EXAMPLE 6 PROBLEM STATEMENT:

The subject building feature is a ground-supported slab system to be constructed as part of a single-story frame-type building. In addition to the superstructure and its structural foundation, two distinct slab-on-ground areas were considered and designed by the project’s structural engineer of record.

The first slab area supports uniformly distributed light storage live load accessed by manually operated pallet jacks. The second slab area is subjected to vertical rack storage accessible by forklift. Both areas are to be exposed concrete surface with no applied finish material. Stored materials are considered to be sensitive to moisture, and in some cases may be in direct contact with the exposed slab surface. Based on available preliminary information related to construction sequencing, it is anticipated that the slabs will be constructed prior to a watertight roofing system being in place.

The slabs-on-ground do not transmit vertical loads or lateral forces from other portions of the superstructure or shallow foundation system to the soil. As such, ACI 318-19 was not the appropriate standard by which the slabs were designed by the structural engineer. Instead, ACI 360-10 “Guide to Design of Slabs-on Ground” was used by the structural engineer as a reference in formulating the slab designs.

Design criteria are as follows:  
\[ f'_c = 4,000 \text{ psi} \]  
\[ f_y = 70,000 \text{ psi (welded deformed wire reinforcement yield strength will be used)} \]  
\[ \text{Concrete Density} = 0.145 \text{ kcf, normalweight concrete } \lambda = 1.0 \]  
\[ \text{Relevant Soil Criteria 150 pounds/in}^2/\text{vertical inch of displacement (subgrade modulus); 0.120 kcf density} \]  
\[ \text{LL Loading = varies} \]

The example includes the following steps:

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<thead>
<tr>
<th>Step 1 – Engineer’s Design Methodology</th>
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<tr>
<td>Step 2 – Comparison to WRI TF-705 Methods</td>
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<td>Step 3 – Reinforcement Detailing</td>
</tr>
</tbody>
</table>
Figure 1: Excerpt from contract structural drawings: foundation plan

SLAB TYPE 1:
- 4" THICKNESS
- CONTROL JOINTS: 15'-0" MAXIMUM SPACING
- 0.067 in² PER FOOT, EACH DIR., FIELD
- 0.048 in² PER FOOT, ACROSS JOINTS

SLAB TYPE 2:
- 6" THICKNESS
- NO CONTROL JOINTS
- REINFORCEMENT: 0.36 in² PER FOOT, EACH DIR.
Figure 2: Excerpt from contract structural drawings: 1/S2 exterior wall section, showing integration of slab-on-ground with perimeter stem wall and supporting foundation. An alternative condition in which the slab-on-ground is held clear of the exterior wall assembly was also considered by the structural engineer but not selected. The edge restraint of the slab-on-ground resulting from the arrangement shown here is reflected in the slab FEA modeling results.
Figure 3: Excerpt from contract structural drawings: 2/S2 and 5/S2 spread footings, showing isolation blockout around building column
Figure 4: Excerpt from contract structural drawings: typical details for isolation joints / blockouts at columns

**Interior (Typical) Blockout**
- Welded Wire Reinforcement Design and Detailing Guide
- Control joint to align with blockout vertex, typ. See plan and schedules for column, baseplate, and anchor rods.
- 3/8" Premolded joint filler all around, full depth of surrounding slab. Cut out top 5/8" of filler and fill with polyurethane sealant.

**Perimeter Blockout**
- Structural foundation wall 3/8" premolded joint filler, provide depths to match adjoining slab thickness and foundation wall height.
- Blockout to be flush with adjoining foundation wall face to suit architectural building envelope details.
- Note: Steel column surfaces below the top of slab but not encapsulated by a minimum of 3 inches of concrete shall be considered to be steel subjected to corrosive conditions. These surfaces shall be painted per the general notes and project specifications.

**Isolation Blockout Notes**
1. Details here are intended to show typical blockout configurations. Project specific variations may apply.
2. Place concrete in blockout after column, brace (as applicable), and all related connections are set, and after slab-on-ground is placed.
3. Plan position of columns shown in typical details is illustrative only. The extent to which column base assemblies extend beyond the protected building envelope - if at all - will be project specific. Refer to foundation plan for column locations in conjunction with other project information to determine need for formed projections as well as required geometry.
4. Where columns are supported on structural piers (not shown) with top elevations below slab on ground, the isolation blockouts' plan geometry shall taper starting from the bottom of slab down to top of structural pier elevation. Three inches (min.) of concrete coverage over steel elements shall be maintained.

**Inside Corner Blockout**
- Notes shown for perimeter location are also applicable to inside corner location.

**Braced Frame Blockout Requirements**
- Hatch represents 1" styrofoam adhered tightly to each side of gusset and all surfaces of brace within blockout.
- An anchor rod as scheduled.

**Outside Corner Blockout**
- Notes shown for perimeter location are also applicable to outside corner location.
Figure 5: Excerpt from contract structural drawings: typical detail for re-entrant crack control bars or deformed wires. These will be required at column isolation blockouts in the area of Slab Type 2, where there are no control joints.
1/4" WIDE SAWCUT JOINT (+1/16" & -0"). LOCATE JOINTS A MAXIMUM OF 15'-0" IN BOTH DIRECTIONS.

FILL FULL DEPTH OF JOINT WITH JOINT FILLER.

SEE PLAN NOTES, GENERAL NOTES, AND TYPICAL DETAILS FOR SLAB-ON-GROUND AND REINFORCEMENT.

TERMINATE REINFORCEMENT ±3" FROM FUTURE SAWCUT, EACH SIDE.

SUPPORT BAR AS REQUIRED

SAWCUT JOINT SHALL BE ONE-QUARTER THE DEPTH OF SLAB ± 1/16" OR 1", WHICHEVER IS GREATER.

THICKNESS OF CONCRETE SLAB ON GROUND.

INTERLOCK MAT. PROVIDE STEEL AREA EQUAL TO 0.10% OF SLAB CROSS-SECTIONAL AREA, PERPENDICULAR TO JOINT. POSITION AT A DEPTH 3/4" CLEAR FROM SAWCUT JOINT. EXTEND 16" ON EACH SIDE OF JOINT.

*Figure 6: Excerpt from contract structural drawings: 3/S2 typical detail for sawcut control joint*
Figure 7: Excerpt from contract structural drawings: 4/S2 typical detail for construction joint transition between slab types. Typical detail shows same slab depth each side of joint, while slabs in example will require a depth transition (4” and 6”). Load transfer dowels will be centered in the 4” thick slab depth.
CONTROL AND CONSTRUCTION JOINT NOTES

1. USING AN EARLY-ENTRY SAW, SAWCUT JOINTS AT CONTROL JOINTS SHALL BE MADE AS SOON AS THE CONCRETE HAS HARDENED SUFFICIENTLY TO PREVENT RAVELING OUT OF THE AGGREGATE OR DAMAGE TO THE EDGES, BUT NO LATER THAN 36 HOURS AFTER THE FINISHING OF THE SLAB SURFACE HAS BEEN COMPLETED.

2. SAWCUT JOINTS SHALL BE MADE AT ALL CONSTRUCTION JOINTS. JOINTS MAY BE SAWCUT WHEN CONTROL JOINTS ARE SAWCUT OR AT ANY TIME PRIOR TO THE TIME THAT JOINT FILLERS ARE TO BE INSTALLED IN THE CONSTRUCTION JOINTS.

3. IMMEDIATELY AFTER SAWCUTTING, CLEAN THE JOINT AND SLAB SURFACE WITH A HIGH PRESSURE WATER BLASTER (1000 PSI MIN.). WATER BLASTING SHALL REMOVE ALL LAITANCE AND OTHER CONTAMINANTS FROM THE JOINT AND SLAB SURFACE.

4. CONTINUE CURING THE SLAB SURFACE AND JOINT AS SPECIFIED.

5. AFTER A MINIMUM OF 60 DAYS AFTER THE JOINTS ARE SAWCUT AND AFTER THE BUILDING SHELL IS COMPLETE, RE-CLEAN ALL JOINTS WITH A WATER BLASTER AND BLOW CLEAN WITH COMPRESSED AIR.

6. IMMEDIATELY AFTER BLOWING SURFACE DRY, PRIME THE SURFACES TO RECEIVE THE JOINT FILLER. USE OF BACKER ROD IS PROHIBITED. CONTRACTOR IS PERMITTED TO CHOKE-OFF THE SHRINKAGE CRACK AT THE BASE OF THE JOINT USING 1/4” MAXIMUM LAYER OF CLEAN, DRY SILICA SAND.

7. JOINT FILLER SHALL BE TWO-COMPONENT 100% SOLIDS EPOXY HAVING A MINIMUM SHORE-A HARDNESS OF 80 COMPLIANT WITH ASTM D2240. INSTALL JOINT FILLER IN STRICT ACCORDANCE WITH MANUFACTURER’S FULL WRITTEN INSTRUCTIONS.

8. JOINTS SHALL BE CHECKED PERIODICALLY TO ENSURE NO SEEPAGE OCCURS INTO SHRINKAGE CRACK, RESULTING IN LOW SPOTS. CROWNED MATERIAL SHALL BE ALLOWED TO SET FOR A TIME AS NOTED BY THE MANUFACTURER, AT WHICH POINT IT SHALL BE SHAVED OR GRINDED FLUSH TO THE FLOOR SURFACE.

Figure 8: Excerpt from contract structural drawings: example of notes outlining slab joint requirements. Note that the requirements for joint sealant/filler align with recommendations made in ACI 360 regarding Shore A hardness that is appropriate for wheel traffic.
CONCRETE SLAB-ON-GROUND

1. SLABS-ON-GROUND ARE TO BE OF A THICKNESS AND REINFORCEMENT AS DEFINED IN THE FOUNDATION PLAN NOTES AND ASSOCIATED SECTIONS, UNLESS OTHERWISE EXPLICITLY NOTED. SLAB REINFORCEMENT SHALL BE SUPPORTED ON CHAIRS AND/OR BOLSTERS SO THAT THE BARS/WIRES ARE POSITIONED 1” CLEAR FROM THE TOP OF 4-INCH THICK SLABS AND 1 ½” CLEAR FROM THE TOP OF 6-INCH THICK SLABS. REINFORCING BARS/WIRES SHALL BE SPACED AT 4” MINIMUM / 18” MAXIMUM.

2. ALL SUBSURFACE AND SITE PREPARATION REQUIREMENTS ASSOCIATED WITH SUPPORT OF SLABS-ON-GROUND DETAILED HEREIN SHALL BE IN ACCORDANCE WITH THE RECOMMENDATIONS AND REQUIREMENTS NOTED IN THE APPLICABLE PROJECT GEOTECHNICAL REPORT. SLABS-ON-GROUND ARE DESIGNED BASED ON THE PRESENCE OF UNIFORM SOIL SUPPORT THROUGHOUT THE SLAB AREA, AS REPRESENTED BY A MODULUS OF SUBGRADE REACTION EQUAL TO 150 LB/IN² PER INCH (PCI) OF SUBSURFACE DEPTH.

3. CONCRETE FOR SLABS-ON-GROUND SHALL BE PLACED IN A SEQUENCE AND MANNER THAT IS CONSISTENT WITH THE RECOMMENDATIONS OF THE AMERICAN CONCRETE INSTITUTE.

4. SLABS-ON-GROUND REQUIRE CONTROL JOINTS PER THE TYPICAL DETAILS UNLESS OTHERWISE NOTED. LOCATE CONSTRUCTION AND CONTROL JOINTS IN SUCH A WAY TO MINIMIZE THE EFFECTS OF SHRINKAGE AND CURLING OF THE CONCRETE SLAB SECTIONS. SUBMIT TO THE ARCHITECT/ENGINEER THE SEQUENCE AND METHOD OF CASTING CONCRETE SLABS-ON-GROUND PRIOR TO PLACING THESE ELEMENTS.

5. JOINT SPACING SHALL NOT EXCEED 15 FEET. AND THE RESULTING PLAN ASPECT RATIO (PANEL LENGTH TO PANEL WIDTH) OF ANY GIVEN SECTION OF SLAB SHALL NOT EXCEED 1.5.

6. SEE THE TYPICAL DETAILS ON THE DRAWINGS FOR JOINT AND COLUMN ISOLATION BLOCKOUT CONSTRUCTION. WHERE POSSIBLE, CONTROL JOINT LAYOUT SHALL BE DESIGNED SUCH THAT JOINTS ALIGN WITH THE VERTICES OF COLUMN ISOLATION BLOCKOUTS AND SHALL BE ARRANGED IN A GENERALLY ORTHOGONAL MANNER. JOINT SKEW BETWEEN ISOLATION BLOCKOUT VERTICES RESULTING FROM OFFSET IN PLAN ALIGNMENT OF COLUMNS IS PERMITTED PROVIDED THE SKEW COMPLIES WITH THE TYPICAL SKEWED CONTROL JOINT DETAIL.

7. IN AREAS WHERE NON-STRUCTURAL BONDED OVERLAY MATERIAL OR GROUT-BEDDED FLOORING ARE TO BE INSTALLED, FEATURES SUCH AS CRACK-SUPPRESSION MEMBRANES AND FLOORING DIVIDER STRIPS ALIGNED WITH SLAB CONTROL JOINTS SHALL BE DESIGNED AND INSTALLED BY OTHERS AS PART OF THE NON-STRUCTURAL FLOORING ASSEMBLY IN ORDER TO MITIGATE SUBSTRATE-INDUCED CRACKING.

8. WHERE SLAB-ON-GROUND SURFACES ARE REQUIRED TO SLOPE, THE MINIMUM SLAB THICKNESS SHALL BE MAINTAINED THROUGHOUT THE EXTENT OF THE SLOPED REGION.

9. REFER TO CAST-IN-PLACE CONCRETE GENERAL NOTES FOR OTHER SLAB REQUIREMENTS.

Figure 9: Excerpt from contract structural drawings: example of notes outlining slab-on-ground requirements
Author Statement:

Per ACI 318-19 Section 1.4.8, detailed recommendations for design and construction of slabs-on-ground and floors that do not transmit vertical loads or lateral forces from other portions of the structure to the soil are given in ACI 360R-10. This example incorporates ACI 360 recommendations in numerous instances as a guiding standard for the slab types in question.

It cannot be overstated the importance of pre-design coordination between the design team and the building owner for the purposes of establishing not only the applicable design criteria for the slabs, but also the functional and aesthetic expectations for the building that are related to the performance of the slabs. It is imperative that the project’s geotechnical engineer be included in pre-design coordination discussions in order to have reliable recommendations for subgrade and base material preparation defined.

It should be understood that even with the most stringently designed and constructed high performance slabs-on ground, there is the potential for visible cracks to form. The design professional may be doing the owner a disservice and putting themself in a position of considerable liability exposure by suggesting or guaranteeing that a slab will be free or nearly free of visible cracking for its entire service life. It is strongly suggested that candid conversations take place early and often regarding the criticality of a slab-on-ground’s performance in light of the subject building’s intended use, and that a commensurate discussion take place regarding the potential – often likely - presence of visible cracks not necessarily being an indicator that the slab was poorly designed and/or constructed.

Geotechnical engineers, slab design engineers, contractors, and owners all play a role in how closely a constructed slab-on-ground conforms to its originally intended purpose. The success of the slab-on-ground component of a building project is a shared responsibility.

For all of the extremely valuable information and insights made available to the designer in ACI 360-10, like other publications – and for a plethora of justified reasons - it tends to lack a formal, definitive, and quantitative correlation between acceptable slab cracking and specific steel reinforcement area required to achieve acceptable crack control. As a result, engineers often make the mistake of relying upon ACI 318-19 as a reference for establishing required amounts of reinforcement to resist shrinkage and temperature effects in slabs-on-ground. It is well documented that such an approach can result in unacceptably large, visually unappealing crack widths given in part that ACI 318 is based on a mobilization of reinforcement corresponding to high steel reinforcement stresses. This is not to say that all instances of use of ACI 318 by engineers in prescribing slab-on-ground reinforcement requirements have resulted in failures or unacceptable performance; rather, it is more a statement on the importance of an engineer’s understanding of the limitations of the ACI 318 design and prescriptive methodologies on concrete cracking not being in alignment with – nor specifically calibrated for – typical slab-on-ground construction.

Beyond ACI standards, the Wire Reinforcement Institute’s own TF-705 “Innovative Ways to Reinforce Slabs-on-Ground” offers several methods for deriving steel areas, but there is a broad variation in the results depending on the method selected therein. This lends credence to the notion that, unlike design of flexural and shear reinforcement for, say, a structural beam designed per ACI 318, there really is not a “one size fits all” approach to slab-on-ground reinforcement design.

For this example, prescriptions for minimum reinforcement defined in ACI 360-10 are adopted where deemed appropriate and ultimately supplemented by engineering judgment for the calculation and selection of other reinforcement attributes.
### Chapter 4

#### Section 1.4.3 and professional engineering judgment

- Geotechnical engineering report provided
- Subsurface preparation recommendations defined
- Modulus of subgrade reaction = 150 pounds/in² per inch vertical depth

In lieu of using simplified chart or graph-based methods by PCI, WRI, and/or COE, the variability of the loading magnitude and location incurred by the slabs in combination with perimeter slab restraint conditions prompted the engineer to model the slabs using FEA, assuming meshed plate supported by an elastic subgrade.

The Subgrade is represented by linear compression-only springs, with control joints occurring along adjoining plate edges accounted for in the form of release of plate bending fixity (rotation free, translation pinned condition). Spring constants in terms of kips per vertical inch are derived directly from the modulus of subgrade reaction provided by the geotechnical engineer and the tributary area of slab associated with the springs based on the spring layout.

Because of the perimeter building condition being characterized by a termination of the slab-on-grade over the top of the building’s foundation stem wall and shallow foundation system, a continuous restraint against vertical translation has been imposed around the modeled slab’s perimeter. The effects of this restraint are captured in the FEA model’s plate stress results.

### Description

Close coordination between the structural engineer, geotechnical engineer, and building owner has taken place regarding the building’s use in conjunction with functional and aesthetic expectations. As part of this coordination, loading and location of pallets, rack systems, manual pallet jack travel, and forklift travel have been established and agreed upon.

Facility use and proposed floor elevations, anticipated loads, environmental conditions of the building space, floor-levelness and flatness criteria, and floor covering requirements have been provided to the geotechnical engineer and recommendations for subgrade and base material preparation have been provided to the structural engineer.

Based on function, loading, and aesthetics, the structural engineer, owner, and building architect have agreed upon the following:

1. Slab Type 1 is permitted to have contraction (control) joints at a spacing not to exceed 15'-0”.
2. Slab Type 2 shall be free of control joints.

The structural engineer uses Finite Element Analysis (FEA) modeling to determine service stress levels due to applied loads from pallets, racks, and vehicles (hand-operated pallet jacks and powered-forklifts).
### ACI 360-10 Calculations Description

**STEP 1: Engineer’s Design Methodology (continued)**

<table>
<thead>
<tr>
<th>ACI 360-10</th>
<th>Calculations</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Section 3.2.2</strong>&lt;br&gt;<strong>Section 7.1</strong>&lt;br&gt;<strong>Table 5.2</strong></td>
<td>Slabs thicknesses are derived based on uncracked section analysis, using the modulus of rupture of concrete with a factor of safety equal to 2.0 and checked against service-level bending stresses that result from the applied pallet, rack, and vehicle loading.</td>
<td></td>
</tr>
<tr>
<td><strong>Section 6.2</strong>&lt;br&gt;<strong>Figure 6.6</strong>&lt;br&gt;and <strong>professional engineering judgment</strong></td>
<td>The slabs will be provided with reinforcement as a crack control measure.</td>
<td></td>
</tr>
<tr>
<td><strong>Table 3.1</strong>&lt;br&gt;<strong>Section 8.3</strong></td>
<td>For Slab Type 1, joints are relied upon as the primary method of managing shrinkage and curling. Joint spacing will be kept to a maximum of 15'-0&quot;, and will be supplemented by a small percentage (0.1% of the slab cross-sectional area) of deformed reinforcement extended through the sawcut contraction joints to achieve enhanced aggregate interlock for load transfer at the joints without excessive restriction to activation of the joint. The field of the slab “panels” will then be provided with nominal reinforcement placed in the upper one-third of the slab depth (reinforcement area derived based on engineer’s judgment and anticipated maximum curling stress at mid-panel).</td>
<td></td>
</tr>
<tr>
<td></td>
<td>For Slab Type 2, no control joints will be used, and the slab will be designed as continuously reinforced using a reinforcement with area equal to 0.50% of the slab cross-sectional area (again, an ACI 360 recommendation in combination with engineer’s judgment with due consideration for use of the space, durability, and shrinkage and curling).</td>
<td></td>
</tr>
</tbody>
</table>
### Section 1.4.3

**Overall plan view of plate model (plate sub-meshing not shown):**

![Overall plan view of plate model](image)

**A representative illustration of an external applied load case:**

![A representative illustration of an external applied load case](image)
### ACI 360-10 Calculations

<table>
<thead>
<tr>
<th>Section 1.4.3</th>
<th><strong>FEA plate model - flexural contours (north-south):</strong></th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><img src="image1.png" alt="Image" /></td>
<td>Service-level load combinations are generated and summarized.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Section 1.4.3</th>
<th><strong>FEA plate model - flexural contours (east-west):</strong></th>
<th></th>
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<tr>
<td></td>
<td><img src="image2.png" alt="Image" /></td>
<td></td>
</tr>
</tbody>
</table>

**Moment X**
-0.203
-0.235
-0.177
-0.119
-0.081
-0.033
-0.055
-0.113
-0.171
-0.229
-0.287

**Moment Z**
-0.283
-0.226
-0.169
-0.112
-0.055
-0.002
-0.059
-0.110
-0.173
-0.23
-0.287
### Slab thickness Design

#### Slab Type 1: 4” thickness

\[
f_b = \frac{M_{\text{service}}}{S_{\text{uncracked}}} = \frac{0.287 \times 12}{12 \times 4^2} = 0.108 \text{ ksi}
\]

\[
F_b = \frac{f_r}{F.S} = \frac{7.5 \lambda \sqrt{f_c}}{2.0} = \frac{7.5 \times 1.0 \times \sqrt{4000}}{2.0} = 0.237 \text{ ksi}
\]

\[f_b < F_b \therefore OK\]

#### Slab Type 2: 6” thickness

\[
f_b = \frac{M_{\text{service}}}{S_{\text{uncracked}}} = \frac{0.293 \times 12}{12 \times 6^2} = 0.049 \text{ ksi}
\]

\[
F_b = \frac{f_r}{F.S} = \frac{7.5 \lambda \sqrt{f_c}}{2.0} = \frac{7.5 \times 1.0 \times \sqrt{4000}}{2.0} = 0.237 \text{ ksi}
\]

\[f_b < F_b \therefore OK\]

Uncracked section analysis is used for determination of slab thicknesses, with the maximum allowable flexural stress being equal to the concrete’s modulus of rupture “reduced by” a factor of safety. Flexural demand is based on service-level (unfactored) stresses calculated from the reported output of the structural model and resulting from applied external loading.

Satisfactory slab-on-ground thicknesses are derived based on the premise that applied external loading does not impose a demand that exceeds a modulus of rupture with a safety factor incorporated.
### Designer's project-specific reinforcement design

**Slab Type 1: curling and shrinkage reinforcement**

Curling stresses for slabs configured with 15-foot joint spacing are assumed to not exceed 300 psi for this example*, with the highest magnitude occurring in the central “backspan” region of a slab panel, away from the slab edges and corners that are lifting upward. Slab curling can be idealized conceptually as a flexural tension force distributed near the slab’s upper surface.

Assuming a worst-case scenario in which the effects of curling are cumulative with the effects of previously calculated stresses that result from applied pallet and vehicle loading, it is worthwhile to mathematically check to see if there remains available resistance from the uncracked section under quasi-ultimate loading:

**Check A:**

\[
300 \text{ psi} + 108 \text{ psi} = 408 \text{ psi} < 474 \text{ psi modulus of rupture, unreduced} \\
\therefore \text{ previously calculated 4" thick slab proportioning acceptable}
\]

**Check B:**

For determination of area of steel reinforcement required for this example slab, the engineer carries out a check predicated on an idealized linear curling stress distribution, the neutral axis of which is eccentric to the slab’s gross-section’s centroid, resulting in a bending moment. This bending moment then serves as the basis for a nominal amount of quasi-flexural steel reinforcement at a maximum service-level stress of 24,000 psi.

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<th><strong>Calculations</strong></th>
<th><strong>Description</strong></th>
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<tr>
<td><strong>STEP 1: Engineer’s Design Methodology (continued)</strong></td>
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</table>

*Designer’s project-specific reinforcement design (continued)*

**Slab Type 1: curling and shrinkage reinforcement (continued)**

**Check B (continued):**

![Diagram](image)

\[
M = 300 \frac{lb}{in^2} \times 4'' \times 12'' \times \left(\frac{4''}{2} - \frac{4''}{3}\right) = 4,800 \text{ lb} - \text{in, per linear ft}
\]

\[
M = f_s \times A_s \times j d
\]

*Set* \( f_s = 24,000 \text{ psi} \)

*Set* \( jd = 0.75h \)

\[
A_s = \frac{4,800}{24,000 \times 0.75 \times 4''} = \textcolor{blue}{0.067 \text{ in}^2 \text{ per linear foot}} > 0.001bh
\]

A lower bound steel area equal to the ACI 360 recommendation for enhanced aggregate interlock reinforcement is exceeded. As such, reinforcement in the field of the slab panels bounded by control joints will be furnished separately from the reinforcement ladders used for enhancing aggregate interlock.

Theoretically, if the resistance available through uncracked section behavior is exceeded, cracking will result, at which point there is a fundamental change in how capacity is calculated. Cracked section behavior is predicated on a linear strain relationship and strain compatibility between a compressed concrete section and an equal but opposite tensile force that combine to create a resisting flexural couple.

The simplification shown here is an approach intended to yield an amount of steel reinforcement capable of resisting flexural-type tension force calculated at service level and based on an idealized triangular stress distribution. Note that, not unlike other methods used to calculate steel reinforcement for crack control, no direct correlation is made between resulting crack size and corresponding steel area.
### Designer's project-specific reinforcement design (continued)

### Slab Type 2: curling and shrinkage reinforcement

Curling stresses can be minimized if slabs are properly designed and constructed with continuous reinforcement and no contraction joints. At the same time, special attention should still be paid to the terminal edges of jointless slabs if these edges are located in areas subjected to load cycling due to vehicular / mechanized traffic.

For the subject building, curling is reasonably restrained along the plan north and east building walls as a result of the slab's termination being integrated into the vertical construction. At the south and west edges, however, curling may still occur due to an absence of restraint, and there is a possibility that occasional incidental cross-traffic between the Slab Type 1 and Slab Type 2 regions could create exacerbating effects.

A curling stress of 400 psi is assumed in the design of Slab Type 2 and is based on its 30-foot dimension.

Assuming a worst-case scenario in which the effects of curling are cumulative with the effects of previously calculated stresses that result from applied pallet and vehicle loading, it is worthwhile to mathematically check to see if there remains available resistance from the uncracked section under quasi-ultimate loading:

#### Check A:

\[
400 \text{ psi} + 49 \text{ psi} = 449 \text{ psi} < 474 \text{ psi} \text{ modulus of rupture, unreduced} \\
\therefore \text{ previously calculated 6" thick slab proportioning acceptable}
\]
## ACI 360-10 Calculations

### Description

#### STEP 1: Engineer’s Design Methodology (continued)

**Designer’s project-specific reinforcement design (continued)**

**Slab Type 2: curling and shrinkage reinforcement (continued)**

**Check B:**

\[
M = 400 \frac{lb}{in^2} \times 6^\circ \times 12^\circ \times \frac{1}{2} \times \left( \frac{6^\circ}{2} - \frac{6^\circ}{3} \right) = 14,400 \text{ lb-in, per linear ft}
\]

\[
M = f_s \times A_s \times jd
\]

*Set \( f_s = 24,000 \text{ psi} \)*

*Set \( jd = 0.75h \)*

\[
A_s = \frac{14,400}{24,000 \times 0.75 \times 6''} = 0.133 \text{ in}^2 \text{ per linear foot} < 0.005bh
\]

A lower bound steel area equal to the ACI 360 recommendation for minimum reinforcement of continuously-reinforced jointless slabs is not satisfied. As such, reinforcement cross-sectional area equal to \(0.005 \times 12 \times 6 = 0.36 \text{ in}^2 \text{ per linear foot width of slab shall be used.}\)
## STEP 2: Comparison to WRI TF-705 Methods

<table>
<thead>
<tr>
<th>Procedure (use 70 ksi reinforcement)</th>
<th>Slab Type 1 Steel Area</th>
<th>Slab Type 2 Steel Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design &amp; Detailing Guide</td>
<td>0.067 in²</td>
<td>0.133 in²</td>
</tr>
<tr>
<td>Design &amp; Detailing Guide + ACI 360 prescriptions*</td>
<td>0.067 in²/@ joint 0.048 in² @ joint</td>
<td>0.360 in²</td>
</tr>
<tr>
<td>Subgrade Drag</td>
<td>0.014 in²</td>
<td>0.107 in²</td>
</tr>
<tr>
<td>Confirmed Capacity</td>
<td>0.052 in²</td>
<td>0.079 in²</td>
</tr>
<tr>
<td>Temperature**</td>
<td>0.096 in²</td>
<td>0.144 in²</td>
</tr>
<tr>
<td>Equivalent Strength</td>
<td>0.173 in²</td>
<td>0.260 in²</td>
</tr>
<tr>
<td>Crack Restraint</td>
<td>0.535 in²</td>
<td>0.802 in²</td>
</tr>
</tbody>
</table>

* ACI 360 recommendations of 0.1% (enhanced aggregate interlock) and 0.5% minimum (continuously reinforced, jointless) incorporated.

** range of temperature to which slab is subjected: use 30 °F: 

\[ \text{CTE} = 6 \times 10^{-6}/\text{°F} \]

The Wire Reinforcement Institute’s TF-705 “Innovative Ways to Reinforce Slabs-on-Ground” Tech Fact contains five (5) methods for the determination of slab-on-ground reinforcement quantities. Results for these previously published methods are presented here as a comparison to the simplified and idealized method used in the WRI Design and Detailing Guide example.

It is not the intent of the WRI Design and Detailing Guide to marginalize or replace the methods found in TF-705. In the end, the determination of slab-on-ground reinforcement is carried out at the discretion of – and is the responsibility of – the slab design engineer, whether based on longstanding published methods or derived from proprietary-like procedures and empirical- or experience-based data.

**Author Note:**

The Subgrade Drag procedure was removed from ACI 360 starting with ACI 360-06, as there were indications that its use was resulting in insufficient slab reinforcement designs. TF-705 was published prior to the release of ACI 360-06, hence the procedure’s continued inclusion in that document. With this stated, designers should be aware of the procedure’s limited applicability and potentially insufficient resulting reinforcement requirements.
### STEP 3: Reinforcement Detailing

<table>
<thead>
<tr>
<th>Slab Type 1 Steel Area</th>
<th>Slab Type 2 Steel Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.067 in²/0.048 in² @ joint</td>
<td>0.360 in²</td>
</tr>
</tbody>
</table>

Three options are considered, two of which are comprised of welded deformed wire reinforcement that will be detailed herein.

**For field reinforcement:**  
Option A: D4.5 @ 8” on center (8x8 D4.5/D4.5) \( A_s = 0.068 \text{ in}^2 \)  
Option B: D9.0 @ 16” on center (16x16 D9.0/D9.0) \( A_s = 0.068 \text{ in}^2 \)  
Option C: #3 rebar @ 18” on center \( A_s = 0.073 \text{ in}^2 \)

**For joint reinforcement (enhanced aggregate interlock):**  
D4.8 @ 12” on center \( A_s = 0.048 \text{ in}^2 \)  
or  
#3 rebar @ 27.5” on center \( A_s = 0.048 \text{ in}^2 \)  

In the interest of time and labor savings on the jobsite, welded wire reinforcement solutions have long been selected as a more time-efficient option than placement of individual loose bars/wires. Welded wire reinforcement options will be detailed.

Refer to Figure 10 for the engineer’s Typical WWR Lap Splice Detail Refer to figure 11 for related excerpt from the engineer’s mild reinforcement general notes.

**Author Note:**  
It is important for the design professional / specifying engineer to understand that the steps shown here for determining overall mat geometries are most commonly carried out by the WWR manufacturer / detailer. As long as the engineer defines the basic reinforcement requirements (steel size/area, spacing, and terminations) there is no need for the designer to spend additional time on the matter. Mat geometries and layouts will be derived by the WWR detailer based on satisfying the engineer’s design-driven requirements in conjunction with the contractor’s preferred handling and placement criteria (means and methods). This information is then presented on the project’s reinforcement shop drawings for review.
Figure 10: Excerpt from contract structural drawings: Typical WWR Lap Splice Detail

**ACI 318-19 LAP SPLICE LENGTH:**

<table>
<thead>
<tr>
<th>WIRE SIZE</th>
<th>LAP SPLICE</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ D7.0</td>
<td>12 INCHES</td>
</tr>
<tr>
<td>≤ D11.0</td>
<td>16 INCHES</td>
</tr>
<tr>
<td>≤ D16.0</td>
<td>18 INCHES</td>
</tr>
<tr>
<td>≤ D20.0</td>
<td>22 INCHES</td>
</tr>
<tr>
<td>≤ D31.0</td>
<td>30 INCHES</td>
</tr>
</tbody>
</table>
THROUGH MILL REPORT DOCUMENTATION PROVIDED AS PART OF THE PROJECT REINFORCEMENT SUBMITTAL.

3. UNLESS OTHERWISE NOTED ON THE DRAWINGS OR IN THE PROJECT SPECIFICATIONS, LAP CONTINUOUS WALL FOOTING BARS 24” MINIMUM INTO SPREAD FOOTINGS.

4. WELDED DEFORMED WIRE REINFORCEMENT SHALL CONFORM TO ASTM A1064 (GRADE 70) AND SHALL BE PROVIDED IN SHEET FORM, UNLESS OTHERWISE NOTED. REINFORCEMENT SHEETS SHALL BE MANUFACTURED WITH OVERHANG LENGTHS SUFFICIENT TO ACHIEVE A LAP SPLICE LENGTH EQUAL TO THAT NOTED IN THE TYPICAL WWR LAP SPLICE DETAIL, UNLESS OTHERWISE NOTED ELSEWHERE IN DETAILS. SHEETS AND ASSOCIATED LAP REGIONS SHALL BE INSTALLED COPLANAR SO AS TO NOT “STACK”.

5. MANUFACTURER OVER-STEELING OF WELDED WIRE REINFORCEMENT IS NOT PERMITTED. ORDERED WIRE SIZES SHALL MATCH THOSE SPECIFIED BY THE STRUCTURAL ENGINEER OF RECORD.

6. ALL REINFORCING STEEL SHALL BE SECURELY TIED AND ANCHORED IN PLACE TO MAINTAIN SPECIFIED CONCRETE COVER AND TO PREVENT DISLOCATION DURING THE PLACING OPERATION. ALL BOLSTERS AND CHAIRS SHALL BE NON-CORROSIVE AND NON-REACTIVE MATERIAL COMPATIBLE WITH CAST-IN-PLACE CONCRETE.

7. REINFORCING STEEL SHALL BE CLEAN OF MUD, DEBRIS, LOOSE RUST, CEMENT GROUT, OR ANY OTHER MATERIAL WHICH MAY INHIBIT BOND BETWEEN THE STEEL AND CONCRETE.

Figure 11: Excerpt from contract structural drawings: selection from mild reinforcement notes outlining lap splice requirements
**Slab Type 1 Steel Area**

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>0.067 in²/</td>
<td>0.048 in² @ joint</td>
</tr>
</tbody>
</table>

*For field reinforcement:*

Option A: D4.5 @ 8" on center (8x8 D4.5/D4.5) → $A_s = 0.068$ in²

Option B: D9.0 @ 16" on center (16x16 D9.0/D9.0) → $A_s = 0.068$ in²

Slab area, per “panel” bounded by control joints and/or slab terminations at building perimeter:

15'-0" x 15'-0" typical slab panel area

Subtract 2" around perimeter to maintain specified perimeter cover and to terminate mats clear of control joints:

**14'-8" x 14'-8" reinforcement “coverage” area**

See Figures 12 and 13 for slab panel reinforcement using Option A.

See Figures 14 and 15 for slab panel reinforcement using Option B.
Figure 12: Slab Type 1, Option A: WWR mat configuration shown. For each 15 ft x 15 ft slab “panel” there will be a “left” and “right” mat. Note how mat central overhangs are configured to achieve the lap splice requirement while maintaining a typical wire spacing at the transition from one mat to the other.
Figure 13: Slab Type 1, Option A: WWR mats “in panel”. Dimensions between the concrete panel boundaries (at control joints and slab terminations) and steel wires are illustrated.

Notice that there is slight projection of short wire overhangs (ends) beyond the 2” offset distance that was established for definition of reinforcement coverage area, resulting in a modest encroachment outside of the previously defined coverage area. These short overhangs are inherent to manufacture of WWR mats and are a function of the welding equipment programming for traction (pulling) of wires through the machine itself, as well as execution of the automatic welding operation. ASTM A1064 provides for special requirements that, if requested by the designer and/or purchaser, would in turn allow these small overhangs to be minimized/eliminated by the manufacturer during fabrication, prior to shipment.
Figure 14: Slab Type 1, Option B: WWR mat configuration shown. For each 15 ft x 15 ft slab “panel” there will be two identical mats, one of which will simply be rotated end for end in the field.

Note how mat central overhangs are configured to achieve the lap splice requirement while maintaining a typical wire spacing at the transition from one mat to the other.
Figure 15: Slab Type 1, Option B: WWR mats “in panel”. Dimensions between the concrete panel boundaries (at control joints and slab terminations) and steel wires are illustrated. See Figure 13 for additional information on small overhang projections around perimeter.
**ACI 360-10 Calculations**  
**Description**

<table>
<thead>
<tr>
<th><strong>STEP 3: Reinforcement Detailing (continued)</strong></th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Slab Type 1 Steel Area</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.067 in²/0.048 in² @ joint</td>
<td>Enhanced aggregate interlock is achieved by introducing separate, purpose-specific mats that are discontinuous from – and in this example, of lesser cross-sectional area than - those used in the field of the slab panel.</td>
</tr>
</tbody>
</table>

*For joint reinforcement (enhanced aggregate interlock):*

D4.8 @ 12” on center → A_s = 0.048 in²

The WWR mat detailed as the “interlock mat” noted in Figure 6 will be a ladder configuration. See Figure 16.

The steel area recommended to achieve enhanced aggregate interlock in conjunction with correctly-spaced control joints is outlined in ACI 360.
Figure 16: Slab Type 1, joint reinforcement. Ladder-type mats are used across the control joints to promote enhanced aggregate interlock per ACI 360. The W3.0 wires are simply for supporting the "structural" D4.8 wires and are not relied upon for anything other than maintaining the shape of the mat and contributing to the reinforcement placement operation itself.

These ladder mats, along with the previously detailed field mats for Slab Type 1, are easily trimmable on the jobsite to avoid encroachment into the column isolation blockouts.
**ACI 360-10 Calculations**

**Description**

**STEP 3: Reinforcement Detailing (continued)**

<table>
<thead>
<tr>
<th>Slab Type 2 Steel Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.360 in²</td>
</tr>
</tbody>
</table>

For Slab Type 2

\[ A_s = 0.360 \text{ in}^2 \]

Options:

- D30.0 @ 10” on center \( \rightarrow A_s = 0.360 \text{ in}^2 \)
- D24.0 @ 8” on center \( \rightarrow A_s = 0.360 \text{ in}^2 \)
- D18.0 @ 6” on center \( \rightarrow A_s = 0.360 \text{ in}^2 \)

For overall project uniformity, the 8” spacing option is selected.

Use D24.0 @ 8” on center, each direction 30” lap splice required* per structural engineer (Figure 10)

Slab area bounded by doweled joint at interior edges and slab terminations at building perimeter:

75'-0” x 60'-0” slab area

Subtract 2" around perimeter to maintain specified perimeter cover and to terminate mats clear of control joints:

74’-8” x 59’-8” reinforcement “coverage” area

*A pre-defined, prescriptive range of lap splice lengths has been provided on the contract drawings. This type of simplification is a common practice when the design professional wants to maximize the repeatability of a particular detail or schedule from one project to another, as their design fees are often tied back to and checked against the accrual of chargeable hours. As such, any opportunities to streamline design tasks for the sake of efficiency are taken advantage of and implemented. In this case, if the WWR fabricator were to inspect more closely the lap splice length requirements per ACI 318-19 (Section 25.4.2.4 in conjunction with Section 25.5.3.1.1 and Section 25.5.2), they would find that the actual require Class B lap splice is just under 22”. See Figure 17.

WWR manufacturers’ technical staff are encouraged to broach the subject of refined lap splicing as well as any other potential material and cost savings measures with both the contractor and engineer of record. With the implementation of WWR into a project itself typically being a value-based solution (time and labor savings), it is important for there to be a collaborative approach to striking the ideal balance between design intent, constructability, and installed cost.

For this example, the engineer’s specified lap splice will be used in the WWR detailing.
Figure 17: Automated calculation of Class B lap splice length using ACI 318-19 Sections 25.4.2.4, 25.5.3.1.1 and 25.5.2. Note that the reinforcement grade modification factor is conservatively listed as 1.15, which adopts the same value as that established in ACI 318-19 for 80 ksi reinforcement instead of a 1.0 value associated with 60 ksi. Future editions of ACI 318 will be expected to provide clarity on this matter.
**Slab Type 2 Steel Area**

| 0.360 in² |

**74’-8” (plan east-west) x 59’-8” (plan north-south) “coverage” area**

Plan east-west mat width/quantity determination (896 inches coverage):

- **Try ten** (10) intermediate 8” spaces between mats (80 inches)
- **Remaining width for eleven** (11) mats: 896 – 80 = 816 inches
- **816” is divisible by 8”; therefore, a solution exists without atypical spaces** (i.e., atypical = wire spaces not equal to 8”)
- **816 ÷ 11 = eleven mats, each 74.18 inch wide. But avoid atypical spaces.**
  - Option EW-1: (10) mats @ 72” + (1) mat @ 96”
  - Option EW-2: (10) mats @ 80” + (1) mat @ 16”

- **Try eleven** (11) intermediate 8” spaces between mats (88 inches)
- **Remaining width for twelve** (12) mats: 896 – 88 = 808 inches
- **808” is divisible by 8”; therefore, a solution exists without atypical spaces** (i.e., atypical = wire spaces not equal to 8”)
- **808 ÷ 12 = twelve mats, each 67.33 inch wide. But avoid atypical spaces.**
  - Option EW-3: (11) mats @ 72” + (1) mat @ 16”
  - Option EW-4: (11) mats @ 64” + (1) mat @ 104”

- **Try twelve** (12) intermediate 8” spaces between mats (96 inches)
- **Remaining width for thirteen** (13) mats: 896 – 96 = 800 inches
- **800” is divisible by 8”; therefore, a solution exists without atypical spaces** (i.e., atypical = wire spaces not equal to 8”)
- **800 ÷ 13 = thirteen mats, each 61.50 inch wide. But avoid atypical spaces.**
  - Option EW-5: (12) mats @ 64” + (1) mat @ 32”
  - Option EW-6: (12) mats @ 56” + (1) mat @ 128”

Coverage areas must be subdivided into WWR mat arrangements. WWR detailers have expedited methodologies for deriving the most appropriate “roster” of WWR mats types to suit project-specified coverage areas. A detailed view of one such method is shown here, presented in longhand form for clarity.
### ACI 360-10 Calculations Description

#### STEP 3: Reinforcement Detailing (continued)

<table>
<thead>
<tr>
<th>Slab Type 2 Steel Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.360 in²</td>
</tr>
</tbody>
</table>

74'-8" (plan east-west) x **59'-8" (plan north-south)** "coverage" area

Plan north-south mat width/quantity determination (716 inches coverage):

- Try one (1) intermediate 8" space between mats (8 inches)
- Remaining length for two (2) mats: 716– 8 = 708 inches
- 708" is not divisible by 8"; therefore, mats would be configured with an atypical space – not exceeding 8" increments – to provide the required coverage
- 708 ÷ 2 = two mats, each 354 inch long
  - Option NS-1: (2) mats @ 354" (each with a 5" space each end, with remaining spaces at 8")

- Try two (2) intermediate 8" spaces between mats (16 inches)
- Remaining length for three (3) mats: 716– 16 = 700 inches
- 700" is not divisible by 8"; therefore, mats would be configured with an atypical space – not exceeding 8" increments – to provide the required coverage
- 700 ÷ 3 = three mats, each 233.33-inch-long (avoid fractional spaces)
  - Option NS-2: (2) mats @ 240" + (1) mat @ 220" (6" space each end, with remaining spaces at 8")

The WWR detailer, with guidance from the contractor on installation preferences, means, and methods, will select the mat geometries and proposed plan placement and in turn presents this information on the shop drawing submittal that will ultimately be reviewed by the engineer for general conformance with the design intent. Shop drawing submittals must always first pass through the contractor, however, to ensure the proposed reinforcement attributes align with the contractor’s means, methods, and constructability considerations.

Refer to the following figures for the culmination of the WWR detailing effort:

- Figure 18 – WWR detailer’s mat plan nomenclature
- Figure 19 – WWR Solution: Option A
- Figure 20, 21 – WWR Solution: Option A Detailing
- Figure 22 – WWR Solution: Option B
- Figure 23 – WWR Solution: Option B Detailing

WWR Option A is a combination of EW-5 and NS-2. WWR Option B is a combination of EW-1 and NS-1.
**Figure 18:** WWR detailing nomenclature – For this example, the WWR detailer utilizes a compass-type system that helps the reviewer quickly identify on the placement plan the presence and general extent of wire overhangs.

The presence of overhangs is represented by an arrow. Where overhangs extend past the first wire of the adjacently-placed WWR mat, the arrow is shown in the symbol to be beyond a representative crossing line. Where overhangs extend to and terminate at the first wire of the adjacently-placed WWR mat, the arrow is shown in the symbol to abut the crossing line.

The absence of overhangs on a given mat is simply represented by the absence of a directional arrow. If the mat has no overhangs on any of its four sides (i.e., wires project beyond the mat’s perimeter wire centerlines by no more than one inch), the symbol is a rectangle with an inscribed “X”.

Figure 19: WWR Solution: Option A
Note that overhang dimensions are presented on the individual WWR mat details in Figures 20 and 21. Overhang lengths are configured in conjunction with the typical 8” space between mats to ensure the specified Class B lap splice is achieved.
Figure 20:
WWR Solution: Option A
Mat configurations 1-3

WWR OPTION A: MAT 1 (WEIGHT = 358.3 LBS)
8X8 D24/D24 64" (+0.5", +30") X 20'-9" (8", 1")

WWR OPTION A: MAT 2 (WEIGHT = 374.6 LBS)
8X8 D24/D24 64" (+8", +30") X 20'-9" (8", 1")

WWR OPTION A: MAT 3 (172.9 LBS)
8X8 D24/D24 32" (+8", +0.5") X 20'-9" (8", 1")
Figure 21:
WWR Solution:
Option A
Mat configurations 4-6

WWR OPTION A: MAT 4 (WEIGHT = 357.9 LBS)
8XV D24/D24 64" (+6.5",+30") X 23'-4" (30",30")
V = 30° OH, 6", 26 @ 8", 6", 30° OH

WWR OPTION A: MAT 5 (WEIGHT = 372.8 LBS)
8X8 D24/D24 64" (+8",+30") X 23'-4" (30",30")
V = 30° OH, 6", 26 @ 8", 6", 30° OH

WWR OPTION A: MAT 6 (WEIGHT = 175.2 LBS)
8X8 D24/D24 32" (+8",+0.5") X 23'-4" (30",30")
V = 30° OH, 6", 26 @ 8", 6", 30° OH
Figure 22: WWR Solution: Option B
Note that overhang dimensions are presented on the individual WWR mat details in Figure 23. Overhang lengths are configured in conjunction with the typical 8” space between mats to ensure the specified Class B lap splice is achieved.
Figure 23:
WWR Solution: Option B
Mat configurations 1-3

WWR OPTION B: MAT 1 (WEIGHT = 575.4 LBS)
8XV D24/D24 72" (+0.5";+30") X 31;2" (19";1")
V = 19" OH, 5", 43 @ 8", 5", 1" OH

WWR OPTION B: MAT 2 (WEIGHT = 598.9 LBS)
8XV D24/D24 72" (+8";+30") X 31;2" (19";1")
V = 19" OH, 5", 43 @ 8", 5", 1" OH

WWR OPTION B: MAT 3 (WEIGHT = 658.0 LBS)
8XV D24/D24 96" (+8";+0.5") X 31;2" (19";1")
V = 19" OH, 5", 43 @ 8", 5", 1" OH
The following is a comparison of Option A and B, outlining considerations relevant to the contractor’s handling and placement operations.

<table>
<thead>
<tr>
<th>Attribute</th>
<th>Option A</th>
<th>Option B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coverage area</td>
<td>~4,455 ft²</td>
<td>~4,455 ft²</td>
</tr>
<tr>
<td>Total number of mats</td>
<td>39</td>
<td>22</td>
</tr>
<tr>
<td>Average coverage per mat</td>
<td>~144.2 ft²</td>
<td>202.5 ft²</td>
</tr>
<tr>
<td>Unique mat types</td>
<td>6</td>
<td>3</td>
</tr>
<tr>
<td>Mat weight range</td>
<td>173 – 373 lbs</td>
<td>575 – 658 lbs</td>
</tr>
<tr>
<td>Mechanized installation</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>Total steel weight</td>
<td>13,937.50 lbs</td>
<td>13,247.50</td>
</tr>
<tr>
<td>Standard freight?</td>
<td>Yes</td>
<td>No</td>
</tr>
</tbody>
</table>

1. Mechanized assistance during installation likely required due to weight of individual WWR mats, typically through use of telehandler and spreader beam.
2. Standard width includes material that is 8'-6” in width. Wider loads require oversize considerations and additional permit.

Two viable WWR configurations are presented here for the Slab Type 2 area, each of which satisfies the engineer’s design intent. Ultimately, the selection of the option used for this project will depend heavily on the contractor’s transport, handling, and placement operations.

On many projects, mechanized equipment is on site by default due to the plethora of other construction activities that benefit from its availability, so the need for a telehandler or comparable means of mechanized installation is not necessarily a construction cost that is specific to WWR alone.

Contractors are encouraged to work closely with a WWR manufacturer’s technical staff to identify and maximize efficiencies of the WWR package specific to a project’s needs. All major manufacturers employ engineering and detailing professionals who are well-versed in the manufacture, submittal, and installation of welded wire reinforcement, and these services are inherently a value-added aspect of the reinforcement product.