# 4.4 Procedures for Lateral-Force-Resisting Systems

This section provides Tier 2 evaluation procedures that apply to lateral force resisting systems: moment frames, shear walls and braced frames.

## 4.4.1 Moment Frames

**Commentary:**

Moment frames develop their resistance to lateral forces through the flexural strength and continuity of beam and column elements.

In an earthquake, a frame with suitable proportions and details can develop plastic hinges that will absorb energy and allow the frame to survive actual displacements that are larger than calculated in an elastic-based design.

In modern moment frames, the ends of beams and columns, being the locations of maximum seismic moment, are designed to sustain inelastic behavior associated with plastic hinging over many cycles and load reversals. Frames that are designed and detailed for this ductile behavior are called "special" moment frames.

Frames without special seismic detailing depend on the reserve strength inherent in the design of the members. The basis of this reserve strength is the load factors in strength design or the factors of safety in working-stress design. Such frames are called "ordinary" moment frames. For ordinary moment frames, failure usually occurs due to a sudden brittle mechanism, such as shear failure in concrete members.

For evaluations using this Handbook, it is not necessary to determine the type of frame in the building. The performance issue is addressed by appropriate acceptance criteria in the specified procedures. The fundamental requirements for all ductile moment frames are that:

1. They have sufficient strength to resist seismic demands,
2. They have sufficient stiffness to limit interstory drift,
3. Beam-column joints have the ductility to sustain the rotations they are subjected to,
4. Elements can form plastic hinges, and
5. Beams will develop hinges before the columns at locations distributed throughout the structure (the strong column/weak beam concept).

These items are covered in more detail in the evaluation statements that follow.

It is expected that the combined action of gravity loads and seismic forces will cause the formation of plastic hinges in the structure. However, a concentration of plastic hinge formation at undesirable locations can severely undermine the stability of the structure. For example, in a weak column situation (see Figure 4-13 next page), hinges can form at the tops and bottoms of all the columns in a particular story, and a story mechanism develops. This condition results in a concentration of ductility demand and displacement in a single story that can lead to collapse.

In a strong column situation (see Figure 4-13 next page) the beams hinge first, yielding is distributed throughout the structure, and the ductility demand is more dispersed.

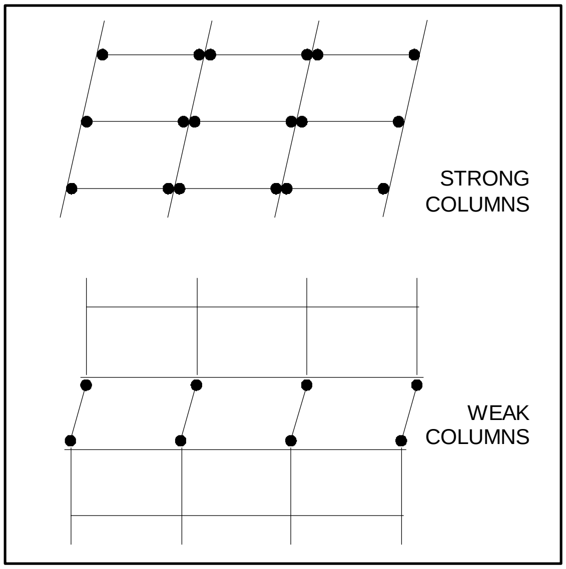



Figure 4-13. Plastic Hinge Formation

### 4.4.1.1 General

**4.4.1.1.1 REDUNDANCY: The number of lines of moment frames in each direction shall be greater than equal to 2 for Life Safety and for Immediate Occupancy. The number of bays of moment frames in each line shall be greater than or equal to 2 for Life Safety and 3 for Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with the procedures in Section 4.2 shall be performed. The adequacy of all elements and connections in the frames shall be evaluated.

**Commentary:**

Redundancy is a fundamental characteristic of lateral force resisting systems with superior seismic performance. Redundancy in the structure will ensure that if an element in the lateral force resisting system fails for any reason, there is another element present that can provide lateral force resistance. Redundancy also provides multiple locations for potential yielding, distributing inelastic activity throughout the structure and improving ductility and energy dissipation. Typical characteristics of redundancy include multiple lines of resistance to distribute the lateral forces uniformly throughout the structure, and multiple bays in each line of resistance to reduce the shear and axial demands on any one element (see Figure 4-14).

A distinction should be made between redundancy and adequacy. For the purpose of this Handbook, redundancy is intended to mean simply "more than one." That is not to say that for large buildings two elements is adequate, or for small buildings one is not enough. Separate evaluation statements are present in the Handbook to determine the adequacy of the elements provided.

When redundancy is not present in the structure, an analysis which demonstrates the adequacy of the lateral force elements is required.

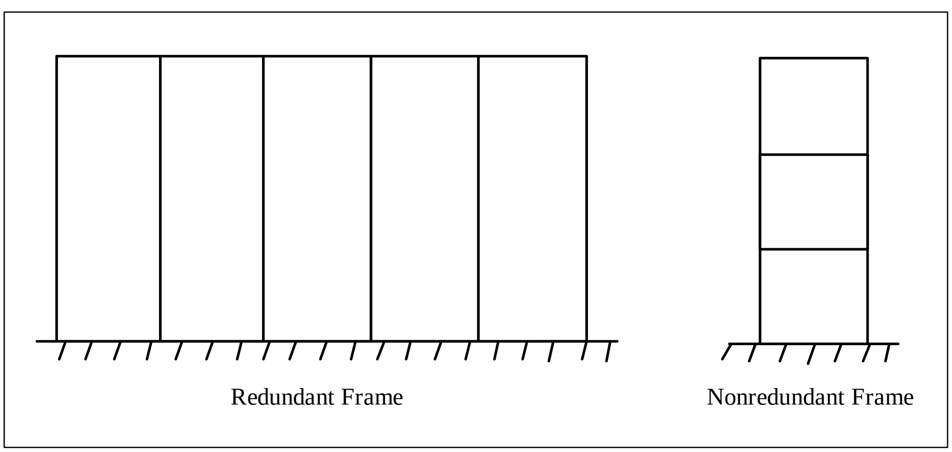



Figure 4-14. Redundancy Along a Line of Moment Frame

### 4.4.1.2 Moment Frames with Infill Walls

**Commentary:**

Infill walls used for partitions, cladding or shaft walls that enclose stairs and elevators should be isolated from the frames. If not isolated, they will alter the response of the frames and change the behavior of the entire structural system. Lateral drifts of the frame will induce forces on walls that interfere with this movement. Cladding connections must allow for this relative movement. Stiff infill walls confined by the frame will develop compression struts that will impart loads to the frame and cause damage to the walls. This is particularly important around stairs or other means of egress from the building.

**4.4.1.2.1 INTERFERING WALLS: All infill walls placed in moment frames shall be isolated from structural elements.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The demands imparted by the structure to the interfering walls, and the demands induced on frame shall be calculated. The adequacy of the interfering walls and the frame to resist the induced forces shall be evaluated.

**Commentary:**

When an infill wall interferes with the moment frame, the wall becomes an unintended part of the lateral-force-resisting system. Typically these walls are not designed and detailed to participate in the lateral-force-resisting system and may be subject to significant damage.

Interfering walls should be checked for forces induced by the frame, particularly when damage to these walls can lead to falling hazards near means of egress. The frames should be checked for forces induced by contact with the walls, particularly if the walls are not full height, or do not completely infill the bay.

### 4.4.1.3 Steel Moment Frames

**Commentary:**

The following are characteristics of steel moment frames that have demonstrated acceptable seismic performance:

1. The beam end connections develop the plastic moment capacity of the beam or panel zone,
2. There is a high level of redundancy in the number of moment connections,
3. The column web has sufficient strength to sustain the stresses in the beam-column joint,
4. The lower flanges have lateral bracing sufficient to maintain stability of the frame, and
5. There is flange continuity through the column.

Prior to the 1994 Northridge earthquake, steel moment-resisting frame connections generally consisted of complete penetration flange welds and a bolted or welded shear tab connection at the web. This type of connection, which was an industry standard from 1970 to 1995, was thought to be ductile and capable of developing the full capacity of the beam sections. However, over 200 buildings experienced extensive brittle damage to this type of connection during the Northridge earthquake. As a result, an emergency code change was made to the *1994 UBC* (ICBO, 1994) removing the prequalification of this type of connection. The reasons for this unexpected performance are still under investigation. A full discussion of the various fractures mechanisms and ways of preventing or repairing them is given in *FEMA 267* (SAC, 1995) and *FEMA 267A* (SAC, 1997).

**4.4.1.3.1 DRIFT CHECK: The drift ratio of the steel moment frames, calculated using the Quick Check Procedure of Section 3.5.3.1, shall be less than the 0.025 for Life Safety and 0.015 for Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the beams and columns, including P-Δ effects, shall be evaluated using the *m*-factors in Table 4-3.

> **Commentary:**
>
> Moment-resisting frames are more flexible than shear wall or braced frame structures. This flexibility can lead to large interstory drifts that may potentially cause extensive structural and nonstructural damage to welded beam-column connections, partitions, and cladding. Drifts may also induce large P-Δ demands, and pounding when adjacent buildings are present.
>
> An analysis of non-compliant frames is required to demonstrate the adequacy of frame elements subjected to excessive lateral drifts.

**4.4.1.3.2 AXIAL STRESS CHECK: The axial stress due to gravity loads in columns subjected to overturning forces shall be less than $0.10F_y$ for Life Safety and Immediate Occupancy. Alternatively, the axial stress due to overturning forces alone, calculated using the Quick Check Procedure of Section 3.5.3.6, shall be less than $0.30F_y$ for Life Safety and Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The gravity and overturning demands for non-compliant columns shall be calculated and the adequacy of the columns to resist overturning forces shall be evaluated using the *m*-factors in Table 4-3.

> **Commentary:**
>
> Columns that carry a substantial amount of gravity load may have limited additional capacity to resist seismic forces. When axial forces due to seismic overturning moments are added, the columns may buckle in a nonductile manner due to excessive axial compression.
>
> The alternative calculation of overturning stresses due to seismic forces alone is intended to provide a means of screening out frames with high gravity loads, but are known to have small seismic overturning forces.
>
> When both demands are large, the combined effect of gravity and seismic forces must be calculated to demonstrate compliance.

**4.4.1.3.3 MOMENT-RESISTING CONNECTIONS: All moment connections shall be able to develop the strength of the adjoining members or panel zones.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the members and connections shall be evaluated using the *m*-factors in Table 4-3.

> **Commentary:**
>
> Prior to the 1994 Northridge earthquake, steel moment-resisting frame connections generally consisted of full penetration flange welds and a bolted or welded shear tab connection at the web. This type of connection, (see Figure 4-15 on the following page) which was an industry standard from 1970 to 1995, was thought to be ductile and capable of developing the full capacity of the beam sections. However, over 200 buildings experienced extensive brittle damage to this type of connection during the Northridge earthquake. As a result, an emergency code change was made to the *1994 UBC* (ICBO, 1994) removing the prequalification of this type of connection. The reasons for this

unexpected performance are still under investigation. A full discussion various fracture mechanisms and ways of preventing or repairing them is given in *FEMA 267* (SAC, 1995) and *FEMA 267A* (SAC, 1997).

For this Handbook, the Tier 1 evaluation statement is considered non-compliant for full penetration flange welds and a more detailed analysis is required to determine the adequacy of these moment-resisting connections.

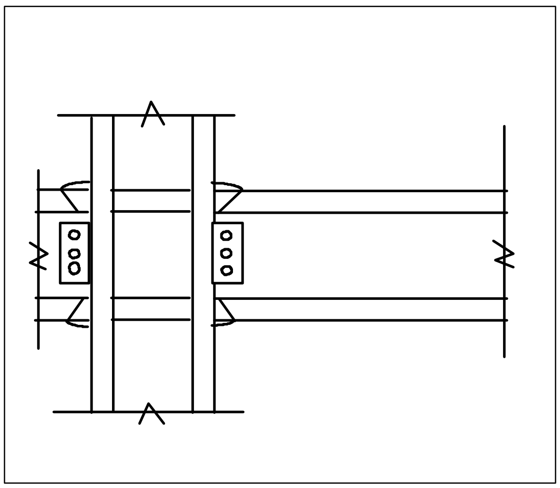


Figure 4-15. Northridge-Type Connection

**4.4.1.3.4 PANEL ZONES: All panel zones shall have the shear capacity to resist the shear demand required to develop 0.8$\Sigma M_p$ of the girders framing in at the face of the column.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The demands in non-compliant joints shall be calculated and the adequacy of the panel zones for web shear shall be evaluated using the *m*-factors in Table 4-3.

**Commentary:**

Panel zones with thin webs may yield or buckle before developing the capacity of the adjoining members, reducing the inelastic performance and ductility of the moment frames.

When panel zones cannot develop the strength of the beams, compliance can be demonstrated by checking the panel zones for actual shear demands.

**4.4.1.3.5 COLUMN SPLICES: All column splice details located in moment resisting frames shall include connection of both flanges and the web for Life Safety, and the splice shall develop the strength of the column for Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The gravity and seismic demands shall be calculated and the adequacy of the splice connection shall be evaluated.

**Commentary:**

The lack of a substantial connection at the splice location may lead to separation of the spliced sections and misalignment of the columns resulting in loss of vertical support and partial or total collapse of the building. Tests on partial-penetration weld splices have shown limited ductility.

An inadequate connection also reduces the effective capacity of the column. Splices are checked against calculated demands to demonstrate compliance.

**4.4.1.3.6 STRONG COLUMN/WEAK BEAM: The percent of strong column/weak beam joints in each story of each line of moment resisting frames shall be greater than 50% for Life Safety and 75% for Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the columns to resist calculated demands shall be evaluated using an *m*-factor equal to 2.5. Alternatively, the story strength shall be calculated, and checked for the capacity to resist one half the total pseudo lateral force.

**Commentary:**

When columns are not strong enough to force hinging in the beams, column hinging can lead to story mechanisms and a concentration of inelastic activity at a single level. Excessive story drifts may result in an instability of the frame due to P-$\Delta$ effects. Good post-elastic behavior consists of yielding distributed throughout the frame. A story mechanism will limit forces in the levels above, preventing the upper levels from yielding. Joints at the roof level need not be considered.

If it can be demonstrated that non-compliant columns are strong enough to resist calculated demands with sufficient overstrength, acceptable behavior can be expected.

The alternative procedure checks for the formation of a story mechanism. The story strength is the sum of the shear capacities of all the columns as limited by the controlling action. If the columns are shear critical, a shear mechanism forms at the shear capacity of the columns. If the columns are controlled by flexure, a flexural mechanism forms at a shear corresponding to the flexural capacity.

Should additional study be required, a Tier 3 evaluation would include a non-linear pushover analysis. The formation of a story mechanism would be acceptable, provided the target displacement is met .

**4.4.1.3.7 COMPACT MEMBERS: All moment frame elements shall meet compact section requirements set forth by the *Load and Resistance Factor Design Specification For Structural Steel Buildings* (AISC, 1993). This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of non-compliant beams and columns shall be evaluated using the *m*-factors in Table 4-3.

**Commentary:**

Noncompact frame elements may experience premature local buckling prior to development of their full moment capacities. This can lead to poor inelastic behavior and ductility.

The adequacy of the frame elements can be demonstrated using reduced *m*-factors in consideration of reduced capacities for noncompact sections.

**4.4.1.3.8 BEAM PENETRATIONS: All openings in frame-beam webs shall be less than 1/4 of the beam depth and shall be located in the center half of the beams. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The shear and flexural demands on non-compliant beams shall be calculated. The adequacy of the beams considering the strength around the penetrations shall be evaluated.

**Commentary:**

Members with large beam penetrations may fail in shear prior to the development of their full moment capacity, resulting in poor inelastic behavior and ductility.

The critical section is at the penetration with the highest shear demand. Shear transfer across the web opening will induce secondary moments in the beam sections above and below the opening that must be considered in the analysis.

**4.4.1.3.9 GIRDER FLANGE CONTINUITY PLATES: There shall be girder flange continuity plates at all moment resisting frame joints. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** The adequacy of the column flange to transfer girder flange forces to the panel zone without continuity plates shall be evaluated.

**Commentary:**

The lack of girder flange continuity plates may lead to a premature failure at the column web or flange at the joint. Beam flange forces are transferred to the column web through the column flange, resulting in a high stress concentration at the base of the column web. The presence of continuity plates, on the other hand, transfers the beam flange forces along the entire length of the column web.

Adequate force transfer without continuity plates will depend on the strength and stiffness of the column flange in weak-way bending.

**4.4.1.3.10 OUT-OF-PLANE BRACING: Beam-column joints shall be braced out-of-plane. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The axial demands on non-compliant columns shall be calculated and the adequacy of the column to resist buckling between points of lateral support shall be evaluated considering a horizontal out-of-plane force equal to 6% of the critical column flange compression force acting concurrently at the non-compliant joint.

**Commentary:**

Joints without proper bracing may buckle prematurely out-of-plane before the strength of the joint can be developed. This will limit the ability of the frame to resist seismic forces.

The combination of axial load and moment on the columns will result in higher compression forces in one of the column flanges. The tendency for highly loaded joints to twist out-of-plane is due to compression buckling of the critical column compression flange.

Compliance can be demonstrated if the column section can provide adequate lateral restraint for the joint between points of lateral support.

**4.4.1.3.11 BOTTOM FLANGE BRACING: The bottom flange of beams shall be braced out-of-plane. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the beams shall be evaluated considering the potential for lateral torsional buckling of the bottom flange between points of lateral support.

**Commentary:**

Beams flanges in compression require out-of-plane bracing to prevent lateral torsional buckling. Buckling will occur before the full strength of the beam is developed, and the ability of the frame to resist lateral forces will be limited.

Top flanges are typically braced by connection to the diaphragm. Bottom flange bracing occurs at discrete locations, such as at connection points for supported beams. The spacing of bottom flange bracing may not be close enough to prevent premature lateral torsional buckling when seismic loads induce large compression forces in the bottom flange.

Note that this condition is not considered a life-safety concern, and need only be examined for the Immediate Occupancy Performance Level.

### 4.4.1.4 Concrete Moment Frames

**Commentary:**

Concrete moment frame buildings typically are more flexible than shear wall buildings. This flexibility can result in large interstory drifts that may lead to extensive nonstructural damage and P-delta effects. If a concrete column has a capacity in shear that is less than the shear associated with the flexural capacity of the column, brittle column shear failure may occur and result in collapse. This condition is common in buildings in zones of moderate seismicity and in older buildings in zones of high seismicity. The columns in these buildings often have ties at standard spacing equal to the depth of the column, whereas current code maximum spacing for shear reinforcing is d/2. The following are the characteristics of concrete moment frames that have demonstrated acceptable seismic performance:

1. Brittle failure is prevented by providing a sufficient number of beam stirrups, column ties, and joint ties to ensure that the shear capacity of all elements exceeds the shear associated with flexural capacity,
2. Concrete confinement is provided by beam stirrups and column ties in the form of closed hoops with 135-degree hooks at locations where plastic hinges will occur.
3. Overall performance is enhanced by long lap splices that are restricted to favorable locations and protected with additional transverse reinforcement.
4. The strong column/weak beam requirement is achieved by suitable proportioning of the members and their longitudinal reinforcing.

Older frame systems that are lightly reinforced, precast concrete frames, and flat slab frames usually do not meet the detail requirements for ductile behavior.

**4.4.1.4.1 SHEAR STRESS CHECK: The shear stress in the concrete columns, calculated using the Quick Check Procedure of Section 3.5.3.2, shall be less than 100 psi or $2\sqrt{f'c}$ or Life Safety and Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the concrete frame elements shall be evaluated using the *m*-factors in Table 4-4.

**Commentary:**

The shear stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building.

**4.4.1.4.2 AXIAL STRESS CHECK: The axial stress due to gravity loads in columns subjected to overturning forces shall be less than $0.10f'_c$ for Life Safety and Immediate Occupancy. Alternatively, the axial stress due to overturning forces alone, calculated using the Quick Check Procedure of Section 3.5.3.6, shall be less than $0.30f'_c$ for Life Safety and Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The gravity and overturning demands for non-compliant columns shall be calculated and the adequacy of the columns to resist overturning forces shall be evaluated using the *m*-factors in Table 4-4.

> **Commentary:**
>
> Columns that carry a substantial amount of gravity load may have limited additional capacity to resist seismic forces. When axial forces due to seismic overturning moments are added, the columns may crush in a nonductile manner due to excessive axial compression.
>
> The alternative calculation of overturning stresses due to seismic forces alone is intended to provide a means of screening out frames with high gravity loads, but are known to have small seismic overturning forces.
>
> When both demands are large, the combined effect of gravity and seismic forces must be calculated to demonstrate compliance.

**4.4.1.4.3 FLAT SLAB FRAMES: The lateral-force-resisting system shall not be a frame consisting of columns and a flat slab/plate without beams.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the slab-column system for resisting seismic forces and punching shear shall be evaluated using the *m*-factors in Table 4-4.

> **Commentary:**
>
> The concern is the transfer of the shear and bending forces between the slab and column, which could result in a punching shear failure and partial collapse. The flexibility of the lateral-force-resisting system will increase as the slab cracks.
>
> Continuity of some bottom reinforcement through the column joint will assist in the transfer of forces and provide some resistance to collapse by catenary action in the event of a punching shear failure.

**4.4.1.4.4 PRESTRESSED FRAME ELEMENTS: The lateral-load-resisting frames shall not include any prestressed or post-tensioned elements.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the concrete frame including prestressed elements shall be evaluated using the *m*-factors in Table 4.4.

> **Commentary:**
>
> Frame elements that are prestressed or post-tensioned may not behave in a ductile manner. The concern is the inelastic behavior of prestressed elements.

**4.4.1.4.5 SHORT CAPTIVE COLUMNS: There shall be no columns at a level with height/depth ratios less than 50% of the nominal height/depth ratio of the typical columns at that level for Life Safety and 75% for Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** The adequacy of the columns for the shear force required to develop the moment capacity at the top and bottom of the clear height of the columns shall be evaluated. Alternatively, evaluate the columns as force controlled elements in accordance with the alternative equations in Section 4.2.4.3.2.

**Commentary:**

Short captive columns tend to attract seismic forces because of high stiffness relative to other columns in a story. Significant damage has been observed in parking structure columns adjacent to ramping slabs, even in structures with shear walls. Captive column behavior may also occur in buildings with clerestory windows, or in buildings with partial height masonry infill panels.

If not adequately detailed, the columns may suffer a non-ductile shear failure which may result in partial collapse of the structure.

A captive column that can develop the shear capacity to develop the flexural strength over the clear height will have some ductility to prevent sudden non-ductile failure of the vertical support system.

**4.4.1.4.6 NO SHEAR FAILURES: The shear capacity of frame members shall be able to develop the moment capacity at the top and bottom of the columns.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The shear demands shall be calculated for non-compliant columns and the adequacy of the columns for shear shall be evaluated.

**Commentary:**

If the shear capacity of a column is reached before the moment capacity, there is a potential for a sudden non-ductile failure of the column, leading to collapse.

Columns that cannot develop the flexural capacity in shear should be checked for adequacy against calculated shear demands. Note that the shear capacity is affected by the axial loads on the column and should be based on the most critical combination of axial load and shear.

**4.4.1.4.7 STRONG COLUMN/WEAK BEAM: The sum of the moment capacity of the columns shall be 20% greater than that of the beams at frame joints.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the columns to resist calculated demands shall be evaluated using an *m*-factor equal to 2.0. Alternatively, the story strength shall be calculated, and checked for the capacity to resist one half the total pseudo lateral force.

**Commentary:**

When when columns are not strong enough to force hinging in the beams, column hinging can lead to story mechanisms and a concentration of inelastic activity at a single level. Excessive story drifts may result in an instability of the frame due to P-Δ effects. Good post-elastic behavior consists of yielding distributed throughout the frame. A story mechanism will limit forces in the levels above, preventing the upper levels from yielding. Joints at the roof level need not be considered.

If it can be demonstrated that non-compliant columns are strong enough to resist calculated demands with sufficient overstrength, acceptable behavior can be expected. Reduced *m*-factors are used to check the columns at near elastic levels.

The alternative procedure checks for the formation of a story mechanism. The story strength is the sum of the shear capacities of all the columns as limited by the controlling action. If the columns are shear critical, a shear mechanism forms at the shear capacity of the columns. If the columns are controlled by flexure, a flexural mechanism forms at a shear corresponding to the flexural capacity.

**4.4.1.4.8 BEAM BARS: At least two longitudinal top and two longitudinal bottom bars shall extend continuously throughout the length of each frame beam. At least 25% of the longitudinal bars provided at the joints for either positive or negative moment, shall be continuous throughout the length of the members for Life Safety and Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The flexural demand at several sections along the length of the non-compliant beams shall be calculated, and the adequacy of the beams shall be evaluated using an *m*-factor equal to 1.0.

**Commentary:**

The requirement for two continuous bars is a collapse prevention measure. In the event of complete beam failure, continuous bars will prevent total collapse of the supported floor, holding the beam in place by catenary action.

Previous construction techniques used bent up longitudinal bars as reinforcement. These bars transitioned from bottom to top reinforcement at the gravity load inflection point. Some amount of continuous top and bottom reinforcement is desired because moments due to seismic forces can shift the location of the inflection point.

Because non-compliant beams are vulnerable to collapse, the beams are required to resist demands at an elastic level. Continuous slab reinforcement adjacent to the beam may be considered as continuous top reinforcement.

**4.4.1.4.9 COLUMN-BAR SPLICES: All column bar lap splice lengths shall be greater than 35 $d_b$ for Life Safety and 50 $d_b$ for Immediate Occupancy, and shall be enclosed by ties spaced at or less than 8 $d_b$ for Life Safety and Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The flexural demands at non-compliant column splices shall be calculated and the adequacy of the columns shall be evaluated using the *m*-factors in Table 4-4.

**Commentary:**

Located just above the floor level, column bar splices are typically located in regions of potential plastic hinge formation. Short splices are subject to sudden loss of bond. Widely spaced ties can result in a spalling of the concrete cover and loss of bond. Splice failures are sudden and non-ductile.

Columns with non-compliant lap splices are checked using reduced *m*-factors to account for this potential lack of ductility. If the members have sufficient capacity, the demands on the splices are less likely to exceed the capacity of the bond.

**4.4.1.4.10 BEAM-BAR SPLICES: The lap splices for longitudinal beam reinforcing shall not be located within $l_b/4$ of the joints and shall not be located in the vicinity of potential plastic hinge locations.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The flexural demands in non-compliant beams shall be calculated and the adequacy of the beams shall be evaluated using the *m*-factors for non-ductile beams in Table 4-4.

**Commentary:**

Lap splices located at the end of beams and in vicinity of potential plastic hinges may not be able to develop the full moment capacity of the beam as the concrete degrades during multiple cycles.

Beams with non-compliant lap splices are checked using reduced *m*-factors to account for this potential lack of ductility. If the members have sufficient capacity, the demands are less likely to cause degradation and loss of bond between concrete and the reinforcing steel.

**4.4.1.4.11 COLUMN-TIE SPACING: Frame columns shall have ties spaced at or less than d/4 for Life Safety and Immediate Occupancy throughout their length and at or less than 8 $d_b$ for Life Safety and Immediate Occupancy at all potential plastic hinge locations.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The flexural demand in non-compliant columns shall be calculated and the adequacy of the columns shall be evaluate using the *m*-factors in Table 4-4.

**Commentary:**

Widely spaced ties will reduce the ductility of the column, and it may not be able to maintain full moment capacity through several cycles. Columns with widely spaced ties have limited shear capacity and non-ductile shear failures may result.

Elements with non-compliant confinement are checked using reduced *m*-factors to account for this potential lack of ductility.

**4.4.1.4.12 STIRRUP SPACING: All beams shall have stirrups spaced at or less than d/2 for Life Safety and Immediate Occupancy throughout their length. At potential plastic hinge locations stirrups shall be spaced at or less than the minimum of 8 $d_b$ or d/4 for Life Safety and Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The flexural demand in non-compliant beams shall be calculated and the adequacy of the beams shall be evaluate using the *m*-factors in Table 4-4.

**Commentary:**

Widely spaced ties will reduce the ductility of the column, and it may not be able to maintain full moment capacity through several cycles. Columns with widely spaced ties have limited shear capacity and non-ductile shear failures may result.

Elements with non-compliant confinement are checked using reduced *m*-factors to account for this potential lack of ductility.

**4.4.1.4.13 JOINT REINFORCING: Beam-column joints shall have ties spaced at or less than 8$d_b$ for Life Safety and Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The joint shear demands shall be calculated and the adequacy of the joint to develop the adjoining members forces shall be evaluated.

**Commentary:**

Beam-column joints without shear reinforcement may not be able to develop the strength of the connected members, leading to a non-ductile failure of the joint. Perimeter columns are especially vulnerable because the confinement of joint is limited to three sides (along the exterior) or two sides (at a corner).

The shear capacity of the joint my be calculated as follows:

$$Q_{cl}=\lambda\gamma A_j(f'_c)^{1/2} \text{ psi, where } \gamma \text{ is:}$$

| | $\rho''<0.003$ | $\rho''\geq 0.003$ |
|---|---|---|
| Int. joints w/ transverse beams | 12 | 20 |
| Int. joints w/o transverse beams | 10 | 15 |
| Ext. joints w/ transverse beams | 8 | 15 |
| Ext. joints w/o transverse beams | 6 | 12 |
| Corner joints | 4 | 8 |

$\lambda$ = 0.75 for lightweight concrete

$A_j$ = joint cross-sectional area

**4.4.1.4.14 JOINT ECCENTRICITY: There shall be no eccentricities larger than 20% of the smallest column plan dimension between girder and column centerlines for Immediate Occupancy. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The joint shear demands including additional shear stresses from joint torsion shall be calculated and the adequacy of the beam-column joints shall be evaluated.

**Commentary:**

Joint eccentricities can result in high torsional demands on the joint area, which will result in higher shear stresses.

**4.4.1.4.15 STIRRUP AND TIE HOOKS: The beam stirrups and column ties shall be anchored into the member cores with hooks of 135° or more. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The shear and axial demands in non-compliant members shall be calculated and the adequacy of the beams and columns shall be evaluated using the *m*-factors in Table 4-4.

**Commentary:**

To be fully effective, stirrups and ties must be anchored into the confined core of the member. 90° hooks that are anchored within the concrete cover are unreliable if the cover spalls during plastic hinging. The amount of shear resistance and confinement will be reduced if the stirrups and ties are not well anchored.

Elements with non-compliant confinement are checked using reduced *m*-factors to account for this potential lack of ductility.

### 4.4.1.5 Precast Concrete Moment Frames

**4.4.1.5.1 PRECAST CONNECTION CHECK: The precast connections at frame joints shall have the capacity to resist the shear and moment demands calculated using the QuickCheck Procedure of Section 3.5.3.5.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the precast connections shall be evaluated as force controlled elements using the procedures in Section 4.2.4.3.2.

**Commentary:**

Precast frame elements may have sufficient strength to meet lateral force requirements, but connections often cannot develop the strength of the members, and may be subject to premature non-ductile failures. Failure mechanisms may include fractures in the welded connections between inserts, pull out of embeds, and spalling of concrete.

Since full member capacities cannot be realized, the behavior of this system is entirely dependent on the performance of the connections.

**4.4.1.5.2 PRECAST FRAMES: For buildings with concrete shear walls, lateral forces shall not be resisted by precast concrete frame elements.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the precast frame elements shall be evaluated as force controlled elements using the procedures in Section 4.2.4.3.2.

**Commentary:**

Precast frame elements may have sufficient strength to meet lateral force requirements, but connections often cannot develop the strength of the members, and may be subject to premature non-ductile failures. Failure mechanisms may include fractures in the welded connections between inserts, pull out of embeds, and spalling of concrete.

Since full member capacities cannot be realized, the behavior of this system is entirely dependent on the performance of the connections.

**4.4.1.5.3 PRECAST CONNECTIONS: For buildings with concrete shear walls, the connection between precast frame elements such as chords, ties, and collectors in the lateral-force-resisting system shall develop the capacity of the connected members.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the connections for seismic forces shall be evaluated as force controlled elements using the procedures in Section 4.2.4.3.2.

**Commentary:**

Precast frame elements may have sufficient strength to meet lateral force requirements, but connections often cannot develop the strength of the members, and may be subject to premature non-ductile failures. Failure mechanisms may include fractures in the welded connections between inserts, pull out of embeds, and spalling of concrete.

Since full member capacities cannot be realized, the behavior of this system is entirely dependent on the performance of the connections.

### 4.4.1.6 Frames Not Part of the Lateral-Force-Resisting System

**Commentary:**

This section deals with secondary components consisting of frames that were not designed to be part of the lateral-force-resisting system. These are basic structural frames of steel or concrete that are designed for gravity loads only. Shear walls or other vertical elements provide the resistance to lateral forces. In actuality, however, all frames act as part of the lateral-force-resisting system. Lateral drifts of the building will induce forces in the beams and columns of the secondary frames. Furthermore, in the event that the primary elements fail, the secondary frames become the primary lateral force resisting components of the building.

If the walls are concrete (infilled in steel frames or monolithic in concrete frames), the building should be treated as a concrete shear wall building (Types C2 or C2A) with the frame columns as boundary elements. If the walls are masonry infills, the frames should be treated as steel or concrete frames with infill walls of masonry (Types S5, S5A, C3 or C3A). Research is continuing on the behavior of infill frames. Lateral forces are resisted by compression struts that develop in the masonry infill and induce forces on the frame elements eccentric to the joints.

The concern for secondary frames is the potential loss of vertical-load-carrying capacity due to excessive deformations and p-delta effects.

**4.4.1.6.1 COMPLETE FRAMES: Steel or concrete frames classified as secondary components shall form a complete vertical load carrying system.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The gravity and seismic demands for the shear walls shall be calculated and the adequacy of the shear walls shall be evaluated.

**Commentary:**

If the frame does not form a complete vertical load carrying system, the walls will be required to provide vertical support as bearing walls. (see Figure 4-16). A frame is incomplete if there are no columns cast into the wall, there are no columns adjacent to the wall, and beams frame into the wall, supported solely by the wall.

During an earthquake, shear walls might become damaged by seismic forces, limiting their ability to support vertical loads. Loss of vertical support may lead to partial collapse.

Compliance can be demonstrated if the wall is judged adequate for combined vertical and seismic forces.

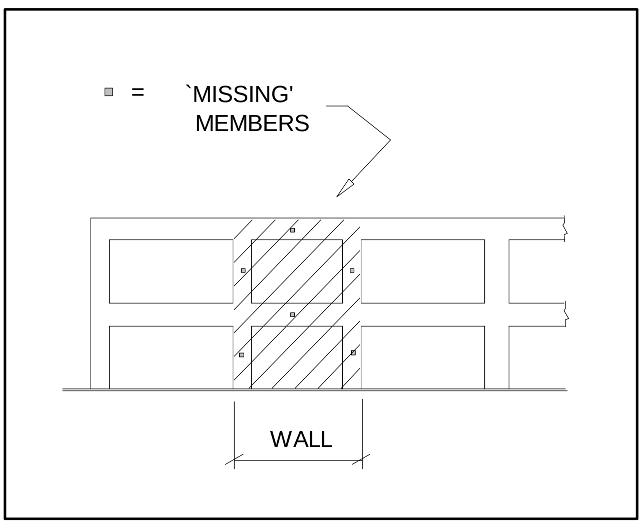



Figure 4-16. Incomplete Frame

**4.4.1.6.2 DEFLECTION COMPATIBILITY: Secondary components shall have the shear capacity to develop the flexural strength of the elements for Life Safety and shall have ductile detailing for Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The flexural and shear demands at maximum interstory drifts for non-compliant elements shall be calculated and the adequacy of the elements shall be evaluated.

**Commentary:**

Frame components, especially columns, that are not specifically designed to participate in the lateral-force-resisting system will still undergo displacements associated with overall seismic interstory drifts. If the columns are located some distance away from the lateral-force-resisting elements, the added deflections due to semi-rigid floor diaphragms will increase the drifts. Stiff columns, designed for potentially high gravity loads, may develop significant bending moments due to the imposed drifts. The moment-axial force interaction may lead to a nonductile failure of the columns and a collapse of the building.

**4.4.1.6.3 FLAT SLABS: Flat slab/plates classified as secondary components shall have continuous bottom steel through the column joints for Life Safety. Flat slabs/plates shall not be permitted for the Immediate Occupancy Performance Level.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the joint for punching shear for all gravity and seismic demands, and shear transfer due to seismic moments, shall be evaluated.

**Commentary:**

Flat slabs not designed to participate in the lateral-force-resisting system may still experience seismic forces due to displacements associated with overall building drift. The concern is the transfer of the shear and bending forces between the slab and column, which could result in a punching shear failure.

Continuity of some bottom reinforcement through the column joint will assist in the transfer of forces and provide some resistance to collapse by catenary action in the event of a punching shear failure (see Figure 4-17). Bars can be considered continuous if they have proper lap splices, mechanical couplers, or are developed beyond the support.

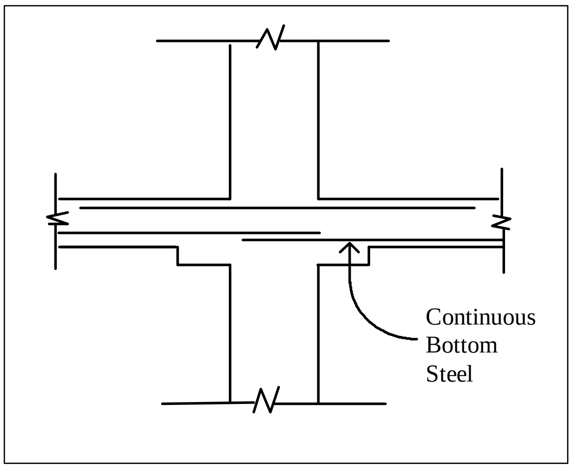



Figure 4-17. Continuous Bottom Steel

## 4.4.2 Shear Walls

### 4.4.2.1 General

**Commentary:**

Shear walls, as the name implies, resist lateral forces primarily in shear. In the analysis of shear walls, it is customary to consider the shear taken by the length of the wall and the flexure taken by vertical reinforcement added at each end, much as flexure in diaphragms is designed to be taken by chords at the edges. Squat walls that are long compared to their height, are dominated by shearing behavior. Flexural forces require only a slight local reinforcement at each end. Slender walls that are tall compared to their length are usually dominated by flexural behavior, and may require substantial boundary elements at each end.

It is a good idea to sketch a complete free-body diagram of the wall (as indicated in Figure 4-18) so that no forces are inadvertently neglected. An error often made in the design of wood shear walls is to treat the walls one story at a time, considering only the shear force in the wall and overlooking the accumulation of overturning forces from the stories above.

When the earthquake direction being considered is parallel to a shear wall, the wall develops in-plane shear and flexural forces as described above. When the earthquake direction is perpendicular to a shear wall, the wall contributes little to the lateral force resistance of the building and the wall is subjected to out-of-plane forces tending to separate it from the rest of the structure. This section addresses the in-plane behavior of shear walls. Out-of-plane strength and anchorage of shear walls to the structure is addressed in Section 4.5.

Solid shear walls usually have sufficient strength, though they may be lightly reinforced. Problems with shear wall systems arise when walls are not continuous to the foundation, or when numerous openings break the walls up into small piers with limited shear and flexural capacity.

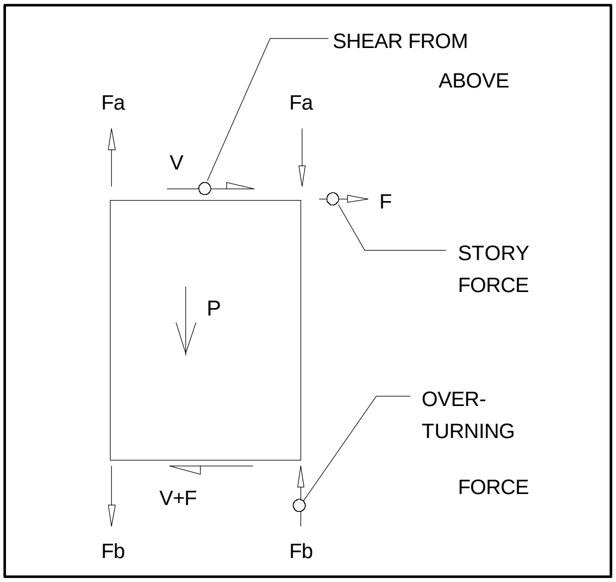



Figure 4-18. Wall Free-Body Diagram.

**4.4.2.1.1 REDUNDANCY: The number of lines of shear walls in each direction shall be greater than or equal to 2 for Life Safety and for Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with the procedures in Section 4.2 shall be performed. The adequacy of all walls and connections shall be evaluated.

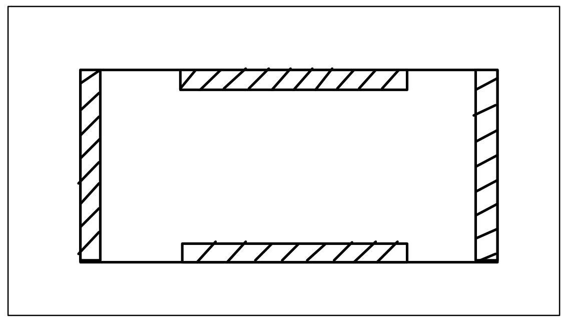
**Commentary:**

Redundancy is a fundamental characteristic of lateral force resisting systems with superior seismic performance. Redundancy in the structure will ensure that if an element in the lateral force resisting system fails for any reason, there is another element present that can provide lateral force resistance. Redundancy also provides multiple locations for potential yielding, distributing inelastic activity throughout the structure and improving ductility and energy absorption. Typical characteristics of redundancy include multiple lines of resistance to distribute the lateral forces uniformly throughout the structure, (see Figure 4-19) and multiple bays in each line of resistance to reduce the shear and axial demands on any one element.

A distinction should be made between redundancy and adequacy. For the purpose of this Handbook, redundancy is intended to mean simply "more than one". That is not to say that for large buildings two elements is adequate, or for small buildings one is not enough. Separate evaluation statements are present in the Handbook to determine the adequacy of the elements provided.

When redundancy is not present in the structure, an analysis which demonstrates the adequacy of the lateral force elements is required.

Figure 4-19. Redundancy in Shear Walls



#### 4.4.2.2 Concrete Shear Walls

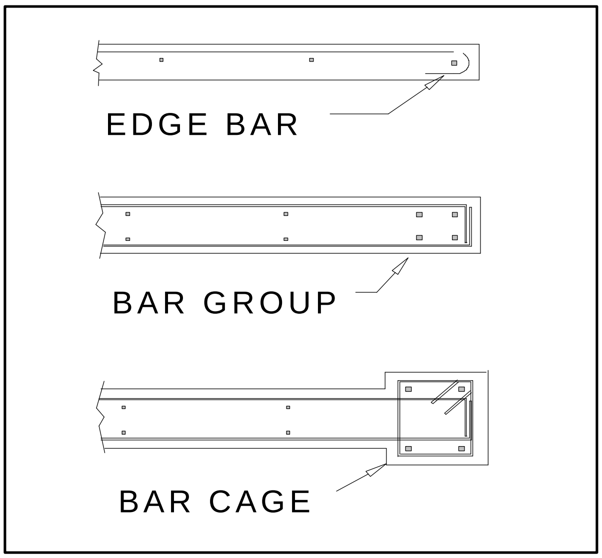
**Commentary:**

In highly redundant buildings with many long walls, stresses in concrete shear walls are usually low. In less redundant buildings with large openings and slender walls, the stresses can be high. In the ultimate state, when overturning forces are at their highest, a thin wall may fail in buckling along the compression edge, or it may fail in tension along the tension edge. Tension failures may consist of slippage in bar lap splices, or bar yield and fracture if adequate lap splices have been provided.

In the past, designs have been based on liberal assumptions about compression capacity, and have simply packed vertical rebar into the ends of the walls to resist the tensile forces. Recent codes, recognizing the importance of boundary members, have special requirements for proportions, bar splices, and transverse reinforcement. Examples of boundary members with varying amounts of reinforcing are shown in Figure 4-20. Existing buildings often do not have these elements, and the acceptance criteria are designed to allow for this.

Another development in recent codes is the requirement to provide shear strength compatible with the flexural capacity of the wall to ensure ductile flexural yielding prior to brittle shear failure. Long continuous walls and walls with embedded steel or large boundary elements can have high flexural capacities with the potential to induce correspondingly high shear demands that are over and above the minimum design shear demands.

Figure 4-20. Boundary Elements.




**4.4.2.2.1 SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 3.5.3.3, shall be less than 100 psi or $2\sqrt{f'c}$ for Life Safety and Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the concrete shear wall elements shall be evaluated using the *m*-factors in Table 4-4.

**Commentary:**

The shear stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building.

**4.4.2.2.2 REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area shall be greater than 0.0015 in the vertical direction and 0.0025 in the horizontal direction for Life Safety and Immediate Occupancy. The spacing of reinforcing steel shall be equal to or less than 18" for Life Safety and for Immediate Occupancy**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the concrete shear wall elements shall be evaluated using the *m*-factors in Table 4-4.

**Commentary:**

If the walls do not have sufficient reinforcing steel, they will have limited capacity in resisting seismic forces. The wall will also behave in a nonductile manner for inelastic forces.

**4.4.2.2.3 COUPLING BEAMS: The stirrups in all coupling beams over means of egress shall be spaced at or less than d/2 and shall be anchored into the core with hooks of 135° or more for Life Safety and Immediate Occupancy. In addition, the beams shall have the capacity in shear to develop the uplift capacity of the adjacent wallfor Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The shear and flexural demands on non-compliant coupling beams shall be calculated and the adequacy of the coupling beams shall be evaluated. If the coupling beams are inadequate, the adequacy of the coupled walls shall be evaluated as if they were independent.

**Commentary:**

Coupling beams with sufficient strength and stiffness can increase the lateral stiffness of the system significantly beyond the stiffnesses of the independent walls. When the walls deflect laterally, large moments and shears are induced in the coupling beams as they resist the imposed deformations. Coupling beams also link the coupled walls for overturning resistance (see Figure 4-21).

Coupling beam reinforcement is often inadequate for the demands that can be induced by the movement of the coupled walls. Seismic forces may damage and degrade the beams so severely that the system degenerates into a pair of independent walls. This changes the distribution of overturning forces which may result in potential stability problems for the independent walls. The boundary reinforcement may also be inadequate for flexural demands if the walls act independently.

If the beams are lightly reinforced, their degradation could result in falling debris that is a potential life-safety hazard, especially at locations of egress.

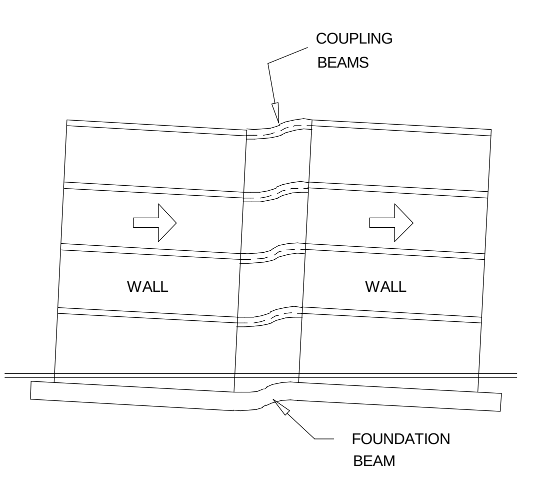



Figure 4-21. Coupled Walls.

**4.4.2.2.4 OVERTURNING: All shear walls shall have aspect ratios less than 4 to 1. Wall piers need not be considered. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The overturning demands for non-compliant walls shall be calculated and the adequacy of the shear walls shall be evaluated.

**Commentary:**

Tall, slender shear walls may have limited overturning resistance. Displacements at the top of the building will be greater than anticipated if overturning forces are not properly resisted.

Often sufficient resistance can be found in immediately adjacent bays, if a load path is present to activate the adjacent column dead loads.

**4.4.2.2.5 CONFINEMENT REINFORCING: For shear walls with aspect ratios greater than 2.0, the boundary elements shall be confined with spirals or ties with spacing less than 8 $d_b$. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The shear and flexural demands on the non-compliant walls shall be calculated and the adequacy of the shear walls shall be evaluated.

**Commentary:**

Fully effective shear walls require boundary elements to be properly confined with closely spaced ties (see Figure 4-20). Degradation of the concrete in the vicinity of the boundary elements can result in buckling of rebar in compression and failure of lap splices in tension. Non-ductile failure of the boundary elements will lead to reduced capacity to resist overturning forces.

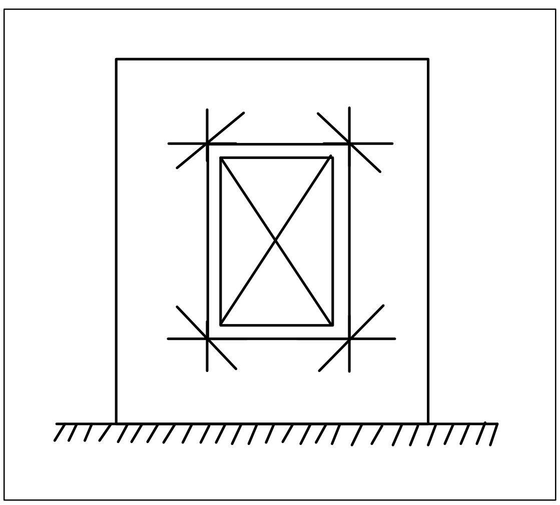
**4.4.2.2.6 REINFORCING AT OPENINGS: There shall be added trim reinforcement around all**

**openings. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The flexural and shear demands around the openings shall be calculated and the adequacy of the piers and spandrels shall be evaluated.

Figure 4-22. Conventional Trim Steel

> **Commentary:**
>
> Conventional trim steel is adequate only for small openings (see Figure 4-22). Large openings will cause significant shear and flexural stresses in the adjacent piers and spandrels. Inadequate reinforcing steel around these openings will lead to strength deficiencies, nonductile performance and degradation of the wall.



**4.4.2.2.7 WALL THICKNESS: Thickness of bearing walls shall not be less than 1/25 the minimum unsupported height or length, nor less than 4". This statement shall apply to the Immediate Occupancy Performance Level only**

**Tier 2 Evaluation Procedure:** The adequacy of the walls to resist out-of-plane forces in combination with vertical loads shall be evaluated.

> **Commentary:**
>
> Slender bearing walls may have limited capacity for vertical loads and higher potential for damage due to out-of-plane forces and magnified moments. Note that this condition is not considered a life-safety concern and need only be examined for the Immediate Occupancy performance level.

**4.4.2.2.8 WALL CONNECTIONS: There shall be a positive connection between the shear walls and the steel beams and columns for Life Safety, and the connection shall be able to develop the strength of the walls for Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The shear and flexural demands on the shear walls shall be calculated and the adequacy of the connection to transfer shear between the walls and the steel frame shall be evaluated.

> **Commentary:**
>
> Insufficient shear transfer between the steel and concrete elements will limit the ability of the steel to contribute to the performance of the shear walls. The connections to the column are especially important as the columns will develop a portion of the shear wall overturning moment. The connections should include welded studs, welded reinforcing steel, or fully encased steel elements with longitudinal reinforcing and ties.

Shear friction between the concrete and steel should only be used when the steel is completely encased in the concrete.

**4.4.2.2.9 COLUMN SPLICES: Steel columns encased in shear wall boundary elements shall have splices that develop the tensile strength of the column. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The tension demands due to overturning forces on non-compliant columns shall be calculated and the adequacy of the splice connections shall be evaluated.

**Commentary:**

Columns encased in shear wall boundary elements may be subjected to high tensile forces due to shear wall overturning moments. If the splice cannot develop the strength of the column, the ability of the column to contribute to overturning resistance will be limited.

The presence of axial loads may reduce the net tensile demand on the boundary element columns to a level below the capacity of the splice.

### 4.4.2.3 Precast Concrete Shear Walls

**Commentary:**

Precast concrete shear walls are constructed in segments that are usually interconnected by embedded steel elements. These connections usually possess little ductility, but are important to the overall behavior of the wall assembly. Interconnection between panels increases the overturning capacity by transferring overturning demands to end panels. Panel connections at the diaphragm are often used to provide continuous diaphragm chords. Failure of these connections will reduce the capacity of the system.

**4.4.2.3.1 SHEAR STRESS CHECK: The shear stress in the precast panels, calculated using the Quick Check Procedure of Section 3.5.3.3, shall be less than 100 psi or $2\sqrt{f'c}$ for Life Safety and Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the concrete shear wall elements shall be evaluated using the *m*-factors in Table 4-4.

**Commentary:**

The shear stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building.

**4.4.2.3.2 REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area shall be greater than 0.0015 in the vertical direction and 0.0025 in the horizontal direction for Life Safety and Immediate Occupancy. The spacing of reinforcing steel shall be equal to or less than 18" for Life Safety and Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the concrete shear wall elements shall be evaluated using the *m*-factors in Table 4-4.

**Commentary:**

If the walls do not have sufficient reinforcing steel, they will have limited capacity in resisting seismic forces. The wall will also behave in a nonductile manner for inelastic forces.

**4.4.2.3.3 WALL OPENINGS: Openings shall constitute less than 75% of the length of any perimeter wall for Life Safety and 50% for Immediate Occupancy with the wall piers having aspect ratios of less than 2.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the remaining wall shall be evaluated for shear and overturning resistance, and the adequacy of the shear transfer connection between the diaphragm and the wall shall be evaluated. The adequacy of the connection between any collector elements and the wall shall also be evaluated.

**Commentary:**

In tilt-up construction, typical wall panels are often of sufficient length that special detailing for collector elements, shear transfer, and overturning resistance is not provided. Perimeter walls that are substantially open, such as at loading docks, have limited wall length to resist seismic forces, and may be subject to overturning or shear transfer problems that were not accounted for in the original design.

Walls will be compliant if an adequate load path for shear transfer, collector forces and overturning resistance can be demonstrated.

**4.4.2.3.4 CORNER OPENINGS: Walls with openings at a building corner larger than the width of a typical panel shall be connected to the remainder of the wall with collector reinforcing.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the diaphragm to transfer shear and spandrel panel forces to the remainder of the wall beyond the opening shall be evaluated.

**Commentary:**

Open corners often are designed as entrances with the typical wall panel replaced by a spandrel panel and a glass curtain wall. Seismic forces in these elements are resisted by adjacent panels and, therefore, must be delivered through collectors.

If the spandrel and other wall elements are adequately tied to the diaphragm, panel forces can be transferred back to adjacent wall panels through collector elements in the diaphragm.

**4.4.2.3.5 PANEL-TO-PANEL CONNECTIONS: Adjacent wall panels shall be interconnected to transfer overturning forces between panels by methods other than steel welded inserts. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The overturning demands shall be calculated and the adequacy of the welded inserts to transfer overturning forces shall be evaluated as force controlled elements in accordance with Section 4.2.4.3.2.

**Commentary:**

Welded steel inserts can be brittle and may not be able to transfer the overturning forces between panels. Latent stresses may be present due to shrinkage and temperature effects. Brittle failure may include weld fracture, pull-out of the embedded anchors, or spalling of the concrete.

Failure of these connections will result in separation of the wall panels, and a reduction in overturning resistance.

**4.4.2.3.6 WALL THICKNESS: Thickness of bearing walls shall not be less than 1/25 the minimum unsupported height or length, nor less than 4". This statement shall apply to the Immediate Occupancy Performance Level only**

**Tier 2 Evaluation Procedure:** The adequacy of the walls to resist out-of-plane forces shall be evaluated.

> **Commentary:**
>
> Slender bearing walls may have limited capacity for vertical loads and higher potential for damage due to out-of-plane forces and magnified moments. Note that this condition is not considered a life-safety concern and only needs to be examined for the Immediate Occupancy performance level.

### 4.4.2.4 Reinforced Masonry Shear Walls

**4.4.2.4.1 SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check Procedure of Section 3.5.3.3, shall be less than 50 psi for Life Safety and Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the reinforced masonry shear wall elements shall be evaluated using the *m*-factors in Table 4-5.

> **Commentary:**
>
> The shear stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building.

**4.4.2.4.2 REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls shall be greater than 0.002 for Life Safety and 0.003 for Immediate Occupancy with the minimum of 0.0007 for Life Safety and 0.001 for Immediate Occupancy in either of the two directions; the spacing of reinforcing steel shall be less than 48" for Life Safety and 24" for Immediate Occupancy, and all vertical bars shall extend to the top of the walls.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the reinforced masonry shear wall elements shall be evaluated using the *m*-factors in Table 4-5.

> **Commentary:**
>
> If the walls do not have sufficient reinforcing steel, they will have limited capacity in resisting seismic forces. The wall will also behave in a nonductile manner for inelastic forces.

**4.4.2.4.3 REINFORCING AT OPENINGS: All wall openings that interrupt rebar shall have trim reinforcing on all sides. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The flexural and shear demands around the openings shall be calculated and the adequacy of the walls shall be evaluated using only the length of the piers between reinforcing steel.

> **Commentary:**
>
> Conventional trim steel is adequate only for small openings. Large openings will cause significant shearing and flexural stresses in the adjacent piers and spandrels. Inadequate reinforcing steel around these openings will lead to strength deficiencies, non-ductile performance and degradation of the wall.

**4.4.2.4.4 PROPORTIONS: The height-to-thickness ratio of the shear walls at each story shall be less than 30. This statement shall apply to the Immediate Occupancy Performance Level only**

**Tier 2 Evaluation Procedure:** The adequacy of the walls to resist out-of-plane forces in combination with vertical loads shall be evaluated.

> **Commentary:**
>
> Slender bearing walls may have limited capacity for vertical loads and higher potential for damage due to out-of-plane forces and magnified moments. Note that this condition is not considered a life-safety concern and need only be examined for the Immediate Occupancy performance level.

### 4.4.2.5 Unreinforced Masonry Shear Walls

**4.4.2.5.1 SHEAR STRESS CHECK: The shear stress in the unreinforced masonry shear walls, calculated using the Quick Check Procedure of Section 3.5.3.3, shall be less than 15 psi for clay units and 30 psi for concrete units for Life Safety and Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the unreinforced masonry shear wall elements shall be evaluated using the *m*-factors in Table 4-5.

> **Commentary:**
>
> The shear stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building.

**4.4.2.5.2 PROPORTIONS: The height-to-thickness ratio of the shear walls at each story shall be less than the following for Life Safety and for Immediate Occupancy:**

| | |
|---|---|
| **Top story of multi-story building:** | **9** |
| **First story of multi-story building:** | **15** |
| **All other conditions:** | **13** |

**Tier 2 Evaluation Procedure:** No Tier 2 evaluation procedure is available for unreinforced masonry shear wall proportions in non-compliance. A Tier 3 evaluation is necessary to achieve the selected performance level.

> **Commentary:**
>
> Slender unreinforced masonry bearing walls with large height-to-thickness ratios have a potential for damage due to out-of-plane forces which may result in falling hazards and potential collapse of the structure.

**4.4.2.5.3 MASONRY LAY-UP: Filled collar joints of multiwythe masonry walls shall have negligible voids.**

**Tier 2 Evaluation Procedure:** No Tier 2 evaluation procedure is available for unreinforced masonry shear wall proportions in non-compliance. A Tier 3 evaluation is necessary to achieve the selected performance level.

> **Commentary:**
>
> When walls have poor collar joints, the inner and outer wythes will act independently. The walls may be inadequate to resist out-of-plane forces due to a lack of composite action between the inner and outer wythes.
>
> Mitigation to provide out-of-plane stability and anchorage of the wythes may be necessary to achieve the selected performance level.

### 4.4.2.6 Infill Walls in Frames

**4.4.2.6.1 WALL CONNECTIONS: All infill walls shall have a positive connection to the frame to resist out-of-plane forces for Life Safety, and the connection shall be able to develop the out-of-plane strength of the wall for Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The out-of-plane demands on the wall shall be calculated and the adequacy of the connection to the frame shall be evaluated.

**Commentary:**

Performance of frame buildings with masonry infill walls is dependent upon the interaction between the frame and infill panels. In-plane lateral force resistance is provided by a compression strut developing in the infill panel that extends diagonally between corners of the frame. If gaps exist between the frame and infill, this strut cannot be developed (see Figure 4-23). If the infill panels separate from the frame due to out-of-plane forces, the strength and stiffness of the system will be determined by the properties of the bare frame, which may not be detailed to resist seismic forces. Severe damage or partial collapse due to excessive drift and p-delta effects may occur.

A positive connection is needed to anchor the infill panel for out-of-plane forces. In this case, a positive connection can consist of a fully grouted bed joint in full contact with the frame, or complete encasement of the frame by the brick masonry. The mechanism for out-of-plane resistance of infill panels is discussed in the commentary to Section 4.4.2.6.2.

If the connection is non-existent, mitigation with adequate connection to the frame is necessary to achieve the selected performance level.

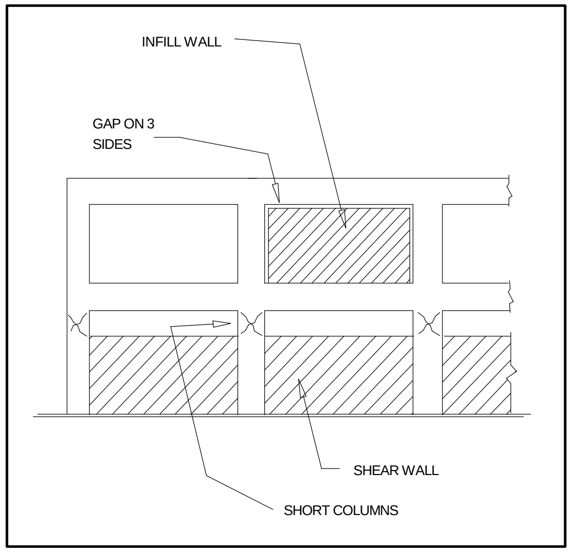



Figure 4-23. Infill Wall

**4.4.2.6.2 PROPORTIONS: The height-to-thickness ratio of the infill walls at each story shall be less than 9 for Life Safety in regions of high seismicity, 13 for Immediate Occupancy in regions of moderate seismicity, and 8 for Immediate Occupancy in regions of high seismicity.**

**Tier 2 Evaluation Procedure:** No Tier 2 evaluation procedure is available for unreinforced masonry shear wall proportions in non-compliance. A Tier 3 evaluation is necessary to demonstrate compliance with the selected performance level.

**Commentary:**

Slender masonry infill walls with large height-to-thickness ratios have a potential for damage due to out-of-plane forces. Failure of these walls out-of-plane will result in falling hazards and degradation of the strength and stiffness of the lateral force resisting system.

The out-of-plane stability of infill walls is dependent on many factors including flexural strength of the wall and confinement provided by the surrounding frame. If the infill is unreinforced, the flexural strength is limited by the flexural tension capacity of the material. The surrounding frame will provide confinement, induce infill thrust forces and develop arching action against out-of-plane forces. The height-to-thickness limits in the evaluation statement are based on arching action models that will exceed any plausible acceleration levels in various seismic zones.

Further investigation of non-compliant infill panels requires a Tier 3 level analysis.

**4.4.2.6.3 SOLID WALLS: The infill walls shall not be of cavity construction.**

**Tier 2 Evaluation Procedure:** No Tier 2 evaluation procedure is available for infill walls in non-compliance.

**Commentary:**

When the infill walls are of cavity construction, the inner and outer wythes will act independently. due to a lack of composite action, increasing the potential for damage from out-of-plane forces. Failure of these walls out-of-plane will result in falling hazards and degradation of the strength and stiffness of the lateral force resisting system.

Mitigation to provide out-of-plane stability and anchorage of the wythes is necessary to achieve the selected performance level.

**4.4.2.6.4 INFILL WALLS: The infill walls shall be continuous to the soffits of the frame beams.**

**Tier 2 Evaluation Procedure:** The adequacy of the columns adjacent to non-conforming infill walls shall be evaluated for the shear force required to develop the flexural capacity of the column over the clear height above the infill.

**Commentary:**

Discontinuous infill walls occur when full bay windows or ventilation openings are provided between the top of the infill and bottom soffit of the frame beams. The portion of the column above the infill is a short captive column which may attract large shear forces due to increased stiffness relative to other columns (see Figure 4-24). Partial infill walls will also develop compression struts with horizontal components that are highly eccentric to the beam column joints. If not adequately detailed, concrete columns may suffer a non-ductile shear failure which may result in partial collapse of the structure. Because steel columns are not subject to the same kind of brittle failure, this is not generally considered a concern in steel frame infill buildings.

A column that can develop the shear capacity to develop the flexural strength over the clear height above the infill will have some ductility to prevent sudden catastrophic failure of the vertical support system.

### 4.4.2.7 Walls in Wood-Frame Buildings

**4.4.2.7.1 SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check Procedure of 3.5.3.3, shall be less than the following values for Life Safety and Immediate Occupancy:**

| | |
|---|---|
| **Structural panel sheathing:** | **1000 plf** |
| **Diagonal sheathing:** | **700 plf** |
| **Straight sheathing:** | **80 plf** |
| **All other conditions:** | **100 plf** |

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the wood shear wall elements shall be evaluated using the *m*-factors in Table 4-6.

**Commentary:**

The shear stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building.

**4.4.2.7.2 STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multistory buildings shall not rely on exterior stucco walls as the primary lateral-force-resisting system.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The overturning and shear demands for non-compliant walls shall be calculated and the adequacy of the stucco shear walls shall be evaluated using the *m*-factors in Table 4-6.

**Commentary:**

Exterior stucco walls are often used (intentionally and unintentionally) for resisting seismic forces. Stucco is relatively stiff, but brittle, and the shear capacity is limited. Building movements due to differential settlement, temperature changes and earthquake or wind forces can cause cracking in the stucco and loss of lateral strength. Lateral force resistance is unreliable because sometimes the stucco will delaminate from the framing and the system is lost. Multistory buildings should not rely on stucco walls as the primary lateral-force-resisting system.

**4.4.2.7.3 GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard shall not be used as shear walls on buildings over one story in height.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The overturning and shear demands for non-compliant walls shall be calculated and the adequacy of the gypsum wallboard or plaster shear walls shall be evaluated using the *m*-factors in Table 4-6.

**Commentary:**

Gypsum wallboard or gypsum plaster sheathing tends to be easily damaged by differential foundation movement or earthquake shaking.

Though the capacity of these walls is low, most residential buildings have numerous walls constructed with plaster or gypsum wallboard. As a result, plaster and gypsum wallboard walls may provide adequate resistance to moderate earthquake shaking.

One problem that can occur is incompatibility with other lateral-forcing-resisting elements. For example, narrow plywood shear walls are more flexible than long stiff plaster walls; as a result, the plaster or gypsum walls will take all the load until they fail and then the plywood walls will start to resist the lateral loads. In multistory buildings, plaster or gypsum wallboard walls should not be used for shear walls except in the top story.

**4.4.2.7.4 NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2 to 1 for Life Safety and 1.5 to 1 for Immediate Occupancy shall not be used to resist lateral forces developed in the building.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The overturning and shear demands for non-compliant walls shall be calculated and the adequacy of the narrow shear walls shall be evaluated using the *m*-factors in Table 4-6.

**Commentary:**

Narrow shear walls are highly stressed and subject to severe deformations that will damage the capacity of the walls. Most of the damage occurs at the base, and consists of sliding of the sill plate and deformation of hold-down anchors when present. As the deformation continues, the plywood pulls up on the sill plate causing splitting. Splitting of the end studs at the bolted attachment of hold down anchors is also common.

**4.4.2.7.5 WALLS CONNECTED THROUGH FLOORS: Shear walls shall have interconnection between stories to transfer overturning and shear forces through the floor.**

**Tier 2 Evaluation Procedure:** No Tier 2 evaluation procedure is available for walls in non-compliance.

**Commentary:**

In platform construction, wall framing is discontinuous at floor levels. The concern is that this discontinuity will prevent shear and overturning forces from being transferred between shear walls in adjacent stories.

Mitigation with elements or connections needed to complete the load path is necessary to achieve the selected performance level.

**4.4.2.7.6 HILLSIDE SITE: For a sloping site greater than one-half story, all shear walls on the downhill slope shall have an aspect ratio less than 1 to 1 for Life Safety and 1 to 2 for Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The shear and overturning demands on the downhill slope walls shall be calculated including the torsional effects of the hillside. The adequacy of the shear walls on the downhill slope shall be evaluated.

**Commentary:**

Buildings on a sloping site will experience significant torsion during an earthquake. Taller walls on the downhill slope are more flexible than the supports on the uphill slope. Therefore, significant displacement and racking of the shear walls on the downhill slope will occur. If the walls are narrow, significant damage or collapse may occur.

**4.4.2.7.7 CRIPPLE WALLS: All cripple walls below first floor level shear walls shall be braced to the foundation with shear elements.**

**Tier 2 Evaluation Procedure:** No Tier 2 evaluation procedure is available for cripple walls in non-compliance.

**Commentary:**

Cripple walls are short stud walls that enclose a crawl space between the first floor and the ground. Often there are no other walls at this level, and these walls have no stiffening elements other than architectural finishes. If this sheathing fails, the building will experience significant damage and, in the extreme case, may fall off its foundation. To be effective, all exterior cripple walls below the first floor level should have adequate shear strength, stiffness, and proper connection to the floor and foundation. Cripple walls that change height along their length, such as along sloping walls on hillside sites, will not have a uniform distribution of shear along the length of the wall, due to the varying stiffness. These walls may be subject to additional damage on the uphill side due to concentration of shear demand.

Mitigation with shear elements needed to complete the load path is necessary to achieve the selected performance level.

**4.4.2.7.8 OPENINGS: Walls with garage doors or other large openings shall be braced with plywood shear walls or shall be supported by adjacent construction through substantial positive ties. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The overturning and shear demands on non-compliant walls shall be calculated and the adequacy of the shear walls shall be evaluated using the *m*-factors in Table 4-6

**Commentary:**

Walls with large openings may have little or no resistance to shear and overturning forces. They must be specially detailed to resist these forces, or braced to other parts of the structure with collectors. Special detailing and collectors are not part of conventional construction procedures. Lack of this bracing can lead to collapse of the wall.

**4.4.2.7.9 HOLD-DOWN ANCHORS: All walls shall have properly constructed hold-down anchors. This statement shall apply to the Immediate Occupancy Performance Level only**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The overturning and shear demands for non-compliant walls shall be calculated and the adequacy of the shear walls shall be evaluated using the *m*-factors in Table 4-6.

**Commentary:**

Buildings without hold-down anchors may be subject to significant damage due to uplift and racking of the shear walls. Note that this condition is not considered a life-safety concern and only needs to be examined for the Immediate Occupancy performance level.

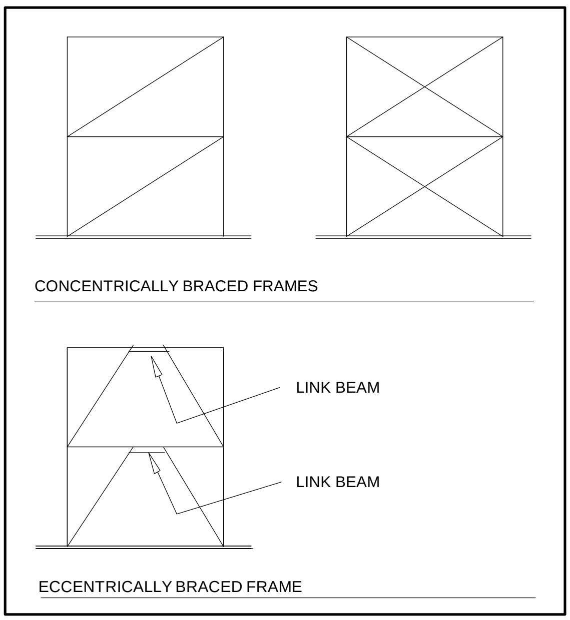
## 4.4.3 Braced Frames

**Commentary:**

Braced frames develop their lateral force resistance through axial forces developed in the diagonal bracing members. The braces induce forces in the associated beams and columns, and all are subjected to stresses that are primarily axial. When the braces are eccentric to beam/column joints, members are subjected to shear and flexure in addition to axial forces. A portal frame with knee braces near the frame joints is one example.

Braced frames are classified as either concentrically braced frames or eccentrically braced frames (see Figure 4-24). Concentrically braced frames have braces that frame into beam/column joints or concentric connections with other braces. Minor connection eccentricities may be present and are accounted for in the design. Eccentrically braced frames have braces that are purposely located away from joints, and connections that are intended to induce shear and flexure demands on the members. The eccentricity is intended to force a concentration of inelastic activity at a predetermined location that will control the behavior of the system. Modern eccentrically braced frames are designed with strict controls on member proportions and special out-of-plane bracing at the connections to ensure the frame behaves as intended.

Figure 4-24. Braced Frames




### 4.4.3.1 General

**4.4.3.1.1 REDUNDANCY: The number of lines of braced frames in each principal direction shall be greater than or equal to 2 for Life Safety and Immediate Occupancy. The number of braced bays in each line shall be greater than 2 for Life Safety and 3 for Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with the procedures in Section 4.2 shall be performed. The adequacy of all elements and connections in the braced frames shall be evaluated.

> **Commentary:**
>
> Redundancy is a fundamental characteristic of lateral force resisting systems with superior seismic performance. Redundancy in the structure will ensure that if an element in the lateral force resisting system fails for any reason, there is another element present that can provide lateral force resistance. Redundancy also provides multiple locations for potential yielding, distributing inelastic activity throughout the structure and improving ductility and energy absorption. Typical characteristics of redundancy include multiple lines of resistance to distribute the lateral forces uniformly throughout the structure, and multiple bays in each line of resistance to reduce the shear and axial demands on any one element.
>
> A distinction should be made between redundancy and adequacy. For the purpose of this Handbook, redundancy is intended to mean simply "more than one". That is not to say that for large buildings two elements is adequate, or for small buildings one is not enough. Separate evaluation statements are present in the Handbook to determine the adequacy of the elements provided.
>
> When redundancy is not present in the structure, an analysis which demonstrates the adequacy of the lateral force elements is required.

**4.4.3.1.2 AXIAL STRESS CHECK: The axial stress in the diagonals, calculated using the Quick Check Procedure of Section 3.5.3.4, shall be less than 18 ksi or 0.50$F_y$ for Life Safety and Immediate Occupancy.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the braced frame elements shall be evaluated using the *m*-factors in Table 4-3.

> **Commentary:**
>
> The axial stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building.

**4.4.3.1.3 STIFFNESS OF DIAGONALS: All diagonal elements required to carry compression shall have Kl/r ratios less than 120. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The compression demands in non-compliant braces shall be calculated and the adequacy of the braces shall be evaluated for buckling.

> **Commentary:**
>
> Code design requirements have allowed compression diagonal braces to have Kl/r ratios of up to 200. Cyclic test have demonstrated that elements with high Kl/r ratios are subjected to large buckling deformations resulting in brace or connection fractures. They cannot be expected to provide adequate performance. Limited energy dissipation and premature buckling can significantly reduce strength, increase the building displacements and jeopardize the performance of the framing system.

**4.4.3.1.4 CONNECTION STRENGTH: All the brace connections shall develop the yield capacity of the diagonals. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The demands on the non-compliant connections shall be calculated and the adequacy of the brace connections shall be evaluated.

**4.4.3.1.5 COLUMN SPLICES: All column splice details located in braced frames shall develop the tensile strength of the column. This statement shall apply to the Immediate Occupancy Performance Level only.**

> **Commentary:**
>
> Since connection failures are usually nonductile in nature, it is more desirable to have inelastic behavior in the members.

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The tension demands on non-compliant columns shall be calculated and the adequacy of the splice connections shall be evaluated.

> **Commentary:**
>
> Columns in braced frames may be subject to large tensile forces. A connection that is unable to resist this tension may limit the ability of the frame to resist lateral forces. Columns may uplift and slide off bearing supports, resulting in a loss of vertical support and partial collapse.

**4.4.3.1.6 OUT-OF-PLANE BRACING: Braced frame connections attached to beam bottom flanges located away from beam-column joints shall be braced out-of-plane at the bottom flange of the beams. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The demands shall be calculated and the adequacy of the beam shall be evaluated considering a horizontal out-of-plane force equal to 2% of the brace compression force acting concurrently at the bottom flange of the beam.

> **Commentary:**
>
> Brace connections at beam bottom flanges that do not have proper bracing may have limited ability to resist seismic forces. Out-of-plane buckling may occur before the strength of the brace is developed. Connections to beam top flanges are braced by the diaphragm, so V-bracing need not be considered.
>
> This statement is intended to target chevron type bracing, where braces intersect the beam from below at a location well away from a column. Here only the beam can provide out-of-plane stability for the connection. At beam/column joints, the continuity of the column will provide stability for the connection.
>
> To demonstrate compliance, the beam is checked for the strength required to provide out-of-plane stability using the 2% rule.

### 4.4.3.2 Concentrically Braced Frames

> **Commentary:**
>
> Common types of concentrically braced frames are shown in Figure 4-25.
>
> Braces can consist of light tension-only rod bracing, double angles, pipes, tubes or heavy wide-flange sections.

Concrete braced frames are rare and are not permitted in some jurisdictions because it is difficult to detail the joints with the kind of reinforcing that is required for ductile behavior.

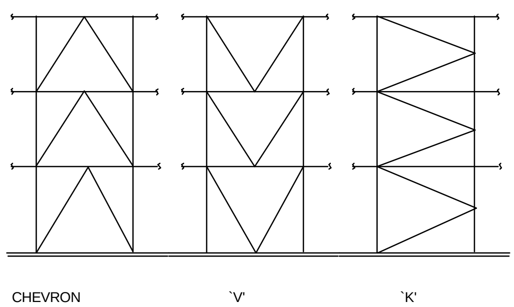



Figure 4-25. Bracing Types

**4.4.3.2.1 K-BRACING: The bracing system shall not include K-braced bays.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the columns shall be evaluated for all demands including concurrent application of the unbalanced force that can be applied to the column by the braces. The unbalanced force shall be taken as the horizontal component of the tensile capacity of one brace, assuming the other brace has buckled in compression. The *m*-factors in Table 4-3 shall be used.

**Commentary:**

In K-brace configurations, diagonal braces intersect the column between floor levels (see Figure 4-25). When the compression brace buckles, the column will be loaded with the horizontal component of the adjacent tension brace. This will induce large midheight demands that can jeopardize the stability of the column and vertical support of the building.

In most cases, columns have not been designed to resist this force. The risk to the vertical support system makes this an undesirable bracing configuration.

**4.4.3.2.2 TENSION-ONLY BRACES: Tension-only braces shall not comprise more than 70% of the total lateral-force-resisting capacity in structures over two stories in height. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the tension-only braces shall be evaluated using the *m*-factors in Table 4-3.

**Commentary:**

Tension-only brace systems may allow the brace to deform with large velocities during cyclic response after tension yielding cycles have occurred. Limited energy dissipation and premature fracture can significantly reduce the strength, increase the building displacements and jeopardize the performance of the framing system.

**4.4.3.2.3 CHEVRON BRACING: The bracing system shall not include chevron-, or V-braced bays. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The adequacy of the beams shall be evaluated for all demands including concurrent application of the unbalanced force that can be applied to the beams by the braces. The unbalanced force shall be taken as the vertical component of the tensile capacity of one brace, assuming the other brace has buckled in compression. The *m*-factors in Table 4-3 shall be used.

**Commentary:**

In chevron and V-brace configurations, diagonal braces intersect the beam between columns (see Figure 4-25). When the compression brace buckles, the beam will be loaded with the vertical component of the adjacent tension brace. This will induce large midspan demands on the beam that can jeopardize the support of the floor.

In most cases, beams have not been designed to resist this force. The risk to the vertical support system makes chevron and V-bracing undesirable bracing configurations.

**4.4.3.2.4 CONCENTRIC JOINTS: All the diagonal braces shall frame into the beam-column joints concentrically. This statement shall apply to the Immediate Occupancy Performance Level only.**

**Tier 2 Evaluation Procedure:** An analysis in accordance with Section 4.2 shall be performed. The axial, flexural, and shear demands including the demands due to eccentricity of the braces shall be calculated. The adequacy of the joints shall be evaluated.

**Commentary:**

Frames that have been designed as concentrically braced frames may have local eccentricities within the joint. A local eccentricity is where the lines of action of the bracing members do not intersect the centerline of the connecting members. These eccentricities induce additional flexural and shear stresses in the members that may not have been accounted for in the design. Excessive eccentricity can cause premature yielding of the connecting members or failures in the connections, thereby reducing the strength of the frames.

### 4.4.3.3 Eccentrically Braced Frames

No evaluation statements or Tier 2 procedures have been provided specifically for eccentrically braced frames. Eccentrically braced frames shall be checked for the general braced frame evaluation statements and Tier 2 procedures in Section 4.4.3.1.

**Commentary:**

Eccentrically braced frames have braces that are purposely located away from joints, and connections that are intended to induce shear and flexure demands on the members. The eccentricity is intended to force a concentration of inelastic activity at a predetermined location that will control the behavior of the system. Modern eccentrically braced frames are designed with strict controls on member proportions and special out-of-plane bracing at the connections to ensure the frame behaves as intended.

The eccentrically braced frame is a relatively new type of frame that is recognizable by a diagonal with one end significantly offset from the joints (Figure 4-26). As with any braced frame, the function of the diagonal is to provide stiffness and transmit lateral forces from the upper to the lower level. The unique feature of eccentrically braced frames an offset zone in the beam, called the "link". The link is specially detailed for controlled yielding This detailing is subject to very specific requirements, so an ordinary braced frame that happens to have an offset zone that looks like a link may not necessarily behave like an eccentrically braced frame.

An eccentrically braced frame has the following essential features:

1. There is a link beam at one end of each brace.
2. The length of the link beam is limited to control shear deformations and rotations due to flexural yielding at the ends of the link.
3. The brace and the connections are designed to develop forces consistent with the strength of the link.
4. When one end of a link beam is connected to a column, the connection is a full moment connection.
5. Lateral bracing is provided to prevent out-of-plane beam displacements that would compromise the intended action.

In most cases where eccentrically braced frames are used, the frames comprise the entire lateral force resisting system. In some tall buildings, eccentrically braced frames have been added as stiffening elements to help control drift in moment resisting steel frames.

There are no evaluation statements for eccentrically braced frames because their history is so short, but the engineer is alerted to their possible presence in a building. For guidance in dealing with eccentrically braced frames, the evaluating engineer is referred to the Recommended Lateral Force Requirements and Commentary (SEAOC, 1996). It should be noted that some engineers who were familiar with current research, designed eccentrically braced frames before the SEAOC provisions were developed. These frames may not satisfy all of the detailing requirements present in the current code. Any frame that was clearly designed to function as proper eccentrically braced frame should be recognized and evaluated with due regard for any possible shortcomings that will affect the intended behavior.

Figure 4-26. Eccentrically Braced Frames