Comparative analysis of single-wythe, non-composite double-wythe, and composite double-wythe tilt-up panels

by

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Abstract

Insulated precast concrete sandwich panels are commonly used for exterior cladding on a building. In recent years, insulated tilt-up concrete sandwich panels are being used for the exterior load-bearing walls on a building. The insulation is sandwiched between exterior and interior concrete layers to reduce the heating and cooling costs for the structure. The panels can be designed as composite, partially composite, or non-composite. The shear ties are used to achieve these varying degrees of composite action between the concrete layers. A parametric study analyzing the standard, solid single-wythe tilt-up concrete wall panel and solid sandwich (double-wythe separated by rigid insulation) tilt-up concrete wall panels subjected to eccentric axial loads and out-of-plane seismic loads is presented. The sandwich tilt-up panel is divided into two categories – non-composite and composite wall panels. The height and width of the different types of tilt-up wall panel is 23 feet (21 feet plus 2-foot parapet) and 16 feet, respectively. The solid standard panel (non-sandwich) is 5.5 inches in thickness; the non-composite sandwich panel is composed of 3.5-inch architectural wythe, 2.5-inch rigid insulation, and 5.5-inch interior load bearing concrete wythe; and the composite sandwich panel is composed of 3.5-inch exterior, load bearing concrete wythe, 2.5-inch insulation, and 5.5-inch interior, load bearing concrete wythe. The procedure used to design the tilt-up wall panels is the Alternative Method for Out-of-Plane Slender Wall Analysis per Section 11.8 of ACI 318-14 *Building Code Requirements for Structural Concrete and Commentary*.

The results indicated that for the given panels, the applied ultimate moment and design moment strength is the greatest for the composite sandwich tilt-up concrete panel. The standard tilt-up concrete panel exhibits the greatest service load deflection. The non-composite sandwich tilt-up concrete panel induced the greatest vertical stress.

Additionally, the additional requirements regarding forming materials, casting, and crane capacity is covered in this report. Lastly, the energy efficiency due to the heat loss and heat gain of sandwich panels is briefly discussed in this report. The sandwich tilt-up panels exhibit greater energy efficiency than standard tilt-up panels with or without insulation.

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Notations and Terminology

A_g = gross area of concrete section, in.^{2} For a hollow section, the gross area of the concrete does not include the area of the void(s)

A_s = area of longitudinal tension reinforcement

$A_{s,min}$ = minimum area of flexural reinforcement, in.^{2}

A_{se} = effective area of longitudinal tension reinforcement

A'_s = area of longitudinal compression reinforcement

a = depth of equivalent rectangular stress block

b = width of the compression face of member

b_w = web width, in.

C = compression force

c = distance from extreme compression fiber to neutral axis, in.

cfm = cubic feet per minute

D = dead load

D_{wall} = weight of the structural wall

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

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

E = earthquake load

E_c = modulus of elasticity of concrete

E_s = modulus of elasticity of reinforcement and structural steel

e_{cc} = eccentricity of applied loads, in.

f_c' = specified compressive strength of concrete, psi

f_r = modulus of rupture of concrete, psi

f_y = specified yield strength of reinforcement

F_p = seismic out-of-plane design force

h = overall thickness, height, or depth of member, in.

h_w = height of entire wall from base to top

I_{cr} = moment of inertia of cracked section transformed to concrete, in.^{4}

I_e = importance factor, assume 1.0 for this report, Occupancy Category II

I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, in.^{4}

kips = 1 kip = 1000 lbs

klf = units in kips per linear foot

L = live load

L_r = roof live load

l_c = length of compression member, measure center-to-center of joints, in.

l_w = length of entire wall considered in direction of shear force

M_a = maximum moment in member due to service loads at stage deflection is calculated

M_{cr} = cracking moment

M_n = nominal flexural strength at section

M_{sa} = maximum moment in wall due to service loads, excluding $P\Delta$ effects

M_u = factored moment at section

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

O.C. = on center

pcf = unit in pounds per cubic foot

plf = unit in pounds per linear foot

psf = unit in pounds per square foot

P_a = unfactored axial force not including the panel self-weight

P_{sm} = unfactored axial force including panel self-weight at mid-height of panel

P_u = factored axial force

P_{ua} = factored axial force not including the panel self-weight

P_{um} = factored axial force including panel self-weight at mid-height of panel

$P\Delta$ = secondary moment due to lateral deflection

Q_E = effects of horizontal seismic forces from V or F_P

R = rain load

S = snow load

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

T = tension force

W = wind load

w_s = unfactored load per unit length

w_u = factored load per unit length

y_t = distance from centroidal axis of gross section, neglecting reinforcement, to tension face, in.

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

Δ_{cr} = calculated out-of-plane deflection at mid-height of wall corresponding to cracking moment, M_{cr}, in.

Δ_n = calculated out-of-plane deflection at mid-height of wall corresponding to nominal flexural strength moment, M_n, in.

Δ_s = out-of-plane deflection due to service loads, in.

Δ_u = calculated out-of-plane deflection at mid-height of wall due to factored loads, in.

ε_t = net tensile strain in extreme layer of longitudinal tension reinforcement at nominal strength

ε_{ty} = value of net tensile strain in the extreme layer of longitudinal tension reinforcement used to define a compression-controlled section.

γ_c = density of concrete, lbs/ft^3

ϕ = strength reduction factor

λ = modification factor to reflect the reduced mechanical properties of lightweight concrete relative to normal weight concrete of the same compressive strength

ρ = redundancy factor

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

ρ_t = ratio of area of distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement

Chapter 1 - Introduction

1.1 Scope of the Report

In the built environment energy efficiency requirements and guidelines are growing. The use of sandwich tilt-up panels to meet these demands is on the rise. This report focuses on the comparison of single-wythe tilt-up panels and sandwich, double-wythe tilt-up panels, composite and non-composite. The advantages and disadvantages of the design, construction, and energy efficiency of these are presented. The energy efficiency, the amount of heating and cooling required in a building, of the panels is discussed in Chapter 3. Construction considerations, such as, forming materials, casting, and crane capacity, are presented in Chapter 4. For the design of the panels, vertical stresses, design moment strengths, cracking moments, applied ultimate moments, and service load deflections are compared in Chapters 6, 7, 8. In addition, projects that used standard tilt-up panels and sandwich tilt-up panels are discussed in Chapter 2.

A parametric study is done in this report by analyzing three tilt-up panels: the solid single-wythe panel, solid non-composite sandwich panel, and solid composite sandwich panel. Both sandwich panels are double-wythe separated by rigid insulation. The design and analysis of the different types of panels are in Chapters 5, 6, 7, and 8. The one-story, building used for the parametric study was an example building used in the National Council of Structural Engineers Associations' *2015 IBC SEAOC Structural/Seismic Design Manual Volume 2*. The roof framing plan shown in Chapter 5 is used to determine the joist gravity loads. The tilt-up panel analyzed is 21-foot wall with two feet parapet, 23 feet total in height. The three types of panels are subjected to the same eccentric axial loads from the roof joists and out-of-plane seismic loads. These loads are outlined in Chapter 5.

The standard tilt-up panel, solid panel without insulation, analyzed in Chapter 6 is a 5.5-inch thick concrete wall. This thickness was chosen for strength purposes and the use of standard 2x6 wood lumber, forms, without ripping lumber to a smaller size. The 5.5-inch panel (wythe) is constant in the parametric study. For the non-composite sandwich panel analyzed in Chapter 7, a 3.5-inch exterior non-load-bearing wythe (architectural) and 2.5-inch rigid insulation is added to the 5.5-inch interior load-bearing wythe (structural). Chapter 8 presents the analysis of the composite sandwich panel consisting of the same thickness and composition of wythes as the non-composite panel, 3.5-inch exterior wythe, 2.5-inch rigid insulation, and 5.5-inch interior load-bearing wythe, except the exterior wythe is load-bearing. The 2.5-inch rigid insulation was chosen to provide the insulation required by current energy codes.

1.2 Tilt-Up

Tilt-up concrete panels are known by several names. Originally, they were called "site-cast precast panels" or simply "precast panels". Today they may be known as "slender concrete panels" or "tilt-up panels". "Tilt-up panel" is used throughout this report. The first tilt-up building was built by Robert Hunter Aiken in 1893 at Camp Logan in Illinois. He used a specially-designed tipping table to make and position wall panels which was called "Aiken Method of House Building". As a result, he has been known as the "Father of Tilt-Up Construction". Robert Aiken also developed the first insulated tilt-up wall panel. His design of the insulated tilt-up wall panel consisted of two wythes of 2-inch thick concrete separated by two inches of sand. When these panels were tilted into the vertical position, the sand would pour out of the cavity leaving a two-inch air space acting as insulation. Several buildings were constructed using Aiken's method such as the Memorial United Methodist Church in Zion, Illinois in 1906, the Camp Perry Commissary Building 2009 located near Port Clinton, Ohio in

1908, and the Paint Shop building of the Los Angeles Railway Company in 1911 (Dayton Superior).

In 1910, Thomas Fellows introduced a variation to the Aiken method by casting the walls on the ground and later positioning them upright using a mechanical crane. Several buildings were built using Fellow's variations while some still used Aiken's method; such as, the Banning House in Los Angeles, and the La Jolla Women's Club buildings built by Irving Gill in 1912 and 1913, respectively (Dayton Superior).

Aiken's method was deemed obsolete after 1913 because it did not meet the rising demand of faster construction and bigger panels. Fellow's modern tilt-up construction technique has been used since the beginning of World War II when ready-mix concrete and mobile cranes were readily available. Today, more than 650 million square feet of tilt-up buildings are built every year (Tilt-Up Concrete Association, 2017).

Now, a tilt-up panel is an on-site, precast, concrete wall used in constructing buildings by lifting the panel from the horizontal position to the vertical position to form the perimeter load-bearing walls of a building. In some panels, openings for doors and windows are formed. These wall panels are prefabricated horizontally on a smooth concrete casting bed, typically the slab-on-grade for the building, and dimension lumber is used as the side formwork as shown in *Figure 1-1*. Panels may be cast individually or in long sections using a common form divider between the panels. Stack casting panels, forming and casting panels on top of each other, is used when the slab-on-grade does not provide enough casting surface and waste slabs, casting slabs outside the building perimeter are not used. The concrete for the panels is tested for compressive strength before lifting by testing concrete cylinders or, for tensile strength or flexural strength, by testing concrete beams. Typically, the tilt-up panel is lifted when the compressive strength of the concrete reaches at least 2500 psi or 75% of the specified minimum 28-day compressive strength, if not specified differently in the panel erection manual (the lifting insert may require higher strength). The panels are lifted by cranes and positioned on footings/setting pads as shown in *Figure 1-2*. Panels are temporarily braced using post-installed braces as shown in *Figure 1-4 and Figure 1-5*. After, the roof and floor systems are installed, anchored to the load-bearing panels; they become an integral part of the building structure, see *Figure 1-3*. Typically, once the roof and floor systems are attached to the panels and are complete, the temporary bracing is removed.

Figure 1-1. Formwork for casting the wall panel using dimension lumber. Photo courtesy of (constructionphotographs.com, 2012).

Figure 1-2. Tilt-up wall panels positioned on footing pads. Photo courtesy of (constructionphotographs.com, 2012).

Figure 1-3. Roof framing installed after wall panels are in place. Photo courtesy of (constructionphotographs.com, 2012).

Figure 1-4. Braces screwed onto the slab. Photo courtesy of (constructionphotographs.com, 2012).

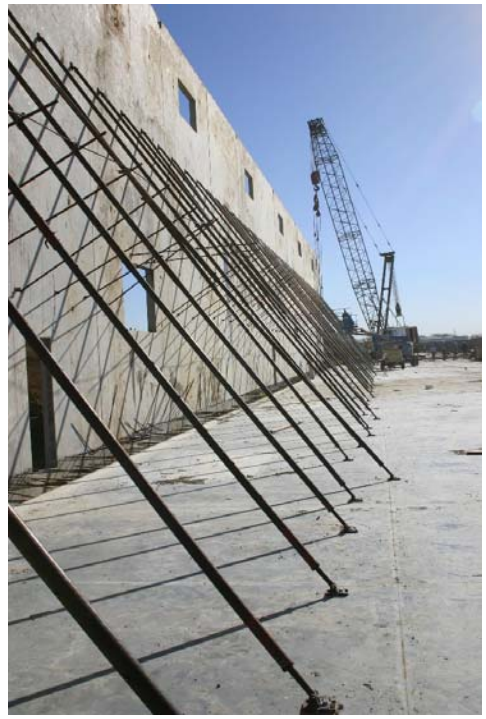

Figure 1-5. Tilt-up wall panel bracing. Photo courtesy of (constructionphotographs.com, 2012).

Presently, tilt-up panels are being used extensively in the United States of America and other countries because it is an efficient way to build structures. Architects and engineers use it due to the speed of construction and its durability while owners and developers considers its economic aspect, aesthetics, and cost-effectiveness as its best features.

1.3 Sandwich Tilt-Up Panels

A sandwich tilt-up panel also known as insulated tilt-up panel is composed of three layers – two layers of concrete and a layer of insulation sandwiched between the concrete wythes. A series of wythe connectors are cast in the first concrete wythe to be constructed, then inserted through the insulation layer, and lastly cast in the second concrete wythe. These shear ties provide integrity between the interior and exterior concrete sections, or wythes; shown in *Figure 1-6* are non-composite wall ties. The shear ties allow the panels to be lifted and handled during building erection and behave as composite elements against flexural demands. The insulation is sandwiched between an exterior concrete layer and an interior one to create a thermal break and limit damage to the insulation during construction and service. This allows for a high thermal resistance (R-Value) and it also prevents moisture migration. Thermal bridges can occur if the insulation is not continuous which may result in a high probability of moisture transfer and a lower R-Value. The main advantage of using sandwich, tilt-up panels is that it provides energy efficiency, low maintenance, and speedy construction.

When compared to standard tilt-up panels post-insulated with metal studs and batt insulation, sandwich tilt-up panels prevent thermal bridging due to the absence of metal studs. By having a built-in insulation in the panel, construction requires less labor, coordination, and labor crews compared to standard tilt-up panels post-insulated.

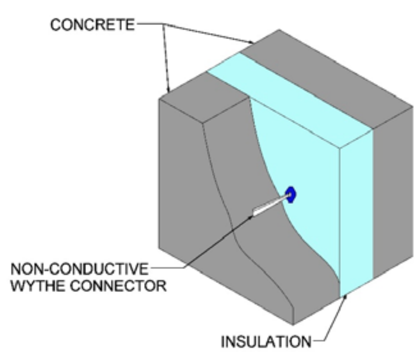

Figure 1-6. Non-conductive wythe connector essential in sandwich panels to tie the two layers of concrete together.

There are types of sandwich panels – composite (partial or full) and non-composite sandwich panels. For composite sandwich panels, the two layers of concrete act together, based on their stiffness and capacity of the connector, to carry the loads that are subjected to the wall.

Special shear connectors transfer the loads between the two concrete layers. Varying the type and arrangement of the shear ties controls the amount of composite between the two wythes. For non-composite sandwich panels, the two layers of concrete act independently. Typically, the exterior layer serves as the exterior finish and is non-load bearing, except for carrying its self-weight, while the interior layer carries the loads that are subjected to the wall system including stability of the exterior architectural wythe. Depending on the moment of inertia of each wythe, some designs for non-composite sandwich panels have both wythes carry the loads. The load-bearing wythe is usually thicker than the non-load bearing wythe due to the strength required.

There are several functions of a wythe connector. The connectors resist the tensile forces due to the weight and the suction of the lower wythe for the tilt-up panels that are lifted from the slab-on-grade form. After the panel has been placed in its final position, the wythe connectors have to resist the tensile forces due to the out-of-plane wind and seismic loading. Also, these connectors must resist the horizontal, in-plane shear due to flexural bending of the wall. The flexural demands placed on sandwich panels produce internal compression, tension, and shear within the section. To support these internal demands as a composite section, the sandwich panel must have adequate tie reinforcement between the interior and the exterior concrete wythes. This is accomplished by the placement of shear ties or the use of solid concrete zones between wythes – solid zones cause a thermal bridging which reduces the energy efficiency of the panel (PCI Committee on Precast Sandwich Wall Panels).

The shear connectors allow the transfer of in-plane shear forces between the two wythes. Typically, the sandwich panels are designed as one-way structural members in which the shear forces are produced due to the bending of the panels longitudinally. The shear connectors can be used to transfer the weight of the non-load-bearing wythe to the load-bearing wythe. Shear connectors that are designed to be flexible in the orthogonal direction and rigid in one direction are called one-way shear connectors. Shear connectors that can transfer both longitudinal and transverse horizontal shears are designed to be rigid in at least two perpendicular directions. These connectors are typically used at the panel edges and at lifting points. Non-composite connectors are typically only able to transfer the tension forces between the wythes (PCI Committee on Precast Sandwich Wall Panels).

The shear ties are available in a variety of materials and configurations. These include carbon steel, stainless steel, galvanized carbon steel, carbon fiver reinforced polymer, glass fiver reinforced polymer, and basalt fiver reinforced polymer. Shear ties are produced as trusses, pins, rods, and grids resulting in a broad range of deformation ability. A thin steel rod results in a flexible response with large ductility while a FRP truss tie produces a stiff brittle response. The flexural performance of a wall panel can vary significantly on the basis of the tie used. The connectors used in tilt-up panels must be compatible with concrete. They also must be thermally non-conductive and durable. Connectors that are prone to alkaline attack or have higher thermal coefficient of expansion than concrete cannot be used in sandwich panels as wythe ties (Thermomass).

Using an insulated tilt-up panel improves the R-value of the wall panel. According the 2015 International Energy Conservation Code, the R-value or thermal resistance is "the inverse of the time rate of heat flow through a body from one of its bounding surfaces to the other surface for a unit temperature difference between the two surfaces, under steady state conditions per unit area." Simply, it is the measure of thermal resistance which is the ability to prevent transfer of heat. The insulation between the concrete layers in a sandwich panel significantly slows down the heat of one side of the wall from transferring to the other side of the wall. As the R-Value of a material increases, the ability of that material to resist heat transfer also increases.

The R-value of an 8-inch thick normal weight concrete wall ranges from 0.4 to 1.14 $h{\cdot}ft^2{\cdot}°F/Btu$ while a 1-inch thick extruded polystyrene insulation has an R-vale of 5.0 $h{\cdot}ft^2{\cdot}°F/Btu$ (Howell, Coad, & Sauer, Jr., 2013). It is evident that by adding insulation in between the concrete layers, the wall yields a much higher thermal resistance compared to an uninsulated solid single wythe panel. The energy needed to heat or cool a building decreases if there is low thermal loss, therefore saving energy.

As a result of the density of concrete, the panel has the capacity to absorb and accumulate large amount of heat. Due to this thermal mass, concrete reacts gradually to the changes in outside temperature. It reduces the cooling and heating load peaks and it also delays the time of when the peak loads occur. The benefits of the thermal mass of concrete is prominent in regions where the difference between the inside and the outside temperature vary significantly (PCI). Since sandwich panels are typically thicker than standard tilt-up panels and they have integral insulation, it is apparent that sandwich panels will provide more energy efficiency into the building than single-wythe tilt-up panels. Approximately 20% - 30% of the tilt-up structures today use sandwich tilt-up panels (Baty, 2017).

Chapter 2 - Tilt-Up Concrete Structures

Tilt-up structures vary from small banks to large distribution centers. The ideal project for tilt-up structures are warehouses due to their simple dimensions and it allows for fast and economical construction (Tilt-Up Concrete Association, 2017). Tilt-up is not only for simple dimension and plain structures. Architectural reveals can also be made on the exterior face of the walls. Brick, stone, and other architectural finishes can be incorporated in the wall. Curved tilt-up walls can also be constructed to offer a variety other than flat panels. In this chapter, examples of standard tilt-up structures and sandwich tilt-up structures are presented.

Figure 2-1. Cinemark 12 Movie Theater in Mansfield, Texas. Architecturally designed by Beck Architecture, developed by Kossman Development Company, and constructed by Bob Moore Construction. Photo courtesy of (Bob Moore Construction, 2014).

The Cinemark 12 Movie Theater in Mansfield, Texas is a 42,265 square-foot standard tilt-up structure which houses 12 screens, a large concession area and seats 2,010 people. The largest panel area was 1,027 square feet, with the tallest panel height of 51 feet and 11 inches, and the heaviest panel is 123,240 pounds. By choosing tilt-up, it allowed for a fast construction due to the overlapping of trades. Cinemark finished construction almost two weeks before the original schedule (Bob Moore Construction, 2014).

Rooms To Go Distribution Center/Showroom shown in *Figures 2-2* and *2-3* houses offices, breakrooms, retail stores, a warehouse, and distribution center in Brookshire, Texas. The structure is 1,252,000 square feet and the main distribution center/showroom is 988,000 square feet which is of standard tilt-up construction. It also includes a classroom with auditorium style seating for employee trainings. The structure comprises floor areas that are in several different elevations. Seven bays are at ground level where customer pick-up is located, six bays at 18 inches, 35 bays at 54 inches, and 125 bays at 44 inches above the ground level. Due to the heavy moving loads, the floor joints are finished with armored hinging (Bob Moore Construction, 2014).

Figure 2-2. Northeast view of the Rooms To Go Distribution Center/Showroom in Brookshire, Texas. Architecturally designed by MacGregor Associates Architects, Inc. and constructed by Bob Moore Construction. Photo courtesy of (Bob Moore Construction, 2014).

Figure 2-3. Southeast view of the Room To Go Distribution/Showroom in Brookshire, Texas. Photo courtesy of (Bob Moore Construction, 2014).

Tilt-up walls can also be constructed as curved wall panels. *Figures 2-4, 2.5,* and *2-6* show finished structures that feature the capability of the construction of curved tilt-up walls. The International Parkway Tech Center in Carrollton, Texas is a 118,000 square-foot standard tilt-up structure. The exterior wall also features stainless steel finish. The Flooring Services, Inc. headquarters/distribution center is shown in *Figure 2-6*. It is a 421,085 square-foot tilt-up structure that houses office spaces, warehouse, and distribution center. The entrance features curved tilt-up walls carried by columns underneath the floor (Bob Moore Construction, 2014).

Figure 2-4. Southwest corner of the International Parkway Tech Center in Carrollton, Texas. Architecturally designed by Hardy McCullah/MLM Architects, Inc., developed by CMC - Commercial Realty Group, and constructed by Bob Moore Construction. Photo courtesy of (Bob Moore Construction, 2014).

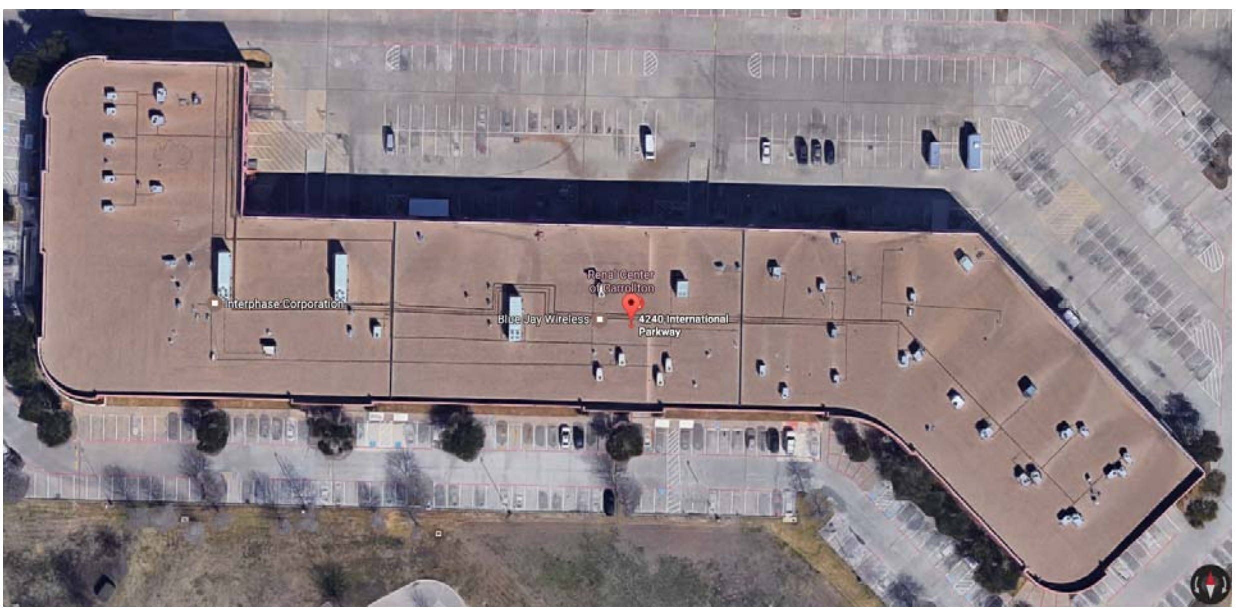

Figure 2-5. Aerial plan view of the International Parkway Tech Center in Carrollton, Texas. Photo courtesy of (Google, 2017).

Figure 2-6. Flooring Services, Inc. Headquarters/Distribution Center in Lewisville, Texas. Architecturally designed by Pross Design Group and constructed by Bob Moore Construction. Photo courtesy of (Bob Moore Construction, 2014).

The Ave Maria University Oratory that seats 1,110 people is located in Southwest Florida. Constructability studies were conducted prior to the construction of the oratory. The standard tilt-up panels are welded to the steel frame of the structure which is 100 feet in height. Each of the panels have a five-feet radius return wall. One of the advantages of using the tilt-up panels is that it resisted the uplift loads due to the heavy weight of the panels. The stone block finish applied to the exterior face of the entrance wall was imported from Italy. The finish matched the Italian themed buildings around it (Woodland Tilt-Up, 2015). The tallest panel is 42 feet and 3 inches; the widest panels is 34 feet and 6 inches; the largest panel is 462 square feet; and the heaviest panel is 110,956 lbs (Tilt-Up Concrete Association, 2017).

Figure 2-7. Ave Maria University Oratory in Southwest Florida. 2008 TCA Tilt-Up Achievement Award - Spiritual Division. Photo courtesy of (Google, 2017).

Figure 2-8. Southwest view of the Ave Maria University Oratory. Photo courtesy of (Google, 2017).

One of the many structures that utilized sandwich tilt-up panels is the test hall of the United Technologies Fire and Security Innovation in Jupiter Florida. This LEED Gold Certified Project is a 19,750 square feet manufacturing/industrial facility. This building received the 2013 TCA Tilt-Up Achievement Award – Manufacturing/Industrial Division. The panels consist of 12 inches of structural wythe for the interior, 2 inches of insulation, and 3.75 inches of architectural wythe for the exterior. To lift the panels in place, 300-ton and 275-ton crawler cranes were used (Woodland Tilt-Up, 2015). The tallest panel is 67 feet; the widest panels is 25 feet and 9 inches; the largest panel is 1,758 square feet; and the heaviest panel is 268 lbs (Tilt-Up Concrete Association, 2017).

Figure 2-9. Rutledge Elementary School in Austin, Texas. Utilized 150 sandwich tilt-up panels. Photo courtesy of (Google, 2017).

Another example of a structure that uses sandwich panels is the Rutledge Elementary School in Austin, Texas. The project used a total of 150 tilt-up panels. The main priorities of the school district for this structure is energy-efficiency, durability, high resistance to mold due to humidity, and matching façade to the other schools in the district. Choosing tilt-up construction sped up the building process. The structure was completed on time even with the three-week delay due to Texas' wettest winter. The panels were site-casted in two separate areas that helped with the construction schedule. The sandwich panels used in this project consist of concrete interior wall, 2 inches of insulation, and exterior brick veneer. This system yielded an R-value of 24 which answered the energy-efficiency concern of the owners. Moreover, the interior concrete wall of the panel offered a more durable solution than metal studs and drywall (Thermomass). The tallest panel is 44 feet; the largest panel is 898 square feet; and the heaviest panel is 112,000 lbs (Tilt-Up Concrete Association, 2017).

Figure 2-10. St. Sarkis Armenian Apostolic Church in Charlotte, North Carolina completed in 2005. Architectrually designed by SAA Architecture, LLC and constructed by Seretta Construction. Photo courtesy of (Google, 2017).

The 15,000 square-foot St. Sarkis Armenian Apostolic Church in Charlotte, North Carolina utilized site-casted sandwich tilt-up panels and received the American Institute of Architects Southeast Award of Merit for Concrete and the 2006 TCA Tilt-Up Award. The main goal for the construction of this project was the preservation of the Armenian history, energy-efficiency, and cost effectiveness. The exterior face of the wall panels was formed and colored to resemble volcanic stones in Caucasus Mountains in Eastern Europe. Tilt-up concrete provided the variety of choices for the exterior finish at a lower cost and uncompromised durability. Since the interior walls were made out of concrete, they provided more strength than metal studs. Moreover, the interior walls can be designed to address acoustical issues regarding reverberance (Thermomass). The tallest panel is 39 feet and 3 inches; the largest panel is 707 square feet; and the heaviest panel is 81,600 lbs (Tilt-Up Concrete Association, 2017).

Chapter 3 - Energy Efficiency

Comfortable environment within the structure is essential especially in regions where major differences between the indoor and outdoor conditions occur. Proper design and construction of the buildings is vital to provide the ideal conditions possible. Codes and standards are established to help engineers, contractors, and designers meet these conditions. Within these standards, minimum thermal resistance values are defined for floor, ceiling, wall, and roof systems.

For this report, the energy efficiency, in terms of the amount of heating and cooling required in a building, of the standard solid wall panel and sandwich panel presented in Chapter 1 are considered. Based on the *2015 International Energy Conservation Code*, Springfield, Missouri requires a minimum R-Value of 9.5 for concrete walls. Per the *2013 American Society of Heating, Refrigerating and Air-Conditioning Engineers Handbook*, for 5.5-inch and 3.5-inch thick concrete walls with 15.0 Btu·in/h·ft^2·°F conductivity, the R-Values are 0.367 h·ft^2·°F/Btu and 0.233 h·ft^2·°F/Btu, respectively. These are shown in *Equation 3-1* and *Equation 3-2*. For a wall composition of 3.5 inches of exterior concrete and 5.5 inches of interior concrete, the R-Value is 0.6 h·ft^2·°F/Btu. Insulation is added to make up the difference of the R-Value. Extruded polystyrene insulation has a conductivity of 0.20 Btu·in/h·ft^2·°F. Using *Equation 3-3*, the required insulation thickness is 2.5 inches.

$$R = \frac{thickness}{conductivity} = \frac{5.5\ in}{15.0\ Btu \cdot \frac{in}{h} \cdot ft^2 \cdot \text{°F}}$$ *Equation 3-1*

$$R = 0.367\ h \cdot ft^2 \cdot \text{°F}/Btu$$

$$R = \frac{thickness}{conductivity} = \frac{3.5\ in}{15.0\ Btu \cdot \frac{in}{h} \cdot ft^2 \cdot °\mathrm{F}}$$ *Equation 3-2*

$$R = 0.233\ h \cdot ft^2 \cdot °\mathrm{F}/Btu$$

$$thickness = (R_{required} - R_{wall}) \times conductivity$$ *Equation 3-3*

$$= (13 - 0.367 - 0.233)\ h \cdot ft^2 \cdot °\mathrm{F}/Btu \times (0.20\ Btu \cdot in/h \cdot ft^2 \cdot °\mathrm{F})$$

$$thickness = 2.48\ inches \rightarrow 2.5\ inches$$

With the same wall panel compositions for the standard solid tilt-up panel, non-composite sandwich tilt-up panel, and composite sandwich tilt-up panel used in Chapters 6, 7, and 8, the R-Values are tabulated in *Table 3-1*. The R-Value of the standard solid panel without insulation is 95% less than the sandwich panels while the standard solid panel with insulation is 2% less.

Table 3-1. R-Values of Wall Composition

Tilt-Up Wall Panel Type	*R-Value ($h \cdot ft^2 \cdot °F/Btu$)*
Standard Solid without insulation • *5.5 in. concrete*	0.4
Standard Solid with insulation • *5.5 in. concrete and 2.5 in. insulation*	12.9
Non-Composite Sandwich • *9 in. concrete and 2.5 in. insulation*	13.1
Composite Sandwich • *9 in. concrete and 2.5 in. insulation*	13.1

A study was performed using TRACE 700 software to measure the total building loads required for the HVAC system per tilt-up wall panel type. The building mentioned in Chapter 1 is a 40,320 square-foot warehouse located in Springfield, Missouri (see *Figure 5.1*). Based on the location, the outdoor cooling dry bulb temperature is 93°F and the wet bulb temperature is 77°F while the heating dry bulb temperature is 9°F. For the indoor conditions, the cooling dry bulb temperature is 75°F, the heating dry bulb temperature is 70°F, and the relative humidity is 50%. The airflows for cooling and heating ventilation is 0.05 cfm/sf. The study calculated the total building cooling and heating loads using the same roof components, floor components, internal loads, airflows, building orientation, and location. Only the exterior tilt-up wall panels were changed to compare the total building loads for each different panel type.

The total building cooling loads for the different types of tilt-up wall panels are tabulated in *Table 3-2*. By using sandwich tilt-up panels, it is 50% more efficient in cooling than standard solid panels without insulation and 68% more efficient in heating. Moreover, sandwich panels are about 8% and 2% more efficient in cooling and heating, respectively, compared to standard solid panels with insulation.

Table 3-2. Total Building Cooling and Heating Loads Per Wall Panel Type.

Tilt-Up Wall Panel Type	*Total Building Cooling Load (MBh)*	*Total Building Heating Load (MBh)*
Standard Solid without insulation	994.2	1,741.1
Standard Solid with insulation	544.2	568.8
Non-composite Sandwich	496.8	556.1
Composite Sandwich	496.8	556.1

Chapter 4 - Construction

Several differences between the construction methods of the standard and sandwich tilt-up panels exist. This chapter presents the differences of the forming materials, the casting of concrete, the lifting cranes capacity needed between the standard single wythe tilt-up panel and the sandwich panel. Proper construction is imperative to achieve the full extent of the structural design and the energy efficiency of the structure.

4.1 Forming Materials

The most common forming material used is dimension lumber. Dimension lumber can also be ripped to the desired dimension. The number of splices should be minimized when ordering the lengths of the lumber. Knots in the lumber are not a major concern since the sides of the wall panels are hidden in between the joints. These side forms should not be oiled because oils are not compatible with the bond breakers used to prevent the wall panel from adhering to the slab where it is casted.

The size of the lumber is determined by the thickness of the wall panel. If the desired wall panel thickness is 5.5 inches, then a 2x6 dimension lumber should be used as the edge form of the panel. Moreover, for a 7.25-inch thick panel, a 2x8 dimension lumber should be used. If the desired panel thickness does not coincide with the depth of the dimension lumber, nailing a strip of plywood or a smaller size of lumber is done. For example, if the required panel thickness is 6.25 inches, then nailing a 1x2 dimension lumber to the top of the 2x6 will suffice. If constructing an 8-inch thick panel, use a 2x8 and 1x2. Although, the following method is not very efficient, ripping the lumber lengthwise to the desired thickness can also solve the problem. Using deeper edge forms and having the concrete below the full depth of the form makes levelling and finishing difficult.

Steel channels or angles are also used as edge forms but are not as common as dimension lumber and gives less flexibility when it comes to its dimension. Engineered lumber made of plywood or pressed wood can also be used as edge forms. Dimension lumber can be used no more than three times while the engineered lumber and steel edge forms can be used continually.

Since sandwich panels are thicker than regular solid single wythe panels, wider dimension lumbers are used as edge forms when casting the panels. Wider dimension lumber can accommodate the two layers of concrete and the insulation in between. Although, using a sheeted form, such as plywood, is more efficient than using wider dimension lumber.

4.2 Casting

When casting sandwich wall panels, it is typically done in two pours. The exterior layer with reinforcement is poured and the insulation and wythe connectors are placed simultaneously. The second pour for the interior layer is done after the installation of the reinforcing steel, lifting inserts, and anchor braces.

The anchorage capacity and the mix design for a sandwich panel is different from a standard solid single wythe panel. Typically, the exterior wythe is non-load bearing and the wythe connectors are installed in a plastic mix. To ensure appropriate performance of the connectors, the concrete mix is recommended to have a slump of 5 to 7 inches. The concrete around the anchorage point of the connector must be properly consolidated.

The exterior concrete layer should be at least 3000 psi to obtain adequate strength. Continuous pour of the concrete is crucial to prevent cold joints. The concrete must have the minimum compressive strength at 28 days specified in the drawings for panel erection. The concrete slump of the exterior concrete layer should be within the 4 to 7-inch range and should be maintained throughout the installation of the insulation system. The maximum aggregate size for the exterior wythe should be ½ inch whereas the maximum size of aggregate for a standard single wythe is typically ¾ inch or 1 inch (Tilt-Up Concrete Association, 2011). Aggregate sizes larger than specified obstruct the placement of the wythe connectors. Placing the insulation edge-to-edge helps to attain continuous thermal and moisture resistance. Thermal bridges and moisture migration is a result of improper installation of the insulation. Foaming insulation can be used to fill the gaps that are greater than ¼ inch between the insulation sheets to avoid thermal bridges. Once the exterior concrete layer reaches 25% of the 28-day specified strength, a pull-out test should be performed on the connectors. See *Table 4.1* for the minimum time from casting to anchorage test. This period depends on the ambient temperature (Tilt-Up Concrete Association, 2011).

Table 4-1. Minimum Period Prior to Pull-out Test Performed on the Connectors.

Ambient Temperature Range (℉)	*Minimum Time from Casting to Anchorage Test (hours)*
70 ≤	12
60 – 70	24
40 – 60	36
< 40	Field-Cured Cylinder Test Required

Once the proper tests are performed for the anchorage of the connectors and interior concrete layer with reinforcement is poured, the panel is then finished, erected, and braced similarly as the standard single wythe panel.

4.3 Cranes

The crane capacity between standard single-wythe panels and sandwich panels differ due to the weight of the panels. When selecting the crane lifting capacity, the heaviest panel lift, the boom reach distance, and the distance required to carry the panel while the crane is moving are taken into consideration. The boom reach distance is based on the panel heights and the lengths of slings to pick the panels.

Formerly, truck cranes were the most common cranes being used to erect tilt-up panels. Truck cranes, shown in *Figure 4-1*, can be driven on the highway legally to the jobsite. The crane is prepared within an hour after arriving the jobsite. Hydraulic controlled outriggers extend out from the crane to stabilize a truck crane. These outriggers have pads at the ends to be set on the floor slab. While carrying the panel from the casting bed to its final position, almost all the weight is being carried by the pads or on the eight rear tires during lifting. Because of this, the design of the floor slab is crucial.

The use of crawler cranes, shown in *Figure 4-2*, has increased due to their greater lifting capacity of 250 to 300 tons and their capability to work off the slab. Crawler cranes can carry or "walk" a panel that weighs 100 tons. Unlike truck cranes, crawler cranes are transported on trailers to the jobsite. Once it has arrived on the jobsite, it is assembled which sometimes requires an additional crane. The cost of transportation of crawler cranes alone may cost $10,000 or more.   Purchasing a new 300-ton crawler crane plus spreader bars, blocks, slings, and shackles can cost $2,000,000 or more (Tilt-Up Concrete Association, 2011).

The heaviest panel of up to 30 feet in height should not weigh more than half the capacity of the crane and the heaviest panel of more than 30 feet in height should not weigh more than a third of the capacity of the crane (The Construction of Tilt-Up, 2011). Since the panel weight of a sandwich panel would be much larger than that of a single wythe panel due to the additional layer of concrete, a higher crane lifting capacity is need to perform the erection of the panels.

Figure 4-1. Truck crane with outriggers on the slab. Photo courtesy of (constructionphotographs.com, 2012).

Figure 4-2. Crawler crane off the slab. Photo courtesy of (constructionphotographs.com, 2012).

Chapter 5 - Slender Reinforced Concrete Wall (Tilt-Up) Design

A wall panel must resist vertical and horizontal loads. It is designed to resist vertical eccentric and/or concentric axial loads, horizontal out-of-plane loads that are applied normal to the face of the wall, and in-plane loads horizontal along the wall panel, see *Figure 5-1*. A slender wall should take into consideration the additional bending induced in the wall due to its deflection, *P*-Δ effects. In this report, a 21-foot wall with two feet parapet, 23 feet total in height, will be designed assuming a pin-pin connection at the base and a roof diaphragm location. The design example in this chapter and the following chapters uses the tilt-up design example in the *2015 IBC SEAOC Structural/Seismic Design Manual*.

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deflection, P-Δ effects. In this report, a 21-foot wall with two feet parapet, 23 feet total in height, will be designed assuming a pin-pin connection at the base and a roof diaphragm location. The design example in this chapter and the following chapters uses the tilt-up design example in the 2009 IBC SEAOC Structural/Seismic Design Manual.

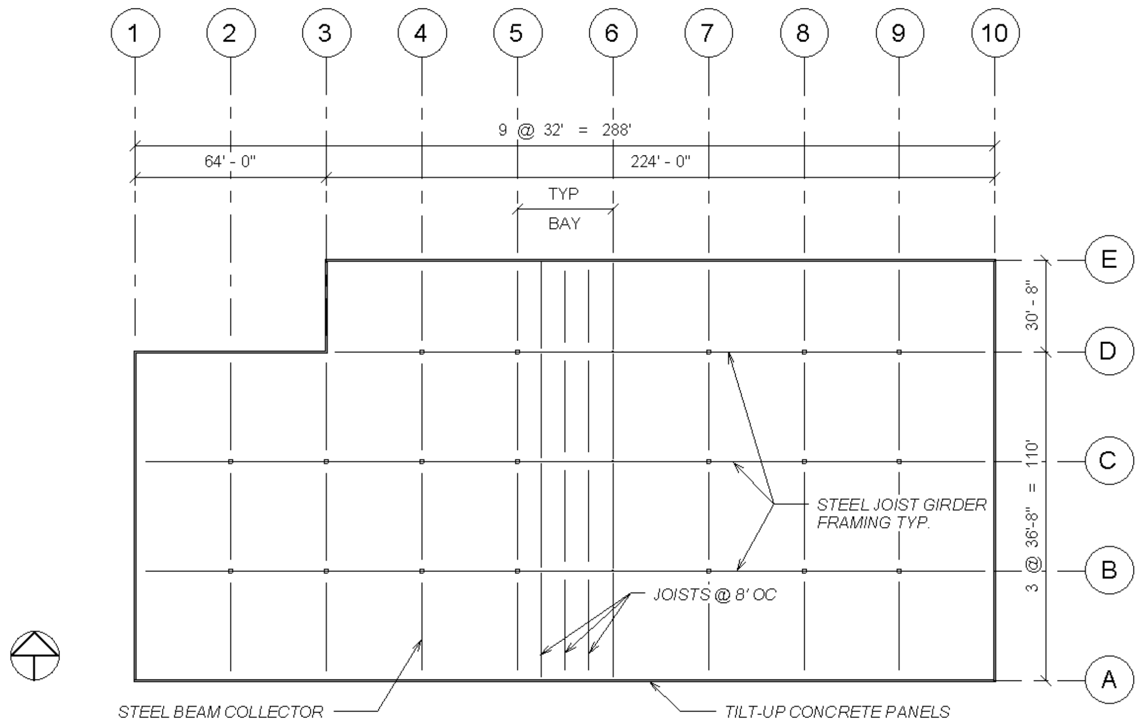


Figure 5-1. Roof framing plan of tilt-up building. Adapted from (Structural Engineers Association of California (SEAOC), 2012)

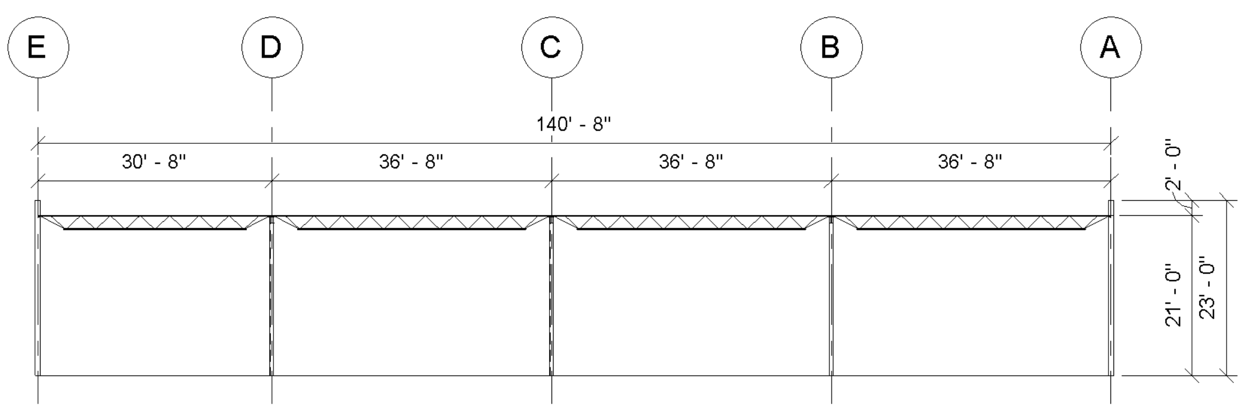


Figure 5-2. Typical cross-section. Adapted from (Structural Engineers Association of California (SEAOC), 2012)

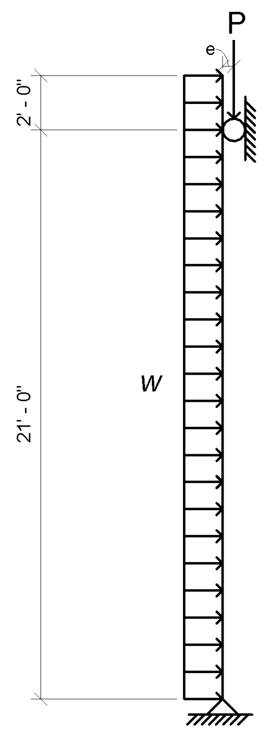


Figure 5-3. Eccentric axial load applied at distance e from the center of the wall and out-of-plane load applied normal to the face of the wall. The moment caused by the eccentric loading and out-of-plane loading is modeled in *Figure 5-3*.

5.1 Loads

Determining the loads being carried by the wall panel is the first step in design. The three loading directions a wall panel may be subjected to are axial, out-of-plane, and in-plane. Axial loading is applied in the vertical direction while the out-of-plane loading and in-plane shear are applied in the horizontal direction. In this report, the wall panel is examined and designed for gravity loads and out-of-plane loading lateral seismic load per the design example in the *2015 IBC SEAOC Structural/Seismic Design Manual*.

5.1.1 Gravity Loads

The wall panel is subjected to gravity loads such as dead loads, roof live loads, and snow loads. In this report, snow loads do not govern the design the wall panel per the design example in the *2015 IBC SEAOC Structural/Seismic Design Manual.* Also, steel joists span 36'-8" and are spaced at 8'-0" on center. It is also bearing at the interior face of the wall, which yields an eccentricity of half the thickness of the interior wall thus inducing and eccentric load on the wall.

5.1.1.1 Dead Loads

Dead loads (*D*), per the ASCE 7, consist of the weight of all materials of construction in a building. Some examples of dead loads are the walls, floors, roofs, ceilings, stairways, partitions, architectural finishes, structural members, and service equipment. Minimum design dead loads can be obtained from Table C3-1 of ASCE 7-10. In this study, the dead load from the roof will be taken as 14 psf per the design example in the *2015 IBC SEAOC Structural/Seismic Design Manual*. This includes the weight of the structural members. The wall dead load is calculated separately in the next chapters based on the composition of the wall.

5.1.1.2 Roof Live Loads

The roof live load (L_r), as defined in the ASCE 7, is "a load on a roof produced during maintenance by workers, equipment and materials and during the life of the structure by movable objects, such as planters or other similar small decorative appurtenances that are not occupancy related." Minimum uniformly distributed live loads and minimum concentrated live loads can be obtained in Table 4-1 and Table C4-1 of ASCE 7-10. Reduction of the roof live loads are allowed based on the slope of the roof and the tributary area of the member. For this report, even though the roof live load reduction can be applied, the roof live load is taken as 20 psf.

5.1.2 Lateral Loads

The wall panel is subjected to lateral loads such as wind loads and seismic loads. These loads are applied horizontally on the panel. Out-of-plane and in-plane loading can occur from these loads. For this report, seismic loads govern over the wind loads.

5.1.2.1 Seismic Loads

Seismic Load (E) consists of two load effects, vertical seismic load effect and horizontal seismic load effect. The horizontal seismic load effect, E_h, and vertical seismic load effect, E_v, are determined using these equations from ASCE 7-10 Section 12.4.2:

$$E_h = \rho Q_E \qquad \textit{Equation 5-1}$$

$$E_h = 0.2 S_{DS} D \qquad \textit{Equation 5-2}$$

For this report ρ is taken as 1.0 and S_{DS} is 1.0 for simplification. S_{DS} = 1.0 tends to be in regions with seismic design category D or higher. Assuming that the seismic force-resisting systems in the building consist of two bays of seismic force-resisting perimeter framing on each side of the structure, the redundancy factor, ρ, may be taken as 1.0 per ASCE 7-10. In ASCE 7-10 Section 12.11.1, the out-of-plane force F_P shall be calculated using the following equation and should not be less than 10% of the weight of the structural wall:

$$F_P = 0.4 S_{DS} I_E D_{wall} \qquad \textit{Equation 5-3}$$

5.2 Alternative Method for Out-of-Plane Slender Wall Analysis – ACI 318-14, Section 11.8

According to Section 11.8 of ACI 318-14, walls should satisfy the following conditions to analyze the out-of-plane slenderness effects:

a) The cross section is constant over the height of the wall

b) The 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 ACI 318-14 Section 19.2.3

d) P_u at the mid-height section does not exceed $0.06 f'_c A_g$

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

5.2.1 Load Cases

The design strength of the structural members must be equal to or greater than the factored loads using strength design in the following combinations based on ASCE 7-10 Section 2.3.2:

1) $1.4D$ *Equation 5-4*

2) $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$ *Equation 5-5*

3) $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$ *Equation 5-6*

4) $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$ *Equation 5-7*

5) $1.2D + 1.0E + L +0.2S$ *Equation 5-8*

6) $0.9D + 1.0W$ *Equation 5-9*

7) $0.9D + 1.0E$ *Equation 5-10*

For load cases 5 and 7, the following load combinations for strength design are used to take into consideration the horizontal and vertical seismic load effects based on ASCE 7-10 Section 12.4.2.3:

5) $(1.2+0.2S_{DS})D + \rho Q_E + L +0.2S$ *Equation 5-11*

7) $(0.9D - 0.2S_{DS})D + \rho Q_E$ *Equation 5-12*

The following load combinations are the allowable stress design, and for serviceability conditions, based on ASCE 7-10 Section 2.4.1:

1) D *Equation 5-13*

2) $D + L$ *Equation 5-14*

3) $D + (L_r \text{ or } S \text{ or } R)$ *Equation 5-15*

4) $D + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$ *Equation 5-16*

5) $D + (0.6W \text{ or } 0.7E)$ *Equation 5-17*

6) $D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R)$ *Equation 5-18*

$D + 0.75L + 0.75(0.7E) + 0.75S$

7) $0.6D + 0.6W$ *Equation 5-19*

8) $0.6D + 0.7E$ *Equation 5-20*

For load cases 5, 6, and 8, the following load combinations for stress design are used to take into consideration the horizontal and vertical seismic load effects based on ASCE 7-10 Section 12.4.2.3:

5) $(1.0 + 0.14S_{DS})D + 0.7\rho Q_E$ *Equation 5-21*

6) $(1.0 + 0.10S_{DS})D + 0.525\rho Q_E + 0.75L + 0.75S$ *Equation 5-22*

8) $(0.6 - 0.14S_{DS})D + 0.7\rho Q_E$ *Equation 5-23*

For this study, load case 3 determines the greatest applied force due to gravity loads for both strength and stress designs. For the strength design of the lateral loads, load cases 5 and 7 yield the greatest applied force due to seismic loads. For the stress design of lateral loads, load cases 5 and 8 yield the greatest applied force due to seismic.

5.2.2 Design Moment Strength

The design moment strength of the wall panel shall be calculated using the following equation:

$$\phi M_n = \phi A_{se} f_y \left(d - \frac{a}{2}\right)$$ *Equation 5-24*

Based on ACI Section 21.2, the strength reduction factor, ϕ, for moment, axial force, or combined moment and axial force is 0.90 for tension-controlled sections. This is a requirement for the alternate method analysis. The section is considered tension-controlled when net tensile strain in the extreme fiber in tension is greater than 0.0005 in/in. This allows for a ductile failure showing excessive cracking and deflection before failure happens.

The following equation is used to calculate the effective area of reinforcement:

$$A_{se} = A_s + \frac{P_{um}}{f_y}\frac{h}{2d} \qquad \textit{Equation 5-25}$$

The axial forces applied to the wall counteract a percentage of the flexural tension stresses in the member cross-section. This results to an increase in the bending moment resistance of the member. For axial stresses less than 10% of the specified compressive strength of concrete, *f_c'*, the modification of the area of reinforcement accounts for the increase in the bending moment resistance. The equation used before 2008 to calculate the effective area of reinforcement overestimated the axial load contribution for walls that have two layers of reinforcement; therefore, the *h/2d* factor has been added (PCA 2013). For a single layer of reinforcement, the *h/2d* factor is close to 1.0. For two layers of reinforcement, the factor will be lower which lowers the effective area of reinforcement.

The equivalent rectangular stress distribution is used to analyze the section as permitted by ACI 318-14. *Figure 5-2* shows a diagram of the rectangular stress distribution and internal couple for simply-supported member with a downward force applied at the top.

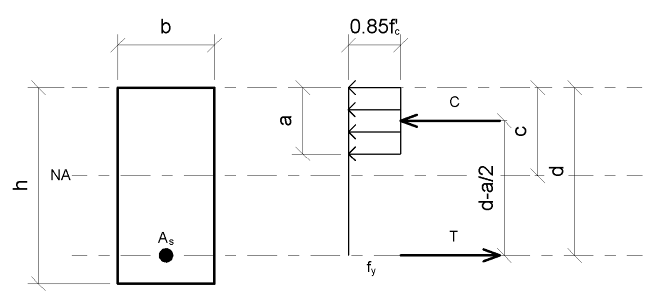


Figure 5-4. Rectangular stress distribution for simply-supported member.

To calculate the internal couple's total tension force in the member, the effective area of longitudinal, flexural, reinforcement is multiplied by the specified minimum yield strength of reinforcement. The total compression force is equal to average stress in the compression region, 85% of the specified 28-day minimum compressive strength of concrete, times the area of the compression block, the depth of equivalent rectangular stress block by width of the compression face of the member. The tension force is equal to the compression force to achieve equilibrium at the cross-section. By equating both forces, the depth of the equivalent rectangular stress block, a, can be obtained, see *Equations 5-26, 5-27, 5-28,* and *5-29*.

$$T = A_{se}f_y \quad \textit{Equation 5-26}$$

$$C = 0.85f_c'ab \quad \textit{Equation 5-27}$$

$$T = C \rightarrow A_{se}f_y = 0.85f_c'ab \quad \textit{Equation 5-28}$$

$$a = \frac{A_{se}f_y}{0.85f_c'b} \quad \textit{Equation 5-29}$$

The depth of the rectangular stress block allows for the calculation of the distance or moment arm, *d-a/2*, between the centroid of tension and compression forces needed to calculate the nominal moment strength of the section in *Equation 5-24*.

5.2.2.1 Cracking Moment

The nominal moment strength of a member has to be equal or greater than the cracking moment, M_{cr}, as stated in *ACI 318-14* Section 11.8 to prevent abrupt failure at the point when cracking starts to occur. The cracking moment is calculated using the following equation:

$$M_{cr} = \frac{f_r I_g}{y_t} \quad \textit{Equation 5-30}$$

The modulus of rupture, f_r, of the concrete section depends on the specified compressive strength of concrete and the type of concrete, whether it is lightweight, normal weight, or heavyweight concrete. For normal weight concrete, the modification factor, λ, is taken as 1.0. The modulus of rupture is calculated using the following equation based on *ACI 318-14* Sections 19.2.3 and 19.2.4:

$$f_r = 7.5\lambda\sqrt{f_c'}$$ *Equation 5-31*

5.2.2.2 Flexural Minimum Reinforcement

If the cross-section of a member is much larger than the strength required, the member could fail abruptly due to the low amount of tensile reinforcement. The calculated moment strength of such member could become less than the moment strength of an unreinforced concrete member computed from its modulus of rupture. To this failure, a specified minimum reinforcement is set. The flexural reinforcement in the member should be equal to or greater than the minimum reinforcement based on *ACI 318-14* Section 9.6.1.2. The governing minimum steel reinforcement shall be calculated using the following equations, whichever is greater:

Equation 5-32

$$A_{s,min} = \frac{3\sqrt{f_c'}}{f_y} b_w d$$

$$A_{s,min} = \frac{200}{f_y} b_w d$$

Equation 5-33

5.2.3 Minimum Vertical and Horizontal Reinforcement

The horizontal distributed reinforcement ratio, ρ_t, should be at least 0.0025. In accordance *to ACI 318-14* Section 11.6 the minimum vertical distributed reinforcement ratio, ρ_l, should be the greater of *Equation 5-34* and 0.0025 but should not be greater than the horizontal distributed reinforcement ratio, ρ_t. Tests have been conducted for walls with low height-to-length ratios. The results showed that the horizontal shear reinforcement is less effective for the shear resistance than the vertical reinforcement. *Equation 5-34* takes into account the change of effectiveness between the horizontal and vertical reinforcement. If the h_w/l_w is less than 0.5, the vertical reinforcement is equal to the horizontal reinforcement. Only the minimum vertical reinforcement is required for h_w/l_w greater than 2.5 (ACI Committee 318, 2014).

$$\rho_l \geq 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_t - 0.0025) \qquad \textit{Equation 5-34}$$

5.2.4 Applied Ultimate Moment

The wall is idealized as simply-supported for the alternative method for out-of-plane slender wall analysis. It is analyzed with axial load and uniform lateral load with maximum moments and deflections located at mid-height. Because of the uniform lateral load applied on the member as seen in *Figure 5-3,* the wall will yield a deflected shape. The axial load on the wall will increase the moment that the member experiences due to the deflected shape. This is called the *P-Δ* effect. The maximum bending moment has two components, the primary moment due to the applied loads and the secondary moment due to the *P-Δ* effects. The contributors to the primary moment that a panel experiences are the eccentric axial loads, out-of-plane lateral loads, and initial lateral deflections due to panel out-of-straightness (ACI 551.2R-15).

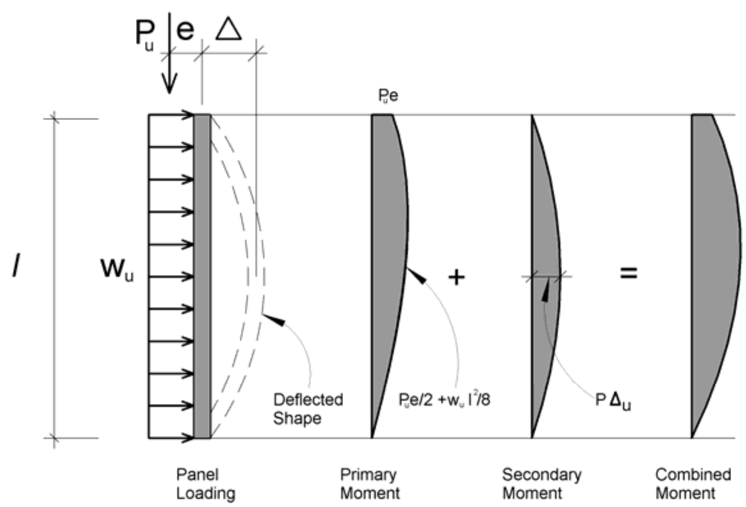


Figure 5-5. Panel design model with lateral force acting with eccentric axial load adapted from (ACI Committee 318, 2014).

Equation 5-35 yields the maximum factored applied primary moment at midheight due to the eccentric axial loads and lateral loads, not including *P-Δ* effects. In the given equation, w_u is the factored uniform lateral load which, in this study, is the seismic loads. l_c is the unbraced length of the wall panel, P_{ua} is the factored applied axial load, and the e_{cc} is the eccentricity or the distance from the center of the wall to the axial load applied to the wall panel.

$$M_{ua} = \frac{w_u l_c^2}{8} + \frac{P_{ua} e_{cc}}{2} \qquad \textit{Equation 5-35}$$

ACI 318-14 Section 11.8.3 accounts for the deflected shape by either using the iterative calculation or by direct calculation. *Equation 5-36* utilizes the iterative calculation where it considers the primary moment, in this case, M_{ua} and the secondary moment $P_u \Delta_u$. The ultimate deflection, Δ_u, is calculated using *Equation 5-37*. The reduction factor of 0.75 in *Equation 5-37* is used to reduce the bending stiffness of the concrete section. It accounts for the disparity in construction and material properties (ACI Committee 551, 2015).

$$M_u = M_{ua} + P_u \Delta_u \qquad \textit{Equation 3-36}$$

$$\Delta_u = \frac{5 M_u l_c^2}{(0.75) 48 E_c I_{cr}} \qquad \textit{Equation 5-37}$$

5.2.4.1 Moment Magnifier Method

ACI 318-14 Section 11.8.3 determines the maximum combined moment of the wall using the direct calculation equation:

$$M_u = \frac{M_{ua}}{\left(1 - \frac{5 P_u l_c^2}{(0.75) 48 E_c I_{cr}}\right)} \qquad \textit{Equation 5-38}$$

Where the equation calculates the cracking moment of inertia:

$$I_{cr} = \frac{E_s}{E_c}(A_{se})(d-c)^2 + \frac{l_w c^3}{3} \qquad \textit{Equation 5-39}$$

The cracking moment of inertia, see *Equation 5-39*, is calculated based on the rectangular stress block derivation using the effective area of reinforcement, A_{se}. The concrete is in compression and the steel reinforcement is in tension. Since the section is cracked, in ultimate strength design, the tensile force in the concrete section is transferred to the steel reinforcement.

5.2.4.2 Iteration Method

Due to the service eccentric axial loads and lateral loads, including P_s-Δ_s effects, the maximum moment at midheight of the wall, with iteration of deflections, is calculated using the following equation provided in Section 11.8.4.2 of ACI 318-14:

$$M_a = M_{sa} + P_s\Delta_s \qquad \textit{Equation 5-40}$$

Where the service load deflection is calculated using the formula when the unfactored applied load, M_a, is less than or equal to $2/3\ M_{cr}$:

$$\Delta_s = \frac{M_a}{M_{cr}}\Delta_{cr} \qquad \textit{Equation 5-41}$$

Based on test data, the out-of-plane deflections tend to increase rapidly when M_a is greater than $2/3\ M_{cr}$ (ACI Committee 318, 2014). When M_a is greater than $2/3\ M_{cr}$, the service load deflection is determined from the following equation:

$$\Delta_s = \frac{2}{3}\Delta_{cr} + \frac{(M_a - \frac{2}{3}M_{cr})}{(M_n - \frac{2}{3}M_{cr})}\left(\Delta_n - \frac{2}{3}\Delta_{cr}\right) \qquad \textit{Equation 5-42}$$

5.2.5 Service Load Deflection

The deflection limit is recommended to prevent detrimental effects on nonstructural components and residual deformations (ACI Committee 551, 2015). As defined in Section

11.8.1.1 of ACI 318-15, the maximum service load deflection should be checked to not exceed the maximum deflection of the wall:

$$\Delta_s = \frac{l_c}{150} \qquad \textit{Equation 5-43}$$

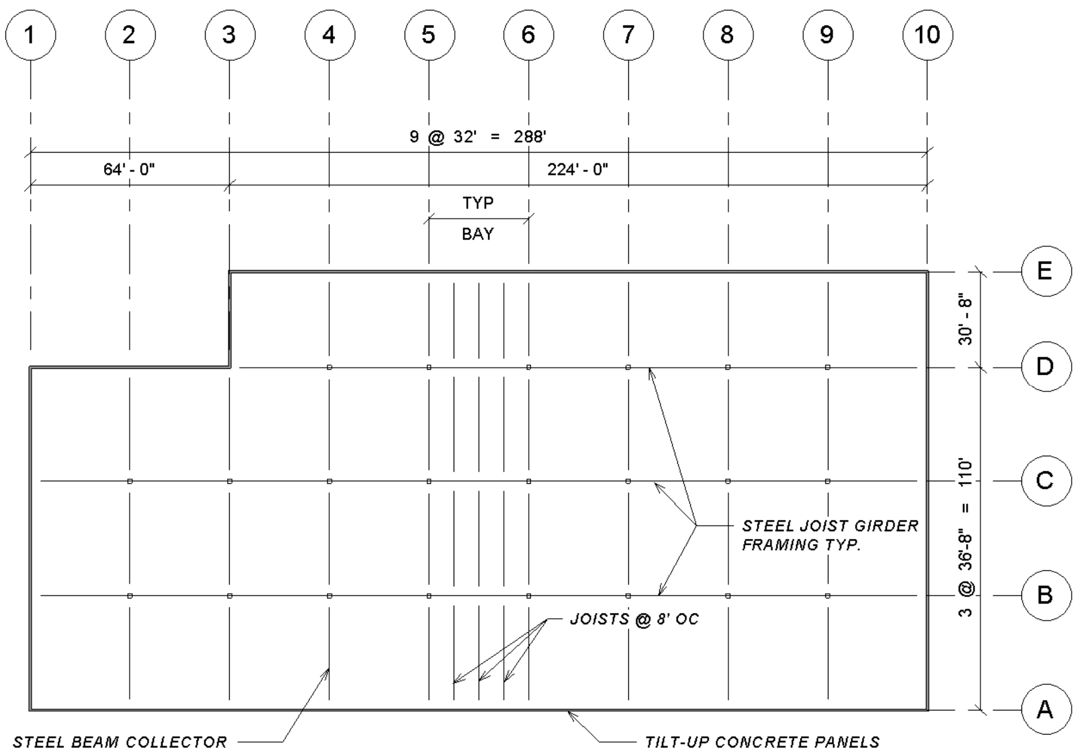
To simplify the design of slender walls that have applied unfactored moments, M_a, greater than the cracking moment, M_{cr}, the service load deflection can be interpolated between the Δ_{cr}, calculated out-of-plane deflection at midheight of the wall corresponding to cracking moment, and the Δ_n, out-of-plane deflection at midheight of the wall corresponding to the nominal flexural strength moment:

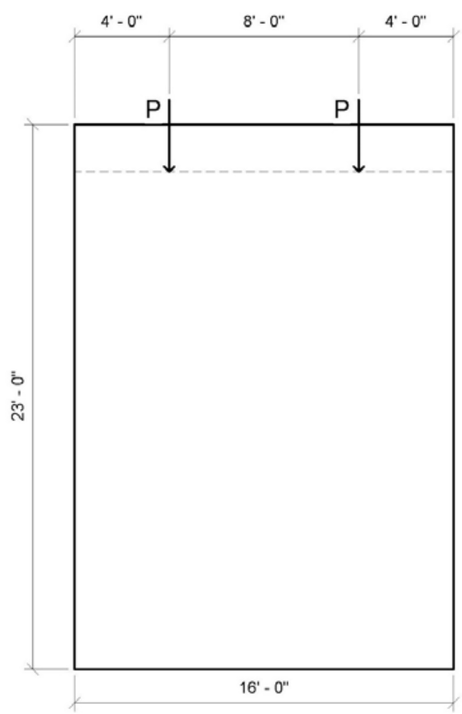
$$\Delta_{cr} = \frac{5M_{cr}l_c^2}{48E_cI_g} \qquad \textit{Equation 5-44}$$

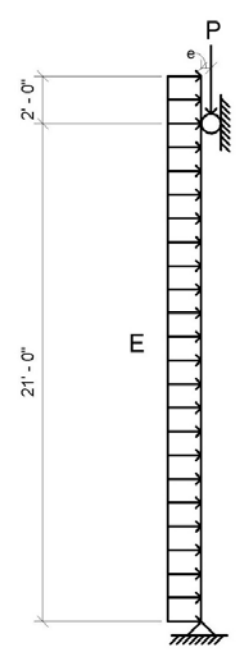
$$\Delta_n = \frac{5M_nl_c^2}{(0.75)48E_cI_{cr}} \qquad \textit{Equation 5-45}$$

Chapter 6 - Solid Single-Wythe Tilt-Up Wall Panel Design Example

6.1 Panel Design Properties and Applied Loads







Roof loads from each joist:

- D = 14 psf (2.1 kips)
- L_r = 20 psf (2.9 kips)
- e_{cc} = 2.75 in.

Wall:

- γ_c = 150 pcf
- f_c' = 4,000 psi
- Thickness = 5.5 in.
- Unbraced length = 21 ft.
- Parapet = 2 ft.
- Width = 16 ft.
- E_c = 3605 ksi

Seismic coefficients:

- S_{DS} = 1g
- ρ = 1.0
- I_e = 1.0

Reinforcement:

- d = 2.75 in. (centered in the wall thickness)
- f_y =60,000 psi
- E_s = 29,000 ksi
- $A_{s,vert}$ = 12 No. 5 rebars at 16 in. O.C.
  = 12(0.31 in^2) = 3.72 in^2
- $A_{s,horiz}$ = 19 No. 4 rebars at 14 in. O.C.
  = 19(0.20 in^2) = 3.80 in^2

$$D_{wall} = concrete\ density \times wall\ thickness = 150\ pcf \times \left(\frac{5.5}{12}\right) ft = 68.75\ psf$$

$$= 68.75\ psf \times \left(\frac{21\ ft}{2} + 2ft\right) \times 16ft \times \frac{1\ kip}{1000\ lbs} = 13.75\ kips$$

$$Q_E = F_p = 0.4 S_{DS} I_e D_{wall} = 0.4 \times 1.0 \times 1.0 \times 68.75\ psf \geq 0.10 D_{wall}$$

$$= 27.5\ psf \geq 6.88\ psf \qquad OK$$

$$D_{roof} = 14\ psf \times \frac{36.67\ ft}{2} \times 8ft \times 2 = 4107\ lbs = 4.11\ kips$$

$$L_{roof} = 20\ psf \times \frac{36.67\ ft}{2} \times 8ft \times 2 = 5867\ lbs = 5.87\ kips$$

$$D = D_{wall} + D_{roof} = 13.75\ kips + 4.11\ kips = 17.86\ kips$$

6.2 Load Case

The governing load cases for:

- Gravity loads in ultimate design

Load case 3: $1.2D + 1.6L_r$ *Equation 6-1*

- Lateral loads in ultimate design

  Load case 5: $(1.2 + 0.2S_{DS})D + \rho Q_E + L + 0.2S$ *Equation 6-2*

  Load case 7: $(0.9 - 0.2S_{DS})D + \rho Q_E$ *Equation 6-3*

- Gravity loads in service design

  load case 3: $D + L_r$ *Equation 6-4*

- Lateral loads in service design

  load case 5: $(1.0 + 0.14S_{DS})D + 0.7\rho Q_E$ *Equation 6-5*

  load case 8: $(0.6 - 0.14S_{DS})D + 0.7\rho Q_E$ *Equation 6-6*

Factored axial force without panel weight:

$$P_{ua} = 1.2D + 1.6L_r \qquad \textit{Equation 6-7}$$

$$P_{ua} = 1.2(4.11\ kips) + 1.6(5.87\ kips) = 14.31\ kips$$

Factored axial force with panel weight:

$$P_{um} = P_{ua} + 1.2D_{wall} \qquad \textit{Equation 6-8}$$

$$P_{um} = 14.31\ kips + 1.2(13.75\ kips) = 30.81\ kips$$

Factored out-of-plane load:

$$w_u = \rho Q_E \qquad \textit{Equation 6-9}$$

$$w_u = 1.0 \times 27.5\ psf \times 16\ ft = 440\ plf = 0.440\ klf$$

6.2.1 Vertical Stress

Check vertical stress at the midheight section of the panel:

$$\frac{P_{um}}{A_g} \le 0.06f_c' \qquad \textit{Equation 6-10}$$

$$\frac{30.81\ kips}{(16\ \times 12)in\ \times 5.5in} \leq 0.06(4\ ksi)$$

$$0.029\ ksi \leq 0.24\ ksi \qquad OK$$

Check if the effective reinforcement area equation needs to be calculated:

$$\frac{P_{um}}{A_g} \leq 0.10f_c' \qquad \textit{Equation 6-11}$$

$$\frac{30.81\ kips}{(16\ \times 12)in\ \times 5.5in} \leq 0.10(4\ ksi)$$

$0.029\ ksi \leq 0.40\ ksi$ calculate the effective reinforcement area

6.2.2 Design Moment Strength

$$A_{se} = A_s + \frac{P_{um}}{f_y}\frac{h}{2d} \qquad \textit{Equation 6-12}$$

$$A_{se} = 3.72\ in.^2 + \frac{30.81\ kips}{60\ ksi}\frac{5.5\ in.}{2(2.75\ in.)} = 4.23\ in.^2$$

$$a = \frac{A_{se}f_y}{0.85f_c'b} \qquad \textit{Equation 6-13}$$

$$a = \frac{(4.23\ in.^2)(60\ ksi)}{0.85(4\ ksi)(16 \times 12)in.} = 0.39\ in$$

$$c = \frac{a}{\beta_1} \qquad \textit{Equation 6-14}$$

$$c = \frac{0.39\ in.}{0.85} = 0.46\ in.$$

$$\varepsilon_t = 0.003\ \times \frac{d-c}{c} \qquad \textit{Equation 6-15}$$

$$\varepsilon_t = 0.003\ \times \frac{2.75\ in. - 0.46\ in.}{0.46\ in.} = 0.015\ > 0.005$$

$$\therefore tension - controlled\ \phi = 0.90$$

$$\phi M_n = \phi A_{se} f_y \left(d - \frac{a}{2}\right) \qquad \textit{Equation 6-16}$$

$$\phi M_n = 0.90(4.23\ in.^2)(60\ ksi)\left(2.75\ in. - \frac{0.39\ in.}{2}\right) = 584.21\ k - in.$$

6.2.3 Cracking Moment

$$f_r = 7.5\lambda\sqrt{f_c'} \qquad \textit{Equation 6-17}$$

$$f_r = 7.5(1.0)\sqrt{4{,}000\ psi} = 474\ psi = 0.474\ ksi$$

$$I_g = \frac{1}{12} bh^3 \qquad \textit{Equation 6-18}$$

$$I_g = \frac{1}{12}(16 \times 12)in.(5.5\ in.)^3 = 2662\ in.^4$$

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad \textit{Equation 6-19}$$

$$M_{cr} = \frac{(0.474\ ksi)(2662\ in.^4)}{2.75\ in.} = 459.16\ k - in.$$

$$M_{cr} = 459.16\ k - in. < \phi M_n = 584.21\ k - in. \qquad OK$$

6.2.4 Minimum Vertical and Horizontal Reinforcement

$$\rho_t = \frac{A_{s,horizontal}}{A_g} \geq 0.0025 \qquad \textit{Equation 6-20}$$

$$\rho_t = \frac{3.80\ in.^2}{(23 \times 12)in(5.5\ in)} = 0.0025 \geq 0.0025 \qquad OK$$

$$\rho_l \geq 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_t - 0.0025) \qquad \textit{Equation 6-21}$$

$$or\ 0.0025\ whichever\ is\ greater$$

$$\rho_{l,min} = 0.0025 + 0.5\left(2.5 - \frac{23\,ft}{16\,ft}\right)(0.0025 - 0.0025) = 0.0025$$

$$\rho_{l,min} = 0.0025$$

$$\rho_l = \frac{A_{s,vertical}}{A_g} \geq 0.0025$$ *Equation 6-22*

$$\rho_l = \frac{3.72\ in.^2}{(16 \times 12)in.\,(5.5\ in.)} = 0.0035 \geq 0.0025 \quad OK$$

Check minimum flexural reinforcement:

$$A_{s,min} = \frac{3\sqrt{f_c'}}{f_y} b_w d \ or\ \frac{200}{f_y} b_w d$$ *Equation 6-23*

$$A_{s,min} = \frac{3\sqrt{4{,}000\ psi}}{60{,}000\ psi} 12\ in.\,(2.75\ in.) = 0.10\ in.^2/ft$$

$$or\ \frac{200}{60{,}000\ psi} 12\ in.\,(2.75\ in.) = 0.11\ in.^2/ft$$

$$A_{s,min} = 0.11\ in.^2/ft\ <\ A_s = 0.23\ in.^2/ft \quad OK$$

Check spacing:

$$s_{max} = 3h\ or\ 18\ in. \quad whichever\ is\ smaller$$ *Equation 6-24*

$$s_{max} = 3(5.5\ in.) = 16.5\ in.\,or\ 18\ in.$$

$$s_{max} = 16.5\ in. > s_{vert} = 16\ in. \quad OK$$

$$s_{max} = 16.5\ in. > s_{horiz} = 14\ in. \quad OK$$

6.2.5 Applied Ultimate Moment

$$I_{cr} = \frac{E_s}{E_c}(A_{se})(d - c)^2 + \frac{l_w c^3}{3}$$ *Equation 6-25*

$$I_{cr} = \frac{29{,}000\ ksi}{3605\ ksi}(4.23\ in.^2)(2.75in - 0.46in)^2 + \frac{(16 \times 12)in\ (0.46in.)^3}{3}$$

$I_{cr} = 185.1\ in.^4$

$$M_{ua} = \frac{w_u l_c^2}{8} + \frac{P_{ua} e_{cc}}{2}$$ *Equation 6-26*

$$M_{ua} = \frac{0.44\ klf\ (21\ ft)^2}{8} + \frac{14.31\ kips\ \times (2.75/12)\ ft}{2}$$

$$= 25.90\ k - ft\ or\ 310.74\ k - in.$$

$$M_u = \frac{M_{ua}}{\left(1 - \frac{5P_{um} l_c^2}{(0.75)48 E_c I_{cr}}\right)}$$ *Equation 6-27*

$$M_u = \frac{310.74\ k - in}{\left(1 - \frac{5(30.81\ kips)(21 \times 12\ in)^2}{(0.75)48(3605\ ksi)185.1\ in.^4}\right)} = 524.32\ k - in.$$

$$M_u = 524.32\ \mathrm{k-in}\ < \phi M_n = 584.21\ k - in.\ OK$$ *Equation 6-28*

$$\Delta_u = \frac{5 M_u l_c^2}{(0.75)48 E_c I_{cr}}$$ *Equation 6-29*

$$\Delta_u = \frac{5(524.32\ k - in)(21 \times 12\ in)^2}{(0.75)48(3605\ ksi)185.1\ in.^4} = 6.93\ in.$$

6.2.6 Service Load Deflection

Unfactored axial force without panel weight:

$$P_a = D + L_r$$ *Equation 6-30*

$$P_a = 4.11\ kips + 5.87\ kips = 9.97\ kips$$

Unfactored axial force with panel weight:

$$P_{sm} = P_a + D_{wall}$$ *Equation 6-31*

$$P_{sm} = 9.97\ kips + 13.75\ kips = 23.72\ kips$$

Factored out-of-plane load:

$$w_s = 0.7\rho Q_E$$ *Equation 6-32*

$$w_s = 0.7 \times 1.0 \times 27.50\, psf \times 16\, ft = 308\, plf = 0.308\, klf$$

$$A_{se} = A_s + \frac{P_{sm}}{f_y}\frac{h}{2d}$$ *Equation 6-33*

$$A_{se} = 3.72\, in.^2 + \frac{23.72\, kips}{60\, ksi}\frac{5.5\, in.}{2(2.75\, in.)} = 4.12\, in.^2$$

$$a = \frac{A_{se}f_y}{0.85f_c'b}$$ *Equation 6-34*

$$a = \frac{(4.12\ \ in.^2)(60\, ksi)}{0.85(4\, ksi)(16 \times 12)in.} = 0.38\, in$$

$$c = \frac{a}{\beta_1}$$ *Equation 6-35*

$$c = \frac{0.38\, in.}{0.85} = 0.45\, in.$$

$$\varepsilon_t = 0.003\ \times \frac{d - c}{c}$$ *Equation 6-36*

$$\varepsilon_t = 0.003\ \times \frac{2.75\, in. - 0.45\, in.}{0.45\, in.} = 0.0155\ > 0.005$$

$$\therefore tension - controlled\ \phi = 0.90$$

$$\phi M_n = \phi A_{se}f_y\left(d - \frac{a}{2}\right)$$ *Equation 6-37*

$$\phi M_n = 0.90(4.12\, in.^2)(60\, ksi)\left(2.75\, in. - \frac{0.38\, in.}{2}\right) = 569.11\, k - in.$$

$$M_n = 632.34\, k - in$$

$$M_{cr} = 459.16\, k - in. < \ \phi M_n = 569.11\, k - in. \qquad OK$$

$$I_{cr} = \frac{E_s}{E_c}(A_{se})(d - c)^2 + \frac{l_w c^3}{3}$$ *Equation 6-38*

$$I_{cr} = \frac{29{,}000\ ksi}{3605\ ksi}(4.12\ in.^2)(2.75\ in. - 0.45\ in)^2$$

$$+ \frac{(16 \times 12)in\ (0.45in.)^3}{3}$$

$$I_{cr} = 181.5\ in.^4$$

$$\Delta_{allow} = \frac{l_c}{150}$$ *Equation 6-39*

$$\Delta_{allow} = \frac{21 \times 12\ in.}{150} = 1.68\ in.$$

$$\Delta_{cr} = \frac{5M_{cr}l_c^2}{48E_cI_g}$$ *Equation 6-40*

$$\Delta_{cr} = \frac{5(459.16\ k - in)(21 \times 12\ in)^2}{48(3605\ ksi)(2662\ in^4)} = 0.32\ in.$$

$$\Delta_n = \frac{5M_nl_c^2}{(0.75)48E_cI_{cr}}$$ *Equation 6-41*

$$\Delta_n = \frac{5(632.34\ k - in)(21 \times 12\ in)^2}{0.75(48)(3605\ ksi)(181.5\ in^4)} = 6.39\ in.$$

$$M_{sa} = \frac{w_sl_c^2}{8} + \frac{P_ae_{cc}}{2}$$ *Equation 6-42*

$$M_{sa} = \frac{0.308\ klf\ (21\ ft)^2}{8} + \frac{9.97\ kips\ \times (2.75/12)\ ft}{2}$$

$$= 18.12\ k - ft\ or\ 217.46\ k - in.$$

For the first iteration, assume $M_a \leq 2/3\ M_{cr}$:

$$\Delta_s = \frac{M_a}{M_{cr}}\Delta_{cr}$$ *Equation 6-43*

$$\Delta_s = \frac{217.46\ k - in}{459.16\ k - in}0.32\ in = 0.150\ in.$$

$$M_a = M_{sa} + P_{sm}\Delta_s$$ *Equation 6-44*

$$M_a = 217.46\ k - in + (23.72\ kips)(0.150\ in.) = 221.01\ k - in$$

$$M_a = 221.01\ k - in\ < \frac{2}{3} M_{cr} = 306.11\ k - in$$ *Equation 6-45*

For the second iteration:

$$\Delta_s = \frac{M_a}{M_{cr}} \Delta_{cr}$$ *Equation 6-46*

$$\Delta_s = \frac{221.01\ k - in}{459.16\ k - in} 0.32\ in = 0.152\ in.$$

$$M_a = M_{sa} + P_{sm} \Delta_s$$ *Equation 6-47*

$$M_a = 217.46\ k - in + (23.72\ kips)(0.152\ in.) = 221.07\ k - in$$

$$M_a = 221.07\ k - in\ < \frac{2}{3} M_{cr} = 306.11\ k - in$$ *Equation 6-48*

For the third and last iteration:

$$\Delta_s = \frac{M_a}{M_{cr}} \Delta_{cr}$$ *Equation 6-49*

$$\Delta_s = \frac{221.07\ k - in}{459.16\ k - in} 0.32\ in = 0.152\ in.$$

$$M_a = M_{sa} + P_{sm} \Delta_s$$ *Equation 6-50*

$$M_a = 217.46\ k - in + (23.72\ kips)(0.152\ in.) = 221.07\ k - in$$

$$M_a = 221.07\ k - in\ < \frac{2}{3} M_{cr} = 306.11\ k - in$$ *Equation 6-51*

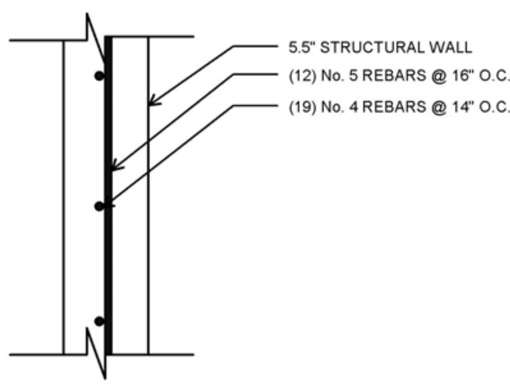
6.3 Summary

The vertical stress in the single wythe tilt-up panel is 29 psi and the design moment strength, ϕM_n, is 584 k-in. The maximum ultimate moment due to applied loads is 524 k-in which is less than 90% of the design moment strength. The service load deflection is 0.152

inches which is less than the allowable deflection of 1.68 inches. The cracking moment, M_{cr}, for this wall panel is 459 k-in, which is less than the maximum ultimate moment, thus the wall has cracked. A tabulated summary is provided in *Table 6-1*.

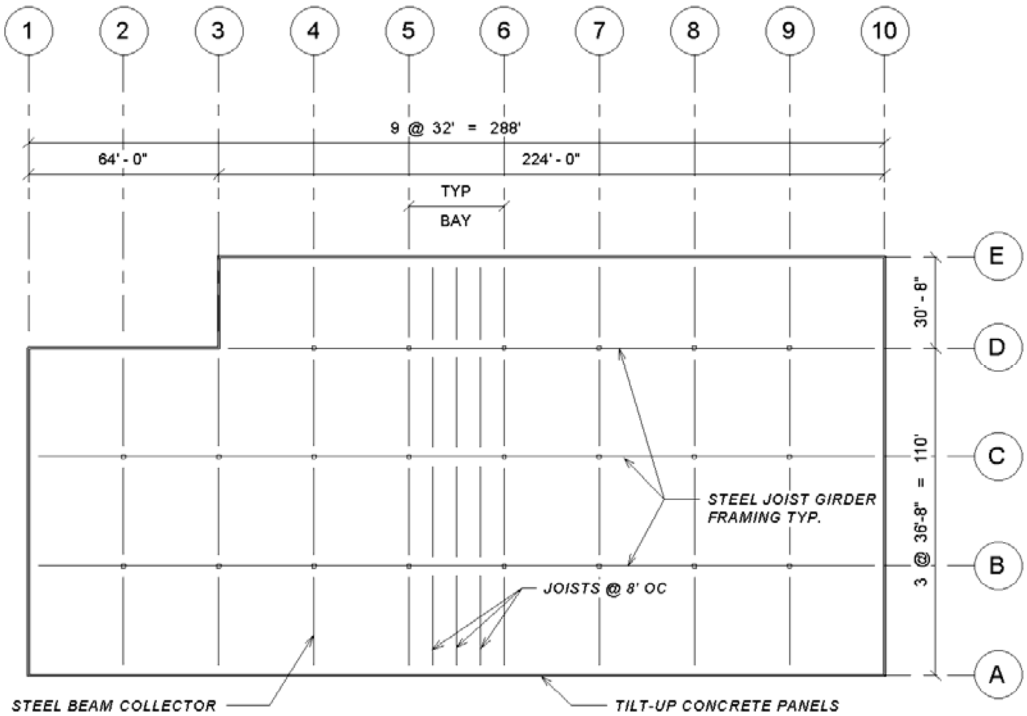
Table 6-1. Summary of Solid Single Wythe Tilt-Up Wall Panel Design.

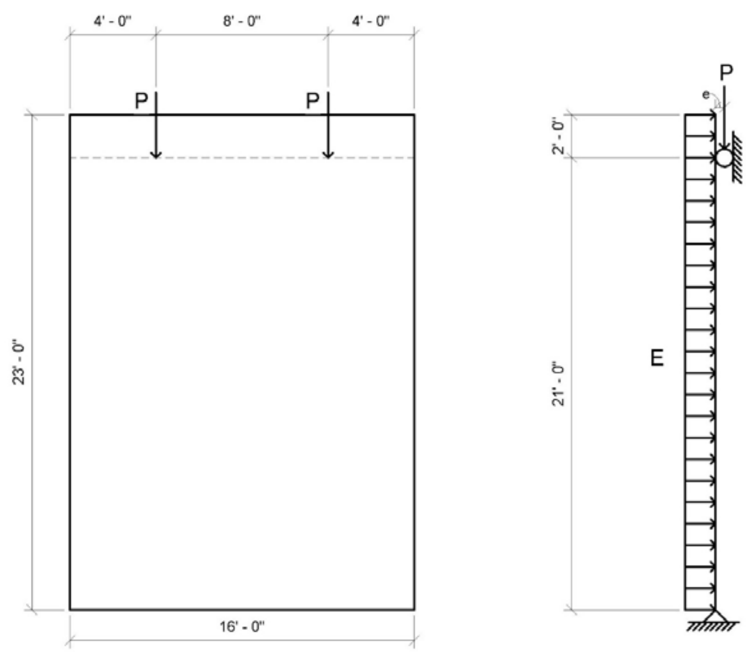
	Ultimate Design	***Service Design***
P (k)	14.31	9.97
P_m (k)	30.81	23.72
W (klf)	0.44	0.308
M_{cr} (k-in)	459.16	459.16
ϕM_n (k-in)	584.21	569.11
$A_{s\ vert}$ (in^2)	3.72	3.72
A_{se} (in^2)	4.23	4.12
$A_{s\ horiz}$ (in^2)	3.80	3.80
I_{cr} (in^4)	185.1	181.5
M (k-in)	524.32	217.46
Δ (in)	6.93	0.152



Chapter 7 - Non-Composite Sandwich Tilt-Up Wall Panel Design Example

7.1 Panel Design Properties and Applied Loads





Roof loads from each joist:

- D = 14 psf (2.05 kips)
- L_r = 20 psf (2.93 kips)
- e_{cc} = 2.75 in.

Seismic coefficients:

- S_{DS} = 1g
- ρ = 1.0
- I_e = 1.0

Wall:

- γ_c = 150 pcf
- f_c' = 4,000 psi
- Structural wall thickness = 5.5 in.
- Insulation: 2.5 in. (0.75 psf per ½")
- Non-structural wall thickness = 3.5 in.
- Unbraced length = 21 ft.
- Parapet = 2 ft.
- Width = 16 ft.
- E_c = 3605 ksi

Reinforcement (non-structural wall):

- d = 1.75 in.
- f_y =60,000 psi
- E_s = 29,000 ksi
- $A_{s,vert}$ = 20 No. 5 rebars at 9" O.C. = 20(0.31 in^2) = 6.2 in^2
- $A_{s,horiz}$ = 19 No. 4 rebars at 14" O.C. = 19(0.20 in^2) = 3.8 in^2

Reinforcement (structural wall):

- d = 2.75 in.
- f_y =60,000 psi
- E_s = 29,000 ksi
- $A_{s,vert}$ = 20 No. 5 rebars at 9" O.C. = 20(0.31 in^2) = 6.2 in^2
- $A_{s,horiz}$ = 19 No. 4 rebars at 14" O.C. = 19(0.20 in^2) = 3.8 in^2

$$D_{wall} = concrete\ density\ \times\ wall\ thickness + insulation$$

$$D_{wall} = 150\, pcf\ \times \left(\frac{3.5\, in + 5.5\, in}{12}\right) ft + \frac{0.75 psf}{0.5\, in}(2.5\, in.) = 116.25\, psf$$

$$D_{wall} = 116.25\, psf \times \left(\frac{21\, ft}{2} + 2ft\right) \times 16ft \times \frac{1\, kip}{1000\, lbs} = 23.25\, kips$$

$$Q_E = F_p = 0.4 S_{DS} I_e D_{wall} = 0.4 \times 1.0 \times 1.0 \times 116.25\, psf \geq 0.10 D_{wall}$$

$$Q_E = 46.50\, psf \geq 11.63\, psf \qquad OK$$

$$D_{roof} = 14\, psf\ \times \frac{36.67\, ft}{2} \times 8ft \times 2 = 4107\, lbs = 4.11\, kips$$

$$L_{roof} = 20\, psf\ \times \frac{36.67\, ft}{2} \times 8ft \times 2 = 5867\, lbs = 5.87\, kips$$

$$D = D_{wall} + D_{roof} = 23.25\, kips + 4.11\, kips = 27.36\, kips$$

7.2 Load Case

The governing load cases for:

- Gravity loads in ultimate design

Load case 3: $1.2D + 1.6L_r$ *Equation 7-1*

- Lateral loads in ultimate design

Load case 5: $(1.2 + 0.2S_{DS})D + \rho Q_E + L + 0.2S$ *Equation 7-2*

Load case 7: $(0.9 - 0.2S_{DS})D + \rho Q_E$ *Equation 7-3*

- Gravity loads in service design

load case 3: $D + L_r$ *Equation 7-4*

- Lateral loads in service design

load case 5: $(1.0 + 0.14S_{DS})D + 0.7\rho Q_E$ *Equation 7-5*

load case 8: $(0.6 - 0.14S_{DS})D + 0.7\rho Q_E$ *Equation 7-6*

Factored axial force without panel weight:

$P_{ua} = 1.2D + 1.6L_r$ *Equation 7-7*

$P_{ua} = 1.2(4.11\ kips) + 1.6(5.87\ kips) = 14.31\ kips$

Factored axial force with panel weight:

$P_{um} = P_{ua} + 1.2D_{wall}$ *Equation 7-8*

$P_{um} = 14.31\ kips + 1.2(23.25\ kips) = 42.21\ kips$

Factored out-of-plane load:

$w_u = \rho Q_E$ *Equation 7-9*

$w_u = 1.0 \times 46.5\ psf \times 16\ ft = 744\ plf = 0.744\ klf$

7.2.1 Vertical Stress

Check vertical stress at the midheight section of the panel:

$$\frac{P_{um}}{A_g} \leq 0.06 f_c' \qquad \textit{Equation 7-10}$$

$$\frac{42.21\ kips}{(16\ \times 12)in\ \times 5.5in} \leq 0.06(4\ ksi)$$

$$0.040\ ksi \leq 0.24\ ksi \qquad OK$$

Check if the effective reinforcement area equation needs to be calculated:

$$\frac{P_{um}}{A_g} \leq 0.10 f_c' \qquad \textit{Equation 7-11}$$

$$\frac{42.21\ kips}{(16\ \times 12)in\ \times 5.5in} \leq 0.10(4\ ksi)$$

$0.040\ ksi \leq 0.40\ ksi$ calculate the effective reinforcement area

7.2.2 Design Moment Strength

$$A_{se} = A_s + \frac{P_{um}}{f_y}\frac{h}{2d} \qquad \textit{Equation 7-12}$$

$$A_{se} = 6.2\ in.^2 + \frac{42.21\ kips}{60\ ksi}\frac{5.5\ in.}{2(2.75\ in.)} = 6.90\ in.^2$$

$$a = \frac{A_{se} f_y}{0.85 f_c' b} \qquad \textit{Equation 7-13}$$

$$a = \frac{(6.90\ in.^2)(60\ ksi)}{0.85(4\ ksi)(16 \times 12)in.} = 0.63\ in$$

$$c = \frac{a}{\beta_1} \qquad \textit{Equation 7-14}$$

$$c = \frac{0.63\ in.}{0.85} = 0.75\ in.$$

$$\varepsilon_t = 0.003 \times \frac{d - c}{c}$$ *Equation 7-15*

$$\varepsilon_t = 0.003 \times \frac{2.75\ in. - 0.75\ in.}{0.75\ in.} = 0.0081 > 0.005$$

$$\therefore tension - controlled\ \phi = 0.90$$

$$\phi M_n = \phi A_{se} f_y \left(d - \frac{a}{2}\right)$$ *Equation 7-16*

$$\phi M_n = 0.90(6.90\ in.^2)(60\ ksi)\left(2.75\ in. - \frac{0.63\ in.}{2}\right) = 906.91\ k - in.$$

7.2.3 Cracking Moment

$$f_r = 7.5\lambda\sqrt{f_c'}$$ *Equation 7-17*

$$f_r = 7.5(1.0)\sqrt{4{,}000\ psi} = 474\ psi = 0.474\ ksi$$

$$I_g = \frac{1}{12} bh^3$$ *Equation 7-18*

$$I_g = \frac{1}{12}(16 \times 12) in. (5.5\ in.)^3 = 2662\ in.^4$$

$$M_{cr} = \frac{f_r I_g}{y_t}$$ *Equation 7-19*

$$M_{cr} = \frac{(0.474\ ksi)(2662\ in.^4)}{2.75\ in.} = 459.16\ k - in.$$

$$M_{cr} = 459.16\ k - in. < \phi M_n = 906.91\ k - in. \quad OK$$

7.2.4 Minimum Vertical and Horizontal Reinforcement

$$\rho_t = \frac{A_{s,horizontal}}{A_g} \geq 0.0025$$ *Equation 7-20*

$$\rho_t = \frac{3.80\ in.^2}{(23 \times 12)in(5.5\ in)} = 0.0025\ \geq 0.0025 \quad OK$$

$$\rho_l \geq 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_t - 0.0025)$$ *Equation 7-21*

$or\ 0.0025\ whichever\ is\ greater$

$$\rho_{l,min} = 0.0025 + 0.5\left(2.5 - \frac{23\ ft}{16\ ft}\right)(0.0025 - 0.0025) =$$

$0.0025\ or\ 0.0025$

$$\rho_{l,min} = 0.0025$$

$$\rho_l = \frac{A_{s,vertical}}{A_g} \geq 0.0025$$ *Equation 7-22*

$$\rho_l = \frac{6.2\ in.^2}{(16 \times 12)in.(5.5\ in.)} = 0.0059 \geq 0.0025 \quad OK$$

Check minimum flexural reinforcement:

$$A_{s,min} = \frac{3\sqrt{f_c'}}{f_y} b_w d\ or\ \frac{200}{f_y} b_w d$$ *Equation 7-23*

$$A_{s,min} = \frac{3\sqrt{4{,}000\ psi}}{60{,}000\ psi} 12\ in.(2.75\ in.) = 0.10\ in.^2/ft$$

$$or\ \frac{200}{60{,}000\ psi} 12\ in.(2.75\ in.) = 0.11\ in.^2/ft$$

$$A_{s,min} = 0.11\ in.^2/ft\ <\ A_s = 0.39\ in.^2/ft \quad OK$$

Check spacing:

$$s_{max} = 3h\ or\ 18\ in. \quad whichever\ is\ smaller$$ *Equation 7-24*

$$s_{max} = 3(5.5\ in.) = 16.5\ in.\ or\ 18\ in.$$

$$s_{max\ vert} = 16.5\ in. > s = 9\ in. \quad OK$$

$s_{max\ horiz} = 16.5\ in. > s = 14\ in. \quad OK$

7.2.5 Applied Ultimate Moment

$$I_{cr} = \frac{E_s}{E_c}(A_{se})(d-c)^2 + \frac{l_w c^3}{3} \qquad \textit{Equation 7-25}$$

$$I_{cr} = \frac{29{,}000\ ksi}{3605\ ksi}(6.90\ in.^2)(2.75in - 0.75in)^2 + \frac{(16 \times 12)in\ (0.75in.)^3}{3}$$

$$I_{cr} = 249.5\ in.^4$$

$$M_{ua} = \frac{w_u l_c^2}{8} + \frac{P_{ua} e_{cc}}{2} \qquad \textit{Equation 7-26}$$

$$M_{ua} = \frac{0.744\ klf\ (21\ ft)^2}{8} + \frac{14.31\ kips\ \times (2.75/12)\ ft}{2}$$

$$= 42.65\ k - ft\ or\ 511.84\ k - in.$$

$$M_u = \frac{M_{ua}}{\left(1 - \frac{5P_{um} l_c^2}{(0.75)48 E_c I_{cr}}\right)} \qquad \textit{Equation 7-27}$$

$$M_u = \frac{511.84\ k - in}{\left(1 - \frac{5(42.21\ kips)(21 \times 12\ in)^2}{(0.75)48(3605\ ksi)249.5\ in.^4}\right)} = 873.28\ k - in.$$

$$M_u = 873.28\ \mathrm{k - in} < \phi M_n = 906.91\ k - in. \quad OK \qquad \textit{Equation 7-28}$$

$$\Delta_u = \frac{5 M_u l_c^2}{(0.75)48 E_c I_{cr}} \qquad \textit{Equation 7-29}$$

$$\Delta_u = \frac{5(873.28\ k - in)(21 \times 12\ in)^2}{(0.75)48(3605\ ksi)249.5\ in.^4} = 8.56\ in.$$

7.2.6 Service Load Deflection

Unfactored axial force without panel weight:

$$P_a = D + L_r \qquad \textit{Equation 7-30}$$

$P_a = 4.11\ kips + 5.87\ kips = 9.97\ kips$

Unfactored axial force with panel weight:

$$P_{sm} = P_a + D_{wall} \qquad \textit{Equation 7-31}$$

$$P_{sm} = 9.97\ kips + 23.25\ kips = 33.22\ kips$$

Factored out-of-plane load:

$$w_s = 0.7\rho Q_E \qquad \textit{Equation 7-32}$$

$$w_s = 0.7 \times 1.0 \times 46.50\ psf \times 16\ ft = 521\ plf = 0.521\ klf$$

$$A_{se} = A_s + \frac{P_{sm}}{f_y}\frac{h}{2d} \qquad \textit{Equation 7-33}$$

$$A_{se} = 6.2\ in.^2 + \frac{33.22\ kips}{60\ ksi}\frac{5.5\ in.}{2(2.75\ in.)} = 6.75\ in.^2$$

$$a = \frac{A_{se}f_y}{0.85f_c'b} \qquad \textit{Equation 7-34}$$

$$a = \frac{(6.75\ in.^2)(60\ ksi)}{0.85(4\ ksi)(16 \times 12)in.} = 0.62\ in$$

$$c = \frac{a}{\beta_1} \qquad \textit{Equation 7-35}$$

$$c = \frac{0.62\ in.}{0.85} = 0.73\ in.$$

$$\varepsilon_t = 0.003 \times \frac{d - c}{c} \qquad \textit{Equation 7-36}$$

$$\varepsilon_t = 0.003 \times \frac{2.75\ in. - 0.73\ in.}{0.73\ in.} = 0.0083 > 0.005$$

$$\therefore tension - controlled\ \phi = 0.90$$

$$\phi M_n = \phi A_{se} f_y \left(d - \frac{a}{2}\right) \qquad \textit{Equation 7-37}$$

$$\phi M_n = 0.90(6.75\ in.^2)(60\ ksi)\left(2.75\ in. - \frac{0.62\ in.}{2}\right) = 889.73\ k - in.$$

$$M_n = 988.59\ k - in$$

$$M_{cr} = 459.16\ k - in. < \ \phi M_n = 889.73\ k - in. \quad OK$$

$$I_{cr} = \frac{E_s}{E_c}(A_{se})(d-c)^2 + \frac{l_w c^3}{3}$$ *Equation 7-38*

$$I_{cr} = \frac{29{,}000\ ksi}{3605\ ksi}(6.75\ in.^2)(2.75in. - 0.73\ in)^2 + \frac{(16 \times 12)in\ (0.73in.)^3}{3}$$

$$I_{cr} = 246.5\ in.^4$$

$$\Delta_{allow} = \frac{l_c}{150}$$ *Equation 7-39*

$$\Delta_{allow} = \frac{21 \times 12\ in.}{150} = 1.68\ in.$$

$$\Delta_{cr} = \frac{5 M_{cr} l_c^2}{48 E_c I_g}$$ *Equation 7-40*

$$\Delta_{cr} = \frac{5(459.16\ k - in)(21 \times 12\ in)^2}{48(3605\ ksi)(2662\ in^4)} = 0.32\ in.$$

$$\Delta_n = \frac{5 M_n l_c^2}{(0.75)48 E_c I_{cr}}$$ *Equation 7-41*

$$\Delta_n = \frac{5(988.59\ k - in)(21 \times 12\ in)^2}{0.75(48)(3605\ ksi)(246.5\ in^4)} = 7.36\ in.$$

$$M_{sa} = \frac{w_s l_c^2}{8} + \frac{P_a e_{cc}}{2}$$ *Equation 7-42*

$$M_{sa} = \frac{0.521\ klf\ (21\ ft)^2}{8} + \frac{9.97\ kips\ \times (2.75/12)\ ft}{2}$$

$$= 29.85\ k - ft\ or\ 358.22\ k - in.$$

For the first iteration, assume $M_a \leq 2/3\ M_{cr}$:

$$\Delta_s = \frac{M_a}{M_{cr}} \Delta_{cr} \qquad \textit{Equation 7-43}$$

$$\Delta_s = \frac{358.22\ k - in}{459.16\ k - in} 0.32\ in = 0.247\ in.$$

$$M_a = M_{sa} + P_{sm}\Delta_s \qquad \textit{Equation 7-44}$$

$$M_a = 358.22\ k - in + (33.22\ kips)(0.247\ in.) = 366.43\ k - in$$

$$M_a = 366.43\ k - in\ < \frac{2}{3} M_{cr} = 306.11\ k - in \qquad \textit{Equation 7-45}$$

For the second iteration:

$$\Delta_s = \frac{2}{3}\Delta_{cr} + \frac{(M_a - \frac{2}{3}M_{cr})}{(M_n - \frac{2}{3}M_{cr})}\left(\Delta_n - \frac{2}{3}\Delta_{cr}\right) \qquad \textit{Equation 7-46}$$

$$\Delta_s = \frac{2}{3}(0.32\ in.) + \frac{(366.43 - 306.11)k - in.}{(988.59 - 306.11)k - in.}\left(7.36 - \frac{2}{3}(0.32\ in.)\right)$$

$$\Delta_s = 0.843\ in.$$

$$M_a = M_{sa} + P_{sm}\Delta_s \qquad \textit{Equation 7-47}$$

$$M_a = 358.22\ k - in + (33.22\ kips)(0.843\ in.) = 386.22\ k - in$$

$$M_a = 386.22\ k - in > \frac{2}{3} M_{cr} = 306.11\ k - in \qquad \textit{Equation 7-48}$$

For the third iteration:

$$\Delta_s = \frac{2}{3}\Delta_{cr} + \frac{(M_a - \frac{2}{3}M_{cr})}{(M_n - \frac{2}{3}M_{cr})}\left(\Delta_n - \frac{2}{3}\Delta_{cr}\right) \qquad \textit{Equation 7-49}$$

$$\Delta_s = \frac{2}{3}(0.32\ in.) + \frac{(386.22 - 306.11)k - in.}{(988.59 - 306.11)k - in.}\left(7.36 - \frac{2}{3}(0.32\ in.)\right)$$

$$\Delta_s = 1.050\ in.$$

$$M_a = M_{sa} + P_{sm}\Delta_s \qquad \textit{Equation 7-50}$$

$$M_a = 358.22\ k - in + (33.22\ kips)(1.050\ in.) = 393.10\ k - in$$

$$M_a = 393.10\ k - in > \frac{2}{3}M_{cr} = 306.11\ k - in \qquad \textit{Equation 7-51}$$

For the fourth iteration:

$$\Delta_s = \frac{2}{3}\Delta_{cr} + \frac{(M_a - \frac{2}{3}M_{cr})}{(M_n - \frac{2}{3}M_{cr})}\left(\Delta_n - \frac{2}{3}\Delta_{cr}\right) \qquad \textit{Equation 7-52}$$

$$\Delta_s = \frac{2}{3}(0.32\ in.) + \frac{(393.10 - 306.11)k - in.}{(988.59 - 306.11)k - in.}\left(7.36 - \frac{2}{3}(0.32\ in.)\right)$$

$$\Delta_s = 1.122\ in.$$

$$M_a = M_{sa} + P_{sm}\Delta_s \qquad \textit{Equation 7-53}$$

$$M_a = 358.22\ k - in + (33.22\ kips)(1.122\ in.) = 395.50\ k - in$$

$$M_a = 395.50\ k - in > \frac{2}{3}M_{cr} = 306.11\ k - in \qquad \textit{Equation 7-54}$$

For the fifth iteration:

$$\Delta_s = \frac{2}{3}\Delta_{cr} + \frac{(M_a - \frac{2}{3}M_{cr})}{(M_n - \frac{2}{3}M_{cr})}\left(\Delta_n - \frac{2}{3}\Delta_{cr}\right) \qquad \textit{Equation 7-55}$$

$$\Delta_s = \frac{2}{3}(0.32\ in.) + \frac{(395.50 - 306.11)k - in.}{(988.59 - 306.11)k - in.}\left(7.36 - \frac{2}{3}(0.32\ in.)\right)$$

$$\Delta_s = 1.147\ in.$$

$$M_a = M_{sa} + P_{sm}\Delta_s \qquad \textit{Equation 7-56}$$

$$M_a = 358.22\ k - in + (33.22\ kips)(1.147\ in.) = 396.33\ k - in$$

$$M_a = 396.33\ k - in > \frac{2}{3}M_{cr} = 306.11\ k - in \qquad \textit{Equation 7-57}$$

For the sixth iteration:

$$\Delta_s = \frac{2}{3}\Delta_{cr} + \frac{(M_a - \frac{2}{3}M_{cr})}{(M_n - \frac{2}{3}M_{cr})}\left(\Delta_n - \frac{2}{3}\Delta_{cr}\right)$$ *Equation 7-58*

$$\Delta_s = \frac{2}{3}(0.32\ in.) + \frac{(396.33 - 306.11)k - in.}{(988.59 - 306.11)k - in.}\left(7.36 - \frac{2}{3}(0.32\ in.)\right)$$

$$\Delta_s = 1.156\ in.$$

$$M_a = M_{sa} + P_{sm}\Delta_s$$ *Equation 7-59*

$$M_a = 358.22\ k - in + (33.22\ kips)(1.156\ in.) = 396.62\ k - in$$

$$M_a = 396.62\ k - in > \frac{2}{3}M_{cr} = 306.11\ k - in$$ *Equation 7-60*

For the seventh iteration:

$$\Delta_s = \frac{2}{3}\Delta_{cr} + \frac{(M_a - \frac{2}{3}M_{cr})}{(M_n - \frac{2}{3}M_{cr})}\left(\Delta_n - \frac{2}{3}\Delta_{cr}\right)$$ *Equation 7-61*

$$\Delta_s = \frac{2}{3}(0.32\ in.) + \frac{(396.62 - 306.11)k - in.}{(988.59 - 306.11)k - in.}\left(7.36 - \frac{2}{3}(0.32\ in.)\right)$$

$$\Delta_s = 1.159\ in.$$

$$M_a = M_{sa} + P_{sm}\Delta_s$$ *Equation 7-62*

$$M_a = 358.22\ k - in + (33.22\ kips)(1.159\ in.) = 396.72\ k - in$$

$$M_a = 396.72\ k - in > \frac{2}{3}M_{cr} = 306.11\ k - in$$ *Equation 7-63*

For the eighth and last iteration:

$$\Delta_s = \frac{2}{3}\Delta_{cr} + \frac{(M_a - \frac{2}{3}M_{cr})}{(M_n - \frac{2}{3}M_{cr})}\left(\Delta_n - \frac{2}{3}\Delta_{cr}\right)$$ *Equation 7-64*

$$\Delta_s = \frac{2}{3}(0.32\ in.) + \frac{(396.72 - 306.11)k - in.}{(988.59 - 306.11)k - in.}\left(7.36 - \frac{2}{3}(0.32\ in.)\right)$$

$$\Delta_s = 1.159\ in.$$

$$M_a = M_{sa} + P_{sm}\Delta_s \qquad \textit{Equation 7-65}$$

$$M_a = 358.22\ k - in + (33.22\ kips)(1.159\ in.) = 396.72\ k - in$$

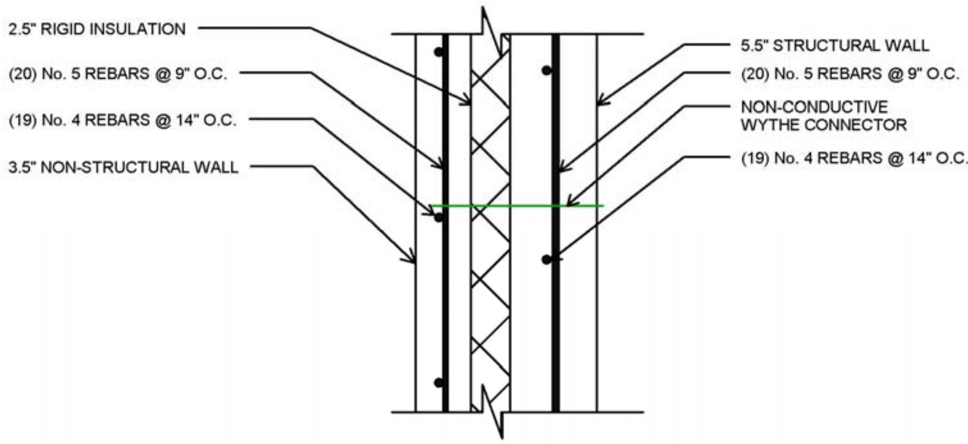
$$M_a = 396.72\ k - in > \frac{2}{3}M_{cr} = 306.11\ k - in \qquad \textit{Equation 7-66}$$

7.3 Summary

The vertical stress in the non-composite tilt-up panel is 40 psi and the design moment strength, ϕM_n, is 907 k-in. The maximum ultimate moment due to applied loads is 873 k-in which is about 96% of the design moment strength. The service load deflection is 1.16 inches which is less than the allowable deflection of 1.68 inches. The cracking moment, M_{cr}, for this wall panel is 459 k-in, which is less than the maximum ultimate moment, thus the wall has cracked. A tabulated summary is provided in *Table 7-1*.

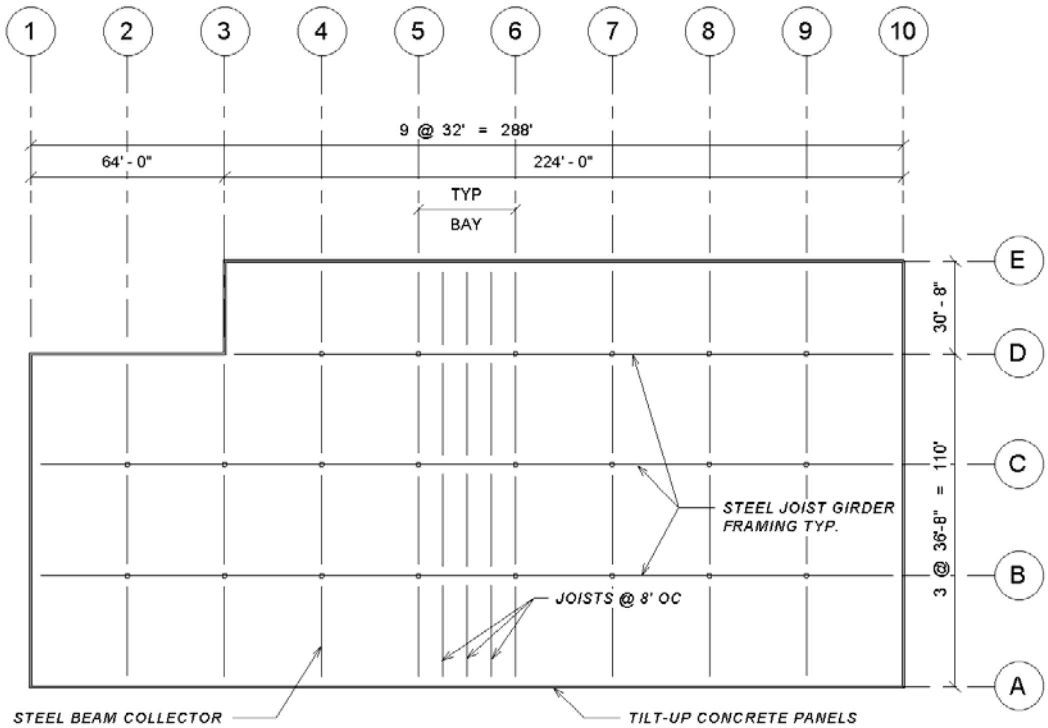
Table 7-1. Summary of Non-Composite Sandwich Tilt-Up Wall Panel Design.

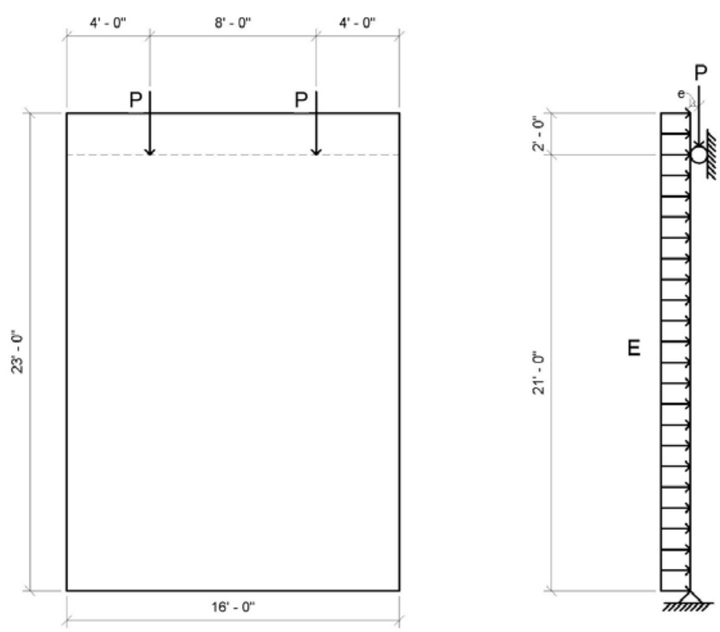
	Ultimate Design	***Service Design***
P (k)	14.31	9.97
P_m (k)	42.21	33.22
W (klf)	0.744	0.521
M_{cr} (k-in)	459.16	459.16
ϕM_n (k-in)	906.91	889.73
$A_{s\ vert}$ (in^2)	6.20	6.20
A_{se} (in^2)	6.90	6.75
$A_{s\ horiz}$ (in^2)	3.80	3.80
I_{cr} (in^4)	249.5	246.5
M (k-in)	873.28	358.22
Δ (in)	8.56	1.16


2.5" RIGID INSULATION
(20) No. 5 REBARS @ 9" O.C.
(19) No. 4 REBARS @ 14" O.C.
3.5" NON-STRUCTURAL WALL
5.5" STRUCTURAL WALL
(20) No. 5 REBARS @ 9" O.C.
NON-CONDUCTIVE
WYTHE CONNECTOR
(19) No. 4 REBARS @ 14" O.C.

Chapter 8 - Composite Sandwich Tilt-Up Wall Panel Design Example

8.1 Panel Design Properties and Applied Loads





Roof loads from each joist:

- D = 14 psf (2.05 kips)
- L_r = 20 psf (2.93 kips)
- e_{cc} = 5.75 in.

Seismic coefficients:

- S_{DS} = 1g
- ρ = 1.0
- I_e = 1.0

Wall:

- γ_c = 150 pcf
- f_c' = 4,000 psi
- Interior wall thickness = 5.5 in.
- Insulation: 2.5 in. (0.75 psf per ½")
- Exterior wall thickness = 3.5 in.
- Unbraced length = 21 ft.
- Parapet = 2 ft.
- Width = 16 ft.
- E_c = 3605 ksi

Reinforcement (exterior wall):

- d = 2.75 in.
- f_y =60,000 psi
- E_s = 29,000 ksi
- $A_{s,vert}$ = 18 No. 5 rebars at 10" O.C. = 18(0.31 in^2) = 5.58 in^2
- $A_{s,horiz}$ = 26 No. 4 rebars at 10" O.C. = 26(0.20 in^2) = 5.20in^2

Reinforcement (interior wall):

- d = 2.75 in.
- f_y =60,000 psi
- E_s = 29,000 ksi
- $A_{s,vert}$ = 19 No. 5 rebars at 9.5 in. O.C. = 19(0.31 in^2) = 5.89 in^2
- $A_{s,horiz}$ = 19 No. 4 rebars at 14 in. O.C. = 19(0.20 in^2) = 3.80 in^2

$$D_{wall} = concrete\ density\ \times\ wall\ thickness + insulation$$

$$D_{wall} = 150\,pcf\ \times\left(\frac{3.5\ in + 5.5\ in}{12}\right)ft + \frac{0.75psf}{0.5\ in}(2.5\ in.) = 116.25\ psf$$

$$D_{wall} = 116.25\ psf \times\left(\frac{21\ ft}{2} + 2ft\right)\times 16ft \times \frac{1\ kip}{1000\ lbs} = 23.25\ kips$$

$$Q_E = F_p = 0.4 S_{DS} I_e D_{wall} = 0.4 \times 1.0 \times 1.0 \times 116.25\ psf \geq 0.10 D_{wall}$$

$$Q_E = 46.50\ psf \geq 11.63\ psf \qquad OK$$

$$D_{roof} = 14\ psf\ \times \frac{36.67\ ft}{2} \times 8ft \times 2 = 4107\ lbs = 4.11\ kips$$

$$L_{roof} = 20\ psf\ \times \frac{36.67\ ft}{2} \times 8ft \times 2 = 5867\ lbs = 5.87\ kips$$

$$D = D_{wall} + D_{roof} = 23.25\ kips + 4.11\ kips = 27.36\ kips$$

8.2 Load Case

The governing load cases for:

- Gravity loads in ultimate design

Load case 3: $1.2D + 1.6L_r$ *Equation 8-1*

- Lateral loads in ultimate design

Load case 5: $(1.2 + 0.2S_{DS})D + \rho Q_E + L + 0.2S$ *Equation 8-2*

Load case 7: $(0.9 - 0.2S_{DS})D + \rho Q_E$ *Equation 8-3*

- Gravity loads in service design

load case 3: $D + L_r$ *Equation 8-4*

- Lateral loads in service design

load case 5: $(1.0 + 0.14S_{DS})D + 0.7\rho Q_E$ *Equation 8-5*

load case 8: $(0.6 - 0.14S_{DS})D + 0.7\rho Q_E$ *Equation 8-6*

Factored axial force without panel weight:

$$P_{ua} = 1.2D + 1.6L_r \quad \text{Equation 8-7}$$

$$P_{ua} = 1.2(4.11\ kips) + 1.6(5.87\ kips) = 14.31\ kips$$

Factored axial force with panel weight:

$$P_{um} = P_{ua} + 1.2D_{wall} \quad \text{Equation 8-8}$$

$$P_{um} = 14.31\ kips + 1.2(23.25\ kips) = 42.21\ kips$$

Factored out-of-plane load:

$$w_u = \rho Q_E \quad \text{Equation 8-9}$$

$$w_u = 1.0 \times 46.50\ psf \times 16\ ft = 744\ plf = 0.744\ klf$$

8.2.1 Vertical Stresses

Check vertical stress at the midheight section of the panel:

$$\frac{P_{um}}{A_g} \leq 0.06 f_c' \qquad \textit{Equation 8-10}$$

$$\frac{42.21\ kips}{(16\ \times 12)in\ \times (5.5in + 3.5in)} \leq 0.06(4\ ksi)$$

$$0.0244\ ksi \leq 0.24\ ksi \qquad OK$$

Check if the effective reinforcement area equation needs to be calculated:

$$\frac{P_{um}}{A_g} \leq 0.10 f_c' \qquad \textit{Equation 8-11}$$

$$\frac{42.21\ kips}{(16\ \times 12)in\ \times (5.5in + 3.5in)} \leq 0.10(4\ ksi)$$

$0.0244\ ksi \leq 0.40\ ksi$ calculate the effective reinforcement area

8.2.2 Design Moment Strength

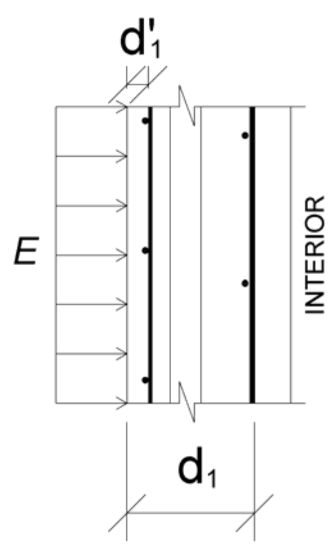


Figure 8-1. Exterior wall is in compression while interior wall is in tension.

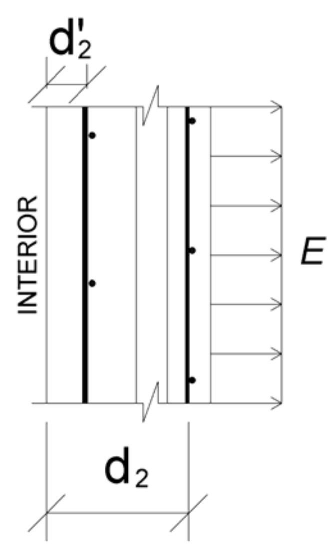


Figure 8-2. Exterior wall is in tension while interior wall is in compression

$$A_{se} = A_s + \frac{P_{um}}{f_y}\frac{h}{2d}$$ *Equation 8-12*

$$A_{se,int} = 5.89\ in.^2 + \frac{42.21\ kips}{60\ ksi}\frac{(5.5\ in + 3.5in)}{2(8.75\ in.)} = 6.25\ in.^2$$

$$A_{se,ext} = 5.58\ in.^2 + \frac{42.21\ kips}{60\ ksi}\frac{(5.5\ in + 3.5in)}{2(9.75\ in.)} = 5.90\ in.^2$$

$$a = \frac{A_{se}f_y}{0.85f_c'b}$$ *Equation 8-13*

$$a_{int} = \frac{(6.25\ \ in.^2)(60\ ksi)}{0.85(4\ ksi)(16 \times 12)in.} = 0.57\ in$$

$$a_{ext} = \frac{(5.90\ \ in.^2)(60\ ksi)}{0.85(4\ ksi)(16 \times 12)in.} = 0.54\ in$$

$$c = \frac{a}{\beta_1}$$ *Equation 8-14*

$$c_{int} = \frac{0.57\ in.}{0.85} = 0.68\ in.$$

$$c_{ext} = \frac{0.54\ in.}{0.85} = 0.64\ in.$$

$$\varepsilon_t = 0.003 \times \frac{d - c}{c}$$ *Equation 8-15*

$$\varepsilon_{t,int} = 0.003 \times \frac{8.75\ in. - 0.68\ in.}{0.68\ in.} = 0.0358 > 0.005$$

$$\therefore tension - controlled\ \phi = 0.90$$

$$\varepsilon_{t,ext} = 0.003 \times \frac{9.75\ in. - 0.64\ in.}{0.64\ in.} = 0.0428 > 0.005$$

$$\therefore tension - controlled\ \phi = 0.90$$

$$\phi M_n = \phi\left[A_{se}f_y\left(d - \frac{a}{2}\right) + A'_s f_y\left(d' - \frac{a}{2}\right)\right]$$ *Equation 8-16*

$$\phi M_{n,int} = 0.90\left((6.25\ in.^2)(60\ ksi)\left(8.75\ in. - \frac{0.57\ in.}{2}\right)\right.$$

$$\left. + (5.58\ in.^2)(60\ ksi)\left(1.75\ in. - \frac{0.57\ in.}{2}\right)\right)$$

$$= 3297.74\ k - in.$$

$$\phi M_{n,ext} = 0.90((5.90\ in.^2)(60\ ksi)\left(9.75\ in. - \frac{0.54\ in.}{2}\right)$$

$$+ (5.89\ in.^2)(60\ ksi)\left(2.75\ in. - \frac{0.54\ in.}{2}\right)$$

$$= 3810.67\ k - in.$$

$$\phi M_{n,min} = 3297.74\ k - in.$$

8.2.3 Cracking Moment

$$f_r = 7.5\lambda\sqrt{f_c'}$$ *Equation 8-17*

$$f_r = 7.5(1.0)\sqrt{4{,}000\ psi} = 474\ psi = 0.474\ ksi$$

$$I_g = \frac{1}{12}bh^3$$ *Equation 8-18*

$$I_g = \frac{1}{12}(16 \times 12)in.(7.25\ in. + 2.5\ in + 3.5\ in)^3 - (2.5\ in)^3$$

$$= 36969.25\ in.^4$$

$$M_{cr} = \frac{f_r I_g}{y_t}$$ *Equation 8-19*

$$M_{cr,int} = \frac{(0.474\ ksi)(24084\ in.^4)}{5.47\ in.} = 2087.64\ k - in.$$

$$M_{cr,int} = 2087.64\ k - in. < \phi M_{n,int} = 3297.74\ k - in. \qquad OK$$

$$M_{cr,ext} = \frac{(0.474\ ksi)(24084\ in.^4)}{6.03\ in.} = 1895.23\ k-in.$$

$$M_{cr,ext} = 1895.23\ k-in. < \ \phi M_{n,ext} = 3810.67\ k-in. \quad OK$$

$$M_{cr,min} = 1895.23\ k-in$$

8.2.4 Minimum Vertical and Horizontal Reinforcement

$$\rho_t = \frac{A_{s,horizontal}}{A_g} \geq 0.0025$$ *Equation 8-20*

$$\rho_{t,int} = \frac{3.80\ in.^2}{(23 \times 12)in(5.5\ in)} = 0.0025\ \geq 0.0025 \quad OK$$

$$\rho_{t,ext} = \frac{5.20\ in.^2}{(23 \times 12)in(3.5\ in)} = 0.0054\ \geq 0.0025 \quad OK$$

$$\rho_l \geq 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_t - 0.0025)$$ *Equation 8-21*

$or\ 0.0025\ whichever\ is\ greater$

$$\rho_{l,min} = 0.0025 + 0.5\left(2.5 - \frac{23\ ft}{16\ ft}\right)(0.0025 - 0.0025) =$$

$0.0025\ or\ 0.0025$

$$\rho_{l,min} = 0.0025$$

$$\rho_l = \frac{A_{s,vertical}}{A_g} \geq 0.0031$$ *Equation 8-22*

$$\rho_{l,int} = \frac{5.89\ in.^2}{(16 \times 12)in.(5.5\ in.)} = 0.0056 \geq 0.0025 \quad OK$$

$$\rho_{l,ext} = \frac{5.58\ in.^2}{(16 \times 12)in.(3.5\ in.)} = 0.0083 \geq 0.0025 \quad OK$$

Check minimum flexural reinforcement:

$$A_{s,min} = \frac{3\sqrt{f_c'}}{f_y} b_w d \ or \ \frac{200}{f_y} b_w d$$ *Equation 8-23*

$$A_{s,min,int} = \frac{3\sqrt{4{,}000\ psi}}{60{,}000\ psi} 12\ in.(8.75\ in.) = 0.27\ in.^2/ft$$

$$or\ \frac{200}{60{,}000\ psi} 12\ in.(8.75\ in.) = 0.29\ in.^2/ft$$

$$A_{s,min,int} = 0.29\ in.^2/ft < A_s = 0.37\ in.^2/ft \quad OK$$

$$A_{s,min,ext} = \frac{3\sqrt{4{,}000\ psi}}{60{,}000\ psi} 12\ in.(9.75\ in.) = 0.32\ in.^2/ft$$

$$or\ \frac{200}{60{,}000\ psi} 12\ in.(9.75\ in.) = 0.34\ in.^2/ft$$

$$A_{s,min,ext} = 0.34\ in.^2/ft < A_s = 0.35\ in.^2/ft \quad OK$$

Check spacing:

$$s_{max} = 3h\ or\ 18\ in. \quad whichever\ is\ smaller$$ *Equation 8-24*

$$s_{max,int} = 3(5.5\ in.) = 16.5\ in.\ or\ 18\ in.$$

$$s_{max,int} = 16.5\ in. > s = 9.5\ in. \quad OK$$

$$s_{max,ext} = 3(3.5\ in.) = 10.5\ in.\ or\ 18\ in.$$

$$s_{max,ext} = 10.5\ in. > s = 10\ in. \quad OK$$

8.2.5 Applied Ultimate Moment

$$I_{cr} = \frac{E_s}{E_c}(A_{se})(d-c)^2 + \frac{l_w c^3}{3} + \left(\frac{E_s}{E_c} - 1\right)(A'_s)(c-d')^2$$ *Equation 8-25*

$$I_{cr,int} = \frac{29{,}000\ ksi}{3605\ ksi}(6.25\ in.^2)(8.75\ in - 0.68in)^2$$

$$+ \frac{(16 \times 12)in\ (0.68in.)^3}{3}$$

$$+ \left(\frac{29{,}000\ ksi}{3605\ ksi} - 1\right)(5.58\ in.^2)(0.68in - 1.75in)^2$$

$$I_{cr,int} = 3343.6\ in.^4$$

$$I_{cr,ext} = \frac{29{,}000\ ksi}{3605\ ksi}(5.90\ in.^2)(9.75\ in - 0.64in)^2$$

$$+ \frac{(16 \times 12)in\ (0.64in.)^3}{3}$$

$$+ \left(\frac{29{,}000\ ksi}{3605\ ksi} - 1\right)(5.89\ in.^2)(0.64in - 2.75in)^2$$

$$I_{cr,ext} = 4145.1\ in.^4$$

$$I_{cr,min} = 3343.6\ in.^4$$

$$M_{ua} = \frac{w_u l_c^2}{8} + \frac{P_{ua} e_{cc}}{2}$$ *Equation 8-26*

$$M_{ua} = \frac{0.744\ klf\ (21\ ft)^2}{8}$$

$$+ \frac{14.31\ kips\ \times (\frac{7.25}{12}ft + \frac{2.5}{12}ft + \frac{3.5}{12}ft)/2)}{2}$$

$$= 44.44\ k - ft\ or\ 533.31\ k - in.$$

$$M_u = \frac{M_{ua}}{\left(1 - \frac{5P_{um} l_c^2}{(0.75)48 E_c I_{cr}}\right)}$$ *Equation 8-27*

$$M_u = \frac{533.31\ k - in}{\left(1 - \frac{5(42.21\ kips)(21 \times 12\ in)^2}{(0.75)48(3605\ ksi)3343.6\ in.^4}\right)} = 550.31\ k - in.$$

$$M_u = 550.31\ \ k - in < \phi M_{n,min} = 3297.74\ k - in. \quad OK$$ *Equation 8-28*

$$\Delta_u = \frac{5 M_u l_c^2}{(0.75) 48 E_c I_{cr}}$$ *Equation 8-29*

$$\Delta_u = \frac{5(550.31\ k-in)(21 \times 12\ in)^2}{(0.75)48(3605\ ksi)3343.6\ in.^4} = 0.40\ in.$$

8.2.6 Service Load Deflection

Unfactored axial force without panel weight:

$$P_a = D + L_r$$ *Equation 8-30*

$$P_a = 4.11\ kips + 5.87\ kips = 9.97\ kips$$

Unfactored axial force with panel weight:

$$P_{sm} = P_a + D_{wall}$$ *Equation 8-31*

$$P_{sm} = 9.97\ kips + 23.25\ kips = 33.22\ kips$$

Factored out-of-plane load:

$$w_s = 0.7 \rho Q_E$$ *Equation 8-32*

$$w_s = 0.7 \times 1.0 \times 46.50\ psf \times 16\ ft = 521\ plf = 0.521\ klf$$

$$A_{se} = A_s + \frac{P_{sm}}{f_y}\frac{h}{2d}$$ *Equation 8-33*

$$A_{se,int} = 5.89\ in.^2 + \frac{33.22\ kips}{60\ ksi}\frac{(5.5\ in + 3.5 in)}{2(8.75\ in.)} = 6.17\ in.^2$$

$$A_{se,ext} = 5.58\ in.^2 + \frac{33.22\ kips}{60\ ksi}\frac{(5.5\ in + 3.5 in)}{2(9.75\ in.)} = 5.84\ in.^2$$

$$a = \frac{A_{se} f_y}{0.85 f_c' b}$$ *Equation 8-34*

$$a_{int} = \frac{(6.17\ in.^2)(60\ ksi)}{0.85(4\ ksi)(16 \times 12) in.} = 0.57\ in$$

$$a_{ext} = \frac{(5.84\ \ in.^2)(60\ ksi)}{0.85(4\ ksi)(16 \times 12)in.} = 0.54\ in$$

$$c = \frac{a}{\beta_1}$$ *Equation 8-35*

$$c_{int} = \frac{0.57\ in.}{0.85} = 0.67\ in.$$

$$c_{ext} = \frac{0.54\ in.}{0.85} = 0.63\ in.$$

$$\varepsilon_t = 0.003\ \times \frac{d - c}{c}$$ *Equation 8-36*

$$\varepsilon_{t,int} = 0.003\ \times \frac{8.75\ in. - 0.67\ in.}{0.67\ in.} = 0.0363\ > 0.005$$

$$\therefore tension - controlled\ \phi\ =\ 0.90$$

$$\varepsilon_{t,ext} = 0.003\ \times \frac{9.75\ in. - 0.63\ in.}{0.63\ in.} = 0.0410 > 0.005$$

$$\therefore tension - controlled\ \phi\ =\ 0.90$$

$$\phi M_n =\ \phi\left[A_{se}f_y\left(d - \frac{a}{2}\right) + A'_s f_y\left(d' - \frac{a}{2}\right)\right]$$ *Equation 8-37*

$$\phi M_{n,int} = 0.90\left((6.17\ in.^2)(60\ ksi)\left(8.75\ in. - \frac{0.57 in.}{2}\right) + (5.58\ in.^2)(60\ ksi)\left(1.75\ in. - \frac{0.57 in.}{2}\right)\right) = 3264.77\ k - in.$$

$$\phi M_{n,ext} = 0.90((5.84\ in.^2)(60\ ksi)\left(9.75\ in. - \frac{0.54\ in.}{2}\right) + (5.89\ in.^2)(60\ ksi)\left(2.75\ in. - \frac{0.54\ in.}{2}\right) = 3777.28\ k - in.$$

$\phi M_{n,min} = 3264.77\ k - in.$

$M_{n,min} = 3627.52\ k - in$

$M_{cr,int} = 2087.64\ k - in. < \ \phi M_{n,int} = 3264.77\ k - in. \quad OK$

$M_{cr,ext} = 1895.23\ k - in. < \ \phi M_{n,ext} = 3777.28\ k - in. \quad OK$

$$I_{cr} = \frac{E_s}{E_c}(A_{se})(d-c)^2 + \frac{l_w c^3}{3} + \left(\frac{E_s}{E_c} - 1\right)(A'_s)(c-d')^2$$ *Equation 8-38*

$$I_{cr,int} = \frac{29{,}000\ ksi}{3605\ ksi}(6.17\ in.^2)(8.75in - 0.67\ in)^2$$

$$+ \frac{(16 \times 12)in\ (0.67in.)^3}{3}$$

$$+ \left(\frac{29{,}000\ ksi}{3605\ ksi} - 1\right)(5.58\ in.^2)(0.67\ in - 1.75\ in)^2$$

$I_{cr,int} = 3309.9\ in.^4$

$$I_{cr,ext} = \frac{29{,}000\ ksi}{3605\ ksi}(5.84\ in.^2)(9.75in - 0.63\ in)^2$$

$$+ \frac{(16 \times 12)in\ (0.63in.)^3}{3}$$

$$+ \left(\frac{29{,}000\ ksi}{3605\ ksi} - 1\right)(5.89\ in.^2)(0.63\ in - 2.75\ in)^2$$

$I_{cr,ext} = 4106.0\ in.^4$

$I_{cr,min} = 3309.9\ in.^4$

$$\Delta_{allow} = \frac{l_c}{150}$$ *Equation 8-39*

$$\Delta_{allow} = \frac{21 \times 12\ in.}{150} = 1.68\ in.$$

$$\Delta_{cr} = \frac{5 M_{cr} l_c^2}{48 E_c I_g}$$ *Equation 8-40*

$$\Delta_{cr} = \frac{5(2087.64\ k - in)(21 \times 12\ in)^2}{48(3605\ ksi)(24084\ in^4)} = 0.16\ in.$$

$$\Delta_n = \frac{5M_n l_c^2}{(0.75)48E_c I_{cr}}$$ *Equation 8-41*

$$\Delta_n = \frac{5(3627.52\ k - in)(21 \times 12\ in)^2}{48(3605\ ksi)(3309.9\ in^4)} = 2.01\ in.$$

$$M_{sa} = \frac{w_s l_c^2}{8} + \frac{P_a e_{cc}}{2}$$ *Equation 8-42*

$$M_{sa} = \frac{0.521\ klf\ (21\ ft)^2}{8} + \frac{9.97\ kips\ \times (5.75/12)\ ft}{2}$$

$$= 31.10\ k - ft\ or\ 373.18\ k - in.$$

For the first iteration, assume $M_a \leq 2/3\ M_{cr}$:

$$\Delta_s = \frac{M_a}{M_{cr}} \Delta_{cr}$$ *Equation 8-43*

$$\Delta_s = \frac{373.18\ k - in}{1895.23\ \ k - in} 0.16\ in = 0.031\ in.$$

$$M_a = M_{sa} + P_{sm}\Delta_s$$ *Equation 8-44*

$$M_a = 373.18\ k - in + (33.22\ kips)(0.031\ in.) = 374.22\ k - in$$

$$M_a = 374.22\ k - in\ < \frac{2}{3} M_{cr} = 1263.49\ k - in$$ *Equation 8-45*

For the second iteration:

$$\Delta_s = \frac{M_a}{M_{cr}} \Delta_{cr}$$ *Equation 8-46*

$$\Delta_s = \frac{374.22\ k - in}{1895.23\ \ k - in} 0.16\ in = 0.029\ in.$$

$$M_a = M_{sa} + P_{sm}\Delta_s$$ *Equation 8-47*

$$M_a = 373.18\ k - in + (33.22\ kips)(0.029\ in.) = 374.13\ k - in$$

$$M_a = 374.13\ k - in\ < \frac{2}{3} M_{cr} = 1263.49\ k - in \qquad \textit{Equation 8-48}$$

For the third and last iteration:

$$\Delta_s = \frac{M_a}{M_{cr}} \Delta_{cr} \qquad \textit{Equation 8-49}$$

$$\Delta_s = \frac{374.13\ k - in}{1895.23\ \ k - in} 0.16\ in = 0.029\ in.$$

$$M_a = M_{sa} + P_{sm}\Delta_s \qquad \textit{Equation 8-50}$$

$$M_a = 373.18\ k - in + (33.22\ kips)(0.029\ in.) = 374.13\ k - in$$

$$M_a = 374.13\ k - in\ < \frac{2}{3} M_{cr} = 1263.49\ k - in \qquad \textit{Equation 8-51}$$

Check the horizontal shear at the interface of the concrete and insulation:

$$T_u = A_s f_y \qquad \textit{Equation 8-52}$$

$$T_u = 5.89\ in.^2 \times 60\ ksi = 353.4\ k$$

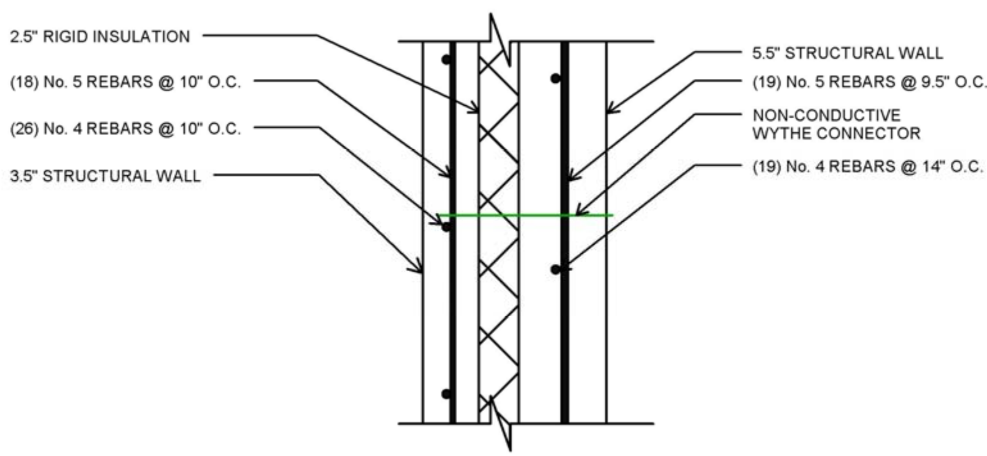
Provide wythe connectors to resist 353.4 kips in each half height of the panel

8.3 Summary

The vertical stress in the composite tilt-up panel is 24 psi and the design moment strength, ϕM_n, is 3298 k-in. The maximum ultimate moment due to applied loads is 550.31 k-in which is about 17% of the design moment strength. It is evident that the wall can resist more load than the applied loads therefore the thickness of the wall can be decreased to maximize its efficiency. The service load deflection is 0.029 inches which is less than the allowable deflection of 1.68 inches. The cracking moment, M_{cr}, for this wall panel is 1895.23 k-in, which is greater than the maximum ultimate moment, thus the wall has not cracked. A tabulated summary is provided in *Table 8-1*.

Table 8-1. Summary of Composite Sandwich Tilt-Up Wall Panel Design.

	Ultimate Design	***Service Design***
P (k)	14.31	9.97
P_m (k)	42.21	33.22
W (klf)	0.744	0.521
M_{cr} (k-in)	1895.23	1895.23
ϕM_n (k-in)	3297.74	3264.77
$A_{s\ vert,\ int}$ (in^2)	5.89	5.89
$A_{s\ vert,\ ext}$ (in^2)	5.58	5.58
$A_{se\ vert,\ int}$ (in^2)	6.25	6.25
$A_{se\ vert,\ ext}$ (in^2)	5.90	5.90
$A_{s\ horiz,\ int}$ (in^2)	3.80	3.80
$A_{s\ horiz\ ext}$ (in^2)	5.20	5.20
I_{cr} (in^4)	3343.6	3309.9
M (k-in)	550.31	373.18
Δ (in)	0.40	0.029



Chapter 9 - Conclusions

As calculated in the parametric study, the vertical stresses in the standard solid tilt-up concrete panel, non-composite sandwich tilt-up panel, and composite sandwich tilt-up panel are 29 psi, 40 psi, and 24 psi, respectively. The composite sandwich panel experiences the least vertical stress among the panels because of the gross area of the wall. Since both the interior and exterior concrete wythes are resisting the loads for a composite sandwich panel, even though the factored axial loads of the sandwich panels are the same, that has approximately 64% more load-bearing wall gross area than the non-composite sandwich panel. Additionally, the composite sandwich panel experiences 19% and 64% less vertical stress than the single wythe panel and the non-composite sandwich panel, respectively. When compared to the standard solid tilt-up panel, the non-composite sandwich panel experiences 37% more vertical stress. The non-composite panel experiences the greatest vertical stress among the three different panels. This is because of the increased axial load at midheight due to the weight of the exterior concrete layer. Moreover, only the 5.5-inch interior concrete layer is resisting the load.

The design moment strengths of the standard solid panel, non-composite panel, and composite panel are 584 k-in, 907 k-in, and 3298 k-in, respectively. The 55% increase in the design moment strength of the non-composite panel, compared to the standard panel, is due to the increase in the effective area of reinforcement. The effective area of reinforcement is dependent on the amount of vertical reinforcement, the factored axial force with the panel weight, the thickness of the wall, and the distance of the tension reinforcement from the extreme fiber in compression. Since the non-composite panel has an additional 3.5-inch exterior concrete layer, the panel weight increased which then increased the factored axial load, thus increasing the effective area of reinforcement. The design moment strength of the composite panel is more than 4.5 times larger than the standard panel.

The cracking moments of the standard solid panel, non-composite panel, and composite panel are 459 k-in, 459 k-in, and 1895 k-in, respectively. It is dependent on the concrete’s modulus of rupture, gross moment of inertia, and the distance from the centroidal axis of gross section to the tension face. The solid panel and the non-composite panel have the same properties therefore they have equivalent cracking moments. The distance from the centroidal axis of the gross section to the tension face of the composite panel is almost twice than that of the standard panel and non-composite panel, which would have decreased the cracking moment of the composite panel. Since the concrete gross area of the composite panel is 64% more than the single-wythe and non-composite sandwich panel, it substantially increased the cracking moment of the composite panel by more than 3 times.

The applied ultimate moments on to the standard solid panel, non-composite panel, and composite panel are 524 k-in, 873 k-in, and 550 k-in, respectively. It is expected for the standard panel to have the least applied moment since the sandwich panels have additional panel weight due to the exterior concrete layer and insulation. The cracked moment of inertia of the composite panel is approximately 13 times more than the non-composite and 17 times more than the standard panel due to the increased gross area of the concrete wall and the steel reinforcements.

The service load deflections on the standard solid panel, non-composite panel, and composite panel are 0.15 inches, 1.16 inches, and 0.03 inches, respectively. It is expected the composite panel will yield the smallest deflection due to the much larger gross area of the concrete. The non-composite panel obtained the largest deflection due to the increased panel weight but the lack of more load bearing wall that can resist the load.

As discussed in Chapter 3, it is evident that the use of sandwich panels significantly increased energy efficiency of the building. The sandwich panels are 50% more efficient in cooling than standard solid panels without insulation and 68% more efficient in heating. Also, sandwich panels are 8% more efficient in cooling than standard solid panels with 2.5-inch insulation and 2% more efficient in heating. Although the results in Chapter 3 showed that post-insulated single-wythe tilt-up panels and sandwich panels have comparable loads, using sandwich panels speeds up the construction due to their integral insulation. It also requires less labor, coordination, and labor crews compared to post-insulated single-wythe tilt-up panels.

In the construction aspect of sandwich panels, they require more materials for the formwork due to the increased panel thickness. Also, an additional amount of concrete is needed to account for the exterior and interior layers of concrete, thus requiring two concrete pours per panel. Moreover, anchorage pullout testing on the wythe connectors is required for sandwich panels. Since the sandwich panels weigh more than the standard panels, higher crane capacity is needed to lift the heavy panels. Smaller widths for the sandwich panels can be designed so that a lower crane capacity can be used. This will increase the amount of panels that the crane needs to lift.

Overall, the composite sandwich tilt-up panel is the most efficient in terms of the structural design. As seen in the results, the composite sandwich tilt-up panel can resist a significant amount of loads compared to the other panel types. The overall thickness of the composite tilt-up panels can be significantly reduced to resist the loads induced in the walls. It is important to ensure that the wythe connectors can resist the horizontal shear at the interface of the concrete and insulation

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Appendix A - Standard Solid Panel Load Combination Results

STANDARD SOLID PANEL DESIGN

LOAD COMBINATIONS

Joist span =	36.67	ft	NWC desnity =	150	pcf
Joist spacing =	8	ft	f'_c =	4000	psi
Wall height =	21	ft	f_y =	60000	psi
	2	ft	I_e =	1	
Wall width =	16	ft			
Wall thickness =	5.5	in			

Input Loads

D_{roof} =	14.00	psf =	4.11	k
D_{wall} =	68.75	psf =	13.75	k
L =	0	psf		
L_r =	20	psf =	5.87	k
S =	0	psf		
R =	0	psf		
W =	0	psf		
Q_E =	27.50	psf		
ρ =	1			
S_{DS} =	1	g		

Roof

D =	14	psf				
	4.1	k				
er joist =	2.1	k	2	joists =	4.11	k
L =	20	psf				
	5.9	k				
er joist =	2.9	k	2	joists =	5.87	k

Walls

D_{wall} =	68.75	psf
	13.75	k
F_p =	27.50	psf

	Factored Load Combinations	Vert. Loads (kips)		Horiz. Loads (PSF)	
1	1.4D	25.00		-	
2.a	$1.2D+1.6L+0.5L_r$	24.36		-	
2.b	1.2D+1.6L+0.5S	21.43		-	
2.c	1.2D+1.6L+0.5R	21.43		-	
3.a	$1.2D+1.6L_r+L$	30.81	<--governs	-	
3.b	$1.2D+1.6L_r+0.5W$	30.81	<--governs	-	
3.c	1.2D+1.6S+L	21.43		-	
3.d	1.2D+1.6S+0.5W	21.43		-	
3.e	1.2D+1.6R+L	21.43		-	
3.f	1.2D+1.6R+0.5W	21.43		-	
4.a	$1.2D+1.0W+L+0.5L_r$	24.36		-	
4.b	1.2D+1.0W+L+0.5S	21.43		-	
4.c	1.2D+1.0W+L+0.5R	21.43		-	
5.a	$(1.2+0.2S_{DS})D+\rho Q_E+L+0.2S$	25.00		27.5	<--governs
6	0.9D+1.0W	16.07		-	
7.a	$(0.9-0.2S_{DS})D+\rho Q_E$	12.50		27.5	<--governs

	Service Load Combinations	Vert. Loads (kips)		Horiz. Loads (PSF)	
1	D	17.86		-	
2	D+L	17.86		-	
3.a	$D+L_r$	23.72	<--governs	-	
3.b	D+S	17.86		-	
3.c	D+R	17.86		-	
4.a	$D+0.75L+0.75L_r$	22.26		-	
4.b	D+0.75L+0.75S	17.86		-	
4.c	D+0.75L+0.75R	17.86		-	
5.a	D+0.6W	17.86		-	
5.b	$(1.0+0.14S_{DS})D+0.7\rho Q_E$	20.36		19.25	<--governs
6.a.1	$D+0.75L+0.75(0.6W)+0.75L_r$	22.26		-	
6.a.2	D+0.75L+0.75(0.6W)+0.75S	17.86		-	
6.a.3	D+0.75L+0.75(0.6W)+0.75R	17.86		-	
6.b	$(1.0+0.10S_{DS})D+0.75L+0.525\rho Q_E+0.75S$	19.64		14.44	
7	0.6D+0.6W	10.71		-	
8	$(0.6-0.14S_{DS})D+0.7\rho Q_E$	8.21		19.25	<--governs

Appendix B - Non-Composite Panel Load Combination Results

NON-COMPOSITE SANDWICH PANEL DESIGN

LOAD COMBINATIONS

Joist span =	36.67	ft
Joist spacing =	8	ft
Wall height =	21	ft
	2	ft
Wall width =	16	ft
Wall thickness =	5.5	in
	3.5	in
Insulation thickness =	2.5	in

NWC desnity =	150	pcf
f'_c =	4000	psi
f_y =	60000	psi
I_e =	1	
Insul. load per 1/2" =	0.75	psf

Input Loads

D_{roof} =	14.00	psf =	4.11	k
D_{wall} =	116.25	psf =	23.25	k
L =	0	psf		
L_r =	20	psf =	5.87	k
S =	0	psf		
R =	0	psf		
W =	0	psf		
Q_E =	46.50	psf		
ρ =	1			
S_{DS} =	1	g		

Roof

D =	14	psf				
	4.1	k				
r joist =	2.1	k	2	joists =	4.11	k
L =	20	psf				
	5.9	k				
r joist =	2.9	k	2	joists =	5.87	k

Walls

D_{wall} =	116.25	psf
	23.25	k
F_p =	46.50	psf

	Factored Load Combinations	Vert. Loads (kips)		Horiz. Loads (PSF)	
1	1.4D	38.30		-	
2.a	$1.2D+1.6L+0.5L_r$	35.76		-	
2.b	1.2D+1.6L+0.5S	32.83		-	
2.c	1.2D+1.6L+0.5R	32.83		-	
3.a	$1.2D+1.6L_r+L$	42.21	<--governs	-	
3.b	$1.2D+1.6L_r+0.5W$	42.21	<--governs	-	
3.c	1.2D+1.6S+L	32.83		-	
3.d	1.2D+1.6S+0.5W	32.83		-	
3.e	1.2D+1.6R+L	32.83		-	
3.f	1.2D+1.6R+0.5W	32.83		-	
4.a	$1.2D+1.0W+L+0.5L_r$	35.76		-	
4.b	1.2D+1.0W+L+0.5S	32.83		-	
4.c	1.2D+1.0W+L+0.5R	32.83		-	
5.a	$(1.2+0.2S_{DS})D+\rho Q_E+L+0.2S$	38.30		46.5	<--governs
6	0.9D+1.0W	24.62		-	
7.a	$(0.9-0.2S_{DS})D+\rho Q_E$	19.15		46.5	<--governs

	Service Load Combinations	Vert. Loads (kips)		Horiz. Loads (PSF)	
1	D	27.36		-	
2	D+L	27.36		-	
3.a	$D+L_r$	33.22	<--governs	-	
3.b	D+S	27.36		-	
3.c	D+R	27.36		-	
4.a	$D+0.75L+0.75L_r$	31.76		-	
4.b	D+0.75L+0.75S	27.36		-	
4.c	D+0.75L+0.75R	27.36		-	
5.a	D+0.6W	27.36		-	
5.b	$(1.0+0.14S_{DS})D+0.7\rho Q_E$	31.19		32.55	<--governs
6.a.1	$D+0.75L+0.75(0.6W)+0.75L_r$	31.76		-	
6.a.2	D+0.75L+0.75(0.6W)+0.75S	27.36		-	
6.a.3	D+0.75L+0.75(0.6W)+0.75R	27.36		-	
6.b	$(1.0+0.10S_{DS})D+0.75L+0.525\rho Q_E+0.75S$	30.09		24.41	
7	0.6D+0.6W	16.41		-	
8	$(0.6-0.14S_{DS})D+0.7\rho Q_E$	12.58		32.55	<--governs

Appendix C - Composite Panel Load Combination Results

COMPOSITE SANDWICH PANEL DESIGN | LOAD COMBINATIONS

Joist span =	36.67	ft
Joist spacing =	8	ft
Wall height =	21	ft
	2	ft
Wall width =	16	ft
Wall thickness =	5.5	in
	3.5	in
Insulation thickness =	2.5	in

NWC desnity =	150	pcf
f'_c =	4000	psi
f_y =	60000	psi
I_e =	1	
Insul. load per 1/2" =	0.75	psf

Input Loads

D_{roof} =	14.00	psf =	4.11	k
D_{wall} =	116.25	psf =	23.25	k
L =	0	psf		
L_r =	20	psf =	5.87	k
S =	0	psf		
R =	0	psf		
W =	0	psf		
Q_E =	46.50	psf		
ρ =	1			
S_{DS} =	1	g		

Roof

D =	14	psf				
	4.1	k				
er joist =	2.1	k	2	joists =	4.11	k
L =	20	psf				
	5.9	k				
er joist =	2.9	k	2	joists =	5.87	k

Walls

D_{wall} =	116.25	psf
	23.25	k
F_p =	46.50	psf

	Factored Load Combinations	Vert. Loads (kips)		Horiz. Loads (PSF)	
1	1.4D	38.30		-	
2.a	$1.2D+1.6L+0.5L_r$	35.76		-	
2.b	1.2D+1.6L+0.5S	32.83		-	
2.c	1.2D+1.6L+0.5R	32.83		-	
3.a	$1.2D+1.6L_r+L$	42.21	<--governs	-	
3.b	$1.2D+1.6L_r+0.5W$	42.21	<--governs	-	
3.c	1.2D+1.6S+L	32.83		-	
3.d	1.2D+1.6S+0.5W	32.83		-	
3.e	1.2D+1.6R+L	32.83		-	
3.f	1.2D+1.6R+0.5W	32.83		-	
4.a	$1.2D+1.0W+L+0.5L_r$	35.76		-	
4.b	1.2D+1.0W+L+0.5S	32.83		-	
4.c	1.2D+1.0W+L+0.5R	32.83		-	
5.a	$(1.2+0.2S_{DS})D+\rho Q_E+L+0.2S$	38.30		46.5	<--governs
6	0.9D+1.0W	24.62		-	
7.a	$(0.9-0.2S_{DS})D+\rho Q_E$	19.15		46.5	<--governs

	Service Load Combinations	Vert. Loads (kips)		Horiz. Loads (PSF)	
1	D	27.36		-	
2	D+L	27.36		-	
3.a	$D+L_r$	33.22	<--governs	-	
3.b	D+S	27.36		-	
3.c	D+R	27.36		-	
4.a	$D+0.75L+0.75L_r$	31.76		-	
4.b	D+0.75L+0.75S	27.36		-	
4.c	D+0.75L+0.75R	27.36		-	
5.a	D+0.6W	27.36		-	
5.b	$(1.0+0.14S_{DS})D+0.7\rho Q_E$	31.19		32.55	<--governs
6.a.1	$D+0.75L+0.75(0.6W)+0.75L_r$	31.76		-	
6.a.2	D+0.75L+0.75(0.6W)+0.75S	27.36		-	
6.a.3	D+0.75L+0.75(0.6W)+0.75R	27.36		-	
6.b	$(1.0+0.10S_{DS})D+0.75L+0.525\rho Q_E+0.75S$	30.09		24.41	
7	0.6D+0.6W	16.41		-	
8	$(0.6-0.14S_{DS})D+0.7\rho Q_E$	12.58		32.55	<--governs

Appendix D - Standard Panel without insulation TRACE700 Results

System Checksums

By ACADEMIC

System - 001 — **Water Source Heat Pump**

COOLING COIL PEAK / CLG SPACE PEAK

Peaked at Time: Mo/Hr: 7 / 17 — Outside Air: OADB/WB/HR: 91 / 75 / 114

CLG SPACE PEAK: Mo/Hr: Sum of — OADB: Peaks

	Space Sens. + Lat. Btu/h	Plenum Sens. + Lat Btu/h	Net Total Btu/h	Percent Of Total (%)	Space Sensible Btu/h	Percent Of Total (%)
Envelope Loads						
Skylite Solar	0	0	0	0	0	0
Skylite Cond	0	0	0	0	0	0
Roof Cond	0	357,997	357,997	36	0	0
Glass Solar	2,624	0	2,624	0	2,479	0
Glass/Door Cond	276	0	276	0	211	0
Wall Cond	482,286	32,170	514,456	52	542,310	81
Partition/Door	0		0	0	0	0
Floor	0		0	0	0	0
Adjacent Floor	0	0	0	0	0	0
Infiltration	0		0	0	0	0
Sub Total ==>	485,186	390,167	875,353	88	544,999	81
Internal Loads						
Lights	0	0	0	0	0	0
People	118,881	0	118,881	12	62,965	9
Misc	0	0	0	0	0	0
Sub Total ==>	118,881	0	118,881	12	62,965	9
Ceiling Load	86,474	-86,474	0	0	63,510	9
Ventilation Load	0	0	0	0	0	0
Adj Air Trans Heat	0		0	0	0	0
Dehumid. Ov Sizing			0	0		
Ov/Undr Sizing	0		0	0	0	0
Exhaust Heat		0	0	0		
Sup. Fan Heat			0	0		
Ret. Fan Heat		0	0	0		
Duct Heat Pkup		0	0	0		
Underflr Sup Ht Pkup			0	0		
Supply Air Leakage		0	0	0		
Grand Total ==>	690,541	303,693	994,234	100.00	671,474	100.00

HEATING COIL PEAK

Mo/Hr: Heating Design — OADB: 9

	Space Peak Space Sens Btu/h	Coil Peak Tot Sens Btu/h	Percent Of Total (%)
Envelope Loads			
Skylite Solar	0	0	0.00
Skylite Cond	0	0	0.00
Roof Cond	0	-255,607	14.68
Glass Solar	0	0	0.00
Glass/Door Cond	-1,031	-1,031	0.06
Wall Cond	-1,310,129	-1,422,146	81.68
Partition/Door	0	0	0.00
Floor	-62,303	-62,303	3.58
Adjacent Floor	0	0	0
Infiltration	0	0	0.00
Sub Total ==>	-1,373,463	-1,741,086	100.00
Internal Loads			
Lights	0	0	0.00
People	0	0	0.00
Misc	0	0	0.00
Sub Total ==>	0	0	0.00
Ceiling Load	-81,481	0	0.00
Ventilation Load	0	0	0.00
Adj Air Trans Heat	0	0	0
Ov/Undr Sizing	0	0	0.00
Exhaust Heat		0	0.00
OA Preheat Diff.		0	0.00
RA Preheat Diff.		0	0.00
Additional Reheat		0	0.00
Underflr Sup Ht Pkup		0	0.00
Supply Air Leakage		0	0.00
Grand Total ==>	-1,454,944	-1,741,086	100.00

TEMPERATURES

	Cooling	Heating
SADB	60.0	102.4
Ra Plenum	81.8	63.6
Return	81.8	63.6
Ret/OA	81.8	63.6
Fn MtrTD	0.0	0.0
Fn BldTD	0.0	0.0
Fn Frict	0.0	0.0

AIRFLOWS

	Cooling	Heating
Diffuser	42,243	42,243
Terminal	42,243	42,243
Main Fan	42,243	42,243
Sec Fan	0	0
Nom Vent	0	0
AHU Vent	0	0
Infil	0	0
MinStop/Rh	0	0
Return	42,243	42,243
Exhaust	0	0
Rm Exh	0	0
Auxiliary	0	0
Leakage Dwn	0	0
Leakage Ups	0	0

ENGINEERING CKS

	Cooling	Heating
% OA	0.0	0.0
cfm/ft²	1.05	1.05
cfm/ton	509.86	
ft²/ton	486.65	
Btu/hr·ft²	24.66	-43.18
No. People	242	

COOLING COIL SELECTION

	Total Capacity ton	Total Capacity MBh	Sens Cap. MBh	Coil Airflow cfm	Enter DB °F	Enter WB °F	Enter HR gr/lb	Leave DB °F	Leave WB °F	Leave HR gr/lb
Main Clg	82.9	994.2	938.3	42,243	81.8	64.5	67.9	60.0	56.8	67.3
Aux Clg	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Opt Vent	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Total	82.9	994.2								

AREAS

	Gross Total	Glass ft²	Glass (%)
Floor	40,320		
Part	0		
Int Door	0		
ExFlr	1,502		
Roof	40,320	0	0
Wall	35,716	0	0
Ext Door	84	84	100

HEATING COIL SELECTION

	Capacity MBh	Coil Airflow cfm	Ent °F	Lvg °F
Main Htg	-1,741.1	42,243	63.6	102.4
Aux Htg	0.0	0	0.0	0.0
Preheat	0.0	0	0.0	0.0
Humidif	0.0	0	0.0	0.0
Opt Vent	0.0	0	0.0	0.0
Total	-1,741.1			

Project Name:
Dataset Name: ROBEE.TRC

TRACE® 700 v6.3 calculated at 02:45 PM on 04/05/2017
Alternative - 1 System Checksums Report Page 1 of 1

Appendix E - Standard Panel with Insulation TRACE700 Results

System Checksums

By ACADEMIC

System - 001 — **Water Source Heat Pump**

COOLING COIL PEAK / CLG SPACE PEAK

Cooling Coil Peak — Peaked at Time: Mo/Hr: 7 / 16; Outside Air: OADB/WB/HR: 92 / 77 / 120

Clg Space Peak — Mo/Hr: Sum of; OADB: Peaks

	Space Sens. + Lat. Btu/h	Plenum Sens. + Lat Btu/h	Net Total Btu/h	Percent Of Total (%)	Space Sensible Btu/h	Percent Of Total (%)
Envelope Loads						
Skylite Solar	0	0	0	0	0	0
Skylite Cond	0	0	0	0	0	0
Roof Cond	0	358,191	358,191	66	0	0
Glass Solar	2,816	0	2,816	1	2,816	1
Glass/Door Cond	285	0	285	0	285	0
Wall Cond	62,788	1,239	64,027	12	62,788	22
Partition/Door	0		0	0	0	0
Floor	0		0	0	0	0
Adjacent Floor	0	0	0	0	0	0
Infiltration	0		0	0	0	0
Sub Total ==>	65,889	359,430	425,319	78	65,889	23
Internal Loads						
Lights	0	0	0	0	0	0
People	118,881	0	118,881	22	62,965	22
Misc	0	0	0	0	0	0
Sub Total ==>	118,881	0	118,881	22	62,965	22
Ceiling Load	159,760	-159,760	0	0	159,760	55
Ventilation Load	0	0	0	0	0	0
Adj Air Trans Heat	0		0	0	0	0
Dehumid. Ov Sizing			0	0		
Ov/Undr Sizing	0		0	0	0	0
Exhaust Heat		0	0	0		
Sup. Fan Heat			0	0		
Ret. Fan Heat		0	0	0		
Duct Heat Pkup		0	0	0		
Underflr Sup Ht Pkup			0	0		
Supply Air Leakage		0	0	0		
Grand Total ==>	344,530	199,670	544,200	100.00	288,614	100.00

HEATING COIL PEAK

Mo/Hr: Heating Design; OADB: 9

	Space Peak Space Sens Btu/h	Coil Peak Tot Sens Btu/h	Percent Of Total (%)
Envelope Loads			
Skylite Solar	0	0	0.00
Skylite Cond	0	0	0.00
Roof Cond	0	-242,720	42.67
Glass Solar	0	0	0.00
Glass/Door Cond	-1,031	-1,031	0.18
Wall Cond	-242,989	-262,717	46.19
Partition/Door	0	0	0.00
Floor	-62,303	-62,303	10.95
Adjacent Floor	0	0	0
Infiltration	0	0	0.00
Sub Total ==>	-306,323	-568,771	100.00
Internal Loads			
Lights	0	0	0.00
People	0	0	0.00
Misc	0	0	0.00
Sub Total ==>	0	0	0.00
Ceiling Load	-116,663	0	0.00
Ventilation Load	0	0	0.00
Adj Air Trans Heat	0	0	0
Ov/Undr Sizing	0	0	0.00
Exhaust Heat		0	0.00
OA Preheat Diff.		0	0.00
RA Preheat Diff.		0	0.00
Additional Reheat		0	0.00
Underflr Sup Ht Pkup		0	0.00
Supply Air Leakage		0	0.00
Grand Total ==>	-422,986	-568,771	100.00

TEMPERATURES

	Cooling	Heating
SADB	56.9	96.5
Ra Plenum	87.5	60.9
Return	87.5	60.9
Ret/OA	87.5	60.9
Fn MtrTD	0.0	0.0
Fn BldTD	0.0	0.0
Fn Frict	0.0	0.0

AIRFLOWS

	Cooling	Heating
Diffuser	15,032	15,032
Terminal	15,032	15,032
Main Fan	15,032	15,032
Sec Fan	0	0
Nom Vent	0	0
AHU Vent	0	0
Infil	0	0
MinStop/Rh	0	0
Return	15,032	15,032
Exhaust	0	0
Rm Exh	0	0
Auxiliary	0	0
Leakage Dwn	0	0
Leakage Ups	0	0

ENGINEERING CKS

	Cooling	Heating
% OA	0.0	0.0
cfm/ft²	0.37	0.37
cfm/ton	331.46	
ft²/ton	889.08	
Btu/hr·ft²	13.50	-14.11
No. People	242	

COOLING COIL SELECTION

	Total Capacity ton	Total Capacity MBh	Sens Cap. MBh	Coil Airflow cfm	Enter DB °F	Enter WB °F	Enter HR gr/lb	Leave DB °F	Leave WB °F	Leave HR gr/lb
Main Clg	45.4	544.2	488.3	15,032	87.5	66.4	68.0	56.9	54.4	62.5
Aux Clg	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Opt Vent	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Total	45.4	544.2								

AREAS

	Gross Total	Glass ft²	Glass (%)
Floor	40,320		
Part	0		
Int Door	0		
ExFlr	1,502		
Roof	40,320	0	0
Wall	35,716	0	0
Ext Door	84	84	100

HEATING COIL SELECTION

	Capacity MBh	Coil Airflow cfm	Ent °F	Lvg °F
Main Htg	-568.8	15,032	60.9	96.5
Aux Htg	0.0	0	0.0	0.0
Preheat	0.0	0	0.0	0.0
Humidif	0.0	0	0.0	0.0
Opt Vent	0.0	0	0.0	0.0
Total	-568.8			

Project Name:
Dataset Name: ROBEE.TRC

TRACE® 700 v6.3 calculated at 02:56 PM on 04/05/2017
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Appendix F - Non-Composite and Composite Sandwich Panel

TRACE700 Results

System Checksums

By ACADEMIC

System - 001 | **Water Source Heat Pump**

COOLING COIL PEAK / CLG SPACE PEAK

Cooling Coil Peak — Peaked at Time: Mo/Hr: 7 / 16; Outside Air: OADB/WB/HR: 92 / 77 / 120

Clg Space Peak — Mo/Hr: Sum of; OADB: Peaks

	Space Sens. + Lat. Btu/h	Plenum Sens. + Lat Btu/h	Net Total Btu/h	Percent Of Total (%)	Space Sensible Btu/h	Percent Of Total (%)
Envelope Loads						
Skylite Solar	0	0	0	0	0	0
Skylite Cond	0	0	0	0	0	0
Roof Cond	0	337,596	337,596	68	0	0
Glass Solar	2,841	0	2,841	1	2,841	1
Glass/Door Cond	265	0	265	0	265	0
Wall Cond	38,163	-918	37,245	7	38,163	14
Partition/Door	0		0	0	0	0
Floor	0		0	0	0	0
Adjacent Floor	0	0	0	0	0	0
Infiltration	0		0	0	0	0
Sub Total ==>	41,269	336,678	377,947	76	41,269	16
Internal Loads						
Lights	0	0	0	0	0	0
People	118,881	0	118,881	24	62,965	24
Misc	0	0	0	0	0	0
Sub Total ==>	118,881	0	118,881	24	62,965	24
Ceiling Load	159,479	-159,479	0	0	159,479	60
Ventilation Load	0	0	0	0	0	0
Adj Air Trans Heat	0		0	0	0	0
Dehumid. Ov Sizing			0	0		
Ov/Undr Sizing	0		0	0	0	0
Exhaust Heat		0	0	0		
Sup. Fan Heat			0	0		
Ret. Fan Heat		0	0	0		
Duct Heat Pkup		0	0	0		
Underflr Sup Ht Pkup			0	0		
Supply Air Leakage		0	0	0		
Grand Total ==>	319,628	177,199	496,828	100.00	263,713	100.00

HEATING COIL PEAK

Mo/Hr: Heating Design; OADB: 9

	Space Peak Space Sens Btu/h	Coil Peak Tot Sens Btu/h	Percent Of Total (%)
Envelope Loads			
Skylite Solar	0	0	0.00
Skylite Cond	0	0	0.00
Roof Cond	0	-240,468	43.24
Glass Solar	0	0	0.00
Glass/Door Cond	-1,031	-1,031	0.19
Wall Cond	-233,483	-252,263	45.37
Partition/Door	0	0	0.00
Floor	-62,303	-62,303	11.20
Adjacent Floor	0	0	0
Infiltration	0	0	0.00
Sub Total ==>	-296,817	-556,065	100.00
Internal Loads			
Lights	0	0	0.00
People	0	0	0.00
Misc	0	0	0.00
Sub Total ==>	0	0	0.00
Ceiling Load	-122,812	0	0.00
Ventilation Load	0	0	0.00
Adj Air Trans Heat	0	0	0
Ov/Undr Sizing	0	0	0.00
Exhaust Heat		0	0.00
OA Preheat Diff.		0	0.00
RA Preheat Diff.		0	0.00
Additional Reheat		0	0.00
Underflr Sup Ht Pkup		0	0.00
Supply Air Leakage		0	0.00
Grand Total ==>	-419,629	-556,065	100.00

TEMPERATURES

	Cooling	Heating
SADB	56.4	99.6
Ra Plenum	87.5	60.4
Return	87.5	60.4
Ret/OA	87.5	60.4
Fn MtrTD	0.0	0.0
Fn BldTD	0.0	0.0
Fn Frict	0.0	0.0

AIRFLOWS

	Cooling	Heating
Diffuser	13,363	13,363
Terminal	13,363	13,363
Main Fan	13,363	13,363
Sec Fan	0	0
Nom Vent	0	0
AHU Vent	0	0
Infil	0	0
MinStop/Rh	0	0
Return	13,363	13,363
Exhaust	0	0
Rm Exh	0	0
Auxiliary	0	0
Leakage Dwn	0	0
Leakage Ups	0	0

ENGINEERING CKS

	Cooling	Heating
% OA	0.0	0.0
cfm/ft²	0.33	0.33
cfm/ton	322.77	
ft²/ton	973.86	
Btu/hr-ft²	12.32	-13.79
No. People	242	

COOLING COIL SELECTION

	Total Capacity ton	Total Capacity MBh	Sens Cap. MBh	Coil Airflow cfm	Enter DB °F	Enter WB °F	Enter HR gr/lb	Leave DB °F	Leave WB °F	Leave HR gr/lb
Main Clg	41.4	496.8	440.9	13,363	87.5	66.4	68.0	56.4	54.1	61.8
Aux Clg	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Opt Vent	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Total	41.4	496.8								

AREAS

	Gross Total	Glass ft²	Glass (%)
Floor	40,320		
Part	0		
Int Door	0		
ExFlr	1,502		
Roof	40,320	0	0
Wall	35,716	0	0
Ext Door	84	84	100

HEATING COIL SELECTION

	Capacity MBh	Coil Airflow cfm	Ent °F	Lvg °F
Main Htg	-556.1	13,363	60.4	99.6
Aux Htg	0.0	0	0.0	0.0
Preheat	0.0	0	0.0	0.0
Humidif	0.0	0	0.0	0.0
Opt Vent	0.0	0	0.0	0.0
Total	-556.1			

Project Name:
Dataset Name: ROBEE.TRC

TRACE® 700 v6.3 calculated at 02:59 PM on 04/05/2017
Alternative - 1 System Checksums Report Page 1 of 1

Appendix G - Reprint Image/Figure Permission

The images contained within this report are property of the author unless otherwise noted. Images provided by others are used by permission of the entities cited in this section.

3/12/2017 RE: Website Inquiry - Other inquiry - Robee Ybanez

RE: Website Inquiry - Other inquiry

Jeff Schaefer

Thu 2/23/2017 1:45 PM

To:Robee Ybanez <robeey@ksu.edu>;

Hi Robee – I'm the photographer for these pictures. You're welcome to use the images on the page you identified for publication as you described. If possible, please give photo credit to tiltup.com.

Send me a copy when it's done – we'd love to see it.

Thanks,
Jeff Schaefer

From: Robee Ybanez [mailto:robeey@ksu.edu]
Sent: Thursday, February 23, 2017 12:50 PM
Subject: Website Inquiry - Other inquiry

Inquiry From GeneralContractor.com:	
Name:	Robee Ybanez
Company:	Kansas State University
Phone:	785-317-1722
Message:	Greetings! I am writing to ask permission to copy some images in digital form and incorporate into an electronic publication for my graduate studies report from the following URL: http://www.tiltup.com/construction-project-photos/tiltup-panel-building.asp The material will be distributed/published as follows: Distribution: Published electronically through K-State Research Exchange Expected distribution/publication date: May 2017 Target market: K-State students, staff, and faculty If you do not solely control copyright in the requested materials, I would appreciate any information you can provide about others to whom I should write, including most recent contact information, if available. Sincerely, Robee Ybanez Graduate Student Architectural Engineering Kansas State University

RE: Website Inquiry - Other inquiry

Jeff Schaefer

Thu 2/23/2017 1:48 PM

To:Robee Ybanez <robeey@ksu.edu>;

I forgot to mention – if you want larger versions of these pictures for printing, look through the tilt-up construction section of constructionphotographs.com at http://constructionphotographs.com/stock-photos/tilt-up-construction-tiltwall-pictures/index.asp?pNum=1

You should find most if not all of the pictures at the original higher resolution.

Just find a photo and request the original image – a link will be sent to you for the file.

From: Robee Ybanez [mailto:robeey@ksu.edu]
Sent: Thursday, February 23, 2017 12:50 PM
To: Phillip Bell <pbell@generalcontractor.com>; Ed McGuire <emcguire@generalcontractor.com>; Kyle Whitesell <kwhitesell@generalcontractor.com>; Larry Knox <lknox@generalcontractor.com>; Mike Moore <mmoore@generalcontractor.com>; Jeff Schaefer <JSchaefer@generalcontractor.com>
Cc: robeey@ksu.edu
Subject: Website Inquiry - Other inquiry

Inquiry From GeneralContractor.com:	
Name:	Robee Ybanez
Company:	Kansas State University
Phone:	785-317-1722
Message:	Greetings! I am writing to ask permission to copy some images in digital form and incorporate into an electronic publication for my graduate studies report from the following URL: http://www.tiltup.com/construction-project-photos/tiltup-panel-building.asp The material will be distributed/published as follows: Distribution: Published electronically through K-State Research Exchange Expected distribution/publication date: May 2017 Target market: K-State students, staff, and faculty If you do not solely control copyright in the requested materials, I would appreciate any information you can provide about others to whom I should write, including most recent contact information, if available. Sincerely, Robee Ybanez Graduate Student Architectural Engineering Kansas State University

Re: Website Inquiry - Other inquiry

Jeff Schaefer

Fri 3/3/2017 5:06 PM

To:Robee Ybanez <robeey@ksu.edu>;

Hi robee- I am the photographer for virtually all the pictures on our website. You are welcome to use the photographs that you specified in your project.

General contractor.com and tilt up.com are both websites on by Bob Moore construction. They share a lot of images. You are welcome to use pictures from either side for your project. Many of the pictures showing construction work can be found on construction photographs.com, which is also our website.

Sent from my iPhone

On Mar 3, 2017, at 4:44 PM, Robee Ybanez <robeey@ksu.edu> wrote:

> Hi Jeff,
>
> I was wondering if you could help me. I would like to use some of the images in the following url: http://www.generalcontractor.com/construction-projects/tilt-up.asp
> Who do I need to contact or ask permission to use these? Thank you.
>
> ---
>
> Tilt-up Construction and Precast Concrete Building ...
>
> www.generalcontractor.com
>
> Tilt-Up Construction and Precast Concrete Construction Project Showcase. Tilt-up Construction (also called Tilt Wall Construction): An innovative, cost-effective way ...
>
> ---