Structural Design with Welded Wire Reinforcement

Building Shallow Footings

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 318-25. 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

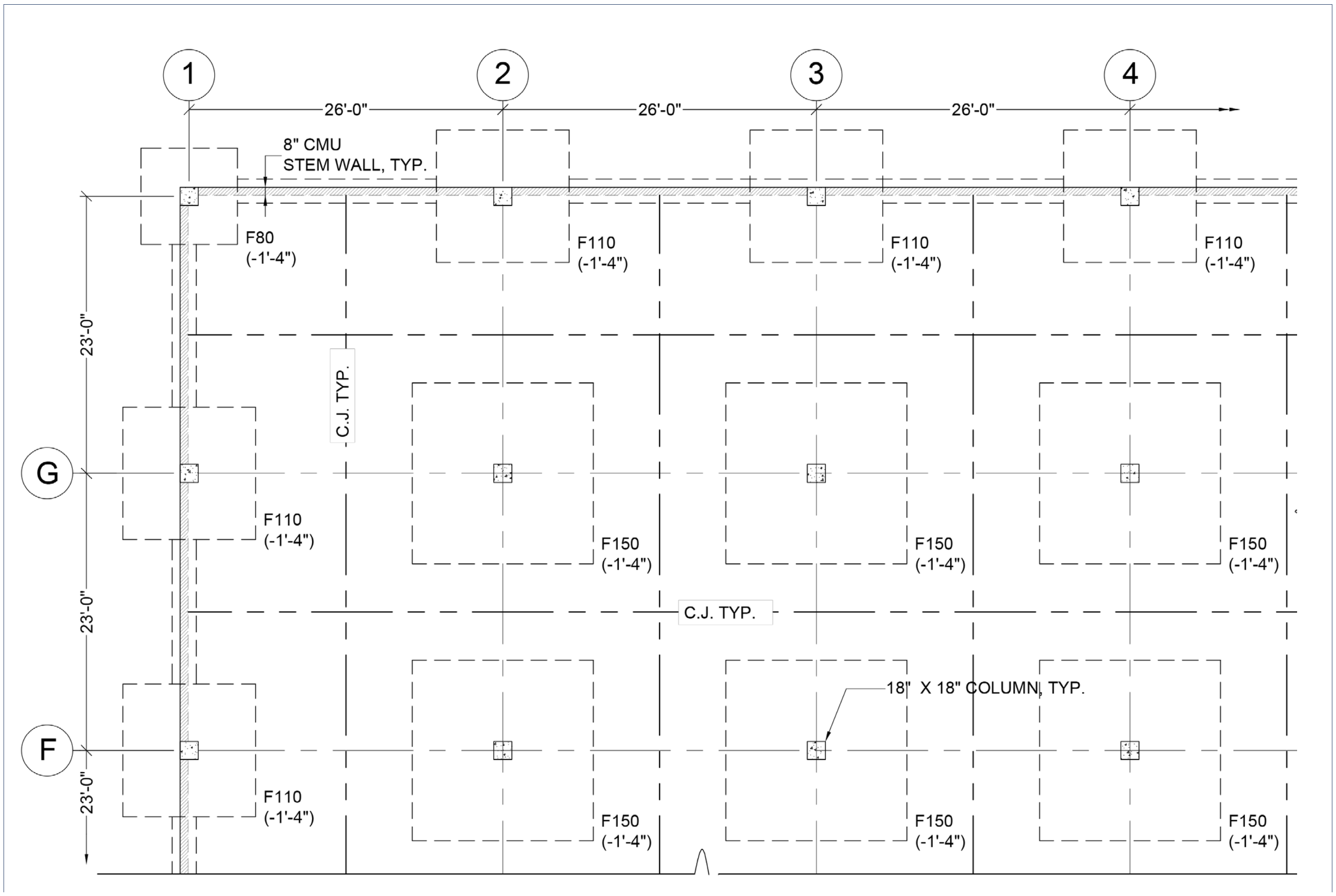


Figure 1: Partial Foundation Plan

ACI 318-25	Calculations	Description
STEP 1: Corner Spread Footing – Proportioning and Loading		
13.3.1.1 Equation 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:* $A_{ftg} = \frac{P_D + P_L}{q_a} = \frac{128\ kips}{2\ ksf} = 64\ ft^2$ $\sqrt{64} = 8$ $\therefore$ *Use 8 foot x 8 foot spread footing.* *The Designer selects 16 inches as trial footing thickness.* $U = 1.2D + 1.6L$ $P_u = 1.2 \times 84 + 1.6 \times 44 = 171\ kips$ $q_u = \frac{P_u}{A_{ftg}} = \frac{171}{64} = 2.67\ ksf \rightarrow$ *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 318-25	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.
8.5.1.1(d) 8.5.1.2 22.6.1.2 22.6.4.1	***Two Way Shear Design*** *Need* $\varphi v_n \geq v_u$ $\varphi = 0.75$ $v_n = v_c$ $d = 16" - 3" - 0.5" = 12.5"$ ***Critical Section Considered as Interior Column Condition*** $b_o = 4 \times (18" + 12.5"/2 + 12.5"/2) = 122"$ *Loaded tributary area* $= 8\ ft \times 8\ ft - \frac{30.5" \times 30.5"}{144} = 57.54\ sf$ $V_u = 2.67\ ksf \times 57.54\ sf = 182\ kips$ $v_{uv} = \frac{V_u}{b_o d} = \frac{182\ kips}{122" \times 12.5"} = 0.119\ ksi$	For two-way (punching) shear design, there will be no shear reinforcement. 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. 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 318-25	Calculations	Description
STEP 2: Corner Spread Footing – Two-Way Shear Design (continued)		
22.6.5.2 22.6.5.3 13.2.6.2	$v_c\ is\ the\ least\ of:$ $v_c = 4\lambda_s\lambda\sqrt{f'_c} = 4 \times 1.0 \times 1.0 \times \sqrt{4000} = 0.253\ ksi\ \leftarrow$ $v_c = \left(2 + \frac{4}{\beta}\right)\lambda_s\lambda\sqrt{f'_c} = 6 \times 1.0 \times 1.0 \times \sqrt{4000} = 0.379\ ksi$ $\beta\ is\ the\ ratio\ of\ column\ sides.\ \ \beta = 1\ for\ square\ column.$ $v_c = \left(2 + \frac{\alpha_s d}{b_o}\right)\lambda_s\lambda\sqrt{f'_c} = \left(2 + \frac{40 \times 12.5}{122}\right) \times 1 \times 1 \times \sqrt{4000} = 0.386\ ksi$ $\alpha_s = 40\ for\ interior\ columns$ $\therefore v_c = 0.253\ ksi$ $v_{uv} = 0.119\ ksi < 0.75 \times 0.253 = 0.190\ ksi$ $\varphi v_n \geq\ v_u\ is\ satisfied$ $\therefore \boldsymbol{Two\ way\ (punching)\ shear\ capacity\ is\ sufficient.}$	For two-way shear strength the size effect factor specified in 22.6 shall be $\lambda_s = 1.0$.

ACI 318-25	Calculations	Description
STEP 3: Corner Spread Footing – Flexural Design		
13.2.7.1	$For\ concentric\ axially\ loaded\ square\ footing:$ $M_u = q_u \times b \times \frac{(l-c)^2}{8}$ $b = 8\ feet\ (footing\ dimension\ perpendicular\ to\ flexure)$ $l = 8\ feet\ (footing\ dimension\ parallel\ to\ flexure)$ $c = 1.5\ feet\ (column\ dimension\ parallel\ to\ flexure)$ $M_u = 2.67\ ksf \times 8\ ft \times \frac{(6.5\ ft)^2}{8} = 113\ k-ft$	Calculate M_u at the face of the column.
21.2.2	$Assume\ tension-controlled\ section\ to\ start: \varphi = 0.90$ $m = \frac{f_y}{0.85 \times f'_c} = \frac{80{,}000}{0.85 \times 4{,}000} = 23.53$ $R_u = \frac{M_u}{\varphi b d^2} = \frac{113 \times 12}{0.9 \times 8 \times 12 \times 12.5^2} = 0.101\ ksi$ $\rho_{reqd} = \frac{1}{m} \times \left(1 - \sqrt{1 - \frac{2mR_u}{f_y}}\right) = 0.0013$	
8.6.1.1	$\rho_{reqd} = 0.0013$ $A_{s,reqd} = 0.0013 \times 96" \times 12.5" = \ 1.56\ in^2$ $A_{s,\min prescriptive} = 0.0018 \times A_g = 0.0018 \times 96" \times 16" = \ 2.77\ in^2$	

ACI 318-25	Calculations	Description
STEP 3: Corner Spread Footing – Flexural Design (continued)		
8.6.1.2 7.7.2.4 21.2 22.2	$Is\ v_{uv} = 0.119\ ksi > \varphi \times 2 \times \lambda_s \times \lambda \times \sqrt{f'_c}\,?$ $\varphi \times 2 \times \lambda_s \times \lambda \times \sqrt{f'_c} = 0.75 \times 2 \times 1.0 \times 1.0 \times \sqrt{4{,}000} = 0.095\ ksi$ $0.119\ ksi > 0.095\ ksi$ $\therefore\ Must\ consider\ flexure\ driven\ punching\ failure.$ $provide\ A_{s,\min FDPS}\ to\ resist\ flexure\ driven\ punching\ shear!$ $A_{s,\min FDPS} = \frac{5 v_{uv} b_{slab} b_o}{\varphi \alpha_s f_y} = \frac{5 \times 0.119\ ksi \times 96" \times 122"}{0.75 \times 40 \times 80\ ksi} = 2.90\ in^2 \leftarrow$ $Maximum\ spacing\ shall\ be\ the\ lesser\ of:$ • $2h = 2 \times 16\ inches = 32\ inches$ • $18\ inches \leftarrow$ ***The Design Professional selects (7) #6 reinforcing bars.*** $A_{s,prov} = 3.08\ in^2, (7)\ bars\ spaced\ @\ \pm 15"\ on\ center\ with\ 3"\ cover$ $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{3.08 \times 80}{0.85 \times 4 \times 96} = 0.76"$ $\varepsilon_t = \frac{0.003 \times \beta_1 \times d}{a} - 0.003 = \frac{0.003 \times 0.85 \times 12.5}{0.76} - 0.003 = 0.039\ in/in$ $\varepsilon_t = 0.039 >> \varepsilon_{ty} + 0.003 = 0.0058 \therefore Tension\ Controlled$ $\varphi M_n = 0.9 A_s f_y (d - a/2) = 224\ k - ft > 113\ k - ft$ $\therefore$ ***Flexural capacity is sufficient.***	The design professional must calculate the required flexural steel area with due consideration for potential flexure-driven punching failure and must also 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 in 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 318-25	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 ANDHORIZONTAL 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 318-25	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	$A_v = 0, \therefore A_v < A_{v,min}$ $V_c = \left[8\lambda_s\lambda({\rho_w}^{1/3})\sqrt{f'_c} + \frac{N_u}{6A_g}\right] \times b_w d \leq 5\lambda\sqrt{f'_c}b_w d$ $\rho_w = \frac{A_s}{b_w d} = \frac{3.08}{96 \times 12.5} = 0.0026$ $V_c = [8 \times 1.0 \times 1.0 \times 0.0026^{1/3} \times \sqrt{4000} + 0] \times 96 \times 12.5 = 83.5\ kips$	
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 318-25	Calculations	Description
STEP 4: Corner Spread Footing – One-Way Shear Design (continued)		
8.5.1.1	$Need\ \varphi V_n \geq V_u$ $0.75 \times 83.5\ kips = 62.6\ kips > V_u = 47\ kips\ \therefore OK$	
22.5.1.2	$Also\ need\ V_u \leq \varphi\left(V_c + 8\sqrt{f'_c}b_w d\right) \rightarrow 47\ kips < 518\ kips\ \therefore OK$ $\therefore$ ***One way shear capacity is sufficient.***	Check to confirm likelihood of diagonal compression failure is precluded and to control cracking under service load. **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.**

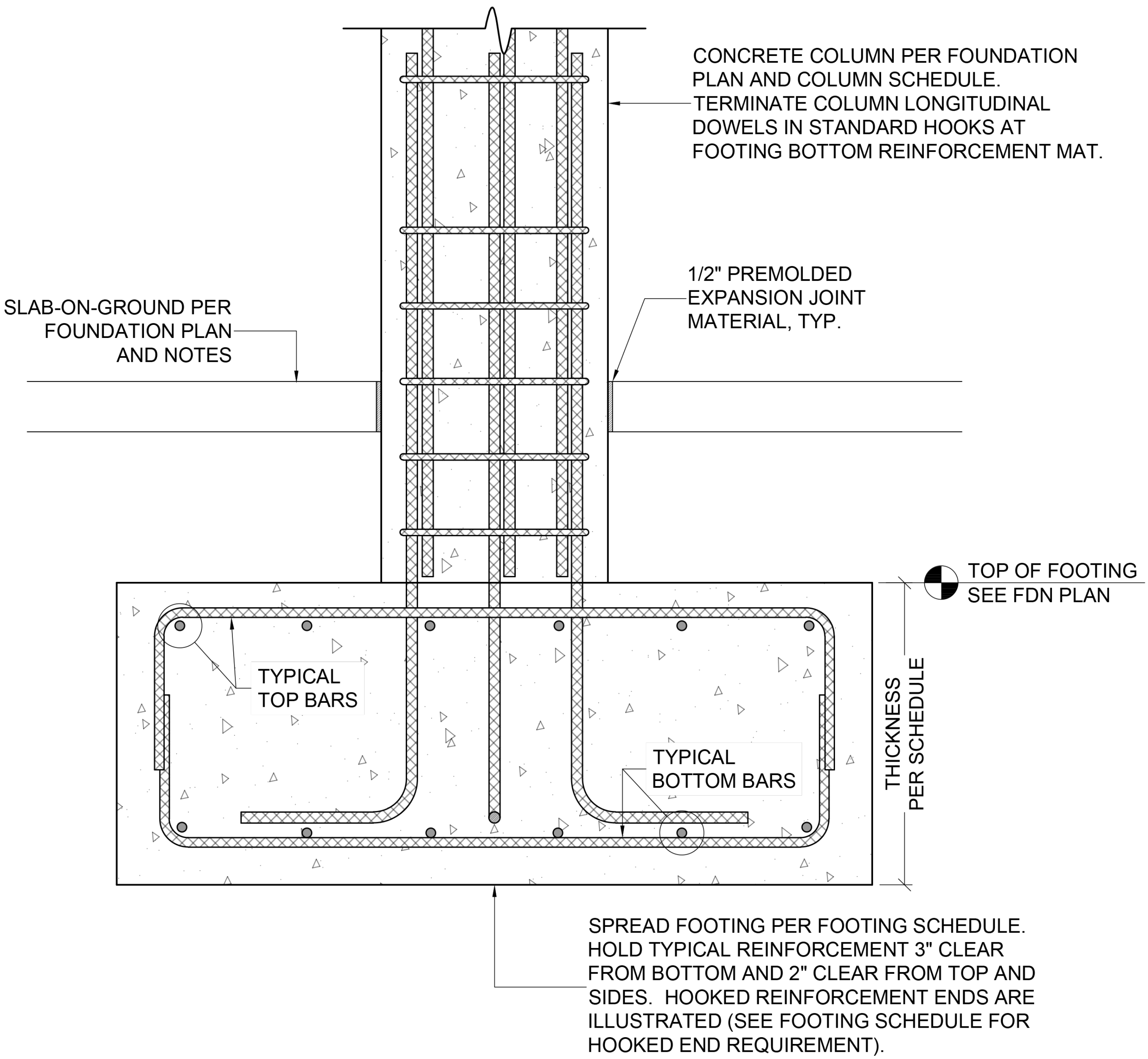


Figure 3: Typical spread footing detail as presented on contract documents

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 318-25	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 318-25 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 material 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 (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**
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 (in^2)**	**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

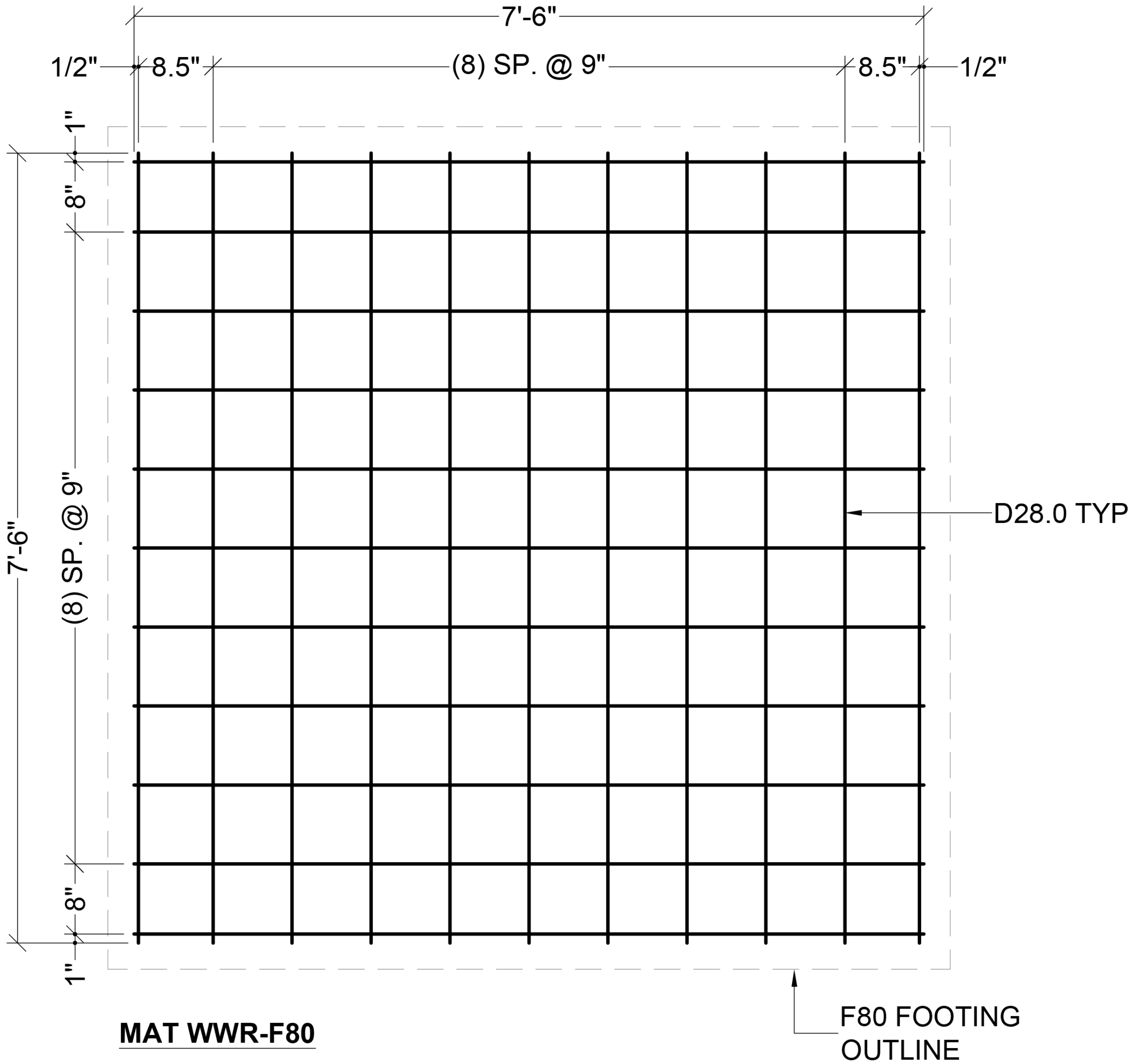
Material: ASTM A1064, 80 ksi yield strength

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 318-25	Calculations	Description
STEP 5: Corner Spread Footing – WWR Detailing (continued)		
Table 20.2.2.4(a) 25.4.2.4	$A_{s,provided} = 3.08\ in^2 = (7)\#6\ bars\ @\ \pm 15"\ on\ center\ each\ way$ $WWR\ options\ providing\ 3.08\ in^2\ steel\ area\ include:$ $(10)\ D30.8\ wires\ @\ \pm 10"\ on\ center\ each\ way$ $(11)\ D28.0\ wires\ @\ \pm 9"\ on\ center\ each\ way \leftarrow\ selected\ by\ detailer$ $(12)\ D25.7\ wires\ @\ \pm 8"\ on\ center\ each\ way$ $\frac{c_b + K_{tr}}{d_b} = \frac{3.299" + 0}{0.597} = 5.53 > 2.5\ \therefore use\ 2.5$ $l_d = \left(\frac{3}{40} \times \frac{80{,}000}{1.0 \times \sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 1.0 \times 1.15}{2.5}\right) \times 0.597 = 26"$ $Wires\ will\ run\ full\ dimension\ of\ footing, minus\ 3"\ cover\ each\ end.$ $Wire\ length\ beyond\ column\ face = \frac{96"}{2} - \frac{18"}{2} - 3" = 36\ inches$ $36"\ \text{extension} > 26"\ \text{required}$ $\therefore \boldsymbol{Flexural\ reinforcement\ is\ sufficiently\ developed.}$	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 318-25	Calculations	Description
STEP 6: Interior Spread Footing – Proportioning and Loading		
    13.3.1.1 Equation 5.3.1(b)	*Interior Spread Footing Load* $P_D = 300\ kips$ $P_L = 150\ kips$ $q_a = 2{,}000\ psf\ allowable\ soil\ bearing\ pressure$ $Required\ Footing\ Area\!:\ A_{ftg} = \frac{P_D + P_L}{q_a} = \frac{450\ kips}{2\ ksf} = 225\ ft^2$ $\sqrt{225} = 15$ $\therefore Use\ 15\ foot\ x\ 15\ foot\ spread\ footing.$ $The\ Designer\ selects\ 28\ inches\ as\ trial\ footing\ thickness.$ $U = 1.2D + 1.6L$ $P_u = 1.2 \times 300 + 1.6 \times 150 = 600\ kips$ $q_u = \frac{P_u}{A_{ftg}} = \frac{600}{225} = 2.67\ ksf \rightarrow strength\ level\ bearing\ pressure$	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 318-25	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.
  8.5.1.1(d) 8.5.1.2 22.6.1.2 22.6.4.1	***Two Way Shear Design*** $Need\ \varphi v_n \geq\ v_u$ $\varphi = 0.75$ $v_n = v_c$ $d = 28" - 3" - 0.5" = 24.5"$ ***Critical Section Considered as Interior Column Condition*** $b_o = 4 \times (18" + 24.5"/2 + 24.5"/2) = 170"$ $Loaded\ tributary\ area = 15\ ft \times 15\ ft - \frac{42.5" \times 42.5"}{144} = 212.5\ sf$ $V_u = 2.67\ ksf \times 212.5\ sf = 567\ kips$ $v_{uv} = \frac{V_u}{b_o d} = \frac{567\ kips}{170" \times 24.5"} = 0.136\ ksi$	For two-way (punching) shear design, there will be no shear reinforcement. 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. 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 318-25	Calculations	Description
STEP 7: Interior Spread Footing – Two-Way Shear Design (continued)		
22.6.5.2 22.6.5.3 13.2.6.2	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* ***∴ Two way (punching) shear capacity is sufficient.***	For two-way shear strength the size effect factor specified in 22.6 shall be $\lambda_s = 1.0$.

ACI 318-25	Calculations	Description
STEP 8: Interior Spread Footing – Flexural Design		
13.2.7.1	*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)* $M_u = 2.67\ ksf \times 15\ ft \times \frac{(13.5\ ft)^2}{8} = 913\ k - ft$	Calculate M_u at the face of the column.
21.2.2	*Assume tension − controlled section to start*: $\varphi = 0.90$ $m = \frac{f_y}{0.85 \times f'_c} = \frac{80{,}000}{0.85 \times 4{,}000} = 23.53$ $R_u = \frac{M_u}{\varphi b d^2} = \frac{913 \times 12}{0.9 \times 15 \times 12 \times 24.5^2} = 0.113\ ksi$ $\rho_{reqd} = \frac{1}{m} \times \left(1 - \sqrt{1 - \frac{2mR_u}{f_y}}\right) = 0.0015$	
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 318-25	Calculations	Description
STEP 8: Interior Spread Footing – Flexural Design (continued)		
8.6.1.2 7.7.2.4 21.2 22.2	$Is\ v_{uv} = 0.136\ ksi > \varphi \times 2 \times \lambda_s \times \lambda \times \sqrt{f'_c}\ ?$ $\varphi \times 2 \times \lambda_s \times \lambda \times \sqrt{f'_c} = 0.75 \times 2 \times 1.0 \times 1.0 \times \sqrt{4{,}000} = 0.095\ ksi$ $0.136\ ksi > 0.095\ ksi$ $\therefore Must\ consider\ flexure\ driven\ punching\ failure.$ $provide\ A_{s,\min FDPS}\ to\ resist\ flexure\ driven\ punching\ shear!$ $A_{s,\min FDPS} = \frac{5v_{uv}b_{slab}b_o}{\varphi\alpha_s f_y} = \frac{5 \times 0.136\ ksi \times 180" \times 170"}{0.75 \times 40 \times 80\ ksi} = 8.67\ in^2$ $Maximum\ spacing\ shall\ be\ the\ lesser\ of:$ • $2h = 2 \times 16\ inches = 32\ inches$ • $18\ inches \leftarrow$ ***The Design Professional selects (16) #7 reinforcing bars.*** $A_{s,prov} =\ 9.60\ in^2, (16)\ bars\ spaced\ @\ \pm 11.6"\ on\ center\ with\ 3"\ cover$ $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{9.60 \times 80}{0.85 \times 4 \times 180} = 1.26"$ $\varepsilon_t = \frac{0.003 \times \beta_1 \times d}{a} - 0.003 = \frac{0.003 \times 0.85 \times 24.5}{1.26} - 0.003 = 0.047\ in/in$ $\varepsilon_t = 0.047 >> \varepsilon_{ty} + 0.003 = 0.0058 \therefore Tension\ Controlled$ $\varphi M_n = 0.9 A_s f_y (d - a/2) = 1{,}375\ k - ft > 913\ k - ft$ $\therefore$ ***Flexural capacity is sufficient.***	Per ACI 318-25 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 must also 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 in 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 318-25	Calculations	Description
STEP 8: Interior Spread Footing – Flexural Design (continued)		
13.2.8.3 25.4.2.4	$l_d = \left(\frac{3}{40} \times \frac{f_y}{\lambda\sqrt{f'_c}} \times \frac{\psi_t\psi_e\psi_s\psi_g}{\frac{c_b + K_{tr}}{d_b}}\right) \times d_b$ $f_y = 80{,}000\ psi$ $f'_c = 4{,}000\ psi$ $\lambda = 1.0\ (normalweight\ concrete)$ $\psi_t = 1.0\ (bottom\ reinforcement\ with\ less\ than\ 12\text{"}\ concrete\ below)$ $\psi_e = 1.0\ (uncoated\ reinforcement)$ $\psi_s = 1.0\ (\#6\ bar\ and\ smaller\ bars\ and\ deformed\ wires)$ $\psi_g = 1.15\ (Grade\ 80\ reinforcement)$ $c_b = 3\ \text{cover} + \text{half bar diameter} = 3.44\text{"}$ $K_{tr} = 0\ (permitted\ as\ a\ design\ simplification)$ $d_b = 0.875\text{"}$ $\frac{c_b + K_{tr}}{d_b} = \frac{3.44\text{"} + 0}{0.875} = 3.93 > 2.5\ \therefore use\ 2.5$ $l_d = \left(\frac{3}{40} \times \frac{80{,}000}{1.0 \times \sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 1.0 \times 1.15}{2.5}\right) \times 0.875 = 39\text{"}$ $Bars\ will\ run\ full\ dimension\ of\ footing, minus\ 3\text{"}\ cover\ each\ end.$ $Bar\ length\ beyond\ column\ face = \frac{180\text{"}}{2} - \frac{18\text{"}}{2} - 3\text{"} = 78\ inches$ $78\text{"}\ \text{extension} > 39\text{"}\ \text{required}$ $\therefore \boldsymbol{Flexural\ reinforcement\ is\ sufficiently\ developed.}$	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 318-25	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	$\therefore V_n = V_c$	
7.4.3.2	$V_u = q_u \times b \times \left(\left(\frac{l-c}{2}\right) - d\right)$ $b = 15\ feet\ (footing\ dimension\ parallel\ to\ shear\ line)$ $l = 15\ feet\ (footing\ dimension\ perpendicular\ to\ shear\ line)$ $c = 1.5\ feet\ (column\ dimension\ perpendicular\ to\ shear\ line)$ $d = 24.5\ inches = 2.042\ feet$ $V_u = 2.67\ ksf \times 15\ feet \times \left(\left(\frac{15-1.5}{2}\right) - 2.042\right) = 189\ kips$	Calculate required shear strength along a shear line located a distance "d" from the face of the column.
22.5.5.1	$A_v = 0, \therefore A_v < A_{v,min}$ $V_c = \left[8\lambda_s\lambda({\rho_w}^{1/3})\sqrt{f'_c} + \frac{N_u}{6A_g}\right] \times b_w d \leq 5\lambda\sqrt{f'_c} b_w d$ $\rho_w = \frac{A_s}{b_w d} = \frac{9.60}{180 \times 24.5} = 0.0022$ $V_c = \left[8 \times 1 \times 1 \times 0.0022^{1/3} \times \sqrt{4000} + 0\right] \times 180 \times 24.5 = 290\ kips$	Slab proportioning/depth is appropriate.
22.5.5.1.1	$V_c = 290\ kips < 5 \times 1.0 \times \sqrt{4000} \times 180 \times 24.5 = 1{,}395\ kips$	

ACI 318-25	Calculations	Description
STEP 9: Interior Spread Footing – One-Way Shear Design (continued)		
8.5.1.1	$Need\ \varphi V_n \geq V_u$ $0.75 \times 290\ kips = 218\ kips > V_u = 189\ kips\ \therefore OK$	
22.5.1.2	$Also\ need\ V_u \leq \varphi\left(V_c + 8\sqrt{f'_c} b_w d\right) \rightarrow 189\ kips < 2{,}521\ kips\ \therefore OK$ $\boldsymbol{\therefore One\ way\ shear\ capacity\ is\ sufficient.}$	Check to confirm likelihood of diagonal compression failure is precluded and to control cracking under service load. **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 318-25	Calculations	Description
STEP 10: Interior 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 (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**
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 (in²)**	**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 318-25	Calculations	Description
STEP 10: Interior 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.

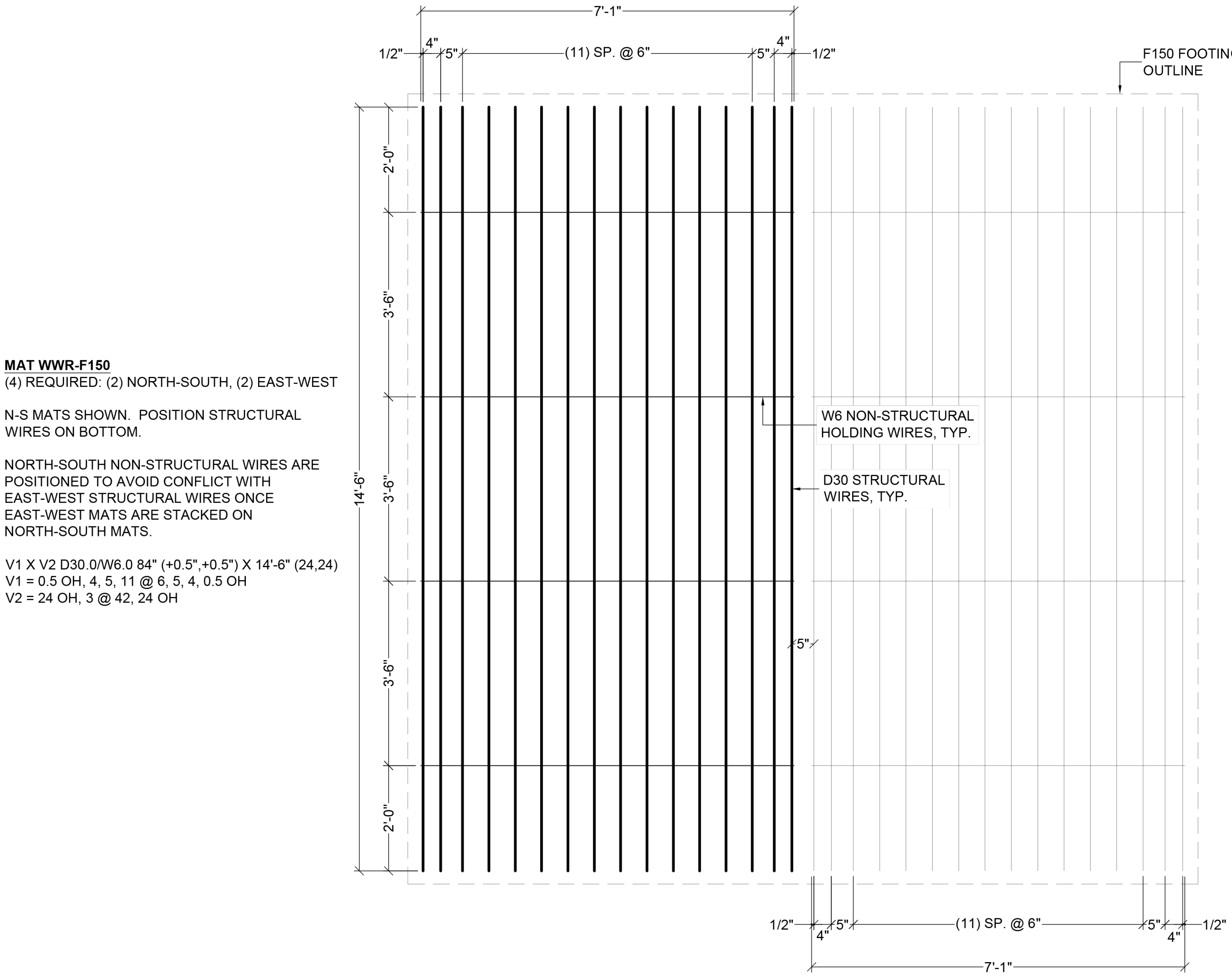


Figure 9: Representation of WWR for footing F150, as would be presented on a set of reinforcement shop/placement drawings

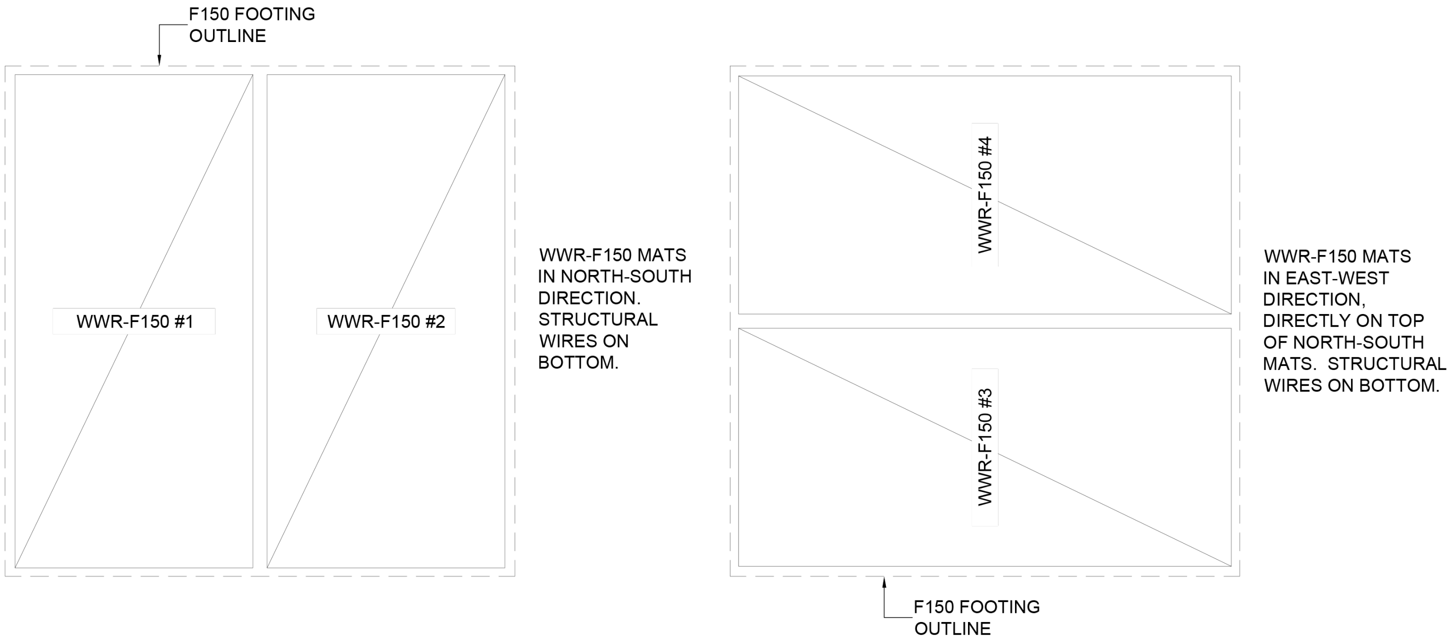


Figure 10: WWR placement information for Footing F150, showing the required four-mat assembly

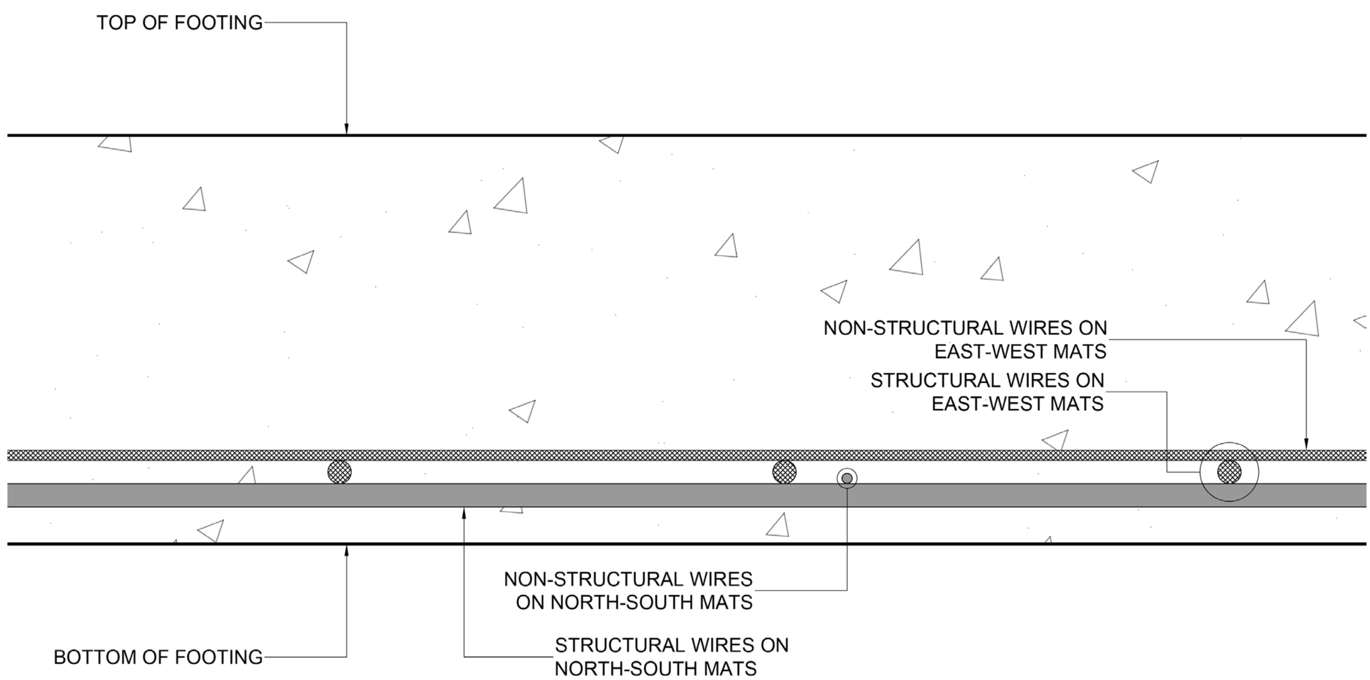


Figure 11: WWR placement information for Footing F150, showing mat stacking. Note that the WWR detailer will arrange non-structural holding wires such that they do not conflict with stacked structural wires that are coplanar.

ACI 318-25	Calculations	Description
STEP 11: Wall Footing – Design		
 13.3.1.1 7.6.1.1 7.6.4.1 24.4.3.2 24.4.3.3	$Continuous\ Wall\ Footing\ Load$ $q_D = 0.200\ kips/foot$ $q_L = 0\ kips/foot$ $q_a = 2{,}000\ psf\ allowable\ soil\ bearing\ pressure$ $Required\ Footing\ Width\!:\ W_{ftg} = \dfrac{q_D + q_L}{q_a \times 1\ ft} = \dfrac{0.200\ kips}{2\ kip/ft} = 0.1\ ft$ $Loading\ does\ not\ govern\ over\ the\ geometry\ of\ the\ cladding\ itself.$ $\therefore\ Use\ 20\text{" wide wall footing, resulting in 4" projection beyond cladding.}$ $The\ Designer\ selects\ 12\ inches\ as\ a\ footing\ thickness.$ $ACI\ 318\ minimums\ will\ be\ provided\ for\ longitudinal\ reinforcement\ .$ $Longitudinal\!:\ 0.0018 \times 16\text{"} \times 12\text{"} = 0.35\ in^2\ required$ $\therefore\ Use\ two\ (2)\ \#4\ bars, spaced\ at\ 20 - 3\text{"} - 3\text{"} = 14\text{"}$ $By\ inspection, nominal\ transverse\ reinforcment\ will\ be\ provided.$ $\therefore\ Use\ \#4\ bars\ @\ 14\text{"}\ on\ center$	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. 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 318-25	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.

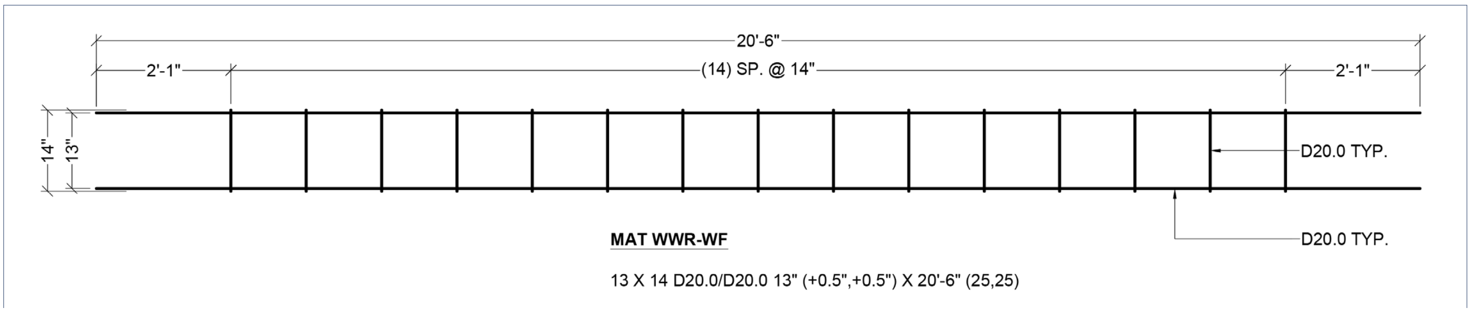


Figure 12: Representation of WWR for wall footing. Note the spacing indicated for the longitudinal wires is shown a total of one inch less than that which was specified in order to accommodate a minimum production "overhang" of one-half inch on each side (as dictated by the needs of the welding machines). 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.