In a strong column situation (see upper sketch, Figure C4-12), the beams hinge first, yielding is distributed throughout the structure, and the ductility demand is more dispersed.

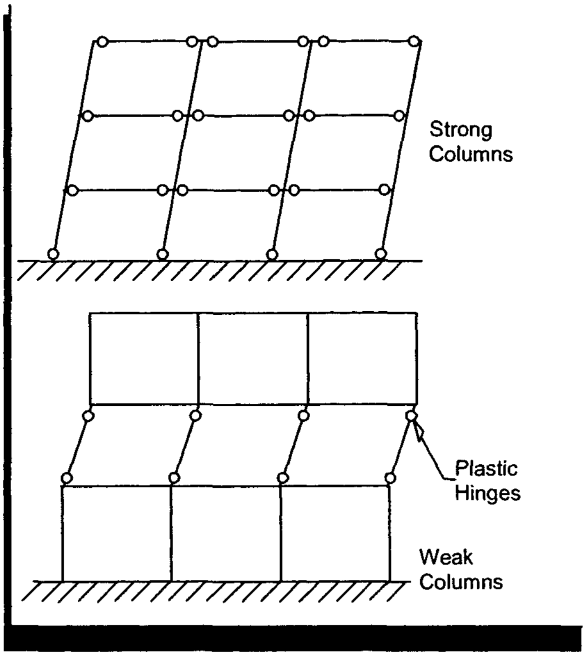

Figure C4-12. Plastic Hinge Formation

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 or equal to 2.0 for Life Safety and for Immediate Occupancy. The number of bays of moment frames in each line shall be greater than or equal to 2.0 for Life Safety and 3.0 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.

C4.4.1.1.1 Redundancy

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 (see Figure C4-13).

A distinction should be made between redundancy and adequacy. For the purpose of this standard, 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 standard to determine the adequacy of the elements provided.

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

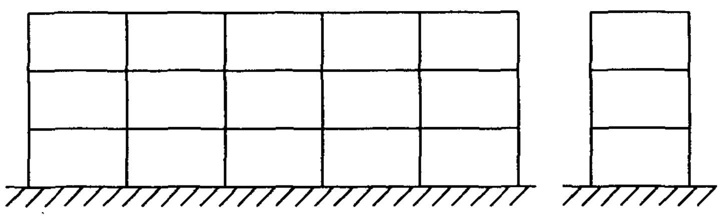

Figure C4-13. Redundancy along a Line of Moment Frame

4.4.1.2 Moment Frames with Infill Walls

C4.4.1.2 Moment Frames with Infill Walls

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 concrete and masonry 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 the frame, shall be calculated. The adequacy of the interfering walls and the frame to resist the induced forces shall be evaluated.

C4.4.1.2.1 Interfering Walls

Where 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. The amount of isolation must be able to accommodate the inter-story drift of the moment frame.

Interfering walls should be checked for forces induced by the frame, particularly where 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 fill the bay.

It should be noted that it is impossible to simultaneously satisfy this section and Section 4.4.2.6.1.

4.4.1.3 Steel Moment Frames

C4.4.1.3 Steel Moment Frames

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.
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, a large number of 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 *Uniform Building Code* (ICBO, 1994a) to remove the prequalification of this type of connection. For a full discussion of these connections, please refer to FEMA 351, *Recommended Seismic Evaluation and Upgrade Criteria for Existing Steel Moment-Frame Buildings* (FEMA, 2000b).

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 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 4-5.

C4.4.1.3.1 Drift Check

Moment-resisting frames are more flexible than shear wall or braced frame structures. This flexibility can lead to large inter-story drifts that may potentially cause extensive structural and nonstructural damage to welded beam-column connections, partitions, and cladding. Drifts also may induce large *P-Δ* demands and pounding where 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 4-5.

C4.4.1.3.2 Axial Stress Check

Columns that carry a substantial amount of gravity load may have limited additional capacity to resist seismic forces. Where 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 identifying frames that are likely to be adequate: frames with high gravity loads, but small seismic overturning forces.

Where 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 4-5.

C4.4.1.3.3 Moment-Resisting Connections

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 (see Figure C4-14). 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, a large number of 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 *Uniform Building Code* removing the prequalification of this type of connection. For a full discussion of these connections, please refer to FEMA 351.

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

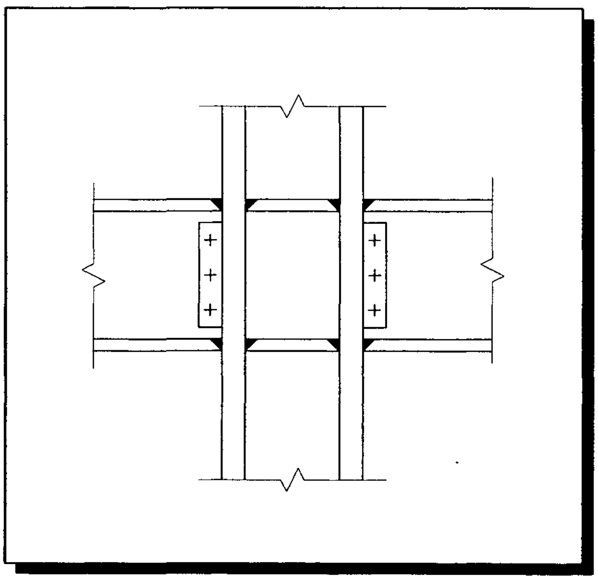

Figure C4-14. Pre–Northridge-Type Connection

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 times the sum of the flexural strengths 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 4-5.

C4.4.1.3.4 Panel Zones

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.

Where 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.

C4.4.1.3.5 Column Splices

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 percentage of strong column/weak beam joints in each story of each line of moment-resisting frames shall be greater than 50 percent 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 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 of the total pseudo lateral force.

C4.4.1.3.6 Strong Column/Weak Beam

Where 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.

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 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 frame elements shall meet section requirements set forth by *Seismic Provisions for Structural Steel Buildings* Table I-9-1 (AISC, 1997).

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 4-5.

C4.4.1.3.7 Compact Members

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 1997 AISC Seismic Provisions explicitly address the section requirements that should be considered.

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 one-fourth 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.

C4.4.1.3.8 Beam Penetrations

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.

C4.4.1.3.9 Girder Flange Continuity Plates

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 percent of the critical column flange compression force acting concurrently at the non-compliant joint.

C4.4.1.3.10 Out-of-Plane Bracing

Columns 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 flanges 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.

C4.4.1.3.11 Bottom Flange Bracing

Beam 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 where seismic loads induce large compression forces in the bottom flange.

4.4.1.4 Concrete Moment Frames

C4.4.1.4 Concrete Moment Frames

Concrete moment frame buildings typically are more flexible than shear wall buildings. This flexibility can result in large inter-story drifts that may lead to extensive nonstructural damage and P-Δ 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 American Concrete Institute (ACI) code maximum spacing for shear reinforcing is much smaller. 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 are expected to 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 the greater of 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 frame elements shall be evaluated using the *m*-factors in Table 4-6.

C4.4.1.4.1 Shear Stress Check

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 compressive 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 compressive 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 4-6.

C4.4.1.4.2 Axial Stress Check

Columns that carry a substantial amount of gravity load may have limited additional capacity to resist seismic forces. Where 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 identifying frames that are likely to be adequate: frames with high gravity loads, but small seismic overturning forces.

Where 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 concrete frame including prestressed elements shall be evaluated using the *m*-factors in Table 4.4.

C4.4.1.4.3 Flat Slab Frames

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-force-resisting frames shall not include any prestressed or post-tensioned elements where the average prestress exceeds the lesser of *700 psi or* $f'_c/6$ at potential hinge locations. The average prestress shall be calculated in accordance with the Quick Check procedure of Section 3.5.3.8.

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 4-6.