Analysis of reinforced concrete tilt-up panels utilizing high-strength reinforcing bars

by

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Abstract

Recent years have witnessed the advent of many innovative materials to the construction industry. These materials often offer benefits to the projects on which they are used, but only if they are utilized in the proper applications. Among these new materials is high-strength reinforcing steel for use in reinforced concrete structural elements. This material is not new from the perspective of chemical composition, but rather the applications that it is being selected for. The following paper details the evaluation of the use of high-strength steel reinforcement in the design of reinforced concrete tilt-up panels and compares those designs to that of standard strength reinforcement. For the purpose of this study, standard strength is defined as reinforcement having a tensile yield stress of 60ksi while high-strength reinforcement refers to reinforcing steel with a tensile yield stress of 80ksi. 120 panels are designed for both high-strength and standard strength reinforcement, and the resulting steel spacings are compared. This study provides data from which designers and contractors can improve their ability to provide quality tilt-up panel designs.

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List of Terms

ACI – American Concrete Institute

A_s – Area of Steel

A_{se} – Equivalent area of steel including effects of an axially applied load

C_e –Exposure factor coefficient

C_t – Thermal factor coefficient

D – Dead load

d – Distance from the extreme fiber in compression to flexural reinforcing

E_c – Modulus of elasticity of concrete

e – Eccentricity of axial load relative to centroid of panel

E_s – Modulus of elasticity for steel

f'_c – 28-day compressive strength of concrete

f_r – Modulus of rupture for concrete

f_y – Specified yield stress for steel

GC_p – External pressure coefficient

GC_{pi} – Internal pressure coefficient

h – Panel height

h_p – Height of parapet

I_s – Snow importance factor

I – Moment of inertia

I_{cr} – Cracked moment of inertia

K_d – Wind directionality factor

K_z – Velocity pressure exposure coefficient

K_{zt} – Topographic factor

L – Live load

l_c – Unbraced length of the panel

L_r – Roof live load

M_{cr} – Cracked moment of inertia of a concrete section

M_n – Nominal moment resisting capacity

MPH – Miles per hour

M_s – Service applied moment including P-Δ effects

M_{sa} – Initial service applied moment

M_u – Maximum factored moment including P-Δ effects

M_{ua} – Maximum factored applied moment

n – Modular ratio, modulus of elasticity of steel to modulus of elasticity

P – Applied force vector

p – Design wind pressure

p_f – Flat roof snow load

p_g – Ground Snow load

P_{sa} – Initial service applied axial load

psi – pounds per square inch

P_u – Ultimate axially applied load

P_{ua} – Axially load applied to the panel

q_z – Velocity pressure

S – Snow load

SEAOC – Structural Engineers Association of Southern California

t – Thickness of the panel

t_{NOM} – Nominal thickness of dimensional lumber forms

V – Wind velocity

w_u – Factored uniform lateral load

y_t – Distance from the centroid to the extreme fiber in tension

Δ_u – Initial deflection exhibited by application of primary moments

Δ_{cr} – Deflection exhibited at the cracked moment of inertia

Δ_n – Deflection exhibited at the nominal moment capacity

Δ_s – Service deflection

Φ – Strength reduction factor

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Chapter 1 - Introduction

The estimated volume of tilt-up construction across the United States and Canada experienced a jump from 123 million square feet in 2015 to 154 million square feet in 2016 (Tilt-Up Concrete Association, 2016). This 25 percent increase occurred in a construction environment that prioritizes sustainable and efficient designs. Many of these designs emphasize the use of less material, reducing the overall carbon footprint of the building. The use of higher strength materials is one means of reducing this footprint, as it decreases the carbon emissions associated with material production and transportation. A study on the use of high-strength reinforcing steel in the design of reinforced concrete tilt-up panels (slender walls) is presented. The demand for the application of the higher strength reinforcement has increased recently, often with the rationale that the use of stronger materials results in material and/or cost savings. The replacement of ASTM A615 carbon-steel reinforcement having a specified specified yield strength of 60 ksi (standard) with ASTM A615 carbon-steel reinforcement having a specified yield strength of 75 ksi or higher (high-strength reinforcing steel) would intuitively be associated with a decrease in material usage, but deeper understanding of the mechanics of slender wall design and high-strength reinforcement reveals that this replacement is more than a linear exchange of reinforcement based on area and tensile stress.

The code or standard that many jurisdictions have adopted for the design of reinforced concrete structures, *ACI 318-14: Building Code Requirements for Structural Concrete*, features a specific method for the analysis of slender wall elements. This method is the basis for design of the panels in this study. Section 11.8 of *ACI 318-14, Alternative method for out-of-plane slender wall analysis,* provides methods to quantify the stresses and deflections experienced by slender wall elements (panels) loaded both axially and laterally (ACI Committee 318, 2014). These panels are subject to secondary effects, or P-Δ effects, as a function of their out-of-plane deflections and continued loading. Chapter 2 of this thesis explores the *Alternative method for out-of-plane slender wall analysis*'s utilization and rationale, as well as other relevant provisions of *ACI 318-14* in regard to the design of tilt-up panels.

A comparison of standard and high-strength reinforcement is included in this study in Chapter 3 to provide a better understanding of the considerations that should be taken prior to the use of ASTM A615 Grade 80 reinforcing steel. These considerations include mechanical behavior, impacts on constructability, and the costs associated with both grades of reinforcement. The results are only relevant if considered relative to the topics discussed in the comparison.

Chapter 4 includes the setup of the parametric study, including a discussion on each of the parameters that were part of it. The study includes five different parameters: panel height, panel thickness, 28 day compressive strength of concrete, bar size and reinforcement tensile yield stress. This section summarizes the results of the study, and identifies relationships that exist between the individual parameters and the results.

Lastly, the conclusions of the study are presented based on the data that was presented as part of the experimental portion of this study. Along with these conclusions, recommendations for further research that would provide a deeper understanding of the concepts identified within this parametric study are discussed.

Chapter 2 - *ACI 318-14 Section 11.8: Alternative Method for Out-of-plane Slender Wall Analysis*

The alternative method of analysis outlined in Section 11.8 of *ACI 318-14* provides a means of designing slender wall elements that accounts for the P-Δ effects that magnify the ultimate moment experienced by the element. The P-Δ effects are the result of continued vertical loading of the wall element after it has deflected out-of-plane. The wall deflects in response to lateral loading, often in the form of wind pressures. As the panel continues to carry vertical load, it occurs at an eccentricity to the centroid of the wall element, resulting in an increased moment along the height of the panel. The method prescribed in *ACI 318-14* Section 11.8 defines these magnified moments and provides a means of approximating the associated deflections.

This method developed as the result of demand for means of design outside of the overly conservative height-to-width ratios that the American Concrete Institute (ACI) imposed on the design of slender wall panels. In 1982, the Structural Engineer's Association of California (SEAOC) conducted full scale testing of twelve slender wall panels with thicknesses ranging from 4.75 inches to 9.5 inches. The full-scale testing was conducted in order to quantify the deflections that the panels could endure prior to yielding. This study served to illustrate the over conservative nature of the height-to-width ratios that governed slender wall design at the time, and provided data on the relationship between load and deflection of the panels. Equations quantifying these deflections were devised, which served as the predecessors to those first introduced to the Uniform Building Code (Lawson, 2007). Over time, the equations in *ACI 318-14* were adapted to correlate more closely to test data.

Requirements for Use

The first section of Section 11.8 of *ACI 318-14* defines limitations to the method, identifying which wall elements may be designed utilizing the method. The limitations are as follows:

1. "Cross section is constant over height of wall,
2. Wall is tension-controlled for out-of-plane moment effect,
3. ϕM_n is at least M_{cr}, where M_{cr} is calculated using f_r as provided in 19.2.3,
4. P_u at the mid-height section does not exceed $0.06 f'_c A_g$,
5. Calculated out-of-plane deflections due to service loads, Δ_s, including PΔ effects, does not exceed $l_c/150$,"

Additionally, the panel must be simply supported, spanning from foundation to roofline or support location. These limitations seek to prevent the application of the method to panels that would develop stress concentrations or deflect differently than the panels from which the equations of the method were derived.

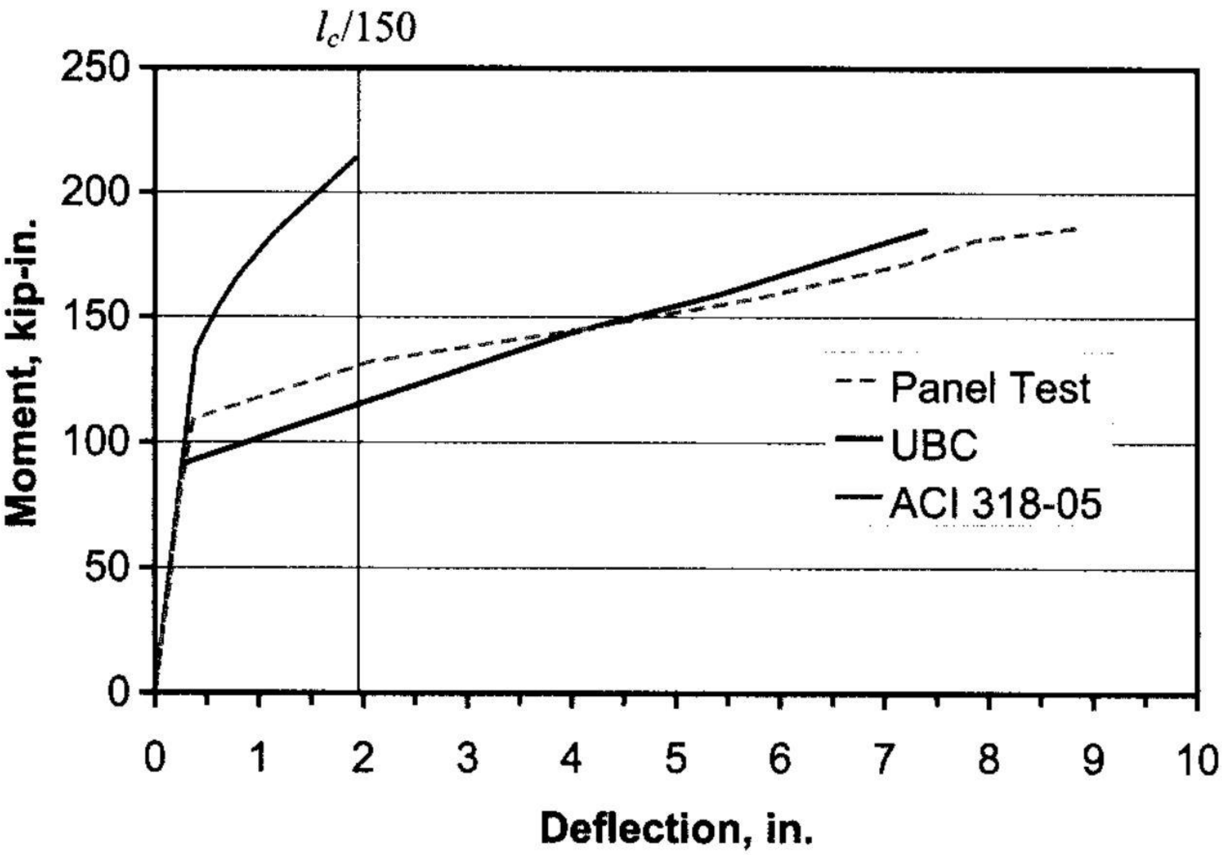

Figure 2-1 Moment curvature comparison between panel tests and UBC and ACI equations, adapted from (Lawson, 2007)

The mandate that the design moment be larger than the cracking moment, as calculated in Equation 2-7, serves to ensure that the wall does not deflect drastically upon cracking. For tilt-up panels, it cannot be determined if the panel will be cracked during the lifting portion of construction, so it is assumed that when the panel is in service, it will have cracked. Therefore, the cracked moment of inertia is used in calculating the bending stiffness of the panel section. Figure 2-1 illustrates how *ACI 318* code provisions approximate the deflections, as a function of the cracked moment of inertia.

Each of the limitations defined are met by the panel designs described in the *Results* section of this parametric study. Maintaining a tension-controlled section and limiting out-of-plane deflections to 150th of the clear span of the panel proved to be the most stringent requirements during this study. Many of the panel designs achieved a design capacity larger than the ultimate forces it experienced, but not without deflecting more than $l_c/150$ or maintaining a transitional or compression-controlled section. Since these panels did not meet all limitations, they were deemed invalid. Chapter 4 identifies the number of panels that were deemed invalid, and their parameters are listed in the design results in Appendix B.

Factored Moment

The *ACI 318-14 Section 11.8.3* of the alternative analysis method is utilized to determine the ultimate moment for which the panel must be able to resist. The moment may be determined iteratively by *ACI 318-14* Equation 11.8.3.1a or by direct calculation using *ACI 318-14* Equation 11.8.3.1d. The moment has two components; the moment at mid-height of the panel resulting from the factored out-of-plane loads, M_{ua}, and the moment resulting from the factored axial load on the panel, P_u, acting at the eccentricity of the initial out-of-plane deflections. The iterative moment is defined in *ACI 318-14* Equation 11.8.3.1a.

$$M_u = M_{ua} + P_u\Delta_u \qquad \text{(\textit{ACI 318-14} 11.8.1.a)}$$

The ultimate moment at mid-height, M_{ua}, has two components, one for the applied lateral loads that cause bending of the panel and one for the effects of axial load and panel slenderness which causes the wall panel to deform. The first component, M_{ua}, shown in Equation 2-1 is the maximum ultimate moment at the midspan of a simply supported panel with an ultimate uniform load, w_u, distributed over its clearspan, l_c. The ultimate uniform load is the components and cladding wind load or the seismic element load. Were the panel not subject to superimposed axial loads, this would be added to the moment resulting from the self-weight of the panel acting at the centroid of the laterally deflected shape. The resulting value would be the ultimate moment for which the panel would be designed. The first component is the primary contributor to the maximum moment that occurs at the mid-height of the panel. If the externally applied vertical loads, such as roof joists, did not act at the centroid of the wall, an additional moment would need to be considered and the location where the maximum moment caused by the lateral load (mid-span for pinned-pined wall panel condition) would be incorporated into Equation 11.8.1a to account for the initial roof load acting at an eccentricity. For this study, the roof joists bearing on the panels are assumed to act at the centroid of the wall panel. The slenderness of the panel generates the secondary moments that make up the second component of Equation 11.8.3.1a above. Limiting the eccentricity of the superimposed axial loads of the panel simplify the analysis of the panel, and ensure that the results of the study emphasize the effects of the slenderness of the panel rather than the eccentric loading by the joists.

$$M_{ua} = \frac{w_u l_c^2}{8} + \frac{P_u e}{2} \tag{2-1}$$

The second component of *ACI 318-14* Equation 11.8.3.1a accounts for the moments created by the vertical loading of the panel including self-weight acting eccentric to the centroid of the panel due to the deformed shape. This eccentricity, Δ_u, is approximated by *ACI 318-14* Equation 11.8.3.1b as the out-of-plane deflection resulting from the ultimate axial load, P_u, at mid-height where maximum moment is occurring. By inspection, the deflections are inversely proportional to the cracked moment of inertia of the panel, I_{cr}. The 0.75 modification to the cracked moment of inertia is applied to reduce the bending stiffness of the panel to account for any imperfections related to material flaws or poor craftmanship (ACI Committee 551, 2015). However, the document also references the design of columns a source for the stiffness reduction. This relates directly to the moment magnifier method used in the design of tied reinforced concrete columns, which may not adequately model the behavior of the untied slender wall panels in this study.

$$\Delta_u = \frac{5M_u l_c^2}{(0.75)48E_c I_{cr}} \qquad (\textit{ACI 318-14}\ 11.8.3.1\text{b})$$

The cracked moment of inertia, I_{cr}, is determined using *ACI 318-14* Equation 11.8.3.1c. The cracked moment of inertia is used to prevent the occurrence of drastic deflections and due to reduction in stiffness at the instance of cracking in the panel, as described previously. Figure 4-1 illustrates how the use of a higher moment of inertia during design would prove inaccurate, due to the uncertainty of the panel cracking during lifting. Additionally, deflections determined with the cracked moment of inertia match closely with deflections that were measured during the full-scale testing of slender walls by SEAOC in the early 1980s (ACI Committee 551, 2015). *ACI 318-14* Equation 11.8.3.1c is based on a rectangular compression block, which is assured by the alternative slender wall analysis limitation of axial loads at the mid-height of the panel to $0.06f'_cA_g$. By limiting the compressive stressed to less than 6% of the concrete compressive capacity, the derivation that is used to determine the stress-strain relationships of the section remains valid. The first component of the cracked moment determination equates the inertial effects of the effective area of reinforcement, calculated with Equation 2-2, to concrete.

$$I_{cr} = \frac{E_s}{E_C}\left(A_s + \frac{P_u}{f_y}\frac{h}{2d}\right)(d-c)^2 + \frac{l_w c^3}{3} \qquad (\textit{ACI 318-14}\ \text{Equation 11.8.3.1c})$$

The effective area of steel in section, $A_{se,w}$, seeks to quantify the initial stresses that the compressive axial loads produce in the panel section and their contribution to resisting some of the tensile stresses that occur in the panel as a result of bending. Of interest for this study is the appearance of the tensile yield stress of the reinforcement in the denominator of the second term. Panels that use standard reinforcement will experience from this effective area of steel and its influence on the bending stiffness of the panel than identical panels that contain high-strength reinforcement. The second term also accounts for the placement of the reinforcement relative to the center of the panel, which can significantly influence the stiffness of the panel (ACI Committee 551, 2015).

$$A_{se,w} = A_s + \frac{P_u}{f_y}\frac{h}{2d} \tag{2-2}$$

The alternative to the iterative method previously described is the direct calculation of the ultimate method in *ACI 318-14* Equation 11.8.3.1d.

$$M_u = \frac{M_{ua}}{\left(1 - \frac{5P_u l_c^2}{(0.75)48E_c I_{cr}}\right)} \qquad (\textit{ACI 318-14}\ 11.8.3.1\text{d})$$

Out-of-plane Deflection for Service Loads

To provide a valid panel design in accordance with the alternative method for out-of-plane analysis of slender walls, the service level deflections must be less than 150th of the clear span of the panel. The limits are imposed to prevent the possibility of exceeding the elastic deformation of the panel. If the panel were loaded to inelastic stresses, the panel would be permanently deformed. As the panel underwent cyclical axial loads in the future, the secondary moments experienced by the panel would increase exponentially, leading to the possibility of failure and collapse. These deflections are determined iteratively, accounting for deflections as a function of out-of-plane loading and service level P-Δ effects. As observed in the full-scale testing of slender tilt-up panels by SEAOC in the 1982, the out-of-plane deflections increase rapidly when the service level moment exceeds two-thirds of the cracking moment as determined in *ACI 318-14* Equation 24.2.3.5b (ACI Committee 318, 2014). The testing conducted by SEAOC suggests that the modulus of rupture, f_r, should be calculated as two-thirds the value in

ACI 318-14; the two-thirds reduction in the cracking moment serves to eliminate this discrepancy in modulus of rupture values to match full scale testing data.

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad \text{(\textit{ACI 318-14} Equation 24.2.3.5b)}$$

Therefore, for moments less than two-thirds of the cracking moment, calculated with Equation 2-3, deflections are determined as proportional to the cracking deflection, Δ_{cr}. This relationship is shown in Equation 2-3.

$$M_a \leq \left(\frac{2}{3}\right) M_{cr} \therefore \Delta_s = \left(\frac{M_a}{M_{cr}}\right) \Delta_{cr} \qquad (2\text{-}3)$$

For service moments larger than two-thirds of the cracking moment, deflections are determined as an interpolation between the deflections at the nominal moment and two-thirds of the cracking moment deflections. Equation 2-4 illustrates the interpolation to determine out-of-plane deflection of the panel as a result of service loads.

$$M_a > \left(\frac{2}{3}\right) M_{cr} \therefore \Delta_s = \left(\frac{2}{3}\right) \Delta_{cr} + \frac{\left(M_a - \left(\frac{2}{3}\right) M_{cr}\right)}{\left(M_n - \left(\frac{2}{3}\right) M_{cr}\right)} \left(\Delta_n - \left(\frac{2}{3}\right) \Delta_{cr}\right) \qquad (2\text{-}4)$$

The deflection at the cracking moment is determined as follows in *ACI 318-14* Equation 11.8.4.3a.

$$\Delta_{cr} = \frac{5 M_{cr} l_c^2}{48 E_c I_g} \qquad \text{(\textit{ACI 318-14} Equation 11.8.4.3a)}$$

For use in Equation 2-4, the deflection of the panel when the applied moment is greater than two-thirds the cracking moment, the nominal deflection is determined with *ACI 318-14* Equation 11.8.4.3b in which the panel is assumed to be fully cracked.

$$\Delta_n = \frac{5 M_n l_c^2}{48 E_c I_{cr}} \qquad \text{(\textit{ACI 318-14} Equation 11.8.4.3b)}$$

The deflections determined in Equation 2-3 or 2-4 are combined with the service axial loading of the panel to approximate the service level moment at the mid-height of the panel, evident in Equation 2-5. This process is repeated with the resulting moment to determine new deflections. Repeated iterations of this procedure will converge on a moment and associated deflections.

$$M_a = M_{sa} + P_s\Delta_s \qquad \text{(\textit{ACI 318-14} Equation 11.8.4.2)}$$

Reinforcement Limits

In addition to the prescriptions of Section 11.8 of *ACI 318-14*, the panels must meet the minimum and maximum reinforcement values defined for cast-in-place walls. *ACI 318-14* Section 11.7.2.1 *ACI 318-14* limits the spacing of longitudinal reinforcement to "the lesser of three times the thickness of the wall, or 18 inches." This value is determined for each panel design. For the design of more compact panels that utilize larger bar sizes, the maximum spacing provisions govern over strength or serviceability considerations. Additionally, minimum area of steel is required in compliance with *ACI 318-14* Table 11.6.1. For cast-in-place walls and bar sizes equal to or less than #5, the longitudinal reinforcement ratio must exceed 0.0012. The minimum value for #6 bars is 0.0015. These limitations are in place to ensure proper interaction of the wall over all sections; the minimum transverse reinforcement ties the vertical rigidity of the wall generated by the longitudinal reinforcement together. The spacing of the longitudinal reinforcement is in place for the same reason; spacing the vertical reinforcement ensures interaction between each vertical bar and allows for the wall to act as a single element than successive strips of varying stiffness. Lastly, the limitation on spacing of the reinforcement prevents the propagation of cracks that may occur in the wall. These minimum values often limit the savings that may otherwise be experienced by replacing standard reinforcement with high-strength reinforcement, as there is no adjustment of the ratios for the increase in tensile yield stress (Timothy W. Mays & Steinbicker, P.E., S.E., 2013).

Chapter 3 - Comparison of ASTM A615 Grade 80 Reinforcement to ASTM A615 Grade 60 Reinforcement

The use of high-strength reinforcement impacts the overall design of a reinforced concrete tilt-up panel in a variety of ways. ASTM A615 Grade 60 carbon-steel reinforcement is the customary reinforcement used in the design of reinforced concrete tilt-up panels. Grade 80 steel reinforcement is the high-strength reinforcement that is most widely used in non-special reinforced concrete building structures. This section compares the two grades of reinforcement on the parameters of behavior, constructability and cost to better understand how using Grade 80 steel reinforcement would impact the design of reinforced concrete tilt-up panels.

Behavior

The behavior of steel reinforcement is a function of the material properties of the steel. Any reinforcement that meets the requirements of ASTM A615 with a tensile specified yield strength that exceeds 60,000 psi is considered a high-strength reinforcement. This section examines the strength of the two grades of reinforcement and the impact that these characteristics have on the overall design of a reinforced concrete tilt-up panel.

ASTM A615 Grade 60

Grade 60 steel reinforcement meeting ASTM A615 exhibits a tensile specified yield strength of 60,000 psi. This value appears on the stress-strain curve in Figure 3-1; Grade 60 steel is shown as light blue with ASTM A615 as a solid line and ASTM A706 as a dotted line. According to the Concrete Reinforcing Steel Institute (CRSI), the plateau seen at the end of the steep slope in the stress-strain curve in Figure 3-1 for the Grade 60 reinforcement indicates that the yield stress occurs just above 60,000 psi with certainty (Concrete Reinforcing Steel Institute, 2016). Steel reinforcement engages in a reinforced concrete tilt-up panel once the concrete has begun to crack. At this point, the steel begins to elongate in tension and stresses occur in the steel. This appears in the sharp rise at the start of the stress-strain curve. Once the steel reaches its tensile specified yield strength, the stress plateaus and elongation continue.

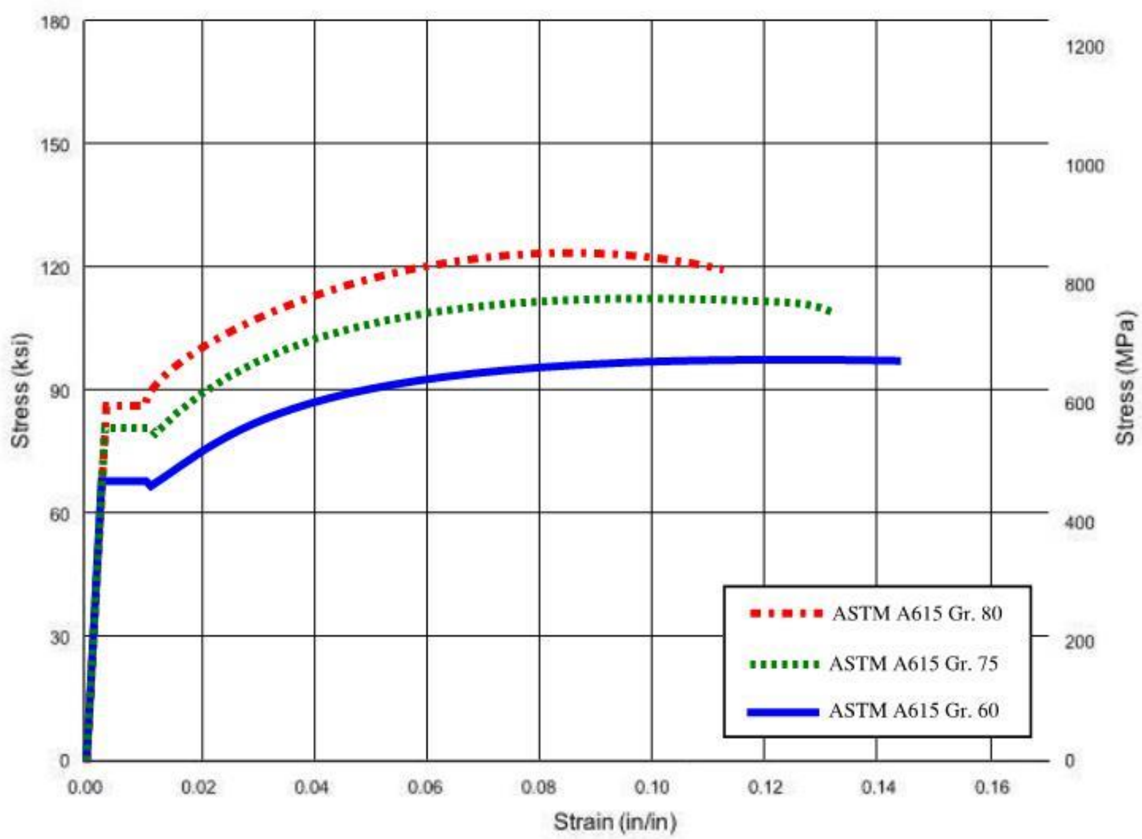

Figure 3-1, Representative stress vs. strain, Grades 60-80, Adapted from (Wiss, Janney, Elstner Associates, Inc., 2008)

Traditional reinforced concrete design limits the strain experienced by concrete in the compression of a flexural member to 0.003 in./in. When a given reinforced section is subjected to external forces perpendicular to the face of the section, tension and compression stresses develop in the section. Steel reinforcement is placed in the section is to carry the tensile stresses that develop while the concrete of the section carries the compressive stresses. The limitation of the concrete strain associated with the internal stresses that develop represents the strain at which concrete crushes. If concrete were to crush, spalling would occur, displacing the material of the section and leading to potential brittle failure of the element. The stresses that develop in the section are idealized as tension and compression forces that exist within the reinforcement and a portion of the concrete, respectively. The forces, when summed as moments around the neutral axis of the section, result in the nominal moment capacity of the section. To attain equilibrium in the section and resolve the external forces introduced by loading, the internal compression forces and tension forces must equal. These forces are comprised of stresses acting over an area of material; the compressive over a block of concrete in the section, and the tensile over the area of reinforcement located opposite the neutral axis of the concrete compression block. The designer adjusts the area of steel in that section in order match the compressive force generated by the external loading. As mentioned, the internal tension force is generated by tension stresses acting over the area of the reinforcement. Therefore, the increase in specified tensile yield stress from Grade 60 to Grade 80 requires less of an area of reinforcement to develop the same magnitude of internal tension force.

ASTM A615 Grade 80

For the purposes of this section and the discussion on mechanical properties of ASTM A615 Grade 80 reinforcement, Grade 80 reinforcement behaves similarly to ASTM A615 Grade 75 reinforcement exhibited on the stress-strain curve in Figure 3-1. A plateau occurs on the strain curve just above 80,000 psi, indicating the yield stress of the steel. This clear plateau indicates with certainty, that the material is yielding. This plateau is not present in high-strength reinforcement, but is part of the reason *ACI 318-14* allows the use of this material in non-special lateral force resisting systems. Since the material demonstrates a clear yield, engineers can be comfortable utilizing it in their designs. As discussed earlier, the reinforcing steel engages in tension after the concrete has cracked, at which point the limitation on the strain of the concrete in the compression region of the member governs the strength capacity of the section. However, according to Hugh Brooks of the Tilt-Up Concrete Association, "the lateral deflection of the panel at mid-height is the controlling condition that requires a minimum area of steel independent of the specified yield strength" (Tilt-Up Concrete Association, 2011). The limitation of the out-of-plane deflections imposed by *ACI 318-14 Section 11.8,* as well as the other limitations found in that provision, often govern the design of tilt-up panels. Therefore, the bending stiffness of the panel becomes the defining characteristic rather than the bending strength. As discussed in *Section 2*, the bending stiffness of the panel is based on the cracked moment of the panel section. The cracked moment is positively related to the equivalent area of reinforcement, which decreases as specified tensile yield stress increases. Therefore, there is no advantage of a higher strength reinforcement for resisting out-of-plane deflections. In a pure tensile loading condition, the superior tensile strength of Grade 80 reinforcement becomes preferable, as deflections associated with compression of a slender element and associated secondary moments are not a design consideration.

Constructability

The constructability of a structural element is of similar importance as the strength of the element. If the element is not easily constructed, it may be constructed incorrectly and therefore, may not be adequate to resist imposed loads. In addition, time is money and contractors may find other solutions that reduce construction difficulty and therefore time. This section details how the two grades of reinforcement compare in terms of ease of construction. Tilt-up panels are generally characterized by a mat of reinforcement with bars running longitudinally and transverse to the plane of the wall. Often, the slab on grade that the panels are cast on will later serve as the slab on the interior of the building. Chairs that support the mat reinforcement mat are placed on the slab, and then the transverse and longitudinal reinforcing steel is placed. The reinforcing steel is generally spaced far enough apart that congestion is not an issue, but traditional vibration of concrete is still necessary. Once the concrete is placed and cured sufficiently, a crane lifts the panels into their permanent vertical position. Prior to the placement of the concrete, inserts are tied to the reinforcement mat. These inserts are the location that the crane rigging later connects to lift the panels. Once the panels have been placed vertically, they are laterally braced until they can be permanently connected to adjacent panels and the lateral force resistance system.

ASTM A615 Grade 60

Engineers traditionally specify ASTM A615 Grade 60 reinforcing steel for all rebar within a concrete tilt-up panel (Tilt-Up Concrete Association, 2011). For singly reinforced panels, laborers tie a primary reinforcing steel mat of vertical and horizontal reinforcement placed at mid-depth of the panel. A mat of reinforcing steel for a typical solid panel without openings can generally be completed by a two-man crew within a half hour (Tilt-Up Concrete Association, 2011). Concrete placers walk on this mat during the placing of the concrete for the panel. The ability to walk on or between the grid of reinforcement is important during the placement of fresh concrete. If it is difficult to do so, it will take longer for the placement of concrete. The stability of this mat depends on the spacing and number of the rebar; smaller bar numbers indicate less stiff reinforcing. Less stiffness in the mat makes it more difficult to travel on. Likewise, increasing the spacing between the bars may result in a less rigid mat that bounces or bends when walked on. For example, a mat that utilized #6 bars at a spacing of 8 inches would be more rigid and more easily walked on than a mat built of #4 bars at 14 inches on center. Where joist and beam bearing occur, engineers specify increased amounts of reinforcement to resist the associated stress concentrations at these locations. This increased volume of reinforcement causes congestion. To handle the additional forces at the interaction of the panel and a beam bearing, reinforcement spacing may be decreased from 10 inches to 8 inches. The additional reinforcement helps to carry larger tension stresses that are introduced at this section by the beam. This congestion potentially creates difficulties in verifying that concrete properly consolidates. Concrete subcontractors must spend extra time at these locations vibrating the concrete to increase consolidation, which can delay the overall construction of the panel.

ASTM A615 Grade 80

The higher tensile specified yield strength of ASTM A615 Grade 80 reinforcing bars allows designers to decrease the bar number or increase the spacing of reinforcement, decreasing the total amount of rebar. Walls are often designed as one-foot wide strips, and the necessary reinforcement is selected for each one-foot section. This design strip is then repeated over the length of the wall. The increase in spacing, or decrease in bar number, would correlate to a decrease in the steel that is present is this one-foot strip. This overall reduction reduces the time needed to complete the tying of the reinforcement mat. However, the increase in spacing of the reinforcement also decreases the stability of the mat, making it harder for laborers to navigate the cage during concrete placement. This decreased time in the reinforcement tying portion of the construction schedule may equally increase the time it takes for concrete placers to complete their portion of the schedule. Therefore, the reinforcement substitution may not save any time. This relationship could be quantified through a full cost/schedule analysis utilizing the results of this study.

At areas of high reinforcement congestion, the smaller rebar sizes or few number of bars increases concrete consolidation and decreases the time needed to vibrate the concrete at these locations.

Cost

ASTM A615 Grade 60

Engineers use Grade 60 reinforcement as a standard when designing reinforced concrete tilt-up panels (Timothy W. Mays & Steinbicker, P.E., S.E., 2013). High-strength reinforcement emerged on the construction market in 1959, but weren't adopted by code provisions for use in reinforced concrete construction until the publication of ACI 318-08 (Concrete Reinforcing Steel Institute, 2016). Since Grade 60 is the most common, concrete subcontractors find it to be available in all building markets. This widespread availability and the volume that the contractor purchases the reinforcement makes Grade 60 reinforcement the least expensive option, with a cost of $2,025/ton including placement costs and contractor markup (RSMeans, 2011). As seen in the previous section detailing the behavior of the ASTM A615 Grade 60 reinforcement, designs that utilize ASTM A615 Grade 80 reinforcement may require less steel volume to be purchased. This reduced volume could inhibit the qualification for reduced unit costs that may be available with higher volume purchases. Although the difference in total tonnage between a panel designed with ASTM A615 Grade 60 reinforcing steel and that of ASTM A615 Grade 80 may be small, that difference is multiplied over the course of the project, and can lead to significant changes in the overall volume of steel.

ASTM A615 Grade 80

Grade 80 reinforcement was first included in ASTM A615 in 2009, and which was later adopted in *ACI 318-11* (Concrete Reinforcing Steel Institute, 2016). However, due to the demand for high-strength reinforcement, Grade 80 is available in most markets. The production process associated with Grade 80 reinforcement includes superior materials or methods that result in a higher price than that of Grade 60. Depending on location, Grade 80 can carry a 2% premium over the price of Grade 60 reinforcement in that same market (Schwinger, 2011). The volume savings however, may justify the material premium. Any ease of construction introduced with Grade 80 correlates to cost savings on the overall construction budget, adding to any savings incurred during the material purchase.

Chapter 4 - Parametric Study

This parametric study seeks to quantify the effects of designing a reinforced concrete tilt-up panel with ASTM A615 Grade 80 reinforcement rather than the current industry standard, ASTM A615 Grade 60. The parameters of the study include panel height, panel thickness, 28 day compressive strength of concrete and size of bar. These parameters mostly commonly vary between construction projects and best demonstrate the effects of difference in reinforcement tensile yield stresses. This study utilizes *Section 11.8 ACI 318-14: Alternative method for out-of-plane slender wall analysis* as the basis of design. Appendix A illustrates the process utilized to design each panel. The contents of this chapter include a description of the conditions for the which the panels are designed, a discussion of each parameter the study evaluates, and the results of the study.

Building Conditions

The parametric study evaluates panel designs utilized in a single-story warehouse type building in an arbitrary location. The arbitrary location allows for freedom in the selection of loading conditions. The selected loading conditions are characteristic of a large portion of the United States of America as shown in Figure 4-1 below. Figure 4-1 illustrates the areas of the United States of America where the wind and snow loads selected for the study coincide.

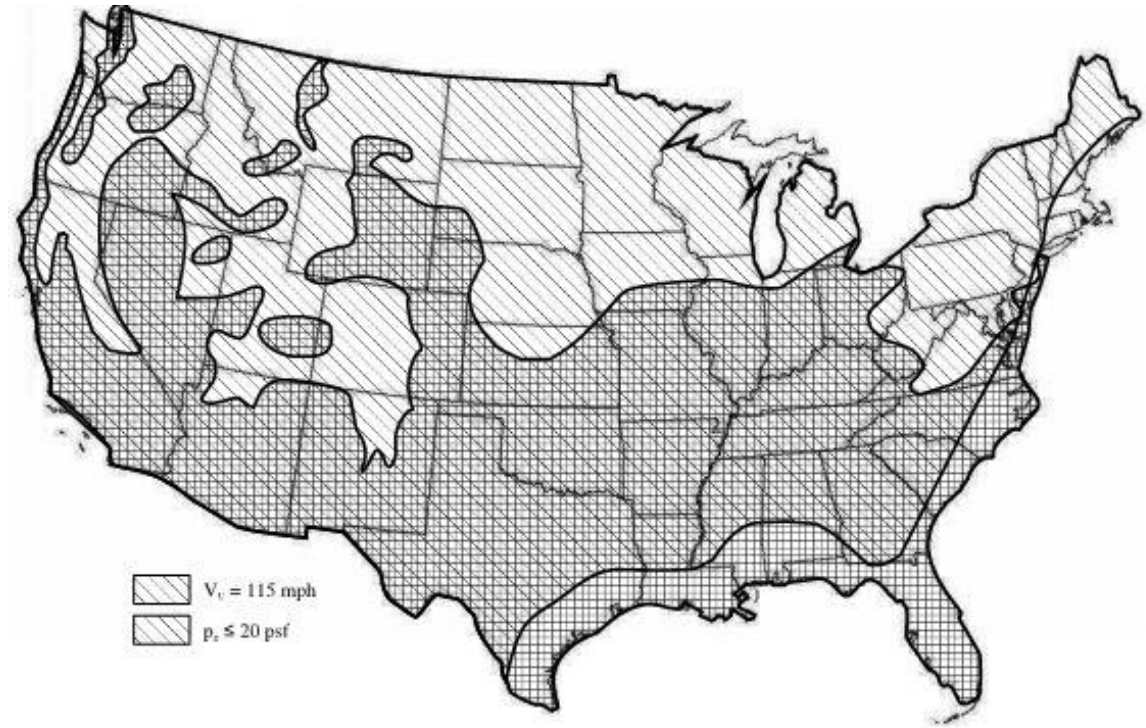

Figure 4-1 Area of United States of America represented by study

The building measures 160 feet by 320 feet, with the panel analyzed located at the middle of the wall running parallel to the 320 foot dimension with roof framing spanning to the wall panel. This specific panel location features uniform wind pressures and vertical loading from the roof structure the panel supports. The building is composed of a steel deck on bar joist roof structure, supported by the wall panels at the exterior and wide flange girders at the interior. The building floor plan is show in Figure 4-2 below.

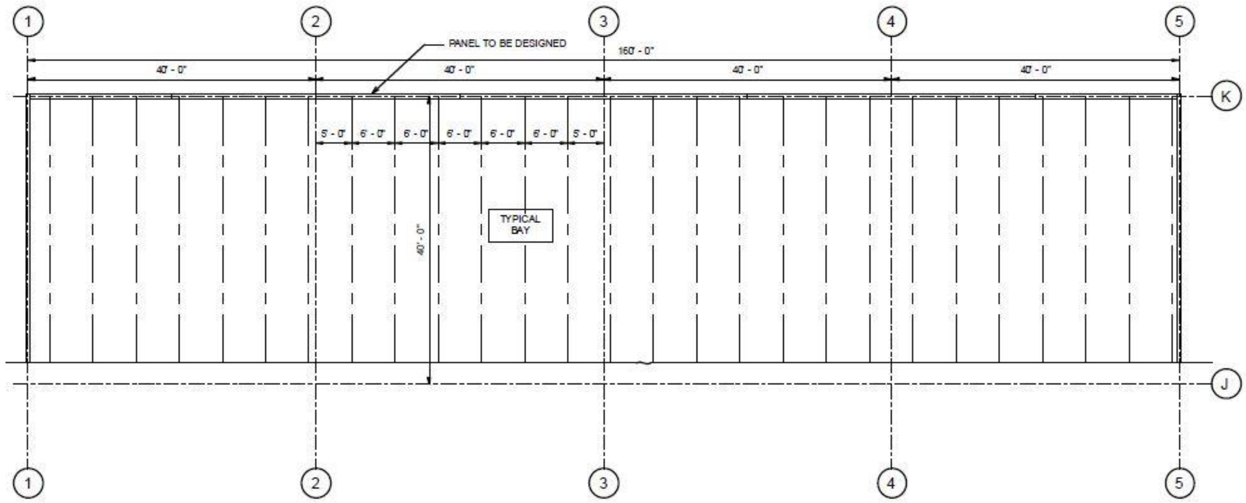

Figure 4-2, Plan view of parametric study case building

Typical bay size is 40 feet by 40 feet, so the panel is influenced by half the span distance of the bar joists at the roof level. The spacing of the bar joists is driven by the allowable span distance of traditional roof decking, and was selected as six feet from center-to-center of joist to meet Factory Mutual Insurance, a large insurer of commercial buildings, requirements (Vulcraft, 2008). It is customary in tilt-up construction that the individual panel dimensions are determined by the lifting capacity of the picking crane. Therefore, a panel width of 20 feet was selected for all panels; this ensures that no panel would achieve a weight that would be outside of the lifting capacity of potential picking cranes. The panels were modeled as spanning from the foundation to the roof level, with pinned boundary conditions at each mechanical connection location. Figure 4-3 shows how the panels in this study are idealized.

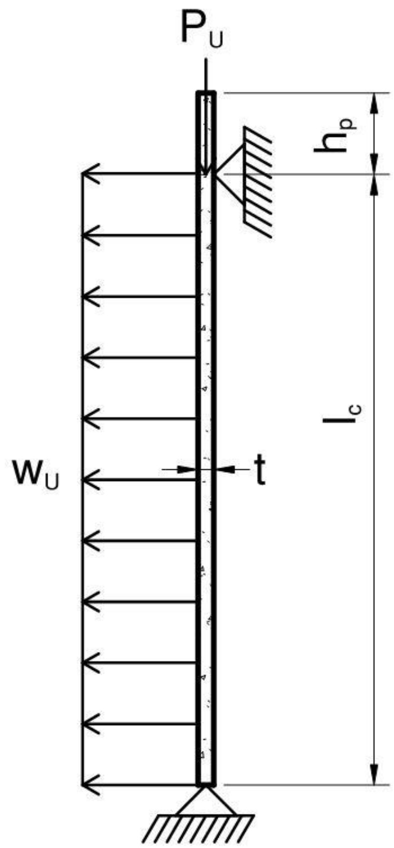

Figure 4-3, Panel loading and boundary conditions

This idealization prevents transmission of moments through the base of the panel into the foundation, and locates the maximum moment at the midspan of the panel. A four-foot parapet runs the perimeter of the building and is integral with the construction of the panel. Conservatively, the effects of the parapet on the stresses that occur within the panel are neglected when creating a favorable condition and included when they lend to a worst-case scenario. The inclusion of these effects is detailed with the appropriate load discussion.

Loads

The panel design portion of this study includes the determination of vertical and horizontal loads for each panel. The loads are derivative of the parameters selected for each panel. These panels were subject to loads in the vertical plane of the wall and out-of-plane loads, specifically dead, live, snow and wind loads. The study neglects in-plane loading of the panels to isolate the relationship between the longitudinal reinforcement of the wall and secondary stresses slender panels experience as a function of out-of-plane loading. All loads are determined in accordance with *ASCE 7-10 Minimum Design Loads for Buildings and Other Structures*. Wind pressure represents the out-of-plane loads that the panels are subject to, and are determined with *ASCE 7-10 Chapter 30: Wind Loads – Components and Cladding (C&C)*. These loads in combination with the Load and Resistance Factor Design (LRFD) Load Combinations from Section 1.1.5 of *ASCE 7-10* to determine the ultimate design stresses.

Gravity Loads

The panels are subject to dead, roof live and snow loading. These vertical loads act upon the roof deck, which translate into the bar joists, and then to the tilt-up panels. These loads determine the axial loading the panel experiences, which have a magnified effect on the stresses experienced by the panel due to the secondary stresses that occur within the panel as a function of its slender nature. The bar joists are spaced at six feet on center, so each panel is subject to loading from four individual joists. The vertical loads were approximated as uniform due to their equal magnitude and spacing.

Dead Load

The dead load experienced by the panel at the location of analysis is the summation of the weights of the roofing materials within the tributary area of the panel, the weight of the parapet that occurs above the roof and is integral with the panel, and the self-weight of the panel. A metal deck on open web bar joists with four inches of rigid insulation is assumed. Values for the weight of the materials were taken in accordance with Table C3-1 Minimum Design Dead Loads of *ASCE 7-10*. The components of the roofing assembly and their associated self-weights are summarized in Table 4-1.

Table 4-1, Dead load at roof

Dead Load				
ROOF: BY PANEL				
			MULT.	psf
METAL DECK =	2	psf	1	**2**
INSULATION =	1.5	psf/in	4	**6**
MEP =	5	psf	1	**5**
JOIST =	12	plf	0.17	**2**
			TOTAL =	15

In accordance with the alternative method out-of-plane analysis of slender walls, the location of analysis is at the midpoint of the panel between the foundation and the roof when the panel is idealized as a pinned-pinned condition. Therefore, only the self-weight of the parapet and half the panel height is considered in the determination of the ultimate vertical stresses since this is the location of the maximum combined stresses in the panel. In the case of dead load, the effects of the parapet are included as they increase the ultimate axial load transferred to the panel. This increase in axial load contributes to the additional stresses that occur due to secondary effects.

Roof Live Load

The live load affecting the design of the panels in this study occurs at the roof level. A minimum construction live load of 20 psf in accordance with *ASCE 7-10* is used over the area of the roof that influences the panel.

Snow Load

A ground snow load of 20 psf was selected for the study; the ground snow maps found in Fig. 7-1 of *ASCE 7-10* show that this value accounts for a large portion of the country, illustrated in Figure 4-1, which fulfills the goal of representing typical values for an average building in the United States of America. This ground snow load is converted to a flat roof snow load by Equation 4-1 below to determine the flat roof snow load:

$$p_f = 0.7 C_e C_t I_s p_g \qquad \text{(}\textit{ASCE 7-10}\text{ EQUATION 7.3-1)}$$

The values aside from the ground snow load in Equation 7.3-1 were selected based on assumed conditions. The exposure factor, C_e, was chosen to be 0.9, based on an assumed Category C exposure and the fully exposed nature of the warehouse roof. With such a large footprint, no obstructions near the roof to prevent the wind from blowing snow off the roof is assumed. In determining the thermal factor, C_t, an insulated roof is assumed, resulting in a value of 1.2. This implies that if the occupiable space immediately below the structure of the roof is heated, the insulating material has a thermal resistance high enough to preclude the transmission of enough heat energy, preventing the melting of snow. Lastly, the warehouse occupancy of the building and classification as a Risk Category II structure leads to a snow importance factor, I_s, of 1.0, as a failure does not lead to an extraordinary loss of life.

In regions of the country where environmental conditions exist such that snow and rain can occur in the same storm, Section 7.10 of *ASCE 7-10*, the ground snow load is less than or equal to 20 psf, so a rain-on-snow surcharge of 5 psf must be applied to the value resulting from Equation 4-1. This surcharge accounts for climates where precipitation may begin as snow, but transition to rain after snow has accumulated, resulting in a denser loading condition, and therefore inability for the wind to blow snow away. Minimum flat roof snow values are also checked in accordance with *Section 7.3.4 Minimum Snow Load for Low-Slope Roofs, p_m.*

The presence of a parapet at the perimeter of the study building introduces opportunity for the formation of a snow drifts. Snow drifts are taken as a surcharge on the flat roof snow load, and dependent on the upwind length of the roof over which snow can be picked up by the wind. This snow is then deposited on the back face of the parapet, leading to an increased snow load. Drift depth, h_d, is calculated with Equation 4-2, which comes from Figure 7-9 of *ASCE 7-10*:

$$h_d = 0.75\left(0.43\sqrt[3]{l_u}\sqrt[4]{p_g + 10} - 1.5\right) \tag{4-1}$$

This value is utilized to determine a maximum drift load, and *ASCE 7-10* assumes a load distribution that decreases from the maximum value to the flat roof snow load as determined above. For this study, an average of the flat roof snow load and maximum drift load was taken over the entirety of the tributary area of the panel.

Lateral Loads

This study focuses on the effects of wind loads as the predominant lateral load experienced by the tilt-up panels. The lateral loads play a significant role in the magnification of the out-of-plane deflections that generally govern the design of the tilt-up panels. Seismic loading of the panels was not considered. The neglection of seismic forces limits the applicability of the results generate in this study to regions that seismic loading does not generate larger forces than wind loading.

Wind Load

Components and cladding loads were determined for the panels utilizing the methods outlined in *ASCE 7-10 Section 30.4: Part 1: Low-Rise Buildings*. With a mean roof height less than 60 feet and the least horizontal dimension, the sample building for this study meets both requirements for use of this section. Components and cladding loads were selected rather than Main Wind Force Resisting System (MWFRS), as the pressures associated with a single element tend to be larger when distributed over a smaller effective wind area. This method gives design wind pressures based on *ASCE 7-10* Equation 30.4-1 shown below:

$$p = q_h\left[\left(GC_p\right) - \left(GC_{pi}\right)\right] \quad \text{(\textit{ASCE 7-10} EQUATION 30.4-1)}$$

The design wind pressures are based on the velocity pressure at the mean roof height, q_h, and the internal and external pressure coefficients, GC_p and GC_{pi}, respectively. The warehouse occupancy of the study building implies the use of large overhead doors at loading dock locations, and without certainty as to the balance of openings on leeward and windward sides of the building at any given time, a partially enclosed classification is conservatively taken. This classification results in an internal pressure coefficient of ± 0.55 per *Section 26.11 Internal Pressure Coefficients*. The external pressure coefficient is a function of the location of the panel along the exterior of the building. For simplicity, the study locates panels in Zone 4 of the building. Locating the panels in Zone 4 ensures that they are not subject to the heightened negative wind pressures that are observed at the corner of buildings due to the fluid nature of wind. The external pressure coefficients are interpolated for as a function of the effective wind area of the element in question. As discussed later in this section, the smallest area of any panel in this study is 400 ft^2, which approaches the upward bound of the interpolation, 500 ft^2. As the effective wind area approaches this upper bound, the resulting pressures converge on values that would be experienced by the Main Wind Force Resisting System at this location. Figure 30.4-1 of *ASCE 7-10* illustrates the values used for interpolation of this coefficient. The last component of Equation 4-3, the velocity pressure, is determined using Equation 30.3-1 shown below:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 \qquad \text{(}\textit{ASCE 7-10}\text{ EQ. 30.3-1)}$$

The study building is assumed to have Exposure C, as defined in section 26.7. Exposure C was selected based on the high likelihood that the warehouse would be located in an industrial area with few nearby obstructions of 30 feet or taller. This exposure, in combination with the mean roof height that is defined for each panel, determines the velocity exposure pressure coefficient, K_z. The topographic factor, K_{zt}, takes into account the elevation of the base of the study building relative to the surrounding area. The study building is not found on a hill or in a depression, so K_{zt} is taken at the default value of 1.0. The wind directionality factor, K_d, is taken as 0.85 to account for the possibility of wind acting on the building from a direction other than perfectly orthogonal to its face. Lastly, this study takes the basic wind speed used in determination of the velocity pressure, V, to be 115 mph. In viewing the basic wind speed map for Risk Category II buildings in Fig. 26.5-1A of *ASCE 7-10*, a basic wind speed of 115 mph is characteristic of a majority of the continental United States. This value represents the design wind speed of a 3-second gust that would occur 33 feet above the ground for an Exposure C building. Therefore, no adjustment is necessary for the use of this value in determination of wind pressures.

This study idealizes the wind pressures determined for each panel as a uniform distributed load over a one-foot-wide strip of the panel. Table 4-2 below contains the positive and negative internal pressure coefficients that were calculated for each panel height.

Table 4-2, Internal pressure coefficients by panel height

Panel Height (ft)	Area (ft^2)	GCp(-)	GCp(+)	p (-) psf	p (+) psf
20	400	-0.78	0.68	-34.32	31.96
25	500	-0.72	0.63	-34.35	31.92
30	600	-0.72	0.63	-35.82	33.28
35	700	-0.72	0.63	-36.91	34.30
40	800	-0.72	0.63	-38.01	35.32

As is evident in Fig. 30.4-1 of *ASCE 7-10*, the negative pressure coefficient for a wall element in Zone 4 is always greater in magnitude than the positive coefficient for a given effective wind area. Therefore, the negative wind pressure governs over the positive wind pressure for each panel. With the governing pressures acting away from the panel, the internal face of the panel acts as the extreme compression fiber.

This study neglects the effect of wind pressures on the parapet to maintain the location of the maximum moment at the midspan of the panel. If the effects were accounted for the, the negative moment generated by the cantilevered parapet would relocate the location of the maximum out of plane deflection and moment. As discussed previously, the alternative method of analysis for slender walls can only be utilized if the wall is simply supported and maximum moment and deflections occur at mid-height. Accounting for the wind pressures on the parapet would invalidate the alternative method of analysis. Additionally, neglecting the effects of the parapet wind loading results in a reduced out of plane deflection. The panel designs generated by this study are adequate for larger out of plane deflections, and therefore conservative. By the same reasoning, the effects of uplift on the roof are neglected, as they would reduce the axial load. This reduction would reduce the additional stresses that result from the panel being loaded while deflecting out of plane.

Load Combinations

The panel design of this study determines the ultimate vertical and out of plane loads with the combinations found in Table 5.3.1 of *ACI 318-14*. These loads are used in conjunction with the alternative analysis methods outlined in the Section 11.8. The load combinations are listed below:

1. $U = 1.4D$
2. $U = 1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3. $U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.5W)$
4. $U = 1.2D + 1.0W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $U = 1.2D + 1.0E + 1.0L + 0.2S$
6. $U = 0.9D + 1.0W$
7. $U = 0.9D + 1.0E$

For the purpose of this study, the out of plane wind loads and vertical live and dead loads will contribute most to the ultimate forces to be resisted. By inspection, Combinations 3, 4, and 6 will govern over the other combinations as they account for the effects of roof live load, L_r, and wind load, W. Since the loads of interest act in different directions, the true governing combination cannot be determined until the ultimate moment is determined through the analysis methods described previously. Therefore, the design portion of this study produces a panel design for each load combination, and the design that results in the largest ultimate moment at the mid-height of the panel governs. The study gives additional consideration to the requirements of the analysis method described in Chapter 2 of this paper, and only validates a panel design if all the limitations are met for each of the three load combinations considered.

Parameters

Panel height, panel thickness, 28-day compressive strength of concrete, bar size, and tensile yield stress of reinforcement were selected as the parameters of this study with the intent of representing typical conditions that would be altered during design to meet different building needs. Each parameter is compared to the spacing of the final design for the given panel to understand the relationship between the parameter and the resulting usage of steel. All possible combinations of the five parameters were evaluated, totaling 240 individual panel designs. For some parameter combinations, a viable design was not achievable within the limitations of the alternative analysis method described in Chapter 2 of this paper.

Each panel is designated by a unique name that defines each of the parameters of the panel. Figure 4-4 shows the typical designation and how each parameter of the panel design is incorporated into the designation. For example, a panel with a height of 40', that has a nominal thickness of 8", a 28 day compressive concrete strength of 3000 psi, a tensile yield stress of 80 ksi, and utilizes 2 layers of #4 bars would be designated with 40'.8".3.80.#4.2.

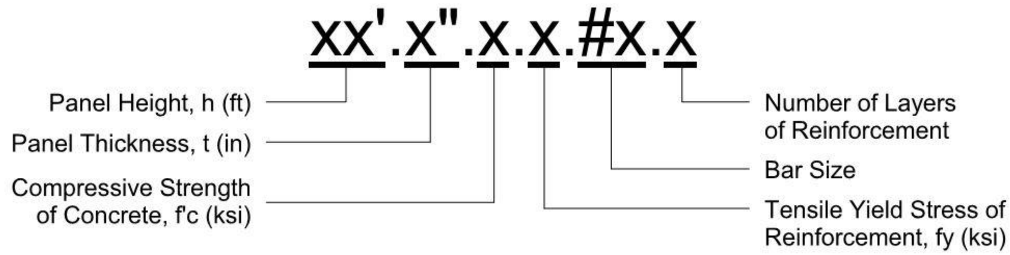

Figure 4-4, Panel designation

Panel Height

Panel height refers to the distance between the base of the panel and the connection to the roof diaphragm. Panel heights were selected on a range from 20 feet to 40 feet in increments of five foot. The height range was selected based on a constant panel width of 20 feet, and adheres to a total panel weight of 60 tons or less which is based on the lifting capacity of a traditional picking crane. The height of the panel has a large impact on both the ultimate moment and out-of-plane deflection of the panel at mid-height. Equation 2-1 and ACI 318 Equation 11.8.3.b1 illustrate that the ultimate moment and deflection of the panel are exponentially proportional to the panel height.

Panel Thickness

This study selects panel thickness dimensions based on the depth of dimensional lumber, without ripping the lumber down, that is typical for use in formwork of tilt-up panels. Three thicknesses were used: 5.5 inches, 7.25 inches, and 9.25 inches. These thicknesses correlate with 6-inch, 8-inch and 10-inch dimensional lumber nomenclature, respectively. Throughout this document, the panels are referred to with the dimensional lumber nomenclature. Adhering to dimensional lumber actual depth equaling the panel depth is typical of a tilt-up construction project, as it allows for economy in the formwork portion of the construction budget, including labor costs. The study uses one layer of reinforcing steel for 6-inch panel solutions and one set of 8-inch panel solutions. Conversely, one set of 8-inch panel solutions and all 10-inch panel solutions use two layers of reinforcement. The inclusion of two sets of 8-inch panel designs, one with one layer of reinforcing steel and one with two layers of reinforcing steel, strives to illustrate the effects of increasing the internal moment arm between the tensile reinforcement and the extreme compression fiber in relation to reinforcement tensile yield stress. Figure 4-5 exhibits sections for a singly reinforced panel and a doubly reinforced panel.

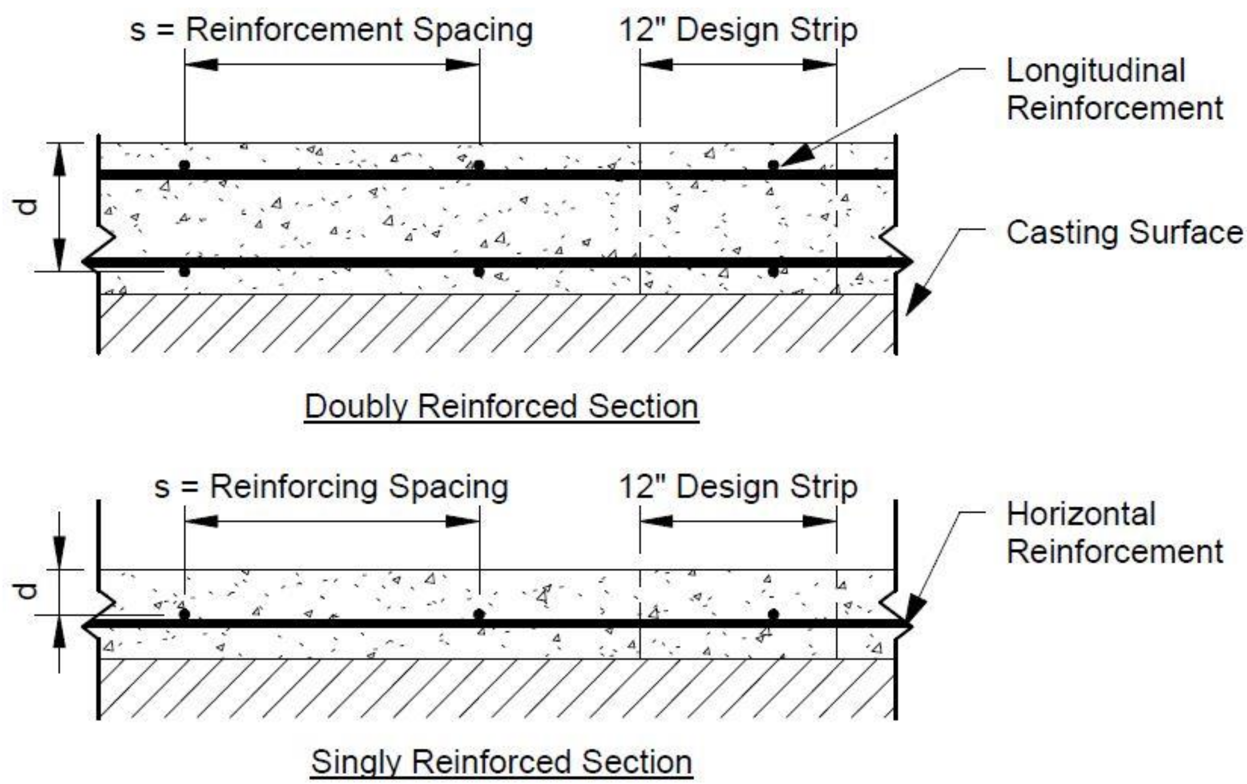

Figure 4-5, Singly and doubly reinforced panel sections

The internal moment arm directly affects the cracking moment of inertia of the panel, which limits the out-of-plane deflection of the panel. The larger the cracking moment of inertia, the stiffer the panel and the more out-of-plane deflections are minimalized (ACI Committee 551, 2015). Panels that have two layers of reinforcing are referred to throughout this study as doubly reinforced. It should be noted that when two layers of reinforcement are present, the design process of this study neglects the contribution of reinforcement in the compression region of the section to the compressive strength of the section. Since panels of this study are designed as untied, the compression reinforcement is uncontained, and buckles easily. Also, if the neutral axis of a section moves to a location such that both layers of reinforcement is subject to tension stresses, the contribution of the layer closest to the neutral axis is neglected. The stresses accumulated in this would be small relative to those in the extreme tension layer, and can conservatively be considered negligible. Additionally, chair height is determined in relation to the thickness of the panel. The chair height and the diameter of the reinforcing steel determines the location of the centroid of the tension steel from the extreme fiber in compression. For doubly reinforced panels, this study selects chairs appropriate to support the longitudinal reinforcement with a cover of 1-inch. In accordance with Table 10.1b of ACI 551: Design Guide for Tilt-Up Panels, a 1-inch concrete cover is appropriate for panels cast against earth and exposed to weather. For singly reinforced panels, the design uses chairs that place the reinforcement closest to the center of the panel.

Concrete Compressive Strength

Concrete compressive strength, f'_c, is limited to 3000 and 4000 psi mixes. 3000 psi mixes represent the minimum strength concrete that should be used for the construction of tilt-up panels (Tilt-Up Concrete Association, 2011). Maintaining a minimum of 3000 psi concrete ensures that panel will be cured in a reasonable amount of time following its casting. The concrete compressive strength plays a role in the strain compatibility that is utilized to determine the depth of compression block in the design of the panels. Utilization of high-strength concrete in conjunction with the use of high-strength reinforcement may provide for improved use of both materials, but falls outside the scope of this study. Introducing the small variance in strengths gives some reference to the relationship between the tensile yield stress of the reinforcement and the compressive strength of the concrete, but does not venture outside of the traditional mixes for tilt-up construction.

Bar Size

Traditionally, bar size within tilt-up construction is limited to #4, #5 and #6 (Tilt-Up Concrete Association, 2011). Therefore, this study limits longitudinal bar sizes to these, and utilizes #4 bars for all transverse reinforcement. The transverse reinforcement is designed per *ACI 318-14* Table 11.6.1 to meet minimum reinforcement ratios for cast-in-place walls with ultimate in-plane shear loads less than half of the design concrete shear capacity. These criteria are satisfied since most tilt-up panels do not experience large in-plane shear loads relative to their capacity; this trend provided the rationale to neglect in-plane loading for this parametric study.

Reinforcement Tensile Yield Stress

This study includes designs for all possible combinations of height, thickness, compressive strength of concrete, and bar size for both ASTM A615 Grade 60 and ASTM A615 Grade 80 reinforcement. This parameter affects the strain compatibility aspect of the design, as well as the determination of the effective area of steel. This effective area is utilized to find the cracking moment of the section, which is inversely related to the out-of-plane deflections of the panel and associated P-Δ effects. As the focal point of this study, the final designs for each grade of steel is compared in the results section of this paper.

Results

Each panel design generates an optimal spacing to meet the ultimate moment applied to the panel while maintaining an out-of-plane deflection that meets the limits imposed by the alternative method of slender wall analysis. The spacing is limited to increments of 1-inch to maintain a level of constructability that was practical; spacing to fraction of an inch would prove difficult to measure in the field during construction and efficiency would be sacrificed. The spacing is inversely related to the area of steel per one-foot design strip of the wall, and therefore total tonnage of reinforcement within the panel. As mentioned previously, some combinations of parameters did not allow for valid designs within the limitations of the alternative methods of slender wall analysis. Additionally, these panels met the minimum and maximum requirements for spacing and reinforcement ratio. Figure 4-3 below categorizes each of the 120 ASTM A615 Grade 80 panel designs produced by the study relative to the designs produced by keeping all parameters constant and changing the reinforcement to ASTM A615 Grade 60. This comparison led to 120 data points that could be categorized as the following: increase in spacing, decrease in spacing, no change in spacing, invalid design, or design becomes valid. Of the 120 unique spacing comparisons, seventy-two produced designs that could meet the limitations of ACI 318 as described in Chapter 2. For the designs that yielded invalid designs, no spacing produced a design within the limitations. Panels 40'.8".4.80.#4.2 and 40'.8".4.80.#5.2 produced valid designs, while panels of the same parameters that utilized Grade 60 reinforcing steel did not. These two panels represent the "DESIGN BECOMES VALID" category below.

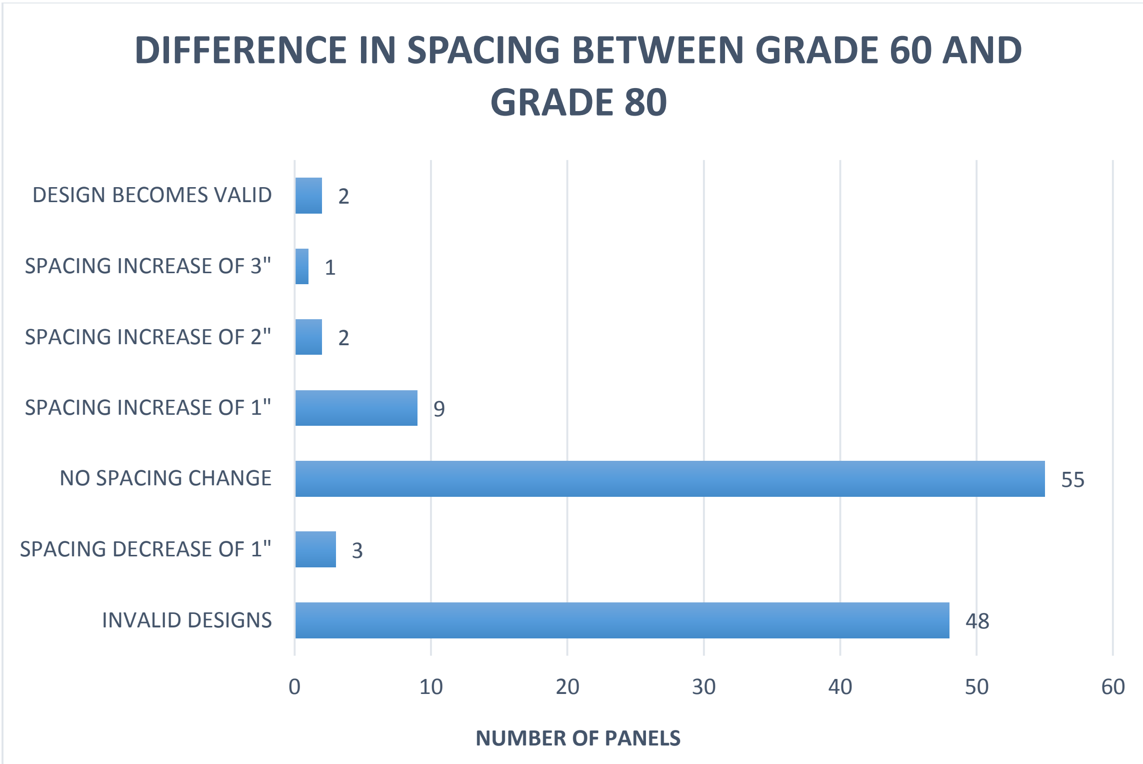

Figure 4-6, Summary of Spacing Differential for All Panels

The data in Figure 4-3 shows that most panel designs, 45.8%, resulted in no spacing change between grades of reinforcing steel. When the number of designs experiencing no change, 55, is compared to the number of designs that produced viable results, 72, it becomes clear that a majority of panels did not experience a reduction in steel tonnage. However, the utilization of ASTM A615 Grade 80 reinforcement did increase required spacing for 20% of the panels that produced designs, whereas spacing was decreased for only three of the 72 viable panel designs. This percentage includes those designs that were unachievable with Grade 60, but were valid for Grade 80. Many of the panels that experienced no change in spacing were limited by the maximum spacing provisions of *ACI 318-14*, and may have had increased reinforcement savings if this limit was not imposed.

As would be expected, as panel height increased, deflections at mid-height of the panel and ultimate moment increased. This relationship is a function of increased curvature in the plane of the panel, as well as increased wind pressures. Increasing the height of the panel for a given thickness results in a more slender member, and therefore less stiff. Figure 4-4 illustrates the need to decrease spacing. For example, an 8-inch panel with two layers of reinforcement with a concrete compressive strength of 3000 psi and #4 bars requires spacing to be decreased as it increases in height because the moments and deflections at mid-height increase. The deflection of the panel at mid-height is inversely proportional to the stiffness of the panel. To remain within the deflection limit as height increases, the bending stiffness of the panel must increase. There are two means of increasing the stiffness; increase the thickness of the panel, or increase the area of steel in the section. Therefore, for a constant thickness, spacing of reinforcement must decrease as panel heights increase.

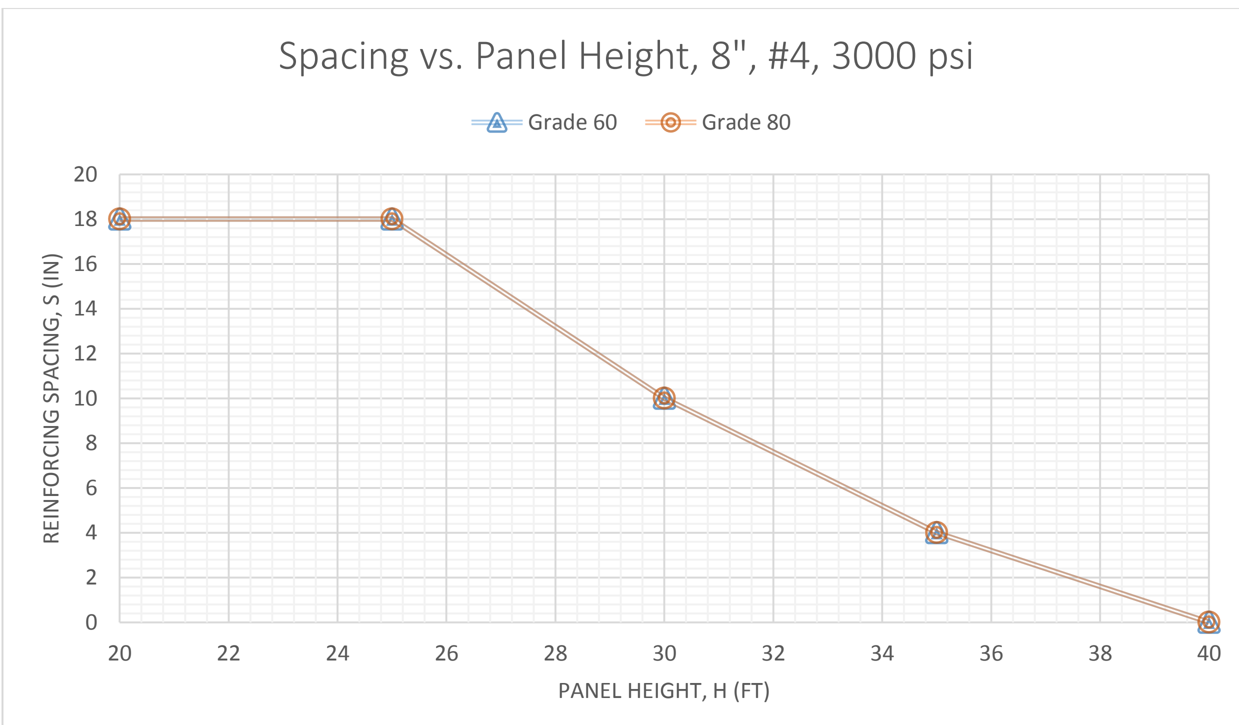

Figure 4-7, Spacing vs. Panel Height, Panels 20-40’.8”.4.#3.2

The need for a decrease in spacing to increase the stiffness of the panel is experienced equally by panels reinforced with Grade 60 and Grade 80 steel. In Figure 4-4 above, the data points depicted by a triangle represent Grade 60, while the circles represent Grade 80. For the given combination of nominal panel thickness, bar size, and concrete compressive strength, the series of data are identical This indicates that for the panel conditions illustrated in Figure 4-4, as height increases, the use of Grade 80 reinforcement provides no benefit relative to the use of Grade 60. Figure 4-5 shows the spacing of reinforcement relative to panel height for panels of heights ranging from 20 feet to 40 feet that have a nominal thickness of 8", use #4 bars, and have a 28 day concrete compressive strength of 4000 psi. This height, thickness, bar size and concrete strength resulted in the largest spacing differences for the two reinforcement grades.

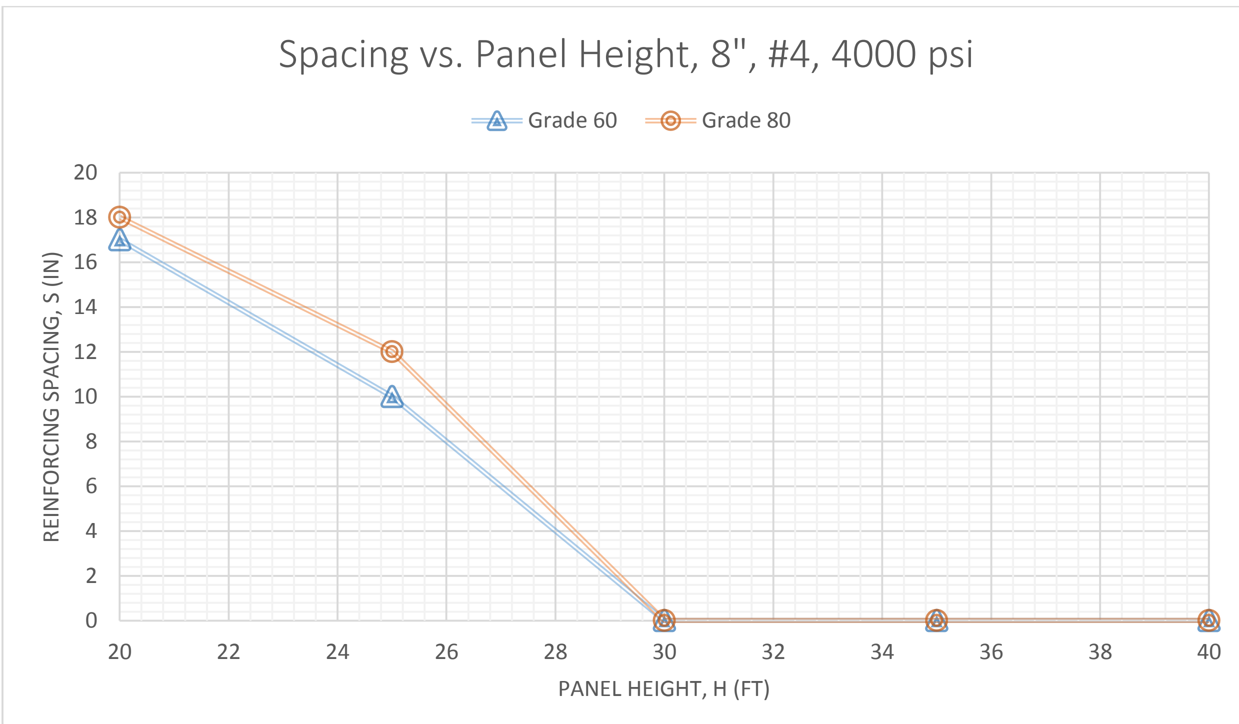

Figure 4-8, Spacing vs. Panel Height, Panels 20-40'.8".4.#4.2

In examining Figure 4-5, Grade 80 reinforcing steel provides an advantage over Grade 60 reinforcing steel for panel heights of 20 feet and 25 feet, but provides no advantage otherwise. Both reinforcement grades provide invalid designs for taller panels. Contrary to the trend in

Figure 4-5, some panel designs result in reduced spacing for Grade 80 relative to Grade 60, indicating an increase in total tonnage of steel. This condition is exhibited in Figure 4-6.

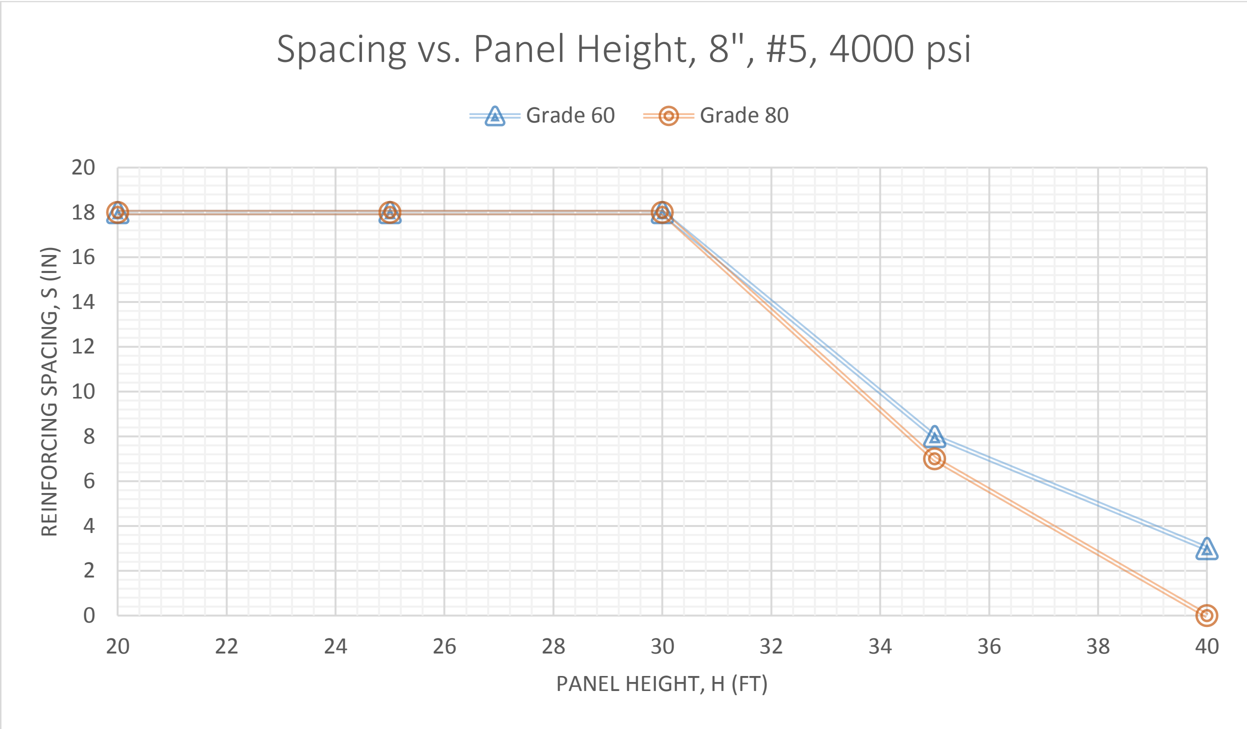

Figure 4-9, Spacing vs. Panel Height, Panels 20-40’.8”.4.#5.2

Figure 4-6 illustrates the effects of panel height on reinforcement spacing for a doubly reinforced, 8-inch panel that utilizes a #5 bar. When the height exceeds 30 feet, the optimal spacing of the reinforcement for the Grade 80 design falls below that of the Grade 60 design. This can be traced to Equation 2-5; the tensile yield stress is inversely related to the effective area of steel, and impacts the cracking moment of inertia of the slender wall section. With a reduced moment of inertia, the out-of-plane deflections are increased. This trend demonstrates a unique condition, where for the given panel parameters, this marginal reduction in the effective area of steel is enough to push the out-of-plane deflections from service loads beyond the allowable amount per the alternative analysis method. Therefore, an increased area of steel is required to increase the stiffness, and produce a valid design.

Panel thickness also had a role in the effectiveness of Grade 80 versus Grade 60 reinforcement. As the panels increased in thickness for a given panel height, they became less slender, resulting in smaller deflections and decreased need for reinforcement.

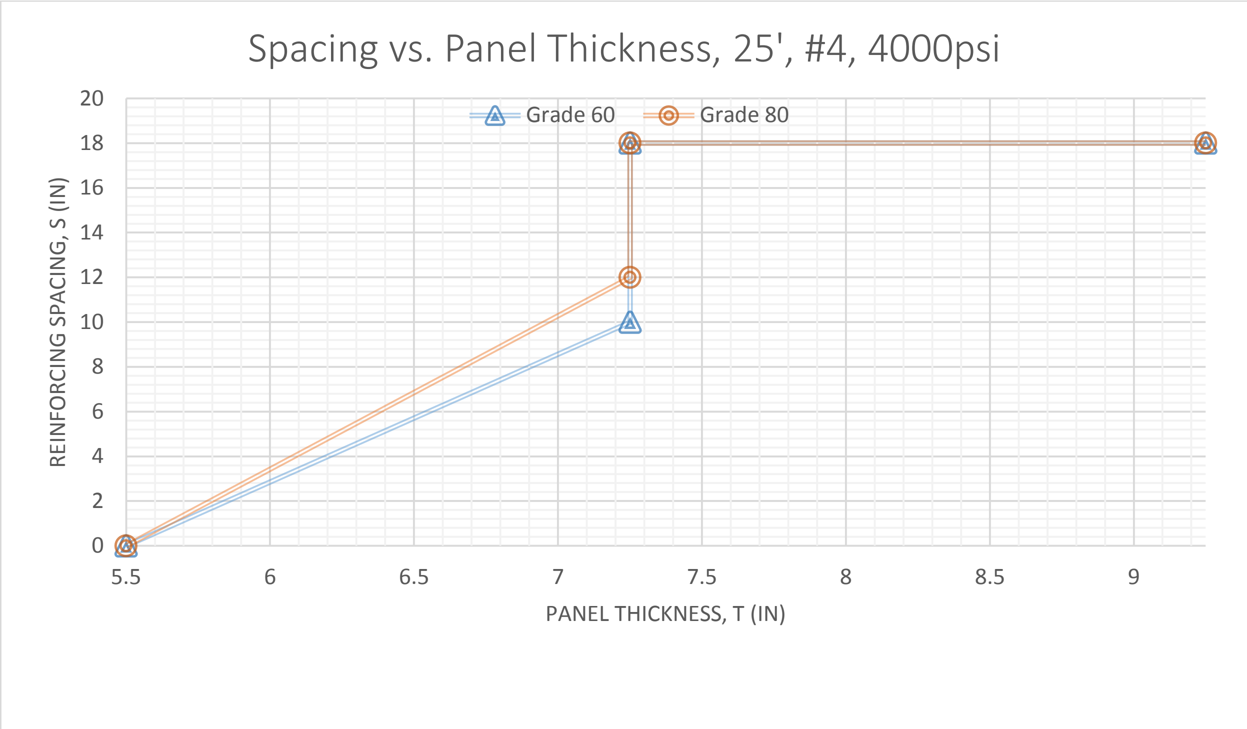

Figure 4-10, Spacing vs. Panel Thickness, Panels 25'.(6-10)".3.#5.(1-2)

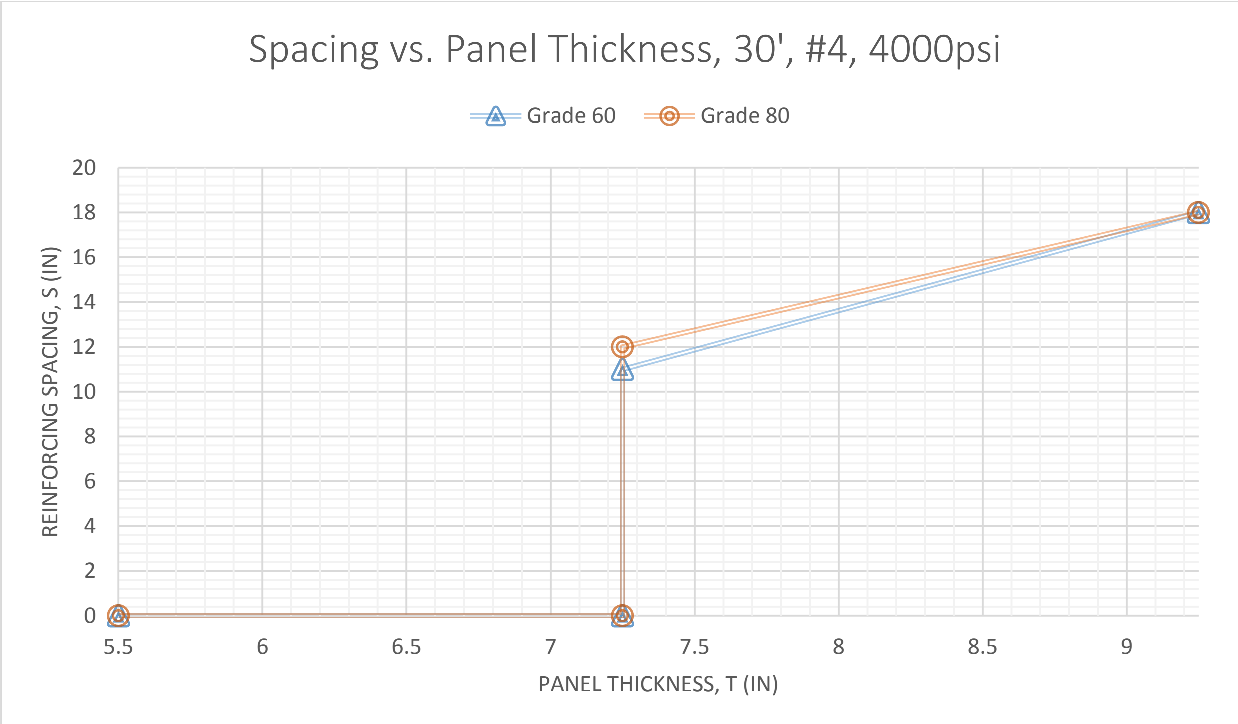

Figure 4-11, Spacing vs. Panel Thickness, Panels 30'.(6-10)".3.#5.(1-2)

Figures 4-7 and 4-8 display the trend observed for all panels that spacing increase with increased thickness. The graphs experience a jump at the 8-inch nominal thickness, as results were included for both singly and doubly reinforced 8-inch panels. The jump in spacing consistently occurs for panels of all heights, and indicates that the inclusion of two layers of steel rather than one serves to increase the stiffness of the panel section. This jump is also the location of the largest differentiation between designs for Grade 60 and Grade 80 reinforcement.

The changing of bar size and concrete compressive strength played a limited role in differentiating between Grade 60 and Grade 80 reinforcement. The effects of the panel height and thickness contributed more than any other parameter to observed spacing differences, and further investigation to isolate these parameters would be needed to observe any definitive effects. These parameters had the largest effect on the spacing of the reinforcement due to their direct effects on the stiffness of the panel section and the deflections and moment the panel is subject to.

Cost

A true cost analysis falls outside the scope of this study, but some observations are made based on the data available. As discussed in Chapter 3 of this paper, Grade 80 reinforcement carries a premium over Grade 60 due to lack of demand and inclusion of superior component materials. Cost projections for individual panel designs are based on rough per unit costs of the two reinforcing types, including the cost of placement and tying. For this study, Grade 60 carries a price of $2,025/ton for material and placement, and Grade 80 carries a price of $2,120/ton for material and placement (RSMeans, 2011). When applied to the total tonnage values that were attained through this study, panels utilizing Grade 80 reinforcement carried an average premium of $76.38 per panel. Table 4-3 illustrates the cost comparison for panels having a 10-inch nominal thickness and a concrete compressive strength of 4000 psi. The cost premium associated with the use of Grade 80 reinforcing steel rather than Grade 60 reinforcing steel is shown. Since the premium is always positive, it can be concluded that no cost savings in term of labor and material are observed. The design results for the panel comparisons are shown in Appendix B.

Table 4-3 Design results, 10” nominal thickness, 4000 psi

	DOUBLY REINFORCED PANELS BY HEIGHT FOR t_{NOM} = 10", f'_c = 4000 psi								
	ASTM A615 GRADE 60				ASTM A615 GRADE 80				
	DESIGNATION	TONNAGE (PER PANEL)	SPACING (in)	TOTAL COST ($/PANEL)	DESIGNATION	TONNAGE (PER PANEL)	SPACING (in)	TOTAL COST ($/PANEL)	COST PREMIUM (%)
#4 BAR	20'.10".4.60.#4.2	0.4134085	18	$ 1,979.13	20'.10".3.80.#4.2	0.4134085	18	$ 2,018.40	1.98%
	25'.10".4.60.#4.2	0.4997475	18	$ 2,439.46	25'.10".3.80.#4.2	0.4997475	18	$ 2,486.93	1.95%
	30'.10".4.60.#4.2	0.5860865	18	$ 2,899.79	30'.10".3.80.#4.2	0.5860865	18	$ 2,955.47	1.92%
	35'.10".4.60.#4.2	0.8277355	12	$ 3,674.62	35'.10".3.80.#4.2	0.8018505	13	$ 3,698.38	0.65%
	40'.10".4.60.#4.2	1.5186145	6	$ 5,359.14	40'.10".3.80.#4.2	1.5186145	6	$ 5,503.41	2.69%
#5 BAR	20'.10".4.60.#5.2	0.538096	18	$ 2,231.62	20'.10".3.80.#5.2	0.538096	18	$ 2,282.74	2.29%
	25'.10".4.60.#5.2	0.650685	18	$ 2,745.11	25'.10".3.80.#5.2	0.650685	18	$ 2,806.92	2.25%
	30'.10".4.60.#5.2	0.763274	18	$ 3,258.59	30'.10".3.80.#5.2	0.763274	18	$ 3,331.10	2.23%
	35'.10".4.60.#5.2	0.875863	18	$ 3,772.08	35'.10".3.80.#5.2	0.875863	18	$ 3,855.29	2.21%
	40'.10".4.60.#5.2	1.4447645	10	$ 5,209.60	40'.10".3.80.#5.2	1.4447645	10	$ 5,346.85	2.63%
#6 BAR	20'.10".4.60.#6.2	0.6907135	18	$ 2,540.67	20'.10".3.80.#6.2	0.6907135	18	$ 2,606.29	2.58%
	25'.10".4.60.#6.2	0.8354325	18	$ 3,119.22	25'.10".3.80.#6.2	0.8354325	18	$ 3,198.59	2.54%
	30'.10".4.60.#5.2	0.763274	18	$ 3,258.59	30'.10".3.80.#5.2	0.763274	18	$ 3,331.10	2.23%
	35'.10".4.60.#5.2	0.875863	18	$ 3,772.08	35'.10".3.80.#5.2	0.875863	18	$ 3,855.29	2.21%
	40'.10".4.60.#6.2	1.5324395	14	$ 5,387.14	40'.10".3.80.#6.2	1.5324395	14	$ 5,532.72	2.70%

Despite the savings associated with increased spacing of reinforcement in some of the panel designs, the overwhelming majority of panels did not experience any tonnage savings between standard reinforcing steel and high-strength reinforcing steel designs. This lack of savings, in conjunction with the premium of Grade 80 steel, results in higher cost per panel, which could significantly increase construction costs when applied over an entire project.

Chapter 5 - Conclusion

Based on the results found within this study, it can be concluded that for a majority of panel designs, the substitution of ASTM A615 Grade 80 reinforcement for ASTM A615 Grade 60 reinforcement does not provide substantial reduction in steel reinforcing tonnage. Of the 120 unique panels that were designed using both standard reinforcement and high-strength reinforcement, 72 yielded designs that met the limitations of the applicable body of code, *ACI 318-14*. Of these, 80% demonstrated no reduction in volume of steel reinforcement or an increase in the volume of necessary steel reinforcement. These panels considered a variety of commonly varied parameters, including panel height, thickness, concrete compressive strength and bar size. The strongest relationships between any given parameter and the resulting volume of steel were found to be panel height and panel thickness. For a given set of conditions, full design with both standard and high-strength reinforcement should be conducted in order to verify which provides the better solution in terms of strength, serviceability, and economy.

Recommendations

Further research is needed to better understand the mechanics of high-strength reinforcement utilization within slender tilt-up panels. Within this study, the height and width of panel were the parameters with the largest range of values. With the information of this study, it is concluded that the benefits associated with ASTM A615 Grade 80 reinforcing steel do not justify its use for panels with heights ranging from 20 feet to 40 feet, and panel thicknesses of 5.5 inches, 7.25 inches and 9.25 inches. Additional studies on the effects of combining high-strength reinforcement with high-strength concrete may lead to better utilization of the two materials. Additionally, a full cost/schedule analysis that details all costs associated with the use of high-strength reinforcement, including constructability and availability, and the impacts of its use on the construction schedule, would provide information that could be used to more accurately select panel designs that demonstrate the highest level of economy. Lastly, further evaluation of the equations in *ACI 318-14 Section 11.8* should be further evaluated to verify their validity. After investigation of the origins of Equation 11.8.3.1b revealed that the reduction to bending stiffness found in the denominator is derivative of practices to approximate the deflection of tied columns. The panels that were part of this study were untied, yet still utilized the same stiffness reduction factor. Therefore, further evaluation is necessary to identify appropriate reductions of the stiffness to best estimate the ultimate deflections for slender walls. Continued research in the topic of high-strength reinforcement usage in slender tilt-up panels is necessary to ensure improved economy and quality of design for future tilt-up construction projects.

Chapter 6 - References

ACI Committee 318. (2014). *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary on Building Code Requirements for Structural Concrete (ACI 318R-14).* Farmington Hills, Michigan: American Concrete Institute.

ACI Committee 551. (2015). *Design Guide for Tilt-Up Concrete Panels.* Farmington Hills, Michigan: American Concrete Institute.

Concrete Reinforcing Steel Institute. (2016). *Engineering Technical Note: High-Strength Reinforcing Bars.* Schaumburg, IL: Concrete Reinforcing Steel Institute.

Lawson, J. (2007, September). Deflection Limits for Tilt-Up Wall Serviceability - The history behind changes to Chapter 14 of ACI 318. *Concrete International*, pp. 33-38.

RSMeans. (2011). *RSMeans Building Construction Cost Data.* Norwell, Massachusetts: Construction Publishers & Consultants.

Schwinger, C. W. (2011, August). ASTM A615 grade 75 reinforcing steel: When, why and how to use it. *STRUCTURE Magazine*, pp. 34-35.

Tilt-Up Concrete Association. (2011). *The Construction of Tilt-Up.* Mount Vernon, Iowa: Tilt-Up Concrete Association.

Tilt-Up Concrete Association. (2016). *Tilt-Up Market Report.* Mt. Vernon, IA: Tilt-Up Concrete Association.

Timothy W. Mays, P. P., & Steinbicker, P.E., S.E., J. J. (2013). *Engineering Tilt-Up.* Mount Vernon, Iowa: Tilt-Up Concrete Association.

Vulcraft. (2008). *Vulcraft Steel Roof and Floor Deck.* Alpharetta, GA: Nucor.

Wiss, Janney, Elstner Associates, Inc. (2008). *Mechanical properties of ASTM A1035 high strength steel bar reinforcement.* Chicago, Illinois.

Appendix A - Panel Reinforcement Design Example

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CALCULATIONS	REFERENCES
Panel Designation: 20'.6".3.60.#4.1	ASCE 7-10

Panel Parameters

						References
h =	20	ft	f_{yt} =	60,000	psi	
t =	5.5	in	Bar Size:	#4		
f'_c =	3000	psi	d_b =	0.5	in	ACI 318-14
$h_{Parapet}$ =	4	ft	Wt =	0.668	plf	APPENDIX A
w =	20	ft	A_b =	0.2	in^2	ACI 551.2R-15
h_{Chair} =	2.5	in	Layers=	1	layers	TAB. 10.1b

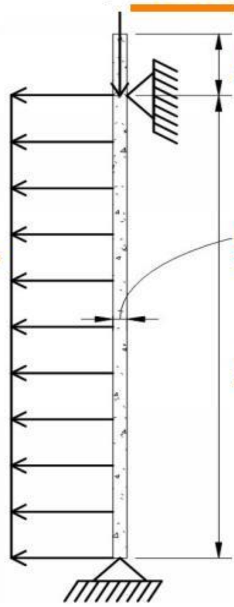
P_U = 0.89235 klf

hp = 4 ft

$w_{W,Horiz}$ = 0.038

t = 5.5 in

h = 20 ft



Solution

						References
A_s =	0.3	in^2	Tonnage =	0.357	tons	
A_s' =	0	in^2				ACI 318-14
s =	8	≤	s_{max} =	16.5		SECTION 11.7.2.1
		GOOD				

APPROXIMATE MULTIPLE PONT LOADS FROM JOIST BEARING AS UNIFORM LOAD

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CALCULATIONS	REFERENCES
	ASCE 7-10

Panel Loads

Wind Loading

UTILIZE COMPONENTS AND CLADDING, ASSUME PANEL IS LOCATED AT INTERIOR OF BUILDING PERIMETER

Part 1:

$h =$ 20 < 60 GOOD — SECTION 30.4

LOW RISE BUILDING: YES GOOD

				Reference
$V =$	115	mph	$p = q_h[(GC_p)-(G_{cpi})]$	FIG. 26.5-1A
$K_d =$	0.85		$q_h = 0.00256K_zK_{zt}K_dV^2$	TAB. 26.6-1
$K_{zt} =$	1.0			SECTION 26.8
$K_{zh} =$	0.9	Exposure C		SECTION 26.7.3
$GC_{pi} =$	0.55	Partially Enclosed		TAB. 26.11-1
$q_h =$	25.90	psf		EQ 30.3-1
				EQ 30.4-1

ZONE	AREA	GCp(-)	GCp(+)	p (-) psf	p (+) psf
4	400	-0.92	0.76	-38.14	33.96

FIG. 30.4-1

Snow Loading

			Reference
Assume $p_g =$	20	psf	FIG. 7-1
$C_e =$	0.9	Exposure C, Assume fully exposed	TAB. 7-2
$C_t =$	1.2	Roof insulated, no benefit from space below	TAB. 7-3
$I_s =$	1.0	Risk Category II	TAB. 1.5-2

$p_f = 0.7C_eC_tI_sp_g =$ 15.12 psf — EQ 7.3-1

Rain on Snow: 5psf Surcharge

$p_g =$ 20 ≤ 20 APPLIES — SECTION 7.10

$p_s =$ 20.12 psf

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CALCULATIONS	REFERENCES
	ASCE 7-10
Minimum Snow	
p_{min} = Ispg = 20 psf	SECTION 7.3.4
p_f > p_{min} USE FLAT ROOF SNOW	
p_s = 20.12 psf	
Drifting	SECTION 7.8
l_u = 160 ft	FIG. 7-8
$\Upsilon = 0.13p_g + 14 =$ 16.6 pcf	EQ 7.7-1
$h_b = \frac{p_f}{\gamma} =$ 0.91 ft	
$h_c = h_{Parapet} - h_b =$ 3.09 ft	
$h_d = 0.75\left(0.43\sqrt[3]{l_u}\sqrt[4]{p_g+10} - 1.5\right) =$ 2.97 ft	FIG. 7-9
h_d < h_c	
$w = 4h_d =$ 11.89 ft	
$p_d = h_d\Upsilon + p_s =$ 69.46 psf	
CONSERVATIVELY, USE AVERAGE OF p_d AND p_s OVER WHOLE AREA	
p_s = 44.79 psf	

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CALCULATIONS	REFERENCES
	ASCE 7-10

Dead Load

ROOF: BY PANEL

			MULT.	psf	
METAL DECK =	2	psf	1	2	TAB C3-1
INSULATION	1.5	psf/in	4	6	
MEP =	5	psf	1	5	
JOIST =	12	plf	0.17	2	
			TOTAL =	15 psf	

Live Load

Construction Live Load: 20 psf — ASCE 37-02

Load Combinations

CRITICAL SECTION OCCURS AT MIDPOINT OF PANEL

$w_{DL} = (W_{Panel} \times Weight_{Roof}) + (Weight_{Concrete} \times t \times h_{Above\ Midpoint})$

w_{DL} =	0.992	Klf	INCLUDES SELF WEIGHT OF WALL AND PARAPET
w_{LL} =	0.000	Klf	NO LIVE LOAD ACTING ON WALL
w_S =	0.896	Klf	SNOW IS AVERAGE OF DRIFT AND FLAT ROOF
w_{Lr} =	0.400	Klf	20 psf CONSTRUCTION LIVE LOAD
w_E =	0.000	Klf	ASSUME WIND GOVERNS OVER SEISMIC
$w_{W,Vert}$ =	0.000	Klf	NO WIND, ONLY ACCOUNT FOR OUT OF PLANE
$w_{W,Horiz}$ =	0.038	Klf	COMPONENTS AND CLADDING LOAD

LOAD COMBINATIONS

LC 3, LC 4 AND LC 6 WILL BE TESTED TO DETERMINE CONTROLLING CASE. WORST CASE IS HIGHLIGHTED.

Applicable Load Combinations — SECTION 2.3.2

Combination	P_U =	$w_{U,Horiz}$ =
1. 1.4D	1.3881	0.0000
2. 1.2D + 1.6L + 0.5(Lr, S, R)	1.6377	0.0000
3. 1.2D + 1.6(Lr, S, R) + (L, 0.5W)	2.6231	0.0191
4. 1.2D + 1.0W + L + 0.5(Lr, S, R)	1.6377	0.0381
5. 1.2D + 1.0E + L + 0.2S	1.3690	0.0000
6. 0.9D + 1.0W	0.8924	0.0381
7. 0.9D + 1.0E	0.8924	0.0000

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CALCULATIONS	REFERENCES
	ACI 318-14

FIND SOLUTIONS FOR LC 3, LC 4 AND LC 5 TO DETERMINE LARGEST REQUIRED VOLUME OF REINFORCEMENT

LC 3

AISC STEEL CONSTRUCTION MANUAL

$M_U = \frac{w_{WU} l^2}{8} =$ **0.953** k-ft — TAB. 3-23

$d = t - d_{chair} - .5d_b =$ **2.75** in — ACI 551.2R-15 FIG. Bb

ACI 318-14 ALTERNATIVE SLENDER WALL ANALYSIS

$\frac{P_{UM}}{A_g} =$ **0.0397** ksi < $0.06f'_c =$ **0.18** ksi — GOOD — SECTION 11.8.1d

2.6231

DESIGN MOMENT STRENGTH

$A_{se,w} = A_s + \frac{P_{UM}}{f_y}\left(\frac{t}{2d}\right) =$ **0.344** in^2 — SECTION R11.8.3

$a = \frac{A_{se} f_y}{0.85 f'_c b} =$ **0.674** in; $c = \frac{a}{\beta_1} =$ **0.793** in — EQ 22.2.2.4.1

$\varepsilon_{ty} = 0.003\left(\frac{d-c}{c}\right) =$ **0.0074** > 0.005 — TENSION CONTROLLED — TAB. 21.2.2

$f_r = 7.5\lambda\sqrt{f'_c} =$ **0.411** ksi; $I_g = \frac{bt^3}{12} =$ **166.38** in^4 — EQ 19.2.3.1

$M_{cr} = \frac{f_r I_g}{\left(\frac{t}{2}\right)} =$ **2.07** k-ft — EQ 24.2.3.5b

$\phi M_n = \phi A_{se,w} f_y \left(d - \frac{a}{2}\right) =$ **3.73** k-ft

CHECK MAX MOMENT, M_{UA}

$E_c = 57\sqrt{f'_c} =$ **3122** ksi — EQ 19.2.2.1.b

$I_{cr} = \frac{E_s}{E_c}\left(A_s + \frac{P_U}{f_y}\left(\frac{t}{2d}\right)\right)(d-c)^2 + \frac{bc^3}{3} =$ 14.22 in^4 — EQ 11.8.3.1c

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CALCULATIONS	REFERENCES
	ACI 318-14
e_{cc} = 0 in	
$M_{UA}=\frac{w_U l_c^2}{8}+\frac{P_{UA}e_{cc}}{2}=$ 0.953 k-ft	
$M_U=\frac{M_{UA}}{1-\frac{5P_{UM}l_c^2}{(0.75)(48)E_cI_{cr}}}=$ 1.808 k-ft	EQ 11.8.3.1d
M_U = 1.808 < ϕM_n = 3.73 GOOD	
$\Delta_U=\frac{5M_Ul_c^2}{(0.75)(48)E_cI_{cr}}=$ 3.908 in	EQ 11.8.3.1b
SERVICE LOAD DEFLECTIONS	
LOAD COMBINATION: D + 0.5L + W_A	
$P_{SM}=P_A+\left(\frac{h}{2}+h_{Parapet}\right)t\rho_{Conc}=$ 1.263 klf	
$A_{se,w}=A_s+\frac{P_{SM}}{f_y}\left(\frac{t}{2d}\right)=$ 0.321 in^2	SECTION R11.8.3
$a=\frac{A_{se}f_y}{0.85f'_cb}=$ 0.629 in; $c=\frac{a}{\beta_1}=$ 0.741 in	EQ 22.2.2.4.1
$\varepsilon_{ty}=0.003\left(\frac{d-c}{c}\right)=$ 0.00814 > 0.005 TENSION CONTROLLED	TAB. 21.2.2
$f_r=7.5\lambda\sqrt{f'_c}=$ 0.411 ksi; $I_g=\frac{bt^3}{12}=$ 166.38 in^4	EQ 19.2.3.1
$M_{cr}=\frac{f_rI_g}{\left(\frac{t}{2}\right)}=$ 2.07 k-ft	EQ 24.2.3.5b
$\phi M_n=\phi A_{se,w}f_y\left(d-\frac{a}{2}\right)=$ 3.52 k-ft	
$I_{cr}=\frac{E_s}{E_c}\left(A_s+\frac{P_U}{f_y}\left(\frac{t}{2d}\right)\right)(d-c)^3+\frac{bc^2}{3}=$ 13.67 in^4	EQ 11.8.3.1c

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CALCULATIONS	REFERENCES
	ACI 318-14
DEFLECTION CHECK	
$\Delta_{SMAX}=\frac{l_c}{150}=$ 1.600 in	SECTION 11.8.1.1e
$\Delta_{cr}=\frac{5M_{cr}l_c^2}{48E_cI_{cr}}=$ 0.287 in	EQ 11.8.4.3a
$\Delta_n=\frac{5M_nl_c^2}{48E_cI_{cr}}=$ 6.597 in	EQ 11.8.4.3b
$M_{SA}=\frac{w_Sl_c^2}{8}+\frac{P_{SA}e_{cc}}{2}=$ 0.953 k-ft	
ASSUME: $M_A\le\frac{2M_{cr}}{3}\therefore\ \Delta_s=\frac{M_A\Delta_{cr}}{M_{cr}}$	SECTION R11.8.4.1
Iteration 1:	
$\Delta_s=\frac{M_A\Delta_{cr}}{M_{cr}}=$ 0.1322 in	TAB. 11.8.4.1a
$M_A=M_{SA}+P_{SM}\Delta_s=$ 1.120 k-ft	EQ 11.8.4.2
Iteration 2:	
$\Delta_s=\frac{M_A\Delta_{cr}}{M_{cr}}=$ 0.1553 in	TAB. 11.8.4.1a
$M_A=M_{SA}+P_{SM}\Delta_s=$ 0.970 k-ft	EQ 11.8.4.2
Iteration 3:	
$\Delta_s=\frac{M_A\Delta_{cr}}{M_{cr}}=$ 0.1344 in	TAB. 11.8.4.1a
$M_A=M_{SA}+P_{SM}\Delta_s=$ 0.968 k-ft	EQ 11.8.4.2
Iteration 4:	
$\Delta_s=\frac{M_A\Delta_{cr}}{M_{cr}}=$ 0.1341 in	TAB. 11.8.4.1a
$M_A=M_{SA}+P_{SM}\Delta_s=$ 0.967 k-ft	EQ 11.8.4.2

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CALCULATIONS	REFERENCES
	ACI 318-14
Iteration 5:	
$\Delta_s = \frac{M_A \Delta_{cr}}{M_{cr}} =$ 0.1341 in	TAB. 11.8.4.1a
$M_A = M_{SA} + P_{SM}\Delta_s =$ 0.967 k-ft	EQ 11.8.4.2
COVERGES TO: $\Delta_S =$ 0.1341 in; $M_A =$ 0.967 k-ft	
$M_A =$ 0.967 < $\frac{2M_{cr}}{3} =$ 1.38071728	TAB. 11.8.4.1
ASSUMPTION CORRECT	
IF INITIAL ASSUMPTION IS INCORRECT:	
ASSUME: $M_a \geq \frac{2M_{cr}}{3} \therefore \Delta_s = \frac{2\Delta_{cr}}{3} + \frac{\left(M_a - \frac{2\Delta_{cr}}{3}\right)}{\left(M_n - \frac{2\Delta_{cr}}{3}\right)}\left(\Delta_n - \frac{2\Delta_{cr}}{3}\right)$	SECTION R11.8.4.1
Iteration 1:	
$\Delta_s = \frac{2\Delta_{cr}}{3} + \frac{\left(M_a - \frac{2\Delta_{cr}}{3}\right)}{\left(M_n - \frac{2\Delta_{cr}}{3}\right)}\left(\Delta_n - \frac{2\Delta_{cr}}{3}\right) =$ 0.000 in	TAB. 11.8.4.1b
$M_A = M_{SA} + P_{SM}\Delta_s =$ 0.000 k-ft	EQ 11.8.4.2
$M_A =$ 0.000 > $\frac{2M_{cr}}{3} =$ 0.000	TAB. 11.8.4.1
NOT APPLICABLE	
$\Delta_S =$ 0.1341 < $\Delta_{SMAX} =$ 1.600	SECTION 11.8.1.1e
GOOD	
CHECK MINIMUM STEEL	
$\rho = \frac{A_s}{bt} =$ 0.00455 > $\rho_{l,MIN} =$ 0.0012	TAB 11.6.1
GOOD	

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CALCULATIONS	REFERENCES
	ACI 318-14
TRANSVERSE REINFORCEMENT	
$\rho_{t,MIN}$ = 0.002 $A_{S,MIN} = \rho_{t,MIN}bt$ = 2.64 in^2	TAB 11.6.1
Use: (14)#4 AT 16 IN O.C.	SECTION 11.7.3.1
STEEL TONNAGE	
#4 @ 8 IN O.C. VERTICAL = 30 Sticks @ 23.75 ft	
#4 @ 16.5 IN O.C. HORIZONTAL = 18 Sticks @ 19.75 ft	
$Tonnage_{Long}$ = 0.238 tons	
$Tonnage_{Trans}$ = 0.119 tons	
$Tonnage_{Total}$ = 0.357 tons	
LC 4	AISC STEEL CONSTRUCTION MANUAL
$M_U=\frac{w_{WU}l^2}{8}$ = 1.907 k-ft	TAB. 3-23
d = t - dchair - .5db = 2.75 in	ACI 551.2R-15 FIG. Bb
ACI 318-14 ALTERNATIVE SLENDER WALL ANALYSIS	
$\frac{P_{UM}}{A_g}$ = 0.0248 ksi < $0.06f'_c$ = 0.18 ksi GOOD	SECTION 11.8.1d
DESIGN MOMENT STRENGTH	
$A_{se,w}=A_s+\frac{P_{UM}}{f_y}\left(\frac{t}{2d}\right)$ = 0.327 in^2	SECTION R11.8.3
$a=\frac{A_{se}f_y}{0.85f'_cb}$ = 0.642 in $c=\frac{a}{\beta_1}$ = 0.755 in	
$\varepsilon_{ty}=0.003\left(\frac{d-c}{c}\right)$ = 0.00793 > 0.005 TENSION CONTROLLED	TAB. 21.2.2
$f_r=7.5\lambda\sqrt{f'_c}$ = 0.411 ksi $I_g=\frac{bt^3}{12}$ = 166.38 in^4	EQ 19.2.3.1

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CALCULATIONS	REFERENCES
	ACI 318-14
$M_{cr}=\frac{f_r I_g}{\left(\frac{t}{2}\right)}=$ **2.07** k-ft	EQ 24.2.3.5b
$\phi M_n=\phi A_{se,w} f_y\left(d-\frac{a}{2}\right)=$ **3.58** k-ft	
CHECK MAX MOMENT, M_{UA}	
$E_c=57\sqrt{f_c'}=$ **3122** ksi	EQ 19.2.2.1.b
$I_{cr}=\frac{E_s}{E_c}\left(A_s+\frac{P_U}{f_y}\left(\frac{h}{2d}\right)\right)(d-c)^2+\frac{bc^3}{3}=$ 13.82 in^4	EQ 11.8.3.1c
$e_{cc}=$ 0 in	
$M_{UA}=\frac{w_U l_c^2}{8}+\frac{P_{UA}e_{cc}}{2}=$ **1.907** k-ft	
$M_U=\frac{M_{UA}}{1-\frac{5P_{UM}l_c^2}{(0.75)(48)E_c I_{cr}}}=$ **1.907** k-ft	EQ 11.8.3.1d
$M_U=$ 1.907 < $\phi M_n=$ 3.58 **GOOD**	
$\Delta_U=\frac{5M_U l_c^2}{(0.75)(48)E_c I_{cr}}=$ **4.242** in	EQ 11.8.3.1b
SERVICE LOAD DEFLECTIONS	
LOAD COMBINATION: D + 0.5L + W_A	
$P_{SM}=P_A+\left(\frac{h}{2}+h_{Parapet}\right)t\rho_{Conc}=$ **1.263** klf	
$A_{se,w}=A_s+\frac{P_{SM}}{f_y}\left(\frac{h}{2d}\right)=$ **0.321** in^2	SECTION R11.8.3
$a=\frac{A_{se}f_y}{0.85f_c'b}=$ **0.629** in; $c=\frac{a}{\beta_1}=$ **0.741** in	
$\varepsilon_{ty}=0.003\left(\frac{d-c}{c}\right)=$ **0.00814** > 0.005 **TENSION CONTROLLED**	TAB. 21.2.2
$f_r=7.5\lambda\sqrt{f_c'}=$ **0.411** ksi; $I_g=\frac{bh^3}{12}=$ **166.38** in^4	EQ 19.2.3.1

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CALCULATIONS	REFERENCES
$M_{cr}=\frac{f_r I_g}{\left(\frac{t}{2}\right)}=$ 2.07 k-ft	ACI 318-14 EQ 24.2.3.5b
$\phi\, M_n=\phi A_{se,w} f_y \left(d-\frac{a}{2}\right)=$ 3.52 k-ft	
$I_{cr}=\frac{E_s}{E_c}\left(A_s+\frac{P_U}{f_y}\left(\frac{h}{2d}\right)\right)(d-c)^2+\frac{bc^3}{3}=$ 13.67 in^4	EQ 11.8.3.1c
DEFLECTION CHECK	
$\Delta_{SMAX}=\frac{l_c}{150}=$ 1.600 in	SECTION 11.8.1.1e
$\Delta_{cr}=\frac{5M_{cr}l_c^2}{48E_cI_g}=$ 0.287 in	EQ 11.8.4.3a
$\Delta_n=\frac{5M_nl_c^2}{48E_cI_{cr}}=$ 6.597 in	EQ 11.8.4.3b
$M_{SA}=\frac{w_Sl_c^2}{8}+\frac{P_{SA}e_{cc}}{2}=$ 1.907 k-ft	
ASSUME: $M_A\le\frac{2M_{cr}}{3}\therefore\ \Delta_s=\frac{M_A\Delta_{cr}}{M_{cr}}$	SECTION R11.8.4.1
Iteration 1:	
$\Delta_s=\frac{M_A\Delta_{cr}}{M_{cr}}=$ 0.2643 in	TAB. 11.8.4.1a
$M_A=M_{SA}+P_{SM}\Delta_s=$ 2.240 k-ft	EQ 11.8.4.2
Iteration 2:	
$\Delta_s=\frac{M_A\Delta_{cr}}{M_{cr}}=$ 0.3106 in	TAB. 11.8.4.1a
$M_A=M_{SA}+P_{SM}\Delta_s=$ 1.939 k-ft	EQ 11.8.4.2
Iteration 3:	
$\Delta_s=\frac{M_A\Delta_{cr}}{M_{cr}}=$ 0.2688 in	TAB. 11.8.4.1a
$M_A=M_{SA}+P_{SM}\Delta_s=$ 1.935 k-ft	EQ 11.8.4.2

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CALCULATIONS	REFERENCES
	ACI 318-14
Iteration 4:	
$\Delta_s = \frac{M_A \Delta_{cr}}{M_{cr}} =$ 0.2682 in	TAB. 11.8.4.1a
$M_A = M_{SA} + P_{SM}\Delta_s =$ 1.935 k-ft	EQ 11.8.4.2
Iteration 5:	
$\Delta_s = \frac{M_A \Delta_{cr}}{M_{cr}} =$ 0.2682 in	TAB. 11.8.4.1a
$M_A = M_{SA} + P_{SM}\Delta_s =$ 1.935 k-ft	EQ 11.8.4.2
COVERGES TO: $\Delta_S =$ 0.2682 in	
$M_A =$ 1.935 k-ft	
$M_A =$ 1.935 > $\frac{2M_{cr}}{3} =$ 1.38071728	TAB. 11.8.4.1
ASSUMPTION INCORRECT	
IF INITIAL ASSUMPTION IS INCORRECT:	
ASSUME: $M_a \geq \frac{2M_{cr}}{3} \therefore \Delta_s = \frac{2\Delta_{cr}}{3} + \frac{\left(M_a - \frac{2\Delta_{cr}}{3}\right)}{\left(M_n - \frac{2\Delta_{cr}}{3}\right)}\left(\Delta_n - \frac{2\Delta_{cr}}{3}\right)$	SECTION R11.8.4.1
Iteration 1:	
$\Delta_s = \frac{2\Delta_{cr}}{3} + \frac{\left(M_a - \frac{2\Delta_{cr}}{3}\right)}{\left(M_n - \frac{2\Delta_{cr}}{3}\right)}\left(\Delta_n - \frac{2\Delta_{cr}}{3}\right) =$ 1.490 in	TAB. 11.8.4.1b
$M_A = M_{SA} + P_{SM}\Delta_s =$ 2.064 k-ft	EQ 11.8.4.2
$M_A =$ 2.064 > $\frac{2M_{cr}}{3} =$ 1.381	TAB. 11.8.4.1
ASSUMPTION CORRECT	
$\Delta_S =$ 1.4901 < $\Delta_{SMAX} =$ 1.600	SECTION 11.8.1.1e
GOOD	

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CALCULATIONS	REFERENCES
	ACI 318-14
CHECK MINIMUM STEEL	
$\rho = \frac{A_s}{bh} =$ 0.00455 > $\rho_{l,MIN} =$ 0.0012	TAB 11.6.1
GOOD	
TRANSVERSE REINFORCEMENT	
$\rho_{t,MIN} =$ 0.002 $A_{S,MIN} = \rho_{t,MIN}bh =$ 2.64 in^2	TAB 11.6.1
Use: (14)#4 AT 16 IN O.C.	SECTION 11.7.3.1
STEEL TONNAGE	
#4 @ 8 IN O.C. VERTICAL = 30 Sticks @ 23.75 ft	
#4 @ 16.5 IN O.C. HORIZONTAL = 18 Sticks @ 19.75 ft	
$Tonnage_{Long} =$ 0.238 tons	
$Tonnage_{Trans} =$ 0.119 tons	
$Tonnage_{Total} =$ 0.357 tons	
LC 6	AISC STEEL CONSTRUCTION MANUAL
$M_U = \frac{w_{WU} l^2}{8} =$ 1.907 k-ft	TAB. 3-23
d = t - dchair - .5db = 2.75 in	ACI 551.2R-15 FIG. Bb
ACI 318-14 ALTERNATIVE SLENDER WALL ANALYSIS	
$\frac{P_{UM}}{A_g} =$ 0.0135 ksi < $0.06f'_c =$ 0.18 ksi	SECTION 11.8.1d
GOOD	
DESIGN MOMENT STRENGTH	
$A_{se,w} = A_s + \frac{P_{UM}}{f_y}\left(\frac{t}{2d}\right) =$ 0.315 in^2	SECTION R11.8.3

CALCULATIONS	REFERENCES
	ACI 318-14
$a = \frac{A_{se} f_y}{0.85 f'_c b} =$ **0.617** in $c = \frac{a}{\beta_1} =$ **0.726** in	
$\varepsilon_{ty} = 0.003\left(\frac{d-c}{c}\right) =$ **0.00836** > 0.005 **TENSION CONTROLLED**	TAB. 21.2.2
$f_r = 7.5\lambda\sqrt{f'_c} =$ **0.411** ksi $I_g = \frac{bt^3}{12} =$ **166.38** in^4	EQ 19.2.3.1
$M_{cr} = \frac{f_r I_g}{\left(\frac{t}{2}\right)} =$ **2.07** k-ft	EQ 24.2.3.5b
$\phi M_n = \phi A_{se,w} f_y \left(d - \frac{a}{2}\right) =$ **3.46** k-ft	
CHECK MAX MOMENT, M_{UA}	
$E_c = 57\sqrt{f'_c} =$ **3122** ksi	EQ 19.2.2.1.b
$I_{cr} = \frac{E_s}{E_c}\left(A_s + \frac{P_U}{f_y}\left(\frac{h}{2d}\right)\right)(d-c)^2 + \frac{bc^3}{3} =$ 13.51 in^4	EQ 11.8.3.1c
$e_{cc} =$ 0 in	
$M_{UA} = \frac{w_U l_c^2}{8} + \frac{P_{UA} e_{cc}}{2} =$ **1.907** k-ft	
$M_U = \frac{M_{UA}}{1 - \frac{5 P_{UM} l_c^2}{(0.75)(48) E_c I_{cr}}} =$ **2.295** k-ft	EQ 11.8.3.1d
$M_U =$ 2.295 < $\phi M_n =$ 3.46 **GOOD**	
$\Delta_U = \frac{5 M_U l_c^2}{(0.75)(48) E_c I_{cr}} =$ **5.224** in	EQ 11.8.3.1b
SERVICE LOAD DEFLECTIONS	
LOAD COMBINATION: D + 0.5L + W_A	
$P_{SM} = P_A + \left(\frac{h}{2} + h_{Parapet}\right) t \rho_{Conc} =$ **1.263** klf	

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CALCULATIONS	REFERENCES
$A_{se,w}=A_s+\frac{P_{SM}}{f_y}\left(\frac{h}{2d}\right)=$ 0.321 in^2	ACI 318-14 SECTION R11.8.3
$a=\frac{A_{se}f_y}{0.85f'_c b}=$ 0.629 in $c=\frac{a}{\beta_1}=$ 0.741 in	
$\varepsilon_{ty}=0.003\left(\frac{d-c}{c}\right)=$ 0.00814 > 0.005 TENSION CONTROLLED	TAB. 21.2.2
$f_r=7.5\lambda\sqrt{f'_c}=$ 0.411 ksi $I_g=\frac{bh^3}{12}=$ 166.38 in^4	EQ 19.2.3.1
$M_{cr}=\frac{f_r I_g}{\left(\frac{h}{2}\right)}=$ 2.07 k-ft	EQ 24.2.3.5b
$\phi M_n=\phi A_{se,w}f_y\left(d-\frac{a}{2}\right)=$ 3.52 k-ft	
$I_{cr}=\frac{E_s}{E_c}\left(A_s+\frac{P_U}{f_y}\left(\frac{h}{2d}\right)\right)(d-c)^2+\frac{bc^3}{3}=$ 13.67 in^4	EQ 11.8.3.1c
DEFLECTION CHECK	
$\Delta_{SMAX}=\frac{l_c}{150}=$ 1.600 in	SECTION 11.8.1.1e
$\Delta_{cr}=\frac{5M_{cr}l_c^2}{48E_cI_{cr}}=$ 0.287 in	EQ 11.8.4.3a
$\Delta_n=\frac{5M_n l_c^2}{48E_cI_{cr}}=$ 6.597 in	EQ 11.8.4.3b
$M_{SA}=\frac{w_s l_c^2}{8}+\frac{P_{SA}e_{cc}}{2}=$ 1.907 k-ft	
ASSUME: $M_A\le\frac{2M_{cr}}{3}\therefore\ \Delta_s=\frac{M_A\Delta_{cr}}{M_{cr}}$	SECTION R11.8.4.1
Iteration 1:	
$\Delta_s=\frac{M_A\Delta_{cr}}{M_{cr}}=$ 0.2643 in	TAB. 11.8.4.1a
$M_A=M_{SA}+P_{SM}\Delta_s=$ 2.240 k-ft	EQ 11.8.4.2

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CALCULATIONS	REFERENCES
	ACI 318-14
Iteration 2:	
$\Delta_s = \frac{M_A \Delta_{cr}}{M_{cr}} =$ 0.3106 in	TAB. 11.8.4.1a
$M_A = M_{SA} + P_{SM}\Delta_s =$ 1.939 k-ft	EQ 11.8.4.2
Iteration 3:	
$\Delta_s = \frac{M_A \Delta_{cr}}{M_{cr}} =$ 0.2688 in	TAB. 11.8.4.1a
$M_A = M_{SA} + P_{SM}\Delta_s =$ 1.935 k-ft	EQ 11.8.4.2
Iteration 4:	
$\Delta_s = \frac{M_A \Delta_{cr}}{M_{cr}} =$ 0.2682 in	TAB. 11.8.4.1a
$M_A = M_{SA} + P_{SM}\Delta_s =$ 1.935 k-ft	EQ 11.8.4.2
Iteration 5:	
$\Delta_s = \frac{M_A \Delta_{cr}}{M_{cr}} =$ 0.2682 in	TAB. 11.8.4.1a
$M_A = M_{SA} + P_{SM}\Delta_s =$ 1.935 k-ft	EQ 11.8.4.2
COVERGES TO: $\Delta_S =$ 0.2682 in; $M_A =$ 1.935 k-ft	
$M_A =$ 1.935 > $\frac{2M_{cr}}{3} =$ 1.381	TAB. 11.8.4.1
ASSUMPTION INCORRECT	
IF INITIAL ASSUMPTION IS INCORRECT:	
ASSUME: $M_a \geq \frac{2M_{cr}}{3} \therefore \Delta_s = \frac{2\Delta_{cr}}{3} + \frac{\left(M_a - \frac{2\Delta_{cr}}{3}\right)}{\left(M_n - \frac{2\Delta_{cr}}{3}\right)}\left(\Delta_n - \frac{2\Delta_{cr}}{3}\right)$	SECTION R11.8.4.1

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CALCULATIONS	REFERENCES
	ACI 318-14
Iteration 1:	
$\Delta_s = \frac{2\Delta_{cr}}{3} + \frac{\left(M_a - \frac{2\Delta_{cr}}{3}\right)}{\left(M_n - \frac{2\Delta_{cr}}{3}\right)}\left(\Delta_n - \frac{2\Delta_{cr}}{3}\right) =$ 1.560 in	TAB. 11.8.4.1b
$M_A = M_{SA} + P_{SM}\Delta_s =$ 1.907 k-ft	EQ 11.8.4.2
M_A = 1.907 > $\frac{2M_{cr}}{3} =$ 1.381	TAB. 11.8.4.1
ASSUMPTION CORRECT	
Δ_S = 1.5596 < Δ_{SMAX} = 1.600	SECTION 11.8.1.1e
GOOD	
CHECK MINIMUM STEEL	
$\rho = \frac{A_s}{bh} =$ 0.00455 > $\rho_{l,MIN}$ = 0.0012	TAB 11.6.1
GOOD	
TRANSVERSE REINFORCEMENT	
$\rho_{t,MIN}$ = 0.002 $A_{S,MIN} = \rho_{t,MIN}bh$ = 2.64 in^2	TAB 11.6.1
Use: (14)#4 AT 16 IN O.C.	SECTION 11.7.3.1
STEEL TONNAGE	
#4 @ 8 IN O.C. VERTICAL = 30 Sticks @ 23.75 ft	
#4 @ 16.5 IN O.C. HORIZONTAL = 18 Sticks @ 19.75 ft	
$Tonnage_{Long}$ = 0.238 tons	
$Tonnage_{Trans}$ = 0.119 tons	
$Tonnage_{Total}$ = 0.357 tons	

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CALCULATIONS	REFERENCES
GOVERNING CASE $M_{U,MAX}$ = 2.295 K-ft **LC 6 GOVERNS** Total Tonnage = 0.3567 tons	ACI 318-14

Appendix B - Panel Reinforcement Deign Results

Table 6-1 Design results, 6” nominal thickness, 3000 psi

	SINGLY REINFORCED PANELS BY HEIGHT FOR t_{NOM} = 6", f'_c = 3000 psi								
	ASTM A615 GRADE 60				ASTM A615 GRADE 80				
	DESIGNATION	TONNAGE (PER PANEL)	SPACING (in)	TOTAL COST ($/PANEL)	DESIGNATION	TONNAGE (PER PANEL)	SPACING (in)	TOTAL COST ($/PANEL)	COST PREMIUM (%)
#4 BAR	20'.6".3.60.#4.1	0.356712	8	$ 1,401.35	20'.6".3.80.#4.1	0.356712	8	$ 1,435.24	2.42%
	25'.6".3.60.#4.1	INVALID	2	$ -	25'.6".3.80.#4.1	INVALID	2	$ -	0.00%
	30'.6".3.60.#4.1	INVALID	2	$ -	30'.6".3.80.#4.1	INVALID	2	$ -	0.00%
	35'.6".3.60.#4.1	INVALID	2	$ -	35'.6".3.80.#4.1	INVALID	2	$ -	0.00%
	40'.6".3.60.#4.1	INVALID	2	$ -	40'.6".3.80.#4.1	INVALID	2	$ -	0.00%
#5 BAR	20'.6".3.60.#5.1	INVALID	2	$ -	20'.6".3.80.#5.1	INVALID	4	$ -	0.00%
	25'.6".3.60.#5.1	INVALID	2	$ -	25'.6".3.80.#5.1	INVALID	2	$ -	0.00%
	30'.6".3.60.#5.1	INVALID	2	$ -	30'.6".3.80.#5.1	INVALID	2	$ -	0.00%
	35'.6".3.60.#5.1	INVALID	2	$ -	35'.6".3.80.#5.1	INVALID	2	$ -	0.00%
	40'.6".3.60.#5.1	INVALID	2	$ -	40'.6".3.80.#5.1	INVALID	2	$ -	0.00%
#6 BAR	20'.6".3.60.#6.1	INVALID	2	$ -	20'.6".3.80.#6.1	INVALID	2	$ -	0.00%
	25'.6".3.60.#6.1	INVALID	2	$ -	25'.6".3.80.#6.1	INVALID	2	$ -	0.00%
	30'.6".3.60.#5.1	INVALID	2	$ -	30'.6".3.80.#5.1	INVALID	2	$ -	0.00%
	35'.6".3.60.#5.1	INVALID	2	$ -	35'.6".3.80.#5.1	INVALID	2	$ -	0.00%
	40'.6".3.60.#6.1	INVALID	2	$ -	40'.6".3.80.#6.1	INVALID	2	$ -	0.00%

Table 6-2 Design results, singly reinforced 8” nominal thickness, 3000 psi

	SINGLY REINFORCED PANELS BY HEIGHT FOR t_{NOM} = 8", f'_c = 3000 psi								
	ASTM A615 GRADE 60				ASTM A615 GRADE 80				
	DESIGNATION	TONNAGE (PER PANEL)	SPACING (in)	TOTAL COST ($/PANEL)	DESIGNATION	TONNAGE (PER PANEL)	SPACING (in)	TOTAL COST ($/PANEL)	COST PREMIUM (%)
#4 BAR	20'.8".3.60.#4.1	0.270707	17	$ 1,443.24	20'.8".3.80.#4.1	0.2627745	18	$ 1,452.14	0.62%
	25'.8".3.60.#4.1	0.4085655	10	$ 1,946.17	25'.8".3.80.#4.1	0.3893605	11	$ 1,944.27	-0.10%
	30'.8".3.60.#4.1	INVALID	2	$ -	30'.8".3.80.#4.1	INVALID	2	$ -	0.00%
	35'.8".3.60.#4.1	INVALID	2	$ -	35'.8".3.80.#4.1	INVALID	2	$ -	0.00%
	40'.8".3.60.#4.1	INVALID	2	$ -	40'.8".3.80.#4.1	INVALID	2	$ -	0.00%
#5 BAR	20'.8".3.60.#5.1	0.32511825	18	$ 1,553.43	20'.8".3.80.#5.1	0.32511825	18	$ 1,584.31	1.99%
	25'.8".3.60.#5.1	0.4179955	15	$ 1,965.27	25'.8".3.80.#5.1	0.403002375	16	$ 1,973.19	0.40%
	30'.8".3.60.#5.1	INVALID	2	$ -	30'.8".3.80.#5.1	INVALID	2	$ -	0.00%
	35'.8".3.60.#5.1	INVALID	2	$ -	35'.8".3.80.#5.1	INVALID	2	$ -	0.00%
	40'.8".3.60.#5.1	INVALID	2	$ -	40'.8".3.80.#5.1	INVALID	2	$ -	0.00%
#6 BAR	20'.8".3.60.#6.1	0.401427	18	$ 1,707.95	20'.8".3.80.#6.1	0.401427	18	$ 1,746.09	2.23%
	25'.8".3.60.#6.1	0.480383	18	$ 2,091.60	25'.8".3.80.#6.1	0.480383	18	$ 2,137.24	2.18%
	30'.8".3.60.#5.1	INVALID	2	$ -	30'.8".3.80.#5.1	INVALID	2	$ -	0.00%
	35'.8".3.60.#5.1	INVALID	2	$ -	35'.8".3.80.#5.1	INVALID	2	$ -	0.00%
	40'.8".3.60.#6.1	INVALID	2	$ -	40'.8".3.80.#6.1	INVALID	2	$ -	0.00%

Table 6-3 Design results, doubly reinforced 8” nominal thickness, 3000 psi

	DOUBLY REINFORCED PANELS BY HEIGHT FOR t_{NOM} = 8", f'_c = 3000 psi								
	ASTM A615 GRADE 60				ASTM A615 GRADE 80				
	DESIGNATION	TONNAGE (PER PANEL)	SPACING (in)	TOTAL COST ($/PANEL)	DESIGNATION	TONNAGE (PER PANEL)	SPACING (in)	TOTAL COST ($/PANEL)	COST PREMIUM (%)
#4 BAR	20'.8".3.60.#4.2	0.3738295	18	$ 1,652.07	20'.8".3.80.#4.2	0.3738295	18	$ 1,687.58	2.15%
	25'.8".3.60.#4.2	0.4469755	18	$ 2,023.95	25'.8".3.80.#4.2	0.4469755	18	$ 2,066.42	2.10%
	30'.8".3.60.#4.2	0.752168	10	$ 2,865.73	30'.8".3.80.#4.2	0.752168	10	$ 2,937.19	2.49%
	35'.8".3.60.#4.2	1.790574	4	$ 5,192.27	35'.8".3.80.#4.2	1.790574	4	$ 5,362.37	3.28%
	40'.8".3.60.#4.2	INVALID	2	$ -	40'.8".3.80.#4.2	INVALID	2	$ -	0.00%
#5 BAR	20'.8".3.60.#5.2	0.498517	18	$ 1,904.56	20'.8".3.80.#5.2	0.498517	18	$ 1,951.92	2.49%
	25'.8".3.60.#5.2	0.597913	18	$ 2,329.60	25'.8".3.80.#5.2	0.597913	18	$ 2,386.40	2.44%
	30'.8".3.60.#5.2	0.774308	15	$ 2,910.57	30'.8".3.80.#5.2	0.73910675	16	$ 2,909.50	-0.04%
	35'.8".3.60.#5.2	1.854124	6	$ 5,320.96	35'.8".3.80.#5.2	1.854124	6	$ 5,497.10	3.31%
	40'.8".3.60.#5.2	INVALID	2	$ -	40'.8".3.80.#5.2	INVALID	2	$ -	0.00%
#6 BAR	20'.8".3.60.#6.2	0.6511345	18	$ 2,213.61	20'.8".3.80.#6.2	0.6511345	18	$ 2,275.47	2.79%
	25'.8".3.60.#6.2	0.7826605	18	$ 2,703.71	25'.8".3.80.#6.2	0.7826605	18	$ 2,778.07	2.75%
	30'.8".3.60.#5.2	0.774308	15	$ 2,910.57	30'.8".3.80.#5.2	0.73910675	16	$ 2,909.50	-0.04%
	35'.8".3.60.#5.2	1.854124	6	$ 5,320.96	35'.8".3.80.#5.2	1.854124	6	$ 5,497.10	3.31%
	40'.8".3.60.#6.2	INVALID	2	$ -	40'.8".3.80.#6.2	INVALID	2	$ -	0.00%

Table 6-4 Design results, 10” nominal thickness, 3000 psi

	DOUBLY REINFORCED PANELS BY HEIGHT FOR t_{NOM} = 10", f'_c = 3000 psi								
	ASTM A615 GRADE 60				ASTM A615 GRADE 80				
	DESIGNATION	TONNAGE (PER PANEL)	SPACING (in)	TOTAL COST ($/PANEL)	DESIGNATION	TONNAGE (PER PANEL)	SPACING (in)	TOTAL COST ($/PANEL)	COST PREMIUM (%)
#4 BAR	20'.10".3.60.#4.2	0.4134085	18	$ 1,979.13	20'.10".3.80.#4.2	0.4134085	18	$ 2,018.40	1.98%
	25'.10".3.60.#4.2	0.4997475	18	$ 2,439.46	25'.10".3.80.#4.2	0.4997475	18	$ 2,486.93	1.95%
	30'.10".3.60.#4.2	0.5860865	18	$ 2,899.79	30'.10".3.80.#4.2	0.5860865	18	$ 2,955.47	1.92%
	35'.10".3.60.#4.2	0.8277355	12	$ 3,674.62	35'.10".3.80.#4.2	0.8018505	13	$ 3,698.38	0.65%
	40'.10".3.60.#4.2	1.5186145	6	$ 5,359.14	40'.10".3.80.#4.2	1.5186145	6	$ 5,503.41	2.69%
#5 BAR	20'.10".3.60.#5.2	0.538096	18	$ 2,231.62	20'.10".3.80.#5.2	0.538096	18	$ 2,282.74	2.29%
	25'.10".3.60.#5.2	0.650685	18	$ 2,745.11	25'.10".3.80.#5.2	0.650685	18	$ 2,806.92	2.25%
	30'.10".3.60.#5.2	0.763274	18	$ 3,258.59	30'.10".3.80.#5.2	0.763274	18	$ 3,331.10	2.23%
	35'.10".3.60.#5.2	0.875863	18	$ 3,772.08	35'.10".3.80.#5.2	0.875863	18	$ 3,855.29	2.21%
	40'.10".3.60.#5.2	1.4447645	10	$ 5,209.60	40'.10".3.80.#5.2	1.4447645	10	$ 5,346.85	2.63%
#6 BAR	20'.10".3.60.#6.2	0.6907135	18	$ 2,540.67	20'.10".3.80.#6.2	0.6907135	18	$ 2,606.29	2.58%
	25'.10".3.60.#6.2	0.8354325	18	$ 3,119.22	25'.10".3.80.#6.2	0.8354325	18	$ 3,198.59	2.54%
	30'.10".3.60.#5.2	0.763274	18	$ 3,258.59	30'.10".3.80.#5.2	0.763274	18	$ 3,331.10	2.23%
	35'.10".3.60.#5.2	0.875863	18	$ 3,772.08	35'.10".3.80.#5.2	0.875863	18	$ 3,855.29	2.21%
	40'.10".3.60.#6.2	1.5324395	14	$ 5,387.14	40'.10".3.80.#6.2	1.5324395	14	$ 5,532.72	2.70%

Table 6-5 Design results, 6" nominal thickness, 4000 psi

	SINGLY REINFORCED PANELS BY HEIGHT FOR t_{NOM} = 6", f'_c = 4000 psi								
	ASTM A615 GRADE 60				ASTM A615 GRADE 80				
	DESIGNATION	TONNAGE (PER PANEL)	SPACING (in)	TOTAL COST ($/PANEL)	DESIGNATION	TONNAGE (PER PANEL)	SPACING (in)	TOTAL COST ($/PANEL)	COST PREMIUM (%)
#4 BAR	20'.6".4.60.#4.1	0.356712	8	$ 1,401.35	20'.6".3.80.#4.1	0.356712	8	$ 1,435.24	2.42%
	25'.6".4.60.#4.1	INVALID	2	$ -	25'.6".3.80.#4.1	INVALID	2	$ -	0.00%
	30'.6".4.60.#4.1	INVALID	2	$ -	30'.6".3.80.#4.1	INVALID	2	$ -	0.00%
	35'.6".4.60.#4.1	INVALID	2	$ -	35'.6".3.80.#4.1	INVALID	2	$ -	0.00%
	40'.6".4.60.#4.1	INVALID	2	$ -	40'.6".3.80.#4.1	INVALID	2	$ -	0.00%
#5 BAR	20'.6".4.60.#5.1	INVALID	2	$ -	20'.6".3.80.#5.1	INVALID	4	$ -	0.00%
	25'.6".4.60.#5.1	INVALID	2	$ -	25'.6".3.80.#5.1	INVALID	2	$ -	0.00%
	30'.6".4.60.#5.1	INVALID	2	$ -	30'.6".3.80.#5.1	INVALID	2	$ -	0.00%
	35'.6".4.60.#5.1	INVALID	2	$ -	35'.6".3.80.#5.1	INVALID	2	$ -	0.00%
	40'.6".4.60.#5.1	INVALID	2	$ -	40'.6".3.80.#5.1	INVALID	2	$ -	0.00%
#6 BAR	20'.6".4.60.#6.1	INVALID	2	$ -	20'.6".3.80.#6.1	INVALID	2	$ -	0.00%
	25'.6".4.60.#6.1	INVALID	2	$ -	25'.6".3.80.#6.1	INVALID	2	$ -	0.00%
	30'.6".4.60.#5.1	INVALID	2	$ -	30'.6".3.80.#5.1	INVALID	2	$ -	0.00%
	35'.6".4.60.#5.1	INVALID	2	$ -	35'.6".3.80.#5.1	INVALID	2	$ -	0.00%
	40'.6".4.60.#6.1	INVALID	2	$ -	40'.6".3.80.#6.1	INVALID	2	$ -	0.00%

Table 6-6 Design results, singly reinforced 8" nominal thickness, 4000 psi

	SINGLY REINFORCED PANELS BY HEIGHT FOR t_{NOM} = 8", f'_c = 4000 psi								
	ASTM A615 GRADE 60				ASTM A615 GRADE 80				
	DESIGNATION	TONNAGE (PER PANEL)	SPACING (in)	TOTAL COST ($/PANEL)	DESIGNATION	TONNAGE (PER PANEL)	SPACING (in)	TOTAL COST ($/PANEL)	COST PREMIUM (%)
#4 BAR	20'.8".4.60.#4.1	0.270707	17	$ 1,443.24	20'.8".3.80.#4.1	0.2627745	18	$ 1,452.14	0.62%
	25'.8".4.60.#4.1	0.4085655	10	$ 1,946.17	25'.8".3.80.#4.1	0.3893605	11	$ 1,944.27	-0.10%
	30'.8".4.60.#4.1	INVALID	2	$ -	30'.8".3.80.#4.1	INVALID	2	$ -	0.00%
	35'.8".4.60.#4.1	INVALID	2	$ -	35'.8".3.80.#4.1	INVALID	2	$ -	0.00%
	40'.8".4.60.#4.1	INVALID	2	$ -	40'.8".3.80.#4.1	INVALID	2	$ -	0.00%
#5 BAR	20'.8".4.60.#5.1	0.32511825	18	$ 1,553.43	20'.8".3.80.#5.1	0.32511825	18	$ 1,584.31	1.99%
	25'.8".4.60.#5.1	0.4179955	15	$ 1,965.27	25'.8".3.80.#5.1	0.403002375	16	$ 1,973.19	0.40%
	30'.8".4.60.#5.1	INVALID	2	$ -	30'.8".3.80.#5.1	INVALID	2	$ -	0.00%
	35'.8".4.60.#5.1	INVALID	2	$ -	35'.8".3.80.#5.1	INVALID	2	$ -	0.00%
	40'.8".4.60.#5.1	INVALID	2	$ -	40'.8".3.80.#5.1	INVALID	2	$ -	0.00%
#6 BAR	20'.8".4.60.#6.1	0.401427	18	$ 1,707.95	20'.8".3.80.#6.1	0.401427	18	$ 1,746.09	2.23%
	25'.8".4.60.#6.1	0.480383	18	$ 2,091.60	25'.8".3.80.#6.1	0.480383	18	$ 2,137.24	2.18%
	30'.8".4.60.#5.1	INVALID	2	$ -	30'.8".3.80.#5.1	INVALID	2	$ -	0.00%
	35'.8".4.60.#5.1	INVALID	2	$ -	35'.8".3.80.#5.1	INVALID	2	$ -	0.00%
	40'.8".4.60.#6.1	INVALID	2	$ -	40'.8".3.80.#6.1	INVALID	2	$ -	0.00%

Table 6-7 Design results, doubly reinforced 8” nominal thickness, 4000 psi

	DOUBLY REINFORCED PANELS BY HEIGHT FOR t_{NOM} = 8", f'_c = 4000 psi								
	ASTM A615 GRADE 60				ASTM A615 GRADE 80				
	DESIGNATION	TONNAGE (PER PANEL)	SPACING (in)	TOTAL COST ($/PANEL)	DESIGNATION	TONNAGE (PER PANEL)	SPACING (in)	TOTAL COST ($/PANEL)	COST PREMIUM (%)
#4 BAR	20'.8".4.60.#4.2	0.3738295	18	$ 1,652.07	20'.8".3.80.#4.2	0.3738295	18	$ 1,687.58	2.15%
	25'.8".4.60.#4.2	0.4469755	18	$ 2,023.95	25'.8".3.80.#4.2	0.4469755	18	$ 2,066.42	2.10%
	30'.8".4.60.#4.2	0.752168	10	$ 2,865.73	30'.8".3.80.#4.2	0.752168	10	$ 2,937.19	2.49%
	35'.8".4.60.#4.2	1.790574	4	$ 5,192.27	35'.8".3.80.#4.2	1.790574	4	$ 5,362.37	3.28%
	40'.8".4.60.#4.2	INVALID	2	$ -	40'.8".3.80.#4.2	INVALID	2	$ -	0.00%
#5 BAR	20'.8".4.60.#5.2	0.498517	18	$ 1,904.56	20'.8".3.80.#5.2	0.498517	18	$ 1,951.92	2.49%
	25'.8".4.60.#5.2	0.597913	18	$ 2,329.60	25'.8".3.80.#5.2	0.597913	18	$ 2,386.40	2.44%
	30'.8".4.60.#5.2	0.774308	15	$ 2,910.57	30'.8".3.80.#5.2	0.73910675	16	$ 2,909.50	-0.04%
	35'.8".4.60.#5.2	1.854124	6	$ 5,320.96	35'.8".3.80.#5.2	1.854124	6	$ 5,497.10	3.31%
	40'.8".4.60.#5.2	INVALID	2	$ -	40'.8".3.80.#5.2	INVALID	2	$ -	0.00%
#6 BAR	20'.8".4.60.#6.2	0.6511345	18	$ 2,213.61	20'.8".3.80.#6.2	0.6511345	18	$ 2,275.47	2.79%
	25'.8".4.60.#6.2	0.7826605	18	$ 2,703.71	25'.8".3.80.#6.2	0.7826605	18	$ 2,778.07	2.75%
	30'.8".4.60.#5.2	0.774308	15	$ 2,910.57	30'.8".3.80.#5.2	0.73910675	16	$ 2,909.50	-0.04%
	35'.8".4.60.#5.2	1.854124	6	$ 5,320.96	35'.8".3.80.#5.2	1.854124	6	$ 5,497.10	3.31%
	40'.8".4.60.#6.2	INVALID	2	$ -	40'.8".3.80.#6.2	INVALID	2	$ -	0.00%

Table 6-8 Design results, 10” nominal thickness, 4000 psi

	DOUBLY REINFORCED PANELS BY HEIGHT FOR t_{NOM} = 10", f'_c = 4000 psi								
	ASTM A615 GRADE 60				ASTM A615 GRADE 80				
	DESIGNATION	TONNAGE (PER PANEL)	SPACING (in)	TOTAL COST ($/PANEL)	DESIGNATION	TONNAGE (PER PANEL)	SPACING (in)	TOTAL COST ($/PANEL)	COST PREMIUM (%)
#4 BAR	20'.10".4.60.#4.2	0.4134085	18	$ 1,979.13	20'.10".3.80.#4.2	0.4134085	18	$ 2,018.40	1.98%
	25'.10".4.60.#4.2	0.4997475	18	$ 2,439.46	25'.10".3.80.#4.2	0.4997475	18	$ 2,486.93	1.95%
	30'.10".4.60.#4.2	0.5860865	18	$ 2,899.79	30'.10".3.80.#4.2	0.5860865	18	$ 2,955.47	1.92%
	35'.10".4.60.#4.2	0.8277355	12	$ 3,674.62	35'.10".3.80.#4.2	0.8018505	13	$ 3,698.38	0.65%
	40'.10".4.60.#4.2	1.5186145	6	$ 5,359.14	40'.10".3.80.#4.2	1.5186145	6	$ 5,503.41	2.69%
#5 BAR	20'.10".4.60.#5.2	0.538096	18	$ 2,231.62	20'.10".3.80.#5.2	0.538096	18	$ 2,282.74	2.29%
	25'.10".4.60.#5.2	0.650685	18	$ 2,745.11	25'.10".3.80.#5.2	0.650685	18	$ 2,806.92	2.25%
	30'.10".4.60.#5.2	0.763274	18	$ 3,258.59	30'.10".3.80.#5.2	0.763274	18	$ 3,331.10	2.23%
	35'.10".4.60.#5.2	0.875863	18	$ 3,772.08	35'.10".3.80.#5.2	0.875863	18	$ 3,855.29	2.21%
	40'.10".4.60.#5.2	1.4447645	10	$ 5,209.60	40'.10".3.80.#5.2	1.4447645	10	$ 5,346.85	2.63%
#6 BAR	20'.10".4.60.#6.2	0.6907135	18	$ 2,540.67	20'.10".3.80.#6.2	0.6907135	18	$ 2,606.29	2.58%
	25'.10".4.60.#6.2	0.8354325	18	$ 3,119.22	25'.10".3.80.#6.2	0.8354325	18	$ 3,198.59	2.54%
	30'.10".4.60.#5.2	0.763274	18	$ 3,258.59	30'.10".3.80.#5.2	0.763274	18	$ 3,331.10	2.23%
	35'.10".4.60.#5.2	0.875863	18	$ 3,772.08	35'.10".3.80.#5.2	0.875863	18	$ 3,855.29	2.21%
	40'.10".4.60.#6.2	1.5324395	14	$ 5,387.14	40'.10".3.80.#6.2	1.5324395	14	$ 5,532.72	2.70%

Appendix C - Permission for Use

RE: Mechanical Properties of High-strength Reinforcement Figure

Graham, Scott <sgraham@wje.com>

Thu 11/2/2017 12:52 PM

To: Sam McConnell <smcconnell@ksu.edu>;

Dear Sam,

Many thanks for reaching out. We don't take any exception to you referencing our figure. Just be sure to cite our report and then also include a "Copywright WJE" on the figure so that it is clear you are using the figure with our permission. Let me know if you have any other questions. Once your thesis is complete, I wouldn't mind reading it!

Thanks,
Scott