh of 75,000 PSI multiplied by the cross sectional area of the wire. The largest wire mesh size for the SSL's retaining wall system is a six wire sheet that is comprised of individual W24 wires. For an individual W24 wire, the cross sectional area of the wire is 0.24 in^2. The TL requirement for an individual W24 wire using an AASHTO LRFD load factor of 0.65 is determined as follows. We have also included second load using a load factor of 0.70 for additional information and data.

- Target Load 1 (TL1) = 1 x 75,000 psi x 0.24 in^2 x 0.65 = 11,700 Lbs. per wire
- Target Load 2 (TL2) = 1 x 75,000 psi x 0.24 in^2 x 0.70 = 12,600 Lbs. per wire

2. TEST PROCEDURE OVERVIEW

The pull rod shown in Figure 3 will undergo an exceedingly small elongation as the TL test load is applied to the pull rod system.

Phase 1: Any slack in the pull rod will be eliminated as pull rod is preloaded one until the free end restrained by the connector pin comes in to contact with the back of the panel connectors.
Phase 2: The eyes of the pull rod will be pulled tightly against the W30 connector pin (see Figure 2).
Phase 3: During testing, the 18 inch pull rod element will elongate in the length between the connector pin and the nut and washer assembly location at the top of the 12 ton jack. 

NOTE: The elongation of the pull rod system for TL1 is as follows = \( \frac{11,700 \text{ LB}}{0.3068 \text{ in}^2} \div 36,000,000 \) = 0.0191 inches.

Due to the configuration of the test apparatus and the position of the hydraulic rams there is a small deflection of the aggregate from the applied load to the steel plate from the hydraulic jacks. This anticipated compression of the aggregate is undetermined. However, measurements being taken to record vertical displacement.

2.2 MATERIALS AND EQUIPMENT

The panel connectors are to be manufactured and fabricated in accordance with ASTM A 1064 and galvanized in accordance with ASTM A123. These connectors are standard panel connectors randomly selected for our production inventory.

Six 12-ton hollow-cylinder calibrated ram and gauge combinations by Technical Associated Services (TAS). The calibration reports provided by TAS are included in Section 6.

The 5’ x 5’ test panel is a standard production panel fabricated as shown in Section 7 using 60 KSI rebar reinforcement configured for a W24 “Y” panel.

The ram and gauge combinations are identified as follows with identical labels for both the ram and gauge:
The six jack numbers as follows - 12K-12-2, 12K-12-3, 234281, 234282, 234283, 234285
The two gauge combinations are 12K-12-2 and 12K-12-3

Based on the two sets of calibration reports which were compiled by using the average of the results of all jacks for each gauge, the following gauge values were used for the target load (TL) for the W24 wire size:

- TL1 – 11,700 Lbs. equals 4,200 PSI gauge pressure.
- TL2 – 12,600 Lbs. equals 4,600 PSI gauge pressure.

2.3 TEST REQUIREMENTS

This test will confirm:

- The capacity of a group of six connectors will meet or exceed the LRFD strength requirement of six wires of the W24 mesh sheet. The W24 wire, which is the largest diameter longitudinal wire used in a mesh sheet in the MSE Plus™ system.

The test will be concluded when one of the following events occurs:

- If the panel show significant visual cracks of approximately 1/8 inch in width in the panel or that the vertical displacement of panel connectors exceed 1/8 inch, or a wire loop is broken.

Below are the Pass-Fail criteria:

- If the panel connector achieves the TL1 as indicated above without any significant visual cracks in the panel or deformation of the precast panel of metallic connectors.

A representative from a qualified independent testing laboratory shall be present during all testing. A statement of qualifications and a report prepared by the lab stating they witnessed the test procedure, and that testing was performed in accordance with this test procedure.

3. TEST PANEL SETUP and PROCEDURE

3.1 TEST PANEL FABRICATION

1. Install panel connector and panel reinforcing steel.
2. Pour the standard test panel.
3. Form concrete cylinders for testing. Connection testing may be performed upon a minimum of 4,000 PSI concrete cylinders breaks.
3.2 PANEL CONNECTOR PREPARATION

1. When six panel connectors are being tested simultaneously, extreme care must be taken to ensure that there are no additional effects caused by the interaction of the testing apparatus and the test panel.
   - Prior to testing, visually inspect the connectors and loops to ensure that they are at a uniform height above the test panel.
   - During testing, maintain a consistent and even load to each loop that is being evaluated as possible.
2. Prior to the test run, the panel connector must be thoroughly inspected with a hand magnification device to look for cracks, condition of welds, condition of galvanizing, and any other structural or cosmetic defects.

3.3 PULL TEST PROCEDURE

1. Attach all pull rods to the six panel connector using a standard W30 connector pin as shown in Section 8 of this report.
2. Install the test frame and fill the test frame cavity with crushed rock.
3. Tamp the crushed rock in the cavity and install and level the plate.
4. Install the tubing beam and one hollow cylinder ram at the end of each rod on top of the plate.
5. For each connector tested, attach measurement clip at the top of the jack and align with magnetic base dial indicator.
6. Apply loading to all six rods simultaneously with the jacks through a hydraulic manifold.
7. Apply an alignment Load of 1,000 PSI gauge pressure. Set the all the dial indicator gauges to zero. After Alignment Load 1 is applied continue applying load on all rams in 500 PSI increments and record the dial indicator reading on all six gauges at each increment.
8. Once Target Load TL1 is reached, record the final dial indication values. Continue applying loading until TL2 has been reached and then record the final dial indicator values.
9. This is the conclusion of the test.

3.4 EVALUATION AND RE-TESTING IN THE EVENT OF FAILURE

1. Identify which element of the connection system failed.
2. If the failure was caused by something other than the panel connectors in the panel, it is highly likely that the test apparatus was not set-up properly. Examine test apparatus, identify problem, and adjust the apparatus. Select a fresh sample and re-test.

4. Summary and Conclusion

4.1 SUMMARY
For this test, the Target Load (TL1) was determined using the yield strength of the steel of 75,000 PSI. To pass the connection system test, as shown in the calculations in Section 1.1, the six panel connectors must simultaneously achieve the TL1 of 6 x 11,700 pounds for a total load of 70,200 pounds.

The test was successful based upon the results as the panel connectors and panel did not show any signs of distress, movement, or cracking of the panel as shown in Section 8 of the test photos.

In addition, we subject the panel connectors to a greater load of TL2 which uses a load 0.70 factor which is greater than AASHTO's requirement of 0.65. TL2 load testing was successful with the connectors and panel showing no signs of distress, movement, or cracking with a total load of 75,600 pounds as shown in Section 8 in Figures 6 through 12.

![Test Table](image)

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<th>Gauge Pressure - PSI</th>
<th>Target Load</th>
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<td>(0.011)</td>
<td>(0.012)</td>
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Date: 20-Jul-23
Witness: VL

**4.2 CONCLUSION**

The test results shown in Figure 4 the capacity was reached but the displacement was not what we expected. As can be seen from the test data the average net displacement at the top of the jacks is approximately -0.012 inches. This is only possible since the aggregate in the testing frame was consolidated further from the applied load of 75,600 pounds under the steel plate. However, what is important is that the net displacement
results are very consistent with the margin of error more likely from reading the dial indicators faces. We also had an error with gauge number 3 as it slipped from the magnetic base. However, the displacement from loading of TL1 to TL2 is consistent with the other five measurements.

In summary, we can conclude that all elements of the panel connection system meet and exceed the AASHTO requirements with respect to connection load capacity.
5. Independent Verification Report

Twining, Inc.- Long Beach Lab
3310 Airport Way, Long Beach, CA 90806
Ph: 562.426.3355
Fax: 562.426.6424
www.twinninginc.com

Daily Field Report

Customer: Surecast West
Pmb 338, 700 E Redlands Blvd., Suite U
Redlands, CA 92373

Project: Surecast West, 8203 Alabama St Redlands
8203 Alabama Street
Redlands, CA 92373

Project No: DFR:W01-23-26183-O1
Date of Issuance: 8/25/2023

Distribution:

Jurisdiction:

Approved Signatory: Hesam Sajed

Test Date: Thursday, July 20, 2023
Inspector: Vanessa Garcia Linares

Inspection No.: 23-26183
Contractor: surecast

Equipment Obsv.: Pull rod

Time Arrived: 7:00 AM
Time Departed: 3:22 PM
Time on Site: 8 Hrs

Lunch Start: 12:00 PM
Lunch End: 12:28 PM
Travel Time:

Mileage: Weather: Clear

Types of Testing / Observations

Panel connection test

Referenced Documents

Observations / Remarks

****Regular: CT: DT: Reason for Travel: Miles to Jobsite: Miles from Jobsite: Travel to Jobsite (start): Travel to Jobsite (end): Travel from Jobsite (start): Travel from Jobsite (end): Expenses: Equipment Used:

On site as requested to observe panel connection test. Test to confirm the capacity of a group of 6 connectors will meet or exceed the LRFD strength requirement of 6 wires of the W24 mesh sheet. The W24 wire which is the largest diameter longitudinal wire used in a mesh sheet in the MSE Plus system.

Observation of test set up and procedure per SSL Panel Connection Test, MSE Plus Wall System.

Remarks:

This report will be distributed to architect, engineer, client, and governing jurisdiction (e.g. DSA) as required by applicable codes and project documents.

Certificate of Compliance

The Work Was Was Not The Work Inspected Met Did Not Meet The Material Sampling Was Was Not

All reports remain the property of Twining Inc., the publication of contents, or derivative works is prohibited without written permission. The information in this report is confidential and is for the use of the client. No other party may use the information in this report without written permission from Twining Inc.
### TWINING

**Daily Field Report**

| Customer: | Surecast West  
| Project: |  
| Jurisdiction: |  
| Distribution: |  

| Report No: | DFR:W01-23-28183-O1  
| Project No.: | 230477 1  
| Permit No.: |  
| DSA File #: |  
| DSA AP #: |  

**Twining, Inc. - Long Beach Lab**

3310 Airport Way, Long Beach, CA 90806

Ph: 562.426.3355

Fax: 562.426.6424

www.twininginc.com

| Date of Issue: | 7/29/2023  
| Issue No: | 1  

Approved Signature: Insert Signature of P. Long Beach Operations
# Daily Field Report

**Customer:** Surecast West  
Pmb. 339, 700 E Redlands Blvd., Suite U  
Redlands, CA, 92373

**Project:** Surecast West, 8203 Alabama St Redlands  
8203 Alabama Street  
Redlands, CA, 92373

**Project No.:** 230477 1  
**Permit No.:**  
**HCA#:**  
**DSA File #:**  
**DSA AP #:**

---

**Approved Signature:** [Signature]

[Twinning, Inc. - Long Beach Lab](#)  
3310 Airport Way, Long Beach, CA 90806  
Ph: 562.426.3355  
Fax: 562.426.6424  
[www.twinning.com](http://www.twinning.com)
6. Technical Information and Data

Jack Calibration Curves
TECHNICAL ASSOCIATED SERVICES
7832 Franklin Drive Huntington Beach, CA. 92648 T. (714) 841-0475 F. (714) 841-4180 http://www.tscalibration.com

Customer: SURECAST WEST
Contact: Cody
Address: 8203 Alabama
City/State/Zip: Highland, CA 92346

CERTIFICATE OF CALIBRATION

Calibration Date: 4/29/2023
PO #: Cody
Due Date: 4/29/2024
Location of Equip. Calibrated: Lab

Event Information
Test Number: HB-23317
Temp./Humidity: 68°F 51%
Procedure: ASTM E4-Caltrans

Equipment Information
Description: Hyd Ram
Manufacturer: Enerpac

Model: Holl-O Ram
Range: 10 Ton
Ram ID: 12K-12-2
Customer ID: n/a

Calibration Results

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AVERAGE: 2.6088
STDEV: 0.2926

COMMENTS: Average of three runs taken

APPROVAL: Terry Summers

1) Test conforms to ANSI/ISA2 and ISO/IEC 17025.
2) Calibration Standards used are traceable to NIST unless otherwise specified.
3) Calibration data results relate only to the specified serial number stated in the equipment information section in this certificate.
4) Calibration on this equipment is the responsibility of the customer/user of this equipment to determine if the results identified meet the specific requirements of accuracy for its intended use.
5) Calibration data results relate only to the specified serial number stated in the equipment information section in this certificate.
6) Calibration Standards used are traceable to NIST unless otherwise specified.

Page 1 of 1
CERTIFICATE OF CALIBRATION

Customer: SURECAST WEST
Contact: Cody
Address: 8203 Alabama
City/St/Zip: Highland, CA 92346

Calibration Date: 4/29/2023
PO #: Cody
Due Date: 4/29/2024
Location of Equip. Calibrated: Lab

Event Information
Test Number: HB-23317
Temp./Humidity: 68°F 51%
Procedure: ASTM E4-CalTrans

Equipment Information
Model: RCH123
Range: 10 Ton
Ram ID: 12K-12-2
Customer ID: n/a

Calibration Results

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AVERAGE: 2.6474
STDEV: 0.0319

ENERPAC RAM
Gauge S/N:2831124454

Remarks:
1.) Test conforms to ANSI/Z540 and ISO/IEC 17025.
2.) Calibration Standards used are traceable to NIST unless otherwise specified.
3.) Calibration data results relate only to the specified serial number stated in the equipment information section in this certificate.
4.) Equipment condition statements are the opinions of Technical Associated Services and are based on, but not limited to, data from measurements made, professional experience, and procedures utilized. It is the responsibility of the customer/user of this equipment to determine if the results identified meet the specific requirements of accuracy for its intended use.
5.) This report shall not be reproduced, except in full, without the written approval of Technical Associated Services.

APPROVAL: Terry Summers

Page 1 of 1
CERTIFICATE OF CALIBRATION

Event Information
- Calibration Date: 4/29/2023
- PO #: Cody
- Due Date: 4/29/2024
- Location of Equip. Calibrated: Lab

Test Number: HB-23317
- Temp./Humidity: 68°F 51%
- Procedure: ASTM E4-Caltrans

Equipment Information
- Model: RCH123
- Range: 10 Ton
- Ram ID: 12K-12-3
- Customer ID: n/a

Calibration Results

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AVERAGE: 2.4863
STDEV: 0.2949

COMMENTS: Average of three runs taken

Remarks:
1) Test conforms to ANSI/ASME and ISO/IEC 17025.
2) Calibration Standards used are traceable to NIST unless otherwise specified.
3) Calibration data results relate only to the specified serial number stated in the equipment information section in this certificate.
4) Equipment condition statements are the opinions of Technical Associated Services and are based on, but not limited to, data from measurements made, professional experience, and procedures utilized. It is the responsibility of the customer/user of this equipment to determine if the results identified meet the specific requirements of accuracy for its intended use.
5) This report shall not be reproduced, except in full, without the written approval of Technical Associated Services.

Signature: Terry Summers

Page 1 of 1
TECHNICAL ASSOCIATED SERVICES
7832 Franklin Drive
Huntington Beach, CA 92645
T. (714) 841-0475 F. (714) 841-4180
http://www.asscalibration.com

Customer: SURECAST WEST
Contact: Cody
Address: 8203 Alabama
City/State/Zip: Highland, CA 92346

CERTIFICATE OF CALIBRATION

Calibration Date: 4/29/2023
PO #: Cody
Due Date: 4/29/2024
Location of Equip. Calibrated: Lab

Test Number: HB-23317
Temp./Humidity: 68°F 51%
Procedure: ASTM E4-Calltrans

Description: Hyd Ram
Manufacturer: Enerpac
Model: RCH123
Range: 10 Ton
Ram ID: 12K-12-3
Customer ID: n/a

Calibration Results

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AVERAGE: 2.6518
STDEV: 0.0266

COMMENTS: Average of three runs taken

Approval: Terry Summers

Remarks:
1. Test conforms to ANSI/IS40 and ISO/IEC 17025
2. Calibration Standards used are traceable to NIST unless otherwise specified
3. Calibration data results relate only to the specified serial number stated in the equipment information section in this certificate.
4. Equipment condition statements are the opinions of Technical Associated Services and are based on, but not limited to, data from measurements made, professional experience, and procedures utilized. It is the responsibility of the customer/user of this equipment to determine if the results identified meet the specific requirements of accuracy for its intended use.
5. This report shall not be reproduced, except in full, without the written approval of Technical Associated Services.
Certificate of Calibration

Calibration Date: 4/29/2023
PO #: Cody
Due Date: 4/29/2024
Location of Equip. Calibrated: Lab

Event Information

Test Number: HB-23317
Temp./Humidity: 68°F 51% Rh
Procedure: ASTM E4-Caltrans

Equipment Information

Model: RCH123
Range: 10 Ton

Ram ID: 234281
Customer ID: n/a

Calibration Results

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AVERAGE: 2.5374
STDEV: 0.2729

Remarks:
1.) Test conforms to ANSI/Z540 and ISO/IEC 17025.
2.) Calibration Standards used are traceable to NIST unless otherwise specified.
3.) Calibration data results relate only to the specified serial number stated in the equipment information section in this certificate.
4.) Equipment condition statements are the opinions of Technical Associated Services and are based on, but not limited to, data from measurements made, professional experience, and procedures utilized. It is the responsibility of the customer/user of this equipment to determine if the results identified meet the specific requirements of accuracy for its intended use.
5.) This report shall not be reproduced, except in full, without the written approval of Technical Associated Services.

COMMENTS: Average of three runs taken

Calibration Standards

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CERTIFICATE OF CALIBRATION

Calibration Date: 4/29/2023
PO #: Cody
Due Date: 4/29/2024

Equipment Information

Description: Hyd Ram
Manufacturer: Enerpac
Model: RCH123
Range: 10 Ton
Ram ID: 234281
Customer ID: n/a

Calibration Results

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AVERAGE: 2.6469
STDEV: 0.0369

COMMENTS: Average of three runs taken

Remain:
1.) Test conforms to ANSI/ISO540 and ISO9001.17025
2.) Calibration Standards used are traceable to NIST unless otherwise specified.
3.) Calibration data results relate only to the specified serial number stated in the equipment information section in this certificate.
4.) Equipment condition statements are the opinions of Technical Associated Services and are based on, but not limited to, data from measurements made, professional experience, and procedures utilized. It is the responsibility of the customer/user of this equipment to determine if the results identified meet the specific requirements of accuracy for its intended use.
5.) This report shall not be reproduced, except in full, without the written approval of Technical Associated Services.

APPROVAL: Terry Summers
COURSE WEST

Customer: JEROME WEST
Contact: Cody

Address: 8203 Alabama
City/State/Zip: Highland, CA 92346

TECHNICAL ASSOCIATED SERVICES
7832 Franklin Drive Huntington Beach, CA 92648 T: (714) 841-0475 F: (714) 841-4180 http://www.tascalibration.com

CERTIFICATE OF CALIBRATION

Calibration Date: 4/29/2023
PO #: Cody
Due Date: 4/29/2024

Location of Equip. Calibrated: Lab

Description: Hyd Ram
Manufacturer: Enerpac
Model: RCH123
Range: 10 Ton
Ram ID: 234282
Customer ID: n/a

Equipment Information

Test Number: HB-23317
Temp./Humidity: 68°F 51%
Procedure: ASTM E4-Cali tran

Calibration Results

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AVERAGE: 2.5529
STDEV: 0.3190

COMMENTS: Average of three runs taken

ENERPAC RAM
Gauge S/N 12K-12-2

Remarks:
1. Test conforms to ANSI B25.40 and ISO/IEC 17025.
2. Calibration Standards used are traceable to NIST unless otherwise specified.
3. Calibration data results relate only to the specified serial number stated in the equipment information section in this certificate.
4. Equipment condition statements are the opinions of Technical Associated Services and are based on, but not limited to, data from measurements made, professional experience, and procedures utilized. It is the responsibility of the customer/user of this equipment to determine if the results identified meet the specific requirements of accuracy for its intended use.
5. This report shall not be reproduced, except in full, without the written approval of Technical Associated Services.

Approval: Terry Summers
TECHNICAL ASSOCIATED SERVICES

Customer: SURECAST WEST
Contact: Cody
Address: 8203 Alabama
City/ST/Zip: Highland, CA 92346

CERTIFICATE OF CALIBRATION

Calibration Date: 4/29/2023
PO #: Cody
Due Date: 4/29/2024
Location of Equip. Calibrated: Lab

Event Information
Test Number: HB-23317
Temp./Humidity: 68°F 51%
Procedure: ASTM E4-Caltrans

Equipment Information
Description: Hyd Ram
Manufacturer: Enerpac
Model: RCH123
Range: 10 Ton
Ram ID: 234282
Customer ID: n/a

Calibration Results

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AVERAGE: 2.6587
STDEV: 0.0416

COMMENTS: Average of three runs taken

Calibration Standards

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<th>Traceability</th>
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Remarks:
1) Test conforms to ANSI/Z540 and ISO/IEC 17025.
2) Calibration Standards used are traceable to NIST unless otherwise specified.
3) Calibration data results relate only to the specified serial number stated in the equipment information section in this certificate.
4) Equipment condition statements are the opinions of Technical Associated Services and are based on, but not limited to, data from measurements made, professional experience, and procedures utilized. It is the responsibility of the customer/user of this equipment to determine if the results identified meet the specific requirements of accuracy for its intended use.
5) This report shall not be reproduced, except in full, without the written approval of Technical Associated Services.
TECHNICAL ASSOCIATED SERVICES
7822 Franklin Drive  Huntington Beach, CA  92648  T:(714) 841-0475  F: (714) 841-4180  http://www.tascalibration.com

Customer: SURECAST WEST
Contact: Cody
Address: 8203 Alabama
City/State/Zip: Highland, CA 92346

CERTIFICATE OF CALIBRATION

Calibration Date: 4/29/2023
PO #: Cody
Due Date: 4/29/2024
Location of Equip. Calibrated: Lab

Test Number: HB-23317
Temp./Humidity: 68°F 51% Rh
Procedure: ASTM F4-Caltrans

Description: Hyd Ram
Manufacturer: Enerpac
Model: RCH123
Range: 10 Ton
Ram ID: 234283
Customer ID: n/a

Calibration Results

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AVERAGE: 2.5200
STDEV: 0.3063

COMMENTS: Average of three runs taken.

ENERPAC RAM
Gauge S/N 12K-12-2

Remarks:
1.) Test conformed to ANSI/Z540 and ISO/IEC 17025.
2.) Calibration Standards used are traceable to NIST unless otherwise specified.
3.) Calibration data results relate only to the specified serial number stated in the equipment information section in this certificate.
4.) Equipment condition statements are the opinions of Technical Associated Services and are based on, but not limited to, data from measurements made, professional experience, and procedures utilized. It is the responsibility of the customer/owner of this equipment to determine if the results identified meet the specific requirements of accuracy for its intended use.
5.) This report shall not be reproduced, except in full, without the written approval of Technical Associated Services.

APPROVAL: Terry Summers

Page 1 of 1
CERTIFICATE OF CALIBRATION

Calibration Date: 4/29/2023
PO #: Cody
Due Date: 4/29/2024
Location of Equip. Calibrated: Lab

Test Number: HB-23317
Temp./Humidity: 68°F 51%
Procedure: ASTM E4-Caltrans

Description: Hyd Ram
Manufacturer: Enerpac
Model: RCH123
Range: 10 Ton
Ram ID: 234283
Customer ID: n/a

<table>
<thead>
<tr>
<th>PSI</th>
<th>LBF</th>
<th>LBF/PSI</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
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<td>0</td>
</tr>
<tr>
<td>300</td>
<td>807</td>
<td>2.690</td>
</tr>
<tr>
<td>600</td>
<td>1,573</td>
<td>2.622</td>
</tr>
<tr>
<td>900</td>
<td>2,378</td>
<td>2.642</td>
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<tr>
<td>1,200</td>
<td>3,144</td>
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<td>2.647</td>
</tr>
<tr>
<td>1,800</td>
<td>4,769</td>
<td>2.649</td>
</tr>
<tr>
<td>2,100</td>
<td>5,575</td>
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<td>7,183</td>
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<tr>
<td>3,000</td>
<td>8,021</td>
<td>2.674</td>
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</table>

AVERAGE: 2.6533
STDEV: 0.0224

COMMENTS: Average of three runs taken

Calibration Standards

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<tr>
<th>SH# or ID#</th>
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<tr>
<td>1233694</td>
<td>9/6/2024</td>
<td>53712-2</td>
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<td>22620</td>
<td>6/13/2024</td>
<td>4165-13770905</td>
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</table>

Remarks:
1.) Test conforms to ANSI/Z540 and ISO/IEC 17025.
2.) Calibration Standards used are traceable to NIST unless otherwise specified.
3.) Calibration data results relate only to the specified serial number stated in the equipment information section in this certificate.
4.) Equipment condition statements are the opinions of Technical Associated Services and are based on, but not limited to, data from measurements made, professional experience, and procedures utilized. It is the responsibility of the customer/user of this equipment to determine if the results identified meet the specific requirements of accuracy for its intended use.
5.) This report shall not be reproduced, except in full, without the written approval of Technical Associated Services.

APPROVAL: Terry Summers

Page 1 of 1
CERTIFICATE OF CALIBRATION

Customer: SURECAST WEST
Contact: Cody
Address: 8203 Alabama
City/State/Zip: Highland, CA 92346

Calibration Date: 4/29/2023
PO #: Cody
Due Date: 4/29/2024
Location of Equip. Calibrated: Lab

Test Number: HB-23317
Temp./Humidity: 68°F 51%
Procedure: ASTM E4-Caltrans

Description: Hyd Ram
Manufacturer: Enerpac
Model: RCH123
Range: 10 Ton
Ram ID: 234284
Customer ID: n/a

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<thead>
<tr>
<th>PSI</th>
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<tr>
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<td>3,000</td>
<td>8,451</td>
<td>2.817</td>
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</table>

AVERAGE: 2.5902
STDEV: 0.2599

COMMENTS: Average of three runs taken

ENERPAC RAM
Gauge S/N 12K-12-2

Remarks:
1.) Test conforms to ANSI/Z540 and ISO/IEC 17025.
2.) Calibration Standards used are traceable to NIST unless otherwise specified.
3.) Calibration data results relate only to the specified serial number stated in the equipment information section in this certificate.
4.) Equipment condition statements are the opinions of Technical Associated Services and are based on, but not limited to, data from measurements made, professional experience, and procedures utilized. It is the responsibility of the customer/user of this equipment to determine if the results identified meet the specific requirements of accuracy for its intended use.
5.) This report shall not be reproduced, except in full, without the written approval of Technical Associated Services.

APPROVAL: Terry Summers
CERTIFICATE OF CALIBRATION

Event Information
Calibration Date: 4/29/2023
PO #: Cody
Due Date: 4/29/2024
Location of Equip. Calibrated: Lab

Test Number: HB-23317
Temp./Humidity: 68°F 51%
Procedure: ASTM E4-Caltrans

Equipment Information
Description: Hyd Ram
Manufacturer: Enerpac
Model: RCH123
Range: 10 Ton
Ram ID: 234284
Customer ID: n/a

Calibration Results

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<td>3,179</td>
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<td>2.680</td>
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</table>

AVERAGE: 2.6763
STDEV: 0.0334

COMMENTS: Average of three runs taken

Approval: Terry Summers
Remarks:
1. Test conforms to ANSI/Z540 and ISO/IEC 17025
2. Calibration Standards used are traceable to NIST unless otherwise specified.
3. Calibration data results relate only to the specified serial number stated in the equipment information section in this certificate.
4. Equipment condition statements are the opinions of Technical Associated Services and are based on, but not limited to, data from measurements made, professional experience, and procedures utilized. It is the responsibility of the customer/user of this equipment to determine if the results identified meet the specific requirements of accuracy for its intended use.
5. This report shall not be reproduced, except in full, without the written approval of Technical Associated Services.
# Cylinder Test Break and Concrete Mix Design

## SURECAST WEST

<table>
<thead>
<tr>
<th>PROJECT</th>
<th>PULL-OUT TEST</th>
<th>DATE</th>
<th>7/14/2023</th>
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<tbody>
<tr>
<td>LOCATION</td>
<td>SURECAST WEST</td>
<td>DAY</td>
<td>FRIDAY</td>
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<table>
<thead>
<tr>
<th>CONC. SUPPLIER</th>
<th>CEMEX</th>
<th>BATCH TIME</th>
<th>8:06 AM</th>
</tr>
</thead>
<tbody>
<tr>
<td>LOAD NUMBER</td>
<td>1</td>
<td>CAST TIME</td>
<td>8:16 AM</td>
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<tr>
<td>TICKET NUMBER</td>
<td>306232806</td>
<td>FINISH TIME</td>
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<table>
<thead>
<tr>
<th>CONCRETE TYPE</th>
<th>TYPE II / V</th>
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</thead>
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<tr>
<td>USE PANELS</td>
<td>1011631</td>
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<table>
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<tr>
<th>WEATHER</th>
<th>SUNNY</th>
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</thead>
<tbody>
<tr>
<td>AIR CONTENT %</td>
<td>N/A</td>
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<tr>
<td>VOLUME %</td>
<td>N/A</td>
</tr>
<tr>
<td>AMBENT TEMP</td>
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</tr>
<tr>
<td>KELLY BALL</td>
<td>4.5</td>
</tr>
<tr>
<td>CONCRETE TEMP.</td>
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<tr>
<td>CEMENTITY</td>
<td>N/A</td>
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<td>CYLINDERS</td>
<td>5</td>
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## COMPRESSION TEST RESULTS

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<tr>
<th>CYLINDER MARK</th>
<th>DIA. &quot;IN&quot;</th>
<th>DATE TESTED</th>
<th>AGE DAYS</th>
<th>MAX LOAD (LBS)</th>
<th>COMPR. STRENGTH (MPa)</th>
<th>TESTED BY</th>
<th>COMMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>6.01</td>
<td>7-20-23</td>
<td>6</td>
<td>131300</td>
<td>4650</td>
<td>DB</td>
<td></td>
</tr>
<tr>
<td>B</td>
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<td>6</td>
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<td>4560</td>
<td>DB</td>
<td></td>
</tr>
<tr>
<td>C</td>
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<td>7-20-23</td>
<td>6</td>
<td></td>
<td></td>
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<td>D</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[
\text{AVERAGE} = 4605
\]

## BREAK TYPES

- **TYPE 1**
- **TYPE 2**
- **TYPE 3**
- **TYPE 4**
- **TYPE 5**

## PRODUCT PRODUCED

- MSE PANELS

## DATE OF REPORT

7-20-23
# Original Test Data Sheet

<table>
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<th>Gauge Pressure - PSI</th>
<th>Target Load</th>
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<td></td>
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<td>1,000</td>
<td>24</td>
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<tr>
<td>1,500</td>
<td>31</td>
</tr>
<tr>
<td>2,000</td>
<td>31</td>
</tr>
<tr>
<td>2,500</td>
<td>31</td>
</tr>
<tr>
<td>3,000</td>
<td>31</td>
</tr>
<tr>
<td>3,500</td>
<td>31</td>
</tr>
<tr>
<td>4,000</td>
<td>31</td>
</tr>
<tr>
<td>4,200</td>
<td>TL1</td>
</tr>
<tr>
<td>4,600</td>
<td>TL2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Gauge 1</th>
<th>Gauge 2</th>
<th>Gauge 3</th>
<th>Gauge 4</th>
<th>Gauge 5</th>
<th>Gauge 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>11</td>
<td>10</td>
<td>19</td>
<td>19</td>
<td>24</td>
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<tr>
<td>84</td>
<td>82</td>
<td>82</td>
<td>82</td>
<td>82</td>
<td>82</td>
</tr>
</tbody>
</table>

Date: **July 24, 2023**

Witness: **L.L.**
7. Test Panel Details

**TYPICAL "Y" PANEL - #3 REBAR**

**SECTION A-A**

**PANEL EMBED TEST CONFIGURATION**
8. Test Photos

Figure 1 – Panel Reinforcement

Figure 2 – Assembling the test frame and pull rods
Figure 3 – Installation of the aggregate and top plate
Figure 4 – Test apparatus complete with dial indicators with connection number 1 is at the top of the photo.
Figure 5 – Disassembling the test setup to view the connectors

Figure 6 – Connection system and the aggregate removed and panel cleaned
Figure 7 – Connection 1
Figure 8 – Connection 2
Figure 9 – Connection 3
Figure 10 – Connection 4
Figure 11 – Connection 5
Figure 12 – Connection 6
1.3 Other Components
1.3.3 ERS Drainage
1.3.4 ERS Coping
CAST-IN-PLACE COPING
CROSS SECTION

- Expansion joints at 30" o.c.
- #4 U-bars @ 12" o.c.
- #4 x 2'-0" long, bowels @ 24" o.c.

NOTE: Expansion joints at 30'-0" o.c.

CAST-IN-PLACE COPING DETAIL

ADHERED ONLY WITH RESPECT TO INITIAL STABILITY OF REINFORCED BATH STRUCTURE
1.3.5 ERS Traffic Barrier
BARRIER WITH MOMENT SLAB DETAIL
1.3.6 ERS Slip Joint Detail
Section 2: ERS Design
2.1
Design Methodology
2.1.3 ERS Obstruction Design Detail
2.2
Design Example
2.2.1 Design Problems
AASHTO 2017-2020
AASHTO LRFD 75 years
MSE+: Update # 2021.14

PROJECT IDENTIFICATION

Title: AASHTO LRFD 75 years
Project Number: Problem 1 - Level Fill
Client: FSH
Designer: FSH

Description:
IDEA Evaluation, Strength 1

Company's information:
Name: SSL, LLC
Street: 4740 Scotts Valley Drive
        Suite E
        Scotts Valley, CA 95126
Telephone #: 831-430-9300
Fax #: 831-430-9340
E-Mail:

File path and name: C:\Users\fran\OneDrive - SSL\Documents\Technical submit.....yrs H=30.0' LC1.BENp

Original date and time of creating this file: Wed Dec 14 13:52:17 2016

PROGRAM MODE:
ANALYSIS of a SIMPLE STRUCTURE using METAL MATS/GRIDS 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 °

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, $\phi_{equiv.}$ 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) = 0.3333  (if batter is less than 10°, $K_a$ is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity is controlled by general shear.
Bearing capacity factors (calculated by MSEW):  $N_c = 30.14$  $N = 22.40$

SEISMICITY

Not Applicable
## INPUT DATA: Metal mats/grids (Analysis)

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<thead>
<tr>
<th>DATA</th>
<th>Metal mats type #1</th>
<th>Metal mats type #2</th>
<th>Metal mats type #3</th>
<th>Metal mats type #4</th>
<th>Metal mats type #5</th>
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</thead>
<tbody>
<tr>
<td>Yield strength of steel, ( F_y ) [kips/in²]</td>
<td>75.0</td>
<td>75.0</td>
<td>75.0</td>
<td>75.0</td>
<td>75.0</td>
</tr>
<tr>
<td>Gross width of grid, ( b ) [in]</td>
<td>24.0</td>
<td>32.0</td>
<td>40.0</td>
<td>24.0</td>
<td>24.0</td>
</tr>
<tr>
<td>Vertical spacing, ( S_v ) [ft]</td>
<td>Varies</td>
<td>Varies</td>
<td>Varies</td>
<td>Varies</td>
<td>Varies</td>
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<tr>
<td>Design cross section area, ( A_c ) [in²]</td>
<td>0.32</td>
<td>0.40</td>
<td>0.48</td>
<td>0.63</td>
<td>0.32</td>
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<td>Thickness of transverse element, ( t ) [in]</td>
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<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
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<tr>
<td>Distance between transverse bars, ( S_t ) [in]</td>
<td>18.0</td>
<td>30.0</td>
<td>36.0</td>
<td>30.0</td>
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<td>34.00</td>
<td>34.00</td>
<td>34.00</td>
<td>34.00</td>
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<tr>
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<td>34.00</td>
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<td>34.00</td>
<td>34.00</td>
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<td>34.00</td>
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<tr>
<td>Pullout resistance factor, ( F^* )</td>
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<td>0.21</td>
<td>0.18</td>
<td>0.21</td>
<td>0.53</td>
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<tr>
<td>@ the top</td>
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<td>0.09</td>
<td>0.11</td>
<td>0.27</td>
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<tr>
<td>@ 19.7 ft or below</td>
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<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
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<tr>
<td>Scale-effect correction factor, ( \alpha )</td>
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<td>1.00</td>
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<td>1.00</td>
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### Variation of Lateral Earth Pressure Coefficient With Depth (Coherent Mass)

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<tr>
<th>( Z ) [ft]</th>
<th>( K / K_a )</th>
</tr>
</thead>
<tbody>
<tr>
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<td>1.56</td>
</tr>
<tr>
<td>3.3</td>
<td>1.47</td>
</tr>
<tr>
<td>6.6</td>
<td>1.37</td>
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<td>9.8</td>
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<td>13.1</td>
<td>1.19</td>
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<tr>
<td>16.4</td>
<td>1.09</td>
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<tr>
<td>19.7</td>
<td>1.00</td>
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![Graph showing variation of lateral earth pressure coefficient with depth](image-url)
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 $\gamma_l = 150.00 \text{ lb/ft}^3$

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<th>Z / Hd</th>
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<tr>
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<td>1.00</td>
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<tr>
<td>0.25</td>
<td>1.00</td>
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<td>0.50</td>
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<tr>
<td>0.75</td>
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</tr>
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<td>1.00</td>
<td>1.00</td>
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</table>

Connection strength, T-lot, is related to T-ult.

<table>
<thead>
<tr>
<th>D A T A (for connection only)</th>
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<th>Type #2</th>
<th>Type #3</th>
<th>Type #4</th>
<th>Type #5</th>
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<td>Product Name</td>
<td>4W11x1.50..</td>
<td>5W11x2.50..</td>
<td>6W11x3.00..</td>
<td>4W20x2.50..</td>
<td>4W11x1.00..</td>
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<tr>
<td>Strength reduction at the connection, $CR_u = \frac{F_{yc}}{F_y}$</td>
<td>0.98</td>
<td>0.98</td>
<td>0.98</td>
<td>0.98</td>
<td>0.98</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 \}

Soil in front of wall is Horizontal.

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/ft$^2$], and live load is 250.0 [lb/ft$^2$]

ANALYZED REINFORCEMENT LAYOUT:

SCALE:

0  5  10  15  20  25  30 [ft]
AASHTO 2017-2020 – Load and Resisting Factors

INTERNAL STABILITY

Load factor for vertical earth pressure, EV:  \( \gamma_{p-EV} \)  1.35
Load factor for earthquake loads, EQ:  \( \gamma_{p-EQ} \)  1.00
Load factor for live load surcharge, LS:  \( \gamma_{p-LS} \)  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
\( \phi \)  Static  Combined static/seismic
Metal Mats:  0.65  0.85

Resistance factor for reinforcement tension in connectors
\( \phi \)  Static  Combined static/seismic
Metal Mats:  0.65  0.85

Resistance factor for Metal Mats pullout
\( \phi \)  0.90  1.20

EXTERNAL STABILITY

Load factor for vertical earth pressure, EV
\( \gamma_{p-EV} \)  1.00  \( \gamma_{p-EQ} \)  1.00
Sliding and Eccentricity
Bearing Capacity  \( \gamma_{p-EV} \)  1.35  \( \gamma_{p-EQ} \)  1.35

Load factor of active lateral earth pressure, EH  \( \gamma_{p-EH} \)  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 (multiplies \( P_{AE} \) and \( P_{IR} \)):  \( \gamma_{p-EQ} \)  1.00

Resistance factor for shear resistance along common interfaces
\( \phi_{\tau} \)  Static  Combined Static/Seismic
Reinforced Soil and Foundation  1.00  1.00
Reinforced Soil and Reinforcement  1.00  1.00

Resistance factor for bearing capacity of shallow foundation
\( \phi_{b} \)  Static  Combined Static/Seismic
  0.65  0.90
ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, CDR = 1.71, factored bearing load = 7956 lb/ft².
Foundation Interface: Direct sliding, CDR = 1.565, Eccentricity, e/L = 0.1879, CDR-overturning = 2.66

<table>
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<tr>
<th>METAL MATS</th>
<th>CONNECTION</th>
<th>Metal mats strength CDR</th>
<th>Pullout resistance CDR</th>
<th>Direct sliding CDR</th>
<th>Eccentricity e/L</th>
<th>Product name</th>
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<tr>
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<td>Length [ft]</td>
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<td>CDR [connection break]</td>
<td>CDR Strength</td>
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BEARING CAPACITY for GIVEN LAYOUT – Using AASHTO 2017-2020 method

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<th>SEISMIC</th>
<th>UNITS</th>
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<td>Factored bearing resistance, $q_n$</td>
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<td>[lb/ft²]</td>
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<tr>
<td>Factored bearing load, $\sigma_v$</td>
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<td>[lb/ft²]</td>
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<tr>
<td>Eccentricity, $e$</td>
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<td>[ft]</td>
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<tr>
<td>Eccentricity, $e/L$</td>
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<td>[ft]</td>
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<tr>
<td>Base length</td>
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</table>

(Water table does not affect bearing capacity)

Unfactored applied bearing pressure $= \frac{(\text{Unfactored } R)}{[\text{L} - 2 \times (\text{Unfactored } e)]} =$

Unfactored $R = 90299.94$ [lb/ft], $L = 21.00$, Unfactored $e = 2.41$ [ft], and Sigma = 5580.02 [lb/ft²]

**SCALE:**

0 5 10 15 20 25 30 [ft]
### DIRECT SLIDING for GIVEN LAYOUT  (for METAL MATS/GRIDS reinforcements)

Along reinforced and foundation soils interface: CDR-static = 1.565

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<th>Metal Mats Elevation [ft]</th>
<th>Metal Mats Length [ft]</th>
<th>CDR Static</th>
<th>CDI: Seismic</th>
<th>Metal Mats Type #</th>
<th>Product name</th>
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<td>21.00</td>
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<td>8.659</td>
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</table>

### ECCENTRICITY for GIVEN LAYOUT  (for Coherent Gravity Mass Method)

At interface with foundation: e/L static = 0.1879; Overturning: CDR-static = 2.66

<table>
<thead>
<tr>
<th>#</th>
<th>Metal Mats Elevation [ft]</th>
<th>Metal Mats Length [ft]</th>
<th>e / L Static</th>
<th>e / L Seismic</th>
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### RESULTS for STRENGTH

[Note: Actual CDR = (Yield stress) / (Actual stress)]

For Coherent Mass Method, Option A

Live Load included in calculating Tmax

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<th>#</th>
<th>Metal Mats Elevation [ft]</th>
<th>Coverage ratio, Rc=b/Sh</th>
<th>Horizontal spacing, Sh [ft]</th>
<th>LTDS = 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>
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### RESULTS for PULLOUT

Live Load included in calculating Tmax

Note: Live load is not included in calculating the overburden pressure used to assess pullout resistance.

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### RESULTS for CONNECTION (static conditions)

Live Load included in calculating Tmax

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AASHTO 2017-2020
AASHTO LRFD 75 years
MSE+ : Update # 2021.14

PROJECT IDENTIFICATION

Title: AASHTO LRFD 75 years
Project Number: Problem 2 - Sloping Fill
Client: FSH
Designer: FSH
Station Number:

Description:
IDEA Evaluation, Strength 1

Company's information:

Name: SSL, LLC
Street: 4740 Scotts Valley Drive
        Suite E
        Scotts Valley, CA 95126
Telephone #: 831-430-9300
Fax #: 831-430-9340
E-Mail:

File path and name: C:\\Users\fran\OneDrive - SSL\Documents\Technical submit.....
.....yrs H=30.0' LC2.BENp
Original date and time of creating this file: Wed Dec 14 13:52:17 2016

PROGRAM MODE:
ANALYSIS
of a SIMPLE STRUCTURE
using METAL MATS/GRIDS 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, \( \gamma_{\text{equiv.}} \) 120.0 lb/ft\(^3\)  
Equivalent internal angle of friction, \( \phi_{\text{equiv.}} \) 30.0 °  
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  
Ka (external stability) = 0.5365

BEARING CAPACITY
Bearing capacity is controlled by general shear.
Bearing capacity factors (calculated by MSEW):  \( N_c = 30.14 \)  
\( N \gamma = 22.40 \)

SEISMICITY
Not Applicable
**INPUT DATA: Metal mats/grids (Analysis)**

<table>
<thead>
<tr>
<th>D A T A</th>
<th>Metal mats type #1</th>
<th>Metal mats type #2</th>
<th>Metal mats type #3</th>
<th>Metal mats type #4</th>
<th>Metal mats type #5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield strength of steel, $F_y$ [kips/in$^2$]</td>
<td>75.0</td>
<td>75.0</td>
<td>75.0</td>
<td>75.0</td>
<td>75.0</td>
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<tr>
<td>Gross width of grid, $b$ [in]</td>
<td>24.0</td>
<td>32.0</td>
<td>40.0</td>
<td>24.0</td>
<td>24.0</td>
</tr>
<tr>
<td>Vertical spacing, $S_v$ [ft]</td>
<td>Varies</td>
<td>Varies</td>
<td>Varies</td>
<td>Varies</td>
<td>Varies</td>
</tr>
<tr>
<td>Design cross section area, $A_c$ [in$^2$]</td>
<td>0.32</td>
<td>0.40</td>
<td>0.48</td>
<td>0.63</td>
<td>0.32</td>
</tr>
<tr>
<td>Thickness of transverse element, $t$ [in]</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Distance between transverse bars, $S_t$ [in]</td>
<td>18.0</td>
<td>30.0</td>
<td>36.0</td>
<td>30.0</td>
<td>12.0</td>
</tr>
<tr>
<td>Friction angle along reinforcement-soil interface, $\rho$</td>
<td>34.00</td>
<td>34.00</td>
<td>34.00</td>
<td>34.00</td>
<td>34.00</td>
</tr>
<tr>
<td>@ the top</td>
<td>34.00</td>
<td>34.00</td>
<td>34.00</td>
<td>34.00</td>
<td>34.00</td>
</tr>
<tr>
<td>@ 19.7 ft or below</td>
<td>34.00</td>
<td>34.00</td>
<td>34.00</td>
<td>34.00</td>
<td>34.00</td>
</tr>
<tr>
<td>Pullout resistance factor, $F^*$</td>
<td>0.35</td>
<td>0.21</td>
<td>0.18</td>
<td>0.21</td>
<td>0.53</td>
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<tr>
<td>@ the top</td>
<td>0.18</td>
<td>0.11</td>
<td>0.09</td>
<td>0.11</td>
<td>0.27</td>
</tr>
<tr>
<td>@ 19.7 ft or below</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
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</table>

**Variation of Lateral Earth Pressure Coefficient With Depth (Coherent Mass)**

<table>
<thead>
<tr>
<th>$Z$ [ft]</th>
<th>$K / K_a$</th>
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<tr>
<td>0</td>
<td>1.56</td>
</tr>
<tr>
<td>3.3</td>
<td>1.47</td>
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<tr>
<td>6.6</td>
<td>1.37</td>
</tr>
<tr>
<td>9.8</td>
<td>1.28</td>
</tr>
<tr>
<td>13.1</td>
<td>1.19</td>
</tr>
<tr>
<td>16.4</td>
<td>1.09</td>
</tr>
<tr>
<td>19.7</td>
<td>1.00</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 \( \gamma_f = 150.00 \text{ lb/ft}^3 \)

<table>
<thead>
<tr>
<th>Z / Hd</th>
<th>To-static / Tmax</th>
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<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>
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</table>

Connection strength, T-lot, is related to T-ult

<table>
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<tr>
<th>Type #1</th>
<th>Type #2</th>
<th>Type #3</th>
<th>Type #4</th>
<th>Type #5</th>
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<tbody>
<tr>
<td>Product Name</td>
<td>4W11x1.50..</td>
<td>5W11x2.50..</td>
<td>6W11x3.00..</td>
<td>4W20x2.50..</td>
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<tr>
<td>CRu = Fyc / Fy</td>
<td>0.98</td>
<td>0.98</td>
<td>0.98</td>
<td>0.98</td>
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</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 }

Soil in front of wall is Horizontal.

- Batter, \(\omega \) 0.0 [deg]
- Backslope, \(\beta \) 26.6 [deg]
- Backslope rise 30.0 [ft]  
  Broken back equivalent angle, \(I = 26.56^\circ\)  (see Fig. 25 in DEMO 82)

UNIFORM SURCHARGE

Uniformly distributed dead load is 0.0 [lb/ft\(^2\)]

ANALYZED REINFORCEMENT LAYOUT:
AASHTO 2017-2020 – Load and Resisting Factors

INTERNAL STABILITY

Load factor for vertical earth pressure, EV: \( \gamma_{p-EV} \) 1.35
Load factor for earthquake loads, EQ: \( \gamma_{p-EQ} \) 1.00

Load factor for live load surcharge, LS: \( \gamma_{p-LS} \) 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 \( \phi \) Static Combined static/seismic
Metal Mats: 0.65 0.85

Resistance factor for reinforcement tension in connectors \( \phi \) Static Combined static/seismic
Metal Mats: 0.65 0.85

Resistance factor for Metal Mats pullout \( \phi \) 0.90 1.20

EXTERNAL STABILITY

Load factor for vertical earth pressure, EV
\begin{align*}
\text{Sliding and Eccentricity} & : \gamma_{p-EV} \quad \gamma_{p-EQ} \quad 1.00 \\
\text{Bearing Capacity} & : \gamma_{p-EV} \quad \gamma_{p-EQ} \quad 1.35
\end{align*}

Load factor of active lateral earth pressure, EH: \( \gamma_{p-EH} \) 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 (multiplies \( P_{AE} \) and \( P_{IR} \)): \( \gamma_{p-EQ} \) 1.00

Resistance factor for shear resistance along common interfaces
\begin{align*}
\text{Reinforced Soil and Foundation} & : \phi_{c} \quad 1.00 \\
\text{Reinforced Soil and Reinforcement} & : \phi_{c} \quad 1.00
\end{align*}

Resistance factor for bearing capacity of shallow foundation \( \phi_{b} \) Static Combined Static/Seismic
Metal Mats: 0.65 0.90
### ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, $CDR = 1.15$, factored bearing load = $11382 \text{ lb/ft}^2$.

Foundation Interface: Direct sliding, $CDR = 1.103$, Eccentricity, $e/L = 0.1873$, $CDR$-overturning = 1.93

<table>
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<th>#</th>
<th>Elevation [ft]</th>
<th>Length [ft]</th>
<th>Type</th>
<th>Connection CDR [connection break]</th>
<th>CDR</th>
<th>Metal mats strength CDR</th>
<th>Pullout resistance CDR</th>
<th>Direct sliding CDR</th>
<th>Eccentricity e/L</th>
<th>Product name</th>
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<td>1.25</td>
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<td>1.02</td>
<td>1.04</td>
<td>1.039</td>
<td>1.201</td>
<td>1.314</td>
<td>0.1723</td>
<td>4W20x2.50W11</td>
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<td>3.75</td>
<td>21.00</td>
<td>4</td>
<td>1.17</td>
<td>1.19</td>
<td>1.194</td>
<td>1.198</td>
<td>1.370</td>
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<td>4W20x2.50W11</td>
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<td>1.04</td>
<td>1.037</td>
<td>1.637</td>
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<td>1.17</td>
<td>1.19</td>
<td>1.194</td>
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<td>1.608</td>
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<td>1.98</td>
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<td>2.017</td>
<td>2.756</td>
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<td>2.44</td>
<td>2.437</td>
<td>2.725</td>
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<td>-0.1092</td>
<td>4W11x1.50W11</td>
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BEARING CAPACITY for GIVEN LAYOUT – Using AASHTO 2017-2020 method

(Water table does not affect bearing capacity)

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<tr>
<th>STATIC</th>
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<th>UNITS</th>
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<tr>
<td>Factored bearing resistance, q-n</td>
<td>13073</td>
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Unfactored applied bearing pressure = (Unfactored R) / [ L - 2 * (Unfactored e) ] =

Unfactored R = 123535.85 [lb/ft], L = 21.00, Unfactored e = 2.73 [ft], and Sigma = 7951.69 [lb/ft²]
**DIRECT SLIDING for GIVEN LAYOUT**  
(for METAL MATS/GRIDS reinforcements)

Along reinforced and foundation soils interface: CDR-static = 1.103

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<th>CDI: Seismic</th>
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**ECCENTRICITY for GIVEN LAYOUT**  
(for Coherent Gravity Mass Method)

At interface with foundation: e/L static = 0.1873; Overturning: CDR-static = 1.93

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### RESULTS for STRENGTH

[Note: Actual CDR = (Yield stress) / (Actual stress)]

For Coherent Mass Method, Option A

**Live Load included in calculating Tmax**

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<th>Tmd [lb/ft]</th>
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### RESULTS for PULLOUT

**Live Load included in calculating Tmax**

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## RESULTS for CONNECTION (static conditions)

Live Load included in calculating Tmax

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AASHTO 2017-2020
AASHTO LRFD 75 years

PROJECT IDENTIFICATION

Title: AASHTO LRFD 75 years
Project Number: Abutment on Piles
Client: FSH
Designer: FSH
Station Number:

Description:

Strength 1

Company's information:

Name: SSL, LLC
Street: 4740 Scotts Valley Drive
        Suite E
        Scotts Valley, CA 95126
Telephone #: 831-430-9300
Fax #: 831-430-9340
E-Mail:

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.....utment on piles.BENp
Original date and time of creating this file: Wed Dec 14 13:52:17 2016

PROGRAM MODE:

ANALYSIS
of a BRIDGE ABUTMENT
using METAL MATS/GRIDS as reinforcing material.
SOIL DATA

Soil above reinforcement has the following properties:

**Unit weight, \( \gamma \)**
- **120.0 lb/ft\(^3\)**

**Design value of internal angle of friction, \( \phi \)**
- **30.0 \(^\circ\)**

**REINFORCED SOIL**

**Unit weight, \( \gamma \)**
- **135.0 lb/ft\(^3\)**

**Design value of internal angle of friction, \( \phi \)**
- **34.0 \(^\circ\)**

**RETAINED SOIL**

**Unit weight, \( \gamma \)**
- **120.0 lb/ft\(^3\)**

**Design value of internal angle of friction, \( \phi \)**
- **30.0 \(^\circ\)**

**FOUNDATION SOIL (Considered as an equivalent uniform soil)**

**Equivalent unit weight, \( \gamma_{\text{equiv.}} \)**
- **120.0 lb/ft\(^3\)**

**Equivalent internal angle of friction, \( \phi_{\text{equiv.}} \)**
- **30.0 \(^\circ\)**

**Equivalent cohesion, \( c_{\text{equiv.}} \)**
- **0.0 lb/ft\(^2\)**

Factored bearing capacity resistance of foundation is given: \( q_{\text{ult-static}} = 13625.0 \text{ lb/ft}^2 \).

LATERAL EARTH PRESSURE COEFFICIENTS

- **\( K_a \) (internal stability) = 0.2827**  (if batter is less than 10\(^\circ\), \( K_a \) is calculated from eq. 15. Otherwise, eq. 38 is utilized)
- **\( K_a \) (external stability) = 0.3333**  (if batter is less than 10\(^\circ\), \( K_a \) is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity is controlled by general shear.

Bearing capacity factors (calculated by MSEW): \( N_c = N/A \) \( N_{\gamma} = N/A \)

SEISMICITY

Not Applicable
INPUT DATA: Metal mats/ grids  
(Analysis)

<table>
<thead>
<tr>
<th>DATA</th>
<th>Metal mats</th>
<th>Metal mats</th>
<th>Metal mats</th>
<th>Metal mats</th>
<th>Metal mats</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>type #1</td>
<td>type #2</td>
<td>type #3</td>
<td>type #4</td>
<td>type #5</td>
</tr>
<tr>
<td>Yield strength of steel, Fy [kips/in²]</td>
<td>75.0</td>
<td>75.0</td>
<td>75.0</td>
<td>75.0</td>
<td>75.0</td>
</tr>
<tr>
<td>Gross width of grid, b [in]</td>
<td>24.0</td>
<td>24.0</td>
<td>24.0</td>
<td>24.0</td>
<td>24.0</td>
</tr>
<tr>
<td>Vertical spacing, Sv [ft]</td>
<td>Varies</td>
<td>Varies</td>
<td>Varies</td>
<td>Varies</td>
<td>Varies</td>
</tr>
<tr>
<td>Design cross section area, Ac [in²]</td>
<td>0.32</td>
<td>0.46</td>
<td>Varies</td>
<td>0.46</td>
<td>0.63</td>
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<tr>
<td>Thickness of transverse element, t [in]</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
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<tr>
<td>Distance between transverse bars, St [in]</td>
<td>12.0</td>
<td>12.0</td>
<td>18.0</td>
<td>24.0</td>
<td>18.0</td>
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<tr>
<td>Friction angle along reinforcement-soil interface, (\phi)</td>
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<td>34.00</td>
<td>34.00</td>
<td>34.00</td>
<td>34.00</td>
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<tr>
<td>Pullout resistance factor, (F^*)</td>
<td>Varies</td>
<td>Varies</td>
<td>Varies</td>
<td>Varies</td>
<td>Varies</td>
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<tr>
<td>Scale-effect correction factor, (\alpha)</td>
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<td>1.00</td>
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Variation of Lateral Earth Pressure Coefficient With Depth (Coherent Mass)

<table>
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<tr>
<th>Z [ft]</th>
<th>K / Ka</th>
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<tr>
<td>0</td>
<td>1.56</td>
</tr>
<tr>
<td>3.3</td>
<td>1.47</td>
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<tr>
<td>6.6</td>
<td>1.37</td>
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<td>9.8</td>
<td>1.28</td>
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<tr>
<td>13.1</td>
<td>1.19</td>
</tr>
<tr>
<td>16.4</td>
<td>1.09</td>
</tr>
<tr>
<td>19.7</td>
<td>1.00</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 \( \gamma_f = 150.00 \text{ lb/ft}^3 \)

Connection strength, T-lot, is related to T-ult

<table>
<thead>
<tr>
<th>Z / Hd</th>
<th>To-static / Tmax</th>
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</thead>
<tbody>
<tr>
<td>0.00</td>
<td>1.00</td>
</tr>
<tr>
<td>0.25</td>
<td>1.00</td>
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<tr>
<td>0.50</td>
<td>1.00</td>
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<td>0.75</td>
<td>1.00</td>
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<tr>
<td>1.00</td>
<td>1.00</td>
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<table>
<thead>
<tr>
<th>Z / H</th>
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<th>0.25</th>
<th>0.50</th>
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<tr>
<td>To-static / Tmax</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
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<th>Type #3</th>
<th>Type #4</th>
<th>Type #5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Product Name</td>
<td>4W11x1.00..</td>
<td>4W15x1.00..</td>
<td>4W15x1.50..</td>
<td>4W15x2.00..</td>
</tr>
<tr>
<td>Strength reduction at the connection, CRu = Fyc / Fy</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>
INPUT DATA: Geometry and Surcharge loads (of a BRIDGE ABUTMENT)

Design height, \(H_d\) 20.00 [ft] \{ Embedded depth is \(E = 0.00\) ft, and height above top of finished bottom grade is \(H = 20.00\) ft \}

Soil in front of wall is Horizontal.

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]

ABUTMENT GEOMETRY (On pile foundation.)
2.17
Footing's dimension: height, \(h' = 10.00\), width, \(h = 1.50\), and thickness, \(t = 3.50\) [ft].
Dimensions of bridge bearing plate: height, \(f_h = 0.30\), width, \(f_w = 1.00\) [ft].

ANALYZED REINFORCEMENT LAYOUT:

SCALE:

0 2 4 6 8 10[ft]
AASHTO 2017-2020 – Load and Resisting Factors

INTERNAL STABILITY

Load factor for vertical earth pressure, EV:
\[ \gamma_{p-EV} = 1.35 \]
Load factor for earthquake loads, EQ:
\[ \gamma_{p-EQ} = 1.00 \]
Load factor for live load surcharge, LS:
\[ \gamma_{p-LS} = 1.75 \]
(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
\[ \phi \]
Metal Mats:
Static: 0.65
Combined static/seismic: 0.85

Resistance factor for reinforcement tension in connectors
\[ \phi \]
Metal Mats:
Static: 0.65
Combined static/seismic: 0.85

Resistance factor for Metal Mats pullout
\[ \phi \]
Static: 0.90
Combined static/seismic: 1.20

EXTERNAL STABILITY

Load factor for vertical earth pressure, EV
\[ \gamma_{p-EV} \]
Sliding and Eccentricity
\[ \gamma_{p-EV} = 1.00 \]
Bearing Capacity
\[ \gamma_{p-EV} = 1.35 \]

Load factor of active lateral earth pressure, EH
\[ \gamma_{p-EH} = 1.50 \]

Load factor of active lateral earth pressure during earthquake (does not multiply \( P_{AE} \) and \( P_{IR} \)):
\[ \gamma_{p-EH}^{AE} = 1.50 \]

Load factor for earthquake loads, EQ (multiplies \( P_{AE} \) and \( P_{IR} \)):
\[ \gamma_{p-EQ} = 1.00 \]

Resistance factor for shear resistance along common interfaces
\[ \phi_{t} \]
Reinforced Soil and Foundation: 1.00
Reinforced Soil and Reinforcement: 1.00

Resistance factor for bearing capacity of shallow foundation
\[ \phi_{b} \]
Static: 0.65
Combined Static/Seismic: 0.90
**ANALYSIS: CALCULATED FACTORS (Static conditions)**

Bearing capacity, CDR = 1.10, factored bearing load = 8081 lb/ft².

Foundation Interface: Direct sliding, CDR = 1.106, Eccentricity, e/L = 0.2926, CDR-overturning = 1.63

<table>
<thead>
<tr>
<th>Elevation</th>
<th>Length</th>
<th>Type</th>
<th>CDR</th>
<th>CDR</th>
<th>CDR</th>
<th>CDR</th>
<th>E/L</th>
<th>Product name</th>
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<tbody>
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<td># 1 1.25</td>
<td>17.00</td>
<td>5</td>
<td>1.06</td>
<td>1.06</td>
<td>1.060</td>
<td>1.302</td>
<td>1.332</td>
<td>0.2685</td>
</tr>
<tr>
<td>2 3.75</td>
<td>17.00</td>
<td>5</td>
<td>1.28</td>
<td>1.28</td>
<td>1.279</td>
<td>1.488</td>
<td>1.418</td>
<td>0.2221</td>
</tr>
<tr>
<td>3 6.25</td>
<td>17.00</td>
<td>3</td>
<td>1.11</td>
<td>1.11</td>
<td>1.108</td>
<td>1.668</td>
<td>1.514</td>
<td>0.1779</td>
</tr>
<tr>
<td>4 8.75</td>
<td>17.00</td>
<td>3</td>
<td>1.33</td>
<td>1.33</td>
<td>1.333</td>
<td>1.828</td>
<td>1.621</td>
<td>0.1355</td>
</tr>
<tr>
<td>5 11.25</td>
<td>17.00</td>
<td>3</td>
<td>1.43</td>
<td>1.43</td>
<td>1.426</td>
<td>1.640</td>
<td>1.741</td>
<td>0.0941</td>
</tr>
<tr>
<td>6 13.75</td>
<td>17.00</td>
<td>3</td>
<td>1.35</td>
<td>1.35</td>
<td>1.355</td>
<td>1.247</td>
<td>1.870</td>
<td>0.0525</td>
</tr>
<tr>
<td>7 16.25</td>
<td>17.00</td>
<td>3</td>
<td>1.31</td>
<td>1.31</td>
<td>1.306</td>
<td>1.001</td>
<td>2.003</td>
<td>0.0080</td>
</tr>
<tr>
<td>8 18.75</td>
<td>17.00</td>
<td>2</td>
<td>1.30</td>
<td>1.30</td>
<td>1.298</td>
<td>1.308</td>
<td>2.117</td>
<td>-0.0456</td>
</tr>
</tbody>
</table>
BEARING CAPACITY for GIVEN LAYOUT – Using AASHTO 2017-2020 method

<table>
<thead>
<tr>
<th>Static</th>
<th>Seismic</th>
<th>Units</th>
</tr>
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<tbody>
<tr>
<td>Factored bearing resistance, q-n</td>
<td>8856</td>
<td>N/A</td>
</tr>
<tr>
<td>Factored bearing load, σv</td>
<td>8080.7</td>
<td>N/A</td>
</tr>
<tr>
<td>Eccentricity, e</td>
<td>3.16</td>
<td>N/A</td>
</tr>
<tr>
<td>Eccentricity, e/L</td>
<td>0.186</td>
<td>N/A</td>
</tr>
<tr>
<td>CDR calculated</td>
<td>1.10</td>
<td>N/A</td>
</tr>
<tr>
<td>Base length</td>
<td>17.00</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Unfactored applied bearing pressure = \( \frac{\text{Unfactored R}}{L - 2 \times \text{Unfactored e}} \)

Unfactored R = 63053.46 [lb/ft], L = 17.00, Unfactored e = 2.75 [ft], and Sigma = 5479.30 [lb/ft²]
DIRECT SLIDING for GIVEN LAYOUT  (for METAL MATS/GRIDS reinforcements)

Along reinforced and foundation soils interface:  CDR-static = 1.106

<table>
<thead>
<tr>
<th>#</th>
<th>Metal Mats Elevation [ft]</th>
<th>Metal Mats Length [ft]</th>
<th>CDR Static</th>
<th>CDI: Seismic</th>
<th>Metal Mats Type #</th>
<th>Product name</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>1.25</td>
<td>17.00</td>
<td>1.332</td>
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<td>5</td>
<td>4W20x1.50W11</td>
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<tr>
<td>2</td>
<td>3.75</td>
<td>17.00</td>
<td>1.418</td>
<td>N/A</td>
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<td>4W20x1.50W11</td>
</tr>
<tr>
<td>3</td>
<td>6.25</td>
<td>17.00</td>
<td>1.514</td>
<td>N/A</td>
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<td>4W15x1.50W11</td>
</tr>
<tr>
<td>4</td>
<td>8.75</td>
<td>17.00</td>
<td>1.621</td>
<td>N/A</td>
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<td>4W15x1.50W11</td>
</tr>
<tr>
<td>5</td>
<td>11.25</td>
<td>17.00</td>
<td>1.741</td>
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<td>6</td>
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<td>7</td>
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<td>17.00</td>
<td>2.117</td>
<td>N/A</td>
<td>2</td>
<td>4W15x1.00W11</td>
</tr>
</tbody>
</table>

ECCENTRICITY for GIVEN LAYOUT  (for Coherent Gravity Mass Method)

At interface with foundation:  e/L static = 0.2926; Overturning: CDR-static = 1.63

<table>
<thead>
<tr>
<th>#</th>
<th>Metal Mats Elevation [ft]</th>
<th>Metal Mats Length [ft]</th>
<th>e / L Static</th>
<th>e / L Seismic</th>
<th>Metal Mats Type #</th>
<th>Product name</th>
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<tbody>
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<tr>
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<td>3.75</td>
<td>17.00</td>
<td>0.2221</td>
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<td>5</td>
<td>4W20x1.50W11</td>
</tr>
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<td>3</td>
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<td>4W15x1.50W11</td>
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<td>8.75</td>
<td>17.00</td>
<td>0.1355</td>
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<td>4W15x1.50W11</td>
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<td>N/A</td>
<td>2</td>
<td>4W15x1.00W11</td>
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</table>
### RESULTS for STRENGTH

[Note: Actual CDR = (Yield stress) / (Actual stress)]

For Coherent Mass Method, Option A

Live Load included in calculating Tmax

<table>
<thead>
<tr>
<th>#</th>
<th>Metal Mats Elevation [ft]</th>
<th>Coverage ratio, Rc=b/Sh</th>
<th>Horizontal spacing, Sh [ft]</th>
<th>LTDS = 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>
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<td>1</td>
<td>1.25</td>
<td>0.400</td>
<td>5.000</td>
<td>6171.8</td>
<td>5822.0</td>
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<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
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<td>0.400</td>
<td>5.000</td>
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<td>N/A</td>
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<td>N/A</td>
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<td>8.75</td>
<td>0.400</td>
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<td>N/A</td>
<td>N/A</td>
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<td>N/A</td>
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<td>N/A</td>
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<tr>
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<td>18.75</td>
<td>0.400</td>
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### RESULTS for PULLOUT

Live Load included in calculating Tmax

NOTE: Live load is not included in calculating the overburden pressure used to assess pullout resistance.

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### RESULTS for CONNECTION (static conditions)

*Live Load included in calculating Tmax*

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Section 3: CONSTRUCTION
3.1
Construction Procedures
3.1.2 ERS Construction Manual
MSEPLUS RETAINING WALL SYSTEMS

PRECAST PANEL FACED
MECHANICALLY STABILIZED
EARTH (MSE) WALL
INSTALLATION MANUAL

STANDARD PANEL SIZES

SSL, LLC. SUPPLIES PRECAST CONCRETE PANELS, WELDED WIRE SOIL REINFORCEMENT AND ACCESSORIES TO BE USED IN CONJUNCTION WITH OTHER MATERIALS IN THE CONSTRUCTION OF MSE PLUS RETAINING WALLS. THESE WALLS ARE DETAILED IN THE SHOP DRAWINGS TO CONFORM TO THE REQUIREMENTS OF THE OWNER'S CONTRACT DOCUMENTS. THE MSE PLUS™ WALL INSTALLATION MANUAL FURNISHED HEREIN PROVIDES ONLY A GENERAL EXPLANATION OF THE SYSTEM.

THE CONTRACTOR IS SOLELY RESPONSIBLE TO DEVISE AND CARRY OUT A PLAN FOR ERECTION AND ASSEMBLY OF COMPONENTS, WHICH ARE PROJECT SPECIFIC. SAID PLAN SHALL PROVIDE FOR UNLOADING, HANDLING, PLACING, AND BRACING COMPONENT MATERIALS WITHOUT RELYING ON THE CONCEPTUAL DRAWINGS AND GUIDELINES PROVIDED BY SSL.

ACKNOWLEDGMENT OF THE RECEIPT AND REVIEW OF THE SSL WALL INSTALLATION MANUAL DOES NOT RELIEVE THE WALL CONTRACTOR OF RESPONSIBILITY TO COMPLY WITH THE PROJECT SPECIFICATIONS AND DRAWINGS OR STRICT COMPLIANCE WITH ALL FALL PROTECTION, LAWS, SAFETY STANDARDS, AND PROCEDURES DURING THE PERFORMANCE OF JOB SITE WORK. PRECAUTIONS SHALL BE TAKEN TO PROTECT INSPECTORS, SUPERVISORS AND TRADESMAN FROM INJURY DURING WALL ERECTION.

SSL LLC
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SCOTTS VALLEY, CA 95066
P: [831] 430-9300 | F: [831] 430-9340

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# INSTALLATION MANUAL

## INTRODUCTION .......................................................................................................................... 6

## TERMINOLOGY .......................................................................................................................... 8

## GENERAL GUIDELINES ............................................................................................................. 10

## ACCEPTANCE OF MATERIAL ..................................................................................................... 10

## STORAGE AND HANDLING OF PANELS .................................................................................... 10

## SETTING OF CONCRETE FASCIA PANELS .............................................................................. 17

## FILTER FABRIC ....................................................................................................................... 20

## INITIAL PLACEMENT OF BACKFILL ....................................................................................... 21

## INSTALLATION OF SOIL REINFORCEMENT .......................................................................... 21

## BEARING PADS ...................................................................................................................... 23

## VERTICAL PANEL JOINT PAD ............................................................................................... 24

## ADDITIONAL PANEL PLACEMENT ......................................................................................... 25

## ADDITIONAL BACKFILL PLACEMENT ..................................................................................... 26

## OBSTRUCTIONS ...................................................................................................................... 28

## DRAINAGE ............................................................................................................................ 28

## FINISH GRADE PLACEMENT .................................................................................................... 28

## INSTALLATION SEQUENCE .................................................................................................... 29

## WALL ERECTION CHECK LIST .................................................................................................. 36

## SAMPLE INSTALLATION PHOTOS .......................................................................................... 38

## PROJECT NOTES ..................................................................................................................... 43
List of Figures

Figure 1: Typical Wall Cross Section .............................................................................................................. 7
Figure 2: Panel Storage ..................................................................................................................................... 11
Figure 3: Panel Handling ................................................................................................................................. 12
Figure 4: Mesh Handling ................................................................................................................................. 13
Figure 5: Standard 5' x 6' Precast Panel ......................................................................................................... 14
Figure 6: Typical Mesh Detail ........................................................................................................................... 15
Figure 7: Typical Soil Reinforcing Bend Detail .................................................................................................. 15
Figure 8: Mesh Callout Detail ........................................................................................................................... 15
Figure 9: Typical Wall Elevation ..................................................................................................................... 16
Figure 10: Bottom Row of Panels ................................................................................................................... 17
Figure 11: Panel Layout Line ........................................................................................................................... 18
Figure 12: Nominal Panel Spacing .................................................................................................................. 18
Figure 13: Corner Panel Layout ....................................................................................................................... 19
Figure 14: Curved Layout Line Panel Spacing .................................................................................................. 19
Figure 15: Panel Batter ..................................................................................................................................... 20
Figure 16: Filter Fabric Placement .................................................................................................................. 21
Figure 17: Initial Placement of Backfill ........................................................................................................... 21
Figure 18: Mesh Connection by Wires ............................................................................................................. 22
Figure 19: Mesh Connection Detail 1 ............................................................................................................... 23
Figure 20: Mesh Connection Detail 2 ............................................................................................................... 23
Figure 21: Bearing Pads ................................................................................................................................... 24
Figure 22: Vertical Joint Pads ........................................................................................................................... 25
Figure 23: Installation of Upper Panels .......................................................................................................... 26
MSEPLUS™ PRECAST PANEL FACED MECHANICALLY STABILIZED EARTH WALL
INSTALLATION MANUAL

Figure 24: Additional Backfill Placement ............................................................................................................. 27
Figure 25: Installation Step One .......................................................................................................................... 29
Figure 26: Installation Step Two .......................................................................................................................... 29
Figure 27: Installation Step Three ...................................................................................................................... 30
Figure 28: Installation Step Four ........................................................................................................................ 30
Figure 29: Installation Step Five .......................................................................................................................... 31
Figure 30: Installation Step Six ............................................................................................................................. 31
Figure 31: Installation Step Seven ........................................................................................................................ 32
Figure 32: Installation Step Eight ......................................................................................................................... 33
Figure 33: Installation Step Eight Back View ....................................................................................................... 33
Figure 34: Installation Step Nine .......................................................................................................................... 34
Figure 35: Installation Step Ten ............................................................................................................................. 35

List of References

American Association of State Highway and Transportation Officials (AASHTO) Bridge Design Specifications

AASHTO Interim Specifications for Highway Bridges

FHWA Design of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes
INTRODUCTION

The MSEPlus™ Precast Panel Faced Mechanically Stabilized Earth (MSE) Wall consists of soil reinforcing mesh, compacted soil, precast facing panels, bearing pads and a connection pin. MSE walls use reinforced soil, a term used when multiple layers of mesh act as reinforcement in soils placed as fill. MSE walls are cost-effective soil retaining structures that can tolerate much larger settlements than reinforced concrete walls. By placing tensile reinforcing elements (soil reinforcing mesh), the strength of the soil can be improved significantly such that the vertical face of the reinforced soil is essentially self-supporting. Use of a facing system (precast concrete panels) to prevent soil raveling between the reinforcing elements allows for very steep slopes and vertical walls to be constructed safely. The fascia panels may be cast with a texture such as fractured fin to provide a pleasing finished appearance. In addition, multiple finishes can be used for the fascia panels to create a larger pattern across the wall.
Figure 1: Typical Wall Cross Section
TERMINOLOGY

Bearing Pad – is a hard rubber or plastic pad placed in the horizontal joints between precast segmental panels.

Connection Pin – is a pin made of structural steel that is used to connect the soil reinforcing mesh to the concrete fascia panels.

Coping – is the reinforced concrete element at the top of the wall that typically overlaps the fascia panels and provides a finished appearance. Coping may be cast-in-place or it can be precast.

Embed Loops – are loops extending from the back face of a concrete fascia panel. The loops are used with the connection pin to join the soil reinforcing mesh to the concrete fascia panel.

Face Of Wall – is the front face of the MSE wall, typically the precast concrete panels.

Filter Fabric – is a geo-textile fabric that is placed along the horizontal and vertical joints on the fill side of the precast panels. The filter fabric prevents the erosion of soil from the MSE mass.

Finish Grade – is the material in front of the MSE wall, which should be graded to drain away from the wall. Typically 24" (610 mm) of material is used to protect the bottom of the wall from erosion and undercutting. For taller walls, the height of fill in front of the wall may be 10 percent of the overall height of the wall.

Formliner Finish – is the texture cast into the fascia panels with a concrete formliner. The fascia panels may be cast with a texture such as fractured fin or ashlar stone to provide a pleasing finished appearance. In addition, multiple finishes can be used for the fascia panels to create a larger pattern across the wall.

Leveling Pad – is the non-structural pad which provides a level surface at a given elevation for the placement of fascia panels. Typically leveling pads are poured concrete 6" (152 mm) high and 12" (305 mm) wide.

Level-Up Concrete – is the concrete poured along the top edge of the fascia panels to provide a uniform surface for the installation of precast coping.

Moment Slab – is a reinforced concrete slab along the top of the MSE wall that overlaps the top edges of the fascia panels. The moment slab acts as coping to provide a finished appearance along the top edge of the wall.

Panel Layout Line – is the line at the structural thickness of the panels along the front face that can be used as an aid during panel placement.

Precast Segmental Panel – is the reinforced concrete panel that is connected to the soil reinforcing mesh which forms the front face of the MSE wall. The panels are used to keep the front face of the MSE wall from raveling between the layers of soil reinforcing mesh and to provide an aesthetically pleasing fascia for the wall.

Prepared Foundation – is the bearing surface of the excavated area underneath the MSE wall. Typically the area should be proof-rolled.
Reinforced Soil/Fill – is the MSE structure consisting of successive layers of engineered fill, or select backfill, and soil reinforcement. Reinforced fill is also called select fill or structural fill.

Retained Fill – is the fill material located behind the MSE wall between the reinforced soil mass and the natural soil.

Soil Reinforcing Mesh – is a manufactured welded wire grid element with varying length, width and wire sizes, which reinforces the select fill.

Traffic Barrier – is the structural element to keep vehicles within the roadway and away from hazards. MSE walls supporting lanes of traffic often include a traffic barrier at the top of the wall. Traffic barriers may also be used at the base of an MSE wall where a roadway exists in front of the wall.

Vertical Panel Joint Pad – is a plastic pad which provides a buffer (in some states) during a seismic event. These pads are placed in the vertical joints between concrete fascia panels. Typically these pads are glued to each panel and installed with an air gap between the pad and the adjacent panel.

Wall Control Line – is the horizontal alignment of the MSE wall that is given in the contract plans. The location of the wall control line may be at the front face of the panels, the front face of the coping or barrier slab, or a different location.
GENERAL GUIDELINES

The erection of the MSE structure is relatively simple. The installation consists mainly of preparing the subgrade, pouring the leveling pad, setting the concrete fascia panels, placing and compacting backfill in normal lift operations, laying the soil reinforcing mesh, and installing the features at the top of the wall such as coping or barrier slab. The layers of soil reinforcing mesh and compacted soil are placed in layers with the reinforcing mesh attached to the concrete fascia panels at each level.

NOTE: The figures contained in this installation manual are for illustrative purposes only. Refer to the shop drawings for design criteria, wall geometry, geotechnical reports, project notes, sections, general details, part information, and information or installation instructions specific to your project.

NOTE: The procedures outlined below are intended to be general guidelines for the installation of the MSEPlus™ Precast Panel Faced MSE Wall. Each wall should be erected as shown in the shop drawings in accordance with the Owner’s Contract Documents.

NOTE: It is the responsibility of the contractor or wall erector to establish and maintain proper on-site safety procedures relating to wall construction, including the protection of tradesmen, supervisors, inspectors, technical representatives, etc.

ACCEPTANCE OF MATERIAL

The contractor or erector should insure that proper equipment for unloading the material from trucks will be available when it arrives onsite. It is the responsibility of the contractor or erector to unload material without causing damage, delays, or incurring additional costs.

The material should be checked off the bill of lading as it arrives. The contractor should notify SSL within 24 hours in writing of any missing or damaged material. See the material handling procedures section of this manual and in the shop drawings. SSL recommends that the contractor or erector sort and store the concrete fascia panels and reinforcing mesh to best facilitate wall erection.

It is the responsibility of the contractor or erector to properly store the material so as to prevent damage. Proper sorting and storage of materials can also significantly aid the erection of the MSE structure(s). The contractor or erector should refer to the shop drawings for the location and placement of materials in the MSE structure and plan the storage and handling of materials accordingly.

STORAGE AND HANDLING OF PANELS

The panels should be stacked one-on-one, separated by non-staining dunnage. The width of dunnage between the panels should be greater than or equal to 2.5 inches (64 mm), or the height of the embed, whichever is greater. The number of panels per stack varies.

Dunnage should be aligned in the vertical direction. Care should be taken not to damage the edges or face of the panels during unloading, storage or setting. The panels may be unloaded supported by the provided pallets.
During panel erection, panels shall be lifted and set by the use of the two lifting anchors located in the top of each panel. When lifting panels from the stack, make sure that an additional piece of dunnage is below the bottom edge of the panel to prevent damage when rotating panels from horizontal to vertical. The lifting line should be vertical to avoid damage to the panel.
STORAGE AND HANDLING OF REINFORCING MESH

Soil reinforcement arrives to the site on a flatbed truck with dunnage separating the different bundles of soil reinforcing. Off-load the soil reinforcing carefully, using at least two balanced pick points spaced no more than 7 feet (2.134 m) apart. The soil reinforcing should be placed on dunnage, not placed directly on the ground, with additional dunnage between bundles as needed.

Ensure that the dunnage under the stacked bundles of soil reinforcement are aligned vertically and are not spaced more 7 feet (2.134 m) apart horizontally. Note that the placement of the dunnage in the figure are shown for clarification purposes only. Dunnage may need to be added or removed based on the length of the soil reinforcement being placed into storage.
SITE PREPARATION

Properly preparing the area underneath the MSE structures is important to the overall performance and longevity of the wall(s). The foundation material must be capable of supporting the entire MSE structure and any additional loads upon each wall. The area should be grubbed and graded where reinforced fill will be placed, with additional area behind the soil reinforcement as required in the shop drawings and/or contract plans. Any unsuitable foundation material which may lead to settlement or other problems should be removed. Refer to the shop drawings and contract plans for any additional foundation remediation. Proper site preparation can also aid wall erection at not only the bottom of the wall but at the upper portions as well. Typically the foundation material is proof rolled and approved by the owner's engineer or geotechnical engineer.

LEVELING PAD

The leveling pad provides a smooth, level surface on which to set the precast concrete fascia panels. Though the leveling pad is not a structural element of the MSE wall, it is important to allow proper alignment and spacing of the bottom row of panels. Leveling pads are typically unreinforced concrete at least 6" (152 mm) high and 12" (305 mm) wide. Each pad is cast to the elevation and length shown in the shop drawings. Properly constructed leveling pads aid wall erection at not only the bottom of the wall but at the upper portions as well.

PRECAST CONCRETE PANELS

The fascia panels are cast to the dimensions and specifications in the shop drawings. The standard panel (usually Type A) can be 5’ x 5’, 5’ x 6’, 5’ x 10’ or 5’ x 12’. An MSE wall usually includes several other types of panels to create the dimensions of the wall shown in the shop drawings. The naming of panels varies by project, but typically panel types used are A2 (half), L
(left), R (right), T (top), TL (top-left), TR (top-right) and C (corner). Additional panels such as A1, A3 and A5 may be used at the bottom of the wall.

Figure 5: Standard 5’ x 6’ Precast Panel

SOIL REINFORCING MESH

The welded wire soil reinforcing mesh is manufactured in sheets to the specification and length shown in the shop drawings. Each sheet consists of longitudinal wires and crosswires, with a 90 degree bend at one end. The number of longitudinal wires, longitudinal wire gage, cross wire spacing, cross wire gage, and overall length vary as needed by design. The longitudinal wires are typically spaced at 8” (203 mm) on center and the spacing of the transverse wires varies from 6” to 48” (152 – 1219 mm) depending on the structural requirements of the system.
Figure 6: Typical Mesh Detail

The soil reinforcing mesh has a special bend at one end which is used to connect the concrete fascia panel to the mesh. Typically each sheet has only one bend, but the other end may be bent if individual sheets are to be joined together. This is usually the case if the required overall length of mesh exceeds 40 feet (12.195m).

Figure 7: Typical Soil Reinforcing Bend Detail

Figure 8: Mesh Callout Detail
The shop drawings and design tables give the mesh type and length to be used in each layer. The length is typically shown on a separate line of the design information. The type is detailed with a designated callout. This designation indicates the size and spacing of the welded wires for each sheet of mesh used in that section of the wall. The designation is ordered as the mesh is to be placed in the structure, with the bottom layers at the bottom and the top layers at the top.

NOTE: the contractor or erector must insure that the mesh is properly installed in the configuration shown in the shop drawings. As a check, typically the thicker longitudinal wire sizes occur at the bottom of the wall and the thinner longitudinal wire sizes occur at the top.

NOTE: where traffic barrier is used above the wall, the top one or two layers of soil reinforcement may have additional length, longitudinal wires, or crosswires.
The wall elevations in the shop drawings show the segmental panels along the front face of the wall. Mesh breaks are also shown, which indicate for each column of mesh where the mesh type changes. In the example shown, the mesh breaks correspond with the mesh callout. Note that the difference in mesh type between the bottom two layers, the middle layers and the top layer is indicated by both the mesh callout and the mesh breaks. Review the shop drawings for your project and notify SSL if any discrepancies exist between the mesh callouts, mesh breaks, or other information shown.

**SETTING OF CONCRETE FASCIA PANELS**

Precast concrete fascia panels are placed directly on the leveling pad, using shims where necessary to level the top edge of the panel. The panels are typically placed along a control line. Care should be taken in the setting of the bottom row of panels to achieve proper alignment and spacing. Proper placement of the bottom course of panels will facilitate the installation of the subsequent courses and improve the overall quality of the installation.

![Figure 10: Bottom Row of Panels](image)

The placement of the segmental panels begins with locating the control line(s). SSL recommends that the wall erecter should use this information to establish a panel layout line, which is at the structural thickness of the panels at the front face. The offset distance between the panel layout line and the wall control line is shown in the typical cross section of the shop drawings. The panel layout line does not include the architectural finish, so no adjustment is necessary if the finish varies in thickness.
Figure 11: Panel Layout Line

The nominal width of the concrete fascia panels determines the correct spacing along the panel layout line. The nominal width is the actual width of the standard panels plus the width of the standard vertical panel gap. For example, if the standard panel width is 5'-11 1/4" (1810 mm) and the vertical panel gap is 3/4" (19 mm) then the nominal panel width is 6'-0" (1829 mm).

Figure 12: Nominal Panel Spacing

Where standard panels occur along the bottom of the wall, SSL recommends that the wall erector mark the leveling pad along the panel layout line at intervals equal to the nominal panel spacing. The spacing is adjusted at the ends of the wall, at corners, or wherever the width of the panels used is not the same as the standard width. Using the leveling pad marks will insure that long sections of wall will be set at the correct spacing and the wall will not "grow" or "shrink." The location of the marks can be the corner of each panel, the middle of the joint, or any point along the length of the panels. Each standard panel should be placed in the same position relative to the marks.

The wall control line(s) and panel layout line may include angles or curved sections. The shop drawings include details showing the placement of panels at the angle point. Typically the panel layout line will extend through the corner element.
Where the panel layout line is curved, the spacing should be continued through the curved section. The panels should be placed at intervals equal to the nominal panel width along the curve. The panel gap should be measured along the panel layout line. At the back face of the segmental panels the spacing varies depending on the radius of the curve and whether the fill side of the wall is toward the center of the curve or away from it.

The segmental precast panels should be set with a batter to the fill side of the wall. The amount of batter depends on the type of select fill, the amount of compaction required near the face of the wall, and various construction methods. The shop drawings show a suggested amount of batter based on typical materials and construction methods.
NOTE: the recommended batter shown in the shop drawings is what is typically required in MSE wall installation. This batter will vary depending on the properties of the select fill, moisture content, compaction effort and other variables. Batter shall be monitored and maintained with each lift of soil reinforcing mesh installed by the erection contractor.

FILTER FABRIC

Prior to the placement of backfill, filter fabric should be attached to the back face of the concrete panels as shown in the shop drawings. The filter fabric should cover each of the panel gaps. This allows moisture to "weep" out from behind the panels. For this reason, "weep holes" are usually not necessary in the precast panels. The filter fabric is attached to the panels with a 1/4" (6 mm) bead of adhesive around the edges. Adjacent pieces of filter fabric should overlap the length shown in the shop drawings and/or contract documents.
INITIAL PLACEMENT OF BACKFILL

Refer to shop drawings, contract plans, and/or project specifications for the type of backfill to be used in the reinforced zone of the MSE wall. This backfill can be referred to as the "select fill," "structural fill" or the "reinforced fill." The depth of backfill to be placed for each "lift" is usually determined by the project specifications. Placement of the backfill close to the back of the precast panels shall be done with care to insure that the wall face remains in proper alignment during the initial placement of backfill. Each lift of backfill material should be compacted according to the shop drawings, contract plans, and/or project specifications.

INSTALLATION OF SOIL REINFORCEMENT

The soil reinforcement should be installed perpendicular to the back face of the segmental panels. Each row of embed loops for each panel must have mesh attached. The soil reinforcing should be centered on the panel, or may be installed on alternating sides. For example, mesh with 2, 4, or 6 longitudinal wires should be centered in a standard embed loop configuration with 6 pairs of loops. If mesh with 3 or 5 longitudinal wires is connected to panels with 6 pairs of loops, it should be shifted to alternating sides as you go up. The bottom sheet would be placed on one
side, the sheet above that would be placed on the other side, the third row of mesh placed above the first, etc.

Figure 18: Mesh Connection by Wires

The mesh shown in the shop drawings is used for standard-width panels. Panels such as type "L," "R," "TL," "TR," and "C" may require mesh with fewer longitudinal wires than the standard panels. Typically, the panels will have fewer pairs of embed loops than the standard panels. The maximum number of longitudinal wires needed will be the same as the number of embed loop pairs. The extra longitudinal wires may be cut from the original sheet of mesh, or the mesh may be installed without the extra longitudinal wires attached to the fascia panels. Where the mesh is cut, the ends should be painted with a zinc-rich primer approved by the owner.

The connection is made by placing the mesh with the longitudinal wires between the pairs of embed loops on the panels. The "bearing bar" of the mesh should be resting on the top edge of the loops. Then the connection pin should be installed, typically one pin on each side of the row of loops. After the connection is made, the soil reinforcing mesh should be tensioned away from the fascia panels. One method is to install wedges between the soil reinforcing mesh and the back face of the panels. Another method is to use stakes to tension the mesh while backfill is spread above the soil reinforcement. Tension in the mesh can be maintained by placing the backfill near the panels and spreading it toward the ends of the soil reinforcement.
**Figure 19: Mesh Connection Detail 1**

**Figure 20: Mesh Connection Detail 2**

**BEARING PADS**

Bearing pads are used on all horizontal joints between panels. Typically the pads are high density polyethylene preformed to be 3/4" (19 mm) high, 2 5/8" (67 mm) wide and 6" (152 mm) long. These pads provide a bearing surface between panels and control the width of horizontal panel joints. A minimum of two pads are required per joint. Additional pads may be required for taller walls or as shown in the shop drawings. Each pad must be temporarily glued to the surface of the panel.
VERTICAL PANEL JOINT PAD

In some cases a pad is also provided for vertical panel joints. Typically these are molded plastic pads 3/8" (10 mm) high, 2 1/2" (64 mm) wide and 6 3/4" (171 mm) long. These provide protection during construction and under a seismic event. Each pad is attached with contact cement or adhesive. When vertical panel joint pads are required, they are usually attached to the panels by the precaster.

Note: Vertical panel joint pads are not used for all projects

Note: The vertical joint pad should not contact the adjacent panel
**Addition Panel Placement**

Bearing pads should be glued to the top surface of the bottom row of panels. In many cases, two bearing pads are used spaced equally on top of the panels. Additional bearing pads may be used for tall walls. The upper panels are placed on top of the lower panels and attached with clamps. Filter fabric should be installed along the back face at each panel joint.
Figure 23: Installation of Upper Panels

ADDITIONAL BACKFILL PLACEMENT

Additional lifts of backfill material are placed and compacted over the initial lifts. The fill should be placed as described above. Each layer of soil reinforcing mesh is installed as described above.
Figure 24: Additional Backfill Placement

The reinforced backfill should be compacted according to the shop drawings, contract plans, and/or project specifications, except within 3 feet (1 m) of the face of the wall. Due to the likelihood of causing distortion at the wall face, the fill within 3 feet (1 m) of the wall face should be placed manually. Great caution should be exercised during compaction efforts in this area. SSL recommends only minimal compaction effort be taken, and no compaction testing be done, within this 3 foot (1 m) zone.

The backfill material outside the 3 foot (1 m) zone from the face of the wall is typically compacted with equipment such as an 8-10 ton (88 kN) roller. SSL recommends smooth wheeled or rubber tire rollers but not grid type rollers. The compaction should be along the face of the wall working toward the tails of the mesh. Proper moisture content should be maintained during compaction. Refer to shop drawings, contract plans, and/or project specifications for the proper moisture content, compaction and compaction testing of the backfill material. SSL recommends that no compaction tests be performed within the 3 foot (1 m) zone of the face of the wall. The gradation of the reinforced fill should be checked periodically according to the shop drawings, contract plans, and/or project specifications. The contractor or wall erector should document all tests as they are performed. The vertical and horizontal alignment of the fascia panels should be checked periodically during compaction and wall erection.
OBSTRUCTIONS

The shop drawings for each MSE structure should include the locations of all obstructions to the soil reinforcing mesh. If additional obstructions are encountered, the contractor or wall erector should notify the Engineer of Record (EOR) and/or SSL. Refer to the shop drawings for general solutions or special details for each obstruction. These solutions may include cutting the crosswires of the soil reinforcing mesh to skew it away from the obstruction. The cut ends should be coated with a zinc-rich primer. Typically the skew angle does not exceed 15 degrees without additional calculations. The soil reinforcement should be bent gradually: abrupt bends could weaken the longitudinal wires and therefore should be avoided. SSL recommends that 6" (152 mm) of backfill material be maintained between soil reinforcement and obstructions. Where mesh is overlapping, SSL typically recommends 3" (76 mm) of backfill between the sheets. In some cases a structural frame may be necessary. For obstructions near the top of the wall, SSL recommends the use of a void former to block out space in the reinforced fill. This includes obstructions such as guardrail posts, which should not be driven into the reinforced fill. Driving posts into the reinforced fill after wall erection could damage the mesh and/or distort the wall face.

DRAINAGE

Proper drainage is essential to MSE structures. The contractor or wall erector must insure that the reinforced fill does not become over-saturated during or after wall erection. The reinforced fill should be sloped to provide drainage away from the face of the wall after each day of construction. Proper drainage should be maintained to prevent water intrusion from the front of the wall and the sides. Propagation of water in the backfill material may also alter the gradation by removing fines. Water intrusion or over-saturation will cause the MSE structure to become distorted or to destabilize and fail. Special care must be exercised during rain events to insure proper drainage and prevent erosion of the reinforced soil. Backfill material that becomes eroded or over-saturated should be replaced with new material conforming to the shop drawings, contract plans and/or contract specifications. SSL does not design drainage.

FINISH GRADE PLACEMENT

In most cases the bottom two feet or more of the MSE wall is buried below finish grade. The area in front of the wall is graded level with the bottom of the wall during construction, but should be filled in before wall erection reaches 20 feet (6.10 meters). Refer to the shop drawings, contract plans, and/or project specifications for additional requirements. Typically finish grade at the base of the wall should form a 4 foot (1.22 m) "bench" with a 2 percent minimum slope away from the wall. Proper drainage at the front of the wall should be maintained during and after wall erection to prevent the damage and/or failures described above. Special materials and/or measures may be required to prevent undercutting of the reinforced fill. After finish grade below the wall has been established, the contractor or wall erector should not proceed with any additional excavation in this area without permission in writing from the Engineer of Record (EOR) and/or SSL.
INSTALLATION SEQUENCE

STEP ONE

1. Excavate foundation to grades, lines and widths as shown on the project shop drawings.
2. Prepare foundation using proof-rolling or other means as specified in contract documents.
3. Pour leveling pad as shown on the project shop drawings (typical leveling pad shown).

![Figure 25: Installation Step One](image)

STEP TWO

1. Place appropriate concrete fascia panels and align to the proper position.
2. Use wood wedges where necessary to level the top edge of panels and to batter panels.
3. Brace panels with temporary bracing as determined by contractor.
4. Attach filter fabric to the back face of panels at panel joints.

![Figure 26: Installation Step Two](image)
STEP THREE

1. Place the first lifts of select backfill.
2. Compact each lift of backfill according to project specifications.
3. Test backfill for compaction, gradation, etc. as directed by project specifications.
4. Continue backfill installation up to the level of the lowest embed loops of the facia panels.

![Figure 27: Installation Step Three](image)

STEP FOUR

1. Select appropriate soil reinforcement for the bottom layer.
2. Modify mesh where necessary for non-standard width panels or obstructions.
3. Connect soil reinforcement to fascia panels with connection pin.
4. Tension mesh away from panels without distorting wall face.

![Figure 28: Installation Step Four](image)
STEP FIVE

1. Place additional lifts of select backfill to the elevation of the next layer of soil reinforcement.
2. Compact each lift of backfill according to project specifications.
3. Test backfill for compaction, gradation, etc. as directed by contract documents.

Figure 29: Installation Step Five

STEP SIX

1. Select appropriate soil reinforcement for the current layer.
2. Modify mesh where necessary for non-standard width panels or obstructions.
3. Connect soil reinforcement to fascia panels with connection pin.
4. Tension mesh away from panels without distorting wall face.

*Note: Check to insure that the proper batter is maintained at the face of the wall*

Figure 30: Installation Step Six
STEP SEVEN

1. Place additional lifts of select backfill to the elevation of the next layer of soil reinforcement. Leave void behind panels up to the second layer of mesh.
2. Compact each lift of backfill according to project specifications.
3. Test backfill for compaction, gradation, etc. as directed by contract documents.

*Note: Check to insure that the proper batter is maintained at the face of the wall*

![Figure 31: Installation Step Seven](image)

STEP EIGHT

1. Glue bearing pads to top surface of bottom row of fascia panels. Refer to shop drawings for the number and placement of bearing pads.
2. Place appropriate panels above the first row.
3. Clamp panels to adjacent panels.
4. Remove temporary bracing only when backfill reaches top edge of bottom panels.
Figure 32: Installation Step Eight

Figure 33: Installation Step Eight Back View
STEP NINE

1. Continue installation of backfill and soil reinforcement per steps 5-7.

Note: Insure that the proper batter has been achieved at the face of the wall

Note: Compaction within the 3’ (1 m) zone of the wall face shall be compacted with at least 3 passes of a lightweight mechanical tamper, roller or vibratory system. Insure that proper compaction is achieved without distortions at the front face of the wall.
STEP TEN

1. Continue panel placement per step 8.
2. Monitor batter along wall as pressure of backfill pushes panels out.
3. At top of wall, install coping or barrier slab as directed by contract documents.

Figure 35: Installation Step Ten

Note: Insure that the proper batter has been achieved at the face of the wall

Note: Compaction tests should not be performed within the 3’ (1 m) zone of the wall face.
WALL ERECTION CHECK LIST

1. Yes □ No □ Do you have an approved copy of shop drawings?

2. Yes □ No □ Do you have backfill test certifications?

3. Yes □ No □ Do you have material test certifications?

4. Yes □ No □ Does the wire mill have material manufactured and inspection certifications?

5. Yes □ No □ Does the galvanizer have galvanization certifications?

6. Yes □ No □ Is all material on site?

7. Yes □ No □ Is material stored properly to prevent onsite damage?

8. Yes □ No □ Has damaged material been recorded and a copy of rejected material given to suppliers?

9. Yes □ No □ Is the foundation excavated and proof rolled per the specifications and geotechnical report to the required width and elevation?

10. Yes □ No □ Has unsuitable material been compacted or removed and replaced?

11. Yes □ No □ Is the bottom row of panels properly placed, aligned and battered?

12. Yes □ No □ Is the first row of soil reinforcing mesh attached properly?

13. Yes □ No □ Are the required number of soil reinforcing mesh and the correct type being used?

14. Yes □ No □ Is the filter fabric being attached properly?

15. Yes □ No □ Are the bearing pads being placed properly between the panels? Are the correct number of pads used for each case?

16. Yes □ No □ Is the equipment being kept off of the grid until at least 6” of material is placed?
17. Yes □ No □ Is proper compaction being met in accordance with project specifications?
   Compaction to be minimum 95% of maximum density. Compaction to be 100% under bridge abutment spread footings.

18. Yes □ No □ Are obstructions to the soil reinforcement or other special situations met according to the details shown in the shop drawings?

19. Yes □ No □ Is the vertical and horizontal alignment of the structure being checked periodically?

20. Yes □ No □ At the end of each days operation is the reinforced volume being protected from runoff and saturation?
SAMPLE INSTALLATION PHOTOS

Sample Installation Photo 1: Handling of Precast Panels
Sample Installation Photo 2: Connecting Soil Reinforcement

Sample Installation Photo 3: Completed Soil Reinforcement Connection
Sample Installation Photo 4: Placement of Reinforced Fill
Sample Installation Photo 5: Compaction of Reinforced Backfill
Sample Installation Photo 6: Nearly Complete Wall Installation
PROJECT NOTES
Section 4: QUALITY CONTROL
4.1 Manufacturing
4.1.1 Facing Unit QA/QC
Prepared by: David C. Sweet
# QUALITY CONTROL MANUAL

## TABLE OF CONTENTS

1. Precast Solutions, Inc. QC Policy Statement & General Information & Key Contacts & Suppliers
2. Precast Solutions, Inc. Organization Chart
3. Description of QC Personnel Responsibilities
4. Training requirements for QC Personnel, production staff
5. Housekeeping plan
6. Product pre, post and final inspection procedures
7. Plant curing procedures for all seasons
8. Minimum strength requirements for stripping and shipping products
9. Product repair policy and procedures
10. Product Tolerances
11. Form tolerances and maintenance policy
12. Mix design qualification and testing procedures
13. Raw material testing policy and procedures
14. Equipment calibration policy and procedures
15. Product performance test policy and procedures
16. Examples of all documentation and forms used by plant to record QC and production processes
17. Documentation of products manufactured under franchise agreements, including all design specifications and drawings
18. Form Preparation
19. Concrete Placement/Finishing/Curing
20. Stripping/Stoning/Repairing/Storing
21. Handling, Storage, Shipping
22. Acceptance Package Transmittals
23. Shipments/Shipping Documents
1. PRECAST SOLUTIONS, INC. CONCRETE QUALITY POLICY STATEMENT:

To assure that the materials, fabrication and workmanship is satisfactory to our customers, the Owners, Management and Employees of PRECAST SOLUTIONS, INC. Precast Solutions, Inc. shall comply with the requirements specified in this manual.
Manufacturing Location: Precast Solutions, Inc.
6145 S. Indianapolis Road
Whitestown, IN 46075
Phone: 317-545-6557
Fax: 317-545-6558

Main Contact Person: Dave Sweet
Email Address: dsweet@precastsolutions-inc.com

ACI Certified Technicians:
Alberto Teran
Concrete Strength Testing Technician- Pending Test Results
PQS-Level I- Signed up for class, pending start
ACI- Field Testing Technician- Level I- Expires December 03, 2026

Mike Sweet
Concrete Strength Testing Technician- Pending Test Results
PQS-Level I- Signed up for class, pending start
ACI- Field Testing Technician- Level I- Taking Class February 2022

Carmelo Franco
PQS-Level I- Signed up for class, pending start
ACI- Field Testing Technician- Level I- Taking Class February 2022

Dave Sweet
ID# Expires May 27, 2024
Concrete Strength Testing Technician

Jose Sanchez
Concrete Field Testing Technician- Grade 1, Taking Class February 2022

Ben Winkles
Concrete Field Testing Technician- Grade 1, Taking Class February 2022

PQS School Graduate: David Sweet, John Davis, Britt Burtner, Bob Williams,

PCI Level I, 2 & III Graduate: Dave Sweet

Products Manufactured: Telephone Equipment Mounting Pads, MSE Wall, Box Culvert
Flush to Grade Utility Vaults, Grease Traps, Fire Protection Vaults, Sound Walls,
Architectural Precast Products, Lintels, Sills, Copings, Wall Caps,
Stair Treads, Splash Blocks, A/C Pads, Custom Precast

Manufacturing Start-Up date: October 1998

Concrete Batch Plant: Mixer Systems
Ready-Mix Concrete Suppliers: Bluestar Ready-Mix-Whitestown, IN
Concrete Industries, Whitestown, IN
Prairie Materials, Lebanon, IN

Testing Laboratory: ATC Associates
7988 Centerpoint Dr., Ste 100
Indianapolis, IN 46256
Gordon Pickett, Manager
PRECAST SOLUTIONS, INC.
In-house testing ability

2. Organizational Responsibilities

**John Davis**
President/CEO
Corporate Secretary & Treasurer

**Dave Sweet**
Vice President of Operations
Engineering, Technical Services,
Specifications, Documentation and Certifications
Plant Safety & Quality

**OPEN**
Production/Shipping Manager

**Alberto Teran**
Quality Control Manager

**Jose (Lalo) Sanchez**
Production Supervisor

Reinforcing Steel Suppliers: Harris Supply- a NUCOR Company
1700 7th. Avenue Suite 2100
Seattle, WA 98101
Liberty Steel and Wire
Peoria, IL

Aggregates: Prairie Materials- Indianapolis, IN
Martin Marietta- Carmel, IN

Chemical Admixtures: BASF Products

Certifications: Material Test reports and/or certificates of compliance
3. DESCRIPTION OF RESPONSIBILITIES FOR QC PERSONNEL

3.0 QC Personnel consist of a Quality Assurance Manager (responsible for scope of QC program), Quality Control Technician, Production Manager (manages QC plan in precast), Production Supervisor (executes QC program in precast), Product Manager (specifies products), Concrete Testing Technicians (QC testers).

4. DESCRIPTION OF TRAINING REQUIREMENTS FOR QC PERSONNEL, PRODUCTION STAFF, FORKLIFT OPERATORS AND DRIVERS

4.0 The QC Manager will attend and pass the National Precast Concrete Association Production and Quality Precast School as well as the ACI Concrete Field Testing Technician, Grade 1 School and Concrete Strength Testing Technician Course.

4.1 The Testing Technician(s) will maintain the ACI Concrete Field Testing Technician, Grade 1 and The PQS Level 1 School.

4.2 The Plant Production Supervisor will review Forklift Safety Practices with the personnel who are driving forklifts in our plant. Safety meetings will be held monthly to address other equipment and procedural safety issues.

5. HOUSEKEEPING PLAN

5.0 Precast Solutions, Inc. will keep the plant clean by sweeping daily, to keep the floors clear from aggregate and concrete mix. Tools and molds will all be assigned storage spaces and are to be kept in those spaces when not in use. Walking areas will be kept neat and free from hazards.

The personnel will keep parts and tools in their assigned storage spaces to eliminate tripping hazards and ensure tools are readily found when needed.

Forklifts will be parked with forks on the ground, with gas canisters turned off. At the end of the day, forklift keys will be removed and put in the lockbox.

6. PRODUCTION PRE, POST AND FINAL INSPECTION PROCEDURES

6A. PRE-POUR CHECKLIST

6A.0 The Production Manager will confirm the production schedules. The schedule will be updated daily on the project production board displayed in the production area.

6A.1 The Production Supervisor will use the daily schedule (Exhibit #3) to verify that pre-pour inspection checkpoints are assured and note and correct any deficiencies. Each checkpoint will be verified with a checkmark on each day’s production. Quality Control will review the checklist and discuss any deficiencies found with setup personnel to insure problems are minimized. Proper rebar placement will be checked on one piece daily.

6A.2 Checklists will be returned to the Q. C. Manager weekly. This document will become part of the production file that is maintained in the Q. C. Manager’s office.

6B. POST-POUR CHECKLIST

6B.0 Post-pour checklist is included on the same document as the pre-pour checklist. Issue and completion is the same as the pre-pour checklist.

6B.1 If deficiencies are found, they will be noted on the “Post-pour Checklist (Exhibit 3). The Q.C. Tech/Product Manager will confirm all repairs are completed satisfactorily.
The product is removed from the form and is inspected. If this is not part of a group/lot (same part, same formula and to be shipped on the same pallet) it will be identified with date and/or name. If the item is part of a group to be added to an existing order of like products, the pallet of like parts will be identified with the project name or a color code for the entire pallet. Some short run parts may be added to Precast Solutions, Inc. inventory as a normally stocked item; these will not be marked. After the visual quality inspection and tolerance check (if not previously verified), the product will be transported to a temporary storage location.

6C. **FINAL INSPECTION**

6C.0 As products are wrapped and strapped for shipping, a final inspection will take place to be sure the products are free of defects and are stacked for safe shipping. If defects are found, that item will be removed from the pallet and set aside for repair or replacement. The QC Technician and/or Plant Manager will perform the final inspection.

7. **PLANT CURING PROCEDURES FOR ALL SEASONS**

7.0 During warm weather months, concrete shall be covered overnight, then placed outside after stripping from its mold.

7.1 During cold weather months, concrete shall be covered (to hold heat and moisture, with winter blankets if needed) and left overnight indoors, when feasible, then placed outside after stripping from molds and achieving a minimum strength. If outside overnight temperatures are to be below 20 degrees F or over 80 degrees F, additional time will be added with the product covered and kept inside before moving product outside to insure adequate initial strength.

7.2 If outside temperature appears to be too high or low for standard curing procedures, as noted in 7.1, we will verify overnight in plant curing temperature to determine if products should be removed from molds. This temperature is recorded by Quality Control personnel each morning as the plant is opened.

8. **MINIMUM STRENGTH REQUIREMENTS FOR STRIPPING AND SHIPPING PRODUCT**

8.0 Unless instructed otherwise by specific customer requirements, a minimum of 1500 psi is required before stripping and 2500 psi is required before shipping products.

9. **PRODUCT REPAIR POLICY AND PROCEDURES**

9.0 If the needed repair is purely cosmetic, defect will be corrected during the stripping process. If it is structural, the production supervisor and/or product manager will determine the repair approach and see that it is performed. A customer may request specific repair materials to be used on their manufactured products.

10. **PRODUCT TOLERANCES**

10.0 All products shall be manufactured within tolerances specified on shop drawings and/or blueprints supplied by the customer.

11. **FORM TOLERANCES AND MAINTENANCE POLICY**

11.0 During the stripping process, forms will be visually checked for defects and if found, noted on the Post-pour portion of the Daily Production Status Report form (Exhibit 3) and/or taken directly to the shop for the required repair or possible replacement.

11.1 Forms shall be cleaned thoroughly after each use to ensure that release agents will not build up and that joints between form parts will remain sealed. Forms that will not be soon used again will be stored (upside down if possible) in a designated area.
12.0 Concrete mix is supplied by the batch plant equipment or by a local approved ready-mix plant. Cement shall be type I or III and shall conform to the requirements of AASHTO M-85. Concrete mix for INDOT shall have a compressive strength at twenty-eight (28) days of 4000 psi. The concrete for ODOT shall have a compressive strength of 4500 psi at twenty-eight (28) days.

12.1 Compressive strength testing shall be performed as required for production lots. Documented mix designs from Precast Solutions, Inc. or supplied by local ready-mix plant technicians are maintained by the Production manager and are available upon request. The appropriate mix design shall be submitted to the State DOT or authorized representatives for approval, when required, prior to the start of any project.

Precast Solutions, Inc. Batch Plant utilizes formulas appropriate for specific customers and products, with compressive strengths from 4000 psi to 9000 psi.

12.2 During the batch plant mixing cycle, a plastic properties test shall be conducted to determine the slump, air content, and temperature and will be executed near the point of placement, to assure the concrete meets applicable specifications. Data recorded by Q.C. technician will include the following:
   a) Slump: 3” - 8” or job specific.
   b) Concrete temperature: 50 min. – 90 max.
   c) Air Content: 3-8% or job specific

12.3 All of the above data is captured once each day for each formula used in the batch plant. It is recorded on the Precast Solutions, Inc. Concrete Testing Report form (Exhibit 2) and turned in to the Quality Control Manager upon form completion.

12.4 Entrained air testing shall be performed with a type “B” pressure meter according to AASHTO T-152/ASTM C-231. Equipment shall be calibrated quarterly and records maintained in the calibration file.

Testing personnel shall maintain an ACI Concrete Field Testing Technician – Grade 1 Certification.

Air Specifications by state are as follows:

1) INDOT - 6 ½% ± 1 ½%  
2) OHDOT - 6 % ± 2 %  
3) PENDOT – 6% ± 2%  
4) WVDOT – 7% ± 2 ½%  
5) SCDOT - 3% - 5%  
6) KYDOT – 6% -± 2%  
7) NCDOT – 4 ½% ± 1 ½%  
8) VDOT - 6 ½% ± 1 ½%  
9) MDDOT - 5% - 8%  
10) IDOT - 5-8%

Note: Specifications are after aggregate correction factor

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**Formula - 4000**

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12.4 CONCRETE SAMPLING
12.4a All samplings will be obtained at the discharge end of the chute or concrete hopper.

12.4b ACI and ASTM Specifications require fresh concrete and compression testing every 150 yd³ for each mix design. PSI’s requirement is every 50 yd³ for each mix design.

12.4c Cylinders will be 6” X 12” if required for DOT jobs if necessary, and 4” X 8” for other precast jobs, prepared in accordance with AASHTO T-141 or ASTM C31/C31M-03a.

12.4d Acceptance of the concrete products, with respect to compressive strength, shall be determined on the basis of production mix design.

12.4e PSI will obtain compressive strength sampling and testing for each production mix design. A strength report will be supplied by PSI or ATC.

12.4f A set of compressive strength samples shall consist of a minimum of 4 (four) cylinders. This allows samples to be tested at several intervals. Box culverts require 6 cylinders minimum.

12.4g Compressive strength cylinders shall be cured in the same manner as the products.

12.5 COMPRESSIVE STRENGTH CRITERIA
12.5a Compressive strength tests shall be performed on test cylinders in a size driven by customer requirements. Currently six (4) inch diameter by twelve (8) inch tall cylinders are approved by INDOT when prepared in accordance with AASHTO T-23. Other jobs may only require (4) four by
10 cylinders, with the same testing procedure. Some jobs may also have different strength requirements and will be tested as requested by client or customer.

12.5b The initial strength of the concrete at seven (7) days will be determined by compressive strength testing in accordance with AASHTO T-22. The samples must meet or exceed the mix design strength on or before twenty-eight (28) days. Minimum strength shall be 4000 p.s.i. (27.6Mpa) for DOT projects (IDOT requires 4500 psi). Each quarter, PSI will complete 1-day strength testing, to confirm de-molding strength in that season is appropriate.

Compressive strength shall be established by a two (2) cylinder average at each testing. PSI will prepare cylinders for compressive strengths utilizing concrete test specimens based on ASTM C 172 and ASTM C31. These test samples will then be taken to: ATC Associates Inc. testing lab (Indianapolis, IN) for compressive strength testing, or tested in-house with CGM compressive strength testing equipment.

12.5c For DOT strength testing requirements, if the initial strength test result indicates a compressive strength greater than or equal to 4000 psi (or 4500 psi for IDOT), then this test result will be utilized as the compressive strength test result for that production mix design and the requirements for additional testing will be waived. A twenty-eight day break will still be made unless previous tests have met our customer’s strength requirements.

12.5d Acceptance of a production mix design will be made if compressive strength test result is greater than or equal to the requirements of our customer. The average of both tested cylinders must exceed the minimum required design strength.

13. RAW MATERIAL TESTING POLICY AND PROCEDURES

13.0 Raw materials, such as gravel and sand, will be checked visually and mill certificates will be collected and filed in the materials records book.

13.1 All materials used for fabrication shall meet the requirements specified by the contract documents and approved drawings submitted for fabrication by customers/clients of Precast Solutions, Inc.. Materials not conforming to contract documents shall not be used without the written consent of the designated State or project authority.

13.2 Reinforcing steel shall meet the requirements of ASTM A-615 or A-706 Grade 60 as required by customer. All reinforcing bar is purchased from one or more of the approved Certified Reinforcing Bar Producers from the IDOT list dated 10-5-07. Steel shall be purchased from suppliers listed in Section 1 of this manual. Rebar size and details shall be determined by approved drawings or requests of customers. Standard rebar jigs may be used to insure the consistent conformity of rebar mats. Material certification shall be maintained in the project file in the Quality Control Manager’s office and available upon request. Epoxy or fiberglass rebar is used as requested per project specification.

13.3 Moisture testing for fine aggregates: We follow the standard test method outlined by ASTM Designation: C566 – 97. This approach allows for a sample from the discharge area of the bulk fine aggregate storage bin of a known quantity to be weighed, heated (we use a hot plate) and weighed again to determine the percentage difference from the moisture removal. We determine the mass of the dried sample to the nearest 1% after it has cooled sufficiently. This new moisture percentage is then factored into the plant batching for that particular day unless a different aggregate source is introduced. If the aggregate source changes, we will retest for current moisture content.
14.0 Certificates of Calibration will be kept in the Quality Control Manager’s office.

Testing equipment to be calibrated:

\textbf{Annually shall include-}

\begin{itemize}
  \item a) Compressive Strength Test Machine
  \item b) Unit weight Scales (\textit{DigiWeigh} model DW-65, digital scales)
  \item c) Reference Thermometer (\textit{Instant Read} model #46430, Digital Thermometer)
  \item d) Admixture Dosing Equipment (BASF)
\end{itemize}

\textbf{Quarterly shall include-}

\begin{itemize}
  \item a) Air Meter (Forney LA-0316) shall be calibrated every three (3) months per ASTM C231 Section A1.9, type “B” Air Meter and documented on Air Meter Calibration Record (Exhibit #12)
\end{itemize}

\textbf{Checked visually daily shall include-}

\begin{itemize}
  \item a) Slump Cone (ELE model EI34-0108) – visual check for dents & out of round
  \item b) Single Use Plastic cylinder molds – visual check each time used
\end{itemize}

*Please see attached Precast Solutions, Inc QC Performance Timeline for additional requirements.

15. \textbf{PRODUCT PERFORMANCE TEST POLICY AND PROCEDURES}

15.0 Precast concrete products shall meet all requirements for function, dimension(s), design, color (if specified) and strength to meet customer’s order request.

15.1 The mold shall be checked in detail after its construction and before the first unit is made. A complete check of the first product from the mold is further verification of the adequacy of the mold. Checking correlating product inspection of appearances and dimensional tolerances to the form is only feasible where an element can positively be identified with the specific form in which it was cast.

16. \textbf{EXAMPLES OF ALL DOCUMENTATION AND FORMS USED BY PLANT TO RECORD QC AND PRODUCTION PROCESSES}

\begin{tabular}{|c|l|}
  \hline
  \textbf{EXHIBIT NO.} & \textbf{TABLE OF CONTENTS - DESCRIPTION} \\
  \hline
  1 & Batch Manufacturing Schedule – Daily plan and mix sizes \\
  2 & Precast Solutions, Inc. Concrete Testing Report – testing data \\
  3 & Precast Solutions, Inc. Daily Production Status Report – pre-pour, post-pour inspection \\
  4 & Precast Solutions, Inc. Unit Weight & Yield of Concrete Report – test data \\
  5 & Precast Solutions, Inc. Aggregate Moisture Record \\
  6 & Bill of Lading – Short Form \\
  \hline
\end{tabular}
17. **DOCUMENTATION OF PRODUCTS MANUFACTURED UNDER FRANCHISE AGREEMENTS, INCLUDING ALL DESIGN SPECIFICATIONS AND DRAWINGS.**

17.0 At present, we do not have any franchise agreements. When applicable, they will be filed with design specifications and drawings for easy reference, with an additional set of drawings available to plant supervision personnel.

17.1 Control of product drawings, when provided, shall be maintained by the Product Manager. Master copies are maintained in the project file. Drawings are dated and marked upon receipt. Drawing transmittals are maintained in the project file. New release or revised drawings are hand-carried to affected personnel and outdated drawings are removed and destroyed. Product Manager will VOID the old master set, marking as such, and maintain VOID drawings with project records. All project records will be stored and maintained for a period of three (3) years unless requested to do otherwise.

18. **FORMS PREPARATION**

18.0 Forms shall be thoroughly cleaned and free from dirt, debris and concrete scale. Once accomplished, the inner portion (all interior surfaces) that will be in contact with the concrete mix, shall be applied with an evenly distributed, uniform coat of release agent, while minimizing any puddling on the form face. This will assist with the stripping process and facilitate the production of a defect-free concrete surface, while improving the working condition of the form. All block-outs are made of a rigid, non-absorptive material that will not harm concrete and will be held firmly in place during the casting and curing process. Block-outs will be made to customers’ specific tolerances, if required.

18.1 After cleaning, each form will be set up to meet the specific product demand. This includes the placement and securing of headers to create special parts, cable block-outs, placement of the rebar, positioning of lifting inserts and location devices to locate attachment hardware. Care shall be taken, by the use of rebar chairs, to insure there is no contact with the release agent to the reinforcing steel. The supervisor will complete the Pre-pour checklist (Exhibit 3) for all molds included in each day’s casting. Products with specified rebar placement will be checked for proper installation of rebar sticks or cage. Cages will utilize rebar ties or welding, whichever is appropriate for the particular job, or detailed in the product plans and drawings. If epoxy-coated rebar is used, care will be taken that the bars are repaired with patching material,
19. CONCRETE PLACEMENT/FINISHING/CURING/MARKING

19.0 The concrete mix will be batched at the Precast Solutions, Inc. batch plant or at a local ready-mix plant (if truck delivered). The ready time is approximately 10 minutes for Precast Solutions, Inc. plant or approximately 30 minutes for ready-mix.

19.1 The concrete mix will be deposited into the forms directly from the bucket or chute at close proximity. The concrete shall be placed via use of shovel, trowel or scoop to insure proper concrete volume, as required to fill remaining voids within each respective mold.

19.2 As a form is being filled, or immediately afterwards, the concrete will be consolidated via the use of vibration. Vibration shall be continued until proper consolidation has occurred. The vibration will be minimized to prevent possible segregation of aggregates.

19.3 The exposed back surface of the precast item will be vibrated to level or rough floated to remove any remaining high or low surfaces, if required.

19.4 If required, upon concluding the vibration/ troweling operation, each item shall be marked on the exposed concrete surface with the cast date. On certain parts a surface description may be required; if so, we will place marking on an area out of view during final placement of concrete part. This part marking will insure the proper tracking of the product.

19.5 After any required identification markings, a curing method will be used to meet any applicable project specifications. Covering products with blankets, or use of curing compound(s) are available.

19.6 The cast products will remain in the form for a minimum of fifteen (15) hours. One (1) day compressive strength cylinders shall be made and tested once every quarter to document stripping strength. Minimum acceptable stripping strength is 1500 p.s.i.

19.7 Proper rebar placement is required. For products with specified reinforcement, plans will be followed. Where no customer specifications are required, but reinforcement will be used as a precaution against breakage, we will use Grade # 60, #4 rebar straights placed at least ½” below concrete finishing surface. Rebar length will be at least 2” shorter than the length or width of the mold. Care will be taken to keep end of rebar from the side of the mold. Typically 2 rebar straights will be placed into sills and or wall caps, depending on their size.

19.8 Embedded items will be placed per job drawings, unless they are placed for ease of de-molding, in which case they will be centered in the most useful way. Either templates or measuring will determine proper placement.

20. STRIPPING/STONING/REPAIRING/STORING

20.0 The following morning, after the previous day’s pour, the forms will be stripped. After the product is removed from the form, it will be visually inspected for any voids or defects. If any deficiencies are identified, the piece will be tagged or set aside and a repair will be made during that production day. Supervisors will complete a “Post-pour Checklist” (Daily Production Status Report, Exhibit 3 of this manual). Any repair failing to satisfy our quality inspection will be marked as a reject and taken to the reject storage area. A replacement will be cast timely, as not to interfere with shipping schedule commitments. Rejected pieces will be indicated as such on the project documentation.
After visual quality inspection and tolerance checks, the product will be stoned with rubbing bricks and any small holes filled. These parts will then be loaded on a pallet and moved to a temporary storage area or if the pallet is complete, it will then be first wrapped and strapped and then moved to the storage area.

21. HANDLING, STORAGE, SHIPPING
21.0 All products shall be handled, stored and shipped in such a manner as to minimize the danger of chipping, cracks, fractures, and excessive bending stresses. All products shall be stored on pallets unless otherwise directed by customer. Any product that is to be stacked will utilize wood or foam spacers. This “spaced” stacking may not apply to small items. A designated staging area at Precast Solutions, Inc. extends along the south edge of our fenced property. This paved area is used to accumulate all parts for specific shipments. Additional long term storage is provided on the graveled, level areas of the yard.

21.1 Three forklifts (8,000, 8,000 and 10,000 lbs.) are on-site to move all products safely inside and outside of the plant. There are also a number of overhead cranes/hoists to assist with the safe lifting of pre-cast products. Lifting equipment is visually checked each time before use. Maintenance records are filed in the office.

22. ACCEPTANCE PACKAGE TRANSMITTAL
22.0 We will provide the following project information as a packet upon request:
   a) Mix Design
   b) Reinforcing Steel Certifications
   c) Attachment Hardware Certifications
   d) Cement Certifications
   e) Aggregate Report – Coarse (pea gravel)
   f) Aggregate Report – Fine (#23 sand)
   g) Compressive Strength Report
   h) Daily Production Report

23. SHIPMENTS/SHIPPING DOCUMENTS
23.0 Precast products will be shipped by a commercial freight line as requested by client or customer. If no method of shipping is provided, PSI will seek the most competitive professional means of shipping each product, given size and time requirements. We employ the use of proven commercial LTL and flatbed carriers. Shipping records and “bills of lading” records, which are attached to customer invoices, will be kept in files at PSI for a minimum of one year.
Batch Manufacturing Schedule 2021
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<th>YDS</th>
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<td>Organize foam</td>
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<th><strong>Actual</strong></th>
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<th><strong>Batch Breakout</strong></th>
<th><strong>Staffing</strong></th>
<th><strong>Total Team Members</strong></th>
<th><strong>Total Man Hours</strong></th>
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16
Date: __________

7 days

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<table>
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<th>C temp -</th>
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<tr>
<td>Slump -</td>
<td>Air -</td>
<td>%</td>
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<th>C temp -</th>
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<tr>
<td>Slump -</td>
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<tr>
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## Precast Solutions, Inc. - Unit Weight and Yield of Concrete Report

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<th>Date:</th>
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<tbody>
<tr>
<td>Test no.</td>
<td>Project:</td>
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</table>

### Batch Weight in Pounds

| (A) Cement | | |
| Water | | |
| Fine Aggregate | | |
| Coarse Aggregate | | |
| (B) Total Weight of Batch | | |

### Weight of Concrete in Pounds per Cubic Foot

| Weight of Container Full of Concrete | | |
| Weight of Container | | |
| (C) Net Weight of Concrete | | |
| (D) Container Factor | | |
| (E) Weight per Cubic Foot (C X D) | | |

### Batch Volume Produced and Cement Factor

| Specified Minimum Cement Factor (cwt/cy) | | |
| (F) Batch Volume Produced (B/E) | | |
| (G) Yield (F/27) | | |
| Actual Cement Factor (.01A/G) cwt/cy | | |

### Comments:

Precast Solutions, Inc. Unit Weight & Yield Report Exhibit 4
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<th>Date</th>
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<th>Sample Size Bin #</th>
<th>Pre-Test Weight</th>
<th>Post-Test Weight</th>
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**SHIPPING TICKET**

**SHIP FROM**

Precast Solutions, Inc.  
6145 S. Indianapolis Road  
Whitetown, In 46075

**Bill of Lading Number:**

**SHIP TO**

[Name]  
[Street Address]  
[City, ST ZIP Code]  
CID No.:  

**Carrier Name:**

Trailer number:  
Serial number(s):  

**THIRD PARTY FREIGHT CHARGES BILL TO**

[Name]  
[Street Address]  
[City, ST ZIP Code]  

**Freight Charge Terms (freight charges are prepaid unless marked otherwise):**

- Prepaid ☐
- Collect ☐
- 3rd Party ☐
- Master bill of lading with attached underlying bills of lading.

**CUSTOMER ORDER INFORMATION**

**Customer Order No.**  

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<th># of Packages</th>
<th>Weight</th>
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<th>Additional Shipper Information</th>
<th>Y</th>
<th>N</th>
<th>Y</th>
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**Grand Total**

**CARRIER INFORMATION**

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COD Amount: $  
Fee terms: Collect ☐  
Prepaid ☐  
Customer check acceptable ☐

**Note:** Liability limitation for loss or damage in this shipment may be applicable. See 49 USC § 14706(c)(1)(A) and (B).

Received, subject to individually determined rates or contracts that have been agreed upon in writing between the carrier and shipper, if applicable, otherwise to the rates, classifications, and rules that have been established by the carrier and are available to the shipper, on request, and to all applicable state and federal regulations.

The carrier shall not make delivery of this shipment without payment of charges and all other lawful fees.

Shipper Signature

This is to certify that the above named materials are properly classified, packaged, marked, and labeled, and are in proper condition for transportation according to the applicable regulations of the DOT.

Trailer Loaded:  
- By shipper ☐  
- By driver ☐

Freight Counted:  
- By shipper ☐  
- By driver/pallets said to contain ☐  
- By driver/pieces ☐

Carrier Signature/Pickup Date

Carrier acknowledges receipt of packages and required placards. Carrier certifies emergency response information was made available and/or carrier has the DOT emergency response guidebook or equivalent documentation in the vehicle. Property described above is received in good order, except as noted.

Shipper Signature/Date

To verify the above statements and to certify that the materials are properly classified, packaged, marked, and labeled, and are in proper condition for transportation according to the applicable regulations of the DOT.
Post Pour Inspection Form-Grease Traps/Vaults

Product or Job: Vault Base Date: _____________________

| Date Poured: | No. of cylinders made: |
| Mix: | Air Content: | Slump: | Temperature of mix: |
| Required Strength: | Admixture: | Concrete | Concrete Supplier |
| Client: | Project | Job# |
| Qty of Cu Yds represented by cylinders: | Sampled By |

<p>| Comments: |</p>
<table>
<thead>
<tr>
<th>No</th>
<th>Crushing Load lbs</th>
<th>Crushing Str PSI</th>
<th>Age to be tested</th>
<th>Date Tested</th>
<th>Cylinder Measurement</th>
<th>Type of Break</th>
<th>Initials of tester</th>
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<td>1000-1500</td>
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CGM Compression Break Card Testing Info 9-06.xls

---

Precast Solutions, Inc. Testing, Inspection and Investigation

Compression Test

| Date Poured: | No. of cylinders made: |
| Mix: | Air Content: | Slump: | Temperature of mix: |
| Required Strength: | Admixture: | Concrete | Concrete Supplier |
| Client: | Project | Job# |
| Qty of Cu Yds represented by cylinders: | Sampled By |

<p>| Comments: |</p>
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<th>Initials of tester</th>
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<td>6&quot; X 12&quot;</td>
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### Table of Contents

<table>
<thead>
<tr>
<th><strong>Product Type</strong></th>
<th><strong>Stripping Strength (psi)</strong></th>
<th><strong>Casting Finish - CS 1800</strong></th>
<th><strong>Coating Finish - Black Roll On</strong></th>
<th><strong>Cast Length (ft/in)</strong></th>
<th><strong>Cast Width (ft/in)</strong></th>
<th><strong>Cast Depth (ft/in)</strong></th>
<th><strong>Wall Thickness</strong></th>
<th><strong>Cracks or Spalls</strong></th>
<th><strong>Honeycombing / Bugholes</strong></th>
<th><strong>Exposed reinforcement</strong></th>
<th><strong>Lifting Devices</strong></th>
<th>**Ht of 2 Opposite Sides- **</th>
<th>**Ht of 2 Opposite Sides- **</th>
<th><strong>Block out #1- 24” x 24” x 1” for sump in Floor</strong></th>
<th><strong>Block out #2- Sump Pit-floor</strong></th>
<th><strong>This will get a sloped floor</strong></th>
</tr>
</thead>
</table>

### Comments:

- ____________________________________________________________________
- ____________________________________________________________________

**QC Inspector:** ___________________________ **Date:** ___________________
Customer Complaint Form

Report Date: ____________________________________

Product or Job: ___________________________________

Number of Units: _________________________________

Casting / Unit Type: _______________________________

Nature of Complaint:_____________________________________________________________________________________
______________________________________________________________________________________________
______________________________________________________________________________________________
______________________________________________________________________________________________
______________________________________________________________________________________________

Corrective Action/Follow Up:_________________________________________________________________________
____________________________________________________________________________
____________________________________________________________________________
____________________________________________________________________________
____________________________________________________________________________

Person Performing Follow Up: _______________________ Date: __________________

Person Performing Corrective Action: _____________________ Date: __________________

QC Inspector: ________________________________________ Date: __________________

Tank Watertightness Testing Form

24
TEST SPECIFICATIONS

Test Requested By: _________________________________
Test Performed By: _________________________________
Test Witnessed By: _________________________________

Description of Test: __*see C-1613-10.9.1.2 Hydrostatic Testing below___

Product Tested To: __ASTM C-1613-10.9.1.2 Performance test methods___

Additional Comments: ____________________________________________

                          Start: _________” End: _________” Time: _____ Min.

(Allow the tank to stabilize before beginning the timed test. Refer to the latest edition of ASTM C 1613 for requirements.)

*"C-1613-10.9.1.2 Hydrostatic Testing-Seal the tank, fill with water to operational lever, let stand for 8-10 hours. If there is a measurable drop in water surface elevation, let stand for another 8-10 hours. There shall be no further measurable drop in the water surface elevation. Tank shall not be rejected for damp spots on exterior concrete surface. If water is dripping in a steady stream, the tank shall be repaired and retested”

1st Attempt:                              Final Attempt if needed:
Pass _________ Fail _________ Pass _________ Fail _________

Signature: _________________________________ Date:__________
Mark Schuhler, Q.C. Manager

Signature: _________________________________ Date:__________
Dave Sweet, VP of Operations
This renewal certificate is valid through December 31, 2020.

Renewal Granted on December 31, 2019

Stated in the (14th Edition) 2-19 of the NPCA Quality Control Manual for Precast Concrete Plants

Precast Concrete Requirements

We are audited during an on-site plant inspection on June 6, 2019, and have met the

46075-9526
6145 S. Indianapolis Road

Precast Solutions Inc.

This is to certify that the quality control procedures of
Latest NPCA Plant Audit is available and located in the Hard copy of the Plant QC Manual.
Latest NPCA Plant Audit response to deficiencies is available and located in the Hard copy of the Plant QC Manual.
## Ambient Temperature Log

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**Pool Coping**

**Box Culvert**
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### Sills

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Precast Solutions, Inc.
QC Performance Timeline

**DAILY QC TASKS**

1. Break strip cylinders
2. Log Outdoor and Indoor Ambient Temperatures
3. Post Pour Inspections- From previous day pour
4. Pre-Pour Inspections – Today’s pour
5. Final Inspection- Prior to loading-STOCK and NON-STOCK Precast Materials
6. Concrete Tests on each mix type
7. Break 7 and 28 day cylinders
8. Log Cylinder Breaks
9. Batch tickets into book- these need to be printed every load, every day.
10. Pre and Post Pour sheets filed properly
11. Fill out Daily Quality Report in Computer prior to 11 a.m.
12. Fill out Work Orders, Copy and give to Plant Manager

**WEEKLY QC TASKS**

1. Weekly Curing Ambient Temperature Charting
2. Check for new Mill Certs on Aggregates, Cement, Admixtures & Reinforcing, place in Materials Book

**MONTHLY QC TASKS**

1. Safety Toolbox Talks
2. Monthly QC meeting

**FIRST QUARTER QC TASKS**

1. Compression Machine Calibration (Ohio Requirement)
2. Bench Scale Calibration (Ohio Requirement)
3. Air Meter Calibration (Ohio Requirement)
4. Unit Weight Bucket (Ohio Requirement)
5. Thermometers, 25 ° - 125° F Calibration/Replacement (Ohio Requirement)
6. Slump Cone Calibration (Ohio Requirement)
7. Quarterly QC Meeting

**BI-ANNUAL & SECOND QUARTER QC TASKS**

1. Bi-Annual QC meetings
2. Aggregate Bin Scale Load Cell Calibration (Ohio Requirement)
3. Cement Scale Load Cell Calibration (Ohio Requirement)
4. Admixture Dispensers Calibration (Ohio Requirement)
5. Compression Machine Calibration (Ohio Requirement)
6. Bench Scale Calibration (Ohio Requirement)
7. Air Meter Calibration (Ohio Requirement)
8. Unit Weight Bucket (Ohio Requirement)
9. Thermometers, 25 ° - 125° F Calibration/Replacement (Ohio Requirement)
10. Slump Cone Calibration (Ohio Requirement)
Precast Solutions, Inc.
QC Performance Timeline

THIRD QUARTER QC TASKS
1. Compression Machine Calibration (Ohio Requirement)
2. Bench Scale Calibration (Ohio Requirement)
3. Air Meter Calibration (Ohio Requirement)
4. Unit Weight Bucket (Ohio Requirement)
5. Thermometers, 25 ° - 125° F Calibration/Replacement (Ohio Requirement)
6. Slump Cone Calibration (Ohio Requirement)
7. Quarterly QC Meeting

FOURTH QUARTER QC TASKS
1. Compression Machine Calibration (Ohio Requirement)
2. Bench Scale Calibration (Ohio Requirement)
3. Air Meter Calibration (Ohio Requirement)
4. Unit Weight Bucket (Ohio Requirement)
5. Thermometers, 25 ° - 125° F Calibration/Replacement (Ohio Requirement)
6. Slump Cone Calibration (Ohio Requirement)
7. Quarterly QC Meeting

ANNUAL QC TASKS
1. GT Watertightness Testing (1 of each size, 1000 gal, 2000 gal, etc.)
2. Bi-Annual QC Meetings
3. Check Boots, Profile Gaskets, mastic for Conformance to Standards on Manufacturers Websites
4. Annual Safety Refresher Course for OSHA topics
5. Check Dimensional Tolerances of Forms
6. Vibrator Training Course
4.1.2 Soil Reinforcing QA/QC
Quality Control Manual

SSL Retaining Walls
4740 Scotts Valley Drive, Suite E
Scotts Valley, CA

Prepared By:
Steve Ruel, P.E.
Fransiscus Hardianto, P.E.
Scott Thompson Jr.

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# Table of Contents

1. Introduction ................................................................. Page 1
2. Production Process and Responsible Parties ....................... Page 2
3. Supplier Responsibilities ................................................. Page 2
4. SSL Personnel Responsibilities ......................................... Page 3
5. Material Testing and Inspection (SSL) ................................ Page 4
6. Quality Control Program Flowchart .................................. Page 8
7. Material Testing and Inspection (Suppliers) ......................... Page 10
8. Conclusion .................................................................. Page 13
9. Sample Certificates of Compliance .................................... Page 14
10. Sample Inspection Report ................................................ Page 17
11. Attachments ................................................................ Page 19
   - Concrete Reinforcements, Inc: QA/QC Program ............ Page 20
   - AZZ Galvanizing Services: QA/QC Program ................ Page 30
   - Precast Quality Control Checklist ............................... Page 48
   - SSL Connection Test Procedure ................................. Page 50
1. INTRODUCTION

SSL is an engineering design firm involved in the design and supply of Mechanically Stabilized Earth retaining walls. SSL's products employ welded wire mesh as soil reinforcement combined with a facing element that will either be precast concrete panels or welded wire mesh panels. The production of the welded wire mesh and precast concrete elements is performed by supplier organizations that specialize in those materials. SSL has coordinated and implemented an extensive Quality Control Program that will provide our customers with a superior product that meets or exceeds all applicable standards and specifications including those set forth by the American Association of State Highway and Transportation Officials (AASHTO), the American Society for Testing Materials (ASTM) and the Buy America Act.

This manual will outline the Quality Control procedures in place at SSL along with the procedures in place at our suppliers. The Quality Control measures required by SSL of the Contractor are listed in the Standard Details of our Shop Drawings. The Quality Control Program in its entirety is intended to provide several check points at which any potential production or manufacturing problems will become evident. Should any problems arise, the program is intended to isolate and correct the production process responsible.

SSL's Quality Control Program was developed with input from industry experts and reflects the latest in design and testing methods. Any changes to the Quality Control Program will be detailed in a numbered revision to the Quality Control Manual which will be re-submitted to Caltrans for review.
2. PRODUCTION PROCESS AND RESPONSIBLE PARTIES

3. SUPPLIER RESPONSIBILITIES

Upon acceptance of SSL's purchase order, the material supplier agrees to perform all Quality Control procedures put forth in this manual and any others that may be required by the project Owner’s specifications or the American Society for Testing Materials (ASTM). No welded wire soil reinforcement or precast concrete panels will be shipped to the jobsite without a Certificate of Compliance signed by a California Licensed PE from SSL.

Steel suppliers must ensure that all material is produced and fabricated according to ASTM guidelines (ASTM-A82 and ASTM-A185) along with any other requirements of the project Owner’s specifications. All material must be properly identified and tagged. Steel suppliers shall provide wire coupons (wire rod samples, 1 from each spool) for production go-ahead testing prior to mesh fabrication, which will not proceed until wire coupons achieve a satisfactory result. Production go-ahead testing shall be performed by a qualified third party testing agency¹. Steel suppliers shall provide mesh coupons

¹ Testing agency certified in accordance with ISO 17025, NADCAP AC-7101 or similar.
(fabricated mesh samples, 1 per change in wire size or change number of longitudinal wires per sheet) to be galvanized and sent to the contractor as a control lot.

Steel galvanizers must ensure that all material is coated according to ASTM-A123 guidelines and tested according to ASTM-A143, along with any other requirements of the project Owner's specifications. All material must be properly identified and tagged to ensure proper installation at the jobsite.

Concrete panel suppliers must ensure that all material is produced according to the project Owner's approved Quality Control procedure along with any other requirements of the project Owner's specifications. Concrete panel supplier’s Quality Control Manager must complete and sign the Precast Quality Control Checklist included in Section 10.

For complete details, please see Section 10 containing the current Quality Control Programs for; Concrete Reinforcements, Inc; and AZZ Galvanizing Inc. An individual Quality Control Program for concrete panel suppliers has not been included because concrete suppliers are selected on a job by job basis. The individual Quality Control Program of the selected supplier will be subject to review and will comply with SSL's Quality Control Program.

4. SSL PERSONNEL RESPONSIBILITIES

**General Management**: The general managers are responsible for developing a Quality Control Policy that will ensure that each element of the product supplied to customers meets or exceeds all applicable standards. The general managers are responsible for allocating a sufficient amount of time and resources to the development and implementation of a Quality Control Program that will reflect the Quality Control Policy.
Project Management: The Project Managers are responsible for the implementation and management of SSL’s Quality Control Policy, this includes making material suppliers aware of the program and proficient with all aspects. The project managers will ensure each material supplier has their own Quality Control Program in place and that it reflects SSL's own Quality Control Policy. The project managers will provide a Certificate indicating Compliance (COC) with SSL's Quality Control Program for each load of material delivered to the jobsite, the COC must be signed by a Professional Engineer Licensed in the State of California. When specified, the project managers will coordinate with the contractor’s Quality Control Manager.

Material Supply: At the direction of project management, the production and delivery of materials will be coordinated with the material supplier. Project management must ensure that all material is being produced in accordance with SSL’s Quality Control Program and attain verification that all material is being produced in accordance with each supplier's own Quality Control Program. Project management may at any time in the production process, request changes to the supplier’s Quality Control Program or request additional Quality Control or testing procedures to be performed by the supplier.

Document Control: At the direction of project management, any and all necessary or requested material compliance certifications and testing logs will be collected from material suppliers. All records will be stored on a project specific basis such that information can be quickly and easily verified or distributed as needed.

Material Testing and Inspection: At the direction of project management, specific material components or elements of the entire system may be tested by SSL's material testing personnel. Testing may be performed as proof of design concept or as production compliance verification. Inspection may be performed at the job-site or at each material supplier's location.

5. MATERIAL TESTING AND INSPECTION PERFORMED BY SSL

SSL personnel have the option to perform different testing and inspection methods on the material elements of the system at any stage of production or installation. These tests are in addition to those required by ASTM and AASHTO. These tests were designed to reflect those prescribed by ASTM but modified to allow for on-site testing performed outside of a lab. When specified, SSL will coordinate with a qualified third party\(^2\) to perform additional testing. A report detailing any results will be distributed to the parties involved.

\(^2\) Testing agency with expertise in metallurgy, galvanization or precast concrete.
The list below contains methods that are currently being employed, SSL is not limited to these methods and may adopt others at any point in time. If other methods are employed at a later date, SSL will revise the Quality Control Program to reflect the changes. SSL will determine which tests will be employed and the frequency of testing on a project specific basis, a summary and outline of the project specific program will be included in Shop Drawings that are submitted for approval at the beginning of the project.

**Connection Test:** SSL has developed a test procedure that verifies the load capacity of the concrete panel to soil reinforcement connection elements. Please see the attached Test Procedure for more details. This test will be performed for System Qualification and for evaluation of Steel Suppliers.

**Bend Test:** A bend test verifies the ductile strength of the steel and identifies embrittlement. This test can be performed either on single wire coupons both before and after the galvanizing process, and on welded-wire mesh after fabrication. This test may be performed at any point in the production process or on samples selected from material that has been shipped and is ready for installation. Questionable results will require further evaluation and may be cause for rejection of material. SSL may submit questionable material to a qualified independent testing laboratory for further evaluation.

If the wire has not been bent during mesh fabrication, a single wire is isolated and restrained by a vice or clamps, the wire is bent (max 1 inch bend radius) through 90° and back so that the straightened piece will be examined. The steel must complete the bend test without exhibiting any visible cracking at the bend.

If the wire has been bent during mesh fabrication, a single wire is isolated and restrained by a vice or clamps leaving the bent portion free. Using a minimum 3' long section of 1" metal pipe, the wire is bent to straight so that the straight piece is examined. The steel must complete the bend test without any visible cracking occurring at the bend.

**Production Go-Ahead Testing:** The bend test discussed above will be performed at a rate of 1 sample per wire spool for all wire produced. Samples will be taken from a recently drawn spool, galvanized and bend tested prior to mesh fabrication. Mesh fabrication will not proceed without satisfactory results.

Unsatisfactory results will require that the material in question be removed from production pending further evaluation.

**Impact Test:** An impact test is performed at room temperature on steel samples that have been bent then galvanized and is intended to identify embrittlement.
This test will be performed once for each project during site-visits by SSL personnel, and may also be performed at any time during the production or installation process. Questionable results will be cause for further evaluation.

As detailed in Figure 1 below, a single wire is isolated and restrained by a vice or clamp. Using a small metal mallet or hammer (20 oz or larger), the wire sample is struck up to three times directly on the bent portion: The first strike will be a light tap with the swing initiated at the Tester’s wrist, if no cracking occurs continue on to the second strike; The second strike will be stronger and initiated at the Tester’s elbow, if no cracking occurs repeat the strike with the same strength. The steel must complete the impact test without any visible cracking occurring at the bend.

**FIGURE 1**

**Visual Inspection at Jobsite:** A thorough visual inspection of any SSL supplied material on site is performed during SSL 's onsite technical representation or by the contractor. The inspection is intended to verify that no deformations or broken bends or welds are visibly present in the reinforcing mesh. This test will only identify those issues that are readily apparent as some of the material is typically obscured by packaging or not accessible.

In the absence of an SSL representative, any material found by the Contractor to be out of SSL tolerances shall be tagged and removed from the installation process. The contractor shall notify SSL immediately for further evaluation of the questionable material.

The steel identification tags attached by the galvanizer are checked against actual sizes and styles of reinforcing mesh. If discrepancies exist, the actual size
and styles of the reinforcing mesh can be determined by the contractor by measuring the wire size, longitudinal wire quantity and the distance of transverse wire spacing. Proper identification labels should be attached to ensure proper installation.

If any broken wires or welds are present they will be flagged and the mesh sheet will be evaluated by an SSL engineer for integrity. If SSL determines that the integrity of the mesh sheet is compromised, the sheet will be removed from the installation process and tagged.

**Process Audit at Steel Supplier:** SSL personnel may elect at any time during the production process to perform an on-site inspection of the entire production process at each steel supplier's location. *Process Audits will take place* *a quarterly* *(minimum of four times a year).* The inspection is intended to verify that the production process is in compliance with all aspects of both SSL's and the Supplier's QCP. SSL personnel may decide to include as an agent, an industry expert not affiliated with the supplier in the inspection to provide an independent opinion. A detailed report of the audit will be generated by SSL and kept as a record by SSL and the reinforcement supplier facility. A copy will be supplied to Caltrans upon request.

If SSL's personnel or agent determines that changes to the production process are necessary, the Supplier in question will implement the necessary changes without any delays to the production process.

**First-Article Inspection and Process Audit at Precast Supplier:** For Quality Validation, SSL personnel will perform at least four First-Article Inspection and Process-Audits per year and at the start of production for a specific project if required by the Project Specifications. SSL may also elect at any time during the production process to perform an on-site inspection of the any aspects of the entire production process at the concrete panel supplier's location.

The inspection will involve an examination of the casting and testing process for one representative panel from each potential type of panel for the manufacturing period or project (e.g. “A” panels, “X” panels, top-panels, L/R panels or corner-panels) and any unique product/architectural finish as well. The inspection will focus on the following items: embed loop clearance and spacing, panel identification and tracking, verification of concrete strength, process documentation and general condition of formwork. The inspection is intended to verify that the production process is in compliance with all aspects of both SSL's and the Supplier's QCP and that the Supplier can produce material in accordance with SSL Shop Drawings and Project Specifications.
A report of the inspection will be kept on file at the precast facility and submitted along with the Certificate of Compliance (COC) that accompanies the material to the jobsite (see sample First Article Inspection and Process Audit Report in). A copy will be supplied to Caltrans upon request.

SSL personnel may decide to include as an agent, an industry expert not affiliated with the supplier in the inspection to provide an independent opinion. If SSL’s personnel or agent determines that changes to the production process are necessary, the Supplier in question will implement the necessary changes without any delays to the production process.

6. QUALITY CONTROL PROGRAM FLOWCHART

Below is a flowchart summarizing the entire QC process as required by SSL’s QCP and the individual QCP’s of the material suppliers. This chart combines the steps taken by SSL along with the steps taken at each supplier’s location. For detailed flowcharts that cover the procedures of the material suppliers, please see Section 7.
7. MATERIAL TESTING AND INSPECTION PERFORMED BY SUPPLIERS

Below are the QC Procedure Flowcharts for two of SSL's Suppliers, they reference the attached individual QC Programs in place at those Suppliers and are made part of SSL's QC Program. Should SSL elect to use a different supplier as a substitute, SSL will examine their QC program and perform the necessary QV inspections to ensure that it meets or exceeds the requirements laid out below.
Concrete Reinforcements, Inc: QC Process Flowchart for SSL Soil Reinforcing

**INSPECTION OF RAW MATERIAL**

Implement changes to production process.

**PRODUCTION OF ROLLED WIRE**

Tests performed:
- Diameter
- Bend
- Tensile
- Reduction of Area
- Embrittlement

Performed by: Rolling Line Operator, Quality Control Manager

Frequency: 1 sample per spool (per SSL). At start, then every 3rd spool or 9 tons by line operator, daily by QC Manager (per ASTM)

(all tests performed in accordance with ASTM-A82)

**DATA RECORDING**

- Date
- Wire Size
- Carbon Grade
- Heat Number
- Wire Test Result
- Bend Test Result
- Diameter Size
- Tensile Load
- Tensile Strength
- Reduction of Area

Performed by: Quality Control Manager

**FAIL**

- Hold lot for further testing and evaluation.
- Determine cause of unsatisfactory performance.
- Identify necessary changes to production process.

**PASS/FAIL?**

**PASS**

**STRAIGHTEN & CUT**

Production go-ahead testing:
- 1 wire coupon per spool sent to third-party test for embrittlement testing.

**PROCEED WITH FABRICATION BASED UPON POSITIVE BEND TEST RESULT**

Tests performed:
- Tensile Strength
- Bend
- Weld Force
- Reduction of Area

Performed by: Operator, Quality Control Manager

Frequency: For shear test: no less than 4 cross-sectional samples from initial sheet, no less than 4 samples taken at random from every 300,000 sf. Fe tensile strength test: no less than 1 sample taken from longitudinal and transverse wire's (fibers different) at test sheet, no less than 1 sample per 75,000 sf.

(All tests performed in accordance with ASTM-1165)

**FAIL**

- Test samples are rejected by styx and heat number for rejection.

- MESI COUPONS:
  - 1 coupon per 100 mesh sheets or
  - 1 coupon per every wire size change or mesh in sheet width change.

**PASS/FAIL?**

**PASS**

**BEND & SHIP**

Certificate of Compliance with ASTM issued by CRI

**FAIL**

- Notify SSL

Certificate of Compliance issued by SSL
AZZ Galvanizing Services: QC Process Flowchart for SSL Soil Reinforcing

1. Receive raw wire mesh and sample mesh components.
2. Visually inspect for damage during shipping.
3. Verify receipt count.
4. Bare steel loaded into dipping racks and cleaned (pickled).
5. Steel dipped in zinc bath to galvanize.
6. Steel removed from zinc bath.
7. Wire mesh and test coupons tested according to ASTM A 123 and ASTM A 143.
8. Hold material and notify SSL if not satisfactory.
9. Certificate of ASTM compliance issued by AZZ.
10. Mesh and mesh coupons loaded and shipped to contractor.

Material, quantity and weight entered in Oracle system and recorded daily.

Lead operator inspects material to verify proper pickling, fluxing and venting.

Inspect for any ungalvanized areas, sharp projections and general surface condition.
8. CONCLUSION

SSL feels that our QCP in conjunction with the QCP's in place at our Material Suppliers, will ensure that the products we provide meet or exceed all applicable standard and specifications. If after review, any aspects of our QCP or the individual QCP's of our Material Suppliers are found to be insufficient, SSL will make any necessary changes to our QCP and notify our Material Suppliers of any necessary changes. Should SSL become aware of any additional requirements, SSL's and the individual QCP's of our Material Suppliers will be revised to reflect those requirements.
3/7/2012

Scott Thompson Jr.
SSL LLC
4740 Scotts Valley Dr. STE E
Scotts Valley, CA. 95066

Regarding: Increase Quality Practices for SSL Soil Reinforcing Mesh

Mr. Scott Thompson

Attached is Concrete Reinforcements General Quality Plan as well an Enhanced Quality Plan designed for production of SSL Soil Reinforcing Mesh. This plan was implemented February 2012.

Best Regards,

JT Wright
Concrete Reinforcements, Inc.
Raw Material Inspection for Wire Rod

**Purpose:** This Raw Material Inspection is designed to verify materials strength and dimensional properties as well as verifying the chemical properties are within specification.

**Frequency:** One sample for every different heat number from the steel mill

**Method:**

1. As a shipment is received, verify that Mill Certificates have been provided for each heat number that has been delivered.
2. Obtain one sample (approx. 12") for each different heat number for testing the diameter. Refer to ASTM A510 Table 8 for diameter conformance. Refer to ASTM A510 Section 11 if material does not meet the requirements. Otherwise, proceed to step 3.
3. Using the same sample from the diameter test, test the tensile strength per ASTM A370. Verify it to be within the range specified on the Mill Cert.
4. Verify that the chemical composition of each heat is within the parameters required by A510 using Table 3. Material ¥frac{1}{2}" or greater must be Grade 1010. Material less than ¥frac{1}{2}" will either be 1010 or 1008.

If the tests and inspections are within limits, all material within the heat shall be tagged with a color code. If anything is out of limit, the material shall be put through further processing tests to verify it will meet ASTM Standards for welded wire reinforcing as well as passing the galvanized embrittlement per ASTM A143.

**Recorded Data:**

1. Material Tensile noted on Mill Cert
2. Chemical verification signed off
3. Diameter noted on Mill Cert
Rolled Wire Inspection - SSL Specific Requirements

**Purpose:** This inspection is designed to verify material conformance to ASTM A82 and A143 for Embrittlement

**Frequency:** One sample (approx 2 ft) per spool.
   Note: additional samples may be requested for outside testing

**Method:**

1. Check the diameter around the entire wire using Venier Calipers to verify the wire is within range according to ASTM A82 Table 4 & section 6
2. Perform a Bend Test on each sample per A82 section 5.2
3. Perform a Tension Test on each sample per A82 section 5.1
4. Perform a Reduction of Area Test Per A82 section 5.3
5. Label the bent portion of each sample with the Heat Number, Wire Size & Date and submit them to the galvanizer for the appropriate Embrittlement Test. Wire sizes that do not get bent prior to galvanizing are not required to be tested for Embrittlement.

If the tests and inspections are within limits, material may be staged and tagged for the Straighten and Cut Process. If anything is out of limit, the material shall be placed in a non-conforming area and may not be used for SSL Soil Reinforcing unless additional testing validates the quality of the finished product

**Recorded Data:**

1. Date of Rolling
2. Diameter Min & Max
3. Rod Size & Heat Number
4. Carbon Content
5. Wire Size
6. Lbs on spool
7. Bend Test Verification
8. Reduction of Area
9. Tensile Strength
10. Embrittlement Test Verification / Certification
Weld Shear and Tensile Test Across Welds - SSL Specific Requirements

Purpose: This test is designed to verify weld shear integrity and tensile strength of wire that has been welded

Frequency: One sample sheet for every different width and/or wire size.

Method:

1. Before each style change (width, wire size or machine positioning) weld a replica test sample of the sheets to be succeeded by this test.
2. Perform a Weld Shear Test per ASTM A185 Section 7 & 10 for each intersection across the width of the test sample
3. Perform a Tensile Test across the welded intersections per ASTM A185 Section 7.1 for every different wire size or 75,000 square feet. If the fractured section is of sufficient distance from the weld to measure diameter, perform a reduction of area test per A185 Section 7.2
4. Bend the remaining portion of the replica sample in the same manner as its job requires. Tag it accordingly and ship it with the order in which it corresponds. (this is intended for further process testing if required by the purchaser)

If the tests and inspections are within limits, material may be staged for bending. If anything is out of limit, the material shall be placed in a non-conforming area and may not be used for SSL Soil Reinforcing unless additional testing validates the quality of the finished product

Recorded Data:

1. Date of Weld
2. Job Number and Styles Tested
3. Wire Size
4. Weld Shear Ultimate PSI
5. Tensile Ultimate PSI
Bend Setup Inspection

**Purpose:** The purpose of this inspection is to verify accuracy of the bender setup according to the job requirement

**Frequency:** Once for each bend diagram provided with each job.

**Method:**

1. Check the vertical and horizontal dimensions to ensure the jig pins adjusted correctly.
2. Verify Mandrel size is correct
3. Verify the angle adjustment is correct

If any of the preceding inspections are incorrect, make the necessary adjustments before proceeding

**Recorded Data:** N/A
QUALITY CONTROL PLAN
(IMPLEMENTED 11/98)

ROLLED WIRE:

1. FREQUENCY OF SAMPLING AND TESTING

   A. Each rolling line operator will initially cut a 12" sample coupon from the start of the rolling process and continue to every third spool of wire produced of the same size to ensure it meets all weight, deformation and tensile spec’s required. Operators will repeat the same process after each change over to a different wire size’s to meet all required specifications.

2. TESTING SMOOTH WIRE SAMPLES

   A. Smooth wire is tested in accordance with ASTM-A82. The following tests are performed.
     - Diameter
     - Bend Test
     - Tensile
     - Reduction of Area

3. RECORDED DATA OF SMOOTH WIRE

   A. Sample coupons are tested daily by Quality Control to ensure all wire samples conform to ATMS-A82 steel welded wire specification for yield, weight and tensile.

   B. Internal data recorded is as follows:
     - Date
     - Wire Size
     - Carbon Grade
     - Heat Number
     - Bend Test
Concrete Reinforcements Inc.

- Wire ID Number
- Diameter Size
- Tensile Load
- Tensile Strength
- Reduction of Area

4. SAMPLE TESTING OF DEFORMED WIRE

A. Deformed wire is tested in accordance with ASTM-A 496.

B. Tests performed are as follows:
   - Weight per Linear Foot
   - Tensile
   - Bend Test
   - Height of Deformations

5. RECORDED DATA OF DEFORMED WIRE SAMPLES

A. Sample coupons are tested daily by Quality Control to ensure all wire samples conform to ASTM A-496 steel welded wire specification for yield, weight and tensile and filed.

B. Data recorded is as follows:
   - Date
   - Wire Size
   - Heat Number
   - Bend Test
   - Tag Number
   - Weight per Foot
   - Tensile Load
   - Tensile Strength
   - Deformation Height
WELDED WIRE FABRIC

1. FREQUENCY OF TESTING WELDED WIRE FABRIC

A. Shear testing of welded wire fabric will constitute of no less than four cross sectional welded samples cut out staggered through out the initial sheet to ensure welded consistency is achieved. No less than four test samples are taken for every 300,000 square feet to conform to the weld shear strength requirements for the welded wire fabric to conform to A-497.

B. Tensile testing of welded wire fabric will constitute a minimum of one sample taken from the longitudinal and horizontal wire’s if differs in wire sizes from welded fabric being tested. Testing of samples will be taken at intervals of no less than 75,000² of welded fabric to conform to ASTM A-497 & A185.

C. The samples are identified by style, order number, and tested in accordance with ASTM-A185 or A497.

D. Test performed are as follows:
   - Tensile Strength
   - Bend
   - Wels Shear
   - Reduction of Areas (Smooth Fabric Only)

2. RECORDED DATA OF DEFORMED WIRE SAMPLES

A. Sample coupons are tested daily by Quality Control to ensure all wire samples conform to ATMS-A82 496 and 497 steel welded wire specification for yield, weight and tensile.

B. Data recorded is as follows:
   - Customer
   - Mesh Style
   - Date
   - Bend Test
   - Tensile Strength

27
Concrete Reinforcements Inc.

- Pass Value of Weld Shear Test
- Weld Shear Load
- Reduction of Area (Smooth Wire Fabric Only)

3. FINAL CERTIFICATION

Final job order certifications are signed only by the Quality Control Manager, certifying the material ordered adheres to the designated ASTM specification, and domestic origin.

13450 West Peoria Avenue  Surprise, Arizona 85379
Phone: 623-975-2970  Fax: 623-975-1278
Quality Management System
Quality Manual

FACILITIES:

CROWLEY
200 NORTH BEVERLY
CROWLEY, TX

CINCINNATI
4454 STEEL PLACE
CINCINNATI, OH

DENVER
4400 EAST 61ST AVE.
COMMERCE, CO

JACKSON
125 AZTEC DR
RICHLAND, MS

MUNCIE
2415 S. WALNUT ST
MUNCIE, IN

TULSA
1800 W. 21ST ST
TULSA, OK

HOUSTON
7407 CE KING PKWY
HOUSTON, TX

DIXON
310 E. PROGRESS DRIVE
DIXON, IL

CANTON
1723 CLEVELAND AVE. S.W.
CANTON, OH

MOSS POINT
4212 DUTCH BAYOU RD.
MOSS POINT, MS

CHELSEA
6022 S INDUSTRIAL RD
CHELSEA, OK

HOUSTON WEST
9103 FAIRBANKS N HOUSTON RD
HOUSTON, TX

ARKANSAS
998 ESCUE DRIVE
PRAIRIE GROVE, AR

PLYMOUTH
2831 JIM NEU DR
PLYMOUTH, IN

ST. LOUIS
1461 KIN ARK CT
ST. LOUIS, MO

WASKOM
990 E. TEXAS AVE
WASKOM, TX

PEORIA
6718 PLANK RD
PEORIA, IL

RSJ - CATOOSA
5101 BIRD CREEK
PORT OF CATOOSA, OK

MOBILE
17640 INDUSTRIAL PARK
CITRONELLE, AL

HAMILTON
7825 HOMESTEAD DR
HAMILTON, IN

NASHVILLE
200 32ND AVE. N.
NASHVILLE, TN

HOBSON
2402 ENGINEERS RD
BELLE CHASSE, LA

JOLIET
625 MILLS RD
JOLIET, IL

LOUISVILLE
6310 KENJOY DR.
LOUISVILLE, KY

ARIZONA
16775 ELWOOD ST
GOODYEAR, AZ

WINSTED
800 6TH ST
WINSTED, MN

HURST
626 W. HURST BLVD
HURST, TX

BEAUMONT
5890 INDUSTRIAL RD.
BEAUMONT, TX

BRISTOL
14871 INDUSTRIAL PARK RD
BRISTOL, VA

KANSAS CITY
7700 E. 12TH ST.
KANSAS CITY, MO

WESTSIDE
3520 S. RIVERVIEW
PORT ALLEN, LA

PILOT
MANILA CREEK RD
POCA, WV

WHEELING
748 MCMECHEN ST SUITE 11
BENWOOD, WV
Contents

0.0 Introduction ................................................................. 4
1.0 Duties of Personnel ......................................................... 4
2.0 Quality Management ....................................................... 5
  2.1 Quality Management Team ............................................. 5
  2.2 General Requirements ................................................ 6
  2.3 Document Control ...................................................... 7
3.0 Management Responsibility ............................................. 8
  3.1 Management Commitment ............................................. 8
  3.2 Customer Focus ......................................................... 8
    3.2.1 Customer Requirements ......................................... 8
  3.3 Quality Policy ........................................................ 9
    3.3.1 Quality Policy Communication and Framework for Quality Objectives ........................................ 9
  3.4 Quality Planning ........................................................ 9
4.0 Product Specification .................................................... 10
  4.1 Planning of Product Specification .................................... 10
  4.2 Customer-Related Processes ......................................... 10
    4.2.1 Determination of Requirements Related to the Product .................................................. 10
    4.2.2 Review of Requirements Related to the Product .................................................. 11
  4.3 Customer Communication .............................................. 11
  4.4 Purchasing ............................................................. 11
    4.4.1 Purchasing Process .............................................. 11
    4.4.2 Purchasing Information .......................................... 12
  4.5 Production and Service Provisions ................................ 12
    4.5.1 Control of Production and Service Provisions ................. 12
    4.5.2 Customer Property .............................................. 12
  4.6 Control of Monitoring and Measuring Equipment .................. 13
    4.6.1 Instruments ...................................................... 13
5.0 Measurement, Analysis and Improvement ............................ 13
  5.1 Monitoring and Measurement ....................................... 13
    5.1.1 Customer Satisfaction .......................................... 13
  5.2 Internal Audit ......................................................... 14
  5.3 Control of Nonconforming Product .................................. 14
  5.4 Improvement ........................................................ 14
0.0 Introduction

AZZ Galvanizing Services is committed to a policy of conformance to quality requirements for each function and product of the organization. The policy has been instituted to ensure client confidence, that its services and products are as requested and meet the rigorous specification requirements. This policy is enhanced with an added degree of excellence attributed to the inclusion of substantial control on the administration, procurement, engineering, operation and quality functions of the organization.

The policy is implemented with the adoption of a Quality Management System (QMS) which ensures the organizational structure, responsibilities, processes and resources consistently provide product meeting customer, statutory and regulation requirements. If procedures are found to be in need of improvement, measures will be initiated to revise officially the anomaly to an acceptable level of clarity and resolution.

AZZ Galvanizing Services has planned for the long-term, a strategy utilizing continuous quality improvement, combined with responsive, near-term actions to enable it as an organization, to be a quality leader.

1.0 Duties of Personnel

Senior Vice President and Chief Operating Officer, Galvanizing Services
The Senior Vice President and COO of Galvanizing is responsible for the overall operation of the Galvanizing Services segment. The Senior Vice President and COO of Galvanizing has been assigned the overall responsibility to plan, implement and maintain the Quality Management System.

Vice President of Operations
The Galvanizing Vice President of Operations is responsible for administration, customer relations, safety and quality of the plant under their division.

Galvanizing Regional Manager
The Galvanizing Regional Manager is responsible for administration, customer relations, safety and quality of the plant under their region.

Plant Manager
The Galvanizing Plant Manager is responsible for administration, customer relations, production, maintenance, safety, and quality of service and coating. The Plant Manager is responsible for seeing that all materials are galvanized in accordance with the applicable specifications.
Office Manager
The Office Manager analyzes, organizes and implements office procedures involving accounting payroll, human resources, information management, filing systems, requisition of supplies and other clerical services.

Plant Supervisor
The Plant Supervisor is responsible for production, maintenance and quality of products produced under shift supervision.

Maintenance Supervisor
The Maintenance Supervisor analyzes and resolves work and production problems, performs preventative maintenance, and troubleshoots equipment failures.

Quality Management/Customer Service Representative
The Quality Management/Customer Service Representative is responsible for proper quality training, document control, document review and adherence monitoring.

2.0 Quality Management

The AZZ Galvanizing Services Quality Manual addresses the requirements of the quality standard and defines the company Quality Management System (QMS). This is the overall responsibility of the Quality Management/Customer Service Representative with input and help from all Quality Management team members, and overseen by the Plant Manager.

2.1 Quality Management Team

This chart is an example of the plant management organization. Actual organization and number of employees may vary at each plant.
2.2 General Requirements

The Quality Management Team implements and maintains the effectiveness and efficiency of the organization's performance by considering the needs of customers through the following steps:

a. Determine the processes needed for the QMS and their application throughout the organization.
b. Determine the sequence and interaction of these processes.
c. Determine criteria and methods needed to ensure that both the operation and control of these processes are effective.
d. Ensure the availability of resources and information necessary to support the operation and monitoring of these processes.

e. Monitor, measure where applicable, and analyze these processes.

f. Implement actions necessary to achieve planned results and continual improvement of these processes. Through the use of the Quality Management Team, all quality related issues are resolved in a timely and efficient manner.

The Quality Management Team directs the Quality Management System. It establishes and reviews all quality policies and is the main policy setting body within the plant. The team interprets the quality policy, and develops quality system objectives, and measures quality performance and institutes corrective action when necessary. The Quality Management Team also controls manufacturing resources and ensures the Quality Management System is in compliance with required regulatory standards.

When chosen to outsource any process that affects product conformity to requirements control over such processes will be ensured. Control of such processes will be identified within the Quality Management System by request for quote, quotation, purchase request, and purchase order. Any required sketches, drawings, specifications, explanations, or standards, will be included with the communications for the outsourced vendor.

2.3 Document Control

The Quality Program Manual and all documents and records required by the manual are controlled and are to be maintained and retained by the appropriate Plant Manager or Vice President, as defined by the manual.

The Quality Program Manual is prepared and distributed by the Manufacturing Engineer Galvanizing Services.

- All controlled manuals will be numbered prior to distribution.
- Controlled in-company distribution of quality manuals will include the Senior Vice President Galvanizing Services, Vice President of Operations, Director of Engineering, Manufacturing Engineer, Galvanizing Marketing Manager, Galvanizing Sales Manager, Galvanizing Regional Manager, Galvanizing Plant Manager, and Sales Manager, each of whom will acknowledge receipt by signature of record to the Senior Vice President.
- Distribution outside the company will be as appropriate and necessary, with receipt acknowledged by signature to the Plant Manager. Manuals distributed outside of the company are considered uncontrolled copies of the Plant Manager’s Quality Manual.
- The Plant Manager will maintain records of the distribution of all uncontrolled manuals and receipt signatures.

Revisions of the Quality Program Manual will originate with the Vice President of Operations, and be distributed with a revised contents page to each manual holder of record.
3.0 Management Responsibility

The Quality Management Team implements the Quality Management System and continually improves its effectiveness by:

a) Communicating the importance of meeting customer and regulatory requirements.

b) Establishing the quality policy.

c) Ensuring that quality objectives are established.

d) Conducting management reviews as necessary.

e) Ensuring the availability of resources.

The Quality Policy, Quality Objectives/Goals and the Quality Management Team support the development, management and continuous improvement in effectiveness of the Quality Management System.

3.1 Management Commitment

The Quality Management Team will formally review the QMS at a minimum of once per year to ensure its continuing suitability and assess opportunity improvements.

3.2 Customer Focus

AZZ Galvanizing Services depends on its customers and focuses to understand current and future customer needs. To that end, customer requirements are determined and fulfilled with the aim of enhancing customer satisfaction.

3.2.1 Customer Requirements

Upon receiving the customer’s material, the receiving supervisor or designee will review the customer designated requirements. If customer requirements surpass specifications by
statutory or regulatory standards such as ASTM A123 and/or ASTM A153, it is the Plant Manager’s responsibility to review these requirements with the customer prior to plant commitment to perform any galvanizing service.

3.3 Quality Policy

AZZ Galvanizing is committed to continually improve the effectiveness of the Quality Management System. This commitment is expressed by:

- **Absolute Quality Provider**
- **Zero Injuries**
- **Zero Customer Complaints**

3.3.1 Quality Policy Communication and Framework for Quality Objectives

The Quality Policy provides the framework for the company Quality Objectives. This Quality Policy is communicated and understood throughout the organization. The policy is posted on bulletin boards, posted at work stations, posted in the offices, posted in the lunchroom and reception area.

3.4 Quality Planning

The Quality Management/Customer Service Representative manages and develops quality objectives, measures quality performance and institutes corrective action when quality objectives are not consistent with the quality policy. The Quality Management Team manages manufacturing resources and ensures that quality objectives, including those needed to meet requirements for product are established at relevant functions and levels within the organization.

The quality objectives are measurable and consistent with the quality policy:

- Pounds shipped
- External/Internal Rework
- Customer Complaints
- Pounds per man hour
- OSHA Accidents
- Sales per month
4.0 Product Specification

The drive for continual improvement of the organization's performance focuses on the improvement of the effectiveness and efficiency of processes, to achieve increased benefits, increased customer satisfaction, improved use of resources and reduction of waste. The standard specification is in accordance with ASTM A123 unless otherwise specified.

4.1 Planning of Product Specification

In planning of product specification, the Quality Management Team plans and develops the processes needed for product realization and evaluates planning of product specification to be consistent with the requirements of the other processes of the Quality Management System.

In planning product specification, the organization shall determine the following, as appropriate:

- Quality objectives and requirements for the product.
- The need to establish processes, document and provide product specific resources.
- Required verification, validation, monitoring, inspection and test activities specific to the product and the criteria for product appearance.

4.2 Customer-Related Processes

The Quality Management Team shall have a full understanding of the process requirements of the customer before initiating its action to comply.

4.2.1 Determination of Requirements Related to the Product

It is a combined effort between AZZ Galvanizing Services and the customer to determine product requirements:

- There are requirements specified by customer including packing and shipping instructions plus delivery instructions.
- The requirements not stated by customer but necessary for specific products to be processed with the required zinc thickness and appearance such as ASTM A123 standards.
4.2.2 Review of Requirements Related to the Product

Prior to the AZZ Galvanizing Services' commitment to supply a product to the customer, the review process of requirements related to the product, includes verification of:

- Product requirements are defined.
- Contract or order requirements differing from those previously expressed are resolved.
- That AZZ Galvanizing has the ability to meet the defined requirements.

Where the customer provides no documented statement of requirement, AZZ Galvanizing confirms the customer requirements before acceptance as default to ASTM A123. Changed requirements are identified during review process and evaluated in order to understand the effect on other processes and the needs and expectations of customers. This understanding and its impact need to be mutually acceptable. Relevant documents are amended and relevant personnel are made aware of the changed requirements.

4.3 Customer Communication

The Quality Management Team determines and implements ways for communicating to the customer in relation to:

- Product information.
- Inquiries, contracts or order handling, including amendments.
- Customer feedback, including customer complaints.

Customer communications include quotes, purchase orders, phone calls, fax documents, e-mails, visits, letters, bill of lading and packing slips.

4.4 Purchasing

4.4.1 Purchasing Process

Products will be purchased from approved vendors unless it is a one-time only purchase. The Quality Management Representative or the Plant Manager must approve one-time-only purchases.

Selection criteria for approved vendors are based on:

- Cost
- Product/Service quality
4.4.2 Purchasing Information

Purchasing information will describe the product to be purchased. The purchasing process is:

- Fill out purchasing order requisition form and/or update purchase order log regarding items to be purchased.
- Plant Manager or designee must approve the purchase order.
- Once the order is placed a copy is given to the requestor and the original is filed in purchasing A/P files.

4.5 Production and Service Provisions

4.5.1 Control of Production and Service Provisions

Controlled conditions for production and service provisions include:

- Availability of information that describe the product characteristics.
- Use of suitable process equipment.
- Control of process equipment by monitoring and measurement equipment where applicable.
- Availability and use of monitoring and measuring devices.
- Implementation of release, delivery and post-delivery activities.

4.5.2 Customer Property

Care will be exercised when handling customer-supplied product. Incoming material will be checked visually for damage in transit. The customer will be notified of any damage prior to unloading damaged material. Permission from the customer to unload damaged material must
be confirmed and documented. Digital pictures shall be taken prior to unloading damaged material and kept on file. If customer property is lost, damaged or otherwise found unsuitable for use, this will be reported to the customer and records maintained.

4.6 Control of Monitoring and Measuring Equipment

Each Galvanizing Plant Manager is responsible for all testing equipment and all records relating to testing equipment.

4.6.1 Instruments

Thickness gauge

- Gauge will be calibrated and/or tested for accuracy according to the manufacturer's recommendations against standards traceable to the National Institute of Standards and Technology (NIST).

The zinc bath temperature probes will be checked for accuracy and/or calibrated annually.

Weight scales will be checked for accuracy and/or calibrated annually.

5.0 Measurement, Analysis and Improvement

The Quality Management Team ensures that an effective and efficient monitoring, measurement, collection, analysis, improvement and validation of data are in place, to ensure that AZZ Galvanizing Services' performance achieves satisfaction of customer by:

- Demonstrating conformity of the product.
- Ensuring conformity of the QMS.
- Continually improve the effectiveness of the QMS.

5.1 Monitoring and Measurement

5.1.1 Customer Satisfaction

It is the responsibility of the Plant Manager to ensure customer satisfaction. The Quality Management Team monitors information related to customer perception of having needs met.
Appropriate actions are to be taken to correct any complaints and to guard against in the future.

5.2 Internal Audit

Internal audits are conducted at planned intervals to determine whether the QMS conforms to planned arrangements in product realization and is effectively implemented and maintained.

The internal audit program is planned on the basis of the status and importance of the processes and areas to be audited, as well as the results of the previous audit (except for the initial audit). The audit criteria, scope frequency and methods are defined. Selection of auditors and performance of audits will ensure the objectivity and impartiality of the audit process. Auditors shall not audit their own work.

The Quality Management Team is responsible to ensure that actions are taken without undue delay to eliminate detected nonconformities and their causes. Follow-up activities will include the verification of the actions taken and the reporting of the verification results.

5.3 Control of Nonconforming Product

Product that does not conform to requirements is identified and controlled to prevent its unintended use or delivery.

Nonconforming product is managed by one or more of the following ways:

- By taking action to eliminate the nonconformity.
- By authorizing its use, release or acceptance under concession by the Plant Manager and where applicable customer consent.
- By taking action to prohibit its original intended use or application, unless authorized by the customer.
- Records of the nature of the nonconformities and any subsequent action taken, including concession obtained, are maintained.
- When nonconforming product is corrected it will be re-verified to demonstrate conformity to requirements.
- When nonconforming product is detected after delivery or use has started, actions will be taken appropriate to the effects, or potential effect, of the nonconformity.

5.4 Improvement
5.4.1 Continual Improvement

The Quality Management Team continually seeks to improve the effectiveness of the Quality Management System through the use of the quality policy, quality objectives, audit results, analysis of data, corrective and preventive actions and management review.

5.4.2 Corrective Action

In order to prevent reoccurrence of nonconformities, the following actions are taken to eliminate nonconformities:

- Reviewing nonconformities (including customer complaints).
- Determining the causes of the nonconformities.
- Evaluating the need for action to ensure that nonconformities do not reoccur.
- Determining and implementing action needed.
- Records of the results of action taken.
- Reviewing the effectiveness of corrective action taken.

5.4.3 Preventive Action

Preventive action is taken to eliminate the causes of potential nonconformities in order to prevent their occurrence. The preventive action requirements are:

- Determining potential nonconformities and their causes.
- Evaluating the need for action to prevent the occurrence of nonconformities.
- Determining and implementing action needed.
- Reviewing the effectiveness of the preventive action taken.

6.0 Work Instruction

6.1 Receiving

Incoming material will be checked visually for damage in transit. The customer will be notified of any damage prior to unloading damaged material. Permission from the customer to unload damaged material must be confirmed and documented. Digital pictures should be taken prior to unloading damaged material and kept on file.

Incoming paperwork will be checked for customer identification. Piece counts will be verified when deemed feasible by the Plant Manager.
Material will be removed carefully from the trucks with forklifts or cranes.

Material is inspected for drainage provisions and mounting holes. If these items are not properly addressed the customer will be called and arrangements made to correct the materials.

The material, number of pieces and weights are entered into the Oracle computer system and recorded on daily shipping and receiving sheets.

Any special galvanizing instructions or requests for fast turnaround times are communicated to the Plant Manager and/or Production Supervisor.

6.2 Racking

Bare steel is loaded into pickle baskets or affixed to a fixture via wire, brackets, chains, pins, hooks, etc.

Products are to be oriented in such that they will gravity drain liquids when removed from the different baths. A lead person or supervisor must be contacted for instruction if products are noticed without drainage provisions.

6.3 Pickling

All tanks are to be monitored per the SOP for temperature and chemical strength. Records of temperature, chemical strength and remedial action will be kept per the SOP.

Alkaline cleaning tank or Acid Degreasing tank - when required, material will be immersed in the cleaning solution to remove oils, greases and paints.

Acid Tank

Material may be chemically cleaned in either:

Hydrochloric Acid (or)
Sulfuric Acid

Material is submerged until all scale, rust or coatings are removed.

Abrasive cleaning may be used as an alternative cleaning method to acid cleaning.

Rinse tanks for the removal of cleaning chemicals will contain water.

Pre-flux tank (zinc ammonium chloride) to prevent oxidation until material is galvanized.
Hoisting Procedure:

- Material is lowered slowly into tanks.
- Material is moved onto the next tank only after a minimum amount of drips are present falling from the lowest point of the load.

6.4 Galvanizing

- The zinc bath temperature will be maintained per the SOP and may be varied to meet the requirement of products being galvanized.
- Kettle lead person will inspect material to verify proper pickling, fluxing, and venting, prior to its entering the kettle.
- All material will remain in the kettle zinc bath until it reaches the temperature of the bath.
- All material will be withdrawn from the zinc bath in a manner that will minimize ash inclusion.
- Major uncoated areas are cause for rejection and return to pickling for regalvanizing if the bare spots are larger than 1 inch in the narrowest dimension (ASTM A123). Minor uncoated areas, one inch or less in the narrowest dimension, will be repaired according to ASTM A780 (latest revision). See the Inspection of Products Hot Dip Galvanized After Fabrication (published by American Galvanizers Association).

6.5 Control and De-racking

- Operators remove materials from fixtures or baskets while inspecting for ungalvanized areas, sharp projections and or surface condition.
- If needed, materials are cleaned using files or grinders, and any needed repairs are conducted in accordance to the ASTM A123 standards.
- Product is weighed and the weights recorded.

6.6 Shipping

- Product is prepared for shipment and thoroughly banded and or shrink wrapped if applicable.
- Final product count, identification, weighing, orientation etc must be checked for compliance with customer specifications.
- Customer is contacted for pickup.
- Materials should be loaded as they were received unless noted not feasible.
- Careful loading and tight bundling is important.
• Carrier is responsible for product once it is loaded and must inspect the load.
PRECAST QUALITY CONTROL CHECKLIST

SUPPLIER:

PROJECT:

DATE:

WALL NO:

CASTING DATES:

RELEASE DATE:

PANEL QTY:

Quality Control Checklist Included?

Concrete Strength Test Results Included?

Quality Control Manager:

Signature:
<table>
<thead>
<tr>
<th>PANEL ID</th>
<th>COMMENTS</th>
<th>READY TO CAST?</th>
<th>CASTING</th>
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<tr>
<td></td>
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</tbody>
</table>

*For a panel to be READY TO CAST it must be checked for the following: Length, width, square, level, clean, oil, rebar size, rebar qty, cover, lifting inserts, embed size, embed placement, embed clearance.

*For a panel to be READY TO SHIP it must be checked for the following: Front and back finish, cracks, spalls, broken edges, lifting inserts, embed placement, length, width, square.

*For a panel to CAST properly it must be witnessed that the concrete is placed, finished, vibrated and cured per all applicable Specifications.
4.1.3 Miscellaneous Component QA/QC
# MECHANICAL TESTS

**Client No. SSL000**  
**P.O. No.: Verbal**  
**Project: SSL**  
**Plastic Pad Compression**

<table>
<thead>
<tr>
<th>Description or Mark No.</th>
<th>CAV-1A</th>
<th>CAV-2A</th>
<th>CAV-2B</th>
<th>Specified Requirements</th>
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</thead>
<tbody>
<tr>
<td>Length x Width, Ins.</td>
<td>5.978 x 2.647</td>
<td>5.972 x 2.646</td>
<td>5.964 x 2.644</td>
<td></td>
</tr>
<tr>
<td>Height, Ins.</td>
<td>0.766</td>
<td>0.769</td>
<td>0.761</td>
<td></td>
</tr>
<tr>
<td>Displacement at 40,000 lbf, in</td>
<td>0.1449</td>
<td>0.1281</td>
<td>0.1138</td>
<td></td>
</tr>
<tr>
<td>Mass, g</td>
<td>135.7</td>
<td>136.1</td>
<td>133.3</td>
<td></td>
</tr>
</tbody>
</table>

Test Equipment: Instron 5985 SN B17048 and 6" Fowler SN TE0601. Tested at 0.05 in/mm.  
Scale: Torbal AD500 S/N 105001062

Remarks: Graphs are presented in Figures 1, 2, and 3. Photograph of specimens before and after test are presented in Figure 4. Displacement at max load was calculated based on zero displacement at 30 lbf.

Results Reported To: SSL

![Signature](signature)  
Colin Subasa, Materials Testing Technician

The results presented in this report relate only to the item(s) tested. 
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Samples pertaining to this report will be discarded 30 days from the date of this report unless otherwise advised.
Figure 1. Load-displacement graph of Specimen CAV-1 A.

Figure 2. Load-displacement graph of Specimen CAV-2 A.

Figure 3. Load-displacement graph of Specimen CAV-2 B.
Figure 4. Photographs of CAV-1 A, CAV-2 A, and CAV-2 B before and after testing.
Section 5: PERFORMANCE
5.3
Performance History
5.1.4 System Approval by User
<table>
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<td>Alabama</td>
<td>Y</td>
<td>28-Aug-2006</td>
<td>B.E. Cox Jr., P.E. Geotechnical Engineer, AK DOT</td>
<td>(334) 206-2270</td>
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<td>Alaska</td>
<td>Y</td>
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<td>Kieth Korri Materials and Testing Headquarters, AK DOT</td>
<td>(907) 269-6243</td>
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<td>Arkansas</td>
<td>Y</td>
<td>27-Sep-2001</td>
<td>Jim Gee Materials Engineer, AR DOT</td>
<td>(501) 569-2400</td>
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<tr>
<td>California</td>
<td>Y</td>
<td>14-Aug-2012</td>
<td>Kathryn Griswell Earth Retaining System Specialist, Caltrans</td>
<td>(916) 227-7330</td>
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<tr>
<td>Colorado</td>
<td>Y</td>
<td>Letter not issued</td>
<td>David Kotzer Product Evaluation Coordinator</td>
<td>(303) 757-9421</td>
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<td>John V Eustis Jr. Competitively Bid Contracts Manager</td>
<td>(302) 760-2030</td>
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<tr>
<td>Florida</td>
<td>Y</td>
<td>10-Dec-2012</td>
<td>Barry Smith Product Evaluation Administrator</td>
<td>(850) 414-4533</td>
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<td>Georgia</td>
<td>N</td>
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<td>Brennan Roney New Product Evaluation Engineer, GA DOT</td>
<td>(404) 608-4816</td>
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<td>Hawaii</td>
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<td>Clarence Miyashiro Highways Division, Materials Testing, HI DOT</td>
<td>(808) 832-3409</td>
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<td>Idaho</td>
<td>Y</td>
<td>22-Dec-1998</td>
<td>Jon Ingram Geotechnical Engineer, Headquarters Construction/Materials</td>
<td>(208) 334-8436</td>
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<tr>
<td>Illinois</td>
<td>Y</td>
<td>17-Sep-1999</td>
<td>Ralph E Anderson Engineer of Bridges and Structures, IL DOT</td>
<td>(217) 785-1462</td>
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<td>Indiana</td>
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<td>31-Aug-2020</td>
<td>Aamir Turk, PE Geotechnical Engineer, IN DOT</td>
<td>(317) 522-9728</td>
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<td>Kansas</td>
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<td>10-Oct-2000</td>
<td>Lon S Ingram, P.E. Chief, Bureau of Materials and Research, KS DOT</td>
<td>(785) 296-3008</td>
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<tr>
<td>Kentucky</td>
<td>N</td>
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<td>William H Phillips T.E. Specialist, Division of Bridge Design, KY DOT</td>
<td>(502) 564-4560</td>
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<td>Massachusetts</td>
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<td>Richard Carpenito Technical Services Engineer, Mass DOT</td>
<td>(617) 951-1348</td>
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<tr>
<td>Michigan</td>
<td>Y</td>
<td>Letter not issued</td>
<td>Rich Endress Office of Construction &amp; Technology</td>
<td>(517) 322-1207</td>
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<td>Montana</td>
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<td>Judy Kempker Executive Director, Division of Professional Registration, MO DOT</td>
<td>(573) 751-0047</td>
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<td>Nebraska</td>
<td>Y</td>
<td>24-Apr-2008</td>
<td>Omar A Qudus, P.E. Geotechnical Engineer, Materials &amp; Tests Division, NE DOT</td>
<td>(402) 471-4567</td>
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<tr>
<td>Nevada</td>
<td>Y</td>
<td>17-Feb-2000</td>
<td>Alan R Hilton Research Division Chief, NV DOT</td>
<td>(775) 888-7220</td>
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<td>New Hampshire</td>
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<td>Scott Hidden Geotechnical Engineering Unit</td>
<td>(919) 707-6856</td>
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<td>North Dakota</td>
<td>Y</td>
<td>Letter not issued</td>
<td>Paul Murdoff Administrator, Office of Structural Engineering, OH DOT</td>
<td>(701) 328-2569</td>
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<tr>
<td>Ohio</td>
<td>Y</td>
<td>15-Nov-2006</td>
<td>Tim Keller, P.E. Administrator, Office of Structural Engineering, OH DOT</td>
<td>(614) 728-2057</td>
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<tr>
<td>Oklahoma</td>
<td>Y</td>
<td>15-Apr-2002</td>
<td>Robert J. Rusch, P.E. Bridge Engineer, OK DOT</td>
<td>(405) 521-2606</td>
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<tr>
<td>Oregon</td>
<td>Y</td>
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<td>Jon Guido, P.E., G.E. State Bridge Engineer, Bridge Engineering Section, OR DOT</td>
<td>(503) 986-3993</td>
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<td>Pennsylvania</td>
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<td>Michael McGonagle New Products Coordinator, PennDOT</td>
<td>(717) 214-4035</td>
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<td>Rhode Island</td>
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<td>South Dakota</td>
<td>Y</td>
<td>3-Jan-2002</td>
<td>Lawrence L. Weiss Chief Engineer, Division of Engineering, SD DOT</td>
<td>(605) 773-3174</td>
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<tr>
<td>Tennessee</td>
<td>Y</td>
<td>19-Jun-2002</td>
<td>Edward P Wasserman Civil Engineering Director, Division of Structures, TN DOT</td>
<td>(615) 741-3351</td>
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<tr>
<td>Texas</td>
<td>Y</td>
<td>3-Nov-2000</td>
<td>Mark P McClelland, P.E. Geotechnical Branch Manager, Bridge Design Section, TX DOT</td>
<td>(512) 416-2226</td>
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<tr>
<td>Utah</td>
<td>Y</td>
<td>6-Oct-2004</td>
<td>Jim McMinimee P.E. Project Development Director, UT DOT</td>
<td>(801) 965-4000</td>
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<td>Vermont</td>
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<td>Virginia</td>
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<td>14-Mar-2008</td>
<td>D. Ashton Lawler, P.E. Program Manager, Geotechnical Design of Structures, VA DOT</td>
<td>(804) 786-2355</td>
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<tr>
<td>Washington</td>
<td>Y</td>
<td>5-Aug-2013</td>
<td>Monique Pawelka, P.E. Division of Bridge and Structure Design, WA DOT</td>
<td>(360) 705-7754</td>
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<tr>
<td>West Virginia</td>
<td>Y</td>
<td>17-Apr-2002</td>
<td>Barney C. Stinnett, P.E. Director, Materials Control, Soils and Testing Division, WV DOT</td>
<td>(304) 558-3505</td>
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<td>Wisconsin</td>
<td>Y</td>
<td>5-Feb-2004</td>
<td>Lee J Schuchardt, P.E. Bridge Research and Automation Engineer, WI DOT</td>
<td>(608) 266-8494</td>
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<tr>
<td>Wyoming</td>
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<td>B. Patrick Collins, P.E. Bridge Program, WY DOT</td>
<td>(307) 777-4427</td>
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</table>
Appendix C4
Initial Technical Evaluation Checklist for Precast Concrete Panel Paired with Inextensible Reinforcement

Guidelines for the Applicant to use this checklist:

1. Provide your submittal in Adobe portable document format (i.e. PDF).

2. Organize the submittal based on the numbered outline shown in the checklist below. Use the numbered outline for a table of contents (TOC). Provide the response for each item in your report. Create links between the items in the TOC and the items in the report and appendices.

3. If reports, drawings or calculations are requested for a section, provide them in the appendix tabbed for that section. For example, design calculations are required for Item 2.3.1. They should be included in Appendix 2.3.1.

4. Mark the checklist at each item to indicate “yes” you have included the relevant information. If you must check “no”, please provide a brief explanation if appropriate.

Section 1: ERS Components

1.1 Facing Unit

1.1.1 Yes No Item

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

1.1.2 ☒ ☐ List each type of facing unit.

1.1.3 ☒ ☐ Provide specifications for each facing unit.

1.1.4 ☒ ☐ Provide standard dimensions, tolerances and typical steel reinforcement schedule (if any is used) for each type of unit (e.g. standard, crest, corner, base, etc.) in plan and section drawings.

1.1.5 ☒ ☐ Provide the target 28-day minimum compressive strength.

1.1.6 ☒ ☐ Provide the target percent air range.

1.1.7 ☒ ☐ Producers will change mix design to accommodate state requirements.

1.1.8 ☒ ☐ Describe with text any unit shear, alignment or bearing devices. Provide specifications and detail drawings.

1.1.9 ☒ ☐ Describe with text any filter which is used to prevent migration of fill soil through wall face. Provide specifications.

1.1.10 ☒ ☐ Describe with text the aesthetic facing options that are available. Provide photos, drawings and brochures as appropriate.

1.1.11 ☒ ☐ Describe any limits on the facing units that are created by curved wall sections and corners.

1.2 Inextensible Reinforcement
### Appendix C4
Initial Technical Evaluation Checklist for Precast Concrete Panel Paired with Inextensible Reinforcement

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<td>1.2.1</td>
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<td>Does the wall system contain what you consider to be an innovation that is related to the reinforcement? If yes, please describe the innovation briefly. As items below apply to the innovation, please describe the innovation in further detail.</td>
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<tr>
<td>1.2.2</td>
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<td>List each reinforcement type that is to be used with the facing system.</td>
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<td>1.2.3</td>
<td>☒</td>
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<td>For each type provide physical property specifications. Address ultimate and yield strengths as well as welds if they are applicable.</td>
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<td>1.2.4</td>
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<td>For each reinforcement type describe corrosion protection measures. If coatings or galvanization are used, provide minimum thickness for 75-year design life (based on the electrochemical requirements listed in AASHTO).</td>
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<td>1.2.5</td>
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<td>For each reinforcement type provide sacrificial steel thickness for 75 and 100-year design life.</td>
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<td>1.2.6</td>
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<td>☒</td>
<td>For each reinforcement type provide the results of any corrosion tests that have been performed.</td>
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<td>1.2.7</td>
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<td>For each reinforcement type provide detail drawings that show dimensional tolerances.</td>
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<td>1.2.8</td>
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<td>Describe with text and drawing details how the reinforcement connects to facing units.</td>
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<td>1.2.9</td>
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<td>List each connection device that is used to connect the facing unit and reinforcement.</td>
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<td>1.2.10</td>
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<td>For each connection device provide physical property specifications. Address ultimate and yield strengths as well as welds if they are applicable.</td>
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<td>For each connection device describe corrosion protection measures and provide specifications. If coatings or galvanization are used, provide minimum thickness for 75-year design life (based on the electrochemical requirements listed in AASHTO).</td>
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<td>For each connection device provide detail drawings that show dimensional tolerances.</td>
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<td>1.2.15</td>
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<td>List facing unit-reinforcement connection strength tests performed, provide test results and strength envelopes the Applicant recommends for design.</td>
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<td>1.2.16</td>
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<td>List reinforcement pullout (ASTM D6706) tests performed and provide results. Provide test soil properties, corresponding pullout friction factors (F*) and scale effect correction factors (α) Applicant recommends for design (it is recognized that for inextensible reinforcement the value of α may be 1.0). Discuss how test results support these recommendations based on Appendix B at FHWA-NHI-10-025. If no tests have been performed, list the default values that should be used based on FHWA-NHI-10-024/025</td>
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### Appendix C4

**Initial Technical Evaluation Checklist for Precast Concrete Panel Paired with Inextensible Reinforcement**

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<tr>
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<th>List soil-reinforcement interface shear (ASTM D5321) tests performed and provide results. List interface friction angle (□) Applicant recommends for design. Discuss how test results support these recommendations. If no tests have been performed, list the default values that should be used based on FHWA-NHI-10-024/025.</th>
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Appendix C4
Initial Technical Evaluation Checklist for Precast Concrete Panel Paired with Inextensible Reinforcement

2.2 Design Example

| Item | Yes | No | 2.2.1 Problems 1 and 2—provide complete calculations for both problems using MSEW. If the design is performed with software that is not commercially available or is proprietary, please provide sample calculations with references to support the analysis. |

Section 3: Construction

3.1 Construction Procedures

| Item | Yes | No | 3.1.1 Does the wall system contain what you consider to be an innovation that is related to the construction procedures? If yes, please describe the innovation briefly. As items below apply to the innovation, please describe the innovation in further detail. |
| Item | Yes | No | 3.1.2 Provide the construction manual for the wall system and at a minimum they should include the following items. |
| Item | Yes | No | 3.1.3 Describe any limitations of facing unit installation at inside and outside curved sections of the wall and at corners as well as any modifications that are required to be made to the facing unit. |
| Item | Yes | No | 3.1.4 Describe procedures to install earth reinforcement at curved sections of the wall and at corners. Specifically address any measures that are to be taken at intersection or overlapping panels of reinforcement. |
| Item | Yes | No | 3.1.5 Describe measures that are required to maintain the design vertical and horizontal alignment of the wall face. |
| Item | Yes | No | 3.1.6 Describe the procedures to install soil in the reinforced soil zone. |
### Appendix C4

**Initial Technical Evaluation Checklist for Precast Concrete Panel Paired with Inextensible Reinforcement**

3.1.7  | ☒  | ☐  | Describe measures that are required to prevent erosion behind and in front of the wall during construction.

3.1.8  | ☒  | ☐  | Describe experience or other special qualifications that are required of the wall construction contractor.

3.1.9  | ☒  | ☐  | Describe the procedures to install soil in the reinforced soil zone.

### Section 4: Quality Control

#### 4.1 Manufacturing

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<td>4.1.1</td>
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<td>Describe the quality control measures that are required for the manufacturing of facing units. You may do this by providing a manufacturing QC manual.</td>
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<td>4.1.2</td>
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<td>Describe the quality control measures that are required for the manufacturing of earth reinforcement components. You may do this by providing a manufacturing QC manual.</td>
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<td>4.1.3</td>
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<td>Describe the quality control measures that are required for the manufacturing of any shear, alignment, bearing or connection devices. You may do this by providing a manufacturing QC manual.</td>
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#### 4.2 Construction

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<td>4.2.1</td>
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<td>Describe the quality control measures that are required during construction of the wall system. If these measures are described in the system’s construction manual then state that they are so included and refer the reviewer to the appropriate section of the submittal.</td>
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### 5: Performance

#### 5.1 Performance History

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<td>5.1.1</td>
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<td>Provide a description of the system’s development and usage history. Then describe the following:</td>
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<td>The oldest three structures.</td>
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<td>5.1.3</td>
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<td>☐</td>
<td>The tallest three structures.</td>
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<td>5.1.4</td>
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<td>Provide a list of private- and public sector users who have approved the use of the system. Also provide the contact information for a person at the user agency who may be contacted regarding the wall system’s performance.</td>
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### 6: Other

#### 6.0 Other Information

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<td>6.1</td>
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<td>In this section, please include anything you think will better help a reviewer understand your ERS that has not been adequately address in the previous questions.</td>
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