Engineering Design Manual_

ThermoWaLL®
_ panel system.

DISCLOSURE

The construction principles discussed within the pages of this manual are to be viewed as a reference source only. All materials contained within this manual, including but not limited to, calculations, details and schedules, are not intended to set limitations on design nor be interpreted as being the only design criterion. Structural design should be in accordance with local Building Code considerations and therefore, may not necessarily follow the guidelines suggested within this manual.

ACCREDITATION

All engineering calculations, equations, examples and details cited in this manual have been compiled by Namdar Structural Engineering, Inc., Riverside, California. 2002.

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SECTION 1

Introduction

1.0 INTRODUCTION TO STRUCTURAL CALCULATIONS

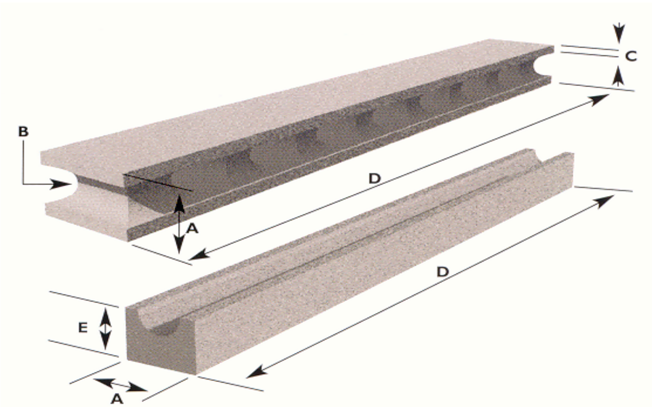


Figure 1

The Perform Wall™ panel system is made of a lightweight material, consisting of beaded, cement coated EPS and is a permanent formwork for reinforced concrete beams, lintels, load bearing walls, roofs, floors, foundation stem walls, basement and retaining walls. A wide variety of Perform Wall™ structures have been built including, clinics, restaurants, commercial buildings, schools, manufacturing plants, truck wash stations, cold storage facilities, apartments and residential homes. The Perform Wall™ panel system is composed of standard panels and end panels, as seen in (figure 1). If a grouted Perform Wall™ panel exterior form were to be stripped away, it would reveal concrete columns with the appearance of a large "reinforced concrete grid." The "grid" is formed by openings or (cells) within the panel, which when stacked together to form a wall, run both vertically and horizontally throughout the structure, creating concrete columns and beams, which form the structural strength of the wall, (see figure 2).

Perform Wall™ panels are available in various thicknesses, ranging from 8 1/2 inches to 14 inches. The 8 1/2 inch panel openings, which are slightly elliptical in shape, measure 5 1/4 inches within the plane of the wall and 5 inches out of plane of the wall. The 10, 12, and 14 inch panel openings are circular and 6-inch in diameter. Once grouted with concrete, the panel becomes a natural insulating barrier against fire and sound. For special structures, the 12 and 14-inch panels can be produced with an 8-inch diameter core for greater structural loads.

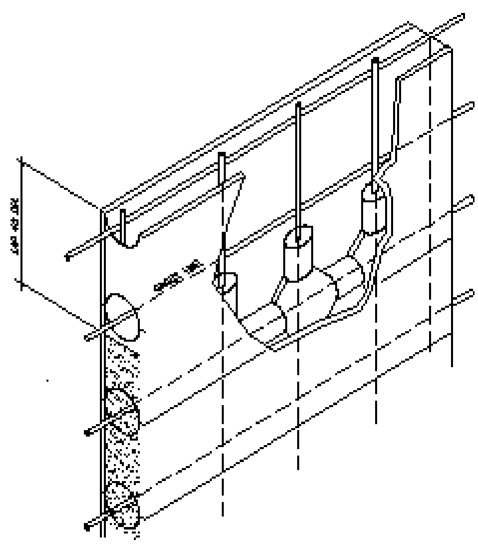
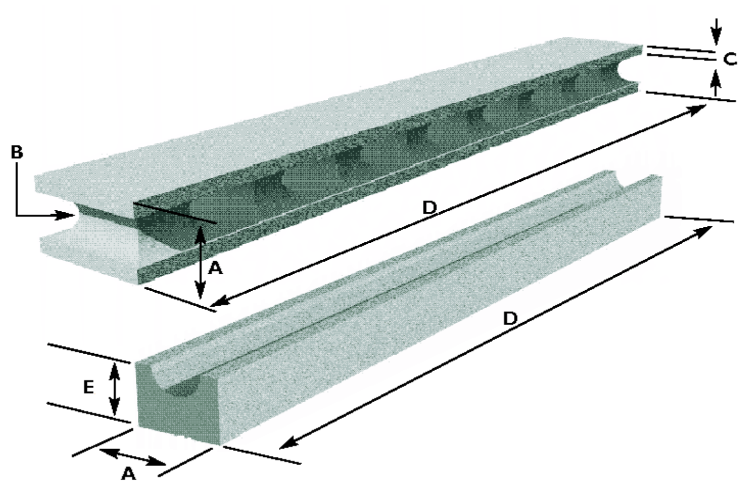


Figure 2

1.1 PHYSICAL PROPERTIES OF PERFORM WALL™ PANELS

Perform Wall™ panels are manufactured in four widths, comprising of two distinct shapes, the main panel and the end panel. The various dimensions and weights of these panels are specified below.



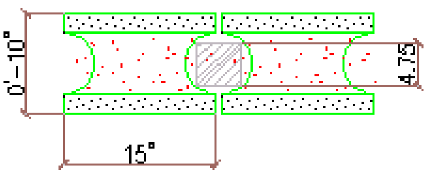
DIMENSIONS - inches (mm)								Panel Weight
PANEL	A	B	C	D	E	Standard Width	Double Width	lbs(kg)±10%
8.5"x15"x120"	8.5" (216)	5" (127)	1.75" (44)	120" (3048)	N.A.	15" (381)	30" (762)	149 (67)
10"x15"x120"	10" (254)	6" (152)	2" (51)		7.5" (191)			158 (72)
12"x15"x120"	12" (305)	6" (152)	3" (76)					197 (90)
14"x15"x120"	14" (356)	6" (152)	4" (102)					248 (110)

TOLERANCES ON MANUFACTURE - inches (mm)							
PANEL	A	B	C	D	E	Standard Width	Double Width
8.5"x15"x120"	±1/8" (3.18)	+3/4" (19.10)	±3/8" (9.53)	+1/2" (12.70) -3/8" (9.53)	+1/2" (12.70) -5/8" (15.88)	±3/32" (2.38)	±3/32" (2.38)
10"x15"x120"	±1/8" (3.18)	-1/2" (12.70)	+1/2",-3/8" (12.70, 9.53)				
12"x15"x120"	±1/8" (3.18)		±1/2" (12.70)				
14"x15"x120"	±3/16" (4.76)		±5/8" (15.88)				

1.2 BASIC CALCULATION METHODS

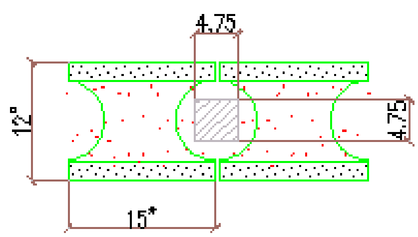
In order to specify structural design calculations for the Perform Wall™ panel system, we replace the circular area of the actual columns and beams, which are spaced at 15 inches on center, with a calculated equivalent rectangular section for an average continuous wall thickness. The dimensions for these equivalent thicknesses are as shown below:

1.3.1 For the **8½ inch** thick panel, calculations are based on the net concrete core of 5 inches within the plane perpendicular to that of the wall and 5 1/4 inches to that within the actual plane of the wall. Using the equivalent block equation of 0.7854 diameter, the circle will result in a rectangular block of Seq=4.25 inches



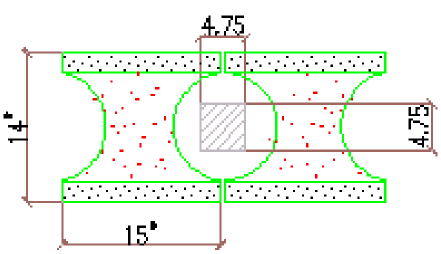
1.3.2 For the **10 inch** thick panel, the minimum net concrete core of 6 inches is calculated by using the equivalent square block equation, which will result in the “S_{eq}=4.75 inches”

1.3.3 For the **12 inch** panel, the minimum net concrete core of 6 inches is calculated using the equivalent square block, which will result in the “S_{eq}=4.75 inches”

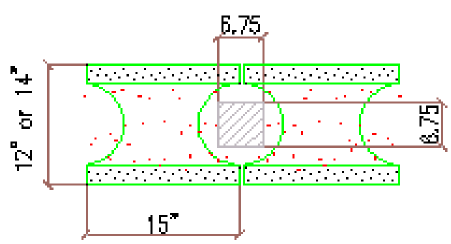


1.3.4 For the **14 inch** panel, the minimum net concrete core of 6 inches, is calculated by using the equivalent square, will result in the “S_{eq}=4.75inches"

1.3.5 For the **12 and 14 inch** panels, a concrete core of 8 inches diameter is also available. This will result in an equivalent square block of “S_{eq}=6.75 inches”. These panels are useful when designing multi level projects and / or slender walls exceeding an unsupported height of 20 feet. This size also allows for easier placement of two layers of reinforcement.



In calculations for the flexure design of walls (except for the shear design in shear walls), the concrete in the horizontal cores between the vertical panels is completely ignored. Although, these panels provide lateral stability, stiffness and strength to the wall, for simplicity of design, they are not considered. The flexural design of lintels is only based upon the horizontal cores. For shallow lintels, which require multiple layers of reinforcement, these cores can be widened to allow for additional placement of reinforcement. The vertical cores are the resisting component for the horizontal shear in



the lintels. For lintels where the shear capacity exceeds the actual shear force of the concrete core on the lintel, shear reinforcement can be added. In addition, only those columns and beams containing steel reinforcement are considered within the calculations within this manual. In the working stress design method, where there is no tension in the core, all cores can be considered in the design. These factors result in a conservative design method.

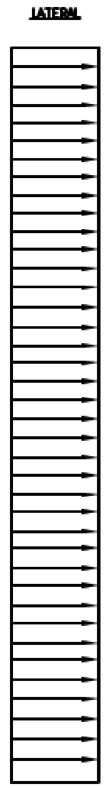
1.3 MINIMUM REINFORCEMENT

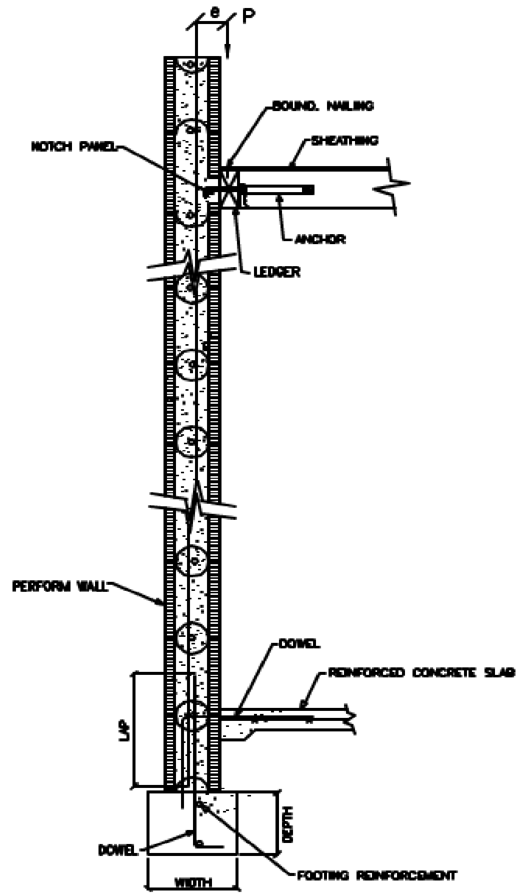
The minimum steel reinforcement for seismic zones I and II and in areas where the basic wind speed is not high as determined by the area building officials, the minimum reinforcement for walls is one #4 rebar placed in every vertical core (on 15-inch centers) and one #4 rebar placed in every other horizontal core (on 30-inch centers). In addition, a minimum of two #5 rebar is required around all windows and door openings. All reinforcement is to be placed in the center of vertical cores and on the bottom surface of horizontal cores, unless otherwise determined by design or as required by local building codes.

1.4 STRUCTURAL VALUES & TECHNICAL INFORMATION

In the following chapters, all necessary structural values and technical information pertaining to the Perform Wall™ panel system will be discussed. The calculations will include analysis, design and tables for allowable loads for a variety of structural members with complete design examples for actual buildings.

1.5 PERFORM WALL™ WALL SYSTEMS





Perform Wall™ is a concrete forming system, which produces a continuous insulated monolithic reinforced concrete wall, which requires no stripping and is ready to receive finish materials. The form produces a waffle shaped concrete wall with concrete cores at 15 inches on center both horizontally and vertically. This forming system optimizes the use of concrete and insulating materials to produce the finished wall.

Because of the unique shape of the resulting concrete wall, requirements for minimum reinforcing steel must be met. If the wall is to be subjected to design loads or longer spans than those required for minimum reinforcement, structural calculations must be performed by a qualified registered engineer to determine the required steel reinforcing for the design conditions.

The concrete system, if properly designed and reinforced, will perform adequately for walls with the following design considerations: Axial and lateral loads, columns which carry concentrated loads, especially on the sides of openings. Lintels and beams carrying gravity as well as lateral loads, elevated slabs with reinforcement or post tensioned slabs carrying gravity as well as lateral loads, and shear walls built to resist in-plane shear loads from shear diaphragms or other shear elements.

The Perform Wall™ panel system can be used for exterior load bearing and non-load bearing wall construction, as well as for interior load bearing and non-load bearing walls. Foundation stem walls, basement walls, freestanding fence walls and retaining walls are also easily possible with this system. Section 3 of this manual provides methods for the proper design of Perform Wall™ construction options.

1.6 PERFORM WALL™ LINTEL

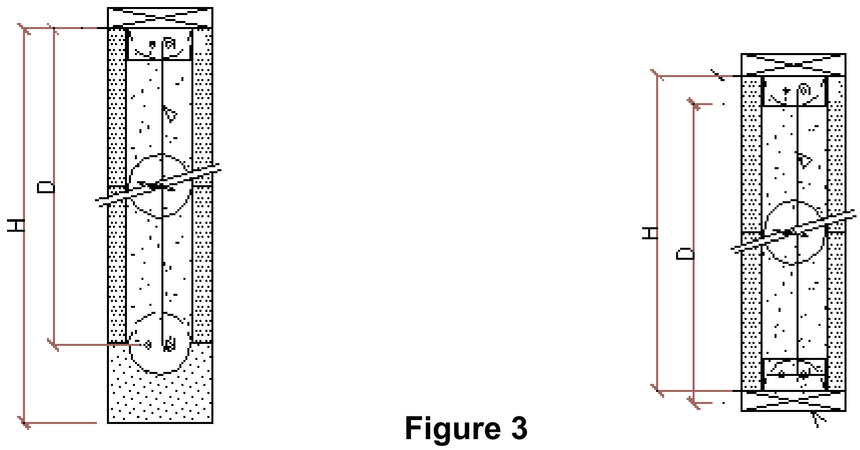


Figure 3

The Perform Wall™ lintel is an integral part of the Perform wall™ system. The lintel is also a concrete forming component, which also produces a continuous insulated monolithic concrete section, which requires no stripping and is ready to receive finish materials. The lintel can have either an end panel at the bottom of the lintel, which houses the flexural reinforcement of the lintel, as shown in figure 3, or it can be built with the use of standard panels, which will have to be either widened or deepened for proper placement of the flexural reinforcement see figure 4.

The Perform Wall™ lintel can simply be designed to support typical headers for a residence, as well as large spans of up to 20 feet or more for low or multi level structures. Perform Wall™ lintels can be either designed as simple span beams, fixed beams or continuous members. Because of the unique shape of the concrete, requirements for minimum reinforcing steel must be met. This is necessary to maintain the structural integrity of the system throughout its useful life. If the wall is to be subjected to design loads or longer spans than those required for minimum reinforcement, structural calculations must be performed by a qualified registered engineer to determine the required steel reinforcing for the appropriate design conditions.

The Perform Wall™ wall lintel system, if properly designed and reinforced, can be used for lintels, beams, girders and spandrels carrying gravity as well as lateral load The lintel design theory can also be applied to the design of elevated slabs with reinforcement or post tensioned slab carrying gravity as well as lateral loads. The Perform Wall™ lintel and slab system can be used for the construction of residential, commercial, parking structure, movie theaters, hospitals, auditoriums and virtually any other type of building. The use of Perform Wall™ is especially versatile in structures requiring high sound, fire and thermodynamic requirements, as the forms act as a natural barrier for these properties.

1.7 MINIMUM REINFORCEMENT REQUIREMENTS

The Perform Wall™ Lintel system requires the use of reinforcement to meet the American Concrete Institute's Building Code Requirements for Reinforced Concrete (ACI 318), Uniform Building Code chapter 19 or SBCCI code chapter. The following recommendations conform to the requirements of these Codes with the exception of ACI Section 14.3.5 or UBC section 19.14.3.5, which requires a maximum spacing of reinforcing of 18 inches in both the horizontal and vertical directions. This criteria is usually not required to be met in most residential, light commercial or light industrial applications.

Minimum reinforcing for the Perform Wall™ lintel is one #4 (1/2") bar placed in every horizontal core (15" inch centers) and one #4 bar placed in every vertical core (15 inch centers). This minimum reinforcing exceeds the requirements of the CABO One and Two Family Dwelling Code, which are accepted by ICBO, BOCA, and SBCCI for residential construction. Every opening less than 4 feet wide must have a reinforced lintel over the door of at least 15" of concrete depth, with two #5 bars, each with 3/4" clear concrete cover on the bar at the top and bottom of the lintel; and two vertical #5 bars on each side of the opening to meet the minimum requirements of ACI for reinforcement around openings.

SECTION 2

Loads, Gravity, Wind and Earthquake

2.1 INTRODUCTION

The design of structures involves the application of external resistance provided by the construction members. Simply stated, the internal resistance provided by the members, and the interconnections of these members, must be equal to or greater than the application of external loads. The gravity loads are defined as dead and live loads and these depend upon the members weight or any other superimposed permanent load combined with the occupancy loads, respectively. The lateral loads are defined as, though not limited to, wind and seismic loads. Chapter 3 describes the seismic loads and their applications in detail.

The gravity loads, when combined with the lateral loads, produce, at times, maximum stresses. However, since the lateral loads are not always acting permanently, an increase in the allowable stress is generally permitted. Similarly, the gravity live loads are generally reduced, depending upon the probability of all areas being occupied at the same time.

This chapter deals with the material weights, occupancy loads, wind loads, and a combination of loads with appropriate stress increases. The wind and snow loads are described in detail with assumed intended loading diagrams. For serviceability purposes, deflection limitations are also defined.

2.2 LOADS AND COMBINATIONS

The material in section 2.2 is taken from the International Conference of Building Officials, Unified Building Code (ICBO - UBC), the Southern Building Code Congress International, Standard Building Code (SBCCI – STC) and Building Officials Code Administrators (BOCA). All subsections and equations have the same numbering as it appears within the UBC.

DEAD LOAD is the vertical load due to the weight of all permanent structural and nonstructural components of a building, such as walls, floors, roofs and fixed service equipment.

LIVE LOAD is the load superimposed by the use and occupancy of the building not including the wind load, earthquake load or dead load.

LOAD DURATION is the time period of continuous application of a given load, or the aggregate of periods of intermittent application of the same load.

(1) **GENERAL**. All buildings and portions thereof, shall be designed and constructed to sustain, within the stress limitations specified in this code, all dead loads and all other loads specified in this chapter or elsewhere in this code. Impact loads shall be considered in the design of any structure where impact loads occur.

EXCEPTION: Unless otherwise required by the building official, buildings or portions thereof, which are constructed in accordance with the conventional framing requirements specified in Chapter 25 of the UBC shall be deemed to meet the requirements of this section.

(2) **RATIONALITY**. Any system or method of construction to be used shall be based on a rational analysis in accordance with well-established principles of mechanics. Such analysis shall result in a system, which provides a complete load path capable of transferring all loads and forces from their point of origin to the load resisting elements. The analysis shall include, but not be limited to, the following:

DISTRIBUTION OF HORIZONTAL SHEAR

The total lateral force shall be distributed to the various vertical elements of the lateral force-resisting system in proportion to their rigidities, considering the rigidity of the horizontal bracing system or diaphragm. Rigid elements that are assumed not to be part of the lateral force-resisting system may be incorporated into buildings, provided that their effect on the action of the system is considered and provided for within the design.

HORIZONTAL TORSIONAL MOMENTS

Provisions shall be made for the increased forces induced on resisting elements of the structural system resulting from torsion due to eccentricity between the center of application of lateral forces and the center of rigidity of the lateral force-resisting system. Forces shall not be decreased due to torsional effects. For accidental torsion requirements for seismic design, see UBC Section 2334 (f) 6.

STABILITY AGAINST OVERTURNING

Every building or structure shall be designed to resist the overturning effect caused by the lateral forces specified in this chapter. See UBC Section 2317 for wind and UBC Section 2334 (g) for seismic.

ANCHORAGE

Anchorage of the roof to walls and columns to foundations shall be provided to resist the uplift and sliding forces, which result from the application of the prescribed forces. For additional requirements for masonry or concrete walls, see UBC Section 2310.

CRITICAL DISTRIBUTION OF LIVE LOADS
Where structural members are arranged so as to crate continuity, the loading conditions, which would cause maximum shear and bending moments along the members, shall be investigated.

STRESS INCREASES
All allowable stresses and soil-bearing values specified in this code for working stress design may be increased one-third, when considering wind or earthquake forces either acting along or when combined with vertical loads. No increases will be allowed for vertical loads acting alone.

LOAD FACTORS
Load factors for ultimate strength design of concrete and plastic design of steel shall be as indicated in the appropriate chapters on the materials.

LOAD COMBINATIONS
Every building component shall be provided with strength adequate to resist the most critical effect resulting from the following combination of loads (floor live load shall not be included where its inclusion results in lower stresses in the member under investigation):

1. Dead plus floor live plus roof live (or snow)[2]
2. Dead plus floor live plus wind[2] (or seismic)
3. Dead plus floor live plus wind plus snow/2[2]
4. Dead plus floor live plus snow plus wind/2[2]
5. Dead plus floor live plus snow[3] plus seismic

2.3 FLOOR DESIGN

2.3.1 GENERAL. Floor shall be designed for the unit loads set forth in Table 16-A. or 1604.1, 1604.2 & 1604.3 of SBCCI Code. These loads shall be taken as the minimum live loads in pounds per square foot of horizontal projection to be used in the design of buildings for the occupancies listed, and loads being at least equal, shall be assumed for uses not listed in this section but which create or accommodate similar loadings. When designing floors where the actual live load will be greater than the value shown in Tables above, actual live load shall be used in the design of such buildings or part thereof and special provisions shall be made for machine or apparatus loads.

2.3.2 DISTRIBUTION OF UNIFORM FLOOR LOADS. Where uniform floors loads are involved, consideration may be limited to full dead load on all spans in combination with full live load on adjacent spans and on alternate spans.

2.2.3 CONCENTRATED LOADS. Provision shall be made in designing floors for concentrated loads as set forth in Table 16-A (UBC) placed upon any space 2 ½ feet square, wherever this load acts upon an otherwise unloaded floor would produce stresses greater than those caused by the uniform load required therefore.

Provision shall be made in areas where vehicles are used or stored for concentrated loads consisting of two or more loads space 5 feet nominally on center without uniform live loads. Each load shall be 40 percent of the gross weigh of the maximum size vehicle to be accommodated. The condition of concentrated or uniform live load producing greater stresses shall govern. Parking garage for the storage of private or recreational-type motor vehicles with no repair or fueling shall have a floor system designed for a concentrated wheel load of not less than 2000 pounds without uniform live loads. The condition of concentrated or uniform live load providing the greater stresses shall govern.

Provisions shall be made for special vertical and lateral loads as set forth.

2.3.4 PARTITION LOADS. Floors in the office buildings and in other buildings where partition locations are subject to change shall be designed to support, in addition to all other loads, a uniformly distributed dead load equal to 20 pounds per square foot. Access floor systems may be designed to support, in addition to all other loads, a uniformly distributed dead load equal to 10 pounds per square foot.

2.3.5 LIVE LOADS POSTED. The live loads for which each floor or part thereof of a commercial or industrial building, shall have such designed live loads conspicuously posted by the owner in that part of each story in which they apply, using durable metal signs, and it shall be unlawful to remove or deface such notices. The occupant of the building shall be responsible for keeping the actual load below the allowable limits.

2.4 ROOF DESIGN

2.4.1 GENERAL. Roofs shall sustain, within the stress limitations of this code, all “dead loads” plus unit “live loads” as set forth in Table 16-C (UBC). The live loads shall be assumed to act vertically upon the area projected upon a horizontal plane.

2.4.2 DISTRIBUTION OF LOADS. Where uniform roof loads are involved in the design of structural members arranged so as to create continuity, consideration may be limited to full dead loads on all spans in combination with full live loads on adjacent spans and on alternate spans.

EXCEPTION: Alternate span loading need not be considered where the uniform roof live load is 20 pounds per square foot or more or the provisions of UBC Section 1605 (d) are met.

2.4.3 UNBALANCED LOADING. Unbalanced loads shall be used where such loading will result in larger members or connections. Trusses and arches shall be designed to resist the stresses caused by unit live loads on one-half of the span, if such loading results in reverse stresses, or stresses are greater in any portion than the stresses produced by the required unit live load upon the entire span. For roofs whose structure is composed of a stressed shell, framed or solid, wherein stresses caused by any point loading are distributed throughout the area of the shell, the requirements for unbalance unit live load design may be reduced 50 percent.

2.4.4 SNOW LOADS. Snow loads full or unbalanced shall be considered in place of loads set forth in section 1608 of UBC Code or section 1605 of SBCCI Code, where such loading will result in large members of connections. Potential accumulation of snow at valleys, parapets, roof structures and offsets in roofs of uneven configuration shall be considered, where snow loads occur, the snow loads shall be determined by the building official. Snow loads in excess of 20 pounds per square foot may be reduced for each degree of pitch over 20 degrees as determined by the following formula:

$$R_s = \frac{S}{40} - \frac{1}{2}$$

FOR **SI**: $$R_s = \frac{S}{40} - 0.024$$

WHERE:

$R_s =$ Snow load reduction in pounds per square foot (kN/m^2)

$S =$ Total snow load in pounds per square foot (kN/m^2)

2.4.5 SPECIAL-PURPOSE ROOFS. Roofs to be used for special purposes shall be designed for appropriate loads as approved by the building official. Greenhouse roof bars, purlin and rafters shall be designed to carry a 100-pound minimum concentrated load in addition to the live load.

2.4.6 WATER ACCUMULATION. All roofs shall be designed with sufficient slope or camber to assure adequate drainage after the long time deflection from dead load or shall be designed to support maximum loads, including snow, due to deflection. See UBC Section 2307 for deflection criteria.

2.4.7 DEFLECTION. The deflection of any structural member shall not exceed the values set forth in section 1613 of UBC Code and section 1610 of SBCCI Code. The deflection criteria representing the most restrictive condition shall apply. Deflection criteria for materials not specified shall be developed in a manner consistent with the provisions of this section.

2.5 SPECIAL DESIGN

2.5.1 GENERAL. In addition to the design loads specified in this chapter, the design of all structures shall consider the special loads set forth in Table 16-B (UBC).

2.5.2. RETAINING WALLS. Retaining walls shall be designed to resist the lateral pressure of the retained material in accordance with accepted engineering practice. Walls retaining drained earth may be designed for pressure equivalent to that exerted by a fluid weighing not less than 30 pounds per cubic foot and having a depth equal to that of the retained earth. Any surcharge shall be in addition to the equivalent fluid pressure. Retaining walls shall be designed to resist sliding and overturning by at least 1.5 times the lateral force and overturning moment.

2.6 WIND LOADS

The material in Section is taken from the UBC. All subsection, and equations have the same numbering as it appears in the UBC.

(a) **General**. Every building or structure and every portion thereof shall be designed and constructed to resist the wind effects determined in accordance with the requirements of this section. Wind shall be assumed to come from any horizontal direction. No reduction in wind pressure shall be taken for the shielding effect of adjacent structures.

Structures sensitive to dynamic effects, such as buildings with a height-width ratio greater than five, structures sensitive to wind-excited oscillations, such as vortex shedding or icing, and building over 400 feet in height, shall be, and any structure may be, designed in accordance with approved national standards.

(b) **Basic Wind Speed**. The minimum basic wind speed for determining design wind pressure shall be taken from section 1615 of UBC Code or section 1606 of SBCCI Code where terrain features and local records indicate that 50-year wind speeds at standard height are higher than those shown in Figure 16-1 of UCB 1997, these higher values shall be the minimum basic wind speeds.

(c) **Exposure**. An exposure shall be assigned to each site for which a building or structure is to be designed. Exposure D represents the most severe exposure in areas with basic wind speeds of 80 miles per hour (mph) or greater and has terrain which is flat and unobstructed facing large bodies of water over one mile or more in width relative to any quadrant of the building site. Exposure D extends inland from the shoreline ¼ mile or 10 times the building height- whichever is greater. Exposure C represents the terrain, which is flat and generally open, extending one-half mile or more from the site in any full quadrant. Exposure B has terrain, which has buildings, forest and surface irregularities 20 feet or more in height covering at least 20 percent of the area extending one mile or more from the site.

(d) **Design wind pressures**. Design wind pressures for structures or elements of structures shall be determined for any height in accordance with the following formula:

$$P = C_e C_q Q_s I_w$$

Where:

$C_e =$ Combined height, exposure and gust factor coefficient as given in Table 16-G (UBC)

$C_q =$ Pressure coefficient for the structure or portion of structure under Consideration.

$I_w =$ Importance factor.

$P =$ design wind pressure.

$q_s =$ wind stagnation pressure at the standard height of 33 feet (10,000 mm) .

(e) **Primary Frames and System**. The primary frames or load-resisting system of every structure shall be designed for the pressures calculated using formulas in section 1621 of UBC Code or 1606.24.1 of SBBCI Code and the pressure coefficients, Cq, of either Method 1 or Method 2. In addition, design of the overall structure and its primary load-resisting system shall conform to gravity and seismic loading.

The base overturning moment for the entire structure, or for any one of its individual primary lateral resisting elements, shall not exceed two-thirds of the dead-load-resisting moment. For an entire structure with a height-to-width ratio of 0.5 or less in the wind direction and a maximum height of 60 feet, the combination of the effect of uplift and overturning moments may be reduced by one-third. The weight of earth superimposed over footings may be used to calculate the dead-load-resisting moment.

1. **Method 1** (Normal Force Method). Method 1 shall be used for the design of gabled rigid frames and may be used for any structure. In the Normal Force Method, the wind pressures shall be assumed to act simultaneously normal to all exterior surfaces. For pressures on leeward walls, Cs shall be evaluated at the mean roof height.

2. **Method 2** (Projects Area Method). Method 2 may be used for any structure less than 200 feet in height except those using gabled rigid frames. This method may be used in stability determinations for any structure less than 200 feet high.

In the Projected Area Method, horizontal pressures shall be assumed to act upon the full vertical projected area of the structure, and the vertical pressures shall be assumed to act simultaneously upon the full horizontal projected area.

(f) **Elements and Components of Structures**. Design wind pressures for each element or component of a structure shall be determined from section 1622 of UBC Code or 1606 of SBCCI Code.

2.7 SNOW LOAD DESIGN

For snow loading on roof, refer to UBC, BOCA and SBCCI code.

2.8 EARTHQUAKE & LOADS

The material in section 2.8 is taken from the International Conference of Building Officials, Unified Building Code (ICBO-UBC), the Southern Building Code Congress International, Standard Building Code (SBCCI-STC) and Building Officials Code Administrators (BOCA). All subsections and equations have the same numbering as it appears within the UBC.

The subject of lateral load design is a combination of engineering analysis, statics and dynamics, and the engineering design for all the various materials that are utilized in the lateral load resisting system. The material contained within this section is only an introduction. The total subject could fill several volumes and even then the practical experience learned from real earthquakes would be hard to include. Seismic loads have been the emphasis of the research at our civil engineering schools and complete texts deal with the calculation of these loads.

For this document to be effective in increasing the reader’s knowledge of seismic design it must be realized that the minimum UBC earthquake forces presented in this section are not representative of the real forces induced in structures when subjected to strong ground shaking. The minimum UBC requirements should not be treated as a “maximum” criterion for design purposes. The representation of forces as a set of static equivalent forces, which can significantly understate the actual earthquakes forces, is not always the most conservative engineering approach. The result is that structures designed by the UBC approach will experience demands greater than the capacity of their seismic resisting elements and the structures will thus perform in the inelastic or non-linear range. Designing for forces that are significantly greater than the UBC, maybe 3 to 4 times the UBC forces, to ensure the elastic behavior of the structure in all earthquakes is not justified by the performance of properly designed structures in recent earthquakes, nor can the economic impact of these higher forces always be justified. An increase in design levels above the minimum UBC levels may provide for more damage control, as is the intent of the more stringent hospital design regulations in California. The increased construction costs can be evaluated against the benefits from the reduction of damage in future earthquake.

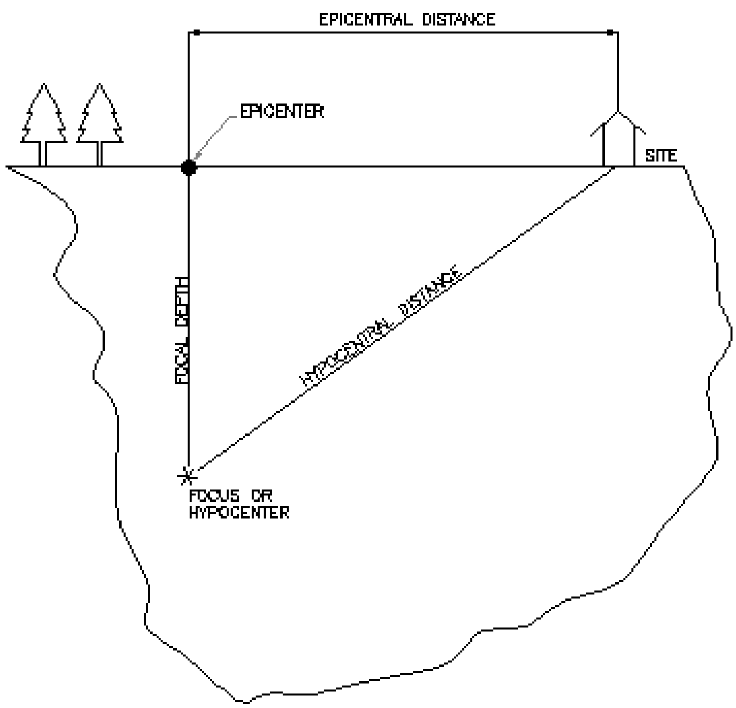
Perform Wall™ structures of substantial size can be designed to perform adequately under major earthquakes, provided careful design and detailing requirements are followed. Earthquakes have a tendency to detect weak links in structural systems and concentrate the damage in these elements. By virtue of the form of Perform Wall™ construction has a larger number of potential weak links than other materials.

Careful conservative design is not enough if proper construction techniques are not strictly followed. The brevity of this document and the referenced Uniform Building Code requires that proper seismic design and construction practices be based upon:

1. More than a minimum literal code interpretation.
2. An understanding of the intent of the code provisions.
3. Good engineering principles and sound judgment.
4. The designer should be familiar with such concepts as load paths, redundancy, regular vs. irregular buildings, strength, stiffness, ductility, and static versus dynamic loading.

2.8.1 EARTHQUAKE CHARACTERISTICS

The Figure below shows a cross section of the earth.



The following are common earthquake engineering terms:

Focus (or hypo center): the center of the initial rupture causing the earthquake.
Epicenter: distance from the epicenter to the site.
Focal Depth: the depth of the focus beneath the surface.
Hypo Central: distance from the focus or hypocenter to the site.

Perhaps the most familiar term used to characterize an earthquake is its Richter magnitude (M). This magnitude is based on an experimental reading obtained on an instrument called a Wood-Anderson seismograph at a specified distance of 100 Km. from the epicenter of the earthquake. If there is no instrument at this distance from the epicenter, empirical formulas are used to estimate the 100 Km. reading. The Richter magnitude is a measure of the energy released by an earthquake. A Modified Mercalli Intensity (MMI) is used to describe the observed effect of ground shaking at a particular site. The value of MMI, assigned after an earthquake at a particular site, is a subjective evaluation of damage made by an observer and is valuable in lieu of instrument records of ground motion.

In recent years, the acquisition of time histories of earthquake ground motion has become common. Instruments called strong-motion accelerographs are placed on the ground to measure earthquake-induced ground motion. The accelergoraphs measure the three orthogonal components of ground acceleration. The integrated velocity and displacement time histories can be obtained. An acceleration versus time trace is called an accelerogram. The earthquake accelerogram can be interpreted directly to obtain estimates of peak ground acceleration, duration of strong shaking, and frequency content.

TABLE 2.8.1 MODIFIED MERCALLI INTENSITY SCALE

MMI SCALE	**DESCRIPTION**
I.	Not felt except under especially favorable circumstances.
II.	Felt by persons at rest, on upper floors, or favorably placed.
III.	Felt indoors, Hanging objects swing. Vibration like passing of light trucks. May not be recognized as an earthquake.
IV.	Hanging objects swing. Vibration like passing of heavy truck or sensation of a jolt like a heavy ball striking the wall. Standing motorcars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. Wooden walls and frames creak.
V.	Felt outdoors; direction estimated. Sleepers wakened. Liquids disturbed, some pilled. Small unstable objects displaced or upset. Doors swing close, shutters open and pictures move.
VI.	Felt by all. Persons walk unsteadily. Windows, dishes, and glassware are broken. Knickknacks, books and so forth, fall off shelves, pictures off walls. Furniture moved or overturned.
VII.	Difficult to stand. Noticed by drivers of automobiles. Hanging objects quiver. Furniture broken. Weak chimneys broken at roofline. Fall of plaster, loose bricks, stones, tiles, cornices, also unbraced parapet and architectural ornaments.

VIII Steering of automobiles affected. Twisting, fall of chimneys, factory stacks, monuments, towers, and elevated tanks. Frame houses move off foundations if not bolted down.

IX. General panic, general damage to foundations, frame structures, if not bolted, shifted off foundations and frames racked, serious damage to reservoirs and underground pipes broken. Conspicuous cracks in ground. In alleviated area sand and mud ejected earthquake foundations, and craters.

X Most Perform Wall™ and frame structures damaged with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankment. Large landslides. Water thrown on the banks of canals, rivers and lakes. Sand and mud shifted horizontally on beaches and flat land. Rails bent slightly.

XI. Rails bent greatly. Underground pipelines completely out of service.

XII. Damage nearly total. Large rock masses displaced. Lines of sight and level distorted. Objects thrown into the air.

2.8.3 RESPONSE SPECTRA

Earthquake accelerogram show the irregularity of the accelerations as a function to time. Although the values for duration of strong shaking and peak ground acceleration provide basic information about the earthquake ground motion, the structural engineer must have a more meaningful characterization for use in structural design. This is provided by a response spectrum. The response spectrum is obtained from an accelerogram and indicates how a single-degree-of-freedom oscillator would respond if it were excited by the earthquake ground motion described in the accelerogram.

The structural engineer usually relies on a smoothed version of a response spectrum for design purposes. This representation characterizes the earthquake motion by defining maximum values of acceleration, velocity and displacement response that are constant for broad period bands. This smoothing of the response spectrum is also consistent with the engineer's inability to predict the natural frequencies of structure as well as the observed dependence of the period on the force level to which the structure is subjected. Thus, peaks and valleys occurring over narrow period bands have little structural significance.

The development of a response spectrum for use in the design of a building, at a specific site, is usually not within the scope of the structural engineer's responsibility. Such a spectrum is usually prepared by a geotechnical consultant and has become a document requiring special expertise. The structural engineer must ensure that he or she understands, in a quantitative way, the information provided by the geotechnical consultant.

2.8.4 SEISMIC LOADS

Design for Seismic Forces
Every structure shall be designed and constructed to resist stresses induced by seismic forces. These forces are applied horizontally at all floor levels and at roof level above the base and shall be assumed to come from any horizontal at all floor levels and at roof level above the base and shall be assumed to come from any horizontal direction. Code prescribed forces provides minimum standards for structures to be earthquake resistant. Seismic forces and wind forces are assumed to not act concurrently, thus, if wind forces produced higher stresses such forces shall be used for analysis and design. When wind forces govern the stress or drift design, the resisting system must conform to the ductility, design, and special requirements similar to those required for seismic systems.

The following material presented in Section 2.8 (below) is excerpted from Sections 1626 thru 1635 of the UBC. All sub-sections, table, figures and equations have the same numbering as appear in the UBC. See the 1997 UBC for full text.

UBC EARTHQUAKE LOADS AND MODELING REQUIREMENTS

1629 - CRITERIA SELECTION

1629.1 Basis for Design. The procedures and the limitations for the design of structures shall be determined considering seismic zoning, site characteristics, occupancy, configuration, structural system and height in accordance with this section. Structures shall be designed with adequate strength to withstand the lateral displacements induced by the Design Basis Ground Motion, considering the inelastic response of the structure and the inherent redundancy, overstrength and ductility of the lateral-force-resisting system. The minimum design strength shall be based on the Design Seismic Forces determined in accordance with the static lateral force procedure of Section 1630, except as modified by Section 1631.5.4. Where strength design is used, the load combinations of Section 1612.2 shall apply. Where Allowable Stress Design is used, the load combinations of Section 1612.3 shall apply. Allowable Stress Design may be used to evaluate sliding or overturning at the soil-structure interface regardless of the design approach used in the design of the structure, provided load combinations of Section 1612.3 are utilized. One- and two-family dwellings in Seismic Zone 1 need not conform to the provisions of this section.

1629.2 Occupancy Categories. For purposes of earthquake- resistant design, each structure shall be placed in one of the occupancy categories listed in Table 16-K. Table

16-K assigns importance factors, *I* and *Ip,* and structural observation requirements for each category.

1629.3 Site Geology and Soil Characteristics. Each site shall be assigned a soil profile type based on properly substantiated geotechnical data using the site categorization procedure set forth in Division VI, Section 1636 and Table 16-J.

> **EXCEPTION:** When the soil properties are not known in sufficient detail to determine the soil profile type, Type *SD* shall be used. Soil Profile Type *SE* or *SF* need not be assumed unless the building official determines that Type *SE* or *SF* may be present at the site or in the event that Type *SE* or *SF* is established by geotechnical data.

1629.3.1 Soil profile type. Soil Profile Types *SA, SB, SC, SD* and *SE* are defined in Table 16-J and Soil Profile Type *SF* is defined as soils requiring site-specific evaluation as follows:

1. Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils.

2. Peats and/or highly organic clays, where the thickness of peat or highly organic clay exceeds 10 feet (3048 mm).

3. Very high plasticity clays with a plasticity index, *PI* > 75, where the depth of clay exceeds 25 feet (7620 mm).

4. Very thick soft/medium stiff clays, where the depth of clay exceeds 120 feet (36 576 mm).

1629.4 Site Seismic Hazard Characteristics. Seismic hazard characteristics for the site shall be established based on the seismic zone and proximity of the site to active seismic sources, site soil profile characteristics and the structure's importance factor.

1629.4.1 Seismic zone. Each site shall be assigned a seismic zone in accordance with Figure 16-2. Each structure shall be assigned a seismic zone factor *Z,* in accordance with Table 16-I.

1629.4.2 Seismic Zone 4 near-source factor. In Seismic Zone 4, each site shall be assigned a near-source factor in accordance with Table 16-S and the Seismic Source Type set forth in Table 16-U. The value of *Na* used to determine *Ca* need not exceed

1.1 for structures complying with all the following conditions:

1. The soil profile type is *SA, SB, SC* or *SD.*
2. ρ = 1.0.
3. Except in single-story structures, Group R, Division 3 and Group U, Division 1 Occupancies, moment frame systems designated as part of the lateral-force-resisting system shall be special moment-resisting frames.
4. The exceptions to Section 2213.7.5 shall not apply, except for columns in one-story buildings or columns at the top story of multistory buildings.
5. None of the following structural irregularities is present: Type 1, 4 or 5 of Table 16-L, and Type 1 or 4 of Table 16-M.

1629.4.3 Seismic response coefficients. Each structure shall be assigned a seismic coefficient, *Ca,* in accordance with Table 16-Q and a seismic coefficient, *Cv,* in accordance with Table 16-R.

1629.5 Configuration Requirements.

1629.5.1 General. Each structure shall be designated as being structurally regular or irregular in accordance with Sections 1629.5.2 and 1629.5.3.

1629.5.2 Regular structures. Regular structures have no significant physical discontinuities in plan or vertical configuration or in their lateral-force-resisting systems such as the irregular features described in Section 1629.5.3.

1629.5.3 Irregular structures.

1. Irregular structures have significant physical discontinuities in configuration or in their lateral-force-resisting systems. Irregular features include, but are not limited to, those described in Tables 16-L and 16-M. All structures in Seismic Zone 1 and Occupancy Categories 4 and 5 in Seismic Zone 2 need to be evaluated only for vertical irregularities of Type 5 (Table 16-L) and horizontal irregularities of Type 1 (Table 16-M).

2. Structures having any of the features listed in Table 16-L shall be designated as if having a vertical irregularity.

EXCEPTION: Where no story drift ratio under design lateral forces is greater than 1.3 times the story drift ratio of the story above, the structure may be deemed to not have the structural irregularities of Type 1 or 2 in Table 16-L. The story drift ratio for the top two stories need not be considered. The story drifts for this determination

may be calculated neglecting torsional effects.
3. Structures having any of the features listed in Table 16-M shall be designated as having a plan irregularity.

1629.6 Structural Systems.

1629.6.1 General. Structural systems shall be classified as one of the types listed in Table 16-N and defined in this section.
1629.6.2 Bearing wall system. A structural system without a complete vertical load-carrying space frame. Bearing walls or bracing systems provide support for all or most gravity loads. Resistance to lateral load is provided by shear walls or braced frames.
1629.6.3 Building frame system. A structural system with an essentially complete space frame providing support for gravity loads. Resistance to lateral load is provided by shear walls or braced frames.

1629.6.4 Moment-resisting frame system. A structural system with an essentially complete space frame providing support for gravity loads. Moment-resisting frames provide resistance to lateral load primarily by flexural action of members.
1629.6.5 Dual system. A structural system with the following features:

1. An essentially complete space frame that provides support for gravity loads.

2. Resistance to lateral load is provided by shear walls or braced frames and moment-resisting frames (SMRF, IMRF, MMRWF or steel OMRF). The moment-resisting frames shall be designed to independently resist at least 25 percent of the design base shear.

3. The two systems shall be designed to resist the total design base shear in proportion to their relative rigidities considering the interaction of the dual system at all levels.

1629.6.6 Cantilevered column system. A structural system relying on cantilevered column elements for lateral resistance.

1629.6.7 Undefined structural system. A structural system not listed in Table 16-N.

1629.6.8 Non-building structural system. A structural system conforming to Section 1634.

1629.7 Height Limits. Height limits for the various structural systems in Seismic Zones 3 and 4 are given in Table 16-N.

EXCEPTION: Regular structures may exceed these limits by not more than 50 percent for unoccupied structures, which are not accessible to the general public.

1629.8 Selection of Lateral-force Procedure.

1629.8.1 General. Any structure may be, and certain structures defined below shall be, designed using the dynamic lateral-force procedures of Section 1631.

1629.8.2 Simplified static. The simplified static lateral-force procedure set forth in Section 1630.2.3 may be used for the following structures of Occupancy Category 4 or 5:

1. Buildings of any occupancy (including single-family dwellings) not more than three stories in height excluding basements that use light-frame construction.

2. Other buildings not more than two stories in height excluding basements.

1629.8.3 Static. The static lateral force procedure of Section 1630 may be used for the following structures:

1. All structures, regular or irregular, in Seismic Zone 1 and in Occupancy Categories 4 and 5 in Seismic Zone

2. Regular structures under 240 feet (73 152 mm) in height with lateral force resistance provided by systems listed in Table 16-N, except where Section 1629.8.4, Item 4, applies.

3. Irregular structures not more than five stories or 65 feet (19 812 mm) in height.

4. Structures having a flexible upper portion supported on a rigid lower portion where both portions of the structure considered separately can be classified as being regular, the average story stiffness of the lower portion is at least 10 times the average story stiffness of the upper portion and the period of the entire structure is not greater than 1.1 times the period of the upper portion considered as a separate structure fixed at the base.

1629.8.4 Dynamic. The dynamic lateral-force procedure of Section 1631 shall be used for all other structures, including the following:

1. Structures 240 feet (73 152 mm) or more in height, except as permitted by Section 1629.8.3, Item 1.

2. Structures having a stiffness, weight or geometric vertical irregularity of Type 1, 2

or 3, as defined in Table 16-L, or structures having irregular features not described in Table 16-L or 16-M, except as permitted by Section 1630.4.2.

3. Structures over five stories or 65 feet (19 812 mm) in height in Seismic Zones 3 and 4 not having the same structural system throughout their height except as permitted by Section 1631.2.

4. Structures, regular or irregular, located on Soil Profile Type *SF that* has a period greater than 0.7 second. The analysis shall include the effects of the soils at the site and shall conform to Section 1631.2, Item 4.

1629.9 System Limitations.

1629.9.1 Discontinuity. Structures with a discontinuity in capacity, vertical irregularity Type 5 as defined in Table 16-L, shall not be over two stories or 30 feet (9144 mm) in height where the weak story has a calculated strength of less than 65 percent of the story above.

> **EXCEPTION:** Where the weak story is capable of resisting a total lateral seismic force of Ω_o times the design force prescribed in Section 1630.

1629.9.2 Undefined structural systems. For undefined structural systems not listed in Table 16-N, the coefficient *R* shall be substantiated by approved cyclic test data and analysis. The following items shall be addressed when establishing *R:*

1. Dynamic response characteristics,

2. Lateral force resistance,

3. Overstrength and strain hardening or softening,

4. Strength and stiffness degradation,

5. Energy dissipation characteristics,

6. System ductility, and

7. Redundancy.

1629.9.3 Irregular features. All structures having irregular features described in Table 16-L or 16-M shall be designed to meet the additional requirements of those sections referenced in the tables.

1629.10 Alternative Procedures.

1629.10.1 General. Alternative lateral-force procedures using rational analyses based on well-established principles of mechanics may be used in lieu of those prescribed in these provisions.

1629.10.2 Seismic isolation. Seismic isolation, energy dissipation and damping systems may be used in the design of structures when approved by the building official and when special detailing is used to provide results equivalent to those obtained by the use of conventional structural systems. For alternate design procedures on seismic isolation systems, refer to Appendix Chapter 16, Division III, and Earthquake Regulations for Seismic-isolated Structures.

SECTION 1630 MINIMUM DESIGN LATERAL FORCES AND RELATED EFFECTS

1630.1 Earthquake Loads and Modeling Requirements.

1630.1.1 Earthquake loads. Structures shall be designed for ground motion producing structural response and seismic forces in any horizontal direction. The following earthquake loads shall be used in the load combinations set forth in Section 1612:

$$E = \rho E_h + E_v \qquad (30\text{-}1)$$

$$E_m = \Omega_o E_h \qquad (30\text{-}2)$$

WHERE:

E = the earthquake load on an element of the structure resulting from the combination of the horizontal component, *Eh,* and the vertical component, *Ev.*

Eh = the earthquake load due to the base shear, *V,* as set forth in Section 1630.2 or the design lateral force, *Fp,* as set forth in Section 1632.

Em = the estimated maximum earthquake force that can be developed in the structure as set forth in Section 1630.1.1.

Ev = the load effect resulting from the vertical component of the earthquake ground motion and is equal to an addition of 0.5*CaID* to the dead load effect, *D,* for Strength Design, and may be taken as zero for Allowable Stress Design.

Ωo = the seismic force amplification factor that is required to account for structural over strength, as set forth in Section 1630.3.1.

ρ= Reliability/Redundancy Factor as given by the following formula:

$$\rho = 2 - \frac{20}{r_{max}\sqrt{A_B}} \qquad (30\text{-}3)$$

For SI: $$\rho = 2 - \frac{6.1}{r_{max}\sqrt{A_B}}$$

WHERE:

r_{max} = the maximum element-story shear ratio. For a given direction of loading, the element-story shear ratio is the ratio of the design story shear in the most heavily loaded single element divided by the total design story shear For any given Story Level *i*, the element-story shear ratio is denoted as *ri*. The maximum element-story shear ratio r_{max} is defined as the largest of the element story shear ratios, *ri*, which occurs in any of the story levels at or below the two-thirds height level of the building. For braced frames, the value of *ri* is equal to the maximum horizontal force component in a single brace element divided by the total story shear.

For moment frames, *ri* shall be taken as the maximum of the sum of the shears in any two adjacent columns in a moment frame bay divided by the story shear. For columns common to two bays with moment-resisting connections on opposite sides at Level *i* in the direction under consideration, 70 percent of the shear in that column may be used in the column shear summation.

For shear walls, *ri* shall be taken as the maximum value of the product of the wall shear multiplied by 10/*lw* (For **SI:** 3.05/*lw*) and divided by the total story shear, where *lw* is the length of the wall in feet (m).

For dual systems, *ri* shall be taken as the maximum value of *ri* as defined above considering all lateral-load-resisting elements. The lateral loads shall be distributed to elements based on relative rigidities considering the interaction of the dual system. For dual systems, the value of ? need not exceed 80 percent of the value calculated above.

ρ shall not be taken less than 1.0 and need not be greater than 1.5, and *AB* is the ground floor area of the structure in square feet (m2). For special moment-resisting frames, except when used in dual systems, ρ shall not exceed 1.25. The number of bays of special moment-resisting frames shall be increased to reduce *r,* such that ρ is less than or equal to 1.25.

EXCEPTION: *AB* may be taken as the average floor area in the upper setback portion of the building where a larger base area exists at the ground floor.

When calculating drift, or when the structure is located in Seismic Zone 0, 1 or 2, ρ shall be taken equal to 1.

The ground motion producing lateral response and design seismic forces may be assumed to act concurrently in the direction of each principal axis of the structure, except as required by Section 1633.1.

Seismic dead load, *W,* is the total dead load and applicable portions of other loads listed below.

1. In storage and warehouse occupancies, a minimum of 25 percent of the floor live load shall be applicable.

2. Where a partition load is used in the floor design, a load of not less than 10 psf (0.48 kN/m2) shall be included.

3. Design snow loads of 30 psf (1.44 kN/m2) or less need not be included. Where design snow loads exceed 30 psf (1.44 kN/m2), the design snow load shall be included, but may be reduced up to 75 percent where consideration of sitting, configuration and load duration warrant when approved by the building official.

4. Total weight of permanent equipment shall be included.

1630.1.2 Modeling requirements. The mathematical model of the physical structure shall include all elements of the lateral- force-resisting system. The model shall also include the stiffness and strength of elements, which are significant to the distribution of forces, and shall represent the spatial distribution of the mass and stiffness of the structure. In addition, the model shall comply with the following:

1. Stiffness properties of reinforced concrete and masonry elements shall consider the effects of cracked sections.

2. For steel moment frame systems, the contribution of panel zone deformations to overall story drift shall be included.

1630.1.3 *P*Δ effects. The resulting member forces and moments and the story drifts induced by $P\Delta$ effects shall be considered in the evaluation of overall structural frame stability and shall be evaluated using the forces producing the displacements of Δ_S. $P\Delta$ need not be considered when the ratio of secondary moment to primary moment does not exceed 0.10; the ratio may be evaluated for any story as the product of the total dead, floor live and snow load, as required in Section 1612, above the story times the seismic drift in that story divided by the product of the seismic shear in that story times the height of that story. In Seismic Zones 3 and 4, $P\Delta$ need not be considered when the story drift ratio does not exceed 0.02/*R.*

1630.2 Static Force Procedure.

1630.2.1 Design base shear. The total design base shear in a given direction shall be determined from the following formula:

$$V = \frac{C_v I}{RT} W \qquad (30\text{-}4)$$

The total design base shear need not exceed the following:

$$V = \frac{2.5 C_a I}{R} W \qquad (30\text{-}5)$$

The total design base shear shall not be less than the following

$$V = 0.11 C_a I W \qquad (30\text{-}6)$$

In addition, for seismic zone 4, the total base shear shall also not be less than:

$$V = \frac{0.8 Z N_v I}{R} W \qquad (30\text{-}7)$$

1630.2.2 Structure period. The value of *T* shall be determined from one of the following methods:

1. **Method A:** For all buildings, the value *T* may be approximated from the following formula:

$$T = C_t (h_n)^{3/4} \qquad (30\text{-}8)$$

Where:

C_t = 0.035 (0.0853) for steel moment-resisting frames.

C_t = 0.030 (0.0731) for reinforced concrete moment resisting frames and eccentrically braced frames.

C_t = 0.020 (0.0488) for all other buildings.

Alternatively, the value of C_t for structures with concrete or masonry shear walls may be taken as $0.1/\sqrt{A_c}$ (For SI: $0.0743/\sqrt{A_c}$ for A_c in m^2). The value of A_c shall be determined from the following formula:

$$A_c = \Sigma A_e \left[0.2 + (D_e / h_n)^2\right] \qquad (30\text{-}9)$$

The value of D_e / h_n used in formula (30-9) shall not exceed 0.9

2. Method B: The fundamental period *T* may be calculated using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The analysis shall be in accordance with the requirements of Section 1630.1.2. The value of *T* from Method B shall not exceed a value 30 percent greater than the value of *T* obtained from Method A in Seismic Zone 4, and 40 percent in Seismic Zones 1, 2 and 3.

The fundamental period *T* may be computed by using the following formula:

$$T=2\pi\sqrt{\left(\sum_{i=1}^{n} W i \delta i^{2}\right) \div \left(g \sum_{i=1}^{n} f i \delta i\right)} \qquad (30\text{-}10)$$

The values of *fi* represent any lateral force distributed approximately in accordance with the principles of Formulas (30-13), (30-14) and (30-15) or any other rational distribution. The elastic deflections, *δi*, shall be calculated using the applied lateral forces, *fi*.

1630.2.3 Simplified design base shear.

1630.2.3.1 General. Structures conforming to the requirements of Section 1629.8.2 may be designed using this procedure.

1630.2.3.2 Base shear. The total design base shear in a given direction shall be determined from the following formula:

$$V = \frac{3.0C_a}{R} W \qquad (30\text{-}11)$$

Where the value of *Ca* shall be based on Table 16-Q for the soil profile type. When the soil properties are not known in sufficient detail to determine the soil profile type, Type *SD* shall be used in Seismic Zones 3 and 4, and Type *SE* shall be used in Seismic Zones 1, 2A and 2B. In Seismic Zone 4, the Near-Source Factor, *Na*, need not be greater than 1.3 if none of the following structural irregularities are present: Type 1, 4 or 5 of Table 16-L, or Type 1 or 4 of Table 16-M.

1630.2.3.3 Vertical distribution. The forces at each level shall be calculated using the following formula:

$$F_x = \frac{3.0C_a}{R} w_i \qquad (30\text{-}12)$$

where the value of *Ca* shall be determined in Section 1630.2.3.2.
1630.2.3.4 Applicability. Sections 1630.1.2, 1630.1.3, 1630.2.1, 1630.2.2, 1630.5, 1630.9, 1630.10 and 1631 shall not apply when using the simplified procedure.

> **EXCEPTION:** For buildings with relatively flexible structural systems, the building official may require consideration of *P*Δ effects and drift in accordance with Sections 1630.1.3, 1630.9 and 1630.10. Δ*s* shall be prepared using design seismic forces from Section 1630.2.3.2.

Where used, Δ*M* shall be taken equal to 0.01 times the story height of all stories. In Section 1633.2.9, Formula (33-1) shall read:

$$F_{px} = \frac{3.0\ C_a}{R} W_{px}$$

and need not exceed 1.0 *Ca wpx*, but shall not be less than 0.5 *Ca wpx. R* and Ω*o* shall be taken from Table 16-N.

1630.3 Determination of Seismic Factors.

1630.3.1 Determination of Ω*o*. For specific elements of the structure, as specifically identified in this code, the minimum design strength shall be the product of the seismic force over strength factor Ω*o* and the design seismic forces set forth in Section 1630. For both Allowable Stress Design and Strength Design, the Seismic Force Over strength Factor, Ω*o*, shall be taken from Table 16-N.

1630.3.2 Determination of *R*. The notation *R* shall be taken from Table 16-N

1630.4 Combinations of Structural Systems.

1630.4.1 General. Where combinations of structural systems are incorporated into the same structure, the requirements of this section shall be satisfied.

1630.4.2 Vertical combinations. The value of *R* used in the design of any story shall be less than or equal to the value of *R* used in the given direction for the story above.

> **EXCEPTION:** This requirement need not be applied to a story where the dead weight above that story is less than 10 percent of the total dead weight of the structure.
>
> Structures may be designed using the procedures of this section under the following conditions:

1. The entire structure is designed using the lowest *R* of the lateral-force-resisting systems used, or

2. The following two-stage static analysis procedures may be used for structures conforming to Section 1629.8.3, Item 4.

 2.1 The flexible upper portion shall be designed as a separate structure, supported laterally by the rigid lower portion, using the appropriate values of *R* and ρ.

 2.2 The rigid lower portion shall be designed as a separate structure using the appropriate values of *R* and ρ. The reactions from the upper portion shall be those determined from the analysis of the upper portion amplified by the ratio of the *(R/*ρ) of the upper portion over *(R/*ρ) of the lower portion.

1630.4.3 Combinations along different axis. In Seismic Zones 3 and 4 where a structure has a bearing wall system in only one direction, the value of *R* used for design in the orthogonal direction shall not be greater than that used for the bearing wall system.

Any combination of bearing wall systems, building frame systems, dual systems or moment-resisting frame systems may be used to resist seismic forces in structures less than 160 feet (48 768 mm) in height. Only combinations of dual systems and special moment-resisting frames shall be used to resist seismic forces in structures exceeding 160 feet (48 768 mm) in height in Seismic Zones 3 and 4.

1630.4.4 Combinations along the same axis. For other than dual systems and shear wall-frame interactive systems in Seismic Zones 0 and 1, where a combination of different structural systems is utilized to resist lateral forces in the same direction, the value of *R* used for design in that direction shall not be greater than the least value for any of the systems utilized in that same direction.

1630.5 Vertical Distribution of Force. The total force shall be distributed over the height of the structure in conformance with Formulas (30-13), (30-14) and (30-15) in the absence of a more rigorous procedure.

$$V = F_t + \sum_{i=1}^{n} F_i \qquad (30\text{-}13)$$

The concentrated force F_t at the top, which is in addition to F_n, shall be determined from the following formula:

$$F_t = 0.07TV \quad (30\text{-}14)$$

The value of *T* used for the purpose of calculating *Ft* shall be the period that corresponds with the design base shear as computed using Formula (30-4). *Ft* need not exceed 0.25*V* and may be considered as zero where *T* is 0.7 second or less. The remaining portion of the base shear shall be distributed over the height of the structure, including Level *n,* according to the following formula:

$$F_x = \frac{(V - F_t) w_x h_x}{\sum_{i=1}^{n} w_i h_i} \quad (30\text{-}15)$$

At each level designated as *x,* the force *Fx* shall be applied over the area of the building in accordance with the mass distribution at that level. Structural displacements and design seismic forces shall be calculated as the effect of forces *Fx* and *Ft* applied at the appropriate levels above the base.

1630.6 Horizontal Distribution of Shear. The design story shear, *Vx*, in any story is the sum of the forces *Ft* and *Fx* above that story. *Vx* shall be distributed to the various elements of the vertical lateral-force-resisting system in proportion to their rigidities, considering the rigidity of the diaphragm. See Section 1633.2.4 for rigid elements that are not intended to be part of the lateral-force- resisting systems.

Where diaphragms are not flexible, the mass at each level shall be assumed to be displaced from the calculated center of mass in each direction a distance equal to 5 percent of the building dimension at that level perpendicular to the direction of the force under consideration. The effect of this displacement on the story shear distribution shall be considered.

Diaphragms shall be considered flexible for the purposes of distribution of story shear and torsional moment when the maximum lateral deformation of the diaphragm is more than two times the average story drift of the associated story. This may be determined by comparing the computed midpoint in-plane deflection of the diaphragm itself under lateral load with the story drift of adjoining vertical-resisting elements under equivalent tributary lateral load.

1630.7 Horizontal Torsional Moments. Provisions shall be made for the increased shears resulting from horizontal torsion where diaphragms are not flexible. The most severe load combination for each element shall be considered for design.

The torsional design moment at a given story shall be the moment resulting from eccentricities between applied designs lateral forces at levels above that story and the vertical-resisting elements in that story plus an accidental torsion.
The accidental torsional moment shall be determined by assuming the mass is displaced as required by Section 1630.6.

Where torsional irregularity exists, as defined in Table 16-M, the effects shall be accounted for by increasing the accidental torsion at each level by an amplification factor, *Ax*, determined from the following formula:

$$A_x = \left[\frac{\delta_{max}}{1.2\delta_{avg}}\right]^2 \qquad (30\text{-}16)$$

WHERE:

δ_{avg} = the average displacements at the extreme points of the structure at Level *x*

δ_{max} = The maximum displacement at Level *x*

The value of A_x need not exceed 3.0.

1630.8 Overturning.

1630.8.1 General. Every structure shall be designed to resist the overturning effects caused by earthquake forces specified in Section 1630.5. At any level, the overturning moments to be resisted shall be determined using those seismic forces (*Ft* and *Fx*) that act on levels above the level under consideration. At any level, the incremental changes of the design overturning moment shall be distributed to the various resisting elements in the manner prescribed in Section 1630.6. Overturning effects on every element shall be carried down to the foundation. See Sections 1615 and 1633 for combining gravity and seismic forces.

1630.8.2 Elements supporting discontinuous systems.

1630.8.2.1 General. Where any portion of the lateral-load- resisting system is discontinuous, such as for vertical irregularity Type 4 in Table 16-L or plan irregularity Type 4 in Table 16-M, concrete, masonry, steel and wood elements supporting such discontinuous systems shall have the design strength to resist the combination loads resulting from the special seismic load combinations of Section 1612.4.

EXCEPTIONS: 1. The quantity *Em* in Section 1612.4 need not exceed the maximum force that can be transferred to the element by the lateral-force-resisting system.

2. Concrete slabs supporting light-frame wood shear wall systems or light-frame steel and wood structural panel shear wall systems.

For Allowable Stress Design, the design strength may be determined using an allowable stress increase of 1.7 and a resistance factor, Φ, of 1.0. This increase shall not be combined with the one- third stress increase permitted by Section 1612.3, but may be combined with the duration of load increase permitted in Chapter 23, Division III.

1630.8.2.2 Detailing requirements in Seismic Zones 3 and 4. In Seismic Zones 3 and 4, elements supporting discontinuous systems shall meet the following detailing or member limitations:

1. Reinforced concrete or reinforced masonry elements designed primarily as axial-load members shall comply with Section 1921.4.4.5.

2. Reinforced concrete elements designed primarily as flexural members and supporting other than light-frame wood shear wall system or light-frame steel and wood structural panel shear wall systems shall comply with Sections 1921.3.2 and 1921.3.3. Strength computations for portions of slabs designed as supporting elements shall include only those portions of the slab that comply with the requirements of these sections.

2.Masonry elements designed primarily as axial-load carrying members shall comply with Sections 2106.1.12.4, Item 1, and 2108.2.6.2.6.

4. Masonry elements designed primarily as flexural members shall comply with Section 2108.2.6.2.5.

5. Steel elements designed primarily as axial-load members shall comply with Sections 2213.5.2 and 2213.5.3.

6. Steel elements designed primarily as flexural members or trusses shall have bracing for both top and bottom beam flanges or chords at the location of the support of the discontinuous system and shall comply with the requirements of Section 2213.7.1.3.

7. Wood elements designed primarily as flexural members shall be provided with lateral bracing or solid blocking at each end of the element and at the connection location(s) of the discontinuous system.

1630.8.3 At foundation. See Sections 1629.1 and 1809.4 for overturning moments to be resisted at the foundation soil interface.

1630.9 Drift. Drift or horizontal displacements of the structure shall be computed where required by this code. For both Allowable Stress Design and Strength Design, the Maximum Inelastic Response Displacement, ΔM, of the structure caused by the Design Basis Ground Motion shall be determined in accordance with this section. The drifts corresponding to the design seismic forces of Section 1630.2.1, ΔS, shall be determined in accordance with Section 1630.9.1. To determine ΔM, these drifts shall be amplified in accordance with Section 1630.9.2.

1630.9.1 Determination of ΔS. A static, elastic analysis of the lateral force-resisting system shall be prepared using the design seismic forces from Section 1630.2.1. Alternatively, dynamic analysis may be performed in accordance with Section 1631. Where Allowable Stress Design is used and where drift is being computed, the load combinations of Section 1612.2 shall be used. The mathematical model shall comply with Section 1630.1.2. The resulting deformations, denoted as ΔS, shall be determined at all critical locations in the structure. Calculated drift shall include translational and torsional deflections.

1630.9.2 Determination of ΔM. The Maximum Inelastic Response Displacement, ΔM, shall be computed as follows:

$$\Delta_M = 0.7 R \Delta_s \quad (30\text{-}17)$$

Exception: Alternatively, ΔM may be computed by nonlinear time history analysis in accordance with Section 1631.6.

The analysis used to determine the Maximum Inelastic Response Displacement ΔM shall consider $P\Delta$ effects.

1630.10 Story Drift Limitation.

1630.10.1 General. Story drifts shall be computed using the Maximum Inelastic Response Displacement, ΔM.

1630.10.2 Calculated. Calculated story drift using ΔM shall not exceed 0.025 times the story height for structures having a fundamental period of less than 0.7 second. For structures having a fundamental period of 0.7 second or greater, the calculated story drift shall not exceed 0.020 times the story height.

EXCEPTIONS: 1. These drift limits may be exceeded when it is demonstrated that greater drift can be tolerated by both structural elements and nonstructural elements that could affect life safety. The drift used in this assessment shall be based upon the Maximum Inelastic Response Displacement, ΔM.

2. There shall be no drift limit in single-story steel-framed structures classified as Groups B, F and S Occupancies or Group H, Division 4 or 5 Occupancies. In Groups B, F and S Occupancies, the primary use shall be limited to storage, factories or workshops. Minor accessory uses shall be allowed in accordance with the provisions of Section 302. Structures on which this exception is used shall not have equipment attached to the structural frame or shall have such equipment detailed to accommodate the additional drift. Walls that are laterally supported by the steel frame shall be designed to accommodate the drift in accordance with Section 1633.2.4.

1630.10.3 Limitations. The design lateral forces used to determine the calculated drift may disregard the limitations of Formula (30-6) and may be based on the period determined from Formula (30-10) neglecting the 30 or 40 percent limitations of Section 1630.2.2, Item 2.

1630.11 Vertical Component. The following requirements apply in Seismic Zones 3 and 4 only. Horizontal cantilever components shall be designed for a net upward force of $0.7CaIWp$.

In addition to all other applicable load combinations, horizontal pre-stressed components shall be designed using not more than 50 percent of the dead load for the gravity load, alone or in combination with the lateral force effects.

SECTION 1631 - DYNAMIC ANALYSIS PROCEDURES

1631.1 General. Dynamic analyses procedures, when used, shall conform to the criteria established in this section. The analysis shall be based on an appropriate ground motion representation and shall be performed using accepted principles of dynamics. Structures that are designed in accordance with this section shall comply with all other applicable requirements of these provisions.

1631.2 Ground Motion. The ground motion representation shall, as a minimum, be one having a 10-percent probability of being exceeded in 50 years, shall not be reduced by the quantity R and may be one of the following:

1. An elastic design response spectrum constructed in accordance with Figure 16-3, using the values of Ca and Cv consistent with the specific site. The

design acceleration ordinates shall be multiplied by the acceleration of gravity, 386.4 in./sec.2 (9.815 m/sec.2).

2. A site-specific elastic design response spectrum based on the geologic, tectonic, and seismologic and soil characteristics associated with the specific site. The spectrum shall be developed for a damping ratio of 0.05, unless a different value is shown to be consistent with the anticipated structural behavior at the intensity of shaking established for the site.

3. Ground motion time histories developed for the specific site shall be representative of actual earthquake motions. Response spectra from time histories, either individually or in combination, shall approximate the site design spectrum conforming to Section 1631.2, Item 2.

4. For structures on Soil Profile Type *SF*, the following requirements shall apply when required by Section 1629.8.4,
Item 4:

4.1 The ground motion representation shall be developed in accordance with items 2 and 3.

4.2 Possible amplification of building response due to the effects of soil-structure interaction and lengthening of building period caused by inelastic behavior shall be considered.

5. The vertical component of ground motion may be defined by scaling corresponding horizontal accelerations by a factor of two- thirds. Alternative factors may be used when substantiated by site- specific data. Where the Near Source Factor, *Na,* is greater than 1.0, site-specific vertical response spectra shall be used in lieu of the factor of two-thirds.

1631.3 Mathematical Model. A mathematical model of the physical structure shall represent the spatial distribution of the mass and stiffness of the structure to an extent that is adequate for the calculation of the significant features of its dynamic response. A three-dimensional model shall be used for the dynamic analysis of structures with highly irregular plan configurations such as those having a plan irregularity defined in Table 16-M and having a rigid or semi rigid diaphragm. The stiffness properties used in the analysis and general mathematical modeling shall be in accordance with Section 1630.1.2.

1631.4 Description of Analysis Procedures.

1631.4.1 Response spectrum analysis. An elastic dynamic analysis of a structure utilizing the peak dynamic response of all modes having a significant contribution to total structural response. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve, which correspond to the modal periods. Maximum modal contributions are combined in a statistical manner to obtain an approximate total structural response.

1631.4.2 Time-history analysis. An analysis of the dynamic response of a structure at each increment of time when the base is subjected to a specific ground motion time history.

1631.5 Response Spectrum Analysis.

1631.5.1 Response spectrum representation and interpretation of results. The ground motion representation shall be in accordance with Section 1631.2. The corresponding response parameters, including forces, moments and displacements, shall be denoted as Elastic Response Parameters. Elastic Response Parameters may be reduced in accordance with Section 1631.5.4.

1631.5.2 Number of modes. The requirement of Section 1631.4.1 that all significant modes be included may be satisfied by demonstrating that for the modes considered, at least 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction.

1631.5.3 Combining modes. The peak member forces, displacements, story forces, story shears and base reactions for each mode shall be combined by recognized methods. When three- dimensional models are used for analysis, modal interaction effects shall be considered when combining modal maxima.

1631.5.4 Reduction of Elastic Response Parameters for design. Elastic Response Parameters may be reduced for purposes of design in accordance with the following items, with the limitation that in no case shall the Elastic Response Parameters be reduced such that the corresponding design base shear is less than the Elastic Response Base Shear divided by the value of *R.*

1. For all regular structures where the ground motion representation complies with Section 1631.2, Item 1, and Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 90 percent of the base shear determined in accordance with Section 1630.2.

2. For all regular structures where the ground motion representation complies with Section 1631.2, Item 2, and Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 80 percent of the base shear determined in accordance with Section 1630.2.

3. For all irregular structures, regardless of the ground motion representation, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 100 percent of the base shear determined in accordance with Section 1630.2.

The corresponding reduced design seismic forces shall be used for design in accordance with Section 1612.

1631.5.5 Directional effects. Directional effects for horizontal ground motion shall conform to the requirements of Section 1630.1. The effects of vertical ground motions on horizontal cantilevers and pre-stressed elements shall be considered in accordance with Section 1630.11. Alternately, vertical seismic response may be determined by dynamic response methods; in no case shall the response used for design be less than that obtained by the static method.

1631.5.6 Torsion. The analysis shall account for torsional effects, including accidental torsional effects as prescribed in Section 1630.7. Where three-dimensional models are used for analysis, effects of accidental torsion shall be accounted for by appropriate adjustments in the model such as adjustment of mass locations, or by equivalent static procedures such as provided in Section 1630.6.

1631.5.7 Dual systems. Where the lateral forces are resisted by a dual system as defined in Section 1629.6.5, the combined system shall be capable of resisting the base shear determined in accordance with this section. The moment-resisting frame shall conform to Section 1629.6.5, Item 2, and may be analyzed using either the procedures of Section 1630.5 or those of Section 1631.5.

1631.6 Time-history Analysis.

1631.6.1 Time history. Time-history analysis shall be performed with pairs of appropriate horizontal ground-motion time- history components that shall be selected and scaled from not less than three recorded events. Appropriate time histories shall have magnitudes, fault distances and source mechanisms that are consistent with those that control the design-basis earthquake (or maximum capable earthquake). Where three appropriate recorded ground-motion time-history pairs are not available, appropriate simulated ground-motion time-history pairs may be used to make up the total number required. For each pair of horizontal ground- motion components, the square root of the sum of the squares (SRSS) of the 5 percent-damped site-specific spectrums of the scaled horizontal components shall be constructed. The motions shall be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5 percent-damped spectrum of the design-basis earthquake for periods from 0.2*T* second to 1.5*T* seconds. Each pair of time histories shall be applied simultaneously to the model considering torsional effects. The parameter of interest shall be calculated for each time- history analysis. If three time-history analyses are performed, then the maximum response of the parameter of interest shall be used for design. If seven or more time-history analyses are performed, then the average value of the response parameter of interest may be used for design.

1631.6.2 Elastic time-history analysis. Elastic time history shall conform to Sections 1631.1, 1631.2, 1631.3, 1631.5.2, 1631.5.4, 1631.5.5, 1631.5.6, 1631.5.7 and 1631.6.1. Response parameters from elastic time-history analysis shall be denoted as Elastic Response Parameters. All elements shall be designed using Strength Design. Elastic Response Parameters may be scaled in accordance with Section 1631.5.4.

1631.6.3 Nonlinear time-history analysis.

1631.6.3.1 Nonlinear time history. Nonlinear time-history analysis shall meet the requirements of Section 1629.10, and time histories shall be developed and results determined in accordance with the requirements of Section 1631.6.1. Capacities and characteristics of nonlinear elements shall be modeled consistent with test data or substantiated analysis, considering the Importance Factor. The maximum inelastic response displacement shall not be reduced and shall comply with Section 1630.10.

1631.6.3.2 Design review. When nonlinear time-history analysis is used to justify a structural design, a design review of the lateral- force-resisting system shall be performed by an independent engineering team, including persons licensed in the appropriate disciplines and experienced in seismic analysis methods. The lateral-force-resisting system design review shall include, but not be limited to, the following:

1. Reviewing the development of site-specific spectra and ground-motion time histories.

2. Reviewing the preliminary design of the lateral-force-resisting system.

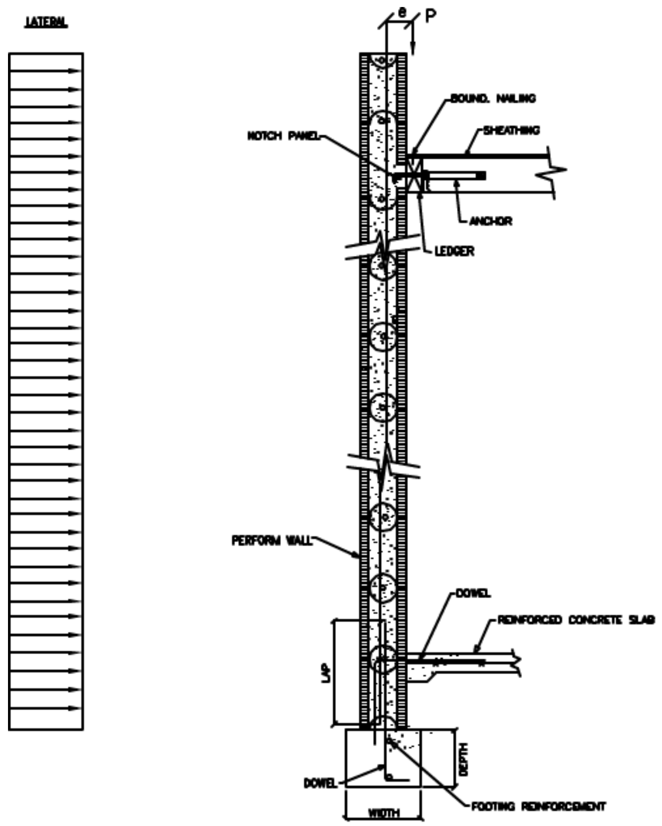
3. Reviewing the final design of the lateral-force-resisting system and all supporting analyses.

The engineer of record shall submit with the plans and calculations a statement by all members of the engineering team doing the review stating that the above review has been performed.

SECTION 3

Design Procedures

3.0 PERFORM WALL™ WALL SYSTEMS



The Perform Wall™ wall system is a concrete forming system which, produces a continuous insulate monolithic concrete wall which requires no stripping and is ready to receive finish materials. The Perform Wall™ form produces a grid shaped concrete wall with vertical concrete cores 15" inches on center and horizontal concrete cores 15" inches on center.

This creates a monolithic reinforced concrete wall. This forming system optimizes the use of concrete and insulating materials to produce the finished wall. Because of the unique shape of the resulting concrete wall, requirements for minimum reinforcing steel must be met. This is necessary to maintain the structural integrity of the system throughout its useful life. If the wall is to be subjected to design loads or longer spans than those required for minimum reinforcement, structural calculations must be performed by a qualified registered engineer to determine the required steel reinforcing for the design conditions.

This concrete system, if properly designed and reinforced, will perform adequately for walls with axial and lateral loads; columns to carry concentrated loads, especially on the sides of openings; lintels and beams carrying gravity as well as lateral loads, elevated slabs with reinforcement or post tensioned slab carrying gravity as well as lateral loads; and shear walls to resist in-plane shear loads from shear diaphragms or other shear elements. This Manual provides design information methods for the proper design of a Perform Wall™ wall system.

3.1 MINIMUM RENFORCEMENT REQUIREMENT

The Perform Wall™ wall system requires the use of reinforcement to meet the requirements of the American Concrete Institute's Building Code Requirements for Reinforced Concrete (ACI 318) or Uniform Building Code chapter 19 or SBCCI code. The following recommendations conform to the requirements of these Codes with the exception of ACI Section 14.3.5 or UBC section 19.14.3.5, which requires a maximum spacing of reinforcing of 18 inches in both the horizontal and vertical directions. This criteria is usually not required to be met in most residential, light commercial and light industrial applications.

The following is the minimum reinforcement requirement for the wall System in seismic zone 3 and 4, and for wind speed greater than 70 MPH; there must be one #4 (1/2") bar placed in every vertical core (15" inch on centers) and one #4 bar placed in every horizontal core (15 on inch centers), with a minimum of 3 bars in the wall. Calculation and tables provided in this Manual are for common load applications.

3.2 RETAINING WALL

The inclusion of a table for backfill for an equivalent fluid pressure of 30 psf/ft of depth is included as this value is sometimes used for local practice. However, this lower value of equivalent fluid pressure is generally associated with retaining walls, whose top is free to deflect and mobilize the shear strength of the soil, generally referred to as "active" pressure.

3.3 FOR BASEMENT WALLS

The top is restrained from movement and "at rest" pressure should be used. A equivalent fluid pressure of 50 psf/ft. of depth, which is the minimum for well-drained uncompacted granular material placed next to the wall, and is the recommended value to use for basement wall applications.

3.4 LOAD BEARING WALLS (LOAD CAPACITIES OF AXIAL AND OUT OF PLANE LOADS)

With a minimum of 2,500 psi concrete and Grade 40 reinforcing, these minimum-reinforcing requirements will carry the following services loads:

3.4.1 8 1/2 inch panel:

3.4.1.1 Wall axial loads up to 2,500 pounds per lineal foot (plf) for an 8-foot height with no lateral loads (ACI Equation 14-1).
3.4.1.2 Lateral Wind Loads to 20 pounds per square foot for walls up to 10 feet high
3.4.1.3 No Backfill is allowed for the 8 1/2" thick Perform Wall™ wall system.

3.4.2 10, 12 and 14 inch panel

3.4.2.1 Wall axial loads up to 10,000 pounds per lineal foot (plf) for a 16-foot height with no lateral loads (ACI Equation 14.1)
3.4.2.2 Lateral Wind Loads to 40 pounds per square foot for walls up to 16 feet high.
3.4.2.3 Backfill up to 5.5 feet on walls 8 feet high or an equivalent fluid pressure of 40 pounds per cubic foot.

3.4.2.4 Cantilever retaining walls up to 5 feet for an equivalent fluid pressure of 30 pounds per cubic foot.

3.4.3 CALCULATION METHODS
Standard engineering methods are used to determine the structural capacities of the concrete wall system. Load combinations and load factors conforming to the ACI 318-95 Building Code or 1997 UBC Chapter 19 requirements are used to determine the design requirements of the wall. The UBC requirements are identical to the ACI requirements except for additional provisions in 1918.8 Alternate Design Slender Walls, usually defined as the Slender Wall Method.

The following sections include drawings, sample calculation worksheets, design tables to aid in the calculation procedure, and uniaxial interaction curves for combinations of axial and moment loads.

A thorough application of the following steps will result in the safe and economical design of a Perform Wall™ wall system. Although this may seem a lengthy procedure, several steps will be an obvious necessity, no matter what system is used, and after a couple of applications, it can be accomplished relatively easily and quickly. Trilogy Materials, Ltd. (holding company of Perform Wall™, L.L.C.), is presently developing computer software to perform the more tedious calculations and many of these have been reduced to design charts and diagrams presented in this Manual. The references are to the Building Code Requirements for Reinforced Concrete, ACI 318-95 & UBC 1997.

3.4.3.1 Determine the structural system, which will bear on and / or support the wall. This would include floor and roof members.

3.4.3.2 Identify the spans which will bear on walls and which walls will be non-load bearing. Also identify shear walls, which will provide reactions to floors and roof diaphragms to resist lateral loads.

3.4.3.3 Determine the design loads for the floors and roof members bearing on the walls and lateral loads on the walls due to wind, seismic, or soil backfill pressures. Use the local building code requirements for minimum values.

3.4.3.4 Sketch attachment details for connection of floor and roof members to the wall system. This will identify off center eccentricities for connections, which are necessary for a correct analysis of the loads transferred to the wall. Typically this would be ledger systems for floors attached to the side of the system. For example, wood ledgers bolted to a 10 inch panel wall supports joists. If a 4x wood member is used for the ledger, the resulting eccentricity for the floor load for a 10" panel wall would be 3.5"/2 + 10"/2 =6.75". This will introduce a moment into the wall which must be designed for. This condition must be conducive with other loads, which also produce moments in the wall. Often this condition will produce counterbalancing moments to soil backfill pressures or wind pressure. Wind suction and seismic loads will generally combine with floor or eccentric loads to produce additive moments.

3.4.3.5 Determine the design load for the wall from the floor, roof, and lateral loads identified above. This is usually the multiplication of contributory span lengths, times the unit load on the floor or roof members. This must be separated into dead load, live load, and short-term load because of the ACI 318 & UBC required load combinations (ACI Section 9.2, Equations 9-1,9-2 and 9-3 and Chapter 19 of UBC). These loads must be multiplied by their respective eccentricities to determine design moments on the wall. Be careful of sign convention to make sure correct moments are added or subtracted. Include the weight of the wall, calculated using 42 psf for the 81/2 inch wall, 57 psf for the 10 inch wall, 60 psf for the 12 inch wall and 64 psf for the 14" inch wall.

3.4.3.6 Determine the lateral design loads for the wall. This would generally include wind, seismic, and soil backfill pressures. Be sure to include any lateral pressures from a surcharge on the backfill such as a parking lot next to the wall. Typically wind and seismic loads are modeled as uniform loads and soil pressures are modeled as triangular or trapezoidal loads. Trapezoidal varying loads are usually broken into combination of triangular and uniform load.

3.4.3.7 Determine the shear and moments induced in the wall from the lateral loads. Again, the different live, dead, and short-term components should be kept separated for differing combinations and load factors.

3.4.3.8 Multiply the various loads by the appropriate load factors and add together in the following. The ultimate load combinations "U" is the largest of the following equations:

$$U=1.4D+1.7L \qquad (9\text{-}1)$$
$$U=.76\,(1.4D+1.7L+1.7W) \qquad (9\text{-}2)$$
$$U=.9D+1.3W \qquad (9\text{-}3)$$

WHERE:

U= ultimate design load, axial, moment and shear
D= dead load
L= live load
W=wind load (substitute 1.1E for seismic load).

The combinations above must be checked for both:

1. Maximum axial load with the corresponding moment for the same load case; and

2. The maximum moment with the corresponding axial load. Along with each of the values for axial load and moment, the corresponding maximum shear values should be determined.

3.4.3.9 We suggest a check of the wall thickness for shear at this point. The wall may be capable of being reinforced to resist flexural and axial load but must be designed to resist the shear induced by the lateral loads. The nominal shear capacity of the wall is determined by Equation 11-3 of the ACI Code, Section 11.3. The shear capacity is per cell, which has vertical and horizontal reinforcing in it.

The capacities can be increased by applying ACI Equation 11-4 to account for the increase in shear capacity due to axial load, however the increase for most applications is so small it does not justify the extra calculation effort.

3.4.10 At this stage in the calculations, a check should be made for conformance to ACI Section 10.11.5.4 for minimum eccentricities. Although this Code provision is intended for columns and not required for walls, a conservative design would consider minimum eccentricities, especially under high axial loads. Eccentricity is the distance off the centroid of the section that the axial load is applied. The Code specifies a minimum eccentricity of (0.6" + 0.03T) inches. For the Perform Wall™ wall system this is 0.77" inches for the 8 1/2 inch wall, and .79" inches for the nominal 10, 12 and 14 inch wall. Compute M_U/P_U being careful to use consistent units. This will give you the equivalent eccentricity to be compared to the Code minimum. If the design eccentricity are less than the Code minimums, use design moments of $0.75P_U$ for the 10", 12, 14" inch walls and 0 $.81P_U$ for the 8 1/2 inch wall (units of inch-kips or inch-pounds). Determine the two load cases to be checked:

(1) Maximum P_U and the corresponding M_U
(2) Maximum M_U and the corresponding P_U

If the axial loads are relatively light with the resultant of all factored loads located within the middle-third of the wall (maximum eccentricity = 0.833 inch for the 6" core and 1.667 for the 8" core), ACI Section 14.5 Empirical design method applies and Equation 14-1 may be used.

3.4.3.11 MOMENT MAGNIFIER METHOD
A check for slenderness is generally necessary because of the shape of the wall system. The Equivalent Core thickness may be used to calculate structural properties. In calculations, r = 0.30h is used, as it results in a more conservative value. This results in a radius of gyration, r = 1.44 inches for the 8 1/2 inch thick wall, and r = 2.02 inches for the 10, 12 and 14 inch thick wall. Another conservative assumption made, is that the walls are designed as simple spans vertically, with no end moments. Requirements of ACI section 10.11.4 then reduces to considering the slenderness only when Klu/r is greater than 34 because the end moments M_{1b} and M_{2b} are assumed zero. Usually the only time this is not a conservative assumption is in a continuous two story wall with soil pressure pushing inward on the lower level with wind suction or seismic loads pulling outward on the upper level resulting in reverse curvature of the wall. In these cases, a two span continuous analysis should be analyze.

To meet the requirements of ACI 10.11.4 and 10.11.5 using the assumptions stated above, slenderness must be considered when the unsupported height of the wall, L_u is greater than 49 inches for a 6 inch core wall, and 68.7 inches for an 8 inch core wall.

Although this appears to be a complicated procedure from the Code equations, much of the procedure is greatly simplified because a wall has to be checked for slenderness only in one direction and is generally always braced against sideways. The following procedure is used:

1. Determine the ratio of maximum factored axial dead load to the maximum total factored axial load. This value I. βd in Chapter 10 of the ACI Code.

2. Substitute the value for βd in Equation 10-13 of the Code to find the equivalent stiffness of the wall section.

3. Substitute the value for EI in Equation 10-9 of the Code to find the critical buckling load.

4. Substitute the value for P in Equation 10-11 of the Code to find the moment magnifier of the design moment M_u (Cm =1).

5. Substitute the value for ςb in Equation 10-6 of the Code to find the value of the magnified design moment Mc.

The second term of Equation 10-6 is generally zero because most walls are braced against side sway. For various concrete strengths, factored dead load to total load ratios (DL/TL = βd), and wall heights. Because of the large number of possible combinations of possible design axial loads and wall heights, a tabulation of moment magnifier coefficients would result in many pages of tables. This tabulation is much more limited but the designer has to substitute only one value from the tables into ACI Equation 10-7 to determine the moment magnifier. This procedure greatly simplifies the determination of the moment magnifier.

3.4.3.12 Determine the amount reinforcement steel necessary to resist the two design load cases for axial load and moment. Note that each load case will have a different moment magnifier. Generally, if the reinforcing is selected on the basis of the maximum magnified moment, the corresponding axial load capacity is adequate for the typical application of the wall system. There are three basic ways to determine the required reinforcing steel. If the amount of reinforcing is selected on the basis of the design moment, the load cased should be checked by plotting on the interaction diagram for the selected wall thickness and steel reinforcing size and position.

The three methods are:

1. Substitute the maximum ultimate design moment M_u in the following equation to determine the coefficient of resistance, R_u:

$$R_v = \frac{M_U}{bd^2}$$

where:
b = 4.0" for 8 ½" Wall.
b = 4.75" for 10", 12" and 14" Wall.

d = 2.0" for 8 1/2" walls with Reinforcement placed in the center of wall, and
d = 2.5" for 8 1/2" walls with Reinforcement placed at the edge of the wall system.
d = 2.2" for 10", 12" and 14" walls (center), and
d = 3.2" for 10", 12" and 14" walls (at edge)
and
φ = .9
Substitute the value for R_u in the following equation for the required *"%* " of steel:

$$R_v = \rho \cdot f_Y \cdot (1 - .59 \cdot \rho \cdot m)$$

where:

$$m = \frac{f_y}{0.85 \cdot f'_c}$$

and:

f_c = concrete strength, and

f_y = yield strength of reinforcing steel (40 ksi for Grade 40 and 60 ksi for Grade 60)

Then the required area of steel is:

$$A_S = \rho \cdot b \cdot d$$

Obtain the allowable moment from the Axial Load Interaction Diagram for the closest selected area of steel. Plot the two load cases to make sure the combination of axial load and moment fall within the curve.

2. Choose a reinforcing bar size, determine the flexural capacity and then compare to see if it exceeds the load induced design moment, revising and recalculating if necessary.

The nominal section capacity equation is, $\varphi = 0.9$ and other variables as defined in the above. Select Moment-Axial load interaction diagram for the selected area of steel plot on the two load cases to make sure the combination of axial load and moment fall within the curve.

3. Select the Moment-Axial Interaction Diagram in which both load cases plot inside of the interaction curve. Use concrete strength and reinforcing as specified for curve.

Other geometries and material properties are available upon request. The Manual does not include tables for reinforcing off center for the 8 ½" Perform Wall™ system. Double reinforcement shall be used for 10", 12" and 14" Perform Wall™ system, which has a minimum of 5 3/4" "d" distance and core. And the special order, 8" diameter core.

Note the limit of application of the Moment Magnifier method is slenderness ratio, which is limited to 100 (ACI 10.11.4.3). Along with this slenderness ratio, the Code effectively limits the maximum ultimate axial load to 70% of the critical buckling load PC as you get a negative Moment Magnifier. Stability is critical when the axial load exceeds 50% of PC because the Moment Magnifier increases rapidly. Stability failure occurs at 70% of PC because the Moment Magnifier gets very large as it approaches 70% and then goes negative.

3.5 Alternate Design Slender Walls

1 When flexural tension control design of walls, the requirements of section 1910.10 of UBC Code may be satisfied by complying with the limitations and procedures set forth in this section (see figure 1).

2 The following limitations apply when this section is employed:

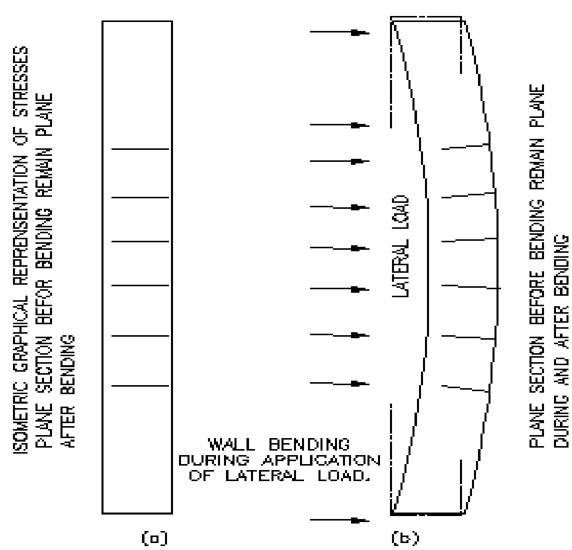


Figure1.

a. Vertical service load stress at the location of maximum horizontal stress.

b. Maximum moment does not exceed 0.04 f'_c.

c. The reinforcement ratio ρ does not exceed 0.6 ρ_b.

d. Sufficient reinforcement is provided so that the nominal moment capacity times the φ factor is greater than M_{cr}.

e. Distribution of concentrated load does not exceed the width of bearing plus a width increasing at a slope of 2 vertical to 1 horizontal down to the design flexural section.

3 The required factor moment, M_u at the midheight cross section for combined axial and lateral factored loads, including the PΔ moments, shall be as set forth in formula (14-2).

$$Mu \leq \varphi M_n \quad (14\text{-}2)$$

Unless a more comprehensive analysis is used, the PΔ moment shall be calculated using the maximum potential deflection, Δ_n as define in section 1914.8.4 of UBC Code.

4 The midheight deflection Δ_s under service lateral and vertical loads (without load factors), shall be limited by the relation.

$$\Delta_s = \frac{I_c}{150}$$

Unless a more comprehensive analysis is used, the midheight deflection shall be computed with the following formulas:

$$\Delta_s = \Delta_{cr} + \frac{M_S - M_{cr}}{M_N - M_{cr}} \cdot \left(\Delta_n - \Delta_{cr}\right)$$

for $M_s > M_{cr}$

$$\Delta_s = \frac{5 \cdot M_S \cdot I_c^2}{48 \cdot E_c \cdot I_g}$$

for $M_s < M_{cr}$

$$I_{cr} = \left[n \cdot A_{se} \cdot (d-c)^2\right] + \frac{s_{eq} \cdot c^3}{3} \qquad A_{se} = \frac{\left(P_u + A_s \cdot f_y\right)}{f_y}$$

$$\Delta_n = \frac{5 \cdot M_n \cdot I_c^2}{48 \cdot E_c \cdot I_{cr}} \qquad \Delta_n = \frac{5 \cdot M_n \cdot I_c^2}{48 \cdot E_c \cdot I_{cr}}$$

Where:

M_s = the maximum moment in the wall resulting from the application of the unfactored load combinations.

3.6 PERFORM WALL™ SHEAR WALL PERFORMANCE AND ANALYSIS.

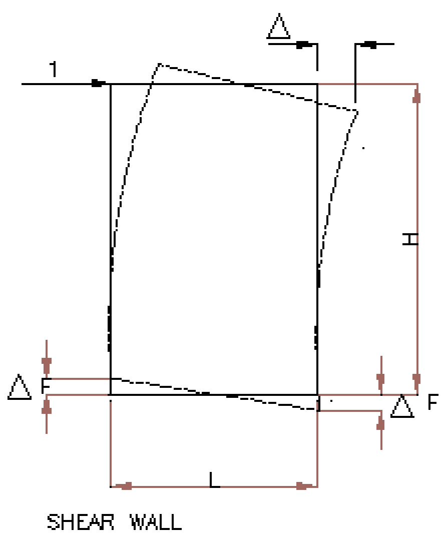


Figure 2.

Perform Wall™ shear wall response to earthquakes, if stressed enough to cause damage, can be described by the following damage pattern shown in yielding in flexure, rocking or uplifting of the foundation and sliding along a

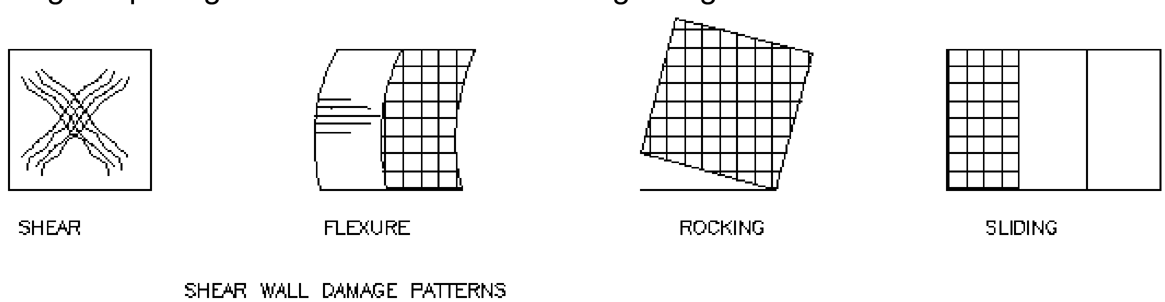


Figure 3.

construction joint. Each pattern of damage or cracking can be minimized by proper reinforcing: horizontal reinforcing for shear, flexible or jamb reinforcing at the end of the wall for flexure; anchorage of the flexure reinforcing into an adequate foundation to prevent uplift; and dowels and proper roughened surfaces across the construction joints to prevent sliding. Yielding and cracking is a mechanism to dissipate the seismic energy, which has been imparted into the structure. If the energy is successfully dissipated, the structure will be stable and only nominally cracked or yielded. If not, the structure may collapse or be severely damaged. Perform Wall™ walls that fail in shear and develop diagonal shear cracks have very limited ductility capacity. Ductility as used here is measured as the deflection of the wall at failure divided by the deflection at yield. Increasing the horizontal shear reinforcing does have a moderate effect on increasing the ductility capacity to a range of 1 to 3. However, Perform Wall™ walls with a flexural mode of response have a ductility capacity significantly greater than walls with a shear mode of response, in the range of 3 to 6 as shown by tests in the laboratory. Ductilities higher than 4 can be developed provided a sudden compression failure is avoided at the toe of the wall and the jamb is properly reinforced, so that the Perform Wall™ is properly confined.

The preference is to design walls such that the damage mechanism will be flexural yielding rather than shear yielding thus giving the wall more ductility capacity. Multi-story walls with openings should also be designed with the intent to provide the maximum ductility capacity. Damage to the vertical elements in a wall as shown in this figure may affect the stability of the structure and cause an early mode of collapse. Designing the vertical elements to be stronger than the spandrels will force the damage into the spandrels, which can absorb energy but not threaten the stability of the wall. This is called the strong column-weak beam concept. Even for this wall with openings, in which yielding is preferred over the shear yielding shown in figures 4 a and 4 b.

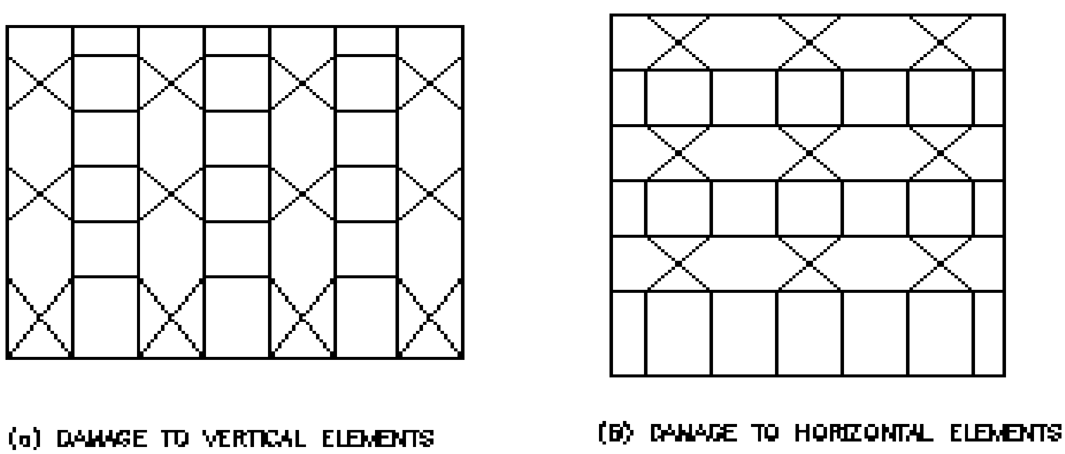


Figure 4.

3.7 SHEAR WALL BUILDING CONFIGURATION

One of the most important aspects of shear wall and seismic design are qualitative elements regarding building configuration: symmetry and location of resisting shear walls, relative deflections, anchorage, and discontinuities. The significant failures observed in past earthquakes usually have occurred in buildings with significant irregularities. Configuration problems can be classified as three basic types:

1. Vertical discontinuities (soft stories, discontinuous or offset shear walls, etc.)

2. Plan irregularities (diaphragms with discontinuous or offsets, unbalanced resistance especially poor distribution of shear walls, etc).

3. Detail problems (strong beam-weak column, lack of drags and chords, etc.)

Through shear and flexural resistance, shear walls resist horizontal forces acting parallel to the plane of the wall. The location of these walls in relation to the direction of the applied force is critical. Since ground motions may occur in perpendicular directions, the location of resisting elements must coincide with these forces. The building shear walls should be symmetrical to eliminate torsional motion. In building design the shear wall layout is very important in order to produce good performance in a seismic event. The following examples are of different design scenarios under earthquake forces:

1. The shear resisting elements are at one corner of the building. In this case even though the shear walls may be of ample strength for shear design, because of the eccentricity of the shear walls in relation to the structure, the building will tend to rotate and cause significant damage to torsion.

2. The shear resistance of the structure is of ample strength; however, because of the location in relation to the structure, has no torsional resistance.

3. The shear walls are at the perimeter with different rigidities. This is preferable for shear design, but because of the different rigidities do not have overall torsional rigidity.

4. The shear walls are at the perimeter and the rigidity of the walls are the same. This is the most desirable situation for the structural design, because they can provide both shear and torsional resistance.

The discontinuous shear walls as shown in the following figure (5a and 5b), create an earthquake lateral force demand on the soft story for energy dissipation, which cannot be easily satisfied. Strengthening the supporting columns alone will not provide enough capacity to solve this problem. All the shear forces in wall (a) must be transferred to another adjacent wall and the overturning forces must be supported by the columns. The column design forces are amplified to provide stability in a major earthquake by increasing the design forces by 3(Rw/8) as specified in UBC. The shear wall with large openings at the base is considered to have soft story due to the loss of a large percentage of the wall. The wall could be thickened at the first story to improve the situation and the isolated column should also be designed for amplified forces. The best design procedure is to avoid discontinuous walls.

Intersecting walls also potentially suffer more damage in earthquakes. Since earthquakes occur in both directions simultaneously, intersecting walls will both be stressed to some degree at the same time. The overturning effects may be accumulative at the intersecting corner. Thus, these effects should be combined by designing for 100 percent of the seismic forces in one direction plus 30 percent of the forces in the perpendicular direction as per UBC.

3.8 RIGIDITY OF SHEAR WALLS

Horizontal forces due to lateral loads will be distributed to individuals shear walls in an assemblage of shear walls based on the relative stiffness of the walls. A stiff wall will resist a proportionally larger force to cause the same deflection as a flexible wall. Therefore, any lateral forces will be distributed proportionally to relative stiffness is typically measured as the rigidity of wall, which is defined
as:

$$R = \frac{1}{\Delta}$$

where Δ is the deflection of the wall due to a unit force. A shear wall's span is characterized as a deep beam, where the span is only several times the depth at the most. Consider the shear wall shown in Figure 5. The wall deforms because of bending and shear and foundation rotation. Because it is difficult to determine the exact stiffness of the foundations, the deflection due to foundation is ignored, therefore The deflection is:

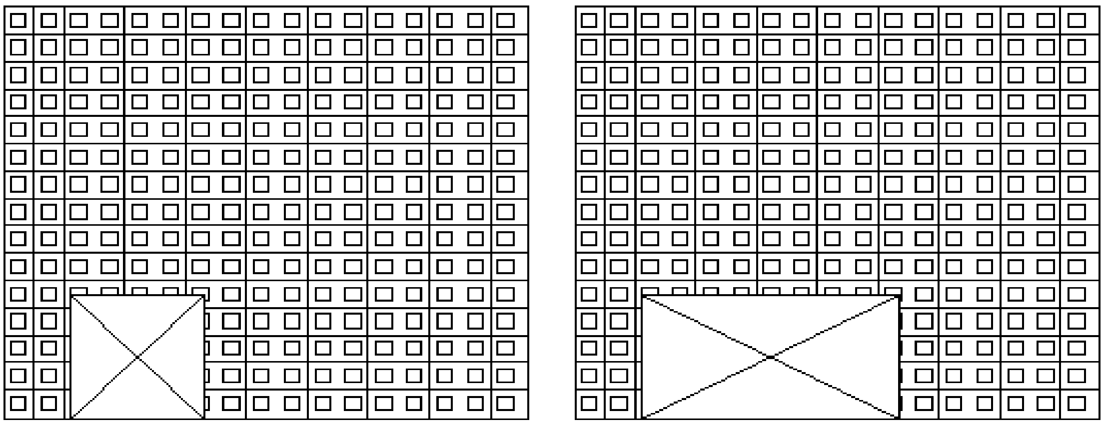
$$\text{Moment}\cdots\Delta = \frac{h^3}{3\cdot E\cdot I} + \frac{1.2\cdot h}{A\cdot G}\cdots\text{Shear}$$

The first term represents the influence of bending, the second term considers shear. The bending term considers the wall as a simple vertical cantilever beam with a moment of inertia, I, which includes returns or pilasters at the ends of the wall. The shear-deformation term contains the constant 1.2 which is for rectangular cross sections and which decreases for walls with flanges.

The area of the wall, A, is the area of the web of the wall and omits the flange areas. E and G are the Young modules and shear modules, respectively. For Perform Wall™, these terms are:

Em = 750 f' m

Gm - 0.4 Em



a Figure 5 b

The foundation rotation term is typically ignored in most design situations even though its contribution may be significant. Foundation distortions and the resulting wall deflection can be as large as the shear or bending effects since foundations in many situations are relatively flexible when compared to the very rigid nature of the wall itself. Difficulties in evaluating the stiffness of various foundation systems also restricts the designer's ability to include the foundation rotation term.

Since only relative rigidities are desired, relative deflection can also be determined. If the walls are all of the same strength, then the deflection equation can be simplified to:

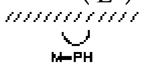
$$\Delta = \frac{h^3}{3 \cdot I} + \frac{3 \cdot h}{A}$$

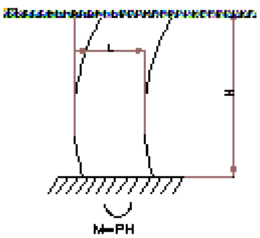
and the relative rigidity is:

$$R = \frac{1}{b \cdot E \cdot \Delta}$$

If all the walls are the same thickness, the deflection equation can be further simplified for cantilever walls deflection to:

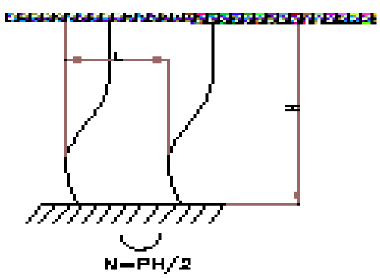
$$\Delta = 4 \cdot \left(\frac{h^3}{L}\right) + 3 \cdot \left(\frac{h}{L}\right)$$





For fixed walls or piers to:

$$\Delta = \left(\frac{h^3}{L}\right) + 3 \cdot \left(\frac{h}{L}\right)$$



the relative rigidity is:

$$R = \frac{1}{E \cdot \Delta}$$

The distribution of the lateral force to an assemblage of walls some shear walls relies on the ability of the drag members to force all the walls to deflect the same and, thus, share the force based on their relative rigidity. The drag members also collect and distribute the shear from the diaphragm, noting that only a portion of the diaphragm transfers directly to the wall. The force in the pier A is:

$$F_A = \frac{R_A}{\sum_{i=1}^{n} R}$$

Design of shear walls requires consideration of both rigidity and strength. If the lateral loads and unit stresses they produce are small. The resisting elements are essentially uncracked and the deformations can be calculated with reasonable accuracy by elementary theory. As the loads increase, the structure develops cracks and the response becomes nonlinear. The point at which cracking begins is difficult to predict. Cracking is a complex process, with great variations in degree. Once a crack is formed, reinforcing steel across the crack tends to limit the opening of crack, but the rigidity of the wall changes as the section changes from an uncracked to a cracked section. Rigidity of walls and, thus, the distribution of lateral forces will not be constant or completely predictable in a major earthquake that causes cracking in the walls. A good design utilizing walls of similar rigidity placed in a symmetrical pattern has a more predictable seismic behavior even if there is a degree of cracking expected in a major earthquake.

3.9 DESIGN OF THE PERFORM WALL™ SHEAR WALLS

Perform Wall™ shear wall shall be designed shall be designed in accordance with UBC 1911.10.

Allowable shear force is:

$$V_c = 2 \cdot \varphi \cdot \sqrt{f_c} \cdot \left(b_w L_{wall} \cdot \Psi\right)$$

Where:

f'_c = Specified the concrete compressive strength.

b_w =The equivalent width of the Perform Wall™ concrete core.

L_w = The equivalent Length of the Perform Wall™ concrete core.

ψ = Perform Wall™ shape reduction factor, .85.
φ = Strength reduction factor, .85.

If the allowable stress of concrete as determined by the above equation is less than the actual shear stress, therefore shear reinforcement is required. The required shear force to be resisted by the reinforcement is:

$$V_s = V_n - \varphi \cdot V_c$$

$$A_v = \frac{V_s \cdot S}{f_v \cdot d}$$

Where:

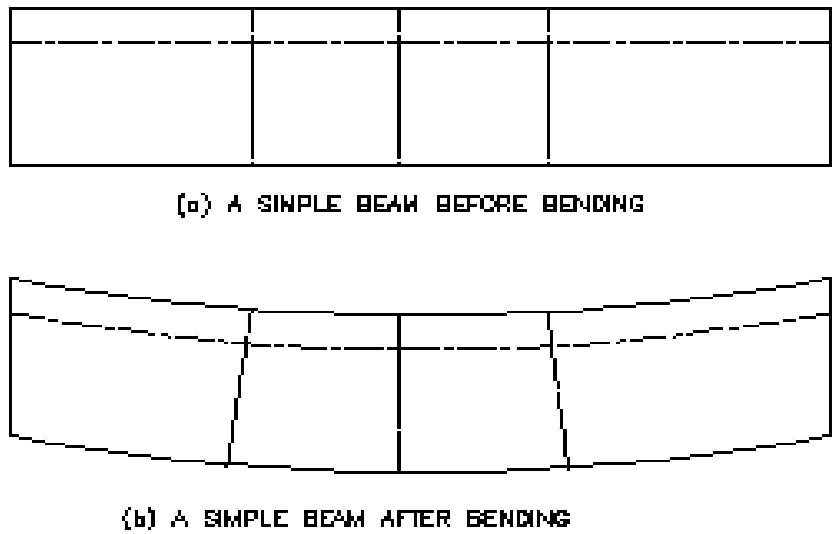
V_s = The shear force to be resisted by the reinforcement.

V_n = The actual shear force on the shear wall.

A_v = The effective area of shear wall.

f_v = The steel strength of the shear reinforcement.

The Perform Wall™ lintel can simply be designed to support typical headers for a residence, as well as large spans of 20 feet or more for low or multi level structure (figure 6).



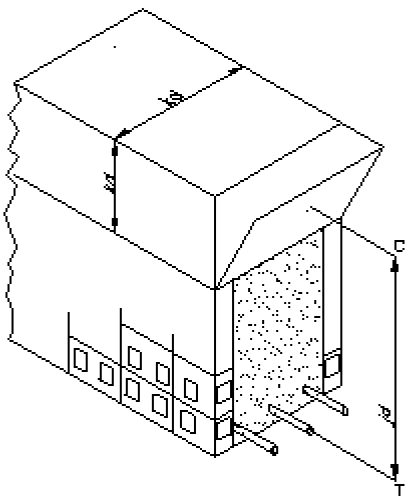


Figure 6a and b

Perform Wall™ lintels can be either designed as simple span beams, fixed beams or continuous members. Because of the unique shape of the

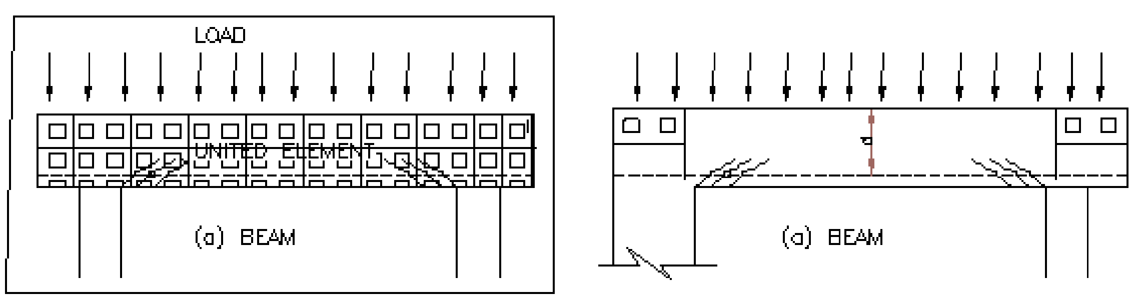


Figure 7

concrete requirements for minimum reinforcing steel must be met. This is necessary to maintain the structural integrity of the system throughout its useful life (figure 7). If the wall is to be subjected to design loads or longer spans than those required for minimum reinforcement, structural calculations must be performed by a qualified registered engineer to determine the required steel reinforcing for the design conditions.

3.9.1 MINIMUM REINFORCEMENT REQUIREMENTS

The Perform Wall™ Lintel system requires the use of reinforcement to meet the requirements of the American Concrete Institute's Building Code Requirements for Reinforced Concrete (ACI 318). Uniform Building Code chapter 19 or SBCCI code section 1908. The following recommendations conform to the requirements of these Codes with the exception of ACI Section 14.3.5 or UBC section 19.14.3.5, which requires a maximum spacing of reinforcing of 18 inches in both the horizontal and vertical directions. This criteria is usually not required to be met in most residential, light commercial, and light industrial applications.

Minimum reinforcing for the lintel is one #4 (1/2") bar placed in every horizontal core (15" inch centers) and one #4 bar placed in every vertical core (15 inch centers). This minimum reinforcing exceeds the requirements of the CABO One and Two Family Dwelling Code, which is accepted by ICBO, BOCA, and SBCCI for residential construction. Every opening less than 4 feet wide must have a reinforced lintel over the door of at least 15" of depth of concrete with two #5 bars, each with 3/4" clear concrete cover on the bar at the top and bottom of the lintel and two vertical #5 bars on each side of the opening to meet the minimum requirements of ACI for reinforcement around openings.

The design of lintels can be summarized as follows:

1. Lintels shall be designed for maximum moment. Determine the maximum moment for the lintel, and then the actual reinforcement will be found.

2. By knowing the reinforcement for flexure, the next step is to check the shear for the imposed loads. Add shear reinforcement if the actual shear force exceeds the allowable shear in the lintel.

3. Finally the deflection of the lintel must be found and compared to the allowable deflection

SECTION 4

Sample Element Calculation

Slender wall supporting roof grading

A 10" Perform Wall® 10' high is supporting a roof dead load of 700 plf, Live load of 700 plf, and is located in seismic zone 4.

Design Data Input

Wall thickness, (Nominal) $t_g = 10 \cdot in$ Concrete core thickness $t_c := 6 \cdot in$

Wall height $H_t := 10 \cdot ft$ Equivalent square sides $S_{eq} = 4.75in$

Rebar size $NO := 5$

Rebar spacing Vertical $spcacing = 15 \cdot in$

Rebar location edge/Center (1/0) $location = 0$

Rebar depth $d = 2.38in$

Concrete strength $f_c = 2500 \cdot psi$

Modulus of rupture $f_r = 250psi$

Steel strength $Fy = 60 \cdot ksi$

Wall Weight Multiplier $factor = 1.00$

Wall weight

Effective strip width

Loads Input

$W_{wall} = 52 \cdot psf$

$W_{eff} = 15$

Vertical Load Input

Dead load $W_{dl}\_e = 700 \cdot plf$ Dead Load $P_{dl}\_c = 0 \cdot lb$

Live load $W_{ll}\_e = 700 \cdot plf$ Live Load $P_{ll}\_c = 0 \cdot lb$

Eccentricity $e = 6.75 \cdot in$

Width of bearing $pl^{W_{pl}=0 \cdot in}$

Slender wall supporting roof grading

Lateral Loads

Seismic , $Z := .4$ $I := 1$ $C_p := .75$

Seismic load Factor $Z \cdot I \cdot C_P wall_w$

Seismic Force on the wall $p_v := Z \cdot I \cdot C_{p \cdot w} wall$ $p_v = 15.6^{\circ} psf$

Wind

speed, $V = 70 \cdot mph$ Exposure $exp = c$ $C_q = 1.3$ $I = 1$

Pressure $q_s = 12.6psf$

Wind force on the wall: $P = 17.36psf$

Check axial load limitation

$\frac{p_w + P_o}{A_g} = 73.39psi$ must be < $.04 \cdot f_c = 100psi$

Factored loads

Seismic

$U = .75(1.4 \cdot w_{dl} + 1.7 \cdot w_{11}(floor) + 1.87 \cdot E)$ $U = .9 \cdot w_{dl} + 1.43 \cdot E$

OR

Wind

$U = .75(1.4 \cdot w_{dl} + 1.7 \cdot w_{11}(floor) + 1.7 \cdot w)$ $U = .9 \cdot w_{dl} + 1.3 \cdot W$

Factored load on the wall $p_{ou} = 2034.38lb$

Factored wall load at mid ht $p_{wu} = 341.25lb$

Factored wall load at mid ht $p_u = 2375.63lb$

Factored Lateral Load $w_u = 27.67plf$

Slender wall supporting roof grading

Cracked section Analysis

$$A_{se} = \frac{(p_u + A_s \cdot F_y)}{F_y} \qquad A_{se} = 0.35 in^2$$

$$M_{cr} = \frac{f_r \cdot I_g}{y_t} \qquad M_{cr} = 6.69 in \cdot kips$$

$$I_{cr} = \left[n \cdot A_{se} \cdot (d - c)^2\right] + \frac{S_{eq} \cdot c^3}{3} \qquad I_{cr} = 22.49 in^4$$

$$\Delta_{cr} = \frac{5 \cdot M_{cr} \cdot Ht^2}{48 \cdot E_c \cdot I_g} \qquad \Delta_{cr} = 0.06 in$$

Service Moment

First try: $\Delta_s = .18 in$

$$M_s = \frac{w_{st} \cdot Ht^2}{8} + p_0 \cdot \frac{e}{2} + (p_0 + p_w) \cdot \Delta_s \qquad M_s = 9.54 in \cdot kips$$

$$\Delta_s = \frac{5 \cdot M_s \cdot Ht^2}{48 \cdot E_c \cdot I_g} + \frac{5 \cdot (M_s - M_{cr}) \cdot Ht^2}{48 \cdot E_c \cdot I_{cr}}$$

$\Delta_s = 0.15 in$ Trial="Trial o.k."

Nominal Moment Strength $\phi = .90$

$$M_n = (A_{se} \cdot F_y) \cdot \left(d - \frac{a}{2}\right)$$

$$M_n = 27.99 in \cdot kips \qquad \phi \cdot M_n = 25.19 in \cdot kips$$

$$\Delta_n = \frac{5 \cdot M_n \cdot Ht^2}{48 \cdot E_c \cdot I_{cr}} \qquad \Delta_n = 0.65 in$$

Slender wall supporting roof grading

Factored Moment

$$M_u = \frac{w_u \cdot Ht^2}{8} + p_u \cdot \frac{e}{2} + (p_{ou} + p_{wu}) \cdot \Delta_n$$

$M_u = 1143.71 lb \cdot ft$ Must be < $\phi \cdot M_n = 2099.51 lb \cdot ft$

Moment= "Moment less than allowable, wall O.K.

Check deflection at service load

$M_s = 9.54 in \cdot kips$ $M_n = 27.99 in \cdot kips$

$$\Delta_s = \Delta_{cr} + \frac{M_s - M_{cr}}{M_n - M_{cr}} \cdot (\Delta_n - \Delta_{cr})$$

$\Delta_s = .14 in$ Must be < $\frac{Ht}{150} = .8 in$

Deflection= "Deflection < L/150 O.K."

Slender wall supporting girder

Design Data Input

Wall thickness, (Nominal) $t_g = 10\text{in}$ Concrete Core thickness $T_c = 6\text{in}$

Wall Height $Ht = 12\text{ft}$ Equivalent square sides $S_{eq} = 4.75\text{in}$

Rebar Size $No = 6$ Rebar spacing Vertical $spacing = 15\text{in}$

Rebar location Edge/Center (1/0) $locaiton = 1$

Rebar depth $d = 3.38\text{in}$

Concrete Strength $f_c = 2500\text{psi}$

Modulus of Rupture $f_r = 250\text{psi}$

Steel Strength $F_y = 60\text{ksi}$

Wall Weight Multiplier $factor = 1.00$

Wall weight $W_{wall} = 52\text{psf}$

Effective Strip Width $W_{eff} = 15$

Loads Input

Dead Load $W_{dl\_e} = 700\text{plf}$ Dead Load $p_{dl\_c} = 1000\text{lb}$

Live Load $W_{ll\_e} = 700\text{plf}$ Live Load $p_{11\_c} = 2000\text{lb}$

Eccentricity $e = 6.75\text{in}$

Width of Bearing Pl $W_{pl} = 15\text{in}$

Slender wall supporting girder

Lateral Loads

Seismic $Z = 4$ $I = 1$ $C_p = .75$

Seismic load Factor $ZIC_p = 0.3$

Seismic Force on the wall $v_p = ZIC_p{}^{w}wall$ $v_p = 15.6psf$

Wind

Speed $V = 70mph$ Exposure $exp = c$ $C_q = 1.3$ $I = 1$

Pressure $q_s = 12.6psf$

Wind force on the wall $P = 17.36psf$

Check axial load limitation

$$\frac{p_w + p_o}{A_g} = 75.69psi$$ Must be < $.04f_c = 100psi$

Factored loads

Seismic

$$U = .75(1.4 \cdot w_{dl} + 1.7 \cdot w_{11}(floor) + 1.87 \cdot E) \qquad U = .9w_{dl} + 1.43E$$

Wind

$$U = .75(1.4 \cdot w_{dl} + 1.7 \cdot w_{11}(floor) + 1.7W) \qquad U = .9w_{dl} + 1.3w$$

Factored load on the wall $p_{ou} = 3622.24lb$

Factored wall load at mid ht $p_{wu} = 409.5lb$

Factored wall load at mid ht $p_u = 4031.74lb$

Factored lateral load $w_u = 27.67plf$

Slender wall supporting girder

Factored Lateral Load

Cracked section Analysis

$A_{se} = \dfrac{p_u + A_s F_y}{F_y}$ $A_{se} = .51 in^2$

$M_{cr} = \dfrac{f_r I_g}{y_t}$ $M_{cr} = 6.69 in \cdot kips$

$I_{cr} = \left[n \cdot A_{se}(d-c)^2\right] + \dfrac{s_{eq} c^3}{3}$ $I_{cr} = 71.51 in^4$

$\Delta_{cr} = \dfrac{5 M_{cr} Ht^2}{48 E_c I_g}$ $\Delta_{cr} = .08 in$

Service Moment

First Try: $\Delta_s = .18 in$

$M_s = \dfrac{w_{st} Ht^2}{8} + p_o \dfrac{e}{2} + (p_o + p_w)\Delta_s$ $M_s = 10.98 in \cdot kips$

$\dfrac{5 M_s Ht^2}{48 E_c I_g} + \dfrac{5(M_s - M_{cr}) Ht^2}{48 E_c I_{cr}}$

$\Delta_s = .18 in$ Trial= "Trial o.k."

Nominal Moment Strength $\phi = .90$

$M_n = (A_{se} F_y)\left(d - \dfrac{a}{2}\right)$

$M_{n=} 56.95 in \cdot kips$ $\phi \cdot M_n = 51.26 in \cdot kips$

$\Delta_n = \dfrac{5 M_n Ht^2}{48 E_c I_{cr}}$ $\Delta_n = .6 in$

Slender wall supporting girder

Factored Moment

$$M_u = \frac{w_u Ht^2}{8} + p_u \cdot \frac{e}{2} + (p_{ou} + p_{wu})\Delta_n$$

$M_u = 1834.81 lb \cdot ft$ Must be < $\phi \cdot M_n = 4271.34 lb \cdot ft$

Moment="Moment less than allowable, Wall O.K."

Check deflection at service load

$M_s = 10.98 in \cdot kips$ $M_n = 56.95 in \cdot kips$

$$\Delta_s = \Delta_{cr} + \frac{M_s - M_{cr}}{M_n - M_{cr}}(\Delta_n - \Delta_{cr})$$

$\Delta_s = .12 in$ Must be < $\frac{Ht}{150} = .96 in$

deflection = "Deflection< L/150 O.K."

Typical Garage Door Opening

Design Data Input

Wall thickness, (Nominal)	$t_g = 10in$	Concrete Core thickness	$t_c = 6in$
Lintel length	$L = 16f$	Equivalent square sides	$s_{eq} = 4.75in$
Lintel height	$h = 30in$	Rebar depth	$d = 25in$
Rebar size	$reinf = 5$	No of renf.	$No = 2$

Concrete Strength	$f_c = 2500psi$
Modulus of rupture	$f_r = 250psi$
Steel Strength	$F_y = 60ksi$
Lintel Weight Multiplier	$factor = 1.00$
Lintel weight	$w_{wall} = 52psf$
Strength Reduction Factor	$\phi = .90$

Loads Input

Vertical load Input		Concentric Load	
Dead load	$w_{dl} = 300plf$	Dead load	$p_{dl} = 0lb$
Live load	$w_{ll} = 300plf$	Live load	$p_{ll} = 0lb$
		Distance from support	$x = 0ft$

Typical Garage Door Opening

Analysis,

$$U = (1.4w_{dl} + 1.7w_{ll})$$

Factored Ultimate Moment $M_u = 29.76\text{ft} \cdot \text{kps}$

Allowable Moment $M_n = 63.98\text{ft} \cdot \text{kips}$ Result= "Bending O.K."
Use No=2 reinf= 5

Check Shear

Area resisting shear $A_c = 118.8\text{in}^2$

Allowable Shear force $V_c = 2\phi\sqrt{f_c} \cdot A_c$ $V_c = 10.69\text{kips}$

Factored Shear force $V_u = 8.9\text{kips}$ Result= "Shear O.K."

Shear reinforcement answer= "No shear reinforcement is required" $A_v = 0\text{in}^2$

Typical Garage Door Opening with Girder Loading

Design data Input

Wall thickness, (Nominal) $t_g = 10\text{in}$ Concrete Core Thickness $t_c = 6\text{in}$

Lintel Length $L = 16\text{ft}$ Equivalent square sides $s_{eq} = 4.75\text{in}$

Lintel Height $h = 30\text{in}$ Rebar Depth $d = 25\text{in}$

Rebar Size $\text{reinf} = 6$ No. of reinf $\text{No} = 2$

Concrete Strength $f_c = 2500\text{psi}$

Modulus of rupture $f_r = 250\text{psi}$

Steel strength $F_y = 60\text{ksi}$

Lintel Weight Multiplier $\text{factor} = 1.00$

Lintel Weight $w_{wall} = 52\text{psf}$

Strength reduction factor $\phi = .90$

Loads input

Vertical load Input Concentric Load

Dead load $w_{dl} = 100\text{plf}$ Dead load $p_{dl} = 4000\text{lb}$

Live load $w_{ll} = 100\text{plf}$ Live load $p_{ll} = 6000\text{lb}$

Distance from Support $x = 8\text{ft}$

Typical Garage Door Opening with Girder Loading

Analysis $U = (1.4w_{dl} + 1.7w_{ll})$

Factored ultimate Moment $M_u = 29.76 ft \cdot kips$

Allowable Moment $A_c = 118.8 in^2$ Result= "Bending O.K."

Use No=2 Reinf=5

Check Shear

Area resisting shear $V_c = 2\phi\sqrt{f_c} \cdot A_c$

Allowable shear force $V_c = 10.69 kips$

Factored shear force $V_u = 8.9 kips$

Shear reinforcement answer= "No shear reinforcement is required" $A_v = 0 in^2$

Shear Wall Design

Design data input

Wall thickness $t_g = 10\text{in}$ Concrete Core Thickness $t_c = 6\text{in}$

Wall height $Ht = 10\text{ft}$ Equivalent Square Sides

Wall length $L_{wall} = 90\text{in}$ $d_{wall} = .8L_{wall}$ $d_{wall} = 6\text{ft}$

Wall weight $w_{wall} = 52\text{psf}$ Weight Factor $\text{factor} = 1.00$

Rebar Size $No = 5$ Rebar spacing Vertical $\text{spacing} = 15\text{in}$

Rebar location Edge/Center (1/0) $\text{location} = 1$

Rebar depth $d = 3.44\text{in}$

Concrete Strength $f_c = 2500\text{psi}$

Modulus of rupture $f_r = 250\text{psi}$

Steel Strength $F_y = 60\text{ksi}$

Strength Reduction Factor Perform Wall shape Reduction Factor

Lateral Loads,

Seismic, $Z = 4$ $I = 1$ $C = 1$ $R_w = 6$

Pier Seismic force $v_p = 10\text{kips}$ $h_p = \dfrac{Ht}{2}$

Additional force on the pier, $V = 0\text{lb}$

Distance from the base of the wall $x = 10\text{ft}$

Shear Wall Design

Forces

Reaction $V = V + v_p$ $V = 1000\text{lb}$

Moment $M = Vx + v_p \frac{h_p}{2}$ $M = 125\text{ft}\cdot\text{kips}$

Axial $P_{axial} = h_{p_{bw\,wall}}$ $P_{axial} = 102.96$

Shear force $V_u = 1.4V$ $V_u = 14000\text{lb}$

Moment $M_u = 1.4M$ $M = 125000\text{lbft}$

Axial $P_u = 1.4P_{axial}$ $P_u = 144.14\text{lb}$

Concrete Shear strength $Vc = 2\phi\sqrt{f_c psi}bd = 3.13\text{kips}$

Boundary Reinforcement

Rebar Size Reinf= 5 No of Reinf No=2

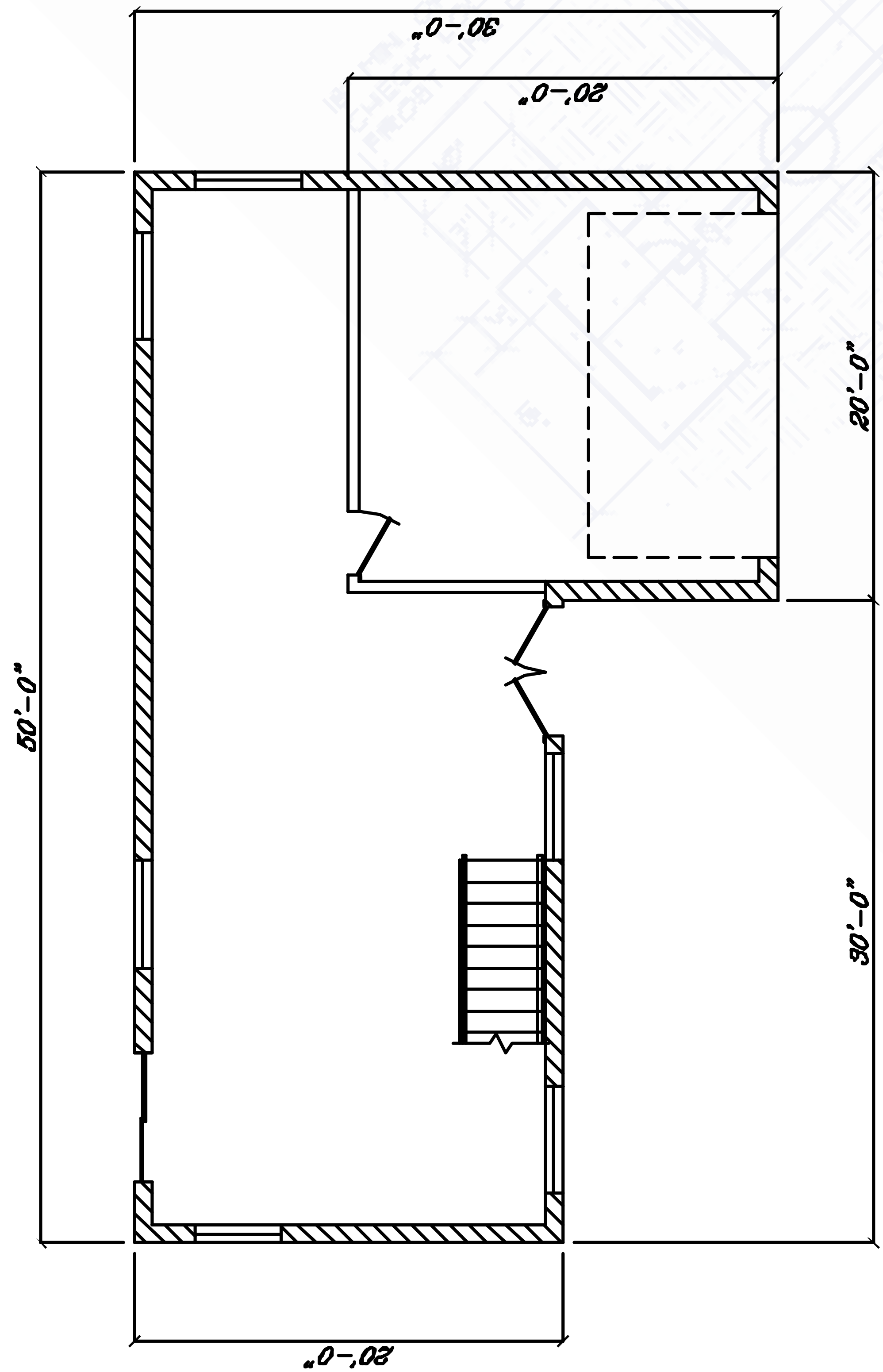
Allowable Moment $M_n = .9s_{eq^d}\,\text{wall}^2 Fy\rho\left(1 - .59\rho\frac{F_y}{f_c}\right)$

$M_n = 166.45\text{ft}\cdot\text{kips}$ Result="Bending O.K." Use No=2 Reinf=5

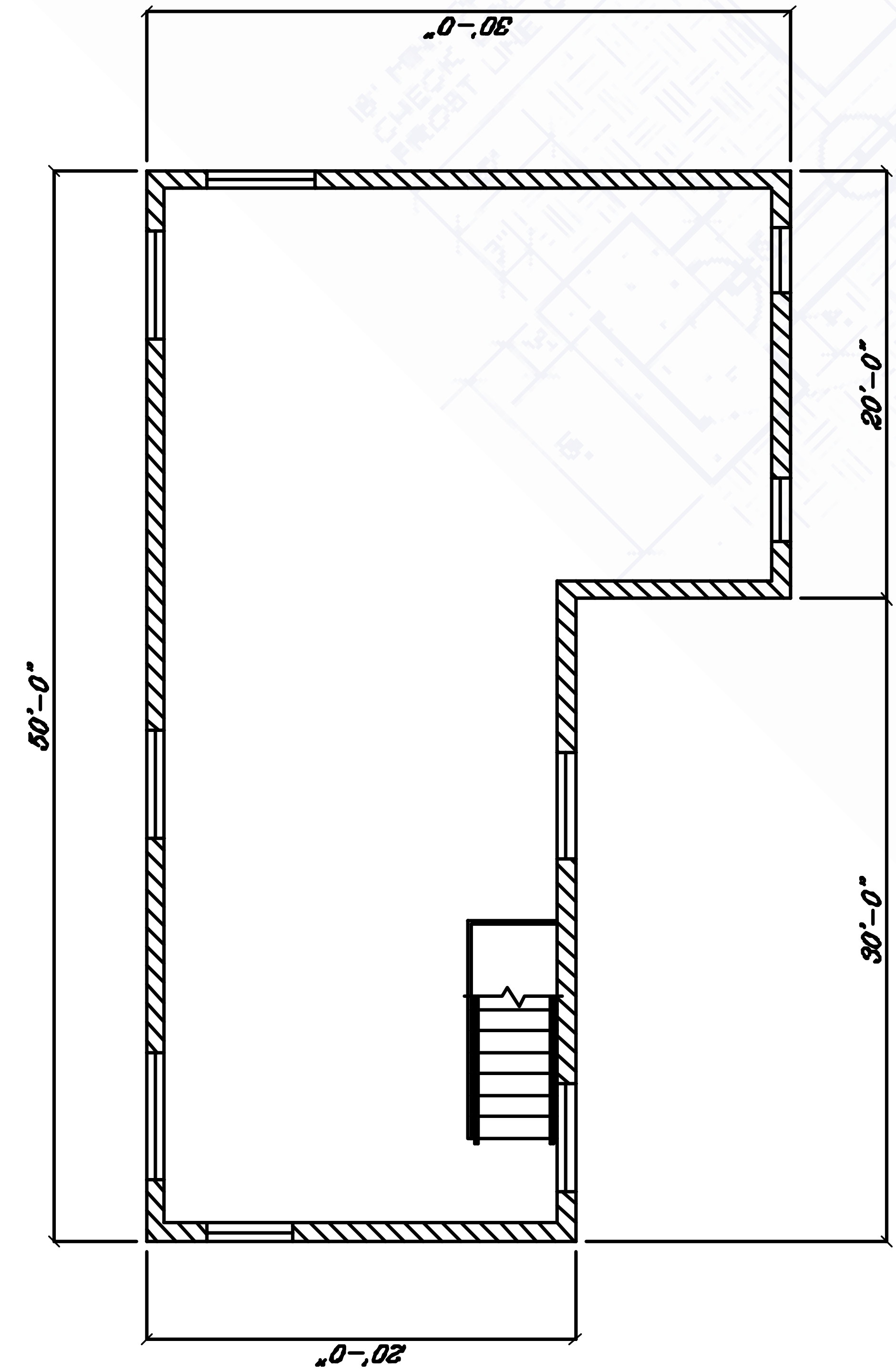
SECTION 5

Design of a Two-Story Residence

First Floor Plan

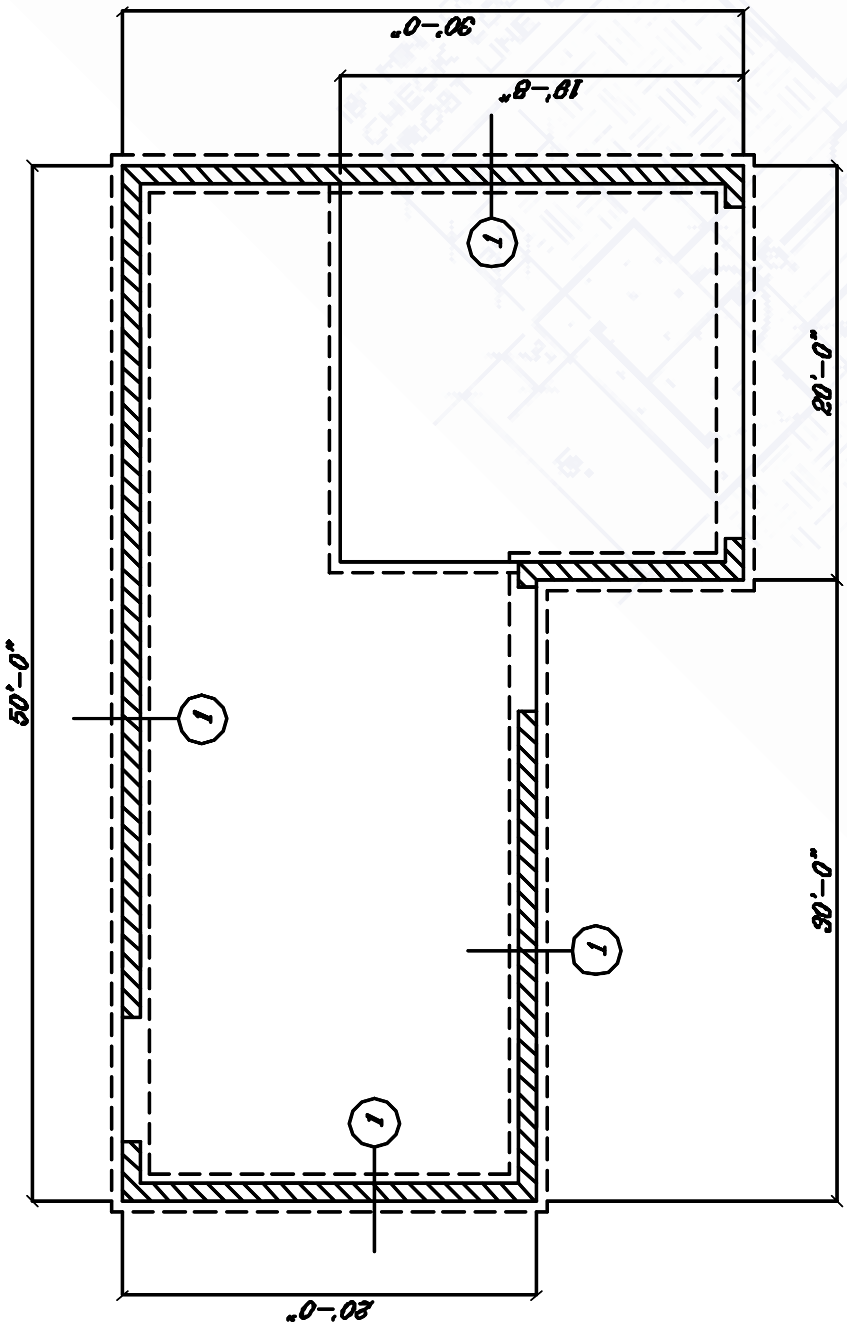


Second Floor Plan



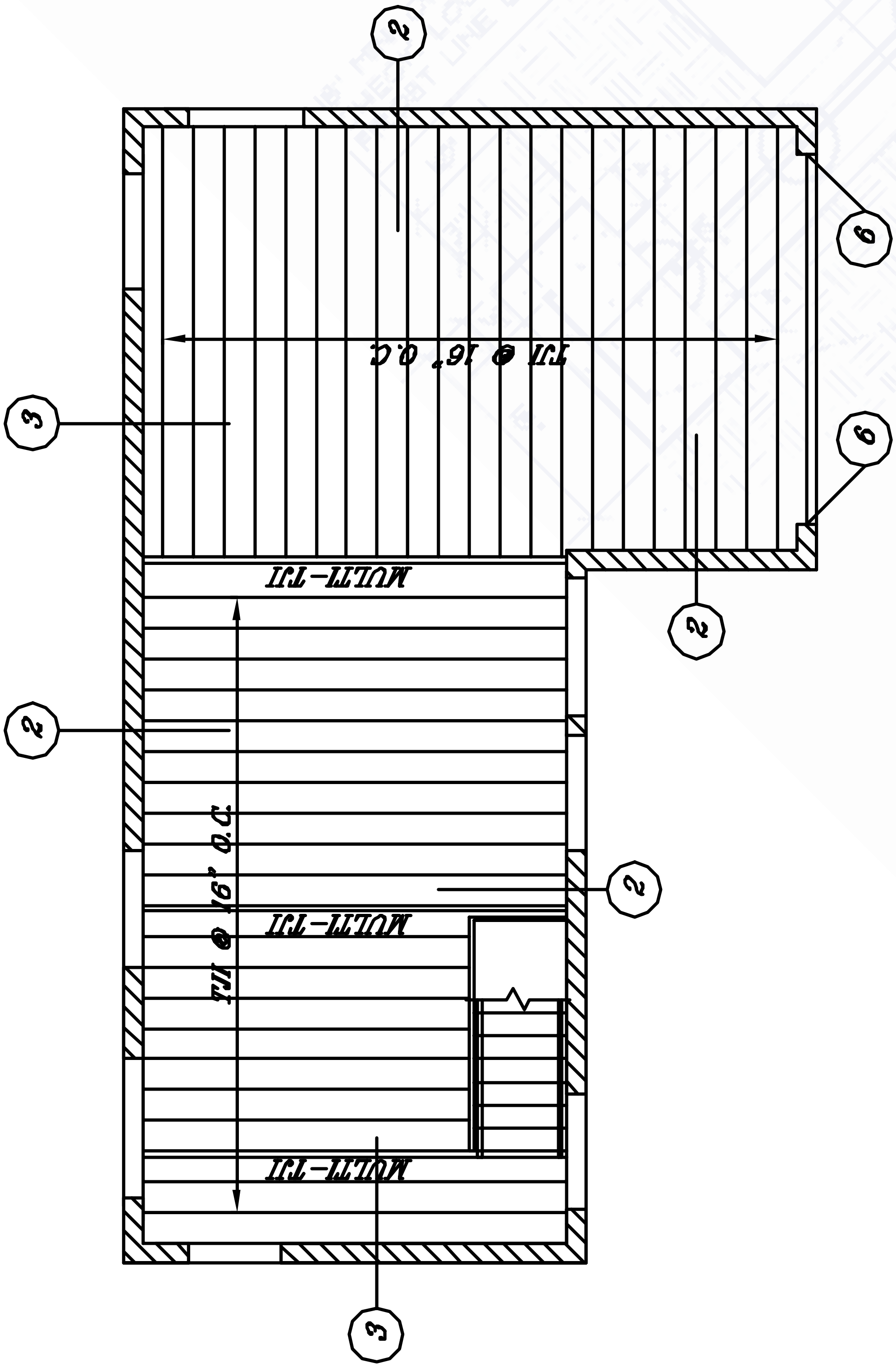
PERFORM WALL PANEL SYSTEM

Foundation Plan

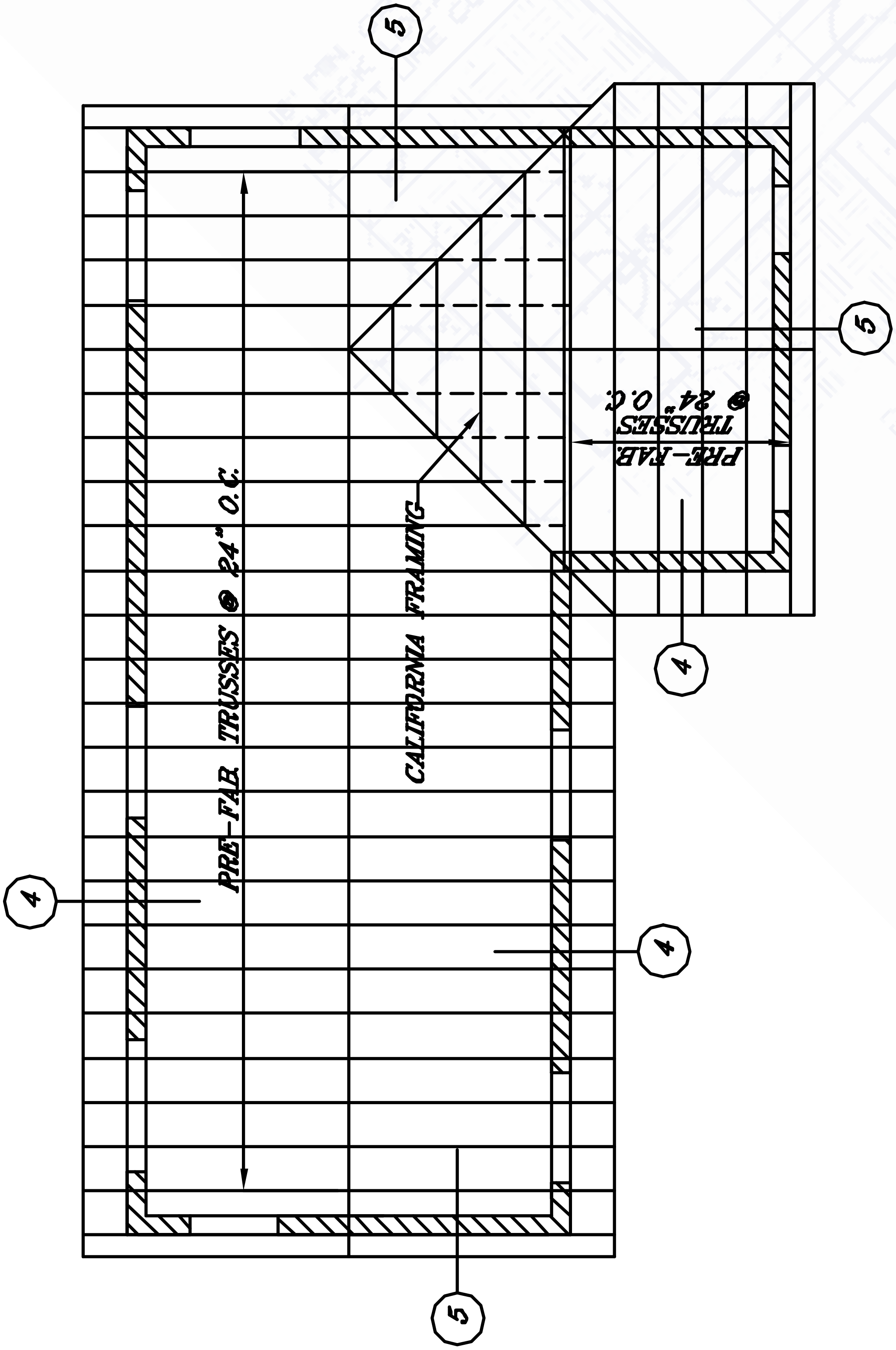


PERFORM WALL PANEL SYSTEM

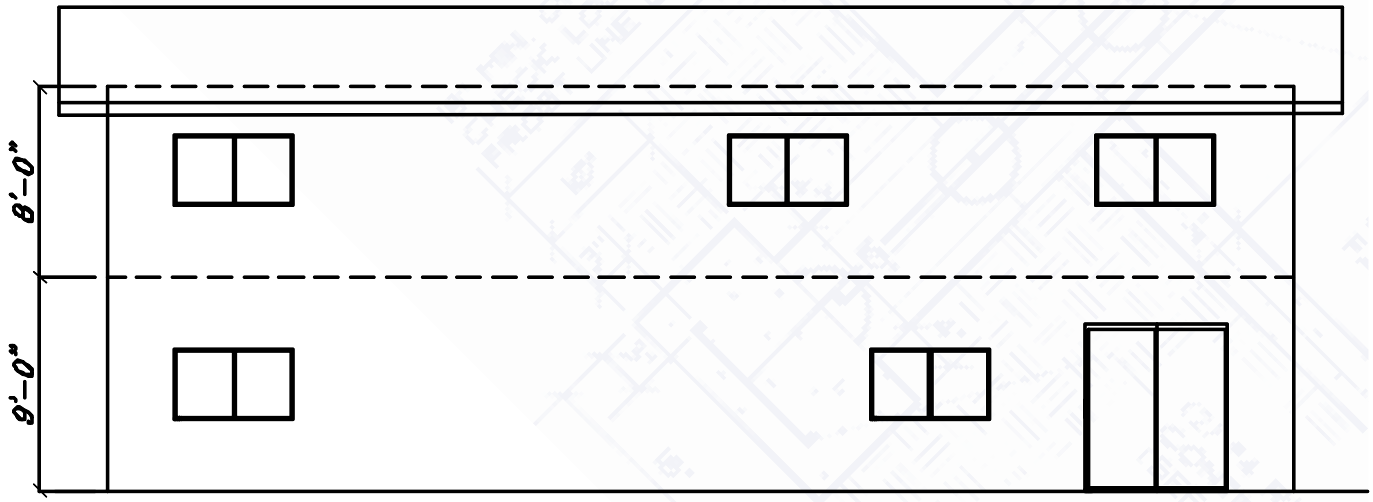
Floor Framing Plan



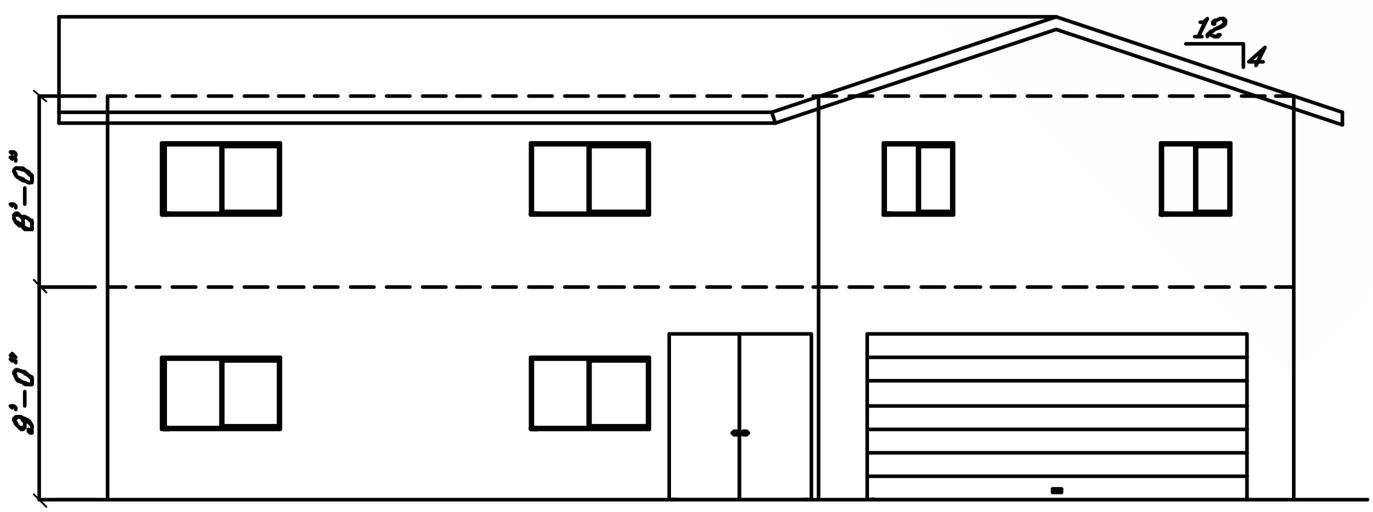
Roof Framing Plan



Back Elevation

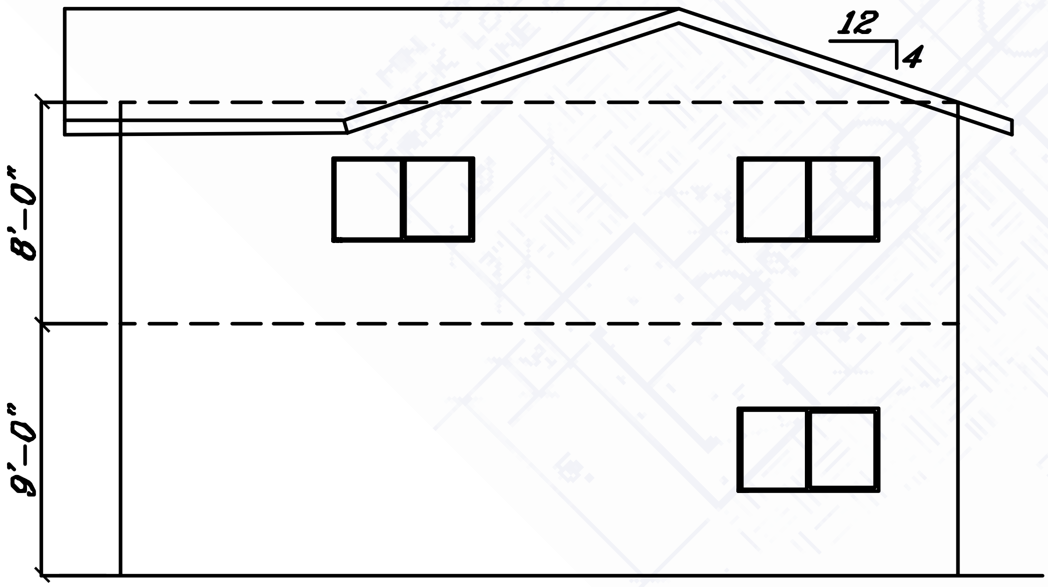


Front Elevation

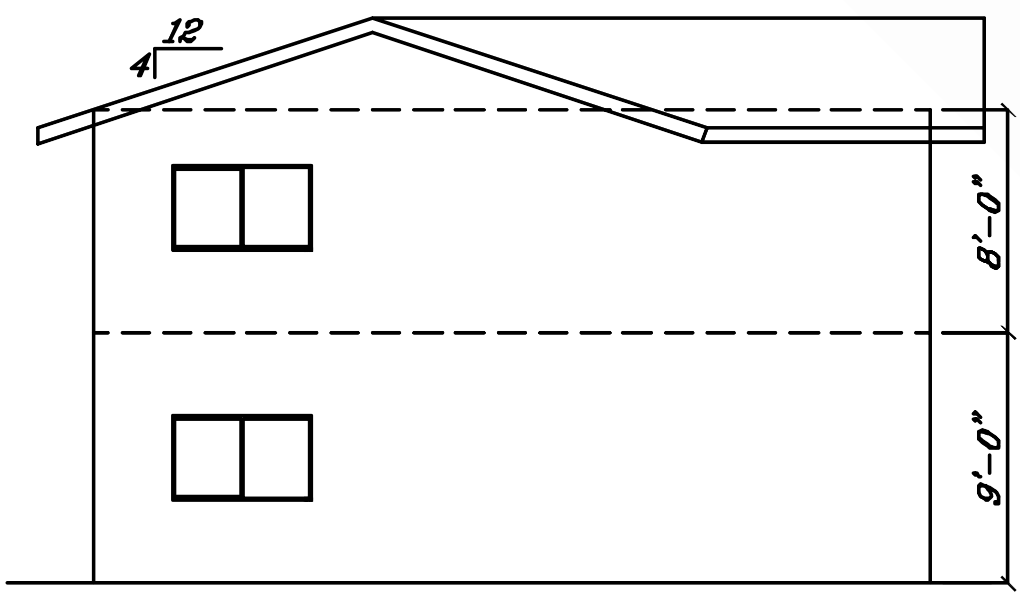


PERFORM WALL PANEL SYSTEM

Right Side Elevation



Left Side Elevation



Example of a residence

Roof Load:

1- Dead Load

Roofing, Built-up/concrete tile/Shingle Rf=10) $Rf_{wt} = 9.5psf$

Sheathing, plywood, thickness (t) $t = .5in$ $St = 1.5psf$

Framing, framing $Fr = 2.20$

Ceiling, Plaster=p
Suspend'd=s
Gypsum=g for gypsum give thickness clg=. 5in
No ceiling=n
Ceiling Type

Ceiling Weight $c\lg_{type} = "s"$

Insulation 8" bat. Insulation $C\lg_{wt} = 1.8psf$

Electrical/Mechanical $In = 2.0psf$

Misc $Ee = 1psf$

Total Dead Load $Ms = 1.5psf$

Roof Pitch $w_{dl\_r} = 20psf$

Live Load $w_{ll\_r} = 16psf$

Snow Load $w_{snow} = 0psf$

Wall weight $w_{wall} = 15psf$

Partition wall load $w_{par} = 0psf$

$$pitch_{factor} = \frac{\sqrt{0^2 + 12^2}}{12}$$ $pitch_{factor} = 1.12$

Description: Example of a residence/Loading Criteria Floor Load

1-Dead Load

Flooring,			$Fi = 1.0psf$
Sheathing, plywood, thickness (t),		$t = \frac{3}{4} in$	$St = 2.25psf$
Lightweight Concrete yes/no	answer ="yes"	$t_{conc} = 1.5in$	$w_{conc} = 15psf$
Framing,	trusses/framing		$Fr = 3.5psf$
Ceiling,	suspend'd/gypsum/no ceiling, (2/1/0)	$C\lg = 2$	
	for gypsum give thickness,	$T_{c\lg} = 0.0in$	$C\lg_{wt} = 1.8psf$
Insulation	8" bat. insulation		$\ln = 1.1psf$
Electrical/Mechanical			$Ee = 2.0psf$
Sprinkler loading			$sp = 1.0psf$
Misc.			$Ms = 2.0psf$
Total Dead Load			$W_{dl\_f} = 30osf$
Partition Load			$W_{par} = 20psf$
Live Load			$W_{ll\_f} = 40psf$

Description: Lateral Analysis

Building Data:

Length, $L_r = 47\text{ft}$ Width, $b_r = 46\text{ft}$

Roof Weight $W_r = W_{dl}\text{pitch}_{factor}$ Fascia height $F_h = 5.5\text{in}$

Roof/floor ht, $H_{wall\_rr} = 8\text{ft}$ Wall weight $W_{wall} = 15\text{psf}$

Roof Pitch $0 = 6/12$ Parapet height $H_p = 0\text{ft}$

Gable/Parapet height $h_g = 11.96\text{ft}$

Roof Height including parapet/or sloped Roof $H_{tot} = 19.96\text{ft}$

Overhang length $L_{oh} = 2\text{ft}$

Roof Weight $W_{roof} = [(L_r + L_{oh^2})(b_r + L_{oh^2})]w_r$ $W_{roof} = 84531.75\text{lb}$

Reduction for snow for seismic load reduction $= 75\%$

Snow Load $W_{snow} = (L_r b_r)[w_{snow}(100\% - \text{reduction})]$ $W_{snow} = 0\text{lb}$

Wall weight:

With Parapet $W_{wall} = w_{wall}\dfrac{H_{r^2}}{2h_r}$ No parapet $W_{wall} = W_{wall}\left(\dfrac{h_r}{2} + h_g\right)$

Long Direction $W_{wall\_F\_B} = 22501.25\text{lb}$ Short Direction $W_{wall\_L\_R} = 22022.5\text{lb}$

Partition wall/ Interior wall weight $W_{par} = (L_r b_r)\dfrac{w_{par}}{2}$ $W_{par} = 21620\text{lb}$

Total load applied at roof level:

Long Direction $W_{r\_F\_B} = 128653\text{lb}$ Short Direction $W_{r\_L\_R} = 128174.25\text{lb}$

Total load applied at roof level $W_r = W_{roof} + W_{wall} + W_{par}$ $W_r = 1286563$

Description: Lateral Analysis

Floor level,

Length $L_f = 49ft$ Width $b_f = 28ft$

Floor height $H_{wall\_fl} = 11ft$ Wall weight $w_{wall} = 15psf$

Floor weight $w_f = (w_{dl\_f})$

Partition wall/Interior wall weight $W_{par} = (L_r b_r)\frac{w_{par}}{2}$ $W_{par} = 21620lb$

Floor weight $W_{floor} = (L_f b_f)w_f$ $W_{floor} = 41160lb$

Wall weight $W_{wall} = \left[L_f w_{wall}\left(\frac{H_{wall\_rf}}{2} + \frac{H_{wall\_fl}}{2}\right)\right]\cdot 2$ $W_{wall} = 13965lb$

Partition wall/Interior wall weight $W_{in} = (L_r b_r)w_{par}$ $W_{par} = 21620lb$

Total load applied at floor level $W_f = W_{floor} + W_{wall} + W_{par}$ $W_f = 76745$

Total Load of the roof and floor $W_{tot} = W_r + W_f$

Description: Lateral Analysis

Seismic Analysis

Seismic zone factor $Z = 4$

Building importance Factor $L = 1$

"R" Coefficient $R = 4.5$

Lateral resistance system:
Mas. or conc. shear wall/steel MSRF/ Concrete MSRF/ Others (1/2/3/4) $Lrs = 1$

For concrete/masonry shear wall system enter: $A_e = 12^2 in^2 \cdot 2 \;\; D_e = 15ft$

Building period $T = C_t h_{tot}^{\frac{3}{4}}$ $T = 0.71 sec$

Distance from site to the source distance=16km

Seismic fault type fault = B

Near Source Factor $N_a = 1$ $N_v = 1$

Seismic force coefficients $C_a = 44N_a$ $C_v = .64N_v$

$coef_1 = \frac{2.5C_a 1}{R}$ $coef_1 = .24$

$coef_2 = .11C_a 1$ $coef_2 = .05$

$coef_3 = \frac{.8N_v 1}{R}$ $coef_3 = .18$

Seismic coefficient $coef = .2444$

Total seismic force $V = coefW_{tot}$ $V = 50208.4lb$

Description: Lateral Analysis

Multi-story analysis,

FT=0 if T<.7, Ft max= .25xV $F_t = 2492.61\text{lb}$

Number of Levels, $n = 2$

$$F_x = \frac{(V - F_t) w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$

	Level No	Height	Force	Unit force	Overturning Moment
	$x=$	$h_x = \text{ft}$	$F_x =$	$f_x = \text{lb} \cdot \text{ft}^{-1}$	$M_{ot_y = \text{lb}\cdot\text{ft}}$
Roof	2	19	35466.99	754.62	
Floor	1	11	12248.8	260.1	134736.8
Base	0	0	47715.79	1015.23	808609.7

Description: Lateral Analysis

SEISMIC LOAD SUMMARY, INCLUDING RELIABILITY FACTORS

Roof Level

Reliability/redundancy factor

Ave. Area $A_B = L_r b_r$ $A_B = 2162\text{ft}^2$

Assume $r_{max} = .15$

$\rho = 1$

$$\rho = 2 - \frac{20}{r_{max}\sqrt{A_B}}$$

Seismic Roof Level, $F_2 = \rho F_2$ $F_2 = 35.47\text{kips}$

Floor Level

Reliability/redundancy factor

Ave. Area $A_B = 1372\text{ft}^2$ $A_B = L_f b_f$

Assume $r_{max} = .15$

$\rho = 1$

$$\rho = 2 - \frac{20}{r_{max}\sqrt{A_B}}$$

Seismic Roof Level $F_1 = \rho F_1$ $F_1 = 12.25\text{kips}$

Seismic Base Level $F_o = F_1 + F_2$ $F_o = 47.72\text{kips}$

Description: Lateral Analysis

WIND ANALYSIS

Wind speed,	$V = 70mph$
Basic wind pressure	$q_s = 12.6psf$
Exposure	$exp = "C"$
Pressure Coeficient,	$C_q = 1.3$
Building inportance Factor	$I = 1$

Front Back Direction

	Height	Gust Factor		Wind Pressure	Wind Force	Unit Force
	$h_j = ft$	$k_i =$	$C_{e_{i=}}$	$p_i = lb \cdot ft^{-2}$	$V_{F\_b_j} = lb \cdot ft^{-1}$	$V_{f\_b_j} = lb \cdot ft^{-1}$
Roof	19	40	1.06	17.36	16779.11	357
Floor	11	35	1.13	18.51	8082.38	171.97
Base	0	30	1.19	19.49	29540.76	628.53
		25	1.23	20.15		

Left-Right direction

	Height	Gust Factor		Wind Pressure	Wind Force	Unit Force
	$h_j = ft$	$k_i =$	$C_{e_{i=}}$	$p_i = lb \cdot ft^{-2}$	$V_{L\_R_j} = lb$	$V_{l\_r_j} = lb \cdot ft^{-1}$
Roof	19	40	1.06	17.36	7970.25	169.58
Floor	11	35	1.13	18.51	4618.5	98.27
Base	0	30	1.19	19.49	15262.62	324.74
		25	1.23	20.15		

Description: Lateral Analysis

LATERAL ANALYSIS SUMMARY

Roof Level

Front to back Direction Seismic_or_wind = "Seismic Governs"

Lateral force front-back, $F_{F\_B_2} = 35466.99\text{lb}$ $f_{f\_b_2} = 754.62\text{plf}$

Left to Right Direction Seismic_or_wind = "Seismic Governs"

Lateral force front-back $F_{L\_R_2} = 35466.99\text{lb}$ $f_{l\_r_2} = 771.02\text{plf}$

Second Floor Level

Front to back Direction Seismic_or_wind = "Seismic Governs"

Lateral force front-back, $F_{F\_B_1} = 12248.8\text{lb}$ $f_{f\_B_1} = 260.61\text{plf}$

Left to Right Direction Seismic_or_wind = "Seismic Governs

Lateral force front-back, $F_{L\_R_1} = 12248.8\text{lb}$ $f_{l\_r_1} = 266.28\text{plf}$

Description: Lateral Analysis

LATERAL ANALYSIS, SUMMARY

At the base of the building

Front to back Direction Seismic_or_wind = "Seismic Governs"

Lateral force front-back, $F_{F\_B_O}$ 47715.79lb $f_{f\_b_o} = 1015.23\text{plf}$

Left to Right Direction Seismic_or_wind = "Seismic Governs

Lateral force front-back, $F_{L\_R_O} = 47715.79\text{lb}$ $f_{l\_r_o} = 1037.3\text{plf}$

Summary of the forces in the front to back

	Level No. $x =$	Height $h_x = \text{ft}$	Story Force $F_{F\_B_x} = \text{lb}$	Unit Story Force $f_{f\_b_x} = \text{lb} \cdot \text{ft}^{-1}$
Roof	2	19	35466.99	754.62
Floor	1	11	12248.8	260.61
Base	0	0	47715.79	1015.23

Summary of the forces in the left to right

	Level No. $x =$	Height $h_x = \text{ft}$	Story Force $F_{L\_R_x} = \text{lb}$	Unit Story Force $f_{l\_R_x} = \text{lb} \cdot \text{ft}^{-1}$
Roof	2	19	35466.99	771.02
Floor	1	11	12248.8	266.28
Base	0	0	47715.79	1037.3

Description: Slender Wall Design

BEARING WALL AT THE RESIDENCE at the second floor

Design data input

Wall thickness	$t_g = 10\text{in}$	Concrete Core Thickness	$t_c = 6\text{in}$
Wall height	$Ht = 10\text{ft}$	Equivalent Square Sides	
Rebar Size	$No = 5$	Rebar spacing Vertical	$spacing = 15\text{in}$

Rebar location Edge/Center (1/0)	$location = 0$	$S_{eq} = 4.75\text{in}$
Rebar depth	$d = 2.38\text{in}$	
Concrete Strength	$f_c = 2500\text{psi}$	
Modulus of rupture	$f_r = 250\text{psi}$	
Steel Strength	$F_y = 60\text{ksi}$	
Wall Weight Multiplier	$factor = 1.00$	
Wall Weight	$w_{wall} = 52\text{psf}$	
Effective strip Width	$w_{eff} = 15$	

Loads input

Vertical Load input

Dead Load $\quad w_{dl\_e} = \dfrac{20\text{ft}w_{dl\_r}}{2} \quad w_{dl\_e} = 200\text{plf}$

Live load $\quad w_{ll\_e} = \dfrac{20\text{ft}w_{ll\_r}}{2} \quad w_{ll\_e} = 160\text{plf}$

Eccentricity $\quad e = 6.25\text{in}$

Description: Slender Wall Design

Lateral Loads

Seismic $Z = 4$ $I = 1$ $C_p = .75$

Seismic Load Factor $v_p = ZIC_{p}{}_{w\,wall}$

Seismic Force on the wall $v_p = 15.6psf$

Additional Load

Point Load $P_1 = 0lb$ $P_2 = 0lb$

Height above Ground $h_1 = 10ft$ $h_2 = 0lb$

Uniform Load $w_{seis} = 0plf$

Height from Bottom $h_{bott} = 1ft$

Height to top $h_{top} = 10ft$

Wind

Speed $V = 80mph$ Exposure $\exp = C$ $C_q = 1.3$ $I = 1$

Pressure $q_s = 16.4psf$

Wind force on the wall $P = 22.6psf$

Additional Load

Point Load $P_1 = 0lb$ $P_2 = 0lb$

Height above Ground $h_c = 10ft$ $h_2 = 0lb$

Uniform Load $w_{wind} = 0plf$

Height from Bottom $h_{bott} = 1ft$

Height to Top $h_{top} = 10ft$

Description: Slender Wall Design

Check axial load limitation

$$\frac{p_w + p_o}{A_g} = 27.41\text{psi} \qquad \text{Must be <} \qquad .04f_c = 100\text{psi}$$

Factored loads

Seismic		Wind
$U = .75(1.4w_{dl} + 1.7w_{ll}(\text{floor}) + 1.87E)$		$U = .75(1.4w_{dl} + 1.7w_{ll}(\text{floor}) + 1.7W)$
	OR	
$U = .9w_{dl} + 1.43E$		$U = .9w_{dl} + 1.3W$

Factored load on the wall $\quad p_{ou} = 517.5\text{lb}$

Factored wall load at mid ht $\quad p_{wu} = 341.25\text{lb}$

Factored wall load at mid ht $\quad p_u = 858.75$

Factored Lateral Load $\quad w_u = 36.02$

Description: Slender Wall Design

BEARING WALL AT THE RESIDENCE BETWEEN THE WINDOWS

Design Data Input

Wall thickness, (Nominal) $t_g = 10in$ Concrete Core Thickness $t_c = 6in$

Wall height $Ht = 10ft$ Equivalent square sides $S_{eq} = 4.75in$

Rebar Size $No = 5$ Rebar spacing vertical $spacing = 15in$

Rebar location Edge/Center (1/0) $location = 0in$

Rebar Depth $d = 2.38in$

Concrete Strength $f_c = 2500psi$

Modulus of Rupture $f_r = 250psi$

Steel Strength $F_y = 60ksi$

Wall Weight Multiplier $factor = 1.00$

Wall Weight $w_{wall} = 52psf$

Effective strip width $w_{eff} = 15$

Loads Input

Dead Load $w_{dl_e} = \left(\frac{20ft w_{dl\_r}}{2}\right) 6 \frac{ft}{2 \cdot 2.5ft}$ $w_{dl_e} = 240plf$

Live load $w_{ll_e} = \frac{20ft w_{ll\_r}}{2} 6 \frac{ft}{2 \cdot 2.5ft}$ $w_{ll_e} = 192plf$

Eccentricity $e = 6.25in$

Description: Slender Wall Design

Lateral Loads

Seismic $Z = 4$ $I = 1$ $C_p = .75$

Seismic load Factor $ZIC_p = .3$

Seismic Force on the wall $v_p = ZIC_{p\, w_{wall}}$ $v_p = 15.6\text{psf}$

Additional Load

Point Load $P_1 = 0\text{lb}$ $P_2 = 0\text{lb}$

Height Above Ground $h_1 = 10\text{ft}$ $h_2 = 0\text{lb}$

Uniform Load $w_{seis} = 0\text{plf}$

Height from Bottom $h_{bott} = 1\text{ft}$

Height to Top $h_{top} = 10\text{ft}$

Wind

Speed $V = 80\text{mph}$ Exposure $\exp = C$ $C_q = 1.3$ $I = 1$

Pressure $q_s = 16.4\text{psf}$

Wind force on the wall $P = 22.6\text{psf}$

Additional Load

Point Load $P_1 = 0\text{lb}$ $p_2 = 0\text{lb}$

Height Above Ground $h_c = 10\text{ft}$

Uniform Load $w_{wind} = 0\text{plf}$

Height from Bottom $h_{bott} = 1\text{ft}$

Height to Top $h_{top} = 10\text{ft}$

Example: Slender Wall Design

Check axial load limitation

$\frac{p_w + p_o}{A_g} = 22.46\text{psi}$ Must be $< .04f_c = 100\text{psi}$

Factored Loads

Seismic

$U = .75(1.4w_{dl} + 1.7w_{ll}(\text{floor}) + 1.87E)$

$U = .9w_{dl} + 1.43E$

OR

Wind

$U = .75(1.4w_{dl} + 1.7w_{ll}(\text{floor}) + 1.7W)$

$U = .9w_{dl} + 1.3W$

Factored Load on the wall $p_{ou} = 2071\text{lb}$

Factored wall load at mid ht $p_{wu} = 477.75\,\text{lb}$

Factored wall load at mid ht $p_u = 684.75\text{lb}$

Factored Lateral Load $w_u = 36.02\text{plf}$

Description: Slender Wall Design

Cracked Section Analysis

$$A_{se} = \frac{(p_u + A_s F_y)}{F_y} \qquad A_{se} = .32\text{in}^2$$

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad M_{cr} = 6.69\text{in} \cdot \text{kips}$$

$$I_{cr} = \left[nA_{se}(d-c)^2\right] + \frac{s_{eq}c^3}{3} \qquad I_{cr} = 8.32\text{in}^4$$

$$\Delta_{cr} = \frac{5M_{cr}Ht^2}{48E_cI_g} \qquad \Delta_{cr} = .11\text{in}$$

Service Moment

$$M_s = \frac{w_{st}Ht^2}{8} + p_o\frac{e}{2} + (p_o + p_w)\Delta_s \qquad M_s = 10.33\text{in} \cdot \text{kips}$$

$$\Delta_s = \frac{5M_sHt^2}{48E_cI_g} + \frac{5(M_s - M_{cr})Ht^2}{48E_cI_{cr}} \qquad \Delta_s = 62\text{in}$$

Nominal Moment Strength $\phi = 7.2$

$$M_n = (A_{se}F_y)\left(d - \frac{a}{2}\right)$$

$$M_n = 38.22\text{in} \cdot \text{kips} \qquad \phi M_n = 27.52\text{in} \cdot \text{kips}$$

$$\Delta_n = \frac{5M_nHt^2}{48E_cI_{cr}} \qquad \Delta_n = 4.74\text{in}$$

Example: Slender Wall Design

Factored Moment

$$M_u = \frac{w_u Ht^2}{8} + p_u \frac{e}{2} + (p_{ou} + p_{wu})\Delta_n$$

$M_u = 1331.1 lbft$ Must be < $\phi M_n = 2292.98 lbft$

Check deflection at service load

$M_s = 10.33 in \cdot kips$ $M_n = 38.22 in \cdot kips$

$\Delta_s = \Delta_{cr} + \frac{M_s - M_{cr}}{M_n - M_{cr}}(\Delta_n - \Delta_{cr})$ $\Delta_s = .64 in$ Must be < $\frac{Ht}{150} = 1.12 in$

Example: Slender Wall Design

Design Data Input

Wall thickness, (Nominal) $t_g = 10in$ Concrete Core Thickness $t_c = 6in$

Wall height $Ht = 10ft$ Equivalent square sides $S_{eq} = 4.75in$

Rebar Size $No = 5$ Rebar spacing vertical $spacing = 15in$

Rebar location Edge/Center (1/0) $location = 0in$

Rebar Depth $d = 2.38in$

Concrete Strength $f_c = 2500psi$

Modulus of Rupture $f_r = 250psi$

Steel Strength $F_y = 60ksi$

Wall Weight Multiplier $factor = 1.00$
Wall Weight $w_{wall} = 52psf$

Effective strip width $w_{eff} = 15$

Loads Input

Loads Input

Dead Load

$w_{dl\_e} = 916plf$ $$w_{dl\_e} = \frac{20ft\left(w_{dl\_r} + w_{dl\_f}\right)}{2} + H_{wall\_rf} w_{wall}$$

Live load

$w_{ll\_e} = 560plf$ $$w_{ll\_e} = \frac{20ft\left(w_{ll\_r} + w_{ll\_f}\right)}{2}$$

Eccentricity $e = 6.25in$

Example: Slender Wall Design

Design Data Input

Wall thickness, (Nominal) $t_g = 10\text{in}$ Concrete Core Thickness $t_c = 6\text{in}$

Wall height $Ht = 10\text{ft}$ Equivalent square sides $S_{eq} = 4.75\text{in}$

Rebar Size $No = 5$ Rebar spacing vertical
$spacing = 15\text{in}$

Rebar location Edge/Center (1/0) $location = 0\text{in}$

Rebar Depth $d = 2.38\text{in}$

Concrete Strength $f_c = 2500\text{psi}$

Modulus of Rupture $f_r = 250\text{psi}$

Steel Strength $F_y = 60\text{ksi}$

Wall Weight Multiplier $factor = 1.00$
Wall Weight $w_{wall} = 52\text{psf}$

Effective strip width $w_{eff} = 15$

Loads Input

Loads Input

Dead Load

$w_{dl\_e} = 916\text{plf}$

$$w_{dl\_e} = \frac{20\text{ft}\left(w_{dl\_r} + w_{dl\_f}\right)}{2} + H_{wall\_rf} w_{wall}$$

Live load

$w_{ll\_e} = 560\text{plf}$

$$w_{ll\_e} = \frac{20\text{ft}\left(w_{ll\_r} + w_{ll\_f}\right)}{2}$$

Eccentricity $e = 6.25\text{in}$

Description: Slender Wall Design

Lateral Loads

Seismic $Z = 4$ $I = 1$ $C_p = .75$

Seismic load Factor $ZIC_p = .3$

Seismic Force on the wall $v_p = ZIC_{p\,w_{wall}}$ $v_p = 15.6\text{psf}$

Additional Load

Point Load $P_1 = 0\text{lb}$ $P_2 = 0\text{lb}$

Height Above Ground $h_1 = 10\text{ft}$ $h_2 = 0\text{lb}$

Uniform Load $w_{seis} = 0\text{plf}$

Height from Bottom $h_{bott} = 1\text{ft}$

Height to Top $h_{top} = 10\text{ft}$

Wind

Speed $V = 80\text{mph}$ Exposure $\exp = C$ $C_q = 1.3$ $I = 1$

Pressure $q_s = 16.4\text{psf}$

Wind force on the wall $P = 22.6\text{psf}$

Additional Load
Point Load $P_1 = 0\text{lb}$ $p_2 = 0\text{lb}$

Height Above Ground $h_c = 10\text{ft}$

Uniform Load $w_{wind} = 0\text{plf}$

Height from Bottom $h_{bott} = 1\text{ft}$

Height to Top $h_{top} = 10\text{ft}$

Description: Slender Wall Design

Check axial load limitation

$\frac{p_w + p_o}{A_g} = 76.75\text{psi}$ Must be $< .04f_c = 100\text{psi}$

Factored Loads

Seismic

$U = .75(1.4w_{dl} + 1.7w_{ll}(\text{floor}) + 1.87E)$

$U = .9w_{dl} + 1.43E$

OR

Wind

$U = .75(1.4w_{dl} + 1.7w_{ll}(\text{floor}) + 1.7W)$

$U = .9w_{dl} + 1.3W$

Factored Load on the wall $p_{ou} = 2094.751\text{lb}$

Factored wall load at mid ht $p_{wu} = 341.25\text{lb}$

Factored wall load at mid ht $p_u = 2436\text{lb}$

Factored Lateral Load $w_u = 36.02\text{plf}$

Description: Slender Wall Design

Cracked Section Analysis

$$A_{se} = \frac{(p_u + A_s F_y)}{F_y} \qquad A_{se} = .35\text{in}^2$$

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad M_{cr} = 6.69\text{in} \cdot \text{kips}$$

$$I_{cr} = \left[nA_{se}(d-c)^2\right] + \frac{s_{eq}c^3}{3} \qquad I_{cr} = 8.48\text{in}^4$$

$$\Delta_{cr} = \frac{5M_{cr}Ht^2}{48E_cI_g} \qquad \Delta_{cr} = .06\text{in}$$

Service Moment

$$M_s = \frac{w_{st}Ht^2}{8} + p_o\frac{e}{2} + (p_o + p_w)\Delta_s \qquad M_s = 14.99\text{in} \cdot \text{kips}$$

$$\Delta_s = \frac{5M_sHt^2}{48E_cI_g} + \frac{5(M_s - M_{cr})Ht^2}{48E_cI_{cr}} \qquad \Delta_s = .14\text{in}$$

Nominal Moment Strength $\quad \phi = .72$

$$M_n = (A_{se}F_y)\left(d - \frac{a}{2}\right)$$

$$M_n = 41.01\text{in} \cdot \text{kips} \qquad \phi M_n = 29.52\text{in} \cdot \text{kips}$$

$$\Delta_n = \frac{5M_nHt^2}{48E_cI_{cr}} \qquad \Delta_n = 2.55\text{in}$$

Description: Slender Wall Design

Factored Moment

$$M_u = \frac{w_u Ht^2}{8} + p_u \frac{e}{2} + (p_{ou} + p_{wu})\Delta_n$$

$M_u = 1601.25\text{lbft}$ Must be < $\phi M_n = 2460.36\text{lbft}$

Check deflection at service load

$M_s = 14.99\text{in} \cdot \text{kips}$ $M_n = 41.01\text{in} \cdot \text{kips}$

$$\Delta_s = \Delta_{cr} + \frac{M_s - M_{cr}}{M_n - M_{cr}}(\Delta_n - \Delta_{cr})$$ $\Delta_s = .66\text{in}$ Must be < $\frac{Ht}{150} = .8\text{in}$

Description: Slender Wall Design

BEARING WALL AT THE RESIDENCE FIRST FLOOR BTWN WINDOW PIER

Design Data Input

Wall thickness, (Nominal) $t_g = 10in$ Concrete Core Thickness $t_c = 6in$

Wall height $Ht = 10ft$ Equivalent square sides $S_{eq} = 4.75in$

Rebar Size $No = 5$ Rebar spacing vertical $spacing = 15in$

Rebar location Edge/Center (1/0) $location = 0in$

Rebar Depth $d = 2.38in$

Concrete Strength $f_c = 2500psi$

Modulus of Rupture $f_r = 250psi$

Steel Strength $F_y = 60ksi$

Wall Weight Multiplier $factor = 1.00$

Wall Weight $w_{wall} = 52psf$

Effective strip width $w_{eff} = 15$

Loads Input

Loads Input

Dead Load

$$w_{dl} = 2ft\left(w_{dl\_r} + w_{dl\_f}\right) + 4ftw_{wall}$$

Live load

$$w_{ll} = 2\left(w_{ll\_r} + w_{ll\_f}\right)$$

Description: Slender Wall Design

Lateral Loads

Seismic $Z = 4$ $I = 1$ $C_p = .75$

Seismic load Factor $ZIC_p = .3$

Seismic Force on the wall $v_p = ZIC_{p w_{wall}}$ $v_p = 15.6psf$

Additional Load

Point Load $P_1 = 0lb$ $P_2 = 0lb$

Height Above Ground $h_1 = 10ft$ $h_2 = 0lb$

Uniform Load $w_{seis} = 0plf$

Height from Bottom $h_{bott} = 1ft$

Height to Top $h_{top} = 10ft$

Wind

Speed $V = 80mph$ Exposure $exp = C$ $C_q = 1.3$ $I = 1$

Pressure $q_s = 16.4psf$

Wind force on the wall $P = 22.6psf$

Additional Load

Point Load $P_1 = 0lb$ $p_2 = 0lb$

Height Above Ground $h_c = 10ft$

Uniform Load $w_{wind} = 0plf$

Height from Bottom $h_{bott} = 1ft$

Height to Top $h_{top} = 10ft$

Description: Slender Wall Design

Check axial load limitation

$$\frac{p_w + p_o}{A_g} = 99.24\text{psi} \qquad \text{Must be} < .04f_c = 100\text{psi}$$

Factored Loads

Seismic

$$U = .75(1.4w_{dl} + 1.7w_{ll}(\text{floor}) + 1.87E)$$

$$U = .9w_{dl} + 1.43E$$

OR

Wind

$$U = .75(1.4w_{dl} + 1.7w_{ll}(\text{floor}) + 1.7W)$$

$$U = .9w_{dl} + 1.3W$$

Factored Load on the wall $p_{ou} = 2757.22\text{lb}$

Factored wall load at mid ht $p_{wu} = 341.25\text{lb}$

Factored wall load at mid ht $p_u = 3098.47\text{lb}$

Factored Lateral Load $w_u = 36.02\text{plf}$

Description: Slender Wall Design

Cracked Section Analysis

$$A_{se} = \frac{(p_u + A_s F_y)}{F_y} \qquad A_{se} = .36\text{in}^2$$

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad M_{cr} = 6.69\text{in} \cdot \text{kips}$$

$$I_{cr} = \left[nA_{se}(d-c)^2\right] + \frac{s_{eq}c^3}{3} \qquad I_{cr} = 8.53\text{in}^4$$

$$\Delta_{cr} = \frac{5M_{cr}Ht^2}{48E_cI_g} \qquad \Delta_{cr} = .06\text{in}$$

Service Moment

$$M_s = \frac{w_{st}Ht^2}{8} + p_o\frac{e}{2} + (p_o + p_w)\Delta_s \qquad M_s = 18.44\text{in} \cdot \text{kips}$$

$$\Delta_s = \frac{5M_sHt^2}{48E_cI_g} + \frac{5(M_s - M_{cr})Ht^2}{48E_cI_{cr}} \qquad \Delta_s = .88\text{in}$$

Nominal Moment Strength $\qquad \phi = .72$

$$M_n = (A_{se}F_y)\left(d - \frac{a}{2}\right)$$

$$M_n = 42.03\text{in} \cdot \text{kips} \qquad \phi M_n = 30.26\text{in} \cdot \text{kips}$$

$$\Delta_n = \frac{5M_nHt^2}{48E_cI_{cr}} \qquad \Delta_n = 2.59\text{in}$$

Description: Slender Wall Design

Factored Moment

$$M_u = \frac{w_u Ht^2}{8} + p_u \frac{e}{2} + (p_{ou} + p_{wu})\Delta_n$$

$M_u = 1926.62 \text{lbft}$ Must be < $\phi M_n = 2521.79 \text{lbft}$

Check deflection at service load

$M_s = 18.44 \text{in} \cdot \text{kips}$ $M_n = 42.03 \text{in} \cdot \text{kips}$

$$\Delta_s = \Delta_{cr} + \frac{M_s - M_{cr}}{M_n - M_{cr}}(\Delta_n - \Delta_{cr})$$

$\Delta_s = .9 \text{in}$ Must be < $\frac{Ht}{150} = .8 \text{in}$

Example: Lintel Design

Analysis

$U = (1.4w_{dl} + 1.7w_{ll})$

$$M_u = 1926.62 lb \cdot ft$$

Design;

Given $M_u = .9bd^2 F_y \rho \left(1 - .59\rho \frac{F_y}{f_c}\right)$ $\rho = \mathrm{Find}(\rho)$ $\rho = .0004$

$A_{s\_req} = \rho bd$ $A_{s\_req} = .03 in^2$ Reinf. Size $\mathrm{Reinf} = 5$

Use No of reinf $No = 1$

Description: Lintel Design

LINTEL AT THE ROOF WINDOW HEADER, BEARING

Design Data Input

Wall thickness, (Nominal) $t_g = 10in$ Concrete Core Thickness $t_c = 6in$

Lintel Length $L = 7.5ft$ Equivalent square sides $b = 4.75in$

Lintel Height $h = 30in$ Rebar depth $d = 15in$

Rebar Size $No = 5$

Concrete Strength $f_c = 2500psi$

Modulus of Rupture $f_r = 250psi$

Steel Strength $F_y = 60ksi$

Wall Weight Multiplier $factor = 1.00$

Wall Weight $w_{wall} = 52psf$

Effective strip width $w_{eff} = 15$

Loads Input

Vertical load input

Dead Load

$$w_{dl} = \frac{20ft w_{dl\_r}}{2}$$

Live load

$$w_{ll} = \frac{20ft w_{ll\_r}}{2}$$

Concentric Load

Dead Load

$$p_{dl\_c} = 0lb$$

Live Load

$$p_{ll\_c} = 0lb$$

Example: Lintel Design

Analysis

$$U = (1.4w_{dl} + 1.7w_{ll})$$

$$M_u = 3881.25\,lb \cdot ft$$

Design;

Given $M_u = .9bd^2F_y\rho\left(1 - .59\rho\frac{F_y}{f_c}\right)$ $\rho = Find(\rho)$ $\rho = .0008$

$A_{s\_req} = \rho bd$ $A_{s\_req} = .06in^2$ Reinf. Size $Reinf = 5$

Use No of reinf $No = 1$

Description: Lintel Design

LINTEL AT THE ROOF WINDOW HEADER, BEARING

Design Data Input

Wall thickness, (Nominal) $t_g = 10in$ Concrete Core Thickness $t_c = 6in$

Lintel Length $L = 7.5ft$ Equivalent square sides $b = 4.75in$

Lintel Height $h = 30in$ Rebar depth $d = 15in$

Rebar Size $No = 5$

Concrete Strength $f_c = 2500psi$

Modulus of Rupture $f_r = 250psi$

Steel Strength $F_y = 60ksi$

Wall Weight Multiplier $factor = 1.00$

Wall Weight $w_{wall} = 52psf$

Effective strip width $w_{eff} = 15$

Loads Input

Vertical load Input

Dead Load $$w_{dl} = \left[\frac{20ft\left(w_{dl\_r} + w_{ll\_r}\right)}{2} + w_{wall}5ft\right]$$

Live Load $$w_{ll} = \frac{20ft\left(w_{ll\_r} + w_{ll\_f}\right)}{2}$$

Example: Lintel Design

Analysis

$U = (1.4w_{dl} + 1.7w_{ll})$

$M_u = 8015.63 lb \cdot ft$

Design;

Given $M_u = .9bd^2F_y\rho\left(1 - .59\rho\frac{F_y}{f_c}\right)$ $\rho = Find(\rho)$ $\rho = .0017$

$A_{s\_req} = \rho bd$ $A_{s\_req} = .12in^2$ Reinf. Size $Reinf = 5$

Use No of reinf $No = 1$

Description: Shear Wall Design

SHEAR WALL AT THE SIDE OF THE HOUSE AT THE SECOND FLOOR

Wall weight $w_{wall} = 52psf$ Tributary Width $b = 4.75in$

Thickness, nominal $d = 4.5in$ Reinf location center (c), edge/ Location= c

Wall length $l_{wall} = 6ft$ $d_{wall} = .8l_{wall}$ $d_{wall} = 4.8ft$

Wall height $h_p = 10ft$

Perform Wall data Special inspection (y/n) answer = no

$f_c = 2500psi$ $\phi = .85$

$F_y = 60000psi$

Lateral Loads

Seismic Z=4 I=1 C=1 $R_w = 6$

Pier seismic force $v_p = \frac{F_{F\_B_2}}{6}$

Additional force on the pier, V = 0 lb

Distance from the base of the wall H_{wall_rf}

Description: Shear Wall Design

Forces

Reaction $R = V + v_p$ $R = 5911.17lb$

Moment $M = Vx + v_p \dfrac{h_p}{2}$ $M = 29.56ft \cdot kips$

Axial $P_{axial} = h_{p^{bw_{wall}}}$ $P_{axial} = 205.92$

Shear Strength factor= 7 $2\phi\sqrt{f_c psi}bd = 3.13kips$

$$v_c = \left[\left(2\phi\sqrt{f_c psi}bd\right)\frac{l_{wall}}{15in}\right]$$ $V_c = 15.03kips$

Reinforce, req $d = .65ft$ $F_y = 60ksi$ $\rho = .01$

$b = .4ft$ $f_c = 2.5ksi$

Design;

$M_u = 1.7M$ $M_u = 50.24kips \cdot ft$

Boundary Reinforcement

Rebar Size Reinf= 5 No of Reinf No=2

Allowable Moment $M_n = .9s_{eq^d}wall^2Fy\rho\left(1 - .59\rho\dfrac{F_y}{f_c}\right)$

$M_n = 1066.53ft \cdot kips$ Result="Bending O.K." Use No=2 Reinf=5

Description: Shear Wall Design

INTERIOR SHEAR WALL AT THE ROOF LEVEL

Wall weight $w_{wall} = 52psf$ Tributary Width $b = 4.75in$

Thickness, nominal $d = 4.5in$ Reinf location center (c), edge/ Location= c

Wall length $l_{wall} = 10ft$ $d_{wall} = .8 l_{wall}$ $d_{wall} = 8ft$

Wall height $h_p = 10ft$

Perform Wall data Special inspection (y/n) answer = no

$f_c = 2500psi$ $\phi = .85$

$F_y = 60000psi$

Lateral Loads

Seismic Z=4 I=1 C=1 $R_w = 6$

Pier seismic force $v_p = \dfrac{F_{F\_B_2}}{2}$

Additional force on the pier, V = 0 lb

Distance from the base of the wall $x = 10ft$

Description: Shear Wall Design

Forces

Reaction $R = V + v_p$ $R = 17733.5\text{lb}$

Moment $M = Vx + v_p \dfrac{h_p}{2}$ $M = 88.67\text{ft} \cdot \text{kips}$

Axial $P_{axial} = h_{p^{bw\,wall}}$ $P_{axial} = 205.92$

Shear Strength factor= 7 $2\phi\sqrt{f_c psi}bd = 3.13\text{kips}$

$$v_c = \left[\left(2\phi\sqrt{f_c psi}bd\right)\frac{l_{wall}}{15in}\right]$$ $V_c = 25.04\text{kips}$

Reinforce, req $d = .65\text{ft}$ $F_y = 60\text{ksi}$ $\rho = .01$

$b = .4\text{ft}$ $f_c = 2.5\text{ksi}$

Design;

$M_u = 1.7M$ $M_u = 150.73\text{kips} \cdot \text{ft}$

Boundary Reinforcement

Rebar Size Reinf= 5 No of Reinf No=2

Allowable Moment $M_n = .9s_{eq^d}\,\text{wall}^2 Fy\rho\left(1 - .59\rho\dfrac{F_y}{f_c}\right)$

$M_n = 2962.58\text{ft} \cdot \text{kips}$ Result="Bending O.K." Use No=2 Reinf=5

Description: Shear Wall Design

THE FIRST LEVEL AT THE SIDE OF THE HOUSE

Wall weight $w_{wall} = 52psf$ Tributary Width $b = 4.75in$

Thickness, nominal $d = 4.5in$ Reinf location center (c), edge/ Location= c

Wall length $l_{wall} = 11.3ft$ $d_{wall} = .8l_{wall}$ $d_{wall} = 9.06ft$

Wall height $h_p = 10ft$

Perform Wall data Special inspection (y/n) answer = no

$f_c = 2500psi$ $\phi = .85$

$F_y = 60000psi$

Lateral Loads

Seismic Z=4 I=1 C=1 $R_w = 6$

Pier seismic force $v_p = \frac{F_{F\_B_2}}{2}$

Additional force on the pier, V = 0 lb

Distance from the base of the wall $x = 10ft$

Description: Shear Wall Design

Forces

Reaction $R = V + v_p$ $R = 47715.79\text{lb}$

Moment $M = Vx + v_p \frac{h_p}{2}$ $M = 238.58\text{ft}\cdot\text{kips}$

Axial $P_{axial} = h_{p^{bw\,wall}}$ $P_{axial} = 205.92$

Shear Strength factor= 7 $2\phi\sqrt{f_c \text{psi}}bd = 3.13\text{kips}$

$v_c = \left[\left(2\phi\sqrt{f_c\text{psi}}bd\right)\frac{l_{wall}}{15\text{in}}\right]$ $V_c = 28.37\text{kips}$

Reinforce, req $d = .65\text{ft}$ $F_y = 60\text{ksi}$ $\rho = .01$

$b = .4\text{ft}$ $f_c = 2.5\text{ksi}$

Design;

$M_u = 1.7M$ $M_u = 405.58\text{kips}\cdot\text{ft}$

Boundary Reinforcement

Rebar Size Reinf= 5 No of Reinf No=2

Allowable Moment $M_n = .9s_{eq^d}\text{wall}^2 Fy\rho\left(1 - .59\rho\frac{F_y}{f_c}\right)$

$M_n = 3803.02\text{ft}\cdot\text{kips}$ Result="Bending O.K." Use No=2 Reinf=5

Description: Shear Wall Design

Wall weight $w_{wall} = 52psf$ Tributary Width $b = 4.75in$

Thickness, nominal $d = 4.5in$ Reinf location center (c), edge/ Location= c

Wall length $l_{wall} = 1.25ft$ $d_{wall} = .8l_{wall}$ $d_{wall} = 1ft$

Wall height $h_p = 10ft$

Perform Wall data Special inspection (y/n) answer = no

$f_c = 2500psi$ $\phi = .85$

$F_y = 60000psi$

Lateral Loads

Seismic Z=4 I=1 C=1 $R_w = 6$

Pier seismic force $v_p = 1kips$

Additional force on the pier, V = 0 lb

Distance from the base of the wall $x = 10ft$

Description: Shear Wall Design

Forces

Reaction $R = V + v_p$ $R = 1000lb$

Moment $M = Vx + v_p \frac{h_p}{2}$ $M = 5ft \cdot kips$

Axial $P_{axial} = h_{p^{bw\,wall}}$ $P_{axial} = 205.92$

Shear Strength factor= 7 $2\phi\sqrt{f_c psi} bd = 3.13kips$

$v_c = \left[\left(2\phi\sqrt{f_c psi} bd \right) \frac{l_{wall}}{15in} \right]$ $V_c = 3.13kips$

Reinforce, req $d = .65ft$ $F_y = 60ksi$ $\rho = .01$

$b = .4ft$ $f_c = 2.5ksi$

Design;

$M_u = 1.7M$ $M_u = 8.5kips \cdot ft$

Boundary Reinforcement

Rebar Size Reinf= 5 No of Reinf No=2

Allowable Moment $M_n = .9s_{eq^d} wall^2 Fy\rho \left(1 - .59\rho \frac{F_y}{f_c} \right)$

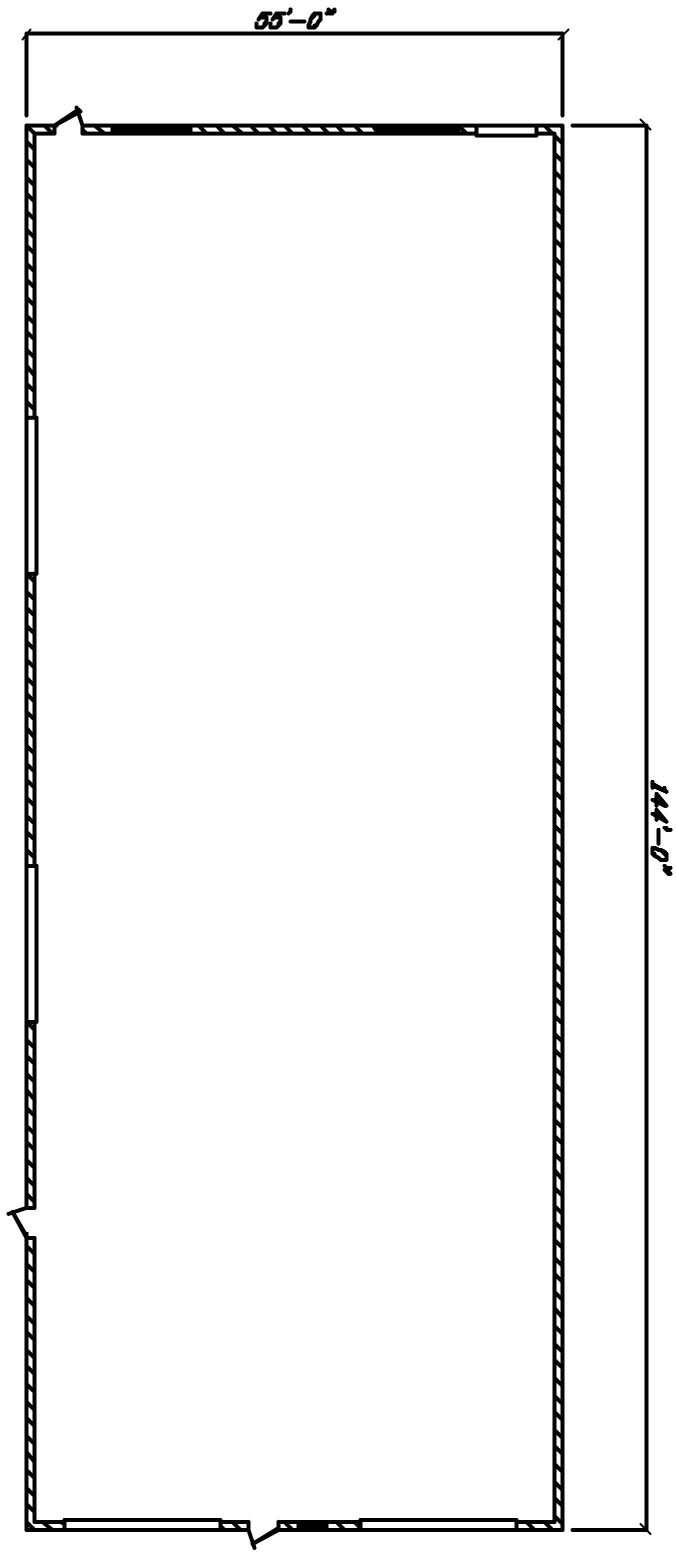
$M_n = 2962.58ft \cdot kips$ Result="Bending O.K." Use No=2 Reinf=5

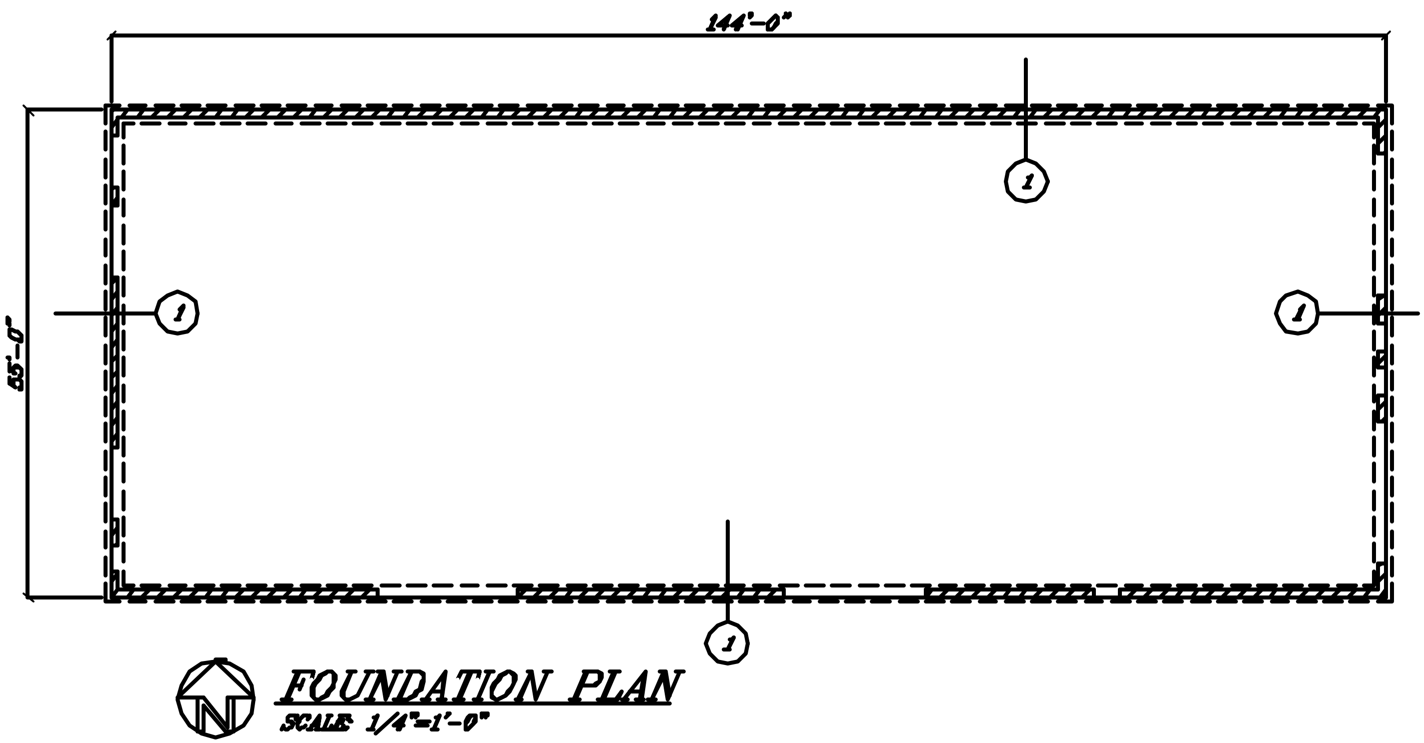
SECTION 6

Design of a Single Story Industrial Building

FLOOR PLAN

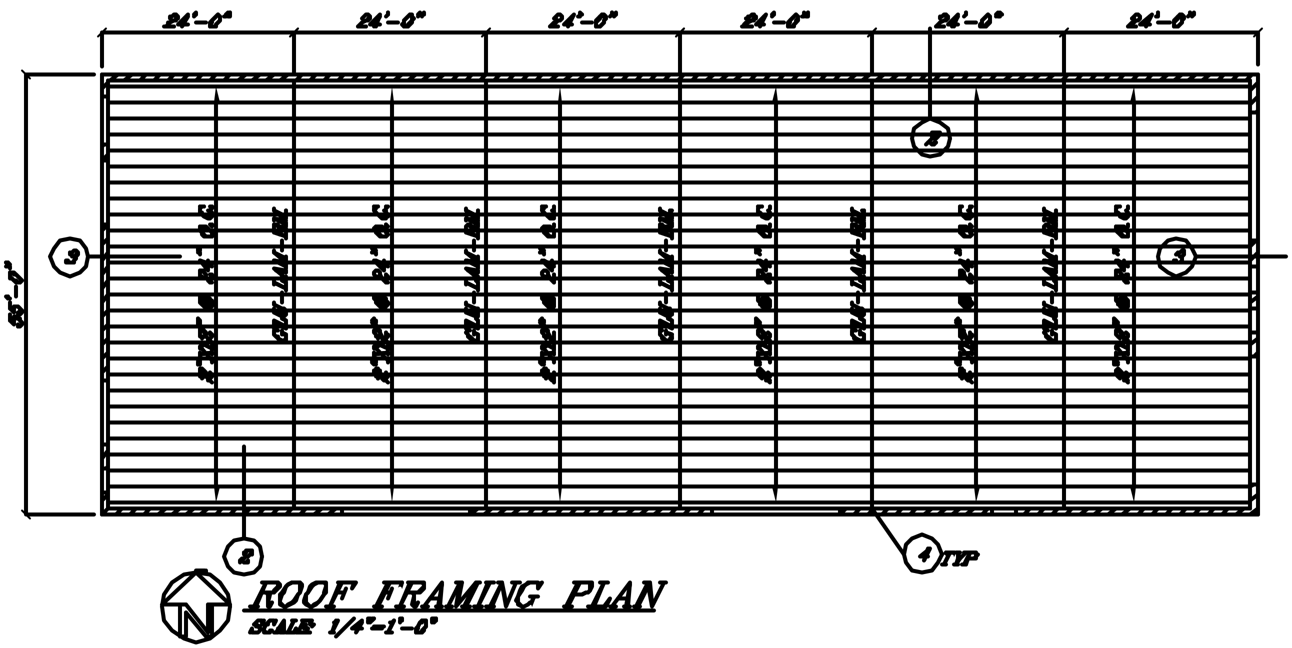
SCALE: 1/16"=1'-0"





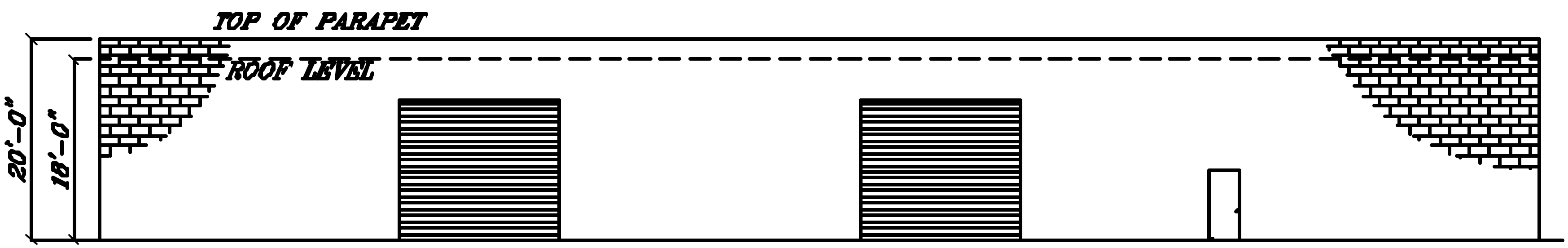
FOUNDATION PLAN

SCALE: 1/4"=1'-0"

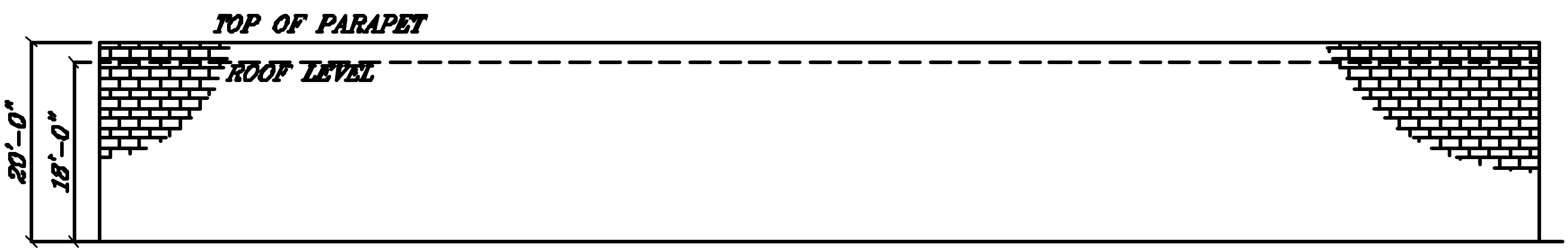


ROOF FRAMING PLAN

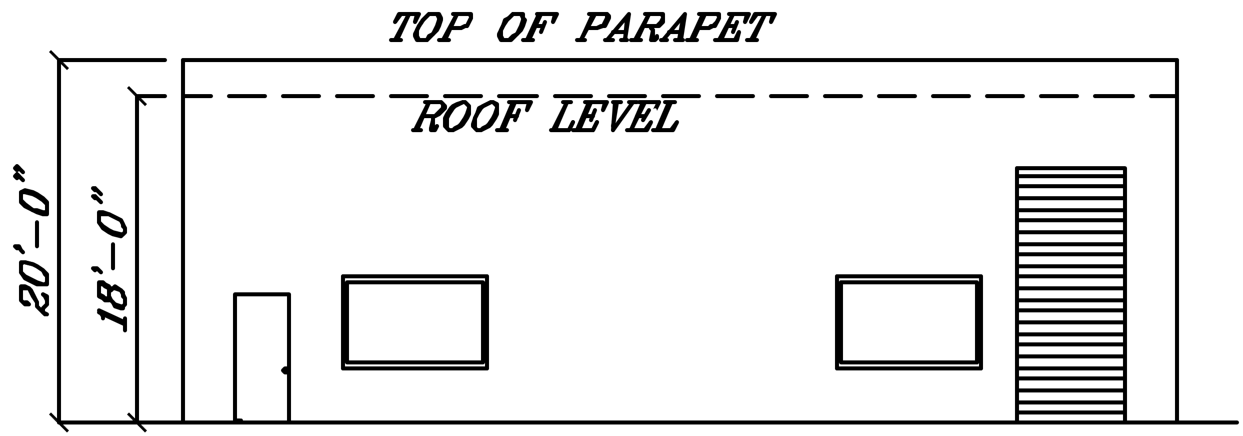
SCALE: 1/4"=1'-0"



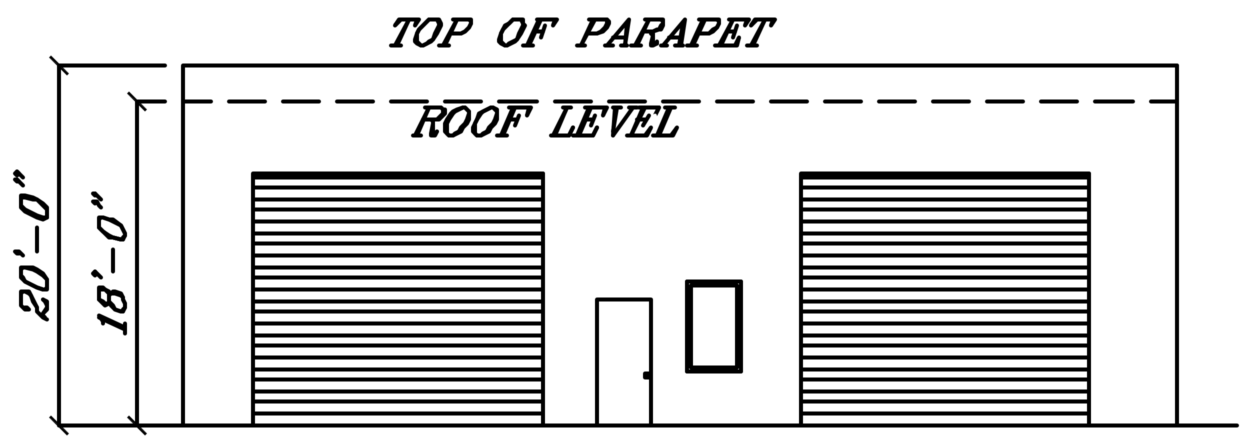
SOUTH ELEVATION



NORTH ELEVATION



WEST ELEVATION



EAST ELEVATION

Description: Typical Industrial Building

LOADING CRITERIA

Roof Load:

	Stiffener	Purlins	Girders and Seismic
Roofing,	Rf= 6 psf	Rf= 6.0 psf	RF=6.0 psf
Sheathing	St= 1.5 psf	St= 1.5 psf	St=1.5 psf
Framing, Stiff	$Fr_s = 1.0psf$	$Fr_s = 1.0psf$	$Fr_s = 1.0psf$
Framing, Purl		$Fr_p = 2.5psf$	$Fr_p = 2.5psf$
Framing, BM			
Framing, Gir			$Fr_g = 3.0psf$
Ceiling,	Cl= 0 psf	Cl= 2.2 psf	Cl= 2.2 pf
Insulation	In= 0 psf	In= 1 psf	In= 1 psf
Elect/Mech	Ee= 0 psf	Ee=1.0 psf	Ee= 1.0 psf
Sprinkler	Sp= 0 psf	Sp= 1.0 psf	Sp=1.0 psf
misc	$Ms_s = .5psf$	Ms= 1.0 psf	Ms= 1.0 psf
Total Dead Load,	$w_{d\_s} = 9psf$	$w_{d\_p} = 18psf$	$w_{d\_g} = 20psf$
Total Live Load,	$w_{l\_s} = 20psf$	$w_{l\_p} = 20psf$	$w_{l\_g} = 12psf$

Description: Lateral Analysis

Roof Load:
1-Dead Load

Roofing,	Built up/ Concrete Tile/ Shingle	Rf=20	$Rf_{wt} = 5.5psf$
Sheathing, plywood, thickness (t)		t= .5 in	St= 1.5 psf
Framing, framing			Fr=2.20 psf

Ceiling, Plaster=p
Suspend'd=s
Gypsum=g For gypsum give thickness $t_{clg} = .5in$
No ceiling= n
Ceiling Type $clg_{type} = "s"$

Ceiling Weight $Clg_{wt} = 1.8psf$

Insulation 8" bat. insulation	In=1 psf
Electrical/Mechanical	Ee=1 psf
misc.	Ms=1.5 psf

Total Dead Load

Roof pitch	$w_{dl\_r} = 15psf$
Live Load	$w_{ll\_r} = 16psf$
Snow Load	$w_{snow} = 0psf$
Wall weight	$w_{wall} = 15psf$
Partition Wall load	$w_{par} = 0psf$

$$pitch_{factor} = \frac{\sqrt{0^2 + 12^2}}{12}$$

$pitch_{factor} = 1.08$

Description: Lateral Analysis

Building Data:

Length $L_r = 144\text{ft}$ Width $b_r = 55\text{ft}$

Roof Weight $w_r = w_{dl^{pitch_{factor}}}$ Fascia height $F_h = 5.5\text{in}$

Roof/Floor ht $H_{wall} = 8\text{ft}$ Wall weight $w_{wall} = 15\text{psf}$

Roof Pitch 0= 5/12 Parapet Height $H_p = 0\text{ft}$

Gable/ Parapet height $h_g = 11.92\text{ft}$

Roof Height including parapet/or sloped roof $H_{tot} = 19.92\text{ft}$

Overhang Length $L_{oh} = 2\text{ft}$

Roof Weight $W_{roof} = \left[\left(L_r + L_{oh^2}\right)\left(b_r + L_{oh^2}\right)\right]w_r$ $w_{roof} = 137165.17\text{lb}$

Reduction for snow for seismic load reduction = 75%

Snow load $W_{snow} = (L_r b_r)\left[w_{snow}(100\% - \text{reduction})\right]$ $W_{snow} = 0\text{lb}$

Wall weight

With Parapet $W_{wall} = w_{wall}\dfrac{Hr^2}{2h_r}$ No Parapet $W_{wall} = W_{wall}\left(\dfrac{h_r}{2} + h_g\right)$

Long Direction $W_{wall\_F\_B} = 68760\text{lb}$ Short Direction $W_{wall\_L\_R} = 26262.5\text{lb}$

Partition wall/ Interior wall weight $W_{par} = (L_r b_r)\dfrac{w_{par}}{2}$ $W_{par} = 0\text{lb}$

Total load applied at roof level

Long Direction $W_{r\_F\_B} = 205925.17\text{lb}$ Short Direction $W_{r\_L\_R} = 163427.67\text{lb}$

Description: Lateral Analysis

Seismic Analysis

Seismic Zone Factor Z=4

Building Importance Factor I=1

"R" Coefficient R=4.5

Lateral Resistance System:
Mas. or Conc. Shear wall/ Steel MSRF/ Concrete MSRF/ Others (1/2/3) Lrs=1

For concrete/masonry shear wall system enter: $A_e = 12^2 in^2 2$ $D_e = 15ft$

Building Period $T = C_t H_{wall}^{\frac{3}{4}}$ T= .33 seconds

Distance from site to the source: distance=16km

Seismic fault type fault= B

Near Source Factor $N_a = 1$ $N_v = 1$

Seismic Force Coefficients $C_a = .44N_a$ $C_v = .64N_v$

$coef_1 = \dfrac{2.5C_a I}{R}$ $coef_1 = .24$

$coef_2 = .11C_a I$ $coef_2 = .05$

$coef_3 = \dfrac{.8N_v I}{R}$ $coef_3 = .18$

Seismic coefficient coef= .2444

Description: Lateral Analysis

Seismic Force

Total Force= W.coef Uniform Load= W. coef/ (L or B)

Long Direction $V_{F\_B} = 50337.26lb$ $v_{f\_b} = 349.56plf$

Short Direction $V_{L\_R} = 39948.99lb$ $v_{l\_r} = 726.35plf$

Description: Lateral Analysis

Wind Analysis

Wind speed, V=70 mph Exposure exp="C"

Wind parameter $q_s = 12.6\text{psf}$ $C_q = 1.3$ $I = 1$ $C_{e\_roof} = 1.06$ $C_{e\_parp} = 1.13$

Wind pressure at roof level $p_{roof} = C_{e\_roof} C_q q_s$ $p_{parp} = 18.51\text{psf}$

Wind pressure at top of parapet level

Wind force at the roof level

With Parapet $W_{wind} = w_{wall} \dfrac{H_{r^2}}{2H_r}$ No Parapet $W_{wind} = w_{wall} \left(\dfrac{h_r}{2} + h_g \right)$

Long Direction $W_{wind\_F\_B} = 41763.1$ Short Direction $W_{wind\_L\_R} = 15951.19$

Description: Lateral Analysis

LATERAL ANALYSIS SUMMARY

Front to back direction. Seismic_or_wind= "Seismic Governs"

Lateral force front-back, $F_{F\_B} = 50337.26\text{lb}$ $f_{f\_b} = 349.56\text{plf}$

Left to right direction. Seismic_or_wind= "Seismic Governs"

Lateral force front-back, $F_{L\_R} = 39948.99\text{lb}$ $f_{l\_r} = 726.35\text{plf}$

Description: Slender Wall Design

BEARING WALL SUPPORTING THE PURLINS

Design Data Input

Wall thickness, (Nominal) $t_g = 10in$ Concrete Core Thickness $t_c = 6in$

Wall height $Ht = 16ft$ Equivalent square sides $S_{eq} = 4.75in$

Rebar Size $No = 5$ Rebar spacing vertical $spacing = 15in$

Rebar location Edge/Center (1/0) $location = 1in$

Rebar Depth $d = 3.75in$

Concrete Strength $f_c = 2500psi$

Modulus of Rupture $f_r = 250psi$

Steel Strength $F_y = 60ksi$

Wall Weight Multiplier $factor = 1.00$
Wall Weight $w_{wall} = 52psf$

Effective strip width $w_{eff} = 15$

Loads Input

Dead Load $w_{dl_e} = \dfrac{20ft w_{d\_p}}{2}$ $p_{dl\_c} = 0lb$

Live load $w_{ll_e} = \dfrac{25ft w_{l\_p}}{2}$ $w_{ll_e} = 192plf$

Eccentricity $e = 6.25in$

Description: Slender Wall Design

Lateral Loads

Seismic $Z = 4$ $I = 1$ $C_p = .75$

Seismic load Factor $ZIC_p = .3$

Seismic Force on the wall $v_p = ZIC_{p}w_{wall}$ $v_p = 15.6psf$

Additional Load

Point Load $P_1 = 0lb$ $P_2 = 0lb$

Height Above Ground $h_1 = 10ft$ $h_2 = 0lb$

Uniform Load $w_{seis} = 0plf$

Height from Bottom $h_{bott} = 1ft$

Height to Top $h_{top} = 10ft$

Wind

Speed $V = 80mph$ Exposure $\exp = C$ $C_q = 1.3$ $I = 1$

Pressure $q_s = 16.4psf$

Wind force on the wall $P = 22.6psf$

Additional Load
Point Load $P_1 = 0lb$ $p_2 = 0lb$

Height Above Ground $h_c = 10ft$

Uniform Load $w_{wind} = 0plf$

Height from Bottom $h_{bott} = 1ft$

Example: Slender Wall Design

Check axial load limitation

$\frac{p_w + p_o}{A_g} = 39.39\text{psi}$ Must be $< .04f_c = 100\text{psi}$

Factored Loads

Seismic

$U = .75(1.4w_{dl} + 1.7w_{ll}(\text{floor}) + 1.87E)$

$U = .9w_{dl} + 1.43E$

OR

Wind

$U = .75(1.4w_{dl} + 1.7w_{ll}(\text{floor}) + 1.7W)$

$U = .9w_{dl} + 1.3W$

Factored Load on the wall $p_{ou} = 253.13\text{lb}$

Factored wall load at mid ht $p_{wu} = 468\text{lb}$

Factored wall load at mid ht $p_u = 721.13\text{lb}$

Factored Lateral Load $w_u = 40.4\text{plf}$

Description: Slender Wall Design

Cracked Section Analysis

$$A_{se} = \frac{(p_u + A_s F_y)}{F_y} \qquad A_{se} = .32\text{in}^2$$

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad M_{cr} = 6.69\text{in} \cdot \text{kips}$$

$$I_{cr} = \left[nA_{se}(d-c)^2\right] + \frac{s_{eq}c^3}{3} \qquad I_{cr} = 15.63\text{in}^4$$

$$\Delta_{cr} = \frac{5M_{cr}Ht^2}{48E_c I_g} \qquad \Delta_{cr} = .014\text{in}$$

Service Moment

$$M_s = \frac{w_{st}Ht^2}{8} + p_o\frac{e}{2} + (p_o + p_w)\Delta_s \qquad M_s = 15.26\text{in} \cdot \text{kips}$$

$$\Delta_s = \frac{5M_s Ht^2}{48E_c I_g} + \frac{5(M_s - M_{cr})Ht^2}{48E_c I_{cr}} \qquad \Delta_s = 1.06\text{in}$$

Nominal Moment Strength $\phi = .72$

$$M_n = (A_{se}F_y)\left(d - \frac{a}{2}\right)$$

$$M_n = 50.21\text{in} \cdot \text{kips} \qquad \phi M_n = 36.15\text{in} \cdot \text{kips}$$

$$\Delta_n = \frac{5M_n Ht^2}{48E_c I_{cr}} \qquad \Delta_n = 4.33\text{in}$$

Example: Slender Wall Design

Factored Moment

$$M_u = \frac{w_u Ht^2}{8} + p_u \frac{e}{2} + (p_{ou} + p_{wu})\Delta_n$$

$M_u = 1740.51\text{lbft}$ Must be < $\phi M_n = 3012.71\text{lbft}$

Check deflection at service load

$M_s = 15.26\text{in} \cdot \text{kips}$ $M_n = 50.21\text{in} \cdot \text{kips}$

$$\Delta_s = \Delta_{cr} + \frac{M_s - M_{cr}}{M_n - M_{cr}}(\Delta_n - \Delta_{cr})$$ $\Delta_s = .97\text{in}$ Must be < $\frac{Ht}{150} = 1.28\text{in}$

Description: Slender Wall Design

BEARING WALL AT THE RESIDENCE BETWEEN THE WINDOWS

Design Data Input

Wall thickness, (Nominal) $t_g = 10in$ Concrete Core Thickness $t_c = 6in$

Wall height $Ht = 18ft$ Equivalent square sides $S_{eq} = 4.75in$

Rebar Size $No = 5$ Rebar spacing vertical $spacing = 15in$

Rebar location Edge/Center (1/0) $location = 1in$

Rebar Depth $d = 3.75in$

Concrete Strength $f_c = 2500psi$

Modulus of Rupture $f_r = 250psi$

Steel Strength $F_y = 60ksi$

Wall Weight Multiplier $factor = 1.00$
Wall Weight $w_{wall} = 52psf$

Effective strip width $w_{eff} = 15$

Loads Input

Dead Load $w_{dl_e} = 4ft w_{d\_s}$ $p_{dlc} = 0lb$

Live load $w_{ll_e} = 4ft w_{l\_s}$ $p_{ll_e} = 0lb$

Eccentricity $e = 6.25in$

Description: Slender Wall Design

Lateral Loads

Seismic $Z = 4$ $I = 1$ $C_p = .75$

Seismic load Factor $ZIC_p = .3$

Seismic Force on the wall $v_p = ZIC_{p^{w_{wall}}}$ $v_p = 15.6psf$

Additional Load

Point Load $P_1 = 0lb$ $P_2 = 0lb$

Height Above Ground $h_1 = 10ft$ $h_2 = 0lb$

Uniform Load $w_{seis} = 0plf$

Height from Bottom $h_{bott} = 1ft$

Height to Top $h_{top} = 10ft$

Wind

Speed $V = 80mph$ Exposure $exp = C$ $C_q = 1.3$ $I = 1$

Pressure $q_s = 16.4psf$

Wind force on the wall $P = 22.6psf$

Additional Load

Point Load $P_1 = 0lb$ $p_2 = 0lb$

Height Above Ground $h_c = 10ft$

Uniform Load $w_{wind} = 0plf$

Height from Bottom $h_{bott} = 1ft$

Example: Slender Wall Design

Check axial load limitation

$$\frac{p_w + p_o}{A_g} = 25.82\text{psi} \qquad \text{Must be} < .04f_c = 100\text{psi}$$

Factored Loads

Seismic

$$U = .75(1.4w_{dl} + 1.7w_{ll}(\text{floor}) + 1.87E)$$

$$U = .9w_{dl} + 1.43E$$

OR

Wind

$$U = .75(1.4w_{dl} + 1.7w_{ll}(\text{floor}) + 1.7W)$$

$$U = .9w_{dl} + 1.3W$$

Factored Load on the wall $p_{ou} = 40.5\text{lb}$

Factored wall load at mid ht $p_{wu} = 526.5\text{lb}$

Factored wall load at mid ht $p_u = 567\text{lb}$

Factored Lateral Load $w_u = 40.4\text{plf}$

Description: Slender Wall Design

Cracked Section Analysis

$$A_{se} = \frac{(p_u + A_s F_y)}{F_y} \qquad A_{se} = .32\text{in}^2$$

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad M_{cr} = 6.69\text{in} \cdot \text{kips}$$

$$I_{cr} = \left[nA_{se}(d-c)^2\right] + \frac{s_{eq}c^3}{3} \qquad I_{cr} = 27.69\text{in}^4$$

$$\Delta_{cr} = \frac{5M_{cr}Ht^2}{48E_cI_g} \qquad \Delta_{cr} = .18\text{in}$$

Service Moment

$$M_s = \frac{w_{st}Ht^2}{8} + p_o\frac{e}{2} + (p_o + p_w)\Delta_s \qquad M_s = 15.86\text{in} \cdot \text{kips}$$

$$\Delta_s = \frac{5M_sHt^2}{48E_cI_g} + \frac{5(M_s - M_{cr})Ht^2}{48E_cI_{cr}} \qquad \Delta_s = .99\text{in}$$

Nominal Moment Strength $\phi = .72$

$$M_n = (A_{se}F_y)\left(d - \frac{a}{2}\right)$$

$$M_n = 64.13\text{in} \cdot \text{kips} \qquad \phi M_n = 46.18\text{in} \cdot \text{kips}$$

$$\Delta_n = \frac{5M_nHt^2}{48E_cI_{cr}} \qquad \Delta_n = 3.95\text{in}$$

Example: Slender Wall Design

Factored Moment

$$M_u = \frac{w_u Ht^2}{8} + p_u \frac{e}{2} + \left(p_{ou} + p_{wu}\right)\Delta_n$$

$$M_u = 1970.31 \text{lbft} \qquad \text{Must be} < \phi M_n = 3848.02 \text{lbft}$$

Check deflection at service load

$$M_s = 15.86 \text{in} \cdot \text{kips} \qquad M_n = 64.13 \text{in} \cdot \text{kips}$$

$$\Delta_s = \Delta_{cr} + \frac{M_s - M_{cr}}{M_n - M_{cr}}\left(\Delta_n - \Delta_{cr}\right) \qquad \Delta_s = .78 \text{in} \qquad \text{Must be} < \frac{Ht}{150} = 1.44 \text{in}$$

Description: Slender Wall Design

PILASTER AT THE GIRDER

Design Data Input

Wall thickness, (Nominal) $t_g = 10in$ Concrete Core Thickness $t_c = 6in$

Wall height $Ht = 16ft$ Equivalent square sides $S_{eq} = 4.75in$

Rebar Size $No = 5$ Rebar spacing vertical $spacing = 15in$

Rebar location Edge/Center (1/0) $location = 1in$

Rebar Depth $d = 3.75in$

Concrete Strength $f_c = 2500psi$

Modulus of Rupture $f_r = 250psi$

Steel Strength $F_y = 60ksi$

Wall Weight Multiplier $factor = 1.00$
Wall Weight $w_{wall} = 52psf$

Effective strip width $w_{eff} = 15$

Loads Input

Dead Load $w_{dl_e} = w_{d\_g} 2ft$ $p_{dl\_c} = \dfrac{24ft55w_{d\_g}}{2}$

Live load $w_{ll_e} = w_{l\_g} 2ft$ $p_{ll_e} = \dfrac{24ft55w_{d\_g}}{2}$

Eccentricity $e = 6.25in$

Description: Slender Wall Design

Lateral Loads

Seismic $Z = 4$ $I = 1$ $C_p = .75$

Seismic load Factor $ZIC_p = .3$

Seismic Force on the wall $v_p = ZIC_{p^{w_{wall}}}$ $v_p = 15.6\text{psf}$

Additional Load

Point Load $P_1 = 0\text{lb}$ $P_2 = 0\text{lb}$

Height Above Ground $h_1 = 10\text{ft}$ $h_2 = 0\text{lb}$

Uniform Load $w_{seis} = 0\text{plf}$

Height from Bottom $h_{bott} = 1\text{ft}$

Height to Top $h_{top} = 10\text{ft}$

Wind

Speed $V = 80\text{mph}$ Exposure $\exp = C$ $C_q = 1.3$ $I = 1$

Pressure $q_s = 16.4\text{psf}$

Wind force on the wall $P = 22.6\text{psf}$

Additional Load

Point Load $P_1 = 0\text{lb}$ $p_2 = 0\text{lb}$

Height Above Ground $h_c = 10\text{ft}$ h2=0lb

Uniform Load $w_{wind} = 0\text{plf}$

Height from Bottom $h_{bott} = 1\text{ft}$

Example: Slender Wall Design

Check axial load limitation

$$\frac{p_w + p_o}{A_g} = 21.22\text{psi} \qquad \text{Must be} < .04f_c = 100\text{psi}$$

Factored Loads

Seismic

$$U = .75\left(1.4w_{dl} + 1.7w_{ll}(\text{floor}) + 1.87E\right)$$

$$U = .9w_{dl} + 1.43E$$

OR

Wind

$$U = .75\left(1.4w_{dl} + 1.7w_{ll}(\text{floor}) + 1.7W\right)$$

$$U = .9w_{dl} + 1.3W$$

Factored Load on the wall $p_{ou} = 40.5\text{lb}$

Factored wall load at mid ht $p_{wu} = 546\text{lb}$

Factored wall load at mid ht $p_u = 586.5\text{lb}$

Factored Lateral Load $w_u = 36.02\text{plf}$

Description: Slender Wall Design

Cracked Section Analysis

$$A_{se} = \frac{(p_u + A_s F_y)}{F_y} \qquad A_{se} = .61\text{in}^2$$

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad M_{cr} = 6.69\text{in} \cdot \text{kips}$$

$$I_{cr} = \left[nA_{se}(d-c)^2\right] + \frac{s_{eq}c^3}{3} \qquad I_{cr} = 34.06\text{in}^4$$

$$\Delta_{cr} = \frac{5M_{cr}Ht^2}{48E_c I_g} \qquad \Delta_{cr} = .14\text{in}$$

Service Moment

$$M_s = \frac{w_{st}Ht^2}{8} + p_o\frac{e}{2} + (p_o + p_w)\Delta_s \qquad M_s = 12.48\text{in} \cdot \text{kips}$$

$$\Delta_s = \frac{5M_s Ht^2}{48E_c I_g} + \frac{5(M_s - M_{cr})Ht^2}{48E_c I_{cr}} \qquad \Delta_s = .49\text{in}$$

Nominal Moment Strength $\phi = .72$

$$M_n = (A_{se}F_y)\left(d - \frac{a}{2}\right)$$

$$M_n = 111.21\text{in} \cdot \text{kips} \qquad \phi M_n = 80.07\text{in} \cdot \text{kips}$$

$$\Delta_n = \frac{5M_n Ht^2}{48E_c I_{cr}} \qquad \Delta_n = 4.4\text{in}$$

Example: Slender Wall Design

Factored Moment

$$M_u = \frac{w_u Ht^2}{8} + p_u \frac{e}{2} + (p_{ou} + p_{wu})\Delta_n$$

$M_u = 1520.28 \text{lbft}$ Must be < $\phi M_n = 6672..57 \text{lbft}$

Check deflection at service load

$M_s = 12.48 \text{in} \cdot \text{kips}$ $M_n = 111.21 \text{in} \cdot \text{kips}$

$$\Delta_s = \Delta_{cr} + \frac{M_s - M_{cr}}{M_n - M_{cr}}(\Delta_n - \Delta_{cr}) \qquad \Delta_s = .97\text{in} \qquad \text{Must be} < \frac{Ht}{150} = 1.28\text{in}$$

Description: Slender Wall Design

BEARING WALL SUPPORTING THE PURLINS

Design Data Input

Wall thickness, (Nominal) $t_g = 10\text{in}$ Concrete Core Thickness $t_c = 6\text{in}$

Lintel Length $L = 12\text{ft}$ Equivalent square sides $S_{eq} = 4.75\text{in}$

Lintel Height $h=60\text{in}$ Rebar depth $d=15\text{in}$

Rebar Size $No = 6$

Concrete Strength $f_c = 2500\text{psi}$

Modulus of Rupture $f_r = 250\text{psi}$

Steel Strength $F_y = 60\text{ksi}$

Wall Weight Multiplier $factor = 1.00$

Wall Weight $w_{wall} = 52\text{psf}$

Effective strip width $w_{eff} = 15$

Loads Input

Dead Load $w_{dl} = (4w_{d\_g}\text{ft}) + 12\text{ft}w_{wall}$ $p_{dl\_c} = 0\text{lb}$

Live load $w_{ll_e} = 4w_{l\_g}\text{ft}$ $p_{ll\_c} = 0\text{lb}$

Example: Lintel Design

Analysis

$$U = (1.4w_{dl} + 1.7w_{ll})$$

$$M_u = 19209.6 lb \cdot ft$$

Design;

Given $M_u = .9bd^2F_y\rho\left(1 - .59\rho\frac{F_y}{f_c}\right)$ $\rho = Find(\rho)$ $\rho = .0042$

$A_{s\_req} = \rho bd$ $A_{s\_req} = .3in^2$ Reinf. Size $Reinf = 6$

Use No of reinf $No = 1$

Description: Shear Wall Design

EAST SHEAR WALL

Wall weight $w_{wall} = 52psf$ Tributary Width $b = 4.75in$

Thickness, nominal $d = 5in$ Reinf location center (c), edge/ Location= c

Wall length $l_{wall} = 2.5ft$ $d_{wall} = .8 l_{wall}$ $d_{wall} = 2ft$

Wall height $h_p = 12ft$

Perform Wall data Special inspection (y/n) answer = no

$f_c = 2500psi$ $\phi = .85$

$F_y = 60000psi$

Lateral Loads

Seismic Z=4 I=1 C=1 $R_w = 6$

Pier seismic force $v_p = \dfrac{F_{F\_B_2}}{6}$

Additional force on the pier, V = 0 lb

Distance from the base of the wall $x = 10ft$

Description: Shear Wall Design

Forces

Reaction $R = V + v_p$ $R = 6292.16\text{lb}$

Moment $M = Vx + v_p \dfrac{h_p}{2}$ $M = 37.75\text{ft}\cdot\text{kips}$

Axial $P_{axial} = h_{p_{bwwall}}$ $P_{axial} = 247.1\text{lb}$

Shear Strength factor= 7 $2\phi\sqrt{f_c\text{psi}}bd = 3.13\text{kips}$

$v_c = \left[\left(2\phi\sqrt{f_c\text{psi}}bd\right)\dfrac{l_{wall}}{15\text{in}}\right]$ $V_c = 6.26\text{kips}$

Reinforce, req $d = .65\text{ft}$ $F_y = 60\text{ksi}$ $\rho = .01$

Rebar Size Reinf= 5 No of Reinf No=2

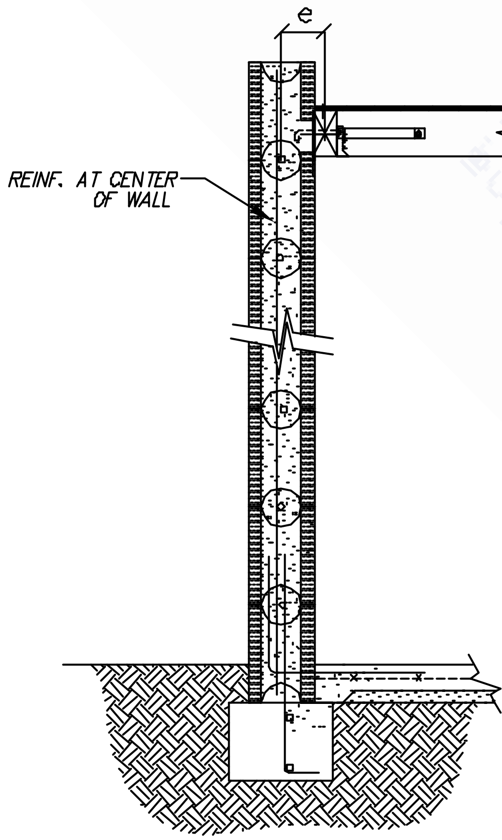
Allowable Moment $M_n = .9s_{eq^d}\,\text{wall}^2 Fy\rho\left(1 - .59\rho\dfrac{F_y}{f_c}\right)$

$M_n = 185.16\text{ft}\cdot\text{kips}$ Result="Bending O.K." Use No=2 Reinf=5

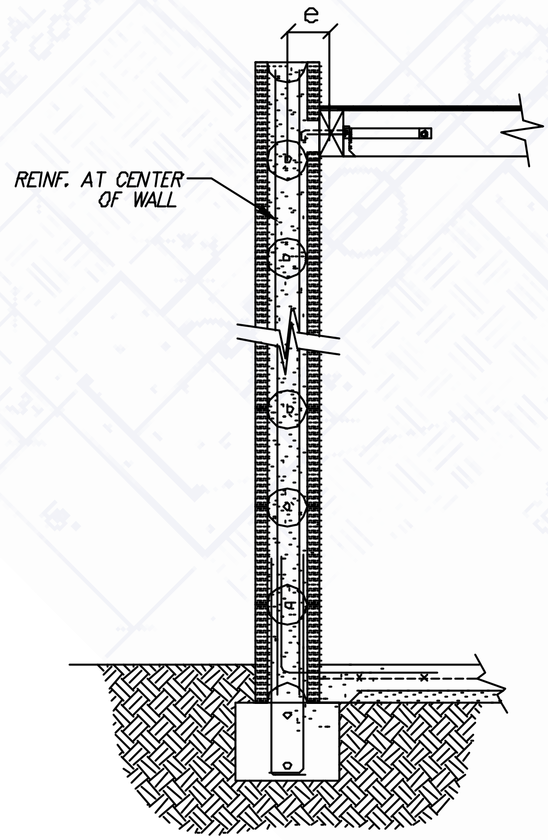
SECTION 7

Schedules

10" Perform Wall™ Uniform Loads



DETAIL-1



DETAIL-2

ALLOWABLE UNIFORM LOADS

ALLOWABLE AXIAL LOAD IN POUNDS PER LINEAL FOOT

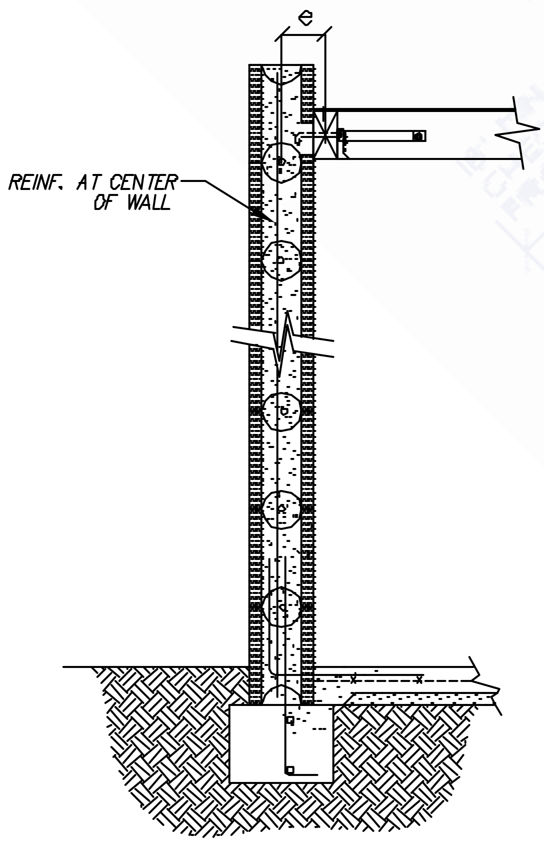
HEIGHT OF WALL FT.	REBAR & SPACING				
	70 MPH (16.03 PSF)		80 MPH (28.86 PSF)		
	#4 @ 15" O.C.	#5 @ 15" O.C.	#5 @ 15" O.C.	#5 @ 15" O.C. (EACH FACE)	#6 @ 15" O.C. (EACH FACE)
8'-0"	1,000	2,100	1,800	2,700	3,000
10'-0"	800	1,800	1,500	2,500	2,700
12'-0"	600	1,500	1,200	2,400	2,500
14'-0"	400	1,200	750	2,200	2,400
16'-0"		600	400	1,500	1,800
18'-0"				600	1,500
20'-0"					1,000
22'-0"					600
	SEE DETAIL-1			SEE DETAIL-2	

DESIGN PARAMETERS

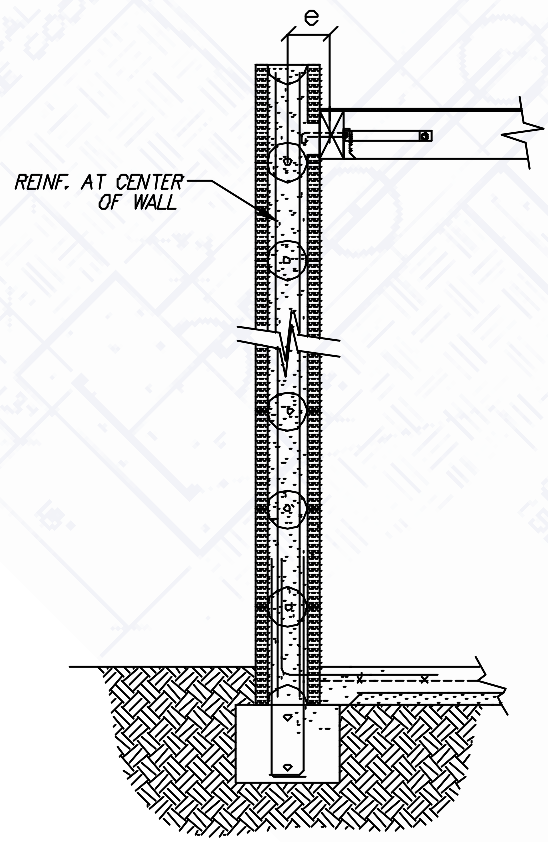
DESIGN PARAMETERS
ECCENTRICITY: e = 6 3/4"
STEEL: 60,000 psi.
CONCRETE: 2,500 psi.
SEISMIC ZONE: ZONE-4, %=0.40
WIND EXPOSURE: EXPOSURE-C

PERFORM WALL PANEL SYSTEM

12" Perform Wall™ Uniform Loads



DETAIL-1



DETAIL-2

ALLOWABLE UNIFORM LOADS

ALLOWABLE AXIAL LOAD IN POUNDS PER LINEAL FOOT

HEIGHT OF WALL FT.	REBAR & SPACING				
	70 MPH (16.03 PSF)		80 MPH (28.86 PSF)		
	#4 @ 15" O.C.	#5 @ 15" O.C.	#5 @ 15" O.C.	#5 @ 15" O.C. (EACH FACE)	#6 @ 15" O.C. (EACH FACE)
8'-0"	1,400	2,400	2,400	3,400	3,600
10'-0"	1,400	2,000	1,800	3,000	3,400
12'-0"	800	1,700	1,400	3,000	3,200
14'-0"	400	1,200	800	2,800	3,000
16'-0"	300	800	300	1,200	1,800
18'-0"				300	1,600
20'-0"					1,200
22'-0"					500
	SEE DETAIL-1			SEE DETAIL-2	

DESIGN PARAMETERS

ECCENTRICITY:
e = 7 3/4"

STEEL:
60,000 psi.

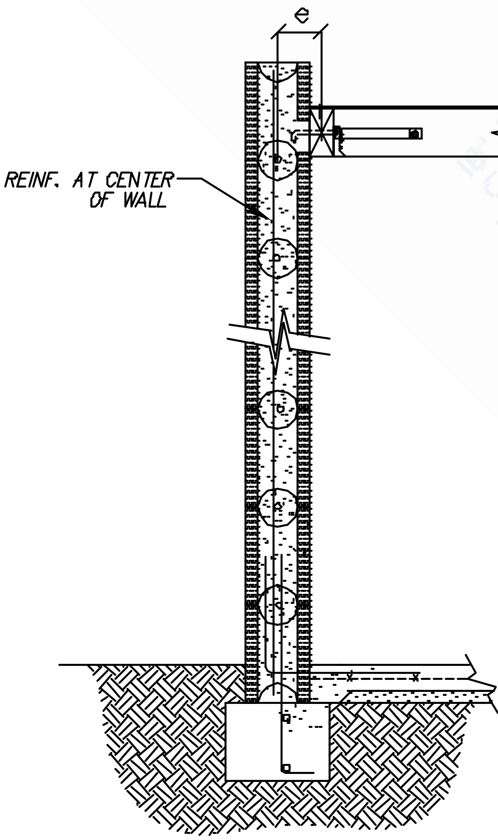
CONCRETE:
2,500 psi.

SEISMIC ZONE:
ZONE-4, %=0.40

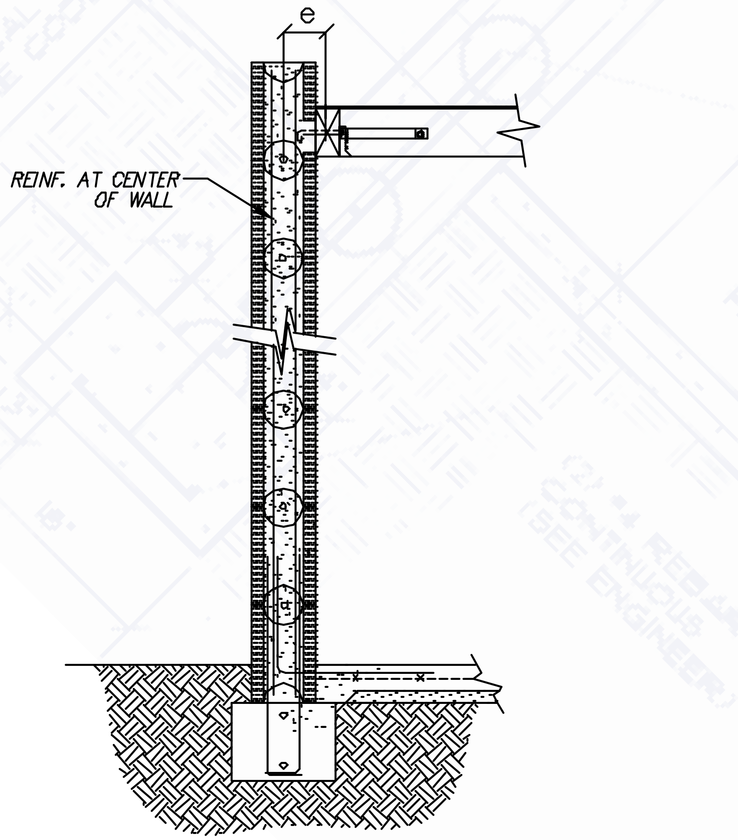
WIND EXPOSURE:
EXPOSURE-C

PERFORM WALL PANEL SYSTEM

14” Perform Wall™ Uniform Loads



DETAIL-1



DETAIL-2

ALLOWABLE UNIFORM LOADS

ALLOWABLE AXIAL LOAD IN POUNDS PER LINEAL FOOT

HEIGHT OF WALL FT.	REBAR & SPACING				
	70 MPH (16.03 PSF)		80 MPH (28.86 PSF)		
	#4 @ 15" O.C.	#5 @ 15" O.C.	#5 @ 15" O.C.	#5 @ 15" O.C. (EACH FACE)	#6 @ 15" O.C. (EACH FACE)
8'-0"	1,400	2,000	2,000	3,000	3,200
10'-0"	1,400	1,800	1,600	2,800	3,000
12'-0"	800	1,400	1,200	2,600	2,800
14'-0"	400	1,000	800	2,000	2,400
16'-0"		600	200	1,000	1,400
18'-0"					1,200
20'-0"					1,000
22'-0"					400
	SEE DETAIL-1			SEE DETAIL-2	

DESIGN PARAMETERS

ECCENTRICITY:
e = 8 3/4"

STEEL:
60,000 psi.

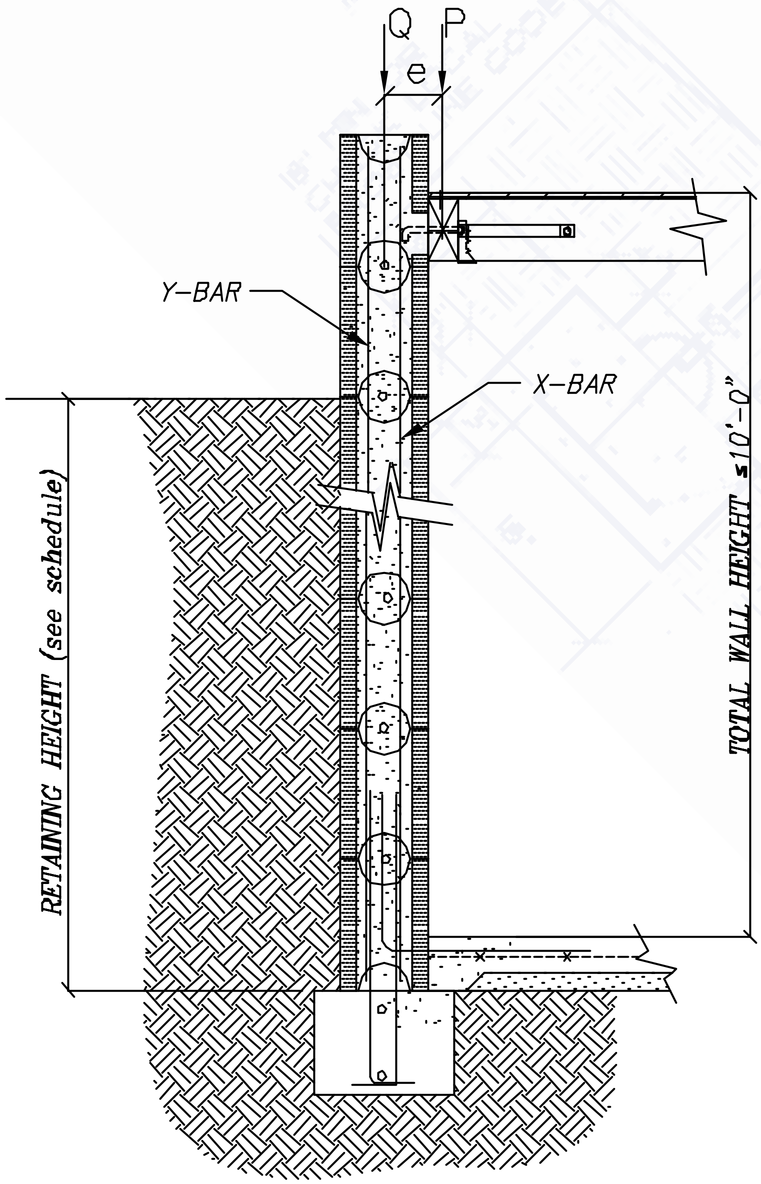
CONCRETE:
2,500 psi.

SEISMIC ZONE:
ZONE-4, %=0.40

WIND EXPOSURE:
EXPOSURE-C

PERFORM WALL PANEL SYSTEM

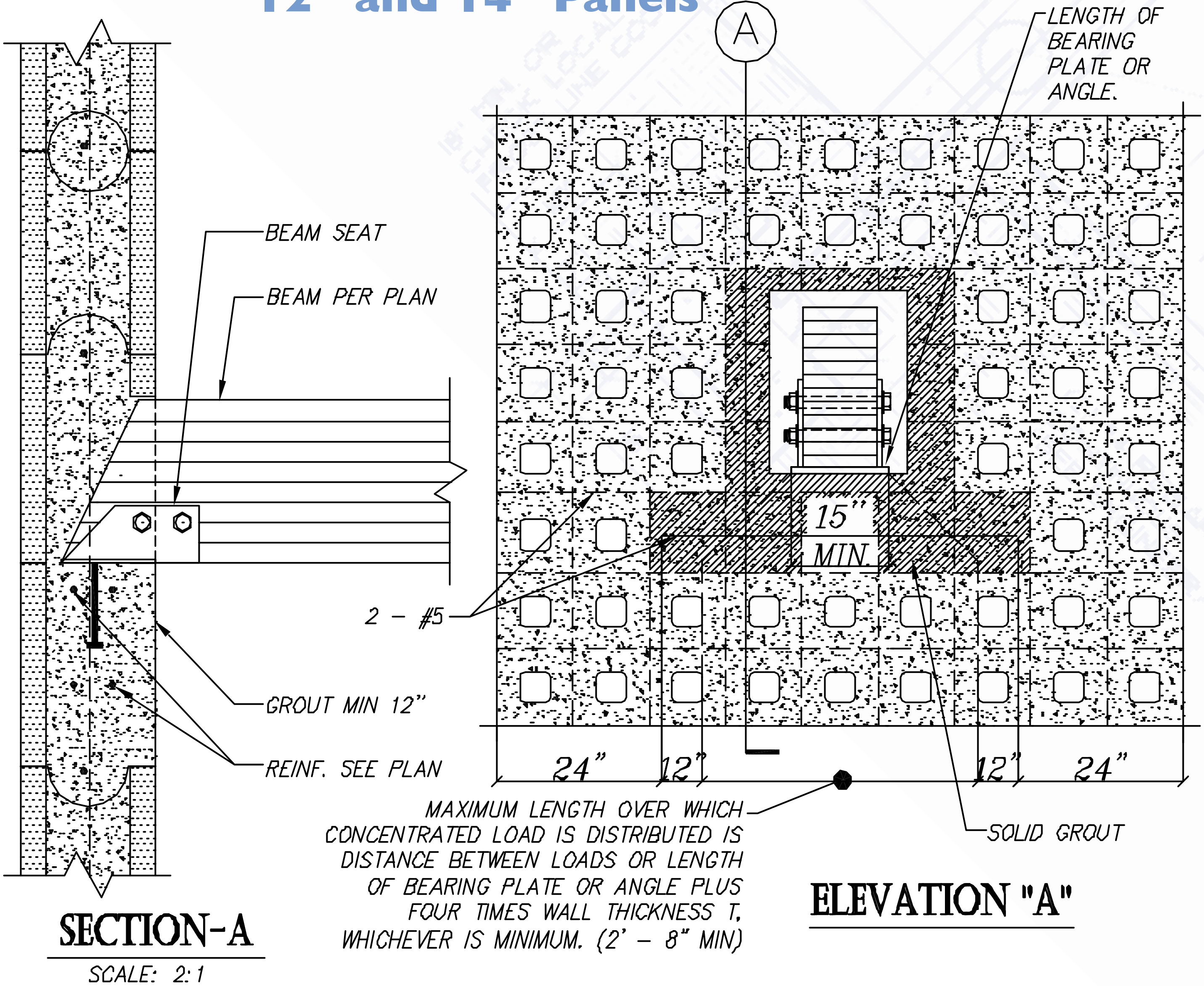
10” & 12” Perform Wall™ Basement Wall



BASEMENT WALL SHEDULE							DESIGN PARAMETERS
RETAINING HEIGHT	ECCENTRIC AXIAL LOAD (P)–(PLF)	AXIAL CENTRIC LOAD (Q)–(PLF)	X–BAR	Y–BAR	CONCRETE f'c	EQUIVALENT FLUID PRESSURE	ECCENTRICITY: e = 7 3/4"
3'–0"	500	≥ 10,000	1–#4 @ 15" O.C.	1–#5 @ 15" O.C.	2,500 PSI	50 PCF	CONCRETE: PER SCHEDULE
4'–0"	600						
6'–0"	700		1–#6 @ 15" O.C.	1–#6 @ 7.5" O.C.	4,000 PSI		STEEL: 60,000 psi.
8'–0"	600						

PERFORM WALL PANEL SYSTEM

Allowable Concentrated Loads For 10," 12" and 14" Panels



ALLOWABLE CONCENTRATED LOADS

ALLOWABLE AXIAL LOAD IN POUNDS PER LINEAL FOOT

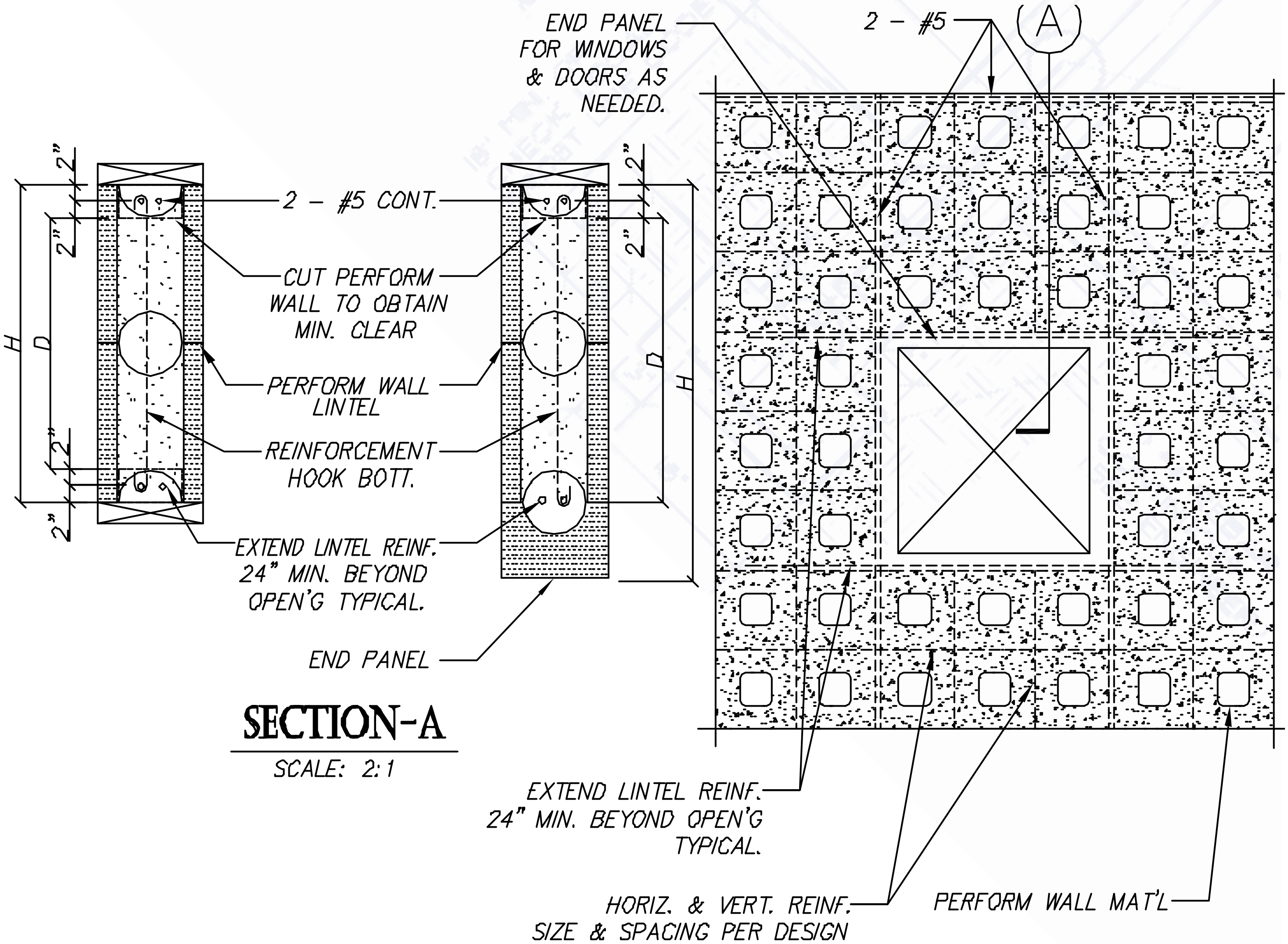
HEIGHT OF WALL FT.	#4 @ 15" O.C.	#5 @ 15" O.C.	#5 @ 15" O.C. EA. FACE	#6 @ 15" O.C. EA. FACE
8'–0"	3800	6000	7400	8000
10'–0"	3800	4800	6400	7400
12'–0"	2200	3800	6000	7000
14'–0"	1200	2800	5000	6500
16'–0"		1600	2600	3700
18'–0"				3200
20'–0"				2700

DESIGN PARAMETERS

ECCENTRICITY:	e = 4"
CONCRETE: 2,500 psi	STEEL: 60,000 psi.
SEISMIC ZONE: ZONE–4, %=0.40	WIND EXPOSURE: EXPOSURE–C

PERFORM WALL PANEL SYSTEM

Lintel Schedule



LINTEL SCHEDULE

FOR 8" OR 10" PERFORM WALLS – RESPECTIVELY

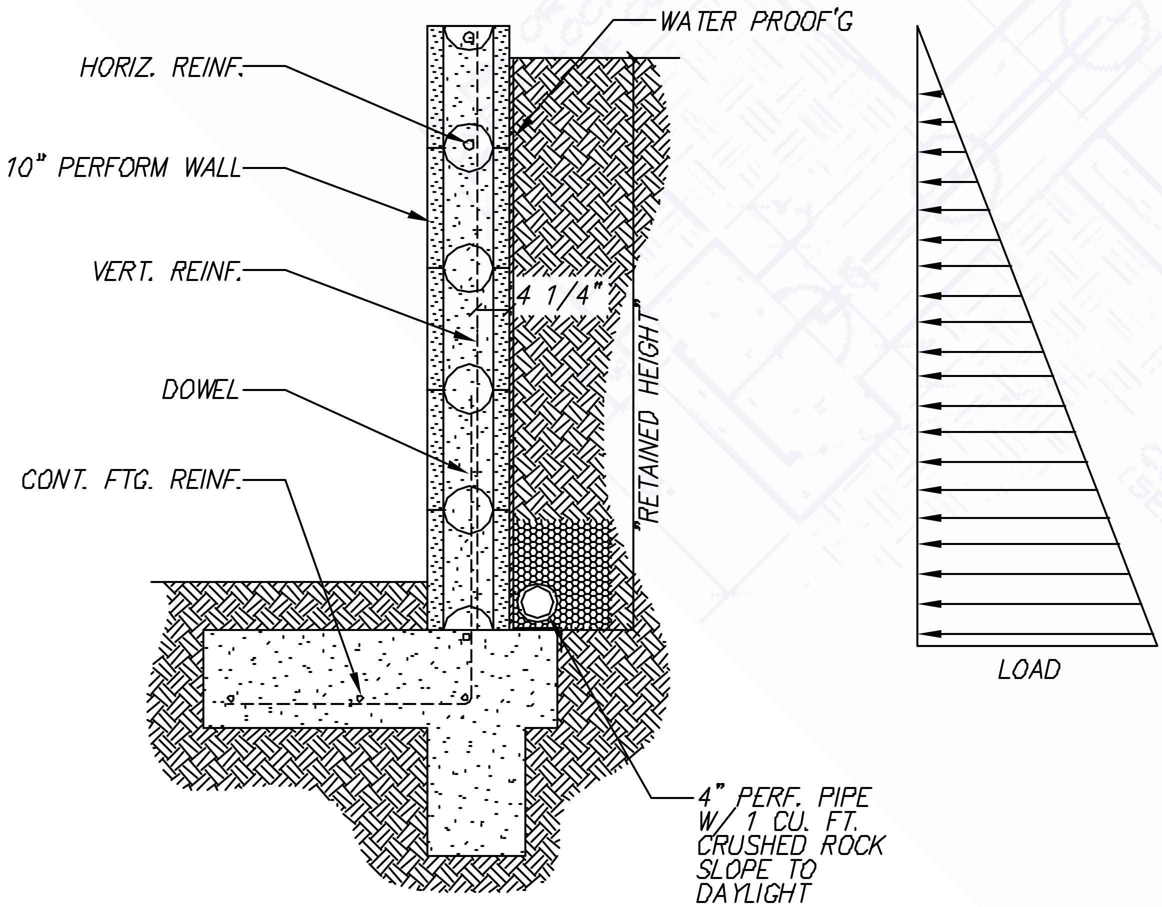
MAX. SPAN	UNIFORM LOAD (PLF) 15" LINTELS H=15",D=13"	UNIFORM LOAD (PLF) 30" LINTELS H=30",D=28"	TOP REINF.	BOTT. REINF.	REMARKS
3'–0"	1800	2600	2–#5	2–#5	24" BEARING EACH END
4'–0"	1400	2000			
6'–0"	800	1600			
8'–0"	600	1400			
10'–0"	500	1000			
12'–0"	400	800			
16'–0"	200	600			

UNLESS OTHERWISE DRAWN OR NOTED

DESIGN PARAMETERS

CONCRETE:	2,500 psi.
STEEL:	60,000 psi.
SEISMIC ZONE:	ZONE–4, %=0.40
WIND EXPOSURE:	EXPOSURE–C

Retaining Wall Schedule

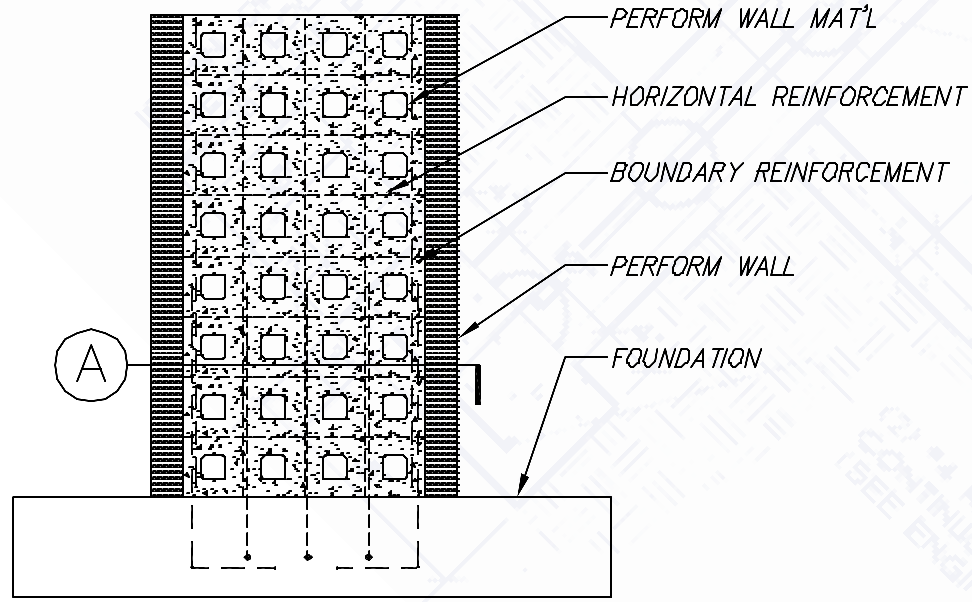


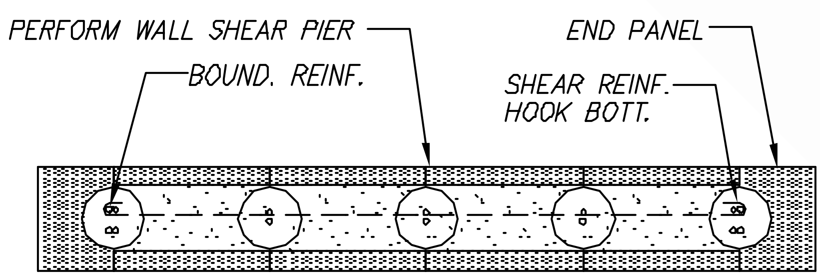
RETAINING WALL SCHEDULE

STEEL: 60,000 PSI

RETAINED HT.	SIZE	HORIZ. REINF.	VERT. REINF.	REMARKS	EQUIV. FLUID PRES.
3'–0"	10"	1–#4	#5 @ 15" O.C.	2,500 PSI CONCRETE	30 PCF (38 PCF FOR WALL)
4'–0"		1–#5			
6'–0"			#6 @ 15" O.C.		
8'–0"			2–#6 @ 15" O.C.	3,000 PSI CONCRETE	

Shear Wall Schedule





SECTION-A

SCALE: 2:1

SHEAR WALL SCHEDULE

FOR 8" OR 10" PERFORM WALL – RESPECTIVELY

MAX. SPAN	LOADING (PLF)	BOUNDARY REINF.	REMARKS
15"	2340	2–#5	#5 @ 15" O.C. HORIZONTALLY
30"	4690		
45"	7030		
60"	11710		
75"	14060		
120"	18740		

DESIGN PARAMETERS

CONCRETE: 2,500 psi.	STEEL: 60,000 psi.
SEISMIC ZONE: ZONE–4, %=0.40	WIND EXPOSURE: EXPOSURE–C

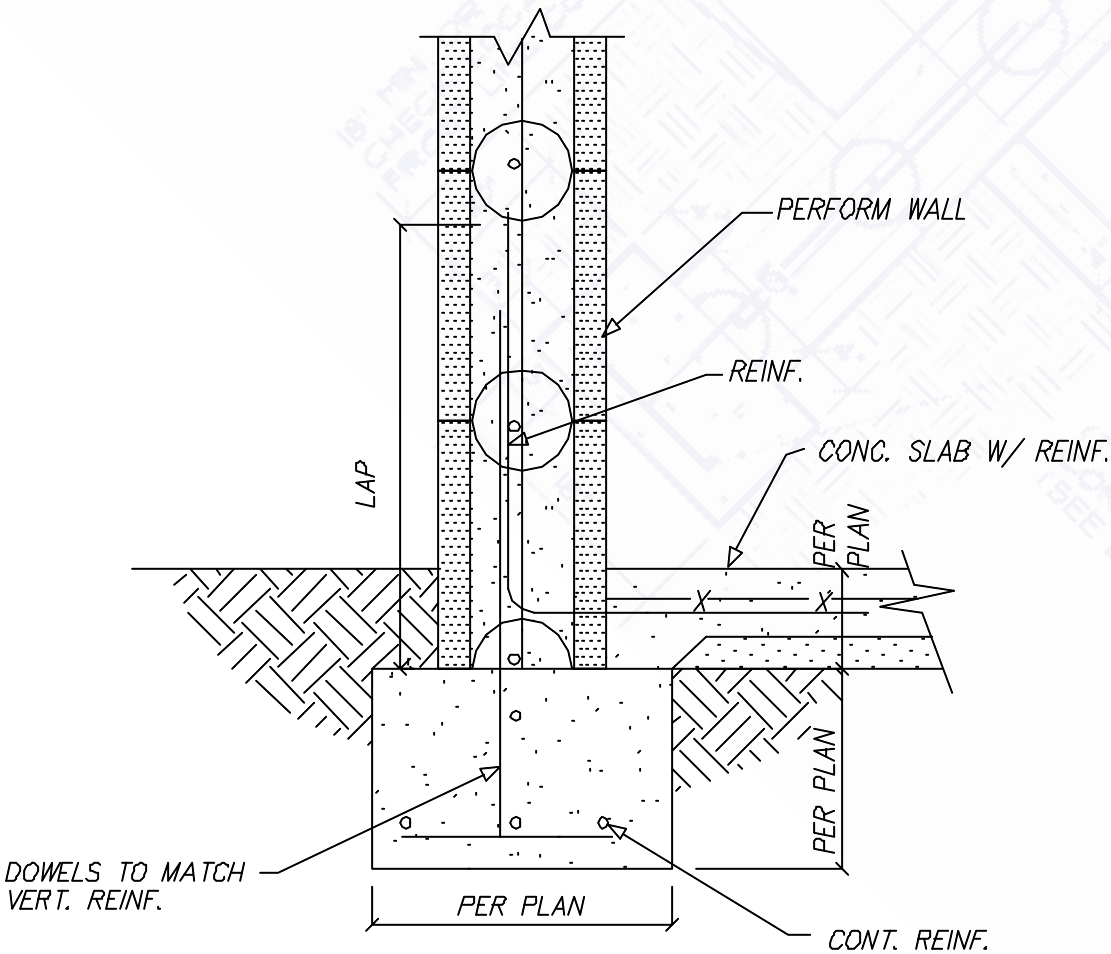
PERFORM WALL PANEL SYSTEM

SECTION 8

Typical Details

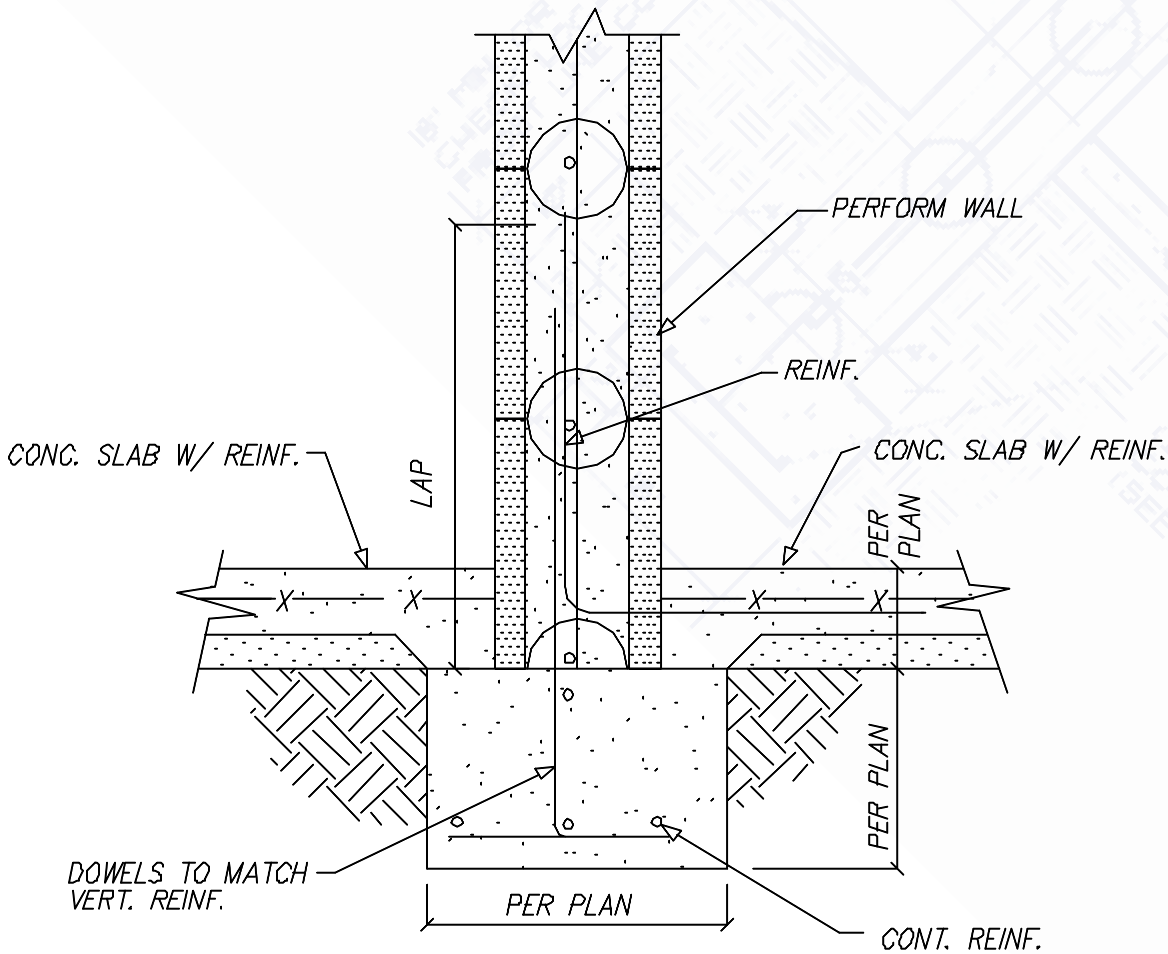
Detail No. 1

Exterior Footing



Detail No. 2

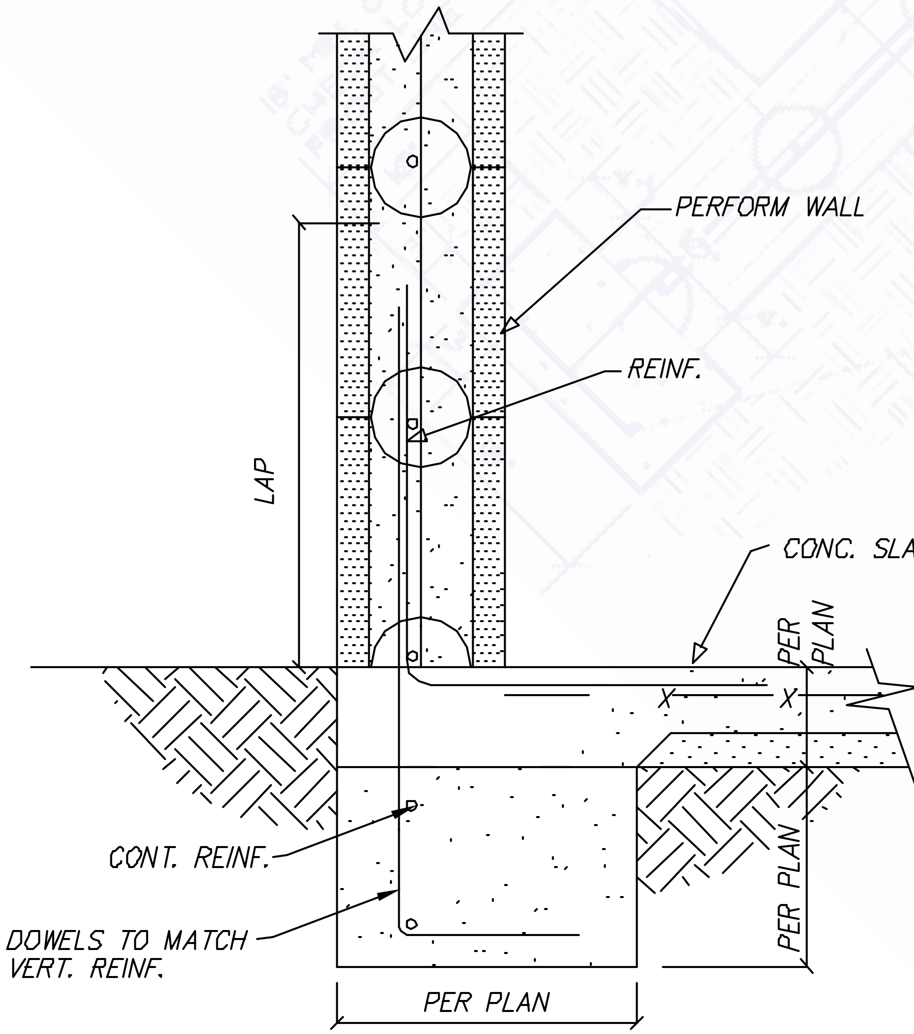
Interior Footing



PERFORM WALL PANEL SYSTEM

Detail No. 3

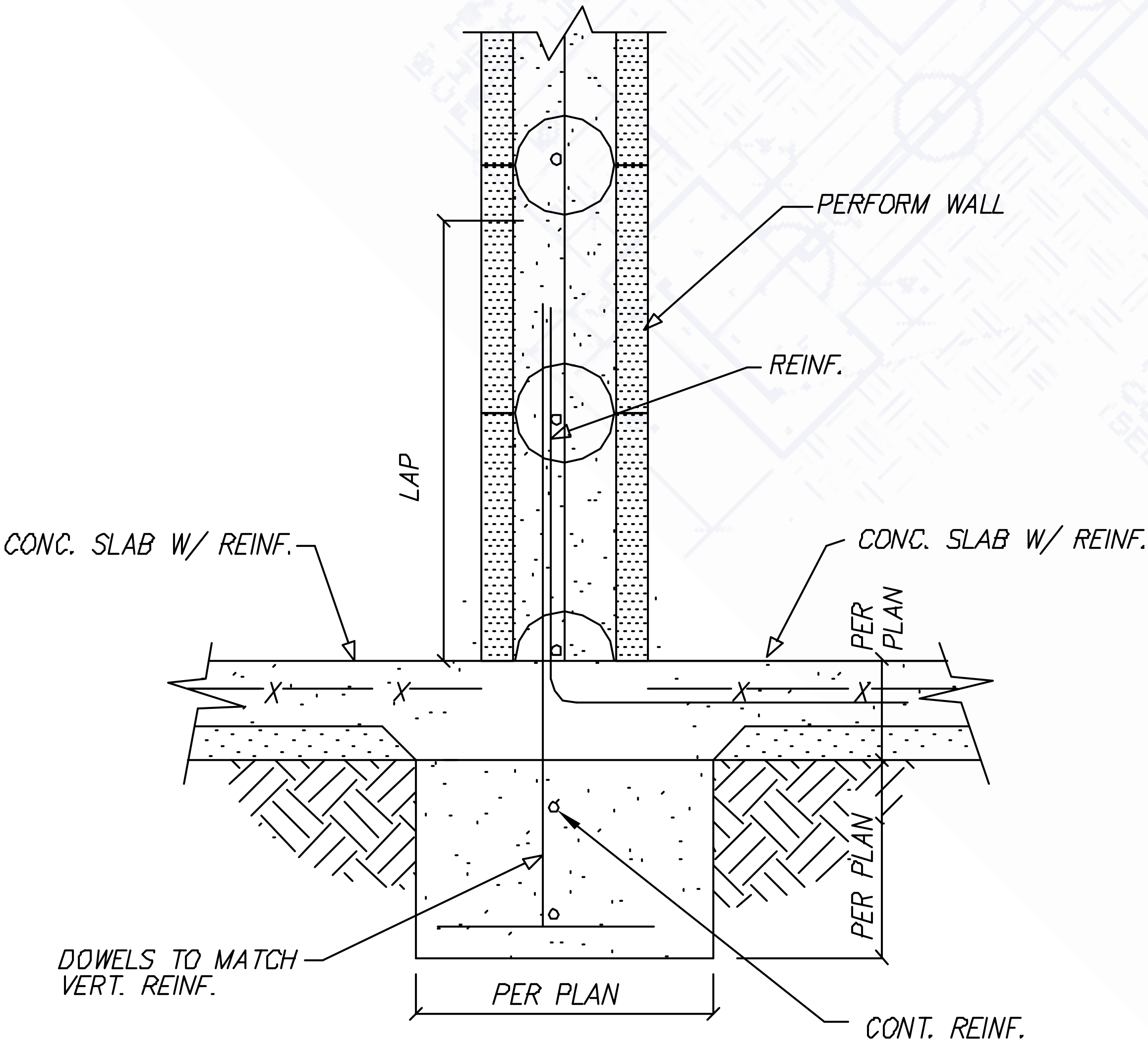
Edge Footing



PERFORM WALL PANEL SYSTEM

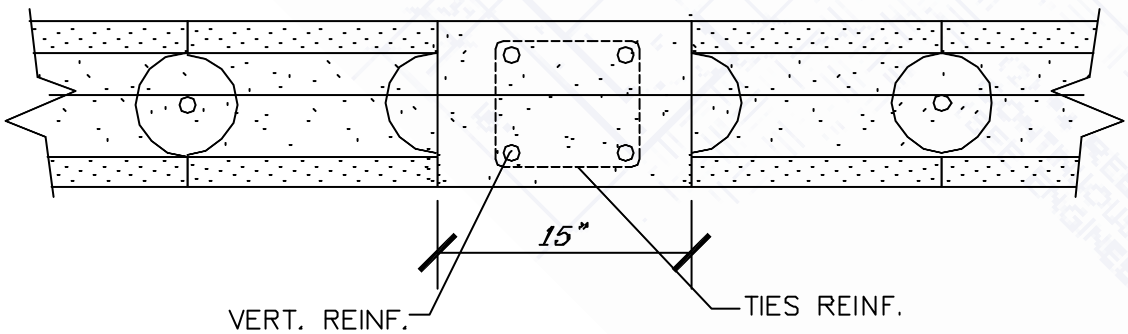
Detail No. 4

Interior Footing with Wall, Resting on Slab



PERFORM WALL PANEL SYSTEM

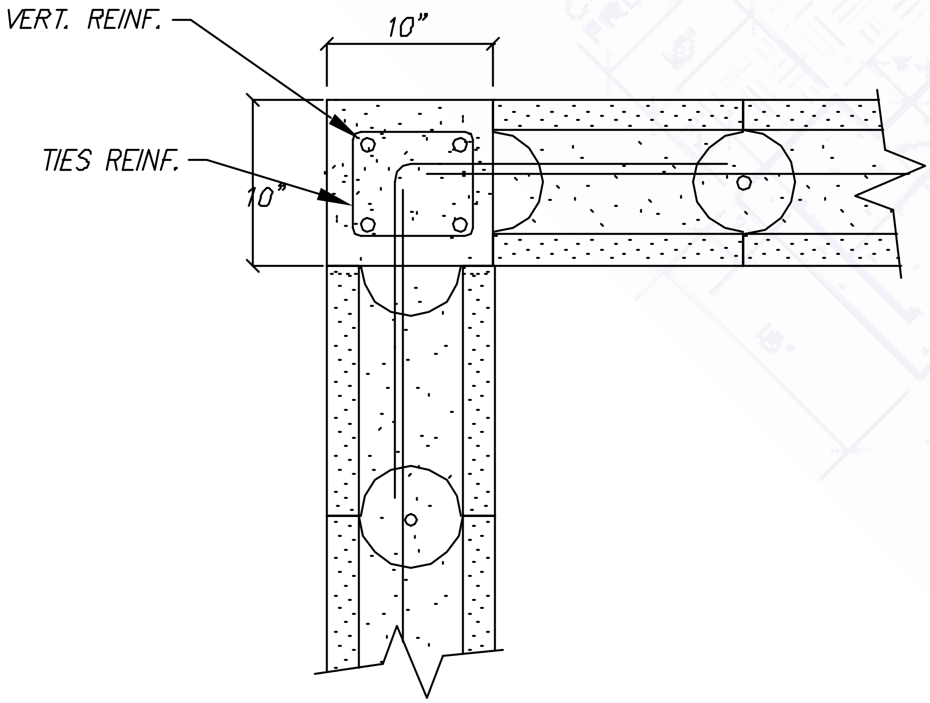
Flush Pilaster



PERFORM WALL PANEL SYSTEM

Detail No. 6

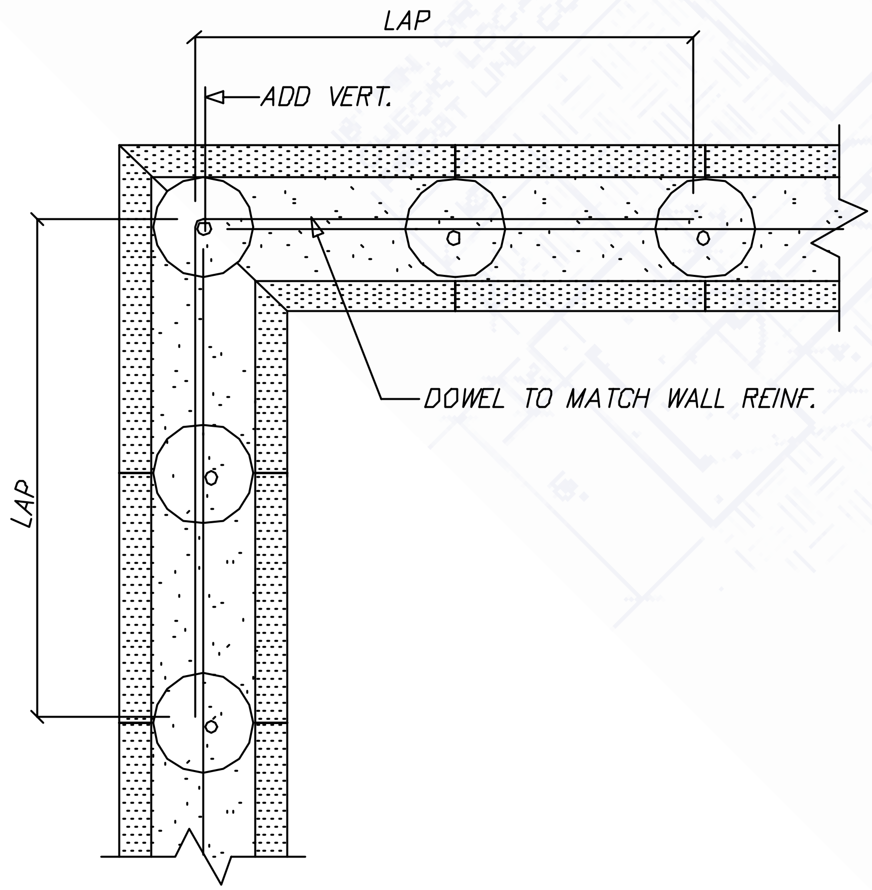
Flush Pilaster at Corner



PERFORM WALL PANEL SYSTEM

Detail No. 7

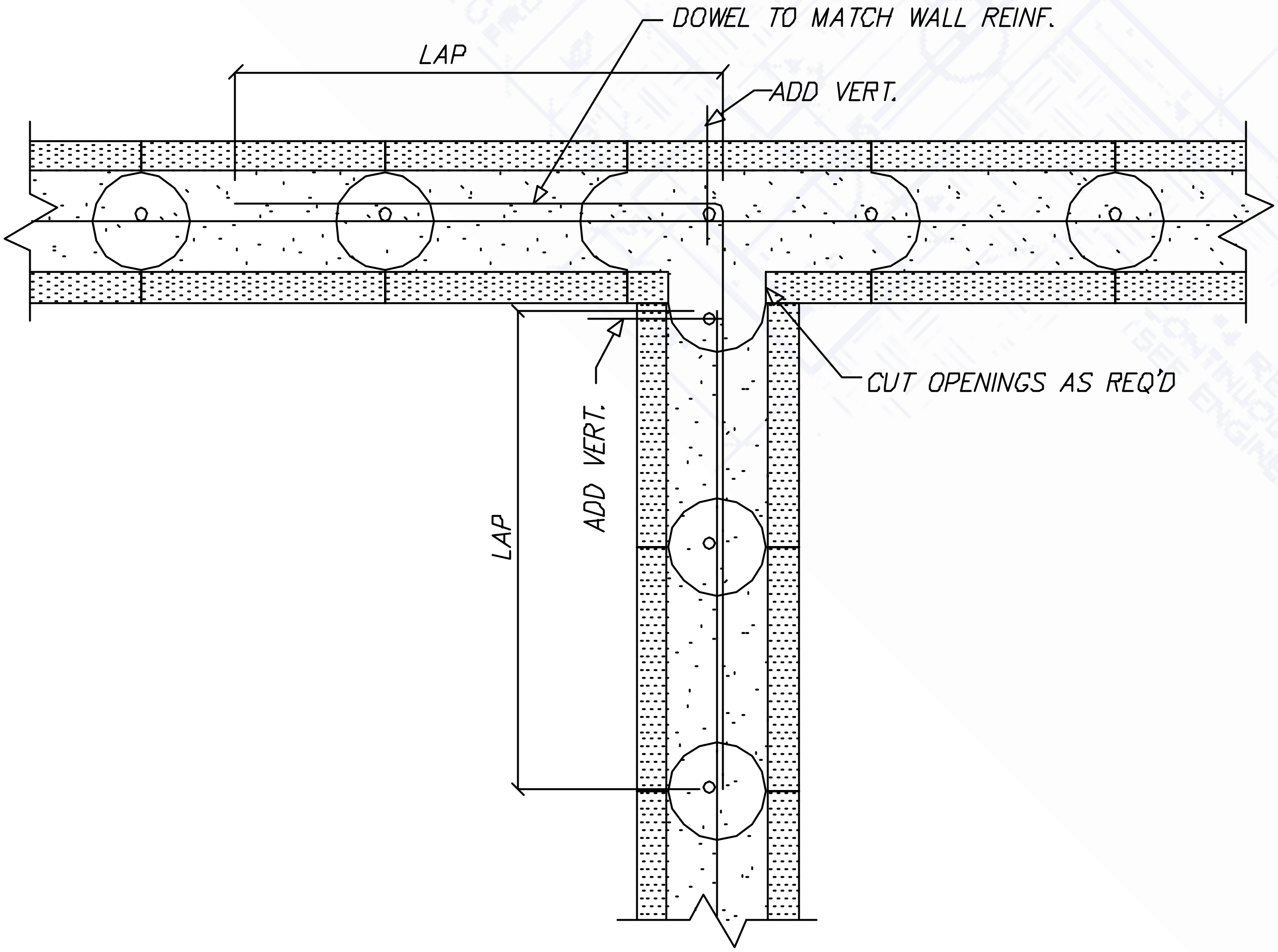
Corner Connection



PERFORM WALL PANEL SYSTEM

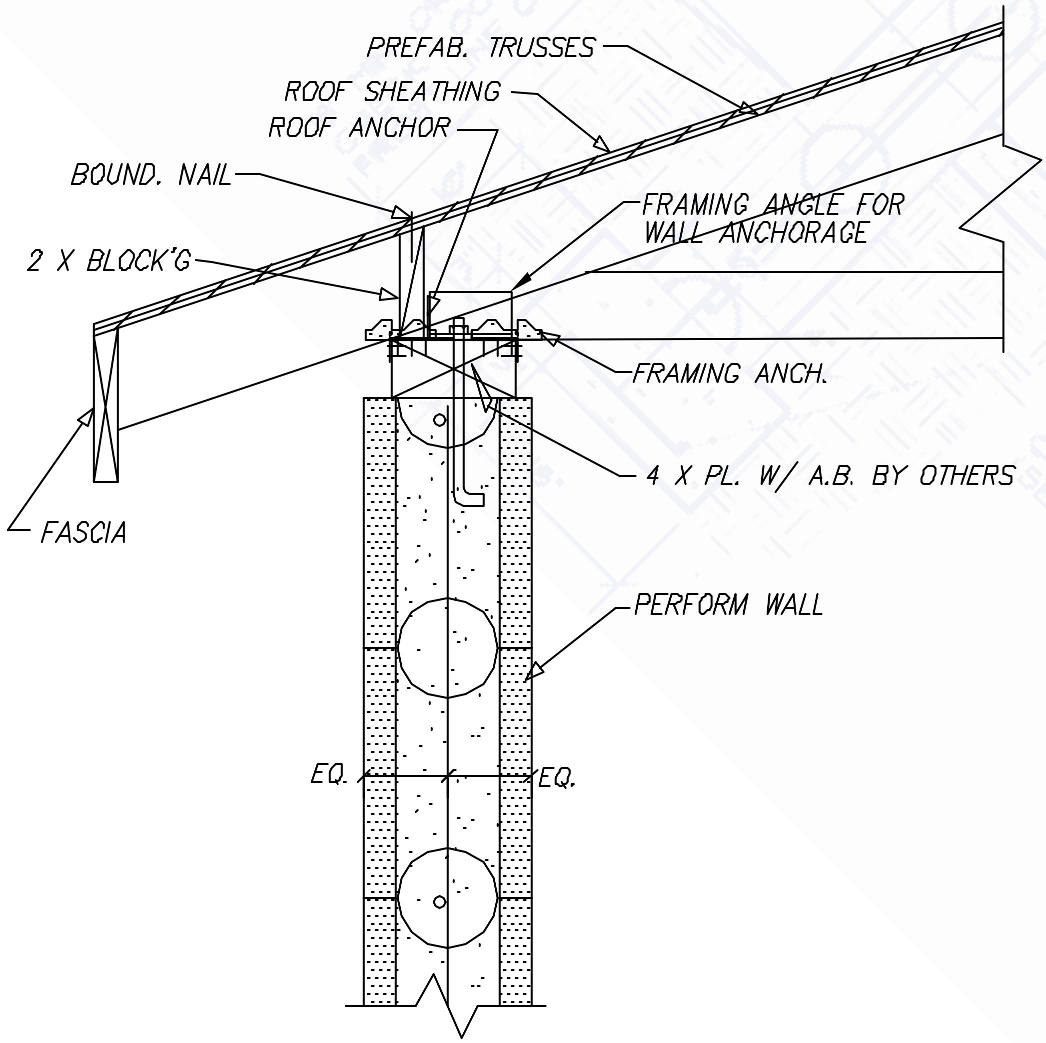
Detail No. 8

Interior Wall to Exterior Wall Connection



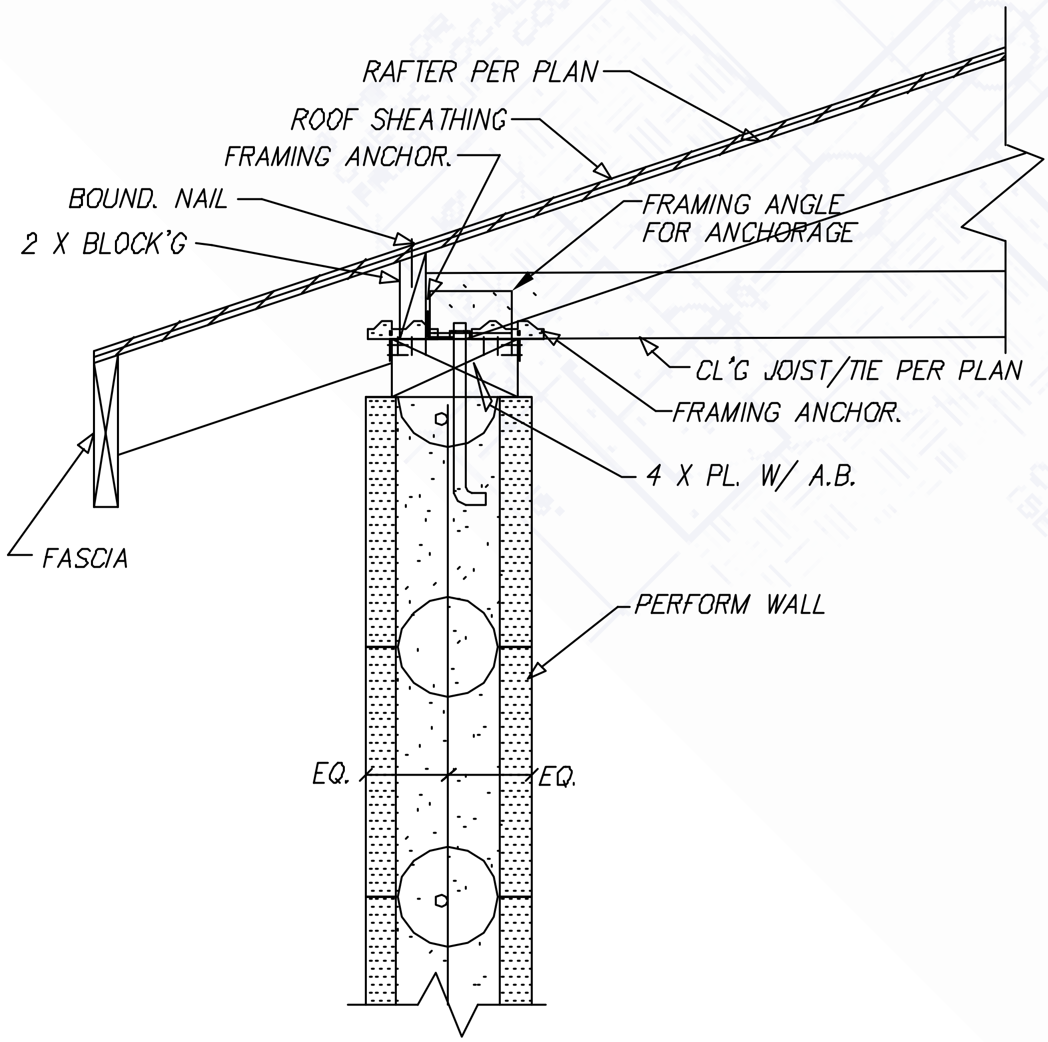
PERFORM WALL PANEL SYSTEM

Roof Truss/Wall Connection



Detail No. 10

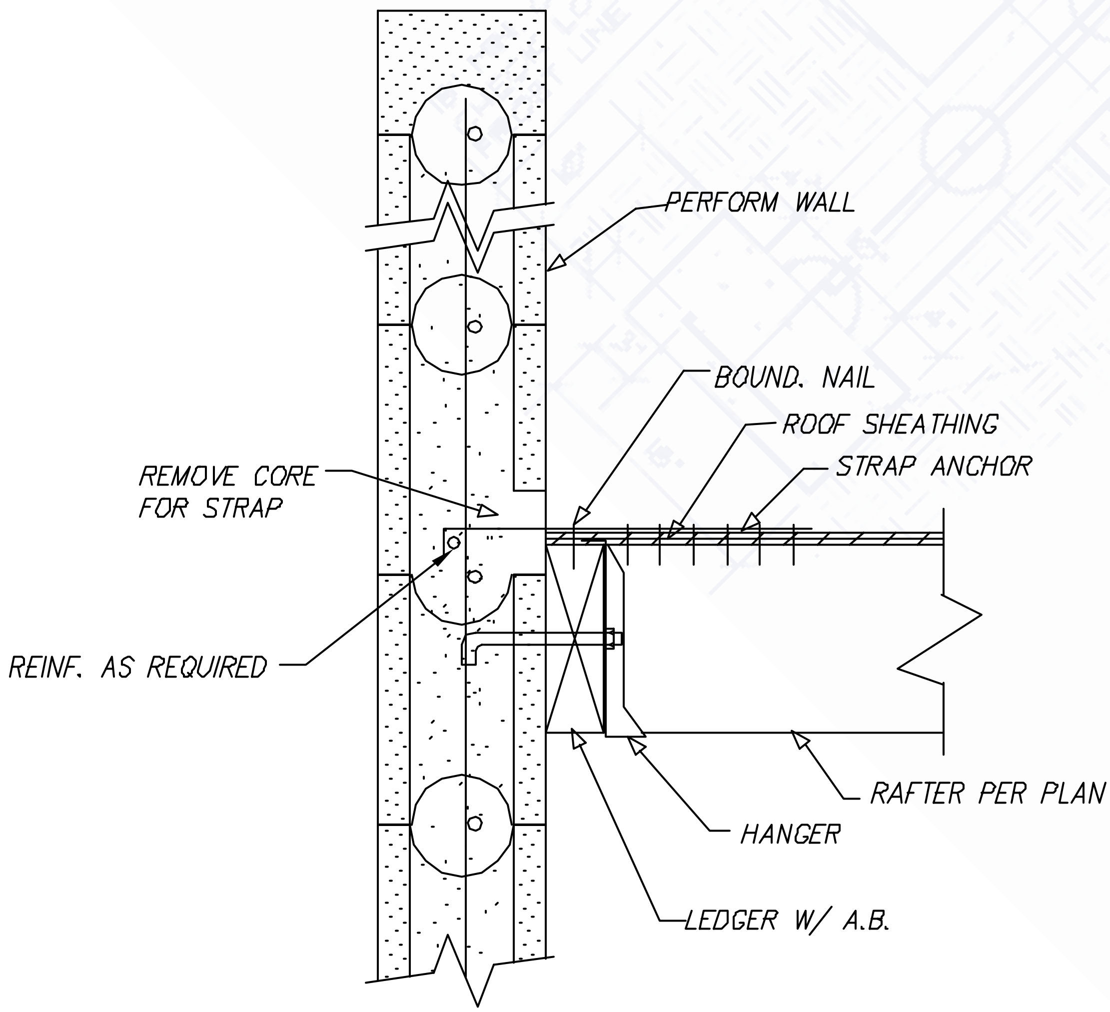
Roof Rafter/Wall Connection



PERFORM WALL PANEL SYSTEM

Detail No. 11

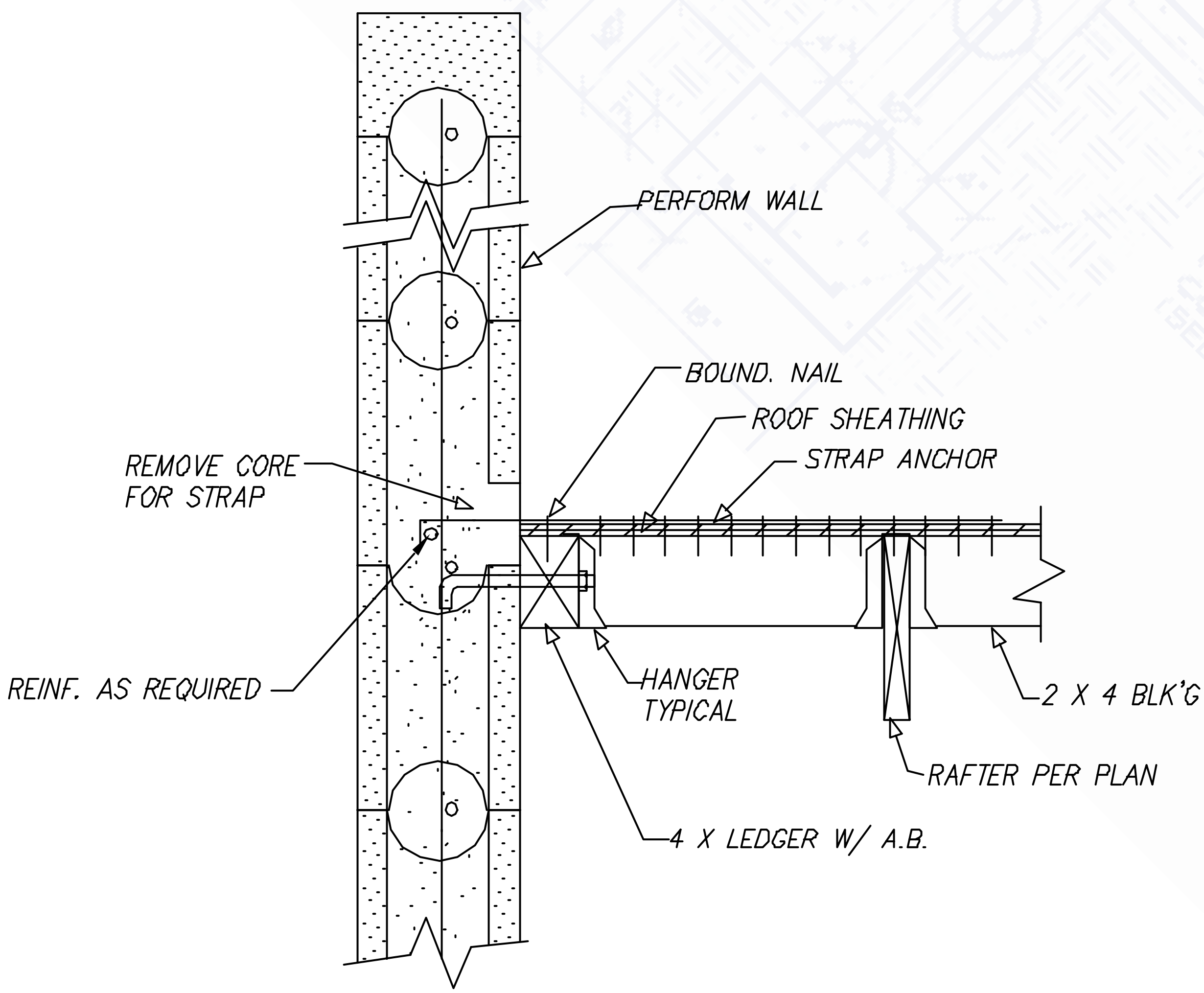
Wall with Parapet/Roof Connection



PERFORM WALL PANEL SYSTEM

Detail No. 12

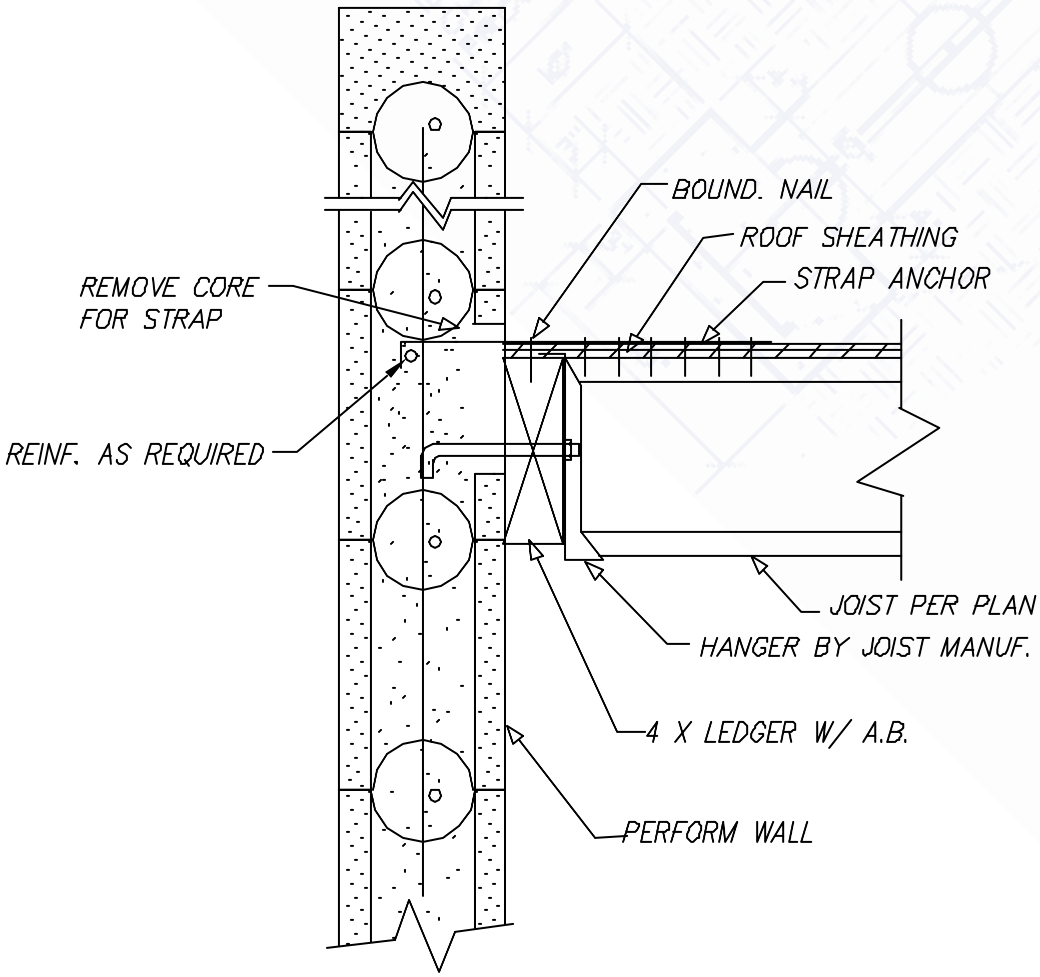
Wall with Parapet to Roof Connection, Wall Parallel to Joist



PERFORM WALL PANEL SYSTEM

Detail No. 13

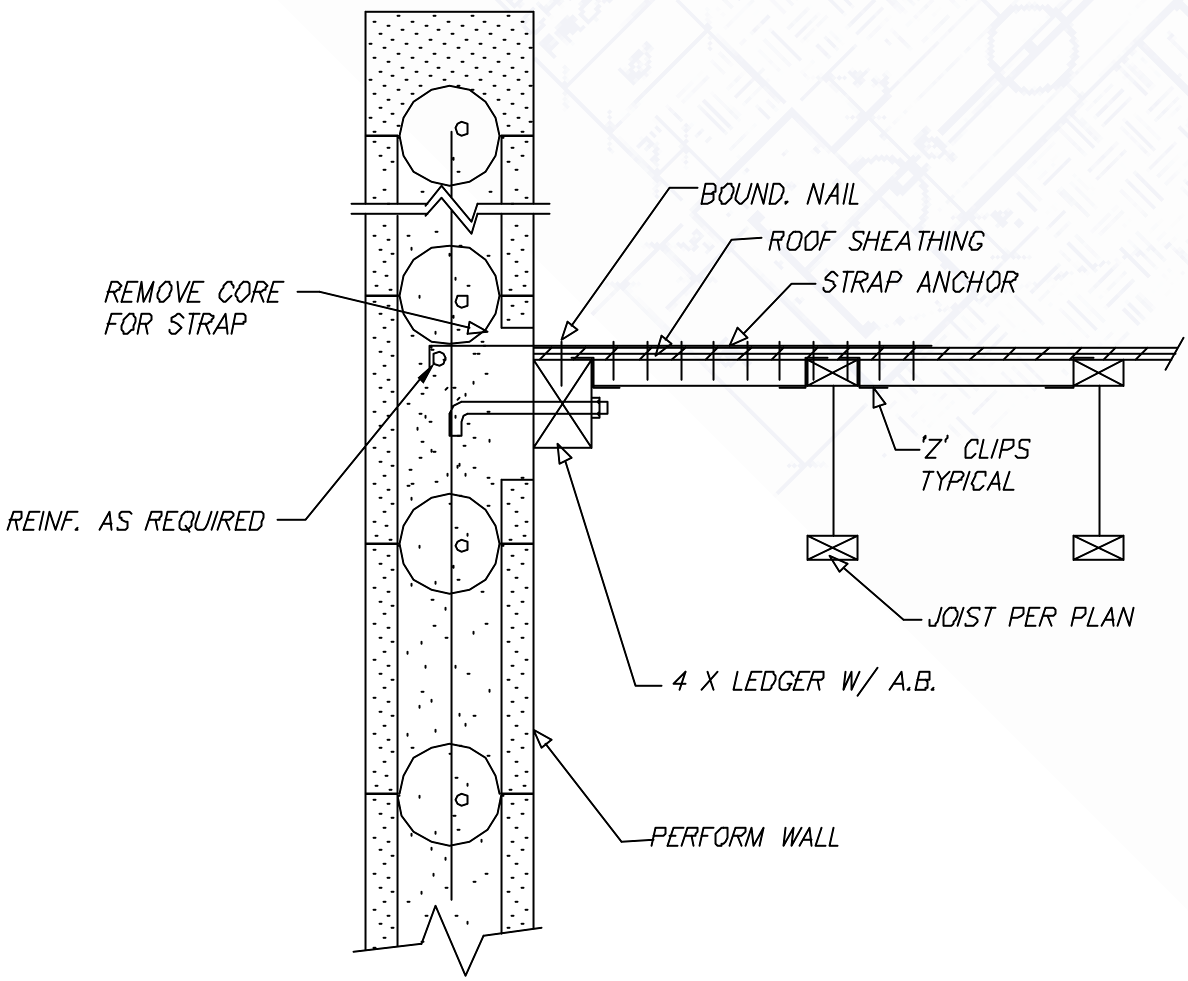
Wall with Parapet/Roof Joist Connection, Joist Parallel to Wall



PERFORM WALL PANEL SYSTEM

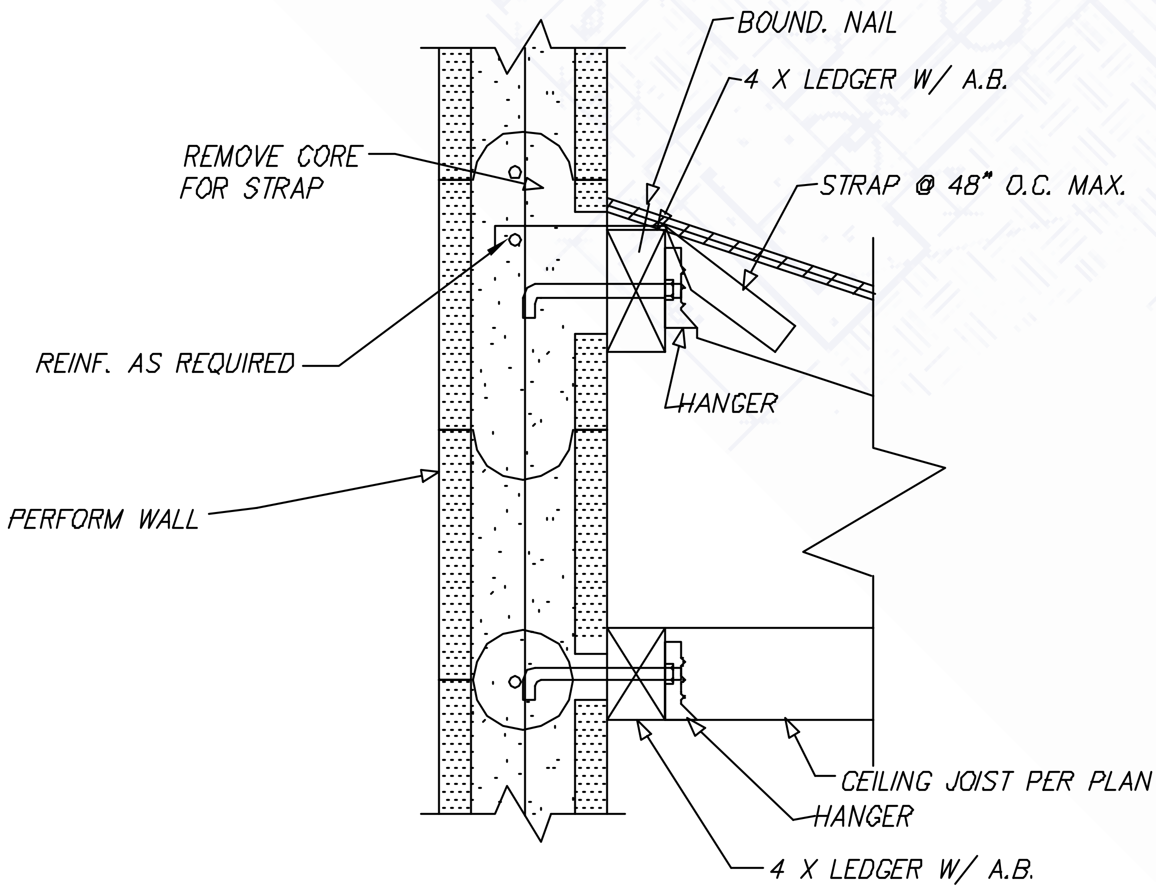
Detail No. 14

Wall with Parapet/Roof Joist Connection, Pre-fab Joist Parallel to Wall



PERFORM WALL PANEL SYSTEM

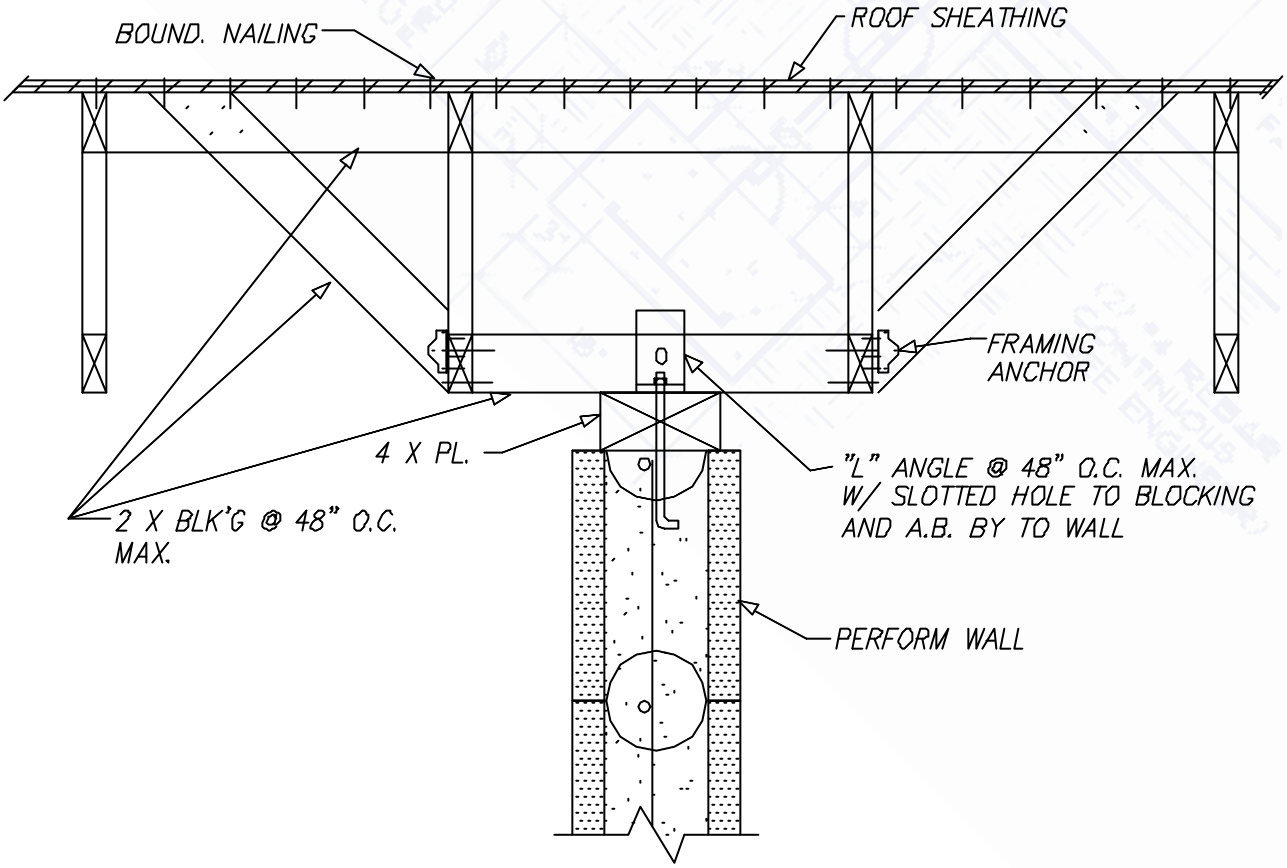
Roof to Wall Connection



PERFORM WALL PANEL SYSTEM

Detail No. 16

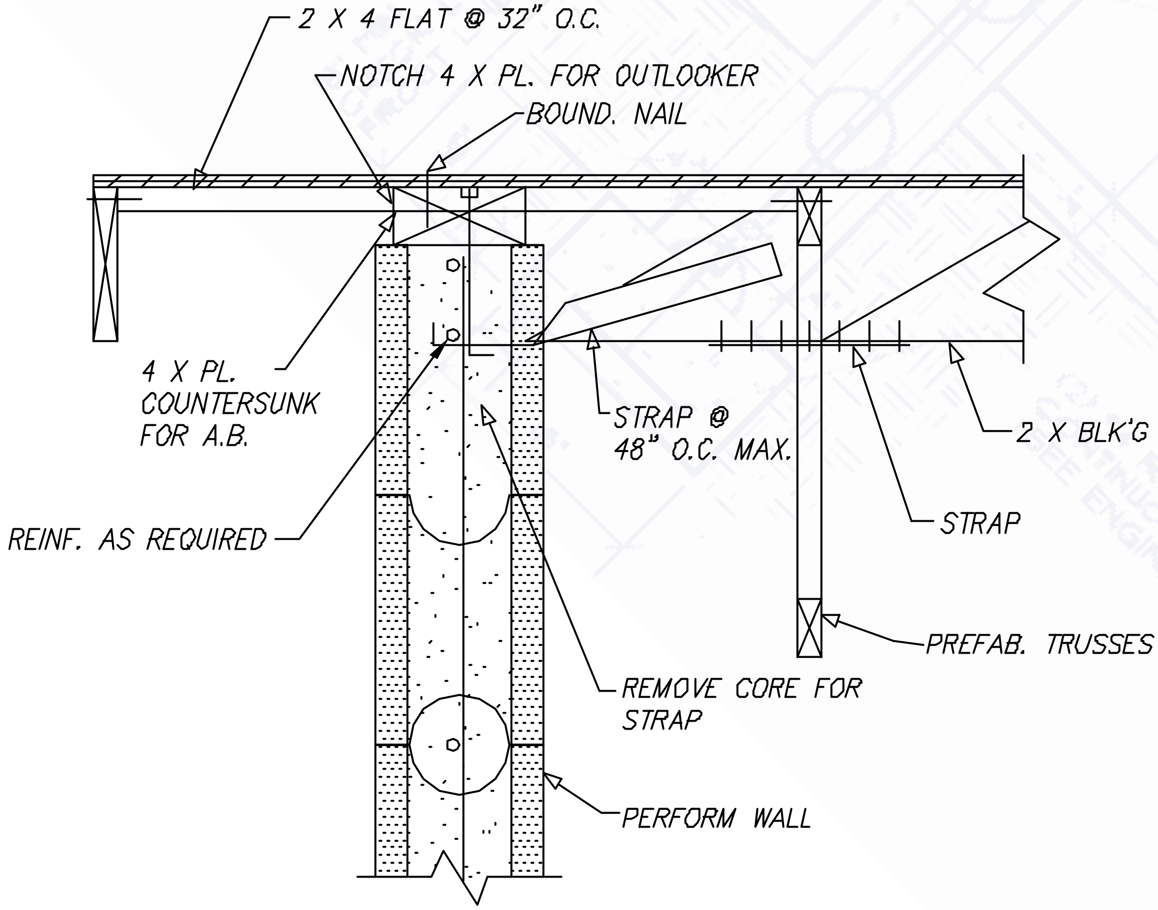
Interior Non-bearing Wall to Roof Connection



PERFORM WALL PANEL SYSTEM

Detail No. 17

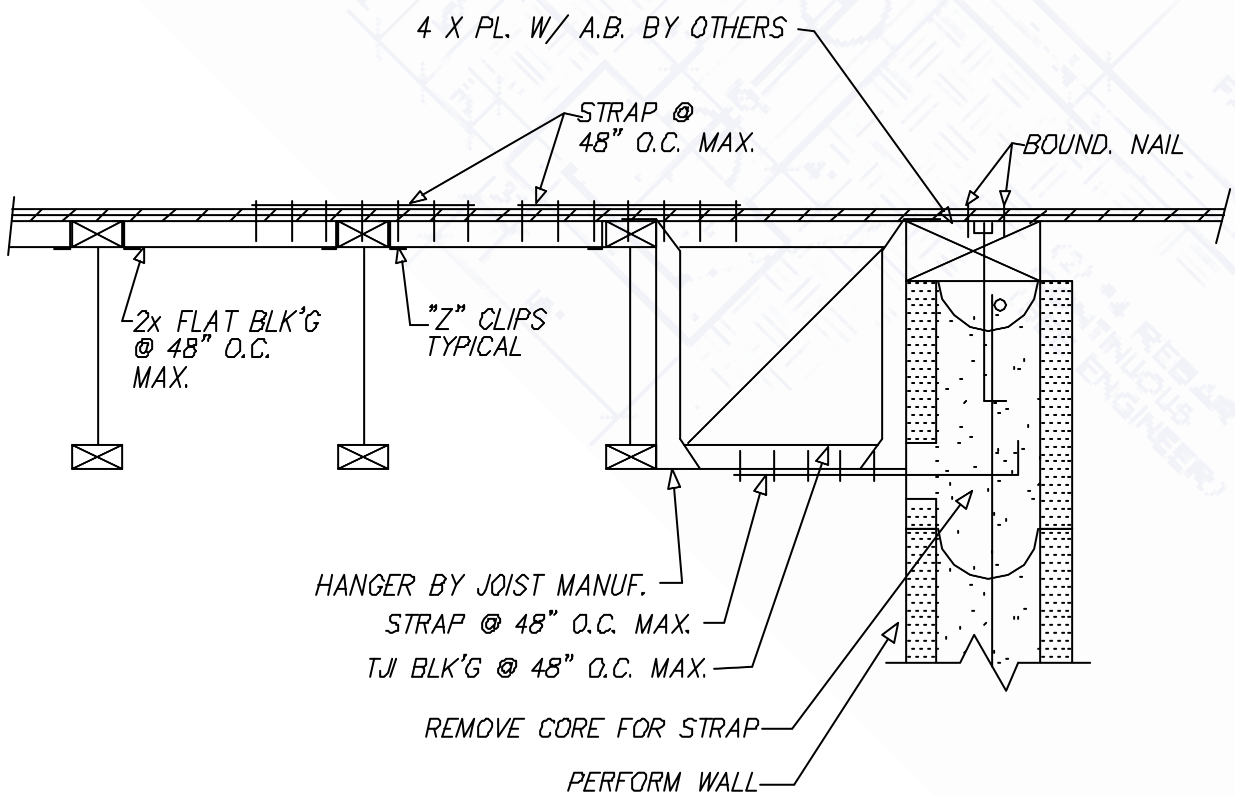
Gable End Wall to Roof Connection



PERFORM WALL PANEL SYSTEM

Detail No. 18

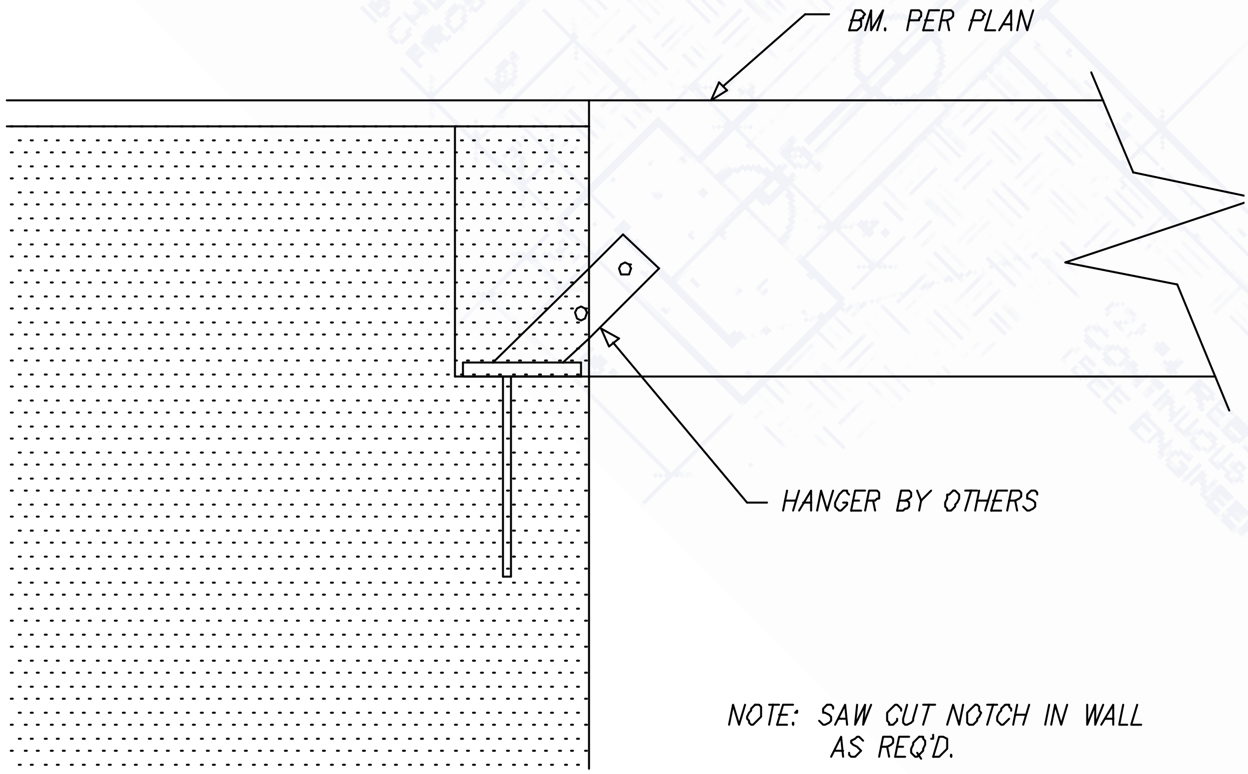
Interior Non-bearing Wall to Roof Connection



PERFORM WALL PANEL SYSTEM

Detail No. 19

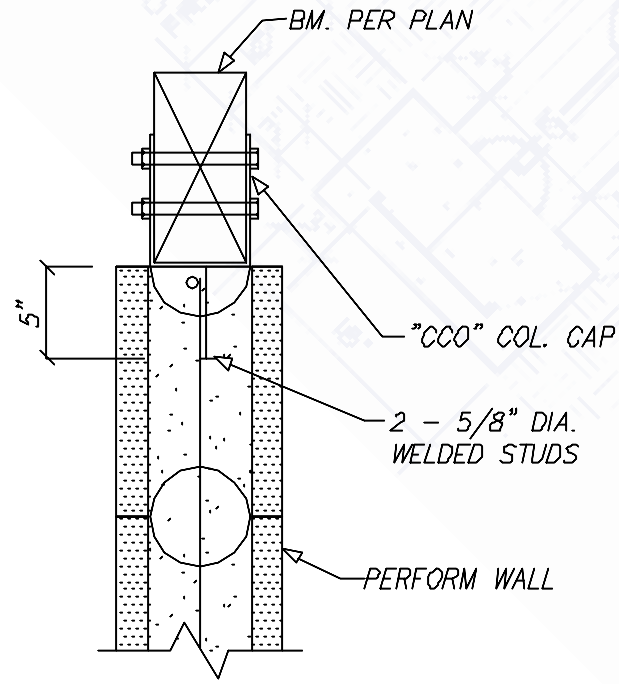
Beam/Wall Connection



PERFORM WALL PANEL SYSTEM

Detail No. 20

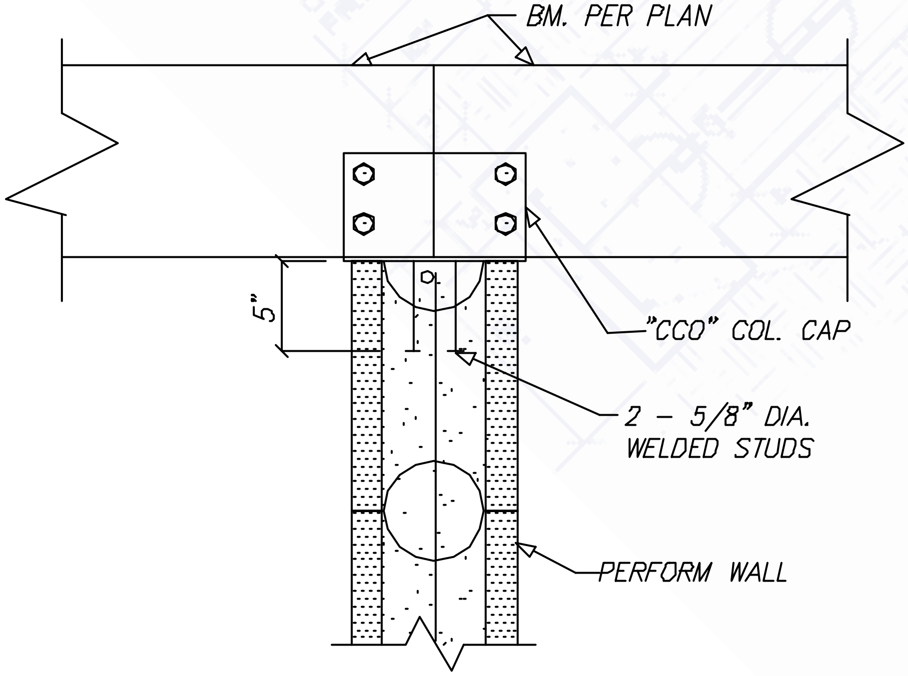
Wood Beam to Wall Connection



PERFORM WALL PANEL SYSTEM

Detail No. 21

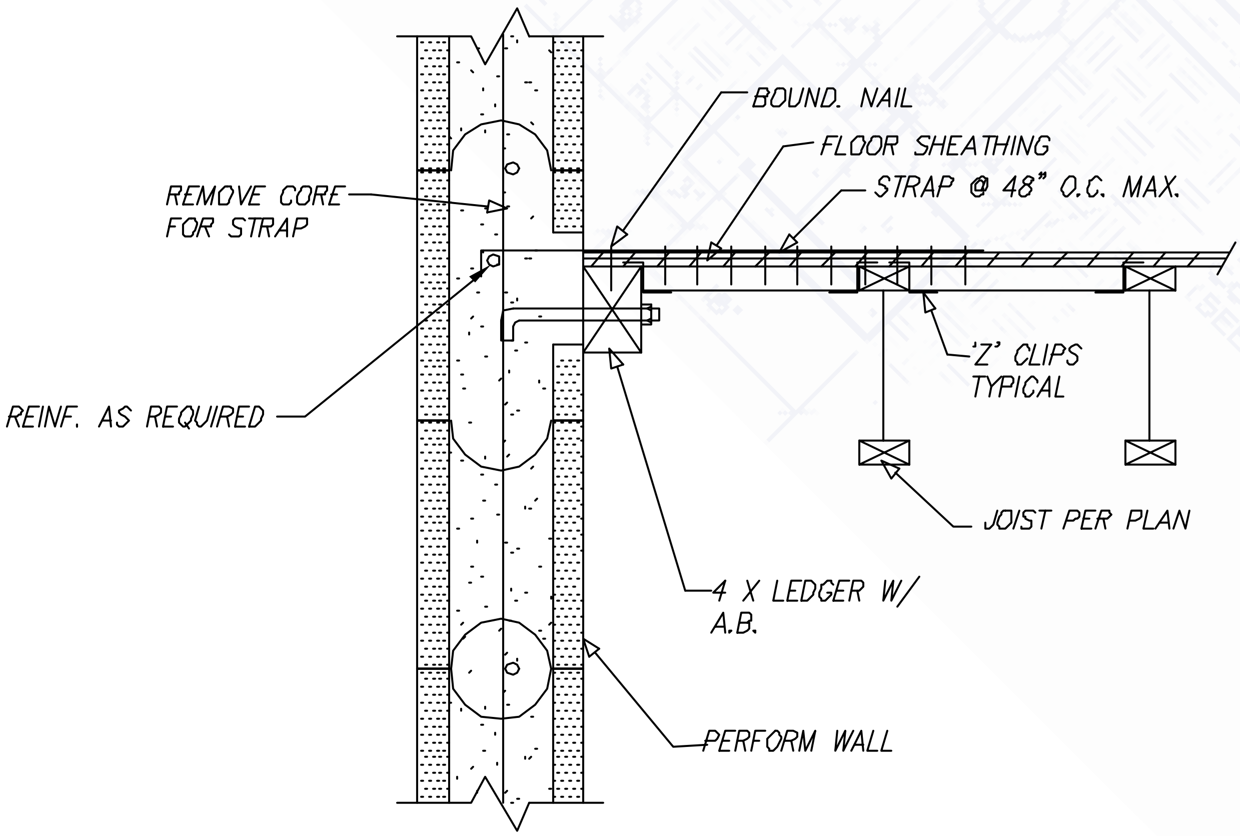
Wood Beam to Wall Connection



PERFORM WALL PANEL SYSTEM

Detail No. 22

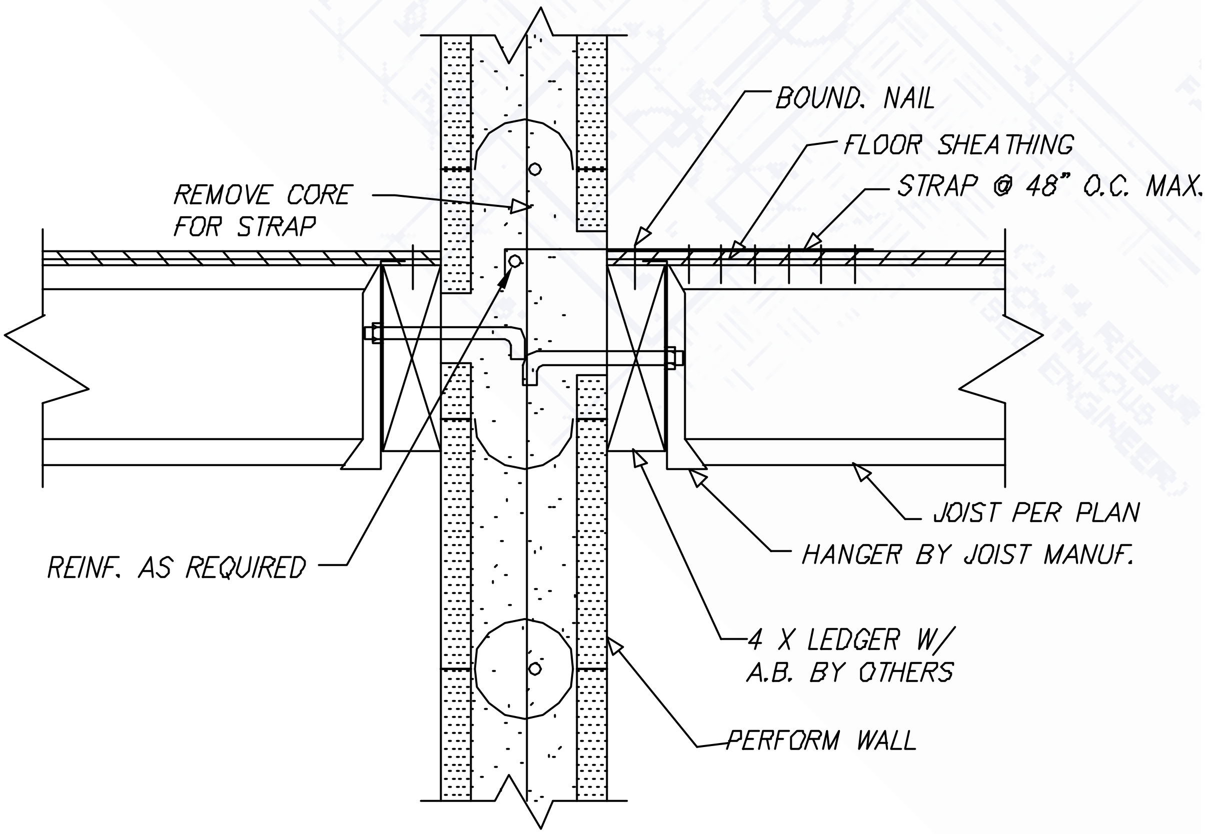
Wall to Floor Connection, Wall Parallel to Joist



PERFORM WALL PANEL SYSTEM

Detail No. 23

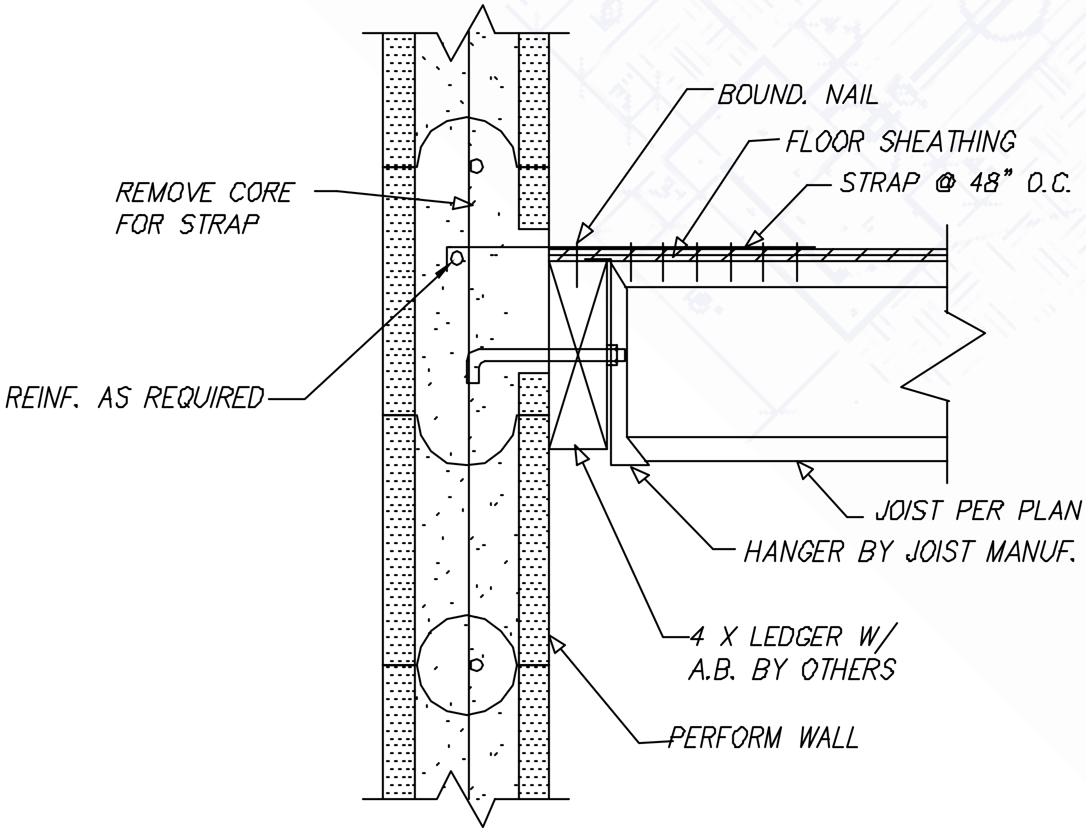
Interior Wall/Joist Connection, Wall Perpendicular to Joist



PERFORM WALL PANEL SYSTEM

Detail No. 24

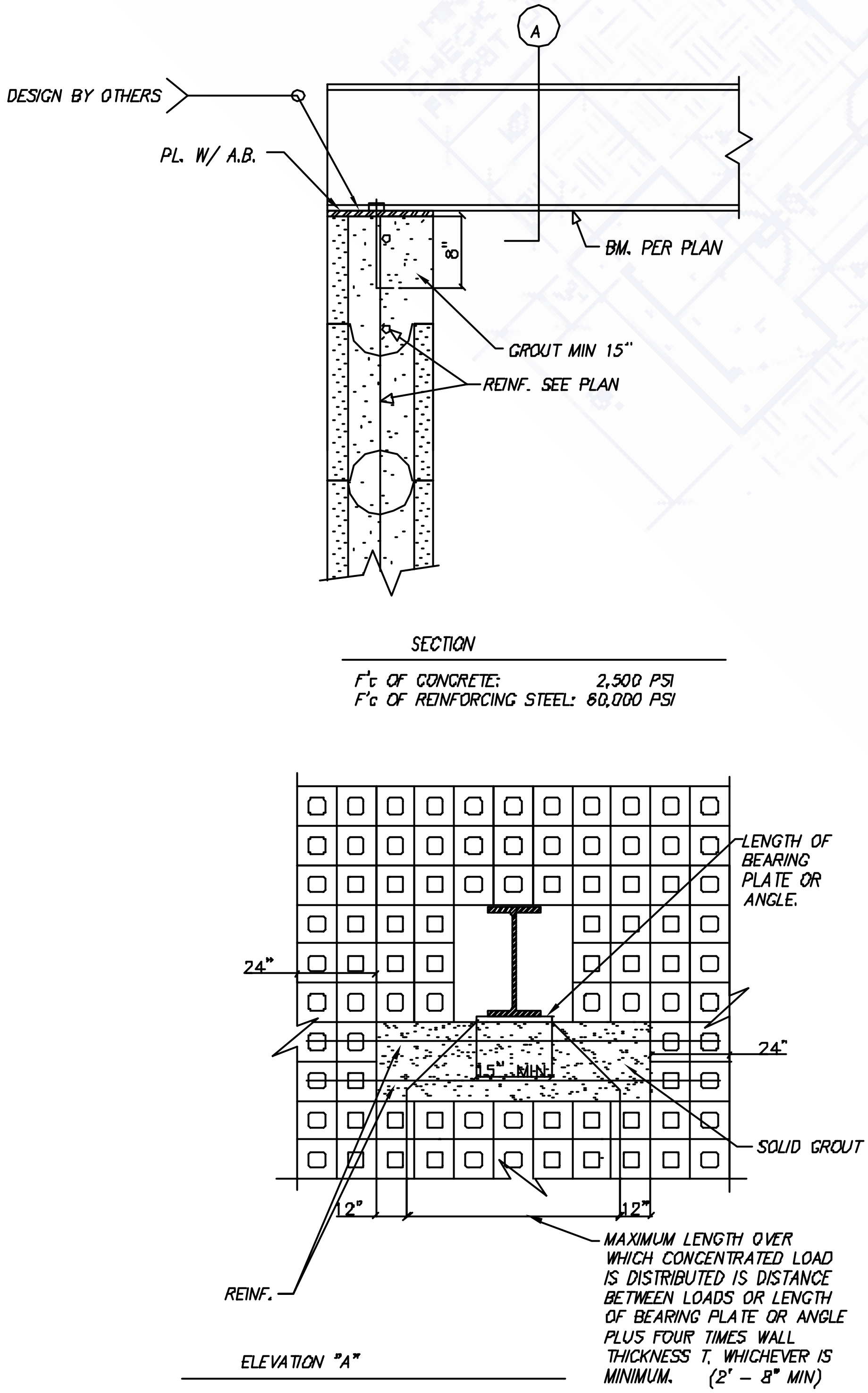
Wall to Floor Joist Connection



PERFORM WALL PANEL SYSTEM

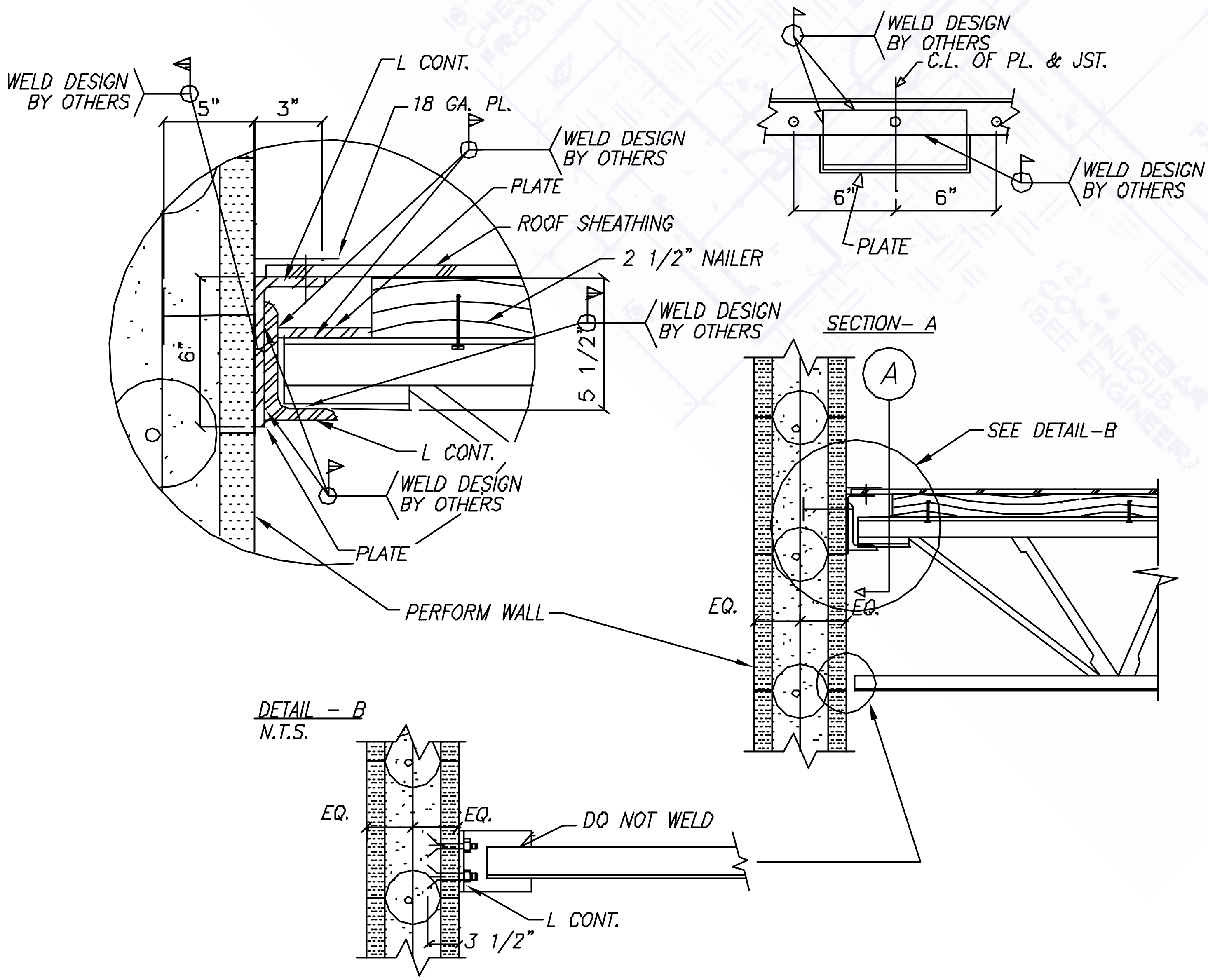
Detail No. 25

Steel Beam to Wall Connection



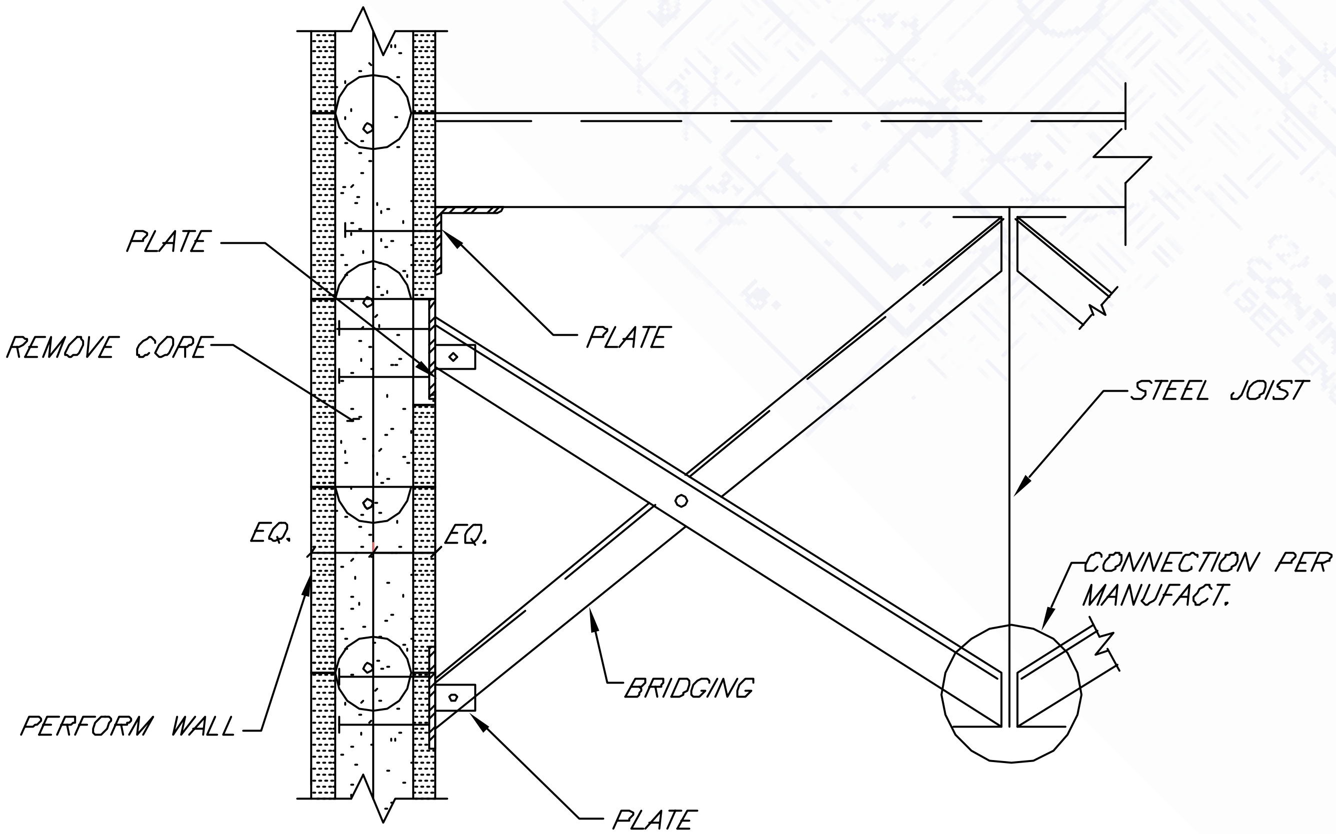
Detail No. 26

Steel Truss to Wall Connection



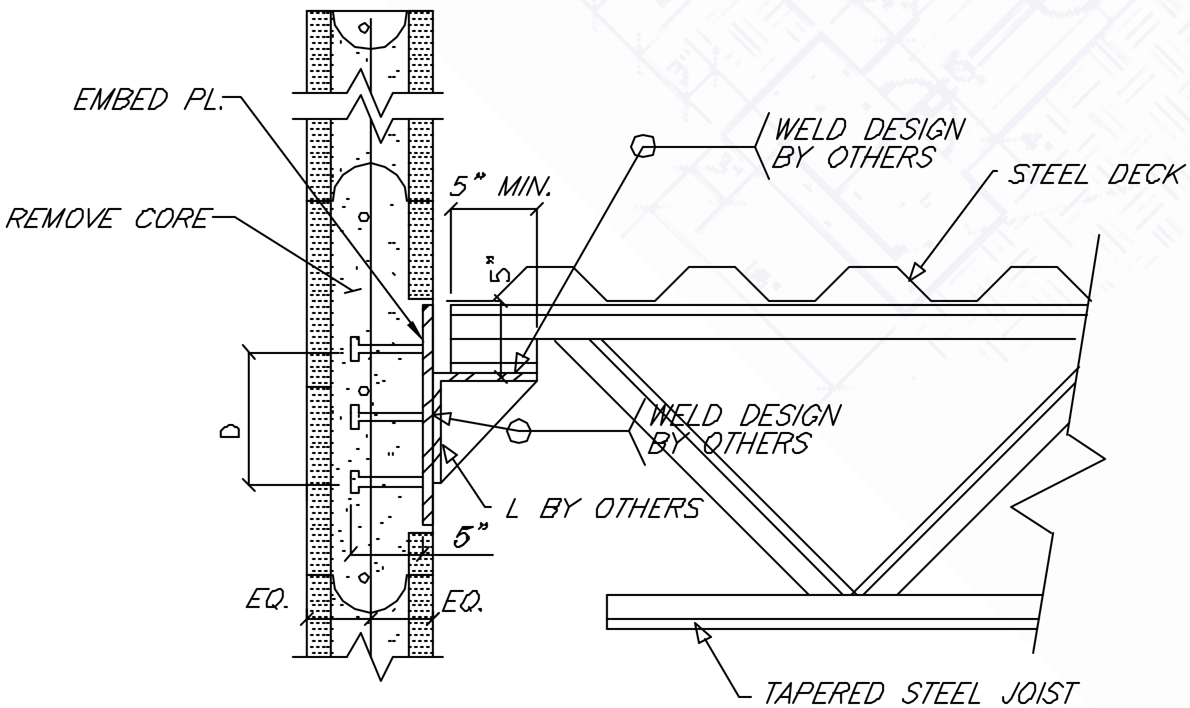
Detail No. 27

Steel Bracing/Bridging to Wall



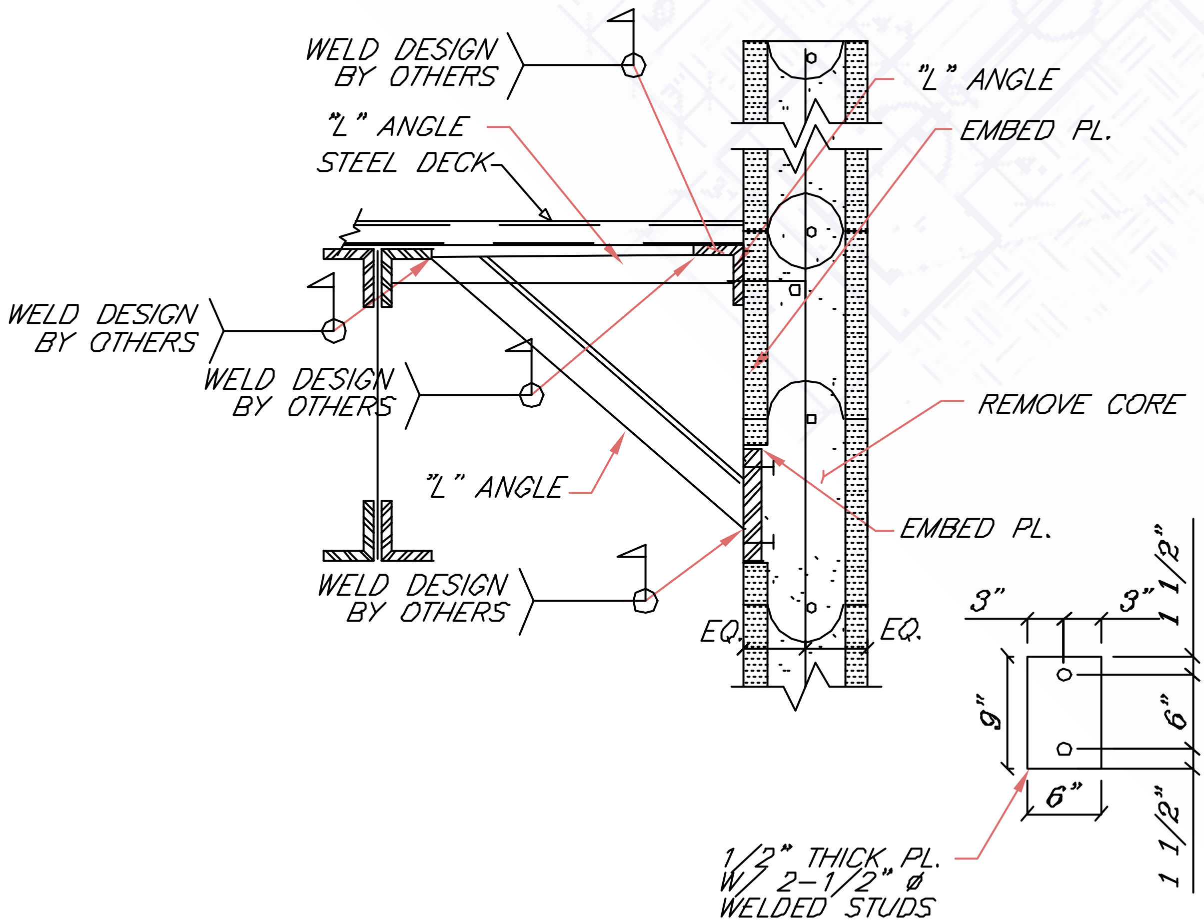
PERFORM WALL PANEL SYSTEM

Steel Joist to Wall



Detail No. 29

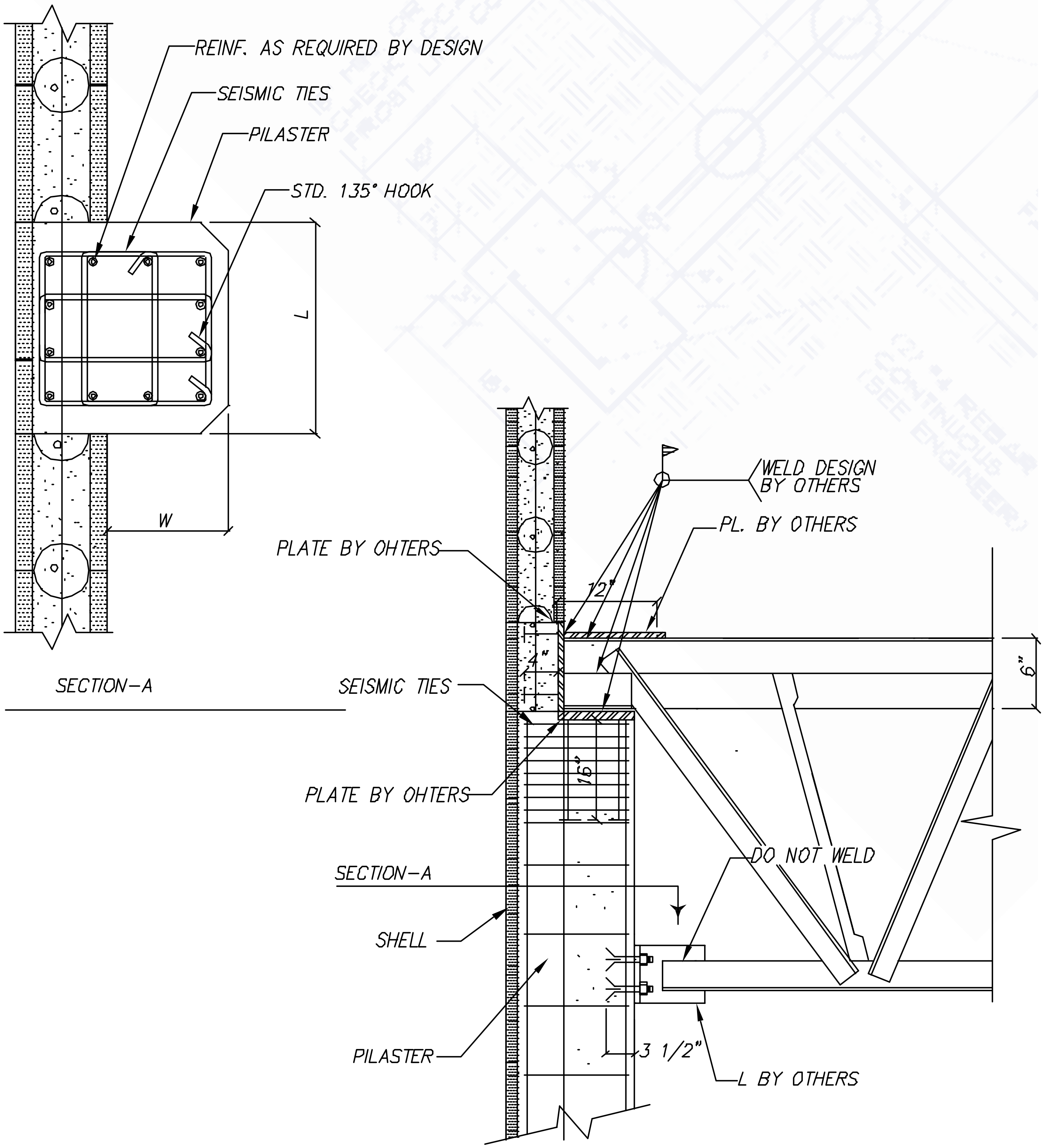
Steel Bridging to Wall



PERFORM WALL PANEL SYSTEM

Detail No. 30

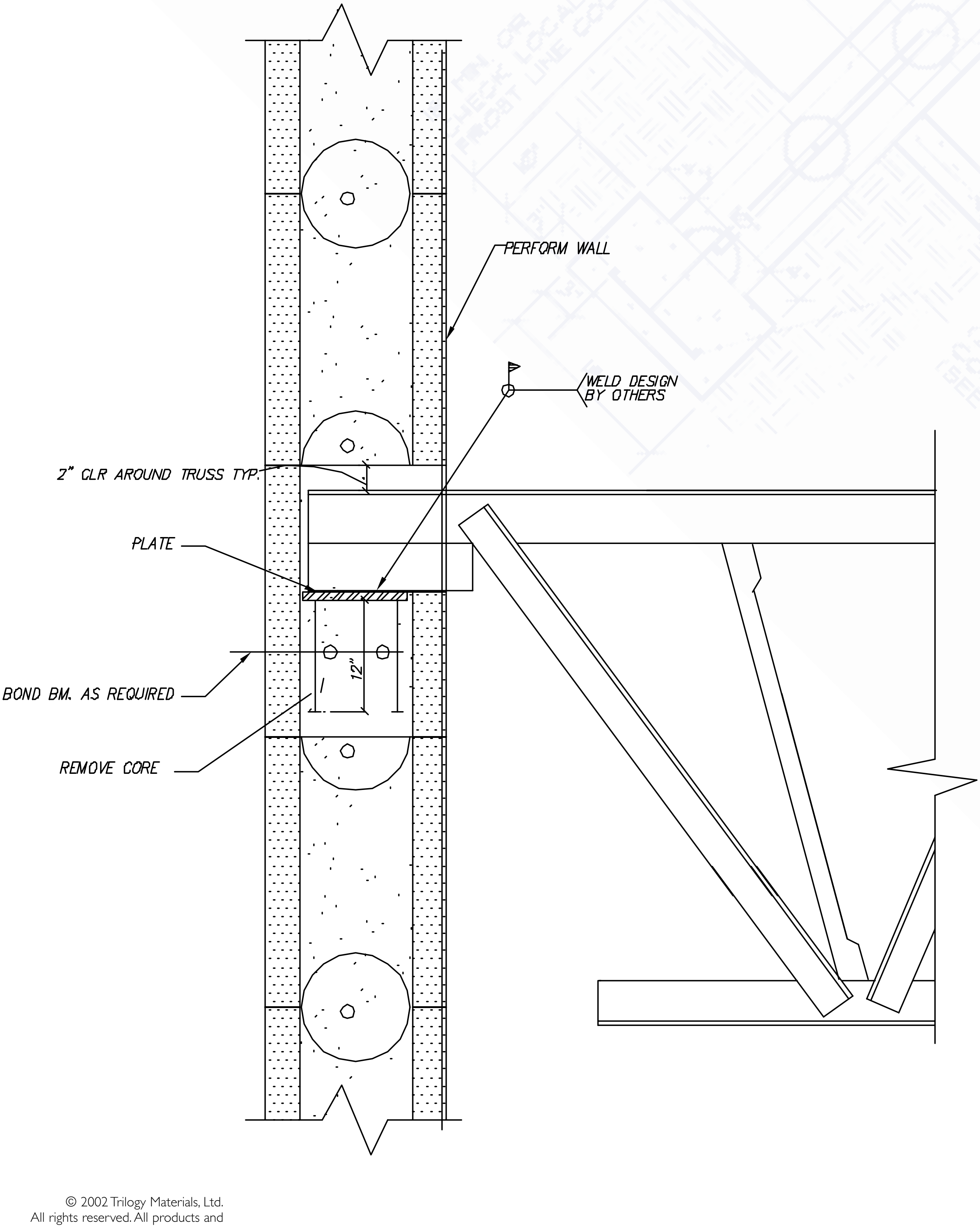
Steel Girders to Pilaster



PERFORM WALL PANEL SYSTEM

Detail No. 31

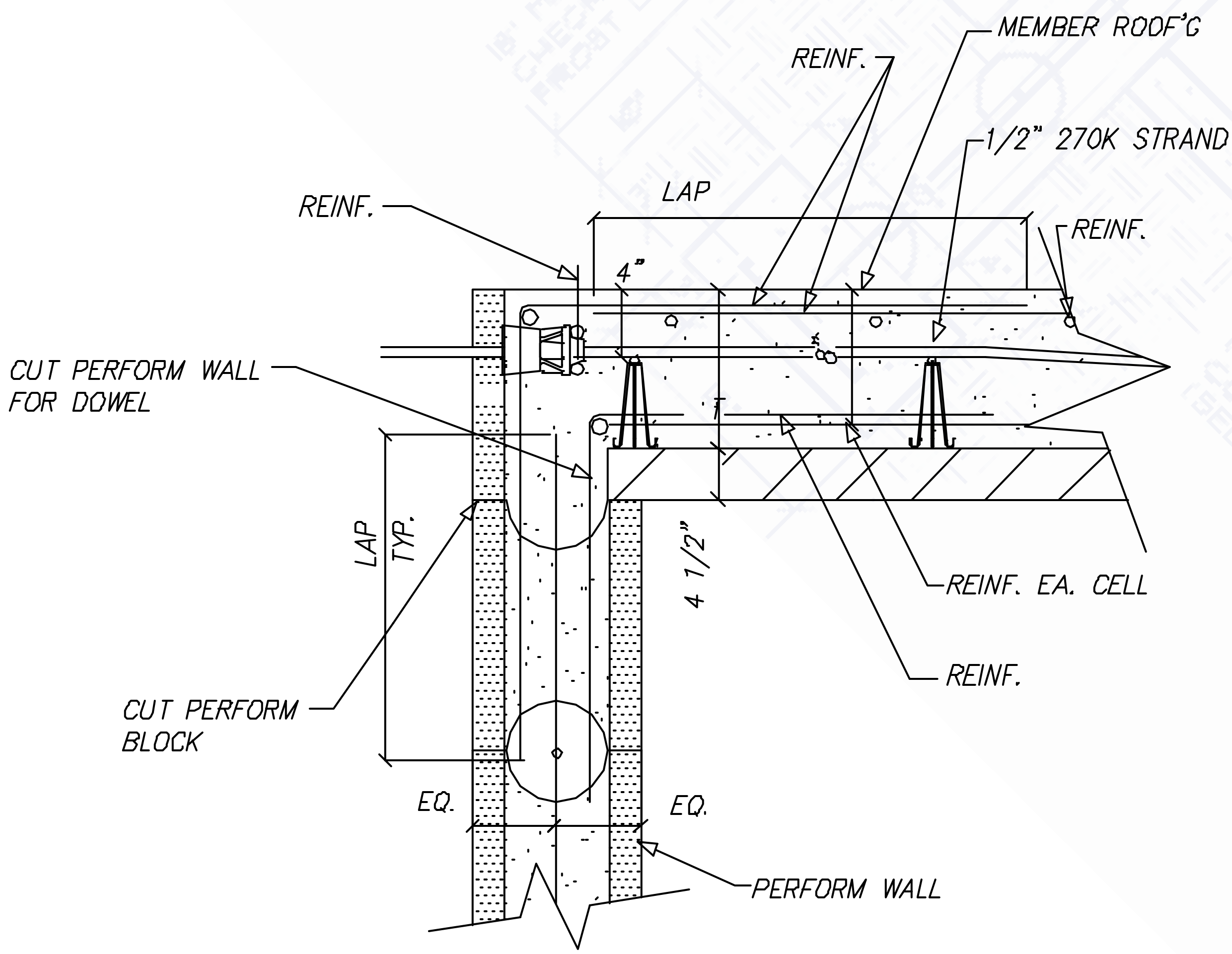
Typical Steel Joist to Wall Connection



PERFORM WALL PANEL SYSTEM

Detail No. 32

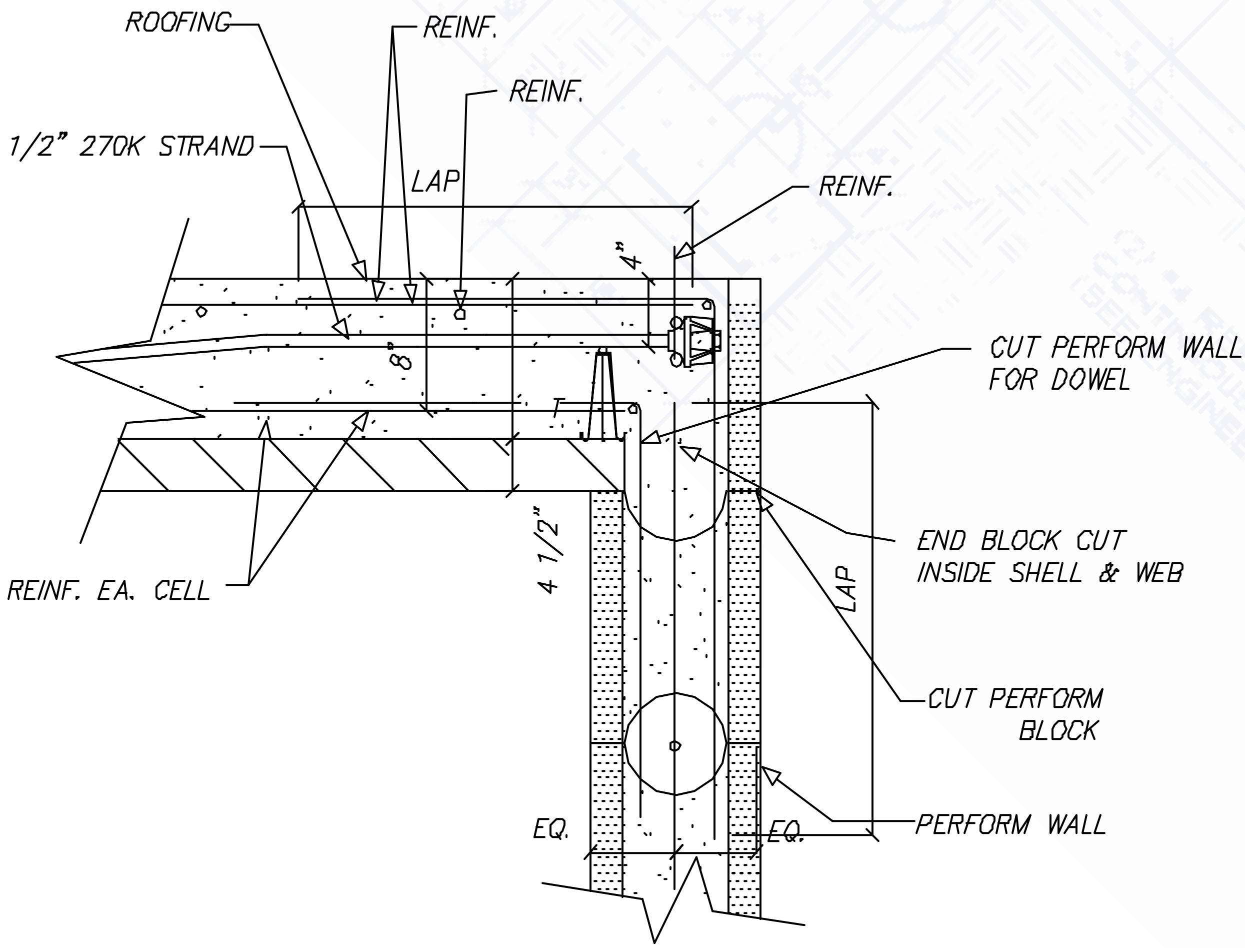
Post Tension Slab to Wall Connection



PERFORM WALL PANEL SYSTEM

Detail No. 33

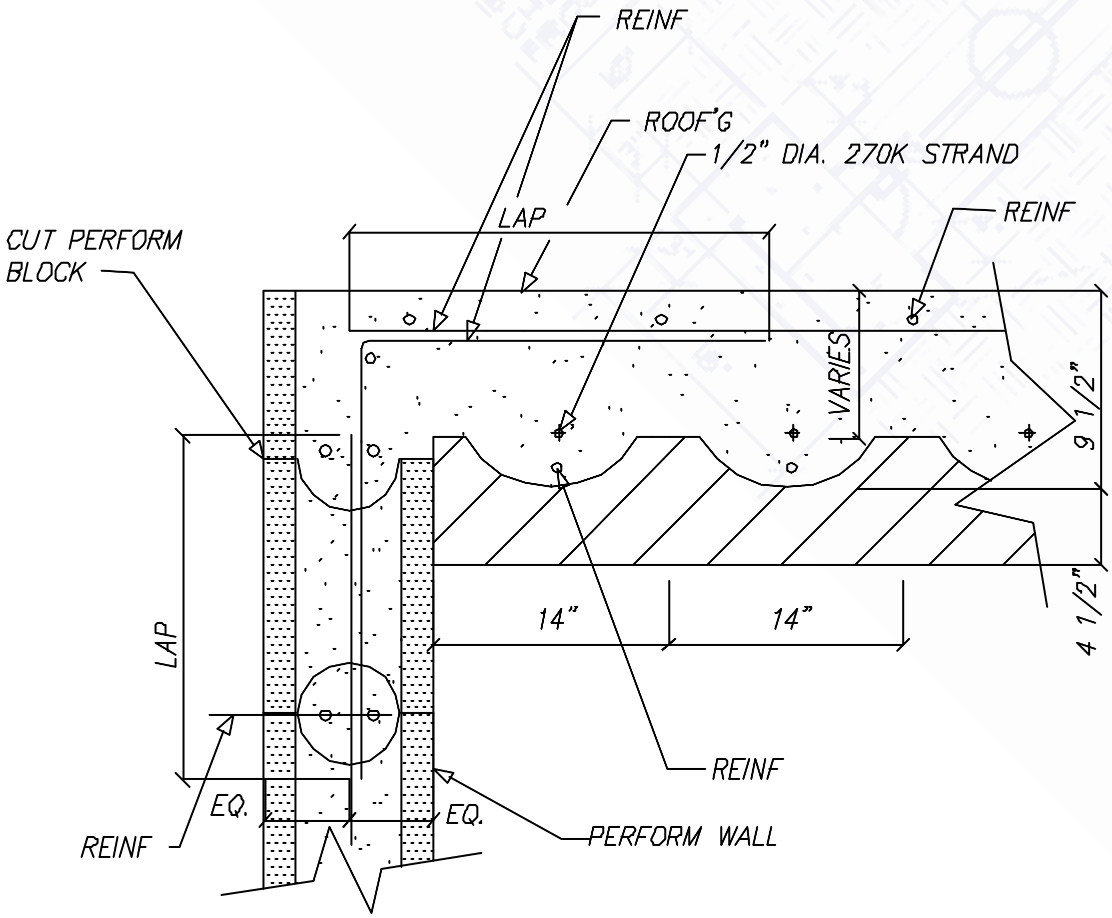
Post Tension Slab to Wall Connection



PERFORM WALL PANEL SYSTEM

Detail No. 34

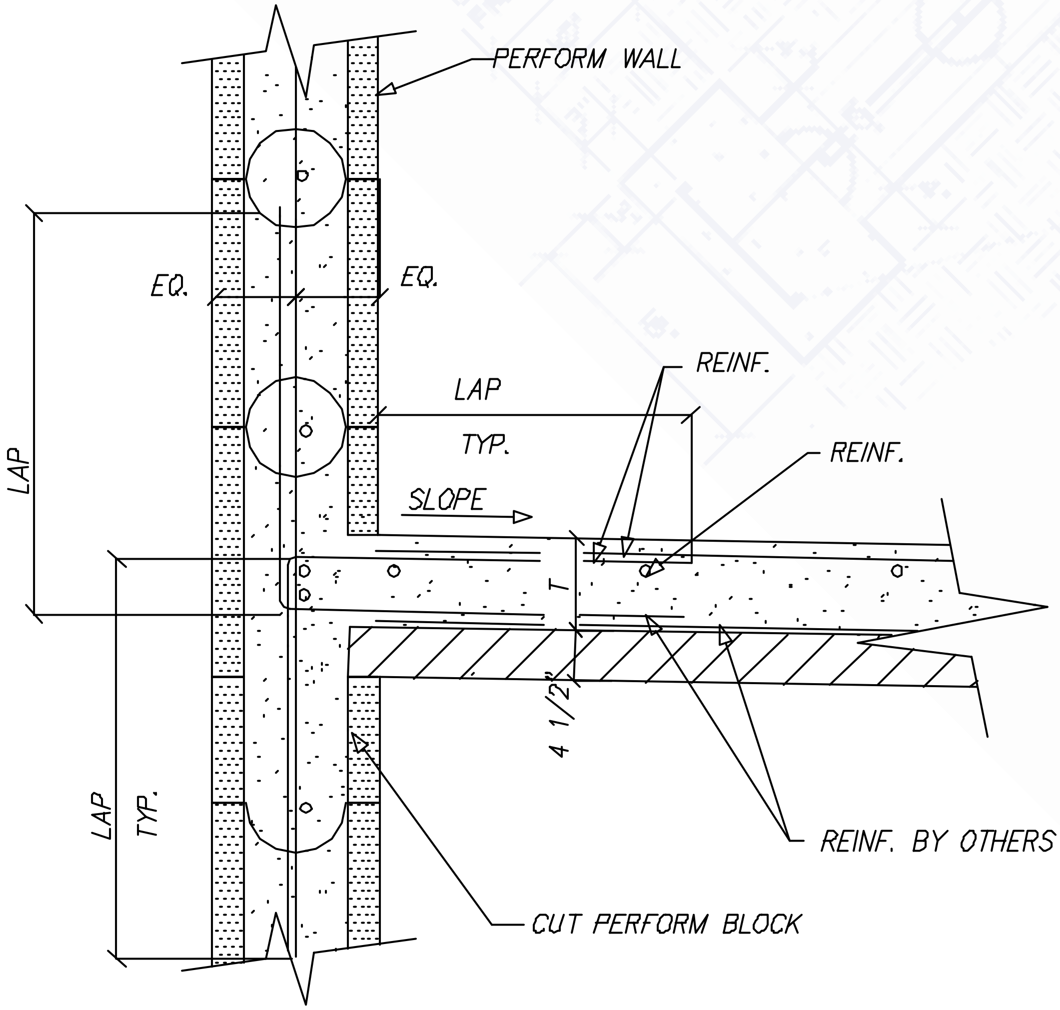
Concrete Slab to Wall Connection



PERFORM WALL PANEL SYSTEM

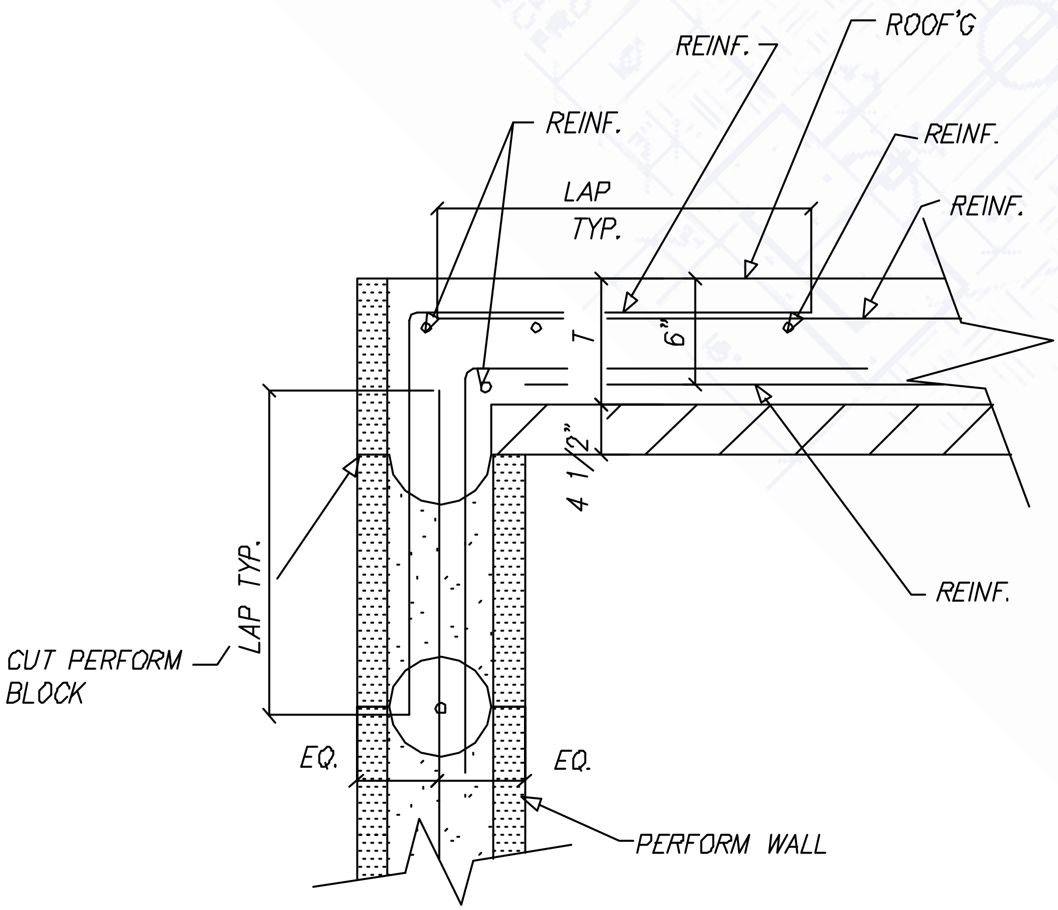
Detail No. 35

Concrete Slab to Wall Connection



Detail No. 36

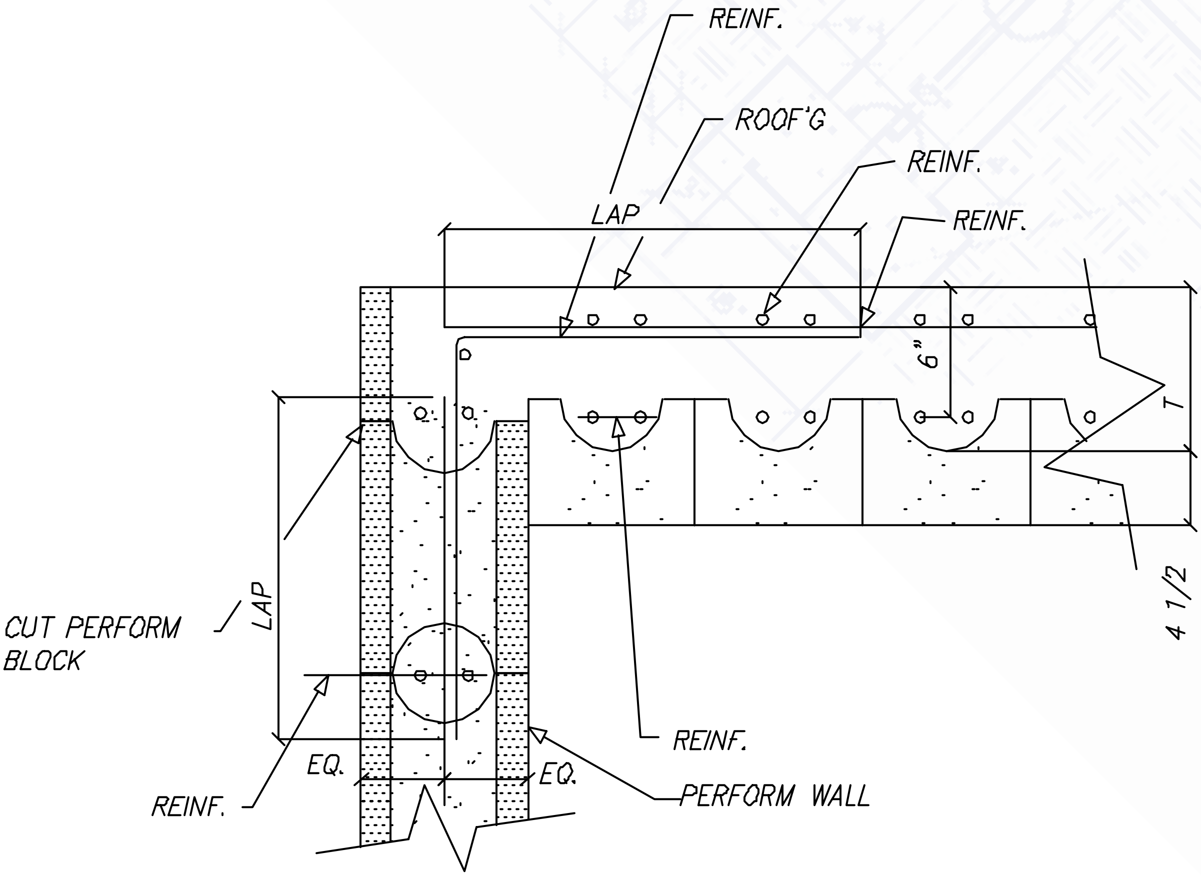
Concrete Slab to Wall Connection



PERFORM WALL PANEL SYSTEM

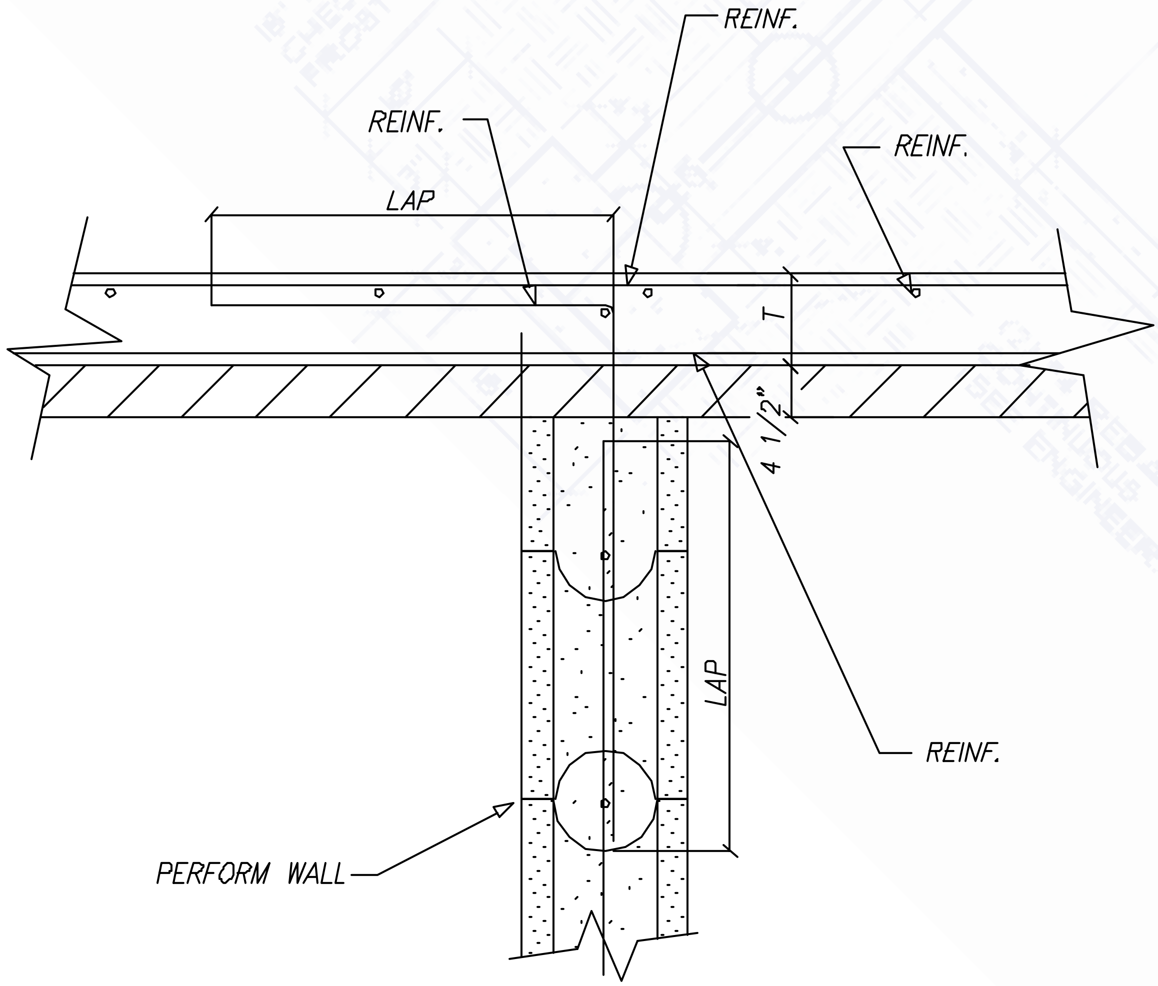
Detail No. 37

Concrete Slab to Wall Connection



Detail No. 38

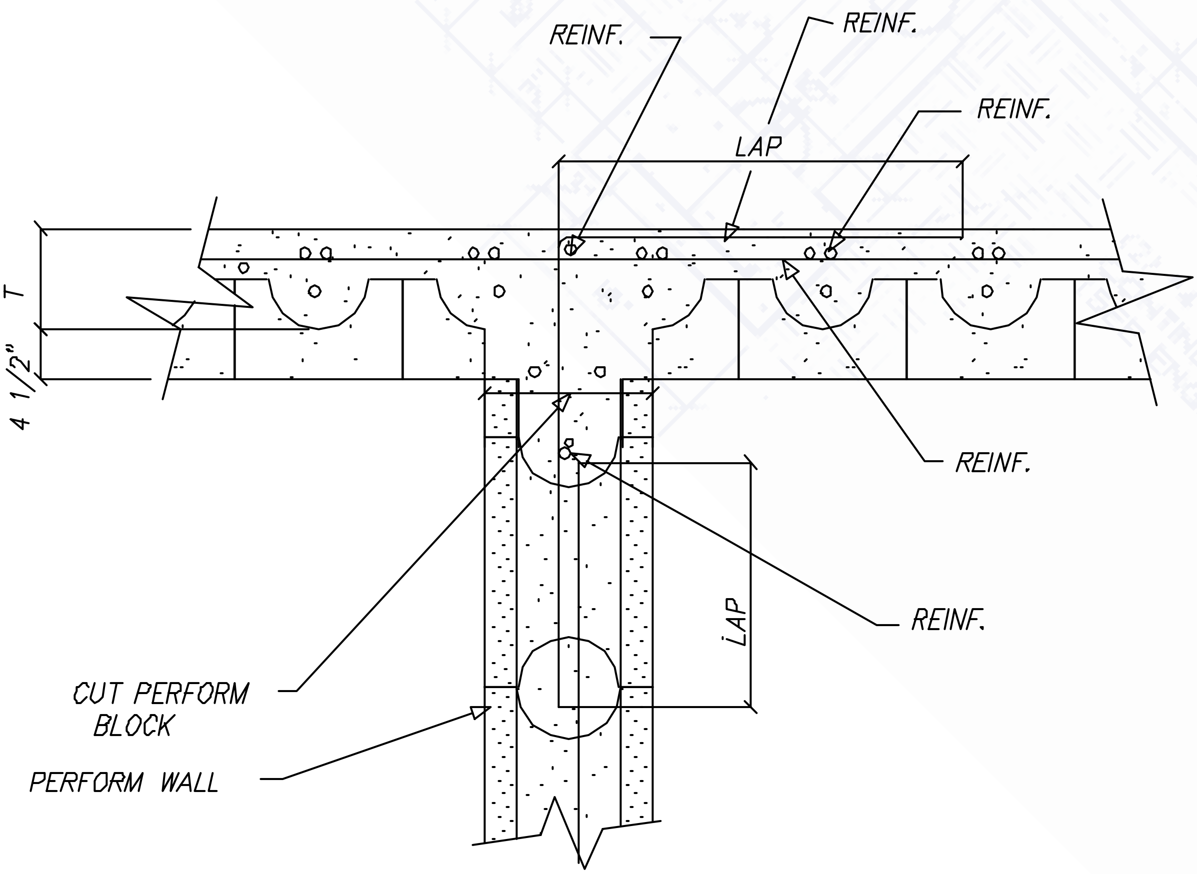
Concrete Slab to Wall Connection



PERFORM WALL PANEL SYSTEM

Detail No. 39

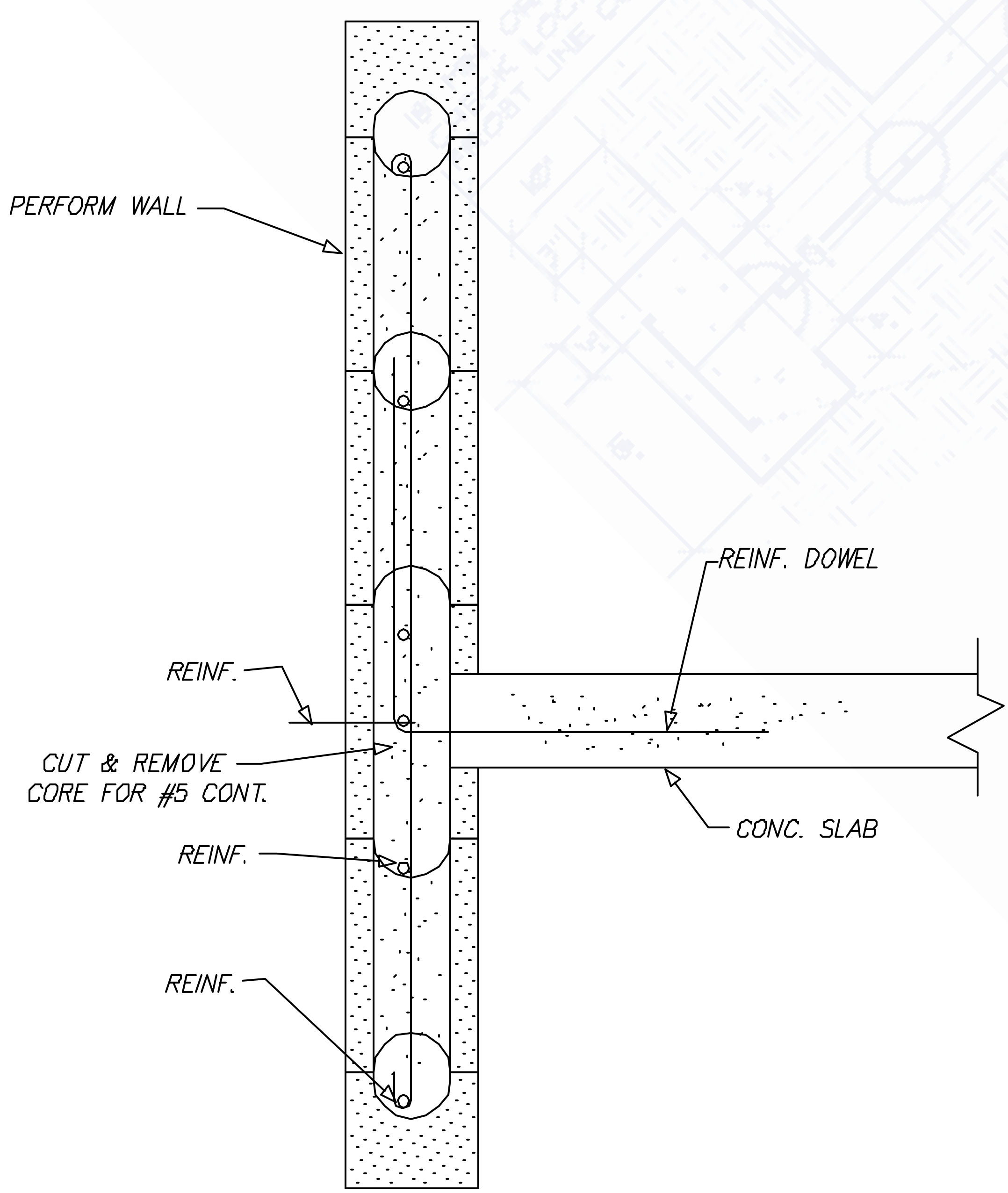
Concrete Slab to Wall Connection



PERFORM WALL PANEL SYSTEM

Detail No. 40

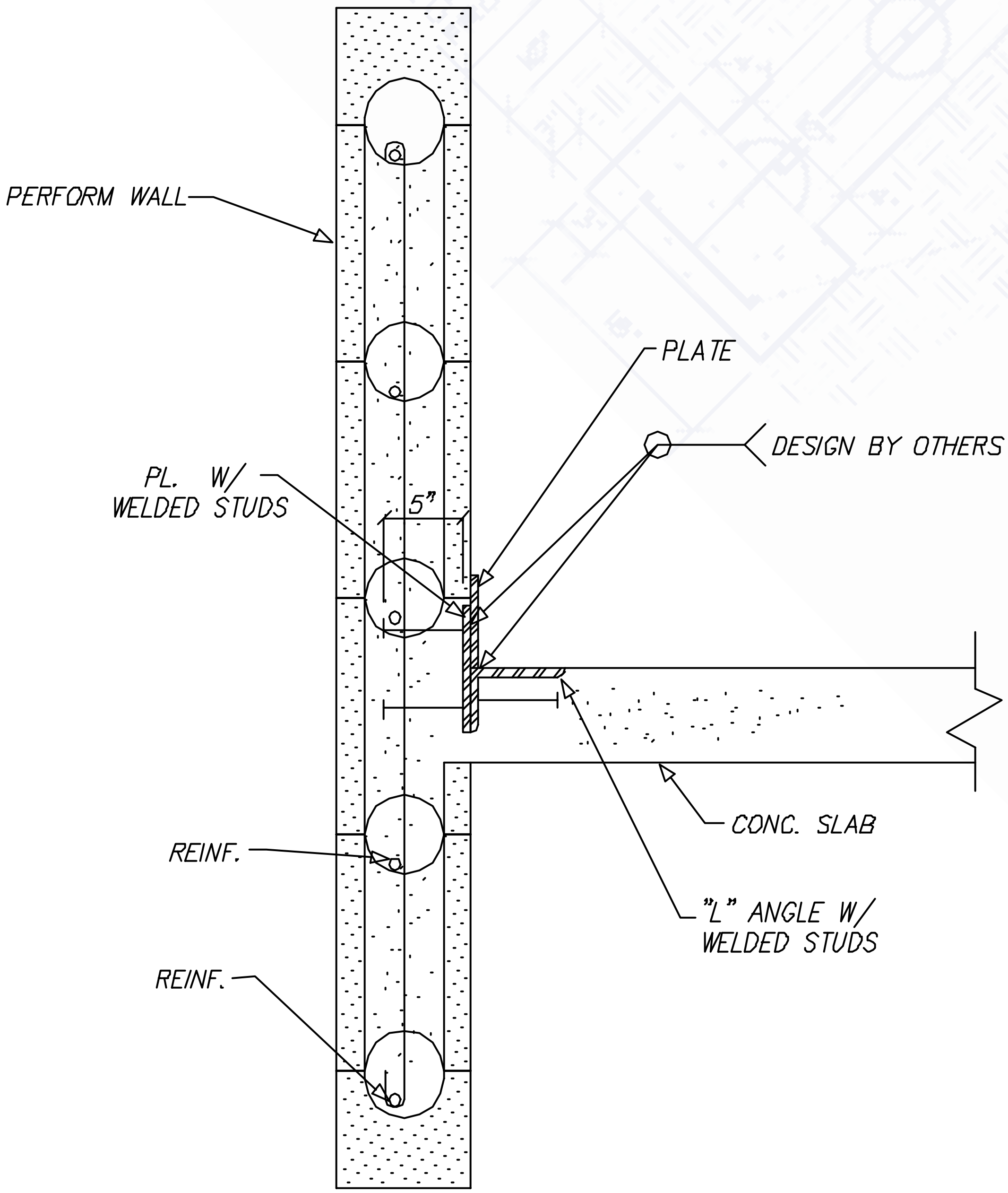
Slab to Spandrell Panel Connection



PERFORM WALL PANEL SYSTEM

Detail No. 41

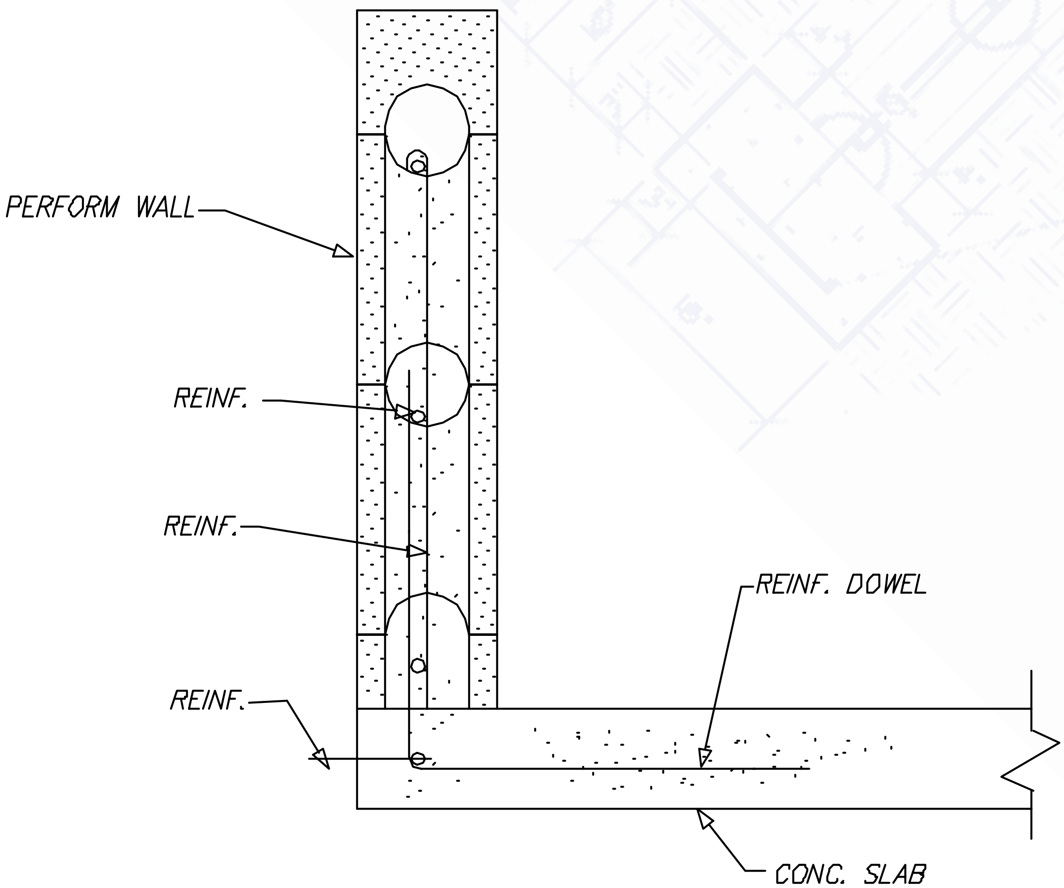
Slab to Spandrell Panel Connection



PERFORM WALL PANEL SYSTEM

Detail No. 42

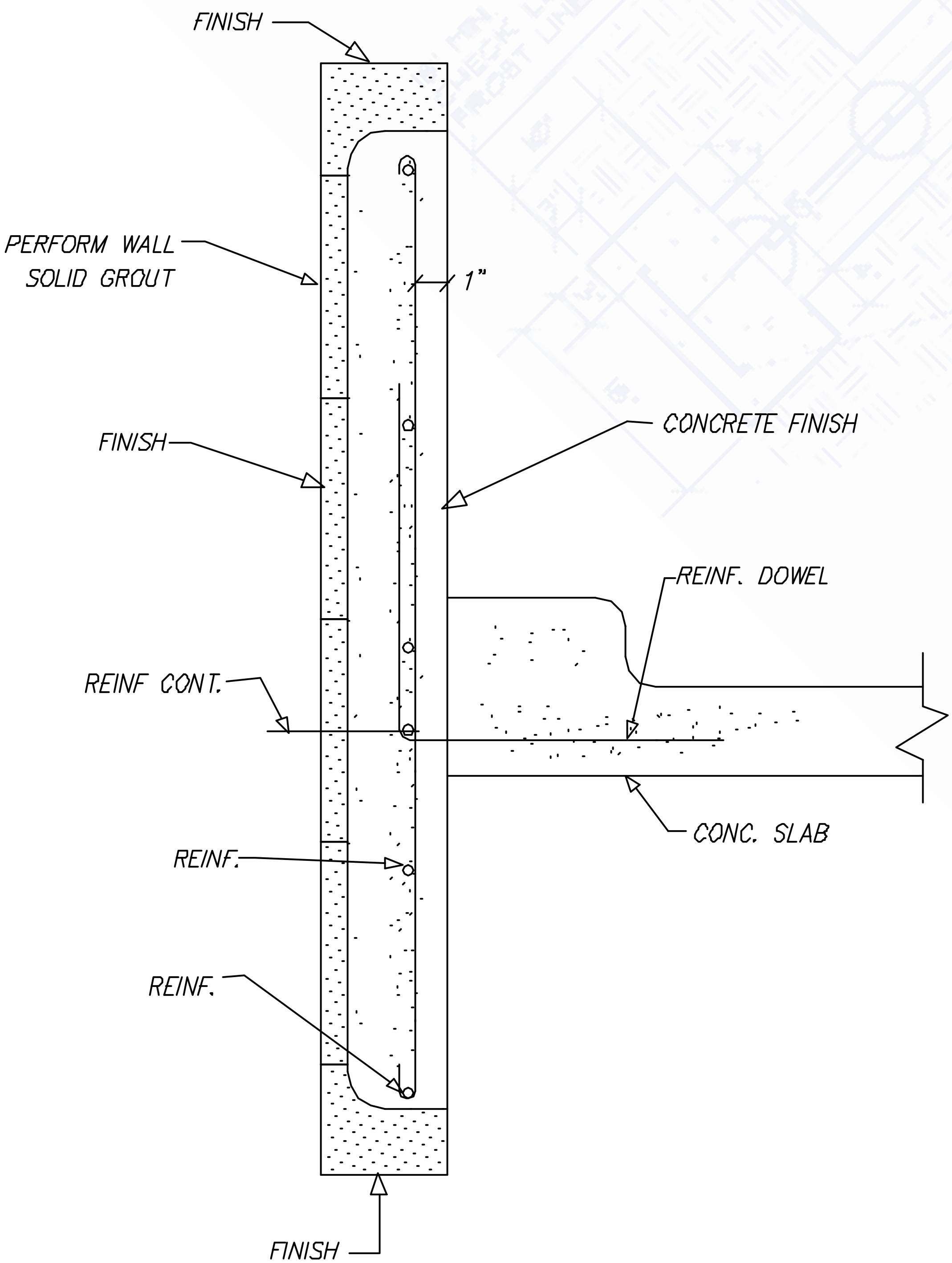
Slab to Spandrell Panel Connection



PERFORM WALL PANEL SYSTEM

Detail No. 43

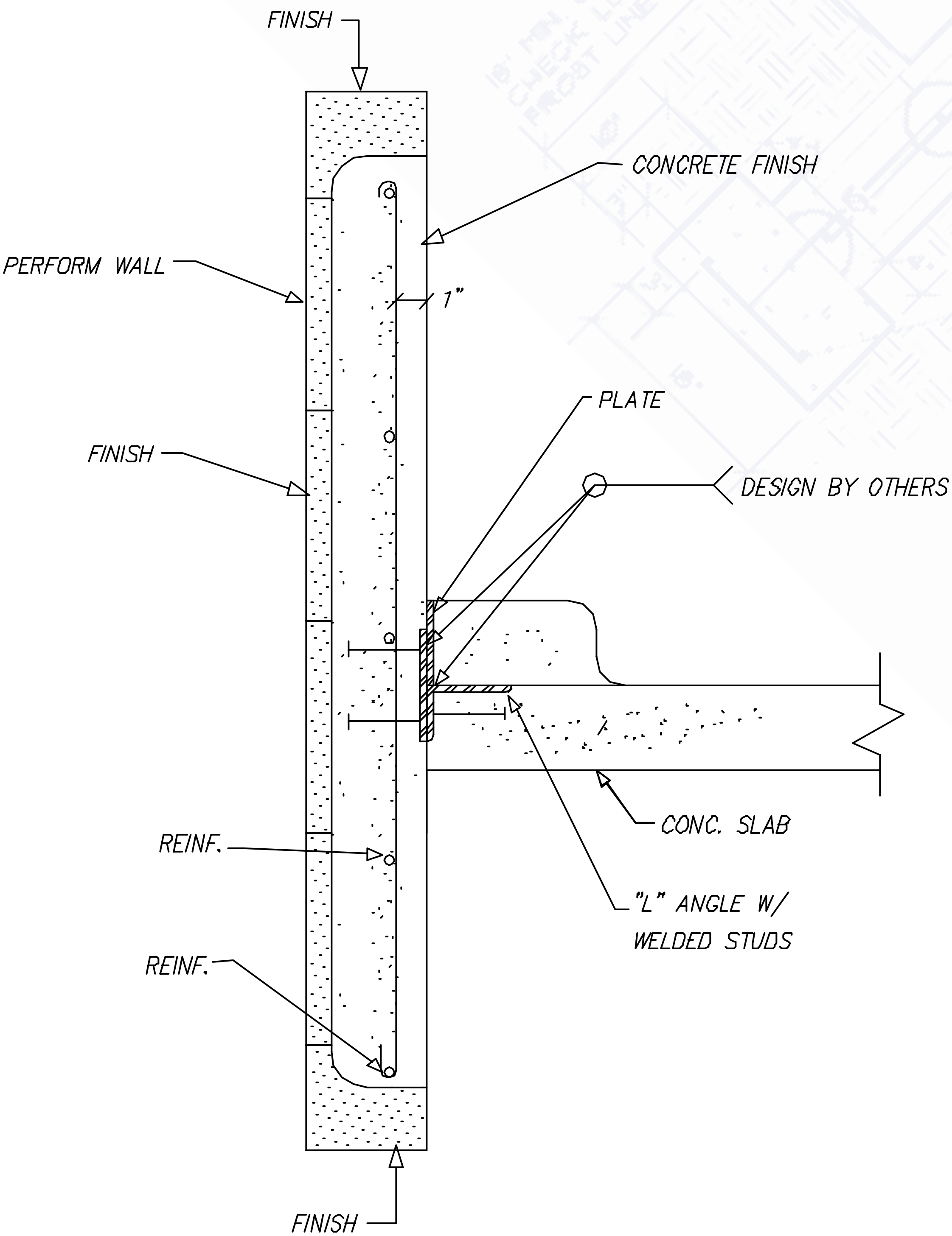
Slab to Spandrell Panel Connection



PERFORM WALL PANEL SYSTEM

Detail No. 44

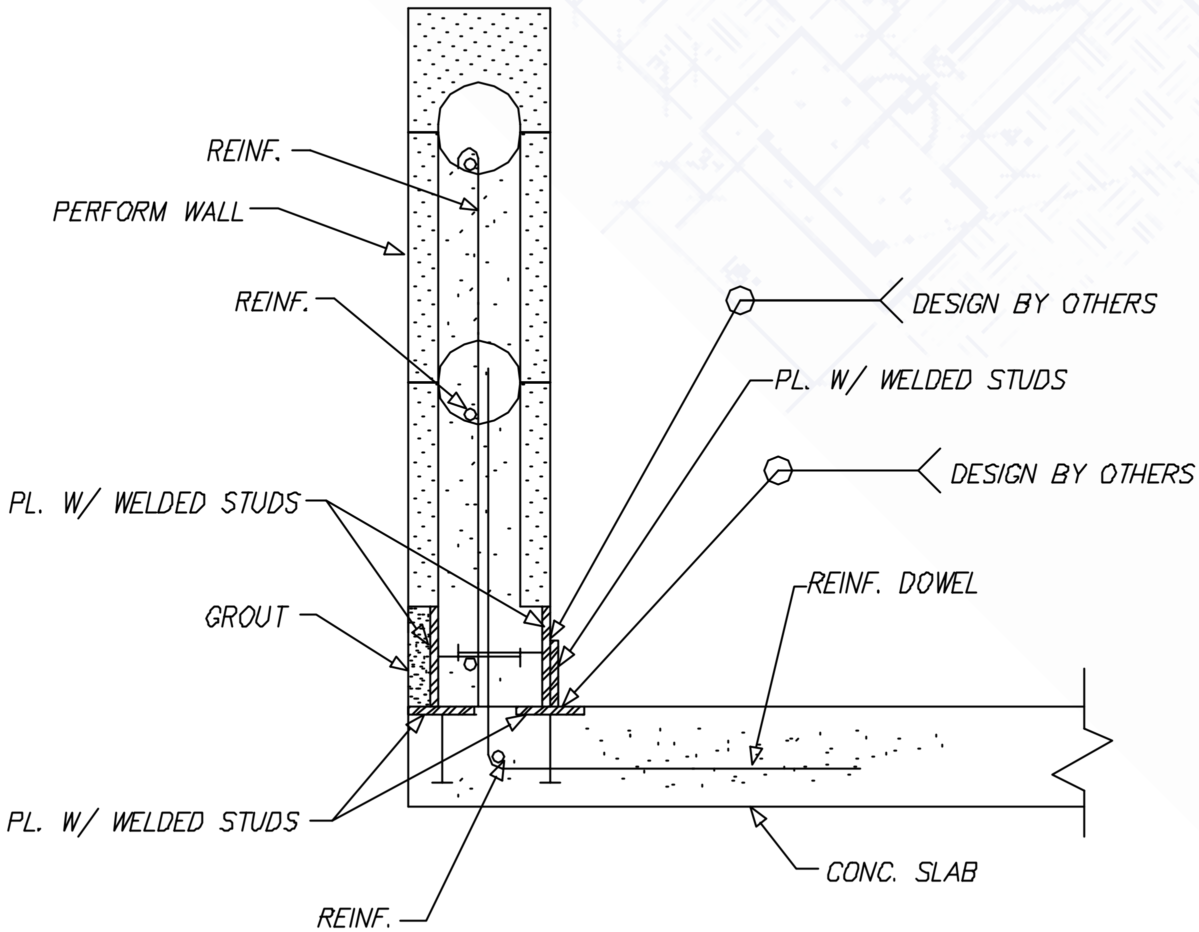
Slab to Spandrell Panel Connection



PERFORM WALL PANEL SYSTEM

Detail No. 45

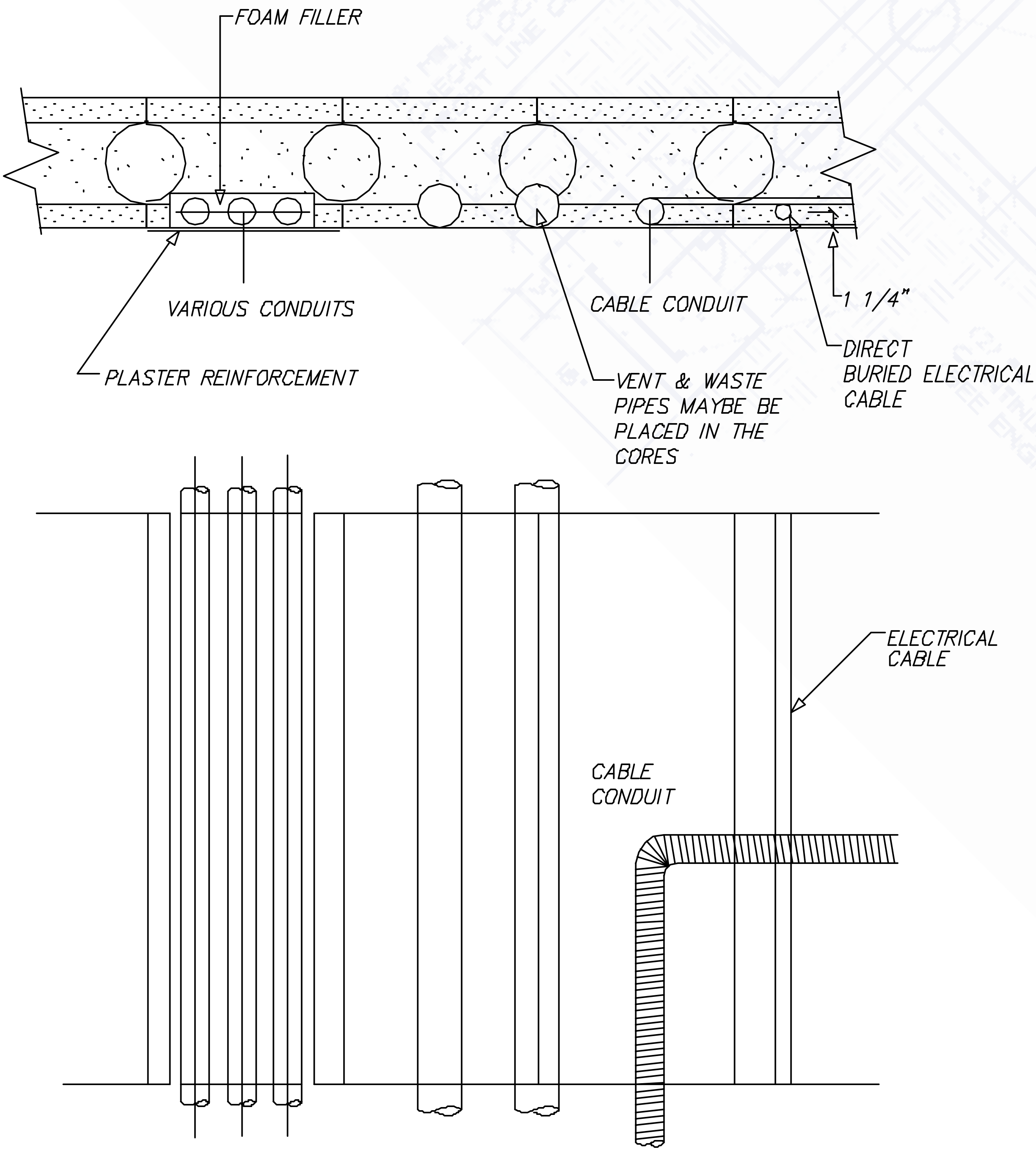
Slab to Spandrell Panel Connection



PERFORM WALL PANEL SYSTEM

Detail No. 46

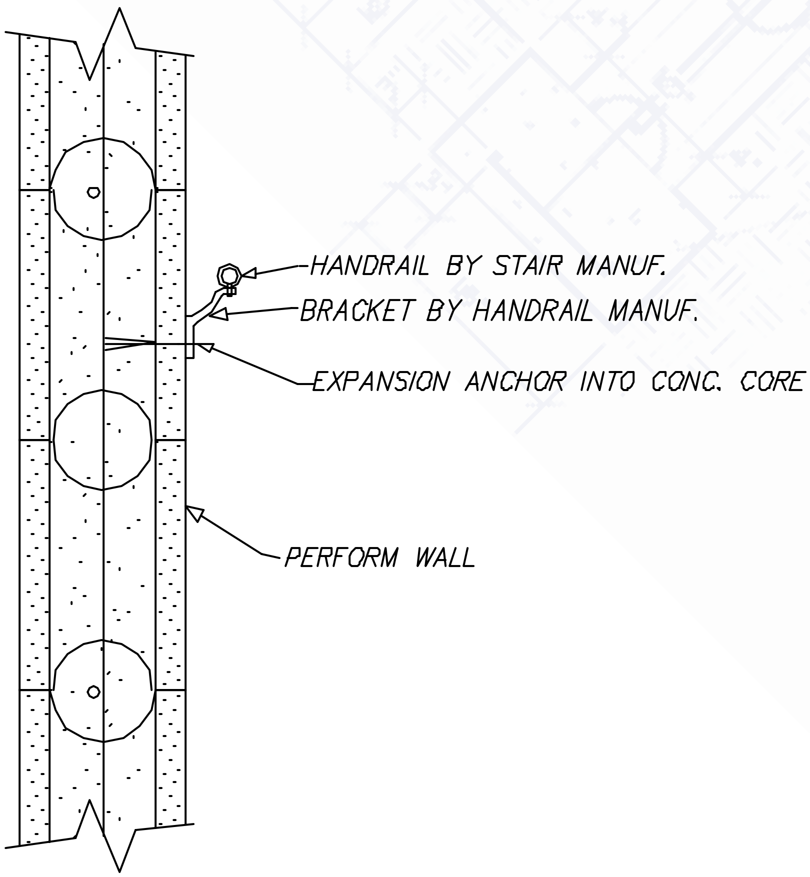
Typical Conduit Embed



PERFORM WALL PANEL SYSTEM

Detail No. 47

Typical Rail Connection



PERFORM WALL PANEL SYSTEM