Study of multistory tilt-up with varying parameters

by

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Abstract

Historically, tilt-up concrete construction has been confined to large, single-story warehouses and industrial buildings with a large footprint and few openings. Now, however, given the speed of construction, durability, and economics of tilt-up construction, a demand for selecting tilt-up concrete as a structural system for multistory office, retail, and apartment buildings is on the rise.

This report provides an overview of the effect of the construction location, reinforcing steel bar size, reinforcing steel yield strength, number of layers of steel, and number of stories on the design of multistory tilt-up using the slender wall method, when out-of-plane wind loading and gravity loads are applied. An exterior multistory panel is assessed to determine the effect of sundry panel designs for loadbearing walls in multistory tilt-up construction. A two-story, three-story, and four-story panel with a consistent thickness, width, and concrete strength in adjunction to varying reinforcement size, reinforcement strength, layers of reinforcement, and out-of-plane loading is analyzed to determine the impact the design has on the panel.

Two-story, three-story, and four-story panels with a floor-to-floor height of 14 feet are used in the parametric study done in this report. All panels have a width of 15 feet and concrete compressive strength of 4,000 psi. The panel thickness is used in conjunction with varying steel reinforcement sizes of a #5 and #6 bar as well as strengths of 60 ksi and 80 ksi.

Based on the results found in the parametric study it was concluded that using the alternative method for slender wall analysis is only valid for two- or three-story buildings when normal weight concrete is utilized. It is also apparent that using the same number of bars in a doubly reinforced panel as opposed to a singly reinforced panel resulted in lesser moments and out-of-plane deflections.

Further research should be done to analyze how designing multistory tilt-up as a column with varying parameters affects the panel. This study showed that regardless of the applied lateral loads and the amount of reinforcement placed within the panel, the four-story panel design was invalid and should be designed as a column. Increasing the concrete strength may result in a valid panel design and should also be further investigated.

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ACI = American Concrete Association

ASCE = American Society of Civil Engineers

A_g = gross area of concrete section

A_{se} = effective cross-sectional area of steel reinforcement

b = width of compression face of panel

c = distance from extreme compression fiber to neutral axis

C_e = exposure factor

C_t = thermal factor

d = distance from extreme compression fiber to centroid of longitudinal reinforcement

D = effect of service dead load

e_{cc} = eccentricity of applied loads

E = effect of horizontal and vertical earthquake-induced forces

E_c = modulus of elasticity of concrete

f'_c = specified compressive strength of concrete

f_r = modulus of rupture of concrete

f_y = specified yield strength for reinforcement

Gc_p = external pressure coefficient

Gc_{pi} = internal pressure coefficient

h = overall height of panel

h_c = clear height from top of balanced snow load to top of parapet

h_d = height of snow drift

h_p = height of parapet

I_{cr} = moment of inertia of cracked section

I_g = moment of inertia of gross concrete section about centroidal axis

I_s = snow importance factor

k = effective length factor

K = modulus of rupture factor

K_b = bending stiffness

K_d = wind directionality factor

K_e = ground elevation factor

K_z = velocity pressure exposure coefficient

K_{zt} = topographic factor

l = vertical span of member between supports

l_c = cantilever height

l_u = length of roof upwind of snow drift

l_u = unbraced length of panel

l_w = length of panel

L = effect of service live load

L_r = effect of service live load

mph = miles per hour

M_A = maximum moment in member due to service loads, where deflection is calculated

M_{cr} = cracking moment

M_n = nominal flexure flexural strength at section

M_{SA} = maximum moment in panel due to service loads, excluding P-Δ effects

M_u = factored moment at section

M_{ua} = moment at midheight of panel due to factored lateral and eccentric vertical loads

p = design wind pressure

p_g = ground snow load

p_m = minimum snow load for low-sloped roofs

P_{SA} = initial service applied axial load

psf = pounds per square foot

P_u = factored axial force

P_{ua} = axial load applied to the panel

P-Δ = secondary moment due to lateral deflection

q_h = wind velocity pressure evaluated at height z = h

R = cumulative load effect of service rain load

S = effect of service snow load

SEAOSC = Structural Engineers Association of Southern California

t = thickness of panel

V = basic wind speed

w_s = service uniform lateral load on panel

w_u = factored uniform lateral load on panel

W = effect of wind load

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

γ_c = unit weight of concrete

Δ_{cr} = calculated out-of-plane deflection at midheight of the panel corresponding to M_{cr}

Δ_n = calculated out-of-plane deflection at midheight of the panel corresponding to M_n

Δ_s = out-of-plane deflection due to service loads

Δ_u = calculated out-of-plane deflection at midheight of the panel due to factored loads

λ = modification factor

ρ_l = minimum longitudinal steel

ρ_t = minimum transverse steel

Φ = strength reduction factor

Acknowledgements

A million thanks to my homies that have helped me get to this point; encouraging, helping, and keeping me sane along the way.

I would also like to thank my major professor Kimberly Kramer for her contributions, guidance, and support in the composition of this report. Additionally, to the members of my graduate committee, Bill Zhang and Shannon Casebeer for their guidance and flexibility.

Dedication

I dedicate this report to my father, Todd Mills, for being my inspiration to do the impossible and teaching me that hard work and determination always pay off in the end – even when there are obstacles and road bumps along the way.

And to my mother, Anne Mills, for instilling in me her grit and work ethic that made completing this report possible. And for always answering my daily phone calls.

Chapter 1 – Introduction

Historically, tilt-up concrete construction has been confined to large, single-story warehouses and industrial buildings with a large footprint and few openings. Now, however, given the speed of construction, durability, and economics of tilt-up construction, multistory office, retail, and apartment buildings are beginning to have more demand. These multistory buildings are, for the most part, being constructed in southern environments where the warmer climate allows for construction schedules to be met. The small window of opportunity in northern climates prevents tilt-up construction from being the method of choice, as construction schedules are difficult to meet and the extreme cold creates an increase in cost due to the additional expense of cold weather concreting.

This report provides an overview of the effect of the construction location, reinforcing steel bar size, reinforcing steel yield strength, number of layers of steel, and number of stories on the design of multistory tilt-up when out-of-plane wind loading and gravity loads are applied. An exterior multistory panel was assessed to determine the effect of sundry panel designs for loadbearing walls in multistory tilt-up construction. A two-story, three-story, and four-story panel with a consistent thickness, width, and concrete strength in conjunction to varying reinforcement size, reinforcement strength, layers of reinforcement, and out-of-plane loading were analyzed to determine the impact the design has on the panel.

Two-story, three-story, and four-story panels with a floor-to-floor height of 14 feet were used in the parametric study and can be seen in the sections shown in Figure 1. The pinned supports represent the floor system and are used instead of springs to simplify the design such that no horizontal drift is considered at these locations. All panels had a width of 15 feet and concrete compressive strength of 4,000 psi. The panel thickness was used in conjunction with varying steel reinforcement sizes of a #5 and #6 bar as well as strengths of 60 ksi and 80 ksi.

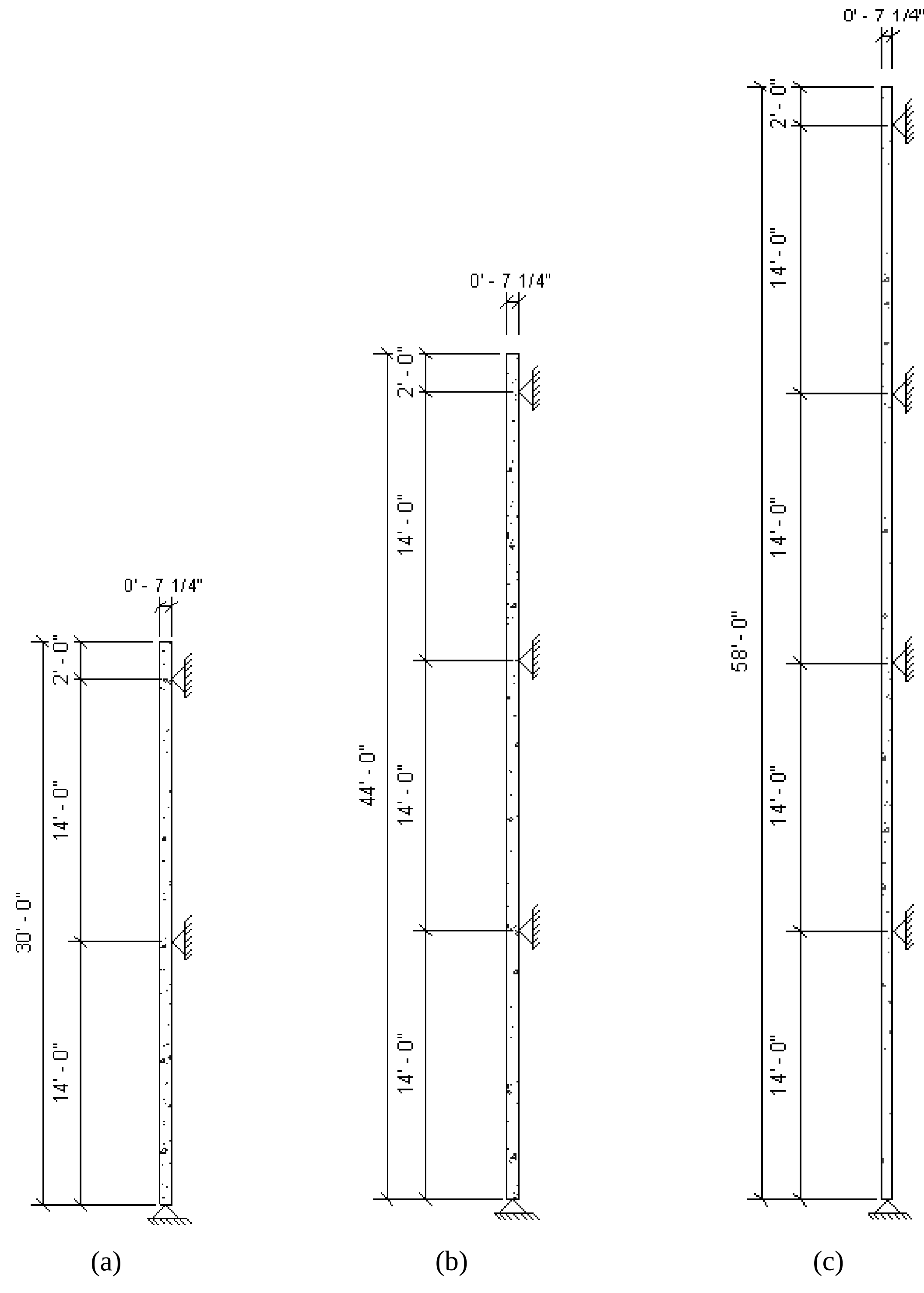

Figure 1: (a) Section of typical 2-story panel, (b) Section of typical 3-story panel, and (c) Section of typical 4-story panel

1.1 Objective

The purpose of this report is to provide a design comparison for multistory tilt-up panels when out-of-plane wind and gravity loads are applied. The comparison is based on three construction locations, two reinforcing steel bar sizes, two reinforcement strengths, three panel heights, and one and two layers of steel with a constant concrete strength, panel width, and panel thickness. In total, 72 design combinations are assessed.

1.2 Scope of Report

This report contains five chapters. Chapter one provides the introduction, objective, and scope of the report. Chapter two presents the paucity of current research available on multistory tilt-up construction in way of a literature review on "Loadbearing Architectural Precast Concrete Wall Panels" by Sidney Freedman, "Investigation of Reinforced Concrete Wall" by Gerold D. Oberlander and Noel J. Everard, and "Interesting Aspects of the Empirical Wall Design Equation" by K.M. Kripanarayanan. Chapter three describes the parametric study in which the building conditions, panel parameters, and applied loading are discussed. Chapter four presents the results found from the parametric study and discusses the findings of the design comparison. Chapter five details the findings of the tilt-up panel design comparison and conclusions of the study.

Chapter 2 – Literature Review

The ACI describes tilt-up as "a construction technique for casting concrete elements in a horizontal position at the job site and then tilting them to their final position in a structure." As tilt-up is a "construction technique" what is done in the industry today is based upon what has worked in the past. Since tilt-up first began in the early 1900's, this method of construction has undergone numerous innovations and refinements to develop into the process that is now used in the construction industry. Despite the advancement of industry practices, very little lab testing has been conducted on the tilt-up construction method. Around the same time tilt-up was gaining popularity, precast concrete was beginning to be widely used. While similar to tilt-up in many aspects, particularly in how they are designed, precast concrete, however, is cast in a reusable form, cured in a controlled environment and then transported from the manufacturer to the construction site. Considering the similarity of design of the two methods and how little research has been done on tilt-up construction itself, the testing done on precast walls will be presented and reviewed in this chapter.

2.1 Research on the Empirical Method

Garold D. Oberlander and Noel J. Everard conducted an experimental research investigation of precast reinforced concrete loadbearing walls between the years of 1971 and 1973. Fifty-four precast concrete walls were tested to failure and the results of testing were compared to the analytical methods prescribed by ACI 318-71. The intent of the testing was to "determine the strength of reinforced concrete load-bearing wall panels with respect to geometric configuration (and loading conditions)." Oberlander and Everard's research has provided experimental information on the ultimate strength and structural behavior of reinforced concrete loadbearing panels. The results from the testing were used to verify the empirical method presented in Chapter 14 and the method of designing walls as columns presented in Chapter 10 of ACI 318-71, and to make recommendations for modifications to the analytical methods. This report is focused on the usage of the empirical method as an approach to analyze slender walls, and therefore the results of Oberlander and Everard's research will be reviewed in this chapter. Oberlander and Everard's research has been a building block for further research and discussion. K. M. Kripanarayanan discussed his thoughts on the empirical formula and further refined it.

2.1.1 Oberlander & Everard 1977 *Investigation of Reinforced Concrete Walls*

In Oberlander and Everard's research, 54 concrete wall panels with a constant thickness of three inches were tested to failure with varying heights, amount of reinforcement, location of reinforcement, and loading conditions. The structural behavior of the panels was observed by measuring axial deflections, lateral deflections, and surface strains. The panels were formed per the requirements of ACI 318-71, and were used to predict the structural behavior and ultimate strength of walls used in practice at the time of testing.

At the time, ACI 318 required a minimum thickness of a precast panel to be six inches, so a panel thickness of three inches was chosen as a half-scale model. Presently, ACI 318 requires the minimum thickness of bearing walls designed in accordance with the simplified design method to be the greater of four inches or one twenty-fifth the lesser of unsupported length and unsupported height (ACI 318-19: Table 11.3.1.1 – Minimum wall thickness *h*). To provide a range of height-to-thickness ratios from eight to 28, panel heights of two to seven feet were used in one foot increments. When using the empirical method of ACI 318-71 Chapter 14, a height-to-wall thickness ratio was limited to 25, however, as the method of designing walls as columns under Chapter 10 of the same code did not limit the ratio, the ratio 28 was still used. ACI 318-19 does not specify a limit to the height-to-wall thickness as the minimum wall thickness accounts for the unbraced length.

Smooth round welded wire fabric (4 x 4-6 x 6 and 4 x 4-8 x 8) were used as the two sizes of reinforcement in the study. Two layers of reinforcement with a 5/8-inch clear cover were placed in the majority of the panels. The #6-gauge reinforcement resulted in an area of steel to gross area of concrete ratio of 0.0047. The #8-gauge reinforcement resulted in a ratio of 0.0033. These A_s/A_y ratios satisfied the requirements set by Chapter 14 of ACI 318-71, and were selected to "obtain test results in the usual ranges of reinforcement used in wall design." Six of the 54 panels were cast with a 3/8-inch clear cover on the steel instead of 5/8-inch to determine the effect that location has on the reinforcement. Two panels of heights three, five, and seven feet made up these six panels, and they were constructed with only the #8-gauge reinforcement. Eccentric loading conditions were applied to one of the panels of each height, and concentric loading was applied to the other.

Tensile tests were done on six samples of each of the #6- and #8-gauge reinforcements, conforming to ASTM A185 specifications. The tensile tests resulted in tensile strengths varying from 73 ksi to 86 ksi. The concrete compressive strength was also tested. Cylinder tests were done on each wall mix and the concrete strength determined was used in the analysis. The compressive strength of the concrete varied between 4,000 and 6,000 psi, which was within the industry wall design norms.

A uniformly distributed axial load was applied to all panels in 10 kip increments until destruction. The load was applied concentrically along the centroidal axis on half of the panels and eccentrically for the other half. The eccentric load was applied 1/2-inch from the centroidal axis in the transverse direction. The ultimate load of each panel was obtained as a dimensionless quantity so it could be compared with the empirical design method of analysis presented in ACI 318-71 Chapter 14 and the method of designing walls as columns presented in ACI 318-71 Chapter 10.

Testing data showed that for height-to-thickness ratios of eight through 12 the ultimate failure loads were comparably greater than the values obtained using the empirical equation. The testing data for height-to-thickness ratios between 16 and 20 coincided with the values obtained from the empirical equation. A strength reduction factor, ϕ, of 0.7 was applied per ACI 318-71: Section 14.2. Presently, when using the empirical method, a strength reduction factor of 0.65 is used. Panels with a height-to-thickness ratio exceeding 20 showed test failure loads less than the empirical equation values when the ϕ factor is not applied, but failure loads were greater than values obtained from the equation when the factor was applied. Furthermore, panels with height-to-thickness ratios greater than 16 exhibited buckling failures when eccentric loads were applied. Tilt-up-panels with height-to-thicknesses greater than 16 have since been shown to perform without failing when eccentric loads are applied. Two-story panels have a recommended height-to-thickness ratio of l'/30, three-story panels are recommended to be l'/28, and four-story panels are recommended to be l'/26 (*Design of Multi-Story Tilt-Up Panels for Out-of-Plane Wind Loads: White Paper*, 2018). These height-to-thickness ratios used for tilt-up are meant to be used as a general guideline based on what has been successful in the past to perform preliminary calculations and are not required by code.

It was concluded that when the overstrength factor is applied to the empirical equation, the equation results in lesser loads than all the test load results and is therefore adequate. However, the panels in the test had considerably higher ratios of area of steel to concrete (0.0033 and 0.0047) compared to the required minimum of 0.0015 by ACI 318-71 Chapter 14. Had smaller ratios been used in the testing, failures could be expected to exceed the empirical equation values for large height-to-thickness ratios (24 to 28).

The equation:

$$P_u = 0.60\varphi f'_c bh\left[1-\left(\frac{l}{30h}\right)^2\right] \qquad \text{Eq. 2-1}$$

was derived by Oberlander and Everard's research results to predict the ultimate failure load of reinforced concrete panels. In order to more accurately represent the ultimate failure load of the panel, the equation was modified from the one presented in the ACI 318-71 code which was:

$$P_u = 0.55\phi f'_c bh\left[1-\left(\frac{l}{40h}\right)^2\right] \qquad \text{Eq. 2-2}$$

The modified equation was limited to use on walls with two rows of reinforcement as all the panels tested had two layers of reinforcement. Further testing has since been done and the equation developed in this research has been modified to ACI 318-19: Equation 11.5.3.1, shown as Eq. 2-3 here:

$$P_n = 0.55 f'_c A_g\left[1-\left(\frac{kl_c}{32h}\right)^2\right] \qquad \text{Eq. 2-3}$$

Setting the design nominal axial compressive strength, ϕP_n, to the ultimate axial load, P_u, results in the equation:

$$P_u = 0.55\phi f'_c A_g\left[1-\left(\frac{kl_c}{32h}\right)^2\right] \qquad \text{Eq. 2-4}$$

2.1.2 Kripanarayanan 1977 *Interesting Aspects of the Empirical Equation*

A few years after Oberlander and Everard's testing had concluded, K. M. Kripanarayanan in 1977 provided an evaluation of the empirical wall design equation. Kripanarayanan dissects the equation given in ACI 1971 (Eq. 2-4) to determine the factored vertical load on a load bearing wall. The equation is considered as a product of two functions:

$$\frac{P_u}{\phi f'_c bh} = F_1 \times F_2 \qquad \text{Eq. 2-5}$$

where:

F_1 = 0.55, and is a function of eccentricity,

$F_2 = [1 - (l_u/40h)^2]$, and is a function of slenderness.

Interaction diagrams were developed for walls with thicknesses of eight, 10, and 12 inches to evaluate the effect of eccentric loading on the wall. The walls contained 0-1% of reinforcement to gross area of the section, which was found to not have a substantial impact on the capacity of the walls. The interaction diagrams confirmed that the use of 0.55 for F_1 is rational, but only valid if lateral loads are not applied to the wall. This limitation is not practical for the majority of tilt-up design as lateral loads are typically applied to the panel.

The slenderness effects on load bearing walls were analyzed as well. The Salse-Fintel stress-strain relationship for concrete was utilized to generate a curve for the slenderness procedure given by ACI 318-71: Section 10.11. The approximation was declared "reasonable" for walls with a height-to-thickness ratio less than 12, however, for height-to-thickness ratios between 12 and 15, the use of the empirical design equation results in wall capacities greater than what is determined by laboratory testing. Again, present recommendations for height-to-thickness ratios are substantially greater than what Kripanarayanan deems acceptable for use. The variation between the results of the empirical equation and laboratory testing results is especially apparent in pin-ended walls, as confirmed by the testing done by Oberlander and Everard. The discrepancy in the results that were obtained by the empirical equation and testing is due to the equation not considering P-Δ effects on the panel, which are explained in section 2.2 of this report.

Kripanarayanan suggested a modification to the empirical equation given by ACI 318-71 be modified to Eq. 2-5. Incorporating the effective length factor, k, allows the equation to be applied to conditions where the support conditions are not pin-pinned. Kripanarayanan does not recommend k factors for use in his report. However, research has been done since then, and Table 11.5.3.2 in ACI 318-19 provides the k value to be used for given boundary conditions. Walls braced top and bottom against lateral translation and restrained against rotation at the top or bottom, or top and bottom, shall use a k value of 0.8. Walls braced top and bottom against lateral translation and unrestrained against rotation at both ends shall use a k value of 1.0. Walls that are not braced against lateral translation shall have a k value of 2.0. The modifications result in conservative load capacities for the walls with height-to-thickness ratios between 12 and 15.

2.2 Alternate Method of Analysis

Tilt-up construction was invented and developed in the United States around 1893, at Camp Logan, Illinois, and patented in 1908 by Robert Akin (Dayton Concrete Accessories, 2008). However, it was not until nearly a century passed that the 1999 edition of ACI 318 specifically addressed the requirements for design of slender walls. ACI 318-08 and ACI 318-11, Section 14.8: Alternative design of slender walls, presents a method of analysis as an alternative to the requirements Chapter 10. ACI 318-08 and ACI 318-11 Section, 10.10 – Slenderness effects in compression members, requirements for slender wall design are more general than Section 14.8 and should be applied with discretion. In the 2014 and 2019 editions of ACI 318 Section 14.8 has been updated to be Section 11.8: Alternative method for out-of-plane slender wall analysis.

2.2.1 ACI 318-19 Section 11.8 – Alternative method for out-of-plane slender wall analysis

The alternative method of analysis provides a means of designing slender walls that accounts for the second-order effects of P-Δ that magnify the ultimate moment the wall experiences. The initial deflection of the wall due to lateral loads is furthered as a result of continued vertical loading, resulting in the P-Δ effects. The eccentricity of these applied axial forces in the member causes the additional moment. ACI 318-19 Section 11.8 defines the magnified moment within the wall and provides a means of approximating the ultimate deflection resulting from out-of-plane loads and the out-of-plane deflection due to service loads.

In order to use the alternative method for out-of-plane slender wall analysis, walls must satisfy the following conditions per ACI 318-19 Section 11.8.1.1:

a) Cross section is constant over the height of the wall

b) Wall is tension-controlled for out-of-plane moment effect

c) ϕM_n is at least M_{cr}, where M_{cr} is calculated using f_r as provided in 19.2.3

d) P_u at the midheight section does not exceed $0.06f'_cA_g$

e) Calculated out-of-plane deflection due to service loads, Δ_s, including P-Δ effects, does not exceed $l_c/150$

The panel must also be simply supported, spanning from the foundation to the roofline or support location. These limitations are set based on the testing of full-scale slender tilt-up panels by SEAOSC in 1982. By satisfying these limitations, the application of the alternative method to panels will not result in stress concentrations or deflections that are unlike those that the panels from which the equations used within the alternative method were derived.

The requirement that the design moment be greater than the cracking moment ensures that the wall does not undergo drastic deflection upon cracking. Furthermore, it is conservatively assumed that the panel cracks during the lifting stage of construction and is designed as cracked. The mandate that P_u be less than $0.06f'_cA_g$ ensures that the flexural compression stresses in the concrete and the flexural tension stresses in the steel reinforcing bars govern the wall capacity. The P-Δ effects have a major effect on the wall since it experiences increased cracking and reduced flexural stiffness, so a six percent reduction is taken to account for the slenderness of it.

The multistory panels investigated in the parametric study presented in this report are simply supported and spans from the foundation to each story. The relationship of the moment a 28-foot panel experiences compared to a two-story panel, where the floor-to-floor height is 14 feet for both stories, is shown in Figure 2. The single span panel experiences a much higher maximum moment, shown in black, than the panel that is simply supported at midheight, shown in green. The same relationship is seen in three- and four-story panels when compared to a single span panel of the equal total height.

The design moment must be greater than the maximum positive moment that the panel experiences. As the multi span panel has a lesser maximum moment than the single span panel, the design moment may also be lesser. The multistory panel experiences a negative moment whereas the single span panel does not, so the design moment strength must also be compared to the maximum negative moment.

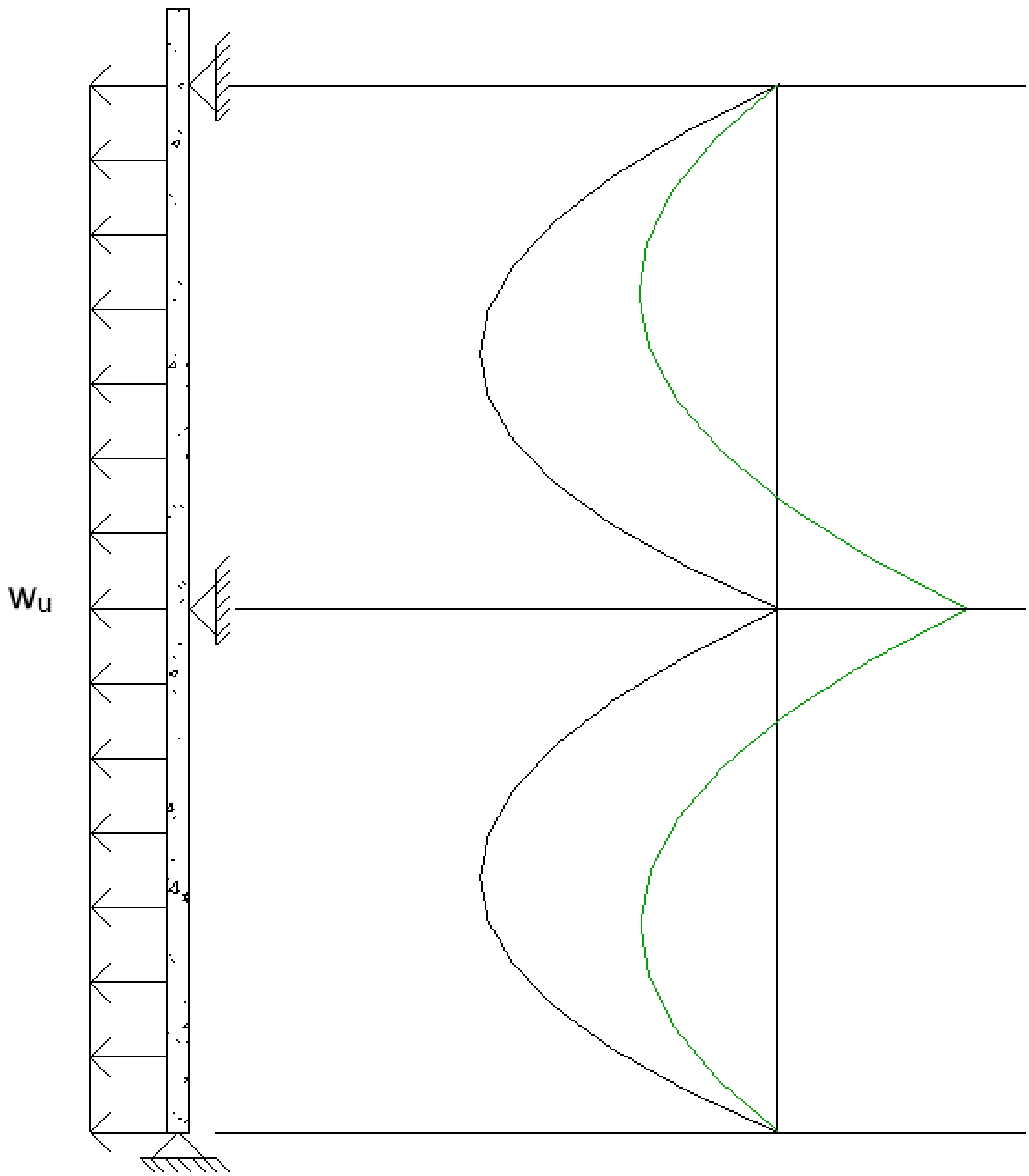

Figure 2: Moment diagram of two-story panel compared to single span panel of equal total length

2.2.1.1 Factored Design Moment

In the parametric study presented in this report, the maximum cracking and nominal deflection that the panel experiences are calculated for the maximum positive moment that the panel experiences as opposed to the maximum negative moment. The cracking and nominal deflections were determined for both the maximum positive and negative moments and it was determined that using the positive moment was more conservative. The secondary moment resulting from axial loads acting eccentrically on the deflection shape is added to the bending moment, resulting in the ultimate moment, M_u. The eccentricity of the applied axial loads causes a bending moment that is additive to the moment from the applied lateral loads to result in the primary moment. The primary moment, M_{ua}, is determined using Eq. 2-8. The ultimate moment that the panel must be able to resist is determined using ACI 318-19 Section 11.8.3. The moment can be determined iteratively using Equation 11.8.3.1a, shown as Eq. 2-6 here:

$$M_u = M_{ua} + P_u \Delta_u \qquad \text{Eq. 2-6}$$

or by using the moment magnifier method by the direct calculation of Equation 11.8.3.1d as is done in the parametric study in this report:

$$M_u = \frac{M_{ua}}{\left(1 - \frac{5P_u l_c^2}{(0.75)48E_c I_{cr}}\right)} \qquad \text{Eq. 2-7}$$

The moment magnifier equation is obtained by combining the moment at the midheight of the wall equation and the out-of-plane deflection equation used in the iteration method. The equation results in values identical to those determined by the iteration method, given that the loading conditions and material properties are consistent.

Examining the iterative method for P-Δ effects, the ultimate moment is composed of two main components; the moment at the midheight of the wall due to factored lateral and vertical loads, and the moment that results from the factored axial load on the panel that produce P-Δ effects. As apparent in Equation 11.8.3.1a, the maximum moment in a slender wall is the sum of the applied moment and the P-Δ moment.

The moment at the midheight of the wall is the resultant of the applied lateral loads and the eccentrically applied axial loads that cause the first-order effects. The following equation is used to determine the moment:

$$M_{ua} = \frac{w_u l_c^2}{8} + \frac{P_{uA} e_{cc}}{2} \qquad \text{Eq. 2-8}$$

The maximum positive moment occurs at the midheight of a simply supported panel with a uniform load, w_u, distributed over the clearspan, l_c. In the case of a multistory structure, the clearspan is taken as the largest floor-to-floor height as it results in the largest moment. The dead and live loads from each floor are added per the number of floors that are above where the maximum positive moment occurs. When the maximum positive moment, not including P-Δ effects, under examination occurs at the midheight of the largest story, only the roof loads are considered as the axial loads when calculating the moment at the midheight of the wall. When the maximum positive moment that is being inspected occurs at the midheight of the first story, the roof loads and all floor dead and live loads are added together to result in the applied axial load that is used when determining the moment at the midheight of the wall.

The second component in determining the ultimate moment accounts for the secondary moment resulting from the applied axial loading including the self-weight. The loads are acting eccentrically to the centroid of the panel. The calculated out-of-plane deflection at midheight of the wall due to factored loads is determined by ACI 318-19 Equation 11.8.3.1b, which is shown as Eq. 2-9 here:

$$\Delta_u = \frac{5 M_u l^2}{(0.75) 48 E_c I_{cr}} = \frac{M_u}{K_b} \qquad \text{Eq. 2-9}$$

where K_b is permitted to be used for walls subject to out-of-plane bending forces due to uniformly applied lateral loading and is calculated by:

$$K_b = \frac{M_u}{\Delta_u} = \frac{48E_cI_{cr}}{5l^2} \qquad \text{Eq. 2-10}$$

where, for normal weight concrete, the modulus of elasticity, E_c, is calculated as:

$$E_c = 57{,}000\sqrt{f'_c} \qquad \text{Eq. 2-11}$$

and the cracked moment of inertia, I_{cr}, is found using ACI 318-19 Equation 11.8.3.1c, shown as Eq. 2-12 in this report:

$$I_{cr} = \frac{E_s}{E_c}A_s + \frac{P_{um}}{f_y}\left(\frac{h}{2d}\right)(d-c)^2 + \frac{l_w c^3}{3} \qquad \text{Eq. 2-12}$$

The cracked moment of inertia is utilized to prevent major deflections of the section to occur. A modification factor of 0.75 is applied to the cracked moment of inertia to reduce the bending stiffness of the panel. The modification accounts for any variations in material properties or due to poor craftsmanship (ACI 551 Committee, 2015).

A modification in the area of reinforcement, A_{se}, is used to partially account for the increased bending resistance due to applied axial loads.

$$A_{se} = A_s + \frac{P_{um}}{f_y}\left(\frac{h}{2d}\right) \qquad \text{Eq. 2-13}$$

The use of A_{se} is permitted by ACI 318-19 Section 11.8 where the vertical stress is limited to $0.06f'_c$ at the midheight of the section.

2.2.1.2 Out-of-Plane Deflection for Service Loads

Out-of-plane deflections are limited for tilt-up walls in order to "avoid excessive elastic deformation due to permanent loads and residual deformation due to an inelastic response" (ACI 551 Committee, 2015). Were the panel to be loaded to the point where inelastic stresses occur, the panel would be permanently deformed and secondary moments would increase causing potential structure failure. The out-of-plane deflection limits must be met in order for the design to be considered adequate by the alternative method for out-of-plane loading. The service level deflections must be less than l_c / 150 per ACI 318-19 11.8.1.1e. This limitation was determined by the full-scale testing on slender walls done by SEAOSC in 1982. These deflections are determined iteratively, accounting for deflections as a function of out-of-plane loading and the P-Δ effect.

In the testing of full-scale slender tilt-up panels by SEAOSC in 1982, it was observed that the out-of-plane deflections increase rapidly when the induced moment exceeds $2/3M_{cr}$, as determined by ACI 318-19 Equation 24.2.3.5, which is shown as Eq. 2-14 here:

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad \text{Eq. 2-14}$$

By reducing the modulus of rupture by two-thirds, the discrepancy when compared to the modulus of rupture determined in the full-scale testing is eliminated. The two-thirds reduction is used to determine the true modulus of rupture, based on test data. SEAOSC proved that the modulus of rupture is:

$$f_r = 5\sqrt{f'_c} \qquad \text{Eq. 2-15}$$

This is two-thirds of the modulus of rupture given by ACI 318, which is:

$$f_r = 7.5\sqrt{f'_c} \qquad \text{Eq. 2-16}$$

For moments less than two-thirds of the cracking moment, the out-of-plane deflection caused by service loads is proportional to the cracking deflection. This relationship is shown as:

$$M_A \leq \frac{2M_{cr}}{3} \quad \therefore \ \Delta_s = \frac{M_A \Delta_{cr}}{M_{cr}} \qquad \text{Eq. 2-17}$$

When the maximum moment at midheight of the wall due to service lateral and eccentric vertical loads exceeds two-thirds the cracking moment, deflections are determined by interpolating between the nominal moment deflection and two-thirds of the cracking moment deflection. The interpolation to determine the out-of-plane deflection caused by service loads is shown below in Eq. 2-16.

$$M_A \geq \frac{2M_{cr}}{3} \quad \therefore \ \Delta_s = \frac{2\Delta_{cr}}{3} + \frac{\left(M_A - \frac{2M_{cr}}{3}\right)}{\left(M_n - \frac{2M_{cr}}{3}\right)}\left(\Delta_n - \frac{2\Delta_{cr}}{3}\right) \qquad \text{Eq. 2-18}$$

where the deflection at the cracking moment is determined by ACI 318-19 Equation 11.8.4.3a.

$$\Delta_{cr} = \frac{5M_{cr}l_c^2}{48E_cI_g} \qquad \text{Eq. 2-19}$$

The potential midheight deflection, Δ_n, is required when the applied moment is greater than two-thirds the cracking moment. The panel is assumed to be fully cracked, and the nominal deflection is determined by ACI 318-19 Equation 11.384.3b.

$$\Delta_n = \frac{5M_nl_c^2}{48E_cI_{cr}} \qquad \text{Eq. 2-20}$$

The design moment strength is checked and compared to both the maximum positive and maximum negative moment. This results in two cracking moments and nominal moments for each load case. Δ_{cr} and Δ_n are calculated using both the positive and negative cracking and nominal moment. The determined values are comparable and the moments resulting from the case where the maximum positive moment was checked was then used to determine the service level deflections for consistency.

The deflection caused by service level loading is multiplied by the service axial loading of the panel and combined with the service moment at the midheight of the panel to result in the maximum moment.

$$M_A = M_{SA} + P_{SM}\Delta_s \qquad \text{Eq. 2-21}$$

where the service moment at the midheight of the panel is

$$M_{SA} = \frac{w_s l_c^2}{8} + \frac{P_{SA} e_{cc}}{2} \qquad \text{Eq. 2-22}$$

and P_{SM} is

$$P_{SM} = P_{SA} + \left(\frac{h}{2} + h_p\right) t\gamma_c \qquad \text{Eq. 2-23}$$

2.3 Reinforcement Limitations with Lifting and Bracing Considerations

ACI 551.2R-15: The Design Guide for Tilt-Up Concrete Panels presents information that "expands on the provisions of ACI 318 applied to the design of site-cast precast, or tilt-up, concrete panels, and provides a comprehensive procedure for the design of these … structural elements," (ACI 551.2R). ACI 551.2R presents design example B.5 for the slender wall analysis of a solid three-story panel considering dead and live loads from the roof and floors. This multistory panel design example was modified with various panel heights, thicknesses, steel strengths, and concrete strengths combinations to determine the effect of each on multistory tilt-up in the parametric study. The calculations done in the parametric study determine if the final in-service condition is adequately designed. Only the reinforcing steel required for this condition is examined, and the design of such for lifting and bracing stresses are outside the scope of this report.

The panels designed were to meet the minimum and maximum reinforcement values defined for cast-in-place walls per ACI 318-19 Sections 11.6 – 11.7. The requirements for cast-in-place walls are more stringent and thus are used instead of those for precast walls. The minimum longitudinal steel, ρ_l, and minimum transverse steel, ρ_t, must meet the requirements given in ACI 318-19 Table 11.6.1.

The practicing engineer should consider stresses on the panel during lifting and bracing when the supporting structural steel system has yet to be attached to determine if additional vertical reinforcing steel is required to be included in the panel so the lifting and bracing engineer may proceed barring any complications. A tilt-up panel undergoes the greatest stress it will experience during the lifting process; as the panel is tilted, the dead weight of the panel induces a flexural moment with associated stresses. The moment a multistory tilt-up panel experiences is greater during lifting as the unbraced length is significantly larger than the floor-to-floor heights of the multistory building. The stiffness of the panel must be sufficient enough to withstand these substantial lifting stresses and unbraced length when temporarily braced prior to the final panel connections to the floor and roof diaphragms. The design of a panel for these stresses is outside of the scope of this report.

The thickness of the panel in relation to vertical reinforcement is often adequate to resist lifting stresses; however, in some cases the use of strongbacks may be required. Strongbacks are a structural steel method used to reinforce uniquely shaped panels during erection. Strongbacks are costly and require additional time to be installed and uninstalled, so the use of them is best to be limited or avoided whenever possible. A panel can be designed to resist the stresses during lifting by increasing the compressive strength of the concrete and selecting a sufficient panel thickness and adequate vertical reinforcement based on requirements from the lifting engineer. Adding additional rebar to a panel or using higher strength concrete to obtain a greater stiffness requires less expense and time than using strongbacks.

In order to erect the panel from its horizontal casting position to its final placement, lifting inserts are embedded within the tilt-up panel prior to casting. Typically, two lifting inserts at each floor level are used to resist the loads applied as direct tension, shear, or a combination of the two. The common placement of the lifting inserts is seen in **Error! Reference source not found.**. The floor-to-floor height may change between stories so the floor height-to-panel thickness ratio must be considered to ensure that the stresses acting on the panel during lifting do not exceed its capacity. Additional lifting inserts may be added to a panel to distribute the forces, though doing so can cause additional construction time as the rigging of the crane must be reconfigured if the number or location of inserts vary between panels.

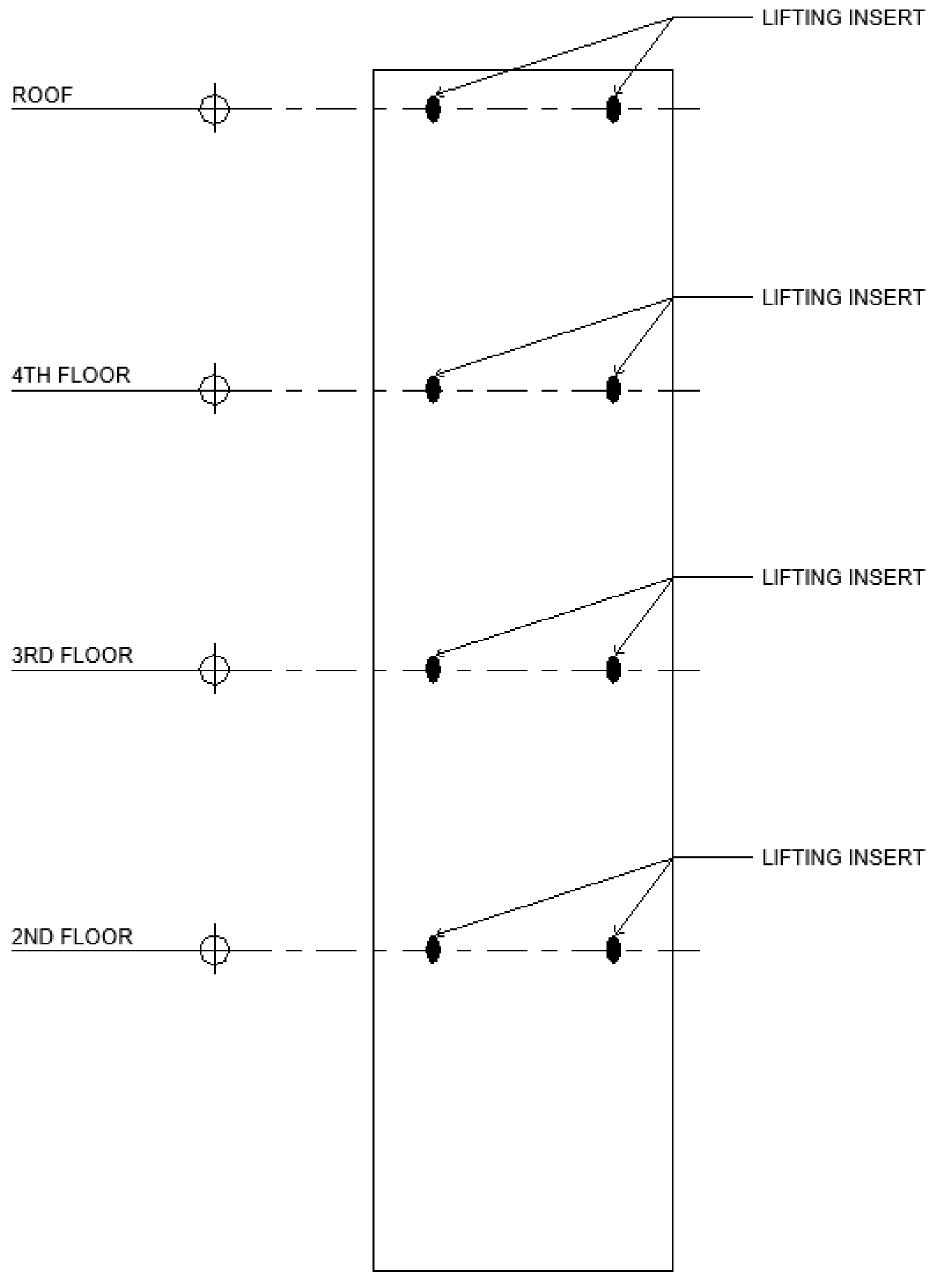

Figure 3: Lifting insert locations

Prior to lifting the panel, temporary braces are attached to the face of the panel, the location of such is seen in **Error! Reference source not found.**. Once the panel is lifted into its f inal vertical position the braces are properly anchored to the slab or an alternative foundation system. The crane then releases the panel when the braces are securely anchored. The bracing holds the panel plumb and keeps it from collapsing in response to applied lateral forces until the supporting structure is in place and fully attached. There is currently no code to design the bracing to, though the TCA Bracing Guidelines suggest using a safety factor of 1.5 times the applied governing wind or seismic load to select braces and a minimum of two braces per panel are required to be installed. The bracing can be removed when the engineer of record confirms that the steel structure is in place and is fully supporting the panel.

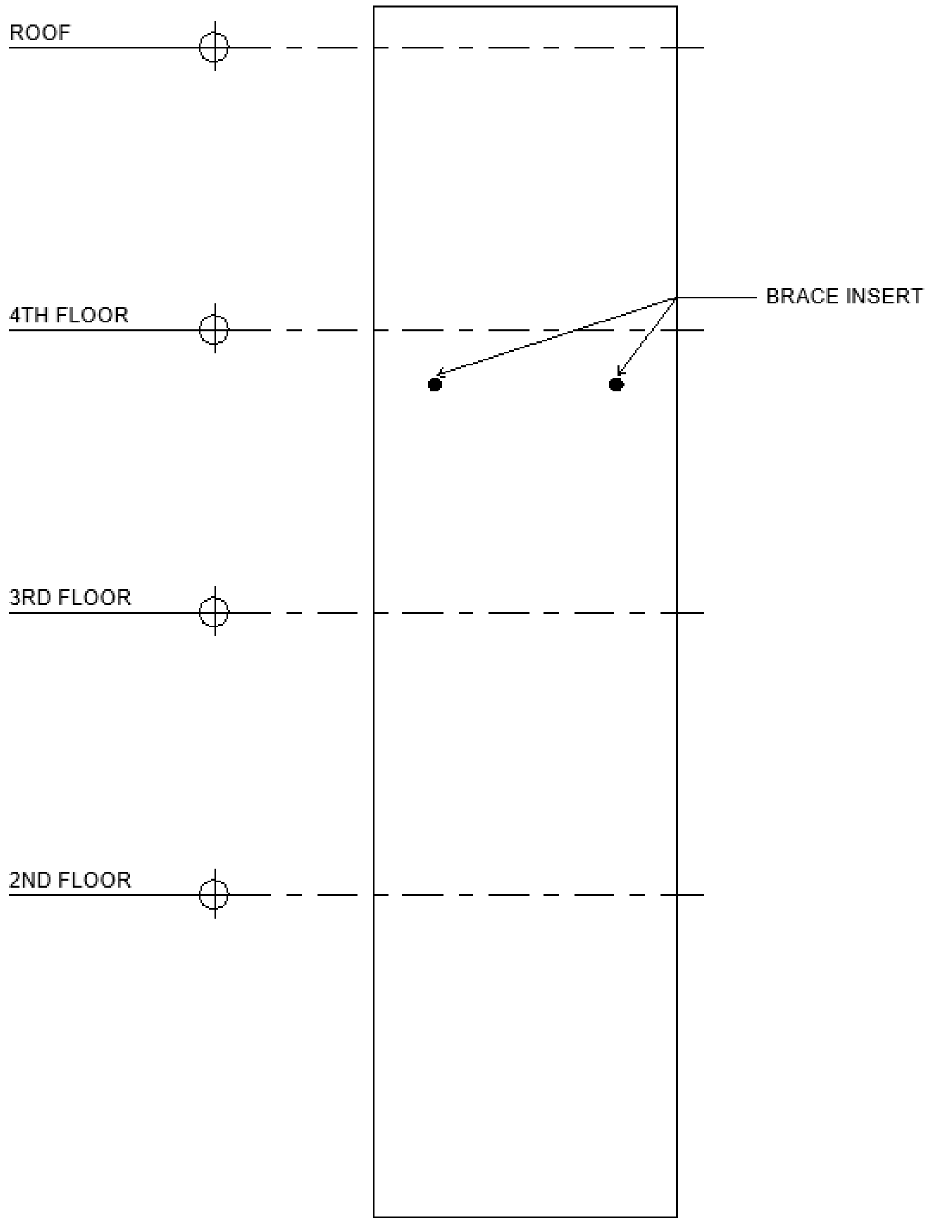

Figure 4: Brace insert location

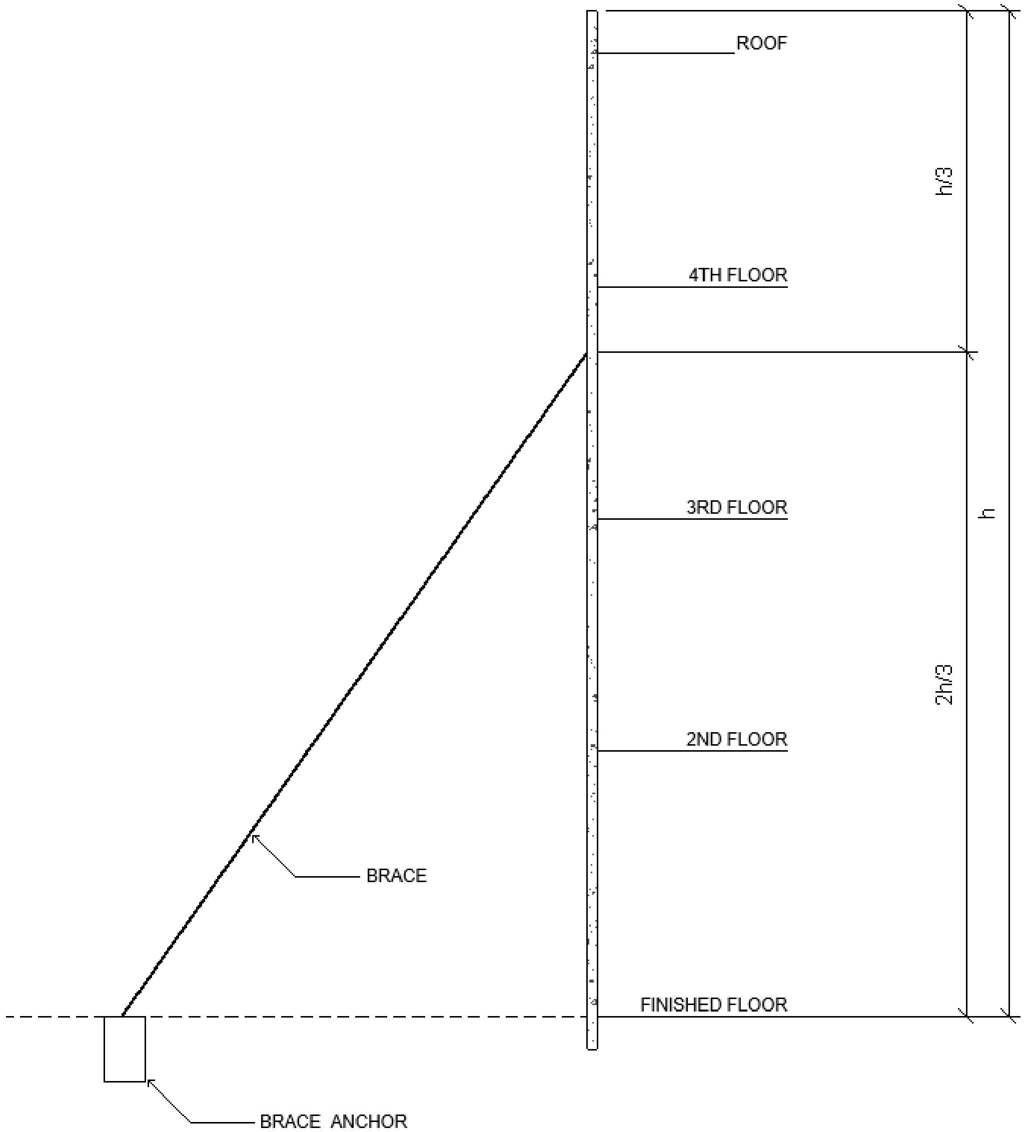

Figure 5: Bracing diagram

Chapter 3 – Parametric Study

This parametric study evaluates the effects of designing a multistory reinforced concrete tilt-up panel with varying parameters. Eight typical variables in multistory tilt-up construction are examined as parameters in this study. These eight parameters are the:

1. location of construction site
2. compressive strength of concrete
3. reinforcing steel size
4. strength of the reinforcement
5. number of stories constructed
6. panel width
7. thickness of the panel
8. number of layers of reinforcement.

Aside from the strength of the reinforcing steel, these parameters commonly vary between construction projects and best demonstrate the effects of difference in the number of stories. ASTM A615 Grade 60 steel is most commonly used in tilt-up panel design today; however, the use of ASTM A615 Grade 80 steel is gaining momentum and thus is also evaluated in this study to determine the effects it has on multistory tilt-up. All possible combinations of the eight parameters were evaluated, totaling 72 varied panel designs.

ACI 318-19, Section 11.8: Alternative method for out-of-plane slender wall analysis, in adjacency with Design example B.5 from ACI 551.2R-15: Design Guide for Tilt-Up Concrete Panels, was utilized in this study as the basis of design. The panels are idealized as pin-pinned rather than a spring in order to meet the limitations set by ACI 318-19 Section 11.8. This is an unconservative method as the panel experiences some movement in the x-direction. Appendix A demonstrates the calculations done to design each panel. For some parameter combinations, a viable design was not achievable within the limitations of the alternative analysis method presented by ACI 318-19: Section 11.8 and ACI 551.2R-15, as described in Chapter 2 of this report and was deemed as “NP” for Not Permittable.

3.1 Building Conditions

A panel design of a multistory office building was evaluated in this parametric study. The building measures 90 feet by 180 feet, with the panel analyzed located at the interior of the 180-foot wall. An exterior elevation of the wall of which the panel designed within is shown in **Error! Reference source not found.** below. The roof system of the office building is composed o f a steel deck on joists, supported by the wall panels at the exterior and wide flange girders at the interior. Similarly, the floor system is composed of a composite deck with three and a half inches of lightweight concrete topping on joists, supported by the wall panels at the exterior and wide flange girders at the interior. The roof and floor framing spans into the wall that the panel designed is located.

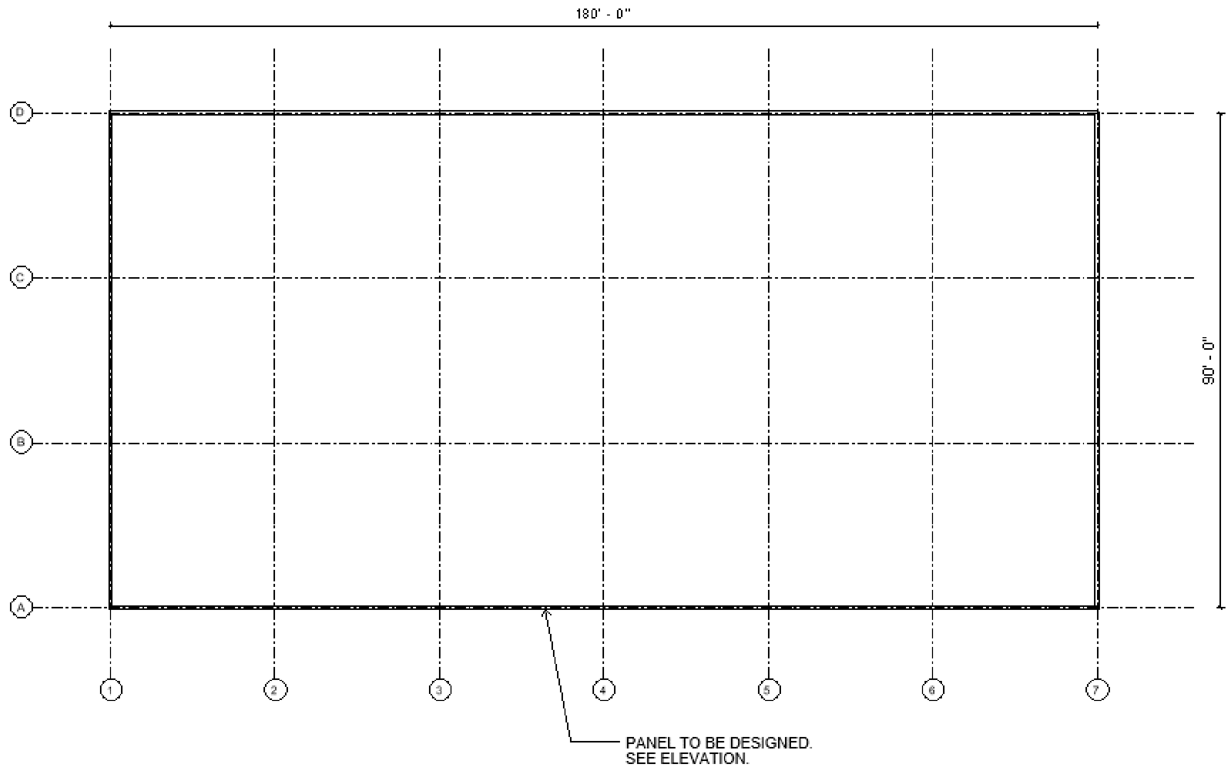

Figure 6: Floor plan of parametric study building

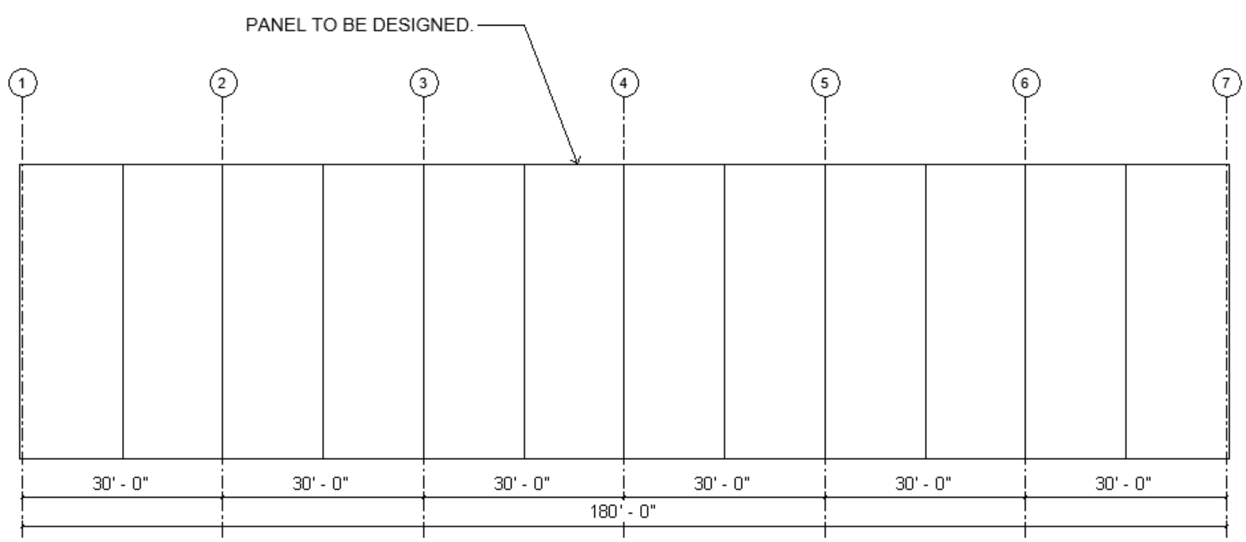

Figure 7: Elevation of parametric study building

The typical bay size is 30 feet by 30 feet and two tilt-up panels are installed within each exterior bay. The joists are spaced five feet on center for both the roof and floor systems. The spacing was determined by the allowable span distance of roof and floor decking provided by Vulcraft Steel Roof Deck and Composite Deck for the roof and floor systems, respectively. The width of the panel was selected based on the bay dimensions and the lifting capacity of the assumed picking crane that would theoretically be used in construction. Using a panel width of 15 feet ensures that it would not exceed a weight that the crane would not be able to safely erect.

3.1.1 Panel Parameters

The construction site location, compressive strength of concrete, bar size of reinforcing steel, strength of reinforcement, number of stories constructed, panel width, thickness of the panel, and how many layers of reinforcement are implemented were used as the parameters of this study. Discussion on why each parameter was chosen and how it affects the tilt-up panel is presented in this section.

3.1.1.1 Construction Site Location

Three locations with varying wind speeds were chosen to evaluate how out-of-plane loading effects multistory tilt-up. The locations are Manhattan, Kansas, Houston, Texas, and Miami, Florida. Their respective wind speeds are 114 mph, 140 mph, and 169 mph. The wind speed was determined using ASCE 7-16 Figure 26.5-1B Basic Wind Speeds for Risk Category II Buildings and Other Structures. The wind speeds determined from ASCE 7-16 were reviewed with the ATC Hazards by Location website and were found to be relatively the same. The value used for calculations in this parametric study are those from ASCE 7-16 as they are higher and thus more conservative.

The main purpose of varying the location of the construction site was to analyze how the panels are affected by out-of-plane loading, however, snow loads also varied between the locations. Houston, Texas and Miami, Florida do not experience a snow load, however, Manhattan, Kansas has a 20 psf ground snow load. The snow load acts as an additional axial load applied at the roof that ultimately results in a lesser out-of-plane deflection than that caused from the wind pressures acting upon the panel.

3.1.1.2 Concrete Compressive Strength

Panels should have a minimum of 3,000 psi compressive strength to ensure that they obtain the sufficient strength within five to seven days following casting operations (Tilt-Up Concrete Association, 2011). A 3,000 psi mix is considered lightweight. The concrete compressive strength is further limited to a maximum strength of 4,000 psi, which is considered normal weight. For most projects, a minimum sufficient concrete compressive strength of 2,500 psi at lifting is specified by the specialty lifting engineer (STRUCTURE 2018). Previous research conducted by Lewer and Kramer (2019), Analysis of Slender Reinforced Concrete Wall Panels (Tilt-Up) Utilizing Lightweight Concrete, presents the effects of using lightweight concrete on the performance of tilt-up panels. The use of lightweight concrete results in a minimal difference in deflections for lateral loading prior to reaching the cracking moment. Once the cracking moment was reached, larger deflections were observed in panels with small a small modification factor, λ, and modulus of rupture factor, K. The effects of lightweight concrete on multistory panel design falls outside the scope of this study and is thus not used as a variable parameter. Only a 4,000 psi concrete compressive strength is used in this parametric study as it is most commonly used for tilt-up panels.

3.1.1.3 Reinforcement Bar Size

While the practicing engineer does not design to the highest stress the panel will experience during lifting, such must be considered. To resist the stresses of lifting and to add additional structural integrity, reinforcing bars are used in tilt-up panels. Commonly, the steel reinforcement within tilt-up panels are #4, #5, or #6 bars (Tilt-Up Concrete Association, 2011). This study will utilize #5 and #6 bars for longitudinal reinforcement and will use a #5 bar for all transverse reinforcement. By comparing the loading applied, panel height, and panel thickness in this study to the loads applied in the study done by McConnell and Kramer (2017), Analysis of Reinforced Concrete Tilt-Up Panels utilizing High-Strength Reinforcing Bars, it was assumed that a #4 bar will not provide sufficient longitudinal reinforcement for many of the panels evaluated. A #4 bar therefore was not used as a longitudinal reinforcement parameter in this study. The reinforcement within the panels in this study was designed per ACI 318-19: Table 11.6.1 to satisfy the minimum required reinforcement ratios for cast-in-place walls.

3.1.1.4 Reinforcement Tensile Yield Stress

A higher strength of steel used as reinforcement in tilt-up panels will amount to a higher structural integrity. However, high strength steel is not commonly used in the construction industry at this time. The use of such is becoming more prevalent, and understanding how it affects multistory tilt-up panel design is beneficial to industry professionals. ASTM A615 Grade 60 and ASTM A615 Grade 80 reinforcement are used as parameters to evaluate the resulting structural integrity of the panel.

The strength of steel directly impacts the effective area of steel, which is used to determine the cracking moment of the section. The cracking moment is inversely related to the out-of-plane deflections of the panel and is associated with P-Δ effects. P-Δ effects, also referred to as secondary effects, occur when an axial load is applied to a member that has already deflected.

3.1.1.5 Number of Stories

Panels were selected to range from two to four stories. The floor-to-floor height of all stories is 14 feet. The heights of the stories were selected based on the lifting capacity of a traditional picking crane. With a constant width of 15 feet and thickness of 7 1/4-inches, the three panel heights all maintain a total weight less than 60 tons. A four-story panel can be cast on the building slab, or adjacent casting bed, and lifted into place. Should a building exceed four stories, an additional casting sequence must be performed for the additional story or stories. Once the four-story panels are tilted into place the additional panels are casted, and once cured, are then lifted and attached to the top of the previously erected four-story panels. Evaluating how the stacked panel system of varying parameters is affected by out-of-plane loads is outside of the scope of this report. Therefore, panels exceeding four stories are not evaluated in this parametric study.

3.1.1.6 Panel Thickness

In tilt-up construction panel thickness is limited to a minimum of 5 1/2-inches to resist axial forces from loads imposed by roof or floor framing, and from the self-weight of the panel, such that the panel will not be overstressed in compression. The thinner the panel, the more likely it is to buckle. A panel thickness of 7 1/4-inches was selected to resist buckling based on the depth of dimensional lumber that is commonly used as formwork for tilt-up panels. This thickness correlates to nominal 2x8 inch dimensional lumber. Using the actual depth of dimensional lumber as the panel thickness is economical as the lumber is not required to be ripped, saving time and labor costs.

3.1.1.7 Number of Layers of Reinforcement

Using a panel thickness of 7 1/4-inches allows for two layers of steel reinforcement to be used. Varying the number of layers of reinforcing steel used in the panel illustrates the effects of increasing the internal moment arm as the distance from the extreme fiber in compression to the flexural reinforcement, d, increases. The moment arm between the tensile reinforcement and the extreme compression fiber in relation to reinforcement tensile yield stress increases as the amount of steel increases. The cracking moment of inertia and out-of-plane deflection of the panel is directly affected by the internal moment arm. A larger cracking moment results in a stiffer panel and lesser out-of-plane deflections.

The height of the reinforcing steel support chairs varies based on if the panel is singly or doubly reinforced. Singly reinforced panels have a chair height of half of the thickness as it places the reinforcement closest to the center of the panel. Doubly reinforced panels have a chair height of one inch, in accordance with the clear cover recommendation specified by ACI 551, Table 10.1b. A 3/4-inch concrete cover is recommended for precast concrete panels and a 1.5-inch concrete clear cover is recommended for cast-in-place panels when cast against earth and exposed to weather, such as the panel in this parametric study is. ACI 551 does not specify a clear cover to be used for tilt-up panels, so a minimum clear cover of 1.5 inches was used in this parametric study as a worst case. Figure 8 exhibits sections for a singly and doubly reinforced panel section, respectively.

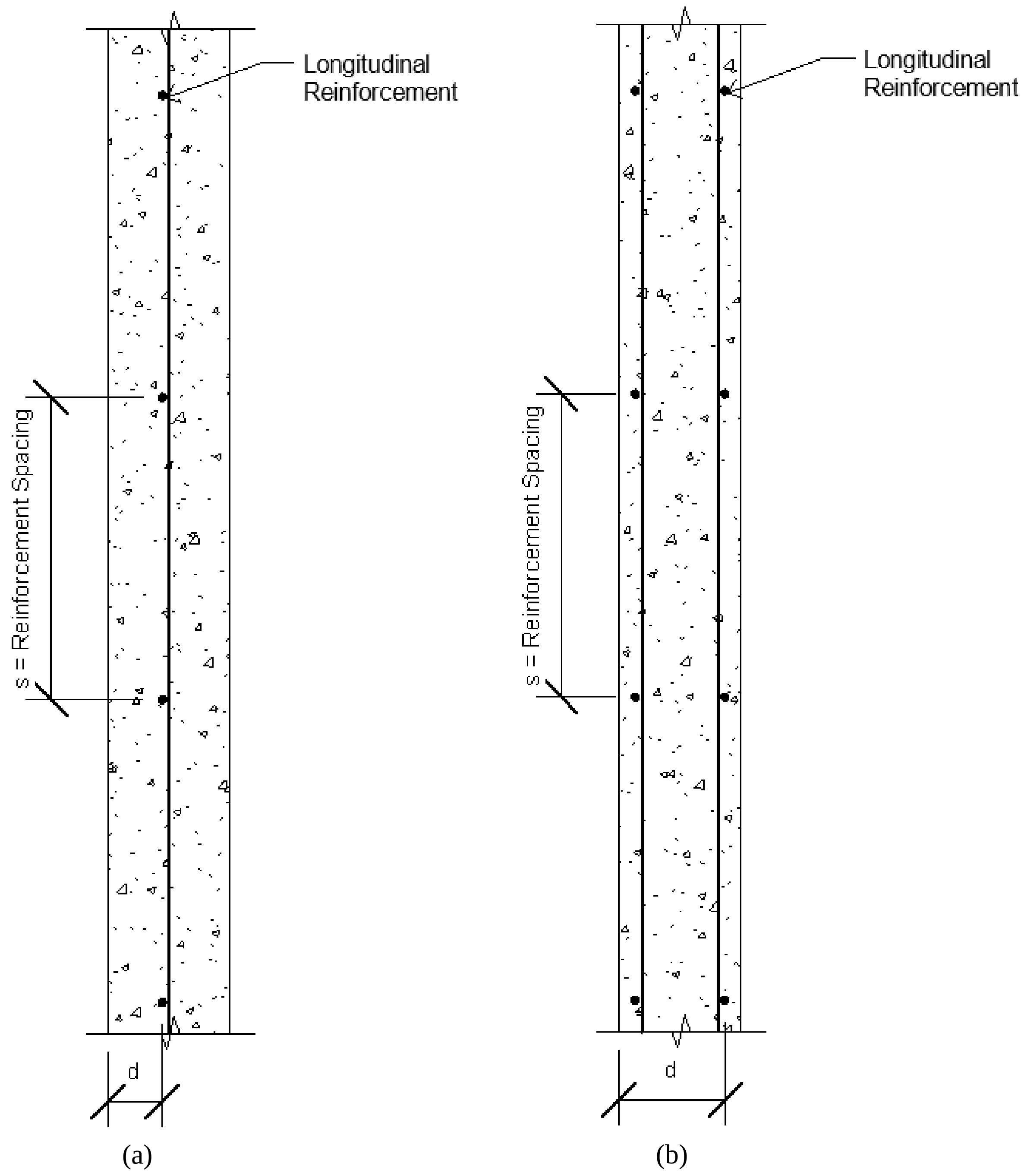

Figure 8: (a) Singly reinforced section and (b) Doubly reinforced section

The same number of bars are used in both the singly and doubly reinforced panel design. As a result, the spacing between the reinforcement will change between the two designs. The maximum allowed spacing is 18 inches, per ACI 318-19 Section 11.7.3.1, so the spacing of the bars in the doubly reinforced panel is used to be 18 inches. The spacing in the singly reinforced panel is halved, so the bars are spaced at nine inches on center. The number of bars in tension in the doubly reinforced panel is half of what is present in the singly reinforced panel as half of the bars are in compression.

The design process of this study neglects the contribution of reinforcement in the compression region. The compression reinforcement is uncontained and buckles easily as the panels were designed as untied. Additionally, should the neutral axis of the section move to a location such that both layers of reinforcement are subject to tension stresses, the layer closest to the neutral axis is neglected. The stresses can conservatively be neglected as the stresses accumulated would be relatively small in comparison to those in the extreme layer in tension.

3.2 Panel Designation

The naming convention shown in Figure 10 is used to designate the 72 panels analyzed in the parametric study. The panel height given in the designation is the total height of the panel. The clear span is 14 feet for all panels designed in this parametric study. As an example, a panel constructed in Houston, Texas with a width of 30 feet with a thickness of 7.25 inches, a concrete compressive strength of 4000 psi, with a single layer of #5 longitudinal reinforcement that is of 60 ksi steel would be described as Panel TX.30'.7.25"4.60.5.1.

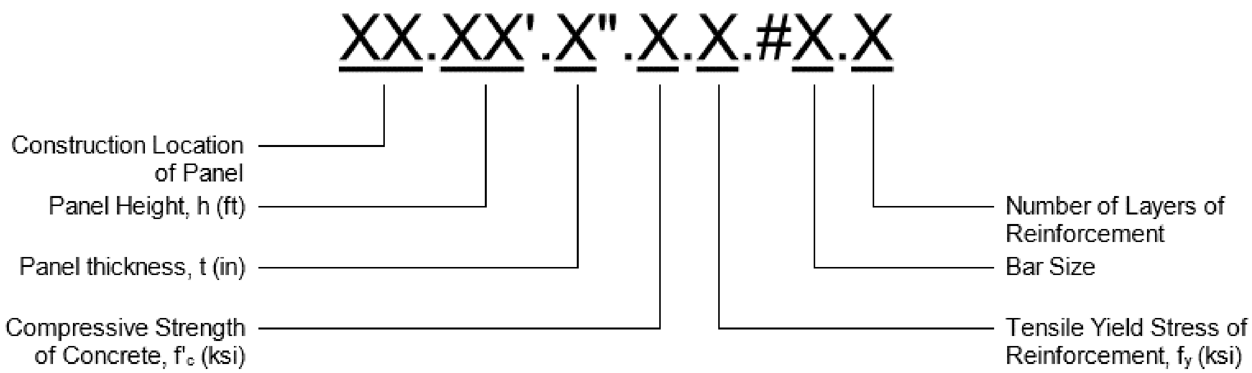

Figure 9: Panel Designation

3.3 Applied Loading

The loads applied to the panel are determined in accordance with ASCE 7-16 Minimum Design Loads for Buildings and Other Structures. The panel being designed in this study resists mainly gravity dead and live loads and out-of-plane wind loads. The in-plane loading that the panel experiences is neglected as most tilt-up panels do not experience large in-plane shear loads relative to their capacity. By neglecting in-plane loads on the panels, the relationship between the longitudinal reinforcement of the wall and secondary stresses from out-of-plane loads is isolated. Since the panel is located at the interior of the wall it will undergo uniform vertical loading from the roof and floor and out-of-plane wind loads.

In order to design the panel, it was modeled as spanning from the foundation to the roof, with pinned connections where mechanical connections occur. The pinned connections were thus implemented at the foundation, each floor level, and the roof level, as shown in Figure 10 below. Idealizing the panel in this way prevents moments that occur in the panel to be transferred to the foundation.

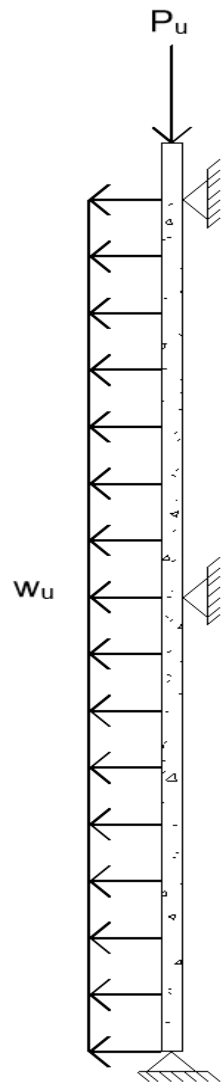

Figure 10: Panel loading and boundary conditions for a 2-story panel

3.3.1 Gravity Loads

The panel designed is subjected to snow, roof live, roof dead, floor live and floor dead loads. These vertical loads act upon the roof or floor deck, transfer to the joists, and then into the tilt-up panels. The panel is influenced by half the span distance of the joists at the roof and floor levels as the framing systems span into the panel. The axial load that the panel experiences is a result of the dead and live loads acting within the tributary area, in addition to the self-weight of the panel above the location of which the maximum positive moment occurs. The joists were spaced five feet on center, so each panel is subjected to the load from three joists. Due to the equally spaced joists and magnitude of loads that are applied, the vertical loads were approximated as uniform.

3.3.1.1 Dead Load

The dead load acting upon the panel is a summation of the weights of the roofing and floor materials within the tributary area of the panel, the weight of the parapet, and the self-weight of the panel. The assumed roofing materials and the corresponding weight of each are listed in Table 1. Similarly, the assumed flooring materials and the corresponding weights are listed in

Table 2

Table 2: Floor Dead Loads. The weights of the materials were determined in accordance with ASCE 7-16: Table C3.1-1, Minimum Design Dead Loads.

Table 1: Roof Dead Loads

ROOF DEAD LOADS	
COMPONENT	WEIGHT (psf)
SINGLE-PLY MEMBRANE ROOFING	3
8” RIGID INSULATION	4
MECHANICAL/ ELECTRICAL SYSTEMS	5
CEILING SYSTEM	2
MISC.	5

ROOF DECK	2.7
TOTAL	21.7

Table 2: Floor Dead Loads

FLOOR DEAD LOADS	
COMPONENT	WEIGHT (psf)
CARPET	6.8
MECHANICAL/ ELECTRICAL SYSTEMS	5
CEILING SYSTEM	2
MISC.	5
COMPOSITE DECK W/ CONCRETE TOPPING	34
TOTAL	52.8

3.3.1.2 Live Load

The live load acting upon the panel occurs at both the roof and the floors. The roof live load was considered a separate live load than the floor live load and is thus a different factor in the load combinations. A minimum construction live load for an ordinary roof and occupancy live load for the floors are shown in Table 3 and Table 4, respectively. The live loads were determined in accordance with ASCE 7-16: Table 4.3-1.

Table 3: Roof Live Loads

ROOF LIVE LOADS	
LOAD TYPE	WEIGHT (psf)
ORDINARY ROOF	20

TOTAL	20

Table 4: Floor Live Loads

FLOOR LIVE LOADS	
LOAD	WEIGHT (psf)
OFFICES	50
PARTITIONS	15
TOTAL	65

Reduction in the uniform live loads are not permitted to be taken per the requirements of ASCE 7-16: Sections 4.7.2 through 4.7.6.

3.3.1.3 Snow Load

The snow load acting upon the panel was calculated in accordance with ASCE 7-16: Chapter 7. The ground snow load was determined using the ground snow maps found in Figure 7.2-1 of ASCE 7-16 and checked with ATC Hazards by Location. The snow load varies based on which construction site location the panel is being designed in, and in two of the locations, Houtson, Texas and Miami, Florida, the load is zero. The ground snow load for Manhattan, Kansas was converted to a flat roof snow load by Equation 7.3-1 of ASCE 7-16:

$$p_f = 0.7 C_e C_t I_s p_g \qquad \text{Eq. 3-1}$$

The exposure factor, C_e, thermal factor, C_t, and snow importance factor, I_s, were selected based on assumed conditions for a typical office building. The exposure factor was determined using

Table 7.3-1 of ASCE 7-16 and is found to be 1.0. This is based on an assumed Exposure B, per Section 26.7.3 of ASCE 7-16 and that the roof is partially exposed. The roof is partially exposed as the office building is assumed to not be located on the edge of town, exposed on all sides, or in a dense city, obstructed by surrounding structures. The thermal factor is determined via Table 7.3-2 of ASCE 7-16. It was assumed the roof is insulated with no thermal benefit from the space below, resulting in a factor of 1.1. This indicates that the occupied space below the structure of the roof is heated and that the rigid insulation has a thermal resistance high enough to prevent heat from transferring through and melting the snow. The importance factor of a structure is determined based on the risk category that it is classified as. Office buildings are commonly categorized within Risk Category II, as the risk to human life is between "low" and "substantial" should failure of the structure occur. The snow importance factor was then found to be 1.0 per Table 1.5-2 of ASCE 7-16.

When the ground snow load is less than 20 psf, but not zero psf, five psf must be added to the resulting flat roof snow load value, per ASCE 7-16: Section 7.10, to account for rain-on-snow surcharge load. Adding the additional load considers that rain and snow could occur during the same storm. The minimum snow load for a flat roof such as is assumed in this parametric study is checked with ASCE 7-16: Section 7.3.4 – Minimum Snow Load for Low-Sloped Roofs, p_m. Should the flat roof snow load not exceed the required minimum load, the minimum was used.

Snow drift is present on the roof due to the parapet along the perimeter of the office building. This is only applicable to the Manhattan, Kansas location as the other two locations do not experience a snow load. The snow drift load is a surcharge load on the flat roof snow load as the snow is pushed and deposited on the back face of the parapet, resulting in an increased snow load. The load of the snow drift must be considered if the ratio of the clear height from the top of the balanced snow to the top of the parapet, h_c, to the height of the balanced snow load, h_d, is greater than 0.2 (ASCE 7-16: Section 7.7.1 – Lower Roof of a Structure). The depth of the drift is calculated using ASCE 7-16: Figure 7.6-1 – Graph and Equation for Determining Drift Height:

$$h_d = \left(0.43\sqrt[3]{l_u}\sqrt[4]{p_g + 10}\right) - 1.5 \qquad \text{Eq. 3-2}$$

The upwind length, l_u, of the roof is the length of which the snow can be picked up by the wind before it is deposited along the parapet. How far the drift extends from parapet is considered the width of the drift and is determined based on the relationship of h_d and h_c. The snow load on the roof can then be calculated based on the depth and width of the drift, however, an average of the flat roof snow load and maximum drift load was taken over the roof for this study.

3.3.2 Lateral Loads

The panel designed is also subjected to wind loads. The lateral wind loads applied to a tilt-up panel is the main factor in the out-of-plane deflection the panel experiences. This deflection typically governs tilt-up design. Theoretically, seismic forces are also acting upon the building, however, it is assumed that wind loads govern over Seismic Design Category B in the locations selected and thus panel design for seismic loading is not within the scope of this study.

3.3.2.1 Wind Load

Wind pressures on components and cladding of the office building were determined for the panel per ASCE 7-16 Section 30.3: Part 1: Low-Rise Buildings. The office building meets the requirements of using this analytical method as it is partially enclosed and has a mean roof height less than 60 feet for all panel heights. The wind pressure on the panel was determined by Equation 30.3-1 of ASCE 7-16, shown as Eq. 3-3 here:

$$p = q_h\left[\left(Gc_p\right) - \left(Gc_{pi}\right)\right] \qquad \text{Eq. 3-3}$$

The pressure is based on the velocity pressure evaluated at the mean roof height, q_h, and the external and internal pressure coefficients and gust-effect factors, Gc_p and GC_{pi}, respectively. The external pressure coefficient is found in ASCE 7-16: Figure 30.3-1. The panel designed in this study is located within Zone 4, determined by the diagram of Figure 30.3-1. Both the positive and negative pressure given in the figure were analyzed to determine which pressure governs. The external pressure coefficient is determined based on the area of the panel. When the area of the panel is less than 500 square feet the external pressure coefficient should be interpolated as a function of the effective wind area of the panel. The two-story panel has an effective wind area of 420 square feet, so the external pressure coefficient was interpolated and found to be -0.83 and 0.73. The three- and four-story panels have an area greater than 500 square feet so no interpolation was required and the external pressure coefficient was determined to be -0.8 and 0.7. The internal pressure coefficient is given in Table 26.13-1 of ASCE 7-16 and determined to be +/- 0.55 for partially enclosed buildings.

The velocity pressure at the mean roof height is calculated using Equation 26.10-1 of ASCE 7-16, shown in this report as Eq, 3-4:

$$q_h = 0.00256 K_z K_{zt} K_d K_e V^2 \qquad \text{Eq. 3-4}$$

The velocity pressure exposure coefficient, K_z, is determined based on the height of the panel and the exposure of the building and is determined using ASCE 7-16: Table 26.10-1 – Velocity Pressure Coefficients. The height above ground level varies in the panel design based on the number of stories that are present. Exposure B is assumed for this office building, the same exposure category used in the determination of snow loads. The topographic factor, K_{zt}, accounts for the elevation of the base of the building relative to the surrounding environment. The building in this study is not located within a hill, ridge, or escarpment so K_{zt} was taken to be 1.0 (ASCE 7-16: Section 26.8.2 – Topographic Factor). The wind directionality factor, K_d, was taken as 0.85 for components and cladding structure type, specified in ASCE 7-16: Table 26.6-1. The ground elevation factor, K_e, adjusts the wind pressure acting on the building for air density. K_e is determined in accordance with Table 26.9-1 of ASCE 7-16, or is permitted to be 1.0 for all elevations. Using 1.0 for K_e is conservative for most locations, however, the factor was calculated for this study. It was found to cause minimal variance in the final wind pressure that is applied to the building. The basic wind speed, V, varies based on which location the office building was designed in. For Manhattan, Kansas the wind speed is 114 mph, in Houston, Texas the wind speed is 140 mph, and in Miami, Florida it is 169 mph. These values represent the design three second gust wind speed that occurs 33 feet above ground for an Exposure C building. The K_z factor adjusts the velocity to account for the building being designed as Exposure B.

The resulting negative and positive wind pressures acting on the panel are shown in

Table 5: Wind Pressure on Panel (Manhattan, Kansas)

Panel Height (ft)	Area (ft^2)	GC_p (-)	GC_p (+)	GC_{pi} (+/-)	p (-) (psf)	p (+) (psf)
28	420	-0.83	0.73	0.55	-26.2	24.3
42	630	-0.8	0.7	0.55	-28.2	26.1
56	840	-0.8	0.7	0.55	-30.5	28.3

, **Error! Reference source not found.**, and Table 7.

Table 5: Wind Pressure on Panel (Manhattan, Kansas)

Panel Height (ft)	Area (ft^2)	GC_p (-)	GC_p (+)	GC_{pi} (+/-)	p (-) (psf)	p (+) (psf)

28	420	-0.83	0.73	0.55	-26.2	24.3
42	630	-0.8	0.7	0.55	-28.2	26.1
56	840	-0.8	0.7	0.55	-30.5	28.3

Table 6: Wind Pressure on Panel (Houston, Texas)

Panel Height (ft)	Area (ft^2)	GC_p (-)	GC_p (+)	GC_{pi} (+/-)	p (-) (psf)	p (+) (psf)
28	420	-0.83	0.73	0.55	-41.1	38.2
42	630	-0.8	0.7	0.55	-44.2	41.0
56	840	-0.8	0.7	0.55	-47.9	44.4

Table 7: Wind Pressure on Panel (Miami, Florida)

Panel Height (ft)	Area (ft^2)	GC_p (-)	GC_p (+)	GC_{pi} (+/-)	p (-) (psf)	p (+) (psf)
28	420	-0.83	0.73	0.55	-60.1	55.7
42	630	-0.8	0.7	0.55	-64.6	59.8
56	840	-0.8	0.7	0.55	-70.0	64.8

By comparing the negative and positive wind pressures, it was evident that the negative governs over the positive for all of the panels. As the negative pressure governs, the wind pressure acting away from the panel is greater than that acting upon the panel.

This study neglects the effect of the wind pressure on the parapet. This reduces the out-of-plane deflection the panel experiences, however, the panels are designed for larger out-of-plane deflections so this is an acceptable method. The effects of uplift on the roof are also neglected as they reduce the axial loading on the panel. This in turn reduces the additional stresses acting on the panel while it is loaded, resulting in out-of-plane deflection.

3.4 Load Combinations

The gravity and out-of-plane loads acting on the panel analyzed in this parametric study are determined from the load combinations set forth in ACI 318-19: Table 5.3.1 – Load Combinations. The same load combinations are also found in ASCE 7-16: Section 2.3 – Load Combinations for Strength Design. The following load combinations were evaluated to determine which governs:

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

Load combinations 2, 3, 4, and 5 can be iterated depending on which loads are acting upon the structure. For this parametric study all possible iterations are examined for two cases: when the maximum moment occurs at the midheight of the first story, and when it occurs at the midheight of the largest story. The number of stories and self-weight of the panel varies depending on where the maximum moment occurs, affecting the dead and live loads input to the load combinations. By inspection combinations 4b and 6 will govern over the other combinations when the maximum moment occurs at the midheight of the first story as they account for the effects of the snow load on the roof and the wind load applied on the panel. When the maximum moment occurs at the midheight of the largest story, load combinations 4a and 6 govern as they account for the effects of live load on the roof. Should the building be located in Manhattan, Kansas load combination 4b governs when the maximum occurs at the midheight of the largest story, but 4a governs when the building is located in Houston, Texas or Miami, Florida. The resulting loads from 4a and 4b differ by less than five percent thus 4a is used as the governing load combination for the case. The single most governing combination cannot be determined until the ultimate moment within the panel is calculated because the gravity loads and lateral loads are acting in perpendicular directions. Therefore, each of the 72 panels are designed for load combinations 4b and 6 when the moment occurs at the midheight of the first story, and load combinations 4a and 6 when the moment occurs at the midheight of the largest story. The design that results in the largest ultimate moment will govern for the panel under investigation. It is possible that the governing load combination could vary between the 72 panels designed.

Once the governing load combination was determined for each panel, the corresponding load combination for allowable stress design was determined to calculate the service level deflection. The service level load combinations are given by ASCE 7-16: Section 2.4 – Load Combinations for Allowable Stress Design.

1. U = D

2. $U = D + L$
3. $U = D + (L_r \text{ or } S \text{ or } R)$
    a. $D + L_r$
    b. $D + S$
4. $U = D + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$
    a. $D + 0.75L + 0.75L_r$
    b. $D + 0.75L + 0.75S$
5. $U = D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R)$
    a. $D + 0.75L + 0.75(0.6W) + 0.75(L_r)$
    b. $D + 0.75L + 0.75(0.6W) + 0.75(R)$
6. $U = D + (0.6W)$
7. $U = 0.6D + 0.6W$

Load combinations 3, 4, and 5 can be iterated depending on which loads are acting upon the structure. For this parametric study it was not necessary to calculate all possible iterations. The unfactored combination that correlated with the governing factored combination was analyzed for each panel designed.

3.5 Parametric Study Methods

A two-story, three-story, and four-story panel with a consistent thickness and concrete strength but with varying reinforcement strength, layers of reinforcement, and out-of-plane loading were analyzed to determine the impact the design has on the panel. ACI 318-19 Section 11.8 – Alternative Method for Out-of-Plane Slender Wall Analysis was used as the code the panels were designed to in conjunction with ACI 551.2R-15 as a design guide. Results of the 72 panels are presented in Chapter 4.

Chapter 4 – Results and Discussion

4.1 Results

Each of the 72 panels were designed with the same number of reinforcement bars to clearly compare the moment and deflection that each would experience. To maintain an equal number of bars in order to compare the moment and deflection between the size of reinforcement and how many layers of reinforcement was used, 20 bars were placed in singly reinforced panels and 10 bars were placed in each layer should the panel be doubly reinforced. The use of 20 bars per panel provided an adequate area of steel and satisfied spacing requirements. In the doubly reinforced panels, bars were spaced at 18 inches on center, which is the maximum spacing allowable per ACI 318-19 Section 11.7.3.1. Spacing the bars at a whole inch provided a level of constructability that was practical; spacing to a fraction of an inch would be difficult to measure and install without error or loss of time during the construction process. As seen in **Error! Reference source not found.**, **Error! Reference source not found.**, and * Wall design does not meet the criteria to be designed to be as a slender wall per ACI 318-19 Section 11.8. Wall may be viably designed as a column; outside the scope of this report., the number of reinforcement bars per layer and spacing of the reinforcement did not change between construction site locations.

Some design combinations resulted in an invalid design per ACI 318-19 Alternative Methods of Slender Wall Analysis. If a panel did not meet any of the various limitations the entirety of the design was defined "NP" for not permittable. Of the 72 panels designed, 44 produced valid designs. The 28 panels that were invalid did not meet one or two of three of the limitations defined by ACI 318-19. The four-story panel was invalid regardless of the location due to the axial and self-weight to gross area ratio exceeding 0.06 times the compressive strength of the concrete. This signifies that the wall must be designed as a column per ACI 318-19 Section 6.6 or as a wall per Chapter 14 instead of using the alternative method of slender walls. When the construction location is based in Miami, Florida the panels were deemed invalid for failure to meet further limitations when the panel was singly reinforced. When two layers of reinforcement were used the allowable service deflection was greater than what the panel actually experienced, so the panel in which two layers of reinforcement were used was deemed valid when using the Alternative method for out-of-plane slender wall analysis for the three-story panel. Reference Table 8: Miami, Florida Design Checks for which limitations were deemed "OK" or "NG" for no good. Increasing the amount of steel used within the panel did not result in any of the invalid panels becoming valid.

Table 8: Miami, Florida Design Checks

	OVERALL DESIGN		OVERALL DESIGN
	$P_u \leq 0.06f'_cA_g$	$\Delta_s \leq l_c/150$	
2-STORY W/ 1 LAYER REINFORCEMENT	OK	OK	OK
3-STORY W/ 1 LAYER REINFORCEMENT	OK	NG	NP*
4-STORY W/ 1 LAYER REINFORCEMENT	NG	NG	NP*
2-STORY W/ 2 LAYERS REINFORCEMENT	OK	OK	OK
3-STORY W/ 2 LAYERS REINFORCEMENT	OK	OK	OK
4-STORY W/ 2 LAYERS REINFORCEMENT	NG	OK	NP*

* Wall design does not meet the criteria to be designed to be as a slender wall per ACI 318-19 Section 11.8. Wall may be viably designed as a column; outside the scope of this report.

Table 9: Manhattan, Kansas reinforcement

	MANHATTAN, KANSAS					
	DESIGNATION	STRENGTH OF REINFORCEMENT (ksi)	BAR SIZE	LAYERS OF REINFORCEMENT	# OF BARS/ LAYER	SPACING (in O.C.)
2 STORIES	KS.30'.7.25".4.60.#5.1	60	5	1	20	9
	KS.30'.7.25".4.60.#6.1	60	6	1	20	9
	KS.30'.7.25".4.60.#5.2	60	5	2	10	18
	KS.30'.7.25".4.60.#6.2	60	6	2	10	18
	KS.30'.7.25".4.80.#5.1	80	5	1	20	9
	KS.30'.7.25".4.80.#6.1	80	6	1	20	9
	KS.30'.7.25".4.80.#5.2	80	5	2	10	18
	KS.30'.7.25".4.80.#6.2	80	6	2	10	18
3 STORIES	KS.44'.7.25".4.60.#5.1	60	5	1	20	9
	KS.44'.7.25".4.60.#6.1	60	6	1	20	9
	KS.44'.7.25".4.60.#5.2	60	5	2	10	18
	KS.44'.7.25".4.60.#6.2	60	6	2	10	18
	KS.44'.7.25".4.80.#5.1	80	5	1	20	9
	KS.44'.7.25".4.80.#6.1	80	6	1	20	9
	KS.44'.7.25".4.80.#5.2	80	5	2	10	18
	KS.44'.7.25".4.80.#6.2	80	6	2	10	18
4 STORIES	KS.58'.7.25".4.60.#5.1	NP*	NP*	NP*	NP*	NP*
	KS.58'.7.25".4.60.#6.1	NP*	NP*	NP*	NP*	NP*
	KS.58'.7.25".4.60.#5.2	NP*	NP*	NP*	NP*	NP*
	KS.58'.7.25".4.60.#6.2	NP*	NP*	NP*	NP*	NP*
	KS.58'.7.25".4.80.#5.1	NP*	NP*	NP*	NP*	NP*
	KS.58'.7.25".4.80.#6.1	NP*	NP*	NP*	NP*	NP*
	KS.58'.7.25".4.80.#5.2	NP*	NP*	NP*	NP*	NP*
	KS.58'.7.25".4.80.#6.2	NP*	NP*	NP*	NP*	NP*

* Wall design does not meet the criteria to be designed to be as a slender wall per ACI 318-19 Section 11.8. Wall may be viably designed as a column; outside the scope of this report.

Table 10: Houston, Texas reinforcement

	HOUSTON, TEXAS					
	DESIGNATION	STRENGTH OF REINFORCEMENT (ksi)	BAR SIZE	LAYERS OF REINFORCEMENT	# OF BARS/ LAYER	SPACING (in O.C.)
2 STORIES	TX.30'.7.25".4.60.#5.1	60	5	1	20	9
	TX.30'.7.25".4.60.#6.1	60	6	1	20	9
	TX.30'.7.25".4.60.#5.2	60	5	2	10	18
	TX.30'.7.25".4.60.#6.2	60	6	2	10	18
	TX.30'.7.25".4.80.#5.1	80	5	1	20	9
	TX.30'.7.25".4.80.#6.1	80	6	1	20	9
	TX.30'.7.25".4.80.#5.2	80	5	2	10	18
	TX.30'.7.25".4.80.#6.2	80	6	2	10	18
3 STORIES	TX.44'.7.25".4.60.#5.1	60	5	1	20	9
	TX.44'.7.25".4.60.#6.1	60	6	1	20	9
	TX.44'.7.25".4.60.#5.2	60	5	2	10	18
	TX.44'.7.25".4.60.#6.2	60	6	2	10	18
	TX.44'.7.25".4.80.#5.1	80	5	1	20	9
	TX.44'.7.25".4.80.#6.1	80	6	1	20	9
	TX.44'.7.25".4.80.#5.2	80	5	2	10	18
	TX.44'.7.25".4.80.#6.2	80	6	2	10	18
4 STORIES	TX.58'.7.25".4.60.#5.1	NP*	NP*	NP*	NP*	NP*
	TX.58'.7.25".4.60.#6.1	NP*	NP*	NP*	NP*	NP*
	TX.58'.7.25".4.60.#5.2	NP*	NP*	NP*	NP*	NP*
	TX.58'.7.25".4.60.#6.2	NP*	NP*	NP*	NP*	NP*
	TX.58'.7.25".4.80.#5.1	NP*	NP*	NP*	NP*	NP*
	TX.58'.7.25".4.80.#6.1	NP*	NP*	NP*	NP*	NP*
	TX.58'.7.25".4.80.#5.2	NP*	NP*	NP*	NP*	NP*
	TX.58'.7.25".4.80.#6.2	NP*	NP*	NP*	NP*	NP*

* Wall design does not meet the criteria to be designed to be as a slender wall per ACI 318-19 Section 11.8. Wall may be viably designed as a column; outside the scope of this report.

Table 11: Miami, Florida reinforcement

	MIAMI, FLORIDA					
	DESIGNATION	STRENGTH OF REINFORCEMENT (ksi)	BAR SIZE	LAYERS OF REINFORCEMENT	# OF BARS/ LAYER	SPACING (in O.C.)
2 STORIES	FL.30'.7.25".4.60.#5.1	60	5	1	20	9
	FL.30'.7.25".4.60.#6.1	60	6	1	20	9
	FL.30'.7.25".4.60.#5.2	60	5	2	10	18
	FL.30'.7.25".4.60.#6.2	60	6	2	10	18
	FL.30'.7.25".4.80.#5.1	80	5	1	20	9
	FL.30'.7.25".4.80.#6.1	80	6	1	20	9
	FL.30'.7.25".4.80.#5.2	80	5	2	10	18
	FL.30'.7.25".4.80.#6.2	80	6	2	10	18
3 STORIES	FL.44'.7.25".4.60.#5.1	NP*	NP*	NP*	NP*	NP*
	FL.44'.7.25".4.60.#6.1	NP*	NP*	NP*	NP*	NP*
	FL.44'.7.25".4.60.#5.2	60	5	1	20	9
	FL.44'.7.25".4.60.#6.2	60	6	1	20	9
	FL.44'.7.25".4.80.#5.1	NP*	NP*	NP*	NP*	NP*
	FL.44'.7.25".4.80.#6.1	NP*	NP*	NP*	NP*	NP*
	FL.44'.7.25".4.80.#5.2	80	5	2	10	18
	FL.44'.7.25".4.80.#6.2	80	6	2	10	18
4 STORIES	FL.58'.7.25".4.60.#5.1	NP*	NP*	NP*	NP*	NP*
	FL.58'.7.25".4.60.#6.1	NP*	NP*	NP*	NP*	NP*
	FL.58'.7.25".4.60.#5.2	NP*	NP*	NP*	NP*	NP*
	FL.58'.7.25".4.60.#6.2	NP*	NP*	NP*	NP*	NP*
	FL.58'.7.25".4.80.#5.1	NP*	NP*	NP*	NP*	NP*
	FL.58'.7.25".4.80.#6.1	NP*	NP*	NP*	NP*	NP*
	FL.58'.7.25".4.80.#5.2	NP*	NP*	NP*	NP*	NP*
	FL.58'.7.25".4.80.#6.2	NP*	NP*	NP*	NP*	NP*

* Wall design does not meet the criteria to be designed to be as a slender wall per ACI 318-19 Section 11.8. Wall may be viably designed as a column; outside the scope of this report.

The largest factored moment determined which of the four load cases under investigation governed. Load case 4a or 4b governed for all combinations to determine the maximum moment and deflection due to dead, live, and either snow loads or roof live loads being accounted for in addition to the wind load acting laterally. The applied axial and lateral load that coincided with the determined governing load are shown in **Error! Reference source not found.**, **Error! Reference source not found.**, and **Error! Reference source not found.**. The axial and lateral load that is applied to the panel is consistent where the construction location and number of stories are the same, though the resulting moment and deflection vary.

Table 12: Moment and deflection for panels designed in Manhattan, Kansas

	MANHATTAN, KANSAS						
	DESIGNATION	APPLIED AXIAL LOAD (k)	APPLIED LATERAL LOAD (klf)	M_u (k-ft)	Δ_u (in)	M_s (k-ft)	Δ_s (in)
2 STORIES	KS.30'.7.25".4.60.#5.1	37.0	0.73	24.0	0.545	14.0	0.238
	KS.30'.7.25".4.60.#6.1	37.0	0.73	23.9	0.465	13.9	0.203
	KS.30'.7.25".4.60.#5.2	37.0	0.73	23.6	0.340	13.6	0.146
	KS.30'.7.25".4.60.#6.2	37.0	0.73	23.4	0.252	13.4	0.108
	KS.30'.7.25".4.80.#5.1	37.0	0.73	24.1	0.605	14.2	0.266
	KS.30'.7.25".4.80.#6.1	37.0	0.73	23.9	0.500	13.9	0.219
	KS.30'.7.25".4.80.#5.2	37.0	0.73	23.6	0.363	13.6	0.157
	KS.30'.7.25".4.80.#6.2	37.0	0.73	23.4	0.271	13.4	0.117
3 STORIES	KS.44'.7.25".4.60.#5.1	65.9	1.18	48.5	0.899	25.1	0.427
	KS.44'.7.25".4.60.#6.1	65.9	1.18	48.0	0.766	24.5	0.358
	KS.44'.7.25".4.60.#5.2	65.9	1.18	47.0	0.561	23.4	0.252
	KS.44'.7.25".4.60.#6.2	65.9	1.18	46.5	0.415	22.8	0.184
	KS.44'.7.25".4.80.#5.1	65.9	1.18	49.0	0.997	25.7	0.482
	KS.44'.7.25".4.80.#6.1	65.9	1.18	48.2	0.825	24.8	0.389
	KS.44'.7.25".4.80.#5.2	65.9	1.18	47.2	0.599	23.6	0.271
	KS.44'.7.25".4.80.#6.2	65.9	1.18	46.6	0.446	22.9	0.199
4 STORIES	KS.58'.7.25".4.60.#5.1	NP*	NP*	NP*	NP*	NP*	NP*
	KS.58'.7.25".4.60.#6.1	NP*	NP*	NP*	NP*	NP*	NP*
	KS.58'.7.25".4.60.#5.2	NP*	NP*	NP*	NP*	NP*	NP*
	KS.58'.7.25".4.60.#6.2	NP*	NP*	NP*	NP*	NP*	NP*
	KS.58'.7.25".4.80.#5.1	NP*	NP*	NP*	NP*	NP*	NP*
	KS.58'.7.25".4.80.#6.1	NP*	NP*	NP*	NP*	NP*	NP*
	KS.58'.7.25".4.80.#5.2	NP*	NP*	NP*	NP*	NP*	NP*
	KS.58'.7.25".4.80.#6.2	NP*	NP*	NP*	NP*	NP*	NP*

* Wall design does not meet the criteria to be designed to be as a slender wall per ACI 318-19 Section 11.8. Wall may be viably designed as a column; outside the scope of this report.

Table 13: Moment and deflection for panels designed in Houston, TX

	HOUSTON, TEXAS						
	DESIGNATION	APPLIED AXIAL LOAD (k)	APPLIED LATERAL LOAD (klf)	M_u (k-ft)	Δ_u (in)	M_s (k-ft)	Δ_s (in)
2 STORIES	TX.30'.7.25".4.60.#5.1	37.0	1.15	34.6	0.792	20.7	0.360
	TX.30'.7.25".4.60.#6.1	37.0	1.15	34.4	0.675	20.5	0.304
	TX.30'.7.25".4.60.#5.2	37.0	1.15	34.0	0.494	20.1	0.224
	TX.30'.7.25".4.60.#6.2	37.0	1.15	33.7	0.366	19.8	0.164
	TX.30'.7.25".4.80.#5.1	37.0	1.15	34.8	0.879	20.9	0.400
	TX.30'.7.25".4.80.#6.1	37.0	1.15	34.5	0.727	20.6	0.328
	TX.30'.7.25".4.80.#5.2	37.0	1.15	34.0	0.528	20.2	0.238
	TX.30'.7.25".4.80.#6.2	37.0	1.15	33.7	0.393	19.9	0.175
3 STORIES	TX.44'.7.25".4.60.#5.1	65.9	1.86	65.4	1.297	37.1	0.644
	TX.44'.7.25".4.60.#6.1	65.9	1.86	64.7	1.106	36.1	0.535
	TX.44'.7.25".4.60.#5.2	65.9	1.86	63.6	0.809	34.6	0.385
	TX.44'.7.25".4.60.#6.2	65.9	1.86	62.9	0.600	33.6	0.277
	TX.44'.7.25".4.80.#5.1	65.9	1.86	66.0	1.439	37.9	0.723
	TX.44'.7.25".4.80.#6.1	65.9	1.86	65.1	1.190	36.5	0.581
	TX.44'.7.25".4.80.#5.2	65.9	1.86	63.8	0.864	34.9	0.411
	TX.44'.7.25".4.80.#6.2	65.9	1.86	63.0	0.643	33.8	0.298
4 STORIES	TX.58'.7.25".4.60.#5.1	NP*	NP*	NP*	NP*	NP*	NP*
	TX.58'.7.25".4.60.#6.1	NP*	NP*	NP*	NP*	NP*	NP*
	TX.58'.7.25".4.60.#5.2	NP*	NP*	NP*	NP*	NP*	NP*
	TX.58'.7.25".4.60.#6.2	NP*	NP*	NP*	NP*	NP*	NP*
	TX.58'.7.25".4.80.#5.1	NP*	NP*	NP*	NP*	NP*	NP*
	TX.58'.7.25".4.80.#6.1	NP*	NP*	NP*	NP*	NP*	NP*
	TX.58'.7.25".4.80.#5.2	NP*	NP*	NP*	NP*	NP*	NP*
	TX.58'.7.25".4.80.#6.2	NP*	NP*	NP*	NP*	NP*	NP*

* Wall design does not meet the criteria to be designed to be as a slender wall per ACI 318-19 Section 11.8. Wall may be viably designed as a column; outside the scope of this report.

Table 14: Moment and deflection for panels designed in Miami, Florida

	MIAMI, FLORIDA						
	DESIGNATION	APPLIED AXIAL LOAD (k)	APPLIED LATERAL LOAD (klf)	M_u (k-ft)	Δ_u (in)	M_s (k-ft)	Δ_s (in)
2 STORIES	FL.30'.7.25".4.60.#5.1	37.0	1.68	48.3	1.105	29.2	0.507
	FL.30'.7.25".4.60.#6.1	37.0	1.68	47.9	0.942	28.9	0.428
	FL.30'.7.25".4.60.#5.2	37.0	1.68	47.4	0.690	28.3	0.315
	FL.30'.7.25".4.60.#6.2	37.0	1.68	47.0	0.511	27.9	0.231
	FL.30'.7.25".4.80.#5.1	37.0	1.68	48.6	1.226	29.5	0.563
	FL.30'.7.25".4.80.#6.1	37.0	1.68	48.1	1.014	29.0	0.462
	FL.30'.7.25".4.80.#5.2	37.0	1.68	47.5	0.736	28.4	0.335
	FL.30'.7.25".4.80.#6.2	37.0	1.68	47.1	0.548	28.0	0.247
3 STORIES	FL.44'.7.25".4.60.#5.1	NP*	NP*	NP*	NP*	NP*	NP*
	FL.44'.7.25".4.60.#6.1	NP*	NP*	NP*	NP*	NP*	NP*
	FL.44'.7.25".4.60.#5.2	65.9	2.71	85.5	1.125	48.7	0.542
	FL.44'.7.25".4.60.#6.2	65.9	2.71	84.5	0.833	47.2	0.390
	FL.44'.7.25".4.80.#5.1	NP*	NP*	NP*	NP*	NP*	NP*
	FL.44'.7.25".4.80.#6.1	NP*	NP*	NP*	NP*	NP*	NP*
	FL.44'.7.25".4.80.#5.2	65.9	2.71	85.8	1.200	49.1	0.578
	FL.44'.7.25".4.80.#6.2	65.9	2.71	84.8	0.894	47.5	0.419
4 STORIES	FL.58'.7.25".4.60.#5.1	NP*	NP*	NP*	NP*	NP*	NP*
	FL.58'.7.25".4.60.#6.1	NP*	NP*	NP*	NP*	NP*	NP*
	FL.58'.7.25".4.60.#5.2	NP*	NP*	NP*	NP*	NP*	NP*
	FL.58'.7.25".4.60.#6.2	NP*	NP*	NP*	NP*	NP*	NP*
	FL.58'.7.25".4.80.#5.1	NP*	NP*	NP*	NP*	NP*	NP*
	FL.58'.7.25".4.80.#6.1	NP*	NP*	NP*	NP*	NP*	NP*
	FL.58'.7.25".4.80.#5.2	NP*	NP*	NP*	NP*	NP*	NP*
	FL.58'.7.25".4.80.#6.2	NP*	NP*	NP*	NP*	NP*	NP*

* Wall design does not meet the criteria to be designed to be as a slender wall per ACI 318-19 Section 11.8. Wall may be viably designed as a column; outside the scope of this report.

4.2 Discussion

For the most part, the limitations set by ACI 318-19 Sections 11.8.1.1.a-e did not prove to be cumbersome to meet. In the case of the four-story panel designs, the axial load applied to the slender panel caused the axial stress check for any of the four-story designs to not pass, limitation d. If a thicker panel were to be used the limitation would be met. An eight-inch panel at minimum would need to be used to satisfy this limitation; however, for construction purposes it is recommended to use a 9 1/4-inch panel. The actual dimensions of 2x10 lumber correlate with 9 1/4 inches so the boards would not have to be ripped, simplifying the construction forming process. Concrete with a higher compressive strength could also be used, but the use of high strength concrete is not commonly used in tilt-up design. When the panels were located in Miami, Florida and were singly reinforced the actual service load deflection was greater than the allowable service load deflection, thus limitation e failed for such cases. Doubly reinforcing the panels results in the limitation passing, and should be done in regions where the lateral load is greater than 2.68 klf, the lateral load that Houston, Texas experiences. Miami, Florida experiences a lateral load of 2.71 klf. Further research should be done to determine the exact lateral load that can be applied before the service level deflection exceeds what is allowable.

Tables 15 – 18 present break downs of the results found in the parametric study.

Table 15: Panel Designed in Varying Construction Site Locations

DESIGNATION	APPLIED AXIAL LOAD (k)	APPLIED LATERAL LOAD (klf)	M_u (k-ft)	Δ_u (in)	M_s (k-ft)	Δ_s (in)
KS.30'.7.25".4.60.#5.1	37.0	0.73	24.0	0.545	14.0	0.238
TX.30'.7.25".4.60.#5.1	37.0	1.15	34.6	0.792	20.7	0.36
FL.30'.7.25".4.60.#5.1	37.0	1.68	48.3	1.105	29.2	0.507

The axial load applied for each panel is equal as the gravity loads acting on the panel are consistent between construction site locations. While a snow load is applied to the panel in Manhattan, Kansas, the load combination that includes snow load does not govern. The applied lateral load increases as the wind speed increases between locations. The moment and deflection in turn also increase as the out-of-plane wind load increase.

Table 16: Panel Designed with Varying Number of Stories

DESIGNATION	APPLIED AXIAL LOAD (k)	APPLIED LATERAL LOAD (klf)	M_u (k-ft)	Δ_u (in)	M_s (k-ft)	Δ_s (in)
KS.30'.7.25".4.60.#5.1	37.0	0.73	24.0	0.545	14.0	0.238
KS.44'.7.25".4.60.#5.1	65.9	1.18	48.5	0.899	25.1	0.427
TX.30'.7.25".4.60.#5.1	37.0	1.15	34.6	0.792	20.7	0.360
TX.44'.7.25".4.60.#5.1	65.9	1.86	65.4	1.297	37.1	0.644

Results from two- and three-story panels are presented in Table 16 for construction sites located in Manhattan, Kansas and Houston, Texas. When the height of the panel increases and all other parameters remain consistent the moment and deflection the panel experiences increases. This is expected as the applied axial and lateral loads increase as the number of stories increases.

Table 17: Panel Designed with Varying Strength of Reinforcing Steel

DESIGNATION	APPLIED AXIAL LOAD (k)	APPLIED LATERAL LOAD (klf)	M_u (k-ft)	Δ_u (in)	M_s (k-ft)	Δ_s (in)
KS.30'.7.25".4.60.#5.1	37.0	0.73	24.0	0.545	14.0	0.238
KS.30'.7.25".4.80.#5.1	37.0	0.73	24.1	0.605	14.2	0.266
TX.30'.7.25".4.60.#5.1	37.0	1.15	34.6	0.792	20.7	0.360
TX.30'.7.25".4.80.#5.1	37.0	1.15	34.8	0.879	20.9	0.400
FL.30'.7.25".4.60.#5.1	37.0	1.68	48.3	1.105	29.2	0.507
FL.30'.7.25".4.80.#5.1	37.0	1.68	48.6	1.226	29.5	0.563

When high strength reinforcement is used instead of 60 ksi reinforcement and all other parameters remain the same, the moment and deflection increase. This is due to P-Δ effects occurring in the panel. Eq. 2-12 and Eq. 2-13 show why. The effective area of the steel decreases when a higher strength of steel is used, resulting in a lesser cracked moment of inertia. This ultimately results in the moment and deflection of the panel to be greater.

Table 18: Panel Designed with Varying Size of Reinforcing Bars

DESIGNATION	APPLIED AXIAL LOAD (k)	APPLIED LATERAL LOAD (klf)	M_u (k-ft)	Δ_u (in)	M_s (k-ft)	Δ_s (in)
KS.30'.7.25".4.60.#5.1	37.0	0.73	24.0	0.545	14.0	0.238
KS.30'.7.25".4.60.#6.1	37.0	0.73	23.3	0.199	13.3	0.085
TX.30'.7.25".4.60.#5.1	37.0	1.15	34.6	0.792	20.7	0.360
TX.30'.7.25".4.60.#6.1	37.0	1.15	34.4	0.675	20.5	0.304
FL.30'.7.25".4.60.#5.1	37.0	1.68	48.3	1.105	29.2	0.507
FL.30'.7.25".4.60.#6.1	37.0	1.68	47.9	0.942	28.9	0.428

When the panel was designed with a #6 bar instead of a #5 bar, and all other parameters remained the same, the moment and deflection decreased. This is expected as the area of steel present within the panel increases when a larger bar size is used. No limitations fail to be met due to the use of a #5 bar instead of a #6 bar, so for the panel designed in this parametric study it is recommended that the #5 bar be used as it lessen the weight of the panel and saves money.

Table 19: Panel Designed with Varying Layers of Reinforcement

DESIGNATION	APPLIED AXIAL LOAD (k)	APPLIED LATERAL LOAD (klf)	M_u (k-ft)	Δ_u (in)	M_s (k-ft)	Δ_s (in)
KS.30'.7.25".4.60.#5.1	37.0	0.73	24.0	0.545	14.0	0.238
KS.30'.7.25".4.60.#5.2	37.0	0.73	23.6	0.34	13.6	0.146
TX.30'.7.25".4.60.#5.1	37.0	1.15	34.6	0.792	20.7	0.360
TX.30'.7.25".4.60.#5.2	37.0	1.15	34	0.494	20.1	0.224
FL.30'.7.25".4.60.#5.1	37.0	1.68	48.3	1.105	29.2	0.507
FL.30'.7.25".4.60.#5.2	37.0	1.68	47.4	0.690	28.3	0.315

Doubly reinforcing the panel instead of singly reinforcing it proved to result in a lesser moment and deflection that the panel experiences even though the number of bars in tension and used to determine the effective area of steel is half. The distance from extreme compression fiber to centroid of longitudinal reinforcement, *d*, used in Eq. 2-12 proved to be a critical factor as both the moment and deflection depend on I_{cr}.

The moment and deflection due to service loads are lesser than the calculated factored moment and out-of-plane deflection for the same panel as the service loads are unfactored, and thus less. Load case 6 governs when determining the moment and deflection due to service loads. The resulting moment and out-of-plane deflections from the service loads exhibit the same trends as the factored moments and deflections.

Chapter 5 – Conclusion & Recommendations

5.1 Conclusion

The parametric study performed in this report calculated the out-of-plane deflection based on the factored moment as well as the moment due to service loads for 24 panels with varying parameters designed for three locations with varying wind speeds. Based on the results found in the parametric study it can be concluded that using the alternative method for slender wall analysis is only valid for two- or three-story buildings when a 7.25" panel of normal weight concrete is utilized. The location of the construction site is crucial when designed a multistory tilt-up panel. When the construction site experiences relatively high wind speeds multiple limitations of the alternative method may fail to be met. The use of two layers of reinforcement instead of one layer resulted in a lesser moment and deflection due to the difference in d used in the calculation of the cracked moment of inertia. The use of high strength steel, 80 ksi, resulted in greater moments and deflections that the panel experiences compared to when 60 ksi steel is used due to the effects of P-Δ on the panel.

5.2 Recommendations

The equations presented in ACI 318-19 Section 11.8 – Alternative Method for Out-of-Plane Slender Wall Analysis should be further evaluated to determine the validity and accuracy of the moment and deflection equations. Until full scale test data on multistory tilt-up panels is available it is not possible to determine the precision of these equations used in the Alternative method for out-of-plane slender wall analysis. Updated equations should be generated from this full-scale testing and implemented into the analysis method.

Further research should be done to analyze how designing multistory tilt-up as a column with varying parameters affects the panel. This study showed that regardless of the applied lateral loads and the amount of reinforcement placed within the panel, the four-story panel design was invalid and should be designed as a column.

References

ACI Committee 318. (2008). *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary on Building Code Requirements for Structural Concrete (ACI 318R-08).* American Concrete Institute.

ACI Committee 318. (2011). *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary on Building Code Requirements for Structural Concrete (ACI 318R-11).* American Concrete Institute.

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

ACI Committee 318. (2019). *Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary on Building Code Requirements for Structural Concrete (ACI 318R-19).* American Concrete Institute.

ACI Committee 551. (2015). *Design Guide for Tilt-Up Concrete Panels*. American Concrete Institute.

American Institute of Steel Construction (AISC). (2017). *Steel Construction Manual, Fifteenth Edition*. American Institute of Steel Construction.

American Society of Civil Engineers (ASCE). (2017). *Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE/SEI 7-16).* American Society of Civil Engineers (ASCE).

Baty II, J. R. (2014, July). *Changes to ACI 318 for Tilt-Up Wall Panels*. STRUCTURE Magazine.

Bloomquist, M. (2016, January 28). *World's Tallest Tilt-Up Panel Erected Near Miami, Florida: TILT-UP TODAY – A publication of the Tilt-Up Concrete Association (TCA)*. TILT-up TODAY.

Dayton Concrete Accessories. (2008). *Tilt-Up Construction General and Technical Information*.

Design of Multi-Story Tilt-Up Panels for Out-of-Plane Wind Loads: White Paper. (2018). ACI Committee 551.

Freedman, S. (1999, October). *Loadbearing Architectural Precast Concrete Wall Panels*. PCI Journal.

Griffin, Ph. D., P.E., P.M.P., J., & Bloomquist, M. (2014, September). *Tilt-Up Pushes the Height Envelope on a Six-Story Office Building in Texas*. STRUCTURE Magazine.

Kripanarayanan, K. M. (1977). Interesting Aspects of the Empirical Equation. *ACI Journal*, *Title No. 74-20*, 204–207.

Lawson, J. (2007). Deflection Limits for Tilt-Up Wall Serviceability - The History Behind Changes to Chapter 14. *Concrete International*, 33–38.

Lawson, J. W. (2015, January). *Crack Control Measures for Tilt-Up Concrete Panels*. STRUCTURE Magazine.

MacKinnon, J. (2007, October 1). *Considerations When Designing and Building A Tilt-Up Project : TILT-UP TODAY – A publication of the Tilt-Up Concrete Association (TCA)*. TILT-up TODAY.

Oberlender, G. D., & Everard, N. J. (1977). Investigation of Reinforced Concrete Walls. *ACI Journal*, *Title No, 74-28*, 256–263.

Remmetter, P.E., Mark. E., Walters, P.E., S. A., & Steinbicker, P.E., S.E., J. J. (2008, October 1). *Multi-Story Tilt-Up Buildings — A Design Approach: TILT-UP TODAY – A publication of the Tilt-Up Concrete Association (TCA)*. TILT-up TODAY.

Salse, E. A. B., & Fintel, M. (1973). *Strength, Stiffness and Ductility Properties of Slender Shear Walls*. 5th WCEE, Rom, 919-928.

Tilt Wall Ontario Inc, & Overbeek, L. (2017, July 31). *Tilt-Up Construction vs. Precast: What is the Difference?* Tiltwall Tilt-up Construction; Tilt Wall Ontario Inc.

Tilt-Up Concrete Association. (n.d.). *Tilt-Up Basics | Tilt-up Concrete Association*. Tilt-Up.org.

Verity, Wells, B., & Gainey, T. (2019, July 24). *7 Factors to Consider When Choosing Precast Concrete vs. Tilt Wall Construction Method*. Verity Commercial.

Vulcraft. (2008). *Vulcraft Roof and Floor Deck*. Nucor.

Williams, P.E., C.E., S.E., AIA, LEED AP, J. M. (2018, April). *A New Trend in Multi-Story Office Construction*. STRUCTURE Magazine.

Appendix A – Design Example

A design example of panel KS.28’7.25”4.60.#5.1 is presented in this appendices.

STEP	COMPUTATIONS	REF.
PANEL PARAMETERS	LOCATION: **MANHATTAN, KS**	
	# OF STORIES = 2	
	h_1 = 14 ft	
	h_2 = 14 ft	
	h_3 = 0 ft	
	h_4 = 0 ft	
	h_p = 2 ft	
	$\Sigma h + h_p$ = 30 ft	
	b = 15 ft	
	t = 7.25 in	
	# OF LAYERS OF STEEL = 1	
	d = t - dchair - 0.5db = 3.9 in	
	CONCRETE DENSITY = NW	
	γ_c = 150 pcf	
	$f_r = 0.75\lambda \text{sqrt}(f'_c)$ = 0.474 ksi	
	λ = 1	
	f'_c = 4000 psi	
	f_y = 60 ksi	
	E_s = 29000 ksi	
	E_c = 3605 ksi	

STEP	COMPUTATIONS			REF. ASCE 7-16
DEAD LOADING	ROOF			
	SINGLE-PLY MEMBRANE ROOFING	3	psf	Tb. C3.1-1a
	8" RIGID INSULATION	4	psf	
	MECHANICAL/ ELECTRICAL	5	psf	
	CEILING SYSTEM	2	psf	
	MISC.	5	psf	
	TOTAL LOAD ON ROOF DECK	**19**	**psf**	
	1.5B22 ROOF DECK	2.7	psf	
	TOTAL ROOF DEAD LOAD	**21.7**	**psf**	
	FLOORS			
	CARPET	6.8	psf	Tb. C3.1-1a
	MECHANICAL/ ELECTRICAL	5	psf	
	CEILING SYSTEM	2	psf	
	MISC.	5	psf	
	TOTAL LOAD ON FLOOR DECK	**18.8**	**psf**	
	COMPOSTIE DECK W/ 3.5" LW TOPPING	34	psf	
	TOTAL FLOOR DEAD LOAD	**52.8**	**psf**	
LIVE LOADING	ROOF			
	ORDINARY ROOF	20	psf	Tb. 4.3-1
	TOTAL ROOF LIVE LOAD	**20**	**psf**	
	FLOORS			
	OFFICES	65	psf	Tb. 4.3-1
	TOTAL FLOOR LIVE LOAD	**65**	**psf**	
ROOF DECK	$W_u = 1.2D + 1.6L =$	54.8	psf	
	span =	5	ft	
	ROOF DECK SELECTION =	**3N20 ROOF DECK**		VULCRAFT DESIGN MANUAL
	ROOF WEIGHT =	2.71	psf	
	3 SPANS, CAPACITY =	90	psf	
FLOOR DECK	$W_u = 1.2D + 1.6L =$	126.6	psf	
	span =	5	ft	
	FLOOR DECK SELECTION =	**1.5VLR18 2.5" LW TOPPING**		VULCRAFT DESIGN MANUAL
	SLAB WEIGHT =	34	psf	
	CAPACITY =	209	psf	

STEP	COMPUTATIONS	REF. ASCE 7-16
SNOW LOADING	p_g = 20 psf	Fig. 7.2-1
	Exposure B, Assume Partially Exposed, C_e = 1.0	Tb. 7.3-1
	Roof Insulated, No benefit from space below, C_t = 1.1	Tb. 7.3-2
	Risk Category II, I_s = 1.0	Tb. 1.5-2
	$p_f = 0.7C_eC_tI_sp_g$	
	= 0.7 * 1 * 1.1 * 1 * 20 = 15.4 psf	Eq. 7.3-1
	FOR LOCATIONS WHERE p_g IS 20 PSF OR LESS, BUT NOT 0, ALL ROOFS SHALL INCLUDE A 5 PSF RAIN-ON-SNOW SURCHARGE LOAD.	Sec. 7.10
	pg = 20 psf ≤ 20 psf	
	APPLIES	
	$p_s = p_f + 5$ = 20.4 psf	
	$p_{min} = I_sp_g$	
	= 1 * 20 = 20.0 psf	Sec. 7.3.4
	CHECK: $p_{min} \le p_s$ **OK**	
	l_u = 90 ft	
	$\gamma = 0.13p_g + 14 \le 30$	Eq. 7.7-1
	= 0.13 * 20 + 14 = 16.6 pcf	
	$h_b = p_f / \gamma$	Sec. 7.7.1
	= 15.4 / 16.6 = 0.9 ft	
	$h_c = h_p - h_b$	
	= 2 - 0.9 = 1.1 ft	
	h_c / h_b	
	= 1.1 / 0.9 = 1.2	
	CHECK: IF $h_c / h_b > 0.2$ CONSIDER DRIFT	Sec. 7.7.1

STEP	COMPUTATIONS	REF. ASCE 7-16
	$h_d = 0.75\left(0.43\sqrt[3]{l_u}\sqrt[4]{p_g + 10} - 1.5\right)$	
	= 0.75 * (0.43 * 90^(1/3)* (20 + 10)^(1/4) - 1.5) = $\boxed{2.3}$ ft	Fig. 7.6-1
	h_d > h_c	
	w = 4hd^2 / hc = $\boxed{19.0}$ ft	Sec. 7.7.1
	$p_d = h_d \gamma + p_s$	
	2.3 * 16.6 + 20.4 = $\boxed{57.9}$ psf	Sec. 7.7.1
	CONSERVATIVELY, USE THE AVERAGE OF p_d AND p_s OVER THE ENTIRE ROOF AREA.	
	$p_s = (p_d + p_s) / 2$	
	(57.9 + 20.4) / 2 = $\boxed{39.1}$ psf	

STEP	COMPUTATIONS	REF. ASCE 7-16
WIND LOADING	*UTILIZE COMPONENTS & CLADDING.*	
	$h = \Sigma h =$ 28 ft	
	CHECK: $\Sigma h \leq 60$ ft **OK**	
	LOW RISE BUILDING: YES	Sec. 30.4
	$V =$ 114 mph	Fig. 26.5-1A
	EXPOSURE B, $K_z =$ 0.70	Tb. 26.10-1
	$K_{zt} =$ 1.0	Sec. 26.8
	$K_d =$ 0.85	Tb. 26.6-1
	$K_e =$ 0.96	Tb. 26.9-1
	PARTIALLY ENCLOSED, $GC_{pi} =$ 0.55	Tb. 26.13-1
	$q_h = 0.00256 K_z K_{zt} K_d K_e V^2$	Eq. 26.10-1
	= 0.00256 * 0.7 * 1 * 0.85 * 0.96 * 114^2 = 19.0 psf	
	$p = q_h[(GC_p) - (GC_{pi})]$	Eq. 30.3-1
	(see table below)	Fig. 30.3-1

ZONE	AREA	GC_p (-)	GC_p (+)	p (-) psf	p (+) psf
5	420	-0.83	0.73	-26.2	24.3

STEP	COMPUTATIONS	REF. ACI318-19

APPLIED LOADS

b =	5	ft
BAY LENGTH =	30	ft
BEAMS FRAMING INTO ROOF =	3	
BEAMS FRAMING INTO FLOOR =	3	
# OF STORIES =	2	

ASSUME WIND LOADS GOVERN OVER SEISMIC LOADS.
WIND LOADS ONLY APPLIED OUT-OF-PLANE, NOT VERTICAL.

$P_{DL,ROOF}$ =	4.9	k
$P_{LL,ROOF}$ =	4.5	k
$P_{DL,FLOOR}$ =	11.9	k
$P_{LL,FLOOR}$ =	14.6	k
P_{SNOW} =	8.8	k
$P_{WIND,HORIZ}$ =	0.7	klf

$h = \Sigma h$ =	28	ft
$e_{cc,ROOF}$ =	3	in

LOAD COMBOS

MAX MOMENT OCCURS MIDHEIGHT OF FIRST STORY:

	$P_{u,VERT}$		$w_{u,HORIZ}$		REF.
1. 1.4D =	23.5	k	0	klf	Tb. 5.3.1
2a. 1.2D + 1.6L + 0.5L_r =	45.8	k	0	klf	
2b. 1.2D + 1.6L + 0.5S =	36.5	k	0	klf	
3a. 1.2D + 1.6L_r + L =	27.3	k	0	klf	
3b. 1.2D + 1.6S + L =	48.8	k	0	klf	
3c. 1.2D + 1.6L_r + 0.5W =	27.3	k	0.37	klf	
3d. 1.2D + 1.6S + 0.5W =	34.2	k	0.37	klf	
4a. 1.2D + 1.0W + L + 0.5L_r =	37.0	k	0.73	klf	
4b. 1.2D + 1.0W + L + 0.5S =	**39.1**	k	**0.73**	klf	
5. 1.2D + 1.0E + L + 0.2S =	36.5	k	0	klf	
6. 0.9D + 1.0W =	**15.1**	k	**0.73**	klf	
7. 0.9D + 1.0E =	15.1	k	0	klf	

LC 4b AND LC 6 WILL BE EVALUATED TO DETERMINE THE CONTROLLING CASE FOR WHEN THE MAX MOMENT OCCURS AT THE MIDHEIGHT OF THE FIRST STORY.

STEP	COMPUTATIONS	REF. ACI318-19
	MAX MOMENT OCCURS MIDHEIGHT OF LARGEST STORY:	

	$P_{u, VERT}$		$w_{u, HORIZ}$		
1. 1.4D =	23.5	k	0	klf	Tb. 5.3.1
2a 1.2D + 1.6L + 0.5L_r =	45.8	k	0	klf	
2b. 1.2D + 1.6L + 0.5S =	36.5	k	0	klf	
3a 1.2D + 1.6L_r + L =	27.3	k	0	klf	
3b. 1.2D + 1.6S + L =	48.8	k	0	klf	
3c. 1.2D + 1.6L_r + 0.5W =	27.3	k	0.37	klf	
3d. 1.2D + 1.6S + 0.5W =	34.2	k	0.37	klf	
4a 1.2D + 1.0W + L + 0.5L_r =	**37.0**	k	**0.73**	klf	
4b. 1.2D + 1.0W + L + 0.5S =	39.1	k	0.73	klf	
5. 1.2D + 1.0E + L + 0.2S =	36.5	k	0	klf	
6. 0.9D + 1.0W =	**15.1**	k	**0.73**	klf	
7. 0.9D + 1.0E =	15.1	k	0	klf	

LC 4a AND LC 6 WILL BE EVALUATED TO DETERMINE THE CONTROLLING CASE FOR WHEN THE MAX MOMENT OCCURS AT THE MIDHEIGHT OF THE LARGEST STORY.

LC 4b:	+ M =	18.0	k-ft
	+ x =	14	ft *FROM EDGE*
	- M =	-10.1	k-ft
	- x =	5.3	ft *FROM EDGE*
LC $6_{1ST\ STORY}$:	+ M =	18.0	k-ft
	+ x =	14	ft *FROM EDGE*
	- M =	-10.1	k-ft
	- x =	5.3	ft *FROM EDGE*
LC 4a:	+ M =	18.0	k-ft
	+ x =	14	ft *FROM EDGE*
	- M =	-10.1	k-ft
	- x =	5.3	ft *FROM EDGE*
LC $6_{LARGEST\ STORY}$:	+ M =	18.0	k-ft
	+ x =	14	ft *FROM EDGE*
	- M =	-10.1	k-ft
	- x =	5.3	ft *FROM EDGE*

STEP	COMPUTATIONS	REF.
PANEL WEIGHT	*PANEL WEIGHT @ MAX POSITIVE MOMENT, WHEN MOMENT OCCURS AT MIDHEIGHT OF FIRST STORY:* $t\gamma_c b\,(x^{+})$ $= 7.25/12 * 150 * 15 * (28 - 14) / 1000 = \boxed{19.0}$ k *PANEL WEIGHT @ MAX NEGATIVE MOMENT, WHEN MOMENT OCCURS AT THE FIRST STORY:* $t\gamma_c b\,(x^{-})$ $= 7.25/12 * 150 * 15 * (28 - 5.3) / 1000 = \boxed{30.9}$ k *PANEL WEIGHT @ MAX POSITIVE MOMENT, WHEN MOMENT OCCURS AT MIDHEIGHT OF LARGEST STORY:* $t\gamma_c b\,(x^{+})$ $= 7.25/12 * 150 * 15 * (28 - 14) / 1000 = \boxed{19.0}$ k *PANEL WEIGHT @ MAX NEGATIVE MOMENT, WHEN MOMENT OCCURS AT THE LARGEST STORY:* $t\gamma_c b\,(x^{-})$ $= 7.25/12 * 150 * 15 * (28 - 5.3) / 1000 = \boxed{7.1}$ k	

STEP	COMPUTATIONS	REF. ACI318-19
MAX MOMENT @ MID-HEIGHT OF 1ST STORY: LC 4b	*LC 4b AND LC 6 WILL BE EVALUATED TO DETERMINE THE CONTROLLING CASE FOR WHEN THE MAX MOMENT OCCURS AT THE MIDHEIGHT OF THE FIRST STORY.* *ASSUME REINFORCEMENT REQUIRED:* BAR SIZE = 5 # OF BARS = 20 d_b = 0.625 in A_s = 0.31 $A_{s,prov}$ = 6.2 in^2 *CHECK VERTICAL STRESS AT MIDHEIGHT SECTION OF THE 1ST STORY PANEL PER ACI 318-19, SECTION 11.8.1.1:* PANEL S.W. = $t\gamma_c b\,(\Sigma h - h_1 / 2)$ = 7.25 / 12 * 150 * 15 * (28 - 14 / 2) / 1000 = 28.5 k $P_{um} = P_u + 1.2$(PANEL S.W.) = Pu + 1.2 * 28.5 = 73.4 k $w_u = 0.5P_{u,HORIZ}b$ = 0.5 * Pu,HORIZ * 15 / 1000 = 0.006 klf P_{um} / A_g = 73.4 * 1000 / (7.25 * 15 * 12) = 56.2 psi $0.06f'_c$ = 0.06 * 4000 = 240.0 psi **CHECK:** $P_{um} / A_g \le 0.06f'_c$ **OK**	Appendix B 11.8.1d

STEP	COMPUTATIONS	REF. ACI318-19
	CHECK DESIGN MOMENT STRENGTH TO COMPARE TO THE MAXIMUM POSITIVE MOMENT:	
	Pum = 1.2 * (4.9 + 19) + 0.5 * 8.8 = $\boxed{33.1}$ k	
	$A_{se} = A_s + \frac{P_{um}}{f_y}\left(\frac{h}{2d}\right)$ = 6.2 + 33.1 / 60 * (7.25 / (2*3.9)) = $\boxed{6.7}$ in^2	R11.8.3
	$a = \frac{A_{se} f_y}{0.85 f'_c b}$ = 6.7 * 60 / (0.85 * 4 * 15 * 12) = $\boxed{0.66}$ in	22.2.2.4.1
	c = a / 0.85 = 0.66 / 0.85 = $\boxed{0.77}$ in	EQ. 22.2.2.4..1
	c / d = 0.77 / 3.9 = $\boxed{0.20}$	
	CHECK: IF c /d ≤ 0.375 **OK** TENSION-CONTROLLED	Tb. 21.2.2
	$I_{cr} = \frac{E_s}{E_c} A_{se}(d-c)^2 + \frac{l_w c^3}{3}$ = 8.044*6.7*(3.9-0.77)^2 + 15*12*0.77^3/3 = $\boxed{568}$ in^4	EQ. 11.8.3.1c
	$M_{cr} = \frac{f_r I_g}{y_t} = f_f S = f_r\left(\frac{1}{6} bt^2\right)$ = 0.474 * 1/6 * 15 * 7.25^2 = $\boxed{62.3}$ k-ft	EQ. 24.2.3.5b
	$\phi M_n = \phi A_{se} f_y \left(d - \frac{a}{2}\right)$ = 0.9 * 6.7 * 60 * (7.25 - 0.66 / 2) / 12 = $\boxed{208.9}$ k-ft	
	CHECK: $\phi M_n \geq M_{cr}$ **OK**	

STEP	COMPUTATIONS	REF. ACI318-19
	CHECK MINIMUM REINFORCEMENT REQUIRED BY ACI 318-19, SECTION 11.6.2:	
	$\rho = \frac{A_S}{bh}$	Tb. 11.6.1
	= 6.2 / (15 * 12 * 7.25) = $\boxed{0.00475}$	
	CHECK: $\rho \geq \rho_t$ **OK**	
	$A_{smin} = \rho_t bh$	
	=0.002 * 15 * 12 * 7.25 = $\boxed{2.61}$ in^2	
	CHECK: $A_{s,prov} \geq A_{s,min}$ **OK**	
	SPACING = b / # OF BARS	
	= 15 * 12 / 20 = $\boxed{9}$ in O.C.	11.7.3.1
	CHECK APPLIED MOMENT PER ACI 318-19, SECTION 11.8.3:	
	$K_b = \frac{48 E_c I_{cr}}{5 l_c^2}$	
	= 48 * 3605 * 568 / (5*(14*12)^2) = $\boxed{696}$ k	
	$M_{ua} = \frac{w_u l_c^2}{8} + \frac{P_{UA} e_{cc}}{2}$	
	= 0.73 * 14^2 / 8 + 39.1 * 3 /12 / 2 = $\boxed{22.9}$ k-ft	
	$M_u = \frac{M_{uA}}{\left(1 - \frac{P_{um}}{K_b}\right)}$	EQ. 11.8.3.1d
	= 22.9 / (1 - 33.1 / 696.4) = $\boxed{24.0}$ k-ft	
	CHECK: $M_u \leq \phi M_n$ **OK**	
	$\Delta_u = \frac{M_u}{0.75 K_b}$	EQ. 11.8.3.1b
	= 24 * 12 / (0.75 * 696) = $\boxed{0.552}$ in	

STEP	COMPUTATIONS	REF. ACI 318-19
	CHECK DESIGN MOMENT STRENGTH TO COMPARE TO THE MAXIMUM NEGATIVE MOMENT:	
	Pum = 1.2 * (4.9 + 1*11.9 + 30.9) + 0.5*8.8 + 1*14.6 = $\boxed{76.3}$ k	
	$A_{se} = A_s + \frac{P_{um}}{f_y}\left(\frac{h}{2d}\right)$ = 0 + 76.3 / 60 * (7.25 / (2*3.9)) = $\boxed{7.4}$ in^2	R11.8.3
	$a = \frac{A_{se} f_y}{0.85 f'_c b}$ = 7.4 * 60 / (0.85 * 4 * 15 * 12) = $\boxed{0.72}$ in	22.2.2.4.1
	c = a / 0.85 = 0.72 / 0.85 = $\boxed{0.85}$ in	EQ. 22.2.2.4..1
	c / d = 0.85 / 3.9 = $\boxed{0.22}$	
	CHECK: IF c /d ≤ 0.375 **OK** TENSION-CONTROLLED	Tb. 21.2.2
	$I_{cr} = \frac{E_s}{E_c} A_{se}(d-c)^2 + \frac{l_w c^3}{3}$ = 8.044*7.4*(3.9-0.85)^2 + 15*12*0.85^3/3 = $\boxed{602}$ in^4	EQ. 11.8.3.1c
	$M_{cr} = \frac{f_r I_g}{y_t} = f_r S = f_r\left(\frac{1}{6} b t^2\right)$ = 0.474 * 1/6 * 15 * 7.25^2 = $\boxed{62.3}$ k-ft	EQ. 24.2.3.5b
	$\phi M_n = \phi A_{se} f_y \left(d - \frac{a}{2}\right)$ = 0.9 * 7.4 * 60 * (7.25 - 0.72 / 2) / 12 = $\boxed{228.5}$ k-ft	
	CHECK: $\phi M_n \geq M_{cr}$ **OK**	

STEP	COMPUTATIONS	REF. ACI 318-19
	CHECK MINIMUM REINFORCEMENT REQUIRED BY ACI 318-19, SECTION 11.6.2:	
	$\rho = \frac{A_S}{bh}$ = 6.2 / (15 * 12 * 7.25) = $\boxed{0.00475}$	Tb. 11.6.1
	CHECK: $\rho \geq \rho_l$ **OK**	
	CHECK APPLIED MOMENT PER ACI 318-19, SECTION 11.8.3:	
	$K_b = \frac{48E_cI_{cr}}{5l_c^2}$ = 48 * 3605 * 602 / (5*(*12)^2) = $\boxed{738}$ k	
	$M_{ua} = \frac{w_u l_c^2}{8} + \frac{P_{UA}e_{cc}}{2}$ = 0.73 * 14^2 / 8 + 39.1 / 12 * 3 / 2 = $\boxed{22.9}$ k-ft	
	$M_u = \frac{M_{uA}}{\left(1 - \frac{P_{um}}{K_b}\right)}$ = 22.9 / (1 - 76.3 / 738.2) = $\boxed{25.5}$ k-ft	EQ. 11.8.3.1d
	CHECK: $M_u \leq \phi M_n$ **OK**	
	$\Delta_u = \frac{M_u}{0.75K_b}$ = PINNED AT SUPPORTS = $\boxed{0.0}$ in	EQ. 11.8.3.1b

STEP	COMPUTATIONS	REF. ACI318-19
Δ CHECKS	$\Delta_{ALLOWABLE} = l_c / 150$ $= 14 * 12 / 150 =$ $\boxed{1.120}$ in	11.8.1.1e
	$\Delta_{cr} = \frac{5 M_{cr} l_c^2}{48 E_c I_g}$ + $= 5*62.3*12*(14*12)\text{^}2 / (48*3605*5716) =$ $\boxed{0.107}$ in - $= 5*62.3*12*(14*12)\text{^}2 / (48*3605*5716) =$ $\boxed{0.107}$ in	Eq. 11.8.4.3a
	$\Delta_n = \frac{5 M_n l_c^2}{48 E_c I_{cr}}$ + $= 5*232.1*12*(14*12)\text{^}2 / (48*3605*568) =$ $\boxed{4.00}$ in - $= 5*253.9*12*(14*12)\text{^}2 / (48*3605*602) =$ $\boxed{4.13}$ in	Eq. 11.8.4.3b
	$P_{SA} = D + 0.75L + 0.75S =$ $\boxed{31.1}$ k $w_s =$ $\boxed{0.0}$ klf	
	$M_{SA} = \frac{w_s l_c^2}{8} + \frac{P_{SA} e_{cc}}{2}$ $= 0 * 14\text{^}2 / 8 + 31.1 * 3 / 12 / 2 =$ $\boxed{3.89}$ k-ft	
	$P_{SM} = P_{SA} + (h/2 + h_p) t \gamma_c$ $= 31.1 + (28 / 2 + 2) * 7.25/12 * 150 / 1000 * 28 =$ $\boxed{71.7}$ k	

STEP	COMPUTATIONS	REF. ACI318-19
	$M_A \leq \frac{2M_{cr}}{3} \quad \therefore \Delta_s = \frac{M_A \Delta_{cr}}{M_{cr}}$	R11.8.4.1
	$\Delta_s = \frac{M_A \Delta_{cr}}{M_{cr}}$ $M_A = M_{SA} + P_{SM}\Delta_s$	Tb. 11.8.4.1a Eq. 11.8.4.2
	ITERATION 1: $\Delta_s =$ 0 in; $M_A =$ 3.88856 k-ft	
	ITERATION 2: $\Delta_s =$ 0.0067 in; $M_A =$ 3.92835 k-ft	
	ITERATION 3: $\Delta_s =$ 0.0067 in; $M_A =$ 3.92875 k-ft	
	ITERATION 4: $\Delta_s =$ 0.0067 in; $M_A =$ 3.92876 k-ft	
	ITERATION 5: $\Delta_s =$ 0.00673 in; $M_A =$ 3.92876 k-ft	
	CONVERGES TO: $\Delta_s =$ 0.00673 in; $M_A =$ 3.92876 k-ft	
	CHECK: $M_A \leq 2/3M_{cr}$ **OK**	

STEP	COMPUTATIONS	REF. ACI318-19
	$M_A > \frac{2M_{cr}}{3} \quad \therefore \; \Delta_s = \frac{2\Delta_{cr}}{3} + \frac{\left(M_a - \frac{2M_{cr}}{3}\right)}{\left(M_n - \frac{2M_{cr}}{3}\right)}\left(\Delta_n - \frac{2\Delta_{cr}}{3}\right)$	R11.8.4.1
	$\Delta_s = \frac{2\Delta_{cr}}{3} + \frac{\left(M_a - \frac{2M_{cr}}{3}\right)}{\left(M_n - \frac{2M_{cr}}{3}\right)}\left(\Delta_n - \frac{2\Delta_{cr}}{3}\right)$	Tb. 11.8.4.1b
	$M_A = M_{SA} + P_{SM}\Delta_s$	Eq. 11.8.4.2
	ITERATION 1: $\Delta_s =$ N.A. in; $M_A =$ N.A. k-ft	
	ITERATION 2: $\Delta_s =$ N.A. in; $M_A =$ N.A. k-ft	
	ITERATION 3: $\Delta_s =$ N.A. in; $M_A =$ N.A. k-ft	
	ITERATION 4: $\Delta_s =$ N.A. in; $M_A =$ N.A. k-ft	
	ITERATION 5: $\Delta_s =$ N.A. in; $M_A =$ N.A. k-ft	
	CONVERGES TO: $\Delta_s =$ N.A. in; $M_A =$ N.A. k-ft	
	CHECK: $\Delta_s \le \Delta_{ALLOWABLE}$ **OK**	11.8.1.1e

STEP	COMPUTATIONS	REF. ACI318-19
MAX MOMENT @ MID-HEIGHT OF 1ST STORY: LC 6	*LC 4b AND LC 6 WILL BE EVALUATED TO DETERMINE THE CONTROLLING CASE FOR WHEN THE MAX MOMENT OCCURS AT THE MIDHEIGHT OF THE FIRST STORY.* *ASSUME REINFORCEMENT REQUIRED:* BAR SIZE = 5 # OF BARS = 20 d_b = 0.625 in A_s = 0.31 $A_{s,prov}$ = 6.2 in^2 *CHECK VERTICAL STRESS AT MIDHEIGHT SECTION OF THE 1ST STORY PANEL PER ACI 318-19, SECTION 11.8.1.1:* PANEL S.W. = $t\gamma_c b(\Sigma h - h_1 / 2)$ = 7.25 / 12 * 150 * 15 * (28 - 14 / 2) / 1000 = 28.5 k $P_{um} = P_u + 0.9$(PANEL S.W.) = Pu + 0.9 * 28.5 = 40.8 k $w_u = 0.5P_{u,HORIZ}b$ = 0.5 * Pu,HORIZ * 15 / 1000 = 0.006 klf P_{um} / A_g = 40.8 * 1000 / (7.25 * 15 * 12) = 31.2 psi $0.06f'_c$ = 0.06 * 4000 = 240.0 psi **CHECK:** $P_{um} / A_g \leq 0.06f'_c$ **OK**	Appendix B 11.8.1d

STEP	COMPUTATIONS	REF. ACI318-19
	CHECK DESIGN MOMENT STRENGTH TO COMPARE TO THE MAXIMUM POSITIVE MOMENT:	
	Pum = 0.9 * (4.9 + 19) = 21.5 k	
	$A_{se} = A_s + \frac{P_{um}}{f_y}\left(\frac{h}{2d}\right)$ = 6.2 + 21.5 / 60 * (7.25 / (2*3.9)) = 6.5 in^2	R11.8.3
	$a = \frac{A_{se} f_y}{0.85 f'_c b}$ = 6.5 * 60 / (0.85 * 4 * 15 * 12) = 0.64 in	22.2.2.4.1
	c = a / 0.85 = 0.64 / 0.85 = 0.75 in	EQ. 22.2.2.4..1
	c / d = 0.75 / 3.9 = 0.19	
	CHECK: IF c / d ≤ 0.375 **OK** TENSION-CONTROLLED	Tb. 21.2.2
	$I_{cr} = \frac{E_s}{E_c} A_{se}(d-c)^2 + \frac{l_w c^3}{3}$ = 8.044 *6.5*(3.9-0.75)^2 + 15*12*0.75^3/3 = 558 in^4	EQ. 11.8.3.1c
	$M_{cr} = \frac{f_r I_g}{y_t} = f_f S = f_r \left(\frac{1}{6} bt^2\right)$ = 0.474 * 1/6 * 15 * 7.25^2 = 62.3 k-ft	EQ. 24.2.3.5b
	$\phi M_n = \phi A_{se} f_y \left(d - \frac{a}{2}\right)$ = 0.9 * 6.5 * 60 * (7.25 - 0.64 / 2) / 12 = 203.6 k-ft	
	CHECK: $\phi M_n \geq M_{cr}$ **OK**	

STEP	COMPUTATIONS	REF. ACI318-19
	CHECK MINIMUM REINFORCEMENT REQUIRED BY ACI 318-19, SECTION 11.6.2:	
	$\rho = \frac{A_S}{bh}$	Tb. 11.6.1
	$= 6.2 / (15 * 12 * 7.25) = \boxed{0.00475}$	
	CHECK: $\rho \geq \rho_l$ **OK**	
	$A_{smin} = \rho_l bh$	11.7.3.1
	$= 0.002 * 15 * 12 * 7.25 = \boxed{2.61}\ in^2$	
	CHECK: $A_{s,prov} \geq A_{s,min}$ **OK**	
	SPACING = b / # OF BARS	
	$= 15 * 12 / 20 = \boxed{9}$ in O.C.	
	CHECK APPLIED MOMENT PER ACI 318-19, SECTION 11.8.3:	
	$K_b = \frac{48 E_c I_{cr}}{5 l_c^2}$	
	$= 48 * 3605 * 558 / (5*(14*12)^2) = \boxed{685}$ k	EQ. 11.8.3.1d
	$M_{ua} = \frac{w_u l_c^2}{8} + \frac{P_{UA} e_{cc}}{2}$	
	$= 0.73 * 14^2 / 8 + 15.1 * 3 /12 / 2 = \boxed{19.9}$ k-ft	
	$M_u = \frac{M_{uA}}{\left(1 - \frac{P_{um}}{K_b}\right)}$	EQ. 11.8.3.1b
	$= 19.9 / (1 - 21.5 / 684.6) = \boxed{20.5}$ k-ft	
	CHECK: $M_u \leq \phi M_n$ **OK**	
	$\Delta u = \frac{M_u}{0.75 K_b}$	
	$= 21 * 12 / (0.75 * 685) = \boxed{0.480}$ in	

STEP	COMPUTATIONS	REF. ACI318-19
	CHECK DESIGN MOMENT STRENGTH TO COMPARE TO THE MAXIMUM NEGATIVE MOMENT:	
	Pum = 0.9 * (4.9 + 1*11.9 + 30.9) = $\boxed{42.9}$ k	
	$A_{se} = A_s + \frac{P_{um}}{f_y}\left(\frac{h}{2d}\right)$	R11.8.3
	= 0 + 42.9 / 60 * (7.25 / (2*3.9)) = $\boxed{6.9}$ in^2	
	$a = \frac{A_{se}f_y}{0.85f'_c b}$	22.2.2.4.1
	= 6.9 * 60 / (0.85 * 4 * 15 * 12) = $\boxed{0.67}$ in	
	c = a / 0.85	EQ.
	= 0.67 / 0.85 = $\boxed{0.79}$ in	22.2.2.4..1
	c / d	
	= 0.79 / 3.9 = $\boxed{0.20}$	
	CHECK: IF c /d ≤ 0.375 **OK** TENSION-CONTROLLED	Tb. 21.2.2
	$I_{cr} = \frac{E_s}{E_c}A_{se}(d-c)^2 + \frac{l_w c^3}{3}$	EQ. 11.8.3.1c
	= 8.044*6.9*(3.9-0.79)^2 + 15*12*0.79^3/3 = $\boxed{576}$ in^4	
	$M_{cr} = \frac{f_r I_g}{y_t} = f_f S = f_r\left(\frac{1}{6}bt^2\right)$	EQ. 24.2.3.5b
	= 0.474 * 1/6 * 15 * 7.25^2 = $\boxed{62.3}$ k-ft	
	$\phi M_n = \phi A_{se} f_y\left(d - \frac{a}{2}\right)$	
	= 0.9 * 6.9 * 60 * (7.25 - 0.67 / 2) / 12 = $\boxed{213.4}$ k-ft	
	CHECK: $\phi M_n \geq M_{cr}$ **OK**	

STEP	COMPUTATIONS	REF. ACI318-19
	CHECK MINIMUM REINFORCEMENT REQUIRED BY ACI 318-19, SECTION 11.6.2:	
	$\rho = \frac{A_S}{bh}$	Tb. 11.6.1
	= 6.2 / (15 * 12 * 7.25) = $\boxed{0.00475}$	
	CHECK: $\rho \geq \rho_l$ **OK**	
	CHECK APPLIED MOMENT PER ACI 318-19, SECTION 11.8.3:	
	$K_b = \frac{48 E_c I_{cr}}{5 l_c^2}$	
	= 48 * 3605 * 576 / (5*(14*12)^2) = $\boxed{706}$ k	
	$M_{ua} = \frac{w_u l_c^2}{8} + \frac{P_{UA} e_{cc}}{2}$	
	= 0.73 * 14^2 / 8 + 15.1 * 3 / 12 / 2 = $\boxed{19.9}$ k-ft	
	$M_u = \frac{M_{uA}}{\left(1 - \frac{P_{um}}{K_b}\right)}$	EQ. 11.8.3.1d
	= 19.9 / (1 - 42.9 / 706.2) = $\boxed{21.2}$ k-ft	
	CHECK: $M_u \leq \phi M_n$ **OK**	
	$\Delta_u = \frac{M_u}{0.75 K_b}$	EQ. 11.8.3.1b
	= PINNED AT SUPPORTS = $\boxed{0.0}$ in	

STEP	COMPUTATIONS	REF. ACI 318-19
Δ CHECKS	$\Delta_{ALLOWABLE} = l_c / 150$	11.8.1.1e
	= 14 * 12 / 150 = 1.120 in	
	$\Delta_{cr} = \frac{5M_{cr}l_c^2}{48E_cI_{cr}}$	Eq. 11.8.4.3a
	+ = 5*62.3*12*(14*12)^2 / (48*3605*5716) = 0.107 in	
	- = 5*62.3*12*(14*12)^2 / (48*3605*5716) = 0.107 in	
	$\Delta_n = \frac{5M_nl_c^2}{48E_cI_{cr}}$	Eq. 11.8.4.3b
	+ = 5*226.3*12*(14*12)^2 / (48*3605*558) = 3.97 in	
	- = 5*237.1*12*(14*12)^2 / (48*3605*576) = 4.03 in	
	$P_{SA} = D + 0.6W =$ 16.8 k	
	$w_s =$ 0.4 klf	
	$M_{SA} = \frac{w_sl_c^2}{8} + \frac{P_{SA}e_{cc}}{2}$	
	= 0.44 * 14^2 / 8 + 16.8 * 3 / 12 / 2 = 12.89 k-ft	
	$P_{SM} = P_{SA} + (h/2 + h_p)t\gamma_c$	
	= 16.8 + (28 / 2 + 2) * 7.25/12 * 150 / 1000 * 28 = 57.4 k	

STEP	COMPUTATIONS	REF. ACI318-19
	$M_A \leq \frac{2M_{cr}}{3} \quad \therefore \Delta_s = \frac{M_A \Delta_{cr}}{M_{cr}}$	R11.8.4.1
	$\Delta_s = \frac{M_A \Delta_{cr}}{M_{cr}}$ $M_A = M_{SA} + P_{SM}\Delta_s$	Tb. 11.8.4.1a Eq. 11.8.4.2
	ITERATION 1: $\Delta_s =$ 0 in; $M_A =$ 12.8869 k-ft	
	ITERATION 2: $\Delta_s =$ 0.0221 in; $M_A =$ 12.9923 k-ft	
	ITERATION 3: $\Delta_s =$ 0.0222 in; $M_A =$ 12.9932 k-ft	
	ITERATION 4: $\Delta_s =$ 0.0222 in; $M_A =$ 12.9932 k-ft	
	ITERATION 5: $\Delta_s =$ 0.02225 in; $M_A =$ 12.9932 k-ft	
	CONVERGES TO: $\Delta_s =$ 0.02225 in; $M_A =$ 12.9932 k-ft	
	CHECK: $M_A \leq 2/3M_{cr}$ **OK**	

STEP	COMPUTATIONS	REF. ACI 318-19
	$M_A > \frac{2M_{cr}}{3} \quad \therefore \ \Delta_s = \frac{2\Delta_{cr}}{3} + \frac{\left(M_a - \frac{2M_{cr}}{3}\right)}{\left(M_n - \frac{2M_{cr}}{3}\right)}\left(\Delta_n - \frac{2\Delta_{cr}}{3}\right)$	R11.8.4.1
	$\Delta_s = \frac{2\Delta_{cr}}{3} + \frac{\left(M_a - \frac{2M_{cr}}{3}\right)}{\left(M_n - \frac{2M_{cr}}{3}\right)}\left(\Delta_n - \frac{2\Delta_{cr}}{3}\right)$	Tb. 11.8.4.1b
	$M_A = M_{SA} + P_{SM}\Delta_s$	Eq. 11.8.4.2
	ITERATION 1: Δ_s = N.A. in; M_A = N.A. k-ft	
	ITERATION 2: Δ_s = N.A. in; M_A = N.A. k-ft	
	ITERATION 3: Δ_s = N.A. in; M_A = N.A. k-ft	
	ITERATION 4: Δ_s = N.A. in; M_A = N.A. k-ft	
	ITERATION 5: Δ_s = N.A. in; M_A = N.A. k-ft	
	CONVERGES TO: Δ_s = N.A. in; M_A = N.A. k-ft	
	CHECK: $\Delta_s \leq \Delta_{ALLOWABLE}$ **OK**	11.8.1.1e

STEP	COMPUTATIONS	REF. ACI 318-19
MAX MOMENT @ MID-HEIGHT OF LARGEST STORY: LC 4a	*LC 4a AND LC 6 WILL BE EVALUATED TO DETERMINE THE CONTROLLING CASE FOR WHEN THE MAX MOMENT OCCURS AT THE MIDHEIGHT OF THE LARGEST STORY.* *ASSUME REINFORCEMENT REQUIRED:* BAR SIZE = 5 # OF BARS = 20 d_b = 0.625 in A_s = 0.31 $A_{s,prov}$ = 6.2 in^2	Appendix B
	CHECK VERTICAL STRESS AT MIDHEIGHT SECTION OF THE 1ST STORY PANEL PER ACI 318-19, SECTION 11.8.1.1: PANEL S.W. = $t\gamma_c b\,(\Sigma h - h_1 / 2)$ = 7.25 / 12 * 150 * 15 * (28 - 14 / 2) / 1000 = 28.5 k $P_{um} = P_u + 1.2$(PANEL S.W.) = Pu + 1.2 * 28.5 = 71.2 k $w_u = 0.5P_{u,HORIZ}b$ = 0.5 * Pu,HORIZ * 15 / 1000 = 0.006 klf P_{um} / A_g = 71.2 * 1000 / (7.25 * 15 * 12) = 54.6 psi $0.06f'_c$ = 0.06 * 4000 = 240.0 psi	11.8.1d
	CHECK: $P_{um} / A_g \leq 0.06f'_c$ **OK**	

STEP	COMPUTATIONS	REF. ACI318-19
	CHECK DESIGN MOMENT STRENGTH TO COMPARE TO THE MAXIMUM POSITIVE MOMENT:	
	Pum = 1.2 * (4.9 + 19) + 1.6 * 27.3 = $\boxed{35.9}$ k	
	$A_{se} = A_s + \frac{Pum}{f_y}\left(\frac{h}{2d}\right)$ = 6.2 + 35.9 / 60 * (7.25 / (2*3.9)) = $\boxed{6.8}$ in^2	R11.8.3
	$a = \frac{A_{se}f_y}{0.85f'_cb}$ = 6.8 * 60 / (0.85 * 4 * 15 * 12) = $\boxed{0.66}$ in	22.2.2.4.1
	c = a / 0.85 = 0.66 / 0.85 = $\boxed{0.78}$ in	EQ. 22.2.2.4..1
	c / d = 0.78 / 3.9 = $\boxed{0.20}$	
	CHECK: IF c /d ≤ 0.375 **OK** TENSION-CONTROLLED	Tb. 21.2.2
	$I_{cr} = \frac{E_s}{E_c}A_{se}(d-c)^2 + \frac{l_w c^3}{3}$ = 8.044 *6.8*(3.9-0.78)^2 + 15* 12*0.78^3/3 = $\boxed{570}$ in^4	EQ. 11.8.3.1c
	$M_{cr} = \frac{f_r I_g}{y_t} = f_r S = f_r\left(\frac{1}{6}bt^2\right)$ = 0.474 * 1/6 * 15 * 7.25^2 = $\boxed{62.3}$ k-ft	EQ. 24.2.3.5b
	$\phi M_n = \phi A_{se} f_y \left(d - \frac{a}{2}\right)$ = 0.9 * 6.8 * 60 * (7.25 - 0.66 / 2) / 12 = $\boxed{210.2}$ k-ft	
	CHECK: $\phi M_n \geq M_{cr}$ **OK**	

STEP	COMPUTATIONS	REF. ACI 318-19
	CHECK MINIMUM REINFORCEMENT REQUIRED BY ACI 318-19, SECTION 11.6.1:	
	$\rho = \frac{A_s}{bh}$	Tb. 11.6.1
	$= 6.2 / (15 * 12 * 7.25) = \boxed{0.00475}$	
	CHECK: $\rho \geq \rho_t$ **OK**	
	$A_{smin} = \rho_t bh$	
	$= 0.002 * 15 * 12 * 7.25 = \boxed{2.61}$ in^2	
	CHECK: $A_{s,prov} \geq A_{s,min}$ **OK**	
	SPACING = b / # OF BARS	
	$= 15 * 12 / 20 = \boxed{9}$ in O.C.	11.7.3.1
	CHECK APPLIED MOMENT PER ACI 318-19, SECTION 11.8.3:	
	$K_b = \frac{48E_cI_{cr}}{5l_c^2}$	
	$= 48 * 3605 * 570 / (5*(14*12)^2) = \boxed{699}$ k	
	$M_{ua} = \frac{w_u l_c^2}{8} + \frac{P_{UA} e_{cc}}{2}$	
	$= 0.73 * 14^2 / 8 + 37 / 12 * 3 / 2 = \boxed{22.6}$ k-ft	
	$M_u = \frac{M_{uA}}{\left(1 - \frac{P_{um}}{K_b}\right)}$	EQ. 11.8.3.1d
	$= 22.6 / (1 - 35.9 / 699.2) = \boxed{23.8}$ k-ft	
	CHECK: $M_u \leq \phi M_n$ **OK**	
	$\Delta_u = \frac{M_u}{0.75K_b}$	EQ. 11.8.3.1b
	$= 24 * 12 / (0.75 * 699) = \boxed{0.545}$ in	

STEP	COMPUTATIONS	REF. ACI318-19
	CHECK DESIGN MOMENT STRENGTH TO COMPARE TO THE MAXIMUM NEGATIVE MOMENT:	
	Pum = 1.2 * (4.9 + 11.9 + 7.1) + 1.6 * 4.5 = $\boxed{35.9}$ k	
	$A_{se} = A_s + \frac{P_{um}}{f_y}\left(\frac{h}{2d}\right)$ = 0 + 35.9 / 60 * (7.25 / (2*3.9)) = $\boxed{6.8}$ in^2	R11.8.3
	$a = \frac{A_{se} f_y}{0.85 f'_c b}$ = 6.8 * 60 / (0.85 * 4 * 15 * 12) = $\boxed{0.66}$ in	22.2.2.4.1
	c = a / 0.85 = 0.66 / 0.85 = $\boxed{0.78}$ in	EQ. 22.2.2.4..1
	c / d = 0.78 / 3.9 = $\boxed{0.20}$	
	CHECK: IF c /d ≤ 0.375 **OK** TENSION-CONTROLLED	Tb. 21.2.2
	$I_{cr} = \frac{E_s}{E_c} A_{se}(d-c)^2 + \frac{l_w c^3}{3}$ = 8.044 *6.8*(3.9-0.78)^2 + 15*12*0.78^3/3 = $\boxed{570}$ in^4	EQ. 11.8.3.1c
	$M_{cr} = \frac{f_r I_g}{y_t} = f_f S = f_r\left(\frac{1}{6} bt^2\right)$ = 0.474 * 1/6 * 15 * 7.25^2 = $\boxed{62.3}$ k-ft	EQ. 24.2.3.5b
	$\phi M_n = \phi A_{se} f_y \left(d - \frac{a}{2}\right)$ = 0.9 * 6.8 * 60 * (7.25 - 0.66 / 2) / 12 = $\boxed{210.2}$ k-ft	
	CHECK: $\phi M_n \geq M_{cr}$ **OK**	

STEP	COMPUTATIONS	REF. ACI318-19
	CHECK MINIMUM REINFORCEMENT REQUIRED BY ACI 318-19, SECTION 11.6.2:	
	$\rho = \frac{A_S}{bh}$	Tb. 11.6.1
	$= 6.2 / (15 * 12 * 7.25) = \boxed{0.00475}$	
	CHECK: $\rho \geq \rho_l$ **OK**	
	CHECK APPLIED MOMENT PER ACI 318-19, SECTION 11.8.3:	
	$K_b = \frac{48 E_c I_{cr}}{5 l_c^2}$	
	$= 48 * 3605 * 570 / (5*(14*12)^2) = \boxed{699}$ k	
	$M_{ua} = \frac{w_u l_c^2}{8} + \frac{P_{UA} e_{cc}}{2}$	
	$= 0.73 * 14^2 / 8 + 37 / 12 * 3 / 2 = \boxed{22.6}$ k-ft	
	$M_u = \frac{M_{uA}}{\left(1 - \frac{P_{um}}{K_b}\right)}$	EQ. 11.8.3.1d
	$= 22.6 / (1 - 35.9 / 699.2)) = \boxed{23.8}$ k-ft	
	CHECK: $M_u \leq \phi M_n$ **OK**	
	$\Delta_u = \frac{M_u}{0.75 K_b}$	EQ. 11.8.3.1b
	= PINNED AT SUPPORTS = $\boxed{0.0}$ in	

STEP	COMPUTATIONS	REF. ACI318-19
Δ CHECKS	$\Delta_{ALLOWABLE} = l_c / 150$ $= 14 * 12 / 150 = \boxed{1.120}$ in	11.8.1.1e
	$\Delta_{cr} = \dfrac{5M_{cr}l_c^2}{48E_cI_{cr}}$ + $= 5*62.3*12*(14*12)^2 / (48*3605*5716) = \boxed{0.107}$ in - $= 5*62.3*12*(14*12)^2 / (48*3605*5716) = \boxed{0.107}$ in	Eq. 11.8.4.3a
	$\Delta_n = \dfrac{5M_nl_c^2}{48E_cI_{cr}}$ + $= 5*233.5*12*(14*12)^2 / (48*3605*570) = \boxed{4.01}$ in - $= 5*233.5*12*(14*12)^2 / (48*3605*570) = \boxed{4.01}$ in	Eq. 11.8.4.3b
	$P_{SA} = D + 0.75L + 0.75L_r = \boxed{31.1}$ k $w_s = \boxed{0.0}$ klf	
	$M_{SA} = \dfrac{w_sl_c^2}{8} + \dfrac{P_{SA}e_{cc}}{2}$ $= 0 * 14^2 / 8 + 31.1 * 3 / 12 / 2 = \boxed{3.89}$ k-ft	
	$P_{SM} = P_{SA} + (h/2 + h_p)t\gamma_c$ $= 31.1 + (28 / 2 + 2) * 7.25/12 * 150 / 1000 * 28 = \boxed{71.7}$ k	

STEP	COMPUTATIONS	REF. ACI 318-19
	$M_A \le \frac{2M_{cr}}{3} \quad \therefore \Delta_s = \frac{M_A \Delta_{cr}}{M_{cr}}$	R11.8.4.1
	$\Delta_s = \frac{M_A \Delta_{cr}}{M_{cr}}$ $M_A = M_{sa} + P_{sm}\Delta_s$	Tb. 11.8.4.1a Eq. 11.8.4.2
	ITERATION 1: $\Delta_s = $ 0 in; $M_A = $ 3.88856 k-ft	
	ITERATION 2: $\Delta_s = $ 0.0067 in; $M_A = $ 3.92835 k-ft	
	ITERATION 3: $\Delta_s = $ 0.0067 in; $M_A = $ 3.92875 k-ft	
	ITERATION 4: $\Delta_s = $ 0.0067 in; $M_A = $ 3.92876 k-ft	
	ITERATION 5: $\Delta_s = $ 0.0067 in; $M_A = $ 3.92876 k-ft	
	CONVERGES TO: $\Delta_s = $ 0.0067 in; $M_A = $ 3.92876 k-ft	
	CHECK: $M_A \le 2/3M_{cr}$ **OK**	

STEP	COMPUTATIONS	REF. ACI318-19
	$M_A > \frac{2M_{cr}}{3} \quad \therefore \Delta_s = \frac{2\Delta_{cr}}{3} + \frac{\left(M_a - \frac{2M_{cr}}{3}\right)}{\left(M_n - \frac{2M_{cr}}{3}\right)}\left(\Delta_n - \frac{2\Delta_{cr}}{3}\right)$	R11.8.4.1
	$\Delta_s = \frac{2\Delta_{cr}}{3} + \frac{\left(M_a - \frac{2M_{cr}}{3}\right)}{\left(M_n - \frac{2M_{cr}}{3}\right)}\left(\Delta_n - \frac{2\Delta_{cr}}{3}\right)$	Tb. 11.8.4.1b
	$M_A = M_{SA} + P_{SM}\Delta_s$	Eq. 11.8.4.2
	ITERATION 1: Δ_s = N.A. in; M_A = N.A. k-ft	
	ITERATION 2: Δ_s = N.A. in; M_A = N.A. k-ft	
	ITERATION 3: Δ_s = N.A. in; M_A = N.A. k-ft	
	ITERATION 4: Δ_s = N.A. in; M_A = N.A. k-ft	
	ITERATION 5: Δ_s = N.A. in; M_A = N.A. k-ft	
	CONVERGES TO: Δ_s = N.A. in; M_A = N.A. k-ft	
	CHECK: $\Delta_s \le \Delta_{ALLOWABLE}$ **OK**	11.8.1.1e

<table>
<tr><th>STEP</th><th>COMPUTATIONS</th><th>REF.
ACI 318-19</th></tr>
<tr><td>MAX MOMENT @ MID-HEIGHT OF LARGEST STORY:
LC 6</td><td>

LC 4a AND LC 6 WILL BE EVALUATED TO DETERMINE THE CONTROLLING CASE FOR WHEN THE MAX MOMENT OCCURS AT THE MIDHEIGHT OF THE LARGEST STORY.

ASSUME REINFORCEMENT REQUIRED:

BAR SIZE = 5
OF BARS = 20

d_b = 0.625 in
A_s = 0.31
$A_{s,prov}$ = 6.2 in^2

CHECK VERTICAL STRESS AT MIDHEIGHT SECTION OF THE 1ST STORY PANEL PER ACI 318-19, SECTION 11.8.1.1:

PANEL S.W. = $t\gamma_c b\,(\Sigma h - h_1 / 2)$
= 7.25 / 12 * 150 * 15 * (28 - 14 / 2) / 1000 = 28.5 k

$P_{um} = P_u + 1.2$(PANEL S.W.)
= Pu + 1.2 * 28.5 = 49.3 k

$w_u = 0.5P_{u,HORIZ}b$
= 0.5 * Pu,HORIZ * 15 / 1000 = 0.006 klf

P_{um} / A_g
= 49.3 * 1000 / (7.25 * 15 * 12) = 37.8 psi

$0.06f'_c$
= 0.06 * 4000 = 240.0 psi

CHECK: $P_{um} / A_g \leq 0.06f'_c$ **OK**

</td><td>Appendix B

11.8.1d</td></tr>
</table>

STEP	COMPUTATIONS	REF. ACI318-19
	CHECK DESIGN MOMENT STRENGTH TO COMPARE TO THE MAXIMUM POSITIVE MOMENT:	
	Pum = 1.2 * (4.9 + 19) + 1.6 * 8.8 = 42.8 k	
	$A_{se} = A_s + \frac{P_{um}}{f_y}\left(\frac{h}{2d}\right)$ = 6.2 + 42.8 / 60 * (7.25 / (2*3.9)) = 6.9 in^2	R11.8.3
	$a = \frac{A_{se} f_y}{0.85 f'_c b}$ = 6.9 * 60 / (0.85 * 4 * 15 * 12) = 0.67 in	22.2.2.4.1
	c = a / 0.85 = 0.67 / 0.85 = 0.79 in	EQ. 22.2.2.4..1
	c / d = 0.79 / 3.9 = 0.20	
	CHECK: IF c /d ≤ 0.375 **OK** TENSION-CONTROLLED	Tb. 21.2.2
	$I_{cr} = \frac{E_s}{E_c} A_{se}(d-c)^2 + \frac{l_w c^3}{3}$ = 8.044*6.9*(3.9-0.79)^2 + 15*12*0.79^3/3 = 576 in^4	EQ. 11.8.3.1c
	$M_{cr} = \frac{f_r I_g}{y_t} = f_r S = f_r\left(\frac{1}{6}bt^2\right)$ = 0.474 * 1/6 * 15 * 7.25^2 = 62.3 k-ft	EQ. 24.2.3.5b
	$\phi M_n = \phi A_{se} f_y \left(d - \frac{a}{2}\right)$ = 0.9 * 6.9 * 60 * (7.25 - 0.67 / 2) / 12 = 213.3 k-ft	
	CHECK: $\phi M_n \geq M_{cr}$ **OK**	

STEP	COMPUTATIONS	REF. ACI 318-19
	CHECK MINIMUM REINFORCEMENT REQUIRED BY ACI 318-19, SECTION 11.6.2:	
	$\rho = \frac{A_S}{bh}$ $= 6.2 / (15 * 12 * 7.25) = \boxed{0.00475}$	Tb. 11.6.1
	CHECK: $\rho \geq \rho_t$ **OK**	
	$A_{smin} = \rho_t bh$ $= 0.002 * 15 * 12 * 7.25 = \boxed{2.61}$ in^2	
	CHECK: $A_{s,prov} \geq A_{s,min}$ **OK**	
	SPACING = b / # OF BARS = 15 * 12 / 20 = $\boxed{9}$ in O.C.	11.7.3.1
	CHECK APPLIED MOMENT PER ACI 318-19, SECTION 11.8.3:	
	$K_b = \frac{48E_cI_{cr}}{5l_c^2}$ $= 48 * 0 * 576 / (5*(3*12)\text{^}2) = \boxed{15376}$ k	
	$M_{ua} = \frac{w_u l_c^2}{8} + \frac{P_{UA} e_{cc}}{2}$ $= 0.73 * 14\text{^}2 / 8 + 15.1 * 3 / 12 / 2 = \boxed{19.9}$ k-ft	
	$M_u = \frac{M_{uA}}{\left(1 - \frac{P_{um}}{0.75K_b}\right)}$ $= 19.9 / (1 - 42.8 / (0.75 * 15376.3)) = \boxed{19.9}$ k-ft	EQ. 11.8.3.1d
	CHECK: $M_u \leq \phi M_n$ **OK**	
	$\Delta_u = \frac{M_u}{0.75K_b}$ $= 20 * 12 / (0.75 * 15376) = \boxed{0.021}$ in	EQ. 11.8.3.1b

STEP	COMPUTATIONS	REF. ACI318-19
	CHECK DESIGN MOMENT STRENGTH TO COMPARE TO THE MAXIMUM NEGATIVE MOMENT:	
	Pum = 1.2 * (4.9 + 11.9 + 7.1) + 1.6 * 8.8 = $\boxed{42.8}$ k	
	$A_{se} = A_s + \frac{P_{um}}{f_y}\left(\frac{h}{2d}\right)$ = 0 + 42.8 / 60 * (7.25 / (2*3.9)) = $\boxed{6.9}$ in^2	R11.8.3
	$a = \frac{A_{se} f_y}{0.85 f'_c b}$ = 6.9 * 60 / (0.85 * 4 * 15 * 12) = $\boxed{0.67}$ in	22.2.2.4.1
	c = a / 0.85 = 0.67 / 0.85 = $\boxed{0.79}$ in	EQ. 22.2.2.4..1
	c / d = 0.79 / 3.9 = $\boxed{0.20}$	
	CHECK: IF c /d ≤ 0.375 **OK** TENSION-CONTROLLED	Tb. 21.2.2
	$I_{cr} = \frac{E_s}{E_c} A_{se}(d-c)^2 + \frac{l_w c^3}{3}$ = 8.044*6.9*(3.9-0.79)^2 + 15*12*0.79^3/3 = $\boxed{576}$ in^4	EQ. 11.8.3.1c
	$M_{cr} = \frac{f_r I_g}{y_t} = f_r S = f_r\left(\frac{1}{6} b t^2\right)$ = 0.474 * 1/6 * 15 * 7.25^2 = $\boxed{62.3}$ k-ft	EQ. 24.2.3.5b
	$\phi M_n = \phi A_{se} f_y \left(d - \frac{a}{2}\right)$ = 0.9 * 6.9 * 60 * (7.25 - 0.67 / 2) / 12 = $\boxed{213.3}$ k-ft	
	CHECK: $\phi M_n \geq M_{cr}$ **OK**	

STEP	COMPUTATIONS	REF. ACI 318-19
	CHECK MINIMUM REINFORCEMENT REQUIRED BY ACI 318-19, SECTION 11.6.2:	
	$\rho = \frac{A_s}{bh}$	Tb. 11.6.1
	$= 6.2 / (15 * 12 * 7.25) =$ $\boxed{0.00475}$	
	CHECK: $\rho \geq \rho_l$ **OK**	
	CHECK APPLIED MOMENT PER ACI 318-19, SECTION 11.8.3:	
	$K_b = \frac{48 E_c I_{cr}}{5 l_c^2}$	
	$= 48 * 3605 * 576 / (5*(3*12)\text{^}2) =$ $\boxed{706}$ k	
	$M_{ua} = \frac{w_u l_c^2}{8} + \frac{P_{UA} e_{cc}}{2}$	
	$= 0.73 * 14\text{^}2 / 8 + 15.1 * 3 / 12 / 2 =$ $\boxed{19.9}$ k-ft	
	$M_u = \frac{M_{uA}}{\left(1 - \frac{P_{um}}{K_b}\right)}$	EQ. 11.8.3.1d
	$= 19.9 / (1 - 42.8 / 706) =$ $\boxed{21.2}$ k-ft	
	CHECK: $M_u \leq \phi M_n$ **OK**	
	$\Delta_u = \frac{M_u}{0.75 K_b}$	EQ. 11.8.3.1b
	= PINNED AT SUPPORTS = $\boxed{0.0}$ in	

STEP	COMPUTATIONS	REF. ACI318-19
Δ CHECKS	$\Delta_{ALLOWABLE} = l_c / 150$	11.8.1.1e
	$= 14 * 12 / 150 = \boxed{1.120}$ in	
	$\Delta_{cr} = \frac{5M_{cr}l_c^2}{48E_cI_{cr}}$	Eq. 11.8.4.3a
+	$= 5*62.3*12*(14*12)^2 / (48*3605*576) = \boxed{1.06}$ in	
-	$= 5*62.3*12*(14*12)^2 / (48*3605*576) = \boxed{1.06}$ in	
	$\Delta_n = \frac{5M_nl_c^2}{48E_cI_{cr}}$	Eq. 11.8.4.3b
+	$= 5*237*12*(14*12)^2 / (48*3605*576) = \boxed{4.03}$ in	
-	$= 5*237*12*(14*12)^2 / (48*3605*576) = \boxed{4.03}$ in	
	$P_{SA} = D + 0.6W = \boxed{16.8}$ k	
	$w_s = \boxed{0.4}$ klf	
	$M_{SA} = \frac{w_sl_c^2}{8} + \frac{P_{SA}e_{cc}}{2}$	
	$= 0.44 * 14^2 / 8 + 16.8 * 3 / 12 / 2 = \boxed{12.89}$ k-ft	
	$P_{SM} = P_{SA} + (h/2 + h_p)t\gamma_c$	
	$= 16.8 + (28 / 2 + 2) * 7.25/12 * 150 / 1000 * 28 = \boxed{57.4}$ k	

STEP	COMPUTATIONS	REF. ACI 318-19
	$M_A \leq \frac{2M_{cr}}{3} \quad \therefore \Delta_s = \frac{M_A \Delta_{cr}}{M_{cr}}$	R11.8.4.1
	$\Delta_s = \frac{M_A \Delta_{cr}}{M_{cr}}$ $M_A = M_{SA} + P_{SM}\Delta_s$	Tb. 11.8.4.1a Eq. 11.8.4.2
	ITERATION 1: Δ_s = 0 in; M_A = 12.8869 k-ft	
	ITERATION 2: Δ_s = 0.2190 in; M_A = 13.9339 k-ft	
	ITERATION 3: Δ_s = 0.2368 in; M_A = 14.019 k-ft	
	ITERATION 4: Δ_s = 0.2383 in; M_A = 14.0259 k-ft	
	ITERATION 5: Δ_s = 0.23838 in; M_A = 14.0264 k-ft	
	CONVERGES TO: Δ_s = 0.23839 in; M_A = 14.0265 k-ft	
	CHECK: $M_A \leq 2/3M_{cr}$ **OK**	

STEP	COMPUTATIONS	REF. ACI318-19
	$M_A > \frac{2M_{cr}}{3} \quad \therefore \; \Delta_S = \frac{2\Delta_{cr}}{3} + \frac{\left(M_a - \frac{2M_{cr}}{3}\right)}{\left(M_n - \frac{2M_{cr}}{3}\right)}\left(\Delta_n - \frac{2\Delta_{cr}}{3}\right)$	R11.8.4.1
	$\Delta_S = \frac{2\Delta_{cr}}{3} + \frac{\left(M_a - \frac{2M_{cr}}{3}\right)}{\left(M_n - \frac{2M_{cr}}{3}\right)}\left(\Delta_n - \frac{2\Delta_{cr}}{3}\right)$	Tb. 11.8.4.1b
	$M_A = M_{SA} + P_{SM}\Delta_s$	Eq. 11.8.4.2
	ITERATION 1: Δ_s = N.A. in; M_A = N.A. k-ft	
	ITERATION 2: Δ_s = N.A. in; M_A = N.A. k-ft	
	ITERATION 3: Δ_s = N.A. in; M_A = N.A. k-ft	
	ITERATION 4: Δ_s = N.A. in; M_A = N.A. k-ft	
	ITERATION 5: Δ_s = N.A. in; M_A = N.A. k-ft	
	CONVERGES TO: Δ_s = N.A. in; M_A = N.A. k-ft	
	CHECK: $\Delta_s \le \Delta_{ALLOWABLE}$ **OK**	11.8.1.1e

STEP	COMPUTATIONS	REF.
AXIAL LOAD	$P_{u: 4b}$ - MAX MOMENT AT MIDHEIGHT OF 1ST STORY = 39.1 k $P_{u: 6}$ - MAX MOMENT AT MIDHEIGHT OF 1ST STORY = 15.1 k $P_{u: 4a}$ - MAX MOMENT AT LARGEST STORY = 37.0 k $P_{u: 6}$ - MAX MOMENT AT LARGEST STORY = 15.1 k	
LATERAL LOAD	$w_{u: 4b}$ - MAX MOMENT AT MIDHEIGHT OF 1ST STORY = 0.73 klf $w_{u: 6}$ - MAX MOMENT AT MIDHEIGHT OF 1ST STORY = 0.73 klf $w_{u: 4a}$ - MAX MOMENT AT LARGEST STORY = 0.73 klf $w_{u: 6}$ - MAX MOMENT AT LARGEST STORY = 0.73 klf	
GOVERNING CASE	$M_{u: 4b}$ - MAX MOMENT AT MIDHEIGHT OF 1ST STORY = 24.0 k-ft $M_{u: 6}$ - MAX MOMENT AT MIDHEIGHT OF 1ST STORY = 20.5 k-ft $M_{u: 4a}$ - MAX MOMENT AT LARGEST STORY = 23.8 k-ft $M_{u: 6}$ - MAX MOMENT AT LARGEST STORY = 19.9 k-ft	
Δ_u COMPARISON	$\Delta_{u: 4b}$ - MAX MOMENT AT MIDHEIGHT OF 1ST STORY = 0.552 in $\Delta_{u: 6}$ - MAX MOMENT AT MIDHEIGHT OF 1ST STORY = 0.480 in $\Delta_{u: 4a}$ - MAX MOMENT AT LARGEST STORY = 0.545 in $\Delta_{u: 6}$ - MAX MOMENT AT LARGEST STORY = 0.021 in	
M_s COMPARISON	$M_{s: 4b}$ - MAX MOMENT AT MIDHEIGHT OF 1ST STORY = 3.9 k-ft $M_{s: 6}$ - MAX MOMENT AT MIDHEIGHT OF 1ST STORY = 13.0 k-ft $M_{s: 4a}$ - MAX MOMENT AT LARGEST STORY = 3.9 k-ft $M_{s: 6}$ - MAX MOMENT AT LARGEST STORY = 14.0 k-ft	
Δ_s COMPARISON	$\Delta_{s: 4b}$ - MAX MOMENT AT MIDHEIGHT OF 1ST STORY = 0.007 in $\Delta_{s: 6}$ - MAX MOMENT AT MIDHEIGHT OF 1ST STORY = 0.022 in $\Delta_{s: 4a}$ - MAX MOMENT AT LARGEST STORY = 0.007 in $\Delta_{s: 6}$ - MAX MOMENT AT LARGEST STORY = 0.238 in	
FINAL DESIGN	APPLIED AXIAL LOAD = 37.0 k APPLIED LATERAL LOAD = 0.73 klf M_u = 24.0 k-ft Δ_u = 0.545 in M_s = 14.0 k-ft Δ_s = 0.238 in BAR SIZE = 5 # OF BARS = 20	