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Welded Wire Reinforcement Design and Detailing Guide

Welded Wire Reinforcement Design and Detailing

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Welded Wire Reinforcement Design and Detailing Guide

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Chapter One

Introduction and Document Usage

1.1 Introduction

Clarity of communication is at the heart of any successfully delivered building construction project. Owners and developers, jurisdictional plan reviewers and inspectors, architectural and engineering practitioners, construction managers and general contractors, and subcontractors and vendors: each is a project stakeholder occupying a distinct role characterized by a unique set of proficiencies. The effectiveness with which these proficiencies are respectively deployed will have a profound impact on how closely the finished building resembles the vision upon which the whole undertaking was originally based. Quite simply, poor communication breeds unfulfilled expectations.

The Welded Wire Reinforcement Design and Detailing Guide's target audience is two-fold. The first audience includes the practicing structural engineer and affiliated drafting technician. Each of these two professionals recognizes the need to balance well-communicated structural detailing with constructability of the detailed configuration through a repeatable contract document preparation and delivery routine. The second audience is the construction contractor.

The Focus of this guide will be on welded **deformed** wire reinforcement (WWR) in building type applications only, deferring the specification of welded plain wire reinforcement to other texts. Our objective is to provide to the designer and contractor alike a comprehensive benchmark for the incorporation of WWR into contract documents serving as the basis for construction on site. Material and manufacture requirements for WWR are defined in ASTM A1064 *Standard Specification for Carbon-Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete*.

This document provides WWR knowledge without compromising the current structural detailing process familiarity and comfort. If there is a theme that will be repeated throughout this document, it is one predicated on eliminating the notion that (a) the structural Engineer-of-Record (EOR) needs proprietary-like knowledge in order to implement WWR into their contract drawings, and (b) the protocol-in-use for reinforced concrete design and detailing should somehow be overhauled in order to accommodate WWR usage.

The Welded Wire Reinforcement Design and Detailing Guide ("The Guide") is specifically geared towards detailing of cast-in-place reinforced concrete building-type structures and related appurtenances that are designed in accordance with the provisions of the American Concrete Institute's ACI 318-19 "*Building Code Requirements for Structural Concrete*" Standard and ACI 318R-19 "*Commentary on Building Code Requirements for Structural Concrete*". Note that for the sake of completeness, *The Guide* also includes ground-supported slabs designed using ACI 360-10 "*Guide to Design of Slabs-on-Ground*" as the primary reference.

1.2 Detailing Guide Format and Presentation

Design of cast-in-place building structures comprised of mild-reinforced structural concrete members and components is most commonly based on the use of individual deformed reinforcing bars ("rebar") arranged manually in the field to achieve a specified pattern or configuration. There exist numerous prevailing factors that make the use of rebar the standard by which other structural reinforcements are judged, the details of which are beyond the scope of this guide, but suffice it to say that rebar is deformed reinforcement in its most basic and spatially intuitive form, having a longstanding default presence in engineering academia and practice.

In laying the groundwork for any technically meaningful discussion centered on WWR, it is critically important to acknowledge the role rebar continues to play in the structural design and construction communities. In the case of *The Guide*, this acknowledgement shapes the formatting of its content. Because structural design professionals are historically inclined to present their design intent through the use of details configured using deformed reinforcing bars as a default, any comparative reinforcement solution utilizing WWR will, at least in principle, be based on this same detailing methodology. This is illustrated below in the form of a comparative example.

Assume a structural engineer designs and subsequently details a fixed-free cantilever retaining wall. The detailed design intent is shown in Figure 1-1, using reinforcing bars:

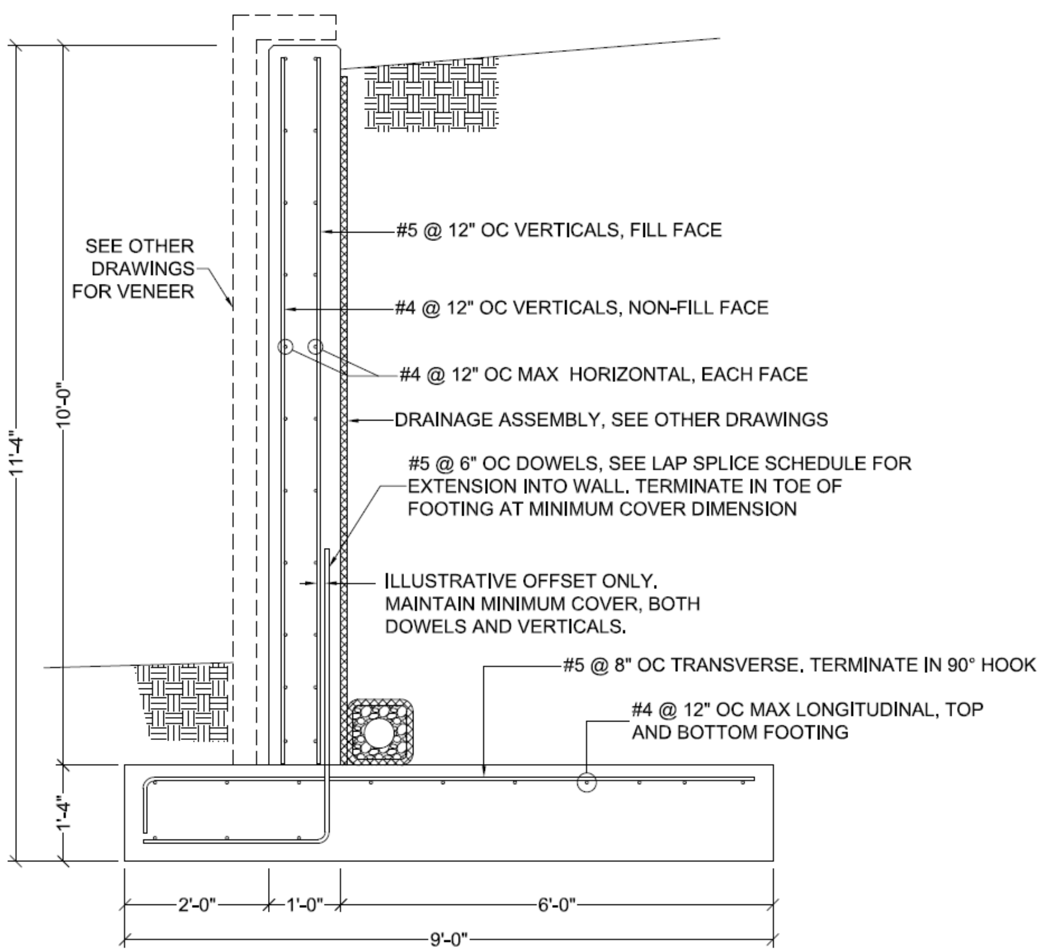


Figure 1-1: Cantilever Retaining Wall Section

What *The Guide* seeks to impress upon the structural engineering practitioner is that the leap required to go from a detail illustrating a rebar-based design to a detail invoking the usage of WWR is, in fact, not much of a leap at all, as shown in Figure 1-2 below:

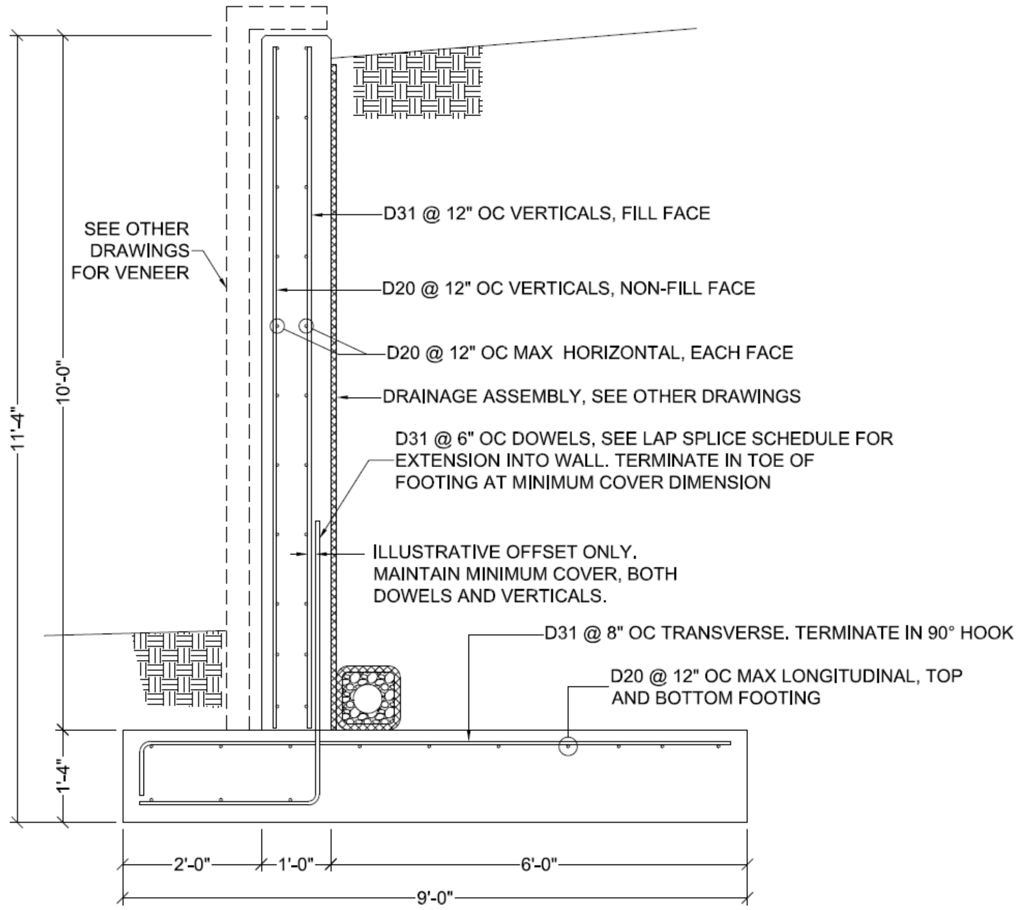


Figure 1-2: Cantilever Retaining Wall Section (WWR)

When comparing the two figures, it is apparent that no “proprietary” knowledge of WWR is required on the part of the design professional. Geometries and alignment originally shown for rebar are maintained, and reinforcement call-outs are simply swapped to reflect rudimentary wire size (#4 rebar → 0.20 in^2 → D20; #5 rebar → 0.31 in^2 → D31). Similarly, directions on reinforcement spacing remain unchanged. These are quite literally identical details from a design intent and structural performance standpoint, and from the perspective of the practicing engineer and affiliated detailer, the change in routine is miniscule. In Chapter 2 we will touch on ACI 318’s acceptance of WWR that serves as the basis for the interchangeability shown above.

Showing rudimentary WWR inclusion in a “familiar framework” is a fundamental goal of *The Guide*, whether it be in the form of “Direct Specification” (as shown in Figure 1-2) or as a “Pre-Approved Equal” (See Chapter 4 for more information), it is also critical that an illustrative light be shined on downstream WWR detailing efforts – carried out by highly capable WWR producers and fabricators – so that the engineering reader is not left feeling as if the specification of WWR on structural contract drawings puts them on an island without a knowledgeable counterpart. To accomplish this, *The Guide* will provide design and detailing examples comprised of both the initial design effort itself and accompanying WWR fabrication drawing and relevant placement information excerpts. The inclusion of fabrication and placement information is intended to provide educational assurance to the EOR that her/his structural information is, in fact, being fleshed out in WWR form - by WWR detailers – for subsequent shop drawing review and contractor placement purposes.

For Figure 1-2, *excerpts* of its WWR shop drawing counterpart are reflected in Figure 1-3.

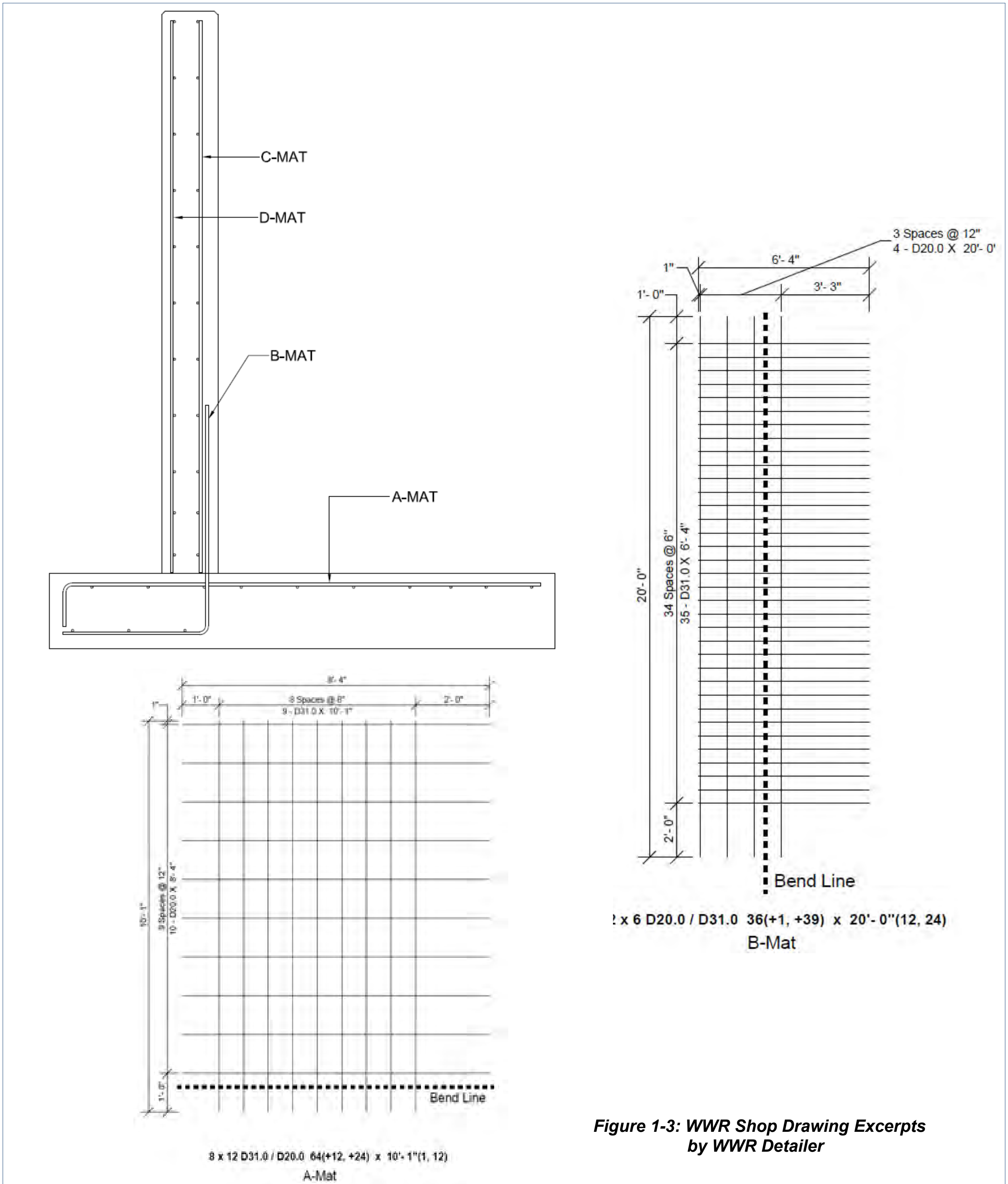


Figure 1-3: WWR Shop Drawing Excerpts by WWR Detailer

Using the above concept, the format and flow of Chapters 5 through 10 generally presents as follows:

A. Project design criteria and the spotlighted reinforced concrete element

B. Structural design of the reinforced concrete element

C. Derivation of reinforcement areas and alignment

D. Engineer's WWR contract document requirements

E. WWR fabrication drawings

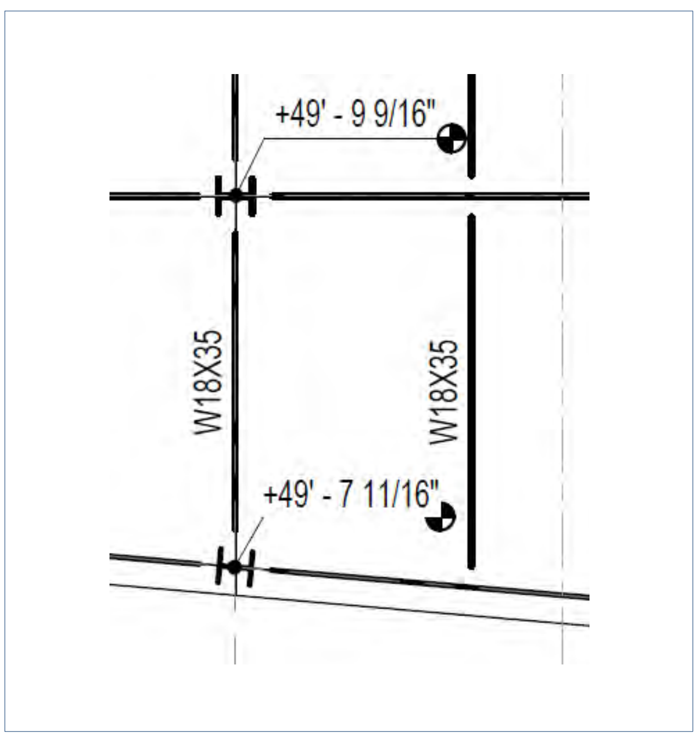
1.3 Catering to the Design Professional

In its efforts to provide WWR-based educational and technical guidance in past years, the Wire Reinforcement Institute has placed a heavy emphasis on the manufacturing process and material nomenclature itself. The hope was that WWR would become an integral part of the daily vocabulary of designer and contractor alike by creating contrast with hand-tied individual reinforcing bars.

These ongoing efforts have had significant success, most notably in the roadway precast, civil precast, and slab-on-ground markets. But in the cast-in-place concrete building structure market, the structural engineering community – in general – would prefer to avoid specifying "specialized" WWR criteria. While this information is critical to WWR manufacture, it is not relevant to conveying design intent on a set of detailed deliverables.

The irony is that structural steel is the primary competition to reinforced concrete when it comes to construction material.

In specifying the need for a W18x35 steel beam, structural contract drawings will indicate on the plan the following:



Now imagine a set of structural drawings with every W18x35 beam specified as follows:

W18x35 (6”, 0.425”) x (0.300”, 17.7”) x 22’-4”

As can be seen, the absence of certain implicit “turnkey” properties of manufacture is oftentimes preferred. From the designer’s perspective, applied to WWR:

D31 @ 12” oc vertical, D20 @ 12” oc horizontal

vs.

12x12 D31/D20 60” (+12”, +24”) x 9’-10” (8,2)

Note that most of the information in the second option, even though interpretable by the EOR (WWR mat style nomenclature is explained in Chapter 3), is beyond his/her responsibility to define, as it is specific to manufacturing capabilities and preferred production geometries that best suit fabrication, shipping, and placement. Expecting the EOR to embrace the definition of explicit WWR mat descriptions on structural drawings is analogous to demanding that the engineer build into his/her structural details all of the rebar lengths and bar counts. It is unreasonable and misaligned with standard practice. It is also worth noting that if a design professional were to attempt to specify turnkey WWR mat styles without themselves having an intimate familiarity of the producing manufacturer’s equipment capabilities and capacities, they could inadvertently create undesirable conflicts or delays when it comes time for the reinforcement to actually be procured, as the pre-specified, turnkey mat styles may not be feasibly manufactured.

In addition to *The Guide’s* aim to keep the prospect of detailing WWR a simple and familiar affair for the EOR, it must also maintain focus on those structural reinforcement applications in a cast-in-place concrete building structure that are actually well-suited for WWR implementation, or else its usefulness is greatly diminished. Because WWR is produced in mat form and is comprised of deformed wires up to and including 5/8” diameter, it does many things very well. This will be demonstrated in the details of Chapter 5 through Chapter 10. In contrast, WWR would likely not be the ideal option to replace, say, five (5) #11 reinforcing bars used as bottom flexural reinforcement in a 24” wide floor beam. This sort of application – one that neither the structural engineer nor WWR manufacturer would be highly motivated to pursue using WWR – is, therefore, not presented in *The Guide*. Quite simply, there are applications for which rebar is, and will continue to be, the most natural fit, just as there are those where WWR would provide an exceptional solution through the eyes of both the designer and the contractor.

Usefulness of this document as interpreted by the design professional and construction professional is of paramount importance, as is the integrity of the information presented within. The interplay between WWR and rebar on the same project, and oftentimes in the same structural application, is candidly presented.

1.4 Why Welded Wire Reinforcement?

The allure of implementing welded wire reinforcement into a cast-in-place concrete building project resides almost exclusively in the field. The prospect of placing large quantities of reinforcement at significantly higher installation rates in comparison to individual, hand-tied rebar can be enticing for contractors seeking to streamline labor and reduce duration of placement operations that fall on the critical path. This speaks directly to the ever-tightening project budgets and accelerated construction schedules that pervade today's cast-in-place building structure landscape, combined with the ever-present challenge that is qualified labor shortage.

As will be shown throughout subsequent detailing chapters of this guide, the directly-tangible aspect of WWR implementation – *actually getting the WWR information on the structural contract drawings* – is a relatively elementary undertaking for an engineer. Despite this procedural simplicity, if the engineer tasked with deciding whether or not to implement WWR does so in a "designer's vacuum", there may end up being little motivation to stray from rebar as the reinforcement of choice, even for those structural applications that are well-suited for WWR use. In contrast, if the engineer is engaged on the project in such a way that their proactivity is relied upon to contribute to improvement of project schedule, cost, and constructability, the return on investment of a decision to implement WWR can be significant. Not only do contractors hold in high regard a set of easily buildable contract documents, but the project owners reap obvious dividends as well.

1.5 A Complementary Document

The content of the *Welded Wire Reinforcement Design and Detailing Guide* is presented with the understanding that the engineering reader is already well-versed in structural design of reinforced concrete, structural detailing of reinforced concrete, or both, and that the contracting professional is equally proficient with regard to interpretation of material installation and placement drawing. *The Guide* is not intended to be a reinforced concrete design guide or code standard, nor is it intended to be a standalone resource for wholesale drafting methodology or construction tolerance and sequence. *The Guide* is best utilized as a complement document to design and detailing standard publications, creating value by offering detailing insights specific to the welded wire reinforcement product in a contract document preparation setting that is familiar and scalable to the reader.

Chapter Two

Welded Wire Reinforcement in ACI 318-19

Structural Design Using Welded Wire Reinforcement

The best starting point for gaining familiarity with the permitted use of welded deformed wire reinforcement as mild reinforcement in cast-in-place concrete building structures is Table 20.2.2.4a in ACI 318-19. An abbreviated version is presented below with acceptable usages noted in green.

Usage	Application		Maximum value of f_y or f_{yt} permitted for design calculations, psi	Welded Deformed Wire Reinforcement
Flexure; axial force; and shrinkage and temperature	Special seismic systems	Special moment frames	80,000	(3)
		Special structural walls	100,000	
	Other		100,000	
Lateral support of longitudinal bars; or concrete confinement	Special seismic systems		100,000	(1)
	Spirals		100,000	(2)
	Other		80,000	
Shear	Special seismic systems	Special moment frames	80,000	(1)
		Special structural walls	100,000	
	Spirals		60,000	(2)
	Shear friction		60,000	
	Stirrups, ties, hoops		60,000	
			80,000	
Torsion	Longitudinal and transverse		60,000	
Anchor Reinforcement	Other (non-special seismic system)		80,000	
Regions designed using strut-and-tie method	Longitudinal ties		80,000	
	Other		60,000	

(1) ASTM A1064 welded wire reinforcement is permitted in special seismic systems if the welds themselves are not relied upon to resist stresses in response to confinement, lateral support of longitudinal bars, shear, or other actions. Here the design and manufacturing solution is to have WWR mats fabricated with hooked terminations.

(2) Equipment used to produce ASTM A1064 welded wire reinforcement mats cannot physically form spiral/helical geometry, so WWR use in spiral applications is a non-starter.

(3) WWR is not permitted for flexural, axial, or shrinkage/temperature use in earthquake-resistant applications in which large inelastic rotation capacity must be exhibited by the primary structural members in special moment frames and special structural walls. To ensure that the level of inelastic behavior necessary for energy absorption is exhibited by these members during a design seismic event, ACI 318-19 establishes minimum ductility requirements for the "primary" reinforcement used therein. These ductility requirements are such that acceptable reinforcement is limited to ASTM A706 deformed reinforcing bars and ASTM A615 deformed reinforcing bars exhibiting a minimum tensile-yield ratio, minimum fracture and uniform elongations, and a maximum spread between actual yield strength and specified yield strength. In the case of WWR, the aforementioned ductility identity is not currently established in the ASTM A1064 material specification, and as such WWR is excluded.

2.1 Pertinent Code Sections

The reader is referred to the ACI 318-19 *Quick Reference for Welded Deformed Wire Reinforcement* (available as a free download on the WRI website) for a comprehensive summary of ACI provisions that are applicable to WDWR. These provisions will look very familiar to the practicing structural engineer, as they are also the basis for design using of reinforcing bars.

Obviously, there is strong emphasis on the code-supported direct interchangeability of reinforcement (reinforcing bars ↔ WWR) in *The Guide*. But in addition to this, just as "prohibited" applications were candidly discussed in Section 2.1, *The Guide* would be remiss in not highlighting a unique WWR attribute that in certain instances can offer potential design and detailing advantages over loose rebar. That attribute is the ability to rely upon the presence of welded wire intersections to reduce development length and lap splice dimensions for non-seismic applications, as well as to eliminate altogether the hooked curtailment often seen in flexural and shear reinforcement applications of gravity system applications..

ACI 318-19 Sections 25.4.6 (development length), 25.5.3 (lap splice), and 25.7.1.4 (hook replacement in shear application) illustrate how this unique attribute can be implemented in design if so chosen by the EOR, and Chapters 5 through 10 will contain selected beneficial examples of its use.

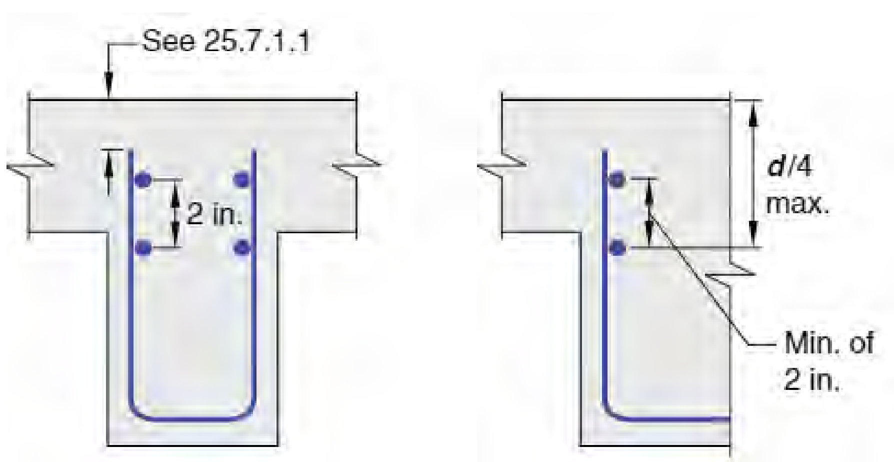


Figure 2-1: Stirrup in shear application using welded wire intersection in lieu of hooks (ACI 318-19 Section 25.7.1.4)

Image by the American Concrete Institute

2.2 Common Designer Questions

Suffice it to say that, despite extensive WWR inclusion throughout the design provisions of the ACI 318-19 Standard, there is still a certain amount of consternation on the part of design professionals as it pertains to usage of WWR. The list below is intended to provide rudimentary design-related clarity that otherwise might not be readily apparent in the black and white of the ACI standard.

Question	Response
1. What methods of corrosion protection are available for welded wire reinforcement?	Welded wire reinforcement can by epoxy-coated, hot-dipped galvanized, or comprised of stainless steel wires. The applicable material specifications are ASTM A1060, ASTM A884, and ASTM A1022.
2. Can welded wire reinforcement be manually welded (stick welded)?	Manual welding of welded wire reinforcement is permitted as noted in several ASTM Standard Specifications, including C1577 (precast monolithic concrete box sections), C478 (precast concrete manhole sections), C76 (reinforced concrete culvert for storm drain and sewer pipe), and C507 (reinforced concrete elliptical culvert). Other instances of manual welding may be permitted by the licensed design professional, as noted in ACI 318-19 Section 26.6.4.
3. With wires welded in an orthogonal pattern, is there any design advantage derived from potential "two-way" interactive flexural behavior?	ACI 318-19 does not recognize any two-way interaction that might exist as a result of orthogonally-arranged WWR. In structural engineering practice, reinforcement for each primary direction is essentially analyzed separately, independent of the presence of welds, with the only exception being those instances in which perpendicular perimeter/edge welded wires are depended upon for development or curtailment.
4. Is WWR permitted for use as flexural reinforcement in beams, walls, and structural slabs?	ACI 318-19 Table 20.2.2.4(a) allows the use of ASTM A1064 WWR as flexural, axial, and/or shrinkage and temperature reinforcement in all conventional beam, wall, and structural slab applications. WWR is not used as flexural, axial, and/or shrinkage and temperature reinforcement in <u>special</u> seismic systems (special moment frames and special structural walls).

Question	Response
5. Is WWR permitted for use as transverse reinforcement in special seismic systems?	ASTM A1064 welded wire reinforcement is permitted as transverse reinforcement in special moment frames and special structural walls per ACI 318-19 Table 20.2.2.4(a), but the welds themselves are not permitted to be relied upon for resistance to any stresses. As such, for WWR used in seismic applications, bond and anchorage of the reinforcement must be derived from wire surface deformations and hooked wire curtailments only, with any potential contribution by welded intersections ignored/disregarded.
6. How is the welding done, and how is weld integrity confirmed?	Welding is carried out by automated welding machines using a controlled cross-wire resistance welding process. Unlike fusion welding characterized by the depositing of a consumable electrode, electrical resistance welding is predicated on welded parts (two wires) being pressed together to allow the flow of electricity across the contact interface, resulting in the material being fused together. This process is acknowledged in ACI 318-19 Section R26.6.4. Confirmation of weld integrity and strength is carried out as part of the material's certification process during manufacture, with ASTM A1064 as the governing material specification, which is also referenced in ACI 318-19.
7. If a design is to rely on a welded intersection for development or curtailment purposes, how strong does the weld need to be to be considered "structural"?	By way of the ACI 318-19 reference to ASTM A1064, the required weld strength shall not be less than 35 ksi x nominal area of the larger wire at the intersection. This is also referenced in ACI 318-19 R20.2.2.4 and confirmed during manufacture.
8. For the purposes of simplifying design, can the engineer ignore potential contribution from the welded intersections despite the intersections being present?	Yes. In fact, save for instances where perimeter/edge welded wires might be depended upon for development or curtailment as a designer's option, the ACI 318 standard treats welded deformed wire reinforcement the same as reinforcing bars or loose, individual deformed wires: familiar rules for straight-line development length, lap splice, and hook development are applicable.

Question	Response
9. Is there a size relationship requirement for deformed wires being welded together?	If the structural design is relying upon welded intersections for the purposes of development or curtailment, then, yes, there exists a wire size relationship: the smaller wire must have a cross-sectional area at least 40% that of the larger wire per ASTM A1064. This is a requisite wire size relationship for the “structural” weld strength provision (Item #7, above) to be relied upon. In this case, the deformed wire must not be less than a D4.0. If the structural design does not rely upon welded intersections, then there is no wire size relationship requirement. Per ASTM A1064, the WWR producer is still required to verify that welded intersections exhibit a weld shear strength of 800 pounds. This is typically for basic transport, handling, and placing purposes.
10. ACI 301 currently requires a 12" support spacing for wire sizes smaller than D4.0 / W4.0. What are WRI's recommendations for support spacing of WWR?	The ACI 301 mandated support spacing does not guarantee conformance with a project's specified acceptable tolerance, nor does it allow for alternative support patterns or methods that would achieve conforming results. Support spacing should be derived on a case-by-case basis with due consideration for attributes such as the reinforcement itself (type, size, and spacing), the intended function/performance of the reinforced concrete element, the selected chair/bolster type, and the substrate upon which the support rests, to name a few. Pre-established tolerances - whether through a combination of ACI 318 and ACI 117 requirements or through a design professional's project-specific requirement - should govern placement of welded wire reinforcement. Refer also to TF 702 in the WRI technical document library. In all instances, the Wire Reinforcement Institute encourages close collaboration between a project’s contractor and design professional of record to ensure appropriate placement criteria and procedures are established and maintained.

Question	Response
11. In Section 20.2.1.7.3 of ACI 318-19, the standard states that, for non-stirrup applications, a maximum spacing of 16” for welded intersections in the direction of calculated stress is applicable. What if I want to specify reinforcement in welded deformed wire reinforcement mat form, but only need structural wires in one direction and not the other? Do I still need to satisfy the spacing requirement for what are essentially non-structural wires?	The need for “single-direction” welded deformed wire reinforcement mats is very common. It is noteworthy that ACI 318-19 acknowledges treatment of welded deformed wire reinforcement in a manner identical to individual loose deformed bars and deformed wires when welded intersections are either absent or are not intentionally-positioned for tensile development or curtailment. With this treatment established, and in light of modern welded wire manufacturing capabilities, it is difficult to find a technical justification for a broadly-applied prescriptive maximum spacing of welded intersections as is done in Section 20.2.1.7.3 ACI 318-19 Sections 25.4.6.4 and 25.5.3.1.1 outline the common scenario in which the absence of intentionally-positioned welded intersections in turn requires calculation of welded deformed wire reinforcement development length and lap splice length, respectively, to be based on the same equations that are used for individual (loose, non-welded) deformed bars and deformed wires. In essence, these ACI 318 provisions direct the designer to disregard any contribution a welded intersection might make to bond and development, and have the designer instead base these attributes on the deformed wire surface's contribution alone. We encourage designers and contractors to continue to take advantage of the highly-customizable welded deformed wire reinforcement mat arrangements capable of being produced by modern automated welding equipment. This includes “single-direction” welded wire reinforcement mats characterized by structural deformed wires in one direction and perpendicular non-structural wire positioned as required in the other direction.

Question	Response
12. Is welded wire reinforcement a proprietary product?	No. welded wire reinforcement is a mild steel reinforcement required to conform to the ASTM A1064 Standard Specification. All welded wire reinforcement manufacturers are held to this common standard. While it is feasible for one welded wire reinforcement producer to pursue different markets or applications than its competitors, or for one producer to have slightly different internal processes and/or automated welding equipment than its competitors, the reinforcement itself must always be compliant with the ASTM Specification’s requirements, and this is confirmed through ASTM A1064 certification and testing measures. Welded wire reinforcement is a manufactured product in the same sense as reinforcing bars or structural steel sections, and as such should not be subject to unique proprietary-like scrutiny on the basis of its inherent pre-assembly.

Chapter Three

Manufacture of Welded Wire Reinforcement

3.1 Process and Nomenclature

The definition of welded deformed wire reinforcement can be derived from content in ASTM A1064 *Standard Specification for Carbon-Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete* and presented as follows:

A product consisting of cold-worked, surface-deformed steel wires, assembled into orthogonal mats by automatic welding machines that assure accurate spacing and alignment of all wires of the finished product.

Hot-rolled steel rod (from billets) is cold-worked by either cold-rolling or cold-drawing the wire to a specified diameter. Deformations can be indented or raised rib (protrusion) type. Wire size and wire spacing on a particular WWR mat can be varied to suit the requirements of the detailed structural design. Welding is carried out using electrical resistance welding which employs the principle of fusion combined with pressure to produce a weldment.

Contrast between production of traditional reinforcing bars and WWR is shown below:

REINFORCING BARS	WELDED WIRE REINFORCEMENT
Metal scrap in electric ARC furnace	Metal scrap in electric ARC furnace
Molten steel cast into rectangular billets	Molten steel cast into rectangular billets
Billets hot rolled to standardized, deformed diameters: • #3 to #18 • Straight or coiled	Billets hot rolled to standardized, smooth rod diameters: • AISI 1018, AISI 1020, etc. • Variable diameters (7/32” - 11/16”) • Coiled
Finished product: ASTM A615 OR ASTM A706 rebar	Hot-rolled rod is run through a series of dies or cassettes for reduction and deforming: • Specific “design” diameter achieved • Surface deformation achieved
	Cold-Worked Wires Ready for Feeding to Automatic Welding Machine
	Finished product: ASTM A1064 welded wire reinforcement

Figure 3-1: Production Comparison Chart

Wire size varies from on the order of 1/8” diameter, up to 5/8” diameter, the latter which is equivalent in size to a #5 reinforcing bar. While larger wire sizes up to 3/4” diameter can be produced, current ACI 318 provisions require these to be analyzed using welded plain wire reinforcement restrictions. As such, anything larger than 5/8” diameter is excluded from the scope of *The Guide* considering any demand that might exist for heavy plain WWR in cast-in-place building structures would be extremely low.

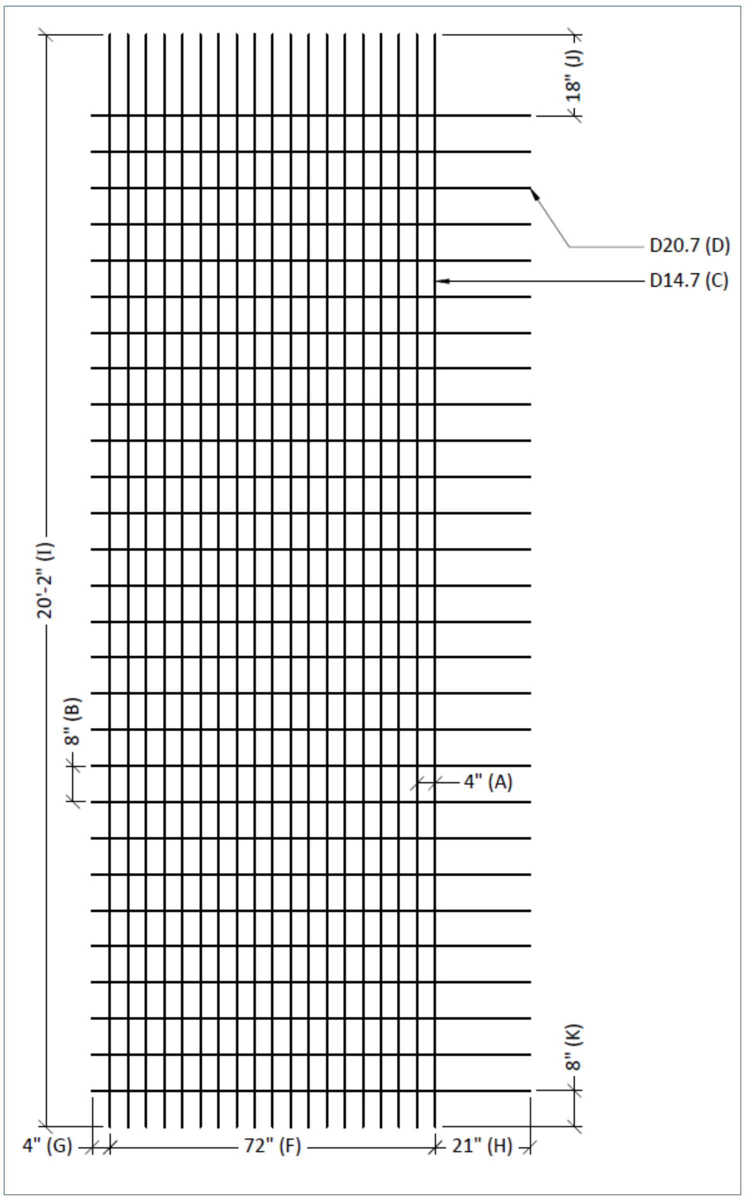
For individual deformed wires, the nomenclature is quite intuitive: the cross-sectional area is built right into the designation:

A wire with cross sectional area equal to 0.247 in^2 → *D24.7*

A wire with cross sectional area equal to 0.144 in^2 → *D14.4*

A wire with cross sectional area equal to 0.280 in^2 → *D28.0*

These wires are in turn welded together to form highly customizable geometries of orthogonal WWR mats. The basic nomenclature form used to describe a WWR mat configuration (or “style”) is shown below.



4x8 D14.7/D20.7 72” (+4”,+21”) x 20’-2” (18,8)

↓

AxB C/D F (G,H) x I (J,K)

A: linewire spacing (inches)

B: crosswire spacing (inches)

C: linewire size (in^2 x 100)

D: crosswire size (in^2 x 100)

F: WWR mat width excluding overhangs (inches)

G: left-side overhang (inches, positive if past leftmost linewire)

H: right-side overhang (inches, positive if past rightmost linewire)

I: WWR mat length including overhangs (ft-in)

J: front end overhang (inches)

K: back end overhang (inches)

E: yield strength of WWR mat

(not shown here as part of mat style)

As was introduced in Section 1.3 of Chapter 1, there are some attributes associated with WWR nomenclature that are generally more relevant to manufacturing interests than they are to a structural engineer's design interests. Distilling the aforementioned mat style down to information that is pertinent to a structural detail found on a set of contract drawings reveals the following:

4x8 **D14.7** /D20.7 72" (+4",+21") x 20'-2" (18,8)

➔ **0.147 in^2 @ 4" oc (0.441 in^2/foot) running in the long direction**

4x **8** D14.7/ **D20.7** 72" (+4",+21") x 20'-2" (18,8)

➔ **0.207 in^2 @ 8" oc (0.331 in^2/foot) running in the short direction**

4x8 D14.7/D20.7 **72" (+4",+21") x 20'-2" (18,8)**

➔ **What the WWR detailer has to do to provide a mat geometry that satisfies the engineer's detailed design intent**

Depending on various geometric and spatial considerations inherent to reinforcement layouts presented on structural contract drawings, the manufacturing nomenclature method described above might prove to be insufficient in capturing a design-compatible reinforcement layout. In this case, the WWR detailer will likely rely on illustrative communication than trying to resolve all of the subtleties of a WWR configuration in a single string of numerical characters. Below is an example of a WWR mat configured to reflect stagger of primary reinforcement.

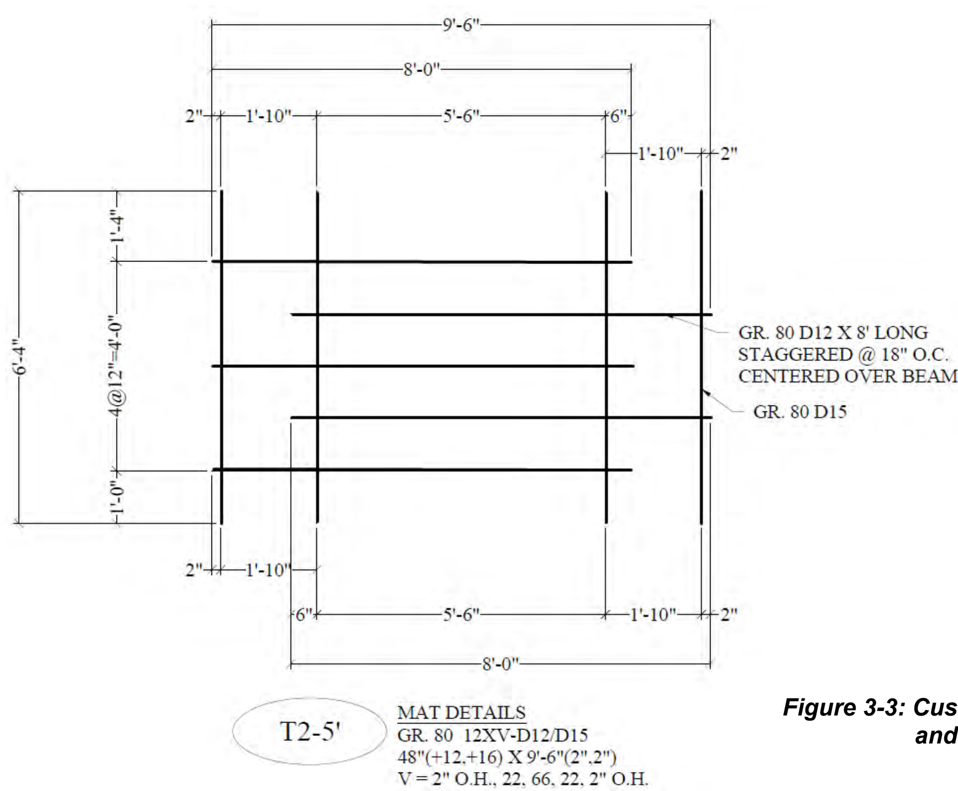


Figure 3-3: Custom WWR Mat Geometry and Nomenclature

To further illustrate WWR variability captured through the combination of mat detail and designation, the figure below is provided. While it is unlikely that the specific arrangement shown in this figure is representative of a structural reinforcement layout that would be commonly encountered on a cast-in-place concrete building structure, it shows the customizable nature of the product and how it is documented internally by the manufacturer.

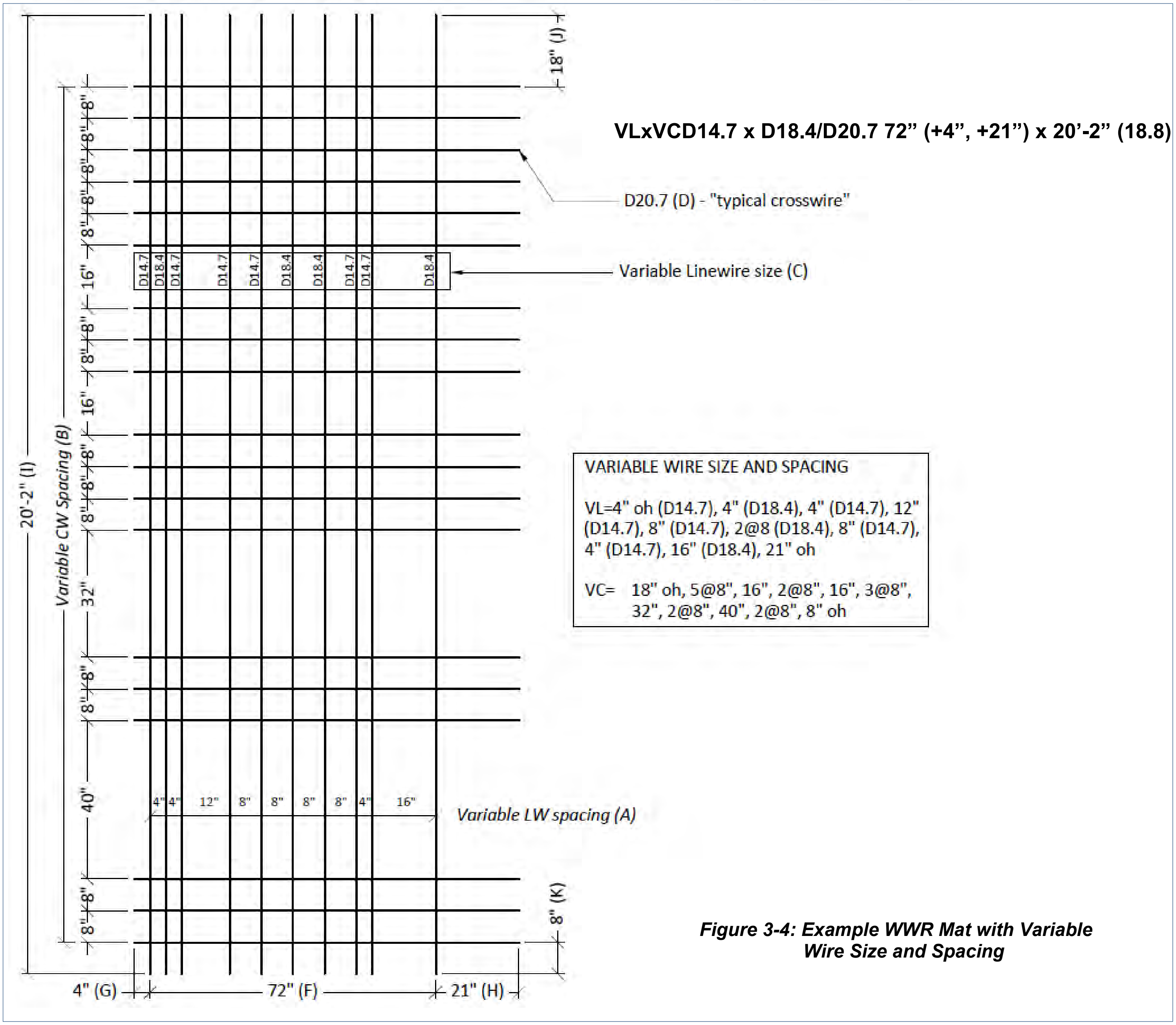


Figure 3-4: Example WWR Mat with Variable Wire Size and Spacing

A design professional would not be expected to derive the specifics of the above mat configuration and present it on a set of contract drawings, but would instead simply be responsible for informing the manufacturer/fabricator of attributes already commonly defined for rebar: steel area (size), spacing, position, and curtailment (hooks, etc.). Similarly, a contractor’s life is made simpler by the WWR manufacturer’s provision of detailed placement drawings and accompanying simplified mat nomenclature (MAT-1A, SHEET-1A, etc.). Examples presented in Chapter 5 through 10 illustrate this very clearly.

3.2 Notes on Manufacturing Tolerances

The manufacture of welded deformed wire reinforcement draws guidance from ASTM A1064 *Standard Specification for Carbon-Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete.*

ASTM limitations placed on dimensional variation of flat WWR mats are presented below. For other manufacturing attributes, refer to ASTM A1064 directly in conjunction with the WRI *Manual of Standard Practice – Structural Welded Wire Reinforcement* (WWR-500).

Mat Width – ASTM A1064 Article 10.1

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. greater or less than the specified width. In case the width of flat sheets are specified as the overall width (tip-to-tip length of transverse wires), the width shall not vary more than ±1 in. from the specified width.

These tolerances apply to a flat sheet, as measured just prior to loading for shipment and delivery to the jobsite.

Mat Length – ASTM A1064 Article 10.2

The overall length of flat sheets, measured on any wire, shall not vary more than ±1 in. or 1%, whichever is greater.

These tolerances apply to a flat sheet, as measured just prior to loading for shipment and delivery to the jobsite.

Transverse Wire Overhang – ASTM A1064 Article 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. unless otherwise specified. When transverse wires are specified to project a specific length beyond the center line of the longitudinal edge wire, the permissible variation shall not exceed 0.5 in. greater than or less than the specified length.

Wire Spacing – ASTM A1064 Article 10.6

The average spacing of wires shall be such that the total number of wires contained in a sheet or roll is equal to or greater than that determined by the specific spacing, but the center-to-center distance between individual wires shall not vary more than 0.25 in. from the specified spacing. Sheets of welded wire reinforcement having the specified length shall not be required to contain an identical number of transverse wires, and therefore, shall be permitted to have various lengths of longitudinal overhang.

The specifying designer is encouraged to utilize Article 4.2 of ASTM A1064, where they are permitted to define special requirements governing the WWR material and physical properties that the purchasing contractor would report to the manufacturer as condition of the reinforcement material order

3.3 Bending of Welded Wire Reinforcement

One of the greatest benefits of WWR is the manufacturer's ability to fabricate it to bent geometries. Bending criteria for WWR are established in ACI 318-19, wherein Section 25.3.1 defines standard hook geometry for reinforcing bar curtailment that are in turn adopted by WWR manufacturers in the fabrication of bent mats, while Section 25.3.3 defines explicitly the bend requirements for WWR used as stirrups or ties.

For a more extensive discussion of welded wire reinforcement bending capabilities and examples, refer to WRI's *Bending Welded Wire Reinforcement* (WWR-400).

Figure 3-5: Hydraulic or pneumatic machines are used to fabricate WWR bent mats

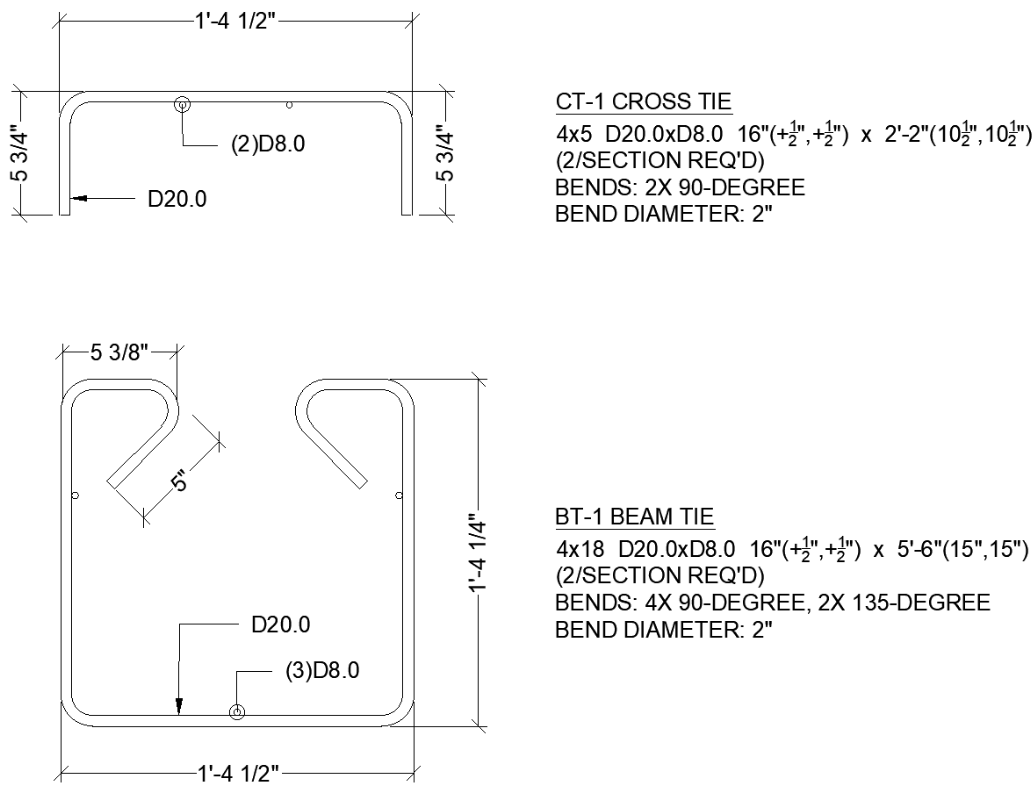


Figure 3-6: An example of an application-specific bend drawing prepared by the manufacturer

Chapter Four

Specifying Welded Deformed Wire Reinforcement

4.1 General Requirements

Practicing engineers are likely familiar with the steps necessary to effectively communicate a design's reinforcement needs in terms of deformed reinforcing bars (rebar) on their structural contract documents. Generally, the delivery of this information comes in the form of "typical" and "project-specific" content.

Information unchanged from one project to the next ("TYPICAL"):

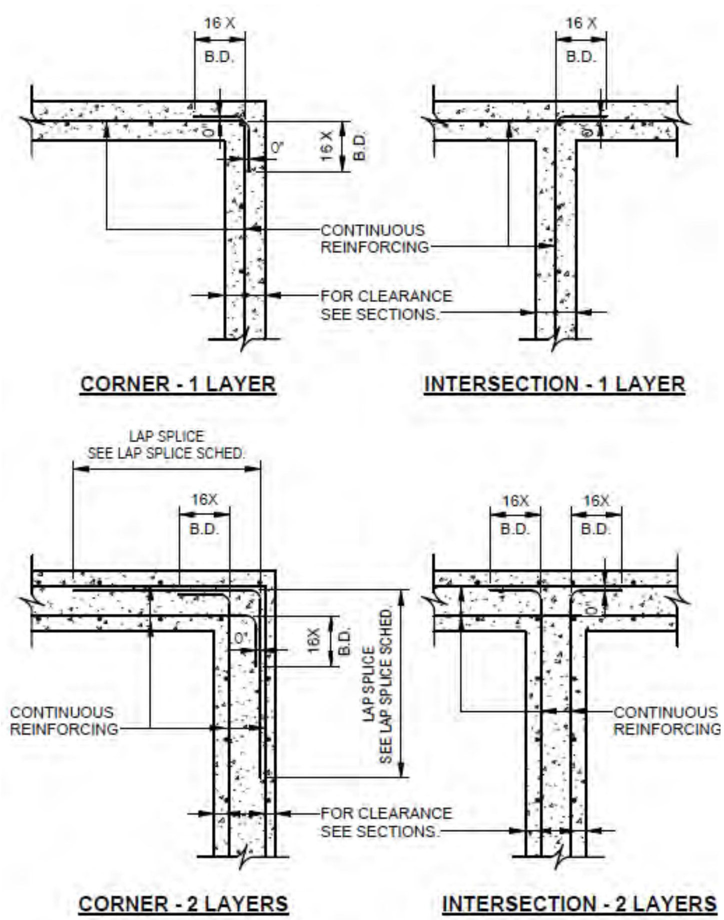


Figure 4-1: Typical reinforcement details

BAR LAP SCHEDULE		
BAR SIZE	CMU WALLS	CAST-IN-PLACE CONCRETE
#4	28"	24"
#5	36"	28"
#6	42"	36"
#7	52"	50"
#8	–	56"

NOTES:

1. LAP SPLICE LENGTHS SHOWN FOR REINFORCED CMU WALLS SHALL BE USED FOR BOTH VERTICAL AND HORIZONTAL REINFORCEMENT.

2. LAP SPLICE LENGTHS SHOWN FOR REINFORCED CONCRETE SHALL APPLY TO ALL REINFORCEMENT FOR CAST-IN-PLACE CONCRETE SLABS, WALLS, AND FOUNDATIONS.

Figure 4-2: Typical splice and reinforcement schedules

C. REINFORCING STEEL

1. REINFORCING STEEL SHALL CONFORM TO ASTM A615, GRADE 60. LAP 30 X BAR DIAMETERS AT THE SPLICES UNLESS OTHERWISE NOTED ON THE DRAWINGS OR IN THE PROJECT SPECIFICATIONS. LAP CONTINUOUS WALL FOOTING BARS INTO SPREAD FOOTINGS 24" MINIMUM.
2. ALL REINFORCING STEEL SHALL BE SECURELY TIED AND ANCHORED IN PLACE TO PREVENT DISLOCATION DURING THE PLACING OPERATION.
3. 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 4-3: General notes covering reinforcement material and usage requirements

Information that varies from one project to the next ("PROJECT-SPECIFIC"):

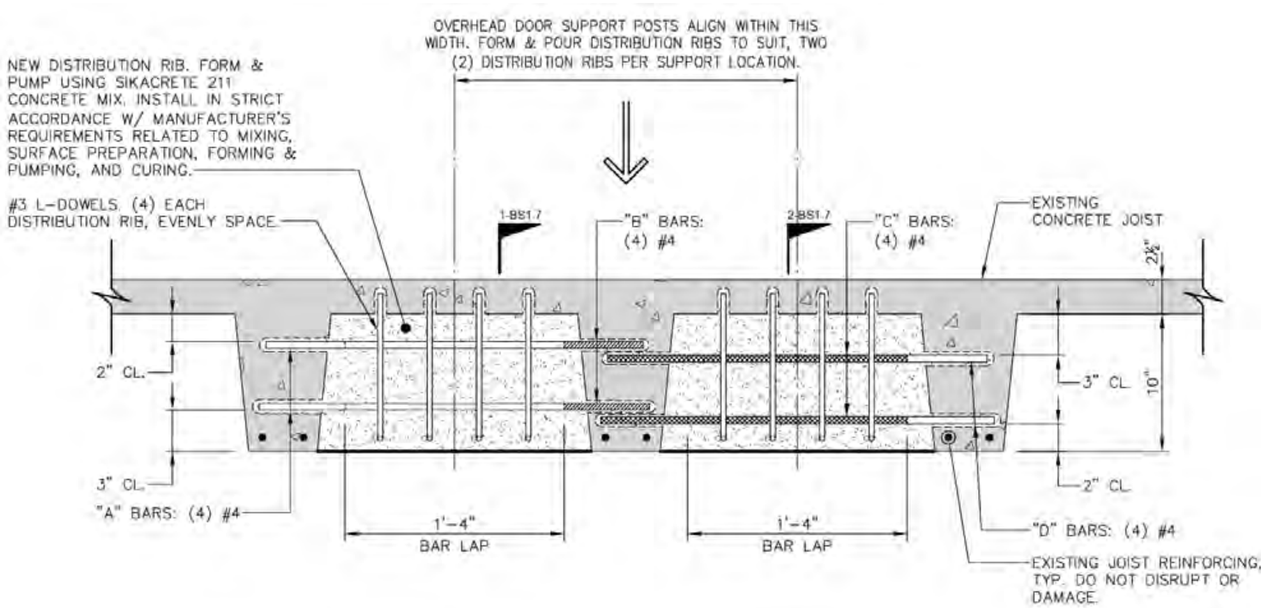


Figure 4-4: Project-specific reinforced concrete detail

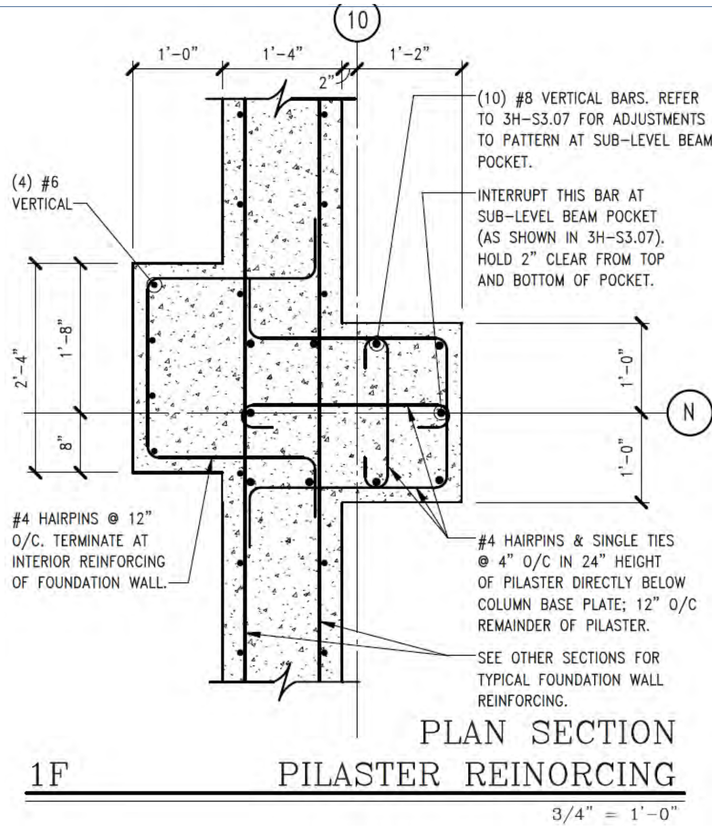


Figure 4-5: Project-specific reinforced concrete details

HEADWALL AND WINGWALL SCHEDULE

DIMENSIONS									REINFORCEMENT													
RETAINED BACKFILL HEIGHT ABOVE TOP OF FOOTING ELEVATION	WALL TYPE	"A"	"B"	"C"	"D"	"E"	"K"	"T"	"L" DOWELS		"V" BARS		"H" BARS	"FT" BARS		"FL" BARS		"KT" BARS	"KL" BARS	LAP	
UP TO 8'-0"	A	12"	1'-6"	4'-0"	6'-6"	N/A	N/A	1'-3"	L1	#7 @ 12"%	V1	#7 @ 12"%	#5 @ 16"%	FT1	#7 @ 12"%	FL1	(5) #5	N/A	N/A	LAP 1	36"
									L2	#5 @ 16"%	V2	#5 @ 16"%		FT2	#7 @ 12"%	FL2	(5) #5			LAP 2	24"
>8'-0" TO 17'-6"	B	18"	3'-0"	8'-6"	13'-0"	1'-0"	3'-0"	2'-0"	L1	#7 @ 8"%	V1	#7 @ 8"%	#5 @ 12"% (WINGWALLS)	FT1	#7 @ 8"%	FL1	(13) #5	#5 @ 12"%	(3) #5	LAP 1	36"
									L2	#5 @ 12"%	V2	#5 @ 12"%	#5 @ 10"% (HEADWALL)	FT2	#7 @ 8"%	FL2	(7) #5			LAP 2	24"

FOOTING SCHEDULE

MARK	SIZE	THICKNESS	REINFORCING
F30	3'-0" X 3'-0"	1' - 4"	(4) #5 EACH WAY, BOTT
F40	4'-0" X 4'-0"	1' - 2"	(5) #5 EACH WAY, BOTT
F46	4'-6" X 4'-6"	1' - 4"	(5) #5 EACH WAY, BOTT
F50	5'-0" X 5'-0"	1' - 4"	(6) #5 EACH WAY, BOTT
F50A	5'-0" X 5'-0"	1' - 4"	(5) #5 EACH WAY, TOP & BOTT
F1038	10'-0" X 38'-0"	3' - 0"	#7 @ 8" O/C EACH WAY, TOP & BOTT
F3040	3'-0" X 4'-0"	1' - 4"	(5) #5 SHORT WAY, BOTT (4) #5 LONG WAY, BOTT
F7346	7'-3" X 4'-6"	1' - 4"	(5) #5 SHORT WAY, (8) #5 LONG WAY, BOTT

Figure 4-6 and 4-7: Project-specific reinforcement schedules

What degree of effort, then, is expected of the design professional as it relates to effective specification of welded deformed wire reinforcement? Just how much change in the conventional design and detailing routine, if any, will a consulting engineer be willing to take on without feeling encumbered?

There are two primary methods by which welded wire reinforcement can be incorporated into structural contract documents: by **Direct Specification** or as a **Pre-Approved Equal**.

4.2 Welded Wire Reinforcement Information by Direct Specification

Direct Specification is exactly as it sounds - welded wire reinforcement information is specified directly and explicitly on the contract drawings by the design professional.

As was discussed in Chapter 1 of *The Guide*, there is a threshold for welded wire information definition beyond which an engineer wouldn't be expected to operate. The following basic "design" attributes would be the responsibility of the design professional to define:

- Wire yield strength
- Wire size
- Wire spacing
- Wire orientation (bent geometry, positioning within the concrete element, etc.)
- Wire lap splice requirements

In effect, all remaining manufacturing attributes related to the manufactured WWR mat width and length would be deferred to the downstream WWR detailer. The result, again as shown in Chapter 1, is intuitive:

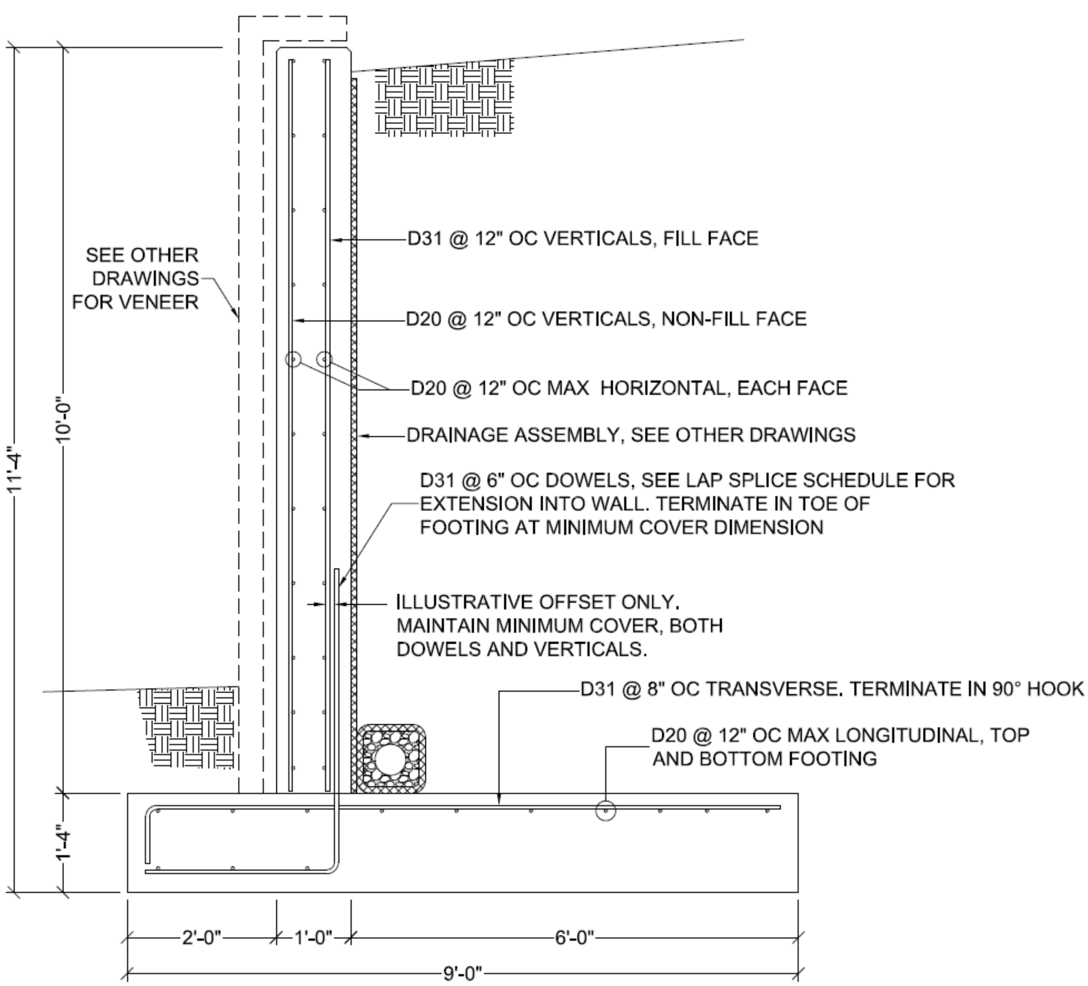


Figure 4-8: Cantilever Retaining Wall Section with WWR directly specified

Slab-on-ground reinforcement applications continue to be excellent examples of directly-specified welded wire reinforcement, with only very simple definitions required for effective implementation.

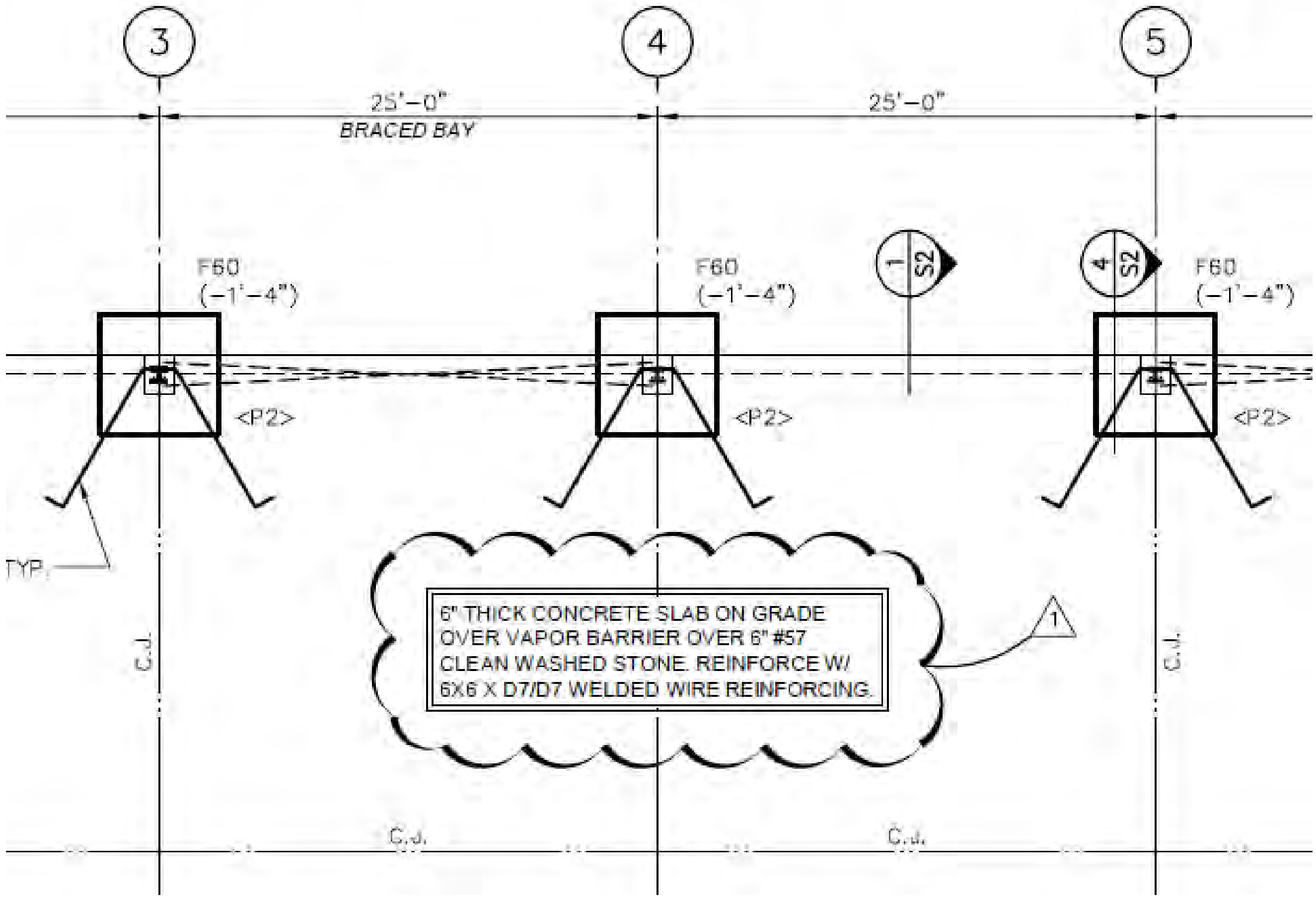


Figure 4-9*: *Excerpt showing directly specified WWR

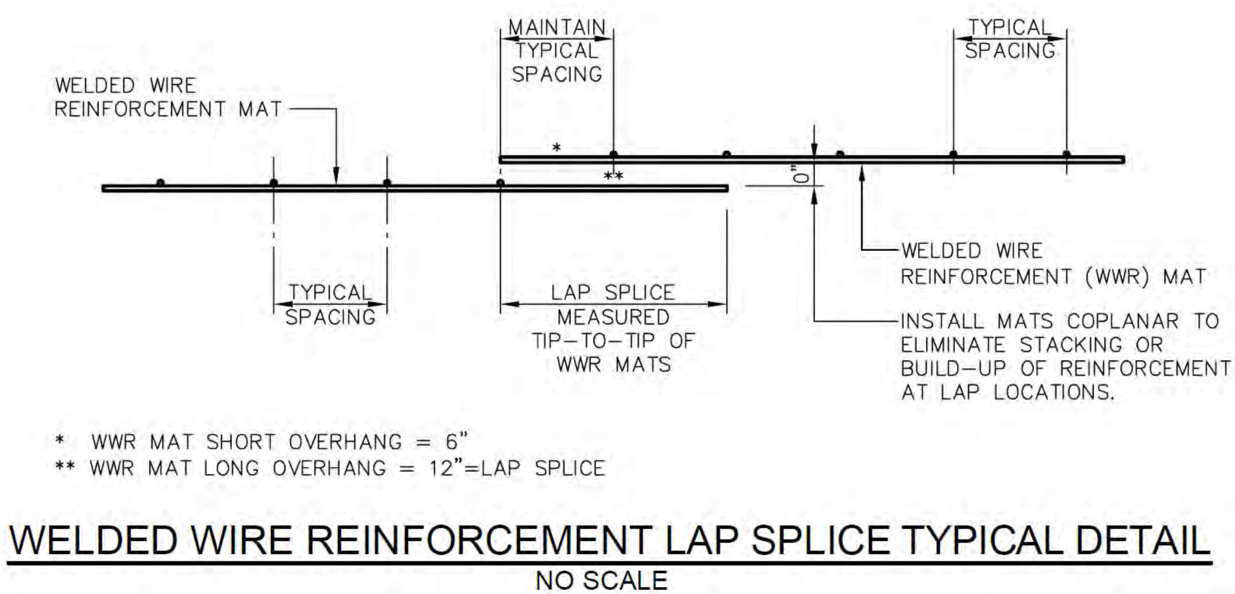


Figure 4-10: Typical detail related to directly specified WWR

In directly specifying welded deformed wire reinforcement, it is important for the language and composition of typical reinforcement details, typical splice and reinforcement schedules, and general notes to be expanded to include appropriate reference to welded wire reinforcement. This is essentially a one-time adjustment that would need to be made by the design professional of record, with modification to "typical" information that is from that moment forward intended to remain unchanged on subsequent projects. Likewise, "project-specific" information would require specific acknowledgement of intended WWR usage as well.

The primary benefit of Direct Specification of welded deformed wire reinforcement is realized by those engineers who wish to have explicit, illustrated control over very specific usages of WWR on the project. In taking this approach, the engineer is actually absorbing quite a bit of the detailing-related burden that would otherwise be expected of the downstream welded wire reinforcement detailer.

The primary drawback of Direct Specification of welded deformed wire reinforcement, other than the additional time spent in preparation of WWR content and detailing by the design professional, is the unintentional restriction placed on the contractor for an application that might be better suited for loose, individual reinforcing bars. The irony here is that the same exact drawback scenario exists – and is likely even more prevalent in the construction industry as a whole - when a design professional of record explicitly defines loose individual reinforcing bars for a structural application that could very well yield significant favorable field placement and labor savings if it were permitted to be installed using WWR.

It should be understood that Direct Specification will still require downstream involvement of a WWR detailer, as it is the detailer who will take the design professional's specified wire sizes, spacings, and orientations and transform this information into mat configurations that will actually be produced on the equipment in the plant. As was alluded to in Section 1.3 of Chapter 1, project design professionals are discouraged from explicitly defining "turnkey" welded deformed wire reinforcement mat styles unless they themselves are intimately familiar with the producing manufacturer's capabilities and capacities.

4.3 Welded Wire Reinforcement Specified as a Pre-Approved Equal

The creation of buildings and related structures in the built environment requires the close collaboration of design and construction professionals. Design and construction are two very distinct disciplines, the marriage of which is inextricably linked to a project's successful completion. Nowhere is this more apparent than with today's fast-based, design-build type project delivery method, where it is critical for all of the moving parts of the design-build team to operate in step.

The specification of welded deformed wire reinforcement as a Pre-Approved Equal is a manifestation of the value of collaboration. The EOR has a very specific design intent needing to be achieved on a project, while the contractor seeks to deploy the optimum use of time and resources on the jobsite to transform that design intent into built elements. Mild reinforcement for reinforced concrete structures essentially comes in two forms: deformed bar and deformed wire, each of which effectively receives equal treatment in the eyes of the ACI 318-19 Standard as it relates to alignment with modern reinforced concrete design methodologies. As such, the engineer is in a unique position to deploy both types of reinforcement, while affording the contractor the necessary oversight related to the means and methods of executing the installation.

Specifying welded deformed wire reinforcement as a Pre-Approved Equal requires very little effort on the part of the design professional of record. This effort comes in the form of a one-time modification to a project's general notes section, wherein permissive language (and/or exclusive language) related to the use of WWR on the project is introduced. All other mild reinforcement content that is presented in terms of rebar is permitted – *and encouraged* - to remain unchanged. This allows for the engineer to continue on with longstanding, familiar design routines without compromise to productivity, while building flexibility into the contract documents that relates directly to constructability and construction schedule using appropriate mild reinforcement.

The following figures provide examples of “pre-approved equal” language that can be easily implemented.

MILD REINFORCING STEEL

1. TYPICAL DEFORMED REINFORCING BARS (REBAR) SHALL CONFORM TO ASTM A615, GRADE 80. BARS SHALL BE LAPPED IN ACCORDANCE WITH THE REBAR LAP SCHEDULE UNLESS OTHERWISE EXPLICITLY DETAILED.
2. LONGITUDINAL REINFORCEMENT IN SPECIAL MOMENT FRAME BEAMS AND COLUMNS, AND VERTICAL AND HORIZONTAL REINFORCEMENT IN SPECIAL STRUCTURAL (SHEAR) WALLS SHALL BE ASTM A706 GRADE 60 OR GRADE 80 AS NOTE. TENSILE AND ELONGATION PROPERTIES SHALL BE CONFIRMED THROUGH MILL REPORT DOCUMENTATION PROVIDED AS PART OF THE PROJECT REINFORCEMENT SUBMITTAL
3. WELDED DEFORMED WIRE REINFORCEMENT SHALL CONFORM TO ASTM A1064 GRADE 80 AND SHALL BE PROVIDED IN SHEET FORM. REINFORCEMENT SHEETS SHALL BE MANUFACTURED WITH OVERHANG LENGTHS SUFFICIENT TO ACHIEVE A LAP SPLICE LENGTH EQUAL TO THE GREATER OF 12 INCHES OR THE LAP SPLICE DIMENSION SHOWN IN THE REBAR LAP SCHEDULE FOR BAR OF EQUAL (OR GREATER) DIAMETER AND GRADE, UNLESS OTHERWISE NOTED. SHEETS AND ASSOCIATED LAP REGIONS SHALL BE INSTALLED COPLANAR SO AS TO NOT "STACK".
4. WELDED DEFORMED WIRE REINFORCEMENT OF EQUAL AREA, EQUAL OR LESSER SPACING, AND IDENTICAL CURTAILMENT (HOOKS AND LAP SPLICES) IS PERMITTED AS A SUBSTITUTION FOR DEFORMED REINFORCING BARS, EXCEPT IN THE FOLLOWING STRUCTURAL APPLICATIONS:
   A. LONGITUDINAL STEEL IN SPECIAL MOMENT FRAMES
   B. VERTICAL AND HORIZONTAL STEEL IN SPECIAL STRUCTURAL WALLS

Figure 4-11: General Notes excerpt showing WWR permissive language and specific exclusions. Note that this engineer is calling for identical curtailment of WWR, which essentially disallows reliance on welded intersections for structural purposes.

1. MILD STEEL REINFORCING BARS SPECIFIED ON THESE STRUCTURAL CONTRACT DRAWINGS ARE PERMITTED TO BE REPLACED BY ASTM A1064 WELDED DEFORMED WIRE REINFORCEMENT (WWR) IN ALL APPLICATIONS, WITH THE FOLLOWING EXCEPTIONS:
   A. SPECIAL MOMENT FRAME LONGITUDINAL REINFORCEMENT
   B. SPECIAL STRUCTURAL WALL VERTICAL REINFORCEMENT
   C. COUPLING BEAM DIAGONAL REINFORCEMENT
2. WWR SUBSTITUTIONS SHALL NOT ALTER THE REINFORCEMENT UNIT CROSS-SECTIONAL AREA,SPACING, AND POSITIONING AS PRESENTED IN THE STRUCTURAL CONRACT DRAWINGS, UNLESS APPROVED IN WRITING BY THE STRUCTURAL ENGINEER OF RECORD.
3. FABRICATOR DETAILING SHALL REFLECT POSITIONING OF STRUCTURAL (AND NON-STRUCTURAL) WIRES THAT BEST MINIMIZES INTERFERENCE WITH OTHER STRUCTURAL COMPONENTS.

Figure 4-12: WWR permissive language and specific exclusions suitable for incorporation into project General Notes

REBAR LAP LENGTH SCHEDULE NORMAL-WEIGHT CONCRETE[A] (145 PCF)						
BAR	MIN CLR COVER	MIN BAR SPACING	f'c=2500 PSI		f'c=4000 PSI	
			TOP[B]	OTHER	TOP[B]	OTHER
#3	1.5	4.5	20	16	16	14
#4	1.5	4.0	26	20	20	16
#5	1.5	3.8	32	24	26	20
#6	2.0	3.0	46	36	38	28
#7	2.0	3.5	68	52	54	42
#8	2.0	3.0	102	78	82	62
#9	2.0	3.4	116	88	92	70
#10	2.0	3.8	130	100	102	80
#11	2.0	4.2	144	110	114	88

NOTES:

A. VALUES IN TABLE ARE IN INCHES. LENGTHS BASED ON 60 GRADE REINF (NOT COATED) WITH CLASS B LAP SPLICES

B. TOP REINF IS DEFINED AS HORIZ REINF WI MORE THAN 12' OF CONC BELOW

C. CONG STRENGTH SELECTED IN TABLE SHALL NOT EXCEED DESIGN CONC STRENGTH

D. DEFORMED WIRE LENGTHS BASED ON 00 GRADE REINF (NOT COATED) WITH CLASS BIAPSPLICES

E. DEFORMED WIRE TO MATCH ANY SPLICE LENGTHS SPECIFIED IN PLAN (INVERT TO ARCH DOWELS)

DEFORMED WIRE LAP LENGTH SCHEDULE NORMAL-WEIGHT CONCRETE (145 PCF)				
WIRE	f'c=2500 PSI		f'c=4000 PSI	
	TOP	OTHER	TOP	OTHER
D8.3	21	17	17	13
D15	29	22	23	18
D23	36	27	28	22

DEFAULT SCHEDULE: FOUNDATIONS, BASEMENTS, CIVIL STRUCTURES

Figure 4-13: WWR permissive language and expanded lap splice schedule. Note the designer's WWR permission based on 80 ksi material in lieu of 60 ksi reinforcing bars

Specifying WWR as a Pre-Approved Equal defers all manufacturing geometries and attributes to the WWR detailer residing downstream of the actual structural design process. This results in minimal disruption to the designer's daily rebar-based structural design routines and affords the contractor the real-time advantage of choice between equivalent mild reinforcement solutions to best suit desired site installation duration and labor savings.

Specifying WWR as a Pre-Approved Equal is generally considered to be a superior method of WWR implementation when compared to Direct Specification, and is made possible by – and maximizes the leveraging of - the presence of the highly qualified engineers and technical personnel who provide the downstream WWR detailing service itself, without compromising the original structural design intent.

4.4 The Role of the Welded Wire Reinforcement Detailer

Both of the previously described methods of specifying welded wire reinforcement require the downstream involvement of welded wire reinforcement detailers. These detailers are most commonly employed directly by the manufacturing companies, though there do exist independent welded wire reinforcement broker-detailers who work in concert with the manufacturer to generate the reinforcement package for the project.

Manufacturers' WWR detailers are required to be familiar with ACI, AASHTO, and ASTM Standards and Specifications that are relevant to structural design, structural reinforcement layout and geometry, and material manufacture. As such, it is extremely common (if not implicitly required by the manufacturer) for the welded wire reinforcing detailer him/herself (or the detailing department's manager) to be a licensed Professional Engineer (PE).

WWR detailers must have the ability to quickly and efficiently review full sets of structural contract documents for the purpose of preparing welded wire reinforcement submittal packages for review and approval during a project's construction administration phase. Submittal packages are not unlike those commonly prepared for loose rebar, though are comparatively far more illustrative given the nature of prefabricated mats of reinforcement versus individual loose bars.

Welded wire reinforcement submittals must include reinforcement material properties, layout and placement plans, reinforcement details, mat piece-marks, and mat quantities. For a project on which the design professional has deployed the Pre-Approved Equal method of specifying WWR, the submittal will also include reinforcement substitution information. This substitution information is necessary to show with full transparency the WWR solution selected to conform with the engineer's contract documents as a replacement for the base design that was presented in terms of loose reinforcing bars. Refer to Figure 4-14 for an example of substitution information submitted to show equivalent WWR used as a replacement for originally detailed reinforcing bars. Chapters 5 through 10 show in greater detail excerpts of the formatting and content of welded wire reinforcement shop drawing submittals.

The welded wire reinforcement detailer will work closely with the contractor (and relevant subcontractors) to ensure timely submittal of the reinforcement package for review. It is increasingly common for the welded wire reinforcement package and loose rebar package to be presented as part of a common submittal to help streamline the review process and to ensure that the project's full reinforcement requirements are duly addressed, minimizing the likelihood of unintentional omissions.

XYZ Welded Wire Reinforcement Manufacturing, Inc.

Welded Wire Reinforcement – Project Conversions Summary

Project: ABC Office Building	**To: Project General Contractor**
Date: June 10, 2021	
By: WWR Designer	**Structural EOR: Project Structural Engineering Firm**

Spread Footing Reinforcement	**WWR Substitution**

Specified Rebar – $f_y = 80$ ksi						
Footing	**Specified Reinforcement**	**Total Reinforcement Area (in²)**	**Hooked Termination?**	**Approximate Bar Spacing Based on Edge Cover (in)**	**Nominal Rebar Tensile Strength (kips) $N_n = A_s \times f_y$**	**WWR ID Utilized**
F80	(7) #6 EW B	3.08	N	15	246.4	WWR-F80
	(0) EW T	0	N/A	N/A	N/A	N/A

Substituted WWR – $f_y = 80$ ksi						
WWR ID	**Mat Type**	**Mats Per Footing**	**Wires in X/Y Directions (For uni-directional mats, Y-Dir non-structural wires)**	**Average Wire Spacing (in)**	**Reinforcement Area Provided, X/Y (in²)**	**Nominal Tensile Strength Provided $N_n = A_s \times f_y$ (kips)**
WWR-F80	B	1	(11) D28.0/ (11) D28.0	9	3.08/3.08	246.4

Mat Type:

A. *Uni-directional* - structural wires in the primary direction; minimal non-structural holding wires at wide intervals in secondary direction, positioned to maintain mat shape.

B. *Bi-directional* - structural reinforcement in both orthogonal directions fabricated on a common WWR mat

Bending: None

Material: ASTM A1064, 80 ksi yield strength

NOTES:

1. See placement drawings for detailed WWR mat description, geometry, and arrangement in the structure.
2. Field trimming of mats is not permitted.

Figure 4-14: Substitution information for WWR as a Pre-Approved Equal. In this example, an isolated spread footing was defined on the structural contract documents to be reinforcement with rebar, but the engineer's pre-approval language allows for the substitution indicated to be made. The burden is on the welded wire reinforcement detailing professional to present proposed substitutions in a clear and concise manner to facilitate the design professional's review for conformance with original design intent.

4.5 Welded Wire Reinforcement as a Non-Proprietary Finished Product

Welded wire reinforcement is a non-proprietary, manufactured product in the same sense that a reinforcing bar or a hot-rolled structural steel beam is. It is required to be produced to satisfy the requirements of the ASTM A1064 Standard Specification, analogous to rebar's required conformance with ASTM A615 or ASTM A706, or a structural steel beam's conformance to ASTM A992. Welded deformed wire reinforcement is referenced explicitly and expansively throughout longstanding industry design standards (ACI 318, AASHTO LRFD Bridge Specification, AREMA, etc.), wherein it is defined as a mild reinforcement for structural concrete. There is no International Code Council Evaluation Service (ICC-ES) report for welded wire reinforcement, as a third-party evaluation of this type would not only be redundant, but at its core unnecessary given the longstanding presence of welded deformed wire reinforcement in the aforementioned cornerstone design standards.

In the past there have been instances of confusion and misrepresentation on the part of jurisdictional authorities and plan review department entities as to what level of added certification might be required for use of welded wire reinforcement in lieu of reinforcing bars. **The project's design professional of record can allay these issues entirely by deploying one (or both) of the specification methods presented in this Guide**. If the design professional her/himself is building welded deformed wire reinforcement into the project's design vocabulary from the start, the plan approval process itself will flow seamlessly without the consternation and confusion that is often associated with other late-breaking "value engineering" proposals that are typically predicated on proprietary product replacements.

4.6 Deferred Submittals

Deferred submittals are elements or assemblies, defined and delegated by the design professional of record, to be designed, detailed, and submitted by a specialty designer, with the deferred submittal's viability being confirmed by EOR approval after issuance of the building permit.

With welded wire reinforcement being specified by the design professional of record as an acceptable and intentional reinforcement for structural concrete, either by Direct Specification or as a Pre-Approved Equal on the structural contract documents themselves, the material is not considered a deferred submittal, even if – in the case of the latter - the extent of its utilization on a project may not be fully defined until after permit issuance.

It is important to understand that welded wire reinforcement is, in theory, not capable of being defined or deferred as a "standalone" or accessory structural element or assembly subject to independent design by others. It is quite literally an implicit component, the presence of which inside hardened concrete produces quantifiable reinforced structural concrete behavior. The EOR, by including welded deformed wire reinforcement as the base reinforcement or as a pre-approved equal reinforcement on the sealed structural contract documents, has acknowledged that the design he or she is presenting is predicated on its acceptability as a mild steel reinforcement, no different than an engineer allowing for either ASTM A615 or A706 reinforcing bars to be utilized in certain applications.

While welded wire reinforcement detailing personnel are commonly licensed as professional engineers, it would be erroneous and misguided to attempt to defer to them any of the project's structural design responsibility, as the detailers themselves are not responsible for deriving the project-specific reinforced concrete design behaviors and interactions, nor are they in a position to make engineering judgments related to same. In contrast to design responsibility, the responsibility for material performance and compliance with ASTM standards is an entirely reasonable and expected demand of the manufacturer. Such a demand would be satisfied through the inclusion of manufacturer-certified mill report results and other material-specific verifications issued as part of the welded wire reinforcement submittal package. Likewise, the accuracy and applicability of reinforcement geometries and layouts are solely the responsibility of the welded wire reinforcement detailer in conjunction with the overseeing contractor.

4.7 WWR Specification as Presented in Guide Chapters 5 Through 10

The Guide uses design and detailing examples as the means by which WWR implementation is best illustrated.

The Direct Specification method is shown in Chapter 6 (Slab-on-Ground) and Chapter 9 (One-Way Post-Tensioned Parking Structure), while Chapter 5 (Shallow Foundations), Chapter 7 (Tilt-Up Wall Panel), Chapter 8 (Cantilever Retaining Wall), and Chapter 10 (Two-Way Mildly Reinforced Slab) utilize the Pre-Approved Equal approach to welded wire reinforcement specification.

Chapter Five

EXAMPLE: Shallow Foundations

EXAMPLE 5 PROBLEM STATEMENT:

The subject structural configuration is comprised of a shallow foundation system. The system employs spread footings for support of primary column loading associated with the concrete frame superstructure, in combination with a perimeter continuous wall footing for support of the building's first floor exterior cladding. A non-structural slab-on-ground terminates over the perimeter wall footing.

The foundations are to be designed per the requirements of ACI 318-19. Column size is 18" x 18" square.

Design Criteria are as follows: $f'_c = 4{,}000\ psi, \beta_1 = 0.85$

$f_y = 80{,}000\ psi$

$q_a = 2{,}000\ psf\ allowable\ soil\ bearing\ pressure$

$Concrete\ Density\ (including\ reinforcement) = 0.150\ kcf, normalweight\ concrete\ \lambda = 1.0$

$Corner\ Spread\ Footing\ Load$

$P_D = 84\ kips$

$P_L = 44\ kips$

$Interior\ Spread\ Footing\ Load$

$P_D = 300\ kips$

$P_L = 150\ kips$

The example includes the following steps:

Step 1 – Corner Spread Footing – Proportioning and Loading
Step 2 – Corner Spread Footing – Two-Way Shear Design
Step 3 – Corner Spread Footing – Flexural Design
Step 4 – Corner Spread Footing – One-Way Shear Design
Step 5 – Corner Spread Footing – WWR Detailing
Step 6 – Interior Spread Footing – Proportioning and Loading
Step 7 – Interior Spread Footing – Two-Way Shear Design
Step 8 – Interior Spread Footing – Flexural Design
Step 9 – Interior Spread Footing – One-Way Shear Design
Step 10 – Interior Spread Footing – WWR Detailing
Step 11 – Wall Footing - Design
Step 12 – Wall Footing – WWR Detailing

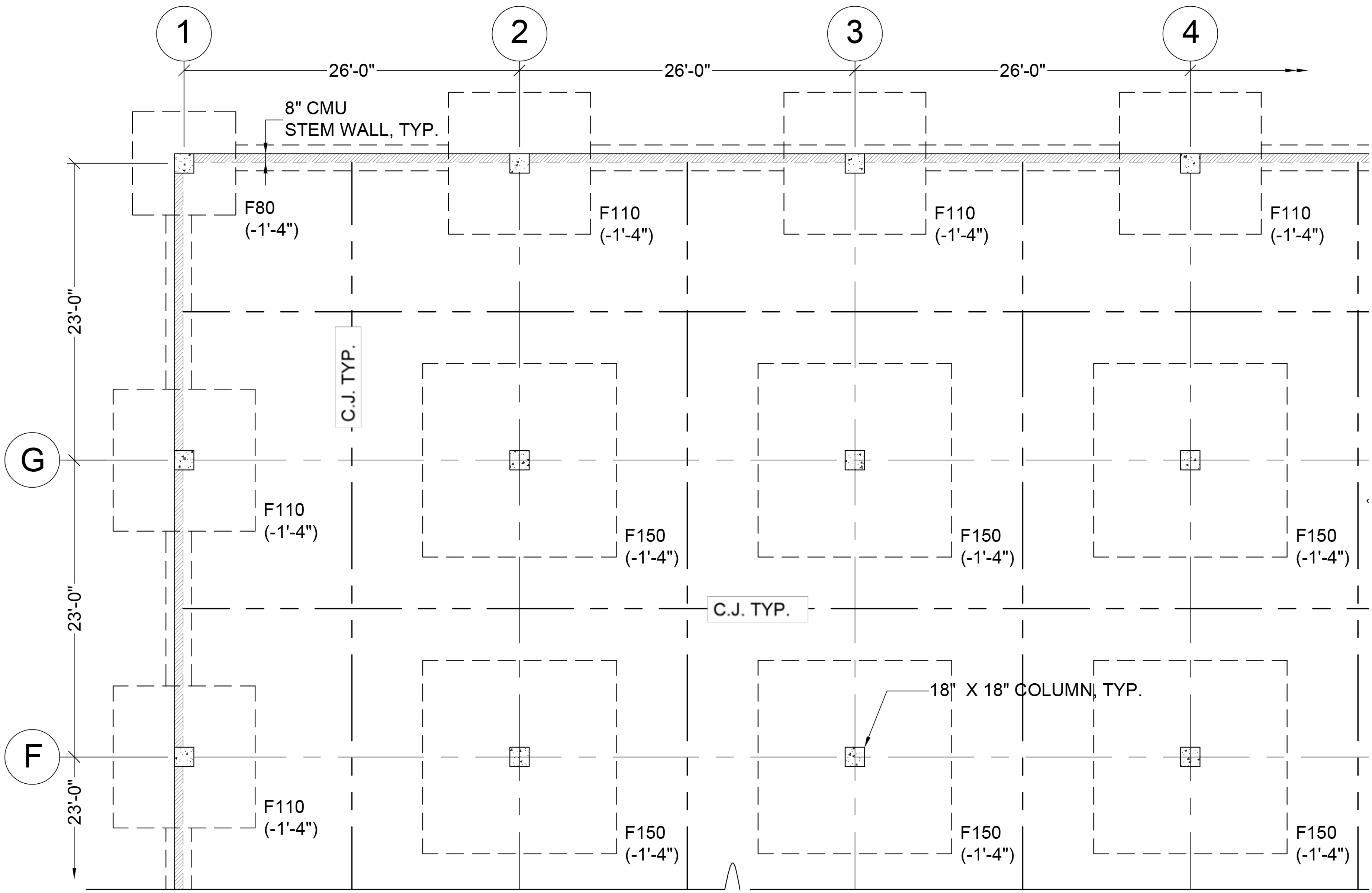


Figure 1: Partial Foundation Plan

ACI 318-19	Calculations	Description
STEP 1: Corner Spread Footing – Proportioning and Loading		
13.3.1.1 5.3.1(b)	$Corner\ Spread\ Footing\ Load$ $P_D = 84\ kips$ $P_L = 44\ kips$ $q_a = 2{,}000\ psf\ allowable\ soil\ bearing\ pressure$ $Required\ Footing\ Area\!: A_{ftg} = \dfrac{P_D + P_L}{q_a} = \dfrac{128\ kips}{2\ ksf} = 64\ ft^2$ $\sqrt{64} = 8$ $\therefore Use\ 8\ foot\ x\ 8\ foot\ spread\ footing.$ $The\ Designer\ selects\ 16\ inches\ as\ trial\ footing\ thickness.$ $U = 1.2D + 1.6L$ $P_u = 1.2 \times 84 + 1.6 \times 44 = 171\ kips$ $q_u = \dfrac{P_u}{A_{ftg}} = \dfrac{171}{64} = 2.67\ ksf \rightarrow strength\ level\ bearing\ pressure$	Spread footing size will be based on a square geometry necessary to maintain service-level soil bearing pressure at or below the defined maximum.

ACI 318-19	Calculations	Description
STEP 2: Corner Spread Footing – Two-Way Shear Design		
13.3.3.1		The design and detailing of two-way isolated square footings shall be in accordance with 13.3.3.2 and the applicable provisions of Chapter 7 and Chapter 8.
	$\boldsymbol{Two\ Way\ Shear\ Design}$	For two-way (punching) shear design, there will be no shear reinforcement.
8.5.1.1(d)	$Need\ \varphi v_n \geq\ v_u$	
8.5.1.2	$\varphi = 0.75$	
22.6.1.2	$v_n = v_c$ $d = 16" - 3" - 0.5" = 12.5"$	Assume 3" clear bottom cover. Assume an additional nominal one-half inch distance to average effective location of the reinforcing mats in both orthogonal directions.
22.6.4.1	$\boldsymbol{Critical\ Section\ Considered\ as\ Interior\ Column\ Condition}$ $b_o = 4 \times (18" + 12.5"/2 + 12.5"/2) = 122"$ $Loaded\ tributary\ area = 8\ ft \times 8\ ft - \dfrac{30.5" \times 30.5"}{144} = 57.54\ sf$ $V_u = 2.67\ ksf \times 57.54\ sf\ = 182\ kips$ $v_{uv} = \dfrac{V_u}{b_o d} = \dfrac{182\ kips}{122" \times 12.5"} = 0.119\ ksi$	This check is based on v_{uv}, the *factored shear stress on the critical section for two-way action*, without moment transfer given this footing is subjected to concentric axial gravity loading only.

ACI 318-19	Calculations	Description
STEP 2: Corner Spread Footing – Two-Way Shear Design (continued)		
22.6.5.2 22.6.5.3 13.2.6.2 8.4.1.9	$v_c\ is\ the\ least\ of:$ $v_c = 4\lambda_s\lambda\sqrt{f'_c} = 4 \times 1.0 \times 1.0 \times \sqrt{4000} = 0.253\ ksi\ \leftarrow$ $v_c = \left(2 + \frac{4}{\beta}\right)\lambda_s\lambda\sqrt{f'_c} = 6 \times 1.0 \times 1.0 \times \sqrt{4000} = 0.379\ ksi$ $\beta\ is\ the\ ratio\ of\ column\ sides.\ \beta = 1\ for\ square\ column.$ $v_c = \left(2 + \frac{\alpha_s d}{b_o}\right)\lambda_s\lambda\sqrt{f'_c} = \left(2 + \frac{40 \times 12.5}{122}\right) \times 1 \times 1 \times \sqrt{4000} = 0.386\ ksi$ $\alpha_s = 40\ for\ interior\ columns$ $\therefore v_c = 0.253\ ksi$ $v_{uv} = 0.119\ ksi < 0.75 \times 0.253 = 0.190\ ksi$ $\varphi v_n \geq\ v_u\ is\ satisfied$ $\therefore \boldsymbol{Two\ way\ (punching) shear\ capacity\ is\ sufficient.}$	For two-way isolated footings it is permissible to neglect the size effect factor specified in 22.6. therefore, $\lambda_s =$ 1.0.

ACI 318-19	Calculations	Description
STEP 3: Corner Spread Footing – Flexural Design		
13.2.7.1 21.2.2 8.6.1.1	$For\ concentric\ axially\ loaded\ square\ footing:$ $M_u = q_u \times b \times \frac{(l-c)^2}{8}$ $b = 8\ feet\ (footing\ dimension\ perpendicular\ to\ flexure)$ $l = 8\ feet\ (footing\ dimension\ parallel\ to\ flexure)$ $c = 1.5\ feet\ (column\ dimension\ parallel\ to\ flexure)$ $M_u = 2.67\ ksf \times 8\ ft \times \frac{(6.5\ ft)^2}{8} = 113\ k-ft$ $Assume\ tension-controlled\ section\ to\ start: \varphi = 0.90$ $m = \frac{f_y}{0.85 \times f'_c} = \frac{80{,}000}{0.85 \times 4{,}000} = 23.53$ $R_u = \frac{M_u}{\varphi b d^2} = \frac{113 \times 12}{0.9 \times 8 \times 12 \times 12.5^2} = 0.101\ ksi$ $\rho_{reqd} = \frac{1}{m} \times \left(1 - \sqrt{1 - \frac{2mR_u}{f_y}}\right) = 0.0013$ $\rho_{reqd} = 0.0013$ $A_{s,reqd} = 0.0013 \times 96" \times 12.5" = 1.56\ in^2$ $A_{s,\min prescriptive} = 0.0018 \times A_g = 0.0018 \times 96" \times 16" = 2.77\ in^2$	Calculate M_u at the face of the column.

ACI 318-19	Calculations	Description
STEP 3: Corner Spread Footing – Flexural Design (continued)		
8.6.1.2 7.7.2.4 21.2 22.2	$Is\ v_{uv} = 0.119\ ksi > \varphi \times 2 \times \lambda_s \times \lambda \times \sqrt{f'_c}\ ?$ $\varphi \times 2 \times \lambda_s \times \lambda \times \sqrt{f'_c} = 0.75 \times 2 \times 1.0 \times 1.0 \times \sqrt{4{,}000} = 0.095\ ksi$ $0.119\ ksi > 0.095\ ksi$ $\therefore Must\ consider\ flexure\ driven\ punching\ failure.$ $provide\ A_{s,\min FDPS}\ to\ resist\ flexure\ driven\ punching\ shear!$ $A_{s,\min FDPS} = \frac{5v_{uv}b_{slab}b_o}{\varphi\alpha_s f_y} = \frac{5 \times 0.119\ ksi \times 96" \times 122"}{0.75 \times 40 \times 80\ ksi} = 2.90\ in^2 \leftarrow$ $Maximum\ spacing\ shall\ be\ the\ lesser\ of:$ • $2h = 2 \times 16\ inches = 32\ inches$ • $18\ inches \leftarrow$ $\boldsymbol{The\ Design\ Professional\ selects\ (7)\ \#6\ reinforcing\ bars.}$ $A_{s,prov} = 3.08\ in^2, (7)\ bars\ spaced\ @\ \pm 15"\ on\ center\ with\ 3"\ cover$ $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{3.08 \times 80}{0.85 \times 4 \times 96} = 0.76"$ $\varepsilon_t = \frac{0.003 \times \beta_1 \times d}{a} - 0.003 = \frac{0.003 \times 0.85 \times 12.5}{0.76} - 0.003 = 0.039\ in/in$ $\varepsilon_t = 0.039 >> \varepsilon_{ty} + 0.003 = 0.0058 \therefore Tension\ Controlled$ $\varphi M_n = 0.9 A_s f_y (d - a/2) = 224\ k-ft > 113\ k-ft$ $\boldsymbol{\therefore Flexural\ capacity\ is\ sufficient.}$	The design professional must calculate the required flexural steel area with due consideration for potential flexure-driven punching failure, and also must maintain prescriptive maximum bar spacing. Section 8.6.1.2 was derived in part from tests on interior column-to-elevated slab connections with lightly reinforced slabs. Because there is currently no language ACI 318 Chapter 13 precluding this provision from being applied to two-way isolated footings, we are showing it as a required check in this example.

ACI 318-19	Calculations	Description
STEP 3: Corner Spread Footing – Flexural Design (continued)		
13.2.8.3 25.4.2.4	$l_d = \left(\frac{3}{40} \times \frac{f_y}{\lambda\sqrt{f'_c}} \times \frac{\psi_t \psi_e \psi_s \psi_g}{\frac{c_b + K_{tr}}{d_b}}\right) \times d_b$ $f_y = 80{,}000\ psi$ $f'_c = 4{,}000\ psi$ $\lambda = 1.0\ (normalweight\ concrete)$ $\psi_t = 1.0\ (bottom\ reinforcement\ with\ less\ than\ 12"\ concrete\ below)$ $\psi_e = 1.0\ (uncoated\ reinforcement)$ $\psi_s = 1.0\ (\#6\ bar\ and\ smaller\ bars\ and\ deformed\ wires)$ $\psi_g = 1.15\ (Grade\ 80\ reinforcement)$ $c_b = 3\text{ cover} + \text{half bar diameter} = 3.375"$ $K_{tr} = 0\ (permitted\ as\ a\ design\ simplification)$ $d_b = 0.750"$ $\frac{c_b + K_{tr}}{d_b} = \frac{3.375" + 0}{0.750} = 4.5 > 2.5\ \therefore use\ 2.5$ $l_d = \left(\frac{3}{40} \times \frac{80{,}000}{1.0 \times \sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 1.0 \times 1.15}{2.5}\right) \times 0.750 = 33"$ $Bars\ will\ run\ full\ dimension\ of\ footing, minus\ 3"\ cover\ each\ end.$ $Bar\ length\ beyond\ column\ face = \frac{96"}{2} - \frac{18"}{2} - 3" = 36\ inches$ $36"\text{ extension} > 33"\text{ required}$ $\therefore \boldsymbol{Flexural\ reinforcement\ is\ sufficiently\ developed.}$	Confirm that the reinforcement is properly developed past the face of the column. This example shows the common practice of reinforcement design and selection based on the use of deformed reinforcing bars (rebar). Refer to Figure 2 for permissive language in the General Notes section of the contract drawings that allows welded wire reinforcement substitutions for certain project reinforcement applications. This method of specifying WWR as a pre-approved equal not only satisfies the design intent of the Engineer of Record but allows contractor consideration for and implementation of potentially beneficial construction and installation considerations downstream of the engineer's design phase.

MILD REINFORCING STEEL

1. TYPICAL DEFORMED REINFORCING BARS (REBAR) SHALL CONFORM TO ASTM A615, GRADE 80. BARS SHALL BE LAPPED IN ACCORDANCE WITH THE REBAR LAP SCHEDULE UNLESS OTHERWISE EXPLICITLY DETAILED.

2. LONGITUDINAL REINFORCEMENT IN SPECIAL MOMENT FRAME BEAMS AND COLUMNS, AND VERTICAL AND HORIZONTAL REINFORCEMENT IN SPECIAL STRUCTURAL (SHEAR) WALLS SHALL BE ASTM A706 GRADE 60 OR GRADE 80 AS NOTED. TENSILE AND ELONGATION PROPERTIES SHALL BE CONFIRMED THROUGH MILL REPORT DOCUMENTATION PROVIDED AS PART OF THE PROJECT REINFORCEMENT SUBMITTAL

3. WELDED DEFORMED WIRE REINFORCEMENT SHALL CONFORM TO ASTM A1064 GRADE 80 AND SHALL BE PROVIDED IN SHEET FORM. REINFORCEMENT SHEETS SHALL BE MANUFACTURED WITH OVERHANG LENGTHS SUFFICIENT TO ACHIEVE A LAP SPLICE LENGTH EQUAL TO THE GREATER OF 12 INCHES OR THE LAP SPLICE DIMENSION SHOWN IN THE REBAR LAP SCHEDULE FOR BAR OF EQUAL (OR GREATER) DIAMETER AND GRADE, UNLESS OTHERWISE NOTED. SHEETS AND ASSOCIATED LAP REGIONS SHALL BE INSTALLED COPLANAR SO AS TO NOT "STACK".

4. WELDED DEFORMED WIRE REINFORCEMENT OF EQUAL AREA, EQUAL OR LESSER SPACING, AND IDENTICAL CURTAILMENT (HOOKS AND LAP SPLICES) IS PERMITTED AS A SUBSTITUTION FOR DEFORMED REINFORCING BARS, EXCEPT IN THE FOLLOWING STRUCTURAL APPLICATIONS:

   A. LONGITUDINAL STEEL IN SPECIAL MOMENT FRAMES

   B. VERTICAL AND HORIZONTAL STEEL IN SPECIAL STRUCTURAL WALLS

Figure 2: Excerpt from contract documents showing the design professional's permissive WWR substitution language

ACI 318-19	Calculations	Description
STEP 4: Corner Spread Footing – One-Way Shear Design		
	One Way Shear Design	One-way shear design can be carried out now that the flexural steel reinforcement amount has been defined.
7.5.1.1	$Need\ \varphi V_n \geq V_u$	
7.5.3.1	$V_n = V_c + V_s;\ V_s = 0\ due\ to\ no\ shear\ reinforcement$	
22.5.1.1	$\therefore V_n = V_c$	
7.4.3.2	$V_u = q_u \times b \times \left(\left(\frac{l-c}{2}\right) - d\right)$ $b = 8\ feet\ (footing\ dimension\ parallel\ to\ shear\ line)$ $l = 8\ feet\ (footing\ dimension\ perpendicular\ to\ shear\ line)$ $c = 1.5\ feet\ (column\ dimension\ perpendicular\ to\ shear\ line)$ $d = 12.5\ inches = 1.042\ feet$ $V_u = 2.67\ ksf \times 8\ feet \times \left(\left(\frac{8-1.5}{2}\right) - 1.042\right) = 47\ kips$	Calculate required shear strength along a shear line located a distance “d” from the face of the column.
22.5.5.1	$A_v = 0, \therefore A_v < A_{v,min}$ $V_c = \left[8\lambda_s\lambda\left(\rho_w{}^{1/3}\right)\sqrt{f'_c} + \frac{N_u}{6A_g}\right] \times b_w d \leq 5\lambda\sqrt{f'_c} b_w d$ $\rho_w = \frac{A_s}{b_w d} = \frac{3.08}{96 \times 12.5} = 0.0026$ $V_c = \left[8 \times 1.0 \times 1.0 \times 0.0026^{1/3} \times \sqrt{4000} + 0\right] \times 96 \times 12.5 = 83.5\ kips$	
22.5.5.1.1	$V_c = 83.5\ kips < 5 \times 1.0 \times \sqrt{4000} \times 96 \times 12.5 = 379\ kips$	Slab proportioning/depth is appropriate.

ACI 318-19	Calculations	Description
STEP 4: Corner Spread Footing – One-Way Shear Design (continued)		
8.5.1.1 22.5.1.2	$Need\ \varphi V_n \geq V_u$ $0.75 \times 83.5\ kips = 62.6\ kips > V_u = 47\ kips\ \therefore OK$ $Also\ need\ V_u \leq \varphi\left(V_c + 8\sqrt{f'_c}b_w d\right) \rightarrow 47\ kips < 518\ kips\ \therefore OK$ $\therefore \boldsymbol{One\ way\ shear\ capacity\ is\ sufficient.}$	Check to confirm likelihood of diagonal compression failure is precluded. **With sectional strength calculations for flexure and shear completed, refer to Figure 3 and Figure 4 for a representation of how this information is typically presented on the Contract Documents.**

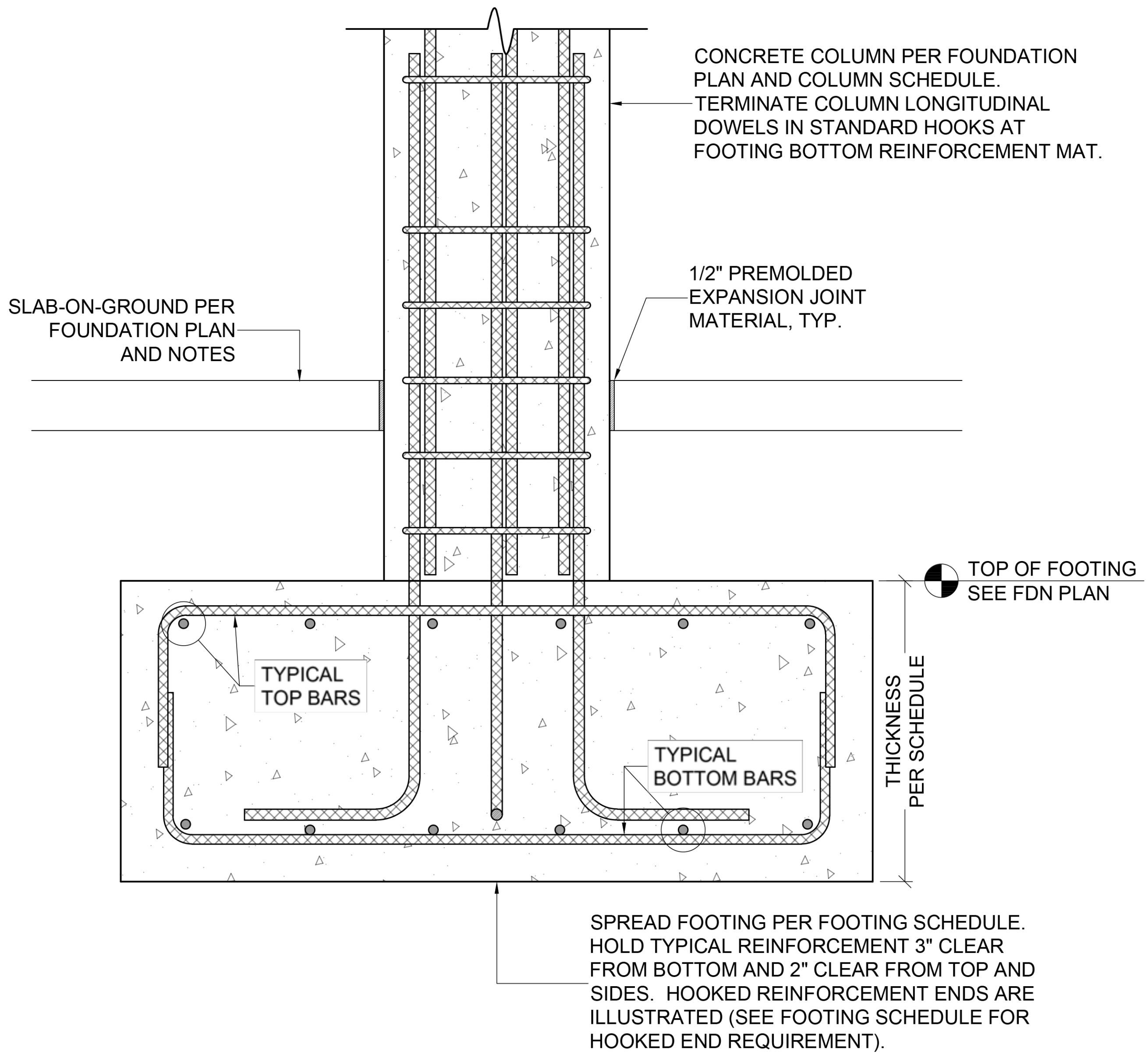


Figure 3: Typical spread footing detail as presented on contract documents

SPREAD FOOTING SCHEDULE			
MARK	SIZE	THICK	TYPICAL REINFORCING
F80	8'-0" x 8'-0"	1'-4"	(7) #6 E.W. BOTT

SPREAD FOOTING SCHEDULE NOTES

1. FOOTING SHALL BE PER TYPICAL SPREAD FOOTING DETAILS.
   HOOKED BAR ENDS ARE DENOTED THUS #4H, #5H, #6H, ETC.
2. E.W. = EACH WAY, S.W. = SHORT WAY; L.W. = LONG WAY

Figure 4: Spread footing schedule as presented on contract documents

ACI 318-19	Calculations	Description
STEP 5: Corner Spread Footing – WWR Detailing		
		The WWR detailer, prompted by the contractor's preference and based on permissive WWR language presented on the contract documents by the design professional , generates a WWR alternative contingent upon his/her familiarity with WWR manufacturing and transport capabilities and the reinforcement's compliance with the ACI 318-19 Standard. In effect, the burden is on the WWR manufacturer's detailer – on behalf of the contractor and associated subcontractors - to confirm acceptability of the substitution through verifying information in the submittal that the design professional's structural intent is not altered or compromised. While this confirmation could be as simple as documented acceptance through electronic mail correspondence, it is more often comprised of a formalized process by which the use of WWR as a substitution is confirmed first through Request for Information (RFI) correspondences, and then followed up by appropriate addendum to the contract drawings. Such an addendum is often in the form of the manufacturer's submittal information itself. An example of a manufacturer's submittal is shown in Figure 5 and would be an accompaniment to the actual illustrative welded wire reinforcement shop drawings submitted for review.

XYZ Welded Wire Reinforcement Manufacturing, Inc.

Welded Wire Reinforcement – Project Conversions Summary

Project: ABC Office Building	**To: Project General Contractor**
Date: June 10, 2021	
By: WWR Designer	**Structural EOR: Project Structural Engineering Firm**

Spread Footing Reinforcement	**WWR Substitution**

Specified Rebar – f_y = 80 ksi							
Footing	**Specified Reinforcement**	**Total Reinforcement Area (in²)**	**Hooked Termination?**	**Approximate Bar Spacing Based on Edge Cover (in)**	**Nominal Rebar Tensile Strength (kips) $N_n = A_s \times f_y$**	**WWR ID Utilized**
F80	(7) #6 EW B	3.08	N	15	246.4	WWR-F80
	(0) EW T	0	N/A	N/A	N/A	N/A

Substituted WWR – f_y = 80 ksi						
WWR ID	**Mat Type**	**Mats Per Footing**	**Wires in X/Y Directions (For uni-directional mats, Y-Dir non-structural wires)**	**Average Wire Spacing (in)**	**Reinforcement Area Provided, X/Y (in²)**	**Nominal Tensile Strength Provided $N_n = A_s \times f_y$ (kips)**
WWR-F80	B	1	(11) D28.0/ (11) D28.0	9	3.08/3.08	246.4

Mat Type:

A. *Uni-directional* - structural wires in the primary direction; minimal non-structural holding wires at wide intervals in secondary direction, positioned to maintain mat shape.

B. *Bi-directional* - structural reinforcement in both orthogonal directions fabricated on a common WWR mat

Bending: None

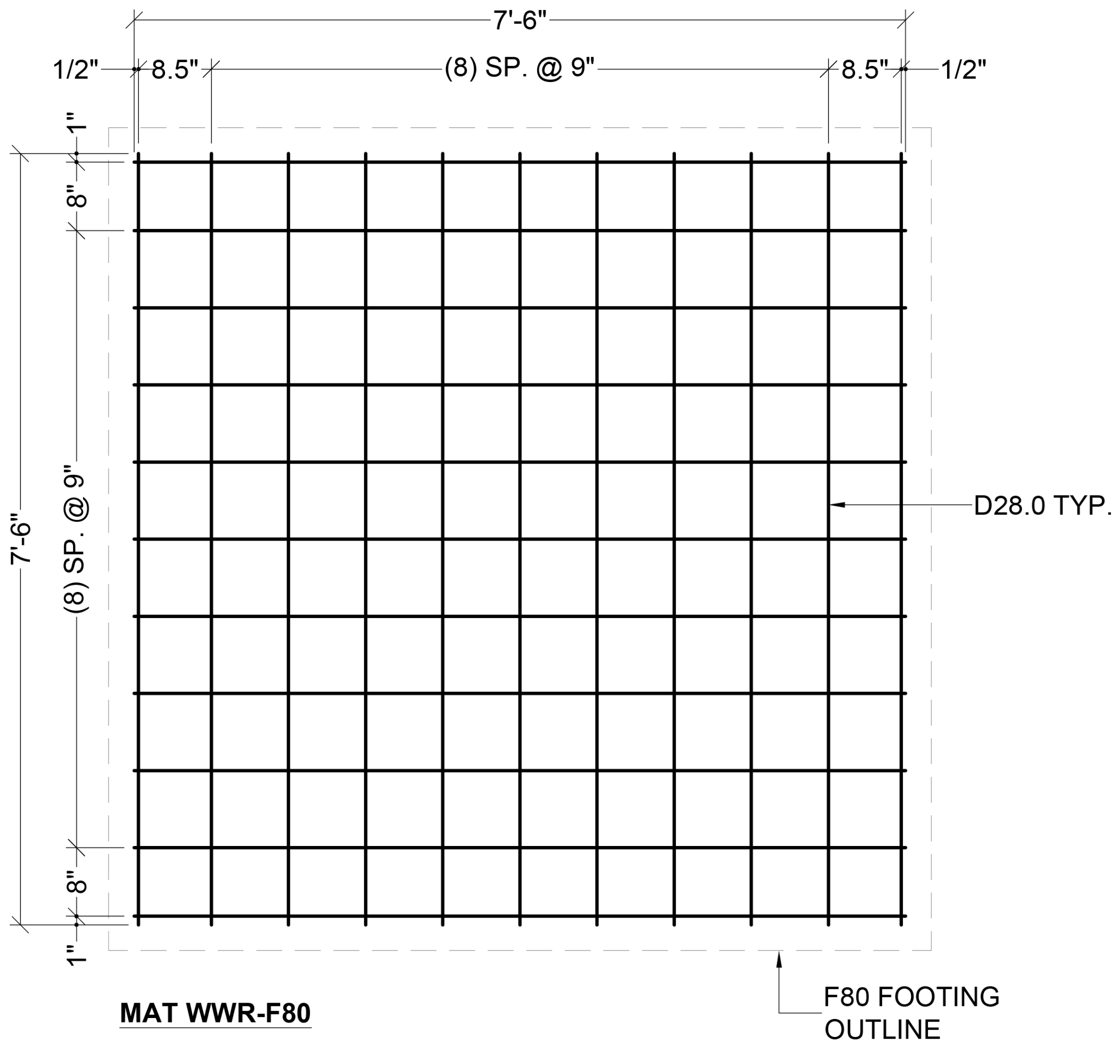
Material: ASTM A1064, 80 ksi yield strength

NOTES:

1. See placement drawings for detailed WWR mat description, geometry, and arrangement in the structure.
2. Field trimming of mats is not permitted.

Figure 5: Manufacturer's submittal information showing WWR substitution

ACI 318-19	Calculations	Description
STEP 5: Corner Spread Footing – WWR Detailing (continued)		
Table 20.2.2.4(a) 25.4.2.4	$A_{s,provided} = 3.08\ in^2 = (7)\#6\ bars\ @\ \pm 15"\ on\ center\ each\ way$ $WWR\ options\ providing\ 3.08\ in^2\ steel\ area\ include:$ $(10)\ D30.8\ wires\ @\ \pm 10"\ on\ center\ each\ way$ $(11)\ D28.0\ wires\ @\ \pm 9"\ on\ center\ each\ way \leftarrow\ selected\ by\ detailer$ $(12)\ D25.7\ wires\ @\ \pm 8"\ on\ center\ each\ way$ $\frac{c_b + K_{tr}}{d_b} = \frac{3.299" + 0}{0.597} = 5.53 > 2.5\ \therefore use\ 2.5$ $l_d = \left(\frac{3}{40} \times \frac{80{,}000}{1.0 \times \sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 1.0 \times 1.15}{2.5}\right) \times 0.597 = 26"$ $Wires\ will\ run\ full\ dimension\ of\ footing, minus\ 3"\ cover\ each\ end.$ $Wire\ length\ beyond\ column\ face = \frac{96"}{2} - \frac{18"}{2} - 3" = 36\ inches$ 36" extension > 26" required $\therefore \boldsymbol{Flexural\ reinforcement\ is\ sufficiently\ developed.}$	Convert the Grade 80 reinforcing bar pattern used as part of the base design into a Grade 80 WWR substitution. The numerical basis for this is presented in Figure 5. Note that the use of smaller diameter wires at closer spacing is checked to confirm development requirements originally checked for rebar in Step 3. The resulting WWR mat assembly is shown in Figure 6. The WWR mat configurations shown here are intended to serve as a general representation of manufacturer capabilities throughout North America. With that said, reinforcement arrangements presented on a submittal by one manufacturer's detailer may vary slightly from those presented by another. Despite this, there exists a common and consistent responsibility for the WWR detailer to ensure that detailed WWR arrangements conform to the reinforcement cross-sectional area, placement, and curtailment requirements established by the design professional, unless explicitly-defined exceptions are made by the design professional of record on a project by project basis.



MAT WWR-F80

V1 X V2 D28/D28 89" (+0.5",+0.5") X 7'-6" (1,1)
V1 = 0.5 OH, 8.5, 8 @ 9, 8.5, 0.5 OH
V2 = 1 OH, 8, 8 @ 9, 8, 1 OH

Figure 6: Representation of WWR for footing F80, as would be presented on a set of reinforcement shop/placement drawings

ACI 318-19	Calculations	Description
STEP 6: Interior Spread Footing – Proportioning and Loading		
13.3.1.1 5.3.1(b)	$Interior\ Spread\ Footing\ Load$ $P_D = 300\ kips$ $P_L = 150\ kips$ $q_a = 2{,}000\ psf\ allowable\ soil\ bearing\ pressure$ $Required\ Footing\ Area\text{:}\ A_{ftg} = \dfrac{P_D + P_L}{q_a} = \dfrac{450\ kips}{2\ ksf} = 225\ ft^2$ $\sqrt{225} = 15$ $\therefore Use\ 15\ foot\ x\ 15\ foot\ spread\ footing.$ $The\ Designer\ selects\ 28\ inches\ as\ trial\ footing\ thickness.$ $U = 1.2D + 1.6L$ $P_u = 1.2 \times 300 + 1.6 \times 150 = 600\ kips$ $q_u = \dfrac{P_u}{A_{ftg}} = \dfrac{600}{225} = 2.67\ ksf \rightarrow strength\ level\ bearing\ pressure$	Spread footing size will be based on a square geometry necessary to maintain service-level soil bearing pressure at or below the defined maximum.

ACI 318-19	Calculations	Description
STEP 7: Interior Spread Footing – Two-Way Shear Design		
13.3.3.1		The design and detailing of two-way isolated square footings shall be in accordance with 13.3.3.2 and the applicable provisions of Chapter 7 and Chapter 8.
	$\boldsymbol{Two\ Way\ Shear\ Design}$	For two-way (punching) shear design, there will be no shear reinforcement.
8.5.1.1(d)	$Need\ \varphi v_n \geq\ v_u$	
8.5.1.2	$\varphi = 0.75$	
22.6.1.2	$v_n = v_c$ $d = 28" - 3" - 0.5" = 24.5"$	Assume 3" clear bottom cover. Assume an additional nominal one-half inch distance to average effective location of the reinforcing mats in both orthogonal directions.
22.6.4.1	$\boldsymbol{Critical\ Section\ Considered\ as\ Interior\ Column\ Condition}$ $b_o = 4 \times (18" + 24.5"/2 + 24.5"/2) = 170"$ $Loaded\ tributary\ area = 15\ ft \times 15\ ft - \frac{42.5" \times 42.5"}{144} = 212.5\ sf$ $V_u = 2.67\ ksf \times 212.5\ sf\ = 567\ kips$ $v_{uv} = \frac{V_u}{b_o d} = \frac{567\ kips}{170" \times 24.5"} = 0.136\ ksi$	This check is based on v_{uv}, the *factored shear stress on the critical section for two-way action*, <u>without moment transfer</u> given this footing is subjected to concentric axial gravity loading only.

ACI 318-19	Calculations	Description
STEP 7: Interior Spread Footing – Two-Way Shear Design (continued)		
22.6.5.2 22.6.5.3 13.2.6.2 8.4.1.9	v_c *is the least of*: $v_c = 4\lambda_s\lambda\sqrt{f'_c} = 4 \times 1.0 \times 1.0 \times \sqrt{4000} = 0.253\ ksi\ \leftarrow$ $v_c = \left(2 + \frac{4}{\beta}\right)\lambda_s\lambda\sqrt{f'_c} = 6 \times 1.0 \times 1.0 \times \sqrt{4000} = 0.379\ ksi$ β *is the ratio of column sides.* $\beta = 1$ *for square column.* $v_c = \left(2 + \frac{\alpha_s d}{b_o}\right)\lambda_s\lambda\sqrt{f'_c} = \left(2 + \frac{40 \times 20.5}{154}\right) \times 1 \times 1 \times \sqrt{4000} = 0.463\ ksi$ $\alpha_s = 40$ *for interior columns* $\therefore v_c = 0.253\ ksi$ $v_{uv} = 0.136\ ksi < 0.75 \times 0.253 = 0.190\ ksi$ $\varphi v_n \geq v_u$ *is satisfied* $\therefore$ ***Two way (punching) shear capacity is sufficient.***	For two-way isolated footings it is permissible to neglect the size effect factor specified in 22.6. therefore, $\lambda_s =$ 1.0.

ACI 318-19	Calculations	Description
STEP 8: Interior Spread Footing – Flexural Design		
13.2.7.1 21.2.2	$For\ concentric\ axially\ loaded\ square\ footing:$ $M_u = q_u \times b \times \frac{(l-c)^2}{8}$ $b = 15\ feet\ (footing\ dimension\ perpendicular\ to\ flexure)$ $l = 15\ feet\ (footing\ dimension\ parallel\ to\ flexure)$ $c = 1.5\ feet\ (column\ dimension\ parallel\ to\ flexure)$ $M_u = 2.67\ ksf \times 15\ ft \times \frac{(13.5\ ft)^2}{8} = 913\ k-ft$ $Assume\ tension-controlled\ section\ to\ start: \varphi = 0.90$ $m = \frac{f_y}{0.85 \times f'_c} = \frac{80{,}000}{0.85 \times 4{,}000} = 23.53$ $R_u = \frac{M_u}{\varphi b d^2} = \frac{913 \times 12}{0.9 \times 15 \times 12 \times 24.5^2} = 0.113\ ksi$ $\rho_{reqd} = \frac{1}{m} \times \left(1 - \sqrt{1 - \frac{2mR_u}{f_y}}\right) = 0.0015$	Calculate M_u at the face of the column.
  8.6.1.1	$\rho_{reqd} = 0.0015$ $A_{s,reqd} = 0.0015 \times 180" \times 24.5" = 6.62\ in^2$ $A_{s,\min prescriptive} = 0.0018 \times A_g = 0.0018 \times 180" \times 28" = 9.10\ in^2 \leftarrow$	

ACI 318-19	Calculations	Description
STEP 8: Interior Spread Footing – Flexural Design (continued)		
8.6.1.2 7.7.2.4 21.2 22.2	$Is\ v_{uv} = 0.136\ ksi > \varphi \times 2 \times \lambda_s \times \lambda \times \sqrt{f'_c}\ ?$ $\varphi \times 2 \times \lambda_s \times \lambda \times \sqrt{f'_c} = 0.75 \times 2 \times 1.0 \times 1.0 \times \sqrt{4{,}000} = 0.095\ ksi$ $0.136\ ksi > 0.095\ ksi$ $\therefore Must\ consider\ flexure\ driven\ punching\ failure.$ $provide\ A_{s,\min FDPS}\ to\ resist\ flexure\ driven\ punching\ shear!$ $A_{s,\min FDPS} = \frac{5 v_{uv} b_{slab} b_o}{\varphi \alpha_s f_y} = \frac{5 \times 0.136\ ksi \times 180" \times 170"}{0.75 \times 40 \times 80\ ksi} = 8.67\ in^2$ $Maximum\ spacing\ shall\ be\ the\ lesser\ of:$ • $2h = 2 \times 16\ inches = 32\ inches$ • $18\ inches \leftarrow$ $\boldsymbol{The\ Design\ Professional\ selects\ (16)\ \#7\ reinforcing\ bars.}$ $A_{s,prov} = 9.60\ in^2, (16)\ bars\ spaced\ @\ \pm 11.6"\ on\ center\ with\ 3"\ cover$ $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{9.60 \times 80}{0.85 \times 4 \times 180} = 1.26"$ $\varepsilon_t = \frac{0.003 \times \beta_1 \times d}{a} - 0.003 = \frac{0.003 \times 0.85 \times 24.5}{1.26} - 0.003 = 0.047\ in/in$ $\varepsilon_t = 0.047 >> \varepsilon_{ty} + 0.003 = 0.0058 \therefore Tension\ Controlled$ $\varphi M_n = 0.9 A_s f_y (d - a/2) = 1{,}375\ k - ft > 913\ k - ft$ $\boldsymbol{\therefore Flexural\ capacity\ is\ sufficient.}$	Per ACI 318-19 Section 8.6.1.2, the design professional must calculate the required flexural steel area with due consideration for potential flexure-driven punching failure, and also must maintain prescriptive maximum bar spacing. Section 8.6.1.2 was derived in part from tests on interior column-to-elevated slab connections with lightly reinforced slabs. Because there is currently no language ACI 318 Chapter 13 precluding this provision from being applied to two-way isolated footings, we are showing it as a required check in this example.

ACI 318-19	Calculations	Description
STEP 8: Interior Spread Footing – Flexural Design (continued)		
13.2.8.3 25.4.2.4	$l_d = \left(\frac{3}{40} \times \frac{f_y}{\lambda\sqrt{f'_c}} \times \frac{\psi_t\psi_e\psi_s\psi_g}{\frac{c_b + K_{tr}}{d_b}}\right) \times d_b$ $f_y = 80{,}000\ psi$ $f'_c = 4{,}000\ psi$ $\lambda = 1.0\ (normalweight\ concrete)$ $\psi_t = 1.0\ (bottom\ reinforcement\ with\ less\ than\ 12"\ concrete\ below)$ $\psi_e = 1.0\ (uncoated\ reinforcement)$ $\psi_s = 1.0\ (\#6\ bar\ and\ smaller\ bars\ and\ deformed\ wires)$ $\psi_g = 1.15\ (Grade\ 80\ reinforcement)$ $c_b = 3\ \text{cover} + \text{half bar diameter} = 3.44"$ $K_{tr} = 0\ (permitted\ as\ a\ design\ simplification)$ $d_b = 0.875"$ $\frac{c_b + K_{tr}}{d_b} = \frac{3.44" + 0}{0.875} = 3.93 > 2.5\ \therefore use\ 2.5$ $l_d = \left(\frac{3}{40} \times \frac{80{,}000}{1.0 \times \sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 1.0 \times 1.15}{2.5}\right) \times 0.875 = 39"$ $Bars\ will\ run\ full\ dimension\ of\ footing, minus\ 3"\ cover\ each\ end.$ $Bar\ length\ beyond\ column\ face = \frac{180"}{2} - \frac{18"}{2} - 3" = 78\ inches$ $78"\ \text{extension} > 39"\ \text{required}$ $\therefore \boldsymbol{Flexural\ reinforcement\ is\ sufficiently\ developed.}$	Confirm that the reinforcement is properly developed past the face of the column. This example shows the common practice of reinforcement design and selection based on the use of deformed reinforcing bars (rebar). Refer to Figure 2 for permissive language in the General Notes section of the contract drawings that allows welded wire reinforcement substitutions for certain project reinforcement applications. This method of specifying WWR as a pre-approved equal not only satisfies the design intent of the Engineer of Record but allows contractor consideration for and implementation of potentially beneficial construction and installation considerations downstream of the engineer's design phase.

ACI 318-19	Calculations	Description
STEP 9: Interior Spread Footing – One-Way Shear Design		
	$\boldsymbol{One\ Way\ Shear\ Design}$	One-way shear design can be carried out now that the flexural steel reinforcement amount has been defined.
7.5.1.1	$Need\ \varphi V_n \geq V_u$	
7.5.3.1	$V_n = V_c + V_s;\ V_s = 0\ due\ to\ no\ shear\ reinforcement$	
22.5.1.1	$\therefore V_n = V_c$	Calculate required shear strength along a shear line located a distance "d" from the face of the column.
7.4.3.2	$V_u = q_u \times b \times \left(\left(\frac{l-c}{2}\right) - d\right)$ $b = 15\ feet\ (footing\ dimension\ parallel\ to\ shear\ line)$ $l = 15\ feet\ (footing\ dimension\ perpendicular\ to\ shear\ line)$ $c = 1.5\ feet\ (column\ dimension\ perpendicular\ to\ shear\ line)$ $d = 24.5\ inches = 2.042\ feet$ $V_u = 2.67\ ksf \times 15\ feet \times \left(\left(\frac{15-1.5}{2}\right) - 2.042\right) = 189\ kips$	
22.5.5.1	$A_v = 0, \therefore A_v < A_{v,min}$ $V_c = \left[8\lambda_s\lambda\left(\rho_w{}^{1/3}\right)\sqrt{f'_c} + \frac{N_u}{6A_g}\right] \times b_w d \leq 5\lambda\sqrt{f'_c} b_w d$ $\rho_w = \frac{A_s}{b_w d} = \frac{9.60}{180 \times 24.5} = 0.0022$	
22.5.5.1.1	$V_c = \left[8 \times 1 \times 1 \times 0.0022^{1/3} \times \sqrt{4000} + 0\right] \times 180 \times 24.5 = 290\ kips$ $V_c = 290\ kips < 5 \times 1.0 \times \sqrt{4000} \times 180 \times 24.5 = 1{,}395\ kips$	Slab proportioning/depth is appropriate.

ACI 318-19	Calculations	Description
STEP 9: Interior Spread Footing – One-Way Shear Design (continued)		
8.5.1.1 22.5.1.2	$Need\ \varphi V_n \geq V_u$ $0.75 \times 290\ kips = 218\ kips > V_u = 189\ kips\ \therefore OK$ $Also\ need\ V_u \leq \varphi\left(V_c + 8\sqrt{f'_c}b_w d\right) \rightarrow 189\ kips < 2{,}521\ kips\ \therefore OK$ $\therefore \boldsymbol{One\ way\ shear\ capacity\ is\ sufficient.}$	 Check to confirm likelihood of diagonal compression failure is precluded. **With sectional strength calculations for flexure and shear completed, refer to Figure 3 and Figure 7 for a representation of how this information is typically presented on the Contract Documents.**

SPREAD FOOTING SCHEDULE			
MARK	SIZE	THICK	TYPICAL REINFORCING
F80	8'-0" x 8'-0"	1'-4"	(7) #6 E.W. BOTT
F150	15'-0" x 15'-0"	2'-4"	(16) #7 E.W. BOTT

SPREAD FOOTING SCHEDULE NOTES

1. FOOTING SHALL BE PER TYPICAL SPREAD FOOTING DETAILS. HOOKED BAR ENDS ARE DENOTED THUS #4H, #5H, #6H, ETC.
2. E.W. = EACH WAY, S.W. = SHORT WAY; L.W. = LONG WAY

Figure 7: Spread footing schedule as presented on contract documents

ACI 318-19	Calculations	Description
STEP 10: Corner Spread Footing – WWR Detailing		
		Same remarks noted in Step 5 apply here. An example of a manufacturer's submittal is shown in Figure 8 and would be an accompaniment to the actual illustrative welded wire reinforcement shop drawings (Figure 9, 10, and 11) submitted for review.

XYZ Welded Wire Reinforcement Manufacturing, Inc.

Welded Wire Reinforcement – Project Conversions Summary

Project: ABC Office Building	To: Project General Contractor
Date: June 10, 2021	
By: WWR Designer	Structural EOR: Project Structural Engineering Firm

Spread Footing Reinforcement	WWR Substitution

Specified Rebar – f_y = 80 ksi						
Footing	Specified Reinforcement	Total Reinforcement Area (in^2)	Hooked Termination?	Approximate Bar Spacing Based on Edge Cover (in)	Nominal Rebar Tensile Strength (kips) $N_n = A_s \times f_y$	WWR ID Utilized
F150	(16) #7 EW B	9.60	N	11.6	768	WWR-F150
	(0) EW T	0	N/A	N/A	N/A	N/A

Substituted WWR – f_y = 80 ksi						
WWR ID	Mat Type	Mats Per Footing	Wires in X/Y Directions (For uni-directional mats, Y-Dir non-structural wires)	Average Wire Spacing (in)	Reinforcement Area Provided, X/Y (in^2)	Nominal Tensile Strength Provided $N_n = A_s \times f_y$ (kips)
WWR-F150	A	Note #1	(16) D30.0/ (5) W6.0	≤ 6	4.80/N.S.	384

Mat Type:

A. *Uni-directional* - structural wires in the primary direction; minimal non-structural holding wires at wide intervals in secondary direction, positioned to maintain mat shape.

B. *Bi-directional* - structural reinforcement in both orthogonal directions fabricated on a common WWR mat)

Bending: None

Material: ASTM A1064, 80 ksi yield strength

NOTES:

1. Provide two (2) WWR-F150 mats side-by-side, each direction bottom, in order to achieve total specified steel area. Total mats required for footing = 4.
2. See placement drawings for detailed WWR mat description, geometry, and arrangement in the structure.
3. Field trimming of mats is not permitted.

Figure 8: Manufacturer's submittal information showing WWR substitution

ACI 318-19	Calculations	Description
STEP 10: Corner Spread Footing – WWR Detailing (continued)		
Table 20.2.2.4(a)	$A_{s,provided} = 9.60\ in^2 = (16)\ \#7\ bars\ @\ \pm 11.6"\ on\ center\ each\ way$ $WWR\ options\ providing\ 9.60\ in^2\ steel\ area\ include:$ $(31)\ D31.0\ wires$ $(32)\ D30.0\ wires\ @\ \pm 6"\ on\ center\ each\ way\ \leftarrow\ selected\ by\ detailer$ $(36)\ D26.7\ wires$ $(Development\ length\ beyond\ column\ face\ is\ OK\ by\ inspection.)$	Convert the Grade 80 reinforcing bar pattern used as part of the base design into a Grade 80 WWR substitution. The numerical basis for this is presented in Figure 8. It will not be possible to transport a 15’-0” x 15’-0” WWR mat. As such, the original reinforcing scheme will be converted into a WWR solution comprised of four (4) identical mats that, once installed as an assembly, provide the overall required steel area. The resulting WWR mat assembly is shown in Figure 9, 10, and 11. The WWR mat configurations shown here are intended to serve as a general representation of manufacturer capabilities throughout North America. With that said, reinforcement arrangements presented on a submittal by one manufacturer’s detailer may vary slightly from those presented by another. Despite this, there exists a common and consistent responsibility for the WWR detailer to ensure that detailed WWR arrangements conform to the reinforcement cross-sectional area, placement, and curtailment requirements established by the design professional, unless explicitly-defined exceptions are made by the design professional of record on a project by project basis.

MAT WWR-F150
(4) REQUIRED: (2) NORTH-SOUTH, (2) EAST-WEST

N-S MATS SHOWN. POSITION STRUCTURAL WIRES ON BOTTOM.

NORTH-SOUTH NON-STRUCTURAL WIRES ARE POSITIONED TO AVOID CONFLICT WITH EAST-WEST STRUCTURAL WIRES ONCE EAST-WEST MATS ARE STACKED ON NORTH-SOUTH MATS.

V1 X V2 D30.0/W6.0 84" (+0.5",+0.5") X 14'-6" (24,24)
V1 = 0.5 OH, 4, 5, 11 @ 6, 5, 4, 0.5 OH
V2 = 24 OH, 3 @ 42, 24 OH

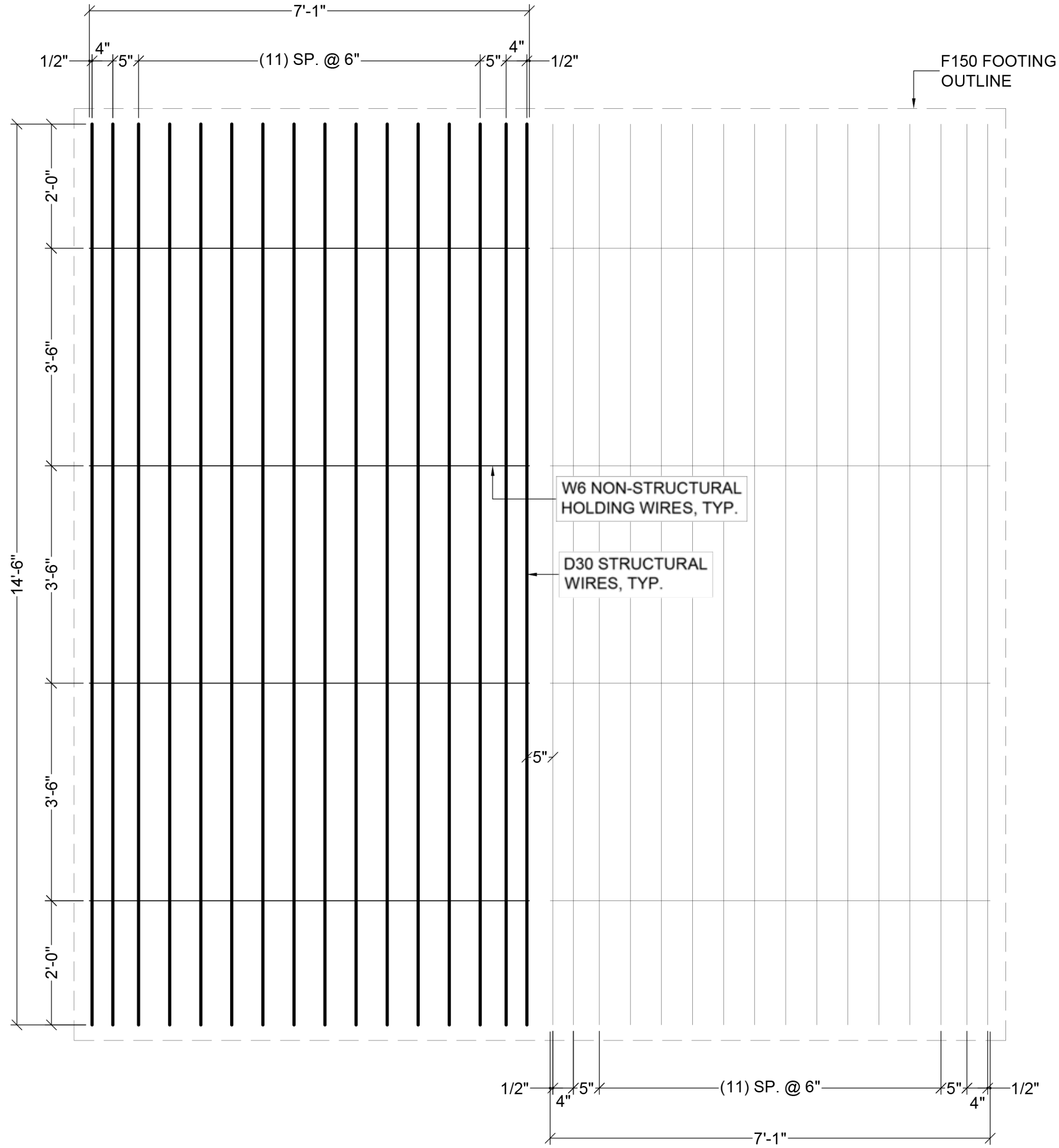


Figure 9: Representation of WWR for footing F150, as would be presented on a set of reinforcement shop/placement drawings

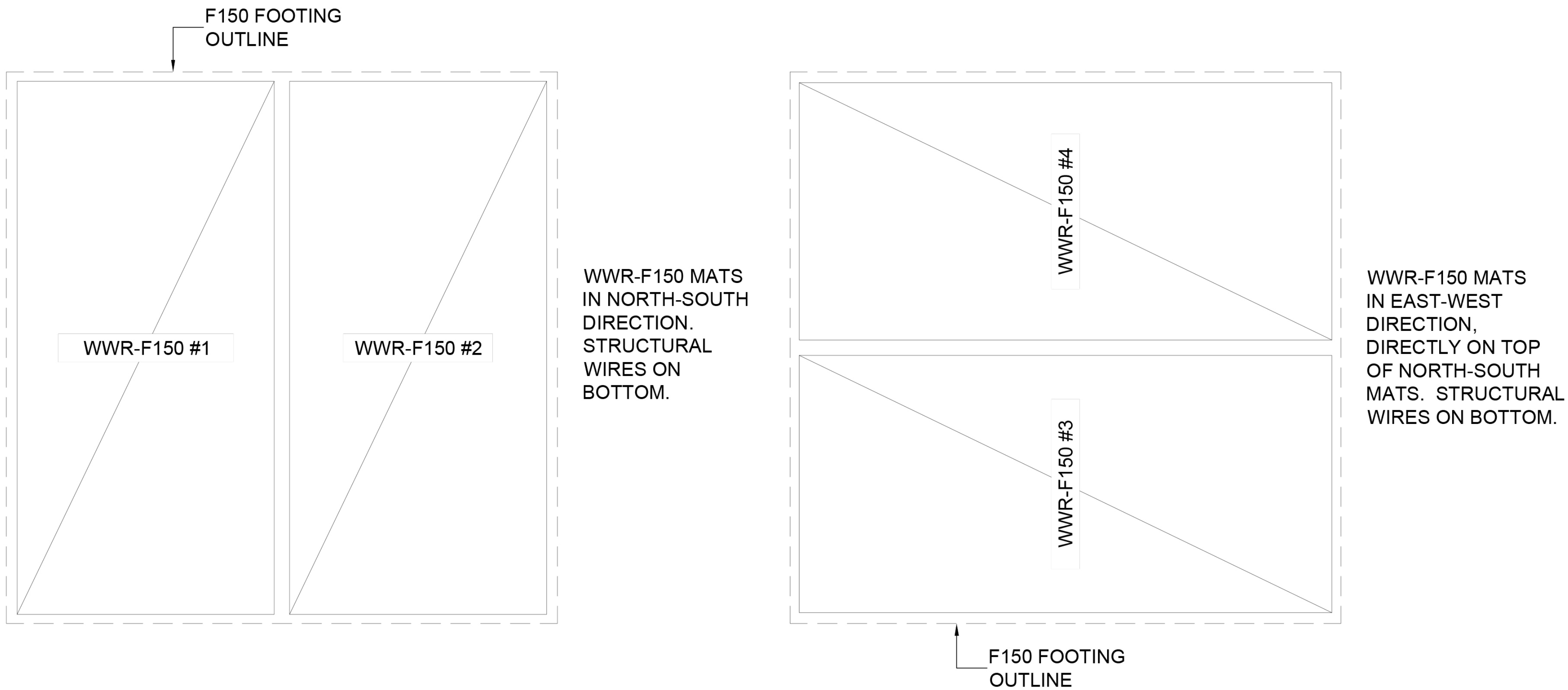


Figure 10: WWR placement information for Footing F150, showing the required four-mat assembly

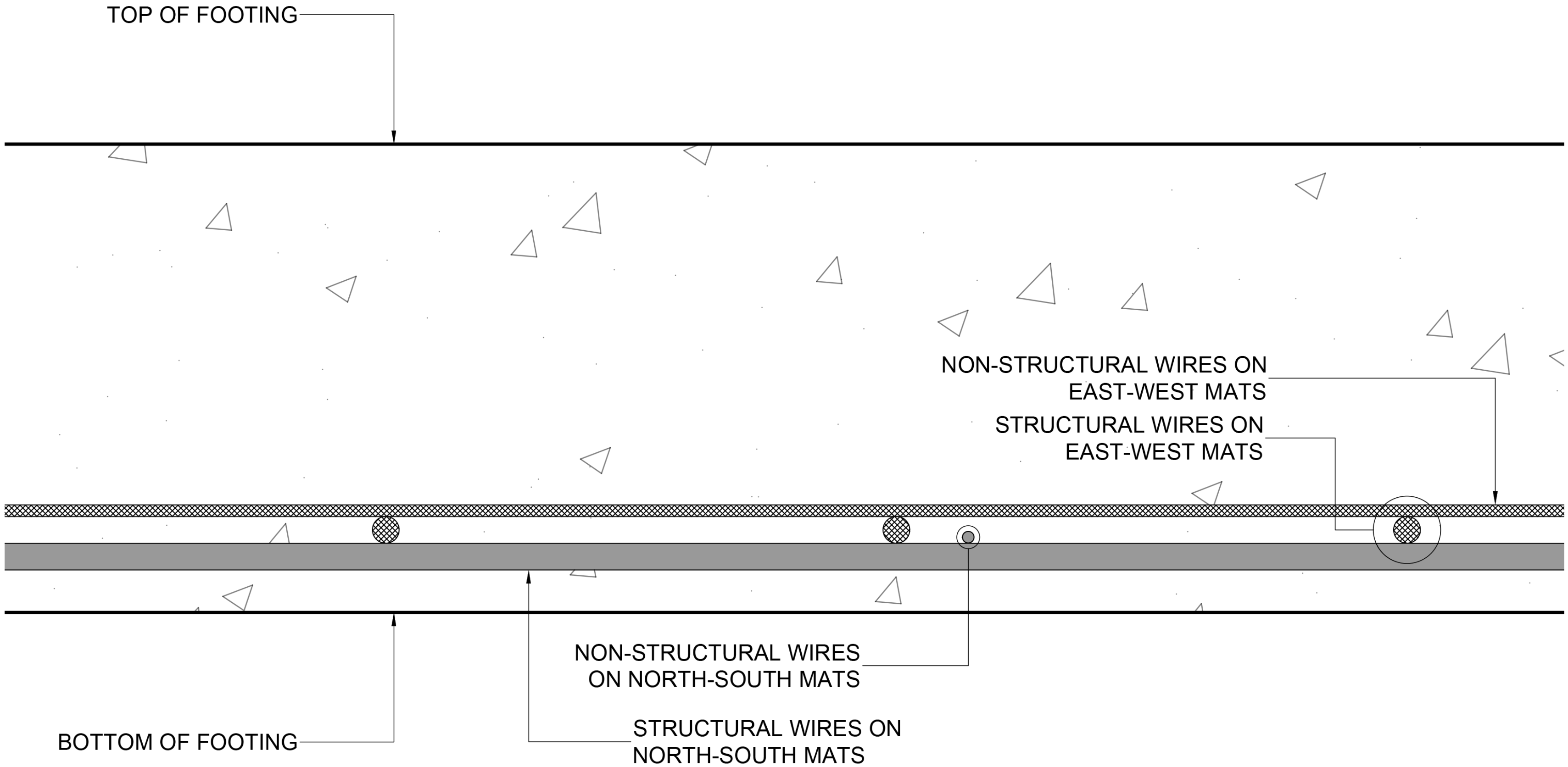


Figure 11: WWR placement information for Footing F150, showing mat stacking. Note that the WWR detailer will arrange non-structural holding wires such that they do not conflict with stacked structural wires that are coplanar.

ACI 318-19	Calculations	Description
STEP 11: Wall Footing – Design		
13.3.1.1	*Continuous Wall Footing Load* $q_D = 0.200\ kips/foot$ $q_L = 0\ kips/foot$ $q_a = 2{,}000\ psf\ allowable\ soil\ bearing\ pressure$ $Required\ Footing\ Width\!:\ W_{ftg} = \dfrac{q_D + q_L}{q_a \times 1\ ft} = \dfrac{0.200\ kips}{2\ kip/ft} = 0.1\ ft$ *Loading does not govern over the geometry of the cladding itself.* $\therefore$ *Use* 20" wide wall footing, resulting in 4" projection beyond cladding. *The Designer selects* 12 *inches as a footing thickness.*	The continuous wall footing located around the building perimeter is subjected to nominal loading due to weight of the first story's exterior cladding. The cladding assembly itself is 12" in width.
7.6.1.1 7.6.4.1 24.4.3.2 24.4.3.3	*ACI* 318 *minimums will be provided for longitudinal reinforcement .* $Longitudinal\!:\ 0.0018 \times 16" \times 12" = 0.35\ in^2\ required$ $\therefore$ *Use two* (2) *#4 bars, spaced at* $20 - 3" - 3" = 14"$ *By inspection, nominal transverse reinforcment will be provided.* $\therefore$ *Use #4 bars @* 14" *on center*	Longitudinal reinforcement is specified to satisfy ACI 318 minimum steel requirements. The design professional specifies a nominal amount of transverse reinforcement deemed appropriate for the intended performance of the wall footing in light of the compact geometry of the footing itself, the available soil bearing capacity, and the low magnitude of applied loading.

ACI 318-19	Calculations	Description
STEP 12: Wall Footing – WWR Detailing		
	$Controlling\ dimension\ between\ spread\ footings:$ $26\ ft - 11\ ft + 2\ ft + 2\ ft = 19'\ ladders$ $26\ ft - \frac{11\ ft}{2} - \frac{8\ ft}{2} + 2\ ft + 2\ ft = 20.5'\ foot\ ladders \leftarrow$ $23\ ft - 11\ ft + 2\ ft + 2\ ft = 16'\ ladders$ $23\ ft - \frac{11\ ft}{2} - \frac{8\ ft}{2} + 2\ ft + 2\ ft = 17.5'\ foot\ ladders$ $\therefore\ Provide\ 20.5'\ ladder\ mats.$	The wall footing reinforcement will be produced by the WWR manufacturer as "ladders", with length to suit the minimum 24" lap into spread footings per Note #5 shown in Figure 2. See Figure 12 for a representative WWR mat for a continuous wall footing.

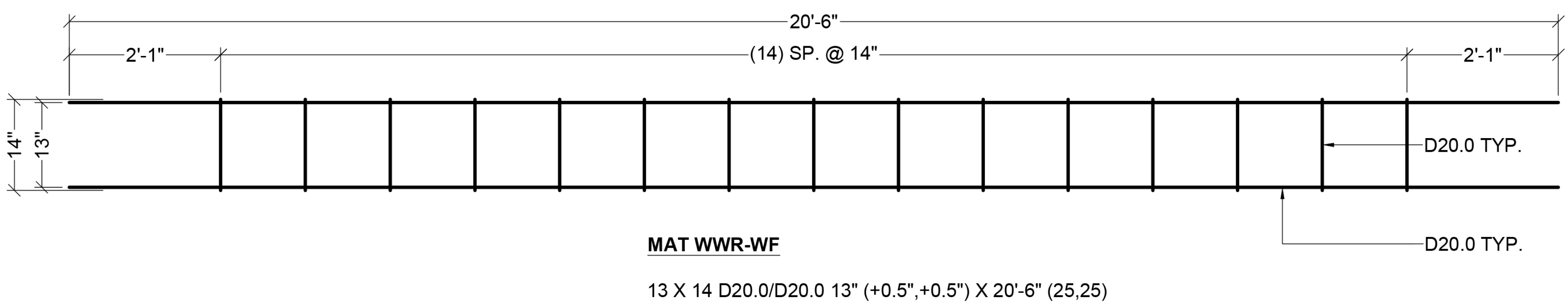


***Figure 12:** Representation of WWR for wall footing. Note the spacing indicated for the longitudinal wires is shown a total of one inch less than that which was specified in order to accommodate a minimum production "overhang" of one-half inch on each side (as dictated by the needs of the welding machines). One alternative that many manufacturers might elect to pursue in lieu of running such a narrow mat is to run a wider mat with a repeating pattern of longitudinals spaced at 14" on center. This mat would then be shop-sheared into several narrow mats satisfying the specified spacing of the longitudinal reinforcement and would simultaneously eliminate any protruding nominal overhangs.*

Chapter Six

Ground Supported Slabs

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: $f'_c = 4{,}000\ psi$

$f_y = 70{,}000\ psi\ (welded\ deformed\ wire\ reinforcement\ yield\ strength\ will\ be\ used)$

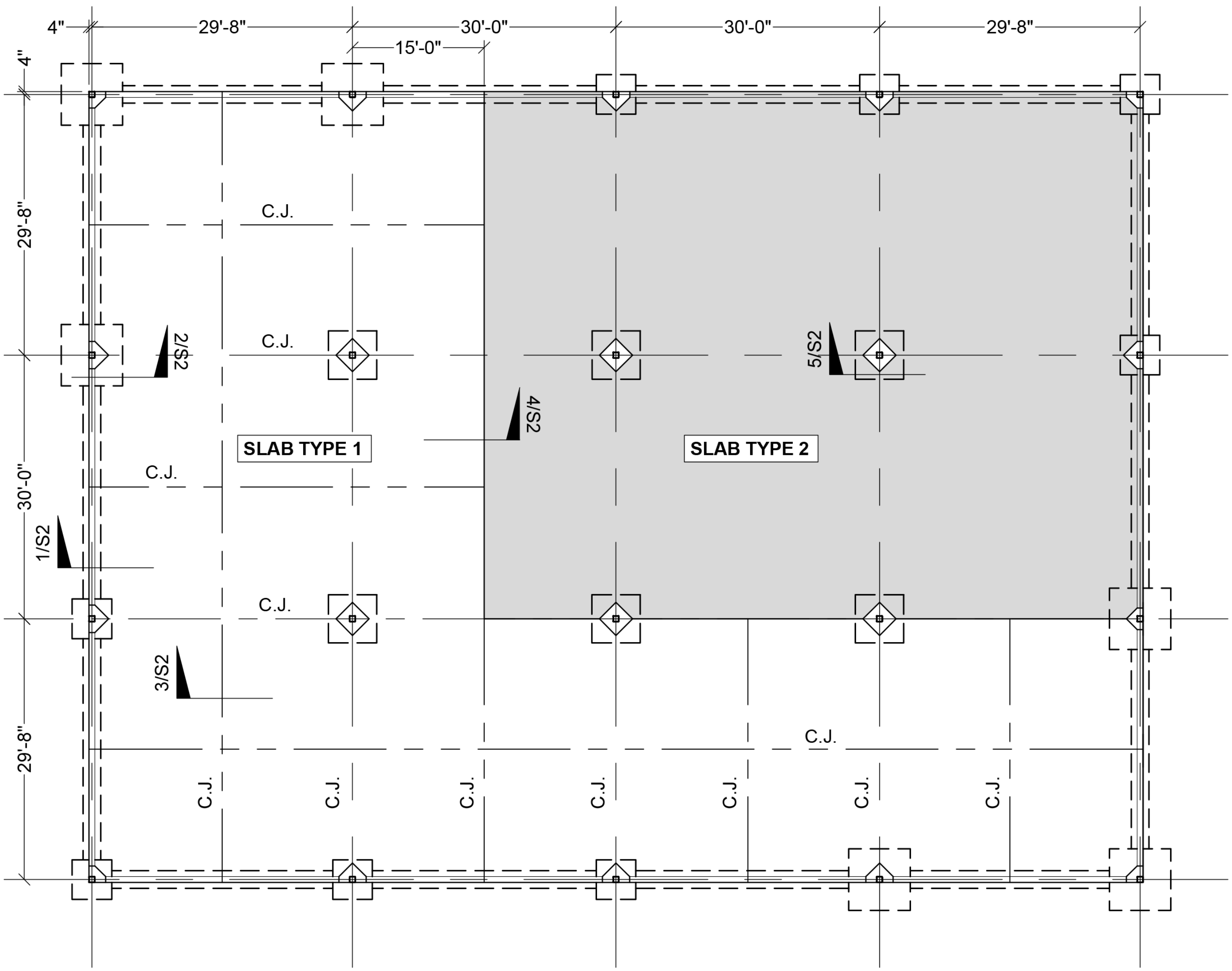
$Concrete\ Density = 0.145\ kcf, normalweight\ concrete\ \lambda = 1.0$

$Relevant\ Soil\ Criteria\ 150\ \ pounds/in^2/vertical\ inch\ of\ displacement\ (subgrade\ modulus);\ 0.120\ kcf\ density$

$LL\ Loading = varies$

The example includes the following steps:

Step 1 – Engineer's Design Methodology
Step 2 – Comparison to WRI TF-705 Methods
Step 3 – Reinforcement Detailing



SLAB TYPE 1:
- 4" THICKNESS
- CONTROL JOINTS: 15'-0" MAXIMUM SPACING
- 0.067 IN^2 PER FOOT, EACH DIR., FIELD
- 0.048 IN^2 PER FOOT, ACROSS JOINTS

SLAB TYPE 2:
- 6" THICKNESS
- NO CONTROL JOINTS
- REINFORCEMENT: 0.36 IN^2 PER FOOT, EACH DIR.

Figure 1: Excerpt from contract structural drawings: foundation plan

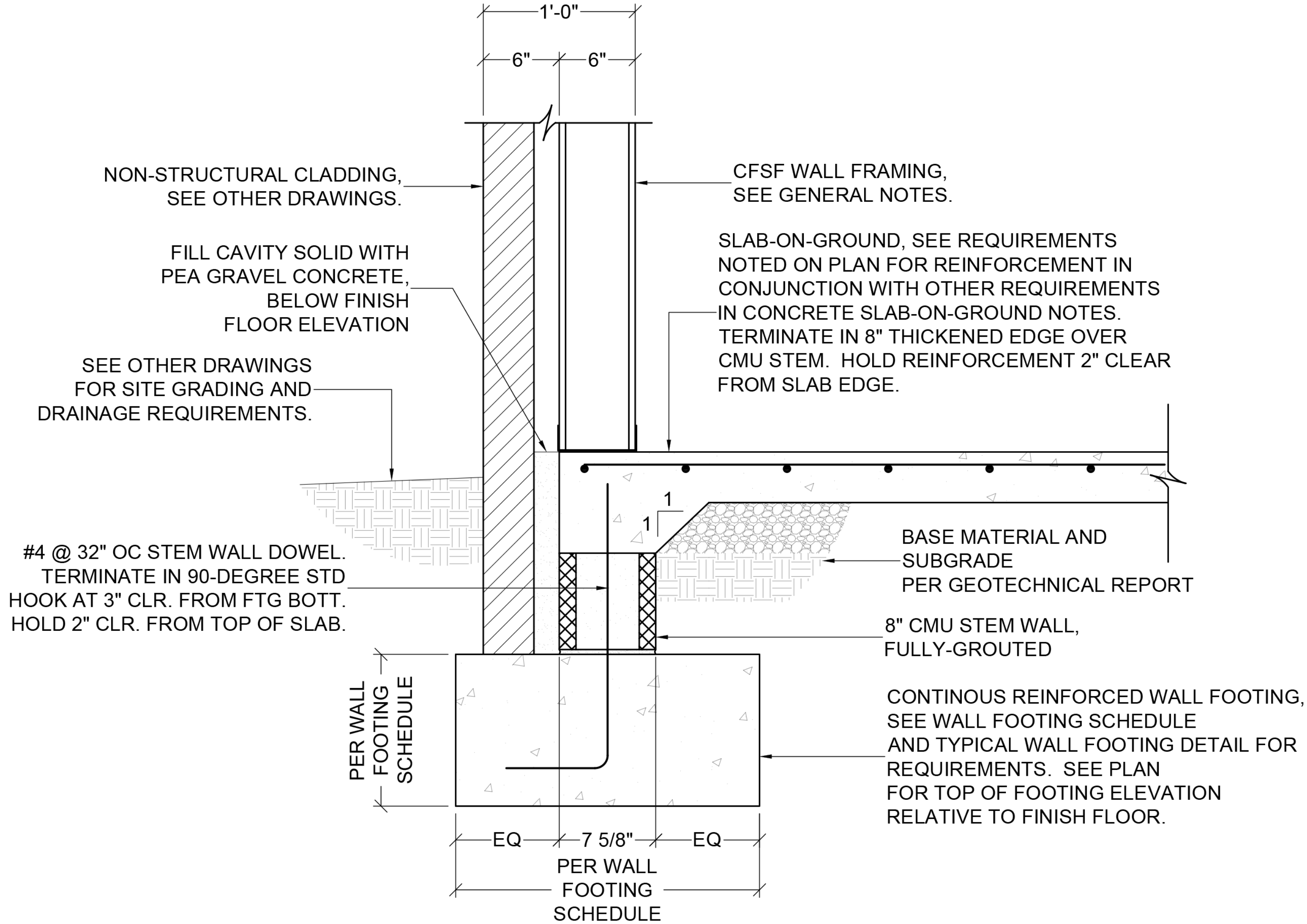


Figure 2: Excerpt from contract structural drawings: 1/S2 exterior wall section, showing integration of slab-on-ground with perimeter stem wall and supporting foundation. An alternative condition in which the slab-on-ground is held clear of the exterior wall assembly was also considered by the structural 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.

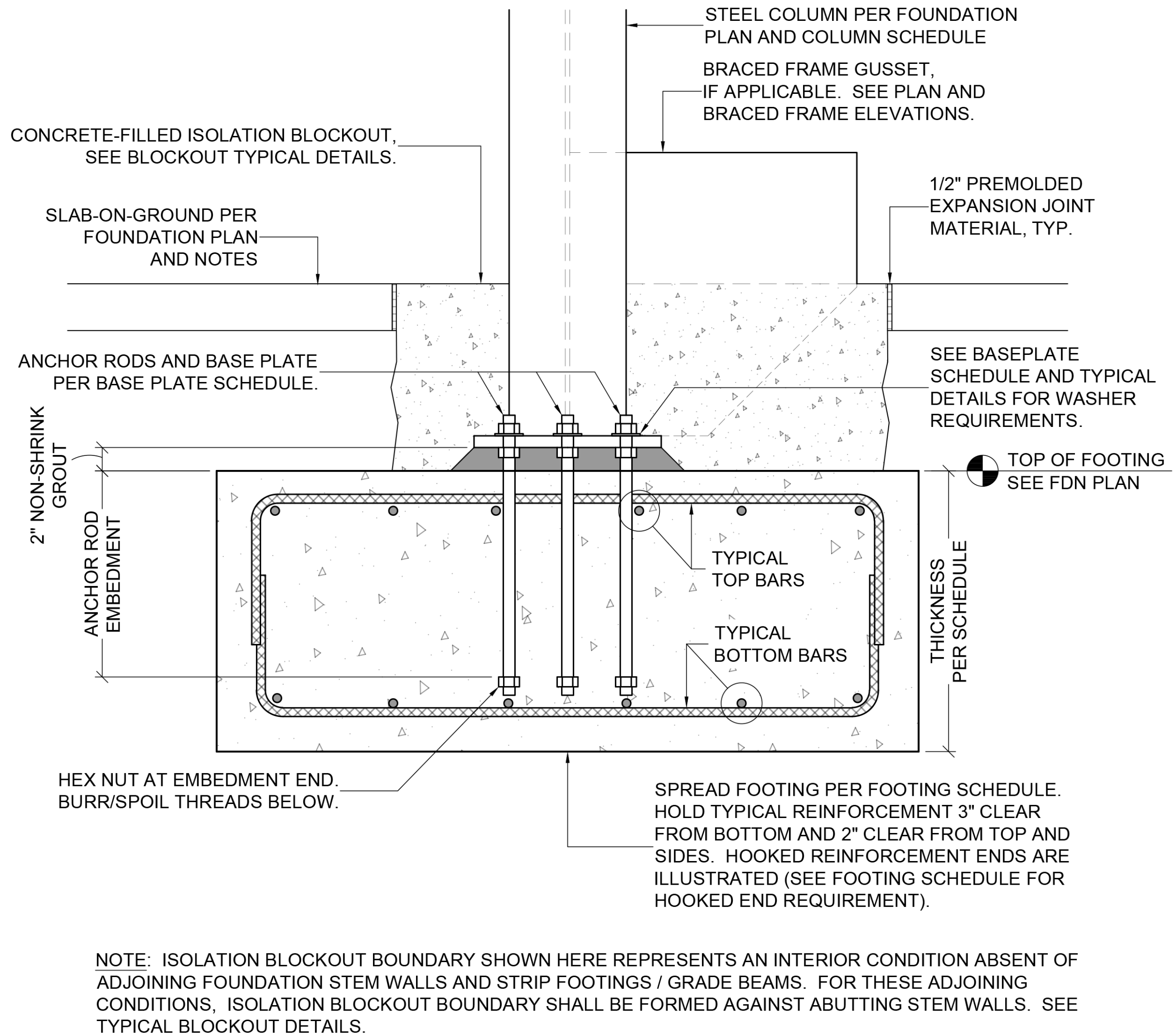


Figure 3: Excerpt from contract structural drawings: 2/S2 and 5/S2 spread footings, showing isolation blockout around building column

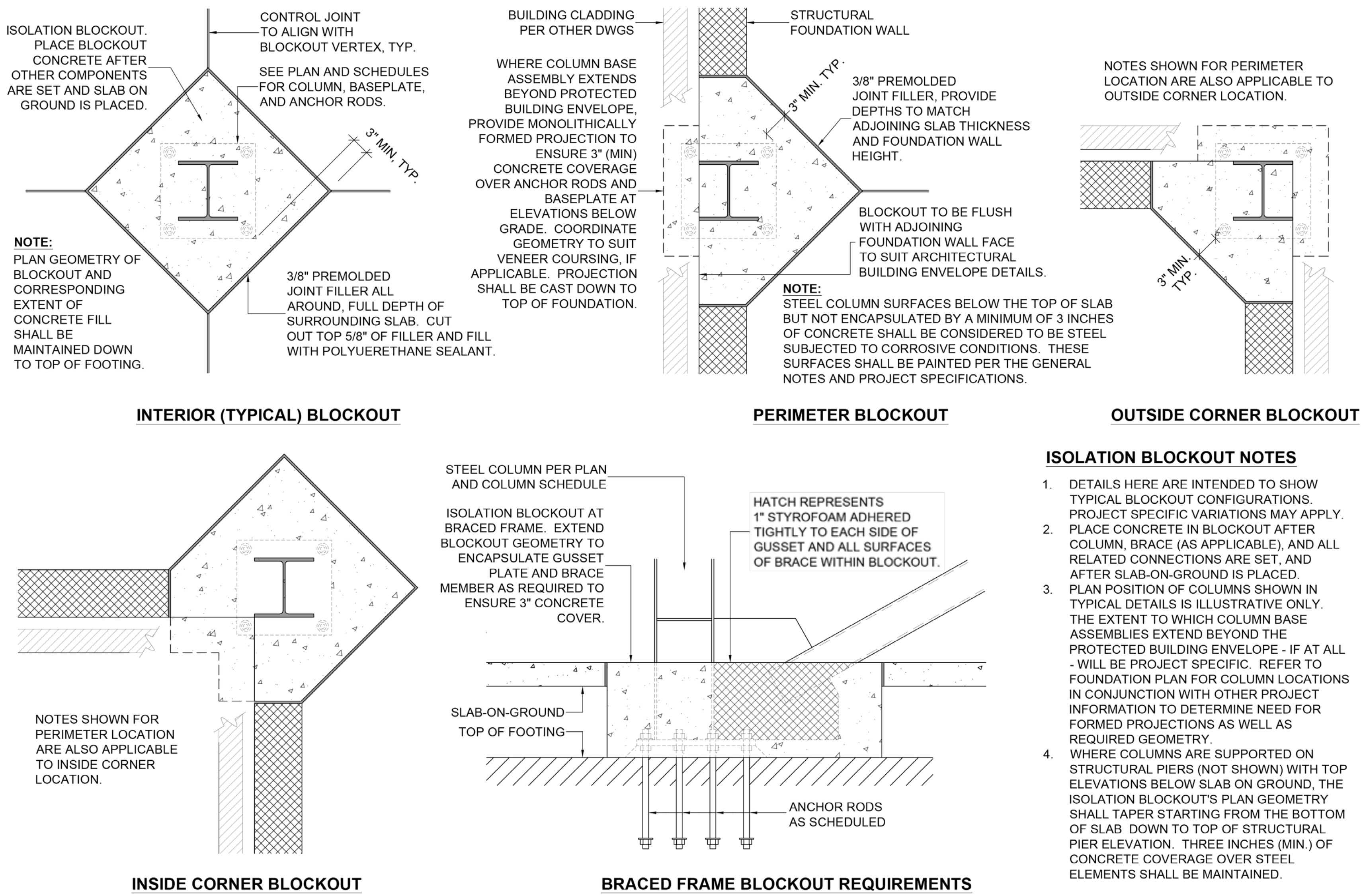


Figure 4: Excerpt from contract structural drawings: typical details for isolation joints / blockouts at columns

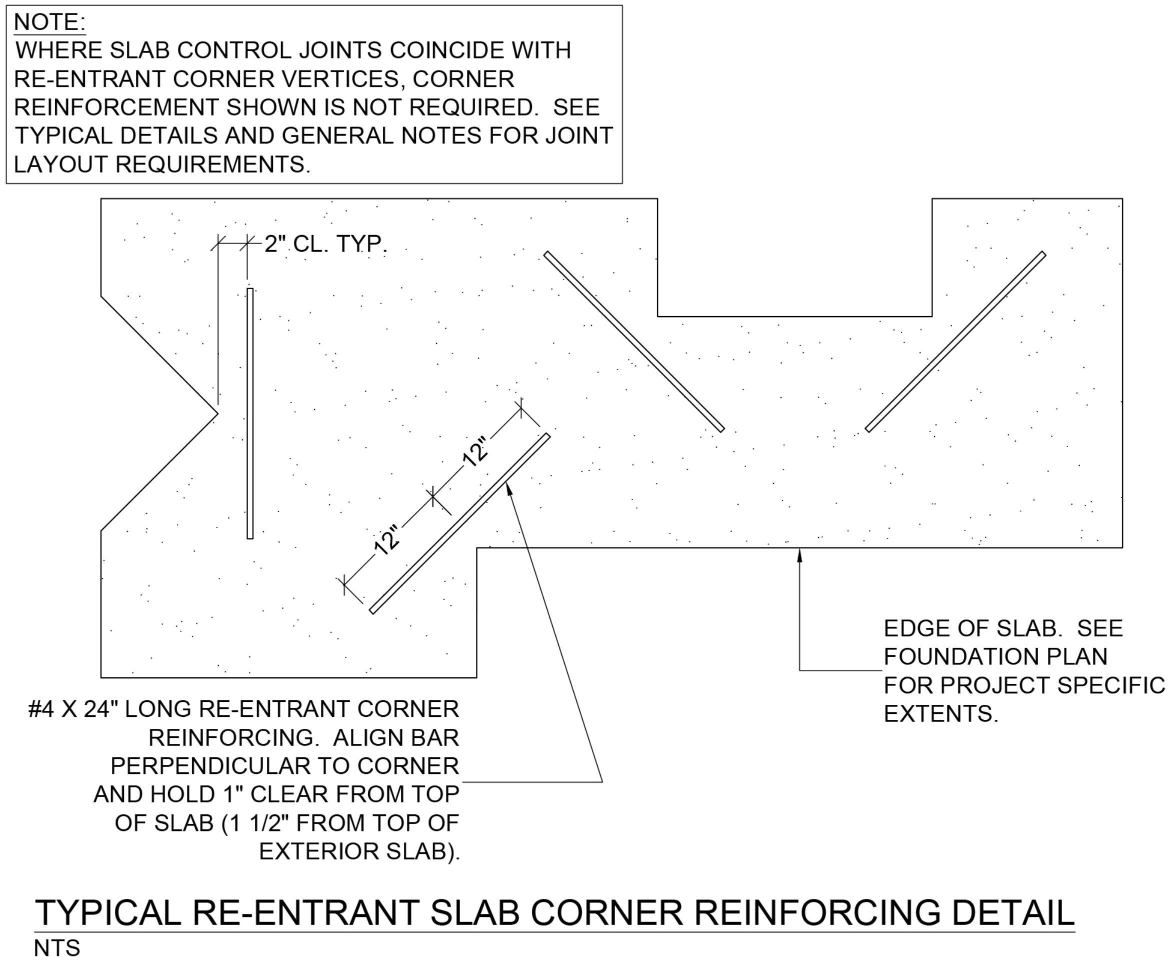


Figure 5: Excerpt from contract structural drawings: typical detail for re-entrant crack control bars or deformed wires. These will be required at column isolation blockouts in the area of Slab Type 2, where there are no control joints.

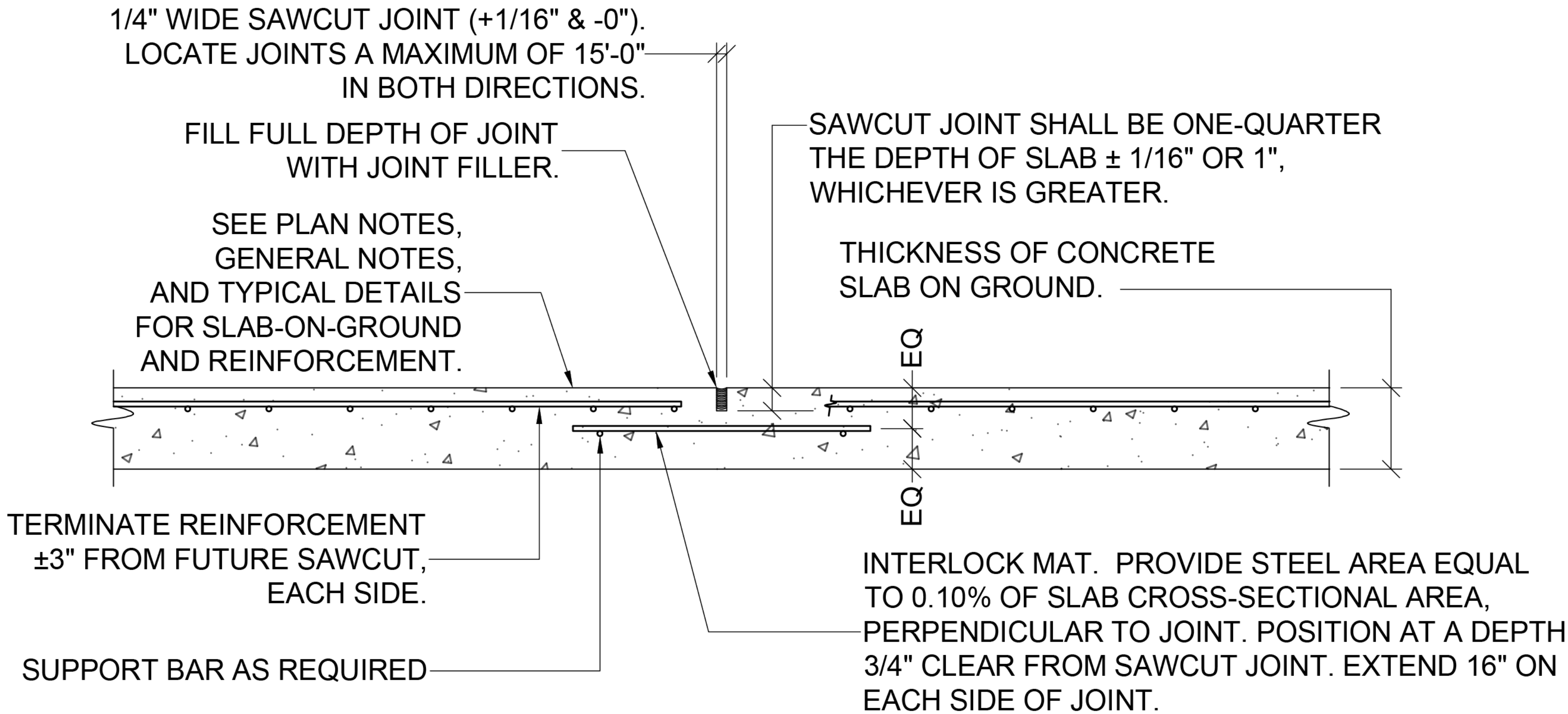


Figure 6: Excerpt from contract structural drawings: 3/S2 typical detail for sawcut control joint

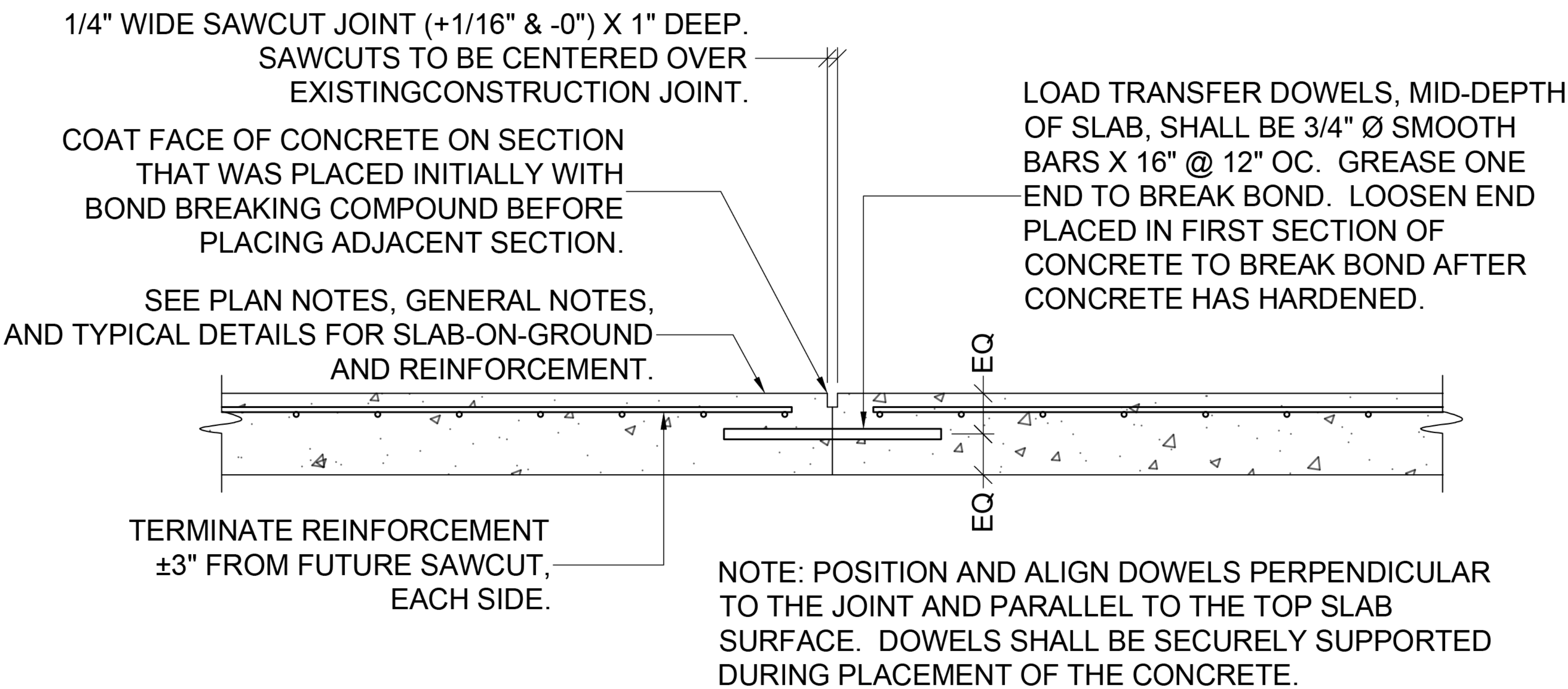


Figure 7: Excerpt from contract structural drawings: 4/S2 typical detail for construction joint transition between slab types. Typical detail shows same slab depth each side of joint, while slabs in example will require a depth transition (4" and 6"). Load transfer dowels will be centered in the 4" thick 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/IN^2 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 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.

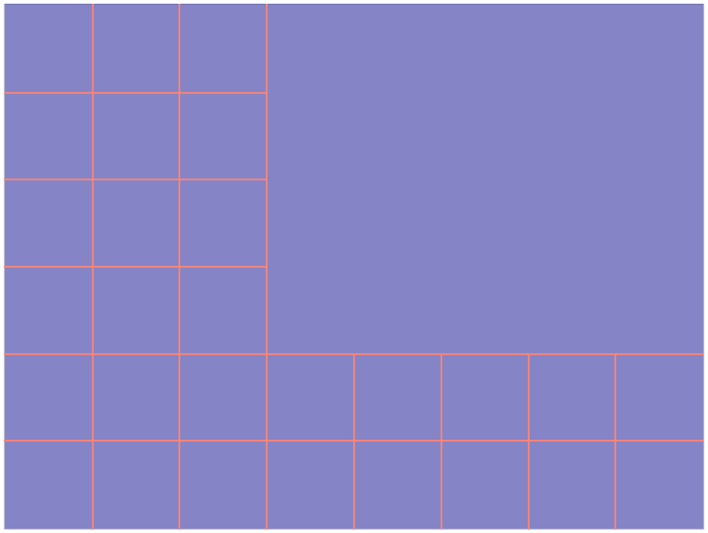
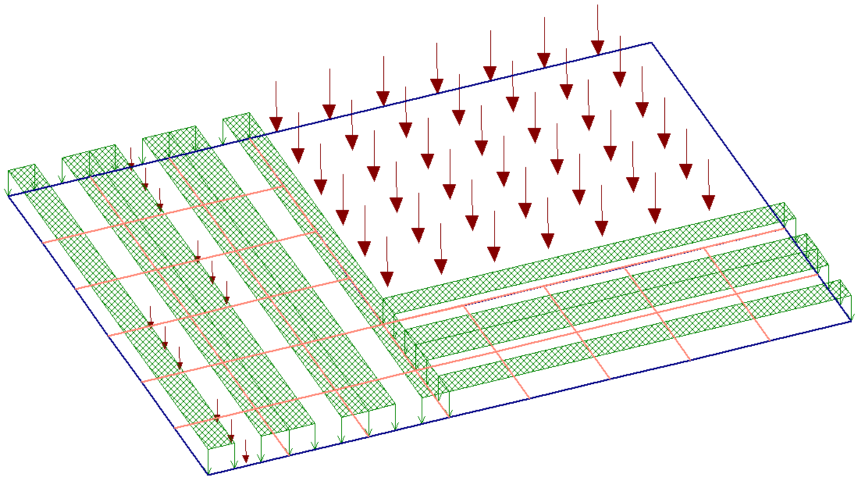
For all of the extremely valuable information and insights made available to the designer in ACI 360-10, like other publications – and for a plethora of justified reasons - it tends to lack a formal, definitive, and quantitative correlation between acceptable slab cracking and specific steel reinforcement 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.

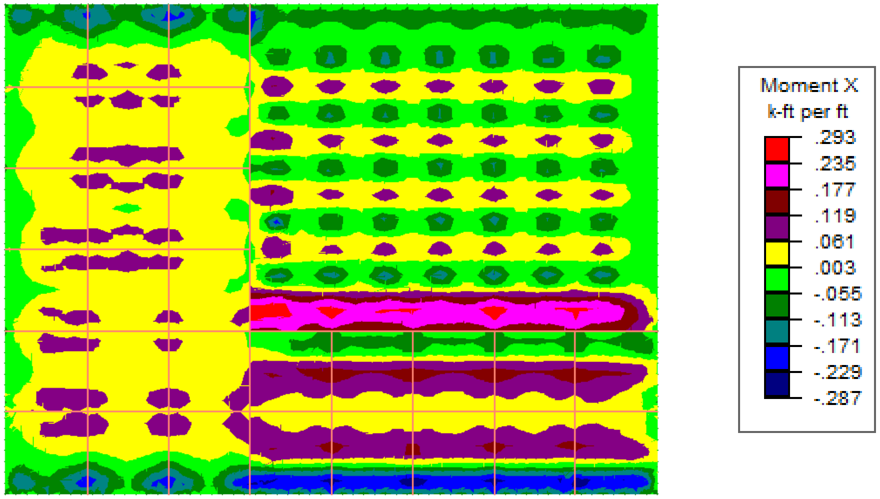
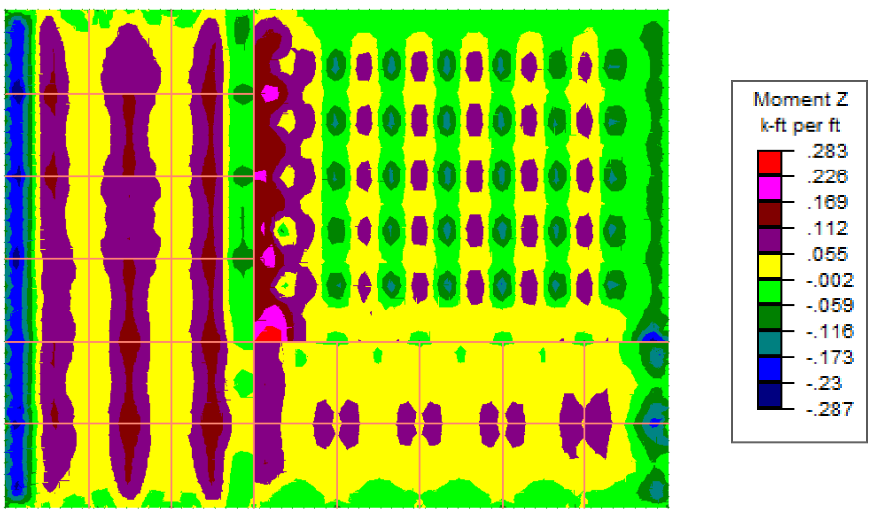
Beyond ACI standards, the Wire Reinforcement Institute's own TF-705 "Innovative Ways to Reinforce Slabs-on-Ground" offers several methods for deriving steel areas, but there is a broad variation in the results depending on the method selected therein. This lends credence to the notion that, unlike design 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.

ACI 360-10	Calculations	Description
STEP 1: Engineer's Design Methodology		
Chapter 4	• Geotechnical engineering report provided • Subsurface preparation recommendations defined • Modulus of subgrade reaction = 150 pounds/in^2 per inch vertical depth	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.
Section 1.4.3 *and* *professional* *engineering judgment*	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.	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).

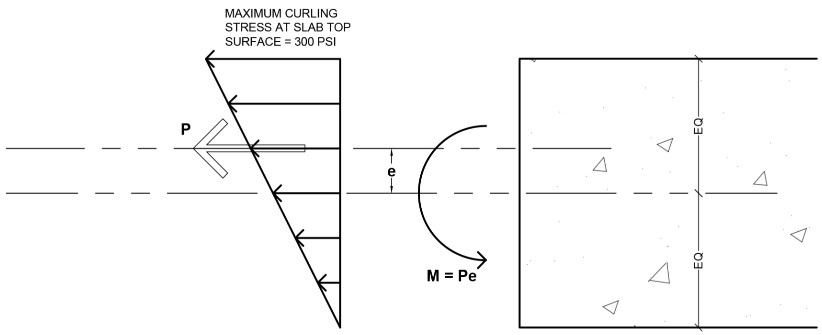
ACI 360-10	Calculations	Description
STEP 1: Engineer's Design Methodology (continued)		
Section 3.2.2 Section 7.1 Table 5.2	Slabs thicknesses are derived based on uncracked section analysis, using the modulus of rupture of concrete with a factor of safety equal to 2.0 and checked against service-level bending stresses that result from the applied pallet, rack, and vehicle loading.	
	The slabs will be provided with reinforcement as a crack control measure.	
Section 6.2 Figure 6.6 *and professional engineering judgment*	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", 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).	
Table 3.1 Section 8.3	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).	

ACI 360-10	Calculations	Description
STEP 1: Engineer's Design Methodology (continued)		
Section 1.4.3	***Overall plan view of plate model (plate sub-meshing not shown):***  ***A representative illustration of an external applied load case:*** 	

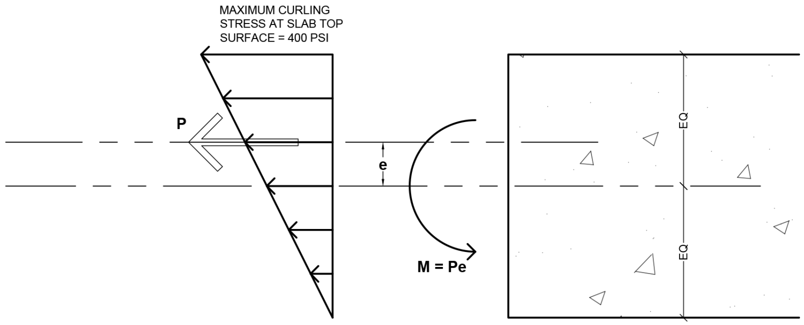
ACI 360-10	Calculations	Description
STEP 1: Engineer's Design Methodology (continued)		
Section 1.4.3	***FEA plate model - flexural contours (north-south):***  ***FEA plate model - flexural contours (east-west):***  	Service-level load combinations are generated and summarized.

ACI 360-10	Calculations	Description
STEP 1: Engineer’s Design Methodology (continued)		
Section 5.9 Table 5.2 Section 7.1 Section 5.9 Table 5.2 Section 7.1	***Slab thickness Design*** **Slab Type 1: 4” thickness** $f_b = \frac{M_{service}}{S_{uncracked}} = \frac{0.287 \times 12}{\frac{12 \times 4^2}{6}} = 0.108\ ksi$ $F_b = \frac{f_r}{F.S} = \frac{7.5\lambda\sqrt{f'_c}}{2.0} = \frac{7.5 \times 1.0 \times \sqrt{4{,}000}}{2.0} = 0.237\ ksi$ $f_b < F_b \therefore OK$ **Slab Type 2: 6” thickness** $f_b = \frac{M_{service}}{S_{uncracked}} = \frac{0.293 \times 12}{\frac{12 \times 6^2}{6}} = 0.049\ ksi$ $F_b = \frac{f_r}{F.S} = \frac{7.5\lambda\sqrt{f'_c}}{2.0} = \frac{7.5 \times 1.0 \times \sqrt{4{,}000}}{2.0} = 0.237\ ksi$ $f_b < F_b \therefore OK$	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.

ACI 360-10	Calculations	Description
STEP 1: Engineer's Design Methodology (continued)		
	<u>Designer's project-specific reinforcement design</u> **<u>Slab Type 1: curling and shrinkage reinforcement</u>** 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:** $300\,psi + 108\,psi = 408\,psi < 474\,psi\ modulus\ of\ rupture, unreduced$ $\therefore\ previously\ calculated\ 4''\ thick\ slab\ proportioning\ acceptable$ **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.	Determination of appropriate curling and shrinkage reinforcement should be carried out on a project-specific basis, and beyond very basic prescriptive reinforcing steel areas defined in ACI 360 will be based on methods deemed applicable by the design professional of record. *Reference: "The First Commandment for Floor Slabs: Thou Shalt Not Curl Nor Crack… (Hopefully)", Walker and Holland, *Concrete International*, January 1999.

ACI 360-10	Calculations	Description
STEP 1: Engineer's Design Methodology (continued)		
	Designer's project-specific reinforcement design (continued) **Slab Type 1: curling and shrinkage reinforcement (continued)** **Check B (continued):**  $M = 300\frac{lb}{in^2} \times 4'' \times 12'' \times \frac{1}{2} \times \left(\frac{4''}{2} - \frac{4''}{3}\right) = 4{,}800\ lb - in, per\ linear\ ft$ $M = f_s \times A_s \times jd$ $Set\ f_s = 24{,}000\ psi$ $Set\ jd = 0.75h$ $A_s = \frac{4{,}800}{24{,}000 \times 0.75 \times 4''} = \mathbf{0.067\ in^2 per\ linear\ foot} > 0.001bh$ 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.

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** 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:** $400\ psi + 49\ psi = 449\ psi < 474\ psi\ modulus\ of\ rupture, unreduced$ $\therefore\ previously\ calculated\ 6"\ thick\ slab\ proportioning\ acceptable$	

ACI 360-10	Calculations	Description
STEP 1: Engineer's Design Methodology (continued)		
	<u>Designer's project-specific reinforcement design (continued)</u> **<u>Slab Type 2: curling and shrinkage reinforcement (continued)</u>** **Check B:**  $M = 400\frac{lb}{in^2} \times 6" \times 12" \times \frac{1}{2} \times \left(\frac{6"}{2} - \frac{6"}{3}\right) = 14{,}400\ lb - in, per\ linear\ ft$ $M = f_s \times A_s \times jd$ $Set\ f_s = 24{,}000\ psi$ $Set\ jd = 0.75h$ $A_s = \frac{14{,}400}{24{,}000 \times 0.75 \times 6"} = 0.133\ in^2 per\ linear\ foot < 0.005bh$ 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 X 12 X 6 = 0.36 in^2 per linear foot width of slab shall be used.**	

ACI 360-10	Calculations	Description
STEP 2: Comparison to WRI TF-705 Methods		

Procedure (use 70 ksi reinforcement)	Slab Type 1 Steel Area	Slab Type 2 Steel Area
Design & Detailing Guide	0.067 in^2	0.133 in^2
Design & Detailing Guide + ACI 360 prescriptions*	0.067 in^2/ 0.048 in^2 @ joint	0.360 in^2
Subgrade Drag	0.014 in^2	0.107 in^2
Confirmed Capacity	0.052 in^2	0.079 in^2
Temperature**	0.096 in^2	0.144 in^2
Equivalent Strength	0.173 in^2	0.260 in^2
Crack Restraint	0.535 in^2	0.802 in^2

* 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: CTE = 6 x 10^{-6}/ °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.

<u>Author Note</u>:
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.

ACI 360-10	Calculations	Description

STEP 3: Reinforcement Detailing

Slab Type 1 Steel Area	Slab Type 2 Steel Area
0.067 in^2/ 0.048 in^2 @ joint	0.360 in^2

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 in^2

Option B: D9.0 @ 16” on center (16x16 D9.0/D9.0) → A_s = 0.068 in^2

~~Option C: #3 rebar @ 18” on center → A_s = 0.073 in^2~~

For joint reinforcement (enhanced aggregate interlock):

D4.8 @ 12” on center → A_s = 0.048 in^2

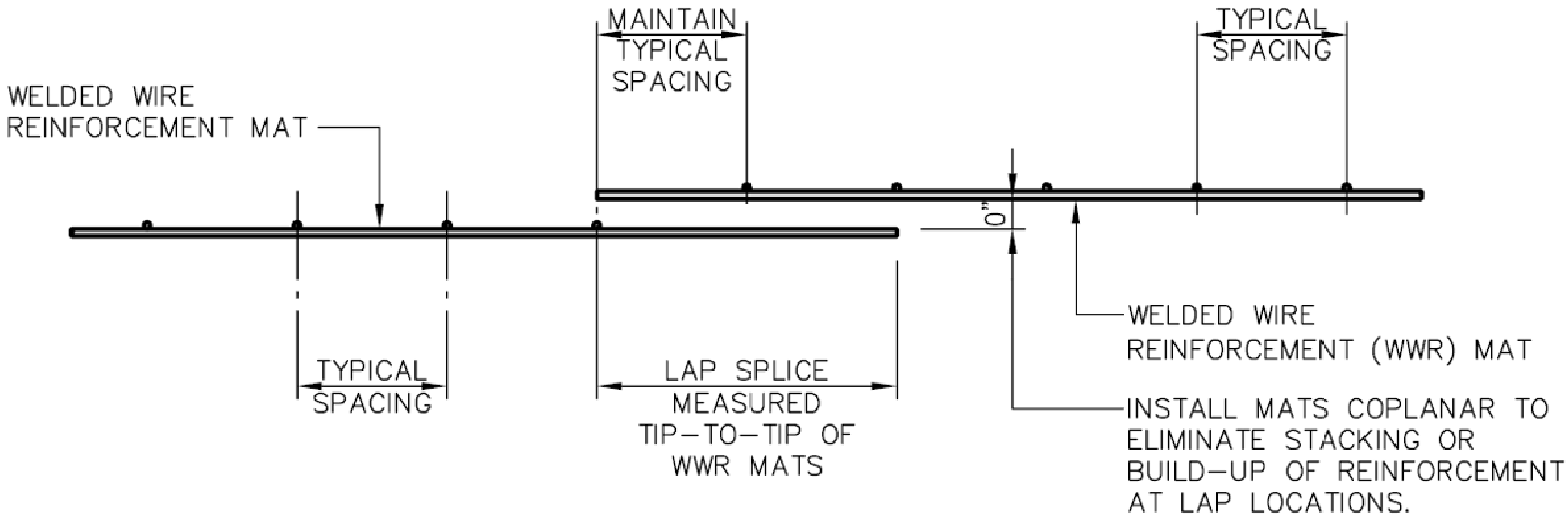
or

~~#3 rebar @ 27.5” on center → A_s = 0.048 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.

<u>Author Note</u>:
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.



ACI 318-19 LAP SPLICE LENGTH:

WIRE SIZE	*LAP SPLICE*
< D7.0	*12 INCHES*
≤ D11.0	*16 INCHES*
≤ D16.0	*18 INCHES*
≤ D20.0	*22 INCHES*
≤ D31.0	*30 INCHES*

Figure 10: Excerpt from contract structural drawings: Typical WWR Lap Splice Detail

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

ACI 360-10	Calculations	Description
STEP 3: Reinforcement Detailing (continued)		
	Slab Type 1 Steel Area 0.067 in^2/ 0.048 in^2 @ joint *For field reinforcement:* Option A: D4.5 @ 8” on center (8x8 D4.5/D4.5) → A_s = 0.068 in^2 Option B: D9.0 @ 16” on center (16x16 D9.0/D9.0) → A_s = 0.068 in^2	
	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.	

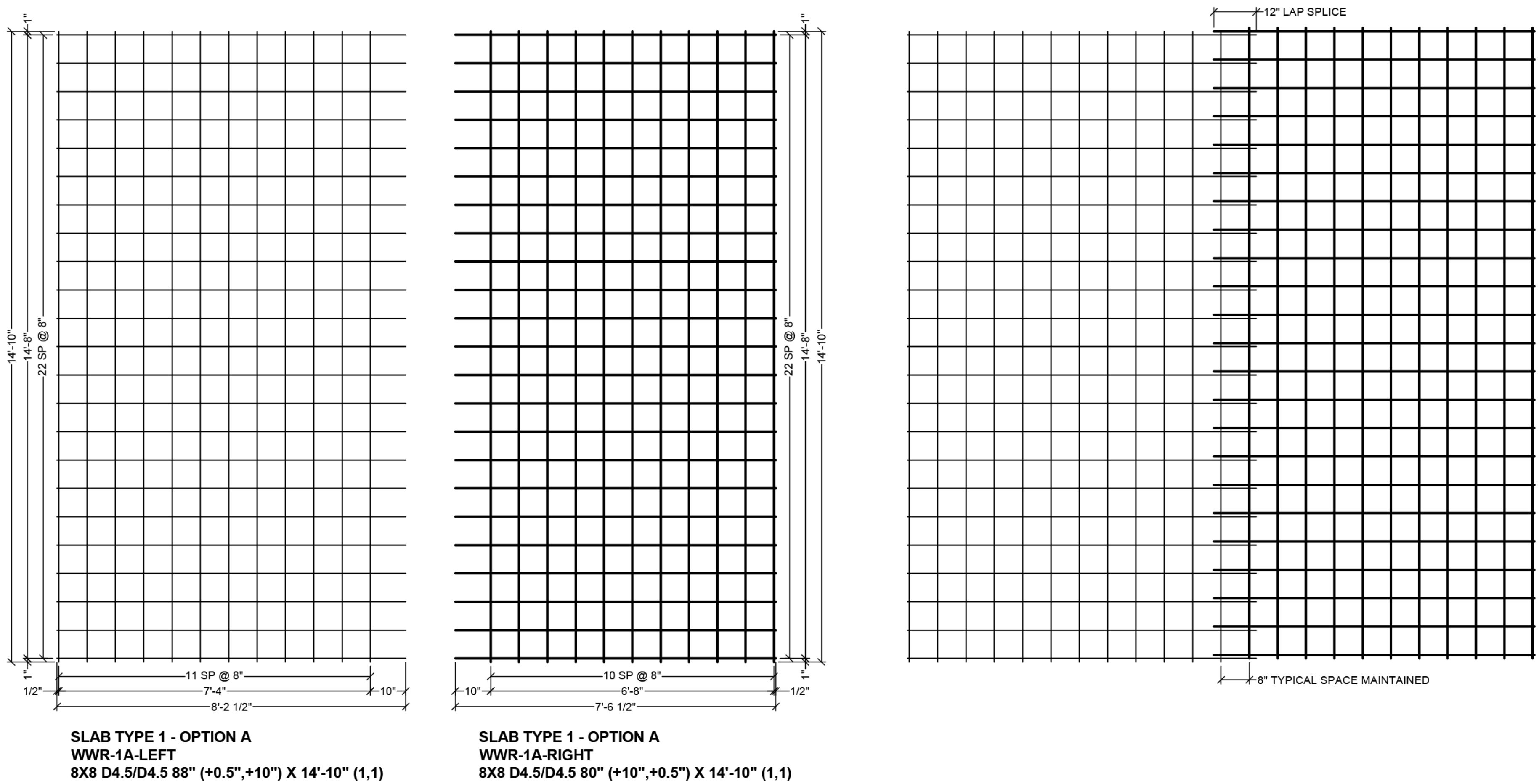


***Figure 12:** Slab Type 1, Option A: WWR mat configuration shown. For each 15 ft x 15 ft slab "panel" there will be a "left" and "right" mat. Note how mat central overhangs are configured to achieve the lap splice requirement while maintaining a typical wire spacing at the transition from one mat to the other.*

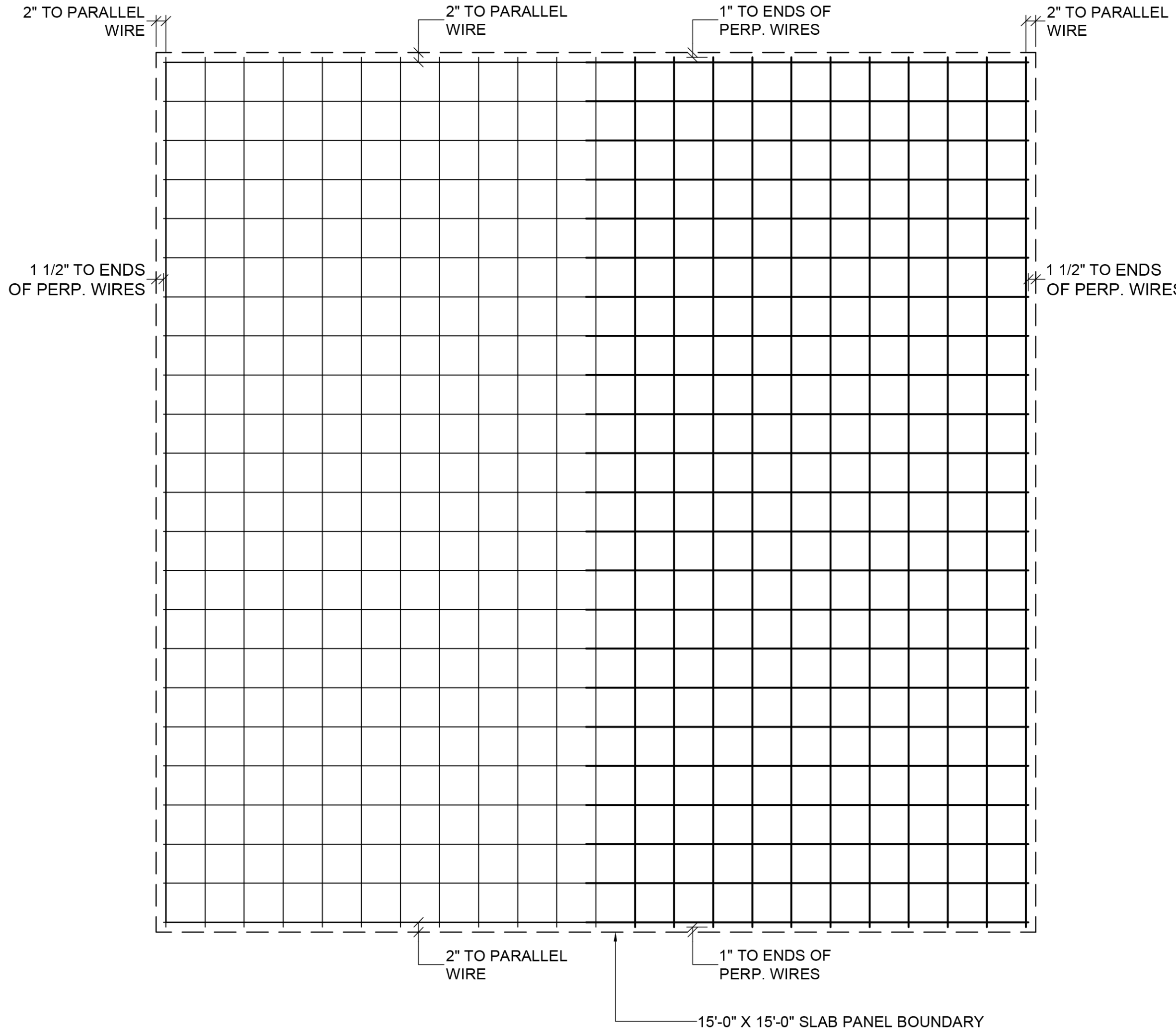


Figure 13: Slab Type 1, Option A: WWR mats "in panel". Dimensions between the concrete panel boundaries (at control joints and slab terminations) and steel wires are illustrated.

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.

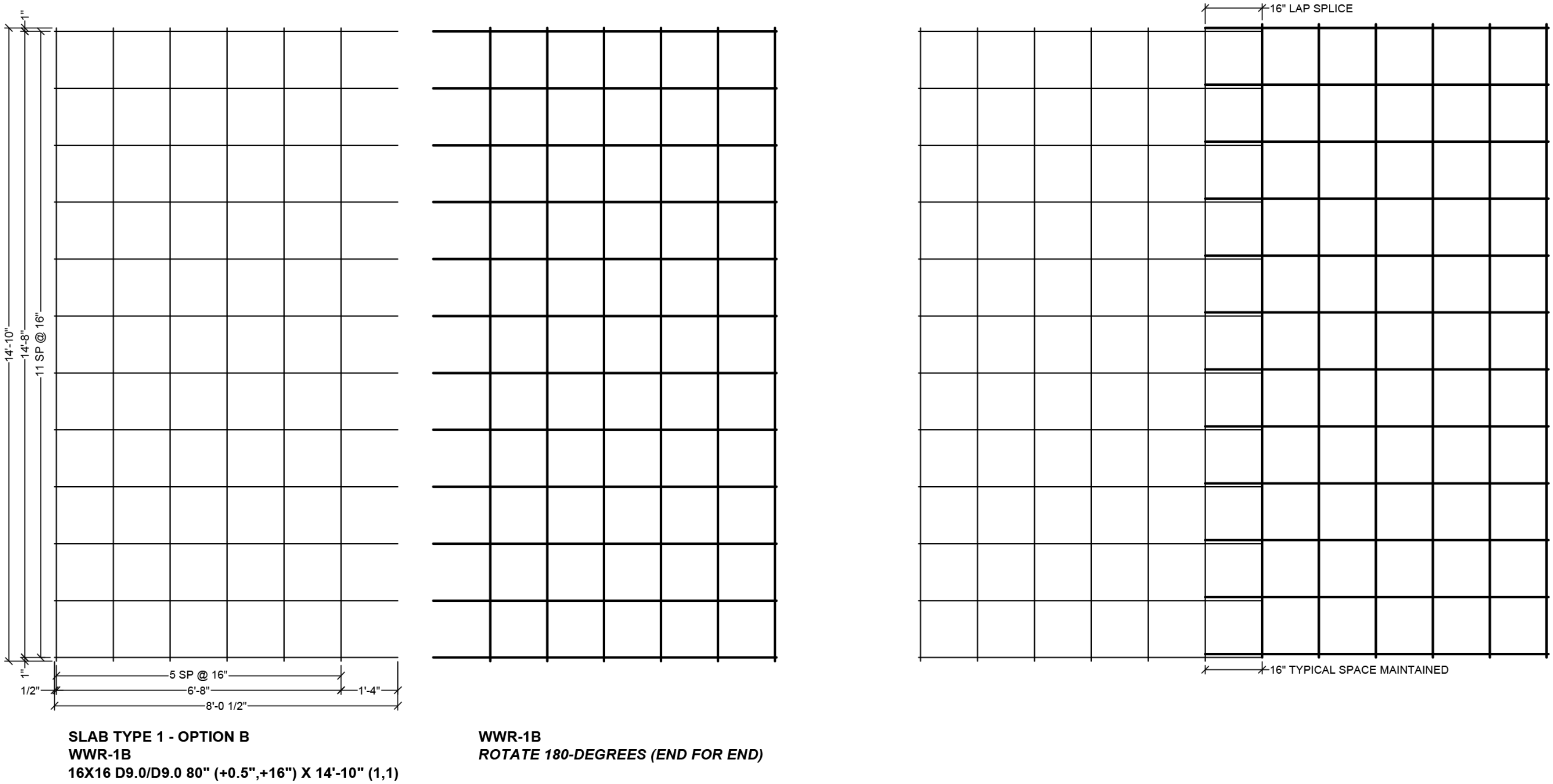


***Figure 14:** Slab Type 1, Option B: WWR mat configuration shown. For each 15 ft x 15 ft slab "panel" there will be two identical mats, one of which will simply be rotated end for end in the field.*

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.

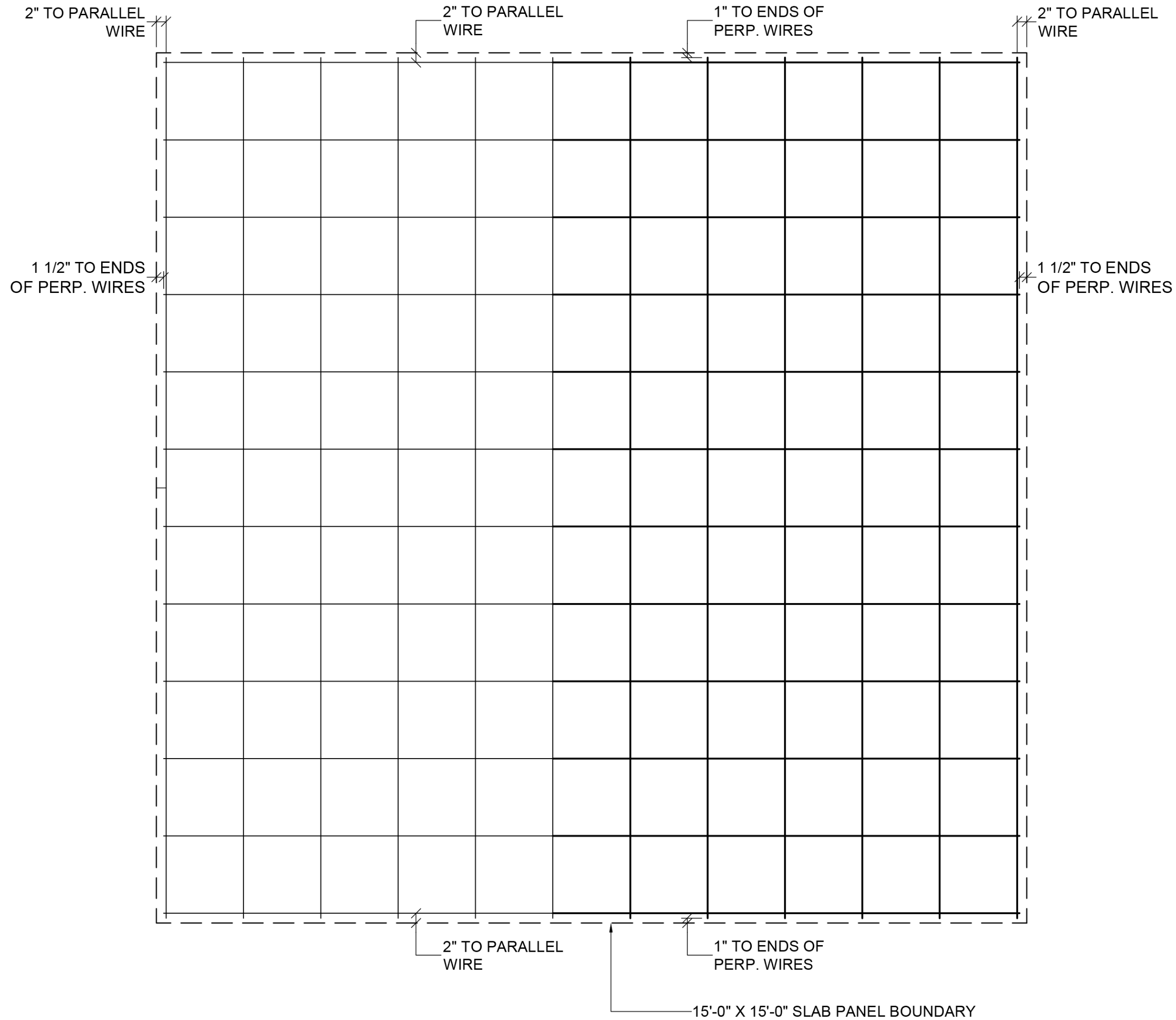
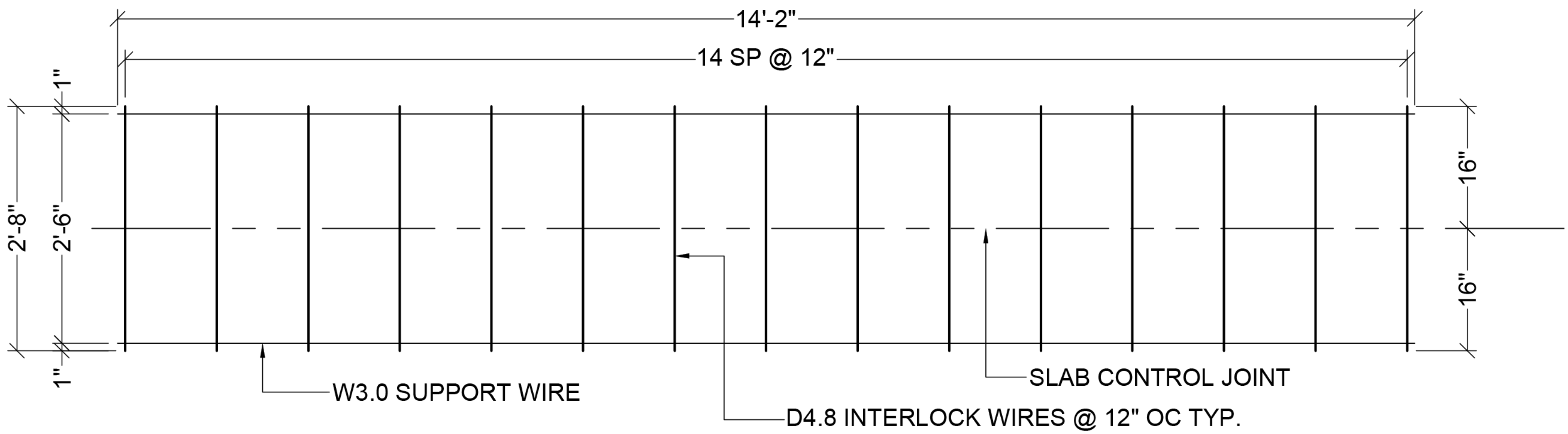


Figure 15: Slab Type 1, Option B: WWR mats "in panel". Dimensions between the concrete panel boundaries (at control joints and slab terminations) and steel wires are illustrated. See Figure 13 for additional information on small overhang projections around perimeter.

ACI 360-10	Calculations	Description
STEP 3: Reinforcement Detailing (continued)		
	Slab Type 1 Steel Area 0.067 in^2/ 0.048 in^2 @ joint *For joint reinforcement (enhanced aggregate interlock):* D4.8 @ 12" on center → A_s = 0.048 in^2 The WWR mat detailed as the "interlock mat" noted in Figure 6 will be a ladder configuration. See Figure 16.	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. The steel area recommended to achieve enhanced aggregate interlock in conjunction with correctly-spaced control joints is outlined in ACI 360.



SLAB TYPE 1 - INTERLOCK MAT @ CONTROL JOINT
WWR-CJ
30X12 W3.0/D4.8 30" (+1",+1") X 14'-2" (1,1)

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)		
	Slab Type 2 Steel Area 0.360 in^2 *For Slab Type 2* A_s = 0.360 in^2 Options: D30.0 @ 10” on center → A_s = 0.360 in^2 D24.0 @ 8” on center → A_s = 0.360 in^2 D18.0 @ 6” on center → A_s = 0.360 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)**	*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.
	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**	

Basic Inputs

Applicable ACI Standard	ACI 318-19	
Reinforcement Type	Welded Deformed Wire Reinforcement	
Wire/Bar Size to be Developed/Spliced	D	24
Diameter of Reinforcement (in)	0.553	
Cross-Sectional Area of Reinforcement (in^2)	0.24	

Development-Specific Inputs

Are Welded Crosswires Available to Contribute to the Development of the Reinforcement?	User Option Per Below
If Welded Deformed Wire Reinforcement, are Available Welded Crosswires Relied Upon by the Designer to Contribute to the Development of the Reinforcement?	NO
Reinforcement Yield Strength, f_y (psi)	70000
Concrete Specified Compressive Strength, f'_c	4000
Modification Factor: Concrete Weight λ	1
Modification Factor: Reinforcement Grade ψ_g	1.15
Modification Factor: Coating ψ_e	1
Modification Factor: Size ψ_s	0.8
Modification Factor: Casting Position ψ_t	1
Product $\psi_t\psi_e$	1

WELDED DEFORMED WIRE REINFORCEMENT			
Spacing of Wires Being Developed	(user entry)	8	inches
Clear Cover Dimension	(user entry)	1.5	inches
Variable c_b		1.7765	inches
Use $K_{tr} = 0$ as a Design Simplification?	(Recommended)	YES	
[redacted]			
Transverse Reinforcement Index K_{tr}		0	
Confinement Term $(c_b + K_{tr})/d_b$		2.5	
Calculated Reference Development Length		16.8928	inches
Prescriptive Minimum Development Length Dimension		12	inches
Welded Deformed Wire Reinforcement Factor ψ_w		1	
REQUIRED MINIMUM DEVELOPMENT LENGTH DIMENSION		**16.9**	**inches**
REQUIRED MINIMUM CLASS B LAP SPLICE LENGTH DIMENSION		**21.97**	**inches**

***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.*

ACI 360-10	Calculations	Description
STEP 3: Reinforcement Detailing (continued)		
	Slab Type 2 Steel Area 0.360 in^2 **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.

<table>
<tr><th>ACI 360-10</th><th>Calculations</th><th>Description</th></tr>
<tr><td colspan="3">STEP 3: Reinforcement Detailing (continued)</td></tr>
<tr><td></td><td>
<table><tr><th>Slab Type 2 Steel Area</th></tr><tr><td>0.360 in^2</td></tr></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”)
</td><td>
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.
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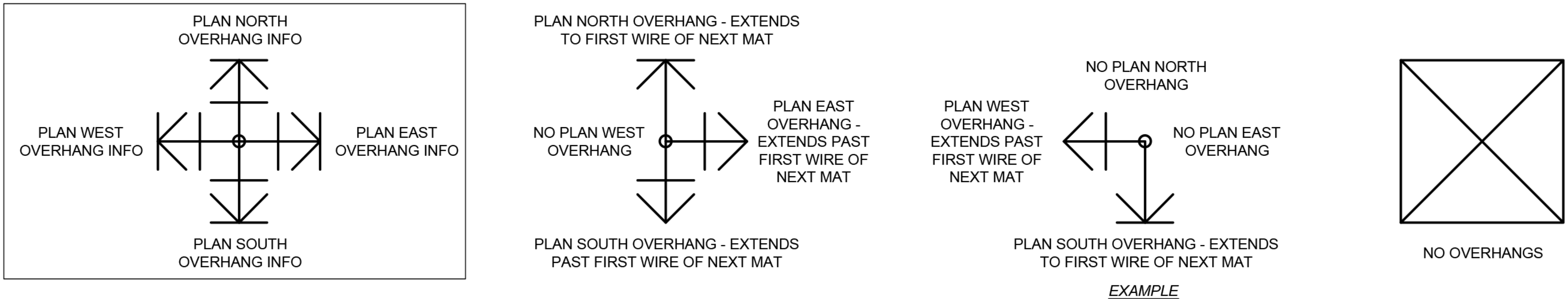


Figure 18: WWR detailing nomenclature – For this example, the WWR detailer utilizes a compass-type system that helps the reviewer quickly identify on the placement plan the presence and general extent of wire overhangs.

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".

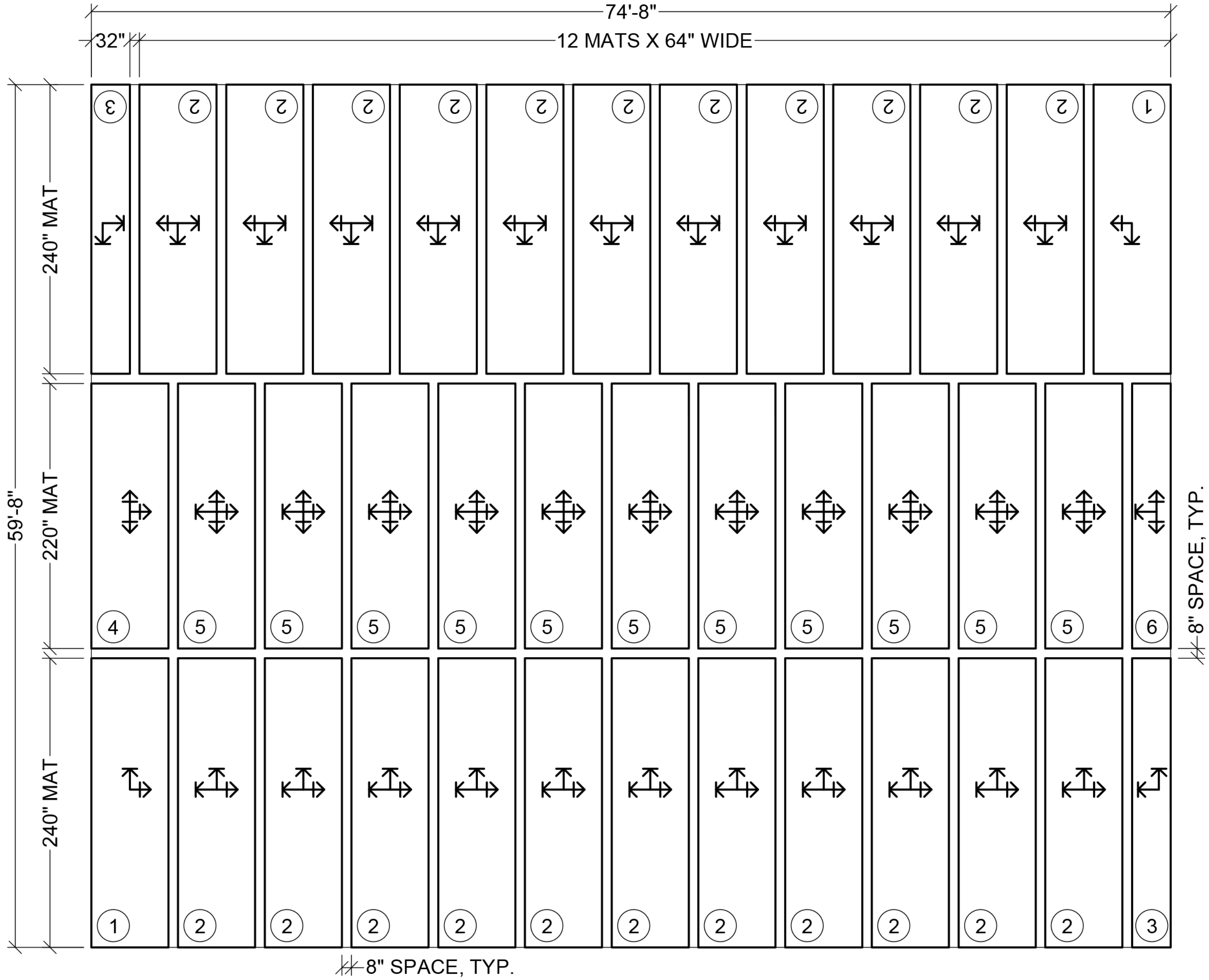
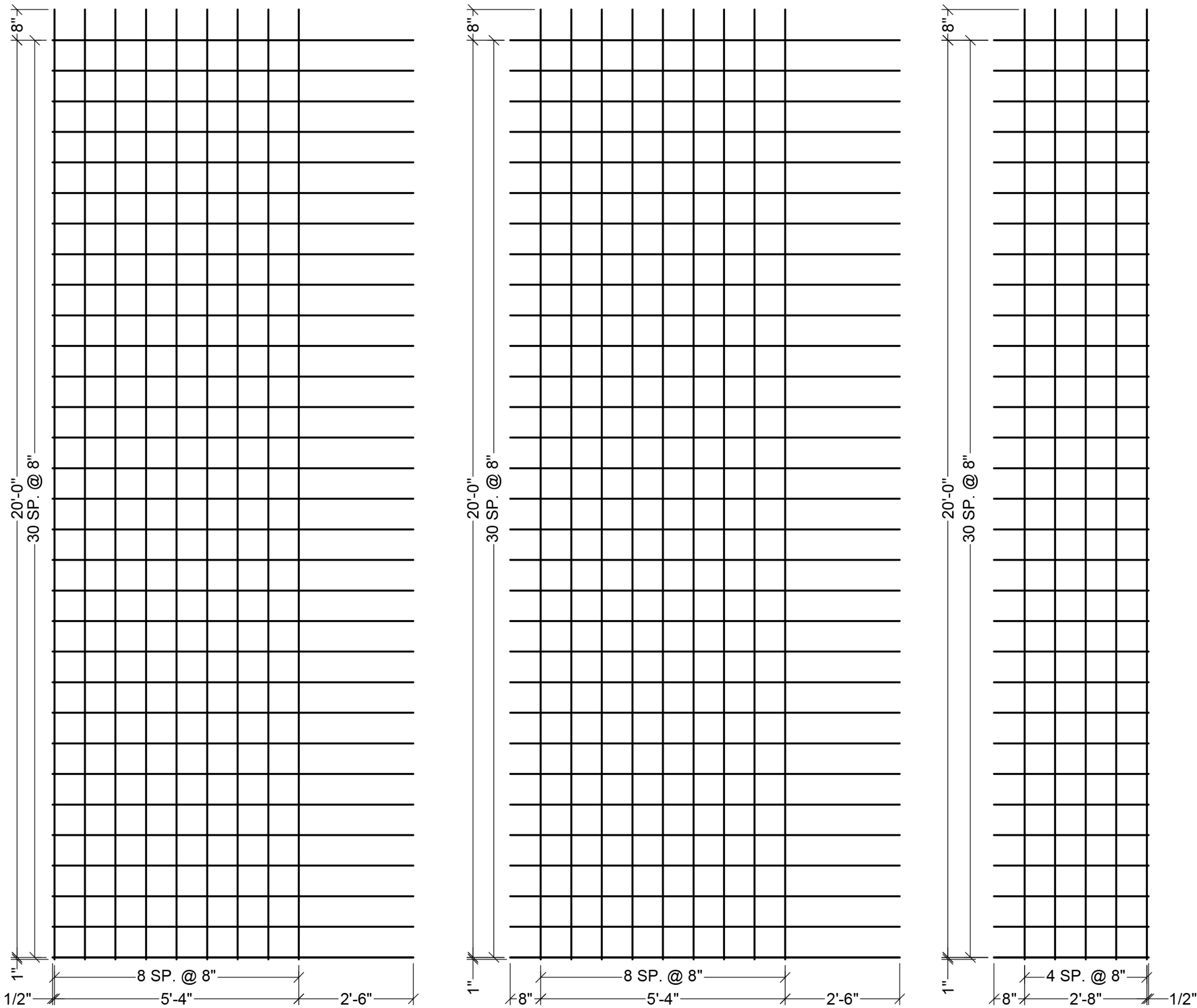


Figure 19: WWR Solution: Option A
Note that overhang dimensions are presented on the individual WWR mat details in Figures 20 and 21. Overhang lengths are configured in conjunction with the typical 8" space between mats to ensure the specified Class B lap splice is achieved.



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 20:
WWR Solution: Option A
Mat configurations 1-3

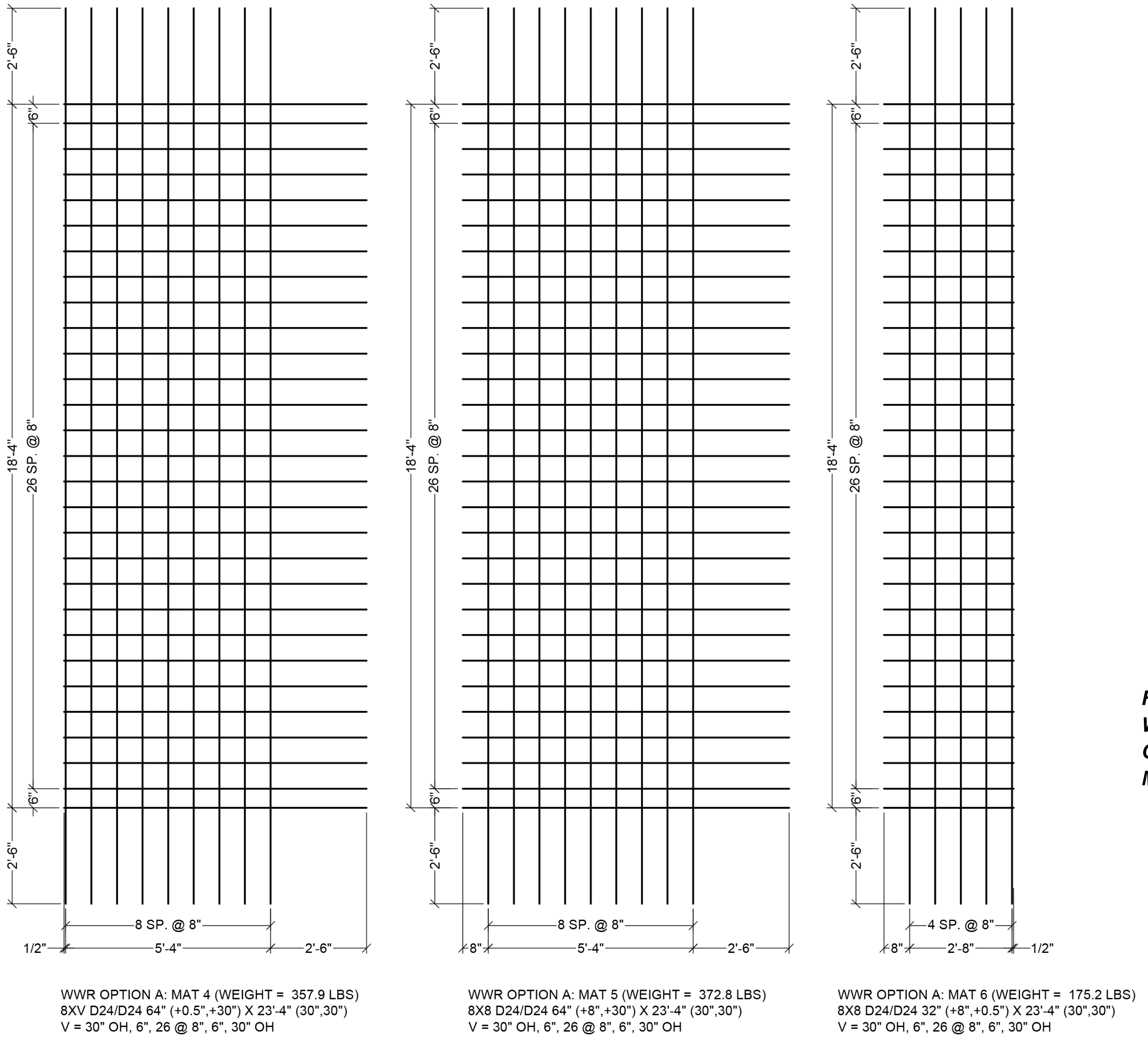


Figure 21:
WWR Solution:
Option A
Mat configurations 4-6

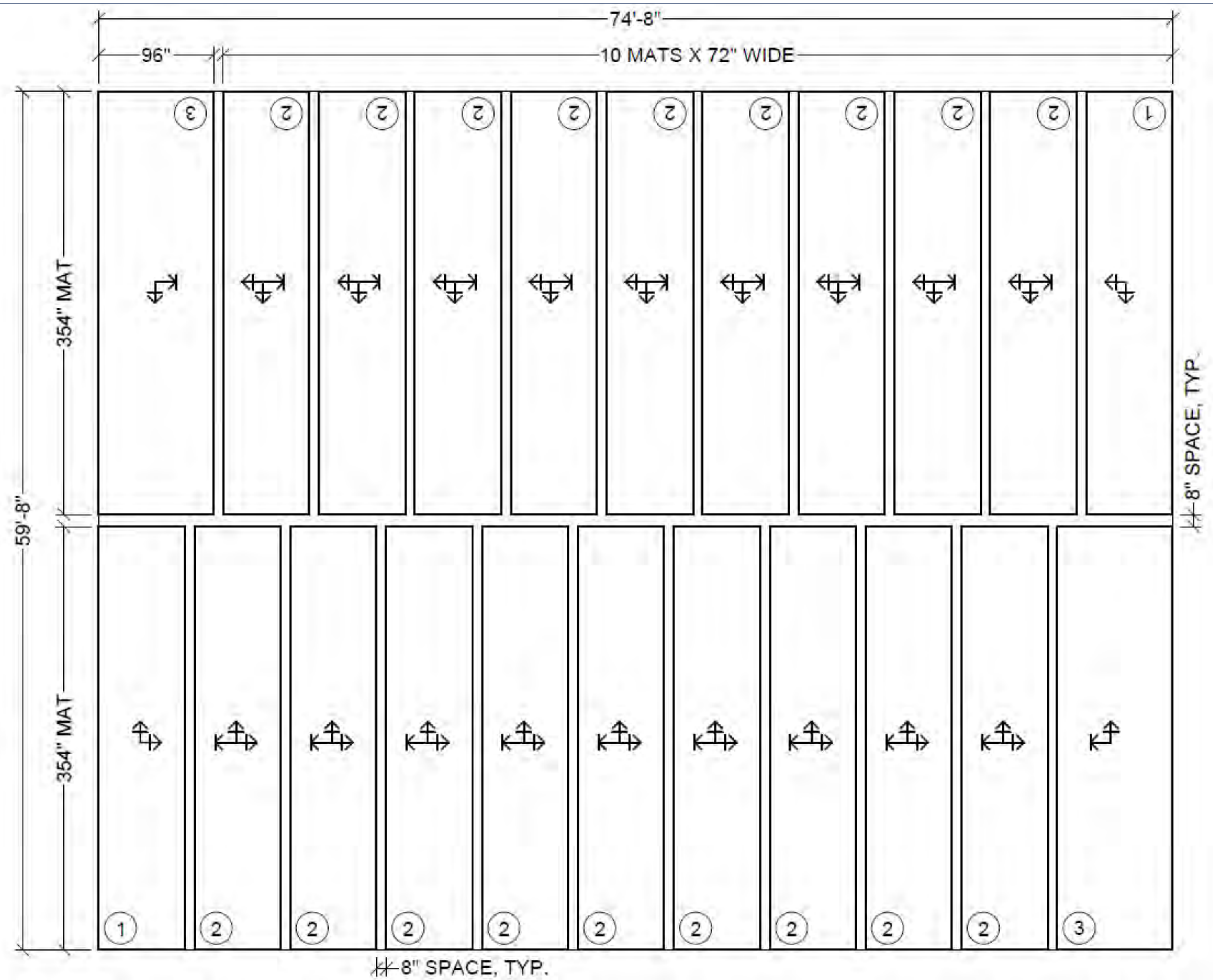
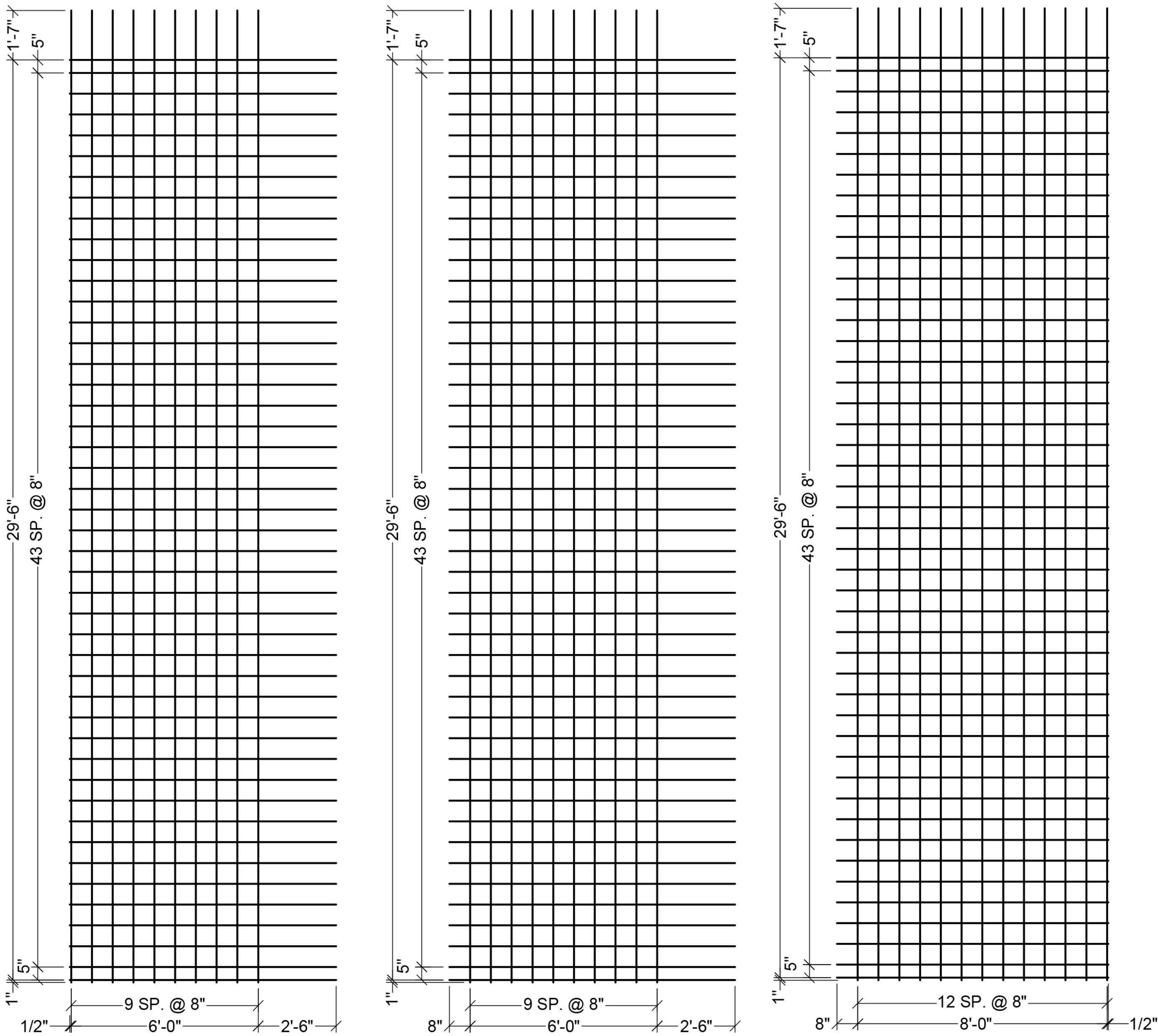


Figure 22: WWR Solution: Option B

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.



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

Figure 23:
WWR Solution: Option B
Mat configurations 1-3

ACI 360-10	Calculations	Description
STEP 3: Reinforcement Detailing (continued)		

Slab Type 2 Steel Area
0.360 in^2

The following is a comparison of Option A and B, outlining considerations relevant to the contractor's handling and placement operations.

Attribute	**Option A**	**Option B**
Coverage area	~4,455 ft^2	~4,455 ft^2
Total number of mats	39	22
Average coverage per mat	~144.2 ft^2	202.5 ft^2
Unique mat types	6	3
Mat weight range	173 – 373 lbs	575 – 658 lbs
Mechanized installation[1]	No	Yes
Total steel weight	13,937.50 lbs	13,247.50
Standard freight[2]?	Yes	No

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.

Chapter Seven

EXAMPLE: Tilt-up Panels

EXAMPLE 5 PROBLEM STATEMENT:

The subject structural wall panel configurations are excerpts from a set of construction drawings for a large-volume, single-story distribution center. Two reinforced concrete tilt-up wall panels are selected for evaluation as part of this example.

The wall panels serve as the primary structural support for the roof structure as well as comprising the building's cladding system. The roof system is comprised of open web steel joists and joist girders, both of which derive direct support from the panels (i.e., there is no structural steel framing line of beams and columns directly adjacent to the wall panels). In addition to providing direct support for gravity loading, the panels are also relied upon for lateral stability under wind or seismic loading. A continuous reinforced concrete shallow footing system is constructed to support the wall panels.

The wall panels are designed by the Engineer of Record per the requirements of ACI 318-19. Walls are considered ordinary reinforced concrete shear walls in accordance with ASCE 7-16, and as such are not categorized as special structural walls noted in Table 20.2.2.4(a) of ACI 318-19.

Design Criteria are as follows: $f'_c = 4{,}000\ psi, \beta_1 = 0.85$

$f_y = 60{,}000\ psi\ (reinforcing\ bar\ yield\ strength)$

$Concrete\ Density = 0.145\ kcf, normalweight\ concrete\ \lambda = 1.0$

$Clear\ cover$: 1.5" interior face, 2.25" exterior face (1.5" clear +0.75" reveal depth)

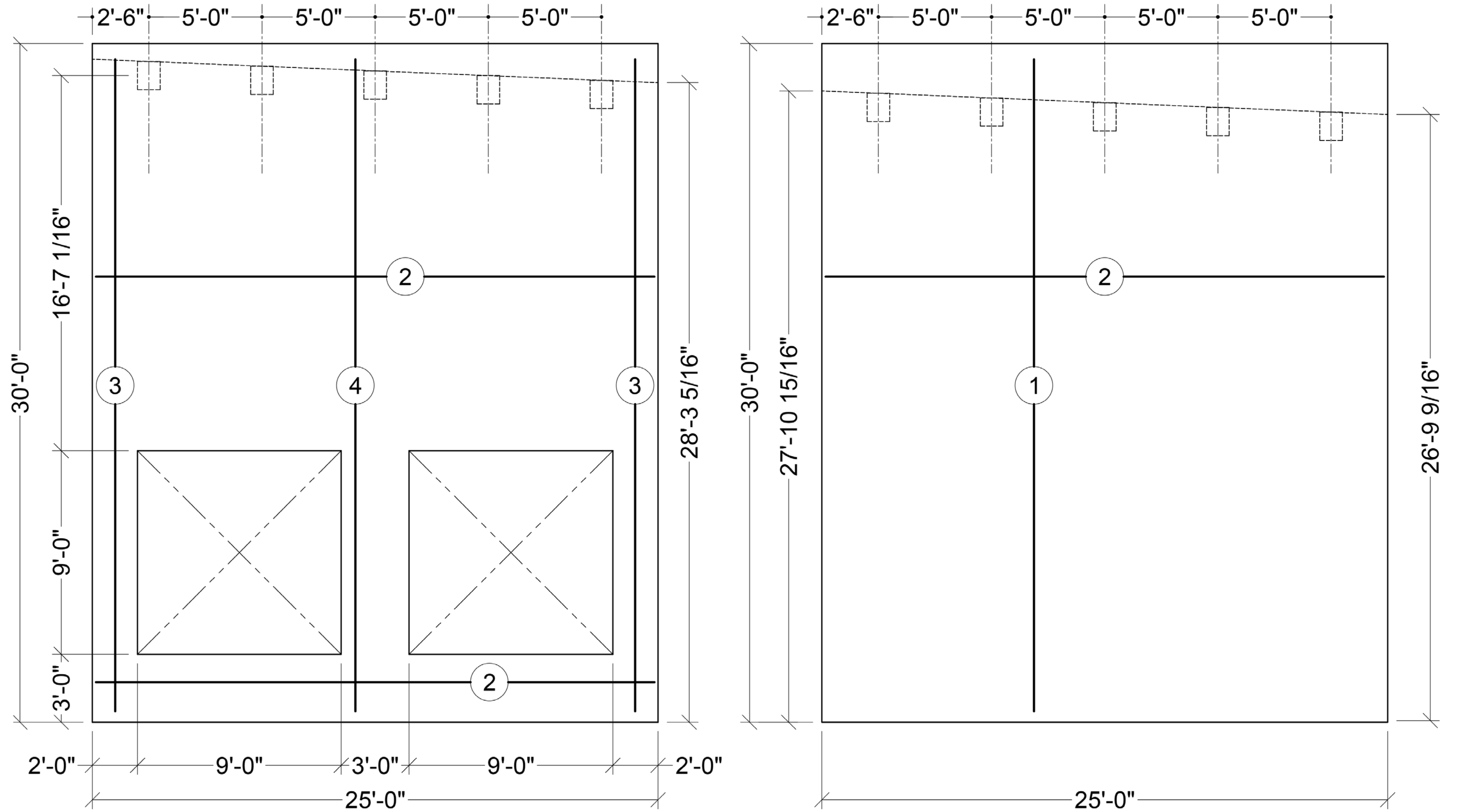


Figure 1: Excerpt from contract structural drawings to be used for construction

The example includes the following steps:

Step 1 – Summary of Engineer's Structural Design Routine and Typical Details
Step 2 – WWR Detailer: Determination of WWR Inclusions and Exclusions
Step 3 – WWR Detailer: Reinforcement Conversion Routine for WP-1
Step 4 – WWR Detailing for WP-1
Step 5 – WWR Detailer: Reinforcement Conversion Routine for WP-2
Step 6 – WWR Detailing for WP-2

ACI 318-19	Calculations	Description
STEP 1: Summary of Structural Design Routine and Typical Details		
		The Structural Engineer of Record's design routine requires reconciliation of both gravity and lateral loading. Due consideration is made by the engineer for the eccentric alignment of roof loading relative to the wall panel centerline, which introduces additional flexure in the wall section. Panel out-of-plane mid-height displacement due to thermal bowing is also acknowledged in the form of additional eccentricity applied to the concentrated forces due to roof joists incurred by the panel. The panel is restrained at three levels: the roof diaphragm, the slab-on-ground, and the foundation. The following figures illustrate design attributes, as well as the resulting content that would be presented on the contract structural drawings. **Figure 2:** Designer's Basic Wall Panel Model with Fixities **Figure 3 through 6:** Designer's "Typical" Details **Figure 7:** Designer's Project-Specific Reinforcement Schedule **Figure 8:** Designer's Project-Specific Panel Elevations **Figure 9:** Designer's General Notes Excerpt

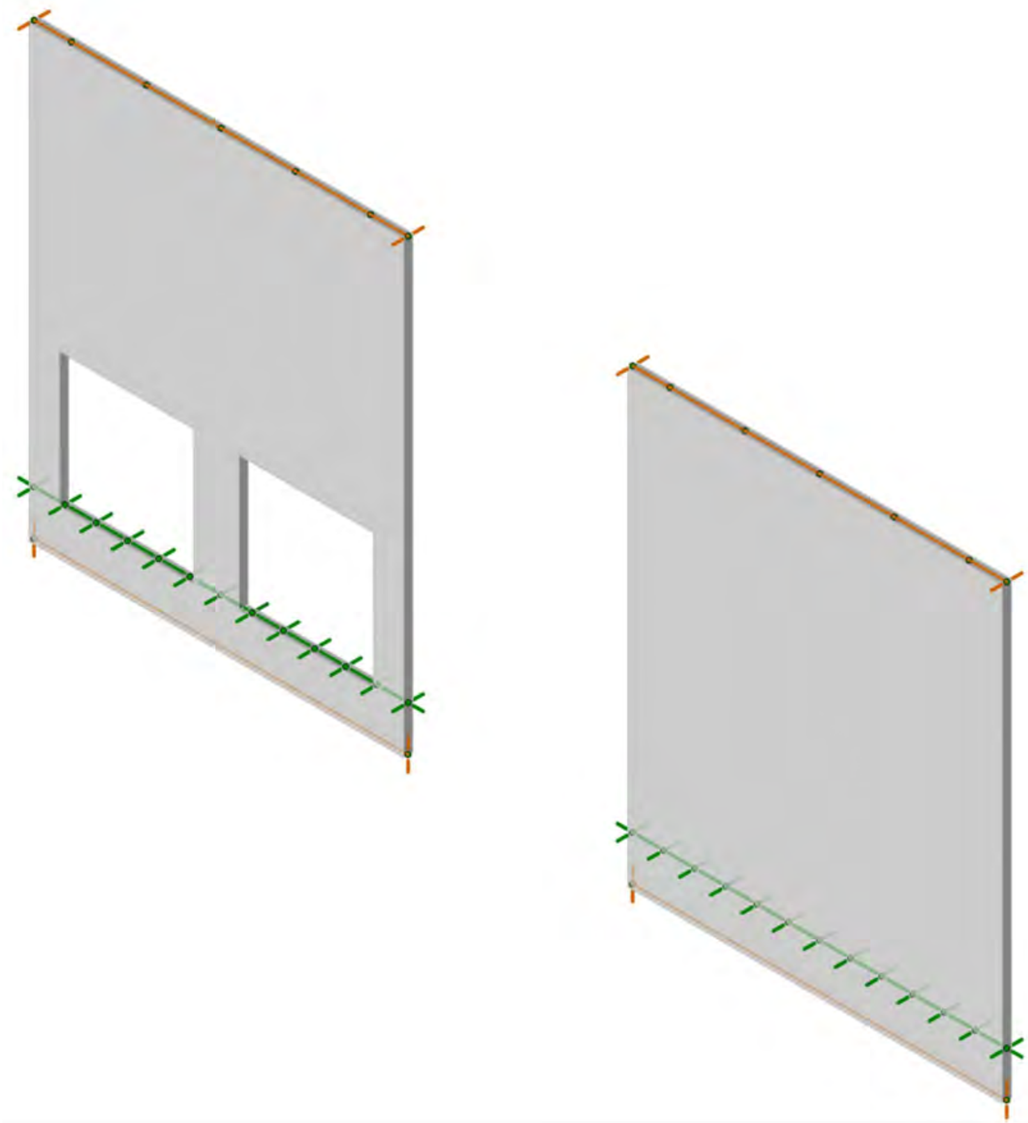


Figure 2: Excerpt from the engineer's design model showing panel proportions and fixities

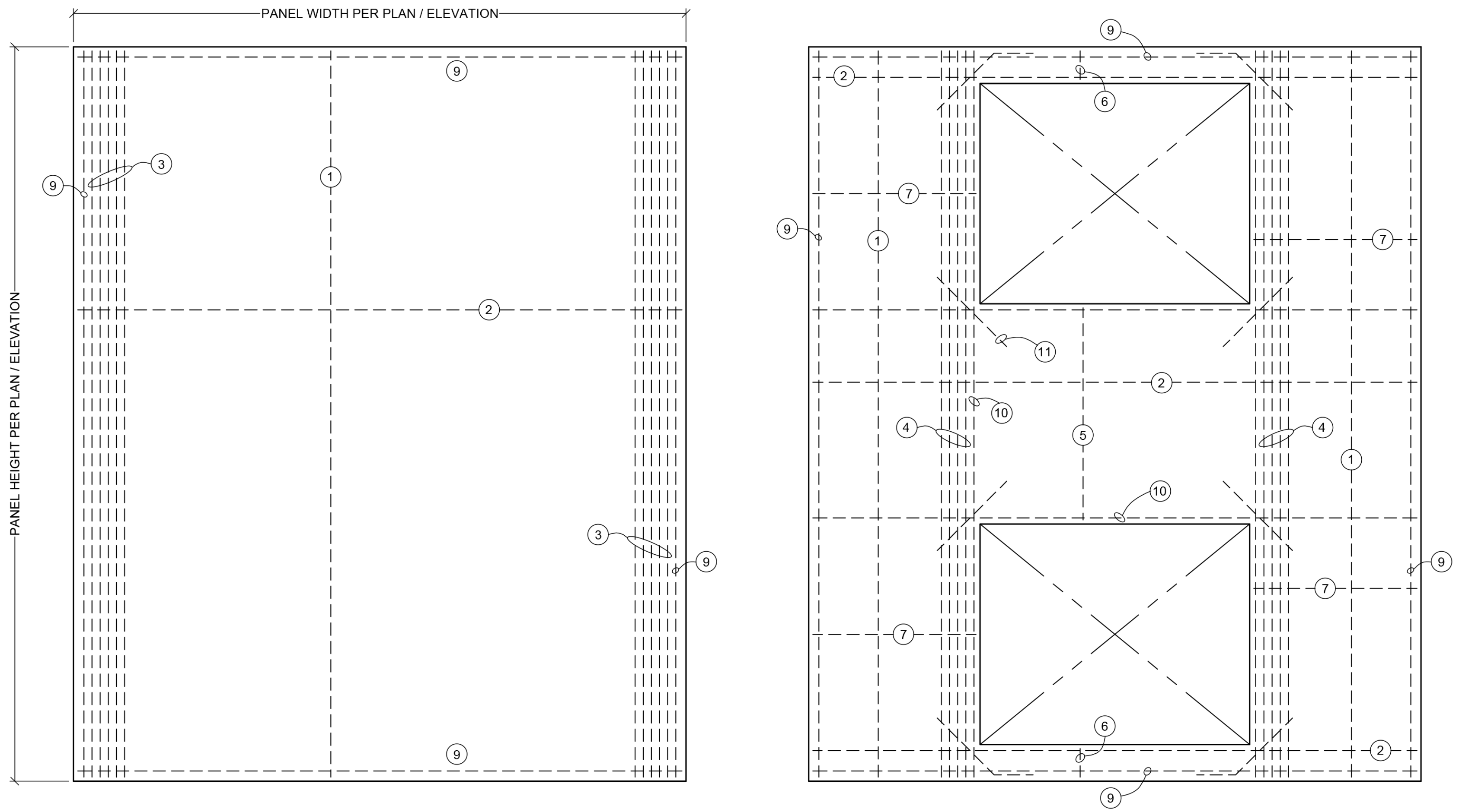
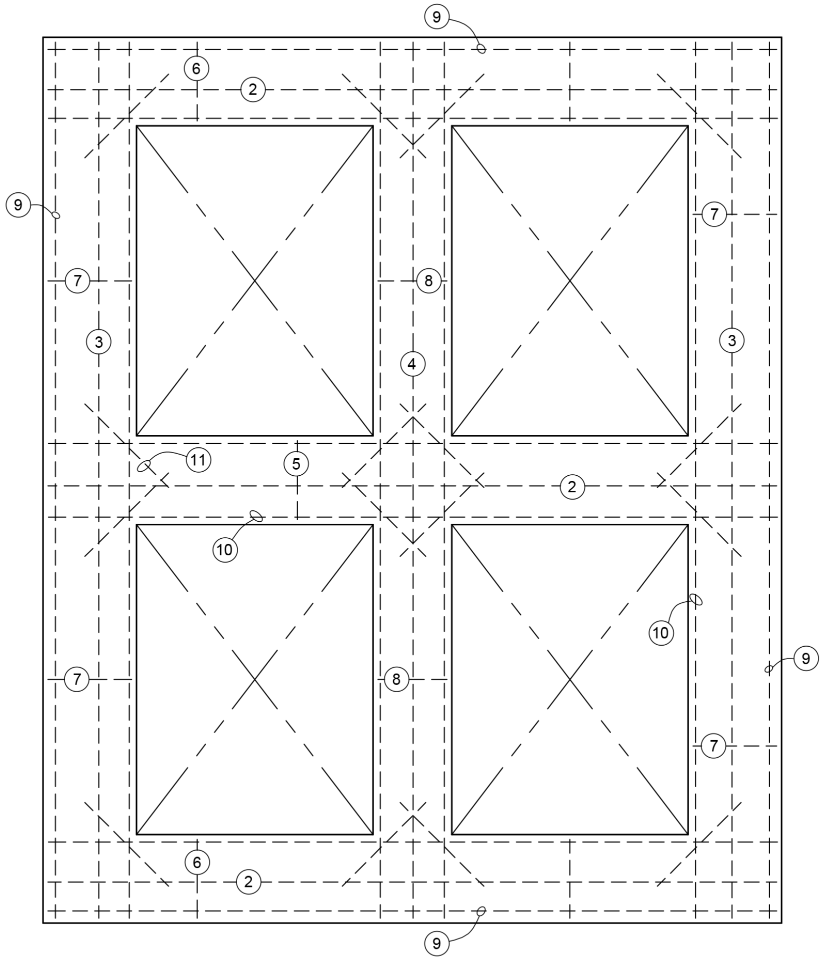


Figure 3: Engineer's reference panel elevations, showing typical reinforcement call-outs (continued on next page)



1 TYPICAL CONTINUOUS VERTICAL BAR, EACH FACE

2 TYPICAL CONTINUOUS HORIZONTAL BAR, EACH FACE

3 END VERTICAL BAR, EACH FACE (IN LIEU OF TYPICAL VERTICALS)

4 JAMB VERTICAL BAR, EACH FACE (IN LIEU OF TYPICAL VERTICALS)

5 SPANDREL VERTICAL BAR 'A', EACH FACE

6 SPANDREL VERTICAL BAR 'B', EACH FACE

7 JAMB HORIZONTAL BAR 'A' (EACH FACE, OR CLOSED HOOP WHERE INDICATED)

8 JAMB HORIZONTAL BAR 'B' (EACH FACE, OR CLOSED HOOP WHERE INDICATED)

9 PANEL PERIMETER TRIM BAR, EACH FACE, 1 1/2" CLEAR FROM PANEL EDGE. TRIM BARS TO MATCH SIZE OF ADJACENT VERTICALS AND HORIZONTALS.

10 ALL OPENINGS SHALL BE PROVIDED WITH (1) #5 TRIM BAR, EACH FACE. TRIM BARS AT OPENINGS SHALL BE HELD 2" CLEAR FROM EDGES OF OPENINGS, SHALL RUN UNINTERRUPTED FOR FULL HORIZONTAL AND VERTICAL DIMENSIONS, AND SHALL TERMINATE IN STANDARD 180-DEGREE HOOKS. COMMON TRIM BARS ARE PERMITTED TO BE USED FOR MULTIPLE OPENINGS SHARING COMMON ALIGNMENT.

11 ALL OPENING CORNERS SHALL BE PROVIDED WITH (1) #4 CORNER BAR X 4'-0" LONG, EACH FACE. WHERE CORNER BAR STRAIGHT LENGTH IS TRUNCATED AT A PANEL BOUNDARY OR ADJOINING OPENING, BAR SHALL BE BENT AS REQUIRED AND TERMINATED WITH 8" EXTENSION.

PERFORATED PANELS SHOWN AT LEFT ARE ILLUSTRATIVE ONLY, AND ARE INTENDED TO SHOW GENERAL REINFORCEMENT DISTRIBUTIONS RELATIVE TO THE PRESENCE OF PENETRATIONS. FIGURES SHOWN DO NOT NECESSARILY REFLECT PROJECT SPECIFIC PERFORATED PANEL CONFIGURATIONS. SEE PROJECT-SPECIFIC PANEL ELEVATIONS FOR MORE INFORMATION.

COORDINATE REINFORCEMENT POSITIONING TO ENSURE COMPATIBILITY WITH ALL EMBEDS, BLOCK-OUTS, AND CAST-IN COMPONENTS. SEE OTHER DETAILS FOR THESE ELEMENTS AND THE ASSOCIATED CONNECTIVITY TO FOUNDATIONS AND ELEVATED DIAPHRAGMS / FRAMING ASSEMBLIES.

Figure 3 (continued): Engineer's reference panel elevations, showing typical reinforcement call-outs. Note that these elevations are illustrative only and are not necessarily project-specific. Their inclusion in the contract drawings is not only worthwhile to give the contractor clarification on general positioning of the various numbered reinforcement "categories", but is required as it relates to the trim (9 and 10) and corner (11) reinforcement that will be present in <u>all</u> wall panels.

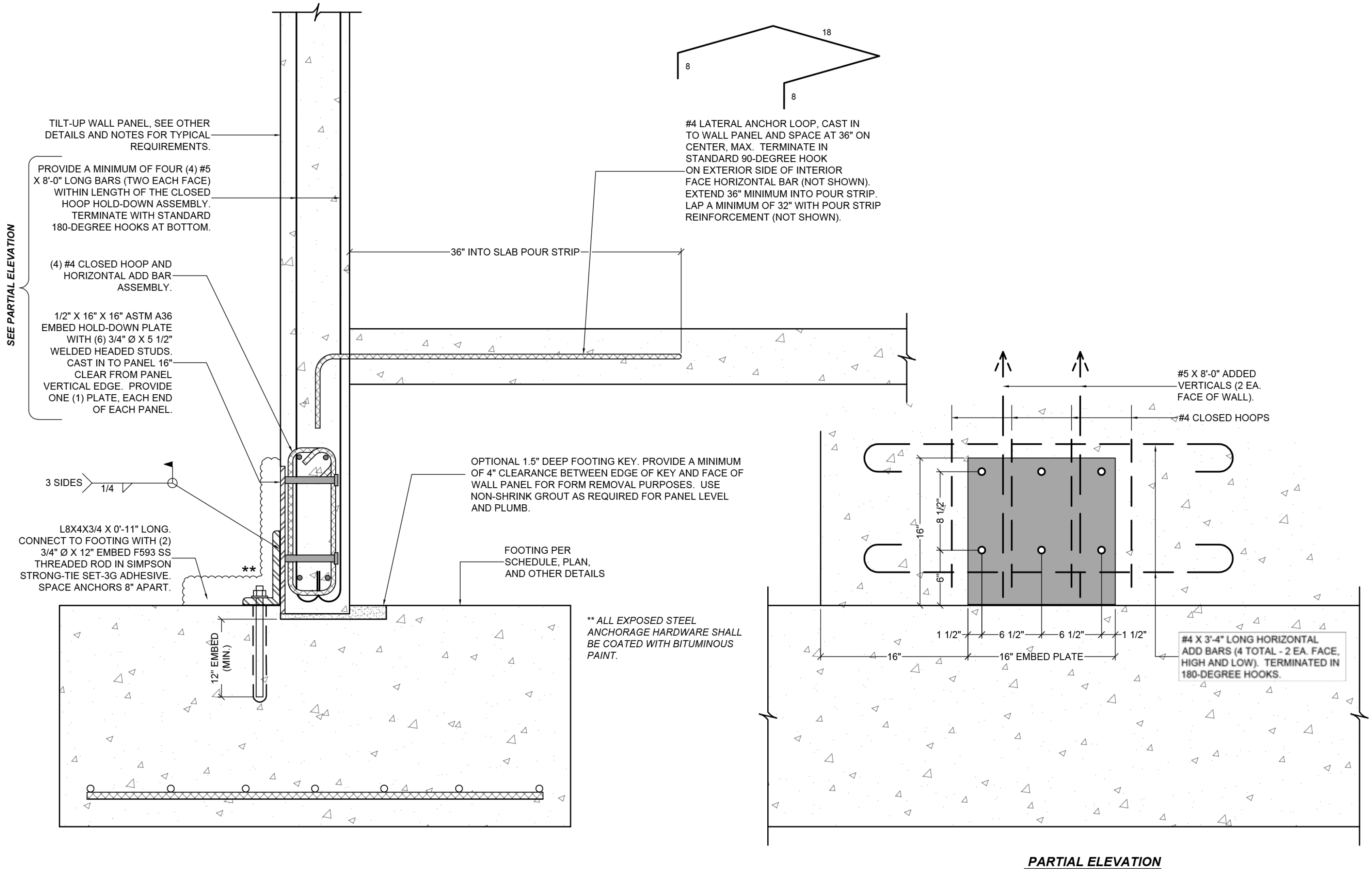


Figure 4: ***Engineer's typical detail showing wall panel base anchorage to the foundation level and to the slab. It is important to note that not all wall panels require physical anchorage to the foundation. Generally, foundation connection is limited to those panels that are subjected to overturning forces not capable of being resisted by the weight of the panel itself.***

Both the foundation and the slab connections indicated are notable from the WWR detailer's standpoint because they represent reinforcement components that (a) can potentially be furnished in WWR form and (b) must be spatially addressed and resolved as they relate to avoidance of interruption/conflict with other "primary" wall reinforcement mats.

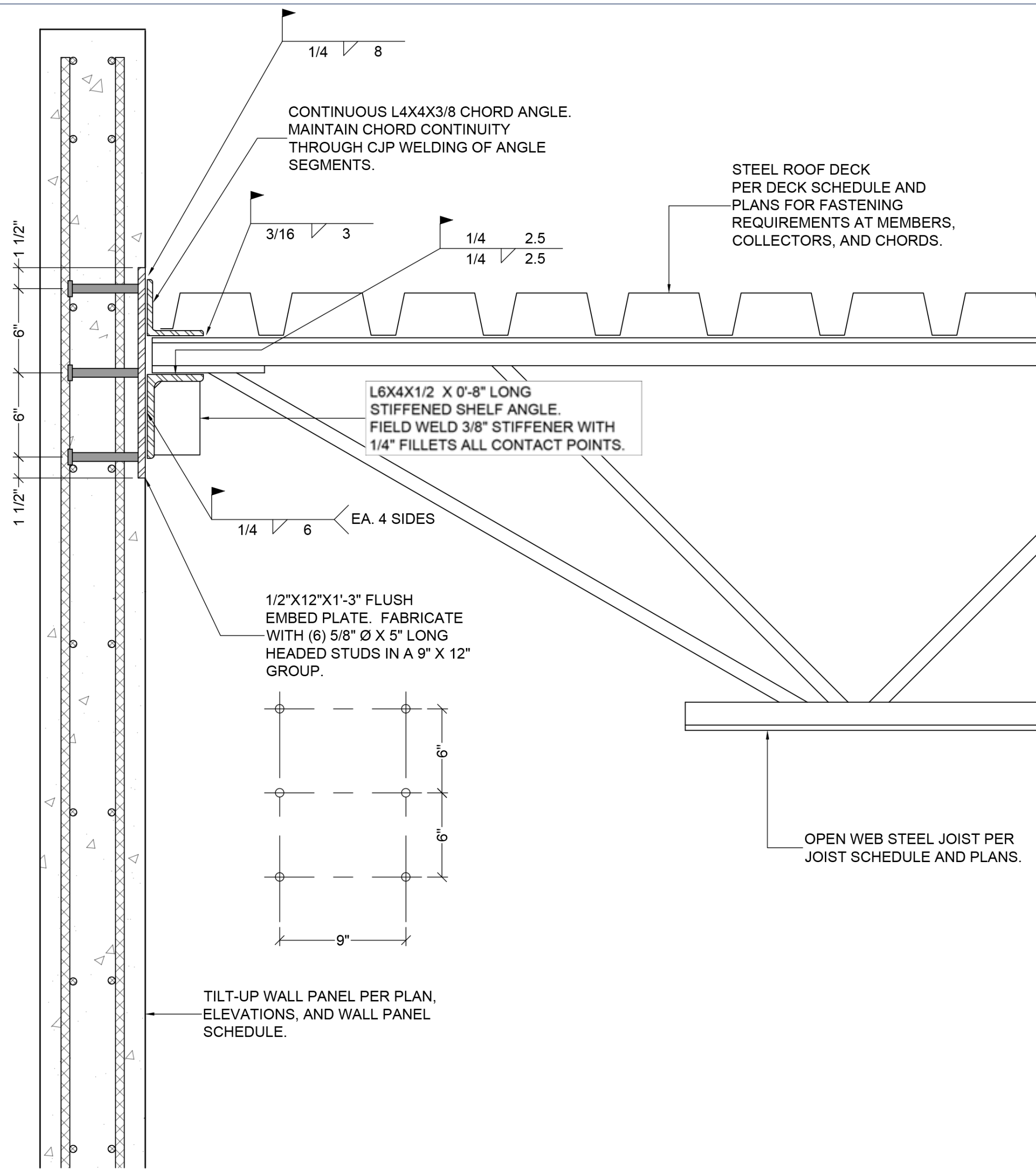


Figure 5: Engineer's typical detail showing roof framing attachment. This information is notable for the WWR detailer in order to identify embedded anchorage features that could result in spatial conflict with intended positioning of the wall reinforcement. The presence of embedded components requires special attention when WWR mat configurations are being derived to ensure compatible alignment. (continued on next page)

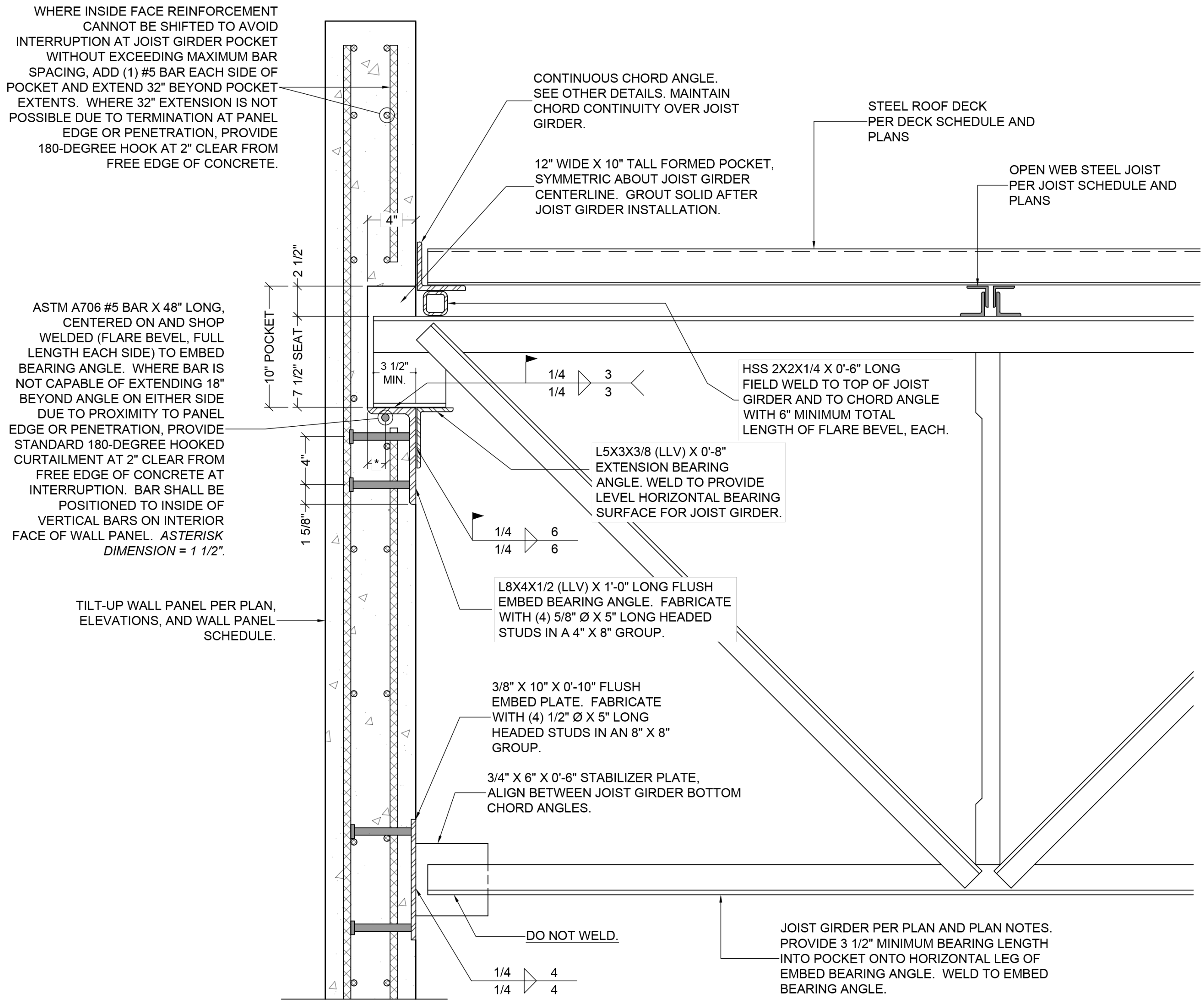


Figure 5 (continued): Engineer's typical detail showing roof framing attachment. This version of the detail shows a formed pocket into which connection hardware and roof framing are ultimately positioned. Again, from the WWR detailer's point-of-view, these cast-in and embedded components are items that must be acknowledged in the geometry of the WWR mats to be used on the project. (Note that this version of the detail ultimately does not apply to this example.)

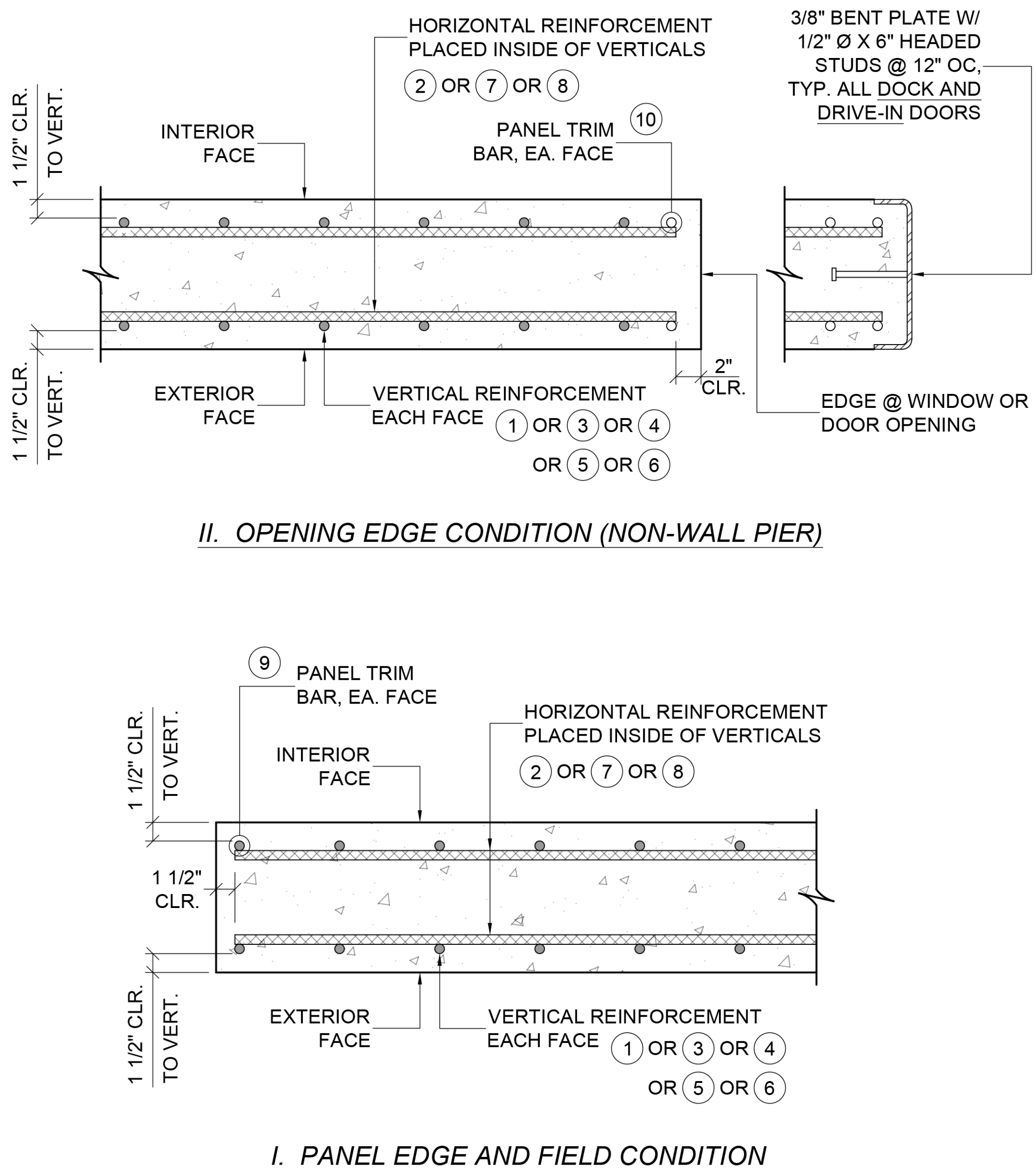


Figure 6: Engineer's typical detail showing panel reinforcing arrangement, used in conjunction with project-specific information in Figure 7 and 8.

TILT-UP WALL PANEL SCHEDULE									
PANEL	THICK.	1	2	3	4	5	6	7	8
WP-1	7.5"	N/A	#4 @ 14" OC E.F.	(4) #5 E.F.	(5) #5 E.F.	#5 @ 14" OC E.F.	#5 @ 14" OC E.F.	N/A	N/A
WP-2	7.5"	#5 @ 14" OC E.F.	#4 @ 14" OC E.F.	N/A	N/A	N/A	N/A	N/A	N/A

TILT-UP WALL PANEL SCHEDULE NOTES

1. REFER TO TYPICAL TILT-UP WALL PANEL REINFORCEMENT DETAILS AND PROJECT-SPECIFIC PANEL ELEVATIONS FOR REINFORCEMENT TYPES DENOTED IN SCHEDULE. REFER TO TYPICAL TILT-UP WALL PANEL REINFORCEMENT DETAILS FOR REQUIRED TRIM BARS AND CORNER BARS, NOT SCHEDULED HERE.
2. INFORMATION SCHEDULED HERE SHALL BE WORKED IN CONJUNCTION WITH FOUNDATION, FLOOR, AND ROOF SECTIONS/DETAILS TO ENSURE FULLY COORDINATED INSTALLATION OF ALL RELATED EMBEDS, BLOCK-OUTS, AND CAST-IN ELEMENTS.
3. PANELS DENOTED WITH AN ASTERISK (*) SHALL BE FURNISHED WITH HOLD-DOWN ASSEMBLIES SHOWN IN THE TYPICAL TILT-UP PANEL FOOTING ANCHORAGE DETAILS.
4. PANEL LIFT DESIGN AND SEQUENCE IS BY OTHERS. LIFTING HARDWARE AND SUPPLEMENTAL CAST-IN FEATURES SHALL NOT COMPROMISE OR ALTER STRUCTURAL REINFORCEMENT AND STRUCTURAL CAST-IN COMPONENTS DEFINED HEREIN. PANEL LIFTING POINTS AND PROCESS SHALL BE AS REQUIRED TO ENSURE 80% OF THE DESIGN STRENGTHS ASSOCIATED WITH DETAILED REINFORCED CONCRETE ASSEMBLIES HEREIN ARE NOT EXCEEDED.

Figure 7: Engineer’s project-specific wall panel reinforcement schedule, used in conjunction with Figure 6 and 8.

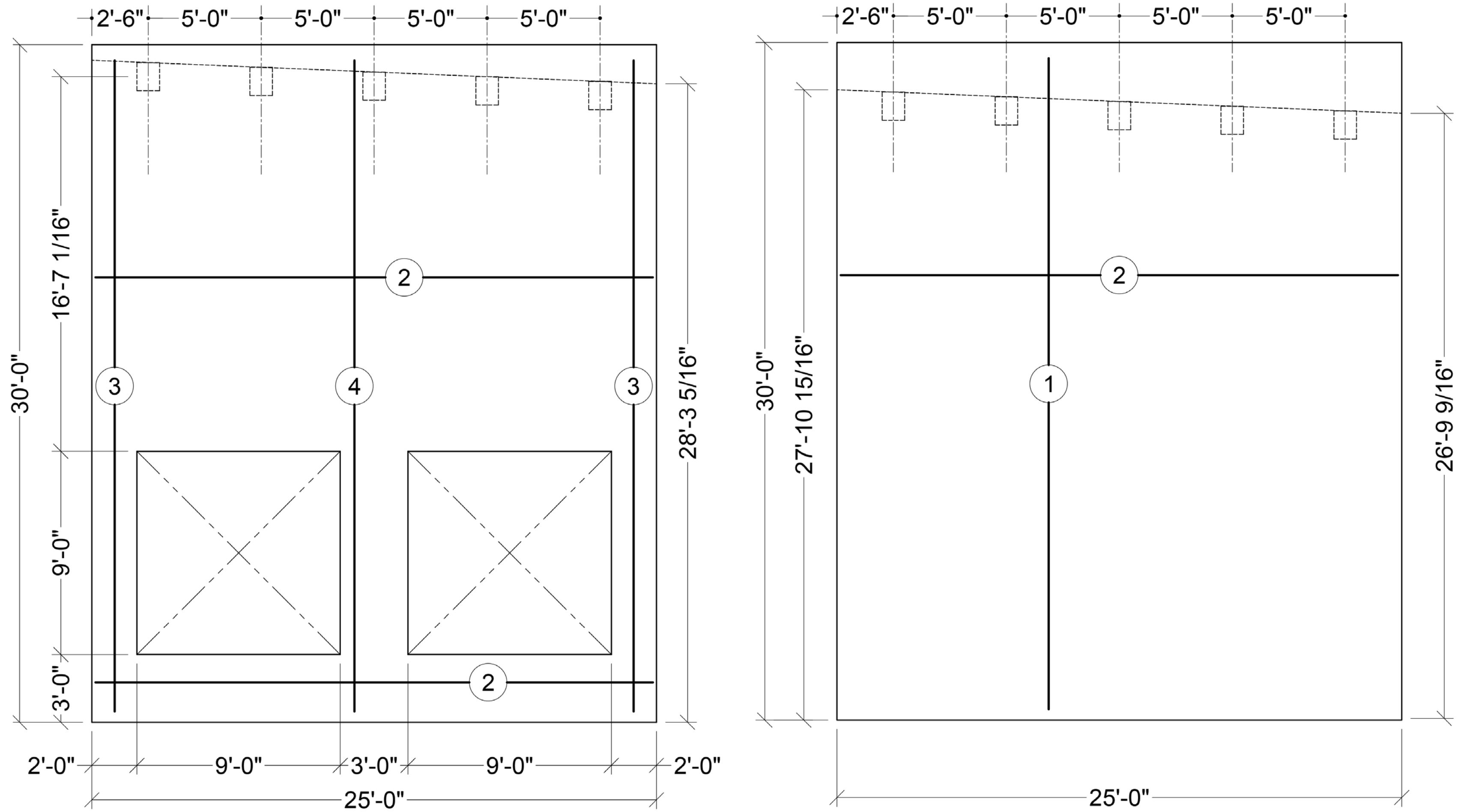


Figure 8: Engineer's project-specific wall panel reinforcement elevations. Use in conjunction with Figure 6 and 7.

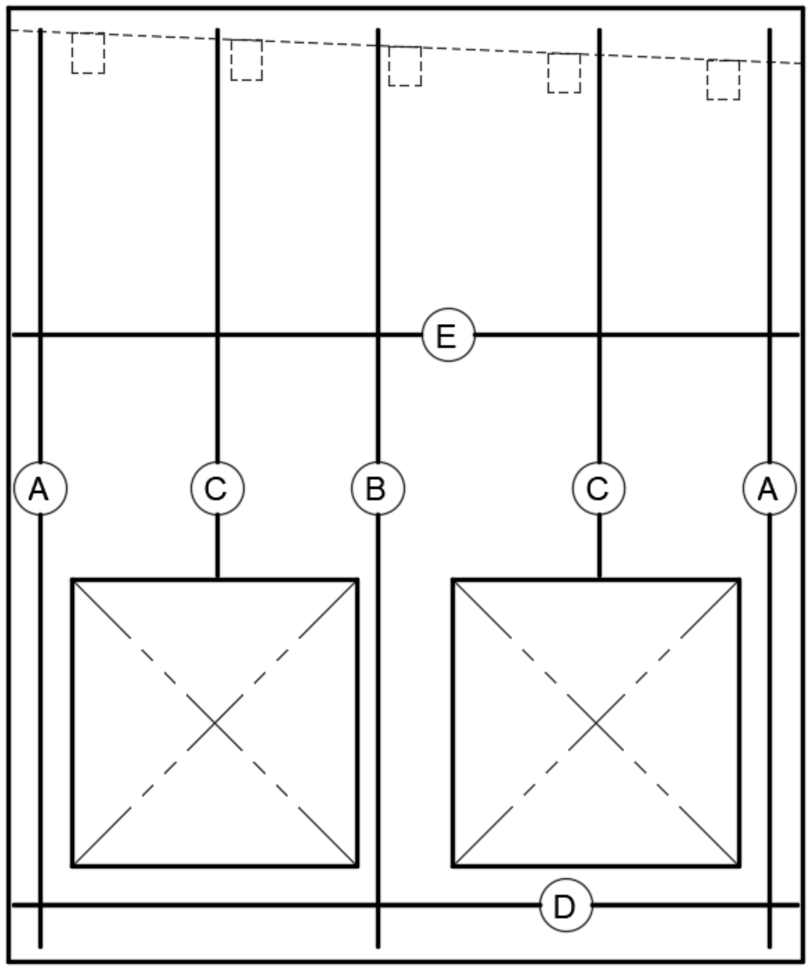
MILD REINFORCING STEEL

1. DEFORMED REINFORCING BARS (REBAR) SHALL CONFORM TO ASTM A615, GRADE 80. BARS SHALL BE LAPPED IN ACCORDANCE WITH THE REBAR LAP SCHEDULES UNLESS OTHERWISE EXPLICITLY DETAILED.
2. LONGITUDINAL REINFORCEMENT IN SPECIAL MOMENT FRAME BEAMS AND COLUMNS, AND VERTICAL AND HORIZONTAL REINFORCEMENT IN SPECIAL STRUCTURAL (SHEAR) WALLS SHALL BE ASTM A706 GRADE 60 OR GRADE 80 AS NOTED. TENSILE AND ELONGATION PROPERTIES SHALL BE CONFIRMED THROUGH MILL REPORT DOCUMENTATION PROVIDED AS PART OF THE PROJECT REINFORCEMENT SUBMITTAL.
3. WELDED DEFORMED WIRE REINFORCEMENT SHALL CONFORM TO ASTM A1064 (GRADE 80) AND SHALL BE PROVIDED IN SHEET FORM. REINFORCEMENT SHEETS SHALL BE MANUFACTURED WITH OVERHANG LENGTHS SUFFICIENT TO ACHIEVE A LAP SPLICE LENGTH EQUAL TO THE GREATER OF 12 INCHES OR THE LAP SPLICE DIMENSION SHOWN IN THE REBAR LAP SCHEDULE FOR BAR OF EQUAL (OR GREATER) DIAMETER AND GRADE, UNLESS OTHERWISE NOTED. SHEETS AND ASSOCIATED LAP REGIONS SHALL BE INSTALLED COPLANAR SO AS TO NOT “STACK”.
4. WELDED DEFORMED WIRE REINFORCEMENT OF EQUAL AREA, EQUAL OR LESSER SPACING, AND IDENTICAL CURTAILMENT (HOOKS AND LAP SPLICES) IS PERMITTED AS A SUBSTITUTION FOR DEFORMED REINFORCING BARS, EXCEPT IN THE FOLLOWING STRUCTURAL APPLICATIONS:
   A. LONGITUDINAL STEEL IN SPECIAL MOMENT FRAMES
   B. VERTICAL AND HORIZONTAL STEEL IN SPECIAL STRUCTURAL WALLS
5. ALL REINFORCING STEEL SHALL BE SECURELY TIED AND ANCHORED IN PLACE

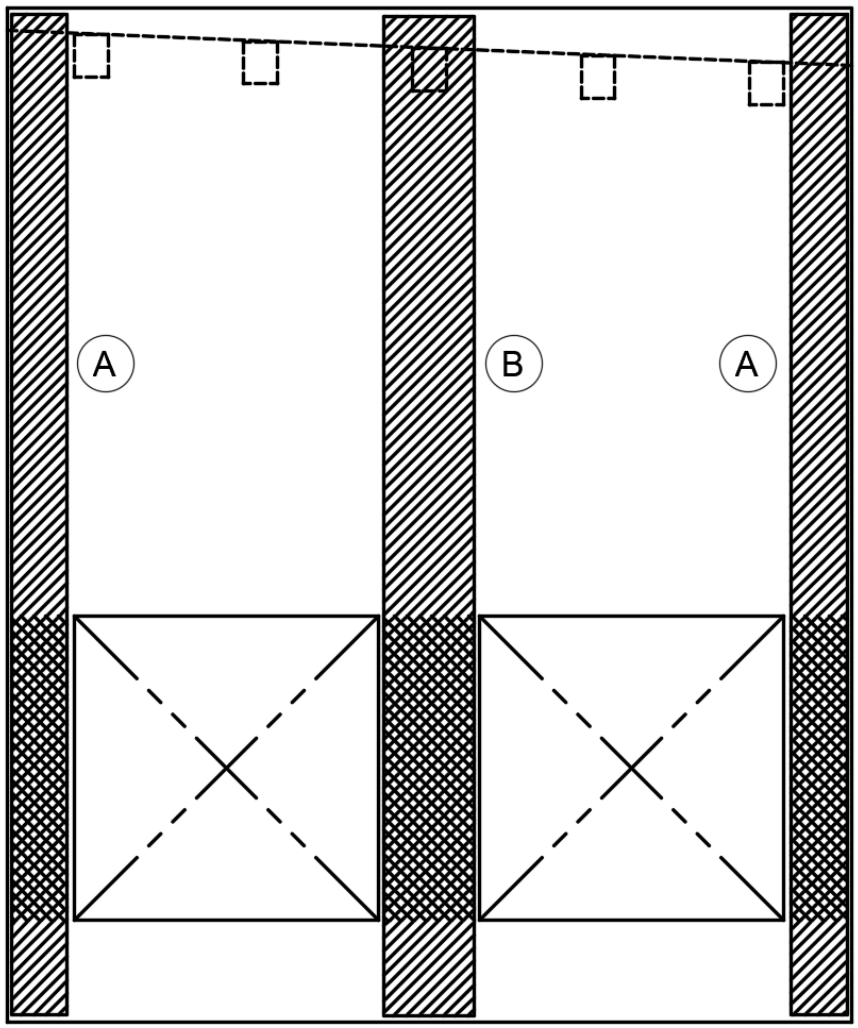
Figure 9: Excerpt from contract documents showing the design professional’s permissive WWR substitution language

ACI 318-19	Calculations	Description
STEP 2: Determination of WWR Inclusions and Exclusions		
20.2.1.7 20.2.2.4		Deformed wire sizes between D4 and D31 are permitted. Welded wire reinforcement is permitted for the tilt-up wall panel application (walls are not “special structural walls”).
	1. The EOR allows for WWR substitution per the Mild Reinforcing Steel General Notes (Figure 9). 2. Individual corner bars (Figure 3) will be excluded from the WWR portion of the reinforcement submittal and will remain reinforcing bars. 3. Panel and opening trim bars will be included in the WWR portion of the reinforcement submittal (bars will be converted to wires). 4. The tilt-up wall panel schedule (Figure 7) indicates that neither wall panel WP-1 nor WP-2 require mechanical hold-down assemblies as detailed in Figure 4. As such, the reinforcement components of these assemblies will not need to be considered for (a) WWR conversion or (b) spatial conflict resolution. 5. The presence of embed plates noted in the first illustration of figure 5 must be coordinated when WWR mat configurations are derived for WP-1 and WP-2. 6. The presence of panel openings in WP-1, resulting in somewhat irregular reinforcement layouts, must be coordinated when WWR mat configurations are derived for WP-1.	The WWR Detailer reviews the structural drawings and identifies notable benchmarks for carrying out the detailing of welded wire reinforcement. As is commonly deployed for numerous other cast-in-place concrete structures requiring reinforcement layout flexibility to achieve the intended design, the WWR detailer for this tilt-up wall panel project will rely heavily on single-direction mats (structural wires in one direction with non-structural “holding” wires in the other direction) to resolve the specified configurations of vertical bars and horizontal bars independent of each other. The use of one-directional WWR mats is prevalent throughout the Welded Wire Reinforcement Design and Detailing Guide and can be seen in several other example chapters.

ACI 318-19	Calculations	Description
STEP 3: WWR Detailer: Reinforcement Conversion Routine for WP-1		
R25.5.2.1		The following figures illustrate the WWR Detailer's allocation of panel reinforcement areas into generalized WWR mat regions. Worth noting is the fact that WWR mat lap splices are not being introduced anywhere in the panel. There exist geometric width limitations related to both transport and manufacture of WWR, with 8'-6" being a maximum width requiring consideration to avoid the extra expense of wide load transport, and 12'-0" being the generally accepted maximum manufactured width of a WWR mat coming off the welding machine at the plant. From a WWR detailing standpoint, one might be tempted to offset the aforementioned geometric limitations by utilizing multiple mats "laced" together by lap splicing, however, this approach could result in the positioning of lap splices in proximity to points of high tensile stress within the panel, a practice that is discouraged in ACI 318-19 and all but prohibited by practicing engineers in the prevailing detailing methodology used on contract structural drawings. While there may be exceptions, it is advisable for lap splices to be avoided unless explicitly detailed and permitted by the Engineer of Record in the original tilt-up wall panel design.

ACI 318-19	Calculations	Description
STEP 3: WWR Detailer: Reinforcement Conversion Routine for WP-1 (continued)		
		A. **EDGE JAMB MATS** - CONTINUOUS VERTICALS. HORIZONTALS SPACED PER DESIGNER REQUIREMENTS DIRECTLY ADJACENT TO OPENING. HORIZONTALS SPACED AS "HOLDING WIRES" OUTSIDE OF THE EXTENTS OF THE OPENING HEIGHT. B. **CENTER JAMB MAT** - CONTINUOUS VERTICALS. HORIZONTALS SPACED PER DESIGNER REQUIREMENTS DIRECTLY ADJACENT TO OPENING. HORIZONTALS SPACED AS "HOLDING WIRES" OUTSIDE OF THE EXTENTS OF THE OPENING HEIGHT. C. **OVER-OPENING VERTICAL MATS** - CONTINUOUS VERTICALS BETWEEN TOP OF PANEL AND TOP OF OPENING. WIDELY-SPACED HORIZONTAL "HOLDING WIRES" THROUGHOUT. D. **BASE HORIZONTAL MAT** - CONTINUOUS HORIZONTALS, FULL PANEL WIDTH. VERTICALS SPACED PER DESIGNER REQUIREMENTS BELOW OPENINGS. VERTICALS OMITTED OUTSIDE OF THE EXTENTS OF THE WIDTHS OF THE OPENINGS. E. **OVER OPENING HORIZONTAL MATS** - CONTINUOUS HORIZONTALS, FULL PANEL WIDTH. WIDELY-SPACED VERTICAL "HOLDING WIRES".

<table>
<tr><th>ACI 318-19</th><th>Calculations</th><th>Description</th></tr>
<tr><td colspan="3">STEP 3: WWR Detailer: Reinforcement Conversion Routine for WP-1 (continued)</td></tr>
<tr><td></td><td>



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Jamb mats A and B are configured predominantly with single directional structural vertical reinforcement in, kept “in shape” by widely-spaced horizontal holding wires. The exception occurs in the regions directly adjacent to the panel openings, where the mats include horizontal wires as required to match the engineer’s specified horizontal reinforcement.

Mats A and B will include the vertical trim bars illustrated in Figure 3, located at the vertical edges of the wall panel and at the vertical edges of the openings.

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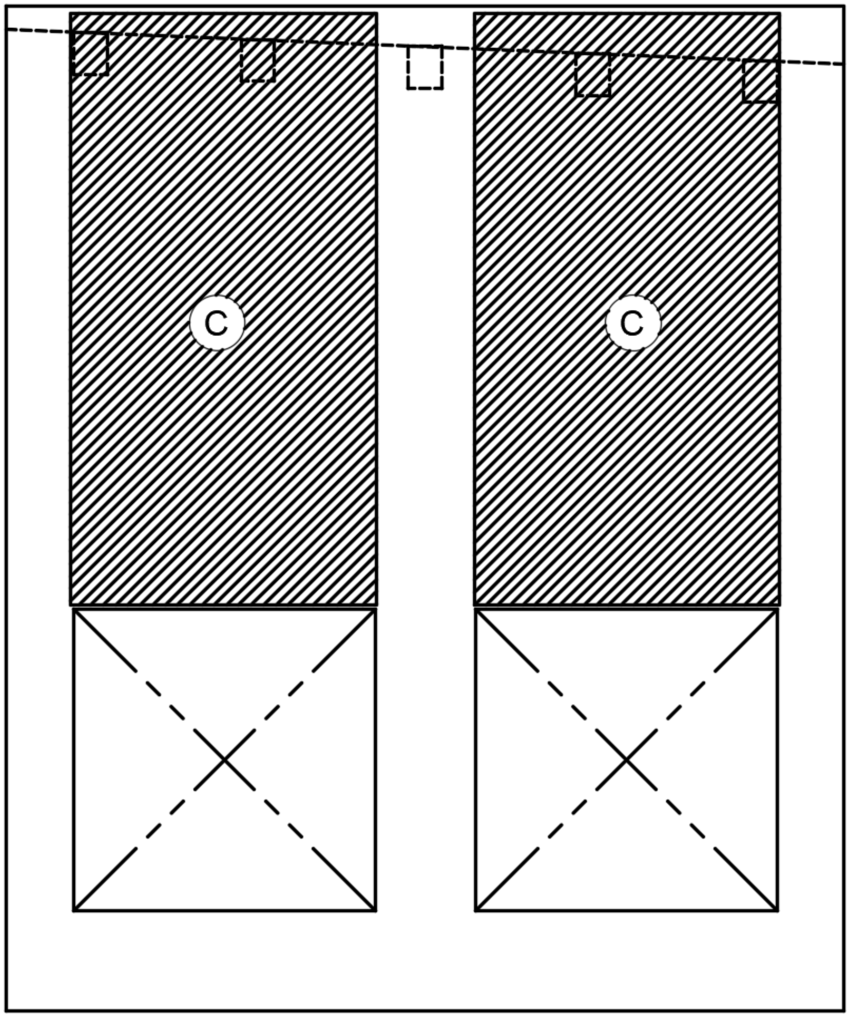
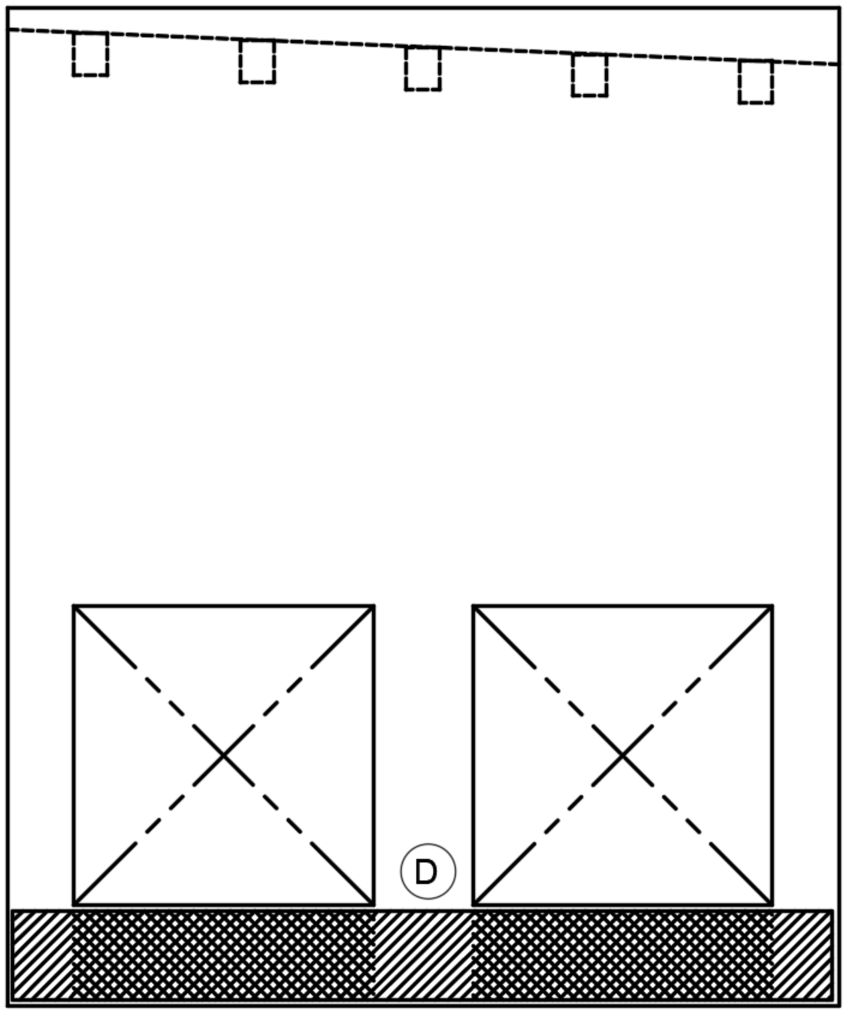
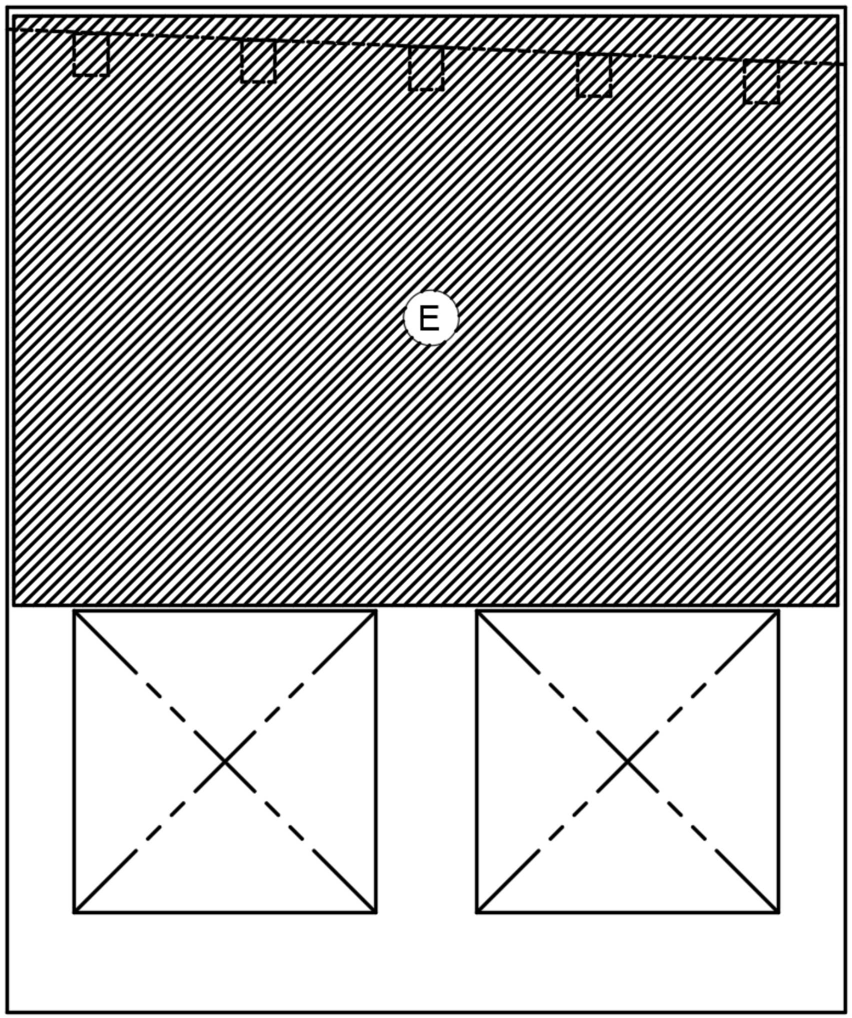
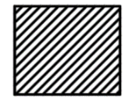
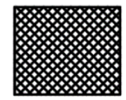
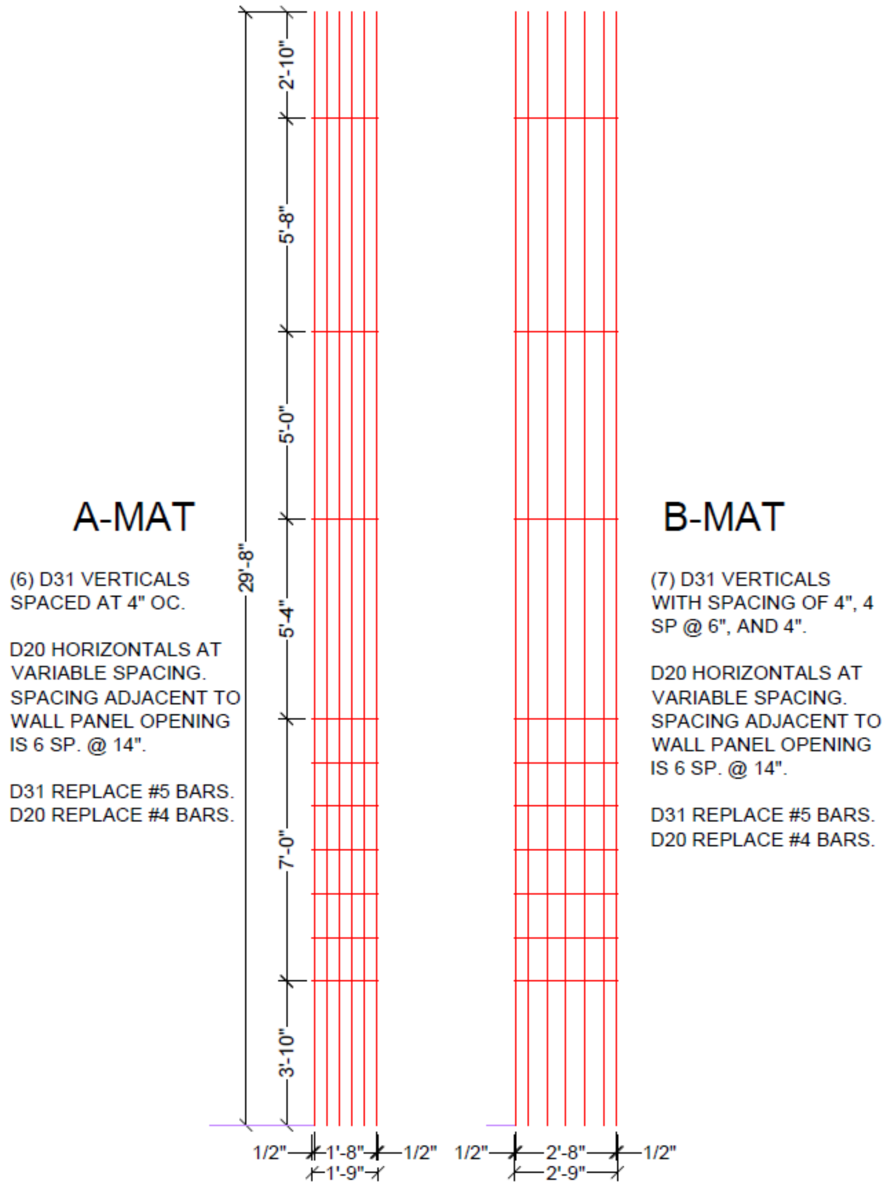
ACI 318-19	Calculations	Description
STEP 3: WWR Detailer: Reinforcement Conversion Routine for WP-1 (continued)		
		 Over-opening vertical mat C is configured with single directional structural vertical reinforcement kept "in shape" by widely-spaced horizontal holding wires. The vertical wires will be duly positioned to suit the alignment of the adjacent vertical trim bars along the vertical edges of the openings, which will be built into previously-described Mats A and B.

ACI 318-19	Calculations	Description
STEP 3: WWR Detailer: Reinforcement Conversion Routine for WP-1 (continued)		
	 	 Base horizontal Mat D has continuous horizontals running the full width of the wall panel. In the wall regions directly below the two openings, welded vertical wires match the engineer's specified requirements. In the wall regions that align with the previously defined A and B jamb regions, vertical wires are omitted from Mat D. Mat D will include the horizontal trim bars illustrated in Figure 3, located at the horizontal bottom edge of the wall panel and at the horizontal bottom edge of the openings.

ACI 318-19	Calculations	Description
STEP 3: WWR Detailer: Reinforcement Conversion Routine for WP-1 (continued)		
		 MAT REGION WITH SINGLE DIRECTION WIRES AND MINIMAL NON-STRUCTURAL HOLDING WIRES  MAT REGION WITH WIRES IN BOTH DIRECTIONS Over-opening horizontal mats E have continuous horizontals running the full width of the wall panel, with widely-spaced vertical holding wires throughout. E-Mats will include the horizontal trim bars illustrated in Figure 3, located at the horizontal top edge of the wall panel and at the top edge of the openings.

<table>
<tr><th>ACI 318-19</th><th>Calculations</th><th>Description</th></tr>
<tr><td colspan="3">STEP 4: WWR Detailer: WWR Detailing for WP-1</td></tr>
<tr><td></td><td>


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With the allocation process done, it is time to actually generate the physical WWR mat configurations to populate the allocated regions.
Mats A and B are shown here. Notice how both mat configurations contain “design” horizontal wire in the regions directly adjacent to the openings, while the remainder of the lengths of the mats contain only widely-spaced non-structural holding wires to maintain shape. Also note that the WWR detailer has kept the lower extent of the A- and B-Mat free of horizontals, which will help avoid potential wire collision at its interface with the D-Mat. Of equal importance is the exclusion of horizontals in the upper extent of the mat, affording the placers some flexibility to slightly shift the verticals for the purpose of avoidance of embedded anchorage items at the roof framing level.
Both A-Mat and B-Mat contain vertical trim bars. A-Mat was scheduled in Figure 6 to have (4) #5. With two (2) trim bars “built in”, it has a total of six (6) verticals. For the B-Mat, scheduled bars (5) plus trim bars (2) bring the total verticals to seven (7). Half-inch dimensions noted are nominal overhangs required for manufacture.
A-MAT:
4 X V2 D31/D20 20” (+0.5”,+0.5”) X 29’-8” (34, 34)
V2= 34” OH, 68”, 60”, 64”, 6 @ 14”, 46” OH
B-MAT:
V1 X V2 D31/D20 32” (+0.5”,+0.5”) X 29’-8” (34, 34)
V1= 0.5” OH, 4”, 4 @ 6”, 4”, 0.5” OH
V2= 34” OH, 68”, 60”, 64”, 6 @ 14”, 46” OH
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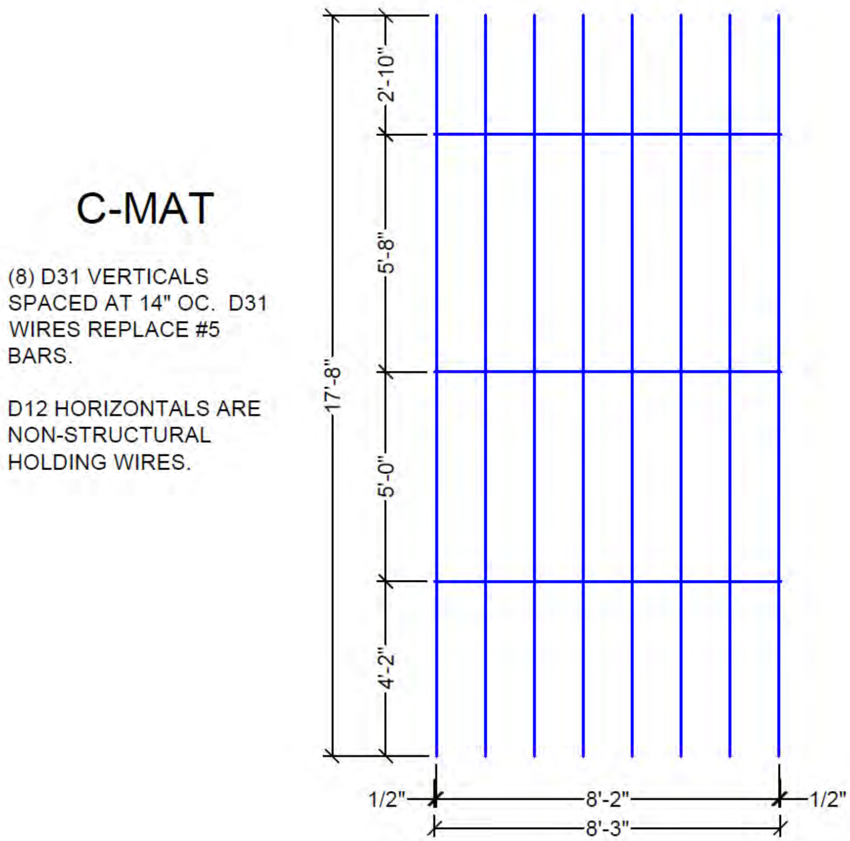
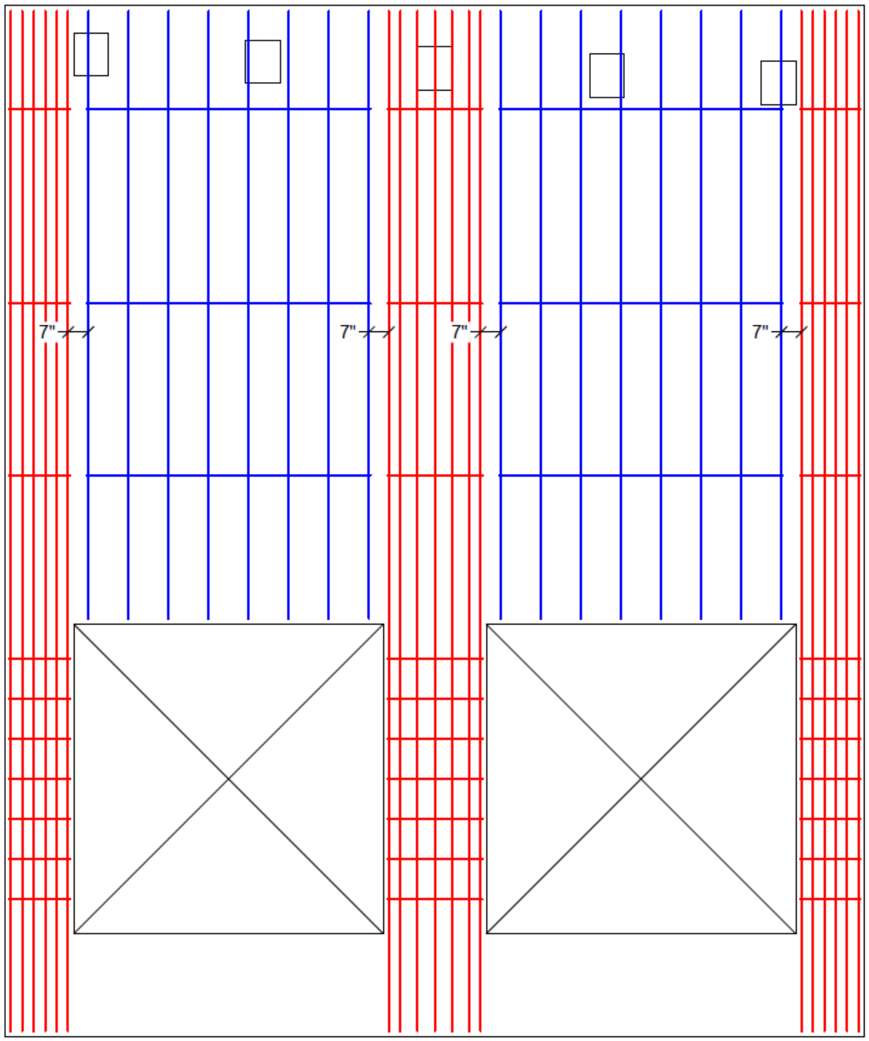
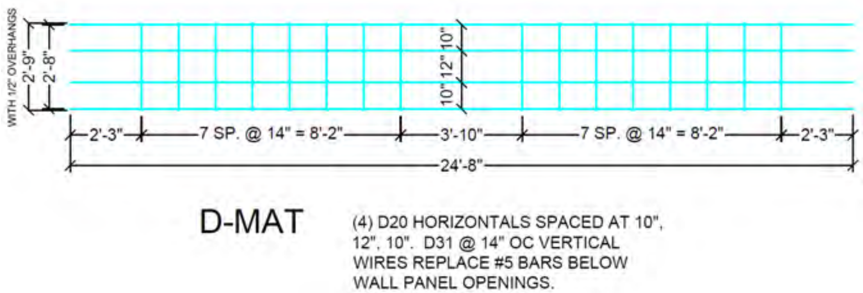
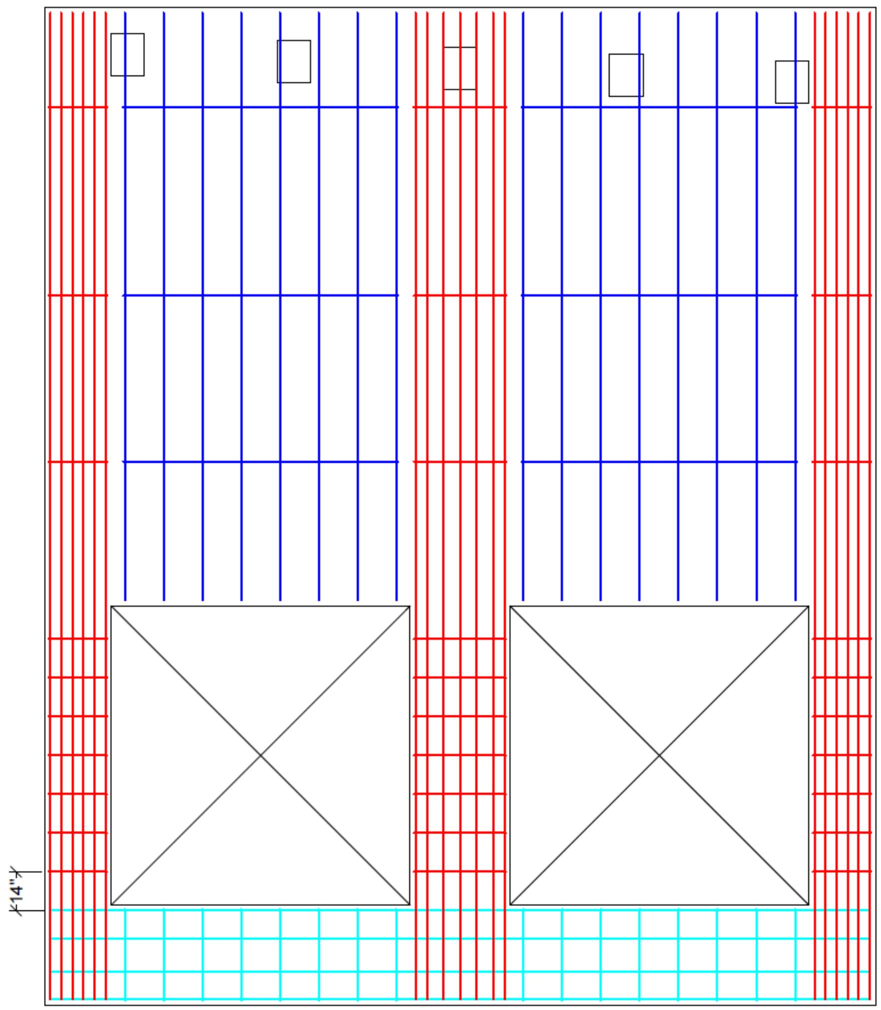
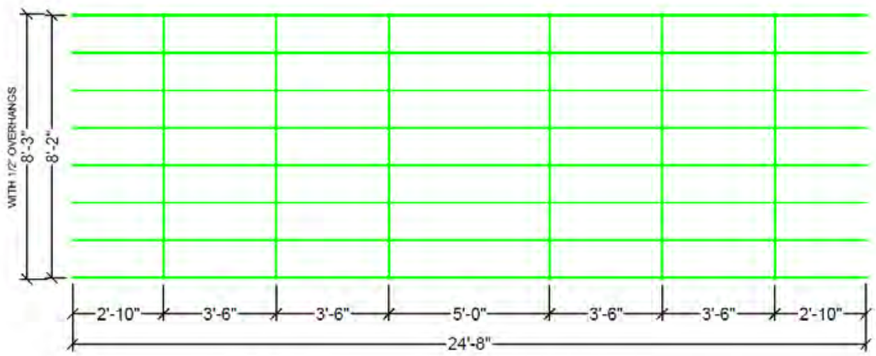
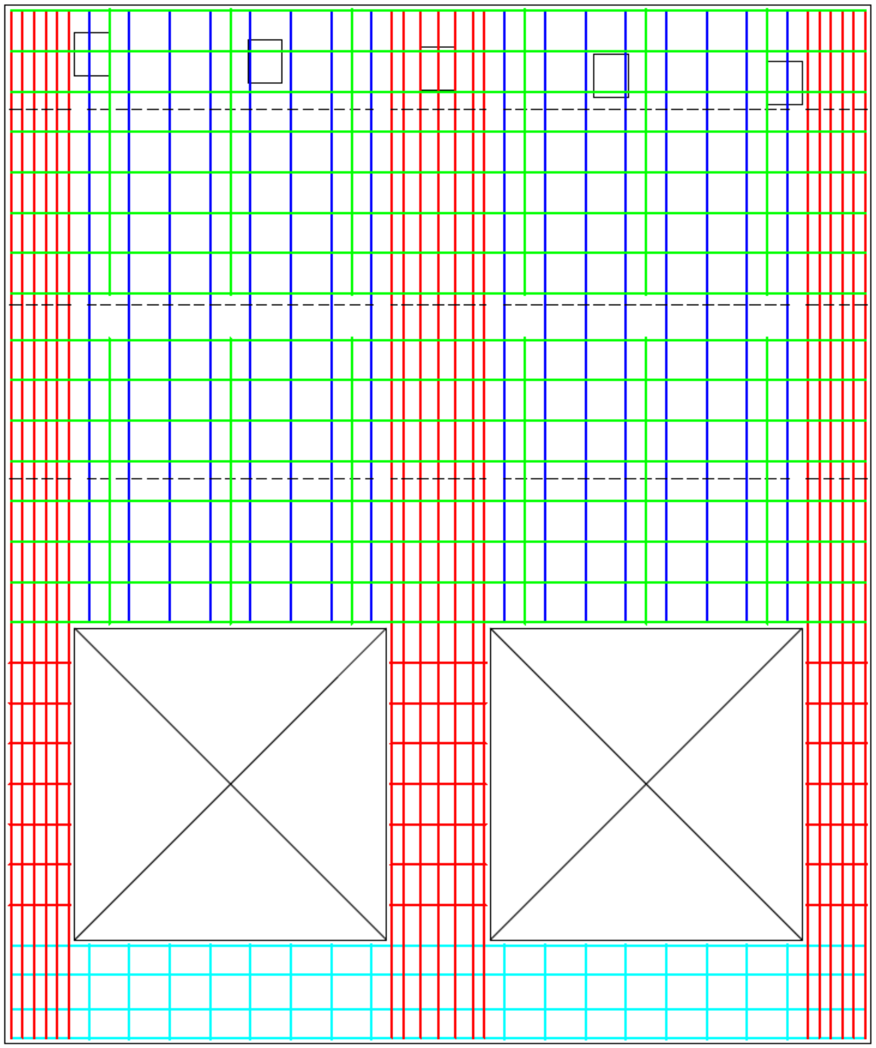
ACI 318-19	Calculations	Description
STEP 4: WWR Detailer: WWR Detailing for WP-1 (continued)		
		For the C-Mats, vertical reinforcement is originally defined for this region to be #5 @ 14" on center (each face of wall panel). $\#5 \, @ \, 14" \, OC \; = \; 0.266 \, \frac{in^2}{ft}$ $0.266 \, \frac{in^2}{ft} \times 9 \, feet \; width = 2.394 \, in^2$ $Use \; (8) \; D31 \; wires \; (A_s = 2.48 \, in^2)$ Eight (8) vertical wires are spaced at 14" on center. Since the WWR detailer is not capturing any of the horizontal structural reinforcement on this particular mat configuration, the only horizontals used will be non-structural holding wires, and their size will be reduced to suit the manufacturer's requirements for maintaining shape during handling, transport, and placement. Finally, note that the overall mat width is still less than the maximum that would require oversize load transport, resulting in a monetary savings. **C-MAT:** 14 X V2 D31/D12 98" (+0.5",+0.5") X 17'-8" (34, 50) V2 = 34" OH, 68", 60", 50" OH

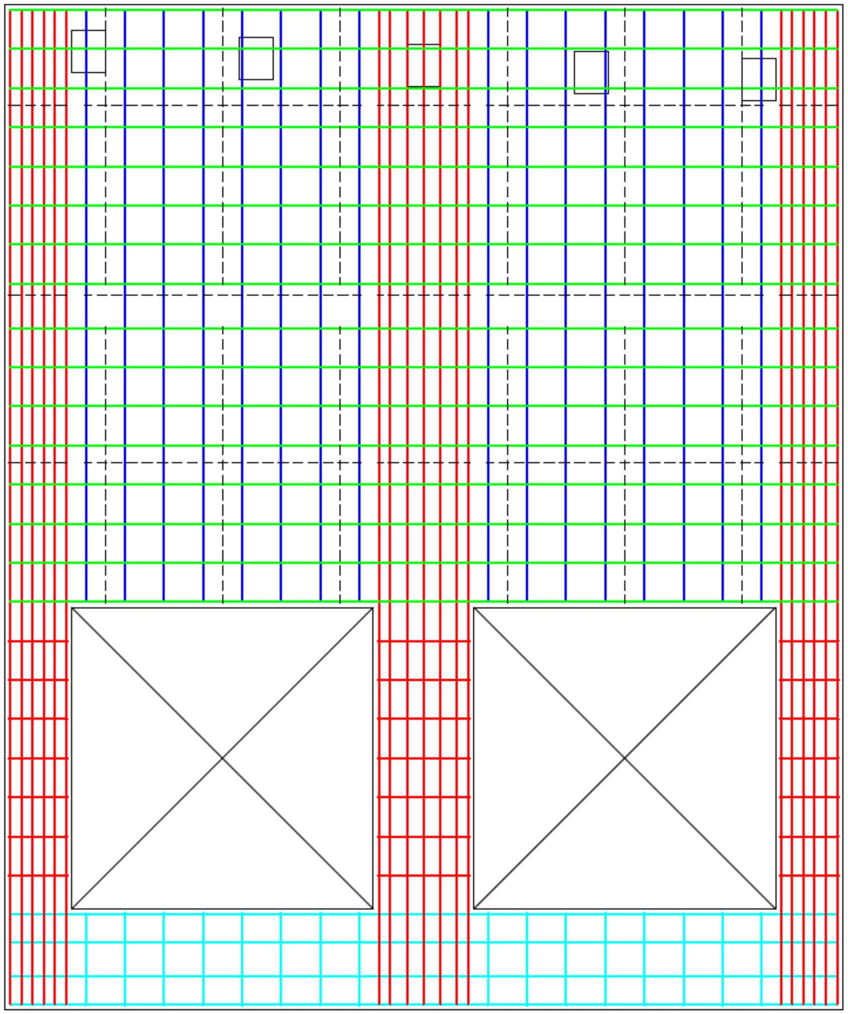
ACI 318-19	Calculations	Description
STEP 4: WWR Detailer: WWR Detailing for WP-1 (continued)		
	 	So far, Mats A, B, and C have been configured and positioned. The figure to the left shows this arrangement. Note the placement of the C-Mats (blue) symmetrically above the wall panel openings, resulting in a 7-inch offset from the A- and B- jamb mats (red). Also noteworthy is the alignment of the non-structural holding wires; the C-Mats were configured such that holding wire position matched that which was previously established for the A-mat and B-mat. Roof structure embedment plates are represented by the five (5) rectangles positioned at varying elevations towards the top of the wall panel. Verticals are capable of slight shifting if necessary to avoid conflict with the embedded headed studs associated with the plates. Non-structural holding wires are not intended to contribute in any way to the structural performance of the reinforced concrete element. As such, once the WWR mats are being positioned in the forms and chaired/bolstered, holding wires can themselves be field cut to allow for added localized adjustment of the structural wire alignment if conflicts with embedded items remain. The permission to field cut non-structural holding wires would be explicitly defined and illustrated by the WWR Detailer on the reinforcement submittal and placement drawings to ensure no confusion.

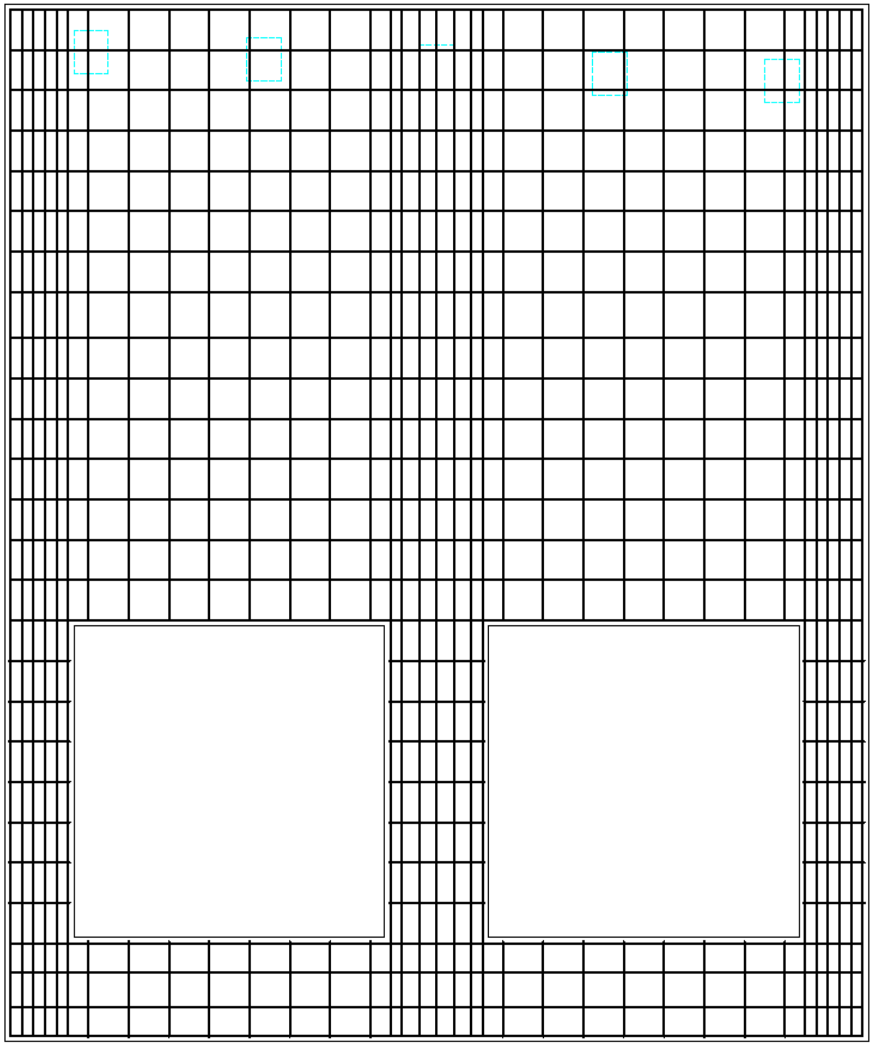
ACI 318-19	Calculations	Description
STEP 4: WWR Detailer: WWR Detailing for WP-1 (continued)		
		For the D-Mat, vertical reinforcement is originally defined for this region to be #5 @ 14” on center and horizontals defined to be #4 @ 14” on center. The pattern of vertical wires to replace the vertical bars will match that used in the C-Mats above the openings. The horizontal wires are determined as follows (each face of wall panel): $\#4 @ 14"\ OC = 0.171\ \frac{in^2}{ft}$ $0.171\ \frac{in^2}{ft} \times 3\ feet\ height = 0.513\ in^2$ $Three\ (3)\ D20 = 0.60\ in^2 \ldots$ $But\ spacing\ would\ be\ \frac{36 - 2" - 2"}{2\ spaces} = 16"$ $Use\ (4)\ D20.\ Resulting\ spacing\ is\ 10", 12", 10".$ Note that panel trim bars are “built in” to the WWR mat configuration. V1 X V2 D20/D31 32” (+0.5”,+0.5”) X 24’-8” (27,27) V1 = 0.5” OH, 10”, 12”, 10”, 0.5” OH V2 = 27” OH, 7 @ 14”, 46”, 7 @ 14”, 27” OH

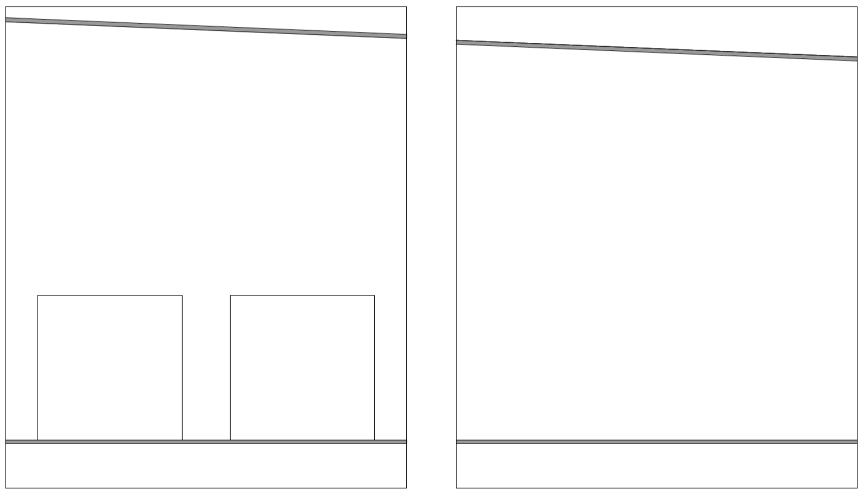
ACI 318-19	Calculations	Description
STEP 4: WWR Detailer: WWR Detailing for WP-1 (continued)		
		A composite of Mats A, B, C, and D is shown at left. While not explicitly noted during the previous configuration exercise for A-Mat and B-Mat, the positioning of the horizontal structural wires on these mats, adjacent to openings, was done in a manner that anticipated the presence of other WWR mats containing the horizontal trim wires at the bottom (shown) and top (subsequent, on E-Mat) of the wall panel openings. As can be seen, the placement of D-Mat is such that, not only are horizontal trim wires captured thereon, but the engineer's prescribed 14" spacing is maintained at the "transition" from D-Mat horizontals upward to A-Mat and B-Mat horizontals. This is where the use of WWR mats can prove to be far superior to placement of individual loose bars. The tilt-up wall panel site fabrication process itself is innately modular, and when coupled with WWR assemblies the labor and time expenditure, as well as the tolerance control, results in an extremely efficient and streamlined construction operation. One of the keys for both the design professional and contractor is the ever-increasing contributions of highly-skilled and knowledgeable WWR detailers throughout the welded wire reinforcement manufacturing industry. Without the WWR detailing professional, the implementation of WWR into structural projects would be a considerably more arduous undertaking.

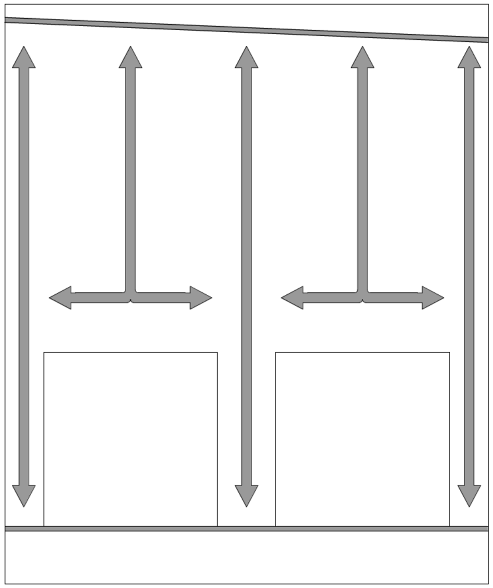
ACI 318-19	Calculations	Description
STEP 4: WWR Detailer: WWR Detailing for WP-1 (continued)		
	 E-MAT (8) D20 HORIZONTALS SPACED AT 14". D12 NON-STRUCTURAL VERTICAL HOLDING WIRES AT VARIABLE SPACING.	The E-Mat configuration is shown in the figure at the left. Two E-Mats will be used over the openings to complete the panel's primary structural reinforcement arrangement. E-Mat contains structural horizontal wires to replace horizontal #4 @ 14" on center, with non-structural vertical holding wires positioned to avoid conflict with the structural wires of other mats as well as embedded wall panel items. Notice that E-Mat contains trim reinforcement, as is readily apparent in subsequent illustrations.

ACI 318-19	Calculations	Description
STEP 4: WWR Detailer: WWR Detailing for WP-1 (continued)		
		The full WWR reinforcement assembly is shown at left, with non-structural holding wires of A-, B-, and C-Mats represented by hidden (dashed) lines.

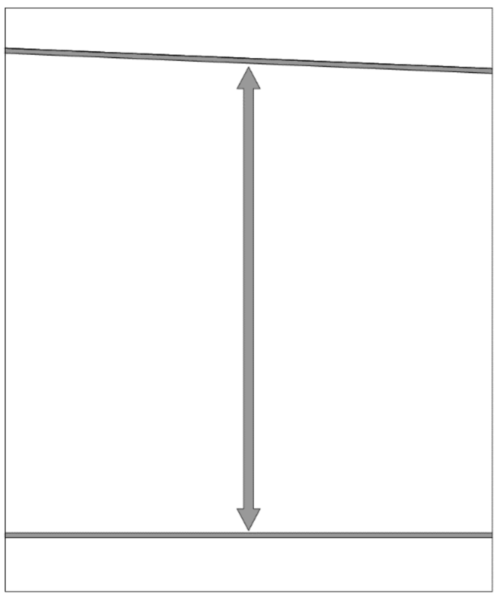
ACI 318-19	Calculations	Description
STEP 4: WWR Detailer: WWR Detailing for WP-1 (continued)		
		The full WWR reinforcement assembly is shown at left, with all non-structural holding wires represented by hidden (dashed) lines. Layout of non-structural holding wires is a critical component of the WWR mat detailing process, as it is the WWR detailer's responsibility to provide "workable" sheets of reinforcement for the crew in the field without compromise to the engineer's structural design intent nor disruption of intended geometric tolerances. In this regard, not only is the WWR detailing professional required to have a good technical grasp of the structural requirements themselves, but they must also have intimate familiarity with manufacturing capabilities and an artistic yet concise methodology of presenting the detailed information to project stakeholders. Note that within the thickness of the wall, holding wires physically occupy the same plane as structural wires, hence all of the attention paid to their arrangement. The result is that positioning of structural vertical and horizontal reinforcement shown in Figure 6 is maintained. Structural vertical wires are positioned outermost in the panel (nearest the surface), and the horizontal holding wires that are welded to them will occupy the same plane as the structural horizontal wires from the other mats placed in direct contact with said outermost mats. This then leaves the non-structural vertical wires associated with the structural horizontal mats. These non-structural vertical wires simply align to the inside of the structural horizontals and occur in a plane where no other reinforcement is positioned.

ACI 318-19	Calculations	Description
STEP 4: WWR Detailer: WWR Detailing for WP-1 (continued)		
		The full WP-1 WWR reinforcement assembly is shown at left, with all non-structural holding wires removed from view, and color-coding turned off. The configuration shown would be placed in each face of the wall panel, and the contractor would be responsible for adding the individual diagonal corner bars noted previously in Figure 3. In the end, for WP-1, the comparison is as follows (excluding aforementioned diagonal corner bars): **Reinforcing Bar Solution**: • Placement and tying of **184 loose reinforcing bars** (51 verticals and 41 horizontals, each face) **Welded Wire Reinforcement Solution:** • Placement of **16 prefabricated WWR mats** For clarification and transparency, the WWR shop/placement drawings for a tilt-up wall panel project will typically include submittal documentation that quantitatively supports the conversion of reinforcing bars over to welded wire reinforcement mats. For an example of this support documentation, refer to the *Shallow Foundations* example chapter of this guide.

ACI 318-19	Calculations	Description
STEP 5: WWR Detailer: Reinforcement Conversion Routine for WP-2		
	 *Heavy lines represent external “horizontal” planes (roof and slab) providing restraint to the wall panels.*	The following figures illustrate the WWR Detailer’s allocation of panel reinforcement areas into generalized WWR mat regions. Both WP-1 and WP-2 derive the entirety of their structural restraint from attachment to horizontally-oriented external “planes” (roof diaphragm level and slab level). As such, an individual wall panel, buffered on each of its vertical edges by a flexible control joints that creates physical separation (structural discontinuity) between itself and abutting wall panels, can only reliably span vertically to external supports; it has no quantifiable ability to span horizontally because no such external supports exist along the vertical panel edges. In this regard, the wall panels in this example are truly one-way spanning elements: resolution of the effects of out-of-plane flexural loading is achieved entirely through spanning in the vertical direction.

<table>
<tr><th>ACI 318-19</th><th>Calculations</th><th>Description</th></tr>
<tr><td colspan="3">STEP 5: WWR Detailer: Reinforcement Conversion Routine for WP-2 (continued)</td></tr>
<tr><td></td><td>


WP-1 relies on vertically continuous portions of the wall to deliver effects of out-of-plane flexural loading to the roof and floor diaphragm. Spandrel portions of the wall rely on horizontal distribution of out-of-plane flexure to the continuous portions of wall, especially in closer proximity to the tops of the wall panel openings.



WP-1 is entirely vertically-spanning. There is no expectation for the wall panel to have any internal horizontal redistribution of out-of-plane flexural effects.
</td><td>
Because of the presence of openings within the field of Wall Panel WP-1, there is invariably some *internal* resolution of out-of-plane flexure by way of “spandrel” portions of the wall (above/below openings) spanning horizontally between the full height vertical jambs/piers. While the jambs/piers do receive this force by way of horizontal redistribution, in the end they still depend entirely on their own vertical spanning ability to seek out restraint from external horizontal planes.

WP-2, on the other hand, has no internal discontinuities that create the need for horizontal redistribution of out-of-plane flexural effects; the internal flexural load path, then, is entirely vertically-spanning.

The offshoot of this WP-2 behavior is that the horizontal reinforcement is essentially relegated to shrinkage and temperature effects (and in extreme instances, in-plane shear resistance). This shift in responsibility of the horizontal reinforcement may allow for the WWR mats themselves to be spliced in the horizontal direction. With that said, lap splices other than those explicitly illustrated and/or defined by the design professional or record would of course require the designer’s approval, so for this example a solution comprised of mats with lap spliced horizontal wires is not illustrated.
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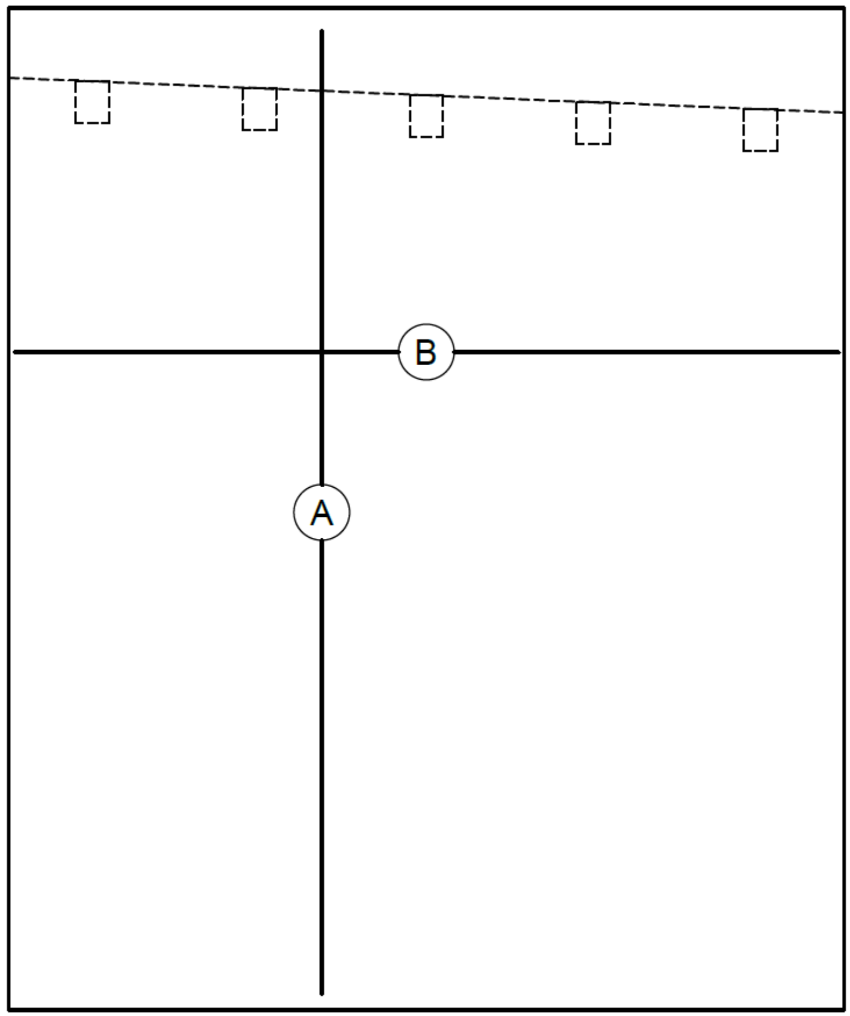
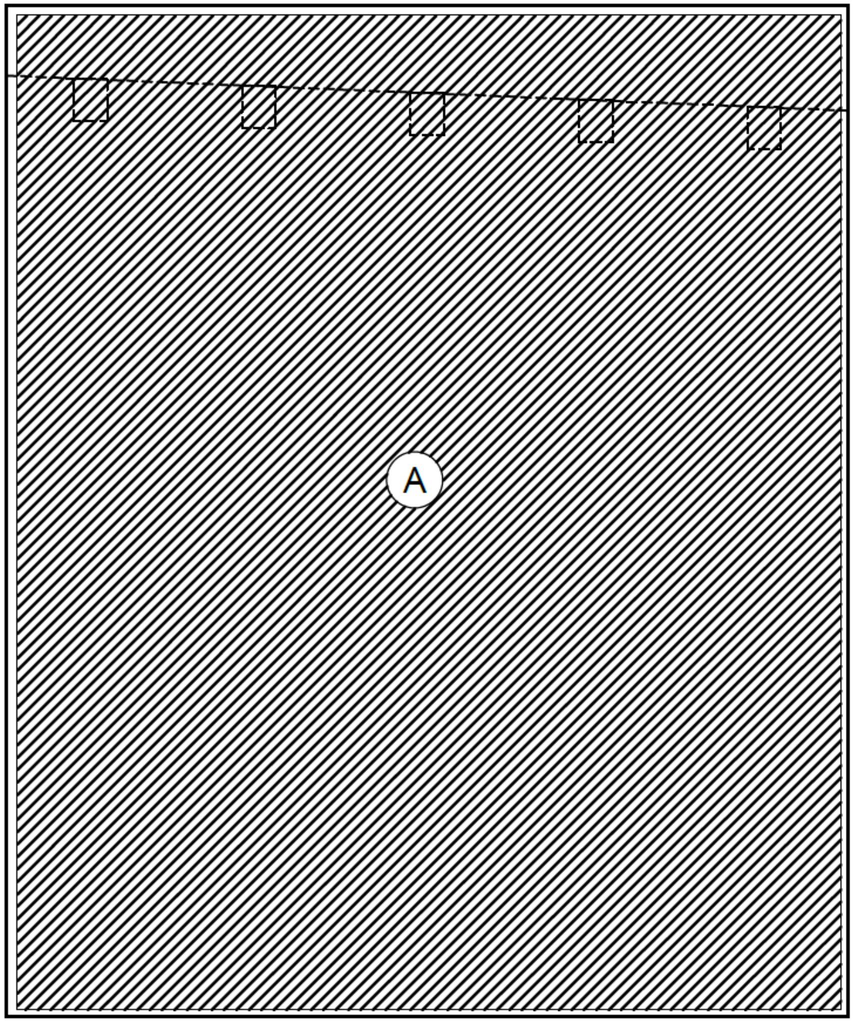
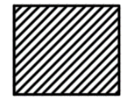
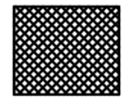
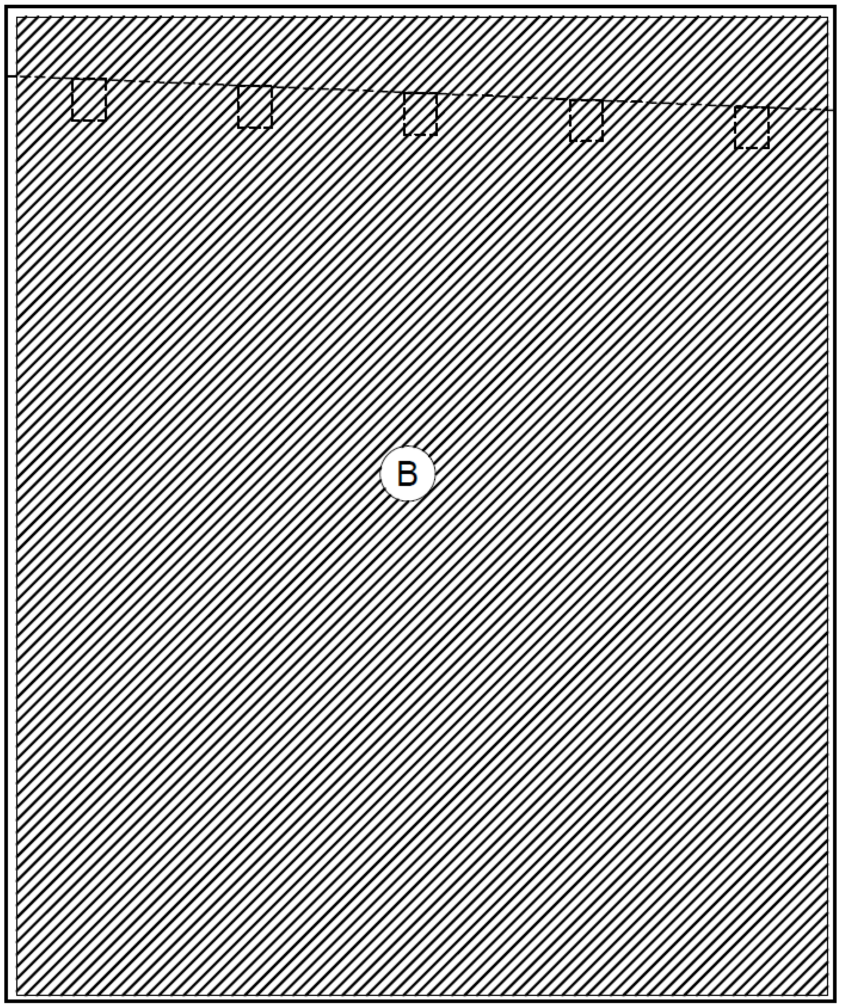
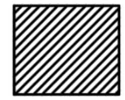
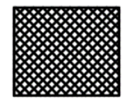
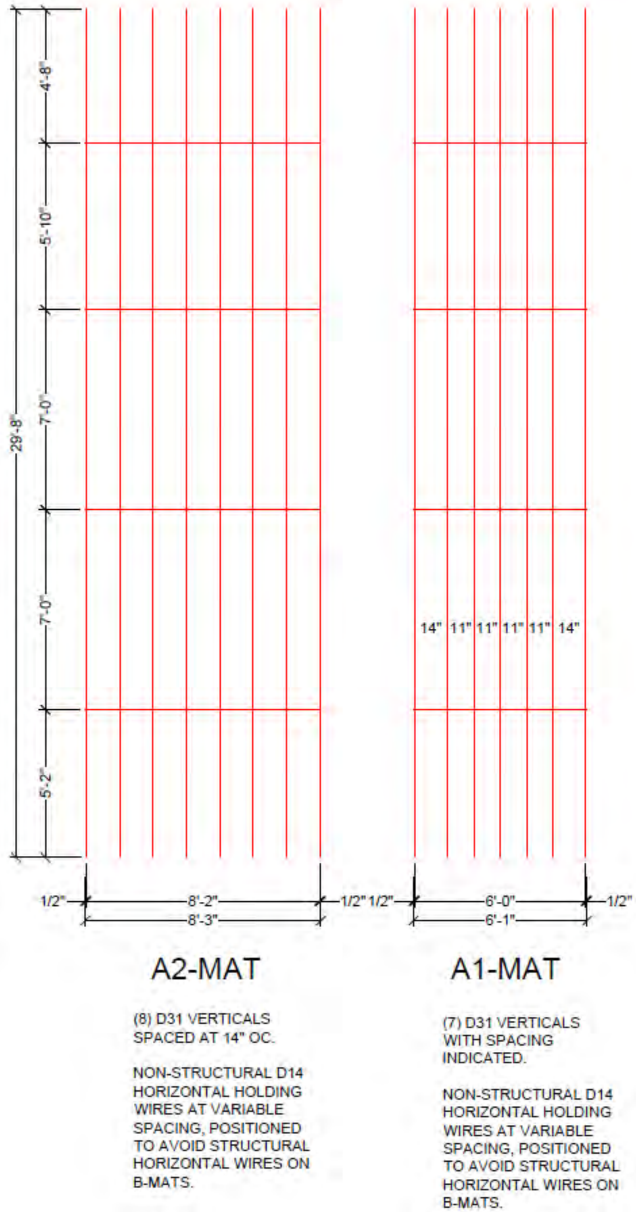
<tr><th>ACI 318-19</th><th>Calculations</th><th>Description</th></tr>
<tr><td colspan="3">STEP 5: WWR Detailer: Reinforcement Conversion Routine for WP-2 (continued)</td></tr>
<tr><td></td><td>
</td><td>A. VERTICAL MATS - CONTINUOUS VERTICALS. WIDELY-SPACED, NON STRUCTURAL HORIZONTAL "HOLDING WIRES".

B. HORIZONTAL MATS - CONTINUOUS HORIZONTALS. WIDELY-SPACED, NON STRUCTURAL VERTICAL "HOLDING WIRES".</td></tr>
</table>

ACI 318-19	Calculations	Description
STEP 5: WWR Detailer: Reinforcement Conversion Routine for WP-2 (continued)		
		 MAT REGION WITH SINGLE DIRECTION WIRES AND MINIMAL NON-STRUCTURAL HOLDING WIRES  MAT REGION WITH WIRES IN BOTH DIRECTIONS A-Mats are configured as single directional structural vertical reinforcement kept “in shape” by widely-spaced horizontal holding wires. Panel vertical trim bars will be configured as part of the mats.

ACI 318-19	Calculations	Description
STEP 5: WWR Detailer: Reinforcement Conversion Routine for WP-2 (continued)		
		 MAT REGION WITH SINGLE DIRECTION WIRES AND MINIMAL NON-STRUCTURAL HOLDING WIRES  MAT REGION WITH WIRES IN BOTH DIRECTIONS B-Mats are configured as single directional structural horizontal reinforcement kept “in shape” by widely-spaced vertical holding wires. Panel horizontal trim bars will be configured as part of the mats.

<table>
<tr><th>ACI 318-19</th><th>Calculations</th><th>Description</th></tr>
<tr><td colspan="3">STEP 6: WWR Detailer: WWR Detailing for WP-2</td></tr>
<tr><td></td><td>


</td><td>

For the A-Mats, vertical reinforcement is originally defined to be #5 @ 14” on center (each face of wall panel).

$$\#5\ @\ 14"\ OC\ =\ 0.266\ \frac{in^2}{ft}$$

$$0.266\ \frac{in^2}{ft} \times 25\ feet\ width = 6.65\ in^2$$

$$(22)\ D31\ wires\ (A_s = 6.82\ in^2), required, min.$$

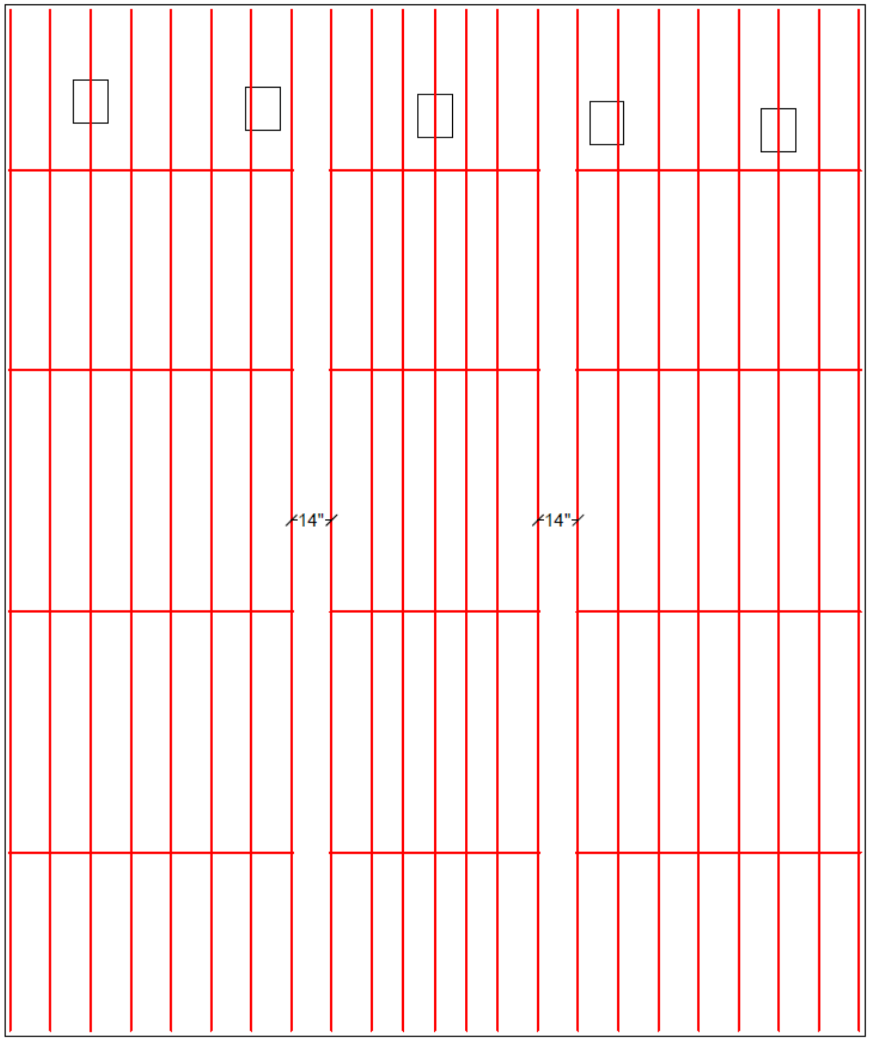
In configuring the mats, based on the overall wall panel width, it is clear that three mats will be necessary in order to avoid monetary ramifications of oversize transport widths. The solution comes in the form of a three-mat arrangement comprised of one (1) A1 mat with seven D31 wires and two (2) A2 mats, each with eight D31 wires. This results in a total of (23) D31 wires.

Horizontal wires are non-structural, sized as D14 to maintain shape of the WWR and spaced to avoid conflict with the structural horizontal wires on the B-mats.

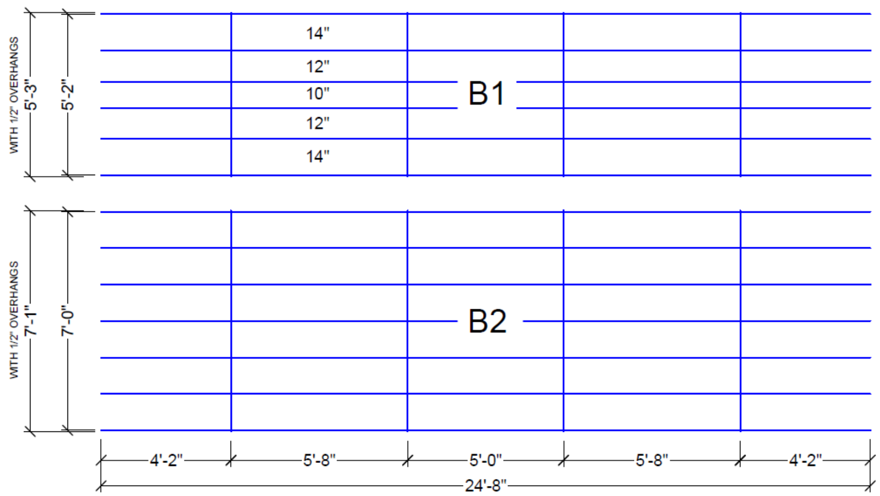
A1-MAT:
V1 X V2 D31/D14 72” (+0.5”,+0.5”) X 29’-8” (56, 62)
V1 = 0.5” OH, 14”, 4 @ 11”, 14”, 0.5” OH
V2 = 56” OH, 70”, 2 @ 84”, 62” OH

A2-MAT:
14 X V2 D31/D14 98” (+0.5”,+0.5”) X 29’-8” (56, 62)
V2 = 56” OH, 70”, 2 @ 84”, 62” OH

</td></tr>
</table>

ACI 318-19	Calculations	Description
STEP 6: WWR Detailer: WWR Detailing for WP-2 (continued)		
		The A-Mat layout within the wall panel is shown at left. A single A1-Mat is centered in the panel and flanked on each side by an A2-Mat. The symmetrical arrangement of vertical wires minimizes guesswork in the field as the workers place the mats in the forms. Non-structural holding wire offset from the top of the panel allows for localized flexibility in the event that slight shifting of the verticals is required to avoid spatial conflict with embedded elements.

<table>
<tr><th>ACI 318-19</th><th>Calculations</th><th>Description</th></tr>
<tr><td colspan="3">STEP 6: WWR Detailer: WWR Detailing for WP-2 (continued)</td></tr>
<tr><td></td><td>



B1-MAT

(6) D20 HORIZONTALS WITH SPACING INDICATED.

NON-STRUCTURAL D14 VERTICAL HOLDING WIRES AT VARIABLE SPACING, POSITIONED TO AVOID STRUCTURAL VERTICAL WIRES ON A-MATS.

B2-MAT

(7) D20 HORIZONTALS SPACED AT 14" OC.

NON-STRUCTURAL D14 VERTICAL HOLDING WIRES AT VARIABLE SPACING, POSITIONED TO AVOID STRUCTURAL VERTICAL WIRES ON A-MATS.

</td><td>

For the B-Mats, horizontal reinforcement is originally defined to be #4 @ 14” on center (each face of wall panel).

$$\#4\ @\ 14"\ OC\ =\ 0.172\ \frac{in^2}{ft}$$

$$0.172\ \frac{in^2}{ft} \times 30\ feet\ height = 5.15\ in^2$$

$$(26)\ D20\ wires\ (A_s = 5.20\ in^2), required, min.$$

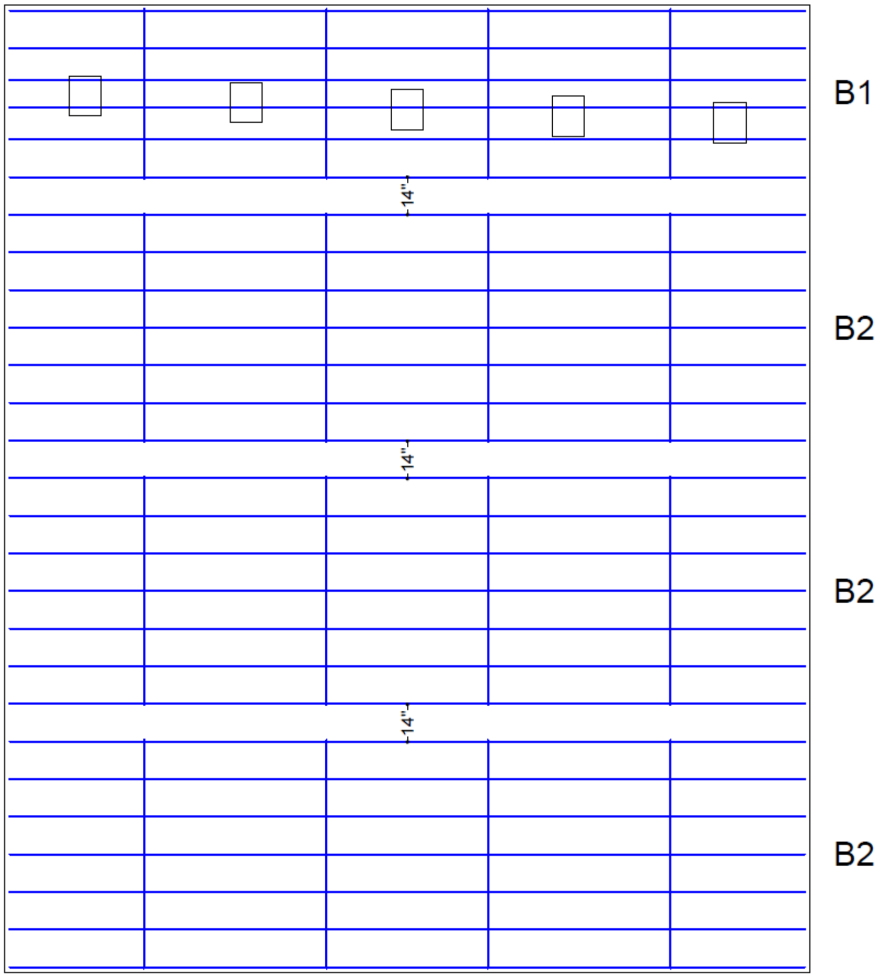
In configuring the mats, based on the overall wall panel width, it is clear that four mats will be necessary in order to avoid monetary ramifications of oversize transport widths. The solution comes in the form of a four-mat arrangement comprised of one (1) B1 mat with six D20 wires and three (3) B2 mats, each with seven D20 wires. This results in a total of (27) D20 wires.

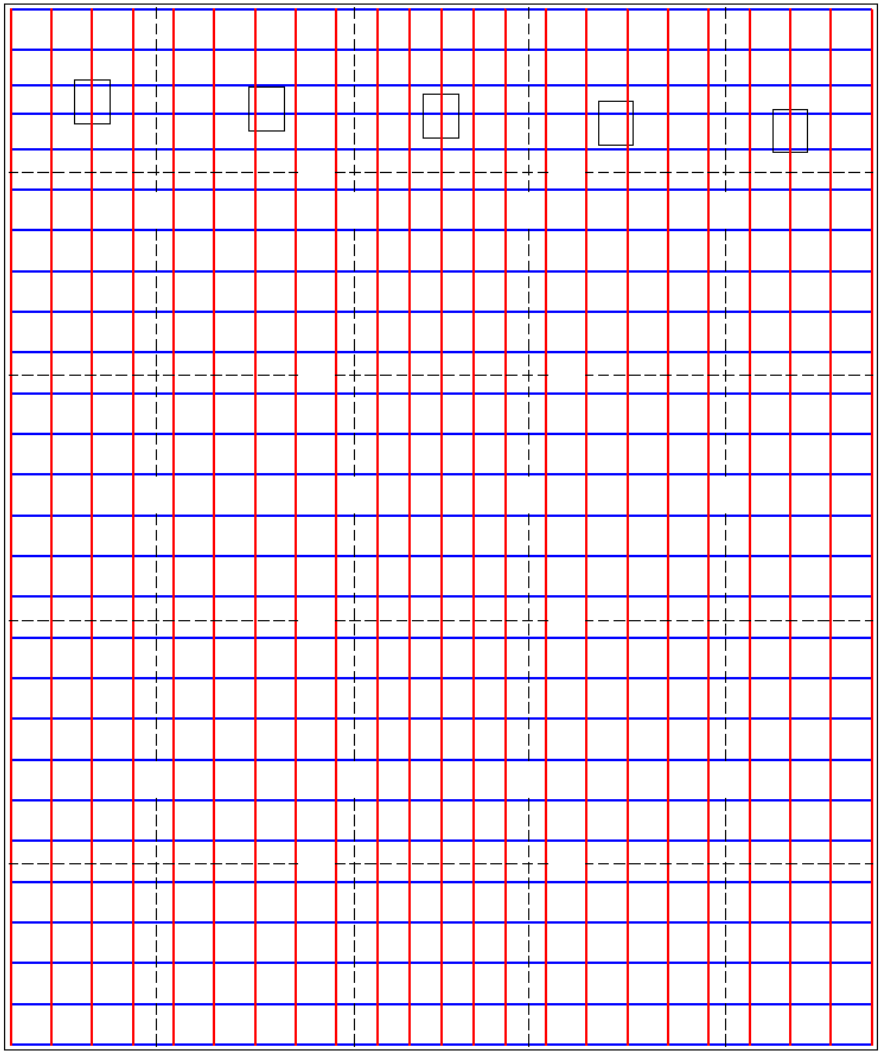
vertical wires are non-structural, sized as D14 to maintain shape of the WWR and spaced to avoid conflict with structural vertical wires on the A-mats.

B1-MAT:
V1 X V2 D20/D14 62” (+0.5”,+0.5”) X 24’-8” (50, 50)
V1 = 0.5” OH, 14”, 12”, 10”, 12”, 14”, 0.5” OH
V2 = 50” OH, 68”, 60”, 68”, 50” OH

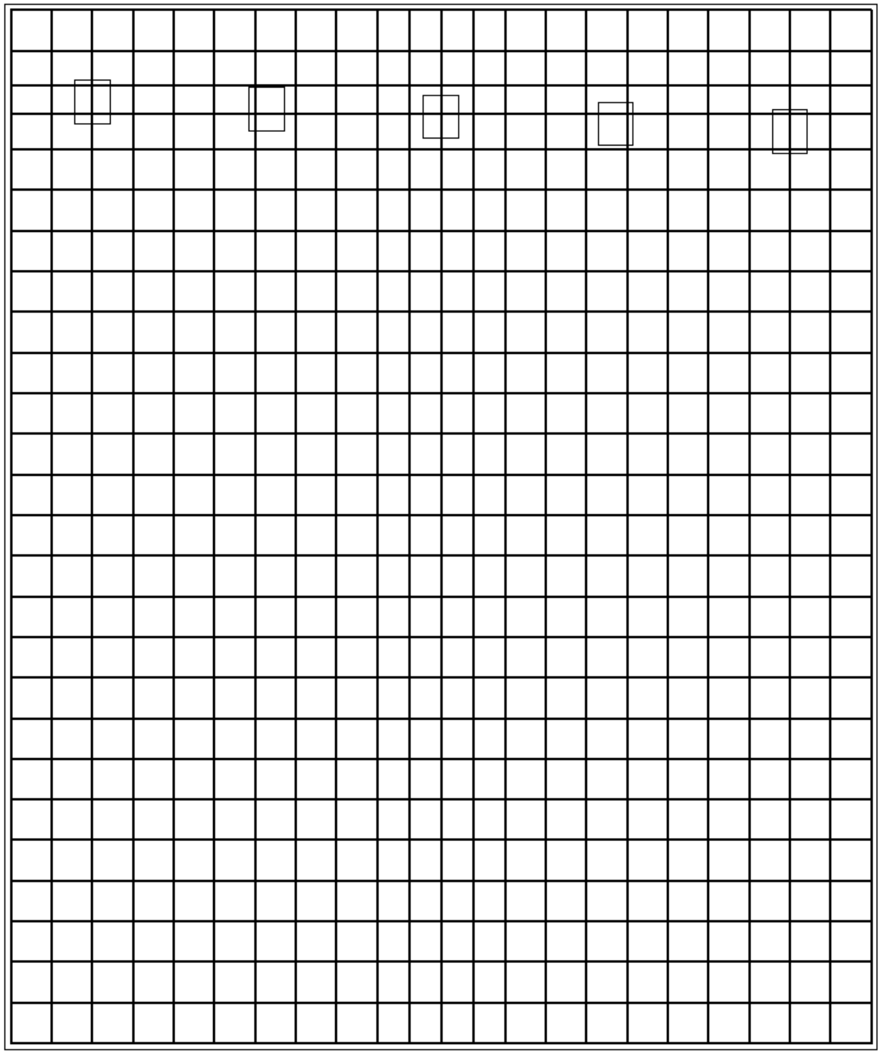
B2-MAT:
14 X V2 D20/D14 84” (+0.5”,+0.5”) X 24’-8” (56, 62)
V2 = 50” OH, 68”, 60”, 68”, 50” OH

</td></tr>
</table>

ACI 318-19	Calculations	Description
STEP 6: WWR Detailer: WWR Detailing for WP-2 (continued)		
		The B-Mat layout within the wall panel is shown at left. A single B1-Mat aligns at the top of the wall panel, with the balance of the panel covered by three (3) B2-Mats. The symmetrical arrangement of wires on each mat minimizes guesswork in the field as the workers place the mats in each face of the form. Non-structural holding wires offset from the embedded item positions allow for localized flexibility in the event that slight shifting of the horizontals is required to avoid spatial conflict with embedded elements.

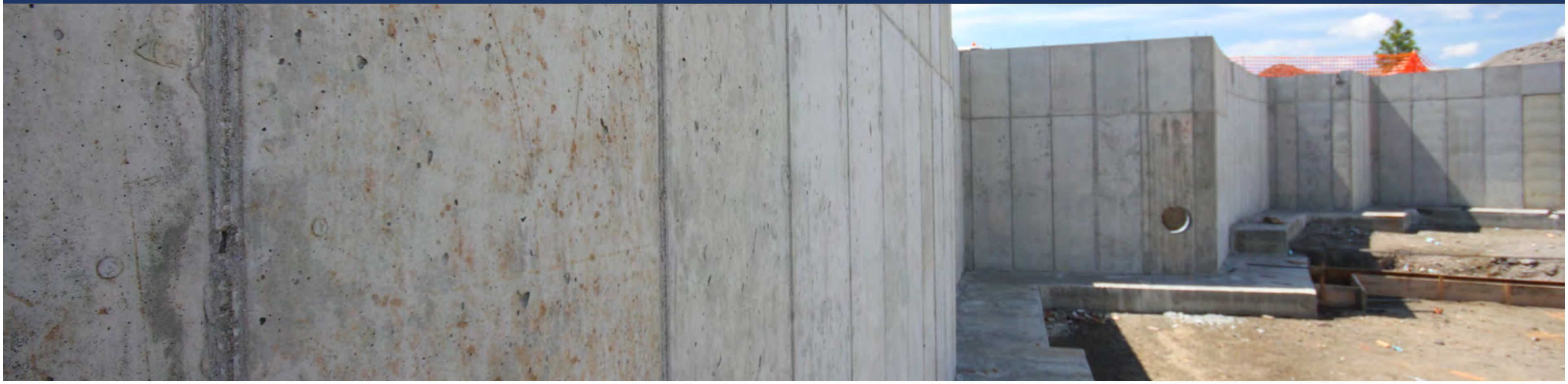
<table>
<tr><th>ACI 318-19</th><th>Calculations</th><th>Description</th></tr>
<tr><td colspan="3">STEP 6: WWR Detailer: WWR Detailing for WP-2 (continued)</td></tr>
<tr><td></td><td></td><td>The full WP-2 WWR reinforcement assembly is shown at left, with all non-structural holding wires represented by hidden (dashed) lines.

Layout of non-structural holding wires is a critical component of the WWR mat detailing process, as previously noted for WP-1.</td></tr>
</table>

ACI 318-19	Calculations	Description
STEP 6: WWR Detailer: WWR Detailing for WP-2 (continued)		
		The full WWR reinforcement assembly is shown at left, with all non-structural holding wires removed from view, and color-coding turned off. The configuration shown would be placed in each face of the wall panel. In the end, for WP-1, the comparison is as follows: **Reinforcing Bar Solution**: • Placement and tying of **100 loose reinforcing bars** (23 verticals and 27 horizontals, each face) **Welded Wire Reinforcement Solution:** • Placement of **14 prefabricated WWR mats** For clarification and transparency, the WWR shop/placement drawings for a tilt-up wall panel project will typically include submittal documentation that quantitatively supports the conversion of reinforcing bars over to welded wire reinforcement mats. For an example of this support documentation, refer to the *Shallow Foundations* example chapter of this guide.

Chapter Eight

EXAMPLE: Cantilever-Type Site Retaining Wall



EXAMPLE 8 PROBLEM STATEMENT:

The subject structure is a reinforced concrete site retaining wall that provides separation between an upper on-grade parking lot and a switch-back concrete ramp providing access to the ground level of a reinforced concrete parking structure. The cantilever-type retaining wall structure is kept separate from components of the parking structure by way of a weakened plane control joint, resulting in the wall's behavior being that of a purely fixed-base, free-top element. The wall is properly drained so as to eliminate the accumulation of hydrostatic pressure and is designed by the Engineer of Record to develop active soil pressure. Rotation of the top of the wall is deemed to be not detrimental to the structure considering the wall itself is not relied upon to support elevated building structure above.

The structural contract drawings provide retaining wall designs that utilize grade 60 deformed reinforcing bars, but also include permissive language for the use of welded wire reinforcement as a replacement to specified loose reinforcing bars (*Pre-Approved Equal Method*). The structure is designed by the Engineer of Record per the requirements of ACI 318-19.

For this example, both wall and footing reinforcement will be presented in the form of deformed welded wire reinforcement solutions.

Design criteria are as follows: $f'_c = 4{,}000\ psi, \beta_1 = 0.85\ (footing\ and\ wall)$

$f_y = 60{,}000\ psi\ (reinforcing\ bar\ yield\ strength)$

$Concrete\ Density = 0.145\ kcf, normalweight\ concrete\ \lambda = 1.0$

$Clear\ cover$: 2" *to flexural reinforcement, typ., 3" clear at footing bottom and sides*

$Soil\ Criteria = 3\ ksf\ net\ allowable\ bearing, 0.120\ kcf\ density, \varphi = 30°, k_a = 0.33, k_p = 3.0, base\ friction\ coefficient = 0.35$

$LL\ Surcharge = 50\ psf$

The example includes the following steps:

Step 1 – Retaining Wall and Footing: Summary of Engineer's Relevant Design and Details
Step 2 – WWR Detailing Procedure – Wall Type 1
Step 3 – Closing Notes – Other Wall Types

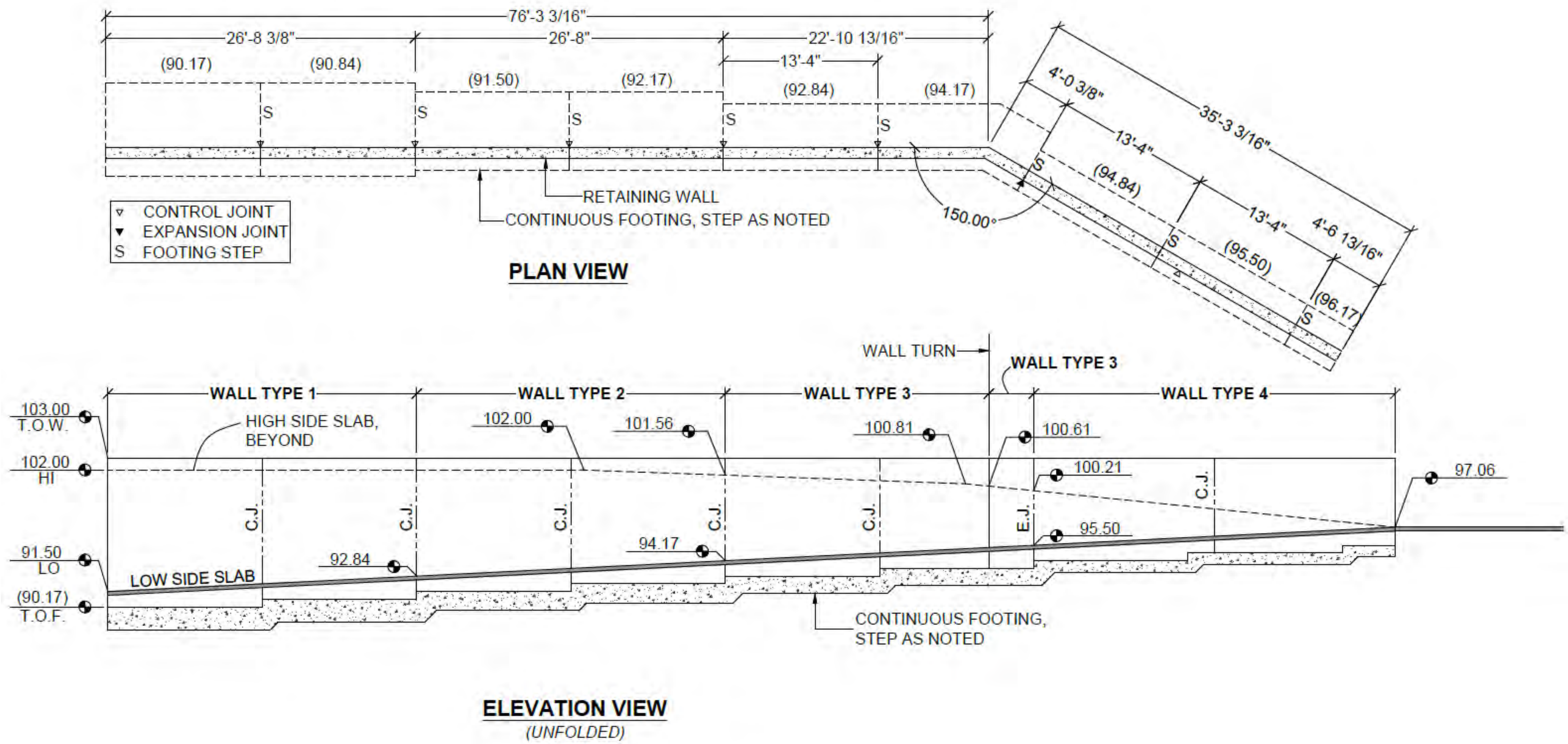


Figure 1: Excerpt from contract structural drawings: wall plan and elevation

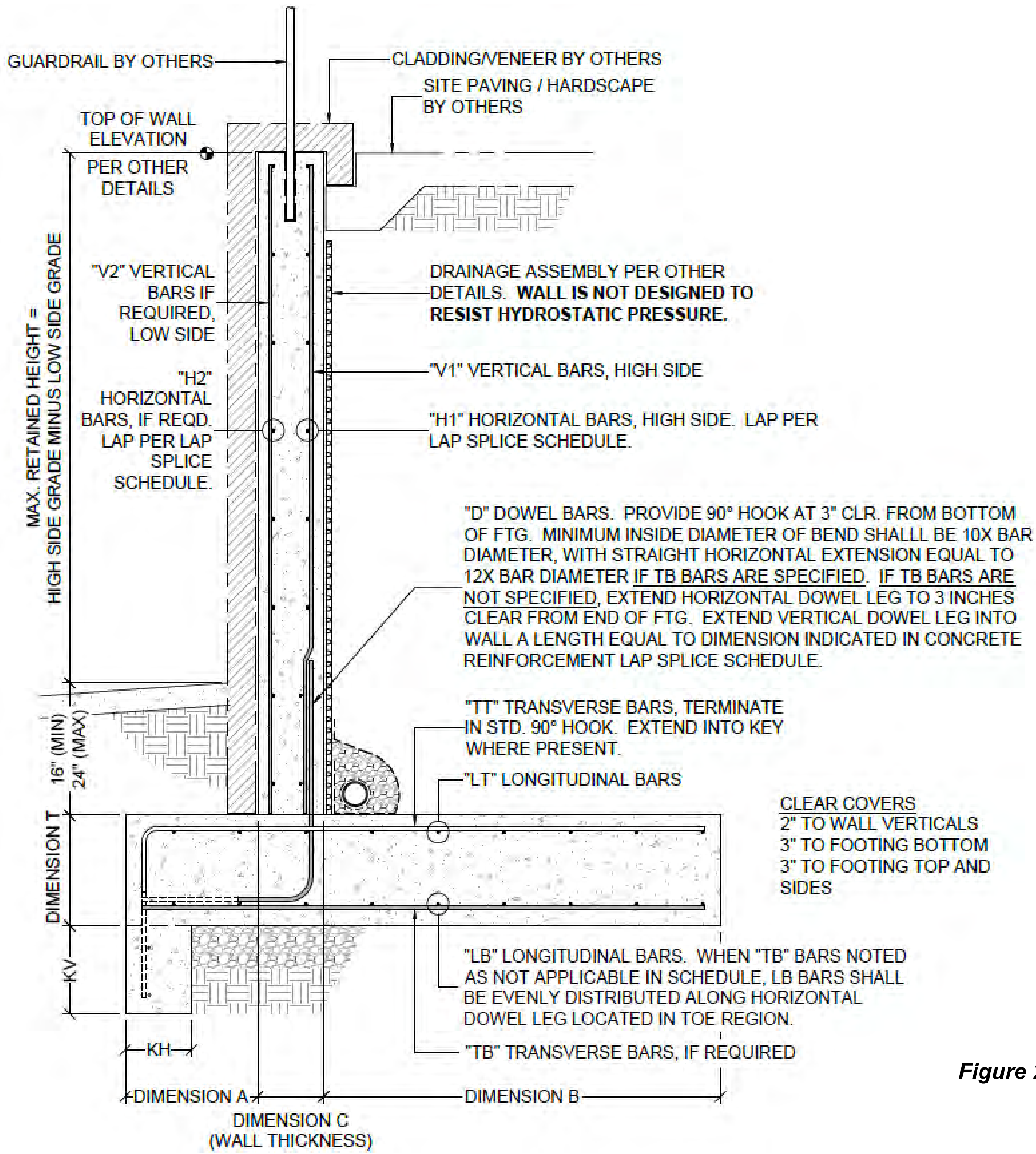


Figure 2: Excerpt from contract structural drawings: typical retaining wall section

ACI 318-19	Calculations	Description
STEP 1: Retaining Wall and Footing: Summary of Engineer's Relevant Design and Details		
11.1.4 13.3.6.1 13.3.6.3 7.6.1.1 22.5 13.2.6.2 13.2.7.1 13.2.7.2 7.4.3 13.3.2.1	**Stem** *Design of cantilever retaining walls shall be in accordance with Chapter 13.* *The stem of a cantilever retaining wall shall be designed as a one-way slab in accordance with the applicable provisions of Chapter 7.* *For walls of uniform thickness, the critical section for shear and flexure shall be at the interface between the stem and the footing.* *Minimum flexural steel threshold is based on that required for one-way slabs and not on requirements for beams due to its explicit definition as the former in Section 13.3.6.1.* *One-way shear strength is calculated per Section 22.5 via reference from Section 7.5.3.* **Footing** *Retaining wall footing is designed for both one-way shallow foundation requirements and beam previsions.* *Neglect size effect factor* *Calculate flexural demand at the face of the stem intersection.* *Calculate shear demand at a horizontal distance d from the face of the stem.* *The design and detailing of shall be in accordance with provisions of Chapter 7 and Chapter 9. Shear will be per Section 22.5, while the designer elects to invoke minimum flexural steel requirements for beams per Section 9.6.*	The Structural Engineer of Record has specified grade 60 deformed reinforcing bars, but the structural contract documents include permissive language for welded wire reinforcement substitution (Figure 3). The WWR detailer, prompted by the contractor's preference and based on permissive WWR language presented on the contract documents by the design professional , generates a WWR alternative contingent upon his/her familiarity with WWR manufacturing and transport capabilities and the reinforcement's compliance with the ACI 318-19 Standard. In effect, the burden is on the WWR manufacturer's detailer – on behalf of the contractor and associated subcontractors - to confirm acceptability of the substitution through verifying information in the submittal that the design professional's structural intent is not altered or compromised. While this confirmation could be as simple as documented acceptance through electronic mail correspondence, it is more often comprised of a formalized process by which the use of WWR as a substitution is confirmed first through Request for Information (RFI) correspondences, and then followed up by appropriate addendum to the contract drawings. Such an addendum is often in the form of the manufacturer's submittal information itself. Refer to Chapter 5 for an example of a manufacturer's submittal that would be an accompaniment to the actual illustrative welded wire reinforcement shop drawings submitted for review.

MILD REINFORCING STEEL

1. TYPICAL DEFORMED REINFORCING BARS (REBAR) SHALL CONFORM TO ASTM A615, GRADE 80. BARS SHALL BE LAPPED IN ACCORDANCE WITH THE REBAR LAP SCHEDULES UNLESS OTHERWISE EXPLICITLY DETAILED.
2. LONGITUDINAL REINFORCEMENT IN SPECIAL MOMENT FRAME BEAMS AND COLUMNS, AND VERTICAL AND HORIZONTAL REINFORCEMENT IN SPECIAL STRUCTURAL (SHEAR) WALLS SHALL BE ASTM A706 GRADE 60 OR GRADE 80 AS NOTED. TENSILE AND ELONGATION PROPERTIES SHALL BE CONFIRMED THROUGH MILL REPORT DOCUMENTATION PROVIDED AS PART OF THE PROJECT REINFORCEMENT SUBMITTAL.
3. WELDED DEFORMED WIRE REINFORCEMENT SHALL CONFORM TO ASTM A1064 GRADE 80 AND SHALL BE PROVIDED IN SHEET FORM. REINFORCEMENT SHEETS SHALL BE MANUFACTURED WITH OVERHANG LENGTHS SUFFICIENT TO ACHIEVE A LAP SPLICE LENGTH EQUAL TO THE GREATER OF 12 INCHES OR THE LAP SPLICE DIMENSION SHOWN IN THE REBAR LAP SCHEDULE FOR BAR OF EQUAL (OR GREATER) DIAMETER AND GRADE, UNLESS OTHERWISE NOTED. SHEETS AND ASSOCIATED LAP REGIONS SHALL BE INSTALLED COPLANAR SO AS TO NOT “STACK”.
4. WELDED DEFORMED WIRE REINFORCEMENT OF EQUAL AREA, EQUAL OR LESSER SPACING, AND IDENTICAL CURTAILMENT (HOOKS AND LAP SPLICES) IS PERMITTED AS A SUBSTITUTION FOR DEFORMED REINFORCING BARS, EXCEPT IN THE FOLLOWING STRUCTURAL APPLICATIONS:
   A. LONGITUDINAL STEEL IN SPECIAL MOMENT FRAMES
   B. VERTICAL AND HORIZONTAL STEEL IN SPECIAL STRUCTURAL WALLS
5. UNLESS OTHERWISE NOTED ON THE DRAWINGS OR IN THE PROJECT SPECIFICATIONS

Figure 3: Excerpt from contract structural drawings: WWR permissive language in General Notes section

ACI 318-19	Calculations	Description

STEP 1: Retaining Wall and Footing: Summary of Engineer’s Relevant Design and Details (continued)

Calculations

Design Parameters		
concrete unit weight	145	pcf
soil unit weight	120	pcf
internal friction angle, phi	30	degrees
sloped backfill?	N	(Y/N)
retained backfill angle, beta	0	degrees
friction coeff. at base, f	0.35	
Allowable Soil bearing	3000	psf
At-rest Coefficient, Ko for design	0.5	
Active Coefficient Ka for design	0.3333	
Passive Coefficient Kp for design	3	

4000	(footing f'c, psi)	
4000	(wall f'c, psi)	

Calculated Rankine Coefficients		
Ka=	0.333333	
Kp=	3	
variable	0.5	
friction coeff=	0.36397	(Bowles)
Uniform LL Surcharge on Backfill:	50	psf

Wall and Soil Geometry		
high side soil elevation	102	absolute elevation, at back face of wall
top of wall elevation	103	absolute elevation, typically same as high side soil elevation
low side soil elevation	91.5	absolute elevation, at front face of wall
top of footing	90.17	absolute elevation
bottom of footing	88.17	absolute elevation
footing thickness	2	feet
wall thickness	1	feet
toe dimension	1.5	feet, measured beyond front face of wall
heel dimension	5.5	feet, measured beyond back face of wall
overall footing dimension	8	feet
Stability Soil Wedge Height, H	13.83	feet, measured at back of heel
internal Force Wedge Height, H'	11.83	feet, measured at back of heel

Stem Height	12.83	feet

Wall and Soil Loading	Force	Arm	Mr	
wall	1860.35	2	3720.7	
rectangular backfill	7807.8	5.25	40990.95	
triangular backfill	0	6.166666667	0	
footing	2320	4	9280	
Pv contribution to Mr	0	8	0	
uniform surcharge	275	5.25	1443.75	
miscellaneous surcharge	0	0	0	**
miscellaneous veneer	0	0	0	**
∑W=	12263.15	∑Mr=	55435.4	
	pounds		lb-ft	

Location of Resultant Force, Xr=	2.95262	ft		
Eccentricity of Force, e=	1.04738	ft	d/6=	1.333333 ft
	ACCEPTABLE			

Stability Driving & Overturning				
At-rest design?	N	(Y/N)		
Pa=	3824.995	lbs		
Pv=	0			
Ph=	3824.995		arm=	4.61
Mo,backfill	17633.23	lb-ft		
P,LLsrchg=	230.477	lbs		
Mo,LLsurchg=	1593.748	lb-ft		
P,misc srchg=	0	lbs**		
Mo, misc srchg=	0	lb-ft**		
∑Mo=	19226.98	lb-ft		

Description

Design results for Wall Type 1 are presented here.

- Design parameters
- Wall and soil geometric parameters
- Applied loading

ACI 318-19	Calculations	Description

STEP 1: Retaining Wall and Footing: Summary of Engineer's Relevant Design and Details (continued)

Stability Check

OVERTURNING FACTOR OF SAFETY = 2.883209
AND IF OVERTURNING RESISTANCE FROM LL SURCHARGE IGNORED= 2.80812

SLIDING FACTOR OF SAFETY 1.550523
AND IF SLIDING RESISTANCE FROM LL SURCHARGE IGNORED= 1.52679

Sliding Calculations

f∑W =	4292.1025	pounds, shear friction
Ineffective ht of passive-side soil =	0	ft
key dimension=	0	ft, measured from bottom of ftg
Pp=	1996.002	pounds, passive pressure
∑P,driving=	4055.472412	

Bearing Pressures

	Service		Factored (1.6x)	
At toe:	2737.035362	psf	4379.256578	psf
At heel:	328.7521384	psf	526.0034215	psf

Internal Forces

Footing Heel
(downward Pv is always ignored)
(upward trapezoidal base pressure is ignored)

Vu	14406.48	pounds due to rectangular backfill & footing
	0	pounds due to triangular backfill
	440	pounds due to uniform LL surcharge
		factored pounds due to misc. surcharge**
∑Vu=	14846.48	pounds
$\sum Vu_{@d}$=	10347.55	pounds
Mu	39617.82	lb-ft due to rectangular backfill & footing
	0	lb-ft due to triangular backfill
	1210	lb-ft due to uniform LL surcharge
		factored lb-ft due to misc. surcharge**
∑Mu=	40827.82	lb-ft

FLEXURE OK

β_1 =	0.85	*(0.85 for 4000 psi and lower)*	
d =	20	(ftg thickness minus 4")	
f_y =	60000	psi	
Footing f'c =	4000	psi	
Ru =	0.11341061	ksi	m = 17.64706
ρ_{reqd} =	0.0019228		
$A_{s,reqd}$ =	0.46147171	in^2/foot	(12" unit width)
$A_{s,min1}$ =	0.75894664	in^2/foot	ACI 318-19 Section 9.6.1.2
$A_{s,min2}$ =	0.8	in^2/foot	
$A_{s,4/3*reqd}$ =	0.61529562	in^2/foot	
$A_{s,provided}$ =	0.6626	in^2/foot	ρ_w = 0.002761
a =	0.97441176	inches	ε_t = 0.049339 in/in
φMn =	58.1812982	k-ft	#6 @ 8" oc

SHEAR OK OK

$$\varphi V_c = \left[8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{f'_c} + \frac{N_u}{6A_g}\right] \times b_w \times d$$

Assume the following:
ACI 318-19 Section 22.5.5
- *No shear reinforcement*
- Size effect factor = 1.0
- NWC, so lambda =1.0
- ρ_w is $\frac{A_{s,provided}}{b \times d}$

ACI 318-19: φVc = 12776.32 pounds
ACI 318-14: φVc = 22768.4

Design results for Wall Type 1 are presented here.

- Retaining wall stability results
- Bearing pressure envelope magnitudes
- Footing heel factored internal forces
- Footing heel moment and shear checks

Note that the cladding acknowledged as a potential feature "by others" in the typical detail presented in Figure 2 has been duly coordinated and had been confirmed by the engineer to be lightweight material having negligible impact on retaining wall stability and strength calculations.

ACI 318-19	Calculations	Description
STEP 1: Retaining Wall and Footing: Summary of Engineer’s Relevant Design and Details (continued)		

Wall Stem
(based on plane at back of wall)

Pa'=	2798.698	pounds
Ph'=	2798.698	pounds
Mu,backfill=	17657.92	lb-ft
Vu,backfill-	4477.917	pounds
Mu,unif. srchg	1865.799	lb-ft
Vu,unif. srchg	315.4351	pounds
Mu,misc srchg		factored lb-ft**
Vu,misc srchg		factored pounds**
ΣMu=	19523.72	**AT BASE OF STEM**
ΣVu=	4793.352	

FLEXURE OK

β_1 =	0.85	*(0.85 for 4000 psi and lower)*		
d =	9	(wall thickness minus 3")		
f_y =	60000	psi		
Wall f'c =	4000	psi		
Ru =	0.26781506	ksi	m =	17.64706
ρ_{reqd} =	0.00465476			
$A_{s,\ reqd}$ =	0.50271431	in^2/foot	(12" unit width)	
$A_{s,min}$=	0.2592	in^2/foot	ACI 318-19 Section 7.6.1.1	
$A_{s,\ 4/3*\ reqd}$ =	0.67028574	in^2/foot		
$A_{s,\ provided}$ =	0.53	in^2/foot	ρ_w =	0.004907
a =	0.77941176	inches	ε_t =	0.026445 in/in
φMn =	20.5355515	k-ft	#6 @ 10" oc	

SHEAR ACI 318-19: OK ACI 318-14: OK

$$\varphi V_c = \left[8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{f'_c} + \frac{N_u}{6A_g}\right] \times b_w \times d$$

Assume the following:
ACI 318-19 Section 22.5.5
- No shear reinforcement
- λ_s can't be neglected (might be < 1.0)
- NWC, so lambda =1.0
- ρ_w is $\frac{A_{s,provided}}{b \times d}$
- No axial force

ACI 318-19: φVc = 6964.486 pounds
ACI 318-14: φVc = 10245.78 pounds

Footing Toe
(based on trapezoidal base pressure)

Vu= 6027.0211 pounds
Mu= 4655.7318 lb-ft
3656.771612 psf at toe/stem interface

d= 20 in
A_s = 0.0018bh 0.5184 in^2/foot

Ru = 0.0129326 ksi
ρ_{reqd} = 0.000216
$A_{s,\ reqd}$ = 0.0518291 in^2/foot
4/3 x$A_{s,\ reqd}$ = 0.0691055 in^2/foot

Design results for Wall Type 1 are presented here.

- Wall stem factored internal forces
- Wall stem moment and shear checks
- Footing toe factored internal forces

ACI 318-19	Calculations	Description
STEP 1: Retaining Wall and Footing: Summary of Engineer's Relevant Design and Details (continued)		
		Using the same automated approach, Wall Designs 2 through 4 are also carried out. The engineer's wall design information is compiled in the schedule shown in Figure 4 and used in conjunction with lap splice information in Figure 5 and footing step requirements illustrated in Figure 6.

RETAINING WALL SCHEDULE

MAXIMUM RETAINED HEIGHT	WALL TYPE	GEOMETRY						FOOTING REINFORCEMENT				DOWELS D	WALL REINFORCEMENT			
		A (TOE)	B (HEEL)	C (WALL)	T (FTG)	KV (KEY)	KH (KEY)	TRANSVERSE BOTTOM TB	LONGITUDINAL BOTTOM LB	TRANSVERSE TOP TT	LONGITUDINAL TOP LT		HIGH SIDE VERTICALS V1	HIGH SIDE HORIZONTALS H1	LOW SIDE VERTICALS V2	LOW SIDE HORIZONTALS H2
10'-6"	1	1'-6"	5'-6"	1'-0"	24"	N/A	N/A	N/A	(3) #4 EQ SP.	#6 @ 8" OC	#6 @ 10" OC	#6 @ 10" OC	#5 @ 10" OC	#4 @ 9" OC	N/A	N/A
5'-0"	2	1'-0"	4'-9"	1'-0"	20"	N/A	N/A	N/A	(3) #4 EQ SP.	#6 @ 10" OC	#6 @ 12" OC	#6 @ 12" OC	#5 @ 12" OC	#4 @ 9" OC	N/A	N/A
7'-0"	3	1'-0"	3'-9"	1'-0"	18"	N/A	N/A	N/A	(3) #4 EQ SP.	#5 @ 12" OC	#6 @ 12" OC	#6 @ 12" OC	#5 @ 12" OC	#4 @ 9" OC	N/A	N/A
9'-0"	4	1'-0"	2'-0"	1'-0"	12"	N/A	N/A	N/A	(3) #4 EQ SP.	#5 @ 12" OC	#5 @ 12" OC	#5 @ 12" OC	#5 @ 12" OC	#4 @ 9" OC	N/A	N/A

RETAINING WALL NOTES

1. DESIGNS PRESENTED HERE REPRESENT CONSTRUCTION REQUIREMENTS FOR STRUCTURAL STABILITY OF SITE RETAINING WALLS ONLY. REFER TO OTHER PLANS FOR SITE RETAINING WALL LAYOUT AND COORDINATION OF WALL TYPES WITH ELEVATIONS.
2. TYPICAL RETAINING WALL DETAIL AND SCHEDULE SHALL BE WORKED WITH SITE/CIVIL GRADING DRAWINGS AND OTHER REQUIREMENTS. SEE ARCHITECTURAL AND LANDSCAPE ARCHITECTURAL DRAWINGS FOR OTHER FEATURES.
3. SOIL ATTRIBUTES LISTED BELOW SHALL BE CONFIRMED BY A GEOTECHNICAL ENGINEER TO BE REPRESENTATIVE OF THE PROJECT SITE CONDITIONS RELEVANT TO THE STRUCTURAL SITE RETAINING WALL DESIGNS PRESENTED HERE. CONFIRMATION SHALL BE CARRIED OUT PRIOR TO FABRICATION AND CONSTRUCTION. NOTIFY THE STRUCTURAL ENGINEER OF RECORD OF ANY DISPARITIES.
4. ALLOWABLE SOIL BEARING PRESSURE = 2000 PSF (PRESUMED)
5. SOIL DENSITY = 120 PCF (PRESUMED)
6. HYDROSTATIC PRESSURE = 0 PSF (WALL IS PROPERLY DRAINED)
7. SOIL INTERNAL FRICTION ANGLE φ = 30 DEGREES
8. ACTIVE PRESSURE COEFFICIENT K_a = 0.33
9. PASSIVE PRESSURE COEFFICIENT K_p = 3.0
10. BASE FRICTION FACTOR BETWEEN FOOTING AND SUPPORTING SOIL f = 0.35.
11. WHERE A SHEAR KEY IS REQUIRED (INDICATED BY KV AND KH), LONGITUDINAL BARS SHALL BE #4 @ 8" ON CENTER, MAX.
12. AT CONTRACTOR'S OPTION, IN LIEU OF HIGH SIDE VERTICALS (V1) LAPPED WITH DOWEL BARS (D) AS SHOWN IN DETAIL, HIGH SIDE VERTICALS CAN BE ELIMINATED AND REPLACED BY DOWEL BARS EXTENDING TO TOP OF WALL. IF FULL-HEIGHT DOWEL BAR SOLUTION IS UTILIZED, THE DOWEL BAR SIZE AND SPACING NOTED IN THE SCHEDULE SHALL APPLY.
13. SEE GENERAL NOTES FOR CONCRETE AND REINFORCEMENT MATERIAL REQUIREMENTS.

Figure 4: Excerpt from contract structural drawings: retaining wall schedule

Note that the transition between wall dowels (D) and high side verticals (V1) was confirmed in a separate calculation, with the extension of dowels into the wall conservatively matching the dimension indicated in the lap splice schedule shown in Figure 5 (satisfying development length requirements beyond maximum flexure occurring at the footing-to-wall interface), and the high side verticals themselves designed to resist the flexural demand corresponding to the location of the dowel's terminated upper end (with sufficient development length measured downward from that point towards the top of the footing).

CONCRETE REINFORCEMENT LAP SPLICE SCHEDULE						
CONCRETE STRENGTH IN PSI (f 'c)						
60 KSI BAR	3,000	3,500	4,000	4,500	5,000	6,000
#3	28 23	26 20	25 19	23 18	22 17	20 16
#4	38 29	35 27	33 25	31 24	29 23	27 21
#5	47 36	43 33	41 31	38 30	36 28	33 26
#6	56 43	52 40	49 37	46 35	44 34	40 31
#7	81 63	75 58	71 54	67 51	63 49	58 45
#8	93 72	86 66	81 62	76 59	72 56	66 51
#9	105 81	97 75	91 70	86 66	81 63	74 57
#10	118 91	109 84	102 79	96 74	92 71	84 64
#11	131 101	121 93	114 87	107 82	102 78	93 71

TOP BARS, TYP.
OTHER BARS, TYP.

CONCRETE REINFORCEMENT LAP SPLICE SCHEDULE NOTES:

1. TABLE SHOWS CLASS B LAP SPLICE LENGTHS PER ACI 318-19.
2. TOP BARS ARE ALL HORIZONTAL OR NEAR-HORIZONTAL BARS PLACED WITH MORE THAN 12 VERTICAL INCHES OF FRESH CONCRETE BELOW THEM.
3. SEE TYPICAL SPLICE DETAILS FOR LAP SPLICE LOCATION AND STAGGER REQUIREMENTS.
4. INCREASE TABULATED LAP LENGTHS 33% FOR LIGHTWEIGHT CONCRETE (<145 PCF).
5. INCREASE TABULATED LAP LENGTHS 20% FOR 3-BAR BUNDLES (WHERE ONE BAR LAPS WITH TWO OTHER BARS).
6. INCREASE TABULATED LAP LENGTHS 33% FOR 4-BAR BUNDLES (WHERE TWO BARS LAP WITH TWO OTHER BARS).
7. TABULATED VALUES ARE BASED ON MINIMUM CLEAR COVER > d_b (3/4" MIN) AND CLEAR SPACING > $2d_b$ (1.5" MIN).
8. INCREASE LAP LENGTHS 50% FOR EPOXY COATED BARS.
9. FOR WELDED WIRE REINFORCEMENT, WIRES D11.1 AND SMALLER SHALL SATISFY #3 BAR LAP REQUIREMENTS, WIRES BETWEEN D11.1 AND D20.0 SHALL SATISFY #4 BAR LAP REQUIREMENTS, AND WIRES BETWEEN D20.1 AND D31 SHALL SATISFY #5 BAR LAP REQUIREMENTS.
10. FOR REINFORCEMENT APPLICATIONS OTHER THAN ELEVATED SLABS, BEAMS, AND COLUMNS, LAP SPLICE LENGTHS NOTED ARE PERMITTED TO BE REPLACED WITH REFINED LAP SPLICE LENGTHS THAT ARE DERIVED FROM ACI 318-19 SECTION 25.4.2.4 DEVELOPMENT LENGTHS, PROVIDED FABRICATOR SUBMITS ACCOMPANYING REPRESENTATIVE CALCULATIONS AS PART OF REINFORCEMENT SUBMITTAL.

Figure 5: Excerpt from contract structural drawings: lap splice schedule

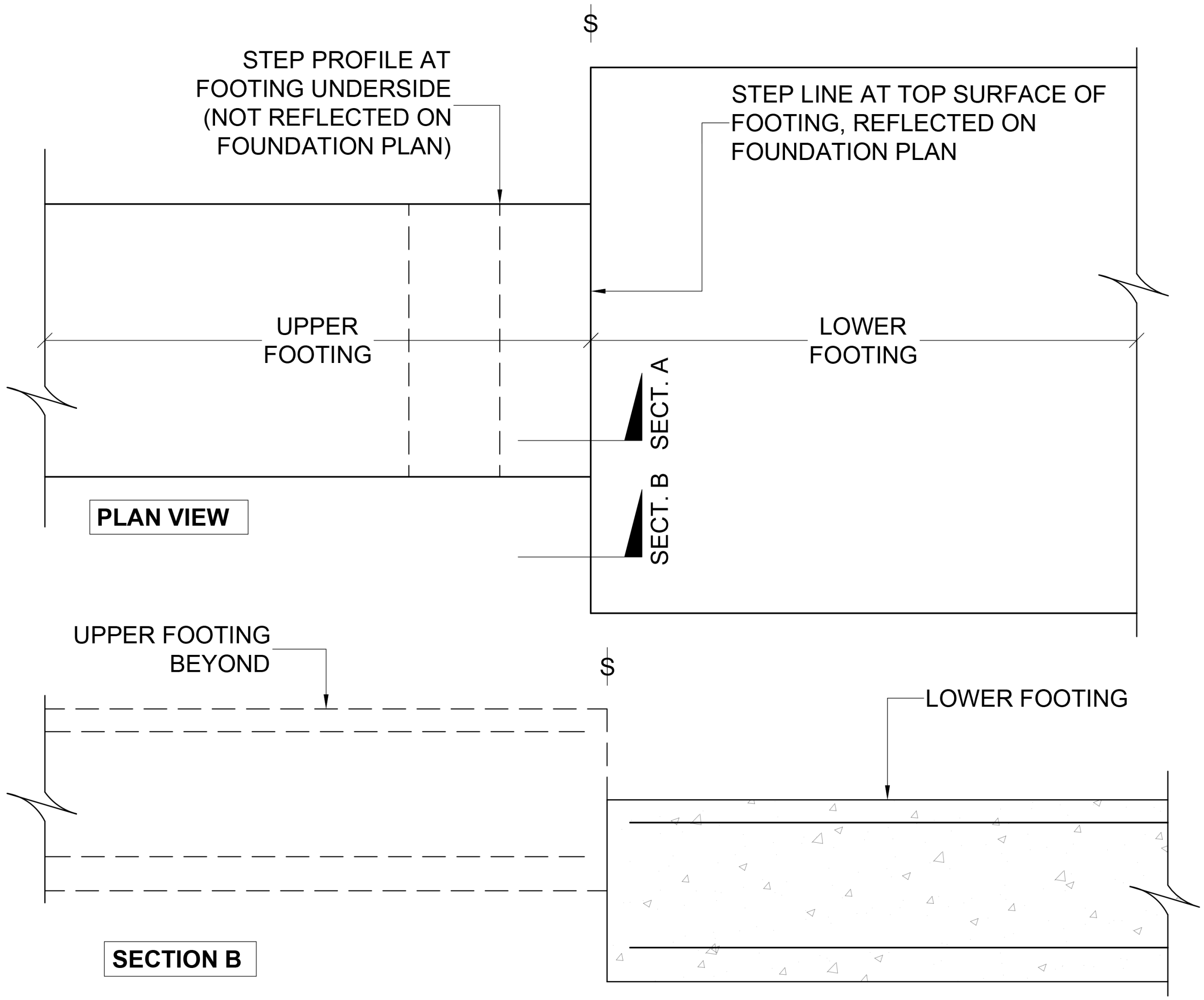
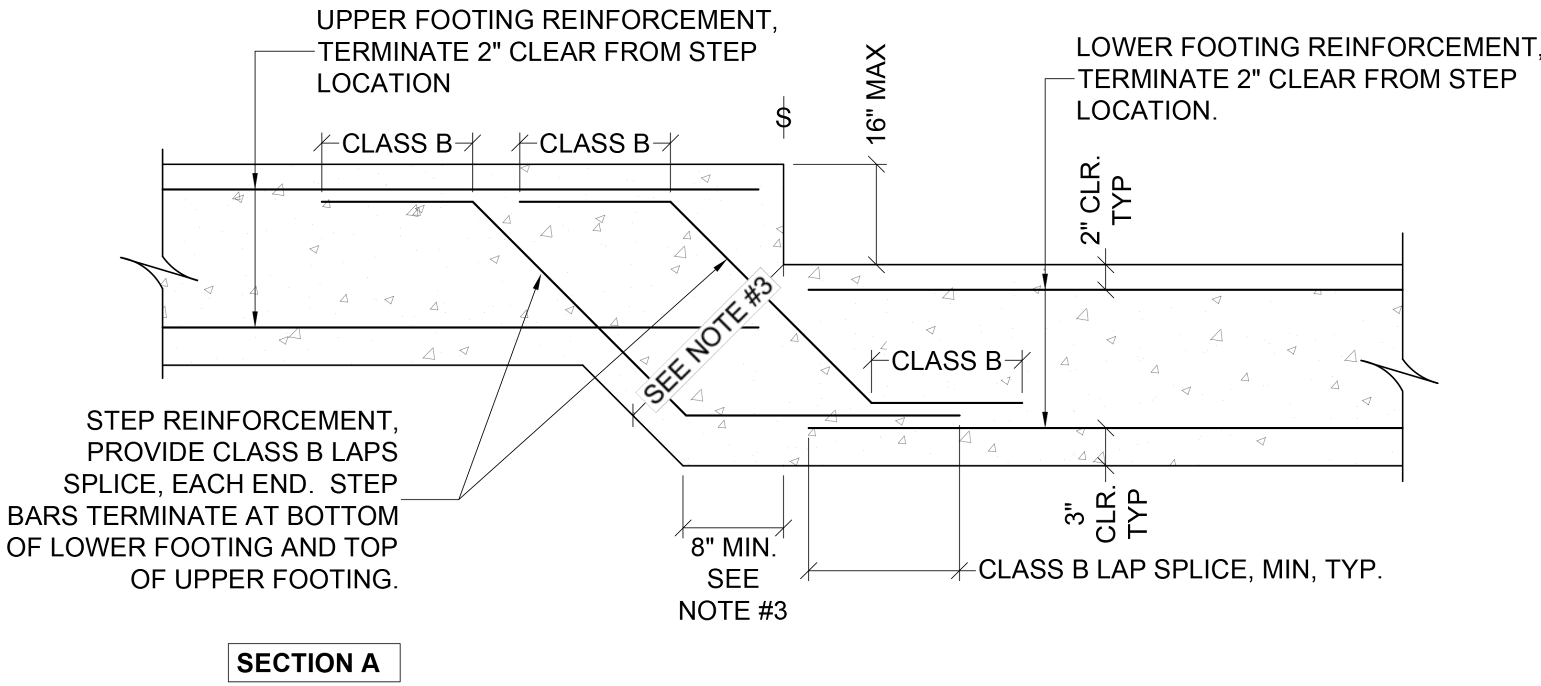


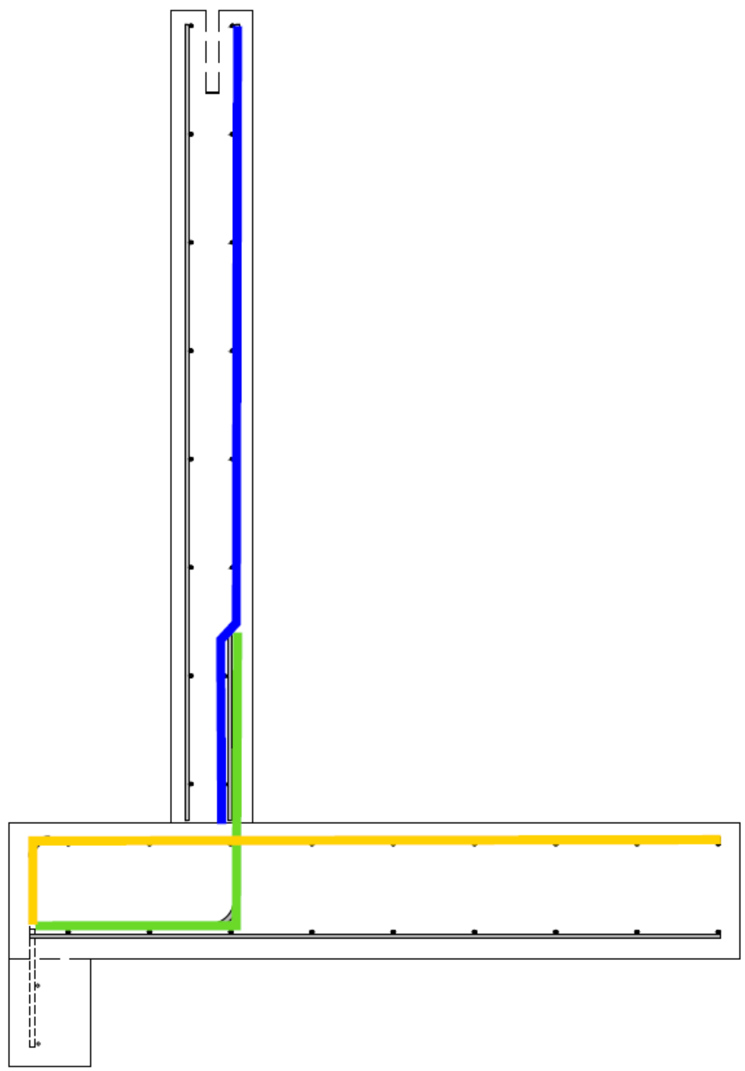
Figure 6: Excerpt from contract structural drawings: typical stepped footing detail



NOTES:

1. FOOTINGS WITH TOP AND BOTTOM REINFORCEMENT SHOWN HERE. WHERE ONLY BOTTOM FOOTING REINFORCEMENT IS REQUIRED PER SCHEDULE, ONLY THE STEP REINFORCEMENT NEAREST THE FOOTING BOTTOM SURFACE IS REQUIRED. STEP REINFORCEMENT SHALL TERMINATE AT BOTTOM OF LOWER FOOTING AND TOP OF UPPER FOOTING.
2. ONLY FOOTING LONGITUDINAL BARS ARE SHOWN. TRANSVERSE BARS (INTO PAGE) ARE REQUIRED PER SCHEDULE AND OTHER DETAILS. PROVIDE TRANSVERSE BARS THROUGH THE STEP TO MATCH THOSE REQUIRED AT NARROWER FOOTING.
3. PROVIDE 8" MINIMUM EXTENSION OF LOWER FOOTING. INCREASE DIMENSION AS REQUIRED TO MAINTAIN STRUCTURAL THICKNESS OF NARROWER FOOTING THROUGH THE STEP.

Figure 6 (continued): Excerpt from contract structural drawings: typical stepped footing detail

ACI 318-19	Calculations	Description
STEP 2: WWR Detailing Procedure – Wall Type 1		
20.2.2.4		With the necessary structural design and detailing information – including permissive WWR language – available on the structural contract drawings, the WWR detailer confirms suitability of a WWR option per Chapter 20 of ACI 318-19. The retaining wall in question is neither a special moment frame nor is it a special structural wall relied upon by the adjacent parking structure for lateral stability, therefore a WWR conversion is appropriate. The first step for the fabricator detailer is to generate a triage grouping of WWR mat types. There are essentially three mat types to configure: 1. Footing (yellow): mat will be configured to capture transverse and longitudinal reinforcement requirements for the footing. Mats to be identified using general form **WWR-F-XX**. 2. Wall dowels (green): mat will be configured to contain the vertical dowels as well as the longitudinal bottom reinforcement that is allocated to the toe region of the footing. Mats to be identified using general form **WWR-D-XX**. 3. Wall Stem (blue): mat will be configured to capture horizontal and vertical reinforcement for the wall. Mats to be identified using general form **WWR-W-XX**.

ACI 318-19	Calculations	Description

STEP 2: WWR Detailing Procedure – Wall Type 1 (continued)

20.2.2.4

Location	Specified	$A_{s,spec}$	WWR	$A_{s,WWR\ prov'd}$
1 – FTG TRANS	#6 @ 8"	0.66 in^2/ft	D22 @ 4"	0.66 in^2/ft
1 – FTG LONG	#6 @ 10"	0.528 in^2/ft	D22 @ 5"	0.528 in^2/ft
1 – DOWEL	#6 @ 10"	0.528 in^2/ft	D22 @ 5"	0.528 in^2/ft
1 – DOWEL LONG	(3) #4	0.60 in^2	(3) D20	0.60 in^2
1 – STEM VERT	#5 @ 10"	0.372 in^2/ft	D31 @ 10"	0.372 in^2/ft
1 – STEM HORIZ	#4 @ 9"	0.267 in^2/ft	D20 @ 9"	0.267 in^2/ft
2 – FTG TRANS	#6 @ 10"	0.528 in^2/ft	D22 @ 5"	0.528 in^2/ft
2 – FTG LONG	#5 @ 12"	0.31 in^2/ft	D31 @ 12"	0.31 in^2/ft
2 – DOWEL	#6 @ 12"	0.44 in^2/ft	D22 @ 6"	0.44in^2/ft
2 – DOWEL LONG	(3) #4	0.60 in^2	(3) D20	0.60 in^2
2 – STEM VERT	#5 @ 12"	0.31 in^2/ft	D31 @ 12"	0.31 in^2/ft
2 – STEM HORIZ	#4 @ 9"	0.267 in^2/ft	D20 @ 9"	0.267 in^2/ft
3 – FTG TRANS	#5 @ 12"	0.31 in^2/ft	D31 @ 12"	0.31 in^2/ft
3 – FTG LONG	#5 @ 12"	0.31 in^2/ft	D31 @ 12"	0.31 in^2/ft
3 – DOWEL	#6 @ 12"	0.44 in^2/ft	D22 @ 6"	0.44in^2/ft
3 – DOWEL LONG	(3) #4	0.60 in^2	(3) D20	0.60 in^2
3 – STEM VERT	#5 @ 12"	0.31 in^2/ft	D31 @ 12"	0.31 in^2/ft
3 – STEM HORIZ	#4 @ 9"	0.267 in^2/ft	D20 @ 9"	0.267 in^2/ft
4 – FTG TRANS	#5 @ 12"	0.31 in^2/ft	D31 @ 12"	0.31 in^2/ft
4 – FTG LONG	#5 @ 12"	0.31 in^2/ft	D31 @ 12"	0.31 in^2/ft
4 – DOWEL	#5 @ 12"	0.31 in^2/ft	D31 @ 12"	0.31 in^2/ft
4 – DOWEL LONG	(3) #4	0.60 in^2	(3) D20	0.60 in^2
4 – STEM VERT	#5 @ 12"	0.31 in^2/ft	D31 @ 12"	0.31 in^2/ft
4 – STEM HORIZ	#4 @ 9"	0.267 in^2/ft	D20 @ 9"	0.267 in^2/ft

The fabricator detailer will create a conversion or correlation chart to track basic welded wire reinforcement size and spacing combinations that will be used. This information typically comprises a portion of the manufacturer's submittal documentation that accompanies the reinforcing steel shop drawings presented for the design professional's review.

The reinforcing bar arrangements can be managed comfortably with only three wire size variations (D20, D22, and D31) with spacing adjusted to satisfy required steel area per unit linear foot of coverage defined by the engineer. By providing WWR with identical unit steel areas, equal or lesser spacing, and identical curtailment (hooks and laps) to that of the originally-specified reinforcing bar solution, the detailer is complying with the engineer's permissive language that is highlighted in Figure 3.

ACI 318-19	Calculations	Description
STEP 2: WWR Detailing Procedure – Wall Type 1 (continued)		
20.2.2.4	**<u>Wall Type 1</u>** *<u>Footing longitudinal reinforcement extents</u>* $\frac{320.375"}{2} = 160.1875 \text{ per "panel" (each side of control joint)}$ Accounting for clear cover, and conservatively applying 3" cover at step location (in lieu of the 2" dimension defined): $160.1875" - 3" \, from\, end \, - 3" \, from\, step = \; 154.1875"$ $\therefore \mathbf{12' - 10 \;\; 3/16"}, \boldsymbol{per\ panel}$ *<u>Wall longitudinal reinforcement extents</u>* $160.1875 \text{ per "panel"}$ $160.1875" - 2" - 2" = \; 156.1875"$ $\therefore \mathbf{13' - 0 \;\; 3/16"}, \boldsymbol{per\ panel}$	With WWR steel area conversions established, the geometric requirements of the WWR mats will now be established. For this example, **Wall Type 1 will be the focus of the exercise.** An important consideration for the detailer is continuity and transition of the reinforcement. For the walls, the presence of vertical control joints and expansion joints represents intentional discontinuities in the horizontal reinforcing steel in order to achieve weakened plane behavior for serviceability and durability requirements. This will require the detailer to configure mats such that horizontal wire terminated to each side of each joint locations (the engineer's joint details are not presented herein, but for detailing purposes reinforcing steel shall be held 2" clear from the vertical joints). For the footings, there is no reinforcement discontinuity, but the steel must transition through elevation and configuration change at the footing step locations. This is most commonly handled by the detailer through the provision of individual loose transitional bent wires/bars that accompany the welded wire reinforcement package.

ACI 318-19	Calculations	Description
STEP 2: WWR Detailing Procedure – Wall Type 1 (continued)		
20.2.2.4	**<u>Wall Type 1 (continued)</u>** **<u>WWR-F-1 @ footing</u>** • D22 @ 4” oc transverse • D22 @ 5” oc longitudinal • Per the engineer’s lap schedule, refinement of the lap length is permitted provided the fabricator’s lap splice length calculations are included in the reinforcement submittal.	
25.4.2.1 25.4.2.4	**Longitudinal wires in footing must lap with Class B lap splice per engineer’s typical retaining wall detail:** $l_d = \left(\frac{3 \times f_y \times \psi_t \times \psi_e \times \psi_s \times \psi_g}{40 \times \lambda \times \sqrt{f'_c} \times \left(\frac{c_b + K_{tr}}{d_b} \right)} \right) \times = \mathbf{15.7\ inches} > 12\ inches$ $f_y = 60{,}000\ psi$ $\psi_t = 1.3$ $\psi_e = 1.0$ $\psi_s = 0.8$ $\psi_g = 1.0\ (steel\ area\ being\ used\ is\ based\ on\ 60\ ksi\ rebar)$ $\lambda = 1.0$ $\sqrt{f'_c} = 63.245$ $c_b = 2.5"$ $K_{tr} = 0\ as\ a\ design\ simplification\ permitted\ by\ ACI\ 318$	Contract structural drawings often include prescriptive requirements for reinforcement lap splices based on development length equations set forth in Table 25.4.2.3, which typically result in lap splice requirements significantly larger than those that can be derived from development lengths calculated in accordance with Section 25.4.2.4. for this example, the engineer provides permissive language for the refinement of lap splice lengths noted in the table. The WWR detailer carries out the development length and lap splice length calculations noted here based on the engineer’s above described permitted refinement. Note that the detailer is not relying on welded intersections to contribute to the tension development of wires, so the calculation of development length and lap splice length is based entirely on the deformed wire surface itself, no different than what is done for loose deformed wires and bars.
25.5.2.1 25.5.3.1.1	$d_b = 0.530"$ $\left(\frac{c_b + K_{tr}}{d_b} \right) = use\ 2.5\ max$ $1.3 \times 15.7\ inches = \mathbf{21\ inch\ Class\ B\ lap\ splice}$ *(Compare this to the “unrefined” lap splice of 41 inches in the schedule.)*	

ACI 318-19	Calculations	Description
STEP 2: WWR Detailing Procedure – Wall Type 1 (continued)		
20.2.2.4	**Wall Type 1 (continued)** **WWR-F-1** Reinforcement for each "panel" of Wall Type 1's footing is to be resolved in the form of two (2) WWR mats, (a) and (b).	Refer to Figure 7 for the WWR mat configurations. Refer to Figures 8 and 9 for the combined mat coverage.

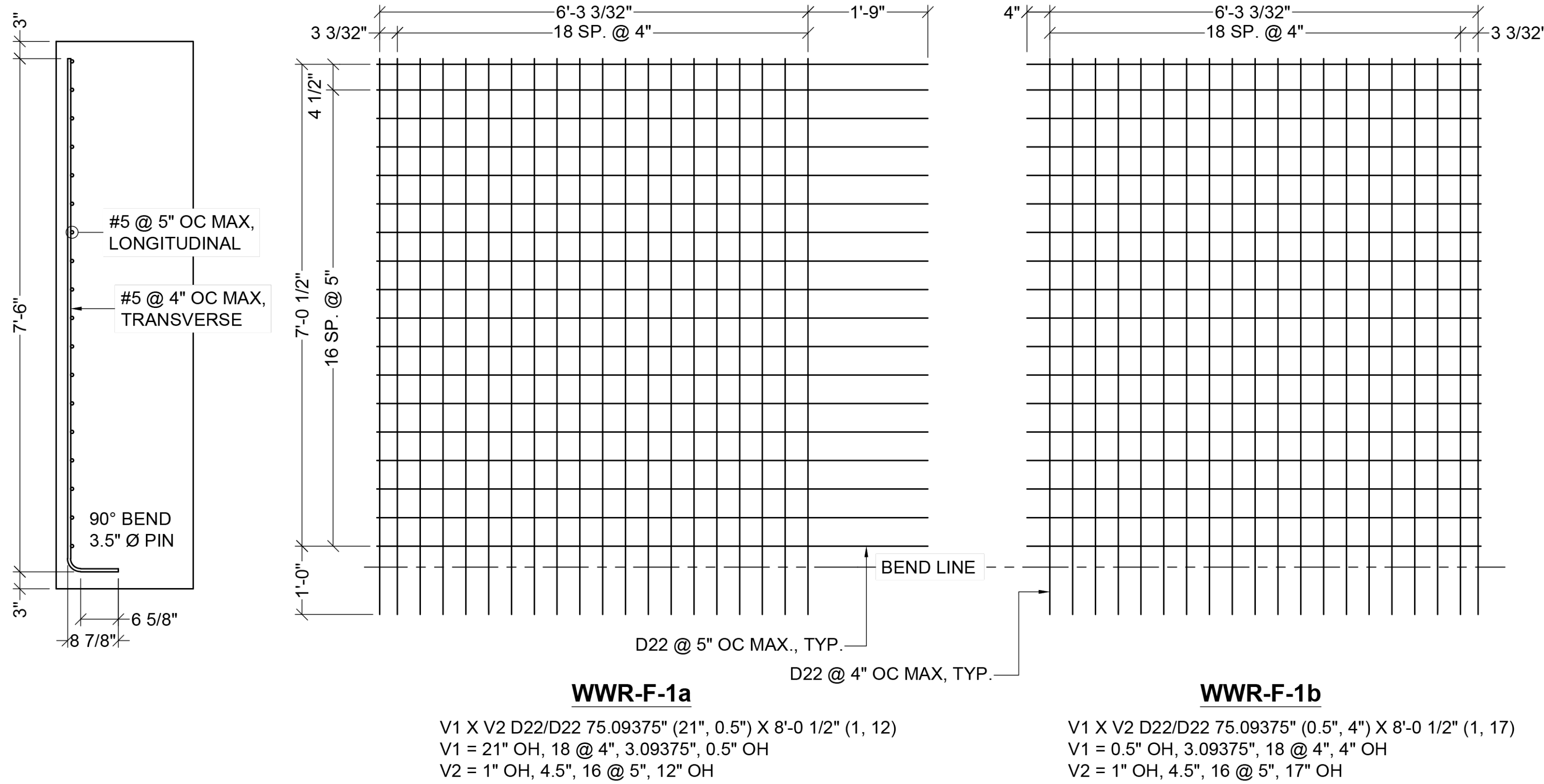


Figure 7: Wall Type 1−Footing Mat WWR-F-1(a) and WWR-F-1(b)

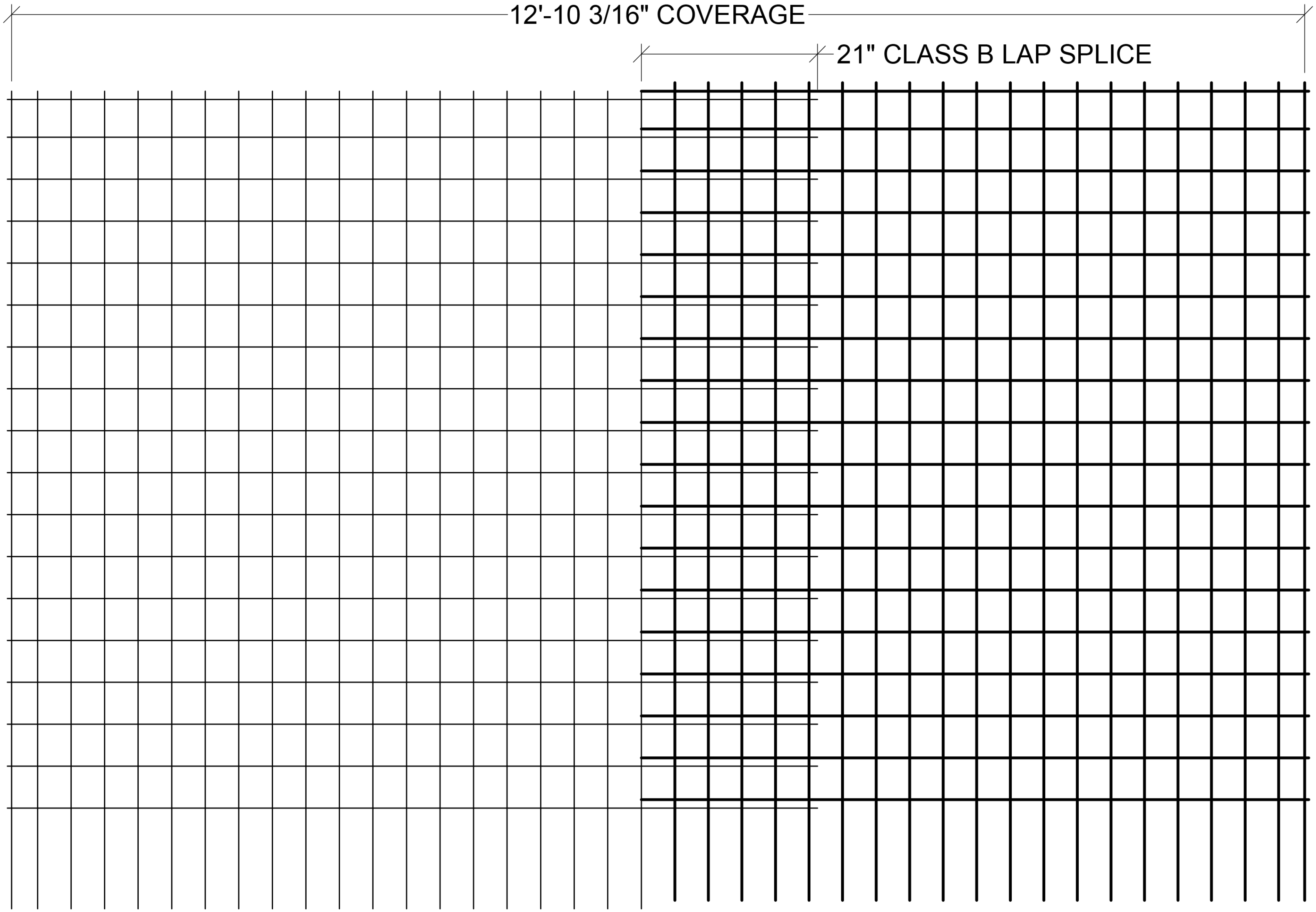


Figure 8: Wall Type 1−Footing Mat coverage. Two mats per footing elevation, resulting in four (4) total mats for Wall Type 1 footing.

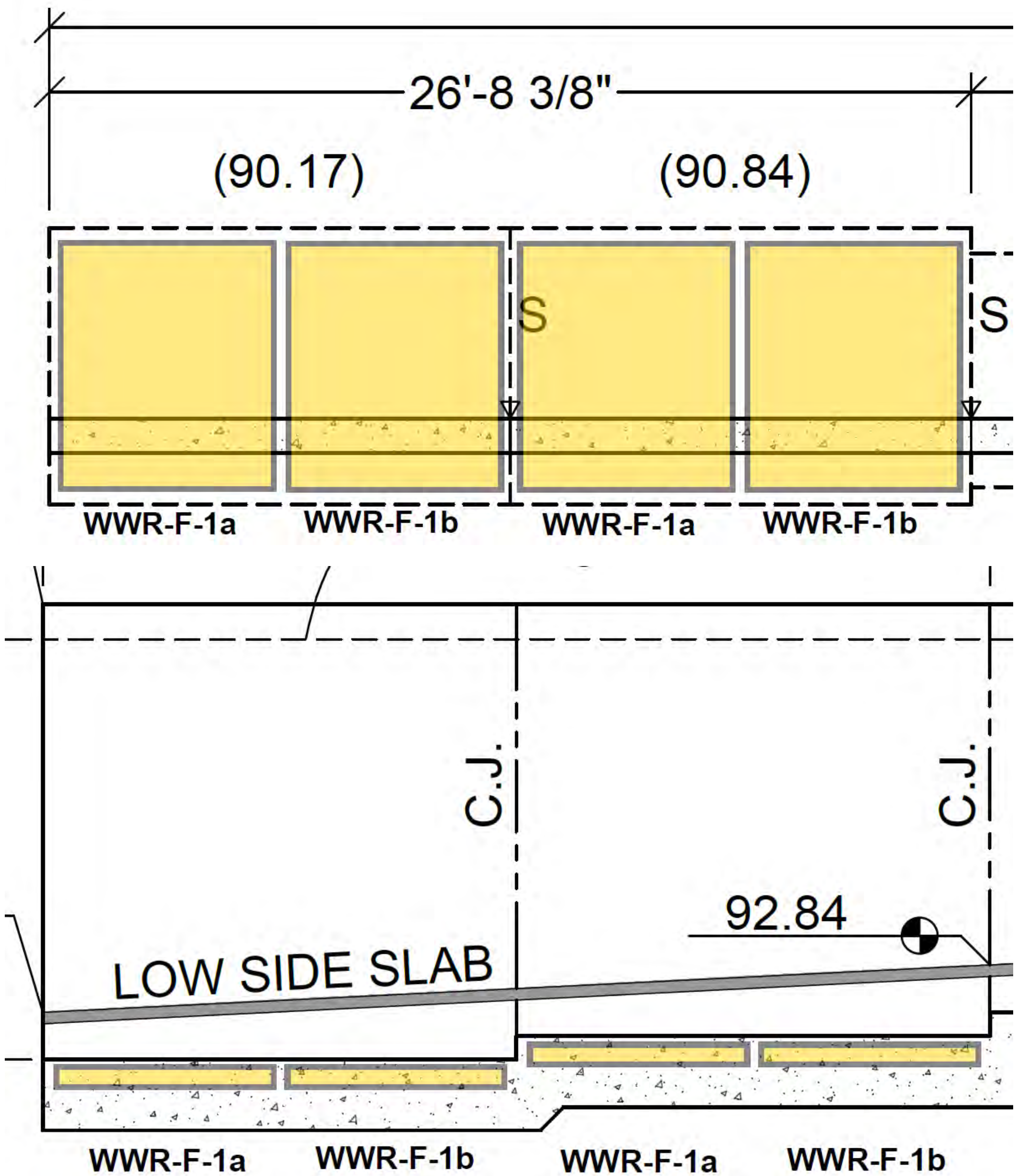


Figure 9: Wall Type 1−Superimposed placement of footing mats

ACI 318-19	Calculations	Description
STEP 2: WWR Detailing Procedure – Wall Type 1 (continued)		
20.2.2.4 25.4.2.1 25.4.2.4 25.5.2.1	**Wall Type 1 (continued)** **WWR-D-1 @ dowels** • D22 @ 5" oc vertical dowels • (3) D20 longitudinal (bottom bars in footing) • Per the engineer's lap schedule, refinement of the lap length is permitted provided the fabricator's lap splice length calculations are included in the reinforcement submittal. **Vertical extension of dowels into footing must be Class B lap splice per engineer's typical retaining wall detail:** $l_d = \left(\frac{3 \times f_y \times \psi_t \times \psi_e \times \psi_s \times \psi_g}{40 \times \lambda \times \sqrt{f'_c} \times \left(\frac{c_b + K_{tr}}{d_b} \right)} \right) \times = \mathbf{12.1\ inches} > 12\ inches$ $f_y = 60{,}000\ psi$ $\psi_t = 1.0$ $\psi_e = 1.0$ $\psi_s = 0.8$ $\psi_g = 1.0\ (steel\ area\ being\ used\ is\ based\ on\ 60\ ksi\ rebar)$ $\lambda = 1.0$ $\sqrt{f'_c} = 63.245$ $c_b = 2.5"$ $K_{tr} = 0\ as\ a\ design\ simplification\ permitted\ by\ ACI\ 318$ $d_b = 0.530"$ $\left(\frac{c_b + K_{tr}}{d_b} \right) = use\ 2.5\ max$ $1.3 \times 12.1\ inches = \mathbf{16\ inch\ Class\ B\ lap\ splice}$	 Calculate the required lap splice length of vertical dowel leg above the top of the footing, into the wall stem.

ACI 318-19	Calculations	Description
STEP 2: WWR Detailing Procedure – Wall Type 1 (continued)		
20.2.2.4 25.4.2.1 25.4.2.4 25.5.2.1	**<u>Wall Type 1 (continued)</u>** **<u>WWR-D-1 @ dowels</u>** • D22 @ 5” oc vertical dowels • (3) D20 longitudinal (bottom bars in footing) • Per the engineer’s lap schedule, refinement of the lap length is permitted provided the fabricator’s lap splice length calculations are included in the reinforcement submittal. **Required lap splice length of bottom footing reinforcement:** $l_d = \left(\frac{3 \times f_y \times \psi_t \times \psi_e \times \psi_s \times \psi_g}{40 \times \lambda \times \sqrt{f'_c} \times \left(\frac{c_b + K_{tr}}{d_b}\right)} \right) \times = \mathbf{11.5\ inches} < 12\ inches$ $f_y = 60{,}000\ psi$ $\psi_t = 1.0$ $\psi_e = 1.0$ $\psi_s = 0.8$ $\psi_g = 1.0\ (steel\ area\ being\ used\ is\ based\ on\ 60\ ksi\ rebar)$ $\lambda = 1.0$ $\sqrt{f'_c} = 63.245$ $c_b = 3.25"$ $K_{tr} = 0\ as\ a\ design\ simplification\ permitted\ by\ ACI\ 318$ $d_b = 0.505"$ $\left(\frac{c_b + K_{tr}}{d_b}\right) = use\ 2.5\ max$ $1.3 \times 11.5\ inches = \mathbf{15\ inches} > 12 inch\ \mathbf{Class\ B\ lap\ splice}$	 Calculate the required lap splice length of the longitudinal wires to be located at the bottom of the footing.

ACI 318-19	Calculations	Description
STEP 2: WWR Detailing Procedure – Wall Type 1 (continued)		
20.2.2.4	**<u>Wall Type 1 (continued)</u>** **<u>WWR-D-1 @ dowels</u>** Reinforcement for each "panel" of Wall Type 1's wall dowels is to be resolved in the form of two (2) WWR mats, (a) and (b).	Refer to Figure 10 for the WWR mat configurations. Refer to Figures 11 and 12 for the combined mat coverage.

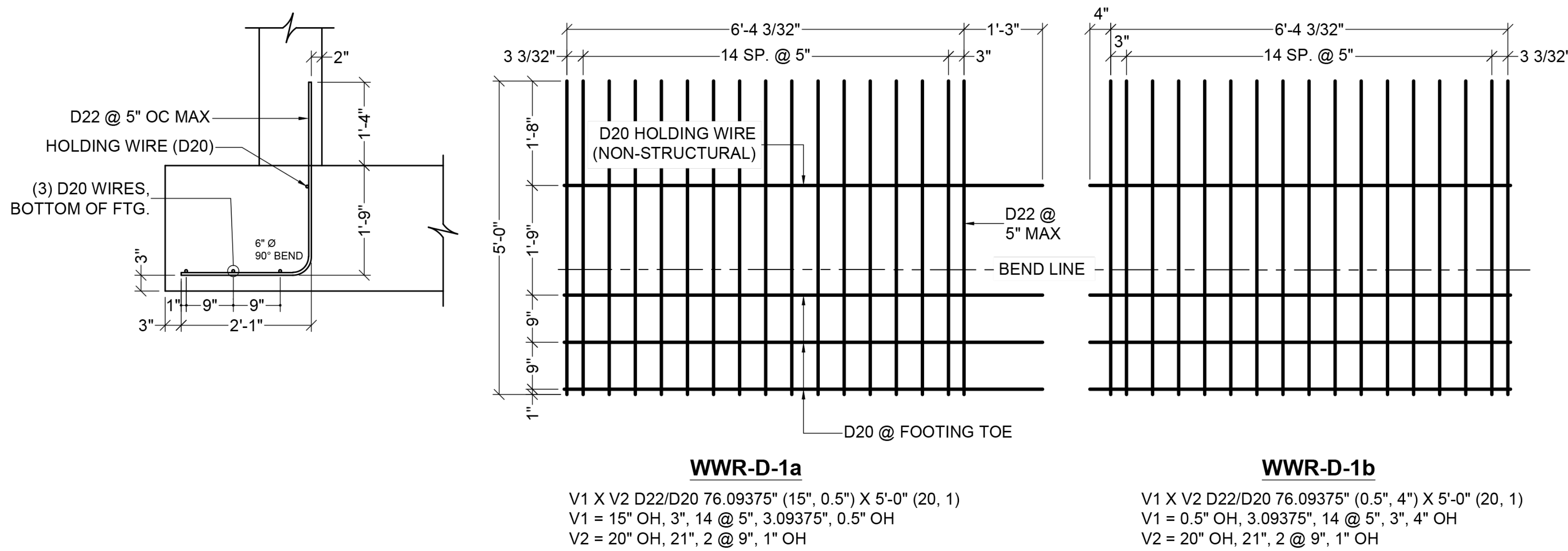


Figure 10: Wall Type 1−Wall Dowel Mat WWR-D-1(a) and WWR-D-1(b)

Note the non-structural holding wire added to assist in maintaining the shape of the bent WWR mat during installation. The WWR detailer can actually configure this mat such that the non-structural holding wire is positioned to serve as a bolster line for support of the previously configured footing mats (WWR-F-1).

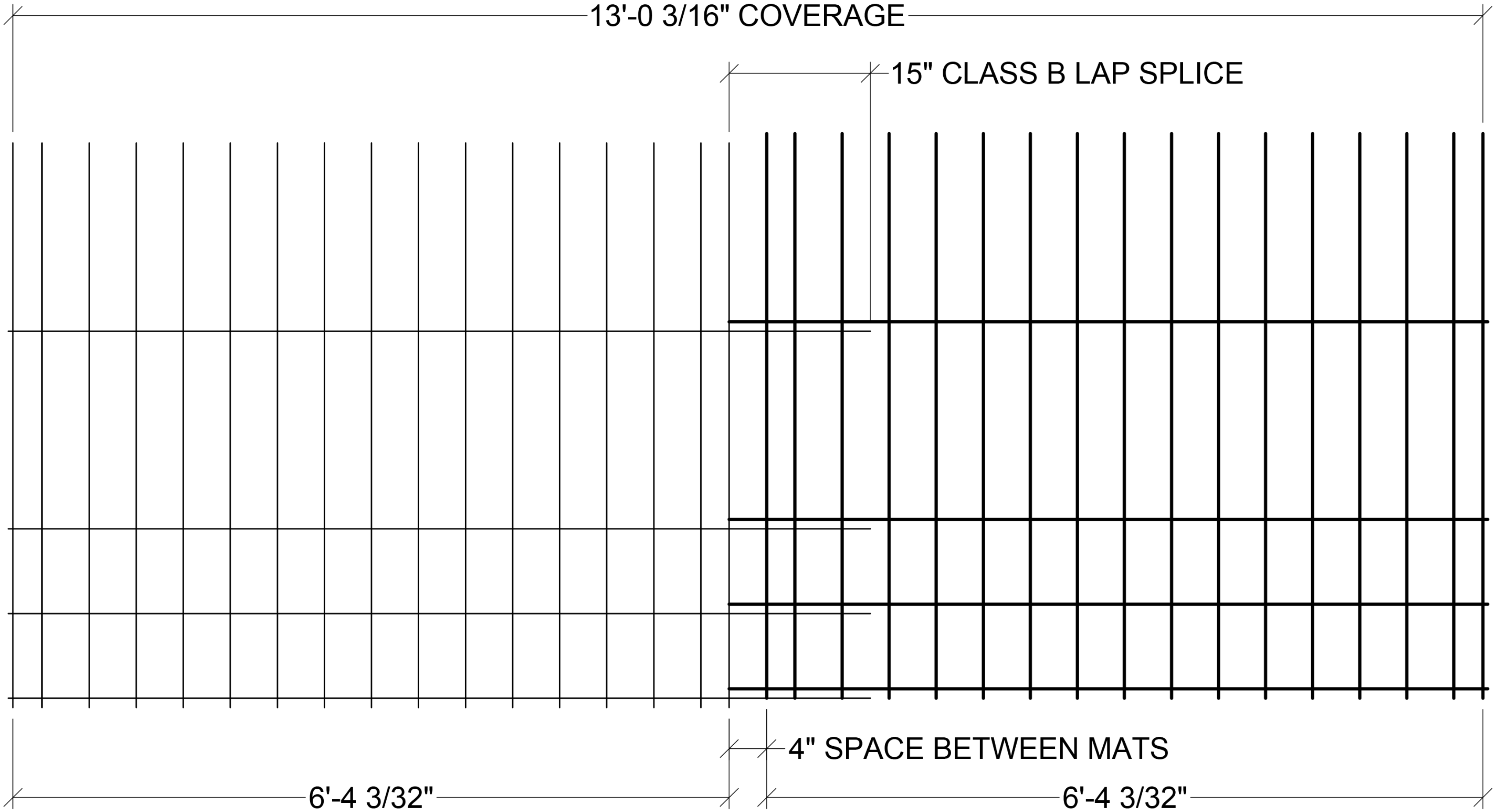


***Figure 11:** Wall Type 1 − Dowel Mat coverage. Two mats per footing elevation, resulting in four (4) total mats for Wall Type 1. Note the 4" space between the two mats that, when cumulative with the two mats wire-to-wire coverage, results in the overall required coverage for the "panel" in question.*

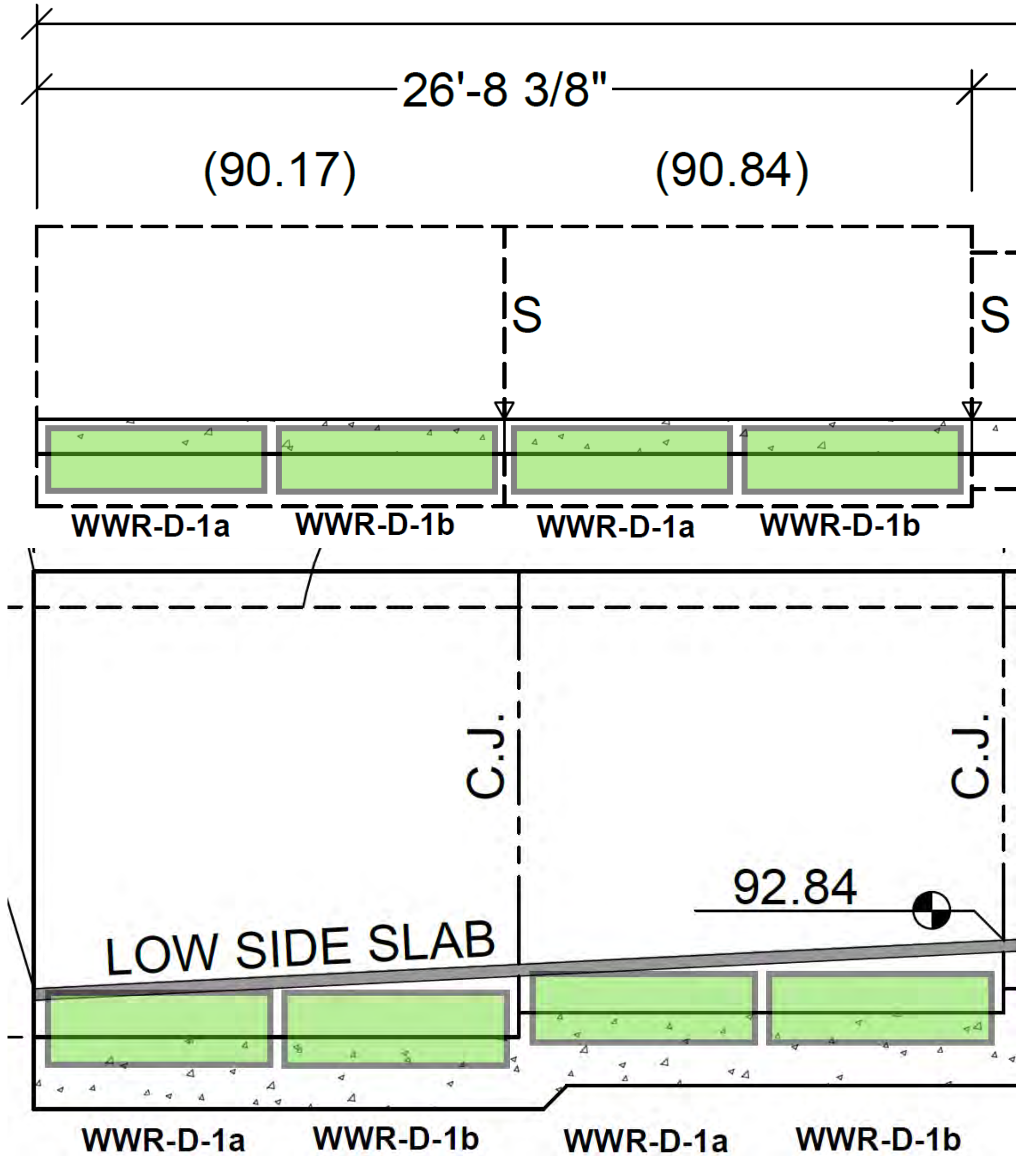
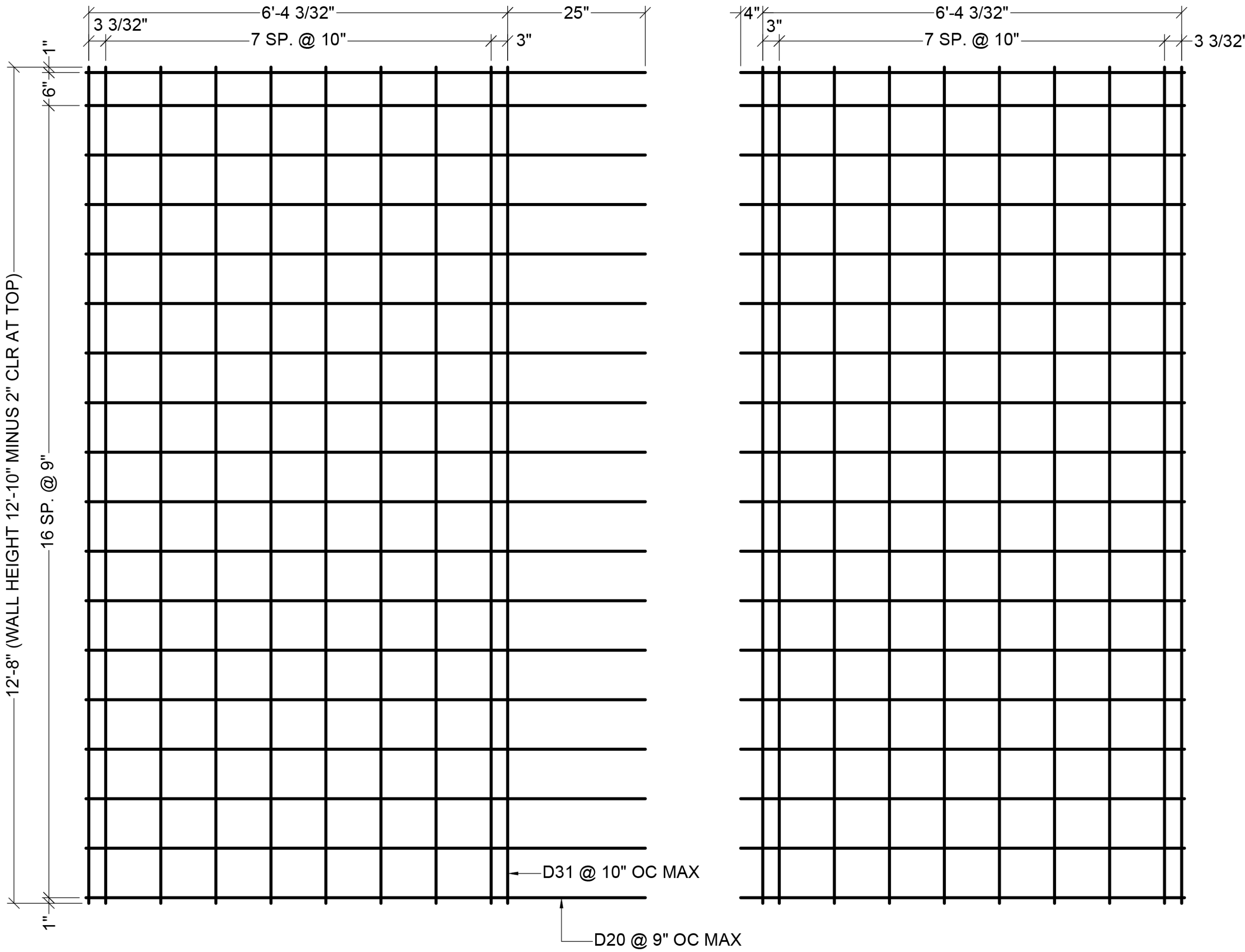


Figure 12: Wall Type 1 – Superimposed placement of dowel mats

ACI 318-19	Calculations	Description
STEP 2: WWR Detailing Procedure – Wall Type 1 (continued)		
20.2.2.4 25.4.2.1 25.4.2.4 25.5.2.1	**<u>Wall Type 1 (continued)</u>** **<u>WWR-W-1 @ wall stem</u>** • D31 @ 10” oc vertical dowels • D20 @ 9” oc horizontal • Per the engineer’s lap schedule, refinement of the lap length is permitted provided the fabricator’s lap splice length calculations are included in the reinforcement submittal. **Horizontal wires in wall stem must lap with Class B lap splice per engineer’s typical retaining wall detail:** $l_d = \left(\frac{3 \times f_y \times \psi_t \times \psi_e \times \psi_s \times \psi_g}{40 \times \lambda \times \sqrt{f'_c} \times \left(\frac{c_b + K_{tr}}{d_b}\right)}\right) \times = \mathbf{18.7\ inches} > 12\ inches$ $f_y = 60{,}000\ psi$ $\psi_t = 1.3$ $\psi_e = 1.0$ $\psi_s = 0.8$ $\psi_g = 1.0\ (steel\ area\ being\ used\ is\ based\ on\ 60\ ksi\ rebar)$ $\lambda = 1.0$ $\sqrt{f'_c} = 63.245$ $c_b = \pm 2.75"$ $K_{tr} = 0\ as\ a\ design\ simplification\ permitted\ by\ ACI\ 318$ $d_b = 0.505"$ $\left(\frac{c_b + K_{tr}}{d_b}\right) = use\ 2.5\ max$ $1.3 \times 18.7\ inches = \mathbf{25\ inch\ Class\ B\ lap\ splice}$	 Calculate the required lap splice length of horizontal wires.

ACI 318-19	Calculations	Description
STEP 2: WWR Detailing Procedure – Wall Type 1 (continued)		
20.2.2.4	**Wall Type 1 (continued)** **WWR-W-1 @ wall stem** Reinforcement for each "panel" of Wall Type 1's wall stem is to be resolved in the form of two (2) WWR mats, (a) and (b). Note that the horizontal coverage and arrangement of wires for the wall stem mats is kept identical to that which was previously calculated for the wall dowel mats to assist with alignment during placement in the field.	Refer to Figure 13 and 14 for the WWR mat configurations. Refer to Figures 15 and 16 for the combined mat coverage. The arrangements of wires presented here, for all three mat regions (footing, dowel, and wall stem), are only one sampling of the numerous wire size and spacing combinations that could be utilized to satisfy the engineer's permissive language noted in Figure 3. This is where the experience of the manufacturer's detailer and affiliated technical staff is so critical to the success of the project. Configuring welded wire reinforcement solutions for projects that utilize the "**specification by pre-approval**" method as opposed to the "**direct specification**" method requires a keen awareness of maintaining steel area equivalencies, curtailment requirements, and maximum and minimum spacing criteria, all while deriving an arrangement of wires that is well-suited for the WWR production equipment and is as intuitive as possible for the project's engineer and contractor alike.



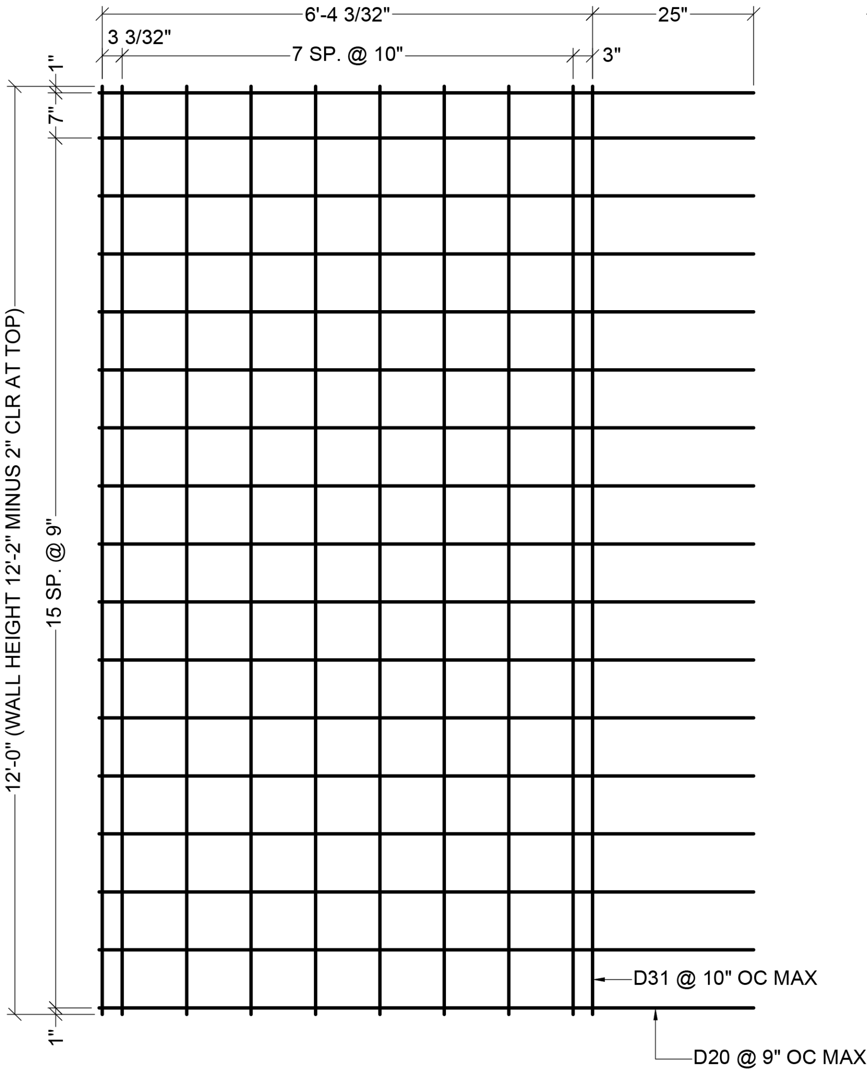
WWR-W-1a

V1 X V2 D31/D20 76.09375" (25", 0.5") X 12'-8" (1, 1)
V1 = 25" OH, 3", 7 @ 10", 3.09375", 0.5" OH
V2 = 1" OH, 6", 16 @ 9", 1" OH

WWR-W-1b

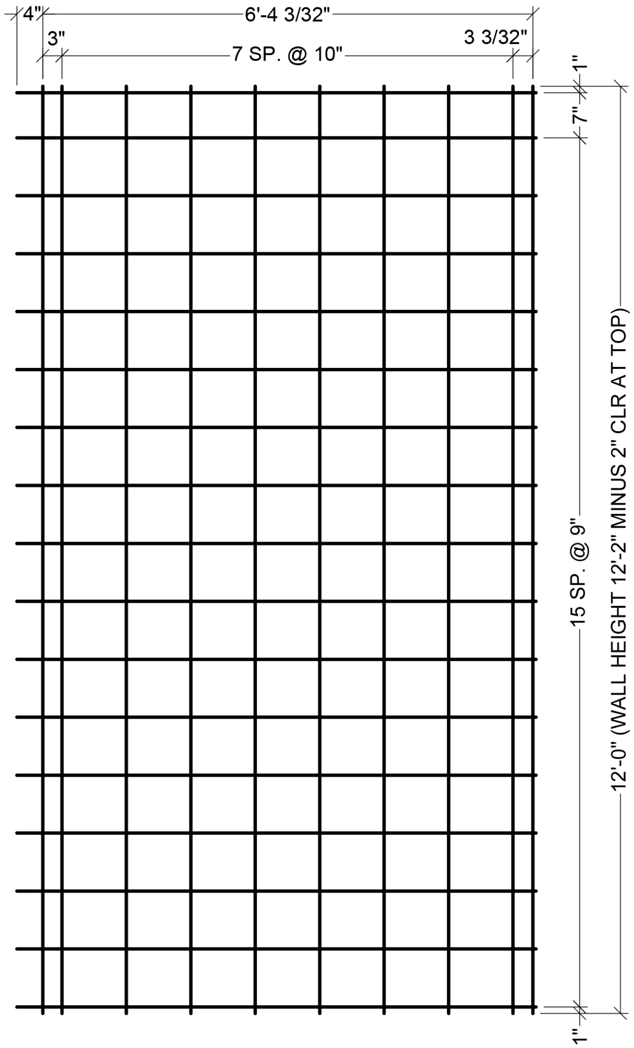
V1 X V2 D31/D20 76.09375" (0.5", 4") X 12'-8" (1, 1)
V1 = 0.5" OH, 3.09375", 7 @ 10", 3", 4" OH
V2 = 1" OH, 6", 16 @ 9", 1" OH

Figure 13: Wall Type 1−Wall Stem Mat WWR-W-1(a) and WWR-W-1(b) @ 12’-10” tall wall



WWR-W-1c

V1 X V2 D31/D20 76.09375" (25", 0.5") X 12'-0" (1, 1)
V1 = 25" OH, 3", 7 @ 10", 3.09375", 0.5" OH
V2 = 1" OH, 7", 15 @ 9", 1" OH



WWR-W-1d

V1 X V2 D31/D20 76.09375" (0.5", 4") X 12'-0" (1, 1)
V1 = 0.5" OH, 3.09375", 7 @ 10", 3", 4" OH
V2 = 1" OH, 7", 15 @ 9", 1" OH

Figure 14: Wall Type 1−Wall Stem Mat WWR-W-1(c) and WWR-W-1(d) @ 12'-2" tall wall

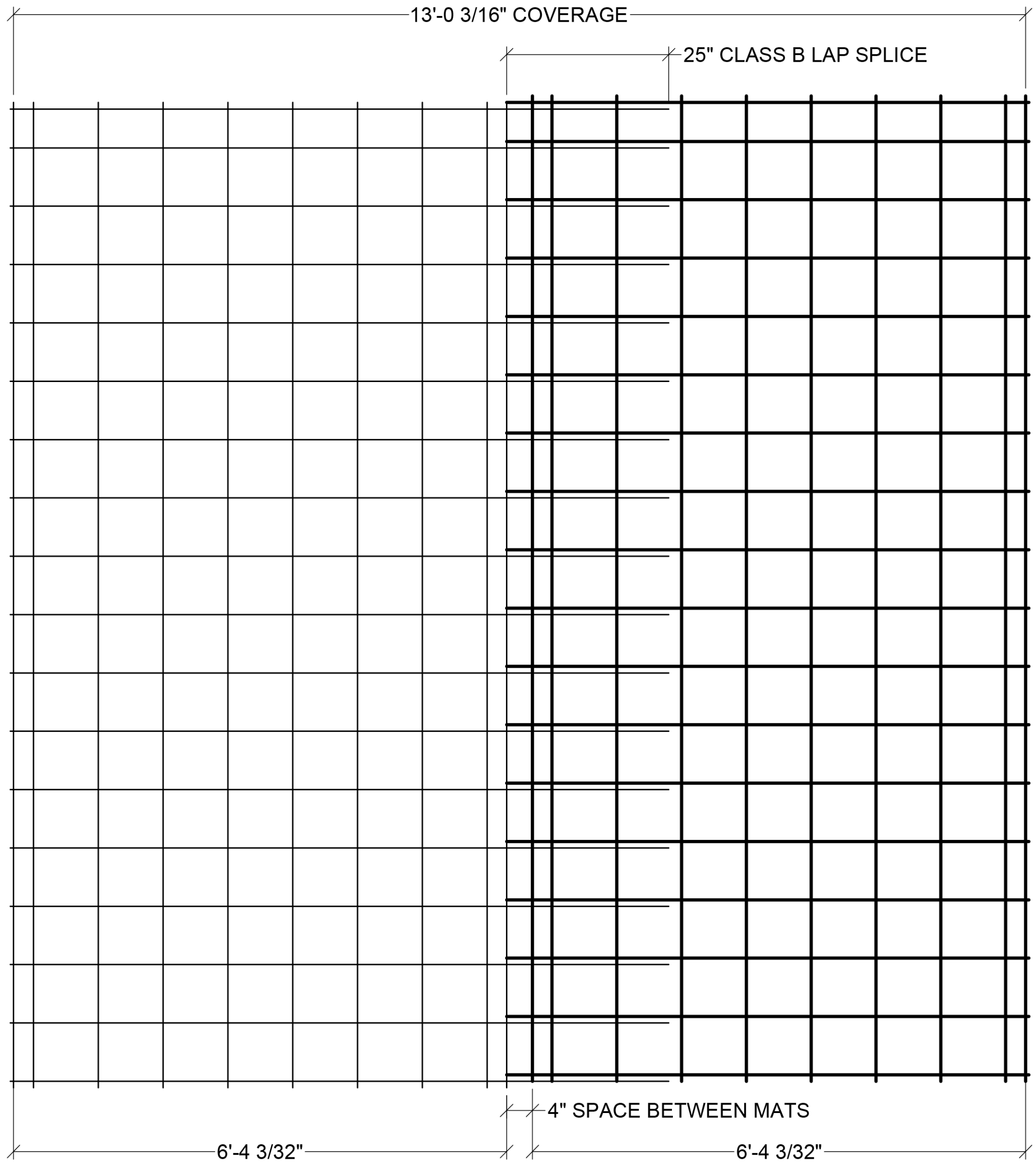


Figure 15: Wall Type 1−Wall Stem Mat coverage. Two mats per footing elevation, resulting in four (4) total mats for Wall Type 1. Mats WWR-W-1(a) and WWR-W-1(b) @ 12'-10" tall wall shown here.

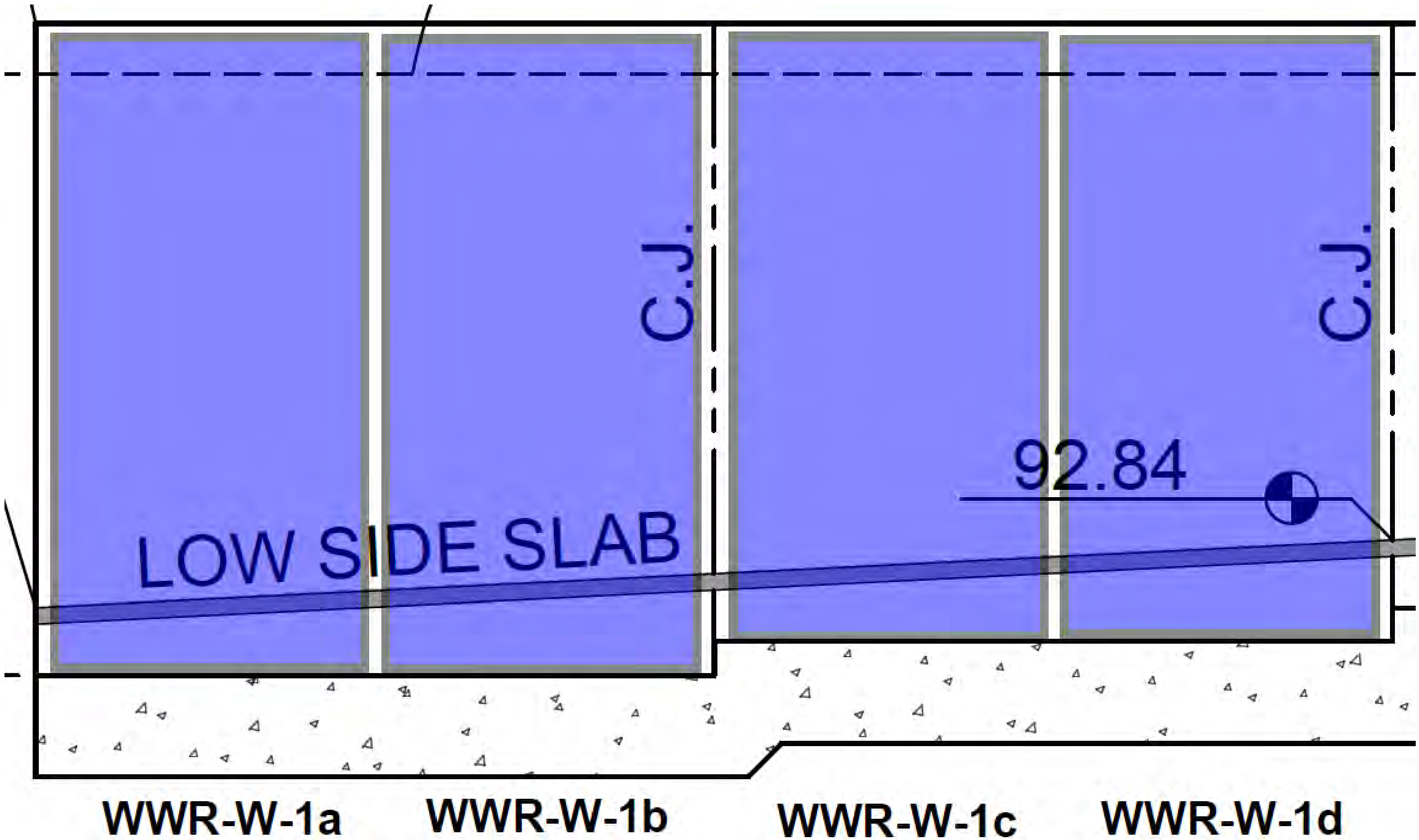
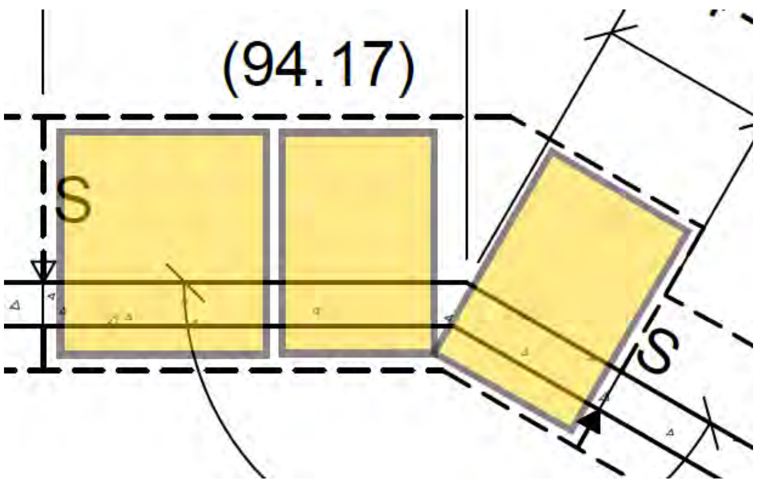
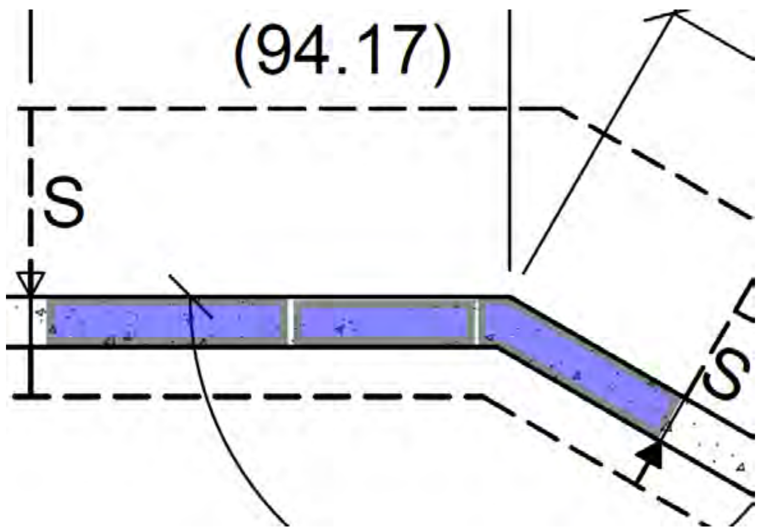


Figure 16: Wall Type 1−Superimposed placement of wall stem mats

ACI 318-19	Calculations	Description
STEP 3: Closing Notes – Other Wall Types		
	<u>Wall Type 2 and 4</u> The WWR takeoff and detailing procedure for these two wall type will be identical to that presented for Wall Type 1, with the resulting mat geometries of course varied slightly to suit the specific wall and footing configurations for the specific regions. **<u>Wall Type 3</u>** The left panel (when viewed in plan) will be handled in the same manner as has been presented for Wall Type 1. The right panel has a geometric variation in that it contains a wall intersection, with the wall (and associated footing) "kinked" at a 150-degree interior angle to suit the site layout. Footing mats will be interrupted at the wall turn, with individual transverse wires/bars oriented in a radial pattern within the resulting triangular gap. Individual longitudinal wires, bent to suit the wall/footing turn angle, would then be placed to lap with the mat longitudinals on each side of the gap.  	

ACI 318-19	Calculations	Description
STEP 3: Closing Notes – Other Wall Types (continued)		
	<u>Wall Type 3 (continued)</u> The dowel mat extents would be similar to those described for the footing mats, with loose bars/wire introduced in the gap region to ensure steel areas and maximum spacings per unit foot are maintained. The wall stem mats would be configured to capture the wall turn angle by way of bent horizontal wires to achieve lap through the turn. No additional loose bars/wires would be required. 	Welded wire reinforcement combined with localized transitional steel bars/wires positioned at footing steps and turns provides an efficient turnkey solution to retaining wall and foundation retaining wall configurations.

Chapter Nine

EXAMPLE: Mild Reinforcement in a Post-tensioned System

EXAMPLE 9 PROBLEM STATEMENT:

The subject structure is a parking garage with floor elements constructed primarily of post-tensioned reinforced concrete. The structure is characterized by one-way slab spanning perpendicular to long-spanning gravity beam members. The beam spans are oriented as required to traverse over both parking and drive aisles, while the spacing of the beams is selected as an interval of the parking spaces.

Lateral stability of the structure is achieved through the use of special moment frames, the columns and beams/girders of which are reinforced primarily with mild reinforcing bars. Moment frame beams/girders contain a nominal amount of post-tensioning positioned at the centroid of the member for the sole purpose of providing a magnitude of pre-compression that will promote improved durability and cracking control.

The structural contract drawings include both directly specified welded wire reinforcement configurations (*Direct Specification Method*) as well as permissive language for the use of welded wire reinforcement as a replacement to specified loose reinforcing bars (*Pre-Approved Equal Method*).

The structure is designed by the Engineer of Record per the requirements of ACI 318-19 with due consideration for longstanding benchmark post-tensioning design procedures and guidelines disseminated by the Post Tensioning Institute.

For this example, transverse reinforcement of gravity beams is directly specified by the Engineer of Record and will be subsequently detailed herein. Likewise, the Engineer of Record has directly noted a specific wire size option to be considered for the mild reinforcement of the one-way slab, and it too will be illustrated. Finally, this example will also show the welded wire reinforcement detailer’s proposed alternative for transverse reinforcement of a moment frame beam member.

The example includes the following steps:

Step 1 – Gravity Beam Transverse Reinforcement: Summary of Relevant Details
Step 2 – Gravity Beam Transverse Reinforcement: Determination of WWR Acceptance
Step 3 – Gravity Beam Transverse Reinforcement: WWR Detailing
Step 4 – One-Way Slab Mild Reinforcement: Summary of Relevant Details
Step 5 – One-Way Slab Mild Reinforcement: Determination of WWR Acceptance
Step 6 – One-Way Slab Mild Reinforcement: WWR Detailing
Step 7 – Moment Frame Beam Transverse Reinforcement: Summary of Relevant Details
Step 8 – Moment Frame Beam Transverse Reinforcement: Determination of WWR Acceptance
Step 9 – Moment Frame Beam Transverse Reinforcement: WWR Detailing

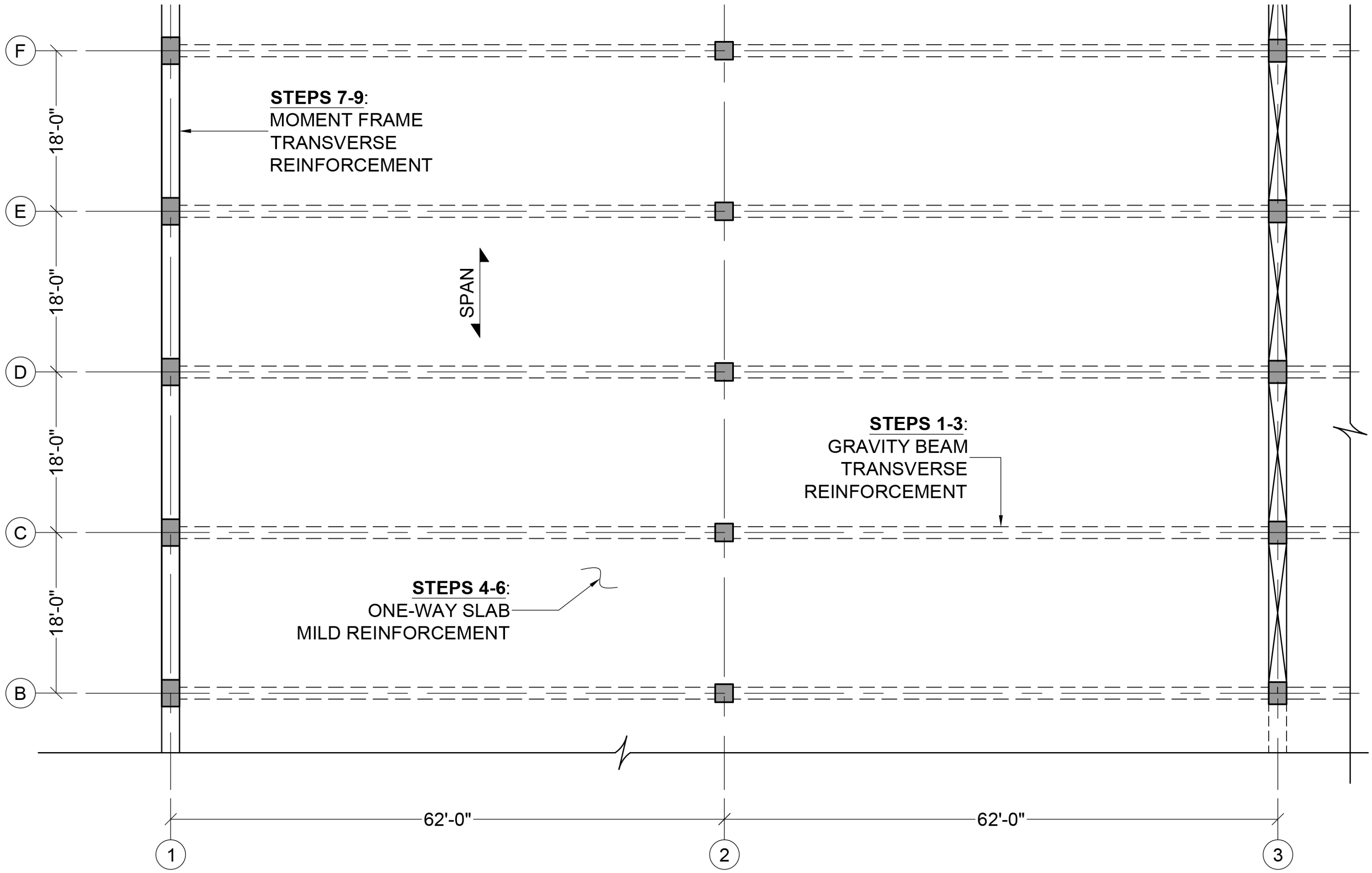


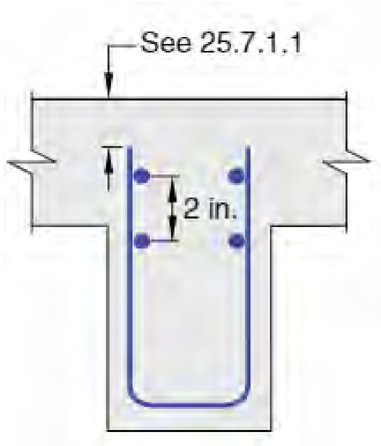
Figure 1: Excerpt from contract structural drawings to be used for construction

Placing crew setting beam stirrup cages in post-tensioned beam. Notice the slab reinforcement mats already in position, comprised of prefabricated WWR mats containing staggered wires per the design professional's detailed requirements.

<table>
<tr><th>ACI 318-19</th><th>Calculations</th><th>Description</th></tr>
<tr><td colspan="3">STEP 1: Gravity Beam Transverse Reinforcement: Summary of Relevant Details</td></tr>
<tr><td>25.7.1.1
25.7.1.4</td><td>

Anchorage of each leg of welded wire reinforcement forming a single U-stirrup shall be in accordance with (a) or (b):

(a) Two longitudinal wires spaced at a 2-inch spacing along the member at the top of the U.



(b) One longitudinal wire located not more than d/4 from the compression face and a second wire closer to the compression face and spaced not less than 2 inches from the first wire.

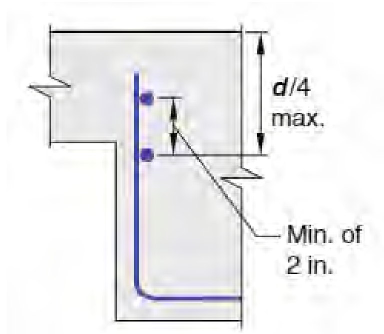


Image Credit: American Concrete Institute

</td><td>

The Structural Engineer of Record has directly specified the use of welded wire reinforcement for shear stirrups located in the gravity beams. The basis for this application is found in ACI 318. For this project, the engineer has detailed the stirrup per provision (b).

The premise for the WWR U-stirrup used in a gravity beam is the anchorage of the stirrup’s vertical shear legs via machine-welded horizontal “anchor” wires positioned at the upper extents of the stirrup. This configuration effectively eliminates the need for hooks. Provided the engineer’s design intent is satisfied on a case-by-case basis, this elimination of hooks can also manifest in a spatial advantage that helps to make the placement of post-tensioning tendons and PT support bars a less cumbersome operation.

Figure 2 and Figure 3 show the geometric requirements for the stirrups, as well as the arrangement of stirrups along the beam length required to satisfy shear demand.

Figure 4 is an excerpt from the contract documents showing both prescriptive and permissive language for WWR.

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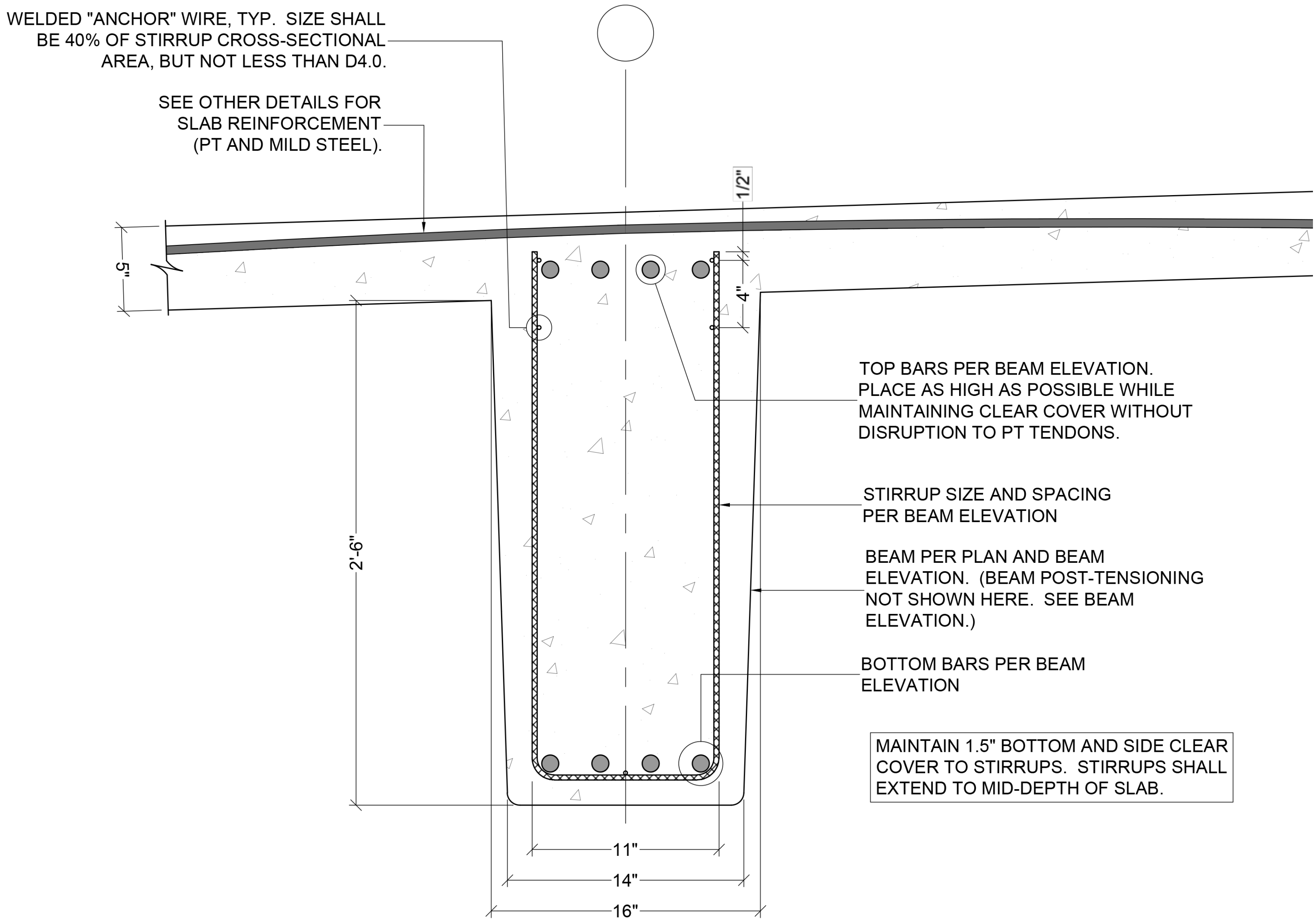


Figure 2: Engineer's typical detail of gravity beam cross-section, showing stirrup geometry and anchor wires

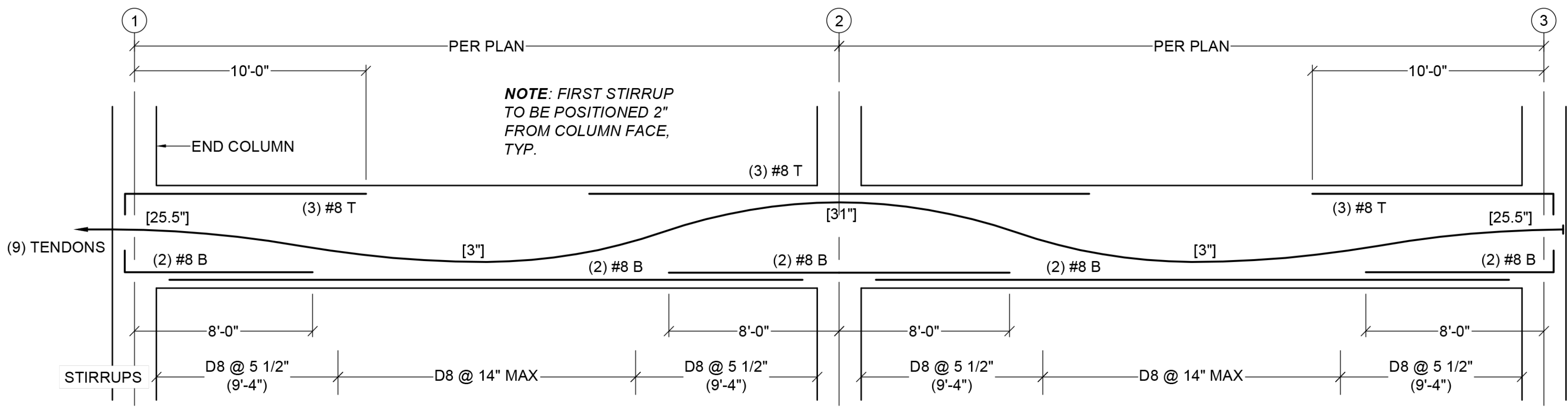


Figure 3: Engineer's beam elevation detail showing post-tensioning and mild steel reinforcement

MILD REINFORCING STEEL

1. TYPICAL DEFORMED REINFORCING BARS (REBAR) SHALL CONFORM TO ASTM A615, GRADE 80. BARS SHALL BE LAPPED IN ACCORDANCE WITH THE REBAR LAP SCHEDULES UNLESS OTHERWISE EXPLICITLY DETAILED.
2. LONGITUDINAL REINFORCEMENT IN SPECIAL MOMENT FRAME BEAMS AND COLUMNS, AND VERTICAL AND HORIZONTAL REINFORCEMENT IN SPECIAL STRUCTURAL (SHEAR) WALLS SHALL BE ASTM A706 GRADE 60 OR GRADE 80 AS NOTED. TENSILE AND ELONGATION PROPERTIES SHALL BE CONFIRMED THROUGH MILL REPORT DOCUMENTATION PROVIDED AS PART OF THE PROJECT REINFORCEMENT SUBMITTAL.
3. WELDED DEFORMED WIRE REINFORCEMENT SHALL CONFORM TO ASTM A1064 GRADE 80 AND SHALL BE PROVIDED IN SHEET FORM. REINFORCEMENT SHEETS SHALL BE MANUFACTURED WITH OVERHANG LENGTHS SUFFICIENT TO ACHIEVE A LAP SPLICE LENGTH EQUAL TO THE GREATER OF 12 INCHES OR THE LAP SPLICE DIMENSION SHOWN IN THE REBAR LAP SCHEDULE FOR BAR OF EQUAL (OR GREATER) DIAMETER AND GRADE, UNLESS OTHERWISE NOTED. SHEETS AND ASSOCIATED LAP REGIONS SHALL BE INSTALLED COPLANAR SO AS TO NOT "STACK".
4. WELDED DEFORMED WIRE REINFORCEMENT OF EQUAL AREA, EQUAL OR LESSER SPACING, AND IDENTICAL CURTAILMENT (HOOKS AND LAP SPLICES) IS PERMITTED AS A SUBSTITUTION FOR DEFORMED REINFORCING BARS, EXCEPT IN THE FOLLOWING STRUCTURAL APPLICATIONS:
   A. LONGITUDINAL STEEL IN SPECIAL MOMENT FRAMES
   B. VERTICAL AND HORIZONTAL STEEL IN SPECIAL STRUCTURAL WALLS
5. UNLESS OTHERWISE NOTED ON THE DRAWINGS OR IN THE PROJECT SPECIFICATIONS

Figure 4: Excerpt from contract documents showing the design professional's permissive WWR substitution language (Note #4, red rectangle) and prescriptive language (Note #3).

ACI 318-19	Calculations	Description
STEP 2: Gravity Beam Transverse Reinforcement: Determination of WWR Acceptance		
20.2.1.7 20.2.2.4		Deformed wire sizes between D4 and D31 are permitted, and WWR is permitted for use in stirrup applications as previously illustrated. This combined with the fact that the Engineer of Record has explicitly defined acceptable WWR sizing and configuration represents a compliant condition. The WWR detailer would proceed accordingly.

ACI 318-19	Calculations	Description
STEP 3: Gravity Beam Transverse Reinforcement: WWR Detailing		
20.2.1.7 20.2.2.4		Based on the engineer's defined geometry and reinforcement extents, the WWR Detailer generates two layout options as shown in Figure 5. As can be seen in Option #1, bent WWR mats are configured with strict adherence to the engineer's specified spacing of 5 ½" in the end regions and 14" in the central region. The extents of the end region stirrups are identical to that defined by the engineer, leaving the balance of the beam length to be populated with stirrups spaced not to exceed 14". As such, and because the balance of the beam length is not an even increment of 14", the WWR detailer configures the central mats (WWR-2) to be offset from the end region mats (WWR-1) by an intermediate space equal to 11 ½". The engineer's requirements are satisfied (intermediate space dimension does not exceed 14"), but the reinforcement placement crew will need to ensure that the atypical intermediate field spacing is maintained. Option #2 represents the WWR Detailer's effort to minimize placement crew bookkeeping. End region mat (WWR-1) stirrup spacing and extents are identical to Option #1, but the configuration of the central mats (WWR-2) is adjusted so that all intermediate field spacings between WWR mats are maintained at 14". This is achieved by configuring WWR-2 with a variable wire spacing comprised predominantly of 14" wire spacing, with 8 ½" closing spacings at each end of the mat. **We will proceed with Option #2.**

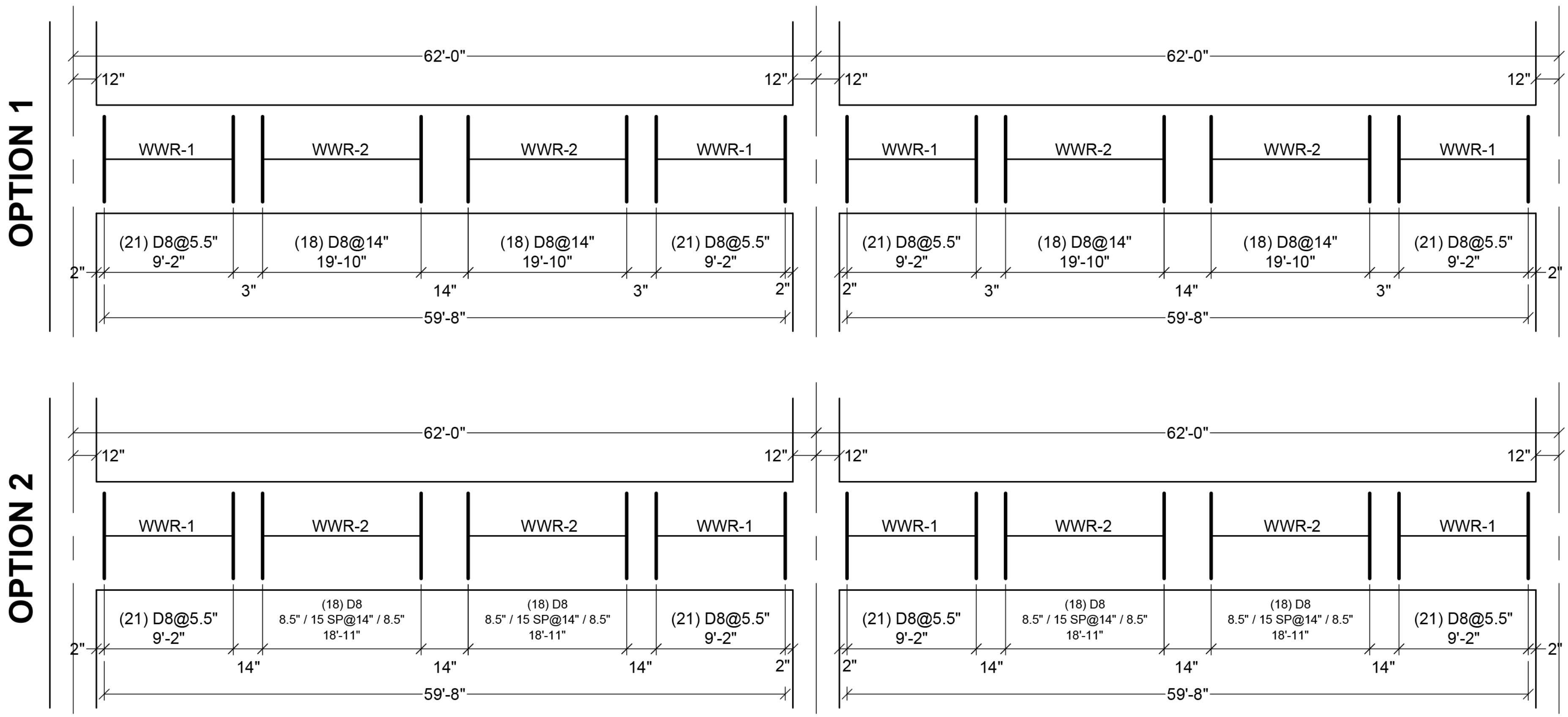
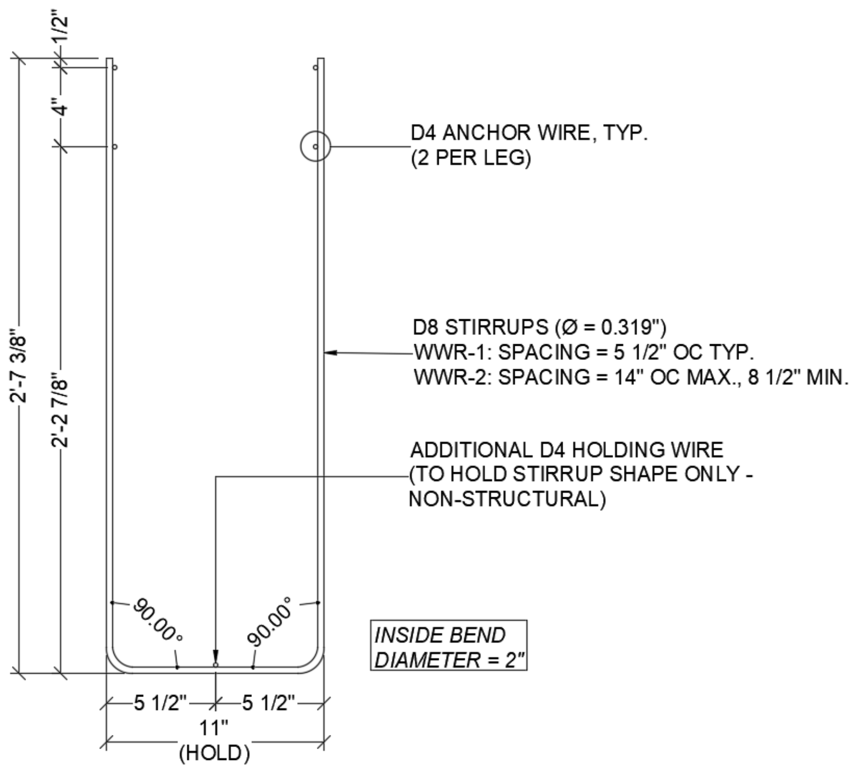


Figure 5: WWR Detailer develops and considers two stirrup layout options based on contract drawing information.

ACI 318-19	Calculations	Description
STEP 3: Gravity Beam Transverse Reinforcement: WWR Detailing (continued)		
25.3.3	 **WWR-1:** V1 X 5.5 D4 X D8 72.25" (+0.5",+0.5") X 9'-4" (1,1) V1 = 0.5" OH, 4", 2 @ 31.625", 4", 0.5" OH **WWR-2:** V1 X V2 D4 X D8 72.25" (+0.5",+0.5") X 19'-1" (1,1) V1 = 0.5" OH, 4", 2 @ 31.625", 4", 0.5" OH V2 = 1" OH, 8.5", 15 @ 14", 8.5", 1" OH	Resulting fabrication information for Option #2 mats WWR-1 and WWR-2 are illustrated here. Note that the ACI requirement dictating a minimum bend diameter equal to $4d_b$ is satisfied. Also observe that that there are no welded wire intersections within $4d_b$ of the bent portion of the wire.

ACI 318-19	Calculations	Description
STEP 4: One-Way Slab Mild Reinforcement: Summary of Relevant Details		
7.6.2.3	$A_{s,min} \geq 0.004 A_{ct}$ $For\ a\ 5"\ thick\ slab, A_{ct} = 2.5 \times 12 = 30\ in^2\ per\ unit\ foot\ width$ $\therefore\ A_{s,min} = 0.004 \times 30 = 0.12\ in^2\ per\ unit\ foot\ width$ $\therefore$ **#4 *bars* @ 20 on center or D20 bars at 20" on center.**	The Structural Engineer of Record has indicated on the contract documents that mild reinforcement of the one-way slab is permitted to be either reinforcing bars or wires, and the size and spacing is defined explicitly. For slabs with unbonded tendons, there is a prescriptive minimum area of bonded deformed longitudinal reinforcement defined by ACI 318-19. Refer to Figure 6 for the engineer's slab reinforcement detail to be used by the WWR Detailer as the basis for takeoff.

ACI 318-19	Calculations	Description
STEP 5: One-Way Slab Mild Reinforcement: Determination of WWR Acceptance		
20.2.1.7 20.2.2.4		Deformed wire sizes between D4 and D31 are permitted. This combined with the fact that the Engineer of Record has explicitly defined a WWR option for the slab's bonded mild reinforcement represents a compliant condition. The WWR detailer would proceed accordingly.

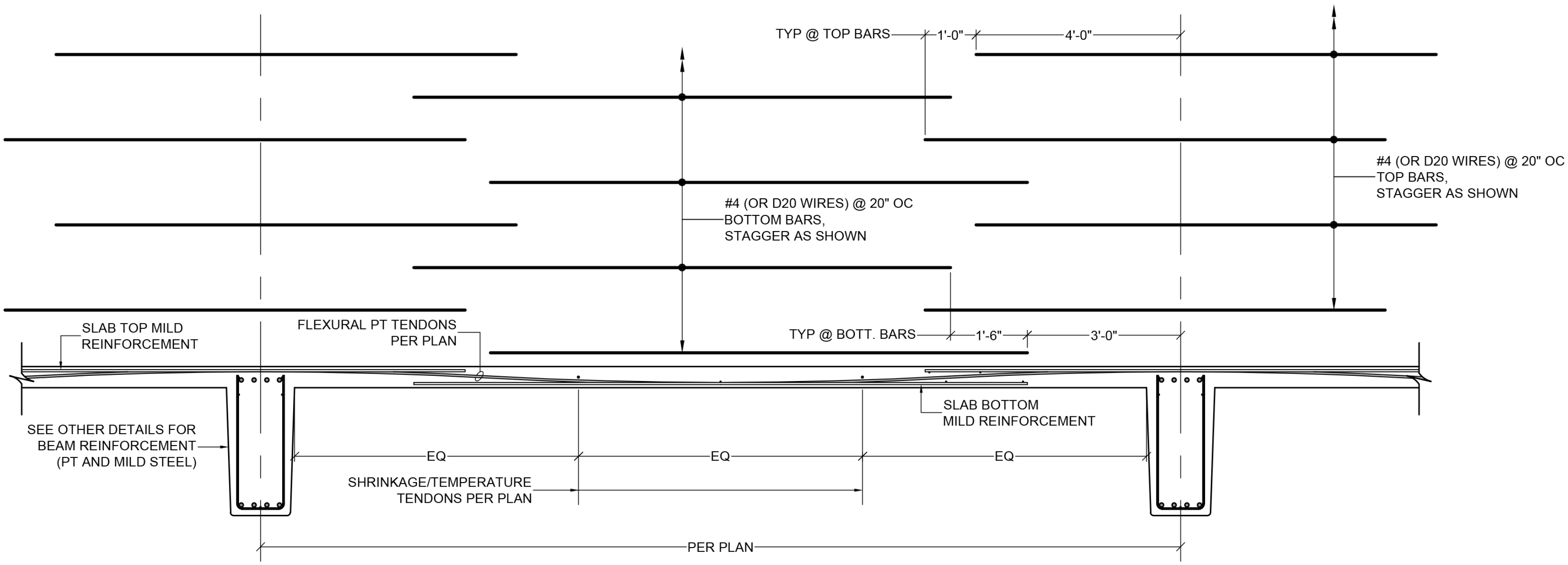


Figure 6: Engineer's slab section view showing typical reinforcement requirements, with projected partial plan view of top and bottom mild reinforcement

ACI 318-19	Calculations	Description
STEP 6: One-Way Slab Mild Reinforcement: WWR Detailing		
20.2.1.7 20.2.2.4		Based on the engineer's defined geometry and reinforcement extents, the WWR Detailer will generate mat configurations for the slab top and bottom mild bonded steel. It is important to note that post-tensioned reinforced concrete systems require the WWR Detailer's close attention to avoidance of the engineer's defined PT tendon profile (drape) and positioning. A change in vertical profile not only results in an altered flexural design strength but can have significant detrimental effects on balance load magnitude and service stresses. These detrimental effects are magnified in relatively thin post-tensioned elements such as the 5-inch slab used in this particular design. Typical construction sequence is for slab bottom reinforcing to be placed before the flexural post-tensioning tendons, with shrinkage/temperature tendons (at mid-depth of slab) then being installed prior to the slab top reinforcing. It is critical that the WWR mats be configured so that holding wires oriented perpendicular to the flexural post-tensioning tendons do not create spatial obstruction to the tendon drape itself. While there is always a possibility that the non-structural holding wires could be field-cut in order to keep the path of post-tensioning tendons clear, it is still the WWR Detailer's responsibility during takeoff to recognize the priority placed on post-tensioning tendon profile, and this acknowledgement must manifest in WWR mat configurations that are spatially compatible. See Figure 7 through Figure 10.

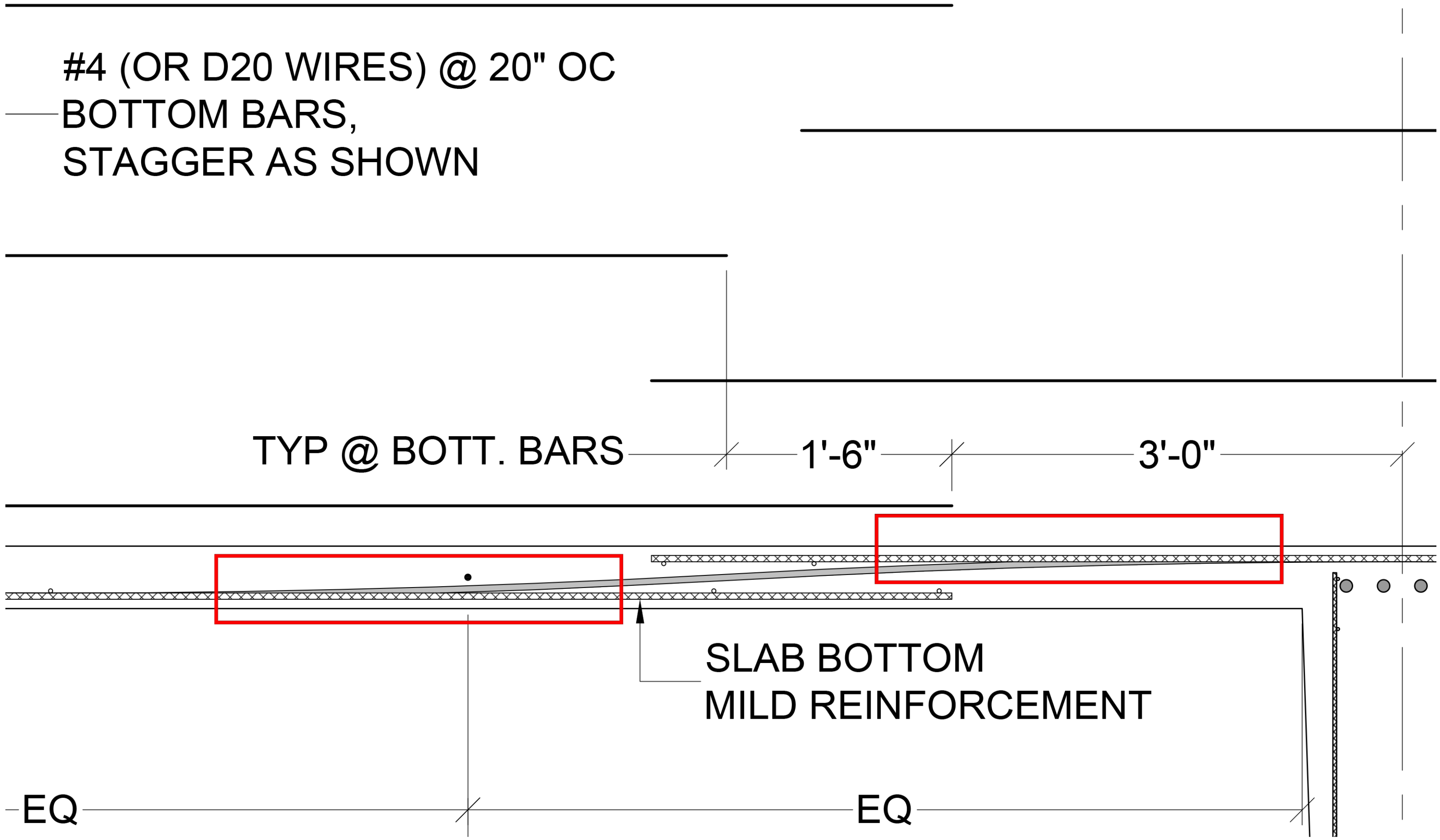
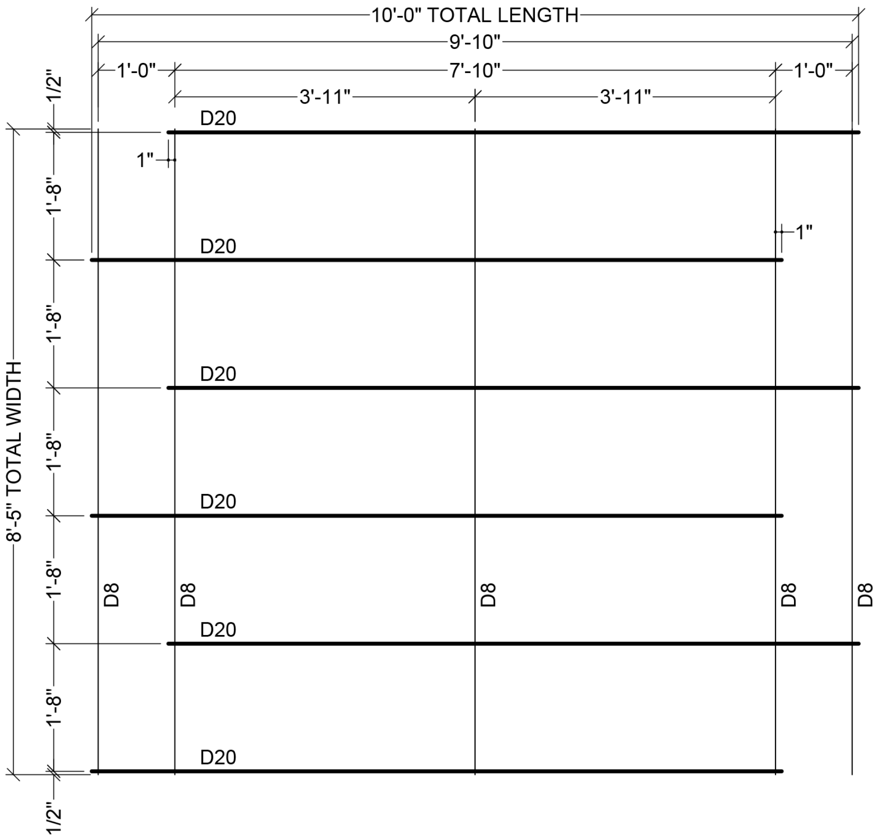


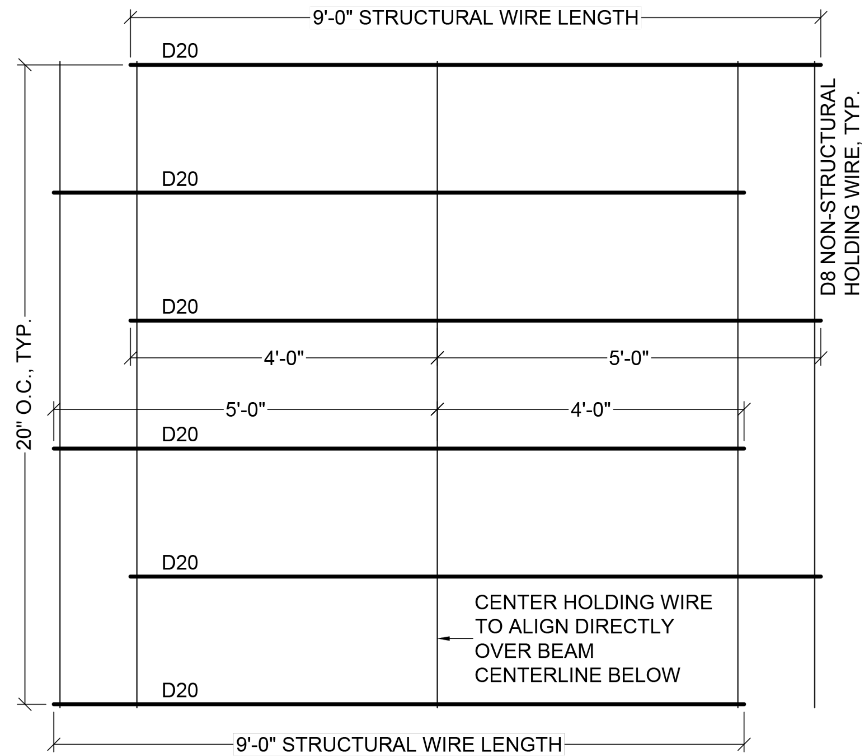
***Figure 7:** Shown in red are potential trouble spots where conflict between WWR non-structural holding wire and post-tensioning tendon could occur. The structural wires themselves are parallel to the flexural post-tensioning tendons and are not a concern. Non-structural holding wires, however, are oriented perpendicular to the flexural post-tensioning tendons, so it is important that they be positioned to avoid causing an abrupt jump or drop in tendon profile.*



WWR-T1
20 X V2 D20/D8 101" (+0.5",+0.5") X 10'-0" (1,1)
V2 = 1" OH, 12", 2 @ 47", 12", 1" OH

MANUFACTURING INFORMATION

NECESSARY FOR PRODUCTION BY PLANT PERSONNEL

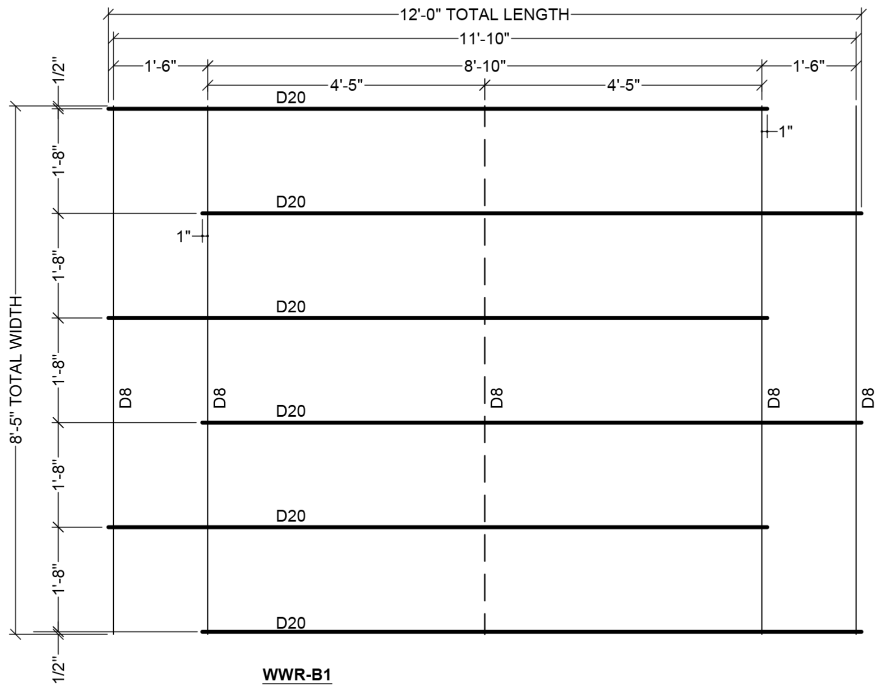


WWR-T1 IS SLAB TOP REINFORCEMENT. INSTALL MAT WITH D20 STRUCTURAL WIRES CLOSEST TO TOP OF SLAB AND D8 NON-STRUCTURAL WIRES BELOW. CENTER HOLDING WIRE SHALL RUN DIRECTLY BELOW HIGHEST POINT OF POST-TENSIONED TENDON PROFILE OVER BEAM CENTERLINE.

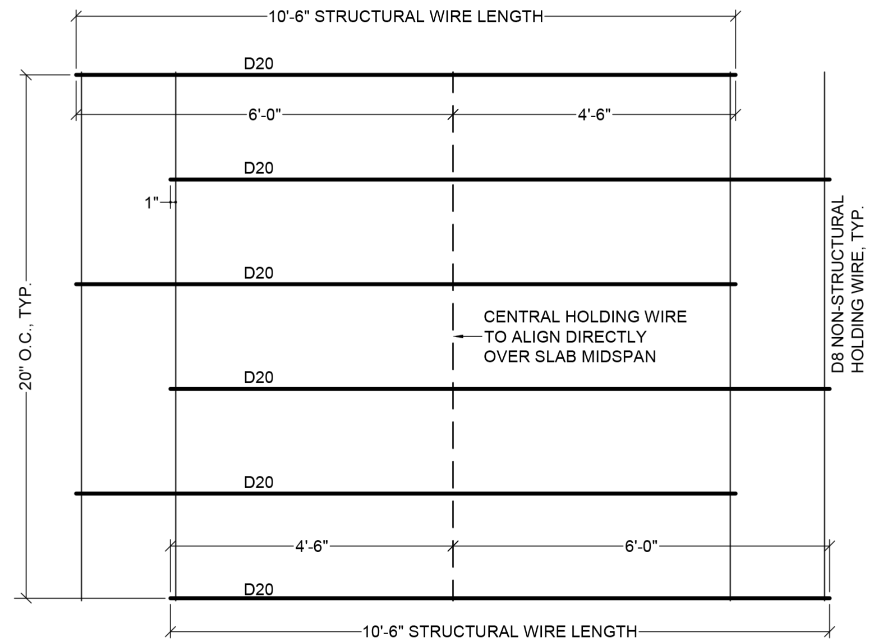
SUBMITTAL INFORMATION

FOR DESIGN INTENT REVIEW BY ENGINEER

Figure 8: Information prepared by the WWR Detailer for the slab top reinforcement is shown here. On the left is the manufacturing information necessary for production, while the illustration on the right is a distilled figure that presents reinforcement size and geometry that is relevant to the Engineer of Record's submittal review. Oftentimes both sets of information are compiled on the submittal as a manner of streamlining, and the submittal document itself can then in turn double as the fabrication drawing.



WWR-B1
20 X V2 D20/D8 101" (+0.5",+0.5") X 12'-0" (1,1)
V2 = 1" OH, 18", 2 @ 53", 18", 1" OH



WWR-B1 IS SLAB BOTTOM REINFORCEMENT. INSTALL MAT WITH D20 STRUCTURAL WIRES CLOSEST TO BOTTOM OF SLAB AND D8 NON-STRUCTURAL WIRES ABOVE. CENTRAL HOLDING WIRE SHALL RUN DIRECTLY ABOVE LOWEST POINT OF POST-TENSIONED TENDON PROFILE AT SLAB MIDSPAN. FIELD CUT NON-STRUCTURAL WIRES AS REQUIRED ONCE MAT IS INSTALLED AND POSITIONED TO AVOID INTERRUPTION OF POST-TENSIONING TENDONS.

MANUFACTURING INFORMATION

NECESSARY FOR PRODUCTION BY PLANT PERSONNEL

SUBMITTAL INFORMATION

FOR DESIGN INTENT REVIEW BY ENGINEER

Figure 9: Information prepared by the WWR Detailer for the slab bottom reinforcement is shown here. On the left is the manufacturing information necessary for production, while the illustration on the right is a distilled figure that presents reinforcement size and geometry that is relevant to the Engineer of Record's submittal review. Oftentimes both sets of information are compiled on the submittal as a manner of streamlining, and the submittal document itself can then in turn double as the fabrication drawing. Note the permissive language to field cut non-structural holding wires to avoid conflict with prioritized post-tensioning drape. This would be a consideration depending on the sequencing placement of mats versus post-tensioning.

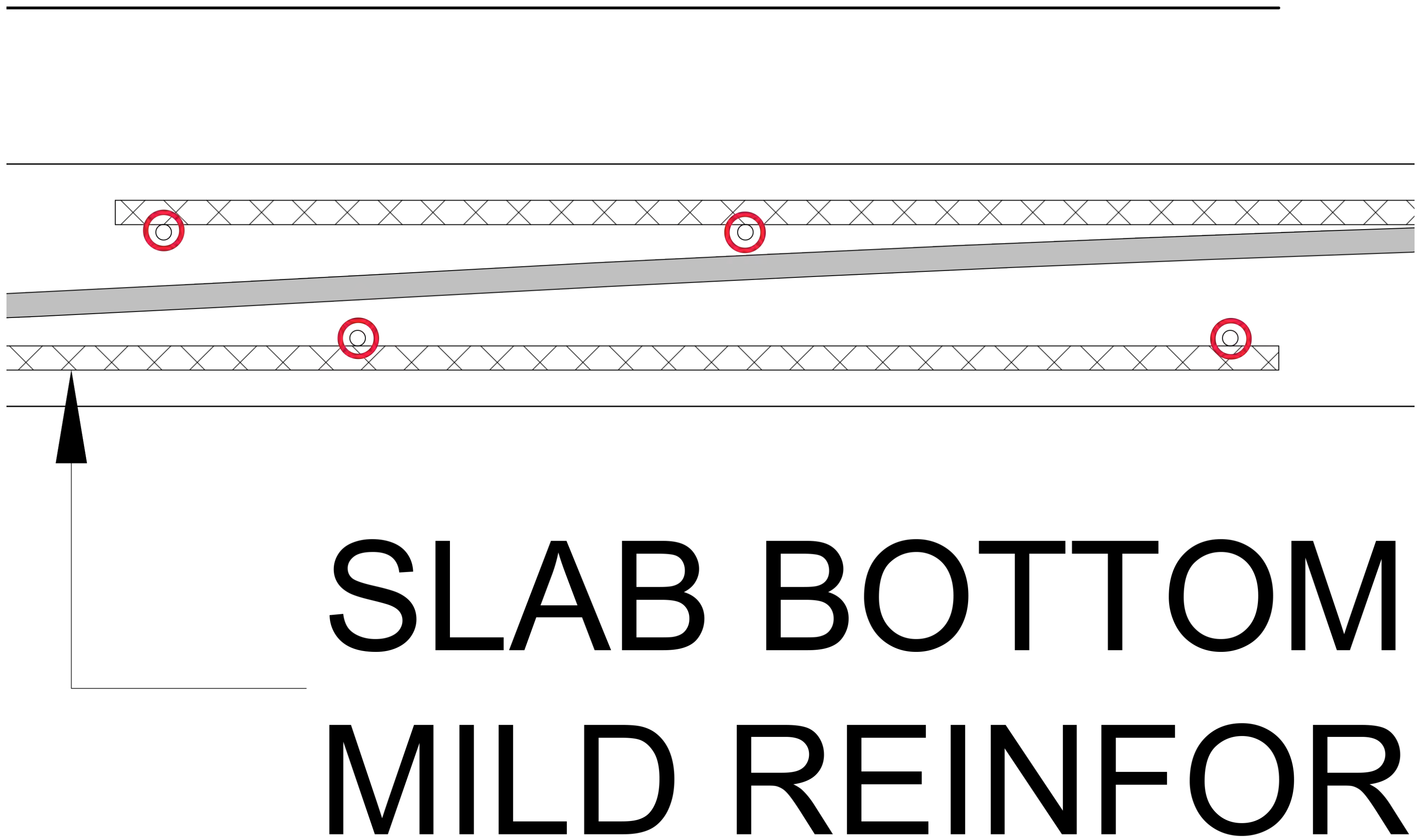


Figure 10: Shown in red are the locations of non-structural holding wires, superimposed on an enlarged excerpt of the previous slab reinforcement drawing from Figure 6 and 7. The WWR Detailer has configured the mats such that these non-structural holding wires do not conflict with the engineer's defined post-tensioning tendon profile.

ACI 318-19	Calculations	Description
STEP 7: Moment Frame Beam Transverse Reinforcement: Summary of Relevant Details		
		The Structural Engineer of Record is showing reinforcing bars for the moment frame beam transverse reinforcement along Gridline 1. The moment frame beam detail is shown in Figure 11. As designed, the engineer does not show welded wire reinforcement as an option. However, as reflected in the previously illustrated Figure 4, the engineer has permissive language for welded wire reinforcement substitutions provided the reinforcing bars being replaced are not serving as (a) longitudinal reinforcement in special moment frames or (b) reinforcement in special structural walls. With this in mind, there is a potential value substitution that could be proposed by the contractor for the moment frame beam transverse reinforcement. Steps 8 and 9 will show this effort.

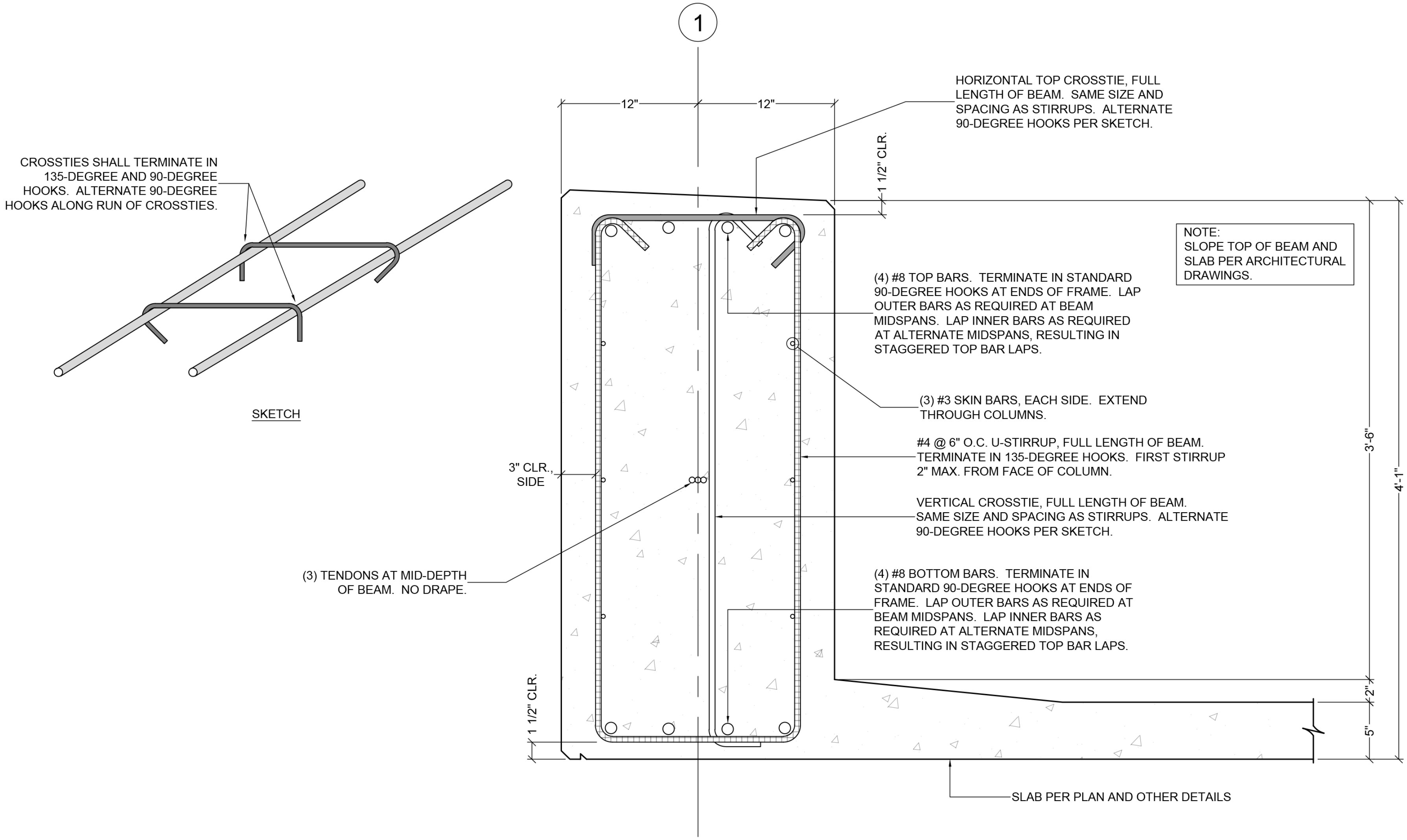


Figure 11: Contract Document section through typical moment frame, showing longitudinal and horizontal mild reinforcement.

ACI 318-19	Calculations	Description
STEP 8: Moment Frame Beam Transverse Reinforcement: Determination of WWR Acceptance		
20.2.1.7 20.2.2.4 25.3.3 25.3.4 18.6.4.4 18.6.4.5 18.7.5		Deformed wire sizes between D4 and D31 are permitted. Welded wire reinforcement is permitted for use in special moment frames (special seismic systems) as lateral support of longitudinal bars, concrete confinement, and shear, and torsion. This permission is predicated on the requirement that the welds are not relied upon for resistance to stresses. In other words, while the welds themselves can be present within the member, they must not be considered structural in nature or relied upon for development or curtailment. The most notable offshoot of this requirement is that the straight-leg welded “anchorage” wire solution shown in Steps 1 through 3 is prohibited. Hooked curtailment of transverse reinforcement in special moment frames must be utilized. With this in mind, there are three potential WWR substitutions that could be made for the moment frame beams represented in Figure 11: (a) U-stirrups with 135-degree hooks (b) Top crosstie with 135- and 90-degree hooks (c) Vertical crosstie with 135- and 90-degree hooks It is possible to manufacture all three configurations. The U-stirrups will be fully detailed herein. For (b) and (c), the bending operation itself will be highlighted to show how the alternated hook pattern can be achieved on a WWR mat.

ACI 318-19	Calculations	Description
STEP 9: Moment Frame Beam Transverse Reinforcement: WWR Detailing		
		Figures 12 through 14 show information relevant to the WWR detailing of the potential substitutions. Note that for contract documents such as those referenced in this example that contain permissive WWR language (i.e., language that constitutes WWR as a “pre-approved equal” in certain applications), the submittal of a request For Information (RFI) to the design professional seeking permission to substitute WWR for reinforcing bars is typically not necessary as long as the WWR submittal itself is thorough and the proposed substitutions are presented and illustrated in a transparent manner. A method of maximizing this transparency is through substitution documentation that would accompany the reinforcement submittal issued for designer review. An example of substitution documentation is shown in Chapter 5, Figure 5 and Figure 8. Because the engineer in Example 9 includes both permissive language and explicitly-detailed WWR applications, however, as a courtesy it would be appropriate to issue an RFI for proposed substitutions that have not been explicitly-detailed. In all “pre-approved equal” cases, prior to communications with the design professional, the WWR manufacturer and the contractor must be in full agreement on those reinforcement applications that are to be pursued as WWR.

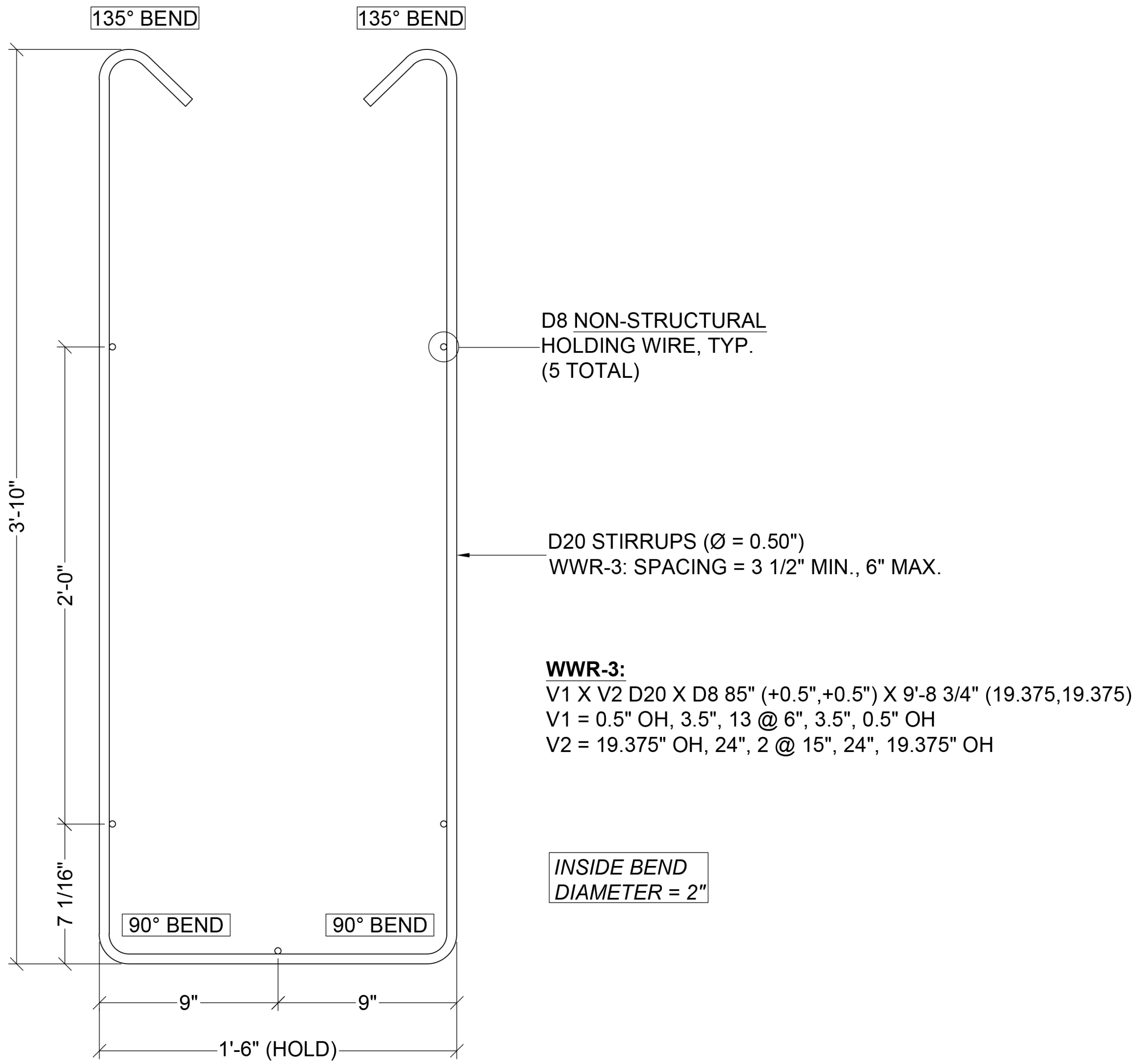


Figure 12: WWR Detailer's bend drawing for the moment frame stirrup is shown here. Notice that there is a subtle adjustment to the endmost stirrup spaces of the WWR cage in order to provide cage "modules" that populate the moment frame beam length without the field workers needing to keep track of odd spaces between cages. See also Figure 13.

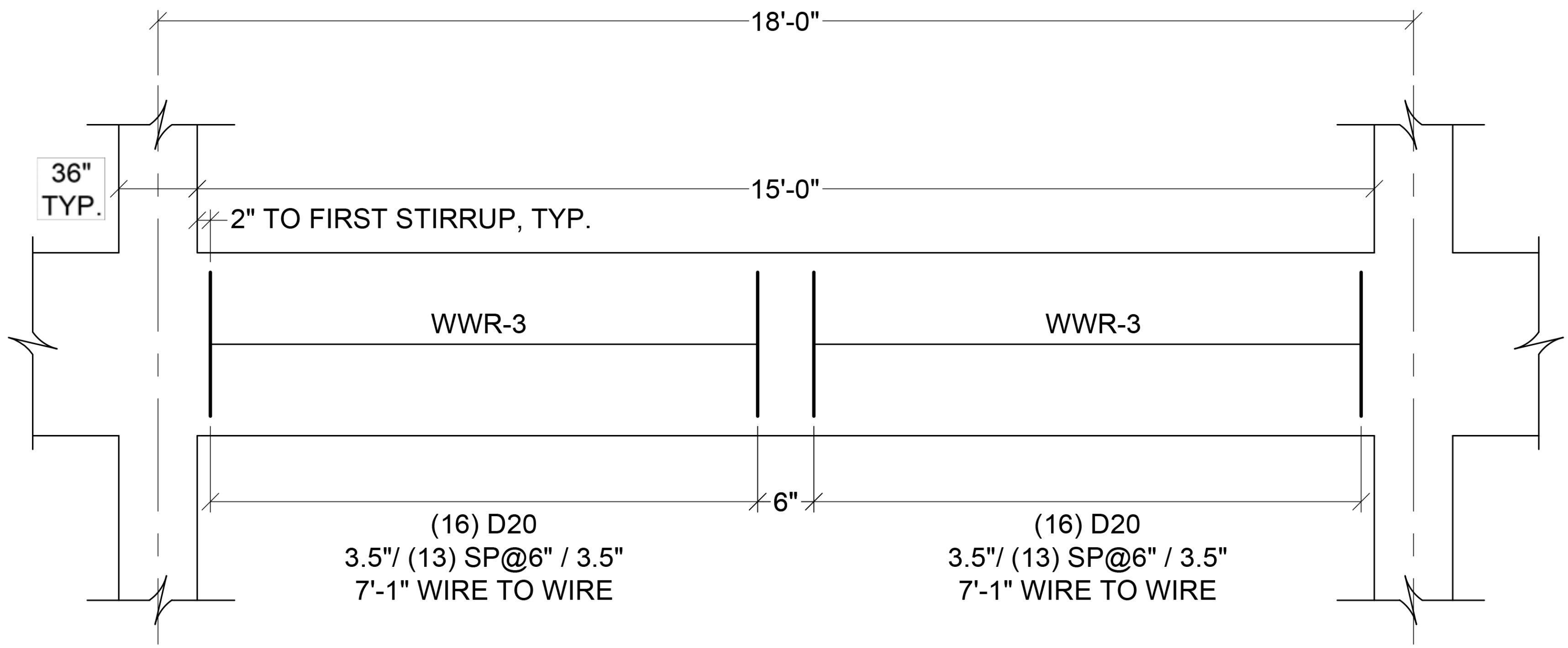


Figure 13: WWR Detailer's stirrup layout view for the typical moment frame beam is shown here. Two WWR-3 stirrup cages are required per beam length, with a field space equal to 6 inches between the cages.

ACI 318-19	Calculations	Description
STEP 9: Moment Frame Beam Transverse Reinforcement: WWR Detailing (continued)		
18.7.5.2(c) 25.3.3		Providing crosstie runs with alternating hooks is a requirement specific to beams of special moment frames in earthquake-resistant structure design, and represents an attribute requiring the WWR Detailer's close attention from a fabrication standpoint more so than from the actual detailing standpoint. Figure 14 provides a schematic representation of how WWR mats with alternating hooks can be achieved on a WWR line bending machine. The finished WWR product will satisfy the alternating hooks requirement, and the crosstie wires will be spaced to match the previously configured stirrup wires on the WWR-3 cage.

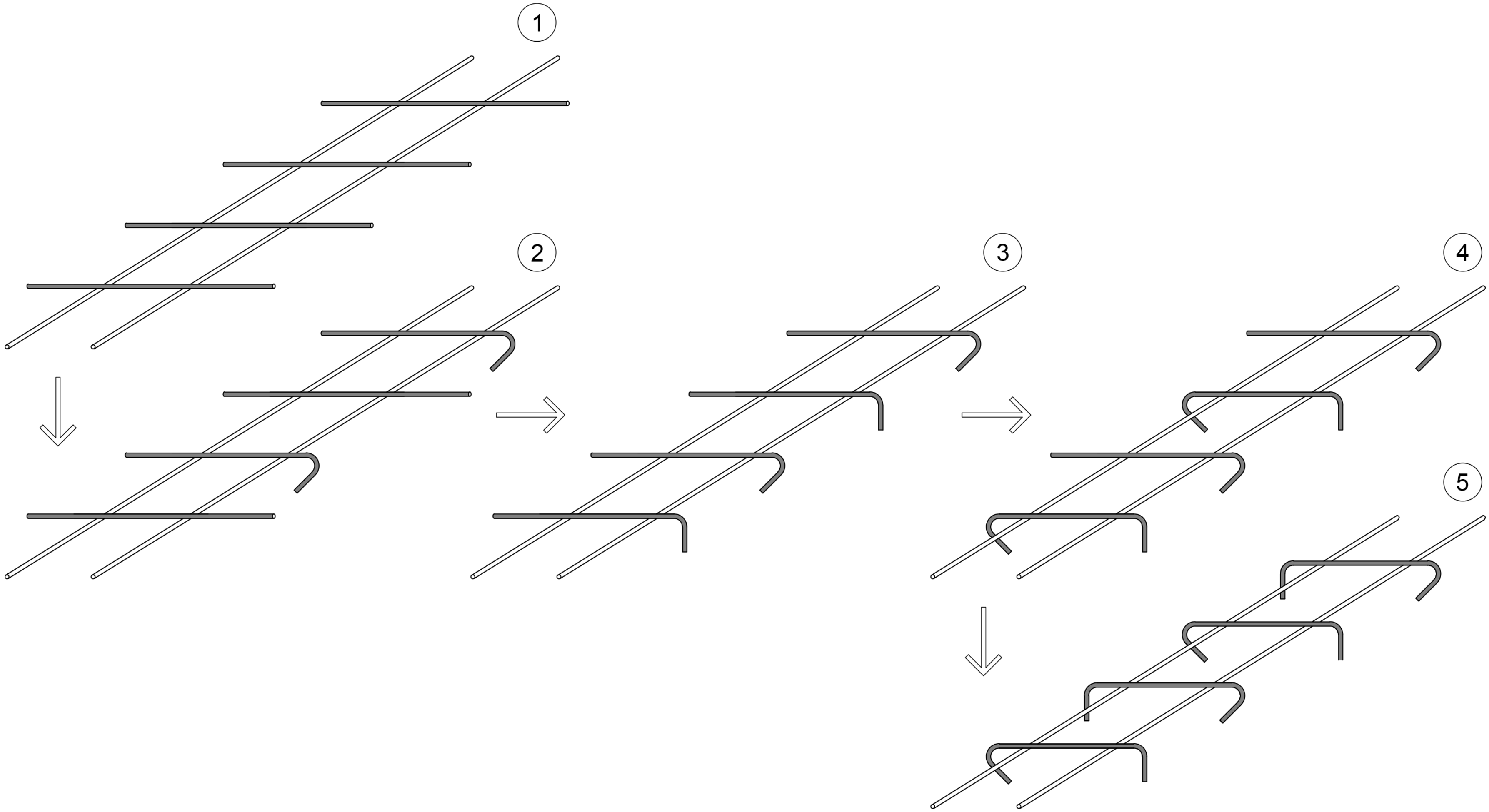


Figure 14: Schematic of sequential bending on a WWR line bender to achieve alternating hooks.

1. ***The bending crew starts with the flat WWR mat, straight from the WWR welding machine.***
2. ***The hydraulic (or pneumatic) line bender is configured such that its mandrels (bending pins) are spaced at twice the typical crosstie spacing. Crossties aligning with the mandrels along the right side are then bent to 135-degree hooks, resulting in every other crosstie wire being bent.***
3. ***The bending crew then shifts the entire mat down the line bender a distance equal to that of the typical stirrup spacing, resulting in the unbent right-side wires now being aligned and saddled into the mandrels. These wires are then bent to 90-degrees, completing the bending operation for the right side.***
4. ***The WWR mat is removed from the bender, rotated end for end, and re-inserted into the line bender. The process noted in Step #2 is then carried out for the left side.***
5. ***The process noted in Step #3 is carried out for the left side, completing the bending operation for the WWR crosstie mat.***

Chapter Ten

EXAMPLE: Conventional Two-Way Mild Reinforced Slab

EXAMPLE 10 PROBLEM STATEMENT:

The subject structural configuration is comprised of a two-way slab with non-prestressed (mild) reinforcement. The system is a flat plate, absent of exterior and interior beams between supports. The lateral forces resisting system is comprised of reinforced concrete shear walls, collector, and chords, the designs of which are beyond the scope of this example.

The slab is to be designed using the Direct Design Method (DDM). While the DDM procedure is no longer explicitly presented in ACI 318-19, it can be found in ACI 318-14 Section 8.10 amongst numerous other reinforced concrete design texts. As such this example contains references to ACI 318-14 where applicable.

The slab is to be characterized by a flat soffit with no drop panels or shear caps. The designer is tasked with gravity design of the reinforced concrete slab. The designer elects to waive the requirements of ACI 318-19 Section 20.2.1.7.3 (see "Common Designer Questions" in Chapter Two of this guide).

Design Criteria are as follows:

$$f'_c = 5{,}000\ psi$$
$$f_y = 80{,}000\ psi$$
$$Concrete\ Density\ (including\ reinforcement) = 0.150\ kcf$$
$$Superimposed\ Uniform\ Dead\ Load, q_{DL} = 0.010\ ksf$$
$$Superimposed\ Uniform\ Live\ Load, q_{LL} = 0.080\ ksf\ (assume\ nonreducible)$$

The example includes the following steps:

Step 1 – Suitability of the Direct Design Method
Step 2 – Slab Thickness Determination
Step 3 – Applied Loading
Step 4 – Defining Design Strips
Step 5 – Preliminary Two-Way (Punching) Shear
Step 6a – DDM – Gridline G Total Factored Static Moments
Step 6b – DDM – Gridline G Moment Distribution to CS and MS
Step 6c – DDM – Gridline G Slab Negative Moment Transfer
Step 7 – Two-Way (Punching) Shear - Revisited
Step 8 – Two-Way Shear Reinforcement Design / Strength Check
Step 9 – One-Way Shear Check
Step 10 –Gridline G Labeled Moments
Step 11 (1C-5C) – Gridline G Column Strip Reinforcement
Step 12 – Gridline G Middle Strip Reinforcement
Step 13 – WWR Detailing for Gridline G Reinforcement

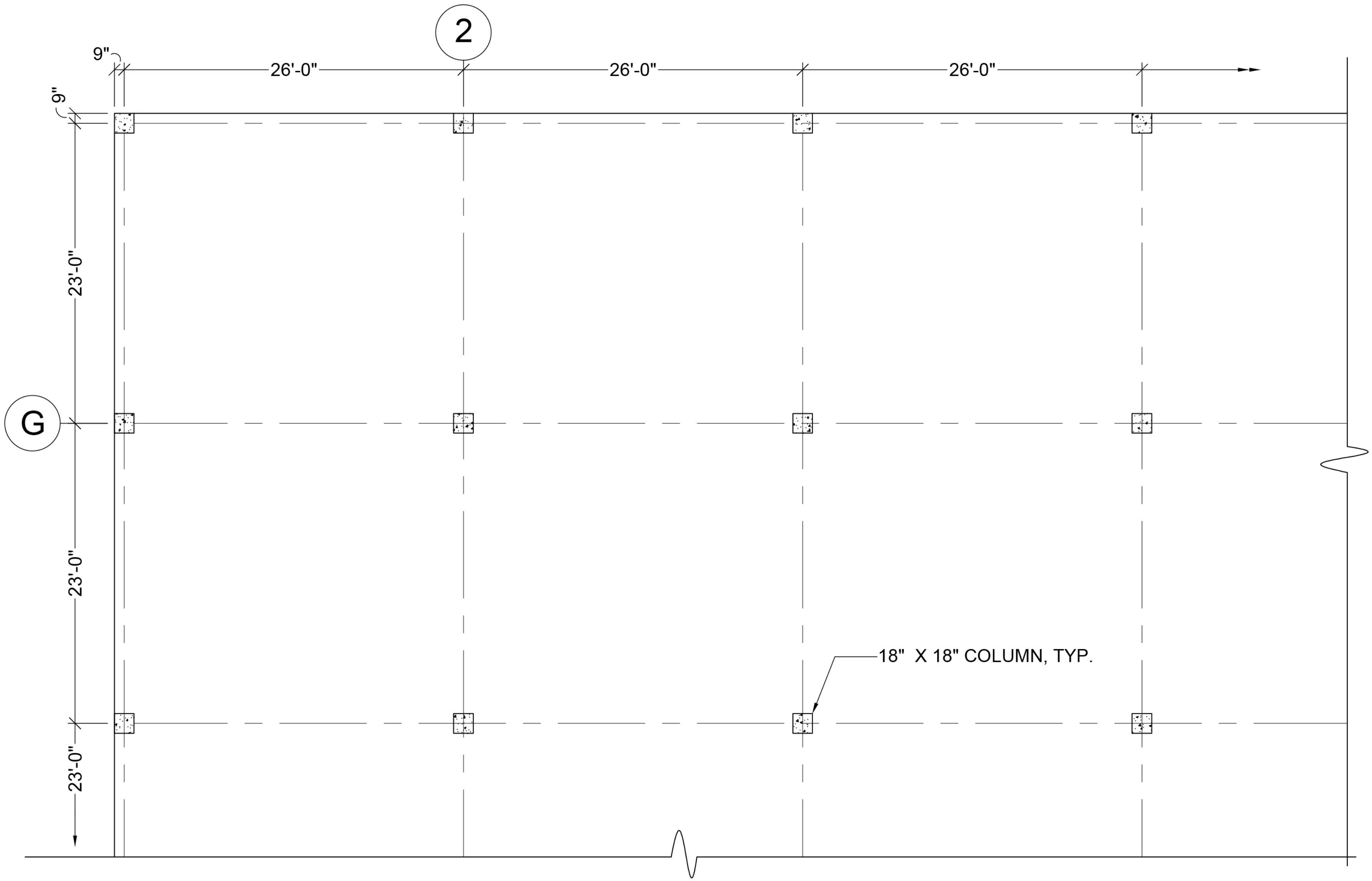


Figure 1: Partial Floor Plan

ACI 318-19	Calculations	Description
STEP 1: Suitability of Direct Design Method		
8.2.1		Direct Design Method (DDM) is permitted.
8.10.2.1 (ACI 318-14)		There are at least three spans in each direction.
8.10.2.2 (ACI 318-14)		Successive span lengths measured center-to-center of supports do not differ by more than one-third the longer span.
8.10.2.3 (ACI 318-14)	$\frac{26\ feet}{23\ feet} = 1.13 < 2.0$	Panels are rectangular, with ratio of longer to shorter panel dimensions, measured center-to-center of support, not exceeding 2.0.
8.10.2.4 (ACI 318-14)		Per ACI, column offsets shall not exceed 10%. Here, we have no column offsets.
8.10.2.5 (ACI 318-14)		All loads are due to gravity only and distributed over an entire panel.
8.10.2.6 (ACI 318-14)	$0.080\ ksf < 2 \times (0.125\ ksf + 0.010\ ksf)\ assuming\ 10"\ slab$ $0.080\ ksf < 0.270\ ksf\ \rightarrow OK$	Unfactored live load does not exceed two times the unfactored dead load.
8.10.2.7 (ACI 318-14)		There are no beam supports.
	$\therefore\ Direct\ Design\ Method\ is\ suitable\ for\ use.$	

ACI 318-19	Calculations	Description
STEP 2: Slab Thickness Determination		
8.3.1 8.3.1.1 Table 8.3.1.1 8.3.1.1(a)	• $f_y = 80{,}000\ psi$ • $No\ drop\ panels$ • $l_n\ is\ the\ clear\ span\ in\ the\ long\ direction, face\ to\ face\ of\ supports$ $l_n = 312" - 9" - 9" = 294" = 24.50\ ft$ $\underline{Exterior\ Panel\ minimum\ slab\ thickness, without\ edge\ beams}$: $l_n/27 = (24.50 \times 12)/27 = 10.88\ inches\ \leftarrow$ $\underline{Interior\ Panel\ minimum\ slab\ thickness, without\ edge\ beams}$: $l_n/30 = (24.50 \times 12)/30 = 9.8\ inches$ $\underline{minimum\ thickness\ of\ slab\ without\ drop\ panels = 5\ inches}$ $\therefore use\ slab\ thickness\ equal\ to\ 11\ inches$	We will use ACI minimum slab thickness provisions. Design will be based on selection of a slab thickness that, based on prescriptive tabulated minimum thicknesses that have been developed through the years, will preclude the need for detailed deflection calculations. Note that there is no floor finish being placed monolithically with the structural slab thickness, as such Section 8.3.1.3 does not apply.

ACI 318-19	Calculations	Description
STEP 3: Applied Loading		
5.3.1 Table 5.3.1 8.4.1.9	$U = 1.2D + 1.6L$ $q_u = 1.2(0.148) + 1.6(0.080) = 0.306\ ksf$	The governing load combination is 1.2D + 1.6L. Dead and live loads are applied uniformly on all slab panels as established in 8.10.2.6 (ACI 318-14). With 8.10.2.6 (ACI 318-14) satisfied, there will be no patterned live loads. All building lateral loading is resisted by shear walls; therefore, the slab does not contribute to the out-of-plane flexural resistance of the building structure and is assumed to only participate as a diaphragm. Consideration for additional slab reinforcement related to chord and collector requirements is beyond the scope of this example.

ACI 318-19	Calculations	Description
STEP 4: Defining Design Strips		
8.4.1.7		A slab panel is bounded by column, beam, or wall centerlines on all sides. Flexural demand will ultimately be distributed to the slab in both directions depending on the aspect ratio of the panel, with further distribution contingent upon proximity to the analytical column lines.
8.4.1.5 8.4.1.6	***Gridline G Direction*** $l_1 = 26.00\ feet$ $l_2 = 23.00\ feet$ $CS\ width\ to\ each\ side\ of\ column = 0.25(lesser\ of\ l_1\ and\ l_2)$ $CS\ total\ width = 2 \times (0.25 \times 23.00') = 11.50\ feet$ $MS\ half\ width\ to\ north = (23.00' - 5.75' - 5.75')/2 = 5.75\ feet$ $MS\ half\ width\ to\ south = (23.00' - 5.75' - 5.75')/2 = 5.75\ feet$ $\therefore$ *Refer to Figure 2 for illustrative representation of design strip G* ***Gridline 2 Direction*** $l_1 = 23.00\ feet$ $l_2 = 26.00\ feet$ $CS\ width\ to\ each\ side\ of\ column = 0.25(lesser\ of\ l_1\ and\ l_2)$ $CS\ total\ width = 2 \times (0.25 \times 23.00') = 11.50\ feet$ $MS\ half\ width\ to\ west = (26.00' - 5.75' - 5.75')/2 = 7.25\ feet$ $MS\ half\ width\ to\ south = (26.00' - 5.75' - 5.75')/2 = 7.25\ feet$ $\therefore$ *Refer to Figure 3 for illustrative representation of design strip 2*	Column strips are sub-design strips with a prescriptive width on each side of a column centerline, while middle strips are sub-design strips bounded by two column strips. For the purposes of the example, then, an interior design strip is comprised of a column strip plus two half-middle strips, while a perimeter design strip consists of a column strip plus one half-middle strip. The design and detailing of Gridline 2, while similar to that shown in subsequent steps of this example for Gridline G, is not carried out here.

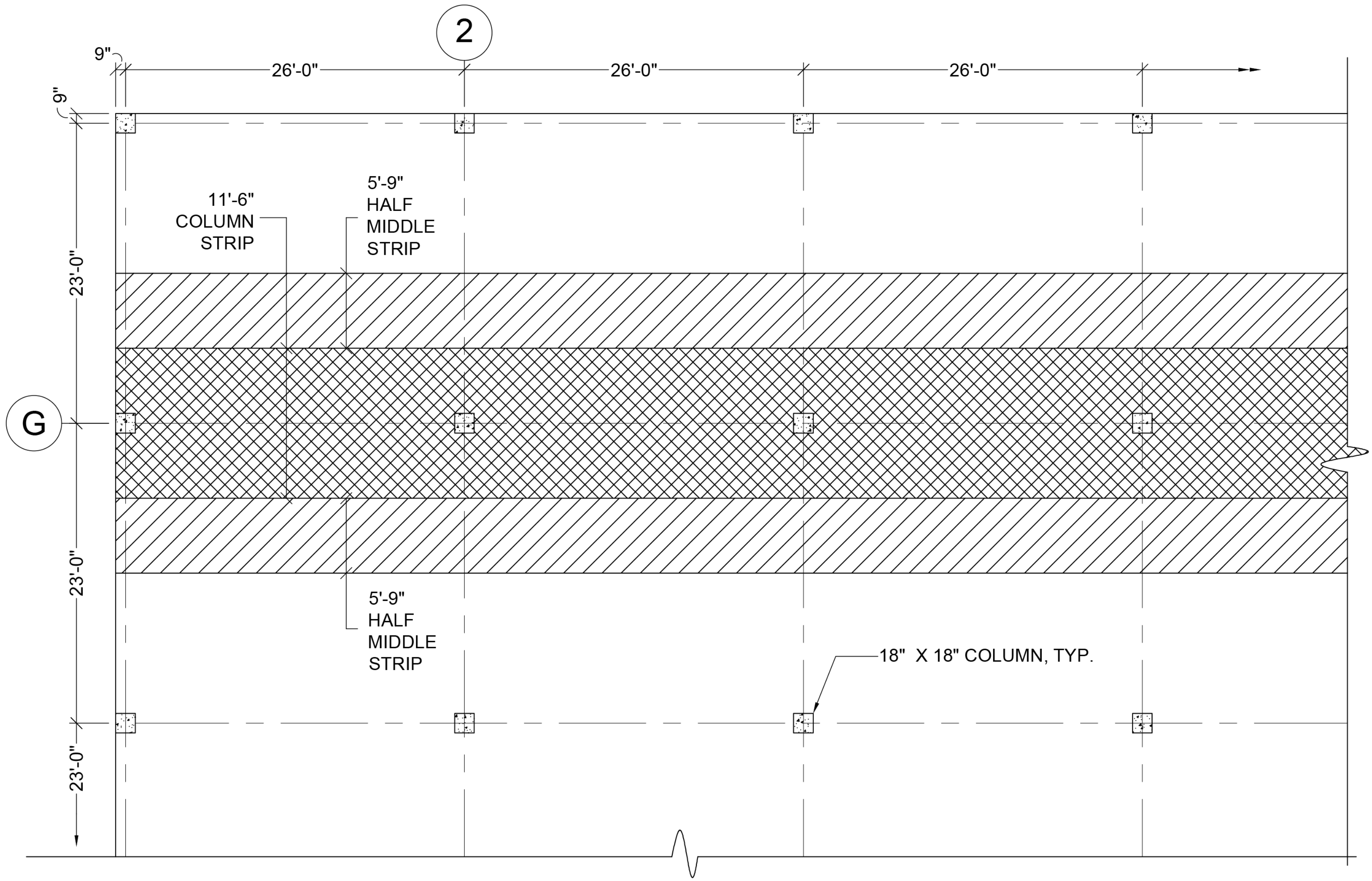


Figure 2: Design Strip G

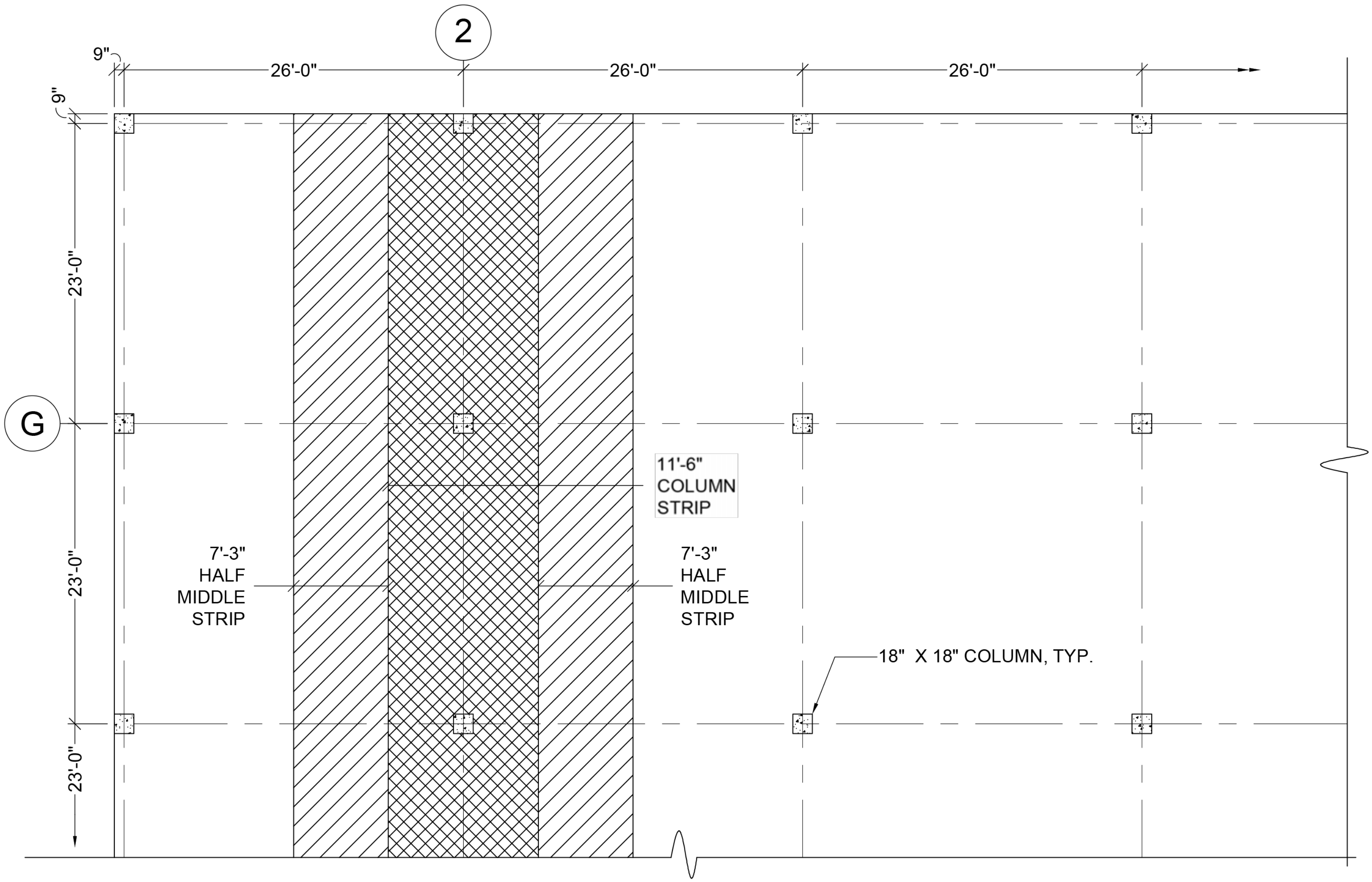


Figure 3: Design Strip 2

ACI 318-19	Calculations	Description
STEP 5: Preliminary Two-Way (Punching) Shear		
8.5.3.1.2 22.6 22.6.2.1 22.6.4.1	$v_{uv} = \frac{V_u}{b_o d}$ • $Assume$ 1" clear cover and two (2) 5/8" $diameter\ bar$ "$mats$" • $Approx.\ effective\ depth\ d$ = 11" − 1" cover - 0.625" = 9.375" ***Interior Column*** $b_o = 4 \times (18" + 9.375/2 + 9.375/2) = 109.5"$ $Loaded\ tributary\ area = 23\ ft \times 26\ ft - \frac{27.375" \times 27.375"}{144} = 593\ sf$ $V_u = 0.306\ ksf \times 593\ sf = 182\ kips$ ***Exterior Column*** $b_o = (18" + 9.375"/2 + 9.375"/2) + 2 \times (18" + 9.375"/2) = 72.75"$ $Loaded\ tributary\ area = 23\ ft \times 13\ ft - \frac{27.375" \times 22.688"}{144} = 295\ sf$ $V_u = 0.306\ ksf \times 295\ sf = 91\ kips$ ***Corner Column*** $b_o = 2 \times (18" + 9.375"/2) = 45.375"$ $Loaded\ tributary\ area = 11.5\ ft \times 13\ ft - \frac{22.688" \times 22.688"}{144} = 146\ sf$ $V_u = 0.306\ ksf \times 146\ sf = 45\ kips$	Carry out a preliminary check of punching shear to ensure slab depth proportioning is appropriate for the loading. This check is based solely on v_{uv}, the *factored shear stress on the slab critical section for two-way action*, from the 1.2D + 1.6L load combination, without moment transfer. The cumulative effect of moment transfer will be fully quantified in a subsequent calculation step. While a favorable result here does not guarantee that the slab alone, without shear reinforcement, will be satisfactory to resist punching shear, it would serve as reasonable confirmation that the introduction of drop panels or a thickening of the slab will be unnecessary.

ACI 318-19	Calculations	Description
STEP 5: Preliminary Two-Way (Punching) Shear - continued		
22.6.1.2	$v_n = v_c$	Preliminarily assume no shear reinforcement in determination of nominal shear strength.
22.5.5.1.3	$\lambda_s = \sqrt{\dfrac{2}{1+\dfrac{9.375"}{10}}} = 1.016 > 1.0 \therefore use\ 1.0$	Determine size effect modification factor.
22.6.5.2 22.6.5.3	$v_c\ is\ the\ least\ of:$ $v_c = 4\lambda_s\lambda\sqrt{f'_c}$ $Interior\ column: v_c = 0.283\ ksi \quad \leftarrow governs$ $Exterior\ column: v_c = 0.283\ ksi \quad \leftarrow governs$ $Corner\ column: v_c = 0.283\ ksi \quad \leftarrow governs$ $v_c = \left(2+\dfrac{4}{\beta}\right)\lambda_s\lambda\sqrt{f'_c}$ $\beta\ is\ the\ ratio\ of\ column\ sides.\ \beta = 1\ for\ square\ column.$ $Interior\ column: v_c = 0.424\ ksi$ $Exterior\ column: v_c = 0.424\ ksi$ $Corner\ column: v_c = 0.424\ ksi$ $v_c = \left(2+\dfrac{\alpha_s d}{b_o}\right)\lambda_s\lambda\sqrt{f'_c}$ $\alpha_s = 40\ for\ interior\ columns, 30\ for\ exterior\ columns,$ $and\ 20\ for\ corner\ columns$ $Interior\ column: v_c = 0.384\ ksi$ $Exterior\ column: v_c = 0.415\ ksi$ $Corner\ column: v_c = 0.434\ ksi$	Refer to table 22.6.5.2 for v_c for two-way members without shear reinforcement.

ACI 318-19	Calculations	Description
STEP 5: Preliminary Two-Way (Punching) Shear - continued		
22.6.1.2 22.6.1.3 21.2.1	$v_{uv} = \frac{V_u}{b_o d}$ $\boldsymbol{Interior\ Column}$ $v_{uv} = \frac{182\ kips}{109.5" \times 9.375"} = 0.177\ ksi$ $\varphi v_n = 0.75 \times 0.283\ ksi = 0.212\ ksi \rightarrow OK$ $\boldsymbol{Exterior\ Column}$ $v_{uv} = \frac{91\ kips}{72.75" \times 9.375"} = 0.134\ ksi$ $\varphi v_n = 0.75 \times 0.283\ ksi = 0.212\ ksi \rightarrow OK$ $\boldsymbol{Corner\ Column}$ $v_{uv} = \frac{45\ kips}{45.375" \times 9.375"} = 0.106\ ksi$ $\varphi v_n = 0.75 \times 0.283\ ksi = 0.212\ ksi \rightarrow OK$ $\therefore$ *Preliminary punching shear check OK. Proceed with flexural design.*	The strength reduction (φ) factor for shear is 0.75. Based on "direct" shear only, the results look favorable. We will proceed with original assumptions for a flat plate slab.

ACI 318-19	Calculations	Description
STEP 6a: Direct Design Method – Gridline G Total Factored Static Moments		
8.10.3.1 (ACI 318-14) 8.10.3.2 (ACI 318-14) Table 8.10.4.2 (ACI 318-14) 8.10.3.2 (ACI 318-14) 8.10.4.1 (ACI 318-14)	$\boldsymbol{End\ Span - Full\ Width\ Design\ Strip\ Moments}$ $M_{o,G\ End} = \frac{q_u l_2 {l_n}^2}{8} = \frac{0.306 \times 23\ ft \times (24.5^2)}{8} = 529\ kip - ft$ $Distribute\ M_o\ as\ follows:$ <table><tr><td></td><td>Slabs without beams between interior supports, without edge beams</td></tr><tr><td>Interior Negative</td><td>0.70</td></tr><tr><td>Positive</td><td>0.52</td></tr><tr><td>Exterior Negative</td><td>0.26</td></tr></table> $M_{u,ext\ (-)} = 0.26 \times 529 = 138\ kip - ft$ $M_{u,int\ (-)} = 0.70 \times 529 = 371\ kip - ft \leftarrow governs\ over\ interior\ span$ $M_{u\ (+)} = 0.52 \times 529 = 275\ kip - ft$ $\boldsymbol{Interior\ Span - Full\ Width\ Design\ Strip\ Moments}$ $M_{o,G\ Int} = \frac{q_u l_2 {l_n}^2}{8} = \frac{0.306 \times 23\ ft \times (24.5^2)}{8} = 529\ kip - ft$ $Distribute\ M_o\ as\ follows:$ $M_{u\ (-)} = 0.65 \times 529 = 344\ kip - ft$ $M_{u\ (+)} = 0.35 \times 529 = 186\ kip - ft$ $Refer\ to\ Fig. 4\ for\ illustrated\ Gridline\ G\ full\ design\ strip\ moments.$	Calculate the total factored static moment for a span with strip bounded laterally by the panel centerline on each side of the centerline of supports. Once calculated, distribute accordingly.

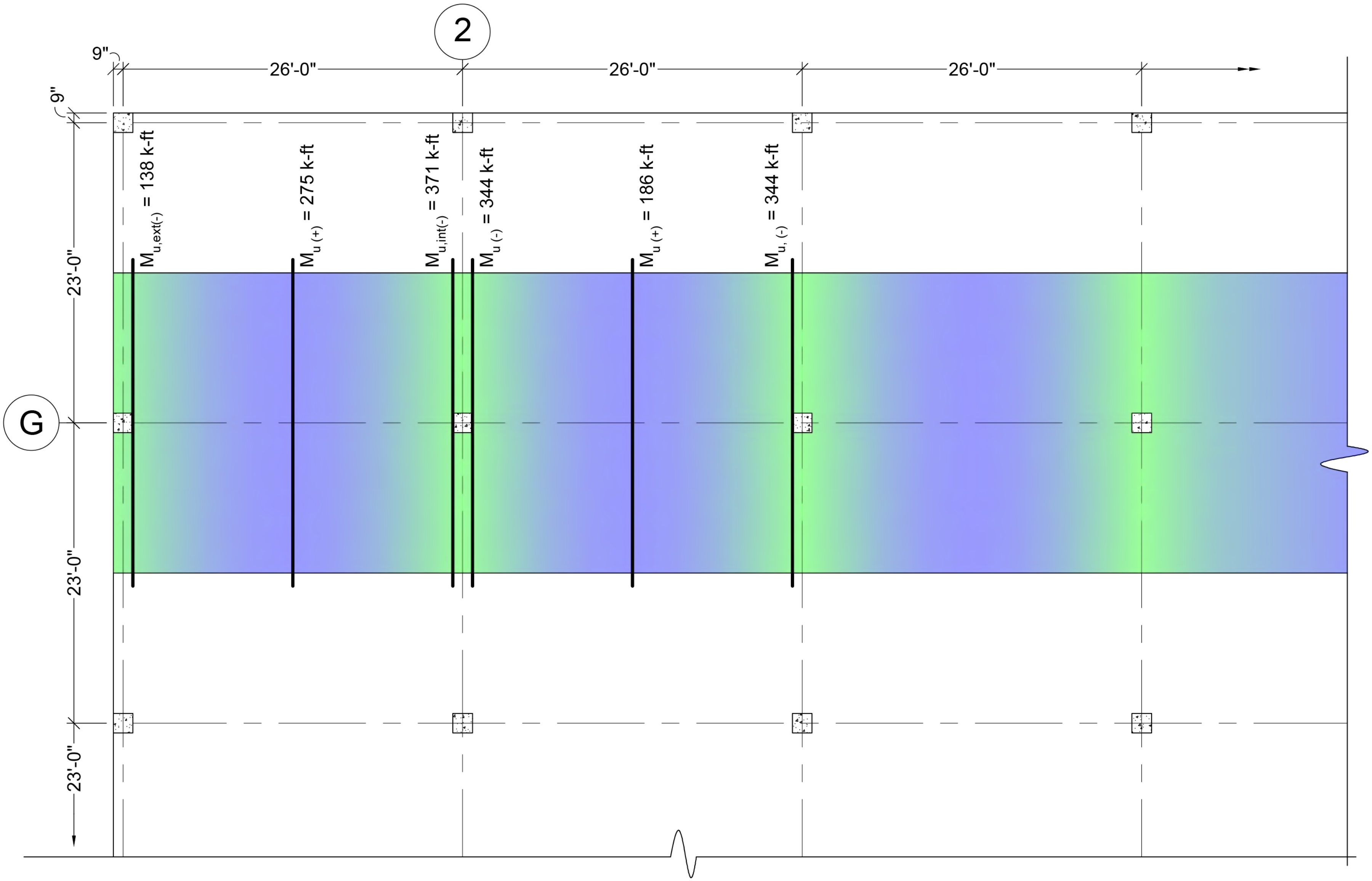


Figure 4: Gridline G Full Design Strip Moments (Total Factored Static Moments)

ACI 318-19	Calculations	Description
STEP 6b: Direct Design Method – Gridline G Distribution of Moment to Column Strip and Middle Strip		
	$l_1 = 26\ feet\ (direction\ in\ which\ moment\ is\ being\ determined)$ $l_2 = 23\ feet$ $\alpha_{f1} = 0\ (no\ beam\ exists, so\ no\ beam/slab\ flexural\ stiffness\ ratio)$	The DDM is now used to further distribute moments to columns strips, beams (*not applicable here*), middle strips, columns, and walls (*not applicable here*).
8.10.5.1 (ACI 318-14)	$\boldsymbol{Portion\ of\ M_{u,int\ (-)}\ to\ Column\ Strip}$ $M_{u,int\ (-)\ CS} = 0.75 \times 371 = 279\ k-ft\ (interior\ support\ @\ End\ Span)$ $M_{u,int\ (-)\ CS} = 0.75 \times 344 = 258\ k-ft\ (all\ other\ interior\ supports)$	Determine portion of $M_{u,int\ (-)}$ in column strip. In the absence of an integral beam, 75% of the moment is assigned to the column strip per Table 8.10.5.1 (ACI 318-14) regardless of the l_2/l_1 ratio.
8.10.6 (ACI 318-14)	$\boldsymbol{Portion\ of\ M_{u,int\ (-)}\ to\ Middle\ Strip}$ $M_{u,int\ (-)\ MS} = (1-0.75) \times 371 = 93\ k-ft\ (int. support\ @\ End\ Span)$ $M_{u,int\ (-)\ MS} = (1-0.75) \times 344 = 86\ k-ft\ (all\ other\ int. supports)$	Determine portion of $M_{u,int\ (-)}$ in middle strip. This magnitude will be divided equally between each of two (2) half middle strips flanking the column strip.
8.10.5.2 (ACI 318-14)	$\boldsymbol{Portion\ of\ M_{u,ext\ (-)}\ to\ Column\ Strip}$ $M_{u,ext\ (-)\ CS} = 138\ k-ft$ $\boldsymbol{Portion\ of\ M_{u,ext\ (-)}\ to\ Middle\ Strip}$ $M_{u,ext\ (-)\ MS} = 0$	Determine portion of $M_{u,ext\ (-)}$ in column strip. Because there are no torsionally-stiff walls or beams at the exterior building edge perpendicular to the design strip, there is no means by which moment can be reliably distributed along the width of the design strip. As such, 100% of the moment is assigned to the column strip.
8.10.5.5 (ACI 318-14)	$\boldsymbol{Portion\ of\ M_{u\ (+)}\ to\ Column\ Strip}$ $M_{u\ (+)\ CS} = 0.60 \times 275 = 165\ k-ft\ @\ End\ Span$ $M_{u\ (+)\ CS} = 0.60 \times 186 = 112\ k-ft\ @\ Interior\ Span$	Determine portion of $M_{u\ (+)}$ in column strip. In the absence of an integral beam, 60% of the moment is assigned to the column strip per Table 8.10.5.1 (ACI 318-14) regardless of the l_2/l_1 ratio.
8.10.6 (ACI 318-14)	$\boldsymbol{Portion\ of\ M_{u\ (+)}\ to\ Middle\ Strip}$ $M_{u\ (+)\ MS} = (1-0.60) \times 275 = 110\ k-ft\ @\ End\ Span$ $M_{u\ (+)\ MS} = (1-0.60) \times 186 = 75\ k-ft\ @\ Interior\ Span$ $Refer\ to\ Fig. 5\ for\ final\ moment\ distribution\ along\ Gridline\ G.$	Determine portion of $M_{u\ (+)}$ in middle strip. This magnitude will be divided equally between each of two (2) half middle strips flanking the column strip.

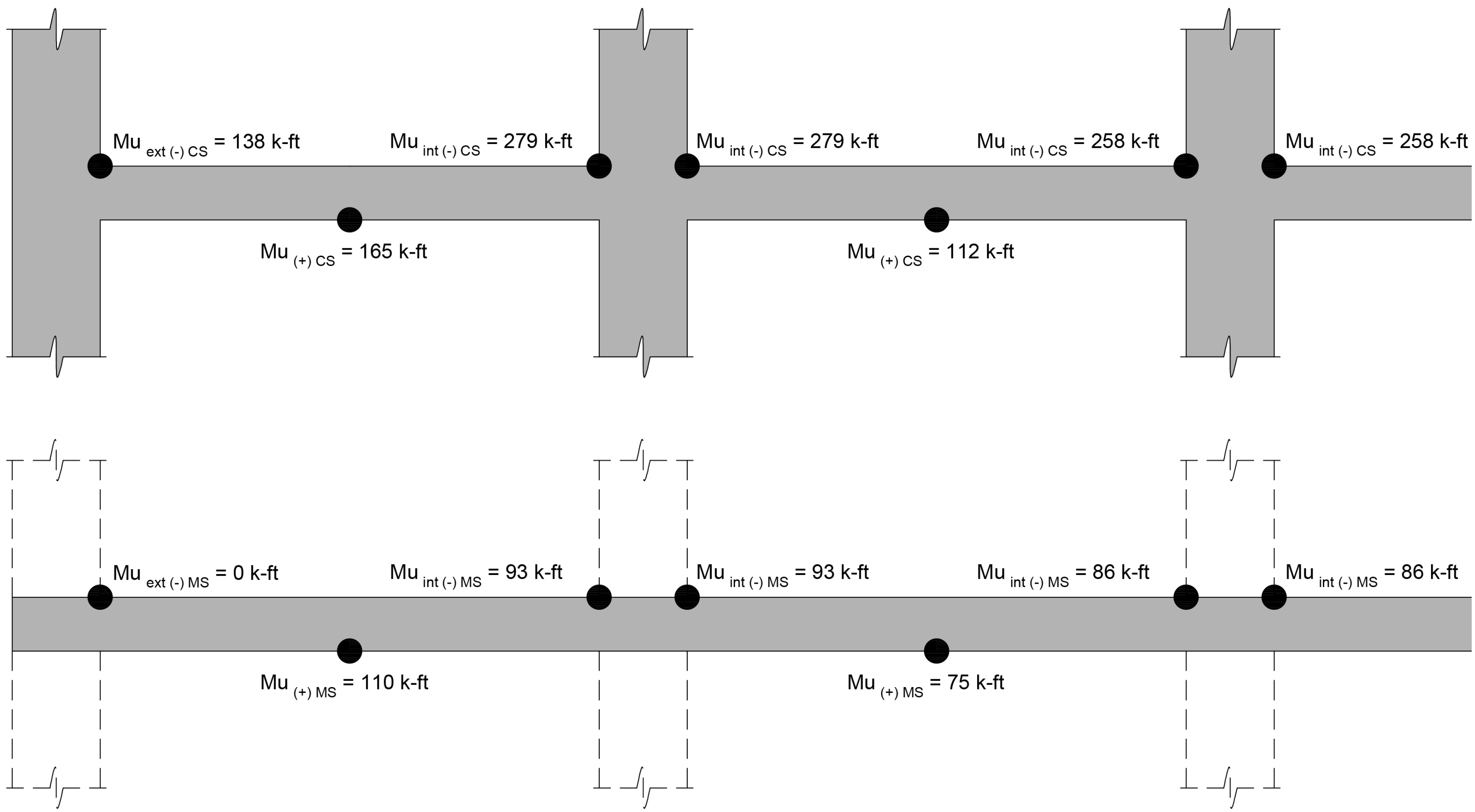


Figure 5 – Gridline G Column Strip and Middle Strip Moments

ACI 318-19	Calculations	Description
STEP 6c: Direct Design Method – Gridline G Slab Negative Moment Transfer to Column		
		When there are slab negative moments at a column that are of unequal magnitude (or in the case of an exterior column, a single "unopposed" slab negative moment) the net difference in slab moments – or unbalanced moment - must be transferred into the column. The mechanism by which this transfer occurs is through a combination of flexure and shear.
8.10.7.2 (ACI 318-14)	$\boldsymbol{At\ Interior\ Column}$ $M_{sc} = 0.07\left[(q_{Du} + 0.5q_{Lu})l_2{l_n}^2 - q'_{Du}l'_2({l'_n}^2)\right]$ $M_{sc} = 0.07[(0.178 + 0.5 \times 0.128)(23)(24.5^2) - 0.178(23)(24.5^2)]$ $M_{sc} = 62\ k - ft$	In this step, our focus is on the flexural component (the shear component will be addressed in Step 8). Calculate this factored slab moment, M_{sc}, resisted by the column at a joint.
8.10.7.3 (ACI 318-14)	$\boldsymbol{At\ Exterior\ Column}$ $M_{sc} = 0.3M_o = 0.3 \times 529 = 159\ k - ft$	
8.4.2.2.1 8.4.2.2.2	$\gamma_f = \dfrac{1}{1 + \left(\frac{2}{3}\right)\sqrt{\frac{b_1}{b_2}}} = 0.60, \quad (b_1 = b_2 = 18")$	Second, determine the fraction of slab moment resisted by the column, transferred by flexure, using the multiplier γ_f.
	$\boldsymbol{Slab\ Moment\ to\ Interior\ Column\ via\ Flexure}$ $\gamma_f M_{sc} = 0.60 \times 62 = 37.2\ k - ft$ $\boldsymbol{Slab\ Moment\ to\ Exterior\ Column\ via\ Flexure}$ $\gamma_f M_{sc} = 0.60 \times 159 = 96\ k - ft$	Note that these ultimate moments are <u>not</u> mathematically cumulative with the previously calculated $M_{u,int\,(-)\,CS}$ and $M_{u,ext\,(-)\,CS}$ column strip ultimate moments. Rather, they will be part of a subsequent check in Steps 6C and 7C to ensure that an appropriate portion of the overall negative moment reinforcement is aligned within a prescribed width proximate to the column itself to ensure the flexural transfer mechanism is achieved.

ACI 318-19	Calculations	Description
STEP 7: Two-Way (Punching) Shear – Revisited		
See Step 5 8.4.4.2.2 8.4.4.2.3	***Interior Column*** $v_{uv} = 0.177\ ksi$ $\varphi v_n = 0.212\ ksi$ ***Exterior Column*** $v_{uv} = 0.134\ ksi$ $\varphi v_n = 0.212\ ksi$ $\gamma_v = 1 - \gamma_f\ \ (\gamma_f = 0.60, from\ Step\ 6c\ and\ 7c)$ $\therefore \gamma_v = 1 - 0.60 = 0.40$ $v_{uv,AB} = v_{uv} + \frac{\gamma_v M_{sc} c_{AB}}{J_c}$ ***Interior Column*** $v_{uv} = 0.177\ ksi$ $\gamma_v = 0.40$ $M_{sc} = 62\ k - ft\ (Gridline\ G\ direction\ governs\ over\ Gridline\ 2)$ $c_{AB} = (18" + 9.375")/2 = 13.69"$ $J_c = 131{,}975\ in^4$ $v_{uv,AB} = 0.177 + \frac{0.40 \times 62 \times 12 \times 13.69"}{131{,}975} = 0.208\ ksi\ < 0.212\ ksi\ \mathbf{OK}$ ***Exterior Column*** $v_{uv} = 0.134\ ksi$ $\gamma_v = 0.40$ $M_{sc} = 159\ k - ft\ (Gridline\ G\ direction\ governs\ over\ Gridline\ 2)$ $c_{AB} = 7.075"$ $J_c = 41{,}960\ in^4$ $v_{u,AB} = 0.134 + \frac{0.40 \times 159 \times 12 \times 7.075"}{41{,}960} = 0.263\ ksi\ > 0.212\ ksi\ \mathbf{NG}$	In Step 5 we calculated v_{uv}, the *factored shear stress on the slab critical section for two-way action <u>without moment transfer</u>*. We also calculated the two-way shear design strength, φv_n, based on $v_n = v_c$ (no shear reinforcement). In Steps 6c we showed that a portion of slab moment is transferred to the supporting column through flexure. In this step, we illustrate how the remainder of slab moment is transferred by eccentricity of shear. The fraction of M_{sc} transferred by eccentricity of shear is shown. The magnitude of direct shear and maximum eccentric shear must be combined. Subscript AB refers to the edge of the linearly-varying shear stress distribution with the highest magnitude, per Fig. R8.4.4.2.3. Judgment might suggest that shear reinforcement is still appropriate at interior columns given the narrow margin of adequacy, but such an effort will not be covered in this example. Results clearly indicate that punching shear demand at an exterior column will exceed the two-way shear strength provided by concrete without shear reinforcement.

ACI 318-19	Calculations	Description
STEP 8: Two-Way (Punching) Shear Reinforcement Design and Strength Check		
22.6.4.1	$\boldsymbol{Critical\ Section\ 1 - At\ Column\ Offset}$	Headed shear reinforcement will be used at exterior columns. Two critical sections must be checked per Figure 8. For this example, a trial arrangement of stud rails is presented. First check the critical section at the column offset.
Table 22.6.6.1	$v_c = 3\lambda_s\lambda\sqrt{f'_c} = 3 \times 1.0 \times 1.0 \times 70.71 = 0.212\ ksi$	Note that upon introduction of shear reinforcement, the two-way shear strength provided by concrete is no longer defined by Section 22.6.5.2 but is instead calculated per Section 22.6.6.1
22.6.1.3 22.6.6.3	$\varphi v_n = 0.75(v_c + v_s)$ $v_u = 0.263\ ksi < \varphi \times 8 \times \sqrt{f'_c} = .424\ ksi \therefore slab\ depth\ confirmed\ OK$	
8.5.1.1	$Need\ \varphi v_n \geq\ v_u$ $0.75(0.212) + 0.75v_{s,reqd} = 0.263$ $v_{s,reqd} = 0.139\ ksi$ $Trial: six\ (6)\ rows\ of\ 1/2"\ \emptyset\ headed\ anchors, f_{yt} = 51\ ksi.$	
22.6.8.2 8.7.7.1.2	$v_s = \dfrac{A_v f_{yt}}{b_{o1} s} = \dfrac{6 \times 0.196 \times 51}{72.75" \times\ 4"} = 0.206\ ksi$ $\varphi v_n = 0.75(0.212 + 0.206) = 0.314\ ksi > 0.263\ ksi$	Align stud rails at each corner of each column. Stud rails are 1.25" wide. Note that as shown in figure 8, the first peripheral line of shear studs is 3 inches from the column face, and the constant spacing of the remaining peripheral lines of shear studs is 4 inches. This arrangement complies with the requirements outlined in Table 8.7.7.1.2.
22.8.6.3	$Confirm\ \dfrac{A_v}{s} \geq 2\sqrt{f'_c} \times \dfrac{b_o}{f_{yt}}$ $\dfrac{6 \times 0.196}{4} = 0.294 > 2 \times 70.71 \times \dfrac{72.75}{51} = 0.202$ $\therefore Trial\ headed\ shear\ stud\ reinforcement\ is\ adequate.$	Two-way shear capacity at Critical Section 1 is satisfactory. Height of shear stud assemblies shall conform to Section 8.7.7.1.1

ACI 318-19	Calculations	Description
STEP 8: Two-Way (Punching) Shear Reinforcement Design and Strength Check - continued		
22.6.4.2 Table 22.6.6.1	$\boldsymbol{Critical\ Section\ 2 - At\ Shear\ Reinforcement\ Boundary}$ $v_c = 2\lambda_s\lambda\sqrt{f'_c} = 2 \times 1.0 \times 1.0 \times 70.71 = 0.141\ ksi$ $b_{o2} = 122.54"$ $Loaded\ tributary\ area = 23\ ft \times 13\ ft - \frac{2{,}224}{144} = 283.6\ sf$ $V_u = 0.306\ ksf \times 283.6\ sf = 87\ kips$ $Need\ \varphi v_n \geq v_u$ $v_u = \frac{87\ kips}{122.54" \times 9.375"} = 0.076\ ksi$ $\varphi v_n = \varphi v_c = 0.75 \times 0.141\ ksi = 0.106\ ksi \rightarrow OK$	Now check two-way shear at the shear reinforcement boundary. Note that upon introduction of shear reinforcement, the two-way shear strength provided by concrete is no longer defined by Section 22.6.5.2 but is instead calculated per Section 22.6.6.1 Critical Section 2 is per Figure 6. Area of the critical section is 2,224 in^2. Note that there is no shear reinforcement contribution to the two-way shear strength at this critical section. Two-way shear capacity at Critical Section 2 is satisfactory. Headed shear stud reinforcement will be provided per Figure 6 and shall comply with Section 8.7.7.

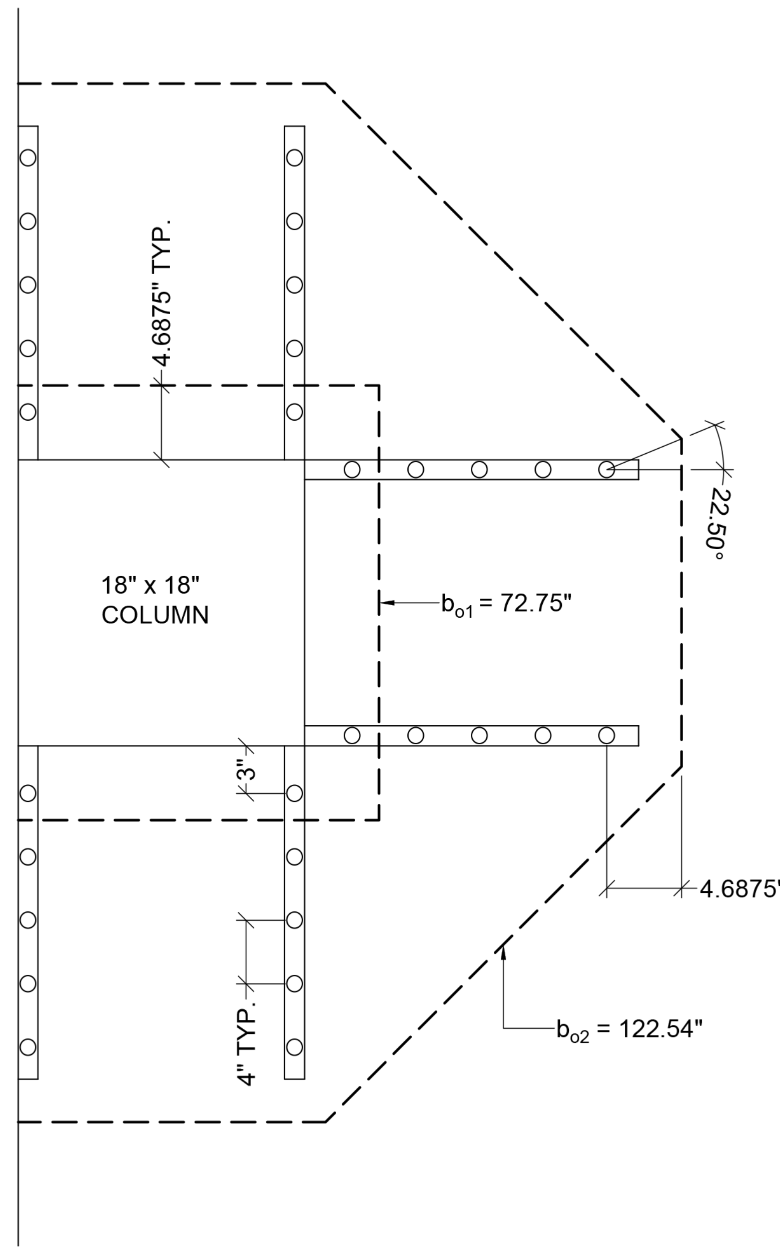


Figure 6: Exterior Column Two-Way Shear Critical Sections and Shear Stud Reinforcement

ACI 318-19	Calculations	Description
STEP 9: One Way Shear Check		
8.4.3.1 Table 6.5.4	$Calculate\ factored\ one\ way\ shear\ at\ the\ face\ of\ the\ support.$ $V_u = 1.15 \times w_u \times \frac{l_n}{2}$ $V_u = 1.15 \times 0.306\ ksf \times 23\ feet \times \frac{24.5\ ft}{2} = 100\ kips$	In this example we will use an approximate value for one-way shear. Generally speaking, one-way shear will rarely govern over two-way shear for a two-way slab, but the check is carried out nonetheless.
22.5.5.1 22.5.5.1.1	$Start\ with\ A_v = 0, \therefore A_v < A_{v,min}$ $V_c = \left[8\lambda_s\lambda\left(\rho_w{}^{1/3}\right)\sqrt{f'_c} + \frac{N_u}{6A_g}\right] \times b_w d \le 5\lambda\sqrt{f'_c} b_w d$ $\rho_w = \frac{A_s}{b_w d}$ $Flexural\ reinforcement\ has\ not\ yet\ been\ determined.$ $Assume\ (10)\ \#5\ bars\ or\ D31\ wires = A_s = 3.10\ in^2.$ $\rho_w = \frac{3.10}{23 \times 12 \times 9.375} = 0.0012$ $V_c = \left[8 \times 1.0 \times 1.0 \times 0.0012^{1/3} \times \sqrt{5000} + 0\right] \times 23 \times 12 \times 9.375$ $V_c = 155\ kips < 5 \times 1.0 \times \sqrt{5000} \times 23 \times 12 \times 9.375 = 914\ kips$	We have not yet carried out flexural design, so we will assume a flexural steel amount for this check. Steel amounts finalized in Steps 11 and 12 confirm that the assumption here is satisfactory.
8.5.1.1 22.5.1.2	$Need\ \varphi V_n \ge V_u$ $0.75 \times 155\ kips = 116\ kips > V_u = 100\ kips\ \therefore OK$ $Also\ need\ V_u \le \varphi\left(V_c + 8\sqrt{f'_c} b_w d\right) \rightarrow 100\ kips < 226\ kips\ \therefore OK$	Even with what is likely a conservative assumption for flexural steel area, one-way shear is satisfactory. Slab proportioning/depth is appropriate.

ACI 318-19	Calculations	Description
STEP 10: Gridline G – Labeled Moments		
	$\boldsymbol{Gridline\ G}$ $d = 11 - 1" - \frac{5/8"}{2} = \ use\ 9.5\ inches$ $Column\ Strip\ Negative\ Moments$ $1C{:}\ M_{u,ext\ (-)\ CS} = 138\ k - ft$ $\left(\gamma_f M_{sc} = 96\ k - ft\right)$ $2C{:}\ M_{u,int\ (-)\ CS} = 279\ k - ft\ (interior\ support\ @\ End\ Span)$ $3C{:}\ M_{u,int\ (-)\ CS} = 258\ k - ft\ (all\ other\ interior\ supports)$ $\left(\gamma_f M_{sc} = 37.2\ k - ft\right)$ $Column\ Strip\ Positive\ Moments$ $4C{:}\ M_{u\ (+)\ CS} = 165\ k - ft\ @\ End\ Span$ $5C{:}\ M_{u\ (+)\ CS} = 112\ k - ft\ @\ Interior\ Span$ $Middle\ Strip\ Negative\ Moments$ $1M{:}\ M_{u,ext\ (-)\ MS} = 0$ $2M{:}\ M_{u,int\ (-)\ MS} = 93\ k - ft\ (int.support\ @\ End\ Span)$ $3M{:}\ M_{u,int\ (-)\ MS} = 86\ k - ft\ (all\ other\ int.supports)$ $Middle\ Strip\ Positive\ Moments$ $4M{:}\ M_{u\ (+)\ MS} = 110\ k - ft\ @\ End\ Span$ $5M{:}\ M_{u\ (+)\ MS} = 75\ k - ft\ @\ Interior\ Span$	Flexural steel reinforcement areas will now be calculated for Gridline G. we will assign the Gridline G reinforcement to outside mats to maximize the effective depth. See Step 6b and Figure 5 for previously calculated ultimate moments. See Step 6c for previously calculated factored slab negative moments ($\gamma_f M_{sc}$) to be transferred to columns. To ensure this transfer, ACI 318 dictates reinforcement be positioned proximate to the column to resist the slab moment, as outlined in this step. See Figure 7 for labeled moments. Labels used here are for bookkeeping of the respective flexural calculations and are also reflected in the subsequent detailing of WWR mats.

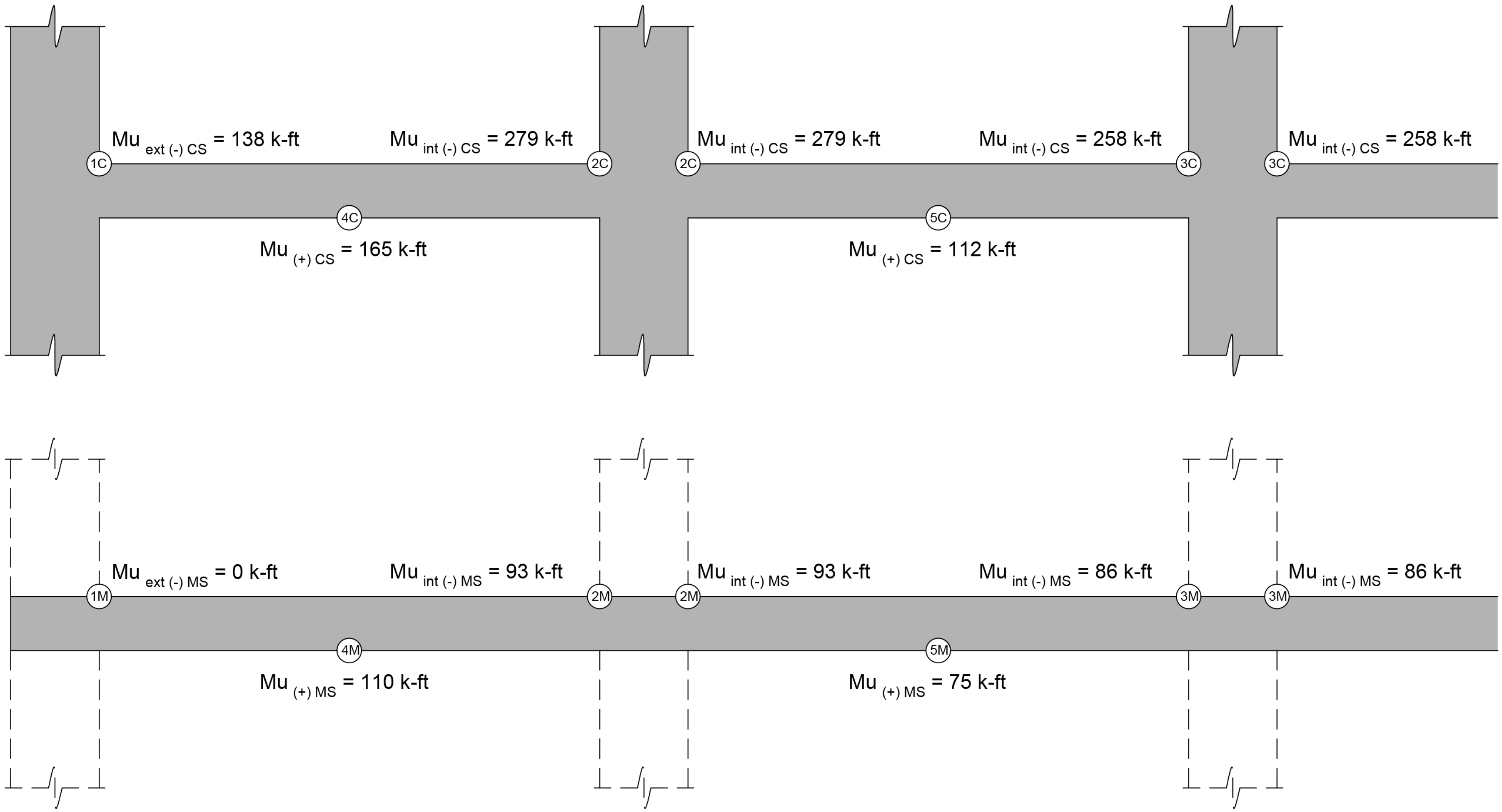


Figure 7: Gridline G – Labeled Moments for subsequent flexural design and WWR mat detailing

ACI 318-19	Calculations	Description
STEP 11-1C: Required Flexural Reinforcement – Gridline G – Location 1C		
21.2.2	$\boldsymbol{Location\ 1C}$: $\boldsymbol{M_{u,ext\,(-)\,CS} = 138\ k - ft}$ $\boldsymbol{(\gamma_f M_{sc} = 96\ k - ft)}$ $Assume\ tension - controlled\ section\ to\ start{:}\ \varphi = 0.90$ $m = \dfrac{f_y}{0.85 \times f'_c} = 18.82$ $R_u = \dfrac{M_u}{\varphi b d^2}$ $\rho_{reqd} = \dfrac{1}{m} \times \left(1 - \sqrt{1 - \dfrac{2mR_u}{f_y}}\right)$	
8.6.1.1	$M_u = 138\ k - ft$ $b_{col\ strip} = 138\ inches$ $R_u = 0.148\ ksi$ $\rho_{reqd} = 0.0019$ $A_{s,reqd} = 0.0019 \times 138" \times 9.5" = 2.49\ in^2$ $A_{s,min} = 0.0018 \times A_g = 0.0018 \times 138" \times 11" = 2.74\ in^2 \leftarrow$ $\boldsymbol{A_{s,reqd-total} = 2.74\ in^2}$	
8.4.2.2.3 8.4.2.2.5	$M_u = 96\ k - ft$ $b_{transfer} = 18" + 1.5(11") + 1.5(11") = 51\ inches$ $R_u = 0.278\ ksi$ $\rho_{reqd} = 0.0036$ $\boldsymbol{A_{s,reqd-transfer} = 0.0036 \times 51" \times 9.5" = 1.75\ in^2}$	

ACI 318-19	Calculations	Description
STEP 11-1C: Required Flexural Reinforcement – Gridline G – Location 1C (continued)		
8.6.1.2 8.7.2.2	$Determine\ minimum\ total\ number\ of\ \#5\ top\ bars\ in\ column\ strip.$ $\frac{2.74\ in^2}{0.31\ in^2} = 8.83\ bars \rightarrow \boldsymbol{use\ ten\ (10)\ \#5\ bars}$ $Determine\ minimum\ total\ number\ of\ \#5\ top\ bars\ in\ transfer\ width.$ $\frac{1.75\ in^2}{0.31\ in^2} = 5.65\ bars \rightarrow \boldsymbol{use\ six\ (6)\ \#5\ bars}\ **$ $We\ must\ confirm\ the\ above\ to\ be\ satisfactory.$ $**\ Check\ potential\ for\ flexure\ driven\ punching\ failure.$ $At\ Exterior\ Column:$ $Check\ v_{uv}\ against\ \varphi 2\lambda_s\lambda\sqrt{f'_c}\ .$ $v_{uv} = 0.134\ ksi\ < 2\lambda_s\lambda\sqrt{f'_c} = 2 \times 1.0 \times 1.0 \times 70.71 = 0.141\ ksi$ $\therefore\ no\ check\ for\ A_{s,min}\ needed\ for\ flexure\ driven\ punching\ shear.$ $Maximum\ Spacing\ (s) = lesser\ of\ 2h\ and\ 18\ inches$ $2h = 2 \times 11 = 22, \therefore\ use\ 18"$ $Refer\ to\ Figure\ 8\ for\ a\ satisfactory\ layout\ of\ reinforcement.$	The design professional must calculate the required steel area with due consideration for potential flexure-driven punching failure, and also must maintain prescriptive maximum bar spacing. Flexure-driven punching shear confirmed to not be a limit state. Proceed with prescriptive spacing check.

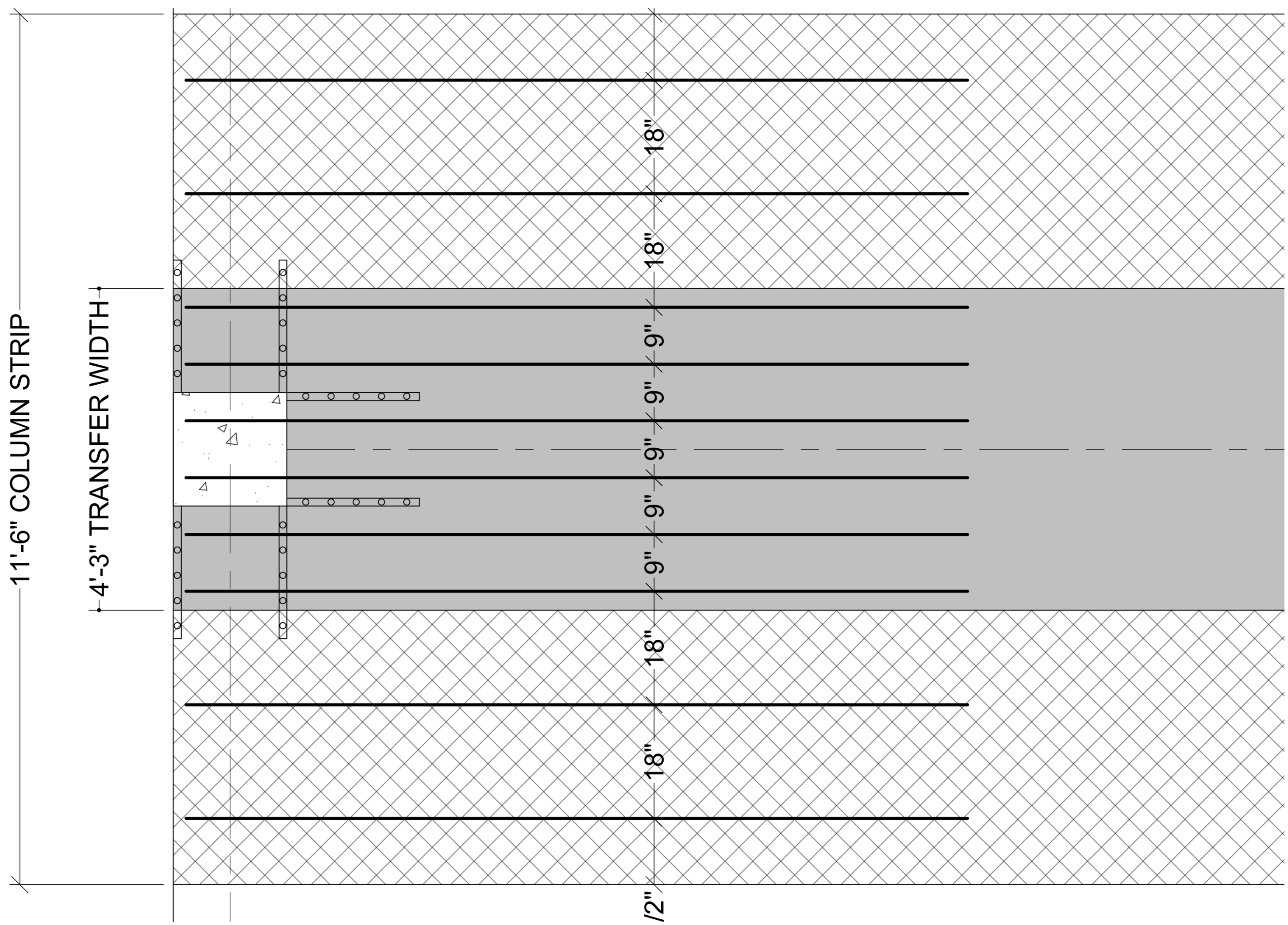


Figure 8: Gridline G – Exterior Column Top Reinforcement "1C"

ACI 318-19	Calculations	Description
STEP 11-1C: Required Flexural Reinforcement – Gridline G – Location 1C (continued)		
	While perhaps unlikely that the design professional will explicitly detail transfer widths and the precise distribution of each reinforcing bar as shown in Figure 8, it is still the designer's responsibility to provide direction to the contractor ensuring such a distribution is satisfied (see Figure 10 for engineer's description of transfer width requirements). The burden of ensuring correct detailing and fabrication of bar quantities, lengths, and curtailments ultimately resides with the reinforcement fabricator/detailer, based on information provided by the design professional. The reinforcement placing contractor is then tasked with ensuring that spacing and distribution requirements are in conformance with language presented on the design professional's contract drawings and reflected on the fabrication/placement drawings. *For welded wire reinforcement manufacture, the responsibilities of the detailer and placer are, to a large extent, combined into one as they relate to how the reinforcement is to be laid out on the job. The WWR manufacturer's detailer must take the design professional's requirements and derive WWR mat configurations consisting of predetermined arrangements of structural wires that are suitable for placement with due consideration for spatial constraints and obstructions, all while satisfying the design professional's intent. In effect, the level of detail related to reinforcement layout in Figure 9 must be built into the WWR submittal drawings, as it is the WWR detailer who is tasked with taking most – if not all – of the burden off of the reinforcement placing contractor as it relates to layout. This of course helps the placing contractor greatly expedite his efforts in the field.* *With that said, and though perhaps not always in explicit detailed form, it is the design professional's responsibility to define expectations related to attributes such as column and middle strip widths, transfer widths, maximum reinforcement spacing, minimum bars through a column core, etc. Figures 9, 10, and 11 are representative of common formats of design information as presented on contract drawing.*	

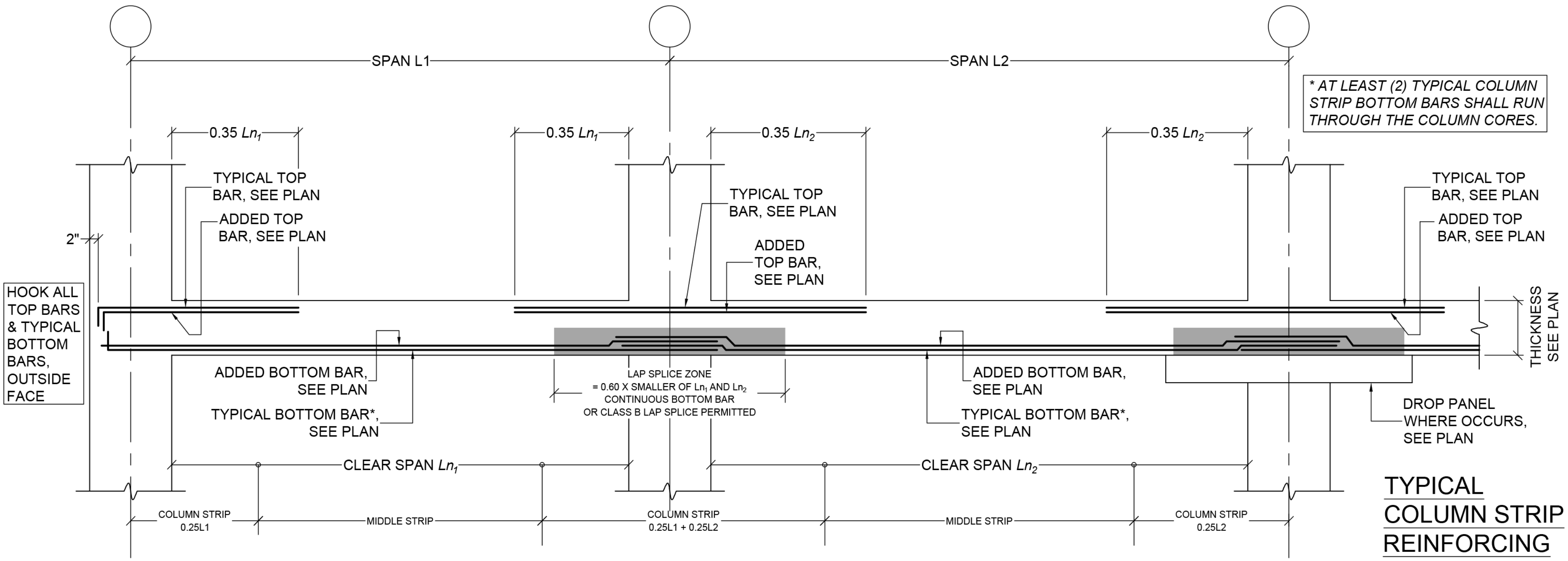


Figure 9: Representative Two-Way Slab Reinforcing Details on Contract Drawings

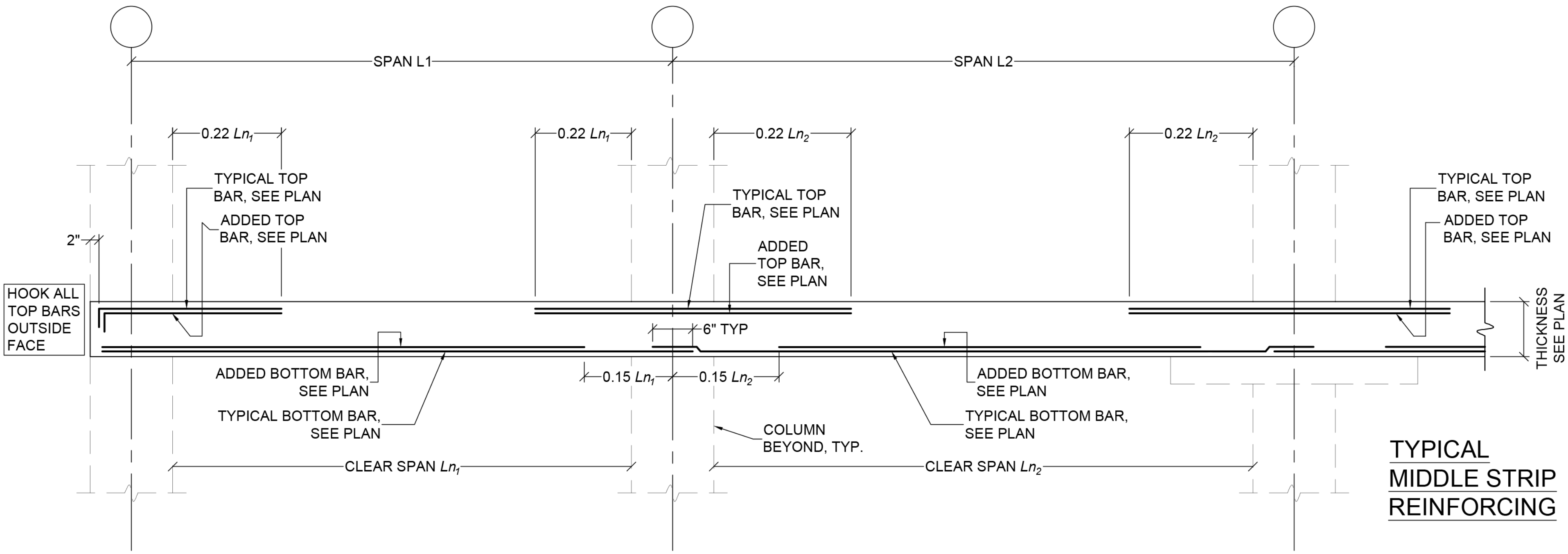


Figure 9 (continued): Representative Two-Way Slab Reinforcing Details on Contract Drawings

TWO-WAY MILD REINFORCED CONCRETE SLAB NOTES

1. SLAB REINFORCEMENT SHALL CONSIST OF DEFORMED REINFORCING BARS OR WELDED DEFORMED WIRE REINFORCEMENT OF EQUAL AREA, EQUAL OR LESSER SPACING, AND IDENTICAL CURTAILMENT (HOOKS AND LAPS).
2. REFER TO PLANS FOR ILLUSTRATIVE BOUNDARIES OF COLUMN STRIP AND MIDDLE STRIP REGIONS. REFER TO TWO-WAY SLAB TYPICAL DETAILS FOR ARRANGEMENT OF TOP AND BOTTOM REINFORCEMENT WITHIN COLUMN STRIPS AND MIDDLE STRIPS.
3. REFER TO DESIGNATIONS ON PLANS FOR REINFORCEMENT REQUIREMENTS. LENGTHS INDICATED SHALL NOT BE LESS THAN PRESCRIPTIVE MINIMUM EXTENTS SHOWN IN THE TWO-WAY SLAB TYPICAL DETAILS.
4. CORE BARS ARE NOTED ON PLAN IN SQUARE BRACKETS […] AND REPRESENT THE PORTION OF THE TOTAL COLUMN STRIP BOTTOM REINFORCEMENT REQUIRED TO PASS THROUGH THE COLUMN CORE BETWEEN THE OUTERMOST LINES OF COLUMN LONGITUDINAL REINFORCEMENT. THE REMAINING TYPICAL COLUMN STRIP BARS/WIRES SHALL BE DISTRIBUTED AT EQUAL SPACING THROUGHOUT THE REMAINDER OF COLUMN STRIP WIDTH OUTSIDE OF THE COLUMN.
5. TRANSFER BARS ARE NOTED ON PLAN IN ANGLE BRACKETS <…> AND REPRESENT THE PORTION OF TOTAL COLUMN STRIP TOP REINFORCEMENT REQUIRED TO BE POSITIONED WITHIN A TRANSFER DIMENSION AS DEFINED BELOW:
   A. COLUMNS WITHOUT DROP PANEL OR SHEAR CAP: TRANSFER DIMENSION IS EQUAL TO THE COLUMN WIDTH PLUS A TRANSVERSE DISTANCE ON EACH SIDE OF THE COLUMN EQUAL TO THE LESSER OF (A) 1.5 X SLAB THICKNESS AND (B) DISTANCE TO EDGE OF SLAB.
   B. COLUMNS WITH DROP PANEL OR SHEAR CAP: TRANSFER DIMENSION IS EQUAL TO THE CAPITAL WIDTH PLUS A TRANSVERSE DISTANCE ON EACH SIDE OF THE CAPITAL EQUAL TO THE LESSER OF (A) 1.5 X DROP/CAP THICKNESS AND (B) DISTANCE TO EDGE OF DROP/CAP PLUS 1.5 SLAB THICKNESS.
6. OUTERMOST BARS/WIRES WITHIN A COLUMN STRIP SHALL BE NOT MORE THAN 4 INCHES AWAY FROM THE ILLUSTRATIVE BOUNDARIES OF THE COLUMN STRIP.
7. MIDDLE STRIP TOP AND BOTTOM BARS SHALL BE EVENLY DISTRIBUTED ACROSS THE ILLUSTRATIVE WIDTH. OUTERMOST BARS/WIRES WITHIN A MIDDLE STRIP SHALL BE NOT MORE THAN 8 INCHES AWAY FROM THE ILLUSTRATIVE BOUNDARY BETWEEN MIDDLE STRIP AND COLUMN STRIP.
8. MAXIMUM BAR/WIRE SPACING SHALL BE THE LESSER OF 2X THE SLAB THICKNESS OR 18 INCHES.
9. SEE PLAN FOR ADDED BARS, AS APPLICABLE.

Figure 10: Representative Two-Way Mild-Reinforced Concrete Slab Notes on Contract Drawings

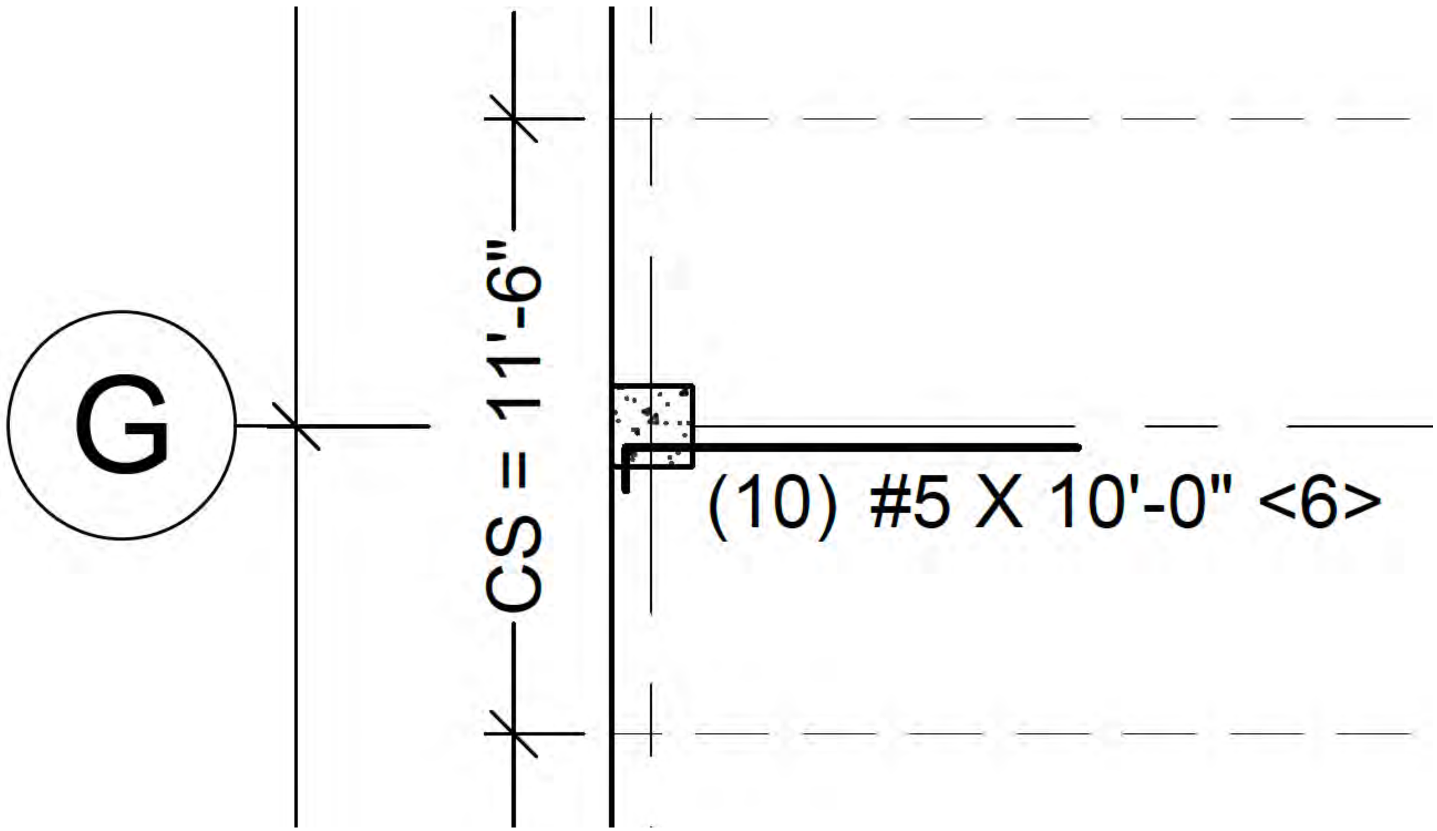


Figure 11: Representative Excerpt of Reinforcement Information as Presented on Floor Plan in Contract Drawings – Location 1C

ACI 318-19	Calculations	Description
STEP 11-1C: Required Flexural Reinforcement – Gridline G – Location 1C (continued)		
21.2 22.2 21.2 22.2	$\boldsymbol{Total\ Top\ Reinforcement} = (10)\ \#5$ $b = 138"$ $d = 9.5"$ $A_s = 3.10\ in^2$ $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{3.10 \times 80}{0.85 \times 5 \times 138} = 0.423"$ $\varepsilon_t = \frac{0.003 \times \beta_1 \times d}{a} - 0.003 = \frac{0.003 \times 0.80 \times 9.5}{0.423} - 0.003 = 0.051$ $\varepsilon_t = 0.051 >> \varepsilon_{ty} + 0.003 = 0.0058 \therefore Tension\ Controlled$ $\varphi M_n = 0.9 A_s f_y (d - a/2) = 173\ k - ft > 138\ k - ft$ $\boldsymbol{Transfer\ Top\ Reinforcement} = < 6 > \#5$ $b = 51"$ $d = 9.5"$ $A_s = 1.86\ in^2$ $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.86 \times 80}{0.85 \times 5 \times 51} = 0.687"$ $\varepsilon_t = \frac{0.003 \times \beta_1 \times d}{a} - 0.003 = \frac{0.003 \times 0.80 \times 9.5}{0.687} - 0.003 = 0.03$ $\varepsilon_t = 0.030 > \varepsilon_{ty} + 0.003 = 0.0058 \therefore Tension\ Controlled$ $\varphi M_n = 0.9 A_s f_y (d - a/2) = 102\ k - ft > 96\ k - ft$ $\therefore Design\ flexural\ strengths\ confirmed.\ Bar\ quantities\ acceptable.$	Close the loop on the design flexural strength check for Location 1C. Note that any potential contribution by compression reinforcement is ignored.

ACI 318-19	Calculations	Description
STEP 11-1C: Required Flexural Reinforcement – Gridline G – Location 1C (continued)		
	$A\ sidebar\ option\ to\ check:$ $\rho_{reqd,transfer} = 0.0036$ $A_{s,reqd-conservative} = 0.0036 \times 138 \times 9.5 = 4.72\ in^2$ $\frac{4.72\ in^2}{0.31\ in^2} = 15.22\ \rightarrow use\ (16)\ \#5\ bars$ $A_s = 4.96\ in^2$ $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{4.96 \times 80}{0.85 \times 5 \times 138} = 0.677''$ $\varepsilon_t = \frac{0.003 \times \beta_1 \times d}{a} - 0.003 = \frac{0.003 \times 0.80 \times 9.5}{0.677} - 0.003 = 0.031$ $\varepsilon_t = 0.031 > \varepsilon_{ty} + 0.003 = 0.0058 \therefore Tension\ Controlled$ $\varphi M_n = 0.9 A_s f_y (d - a/2) = 273\ k - ft > 138\ k - ft$ $\therefore Design\ flexural\ strengths\ confirmed.\ Bar\ quantities\ acceptable.$	The design professional might be tempted to simplify the reinforcement requirements by using the required reinforcement ratio within the transfer width and applying it across the entire column strip, satisfying implicitly the transfer reinforcement requirement but substantially increasing the overall reinforcement in the column strip. Such an approach is shown here to be acceptable from a design perspective (tension-controlled behavior is still maintained), but there is a fairly significant penalty in the form of additional steel beyond that which is required by ACI. While it is certainly the design professional's prerogative to invoke such simplifications during design, cost and benefit as it relates to overall project cost and schedule should also be a consideration. For this example, the sidebar option checked here will be ignored.

ACI 318-19	Calculations	Description
STEP 11-2C: Required Flexural Reinforcement – Gridline G – Location 2C		
	$\boldsymbol{Location\ 2C}$: $\boldsymbol{M_{u,int\,(-)\,CS} = 279\ k - ft}$ $\boldsymbol{\left(\gamma_f M_{sc} = 37.2\ k - ft\right)}$	
21.2.2	$Assume\ tension - controlled\ section\ to\ start{:}\ \varphi = 0.90$ $m = \dfrac{f_y}{0.85 \times f'_c} = 18.82$ $R_u = \dfrac{M_u}{\varphi b d^2}$ $\rho_{reqd} = \dfrac{1}{m} \times \left(1 - \sqrt{1 - \dfrac{2mR_u}{f_y}}\right)$	
8.6.1.1	$M_u = 279\ k - ft$ $b_{col\ strip} = 138\ inches$ $R_u = 0.299\ ksi$ $\rho_{reqd} = 0.0039$ $A_{s,reqd} = 0.0039 \times 138" \times 9.5" = 5.12\ in^2 \leftarrow$ $A_{s,min} = 0.0018 \times A_g = 0.0018 \times 138" \times 11" = 2.74\ in^2$ $\boldsymbol{A_{s,reqd-total} = 5.12\ in^2}$	
8.4.2.2.3 8.4.2.2.5	$M_u = 37.2\ k - ft$ $b_{transfer} = 18" + 1.5(11") + 1.5(11") = 51\ inches$ $R_u = 0.108\ ksi$ $\rho_{reqd} = 0.0014 < \rho_{min} = 0.0018 \leftarrow use\ \rho_{min}\ on\ gross\ area.$ $\boldsymbol{A_{s,reqd-transfer} = 0.0018 \times 51" \times 11" = 1.01\ in^2}$	

ACI 318-19	Calculations	Description
STEP 11-2C: Required Flexural Reinforcement – Gridline G – Location 2C (continued)		
8.6.1.2 8.7.2.2	$Determine\ minimum\ total\ number\ of\ \#5\ top\ bars\ in\ column\ strip.$ $\frac{5.12\ in^2}{0.31\ in^2} = 16.51\ bars \rightarrow \boldsymbol{use\ eighteen\ (18)\ \#5\ bars}$ $Determine\ minimum\ total\ number\ of\ \#5\ top\ bars\ in\ transfer\ width.$ $\frac{1.01\ in^2}{0.31\ in^2} = 3.26\ bars \rightarrow \boldsymbol{use\ four\ (4)\#5\ bars}\ **$ $**\ Check\ potential\ for\ flexure\ driven\ punching\ failure.$ $At\ Interior\ Column:$ $Check\ v_{uv}\ against\ \varphi 2\lambda_s\lambda\sqrt{f'_c}\ .$ $v_{uv} = 0.177\ ksi\ > 2\lambda_s\lambda\sqrt{f'_c} = 2 \times 1.0 \times 1.0 \times 70.71 = 0.141\ ksi$ $\therefore\ Must\ provide\ A_{s,min}\ to\ resist\ flexure\ driven\ punching\ shear!$ $A_{s,min} = \frac{5v_{uv}b_{slab}b_o}{\varphi\alpha_s f_y} = \frac{5 \times 0.177\ ksi \times 51" \times 109.5"}{0.75 \times 40 \times 80\ ksi} = 2.06\ in^2$ $\frac{2.06\ in^2}{0.31\ in^2} = 6.65\ bars \rightarrow \boldsymbol{use\ eight\ (8)\ \#5\ bars\ in\ lieu\ of\ four}.$ $Maximum\ Spacing\ (s) = lesser\ of\ 2h\ and\ 18\ inches$ $2h = 2 \times 11 = 22, \therefore\ use\ 18"$	The design professional must calculate the required steel area with due consideration for potential flexure-driven punching failure, and also must maintain prescriptive maximum bar spacing. Flexure-driven punching shear is confirmed to be a limit state.

ACI 318-19	Calculations	Description
STEP 11-2C: Required Flexural Reinforcement – Gridline G – Location 2C (continued)		
    21.2 22.2 21.2 22.2	$\boldsymbol{Total\ Top\ Reinforcement} = (18)\ \#5$ $b = 138"$ $d = 9.5"$ $A_s = 5.58\ in^2$ $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{5.58 \times 80}{0.85 \times 5 \times 138} = 0.761"$ $\varepsilon_t = \frac{0.003 \times \beta_1 \times d}{a} - 0.003 = \frac{0.003 \times 0.80 \times 9.5}{0.761} - 0.003 = 0.027$ $\varepsilon_t = 0.027 >> \varepsilon_{ty} + 0.003 = 0.0058 \therefore Tension\ Controlled$ $\varphi M_n = 0.9 A_s f_y (d - a/2) = 305\ k - ft > 279\ k - ft$ $\boldsymbol{Transfer\ Top\ Reinforcement} = < 8 > \#5$ $b = 51"$ $d = 9.5"$ $A_s = 2.48\ in^2$ $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{2.48 \times 80}{0.85 \times 5 \times 51} = 0.915"$ $\varepsilon_t = \frac{0.003 \times \beta_1 \times d}{a} - 0.003 = \frac{0.003 \times 0.80 \times 9.5}{0.915} - 0.003 = 0.022$ $\varepsilon_t = 0.022 > \varepsilon_{ty} + 0.003 = 0.0058 \therefore Tension\ Controlled$ $\varphi M_n = 0.9 A_s f_y (d - a/2) = 135\ k - ft > 37.2\ k - ft$ $\therefore Design\ flexural\ strengths\ confirmed.\ Bar\ quantities\ acceptable.$ $See\ Figure\ 12\ for\ requirement\ shown\ on\ contract\ drawing.$	Close the loop on the design flexural strength check for Location 2C. Note that any potential contribution by compression reinforcement is ignored.

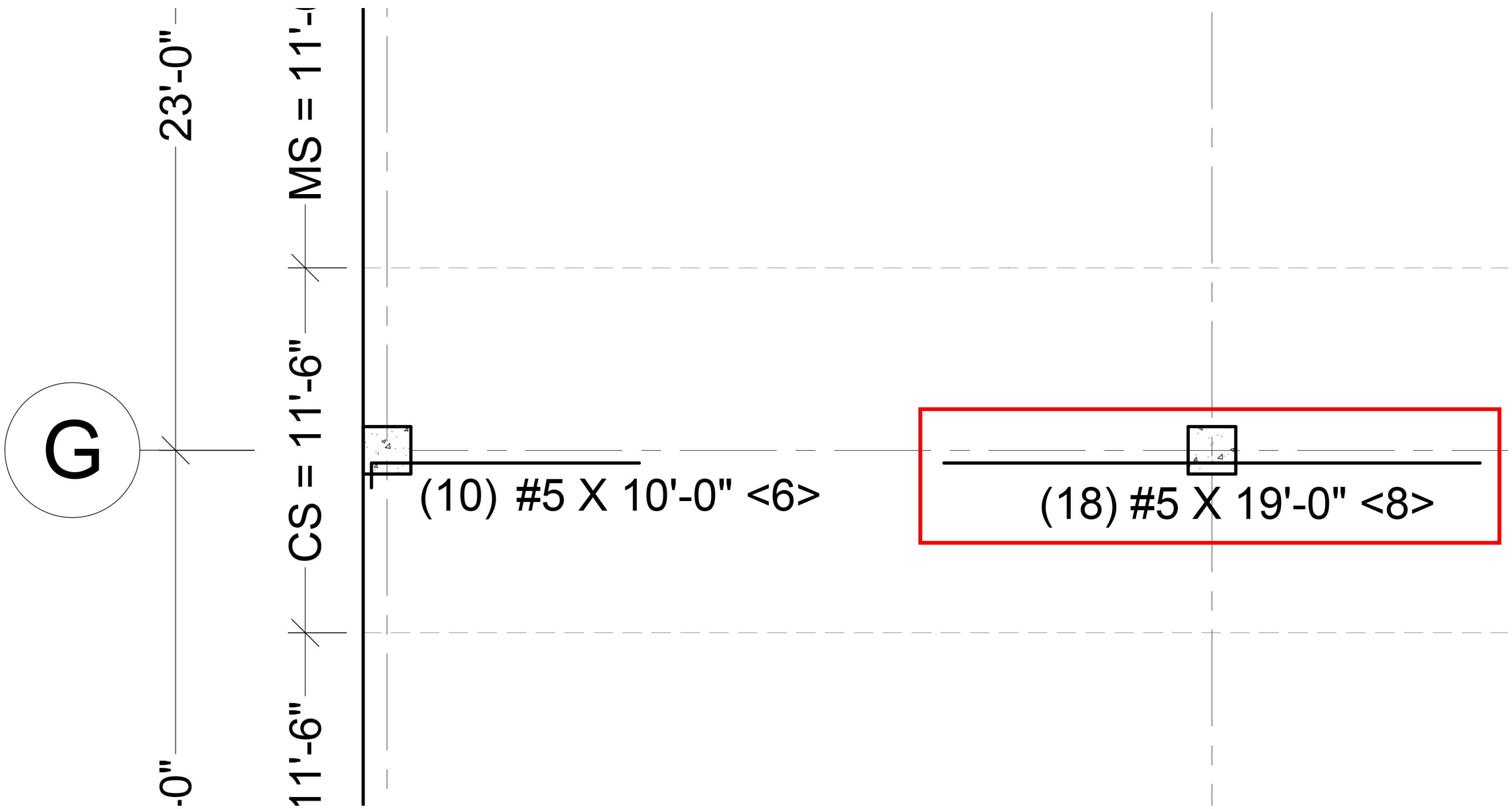


Figure 12: Representative Excerpt of Reinforcement Information as Presented on Floor Plan in Contract Drawings – Location 2C

ACI 318-19	Calculations	Description
STEP 11-3C: Required Flexural Reinforcement – Gridline G – Location 3C		
21.2.2	$\boldsymbol{Location\ 3C}$: $\boldsymbol{M_{u,int\,(-)\,CS} = 258\ k-ft}$ $\boldsymbol{(\gamma_f M_{sc} = 37.2\ k-ft)}$ $Assume\ tension - controlled\ section\ to\ start: \varphi = 0.90$ $m = \dfrac{f_y}{0.85 \times f'_c} = 18.82$ $R_u = \dfrac{M_u}{\varphi b d^2}$ $\rho_{reqd} = \dfrac{1}{m} \times \left(1 - \sqrt{1 - \dfrac{2mR_u}{f_y}}\right)$	
8.6.1.1	$M_u = 258\ k-ft$ $b_{col\ strip} = 138\ inches$ $R_u = 0.277\ ksi$ $\rho_{reqd} = 0.0036$ $A_{s,reqd} = 0.0036 \times 138" \times 9.5" = 4.72\ in^2 \leftarrow$ $A_{s,min} = 0.0018 \times A_g = 0.0018 \times 138" \times 11" = 2.74\ in^2$ $\boldsymbol{A_{s,reqd-total} = 4.72\ in^2}$	
8.4.2.2.3 8.4.2.2.5	$M_u = 37.2\ k-ft$ $b_{transfer} = 18" + 1.5(11") + 1.5(11") = 51\ inches$ $R_u = 0.108\ ksi$ $\rho_{reqd} = 0.0014 < \rho_{min} = 0.0018 \leftarrow use\ \rho_{min}\ on\ gross\ area.$ $\boldsymbol{A_{s,reqd-transfer} = 0.0018 \times 51" \times 11" = 1.01\ in^2}$	

ACI 318-19	Calculations	Description
STEP 11-3C: Required Flexural Reinforcement – Gridline G – Location 3C (continued)		
8.6.1.2 8.7.2.2	$Determine\ minimum\ total\ number\ of\ \#5\ top\ bars\ in\ column\ strip.$ $\frac{4.72\ in^2}{0.31\ in^2} = 15.23\ bars \rightarrow \boldsymbol{use\ sixteen\ (16)\ \#5\ bars}$ $Determine\ minimum\ total\ number\ of\ \#5\ top\ bars\ in\ transfer\ width.$ $\frac{1.01\ in^2}{0.31\ in^2} = 3.26\ bars \rightarrow \boldsymbol{use\ four\ (4)\#5\ bars}\ **$ $**\ Check\ potential\ for\ flexure\ driven\ punching\ failure.$ $At\ Interior\ Column:$ $Check\ v_{uv}\ against\ \varphi 2\lambda_s\lambda\sqrt{f'_c}\ .$ $v_{uv} = 0.177\ ksi\ > 2\lambda_s\lambda\sqrt{f'_c} = 2 \times 1.0 \times 1.0 \times 70.71 = 0.141\ ksi$ $\therefore\ Must\ provide\ A_{s,min}\ to\ resist\ flexure\ driven\ punching\ shear!$ $A_{s,min} = \frac{5v_{uv}b_{slab}b_o}{\varphi\alpha_s f_y} = \frac{5 \times 0.177\ ksi \times 51" \times 109.5"}{0.75 \times 40 \times 80\ ksi} = 2.06\ in^2$ $\frac{2.06\ in^2}{0.31\ in^2} = 6.65\ bars \rightarrow \boldsymbol{use\ eight\ (8)\ \#5\ bars\ in\ lieu\ of\ four}.$ $Maximum\ Spacing\ (s) = lesser\ of\ 2h\ and\ 18\ inches$ $2h = 2 \times 11 = 22, \therefore\ use\ 18"$	The design professional must calculate the required steel area with due consideration for potential flexure-driven punching failure, and also must maintain prescriptive maximum bar spacing. Flexure-driven punching shear confirmed be a limit state.

ACI 318-19	Calculations	Description
STEP 11-3C: Required Flexural Reinforcement – Gridline G – Location 3C (continued)		
   21.2 22.2 21.2 22.2	$\boldsymbol{Total\ Top\ Reinforcement} = (16)\ \#5$ $b = 138"$ $d = 9.5"$ $\varphi M_n = 0.9 A_s f_y (d - a/2) = 305\ k - ft > 279\ k - ft$ $\boldsymbol{Transfer\ Top\ Reinforcement} = < 8 >\ \#5$ $b = 51"$ $d = 9.5"$ $\varphi M_n = 0.9 A_s f_y (d - a/2) = 135\ k - ft > 37.2\ k - ft$ $\therefore Design\ flexural\ strengths\ confirmed.\ Bar\ quantities\ acceptable.$ $See\ Figure\ 13\ for\ requirement\ shown\ on\ contract\ drawing.$	Close the loop on the design flexural strength check for Location 3C. Note that any potential contribution by compression reinforcement is ignored.

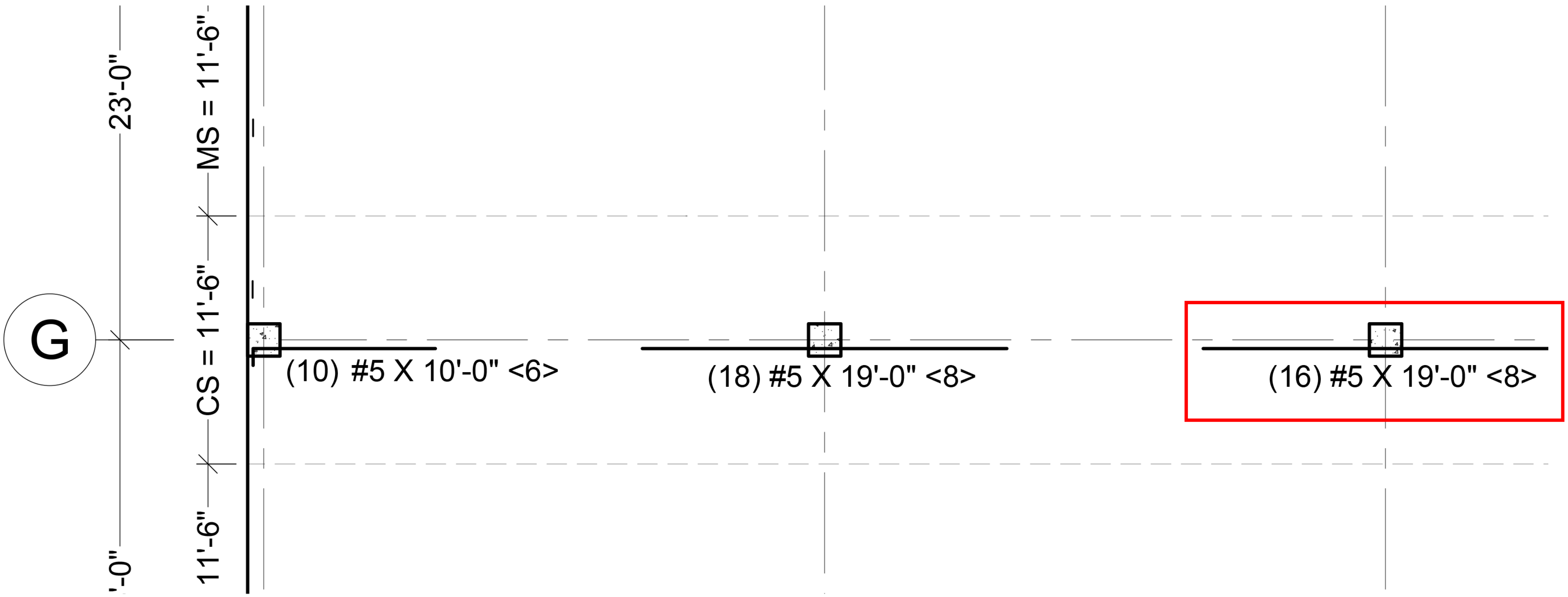


Figure 13: Representative Excerpt of Reinforcement Information as Presented on Floor Plan in Contract Drawings – Location 3C

ACI 318-19	Calculations	Description
STEP 11-4C: Required Flexural Reinforcement – Gridline G – Location 4C		
21.1.2 21.2.2 8.6.1.1	$\boldsymbol{Location\ 4C}$: $\boldsymbol{M_{u\,(+)\,CS} = 165\ k-ft}$ $b_{col\ strip} = 138\ inches$ $m = 18.82$ $R_u = 0.177\ ksi$ $\rho_{reqd} = 0.0023$ $A_{s,reqd} = 0.0023 \times 138" \times 9.5" = 3.02\ in^2 \leftarrow$ $A_{s,min} = 0.0018 \times A_g = 0.0018 \times 138" \times 11" = 2.74\ in^2$ $\boldsymbol{A_{s,reqd-total} = 3.02\ in^2}$ $Determine\ minimum\ total\ number\ of\ \#5\ bottom\ bars\ in\ column\ strip.$ $\frac{3.02\ in^2}{0.31\ in^2} = 9.74\ bars \rightarrow use\ ten\ (10)\ \#5\ bars \rightarrow\ A_{s,prov} = 3.10\ in^2$ $\boldsymbol{Total\ Bottom\ Reinforcement} = (10)\ \#5, b = 138", d = 9.5"$ $\varphi M_n = 0.9 A_s f_y (d - a/2) = 173\ k-ft > 165\ k-ft$	For column strip bottom bars, the process of design is simplified considerably given there is no distribution of reinforcement required within a transfer width.
8.7.2.2	$Maximum\ Spacing\ (s) = lesser\ of\ 2h\ and\ 18\ inches$ $2h = 2 \times 11 = 22, \therefore use\ 18"$	
8.7.4.2.2	$\frac{138" - 18" - 4" - 4"}{2} = 56\ inches\ to\ each\ side\ of\ column\ to\ last\ bar$ $\frac{10\ total\ bars - 2\ core\ bars}{2} = 4\ bars\ each\ side\ of\ column$ $\frac{56"}{3\ spaces} = \sim 18.67"\ spacing$ $\boldsymbol{\therefore Use\ twelve\ (12)\ \#5\ bars \rightarrow\ A_{s,prov} = 3.72\ in^2.\ \ See\ Figure\ 14.}$	ACI 318 requires a minimum of two (2) column strip bottom bars to pass through the core of the column. Carry out a cursory check to ensure that the number of bars selected will facilitate these core bars without exceeding the prescriptive maximum spacing. A quick check shows that the provision of ten (10) bars will be cutting it close. Increase to twelve (12) #5 bars to reduce spacing. Tensile strain in bars is still 0.042 in/in (calculation not shown), confirming tension-controlled behavior still applies.

ACI 318-19	Calculations	Description
STEP 11-5C: Required Flexural Reinforcement – Gridline G – Location 5C		
	$Location\ 5C\text{:}\ M_{u\,(+)\,CS} = 112\ k - ft$ $\therefore$ *Use twelve (12) #5 bars* $\rightarrow A_{s,prov} = 3.72\ in^2$. *See Figure 14.*	For column strip bottom bars at location 5C, the flexural demand is lower than what was previously determined to be necessary at location 4C. Moreover, it was shown at 4C that a bar quantity driven by prescriptive maximum spacing was the governing consideration. As such, at 5C, we will use the same reinforcement as previously defined for 4C.

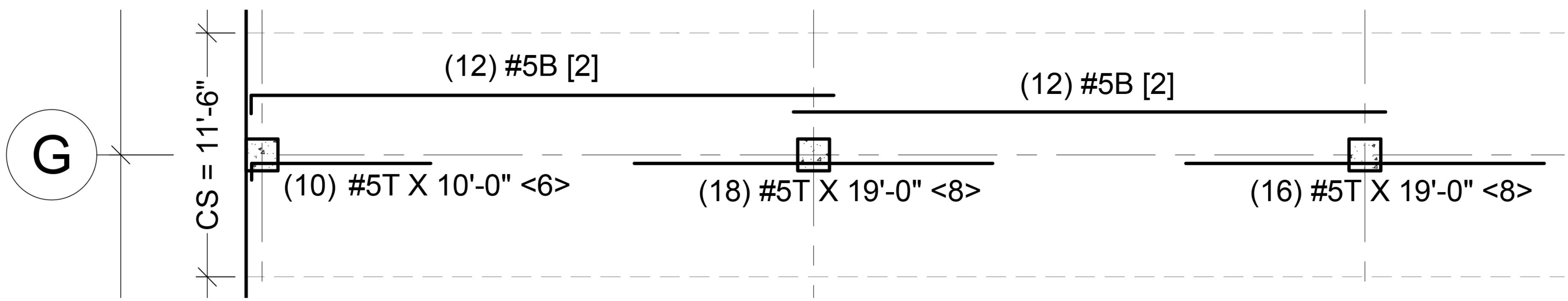


***Figure 14:** Representative Excerpt of Reinforcement Information as Presented on Floor Plan in Contract Drawings – Location 4C and 5C included. Note the "T" and "B" designations for top and bottom, respectively. Also note the angle brackets <…> as indication of the portion of total top bars positioned within the transfer width, and square brackets […] indicating portion of total bottom bars running through the column core.*

ACI 318-19	Calculations	Description
STEP 12: Required Flexural Reinforcement – Gridline G – Middle Strip		
8.6.1.1	$\boldsymbol{Location\ 2M:\ M_{u,int\,(-)\,MS} = 93\ k - ft}$ $Moment\ on\ half\ MS: 46.5\ k - ft$ $Half\ MS\ dimension = 69\ inches$ $Effective\ depth\ d = 9.5\ inches$ $m = 18.82$ $A_{s,calc} = 0.0013 \times 69 \times 9.5 = 0.852\ in^2$ $A_{s,min} = 0.0018 \times 69 \times 11 = 1.37\ in^2 \leftarrow$ $\frac{1.37\ in^2}{0.31\ in^2} = 4.41 \rightarrow use\ five\ (5)\ \#5\ bars$ $Quick\ spacing\ check: \frac{69" - 4" - 4"}{4\ spaces} = 15.25 < 18"\ \text{ok}$ $\boldsymbol{Use\ five\ (5)\ \#5\ bars\ per\ half\ middle\ strip}$ $\boldsymbol{Location\ 3M:\ M_{u\,,int\,(-)\,MS} = 86\ k - ft}$ $\boldsymbol{Use\ five\ (5)\ \#5\ bars\ per\ half\ middle\ strip}$	For middle strip reinforcement design, it is important to remember that the moments indicated are to be distributed between half middle strips to each side of the column strip. Positioning of bars/wires in middle strips is considerably more straightforward than for column strips; limitations such as minimum steel within a transfer width and minimum bars through a column core are not applicable. Note that maximum spacing requirements in Section 8.7.2.2 and minimum steel requirements in Section 8.6.1.1 are still applicable.
8.6.1.1	$\boldsymbol{Location\ 4M:\ M_{u\,(+)\,MS} = 110\ k - ft}$ $Moment\ on\ half\ MS: 55\ k - ft$ $Half\ MS\ dimension = 69\ inches$ $Effective\ depth\ d = 9.5\ inches$ $A_{s,calc} = 0.0015 \times 69 \times 9.5 = 0.983\ in^2$ $A_{s,min} = 0.0018 \times 69 \times 11 = 1.37\ in^2 \leftarrow$ $\boldsymbol{Use\ five\ (5)\ \#5\ bars\ per\ half\ middle\ strip}$ $\boldsymbol{Location\ 5M:\ M_{u\,(+)\,MS} = 75\ k - ft}$ $\boldsymbol{Use\ five\ (5)\ \#5\ bars\ per\ half\ middle\ strip}$ $\boldsymbol{Location\ 1M:\ M_{u,ext\,(-)\,MS} = 0\ k - ft}$ $\boldsymbol{Use\ five\ (5)\ \#4\ bars\ per\ half\ middle\ strip}$ $\boldsymbol{See\ Figure\ 15\ for\ completed\ reinforcement\ layout.}$	While DDM analysis indicates no flexural demand at exterior middle strip locations, we will still provide a nominal amount of reinforcement for slab edge durability.

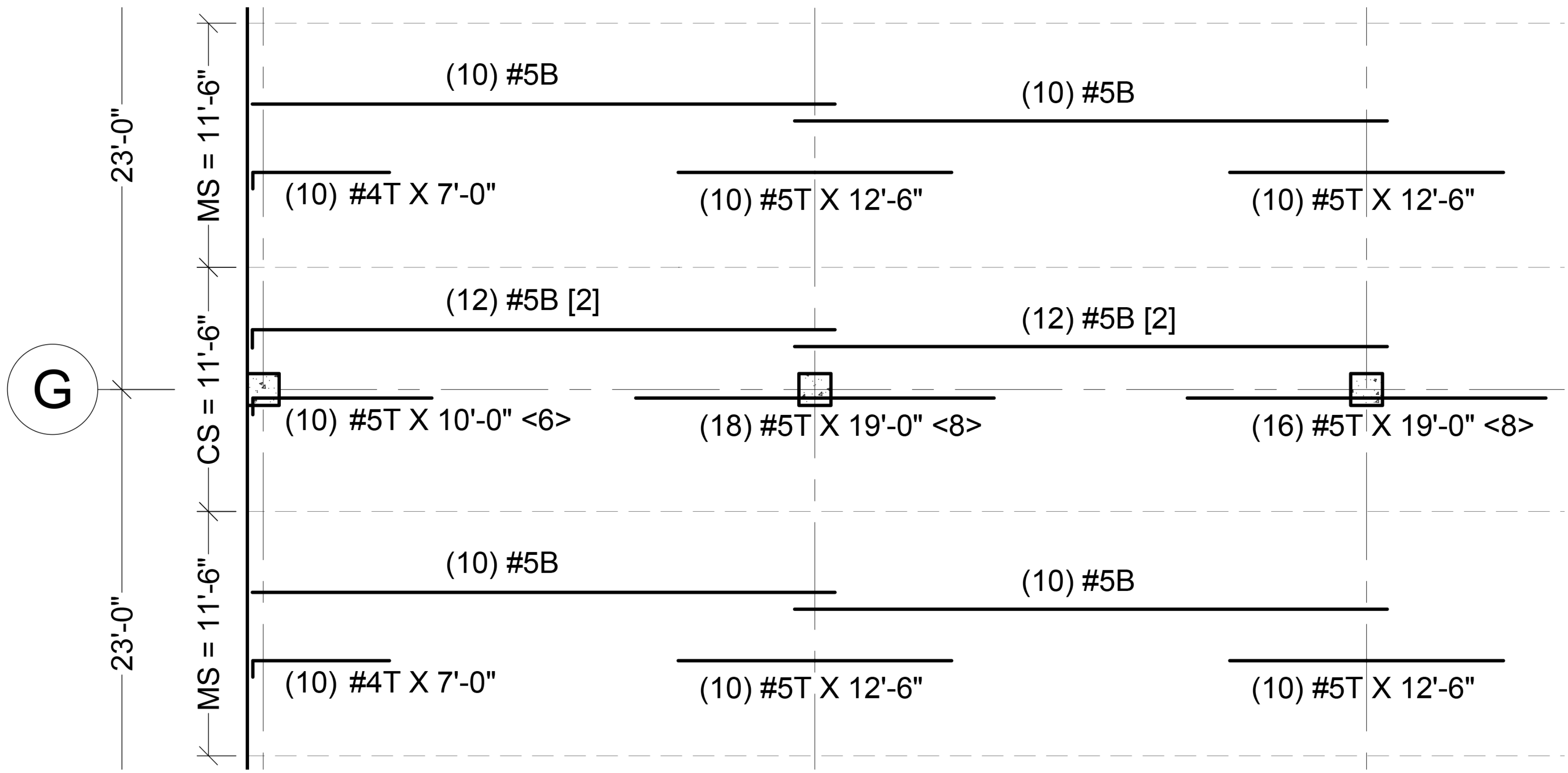
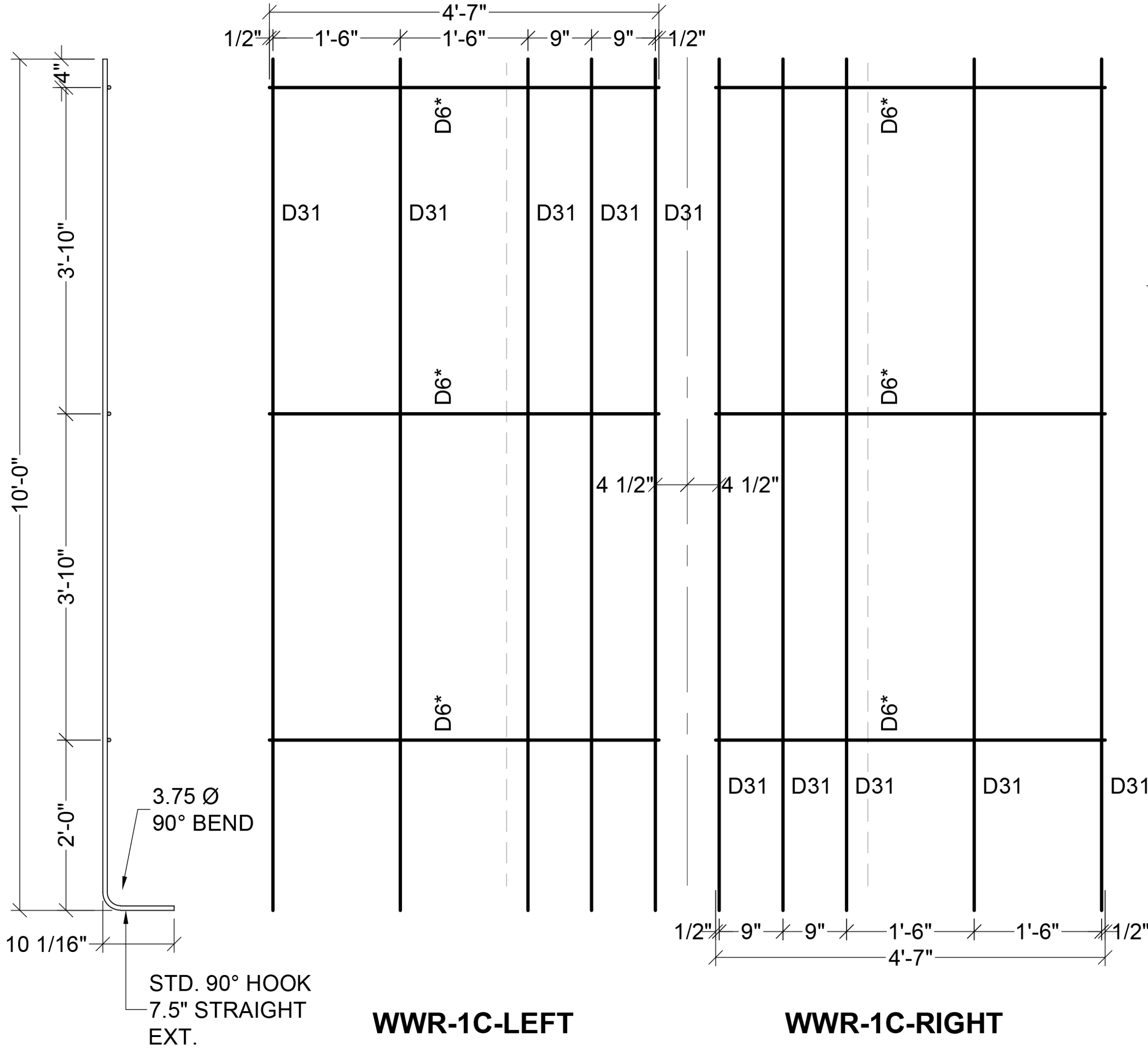


Figure 15: Representative Excerpt of Reinforcement Information as Presented on Floor Plan in Contract Drawings – CS and MS shown. Note that MS bar quantities are doubled that derived from calculations for half middle strips.

ACI 318-19	Calculations	Description
STEP 13: WWR Detailing for Gridline G Reinforcement		
		For the preparation of a WWR submittal, the WWR detailer will require access to the structural contract drawings, drawing files, and specifications, no different than what would be expected of a conventional loose deformed bar detailer. As it relates to this example, representative contract drawing information was presented as follows: Figure 6: Headed Shear Stud Information Figure 9: Typical Two-Way Slab Reinforcing Details Figure 10: Two-Way Slab Notes Figure 15: Two-Way Slab Plan Information
		The WWR mat configurations shown here are intended to serve as a general representation of manufacturer capabilities throughout North America. With that said, reinforcement arrangements presented on a submittal by one manufacturer's detailer may vary slightly from those presented by another. Despite this, there exists a common and consistent responsibility for the WWR detailer to ensure that detailed WWR arrangements conform to the reinforcement cross-sectional area, placement, and curtailment requirements established by the design professional, unless explicitly-defined exceptions are made by the design professional of record on a project by project basis. Refer to Figure 16 through 25 for WWR mat configurations, prepared as a representation of equal reinforcement to that originally specified as deformed reinforcing bars by the design professional.



PLACER NOTES:

1. PLACE MATS INDICATED AS PAIRS, WITH OFFSET DIMENSIONS FROM COLUMN CENTERLINE INDICATED.
2. NON-STRUCTURAL HOLDING WIRES DENOTED WITH ASTERISK.
3. PLACER IS PERMITTED TO FIELD-CUT NON-STRUCTURAL HOLDING WIRES AS REQUIRED.
4. HOLDING WIRES POSITIONED TO AVOID STUD RAILS AND OTHER COMPONENTS.
5. DASHED LINE INDICATED COLUMN STRIP TRANSFER WIDTH.

WWR-1C-LEFT
V1 X V2 D31/D6 54" (+0.5",+0.5) X 10'-8 1/2" (32.5, 4)
V1 = 0.5 OH, 2@18, 2@9, 0.5 OH
V2 = 32.5 OH, 2@46, 4 OH

WWR-1C-RIGHT
V1 X V2 D31/D6 54" (+0.5",+0.5) X 10'-8 1/2" (32.5, 4)
V1 = 0.5 OH, 2@9, 2@18, 0.5 OH
V2 = 32.5 OH, 2@46, 4 OH

Figure 16: Reinforcement for location 1C is shown here as a two-mat assembly. Modern WWR welding machines can produce mats in widths up to 12’-0”, so it would be possible for the configuration shown to be combined into a single mat. This option is not used, however, due to the transport ramifications: 8’-6” is the default maximum width that can be shipped on a flatbed without additional cost associated with “oversize” load.

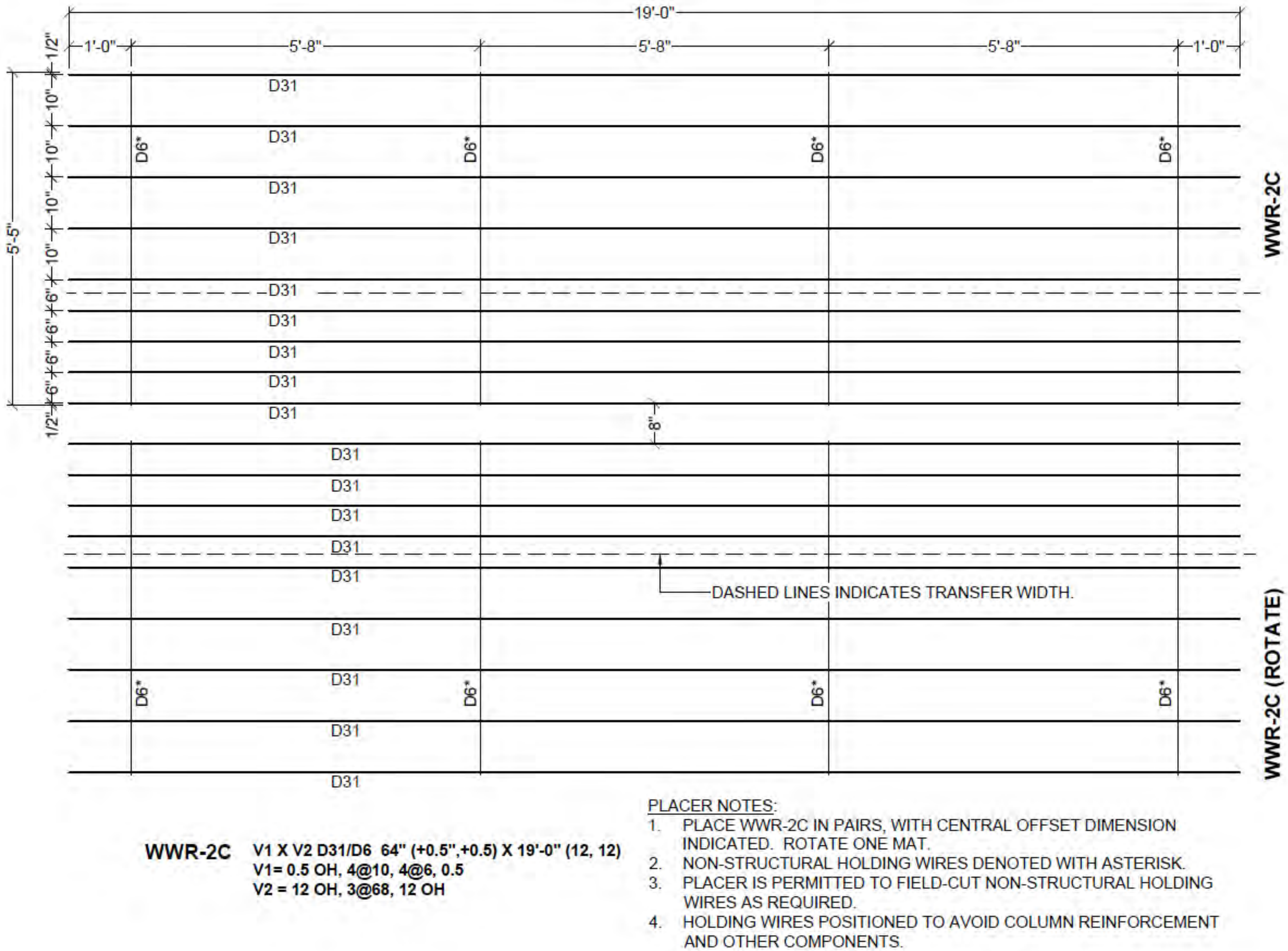
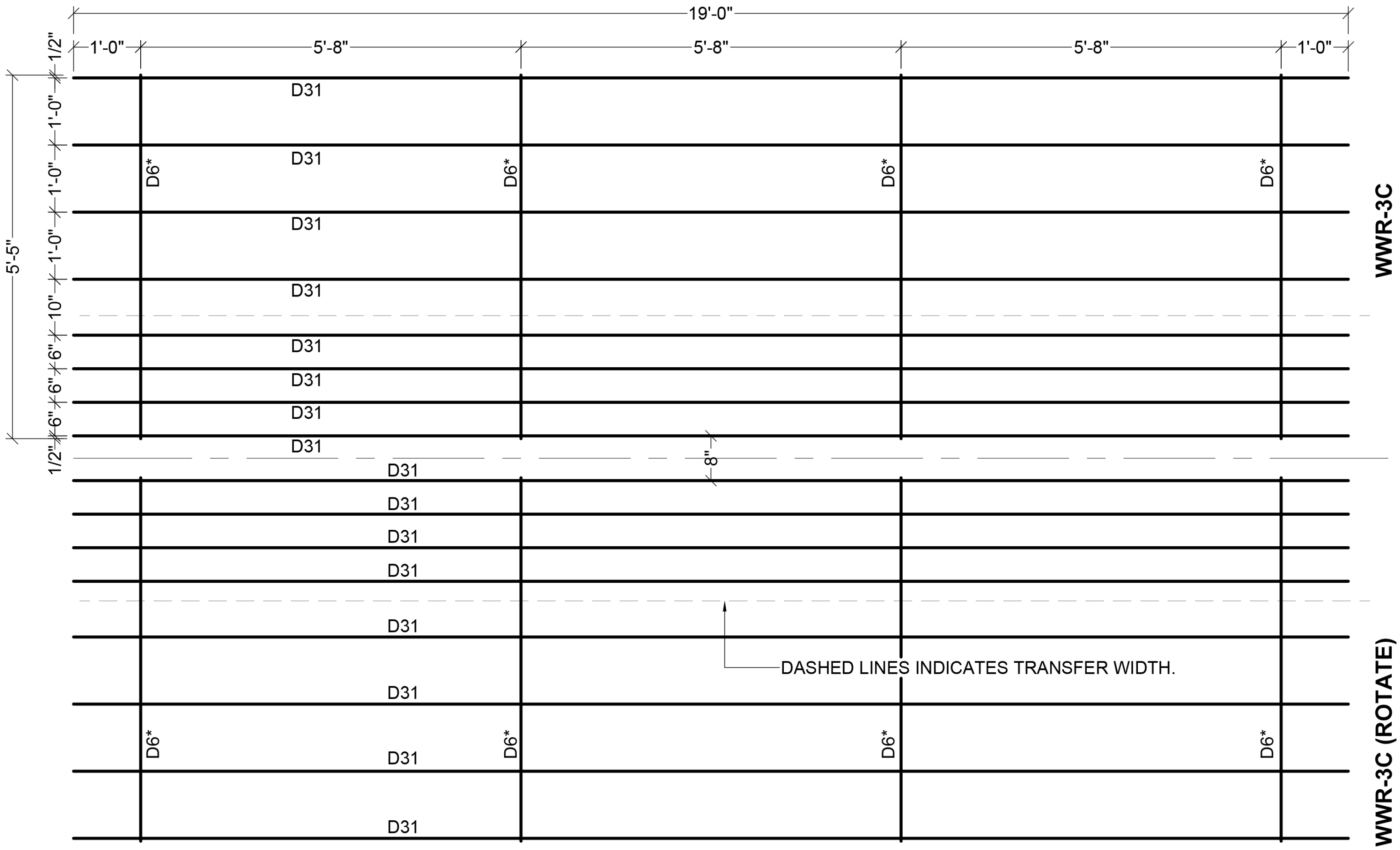


Figure 17: Reinforcement for location 2C is shown here. Two (2) WWR-2C mats are required at each location, one of which would be rotated end-for-end by the placing crew to ensure presence of a minimum of eight <8> wires within the column strip transfer width.

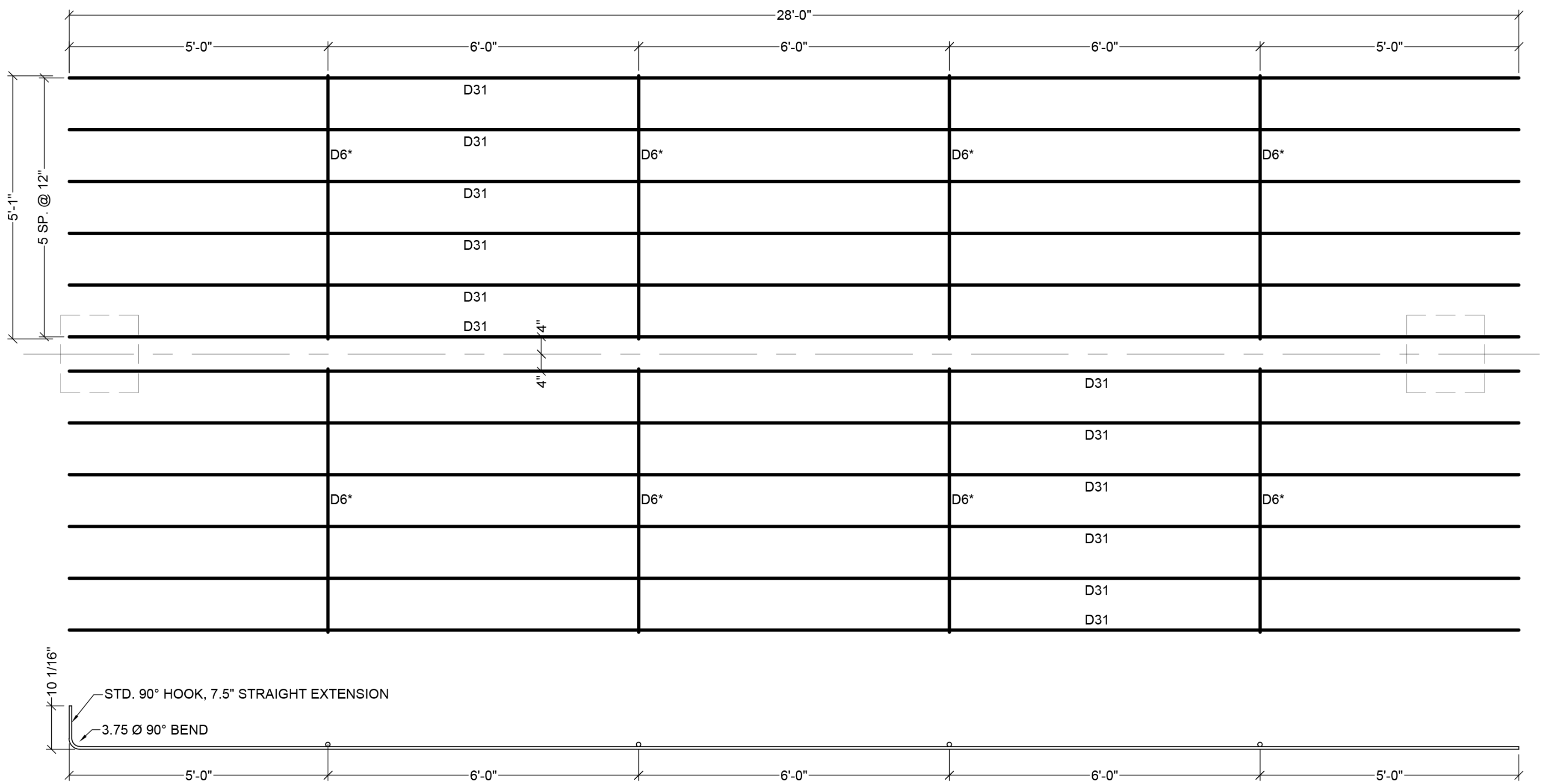


WWR-3C **V1 X V2 D31/D6 64" (+0.5",+0.5) X 19'-0" (12, 12)**
V1= 0.5 OH, 3@12, 10, 3@6, 0.5
V2 = 12 OH, 3@68, 12 OH

PLACER NOTES:

1. PLACE WWR-3C IN PAIRS, WITH CENTRAL OFFSET DIMENSION INDICATED. ROTATE ONE MAT.
2. NON-STRUCTURAL HOLDING WIRES DENOTED WITH ASTERISK.
3. PLACER IS PERMITTED TO FIELD-CUT NON-STRUCTURAL HOLDING WIRES AS REQUIRED.
4. HOLDING WIRES POSITIONED TO AVOID COLUMN REINFORCEMENT AND OTHER COMPONENTS.

Figure 18: Reinforcement for location 3C is shown here. Two (2) WWR-3C mats are required at each location, one of which would be rotated end-for-end by the placing crew to ensure presence of a minimum of eight <8> wires within the column strip transfer width.

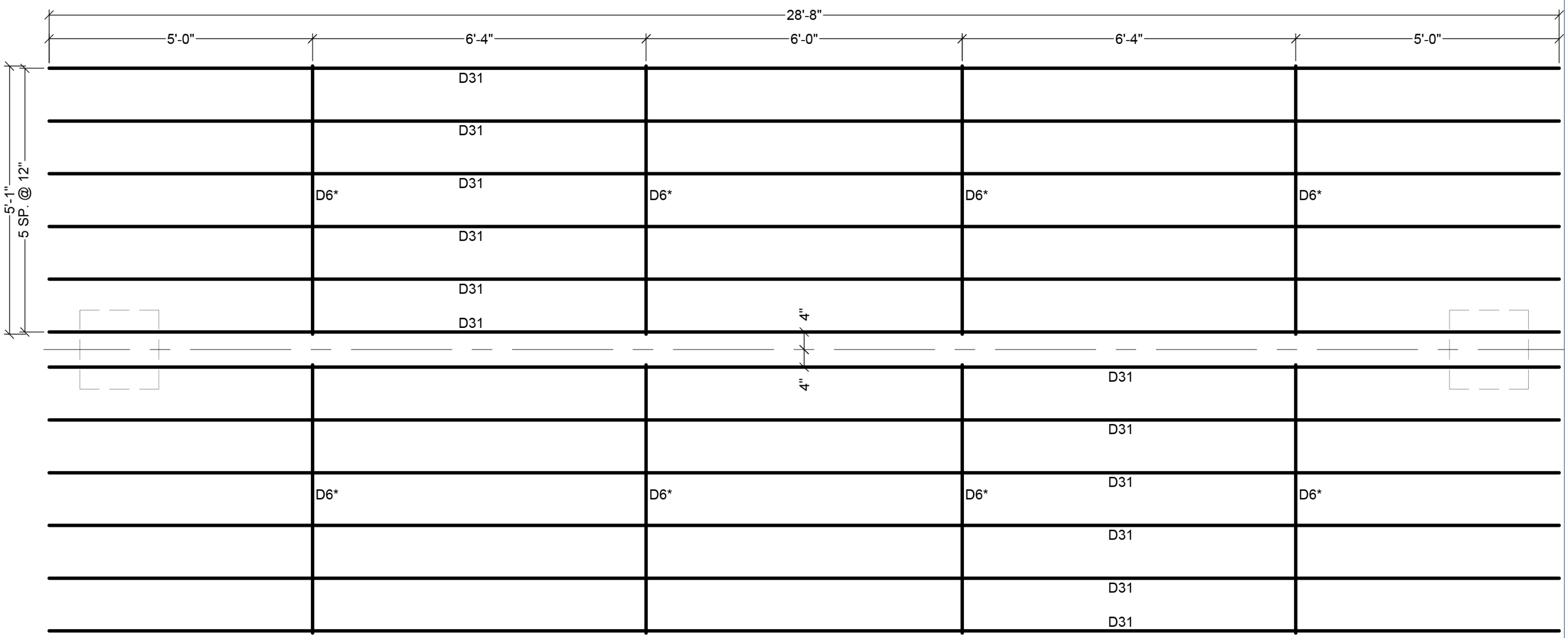


WWR-4C 12 X V2 D31/D6 60" (+0.5",+0.5) X 28'-8 1/2" (68.5, 60)
V2 = 68.5 OH, 3@72, 60 OH

PLACER NOTES:

1. PLACE WWR-4C IN PAIRS, WITH CENTRAL OFFSET DIMENSION INDICATED.
2. NON-STRUCTURAL HOLDING WIRES DENOTED WITH ASTERISK.
3. PLACER IS PERMITTED TO FIELD-CUT NON-STRUCTURAL HOLDING WIRES AS REQUIRED.
4. HOLDING WIRES POSITIONED TO AVOID COLUMN REINFORCEMENT AND OTHER COMPONENTS.

Figure 19: Reinforcement for location 4C is shown here. Two (2) WWR-4C mats are required at each location. Positioning of mats is such that a minimum of two wires pass through the column cores. Mat is hooked at one end to suit slab edge termination, while the overall mat length is such that it achieves a minimum 32" Class B lap splice with the adjoining 5C mats over the first interior column.

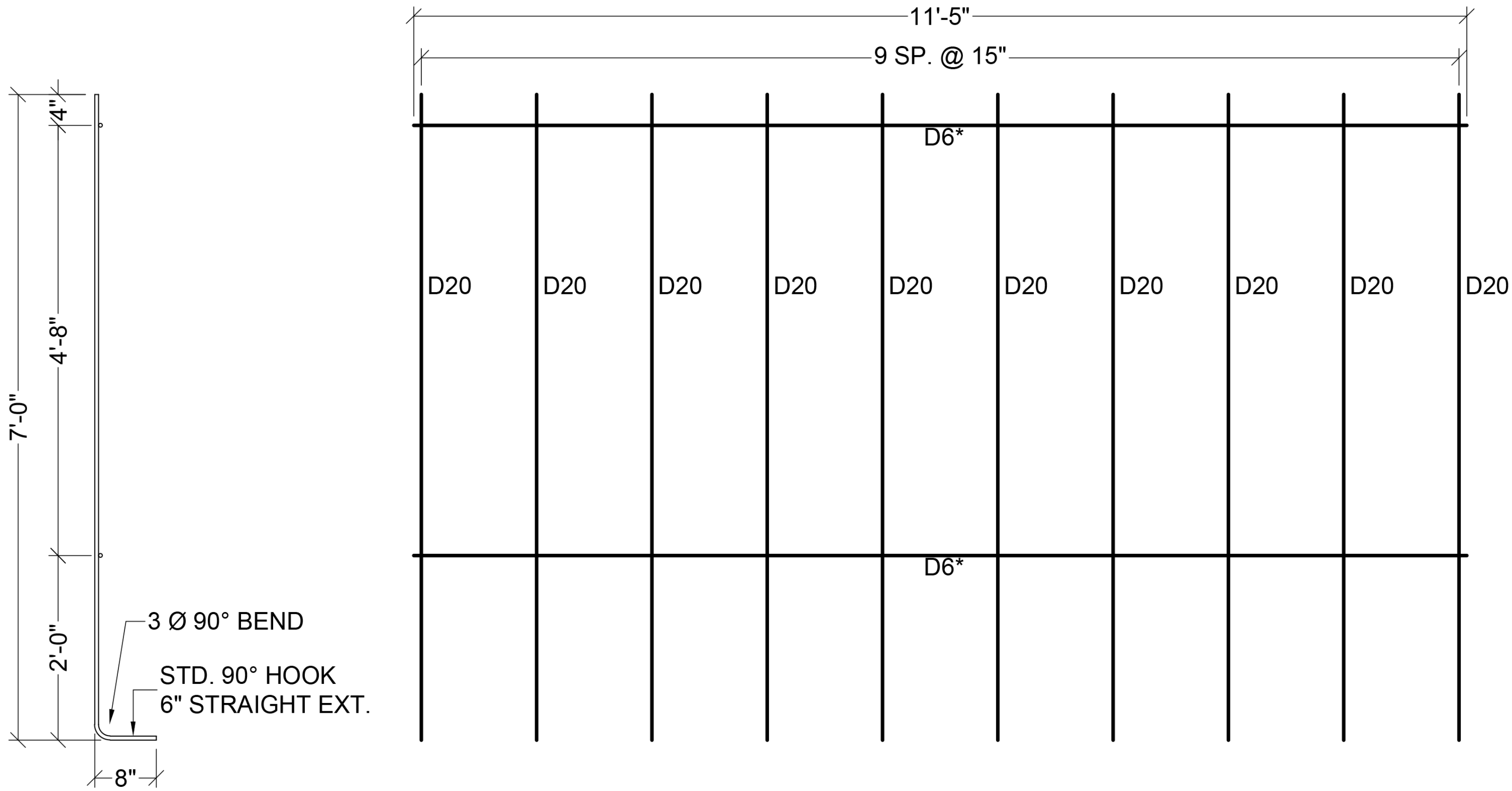


WWR-5C **12 X V2 D31/D6 60" (+0.5",+0.5) X 28'-8" (60, 60)**
V2 = 60 OH, 76, 72, 76, 60 OH

PLACER NOTES:

1. PLACE WWR-5C IN PAIRS, WITH CENTRAL OFFSET DIMENSION INDICATED.
2. NON-STRUCTURAL HOLDING WIRES DENOTED WITH ASTERISK.
3. PLACER IS PERMITTED TO FIELD-CUT NON-STRUCTURAL HOLDING WIRES AS REQUIRED.
4. HOLDING WIRES POSITIONED TO AVOID COLUMN REINFORCEMENT AND OTHER COMPONENTS.

Figure 20: Reinforcement for location 5C is shown here. Two (2) WWR-4C mats are required at each location. Positioning of mats is such that a minimum of two wires pass through the column cores. Overall mat length is such that it achieves a minimum 32" Class B lap splice at each end over columns.

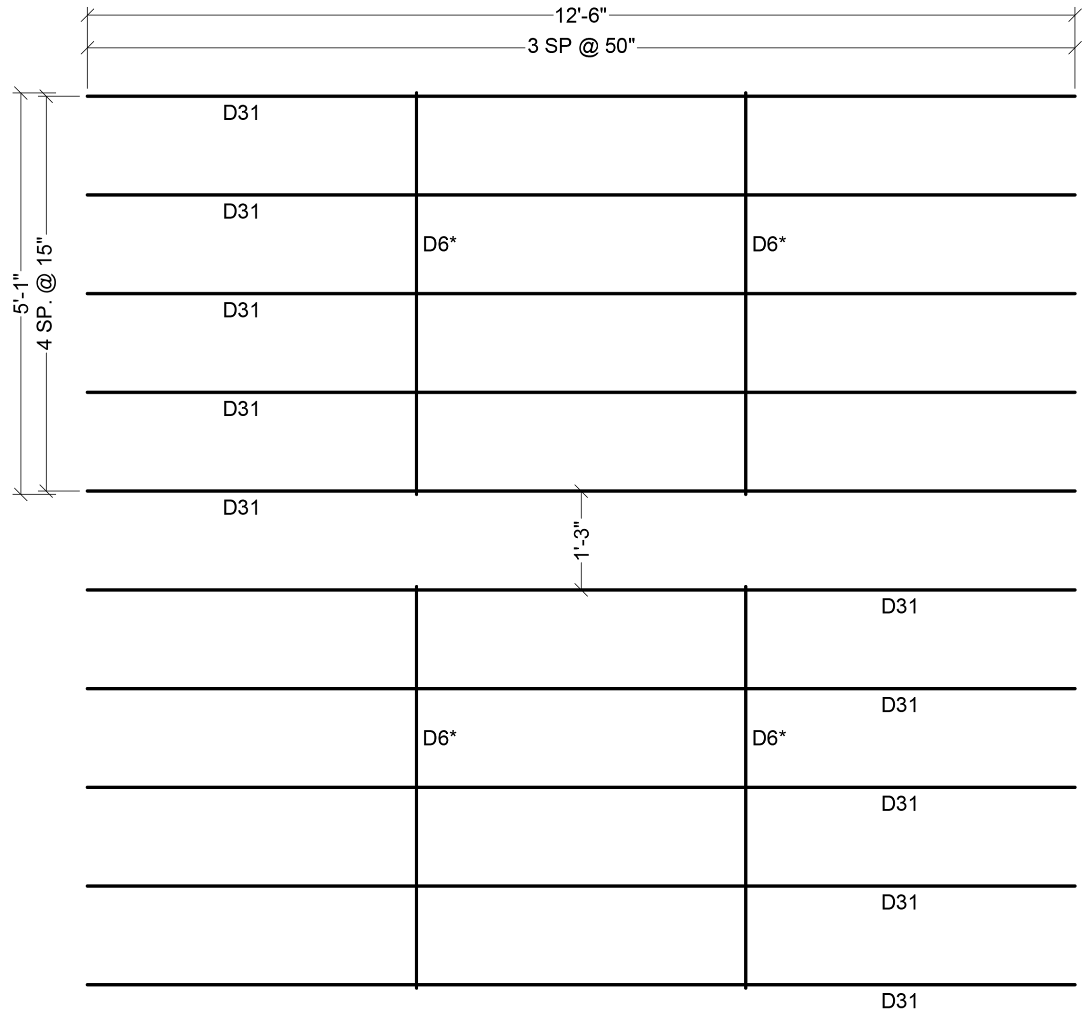


WWR-1M **56 X 15 D6/D20 56" (+4",+30.75") X 11'-5" (1, 1)**

PLACER NOTES:

1. NON-STRUCTURAL HOLDING WIRES DENOTED WITH ASTERISK.
2. PLACER IS PERMITTED TO FIELD-CUT NON-STRUCTURAL HOLDING WIRES AS REQUIRED.
3. HOLDING WIRES POSITIONED TO AVOID OTHER COMPONENTS.

Figure 21: Reinforcement for location 1M is shown here. Because of the relatively short length of the reinforcement specified by the design professional, the WWR detailer can have the mat configured as a single component.



WWR-2/3M **15 X 50 D31/D6 60" (+0.5",+0.5") X 12'-6" (50, 50)**

PLACER NOTES:

1. PLACE MATS AS PAIRS WITH CENTRAL OFFSET DIMENSION INDICATED.
2. NON-STRUCTURAL HOLDING WIRES DENOTED WITH ASTERISK.
3. PLACER IS PERMITTED TO FIELD-CUT NON-STRUCTURAL HOLDING WIRES AS REQUIRED.
4. HOLDING WIRES POSITIONED TO AVOID OTHER COMPONENTS.

Figure 22: Reinforcement for location 2M AND 3M is shown here.

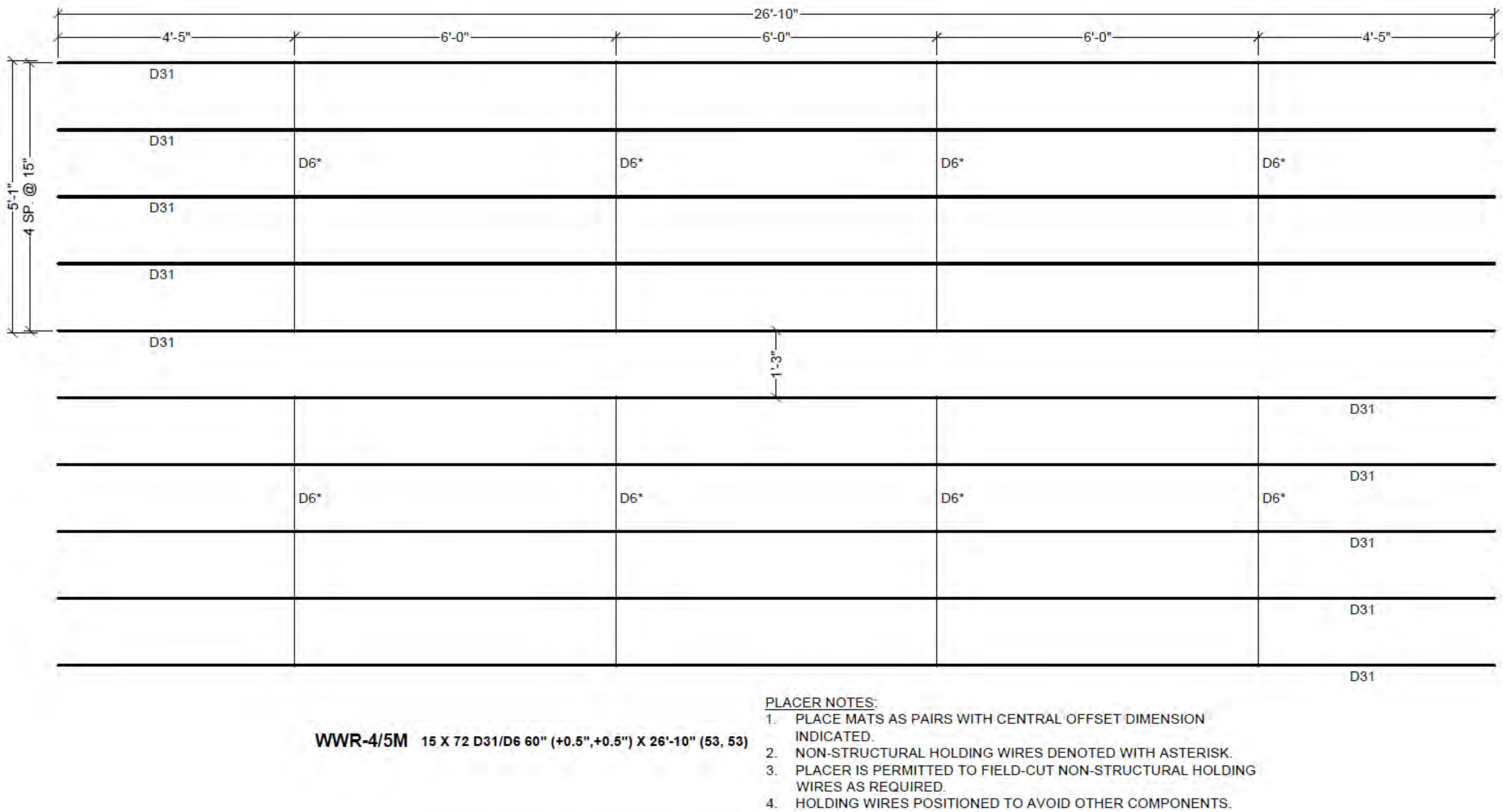


Figure 23: Reinforcement for location 4M AND 5M is shown here. Theoretically, the length shown corresponds to that required for the 4M location (26 feet plus 7" extension past column centerline at slab edge plus 3" extension over interior column centerline). This same length is appropriate for the 5M location (26 feet plus 3" extension past column centerline at each end) as well, so a single length will be used for both, resulting in economy of repetition.

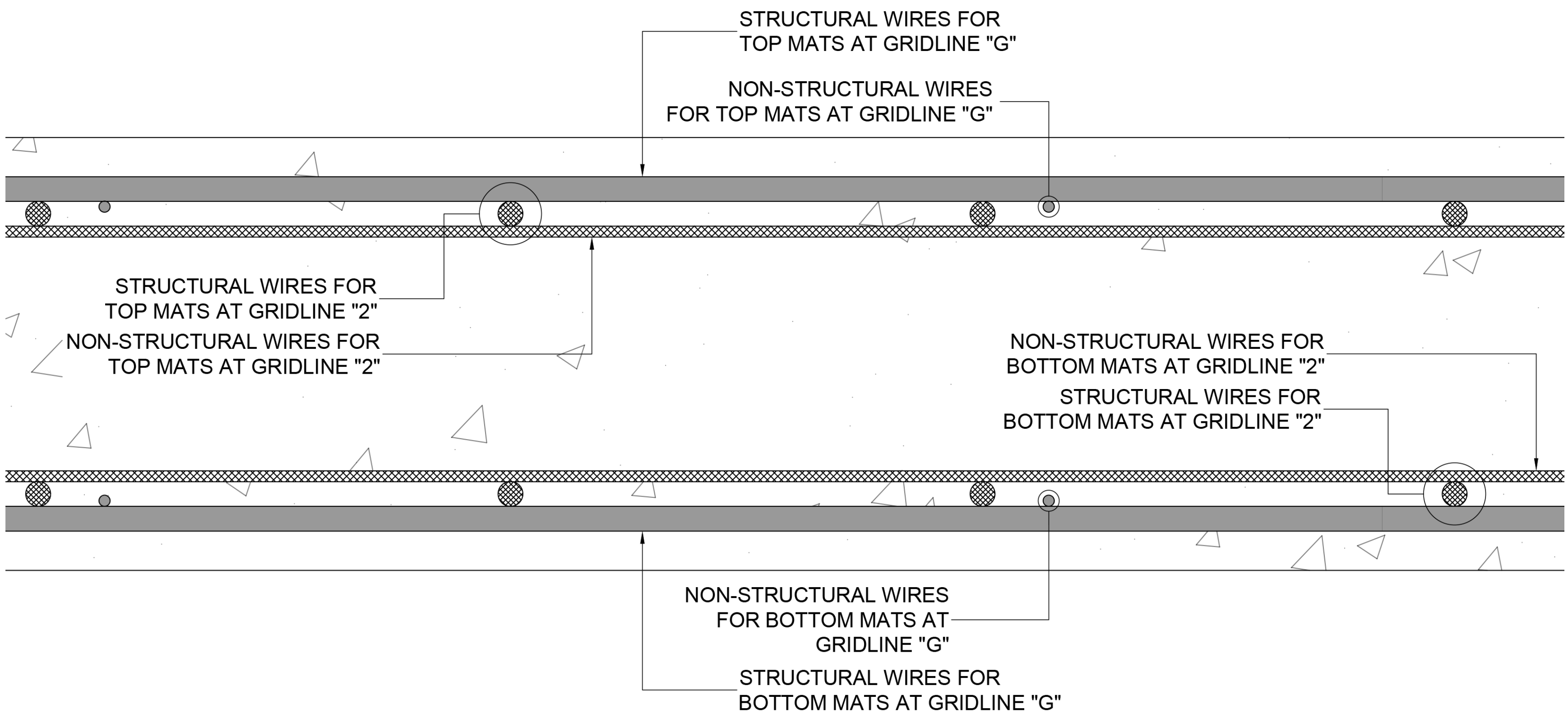


***Figure 24:** Typical detail of relative positioning of WWR mats. This example included design and detailing of Gridline G reinforcement only. These mats would have their structural wires nearest the top and bottom surfaces of the slab, with the non-structural holding wires towards the slab center. Positioning of reinforcement perpendicular to Gridline G (i.e., along Gridline 2 and those gridlines parallel to Gridline 2), would be nested to the inside of the Gridline G mats, again with their structural wires towards the slab top and bottom surfaces and non-structural wires towards the slab center. The WWR detailer is responsible for ensuring that positioning of non-structural holding wires – occurring co-planar with structural wires in the orthogonal direction – does not interfere with said structural wires.*

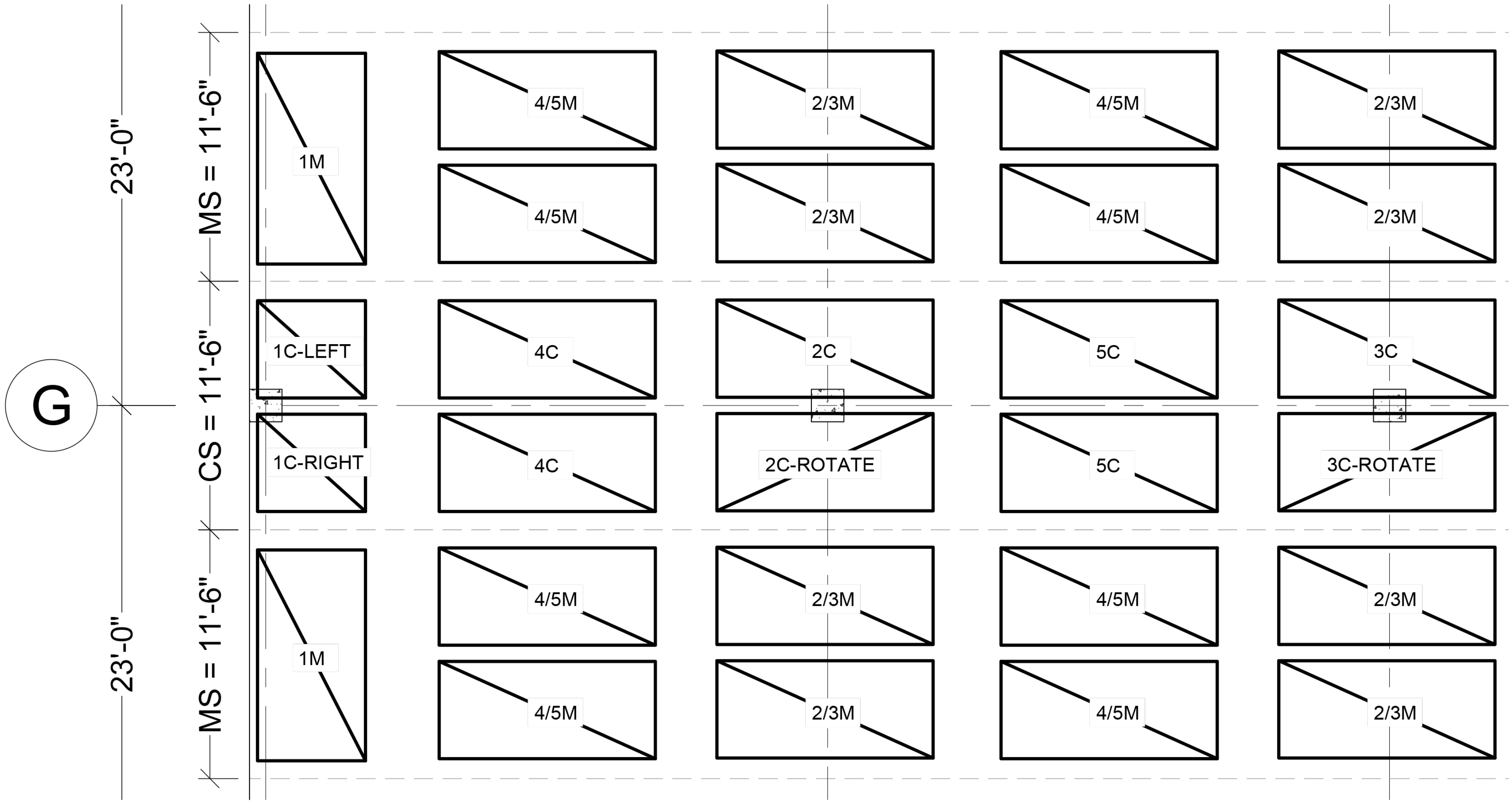


Figure 25: WWR mats are presented on the layout/placement drawing submitted for review by the design professional and contractor during the construction administration phase. The illustrated plan representations (intentionally not to scale to avoid clutter and confusion) combined with the individual WWR mat piece-mark drawings provide an intuitive reinforcement solution for specifier and field personnel alike. The WWR is made possible by the design professional's inclusive contract drawing language (Figure 10), allowing for a WWR solution to be presented as an option to that which is shown in the reinforcing bar-based design of Figure 15. <u>For clarification and transparency, the WWR shop/placement drawings will typically include submittal documentation that quantitatively supports the conversion of reinforcing bars over to welded wire reinforcement mats. For an example of this support documentation, refer to the Shallow Foundations example chapter of this guide.</u>