THE EFFECTS FOUNDATION OPTIONS HAVE ON THE DESIGN OF LOAD-BEARING TILT-UP CONCRETE WALL PANELS

by

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Abstract

Soils conditions vary throughout the United States and effect the behavior of the foundation system for building structures. The structural engineer needs to design a foundation system for a superstructure that is compatible with the soil conditions present at the site. Foundation systems can be classified as shallow and deep, and behave differently with different soils. Shallow foundation systems are typically used on sites with stiff soils, such as compacted sands or firm silts. Deep foundation systems are typically used on sites with soft soils, such as loose sands and expansive clays.

A parametric study is performed within this report analyzing tilt-up concrete structures in Dallas, Texas, Denver, Colorado, and Kansas City, Missouri to determine the most economical tilt-up wall panel and foundation support system. These three locations represent a broad region within the Midwest of low-seismic activity, enabling the use of Ordinary Precast Wall Panels for the lateral force resisting system. Tilt-up wall panels are slender load-bearing walls constructed of reinforced concrete, cast on site, and lifted into their final position. Both a 32 ft (9.75 m) and 40 ft (12 m) tilt-up wall panel height are designed on three foundation systems: spread footings, continuous footings, and drilled piers. These two wall heights are typical for single-story or two-story structures and industrial warehouse projects. Spread footings and continuous footings are shallow foundation systems and drilled piers are a deep foundation system. Dallas and Denver both have vast presence of expansive soils while Kansas City has more abundant stiff soils.

The analysis procedure used for the design of the tilt-up wall panels is the Alternative Design of Slender Walls in the American Concrete Institute standard ACI 318-05 Building Code and Commentary Section 14.8. Tilt-up wall panel design is typically controlled by lateral instability as a result from lateral loads combining with the axial loads to produce secondary moments. The provisions in the Alternative Design of Slender Walls consider progressive collapse of the wall panel from the increased deflection resulting from the secondary moments. Each tilt-up wall panel type studied is designed in each of the three locations on each foundation system type and the most economical section is recommended.

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Acknowledgements

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Dedication

This report is dedicated to my parents, Mike and Marianne Schmitt, who have given me unconditional support and who have inspired me to do the best I can to become a successful asset to society.

1 Introduction

This report includes a parametric study evaluating two load-bearing tilt-up structures to determine the most economical foundation design option for its given location. A common tilt-up structure, a warehouse or two story commercial building, with dimensions of 216 ft x 96 ft (66 m x 29 m) illustrated in Figure 1-1, is evaluated with wall panels at 32 ft (9.75 m) in height and 40 ft (12 m) in height, and designed on three separate foundation systems for each height. These three foundation systems are spread footings, continuous footings, and drilled piers. Each of these structures is designed in three locations: Dallas, Texas, Denver, Colorado, and Kansas City, Missouri.

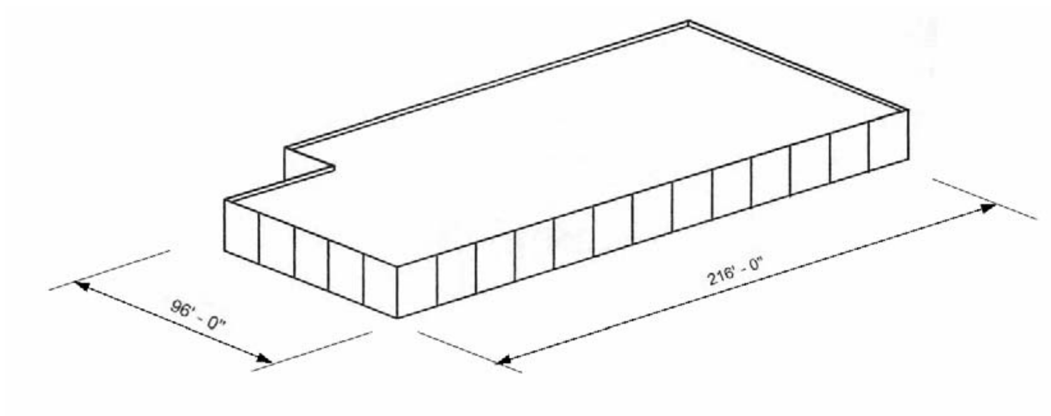

Figure 1-1. Tilt-Up Building Structure
Courtesy of the Structural Engineers Association of California
(SEAOC 2006)

This report begins with an overview of soil types, soil properties, and how the soil conditions vary with region. A discussion of shallow and deep foundations follows with common practices for given soil conditions. Then a discussion of the history of tilt-up construction provides the reader with the practice and evolution of tilt-up panel design to present date. The analysis procedure used to design the tilt-up wall panels in this parametric study is in accordance with the American Concrete Institute standard ACI 318-05 Building Code and Commentary Section 14.8, Alternative Design of Slender Walls. The parametric study, design of the 32 ft (9.75 m) high structure and 40 ft (12 m) high structure, is presented for the three foundation systems in each location, discussing load paths through the system. Common connection details are provided to illustrate the current industry practice of connecting the tilt-up panel to the foundation. The report concludes with recommendations for tilt-up wall panel designs for each respective location.

This report focuses on the connection between the tilt-up wall panel and the foundation. The construction design of tilt-up panels, including lifting, bracing, and panel-to-panel connections, is not within the scope of this report.

2 Scope of Research

This report discusses three foundation systems which are common for supporting tilt-up wall panels, and evaluates the effect each system has on the tilt-up wall panel design. The foundation systems evaluated will include two shallow foundation options and one deep foundation option. The two shallow foundations are spread footings and continuous footings, and drilled piers are used for the deep foundation. The report also evaluates the effect soil condition has on the foundation system chosen. The structure evaluated in this parametric study is designed in three locations: Dallas, Texas, Denver, Colorado, and Kansas City, Missouri. These locations are chosen from the following parameters set.

For evaluation of the load-bearing tilt-up wall panels, the tilt-up building from the 2006 IBC Structural/Seismic Design Manual, Vol. II is used. The scope of this report covers the tilt-up structure located within the region shown in Figure 2-1. This region is formed from the following parameters. The structure is located in Site Class D or E, as defined in ASCE 7-05, Chapter 20. Site class D includes regions with stiff soil and is used in regions where the soil properties are not known in sufficient detail to determine its respective class. This allows the structure to be located within many areas in the Midwest, such Missouri. Site Class E includes regions with soft clay soil. This allows for the structure to be located in regions with expansive soils, such as Colorado and Texas. The structure could also be conservatively located within Site Class A, B, or C, which are defined as hard rock, rock, and very dense soil and soft rock, respectively. These three site classes contain soils of higher strength than Site Class D or E.

Tilt-up panels are slender load-bearing walls constructed of reinforced concrete, cast on site, and lifted into their final position. A seismic force resisting system of Ordinary Precast Shear Walls is chosen for this parametric study. Tilt-up panels can be classified as precast panels because they are not cast in their final position. The bearing wall systems listed in ASCE 7-05 Table 12.2-1 do not include a seismic force-resisting system pertaining directly to tilt-up panels; therefore, the precast shear wall designations can be used for tilt-up panels. Precast wall panels are cast at a manufacturing plant under quality-controlled conditions. This characteristic separates precast panels from tilt-up panels. Tilt-up panels are cast on site and the quality control tolerances are less than precast manufacturing plants. This will exclude any special seismic detailing requirements needed for the tilt-up wall panel design. In order to designate the tilt-up wall panel as Ordinary Precast Shear Walls, the structure must be located in Seismic Design Category A or B. Seismic Design Categories C and higher require Intermediate precast shear walls to be used for the seismic force resisting system. An Occupancy Category II, standard for a warehouse without hazardous materials or a small commercial building, will be used for the structure; therefore, no occupancies listed within ASCE 7-05 Table 1-1 will exist in the structure. For example, this excludes buildings occupying 300 or more people, school facilities occupying more than 250 people, hospital and healthcare facilities, emergency response facilities, and power generation stations. An Importance Factor for wind and seismic loads of 1.0 will be used accordingly with Occupancy Category II. From these parameters, the 0.2s and 1.0s spectral response accelerations, S_s and S_1, respectively, was derived. The structure in this report is located in regions with S_s values less than or equal to 0.3g and S_1 values less than or equal to 0.08g. The structure is also limited to regions where the Basic Wind Speed has a 3-second gust wind speed of less than or equal to 90 mph. This allows for the structure to be located in non-hurricane prone regions, which is typical for Colorado, Missouri, and non-coastal regions of Texas. The areas of Colorado shaded red in Figure 2-1 are special snow and wind regions and shall be examined for unusual snow and wind conditions. Surface Roughness Category B will be used for wind design, due to the generality of the tilt-up structure being located within an urban or suburban area with other structures nearby. The structure will be located within regions of Ground Snow Loads less than or equal to 30 psf.

The combination of these parameters allows for use of the Equivalent Lateral Force Procedure in ASCE 7-05 Section 12.8. From the region defined by the existing parameters, the three regions of Texas, Colorado, and Missouri are chosen for the location of the structure. The design procedures within this parametric study allow for a structure to be located within areas not blacked-out in Figure 2-1.

The structure within this report will be evaluated twice, once with panel heights of 32 ft (9.75 m) and once with panel heights 40 ft (12 m). These two heights were chosen because a 32 ft (9.75 m) panel height is a common height for either a single-story or two-story structure, while a 40 ft (12 m) panel height represents typical warehouse projects. The wall components will consist of normal weight concrete with a compressive strength of 4 ksi and density of 150 pcf and Grade 60 rebar with yield strength of 60 ksi. The bottom of footing elevations for Denver and Kansas City are 36 in (90 cm) below ground surface and the bottom of footing elevation for Dallas is 18 in (45 cm) below ground surface. Setting the bottom of the foundation below the frost depth prevents the foundation from heaving due to moisture. Due to these restraints, the building structures in Denver and Kansas City are 18 in (45 cm) shorter than the building structure in Dallas, but the panel heights are equivalent in all three locations.

Figure 2-1. Applicable Regional Map

(Areas highlighted in white are considered for the parametric study, areas highlighted in red shall be examined for unusual snow and wind conditions, and areas highlighted in black are outside the scope of this report.)

IMAP 1940 – Swelling Clays Map of Conterminous United States

Courtesy of the U.S. Geological Survey

Department of the Interior/USGS

U.S. Geological Survey/Map by Olive, Chleborad, Frahme, Schlocker, Schneider, and Schuster

The USGS home page is http://www.usgs.gov

Figure 2-2 illustrates the presence of expansive soils for the regions applicable to the scope of this parametric study. Over 50 percent of the areas shaded red are underlain by soils with abundant clays of high swelling potential. Less than 50 percent of the areas shaded blue are underlain by soils with clays of high swelling potential. Over 50 percent of the areas shaded orange are underlain by soils with abundant clays of slight to moderate swelling potential. Less than 50 percent of the areas shaded green are underlain by soils with abundant clays of slight to moderate swelling potential. The areas shaded brown are underlain by soils with little to no clays with swelling potential. (Olive 1989)

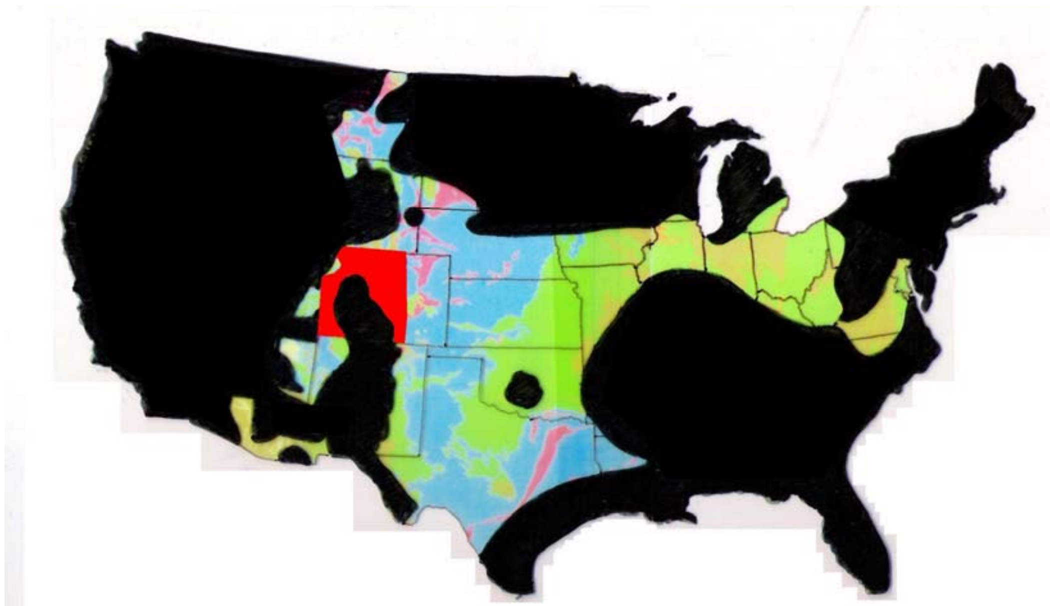

Figure 2-2. Applicable Regional Map for Expansive Soils

IMAP 1940 – Swelling Clays Map of Conterminous United States

Courtesy of the U.S. Geological Survey

Department of the Interior/USGS

U.S. Geological Survey/Map by Olive, Chleborad, Frahme, Schlocker, Schneider, and Schuster

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3 Soil Types and Properties

Rock and soil are two materials that compose the earth's crust. Rock is defined as a natural aggregate of minerals connected by strong and permanent cohesive forces. Soil is defined as a natural aggregate of mineral grains that have resulted from the weathering of rock. The soil types existing today are the result of rock exposure to air, water, chemical solutions, varying temperatures, and wind. Soil is a three-phase particulate material compiled of solids, liquids, and gases. (Terzaghi 1948)

Rock types are classified into three major categories on the basis of their origin and formation process. These three rock types include igneous, sedimentary, and metamorphic.

Igneous rock is the result of the cooling and solidification of molten magma from deep within the earth's mantle. Rapid cooling causes the mineral components to coagulate into small crystals and form a fine texture. This rapid cooling occurs when magma escapes through volcanoes and fissures in the earth's crust. When the magma cools rapidly at or near the earth's surface, the igneous rock is classified as extrusive and includes basalts, rhyolites, and andesites. When the magma is trapped within the earth's surface, it is cooled slowly and causes the mineral components to form large crystals and possess a course texture. This igneous rock is classified as intrusive and includes granites, syenites, diorites, and gabbros. When the igneous rock is composed primarily of quartz or silica, the rock decomposes to a course textured sandy or gravelly soil. Granites and rhyolites possess this characteristic. When the igneous rock is composed primarily of iron, magnesium, calcium, or sodium, but little silica, the rock decomposes to a fine textured silt or clay soil. Clay soil properties and behaviors are different from gravel, sand, and silt soils because the clay soil is a result of the primary igneous rock minerals decomposing into secondary minerals. Gravel, sand, and silt are fragments of the original igneous parent rock. (McCarthy 1977)

Sedimentary rock is the product of gravel, sand, silt, and clay deposits formed by weathering that have become hardened by pressure or cemented by minerals. Pressure from the weight of thick overlying soils or from glaciers compact and consolidate to form strong attractive bonds. Cementing agents such as silica, calcium carbonate, and iron oxides are generally carried in solution by groundwater. They fill the voids between particles to form sedimentary rock. Most of the United States was under water in prehistoric times. Over time, the land in the northeast and along the west coast rose, but the land in the central and southern areas along with the east cost remained beneath shallow seas. Limestone, shales, and sandstones formed from accumulated sediments in these shallow seas. Limestone is predominately crystalline calcium carbonate formed beneath water and often includes impurities such as clays and organic material. Limestone rocks can be a good foundation and construction material when the formation is sound and free of cavities. Shale is predominately formed from deposited clay and silt soils. It is estimated that shale represents approximately 50 percent of the rock at or near the earth's surface. Sandstone is predominately formed from the cementing of quartz with silica, calcium carbonate, or iron compounds. (McCarthy 1977)

Metamorphic rock is the product of metamorphism, the process of changing the composition and texture of rocks by heat and pressure. Metamorphic rock results when the rock structure and mineral composition of igneous or sedimentary rock is changed from combinations of heat and pressure. Under extreme heat and pressure, metamorphic rock may melt to form magma, therefore allowing the cycle to be repeated. (McCarthy 1977)

Bedrock is the term used to describe the parent rock of soil, which is generally rock at a depth within the ground where a structure may be founded. All other rock and soils are formed from this bedrock through the cooling of magma or through weathering. Igneous rock lay at the lowermost part of the bedrock. More recently formed layers of sedimentary rock lay above the igneous rock. In some locations between these two layers, metamorphic rock may exist formed by the intense heat and pressure acting on the sedimentary rock. (Bowles 1988)

Soil is the by-product of mechanical and chemical weathering of rock. Two categories, residual and transported, are used to classify soil. Residual soils are formed from the weathering of parent rock and remain at the original location. Weathering of rock occurs from two methods, mechanical and chemical. Mechanical weathering includes the effects of wind, rain, moving water, and plate tectonic forces. Chemical weathering includes exposure to atmosphere and temperature changes. Residual soils usually contain angular rock fragments of a wide range of sizes, shapes, and composition. Transported soils are formed from the weathering of parent rock at its original location and have been transported by wind, water, glaciers, or gravity to the present site. The following terms are commonly used by engineers and construction personnel as a means of classifying soil types. (McCarthy 1977)

Boulders are cohesionless aggregates fractured from its parent rock material. Fragments with a diameter of 8 in (200 mm) or more fall into this category. Boulders may cause excavation problems at or near the earth's surface and problems in soil exploration or pier drilling at greater depths within the earth. Gravels are cohesionless aggregates with diameters of 1/8 in to 8 in (3 mm to 200 mm). Sands are cohesionless aggregates of rounded subangular fragments with diameters less than 1/8 in (3 mm). Boulders, gravels, and sands are all considered course-grained soil and the individual particles are frequently very irregular in shape. (Terzaghi 1948)

Silts are inert by-products of rock weathering with particle ranges of 2.9×10^{-3} in (0.074 mm) to 3.94×10^{-5} in (0.001 mm). Silt may be found in forms of inorganic silts or organic silts. Inorganic silt is a fine-grained soil with a smooth texture and little or no plasticity. Inorganic silt is impervious and may rise into a drill shaft as a viscous fluid. Organic silt is also a fine-grained plastic soil, but contains an admixture of finely grained organic material. The permeability of organic silt is very low and its compressibility is very high. (Terzaghi 1948)

Clays are fine-grained soils found in particle sizes less than 7.87×10^{-5} (0.002 mm). Clay mineral size overlaps that of silts, but the fundamental difference between the two is that clays are not inert. Almost all clay minerals are crystalline minerals capable of developing cohesion and plasticity. Clays can also be found in forms of inorganic clays or organic clays. Inorganic clay is plastic with an extremely low permeability. Organic clay contains finely graded organic matter. The compressibility of organic clay is very high when saturated, but when dry its strength is very high. The presence or absence of water can produce drastic volume and strength changes because the clay mineral has a high affinity for water and the individual particles may absorb 100 times its volume. (Terzaghi 1948)

Clay soils consist of clay minerals. These clay minerals are complex aluminum silicates. Three principle clay minerals are montmorillonites, illites, and kaolinites. These three principle clay minerals can be classified by their plasticity index. The plasticity index of a soil is the difference between the liquid limit and plastic limit of the soil. The liquid limit is the point of transition of the moisture content of a soil from the plastic to liquid state. The plastic limit is the point of transition of the moisture content of a soil from the semisolid to plastic state. Soil behavior can be classified into four basic states: solid, semisolid, plastic, and liquid. The moisture content of the soil is lowest in the solid state and highest in the liquid state. (Das 2006)

Montmorillonites are the most active of the clay minerals existing in clay soils. The plasticity index of pure montmorillonites is 150 and greater. Montmorillonites are composed of an alumina sheet sandwiched between two silica sheets to form a layer with a weak bond. Clay readily absorbs water between these layers because of the weak bond, which allows the mineral to possess large volume changes. The affinity for water and swell of montmorillonites makes it an ideal drilling mud for soil exploration and pier drilling. Injection of montmorillonites into the ground around basement walls as a water barrier is common because the mineral swells to close off water flow paths. Montmorillonites are found mostly in arid and semiarid regions. Clays weather into less active minerals, therefore montmorillonites weather into illites. (McCarthy 1977)

Illites are intermediate in activity. The plasticity index of pure illites falls in the range of 30 to 50. Illites are composed of an alumina sheet sandwiched between two silica sheets to form a layer bonded by potassium. Illites do not expand when exposed to water unless a deficiency in the potassium bond exists. Illites weather into kaolinites. (Bowles 1988)

Kaolinites are the least active yet most prevalent clay mineral. The plasticity index of pure kaolinites ranges from 15 to 20. Kaolinites are composed of one silica sheet and one alumina sheet. A very strong hydrogen bond holds these two layers together. This mineral is very stable and resists volume change when exposed to water because of the strong hydrogen bond that exists between the silica and alumina sheets. (Bowles 1988)

Soils containing varying degrees of these clay minerals, specifically the active mineral montmorillonite, are known in the engineering profession as expansive soils. Expansive soils have the capacity to undergo volumetric changes when subjected to variances in water content. The expansive soil will swell when the water content is increased, and the soil will shrink when the water content is decreased. The expansion is caused by hydration and attraction of water molecules into the crystal lattice of the clay minerals. When this process is reversed and the water is removed from the crystal lattice, soil shrinkage occurs. Expansive soils are most active in geographic areas where the seasonal climate changes drastically with long droughts alternating with excessive rainfalls. Vegetation may also cause shrinkage to expansive soils. Trees with high water demand are the most common cause of soil shrinkage from vegetation. Other factors that influence expansive soils include changes in the field environment from natural conditions due to construction practices. (Krohn)

4 Soils by Region

Soil site classification used in this report has been set by the American Society of Civil Engineers standard ASCE 7-05 Minimum Design Loads for Buildings and Other Structures in Chapter 20: Site Classification Procedure for Seismic Design. Table 20.3-1 lists six site classes, A through F, for different soil parameters investigated on site. The following classes are listed corresponding with their respective site properties: Site Class A, hard rock; Site Class B, rock; Site Class C; very dense soil and soft rock; Site Class D, stiff soil; Site Class E, soft clay soil; and Site Class F, soils requiring site response analysis, such as liquefiable soils. This parametric study pertains to Site Classes A, B, C, D, and E, but is focused on Site Classes D and E. These site classes allow for Ordinary Precast Shear Walls to be used for the lateral force resisting system in conjunction with the other parameters set.

Stiff soils in Site Class D include those with aggregates structured in a dense manner providing a low void ratio. For construction purposes, it is generally recognized that the smaller the void ratio, or more dense the soil, the higher the strength and the lower the compressibility will be of the soil. Site Class D is generally applicable for shallow foundations. (McCarthy 1977)

Soft clay soils in Site Class E include those with aggregates structured in a loose manner providing a high void ratio. Clay deposits will have high void ratios, low density, and capabilities for high water contents. The clay structure is however quite strong and resistant to external forces due to the attraction between the particles. (McCarthy 1977)

4.1 Region 1: Denver, Colorado

All three rock types – igneous, sedimentary, and metamorphic – are widely spread throughout the state of Colorado. The geologic framework and structure of Colorado can be attributed to two major events in history: the Laramide Orogeny uplift and the Cretaceous-Tertiary boundary. The mountain uplifts and intervening basins were developed during the Laramide Orogeny about 50 to 70 million years ago. Denver is located between the east front of the Southern Rocky Mountains and the Colorado Piedmont, the west edge of the central stable area of the United States known as the Great Plains. The erosion of the Rocky Mountains deposited Tertiary sediment cover onto the Great Plains. This sediment has been eroded by the

South Platte and Arkansas River systems within Denver. Underlying Cretaceous bedrock has been exposed due to the erosion caused by the rivers, creating a broadly rolling topography with local scarps where resistant bedrock units outcrop. The Front Range of the Southern Rocky Mountains, which stretches from Colorado Springs northwards to Fort Collins, lay west of Denver. Precambrian crystalline rocks reaching elevations of 14,000 ft (4.27 km) compose the Front Range. The foothills, where the mountains meet the Great Plains, consist of steeply dipping Paleozoic and Mesozoic sedimentary rocks. The Golden Fault lies here and separates the mountains from the plains. (Costa 1982)

The mountains of the Front Range directly west of Denver consist of Precambrian granites, metamorphic igneous and sedimentary rocks, and volcanic rocks. The Rocky Mountains were created from the uplift in Colorado during the Laramide Orogeny. As the mountains rose, surface land east of the Front Range settled, forming the Denver Basin. Therefore, some rocks at the surface of the mountains a few miles west of Denver lay thousands of feet below the ground surface under Denver. The Denver Basin became the site of deposition for sediments eroded from the mountains. (Costa 1982)

The Denver Formation varies drastically in texture and composition. The majority of sediments deposited here include tuffaceous silty claystone, arkose, and conglomerates. The sediments are composed of compacted volcanic ash. Bedrock of the Denver Formation is roughly 780 ft (240 m) thick and composed of primarily interstratified weakly bonded claystones, siltstones, and sandstones. The bedrock has been highly over-consolidated by the weight of over 980 ft (300 m) of older sedimentary rocks. Clays with high percentages of montmorillonites derived from the Denver Formation. These clays and the volcanic material of the tuffaceous deposits are the source of the expansive soils which exist near Denver today. (Lufkin 2006)(Kumar 1984)(Costa 1982)

Colorado has a semiarid climate with hot summers and cold winters. This alternating pattern is due to the states location far inland from any ocean. The diverse topography influences the patterns of precipitation, temperature, and air movement. Precipitation varies tremendously in Colorado due to its vast differences in topography. This variability in annual precipitation from year-to-year, along with periodic droughts, causes concerns for engineers and construction personnel. The eastern plains, along with the Denver area, receive less precipitation than the mountains, and have high rates of evapotranspiration. The high rates of evapotranspiration are due to the abundant sunshine, clear skies, and low relative humidity in this semiarid region. As a result, the clay-rich soils in Denver are typically dry and may have high swell potentials in their natural state. (Noe, Mathews 2003)

Serious problems may result for structures built on expansive soils from the presence or change in moisture content of the subsurface. In Denver, the amount of subsurface water content increases during late winter and spring. These seasons are when the rates of natural infiltration from precipitation are high. The fall and early winter are the dry seasons, and the subsurface water content decreases. The depth below the ground surface where the alternating moisture content influences the soil is called the active zone. The depth of the active zone to bedrock below the city of Denver averages from 20 ft to 40 ft (6 m to 12 m). (Noe 2003)(Costa 1982)

Figure 4-1 illustrates the presence of expansive soils for the state of Colorado. Over 50 percent of the areas shaded red are underlain by soils with abundant clays of high swelling potential. Less than 50 percent of the areas shaded blue are underlain by soils with clays of high swelling potential. Over 50 percent of the areas shaded orange are underlain by soils with abundant clays of slight to moderate swelling potential. Less than 50 percent of the areas shaded green are underlain by soils with abundant clays of slight to moderate swelling potential. The areas shaded brown are underlain by soils with little to no clays with swelling potential.

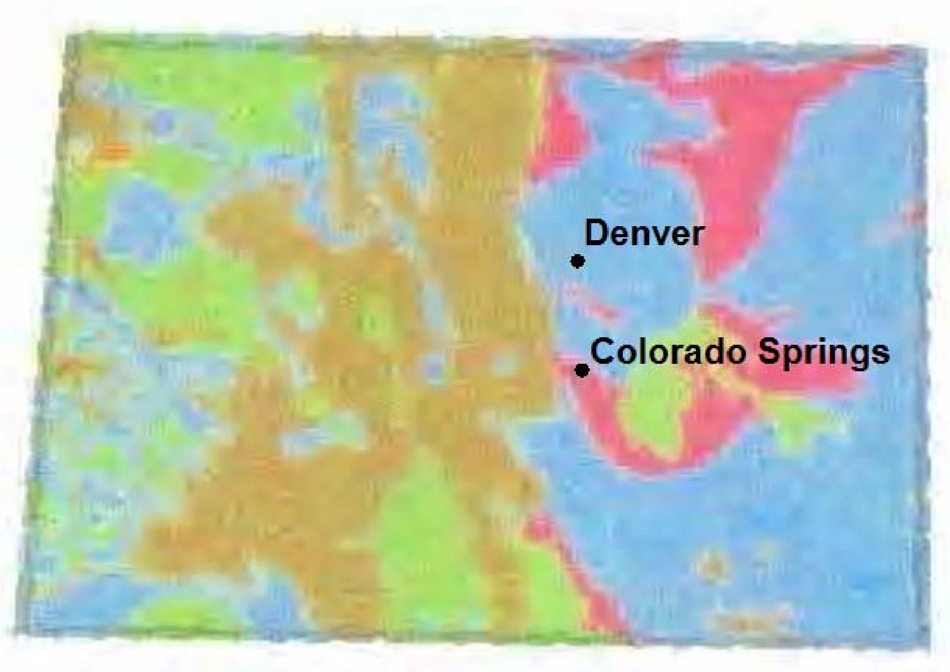

Figure 4-1. Expansive Soils in Colorado

IMAP 1940 – Swelling Clays Map of Conterminous United States

Courtesy of the U.S. Geological Survey

Department of the Interior/USGS

U.S. Geological Survey/Map by Olive, Chleborad, Frahme, Schlocker, Schneider, and Schuster

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4.2 Region 2: Dallas, Texas

The development of the soils lying beneath Dallas, Texas has been impacted by three significant geologic conditions. The city lies in a valley formed by differential erosion of marine bedrock including Cretaceous shale, chalk, and marl. The city has been covered with residual soils due to seventy million years of surface exposure. The Trinity River, including its three branches which join west of the city and flow through Dallas, has carved its main flood plain into the Austin Chalk. This has created valleys now infilled with five terraced alluvial units. (Allen 1986)

Dallas is located in the midst of the Gulf Coastal Plain, the East Texas Embayment, the western limit of the Ouachita Folded Belt, and the Balcones Fault System. Sediments of the Cretaceous age extend from the surface to a depth of 2,000 ft to 4,500 ft (600 m to 1400 m), thickening from west to east. These sediments lay over Paleozoic rock. Some important outcroppings within the city formed include the Eagle Ford Shale, the Austin Chalk, and the Ozan Formation. (Allen 1986)

The Eagle Ford Shale is a weak rock unit consisting mostly of calcareous and noncalcareous clays. The Eagle Ford Shale has a depth of approximately 475 ft (150 m) underneath Dallas. The Austin Chalk is subdivided into the lower chalk, middle marl, and upper chalk. The three divisions consist of weathered chalk beds alternated with beds of marl. The upper and lower divisions of the Austin Chalk have a higher percentage of chalk beds than marl beds. The center division of the Austin Chalk has a higher percentage of marl beds than chalk beds. The maximum thickness of the Austin Chalk under Dallas reaches approximately 550 ft (170 m) below the surface. The Ozan Formation outcrops, positioned in the east portion of Dallas, consist of soft, montmorillonitic, marine shale. The thickness of the Ozan Formation is estimated to reach a depth of 100 ft below the surface. (Allen 1986)

The bedrock throughout most of Dallas is covered with 20 in to 80 in (50 cm to 200 cm) of silty clay and clay residual soils. The thickest layers of these soils lay over the Eagle Ford Shale, the center of the Austin Chalk, and the Ozan Formation. The remainder of the bedrock under the city is covered by alluvium, which ranges from silty clays to impervious clays to clayey sands. The alluvial cover varies in thickness of 5 ft to 15 ft (1.5 m to 4.5 m) along small tributaries and 55 ft to 90 ft (17 m to 27 m) along major streams. Much of downtown Dallas bedrock consists of alluvial deposits lying over the Austin Chalk. (Allen 1986)

Expansive soils within the residual soils and bedrock of the Eagle Ford Shale present the greatest variations of swells and shrinkage. The maximum swell may be on the order of 15 percent with pressures up the 26 kips per square foot. Close to 70 percent of Dallas is built on expansive, montmorillonitic clay soils with plasticity indices ranging from 20 to 60. These expansive soils are predominately residual soils derived from the weathering and erosion of the outcropping chalks and shales. The Austin Chalk contains less hazardous soil zones typically less than 40 in (100 cm). Sandier, less plastic soils are found in these Austin Chalk regions. (Allen 1986)

Dallas has a temperate climate from its location in the extreme northern portion of the humid subtropical belt from the Gulf of Mexico. This climate includes long, hot and dry summers, mild winters, and moderately wet springs and falls. The heaviest rainfall occurs in late summer and early fall from hurricanes moving inland off the Gulf of Mexico. The expansive clay soils in Dallas are compounded by this local climate. (Allen 1986)

Figure 4-2 illustrates the presence of expansive soils for the state of Texas. Over 50 percent of the areas shaded red are underlain by soils with abundant clays of high swelling potential. Less than 50 percent of the areas shaded blue are underlain by soils with clays of high swelling potential. Over 50 percent of the areas shaded orange are underlain by soils with abundant clays of slight to moderate swelling potential. Less than 50 percent of the areas shaded green are underlain by soils with abundant clays of slight to moderate swelling potential. The areas shaded brown are underlain by soils with little to no clays with swelling potential.

Figure 4-2. Expansive Soils in Texas

IMAP 1940 – Swelling Clays Map of Conterminous United States

Courtesy of the U.S. Geological Survey

Department of the Interior/USGS

U.S. Geological Survey/Map by Olive, Chleborad, Frahme, Schlocker, Schneider, and Schuster

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4.3 Region 3: Kansas City, Missouri

Kansas City is located at the juncture of the Kansas and Missouri Rivers, along the boundary that separates two physiographic regions. The region north of this juncture is the Glaciated and Dissected Plains and the region south of this juncture is the Scarped Plains. Kansas City also lays between the Ozarks in central Missouri and the Flint Hills in central Kansas, where several erosion escarpments occur. These escarpments are predominately 50 ft (15 m) or less and the lowlands created between the escarpments cut into shale. (Parizek 1975)

The Missouri River, which runs along the border between northeast Kansas and western Missouri and then extends east through Missouri, depicts a general termination line of where the vast ice sheets swept south from the Canadian Shield several times in the past million years. Glaciations have had a great influence on the formation of the Kansas City area geology because of the city's location at the south boundary of the ice. North of the river, deposits of glacial material are abundant within some uplands and valleys. These glacial deposits generally have a depth of 5 ft (1.5 m) below the surface, but have depths of 20 ft to 60 ft (6 m to 18 m) along the Missouri River bluffs. Windblown loess of 30 ft to 40 ft (9 m to 12 m) covers the surface near the river, and gradually decreases in thickness further from the river. (Parizek 1975)

Pennsylvanian-age rocks extend from north to south through western Missouri and eastern Kansas. Kansas City lies near the center of this 150 mile (240 km) wide band of rocks. The thickness of these rocks reaches a depth near 900 ft (275 m) below the surface, but the deepest exposures reach only depths of 400 ft (120 m). These Pennsylvanian rock layers below Kansas City are composed of alternating limestone and shale layers. The alternating layers have averages depths less than 10 ft (3 m) thick, but some limestone and shale layers reach thicknesses of 20 ft to 40 ft (6 m to 12 m) thick. The repetition of limestone and shale layers exemplifies a cyclic sedimentational feature referred to as a cyclothem. A cyclothem is a geological characteristic of abrupt changes from one rock type to another in vertical sequences. This feature indicates that during the formation of accumulated sediments into alternating layers, widespread and uniform environments existed. (Parizek 1975)

The Swope Formation, the geologic formation under Kansas City with the most influence on foundation engineering, has an average depth of 25 ft to 30 ft (7 m to 9 m) and consists of three rock types. These three rock types are represented by three layers; the lower Middle Creek limestone, the middle Hushpuckney shale, and the upper Bethany Falls limestone. The Middle

Creek has a thickness of 6 in to 2 ½ ft (15 cm to 75 cm) and consists of hard, fine-grained, fossilized limestone. The Hushpuckney has a thickness of 4 ft to 5 ½ ft (1 m to 1.5 m) and consists of fissile and calcareous shale. The Bethany Falls thickness ranges from 12 ft to 30 ft (3.5m to 9m) and is composed of two distinct limestone layers separated by a thin layer of shale. The lower limestone layer thickness averages 8 ft to 10 ft (2.5 m to 3 m) and consists of fine-grained and course-grained limestone. The upper limestone layer thickness averages 10 ft to 12 ft (3 m to 3.5 m) and consists of thick bedded limestone. (Parizek 1975)

Figure 4-3 illustrates the presence of expansive soils for the states of Kansas and Missouri. Over 50 percent of the areas shaded red are underlain by soils with abundant clays of high swelling potential. Less than 50 percent of the areas shaded blue are underlain by soils with clays of high swelling potential. Over 50 percent of the areas shaded orange are underlain by soils with abundant clays of slight to moderate swelling potential. Less than 50 percent of the areas shaded green are underlain by soils with abundant clays of slight to moderate swelling potential. The areas shaded brown are underlain by soils with little to no clays with swelling potential.

Figure 4-3. Expansive Soils in Kansas and Missouri

IMAP 1940 – Swelling Clays Map of Conterminous United States

Courtesy of the U.S. Geological Survey

Department of the Interior/USGS

U.S. Geological Survey/Map by Olive, Chleborad, Frahme, Schlocker, Schneider, and Schuster

The USGS home page is http://www.usgs.gov

5 Structural Foundation Systems

The earth beneath a building provides the support for the structure. The soil beneath the building interacts with the structure and affects the structures stability. Soil is weaker than other building materials, such as concrete, steel, and timber. In order to safely support loads from the structure, the load must be distributed amongst a large area or volume of soil. This load transfer is done through structural foundations. The major function of structural foundations is to properly transfer building loads into the earth such that the supporting soil is not overstressed nor undergoes deformations. These conditions could cause buildings to undergo excessive settlement. The structural foundation used is dependant on the supporting soil properties. Structural foundations perform properly only if the supporting soil behaves as assumed. (McCarthy 1977)

This parametric study evaluates three types of structural foundations: spread footings, continuous footings, and drilled piers. Spread footings and continuous footings are classified as shallow foundation systems. Drilled piers are classified as deep foundation systems. Shallow foundation systems include footings where the depth of the bearing area is generally less than the width of the bearing surface. Deep foundations have a depth greater than four times the width of the bearing surface, and transmit structural loads through upper zones of poor soil conditions to a depth where rock or desired soil conditions exist. This load is resisted by bearing at the base of the pier and/or by side friction along the pier adjacent to the soil. Spread footings support a single column or single load. Continuous footings are elongated shallow foundations that support a wall, a single row of columns, or other types of strip loadings. Spread footings and continuous footings are appropriately used in locations where the soil conditions consist of compacted sands or firm silts. Drilled piers are appropriately used in locations where the soil conditions consist of loose sands and clays. Drilled piers may also be used in compacted sands or firm silts where large loads occur or uplift forces act upon the foundation. (McCarthy 1977)

5.1 Shallow Foundation Systems

The purpose of shallow foundation systems is to distribute the design load transferred into the foundation over a considerable horizontal area just below the earth's surface. The two shallow foundation systems used for the tilt-up panels in this report are spread footings and continuous footings.

It was general practice years ago to make shallow foundations for heavy buildings one continuous bed of concrete, known as a mat or raft foundation. One example is the foundation bed of the Government Post Office and Custom House in Chicago, built in 1877. The foundation was a 3 ½ foot (1 m) thick plain concrete pad. Due to the extensive variation of load values from columns and walls, the building settled about 24 in (60 cm). The settlement was uneven throughout the building's footprint which resulted in the building being replaced after only 18 years in service. This pad is an extensive type of spread footing since it collects the entire structures load into one foundation and distributes into the soil. The spread footing in this report will be of isolated footings, which are typically located at columns to spread point loads laterally into the soil so that the stress intensity is reduced to a safe value for the soil to carry. (Jacoby 1941)

5.1.1 Spread Footings

Spread footings, illustrated in Figure 5-1, can be designed using several different structural materials. These materials include plain concrete, reinforced concrete, and masonry. Reinforced concrete is the most common material for spread footings because of its durability in a potentially hostile environment and economy. Reinforced concrete is the material used for all three foundation systems in this parametric study because it is the most commonly used material in construction practice. Spread footings are constructed as close to the ground surface as the building design allows. Restrictions for depth of spread footings include frost penetration, soil shrinkage and expansion, soil erosion, and local code allowances. (Bowles 1988, Jacoby 1941)

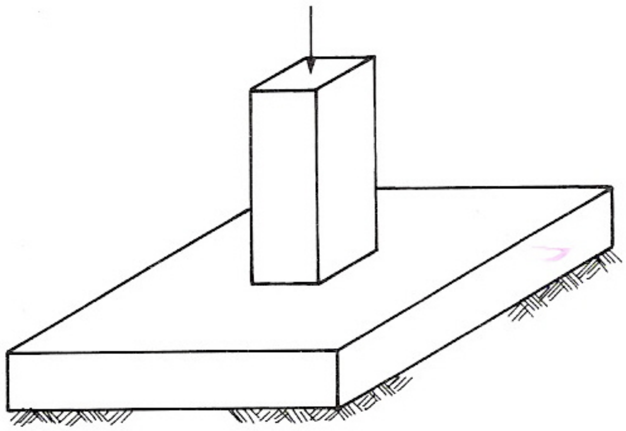

Figure 5-1. Spread Footing

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5.1.1.1 Soil Pressure Behavior

The soil beneath the footing foundation must be able to withstand the greatest pressure induced from the structure loading without failure of the loaded soil or excessive settlement of the footing foundation. The maximum load that can be applied to the foundation soil without rupture is called the bearing capacity. The current methods for determining this bearing capacity have evolved over many years.

Before the 19th century most large buildings had frameworks that could withstand large settlements without damage. These frames consisted of strong, flexible main walls interconnected at right angles by massive partition walls of equal flexibility. Foundation design was not given high consideration, whereas the option to increase support was to increase the wall thickness at the base. (Terzaghi 1948)

Throughout the 19th century, the development of the highly competitive industry led to a demand for large yet inexpensive buildings. These buildings were more susceptible to differential settlements than the massive predecessor buildings. Many regions most desirable for industrial buildings had been avoided in past years due to notoriously poor soil conditions. This led to a need for designers to follow a reliable procedure applicable under all soil conditions to find proportions for footings of a given building to resist the induced loads and experience nearly the same settlement. (Terzaghi 1948)

During the 1870's, the concept of an allowable soil pressure was developed. The concept was based on the fact that under similar soil conditions, footings distributing high intensity pressures to the soil generally settle more than footings distributing low intensity pressures. Designers began to observe the condition of buildings supported by footings that exerted various pressures into the subsoil. These designers concluded that the pressures in the soil beneath the footings that showed signs of damage due to settlement were too great for the given soil conditions. The maximum pressure recorded under footings not experiencing structural damage was considered satisfactory for design. This pressure was termed the allowable soil pressure or allowable bearing pressure. This empirical method consisted of allowable soil pressures for each soil type in a given location. (Terzaghi 1948)

The actual stress distribution beneath symmetrically loaded spread footings is not uniform. Figure 5-2 illustrates the stress distribution. Factors that affect the stress distribution include the footing rigidity and the base soil. Spread footings on loose sands have tendencies to displace the grains near the edge of the footing laterally while the interior soil grains remain confined, causing a greater pressure in the interior of the footing than the exterior, as illustrated in Figure 5-2A. For a more general case of rigid spread footings and for spread footings on clay, the edge pressure is higher than that of the interior pressure because edge shear must occur before any settlement begins. This action causes the soil under the footing to deflect in a bowl-shaped depression as the footing is loaded. This distribution relieves the pressure under the middle of the footing, as illustrated in Figure 5-2B. Soils have low rupture strengths and therefore it is likely that high edge shear stresses will not develop under the spread footings. Because the distribution of soil pressure under the footing is a function of the type of soil and the relative rigidity of the soil and the foundation, it has become common practice to use a linear pressure distribution beneath spread footings, as illustrated in Figure 5-2C. (Bowles 1988)

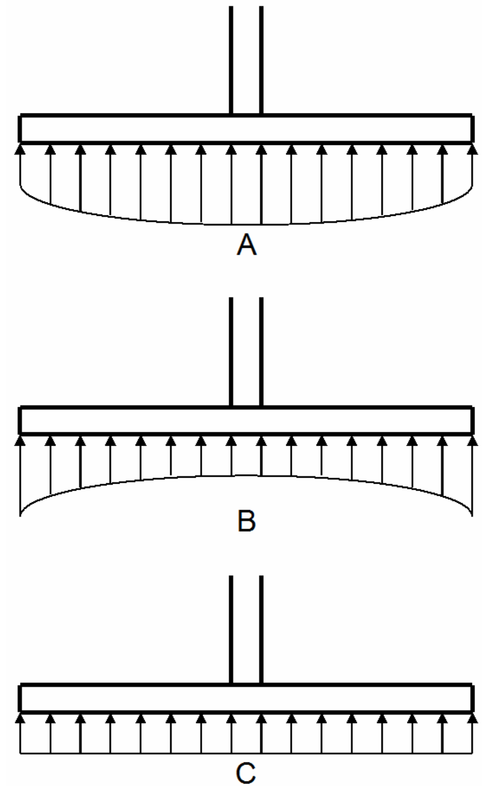

Figure 5-2. Soil Pressure Distribution

Figure Courtesy of John Wiley & Sons, Inc.

(McCormac 2006)

Spread footings may fail by three different primary modes. The first mode is a bearing failure of the footing, which is caused when the soil under the footing moves downwards and outwards from under the footing. The second mode is a serviceability failure, which is caused by excessive differential settlements of adjacent spread footings causing structural and architectural damage. The third mode is excessive total settlement of the entire structure, where multiple spread footings settle large amounts throughout the building. (MacGregor 1997)

5.1.1.2 Design Considerations

The design of a spread footing must consider load transfer from the column to the footing, which induces flexure, shear, bearing, and development of reinforcement. Footings must be designed to safely resist the effects of these actions. The design procedure also must take allowable soil bearing capacity and differential settlement into consideration. As mentioned in the previous section, spread footings may be assumed to be rigid, which results in a uniform soil pressure for concentric loadings, and a linear triangular or linear trapezoidal soil pressure distribution for eccentrically loaded footings.

The base area of the footing is determined by the allowable soil pressure capacity and building service loads. These service loads are unfactored dead, live, wind, earthquake, and other loads induced on the building. The area of the footing base must be large enough such that these external loads do not cause the soil beneath the footing to exceed its allowable soil pressure and cause the soil to fail. To proportion a spread footing in order not to exceed the allowable soil pressure, the base area of the footing, A_f, should be determined by dividing the unfactored service loads, P_a, by the allowable soil pressure, q_a. Unfactored loads are used since allowable stress design is used for soil design. Therefore, for a single concentrically loaded spread footing, $A_f = P_a/q_a$. (PCA 2005)

Flexural strength of spread footings is determined by the critical section for moment that occurs at the face of the column or wall support. This critical section is illustrated in Figure 5-3. The bending moment for each direction of the spread footing must be checked at this location and flexural reinforcement must be provided to resist this moment. Factored loads are used to determine the ultimate bending moment, M_u, in the spread footing. The amount of flexural reinforcement provided is determined by the design flexural strength, ϕM_n, required in order to exceed the total factored moment, M_u. (PCA 2005)

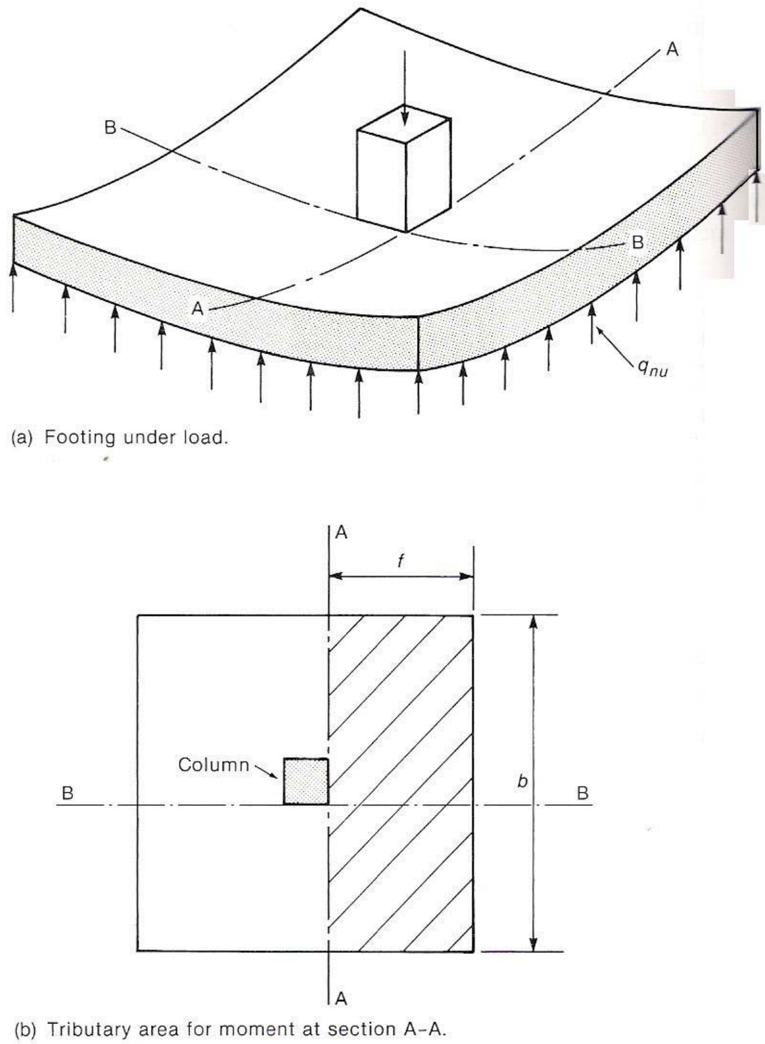

Figure 5-3. Flexural Action at Critical Section of Spread Footing

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Shear strength of spread footings must be determined from the governing case of the two conditions in which shear results. These two shear conditions are one-way shear (beam shear) and two-way shear (punching shear), and are illustrated in Figure 5-4. The depth of the footing is determined by the controlling shear condition. One-way shear assumes the spread footing acts as a wide beam with bending action in one direction. The critical section for one-way shear extends across the entire width of the footing. This critical section is located at a distance $\boldsymbol{d}$ from the face of the support. Two-way shear measures the diagonal tension caused by the column load on the footing. Two-way shear will not exist in spread footings supporting a continuous wall across its length. The critical section for two-way shear, the perimeter $\boldsymbol{b_o}$, is located at $\boldsymbol{d/2}$ from the column support. Factored loads are used to determine the shear force, $\boldsymbol{V_u}$, at the critical section. If $\boldsymbol{V_u}$ exceeds the governing shear strength, $\boldsymbol{\phi V_c}$, shear reinforcement must be provided. (PCA 2005)

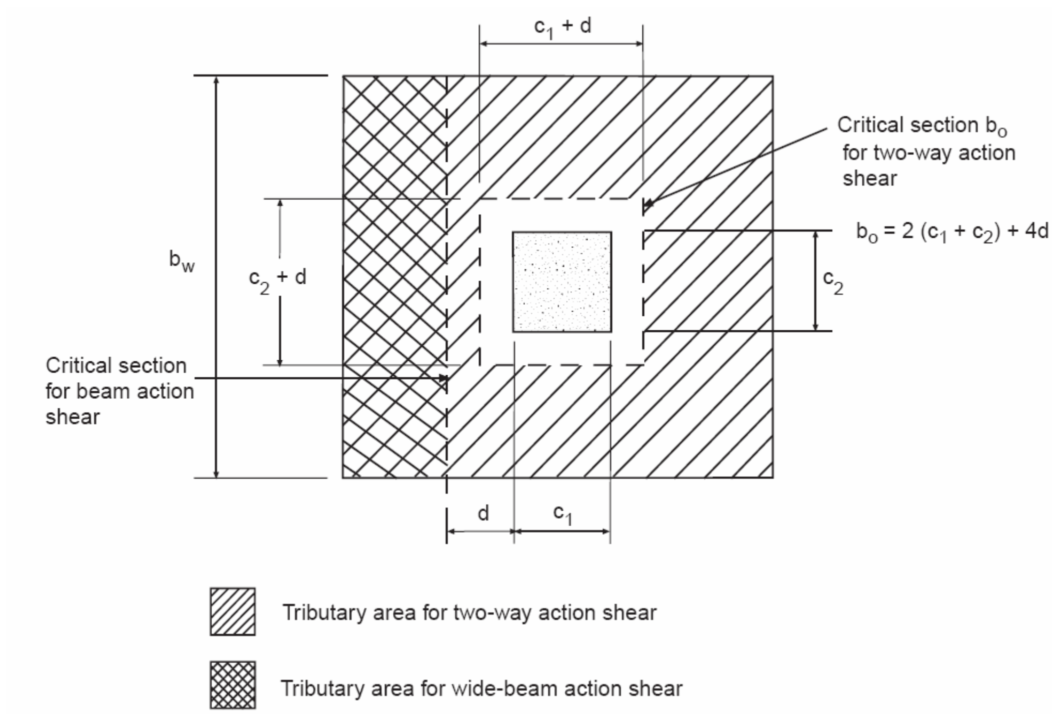

Figure 5-4. Critical Shear Sections in Spread Footings

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Development length is a concept that was first introduced into the American Concrete Institute Building Code Requirements for Reinforced Concrete and Commentary (ACI 318-71) standard in 1971. This concept replaced the dual requirements for flexural bond and anchorage bond. Development length is based on the achievable average bond stress over the length of reinforcement embedment, as illustrated in Figure 5-5. Highly stressed bars tend to split relatively thin sections of restrained concrete; therefore, the reinforcement must extend far enough on each side of the points of maximum bar stress to develop this stress. When the length of available concrete restricts the length of the reinforcement, hooked bars in tension may be used. In compression, hooks are ineffective and may not be used as anchorage. When determining spread footing reinforcement, it is assumed that the reinforcement stress yields along the maximum moment section at the face of the column or support. (MacGregor 1997)(PCA 2005)(ACI 2005)

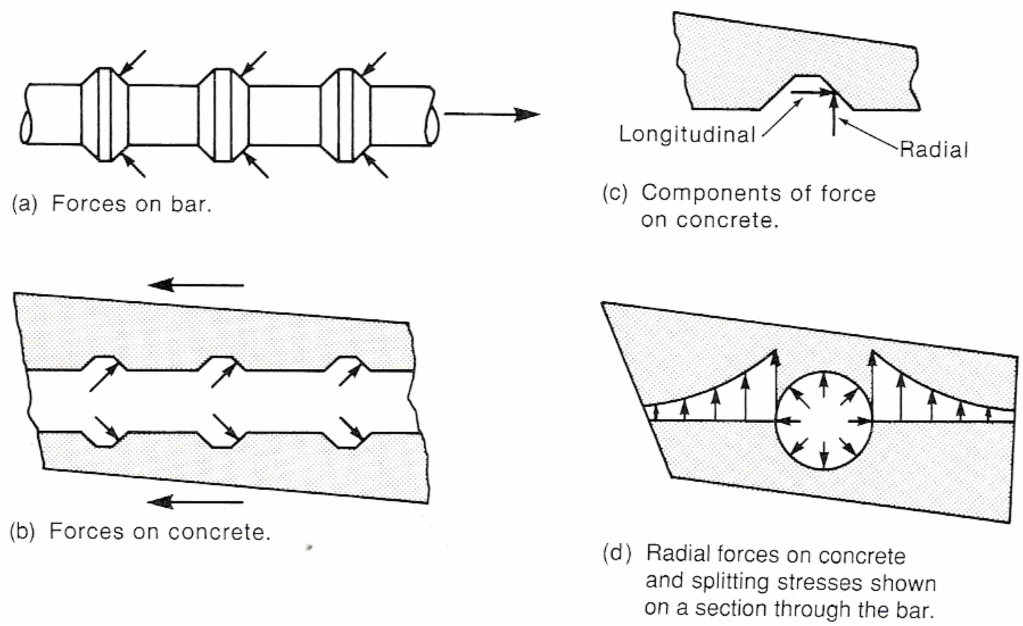

Figure 5-5. Bond Transfer Mechanism

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Load transfer from the column or wall into the footing is transmitted by bearing stresses in the concrete and by stresses in dowels that cross the joint. The bearing stresses occur within the area illustrated in Figure 5-6 and the dowels which cross the joint are illustrated in Figure 5-7. This joint is controlled by four modes of failure: crushing of the concrete at the base of the column or wall, crushing of concrete in the footing under the column or wall, bond failure of the dowels in the footing, and lap splice failure between the dowels and the column bars. The forces applied from the column or wall onto the footing must be transferred through concrete bearing and/or reinforcement. Tensile forces may only be resisted by the reinforcement. When the bearing strength of concrete is exceeded, reinforcing dowels must be provided to transfer the remainder of load. Minimum dowel reinforcement is required by the American Concrete Institute in order to provide resistance to shear at the joint. (MacGregor 1997)(PCA 2005)

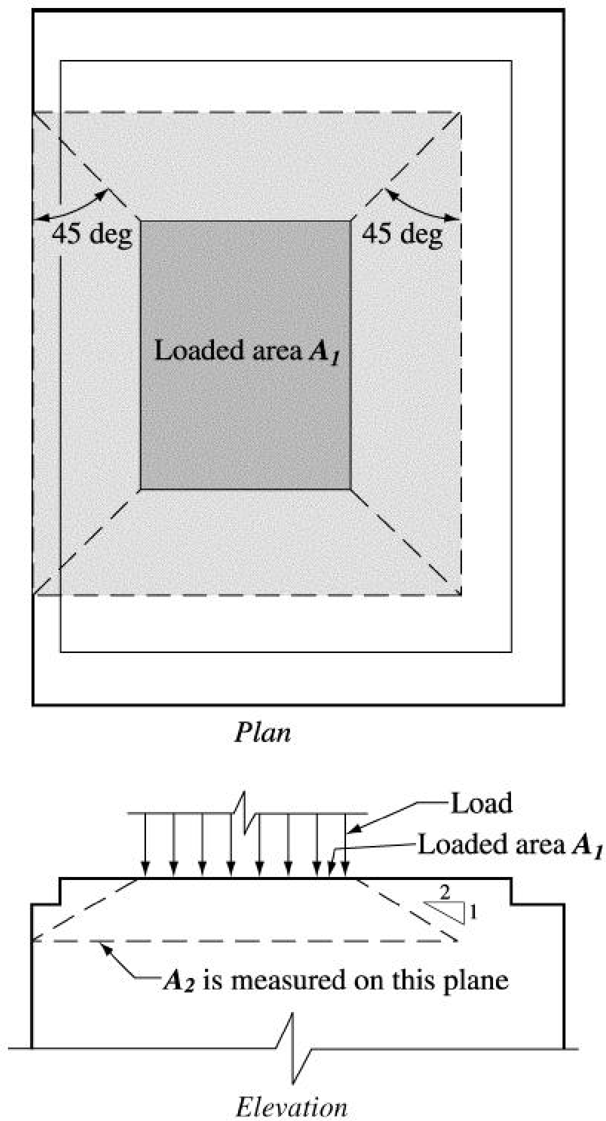

Figure 5-6. Concrete Bearing Area at Column and Footing Interface

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(PCA 2005)

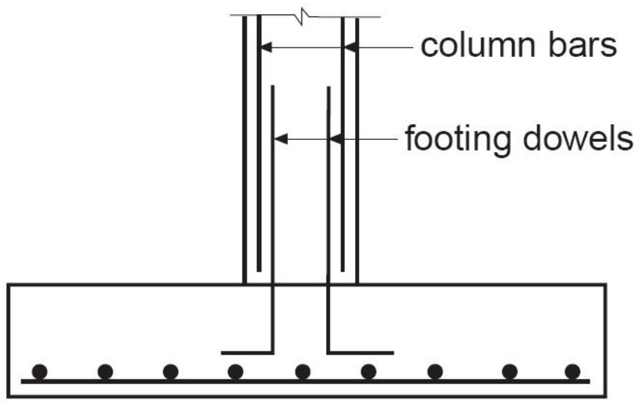

Figure 5-7. Interface of Column and Footing

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(PCA 2005)

5.1.2 Continuous Footings

Continuous footings, illustrated in Figure 5-8, also commonly referred to as wall footings or strip footings, exhibit one-dimensional action. This action exists as a cantilever out from either side from the face of the wall, as illustrated in Figure 5-9. The cantilever action is the result from the soil pressure acting beneath the footing. The continuous wall supported on the footing resists bending action along the length of the continuous footing.

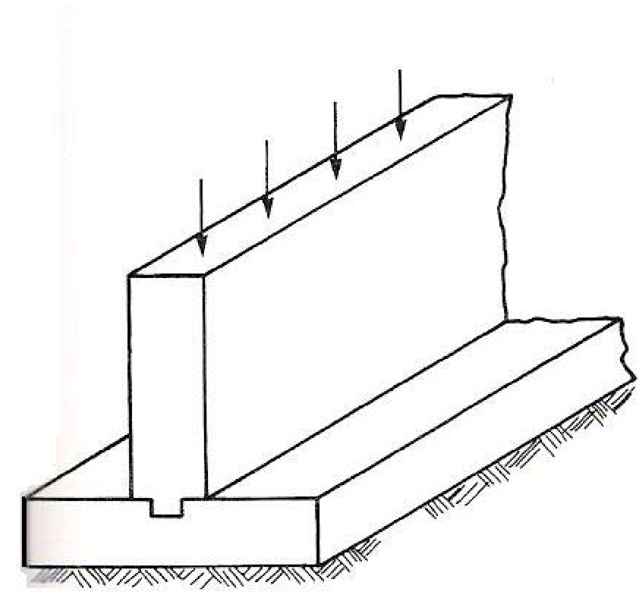

Figure 5-8. Continuous Footing

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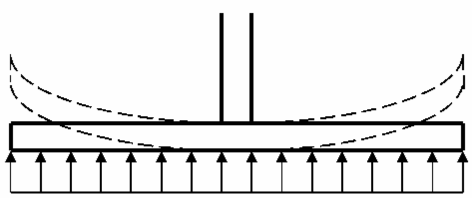

Figure 5-9. Cantilever Action Along Continuous Footing

Reinforced concrete is the most common material for continuous footings because of its durability in a potentially hostile environment and economy. Similar to spread footings, continuous footings are constructed as close to the ground surface as the building design allows, while meeting depth restrictions from frost penetration, soil shrinkage and expansion, soil erosion, and local code allowances. (MacGregor 1997)

5.1.2.1 Soil Pressure Behavior

The stress distribution beneath continuous footings is similar to that of spread footings. The same factors affect the stress distribution under continuous footings as mentioned in Section 5.1.1.1. The soil pressure distribution is similar to the distributions illustrated in Figure 5-1 but only occur within the cross section of the continuous footing, whereas this distribution occurs in both directions for a spread footing.

5.1.2.2 Design Considerations

The design of a continuous footing must consider flexure, shear, development of reinforcement, and load transfer from the wall to the footing. Footings must be designed to safely resist the effects of these actions. Each of these design considerations is similar to those discussed in Section 5.1.1.2 for spread footing design. The only difference for continuous footing design is the elimination of two-way punching shear. The presence of the wall prevents this action from occurring. The critical section for flexure design is at the face of the wall (section A-A in Figure 5-10), and the critical section for one-way shear is at a distance d from the face of the wall (section B-B in Figure 5-10).

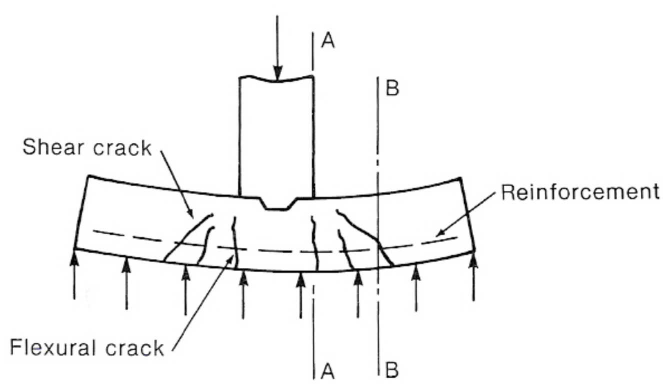

Figure 5-10. Structural Action of Continuous Footing

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5.2 Deep Foundation Systems

The purpose of deep foundation systems is to transmit structural loads through upper zones of poor soil to a depth below the surface where the earth is capable of adequately supporting the structure. This foundation type is common within areas of expansive soils, where the soft clays near the surface cannot sufficiently support the structure. These foundation types are also common in situations where uplift to the structure is a concern. Deep foundations act similar to structural columns. The load from the building structure is transmitted from the top to the bottom of the foundation. Deep foundations are typically considered as slender structural members; however, the soil in which the foundation is embedded provides sufficient lateral support. Under this assumption, buckling under axial loads does not cause concern. (McCarthy 1977)

Two common types of deep foundations include piles and drilled piers. Piles are slender foundation units driven into place. A pile cap is used at the top of a single or multiple piles for the structure to rest on. Drilled piers, which are typically larger in diameter than piles, are constructed from excavating a bore hole and filling with reinforced concrete. This report evaluates drilled piers as the deep foundation for the tilt-up panels. (McCarthy 1977)

5.2.1 Drilled Piers

Four types of drilled piers exist, all similar in construction methods, but differ in the method of how the load transfer from the structure to the earth is assumed. The first type of drilled pier is the straight-shaft end-bearing pier. This pier resists loads through end-bearing on the sound soil in which the pier rests upon. All overlying poor bearing soil, alongside the pier, is assumed to contribute no resistance to the load imposed on the pier. This soil is assumed only to provide lateral support to the pier. The second type of drilled pier is the straight-shaft sidewall shear pier. This pier penetrates far enough into the assigned bearing stratum layer of sound soil and transfers the design loads into the earth through sidewall shear. The overlying poor bearing soil is assumed to carry no load, but rather just laterally brace the pier. The third type of drilled pier is the combination straight-shaft, sidewall shear and end-bearing pier. The construction method is similar to the previous two piers, but the design philosophy is different. It is assumed that both sidewall shear and end-bearing transfer the design load into the sound soil. When this type of pier is carried into rock, it is often referred to as a rock socketed pier. The fourth type of drilled pier is a belled pier. These piers resist the design load through end-bearing only. This report will assume combination straight-shaft, sidewall shear and end-bearing drilled piers, which is common construction practice for Texas, for design analysis of deep foundations. These four types of drilled piers are illustrated in Figure 5-11. (Woodward 1972)

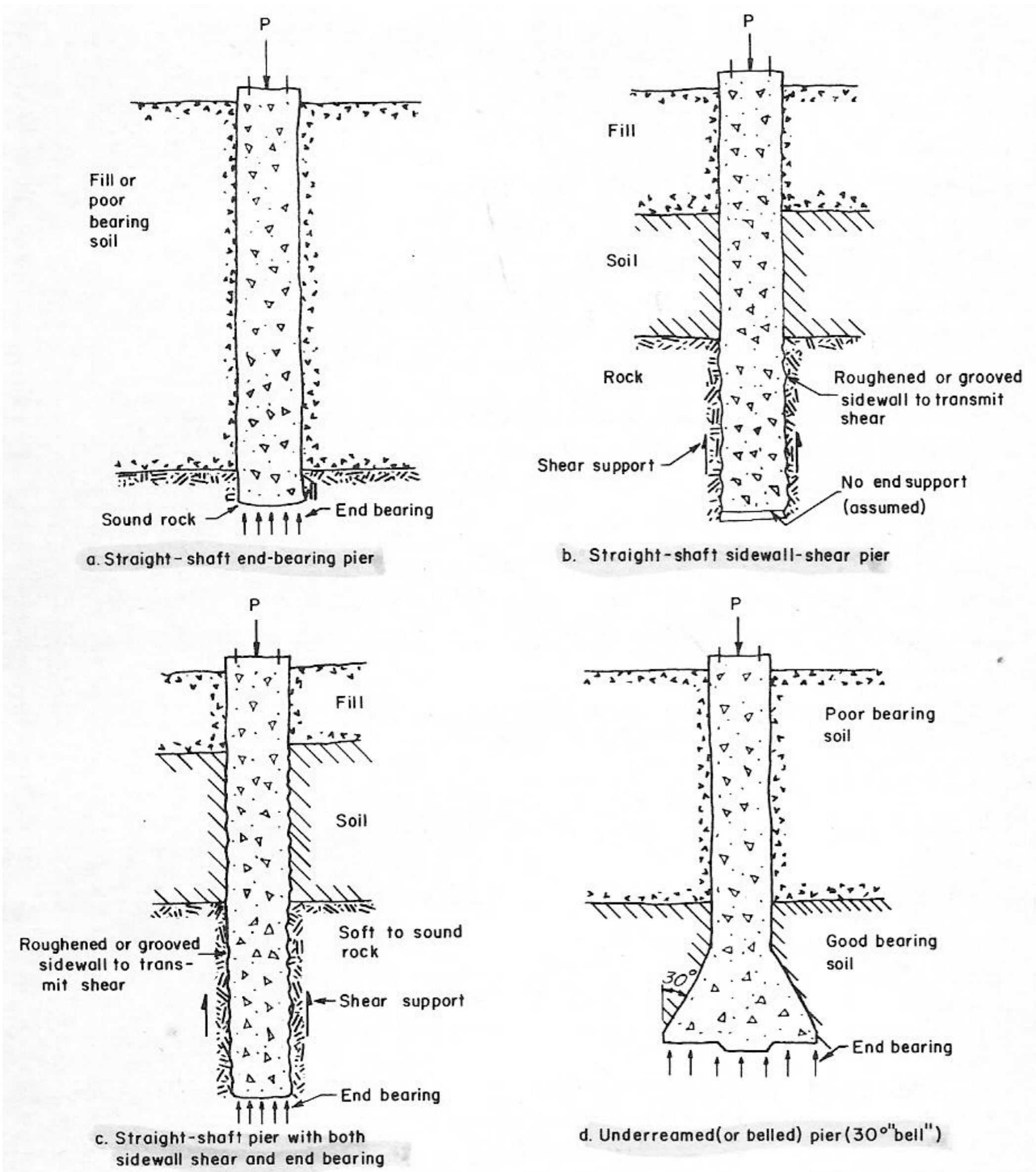

Figure 5-11. Drilled Piers

(Woodward 1972)

With the increase in technology of rotary pier-drilling machines following World War II, drilled piers became the best economical construction choice over all other types of deep foundations in areas where ground conditions required such foundations. In cities such as Houston, Denver, and San Antonio, where expansive soils are common, drilled piers quickly became the prevalent foundation option. As use of drilled piers expanded, some contractors and designers pushed the limits of drilled piers, especially conditions which had led to the advantages of drilled piers over other deep foundations in the first place. This resulted in unsatisfactory construction performance, such as delays, added costs, and contract abandonments, in areas where the geologic formations were not suitable for drilling machines and procedures. The most common difficulties included cohesionless soils caving below the water table, use of casings suitable machines for placing and pulling casings being unavailable, and soils with boulders and stones that could neither be drilled nor removed intact. These difficulties proved to be beneficial for the drilled pier construction industry, because the development of specialized tools, new machines, and better construction techniques advanced to eliminate the construction problems. The economical range thus expanded further throughout the United States, and drilled piers are now used in many areas where they previously would have been uneconomical or unfeasible. (Woodward 1972)

5.2.1.1 Soil Pressure Behavior

The geotechnical engineer on a project recommends design criteria for drilled pier foundations from soil investigations and reports. These recommendations tend to be conservative. Allowable loads for drilled pier design have gradually increased as experience with both full-scale load tests and completed structures have accumulated. This gradual increase in allowable unit loads has been reflected in building code allowable pressures in some cities. However, the unit loads on rock found through experience are still substantially higher than those recommended by geotechnical engineers. Research done on the load distribution of combination straight-shaft, sidewall shear and end-bearing drilled piers has indicated that more load than assumed is carried through sidewall shear. The procedure of using either straight-shaft end-bearing or straight-shaft sidewall shear piers can more often than not produce unnecessarily conservative results. This reasoning, along with the common engineering practice in Texas, is why combination straight shaft drilled piers are used as an option for tilt-up panels: to provide the most economical foundation option. (Woodward 1972)

Combination straight shaft drilled piers require some downward movement, or settlement, in order to activate either the shearing resistance around the shaft or bearing resistance at the base of the shaft. As the shaft continues to settle after its initial loading stages, the ultimate shearing resistance of the soil is fully activated. Further movement will then cause slippage of the pier with respect to the soil along the shaft surface. The ultimate shearing resistance of this pier type will occur first near the top of pier. This is where the shaft displacement is greatest because its confinement is the least. This load will then extend downward. As the load acting on the pier increases, the movement of the pier with respect to the surrounding soil will reach a sufficient level in order to fully activate the ultimate shearing resistance of the surrounding soil. Observations made by Whitaker and Cooke (Whitaker 1966) from combination pier load test have shown that full shaft shear resistance is developed in stiff fissured clays after settlements on the order of ¼ in (6 mm). Pier tests by Reese and O'Neill (Reese 1969) indicate that 0.2 in (5 mm) are required for activation of full shaft resistance. (Woodward 1972)

The amount of settlement necessary to activate full base resistance is dependent on the type and confinement the soil conditions the pier is bearing onto. For given bearing conditions, the base resistance is a function of the base diameter. Load tests have shown that the amount of settlement at the base of the pier required in order to develop ultimate end-bearing capacity ranges from 8 to 10 percent of the base diameter for cohesionless soil materials to 25 percent of the base diameter for cohesive soil materials. (Woodward 1972)

5.2.1.2 Design Considerations

The design of drilled pier foundations is mostly empirical, more so in cases where piers are carried into rock or where sidewall shear is assumed. For projects where subsurface conditions are well established and found to be relatively uniform by soil exploration, if the performance of past construction has been well documented, then the empirical design from experience is usually found to be satisfactory. (Woodward 1972)

Design requirements are usually met if the shaft diameter is large enough to carry the ultimate design load without exceeding the strength of the concrete and steel. Therefore, the design of drilled piers consists of two steps. The first step is to determine the pier size, including the overall concrete depth and diameter dimensions. The second step is to design the concrete pier element. Service loads and soil allowable stresses should be used in the first step to determine pier dimensions. Load combinations should be considered to determine the maximum force that will occur in the interaction between the soil and pier. Once the dimensions have been determined, the strength design method is used to design the concrete. (Lawson 2007)

Drilled piers may be designed and constructed using plain or reinforced concrete. ACI 318.1 governs the design of plain concrete piers. Reinforced concrete piers must be designed in accordance with ACI 318-05 Sections 7.10, 10.2, 10.3, 10.8.4, 10.9, and 10.15. The pier is simply designed as an axially loaded member, with the assumption of being fully braced by the surrounding soil. This assumption can be used because the site soils in the scope of this report will not liquefy under seismic loading. Reinforcing is required when applied tensile forces, usually associated with uplift, exceed the tensile rupture capacity of the pier. Reinforcing is also required to transfer the structural load to the pier in this situation. (Lawson 2007)

6 Common Foundations Used for Tilt-Up Construction Practice

This parametric study analyzes how foundation support affects the design of load-bearing tilt-up panels. Spread footings, continuous footings, and drilled piers are the three foundation design options analyzed to provide support to the tilt-up wall panel. These three foundation options are commonly used in tilt-up construction practice for the three regions within the scope of this report. This section discusses the foundation systems that are commonly used for industrial warehouse buildings or two story commercial buildings similar to the building designed for this parametric study.

A certain level of risk for damage is associated with each of these foundation systems. Damage may occur to the building superstructure and architectural features due to differential foundation movements. Each of these systems also has an associated relative cost of construction. It is typically found from comparison of the various foundation systems that the level of risk is inversely proportional to the level of cost. For example, in areas of prevalent expansive soils, shallow foundation systems typically have a relatively higher level of risk than deep foundations, but are often selected due to economics and ease of construction.

Continuous footings are widely used for tilt-up panels not only within Dallas, Denver, and Kansas City, but throughout the United States. The continuous footing provides a uniform support along the length of the wall at its base. In areas of expansive soils, such as Los Colinas, a suburb of Dallas, and Denver, if it is assumed that a greater risk of foundation movement can be tolerated, this option can be chosen.

Spread footings are used as an alternative shallow foundation option in these three regions as well. Some factors that allow spread footings to be more advantageous than continuous footings include available space for building components that must enter and/or exit the building at the base of the structure, such as electrical duct banks, water lines, plumbing, etc., and more available space for construction equipment and operations. When large amounts of mechanical, electrical, or plumbing systems must cross below the panels, spread footings allow these components to pass through without having to provide multiple sleeves within a continuous footing. Spread footings are also commonly found directly under a panel supporting a largely loaded girder or jamb. In this situation, the spread footing could be an additional isolated footing between the other isolated footings at the end of the wall panel, or the spread footing could be cast within a continuous footing.

Drilled piers are commonly used for tilt-up structures in regions where expansive soils require the substructure to bear on levels of rock or sturdy soil below the active zone. Another situation requiring use of drilled piers is to transfer the loads from the superstructure deep enough into the earth in order to prevent additional loading on adjacent structures. When drilled piers are used, the panels are set directly on the drilled pier. Pile caps are rarely used, unless odd panel configurations requiring unique load paths or adjustments made due to construction errors of the pier placement occur.

Each of these three foundation systems provides advantages and disadvantages. Tilt-up panels are designed and detailed for each of these three foundation systems in this parametric study. The design process and results are evaluated and discussed at the end of the report.

7 Tilt-Up Concrete Panels

Tilt-up construction is a technique for casting concrete elements in a horizontal position at the jobsite and tilting them to their final position in the structure, as defined by ACI 116R, Cement and Concrete Terminology. Tilt-up construction can also be defined, in accordance with ACI 318, Building Code Requirements for Structural Concrete, as structural concrete elements cast elsewhere than their final position in the structure. Several features exist which make the tilt-up construction method unique. Tilt-up panels are only handled once, because the panels are designed to resist the lifting stresses from the crane only one time. The lifting process is one continuous operation, from the panel lying horizontally on the casting bed to being placed vertically in its final position. (ACI 551 Jun 2005)

This technique is used commonly for low-rise industrial and commercial buildings. Buildings with a mean roof height ***h*** less than or equal to 60 ft (18 m) and less than the least horizontal dimension are defined by ASCE 7-05 as low-rise buildings Low-rise buildings using tilt-up construction efficiently are typically limited to four stories. Educational facilities, office buildings, residential apartment homes, retail centers, and churches are all projects which can benefit from the tilt-up construction process. Some of the most advantageous features of tilt-up construction include: the elimination of expensive formwork needed for cast-in-place concrete and scaffolding needed for masonry, a fast and economical construction cycle time, the combination of ease and speed of construction, a durable and low maintenance long-life building, and a wide variety of exterior architectural finishes from colored concrete to exposed aggregates to form line finishes. (ACI 551 Jun 2005)

Tilt-up construction is one of the fastest growing industries in the United States, because of its ability to combine the advantage of low cost with the other advantages previously listed. The tilt-up construction method is used for a minimum of 10,000 buildings each year, enclosing more than 650 million ft^2 (60 km^2). In past decades, tilt-up construction has been dominant in the West and Southwest, but the method is fast gaining advocates in the Midwest, New England, and Canada. (ACI 551 Appendix Jun 2005)

Tilt-up concrete wall panels are typically load-bearing, slender, wall elements; however, can be non-load-bearing. Slender walls are defined as walls that have a significant reduction in the axial load capacity due to moments resulting from lateral deflections of the wall. Tilt-up wall panels can be used as either exterior or interior walls, and can also be designed as shear walls to resist lateral loads from wind and earthquake forces. Because tilt-up panels are load-bearing walls, they can support roof loads and eliminate beams and columns around the periphery of the building. Lateral support to the panels is generally provided by the floor(s) and roof diaphragm of the building. Vertical support is provided by spread footings, continuous footings, or drilled piers. (MacGregor 2005)(ACI 551 Feb 2003)

7.1 History

The practice of using concrete as a structural element dates back to as early as 4700 B.C. when the Villagers in Jarmo, Iraq constructed dwelling walls using Touf, a pressed mud. During the Romans years of dominance, they produced pozzolan cement, which was the mainstream building material for their construction projects. As cementitious materials became more readily available and studied, the quality and durability of concrete construction improved. The development of Portland cement in the nineteenth century allowed concrete structures to become a more competitive building material for construction. By 1890, Portland cement was widely accepted as the standard cementing material for concrete. In the earliest years of the twentieth century, as the concrete technology quickly developed, many pioneers explored new construction ideas, and both the Portland Cement Association (PCA) and the American Concrete Institute (ACI) were established. (ACI 551 Jun 2005)(Glass 2000)

Cast-in-place concrete was commonly used for early structures using Portland cement. Cast-in-place concrete reinforced with mild steel reinforcing bars was second only to structural steel as a building material by 1914. Several entrepreneurs developed during this period, looking for methods to construct concrete structures without the use of massive amounts of formwork used for cast-in-place concrete. Thomas Edison used a track mounted crane to lift tilt-up panels for a housing development in Union, New Jersey in 1908. These houses still stand today. (Glass 2000) (Tilt-Up Construction 2007)

Colonel Robert H. Aiken, an operator for an engineering company in Winthrop Harbor, Illinois, devised an innovative method of casting panels on tilting tables and then lifting them into place by means of specially designed mechanical jacks. Aiken is credited as the first tilt-up pioneer. This tilt table method was used on the Jewett Lumber Company in Des Moines, Iowa, between 1906 and 1912, and on several Army facilities, factory buildings, target abutments, barracks, ammunition and gun houses, mess halls, factory buildings, and churches. (ACI 551 Jun 2005)(ACI 551 Feb 2003)

The first complete tilt-up building was a concrete factory on Aiken's own farm near Zion City, Illinois. The factory walls were cast onsite on a smooth bed of sand, around door and window frames, and within a perimeter form. The finished walls were tipped onto their foundation by block-and-pulley derricks and horsepower. In 1906, Aiken also used this tilt-up method to construct the Memorial United Methodist Church in Zion, illustrated in Figure 7-1, and a two-story ammunition and gun house at Camp Logan. While refining his methods, Aiken used a steel tipping table to construct 15 buildings in five states, including two at Camp Perry in Ohio, illustrated in Figures 7-2 and 7-3. (ACI 551 Jun 2005) (Tilt-Up Construction 2007) (Johnson 2002)

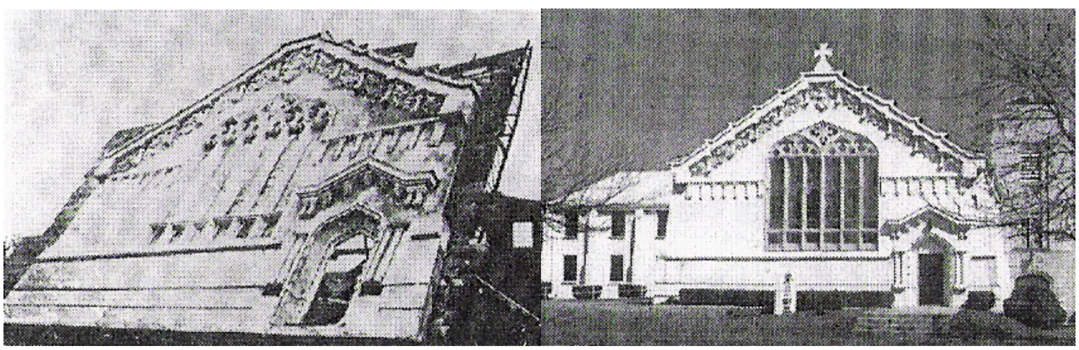

Figure 7-1. Memorial United Methodist Church in Zion.

Left image: Tilting of front wall during construction in 1906.

Right image: Zion Methodist Church in 1987.

(Photos courtesy ACI 551 Feb 2003)

Figure 7-2. Camp Perry, Ohio
(Johnson 2002)

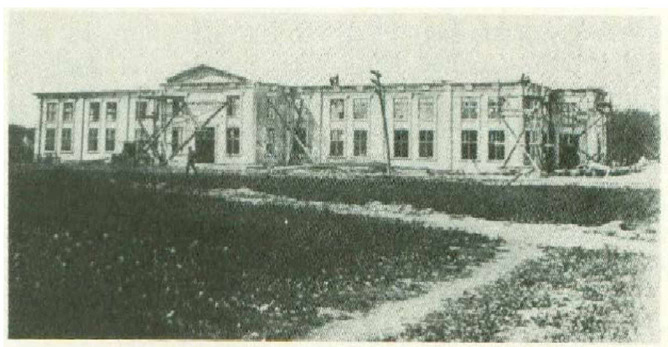

Figure 7-3. Concrete Mess Hall Building at Camp Perry, Ohio
(Aiken 1909)

Prior to World War I, precast concrete construction fell behind the advancements of tilt-up concrete construction because the infrastructure system in the United States was not feasible for transporting off-site precast structural elements. The shortage of steel and labor caused by World War I, along with the development of highway infrastructure, allowed for precast concrete construction to become a more practical method of construction. This advancement in precast concrete stalled the development of tilt-up construction practice. Tilt-up then became a dormant practice during the Great Depression in the 1930s. Because most construction projects were publicly funded, methods of construction that saved labor were not valuable. (ACI 551 Jun 2005)

A construction boom occurred after World War II to house servicemen. Along with the increased construction, three technological advancements created an opportunity for tilt-up construction to emerge in the 1950s. These technological advancements included the heavy-duty truck crane, electric-arc welding, and transit ready-mix concrete. As a result of these innovations, the number of buildings constructed with site-cast concrete elements increased significantly. The first stage of design development methods for tilt-up emerged with this increase of construction. An initial report by ACI Committee 551 in 1979 along with the publication of Brook's comprehensive *Tilt-Up Design and Construction Manual* (Brooks 1994) became widely available to contractors within the United States. (ACI 551 Jun 2005)(Glass 2000)

The spread of this massive development originated in California during 1945 and 1946 and the dollar volume of work in southern California increased dramatically between 1946 and 1952, which spread the tilt-up method throughout the other Sun Belt states. The method soon spread into colder climates and throughout the United States and Canada. Tilt-up construction is currently present in all fifty states, and the construction grew 100 percent between 1995 and 2000. Tilt-up accounted for 15 percent of the annual industrial construction market in the United States during the early 1990s. The twentieth century saw tilt-up evolve from a small-scale construction method to a reliable, economical, and well-understood construction technique. (Glass 2000)

7.2 Construction Process

The advantage of tilt-up construction which allows it to surpass other construction practices is its efficient on-site production operation. Much of the economy is produced as a result of this operation. Success of each tilt-up project depends on the organization and planning prior to construction. Each construction sequence depends on the success of the preceding construction event. This procedure requires articulate scheduling. The following list dictates a proper construction sequence for a typical tilt-up project:

1. Site preparation
2. Underslab system integration
3. Cast and cure interior column footings
4. Cast and cure floor slab
5. Form, cast, and cure panels
6. Form, cast, and cure exterior footings
7. Erect and brace panels
8. Construct roof and floor diaphragms

9. Place concrete pour strip (if necessary)
10. Remove braces
11. Schedule finishing trades

The first four steps are typical for construction projects of all structural systems. A few requirements for these initial steps unique to tilt-up construction include: checking site for crane access and any obstructions that may exist for the crane boom, placing temporary concrete within the interior column blockouts as a form for the panels, and constructing a floor slab which will be used as a casting bed for the panels. (ACI 551 Jun 2005)

The remaining project sequences are exclusive to tilt-up construction. Formwork is placed on the floor slab for the panels. Once the formwork has been set, a bondbreaker is applied to all surfaces that will come into contact with the concrete used to cast the panel. This bondbreaker will allow the tilt-up panel not to stick to the slab, leaving a clean, smooth panel surface once erected. After the bondbreaker is applied, the reinforcement is set, and the concrete is cast. Once the panels have been cured and have reached their desired compressive strength, they are lifted into place by a specified crane and set onto the foundation system selected for the project. A closure strip may need to be poured, depending on the foundation system. Temporary braces are attached to the panels to provide lateral support until the roof or floor diaphragm is constructed. Specialty trades then install the necessary mechanical and electrical equipment, add any architectural finishes specified, and landscape the site. (ACI 551 Jun 2005)

7.3 Design Process and Considerations

A successful tilt-up project is dependent on the five crucial steps of design, planning, construction, erection, and creating finishes. The first of these steps, design, sets a precedent for providing a quick, economical, and versatile method of constructing low and mid-rise structures of four stories or less, with the majority being one and two stories. Tilt-up wall panels are designed as slender load-bearing beam-columns spanning vertically from the ground floor to the roof or intermediate floors. Several methods have been utilized for determining the load carrying capacity of these tilt-up walls. PCA published a design aid for tilt-up load bearing walls in 1974. This design aid contains a series of design charts based on a detailed computer analysis. Coefficients used to determine the maximum axial loadings are given for several combinations of section thickness, reinforcing steel areas, lateral loading, panel height, and concrete strength. In 1979, the PCA published additional variations of these design charts which made it easier to consider special loading conditions or variations in section properties. The Structural Engineers Association of Southern California (SEAOSC) produced the “Recommended Tilt-Up Wall Panel Design” and “Test Report on Slender Walls” documents for a simplified analysis method which gave reasonably accurate but conservative results. The ACI 318-05 provides an analysis procedure, which will be used in this report for design. This procedure is found in Section 14.8, Alternative Design of Slender Walls. ACI Committee 551 uses this procedure for its design guide for the analysis of vertical reinforcing in tilt-up panels. (ACI 551 Feb 2003)

7.3.1 Loading Conditions

Tilt-up panels are subject to forces in three directions: vertical, lateral, and in-plane. The vertical forces are derived from roof and floor joists. Since joist spacing is typically five feet or less, these loads are assumed to act as a uniformly distributed load for purposes of wall panel design. These loads are applied at an eccentricity from the centerline axis of the panel either intentionally or due to bearing irregularities. If these loads are constructed concentrically, a minimum eccentricity of approximately one-third to one-half the panel thickness is recommended, and should be additive to the effect of lateral pressure. The eccentricity at the bottom of the panel is assumed to be zero. Axial load eccentricities should not be used to reduce the bending moment from lateral loads, and axial loads should not be reduced due to wind uplift on the roof structural members. Figure 7-4 illustrates the lateral and axial load application.

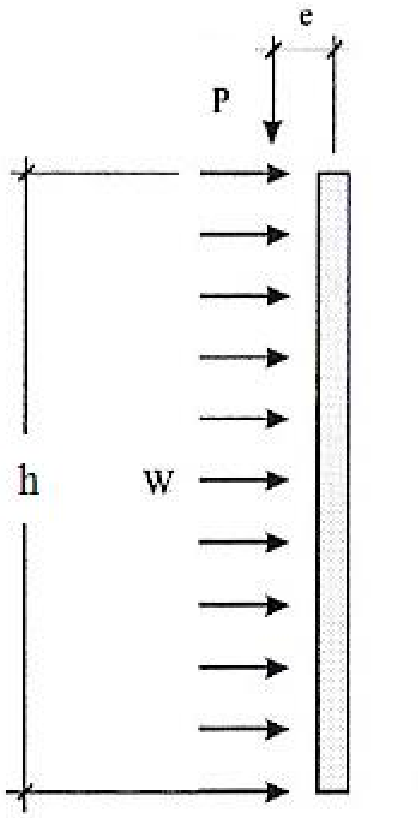

Figure 7-4. Lateral and Axial Load Application

Courtesy of the American Concrete Institute

(ACI 551 Mar 2005)

The effect of panel self weight also must be considered because it is a significant contribution within the vertical loads. A conservative design approach for solid panels is to assume one-half of the total panel weight is applied at the top of the panel as a concentric axial load because the critical section for bending occurs at or above mid-height. The lateral loads are determined by the forces from wind, seismic and soil conditions, whichever one governs. These forces are applied to the wall as a uniformly distributed lateral load. The panel spans similar to a flat slab between its points of support, usually the floor and roof diaphragms, in order to resist the bending from the lateral loads. The effect of lateral loads is often the largest contribution to the total applied bending moment on the tilt-up panel. Tilt-up panels provide lateral resistance through in-plane shear. These shear forces can be significant for long-narrow buildings in moderate and high seismic zones, where large panel overturning moments may occur. Section thickness and reinforcing requirements can considerably increase in panels with large openings and narrow legs. The horizontal reinforcement is critical for the resistance of in-plane shear. Resistance for all three directional forces is provided by the panel thickness and steel reinforcing. The reinforcing is typically placed in the middle of the panel in order to resist bending from forces acting in either direction. When using one layer of reinforcing, placement in the center allows for the greatest moment arm between the compression force of the concrete wall and tension force of the reinforcing steel for either bending direction. Double layers of reinforcing are typically used at jambs or when heavy axial loads are applied. (ACI 551 Feb 2003)(ACI 551 Mar 2005)

7.3.2 Bending Moment and Stiffness

The design bending moment results from the combination of lateral loads, eccentric axial loads, initial out-of-plane straightness, and the ***P-Δ*** effects produced from the axial loads. The maximum bending moment typically occurs at mid-height of a wall for solid panels spanning vertically, assuming a pin-pin connection. This location may change depending on the geometry of the panel, openings within the panel, large axial loads, large eccentricities, and support conditions. Figure 7-5 illustrates this analysis concept for slender walls.

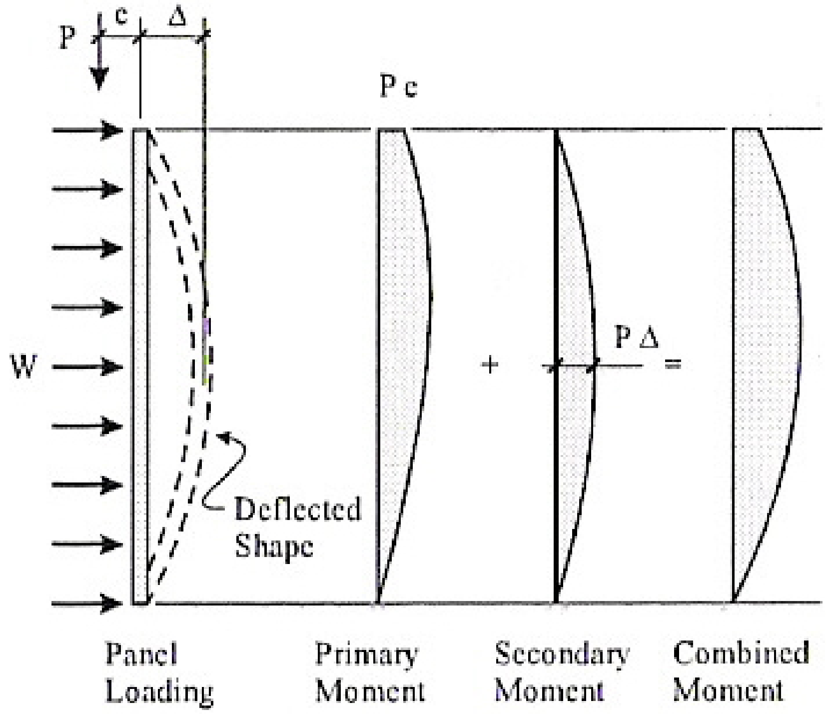

Figure 7-5. Analysis Concept for Slender Walls

Courtesy of the American Concrete Institute

(ACI 551 Mar 2005)

Calculations for the design bending moment depend on the panel bending stiffness. Bending stiffness is the ability for a panel to resist deformation within a linear range. The nonlinear properties of a reinforced concrete panel section make it difficult to precisely calculate the bending stiffness. This will be further discussed in Section 7.4. Tilt-up panels require adequate bending stiffness in order to minimize out-of-plane deflections and coinciding ***P-Δ*** effects. Several variables take place within the bending stiffness of a reinforced concrete section. These variables include the geometry of the concrete section, the concrete modulus of elasticity, the flexural strength of concrete, the axial compression force, the amount and location of reinforcing steel, the grade of reinforcing steel, and the extent of cracking within the panel. (ACI 551 Feb 2003)

7.3.3 In-Plane Shear

Tilt-up panels are designed for in-plane shear forces and may be specified and detailed as the lateral force resisting system for a building structure. Design procedures for in-plane shear forces are distinctly different from the procedure used to design panels to resist out-of-plane bending. Shear stresses, and overturning moments as a result of forces from the roof or floor diaphragms acting parallel to the plane of the wall must be resisted by the tilt-up panels. Panel thickness and reinforcing may be controlled by seismic forces in areas of high seismic activity. The following design procedures must be considered for tilt-up panels subjected to in-plane forces: resistance to panel overturning and sliding, concrete shear resistance, increased axial forces within portions of the panel, load distribution and transfer to the foundation, frame action in panels with openings, and seismic ductility. (ACI 551 Mar 2005)

7.3.4 Temperature and Shrinkage Effects

Tilt-up structures are less susceptible to temperature changes and concrete shrinkage effects than monolithic, cast-in-place concrete structures. A few design techniques should be considered to minimize these effects. Tilt-up panels are usually lifted and tilted into place within a one or two week period after being cast onto the floor slab. Minimum temperature and shrinkage reinforcement, $\boldsymbol{0.0018A_g}$, from ACI 318-05 Section 7.12.2.1, may be insufficient to prevent cracking that may occur as panel connections induce stresses as the panel continues to undergo drying shrinkage. An advantage for tilt-up structures is that each joint between panels acts as an expansion joint. However, excessive restraint and vertical cracking may occur at connections along these vertical joints. (ACI 551 Mar 2005)

7.3.5 Wall Assemblies

The wall assembly chosen for a building will dictate the design and construction process of the tilt-up structure. Three common wall sections are used for tilt-up panel structures. These three wall sections are a plain tilt-up panel, a tilt-up panel with post-installed stud-wall and insulation, and a tilt-up sandwich panel. The plain tilt-up panel offers the lowest cost of the three options, but is the least energy efficient. The sandwich panel is the most expensive wall assembly of the three options, but provides the best energy efficiency. The tilt-up panel with post-installed stud-wall and insulation has a cost and energy efficiency level between the other two options. (ACI 551 Mar 2005)

In order to begin the design of tilt-up panels for a structure, the building must be panelized. Panelizing a building is the process of determining how walls are divided into individual panels, determining the geometry of each panel, and determining where the joints will be located between each panel. The thickness of the panels needs to be set before panelizing the building in order to determine the area of each panel. Panel thicknesses are typically specified in accordance with standard lumber dress sizes, so that the forms do not need to be ripped. A standard industry practice for determining preliminary thickness of the panels is to provide an inch of thickness for every four feet of panel height. For instance, this report provides design analyses for 32 ft (9.75 m) panel heights and 40 ft (12 m) panel heights. Using this method yields preliminary thickness assumptions of 7 ¼ in (18 cm) for the 32 ft (9.75 m) panels and 9 ¼ in (23 cm) for the 40 ft (12 m) panels. (ACI 551 Mar 2005)

7.3.6 Foundation Options

The parametric study within this report analyzes spread footings, continuous footings, and straight-shaft drilled piers. Each of these three foundation options transfers the loads from the structure into the soil or rock supporting the structure. The geotechnical report specifies the permissible soil-bearing capacity or drilled pier capacity, and provides a recommendation for the foundation system to be used for the structure. The geotechnical engineering report recommends a foundation system for the engineer of record to use.

Spread footings are placed at panel joints to provide support for tilt-up wall panels. Common practice for spread footing reinforcement places one layer of reinforcing bars in each direction located 3 in (76 mm) clear from the bottom of the footing, as specified by ACI 318-05 Section 7.7.1. Special detailing within the bottom of the panel can provide distribution to each spread footing, similar to a grade beam. The bottom of the footing is set at or below the frost depth as required by the local building codes. The top of the footing is typically set approximately 1 in to 2 in (25 mm to 50 mm) below the panel base. This allows for the panels to be properly aligned by using grout setting pads placed after erection. The panels are centered on the spread footings unless restrictions such as property lines permit otherwise. (ACI 551 Jun 2005)

Continuous footings are used to provide continuous support to interior and exterior tilt-up wall panels. The continuous reinforcement within the footings helps to distribute panel loads over weak spots in the subgrade soil. Continuous footings with heavier reinforcement are used to span trenches, drainlines, and other site features underneath the building structure. The footing width is inversely proportional to soil strength. Therefore, depending on the site soil conditions, wider footings are necessary for weaker soil conditions and narrower footings are used for stronger soil conditions. The panels are centered along the continuous footing unless restrictions permit otherwise, and the bottom of the footing is set at or below the frost depth in accordance to local building codes. The top of the continuous footing is set one or two inches below the base of the panel and setting pads are used to temporarily support the wall panel. These pads are placed during or immediately after panel erection. Since panel self-weight may contribute up to 75 percent of the total load to the footing, proper distribution of the load to the footing is essential. Once the panels are set on the pads and aligned, the remaining space between the footing and the panel is packed with grout to provide continuous support from the panel to the footing. (ACI 551 Jun 2005)

When soil conditions dictate the use of deep foundations, drilled piers may be used to transfer the load from the structure to the supporting soil or rock. It is common practice for the panels to rest directly upon the pier, but if the pier diameter is not sufficient to support the panels, grade beams can be constructed from pier to pier, similar to continuous footings. When the panels do bear directly onto the pier, the top surface should be set 1 in to 2 in (25 mm to 50 mm) below the panel base and should be finished smooth and level. As with spread footings, a grade beam can be detailed into the bottom of the tilt-up panel to distribute the loads to both supports. (ACI 551 Jun 2005)

7.3.7 Connection Design

Connections must be designed to adequately transmit loads from the roof and floor systems through the load bearing tilt-up walls to the foundation. A wide variety of connection types have resulted due to variations in the type of roof and floor systems, along with designer and contractor preferences. Common connections used in tilt-up are categorized into four main groups: welded embedded metal, embedded inserts, drilled-in anchors, and cast-in-place concrete. (ACI 551 Feb 2003)

This parametric study focuses on connections used at the foundation system to transfer forces from the load bearing panel to the foundation, illustrated in Figure 7-6 and discussed into further detail in Section 9.2.1. In regions of low or negligible seismic activity, friction is often considered to provide sufficient restraint between the panel and foundation without a mechanical connection. ACI 318 Section 15.8.2.1 states a minimum area of reinforcement, $\boldsymbol{0.005A_g}$, must be provided at the connection to the foundation. However, experience throughout the United States indicates that a connection at the base of a panel is unnecessary in these locations. Regions of moderate to high risk seismic activity require a connection mechanism between the tilt-up panel and the foundation. This connection is required to resist a longitudinal displacement of the panel due to seismic forces transmitted from the foundation and wall panel and into the roof or floor diaphragms. Panel displacement is critical in situations involving spread footings or drilled piers, as the panel may slide off the foundation support. (ACI 551 Feb 2003)

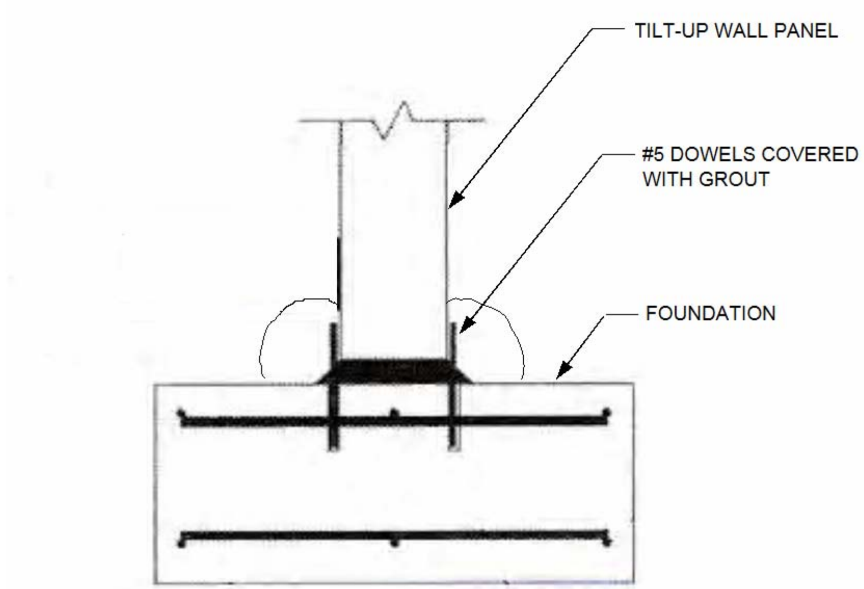

Figure 7-6. Panel Connection to Foundation

Courtesy of the Tilt-Up Concrete Association

(TCA 2006)

The panel is also connected to the slab on grade, illustrated in Figure 7-7 and discussed into further detail in Section 9.2.1. Dowels are cast into the tilt-up panel, and a closure strip is cast around the perimeter of the slab on grade once all the panels are in place. This connection is also critical when the lower portion of the panel acts as a grade beam, and reduces the unsupported length of the panel. Common connection details are illustrated in Figures 9-4, 9-5 and 9-6. (ACI 551 Feb 2003)

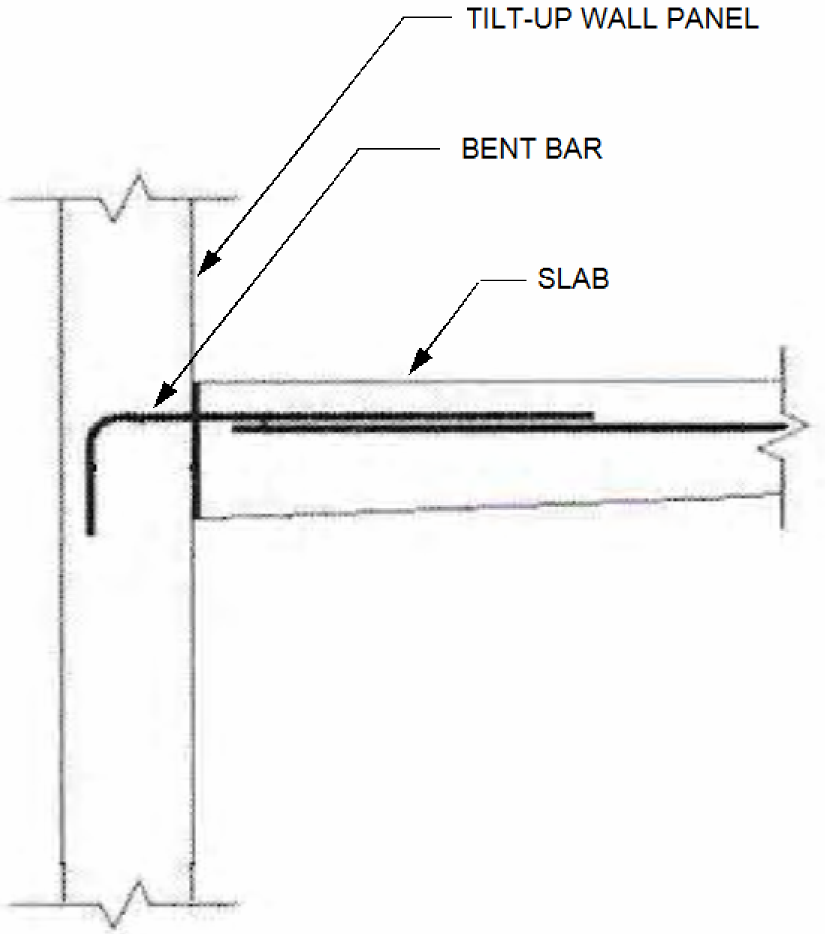

Figure 7-7. Panel Connection to Slab-On-Grade

Courtesy of the Tilt-Up Concrete Association

(TCA 2006)

7.4 Load-Bearing Slender Walls

Tilt-up panels are often classified as slender walls in which flexural tension controls design and moment magnification needs to be considered. Slenderness ratios of 140 to 200 are common. The bending moments due to applied loads can be magnified considerably by the effect of the axial loads on the deflected shape. This increase in moment, the ***P-Δ*** effect, must be taken into account in the proper analysis of out-of-plane deflections. (ACI 551 Feb 2003)

The standards dictating provisions for slender wall panels have been continually updated and revised since their inception in the 1980s. Prior to this time, the concept of slender wall panels was unfamiliar. In the 1960s and 1970s, concrete load-bearing walls were limited by ACI height/thickness (h/t) ratios which specified much thicker walls. As the tilt-up construction industry began to gain momentum in the 1980s, two publications were produced by SEAOSC that provided examples and test results to prove that h/t ratios could be increased with proper second-order effect analyses. (Lawson 2007)

The bending moments resulting from out-of-plane lateral loadings are usually significantly greater than those resulting from eccentric axial loads. The point where the maximum factored bending moment at or near the mid-height of a panel exceeds the ultimate resisting moment of the concrete section is the ultimate strength failure of a slender wall panel. The maximum factored bending moment of a panel can be separated into two components: primary moments and secondary moments. Primary moments are the moments that occur due to applied loadings such as lateral pressures and eccentric axial loads. For a solid panel, this moment occurs at mid-height. Secondary moments, due to ***P-Δ*** effects, are the result of the applied axial load and panel self weight acting on the deflected shape resulting from the primary moments. The deflection is dependent on the bending stiffness of the panel. The bending properties of a concrete section, including both strength and stiffness, vary with changes in axial compression and bending curvature. The solution for the ultimate resisting moment of the concrete section can be determined by an iterative procedure or by direct calculations using ACI 318-05 Equation 14-6. (ACI 551 Mar 2005)

8 Tilt-Up Wall Panel Design

The goal for structural engineers is to design buildings that provide life-safety for the occupants. Codes and standards have been written and revised, from many years of research and experience, which set minimum requirements for building structures. Tilt-up panels follow this same design philosophy: provide life-safety for occupants while incorporating the most economical design option. This section discusses how the loads applied to the building are determined, how the walls are designed to resist these loads, and how the most economical wall section is established. Sample calculations for the tilt-up design process can be found in Appendix A.

8.1 Loads

The parameters of this study have been set to establish a broad region within the Midwest for which this report is applicable. This region, shown in Figure 2-1, covers the cities of Dallas, Denver, and Kansas City, in order to determine how different soil properties affect the design of panels under similar loading conditions.

8.1.1 Gravity Loads

The building used for tilt-up panel design for this parametric study is from the 2006 IBC Structural/Seismic Design Manual, Vol. 2. The example building is a warehouse with tilt-up concrete walls and a panelized roof system. As illustrated in Figure 8-1, the panels along the north and south sides of the building have a larger tributary area than the panels on the east and west sides of the building. Because the panel self-weight is so large with respect to the roof axial loads, this difference in tributary area is negligible with regards to the ***P-Δ*** effects from the applied ultimate moment. It has been determined that the vertical reinforcement for panels along each side of the building can be detailed the same.

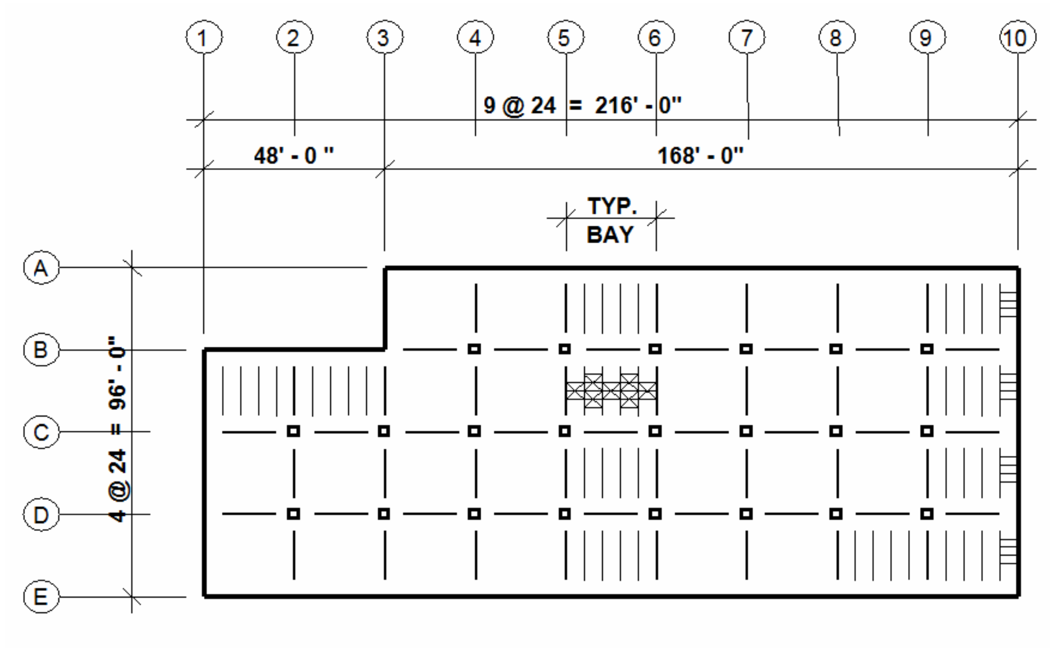

Figure 8-1. Building Framing Plan

Courtesy of the Structural Engineers Association of California

(SEAOC 2006)

The gravity loads are determined from the provisions set in the IBC 2006 and the ASCE 7-05. Roof gravity loads, both dead and roof live, are equivalent in each location. Tables 8-1 and 8-2 illustrate the roof loading determined for design.

Table 8-1. Roof Dead Loads

City	Panel Height (ft)	North & South Panels Roof Dead Load (klf)	East & West Panels Roof Dead Load (klf)
Dallas	32	0.26	0.06
Dallas	40	0.26	0.06
Denver	32	0.26	0.06
Denver	40	0.26	0.06
Kansas City	32	0.26	0.06
Kansas City	40	0.26	0.06

Table 8-2. Roof Live Loads

City	Panel Height (ft)	North & South Panels Roof Live Load (klf)	East & West Panels Roof Live Load (klf)
Dallas	32	0.28	0.08
Dallas	40	0.28	0.08
Denver	32	0.28	0.08
Denver	40	0.28	0.08
Kansas City	32	0.28	0.08
Kansas City	40	0.28	0.08

Snow loads and snow drift loads are determined based on the ground snow load which varies from each location. Ground snow loads of 30 psf or less have been set to establish a general region within the Midwest. Figure 8-2 illustrates the regions within the United States that meet this criterion.

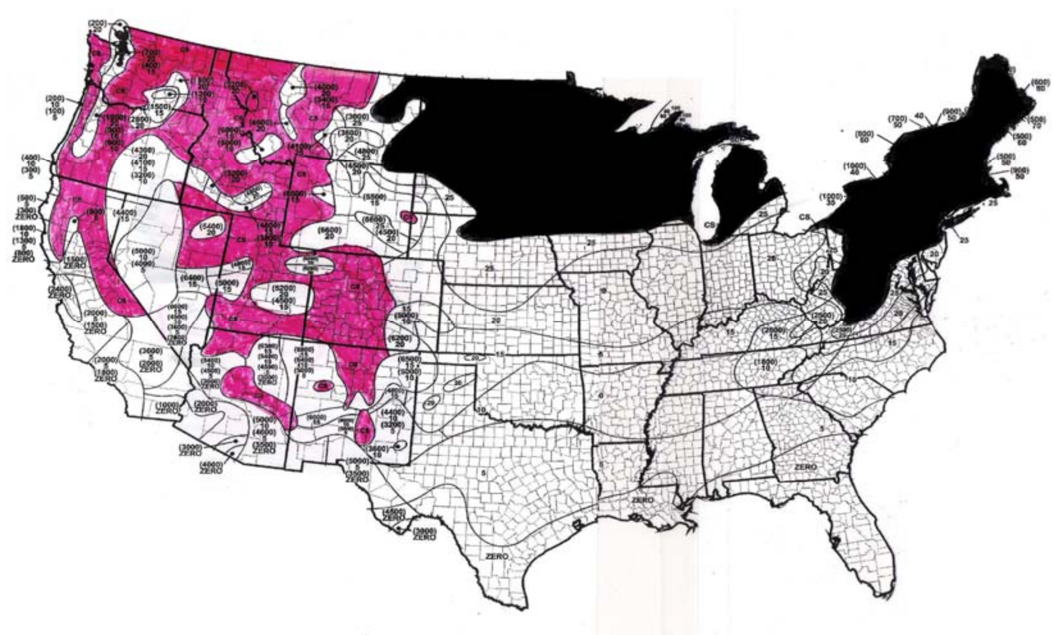

Figure 8-2. Ground Snow Loads

Figure courtesy of ASCE/SEI 7-05

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(ASCE/SEI 2006)

Figure 8-2 is the United States map illustrating ground snow loads from ASCE 7-05 Figure 7-1. The regions that have been darkened exceed a ground snow load of 30 psf. The regions highlighted in red denote Case Study areas. The axial snow loads for the panels can be seen in Table 8-3. The snow load for Denver and Kansas City exceeds the roof live load in these two regions; therefore, the snow load is used for the governing roof live load. In Dallas, the snow load governs for the east and west wall panels while the roof live load governs for the north and south wall panels. Appendix A illustrates the calculations to determine the gravity loads for the 32 ft (9.75 m) panel supported on continuous footings in Dallas.

Table 8-3. Snow Loads

City	Panel Height (ft)	North & South Panels Snow Load (klf)	East & West Panels Snow Load (klf)
Dallas	32	0.23	0.09
Dallas	40	0.23	0.09
Denver	32	0.47	0.11
Denver	40	0.47	0.11
Kansas City	32	0.47	0.11
Kansas City	40	0.47	0.11

Considerations for ponding, the retention of rain water due solely to the deflection of relatively flat roofs, are not investigated for this building structure. Roofs with a slope less than ¼ in/ft (1.19°) shall be analyzed to assure that they possess adequate stiffness to resist progressive deflection as rain falls on the roof or meltwater is created from snow on the roof. The building roof structure within this parametric study has a roof slope of ½ in/ft (2.39°) and therefore is not designed to resist ponding.

8.1.2 Wind Loads

The structure within this parametric study is restricted to regions where the Basic Wind Speed, V, has a 3-second gust wind speed of 90mph or less. This constraint allows for the structure to be located in non-hurricane prone regions. Surface Roughness Category B is used due to the generality of the tilt-up structure used as a warehouse being located within an urban or suburban area with other structures nearby. Figure 8-3 illustrates regions within the United States that meet this criterion.

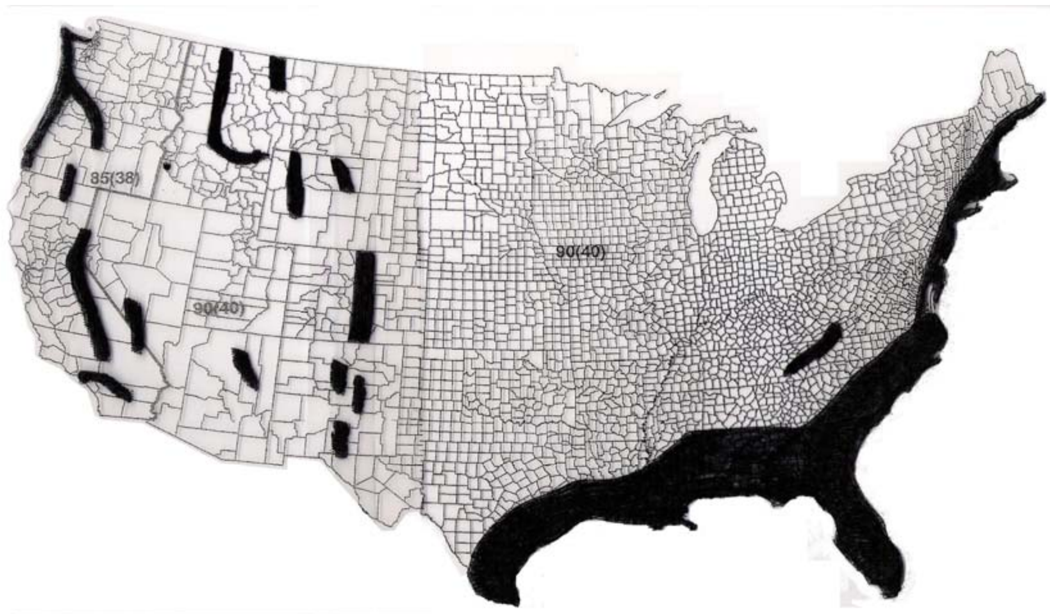

Figure 8-3. Wind Loads

Figure courtesy of ASCE/SEI 7-05

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Figure 8-3 is the United States map illustrating basic wind speed from ASCE 7-05 Figure 6-1. The regions that have been darkened exceed a 3-second gust of 90 mph.

The Analytical Procedure in ASCE 7-05 Section 6.5 is used to determine wind pressures used to calculate both base shear and components and cladding forces. The wind pressure for Components and Cladding (C&C) determined from ASCE 7-05 Section 6.5.11.2.2 is 27.5psf. This wind pressure is applied along a one-foot strip of the wall panel in the out-of-plane direction. The wind pressure for the Main Wind-Force Resisting System (MWRFS) determined from ASCE 7-05 Section 6.5.11.2.1 is used to determine the in-plane shear force for the tilt-up wall panel. Both the C&C and MWFRS forces for out-of-plane and in-plane loads, respectively, are compared to the forces determined from seismic load calculations. This is discussed in Section 8.1.4. Appendix B illustrates the calculations to determine the wind loads for the 32 ft (9.75 m) panel supported on continuous footings in Dallas.

8.1.3 Seismic Loads

Tilt-up panels are load bearing walls. The scope of this report includes a seismic force resisting system of Ordinary Precast Shear Walls. This excludes any special seismic detailing requirements to be used within the wall panel design. In order to designate the tilt-up wall panel as Ordinary Precast Shear Walls, the structure must be located in Seismic Design Category A or B. Seismic Design Categories C and higher require Intermediate Precast Shear Walls to be used for the seismic force resisting system. An Occupancy Category II is used for the structure; therefore, no occupancies listed within ASCE 7-05 Table 1-1 exist in the structure. An Importance Factor for wind and seismic loads of 1.0 is used accordingly with Occupancy Category II. From these parameters, the 0.2s and 1.0s spectral response accelerations, S_s and S_1, can be derived. The structure in this report must be located in regions with S_s values less than or equal to 0.3g and S_1 values less than or equal to 0.08g. Figure 8-4 and Figure 8-5 illustrate the regions within the United States that meet the criteria for S_S and S_1, respectively.

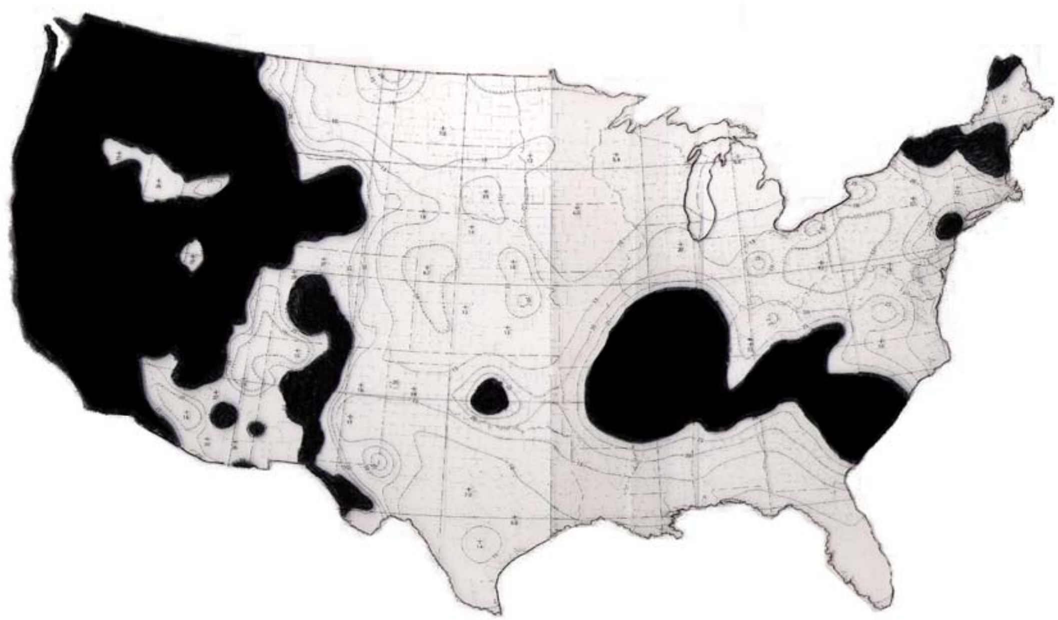

Figure 8-4. S_s Values
Figure courtesy of ASCE/SEI 7-05
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Figure 8-4 is the United States map illustrating maximum considered earthquake ground motion for a 0.2 sec spectral response from the ASCE 7-05 Figure 22-1. The regions that have been darkened exceed an $\boldsymbol{S_s}$ value of 0.3g.

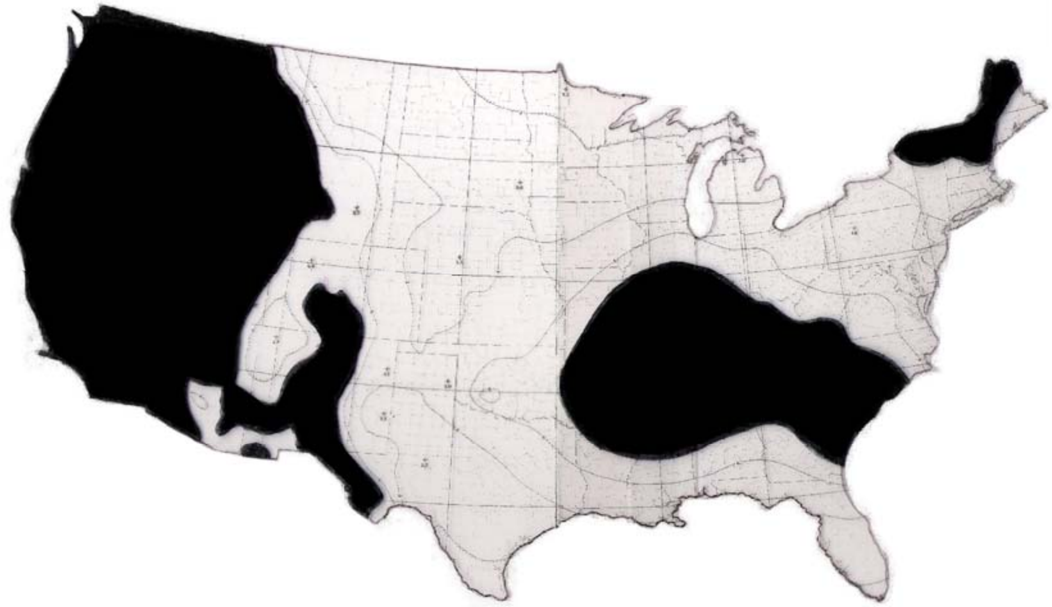

Figure 8-5. S_1 Values
Figure courtesy of ASCE/SEI 7-05
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Figure 8-5 is the United States map illustrating maximum considered earthquake ground motion for a 1.0 sec spectral response from the ASCE 7-05 Figure 22-2. The regions that have been darkened exceed an S_1 value of 0.8g.

The out-of-plane seismic force applied to the wall panel is $0.4\boldsymbol{S_{DS}I}$., in accordance with ASCE 7-05 Section 12.11.1 Table 8-6 illustrates this force for each respective building. This force is compared with the C&C wind pressure to determine the governing out-of-plane loading for the tilt-up wall panels. The Equivalent Lateral Force Procedure (ELFP) in the ASCE 7-05 Section 12.8 is used to determine the base shear. This base shear is used to determine the in-plane forces for the tilt-up wall panels. The ELFP base shear and the MWFRS base shear are compared to determine the governing in-plane shear loading for the tilt-up wall panels. Appendix

C illustrates the calculations to determine the seismic loads for the 32 ft (9.75 m) panel supported on continuous footings in Dallas.

8.1.4 Governing Loads

After all loads induced on the building structure have been determined, an evaluation is done to determine the governing loads for axial, lateral and shear forces. This evaluation compares roof live load versus snow load for axial forces, out-of-plane wind pressures versus out-of-plane seismic pressures for lateral forces, and wind base shear versus seismic base shear for in-plane shear forces.

Table 8-4 illustrates the governing roof live loads from Table 8-2 and Table 8-3. The total axial gravity load is the combination of the roof dead load, Table 8-1, governing live load, Table 8-4, and the effective panel self weight, Table 8-5. The effective panel self weight is the weight of the tilt-up panel above the design section (centerline of the unbraced length).

Table 8-4. Governing Roof Live Loads

City	Panel Height (ft)	North & South Panels Governing Roof Live Load (klf)	East & West Panels Governing Roof Live Load (klf)
Dallas	32	0.28	0.09
Dallas	40	0.28	0.09
Denver	32	0.47	0.11
Denver	40	0.47	0.11
Kansas City	32	0.47	0.11
Kansas City	40	0.47	0.11

Table 8-5. Panel Self-Weight Loads

Panel Height (ft)	Panel Thickness (in)	Panel Self-Weight above Mid-Height (k)	Axial Load Self Weight (klf)
32	7 1/4	37.0	1.54
32	9 1/4	47.2	1.97
32	11 1/4	57.4	2.39
40	7 1/4	45.7	1.90
40	9 1/4	58.3	2.43
40	11 1/4	70.9	2.95

The lateral forces applied to the panel for design are the governing case of the C&C pressure and the seismic out-of-plane pressure. Table 8-6 illustrates the seismic pressure of both the 32 ft (9.75 m) panel and 40 ft (12 m) panel at thicknesses of 7 ¼ in (18 cm), 9 ¼ in (23 cm), and 11 ¼ (28 cm). The C&C analysis yields a wind pressure of 27.5 psf for both panel heights of all three thicknesses in each of the three designated cities. This wind pressure is compared to the seismic pressure in Table 8-6. The wind pressure therefore governs as the lateral force for the panel design.

Table 8-6. Seismic Out-of-Plane Forces

City	Panel Height (ft)	Panel Thickness (in)	$0.4S_{DS}I$ (psf)
Dallas	32	7 1/4	3.34
Dallas	32	9 1/4	4.26
Dallas	32	11 1/4	5.18
Dallas	40	7 1/4	3.34
Dallas	40	9 1/4	4.26
Dallas	40	11 1/4	5.18
Denver	32	7 1/4	8.30
Denver	32	9 1/4	10.59
Denver	32	11 1/4	12.88
Denver	40	7 1/4	8.30
Denver	40	9 1/4	10.59
Denver	40	11 1/4	12.88
Kansas City	32	7 1/4	4.97
Kansas City	32	9 1/4	6.34
Kansas City	32	11 1/4	7.71
Kansas City	40	7 1/4	4.97
Kansas City	40	9 1/4	6.34
Kansas City	40	11 1/4	7.71

Load-bearing tilt-up panels act as shear walls. The tilt-up walls in this report are classified as Ordinary Precast Shear Walls. A Seismic Design Category of A or B is needed in order to use Ordinary Precast Shear Walls. Table 8-7 illustrates the resultant base shear forces determined from both MWFRS and ELFP analyses. To compare these two analyses, the MWFRS shear values must be divided by the wind directionality factor, K_d, and the ELFP shear values must be multiplied by 0.7. The wind directionality factor accounts for two effects: the reduced probability of maximum winds coming from any given direction and the reduced probability of the maximum pressure coefficient occurring for any given wind direction. The wind directionality factor was included in the existing wind load factor 1.3 in ASCE 7-95. The ASCE 7-05 has separated K_d from the load factor. Once the wind directionality factor is removed, the wind base shear in allowable stress design can be compared to the seismic base shear multiplied by 0.7, which is the conversion factor to convert seismic loads in strength design to allowable stress design.

Table 8-7. Governing Base Shear

City	Height (ft)	Direction	Wind Base Shear (k)	Seismic Base Shear (k)	Wind Base Shear / Kd (k)	Seismic Base Shear x 0.7 (k)	Governing Base Shear (k)
Dallas	32	Transverse	205.46	56.89	241.72	39.82	Wind
Dallas	32	Longitudinal	100.89	56.89	118.69	39.82	Wind
Dallas	40	Transverse	256.96	80.16	302.31	56.11	Wind
Dallas	40	Longitudinal	126.25	80.16	148.53	56.11	Wind
Denver	32	Transverse	205.46	142.23	241.72	99.56	Wind
Denver	32	Longitudinal	100.89	142.23	118.69	99.56	Wind
Denver	40	Transverse	256.96	146.53	302.31	102.57	Wind
Denver	40	Longitudinal	126.25	146.53	148.53	102.57	Wind
Kansas City	32	Transverse	205.46	84.68	241.72	59.28	Wind
Kansas City	32	Longitudinal	100.89	84.68	118.69	59.28	Wind
Kansas City	40	Transverse	256.96	98.82	302.31	69.17	Wind
Kansas City	40	Longitudinal	126.25	98.82	148.53	69.17	Wind

8.2 Alternate Design of Slender Walls

Typical tilt-up walls carry very small axial loads. These axial loads are commonly roof loads, but occasionally include floor loads as well. The critical loading condition, whether seismic or wind, will result from lateral loads. Lateral instability controls the design of slender tilt-up panels. This lateral instability occurs when the lateral deflection resulting from the large lateral load moment combines with the small axial load to produce the secondary moment caused by ***P-Δ*** effects. The lateral load moment now includes the ***P-Δ*** secondary moment. This results in increased deflection and the possibility of progressive collapse. (Johnson 1979)

The ACI 318-05 has provisions for the design of slender load-bearing walls. Section 14.8, Alternate Design of Slender Walls, has appeared in the Uniform Building Code (UBC) since 1988 and in the International Building Code (IBC) since 2003. This alternate design method for slender walls is based on the experimental research reported in the document Test Report of Slender Walls (Athey 1982). When the following limitations are met, the alternate design method in Section 14.8 is considered to satisfy Section 10.10 when flexural tension controls the wall design. When one or more of the following limitations are not satisfied, the wall must be designed by the provisions of ACI 318-05 Section 14.4. These limitations are:

1. The wall panel shall be simply supported, axially loaded, and subjected to an out-of-plane uniform lateral load. The maximum moments and deflections shall occur at the mid-height of the wall (14.8.2.1)
2. The cross-section shall be constant over the height of the panel (14.8.2.2)
3. The wall cross-section shall be tension-controlled (14.8.2.3)
4. Reinforcement shall provide a design moment strength ϕM_n greater than or equal to M_{cr}, where M_{cr} is the moment causing flexural cracking due to the applied lateral and vertical loads. The cracking moment shall be obtained using the modulus of rupture, f_r given by Equation 9-10 (14.8.2.4)
5. Concentrated gravity loads applied to the wall above the design flexural section shall be distributed over a width equal to the lesser of (a) the bearing width plus a width on each side that increases at a slope of 2 vertical to 1 horizontal down to the design flexural section or (b) the spacing of the concentrated loads. The distribution width shall not extend beyond the edges of the wall panel (14.8.2.5)
6. The vertical stress P_u/A_g at the mid-height section shall not exceed $0.06f'_c$ (14.8.2.6)

8.2.1 Check Load Cases

Building structures and structural members shall be designed in accordance with ACI 318-05 Section 9.2, Required Strength. In this parametric study, design strengths for the tilt-up panels shall be greater than or equal to the required strengths calculated for the factored loads and forces from the applicable combinations in ACI 318-05 Section 9.2.1. The tilt-up panels must resist the governing axial, lateral, and shear loads of dead, roof live, and wind. The following combinations are used to determine the greatest required strength, ***U***:

Load Case 1	$1.2D + 1.6L_r + 0.8W$	(ACI Equation 9-3)
Load Case 2	$1.2D + 0.5L_r + 1.6W$	(ACI Equation 9-4)
Load Case 3	$0.9D + 1.6W$	(ACI Equation 9-6)

Each of these load case combinations dictates a governing loading condition. Load Case 1 determines the greatest applied force due to gravity loads. Load Case 2 determines the greatest applied force due to lateral sliding. Load Case 3 determines the greatest applied force due to overturning.

The dead load and roof live load in these three load cases determine the axial load applied to the tilt-up panel. The wind load in these three load cases determines the lateral load applied to the tilt-up panel. The combined axial and lateral loads result in secondary moments. The vertical reinforcing must be designed in a manner as to resist this magnified moment.

8.2.2 Check Design Moment Strength

The design moment strength, $\boldsymbol{\phi M_n}$, for combined axial and flexural loads at the mid-height cross-section must be greater than or equal to the total factored moment, $\boldsymbol{M_u}$, at this section. The vertical stress at the mid-height cross-section of the panel must be less than or equal to six percent of the concrete compressive design strength per ACI 318-05 Section 14.8.2.6.

$$\frac{P_{um}}{A_g} < 0.06 f'_c \tag{8.1}$$

Where

P_{um} = total factored axial load (lbs)

A_g = gross area of concrete section (in^2)

f'_c = specified compressive strength of concrete (psi)

The design moment strength of the panel, $\boldsymbol{\phi M_n}$, is directly proportional to the area of effective tension reinforcement, $\boldsymbol{A_{se}}$, by the equation:

$$\phi M_n = \phi A_{se} f_y \left(d - \frac{a}{2} \right) \tag{8.2}$$

where

ϕ = 0.90 for tension-controlled sections as defined by ACI 318-05 Section 14.8.2.3 and ACI 318-05 Section 9.3.2.1

$$A_{se} = \frac{(P_{um} + A_s f_y)}{f_y} \tag{8.3}$$

A_s = area of longitudinal tension reinforcement (in^2)

f_y = specified yield strength of reinforcement (psi)

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

$$a = \frac{A_{se} f_y}{0.85 f'_c l_w} \tag{8.4}$$

l_w = horizontal length of tilt-up wall panel

In order for the structural engineer to design the tilt-up panel by the above method prescribed in ACI 318-05 Section 14.8, the wall must be tension-controlled. According to ACI 318-05 Section 10.3.4, sections are tension-controlled if the net tensile strain in the extreme tension steel, ε_t, is equal to or greater than 0.005 in/in when the concrete in compression reaches its assumed strain limit of 0.003 in/in. The strain in the extreme tension steel, ε_t, can be determined from strain compatibility as shown in Figure 8-6.

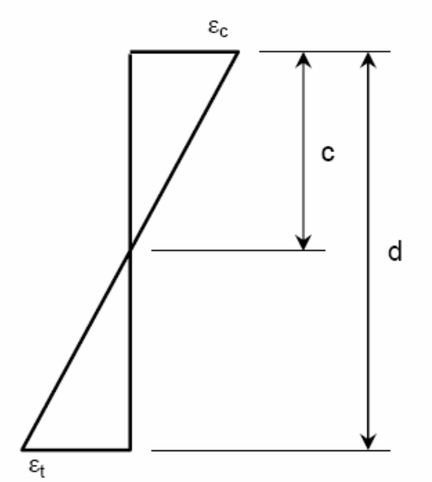


Figure 8-6. Strain Compatibility Diagram

Where

$$\varepsilon_t = \varepsilon_c \left(\frac{d}{c} \right) - \varepsilon_c \tag{8.5}$$

ε_c = 0.003 in/in

$$c = \frac{a}{0.85} \tag{8.6}$$

8.2.3 Check Minimum Vertical Reinforcement

Minimum vertical reinforcement requirements are established for two reasons. The first reason is to ensure the reinforcement within the wall provides a design moment strength greater than the cracking moment according to ACI 318-05 Equation 14-2.

$$\phi M_n \geq M_{cr} \tag{8.7}$$

Where

$$M_{cr} = \left(\frac{f_r I_g}{y_t} \right) \tag{8.8}$$

$$f_r = 7.5\sqrt{f'_c} \tag{8.9}$$

$$I_g = \frac{1}{12} bh^3 \tag{8.10}$$

The second reason minimum wall reinforcement is required is primarily for control of cracking due to shrinkage and temperature stresses. Walls must contain both vertical and horizontal reinforcement to resist these stresses. The minimum ratio of vertical reinforcement area to gross concrete area shown in Equation 8.11 shall be 0.0012 for deformed bars not larger than No. 5 with $\boldsymbol{f_y}$ not less than 60,000 psi or 0.0015 for other deformed bars according to ACI 318-05 Section 14.3.2.

$$\rho_l = \frac{A_s}{b_w t} \tag{8.11}$$

Flexural members shall also provide a minimum amount of tensile reinforcement in accordance with ACI 318-05 Section 10.5.1, where the ratio of vertical reinforcement area to net concrete area shown in Equation 8.12 shall be greater than or equal to the larger values of Equation 8.13 and Equation 8.14.

$$\rho = \frac{A_s}{b_w d} \tag{8.12}$$

$$\rho_{\min} = \frac{3\sqrt{f'_c}}{f_y} \tag{8.13}$$

$$\rho_{\min} = \frac{200}{f_y} \tag{8.14}$$

8.2.4 Check Applied Ultimate Moment

The design moment strength, ϕM_n, for combined axial and flexure loads at the mid-height cross-section must be greater than or equal to the total factored moment, M_u, at this section per ACI 318-05 Section 14.8.3. The total factored moment, M_u, includes the ***P-Δ*** effects and is determined as follows:

$$M_u = M_{ua} + P_u \Delta_u \tag{8.15}$$

where

$$M_{ua} = \frac{w_u l_c^{\,2}}{8} + \frac{P_{u1} e}{2} \tag{8.16}$$

M_{ua} = factored moment at the mid-height section of the wall due to factored lateral and eccentric vertical loads

$$P_u = P_{u1} + \frac{P_{u2}}{2} \tag{8.17}$$

P_{u1} = factored applied gravity load

P_{u2} = factored self-weight of the wall (total)

e = eccentricity of applied gravity load

w_u = factored uniform lateral load

The factored moment, M_u, can be rewritten as:

$$M_u = \frac{w_u l_c^{\,2}}{8} + \frac{P_{u1} e}{2} + \left(P_{u1} + \frac{P_{u2}}{2} \right) \Delta_u \tag{8.18}$$

where

$$\Delta_u = \frac{5 M_u l_c^{\,2}}{(0.75) 48 E_c I_{cr}} \tag{8.19}$$

l_c = vertical distance between supports

E_c = modulus of elasticity of concrete defined in ACI 318-05 Section 8.5

$$I_{cr} = n A_{se} (d - c)^2 + \frac{l_w c^3}{3} \tag{8.20}$$

$$n = \frac{E_s}{E_c} \tag{8.21}$$

E_s = modulus of elasticity of steel reinforcement

The total factored moment, $\boldsymbol{M_u}$, shall be obtained by iteration of deflections or by direct calculation using ACI 318-05 Equation 14-5. This equation provides a conservative result compared with the result derived from the iterative process.

$$M_u = \frac{M_{ua}}{1 - \dfrac{5P_u l_c^{\,2}}{(0.75)48E_c I_{cr}}} \tag{8.22}$$

Figure 8-7 illustrates the analysis of the wall in accordance to the provisions of ACI 318-05 Section 14.8 for the case of additive lateral and gravity load effects.

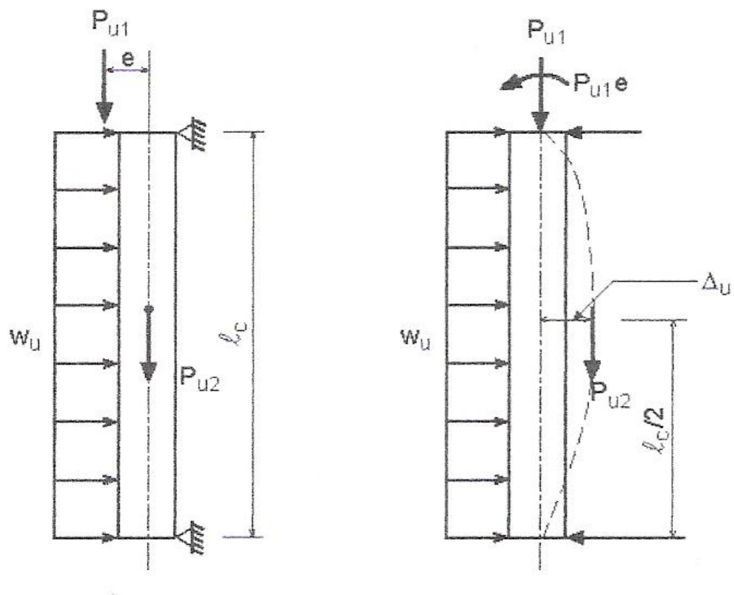


Figure 8-7. Second-Order Effect Wall Analysis

Photo courtesy of Notes on ACI 318-05: Building Code Requirements for Structural Concrete with Design Applications
(PCA 2005)

8.2.5 Check Service Load Deflection

The deflection requirements of ACI 318-05 Section 14.8.4 must also be satisfied in addition to satisfying the strength requirement of ACI 318-05 Equation 14-3. The maximum deflection due to service loads is calculated in accordance to ACI 318-05 Equation 14-8.

$$\Delta_s = \frac{5Ml_c^{\,2}}{48E_c I_e} \tag{8.23}$$

Where

$$M = \frac{M_{sa}}{1 - \frac{5P_s l_c^2}{48E_c I_e}} \tag{8.24}$$

M_{sa} = maximum unfactored applied moment due to service loads, not including ***P-Δ*** effects

P_s = unfactored axial load at the design (mid-height) section including effects of self-weight

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} < I_g \tag{8.25}$$

M_a = M

An iterative process is required to determine the maximum service load deflection, $\boldsymbol{\Delta_s}$, at mid-height. This deflection shall not exceed $\boldsymbol{l_c}$ /150 in accordance with ACI 318-05 Section 14.8.4.

8.2.6 Check Horizontal Reinforcing

Minimum horizontal reinforcement requirements are required in accordance to ACI 318-05 Section 14.3.3. This section states that the minimum ratio of horizontal reinforcement area to gross concrete area, $\boldsymbol{\rho_t}$, shall be 0.0020 for deformed bars not larger than No. 5 with $\boldsymbol{f_y}$ not less than 60,000 psi, or 0.0025 for other deformed bars. The minimum area of horizontal reinforcing steel is shown in Equations 8.26 and 8.27.

$$A_s = 0.002A_g \tag{8.26}$$

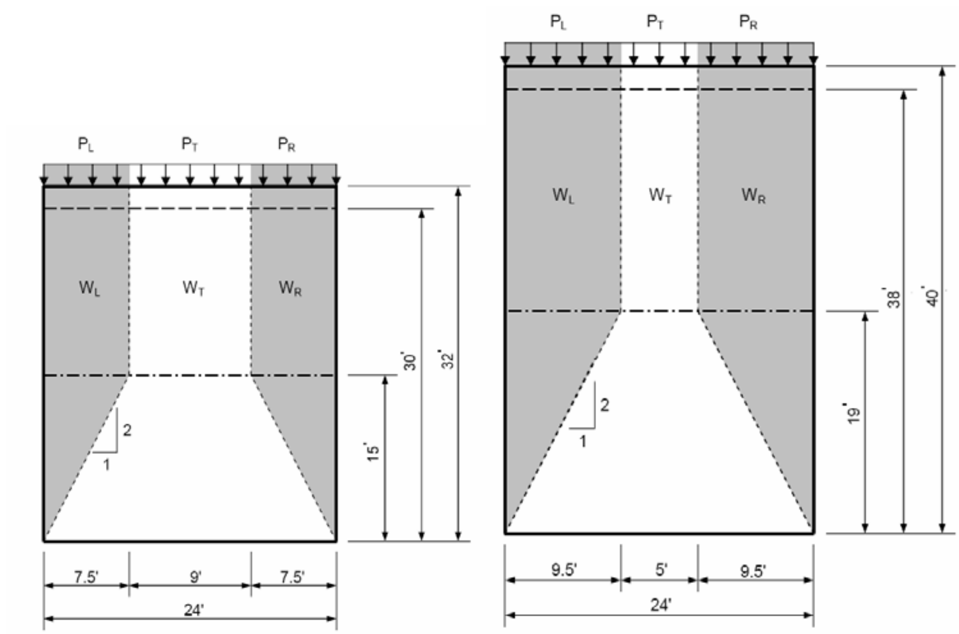
$$A_s = 0.0025A_g \tag{8.27}$$

The horizontal reinforcement shall not be spaced farther apart than three times the wall thickness, or farther apart than 18 in as specified in ACI 318-05 Section 14.3.5.

8.2.7 Check Foundation Support

The foundation support for tilt-up wall panels can either be continuous or isolated. Continuous wall footings provide a uniform bearing support for the tilt-up panel and its supporting loads. Isolated footings, such as spread footings or drilled piers, provide support to the panels near the edge of both sides. The panel must then be designed to span from support to support.

Simplified tilt-up panel design analysis assumes continuous support, so the effective panel width must be reduced when the panel is supported on isolated footings. This reduced effective width is similar to conditions of load concentrations on the panel or large openings within the panel. Since the panel loading is symmetric, the effective panel width at the centerline of unbraced length can be determined from sloping lines of one horizontal to two vertical. Figure 8-8 illustrates this condition.



(a) (b)

Figure 8-8. Panels Supported on Isolated Footings

P_L = roof axial load above reduced effective width for the left isolated support

P_R = roof axial load above reduced effective width for the right isolated support

P_T = roof axial load transferred to the supports through the tension tie

W_L = self weight of reduced effective width for the left isolated support

W_R = self weight of reduced effective width for the right isolated support

W_T = self weight of panel transferred to the supports through the tension tie

A strut-and-tie model can be used to analyze a tilt-up wall panel supported by isolated footings. Strut-and-tie models consist of two concrete compressive struts, longitudinal reinforcement acting as a tension tie, and joints commonly referred to as nodes. The concrete surrounding a node is called a nodal zone, which transfers the forces from the inclined struts to other struts, to ties, and to the reactions. A strut-and-tie model is an idealized model of a portion of the structure being analyzed that satisfies the following:

1. Embodies a system of forces that is in equilibrium with a given set of loads
2. The factored-member forces at every section in the struts, ties, and nodal zones do not exceed the corresponding factored-member strengths for the same sections
3. The structure has sufficient ductility to make the transition from elastic behavior to enough plastic behavior to redistribute the factored internal forces into a set of forces that satisfy items (ACI 551 Jun 2005)(ACI 551 Feb 2003) (MacGregor 2005)

The portions of the two panels shown in Figure 8-8 which are shaded grey, $\boldsymbol{W_L}$, $\boldsymbol{W_R}$, $\boldsymbol{P_L}$, and $\boldsymbol{P_R}$, represent the loads transferred through bearing stress to the isolated footing. The unshaded portions of the two panels, $\boldsymbol{W_T}$ and $\boldsymbol{P_T}$, represent the loads which are analyzed through a strut-and-tie model. Figure 8-9 illustrates the simplified truss used to analyze the strut-and-tie model.

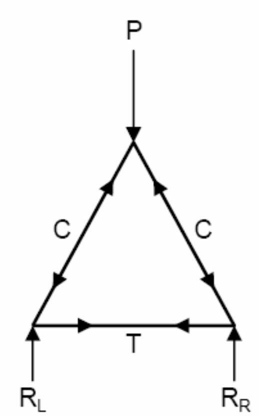


Figure 8-9. Strut-and-Tie Truss Model

The effective self weight of the panel and roof axial loads, $\boldsymbol{W_T}$ and $\boldsymbol{P_T}$, respectively, must be transferred to the isolated footings through the compressive struts and tension tie. $\boldsymbol{W_T}$ and $\boldsymbol{P_T}$ are represented in the strut-and-tie simplified truss by $\boldsymbol{P}$. The compressive struts and tension tie are represented by $\boldsymbol{C}$ and $\boldsymbol{T}$, respectively. The isolated footing reactions are represented by $\boldsymbol{R_L}$ and $\boldsymbol{R_R}$.

The tension tie is designed as a tension member in a strut-and-tie model. This tie consists of reinforcement plus a portion of the surrounding concrete that is concentric with the axis of the tie. The concrete is not used to resist the axial force in the tie, but is included to define the zone in which the forces in the struts and ties are to be anchored. This concrete portion aids in the transfer of loads from struts to ties or to bearing areas through bond with reinforcement. The steel reinforcement alone resists the axial tension within the tie. The nominal strength of the tie without prestressed reinforcement is determined from ACI 318-05 Equation A-6.

$$F_{nt} = A_{ts} f_y \tag{8.28}$$

Where

F_{nt} = nominal strength of a tie, lb

A_{ts} = area of nonprestressed reinforcement in a tie, in^2

The ultimate tension force in the tie, $\boldsymbol{T_u}$, must be less than or equal to the design strength of the tie as illustrated in Equation 8.29.

$$\phi F_{nt} \geq T_u \tag{8.29}$$

Where

T_u = ultimate tension force determined from the idealized truss shown in Figure 8-9

8.2.8 Check In-Plane Shear Forces

The load-bearing tilt-up panels within a building structure provide wall sections to be specifically designed as shear walls to resist lateral forces. The seismic story shearwall force, $\boldsymbol{F_x}$, for the buildings in this parametric study is equal to the seismic base shear force, $\boldsymbol{V}$, because the buildings are one-story structures. Unit shear is one of the criteria used to design shearwalls. Tilt-up wall panels are designated for the Lateral Force Resisting System (LFRS) as ordinary load-bearing precast shearwalls in both the longitudinal and transverse directions of the building structure. The diaphragm structure is permitted to be modeled as flexible for analysis and lateral force distribution. The deflected shape of the roof diaphragm is illustrated in Figure 8-10 A. The reaction of the diaphragm on the end walls of both longitudinal and transverse direction is the reaction of a uniformly loaded simple beam with a span length equal to the distance between the shear walls. For a simple beam, the maximum internal shear is equal to the external reaction. This can be shown by the shear diagram for a simple beam illustrated in Figure 8-10 B. The maximum total shear is converted to a unit shear by distributing the shear along the length of the wall used to resist the lateral force as illustrated in Figure 8-10 C. This unit shear is then multiplied by the length the panel to determine the amount of lateral force the individual panel must resist. The shear force is applied at the roof diaphragm, as illustrated in Figure 8-11.

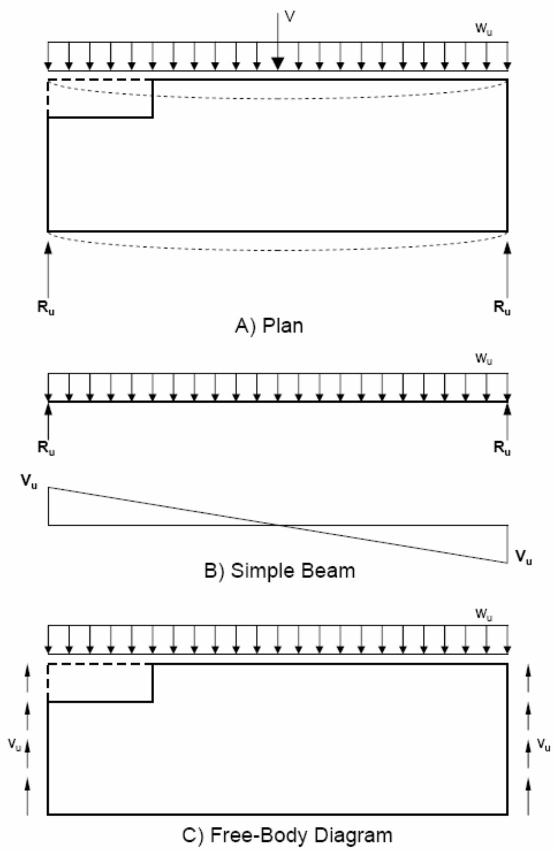


Figure 8-10. Shear Force Model

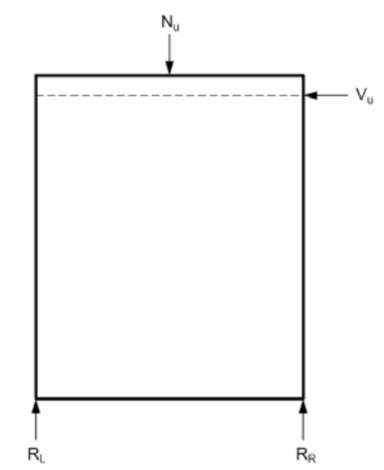


Figure 8-11. Uplift Diagram

Tilt-up panels acting as shear walls need to be checked for overturning. The shear force, V_u, applied at the diaphragm is multiplied by the height to the diaphragm to find the overturning moment. The shear at the roof diaphragm induces a moment which tends to make the wall overturn, and resistance to overturning often is provided by the dead load of the structure. Tilt-up panels are advantageous for resisting the overturning moment because of their extensive self-weight. Through statics, the reactions at the base of the panel, R_L, and R_R, can be found. These forces will be compared to the load-bearing forces to determine the governing load that must be transferred to the foundation through connection design. The reactions determined from overturning are computed using the load combination 0.9D + 1.6W. For isolated footings, the force transferred into the foundation is equal to R_R and R_L. For continuous footings, the reactions are distributed along the base of the wall since the foundation provides uniform support.

8.2.9 Check Load Bearing Forces

The load-bearing force at the connection to the foundation support is determined from the load combination 1.2D + 1.6L. The self-weight of the wall and the axial roof load shall be computed. For isolated footings, half of this load is transferred to each foundation support. For continuous footings, the load is supported uniformly along the foundation. These two scenarios are compared with the reactions determined from the overturning moment to design the connection to the foundation support for the governing condition.

8.3 Design Process

The tilt-up panels in this parametric study are designed in accordance with ACI 318-05 Section 14.8. The alternate design of slender walls analysis is used to determine the flexural reinforcement needed for the panel to resist the total factored moment induced from the governing lateral forces. The panel design process described in Section 8.2 lists the steps taken to design the panels for the parameters given. The parameters of the study are listed in Section 8.3.1. Appendix D illustrates the calculations for the 32 ft (9.75 m) panel supported on continuous footings in Dallas.

8.3.1 Parameters

The building structure shown in Figure 1-1 is located in three regions for comparison of tilt-up panel design to determine how this design is affected by foundation support. Dallas, Denver, and Kansas City are the three locations chosen to represent a broad overview of the region within the United States in which the scope of this report applies. The scope of this report is restricted to Ordinary Precast Load Bearing Walls designed in regions where wind design governs over seismic design. These three regions also provide variances in soil conditions, including stiff soils in Kansas City, and expansive soils in Denver and Dallas, which allows for a comparison of shallow and deep foundation systems.

Two panel heights will be designed at each location. The two heights are 32 ft (9.75 m) and 40 ft (12 m). Tilt-up panels are common structural components for single story warehouse and industrial facilities. These two heights are similar to typical conditions for these facilities. The height difference also allows for a comparison of different slenderness and its affect on ***P-Δ*** effects and reinforcing. Furthermore, these two panel heights are both evaluated with three thicknesses: 7 ¼ in (18 cm), 9 ¼ in (23 cm), and 11 ¼ in (28 cm). To determine the most economical panel design, the cost of concrete and steel required for the panel is evaluated given recent pricing data in the three respective locations. The last variable for the panel design is the reinforcement mats. Single layer mats and double layer mats are evaluated in both the 7 ¼ in (18 cm) panel and 9 ¼ in (23 cm) panel. The 11 ¼ in (28 cm) panel is evaluated with double mats only. ACI 318-05 Section 14.3.4 states that walls more than 10 in (25 cm) thick, except basement walls, shall have reinforcement for each direction placed in two layers parallel with faces of the wall.

8.3.2 Limiting Factors

When the requirements in ACI 318-05 Section 14.8 are all satisfied, the alternative design of slender walls may be used. The panels within this parametric study meet the requirements of ACI 318-05 Section 14.8 and follow the alternative design of slender walls. Several factors have been noted which control design of these panels, and become limiting factors for reinforcement detailing. The following requirements of ACI 318-05 Section 14.8 are found to control the design for the panels:

1. Section 14.3.2: $\rho_l > \rho_{min\ req'd}$
2. Section 14.3.4: Two layers of reinforcement for walls more than 10 in thick
3. Section 14.3.5: Vertical reinforcement spaced no farther apart than 18 in
4. Section 14.8.2.3: Wall shall be tension-controlled
5. Section 14.8.3: $\phi M_n \geq M_u$
6. Section 14.8.4: $\Delta_s \leq l_c / 150$

ACI 318-05 Section 14.3.2 is the limiting design requirement for panels with two layers of reinforcement. The design moment capacity, $\boldsymbol{\phi M_n}$, for these panels exceeds the total factored moment, $\boldsymbol{M_u}$, applied to the panels; however, since two layers of reinforcement are used, the moment arm between the compression force and tension force is larger, which allows for less area of steel. The ratio of area of steel to gross area of concrete needs to exceed the ratios from ACI 318-05 Section 14.3.2 in order to meet minimum steel requirements.

ACI 318-05 Section 14.3.4 is the limiting design requirement for the panel thickness when one layer of reinforcement is utilized. The parametric study consists of three panel thicknesses: 7 ¼ in (18 cm), 9 ¼ in (23 cm), and 11 ¼ in (28 cm). One layer of reinforcement mats can only be utilized in the 7 ¼ in (18 cm) and 9 ¼ in (23 cm) panels. The 11 ¼ (28 cm) in panel must have two layers of reinforcement mats to meet minimum steel requirements.

ACI 318-05 Section 14.3.5 is the limiting design requirement when reinforcement bars could be spaced further than 18 in apart and maintain design moment capacity, $\boldsymbol{\phi M_n}$, larger than total factored applied moment, $\boldsymbol{M_u}$. Vertical and horizontal reinforcement shall not be spaced farther than 18 in (46 cm) apart or three times the wall thickness. For wall thicknesses greater than 6 in (15 cm), the 18 in (46 cm) spacing requirement governs.

ACI 318-05 Section 14.8.2.3 is the limiting design requirement for two situations when the panel height is 40 ft (12 m). These two situations are 7 ¼ in (18 cm) panel with one layer of steel supported on continuous footings and 7 ¼ in (18 cm) panel with one layer of steel supported on isolated footings. The 40 ft (12 m) panels have larger slenderness values and unbraced lengths than the 32 ft (9.75 m) panels. More reinforcement is required to resist the lateral forces and ***P-Δ*** effects. These two wall designs are no longer tension-controlled for the amount of steel reinforcement needed to attain a design moment capacity, ϕM_n, larger than the total factored applied moment, M_u. No panel design solutions are found for these two panels because of this requirement.

ACI 318-05 Section 14.8.3 is the limiting design requirement for all panel design situations when the previous four limiting factors have not yet been reached. The total factored moment, M_u, exceeds the design moment capacity, ϕM_n, with respect to Load Combination 2. This load combination, $1.2D + 0.5L_r + 1.6W$, is the governing combination to determine lateral resistance. This combination governs over the other two combinations because the panel self weight is significantly larger with respect to the roof live loads. Load Combination 3, $0.9D + 1.6W$, can govern for walls of lighter material, because less axial load applied provides less compression to resist bending. For tilt-up concrete wall panels, Load Combination 2 governs over Load Combination 3 because the panel self-weight increases the ***P-Δ*** effect faster than conventional materials for walls.

ACI 318-05 Section 14.8.4 is the limiting design requirement for one panel design situation. This panel is 40 ft (12 m) in height, 7 ¼ in (18 cm) thick, and supported on isolated footings. In order for the service load deflection to be less than l_c /150, the panel needs an amount of reinforcement that no longer allows the panel to be tension-controlled. This is due to the large slenderness, unbraced length, and reduced effective panel width due to isolated footings, as discussed in Section 8.2.7.

Table 8-8 illustrates each panel configuration designed in the parametric study and Appendix E illustrates each panel design along with its corresponding limiting factor.

Table 8-8. Parametric Study Panel Designs

Panel Height (ft)	Panel Thickness (in)	Foundation Support	Rows of Steel	Vertical Reinf. Bar Size	Vertical Spacing (in)
32	7.25	Continuous	1	6	10
32	7.25	Continuous	1	7	12
32	7.25	Continuous	1	8	18
32	9.25	Continuous	1	5	10
32	9.25	Continuous	1	6	12
32	9.25	Continuous	1	7	18
32	7.25	Continuous	2	5	12
32	7.25	Continuous	2	6	18
32	9.25	Continuous	2	5	10
32	9.25	Continuous	2	6	12
32	9.25	Continuous	2	7	18
32	11.25	Continuous	2	6	12
32	11.25	Continuous	2	7	18
32	7.25	Isolated	1	6	10
32	7.25	Isolated	1	7	12
32	7.25	Isolated	1	8	18
32	9.25	Isolated	1	5	10
32	9.25	Isolated	1	6	12
32	9.25	Isolated	1	7	18
32	7.25	Isolated	2	5	12
32	7.25	Isolated	2	6	18
32	9.25	Isolated	2	5	10
32	9.25	Isolated	2	6	12
32	9.25	Isolated	2	7	18
32	11.25	Isolated	2	6	12
32	11.25	Isolated	2	7	18
40	7.25	Continuous	1	NG	NG
40	9.25	Continuous	1	7	10
40	9.25	Continuous	1	8	12
40	7.25	Continuous	2	5	10
40	7.25	Continuous	2	6	12
40	7.25	Continuous	2	7	18
40	9.25	Continuous	2	5	10
40	9.25	Continuous	2	6	12
40	9.25	Continuous	2	7	18
40	11.25	Continuous	2	6	12
40	11.25	Continuous	2	7	18
40	7.25	Isolated	1	NG	NG
40	9.25	Isolated	1	7	10
40	9.25	Isolated	1	8	12
40	7.25	Isolated	2	NG	NG
40	9.25	Isolated	2	5	10
40	9.25	Isolated	2	6	12
40	9.25	Isolated	2	7	18
40	11.25	Isolated	2	6	12
40	11.25	Isolated	2	7	18

8.3.3 Economic Factors

Economics drives structural design. Competition between construction materials and practice is dependent on material and labor cost. In order for one construction method to be more beneficial than another construction method, the method must have a price advantage. Tilt-up concrete provides an economic advantage over cast-in-place concrete and masonry because the amount of required formwork and scaffolding is reduced. Tilt-up concrete also provides an economic advantage in material cost. The more slender the wall, the less material required. Forty-six tilt-up panel design configurations for each location are evaluated in this parametric study. These forty-six configurations are divided into four groups: 32 ft (9.75 m) panels supported on continuous footings, 32 ft (9.75 m) panels supported on isolated footings, 40 ft (12 m) panels supported on continuous footings, and 40 ft (12 m) panels supported on isolated footings. Each division yields a most economical panel design.

The four most economical panel designs are determined from several factors: panel thickness, area of steel reinforcement, and material costs for both concrete and structural rebar. Table 8-9 and Table 8-10 illustrate the price of concrete and structural rebar from Engineering News Record Construction Economics. (ENR 2008, 2009) Prices from each quarter are obtained throughout the previous year and averaged. This average price is used to evaluate the most economic panel for design.

Table 8-9. Concrete Material Prices

4,000 psi Concrete ($/cy)						
City	February-09	November-08	August-08	May-08	February-08	Average
Dallas	97.65	95.62	94.70	94.32	94.00	95.26
Denver	93.20	91.44	89.74	89.00	89.00	90.48
Kansas City	85.00	85.00	85.00	85.00	85.00	85.00

Table 8-10. Structural Rebar Material Prices

Structural Rebar, Grade 60 #4 ($/cwt)						
City	February-09	November-08	August-08	May-08	February-08	Average
Dallas	38.25	38.25	38.03	35.97	35.98	37.30
Denver	40.55	40.55	40.55	40.00	40.00	40.33
Kansas City	42.00	62.00	62.00	44.40	40.00	50.08

Appendix F illustrates each panel configuration and its respective cost per lineal foot. It can be noted from these tables that as the panel increases in thickness, the area of steel reinforcement required decreases. This relationship is due to an increased moment of inertia as the panel thickness increases. A larger moment of inertia provides an increased resistance to bending. With regards to material cost, the concrete cost per lineal foot is substantially greater than the structural rebar cost per lineal foot. Therefore, the thinner panels with more reinforcement become more economical than the thicker panels with less reinforcement. This scenario is not always the case. A thicker panel with less reinforcement could be a more economical design in time periods when the price of steel is higher than average and the price of concrete is lower than average. Table 8-11 illustrates the panel design chosen for each location based on foundation support and panel height.

Table 8-11. Chosen Panel Designs

Location	Foundation Support	Panel Height (ft)	Panel Thickness (in)	Rows of Steel	Rebar Size	Rebar Spacing (in)	Area of Concrete (in^2 / lf)	Area of Steel (in^2 / lf)	Total Cost ($/lf)
Dallas	Continuous	32	7 1/4	1	8	18	87	0.53	$89.45
Dallas	Continuous	40	7 1/4	2	5	10	87	0.74	$122.61
Dallas	Isolated	32	7 1/4	1	8	18	87	0.53	$89.45
Dallas	Isolated	40	9 1/4	1	7	10	111	0.72	$145.37
Denver	Continuous	32	7 1/4	1	8	18	87	0.53	$87.76
Denver	Continuous	40	7 1/4	2	5	10	87	0.74	$121.36
Denver	Isolated	32	7 1/4	1	8	18	87	0.53	$87.76
Denver	Isolated	40	9 1/4	1	7	10	111	0.72	$142.89
Kansas City	Continuous	32	7 1/4	1	8	18	87	0.53	$89.39
Kansas City	Continuous	40	7 1/4	2	5	10	87	0.74	$126.22
Kansas City	Isolated	32	7 1/4	1	8	18	87	0.53	$89.39
Kansas City	Isolated	40	9 1/4	1	7	10	111	0.72	$146.20

8.3.4 Panel Detailing

Tilt-up panel design is controlled by its ability to resist lateral forces. Once the vertical reinforcement is detailed, the panel is checked for in-plane forces. The in-plane shear is resisted by the shear capacity of the tilt-up concrete panel and the horizontal reinforcement within the panel. The nominal shear strength, V_n, at any horizontal section in plane of wall shall not be taken greater than $10\sqrt{(f_c')}hd$, where h is the thickness of the wall and d is 0.8 times the length of the wall. The nominal shear strength of the concrete section, V_c, is determined from Equation (11-29) or Equation (11-30) in from ACI 318-05 Section 11.10.6.

$$V_c = 3.3\sqrt{f_c{}'}hd + \frac{N_u d}{4l_w} \tag{8.30}$$

$$V_c = \left[0.6\sqrt{f_c'} + \frac{l_w\left(1.25\sqrt{f_c'} + 0.2\dfrac{N_u}{l_w h}\right)}{\dfrac{M_u}{V_u} - \dfrac{l_w}{2}} \right] hd \tag{8.31}$$

Where

h = thickness of the wall

d = 0.8 times the length of the wall

l_w = horizontal length of tilt-up wall panel

N_u = total factored axial load (positive for compression, negative for tension)

M_u = total factored moment

V_u = total factored shear force

$M_u / V_u < 0$, Equation (11-30) does not apply

If the total factored shear force, $\boldsymbol{V_u}$, at the section is less than $\boldsymbol{0.5\phi V_c}$, shear reinforcement is not required in accordance with ACI 318-05 Section 11.10.8. Minimum horizontal reinforcement is required in accordance with ACI 318-05 Section 14.3.3, as discussed in Section 8.2.6.

For this parametric study, each panel along all four sides of the building is used as a shear wall. The total factored shear for each panel is much smaller than $\boldsymbol{0.5\phi V_c}$. Using the entire wall of panels as a shear wall allows for the total factored shear at each panel to be small. Therefore, no shear reinforcing is required, but a minimum amount of horizontal reinforcing is needed to meet the requirements of ACI 318-05 Section 14.3.3. Horizontal reinforcement uniformly distributed over the wall height is effective in resisting shear and ensuring ductile flexural failure. (Drysdale 2008) The 7 ¼ in (18 cm) panels for both the 32 ft (9.75 m) and 40 ft (12 m) panels require a #5 reinforcing bar at 18 in on center. The 9 ¼ in (23 cm) panels for both the 32 ft (9.75 m) and 40 ft (12 m) panels require a #5 reinforcing bar at 12 in on center. This horizontal reinforcement is similar for all three locations.

When the panel is supported by isolated footings, detailing is required for the tension tie near the bottom of the panel as discussed is Section 8.2.7. This tension force, determined through strut-and-tie analysis, is predominantly governed by the weight of the panel. From the strut-and-tie analyses, two #4 reinforcing bars are required to resist this tension force from the panels supported on isolated footings. For ease of construction, two #5 reinforcing bars are used for the tension tie to match with the required horizontal reinforcement of #5 reinforcing bars as previously discussed.

The vertical reinforcement in the panel which resists the out-of-plane lateral loads is also used to resist the tension due to uplift from in-plane loads. In reference to Figure 8-11, the reaction $\boldsymbol{R_R}$ may be down to resist uplift of the panel, depending on the values $\boldsymbol{V_u}$, and $\boldsymbol{N_u}$. For the building in this parametric study, uplift does not occur because the entire length of each wall resists the in-plane shear, resulting in a small $\boldsymbol{V_u}$ value for each panel. If the shear was large enough to cause uplift, the vertical reinforcement near the edge of the panel must be sufficient to resist this tension force. Strain compatibility analysis can be used to determine the stress of this vertical reinforcement.

Once the vertical reinforcement is determined to be satisfactory to resist the in-plane forces, the panels can be detailed. Figure 8-12 illustrates the detailed panels for their respective regions.

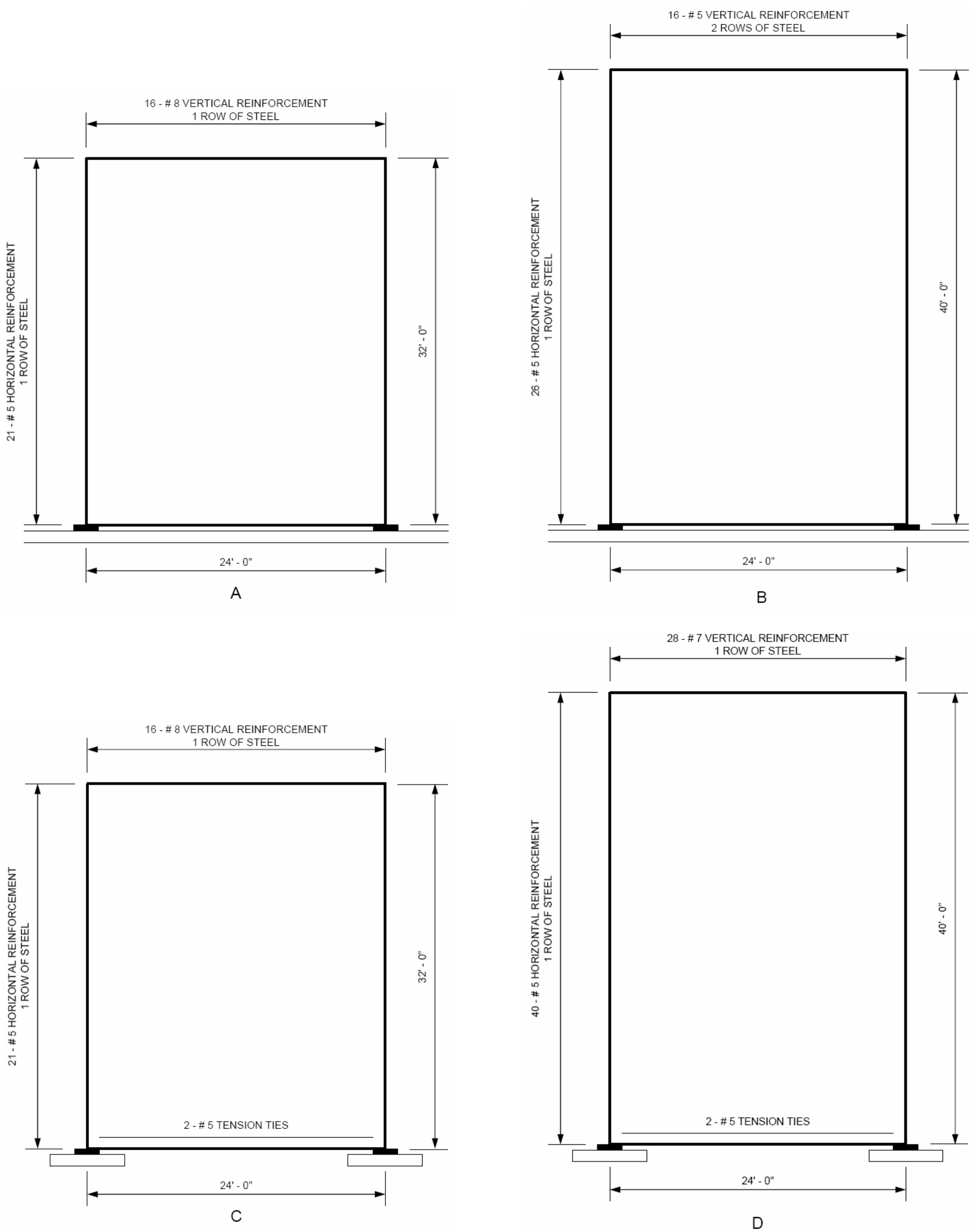


Figure 8-12. Tilt-Up Wall Panel Details

9 Foundation Design for Tilt-Up Panels

The foundation system is designed to resist the governing force resulting from bearing, overturning, or uplift of the panels. Through analysis, the load bearing forces from the panel govern the design of the foundation. For this particular building, overturning and uplift do not govern because of the geometry of the building and the entire length of each wall resists the in-plane shear. The panels are subject to overturning, but the extensive self-weight of the panel resists this action from occurring. The panels are not subject to uplift because the entire length of each wall acts as the shear wall. If only a few panels along each wall are used as shear walls, or if the building has a narrow rectangular shape, each panel would then need to resist a larger unit shear. As the unit shear increases per panel, the possibility of uplift increases. When uplift occurs, the connection to the foundation must resist the tension force resulting from the uplift force. As stated, this action does not occur for the building within this parametric study.

The load bearing forces from the panel are resisted by the foundation through bearing stress. This load takes a different path depending on whether the panel is supported by a continuous footing or an isolated footing. A continuous footing resists the bearing stress from the panel uniformly along the length of the foundation. The one-foot strip of the continuous footing is designed for a one-foot strip of the wall panel. An isolated footing resists the bearing stress resulting from half the panel self-weight and half the axial roof load. The isolated footing supports two panels; therefore, the isolated footing must resist the total bearing stress equivalent to one panel and its entire axial roof load.

9.1 Foundations

The foundations are designed to resist the load bearing stresses resulting from the load combination $1.2D + 1.6L_r + 0.8W$. Each of the three foundation options, continuous footings, spread footings, and drilled piers, is designed to support the panels for the three building locations. Unfactored loads are used to design the bearing area size of the foundations since the soil bearing pressure is an allowable stress. The reinforcement within the foundations is designed for ultimate strength using factored loads.

9.1.1 Continuous Footings

The allowable soil bearing pressure used for the continuous footing design in Dallas, Denver, and Kansas City is 2000 psf, 2500 psf, and 2000 psf, respectively. These allowable soil bearing pressures are determined from geotechnical reports. (Geotechnical Report 1, 2, & 3) The continuous footing size and reinforcement is illustrated in Table 9-1. The larger allowable soil bearing pressure in Denver allows for the footing width to be less than the footing widths in Dallas and Denver. A weaker allowable soil bearing pressure requires a larger area of footing to distribute the load into the earth.

Table 9-1. Continuous Footing Design

Location	Panel Height (ft)	Allowable Soil Bearing Pressure (psf)	Foundation Type	Length (ft)	Width (ft)	Thickness (in)	Reinforcement
Dallas	32	2000	Continuous Footings	-	2.5	12	#5 @ 12" OC
Dallas	40	2000	Continuous Footings	-	3	12	#5 @ 12" OC
Denver	32	2500	Continuous Footings	-	2	12	#5 @ 12" OC
Denver	40	2500	Continuous Footings	-	2.5	12	#5 @ 12" OC
Kansas City	32	2000	Continuous Footings	-	2.5	12	#5 @ 12" OC
Kansas City	40	2000	Continuous Footings	-	3	12	#5 @ 12" OC

A 12 in (30 cm) thickness for the footings is sufficient to resist the one-way shear. A continuous footing supporting a continuous wall will not experience two-way punching shear; therefore, the one-way shear governs. The flexural reinforcement required for the continuous footings is less than the minimum area of steel required for temperature and shrinkage. Reinforcement shall be placed in both directions of the footing using a #5 bar at 12 in (30 cm) on center. The detail in Figure 9-1 illustrates the reinforcement for the continuous footing.

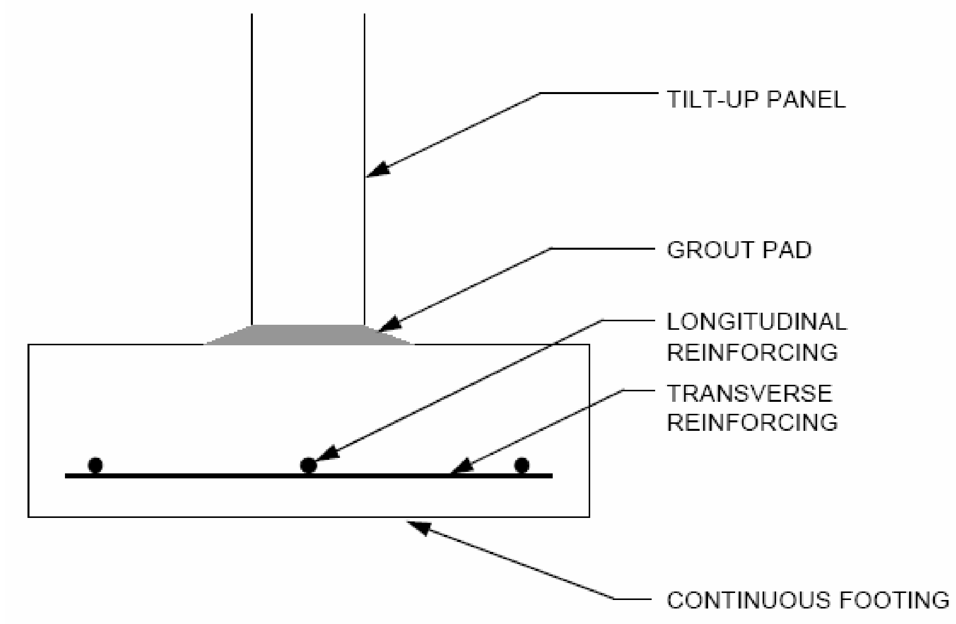


Figure 9-1. Continuous Footing Detail

9.1.2 Spread Footings

The allowable soil bearing pressure used for the spread footing design in Dallas, Denver, and Kansas City is 2000 psf, 2500 psf, and 2000 psf, respectively. These allowable soil bearing pressures are determined from geotechnical reports. (Geotechnical Report 1, 2, & 3) The spread footing size and reinforcement is shown in Table 9-2. When comparing the spread footings determined for the 40 ft (12 m) panel, it can be noted that a weaker allowable soil bearing pressure requires a larger area of footing to distribute the load into the earth. The 32 ft (9.75 m) panel yields that same spread footing design for Dallas and Denver, and a larger spread footing design for Kansas City. The axial roof live loads are larger in Denver and Kansas City than in Dallas. This increase in roof live load is just enough to require an 8'-0" x 8'-0" spread footing in Kansas City with the same allowable soil pressure as Dallas, where a 7'-0" x 7'-0" spread footing is sufficient.

Table 9-2. Spread Footing Design

Location	Panel Height (ft)	Allowable Soil Bearing Pressure (psf)	Foundation Type	Length (ft)	Width (ft)	Thickness (in)	Reinforcement
Dallas	32	2000	Spread Footings	7	7	12	8 - #5 EW
Dallas	40	2000	Spread Footings	9	9	12	9 - #6 EW
Denver	32	2500	Spread Footings	7	7	12	8 - #5 EW
Denver	40	2500	Spread Footings	8	8	12	7 - #6 EW
Kansas City	32	2000	Spread Footings	8	8	12	9 - #5 EW
Kansas City	40	2000	Spread Footings	9	9	12	9 - #6 EW

A 12 in (30 cm) thickness for each spread footing is sufficient to resist both one-way and two-way punching shear. The reinforcement for the spread footings is governed by flexure rather than temperature and shrinkage. The reason the reinforcement is governed by flexure for the spread footings and not the continuous footings is because of the increase in width of the footing which cantilevers out from the base of the wall panel. The continuous footing has a short cantilever distance yielding a smaller moment than the moment resulting from the longer cantilever of the spread footing. This flexural action is illustrated in Figure 5-9. The reinforcement required to resist this bending is dependent on the width of the spread footing and the ultimate bearing capacity. The ultimate bearing capacity, q_u, is the quotient of the ultimate load, P_u, over the area of the footing, A. As illustrated in Table 9-2, the wider footings require more reinforcement to resist the larger moment induced from the longer cantilever. The detail in Figure 9-2 illustrates the reinforcement for the continuous footing.

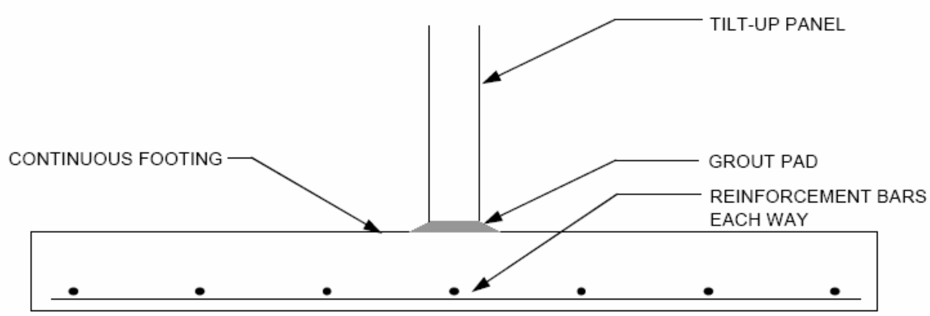


Figure 9-2. Spread Footing Detail

9.1.3 Drilled Piers

Straight-shaft sidewall shear drilled piers are used in this parametric study for deep foundation options. These piers are designed to support the unfactored loads from the superstructure and resist the uplift pressure from the active zone of expansive soils. These two forces are resisted by the end bearing pressure and skin friction of the soil, which are determined from geotechnical reports. (Geotechnical Report 1, 2, & 3) The maximum end bearing pressure used for drilled pier design in Dallas and Kansas City is 30 ksf, and for Denver is 25 ksf. The skin friction for sidewall shear in Dallas and Kansas City is 1 ksf, and for Denver is 2.5 ksf. The uplift pressure at the active zone resulting from the expansive soils in Dallas and Kansas City is 1.5 ksf, and in Denver is 5 ksf. Table 9-3 illustrates the drilled pier design required to support the 32 ft (9.75 m) and 40 ft (12 m) panels in each location.

Table 9-3. Drilled Pier Design

Location	Panel Height (ft)	Maximum End Bearing Pressure (ksf)	Skin Friction (ksf)	Uplift Pressure (ksf)	Diameter (in)	Total Length (ft)	Socket Length (ft)	Reinforcement	Ties
Dallas	32	30	1	1.5	18	54	17	4 - # 8	# 3 @ 12"
Dallas	32	30	1	1.5	24	46	9	6 - # 8	# 3 @ 12"
Dallas	32	30	1	1.5	30	40	3	8 - # 9	# 3 @ 12"
Dallas	40	30	1	1.5	18	63	26	4 - # 8	# 3 @ 12"
Dallas	40	30	1	1.5	24	53	16	6 - # 8	# 3 @ 12"
Dallas	40	30	1	1.5	30	45	8	8 - # 9	# 3 @ 12"
Denver	32	25	2.5	5	18	55	18	4 - # 8	# 3 @ 12"
Denver	32	25	2.5	5	24	52	15	6 - # 8	# 3 @ 12"
Denver	32	25	2.5	5	30	50	13	8 - # 9	# 3 @ 12"
Denver	40	25	2.5	5	18	59	22	4 - # 8	# 3 @ 12"
Denver	40	25	2.5	5	24	55	18	6 - # 8	# 3 @ 12"
Denver	40	25	2.5	5	30	52	15	8 - # 9	# 3 @ 12"
Kansas City	32	30	1	1.5	18	55	18	4 - # 8	# 3 @ 12"
Kansas City	32	30	1	1.5	24	47	10	6 - # 8	# 3 @ 12"
Kansas City	32	30	1	1.5	30	40	3	8 - # 9	# 3 @ 12"
Kansas City	40	30	1	1.5	18	64	27	4 - # 8	# 3 @ 12"
Kansas City	40	30	1	1.5	24	53	16	6 - # 8	# 3 @ 12"
Kansas City	40	30	1	1.5	30	46	9	8 - # 9	# 3 @ 12"

The sizes of the drilled piers are first determined for the unfactored loads of each superstructure and its respective uplift force. Once the size of the drilled pier is set, the reinforcement is then determined from the factored loads of the superstructure. The drilled pier is designed as a compression/tension axial column with full lateral bracing from the surrounding soil. For this parametric study, the drilled piers are designed for compression forces. The uplift force from the expansive soils is resisted by the self-weight of the superstructure. The drilled piers in Table 9-3 are sufficient in size to resist the ultimate loading from the concrete itself. However, minimum reinforcement is required by ACI 318-05 Section 10.9.1. Compression members shall have an area of longitudinal reinforcement, A_{st}, greater than $0.01A_g$ and less than

$\boldsymbol{0.08A_g}$. The design strength, $\boldsymbol{\phi P_n}$, of the drilled pier is determined from ACI 318-05 Equation (10-2) and must be greater than or equal to the ultimate factored load, $\boldsymbol{P_u}$.

$$\phi P_{n,\max} = 0.80\phi\left[0.85 f_c'\left(A_g - A_{st}\right) + f_y A_{st}\right] \tag{9.1}$$

$$\phi P_n \geq P_u \tag{9.2}$$

Where

ϕ = 0.65 for compression controlled sections

A_g = gross area of concrete section (in^2)

A_{st} = total area of longitudinal reinforcement (in^2)

The maximum end bearing pressure for these three locations occurs at 35 ft below the earth's surface. Geotechnical reports advise a minimum length of 40 ft (12 m) for the piers. In theory, the first two feet of the pier socketed into the rock do not provide resistance for sidewall shear. As the pier is socketed further into the earth, more surface area of the pier is available for skin friction. Figure 9-3 illustrates the soil interaction of the drilled pier.

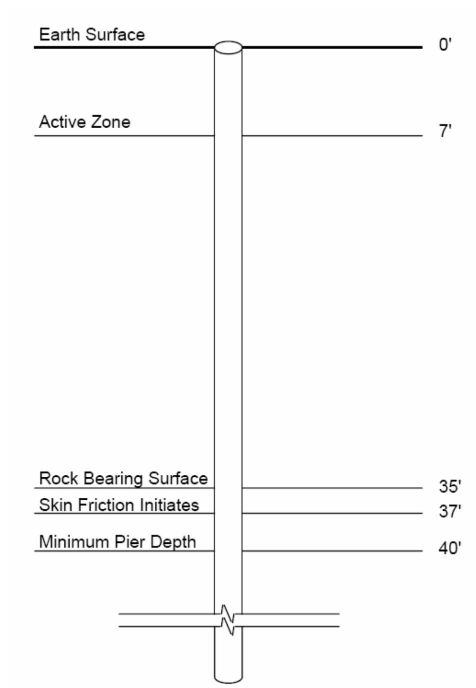


Figure 9-3. Drilled Pier Soil Interaction

9.2 Connections

Tilt-up panels commonly carry vertical and horizontal loadings. These loadings must be adequately transferred to the foundation through means of connection design. In addition to transferring the loads from the roof, floor, and panel into the foundation, the connections must provide a degree of ductility to resist temperature and shrinkage stresses. The primary purpose of the connection is to prevent longitudinal or transverse displacement between the panel and the foundation. The out-of-plane shear forces, illustrated in Table 9-4, must be accounted for either by a positive connection to the slab, matching the spacing of reinforcement in the slab, or by the coefficient of friction of the concrete tilt-up panel bearing on the concrete foundation system.

Table 9-4. Out-of-Plane Shear Force

Panel Height	Foundation Support	Shear Force
32 ft	Isolated	5280 lbs
32 ft	Continuous	440 plf
40 ft	Isolated	6600 lbs
40 ft	Continuous	550 plf

Typical panel-to-foundation connections are discussed in Section 9.2.1. Connections may also be used at the joints between panels. Panel-to-panel connections are not a common practice unless very large shear loads are transferred through the diaphragm and the panels cannot account for this in overturning. Typical panel-to-panel connections are discussed in Section 9.2.2.

9.2.1 Panel-to-Foundation Connections

Connections at foundations are commonly used to provide lateral resistance for lateral loads or prevent the panel from lifting off the foundation. The following situations are typical conditions for panel-to-foundation connections: loading dock walls (where the panel can span at least 4 ft or more below the floor slab), foundations in cold climates (where the panel can span at least 4 ft or more below the floor slab), and panels with excessive overturning moments that can not be resisted by the self weight of the panel and its applied axial loads. (TCA 2006)

Uplift is not a concern for the panels within this parametric study. When a tilt-up panel does need to provide a resistance to excessive lateral forces, tension tie downs may be required at the outside edges of the panel.

Tilt-up wall panels can be attached either at the footing, the floor slab, or both. The detail shown in Figure 9-4 illustrates the common connections used at the panel to foundation juncture. Dowels are cast into the footing on either side of the location where the panel will be set to provide alignment. Once the panels are set, the dowels are covered with grout. These dowels provide no mechanical connection other than means of assisting wall placement. Dowels are commonly cast into the panel and extrude at the foundation elevation. Once the panels have been set, a closure strip is poured between the slab-on-grade and the tilt-up panel encasing the dowels and providing the connection to the panel.

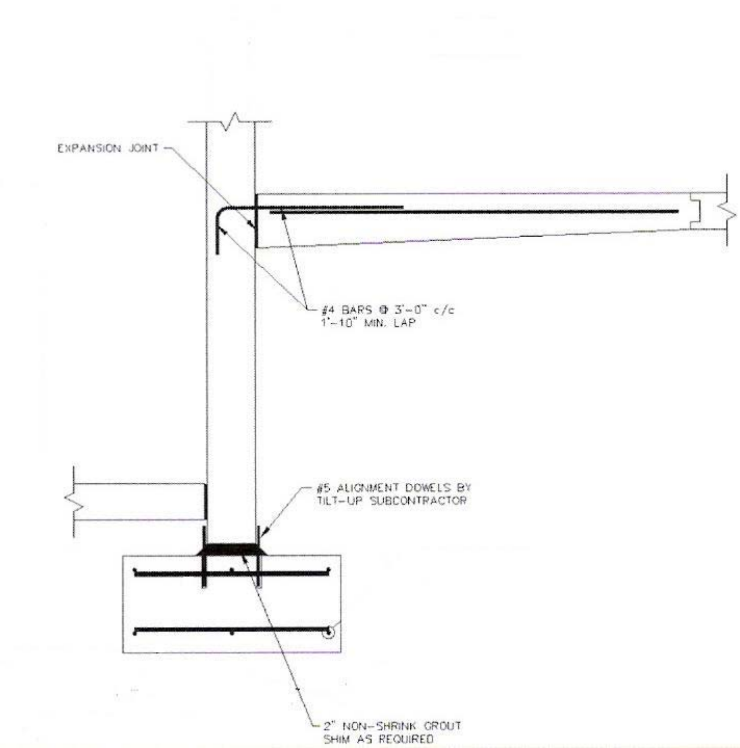
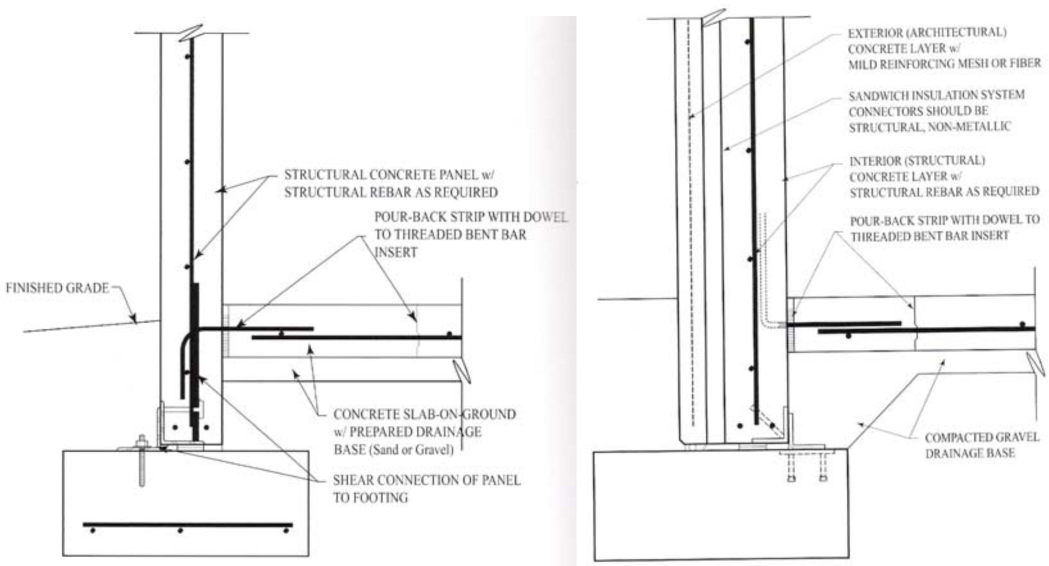


Figure 9-4. Panel-to-Foundation Detail

Courtesy of the Tilt-Up Concrete Association

(TCA 2006)

ACI 318-05 Section 15.8.3 states that anchor bolts or suitable mechanical connectors shall be permitted to satisfy ACI 318-05 Section 15.8.1, which states that forces and moments at the base of a wall shall be transferred to the footing by bearing on concrete or by reinforcement, dowels, and mechanical connectors. The two details in Figure 9-5 illustrate other common panel-to-foundation connections as recommended from The Architecture of Tilt-Up.(TCA 2006) Each of these details provides a connection to the foundation as well as a connection to the slab-on-grade. A combination of inserts, reinforcing bars, and angles are used with welded or bolted final connections. Detail 9-5A provides a bolted angle connection at the foundation. Detail 9-5B provides a welded angle connection at the foundation. Each of these connections transfers the force through shear across the interface. The welded angle provides a stiffer connection which can cause cracking in the concrete during movement, whereas the bolted angle can allow movement resulting from expansion and contraction.



(A) (B)

Figure 9-5. Panel-to-Foundation Mechanical Connections
Courtesy of the Tilt-Up Concrete Association
(TCA 2006)

9.2.2 Panel-to-Panel Connections

One of the major differences between tilt-up panels and precast panels are the panel-to-panel connections, commonly referred to as stitching, or stitchplates. Tilt-up panels are commonly much wider than precast wall panels. This increased width provides more resistance to overturning and uplift resulting from lateral forces. Panel-to-panel stitchplate connections may be required in high seismic regions to resist earthquake loadings. Other panel-to-panel connections include: chord bars, tube sections, and ledger angles.

Figure 9-6 illustrates a stitch plate consisting of steel angles and a steel plate. The steel angles are embedded at the interior face of the wall panels. The steel plate is welded to only one panel embedment to form the connection. This allows for expansion and contraction of the adjacent panel. These stitchplate connections are not used for structural stability, but rather provide a restraint from panels displacing laterally. It is important to detail panel-to-panel connections so not to restrain any more movement with the connection than structurally required. This allows for joint movement between panels to prevent shrinkage and thermal cracking.

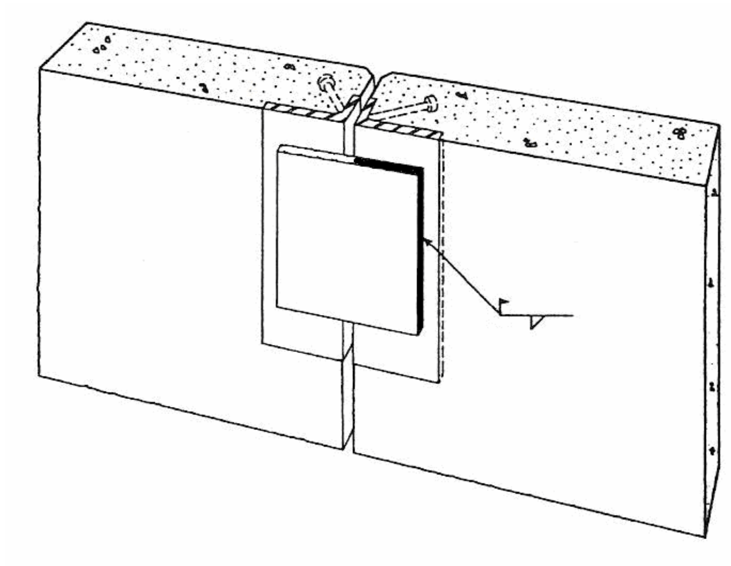


Figure 9-6. Panel-to-Panel Stitchplate Connection

Reprinted with permission of The Portland Cement Association.

(PCA 1987)

10 Recommendations for Tilt-Up Wall Panel Foundation Systems

Table 10-1 illustrates the cost per foundation unit for the three systems designed to support the chosen tilt-up panels in the three locations. From a pure economic standpoint, one may conclude that the least expensive foundation option should be chosen for construction. However, several variables may exist at the construction site which restrict the use of the least expensive foundation option.

Table 10-1. Foundation System Concrete Unit Prices

Location	Concrete Price ($/cy)	Panel Height (ft)	Foundation System							
			Continuous Footing (24' length/panel)	Spread Footing (1/panel)	Drilled Piers					
					18"		24"		30"	
					Length (ft)	Price	Length (ft)	Price	Length (ft)	Price
Dallas	$95.26	32	$211.69	$172.88	54	$336.68	46	$509.86	40	$692.75
Dallas	$95.26	40	$254.03	$285.78	63	$392.79	53	$587.45	45	$779.34
Denver	$90.48	32	$160.85	$164.20	55	$325.70	52	$547.45	50	$822.49
Denver	$90.48	40	$201.07	$214.47	59	$349.39	55	$579.03	52	$855.39
Kansas City	$85.00	32	$188.89	$201.48	55	$305.98	47	$464.84	40	$618.14
Kansas City	$85.00	40	$226.67	$255.00	64	$356.05	53	$524.18	46	$710.86

The prices listed in Table 10-1 are representative of the cost for the foundation support designs illustrated in Table 9-1, Table 9-2, and Table 9-3. The prices listed for the continuous footings represent the cost for the length of the footing underneath one 24 ft (7.25 m) wide panel. The prices listed for the spread footings represent the cost of one spread footing. One spread footing supports two panels, and one panel is supported by two spread footings; therefore, the panel to spread footing ratio for the building structure is 1:1. The prices listed for the drilled piers represent the cost of one drilled pier for the same reason as for the spread footings.

For an analysis based strictly on cost, the continuous footing is the most economical design choice for all situations, except for the 32 ft (9.75 m) panel in Dallas, where the spread footing is the most economical choice. For an analysis based on cost and soil conditions, engineering judgment must be used to determine the most adequate foundation system. In locations where the presence of expansive clays is abundant, such as Dallas and Denver, the 18 in (46 cm) diameter drilled pier foundation system may be the most logical option. Using drilled piers in locations of expansive soils reduces the potential for movement of the superstructure. When drilled piers are used for the foundation system, a void space at least 4 in (10 cm) deep should be provided beneath the panels between piers to allow for the expansive clays to expand and contract without causing movement to the panels. If a greater risk of foundation movement can be tolerated by the superstructure, shallow foundation systems can be considered to support the superstructure. When shallow foundations, spread footings or continuous footings, are used for the foundation system, special provisions are required to ensure that the on-site expansive soils are not allowed to dry out significantly prior to construction. If the foundation supporting expansive soils are allowed to dry out, these soils could exhibit high to very high expansive potential and foundation construction on the soils could experience excessive movement. For an analysis based on material cost, soil conditions, and construction cost, the 18 in (46 cm) drilled piers may not be the most economic choice in locations requiring deep foundation systems. The 24 in (61 cm) or 30 in (76 cm) drilled piers may be a more economical choice due to their decreased depths. The 18 in (46 cm) diameter drilled piers require a deeper socket length into rock, which requires more drilling and excavation, which may require more expensive construction equipment. The 24 in (61 cm) and 30 in (76 cm) diameter drilled piers do not require as much socket length because of their increased circumference surface area.

The panel details in Figure 8-12 illustrate the effect foundation support has on the panel design. The details with continuous support represent panels supported by continuous footings, and the details with isolated support represent panels supported by either spread footings or drilled piers. When the panel is supported by spread footings or drilled piers, two #5 reinforcing bars are placed near the bottom of the panel to simulate this portion of the panel as a grade beam. These reinforcing bars are not required in the panels supported by continuous footings.

The most economical panel designs yield thicknesses of 7 ¼ in (18 cm) for the 32 ft (9.75 m) panels supported by continuous or isolated foundations, 7 ¼ in (18 cm) for the 40 ft (12 m) panels supported by continuous foundations, and 9 ¼ in (23 cm) for 40 ft (12 m) panels supported by isolated foundations. The 9 ¼ in (23 cm) thick panel for the 40 ft (12 m) wall is the most economical choice because the 7 ¼ in (18 cm) panel which correlates to the 40 ft (12 m) panel choice for continuous support cannot be utilized. This 7 ¼ in (18 cm) panel is not tension-controlled due to the required amount of reinforcing needed to provide a design moment strength, ϕM_n, greater than total factored moment, M_u. As illustrated in Table 10-1, the most economical shallow foundation system for the 32 ft (9.75 m) panel in Dallas is the spread footing from Table 9-2, while the continuous footings from Table 9-1 are the most economical shallow foundation system for the remaining situations. In regions of expansive soils, the most economical deep foundation system is the 18 in (46 cm) diameter straight-shaft drilled pier.

11 Conclusions

The foundation system for a building structure has two purposes: to provide a support to the superstructure and to effectively transfer the loads from the superstructure into the earth without overstressing the supporting soil. The soil and rock present at the site for the building structure can control which foundation system must be used for design. Shallow foundation systems, consisting of spread footings and continuous footings, are appropriate systems for site locations where the soil conditions consist of compacted sands or firm silts. Deep foundation systems, such as drilled piers, are appropriate systems for site locations where the soil conditions consist of loose sands and clays. Not all situations allow for these systems to be used for these soil conditions. Deep foundation systems may be required for sites of compacted sands and firms silts where excessively large loads from the superstructure occur or when uplift forces act upon the foundation. Shallow foundation systems may be used at sites of loose sands and clays when the building structure is allowed a higher tolerance of risk for foundation movement and special provisions are used to control the expansive soils.

In one of the fastest growing construction industries in the United States, tilt-up concrete panels are load-bearing, slender, wall elements that are governed by lateral loads. This lateral instability occurs when the lateral deflection produces the secondary moments caused by the $P\text{-}\Delta$ effects. ACI 318-05 Section 14.8, Alternate Design of Slender Walls, provides provisions for designing tilt-up wall panels. The tilt-up panels must be properly designed and detailed in order to resist axial, lateral, and shear loads. The alternate design method may be used when the following design considerations are met:

1. The wall panel is simply supported, axially loaded, and subjected to an out-of-plane uniform lateral load. The maximum moments and deflections occur at the mid-height of the wall
2. The cross-section is constant over the height of the panel
3. The wall cross-section is tension-controlled
4. Reinforcement is provided to ensure a design moment strength ϕM_n greater than or equal to M_{cr}, where M_{cr} is the moment causing flexural cracking due to the applied lateral and vertical loads

5. Concentrated gravity loads applied to the wall above the design flexural section are distributed over a width equal to the lesser of (a) the bearing width plus a width on each side that increases at a slope of 2 vertical to 1 horizontal down to the design flexural section or (b) the spacing of the concentrated loads
6. The vertical stress P_u/A_g at the mid-height section does not exceed $0.06f'_c$

The vertical reinforcement and tilt-up panel thickness required to resist lateral instability resulting from axial and out-of-plane forces are governed by these provisions. After the panel is designed to resist the axial and lateral forces, the panel must be analyzed to check its adequacy of resisting the in-plane shear forces. The foundation support for the panel must also be considered for proper detailing. Isolated foundations require the panel to be designed and reinforced such that the panel acts as a deep beam and spans from footing to footing. The tension ties provided near the bottom of the panel are not required for panels supported on continuous footings.

Tilt-up wall panels can be supported by either spread footings, continuous footings, or drilled piers. The spread footings and drilled piers are placed under the joints between the panels, so that each footing pad supports half of each adjacent panel. The panel spans between footing pads for these two foundation systems. When soil conditions permit shallow foundation systems, continuous footings provide more economy than spread footings. However, if numerous amounts of mechanical, electrical, or plumbing equipment must pass under the tilt-up panels, spread footings may be a more economical choice because of the pipe sleeves and detailing needed for continuous footings.

When the building site consists of expansive soils, the structural engineer should take careful consideration into the decision for the foundation system. If the building can tolerate a higher risk for expansion and contraction at areas such as the panel-to-slab-on-grade connection due to the clayey soils, shallow foundation systems will provide a less expensive option. If the building cannot tolerate risk of expansion and contraction of the clayey soils, drilled piers may be the only option for the foundation system. This deep foundation system may be more expansive up front, but could save the owner money in the future by preventing the risks of settlement and heaving of the structure. The structural engineer needs to use his or her judgment and discuss all options and risks involved with the owner in order to design the most economical structure that the construction site permits.

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Appendix A - Gravity Loads

GRAVITY LOADS			REFERENCE
Roof Gravity Loads			
Roof Dead Load =	14	psf	
Roof Live Load =	20	psf	
Roof Area =	38550	sf	
Roof Dead Load =	539.7	k	
Roof Live Load =	771.0	k	IBC 2006
Reduction in Roof Live Loads			1607.11.2
$L_r = L_o R_1 R_2$			(16-27)
$L_o =$	20.0	psf	
$A_t =$	440.04	sf	
$R_1 =$	0.760		
$R_2 =$	1.000		
F =	0.3	in. rise per foot	
$R_1 = 1$ for $A_t \leq 200$ sf			(16-28)
$R_1 = 1.2 - 0.001A_t$ for 200 sf < A_t < 600 sf			(16-29)
$R_1 = 0.6$ for $A_t > 600$ sf			(16-30)
$R_2 = 1$ for $F \leq 4$			(16-31)
$R_2 = 1.2 - 0.05F$ for $4 < F < 12$			(16-32)
$R_2 = 0.6$ for $F > 12$			(16-33)
$L_r =$	15.20	psf	
...where $12 < L_r < 20$ $L_r =$	$\boxed{15.20}$	psf	
Snow Loads			ASCE 7-05
Ground Snow Load, $p_g =$	5	psf	
Flat Roof Snow Load, $p_f = 0.7 C_e C_t I p_g$ $p_f =$	3.5	psf	(7-1)
Exposure Factor, Ce =	1.0		Table 7-2
Thermal Factor, Ct =	1.0		Table 7-3
Importance Factor, I =	1.0		Table 7-4
Minimum Snow Loads			
$p_{f\,min} = (I)\,p_g$ where $p_g \leq 20$ psf $p_{f\,min} =$	5	psf	
$p_{f\,min} = 20\,(I)$ where $p_g > 20$ psf			
$p_f =$	5	psf	
Rain-on-Snow Surcharge			7.10
For locations where pg is 20 psf or less, but not zero, all roof slopes are 1/2" / 1 ft, and additional 5 psf shall be added for rain-on-snow.			
$p_f =$	10	psf	

GRAVITY LOADS	REFERENCE

Snow Drifts

ASCE 7-05
7.8

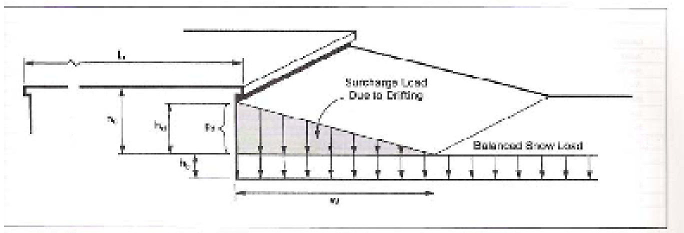


FIGURE 7-8 CONFIGURATION OF SNOW DRIFTS ON LOWER ROOFS

7.7.1

p_g = Ground Snow Load	p_g =	5	psf
$\gamma = 0.13\ p_g + 14 \leq 30$ pcf	γ =	14.65	pcf
l_u = Length of Roof Upwind of the Drift	$l_{u\ LONG}$ =	288	ft
	$l_{u\ TRANS}$ =	140.67	ft
h_c = Height from top of balanced snow load to T.O. Parapet	h_c =	1.32	ft
h_d = Height of Snow Drift = $0.43 * \sqrt[3]{lu} * \sqrt[4]{(Pg+10)} - 1.5$	$h_{d\ LONG}$ =	1.32	ft
Maximum height = Parapet or Wall Height	$h_{d\ TRANS}$ =	1.32	ft
p_d = Maximum Intensity of Drift Surcharge Load	$p_{d\ LONG}$ =	0.0193	ksf
$p_d = h_d\gamma$	$p_{d\ TRANS}$ =	0.0193	ksf
h_b = Height of balanced Snow Load (P_f / γ)	h_b =	0.68	ft
W = Horizontal Distance from	W_{LONG} =	5.27	ft
	W_{TRANS} =	5.27	ft

Snow Drift Applies

Longitudinal Drift Load = 0.0509 klf
Transverse Drift Load = 0.0509 klf

AXIAL PANEL LOADS

North & South Panels			East & West Panels		
DL =	0.26	klf	DL =	0.06	klf
Lr =	0.28	klf	Lr =	0.08	klf
S =	0.23	klf	S =	0.09	klf

Appendix B - Wind Loads

WIND LOADS: BASE SHEAR					ASCE 7-05
Method 2 - Analytical Procedure					6.5
DESIGN PROCEDURE					6.5.3
Step 1	V =	90	mph		Figure 6-1
	k_d =	0.85			Table 6-4
Step 2	I =	1.00			Table 6-1
	Occupancy Category =	II			Table 1-1
Step 3	Exposure Category =	C			6.5.6.2
	Mean Roof Height, h =	32	ft	Low Rise Building Class OK, < 60'	6.5.6.4.2 & 6.2
	Least Horz. Dimension =	140.67	ft	Low Rise Building Class OK	6.5.6.4.2 & 6.2
	GC_{pf}	Roof Angle =	1.70		Figure 6-10

1	2	3	4	5	6	1E	2E	3E	4E
0.40	-0.69	-0.37	-0.29	-0.45	-0.45	0.61	-1.07	-0.53	-0.43

					ASCE 7-05
Step 4	K_{zt} =	1.0			6.5.7.2
Step 5	G =	0.85			
	$T_a = C_t h_n^x$		T_a = 0.36204		(12.8-7)
	C_t =	0.016			Table 12.8-2
	h_n =	32			Table 12.8-2
	x =	0.9			Table 12.8-2
	$f = 1/T_a$	2.76214		Rigid Structure, Frequency > 1 Hz	
Step 6	Building Partially Enclosed				6.5.9.1 & 6.2
Step 7	GC_{pi} =	0.55	&	-0.55	6.5.11.1 & Fig 6-5
Step 8	Refer to Step 3 for GC_{pf} values				6.5.11.2.1 & Fig 6-10
Step 9	$q_h = 0.00256 K_z K_{zt} K_d V^2 I$ psf		q_h = 18.3306 psf		(6-15)
	k_z =	1.04	@	32 ft	Table 6-3
Step 10	$P = q_h\,[(GC_{pf}) - (GC_{pi})]$				(6-18)

	$P_{(+GCpi)}$	$P_{(-GCpi)}$
Zone 1	-2.75 psf	17.41 psf
Zone 2	-22.73 psf	-2.57 psf
Zone 3	-10.08 psf	3.30 psf
Zone 4	-15.40 psf	4.77 psf
Zone 5	-18.33 psf	1.83 psf
Zone 6	-18.33 psf	1.83 psf
Zone 1E	1.10 psf	21.26 psf
Zone 2E	-29.70 psf	-9.53 psf
Zone 3E	-19.80 psf	0.37 psf
Zone 4E	-17.96 psf	2.20 psf

WIND LOADS: BASE SHEAR	ASCE 7-05

BASE SHEAR

Determine "a"	a = 10% least horizontal dimension	a =	14.07	ft	Figure 6-10
	a = 0.4 h	a =	12.80	ft	Footnote #9
	a > 4% least horizontal dimension	a =	5.63	ft	
	a > 3 ft	a =	3.00	ft	
		a =	12.80	ft	

Figure 6-10

Length =	288.00	ft
Width =	140.67	ft
L_1 =	262.40	ft
L_2 =	25.60	ft
L_3 =	115.07	ft
L_4 =	25.60	ft

Transverse

Area 1	8396.8	sf
Area 2	819.2	sf

Longitudinal

Area 3	3682.24	sf
Area 4	819.2	sf

Transverse Base Shear…V = P * Area

V= 121.82 k

V= 205.46 k

Longitudinal Base Shear…V = P * Area

V= 62.19 k

V= 100.89 k

Transverse Wind Base Shear =	**205.46**	**k**
Longitudinal Wind Base Shear =	**100.89**	**k**

COMPONENTS & CLADDING

$p = q_h [(GC_p) - (GC_{pi})]$ psf (6-22)

q_h = 18.3306 psf (6-15)

Area	4	5
GCp^+ =	0.7	0.7
GCp^- =	-0.8	-0.8

Figure 6-11A

GCpi = 0.55 & -0.55 Figure 6-5

Determine "a"	a = 10% least horizontal dimension	a =	14.07	ft
	a = 0.4 h	a =	12.80	ft
	a > 4% least horizontal dimension	a =	5.63	ft
	a > 3 ft	a =	3.00	ft
		a =	12.80	ft

Lateral Wind Load Pressure, p

p = 27.4959 psf

p = -24.746 psf

p = 22.9133 psf

p = -4.5827 psf

w = 27.4959 psf

Appendix C - Seismic Loads

SEISMIC LOADS: BASE SHEAR				REFERENCE
Seismic Ground Motion Values				11.4
Step 1	Maximum Spectral Response Acceleration			11.4.1
	S_s =	0.086		
	S_1 =	0.034		
Step 2	Site Classification			11.4.2
		D		Soils Report
Step 3	Adjusted Maximum Considered Earthquake (MCE)			11.4.3
	$S_{MS} = F_aS_s$			(11.4-1)
	$S_{M1} = F_vS_1$			(11.4-2)
	F_a =	1.6		Table 11.4-1
	F_v =	2.4		Table 11.4-2
	S_{MS} =	0.138		
	S_{M1} =	0.082		
Step 4	Design Spectral Acceleration Paramaters			11.4.4
	$S_{DS} = {}^2/_3\, S_{MS}$			(11.4-3)
	$S_{D1} = {}^2/_3\, S_{M1}$			(11.4-4)
	S_{DS} =	0.092		
	S_{D1} =	0.054		
Step 5	Building Occupancy			11.5.1
	Category	II		Table 1-1
	I =	1.00		Table 11.5-1
Step 6	Seismic Design Category (SDC)			11.6
	Short Period Response	A		Table 11.6-1
	1 Sec Period Response	A		Table 11.6-2
Step 7	Structural Design Basis			12.1
	Basic Requirements: Design seismic forces in accordance to Section 12.6			12.1.1
	Analysis Procedure Selection			12.6
	Equivalent Lateral Force Procedure		**Permitted**	Table 12.6-1
	Modal Response Spectrum Analysis		**Permitted**	Table 12.6-1
	Seismic Response History Procedures		**Permitted**	Table 12.6-1
	SDC			
	$T_a = C_th_n^x$	T_a = 0.362		(12.8-7)
	C_t =	0.016		Table 12.8-2
	h_n =	32		Table 12.8-2
	x =	0.9		Table 12.8-2
	T_s =	1.00	**ELFP Permitted**	
	*This spreadsheet considered null and void if Equivalent Lateral Force Procedure Not Permitted			

SEISMIC LOADS: BASE SHEAR				REFERENCE
Step 8	Equivalent Lateral Force Procedure			12.8
	Seismic Base Shear	V= 56.8912 k		12.8.1
	$V = C_sW$			(12.8-1)
	C_s = 0.031			
	W = k			
Step 9	Seismic Response Coefficient			12.8.1.1
	$C_s = S_{DS} / (^R/_I)$	C_s = 0.031	C_s = 0.031	(12.8-2)
	S_{DS} = 0.092			11.4.4
	R = 3	(Ordinary Precast Shear Walls)		Table 12.2-1
	I = 1			11.5.1
	$C_{s\,MAX} = S_{D1} / [T(^R/_I)]$	for $T \le T_L$	$C_{s\,MAX}$ = 0.05009	(12.8-3)
	$C_{s\,MAX} = S_{D1}T_L / [T^2(^R/_I)]$	for $T > T_L$	$C_{s\,MAX}$ = 1.66016	(12.8-4)
	$C_{s\,MIN} = 0.01$		$C_{s\,MIN}$ = 0.01	(12.8-5)
	$C_{s\,MIN} = 0.5S_1 / (^R/_I)$	for $S_1 \ge 0.6g$	$C_{s\,MIN}$ = 0.00567	(12.8-6)
	T_L = 12		$C_{s\,MAX}$ = 0.05009	
			$C_{s\,MIN}$ = 0.01	
Step 10	Effective Seismic Weight			
	DL_{ROOF} = 539.701 k		W = 1860.54 k	
	DL_{WALLS} = 1320.84 k			
	Roof Area = 38550.1 sf			
	Wall Length = 857.34 ft			
	Wall Height ($^h/_2$) = 17 ft			
	Wall Thickness = 7.25 in			
	γ_{conc} = 150 pcf			
		Transverse Seismic Base Shear =	56.89 k	
		Longitudinal Seismic Base Shear =	56.89 k	

Appendix D - 32' Panel on Continuous Footings in Dallas, TX

NORTH & SOUTH PANEL DESIGN : 1 ROW OF STEEL	REFERENCE
	ASCE 7-05
Load Combinations	2.3.2
1 1.2 D + 1.6 (Lr or S) + 0.8 W	2.3.2. # 3
2 1.2 D + 0.5 (Lr or S) + 1.6 W	2.3.2. # 4
3 0.9 D + 1.6 W	2.3.2. # 6
Loads	
North & South Panel	
Panel Width = 24 ft Panel Weight = 69.6 k	
P_{DL} = 6.2 k	
P_{SW} = 37.0 k	
P_{LL} = 6.7 k	
e_{cc} = 3 in (assumed)	
w = 0.03 ksf	
l_c = 30 ft	
Reinforcing Steel Centered in Panel Thickness	
Assume Steel Reinforcement	
No. = 8	
A_s = 0.79 in^2	
Spacing= 18 in	
$A_{s\,TOT}$ = 12.64 in^2 A_s / ft = 0.526667 in^2	
f_y = 60000 psi	
Load Case 1: 1.2 D + 1.6 (Lr or S) + 0.8 W	
P_{ua} = 18.09 k	
P_{um} = 62.46 k	
w_u = 0.53 klf	
	ACI 318-05
Vertical Stress P_u / A_g at the midheight of section should not exceed 0.06f 'c	14.8.2.6
P_u / A_g = 29.92 psi	
f '$_c$ = 4000 psi	
0.06 f '$_c$ = 240 **Vertical Stress OK**	
Strength Reduction Factor	
The wall shall be Tension-Controlled	14.8.2.3
Φ = 0.9	9.3.2.1

NORTH & SOUTH PANEL DESIGN : 1 ROW OF STEEL						REFERENCE
Design Moment Strength						
Reinforcement Design Strength						14.8.2.4
$\Phi M_n \geq M_{cr}$						(14-2)
$M_{cr} = (f_r I_g) / y_t$						(9-9)
$I_g = {}^1/_{12}\, bh^3$						
$f_r = 7.5 \sqrt{f'_c}$						(9-10)
f'_c =	4000	psi				
f_r =	474.3	psi				
I_g =	9145.9	in^4				
b =	288	in				
h =	7.25	in				
y_t =	3.625	in				
M_{cr} =	1196.76	k-in	=	99.7 k-ft	ΦMn>Mcr - OK	
Check Minimum Reinforcement						
Flexural Members						
$\rho_{min} = 200 / f_y$			ρ_{min} =	0.003333		10.5.1
$\rho_{min} = (3\sqrt{f'_c}) / f_y$			ρ_{min} =	0.000273		(10-3)
Governing ρ_{min} =	0.003333					
$\rho = A_s / (b_w d)$						
ρ =	0.012107				ρ > ρ min - As OK	
Walls						
ρ_{min} =	0.0015					14.3.3
$\rho_l = A_s / (b_w t)$		(gross section)				
ρ =	0.006054				ρ > ρ min - As OK	
Check Tension-Controlled Section						
$\varepsilon_t \geq 0.005$						10.3.4
ε_t =	0.008027				Section Tension Controlled - OK	

NORTH & SOUTH PANEL DESIGN : 1 ROW OF STEEL	REFERENCE
Design Moment Strength for combined flexure and axial loads at midheight cross-section	14.8.3
$\Phi M_n \geq M_u$	(14-3)

According to 14.8.3, the design moment strength ϕM_n for combined flexure and axial loads at the mid-height cross-section must be greater than or equal to the total factored moment M_u at this section. The factored moment M_u includes P-Δ effects and is defined as follows:

$$M_u = M_{ua} + P_u \Delta_u \qquad \text{(14-4)}$$

where M_{ua} = factored moment at the mid-height section of the wall due to factored lateral and eccentric vertical loads

P_u = factored axial load

Δ_u = deflection at the mid-height of the wall due to the factored loads

$= 5M_u \ell_c^2/(0.75)48E_cI_{cr}$ (14-5)

ℓ_c = vertical distance between supports

E_c = modulus of elasticity of concrete (8.5)

I_{cr} = moment of inertia of cracked section transformed to concrete

$= nA_{se}(d-c)^2 + (\ell_w c^3/3)$ (14-7)

n = modular ratio of elasticity = $E_s/E_c \geq 6$

E_s = modulus of elasticity of nonprestressed reinforcement

A_{se} = area of effective longitudinal tension reinforcement in the wall segment

$= (P_u + A_s f_y)/f_y$ (14-8)

A_s = area of longitudinal tension reinforcement in the wall segment

f_y = specified yield stress of nonprestressed reinforcement

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

c = distance from extreme compression fiber to neutral axis

ℓ_w = horizontal length of the wall

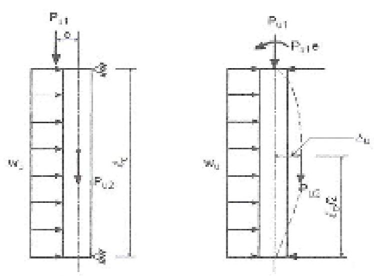


Figure 21-5
PCA 318-05

P_{u1} = factored applied gravity load
P_{u2} = factored self-weight of the wall (total)
e = eccentricity of applied gravity load
w_u = factored uniform lateral load

$$P_u = P_{u1} + \frac{P_{u2}}{2}$$

$$M_u = M_{ua} + P_u \Delta_u = \frac{M_{ua}}{1 - \dfrac{5P_u \ell_c^2}{(0.75)48E_cI_{cr}}}$$

$$M_{ua} = \frac{w_u \ell_c^2}{8} + \frac{P_{u1}e}{2}$$

$$M_u = \frac{w_u \ell_c^2}{8} + \frac{P_{u1}e}{2} + \left(P_{u1} + \frac{P_{u2}}{2}\right)\Delta_u$$

$$\Delta_u = \frac{5M_u \ell_c^2}{(0.75)48E_cI_{cr}}$$

$$a = \frac{A_{se} f_y}{0.85 f'_c \ell_w}$$

NORTH & SOUTH PANEL DESIGN : 1 ROW OF STEEL						REFERENCE
Check Design Moment						
$\Phi M_n = \Phi A_{se} f_y (d - (^a/_2))$						
A_{se} =	13.681	in^2				
f_y =	60	ksi				
a =	0.838	in				
c =	0.986	in				
d =	3.625	in				
ΦM_n =	2368.4	k-in	=	197.4	k-ft	
Check Total Factored Applied Moment						
E_c =	3604.997	ksi				
E_s =	29000	ksi				
n =	8.04					
I_{cr} =	858.4173	in^4				
l_c =	30	ft				
M_{ua} =	61.7	k-ft	=	739.8	k-in	
P_u =	62.5	k				
Δ_u =	4.30	in				
Process Iterations to Compute Mu due to PΔ effects.						
M_u =	1008.64	k-in				
Δ_u =	5.87	in				
M_u =	1106.30	k-in				
Δ_u =	6.43	in				
M_u =	1141.79	k-in				
Δ_u =	6.64	in				
M_u =	1154.68	k-in				
Δ_u =	6.72	in				
M_u =	1159.36	k-in				
Δ_u =	6.74	in				
M_u =	1161.06	k-in				
Δ_u =	6.75	in				
M_u =	1161.68	k-in				
Δ_u =	6.76	in				
M_u =	1161.91	k-in			ΦMn>Mu - OK	
Δ_u =	6.76	in				
Compute Mu with Direct Method Equation						
$M_u = \dfrac{M_{ua}}{1 - \dfrac{5 P_u l_c^2}{(0.75) 48 E_c I_{cr}}}$						(14-6)
M_u =	1162.034	k-in			ΦMn>Mu - OK	

NORTH & SOUTH PANEL DESIGN : 1 ROW OF STEEL	REFERENCE
Check Service Deflection	
$\Delta_s < l_c / 150$	14.8.4
$$\Delta_s = \frac{5M\ell_c^2}{48E_cI_e}$$	(14-8)
$$M = \frac{M_{sa}}{1 - \frac{5P_s\ell_c^2}{48E_cI_e}}$$	(14-9)
$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$	(9-8)

$l_c / 150 =$	2.4	in				
$M_{cr} =$	99.7	k-ft				
$M_a =$	75.84513	k-ft	=	910.1416	k-in	Wall hasn't cracked - Use Ig
$I_e =$	9145.875	in^4				
M =	929.0956	k-in				
$\Delta_s =$	0.3804	in				
M =	948.4443	k-in				
$\Delta_s =$	0.3883	in				
M =	968.196	k-in				
$\Delta_s =$	0.3964	in				
M =	988.359	k-in				
$\Delta_s =$	0.4047	in				
M =	1008.942	k-in				
$\Delta_s =$	0.4131	in				
M =	1029.953	k-in				
$\Delta_s =$	0.4217	in				
M =	1051.403	k-in				
$\Delta_s =$	0.4305	in				
M =	1073.298	k-in				
$\Delta_s =$	0.4395	in				
M =	1095.65	k-in				
$\Delta_s =$	0.4486	in				
M =	1118.467	k-in				
$\Delta_s =$	0.4580	in				Δs < lc/150 - OK

NORTH & SOUTH PANEL DESIGN : 1 ROW OF STEEL	REFERENCE
Load Case 2: 1.2 D + 0.5 (Lr or S) + 1.6 W	
P_{ua} = 10.74 k	
P_{um} = 55.11 k	
w_u = 1.06 klf	
	ACI 318-05
Vertical Stress P_u / A_g at the midheight of section should not exceed **0.06f 'c**	14.8.2.6
P_u / A_g = 26.39 psi	
f '$_c$ = 4000 psi	
0.06 f '$_c$ = 240 — **Vertical Stress OK**	
Strength Reduction Factor	
The wall shall be Tension-Controlled	14.8.2.3
Φ = 0.9	9.3.2.1
Design Moment Strength	
Reinforcement Design Strength	14.8.2.4
$\Phi M_n \geq M_{cr}$	(14-2)
$M_{cr} = (f_r\, I_g) / y_t$	(9-9)
$I_g = {}^1/_{12}\, bh^3$	
$f_r = 7.5 \sqrt{f'_c}$	(9-10)
f '$_c$ = 4000 psi	
f_r = 474.3 psi	
I_g = 9145.9 in^4	
b = 288 in	
h = 7.25 in	
y_t = 3.625 in	
M_{cr} = 1196.76 k-in = 99.7 k-ft — **ΦMn>Mcr - OK**	
Check Minimum Reinforcement	
Flexural Members	
$\rho_{min} = 200 / f_y$ — ρ_{min} = 0.003333	10.5.1
$\rho_{min} = (3\sqrt{f'_c}) / f_y$ — ρ_{min} = 0.003162	(10-3)
Governing ρ_{min} = 0.003333	
$\rho = A_s / (b_w d)$	
ρ = 0.012107 — **ρ > ρ min - As OK**	
Walls	
ρ_{min} = 0.0015	14.3.3
$\rho_l = A_s / (b_w t)$ (gross section)	
ρ = 0.006054 — **ρ > ρ min - As OK**	
Check Tension-Controlled Section	
$\varepsilon_t \geq 0.005$	10.3.4
ε_t = 0.008126 — **Section Tension Controlled - OK**	

NORTH & SOUTH PANEL DESIGN : 1 ROW OF STEEL	REFERENCE
Design Moment Strength for combined flexure and axial loads at midheight cross-section	14.8.3
$\Phi M_n \geq M_u$	(14-3)

According to 14.8.3, the design moment strength ϕM_n for combined flexure and axial loads at the mid-height cross-section must be greater than or equal to the total factored moment M_u at this section. The factored moment M_u includes P-Δ effects and is defined as follows:

$$M_u = M_{ua} + P_u \Delta_u \quad \text{(14-4)}$$

where M_{ua} = factored moment at the mid-height section of the wall due to factored lateral and eccentric vertical loads

P_u = factored axial load

Δ_u = deflection at the mid-height of the wall due to the factored loads

$= 5M_u \ell_c^2/(0.75)48E_cI_{cr}$ (14-5)

ℓ_c = vertical distance between supports

E_c = modulus of elasticity of concrete (8.5)

I_{cr} = moment of inertia of cracked section transformed to concrete

$= nA_{se}(d - c)^2 + (\ell_w c^3/3)$ (14-7)

n = modular ratio of elasticity = $E_s/E_c \geq 6$

E_s = modulus of elasticity of nonprestressed reinforcement

A_{se} = area of effective longitudinal tension reinforcement in the wall segment

$= (P_u + A_s f_y)/f_y$ (14-8)

A_s = area of longitudinal tension reinforcement in the wall segment

f_y = specified yield stress of nonprestressed reinforcement

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

c = distance from extreme compression fiber to neutral axis

ℓ_w = horizontal length of the wall

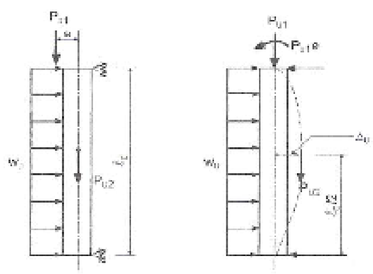


Figure 21-5
PCA 318-05

P_{u1} = factored applied gravity load
P_{u2} = factored self-weight of the wall (total)
e = eccentricity of applied gravity load
w_u = factored uniform lateral load

$$P_u = P_{u1} - \frac{P_{u2}}{2}$$

$$M_u = M_{ua} + P_u \Delta_u = \frac{M_{ua}}{1 - \frac{5P_u \ell_c^2}{(0.75)48E_cI_{cr}}}$$

$$M_{ua} = \frac{w_u \ell_c^2}{8} + \frac{P_{u1}e}{2}$$

$$M_u = \frac{w_u \ell_c^2}{8} + \frac{P_{u1}e}{2} + \left(P_{u1} + \frac{P_{u2}}{2}\right)\Delta_u$$

$$\Delta_u = \frac{5M_u \ell_c^2}{(0.75)48E_cI_{cr}}$$

$$a = \frac{A_{se} f_y}{0.85 f'_c \ell_w}$$

NORTH & SOUTH PANEL DESIGN : 1 ROW OF STEEL	REFERENCE

Check Design Moment

$\Phi M_n = \Phi A_{se} f_y \left(d - \left({}^{a}/_{2} \right) \right)$

A_{se} = 13.558 in^2

f_y = 60 ksi

a = 0.831 in

c = 0.977 in

d = 3.625 in

ΦM_n = 2349.9 k-in = 195.8 k-ft

Check Total Factored Applied Moment

E_c = 3604.997 ksi

E_s = 29000 ksi

n = 8.04

I_{cr} = 854.1916 in^4

l_c = 30 ft

M_{ua} = 120.1 k-ft = 1441.5 k-in

P_u = 55.1 k

Δ_u = 8.43 in

Process Iterations to Compute Mu due to PΔ effects.

M_u = 1905.83 k-in

Δ_u = 11.14 in

M_u = 2055.40 k-in

Δ_u = 12.01 in

M_u = 2103.58 k-in

Δ_u = 12.30 in

M_u = 2119.10 k-in

Δ_u = 12.39 in

M_u = 2124.10 k-in

Δ_u = 12.42 in

M_u = 2125.71 k-in

Δ_u = 12.43 in

M_u = 2126.23 k-in

Δ_u = 12.43 in

M_u = 2126.39 k-in **ΦMn>Mu - OK**

Δ_u = 12.43 in

Compute Mu with Direct Method Equation

$$M_u = \frac{M_{ua}}{1 - \frac{5 P_u l_c^2}{(0.75) 48 E_c I_{cr}}} \qquad (14\text{-}6)$$

M_u = 2126.474 k-in **ΦMn>Mu - OK**

NORTH & SOUTH PANEL DESIGN : 1 ROW OF STEEL	REFERENCE
Check Service Deflection	
$\Delta_s < l_c / 150$	14.8.4
$$\Delta_s = \frac{5 M l_c^2}{48 E_c I_e}$$	(14-8)
$$M = \frac{M_{sa}}{1 - \frac{5 P_s l_c^2}{48 E_c I_e}}$$	(14-9)
$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$	(9-8)

$l_c / 150 =$ 2.4 in

$M_{cr} =$ 99.7 k-ft

$M_a =$ 75.84513 k-ft = 910.1416 k-in **Wall hasn't cracked - Use Ig**

$I_e =$ 9145.875 in^4

M = 929.0956 k-in
$\Delta_s =$ 0.3804 in

M = 948.4443 k-in
$\Delta_s =$ 0.3883 in

M = 968.196 k-in
$\Delta_s =$ 0.3964 in

M = 988.359 k-in
$\Delta_s =$ 0.4047 in

M = 1008.942 k-in
$\Delta_s =$ 0.4131 in

M = 1029.953 k-in
$\Delta_s =$ 0.4217 in

M = 1051.403 k-in
$\Delta_s =$ 0.4305 in

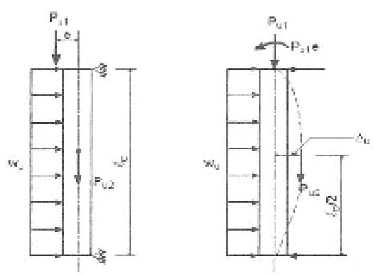
M = 1073.298 k-in
$\Delta_s =$ 0.4395 in

M = 1095.65 k-in
$\Delta_s =$ 0.4486 in

M = 1118.467 k-in
$\Delta_s =$ 0.4580 in

Δs < lc/150 - OK

NORTH & SOUTH PANEL DESIGN : 1 ROW OF STEEL					REFERENCE
Load Case 3: 0.9 D + 1.6 W					
P_{ua} =	5.54	k			
P_{um} =	38.82	k			
w_u =	1.06	klf			
					ACI 318-05
Vertical Stress P_u / A_g at the midheight of section should not exceed 0.06f 'c					14.8.2.6
P_u / A_g =	18.59	psi			
f'_c =	4000	psi			
$0.06 f'_c$ =	240			Vertical Stress OK	
Strength Reduction Factor					
The wall shall be Tension-Controlled					14.8.2.3
Φ =	0.9				9.3.2.1
Design Moment Strength					
Reinforcement Design Strength					14.8.2.4
$\Phi M_n \geq M_{cr}$					(14-2)
$M_{cr} = (f_r I_g) / y_t$					(9-9)
$I_g = {}^1/_{12}\, bh^3$					
$f_r = 7.5 \sqrt{f'_c}$					(9-10)
f'_c =	4000	psi			
f_r =	474.3	psi			
I_g =	9145.9	in^4			
b =	288	in			
h =	7.25	in			
y_t =	3.625	in			
M_{cr} =	1196.76	k-in	= 99.7 k-ft	ΦMn>Mcr - OK	
Check Minimum Reinforcement					
Flexural Members					
$\rho_{min} = 200 / f_y$			ρ_{min} = 0.003333		10.5.1
$\rho_{min} = (3\sqrt{f'_c}) / f_y$			ρ_{min} = 0.003162		(10-3)
Governing ρ_{min} =	0.003333				
$\rho = A_s / (b_w d)$					
ρ =	0.012107			ρ > ρ min - As OK	
Walls					
ρ_{min} =	0.0015				14.3.3
$\rho_l = A_s / (b_w t)$		(gross section)			
ρ =	0.006054			ρ > ρ min - As OK	
Check Tension-Controlled Section					
$\varepsilon_t \geq 0.005$					10.3.4
ε_t =	0.008354			Section Tension Controlled - OK	

NORTH & SOUTH PANEL DESIGN : 1 ROW OF STEEL	REFERENCE
Design Moment Strength for combined flexure and axial loads at midheight cross-section	14.8.3
$\Phi M_n \geq M_u$	(14-3)
According to 14.8.3, the design moment strength ϕM_n for combined flexure and axial loads at the mid-height cross-section must be greater than or equal to the total factored moment M_u at this section. The factored moment M_u includes P-Δ effects and is defined as follows:	
$M_u = M_{ua} + P_u \Delta_u$	(14-4)
where M_{ua} = factored moment at the mid-height section of the wall due to factored lateral and eccentric vertical loads	
P_u = factored axial load	
Δ_u = deflection at the mid-height of the wall due to the factored loads	
$= 5M_u \ell_c^2/(0.75)48E_c I_{cr}$	(14-5)
ℓ_c = vertical distance between supports	
E_c = modulus of elasticity of concrete (8.5)	
I_{cr} = moment of inertia of cracked section transformed to concrete	
$= nA_{se}(d - c)^2 + (\ell_w c^3/3)$	(14-7)
n = modular ratio of elasticity = $E_s/E_c \geq 6$	
E_s = modulus of elasticity of nonprestressed reinforcement	
A_{se} = area of effective longitudinal tension reinforcement in the wall segment	
$= (P_u + A_s f_y)/f_y$	(14-8)
A_s = area of longitudinal tension reinforcement in the wall segment	
f_y = specified yield stress of nonprestressed reinforcement	
d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement	
c = distance from extreme compression fiber to neutral axis	
ℓ_w = horizontal length of the wall	
	Figure 21-5 PCA 318-05
P_{u1} = factored applied gravity load P_{u2} = factored self-weight of the wall (total) e = eccentricity of applied gravity load w_u = factored uniform lateral load	
$P_u = P_{u1} + \frac{P_{u2}}{2}$	
$M_u = M_{ua} + P_u \Delta_u = \frac{M_{ua}}{1 - \frac{5P_u \ell_c^2}{(0.75)48E_c I_{cr}}}$	
$M_{ua} = \frac{w_u \ell_c^2}{8} + \frac{P_{u1} e}{2}$	
$M_u = \frac{w_u \ell_c^2}{8} + \frac{P_{u1} e}{2} + \left(P_{u1} + \frac{P_{u2}}{2}\right)\Delta_u$	
$\Delta_u = \frac{5M_u \ell_c^2}{(0.75)48E_c I_{cr}}$	
$a = \frac{A_{se} f_y}{0.85 f'_c \ell_w}$	

NORTH & SOUTH PANEL DESIGN : 1 ROW OF STEEL							REFERENCE
Check Design Moment							
$\Phi M_n = \Phi A_{se} f_y (d - (^a/_2))$							
A_{se} =	13.287	in^2					
f_y =	60	ksi					
a =	0.814	in					
c =	0.958	in					
d =	3.625	in					
ΦM_n =	2308.9	k-in	=	192.4	k-ft		
Check Total Factored Applied Moment							
E_c =	3604.997	ksi					
E_s =	29000	ksi					
n =	8.04						
I_{cr} =	844.7254	in^4					
l_c =	30	ft					
M_{ua} =	119.5	k-ft	=	1433.7	k-in		
P_u =	38.8	k					
Δ_u =	8.47	in					
Process Iterations to Compute Mu due to PΔ effects.							
M_u =	1762.70	k-in					
Δ_u =	10.42	in					
M_u =	1838.20	k-in					
Δ_u =	10.87	in					
M_u =	1855.52	k-in					
Δ_u =	10.97	in					
M_u =	1859.50	k-in					
Δ_u =	10.99	in					
M_u =	1860.41	k-in					
Δ_u =	11.00	in					
M_u =	1860.62	k-in					
Δ_u =	11.00	in					
M_u =	1860.67	k-in					
Δ_u =	11.00	in					
M_u =	1860.68	k-in				ΦMn>Mu - OK	
Δ_u =	11.00	in					
Compute Mu with Direct Method Equation							
$M_u = \dfrac{M_{ua}}{1 - \dfrac{5 P_u l_c^2}{(0.75) 48 E_c I_{cr}}}$							(14-6)
M_u =	1860.68	k-in				ΦMn>Mu - OK	

NORTH & SOUTH PANEL DESIGN : 1 ROW OF STEEL	REFERENCE

Check Service Deflection

$\Delta_s < l_c / 150$ — 14.8.4

$$\Delta_s = \frac{5M\ell_c^2}{48E_cI_e} \qquad (14\text{-}8)$$

$$M = \frac{M_{sa}}{1 - \frac{5P_s\ell_c^2}{48E_cI_e}} \qquad (14\text{-}9)$$

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad (9\text{-}8)$$

$l_c / 150$ =	2.4	in			
M_{cr} =	99.7	k-ft			
M_a =	75.84513	k-ft	=	910.1416 k-in	Wall hasn't cracked - Use Ig
I_e =	9145.875	in^4			
M =	929.0956	k-in			
Δ_s =	0.3804	in			
M =	948.4443	k-in			
Δ_s =	0.3883	in			
M =	968.196	k-in			
Δ_s =	0.3964	in			
M =	988.359	k-in			
Δ_s =	0.4047	in			
M =	1008.942	k-in			
Δ_s =	0.4131	in			
M =	1029.953	k-in			
Δ_s =	0.4217	in			
M =	1051.403	k-in			
Δ_s =	0.4305	in			
M =	1073.298	k-in			
Δ_s =	0.4395	in			
M =	1095.65	k-in			
Δ_s =	0.4486	in			
M =	1118.467	k-in			
Δ_s =	0.4580	in			Δs < lc/150 - OK

Appendix E - Limiting Factors for Tilt-Up Panels

Table 11-1. Limiting Design Factors for Tilt-Up Panels in Each Location

Panel Height (ft)	Panel Thickness (in)	Foundation Support	Rows of Steel	Vertical Reinf. Bar Size	Vertical Spacing (in)	Limiting Design Factor	ACI 318-05 Section
32	7.25	Continuous	1	6	10	$\phi Mn < Mu$	14.8.3
32	7.25	Continuous	1	7	12	$\phi Mn < Mu$	14.8.3
32	7.25	Continuous	1	8	18	Maximum Spacing	14.3.5
32	9.25	Continuous	1	5	10	$\phi Mn < Mu$	14.8.3
32	9.25	Continuous	1	6	12	$\phi Mn < Mu$	14.8.3
32	9.25	Continuous	1	7	18	Maximum Spacing	14.3.5
32	7.25	Continuous	2	5	12	$\rho < \rho_{min}$	14.3.2
32	7.25	Continuous	2	6	18	Maximum Spacing	14.3.5
32	9.25	Continuous	2	5	10	$\rho < \rho_{min}$	14.3.2
32	9.25	Continuous	2	6	12	$\rho < \rho_{min}$	14.3.2
32	9.25	Continuous	2	7	18	Maximum Spacing	14.3.5
32	11.25	Continuous	2	6	12	$\rho < \rho_{min}$	14.3.2
32	11.25	Continuous	2	7	18	Maximum Spacing	14.3.5
32	7.25	Isolated	1	6	10	$\phi Mn < Mu$	14.8.3
32	7.25	Isolated	1	7	12	$\phi Mn < Mu$	14.8.3
32	7.25	Isolated	1	8	18	Maximum Spacing	14.3.5
32	9.25	Isolated	1	5	10	$\phi Mn < Mu$	14.8.3
32	9.25	Isolated	1	6	12	$\phi Mn < Mu$	14.8.3
32	9.25	Isolated	1	7	18	Maximum Spacing	14.3.5
32	7.25	Isolated	2	5	12	$\rho < \rho_{min}$	14.3.2
32	7.25	Isolated	2	6	18	Maximum Spacing	14.3.5
32	9.25	Isolated	2	5	10	$\rho < \rho_{min}$	14.3.2
32	9.25	Isolated	2	6	12	$\rho < \rho_{min}$	14.3.2
32	9.25	Isolated	2	7	18	Maximum Spacing	14.3.5
32	11.25	Isolated	2	6	12	$\rho < \rho_{min}$	14.3.2
32	11.25	Isolated	2	7	18	Maximum Spacing	14.3.5
40	7.25	Continuous	1	NG	NG	Section Not Tension Controlled	14.8.2.3
40	9.25	Continuous	1	7	10	$\phi Mn < Mu$	14.8.3
40	9.25	Continuous	1	8	12	$\phi Mn < Mu$	14.8.3
40	7.25	Continuous	2	5	10	$\phi Mn < Mu$	14.8.3
40	7.25	Continuous	2	6	12	$\phi Mn < Mu$	14.8.3
40	7.25	Continuous	2	7	18	Maximum Spacing	14.3.5
40	9.25	Continuous	2	5	10	$\rho < \rho_{min}$	14.3.2
40	9.25	Continuous	2	6	12	$\rho < \rho_{min}$	14.3.2
40	9.25	Continuous	2	7	18	Maximum Spacing	14.3.5
40	11.25	Continuous	2	6	12	$\rho < \rho_{min}$	14.3.2
40	11.25	Continuous	2	7	18	Maximum Spacing	14.3.5
40	7.25	Isolated	1	NG	NG	Section Not Tension Controlled	14.8.2.3
40	9.25	Isolated	1	7	10	$\phi Mn < Mu$	14.8.3
40	9.25	Isolated	1	8	12	$\phi Mn < Mu$	14.8.3
40	7.25	Isolated	2	NG	NG	$\Delta_s > l_c / 150$	14.8.4
40	9.25	Isolated	2	5	10	$\rho < \rho_{min}$	14.3.2
40	9.25	Isolated	2	6	12	$\rho < \rho_{min}$	14.3.2
40	9.25	Isolated	2	7	18	Maximum Spacing	14.3.5
40	11.25	Isolated	2	6	12	$\rho < \rho_{min}$	14.3.2
40	11.25	Isolated	2	7	18	Maximum Spacing	14.3.5

Appendix F - Tilt-Up Panel Cost per Lineal Foot

Table 11-2. Dallas Panels

Panel Height (ft)	Panel Thickness (in)	Foundation Support	Rows of Steel	Vertical Reinf. Bar Size	Vertical Spacing (in)	Area of Steel (in^2 / lf)	Area of Concrete (in^2 / lf)	Average Total Price (last 12 months)
32	7.25	Continuous	1	6	10	0.53	87	$89.72
32	7.25	Continuous	1	7	12	0.60	87	$92.60
32	7.25	Continuous	1	8	18	0.53	87	$89.45
32	9.25	Continuous	1	5	10	0.37	111	$101.96
32	9.25	Continuous	1	6	12	0.44	111	$104.95
32	9.25	Continuous	1	7	18	0.40	111	$103.29
32	7.25	Continuous	2	5	12	0.62	87	$93.11
32	7.25	Continuous	2	6	18	0.59	87	$92.11
32	9.25	Continuous	2	5	10	0.74	111	$116.90
32	9.25	Continuous	2	6	12	0.88	111	$122.88
32	9.25	Continuous	2	7	18	0.80	111	$119.55
32	11.25	Continuous	2	6	12	0.88	135	$141.69
32	11.25	Continuous	2	7	18	0.80	135	$138.37
32	7.25	Isolated	1	6	10	0.53	87	$89.72
32	7.25	Isolated	1	7	12	0.60	87	$92.60
32	7.25	Isolated	1	8	18	0.53	87	$89.45
32	9.25	Isolated	1	5	10	0.37	111	$101.96
32	9.25	Isolated	1	6	12	0.44	111	$104.95
32	9.25	Isolated	1	7	18	0.40	111	$103.29
32	7.25	Isolated	2	5	12	0.62	87	$93.11
32	7.25	Isolated	2	6	18	0.59	87	$92.11
32	9.25	Isolated	2	5	10	0.74	111	$116.90
32	9.25	Isolated	2	6	12	0.88	111	$122.88
32	9.25	Isolated	2	7	18	0.80	111	$119.55
32	11.25	Isolated	2	6	12	0.88	135	$141.69
32	11.25	Isolated	2	7	18	0.80	135	$138.37
40	7.25	Continuous	1	NG	NG	NG	NG	NG
40	9.25	Continuous	1	7	10	0.72	111	$145.37
40	9.25	Continuous	1	8	12	0.79	111	$148.61
40	7.25	Continuous	2	5	10	0.74	87	$122.61
40	7.25	Continuous	2	6	12	0.88	87	$130.08
40	7.25	Continuous	2	7	18	0.80	87	$125.92
40	9.25	Continuous	2	5	10	0.74	111	$146.13
40	9.25	Continuous	2	6	12	0.88	111	$153.60
40	9.25	Continuous	2	7	18	0.80	111	$149.44
40	11.25	Continuous	2	6	12	0.88	135	$177.12
40	11.25	Continuous	2	7	18	0.80	135	$172.96
40	7.25	Isolated	1	NG	NG	NG	NG	NG
40	9.25	Isolated	1	7	10	0.72	111	$145.37
40	9.25	Isolated	1	8	12	0.79	111	$148.61
40	7.25	Isolated	2	NG	NG	NG	NG	NG
40	9.25	Isolated	2	5	10	0.74	111	$146.13
40	9.25	Isolated	2	6	12	0.88	111	$153.60
40	9.25	Isolated	2	7	18	0.80	111	$149.44
40	11.25	Isolated	2	6	12	0.88	135	$177.12
40	11.25	Isolated	2	7	18	0.80	135	$172.96

Table 11-3. Denver Panels

Panel Height (ft)	Panel Thickness (in)	Foundation Support	Rows of Steel	Vertical Reinf. Bar Size	Vertical Spacing (in)	Area of Steel (in^2 / lf)	Area of Concrete (in^2 / lf)	Average Total Price (last 12 months)
32	7.25	Continuous	1	6	10	0.53	87	$88.05
32	7.25	Continuous	1	7	12	0.60	87	$91.16
32	7.25	Continuous	1	8	18	0.53	87	$87.76
32	9.25	Continuous	1	5	10	0.37	111	$98.81
32	9.25	Continuous	1	6	12	0.44	111	$102.04
32	9.25	Continuous	1	7	18	0.40	111	$100.24
32	7.25	Continuous	2	5	12	0.62	87	$91.71
32	7.25	Continuous	2	6	18	0.59	87	$90.63
32	9.25	Continuous	2	5	10	0.74	111	$114.96
32	9.25	Continuous	2	6	12	0.88	111	$121.43
32	9.25	Continuous	2	7	18	0.80	111	$117.83
32	11.25	Continuous	2	6	12	0.88	135	$139.30
32	11.25	Continuous	2	7	18	0.80	135	$135.70
32	7.25	Isolated	1	6	10	0.53	87	$88.05
32	7.25	Isolated	1	7	12	0.60	87	$91.16
32	7.25	Isolated	1	8	18	0.53	87	$87.76
32	9.25	Isolated	1	5	10	0.37	111	$98.81
32	9.25	Isolated	1	6	12	0.44	111	$102.04
32	9.25	Isolated	1	7	18	0.40	111	$100.24
32	7.25	Isolated	2	5	12	0.62	87	$91.71
32	7.25	Isolated	2	6	18	0.59	87	$90.63
32	9.25	Isolated	2	5	10	0.74	111	$114.96
32	9.25	Isolated	2	6	12	0.88	111	$121.43
32	9.25	Isolated	2	7	18	0.80	111	$117.83
32	11.25	Isolated	2	6	12	0.88	135	$139.30
32	11.25	Isolated	2	7	18	0.80	135	$135.70
40	7.25	Continuous	1	NG	NG	NG	NG	NG
40	9.25	Continuous	1	7	10	0.72	111	$142.89
40	9.25	Continuous	1	8	12	0.79	111	$146.39
40	7.25	Continuous	2	5	10	0.74	87	$121.36
40	7.25	Continuous	2	6	12	0.88	87	$129.44
40	7.25	Continuous	2	7	18	0.80	87	$124.95
40	9.25	Continuous	2	5	10	0.74	111	$143.70
40	9.25	Continuous	2	6	12	0.88	111	$151.78
40	9.25	Continuous	2	7	18	0.80	111	$147.29
40	11.25	Continuous	2	6	12	0.88	135	$174.12
40	11.25	Continuous	2	7	18	0.80	135	$169.63
40	7.25	Isolated	1	NG	NG	NG	NG	NG
40	9.25	Isolated	1	7	10	0.72	111	$142.89
40	9.25	Isolated	1	8	12	0.79	111	$146.39
40	7.25	Isolated	2	NG	NG	NG	NG	NG
40	9.25	Isolated	2	5	10	0.74	111	$143.70
40	9.25	Isolated	2	6	12	0.88	111	$151.78
40	9.25	Isolated	2	7	18	0.80	111	$147.29
40	11.25	Isolated	2	6	12	0.88	135	$174.12
40	11.25	Isolated	2	7	18	0.80	135	$169.63

Table 11-4. Kansas City Panels

Panel Height (ft)	Panel Thickness (in)	Foundation Support	Rows of Steel	Vertical Reinf. Bar Size	Vertical Spacing (in)	Area of Steel (in^2 / lf)	Area of Concrete (in^2 / lf)	Average Total Price (last 12 months)
32	7.25	Continuous	1	6	10	0.53	87	$89.75
32	7.25	Continuous	1	7	12	0.60	87	$93.62
32	7.25	Continuous	1	8	18	0.53	87	$89.39
32	9.25	Continuous	1	5	10	0.37	111	$97.71
32	9.25	Continuous	1	6	12	0.44	111	$101.72
32	9.25	Continuous	1	7	18	0.40	111	$99.49
32	7.25	Continuous	2	5	12	0.62	87	$94.29
32	7.25	Continuous	2	6	18	0.59	87	$92.96
32	9.25	Continuous	2	5	10	0.74	111	$117.77
32	9.25	Continuous	2	6	12	0.88	111	$125.80
32	9.25	Continuous	2	7	18	0.80	111	$121.33
32	11.25	Continuous	2	6	12	0.88	135	$142.59
32	11.25	Continuous	2	7	18	0.80	135	$138.12
32	7.25	Isolated	1	6	10	0.53	87	$89.75
32	7.25	Isolated	1	7	12	0.60	87	$93.62
32	7.25	Isolated	1	8	18	0.53	87	$89.39
32	9.25	Isolated	1	5	10	0.37	111	$97.71
32	9.25	Isolated	1	6	12	0.44	111	$101.72
32	9.25	Isolated	1	7	18	0.40	111	$99.49
32	7.25	Isolated	2	5	12	0.62	87	$94.29
32	7.25	Isolated	2	6	18	0.59	87	$92.96
32	9.25	Isolated	2	5	10	0.74	111	$117.77
32	9.25	Isolated	2	6	12	0.88	111	$125.80
32	9.25	Isolated	2	7	18	0.80	111	$121.33
32	11.25	Isolated	2	6	12	0.88	135	$142.59
32	11.25	Isolated	2	7	18	0.80	135	$138.12
40	7.25	Continuous	1	NG	NG	NG	NG	NG
40	9.25	Continuous	1	7	10	0.72	111	$146.20
40	9.25	Continuous	1	8	12	0.79	111	$150.55
40	7.25	Continuous	2	5	10	0.74	87	$126.22
40	7.25	Continuous	2	6	12	0.88	87	$136.26
40	7.25	Continuous	2	7	18	0.80	87	$130.67
40	9.25	Continuous	2	5	10	0.74	111	$147.21
40	9.25	Continuous	2	6	12	0.88	111	$157.24
40	9.25	Continuous	2	7	18	0.80	111	$151.66
40	11.25	Continuous	2	6	12	0.88	135	$178.23
40	11.25	Continuous	2	7	18	0.80	135	$172.65
40	7.25	Isolated	1	NG	NG	NG	NG	NG
40	9.25	Isolated	1	7	10	0.72	111	$146.20
40	9.25	Isolated	1	8	12	0.79	111	$150.55
40	7.25	Isolated	2	NG	NG	NG	NG	NG
40	9.25	Isolated	2	5	10	0.74	111	$147.21
40	9.25	Isolated	2	6	12	0.88	111	$157.24
40	9.25	Isolated	2	7	18	0.80	111	$151.66
40	11.25	Isolated	2	6	12	0.88	135	$178.23
40	11.25	Isolated	2	7	18	0.80	135	$172.65