The Use of Welded Wire Reinforcement in Spread Footings

Spread footings are an outstanding application for the implementation of WWR on those projects characterized by shallow foundation systems requiring reinforcement to be installed quickly and efficiently.

This month's blog outlines the step-by-step procedure for design of a concrete spread footing, followed by the design solution presented in the form of welded wire reinforcement (WWR).

Step 1: Establish Footing Design Criteria

- ACI 318-19 (22) procedures apply. Applicable sections noted in *[blue italics]*.
- 28-day compressive strength, f'_c = 4,000 psi normal-weight concrete (λ = 1.0)
- Reinforcement yield strength, f_y = 80 ksi
- Permissible soil bearing pressure, q_a = 3,000 psf
- Applied service-level column loading, P_a = 275 kips
- Applied factored column loading, P_u = 380 kips
- Column section is a W12x65 steel section with an 18" x 18" baseplate
- Column evaluated is considerd to be at an interior location on the footing.
- Assumed effective flexural depth to tensile reinforcement, d = footing thickness minus 3.5 inches (represents 3" clear cover plus portion of reinforcing bar)
- No shear reinforcement of footing is provided.

Step 2: Footing Horizontal Proportioning

[Section 13.3.1.1]

$$Required\ Footing\ Area:\ A_{ftg} = \frac{P_a}{q_a} = \frac{275\ kips}{3\ ksf} = 91.7\ ft^2$$

For a square footing, then the horizontal dimension are:

$\sqrt{92} = 9.59\ ft \rightarrow use\ 10\ ft\ \times 10\ ft$ spread footing

Step 3: Footing Vertical Proportioning - Two-Way Shear Considerations

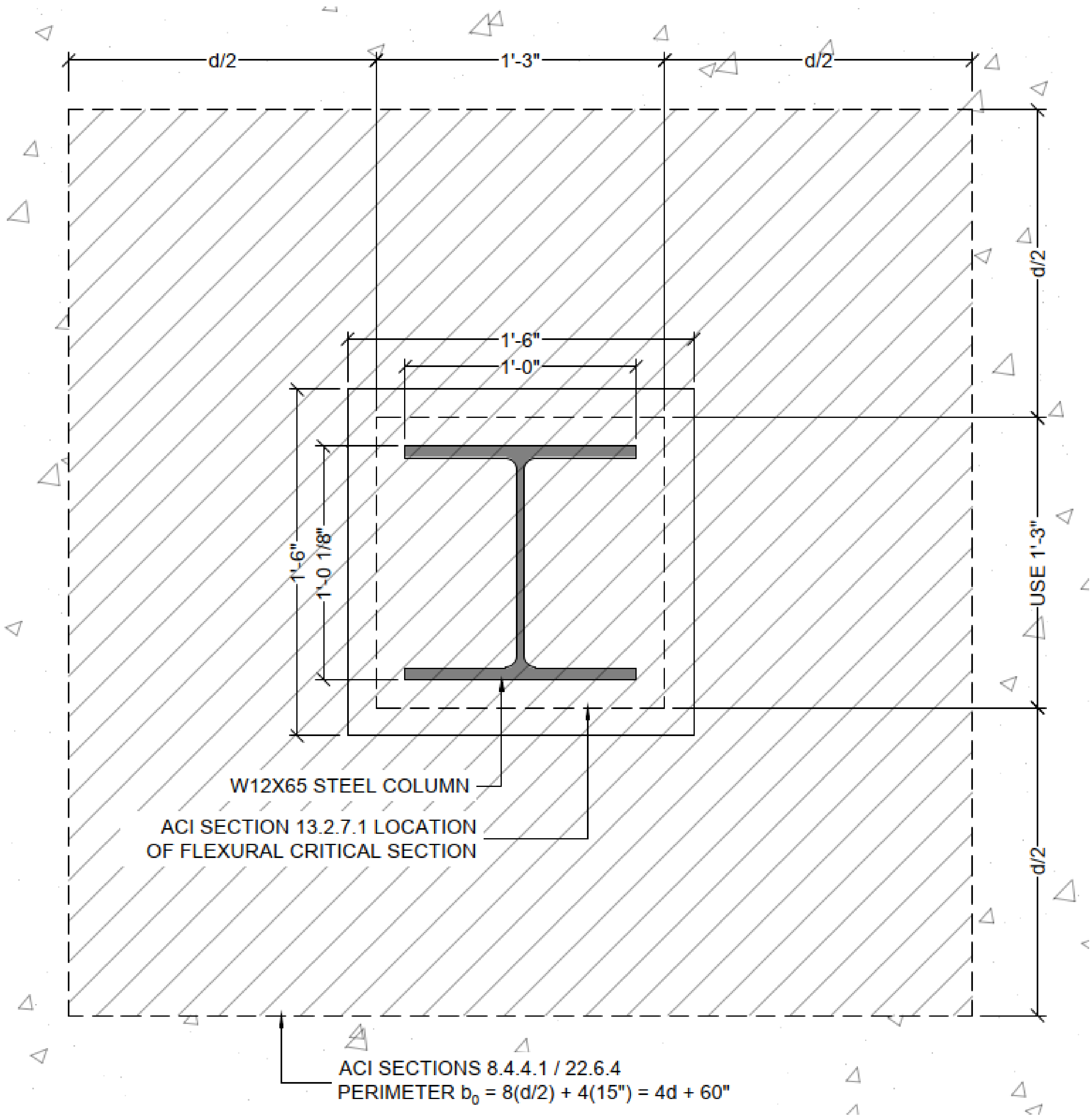
In the proportioning of the vertical thickness of the footing, we first consider two-way (punching) shear.

[13.2.7.2]
[13.2.7.1]

The location of the critical section for two-way shear shall be measured from the location of the critical section for Mu noted in Section 13.2.7.1. For a steel column with steel baseplate, this location is positioned halfway between the face of the column and the edge of the steel baseplate.

[8.4.4.1]
[22.6.4.1]

The critical section perimeter, b_0, is illustrated as follows:



$b_0 = 4d + 60"$

$$q_u = \frac{P_u}{A_{ftg}} = \frac{380}{100} = 3.80\ ksf \rightarrow strength\ level\ bearing\ pressure$$

Assume footing thickness, h, is 24 inches, resulting in d = 20.5 inches.

$$b_0 = 4 \times 20.5" + 60" = 142\ inches$$

Shear, V_u, at the critical perimeter:

$$V_u = P_u - q_u \times area\ within\ critical\ section = 380\ kips - 3.8\ ksf \times (35.5"/12)^2 = 347\ kips$$

$$v_{uv} = \frac{V_u}{b_o d} = \frac{347\ kips}{142" \times 20.5"} = 0.120\ ksi$$

Note that 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.

The above represents the shear stress demand. We must now calculate the shear capacity.

[8.5.1.1(d)]

$$Need\ \varphi v_n \geq v_u$$

[8.5.1.2]

$$\varphi = 0.75$$

[22.6.1.2]

We will have no shear reinforcement. Therefore:

$$Set\ v_n = v_c$$

[22.6.5.2]
[22.6.5.3]
[Table 22.6.5.2]
[13.2.6.2]

Keeping in mind that per Section 13.2.6.2, for two-way isolated footings it is permissible to neglect the size effect factor included in the Table 22.6.5.2 equations (set value equal to 1.0), we have the following:

v_c *is the least of*:

$$(a)\ \ v_c = 4\lambda_s\lambda\sqrt{f'_c} = 4 \times 1.0 \times 1.0 \times \sqrt{4000} = 0.253\ ksi \leftarrow$$

$$(b)\ \ v_c = \left(2 + \frac{4}{\beta}\right)\lambda_s\lambda\sqrt{f'_c} = 6 \times 1.0 \times 1.0 \times \sqrt{4000} = 0.379\ ksi$$

β *is the ratio of column sides.* $\beta = 1$ *for square column.*

$(c)\ \ v_c = \left(2 + \frac{\alpha_s d}{b_o}\right)\lambda_s \lambda \sqrt{f'_c} = \left(2 + \frac{40 \times 20.5"}{142"}\right) \times 1 \times 1 \times \sqrt{4000} = 0.492\ ksi$

$\alpha_s = 40\ for\ interior\ columns$

$\therefore v_c = 0.253\ ksi$

$v_{uv} = 0.120\ ksi < 0.75 \times 0.253 = 0.190\ ksi$

$\varphi v_n \geq\ v_u\ is\ satisfied\ if\ footing\ is\ 24"\ thick.$

Step 3: Footing Vertical Proportioning - Flexural Considerations

For a concentric axially-loaded square footing:

[13.2.7.1]
[21.2.2]
[8.6.1.1]

$$M_u = q_u \times b \times \frac{\left(\frac{l}{2} - \frac{c}{2}\right)^2}{2}$$

$q_u = 3.80\ ksf$
$b = 10\ feet\ (footing\ dimension\ perpendicular\ to\ flexure)$
$l = 10\ feet\ (footing\ dimension\ parallel\ to\ flexure)$
$c = 1.25\ feet\ (column/baseplate$ average$dimension\ parallel\ to\ flexure)$

$$M_u = 3.80\ ksf \times 10\ ft \times \frac{\left(\frac{10'}{2} - \frac{1.25'}{2}\right)^2}{2} = 364\ k - ft$$

$Assume\ tension - controlled\ section\ to\ start: \varphi = 0.90$

$$m = \frac{f_y}{0.85 \times f'_c} = \frac{80{,}000}{0.85 \times 4{,}000} = 23.53$$

$$R_u = \frac{M_u}{\varphi b d^2} = \frac{364 \times 12}{0.9 \times 10 \times 12 \times 20.5^2} = 0.096\ ksi$$

$$\rho_{reqd} = \frac{1}{m} \times \left(1 - \sqrt{1 - \frac{2mR_u}{f_y}}\right) = 0.0012$$

$$\rho_{reqd} = \frac{1}{23.53} \times \left(1 - \sqrt{1 - \frac{2(23.53)(0.096)}{80}}\right) = 0.0012$$

$A_{s,reqd} = \rho_{reqd} \times b \times \mathrm{d}$

$A_{s,reqd} = 0.0012 \times 120" \times 20.5" = \ 2.95\ in^2$

$A_{s,\min prescriptive} = 0.0018 \times A_g = 0.0018 \times 120" \times 24" = \ 5.19\ in^2 \leftarrow$

[13.3.3.1]
[8.6.1.2]
[8.4.2.2.3]
[7.7.2.4]

Before finalizing the flexural reinforcement, however, 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.

Sections 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.

$Is\ v_{uv} = 0.120\ ksi > \varphi \times 2 \times \lambda_s \times \lambda \times \sqrt{f'_c}\ ?$

$\varphi \times 2 \times \lambda_s \times \lambda \times \sqrt{f'_c} = 0.75 \times 2 \times 1.0 \times 1.0 \times \sqrt{4{,}000} = 0.095\ ksi$

$0.120\ ksi > 0.095\ ksi$

$\therefore Must\ consider\ flexure\ driven\ punching\ failure.$

$provide\ A_{s,\min FDPS}\ to\ resist\ flexure\ driven\ punching\ shear, with\ b_{slab}\ per\ Section\ 8.4.2.2.3:$

$b_{slab} = 1.5h + 1.5h + column\ size = 1.5(24")+1.5(24") + 15" = 87\ inches$

$$A_{s,\min FDPS} = \frac{5 v_{uv} b_{slab} b_o}{\varphi \alpha_s f_y} = \frac{5 \times 0.120\ ksi \times 87" \times 142"}{0.75 \times 40 \times 80\ ksi} = 3.09\ in^2$$

$Maximum\ spacing\ shall\ be\ the\ lesser\ of:$

- $2h = 2 \times 24\ inches = 48\ inches$
- $18\ inches \leftarrow$

We end up with two flexural reinforcement solutions to compare, each of which must conform to the above maximum reinforcement spacing limits:

A. 5.19 in^2 across full 120 inches footing width = 0.519 in^2 per linear foot ←
B. 3.09 in^2 across 87 inch punching shear region = 0.426 in^2 per linear foot

We will proceed with $A_{s,design}$ = 5.19 in^2 for flexural reinforcement used in the design. This information, provided with minimum clear cover and the yield strength of the reinforcing steel, constitutes the information necessary on the structural contract documents for the WWR manufacturer to derive the WWR mat configurations (i.e., “styles”).

$A_{s,design} = 5.19\ in^2$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{5.19 \times 80}{0.85 \times 4 \times 120} = 1.018''$$

$$\varepsilon_t = \frac{0.003 \times \beta_1 \times d}{a} - 0.003 = \frac{0.003 \times 0.85 \times 20.5}{1.018} - 0.003 = 0.048\ in/in$$

$\varepsilon_t = 0.048 >> \varepsilon_{ty} + 0.003 = 0.0058 \therefore Tension\ Controlled$

$\varphi M_n = 0.9 A_s f_y (d - a/2)$

$\varphi M_n = 0.9(5.19)(80)(20.5 - 1.018/2) = 623\ k - ft > M_u = 364\ k - ft\ \therefore flexure\ OK!$

Step 4: Footing Vertical Proportioning – One-Way Shear Considerations

We round out the design by checking one-way shear.

[8.5.1.1]

$We\ need\ \varphi V_n \geq V_u$

[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]

$$V_u = q_u \times b \times \left(\left(\frac{l - c}{2} \right) - d \right)$$

$q_u = 3.80\ ksf$
$b = 10\ feet\ (footing\ dimension\ parallel\ to\ shear\ line)$
$l = 10\ feet\ (footing\ dimension\ perpendicular\ to\ shear\ line)$
$c = 1.25\ feet\ (column\ dimension\ perpendicular\ to\ shear\ line)$
$d = 20.5 inches = 1.708\ feet$

$$V_u = 3.80\ ksf \times 10\ feet \times \left(\left(\frac{10 - 1.25}{2} \right) - 1.708 \right) = 101\ kips$$

[22.5.5.1]

$A_v = 0, \therefore A_v < A_{v,min}$

$$V_c = \left[8 \lambda_s \lambda \left(\rho_w^{1/3} \right) \sqrt{f'_c} + \frac{N_u}{6 A_g} \right] \times b_w d \leq 5 \lambda \sqrt{f'_c} b_w d$$

$$\rho_w = \frac{A_s}{b_w d} = \frac{5.19}{120 \times 20.5} = 0.0021$$

$$V_c = \left[8 \times 1 \times 1 \times 0.0021^{1/3} \times \sqrt{4000} + 0\right] \times 120 \times 20.5 = 159.45\ kips$$

[22.5.5.1.1]

$$V_c = 159.4\ kips < 5 \times 1.0 \times \sqrt{4000} \times 120 \times 20.5 = 778 kips$$

$$Need\ \varphi V_n \geq V_u$$

$$0.75 \times 159.4\ kips = 120\ kips > V_u = 101\ kips\ \therefore\ one\ way\ shear\ OK!$$

[22.5.1.2]

$$Also\ need\ V_u \leq \varphi\left(V_c + 8\sqrt{f'_c} b_w d\right) \rightarrow 98.2\ kips < 1{,}404\ kips\ \therefore OK$$

Step 5: Configuring the WWR Mats

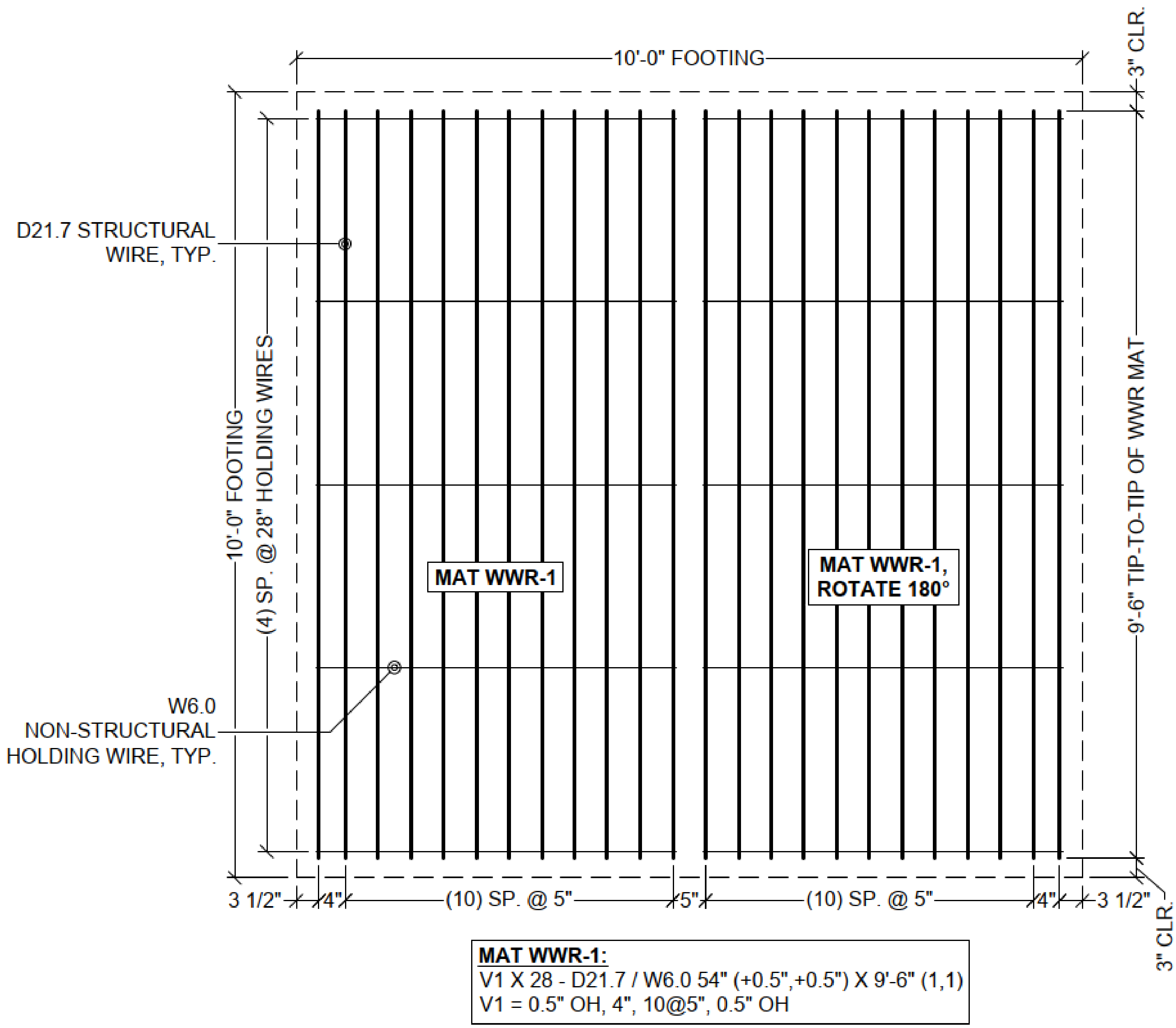
Reinforcement options satisfying the engineer's previously-calculated **A_s = 5.19 in^2** include the following, and would be derived by the manufacturer to satisfy both the engineer's stated design requirements as well as the most efficient and expedient solution from a manufacturing perspective:

Wire Size	Wire quantity	Wire spacing across footing width[1]
D30.6	17	<4”>, (16) sp. @ 7”, <4”>
D26	20	<3.5”>, 5.5”, (17) sp. @ 6”, 5.5”, <3.5”>
D21.7	**24**	**<3.5”>, 4”, (21) sp. @ 5”, 4”, <3.5”>**
D17.9	29	<4”>, (28) sp. @ 4”, <4”>

1. *<X> indicates dimension from edge of footing to nearest parallel wire's centerline*

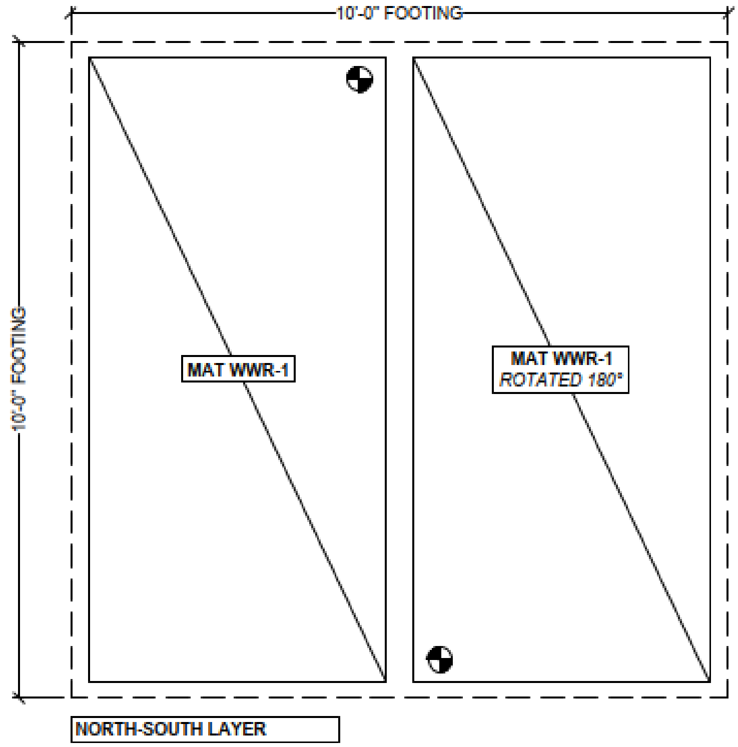
For this particular example we assume that the manufacturer selects the D21.7 option, highlighted in green. A D21.7 wire is a wire with a deformed surface and a cross-sectional area equal to 0.217 in^2.

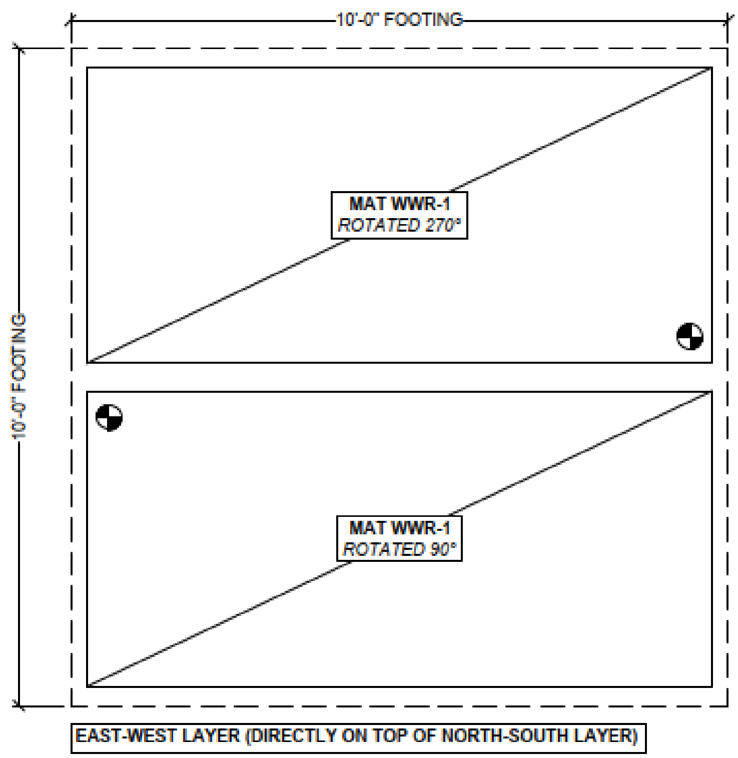
The diagram below shows the use of uni-directional WWR mats: mats with structural wires in one direction only, with non-structural "holding" wires positioned in the perpendicular direction to maintain mat shape. While it is possible to produce WWR mats up to ±12'-0" in width, most manufacturers try to keep mat widths to an 8'-6" maximum to avoid additional transportation permit costs associated with oversize loads. As such, for this 10'-0" wide footing, the manufacturer elects to furnish the reinforcement segmentally as indicated, with width coverage achieved by two identical adjacent mats.



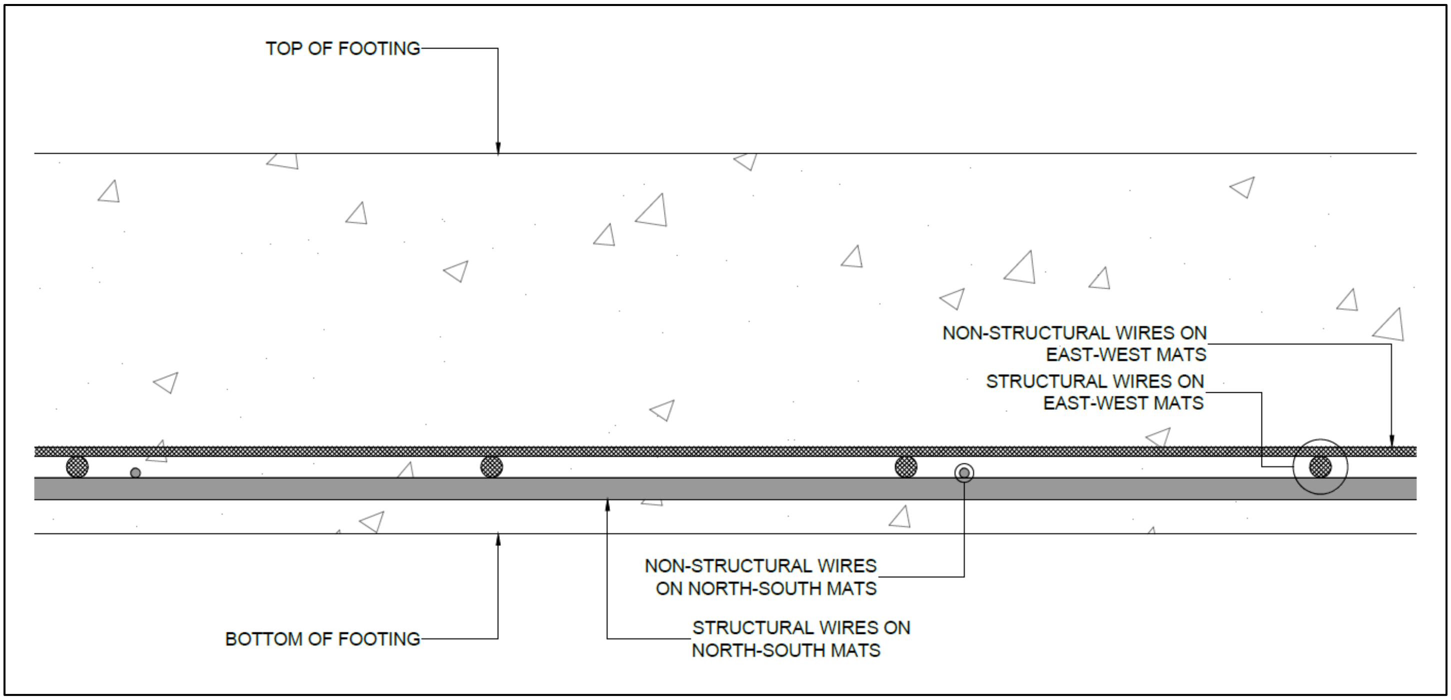
(Note: The illustration above show the reference WWR-1 mat being rotated. This is simply to give a plan-symmetrical reinforcement layout view for the purpose of this article and as idealized in the previous tabulation of wire size/spacing options. It is important to understand that in this case, even if the mat on the right side was not rotated as indicated, the unit cross-sectional area of steel required by the design would still be achieved across the entire footing.)

Because there will be bending in both primary directions of the footing. layering of uni-directional WWR mats will be required as indicated below, comprised of a “north-south” layer and a subsequent “east-west” layer stacked on top of it.





The installed 4-mat assembly would be comprised of structural wires of the east-west layer nesting directly against the top surface of the structural wires of the north-south layer, with the resulting location of the "holding" wires being positioned non-intrusively. The installed layering of mats is illustrated in section view below.



Step 6: Other Considerations

This example shows a fairly streamlined procedure. For the designer there are additional considerations not shown here, briefly summarized below.

- The selected footing thickness could possibly be optimized to a thinner section. A thinner section would of course require re-calculation of the flexural and shear design strengths, and an re-configuration of the flexural steel.
- In this example we simplistically used an effective depth “d” equal to the footing thickness minus 3.5”. in reality, the effective depth used in design calculations could be slightly larger than that shown given the 3.5” dimension is based on the assumption of reinforcement with a 1-inch diameter.
- Development length of reinforcement beyond the critical section would need to be confirmed as adequate (typically not an issue for larger-sized footings such as this).
- Bearing stresses between the top of the footing and the underside of the column baseplate would need to be confirmed as adequate.

This example shows the ease with which concrete spread footings can be designed and constructed using WWR.

If the designer elected to use reinforcing bars, the result would be manual placement and tying of either twenty-four (24) loose #6 bars or thirty-four (34) loose #5 bars. Instead, the placement operation could be greatly simplified by the installation of just four (4) identical WWR mats outlined here.

For more information visit www.wirereinforcementinstitute.org.

References:

1. ACI Committee 318, “Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19),” American Concrete Institute, Farmington Hills, MI, 2019 (Reapproved 2022)