ANALYSIS OF VERTICAL REINFORCEMENT IN SLENDER REINFORCED CONCRETE (TILT-UP) PANELS WITH OPENINGS & SUBJECT TO VARYING WIND PRESSURES

by

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Abstract

This report offers a parametric study analyzing the vertical reinforcement for slender reinforced concrete walls (tilt-up panels) subject to 90 miles per hour (mph), 110 mph, 130 mph, and 150 mph three-second gust wind speeds. Wall panel heights of 32 feet (ft) and 40 ft are considered for one-story warehouse structures. First, solid tilt-up panels serve as the base design used in the comparison process. Next, square openings of 4 ft, 8 ft, 12 ft, and 16 ft centered in the wall panel, are analyzed. A total of 32 tilt-up panel designs are conducted, establishing the most economical design by the least amount of reinforcement and concrete used. In addition to lateral wind pressures, the gravity loads acting on the load bearing tilt-up panel are dead load, roof live load, and snow load. All loads for this report are determined based on a typical 24 ft by 24 ft bay. The procedure to design the tilt-up panels is the Alternative Design of Slender Walls outlined in the American Concrete Institute standard ACI 318-08 *Building Code Requirements for Structural Concrete and Commentary* Section 14.8

In general, an increase in panel height, lateral wind pressure, and/or panel openings, requires an increase in reinforcement to meet strength and serviceability. Typical vertical reinforcement in tilt-up panels is #4, #5, and #6 size reinforcement bars. A double-mat reinforcement scheme is utilized when the section requires an increase in reinforcement provided by use of a single-layer of reinforcement. A thicker tilt-up panel may be needed to ensure tension-controlled behavior. Panel thicknesses of 7.25 inches (in), 9.25 in, and 11.25 in are considered in design.

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a – depth of equivalent stress block

A_g – gross area of concrete section

A_s – area of tension reinforcement

A_{se} – effective area of tension reinforcement

$A_{s,min}$ – minimum area of reinforcement

b– width of the concrete section

c – distance from the extreme fiber to the neutral axis

C_e – exposure factor

C_s – slope factor

C_t – thermal factor

d – distance from the extreme concrete compression fiber to the centroid of tension of reinforcement

d_t – distance from the extreme compression fiber to centroid of extreme layer of longitudinal tension steel

D – dead load

e_{cc} – eccentricity of applied load(s)

E_c – concrete modulus of elasticity

E_s – steel modulus of elasticity

f'_c – compressive strength of concrete

f_y – reinforcement yield stress

f_r – modulus of rupture

G – gust effect factor

GC_p – external pressure coefficient

GC_{pi} – internal pressure coefficient

I – importance factor

I_{cr} – cracked section moment of inertia

I_e – effective moment interia

I_g – gross concrete section moment of inertia

ℓ_c – unbraced height

ℓ_w – width of concrete section

K_b – bending stiffness

K_z – velocity of pressure coefficient at height, z

K_{zt} – topographic factor

K_d – wind directionality factor

L_r – roof live load

M_a – maximum moment at mid-height of wall due to service lateral and eccentric vertical loads, including $P\text{-}\Delta$ effects

M_{cr} – moment causing flexural cracking of the concrete section

M_{max} – maximum moment occurring over the span of the panel due to uniform lateral loads

M_n – nominal moment strength at the mid-height cross-section

M_u – maximum combined moment at mid-height of wall due to factored lateral and eccentric vertical loads including $P\text{-}\Delta$ effects

M_{ua} – maximum moment at mid-height of wall due to factored lateral and eccentric vertical loads not including $P\text{-}\Delta$ effects

n – modular ratio

P – applied axial load at top of panel

$P\text{-}\Delta$ – secondary moment caused by axial load P acting on a deflected shape with displacement Δ

P_{ua} – factored axial load due to factored eccentric vertical loads

P_{um} – factored axial load due to factored eccentric vertical loads, including panel self-weight above mid-height of panel

q_h – effective velocity pressure at mean roof height, h

S – snow load

S_{DS} – design spectral response acceleration parameter at short periods

S_S – mapped short-period spectral acceleration

t – wall thickness

w – uniform lateral load

w_u – factored uniform lateral load on element

W – wind load

y_t – distance from centroidal axis of gross section to tension face

α – coefficient of thermal expansion

β_1 – factor relating depth of equivalent rectangular compressive stress block to neutral axis depth

Δ_{allow} – maximum allowable deflection at mid-height

Δ_{max} – maximum total deflection at mid-height

Δ_n – maximum potential deflection at mid-height

Δ_s – maximum out-of-plane deflection due to service loads, including P-Δ effects

Δ_u – computed deflection at mid-height of wall due to factored loads

ε_t – net tensile strain in extreme layer of tension steel at nominal strength

λ – modification factor reflecting the reduced mechanical properties of lightweight concrete

ρ – ratio of A_s to bd

ρ_l – ratio of area of distributed longitudinal reinforcement to gross concrete area perpendicular to that reinforcement

Φ – strength-reduction factor

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CHAPTER 1 - Introduction

This report includes a parametric study evaluating the vertical reinforcing requirements for concrete tilt-up wall panels with various sized openings centered in the panel subject to varying wind speeds applied perpendicular to the surface. For academia, the technical description of reinforced concrete slender walls is appropriate, whereas tilt-up wall panels is common within the construction industry. The report begins with an overview of concrete material properties, reinforcing steel, and the basic mechanics of reinforced concrete; next, it evaluates the vertical reinforcement resisting out-of-plane forces in accordance with the American Concrete Institute standard ACI 318-08 Building Code and Commentary Section 14.8, Alternative Design of Slender Walls. Two design calculation examples, one of a solid tilt-up wall panel and one of a wall panel incorporating an opening, are presented to demonstrate the analysis process used throughout the study. Finally, the end of this report provides the design of the vertical reinforcement for all panel scenarios discussed in Chapter 2.

CHAPTER 2 - Scope of Research

This report discusses ten different panel configurations experiencing varying wind speeds, and evaluates the reinforcement required for each tilt-up panel, specifically the vertical reinforcement for the panel 'leg'. A panel 'leg' is the portion on each side of the opening. A total of 40 different panels were evaluated for the strength load combinations given in American Society of Civil Engineers' *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-05). Accordingly, Figure 2-1 contains five tilt-up panel configurations with two panel heights.

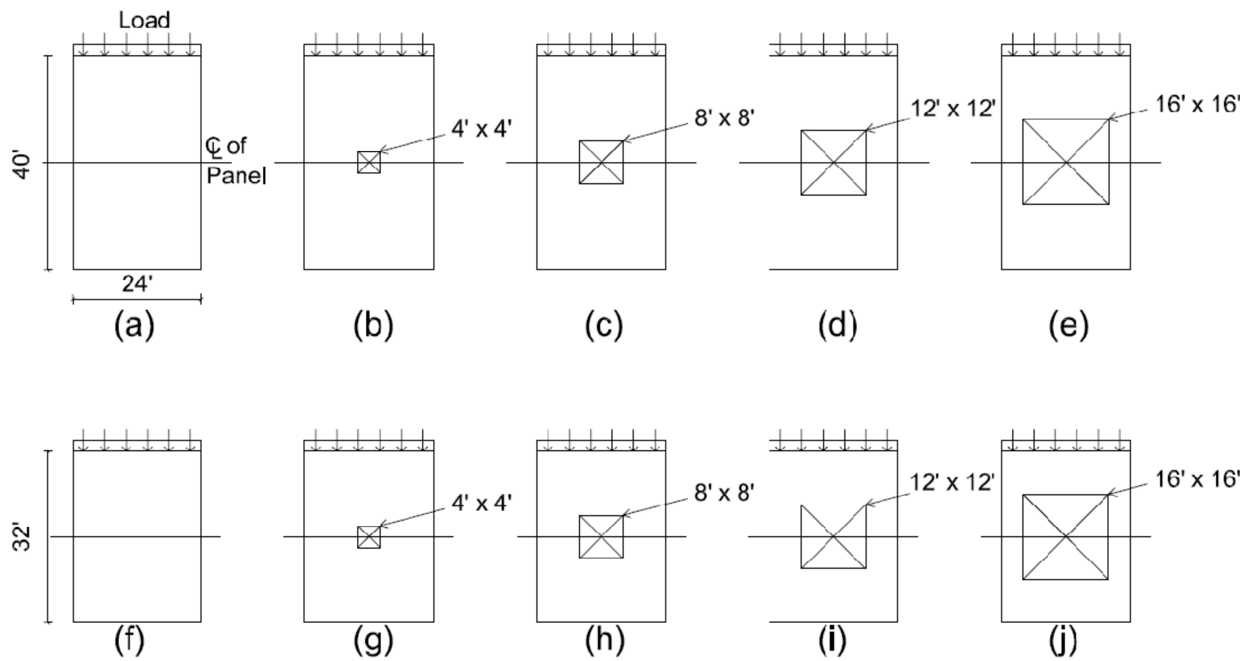

Figure 2-1 Panel Configurations

First, Figure 2-1 offers five basic panel configurations with two different heights as follows:

- solid panel
- 4'-0" x 4'-0" opening centered in panel
- 8'-0" x 8'-0" opening centered in panel
- 12'-0" x 12'-0" opening centered in panel
- 16'-0" x 16'-0" opening centered in panel

The openings are located in the center of the panels where the largest stresses occur, causing the worst case scenario. In addition, Figure 2-1 accounts for panel configurations (a)-(e) that have an unbraced length, ℓ_c, of 32 feet (ft) and panel configurations (f)-(j) that have an unbraced length, ℓ_c, of 40 ft. According to Tilt-Up Concrete Association (TCA), the 32 ft and 40 ft unbraced lengths are common for warehouse structures (Schmitt, 2009). Each of the ten load bearing tilt-up panels are designed for 90 mph, 110 mph, 130 mph, and 150 mph wind speeds using the Analytical Method in Chapter 6 of ASCE 7-05 provisions. The Main Wind Resisting Forces (MWRF) and Components and Cladding (C&C) forces were determined and compared.

Further, this report uses a modified floor plan of the tilt-up structure from the 2006 IBC Structural/Seismic Design Manual, Vol. II, shown in Figure 2-2. Notably, only the bolded load-bearing tilt-up panels are evaluated in the design procedure. Therefore, wind in the transverse direction.

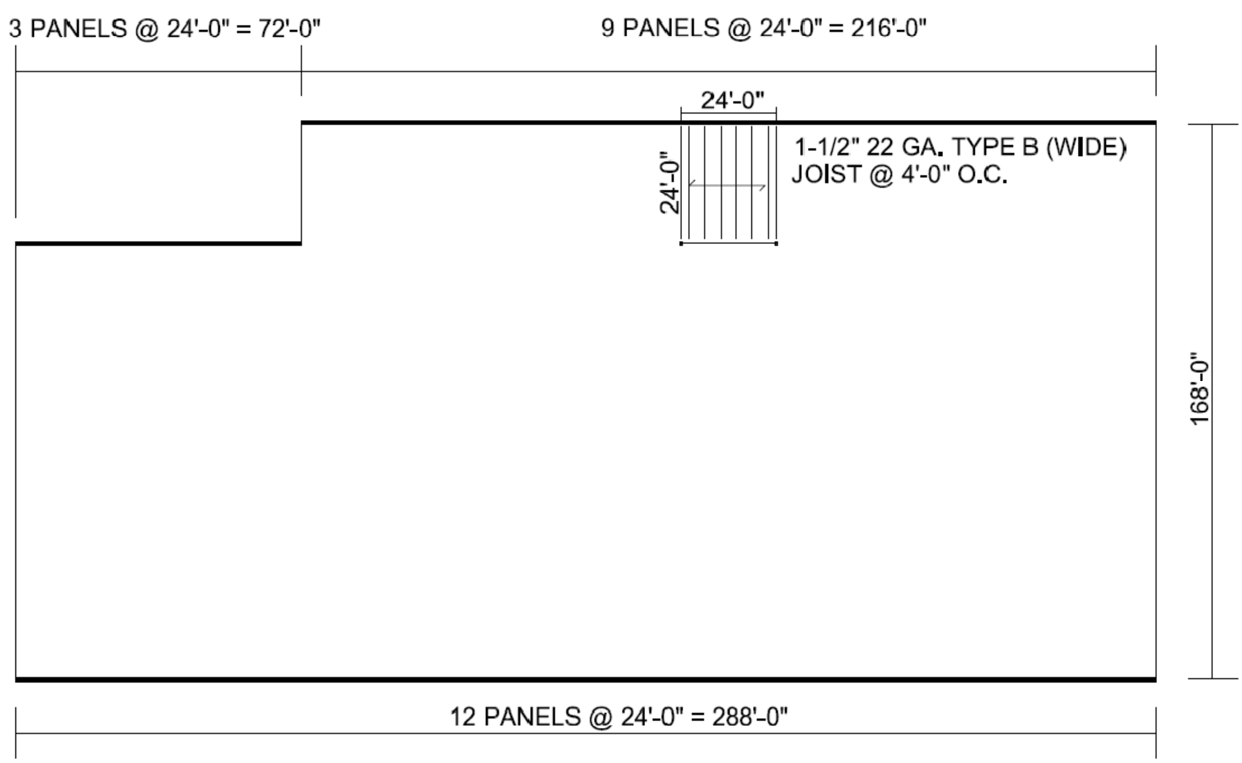

Figure 2-2 Case Study Floor Plan

Analyzing the effect varying wind speeds have on load-bearing tilt-up panels required the short period response acceleration parameter, S_s, to be less than a certain value to ensure that wind pressures due to MWRF and C&C govern over seismic forces. The out-of-plane force due to seismic load for structural walls is defined by Section 12.11.1 of ASCE 7-05. Where the soil properties are not known in sufficient detail, Site Class D is assumed as defined in Section 11.4.2 of ASCE 7-05 provisions. Specifically, the scope of this report covers the tilt-up structure located in regions that meet the short period response acceleration values shown in Table 2-1.

Panel Height	Panel Thickness (in)	$S_s \leq$	$0.4*I*S_{DS}*$panel weight	Wind Speed (3-s gust)	Wind Pressure with K_d = 0.85
32'-0"	7.25	1.410	23.9 psf	90 mph	24 psf
40'-0"	7.25	1.475	24.9 psf	90 mph	25 psf
32'-0"	7.25	2.125	35.9 psf	110 mph	36 psf
40'-0"	9.25	1.710	36.9 psf	110 mph	37 psf
32'-0"	7.25	2.960	49.8 psf	130 mph	50 psf
40'-0"	9.25	2.405	51.9 psf	130 mph	52 psf
32'-0"	9.25	3.055	65.9 psf	150 mph	66 psf
40'-0"	11.25	2.625	68.9 psf	150 mph	69 psf

Table 2-1 Seismic Out-of-Plane Force

Naturally, soil conditions change vastly throughout the desired spectrum of wind speed regions; therefore, for simplicity, the study assumes a shallow foundation (continuous footings) for all panel scenarios. This report does not include design of the foundation, panel/foundation, and foundation/soil interfaces.

CHAPTER 3 - Reinforced Concrete

As with any building material, an understanding of the material properties and mechanics is important when determining the most appropriate design. Such an understanding clearly is important for reinforced concrete, which is used globally. The design of slender reinforced concrete wall panels is directly associated with the basic mechanics and properties of concrete and reinforcing steel. Thus, this chapter offers a broad overview of the materials of concrete and reinforcing steel. Hardened concrete is a brittle material with a tensile strength of approximately one-tenth of its compressive strength. Therefore, in structural concrete, reinforcing steel adds tensile load-carrying capacity and overall toughness. Although recent research and development have revealed other materials beside steel such as fiber reinforced plastic (FRP) to reinforce concrete successfully, the most common practice for tilt-up construction is the use of steel reinforcing. Therefore, this report focuses on billeted reinforcing steel.

3.1 Concrete

Concrete is a mixture of water, aggregates, and cementitious materials mainly cement. Water and portland cement, through the chemical reaction hydration, form a paste that binds the aggregates into a rock-like mass. Hydration can take place under water as well as when exposed to air. Meanwhile, aggregates consist of coarse and fine aggregates and are graded in size from sand to gravel (MacGregor, 2005).

3.1.1 Portland Cement

Most concretes are made with portland cement, a hydraulic cement manufactured mostly from lime and silica with small quantities of gypsum and iron oxide. The process begins with the crushing of the raw materials that begin as massive rocks. After a gradual crushing of the raw materials, appropriate testing determines physical and chemical make-up. Specifically, the material is combined chemically under extremely high temperatures of 3400 degrees Fahrenheit in a kiln, resulting in a substance called ‘clinker’. The clinker is then cooled and ground into the gray powder known as portland cement. In the United States, five basic types of portland cement are defined by the American Society for Testing and Materials, ASTM C 150 (Kosmatha, Kerkhoff, & Panarese, 2002):

- Type I, *regular portland cement* – general all-purpose cement used in reinforced concrete buildings, bridges, and anywhere that special properties of concrete are not desired
- Type II, *moderate sulfate resistance* – modified cement that can withstand moderate sulfate exposure and generates heat of hydration more slowly than Type I
- Type III, *high early strength* – a cement that develops in a week or less the strength that Type I develops at 28 days and therefore has a much higher heat of hydration to reach early strength
- Type IV, *low heat of hydration* – low heat cement develops strength at a slower rate than Type I and typically is used for large concrete structures such as dams
- Type V, *high sulfate resistance* – modified cement that can withstand exposure to high concentrations of sulfate and typically is used for footings and basement walls

Where certain concrete properties are desired, various admixtures can be added to the cement (How Portland Cement is Made, 1963).

3.1.2 Admixtures

Admixtures are added to concrete during or before mixing to attain certain qualities. The main reasons for using admixtures include the following: to reduce the cost of concrete construction, to achieve certain properties in concrete, and to maintain the quality of concrete during construction (Kosmatha, Kerkhoff, & Panarese, 2002). Admixtures typically used for tilt-up concrete buildings follow (Tilt-Up Concrete Association, 2004):

- air-entraining admixtures
- water-reducing admixtures
- accelerating admixtures
- mineral admixtures
- coloring admixtures

3.1.2.1 Air-Entraining Admixture

Air-entraining admixtures, as defined by ASTM C260, improve concrete's resistance to freezing and thawing. Such additives contain minute air bubbles that are distributed uniformly throughout and that provide relief spaces for the water to go to as the water freezes in the concrete. As the concrete temperature increases and the ice melts, the water moves out of the air voids, resulting in less cracking in the concrete (Tilt-Up Concrete Association, 2004) (Nelson, 2006).

3.1.2.2 Water-Reducing Admixtures

Water-reducing admixtures, as specified in ASTM C494, reduce the quantity of mixing water required to produce concrete of a certain slump, which reduces the water-to-cement ratio. As the water-to-cement ratio is reduced, the concrete compressive strength increases. Consequently, the 28-day compressive strength of a water-reduced concrete can be anywhere from 10 to 25 percent greater than for concrete without the water-reducing admixture. Typical water reducers reduce the water content by approximately 5 to 10 percent (Kosmatha, Kerkhoff, & Panarese, 2002).

3.1.2.3 Accelerating Admixtures

An accelerating admixture, as specified in ASTM C494, accelerates the rate of hydration and therefore the strength gain of the concrete. Accelerating admixtures allow for earlier finishing of the concrete in cold weather and the 28-day compressive strength can be reached at approximately seven days when using appropriate concreting methods. Notably, non-chloride accelerators are preferred in tilt-up panels so as not to corrode the reinforcing steel (Tilt-Up Concrete Association, 2004).

3.1.2.4 Mineral Admixtures (pozzolans)

Mineral admixtures, such as fly ash and silica fume, have recently become popular for economical reasons. In general, mineral admixtures replace anywhere from 10 to 50 percent of the portland cement. Fly ash is the most commonly used pozzolan and is specified by ASTM C618. Fly ash is the by-product of coal-burning power plants while silica fume is a by-product of induction arc furnaces in the production of silicon. Using pozzolans for partial replacement of the portland cement can improve the workability of the concrete by producing a more cementitious paste, but tends to cause a slower strength gain since the pozzolanic reaction generates less heat. Therefore, it may not always be desirable for tilt-up construction (Kosmatha, Kerkhoff, & Panarese, 2002).

3.1.2.5 Coloring Admixtures

Natural and synthetic materials color concrete for aesthetic reasons, and ASTM C979 is the specification for pigments for integrally colored concrete. Pigments can be in the form of powder or liquid (Good, 2006).

3.1.3 Water

ACI 318-08 *Building Code Requirements for Structural Concrete and Commentary* Section 3.4.1 specifies any potable water is considered suitable for making concrete. This is because any contaminants in the water can result in altered setting time of the mix or reduced concrete strength as well as corrosion of the steel reinforcement. Moreover, a relatively small amount of water is required to hydrate portland cement. An important characteristic of structural concrete is the water-to- cementitious material (w/cm) ratio, which is defined as the number of pounds of water used per pound of cement. The w/cm parameter can help determine the strength of concrete. Two opposing, yet desirable properties are affected by the w/cm: strength and workability. Concrete mixes that have low w/cm are stronger and more durable than those with high w/cm. Concrete must be both strong and workable, therefore requiring a careful balance of the w/cm. Most reinforced concretes have w/cm between about 0.4 and 0.7. A w/cm of 0.4 corresponds to a 28-day compressive strength of about 4,700 pounds per square inch. The w/cm affects the amount of shrinkage because high water content will decrease the volume taken by the aggregates. Excess water in the concrete also increases bleeding, which is the appearance of water on the fresh concrete surface. As the aggregates and cement particles settle, water used for mixing is forced to the surface (Concrete Bleeding, 1988). Excessive bleeding can lead to reduced strength near the surface, delayed finishing operations, and undesirable results if the concrete surface is finished before bleed water has evaporated (MacGregor, 2005).

3.1.4 Aggregates

Aggregates are generally divided into two groups: fine and coarse. Fine aggregates consist of natural sand or manufactured sand produced by crushing rock to particle sizes ranging up to ⅜ inch. If most of the particles are larger than ¼ inch, the aggregate is considered coarse aggregate, which may be gravel or crushed material, such as stone. Most concrete used in superstructures of building construction has a maximum aggregate size from ¾ inch to 1-1/2 inch. The largest desirable aggregate depends on the size and shape of the member to be made of concrete and on the spacing and location of reinforcing steel. Maximum size aggregates that can be used in reinforced concrete are specified in Section 3.3.2 of ACI 318-08. Nominal maximum size of coarse aggregate should not be larger than:

(a) 1/5 the narrowest dimension between sides of forms, nor
(b) 1/3 the depth of slabs, nor
(c) 3/4 the minimum clear spacing between individual reinforcing bars or wires

The maximum size aggregate commonly used in tilt-up panels is ¾ inch to 1 inch (Tilt-Up Concrete Association, 2004). The aggregate size contributes to the designation of normal weight concrete or light weight concrete. Normal weight concrete is the most commonly used in tilt-up panels, whereas weights equal to or less than 115 lb/ft^3 are considered light weight concretes. Although light weight concrete can be advantageous in that it provides a lighter wall panel, the reduced mechanical properties of the concrete and greater material cost are both undesirable. The aggregates used for these concretes are made from expanded shales of volcanic origin, fired clays, or slag (Nelson, 2006). Finally, aggregates make up 60 to 75 percent of the solid volume of concrete (Kosmatha, Kerkhoff, & Panarese, 2002).

3.1.5 Compressive Strength

The strength of a concrete and its stress-strain curve form the basis of all structural computations since strength of concrete is determined at 28 days by its compressive stress at failure, as defined in ASTM C39. To determine this compressive stress, a sample of the concrete is traditionally formed into a 12 inch long by 6 inch diameter cylinder. The cylinder is left to cure for 28 days and then placed in a compression test machine and loaded at a continuous rate until a well-defined fracture pattern occurs. The compressive stress at ultimate load is known as the 28-day strength and for design purposes labeled as f'_c in pounds per square inch (psi). Concrete will achieve about 60-75% of its design strength at 7 days; however it continues to gain strength for months or even years after placement (MacGregor, 2005).

Concrete is not a truly elastic material; however, it is reasonably elastic within the lower portion of its ultimate strength as shown on the stress-strain curve in Figure 3-1.

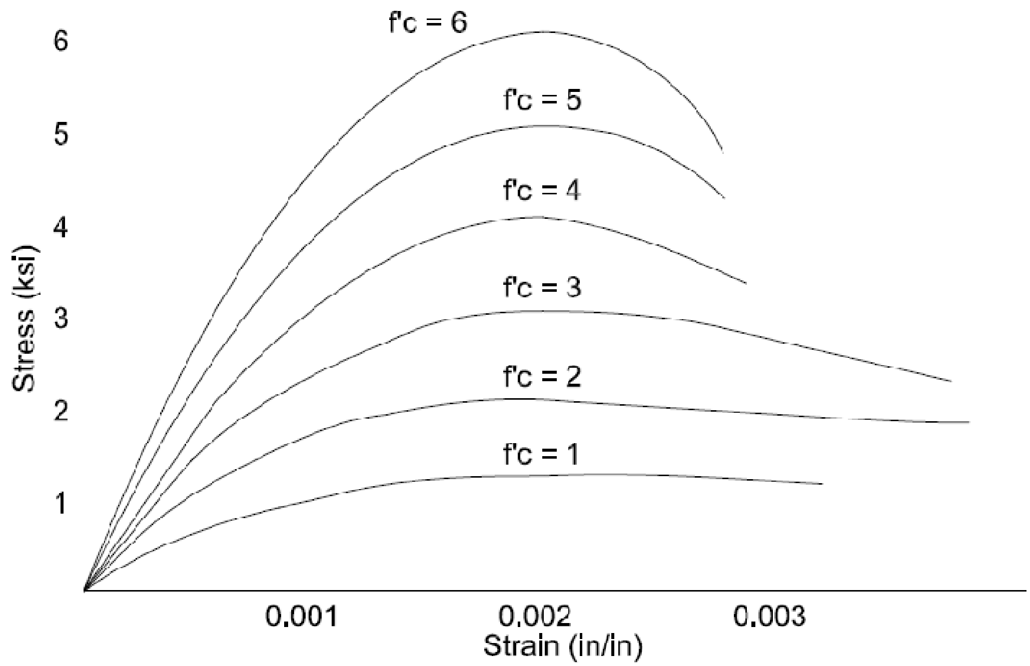

Figure 3-1 Stress-Strain Curve (Nelson, 2006)

For design, a range of about 40 to 45 percent of the compressive strength (f'_c) of concrete is treated as elastic (Nelson, 2006), and an approximation of the slope of the initial portion of the curve is defined as the modulus of elasticity. The value of the modulus of elasticity increases as the strength of the concrete increases as the following equation for normal weight concrete shows:

$$E_c = 57{,}000\sqrt{f'_c} \qquad \textbf{Equation 3-1}$$

where f'_c is the ultimate strength of the concrete in pounds per square inch (psi).

3.1.6 Tensile Strength

The tensile strength of concrete varies from about 8 to 15 percent of its compressive strength (Nelson, 2006). Although the tensile strength of concrete is usually neglected in design, the ultimate tensile strength is important when a reinforced concrete member is subject to flexural loading. ASTM C78 defines the flexural test in which a plain concrete beam spanning

24 inches and measuring 6 inches by 6 inches by 30 inches long is loaded in flexure at the third points. Once failure occurs due to cracking on the tensile face of the test beam, the modulus of rupture (f_r) measured in psi is calculated from the following formula where M is the maximum applied moment, b is the width of the beam, and h is the depth of the beam:

$$f_r = \frac{6M}{bh^2}$$ **Equation 3-2**

ACI 318-08 Section 9.5.2.3 provides the following formula for the modulus of rupture:

$$f_r = 7.5\lambda\sqrt{f'_c}$$ **Equation 3-3**

where λ = 1.0 for normal weight concrete as defined in Section 8.6.1 of ACI 318.

3.1.7 Volume Change

Design should consider three areas, as concrete will experience volume change due to shrinkage, creep, and thermal expansion.

3.1.7.1 Shrinkage

Any excess water that is free to evaporate as the concrete hardens can reduce the size of the member and is called shrinkage. Shrinkage only occurs to the concrete and not the reinforcement and it can produce internal stresses and tension cracking for areas with restrained ends. Consequently, horizontal reinforcing is provided for tilt-up panels to help resist cracking.

3.1.7.2 Creep

When concrete is subjected to loading it experiences deformation. After initial deformation occurs, the additional deformation is called creep or plastic flow. Creep will vary with both time and intensity of load. A tilt-up panel that experiences creep could lead to an increase in deflections and P-Δ effects. Although not included in the panel designs within this report, a minimum initial deflection of $l_c/400$ is recommended to account for creep and panel bowing (ACI Committee 551, 2009) (Nelson, 2006).

3.1.7.3 Thermal Expansion

The coefficient of thermal expansion (α) for normal weight concrete ranges from 5 to 7 x 10^{-6} per unit length per degree Fahrenheit. For steel, the coefficient is 6.5 x 10^{-6} per unit length per degree Fahrenheit. As concrete experiences changes due to temperature effects, steel will experience similar changes since both coefficients of thermal expansion are relatively close (MacGregor, 2005).

3.1.8 Steel Reinforcement

Concrete is strong in compression but weak in tension. Therefore, reinforcement such as steel is placed in the structural member at locations that experience tensile forces. Reinforcement used in tilt-up panels is typically ASTM A615, Grade 60 with minimum yield strength of 60,000 psi. Although reinforcing bars can be plain or deformed, deformed are most commonly used for reinforcement in tilt-up construction as it provides a better bond with the concrete. Reinforcement for tilt-up panels are typically #4, #5, or #6 bars (Tilt-Up Concrete Association, 2004). In some cases, the engineer will allow welded wire reinforcement to replace reinforcement bars. Finally, the modulus of elasticity (E_s) for all steel reinforcing is 29 x 10^6 psi.

CHAPTER 4 - Slender Reinforced Concrete Wall (Tilt-up) Design

Walls are designed to resist concentric or eccentric vertical axial loads, out-of-plane loads applied perpendicular to the face of the wall, and in-plane horizontal loads. Bending in walls results from lateral loads and eccentrically applied vertical axial loads. As the height of the wall increases, the need arises to account for the reduced axial load carrying capacity of the wall due to slenderness effects. The main effect of slenderness is the development of additional bending due to deflection (P-Δ effect). The common analytical model used for design is a load-bearing wall panel that spans vertically from the foundation or slab-on-grade to intermediate floor(s) and/or the roof (ACI Committee 551, 2009). Therefore, the tilt-up wall panels in this report span vertically from the slab-on-grade to the roof level. Due to various foundation depths that may occur due to freeze/thaw requirements, it is assumed that the tilt-up wall panels are braced at the floor slab which is a common construction practice (Robert Drysdale, 2008). Therefore, the additional panel length required for various foundation conditions is not considered.

4.1 Loads

The first step in designing any structural member is to determine the loads acting on the member. A load-bearing tilt-up panel experiences loading in three directions: vertically (axial), out-of-plane (lateral), and in-plane horizontally (shear). In particular, the load bearing tilt-up panels for this report are designed for gravity loads and lateral wind pressure. This lateral wind pressure can be the Main Wind-Force Resisting System (MWFRS) wind pressure which acts perpendicular to the wall surface and simultaneously perpendicular to the roof surface; or the C&C wind pressure which acts only perpendicular to the wall surface. Furthermore, all loads are based on a typical 24'-0" x 24'-0" bay as shown in Figure 2-2. Sample calculations are provided in Appendix A, while ASCE 7-05 and its commentary provide the necessary information to determine the loads acting on the panel.

4.1.1 Gravity Loads

The gravity loads acting on the load-bearing tilt-up panel are dead load, roof live load, and snow load. For this study, steel joists spaced 4'-0" on center result in six axial point loads along the top edge of the panel, resulting in a uniformly distributed load along the panel width at the mid-height of the panel. The panels highlighted along the longitudinal direction in Figure 2-2 will experience higher axial loads and therefore greater *P-Δ* effects because the joists frame into the panel. Meanwhile, the panels along the transverse direction would be designed for a much smaller, uniformly distributed load based on the tributary area concept and a single concentrated load resulting from the beam supporting the joists framing into the panel. This design is outside the scope of this report.

4.1.2 Dead Load

Dead load (D) is the weight of all the materials in the structure: self weight, weight of any fixed equipment, and weight of architectural features. The magnitudes of dead loads for various construction materials are in Table C3-1 of ASCE 7-05 provisions. Table 4-1 depicts the estimated dead load used for panel design in this report.

Bituminous Roofing =	1.5	psf
6" Rigid Insulation =	9	psf
1.5 22 Gauge Deck =	2	psf
Joists =	2.5	psf
M/E/P =	4	psf
Total =	19	psf
Use Dead Load =	20	psf

Table 4-1 Dead Load

The magnitudes provided in the standard are only an estimate; therefore any deviation is at the discretion of the engineer.

4.1.3 Roof Live Load

Roof live load (L_r) is for construction materials and workers and is based on the slope of the roof and the designated tributary area. Although ASCE 7-05 does allow for reduction in roof live loads with the minimum design load being 12 psf, no reduction was considered in this study. Instead, the roof is considered a flat, ordinary roof, and therefore the roof live load is 20 psf as defined in Table 4-1 of ASCE 7-05.

4.1.4 Snow Load

ASCE 7-05 contains maximum measured ground snow loads and ground snow loads with a 2% annual probability of being exceeded (50-year mean recurrence interval). The ground snow load (p_g), given in Table 7-1 of ASCE 7-05, determines design ground snow load. Also, ASCE 7-05 defines the flat roof snow load (p_f), on a roof with a slope equal to or less than 5° from the following equation:

$$p_f = 0.7 C_e C_t I p_g \qquad \textbf{Equation 4-1}$$

The following coefficient values were given in order to establish a reasonable load for designs conducted throughout the study. Exposure factor (C_e) accounts for the slow down effects of site terrain. A factor of 1.0 is based on exposure category C and assumes a partially exposed roof. Thermal factor (C_t) accounts for the thermal condition of the roof and how much heat will pass through the roof and melt snow off the roof. A factor of 1.0 is used based on not meeting the conditions outlined in Table 7-3 of ASCE 7-05. A building category II with an importance factor (I) of 1.0 is used to relate the design load to the consequences of failure.

Given that snowdrifts can occur on roofs in the shadow of higher roofs, or in this case, the parapet, ASCE 7-05 defines snowdrift loads as a surcharge load that is imposed on the balanced snow load, p_f. Snow drift applies only if the ratio $h_c/h_b \geq 0.2$. Appendix A of this report provides load calculations for a roof snow load (p_f) based on a ground snow load (p_g) of 20 psf. This study considers a maximum flat roof snow load without drifting, including rain-on-snow to be 20 psf; equal to the roof live load value.

4.1.5 Lateral Loads

Lateral loads act horizontally for Main Lateral Force Resisting System (MLFRS) and C&C and are known as out-of-plane loads. In-plane loads can also occur for the MLFRS. Wind pressures or equivalent static seismic pressure based on seismic response accelerations are generally applied to the tilt-up walls as uniformly distributed lateral loads. For this report, lateral loading pressures due to wind govern. ASCE 7-05 provisions specify three procedures for determining design wind loads: Simplified Procedure (Method 1), Analytical Procedure (Method

2), and Wind Tunnel Procedure (Method 3). Each procedure has limitations and/or complications regarding suitability for design; however, for this report, the Analytical Procedure (Method 2) will be used for determining the design wind pressures for MLFRS and C&C.

4.1.5.1 Wind Pressure

The Analytical Procedure outlined in Section 6.5 of ASCE 7-05 provides wind pressures for the design of C&C and for the MWFRS. To determine design wind loads in accordance with the Analytical Procedure requires the following conditions of ASCE 7-05 Section 6.5.1:

- the building or other structure must be regular-shaped, having no unusual geometrical irregularity
- the building or other structure must not experience across wind loading, vortex shedding, instability due to galloping or flutter; or be on a site location susceptible to special wind channeling effects

For the MWFRS, this procedure involves determining wind velocity at any height (q_z), wind directionality (K_d), gust effect factor (G), and pressure coefficients (GC_{pf} and GC_{pi}). The procedure also accounts for topographic features (K_{zt}), different wind exposures, and geometric configuration.

The pressure coefficients reflect the loading on each building surface due to wind direction, the transverse direction in this case. For either the MWFRS or C&C, the velocity pressure (q_h) at mean roof height, h is defined by the following equation:

$$q_h = 0.00256 K_z K_{zt} K_d V^2 I \ (lb/ft^2) \qquad \textbf{Equation 4-2}$$

The basic wind speeds (V) in this report are 90 miles per hour (mph), 110 mph, 130 mph, and 150 mph, and they correspond to 3-second gust speeds at 33 feet (10 m) above ground for exposure category C, which ASCE 7-05 defines as open terrain with scattered obstructions of heights generally less than 30 feet. Exposure category C is appropriate for the higher wind speeds, which correspond to hurricane prone regions. The topographic factor (K_{zt}) is a multiplier used to account for higher wind speeds that can be generated due to an isolated hill or escarpment; in this case, the factor of 1.0 is used assuming generally flat ground. Next, ASCE 7-05 contains a single gust effect factor of 0.85 for rigid buildings. A building is considered rigid when the building structure's frequency, in Hertz (Hz), is greater than 1.0. The structure used for this report is said to be rigid as determined in Appendix A. A building structure can be classified as open, partially enclosed, or enclosed, all of which are defined in Section 6.2 of ASCE 7-05. This classification of a structure determines the appropriate internal pressure coefficient in Figure 6-5 of ASCE 7-05. This study assumes an enclosure classification of partially enclosed, resulting in higher internal wind pressures than for the enclosed classification and is common for warehouses with large openings on one side.

The lateral pressure applied to the panel for design is the wind pressure determined for C&C. However, the lateral pressures from the MWFRS should be compared to lateral pressures determined by the C&C method. Appendix B provides the MWFRS lateral pressures that would be used for design. More times than not, the lateral pressure determined by C&C will be larger due to the smaller effective area receiving the wind pressure therefore, a higher average wind pressure occurs. ASCE 7-05 Section 6.5.12.4.1 provides the design wind pressures on C&C elements of low-rise buildings.

$$p = q_h[(GC_p) - (GC_{pi})]\ (lb/ft^2) \qquad \textbf{Equation 4-3}$$

Design wind pressures can be negative or positive, and the designation is determined by the net pressure resulting between the internal and external pressures. For clarity, Figure 4-1 shows the net pressure determination for a windward wall case using partially enclosed classification for MWFRS.

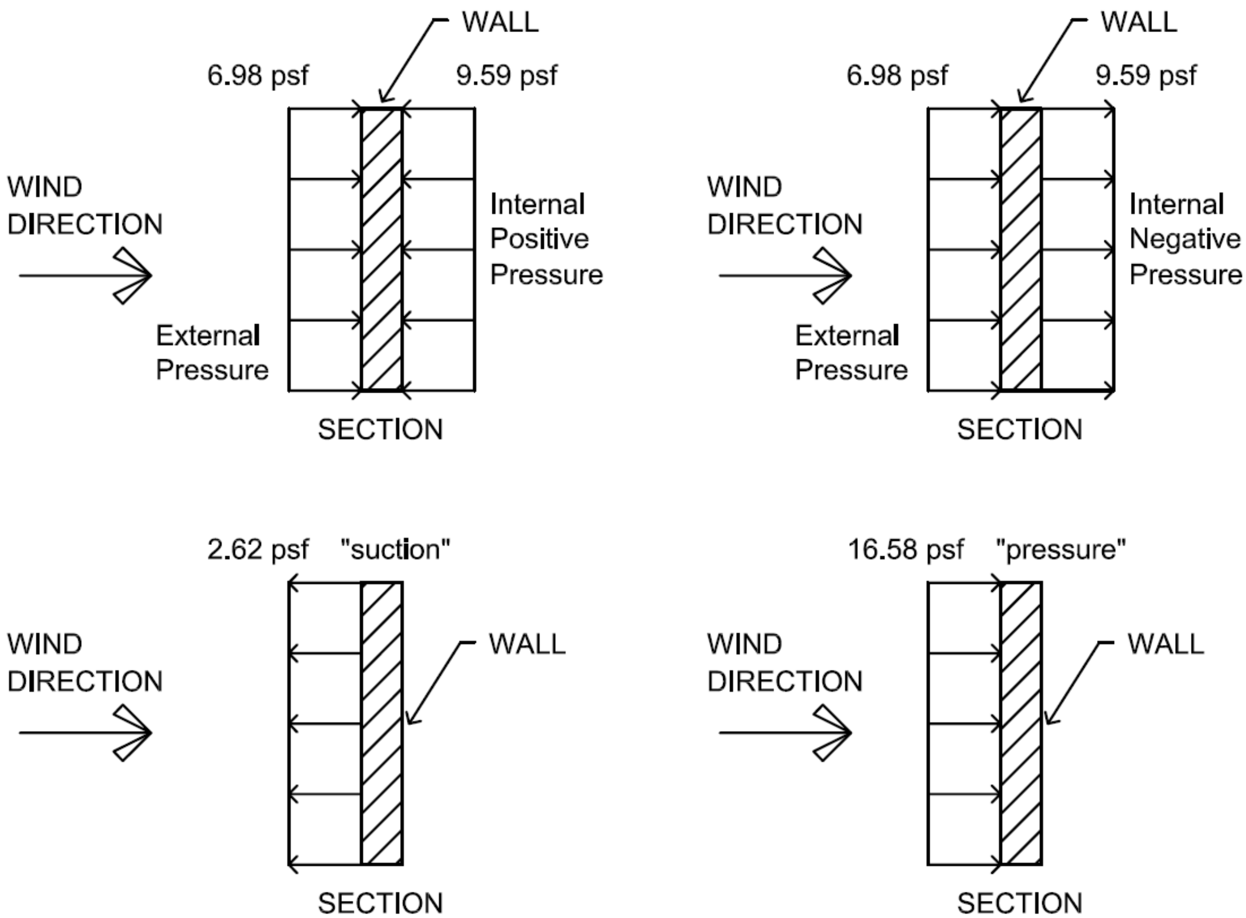

Figure 4-1 Net MWFRS Wind Pressures for a Windward Wall Case

For the partially enclosed classification, both positive and negative internal pressures must be considered, and the net pressures of -2.62 psf and 16.58 psf can be verified in Appendix B under Zone 1 (windward). Notably, a windward case is provided for clarity, whereas the leeward case usually produces the more critical wind pressure. A negative net pressure, or suction, acts away from the element while a positive net pressure, or pressure, acts towards the element. As Figure 4-2 shows, suction is the more critical situation since it will increase the moment at mid-height of the panel with the eccentric (e_{cc}) axial load.

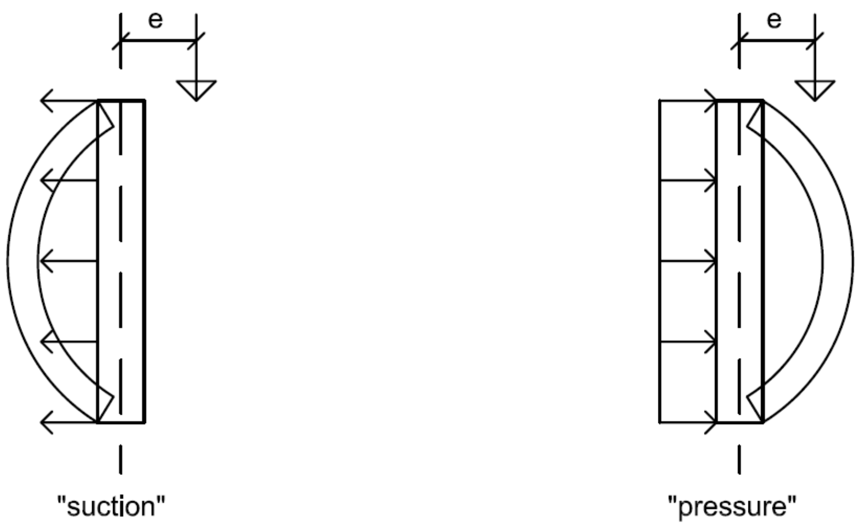

Figure 4-2 Applied Lateral Wind Pressure

Figure 6-10 of ASCE 7-05 provides the appropriate pressure coefficients for the MWFRS. For a roof angle (θ) 0 to 5°, Appendix A shows the external pressure coefficients (GC_{pf}). ASCE 7-05 Section 6.5.12.2.2 provides the design wind pressure for the MWFRS of low-rise buildings where the mean roof height (h) does not exceed 60 feet and does not exceed the least horizontal dimension.

$$p = q_h[(GC_{pf}) - (GC_{pi})]\ (lb/ft^2)$$ **Equation 4-4**

The MWFRS lateral pressures are applied perpendicular to the walls and roof. These forces are transferred to the diaphragm and then into the tilt-up panels parallel to the applied wind direction as in-plane forces; however, this analysis is outside the scope of this report. The MWFRS wind pressures are less than the C&C wind pressures and therefore, do not govern the design.

4.2 Alternative Design of Slender Walls – ACI 318-08, Section 14.8

The design concepts and code limitations for slender walls were tested and confirmed by the American Concrete Institute – Structural Engineers Association of Southern California (ACI-SEAOSC) Task Committee on Slender Walls, 1980 to 1982. The report of the task committee includes design requirements and procedures, analysis methods, and load/deflection relations (Athey, 1982). The alternative design of slender walls was introduced in 1999 in the ACI 318 code, which was based on the provisions of the 1997 Uniform Building Code (UBC). According to Section 14.8.2 of ACI 318-08, walls designed by the alternative design method must satisfy the following limitations:

- the wall panel shall be simply supported, axially loaded subjected to an out-of-plane uniform lateral load, with maximum moments and deflections occurring at midspan
- constant cross-section over the height of the panel
- cross-section of wall must be tension-controlled
- reinforcement must provide a design moment strength (ΦM_n) greater than or equal to the cracking moment (M_{cr})
- concentrated gravity loads applied to the wall above the design flexural section must be distributed over a width equal to the lesser of the bearing width plus a width on each side that increases at a slope of 2 vertical to 1 horizontal down to the design flexural section or the spacing of the concentrated loads but not extending beyond the edges of the wall panel
- vertical stress P_u/A_g at the mid-height section shall not exceed $0.06f'_c$

When one or more of the above limitations is not met, the wall design must meet the provisions outlined in Section 14.4 of ACI 318-08 in which walls subject to axial load or combined flexure and axial load be designed as compression members, requiring tied vertical reinforcement.

4.2.1 Load Cases

The Strength Design Method requires that structural elements at any section have design strengths at least equal to the required strengths calculated by the code-specified factored load combinations as defined in Section 9.2 of ACI 318-08. The following load combinations determine the required strength, *U*:

	$U = 1.4D$	(ACI Equation 9-1)
	$U = 1.2D + 0.5(L_r \text{ or } S)$	(ACI Equation 9-2)
Load Case 1	$U = 1.2D + 1.6(L_r \text{ or } S) + 0.8W$	(ACI Equation 9-3)
Load Case 2	$U = 1.2D + 1.6W + 0.5(L_r \text{ or } S)$	(ACI Equation 9-4)
	$U = 1.2D + 1.0E + 0.2S$	(ACI Equation 9-5)

Load Case 3 $U = 0.9D + 1.6W$ (ACI Equation 9-6)

$U = 0.9D + 1.0E$ (ACI Equation 9-7)

For this parametric study, Load Case 1 determines the greatest applied force due to gravity loads. Load Case 2 and Load Case 3 determine the largest out-of-plane loading applied to wall panel.

4.2.2 Design Moment Strength

The following equation shows the design moment strength of the panel (φM_n):

$$\varphi M_n = \varphi A_{se} f_y (d - \frac{a}{2})$$ **Equation 4-5**

The strength reduction factor (φ) is 0.90 for tension-controlled sections per Section 9.3.2.1 of ACI 318-08, which is required for the alternative design method. The strength reduction factor reflects:

- probability of under-strength members due to variations in material strengths
- inaccuracies in design equations
- degree of ductility and required reliability of the member under the load effects considered
- importance of the member in the structure

Figure 4-3 shows that for a tension-controlled section, the strength reduction factor will be $\varphi = 0.90$.

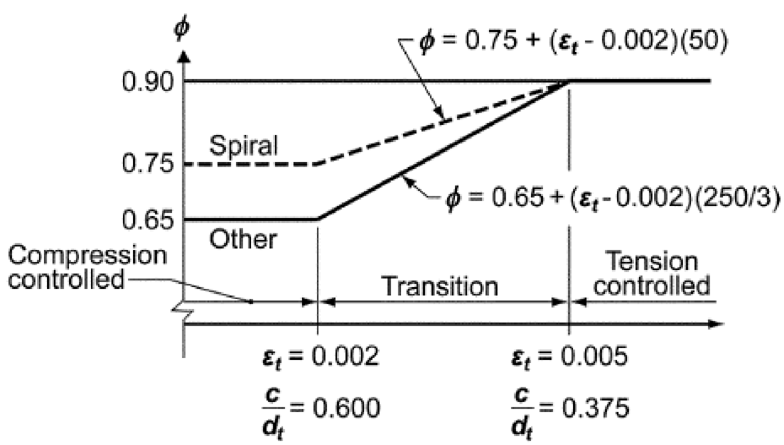

Figure 4-3 Determination of Strength Reduction Factor, φ (ACI Committee 318, 2008)

Two criteria can verify that the section is indeed tension-controlled:

$$\frac{c}{d_t} \leq 0.375 \qquad \text{or} \qquad \varepsilon_t \geq 0.005$$

Figure 4-4 shows, given strain compatibility principles, the nominal flexural strength of a member is reached when the strain in the extreme fiber in compression reaches the assumed strain limit of 0.003 for concrete.

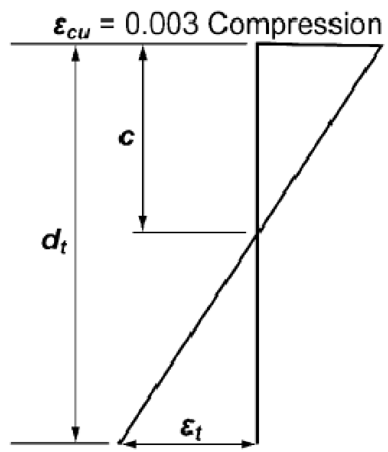

Figure 4-4 Strain Distribution (ACI Committee 318, 2008)

The effective area of reinforcement is defined by the following equation:

$$A_{se} = A_s + \frac{P_{um}}{f_y}\frac{h}{2d} \qquad \textbf{Equation 4-6}$$

The first term is the area of reinforcement (A_s) due to reinforcement placed in the cross-section. The second term contains a compressive force due to the factored applied axial loads (P_{um}) which includes the contributing panel self-weight. The panel self-weight above mid-height is critical because it adds additional axial load at the critical midspan section for a simply supported member. Notably, a single layer of reinforcement and a double layer of reinforcement differently affect the small gain of bending resistance due to the applied axial loads. Before ACI 318-08, the effective area of reinforcement overestimated the contribution of axial load when using two layers of reinforcement; therefore, the ratio *h/2d* has since been added (PCA, 2008).

This ratio will be very close to 1.0 for a single layer of reinforcement where *h* is the panel thickness, and *d* is the distance from the top of the cross-section to the centroid of the tension reinforcement steel. For a double layer of reinforcement, the additional bending resistance reduces.

ACI 318-08 allows an equivalent rectangular compressive stress block to replace the more exact parabolic stress distribution. Figure 4-5 shows such a stress block for a simply-supported member loaded from the top with a downward force.

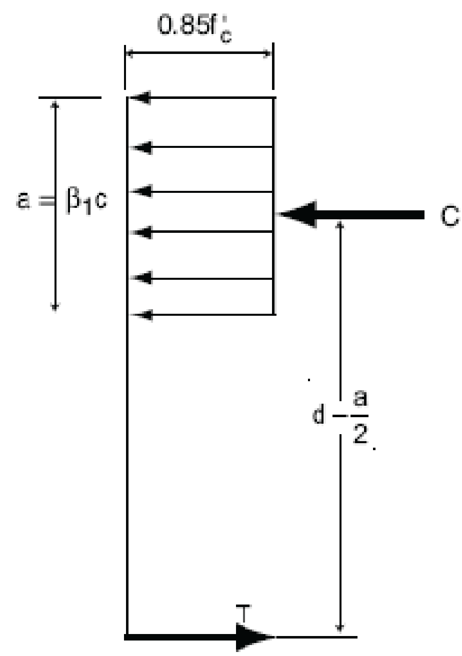

Figure 4-5 Equivalent Rectangular Stress Block (PCA, 2008)

To easily calculate the nominal moment strength, requires a few simple assumptions. At ultimate strength, an average concrete stress of intensity $0.85f'_c$ is uniformly distributed across the equivalent compression zone bounded by the edges of the cross-section and a straight line located parallel to the neutral axis at depth $a = \beta_1 c$ from the fiber of the maximum compressive strain (Mattock, Kriz, & Hognestad, 1961). Determined experimentally, the factor β_1 is a ratio of average stress to maximum stress and is taken as 0.85 for concrete compressive strength of 4,000 psi. Moreover, ACI 318-08 Section 10.2.7.2 defines *c* as the distance from the fiber of maximum strain in compression, to the neutral axis. However, tensile strength of the concrete is typically neglected. Therefore, the total tensile force is taken by the reinforcement steel and defined by the following:

$$T = A_{se} f_y \qquad \textbf{Equation 4-7}$$

The total compressive force (C) is the volume of the equivalent rectangular stress block multiplied by the average concrete stress:

$$C = 0.85 f'_c ab \qquad \textbf{Equation 4-8}$$

The compression force and the tension force must be equal to maintain equilibrium at the cross-section and allow for the determination of the depth of the stress block:

$$0.85 f'_c ab = A_{se} f_y \qquad \textbf{Equation 4-9}$$

$$a = \frac{A_{se} f_y}{0.85 f'_c b} \qquad \textbf{Equation 4-10}$$

Once this expression is found, the distance (or moment arm) between the centroid of the tension and compression force equals ($d - a/2$). To satisfy strength design requirements, the nominal moment strength multiplied by the strength reduction factor, must be greater than or equal to the factored applied moment discussed in Section 4.2.4.

4.2.2.1 Cracking Moment

To prevent sudden failure at the point cracking first occurs, ACI 318-08 Section 14.8.2.4 requires the cross-section of the slender wall have a nominal moment strength greater than or equal to the cracking moment, M_{cr}, defined in Section 9.5.2.3 of ACI 318-08:

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad \textbf{Equation 4-11}$$

(ACI Equation 9-9)

where I_g is the gross moment of inertia of the cross-section, and y_t is the distance from the centroid of the cross-section to the extreme fiber in tension. The modulus of rupture of the concrete is defined by Equation 3.3.

4.2.2.2 Flexural Minimum Reinforcement

When a member has a factored axial compressive load less than $0.10 f'_c A_g$, Section 10.5 of ACI 318-08 specifies that the minimum steel reinforcement ratio, ρ_{min}, not be less than:

$$\rho_{min} = \frac{3\sqrt{f'_c}}{f_y} \qquad \textbf{Equation 4-12}$$

or:

$$\rho_{min} = \frac{200}{f_y} \qquad \textbf{Equation 4-13}$$

Then the governing ρ_{min} is compared to the actual reinforcement ratio, ρ_{actual}, for the cross-section. Equation 4-13 will govern for concrete compressive strengths of 4,000 psi or less.

4.2.3 Minimum Vertical Reinforcement

The minimum ratio of vertical reinforcement area to gross concrete area (ρ_l) shall be as stated in Section 14.3.2 of ACI 318-08:

(a) 0.0012 for deformed bars not larger than No. 5 with yield strength, f_y, not less than 60,000 psi
(b) 0.0015 for other deformed bars
(c) 0.0012 for welded wire reinforcement not larger than W31 or D31 (cross-sectional area of wire is 0.31 in^2)

The minimum vertical reinforcement ratio is compared to the actual vertical reinforcement ratio of the section. The difference between the minimum flexural reinforcement ratio and the minimum vertical reinforcement ratio is the overall section depth. The minimum flexural reinforcement ratio is found by considering the depth of the tensile reinforcement.

4.2.4 Applied Ultimate Moment

For the alternative design of slender walls, the tilt-up wall panel is defined as simply supported, which means when the panel is subjected to a uniform lateral load and an axial load, the maximum moment occurs at mid-span. Next, the bending moment caused by out-of-plane load, wind pressure, is greater than the bending moment caused by eccentric axial loads due to lightly loaded roof. Additionally, the bending moments due to applied loads can be magnified by the effect of the axial loads acting on the deflected shape. This increase in moment is often referred to as the *P-Δ* effect (ACI Committee 551, 2009). The maximum bending moment can be split into two components: primary moment due to applied loads, and secondary moment due to *P-Δ* effect. This is illustrated in Figure 4-6 (ACI Committee 551, 2009):

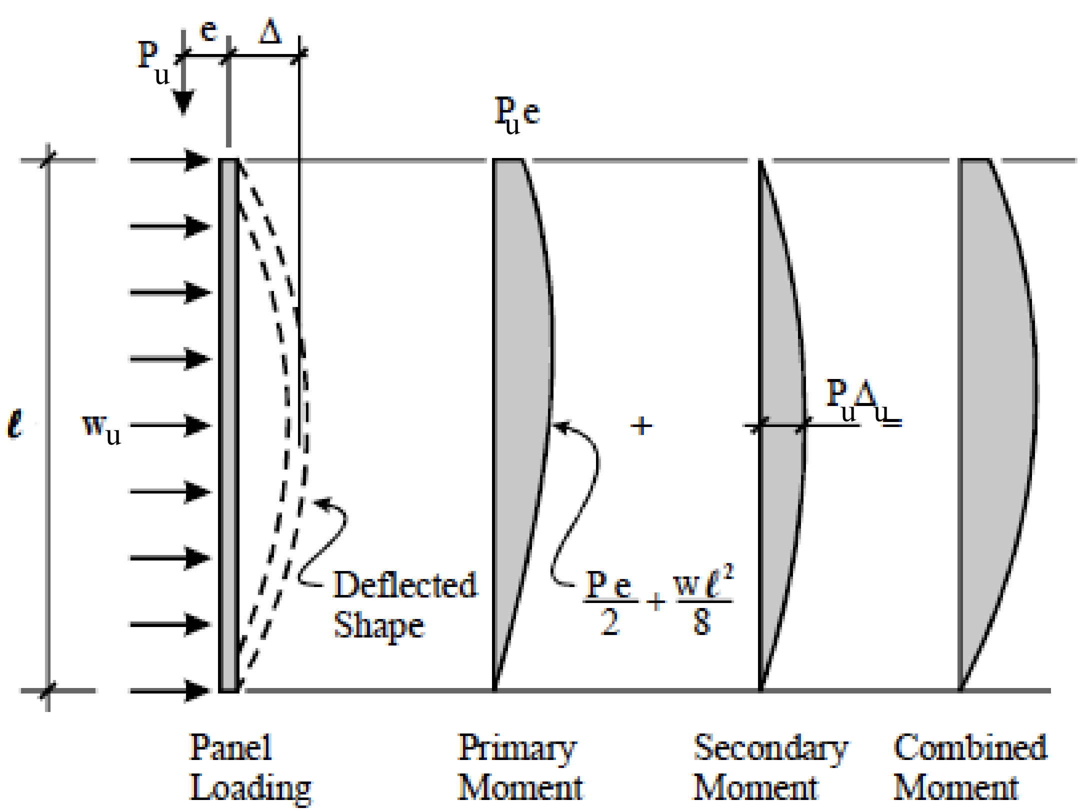

Figure 4-6 Panel Analysis (ACI Committee 551, 2009)

The following calculation generates the maximum factored applied primary moment at midspan due to lateral and eccentric axial loads, not including *P-Δ* effects:

$$M_{ua} = \frac{w_u l_c^{\,2}}{8} + \frac{P_{ua} e_{cc}}{2}$$ **Equation 4-14**

where w_u is the factored uniform lateral load, l_c is the unbraced length, P_{ua} is the factored applied axial load, and e_{cc} is the eccentricity of the factored applied axial load on the panel. Essentially, the primary moment developed in the tilt-up panel occurs due to the following loads (ACI Committee 551, 2009):

- eccentric axial loads
- out-of-plane lateral loads
- initial lateral deflections

The maximum combined moment at mid-span of the panel is the primary moment plus the secondary moment. To account for the axial loads acting on the deflected shape, ACI 318-08 Section 14.8.3 provides the following relationship:

$$M_u = M_{ua} + P_u \Delta_u$$ **Equation 4-15**
(ACI Equation 14-4)

Two approaches to determine the maximum combined moment of the section are direct method and an iterative approach. The direct calculation is based on the moment magnifier method whereas an iterative process calculates the incremental increases in moment and deflection due to *P-Δ* effects until convergence or equilibrium is reached.

4.2.4.1 Moment Magnifier Method

The alternative design of slender walls in Section 14.8 of ACI 318-08 uses the moment magnifier method to determine the maximum combined moment of the wall element using the following equation:

$$M_u = \frac{M_{ua}}{1 - \dfrac{5 P_u \ell_c^2}{(0.75)48 E_c I_{cr}}}$$ **Equation 4-16**

(ACI Equation 14-6)

The moment magnifier method used for slender wall elements is very similar to that used to account for slenderness effects in compression members or columns. For tilt-up panels, the panel is considered to be simply supported with uniform lateral load acting on the element. Given the limitations of the slender wall design, maximum moment (M_{max}) and deflection (Δ_{max}) will occur at mid-height and are defined as follows:

$$M_u = \frac{w_u \ell_c^2}{8} \qquad \textbf{Equation 4-17}$$

$$\Delta_u = \frac{5 w_u \ell_c^2}{384 E_c I_e} \qquad \textbf{Equation 4-18}$$

Substituting Equation 4-17 into Equation 4-18 gives the relation between maximum moment and deflection:

$$\Delta_u = \frac{5 M_u \ell_c^2}{48 E_c I_e} \qquad \textbf{Equation 4-19}$$

Furthermore, substituting Equation 4-19 into Equation 4-15 and solving for the maximum moment yields:

$$M_u = \frac{M_{ua}}{1 - \frac{5 P_u \ell_c^2}{48 E_c I_e}} \qquad \textbf{Equation 4-20}$$

Test results published in *Test Report on Slender Walls* and further research by the 2005 *Slender Wall Task Group*, show that the section stiffness of $E_c I_e$ can be taken as the cracked section stiffness $E_c I_{cr}$ (ACI Committee 551, 2009). Thus, the cracked moment of inertia is defined as:

$$I_{cr} = n(A_{se})(d - c)^2 + \frac{\ell_w c^3}{3}$$ **Equation 4-21**

For a cracked section, the concrete is in compression and reinforcing steel is in tension. In ultimate strength design, the section is assumed to have cracked, and therefore, the tensile force in the concrete is transferred to the steel. The factor *n* is a dimensionless ratio of the moduli of elasticity of steel to concrete and is defined as follows:

$$n = \frac{E_s}{E_c}$$ **Equation 4-22**

The concrete section stiffness is assumed to be constant over the entire height of the panel (ACI Committee 551, 2009). Lastly, the reduction factor of 0.75 in Equation 4-16, accounts for the variability in the stiffness of the section due to material properties and construction.

4.2.4.2 Iteration Method

The second approach in determining the maximum moment and deflection at mid-height of the panel is by a simple iterative process whereby the initially calculated applied factored primary moment from Equation 4-14 determines the deflection from the following expression:

$$\Delta_u = \frac{5 M_u \ell_c^2}{(0.75) 48 E_c I_{cr}}$$ **Equation 4-23**

The maximum moment due to P-Δ effects is determined when the necessary values substitute into Equation 4-15. This process is repeated until both the maximum moment and deflection converge.

4.2.5 Service Load Deflection

In addition to satisfying the strength requirement for combined flexure and axial load at midspan of the panel, engineers must also satisfy the service load deflection requirement such that maximum deflection due to service loads cannot exceed $\ell_c/150$ as defined in Section 14.8.4 of ACI 318-08.

Before the ACI-SEAOSC *Task Committee on Slender Walls*, building codes limited the ratio of height to thickness (h/t) of slender walls (Athey, 1982). However, the committee's test results showed that despite the h/t ratios, the wall panels had more than enough strength for lateral loads while experiencing severe deflections. Nevertheless, very large deflections could result in a panel that is too flexible and perhaps permanently deforms. Therefore, SEAOSC developed service level deflection equations for the load-deflection curve based on the original test results.

$$\Delta_s = \frac{5M_a \ell_c^2}{48E_c I_g} \text{ for } M_a < M_{cr} \qquad \textbf{Equation 4-24}$$

$$\Delta_s = \Delta_{cr} + \left(\frac{M_a - M_{cr}}{M_n - M_{cr}}\right)(\Delta_n - \Delta_{cr}) \text{ for } M_a > M_{cr} \qquad \textbf{Equation 4-25}$$

Based on a cracking moment with a modulus of rupture as follows:

$$M_{cr} = 5\sqrt{f'_c} \qquad \textbf{Equation 4-26}$$

These equations were the basis for the slender wall provisions first incorporated into the 1987 Supplement to the Uniform Building Code (UBC) (Lawson, 2007).

Prior to ACI 318-08, service load deflections for wall panels were calculated using the effective moment of inertia (also known as Branson's equation) defined in Section 9.5.2.3.

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad \textbf{Equation 4-27}$$

While the traditional value for modulus of rupture in ACI 318, defined by Equation 3-3, remains unchanged, the 2008 edition of ACI 318 uses revised deflection equations to better reflect the original test data for slender walls.

$$\Delta_s = \left(\frac{M_a}{M_{cr}}\right)\Delta_{cr} \text{ for } M_a \le (2/3)M_{cr}$$ **Equation 4-28**

$$\Delta_s = (2/3)\Delta_{cr} + \left(\frac{M_a - (2/3)M_{cr}}{M_n - (2/3)M_{cr}}\right)\left(\Delta_n - (2/3)\Delta_{cr}\right) \text{ for } M_a > (2/3)M_{cr}$$ **Equation 4-29**

where

$$\Delta_{cr} = \frac{5M_{cr}\ell_c^{\,2}}{48E_cI_g}$$ **Equation 4-30**

$$\Delta_n = \frac{5M_n\ell_c^{\,2}}{48E_cI_{cr}}$$ **Equation 4-31**

The use of the (2/3) factor reflects the difference in cracking moment based on the modulus of rupture defined in Equation 4-26 and the traditional value used in ACI 318. Ultimately, ACI 318-08 revisions conservatively underestimate the cracking moment by 16% on average (Lawson, 2007). Finally, the maximum moment due to service loads (M_a) is obtained by iteration of deflections, similar to the process defined in Section 4.2.4.2.

4.2.6 Minimum Horizontal Reinforcing

To ensure minimum ductility, the minimum ratio of horizontal reinforcement area to gross concrete area (ρ_t) shall be as stated in Section 14.3.3 of ACI 318-08:

(a) 0.0020 for deformed bars not larger than No. 5 with yield strength, f_y, not less than 60,000 psi

(b) 0.0025 for other deformed bars

(c) 0.0020 for welded wire reinforcement not larger than W31 or D31

The reinforcing accounts for temperature and shrinkage, and for panels used as a shear wall, shear reinforcing is typically required. However, such reinforcing is outside the scope for this report.

CHAPTER 5 - Solid Panel Design Example

For clarity, this chapter provides the design process for the vertical reinforcement for a solid panel using the alternative design of slender walls. Furthermore, the analysis of vertical reinforcement in tilt-up panels is a trial and error process for calculating the panel moment strength based on an assumed effective area of tension reinforcement (ACI Committee 551, 2009).

5.1 Panel Design Properties

Panel width	= 24'-0"	f'_c	= 4,000 psi
Panel height	= 34'-0"	f_y	= 60,000 psi
Unbraced length	= 32'-0"	γ_c	= 150 pcf (normal weight concrete)
Parapet	= 2'-0"	f_r	= 474 psi (Equation 3-3)
"d" (tensile steel)	= 3. 625"	E_c	= 3605 ksi (Equation 3-1)
		n	= 8.044 (Equation 4-22)

Figure 5-1 illustrates the geometry of the panel with no openings. Conservatively, wind forces are neglected in this study. These forces would decrease the moment at mid-height of the panel.

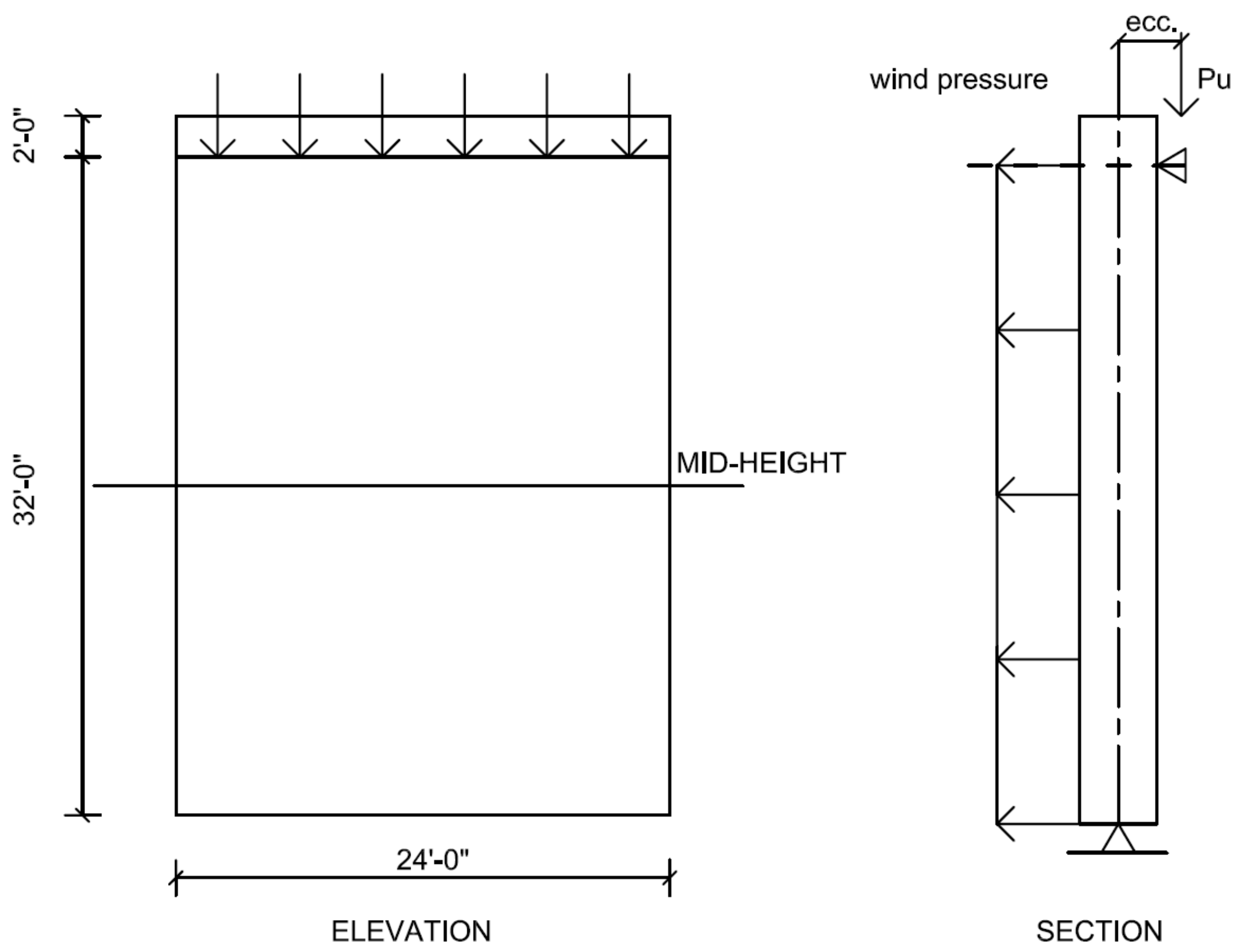

Figure 5-1 Solid Panel Geometry

Figure 5-2 illustrates the cross section of the panel.

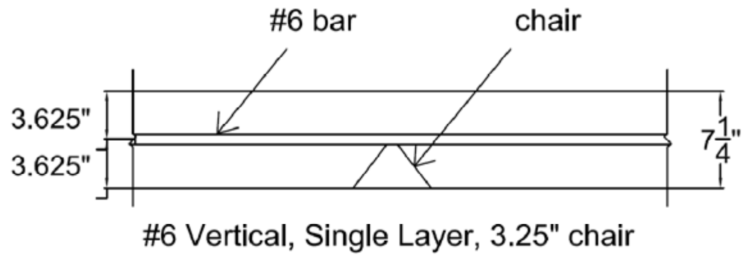

Figure 5-2 Solid Panel Cross Section

The depth of the reinforcement is labeled to reflect the heights achieved in construction. The chair heights are increments of ¼''; and depending on the vertical bar size, two varying depth's can result. Therefore, it is conservative to use the smaller distance for design calculations. Finally, the design should consider 'suction' and 'pressure' load due to wind.

The panel has eccentric axial load from six roof joists assumed to bear on the face of the panel in addition to the wind pressure (lateral load); accordingly, the following loading can be verified in Appendix A:

P_{DL} = 5.76k (20psf)

$P_{(Lr\ or\ S)}$ = 7.69k(20psf)

e_{cc} = 5.125"

Wind = 24 psf (90mph wind speed)

This report assumes an eccentricity of ½ the panel thickness plus 1.5", suggesting a minimum eccentricity of ½ the panel thickness (ACI Committee 551, 2009). Ultimately, the bearing condition is determined based on the roof framing design. Figure 5-3 shows a typical joist to panel connection.

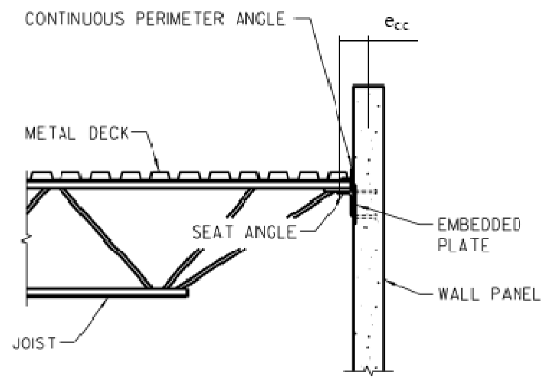

Figure 5-3 Joist to Panel Connection

5.2 Load Case 1 (C&C)

This section addresses the first of the three load combinations discussed in Section 4.2.1:

$$1.2D + 1.6(L_r \text{ or } S) + 0.8W \qquad \textbf{Equation 5-1}$$

The ultimate applied axial load (excluding self-weight) derives from the load case:

$$P_{ua} = 1.2(P_{DL}) + 1.6(P_{(Lr \text{ or } S)}) + 0.8W \qquad \textbf{Equation 5-2}$$

$$P_{ua} = 1.2(5.76k) + 1.6(7.69k)$$

$$P_{ua} = 19.2k$$

The design should account for the effect of panel self-weight since it contributes significantly to the *P-Δ* moment, although only the weight above the mid-height of the panel is considered since the maximum moment is said to act at the mid-span. Next, a panel thickness of 7.25" (l_c/53) was derived by trial and error, while the suggested minimum thickness is (1/50) of the unbraced length for a single mat of reinforcement or (1/65) of the unbraced length for a double mat (ACI Committee 551, 2009). The ultimate axial load including self-weight above mid-span is:

$$P_{um} = P_{ua} + 1.2(\text{panel weight above CL}) \qquad \textbf{Equation 5-3}$$

$$panel\ weight\ above\ CL = \frac{\left(\frac{7.25in}{\frac{12in}{1ft}}\right)(24ft)(150pcf)\left[\left(\frac{32ft}{2}\right)+2ft\right]}{1000k/lb} = 39.2k$$

$$P_{um} = 19.2k + 1.2(39.2k) = 66.2k$$

Determine the factored wind load from C&C:

$$w_u = 0.8(net\ wind\ pressure)(panel\ width)$$ `**Equation 5-4**

$$w_u = 0.8(24psf)(24ft)$$

$$w_u = 461plf = 0.461klf$$

5.2.1 Vertical Stresses

Section 14.8.2.6 of ACI 318-08 states that the vertical stress at the mid-height of the panel shall not exceed $0.06f'_c$. If the vertical stresses exceed this value an increase in the concrete compressive strength or an increase in the panel thickness may be appropriate. If the vertical stresses of the section do not meet the criteria, Section 14.8 is no longer a valid design approach; therefore, the section could be treated as a column with applied moments. Check the stresses:

$$\frac{P_u}{A_g} = \frac{P_u}{width \times thickness}$$ **Equation 5-5**

$$\frac{P_u}{A_g} = \frac{66.2k(1000lb/k)}{(24ft \times 12\frac{in}{ft})7.25in} = 31.7\ psi$$

$$31.7\ psi \leq (0.06x4000psi) = 240\ psi$$

5.2.2 Design Moment Strength

Section 14.8.2.4 of ACI 318-08 states that vertical reinforcement shall provide design strength greater than the ultimate applied moment:

$$\varphi M_n \geq M_{cr}$$ **Equation 5-6**
(ACI Equation 14-2)

Next, define the cracking moment by Equation 4-11:

$$I_g = \frac{bh^3}{12}$$ **Equation 5-7**

$$I_g = \frac{(24ft \times 12\frac{in}{ft})(7.25in)^3}{12}$$

$$I_g = 9{,}146\ in^4$$

$$M_{cr} = \frac{(474psi)(9{,}146in^4)}{\frac{7.25in}{2}}$$

$$M_{cr} = 1{,}195{,}918lb - in = 100k - ft$$

To determine the moment strength of the section trial and error the following for an area of tensile reinforcement:

Use a single-layer #6 bar spaced at 10.125" on center

Account for clear cover of ¾" and center of bar spacing, total number of bars:

$$\frac{\left[24ft \times 12\frac{in}{ft}\right] - (2 \times 0.75in) - (.75in)}{10.125in} = 28.2 \text{ bars}$$

Use (29) #6 bars at 10.125" on center, total area of steel, A_s:

$$A_s = 29bars \times (0.44in^2) = 12.76in^2$$

The minimum flexural reinforcement per Section 10.5.1 of ACI 318-08 calls for the area of steel provided being not less than the maximum value obtained below:

$$\rho_{\min} = \frac{3\sqrt{f'_c}}{f_y}$$ **Equation 5-8**

$$\rho_{\min} = \frac{3\sqrt{4{,}000psi}}{60{,}000psi}$$

$$\rho_{\min} = 0.00316$$

$$\rho_{\min} = \frac{200}{f_y} \qquad \textbf{Equation 5-9}$$

$$\rho_{\min} = \frac{200}{60,000\,psi}$$

$$\rho_{\min} = 0.00333 \leftarrow \text{Governs}$$

compare the actual reinforcement ratio with the minimum:

$$\rho_{\sec} = \frac{A_s}{b_w d} \qquad \textbf{Equation 5-10}$$

$$\rho_{\sec} = \frac{12.76in^2}{\left(24\,ft \times \frac{12in}{1\,ft}\right)(3.625in)} = 0.0122$$

$$0.0122 \geq 0.00333$$

(reinforcement ratio meets the minimum reinforcement ratio requirement)

Check the minimum ratio of vertical reinforcement area to gross concrete area per Section 14.3.2 of ACI 318-08:

$$\rho_{\sec} = \frac{A_s}{b_w t} \qquad \textbf{Equation 5-11}$$

$$\rho_{\sec} = \frac{12.76in^2}{\left((24\,ft \times \frac{12in}{1\,ft}\right)(7.25in)}$$

$$\rho_{\sec} = 0.006 \geq 0.0015$$

(reinforcement ratio meets the minimum reinforcement ratio requirement)

Next, determine the effective area of steel defined by Equation 4-6:

$$A_{se} = 12.76in^2 + \left(\frac{(66.2k)(7.25in)}{2(60ksi)(3.625in)} \right)$$

$$A_{se} = 13.86\ in^2$$

Use the equivalent rectangular stress block, the depth of the stress block, as defined by Equation 4-10:

$$a = \frac{13.86in^2(60ksi)}{0.85(4ksi)24ft\left(\frac{12in}{1ft}\right)}$$

$$a = 0.849\ in$$

The alternative design of slender walls method can be used when the section is considered tension-controlled as addressed in Section 9.3.2 of ACI 318-08. Both criteria addressed in Section 4.2.2 can be used to verify that the section is indeed tension-controlled. For concrete with a compressive strength of 4,000 psi, Section 10.2.7.3 of ACI 318-08 gives
$\beta_1 = 0.85$, therefore:

$$c = \frac{a}{\beta_1}$$ **Equation 5-12**

$$c = \frac{0.849in}{0.85}$$

$$c = 1.0\ in$$

Next, check whether the section is tension-controlled:

$$\frac{c}{d_t} = \frac{1.0}{3.625} = 0.276 \leq 0.375$$

(tension-controlled, φ=0.90)

Alternatively, use similar triangles and check the strain values:

$$\varepsilon_t = 0.003\left(\frac{d_t}{c}\right) - 0.003$$ **Equation 5-13**

$$\varepsilon_t = 0.003\left(\frac{3.625in}{1.0in}\right) - 0.003$$

$$\varepsilon_t = 0.00788 \geq 0.005$$

(tension-controlled, Φ=0.90)

Use Equation 4-5 to determine the design moment strength:

$$\phi M_n = (0.9)(13.86in^2)(60ksi)\left(3.625in - \frac{0.849in}{2}\right)$$

$$\phi M_n = 2{,}395.4k - in = 199.6k - ft$$

Finally, compare design moment strength with the cracking moment:

$$199.6\ k\text{-}ft \geq 100\ k\text{-}ft$$

5.2.3 Applied Moment

Given the two methods discussed in Section 4.2.4 for obtaining the applied moment in the panel, the factored applied moment from Equation 4-14 is as follows:

$$M_{ua} = \left(\frac{0.461klf * (32ft)^2}{8}\right) + \left(\frac{19.2k * 5.125in * \frac{1ft}{12in}}{2}\right)$$

$$M_{ua} = 63.1k - ft = 757.3k - in$$

This is only the primary factored moment due to applied wind loading; however, the total combined moment on the section includes the primary moment and the secondary moment due to *P-Δ* effects. Notably, Figure 5-4 depicts the moment diagram of the panel and typically the parapet is neglected in the calculation of the applied moment because it reduces the applied moment.

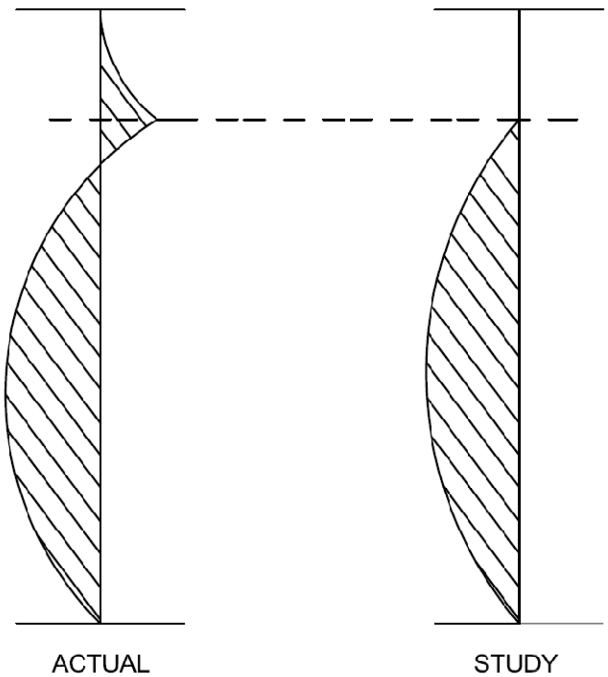

Figure 5-4 Moment Diagram

First, the study evaluated the moment magnifier method and determined the cracked moment of inertia from Equation 4-21 and Equation 4-22:

$$I_{cr} = 8.044(13.86in^2)(3.625in - 1.0in)^2 + \frac{\left(24ft*12\frac{in}{ft}\right)(1.0in)^3}{3}$$

$$I_{cr} = 864in^4$$

The maximum moment calculated by the direct method follows:

$$M_u = \frac{757.3k - in}{1 - \left(\frac{5(66.2k)\left(32ft*\frac{12in}{1ft}\right)^2}{(0.75)(48)(3605ksi)(864in^4)}\right)}$$

$$M_u = 1,341k - in = 111.8k - ft$$

The maximum moment can also be obtained by an iterative process based on Equation 4-15:

$$M_u = M_{ua} + P_u\Delta_u$$

The first iteration will begin with the factored applied moment already found from

Equation 4-14. Next, Equation 4-23 determines the deflection due to this moment:

$$\Delta_u = \frac{5(757.3k - in)\left(32\,ft \times 12\,\dfrac{in}{ft}\right)^2}{(0.75)(48)(3605ksi)(864in^4)}$$

$$\Delta_u = 4.98in$$

Therefore, the first iteration gives the following maximum moment using Equation 4-15:

$$M_u = 757.3k - in + (66.2k)(4.98in)$$

$$M_u = 1{,}087k - in$$

Continuing the iterative process using Equation 4-15, generates a 'new' deflection based on the 'new' maximum moment found. The iteration process should continue until the maximum moment and deflection both converge. The end result of the maximum moment should be very close to the value obtained from the direct method as shown below:

$$\Delta_u = \frac{5(1{,}339k - in)\left(32\,ft \times 12\,\dfrac{in}{ft}\right)^2}{(0.75)(48)(3605ksi)(864in^4)}$$

$$\Delta_u = 8.80in$$

$$M_u = 757.3k - in + (66.2k)(8.80in)$$

$$M_u = 1{,}340k - in = 111.6k - ft$$

Only one of the methods discussed would need to determine the maximum moment on the panel; however, for comparison the report shows both methods. For strength requirements, the following relationship needs to be satisfied:

$$\varphi M_n \geq M_u$$

$$199.6\,k\text{-}ft \geq 111.6\,k\text{-}ft$$

(design strength is greater than required moment)

5.2.4 Service Load Deflection

Now that the panel satisfies strength requirements, it must satisfy serviceability requirements too. Section 14.8.4 of ACI 318-08 states that the maximum out-of-plane deflection due to service loads, including *P-Δ* effects, shall not exceed the following:

$$\Delta_{allow} = \frac{\ell_c}{150}$$

An iterative process similar to that used for obtaining the applied moment will determine the maximum deflection of the panel due to service loads. To begin, the maximum moment at mid-height of the panel due to service loads derives from Equation 4-14:

$$M_{sa} = \left(\frac{24psf \times 24ft \times (32ft)^2}{8 \times \frac{1,000lbs}{1k}} \right) + \left(\frac{(5.76k + 7.69k) \times 5.125in \times \frac{1ft}{12in}}{2} \right)$$

$$M_{sa} = 76.6k - ft = 919.2k - in$$

This moment due to service loads is compared to $(2/3)M_{cr}$ to determine which Δ_s equation is appropriate.

$$\frac{2}{3}(100k - ft) = 67k - ft$$

$$76.6k - ft > 67k - ft \text{ therefore use Equation 4-29}$$

From Equation 4-30:

$$\Delta_{cr} = \frac{5\left(100k - ft * \frac{12in}{1ft}\right)\left(32ft * \frac{12in}{1ft}\right)^2}{48(3605ksi)(9,146in^4)}$$

$$\Delta_{cr} = 0.56in$$

From Equation 4-31:

$$\Delta_n = \frac{5\left(\frac{199.6k - ft}{0.90} \times \frac{12in}{1ft}\right)\left(32ft \times \frac{12in}{1ft}\right)^2}{48(3605ksi)(864in^4)}$$

$$\Delta_n = 13.12in$$

Use Equation 4-29 as determined above:

$$\Delta_s = (2/3)(0.56in) + \left(\frac{76.6k - ft - (2/3)(100k - ft)}{\frac{199.6k - ft}{0.9} - (2/3)(100k - ft)} \right) \left[13.12in - (2/3)(0.56in)\right]$$

$$\Delta_s = 1.20in$$

Now the iterative process begins:

$$M_a = 919.2k - in + (39.2k + 5.76k + 7.69k)(1.20in)$$

$$M_{sa} = 982.4k - in = 81.9k - ft$$

Determine deflection based on a ‘new’ maximum service moment:

$$\Delta_s = (2/3)(0.56in) + \left(\frac{81.9k - ft - (2/3)(100k - ft)}{\frac{199.6k - ft}{0.9} - (2/3)(100k - ft)} \right) \left[13.12in - (2/3)(0.56in) \right]$$

$$\Delta_s = 1.63in$$

Calculate the ‘new’ maximum moment due to service load:

$$M_a = 919.2k - in + (39.2k + 5.76k + 7.69k)(1.63in)$$

$$M_a = 1{,}005k - in = 83.8k - ft$$

Continue iteration until convergence:

$$\Delta_s = 1.79in$$

$$M_a = 1{,}013k - in = 84.4k - ft$$

$$\Delta_s = 1.84in$$

$$M_a = 1{,}016.2k - in = 84.7k - ft$$

$$\Delta_s = 1.86in$$

$$M_a = 1{,}017.3k - in = 84.8k - ft$$

$$\Delta_s = 1.87in$$

$$M_a = 1{,}017.7k - in = 84.8k - ft$$

$$\Delta_s = 1.87in$$

Once the maximum moment and deflection both converge, compare the maximum deflection due to service loads to the allowable deflection.

$$\Delta_{allow} = \frac{32ft * \frac{12in}{1ft}}{150} = 2.56in$$

$$1.87in \leq 2.56in$$

(deflection due to service load meets maximum allowable deflection requirement)

These calculations show the strength and serviceability requirements per Section 14.8 of ACI 318-08 have both been satisfied for Load Case 1. Evaluation of Load Case 2 and Load Case 3 for all panel scenarios can be found in Appendix C.

5.3 Minimum Horizontal Reinforcement

To satisfy minimum horizontal reinforcement requirements defined in Section 14.3.3 of ACI 318-08:

$$A_{s,\min} = 0.002(7.25in)\left(34ft * \frac{12in}{ft}\right)$$

$$A_{s,\min} = 5.916in^2$$

$$\text{\# of bars} = \frac{5.916in^2}{0.2in^2} = 29.58$$

Therefore provide 30-#4 bars for horizontal reinforcement.

5.4 Summary

Figure 5-5 shows that for the given loading, a 7.25" thick panel with vertical reinforcement of (29)-#6 bars at 10.125" on center, satisfied both strength and serviceability requirements per the code and load combinations. While the design process is tedious and requires several trial and error calculations, the goal as a designer is to achieve the most economical design while first ensuring safety. Finally, design results for greater wind speeds and unbraced length are shown in Table 7-1 and Table 7-2.

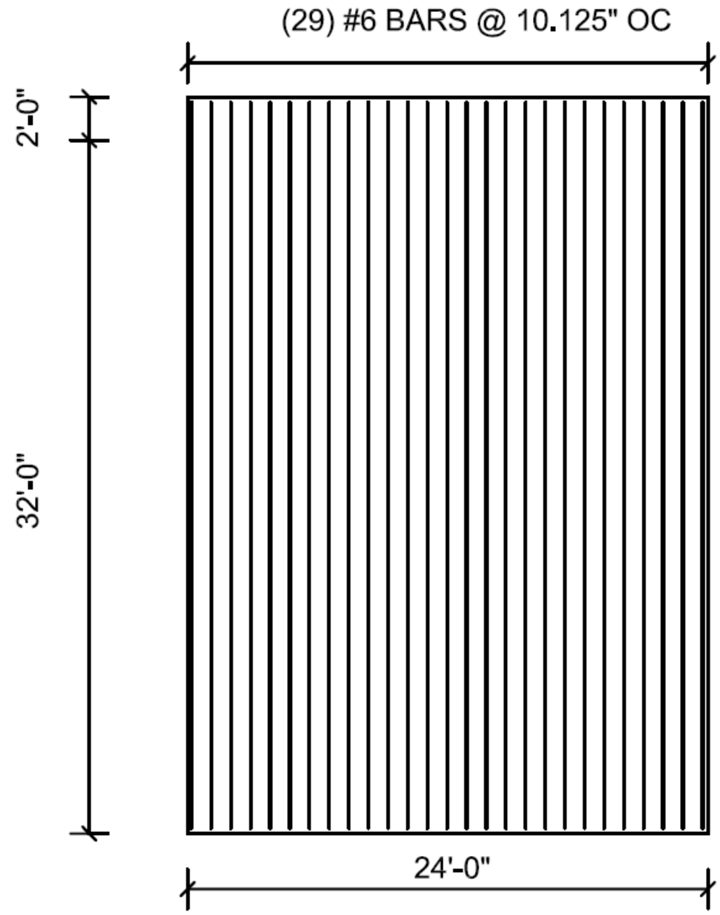

Figure 5-5 Solid Panel Reinforcement Layout

CHAPTER 6 - Panel with Opening Design Example

For clarity, this chapter will provide the design process for vertical reinforcement for a panel with a square opening in the middle using the alternative design of slender walls. An opening placed in the middle of the panel is of interest because maximum moment and deflection occurs at or near mid-height of the panel. Tilt-up panels with an opening does not have a constant cross-section over the height of the panel; therefore, to account for the effect of the opening, the design section is termed the jamb or 'design strip'. Figure 6-1 illustrates the design approach for a panel with an opening centered in the middle. The 'design strips' are to resist tributary wind lateral loads and vertical loads tributary to the strips (Tilt-Up Concrete Association, 2004).

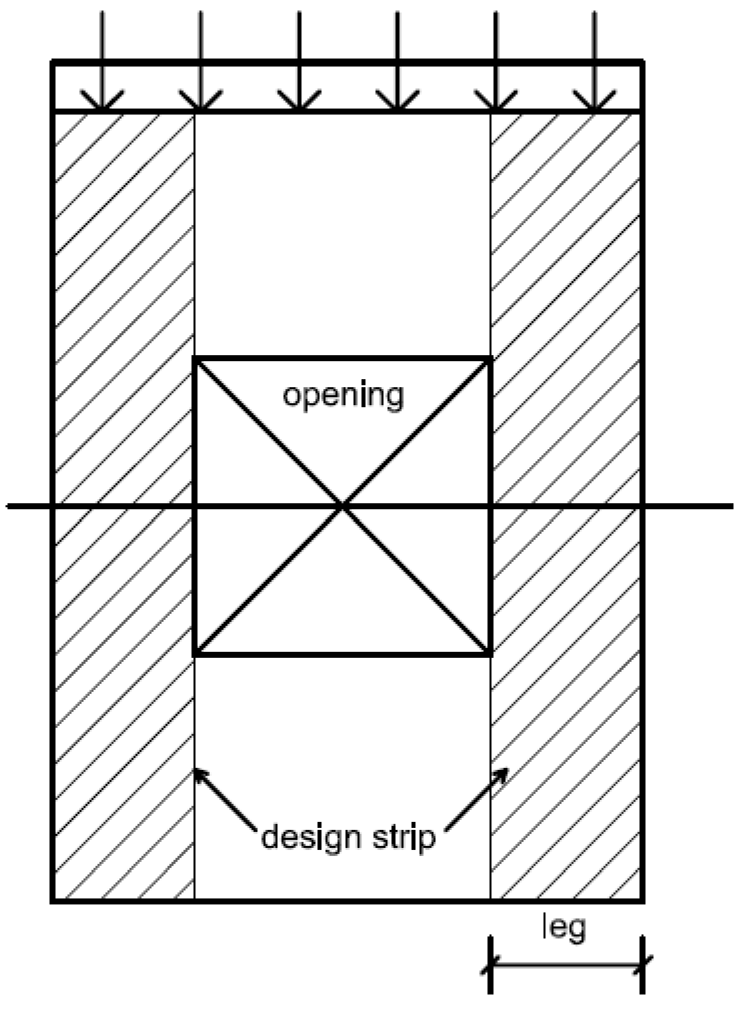

Figure 6-1 Design Model for Panel with Opening

6.1 Panel Design Properties

Panel width	= 24'-0"	f'_c	= 4,000 psi
Panel height	= 34'-0"	f_y	= 60,000 psi
Unbraced length	= 32'-0"	γ_c	= 150 pcf (normal weight concrete)
Parapet	= 2'-0"	f_r	= 474 psi (Equation 3-3)
"d" (tensile steel)	= 5.5"	E_c	= 3605 ksi (Equation 3-1)
Square Opening	= 12'-0"x12'-0"		
n	= 8.044 (Equation 4-22)		

Figure 6-2 illustrates the geometry of the panel with a 12' x 12' opening.

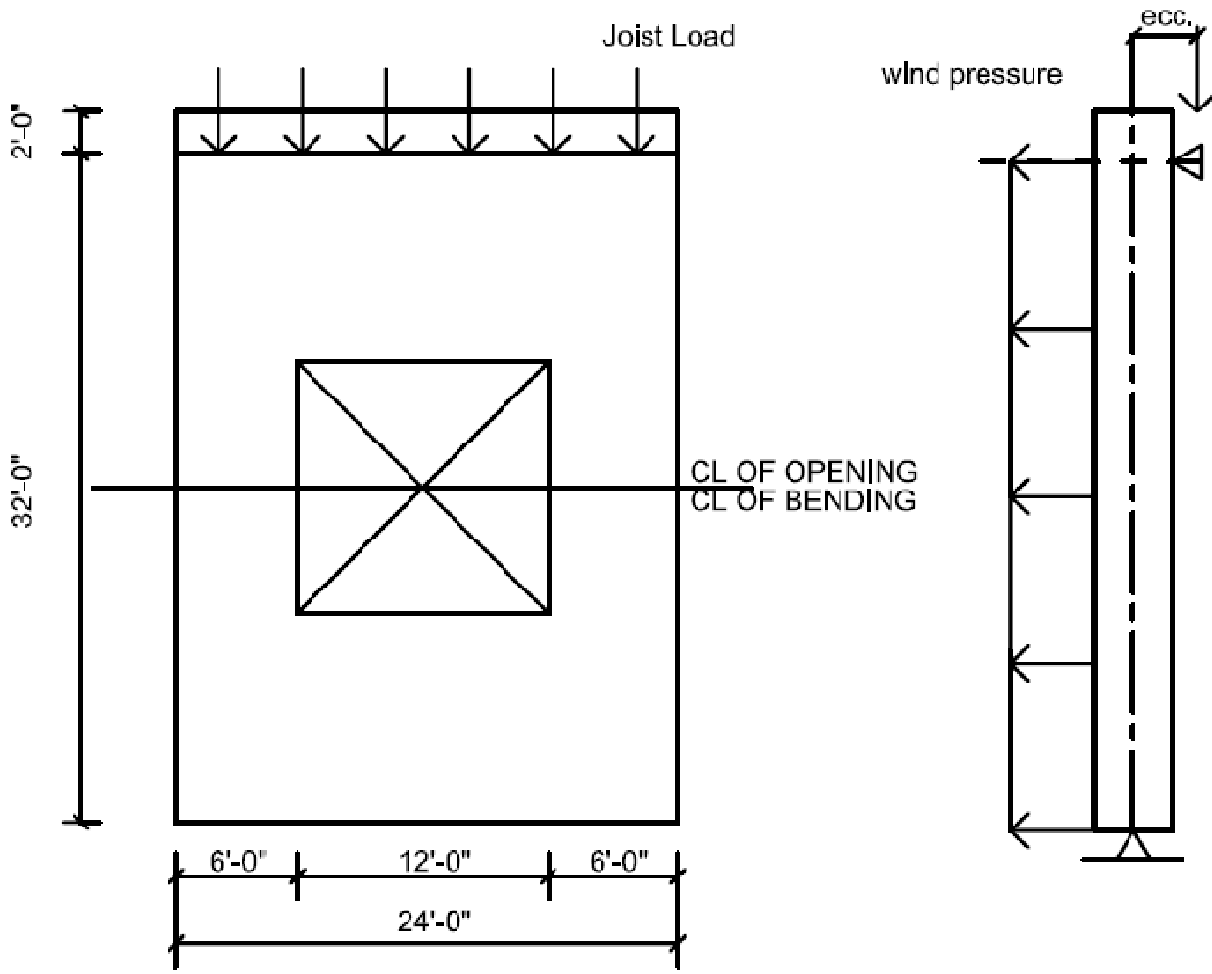

Figure 6-2 Panel with Opening Geometry

Figure 6-3 illustrates the cross-section of the panel. Trial and error of reinforcement configurations determined two layers of steel for adequate strength and deflection control.

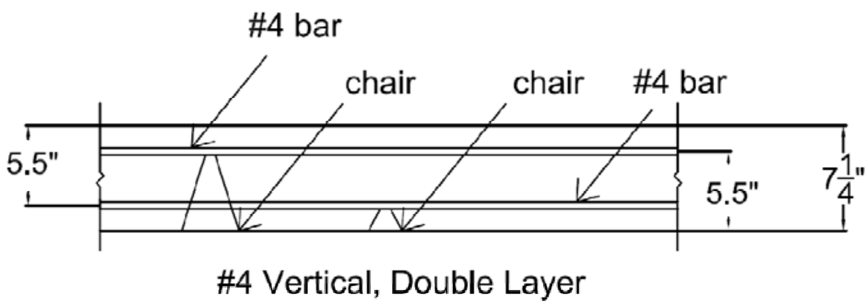

Figure 6-3 Panel with Opening Cross Section

The eccentric axial load and out-of-plane wind pressure remains the same as the previous example. Due to symmetry of the panel, the 'design strips' are equivalent, and therefore the vertical reinforcement will be the same for both panel legs on either side of the opening. Based on the tributary concept, the lateral wind pressure of 24 psf is distributed around the opening to the panel legs.

$$\text{panel leg width (design strip)} = \frac{1}{2}(24\,ft - 12\,ft) = 6\,ft$$

$$\text{tributary width} = 6\,ft + \frac{12\,ft}{2} = 12\,ft$$

6.2 Load Case 1 (C&C)

This section addresses the first of the three load combinations discussed in Section 4.2.1.

$$1.2D + 1.6(L_r \text{ or } S) + 0.8W \qquad \textbf{Equation 6-1}$$

The ultimate applied axial load (excluding self-weight) derives from the load case and considers only three joists contributing to one panel leg:

$$P_{ua} = 1.2(P_{DL}) + 1.6(P_{(Lr\ or\ S)}) + 0.8W \qquad \textbf{Equation 6-2}$$

$$P_{ua} = 1.2(3x0.96k) + 1.6(3x1.28k)$$

$$P_{ua} = 9.61k$$

P_{um}= P_{ua} + 1.2(panel weight above CL) **Equation 6-3**

panel weight above CL =

$$\frac{\left(\frac{7.25in}{\frac{12in}{ft}}(150pcf)\right)\left\{\left[\left(\left(\frac{32ft}{2}+2ft\right)-\frac{12ft}{2}\right)\frac{24ft}{2}\right]+\left[\left(\left(\frac{24ft}{2}-\frac{12ft}{2}\right)\frac{12ft}{2}\right)\right]\right\}}{1{,}000\frac{lbs}{k}} = 16.3k$$

$$P_{um} = 9.61k + 1.2(16.3k) = 29.2k$$

Obtain the factored wind load, which is applied at the opening and transferred over to the 'design strip'. It is suggested that the maximum effective width of the 'design strip' does not exceed 12*h*. However, for the purpose of this study, the entire panel leg width is used for all opening sizes (ACI Committee 551, 2009). Therefore, the tributary width is ½ the opening width plus 'design strip' width:

w_u = 0.8(wind)(tributary width) **Equation 6-4**

$$w_u = 0.8(24psf)(12ft)$$

$$w_u = 230plf = 0.230klf$$

6.2.1 Vertical Stresses

Section 14.8.2.6 of ACI 318-08 states that the vertical stress at the mid-height of the panel shall not exceed $0.06f'_c$. Therefore, check the stresses:

$$\frac{P_u}{A_g} = \frac{P_u}{(design\_strip\_width) \times thickness}$$ **Equation 6-5**

$$\frac{P_u}{A_g} = \frac{29.2k(1000lb/k)}{(6ft*12\frac{in}{ft})7.25in} = 55.9\ psi$$

$$55.9\ psi \leq (0.06*4000psi) = 240\ psi$$

6.2.2 Design Moment Strength

Section 14.8.2.4 of ACI 318-08 states that vertical reinforcement shall provide design strength greater than or equal to the cracking moment:

$$\varphi M_n \geq M_{cr}$$ **Equation 6-6**

(ACI Equation 14-2)

The cracking moment is defined by Equation 4-11:

$$I_g = \frac{bh^3}{12}$$ **Equation 6-7**

$$I_g = \frac{(6ft*12\frac{in}{ft})\left(7.25in\right)^3}{12}$$

$$I_g = 2{,}286\ in^4$$

$$M_{cr} = \frac{(474psi)\left(2{,}286in^4\right)}{\frac{7.25in}{2}}$$

$$M_{cr} = 298{,}914lb-in = 25k-ft$$

To determine the moment strength of the section, trial and error the following area of tensile reinforcement :

Use two-layers of #4 bar spaced at 3. 25" on center each face

Account for clear cover of ¾'' and center of bar spacing, total number of bars:

$$\left(\frac{\left[6ft\times 12\frac{in}{ft}\right]-(2\times 0.75in)-(0.5in)}{3.25in}\right) = 21.5 \text{ bars}$$

Use (22)-#4 bars at 3.625" on center each face, total area of steel, A_s:

$$A_s = 22bars\times(0.2in^2) = 4.4in^2 \text{ each face}$$

Finally, checking the minimum ratio of vertical reinforcement area to gross concrete area per Section 14.3.2 of ACI 318-08:

$$\rho_{sec} = \frac{A_s}{b_w t}$$ **Equation 6-8**

$$\rho_{sec} = \frac{4.4in^2}{\left(6ft * \frac{12in}{1ft}\right)(7.25in)}$$

$$\rho_{sec} = 0.0084 \geq 0.0012$$

(reinforcement ratio meets the minimum reinforcement ratio requirement)

Now, calculate the effective area of steel defined by Equation 4-6:

$$A_{se} = 4.4in^2 + \left(\frac{(29.2k)(7.25in)}{2(60ksi)(5.5in)}\right)$$

$$A_{se} = 4.72 \text{ in}^2$$

Use the equivalent rectangular stress block, the depth of the stress block, as defined by Equation 4-10:

$$a = \frac{4.72in^2(60ksi)}{0.85(4ksi)6ft\left(\frac{12in}{1ft}\right)}$$

$$a = 1.16 \; in$$

Verify if the section is tension-controlled just like in the previous example. For concrete with a compressive strength of 4,000 psi, Section 10.2.7.3 of ACI 318-08 gives

β_1 = 0.85, therefore:

$$c = \frac{a}{\beta_1}$$ **Equation 6-9**

$$c = \frac{1.16in}{0.85}$$

$$c = 1.36 \; in$$

$$\frac{c}{d_t} = \frac{1.36in}{5.5in} = 0.247 \leq 0.375$$

(tension-controlled, φ=0.90)

Or use similar triangles and check the strain values:

$$\varepsilon_t = 0.003\left(\frac{d_t}{c}\right) - 0.003$$ **Equation 6-10**

$$\varepsilon_t = 0.003\left(\frac{5.5in}{1.36in}\right) - 0.003$$

$$\varepsilon_t = 0.009 \geq 0.005$$

(tension-controlled, φ=0.90)

Use Equation 4-5 to determine the nominal moment strength:

$$\phi M_n = (0.9)(4.72in^2)(60ksi)\left(5.5in - \frac{1.16in}{2}\right)$$

$$\phi M_n = 1,254k - in = 104.5k - ft$$

Compare nominal moment strength with the cracking moment:

104.5 *k-ft* ≥ 25 *k-ft (OK)*

6.2.3 Applied Moment

Recall the two methods discussed in Section 4.2.4 for obtaining the applied moment in the panel. First, consider the factored applied moment from Equation 4-14:

$$M_{ua} = \left(\frac{0.230klf * (32ft)^2}{8}\right) + \left(\frac{9.61k * 5.5in * \frac{1ft}{12in}}{2}\right)$$

$$M_{ua} = 31.6k - ft = 379.2k - in$$

Then evaluate the moment magnifier method using the cracked moment of inertia found from Equation 4-21 and Equation 4-22:

$$I_{cr} = 8.044(4.72in^2)(5.5in - 1.36in)^2 + \frac{\left(6ft * 12\frac{in}{ft}\right)(1.36in)^3}{3}$$

$$I_{cr} = 711in^4$$

Then calculate the maximum moment calculated by the direct method:

$$M_u = \frac{379.2k - in}{1 - \left(\frac{5(29.2k)\left(32ft * \frac{12in}{1ft}\right)^2}{(0.75)(48)(3605ksi)(711in^4)} \right)}$$

$$M_u = 494.6k - in = 41.2k - ft$$

The maximum moment can also be obtained by an iterative process using Equation 4-15 as the base equation:

$$M_u = M_{ua} + P_u\Delta_u$$

The first iteration will begin with the factored applied moment from Equation 4-14. The deflection due to this moment is found by Equation 4-23.

$$\Delta_u = \frac{5(379.2k - in)\left(32ft * 12\frac{in}{ft}\right)^2}{(0.75)(48)(3605ksi)(711in^4)}$$

$$\Delta_u = 3.03in$$

Therefore, the first iteration gives the following maximum moment using Equation 4-15:

$$M_u = 379.2k - in + (29.2k)(3.03in)$$

$$M_u = 467.7k - in$$

Continuing the iterative process using Equation 4-15, a 'new' deflection will be determined based on the 'new' maximum moment found. Then, the iteration process should be continued until the maximum moment and deflection both converge. The end result of the maximum moment should be very close to the value obtained from the direct method as shown below:

$$\Delta_u = \frac{5(493.6k - in)\left(32ft * 12\frac{in}{ft}\right)^2}{(0.75)(48)(3605ksi)(711in^4)}$$

$$\Delta_u = 3.94in$$

$$M_u = 379.2k - in + (29.2k)(3.94in)$$

$$M_u = 494.2k - in = 41.2k - ft$$

Only one of the methods would be necessary to determine the maximum moment on the panel, but for comparison, both methods were shown. For strength requirements, the following relationship needs to be satisfied:

$$\varphi M_n \geq M_u$$

$$104.5 k\text{-}ft \geq 41.2 k\text{-}ft$$

(design moment strength is greater than required moment)

6.2.4 Service Load Deflection

Now that the panel satisfies strength requirements, check if the panel satisfies serviceability requirements too. Section 14.8.4 of ACI 318-08 states that the maximum out-of-plane deflection due to service loads, including $P\text{-}\Delta$ effects, shall not exceed:

$$\Delta_{allow} = \frac{\ell_c}{150}$$

An iterative process similar to that used for obtaining the applied moment will determine the maximum deflection of the panel due to service loads. First, calculate the maximum moment at mid-height of the panel due to service loads from Equation 4-14:

$$M_a = \left(\frac{24\,psf \times 12\,ft \times (32\,ft)^2}{8 \times \frac{1{,}000\,lbs}{1k}} \right) + \left(\frac{3 \times (0.96k + 1.28k) \times 5.5in \times \frac{1ft}{12in}}{2} \right)$$

$$M_{sa} = 38.4k - ft = 460.8k - in$$

This moment due to service loads is compared to $(2/3)M_{cr}$ to determine which Δ_s equation is appropriate.

$$\frac{2}{3}(25k - ft) = 16.7k - ft$$

$$38.4k - ft > 16.7k - ft \text{ therefore use Equation 4-29}$$

From Equation 4-30:

$$\Delta_{cr} = \frac{5\left(25k - ft * \frac{12in}{1ft}\right)\left(32\,ft * \frac{12in}{1ft}\right)^2}{48(3605ksi)(2{,}286in^4)}$$

$$\Delta_{cr} = 0.56in$$

From Equation 4-31:

$$\Delta_n = \frac{5\left(\frac{104.5k-ft}{0.90}*\frac{12in}{1ft}\right)\left(32ft*\frac{12in}{1ft}\right)^2}{48(3605ksi)(711in^4)}$$

$$\Delta_n = 8.35in$$

Use Equation 4-29 as determined above:

$$\Delta_s = (2/3)(0.56in) + \left(\frac{38.4k-ft-(2/3)(25k-ft)}{\frac{104.5k-ft}{0.9}-(2/3)(25k-ft)}\right)\left[8.35in-(2/3)(0.56in)\right]$$

$$\Delta_s = 2.11in$$

Now, the iterative process, like before, can begin:

$$M_a = 460.8k-in + (16.3k + 3*(0.96k + 1.28k))(2.11in)$$

$$M_a = 509.4k-in = 42.5k-ft$$

Calculate deflection based on the 'new' maximum service moment:

$$\Delta_s = (2/3)(0.56in) + \left(\frac{42.5k-ft-(2/3)(25k-ft)}{\frac{104.5k-ft}{0.9}-(2/3)(25k-ft)}\right)\left[8.35in-(2/3)(0.56in)\right]$$

$$\Delta_s = 2.44in$$

Determine 'new' maximum moment due to service load:

$$M_a = 460.8k-in + (16.3k + 3*(0.96k + 1.28k))(2.44in)$$

$$M_a = 517k-in = 43.1k-ft$$

Continue iteration until convergence:

$$\Delta_s = 2.49in$$

$$M_a = 517.03k-in = 43.1k-ft$$

$$\Delta_s = 2.50in$$

$$M_a = 517.05k-in = 43.1k-ft$$

Once the maximum moment and deflection both converge, compare the maximum deflection due to service loads to the allowable deflection.

$$\Delta_{allow} = \frac{32ft * \frac{12in}{1ft}}{150} = 2.56in$$

$$2.49in \leq 2.56in$$

(deflection due to service load meets maximum allowable deflection requirement)

Final calculations show the strength and serviceability requirements per Section 14.8 of ACI 318-08 have both been satisfied for Load Case 1.

6.4 Minimum Horizontal Reinforcement

To satisfy minimum horizontal reinforcement requirements defined in Section 14.3.3 of ACI 318-08:

$$A_{s,\min} = 0.002(7.25in)\left(34ft * \frac{12in}{ft}\right)$$

$$A_{s,\min} = 5.916in^2$$

$$\text{\# of bars} = \frac{5.916in^2}{0.2in^2} = 29.58$$

Therefore provide 30-#4 bars for horizontal reinforcement, modifying the lengths where needed on both sides of the opening.

6.5 Summary

Figure 6-4 shows that for the given loading, a 7.25" thick panel with vertical reinforcement in each panel leg of (22)-#4 bars at 3.25" on center and each face, satisfied both strength and serviceability requirements per the code and load combinations. Recall this is only the vertical reinforcement to be placed in the panel legs on both sides of the opening. Meanwhile, vertical reinforcement above and below the opening shall meet the code minimum as defined in ACI 318-08 Section 14.3.2. Finally, design results for greater wind speeds and unbraced length are shown in Table 7-3 and Table 7-4.

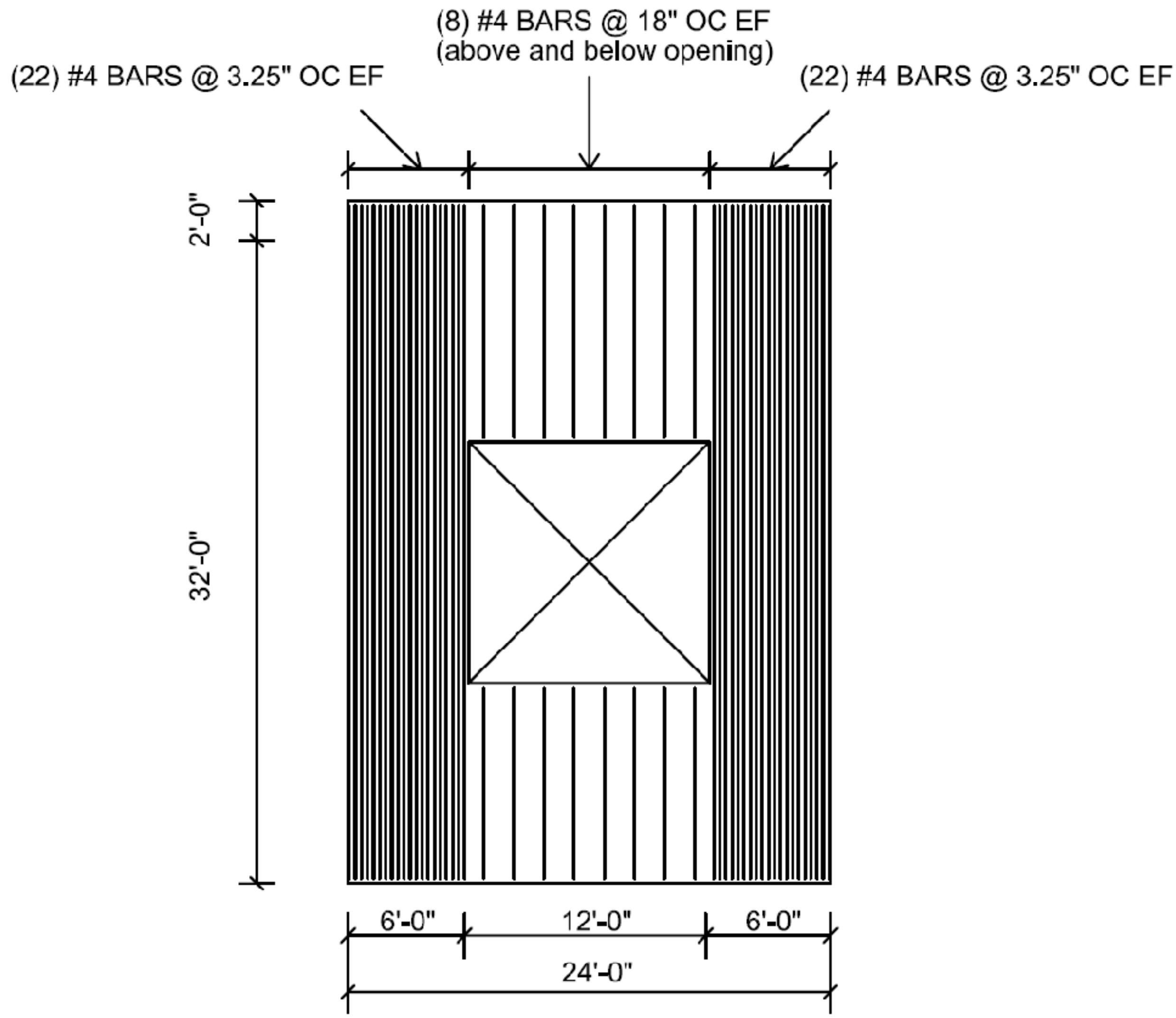

Figure 6-4 Panel with Opening Reinforcement Layout

CHAPTER 7 - Results and Conclusions

With the Alternative Design of Slender Walls method, vertical reinforcement was determined for tilt-up wall panels subject to axial load and out-of-plane uniform lateral wind pressure based on wind speeds of 90 mph, 110 mph, 130 mph, and 150 mph for unbraced lengths of 32 ft and 40 ft. Analysis was performed both for a panel with no openings and for a panel with a square opening centered in the panel varying in size: 4 ft, 8 ft, 12 ft, and 16 ft. For formwork purposes, panel thicknesses were of 7.25", 9.25" and 11.25", and vertical reinforcement included #4, #5, and #6 bars.

Tilt-up wall panels must satisfy strength and serviceability requirements defined by ACI 318-08 Sections 14.8.3 and 14.8.4, respectively and abide by code limitations for reinforcement requirements. The moment magnifier method presented in ACI 318-08 Section 14.8 is considered a trial and error approach in determining the area of tension reinforcement to satisfy strength requirements. Recall from Equation 4-15:

$$M_u = M_{ua} + P_u \Delta_u$$

The maximum moment M_u can be written as:

$$M_u = M_{ua} + \frac{P_u M_u}{K_b}$$

Furthermore, the term K_b is referred to as the panel stiffness:

$$K_b = \frac{M_u}{\Delta_u} = \frac{48 E_c I_{cr}}{5\ell^2}$$

From Equation 4-5, φM_n is directly proportional to the area of effective tension reinforcement, A_{se}:

$$\varphi M_n = \varphi A_{se} f_y (d - \frac{a}{2})$$

Figure 7-1 shows that the factored moment, M_u, is inversely proportional to A_{se}.

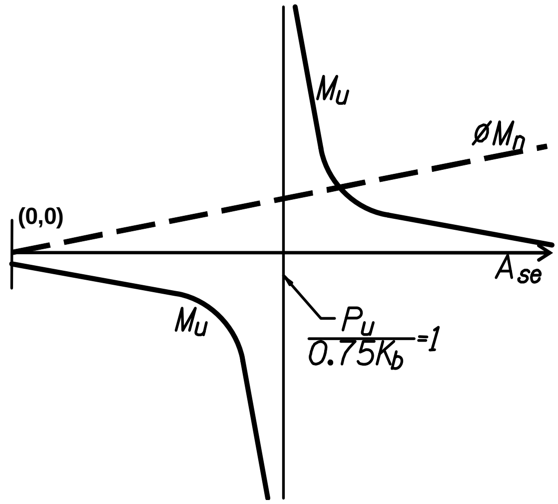

Figure 7-1 Relationship of Nominal Moment, Factored Moment, and Effective Area of Reinforcement (ACI Committee 551, 2009)

The minimum amount of tension reinforcement is defined by the intersecting point of the M_u and φM_n curves. Notably, an area of effective reinforcement to the right of the intersecting point provides adequate strength. In contrast, a small value of A_{se} may result in a negative factored applied moment.

The most economical design uses the least amount of material, both of concrete and reinforcing steel. Therefore, the thinnest panel with the least area of reinforcing steel is desired, while still satisfying strength and serviceability requirements. Spacing of the vertical reinforcement within this report is to 0.125" accuracy. A single layer of reinforcement results in a decreased area of reinforcement by increasing the vertical bar size, while slightly increasing the depth of the tensile steel. In contrast, a double layer of reinforcement yields a greater area of reinforcement by increasing the vertical bar size, while slightly decreasing the depth of the tensile steel.

7.1 Solid Panel Results

Table 7-1 shows the vertical reinforcement for an unbraced length of 32 ft and no openings in the panel.

32'-0 Unbraced Length							
Wind Speed	Panel Thickness	Layers of Steel	Vertical Bar Size	Spacing	No. of Vertical Bars/Layer	Total Vertical Bars in Panel	Total A_s, In^2
90mph	7.25 in	1	6	10.125 in	29	29	12.8
110mph	7.25 in	2	4	6.625 in	44	88	17.6
130mph	7.25 in	2	4	3.500 in	82	164	32.8
150mph	9.25 in	2	4	5.375 in	54	108	21.6

Table 7-1 Solid Panel Results - 32 ft

Table 7-2 shows the vertical reinforcement for an unbraced length of 40 ft and no openings in the panel.

40'-0 Unbraced Length							
Wind Speed	Panel Thickness	Layers of Steel	Vertical Bar Size	Spacing	No. of Vertical Bars/Layer	Total Vertical Bars in Panel	Total A_s, in^2
90mph	7.25 in	2	4	3.750 in	77	154	30.8
110mph	9.25 in	2	4	5.625 in	51	102	20.4
130mph	9.25 in	2	4	3.500 in	82	164	32.8
150mph	9.25 in	2	4	2.000 in	143	286	57.2

Table 7-2 Solid Panel Results - 40 ft

Table 7-1 and Table 7-2 reflect the most economical design for the total vertical reinforcement in the solid panel. For constant panel thickness and wind speed, total vertical area of reinforcement increased by an average of 250% when the unbraced height of the panel increased from 32 ft to 40 ft. For a thinner solid panel the area of reinforcement increases to satisfy serviceability requirements as the wind speed increases. This is due do the fact that the 7.25” panel is quite slender and lacks stiffness. As the panel height increases from 32 ft to 40 ft, an increase in panel thickness is needed for the higher wind speeds. Deflection governs for a 40 ft tall panel subject to high wind speeds. Therefore, the self-weight of the panel greatly contributes to the design in heavy wall panels.

7.2 Panel with Opening Results

Table 7-3 illustrates the wall panel design with different size of openings centered in the unbraced length of 32 ft and subject to varying wind speeds.

32'-0 Unbraced Length								
	Vertical Reinforcement in Panel Legs						Includes above/below opening	
	Opening Size	Panel Thickness	Layers of Steel	Vertical Bar Size	Spacing	No. of Vertical Bars/Layer	Total Vertical Bars in Panel	Total A_s, in^2
Wind Speed = 90mph	4 ft x 4 ft	7.25 in	2	4	7.750 in	16	76	15.2
	8 ft x 8 ft	7.25 in	2	4	6.250 in	16	88	17.6
	12 ft x 12 ft	7.25 in	2	4	3.250 in	22	120	24
	16 ft x 16 ft	9.25 in	2	4	4.500 in	11	88	17.6

32'-0 Unbraced Length								
	Vertical Reinforcement in Panel Legs						Includes above/below opening	
	Opening Size	Panel Thickness	Layers of Steel	Vertical Bar Size	Spacing	No. of Vertical Bars/Layer	Total Vertical Bars in Panel	Total A_s, in^2
Wind Speed = 110mph	4 ft x 4 ft	7.25 in	2	4	4.500 in	27	120	24
	8 ft x 8 ft	7.25 in	2	4	2.750 in	35	164	32.8
	12 ft x 12 ft	9.25 in	2	4	4.875 in	15	92	18.4
	16 ft x 16 ft	9.25 in	2	4	2.750 in	17	112	22.4

32'-0 Unbraced Length								
	Vertical Reinforcement in Panel Legs						Includes above/below opening	
	Opening Size	Panel Thickness	Layers of Steel	Vertical Bar Size	Spacing	No. of Vertical Bars/Layer	Total Vertical Bars in Panel	Total A_s, in^2
Wind Speed = 130mph	4 ft x 4 ft	7.25 in	2	4	2.250 in	53	224	44.8
	8 ft x 8 ft	9.25 in	2	4	4.625 in	21	108	21.6
	12 ft x 12 ft	9.25 in	2	4	3.250 in	22	120	24
	16 ft x 16 ft	11.25 in	2	4	3.000 in	16	108	21.6

32'-0 Unbraced Length								
	Vertical Reinforcement in Panel Legs						Includes above/below opening	
	Opening Size	Panel Thickness	Layers of Steel	Vertical Bar Size	Spacing	No. of Vertical Bars/Layer	Total Vertical Bars in Panel	Total A_s, in^2
Wind Speed = 150mph	4 ft x 4 ft	9.25 in	2	4	4.500 in	27	120	24
	8 ft x 8 ft	9.25 in	2	4	3.375 in	28	136	27.2
	12 ft x 12 ft	9.25 in	2	4	1.875 in	38	184	36.8
	16 ft x 16 ft	11.25 in	2	4	2.125 in	22	132	26.4

Table 7-3 Panel with Opening Results - 32 ft

Table 7-4 illustrates the wall panel design with different size of openings centered in the unbraced length of 40 ft and subject to varying wind speeds.

40'-0 Unbraced Length								
	Vertical Reinforcement in Panel Legs						Includes above/below opening	
Wind Speed = 90mph	Opening Size	Panel Thickness	Layers of Steel	Vertical Bar Size	Spacing	No. of Vertical Bars/Layer	Total Vertical Bars in Panel	Total A_s, in^2
	4 ft x 4 ft	7.25 in	2	4	2.250 in	53	224	44.8
	8 ft x 8 ft	9.25 in	2	4	5.125in	19	100	20
	12 ft x 12 ft	9.25 in	2	4	3.375 in	21	116	23.2
	16 ft x 16 ft	11.25 in	2	4	3.500 in	14	100	20

40'-0 Unbraced Length								
	Vertical Reinforcement in Panel Legs						Includes above/below opening	
Wind Speed = 110mph	Opening Size	Panel Thickness	Layers of Steel	Vertical Bar Size	Spacing	No. of Vertical Bars/Layer	Total Vertical Bars in Panel	Total A_s, in^2
	4 ft x 4 ft	9.25 in	2	4	4.625 in	26	116	23.2
	8 ft x 8 ft	9.25 in	2	4	3.000 in	32	152	30.4
	12 ft x 12 ft	11.25 in	2	4	3.625 in	20	112	22.4
	16 ft x 16 ft	11.25 in	2	4	2.125 in	22	132	26.4

40'-0 Unbraced Length								
	Vertical Reinforcement in Panel Legs						Includes above/below opening	
Wind Speed = 130mph	Opening Size	Panel Thickness	Layers of Steel	Vertical Bar Size	Spacing	No. of Vertical Bars/Layer	Total Vertical Bars in Panel	Total A_s, in^2
	4 ft x 4 ft	9.25 in	2	4	2.375 in	50	212	42.4
	8 ft x 8 ft	11.25 in	2	4	3.500 in	27	132	26.4
	12 ft x 12 ft	11.25 in	2	4	2.375 in	30	152	30.4
	16 ft x 16 ft	NON-TENSION CONTROLLED SECTION						

40'-0 Unbraced Length								
	Vertical Reinforcement in Panel Legs						Includes above/below opening	
Wind Speed = 150mph	Opening Size	Panel Thickness	Layers of Steel	Vertical Bar Size	Spacing	No. of Vertical Bars/Layer	Total Vertical Bars in Panel	Total A_s, in^2
	4 ft x 4 ft	11.25 in	2	4	3.375 in	35	152	30.4
	8 ft x 8 ft	11.25 in	2	4	2.500 in	38	176	35.2
	12 ft x 12 ft	11.25 in	2	4	1.375 in	51	236	47.2
	16 ft x 16 ft	NON-TENSION CONTROLLED SECTION						

Table 7-4 Panel with Opening Results - 40 ft

Table 7-3 and Table 7-4 reflect the most economical design for the total vertical reinforcement in the panel, including vertical reinforcement above and below the opening. Notably, for panels with openings, a double layer of reinforcement is used in all panel thicknesses to satisfy strength and serviceability requirements. As the opening size increases, more reinforcement is required in each panel leg. However, as the opening size increases, the section where the vertical reinforcement is to be placed is less likely to be tension-controlled. Therefore, the panel thickness needs to be increased. For large openings and high wind speeds, no design solution was achieved due to non-tension controlled behavior and therefore deeming ACI 318-08 Section 14.8 an invalid design approach in this case.

Through continued research, engineering of tilt-up concrete panels is becoming more exact and therefore yielding more economical design solutions. As a design engineer, safety is always of most importance. To ensure safe design of tilt-up panels, the Alternative Design of Slender Walls is widely practiced. ACI 318-08 Section 14.8 is appropriate for design when flexural tension controls the out-of-plane design of a wall.

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Appendix A - Sample Load Calculations

Building parameters are based off the Tilt-Up building from 2006 IBC Structural/Seismic Design Manual with modified plan dimensions. All gravity loads are based on a 24'-0" x 24'-0" bay, where the joists framing into tilt-up panel are at 4'-0" O.C.

Number of Joists framing into panel = 6

GRAVITY LOADS				REFERENCE
Roof:				
Dead Load				
	Bituminous Roofing =	1.5	psf	
	6" Rigid Insulation =	9	psf	
	1.5 22 Gauge Deck =	2	psf	
	Joists =	2.5	psf	
	M/E/P =	4	psf	
	Total =	19	psf	
	Use Dead Load =	20	psf	
Live Load (roof)	=	20	psf	ASCE 7-05 Table 4-1
(could be reduced per ASCE 7 Section 4.9)				
Tributary Area of Joist	=	96	sf	
Roof Axial Dead Load/Joist	=	0.96	k	
Roof Axial Live Load/Joist	=	0.96	k	
Total Roof Axial Dead Load	=	5.76	k	
Total Roof Axial Live Load	=	5.76	k	

SNOW LOAD				REFERENCE
Snow:				
Ground Snow Load, p_g	=	20	psf	ASCE 7-05 Figure 7-1
Flat Roof Snow Load, p_f	=	14	psf	ASCE 7-05 Section C6.5.6 (Exposure Category "C")
$p_f = 0.7{*}C_e C_t I p_g$				(7-1)
Exposure Factor, C_e	=	1.0		Table 7-2
Thermal Factor, C_t	=	1.0		Table 7-3
Importance Factor, I	=	1.0		Table 7-4
*For this study, it is assumed roof has a slope equal to or less than 5 degrees				

SNOW LOAD				REFERENCE
Minimum Snow Load				ASCE 7-05 Section 7.3
$p_g \le 20$ psf:				
$p_{fmin} = (I)p_g$	=	20	psf	
$p_g > 20$ psf:				
$p_{fmin} = 20(I)$	=		psf	
p_f =		20	psf	
For locations where p_g is 20 psf or less, but not zero, shall have a 5 psf rain-on-snow surcharge				ASCE 7-05 Section 7.10
Rain-on-Snow				
For this case, C_s =		1.0		ASCE 7-05 Figure 7-2
p_s =		19	psf	
Use Snow Load =		**20**	**psf**	
Axial Balanced Snow Load/Joist	=	0.96	k	
				ASCE 7-05 Section 7.7
Check Snow Drift - Transverse Direction only for this study				
ℓ_u = Length of roof upwind of drift =		168	ft	
Snow Density				
$\gamma = 0.13p_g + 14 \le 30$ pcf =		17	pcf	
Height of Balance Snow Load				
$h_b = p_f/\gamma$ =		1.20	ft	
Clear height from h_b to T.O.P				
h_c = Height of parapet - h_b =		0.80	ft	(2'-0" parapet)
DRIFT LOADS APPLY				
Height of Snow Drift				
h_d=		3.04	ft	ASCE 7-05 Figure 7-9
Max Intensity of Drift Surcharge				
$p_d = h_c\gamma$ =		13	psf	
Width of Snow Drift				
$w = 4h_d$ =		12.16	ft	
Axial Drift Snow Load/joist	=	0.321	k	
Total Axial Snow Load	=	7.69	k	
(includes balanced snow load and drift load)				

WIND LOAD	REFERENCE
(Wind Speeds of 90mph, 110mph, 130mph, and 150mph will be used)	
Method 2 - Analytical Procedure	ASCE 7-05 Section 6.5
Basic Wind speed V = 90 mph	ASCE 7-05 Figure 6-1
Directionality factor K_d = 0.85	ASCE 7-05 Table 6-4
Importance factor I = 1.00	
Occupancy Category II	ASCE 7-05 Table 1-1
Exposure Category C	ASCE 7-05 Section C6.5.6
Velocity Pressure Coefficient	
Roof Mean Height = 32 ft	Unbraced Lengths = 32' and 40'
K_z = 0.99	ASCE 7-05 Table 6-3
Topographic Factor K_{zt} = 1.0	ASCE 7-05 Section 6.5.7
Gust Factor	ASCE 7-05 Section 6.5.8
Fundamental Period, $T_a = C_t h^x{}_n$	ASCE 7-05 Equation 12.8-7
C_t = 0.016	
x = 0.9	
T_a = 0.362039	
Frequency, f = 2.76 Hz	
Rigid Structure > 1 Hz	
G = 0.85	
Enclosure Classification: Partially Enclosed	ASCE 7-05 Section 6.5.9
Internal Pressure Coefficient	
GC_{pi} = 0.55	ASCE 7-05 Figure 6-5
-0.55	
External Pressure Coefficient, GC_{pf}	
For Roof Angle Θ (degrees) 0-5	

1	2	3	4	5	6	1E	2E	3E	4E
0.4	-0.69	-0.37	-0.29	-0.45	-0.45	0.61	-1.07	-0.53	-0.43

Least Horizontal Dimension = 168 ft	
a = 12.8 ft	ASCE 7-05 Figure 6-10 note 9
Velocity Pressure, q_h	
$q_h = 0.00256 K_z K_{zt} K_d V^2 I$	ASCE 7-05 Equation 6-15
q_h = 17.45 psf	
Design Wind Load, p (Low-Rise Building)	ASCE 7-05 Section 6.2
Mean roof height ≤ 60ft	
Mean roof height < least horizontal dimension	
$p = q_h[GC_{pf} - GC_{pi}]$	ASCE 7-05 Equation 6-18

WIND LOAD	REFERENCE

	P, pressures (psf)	
	GC_{pi+}	GC_{pi-}
Zone 1	-2.62	16.58
Zone 2	-21.64	-2.44
Zone 3	-16.05	3.14
Zone 4	-14.66	4.54
Zone 5	-17.45	1.74
Zone 6	-17.45	1.74
Zone 1E	1.05	20.24
Zone 2E	-28.27	-9.07
Zone 3E	-18.85	0.35
Zone 4E	-17.10	2.09

ASCE 7-05 Figure 6-10

Components and Cladding-*Transverse Direction Only*

(Low-Rise Building)

ASCE 7-05 Section 6.2

$p = q_h[GC - GC_{pi}]$

ASCE 7-05 Equation 6-22

$q_h =$ 17.45 psf

ASCE 7-05 Equation 6-15

ASCE 7-05 Figure 6-11A

Area	4	5
GC_{p+}	0.7	0.7
GC_{p-}	-0.8	-0.8

Since all panel configurations have a square footage > 500SF, the pressure coefficients above are valid for all cases

The panel shall be designed for maximum positive and negative pressures, below are the different positive negative cases for partially enclosed:

$p = q_h(0.7 + 0.55) =$ 21.81 psf

$p = q_h(0.7 - 0.55) =$ 2.62 psf

$p = q_h(-0.8 + 0.55) =$ -4.36 psf

$p = q_h(-0.8 - 0.55) =$ -23.56 psf

Use Wind Pressure, w = -23.56 psf

Zone 2	-21.64	-2.44
Zone 3	-16.05	3.14
Zone 2E	-28.27	-9.07
Zone 3E	-18.85	0.35

Use MWFRS Wind Pressure for Uplift, w = -28.27 psf

Uplift Axial Load/joist = -1.36 k

Appendix B - MWFRS and C&C Wind Pressures

Mean Roof Height(h) = 32'-0"	Wind Speed, 90mph	Partially Enclosed P, pressures (psf)		Enclosed P, pressures (psf)	
		GC_{pi+}	GC_{pi-}	GC_{pi+}	GC_{pi-}
	Zone 1	-2.62	16.58	3.84	10.12
	Zone 4	-14.66	4.54	-8.20	-1.92
	Zone 1E	1.05	20.24	7.50	13.78
	Zone 4E	-17.10	2.09	-10.64	-4.36

(a)

Mean Roof Height(h) = 32'-0"	Wind Speed, 110mph	Partially Enclosed P, pressures (psf)		Enclosed P, pressures (psf)	
		GC_{pi+}	GC_{pi-}	GC_{pi+}	GC_{pi-}
	Zone 1	-3.91	24.76	5.73	15.12
	Zone 4	-21.90	6.78	-12.25	-2.87
	Zone 1E	1.56	30.24	11.21	20.59
	Zone 4E	-25.54	3.13	-15.90	-6.52

(b)

Mean Roof Height(h) = 32'-0"	Wind Speed, 130mph	Partially Enclosed P, pressures (psf)		Enclosed P, pressures (psf)	
		GC_{pi+}	GC_{pi-}	GC_{pi+}	GC_{pi-}
	Zone 1	-5.46	34.59	8.01	21.12
	Zone 4	-30.58	9.47	-17.11	-4.00
	Zone 1E	2.18	42.23	15.65	28.76
	Zone 4E	-35.68	4.37	-22.21	-9.10

(c)

Mean Roof Height(h) = 32'-0"	Wind Speed, 150mph	Partially Enclosed P, pressures (psf)		Enclosed P, pressures (psf)	
		GC_{pi+}	GC_{pi-}	GC_{pi+}	GC_{pi-}
	Zone 1	-7.27	46.05	10.66	28.11
	Zone 4	-40.72	12.60	-22.78	-5.33
	Zone 1E	2.91	56.23	20.84	38.29
	Zone 4E	-47.50	5.82	-29.57	-12.12

(d)

Table B-1 MWFRS Wind Pressures - 32ft

Mean Roof Height(h) = 40'-0"	Wind Speed, 90mph	Partially Enclosed		Enclosed	
		P, pressures (psf)		P, pressures (psf)	
		GC_{pi+}	GC_{pi-}	GC_{pi+}	GC_{pi-}
	Zone 1	-2.75	17.41	4.03	10.63
	Zone 4	-15.40	4.77	-8.62	-2.02
	Zone 1E	1.10	21.26	7.88	14.48
	Zone 4E	-17.96	2.20	-11.18	-4.58

(e)

Mean Roof Height(h) = 40'-0"	Wind Speed, 110mph	Partially Enclosed		Enclosed	
		P, pressures (psf)		P, pressures (psf)	
		GC_{pi+}	GC_{pi-}	GC_{pi+}	GC_{pi-}
	Zone 1	-4.11	26.01	6.02	15.88
	Zone 4	-23.00	7.12	-12.87	-3.01
	Zone 1E	1.64	31.76	11.77	21.63
	Zone 4E	-26.84	3.29	-16.70	-6.85

(f)

Mean Roof Height(h) = 40'-0"	Wind Speed, 130mph	Partially Enclosed		Enclosed	
		P, pressures (psf)		P, pressures (psf)	
		GC_{pi+}	GC_{pi-}	GC_{pi+}	GC_{pi-}
	Zone 1	-5.74	36.33	8.41	22.18
	Zone 4	-32.13	9.94	-17.98	-4.21
	Zone 1E	2.29	44.36	16.45	30.21
	Zone 4E	-37.48	4.59	-23.33	-9.56

(g)

Mean Roof Height(h) = 40'-0"	Wind Speed, 150mph	Partially Enclosed		Enclosed	
		P, pressures (psf)		P, pressures (psf)	
		GC_{pi+}	GC_{pi-}	GC_{pi+}	GC_{pi-}
	Zone 1	-7.64	48.37	11.20	29.53
	Zone 4	-42.77	13.24	-23.93	-5.60
	Zone 1E	3.06	59.07	21.89	40.23
	Zone 4E	-49.90	6.11	-31.06	-12.73

(h)

Table B-2 MWFRS Wind Pressures - 40 ft

Mean Roof Height (h) = 32'-0"	Partially Enclosed	
	Wind Speed (V)	C&C Wind Pressure
	90 mph	(-) 24 psf
	110 mph	(-) 36 psf
	130 mph	(-) 50 psf
	150 mph	(-) 66 psf

(a)

Mean Roof Height (h) = 32'-0"	Enclosed	
	Wind Speed (V)	C&C Wind Pressure
	90 mph	(-) 18 psf
	110 mph	(-) 26 psf
	130 mph	(-) 36 psf
	150 mph	(-) 48 psf

(b)

Mean Roof Height (h) = 40'-0"	Partially Enclosed	
	Wind Speed (V)	C&C Wind Pressure
	90 mph	(-) 25 psf
	110 mph	(-) 37 psf
	130 mph	(-) 52 psf
	150 mph	(-) 69 psf

(c)

Mean Roof Height (h) = 40'-0"	Enclosed	
	Wind Speed (V)	C&C Wind Pressure
	90 mph	(-) 18 psf
	110 mph	(-) 27 psf
	130 mph	(-) 38 psf
	150 mph	(-) 50 psf

(d)

Table B-3 C&C Wind Pressures

Appendix C - Load Combination Results

32'-0" (90mph)	Load Combination	P_{um} (k)	w_u (klf)	P_u/A_g (psi)	M_{cr} (k-ft)	A_s (in^2)	A_{se} (in^2)	c/d	ΦM_n (k-ft)	M_{ua} (k-ft)	M_u (k-ft)	Δ_s (in)	$\Delta_s < \Delta_{allow}$
7.25" Panel	1.2D+1.6(L_r or S)+0.8W	66.20	0.46	31.70	100.00	12.76	13.86	0.28	199.65	63.09	111.64	1.88	2.56
	1.2D+1.6W+0.5(L_r or S)	57.70	0.92	27.65	100.00	12.76	13.72	0.27	197.88	120.26	194.43	1.90	
	0.9D+1.6W	40.40	0.92	19.36	100.00	12.76	13.43	0.27	194.26	119.07	163.15	1.59	

32'-0" (110mph)	Load Combination	P_{um} (k)	w_u (klf)	P_u/A_g (psi)	M_{cr} (k-ft)	A_s (in^2)	A_{se} (in^2)	c/d	ΦM_n (k-ft)	M_{ua} (k-ft)	M_u (k-ft)	Δ_s (in)	$\Delta_s < \Delta_{allow}$
7.25" Panel	1.2D+1.6(L_r or S)+0.8W	66.20	0.69	31.70	100.00	8.80	9.53	0.13	223.28	92.58	116.91	2.49	2.56
	1.2D+1.6W+0.5(L_r or S)	57.70	1.38	27.65	100.00	8.80	9.43	0.12	221.23	179.24	219.37	2.52	
	0.9D+1.6W	40.40	1.38	19.36	100.00	8.80	9.24	0.12	217.01	178.05	204.67	2.43	

32'-0" (130mph)	Load Combination	P_{um} (k)	w_u (klf)	P_u/A_g (psi)	M_{cr} (k-ft)	A_s (in^2)	A_{se} (in^2)	c/d	ΦM_n (k-ft)	M_{ua} (k-ft)	M_u (k-ft)	Δ_s (in)	$\Delta_s < \Delta_{allow}$
7.25" Panel	1.2D+1.6(L_r or S)+0.8W	66.20	0.96	31.70	100.00	16.40	17.13	0.22	383.46	126.98	147.64	2.55	2.56
	1.2D+1.6W+0.5(L_r or S)	57.70	1.92	27.65	100.00	16.40	17.03	0.22	381.59	248.06	282.67	2.56	
	0.9D+1.6W	40.40	1.92	19.36	100.00	16.40	16.84	0.22	377.77	246.87	270.18	2.50	

32'-0" (150mph)	Load Combination	P_{um} (k)	w_u (klf)	P_u/A_g (psi)	M_{cr} (k-ft)	A_s (in^2)	A_{se} (in^2)	c/d	ΦM_n (k-ft)	M_{ua} (k-ft)	M_u (k-ft)	Δ_s (in)	$\Delta_s < \Delta_{allow}$
9.25" Panel	1.2D+1.6(L_r or S)+0.8W	79.20	1.27	29.71	162.00	10.80	11.61	0.11	373.36	167.11	187.12	1.97	2.56
	1.2D+1.6W+0.5(L_r or S)	70.70	2.53	26.54	162.00	10.80	11.53	0.11	370.71	327.15	361.93	1.99	
	0.9D+1.6W	50.10	2.53	18.82	162.00	10.80	11.32	0.11	364.24	325.73	349.92	1.97	

Table C-1 Solid Panels - 32 ft

40'-0" (90mph)	Load Combination	P_{um} (k)	w_u (klf)	P_u/A_g (psi)	M_{cr} (k-ft)	A_s (in^2)	A_{se} (in^2)	c/d	ΦM_n (k-ft)	M_{ua} (k-ft)	M_u (k-ft)	Δ_s (in)	$\Delta_s < \Delta_{allow}$
7.25" Panel	1.2D+1.6(L_r or S)+0.8W	76.60	0.48	36.70	100.00	15.40	16.24	0.21	365.62	100.10	135.52	3.11	3.20
	1.2D+1.6W+0.5(L_r or S)	68.20	0.96	32.65	100.00	15.40	16.15	0.21	363.73	194.30	253.43	3.12	
	0.9D+1.6W	48.20	0.96	23.11	100.00	15.40	15.93	0.21	359.28	193.11	231.69	3.00	

40'-0" (110mph)	Load Combination	P_{um} (k)	w_u (klf)	P_u/A_g (psi)	M_{cr} (k-ft)	A_s (in^2)	A_{se} (in^2)	c/d	ΦM_n (k-ft)	M_{ua} (k-ft)	M_u (k-ft)	Δ_s (in)	$\Delta_s < \Delta_{allow}$
9.25" Panel	1.2D+1.6(L_r or S)+0.8W	92.50	0.71	34.71	162.00	10.20	11.15	0.11	359.20	146.98	184.10	2.73	3.20
	1.2D+1.6W+0.5(L_r or S)	84.00	1.42	31.54	162.00	10.20	11.06	0.11	356.52	286.91	351.72	2.76	
	0.9D+1.6W	60.10	1.42	22.57	162.00	10.20	10.82	0.10	348.97	285.48	329.75	2.71	

40'-0" (130mph)	Load Combination	P_{um} (k)	w_u (klf)	P_u/A_g (psi)	M_{cr} (k-ft)	A_s (in^2)	A_{se} (in^2)	c/d	ΦM_n (k-ft)	M_{ua} (k-ft)	M_u (k-ft)	Δ_s (in)	$\Delta_s < \Delta_{allow}$
9.25" Panel	1.2D+1.6(L_r or S)+0.8W	92.50	1.00	34.71	162.00	16.40	17.35	0.17	544.07	204.58	239.44	3.14	3.20
	1.2D+1.6W+0.5(L_r or S)	84.00	2.00	31.54	162.00	16.40	17.26	0.17	541.55	402.11	463.64	3.15	
	0.9D+1.6W	60.10	2.00	22.57	162.00	16.40	17.02	0.16	534.43	400.68	443.19	3.12	

40'-0" (150mph)	Load Combination	P_{um} (k)	w_u (klf)	P_u/A_g (psi)	M_{cr} (k-ft)	A_s (in^2)	A_{se} (in^2)	c/d	ΦM_n (k-ft)	M_{ua} (k-ft)	M_u (k-ft)	Δ_s (in)	$\Delta_s < \Delta_{allow}$
9.25" Panel	1.2D+1.6(L_r or S)+0.8W	92.50	1.33	34.71	162.00	28.60	29.55	0.28	876.94	269.86	301.69	3.17	3.20
	1.2D+1.6W+0.5(L_r or S)	84.00	2.65	31.54	162.00	28.60	29.46	0.28	874.71	532.67	589.22	3.17	
	0.9D+1.6W	60.10	2.65	22.57	162.00	28.60	29.22	2.81	868.41	531.24	570.62	3.14	

Table C-2 Solid Panels - 40 ft

32'-0" (90mph)	Load Combination	P_{um} (k)	w_u (klf)	P_u/A_g (psi)	M_{cr} (k-ft)	A_s (in^2)	A_{se} (in^2)	c/d	ΦM_n (k-ft)	M_{ua} (k-ft)	M_u (k-ft)	Δ_s (in)	$\Delta_s < \Delta_{allow}$
4' x 4'	1.2D+1.6(L_r or S)+0.8W	32.7	0.23	37.54	42	3.2/face	3.56	0.112	83.9	31.54	43.1	1.82	2.56
(7.25"	1.2D+1.6W+0.5(L_r or S)	28.4	0.461	32.68	42	3.2/face	3.51	0.11	82.85	60.13	78.68	1.84	
Panel)	0.9D+1.6W	19.9	0.461	22.85	42	3.2/face	3.42	0.108	80.74	59.54	71.59	1.72	
8' x 8'	1.2D+1.6(L_r or S)+0.8W	31.4	0.23	45.05	33	3.2/face	3.54	0.139	82.53	31.54	43.37	2.45	
(7.25"	1.2D+1.6W+0.5(L_r or S)	27.1	0.461	38.98	33	3.2/face	3.5	0.138	81.51	60.13	78.93	2.48	
Panel)	0.9D+1.6W	18.9	0.461	27.16	33	3.2/face	3.41	0.134	79.54	59.54	71.66	2.35	
12' x 12'	1.2D+1.6(L_r or S)+0.8W	29.2	0.23	55.91	25	4.4/face	4.72	0.247	104.55	31.54	41.13	2.49	
(7.25"	1.2D+1.6W+0.5(L_r or S)	25	0.461	47.8	25	4.4/face	4.67	0.245	103.64	60.13	75.21	0.251	
Panel)	0.9D+1.6W	17.3	0.461	33.09	25	4.4/face	4.59	0.241	101.98	59.54	69.25	2.39	
16' x 16'	1.2D+1.6(L_r or S)+0.8W	30.7	0.23	69.14	27	2.2/face	2.52	0.145	79.66	31.94	40.16	2.05	
(9.25"	1.2D+1.6W+0.5(L_r or S)	26.5	0.461	59.61	27	2.2/face	2.47	0.143	78.38	60.36	73.48	2.08	
Panel)	0.9D+1.6W	18.4	0.461	41.46	27	2.2/face	2.39	0.138	75.91	59.64	68.35	2.01	

32'-0" (110mph)	Load Combination	P_{um} (k)	w_u (klf)	P_u/A_g (psi)	M_{cr} (k-ft)	A_s (in^2)	A_{se} (in^2)	c/d	ΦM_n (k-ft)	M_{ua} (k-ft)	M_u (k-ft)	Δ_s (in)	$\Delta_s < \Delta_{allow}$
4' x 4'	1.2D+1.6(L_r or S)+0.8W	32.7	0.346	37.54	42	5.4/face	5.76	0.181	131.56	46.29	57.11	2.48	2.56
(7.25"	1.2D+1.6W+0.5(L_r or S)	28.4	0.691	32.68	42	5.4/face	5.71	0.18	130.58	89.62	107.44	2.5	
Panel)	0.9D+1.6W	19.9	0.691	22.85	42	5.4/face	5.62	0.177	128.61	89.03	100.85	2.42	
8' x 8'	1.2D+1.6(L_r or S)+0.8W	31.4	0.346	45.05	33	7/face	7.34	0.289	159.47	46.29	55.94	2.53	
(7.25"	1.2D+1.6W+0.5(L_r or S)	27.1	0.691	38.98	33	7/face	7.3	0.287	158.6	89.62	105.41	2.54	
Panel)	0.9D+1.6W	18.9	0.691	27.16	33	7/face	7.21	0.283	156.9	89.03	99.48	2.46	
12' x 12'	1.2D+1.6(L_r or S)+0.8W	29.2	0.346	55.91	41	3/face	3.36	0.129	107.04	46.69	56.08	2.15	
(9.25"	1.2D+1.6W+0.5(L_r or S)	25	0.691	47.8	41	3/face	3.31	0.127	105.73	89.85	105.5	2.18	
Panel)	0.9D+1.6W	17.3	0.691	33.09	41	3/face	3.22	0.124	102.93	89.14	99.76	2.13	
16' x 16'	1.2D+1.6(L_r or S)+0.8W	30.7	0.346	69.14	27	3.4/face	3.72	0.214	113.93	46.69	55.44	2.56	
(9.25"	1.2D+1.6W+0.5(L_r or S)	26.5	0.691	59.61	27	3.4/face	3.67	0.212	112.78	89.85	104.13	2.58	
Panel)	0.9D+1.6W	18.4	0.691	41.46	27	3.4/face	3.59	0.207	110.48	89.14	98.68	2.52	

Table C-3 Panel with Openings - 32 ft

32'-0" (130mph)	Load Combination	P_{um} (k)	w_u (klf)	P_u/A_g (psi)	M_{cr} (k-ft)	A_s (in^2)	A_{se} (in^2)	c/d	ΦM_n (k-ft)	M_{ua} (k-ft)	M_u (k-ft)	Δ_s (in)	$\Delta_s < \Delta_{allow}$
4' x 4'	1.2D+1.6(L_r or S)+0.8W	32.7	0.48	37.54	42	10.6/face	10.96	0.345	231.5	63.49	73.05	2.55	2.56
(7.25"	1.2D+1.6W+0.5(L_r or S)	28.4	0.96	32.68	42	10.6/face	10.91	0.343	230.68	124.03	140.02	2.55	
Panel)	0.9D+1.6W	19.9	0.96	22.85	42	10.6/face	10.82	0.34	229.03	123.43	134.2	2.5	
8' x 8'	1.2D+1.6(L_r or S)+0.8W	37.4	0.48	42.07	54	4.2/face	4.58	0.132	146.02	63.89	73.71	2.15	
(9.25"	1.2D+1.6W+0.5(L_r or S)	33.1	0.96	37.31	54	4.2/face	4.54	0.131	144.71	124.25	141.03	2.17	
Panel)	0.9D+1.6W	23.4	0.96	26.36	54	4.2/face	4.44	0.128	141.71	123.54	135.08	2.15	
12' x 12'	1.2D+1.6(L_r or S)+0.8W	34.6	0.48	51.93	41	4.4/face	4.76	0.183	148.02	63.89	73.53	2.51	
(9.25"	1.2D+1.6W+0.5(L_r or S)	30.4	0.96	45.58	41	4.4/face	4.71	0.181	146.78	124.25	140.52	2.52	
Panel)	0.9D+1.6W	21.3	0.96	32.02	41	4.4/face	4.62	0.178	144.13	123.54	134.63	2.49	
16' x 16'	1.2D+1.6(L_r or S)+0.8W	35.3	0.48	65.29	40	3.2/face	3.55	0.162	141.26	64.29	72.01	2.01	
(11.25"	1.2D+1.6W+0.5(L_r or S)	31	0.96	57.46	40	3.2/face	3.51	0.16	139.72	124.48	137.56	2.03	
Panel)	0.9D+1.6W	21.8	0.96	40.43	40	3.2/face	3.42	0.156	136.36	123.65	132.69	2.01	

32'-0" (150mph)	Load Combination	P_{um} (k)	w_u (klf)	P_u/A_g (psi)	M_{cr} (k-ft)	A_s (in^2)	A_{se} (in^2)	c/d	ΦM_n (k-ft)	M_{ua} (k-ft)	M_u (k-ft)	Δ_s (in)	$\Delta_s < \Delta_{allow}$
4' x 4'	1.2D+1.6(L_r or S)+0.8W	39	0.634	35.16	68	5.4/face	5.8	0.134	184.65	83.55	93.91	2.26	2.56
(9.25"	1.2D+1.6W+0.5(L_r or S)	34.8	1.267	31.35	68	5.4/face	5.76	0.133	183.35	163.57	181.53	2.27	
Panel)	0.9D+1.6W	24.7	1.267	22.21	68	5.4/face	5.65	0.13	180.23	162.86	175.32	2.25	
8' x 8'	1.2D+1.6(L_r or S)+0.8W	37.4	0.634	42.07	54	5.6/face	5.98	0.173	187.15	83.55	93.92	2.52	
(9.25"	1.2D+1.6W+0.5(L_r or S)	33.1	1.267	37.31	54	5.6/face	5.94	0.171	185.9	163.57	181.41	2.54	
Panel)	0.9D+1.6W	23.4	1.267	26.36	54	5.6/face	5.84	0.168	183.01	162.86	175.17	2.52	
12' x 12'	1.2D+1.6(L_r or S)+0.8W	34.6	0.634	51.93	41	7.6/face	7.96	0.306	233.59	83.55	92.53	2.49	
(9.25"	1.2D+1.6W+0.5(L_r or S)	30.4	1.267	45.58	41	7.6/face	7.91	0.304	232.51	163.57	178.85	2.5	
Panel)	0.9D+1.6W	21.3	1.267	32.02	41	7.6/face	7.82	0.301	230.18	162.86	173.33	2.47	
16' x 16'	1.2D+1.6(L_r or S)+0.8W	35.3	0.634	65.29	40	4.4/face	4.75	0.216	184.33	83.95	92.13	2.27	
(11.25"	1.2D+1.6W+0.5(L_r or S)	31	1.267	57.46	40	4.4/face	4.71	0.214	182.87	163.8	177.76	2.28	
Panel)	0.9D+1.6W	21.8	1.267	40.43	40	4.4/face	4.62	0.21	179.69	162.97	172.62	2.26	

Table C-4 Panel with Openings - 32 ft

40'-0" (90mph)	Load Combination	P_{um} (k)	w_u (klf)	P_u/A_g (psi)	M_{cr} (k-ft)	A_s (in^2)	A_{se} (in^2)	c/d	ΦM_n (k-ft)	M_{ua} (k-ft)	M_u (k-ft)	Δ_s (in)	$\Delta_s < \Delta_{allow}$
4' x 4' (9.25" Panel)	1.2D+1.6(L_r or S)+0.8W	37.9	0.24	43.54	42	10.6/face	11.02	0.347	232.5	50.05	65.56	3.09	3.20
	1.2D+1.6W+0.5(L_r or S)	33.7	0.48	38.68	42	10.6/face	10.97	0.345	231.68	97.15	123.07	3.1	
	0.9D+1.6W	23.8	0.48	27.35	42	10.6/face	10.86	0.342	229.8	96.55	113.55	2.99	
8' x 8' (9.25" Panel)	1.2D+1.6(L_r or S)+0.8W	44	0.24	49.57	54	3.8/face	4.25	0.123	136.04	50.45	68.13	2.65	
	1.2D+1.6W+0.5(L_r or S)	39.8	0.48	44.81	54	3.8/face	4.21	0.121	134.72	97.37	127.51	2.68	
	0.9D+1.6W	28.4	0.48	31.98	54	3.8/face	4.09	0.118	131.18	96.66	116.79	2.61	
12' x 12' (9.25" Panel)	1.2D+1.6(L_r or S)+0.8W	41.2	0.24	61.93	41	4.2/face	4.62	0.178	144.27	50.45	67.16	3.14	
	1.2D+1.6W+0.5(L_r or S)	37	0.48	55.58	41	4.2/face	4.58	0.176	143.02	97.37	125.6	3.16	
	0.9D+1.6W	26.3	0.48	39.52	41	4.2/face	4.47	0.172	139.86	96.66	115.4	3.08	
16' x 16' (11.25" Panel)	1.2D+1.6(L_r or S)+0.8W	43.4	0.24	80.29	40	2.8/face	3.23	0.147	129.37	50.85	65.2	2.65	
	1.2D+1.6W+0.5(L_r or S)	39.1	0.48	72.46	40	2.8/face	3.19	0.145	127.81	97.6	122.07	2.68	
	0.9D+1.6W	27.9	0.48	51.68	40	2.8/face	3.08	0.14	123.65	96.77	113.4	2.64	

40'-0" (110mph)	Load Combination	P_{um} (k)	w_u (klf)	P_u/A_g (psi)	M_{cr} (k-ft)	A_s (in^2)	A_{se} (in^2)	c/d	ΦM_n (k-ft)	M_{ua} (k-ft)	M_u (k-ft)	Δ_s (in)	$\Delta_s < \Delta_{allow}$
4' x 4' (7.25" Panel)	1.2D+1.6(L_r or S)+0.8W	45.7	0.355	41.16	68	5.2/face	5.67	0.131	180.71	73.5	92.47	3.17	3.20
	1.2D+1.6W+0.5(L_r or S)	41.5	0.7104	37.35	68	5.2/face	5.63	0.13	179.41	143.45	176.51	3.19	
	0.9D+1.6W	29.6	0.7104	26.71	68	5.2/face	5.5	0.127	175.76	142.74	165.24	3.15	
8' x 8' (9.25" Panel)	1.2D+1.6(L_r or S)+0.8W	44	0.355	49.57	54	6.4/face	6.85	0.198	211.85	73.5	90.29	3.18	
	1.2D+1.6W+0.5(L_r or S)	39.8	0.7104	44.81	54	6.4/face	6.81	0.196	210.63	143.45	172.59	3.19	
	0.9D+1.6W	28.4	0.7104	31.98	54	6.4/face	6.69	0.193	207.33	142.74	162.54	3.14	
12' x 12' (9.25" Panel)	1.2D+1.6(L_r or S)+0.8W	48.1	0.355	59.36	60	4/face	4.47	0.136	180.24	73.89	89.27	2.62	
	1.2D+1.6W+0.5(L_r or S)	43.9	0.7104	54.14	60	4/face	4.43	0.135	178.66	143.68	170.68	2.65	
	0.9D+1.6W	31.4	0.7104	38.83	60	4/face	4.31	0.131	174.02	142.85	161.56	2.63	
16' x 16' (11.25" Panel)	1.2D+1.6(L_r or S)+0.8W	43.4	0.355	80.29	40	4.4/face	4.83	0.22	187.11	73.89	88.9	3.17	
	1.2D+1.6W+0.5(L_r or S)	39.1	0.7104	72.46	40	4.4/face	4.79	0.218	185.66	143.68	169.67	3.19	
	0.9D+1.6W	27.9	0.7104	51.68	40	4.4/face	4.68	0.213	181.79	142.85	160.65	3.16	

Table C-5 Panel with Openings - 40 ft

40'-0" (130mph)	Load Combination	P_{um} (k)	w_u (klf)	P_u/A_g (psi)	M_{cr} (k-ft)	A_s (in^2)	A_{se} (in^2)	c/d	ΦM_n (k-ft)	M_{ua} (k-ft)	M_u (k-ft)	Δ_s (in)	$\Delta_s < \Delta_{allow}$
4' x 4' (9.25" Panel)	1.2D+1.6(L_r or S)+0.8W	45.7	0.5	41.16	68	10/face	10.47	0.242	317.08	102.29	118.51	3.13	3.20
	1.2D+1.6W+0.5(L_r or S)	41.5	0.998	37.35	68	10/face	10.43	0.241	315.91	201.05	229.64	3.14	
	0.9D+1.6W	29.6	0.998	26.71	68	10/face	10.3	0.238	312.65	200.34	220.07	3.1	
8' x 8' (11.25" Panel)	1.2D+1.6(L_r or S)+0.8W	51.5	0.5	47.65	80	5.4/face	5.91	0.134	238.12	102.7	119.31	2.76	
	1.2D+1.6W+0.5(L_r or S)	47.2	0.998	43.73	80	5.4/face	5.87	0.134	236.54	201.28	230.95	2.78	
	0.9D+1.6W	34	0.998	31.46	80	5.4/face	5.74	0.131	231.58	200.45	221.25	2.77	
12' x 12' (11.25" Panel)	1.2D+1.6(L_r or S)+0.8W	48.1	0.5	59.36	60	6/face	6.47	0.197	253.67	102.69	118.55	3.04	
	1.2D+1.6W+0.5(L_r or S)	43.9	0.998	54.14	60	6/face	6.43	0.195	252.18	201.28	229.38	3.06	
	0.9D+1.6W	31.4	0.998	38.83	60	6/face	6.31	0.192	247.81	200.45	220.02	3.04	
16' x 16'	1.2D+1.6(L_r or S)+0.8W												
	1.2D+1.6W+0.5(L_r or S)												
	0.9D+1.6W												

40'-0" (150mph)	Load Combination	P_{um} (k)	w_u (klf)	P_u/A_g (psi)	M_{cr} (k-ft)	A_s (in^2)	A_{se} (in^2)	c/d	ΦM_n (k-ft)	M_{ua} (k-ft)	M_u (k-ft)	Δ_s (in)	$\Delta_s < \Delta_{allow}$
4' x 4' (11.25" Panel)	1.2D+1.6(L_r or S)+0.8W	53.5	0.662	39.62	100	7/face	7.53	0.137	303.06	135.33	152.78	2.89	3.20
	1.2D+1.6W+0.5(L_r or S)	49.3	1.325	36.48	100	7/face	7.49	0.136	301.5	266.56	298.02	2.9	
	0.9D+1.6W	35.5	1.325	26.3	100	7/face	7.35	0.134	296.35	265.73	287.94	2.9	
8' x 8' (11.25" Panel)	1.2D+1.6(L_r or S)+0.8W	51.5	0.662	47.65	80	7.6/face	8.11	0.185	319.42	135.33	152.36	3.09	
	1.2D+1.6W+0.5(L_r or S)	47.2	1.325	43.73	80	7.6/face	8.07	0.184	317.41	266.56	297.14	3.11	
	0.9D+1.6W	34	1.325	31.46	80	7.6/face	7.94	0.181	313.49	265.73	287.23	3.09	
12' x 12' (11.25" Panel)	1.2D+1.6(L_r or S)+0.8W	48.1	0.662	59.36	60	10.2/face	10.67	0.324	393.5	135.33	150.47	3.1	
	1.2D+1.6W+0.5(L_r or S)	43.9	1.325	54.14	60	10.2/face	10.63	0.323	392.2	266.56	293.56	3.1	
	0.9D+1.6W	31.4	1.325	38.83	60	10.2/face	10.51	0.319	388.4	365.73	284.62	3.08	
16' x 16'	1.2D+1.6(L_r or S)+0.8W												
	1.2D+1.6W+0.5(L_r or S)												
	0.9D+1.6W												

Table C-6 Panel with Openings - 40 ft

Appendix D - Reprint Image/Figure Permission

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ACI 318-08 Building Code Requirements for Structural Concrete and Commentary

From: Daniela.Bedward@concrete.org
To: Brian Bartels bbartels@ksu.edu
Date: Wed, Mar 31, 2010 at 11:42 AM
Subject:Re: ACI 318-08

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Please feel free to contact me if you need further assistance.

Have a great day,

Daniela

ACI 551 Design Guide for Tilt-Up Concrete Panels

From: JGriffin@ljbinc.com
To: bbartels@ksu.edu
Date: Tue, Mar 16, 2010 at 6:22 AM
Subject:Re: KSU Architectural Engg Masters Report --Brian Bartels

Brian,

Please find attached the two figures you requested in Word format.

Thank you,

Jeff

PCA Notes on ACI 318-08 Building Code Requirements for Structural Concrete

From: Rabbat, Basile brabbat@cement.org
To: Brian Bartels <bbartels@ksu.edu>
cc: "Novak, Larry" <lnovak@cement.org>
Date: Tue, Apr 6, 2010 at 10:00 AMsubject
Subject:RE: PCA Notes on ACI 318-08

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Best wishes with your report,

Basile Rabbat

Design of Reinforced Concrete ACI 318-05 Code Edition, 7th Edition

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