S.E. Exam Review: Seismic Design

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ASCE | KNOWLEDGE & LEARNING

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ASCE 7-10

Risk Category, Seismic Design Category and Importance Factor

Risk Category

“Risk Category: A categorization of buildings and other structures for determination of flood, snow, ice, and earthquake loads based on the risk associated with unacceptable performance.”

Risk Category

Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads	
Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent low risk to human life in the event of failure.	I
All buildings and other structures except those listed in Risk Categories I, III, and IV.	II
Buildings and other structures, the failure of which could pose a substantial risk to human life. Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure. Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.[a]	III
Buildings and other structures designated as essential facilities. Buildings and other structures, the failure of which could pose a substantial hazard to the community. Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.[a] Buildings and other structures required to maintain the functionality of other Risk Category IV structures.	IV
[a]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.3 that a release of the substances is commensurate with the risk associated with that Risk Category.	

Seismic Design Category

"Seismic Design Category: A classification assigned to a structure based on its Risk Category and the severity of the design earthquake ground motion at the site as defined in Section 11.4."

Importance Factor (I_e)

Importance Factor: "A factor that accounts for the degree of risk to human life, health, and welfare associated with damage to property or loss of use or functionality."

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads[a]

Risk Category from Table 1.5-1	Snow Importance Factor I_s	Ice Importance Factor – Thickness, I_t	Ice importance Factor – Wind, I_w	Seismic Importance Factor, I_s
I	0.8	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.25	1.00	1.25
IV	1.20	1.25	1.00	1.50

[a]The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it depends on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

Value of S_{DS}	Risk Category	
	I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \le S_{DS} < 0.33$	B	C
$0.33 \le S_{DS} < 0.50$	C	D
$0.50 \le S_{DS}$	D	D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

Value of S_{D1}	Risk Category	
	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \le S_{D1} < 0.133$	B	C
$0.133 \le S_{D1} < 0.20$	C	D
$0.20 \le S_{D1}$	D	D

System Overstrength

System Overstrength

System overstrength is the ratio of the lateral forces that cause the formation of a full yield mechanism in the structure to the lateral forces that cause first yielding. It is represented by the symbol Ω_o.

System Overstrength

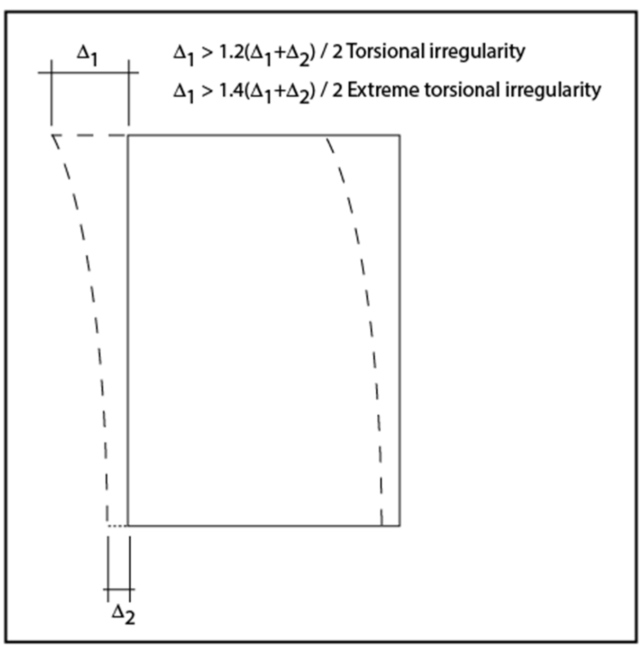
ASCE 7-10 Sections 12.2.5.2 (cantilever column systems), 12.3.3.3 (elements supporting discontinuous wall or frames), 12.10.2.1 (structures in SDC *C*, *D*, *E* or *F*, and collector elements and their connections), 12.13.6.4 (batter piles), and 12.13.6.5 (pile anchorage) require compliance with provisions of Section 12.4.3.

Irregularities

Horizontal Structural Irregularity

Table 12.3-1 ASCE 7-10	
Type	**Irregularity Type and Description**
1a	**Torsional irregularity** – maximum story drift, computed including accidental torsion with A_x = 1.0, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semi-rigid.
1b	**Extreme Torsional irregularity** – maximum story drift, computed including accidental torsion with A_x = 1.0, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semi-rigid.

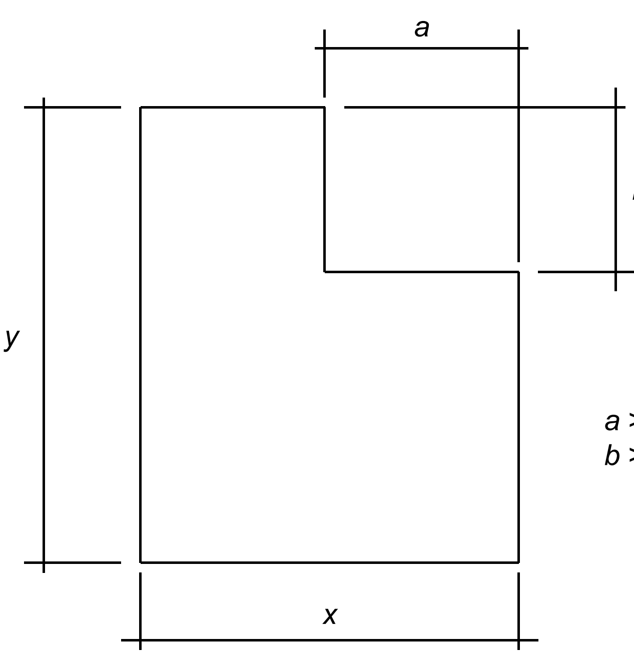
Torsional Irregularity (H-1a or H-1b)



Horizontal Structural Irregularity

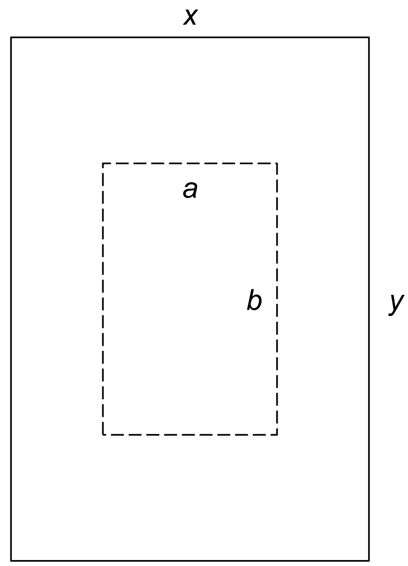
Table 12.3-1 ASCE 7-10	
Type	**Irregularity Type and Description**
2	**Reentrant Corner Irregularity** – both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.
3	**Diaphragm Discontinuity Irregularity** – there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one having a cutout or open area greater than 50% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next.

Reentrant Corner Irregularity (H-2)



$a > 0.15x$
$b > 0.15y$

Diaphragm Discontinuity (H-3)

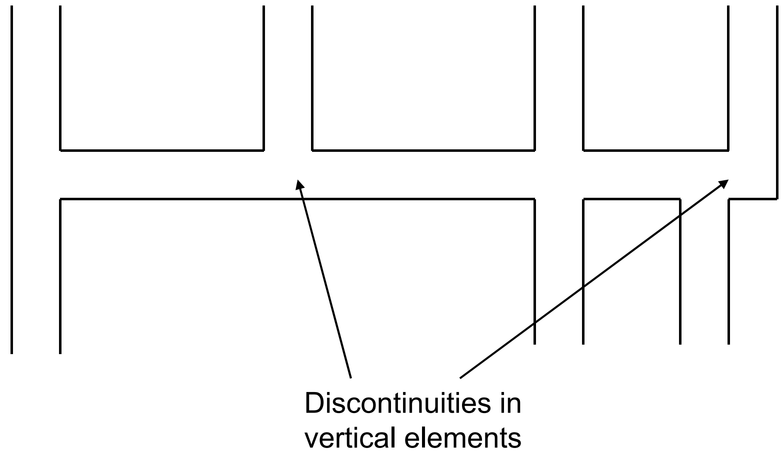


$ab > 0.5xy$

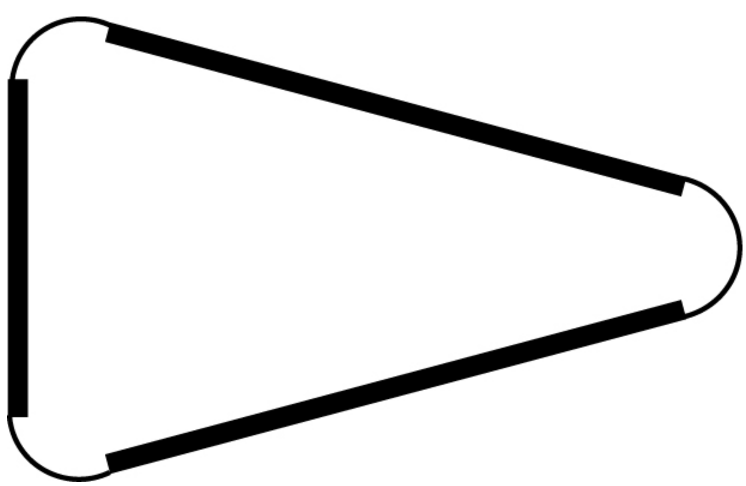
Horizontal Structural Irregularity

Table 12.3-1 ASCE 7-10	
Type	**Irregularity Type and Description**
4	**Out-of-Plane Offsets Irregularity** – there is a discontinuity in a lateral force-resistance path, such as an out-of-plane offset of at least one of the vertical elements.
5	**Nonparallel Systems-Irregularity** – vertical lateral force-resisting elements are not parallel to the major orthogonal axes of the seismic force-resisting system.

Out-of-Plane Offset (H-4)



Non-parallel Shear Walls (H-5)



Vertical Structural Irregularity

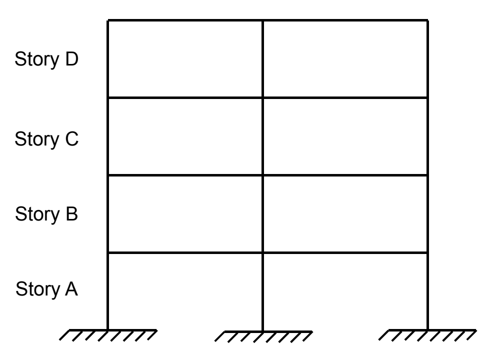
ASCE 7-10 Table 12.3-2	
Type	**Irregularity Type and Description**
1a	**Stiffness-Soft Story Irregularity** –there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.
1b	**Stiffness-Extreme Soft Story Irregularity** – there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.

Stiffness-Soft Story Irregularity (V-1a or V-1b)

Stiffness-Soft Story Irregularity

The stiffness of story A is less than 0.70 times the stiffness of Story B, or,

The stiffness of Story A is less than 0.80 times the average stiffness of Stories B, C and D.



Stiffness-Extreme Soft Story Irregularity

The stiffness of Story A is less than 0.60 times the stiffness of story B, or,

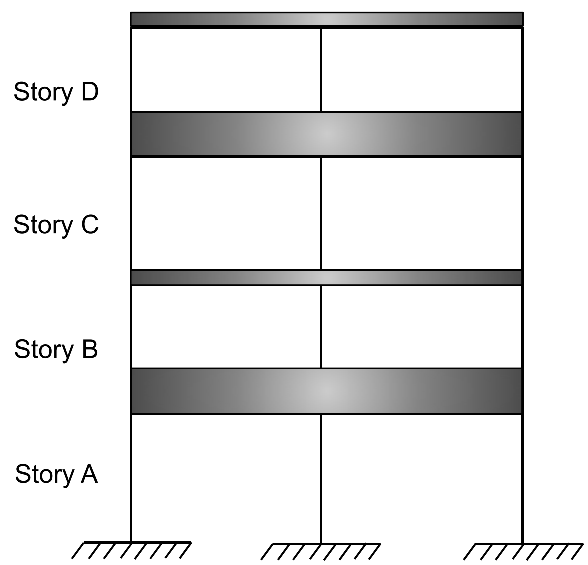
The stiffness of Story A is less than 0.70 times the average stiffness of Stories B, C, and D.

Vertical Structural Irregularity

ASCE 7-10 Table 12.3-2	
Type	**Irregularity Type and Description**
2	**Weight (Mass) Irregularity** – the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.
3	**Vertical Geometric Irregularity** – the horizontal dimension of the seismic force-resisting system in any story is more than 130% of that in an adjacent story.

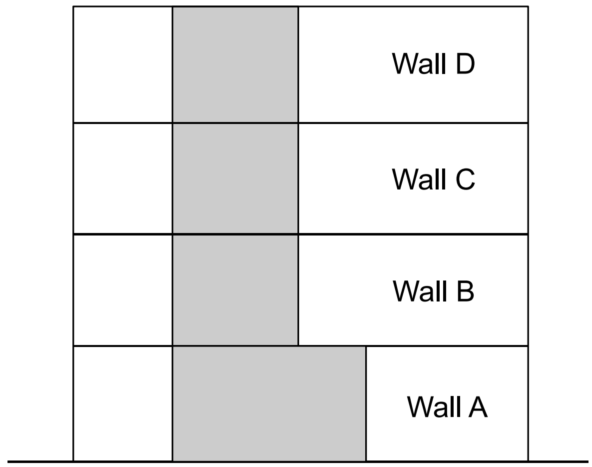
Mass Irregularity (V-2)

Mass on Story B or Story D is more than 1.5 times the mass on Story C.



Vertical Geometric Irregularity (V-3)

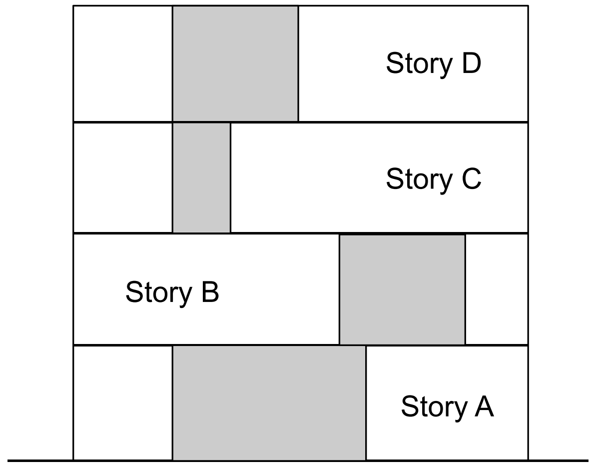
Length of Wall A is more than 1.3 times the length of Wall B.



Vertical Structural Irregularity

ASCE 7-10 Table 12.3-2	
Type	**Irregularity Type and Description**
4	**In-Plane Discontinuity in Vertical Lateral Force – Resisting Element Irregularity** – there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on a supporting structural element.
5a	**Discontinuity in Lateral Strength – Weak Story Irregularity** – the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.
5b	**Discontinuity in Lateral Strength – Extreme Weak Story Irregularity** – the story lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.

In-plane Discontinuity in Vertical Lateral Force Resisting Element (V-4)



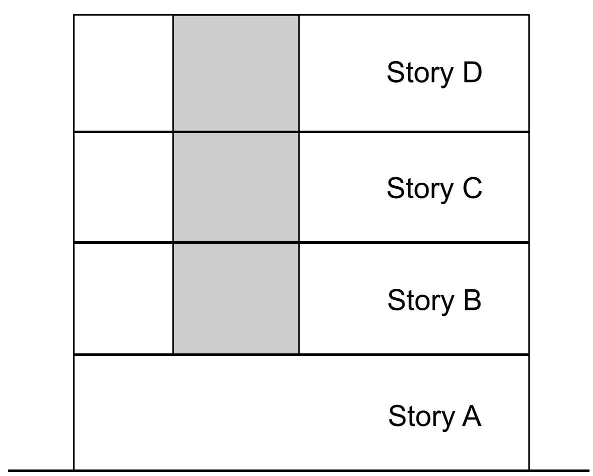
Weak Story Irregularity (V-5)

Weak Story Irregularity:

Lateral Strength of Story A is less than 0.80 of the lateral strength of Story B.

Extreme Weak Story Irregularity:

Lateral strength of Story A is less than 0.65 of the lateral strength of Story B.



Limitations and Additional Requirements for Systems with Irregularities (ASCE 7-10 Section 12.3.3)

- Structures assigned to SDC *E* or *F*, with Type H-1b or Type V-1b, V-5a or V-5b irregularities are not permitted
- Structures assigned to SDC *D* with Type V-5b irregularity are not permitted
- Structures with Type V-5b irregularity shall not be over two stories or 30 feet in height

Limitations and Additional Requirements for Systems with Irregularities (ASCE 7-10 Section 12.3.3)

- Structural elements supporting discontinuous walls or frames of structures having H-4 or V-4 irregularity shall be designed to resist the seismic load effects including overstrength factor (Ω_o)

Limitations and Additional Requirements for Systems with Irregularities (ASCE 7-10 Section 12.3.3)

- Structures assigned to SDC *D*, *E* or *F* having Types H-1a, H-1b, H-2, H-3 or H-4 irregularities or Type V-4 irregularity, the design forces determined from Section 12.10.1.1 shall be increased by 25 percent for following elements of the seismic force-resisting system:

Limitations and Additional Requirements for Systems with Irregularities (ASCE 7-10 Section 12.3.3)

1. Connections of diaphragms to vertical elements and to collectors.
2. Collectors and their connections, including connections to vertical elements, of the seismic force-resisting system.

Forces calculated using the seismic load effects including overstrength factor of Section 12.4.3 need not be increased.

Redundancy Factor

Redundancy Factor

The value of the redundancy factor (ρ) is taken as 1 under any of the following conditions:

1. Structures assigned to SDC B or C.
2. Drift calculation and P-Δ effects.
3. Design of nonstructural components.
4. Design of nonbuilding structures that are not similar to buildings.

Redundancy Factor

5. Design of collector elements, splices, and their connections for which the load combinations with overstrength factor of Section 12.4.3 are used.
6. Design of members or connections where the load combinations with overstrength of Section 12.4.3 are required for design.
7. Diaphragm loads determined using Eq. 12.10-1 (can be both 1 and 1.3 for same diaphragm – transfer forces).

Redundancy Factor

8. Structures with damping systems designed in accordance with Section 18.
9. Design of structural walls for out-of-plane forces, including their anchorage.

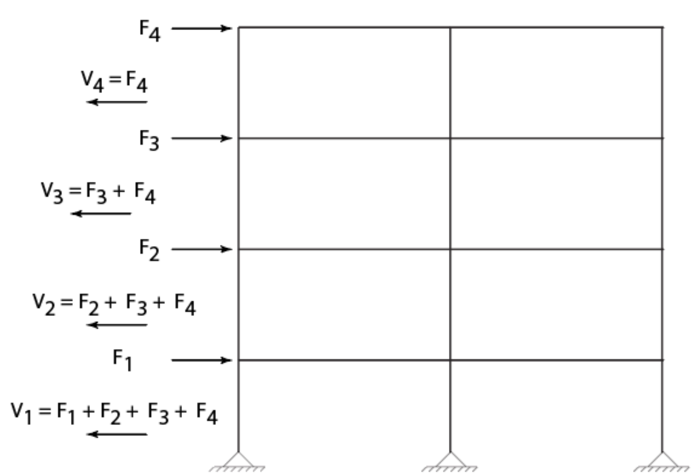
Redundancy Factor

For structures assigned to SDC D having an extreme torsional irregularity (H1-b) shall have *ρ* = 1.3.

For structures assigned to SDC D and structures assigned to SDC E, or F, *ρ* shall equal 1.3 unless *one* of two conditions is met, whereby *ρ* is permitted to be taken as 1.0.

Redundancy Factor

1. Each story resisting more than 35 percent of the base shear in the direction of interest shall comply with Table 12.3-3.



ASCE 7-10 Table 12.3-3

Lateral Force-Resisting Element	Requirement
Braced frames	Removal of an individual brace, or connection thereto, would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type H-1b).
Moment frames	Loss of moment resistance at the beam-to-column connections at both ends of a single beam would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).

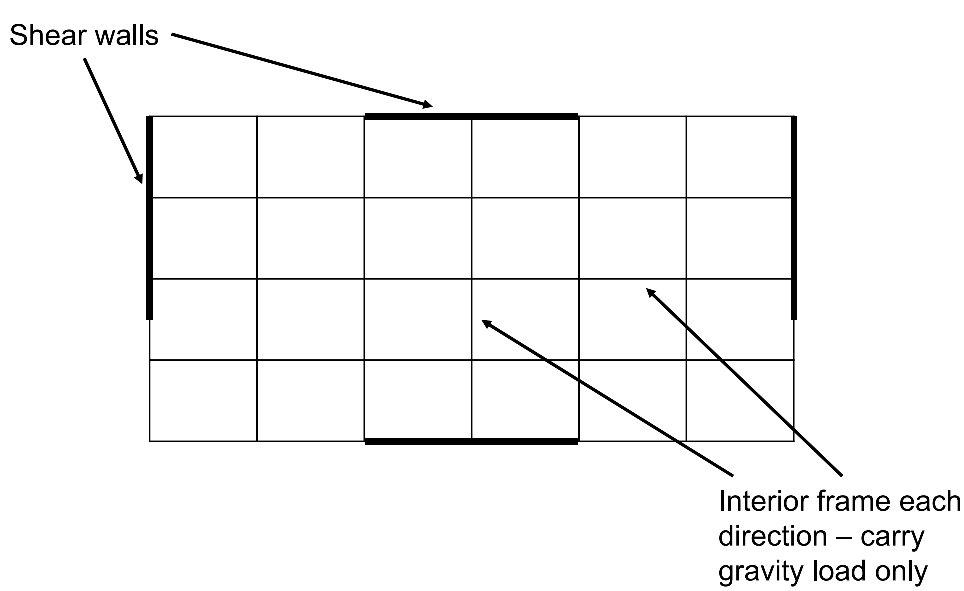
ASCE 7-10 Table 12.3-3

Lateral Force-Resisting Element	Requirement
Shear Walls or Wall Piers with a height-to length ratio of greater than 1.0	Removal of a shear wall or wall pier with a height-to-length ratio greater than 1.0 within any story, or collector connections thereto, would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).
Cantilever columns	Loss of moment resistance at the base connections of any single cantilever column would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).
Other	No requirements

Redundancy Factor

2. Structures that are regular in plan at all levels provided that the seismic force–resisting systems consist of at least two bays of seismic force–resisting perimeter framing on each side of the structure in each orthogonal direction at each story resisting more than 35 percent of the base shear. The number of bays for a shear wall shall be calculated as the length of shear wall divided by the story height or two times the length of shear wall divided by the story height for light-framed construction.

Example Building where ρ = 1



Seismic Load Combinations

Load Combinations

Load combinations involving *W* or *E* require special attention

In the context of a plane frame, wind load can act from right to left or left to right

In the context of a plane frame, earthquake load can act horizontally from right to left or left to right and vertically up or down

Earthquake Load

$E = E_h + E_v$ for use with LC-5

$E = E_h - E_v$ for use with LC-7

$E_h = \rho Q_E$

$Q_E \rightarrow$ horizontal earthquake effect

$\rho \rightarrow$ redundancy factor whose value depends on lateral load resisting system, = 1 or 1.3

$E_v \rightarrow$ vertical load effect $= 0.2S_{DS}D$

Earthquake Load

ASCE 7-10 Section 12.4.3

When use of system overstrength factor is required:

$E = E_m = E_{mh} + E_v$ for use with LC-5

$E = E_m = E_{mh} - E_v$ for use with LC-7

$E_{mh} = \Omega_o Q_E$ (Ω_o ASCE 7 Table 12.2-1)

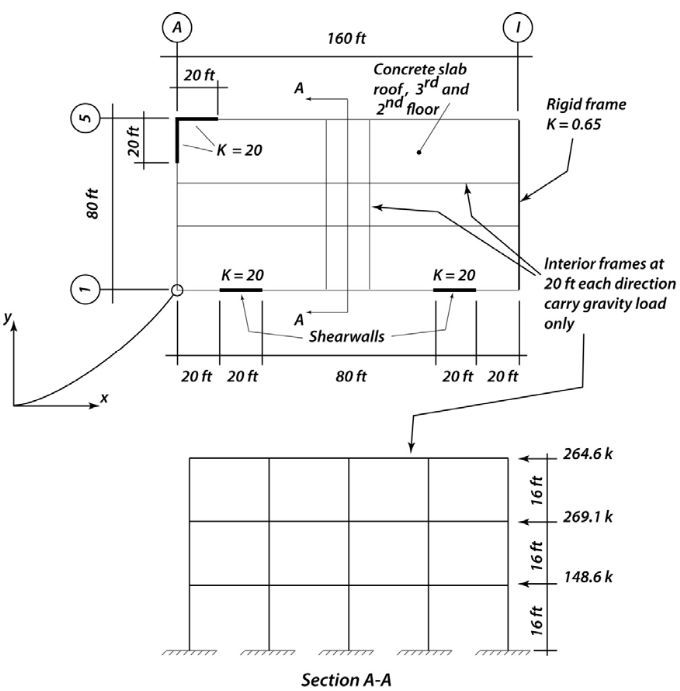
$\Omega_o \rightarrow$ system overstrength factor

$Q_E \rightarrow$ horizontal earthquake effect

$E_v \rightarrow$ vertical load effect $= 0.2S_{DS}D$

Shear Wall Stiffness

Building 2



Shear Wall Stiffness

$K = \frac{Et}{4\left(\frac{h}{l}\right)^3 + 3\left(\frac{h}{l}\right)}$ (cantilever)

$K = \frac{Et}{\left(\frac{h}{l}\right)^3 + 3\left(\frac{h}{l}\right)}$ (fixed-fixed)

$l \rightarrow$ length of the wall in the direction being considered

$h \rightarrow$ height of the wall at the level being considered

$E \rightarrow$ modulus of elasticity

$t \rightarrow$ wall thickness

The definition of stiffness here is the force required to deflect the wall a unit distance at the level h. The first term in the expression in the denominator is related to the flexural rigidity of the wall and the second term is related to its shear rigidity.

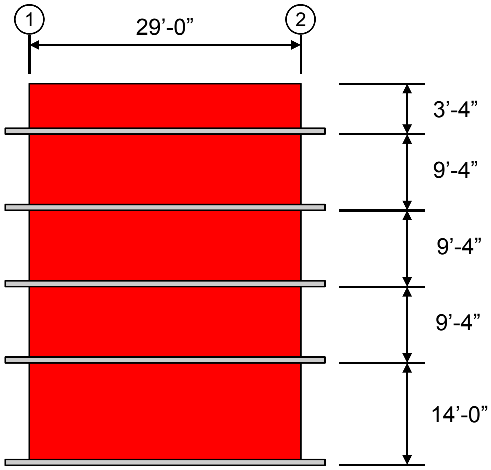
Shear Wall Stiffness

For walls having the same modulus and thickness, a relative value is usually sufficient:

$$K = \frac{1}{4\left(\frac{h}{l}\right)^3 + 3\left(\frac{h}{l}\right)}$$

$$K = \frac{1}{\left(\frac{h}{l}\right)^3 + 3\left(\frac{h}{l}\right)}$$

Shear Wall Stiffness



Level	Relative Stiffness	
Second	0.5268	8.865
Third	0.2224	3.667
Fourth	0.1010	1.813
Roof	0.0607	1

The relative stiffness at the roof level, just below the 3'-4" parapet is:

$$h = 14.0 \text{ ft} + 3(9.333 \text{ ft}) = 42 \text{ ft}$$

$$l = 29 \text{ ft}$$

$$K = \frac{1}{4\left(\frac{42 \text{ ft}}{29 \text{ ft}}\right)^3 + 3\left(\frac{42 \text{ ft}}{29 \text{ ft}}\right)} = 0.0607$$

Because the values are relative, they may be adjusted to any set of values that might be more convenient with which to work, as has been done in the last column of the table.

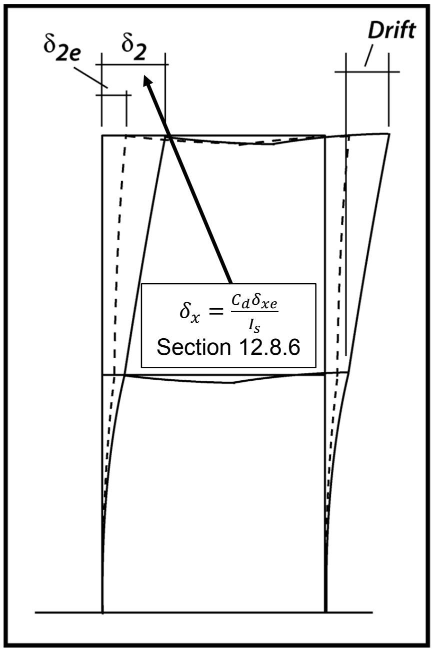
Methods of Analysis

Methods of Analysis

- Equivalent lateral force procedure – ASCE 7-10 Section 12.8
- Modal response spectrum analysis – ASCE 7-10 Section 12.9
- Seismic response history – ASCE 7-10 Chapter 16
- Simplified alternate structural design for simple bearing wall or building frame systems – ASCE 7-10 Section 12.14 (not permitted in some jurisdictions)

Drift

Drift



Drift is calculated based on the respective displacements of the center of mass of the stories.

Drift

- If center of mass of stories align, calculate drift based on displacement of centers of mass
- Where centers of mass do not align vertically, it is permitted to compute the deflection at the bottom of the story based on the vertical projection of the center of mass at the top of the story.

Drift

- For structures assigned to Seismic Design Category C, D, E, or F having horizontal irregularity Type H-1a or H-1b, the design story drift, Δ, shall be computed as the largest difference of the deflections of vertically aligned points at the top and bottom of the story under consideration along any of the edges of the structure.

Allowable Drift - Δ_a

ASCE 7-10 Table 12.12-1			
Structure	**Risk Category I or II**	**Risk Category III**	**Risk Category IV**
Structures, other than masonry shear wall structures, four stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts	$0.025h_{sx}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$
h_{sx} – story height below level x			

Section 12.1.1 - For seismic force-resisting systems solely comprising moment frames in structures assigned to Seismic Design Categories D, E, or F, the design story drift (Δ) shall not exceed Δ_a/ρ for any story.

Drift Example

Minimum Base Shear for Computing Drift

ASCE 7-10 section 12.8.6.1 states:

“The elastic analysis of the seismic force-resisting system for computing drift shall be made using the prescribed seismic design forces of Section 12.8.”

It is not required to consider Eq. 12.8-5 when computing drift.

$$0.044 S_{DS} I_s \geq 0.01 \qquad \text{(Eq. 12.8-5)}$$

Drift Example

Story deflections for Building 2 are calculated using elastic analysis. The elastic displacement calculated under strength-level design earthquake forces at the roof-level deflection is 1.07 in. and at the third story level is 0.61 in. Determine if the corresponding roof drift complies with the limitation of the ASCE 7-10 provisions.

Drift Example

$\delta_x = \frac{C_d \delta_{xe}}{I_s}$

$C_d = 5$

$\Delta_2 = \frac{(\delta_{x2} - \delta_{x1}) C_d}{I_s} = \frac{5(1.07 \text{ in} - 0.61 \text{ in})}{1} = 2.30 \text{ in}$

Using Risk Category II

Drift limit at roof level $= 0.020 h_{sx} =$

$$0.020(16 \text{ ft})(12 \text{ in/ft}) = 3.84 \text{ in} > 2.30 \text{ in}$$

The drift is within limits

If the structure is designed to accommodate drift, the limit would be $0.025 h_{sx}$

Diaphragms and Collectors

Prescriptively Flexible Diaphragms

- Flexible diaphragms – Diaphragms constructed of untopped steel decking and wood structural panels are permitted to be idealized as flexible if any of the following conditions exist:
    a. In structures where the vertical elements are steel braced frames; steel and concrete composite braced frames; or concrete, masonry, steel, or steel and concrete composite shear walls.

Prescriptively Flexible Diaphragms

b. In one- and two-family dwellings.

c. In structures of light-frame construction where all of the following conditions are met:

   1. Topping of concrete or similar materials is not placed over wood structural panel diaphragms except for nonstructural topping no greater than 1 1/2 in. (38mm) thick.

Prescriptively Flexible Diaphragms

   2. Each line of vertical elements of the seismic force resisting system complies with the allowable story drift of Table 12.12-1.

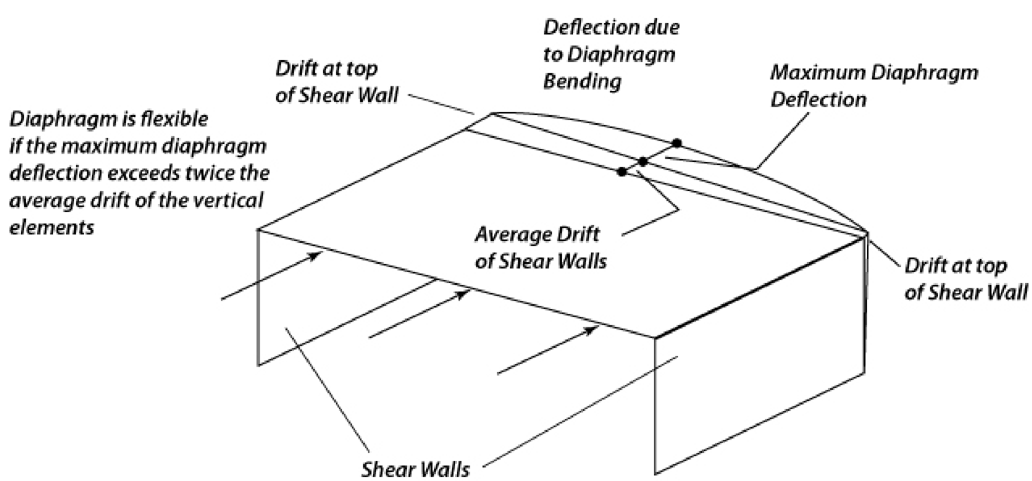
Table 12.12-1 Allowable Story Drift, Δ_a[a,b]

Structure	Risk Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$[c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

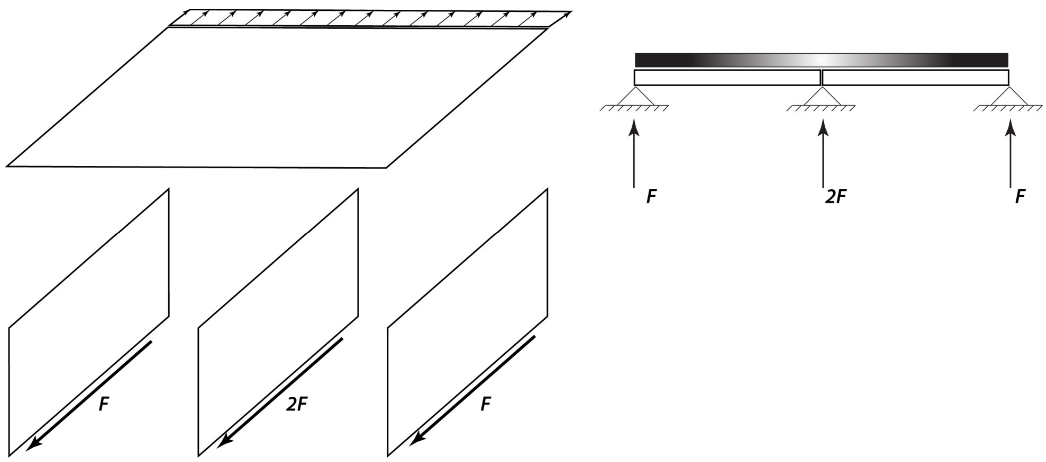
Prescriptively Rigid Diaphragms

- Rigid diaphragms – diaphragms of concrete slabs or concrete filled metal deck with span-to-depth ratio of three or less, in structures that have no horizontal irregularities, are permitted to be idealized as rigid

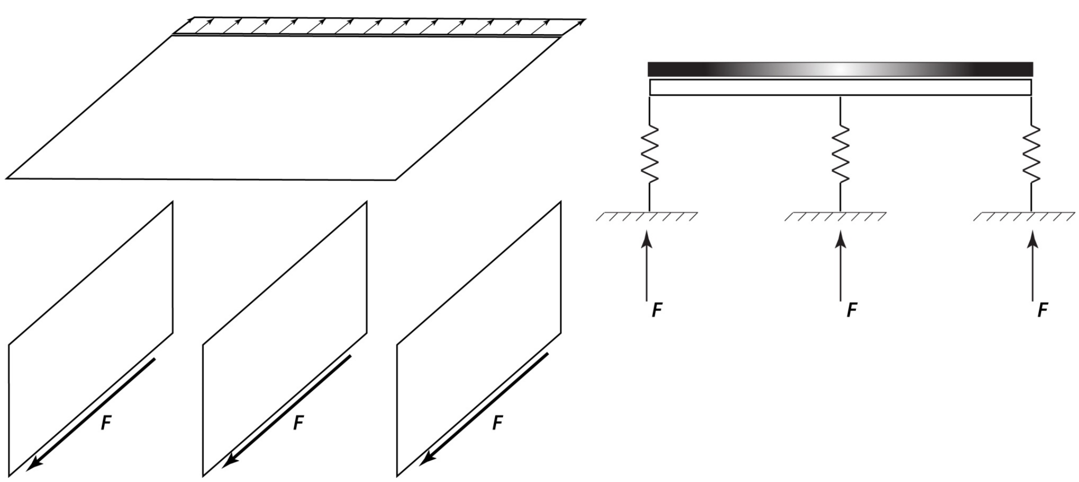
Diaphragm Flexibility by Calculation



Flexible Diaphragm



Rigid Diaphragm



ASCE 7-10 12.3.1 – Diaphragm Flexibility

“The structural analysis shall consider the relative stiffnesses of diaphragms and the vertical elements of the seismic force-resisting system. Unless a diaphragm can be idealized as either flexible or rigid in accordance with Sections 12.3.1.1, 12.3.1.2, or 12.3.1.3, the structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e., semirigid modeling assumption).”

IBC 2015 1604.4 Analysis

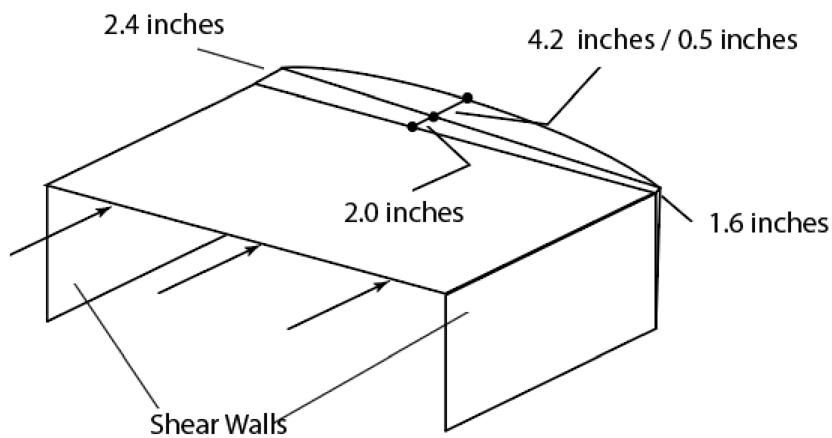
“ . . for in the design. ~~Except where diaphragms are flexible,~~ A diaphragm is rigid for the purpose of distribution of story shear and torsional moment when the lateral deformation of the diaphragm is less than or ~~are permitted~~ equal to ~~be analyzed as flexible,~~ two times the average story drift. Where required by ASCE 7, 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 the lateral forces and the center of rigidity of the lateral force-resisting system.”

Diaphragm Stiffness Classification Example

Diaphragm Stiffness Classification Example

A diaphragm that cannot be classified as prescriptively rigid or prescriptively flexible is supported by two concrete shear walls. Under the action of seismic load one diaphragm drifts 1.6 inches and the other diaphragm drifts 2.4 inches. If the maximum diaphragm displacement is 4.2 inches, classify it as either flexible, rigid or semi-rigid, and then determine how it must be modeled in an analysis if one is following ASCE, IBC 2012 or IBC 2015. Repeat the problem if the maximum diaphragm displacement is 0.5 inches.

Diaphragm Stiffness Classification Example



Diaphragm Stiffness Classification Example

$$\frac{1.6+2.4}{2} = 2 \text{ inches}$$

4.2 inches > (2)(2 inches) = 4 inches

Diaphragm is classified as flexible

For all three codes it is modeled as flexible

Diaphragm Stiffness Classification Example

$$\frac{1.6+2.4}{2} = 2 \text{ inches}$$

0.5 inches < (2)(2 inches) = 4 inches

Diaphragm is not flexible

For ASCE 7 and IBC 2012 it is modeled as semi-rigid

For IBC 2015 it is modeled as rigid

Diaphragm Design

According to ASCE 7 Section 12.4.3.1, diaphragm design forces are earthquake load effects Q_E

Diaphragms – ASCE 7-10 Section 12.10.1

- Design for both bending and shear resulting from design forces
- At discontinuities (for example, openings, reentrant corners), diaphragm must be capable of dissipation or transfer of edge (chord) forces, as well as other applicable forces

Diaphragms Design Forces – ASCE 7-10 Section 12.10.11

Diaphragm forces to be resisted are those indicated by analysis, but not less than…

$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$ (Eq. 12.10-1)

F_{px} → diaphragm design force at level *x*

F_i → design force at level *i*

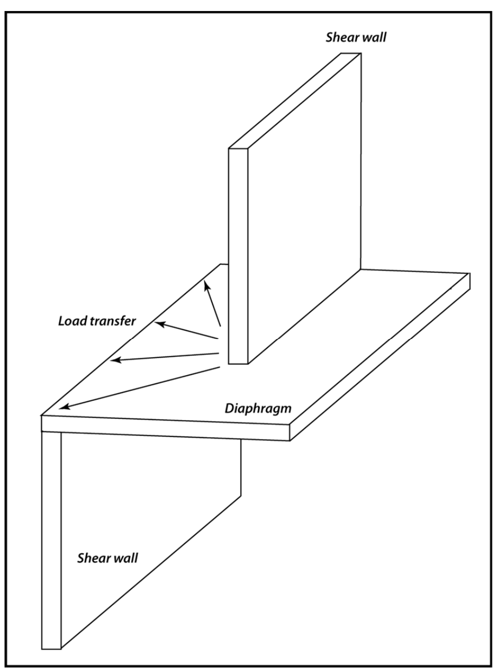
w_i → weight tributary to level *i*

w_{px} → weight tributary to the diaphragm at level *x*

Diaphragms Design Forces – ASCE 7-10 Section 12.10.11

- The diaphragm design force need not exceed $0.4S_{DS}I_s w_{px}$ (Eq. 12.10-2)
- The diaphragm design force can not be less than $0.2S_{DS}I_s w_{px}$ (Eq. 12.10-3)
- Transfer forces due to offsets must be added
- Redundancy factor (ρ) applies to design of diaphragm for structures assigned to SDC *D*, *E*, *F*

Transfer Forces



Diaphragms Design Forces – ASCE 7-10 Section 12.10.11

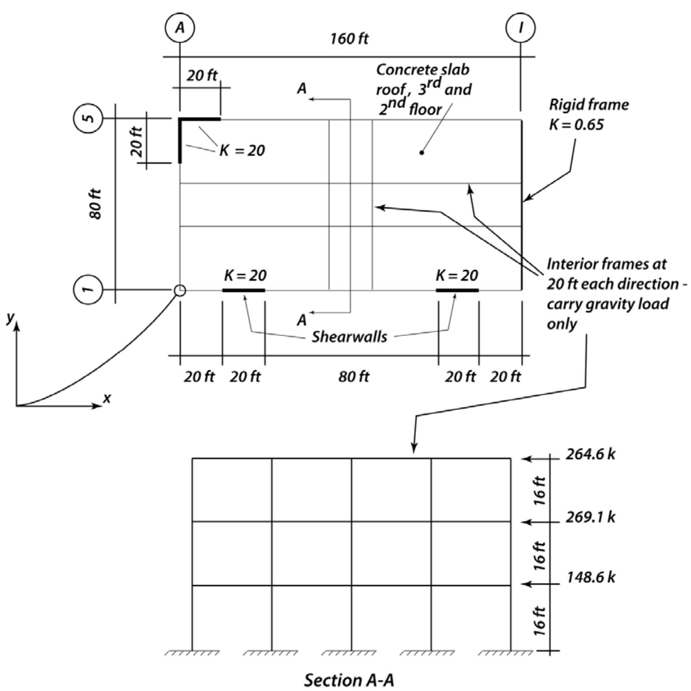
- ρ applies to design of diaphragms in structures assigned to SDC *D*, *E* and *F*
- ρ =1 for inertial forces calculated using Equation 12.10-1
- ρ = same as structure for transfer forces using Equation 12.10-1
- Requirements apply to structures having irregularities defined in Section 12.3.3.4 also apply

Diaphragm Design Force Example

Diaphragm Design Force Example

For Building 2, determine the *minimum* force to be used for design of the diaphragm on the third floor. Investigate load combination 5 and assign the building to SDC *D*.

Building 2



Diaphragm Design Force Example

LC 5

$E = \rho Q_E$

$\rho = 1$ (inertial force)

Loads assigned to roof and 3rd floor are 264.6 k and 269.1 k, respectively

$\rho Q_E = 1(269.1\ \mathrm{k} + 264.6\ \mathrm{k}) = 533.7\ \mathrm{k}$

Diaphragm Design Force Example

$F_{p3} = \frac{\sum_{i=3}^{r} F_i}{\sum_{i=3}^{r} w_i} w_{p3}$

$w_{p3} = (90\ \mathrm{psf})(160\ \mathrm{ft})(80\ \mathrm{ft}) + 89.6\ \mathrm{k} = 1{,}240\ \mathrm{k}$

$\sum_{i=3}^{r} w_i = (60\ \mathrm{psf} + 90\ \mathrm{psf})(160\ \mathrm{ft})(80\ \mathrm{ft}) + 89.6\ \mathrm{k} + 44.8\ \mathrm{k}$

$= 2{,}050\ \mathrm{k}$

$F_{p3} = \frac{533.7\ \mathrm{k}}{2{,}050\ \mathrm{k}}(1{,}240\ \mathrm{k}) = 322.8\ \mathrm{k}$

Diaphragm Design Force Example

$$F_{p3} \leq 0.4 S_{DS} I_e w_{p3} = 0.4(1.0)(1.0)(1{,}240 \text{ k})$$

$$= 496 \text{ k}; \ 322.8 \text{ k} < 496 \text{ k OK}$$

$$F_{p3} \geq 0.2 S_{DS} I_e w_{p3} = 0.2(1.0)(1.0)(1{,}240 \text{ k})$$

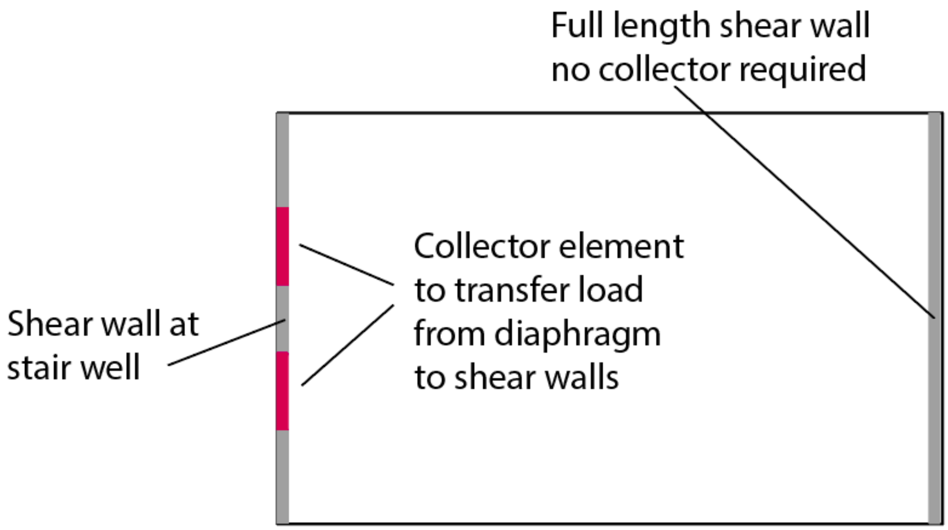
$$= 248 \text{ k}; \ 322.8 \text{ k} > 248 \text{ k OK}$$

The force for which the diaphragm must be designed is that determined by analysis but not less than 322.8 k.

If this building has an irregularity of type H-1a, -1b, -2, -3, -4 or V-4, its connection to vertical elements or collectors forces must be increased by 25%.

Collector Design

Collector Elements



Collectors – ASCE 7-10 Section 12.10.2

- Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces.
- In structures assigned to SDC *C* through *F*, collector elements and their connections, including connections to vertical elements, shall be designed to resist the maximum of the following…

Collectors – ASCE 7-10 Section 12.10.2

1. Forces calculated using the seismic load effects including Ω_0, with seismic forces determined by the equivalent lateral force procedure or the modal response spectrum analysis. (That is, forces determined from the overall building analysis under the design base shear V amplified by the overstrength factor, or $\Omega_0 Q_E$).

Collectors – ASCE 7-10 Section 12.10.2

2. Forces calculated using the seismic load effects including Ω_0, with seismic forces determined by Equation 12.10-1. (That is, forces determined from ASCE 7-10 Eq. (12.10-1), the diaphragm design force at floor level x, F_{px} amplified by Ω_0, or $\Omega_0 F_{px}$).

Collectors – ASCE 7-10 Section 12.10.2

3. Forces calculated using the load combinations of Section 12.4.2.3 with seismic forces determined by Equation 12.10-2. (That is, forces determined from ASCE 7-10 Eq. (12.10-2), the minimum value of F_{px} that can be used in design, or $F_{px,min}$, without any overstrength factor).

Collectors – ASCE 7-10 Section 12.10.2

- The maximum collector forces obtained from the three previous items need not exceed Q_E obtained from Eq. 12.10-3. (That is, $F_{px,max}$ without the overstrength factor).
- Transfer forces as described in Section 12.10.1.1 shall be considered.

Design for Out-of-Plane Forces

Design for Out-of-Plane Forces

- Section 1.4.5 – Anchorage of Structural Walls
- Applies to all walls in all seismic design categories
- Basic structural integrity
- SDC A
- ρ is 1 for design of walls for out-of-plane forces, including anchorages

Section 1.4.5

- For walls that provide for vertical load bearing or lateral shear resistance
- Applies to all walls, not just concrete and masonry walls.
- Use 0.2 times the weight of wall tributary to connection, but not less than 5 psf.

Design for Out-of-Plane Forces

- ASCE 7-10 Section 12.11 – Structural Walls and their Anchorage
- Section 12.11.1 – Design for Out-of-Plane Forces
- Section 12.11.2 – Anchorage of Structural Walls and Transfer of Design Forces into Diaphragms
- Section 12.11.2.1 – Wall Anchorage Forces
- Section 12.11.2.2 – Additional requirements for diaphragms in structures assigned to SDC *C* through *F*

Design for Out-of-Plane Forces

Section 12.11.1 - Structural walls and their anchorage shall be designed for a force normal to the surface equal to:

$$F_p = 0.4 S_{DS} I_s W_p$$

with a minimum force of 10% of the weight of the structural wall.

Design for Out-of-Plane Forces

Section 12.11.2.1 – The anchorage of structural walls to supporting construction shall provide a direct connection capable of resisting the following:

$0.4 S_{DS} k_a I_e W_p$ (ASCE 7-10 12.11-1)

But not less than

$0.2 k_a I_e W_p$

$k_a = 1.0 + \frac{L_f}{100}$ (ASCE 7-10 12.11-2)

$W_p \rightarrow$ weight of the wall tributary to the anchor

Design for Out-of-Plane Forces

L_f = 0 for rigid diaphragms

$L_f \longrightarrow$ the span, in feet, of a flexible diaphragm that provides the lateral support for the wall; the span is measured between vertical elements that provide lateral support to the diaphragm in the direction considered

ASCE | KNOWLEDGE & LEARNING

Design for Out-of-Plane Forces

Where the anchorage is not located at the roof and all diaphragms are not flexible, the value from Eq. 12.11-1 is permitted to be multiplied by the factor:

$$\frac{(1+2z/h)}{3}$$

Where:

$z \rightarrow$ height of the anchor above the base of the structure

$h \rightarrow$ height of the roof above the base

Walls must be designed resist bending between anchors when anchor spacing exceeds 4 feet

ASCE | KNOWLEDGE & LEARNING

OOP Wall Example

Determine the design force acting on an 8-inch masonry wall, which has a weight of approximately 70 psf of wall surface. Then determine the minimum anchor force required to connect the wall to the roof and to one of the supporting floors. The wall is part of a building where it has been determined that S_{DS} = 1 and I_s = 1. The story-to-story height is 12 feet and the wall will be anchored horizontally to the floors at 6 foot intervals. Assume the diaphragms are rigid.

OOP Wall Example

Design wall pressure

$0.4S_{DS}I_eW_p = 0.4(1)(1)(70\text{ psf}) =$

28 psf ← controls

$0.1(70\text{ psf}) = 7\text{ psf}$

Because anchors are spaced at a distance greater than four feet, the wall must be designed to resist bending between the anchors (not done in this example). Anchorages must be designed for a minimum force consistent with 28 psf acting on the wall.

OOP Wall Example

Design anchor loads:

Minimum load based on wall pressure at roof

$$(6\text{ ft})\left(\frac{12\text{ ft}}{2}\right) = 36\text{ ft}^2$$

$$(36\text{ ft}^2)(28\text{ psf}) = 1{,}008\text{ lb}$$

At floor

$$(6\text{ ft})(12\text{ ft}) = 72\text{ ft}^2$$

$$(72\text{ ft}^2)(28\text{ psf}) = 2{,}016\text{ lb}$$

OOP Wall Example

$$L_f = 0$$

$$k_a = 1.0 + \frac{0}{100} = 1.0$$

At roof

$$W_p = (36\text{ ft}^2)(70\text{ psf}) = 2{,}520\text{ lb}$$

$0.4(1)(1.0)(1)(2{,}520\text{ lb}) = 1{,}008\text{ lb} \leftarrow$ controls

$$> 0.2(1.0)(1)(2{,}520\text{ lb}) = 504\text{ lb}$$

OOP Wall Example

At floor

$W_p = (72\ \text{ft}^2)(70\ \text{psf}) = 5{,}040\ \text{lb}$

$0.4(1)(1.0)(1)(5{,}040\ \text{lb}) = 2{,}016\ \text{lb}$ ← controls

$> 0.2(1.0)(1)(5{,}040\ \text{lb}) = 1{,}080\ \text{lb}$

At floor levels the value of 2,016 lb could be reduced according to $\frac{(1+2z/h)}{3}$, but the required load based on wall pressure is 2,016 lb and reduction is not possible

Seismic Design of Extended End Plate Connections

Seismic Steel Design – References

1. *Specification for Structural Steel Buildings*, ASCE 360-10, AISC, Chicago, Il.
2. *Seismic Provisions for Structural Steel Buildings*, AISC 341-10, AISC, Chicago, Il.
3. *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*, AISC 358-10, AISC, Chicago, Il.

Requirements for Seismic Design

According to ASCE 7-10 Table 12.1-1, Item H, non-composite systems of steel assigned to SDC *B* and *C* are exempted from seismic design if $R \le 3$ is used to compute seismic loads. These structures are designed following the requirements of ASCE 360-10. For structures assigned to SDC *A*, seismic design loads do not involve *R*. All other situations are required to follow seismic design.

Extended Moment End-Plate Example

25-ft-long W21x68 beams are to be connected to a W14x145 exterior column using an extended end plate connection. From the ASCE 7-10 load combinations the critical moment and shear are 350 k-ft (M_u) 45 k and (V_u), respectively. The gravity load shear determined from the load combination 1.2D + f_1L + 0.2S is 25 k. The required axial strength of the column is $P_u(P_r)$ = 225 k.

Extended Moment End-Plate Example

The connection will be used in a seismic application (R = 8 – SMF). All steel is ASTM A992. ASTM A325-N fully tensioned bolts are to be used and welds will be made with E70 electrodes.

Difference Between Seismic and Non-seismic Design

1. Additional references must be followed
2. Pre-tensioned bolts must be used
3. Because of possible moment reversal, the same number of bolts must be used top and bottom
4. In certain areas CJP welds are required

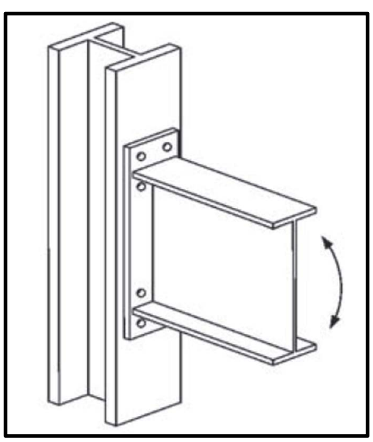
Difference Between Seismic and Non-seismic Design

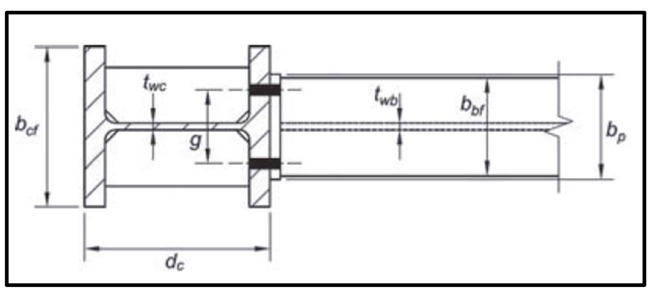
5. For non-seismic applications, design is required only for the moment resulting from load combinations (M_u = 350 k-ft)
6. For seismic design, the design must be for the full nominal flexural strength of the beam (M_u not used)
7. More stringent compactness required must be met

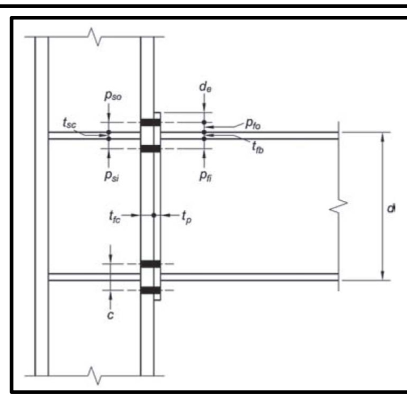
Beam Requirements for Four-Bolt Extended End Plate Connections

ANSI/AISC 358-10		
	Four-Bolt Unstiffened	
Parameter	Maximum	Minimum
t_{bf}	3/4	3/8
b_{bf}	9-1/4	6
d	55	13-3/4
t_p	2-1/4	1/2
b_p	10-3/4	7
g	6	4
p_{fi}, p_{fo}	4-1/2	1-1/2

Four-Bolt Unstiffened Extended End Plate Connection







Steel Moment Resisting Frames – ASCE 7-10

System	*R*	C_d	Ω_o	Comment
Ordinary Steel Moment Frame (OMF)	3.5	3	3	Minimum level of ductility in SDC *D* or higher
Intermediate Steel Moment Frame (IMF)	4.5	4	3	Design may be governed by drift
Special Steel Moment Frame (SMF)	8	5-1/2	3	Design may be governed by drift

Section and Material Properties

Beam data

$d_b = 21.1\text{ in}$

$t_{wb} = 0.430\text{ in}$

$b_{fb} = 8.27\text{ in}$

$t_{fb} = 0.685\text{ in}$

$k_b = 1.19\text{ in}$

$k_1 = \frac{7}{8}\text{ in}$

Workable gage $= 5\frac{1}{2}\text{ in}$

$Z_{xb} = 160\text{ in}^3$

Column Data

$A_c = 42.7\text{ in}^2$

$d_c = 14.8\text{ in}$

$t_{wc} = 0.680\text{ in}$

$b_{fc} = 15.5\text{ in}$

$t_{fc} = 1.09\text{ in}$

$k_c = 1.69\text{ in}$

$k_c = 1.38\text{ in}$

$(h/t_w)_c = 16.8\text{ in}$

Workable gage $= 5\frac{1}{2}\text{ in}$

$Z_{xc} = 260\text{ in}^3$

Material Data

$F_{y,b,c} = 50\text{ ksi}$

$F_{u,b,c} = 65\text{ ksi}$

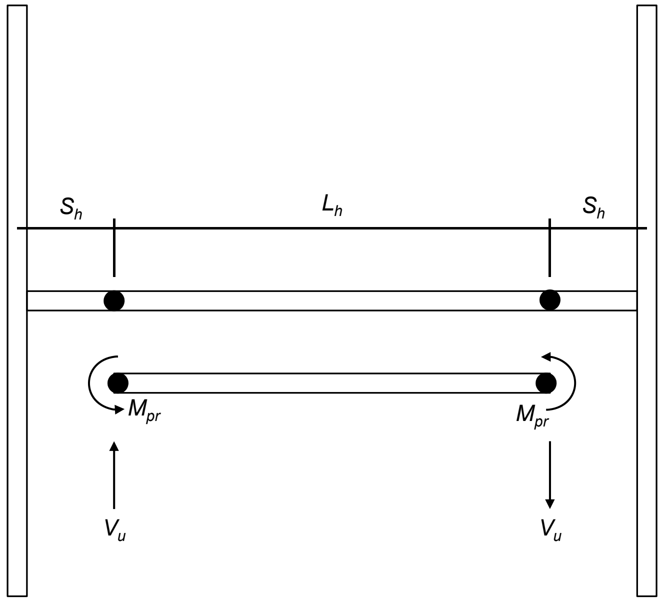
$F_{y,pl} = 50\text{ ksi}$

$F_{u,pl} = 65\text{ ksi}$

Bolt data

$F_t = 90\text{ ksi}$

Beam Plastic Hinge Location



Plastic Hinge Location

AISC 358-10 Section 6.10.1

$$S_h \rightarrow \min\left(d_b/2, 3b_{fb}\right)$$

$$S_h = \min(21.1/2, 3[8.27]) = \boxed{10.55 \text{ in}}$$

Beam-Column Connection Limitations

Note: the symbol d_b is used to represent both beam depth and bolt diameter. The meaning of the symbol applies should be clear from the context.

Sections 6.3 to 6.6 AISC 358-10

$13.75 < d_b = 21.1 < 55$

$3/8 < t_{fb} = 0.685 < 3/4$

$6 < b_{fb} = 8.27 < 9.25$

$L_b/d_b \geq 7$ (SMF); $(25 \text{ ft})\,(12 \text{ in/ft})/21.1 \text{ in} = 14.2$

Beam-Column Connection Limitations

Table D1.1 – AISC 341-10

Highly ductile members - unstiffened elements

$b_{fc}/2t_{fc} = 15.5 \text{ in}/2(1.09 \text{ in}) = 7.11 < \lambda_{hd} = 0.3\sqrt{E/F_y} = 7.22$ Ok

$b_{fb}/2t_{fb} = 8.27 \text{ in}/2(0.685 \text{ in}) = 6.04 < \lambda_{hd} = 7.22$ Ok

Stiffened elements

$C_a = \frac{P_u}{\phi_b P_y}$

$\phi_b P_y = 0.9F_y A_c = 0.9(50 \text{ ksi})(42.7 \text{ in}^2) = 1{,}922 \text{ k}$

Beam-Column Connection Limitations

C_a (column) $= \frac{P_u}{\phi_b P_y} = \frac{225\ k}{1{,}922\ k} = 0.117 < 0.125$

C_a (beam) ≈ 0

$\lambda_{hd,c} = 2.45\sqrt{E/F_y}\,(1 - 0.93C_a) = 52.6$

$\lambda_{hd,b} = 2.45\sqrt{E/F_y}\,(1 - 0.93C_a) = 59.0$

$h_b = d_b - 2k_b = 21.1\text{ in} - 2(1.19\text{ in}) = 18.7\text{ in}$

$h_b/t_{wb} = 18.7\ in/0.430\ in = 43.5 < 59.0$ OK

$h_c = d_c - 2k_c = 14.8\text{ in} - 2(1.69\text{ in}) = 11.42\text{ in}$

$h_c/t_{wc} = 11.42\ in/0.680\ in = 16.79 < 52.6$ OK

Beam-Column Connection Limitations

AISC 341-10 Section E3.4a

$\frac{\sum M^*_{pc}}{\sum M^*_{pb}} > 1.0$

$\sum M^*_{pc} = \sum Z_c\left(F_{yc} - P_{uc}/A_c\right)$

$= 2(260\text{ in}^3)(50\text{ ksi} - 225\text{ k}/42.7\text{ in}^2) = 23{,}260\text{ in}\cdot\text{k}$

$\sum M^*_{pb} = \sum\left(1.1R_yF_{yb}Z_b + M_{uv}\right)$

$= (1.1)(1.1)(50\text{ ksi})(160\text{ in}^3) + \boxed{(96.1\text{ k})}\boxed{(10.55\text{ in})}$

$= 10{,}690\text{ in}\cdot\text{k}$

$\frac{\sum M^*_{pc}}{\sum M^*_{pb}} = \frac{23{,}260\text{ in}\cdot\text{k}}{10{,}690\text{ in}\cdot\text{k}} = 2.18$ OK

Beam Side

Geometric Design Data

$b_p = b_{fb} + 1 \text{ in} = 8.27 \text{ in} + 1 \text{ in} = 9.27 \text{ in}$; Use $b_p = 9.25 \text{ in}$

$g = 5.5 \text{ in}$

$p_{fi} = 2 \text{ in}$

$p_{fo} = 2 \text{ in}$

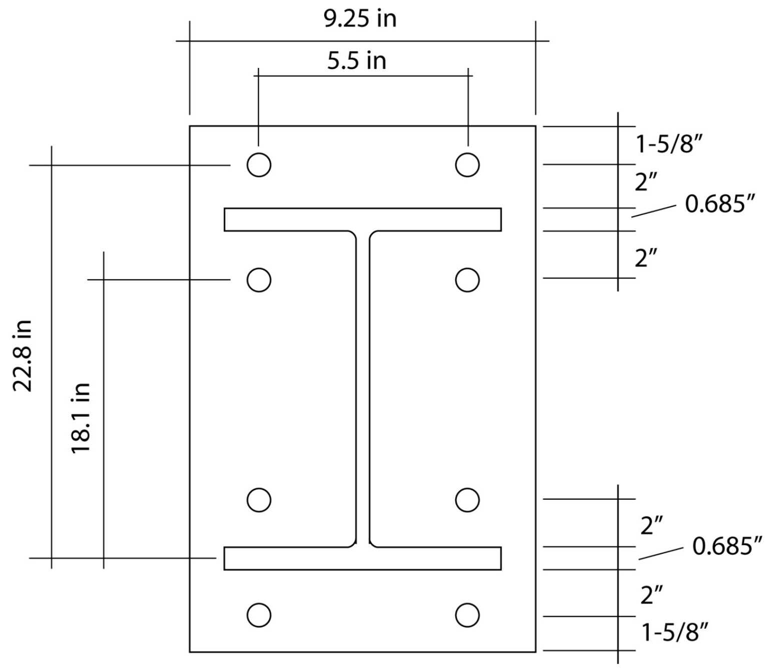
$d_e = 1\frac{5}{8} \text{ in}$

$h_o = d_b + p_{fo} - \frac{t_{fb}}{2} = 21.1 \text{ in} + 2 \text{ in} - \frac{0.685 \text{ in}}{2} = 22.8 \text{ in}$

$h_1 = d_b - t_{fb} - p_{fi} - \frac{t_{fb}}{2} =$

$21.1 \text{ in} - 0.685 \text{ in} - 2 \text{ in} - \frac{0.685 \text{ in}}{2} = 18.1 \text{ in}$

Geometric Design Data



Connection Design Moment

AISC 358-10 Section 6.10.1

$M_f = M_{pr} + V_u S_h$

$V_u = 2M_{pr}/L_h + V_{gravity}$

$C_{pr} = \frac{F_y+F_u}{2F_y} \leq 1.2 = \frac{50\text{ ksi}+65\text{ ksi}}{2(50\text{ ksi})} = 1.15 \leq 1.2$

$M_{pr} = C_{pr} R_y F_y Z_{xb} = (1.15)(1.1)(50\text{ ksi})(160\text{ in}^3) = 10{,}120\text{ k}\cdot\text{in}$

$= 843\text{ k}\cdot\text{ft}$

Location of plastic hinge

$S_h = 10.55\text{ in}$

$L_h = (12\text{ in/ft})(25\text{ ft}) - 2(10.55\text{ in}) = 278.9\text{ in} = 23.2\text{ ft}$

Connection Design Moment

$V_{gravity}$ taken from $1.2D + f_1L + 0.2S = 25\text{ k}$

Required shear resistance

$$V_u = \frac{2M_{pr}}{L_h} + V_{gravity} = \frac{2(843\text{ k·ft})}{23.7\text{ ft}} + 25\text{ k} = \boxed{96.1\text{ k}}$$

$$M_f = M_{pr} + V_uS_h$$

$$M_f = 10{,}120\text{ k}\cdot\text{in} + (96.1\text{ k})(10.55\text{ in}) = 11{,}133\text{ k}\cdot\text{in} = 928\text{ k}\cdot\text{ft}$$

Select Bolt Diameter

Required Bolt Diameter

$$M_f = 11{,}133\text{ k}\cdot\text{in}$$

$$d_{b,reqd} = \sqrt{\frac{2M_f}{\pi\phi_nF_t(h_0+h_1)}} = \sqrt{\frac{2(11{,}133\text{ k·in})}{\pi(0.90)(90\text{ ksi})(22.8\text{ in}+18.1\text{ in})}}$$

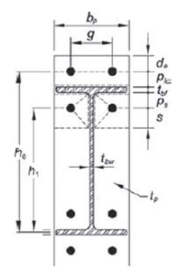
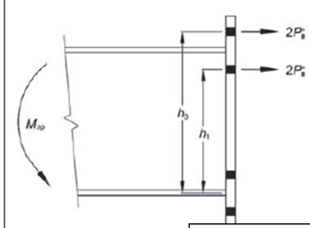
Try $d_b = 1 - \frac{1}{2}$ in

Minimum $p_f = 1.5 + \frac{3}{4} = 2.25\text{ in} > 2\text{ in}$ No good

Increase p_f to $2\frac{1}{4}$"

Connection Information

TABLE 6.2
Summary of Four-Bolt Extended Unstiffened End-Plate Yield Line Mechanism Parameter

End-Plate Geometry and Yield Line Pattern	Bolt Force Model
	
End-Plate	$Y_p = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{fi}}+\frac{1}{s}\right)+h_0\left(\frac{1}{p_{fo}}\right)-\frac{1}{2}\right]+\frac{2}{g}\left[h_1(p_{fi}+s)\right]$ $s=\frac{1}{2}\sqrt{b_p g}$ Note: If $p_{fi} > s$, use $p_{fi} = s$.

Reference 3

Select End Plate Thickness

End plate yield line mechanism parameter

$$s = \frac{1}{2}\sqrt{b_p g} = \frac{1}{2}\sqrt{(9.25\text{ in})(5.5\text{ in})} = 3.57\text{ in} > p_{fi} = 2.25$$

If $p_{fi} > s$, use $p_{fi} = s$

∴ Use $p_{fi} = 2.0$

$$Y_p = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{fi}}+\frac{1}{s}\right)+h_0\left(\frac{1}{p_{fo}}\right)-\frac{1}{2}\right]+\frac{2}{g}\left[h_1(p_{fi}+s)\right] =$$

$$\frac{9.25}{2}\left[(18.41\text{ in})\left(\frac{1}{2\text{ in}}+\frac{1}{3.57\text{ in}}\right)+(23.10\text{ in})\left(\frac{1}{2\text{ in}}\right)-\frac{1}{2}\right]+$$

$$\frac{2}{5.5\text{ in}}\left[(18.41\text{ in})(2\text{ in}+3.57\text{ in})\right] = 154.8$$

Select End Plate Thickness – Flexure

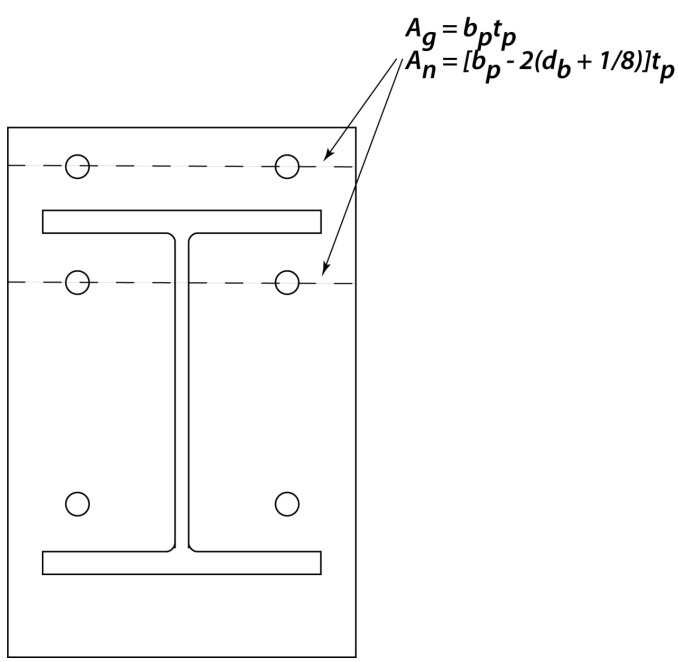
Required end plate thickness

$$t_{p,reqd} = \sqrt{\frac{1.11M_f}{\phi_d F_{y,pl} Y_p}}$$

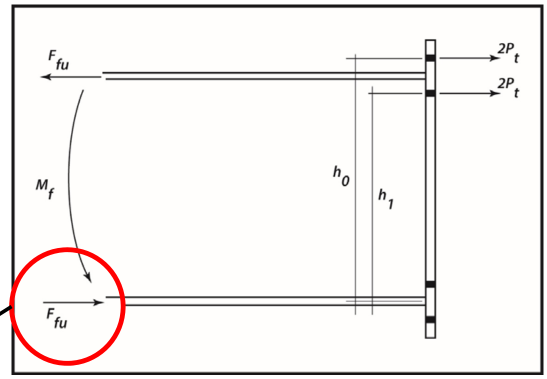
$$= \sqrt{\frac{1.11(11{,}133\ \text{k·in})}{(1)(50\ \text{ksi})(154.8)}} = 1.264\ \text{in}$$

Try $t_p = 1 - \frac{1}{4}$ in

Shear Areas



Check End Plate Thickness – Shear



Factored beam flange force

$$F_{fu} = \frac{M_f}{d_b - t_{fb}} = \frac{11{,}133 \text{ k·in}}{21.1 \text{ in} - 0.685 \text{ in}} = 545 \text{ k}$$

Shear yielding in the extended part of the end plate

$$\phi_d R_n = \phi_d (0.6 F_{y,pl}) b_p t_p$$

$$= (1)(0.6)(50 \text{ ksi})(9.25 \text{ in})(1.25 \text{ in}) = 346.9 \text{ k}$$

$\frac{F_{fu}}{2} = 273 \text{ k} < 246.9 \text{ k}$ OK

Check End Plate Thickness – Shear

Shear rupture in the extended part of the end plate

$$A_n = \left[b_p - 2\left(d_b + \frac{1}{8}\right)\right] t_p = [9.25 - 2(1.5 \text{ in} + 0.125 \text{ in})](1.25 \text{ in})$$

$$= 7.5 \text{ in}^2$$

$$\phi_n R_n = 0.90(0.6 F_{u,pl}) A_n = 0.90(0.6)(65 \text{ ksi})(7.5 \text{ in}^2) = 263.3 \text{ k}$$

$\frac{F_{fu}}{2} = 273 \text{ k} > 263.3 \text{ k}$ No good

Use $t_p = 1 - \frac{1}{2}$ in

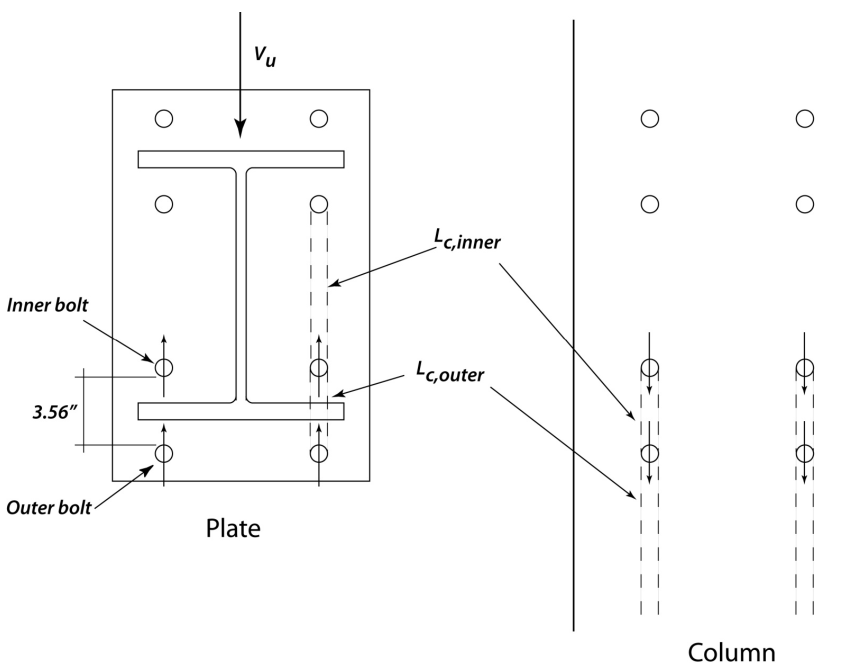
Compression Bolt Shear Rupture Capacity

$V_u = 96.1 \text{ k}$

$\phi_n R_n = \phi_n n_b F_v A_b = (0.90)(4)(48 \text{ ksi})\left[\frac{\pi(1.5 \text{ in})^2}{4}\right] = 305 \text{ k}$

$96.1 \text{ k} < 305 \text{ k}$ OK

Bolt Bearing/Tearout



Bolt Bearing/Tearout Capacity in Endplate

$\phi_n R_n = n_i(\phi R_n)_{inner} + n_o(\phi R_n)_{outer}$

$n_i = n_o = 2$

$R_n = 1.2L_c t_p F_{u,pl} \leq 2.4 d_b t_p F_{u,pl}$

$1.2L_c t_p F_{u,pl} \rightarrow$ tearout strength

$2.4 d_b t_p F_{u,pl} \rightarrow$ bearing strength

Nominal bolt bearing strength – one bolt

$2.4 d_b t_p F_{u,pl} = 2.4(1.5\text{ in})(1.5\text{ in})(65\text{ ksi}) = 351\text{ k/bolt}$

Bolt Bearing/Tearout Capacity in Endplate

Tearout – outer bolts

$$L_c = p_{fo} + t_{fb} + p_{fi} - \left(d_b + \frac{1}{8}\text{ in}\right)$$

$$= 2.25\text{ in} + 0.685\text{ in} + 2.25\text{ in} - \left(1.5\text{ in} + \frac{1}{8}\text{ in}\right) = 3.56\text{ in}$$

$R_{n,outer} = 351\text{ k}$

Tearout – inner bolts

L_c is very long ∴ bearing controls

$R_{n,inner} = 351\text{ k}$

$(n_i + n_o)\phi_n R_n = 4(0.9)(351\text{ k}) = 1{,}264\text{ k} > 96.1\text{ k}$ OK

Bolt Bearing/Tearout Capacity in Column

$t_{fc} = 1.09 \text{ in}$

Nominal bolt bearing strength – one bolt

$2.4d_b t_{fc} F_{u,c} = 2.4(1.5 \text{ in})(1.09 \text{ in})(65 \text{ ksi}) = 255.1 \text{ k/bolt}$

Tearout not possible for outer bolts – bearing controls

Tearout possible for inner bolts

$R_{n,inner} = 1.2L_c t_{fc} F_{u,c} = 1.2(3.56 \text{ in})(1.09 \text{ in})(65 \text{ ksi}) = 302.7 \text{ k}$

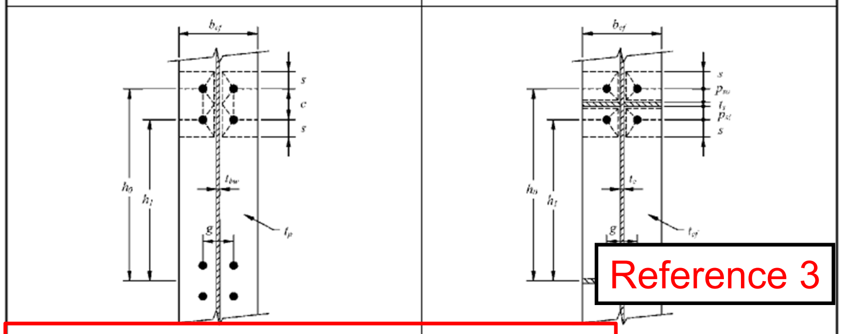
Bearing controls for inner bolts

$(n_i + n_o)\phi_n R_n = 4(0.9)(255.1 \text{ k}) = 918 \text{ k} > 96.1 \text{ k}$ OK

Column Side

Flexural Yielding in Column Flange

Unstiffened Column Flange Geometry and Yield Line Pattern	Stiffened Column Flange Geometry and Yield Line Pattern



Unstiffened Column Flange	$Y_c = \frac{b_{cf}}{2}\left[h_1\left(\frac{1}{s}\right)+h_0\left(\frac{1}{s}\right)\right]+\frac{2}{g}\left[h_1\left(s+\frac{3c}{4}\right)+h_0\left(s+\frac{c}{4}\right)+\frac{c^2}{2}\right]+\frac{g}{2}$ $s=\frac{1}{2}\sqrt{b_{cf}\,g}$
Stiffened Column Flange	$Y_c = \frac{b_{cf}}{2}\left[h_1\left(\frac{1}{s}+\frac{1}{p_{si}}\right)+h_0\left(\frac{1}{s}+\frac{1}{p_{so}}\right)\right]+\frac{2}{g}\left[h_1\left(s+p_{si}\right)+h_0\left(s+p_{so}\right)\right]$ $s=\frac{1}{2}\sqrt{b_{cf}\,g}$ Note: If $p_{si} > s$, use $p_{si} = s$

Flexural Yielding in Column Flange

$$s = \frac{1}{2}\sqrt{b_{fc}g} = \frac{1}{2}\sqrt{(15.5\text{ in})(5.5\text{ in})} = 4.62\text{ in}$$

Try an unstiffened column

$$c = p_{fo} + t_{fb} + p_{fi} = 2.25\text{ in} + 0.685\text{ in} + 2.25\text{ in} = 5.19\text{ in}$$

$$Y_c = \frac{b_{fc}}{2}\left[h_1\left(\frac{1}{s}\right) + h_0\left(\frac{1}{s}\right)\right] +$$

$$\frac{2}{g}\left[h_1\left(s + \frac{3c}{4}\right) + h_0\left(s + \frac{c}{4}\right) + \frac{c^2}{2}\right] + \frac{g}{2}$$

Flexural Yielding in Column Flange

$$Y_c = \frac{15.5\text{ in}}{2}\left[(18.1\text{ in})\left(\frac{1}{4.62\text{ in}}\right) + (22.8\text{ in})\left(\frac{1}{4.62\text{ in}}\right)\right] +$$

$$\frac{2}{5.5\text{ in}}\begin{bmatrix} (18.1\text{ in})\left(4.62\text{ in} + \frac{3(5.19\text{ in})}{4}\right) + \\ (22.8\text{ in})\left(4.62\text{ in} + \frac{5.19\text{ in}}{4}\right) + \frac{(5.19\text{ in})^2}{2} \end{bmatrix} + \frac{5.5\text{ in}}{2}$$

$$= 181.3$$

Flexural Yielding in Column Flange

$$t_{fc,reqd} = \sqrt{\frac{1.11M_f}{\phi_d F_{yc} Y_c}} = \sqrt{\frac{1.11(11{,}133\text{ k·in})}{(1)(50\text{ ksi})(181.3)}} = 1.17\text{ in}$$

$1.17\text{ in} > 1.09\text{ in}$ – Stiffeners are required (continuity plates)

Strength of unstiffened column flange

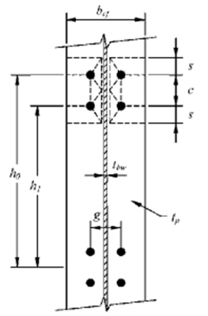
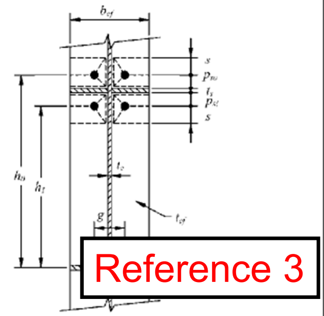
$$\phi_d M_{cf} = \phi_d F_{yc} Y_c t_{fc}^2$$

$$\phi_d M_{cf} = (1)(50\text{ ksi})(181.3)(1.09\text{ in})^2 = 10{,}770\text{ k}\cdot\text{in}$$

$$\phi_d R_n = \frac{\phi_d M_{cf}}{(d_b - t_{fb})} = \frac{10{,}770\text{ k·in}}{(21.1\text{ in} - 0.685\text{ in})} = 528\text{ k}$$

$$545\text{ k} - 528\text{ k} = 17.0\text{ k(tension)}$$

Flexural Yielding in Column Flange

Unstiffened Column Flange Geometry and Yield Line Pattern	Stiffened Column Flange Geometry and Yield Line Pattern
	

Unstiffened Column Flange	$Y_c = \frac{b_{cf}}{2}\left[h_1\left(\frac{1}{s}\right) + h_0\left(\frac{1}{s}\right)\right] + \frac{2}{g}\left[h_1\left(s + \frac{3c}{4}\right) + h_0\left(s + \frac{c}{4}\right) + \frac{c^2}{2}\right] + \frac{g}{2}$ $s = \frac{1}{2}\sqrt{b_{cf}\, g}$
Stiffened Column Flange	$Y_c = \frac{b_{cf}}{2}\left[h_1\left(\frac{1}{s} + \frac{1}{p_{si}}\right) + h_0\left(\frac{1}{s} + \frac{1}{p_{so}}\right)\right] + \frac{2}{g}\left[h_1\left(s + p_{si}\right) + h_0\left(s + p_{so}\right)\right]$ $s = \frac{1}{2}\sqrt{b_{cf}\, g}$ Note: If $p_{si} > s$, use $p_{si} = s$

Flexural Yielding in Column Flange

For stiffened column

Assume $p_{so} = p_{fo}$ and $p_{si} = p_{fi}$

$$Y_c = \frac{b_{fc}}{2}\left[h_1\left(\frac{1}{s} + \frac{1}{p_{si}}\right) + h_0\left(\frac{1}{s} + \frac{1}{p_{so}}\right)\right] +$$

$$\frac{2}{g}\left[h_1(s + p_{si}) + h_0(s + p_{so})\right] =$$

$$\frac{15.5\text{ in}}{2}\left[(18.1\text{ in})\left(\frac{1}{4.62\text{ in}} + \frac{1}{2.25\text{ in}}\right) + (22.8\text{ in})\left(\frac{1}{4.62\text{ in}} + \frac{1}{2.25\text{ in}}\right)\right] +$$

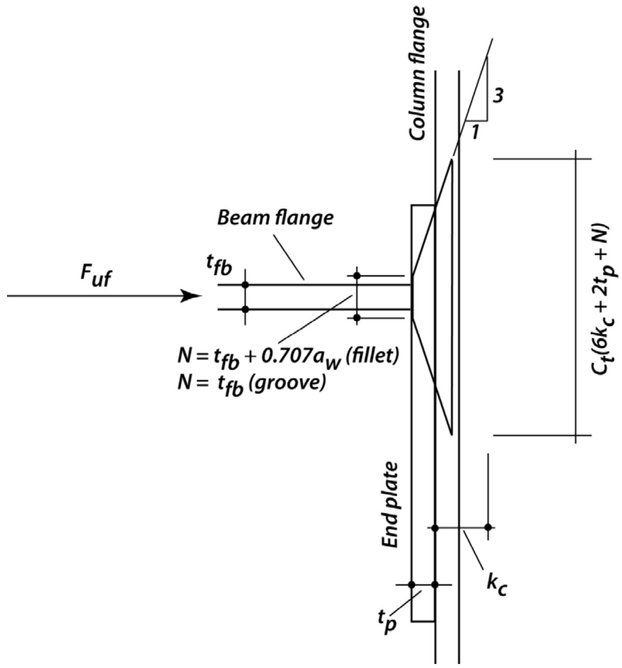
$$\frac{2}{5.5\text{ in}}\left[(18.1\text{ in})(4.62\text{ in} + 2.25\text{ in}) + (22.8\text{ in})(4.62\text{ in} + 2.25\text{ in})\right]$$

$$= 312$$

$$t_{fc,reqd} = \sqrt{\frac{1.11 M_f}{\phi_d F_{yc} Y_c}} = \sqrt{\frac{1.11(11{,}133\text{ k·in})}{(1)(50\text{ ksi})(312)}} = 0.890\text{ in}$$

$0.890\text{ in} < 1.09\text{ in}$ OK

Concentrated Force on Column Flange



Column Local Web Yielding

Assume connection not at top of column ($C_t = 1$)

$\phi_d R_n = \phi_d[C_t(6k_c + 2t_p + t_{bf})]F_{yc}t_{wc}$

$\phi_d R_n =$

$(1)[(1)\{6(1.69 \text{ in}) + 2(1.5 \text{ in}) + 0.685 \text{ in}\}](50 \text{ ksi})(0.680 \text{ in})$

$= 470 \text{ k} < F_{fu} = 545 \text{ k}$

Stiffening is required

$545 \text{ k} - 470 \text{ k} = 75.0 \text{ k}$ (compression)

Column Web Buckling

This check needs be made only if beams frame into column on two sides.

$$h = \left(\frac{h}{t_w}\right)_c t_{wc} = (16.8)(0.680\text{ in}) = 11.4\text{ in}$$

$$\phi R_n = \frac{\phi 24 t_{wc}^3 \sqrt{EF_{yc}}}{h} = \frac{0.75(24)(0.680\text{ in})^3\sqrt{(29{,}000\text{ ksi})(50\text{ ksi})}}{11.4\text{ in}}$$

$$= 598\text{ k} > F_{uf} = 545\text{ k}$$

Stiffening not required

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Column Web Crippling

$$\phi R_n = \phi 0.80 t_{wc}^2 \left[1 + 3\left(\frac{N}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^3\right]\sqrt{\frac{EF_{yc}t_{fc}}{t_{wc}}}$$

$$= 0.75(0.80)(0.680\text{ in})^2 \times$$

$$\left[1 + 3\left(\frac{0.685\text{ in}}{14.8\text{ in}}\right)\left(\frac{0.680\text{ in}}{1.09\text{ in}}\right)^3\right]\sqrt{\frac{(29{,}000\text{ ksi})(50\text{ ksi})(1.09\text{ in})}{0.680\text{ in}}}$$

$$= 437\text{ k} < F_{uf} = 545\text{ k}$$

Stiffening is required

$545\text{ k} - 437\text{ k} = 108\text{ k}$ (compression)

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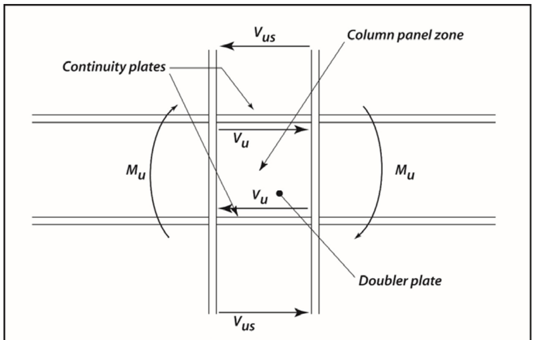
Column Panel Zone



Column Panel Zone

AISC 341-10 Section 6e

Minimum panel zone thickness, or doubler plates, if used:

$t_{wc} \geq \frac{d_z + w_z}{90}$

$d_z = d_b - 2t_{fb} = 21.1 \text{ in} - 2(0.685 \text{ in}) = 19.7 \text{ in}$

$w_z = d_c - 2t_{fc} = 14.8 \text{ in} - 2(1.09 \text{ in}) = 12.6 \text{ in}$

$t_{wc} = 0.645 \text{ in} \geq \frac{d_z + w_z}{90} = \frac{19.7 \text{ in} + 12.6 \text{ in}}{90} = 0.359 \text{ in}$

Column Panel Zone

AISC 341-10 Section 6e

AISC 360-10 Section J10.6

Required shear strength

$P_c = P_y = F_y A_c = (50 \text{ ksi})(42.7 \text{ in}^2) = 2{,}135 \text{ k};\ 0.4P_c = 854 \text{ k}$

$P_r = 225 \text{ k} < 0.4P_c = 854 \text{ k}$

$V_r = \frac{2M_f}{d_c} - V_{us} = \frac{2(11{,}133 \text{ k·in})}{14.8 \text{ in}} - 0 = 1{,}505 \text{ k}$

$\phi_v R_n = \phi_v 0.6 F_y d_c t_{wc}$

$= (1.0)(0.6)(50 \text{ ksi})(14.8 \text{ in})(0.680 \text{ in})$

$= 302 \text{ k}$

Doubler plates are required

$1{,}505 \text{ k} - 302 \text{ k} = 1{,}203 \text{ k}$

Questions

Thank you!