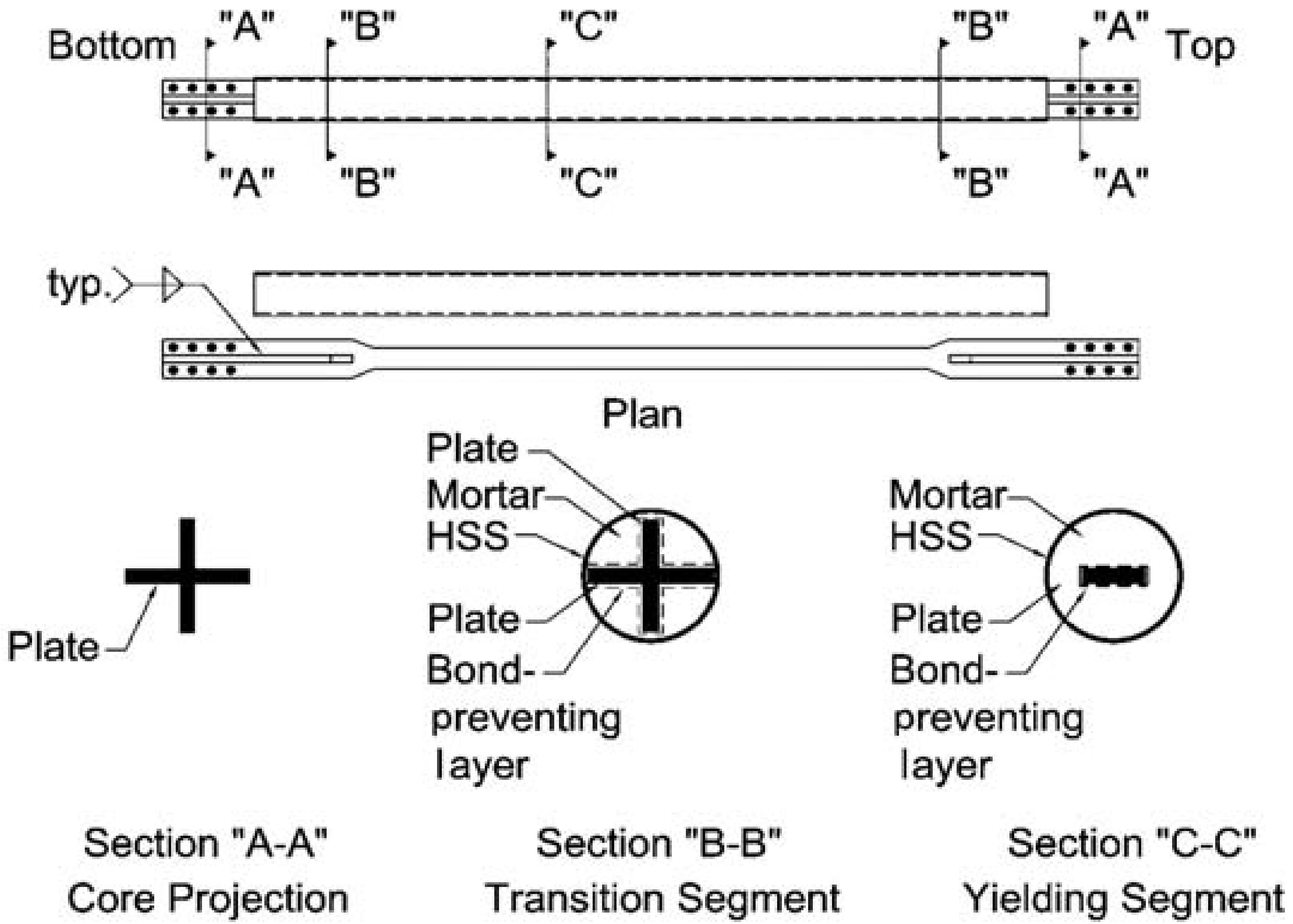
بسم الله الرحمن الرحيم

سیستم های مدرن جذب انرژی در

سازه های فولادی

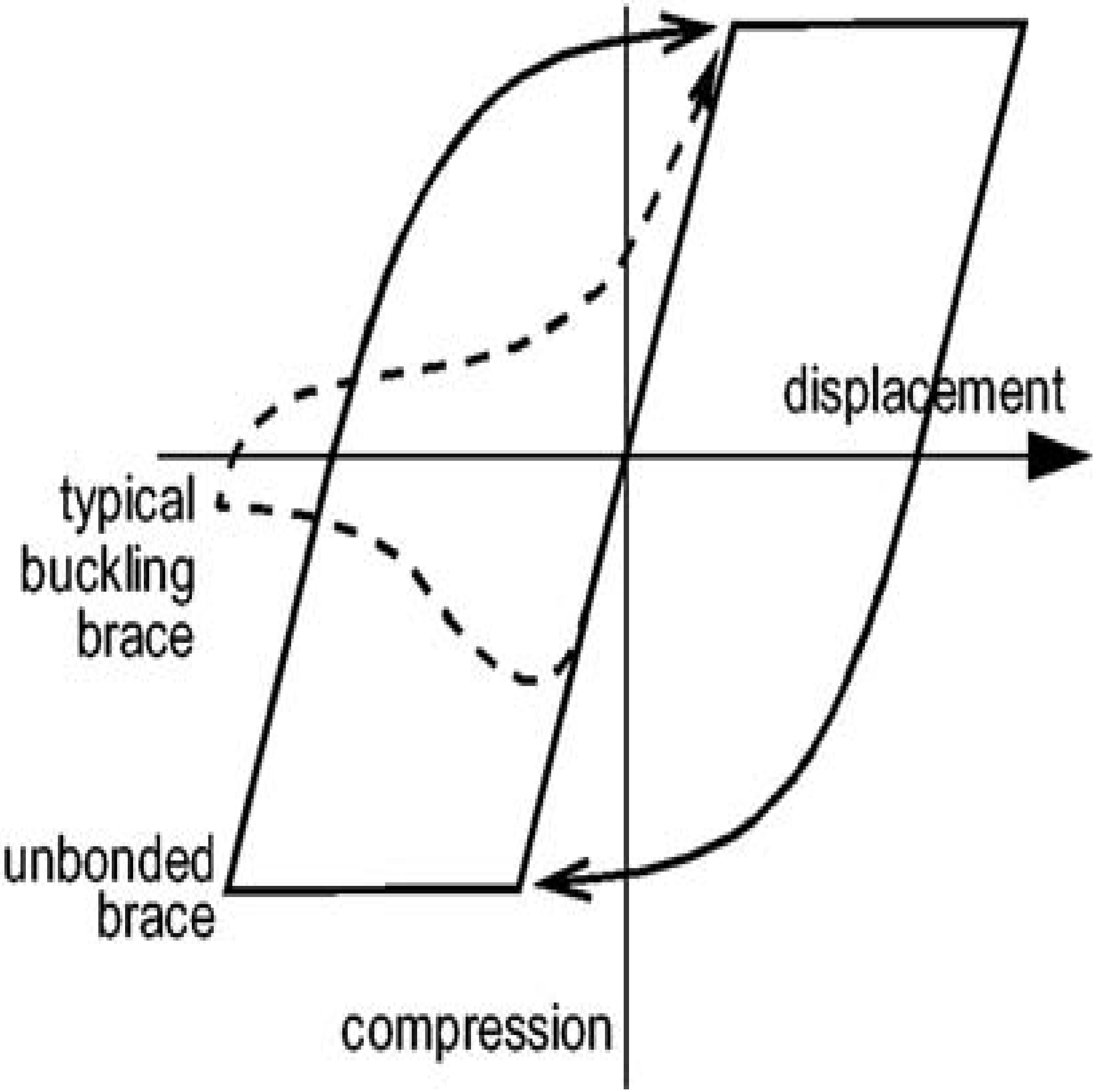
BUCKLING-RESTRAINED BRACED FRAMES (BRBF)

- **1. Scope**
- *BRBF have more ductility and energy absorption than SCBF because overall brace buckling, and its associated strength degradation, is precluded at forces and deformations corresponding to the design story drift.*
- **2. Basis of Design**
- *This section is applicable to frames with specially fabricated braces concentrically connected to beams and columns. Eccentricities less than the beam depth are permitted if the resulting member and connection forces are addressed in the design and do not change the expected source of inelastic deformation capacity.*
- *BRBF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity primarily through brace yielding in tension and compression.*



Bottom
"A"
"B"
"C"
"B"
"A"
Top
"A"
"B"
"C"
"B"
"A"
typ.
Plan
Plate
Mortar
HSS
Plate
Bond-preventing layer
Mortar
HSS
Plate
Bond-preventing layer
Plate
Section "A-A"
Core Projection
Section "B-B"
Transition Segment
Section "C-C"
Yielding Segment

- *In BRBF, the bracing elements dissipate energy through stable tension-compression yield cycles (Clark et al., 1999). Figure C-F4.2 shows the characteristic hysteretic behavior for this type of brace as compared to that of a buckling brace.*
- *This behavior is achieved through limiting buckling of the steel core within the bracing elements. Axial stress is de-coupled from flexural buckling resistance;*
- *axial load is confined to the steel core while the buckling restraining mechanism, typically a casing, resists overall brace buckling and restrains high-mode steel core buckling (rippling).*
- *The steel core within the bracing element is intended to be the primary source of energy dissipation. During a moderate to severe earthquake the steel core is expected to undergo significant inelastic deformations*



Axial force-displacement behavior

- *BRBF can provide elastic stiffness that is comparable to that of EBF.*
- ***The ductility and energy dissipation capability of BRBF is expected to be comparable to that of a SMF and greater than that of a SCBF. This high ductility is attained by limiting buckling of the steel core.***
- *The Provisions are based on the use of brace designs qualified by testing. They are intended to ensure that braces are used only within their proven range of deformation capacity, and that yield and failure modes other than stable brace yielding are precluded at the maximum inelastic drifts corresponding to the design earthquake.*
- ***For analyses performed using linear methods, the maximum inelastic drifts for this system are defined as those corresponding to 200% of the design story drift.***
- *For nonlinear time-history analyses, the maximum inelastic drifts can be taken directly from the analyses results. A minimum of 2% story drift is required for determining expected brace deformations for testing (see Section K3) and is recommended for detailing.*

- *The value of 200% of the design story drift for expected brace deformations represents the mean of the maximum story response for ground motions having a 10% chance of exceedance in 50 years.*
- *Near-fault ground motions, as well as stronger ground motions, can impose deformation demands on braces larger than those required by these provisions.*
- *While exceeding the brace design deformation may result in poor brace behavior such as buckling, this is not equivalent to collapse. Detailing and testing braces for larger deformations will provide higher reliability and better performance.*

- ***2a. Brace Strength***
- *Where required by these Provisions, brace connections and adjoining members shall* ***be designed to resist forces calculated based on the adjusted brace strength.***
- ***The adjusted brace strength in compression shall be***
- $\beta\omega R_y P_{ysc}$,
- where
- β =compression strength adjustment factor
- ω =strain hardening adjustment factor
- P_{ysc} = *axial yield strength of steel core, ksi (MPa)*
- ***The adjusted brace strength in tension shall be***
- $\omega R_y P_{ysc}$,

- The *compression strength adjustment factor, β*, shall be calculated as **the ratio of the maximum compression force to the maximum tension force of the *test specimen* measured** from the qualification tests specified in Section K3.4c for the expected deformations. The larger value of β from the two required brace qualification tests shall be used. In no case shall β be taken as less than 1.0.
- The *strain hardening adjustment factor, ω*, shall be calculated as the **ratio of the maximum tension force measured from the qualification tests to the measured yield force,** $R_y P_{ysc}$ *, of the test specimen.* The larger value of ω from the two required qualification tests shall be used. Where the tested steel core material does not match that of the *prototype, ω shall be* based on coupon testing of the prototype material

- **3. Analysis**
- *Buckling-restrained braces shall not be considered as resisting gravity forces.*
- ***The required strength of columns, beams and connections in BRBF shall be based on the load combinations in the applicable building code that include the amplified seismic load.*** *In determining the amplified seismic load, the effect of horizontal forces including overstrength, E_{mh} , shall be taken as the forces developed in the member assuming the forces in all braces correspond to their adjusted strength in compression or in tension.*
- ***Braces shall be determined to be in compression or tension neglecting the effects of gravity loads.*** *Analyses shall consider both directions of frame loading.*
- *The adjusted brace strength in tension shall be as given in Section F4.2a.*
- *Exceptions:*

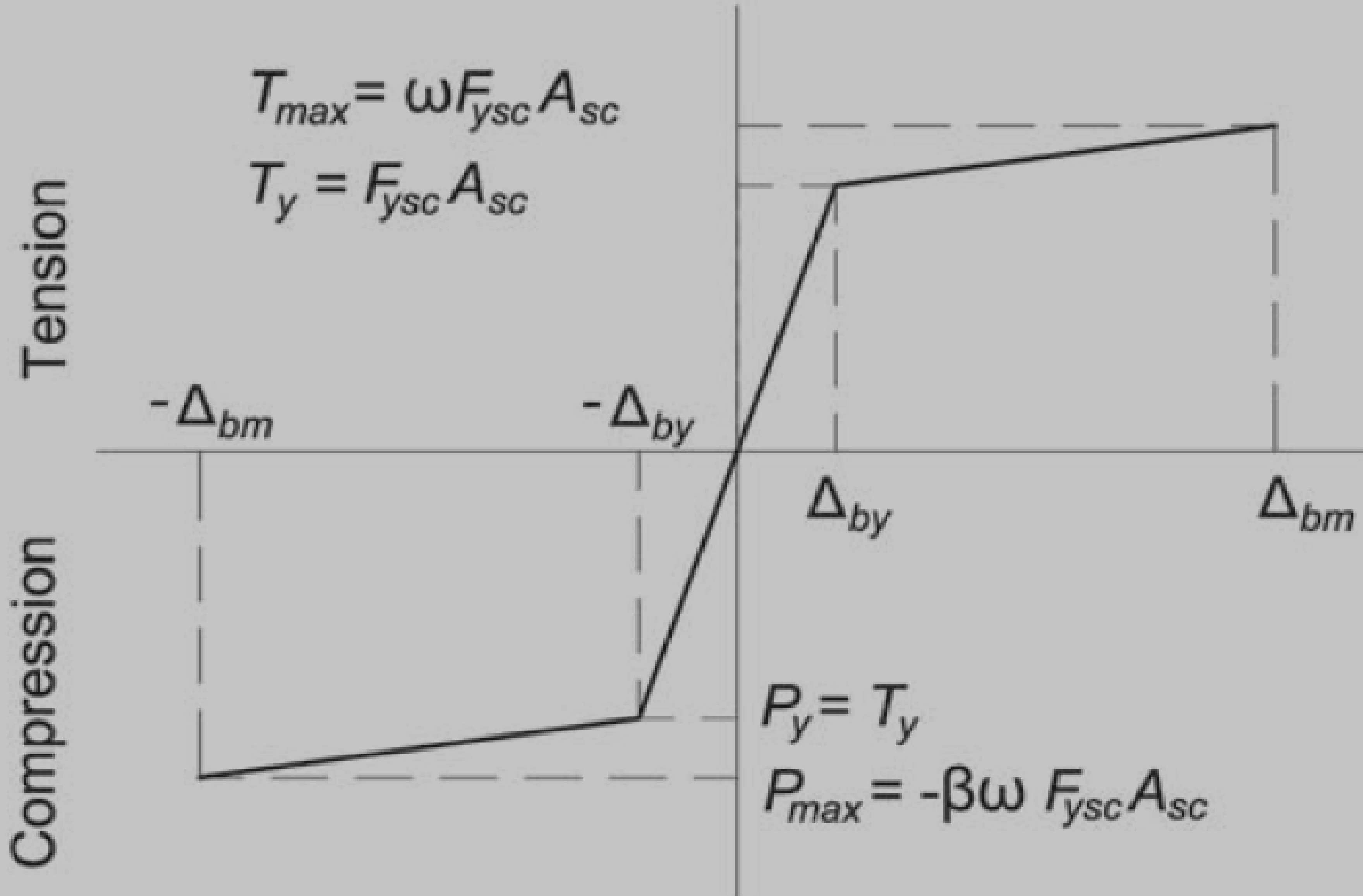


Fig. C-F4.3. Diagram of brace force-displacement.

- *(1) It is permitted to neglect flexural forces resulting from seismic drift in this determination. Moment resulting from a load applied to the column between points of lateral support must be considered.*
- *(2) The required strength of columns need not exceed the lesser of the following:*
- *(a) The forces corresponding to the resistance of the foundation to over turning uplift*
- *(b) Forces as determined from nonlinear analysis as defined in Section C3*
- *The brace deformation shall be determined from the inelastic portion of the design story drift and shall include the effects of beam vertical flexibility. Alternatively, the brace deformation is permitted to be determined from nonlinear analysis as defined in Section C3*

- **4. System Requirements**
- **4a. V- and Inverted V-Braced Frames**
- V-type and inverted-V-type braced frames shall satisfy the following requirements
- (1) The *required strength of beams* intersected by braces, their connections and supporting members shall be determined based on the *load combinations of the* applicable building code assuming that the braces provide no support for dead and live loads. For *load combinations that include earthquake effects, the vertical and horizontal earthquake effect, E, on the beam shall be determined from the* adjusted brace strengths in tension and compression.
- (2) Beams *shall be continuous* between columns. Beams shall be *braced to satisfy the requirements for moderately ductile members in Section D1.2(a).*
- As a minimum, one set of lateral braces is required at the point of intersection of the V-type (or inverted V-type) braces, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

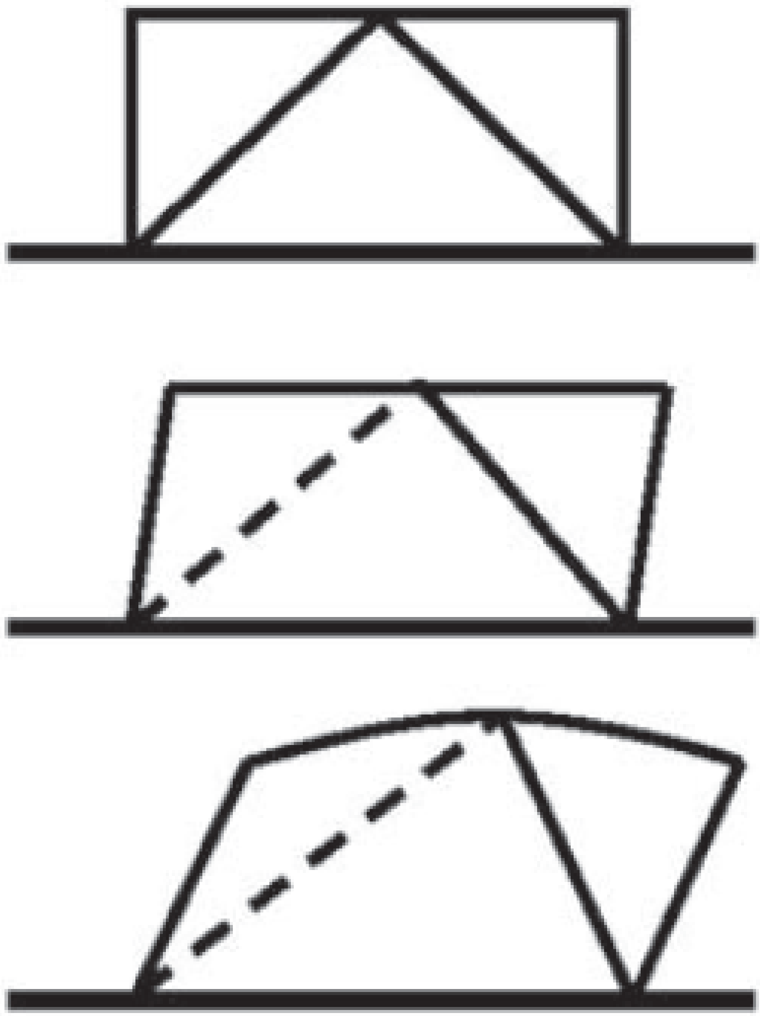


Fig. C-F4.4. Post-yield change in deformation mode for V- and inverted V-BRBF.

- **4b. K-Braced Frames**
- *K-type braced frames are not permitted for BRBF.*
- **5. Members**
- **5a. Basic Requirements**
- *Beam and column members shall satisfy the requirements of Section D1.1 for highly ductile members.*
- **5b. Diagonal Braces**
- ***(1) Assembly***
- *Braces shall be composed of a structural steel core and a system that restrains the steel core from buckling.*
- (1) Steel Core
- ***Plates used in the steel core that are 2 in. (50 mm) thick or greater shall satisfy the minimum notch toughness requirements of Section A3.3.***
- ***Splices in the steel core are not permitted.***

- (2) Buckling-Restraining System
- *The buckling-restraining system shall consist of the casing for the steel core. In stability calculations, beams, columns and gussets connecting the core shall be considered parts of this system.*
- *The buckling-restraining system shall limit local and overall buckling of the steel core for the expected deformations.*
- **(2) Available Strength**
- *The steel core shall be designed to resist the entire axial force in the brace.*
- *The brace design axial strength, φP_{ysc} (LRFD), in tension and compression, in accordance with the limit state of yielding, shall be determined as follows:*
- $P_{ysc} = F_{ysc} A_{sc}$, $\varphi = 0.90$ (LRFD) (F4-1)
- *where*
- A_{sc} = *cross-sectional area of the yielding segment of the steel core, in.^{2} (mm^2)*

- *F_{ysc} = specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon test, ksi (MPa)*
- *Load effects calculated based on adjusted brace strengths should not be amplified by the overstrength factor, Ω_o .*
- **(3) Conformance Demonstration**
- *The design of braces shall be based upon results from qualifying cyclic tests in accordance with the procedures and acceptance criteria of Section K3.*
- *Qualifying test results shall consist of at least two successful cyclic tests: one is required to be a test of a brace subassemblage that includes brace connection rotational demands complying with Section K3.2 and the other shall be either a uniaxial or a subassemblage test complying with Section K3.3. Both test types shall be based upon one of the following:*
- *(a) Tests reported in research or documented tests performed for other projects*
- *(b) Tests that are conducted specifically for the project*

- *Interpolation or extrapolation of test results for different member sizes shall be justified by rational analysis that demonstrates stress distributions and magnitudes of internal strains consistent with or less severe than the tested assemblies and that considers the adverse effects of variations in material properties. Extrapolation of test results shall be based upon similar combinations of steel core and buckling-restraining system sizes. Tests are permitted to qualify a design when the provisions of Section K3 are met.*
- **5c. Protected Zones**
- *The protected zone shall include the steel core of braces and elements that connect the steel core to beams and columns, and shall satisfy the requirements of Section D1.3.*

- **6. Connections**
- **6a. Demand Critical Welds**
- The following welds are *demand critical welds, and shall satisfy the requirements of* Section A3.4b and I2.3:
- *(1) Groove welds at column splices*
- *(2) Welds at the column-to-base plate connections*
- Exception: Where it can be shown that column hinging at, or near, the base plate is precluded by conditions of restraint, and in the absence of net tension under load combinations including the amplified seismic load, demand critical welds are not required.
- **(3) Welds at beam-to-column connections conforming to next Section**.

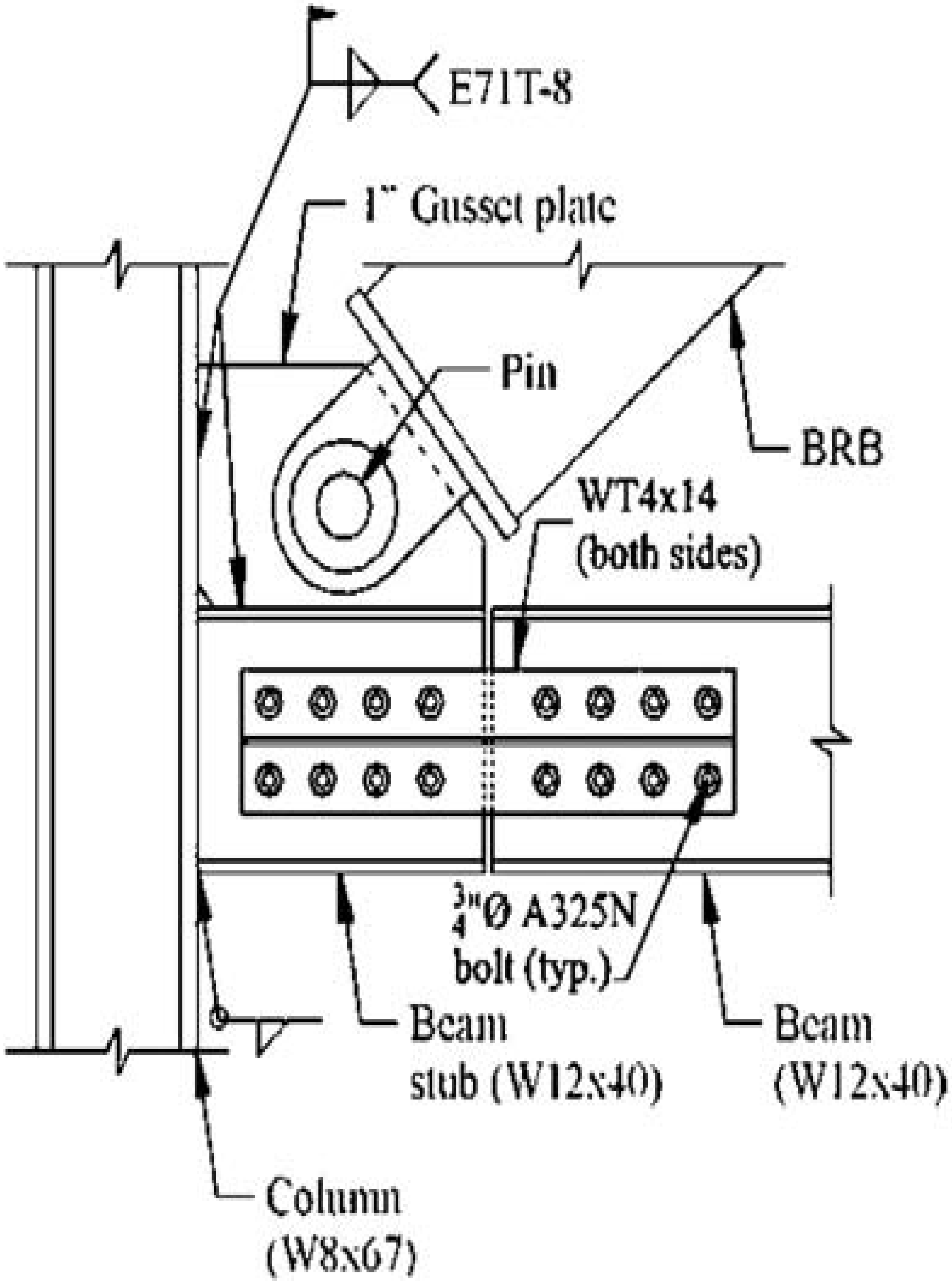
- **6b. Beam-to-Column Connections**
- *Where a brace or gusset plate connects to both members at a beam-to-column connection, the connection shall conform to one of the following:*
- *(a) The connection shall be a simple connection meeting the requirements of Specification Section B3.6a where the required rotation is taken to be 0.025 rad ; or*
- *(b) The connection shall be designed to resist a moment equal to the lesser of the following:*

✓ *(i) A moment corresponding to the expected beam flexural strength multiplied by 1.1 (LRFD), as appropriate. The expected beam flexural strength shall be determined as $R_y M_p$.*

✓ *(ii) A moment corresponding to the sum of expected column flexural strengths multiplied by 1.1 (LRFD), as appropriate. The sum of expected column flexural strengths shall be $\Sigma(R_y F_y Z)$.*

- *This moment shall be considered in combination with the required strength of the brace connection and beam connection, including the amplified diaphragm collector forces.*

- **6c. Diagonal Brace Connections**
- **(1) Required Strength**
- *The required strength of brace connections in tension and compression (including beam-to-column connections if part of the braced-frame system)* ***shall be 1.1 times the adjusted brace strength in compression*** *(LRFD) where the adjusted brace strength is as defined in Section F4.2a.*
- *When oversized holes are used, the required strength for the limit state of bolt slip need not exceed a load effect based upon using the load combinations stipulated by the applicable building code, including the amplified seismic load.*
- **(2) Gusset Plate Requirements**
- *The design of connections shall include considerations of local and overall buckling. Lateral bracing consistent with that used in the tests upon which the design is based is required.*


E71T-8
1" Gusset plate
Pin
BRB
WT4x14
(both sides)
$\frac{3}{4}$"Ø A325N
bolt (typ.)
Beam
stub (W12x40)
Beam
(W12x40)
Column
(W8x67)

- **6d. Column Splices**
- *Column splices shall comply with the requirements of Section D2.5. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds. Column splices shall be designed to develop at least 50% of the lesser available flexural strength of the connected members.*
- The *required shear strength,* V_u *,shall be determined as follows:*
- $V_u=(\Sigma M_{pc}) / H_c$
- H_c *= clear height of the column between beam connections, including a structural slab, if present, in. (mm)*
- ΣM_{pc} *= sum of the nominal plastic flexural strengths,* $F_{yc} Z_c$*, of the columns above and below the splice, kip-in. (N-mm)*

SPECIAL TRUSS MOMENT FRAMES (STMF)

- **1. Scope**
- *Truss-girder moment frames have often been designed with little or no regard for truss ductility. Research has shown that such truss moment frames have very poor hysteretic behavior with large, sudden reductions in strength and stiffness due to buckling and fracture of web members prior to or early in the dissipation of energy through inelastic deformations (Itani and Goel, 1991; Goel and Itani, 1994a). The resulting hysteretic degradation as illustrated in Figure C-E4.1 results in excessively large story drifts in building frames subjected to earthquake ground motions with peak accelerations on the order of 0.4g to 0.5g.*

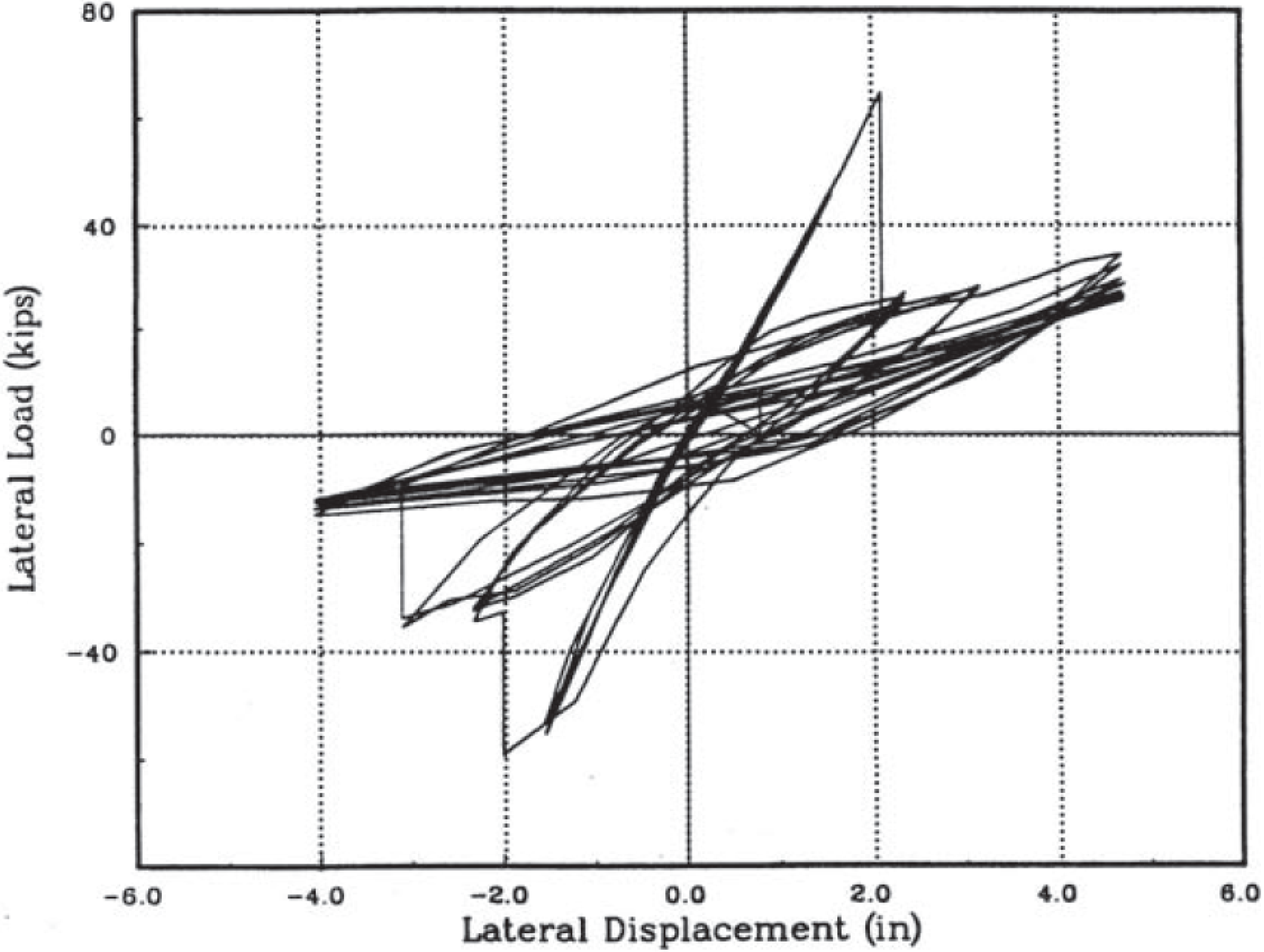


Fig. C-E4.1. Strength degradation in undetailed truss girder.

- *Research led to the development of special truss girders that* ***limit inelastic deformations to a special segment of the truss*** *(Itani and Goel, 1991; Goel and Itani, 1994b; Basha and Goel, 1994). As illustrated in Figure C-E4.2, the chords and web members (arranged in an X pattern) of the special segment are designed to withstand large inelastic deformations, while the rest of the structure remains elastic.*
- *Special truss moment frames (STMF) have been validated by extensive testing of full-scale subassemblages with story-high columns and full-span special truss girders. As illustrated in Figure C-E4.3, STMF are ductile with stable hysteretic behavior. The stable hysteretic behavior continues for a large number of cycles, up to 3% story drifts.*

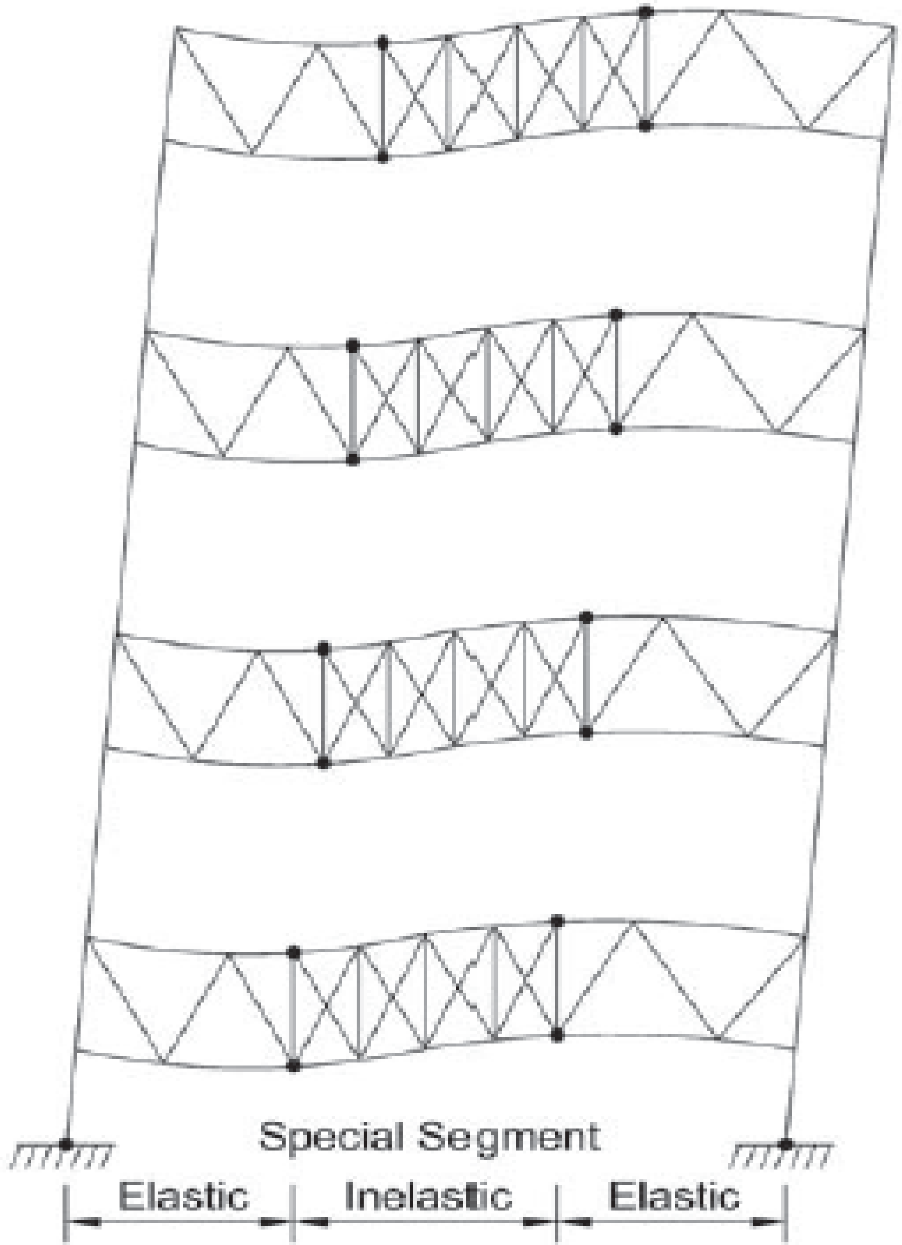


Fig. C-E4.2. Intended yield mechanism of STMF with diagonal web members in special segment.

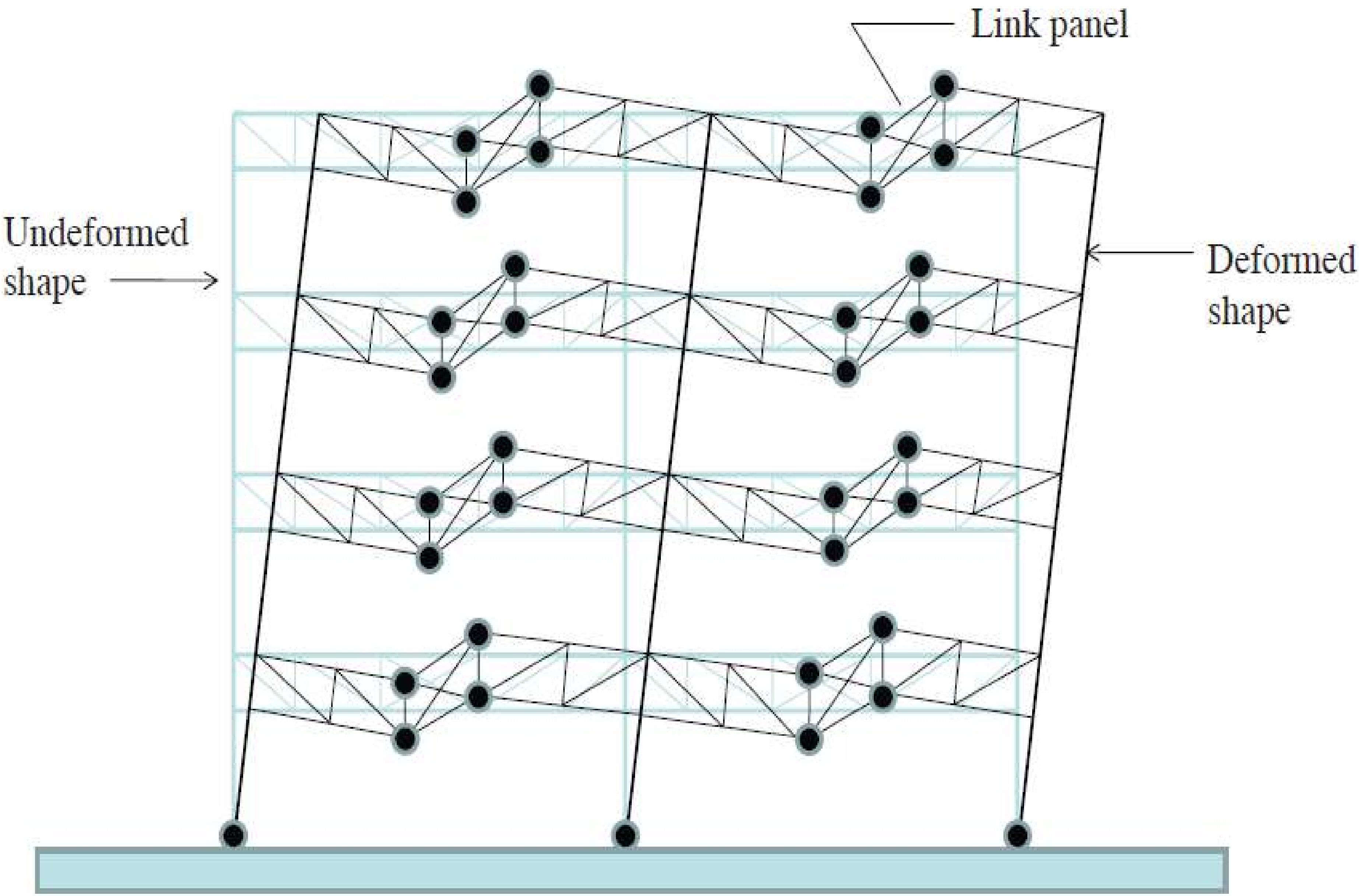


Fig. 6-6. Deformed shape of STMF.

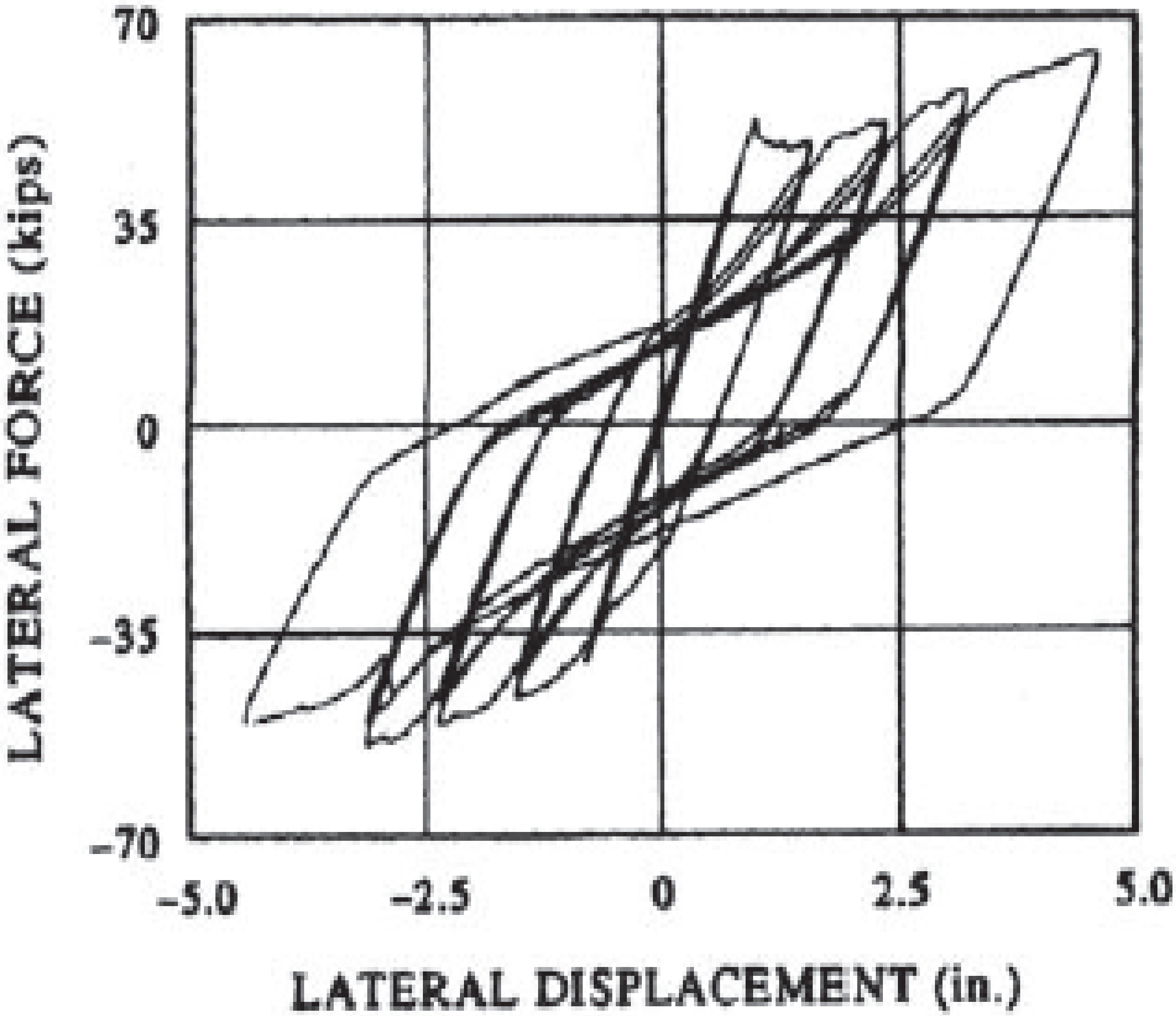


Fig. C-E4.3. Hysteretic behavior of STMF.

- **2. Basis of Design**
- *STMF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity within a special segment of the truss.* ***STMF shall be limited to span lengths between columns not to exceed 65 ft (20 m) and overall depth not to exceed 6 ft (1.8 m).*** ***The columns and truss segments outside of the special segments shall be designed to remain elastic under the forces that can be generated by the fully yielded and strain-hardened special segment.***
- **3. Analysis**
- *Analysis of STMF shall satisfy the following requirements.*
- **3a. Special Segment**
- *The required vertical shear strength of the special segment shall be calculated for the appropriate load combinations in the applicable building code.*

- **3b. Nonspecial Segment**
- The *required strength of nonspecial segment members and connections shall be calculated* based **on the load combinations in the applicable building code that include the *amplified seismic load***. *In determining the amplified seismic load the effect of* horizontal forces including overstrength, E_{mh} *, shall be taken as the lateral forces necessary* to develop the *expected vertical shear strength of the special segment acting* at mid-length and defined in Section E4.5b. Second order effects at maximum design drift shall be included.
- **4. System Requirements**
- **4a. Special Segment**
- *Each horizontal truss that is part of the SFRS shall have a special segment* ***that is located between the quarter points of the span of the truss. The length of the special segment shall be between 0.1 and 0.5 times the truss span length. The length-to-depth ratio of any panel in the special segment shall neither exceed 1.5 nor be less than 0.67.***

- ***Panels within a special segment shall either be all Vierendeel panels or all X-braced panels;*** *neither a combination thereof nor the use of other truss diagonal configurations is permitted. Where diagonal members are used in the special segment, they shall be arranged in an X pattern separated by vertical members.* ***Diagonal members within the special segment shall be made of rolled flat bars of identical sections.*** *Such diagonal members shall be interconnected at points where they cross. The interconnection shall have a required strength equal to 0.25 times the nominal tensile strength of the diagonal member.* ***Bolted connections shall not be used for diagonal members within the special segment.***
- **Splicing of chord members shall not be permitted within the special segment, nor within one-half the panel length from the ends of the special segment.** The *required axial strength of the diagonal web members in the special segment due* to dead and live loads within the special segment shall not exceed $0.03F_y A_g$ *(LRFD)* or $(0.03/1.5)F_y A_g$ *(ASD), as appropriate.*

- **4b. Stability Bracing of Trusses**
- *Each flange of the chord members shall be laterally braced at the ends of the special segment. The required strength of the lateral brace shall be*
- $P_u = 0.06\, R_y\, F_y\, A_f$ *(LRFD)* *(E4-1a)*
- *or*
- $P_a = (0.06/1.5)\, R_y\, F_y\, A_f$ *(ASD)* *(E4-1b)*
- *where*
- A_f *= gross area of the flange of the special segment chord member, in.*^{2} *(mm*^{2}*)*
- **4c. Stability Bracing of Truss-to-Column Connections**
- The columns shall be laterally braced at the levels of top and bottom chords of the trusses connected to the columns. The lateral braces shall have a required strength of
- $P_u = 0.02\, R_y\, P_{nc}$ *(LRFD)* *(E4-2a)*
- Or

- $P_a = (0.02/1.5)\, R_y\, P_{nc}$ *(ASD)* *(E4-2b)*
- where
- P_{nc} = *nominal compressive strength of the chord member at the ends, kips (N)*
- **4d. Stiffness of Stability Bracing**
- *The required brace stiffness shall meet the provisions of Section 6.2 of Appendix 6 of the Specification, where*
- $P_r = R_y\, P_{nc}$ *(LRFD)* *(E4-3a)*
- *or*
- $P_r = R_y\, P_{nc}/1.5$ *(ASD)* *(E4-3b)*
- *where*
- P_r = *required compressive strength, kips (N)*

- **5. Members**
- **5a. Special Segment Members**
- *The available shear strength of the special segment shall be calculated as the sum of the available shear strength of the chord members through flexure, and of the shear strength corresponding to the available tensile strength and 0.3 times the available compressive strength of the diagonal members, when they are used. The top and bottom chord members in the special segment shall be made of identical sections and shall provide at least 25% of the required vertical shear strength.*
- The *available strength,* φP_n *(LRFD) and* P_n/Ω *(ASD), determined in accordance with* the limit state of tensile yielding, shall be equal to or greater than 2.2 times the required strength.
- $\varphi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
- where
- $P_n = F_y A_g$ *(E4-4)*

- **5b. Expected Vertical Shear Strength of Special Segment**
- The expected vertical shear strength of the special segment, V_{ne}*, at mid-length, shall* be:
- $$V_{ne} = \frac{3.6 R_y M_{nc}}{L_s} + 0.036 EI \frac{L}{L_s^{\ 3}} + R_y (P_{nt} + 0.3 P_{nc}) \sin \alpha \qquad \textit{(E4-5)}$$
- *where*
- *E = modulus of elasticity of a chord member of the special segment, ksi (MPa)*
- *I = moment of inertia of a chord member of the special segment, in.*^{4} *(mm*^{4}*)*
- *L = span length of the truss, in. (mm)*
- L_s *= length of the special segment, in. (mm)*
- M_{nc} *= nominal flexural strength of a chord member of the special segment, kip-in.*
- *(N-mm)*
- P_{nt} *= nominal tensile strength of a diagonal member of the special segment, kips*
- *(N)*
- P_{nc} *= nominal compressive strength of a diagonal member of the special segment,*
- *kips (N)*
- R_y *= ratio of the expected yield stress to the specified minimum yield stress*
- α*= angle of diagonal members with the horizontal, degrees*

- **5c. Width-to-Thickness Limitations**
- *Chord members and diagonal web members within the special segment shall satisfy the requirements of Section D1.1b for highly ductile members.* ***The width-to-thickness ratio of flat bar diagonal members shall not exceed 2.5.***
- **5d. Built-Up Chord Members**
- ***Spacing of stitching for built-up chord members in the special segment shall not exceed*** $0.04Er_y/F_y$, *where* r_y *is the radius of gyration of individual components about their weak axis.*
- **5e. Protected Zones**
- *The region at each end of a chord member within the special segment shall be designated as a protected zone meeting the requirements of Section D1.3.* ***The protected zone shall extend over a length equal to two times the depth of the chord member from the connection with the web members. Vertical and diagonal web members from end-to-end of the special segments shall be protected zones.***

- **6. Connections**
- **6a. Demand Critical Welds**
- *The following welds are demand critical welds, and shall satisfy the requirements of Section A3.4b and I2.3:*
- *(1) Groove welds at column splices*
- *(2) Welds at column-to-base plate connections*
- *Exception: Where it can be shown that column hinging at, or near, the base plate is precluded by conditions of restraint, and in the absence of net tension under load combinations including the amplified seismic load, demand critical welds are not required.*
- **6b. Connections of Diagonal Web Members in the Special Segment**
- *The end connection of diagonal web members in the special segment shall have a required strength that is at least equal to the expected yield strength of the web member multiplied by 1.0 (LRFD) or divided by 1.5 (ASD), as appropriate. The expected yield strength of the web member shall be determined as* $R_y F_y A_g$.

- **6c. Column Splices**
- *Column splices shall comply with the requirements of Section D2.5. Where welds are used to make the splice, they shall be complete-joint-penetration groove welds. When bolted column splices are used, they shall have a required flexural strength that is at least equal to $R_y F_y Z_x$ (LRFD) or $R_y F_y Z_x$ /1.5 (ASD), as appropriate, of the smaller column. The required shear strength of column web splices shall be at least equal to ΣM_{pc} /H (LRFD) or ΣM_{pc} /(1.5H) (ASD), as appropriate, where ΣM_{pc} is the sum of the nominal plastic flexural strengths of the columns above and below the splice.*
- *Exception: The required strength of the column splice considering appropriate stress concentration factors or fracture mechanics stress intensity factors need not exceed that determined by a nonlinear analysis as specified in Chapter C.*

ORDINARY CANTILEVER COLUMN SYSTEMS (OCCS)

- **1. Scope**
- *Ordinary cantilever column systems (OCCS) of structural steel shall be designed in* conformance with this section.
- **2. Basis of Design**
- *OCCS designed in accordance with these provisions are expected to provide minimal inelastic drift capacity through flexural yielding of the columns.*
- *OCCS are intended to provide a minimal level of inelastic rotation capability at the base of the column. This system is permitted in seismic design categories B and C only, and to heights not exceeding 35 ft. A low seismic response modification coefficient, R, of 1.25 is assigned due to the system's limited inelastic capacity and lack of redundancy*
- **3. Analysis**
- *There are no additional analysis requirements.*


- **4. System Requirements**
- **4a. Columns**
- *Columns shall be designed using the load combinations including the amplified seismic load. The required axial strength, P_{rc}, shall not exceed 15% of the available axial strength, P_c, for these load combinations only.*
- **4b. Stability Bracing of Columns**
- *There are no additional stability bracing requirements for columns.*
- **5. Members**
- **5a. Basic Requirements**
- *There are no additional requirements*
- **5b. Column Flanges**
- *There are no additional column flange requirements.*
- **5c. Protected Zones**
- *There are no designated protected zones.*

- **6. Connections**
- **6a. Demand Critical Welds**
- *No demand critical welds are required for this system.*
- **6b. Column Bases**
- *There are no additional column base requirements*

SPECIAL CANTILEVER COLUMN SYSTEMS (SCCS)

- **1. Scope**
- *Special cantilever column systems (SCCS) of structural steel shall be designed in conformance with this section.*
- **2. Basis of Design**
- *SCCS designed in accordance with these provisions are expected to provide limited inelastic drift capacity through flexural yielding of the columns.*
- *The SCCS is intended to provide a limited level of inelastic rotation capability at the base of the column. This system is permitted in seismic design categories B thru F, but is limited to heights not exceeding 35 ft. A relatively low seismic response modification coefficient, R, of 2.5 is assigned due to the system's limited inelastic capacity and lack of redundancy.*
- **3. Analysis**
- *There are no additional analysis requirements.*

- **4. System Requirements**
- **4a. Columns**
- *Columns shall be designed using the load combinations including the amplified seismic load. The required strength, P_{rc} , shall not exceed 15% of the available axial strength, P_c , for these load combinations only.*
- *Columns in SCCS would be prone to P-Delta collapse if high axial loads were permitted because even modest rotations at the base of the columns can translate into significant drift at the top where the majority of the gravity load is generally applied.*
- **4b. Stability Bracing of Columns**
- *Columns shall be braced to satisfy the requirements applicable to beams classified as moderately ductile members in Section D1.2a.*
- *Although the columns themselves must satisfy requirements for highly ductile members, the wider spacing of braces permitted is considered to be adequate because of the relatively low inelastic demand expected and the practical difficulty in achieving bracing in many of these structures.*

- **5. Members**
- **5a. Basic Requirements**
- *Column members shall satisfy the requirements of Section D1.1 for highly ductile members.*
- *The intention is to preclude local buckling at the hinging location (bottom of the column), which in this type of structure, with little redundancy, could lead rapidly to collapse.*
- **5b. Column Flanges**
- *Abrupt changes in column flange area are prohibited in the protected zone as designated in Section E6.5c.*
- **5c. Protected Zones**
- *The **region at the base of the column subject to inelastic straining shall be designated as a protected zone**, and shall satisfy the requirements of Section D1.3. The length of the protected zone shall be **two times the column depth**, unless otherwise substantiated by testing.*

- **6. Connections**
- **6a. Demand Critical Welds**
- *The following welds are demand critical welds, and shall satisfy the requirements of Section A3.4b and I2.3:*
- ***(1) Groove welds at column splices***
- ***(2) Welds at column-to-base plate connections***
- **6b. Column Bases**
- *Column bases shall be designed in accordance with Section D2.6.*

COMPOSITE ORDINARY BRACED FRAMES (C-OBF)

- *Composite braced frames consisting of steel, composite and/or reinforced concrete elements have been used in low- and high-rise buildings in regions of low and moderate seismicity. The composite ordinary braced frame (C-OBF) category is provided for systems without special seismic detailing that are used in seismic design categories A, B and C. Thus, the C-OBF systems are considered comparable to structural steel systems that are designed according to the Specification using a seismic response factor of R = 3. Because significant inelastic load redistribution is not relied upon in the design, there is no distinction between frames where braces frame concentrically or eccentrically into the beams and columns.*

- **1. Scope**
- *Composite ordinary braced frames (C-OBF) shall be designed in conformance with this section. Columns shall be structural steel, encased composite, filled composite or reinforced concrete members. Beams shall be either structural steel or composite beams. Braces shall be structural steel or filled composite members. This section is applicable to braced frames that consist of concentrically connected members where at least one of the elements (columns, beams or braces) is a composite or reinforced concrete member.*
- **2. Basis of Design**
- *This section is applicable to braced frames that consist of concentrically connected members. Eccentricities less than the beam depth are permitted if they are accounted for in the member design by determination of eccentric moments. C-OBF designed in accordance with these provisions are expected to provide limited inelastic deformations in their members and connections. C-OBF shall satisfy the requirements of Section F1, except as modified in this section.*

- **3. Analysis**
- *There are no additional analysis requirements.*
- **4. System Requirements**
- *There are no additional system requirements.*
- **5. Members**
- **5a. Basic Requirements**
- *There are no additional requirements.*
- **5b. Columns**
- *There are no additional requirements for structural steel and composite columns. Reinforced concrete columns shall satisfy the requirements of ACI 318, excluding Chapter 21.*
- **5c. Braces**
- *There are no additional requirements for structural steel and filled composite braces.*
- **5d. Protected Zones**
- *There are no designated protected zones.*

- **6. Connections**
- *Connections shall satisfy the requirements of Section D2.7.*
- **6a. Demand Critical Welds**
- *There are no requirements for demand critical welds.*

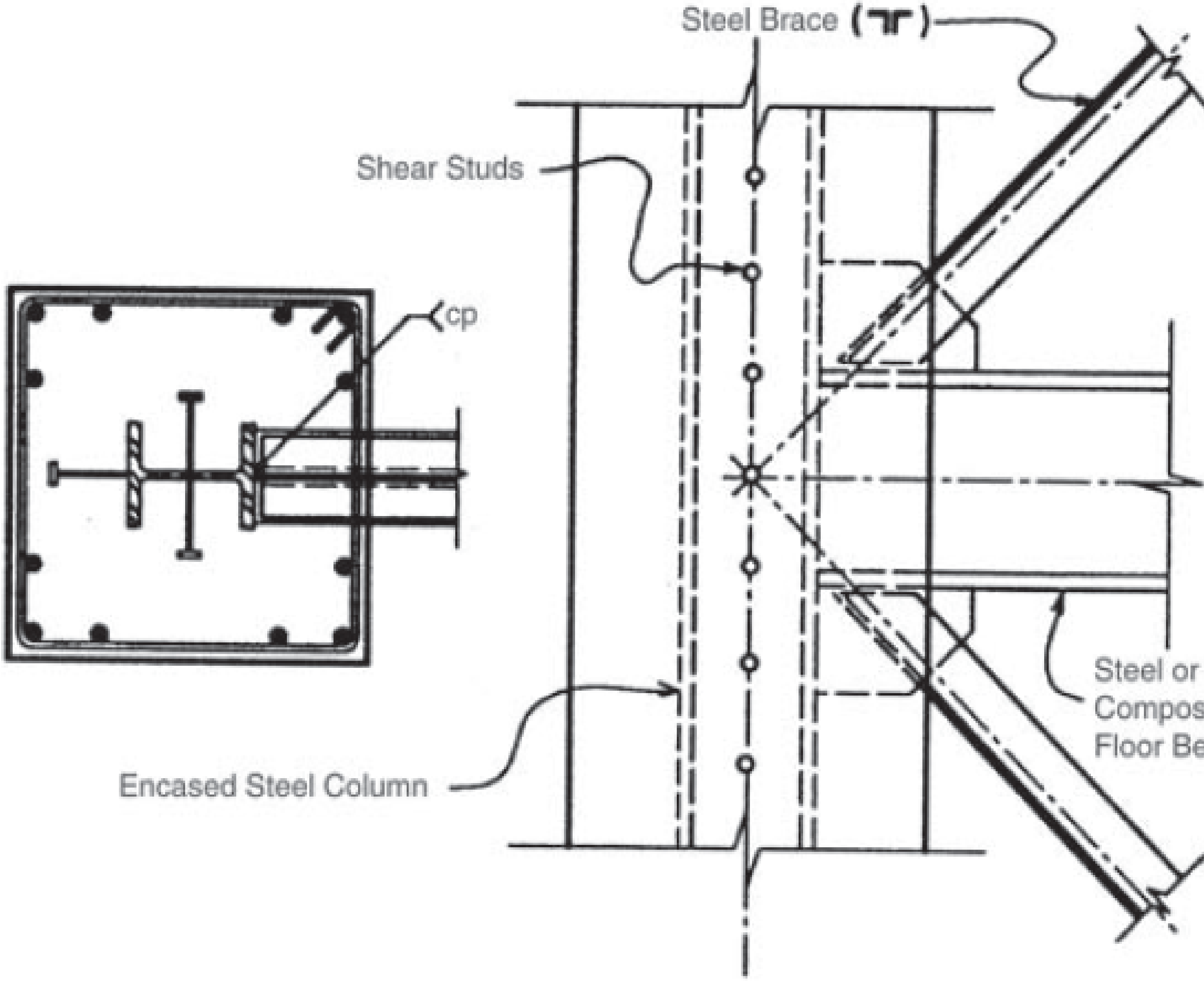


Fig. C-H1.1. Reinforced concrete (or composite) column-to-steel concentric brace.

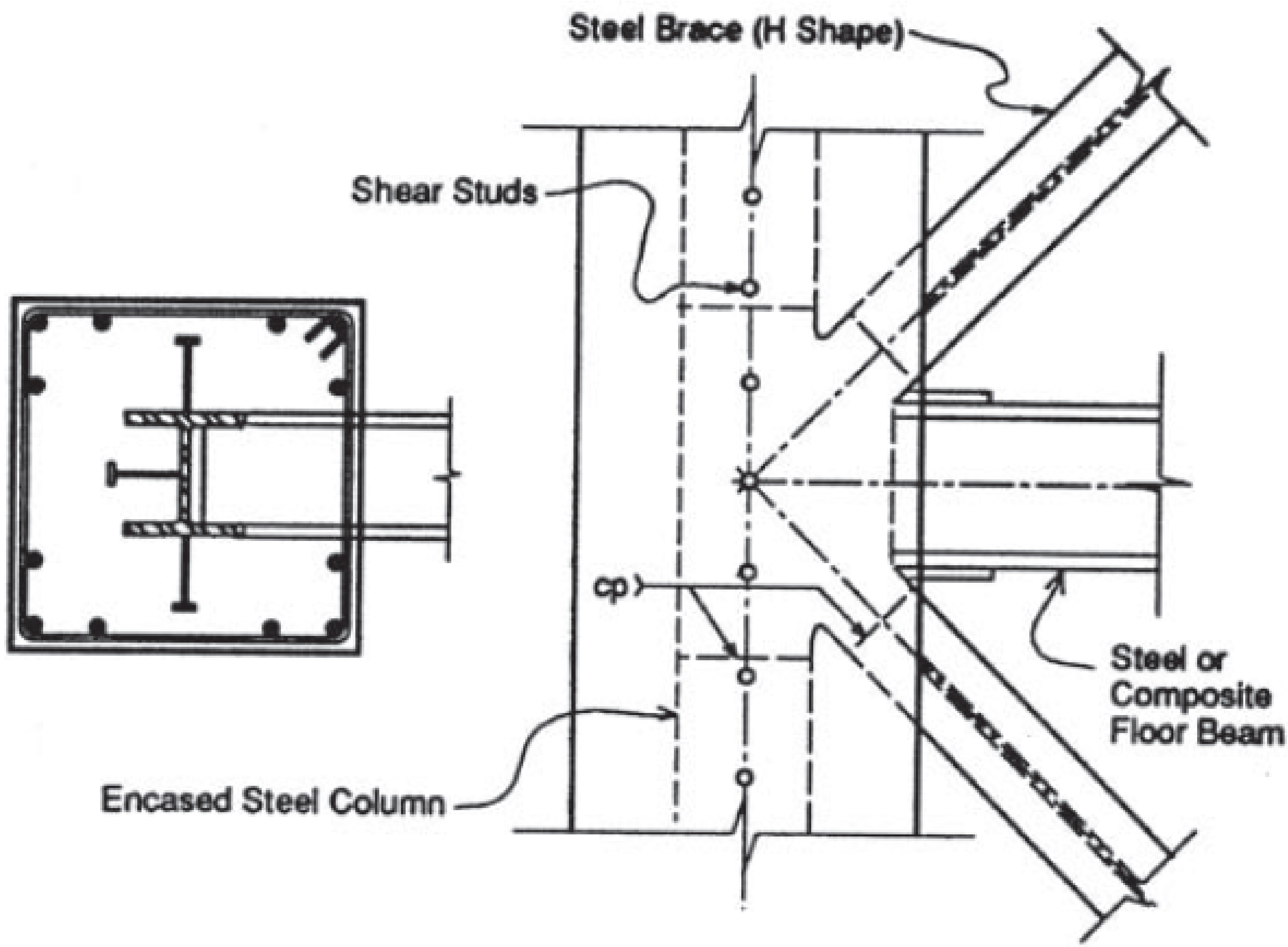


Fig. C-H1.2. Reinforced concrete (or composite) column-to-steel concentric brace.

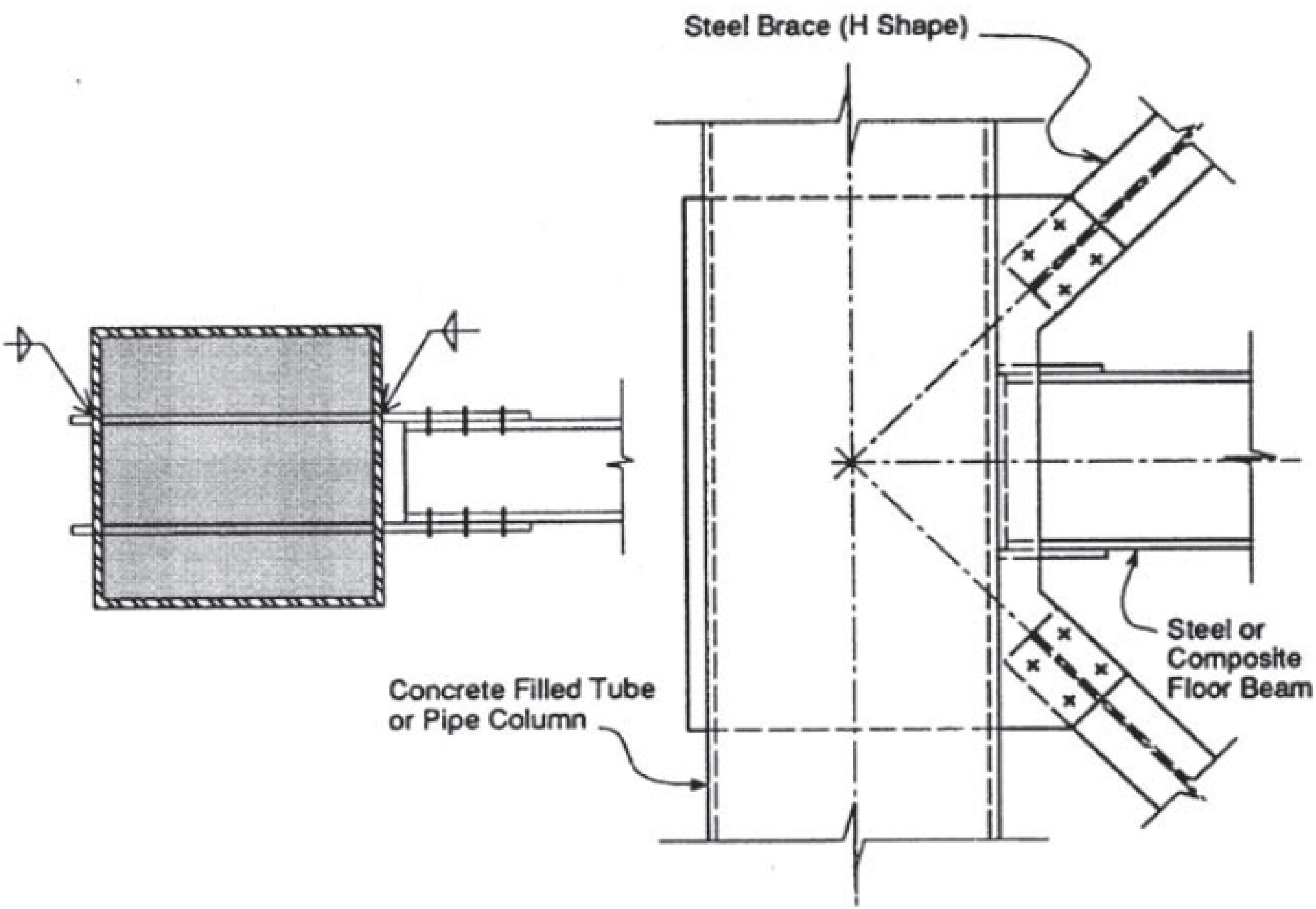


Fig. C-H1.3. Filled HSS or pipe column-to-steel concentric base.

COMPOSITE SPECIAL CONCENTRICALLY BRACED FRAMES (C-SCBF)

- *The composite special concentrically braced frame (C-SCBF) is one of two types of composite braced frames that are specially detailed for seismic design categories D and above; the other is the composite eccentrically braced frame (C-EBF). While experience using C-SCBF is limited in high seismic regions, the design provisions for C-SCBF are intended to provide behavior that is comparable to steel SCBF, wherein the braces often are the elements most susceptible to inelastic deformations (see Commentary Section F2). The R and C_d values and usage limitations for C-SCBF are similar to those for steel SCBF.*

- **1. Scope**
- *Composite special concentrically braced frames (C-SCBF) shall be designed in conformance with this section. Columns shall be encased or filled composite. Beams shall be either structural steel or composite beams. Braces shall be structural steel or filled composite members. This section is applicable to braced frames that consist of concentrically connected members.*
- **2. Basis of Design**
- *This section is applicable to braced frames that consist of concentrically connected members. Eccentricities less than the beam depth are permitted if the resulting member and connection forces are addressed in the design and do not change the expected source of inelastic deformation capacity.*
- *C-SCBF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity primarily through brace buckling and yielding of the brace in tension.*

- **3. Analysis**
- *The analysis requirements for C-SCBF shall satisfy the analysis requirements of Section F2.3.*
- **4. System Requirements**
- *The system requirements for C-SCBF shall satisfy the system requirements of Section F2.4.*
- **5. Members**
- **5a. Basic Requirements**
- *Composite columns and steel or composite braces shall satisfy the requirements of Section D1.1 for highly ductile members. Steel or composite beams shall satisfy the requirements of Section D1.1 for moderately ductile members.*
- *In order to satisfy the compactness requirement of Section F2.5a the actual width-to-thickness ratio of square and rectangular filled composite braces may be multiplied by a factor, [(0.264 + 0.0082KL/r)], for KL/r between 35 and 90; KL/r being the effective slenderness ratio of the brace.*

- **5b. Diagonal Braces**
- *Structural steel and filled composite braces shall satisfy the requirements for SCBF of Section F2.5b. The radius of gyration in Section F2.5b shall be taken as that of the steel section alone.*
- **5c. Protected Zones**
- There are no designated *protected zones.*
- **6. Connections**
- *Design of connections in C-SCBF shall be based on Section D2 and the provisions of this section.*
- **6a. Demand Critical Welds**
- *The following welds are demand critical welds, and shall satisfy the requirements of Section A3.4b and I2.3:*
- *(1) Groove welds at column splices*
- *(2) Welds at the column-to-base plate connections*

- *Exception: Where it can be shown that column hinging at, or near, the base plate is precluded by conditions of restraint, and in the absence of net tension under load combinations including the amplified seismic load, demand critical welds are not required.*
- *(3) Welds at beam-to-column connections conforming to Section H2.6b(b)*
- **6b. Beam-to-Column Connections**
- *Where a brace or gusset plate connects to both members at a beam-to-column connection, the connection shall conform to one of the following:*
- *(a) The connection shall be a simple connection meeting the requirements of Specification Section B3.6a where the required rotation is taken to be 0.025 rad ;*
- *or*
- *(b) Beam-to-column connections shall satisfy the requirements for FR moment connections as specified in Sections D2, G2.6d and G2.6e.*

- *The required flexural strength of the connection shall be determined from analysis and be considered in combination with the required strength of the brace connection and beam connection, including the amplified diaphragm collector forces.*
- **6c. Required Strength of Brace Connections**
- *The required strength of brace connections shall satisfy the requirements of Section F2.6c.*
- **6d. Column Splices**
- *Column splices shall be designed following the requirements of Section G2.6f.*

COMPOSITE ORDINARY SHEAR WALLS (C-OSW)

- **1. Scope**
- *Composite ordinary shear walls (C-OSW) shall be designed in conformance with this section. This section is applicable when reinforced concrete walls are composite with structural steel elements, including structural steel or composite sections acting as boundary members for the walls and structural steel or composite coupling beams that connect two or more adjacent reinforced concrete walls. Examples of such systems are shown in Figure C-H4.1.*
- *This Section also applies to coupled wall systems with steel or composite coupling beams connecting two or more adjacent walls (see Figure C-H4.2). In this case, the walls may or may not have structural steel or composite sections serving as boundary elements. Structural steel or composite boundary elements may be used as wall boundary elements or for erection purposes only*

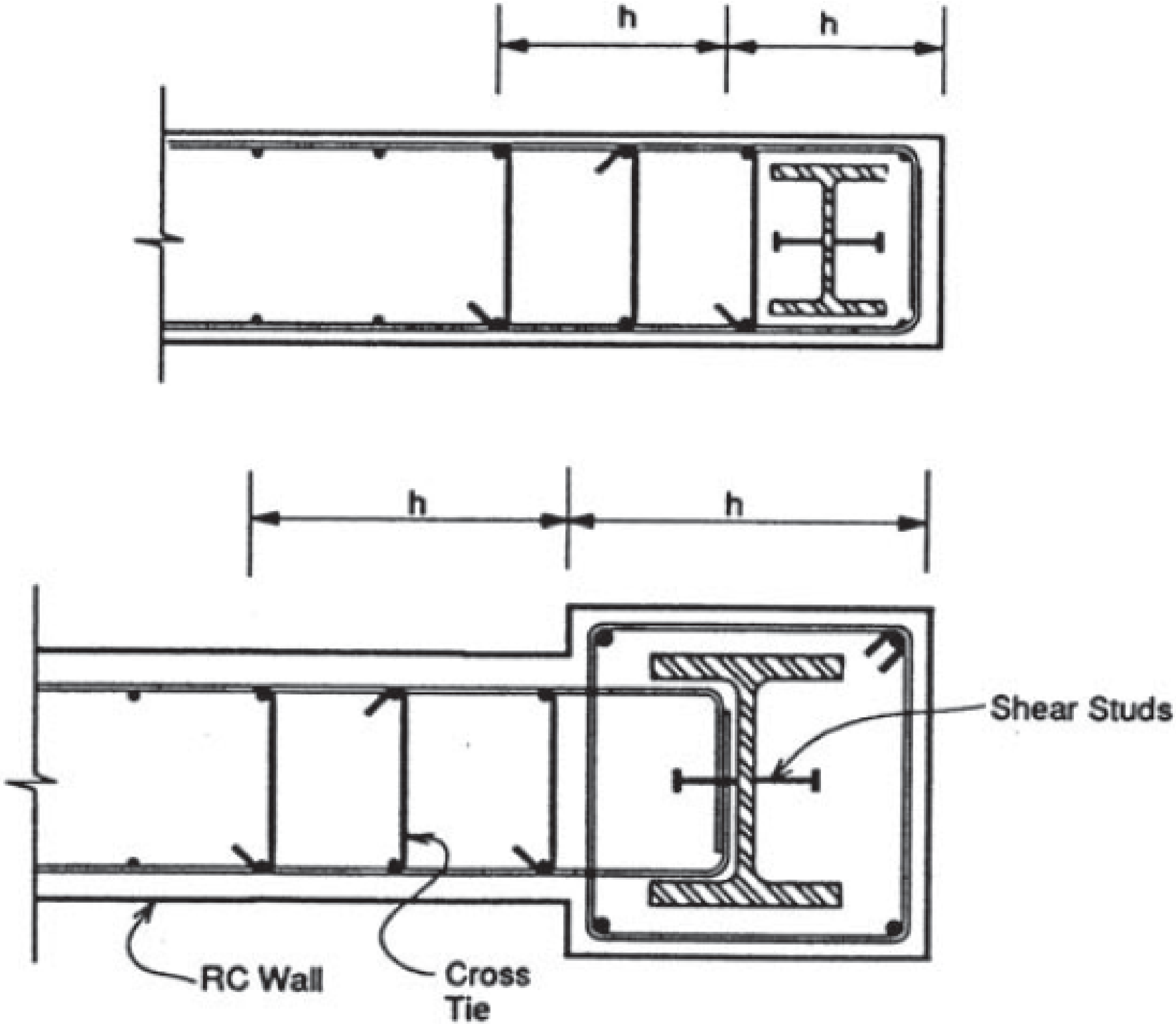


Fig. C-H4.1. Reinforced concrete walls with steel and composite boundary element.

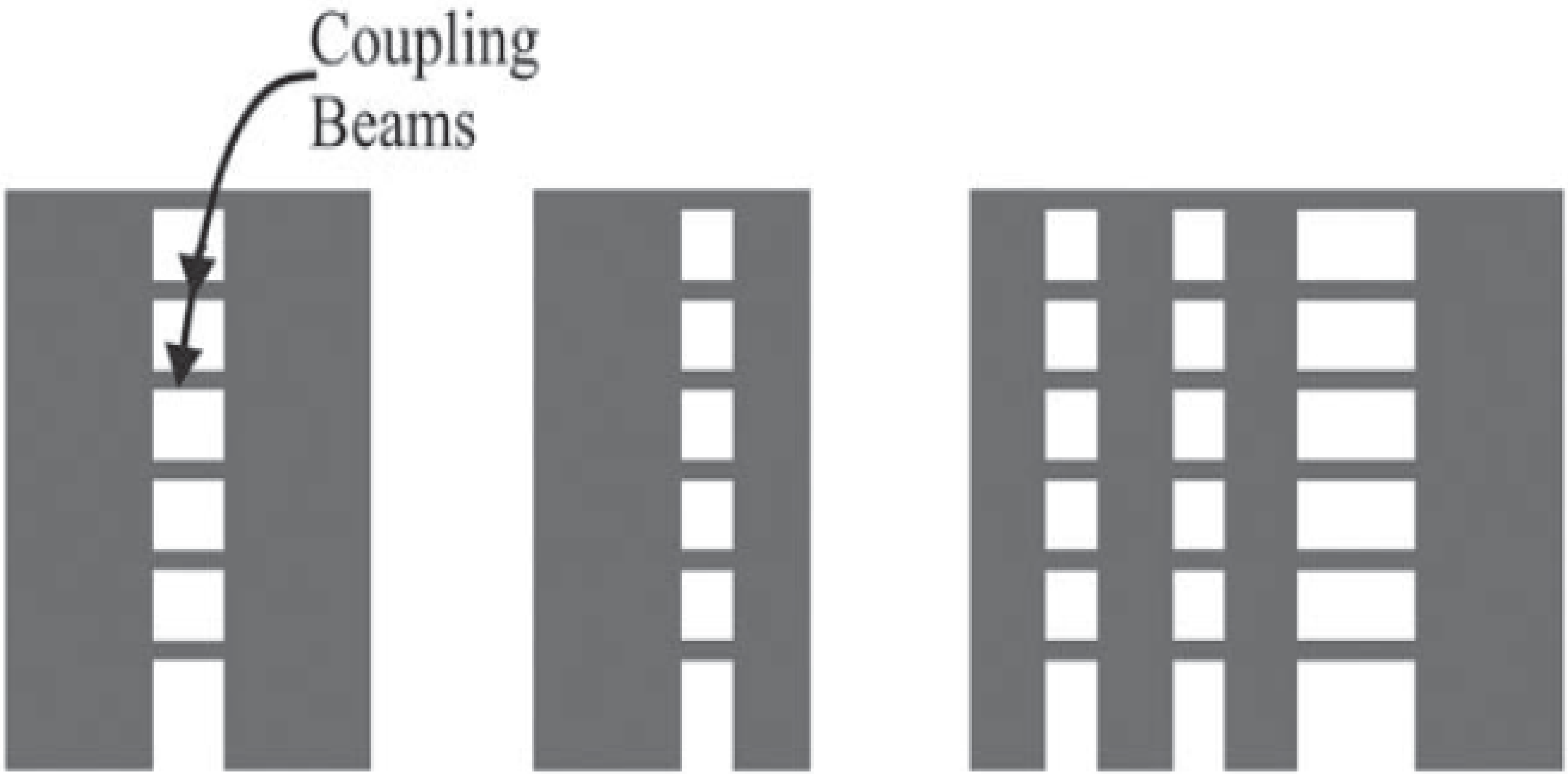
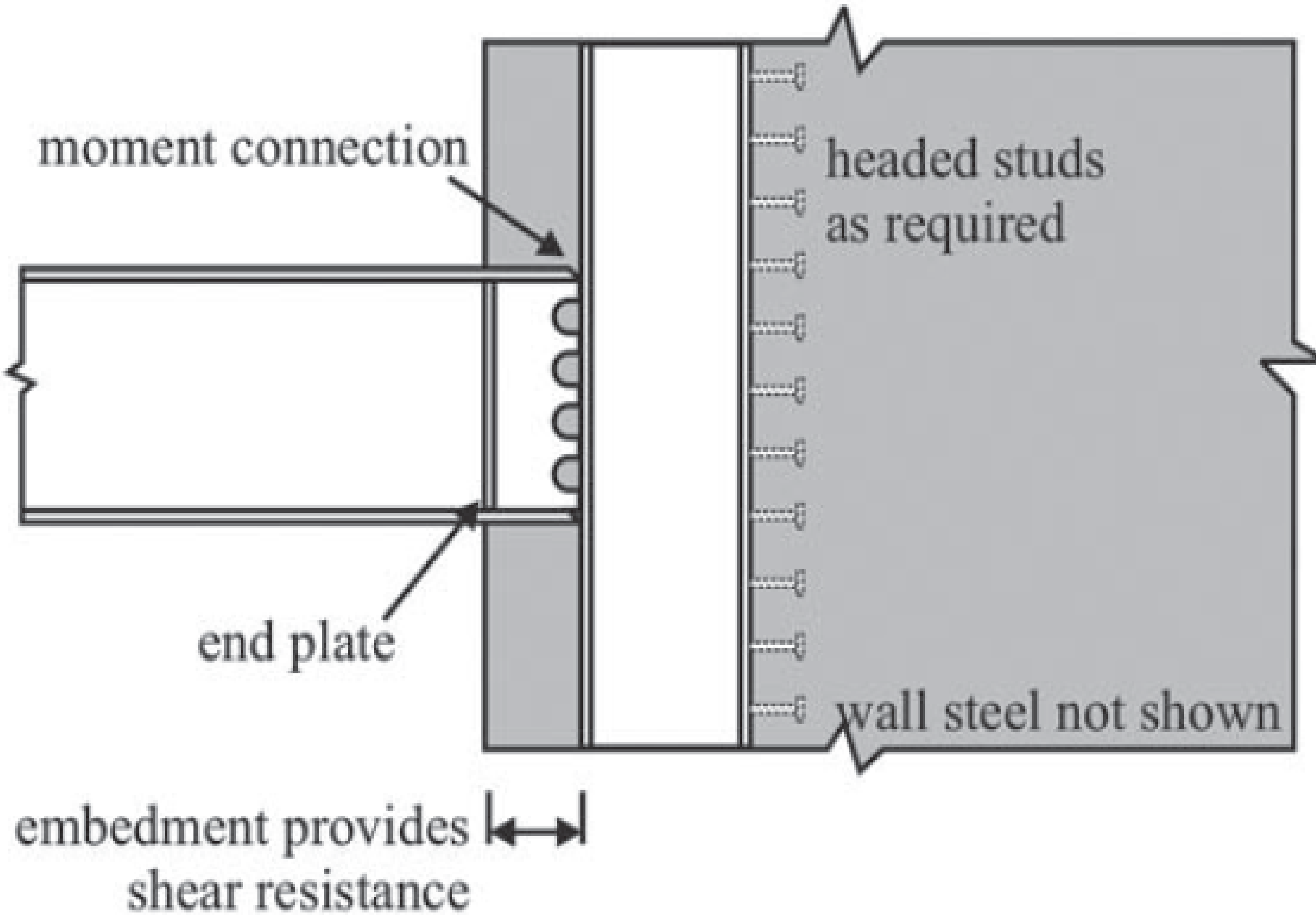
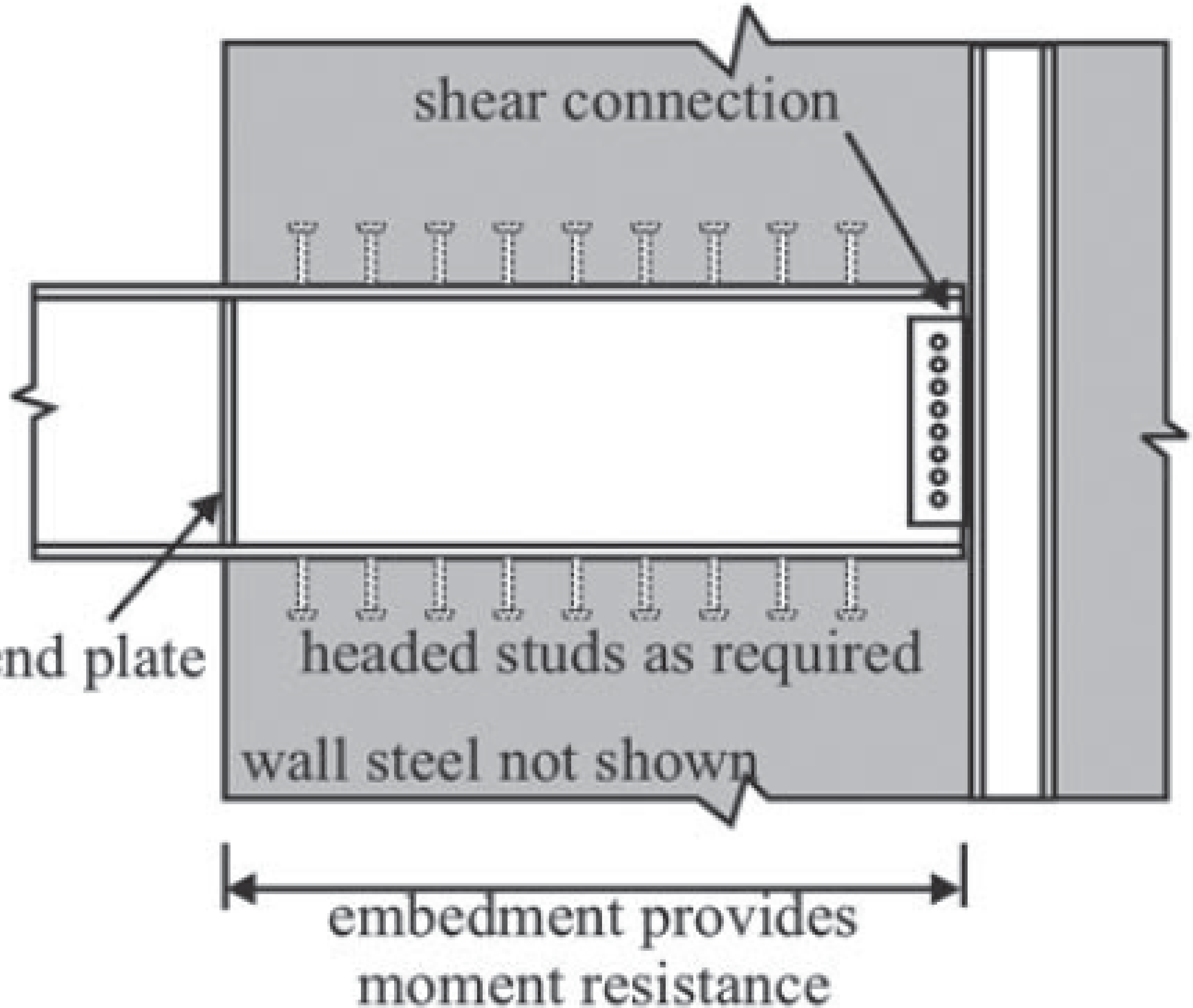


Fig. C-H4.2. Examples of coupled wall geometry.

- *In the latter case, the structural steel members may be relatively small. The detailing of coupling beam-to-wall connections depends on whether structural shapes are embedded in the wall boundaries or the wall has conventional reinforced concrete boundary elements. If steel or composite column boundary elements are used, the coupling beams can frame into the columns and transmit the coupling forces through a moment resisting connection with the steel column (see Figure C-H4.3(a)). The use of a moment connection is, however, not preferred given the cost and difficulty of constructing ductile connections. Alternatively, the coupling beam may be connected to the embedded boundary column with a shear connection while the moment resistance is achieved by a combination of bearing along the embedment length and shear transfer provided by steel headed stud anchors along the coupling beam flanges. In such cases, special reinforcement detailing in the wall boundary region similar to that found in reinforced concrete walls is required. An example is shown in Figure C-H4.3(b).*

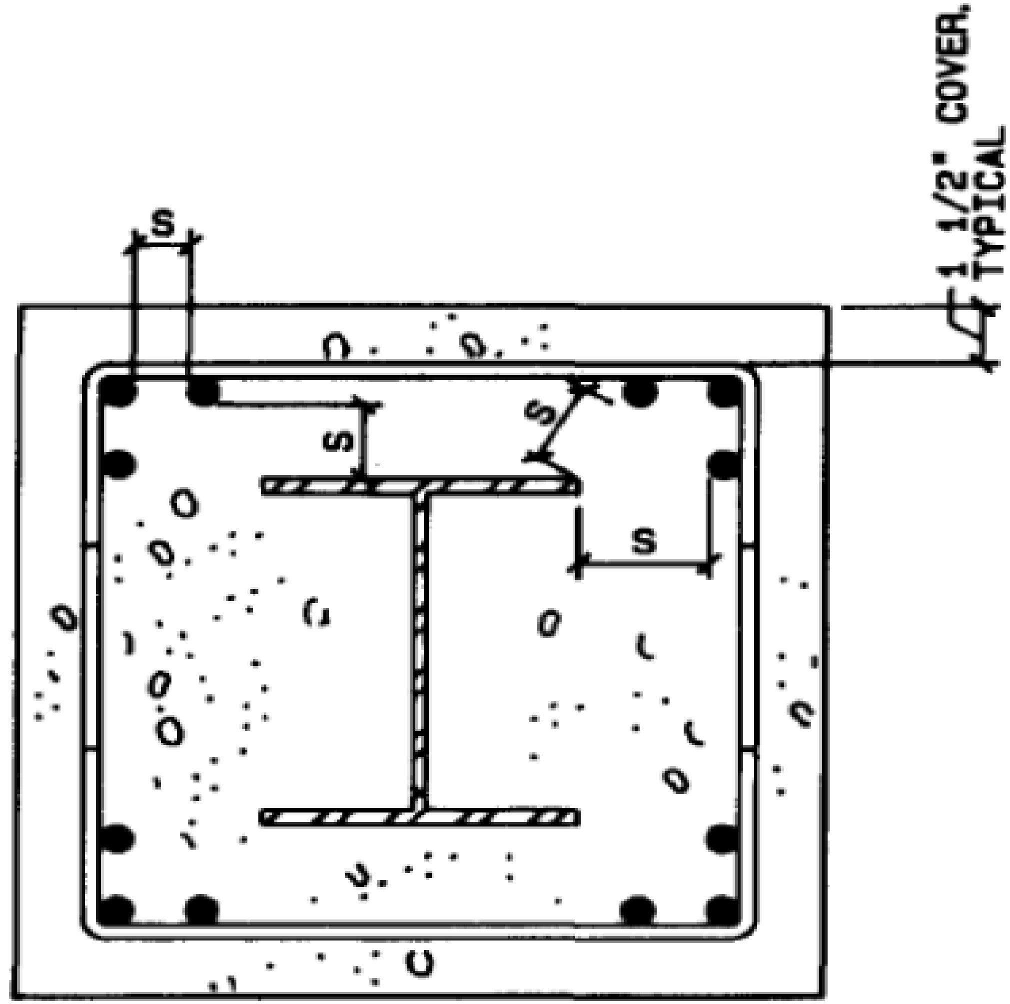


(a) Steel coupling beam attached to steel wall boundary element column



(b) Steel coupling beam attached to steel erection column

Fig. C-H4.3. Steel coupling beam details.

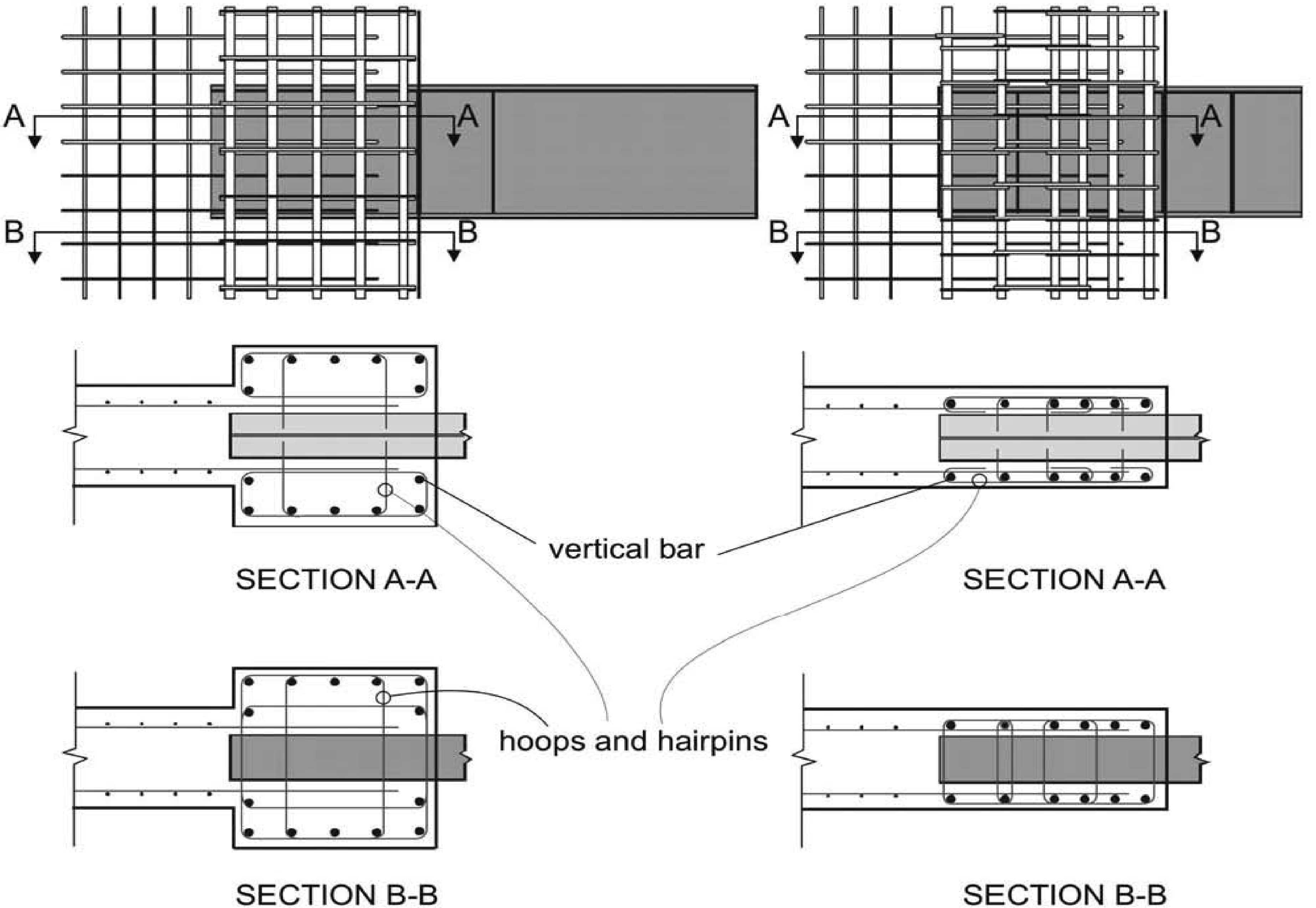


S=CLEAR DISTANCE BETWEEN BARS OR CLEAR DISTANCE BETWEEN ANY BAR AND FACE OF W SHAPE

S $\geq$ 1 1/2xd_b OR 1 1/2", WHICHEVER IS GREATER

d_b =BAR DIAMETER

- *If structural steel or composite boundary elements are not present, the coupling beam should be embedded a sufficient distance into the wall so that the coupling forces are transmitted entirely through the interaction that occurs between the embedded coupling beam and the surrounding concrete. Examples of such embedment regions are as shown in Figure C-H4.4.*
- *It is not necessary, nor is it typically practical, to pass wall boundary transverse reinforcing bars through the web of the embedded coupling beam. A practical alternative is to place hooked ties on either side of the web and provide short vertical bars between the flanges to anchor these ties.*



(a) steel coupling beam framing into "barbell" wall boundary region

(b) steel coupling beam framing into rectangular wall boundary region

- **2. Basis of Design**
- *C-OSW designed in accordance with these provisions are expected to provide limited inelastic deformation capacity through yielding in the reinforced concrete walls and the steel or composite elements. Reinforced concrete wall elements shall be designed to provide inelastic deformations at the design story drift consistent with ACI 318 excluding Chapter 21. Structural steel and composite coupling beams shall be designed to provide inelastic deformations at the design story drift through yielding in flexure or shear. Structural steel and composite boundary elements shall be designed to provide inelastic deformations at the design story drift through yielding due to axial force. Reinforced concrete walls shall satisfy the requirements of ACI 318 excluding Chapter 21, except as modified in this section.*

- **3. Analysis**
- Analysis shall satisfy the requirements of Chapter C as modified in this section.
- (1) Uncracked effective stiffness values for elastic analysis shall be assigned in accordance with ACI 318 Chapter 10 for wall piers and composite coupling beams.
- (2) When *concrete-encased shapes function as boundary members, the analysis shall* be based upon a transformed concrete section using elastic material properties.
- (3) The flexibility of the connection between coupling beams and wall piers and the effect of shear distortions of the coupling beam and walls shall be taken into account.

- **4. System Requirements**
- *In coupled walls, coupling beams are permitted to yield over the height of the structure. The coupling beam-wall connection shall develop the expected flexural and shear strengths of the coupling beam. In coupled walls, it is permitted to redistribute coupling beam forces vertically to adjacent floors. The shear in any individual coupling beam should not be reduced by more than 20% of the elastically determined value. The sum of the coupling beam shear resistance over the height of the building shall be greater than or equal to the sum of the elastically determined values.*
- **5. Members**
- **5a. Boundary Members**
- *Boundary members shall satisfy the following requirements:*
- *(1) The required axial strength of the boundary member shall be determined assuming that the shear forces are carried by the reinforced concrete wall and the entire* gravity and overturning forces are carried by the boundary members in conjunction with the shear wall.

- *(2) When the concrete-encased structural steel boundary member qualifies as a composite column as defined in Specification Chapter I, it shall be designed as a composite column to satisfy the requirements of Chapter I of the Specification.*
- *(3) Headed studs or welded reinforcement anchors shall be provided to transfer required shear strengths between the structural steel boundary members and reinforced concrete walls. Headed studs, if used, shall satisfy the requirements of Specification Chapter I. Welded reinforcement anchors, if used, shall satisfy the requirements of Structural Welding Code-Reinforcing Steel (AWS D1.4/D1.4M).*
- **5b. Coupling Beams**
- **(1) Structural Steel Coupling Beams**
- *Structural steel coupling beams that are used between adjacent reinforced concrete walls shall satisfy the requirements of the Specification and this section. The following requirements apply to wide flange steel coupling beams.*

- *(1) Steel coupling beams shall comply with the requirements of Section D1.1 for moderately ductile members.*
- *(2) The expected shear strength, V_n, of steel coupling beams shall be computed from Equation H4-1.*

$$V_n = \frac{2R_y M_p}{g} \le R_y V_p \qquad \text{(H4-1)}$$

- *where*
- A_{tw} *= area of steel beam web, in.^{2} (mm^2)*
- $M_p = F_y Z$*, kip-in. (N-mm)*
- V_n *= expected shear strength of a steel coupling beam, kips (N)*
- $V_p = 0.6F_y A_{tw}$*, kips (N)*
- g *= coupling beam clear span, in. (mm)*
- *(3) The embedment length, L_e, shall be computed from Equations H4-2 and H4-2M. The embedment length shall be considered to begin inside the first layer of confining reinforcement in the wall boundary member*

$$V_n = 1.54\sqrt{f_c'}\left(\frac{b_w}{b_f}\right)^{0.66}\beta_1 b_f L_e\left[\frac{0.58-0.22\beta_1}{0.88+\dfrac{g}{2L_e}}\right]$$ *(H4-2)* •

$$V_n = 0.004\sqrt{f_c'}\left(\frac{b_w}{b_f}\right)^{0.66}\beta_1 b_f L_e\left[\frac{0.58-0.22\beta_1}{0.88+\dfrac{g}{2L_e}}\right]$$ *(S.I.)* *(H4-2M)* •

- *where*
- L_e = *embedment length of coupling beam, in. (mm)*
- b_w = *thickness of wall pier, in. (mm)*
- b_f = *beam flange width, in. (mm)*
- f'_c = *concrete compressive strength, ksi (MPa)*
- β_1 = *factor relating depth of equivalent rectangular compressive stress block to neutral axis depth, as defined in ACI 318*

- *(4) Vertical wall reinforcement with nominal axial strength equal to the expected shear strength of the coupling beam shall be placed over the embedment length of the beam with two-thirds of the steel located over the first half of the embedment length. This wall reinforcement shall extend a distance of at least one tension development length above and below the flanges of the coupling beam. It is permitted to use vertical reinforcement placed for other purposes, such as for vertical boundary members, as part of the required vertical reinforcement.*
- **(2) Composite Coupling Beams**
- *Encased composite sections serving as coupling beams shall satisfy the requirements of Section H4.5b(1) as modified in this section:*
- *(1) Coupling beams shall have an embedment length into the reinforced concrete wall that is sufficient to develop the expected shear strength, V_n , comp, computed from Equation H4-3.*

$$V_{n,comp} = \frac{2M_{p,\exp}}{g} \le V_{comp} \qquad \text{(H4-3)}$$

-

- *where*

- $M_{p,\,exp}$ = *expected flexural strength of composite coupling beam, kip-in. (N-mm). For concrete-encased or composite beams,* $M_{p,\,exp}$ *shall be calculated using the plastic stress distribution or the strain compatibility method. Appropriate* R_y *factors shall be used for different elements of the cross-section while establishing section force equilibrium and calculating the flexural strength.*
- V_{comp} = *limiting expected shear strength of an encased composite coupling beam as computed by Equations H4-4 and H4-4M, kips (N)*

$$V_{comp} = R_y V_p + \left(2\sqrt{f_c'} b_{wc} d_c + \frac{A_s F_{ysr} d_c}{s} \right) \qquad \text{(H4-4)}$$

$$V_{comp} = R_y V_p + \left(0.166\sqrt{f_c'} b_{wc} d_c + \frac{A_s F_{ysr} d_c}{s} \right) \qquad \text{(S.I.)} \qquad \text{(H4-4M)}$$

- where
- A_s = *area of transverse reinforcement, in.*^{2} *(mm*^{2}*)*
- F_{ysr} = *specified minimum yield stress of transverse reinforcement, ksi* (MPa)
- b_{wc} = *width of concrete encasement, in. (mm)*
- d_c = *effective depth of concrete encasement, in. (mm)*
- s = *spacing of transverse reinforcement, in. (mm)*

- *(2) The required embedment length shall be computed from Equations H4-2 and H4-2M by using $V_{n,comp}$ instead of V_n .*
- **5c. Projected Zones**
- *There are no designated protected zones.*
- **6. Connections**
- *There are no additional requirements beyond Section H4.5.*
- **6a. Demand Critical Welds**
- *There are no requirements for demand critical welds.*

COMPOSITE SPECIAL SHEAR WALLS (C-SSW)

- **1. Scope**
- *Composite special shear walls (C-SSW) shall be designed in conformance with this section. This section is applicable when reinforced concrete walls are composite with structural steel elements, including structural steel or composite sections acting as boundary members for the walls and structural steel or composite coupling beams that connect two or more adjacent reinforced concrete walls.*
- **2. Basis of Design**
- *C-SSW designed in accordance with these provisions are expected to provide significant inelastic deformation capacity through yielding in the reinforced concrete walls and the steel or composite elements. Reinforced concrete wall elements shall be designed to provide inelastic deformations at the design story drift consistent with ACI 318 including Chapter 21.*

- *Structural steel and composite coupling beams shall be designed to provide inelastic deformations at the design story drift through yielding in flexure or shear. Coupling beam connections and the design of the walls shall be designed to account for the expected strength including strain hardening in the coupling beams. Structural steel and composite boundary elements shall be designed to provide inelastic deformations at the design story drift through yielding due to axial force.*
- *C-SSW systems shall satisfy the requirements of Section H4 and the shear wall requirements of ACI 318 including Chapter 21, except as modified in this section.*
- **3. Analysis**
- *Analysis requirements of Section H4.3 shall be met with the following exceptions:*
- *(1) Cracked effective stiffness values for elastic analysis shall be assigned in accordance with ACI 318 Chapter 10 practice for wall piers and composite coupling beams.*
- *(2) Effects of shear distortion of the steel coupling beam shall be taken into account.*

- **4. System Requirements**
- *System requirements of Section H4.4 shall be satisfied with the following exceptions:*
- *(1) In coupled walls, coupling beams shall yield over the height of the structure followed by yielding at the base of the wall piers.*
- *(2) In coupled walls, the axial design strength of the wall at the balanced condition, P_b , shall equal or exceed the total required compressive axial strength in a wall pier, computed as the sum of the required strengths attributed to the walls from the gravity load components of the lateral load combination plus the sum of the expected beam shear strengths increased by a factor of 1.1 to reflect the effects of strain hardening ($1.1R_yV_n$) of all the coupling beams framing into the walls.*
- **5. Members**
- **5a. Ductile Elements**
- *Coupling beams are protected zones, and shall satisfy the requirements of Section D1.3. Welding on steel coupling beams is permitted for attachment of stiffeners, as required in Section F3.5b(4).*

- **5b. Boundary Members**
- *Unencased structural steel columns shall satisfy the requirements of Section D1.1 for highly ductile members and Section H4.5a(1).*
- *In addition to the requirements of Sections H4.3(2) and H4.5a(2), the requirements in this section shall apply to walls with concrete-encased structural steel boundary members. Concrete-encased structural steel boundary members that qualify as composite columns in Specification Chapter I shall meet the highly ductile member requirements of Section D1.4b(2). Otherwise, such members shall be designed as composite compression members to satisfy the requirements of ACI 318 Section 10.13 including the special seismic requirements for boundary members in ACI 318 Section 21.9.6. Transverse reinforcement for confinement of the composite boundary member shall extend a distance of 2h into the wall, where h is the overall depth of the boundary member in the plane of the wall.*
- *Headed studs or welded reinforcing anchors shall be provided as specified in Section H4.5a(3).*

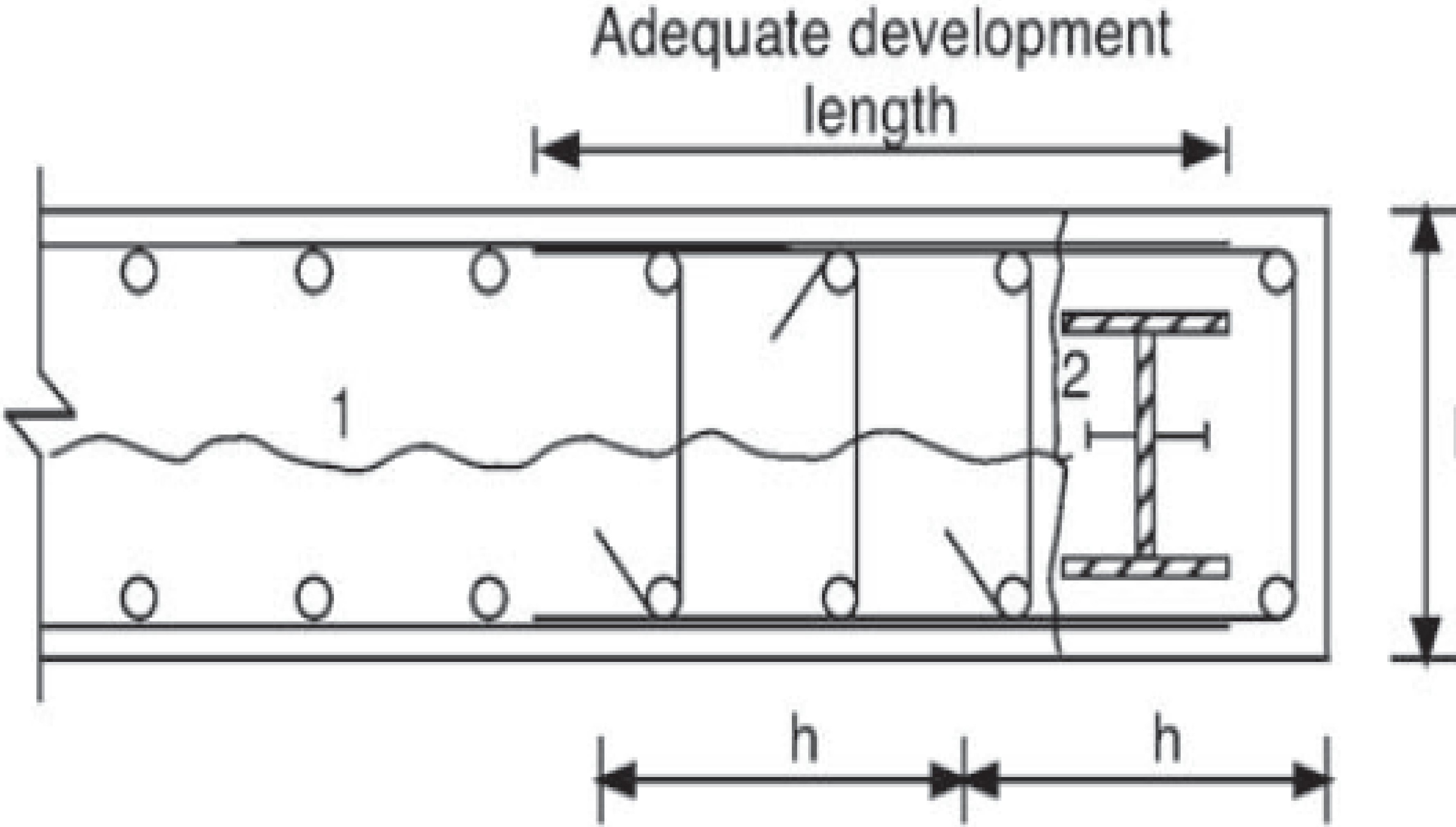


Fig. C-H5.1. Reinforcement to prevent splitting failures.

- **5c. Steel Coupling Beams**
- *In addition to the requirements of Section H4.5b, structural steel coupling beams shall satisfy the requirements of Section F3.5b. When required in Section F3.5b(4), the coupling beam rotation shall be assumed as a 0.08 rad link rotation unless a smaller value is justified by rational analysis of the inelastic deformations that are expected under the design story drift. Face bearing plates shall be provided on both sides of the coupling beams at the face of the reinforced concrete wall. These stiffeners shall meet the detailing requirements of Section F3.5b(4).*
- *Steel coupling beams shall comply with the requirements of Section D1.1 for highly ductile members.*
- *The expected shear strength for which the embedment length is calculated in Equation H4-1 shall be increased by a factor of 1.1 to reflect the effects of strain hardening ($1.1R_yV_n$).*

- *Vertical wall reinforcement as specified in Section H4.5b(1)(4) shall be confined by transverse reinforcement that meets the requirements for boundary members of ACI 318 Section 21.9.6.*
- *Embedded steel members shall be provided with two regions of vertical transfer reinforcement attached to both the top and bottom flanges of the embedded member. The first region shall be located to coincide with the location of longitudinal wall reinforcing bars closest to the face of the wall. The second shall be placed a distance no less than d/2 from the termination of the embedment length. All transfer reinforcement bars shall be fully developed where they engage the coupling beam flanges. It is permitted to use straight, hooked or mechanical anchorage to provide development. It is permitted to use mechanical couplers welded to the flanges to attach the vertical transfer bars. The area of vertical transfer reinforcement required is computed by Equation H5-1:*

$$A_{tb} \geq 0.03 f_c' L_e b_f / F_{ysr} \qquad \text{(H5-1)}$$

- *where*
- A_{tb} *= area of transfer reinforcement required in each of the first and second regions attached to each of the top and bottom flanges, in.*^{2} *(mm*^{2}*)*

- F_{ysr} = *specified minimum yield stress of transfer reinforcement, ksi (MPa)*
- L_e = *embedment length, in. (mm)*
- b_f = *beam flange width, in. (mm)*
- f'_c = *concrete compressive strength, ksi (MPa)*
- *The area of vertical transfer reinforcement shall not exceed that computed by Equation H5-2:*
- $\Sigma A_{tb} < 0.08\, L_e b_w - A_s$ *(H5-2)*
- *where*
- A_{tb}= *total area of transfer reinforcement provided in both the first and second regions attached to either the top or bottom flange, in.2 (mm2)*
- A_s = *area of longitudinal wall reinforcement provided over the embedment length,* L_e, *in.*^{2} *(mm*^{2}*)*
- b_w = *wall width, in. (mm)*

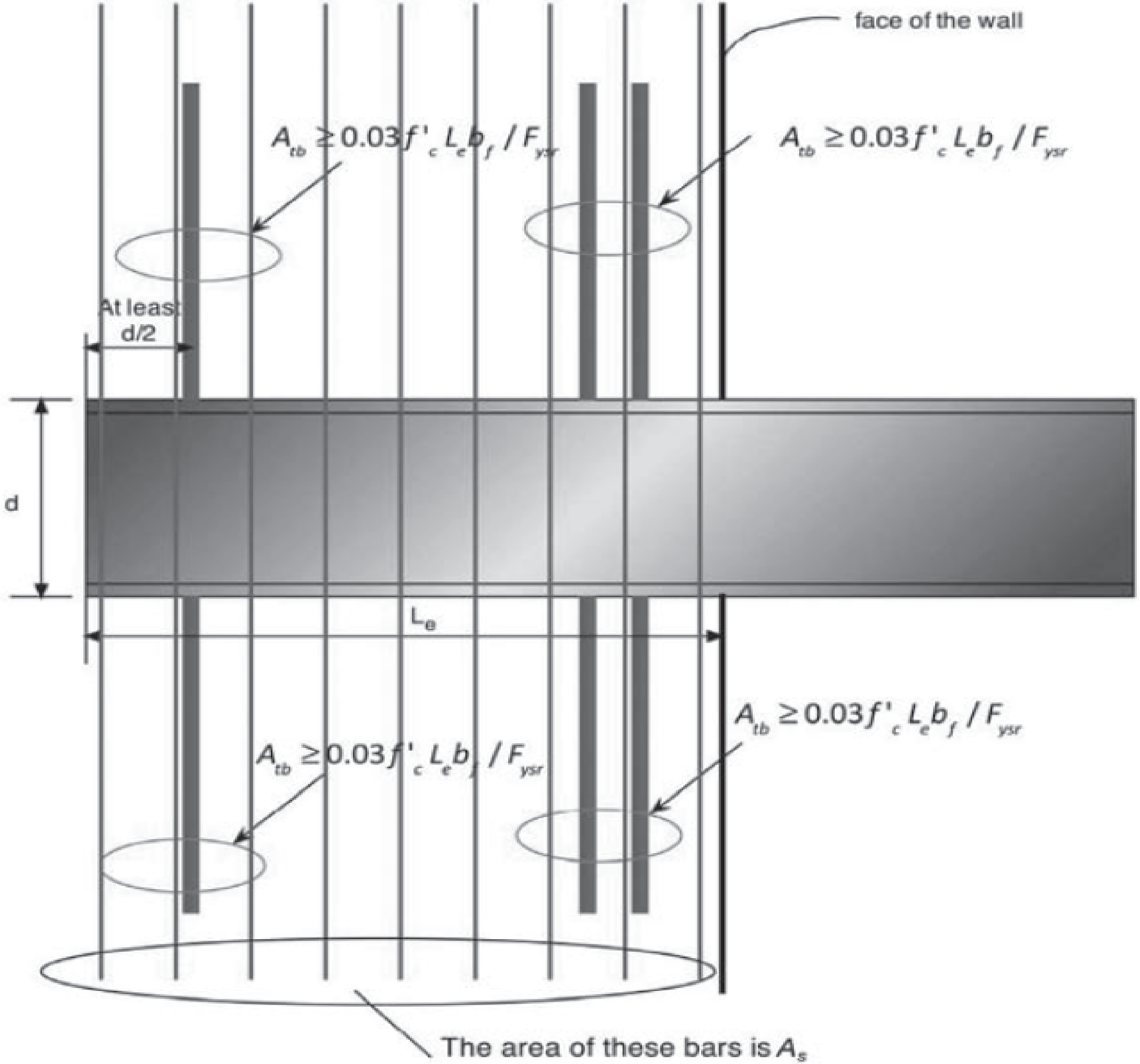


Fig. C-H5.2. Transfer bars.

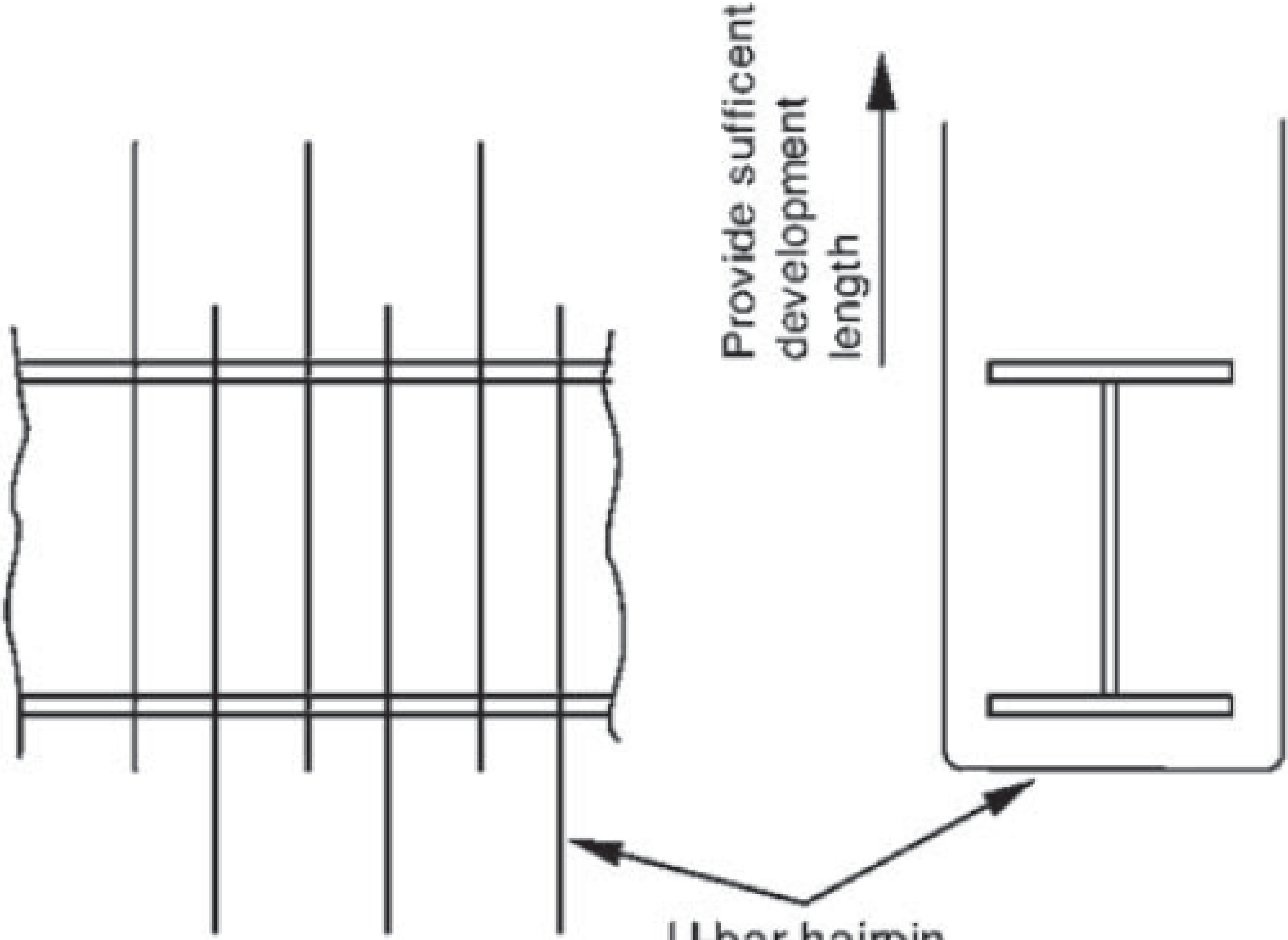


Fig. C-H5.3. Alternating U-shaped hairpins.

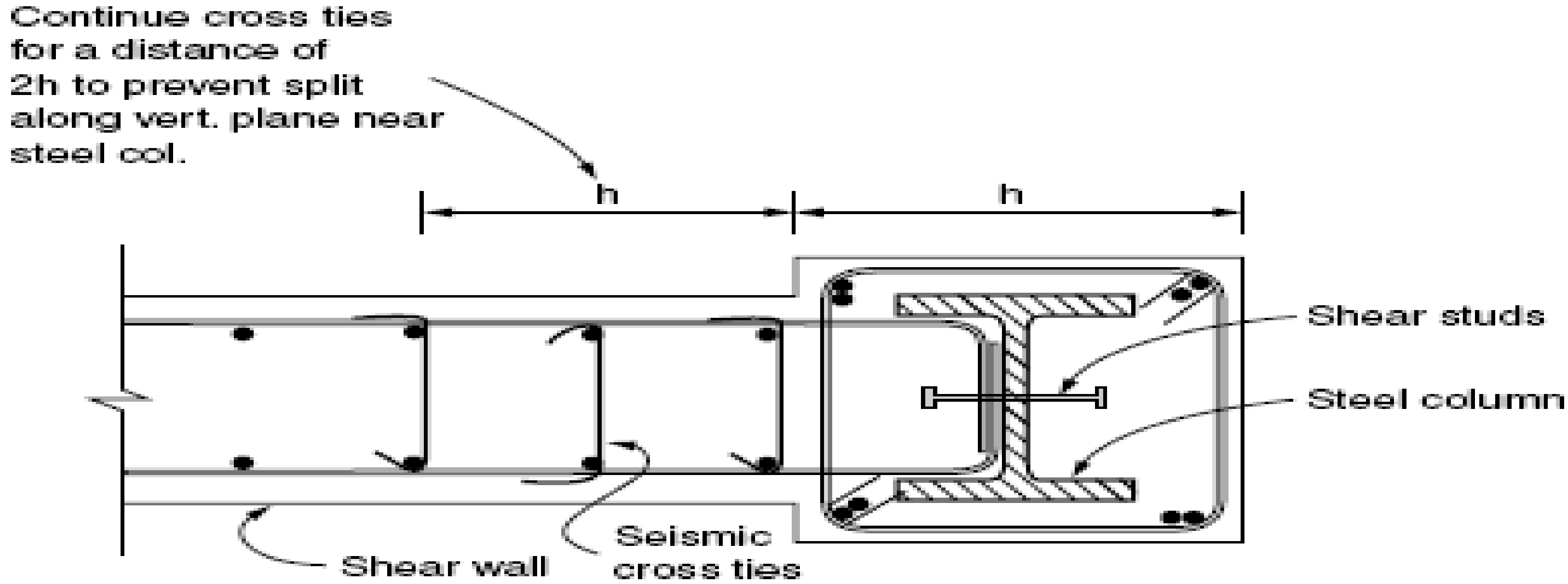
- **5d. Composite Coupling Beams**
- *Encased composite sections serving as coupling beams shall satisfy the requirements of Section H5.5c except the requirements of Section F3.5b(4) need not be met, and Equation H5-3 shall be used instead of Equation H4-4. For all encased composite coupling beams, the limiting expected shear strength, V_{comp}, is:*

$$V_{comp} = 1.1 R_y V_p + 1.56\left(2\sqrt{f_c'}\, b_{wc} d_c + \frac{A_s F_{ysr} d_c}{s}\right) \qquad \textit{(H5-3)}$$

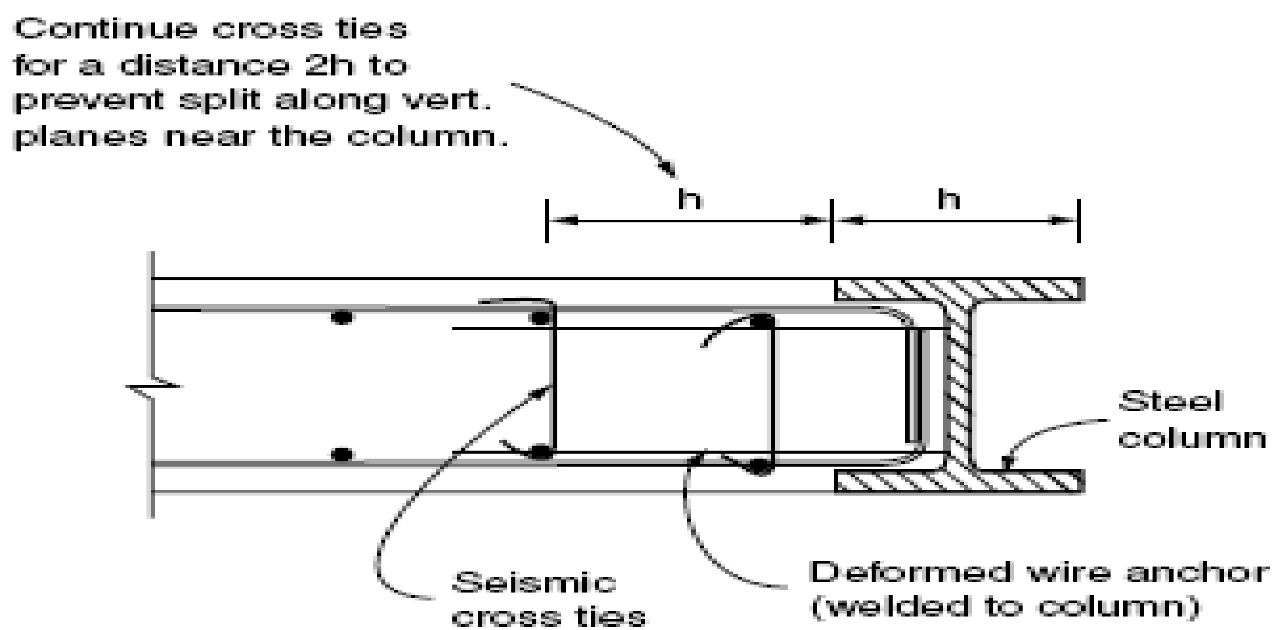
$$V_{comp} = 1.1 R_y V_p + 1.56\left(0.166\sqrt{f_c'}\, b_{wc} d_c + \frac{A_s F_{ysr} d_c}{s}\right) \qquad \textit{(S.I.)} \quad \textit{(H5-3M)}$$

- *where*
- F_{ysr} *= yield stress of transverse reinforcement, ksi (MPa)*
- **5e. Protected Zones**
- *There are no designated protected zones.*

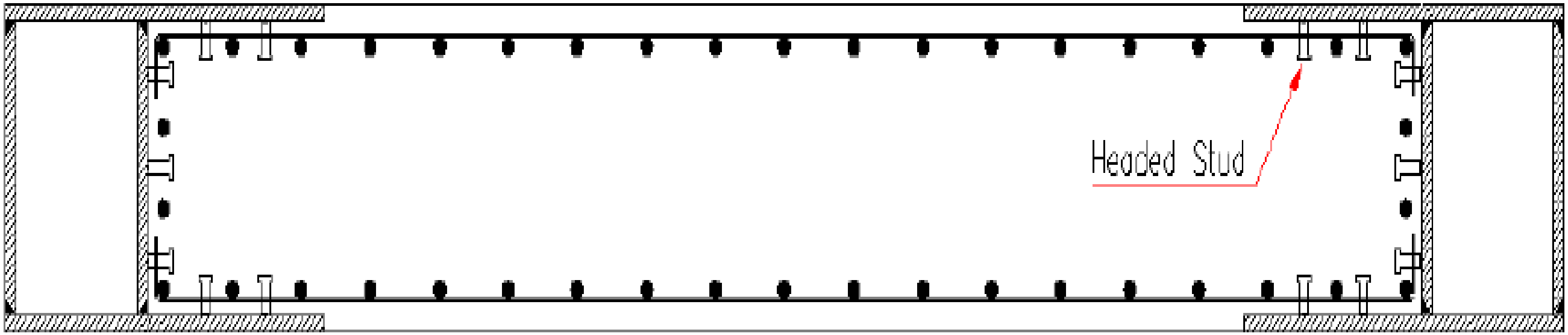
- **6. Connections**
- **6a. Demand Critical Welds**
- *The following welds are demand critical welds, and shall satisfy the requirements of Section A3.4b and I2.3:*
- *(1) Groove welds at column splices*
- *(2) Welds at the column-to-base plate connections.*
- *Exception: Where it can be shown that column hinging at, or near, the base plate is precluded by conditions of restraint, and in the absence of net tension under load combinations including the amplified seismic load, demand critical welds are not required.*
- **6b. Column Splices**
- *Column splices shall be designed following the requirements of Section G2.6f.*


Continue cross ties
for a distance of
2h to prevent split
along vert. plane near
steel col.
h
h
Shear studs
Steel column
Shear wall
Seismic
cross ties

(a)

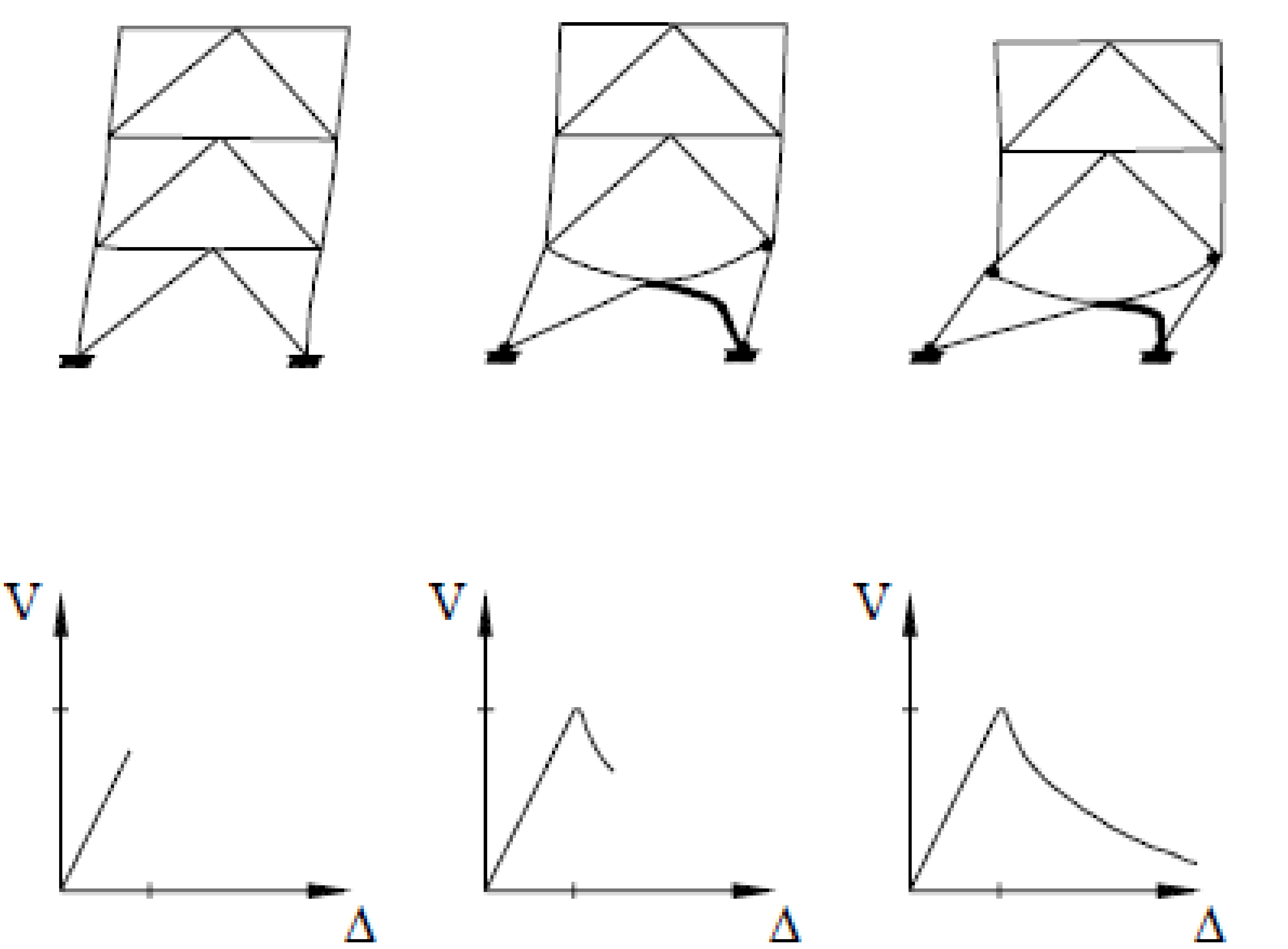

Continue cross ties
for a distance 2h to
prevent split along vert.
planes near the column.
h
h
Steel
column
Seismic
cross ties
Deformed wire anchor
(welded to column)

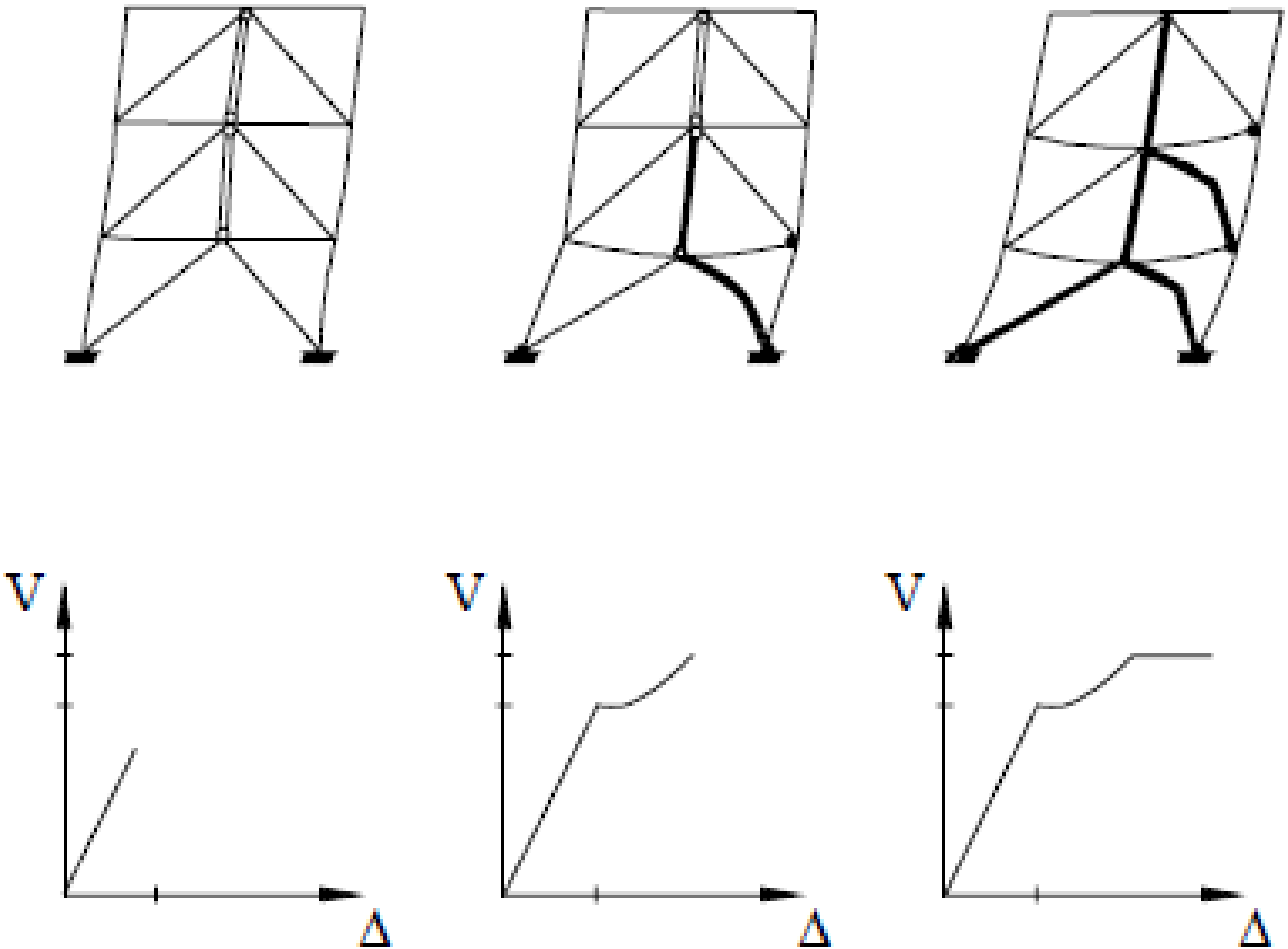
(b)


Headed Stud

Zipper frames

- *Leon and Yang developed a simplified design procedure for suspended zipper frames. Such frames are built from inverted-V-braced frames by adding zipper columns (i.e. vertical members connecting to the intersection points of the braces above the first floor), an elastic hat truss at the top story, and a capacity design procedure that establishes a clear hierarchy of yielding (Figure 7). This is done to distribute uniformly along the frame height the unbalanced vertical force generated when braces buckle.*


V
Δ
V
Δ
V
Δ


V
Δ
V
Δ
V
Δ