Alternative Seismic Design Category Determination *IBC §1613.3.5.1*

Where $S_1 < 0.75$, the *Seismic Design Category* is permitted to be determined from *IBC Table 1613.3.5(1)* alone (i.e, using S_{DS} only) when **all** of the following apply:

- ✓ $T_a < 0.8\ T_S$ in each of the two orthogonal directions, and
- ✓ the fundamental period of the structure used to calculate the story drift - $T < T_S$ … in each of the two orthogonal directions, and
- ✓ *ASCE 7 (12.8-2)* is used to determine the seismic response coefficient … $C_S = \dfrac{S_{DS}}{(R/I_e)}$, and
- ✓ The diaphragms are rigid (per *ASCE 7 – §12.3.1*) **or** for diaphragms that are flexible, the distance between vertical elements of the seismic force-resisting system (SFRS) ≤ 40 feet

Simplified Design Procedure *IBC §1613.3.5.2*

Where the alternate Simplified Design Procedure of *ASCE 7 – §12.14* is used, the *Seismic Design Category* shall be determined in accordance with *ASCE 7*.

3.7 ASCE 7 Seismic Design Criteria *ASCE 7 – Chapter 11*

Scope *ASCE 7 - §11.1.2*

Every structure (e.g., buildings and nonbuilding structures), and portion thereof, including nonstructural components, shall be designed and constructed to resist the effects of earthquake motions as prescribed by the seismic requirements of *ASCE 7*.

Applicability *ASCE 7 - §11.1.3*

Structures and their nonstructural components shall be designed and constructed in accordance with the requirements of the following chapters based on the type of structure or component:

- Buildings: *ASCE 7 – Chapter 12*
- Nonbuilding Structures: *ASCE 7 – Chapter 15*
- Nonstructural Components: *ASCE 7 – Chapter 13*
- Seismically Isolated Structures: *ASCE 7 – Chapter 17*
- Structures with Damping Systems: *ASCE 7 – Chapter 18*

Seismic Importance Factor, I_e *ASCE 7 - §11.5.1*

Each structure shall be assigned an *importance factor* (I_e) in accordance with *ASCE 7 – Table 1.5-2* … based on the *Risk Category* of the building (or other structure) from *IBC – Table 1604.5*.

- ➢ *Risk Category* I → $I_e = 1.0$
- ➢ *Risk Category* II → $I_e = 1.0$
- ➢ *Risk Category* III (high occupancy) → $I_e = 1.25$*
- ➢ *Risk Category* IV (essential facilities) → $I_e = 1.5$*

The seismic *importance factor* (I_e) is used in the *Seismic Response Coefficient* (C_S) equations with the intent to raise the yield level for important structures (e.g., hospitals, fire stations, emergency operation centers, hazardous facilities, etc.).

Use of an *importance factor* greater than one is intended to provide for a lower inelastic demand on a structure which should result in lower levels of structural and nonstructural damage.

Approximate Fundamental Period, T_a *ASCE 7 – §12.8.2.1*

The approximate fundamental period shall be determined by the following:

$$T_a = C_t h_n^x \qquad \textit{ASCE 7 (12.8-7)}$$

where:

C_t & x are determined from *ASCE 7 – Table 12.8-2*

h_n = height in feet, from the base to the uppermost level of the structure

NOTE: See Table C1 – Approximate Fundamental Period, T_a (Appendix C, p. 5-17) for tabulated values of *ASCE 7 (12.8-7)*.

- **Steel Moment-Resisting Frames (SMF, IMF & OMF) –**

$$T_a = 0.028 h_n^{0.8}$$

Or alternatively (for Steel MRF structures ≤ 12 stories and average story height ≥ 10 feet):

$$T_a = 0.1N \qquad \textit{ASCE 7 (12.8-8)}$$

where:

N = number of stories (i.e., levels) above the base

- **Concrete Moment-Resisting Frames (SMF, IMF & OMF) –**

$$T_a = 0.016 h_n^{0.9}$$

Or alternatively (for Concrete MRF structures ≤ 12 stories and average story height ≥ 10 feet):

$$T_a = 0.1N \qquad \textit{ASCE 7 (12.8-8)}$$

- **Steel EBF, Steel BRBF, or Dual System w/ EBF & SMF –**

$$T_a = 0.03 h_n^{0.75}$$

- **All Other Structural Systems –** (e.g., shear walls, CBF, Dual Systems)

$$T_a = 0.02 h_n^{0.75}$$

NOTE: Refer to *ASCE 7 – §12.8.2.1* and equations *(12.8-9)* & *(12.8-10)* for an alternative method of calculating T_a for structures with concrete or masonry shear walls.

4.10 Actual vs. Design Seismic Forces

The *Risk-Targeted Maximum Considered Earthquake* (MCE_R) ground motion is the most severe earthquake effects considered by the *IBC* & *ASCE 7*.

The basis for the mapped MCE_R ground motions in *ASCE 7-10* is significantly different from that of the mapped MCE ground motions in *ASCE 7-05* and previous editions of *ASCE 7*. The MCE_R probabilistic ground motions are based on a uniform collapse risk (e.g., 1% probability of collapse in 50 years), rather than a uniform hazard (e.g., 2% probability of being exceeded in 50 years). The assumption is that buildings designed in accordance with *ASCE 7-10* have a collapse probability of not more than 10% (on average) if MCE_R ground motions were ever to occur at the building site.

In regions of high seismicity (e.g., many areas of California), the seismic hazard is typically controlled by large magnitude events occurring on a limited number of well defined fault systems. For these regions, it is considered more appropriate to use deterministic MCE_R ground motions where a collapse probability of

E_v = effect of vertical seismic forces (i.e., due to vertical ground motions) as defined in *ASCE 7 – §12.4.2.2*. E_v can be positive or negative due to the cyclic nature of (vertical) seismic ground motions.

Horizontal Seismic Load Effect with Overstrength Factor, E_{mh} *ASCE 7 – §12.4.3.1*

The horizontal seismic load effect with overstrength factor (E_{mh}) shall be determined in accordance with the following:

➢ $E_{mh} = \pm\Omega_0 Q_E$ *ASCE 7 (12.4-7)*

where:

Q_E = effects of horizontal seismic forces from the seismic base shear V (per *ASCE 7 – §12.8.1*) or the seismic lateral force F_p (per *ASCE 7 – §13.3.1*). See *ASCE 7 – §12.5.3* & *ASCE 7 – §12.5.4* for consideration of orthogonal effects)

Ω_0 = overstrength factor … per *ASCE 7 – Table 12.2-1*

Exception: E_{mh} need not exceed the maximum force that can develop in the element as determined by … see *ASCE 7 – §12.4.3.1*

5.2 Load Combinations *IBC §1605*

General *IBC §1605.1*

Buildings (and other structures) and portions thereof shall be designed to resist the load combinations specified in:

- *IBC §1605.2* (Strength Design or Load & Resistance Factor Design – SD/LRFD) or
- *IBC §1605.3* (Allowable Stress Design – ASD), and
- *IBC Chapters 18* through *23*, and
- The seismic load effects including overstrength factor (Ω_0) in accordance with *ASCE 7 – §12.4.3* where required by *ASCE 7 – §12.2.5.2*, *§12.3.3.3*, or *ASCE 7 – §12.10.2.1*

NOTE: When using the Simplified Procedure of *ASCE 7 – §12.14*, the seismic load effects including overstrength factor of *ASCE 7 – §12.14.3.2* shall be used (i.e., $\Omega_0 = 2.5$ assumed).

Load combinations are a way of considering the maximum (or minimum) forces on a structural element using principles of superposition.

The load combinations consider combined effects of gravity loads (e.g., dead load, floor live load, roof live load, rain load, snow load) and other load effects as a result of earthquake, wind, flood, earth pressure, fluid pressure, etc.

Notations –

D = Dead load

E = Combined effect of horizontal and vertical earthquake induced forces as defined in *ASCE 7 – §12.4.2*

F = Load due to fluids with well-defined pressures and maximum heights

F_a = Flood load in accordance with *ASCE 7 – Chapter 5*

H = Load due to earth pressure, ground water pressure, or pressure of bulk materials

L = Floor live load, and roof live load > 20 psf

Chapter 8

Diaphragm Design & Wall Rigidity

8.1 Diaphragm Design *ASCE 7 – §12.10.1*

Diaphragms shall be designed for both the shear and bending stresses resulting from design forces.

Diaphragm Design Force, F_{px} Strength Design force level

Floor and roof diaphragms shall be designed to resist design seismic forces from the structural analysis, but shall not be less than that determined in accordance with the following:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$$ *ASCE 7 (12.10-1)*

The diaphragm design force shall not be less than:

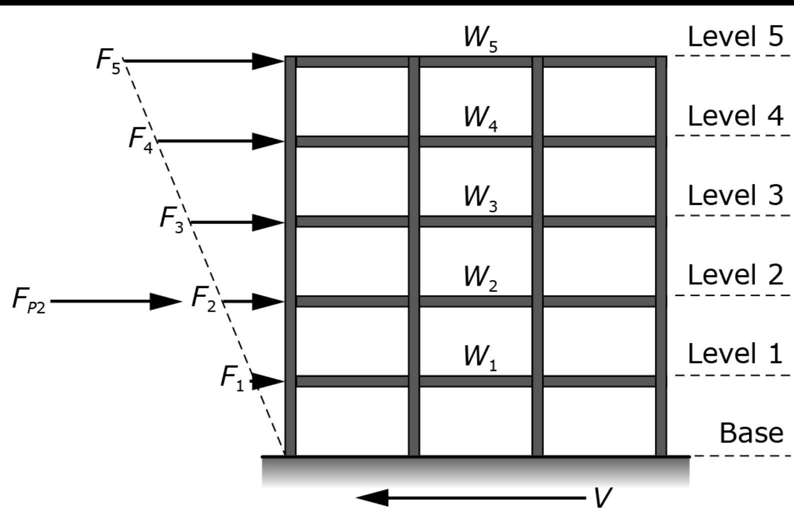
$$F_{px} \geq 0.2 S_{DS} I_e w_{px} \text{ minimum}$$ *ASCE 7 (12.10-2)*

The diaphragm design force need not exceed:

$$F_{px} \leq 0.4 S_{DS} I_e w_{px} \text{ maximum}$$ *ASCE 7 (12.10-3)*

where: w_{px} = weight of the diaphragm and the elements tributary thereto at Level x … including applicable portions of other loads from *ASCE 7 – §12.7.2* (e.g., 25% of floor live load for storage/warehouse, 10 psf minimum for partitions, total operating weight of permanent equipment, 20% of design snow load where P_f > 30 psf, etc.).

Figure 8.1 – Diaphragm Design Force



$$F_{p2} = \left(\frac{F_2 + F_3 + F_4 + F_5}{w_2 + w_3 + w_4 + w_5}\right) w_{p2}$$

The weight of the wall (W_W) can be considered a dead load effect (D). While the weight of the wall might be considered in the determination of the total seismic force on the shear wall (i.e., $V_1 + C_S W_W$), it will not be used to determine E_v because the dead load effect (D) does not contribute to the determination of the calculated unit wall shear.

NOTE: The Redundancy factor (ρ) shall be considered in the design of the shear walls.

Therefore, $E = E_h = \rho\, Q_E = \rho\, V_1$ …

So the ASD calculated unit wall shear:

$$\text{ASD calculated wall } v = \frac{\rho\ (0.7)(\text{total wall shear})}{\sum \text{shear wall width}} = \frac{\rho\ (0.7\ V_1)}{\sum b_s} \quad \text{(units of plf)}$$

NOTE: The equation above is used to determine the drag force, and may be used for shear wall design when the wall weight (W_w) is not significant, not given in a problem statement, or when the diaphragm design force (e.g., $w_s = f_{px} = F_{px}/L$) includes all perimeter walls of the building … essentially when the base shear (V) is used to design the diaphragm.

Otherwise, the weight of the shear wall (W_w) can be included in the ASD calculated unit wall shear:

$$\text{ASD calculated wall } v = \frac{\rho(0.7)(V_1 + C_S W_W)}{\sum b_s} \quad \text{(units of plf)}$$

Using the ASD calculated wall v and *SDPWS Table 4.3A* (see Table 9.5), choose the appropriate:

- ✓ WSP panel grade
- ✓ WSP nominal panel thickness
- ✓ Fastener (nail) size
- ✓ Fastener (nail) spacing

Such that the ASD allowable wall $v \geq$ ASD calculated wall v

Shear Walls in a Line *SDPWS §4.3.3.4*

The shear distribution to individual shear walls in a shear wall line shall provide the same calculated deflection (δ_{sw}) in each shear wall (i.e., the shear force shall be distributed based on the relative stiffnesses of the individual shear walls).

Alternatively, where nominal shear capacities of all WSP shear walls with aspect ratios $h/b_s > 2{:}1$ are multiplied by $2b_s/h$ for design, shear distribution to individual full-height (shear) wall segments shall be permitted to be taken as proportional to the shear capacities of the individual full height wall segments used in design, and the nominal shear capacities need not be reduced by the ARF_{WSP} in *SDPWS §4.3.4.2.*

- ➢ $\boldsymbol{h/b_s \leq 2{:}1}$ → use nominal unit shear capacities from *SDPWS Table 4.3A* with no reduction
- ➢ $\boldsymbol{2{:}1 < h/b_s \leq 3.5{:}1}$ → use nominal unit shear capacities from *SDPWS Table 4.3A* multiplied by $2b_s/h$

Example - given two WSP shear walls on same wall line. Wall A: $h = 12'$ & $b_s = 7'$ and Wall B: $h = 12'$ & $b_s = 4'$. Determine the percent shear in each shear wall with an applied total shear of V_1 assuming both shear walls use the same WSP and nailing with a nominal capacity of 520 plf:

- Wall A: $h/b_s = (12' / 7') = 1.71 \leq 2{:}1$ → no unit shear capacity reduction necessary
  Capacity of Wall A = 520 plf (b_s) = 520 plf (7′) = 3,640 lbs

Hoop - closed tie reinforcement that provides strength for shear and confinement for the concrete core.
Confinement - provides for the ductile performance of members and increases the compressive strength of the confined concrete which helps to compensate for strength loss due to the "spalled" outer concrete shell.

Strong Column - Weak Beam Design ***ACI 318-14 – §18.7.3.2***

Intent is to confine flexural yielding (inelastic) to the beams while the columns remain elastic throughout their seismic response.

At any beam/column joint:

$$\Sigma M_{nc} \geq (6/5)\, \Sigma M_{nb} \qquad \textit{ACI 318 (18.7.3.2)}$$

or $$\sum \begin{pmatrix} \text{nominal flexural strength of} \\ \text{columns framing into the joint} \end{pmatrix} \geq \left(\frac{6}{5}\right) \sum \begin{pmatrix} \text{nominal flexural strength of} \\ \text{beams framing into the joint} \end{pmatrix}$$

Intermediate Moment Frames (IMF) *ACI 318-14 – §18.4*

(Not permitted in *Seismic Design Category* D, E & F)

- ***R* = 5** for reinforced concrete IMF

Limited special detailing of beam-column joint, therefore limited ductility (i.e., lower *R*).

Ordinary Moment Frames (OMF) *ACI 318-14 – §18.3*

(Not permitted in *Seismic Design Category* C, D, E & F)

- ***R* = 3** for reinforced concrete OMF

No special detailing of beam-column joint, therefore very limited ductility (i.e., very low *R*).

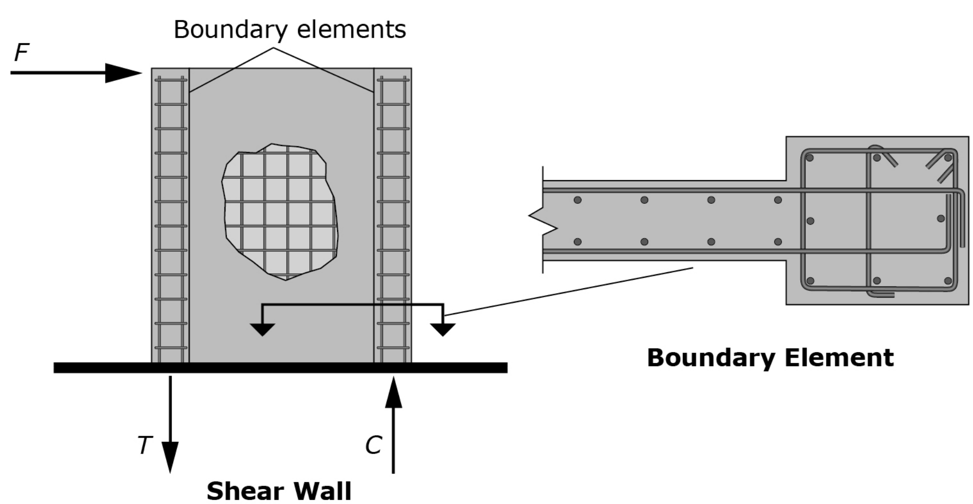
Special Reinforced Concrete Shear Walls *ACI 318-14 – §18.10*

(Typically required in *Seismic Design Category* D, E & F)

Special reinforced concrete structural walls (i.e., shear walls) need to comply with the requirements of *ACI 318 – §18.2.3* through *§18.2.8* and *§18.10.*

- ***R* = 5** for special reinforced concrete shear walls - Bearing Wall System
- ***R* = 6** for special reinforced concrete shear walls - Building Frame System

Figure 10.4 – Special Reinforced Concrete Shear Wall



Intermediate Reinforced Masonry Shear Walls *ACI 530-13 – §7.3.2.5*

(Not permitted in *Seismic Design Category* D, E & F)

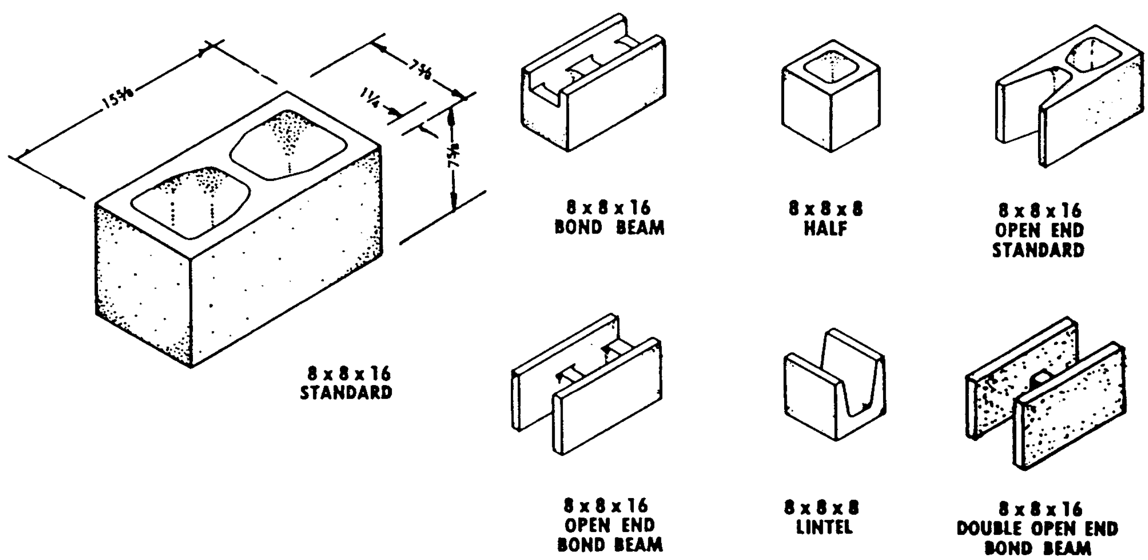
- ***R*** **= 3½** for intermediate reinforced masonry shear walls - Bearing Wall System
- ***R*** **= 4** for intermediate reinforced masonry shear walls - Building Frame System

Ordinary Reinforced Masonry Shear Walls *ACI 530-13 – §7.3.2.4*

(Not permitted in *Seismic Design Category* D, E & F)

- ***R*** **= 2** for ordinary reinforced masonry shear walls - Bearing Wall System
- ***R*** **= 2** for ordinary reinforced masonry shear walls - Building Frame System

Figure 10.6 – 8 x 8 x 16 Concrete Masonry Units (CMU)



10.4 IBC Chapter 22 – Steel

General *IBC §2205.1*

The design, fabrication and erection of structural steel elements of buildings, structures and portions thereof shall be in accordance with *AISC 360-10 – Specification for Structural Steel Buildings* (included in the *AISC Steel Construction Manual*).

Structural Steel Seismic Force-Resisting Systems *IBC §2205.2.1*

The design, detailing, fabrication and erection of structural steel seismic force-resisting systems shall be in accordance with the provisions of *IBC §2205.2.1.1* or *§2205.2.1.2*.

Seismic Design Category B or C – *IBC §2205.2.1.1*

Structures assigned to *SDC* = B or C shall be of any construction permitted in *IBC §2205*.

Where a response modification factor (*R*) in accordance with *ASCE 7-10 – Table 12.2.1* is used for the design … the structures shall be designed and detailed in accordance with the requirements of *AISC 341-10 – Seismic Provisions for Structural Steel Buildings*. See *IBC §2205.2.1.1 - Exception* for "structural steel systems not specifically detailed for seismic resistance, excluding cantilever column systems" per *ASCE 7-10 – Table 12.2-1, item H*.

Seismic Design Category D, E or F – *IBC §2205.2.1.2*

Structures assigned to *SDC* = D, E or F shall be designed and detailed in accordance with *AISC 341-10 – Seismic Provisions for Structural Steel Buildings* (included in the *AISC Seismic Design Manual*), except as permitted in *ASCE 7-10 – Table 15.4-1* (i.e., for nonbuilding structures similar to buildings).

Composite Structural Steel & Concrete Structures *IBC §2206*

General *IBC §2206.1*

Systems of structural steel elements acting compositely with reinforced concrete shall be designed in accordance with:

- ✓ *AISC 360-10 – Specification for Structural Steel Buildings*, and
- ✓ *ACI 318-14 – Building Code Requirements for Structural Concrete* (excluding Chapter 14)

Where required, the seismic design, fabrication and erection of composite steel and concrete systems shall be in accordance with *IBC 2206.2.1*.

Seismic requirements *IBC §2206.2.1*

Where a response modification coefficient R (per *ASCE 7 – Table 12.2-1*) is used for the design of systems of structural steel acting compositely with reinforced concrete, the structures shall be designed and detailed in accordance with the requirements of *AISC 341-10 – Seismic Provisions for Structural Steel Buildings.*

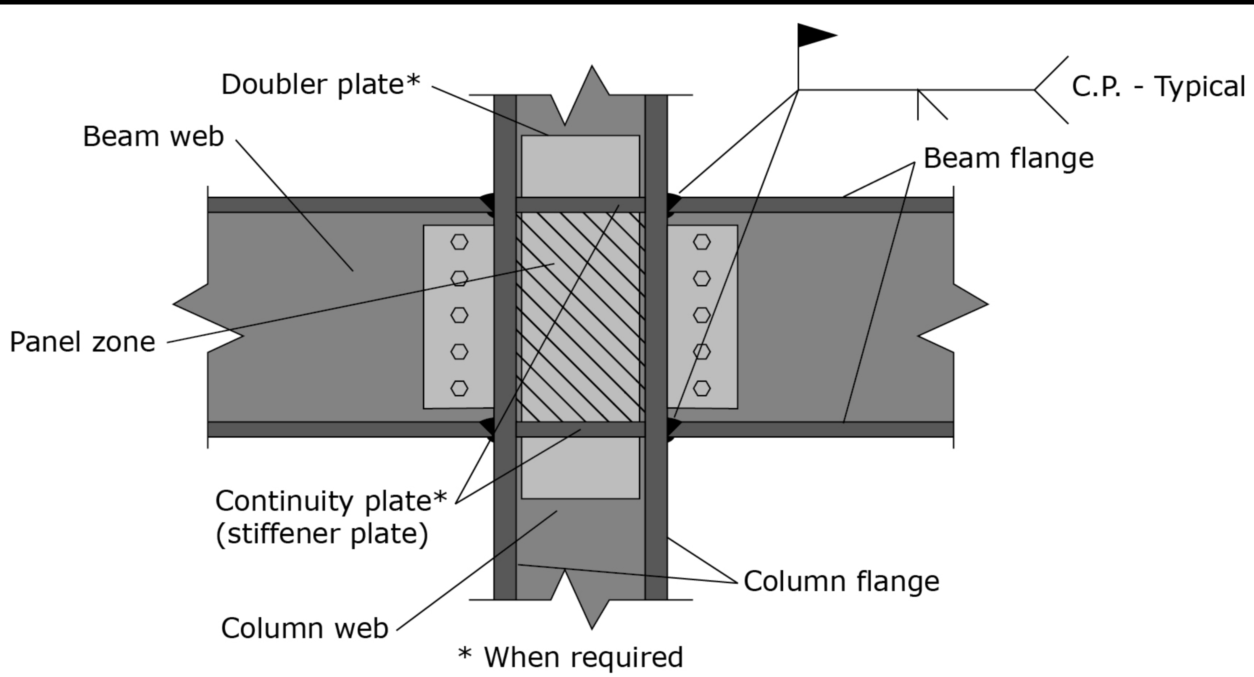
Special Moment Frames (SMF) *AISC 341-10 – §E.3*

(Typically required in *Seismic Design Category* D, E & F)

- $\boldsymbol{R = 8}$ for steel SMF

SMF's are expected to withstand significant inelastic deformations when subjected to the Design Basis Earthquake ground motions. Beam-column joints and members are designed to behave in a ductile manner.

Figure 10.7 – Steel Beam-Column Joint (pre-Northridge)



<u>NOTE</u>: Following the January 17, 1994 Northridge Earthquake, over 100 steel buildings with welded moment-resisting frames were found to have beam-to-column connection fractures. Usually the fractures initiated at the complete joint penetration weld between the beam bottom flange and the column flange. Investigators have suggested that the fractures may be due to a number of factors such as notch effects created by a left in place weld backing bar; sub-standard welding (excessive porosity, slag inclusions, incomplete fusion); and/or pre-earthquake fractures due to initial shrinkage of the weld during cool-down.

In September 1994, the *International Conference of Building Officials* (*ICBO*) adopted an emergency code change to the *1994 Uniform Building Code (UBC)*. This code change omitted the pre-qualified connection (see Figure 10.7) and required that connections be designed to sustain inelastic rotation and develop the strength criteria as demonstrated by cyclic testing or calculation.

In November 2000, the SAC Joint Venture finalized the welded steel moment frame issues which were published by the *Federal Emergency Management Agency* (FEMA) in the following documents:

- **FEMA 350** – Recommended Seismic Design Criteria for <u>New</u> Steel Moment-Frame Buildings
- **FEMA 351** – Recommended Seismic Evaluation and Upgrade Criteria for <u>Existing</u> Welded Steel Moment-Frame Buildings
- **FEMA 352** – Recommended <u>Post-Earthquake</u> Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings
- **FEMA 353** – Recommended <u>Specifications and Quality Assurance</u> Guidelines for Steel Moment-Frame Construction for Seismic Applications

Doubler Plates

Doubler plates are sometimes added to one (or both sides) of the column web and are intended to increase the column's web shear strength and web crippling capacity within the panel zone.

Continuity Plates

Continuity plates are sometimes added to provide "continuity" of the intersecting beam (or girder) flanges across the column web. The requirement for continuity plates is a function of the column yield strength, flange width, and flange thickness, and the intersecting beam yield strength, flange width, and flange thickness.

Intermediate Moment Frames (IMF) *AISC 341-10 – §E.2*

- ***R* = 4½** for steel IMF*

IMF's are expected to withstand <u>limited</u> inelastic deformations (in their members and connections) when subjected to the Design Basis Earthquake ground motions.

Limited special detailing of beam-column joint, therefore limited ductility (i.e., lower *R*).

Ordinary Moment Frames (OMF) *AISC 341-10 – §E.1*

- ***R* = 3½** for steel OMF*

OMF's are expected to withstand <u>minimal</u> inelastic deformations (in their members and connections) when subjected to the Design Basis Earthquake ground motions.

Very limited special detailing of beam-column joint, therefore very limited ductility (i.e., very low *R*).

***<u>NOTE</u>:** Typically steel IMF's and OMF's will not be permitted in structures assigned to *Seismic Design Category* <u>D, E or F</u> … with the exception of steel OMF's meeting the requirements of *ASCE 7 – §12.2.5.6.1* for *SDC* = <u>D or E</u> or *§12.2.5.6.2* for *SDC* = <u>F</u>, and steel IMF's meeting requirements of *ASCE 7 – §12.2.5.7.1* for *SDC* = <u>D</u> or *§12.2.5.7.2* for *SDC* = <u>E</u> or *§12.2.5.7.3* for *SDC* = <u>F</u>.

1. **Structural Steel -** *special inspections* for seismic resistance of:
   - **Seismic force-resisting systems (SFRS)** of buildings & structures assigned to *SDC* = B, C, D, E or F in accordance with the quality assurance requirements of *AISC 341-10*.
   - **Structural steel elements** in the SFRS of buildings & structures assigned to *SDC* = B, C, D, E or F (other than the SFRS elements covered above including struts, collectors, chords and foundation elements) in accordance with the quality assurance requirements of *AISC 341-10*.
2. **Structural wood -** for the SFRS of structures assigned to *SDC* = C, D, E or F per *IBC §1705.12.2*.

   Exception: When sheathing fastener spacing is > 4″ o.c., *special inspections* are not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring, and other fastening to other components of the SFRS.
3. **Cold-formed steel light-frame construction -** periodic special inspection for the SFRS of structures assigned to *SDC* = C, D, E or F per *IBC §1705.12.3*.
4. **Designated seismic systems -** for structures assigned to *SDC* = C, D, E or F per *IBC §1705.12.4*.
5. **Architectural components -** periodic special inspection for the erection and fastening of exterior cladding, interior and exterior nonbearing walls, and interior and exterior veneer in structures assigned to *SDC* = D, E or F.
6. **Plumbing, mechanical and electrical components** - periodic special inspection in structures assigned to *SDC* = C, D, E or F per *IBC §1705.12.6*.
7. **Storage racks -** periodic special inspection for the anchorage of storage racks > 8 feet in height in structures assigned to *SDC* = D, E or F.
8. **Seismic isolation systems -** periodic special inspection for seismically isolated (i.e., base isolated) structures assigned to *SDC* = B, C, D, E or F.
9. **Cold-formed steel special bolted moment frames -** periodic special inspection for the SFRS of structures assigned to *SDC* = D, E or F.

Exception: *Special inspections* noted above are not required for structures designed and constructed in accordance with one of the following:

- Light-frame construction where $S_{DS} \leq 0.5$ and the height of the structure ≤ 35 feet, or
- Reinforced masonry (or reinforced concrete) seismic force-resisting systems where $S_{DS} \leq 0.5$ and the height of the structure ≤ 25 feet, or
- Detached one- or two-family dwellings ≤ two stories (above grade plane), provided the structure does not have any of the following horizontal irregularities or vertical irregularities in accordance with *ASCE 7 – §12.3*:
  - ✓ Torsional or extreme torsional irregularity
  - ✓ Nonparallel systems irregularity
  - ✓ Stiffness-soft story or stiffness-extreme soft story irregularity
  - ✓ Discontinuity in lateral strength-weak story irregularity

11.7 Testing for Seismic Resistance *IBC §1705.13*

Unless exempted from *special inspections* by the exceptions of *IBC §1704.2 … testing* for seismic resistance shall be provided for the following:

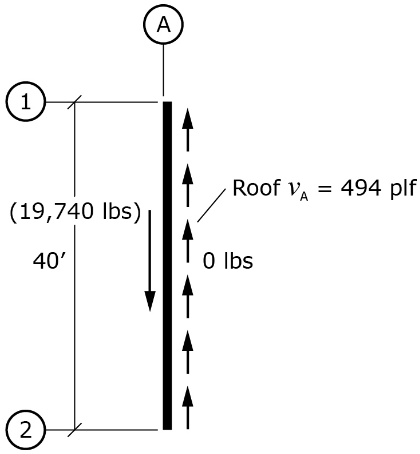
1. **Structural Steel** – nondestructive *testing* for seismic resistance of:
   - **Seismic force-resisting systems (SFRS)** of buildings & structures assigned to *SDC* = B, C, D, E or F in accordance with the quality assurance requirements of *AISC 341-10*.

4. Drag Force Diagram on lines A & B, F_d

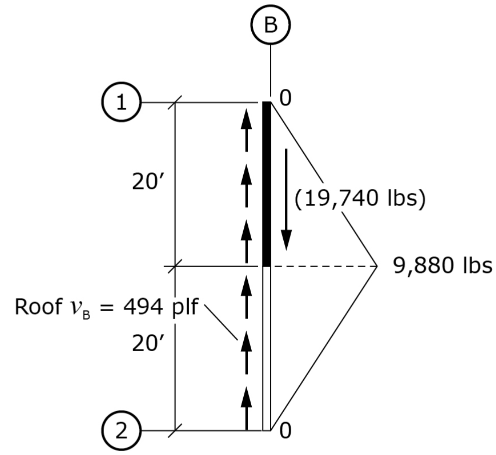
roof $v_A = v_B = 494$ plf

Wall Line A: $F_d = \underline{0 \text{ lbs}}$

Wall Line B: $F_d = (494 \text{ plf})(20') = \underline{9{,}880 \text{ lbs}}$ (SD/LRFD force level)



Drag Force – Line A



Drag Force – Line B

> **NOTE:** The actual design of the "*collector elements and their connections …*" would need to consider the overstrength factor as required by *ASCE 7 - §12.10.2.1* for this structure assigned to *SDC* = D (i.e., design for $\Omega_0 \cdot Q_E$ where Q_E is the drag force determined above … see p. 1-115).

B.) E-W DIRECTION: $L = 40'$, $d = 70'$

1. Design Seismic Force to Diaphragm, $w_s = f_{p1} = F_{p1}/L$

$$w_{p1} = \underbrace{(16 \text{ psf} + 20\% \cdot 100 \text{ psf})(70')(40')}_{\text{roof DL + 20\% snow}} + \underbrace{(85 \text{ psf})(14'/2 + 2')(2 \text{ walls})(40')}_{\text{East \& West exterior walls}}$$
$$= 100{,}800 \text{ lbs} + 61{,}200 \text{ lbs} = 162{,}000 \text{ lbs}$$

$F_{p1} = 0.190\, w_{p1} = 0.190\,(162{,}000 \text{ lbs}) = 30{,}780 \text{ lbs}$

$w_s = f_{p1} = F_{p1} / L = (30{,}780 \text{ lbs}) / (40') = \boxed{770 \text{ plf}}$

2. Unit Roof Shear on lines 1 & 2, v_r

$V_1 = V_2 = w_s\, L / 2 = (770 \text{ plf})(40'/2) = \underline{15{,}400 \text{ lbs}}$

Roof $v_1 = v_2 = V_1 / d = (15{,}400 \text{ lbs}) / 70' = \boxed{220 \text{ plf}}$ (SD/LRFD force level)

3. Maximum Chord Force on lines A & B, *CF*

max. $M = w_s\, L^2 / 8 = (770 \text{ plf})(40')^2 / 8 = 154{,}000$ lb-ft

max. $CF = (154{,}000 \text{ lb-ft}) / 70' = \boxed{2{,}200 \text{ lbs}}$ (SD/LRFD force level)

4. Shear Force to walls 1A & 1B

Relative Rigidities: assume cantilever walls, Table D1 - Relative Rigidity of Cantilever Shear Walls / Piers (Appendix D, p. 5-20)

Wall 1A: $H/D = 14'/11' = 1.27$ → Table D1 (p. 5-20) → $R_{1A} = 0.833$

Wall 1B: $H/D = 14'/22' = 0.64$ → Table D1 (p. 5-20) → $R_{1B} = 3.369$

$\Sigma R = R_{1A} + R_{1B} = 0.833 + 3.369 = 4.202$