Common Engineering Mistakes

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Learning Objectives

- ***The intent of this course is to...***
  1. Highlight some common mistakes or misunderstanding in the current building codes.
  2. Discuss options to address deficiencies to drawings and calculations.

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Site Class Definitions

- ***Site Class F:***

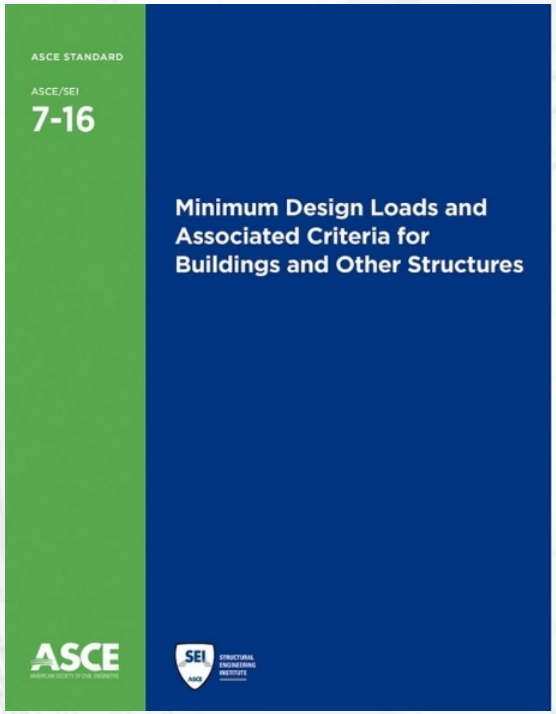
20.3 SITE CLASS DEFINITIONS

Site class types shall be assigned in accordance with the definitions provided in Table 20.3-1 and this section.

20.3.1 Site Class F. Where any of the following conditions is satisfied, the site shall be classified as Site Class F and a site response analysis in accordance with Section 21.1 shall be performed.

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

American Society of Civil Engineers, ASCE 7-16



American Society of Civil Engineers, ASCE 7-16

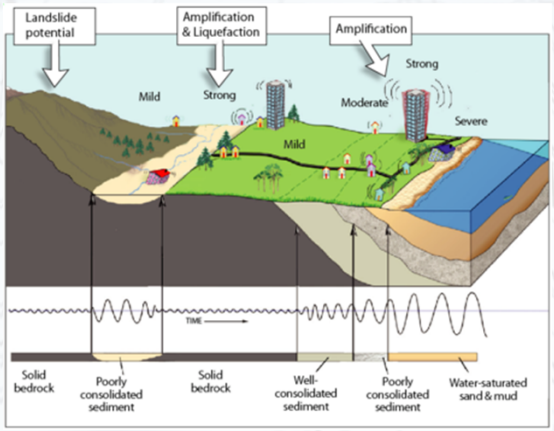
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Site Class Definitions (cont.)

- ***Site Class F:***

EXCEPTION: For structures that have fundamental periods of vibration equal to or less than 0.5 s, site response analysis is not required to determine spectral accelerations for liquefiable soils. Rather, a site class is permitted to be determined in accordance with Section 20.3 and the corresponding values of F_a and F_v determined from Tables 11.4-1 and 11.4-2.

American Society of Civil Engineers, ASCE 7-16



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C20.3 SITE CLASS DEFINITIONS

C20.3.1 Site Class F. Site conditions for which the site coefficients F_a and F_v in Tables 11.4-1 and 11.4-2 may not be applicable for site-response analyses required by Section 11.4.7. For short-period structures, it is permissible to determine values of F_a and F_v assuming that liquefaction does not occur because ground motion data obtained in liquefied soil areas during earthquakes indicate that short-period ground motions generally are attenuated because of liquefaction, whereas long-period ground motions may be amplified. This exception does not affect the requirements in Section 11.8 to assess liquefaction potential as a geologic hazard and to develop hazard mitigation measures as required.

American Society of Civil Engineers, Commentary to ASCE 7-16

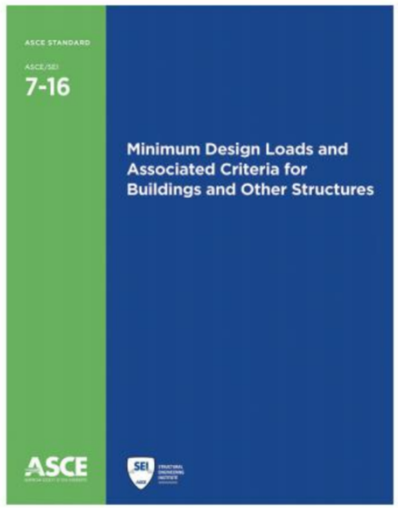
Site Class Definitions (cont.)

❑ *Structure Magazine (January 2022):*

Can a Site Class Change from F to D on a Liquefiable Site?

Q: Does the exception in ASCE 7-16 Section 20.3.1, Site Class F, allow the Site Class of a liquefiable site to change from an F to a D for buildings with fundamental periods equal to or less than 0.5 seconds?

A: Yes, the exception to Condition 1 of Section 20.3.1 does allow the site class to change from Site Class F to Site Class D if the fundamental period of the structure is less than or equal to 0.5 seconds, but ONLY if the soil 1) does not meet any of the other conditions listed under Section 20.3.1 for Site Class F, 2) does not meet the requirements for Site Class E under the exceptions to Section 20.3.1 Conditions 3 and 4, 3) is not classified as Site Class E under the requirements of Section 20.3.2, Soft Clay Site Class E, and 4) is not classified as Site Class E under the requirements of Section 20.3.3, Site Classes C, D, and E. The exception to Condition 1 of Section 20.3.1 does not take precedence over the other requirements of Sections 20.3.1, 20.3.2, and 20.3.3.



Q: As a clarification, is it the intent of this standard that the exception in Section 12.13.9, Requirements for Foundations on Liquifiable Sites, allows foundation ties to be omitted when all the parameters are met, even if the Site Class would have been an F?

A: No. The exception under Section 12.13.9 does not allow foundation ties required by Section 12.13.8.2, Foundation Ties, to be omitted for spread footings founded on Site Class E and F soils. The exception under Section 12.13.9 simply avoids the ADDITIONAL requirements for ties in 12.13.9.2.1.1, Shallow Foundation Design, Foundation Ties, from being applied. Please note that the commentary Section C12.13.9.2 clarifies this requirement; "Shallow foundations are required to be interconnected by ties, regardless of the effects of liquefaction."

Structure Magazine, Just the FAQs by Laura Champion, P.E. and Jennifer Goupil, P.E.

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Concrete Column Ties

- ACI 318-14 Section 10.7.6
  - Details of transverse reinforcement shall be in accordance with 25.7.2
  - Center-to-center spacing shall not exceed the least of $16d_b$ of longitudinal bar, $48d_b$ of tie bar, and the smallest dimension of the member.

Concrete Column Ties (cont)

- ACI 318-14 Section 18.14 Concrete columns not designated as part of the seismic-force-resisting system
  - 18.4.3.2(b) The maximum longitudinal spacing of hoops shall be s_o for the full column length. Spacing shall not exceed the lesser of six diameters of the smallest longitudinal bar enclosed and 6 inches.

Concrete Wall/Pier/Column

- Wall – A vertical element designed to resist axial load, lateral load, or both, with a horizontal length-to-thickness ratio greater than 3, used to enclose or separate spaces.

International Code Council, 2018 IBC©

Concrete Wall/Pier/Column (cont)

- Wall Pier – A vertical wall segment within a structural wall, bounded horizontally by two openings or by an opening and an edge, with ratio of horizontal length to wall thickness (l_w/b_w) less than or equal to 6.0, and ratio of clear wall height to horizontal length (h_w/l_w) greater than or equal to 2.0.

Concrete Pier (cont)

- ACI 318-14 Section 18.10.8: Wall piers shall satisfy the special moment frame requirements for columns of 18.7.4, 18.7.5, and 18.7.6.
  - Wall piers with $l_w/b_w \leq 2.5$ essentially act as concrete columns

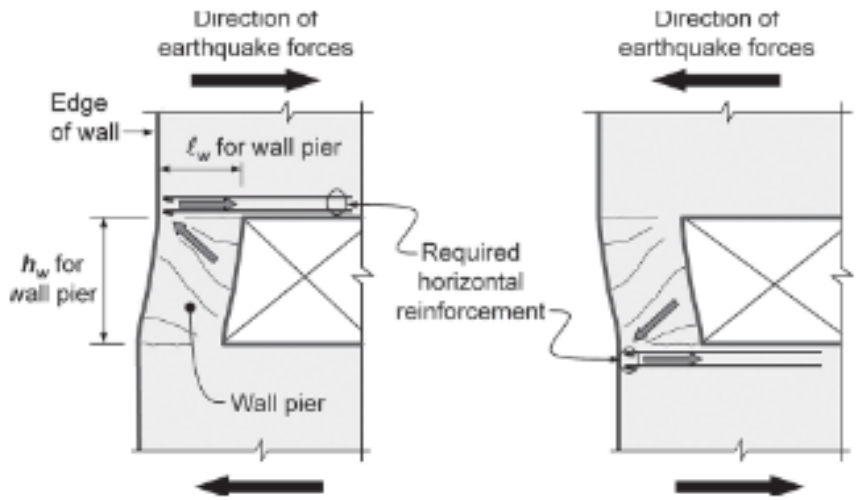
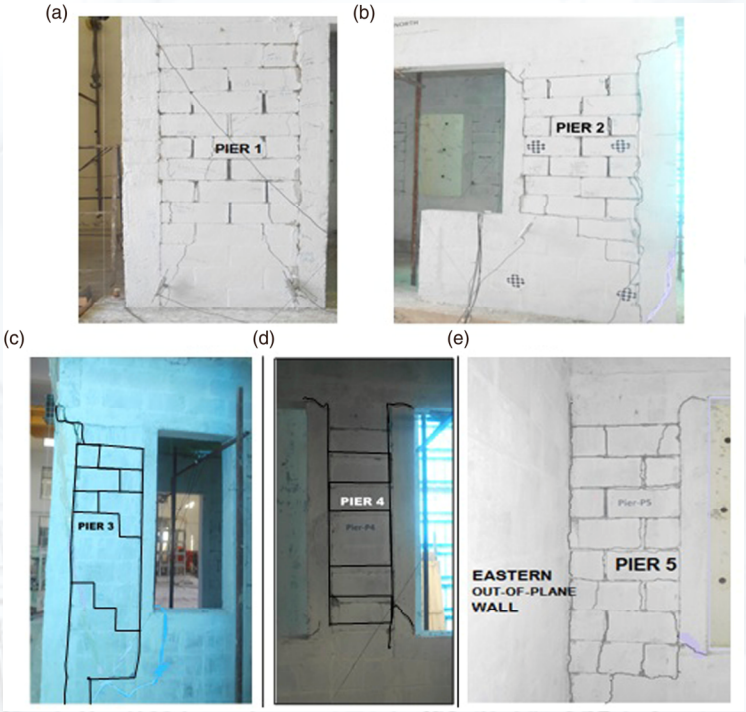


***Fig R18.10.8**—Required horizontal reinforcement in wall segments above and below wall piers at the edge of a wall.*

Masonry Wall/Pier

- TMS 402-16 Section 7.3.2.6: Maximum vertical and horizontal spacing of reinforcement is 1/3 the height or width of the wall pier.
  - The intent of this requirements is to ensure ductility
  - May be designed as a non-participating element and CMU lintel designed to span full opening.



Ahmed, Sadiz, Shahzada: Confined Hollow Concrete Block Masonry Buildings, Sage Journals, April 2022

Slotted Holes

- These are sometimes used for beam connections to address erection tolerances.

Slotted Holes

- They are also used to allow for thicker single plate connections. The *AISC Steel Construction Manual* Table 10-9 limits the plate thickness for single-plate shear connections with standard holes. Short-slotted holes allow for thicker plates.

Table 10-9
Design Values for Conventional Single-Plate Shear Connections

n	Hole Type	e, in.	Maximum t_p or t_w, in.
2 to 5	SSLT	$a/2$	None
	STD	$a/2$	$d/2 + 1/16$
6 to 12	SSLT	$a/2$	$d/2 + 1/16$
	STD	a	$d/2 - 1/16$

Slotted Holes

Table 10-10a

F_y = 36 ksi Plate

Single-Plate Connections

Bolt, Weld and Single-Plate Available Strengths, kips

$^3/_4$-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (l = 35 1/2)	Group A	N	STD	100	150	125	188	–	–	–	–	–	–	–	–
			SSLT	99.5	149	124	187	138	208	138	208	–	–	–	–
		X	STD	100	150	125	188	–	–	–	–	–	–	–	–
			SSLT	99.5	149	124	187	149	224	174	261	–	–	–	–
	Group B	N	STD	100	150	125	188	–	–	–	–	–	–	–	–
			SSLT	99.5	149	124	187	149	224	174	261	–	–	–	–
		X	STD	100	150	125	188	–	–	–	–	–	–	–	–
			SSLT	99.5	149	124	187	149	224	174	261	–	–	–	–
11 (l = 32 1/2)	Group A	N	STD	92.1	138	115	173	–	–	–	–	–	–	–	–
			SSLT	91.4	137	114	171	126	190	126	190	–	–	–	–
		X	STD	92.1	138	115	173	–	–	–	–	–	–	–	–
			SSLT	91.4	137	114	171	137	206	159	239	–	–	–	–
	Group B	N	STD	92.1	138	115	173	–	–	–	–	–	–	–	–
			SSLT	91.4	137	114	171	137	206	159	239	–	–	–	–
		X	STD	92.1	138	115	173	–	–	–	–	–	–	–	–

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Slotted Holes

- Most connections work with standard holes and standard plate thicknesses given in the *AISC Steel Construction Manual* Table 10-10A. Using short slotted holes can lead to problems.

Slotted Holes

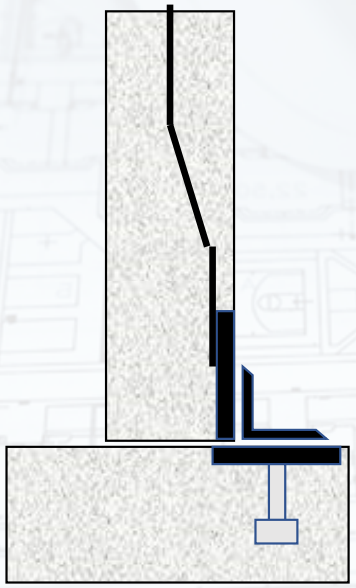
- AISC 360-16 J3.2(e) and (f)
  - Short-slotted holes are permitted in any or all plies of slip-critical or bearing type connections. The slots are permitted without regard to direction of loading in slip-critical connections, but **the length shall be normal to the direction of the loading in bearing type connections**.
  - Long-slotted holes are permitted in only one of the connected parts in either a slip-critical or bearing-type connection at an individual faying surface. Long-slotted holes are permitted without regard to direction of loading for slip-critical connections, but the **length shall be normal to the direction of the loading in bearing-type connections.**

Slotted Holes

- Where might you have forces parallel with the slot direction?
  - Collector (drag) connections.
  - CMU or concrete wall out-of-plane anchorage connections per ASCE 7-16 12.11.2.
- What can you do in these cases?
  - Use standard holes.
  - Use slip-critical bolts.
  - Weld the shear tab to beam.
  - Don't use the beam bolts for your load path. Instead use a strap plate or other load path.

Tilt Panel Base Attachment

- Are concrete tilt wall panels required to be attached to footings?



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Tilt Panel Base Attachment

- **Are concrete tilt wall panels required to be attached to footings?**
  - ASCE 7-16 §12.1.3 requires a complete lateral load path.
    - How is your shear taken out?
    - How are out-of-plane wind/seismic forces taken out?
    - Do you have uplift?
  - If you instead dowel to the slab you must consider:
    - Shear friction to transfer shear forces into the slab.
    - Tension development for out-of-plane forces.
    - Net uplift must be eliminated some other way, usually by tying panel sides to adjacent panels.

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Tilt Panel Base Attachment

- **Problems with tying to slab instead of using footing embeds:**
  - Need calculations showing that the slab develops enough friction with the slab weight alone to transfer the in-plane and out-of-plane forces into the soils.
  - Slab joints need reinforcement to travel through the joints to transfer forces.
  - Greased dowels at joints will not transfer tension forces and cannot be used.
  - The slab requires special inspection and a note that the rebar may not be cut.
  - The slab may crack as the dowels do not allow the slab to crack independent of the footings.

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Tilt Panel Base Attachment

- **Here is a typical review comment you may receive if you don't attach your tilt panels to your footings and haven't provided calculations to justify the omission:**
  - The project calls for tilt-up panels to be directly connected to the floor slab, with no physical connection to the footing. Please provide calculations showing that the portion of the floor slab that is used is capable of transferring the required forces to the supporting soils. In addition, please provide a check of the panel-to-slab connection for side-edge concrete breakout.

Steel Members of Seismic Lateral Force-Resisting System

- **Per Section D2.2 of AISC 341-16, all connections which are part of the SFRS are required to transfer forces through bolts or welds, not both.**
- **Bolted connections are required to be pretensioned.**
  - We often get responses claiming that a project doesn't need special inspection because the bolts are just required to be installed finger tight. Lateral connections require pretensioned bolts.
  - Bolts can be designed as bearing connections, but they require class A faying surfaces (slip-critical)

Steel Members of Seismic Lateral Force-Resisting System

(d) All bolts shall be installed as pretensioned high-strength bolts. Faying surfaces shall satisfy the requirements for slip-critical connections in accordance with *Specification* Section J3.8 with a faying surface with a Class A slip coefficient or higher.

Exceptions: Connection surfaces are permitted to have coatings with a slip coefficient less than that of a Class A faying surface for the following:

(1) End plate moment connections conforming to the requirements of Section E1, or ANSI/AISC 358

(2) Bolted joints where the seismic load effects are transferred either by tension in bolts or by compression bearing but not by shear in bolts

Steel Members of Seismic Lateral Force-Resisting System

- **Section A4 of AISC 341 requires the drawings to clearly designate all members that are part of the seismic-force-resisting-system (SFRS) and their connections.**
  - This helps the inspector know where pretensioned bolts with slip-critical faying surfaces are required.

Diaphragm Collectors

- ***ASCE 7-16 Section 12.10.2 requires collectors.***

12.10.2 Collector Elements. 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.

Diaphragm Collectors

- ***ASCE 7-16 Section 12.10.2 requires collectors.***

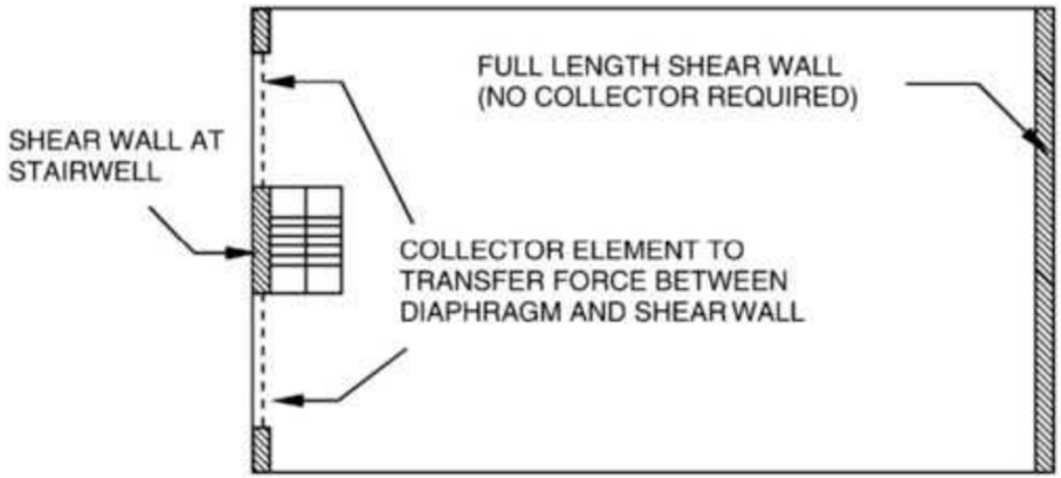


FIGURE 12.10-1 Collectors

Diaphragm Collectors

- **What if the diaphragm has enough shear capacity to directly transfer the entire force into the wall?**
  - What about the diaphragm tension capacity to get the force over to the wall where the deck can transfer the force?
  - ASCE 7-16 §12.10.2 requires collectors to transfer the seismic forces originating in other portions of the structure to the element providing resistance to those forces.

> **12.10.2 Collector Elements.** 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.

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Diaphragm Collectors

- **What if the diaphragm has enough shear capacity to directly transfer the entire force into the wall?**
  - What about the diaphragm tension capacity to get the force over to the wall where the deck can transfer the force?
  - ASCE 7-16 §12.10.2 requires collectors to transfer the seismic forces originating in other portions of the structure to the element providing resistance to those forces.
  - ASCE 7-16 §12.1.3 requires a complete lateral load path to transfer forces from the area they originate into the resisting elements.

Diaphragm Collectors

- **Is there any option to use partial depth diaphragm collectors?**
  - Yes, but the diaphragm will require special detailing.

Diaphragm Collectors

NEHRP Seismic Design Technical Brief No. 3



Seismic Design of Cast-in-Place Concrete Diaphragms, Chords, and Collectors

A Guide for Practicing Engineers

Jack P. Moehle
John D. Hooper
Dominic J. Kelly
Thomas R. Meyer

West Coast Code Consultants, Inc.

Diaphragm Collectors

- **NEHRP Seismic Design Technical Brief No. 3**
  - Pages 14-15 have some useful guidance.

"Full-depth collectors, . . . are designed based on the assumption of uniform slab shear that is transferred directly to the collector. The magnitude of the collector force is zero at the edges of the diaphragms (unless there is lumped mass at the free end) and increases linearly to a maximum where the collector enters the vertical element.

"Partial-depth collectors . . . rely to a greater extent on direct shear transfer between the diaphragm and the vertical element. Thus, partial-depth collector forces are smaller than those of full-depth collectors. The shortest collector and lowest collector force are obtained by designing the diaphragm for maximum permitted shear transfer directly to the vertical element and collector. In some cases, all of the force might theoretically be transferred directly to the wall without a collector, but we recommend extending a collector at least a bay width or 25 ft into the diaphragm, whichever is larger, to **control cracking** near the ends of the wall. The design force in the collector should vary linearly from zero at the end of the collector to a maximum at the face of the vertical element."

Diaphragm Collectors

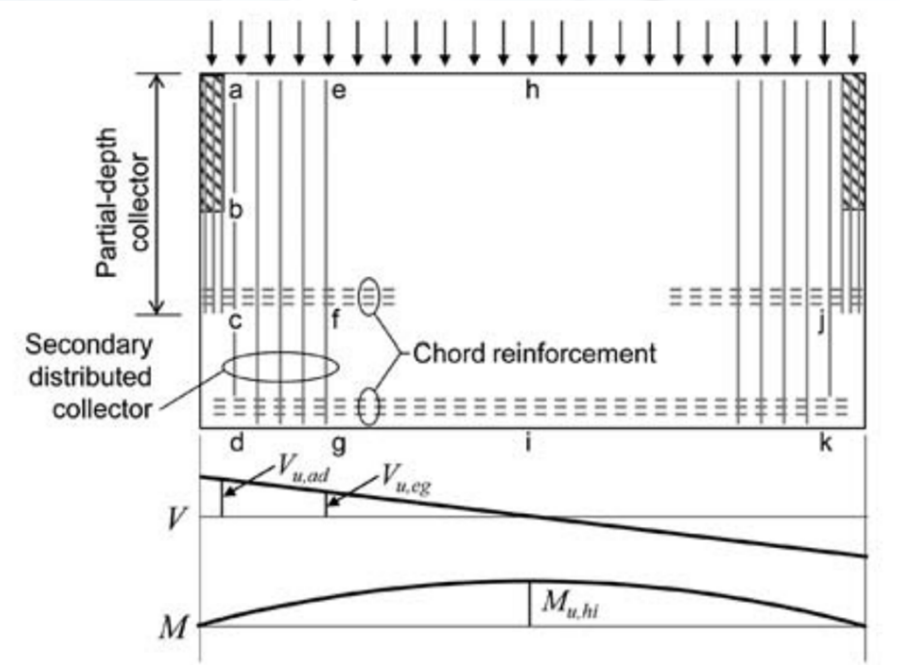


Figure 6-7 – Partial-depth collector.

Diaphragm Collectors

Design of partial-depth collectors requires additional considerations. A load path must be established for inertial forces in all areas of the diaphragm to reach the concentrated area of higher shear adjacent to the vertical elements and partial-depth collectors. In this regard, the load path is similar to a dapped or notched beam for which a concentration of reinforcement is required at the edge of the full-depth section that collects the shear and transfers it to the reduced depth section at the end of the beam. For the diaphragm, the load path requires a secondary distributed collector adjacent to the partial-depth collector. Typical, distributed slab reinforcement parallel to the vertical element serves as this secondary collector. Where necessary, the area of distributed slab reinforcement is increased locally to deliver inertial forces to the higher shear regions. The secondary collector also picks up local inertial forces from area cdfg and the small area to the left of it.

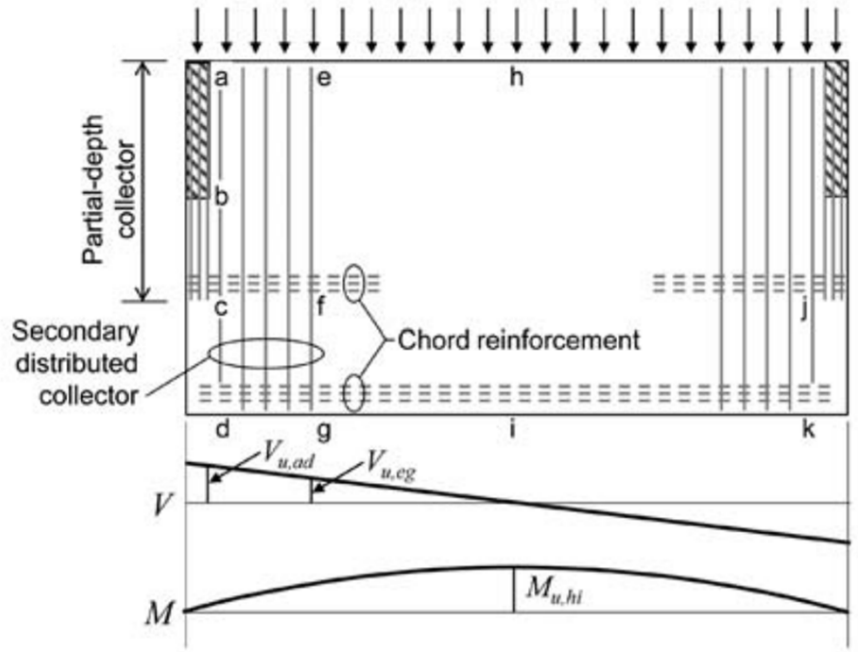


Figure 6-7 – Partial-depth collector.

Diaphragm Collectors

- **Notice that the following items are required:**
  - Secondary chord reinforcement.
  - Secondary distributed collectors.

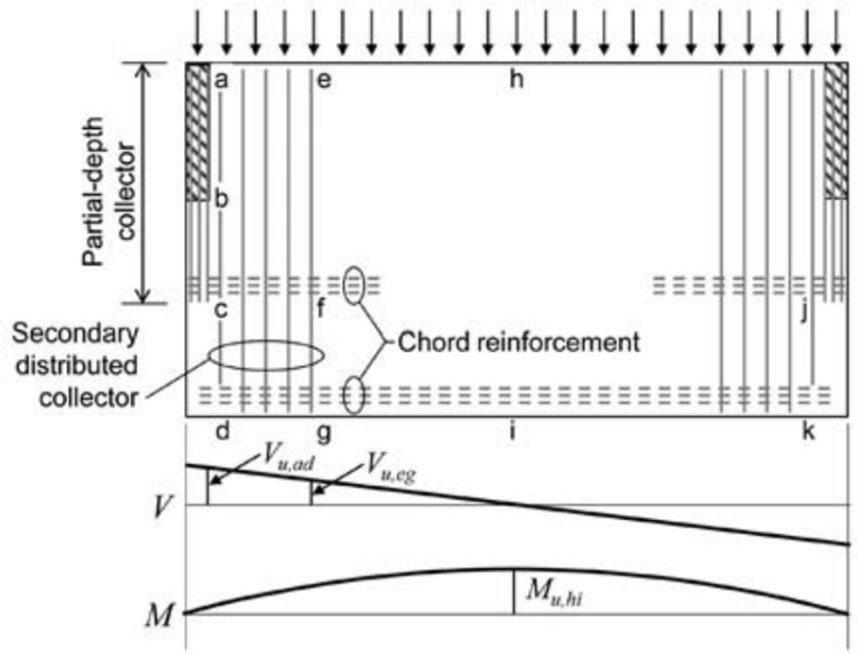


Figure 6-7 – Partial-depth collector.

Type 4 Horizontal Irregularity Requirements

Out-of-Plane Offset Irregularity: Out-of-plane offset irregularity is defined to exist where 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.	12.3.3.3 12.3.3.4 12.7.3 Table 12.6-1 16.3.4

There is one code reference missing from the table. ASCE 7-16 §12.10.1 also has a requirement for this irregularity.

Type 4 Horizontal Irregularity Requirements

All diaphragms shall be designed for the inertial forces determined from Eqs. (12.10-1) through (12.10-3) and for all applicable transfer forces. For structures that have a horizontal structural irregularity of Type 4 in Table 12.3-1, the transfer forces from the vertical seismic force-resisting elements above the diaphragm to other vertical seismic force-resisting elements below the diaphragm shall be increased by the overstrength factor of Section 12.4.3 before being added to the diaphragm inertial forces. For structures that have horizontal or vertical structural irregularities of the types indicated in Section 12.3.3.4, the requirements of that section shall also apply.

EXCEPTION: One- and two-family dwellings of light-frame construction shall be permitted to use $\Omega_0 = 1.0$.

Anchoring to Concrete

- *ACI 318-14 Chapter 17*



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Anchoring to Concrete - Tension

17.2.3.4 *Requirements for tensile loading*

17.2.3.4.1 Where the tensile component of the strength-level earthquake force applied to a single anchor or group of anchors is equal to or less than 20 percent of the total factored anchor tensile force associated with the same load combination, it shall be permitted to design a single anchor or group of anchors to satisfy 17.4 and the tensile strength requirements of 17.3.1.1.

17.2.3.4.2 Where the tensile component of the strength-level earthquake force applied to anchors exceeds 20 percent of the total factored anchor tensile force associated with the same load combination, anchors and their attachments shall be designed in accordance with 17.2.3.4.3. The anchor design tensile strength shall be determined in accordance with 17.2.3.4.4.

17.2.3.4.3 Anchors and their attachments shall satisfy one of options (a) through (d):

Profile:	Rectangular HSS (AISC), HSS12X4X.500; (L x W x T) = 12.000
Base material:	cracked concrete, 2500, f_c' = 2,500 psi; h = 12.000 in.
Reinforcement:	tension: condition B, shear: condition B; edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	no

CBFEM - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

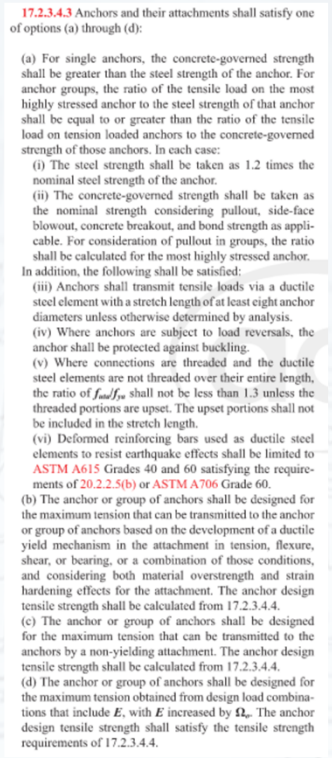
Geometry [in.] & Loading [lb, in.lb]

SIMPSON StrongTie Anchor Designer™ Software
Version 2.9.7376.0

Load and Geometry
Load factor source: ACI 318 Section 9.2
Load combination: U = 0.9D + 1.0E
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: D.3.3.4.2 not applicable
Ductility section for shear: D.3.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

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Anchoring to Concrete - Tension



17.2.3.4.3 Anchors and their attachments shall satisfy one of options (a) through (d):

(a) For single anchors, the concrete-governed strength shall be greater than the steel strength of the anchor. For anchor groups, the ratio of the tensile load on the most highly stressed anchor to the steel strength of that anchor shall be equal to or greater than the ratio of the tensile load on tension loaded anchors to the concrete-governed strength of those anchors. In each case:

(i) The steel strength shall be taken as 1.2 times the nominal steel strength of the anchor.

(ii) The concrete-governed strength shall be taken as the nominal strength considering pullout, side-face blowout, concrete breakout, and bond strength as applicable. For consideration of pullout in groups, the ratio shall be calculated for the most highly stressed anchor.

In addition, the following shall be satisfied:

(iii) Anchors shall transmit tensile loads via a ductile steel element with a stretch length of at least eight anchor diameters unless otherwise determined by analysis.

(iv) Where anchors are subject to load reversals, the anchor shall be protected against buckling.

(v) Where connections are threaded and the ductile steel elements are not threaded over their entire length, the ratio of f_{uta}/f_{ya} shall not be less than 1.3 unless the threaded portions are upset. The upset portions shall not be included in the stretch length.

(vi) Deformed reinforcing bars used as ductile steel elements to resist earthquake effects shall be limited to ASTM A615 Grades 40 and 60 satisfying the requirements of 20.2.2.5(b) or ASTM A706 Grade 60.

(b) The anchor or group of anchors shall be designed for the maximum tension that can be transmitted to the anchor or group of anchors based on the development of a ductile yield mechanism in the attachment in tension, flexure, shear, or bearing, or a combination of those conditions, and considering both material overstrength and strain hardening effects for the attachment. The anchor design tensile strength shall be calculated from 17.2.3.4.4.

(c) The anchor or group of anchors shall be designed for the maximum tension that can be transmitted to the anchors by a non-yielding attachment. The anchor design tensile strength shall be calculated from 17.2.3.4.4.

(d) The anchor or group of anchors shall be designed for the maximum tension obtained from design load combinations that include E, with E increased by Ω_o. The anchor design tensile strength shall satisfy the tensile strength requirements of 17.2.3.4.4.

- ***A) ductile anchor w/ steel strength 1.2 times nominal.***
- ***B) maximum tension for ductile mechanism***
- ***C) maximum tension for non-yielding attachment***
- ***D) Overstrength level (Ω_o) tension***

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Anchoring to Concrete - Tension

- ***A) ductile anchor w/ steel strength 1.2 times nominal.***
  - This requires a stretch length!

In addition, the following shall be satisfied:

(iii) Anchors shall transmit tensile loads via a ductile steel element with a stretch length of at least eight anchor diameters unless otherwise determined by analysis.

(iv) Where anchors are subject to load reversals, the anchor shall be protected against buckling.

(v) Where connections are threaded and the ductile steel elements are not threaded over their entire length, the ratio of f_{uta}/f_{ya} shall not be less than 1.3 unless the threaded portions are upset. The upset portions shall not be included in the stretch length.

(vi) Deformed reinforcing bars used as ductile steel elements to resist earthquake effects shall be limited to ASTM A615 Grades 40 and 60 satisfying the requirements of 20.2.2.5(b) or ASTM A706 Grade 60.

1.3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	1,950	14,529	14	OK
Pullout Strength*	1,950	12,754	16	OK
Concrete Breakout Failure**	3,283	12,799	26	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

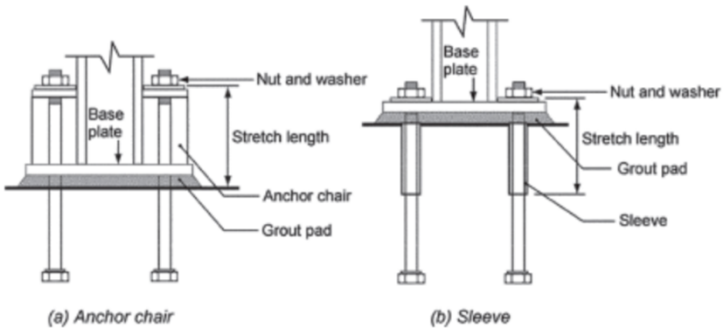


Fig. R17.2.3.4.3—Illustrations of stretch length.

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Anchoring to Concrete - Tension

- ***B) maximum tension for ductile mechanism***
- ***C) maximum tension for non-yielding attachment***

For both options you design for the capacity of the system, not the applied force!

- ***Examples:***
  - Hold down capacity
  - BRB capacity
- ***This uses <u>expected</u> strength, not specified.***
- ***Must consider strain hardening and material overstrength.***

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Anchoring to Concrete - Tension

- ***D) Overstrength level (Ω_o) Tension***

Anchoring to Concrete - Shear

17.2.3.5.1 Where the shear component of the strength-level earthquake force applied to the anchor or group of anchors is equal to or less than 20 percent of the total factored anchor shear force associated with the same load combination, it shall be permitted to design the anchor or group of anchors to satisfy 17.5 and the shear strength requirements of 17.3.1.1.

17.2.3.5.2 Where the shear component of the strength-level earthquake force applied to anchors exceeds 20 percent of the total factored anchor shear force associated with the same load combination, anchors and their attachments shall be designed in accordance with 17.2.3.5.3. The anchor design shear strength for resisting earthquake forces shall be determined in accordance with 17.5.

Profile:	Rectangular HSS (AISC), HSS12X4X.500; (L x W x T) = 12.000
Base material:	cracked concrete, 2500, f_c' = 2,500 psi; h = 12.000 in.
Reinforcement:	tension: condition B, shear: condition B; edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	no

CBFEM - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [lb, in.lb]

SIMPSON Strong-Tie Anchor Designer™ Software
Version 2.9.7376.0

Load and Geometry
Load factor source: ACI 318 Section 9.2
Load combination: U = 0.9D + 1.0E
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: D.3.3.4.2 not applicable
Ductility section for shear: D.3.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

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Anchoring to Concrete - Shear

17.2.3.5.3 Anchors and their attachments shall be designed using one of options (a) through (c):

(a) The anchor or group of anchors shall be designed for the maximum shear that can be transmitted to the anchor or group of anchors based on the development of a ductile yield mechanism in the attachment in flexure, shear, or bearing, or a combination of those conditions, and considering both material overstrength and strain hardening effects in the attachment.

(b) The anchor or group of anchors shall be designed for the maximum shear that can be transmitted to the anchors by a non-yielding attachment.

(c) The anchor or group of anchors shall be designed for the maximum shear obtained from design load combinations that include E, with E increased by Ω_o. The anchor design shear strength shall satisfy the shear strength requirements of 17.3.1.1.

- ***A) maximum shear for ductile mechanism***
- ***B) maximum shear for non-yielding attachment***
- ***C) Overstrength (Ω_o) level shear***

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Anchoring to Concrete - Shear

- ***A) maximum shear for ductile mechanism***
- ***B) maximum shear for non-yielding attachment***

For both options you design for the capacity of the system, not the applied force!

- ***Examples:***
  - Sill plate shear crushing capacity
  - BRB capacity
- ***These are expected strengths, not specified strengths***

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Any Questions?

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