A SunCam online continuing education course

Cast, Lift, and Release
Tilt-Up Concrete Walls
Part 2: Design

by

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Course Outline:
Introduction
Tilt-Up Concrete
Building Code Design
References
Examination

Fig. 1 (from Part 1): Progress photo of a project with tilt-up concrete walls

Introduction

Focus of course:
Design of slender exterior concrete walls subject to gravity loads from floors and roofs combined with out-of-plane lateral loads due to wind and earthquakes.

Ideally, readers should be familiar with statics and mechanics of materials, as well as concrete design.

This is the second of two courses on tilt-up wall panels. The first course, ***Cast, Lift, and Release, Tilt-Up Concrete Walls, Part 1: Construction***, addresses the background and construction phase information about tilt-up concrete walls.

Fig. 2 (from Part 1): Erected tilt-up panels placed side-by-side

Fig. 4 (from Part 1): Panels in various stages, a) Finishing of poured panels, b) Panel ready for pour

Tilt-Up Concrete Wall Design

General Information

There are two types of design that will be done for tilt-up wall panels. They are usually completed separately by two different parties:

1. Building code, Service life:
This tilt-up concrete wall design is strictly for the purposes of the service life of the building to meet the building code in the building's completed condition. **The building design team's structural engineer of record generally performs this role**.

a) Project-specific engineering analysis of walls is done, and drawings and details are produced.
b) A tilt-up wall panel specification lays out performance and prescriptive criteria, as well as acceptable products and methods to be used during construction. The specifications also provide guidance on the engineering analysis for the construction phase.
c) A note such as the following may appear on the drawings:

> 18. CONTRACTOR SHALL SUBMIT DETAILS AND CALCULATIONS FOR PANEL PICK UP POINTS, STRONGBACKS, AND BRACING (CERTIFIED BY A PROFESSIONAL ENGINEER LICENSED IN THE STATE OF TEXAS) TO ENGINEER ACCORDING TO SPECIFICATIONS FOR APPROVAL PRIOR TO POURING PANELS

2. Construction phase:
There is a separate design for lifting, placing, and bracing of the panel during construction that will need to be done and certified by the panel supplier, or someone representing them. This keeps the responsibilty of panels during construction with the contractor, where the expertise in panel erection exists. The wall panel supplier has the freedom to pursue their own method of erection procedures that they are most familiar with.

The considerations and guidelines include:
a) Specific loading from the chosen lifting and bracing methods used
b) Design for various angles from horizontal that the panels will undergo while being subjected to self-weight of the panels while being erected
c) Openings and strongbacks

d) Wind loading from ASCE 37 in the braced position

<u>**NOTE:**</u> **This design for the construction phase may control over the design for the completed condition.** Panel suppliers will add reinforcing in the panels or provide supplemental bracing or strongbacks as necessary. Unless other arrangements are made, the expectation should be that this additional strengthening will not be shown in the Construction Documents issued by the building design team.

Self-weight. **Because design panel bending moments occur at midheight, the axial load due to self-weight at midheight is used.**

Openings will change the effective section available for moment and axial resistance, and engineering judgement will need to be used in determining how to apply loads to the critical section. See Figs. 57, 58, 59a & 59b. For single openings with a maximum dimension of two feet or less, the designer may choose to ignore their impact on wall design due to the ability of the concrete to redistribute loading around small discontinuities. **Generally, though, reinforcing will be added at opening jambs, where loads become concentrated.**

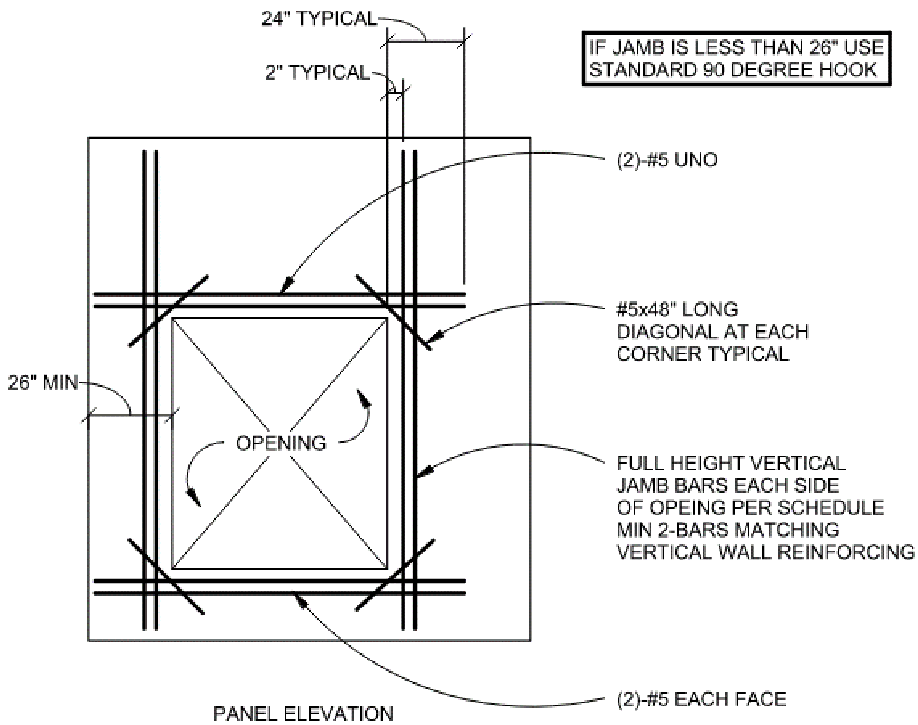


Fig. 57: Elevation view of panel opening with rough schematic of reinforcing

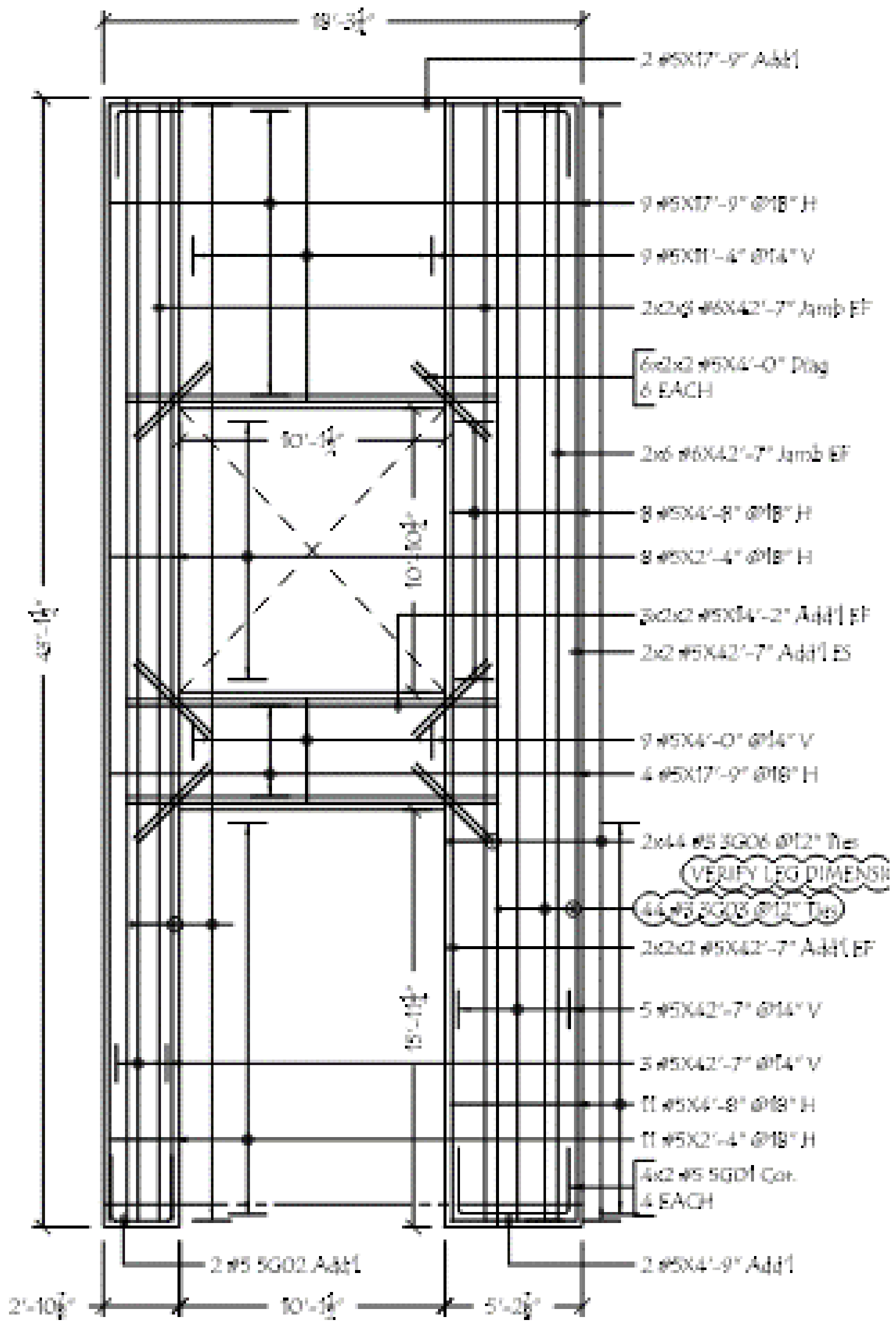


Fig. 58: Typical panel reinforcing in a panel with openings; Elevation view

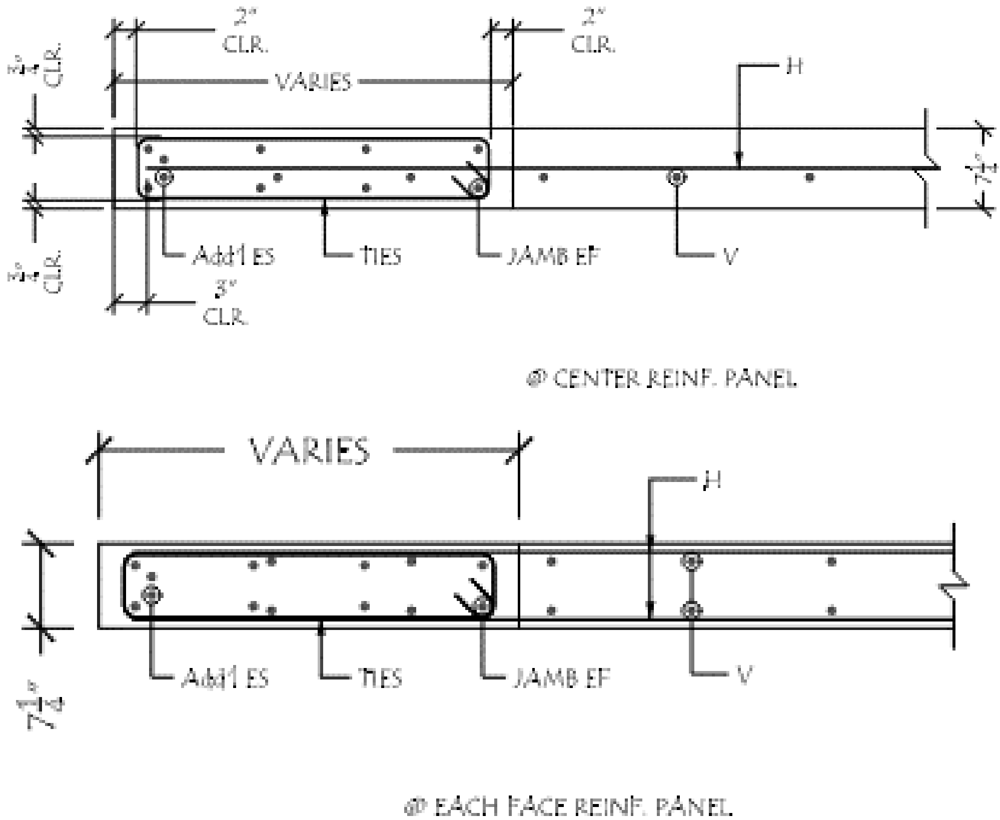


Fig. 59a & 59b: Typical panel reinforcing in a panel with openings; Section views

Panels with wide openings should be designed to span horizontally and vertically to drive loads to critical sections at the jambs. Fig. 60 shows a wide panel opening with added horizontal reinforcing. Or, as in Fig. 61, If the panel opening below is too wide, panels can be supported by adjacent panels.

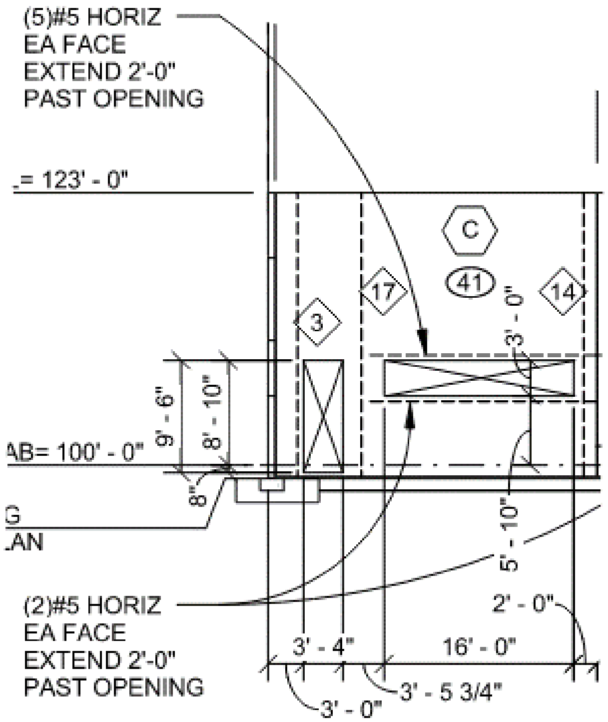


Fig. 60, Wide panel opening with added horizontal reinforcing.

Fig. 61, Panel opening too wide below. Panel supported by adjacent panels.

Integral pilasters can be used for wide openings or at heavy concentrated loads. It is recommended that a minimum dimension of 3-1/2" of added thickness be used for ease of construction, or other dimensions that are convenient for the contractor. The increased bending stiffness of the pilaster relative to the adjacent wall area will tend to invite a higher portion of the loading to the pilaster. See Figs. 62a&b and 63a&b for pilasters.

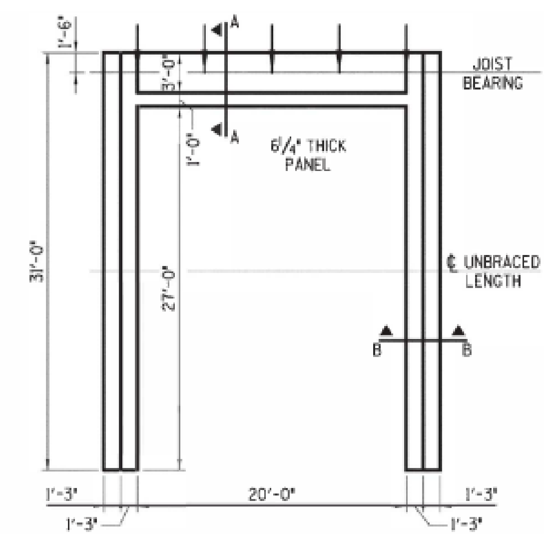


Fig. B.9.1a—Tilt-up panel with stiffening pilaster and header.

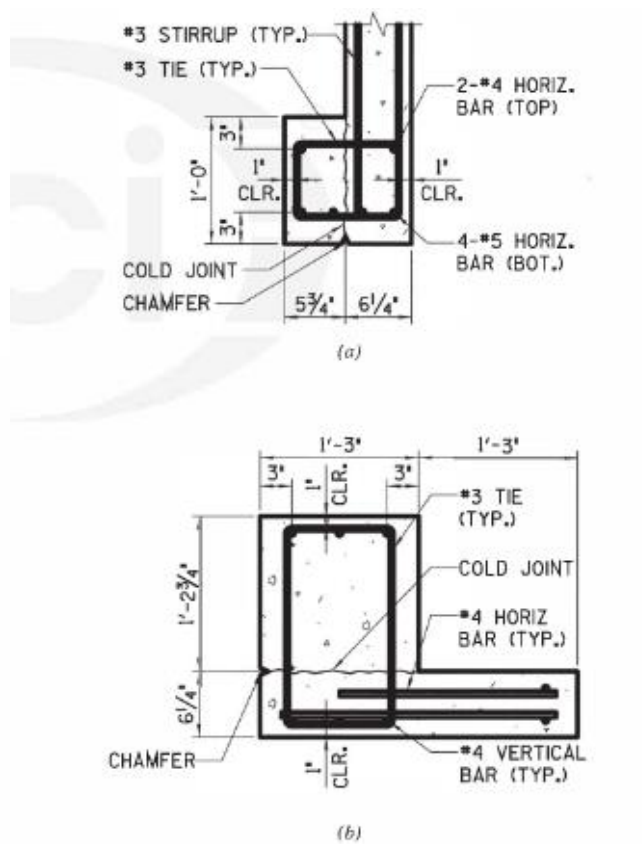
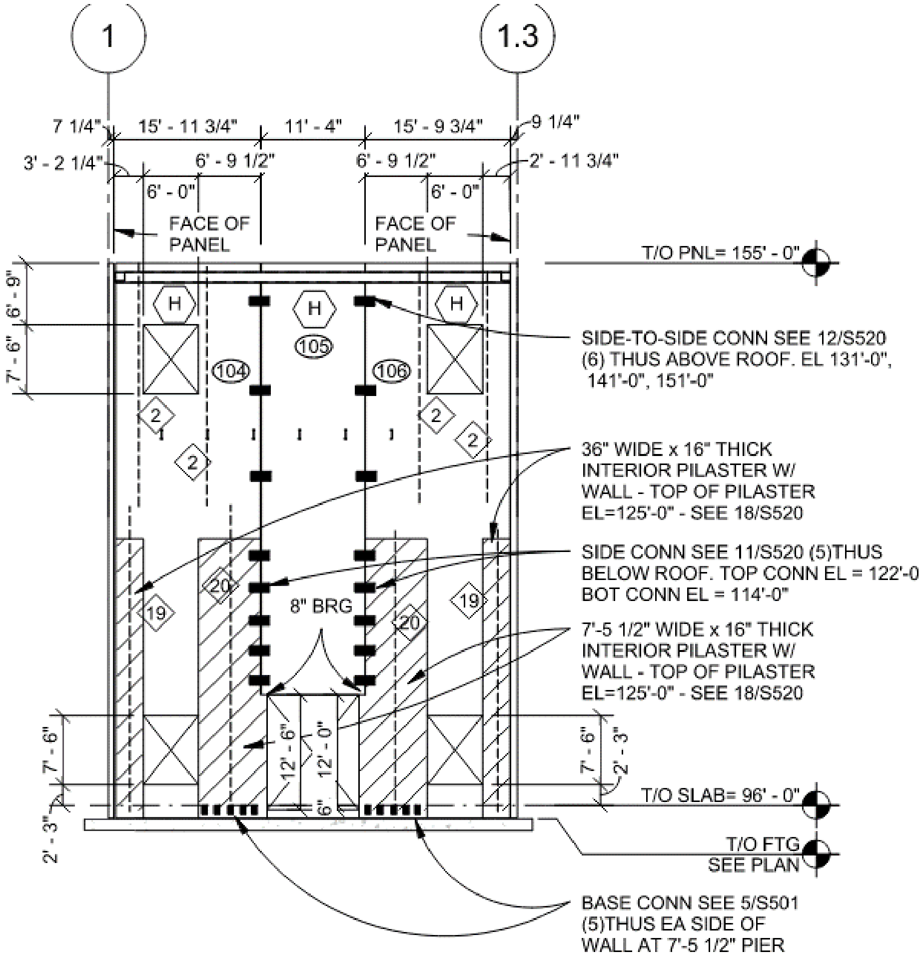
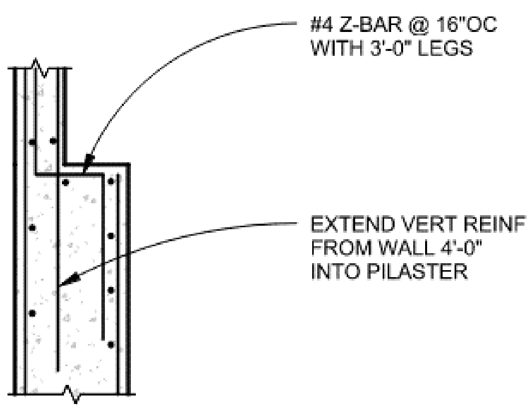


Fig. B.9.1b—Tilt-up panel: (a) Section A-A through header; and (b) Section B-B through pilaster.

Figs. 62a & 62b: Panel with a large opening and integral pilasters and header beam





Figs. 63a & 63b: Panels with pilasters done in a single-pour

Loads at top of wall are often applied at an eccentricity to the panel centerline, meaning a moment would need to be applied there. See a typical detail in Fig. 64. A minimum eccentricity should be used for design even if loads are intended to be concentric. Eccentricity at the bottom of the panel is generally assumed to be zero. See Figs. 65a, 65b, and 66 for options.

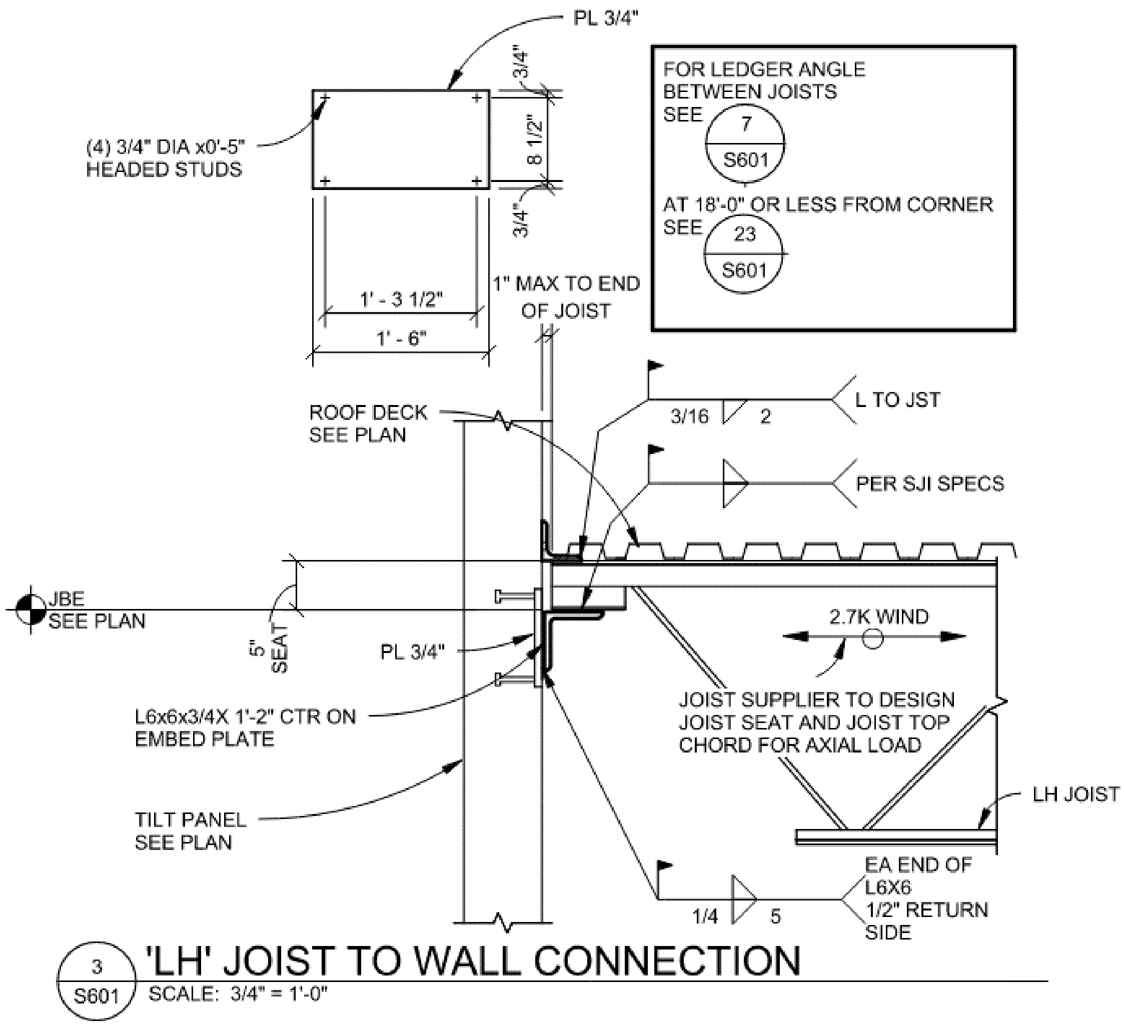
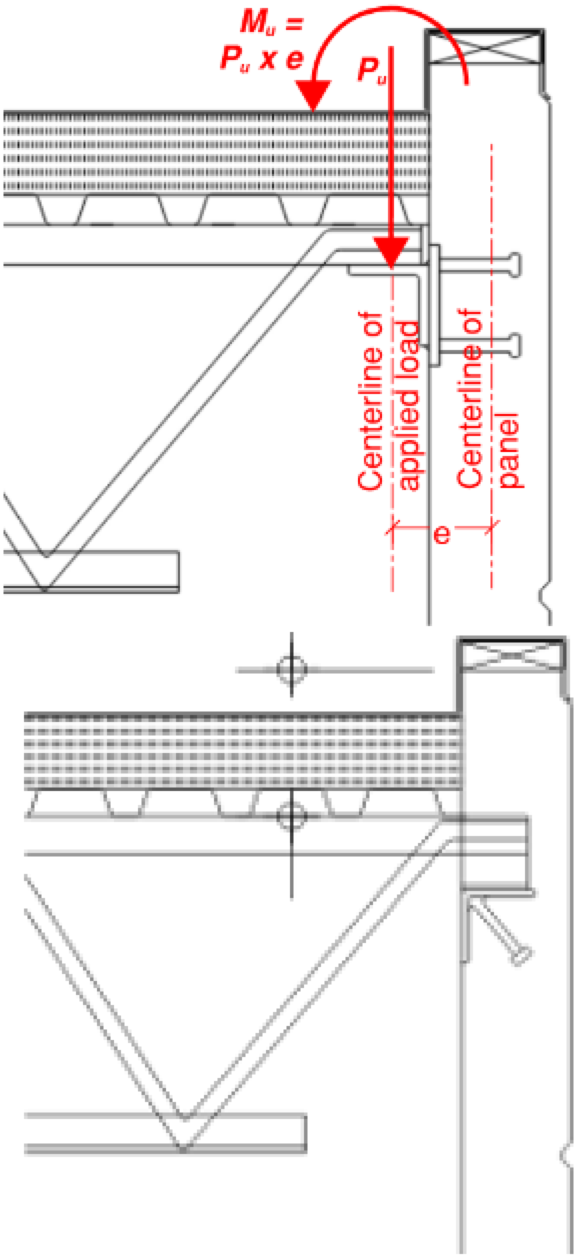


Fig. 64: Typical joist bearing detail



Figs. 65a & 65b:
Top: Angle attached to embed at face of panel
Bottom: Optional pocket in panel to reduce eccentricity

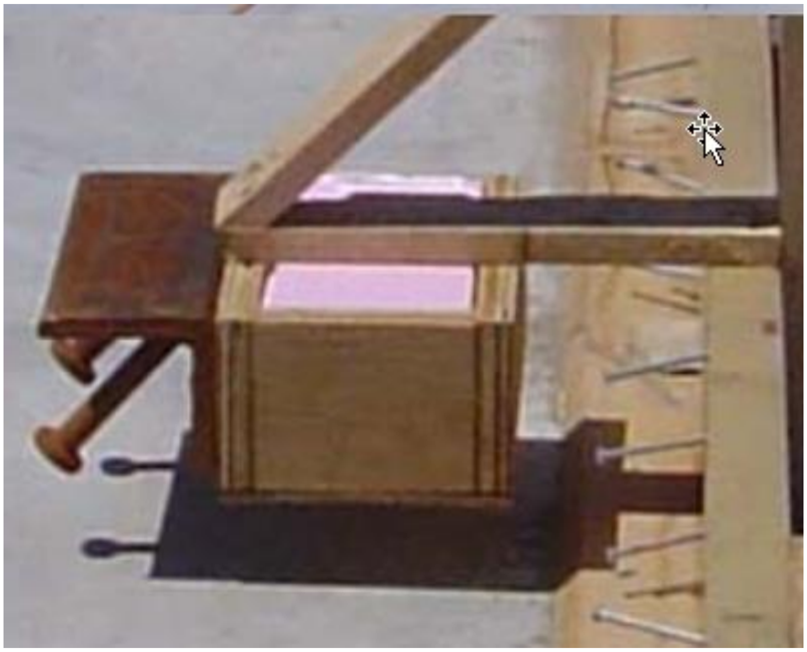


Fig. 66: Forming and angle embed for pocketed bearing prior to pour

Assuming panels will be restrained near bottom corner anchors, use developed reinforcing at the bottom corners of the wall panels to minimize cracking due to shrinkage and temperature movements towards the center of mass. This means bars that are hooked at the corners that lap sufficiently with straight bars. (Fig. 67).

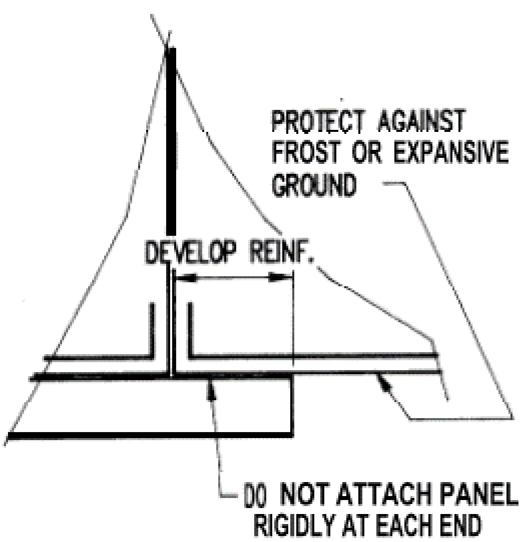


Fig. 67: Corner bars at bottoms of panels (ACI 551)

No construction is perfect, nor does it need to be. **It is recommended that panels follow ACI 117 for construction tolerances. A joint between panels and a gap below panels are used for leveling and shimming to get sufficient alignment**.

The presence of in-plane shear if tilt-up panels are used as shear walls may trigger the need for additional design rules of panels per ACI 318 Chapter 11. For simplicity, those differences are not addressed in this course. Mainly, those additional provisions require greater minimum reinforcing to ensure ductility in the resistance of shear forces (upon diagonal cracking).

There are other items that will require engineeing judgement. A couple of these are:
a. In northern climates, air entrainment may be required.
b**. Some wall panel suppliers prefer to wet-set embed plates into wet concrete during concrete pours**. The most effective method of setting embed plates at the right location, in the proper orientation, where the align with the face of the panel, and so they don't move during concrete placement, is to fasten them into place prior to the pour. Imagine all of the moving parts on pour day, and how those play out with quality control. The plates can't really be inspected for compliance prior to the pour if they are loose on site, and once they're wet set, there's no way to know if they're the right embed plates. Vibrating the concrete around the embed plate wiill be a challenge either way. ACI leaves it to the engineer of record to decide if wet-setting is acceptable.

Tilt-Up Concrete Wall Design per ACI 318

Tilt-up concrete walls are designed per the governing building code and ACI 318. When ACI 318 is mentioned, this refers to the 318-14 version or later, after the chapters were rearranged to get concrete components in their own chapters.

Though local code modifications will govern over it, the International Building Code (IBC) is the base defining body for concrete design, as follows:

Loading:
IBC Chapter 16, with ASCE 7 loading provisions by reference in IBC Chapter 35
These correspond with ACI 318 Chapter 5 load combinations.

Load cases for tilt-up wall panels from ACI 318:

Table 5.3.1—Load combinations

Load combination	Equation	Primary load
$U = 1.4D$	(5.3.1a)	D
$U = 1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$	(5.3.1b)	L
$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.5W)$	(5.3.1c)	L_r or S or R
$U = 1.2D + 1.0W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R)$	(5.3.1d)	W
$U = 1.2D + 1.0E + 1.0L + 0.2S$	(5.3.1e)	E
$U = 0.9D + 1.0W$	(5.3.1f)	W
$U = 0.9D + 1.0E$	(5.3.1g)	E

D = Dead load
L = Live load
Lr = Roof live load
S = Snow load
R = Rain load
W = Wind load
E = Seismic load

Fig. 68: ACI Load combinations

Again, under IBC, ASCE 7 would be the primary reference for the loads. Panel loading in general is shown in Fig. 69, with axial load and moments arising from gravity loads and lateral loading. Due to the slender nature of the wall panels, deflections need to be considered, especially when considering ultimate (failure) loads.

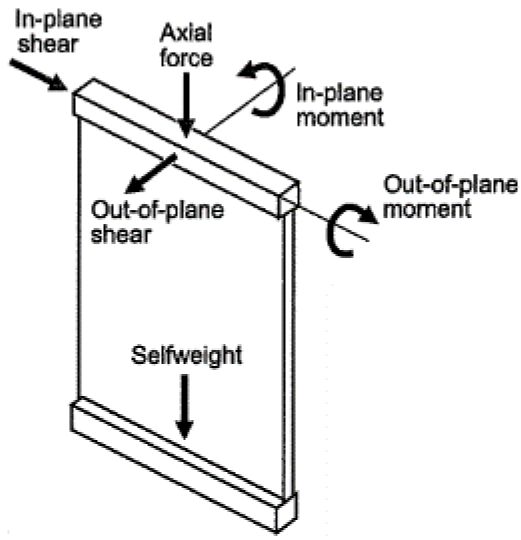


Fig. R11.4.1.3—In-plane and out-of-plane forces.

Fig. 69; ACI 318, Forces on walls

As mentioned above, loads from the structure may be appied off-center on the panel, leading to moments at the top of the panel. These are half their value at midheight. In addition, **P–Δ moments also need to be factored in**. See Fig. 70, 71 & 72.

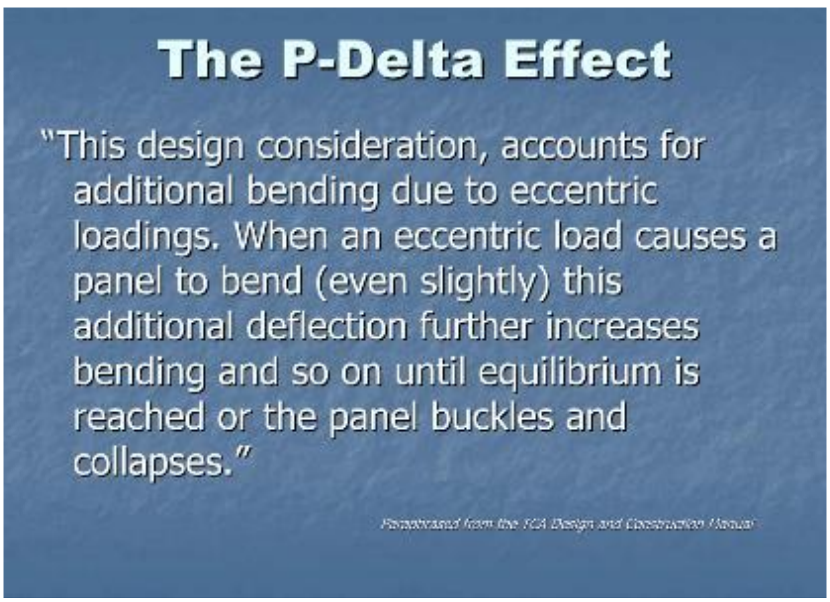


Fig. 70 (Smith)

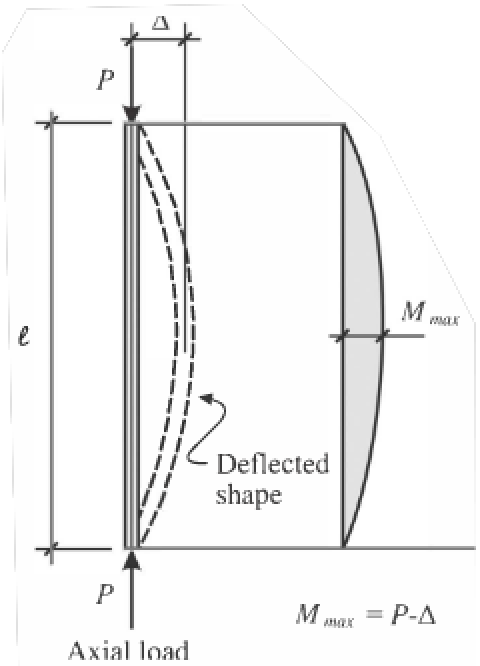


Fig. 71: P-Δ as shown in ACI 551.2

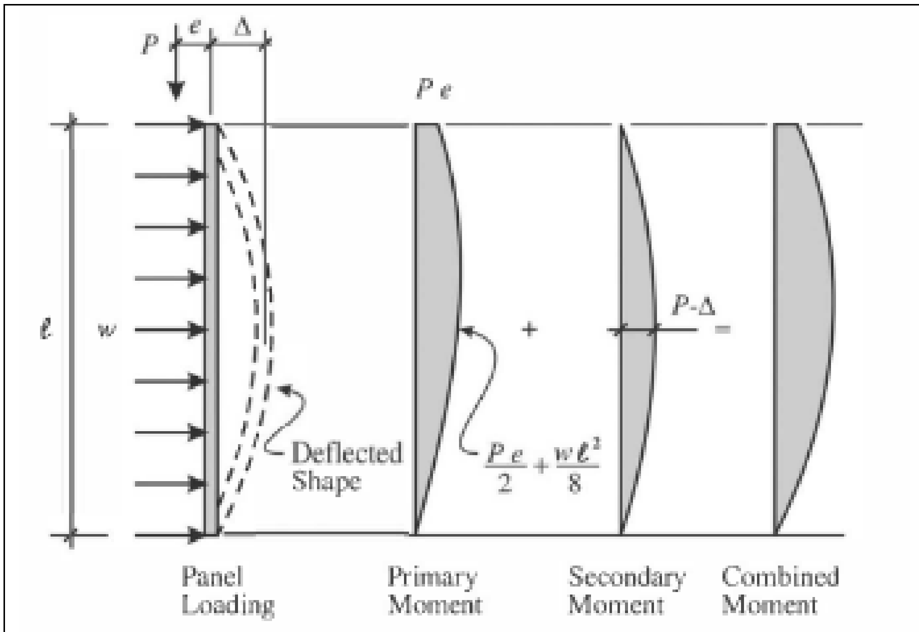


Fig. 3.1—Panel design model—suction force acting with eccentric axial load.

Fig. 72: Moments on a panel from ACI 551.2

Strength and servicability provisions:

Use IBC Chapter 19, with ACI 318 concrete design by reference in IBC Chapter 35. Specifically, use ACI 318, Chapter 11 – Walls

ACI 318 Chapter 11 is focused primarily on the more comprehensive collection of concrete walls. **Section 11.8 addresses "slender walls",** which is a special case of concrete walls that follow certain narrow guidelines. Tilt-up walls fit in this category of "slender walls". Research has been done to support a different set of provisions for walls that meet the requirements below. Note that if the requirements cannot be met, the general ACI 318 provisions would apply, with all of the additional constraints.

> "11.8.1.1 It shall be permitted to analyze out-of-plane slenderness effects in accordance with this section for walls satisfying (a) through (e):
> "(a) Cross section is constant over the height of the wall
> (b) **Wall is tension-controlled for out-of-plane moment effect**
> (c) ϕM_n is at least M_{cr} where M_{cr} is calculated using f_r as provided in 19.2.3
> (d) **P_u at the midheight section does not exceed $0.06f'_c A_g$**
> (e) Calculated out-of-plane deflection due to service loads, Δ_s, including **PΔ** effects, does not exceed $\ell_c/150$"

Where the above are defined as below, for the effective length of wall in question, for ACI 318 code section noted (2014 and later), and standard U.S. units (k=kip or 1,000#):
M_n = bending strength of the concrete wall (out-of-plane), 22.2 (in-k, ft-k)
ϕ =Strength reduction factor for concrete, tension-controlled element, 21.2.1 = 0.9
M_{cr} = the cracking moment where $M_{cr} = Sf_r$, 19.2.3 (in-k, ft-k)
S = Uncracked section modulus of wall (out-of-plane), = $1/6bt^2$ (in^3)
h = thickness of wall panel
b = width of wall (in-plane) (in)
$f_r = 7.5\,\lambda\sqrt{f'c}$, 19.2.3 (psi)
λ = 1 for normal weight concrete. See 19.2.4.2 for lightweight concrete.
f'_c = compressive strength of concrete (psi); units for $\sqrt{f'c}$ remain psi
P_u = Factored axial force in wall the midheight section (k)
A_g = Gross area of concrete, (in^2)

Δ_s = Out-of-plane deflection due to service loads (in)
l_c = clear span of wall between supports (in, ft)

The ACI commentary reads:

> "R11.8.1.1 This procedure is presented as an alternative to the requirements of 11.5.2.1 for the out-of-plane design of slender wall panels, where the panels are restrained against rotation at the top.
> "Panels that have windows or other large openings are not considered to have constant cross section over the height of the panel. Such walls are to be designed taking into account the effects of openings.
> "Many aspects of the design of tilt-up walls and buildings are discussed in ACI 551,2R and Carter et al. (1993)."

The compressive strength of concrete, f'_c, is not the most critical parameter in wall panel design. Because the wall design is controlled by tension, and the wall is slender, the compression block is relatively small. So, lower values of **f'_c such as 3,000 psi** may be suitable. However, 4,000 psi is a common value, and it has a much greater impact on resisting concentrated loads at connections to the concrete wall.

In the above, the author treats jambs of openings under 11.8 slender wall provisions as the constant cross section elements to support the load tributary to the openings.

Analysis modeling of wall per ACI 318 has a couple of requirements:

1. **Wall to be designed as simply supported**:

11.8.2.1 The wall shall be analyzed as a simply supported, axially loaded member subject to an out-of-plane uniformly distributed lateral load, with maximum moments and deflections occurring at midheight.

2. **Concentrated load on walls to be treated as follows:**

11.8.2.2 Concentrated gravity loads applied to the wall above any section shall be assumed to be distributed over a width equal to the bearing width, plus a width on each side that increases at a slope of 2 vertical to 1 horizontal, but not extending beyond (a) or (b):

(a) The spacing of the concentrated loads
(b) The edges of the wall panel

Or, in graphic form:

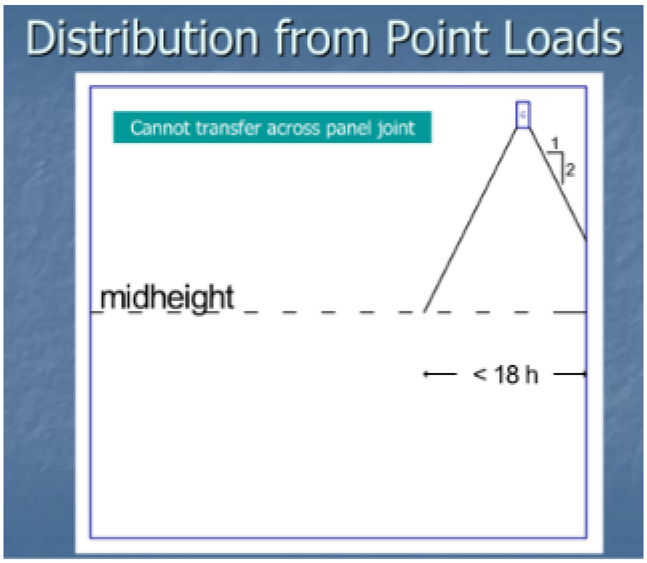


Fig. 73, 1H:2V distribution of concentrated loads (Smith)

ACI 551.2 has the following recommendations (note: not strict provisions) for practical limits on panel slenderness:

Single mat of reinforcement (centered in the panel cross section): ℓ_c / h = 50

Two mats of reinforcment (1 inch clear of each face): ℓ_c / h = 65

where:
h = thickness of wall panel

Design

As mentioned previously, in the normal project workflow, there are two types of tilt-up wall design:
1) Design for lifting, placing, and bracing of the panel during construction that will be done and certified by the panel supplier.
2. Service life of the building to meet the building code in the building's completed condition.

The design below is related to building code design.

Stepping through the 11.8 of ACI 318, the design procedure includes:
Loading on walls, slabs, building geometry, wall openings, finishes, crane limits, contractor preferences, panel joint layout, structural drawings

Structural drawings:
Some excerpts of structural wall panel drawings are shown below.

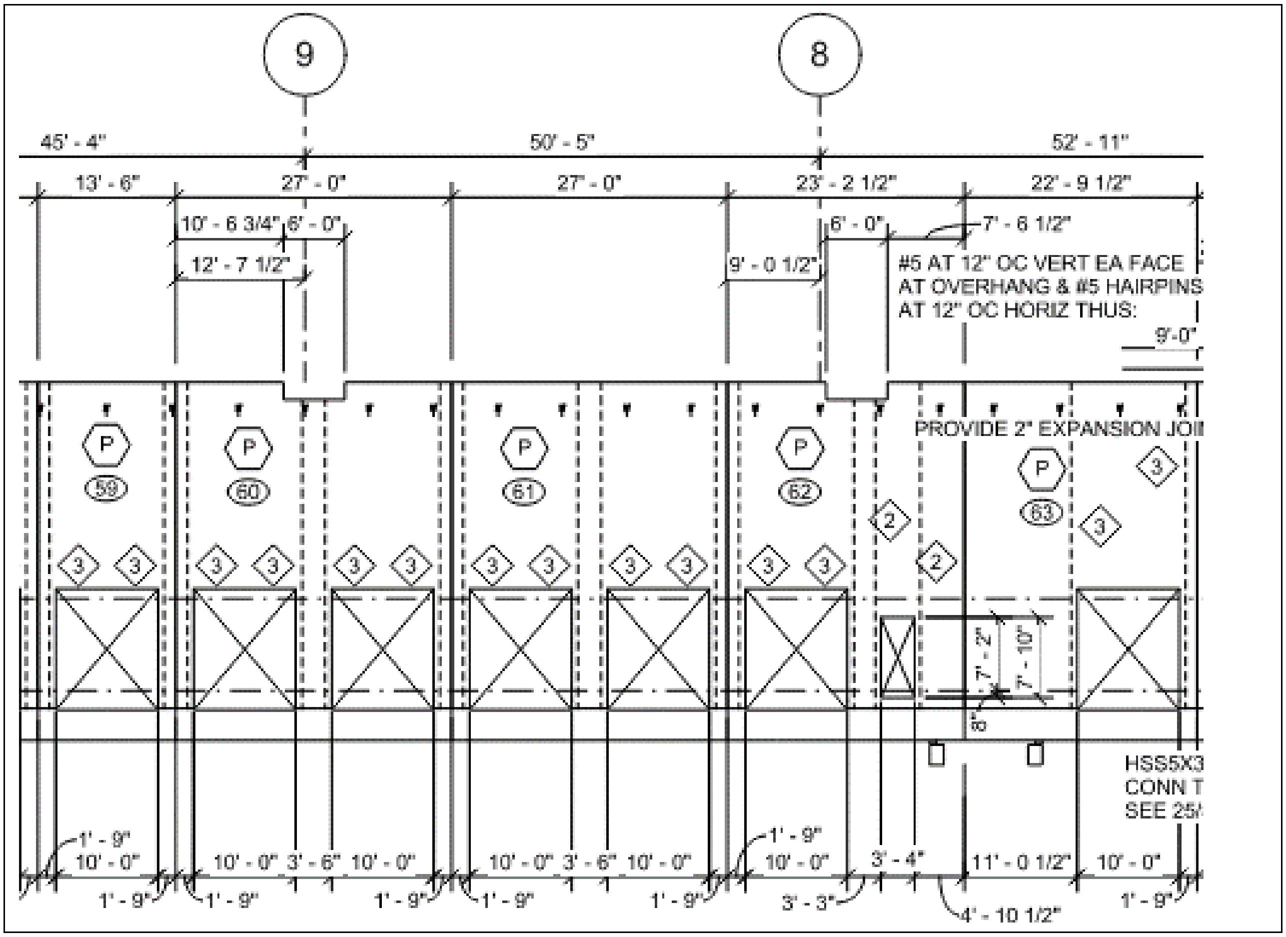


Fig. 74: Elevation of loading dock area from structural drawings; Note dock door openings

PANEL REINFORCING SCHEDULE						
PANEL TYPE	TOTAL PANEL THICKNESS 'T'	HORIZ REINF O.C.	VERT REINF O.C.	HORIZ BOT REINF	TOP & EDGE REINF UNO	COMMENTS
P	0' - 9 1/4"	#4 @ 18"OC	#4 @ 18"OC	(2)- #5	(2)- #5	EACH FACE

NOTES:
1. JAMB REINFORCING MUST EXTEND FROM THE BOTTOM OF THE PANEL TO 2'-0" ABOVE THE ROOF ELEVATION, UNO.
2. JAMB REINFORCING AT ROOF SCUPPERS TO BE HOOKED AT TOP.

Fig. 75: Wall panel reinforcing, with Panel type for elevation shown above

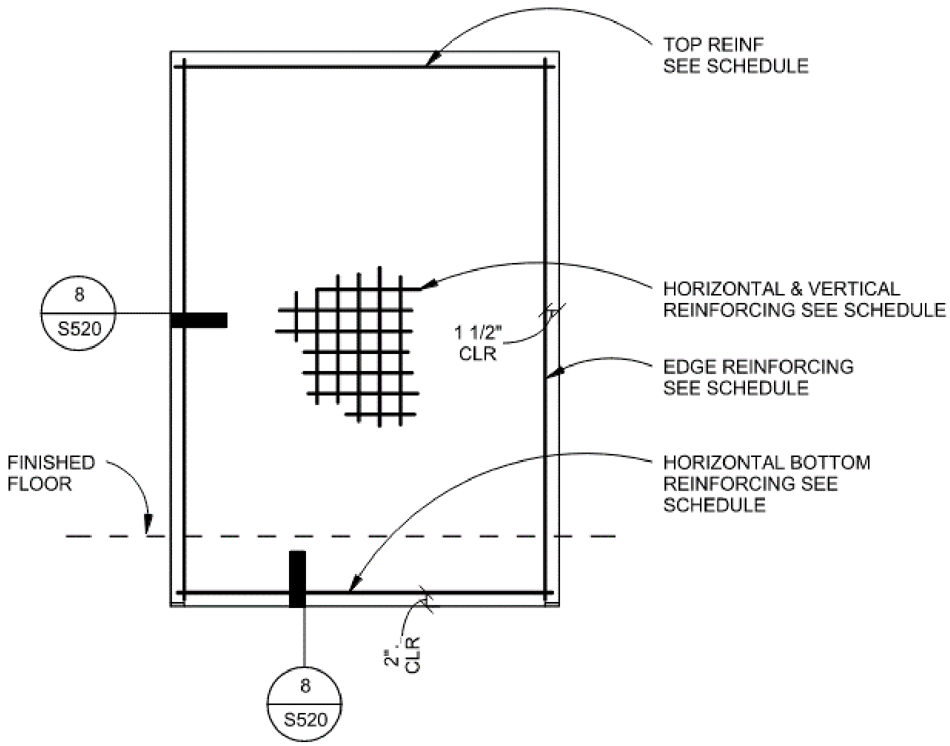


Fig. 76:Detail depicting a typical wall panel with no openings

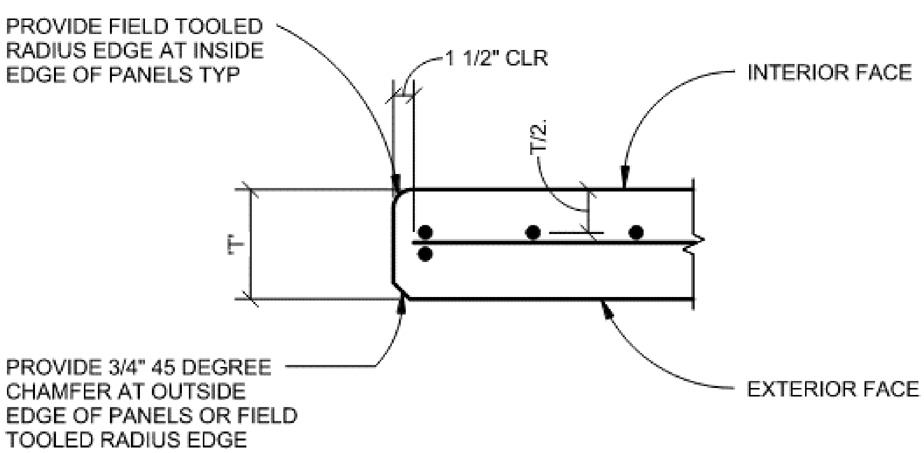


Fig. 77: Detail at edge of panel

JAMB REINFORCING SCHEDULE				
JAMB MARK	# VERT EACH FACE 'N'	SIZE	LENGTH 'L'	DETAIL REF SECTION
2	2	#6	18"	9/S520
3	3	#6	21"	10/S520

Fig. 78: Jamb reinforcing at the side of openings. Refer to elevation in Fig. 75

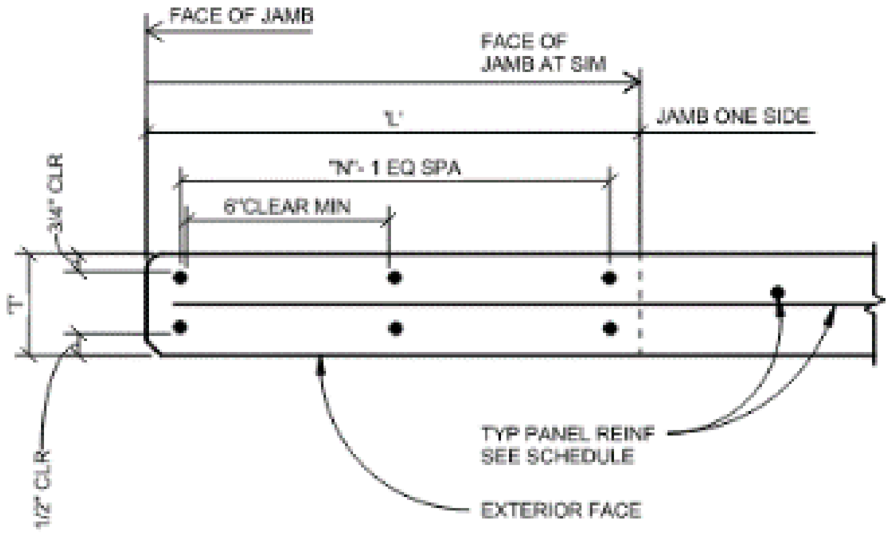


Fig. 79: Section through jamb at openings

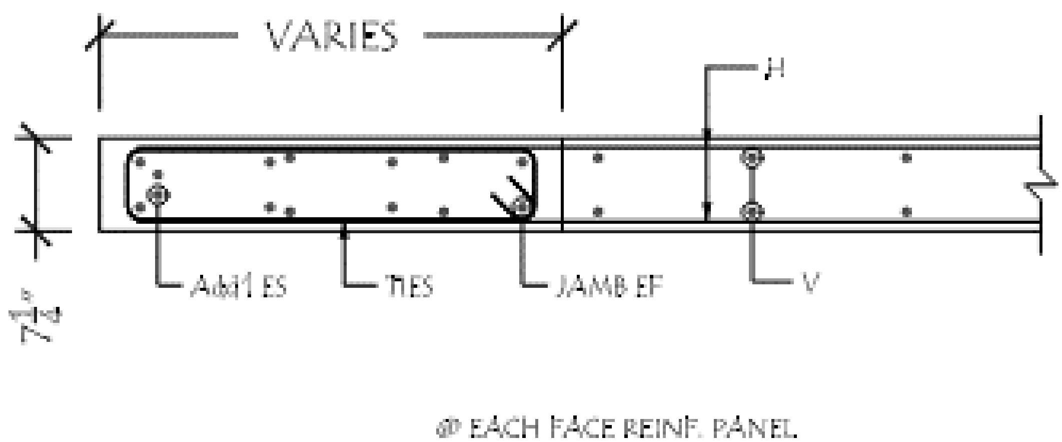


Fig. 80: Section through jamb at openings
<u>Top</u>: No ties
<u>Bottom</u>: Closed Ties with 135 degree hooks for development

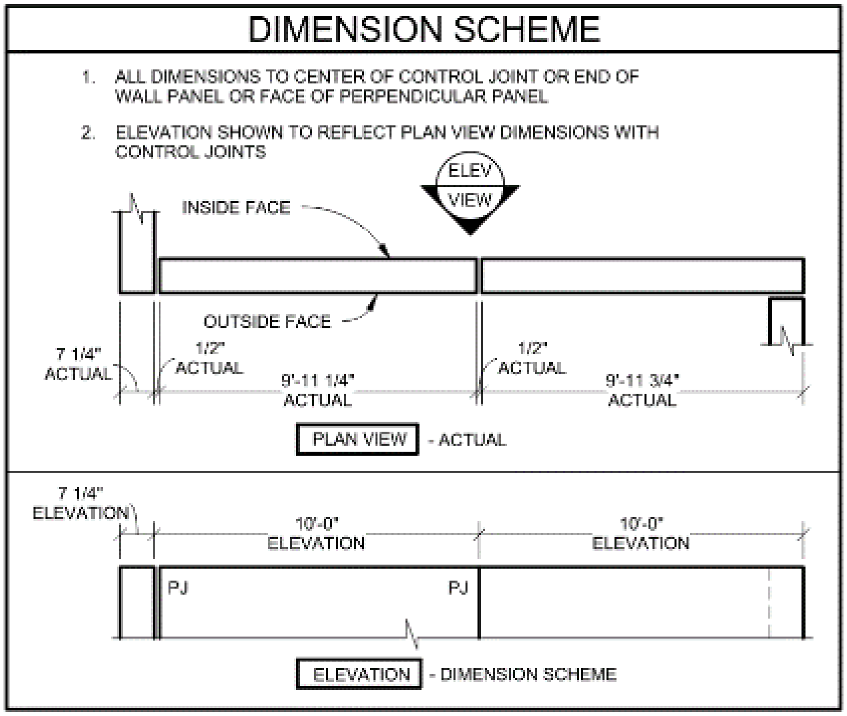


Fig. 81: Nominal vs. actual dimensions

SEQUENCE OF CONSTRUCTION

1. POUR CONCRETE FOUNDATIONS
2. FORM PANELS ON CASTING BEDS AND POUR CONCRETE. NOTIFY SPECIAL INSPECTOR FOR REINFORCING INSPECTION PRIOR TO CASTING PANEL.
3. ERECT PANEL PROVIDING A MINIMUM OF 2 BRACES FROM EACH PANEL TO FLOOR PANEL BRACES SHALL REMAIN IN PLACE UNTIL ROOF STRUCTURE AND ROOF DECK ARE INSTALLED.
4. FIELD WELD ALL PANEL CONNECTIONS.
5. ERECT ROOF STRUCTURE.
6. POUR CONCRETE FLOOR SLAB.

Fig. 82: Assumed sequence (with contractor input)

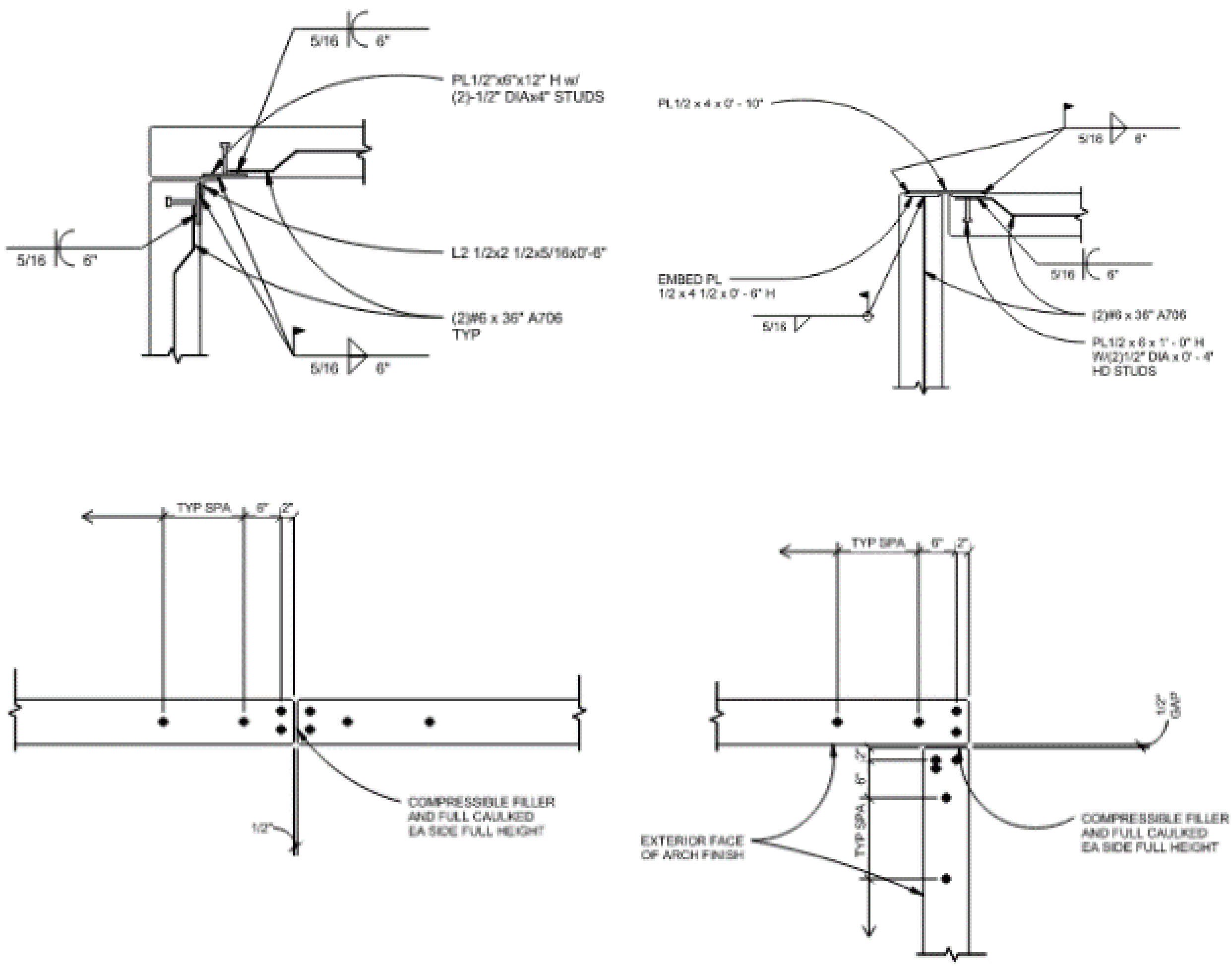


Fig. 83: Typical details

The design building code and structural loading criteria need to be idenfitifed on the structural drawings.

Other types of information on wall panels on structural drawings include:
1) Concrete strength, rebar strength, concrete mix design properties
2) Wind, seismic and gravity load info
3) Roof, floor, foundation info
4) **Wall panel joints between panels, which are normally 1/2" to 3/4"**, connections, reinforcing
5) Shim and grout info at bearing: Normally 1-1/2” gap, 6,000 psi grout, and a note to avoid bearing directly under nearby panel openings

COMPONENT AND CLADDING DESIGN WIND PRESSURE UNO
(1.0W NOT INCLUDING OMEGA FACTOR):

FOR WALLS BELOW ROOFS WITH ELEVATIONS 45 FEET OR LESS ABOVE FINISHED FLOOR:

	TRIBUTARY AREA 20 SQ FT	TRIBUTARY AREA 100 SQ FT
A. TYPICAL PRESSURE	20 PSF	17 PSF
B. TYPICAL SUCTION	-22 PSF	-19 PSF
C. CORNER PRESSURE	20 PSF	17 PSF
D. CORNER SUCTION	-27 PSF	-21 PSF

CORNER VALUES TO BE USED WITHIN 18'-0" OF BUILDING OUTSIDE CORNERS

FOR WALLS BELOW ROOFS WITH ELEVATIONS GREATER THAN 45 FEET ABOVE FINISHED FLOOR:

	TRIBUTARY AREA 20 SQ FT	TRIBUTARY AREA 100 SQ FT
A. TYPICAL PRESSURE	24 PSF	21 PSF
B. TYPICAL SUCTION	-24 PSF	-22 PSF
C. CORNER PRESSURE	24 PSF	21 PSF
D. CORNER SUCTION	-44 PSF	-35 PSF

CORNER VALUES TO BE USED WITHIN 35'-0" OF BUILDING OUTSIDE CORNERS

Fig. 84: Sample of wind loads that would appear on a structural drawing

Basis for design per ACI 318, 11.8

See Fig. 85 for the widely accepted equivalent stress block for a concrete section.

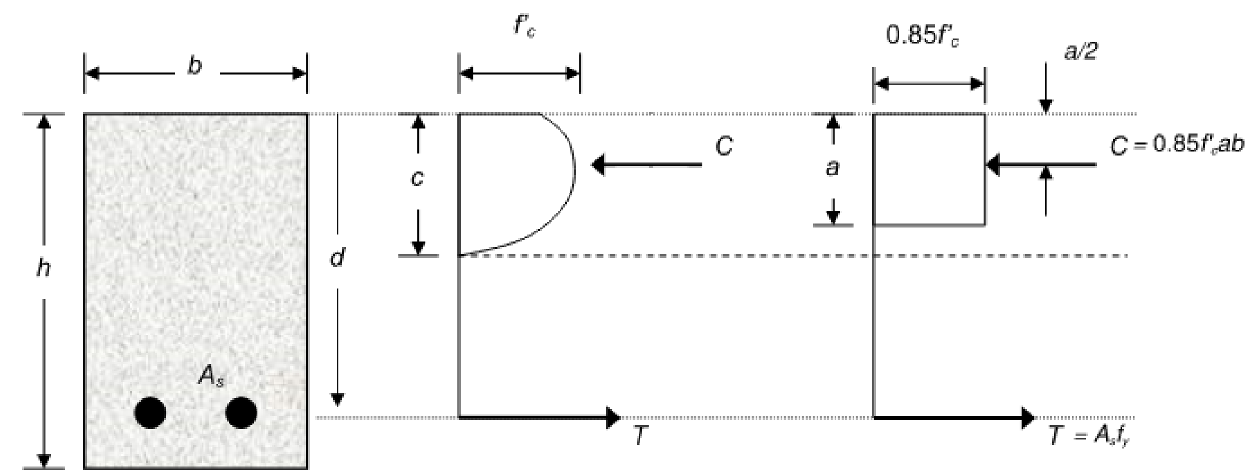


Fig. 85; Stress block for concrete (Whitney)

Design parameters:
f_y = yield strength of reinforcing steel (psi)
This is almost always 60,000 psi. If other yield strengths are proposed, the engineer should check with the contractor on availabililty.

E_c = Young's modulus of concrete for normal weight concrete
= $\mathbf{57{,}000}\sqrt{f'c}$, 19.2.2.1b (psi)
E_s = Young's modulus of reinforcing steel (psi)
A_s = Area of reinforcing steel per unit width (in^2)
A_{se} = Effective area of reinforcing steel (in^2)
= $A_s + \frac{P_u}{f_y}\left(\frac{h/2}{d}\right)$
Note: ACI 551 describes the use of Ase:
"Applied axial forces will counteract a portion of the flexural tension stresses in the concrete section, resulting in increased bending moment resistance. For small axial stresses less than 0.1 fc', this can be accounted for by a simple modification of the area of reinforcement [see above]…A_{se} can also be used to account for the increased bending stiffness when computing ***P-Δ*** deflections."
d = depth from compression face to centerline of the reinforcing that is used to resist bending (in)
a = depth of equivalent rectangular compressive stress block (in)
= $\frac{P_u+A_sF_y}{0.85f'_c b}$
b = width of wall (in-plane) (in)
$c = a/\beta_1$ = distance from extreme compression fiber to neutral axis (in)

$$\beta_1 = \begin{cases} 0.85 \; for \; f'_c \le 4{,}000 \; psi \\ 0.85 - 0.05\left(\frac{f'_c - 4{,}000}{1{,}000}\right) \ge 4{,}000 \; psi \end{cases}$$

ε_1 = axial strain in reinforcing steel
= ***(0.003)d/c-0.003*** per common concrete design methods
Must be ≥ 0.005 for tension control
$M_n = (A_{se}F_y)/\left(d - \frac{a}{2}\right)$ per common concrete design methods
$n = E_s / E_c$

= Shall be at least 6 per 11.8.3.1c

I_{cr} = Cracked moment of inertia of concrete section (in^4)

$$= nA_{se}(d-c)^2 + \frac{1}{3}bc^3$$

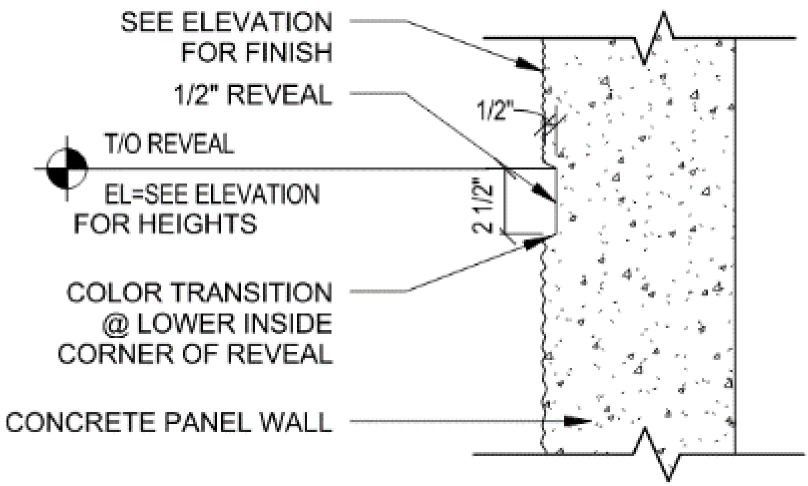
M_{ua} = Maximum factored moment at midheight of wall not including ***P-Δ*** effects

M_u = Maximum factored moment at midheight of wall *including* ***P-Δ*** effects

$$= \frac{M_{ua}}{\left(1 - \frac{5P_u l^2}{(0.75)48E_c I_{cr}}\right)}$$

Note that the depth to the reinforcing steel, *d*, must account for such things as panel reveals (Fig. 86), which are used to break up massive continuous planes of concrete on exposed (or exposed and painted) exterior concrete faces. The distance ***d*** must start at the face within the reveal, not at the greater thickness outside the reveal.

As stated previously, panel thickness will be based on a balance between load demands and the economy of concrete and reinforcing. The decision should be based on a cost analysis with input from the general contractor. **If a thinner choice of panel leads to a reduced number of panels by increasing the length of individual panels that will work with the crane**, the savings in labor and crane time may be worth considering. Options include one layer of reinforcing centered in the wall, or reinforcing in two layers, one layer close to each face. Obviously, two layers of reinforcing will use more reinforcing and result in more labor, but the increase in the distance ***d*** can help the strength of the panel tremendously. Thickness may be controlled by the jamb design, as is the case for many walls at dock doors. The Also, the need to control deflections may influence the thickness of panels.



CONCRETE PANEL REVEAL

SCALE: 1 1/2" = 1'-0"

Fig. 86

Deflections of tilt-up wall panels are somewhat predictable. Fig. 87 shows the evolution of design provisions. The top curve is the earlier UBC curve used to predict panel deflections. The middle dotted line shows test results, and the bottom solid line shows how adjustments were made to code provisions to better match experimental data. **In all cases, there is a strong bilinear relationship where the line breaks when the moment reaches 2/3M_{cr}.**

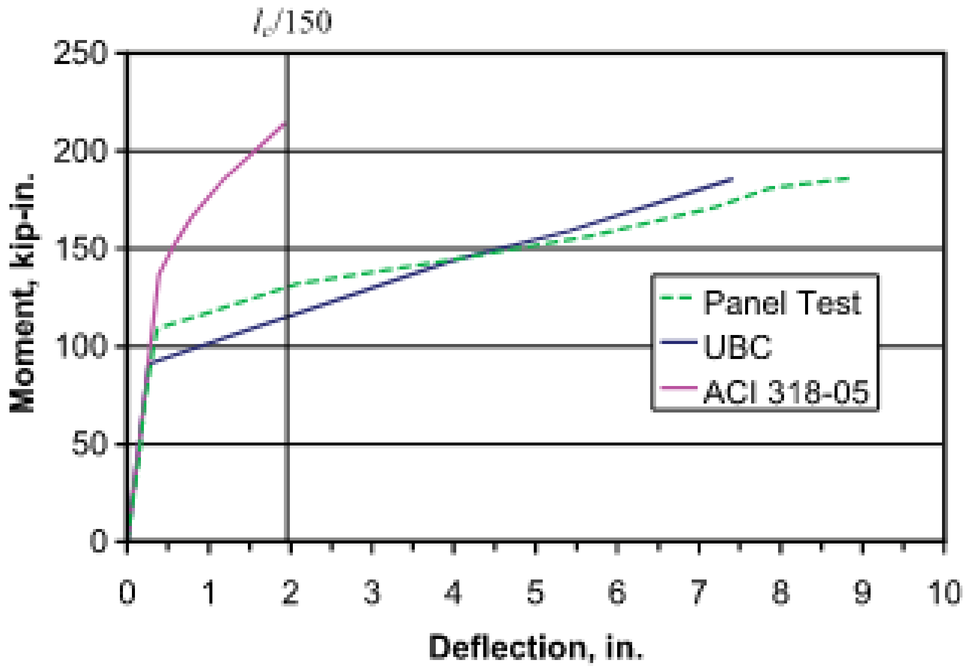


Fig. 1: Comparison of moment-deflection curve from panel test with predicted deflection based on ACI 318-05 equations and UBC 97 equations

Fig. 87, Bilinear deflection curve for tilt-up panels (Lawson)

The latest version of deflection provisions in ACI 318 appears below:

Reproduction of Table 11.8.4.1:

M_a	Δ_s	
$\leq (2/3)M_{cr}$	$\Delta_s = \left[\frac{M_a}{M_{cr}}\right]\Delta_{cr}$	**(a)**
$> (2/3)M_{cr}$	$\Delta_s = (2/3)\Delta_{cr} + \frac{(M_a - (2/3)M_{cr})}{(M_a - (2/3)M_{cr})}(\Delta_n - (2/3)\Delta_{cr})$	**(b)**

M_{sa} = Maximum service moment at midheight of wall not including ***P-Δ*** effects

M_a = Maximum service moment at midheight of wall <u>*including*</u> ***P-Δ*** effects

$= M_{sa} + P_s\Delta_s$

$P_s\Delta_s$ = Service level axial load

Δ_s = Out-of-plane deflection due to service loads, see table above

$$\Delta_{cr} = \frac{5M_{cr}l_c^2}{48E_cI_g}$$

$$\Delta_n = \frac{5M_n l_c^2}{48E_cI_{cr}}$$

Reinforcing steel and clear cover, per ACI 318

Concrete cover:

If standard wall provisions are used, use ACI 318, 20.6.1.3.1, or

Table 20.6.1.3.1—Specified concrete cover for cast-in-place nonprestressed concrete members

Concrete exposure	Member	Reinforcement	Specified cover, in.
Cast against and permanently in contact with ground	All	All	3
Exposed to weather or in contact with ground	All	No. 6 through No. 18 bars	2
		No. 5 bar, W31 or D31 wire, and smaller	1-1/2
Not exposed to weather or in contact with ground	Slabs, joists, and walls	No. 14 and No. 18 bars	1-1/2
		No. 11 bar and smaller	3/4
	Beams, columns, pedestals, and tension ties	Primary reinforcement, stirrups, ties, spirals, and hoops	1-1/2

For more controlled conditions, precast concrete provisions are an option in ACI 318, 20.6.1.3.3,

Table 20.6.1.3.3—Specified concrete cover for precast nonprestressed or prestressed concrete members manufactured under plant conditions

Concrete exposure	Member	Reinforcement	Specified cover, in.
Exposed to weather or in contact with ground	Walls	No. 14 and No. 18 bars; tendons larger than 1-1/2 in. diameter	1-1/2
		No. 11 bars and smaller; W31 and D31 wire and smaller; tendons and strands 1-1/2 in. diameter and smaller	3/4
	All other	No. 14 and No. 18 bars; tendons larger than 1-1/2 in. diameter	2
		No. 6 through No. 11 bars; tendons and strands larger than 5/8 in. diameter through 1-1/2 in. diameter	1-1/2
		No. 5 bar, W31 or D31 wire, and smaller; tendons and strands 5/8 in. diameter and smaller	1-1/4
Not exposed to weather or in contact with ground	Slabs, joists, and walls	No. 14 and No. 18 bars; tendons larger than 1-1/2 in. diameter	1-1/4
		Tendons and strands 1-1/2 in. diameter and smaller	3/4
		No. 11 bar, W31 or D31 wire, and smaller	5/8
	Beams, columns, pedestals, and tension ties	Primary reinforcement	Greater of d_b and 5/8 and need not exceed 1-1/2
		Stirrups, ties, spirals, and hoops	3/8

2. <u>Maximum bar spacing</u>:

a. Again, if standard wall provisions are used, use ACI 318, 11.7.2.1 & 11.7.3.1,

Longitudinal (vertical) bars

11.7.2.1 Spacing s of longitudinal bars in cast-in-place walls shall not exceed the lesser of $3h$ and 18 in. If shear reinforcement is required for in-plane strength, spacing of longitudinal reinforcement shall not exceed $\ell_w/3$.

Transverse (horizontal) bars

11.7.3.1 Spacing s of transverse reinforcement in cast-in-place walls shall not exceed the lesser of $3h$ and 18 in. If shear reinforcement is required for in-plane strength, s shall not exceed $\ell_w/5$.

b. For more controlled conditions, use precast concrete provisions in ACI 318, 11.7.2.2, Longitudinal (vertical) bars

11.7.2.2 Spacing *s* of longitudinal bars in precast walls shall not exceed the lesser of (a) and (b):

(a) **5*h***
(b) 18 in. for exterior walls or 30 in. for interior walls

If shear reinforcement is required for in-plane strength, *s* shall not exceed the smallest of **3*h***, 18 in., and $\ell_w/3$.

Transverse (horizontal) bars

11.7.3.2 Spacing *s* of transverse bars in precast walls shall not exceed the lesser of (a) and (b):

(a) **5*h***
(b) 18 in. for exterior walls or 30 in. for interior walls

If shear reinforcement is required for in-plane strength, *s* shall not exceed the least of **3*h***, 18 in., and $\ell_w/5$

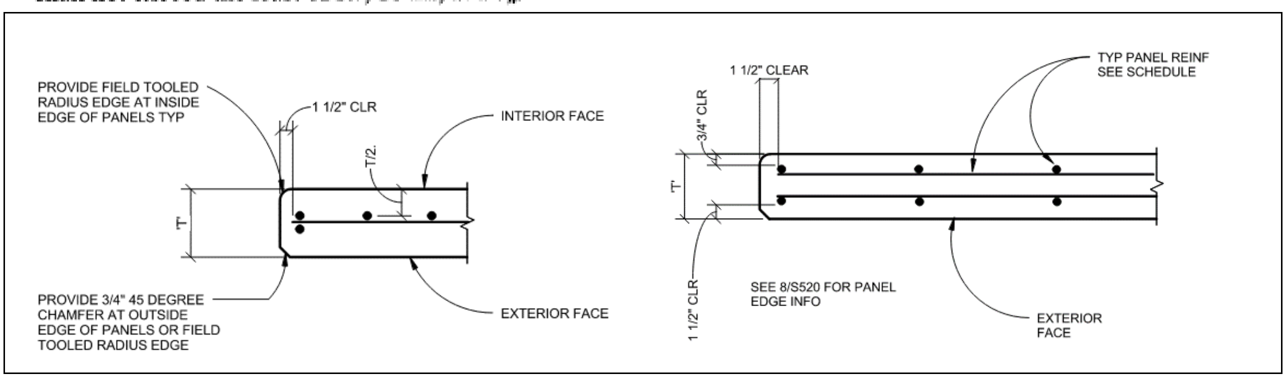


Fig. 88: Vertical (dot), and Horizontal (Line) Reinforcing in wall panels

Bar spacing, 11.7.2.2

Walls with h<10" are permitted to have one layer of reinforcing, with the vertical reinforcing centered in the wall, and the horizontal reinforcing toward one face.

Walls with h >10" require reinforcing in both the interior and exterior faces. These can be placed per 11.7.2.3:

11.7.2.3 For walls with ***h*** greater than 10 in., except basement walls and cantilever retaining walls, distributed reinforcement for each direction shall be placed in two layers parallel with wall faces in accordance with (a) and (b):

(a) One layer consisting of at least one-half and not exceeding two-thirds of total reinforcement required for each direction shall be placed at least 2 in., but not exceeding ***h*/3**, from the exterior surface.
(b) The other layer consisting of the balance of required reinforcement in that direction, shall be placed at least 3/4 in., but not greater than ***h*/3**, from the interior surface.

Minimum reinforcing:
Though not strictly required unless used as shear walls, use ACI 318, 11.6 as practical minimums for shrinkage and temperature and to control cracking.

Wall type	Type of nonprestressed reinforcement	Bar/wire size	f_y, psi	Minimum longitudinal[1], ρ_ℓ	Minimum transverse, ρ_t
Cast-in-place	Deformed bars	≤ No. 5	≥ 60,000	0.0012	0.0020
			< 60,000	0.0015	0.0025
		> No. 5	Any	0.0015	0.0025
	Welded-wire reinforcement	≤ W31 or D31	Any	0.0012	0.0020

Fig. 89, ACI 318, Table 11.6.1

Longitudinal/Vertical: $\rho_l = \boldsymbol{A_{sv}/bh}$**,** where $\boldsymbol{A_{sv}}$ is the vertical bar area
Transverse/Horizontal: $\rho_h = \boldsymbol{A_{sh}/bh}$ where $\boldsymbol{A_{sh}}$ is the horizontal bar area

Reinforcing required at openings, around each side of the opening, and fully developed is noted in 11.7.5.1.
2-#5 bars in walls with reinforcing in each face
1-#5 bar in walls with reinforcing in a single layer

It is common practice to use 2-#5 at openings, no matter whether there is one or two layers of reinforcing (Fig. 90).

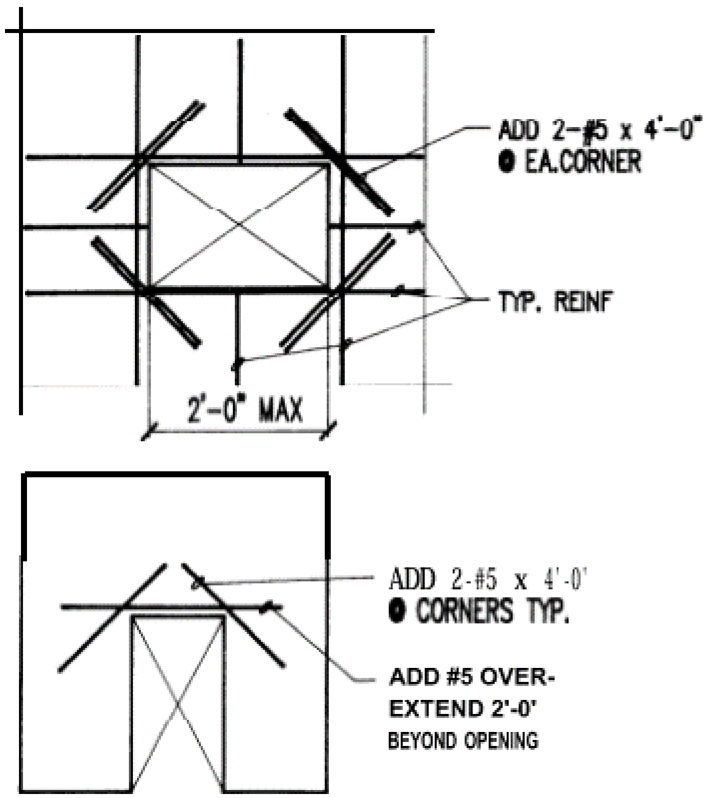


Fig. 90: Corner bars at panel openings (ACI 551)

Closed ties may be required per 11.7.4.1.
Transverse ties are required if A_{st} exceeds $0.01A_g$ where A_{st} is the total amount of longitudinal steel. (Fig. 91).

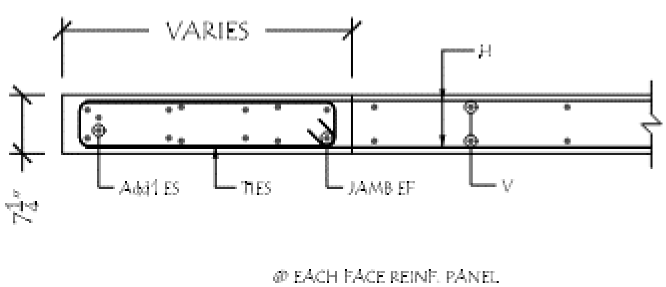


Fig. 91: Transverse ties at a panel region with high axial load

Reinforcing development lengths and splices:
See 25.4 and 25.5 of ACI 318 (318-14 and later).

Design of connections from load bearing elements to the tilt-up wall panels is per concrete design references ACI 318 (Ch. 17), ACI 355, and steel design references AISC (2010a, 2010b). See Fig. 92.

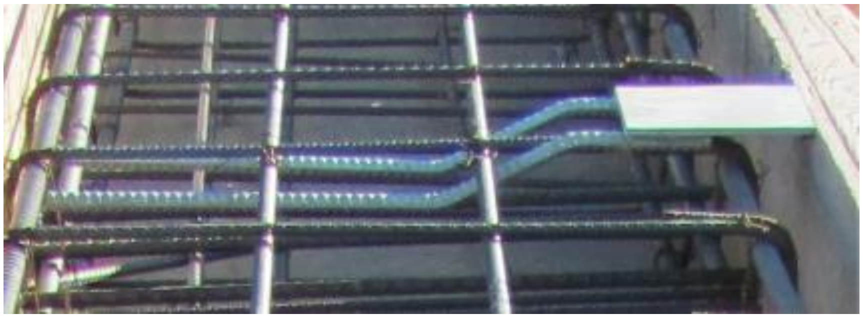


Fig. 92: Embed plate for panel connection

Design Examples

Use the previous equations from the 11.8 slender wall provisions of ACI 318 in wall design.

Example 1: Typical wall

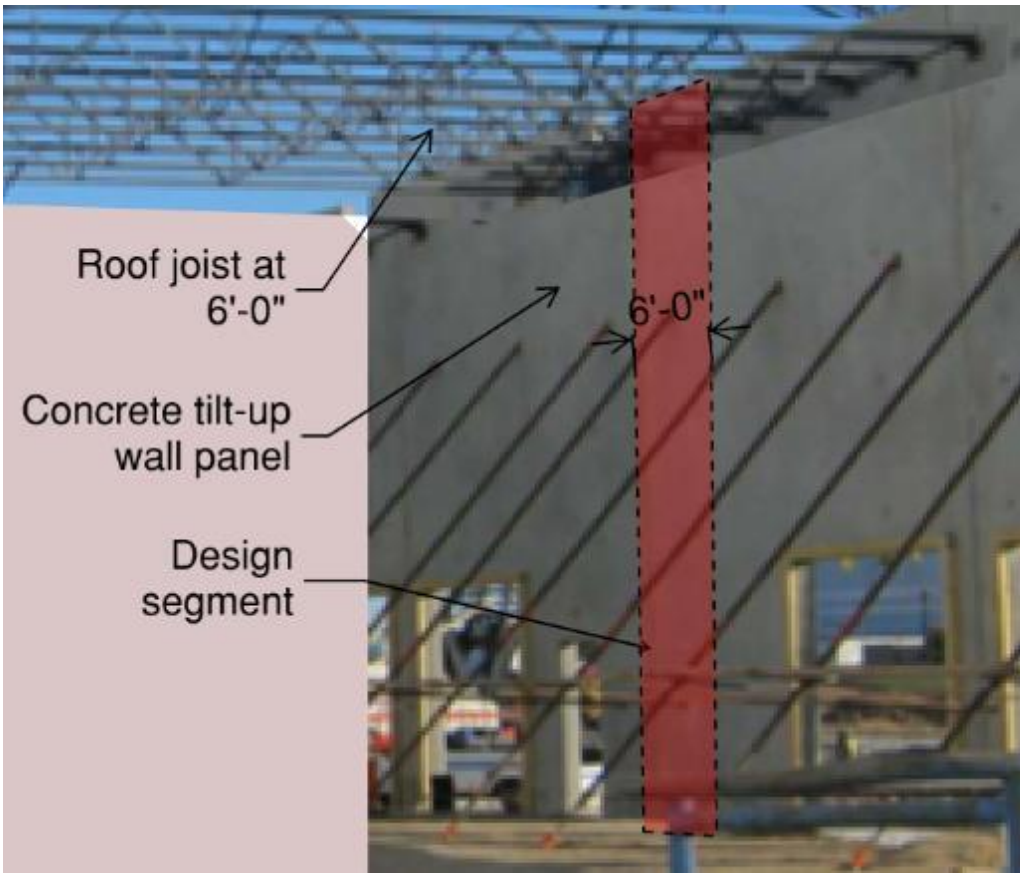


Fig. 93, Typical load bearing exterior wall

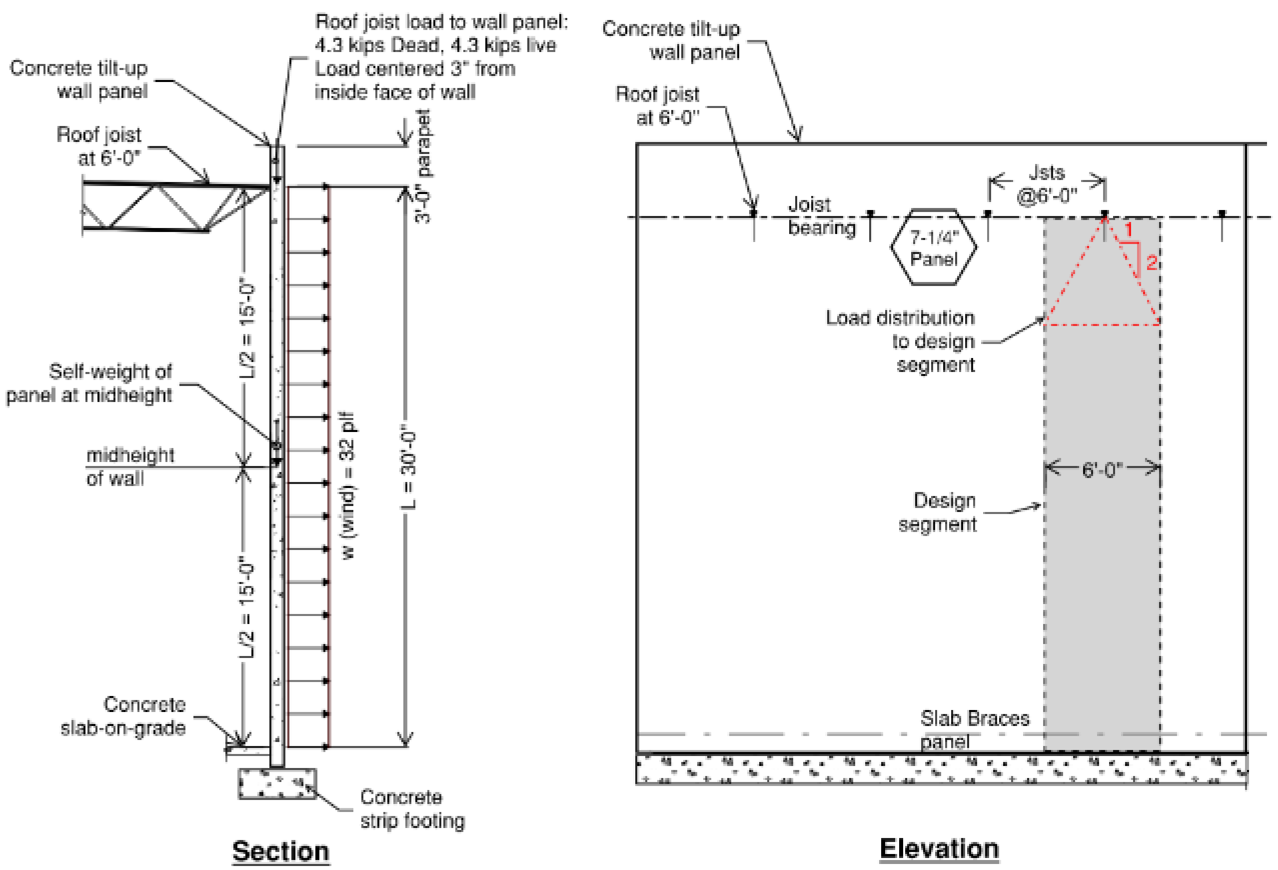


Fig. 94: Example 1: Typical Wall Section and Elevation

Design info:
See Figs. 93 & 94
f'_c =4,000psi
F_y = 60,000 psi
Design with b = 12”
Panel thickness h = 7.25”
Depth to reinforcing (#5, 5/8” bars) = 7.25”-1.5”-0.625”/2 = 5.44”

Load case used in design. See Fig. 68 (others need to be checked as well).:
1.2D + 1.0W + 0.5L

Check also 1.2D + 1.6L for the vertical stress limit of 0.06 $\boldsymbol{f'_c}$.

1. <u>Wall thickness:</u> Base the thickness on using reinforcing each face and a contractor-preferred 7-1/4" thickness. Check slenderness based on ACI 551.2 recommendations:
ℓ_c / h = 30 x 12 / 7.25 = 50 < 65 OK

2. <u>Steel roof joists</u>:
a. Gravity load: With joists at 6'-0" on center, and a load that distributes at a 2:1 height:width ratio, the load is uniform at 6'-0" below bearing.

Its value is P_{dw} = 4.32k / 6ft = 0.72klf service level dead load, and P_{lw} = 4.32k / 6ft = 0.72klf service level live load.

b. Eccentric moment: Assume bearing connection to wall is as shown below. Center of bearing is not at center of wall, so account for eccentric bearing by applying a concentrated moment at the point of bearing.

See Fig. 95. Accounting for ½ thickness of the wall, the gap to the end of joist and knowing the effective bearing will slightly favor toward the supported leg, use e = 7.25"/2 + 3" = 6.625".

This results in concentrated service moments at the top of the wall, that are spread out to the 6'-0" wide wall segment, of M_{dw} = 0.72k x 6.625" x 1"/12ft = 0.40 ft-kip service level dead moment, and M_{lw} = 0.72k x 6.625" x 1"/12ft = 0.40 ft-kip service level live moment.

Concentric bearing is assumed at the base of the wall, where the moment is zero, and the moment is resolved by a constant shear along the wall.

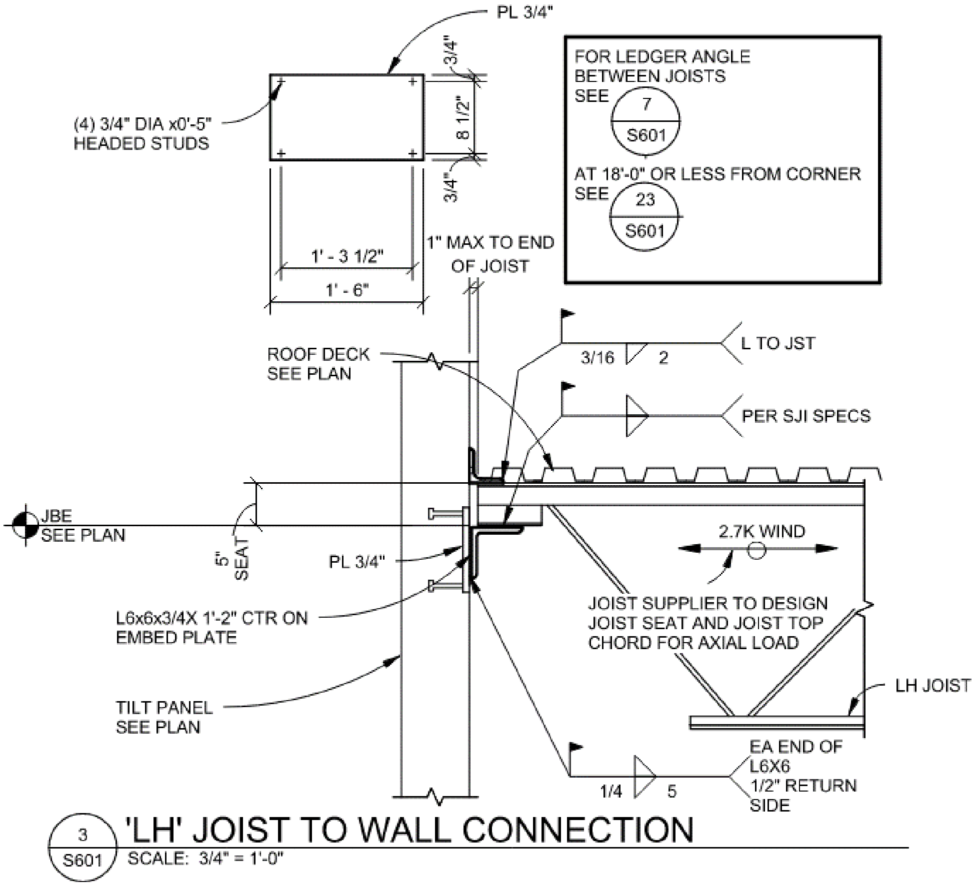


Fig. 95: Top of wall joist-to-wall connection

3. Self-weight: The critical section for bending moment plus axial load is taken at the midheight of the panel span. Therefore, use the self-weight at midheight. In this case, it's $\boldsymbol{P_{sw}}$ = 90.6psf x (15ft+3ft) /1000 = 1.63 klf service level dead load.

4. Lateral load on the panel: Assume wind loading governs. Use 1.0 W case for ultimate loading from ASCE 7-16 for use in LRFD loading. Ultimate level uniform loading is $\boldsymbol{w_u}$ = 0.032 psf.

For deflection checks, use service level loading of 0.6W, or $\boldsymbol{w_s}$ = 0.6 x 0.032 ksf = 0.20ksf.

A couple of notes:

a. Watch for increased wind forces at the primary corners of the building as defined by ASCE 7.
b. This example uses wind as the design load case. Where it applies, seismic loading also needs to be considered, as prescribed by ASCE 7.

5. Panel reinforcing:
Longitudinal (Vertical) steel:
Try #5@16"oc each face
$\rho_l = \boldsymbol{A_{sv}/bh}$ = (2x0.31x12/16)/(12x7.25) = 0.0053 > 0.0015 OK

Transverse (Horizontal) steel:
Try #4@18"oc each face per vertical foot of height
$\rho_h = \boldsymbol{A_{sh}/(12")h}$ = (2x0.2x12/18)/(12x7.25) = 0.0031 > 0.0020

6. Check 1.2D + 1.6L for the vertical stress limit of 0.06 f'_c.
P_u = 1.2 x (0.72k Joists + 1.63k self) + 1.6 x 0.72k = 3.97 k
P_u/A_g = 3.97k / (7.25"x12") = 0.046 ksi < 0.06 f'_c. = 0.06 x 4 = 0.24 ksi
By inspection, load case 1.2D+1.0W+0.5L is OK as well.

7. Check the moment design strength for the 1.2D+1.0W+0.5L load case:

Axial load at midheight:
P_u = 1.2 x (0.72k +1.63k) + 0.5 x 0.72k = 3.18k
A_{se} = Effective area of reinforcing steel (in^2)

$$= A_s + \frac{P_u}{f_y}\left(\frac{h/2}{d}\right) = 0.31x12/16 + \frac{3.18}{60}\left(\frac{7.25/2}{5.44}\right) = 0.268 \text{ in}^2$$

a = depth of compressive stress block (in)

$$= \frac{A_{se}F_y}{0.85f'_c b} = \frac{0.268x60}{0.85x4x12} = 0.394 \text{ in}$$

E_s = 29,000 ksi
$E_c = 57{,}000\sqrt{f'c} = 57{,}000\sqrt{4{,}000}$ / 1000 #/k = 3,605 ksi
$n = E_s / E_c$ = 29,000/3.605 = 8.04 > 6 (per 11.8.3.1c) OK
I_{cr} = Cracked moment of inertia of concrete section (in^4)

$c = a/0.85$ = 0.394/0.85 = 0.464"

$$= nA_{se}(d-c)^2 + \frac{1}{3}bc^3$$

$= \mathbf{8.04x0.268(5.44 - 0.464)^2 + \frac{1}{3}12x0.464^3}$ = 53.75 in^4

$\boldsymbol{f_r}$ **= 7.5** $\lambda \sqrt{f'c}$ = 7.5 x 1.0 x $\sqrt{4{,}000}$ = 0.474 ksi

$\boldsymbol{M_{cr}}$ = $\boldsymbol{Sf_r}$

S = ***1/6bt²*** (in^3)

$\boldsymbol{M_{cr}}$ = *1/6 x 12 x (7.25)^2 x 0.474 / 12 in/ft = 4.15 ft-kip*

$\boldsymbol{\phi M_n} = \emptyset\left(A_{se}F_y\right)\left(d - \frac{a}{2}\right) = \mathbf{0.9(0.268x60)\left(5.44 - \frac{0.394}{2}\right)}/(\frac{12in}{ft})$ = 6.32 ft-kip > $\boldsymbol{M_{cr}}$ =4.15 OK

Check if the wall is tension controlled.

ε_1 = axial strain in reinforcing steel

= ***(0.003)d/c-0.003*** = (0.003)(5.44/0.464)-0.003 = 0.032 > 0.005

Must be ≥ 0.005 for tension control = OK

Check bending moments on the wall against bending strength.

$\boldsymbol{M_{ua}}$ = Maximum factored moment at midheight of wall not including ***P-Δ*** effects, which is as follows:

a. Moment due to wind: $\boldsymbol{M_{uw} = w_u l_c^2/8}$ = 0.032klf x (30ft)^2 / 8 = 3.60 ft-kip
b. Moment due to eccentricity (note does not include dead weight of panel):
   $\boldsymbol{M_{ue} = P_{ue}e/2}$ = (1.2 x 0.72k + 0.5 x 0.72k) x 6.625" / 2 x 1ft/12in= 0.34 ft-kip
c. Sum of moments = $\boldsymbol{M_{ua}}$ = 3.60 ft-kip + 0.34 ft-kip = 3.94 ft-kip

Considering Fig. 96,

$\boldsymbol{M_u}$ = Maximum factored moment at midheight of wall <u>*including*</u> ***P-Δ*** effects

$= \frac{M_{ua}}{\left(1 - \frac{5P_u l^2}{(0.75)48E_c I_{cr}}\right)} = \frac{3.94\ ft-k}{\left(1 - \frac{5x3.18x(30x12)^2}{(0.75)48x3{,}605x53.75}\right)}$ = 5.59 ft-kip < $\boldsymbol{\phi M_n}$ = 6.32 ft-kip OK

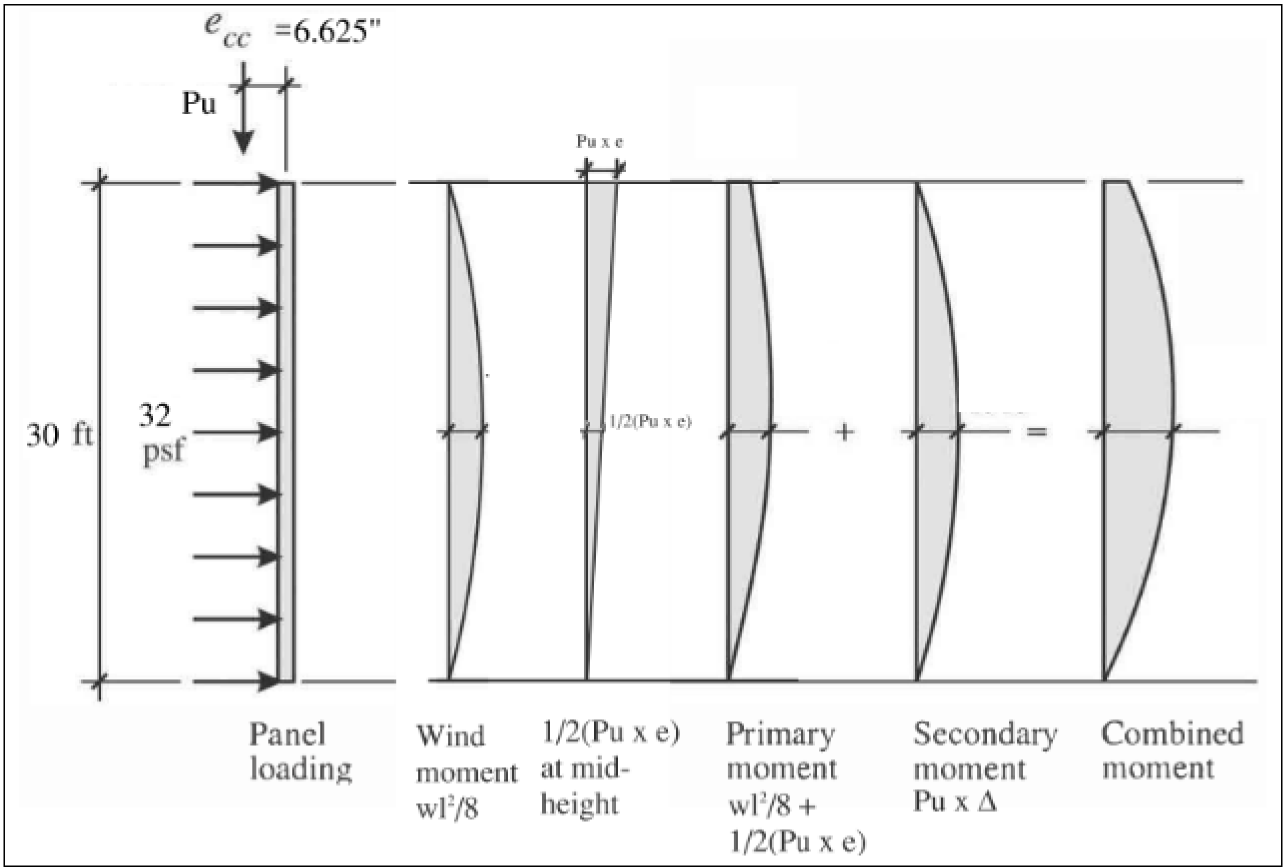


Fig. 96: Moments on the wall

Check deflection per ACI 318, 11.8.1.1(e) and 11.8.4.

Reproduction of Table 11.8.4.1:

M_a	Δ_s	
$\leq (2/3)M_{cr}$	$\Delta_s = \left[\frac{M_a}{M_{cr}}\right]\Delta_{cr}$	**(a)**
$> (2/3)M_{cr}$	$\Delta_s = (2/3)\Delta_{cr} + \frac{(M_a - (2/3)M_{cr})}{(M_n - (2/3)M_{cr})}(\Delta_n - (2/3)\Delta_{cr})$	**(b)**

M_{sa} = Maximum service moment at midheight of wall not including ***P-Δ*** effects
Governing load cases are D + (0.6W) and D + 0.75L + 0.75 (0.6W) from ASCE 7, 2.4.1. Combine for single check and use D + (0.6W) + 0.75L.

Wind: $M_{sw} = w_s l_c^2/8$ = 0.020klf x (30ft)^2 / 8 = 2.25 ft-kip

Eccentric Bearing: M_{se} = $P_{se}e/2$ = (0.72k + 0.75x0.72k) x 6.625"/2 x1ft/12in= 0.35 ft-kip
Sum of moments = M_{sa} = 2.25 + 0.35 = 2.60 ft-kip

M_a = Maximum service moment at midheight of wall *including* P-Δ effects

P_s = 0.72 + 1.63 self + 0.75x0.72 = 2.89k

Δ_s = See table above

And $M_a = M_{sa} + P_s\Delta_s$ (with $P_s\Delta_s$ based on service level axial load)

Now, (2/3)M_{cr} = (2/3) x 4.15 = 2.77 ft-kip

Allowable Δ_s, including PΔ effects, is $l_c/150$ = 30x12/150 = 2.4 in

$I_g = (1/12)bh^3$ = (1/12)12*7.25^3 = 381 in^4

$$\Delta_{cr} = \frac{5M_{cr}l_c^2}{48E_cI_g} = \frac{5x4.15x12x(30x12)^2}{48x3{,}605x381} = 0.49\text{in}$$

Note that, if needed, M_n and Δ_n should be calculated using service loads:

A_{se} = Effective area of reinforcing steel (in^2)

$$= A_s + \frac{P_s}{f_y}\left(\frac{h/2}{d}\right) = \mathbf{0.31x12/16} + \frac{2.89}{60}\left(\frac{7.25/2}{5.44}\right) = 0.265\text{ in}^2$$

a = depth of compressive stress block (in)

$$= \frac{A_{se}F_y}{0.85f'_cb} = \frac{0.265x60}{0.85x4x12} = 0.39\text{ in}$$

$$M_n = (A_{se}F_y)\left(d - \frac{a}{2}\right) = (\mathbf{0.265x60})\left(\mathbf{5.44} - \frac{0.39}{2}\right)/\left(\frac{12in}{ft}\right) = 6.95\text{ ft-kip}$$

$$\Delta_n = \frac{5M_nl_c^2}{48E_cI_{cr}} = \frac{5x6.95x12x(30x12)^2}{48x3{,}605x53.75} = 5.81''$$

Thus, with, starting with $M_a = M_{sa}$ = 2.60 ft-kip < (2/3)M_{cr} = 2.77 ft-kip

$M_a = M_{sa} + P_s\Delta_s$ with $\Delta_s = \left[\frac{M_a}{M_{cr}}\right]\Delta_{cr}$ **per the above table**

$$\Delta_s = \left[\frac{M_a}{M_{cr}}\right]\Delta_{cr} = \left[\frac{2.60}{4.15}\right]\mathbf{0.49}'' = 0.31\text{in}$$

M_a = = 2.60 + 2.89 x 0.31in/12in/ft = 2.675 ft-kip

Now, perform another iteration that includes the new M_a

New M_a = 2.675 ft-kip < (2/3)M_{cr} = 2.77 ft-kip

$M_a = M_{sa} + P_s\Delta_s$ = 2.6 + 2.89 x (2.675/4.15) 0.49in/12in/ft = 2.676 ft-kip

Because the iteration shows convergence with the last value, no more iterations are required. Thus, final $\Delta_s = \left[\frac{M_a}{M_{cr}}\right]\Delta_{cr} = \left[\frac{2.676}{4.15}\right]\mathbf{0.49}''$ = 0.32in. This value is much less than the allowable Δ_s of $l_c/150$ = 2.4 in.

Example 2: Typical jamb at dock door opening

In examining typical walls, with uniform loading and cross section over the height between supports, the design is straightforward. If openings are added to the wall, there are many ways to approach the wall design, both for the load path and for the section and properties used for design. The method below is just one way to go about it, and it echoes the approach used in ACI literature. For example, wind loads are assumed to be driven uniformly from the area above the opening, as well as wind load from the door itself. Obviously, the behavior of the structure and door may cause wind loads to be induced in the jamb slightly differently. Also, the jamb cross section itself at the opening is projected to the roof as the sole resistance to axial load and bending. One could account for the additional width of the wall above the opening in stiffness and strength calculations. However, the simplifications used are rational and reasonable, meet the intent of ACI 318, 11.8, and in the opinion of the author yield a satisfactory design. The reader should decide on a design method suitable for their own purposes.

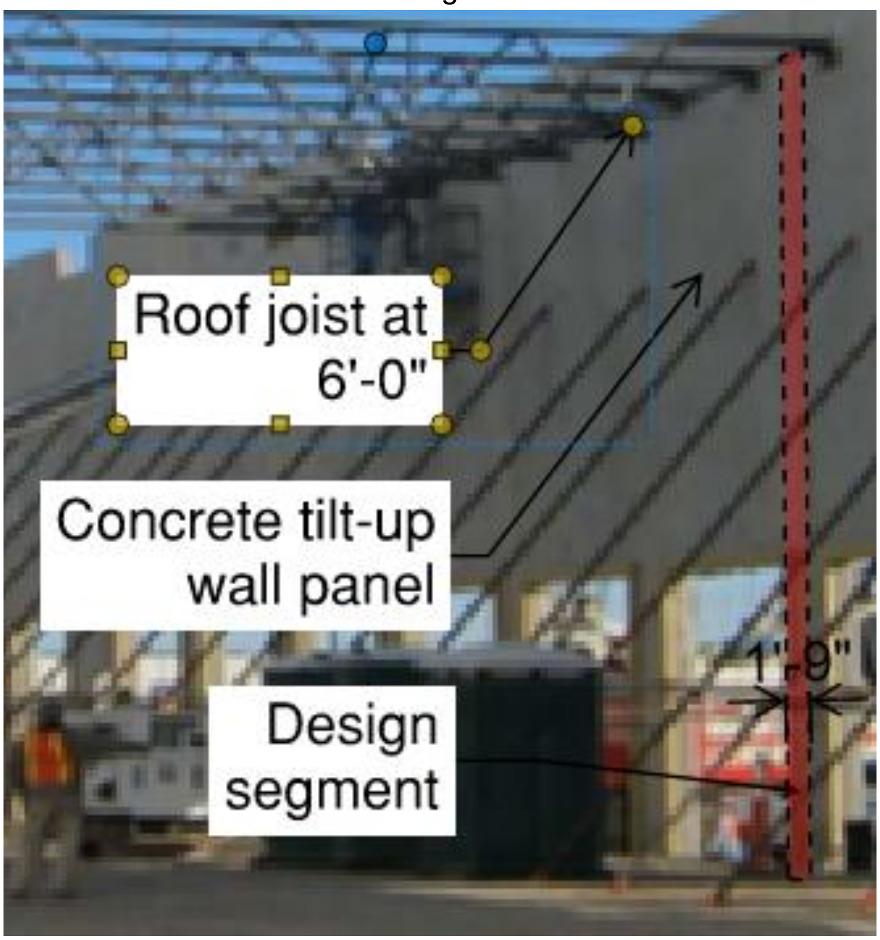


Fig. 97: Example 2, Typical dock door opening jamb

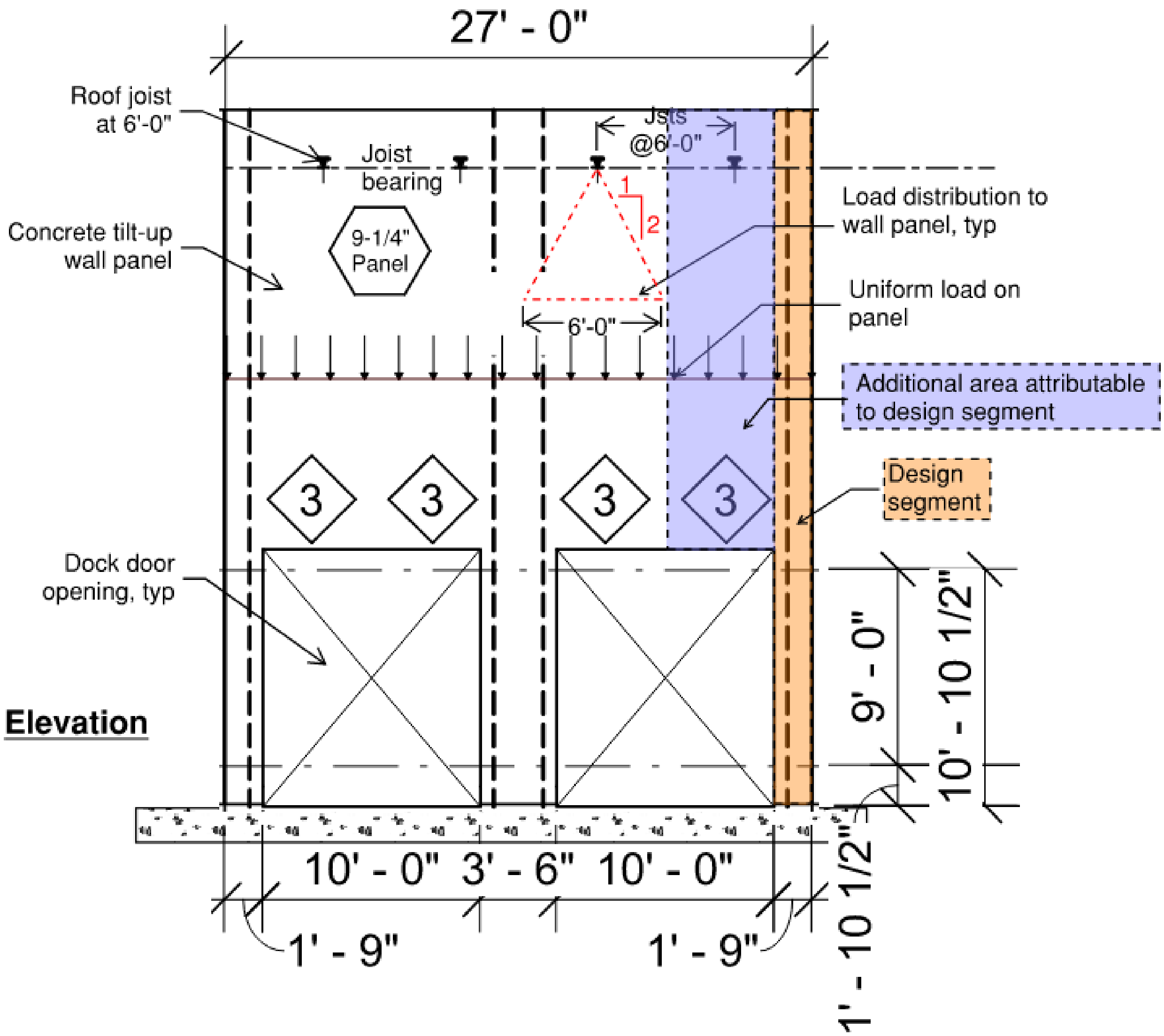


Fig. 98: Example 2 Wall Elevation

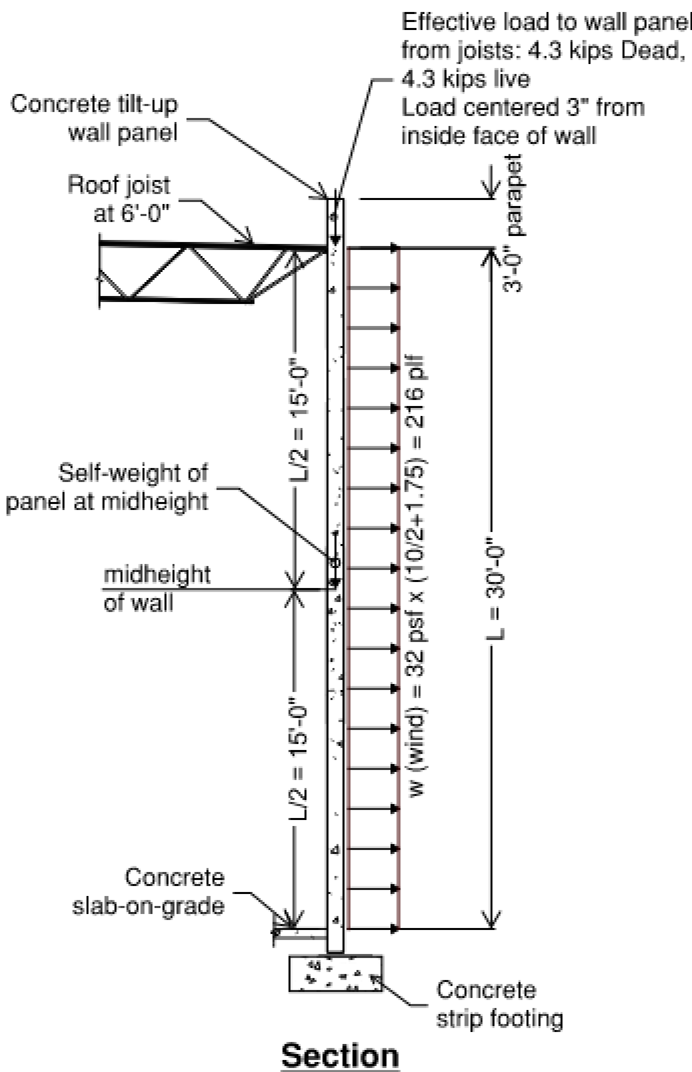


Fig. 99: Example 2 Wall Section

Design info:
See Figs. 97, 98, 99
f'_c =4,000psi
F_y = 60,000 psi
Design with b = 1'-9" = 1.75 ft
Panel thickness h = 9.25"
Depth to reinforcing (#6, 3/4" bars) = 9.25"-1.5"-0.75"/2 = 7.38"

Use previous analysis as a guide:
Load case used in design:
1.2D + 1.0W + 0.5L

Check also ***1.2D + 1.6L*** for the vertical stress limit of 0.06 f'_c.

1. Wall thickness: Use reinforcing each face and a contractor-preferred 9-1/4" thickness. Check slenderness based on ACI 551.2 recommendations:
ℓ_c / h = 30 x 12 / 9.25 = 39 < 65 OK

2. Steel roof joists:
a. Gravity load: Use the uniform load from the previous example and find the load to the jamb. Uniform 0.72klf service level dead load, and 0.72klf service level live load. The load at the jamb due to additional load from the area over the dock door opening P_{dw} = 0.72klf x (10/2 + 1.75) = 4.9k service level dead load, and P_{lw} = 0.72klf x (10/2 + 1.75) = 4.9k service level live load. (Note: The figure shows 4 joists acting on a 27'-0" length of wall, which would result in lower loading than the above, but for simplicity the previous uniform load was used.)
b. Eccentric moment: e = 9.25"/2 + 3" = 7.625". Uniform service moments at the top of the wall of M_{dw} = 0.72k x 7.625" x 1"/12ft x = 0.46 ft-kip service level dead moment, and M_{lw} = 0.72k x 7.625" x 1"/12ft = 0.46 ft-kip service level live moment.

3. Self weight: The self-weight at midheight includes the influence area over the dock door, so for a 9-1/4" PANEL, it's P_{sw} = 115.6psf x (15ft+3ft) x (10/2 + 1.75) /1000 = 14.0k service level dead load.

4. Lateral load on the panel: Use ***1.0W*** case for ultimate loading from ASCE 7-16 for use in LRFD loading. The wind load from the area over the dock door as well as the dock door area itself is assumed to be distributed evenly and uniformly to the jambs on each side. Ultimate level uniform loading is w_u = 0.032 psf x (10/2+1.75) = 0.216klf, For deflection checks, use service level loading of ***0.6W***, or w_s = 0.6 x 0.216 klf = 0.130klf.

5. Panel reinforcing:
Longitudinal (Vertical) steel:
Try 3-#6 each face in 1'-9" jamb width
$\rho_l = A_{sv}/bh$ = (2x3x0.44)/(12x1.75x9.25) = 0.0136 > 0.0015 OK

Transverse (Horizontal) steel:
Try #4@18"oc each face per vertical foot of height
$\rho_h = A_{sh}/(12")h$ = (2x0.2x12/18)/(12x9.25) = 0.0024 > 0.0020 OK

6. Check ***1.2D + 1.6L*** for the vertical stress limit of ***0.06 f'c***.
P_u = 1.2 x (4.9k Joists + 14.0k self) + 1.6 x 4.9k = 30.5 k
P_u/A_g = 30.5k / (9.25"x21") = 0.16 ksi < ***0.06 f'c***. = 0.06 x 4 = 0.24 ksi
By inspection, load case ***1.2D+1.0W+0.5L*** is OK as well.

7. Check the moment design strength for the ***1.2D+1.0W+0.5L*** load case:

Axial load at midheight:
P_u = 1.2 x (4.9k +14.0k) + 0.5 x 4.9k = 25.1k
A_{se} = Effective area of reinforcing steel (in^2)

$$= A_s + \frac{P_u}{f_y}\left(\frac{h/2}{d}\right) = \mathbf{0.44x3(ea.face)} + \frac{25.1}{60}\left(\frac{9.25/2}{7.38}\right) = 1.58 \text{ in}^2$$

a = depth of compressive stress block (in)

$$= \frac{A_{se}F_y}{0.85f'_c b} = \frac{1.58x60}{0.85x4x21} = 1.33 \text{ in}$$

E_s = 29,000 ksi
$E_c = 57{,}000\sqrt{f'c} = 57{,}000\sqrt{4{,}000}$ / 1000 #/k = 3,605 ksi
$n = E_s / E_c$ = 29,000/3.605 = 8.04 > 6 (per 11.8.3.1c) OK
I_{cr} = Cracked moment of inertia of concrete section (in^4)

$c = a/0.85$ = 1.33/0.85 = 1.56"

$$= nA_{se}(d-c)^2 + \frac{1}{3}bc^3$$

$$= \mathbf{8.04x1.58(7.38-1.56)^2 + \frac{1}{3}21x1.56^3} = 457 \text{ in}^4$$

$f_r = 7.5\,\lambda\sqrt{f'c}$ = 7.5 x 1.0 x $\sqrt{4{,}000}$ = 0.474 ksi
$M_{cr} = Sf_r$
$S = 1/6bt^2$ (in^3)
M_{cr} = 1/6 x 21 x (9.25)^2 x 0.474 / 12 in/ft = 11.83 ft-kip
$\phi M_n = \emptyset(A_{se}F_y)\left(d-\frac{a}{2}\right) = \mathbf{0.9(1.58x60)}\left(\mathbf{7.38-\frac{1.33}{2}}\right)/(\frac{12in}{ft})$ = 47.7ft-kip > M_{cr} = 11.83 OK

Check if the wall is tension controlled.
ε_1 = axial strain in reinforcing steel

= ***(0.003)d/c-0.003*** = (0.003)(7.38/1.56)-0.003 = 0.011 > 0.005
Must be ≥ 0.005 for tension control = OK

Check bending moments on the wall against bending strength.
M_{ua} = Maximum factored moment at midheight of wall not including ***P-Δ*** effects, which is as follows:

a. Moment due to wind: $M_{uw} = w_u l_c^2/8$ = 0.216klf x (30ft)^2 / 8 = 24.3 ft-kip
b. Moment due to eccentricity (note does not include dead weight of panel):
$M_{ue} = P_{ue}e/2$ = (1.2x0.46ft-k+0.5x0.46ft-k)x(10/2+1.75)/2 = 2.64 ft-kip
c. Sum of moments = M_{ua} = 24.3 ft-kip + 2.6 ft-kip = 26.9 ft-kip

M_u = Maximum factored moment at midheight of wall *including* ***P-Δ*** effects

$$= \frac{M_{ua}}{\left(1-\frac{5P_u l^2}{(0.75)48E_c I_{cr}}\right)} = \frac{26.9\ ft-k}{\left(1-\frac{5x25.2x(30x12)^2}{(0.75)48x3{,}605x457}\right)} = 37.1 \text{ ft-kip} < \phi M_n = 47.7 \text{ ft-kip OK}$$

Check deflection per ACI 318, 11.8.1.1(e) and 11.8.4.
Governing load cases are D + (0.6W) and D + 0.75L + 0.75 (0.6W) from ASCE 7, 2.4.1.
Combine for single check and use D + (0.6W) + 0.75L.

Wind: $M_{sw} = w_s l_c^2/8$ = 0.135klf x (30ft)^2 / 8 = 15.2 ft-kip

Eccentric Bearing:
$M_{se} = P_{se}e/2$ = (0.72k+0.75x0.72k)x7.625"/2x(10/2+1.75)x1ft/12in= 2.7 ft-kip
Sum of moments = M_{sa} = 15.2 + 2.7 = 17.9 ft-kip

M_a = Maximum service moment at midheight of wall *including* ***P-Δ*** effects

P_s = (0.72 + 0.75x0.72)(10/2+1.75) + 14.0 self = 22.5k
Δ_s = See **Table 11.8.4.1**
And $M_a = M_{sa} + P_s\Delta_s$ (with $P_s\Delta_s$ based on service level axial load)

Now, (2/3)M_{cr} = (2/3) x 11.83 = 7.89 ft-kip
Allowable Δ_s, including PΔ effects, is $l_c/150$ = 30x12/150 = 2.4 in
$I_g = (1/12)bh^3$ = (1/12)21*9.25^3 = 1,385 in^4

$$\Delta_{cr} = \frac{5M_{cr}l_c^2}{48E_c I_g} = \frac{5x11.83x12x(30x12)^2}{48x3605x1385} = 0.38\text{in}$$

M_n and ***Δ_n*** should be calculated using service loads:
A_{se} = Effective area of reinforcing steel (in^2)

$$= A_s + \frac{P_s}{f_y}\left(\frac{h/2}{d}\right) = \mathbf{0.44x3} + \frac{22.5}{60}\left(\frac{9.25/2}{7.38}\right) = 1.56 \text{ in}^2$$

a = depth of compressive stress block (in)

$$= \frac{A_{se}F_y}{0.85f'_c b} = \frac{1.56x60}{0.85x4x21} = 1.31 \text{ in}$$

$$M_n = \left(A_{se}F_y\right)\left(d - \frac{a}{2}\right) = \left(\mathbf{1.56x60}\right)\left(\mathbf{7.38} - \frac{1.31}{2}\right)/\left(\frac{12in}{ft}\right) = 52.5 \text{ ft-kip}$$

$$\Delta_n = \frac{5M_n l_c^2}{48E_c I_{cr}} = \frac{5x52.5x12x(30x12)^2}{48x3{,}605x457} = 5.16''$$

Thus, starting with ***M_a*** = ***M_{sa}*** = 17.9 ft-kip > (2/3)***M_{cr}*** = 7.89 ft-kip

$M_a = M_{sa} + P_s\Delta_s$ with

$$\Delta_s = (2/3)\Delta_{cr} + \frac{(M_a - (2/3)M_{cr})}{(M_n - (2/3)M_{cr})}\left(\Delta_n - (2/3)\Delta_{cr}\right) \text{ in table 11.8.4.1 shown above}$$

$$\Delta_s = (2/3)0.38 + \frac{(17.9 - 7.89)}{(52.5 - 7.89)}\left(5.16 - (2/3)0.38\right) = 1.35\text{in}$$

M_a = 17.9 + 22.5 x 1.35in/12in/ft = 20.4 ft-kip

Now, perform another iteration that includes the new *M_a*

New M_a = 20.4 ft-kip > (2/3)***M_{cr}*** = 7.89 ft-kip

$M_a = M_{sa} + P_s\Delta_s$ with

$$\Delta_s = (2/3)\Delta_{cr} + \frac{(M_a - (2/3)M_{cr})}{(M_n - (2/3)M_{cr})}\left(\Delta_n - (2/3)\Delta_{cr}\right) \text{ in table 11.8.4.1 shown above}$$

$$\Delta_s = (2/3)0.38 + \frac{(20.4 - 7.89)}{(52.5 - 7.89)}\left(5.16 - (2/3)0.38\right) = 1.63\text{in}$$

M_a = 17.9 + 22.5 x 1.63in/12in/ft = 21.0 ft-kip

Again, perform another iteration that includes the new *M_a*

New M_a = 21.0 ft-kip > (2/3)***M_{cr}*** = 7.89 ft-kip

$M_a = M_{sa} + P_s\Delta_s$ with

$$\Delta_s = (2/3)\Delta_{cr} + \frac{(M_a - (2/3)M_{cr})}{(M_n - (2/3)M_{cr})}\left(\Delta_n - (2/3)\Delta_{cr}\right) \text{ in table 11.8.4.1 shown above}$$

$$\Delta_s = (2/3)0.38 + \frac{(21.0 - 7.89)}{(52.5 - 7.89)}\left(5.16 - (2/3)0.38\right) = 1.70\text{in}$$

M_a = 17.9 + 22.5 x 1.70in/12in/ft = 21.1 ft-kip

Because the iteration shows convergence with the last value, no more iterations are required. Thus, final $\Delta_s = (2/3)\Delta_{cr} + \frac{(M_a-(2/3)M_{cr})}{(M_n-(2/3)M_{cr})}(\Delta_n - (2/3)\Delta_{cr})$ = 1.70in. This value is less than the allowable Δ_s of $l_c/150$ = 2.4 in. However, it should be noted that the deflection limit for jambs needs to be watched closely, and may in fact, in some cases, control the design.

Check of closed ties are required per 11.7.4.1. Again, transverse ties are required if A_{st} exceeds $0.01A_g$, where A_{st} is the total amount of longitudinal steel.

A_{st} = 3 x 2 x 0.44 = 2.64 in^2
$0.01A_g$ = 0.01 x 21 x 12 = 2.52 in^2 < A_{st}
Thus, A_{st} > $0.01A_g$, and ties are required. See below.

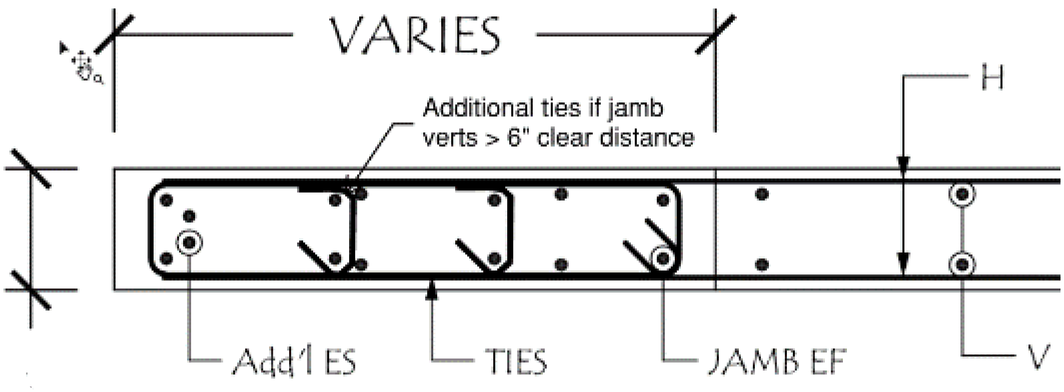


Fig. 100: Transverse ties at a panel region with high axial load

Use ties per ACI 318, Chapter 25. In this case use #3 ties for #6 verticals. With a minimum member size of 9-1/4", use 9" on center spacing. See Figs. 101 through 104 below for ACI guidance from Chapter 25.

25.7.2 *Ties*

25.7.2.1 Ties shall consist of a closed loop of deformed bar with spacing in accordance with (a) and (b):

(a) Clear spacing of at least **(4/3)d_{agg}**
(b) Center-to-center spacing shall not exceed the least of **16d_b** of longitudinal bar, **48d_b** of tie bar, and smallest dimension of member

25.7.2.2 Diameter of tie bar shall be at least (a) or (b):

(a) No. 3 enclosing No. 10 or smaller longitudinal bars
(b) No. 4 enclosing No. 11 or larger longitudinal bars or bundled longitudinal bars

25.7.2.2.1 As an alternative to deformed bars, deformed wire or welded wire reinforcement of equivalent area to that required in 25.7.2.1 shall be permitted subject to the requirements of Table 20.2.2.4a.

25.7.2.3 Rectilinear ties shall be arranged to satisfy (a) and (b):

(a) Every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees
(b) No unsupported bar shall be farther than 6 in. clear on each side along the tie from a laterally supported bar

Fig. 101: ACI 318 tie parameters

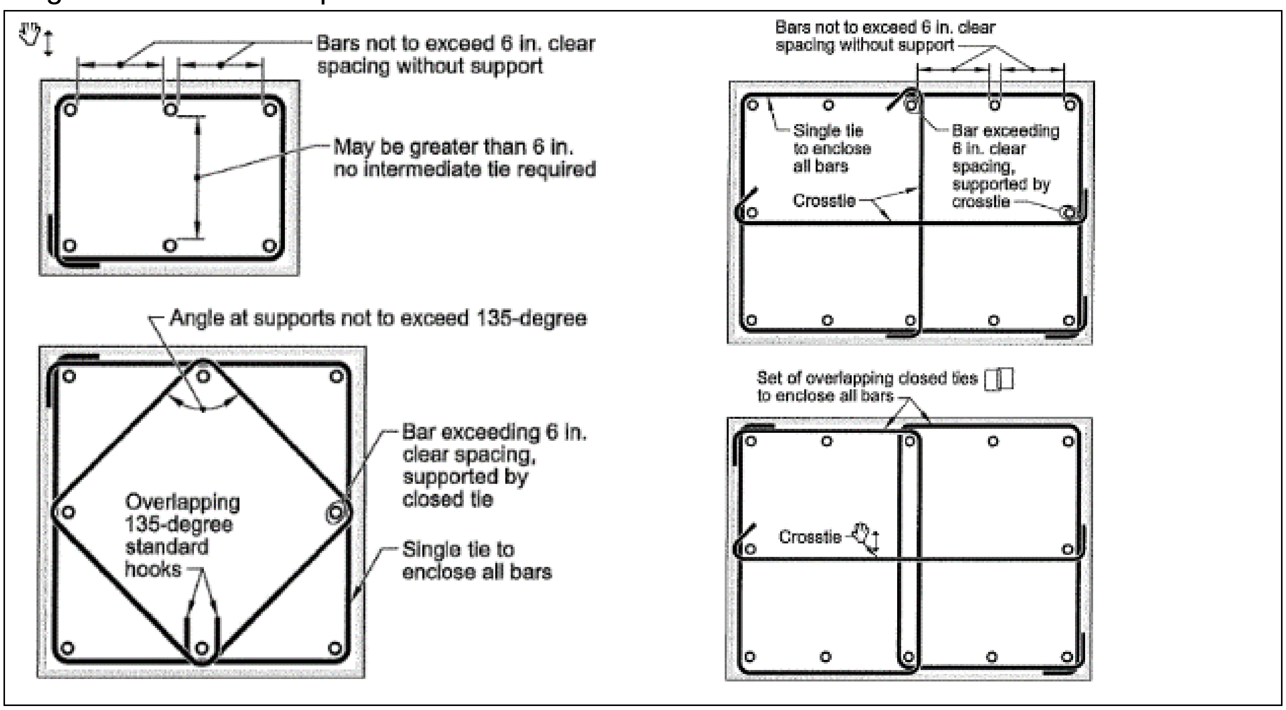


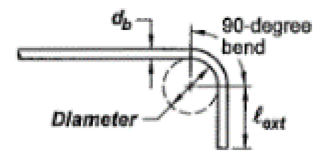
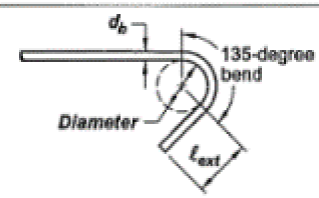
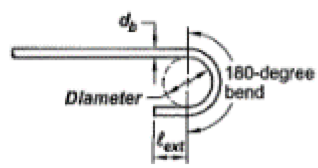
Fig. 102: ACI 318 tie parameters

25.7.2.3.1 Anchorage of rectilinear ties shall be provided by standard hooks that conform to 25.3.2 and engage a longitudinal bar. A tie shall not be made up of interlocking headed deformed bars.

25.3.2 Minimum inside bend diameters for bars used as transverse reinforcement and standard hooks for bars used to anchor stirrups, ties, hoops, and spirals shall conform to Table 25.3.2. Standard hooks shall enclose longitudinal reinforcement.

Fig. 103: ACI 318 tie parameters

Table 25.3.2—Minimum inside bend diameters and standard hook geometry for stirrups, ties, and hoops

Type of standard hook	Bar size	Minimum inside bend diameter, in.	Straight extension[1] ℓ_{ext}, in.	Type of standard hook
90-degree hook	No. 3 through No. 5	$4d_b$	Greater of $6d_b$ and 3 in.	
	No. 6 through No. 8	$6d_b$	$12d_b$	
135-degree hook	No. 3 through No. 5	$4d_b$	Greater of $6d_b$ and 3 in.	
	No. 6 through No. 8	$6d_b$		
180-degree hook	No. 3 through No. 5	$4d_b$	Greater of $4d_b$ and 2.5 in.	
	No. 6 through No. 8	$6d_b$		

[1]A standard hook for stirrups, ties, and hoops includes the specific inside bend diameter and straight extension length. It shall be permitted to use a longer straight extension at the end of a hook. A longer extension shall not be considered to increase the anchorage capacity of the hook.

Fig. 104: ACI 318 ties bends and hooks

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