WRI WIRE REINFORCEMENT INSTITUTE®

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Technical Blog

Welded Wire Reinforcement as a First Choice Solution in Tilt-Up Wall Panels

There has been a substantial increase in welded wire reinforcement usage in tilt-up concrete wall panels in recent years, largely due to the inherent on-site time and labor savings and a notable uptick in the proficiency of reinforcement placing contractors. The WRI's July 2023 *"Technical Roundtable – Engineering and Contracting Perspectives on the Use of Welded Wire Reinforcement in Tilt-Up Concrete Wall Panels"* takes a fairly deep dive into the matter and is a worthwhile read as a precursor to the content of this blog entry. Other relevant WRI documents include the following, all of which are available as free downloads on the WRI website:

- WRI WWR-500 Manual of Standard Practice (2021)
- WRI Tech Fact TF204-R-21 Welded Wire Reinforced Tilt-Up Wall Panels (2021)
- WRI Technical Blog "Leveraging 80 ksi WWR as a Structural Reinforcement Substitution" (April 2022)
- WRI Technical Blog "Substitution Welded Wire Reinforcement Sizes" (June 2022)
- WRI Technical Blog "The ACI Alternative Method for Out-of-Plane Slender Wall Analysis" (July 2022)

A topic that always seems to garner a significant amount of attention is the "value-engineered" substitution of WWR for originally-specified reinforcing bars. At its most basic level, such a substitution is often preliminarily proposed for consideration as an equivalency option that is based on the ratio of the commonly-used 60 ksi reinforcing bar yield strength and the readily available 80 ksi WWR yield strength. For example:

60 ksi #4 reinforcing bars @ 12" on center → $A_s f_y$ = 12 kips (A_s = 0.20 in^2)
80 ksi 12x12 D15.0/D15.0 WWR → $A_s f_y$ = 12 kips (A_s = 0.15 in^2)

The latter option, comprised of a lesser cross-sectional area of steel reinforcement, not only results in less steel tonnage for the project, but considering that it is provided in prefabricated welded mat form, it offers the placement time and labor savings that are so unique to WWR. Note that its nominal unit strength (i.e., the product $A_s f_y$) is identical to that of the reinforcing bars.

With that in mind, appropriate engineering judgment and oversight of such a substitution is always required on the part of the design professional of record to ensure that structural design intent is maintained. *Substitutions must be verified by the designer*. For tilt-up wall panels in particular, this verification is extremely important, as the walls themselves are typically slender elements that are highly sensitive to the amount of cross-sectional steel reinforcement embedded therein. This is because the wall's stiffness, contingent upon a cracked section moment of inertia (I_{cr}), is directly related to steel cross-sectional area, NOT steel yield strength. So a lesser steel area will not only correlate to a lower I_{cr} that produces a larger out-of-plane deflection of the wall panel but also results in a larger flexural demand on the wall in consideration of the increased P-Δ effect.

Compiled below is a hypothetical design for two different panel geometries, with a comparison of an “originally specified” 60 ksi reinforcing bar solution and an 80 ksi WWR option that has equal reinforcement nominal unit strength. The comparison criteria are as follows:

- For each of the two-panel geometries, a field design strip and an edge design strip have been evaluated. For this analysis, the field strip is a 24-inch wide vertical strip of wall that has no openings directly adjacent to it. In contrast, the edge strip is a 24-inch wide vertical strip of wall that has an 8’-0” wide opening directly adjacent to it on one side, which is generally representative of a wall pier “jamb” condition adjacent to a loading dock door.

- Loading applied to the wall is idealized as a 15 psf superimposed service-level roof dead load and a 20 psf superimposed service-level roof live load, each applied over a tributary width of 30 feet (assumes roof framing that spans 60 feet perpendicular to the wall). These loads are incurred at the roof diaphragm level and are applied at a 3-inch eccentricity from the wall centerline.

- A strength-level (ultimate) component and cladding wind pressure / suction equal to 30 psf is applied perpendicularly to the entire wall surface.

- The unbraced panel length is measured vertically as an out-of-plane span between the diaphragm roof restraint and slab-on-ground restraint. A 1-foot parapet is assumed to exist above the point of the diaphragm roof restraint.

- Panel thicknesses indicated are gross thicknesses. For each scenario, a 3/4" deep architectural reveal is assumed, effectively reducing the net structural wall thickness in the proximity of the mid-height of the panel. The effective depth “d” of reinforcement is then derived as a function of the net structural wall thickness, an assumed reinforcement bar/wire diameter, and a constant clear cover of 1 inch.

- Welded wire reinforcement is assumed to be ASTM A1064 grade 80.

- Concrete compressive strength is conservatively specified as 4 ksi.

- Commercially-available tilt-up wall panel design software was used in the derivation of the results compiled herein, with checking of all relevant ACI / IBC load combinations. Note that there exist slight margins of error in the tabulated design moment strengths as a result of number rounding rules embedded into the software itself. These margins of error are largely immaterial.

Table 1: 60 ksi Original Design vs. 80 ksi “Equivalent” Nominal Reinforcement Strength

		Unbraced panel length = 25'-0" Panel thickness = 7"				Uunbraced panel length = 25'-0" panel thickness = 9.5"			
		FIELD STRIP		**EDGE STRIP**		**FIELD STRIP**		**EDGE STRIP**	
f_y		60 ksi	80 ksi	60 ksi	80 ksi	60 ksi	80 ksi	60 ksi	80 ksi
A_s	in^2/ft	0.14	0.105	0.45	0.34	0.17	0.128	.57	0.43
M_u w/ P-Δ	per ft	3350	3801	10795	12647	6591	7501	20866	24580
ϕM_n	per ft	3555	3584	10860	10901	6599	6769	20910	21038
I_{cr}	per ft	29.1	22.1	73.5	56.0	82.3	63.2	218.5	166.2
Δ_s	inches	0.141	0.141	1.089	1.483	0.189	0.189	1.332	1.799
d	inches	5	5	5	5	7.5	7.5	7.438	7.438
A_{se}	in^2/ft	0.16	0.12	0.52	0.39	0.2	0.15	0.67	0.51

Table 1 shows that a simplistic direct swap of reinforcement based on yield strength ratio alone can result in insufficient designs. In all four cases, I_{cr} is significantly reduced and the flexural demand-to-capacity ratio (DCR; M_u / ϕM_n) is exceeded:

- 7-inch panel field strip DCR: 0.942 increased to 1.06
- 7-inch panel edge strip DCR: 0.994 increased to 1.16
- 9.5-inch panel field strip DCR: 0.99 increased to 1.11
- 9.5-inch panel edge strip DCR: 0.99 increased to 1.17

Additionally, in the case of service-level mid-height deflections, including P-Δ effects, both edge strip scenarios see significant increases, though it is worth noting that the increased deflections are still within the limits stated in ACI 318 11.8.1.1(e):

- 7-inch panel mid-height deflection: 1.089" increased to 1.483"
- 9.5-inch panel mid-height deflection: 1.332" increased to 1.799"

While the results of the above hypothetical are obviously not representative of every "yield strength-based" WWR substitution encountered in practice, they do serve as a good indicator of the pitfalls of oversimplified WWR substitutions.

With that said, of particular interest is the fact that a lower cross-sectional area of 80 ksi steel can still be a viable alternative to the originally specified 60 ksi reinforcement. The key item to note here is that the "ratio" would just be larger than the 60/80 ratio (i.e., a 0.75 multiplier) discussed above.

Table 2: 60 ksi Original Design vs. 80 ksi Modified Design

		Unbraced panel length = 25'-0" Panel thickness = 7"				Uunbraced panel length = 25'-0" panel thickness = 9.5"			
		FIELD STRIP		**EDGE STRIP**		**FIELD STRIP**		**EDGE STRIP**	
f_y		60 ksi	80 ksi	60 ksi	80 ksi	60 ksi	80 ksi	60 ksi	80 ksi
A_s (in^2/ft)		0.14	0.11	0.45	0.39	0.17	0.14	.57	0.49
M_u w/ P-Δ	per ft	3350	3726	10795	11964	6591	7213	20866	23209
ϕM_n	per ft	3555	3733	10860	12183	6599	7298	20910	23445
I_{cr}	per ft	29.1	22.8	73.5	60.8	82.3	67.8	218.5	180.5
Δ_s	inches	0.141	0.141	1.089	1.318	0.189	0.189	1.332	1.604
d	inches	5	5	5	5	7.5	7.5	7.438	7.438
Reinforcement		#4 @ 17	D11.0 @ 12	#5 @ 8.2	D19.5 @ 6	#4 @ 14.2	D14.0 @ 12	#5 @ 6.5	D24.5 @ 6

The yellow cells of Table 2 tell a compelling story. 80 ksi WWR substituted at equivalency rates higher than 0.75 (but less than 1.0) are still perfectly acceptable structural options. The WWR placement time and labor advantages would, of course, still be in play, and because of the versatility of the equipment producing the WWR, the available variability of sizes and spacings yield more efficient and economical results than those lockstep sizes and spacings that are typical of field-placed loose reinforcing bars.

For example, while a 17" spacing of bars could be executed properly in the field with frequent measurement and placement tolerance, it is still an uncommon spacing for workers to navigate and would draw more quality control attention than, say, 10-inch or 12-inch spacing. Even more uncommon are fractional spacings of 8.2", 14.2", and 6.5", as shown in the table. Obviously, for the purpose of this discussion, these spacings were derived to correlate as closely as possible to a design solution with a DCR equal to 1.0. In the real world, a design professional would almost certainly round these numbers down to 8”, 14”, and 6” for simplicity. Of course, by rounding down the spacings to help ease the hand placement operation, the designer has effectively specified more reinforcement than the theoretical design actually requires. While this is a fairly common practice with loose individual pieces of reinforcement, the inherent design conservatism that results – not to mention increased project steel tonnage – is noteworthy. With WWR, precise cross-sectional steel areas matching those required by the design can be more easily achieved.

In looking at the WWR modified design, then:

- 7-inch panel field strip equivalency rate: 0.11/0.14 = 0.79
- 7-inch panel edge strip equivalency rate: 0.39/0.45 = 0.87
- 9.5-inch panel field strip equivalency rate: 0.14/0.17 = 0.82
- 9.5-inch panel edge strip equivalency rate: 0.49/0.57 = 0.86

In short, based on the above information, the ideal approach from a designer’s perspective would be to do the following as part of the original tilt-up wall panel design:

- **Assume 80 ksi yield strength material**
- **Specify the minimum steel areas themselves, NOT the actual wire/bar sizes**
- **Specify a maximum / minimum spacing of individual pieces of reinforcement, NOT a predetermined spacing**
- **Specify the required clear cover(s) and/or assumptions for minimum design depth “d”**

With this information in hand, the most economical reinforcement solution satisfying the engineer’s design intent can be derived and submitted for review and approval.

Simplistic conversion of reinforcing bars to WWR based on yield strength ratio alone is generally not appropriate unless the design professional of record is able to validate it on a case-by-case basis, post-design, and pre-construction. Instead, a more savvy solution can be derived by specifying 80 ksi WWR in the original design, as the WWR is generally characterized as being able to offer superior versatility and on-site value in comparison to hand-tied individual reinforcement pieces.

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).

2. “Standard Specification for Carbon-Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete (ASTM A1064/A1064M-22)”, ASTM International, West Conshohocken, PA, 2022.