7. NEW CONSTRUCTION

The building code provisions for earthquake resistive design of Special Moment-Resisting Frames (SMRFs) assume that these structures are extremely ductile and therefore are capable of large plastic rotations at, or near to, their beam-column connections. Based on limited research, and observations of damage experienced in the Northridge Earthquake, it appears that conventionally designed connection assemblies configured such that plastic deformation concentrates at the beam-column connection are not capable of reliably withstanding large plastic rotation demands. The reliability appears to decrease as the size of the connected members increases. Other factors affecting this reliability appear to include the quality of workmanship, joint detailing, toughness of the base and weld metals, relative strengths of the connection elements, and the combined stresses present on these elements. Unfortunately, the quantitative relationship between these factors and connection reliability is not well defined at this time.

In order to attain frames that can reliably perform in a ductile manner, these Interim Guidelines recommend that SMRF connections be configured with sufficient strength so that plastic hinges occur within the beam span and away from the face of the column. All elements of the frame, and the connection itself, should be designed with adequate strength to develop these plastic hinges. The resulting connection assemblies are somewhat complex and the factors limiting their behavior not always evident. Therefore, qualification of connection designs through prototype testing, or by reference to tests of similar connection configurations is recommended.

These procedures should also be applied to the design of Ordinary Moment-Resisting Frames (OMRFs) located in zones of higher seismicity, or for which highly reliable earthquake performance is desired, unless it can be demonstrated that the connections can resist the actual demands from a design earthquake and remain elastic. Interim Guidelines for determining if a design meets this condition are provided. Light, single-story, frame structures, the design of which is predominated by wind loads, have performed well in past earthquakes and may continue to be designed using conventional approaches, regardless of the seismic zone they are located in.

Materials and workmanship are critical to frame behavior and careful specification and control of these factors is essential. Interim Guidelines for the specification of materials and control of workmanship are provided in this Chapter, as well as in Chapters 8, 9, 10 and 11.

7.1 Scope

This Chapter presents interim design guidelines for new welded steel moment frames (WSMFs) intended to resist seismic demands through inelastic behavior. The criteria apply to all SMRF structures designed for earthquake resistance and those OMRF structures located in *Uniform Building Code* (*UBC*) Seismic Zones 3 and 4 {*National Earthquake Hazards Reduction Program* (*NEHRP*) Map Areas 6 and 7}. Light, single-story buildings, the design of which is governed by wind, need not consider these Interim Guidelines. Frames with bolted connections, either fully restrained (type FR) or partially restrained (type PR), are beyond the scope of this document. However, the acceptance criteria for connections may be applied to type FR bolted connections as well.

Commentary: Observation of damage experienced by WSMF buildings in the Northridge Earthquake and subsequent laboratory testing of large scale beam-column assemblies has demonstrated that the standard details for WSMF connections commonly used in the past are not capable of providing reliable service in the post-elastic range. Therefore, structures which are expected to experience significant post-elastic demands from design earthquakes, or for which highly reliable seismic performance is desired, should be designed using the Interim Guidelines presented herein.

In order to determine if a structure will experience significant inelastic behavior in a design earthquake, it is necessary to perform strength checks of the frame components for the combination of dead and live loads expected to be present, together with the full earthquake load. Except for structures with special performance goals, or structures located within the near field (within 10 kilometers) of known active earthquake faults, the full earthquake load may be taken as the minimum design earthquake load specified in the building code, but calculated using a lateral force reduction coefficient (R_w or R) of unity. If all components of the structure and its connections have adequate strength to resist these loads, or nearly so, then the structure may be considered to be able to resist the design earthquake, elastically.

Design of frames to remain elastic under unreduced (R_w {R} taken as unity) earthquake forces may not be an overly oppressive requirement, particularly in more moderate seismic zones. Most frame designs are currently controlled by drift considerations and have substantially more strength than the minimum specified for design by the building code. As part of the SAC Phase 1 research, a number of modern frame buildings designed with large lateral force reduction coefficients (Rw = 12, {R = 8}) were evaluated for unreduced forces calculated using the standard building code spectra. It was determined that despite the nominally large lateral force reduction coefficients used in the original design, the maximum computed demands from the dynamic analyses were only on the order of 2 to 3 times those which would cause yielding of the real structures (Krawinkler, et. al. - 1995; Uang, et. al. - 1995; Engelhardt, et. al. - 1995, Hart, et. al. - 1995; Kariotis and Eimani - 1995). Therefore, it is not unreasonable to expect that OMRF structures (nominally designed with a lateral force reduction coefficient R_w = 6 {R = 4.5}) could resist the design earthquakes with near elastic behavior. Regardless of these considerations, better seismic performance can be expected by designing structures with greater ductility rather than less and engineers are not encouraged to design structures for elastic behavior using brittle or unreliable details..

For structures designed to meet special performance goals, and buildings located within the near field of major active faults, full earthquake loads calculated in accordance with the above procedure may not be adequate. For such structures, the full earthquake load should be determined using a site specific ground motion characterization and a suitable analysis procedure. Recent research (Heaton, et. al. - 1995) suggests that the elastic response spectrum technique, typically used for determining seismic forces for structural design, may not provide an adequate indication of the true earthquake demands produced by the large impulsive ground motions common in the near field of large earthquake events. Further, this research indicates that frame structures, subjected to such impulsive ground motions can experience very large drifts, and potential collapse. Direct nonlinear time history analysis, using an appropriate ground motion representation would be one method of more accurately determining the demands on structures located in the near field. Additional research on these effects is required.

As an alternative to use of the criteria contained in these Interim Guidelines, OMRF structures in zones of high seismicity (UBC seismic zones 3 and 4 and NEHRP map areas 6 and 7) may be designed for the connections to remain elastic (R_w or R taken as 1.0) while the beams and columns are designed using the standard lateral force reduction coefficients specified by the building code. Although this is an acceptable approach, it may result in much larger connections than would be obtained by following these Interim Guidelines.

The use of partially restrained connections may be an attractive and economical alternative to the design of frames with fully restrained connections. However, the design of frames with partially restrained connections is beyond the scope of this document. The AISC is currently working on development of practical design guidelines for frames with partially restrained connections.

7.2 General - Welded Steel Frame Design Criteria

7.2.1 Criteria

Welded Steel Moment Frame (WSMF) systems should, as a minimum, be designed for the provisions of the prevailing building code and these Interim Guidelines. Special Moment-Resisting Frames (SMRF)s and Ordinary Moment-Resisting Frames (OMRF)s with FR connections, should additionally be designed in accordance with the emergency code change to the *1994 UBC* {*NEHRP-1994*}, restated as follows:

2211.7.1.1. Required Strength {NEHRP-1994 Section 5.2, revision to Ref. 8.2c of Ref. 5.3}

The girder-to-column connections shall be adequate to develop the lesser of the following:

1. The strength of the girder in flexure.
2. The moment corresponding to development of the panel zone shear strength as determined by Formula (11-1).

2211.7.1.3-2 Connection Strength

Connection configurations utilizing welds and high strength bolts shall demonstrate, by approved cyclic test results or calculation, the ability to sustain inelastic rotations and to develop the strength criteria in Section 2211.7.1.1 considering the effects of steel overstrength and strain hardening.

*Commentary: At this time, no recommendations are made to change the minimum lateral forces, drift limitations or strength calculations which determine member sizing and overall performance of moment frame systems, except as recommended in Sections 7.2.2, 7.2.3 and 7.2.4. The design of joints and connections is discussed in Section 7.3. The UBC permits OMRF structures with FR connections, designed for 3/8*R_w *times the earthquake forces otherwise required, to be designed without conforming to Section 2211.7.1. However, this is not recommended.*

7.2.2 Strength

When these Interim Guidelines require determination of the strength of a framing element or component, this shall be calculated in accordance with the criteria contained in *UBC-94*, Section 2211.4.2 {*NEHRP-91* Section 10.2, except that the factor ϕ should be taken as 1.0}, restated as follows:

2211.4.1 Member strength. Where this section requires that the strength of the member be developed, the following shall be used:

Flexure	$M_s = Z\,F_y$
Shear	$V_s = 0.55\,F_y\,d\,t$
Axial compression	$P_{sc} = 1.7\,F_a\,A$
Axial tension	$P_{st} = F_y\,A$
Connectors	
Full Penetration welds	$F_y\,A$
Partial Penetration welds	1.7 allowable (see commentary)
Bolts and fillet welds	1.7 allowable

Commentary: Partial penetration welds are not recommended for tension applications in critical connections resisting seismic induced stresses. The geometry of partial penetration welds creates a notch-like condition that can initiate brittle fracture under conditions of high tensile strain.

7.2.3 Configuration

Frames should be proportioned so that the required plastic deformation of the frame may be accommodated through the development of plastic hinges at pre-determined locations within the girder spans, as indicated in Figure 7-1. Beam-column connections should be designed with sufficient strength (through the use of cover plates, haunches, side plates, etc.) to force development of the plastic hinge away from the column face. This condition may also be attained through local weakening of the beam section at the desired location for plastic hinge formation.

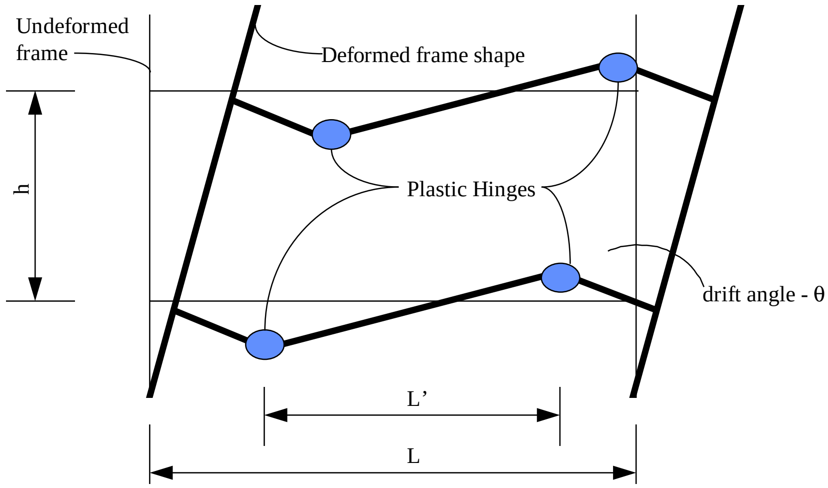


Figure 7-1 - Desired Plastic Frame Behavior

Commentary: Nonlinear deformation of frame structures is accommodated through the development of inelastic flexural or shear strains within discrete regions of the structure. At large inelastic strains these regions can develop into plastic hinges, which can accommodate significant concentrated rotations at constant (or nearly constant) load through yielding at tensile fibers and buckling at compressive fibers. If a sufficient number of plastic hinges develop in a frame, a mechanism is formed and the frame can deform laterally in a plastic manner. This behavior is accompanied by significant energy dissipation, particularly if a number of members are involved in the plastic behavior, as well as substantial local damage to the highly strained elements. The formation of hinges in columns, as opposed to beams, is undesirable, as this results in the formation of mechanisms with relatively few elements participating, so called "story mechanisms" and consequently little energy dissipation occurring. In addition, such mechanisms also result in local damage to critical gravity load bearing elements.

The prescriptive connection contained in the UBC and NEHRP Recommended Provisions prior to the Northridge Earthquake was based on the development of plastic hinges within the beams at the face of the column, or within the column panel zone itself. If the plastic hinge develops in the column panel zone, the resulting column deformation results in very large secondary stresses on the beam flange to column flange joint, a condition which can contribute to brittle failure. If the plastic hinge forms in the beam, at the face of the column, this can result in very large through-thickness strain demands on the column flange material and large inelastic strain demands on the weld metal and surrounding heat affected zones. These conditions can also lead to brittle joint failure. In order to achieve more reliable performance, it is recommended that the connection of the beam to the column be configured to force the inelastic action (plastic hinge) away from the column face. This can be done either by local reinforcement of the *connection, or locally reducing the cross section of the beam, at a distance away from the connection. Plastic hinges in steel beams have finite length, typically on the order of half the beam depth. Therefore, the location for the plastic hinge should be shifted at least that distance away from the face of the column. When this is done through reinforcement of the connection, the flexural demands on the columns, for a given beam size, are increased. Care must be taken to assure that weak column conditions are not inadvertently created by local strengthening of the connections.*

It should be noted that some professionals and researchers believe that configurations which permit plastic hinging to occur adjacent to the column face may still provide reliable service under some conditions. These conditions may include limitations on the size of the connected sections, the use of base and weld metals with adequate notch toughness, joint detailing that minimizes notch effects, and appropriate control of the relative strength of the beam and column materials. Sufficient research has not been performed to date either to confirm these suggestions or define the conditions in which they are valid. Research however does indicate that reliable performance can be attained if the plastic hinge is shifted away from the column face, as suggested above. Consequently, these Interim Guidelines make a general recommendation that this approach be taken. Additional research should be performed to determine the acceptability of other approaches.

It should also be noted that reinforced connection (or reduced beam section) configurations of the type described above, while believed to be effective in preventing brittle connection fractures, will not prevent structural damage from occurring. Brittle connection fractures are undesirable because they result in a substantial reduction in the lateral-force-resisting strength of the structure which, in extreme cases, can result in instability and collapse. Connections configured as described in these Interim Guidelines should experience many fewer such brittle fractures than unmodified connections. However, the formation of a plastic hinge within the span of a beam is not a completely benign event. Beams which have formed such hinges may exhibit large buckling and yielding deformation, damage which typically must be repaired. The cost of such repairs could be comparable to the costs incurred in repairing fracture damage experienced in the Northridge Earthquake. The primary difference is that life safety protection will be significantly enhanced and most structures that have experienced such plastic deformation damage should continue to be safe for occupancy, while repairs are made.

If the types of damage described above are unacceptable for a given building, then alternative structural systems should be considered, that will reduce the plastic deformation demands on the structure during a strong earthquake. Appropriate methods of achieving such goals include the installation of supplemental braced frames, energy dissipation systems, base isolation systems *and similar structural systems. Framing systems incorporating partially restrained connections may also be quite effective in resisting large earthquake induced deformation with limited damage.*

7.2.4 Plastic Rotation Capacity

The plastic rotation capacity of connection assemblies should reflect realistic estimates of the total (elastic and plastic) drift likely to be induced in the frame by earthquake ground shaking, and the geometric configuration of the frame. For frames of typical configuration, and for ground shaking of the levels anticipated by the building code, a minimum plastic rotation capacity of 0.03 radian is recommended.

When the configuration of a frame is such that the ratio L/L' is greater than 1.25, the plastic rotation demand should be taken as follows:

$$q = 0.025\left(1 + (\mathrm{L} - \mathrm{L'})/\mathrm{L'}\right) \qquad (7\text{-}1)$$

where: L is the center to center spacing of columns, and
L' is the center to center spacing of plastic hinges in the bay under consideration

The indicated rotation demands may be reduced when positive means, such as the use of base isolation or energy dissipation devices, are introduced into the design, to control the building's response. When such measurers are taken, nonlinear dynamic analyses should be performed and the connection demands taken as 0.005 radians greater than the rotations calculated in the analyses. The nonlinear analyses should conform to the criteria specified in *UBC-94* Section 1655 {*NEHRP-94* Section 2.6.4.2} for nonlinear dynamic analysis of base isolated structures. Ground motion time histories utilized for these nonlinear analyses should satisfy the scaling requirements of *UBC-94* Section 1655.4.2 {NEHRP-94 Section 2.6.4.4}, except that if the building is not base isolated, the structure period T, calculated in accordance with UBC-94 Section 1628 {*NEHRP-94* Section 2.3.3.1} should be substituted for T_I.

Commentary: Traditionally, engineers have calculated demand in moment frames by sizing the members for strength and drift using code forces (either equivalent static or reduced dynamic forces) and then "developing the strength of the members." Since 1988, "developing the strength" has been accomplished by prescriptive means based on a review of testing of moment frame connections to that date. It was assumed that the prescribed connections would be strong enough that the beam or girder would yield (in bending), or the panel zone would yield (in shear) in a nearly perfectly plastic manner producing the plastic rotations necessary to dissipate the energy of the earthquake.

A realistic estimate of the interstory drift demand for most structures and most earthquakes is on the order of 0.015 to 0.025 times the story height for WSMF structures designed to code allowable drift limits. In such frames, a portion of the drift will be due to elastic deformations of the frame, while the balance must *be provided by inelastic rotations of the beam plastic hinges, by yielding of the column panel zone, or by a combination of the two.*

In the 1994 Northridge Earthquake, many moment-frame connections fractured with little evidence of plastic hinging of the beams or yielding of the column panel zones. Testing of moment frame connections both prior to and subsequent to the earthquake suggests that the standard, pre-Northridge, welded flange-bolted web connection is unable to reliably provide plastic rotations beyond about 0.005 radian for all ranges of beam depths and often fails below that level. Since the elastic contribution to drift may approach 0.01 radian, the necessary inelastic contributions will exceed the capability of the standard connection in many cases. For frames designed for code forces and for the code drift, the necessary plastic rotational demand may be expected to be on the order of 0.02 radian or more and new connection configurations should be developed to accommodate such rotation without brittle fracture.

The recommended connection demand of 0.03 radians was selected both to provide a comfortable margin against the demands actually expected in most cases and because in recent testing of connection assemblies, specimens capable of achieving this demand behaved in a ductile manner through the formation of plastic hinges.

For a given building design, and known earthquake hazard, it is possible to more accurately estimate plastic rotation demands on frame connections. This requires the use of nonlinear analysis techniques. Analysis software, capable of performing such analyses is becoming more available and many design offices will have the ability to perform such analyses and develop more accurate estimates of inelastic demands for specific building designs. However, when performing such analyses, care should be taken to evaluate building response for multiple earthquake time histories, representative of realistic ground motions for sites having similar geologic characteristics and proximity to faults, as the actual building site. Relatively minor differences in the ground motion time history used as input in such an analysis can significantly alter the results. Since there is significant uncertainty involved in any ground motion estimate, it is recommended that analysis not be used to justify the design of structures with non-ductile connections, unless positive measures such as the use of base isolation or energy dissipation devices are taken, to provide reliable behavior of the structure.

It has been pointed out that it is not only the total plastic rotation demand that is important to connection and frame performance, but also the connection mechanism (for example - panel zone yielding, girder flange yielding/buckling, etc.) and hysteretic loading history. These are matters for further study in the continuing research on connection and joint performance.

7.2.5 Redundancy

The frame system should be designed and arranged to incorporate as many moment-resisting connections as is reasonable into the moment frame.

Commentary: Early moment frame designs were highly redundant and nearly every column was designed to participate in the lateral-force-resisting system. In an attempt to produce economical designs, recent practice often produced designs which utilized only a few large columns and beams in a small proportion of the building's frames for lateral resistance, with the balance of the building columns designed not to participate in lateral resistance. This practice led to the need for large welds at the connections and to reliance on only a few connections for the lateral stability of the building. The resulting large framing elements and connections are believed to have exacerbated the poor performance of the pre-Northridge connection. Further, if only a few framing elements are available to resist lateral demands, then failure of only a few connections has the potential to result in a significant loss of earthquake resisting strength. Together, these effects are not beneficial to building performance.

The importance of redundancy to building performance can not be over-emphasized. Even connections designed and constructed according to the improved procedures recommended by these Interim Guidelines will have some potential, albeit greatly reduced, for brittle failures. As the number of individual beams and columns incorporated into the lateral-force-resisting system is increased, the consequences of isolated connection failures significantly reduces. Further, as more framing elements are activated in the building's response to earthquake ground motion, the building develops greater potential for energy absorption and dissipation, and ability to control earthquake induced deformations to acceptable levels.

Incorporation of more of the building framing into the lateral-force-resisting system will lead to smaller members and therefore an anticipated increase in the reliability of individual connections. It will almost certainly lead to improved overall system reliability. Further, recent studies conducted by designers indicate that under some conditions, redundant framing systems can be constructed as economically as non-redundant systems. In these studies, the additional costs incurred in making a greater number of field-welded moment-resisting connections in the more redundant frame were balanced by a reduced total tonnage of steel in the lateral-force-resisting systems and sometimes, reduced foundation costs as well.

The 1994 UBC requirements limit the relative number of weak column/strong beam connections in the moment frame system. There is a divergence of opinion among structural engineers on the desirability of frames in which all beam-column connections are made moment-resisting, including those of beams *framing to the minor axis of columns. Use of such systems as a means of satisfying these Interim Guidelines requires careful consideration by the structural engineer. Limited testing in the past has indicated that moment connections made to the minor axis of wide flange columns are subject to the same types of fracture damage experienced by major axis connections. As of this time, there has not been sufficient research to suggest methods of making reliable connections to the column minor axis.*

7.2.6 System Performance

WSMF design should consider all effects of connection modifications on the response and performance of the frame.

Commentary: Methods developed thus far for improving performance of beam/column connections involve shifting of the hinge point away from the column face either by reinforcement of the connection (e.g. haunches, cover plates, etc.), or reducing the relative strength of the beam locally. These modifications affect the overall stiffness of the frame and, therefore, its seismic response. In fact, it can be shown that the use of smaller beam sizes and haunched connections will result in the same overall frame stiffness as the use of larger beams and unstiffened connections. Additionally, haunching or reinforcement results in magnified moments and shears at the column face which should be included in the strong column/weak beam calculations, panel zone and web connection calculations, and column axial demand calculations. Unsymmetrical haunches, placed on only the bottom (or top) of the beam can also change the relative stiffness of columns above and below the beam resulting in unexpected formation of plastic hinges in one of the columns. In addition, if plastic hinges are forced out into the beam span, away from the column face, the local lateral stability of beams at plastic hinges away from the column should be considered.

7.2.7 Special Systems

When WSMFs are used as components of "Tube" type buildings with beams yielding in shear rather than bending, or in Dual System structures, appropriate consideration should be given to the differences in plastic rotation demands expected (as compared to pure moment frame designs) when applying these provisions. (See discussion in Section 7.10.)

Commentary: Moment frames which are employed in dual systems in low-rise buildings or in the lower levels of taller buildings may have significantly lower rotation demands than those in pure frame buildings. Engineers may consider it appropriate to use less conservative connection designs or qualification requirements for such frames, or for portions of such frames. Appropriate analytical substantiation should be provided for any alternative criteria utilized.

For tube frames with shear-yielding beams, qualification by testing is recommended, but designs and requirements may differ from those presented in these Interim Guidelines. Again appropriate analytical substantiation should be provided for the selected criteria.

7.3 Connection Design & Qualification Procedures - General

7.3.1 Connection Performance Intent

The intent of connection design should be to force the plastic hinge away from the face of the column to a pre-determined location within the beam span. This may be accomplished by local reinforcement of the connection itself (cover plates, haunches, side plates, etc.) or by local reductions of the beam section (drilled holes, trimmed flanges, etc.). All elements of the connection should have adequate strength to develop the forces resulting from the formation of the plastic hinge at the predetermined location, together with forces resulting from gravity loads.

7.3.2 Qualification by Testing

Connection strength and plastic rotation capacity should be demonstrated by approved cyclic testing as described in Section 7.4, except as indicated in paragraph 7.3.3. It is recommended that preliminary design of specimens to be tested be developed using the Interim Guidelines of Section 7.5. Extrapolation and interpolation of test results using the calculation procedures of Section 7.5 is acceptable for connections of elements having similar geometries and material specifications as tested connections.

Commentary: Cyclic testing of connections matching the essential features of those to be used in the actual design is the most reliable method of assuring that the expected connection performance can be attained. Section 7.4 describes testing guidelines in detail. Guidelines for extrapolation by calculation are given in Section 7.5.

7.3.3 Design by Calculation

Connection design by calculations alone may be acceptable under the following conditions:

a) Calculations are based on comparison with previously tested assemblies, or with prototype connections tested for the project;

b) Conditions of the calculated detail, including member property relationships, material properties, welding materials, processes and procedures, and construction sequence, mirror those of the tested detail as closely as possible; or

c) Qualified third party review, in accordance with Section 4.5 is performed.

Commentary: Use of calculations based on engineering principles alone, or to extrapolate data from tests performed on assemblies which do not precisely mirror the conditions of the calculated assembly, requires caution and judgment. Subjective factors affecting the acceptability of such an approach should include:

a) The importance of the structure: Greater caution in applying a calculation-only approach should be exercised for more important facilities, particularly when the facility is expected to remain functional after a major earthquake.

b) Confidence in the lateral forces used in design: Projects which carefully apply seismic forces based on extensively researched, site-specific seismic hazard studies, and having resulting designs with low calculated rotation demands, may warrant more confidence in the application of connection designs using calculations only, as opposed to those that do not use this type of information. Most structures are designed to satisfy the minimum code seismic forces. Structures that are designed assuming higher levels of seismic demand (both strength and stiffness) than found in typical projects, could also possibly be demonstrated to warrant greater latitude in applying a calculation-only approach.

c) The degree of redundancy, regularity and potential over-strength in the structure: Greater care in applying a calculation-only approach should be considered in structures with a limited number of lateral-force-resisting elements in each direction or those with unusual building geometries. Structures with a high degree of redundancy may be demonstrated to be better able to tolerate limited instances of marginal connection performance. Frames designed to limit the rotational demand by relying on elastic or near-elastic behavior may also be more amenable to a calculation-only approach than those that depend on high levels of plastic rotation to dissipate anticipated seismic demands. However, it has not been shown that superior seismic performance results when strength is substituted for ductility, and overly strong, frames with non ductile connections are not the intent of these guidelines.

d) Proximity to active faults: Ground motion records from recent earthquakes clearly demonstrate that sites located close to a fault rupture experience substantially more severe ground motion than is explicitly provided for in current code design provisions. When a building is located within 5 km of an active fault, the plastic rotation demands on connections may exceed those provided for in these Guidelines, and additional caution in design procedures is warranted.

For structures that are essential, contain hazardous materials, are designed with a low degree of conservatism or redundancy, connections qualification by test (either through reference to tests from other projects or project-specific *testing of connections) is strongly recommended. This recommendation should be considered until such time as SAC or other research develops sufficient data to allow formulation of analytical design guidelines for general application.*

For non-essential structures designed with a reasonable degree of redundancy or overstrength and incorporating enhanced welding requirements and quality control, calculations as described above, using proportioning and stress levels compatible with previously completed test programs, may provide sufficient assurance of reliability.

7.4 Guidelines for Connection Qualification by Testing

7.4.1 Testing Protocol

Unless future testing programs reveal significant effects of dynamic loading rate or time history loading, and unless the effects of other factors (e.g., restraint conditions and composite slab effects) are found to be compelling, a testing protocol similar to ATC-24, *Guidelines for Cyclic Seismic Testing of Components of Steel Structures* (Applied Technology Council - 1992), is recommended as the basis for qualification tests.

The testing program should replicate as closely as practical the anticipated conditions in the field, including such factors as:

a) Member sizes.

b) Material specifications.

c) Welding process, details and construction conditions.

d) Cover plates, continuity plates, web tabs, bolts, and doubler plates.

e) Connection configuration (e.g., beams on both sides).

f) Induced stresses because of restraint conditions on the welds and connection members.

g) Axial load, where pertinent.

h) Gravity load, where significant.

The testing program should be organized to provide as much information as possible about the capability of the connections selected. The following program is recommended:

a) Test at least two full size specimens representative of the larger beam/column assemblies in the project.

b) Test one additional full size specimen representative of other beam/column assemblies with significantly different interaction properties, such as beam b/t, panel zone stress/distortion, etc.

If any of the specimens fails to meet the qualification criteria, the connection should be redesigned and retested.

Where two-sided connections are used in the structure, and the type of connection being used can be expected to perform differently in a two-sided use than in one-sided use, it should be tested in the two-sided configuration as well as the one-sided. Two-sided connection assemblies can be expected to behave differently than one-sided assemblies, for example, when panel zone distortions will be significantly different, or when systems involve transfer of stress to the column by plates, welds, or other elements which are connected to the beams on both sides of the column.

The inclusion of axial load should be considered when analysis indicates that significant tension can be expected to occur in a significant number of the columns represented by the specimen and where the connection type relies on the through-thickness strength of the column flanges. If the presence of a floor slab is anticipated to have significant influence on either the location or mechanism of the plastic hinge formed, than this should also be included in the test specimen.

7.4.2 Acceptance Criteria

The minimum acceptance criteria for connection qualification for specimens tested in accordance with these Interim Guidelines should be as follows:

a) The connection should develop beam plastic rotations as indicated in Section 7.2.4, for at least one complete cycle.

b) The connection should develop a minimum strength equal to the plastic strength of the girder, calculated using minimum specified yield strength F_y, throughout the loading history required to achieve the required plastic rotation capacity, as indicated in a), above. If the load limiting mechanism in the test is buckling of the girder flanges, the engineer, upon consideration of the effect of strength degradation on the structure, may consider a minimum of 80% of the nominal strength as acceptable.

Commentary: While the testing of all connection geometries and member combinations in any given building might be desirable, it would not be very practical nor necessary. Test specimens should replicate, within the limitations associated with test specimen simplification, the fabrication and welding procedures, connection geometry and member size, and potential modes of failure. If the testing is done in a manner consistent with other testing programs, reasonable comparisons can be made. On the other hand, testing is expensive and it is difficult to realistically test the beam-column connection using actual boundary conditions and earthquake loading histories and rates.

It was suggested in Interim Recommendation No. 2 by the SEAOC Seismology Committee that three tested specimens be the minimum for qualification of a connection. Further consideration has led to the recognition that while three tests may be desirable, the actual testing program selected should consider the conditions of the project. Since the purpose of the testing program is to "qualify the connection", and since it is not practical for a given project to do enough tests to be statistically meaningful considering random factors such as material, welder skills, and other variables, arguments can be made for fewer tests of identical specimens, and concentration on testing specimens which represent the range of different properties which may occur in the project. Once a connection is qualified, that is, once it has been confirmed that the connection can work, monitoring of actual materials and quality control to assure emulation of the tested design becomes most important.

Because of the cost of testing, use of calculations for interpolation or extrapolation of test results is desirable. How much extrapolation should be accepted is a difficult decision. As additional testing is done, more information may be available on what constitutes "conservative" testing conditions, thereby allowing easier decisions relative to extrapolating tests to actual conditions which are likely to be less demanding than the tests. For example, it is hypothesized that connections of shallower, thinner flanged members are likely to be more reliable than similar connections consisting of deeper, thicker flanged members. Thus, it may be possible to test the largest assemblages of similar details and extrapolate to the smaller member sizes - at least within comparable member group families. Extrapolation or interpolation of results with differences in welding procedures, details or material properties is more difficult.

7.5 Guidelines for Connection Design by Calculation

In conditions where it has been determined that design of connections by calculation is sufficient, or when calculations are used for interpolation or extrapolation, the following guidelines should be used.

7.5.1 Material Strength Properties

In the absence of project specific material property information, the values listed in Table 7-1 should be used to determine the strength of steel shape and plate for purposes of calculation. The permissible strength for weld metal should be taken in accordance with the building code. Additional information on material properties may be found in the Interim Guidelines of Chapter 8.

Table 7-1 - Properties for Use in Connection Design

Material	F_y (ksi)	$F_{y\,m}$ (ksi)	F_u (ksi)
A36	36	use values for Dual Certified	58
Dual Certified Beam			
Axial, Flexural[3]	50		65 min.
Shape Group 1		55[1]	
Shape Group 2		58[1]	
Shape Group 3		57[1]	
Shape Group 4		54[1]	
Through-Thickness	-	-	Note 2
A572 Column/Beam			
Axial, Flexural[3]	50		65 min.
Shape Group 1		58[1]	
Shape Group 2		58[1]	
Shape Group 3		57[1]	
Shape Group 4		57[1]	
Shape Group 5		55[1]	
Through-Thickness	-	-	Note 2’
A913-50			
Axial, Flexural	50	58[1]	65 min.
Through-thickness	-	-	Note 2’
A913--65			
Axial, Flexural	65	75[1]	80 min.

Notes:
1. Based on coupons from web. For thick flanges, the $F_{y\,flange}$ is approximately 0.95 $F_{y\,web}$.
2. See Commentary
3. Values based on (SSPC-1994)

Commentary: The causes for through-thickness failures of column flanges (types C2, C4, and C5), observed both in buildings damaged by the Northridge Earthquake and in some test specimens, are not well understood. They are thought to be a function of the metallurgy and "purity" of the steel; conditions of loading including the presence of axial load and rate of loading application; conditions of tri-axial restraint; conditions of local hardening and embrittlement within the weld's heat affected zone; and by the relationship of the connection components as they may affect flange bending stresses and flange curvature induced by panel zone yielding. Given the many complex factors which can affect the through-thickness strength of the column flange, determination of a reliable basis upon which to set permissible design stresses will require significant research.

Interim Recommendation No. 2 (SEAOC-1995) included a value of 40 ksi, applied to the projected area of beam flange attachment, for the through-thickness strength to be used in calculations. This value was selected because it was consistent with the successful tests of assemblies with cover plates conducted *at the University of Texas at Austin (Engelhardt and Sabol - 1994). However, because of the probable influence of all the factors noted above, this value can only be considered to reflect the specific conditions of those tests and specimens.*

Although reduced stresses at the column face produced acceptable results in the University of Texas tests, the key to that success was more likely the result of forcing the plastic hinge away from the column than reduction of the through-thickness stress by the cover plates. Reduction of through-thickness column flange stress to ever lower levels by the use of thicker cover plates is not recommended, since such cover plates will result in ever higher forces on the face of the column flange.

Notwithstanding all of the above, successful tests using cover plates and other measures of moving hinges (and coincidentally reducing through-thickness stress) continue to be performed. In the interim, engineers choosing to utilize connections relying on through-thickness strength should recognize that despite the successful testing, connections relying on through-thickness strength can not be considered to be fully reliable until the influence of the other parameters discussed above can be fully understood. A high amount of structural redundancy is recommended for frames employing connections which rely on through-thickness strength of the column flange.

7.5.2 Design Procedure

Select a connection configuration, such as one of those indicated in Section 7.9, that will permit the formation of a plastic hinge within the beam span, away from the face of the column, when the frame is subjected to gravity and lateral loads. The following procedure should be followed to size the various elements of the connection assembly:

7.5.2.1 Determine Plastic Hinge Locations

For beams with gravity loads representing a small portion of the total flexural demand, the plastic hinge may be assumed to occur at a distance equal to 1/3 of the beam depth from the edge of the reinforced connection (or start of the reduced beam section), unless specific test data for the connection indicates that a different value is appropriate. Refer to Figure 7-2.

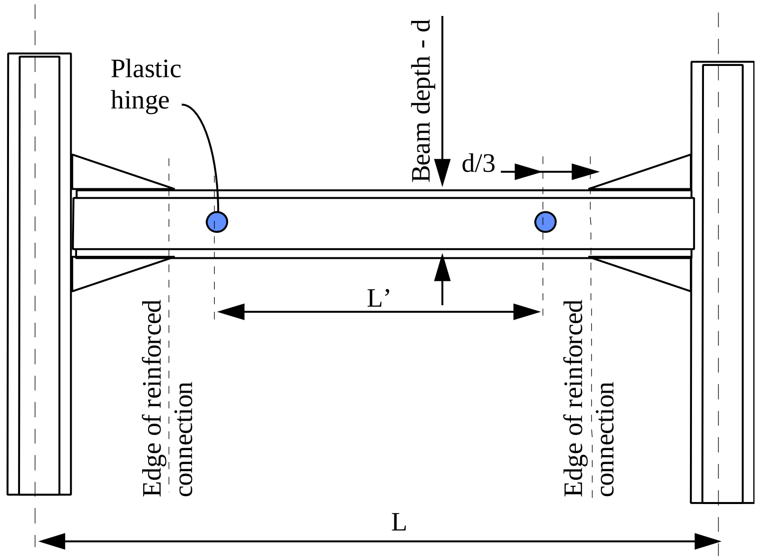


Figure 7-2 - Location of Plastic Hinge

Commentary: The suggested location for the plastic hinge, at a distance d/3 away from the end of the reinforced section (or beginning of reduced section) is based on the observed behavior of test specimens, with no significant gravity load present. If significant gravity load is present, this can shift the locations of the plastic hinges, and in the extreme case, even change the form of the collapse mechanism. If flexural demand on the girder due to gravity load is less than about 30% of the girder plastic capacity, this effect can safely be neglected, and the plastic hinge locations taken as indicated. If gravity demands significantly exceed this level then plastic analysis of the girder should be performed to determine the appropriate hinge locations. In zones of high seismicity (UBC Zones 3 and 4, and NEHRP Map Areas 6 and 7) gravity loading on the girders of earthquake resisting frames typically has a very small effect.

7.5.2.2 Determine Probable Plastic Moment at Hinges

Determine the probable value of the plastic moment, M_{pr}, at the location of the plastic hinges as:

$$M_{pr} = bM_p = bZ_bF_y \qquad (7\text{-}2)$$

where: ß is a coefficient that adjusts the nominal plastic moment to the estimated hinge moment based on the mean yield stress of the beam material and the estimated strain hardening. When designs are based upon calculations alone, an additional factor is recommended to account for uncertainty. In the absence of adequate testing of the type described above, ß should be taken as 1.4 for ASTM A572 and for A913, Grades 50 and 65 steels. Where adequate testing has been performed ß should be permitted to be taken as 1.2 for these materials.

Z_b is the plastic modulus of the section

Commentary: In order to compute b, the expected yield strength, strain hardening and an appropriate uncertainty factor need to be determined. The following assumed strengths are recommended:

<u>Expected Yield</u>: *The expected yield strength, for purposes of computing (M_{pr}) may be taken as:*

$$F_{ye} = 0.95\ F_{ym} \quad (7\text{-}3)$$

The 0.95 factor is used to adjust the yield stress in the beam web, where coupons for mill certification tests are normally extracted, to the value in the beam flange. Beam flanges, being comprised of thicker material, typically have somewhat lower yield strengths than do beam web material.

F_{ym} for various steels are as shown in Table 7-1, based on a survey of web coupon tensile tests (Steel Shape Producers Council - 1994). The engineer is cautioned that there is no upper limit on the yield point for ASTM A36 steel and consequently, dual-certification steel having properties consistent with ASTM A572, Grade 50 is routinely supplied when ASTM A36 is specified. Consequently, it is the recommendation here that the design of connections be based on an assumption of Grade 50 properties, even when A36 steel is specified for beams. It should be noted that at least one producer offers A36 steel with a maximum yield point of 50 ksi in shape sizes ranging up to W 24x62.

<u>Strain Hardening</u>: *A factor of 1.1 is recommended for use with the mean yield stress in the foregoing table when calculating the probable plastic moment capacity M_{pr}.. The 1.1 factor for strain hardening, or other sources of strength above yield, agrees fairly well with available test results. The 1.1 factor could underestimate the over-strength where significant flange buckling does not act as a gradual limit on the beam strength. Nevertheless, the 1.1 factor seems a reasonable expectation of over-strength considering the complexities involved.*

<u>Modeling Uncertainty</u>: *Where a design is based on approved cyclic testing, the modeling uncertainty may be taken as 1.0, otherwise the recommended value is 1.2.*

In summary, for Grade 50 steel, we have:

b = [0.95 (54 ksi to 58 ksi)/50 ksi] (1.1) 1.2) = 1.35 t0 1.45, say 1.4

7.5.2.3 Determine Shear at the Plastic Hinge

The shear at the plastic hinge should be determined by statics, considering gravity loads acting on the beam. A free body diagram of that portion of the beam between plastic hinges, is a useful tool for obtaining the shear at each plastic hinge. Figure 7-3 provides an example of such a calculation. For the purposes of such calculations, gravity load should be based on the load combinations required by the building code in use.

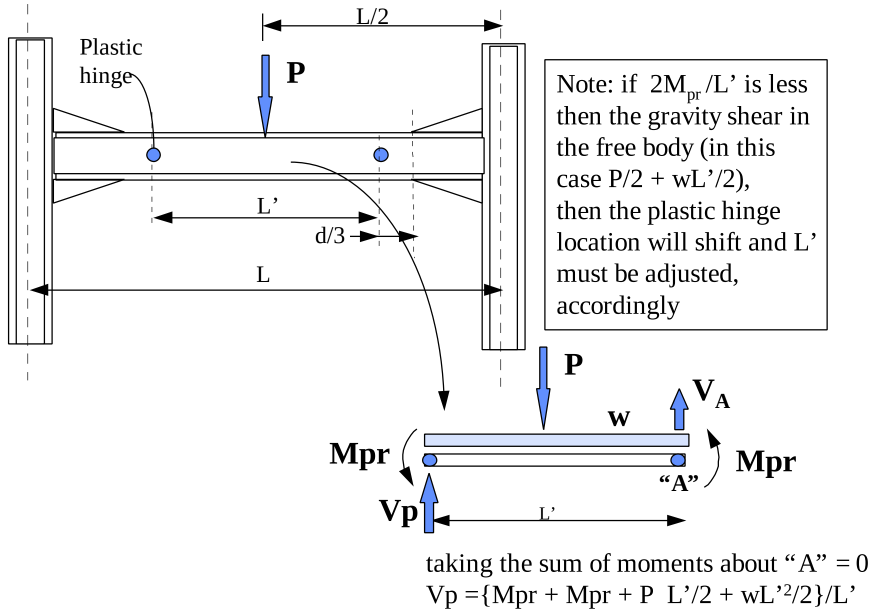


Figure 7-3 - Sample Calculation of Shear at Plastic Hinge

Commentary: The UBC gives no specific guidance on the load combinations to use with strength level calculations while the NEHRP Recommended Provisions do specify specific load factors for the various dead, live and earthquake components of load. For designs performed in accordance with the UBC it is customary to use unfactored gravity loads when checking the strength of elements.

7.5.2.4 Determine Strength Demands at Each Critical Section

In order to complete the design of the connection, including sizing the various plates and joining welds which make up the connection, it is necessary to determine the shear and flexural strength demands at each critical section. These demands may be calculated by taking a free body of that portion of the connection assembly located between the critical section and the plastic hinge. Figure 7-4 demonstrates this procedure for two critical sections, for the beam shown in Figure 7-3.

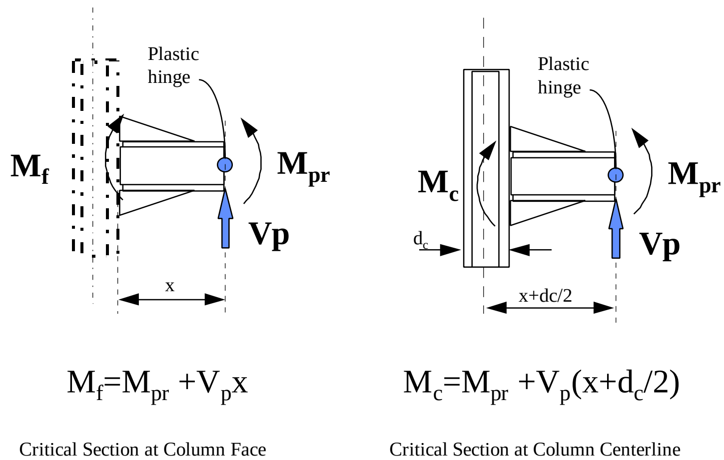


Figure 7-4 - Calculation of Demands at Critical Sections

Commentary: Each unique connection configuration may have different critical sections. The vertical plane that passes through the joint between the beam flanges and column (if such joining occurs) will typically define at least one such critical section, used for designing the joint of the beam flanges to the column, as well as evaluating shear demands on the column panel zone. A second critical section occurs at the center line of the column. Moments calculated at this point are used to check strong column - weak beam conditions. Other critical sections should be selected as appropriate.

7.5.2.5 Check for Strong Column - Weak Beam Condition

When required by the building code, the connection assembly should be checked to determine if strong column - weak beam conditions are satisfied. In lieu of *UBC-94* equation 11-3.1 {NEHRP-91 equation 10-3}, the following equation should be used:

$$\sum Z_c(F_{yc} - f_a) \Big/ \sum M_c > 1.0 \qquad (7\text{-}4)$$

where: Z_c is the plastic modulus of the column section above and below the connection
F_{yc} is the minimum specified yield stress for the column above and below
f_a is the axial load in the column above and below
M_c is the moment calculated at the center of the column in accordance with Section 7.5.2.4

Commentary: The building code provisions for evaluating strong column - weak beam conditions presume that the flexural stiffness of the columns above and below the beam are approximately equal. If non-symmetrical connection configurations are used, such as a haunch on the bottom side of the beam, this can result in an uneven distribution of stiffness between the two column segments.

7.5.2.6 Check Column Panel Zone

The adequacy of the shear strength of the column panel zone should be checked. For this purpose, the term $0.8\Sigma M_f$ should be substituted for the term $0.8\Sigma Ms$ in *UBC-94* Section 2211.7.2.1 {$0.9\Sigma\phi_b M_p$ in *NEHRP-91* Section 10.10.3.1}, repeated below for convenience of reference. M_f is the calculated moment at the face of the column, when the beam mechanism forms, calculated as indicated in Section 7.5.2.4 above.

> **2211.7.2.1 Strength.** The panel zone of the joint shall be capable of resisting the shear induced by beam bending moments due to gravity loads plus 1.85 times the prescribed seismic forces, but the shear strength need not exceed that required to develop $0.8\Sigma M_s$ of the girders framing into the column flanges at the joint. The joint panel zone shear strength may be obtained from the following formula:

$$V = 0.55F_y d_c t \left[1 + \frac{3b_c t_{cf}^{\;2}}{d_b d_c t} \right] \qquad (11\text{-}1)$$

where: b_c = width of column flange
d_b = the depth of the beam (including any haunches or cover plates)
d_c = the depth of the column
t = the total thickness of the panel zone including doubler plates
t_{cf} = the thickness of the column flange

7.6 Metallurgy and Welding

For Guidelines on Metallurgy and Welding for New Structures, see Chapter 8 of these Interim Guidelines. The recommendation for welding electrodes capable of depositing weld metal with specified notch toughness, as described therein, should apply to the critical beam flange to column flange field welded joints. It need not apply to shop welds of continuity plates, etc.

Commentary: This is an area of continuing controversy in the community, requiring additional research for resolution. Some professionals and researchers knowledgeable in fracture mechanics believe it is essential that all weld metal in the beam column connection, including both field and shop welds, welds of continuity plates, doubler plates, etc., as well as the welds of beam to column flanges, should have minimum specified notch toughness. Some of these same professionals believe that the notch toughness requirement should apply to the combined metal, consisting of deposited electrode metal and fused base metal. The current recommendations, which are less restrictive than this position, are based on recommendations of members of the AWS D1.1 committee. These recommendations are consistent with the observation of damage in the Northridge Earthquake, in which most fractures initiated at the root of the beam flange to column flange weld. It is of course possible that if notch tough material is used at this joint and not at others, fractures will initiate in future events at the next critical section, which may be a welded joint using material with low toughness at continuity plates, or other locations.

While there is a lack of agreement as to the extent to which notch toughness specifications should apply to welded joints in the moment connection, there is general agreement that previously acceptable electrodes that had no reported notch toughness values should no longer be used for the critical beam flange to column flange field welded joints. Most of the electrodes that are currently commercially available and have specified notch toughness requirements will meet the notch toughness recommendations contained in Chapter 8 of these Interim Guidelines. Additional research may indicate that alternate criteria are appropriate.

A similar level of disagreement exists with regard to the need for specifying notch toughness in base metals. Most of the fractures which have been investigated have initiated in the weld metal rather than in the base metal. Once these fractures extend into the base metal, they have already reached significant size and material toughness alone may not be able to arrest them. Additional research into the benefits of tough material, both in welds and base metals is clearly called for.

7.7 Quality Control/Quality Assurance

Refer to Chapters 9, 10 and 11 of these Interim Guidelines.

7.8 Guidelines on Other Connection Design Issues

The emphasis thus far in testing of connection assemblages has been on the beam flange/column flange joint. The other components of the connection such as panel zones, web connections and continuity plates have not been studied significantly as independent parameters in the available testing programs to date. It is assumed that the variation of these components will have effects on the performance of the connection and thus on the flange joints, and that an as yet undetermined balance of the sizes and details of these significant components will result in the optimum performance of a particular connection and its various joints. Interim Guidelines for these other critical portions of the connection assembly are presented below.

7.8.1 Design of Panel Zones

No current recommendations are made to supplement or modify the *UBC-1994* {*NEHRP-91*} provisions for the design of panel zones, other than as indicated in Section 7.5.2.6, above. Panel zone demands should be calculated in accordance with Section 7.5.2.6. As with other elements of the connection, available panel zone strength should be computed using minimum specified yield stress for the material, except when the panel zone strength is used as a limit on the required connection strength, in which case F_{ym} should be used.

Where connection design for two-sided connection assemblies is relying on test data for one-sided connection assemblies, consideration should be given to maintaining the level of panel zone deformation in the design to a level consistent with that of the test, or at least assume that the panel zone must remain elastic, under the maximum expected shear demands.

Commentary: At present, no changes are recommended to the code requirements governing the design of panel zones, other than in the calculation of the demand. There is evidence that panel zone yielding may contribute to the plastic rotation capability of a connection. However, there is also concern and some evidence that if the deformation is excessive, a kink will develop in the column flange at the joint with the beam flange and, if the local curvature induced in the beam and column flanges is significant, can contribute to failure of the joint. This would suggest that greater conservatism in column panel zone design may be warranted.

In addition to the influence of the deformation of the panel zone on the connection performance, it should be recognized that the use of doubler plates and especially the welding associated with them is likely to be detrimental to the connection performance. It is recommended that the Engineer consider use of column sizes which will not require addition of doubler plates, where practical.

7.8.2 Design of Web Connections to Column Flanges

Specific modifications to the code requirements for design of shear connections are not made at this time. It should be noted that the emergency code change to the *UBC-94* {*NEHRP-94*} deleted the former requirements for supplemental web welds on shear connections. This is felt to be appropriate since these welds can apparently contribute to the potential for shear tab failure at large induced rotations.

When designing shear connections for moment-resisting assemblies, the designer should calculate shear demands on the web connection in accordance with Section 7.5.2.4, above.

Commentary: Some engineers consider that it is desirable to develop as much bending strength in the web as possible. Additionally, it has been observed in some laboratory testing that pre-mature slip of the bolted web connection can result in large secondary flexural stresses in the beam flanges and the welded joints to the column flange. However, there is some evidence to suggest that if flange connections should fail, welding of shear tabs to the beam web may promote tearing of the tab weld to the column flange or the tab itself through the bolt holes, and some have suggested that welding be avoided and that web connections should incorporate horizontally slotted holes to limit the moment which can be developed in the shear tab, thereby protecting its ability to resist gravity loads on the beam in the event of flexural connection failure.

7.8.3 Design of Continuity Plates

Contrary to current code requirements, it is recommended that continuity plates be provided in all cases and that the thickness be at least equal to the thickness of the beam flange (not including cover plates) or one half the total effective flange thickness (flange plus cover plate). Welds of the continuity plate to the column should develop the strength of the continuity plate.

Where two-sided connection assemblies are designed based on one-sided connection assembly test data, consideration should be given to the effect of the greater distortion of the continuity plates expected in the two sided case.

For reinforced connections using vertical ribs or other configurations of reinforcement, continuity plate sizing should be based on engineering principles and consideration of stress patterns which may occur due to column flange distortion.

For connections incorporating haunches, continuity plates should be provided opposite the joint of the haunch flange with the column flange.

Commentary: The determination of continuity plate thickness requires, in addition to code conformance requirements, engineering judgment based on recognition of two competing factors:

a) Overly thick continuity plates and their welding will contribute to restraint and consequent residual stresses in the column, as well as to the other usual detrimental effects of large welds. Conditions of high restraint tend to be conducive to the initiation of fracture.

b) Omission of continuity plates or the use of overly thin continuity plates will permit column flange distortions which will, in turn, lead to higher stress concentrations in the beam flange joint opposite the column web.

Testing to date has not firmly established an appropriate design criteria for continuity plates, or even that these are definitely needed to obtain good connection performance in all cases. However, tests of specimens reinforced with cover plates to date, have been most successful when continuity plates were present (Engelhardt & Sabol - 1994). Tests using otherwise similar designs but with different continuity plate thicknesses have not been performed. This is an area where further research would be beneficial.

7.8.4 Design of Weak Column and Weak Way Connections

The code permits the use of strong beam/weak column designs under certain circumstances. There is some question as to what should be required for the connections at such conditions. While testing has demonstrated little capability of the pre-Northridge prescriptive connection to develop significant beam yielding without failure, it should be recognized that if the beam is stronger than the column, considering conservative estimates of the column strength including strain hardening, then the beam and its connection can be expected to remain below even this low failure threshold, and it would appear to be unnecessary to provide strengthened connections.

When beam connections are made to the web of columns (weak way) which are stronger than the beams, then connection design should be treated similarly to that of strong direction connections with additional consideration for the unique features of weak direction connections (see Tsai and Popov - 1988). Note that the question of column flange through-thickness strength is not a consideration for this type of connection, but that development of the strength of a cover plated flange through welds in shear to the inside face of the column flange may be difficult. Unless the members so connected represent a very small part of lateral resistance of the structure, testing of such connections should be considered as mandatory. Extrapolation of results from strong way connection testing should not be done. The effect of weak way connection action on the strength and behavior of companion strong way connections, for columns participating in orthogonal lateral-force-resisting frames, has not been tested.

Commentary: Since 1985, the strong column/weak beam principle has been required, but exceptions have been provided which permit weak columns in some instances. These exceptions have not been revoked, and, in fact, the interest in redundancy generated by the Northridge failures has actually increased interest in their use, to the extent permitted, in moment frame systems, where all beam-column connections in the structure are connected for moment resistance and made part of the lateral-force-resisting system. Considering the fact that columns resisting flexural demands about their minor axes will not generally be capable of developing the beam flexural yield strength should permit consideration of the pre-Northridge connections for this use. On the other hand, where specific code exceptions permit use of weak column systems for all or a large part of the lateral resistance a more conservative approach is merited. Use of weak column systems as the primary lateral resistance is strongly discouraged and should not be considered as a desirable or acceptable method of avoiding beam flange connection concerns and reinforcing requirements.

Further, although logic would indicate that the strength demand on connections in weak column structures would be limited by column hinging, and that therefore the beam-column connection should be protected, evidence suggests that this may not be the case. It has been reported that a hospital structure affected by the Northridge Earthquake experienced failure of almost all of its beam-column connections, despite having all or many weak column conditions.

7.9 Moment Frame Connections for Consideration in New Construction

The moment frame connection formerly prescribed by the code was configured to require development of a plastic hinge in the beam adjacent to the beam-to-column connection. The Northridge experience and subsequent testing have shown that as the possible result of a number of factors, it is not reasonable to expect reliable development of plastic hinges at this location, at least within the range of design parameters explored to date. Therefore, connections should be configured to encourage plastic hinging action to other locations.

The types of connections described in the following subsections are felt to offer some promise of providing more reliable inelastic action in WSMFs, consistent with that assumed in the design of such frames. It is of course assumed that the required joints, both welded and bolted, have been installed with appropriate quality control as described previously.

Reference to laboratory testing is provided for those connection configurations for which research has been reported. However, it should be noted that none of these connections has been tested sufficiently at this time to permit unqualified use of the connection.

The figures provided in the following sections are schematic, indicating the general type of connection configuration being described. When designing connections patterned after the reported test data, the test specimen details included in the references should be reviewed to determine specific details not shown.

The SAC Joint Venture does not endorse or specifically recommend any of the connection details shown in this Section. These are presented only to acquaint the reader with available information on representative testing of different connection configurations that have been performed by various parties.

Commentary: With the large interest and availability of funding for research on steel moment frame connections, any lists of connection concepts, such as the above will necessarily become at least partially obsolete by the time they are published. With this in mind, it is very important that there be a publicly accessible center to accumulate testing results as they become available. It is the recommendation of this guideline that as efforts in this area progress, SAC become the repository and distribution group for such information. It is hoped that all engineers, researchers, and contractors responsible for tested connections will willingly share all information on the tests and designs with SAC, with the structural engineering profession, and with the building construction industry.

The various connections suggested in this section were all nominally fully restrained (FR) connections. It has been suggested that partially restrained (PR) connections may be a cost-effective and reliable alternative to these connections. AISC and NSF are currently conducting research into the use of this system and it may become an attractive alternative in the future.

7.9.1 Cover Plate Connections

Figure 7-5 illustrates the basic configuration of cover plated connections. Short cover plates are added to the top and bottom flanges of the beam with fillet welds adequate to transfer the cover plate forces to the beam flanges. The bottom flange cover plate is shop welded to the column flange and the beam bottom flange is field welded to the column flange and to the cover plate. The top flange and the top flange cover plate are both field welded to the column flange with a common weld. The web connection may be either welded or high strength (slip critical) bolted. Limited testing of these connections (Engelhardt & Sabol - 1994), (Tsai & Popov -1988) has been performed.

A variation of this concept which has been tested successfully very recently (Forrel/Elsesser Engineers -1995), uses cover plates sized to take the full flange force, without direct welding of the beam flanges themselves to the column. In this version of the detail, the cover plate provides a cross sectional area at the column face about 1.7 times that of the beam flange area. In the detail which has been tested, a welded shear tab is used, and is designed to resist a significant portion of the plastic bending strength of the beam web.

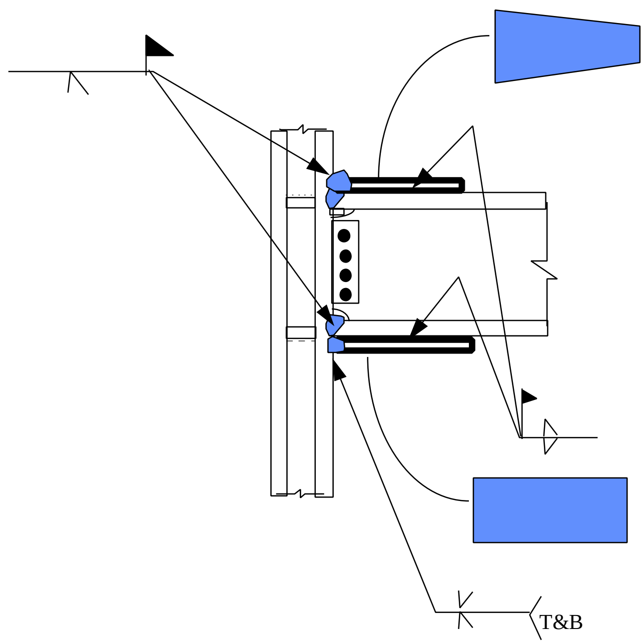


Figure 7-5 - Cover Plate Connection

Design Issues: Approximately eight connections similar to that shown in Figure 7-5 have been recently tested (Engelhardt & Sabol - 1994), and they have demonstrated the ability to achieve acceptable levels of plastic rotation provided that the beam flange to column flange welding is correctly executed and through-thickness problems in the column flange are avoided. This configuration is relatively economical, compared to some other reinforced configurations, and has limited architectural impact.

Six of eight connections tested by the University of Texas at Austin were able to achieve plastic rotations of at least 0.025 radians, or better. Strength loss at the extreme levels of plastic rotation did not reduce the flexural capacity to less than the plastic moment capacity of the section based on minimum specified yield strength. One specimen achieved plastic rotations of 0.015 radians when a brittle fracture of the CJP weld (type W2 failure) occurred. This may partially be the result of a weld that was not executed in conformance with the specified welding procedure specification. The second unsuccessful test specimen achieved plastic rotations of 0.005 radian when a section of the column flange (type C2 failure) occurred. A similar failure occurred in recent testing by Popov of a specimen with cover plates having a somewhat modified plan shape.

<u>Quantitative Results</u>: *No. of specimens tested: 8*
Girder Size: W36 x 150
Column Size: W14 x 455
Plastic Rotation achieved-
6 Specimens : >0.025 radian
1 Specimen: 0.015 radian
1 Specimen: 0.005 radian

Although apparently more reliable than the former prescriptive connection, this configuration is subject to some of the same flaws including dependence on properly executed beam flange to column flange welds, and through-thickness behavior of the column flange. Further these effects are somewhat exacerbated as the added effective thickness of the beam flange results in a much larger groove weld at the joint, and therefore potentially more severe problems with brittle heat affected zones and lamellar defects in the column. Indeed, a significant percentage of connections of this configuration have failed to produce the desired amount of plastic rotation.

7.9.2 Flange Rib Connections

Figure 7-6 demonstrates the basic configuration for connections with flange ribs. The intent of the rib plates is to reduce the demand on the weld at the column flange and to shift the plastic hinge from the column face.

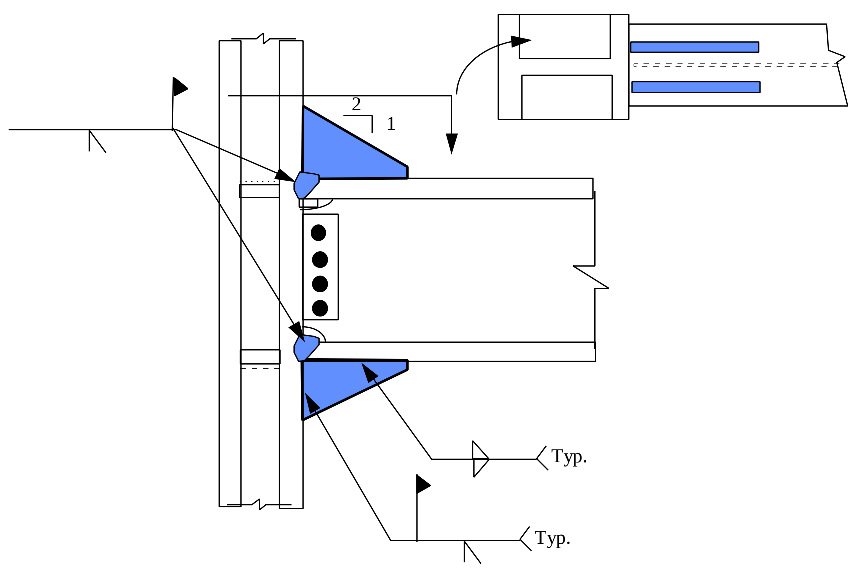


Figure 7-6 - Flange Rib Connection

<u>Design Issues:</u> *There is a limited body of testing of connections similar to these (Engelhardt & Sabol - 1994), (Tsai & Popov - 1988), and they have demonstrated the ability to achieve acceptable levels of plastic rotation provided that the girder flange welding is correctly executed.*

<u>Quantitative Results</u>: *No. of specimens tested: 2*
Girder Size: W36 x 150
Column Size: W14 x 455
Plastic Rotation achieved-
2 Specimens : >0.025 radian

Performance is dependent on properly executed girder flange welds. The joint can be subject to through-thickness failures in the column flange, although it should be somewhat more resistant to such failures than connections reinforced with cover plates, as the weld size is reduced. The size of the specimens tested required the use of two upstanding ribs per flange. This increased the costs significantly above those designs that use only one rib per flange, located above the girder center line. However, limited testing of the design with one rib at the girder centerline (Tsai & Popov) indicated the potential for premature failure of the weld of the rib to the girder at the outstanding edge. It should also be noted that the specimens tested by Engelhardt & Sabol, and reported above, incorporated columns with particularly heavy flanges. The ribs have the potential to cause high local stresses in the column flanges and this configuration may not behave acceptably when used with lighter section. Preliminary reports from fabricators and erectors indicate that the cost of this connection is quite high, relative to other configurations.

7.9.3 Bottom Haunch Connections

Figure 7-7 indicates several potential configurations for single, haunched beam-column connections. As with the cover plated and ribbed connections, the intent is to shift the plastic hinge away from the column face and to reduce the demand on the CJP weld by increasing the depth of the section. To date, the configuration incorporating the triangular haunch has been subjected to limited testing. Testing of configurations incorporating the straight haunch are currently planned, but have not yet been performed.

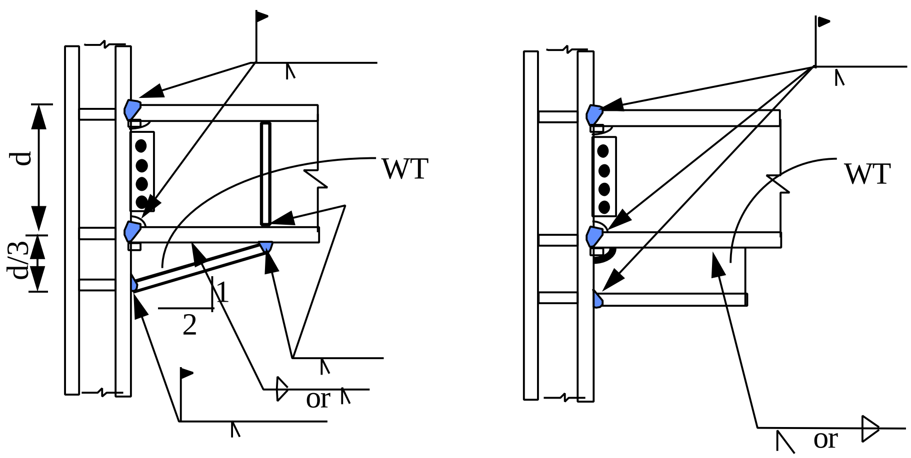


Figure 7-7 - Bottom Haunch Connection Modification

Two tests have been performed to date, both successfully. Both tests were conducted in a repair/modification configuration. In one test, a portion of the girder top flange, adjacent to the column, was replaced with a thicker plate. In addition, the bottom flange and haunch were both welded to the column. This specimen developed a plastic hinge within the beam span, outside the haunched area and behaved acceptably. A second specimen did not have a thickened top flange and the bottom girder flange was not welded to the column. Plastic behavior in this specimen occurred outside the haunch at the bottom flange and adjacent to the column face at the top flange. Failure initiated in the girder at the juncture between the top flange and web, possibly contributed to by buckling of the flange as well as lateral torsional buckling of the section. Fracture progressed slowly along the top fillet of the girder and eventually, traveled into the flange itself.

Design Issues: The haunch can be attached to the girder in the shop, reducing field erection costs. Weld sizes are smaller than in cover plated connections. The top flange is free of obstructions.

Quantitative Results: No. of specimens tested: 2
Girder Size: W30 x 99
Column Size: W14 x 176
Plastic Rotation achieved-
Specimen 1:0.04 radian (w/o bottom flange weld and reinforced top flange)
Specimen 2:0.05 radian (with bottom flange weld and reinforced top flange)

Performance is dependent on properly executed complete joint penetration welds at the column face. The joint can be subject to through-thickness flaws in the column flange; however, this connection may not be as sensitive to this potential problem because of the significant increase in the effective depth of the beam section which can be achieved. Welding of the bottom haunch requires overhead welding when relatively shallow haunches are used. The skewed groove welds of the haunch flanges to the girder and column flanges may be difficult to execute. The increased depth of the beam, resulting from the haunch may have undesirable impact on architectural design. Unless the top flange is prevented from buckling at the face of the column, performance may not be adequate. For configurations incorporating straight haunches, the haunch must be long, in order to adequately develop stress into the haunch, through the web. This tends to increase demands at the column face. Additional testing of all these configurations is recommended.

7.9.4 Top and Bottom Haunch Connections

Figure 7-8 illustrates this connection configuration. Haunches are placed on both the top and bottom flanges. Two tests have been performed on connections utilizing this configuration; both were highly successful.

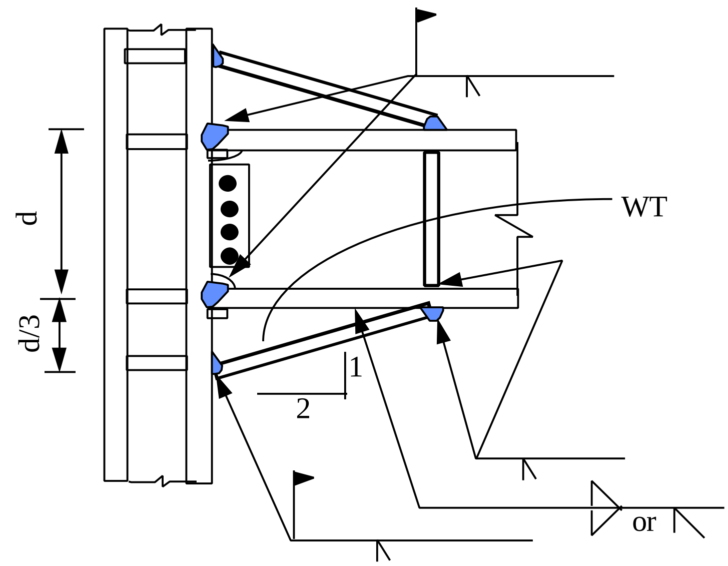


Figure 7-8 - Top and Bottom Haunch Connection

<u>Design Issues</u>: In two tests of this connection configuration performed to date, it has exhibited extremely ductile behavior. Plastic rotations as large as 0.07 radians were obtained. In addition to having very good plastic capacity, the connection is highly redundant. If failure should occur at one of the complete joint penetration welds of the haunch plate, significant residual strength would be available from the remaining girder flange welds.

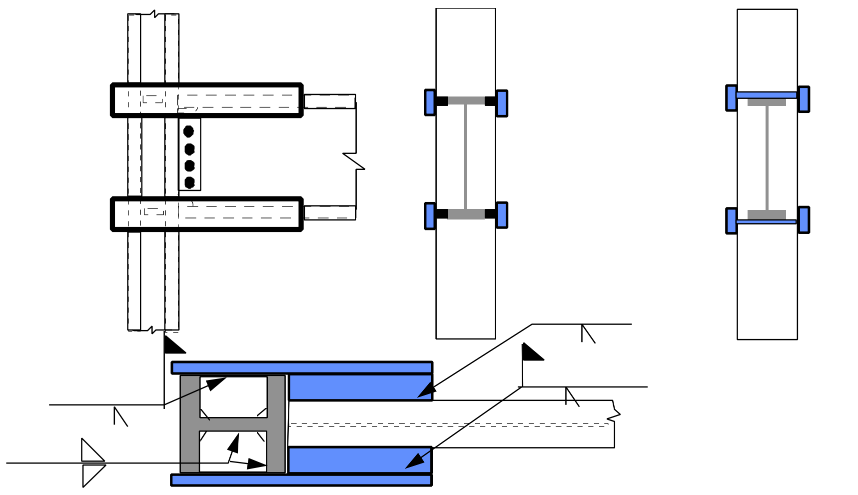
This is one of the more costly connection configurations. Some of this cost could be reduced by eliminating the welds between the girder flanges and columns, however, the performance of the connection in that configuration has not been tested. The presence of the haunch at the top of the girder could be an architectural problem.

Quantitative Results: No. of Specimens Tested: 2
Girder Size: W30 x 99
Column Size W14 x 176
Plastic Rotation achieved - 0.07 radians

7.9.5 Side-Plate Connections

This approach eliminates loading the column in the through-thickness direction by removing the CJP welds at the girder flange and by shifting the plastic hinge from the column face. The tension and compression forces are transferred from the girder flanges into the column through fillet welds. A mechanism to provide a direct connection between the column panel zone and the beam flanges is required; the difficulty appears to be equalizing the width of the beam and column flanges.

At least two configurations of side-plated connections have been tested. One set, shown in Figure 7-9, utilized flat bars at the top and bottom girder flanges, to transfer flange forces to the column (Engelhardt & Sabol - 1994). The girder was widened to the width of the column with the use of filler plates. The specimens achieved plastic rotations of 0.015 radians, however, fractures developed within the welds connecting the beam flange to the transfer plates. Failure of the shear tab, and finally the side plates themselves followed the initiation of these fractures. It is believed that the unsuccessful behavior of this particular specimen was related to the method used to increase the width of the beam flange to equal that of the column flange, using a combination of a filler bar and welding. Other approaches, such as providing a full width cover plate for the girder flanges, may provide better performance.



Tested Configuration

Possible Alternative

Figure 7-9 Side Plate Connection

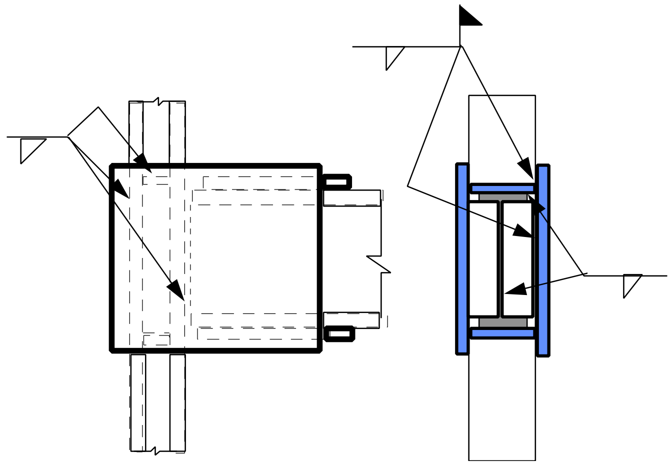
<u>Design Issues:</u> This connection avoids both the large complete joint penetration welds of the beam flange to the column and the potential for through-thickness failure of the column flange. Much of the additional fabrication can be performed in the shop.

This connection did not demonstrate adequate plastic rotation capacity in the configurations tested to date. Additional testing is required to determine if modified configurations will perform in a more acceptable manner.

<u>Quantitative Results</u>:

Separate Top & Bottom Side Plates
No. of specimens tested: 2
Girder Size: W36 x 150
Column Size: W14 x 455
Plastic Rotation achieved-
2 Specimens :0.015 radian

A second, proprietary configuration, is shown in Figure 7-10. Three specimens have undergone full-scale testing to date and achieved large plastic rotations. Loss of strength at large plastic rotation demands was comparable to that of other successful connections. The developer of this connection has applied for US and foreign patents. Further information on technical data for this configuration, may be obtained from the developer.



NOTICE OF CONFIDENTIAL INFORMATION:
WARNING: The information presented in this figure is PROPRIETARY. US and Foreign Patents have been applied for. Use of this information is strictly prohibited except as authorized in writing by the developer. Violators shall be prosecuted in accordance with US and Foreign Patent Intellectual Property Laws.

Figure 7-10 - Proprietary Side Plate Connection

Design Issues: Testing of three prototype specimens (Uang & Latham - 1995) indicates that this proprietary connection has the ability to achieve very satisfactory levels of plastic rotation without relying on sensitive CJP welds between the column and girder flanges or specifying weld material with notch toughness. The elimination of the through-thickness loading of the flange may result in higher levels of connection reliability. Due to the exclusive use of fillet welds, special inspection requirements for welding and bolting can be reduced significantly with this connection.

This connection is proprietary and license fees are associated with its use. The cost of the connection may be greater than some of the other modification methods discussed above; however, this cost differential may not be as great on double-sided connections because much of the cost is associated with the side plates which are similar for both single-sided and double-sided connections. However, double sided connections will require doubling the sizes of the welds which deliver the forces to the columns, and potentially increasing plate thickness as well. The connection of beams framing into the minor axis of the column are made more difficult by this connection, particularly if they must be connected for moment resistance. Publicly bid projects will have to develop performance specifications to permit other connections to be considered for use unless a strong case for sole-sourcing the connection can be made.

Quantitative Results:

No. of specimens tested: 3
Girder Size: W36 x 150

Column Size: W14 x 426
Plastic Rotation achieved-
3 Specimens : >0.042 radian to 0.06 radian

7.9.6 Reduced Beam Section Connections

In this connection, the cross section of the beam is intentionally reduced within a segment, to produce an intended plastic hinge zone or fuse, located within the beam span, away from the column face. Several ways of performing this cross section reduction have been proposed. One method includes removal of a portion of the flanges, symmetrical about the beam centerline, in a so-called "dog bone" profile. Care should be taken with this approach to provide for smoothly contoured transitions to avoid the creation of stress risers which could initiate fracture. It has also been proposed to create the reduced section of beam by drilling a series of holes in the beam flanges. Figure 7-11 illustrates both concepts. The most successful configurations taper the reduced section, through the use of unsymmetrical cut-outs, or variable size holes, to balance the cross section and the flexural demand.

Testing of this concept was first performed by a private party, and US patents were applied for and granted. These patents have now been released. Limited testing of both "dog-bone" and drilled hole configurations have been performed in Taiwan (Chen and Yeh - 1995). The American Institute of Steel Construction is currently performing additional tests of this configuration (Smith-Emery - 1995), however the full results of this testing are not yet available.

There is a concern that the presence of a concrete slab at the beam top flange would tend to limit the effectiveness of the reduced section of that flange, particularly when loading places the top flange into compression. It may be possible to mitigate this effect with proper detailing of the slab.

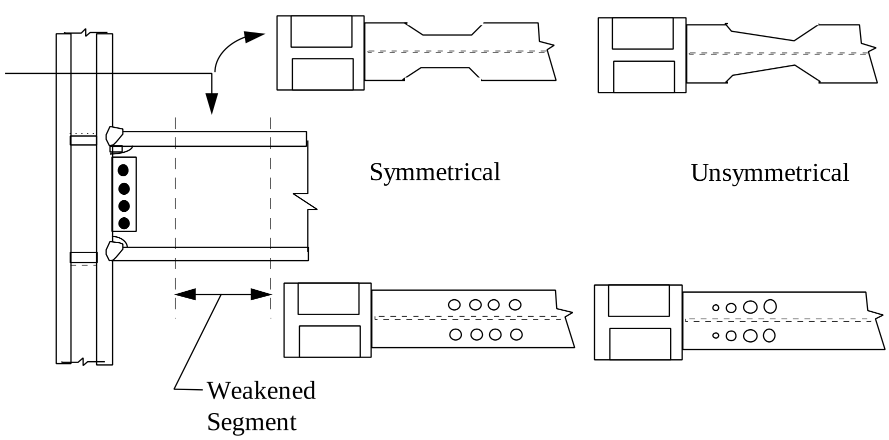


Figure 7-11 - Reduced Beam Section Connection

<u>Design Issues</u>: *This connection type is potentially the most economical of the several types which have been suggested. The reliability of this connection type is dependent on the quality of the complete joint penetration weld of the beam to column flange, and the through-thickness*

behavior of the column flange. If the slab is not appropriately detailed, it may inhibit the intended "fuse" behavior of the reduced section beam segment. It is not clear at this time whether it would be necessary to use larger beams with this detail to attain the same overall system strength and stiffness obtained with other configurations. In limited testing conducted to date of the unsymmetrical "dog-bone" configuration (Smith-Emery - 1995), the plastic hinging which occurred at the reduced section was less prone to buckling of the flanges than in some of the other configurations which have been tested, due to the very compact nature of the flange in the region of the plastic hinge.

Quantitative Results:

No. of specimens tested: 2
Girder Size: W30 x 99
Column Size: W14 x 176
Plastic Rotation achieved- 0.03 radian

7.9.7 Slip - Friction Energy Dissipating Connection

This connection uses high strength bolts and slotted holes to develop the flange forces into the columns. A brass shim in the shear transfer plane provides for controlled friction force. In concept, slip along this bolted connection limits the amount of force which can be transferred to the column and allows plastic deformation to occur in a benign manner. Two alternative configurations have been suggested for attachment of the flanges to the column. One incorporates bolted "T" sections and the other welded plates. Figure 7-12 shows the bolted "T" configuration.

To date, two tests have been performed on the bolted "T" configuration (Popov & Yang-1995). Results were excellent with large inelastic displacements obtained without strength or stiffness degradation.

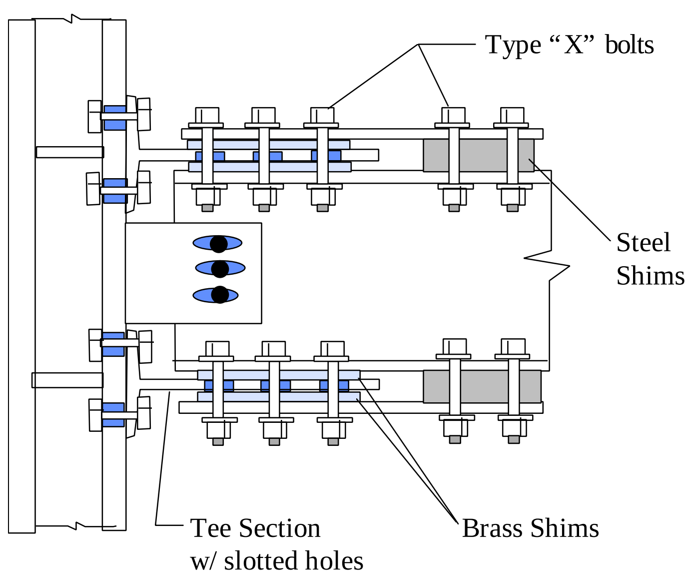


Figure 7-12 - Slip Friction Energy Dissipation Connection

Design Issues: In the limited testing performed, this connection was able to accommodate large inelastic displacements without damage to the connection or beams. This connection can be assembled in the field without welding and can accommodate large plastic rotations without permanent damage to the structure.

The connection is sensitive to fit-up, cleanliness of the faying surfaces and tension in the high-strength bolts, and therefore will require careful field quality control. As with the "reduced section beam" connections, this connection may be sensitive to the presence of a slab and careful detailing of the slab to permit the expected connection rotations to occur may be required. The strength that can be developed by this connection is limited by the number of bolts that can be practically placed. It may not be suitable for use with larger members with high strength demands. The brass shims, used at the slip plane interface are quite costly. Metal parts kept in contact under pressure over a period of years may tend to become partially welded together, potentially reducing the effectiveness of the connection with time. Additional research is required on this effect.

7.9.8 Column-Tree Connection

This concept has been widely used in Japan, with mixed success. Short stubs of girders are fabricated and shop welded to the column. Field connection to the balance of the girder is made with bolted connections. The girder stubs can be intentionally fabricated stronger than the balance of the girder, to force yielding and formation of a plastic hinge away from the column. Figure 7-13 demonstrates the basic concept.

Extensive testing of this connection has not been performed in the US Some variations of this connection, in common use in Japan, performed very poorly in the recent Kobe Earthquake (Watabe-1995). In at least one version of this connection, the beam stub ran continuously through the connection and the columns were shop welded to the top and bottom of this stub. A number of these connections experienced fracture of the shop weld of the column to the beam stub. However, it is reasonable to expect that configurations for this concept can be developed that would permit more favorable behavior.

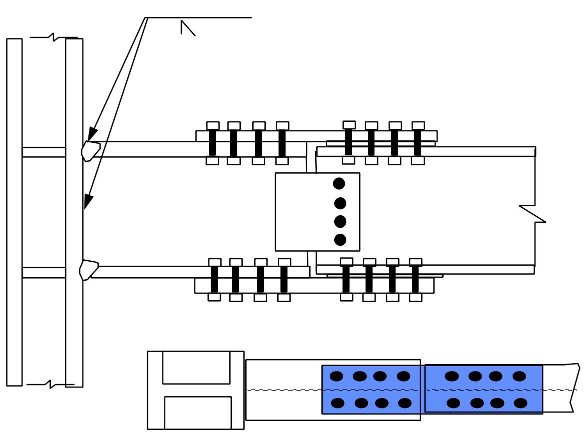


Figure 7-13 - Column Tree Connection

Design Issues: The basic advantage to this connection is that critical welding can be performed in the fabrication shop where it should be possible to attain better quality control. In addition, field erection costs are reduced through the use of bolted field connections.

Testing of this connection in configurations similar to US construction practice has not been performed. Some configurations utilized in Japan performed poorly in the Kobe Earthquake. The connection is dependent on the quality of beam flange to column flange welding and the through-thickness behavior of the column flange. Transportation and handling of tree columns is probably somewhat more difficult and expensive than for standard columns.

7.9.9 Slotted Web Connections

In the former prescriptive connection, in which the beam flanges were welded directly to the column flanges, beam flexural stress was transferred into the column web through the combined action of direct tension across the column flange, opposite the column web, and through flexure of the column flange. This stress transfer mechanism results in a large stress concentration at the center of the beam flange, opposite the column web. Recent research (Allen, et. al. - 1995) indicates that the provision of continuity plates within the column panel zone reduces this stress concentration somewhat, but not completely. The intent of slotted web connections is to further reduce this stress concentration and to achieve a uniform distribution of flexural stress across the beam flange at the connection. Figure 7-14 indicates one configuration for this connection type that has been successfully tested. In this configuration, vertical plates are placed between the column flanges, opposite the edges of the top and bottom beam flanges to stiffen the outstanding column flanges and draw flexural stress away from the center of the beam flange. Horizontal plates are placed between these vertical plates and the column web to transfer shear stresses to the panel zone. The web itself is softened with the cutting of a vertical slot in the column web, opposite the beam flange. High fidelity finite element models were utilized to confirm that a nearly uniform distribution of stress occurs across the beam flange.

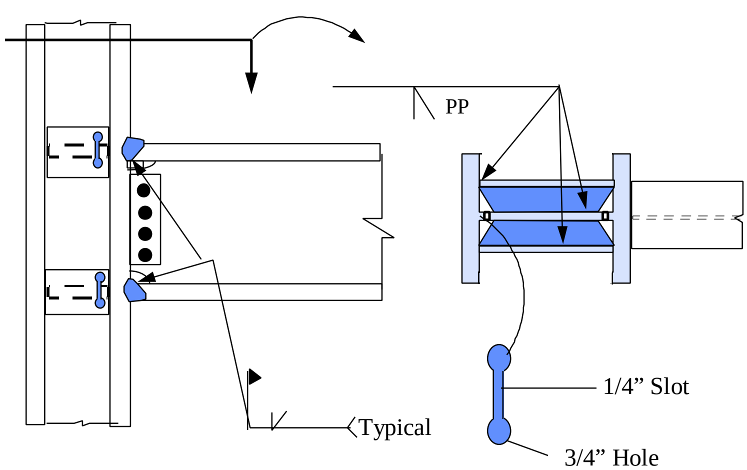


Figure 7-14 - Slotted Web Connection

Design Issues: This detail is potentially quite economical, entailing somewhat more shop fabrication than the former prescriptive connection, but similar levels of field erection work. Contrary to the recommendations contained in these Interim Guidelines, this connection does not shift the location of plastic hinging away from the column face. However, two connections similar to that shown in Figure 7-14 have recently been tested succesfully (Allen. - 1995). The connection detail is sensitive to the quality of welding employed in the critical welds, including those between the beam and column flanges, and between the vertical and horizontal plates and the column elements. It has been reported that one specimen, with a known defect in the beam flange to column flange weld was informally tested and failed at low levels of loading.

The detail is also sensitive to the balance in stiffness of the various plates and flanges. For configurations other than those tested, detailed finite element analyses may be necessary to confirm that the desired uniform stress distribution is achieved. The developer of this detail indicates that for certain column profiles, it may be possible to omit the vertical slots in the column web and still achieve the desired uniform beam flange stress distribution.

This detail may also be sensitive to the toughness of the column base metal at the region of the fillet between the web and flanges. In heavy shapes produced by some rolling processes the metal in this region may have substantially reduced toughness properties relative to the balance of the section. This condition, coupled with local stress concentrations induced by the slot in the web may have the potential to initiate premature fracture. The developer believes that it is essential to perform detailed analyses of the connection configuration, in order to avoid such problems. Popov tested one specimen incorporating a locally softened web, but without the vertical and horizontal stiffener plates contained in the detail shown in Figure 7-14. That specimen failed by brittle fracture through the column flange which progressed into the holes cut into the web. The stress patterns induced in that specimen, however, were significantly different than those which occur in the detail shown in the figure.

Quantitative Results: Number of specimens tested: 2
Girder Size: W 27x94
Column Size: W 14x176
Plastic Rotation Achieved:
Specimen 1: 0.025 radian
Specimen 2: 0.030 radian

7.10 Other Types of Welded Connection Structures

These Interim Guidelines have focused on the design of the moment and shear resisting FR connections in moment frame systems in which the lateral forces are resisted by bending in beams and columns. In addition to moment frame systems, there are a number of other system types which conceivably could exhibit connection or joint distress similar to that seen in moment frames in Northridge when deformed under high intensity earthquake motions. Except for one detail of a welded base plate at the base of a braced frame, there has been no reported damage to the following systems from the Northridge earthquake. The response to earthquake motions, however, presents similar potential conditions to those found in moment frame connections.

Therefore when designing new construction, close attention should be given to these structural systems and details so that damage similar to that observed on WSMF systems can be avoided in future earthquakes.

7.10.1 Eccentrically Braced Frames (EBF)

EBF provisions in the code require the use of "link beam" elements. Link beams are usually designed to yield in shear in the web, but can be designed to yield in flexure. In some configurations, the link beam is connected to the column flange in a manner nearly identical to the connection of the WSMF. The connection of the brace element to the beam also connects to the beam flange, but the connection has additional design requirements which modify the type of connection. In addition to connection concerns for EBFs, the currently recognized variability of steel strengths should be considered in designing the components of the EBF. It is recommended that connections in EBFs intended to resist plastic rotation demands be designed the same as WSMF connections, as previously defined in these Interim Guidelines, with due consideration given to the additional shear forces which may be induced in the link beams.

Commentary: Although the code provisions are intended to cause the link beams connected to columns to yield in shear (length of link limited to $1.6M_S/V_S$), the link may be at or near its bending strength when shear yielding would occur. Thus the connection to the column flanges may be vulnerable to the same or similar problems as those exhibited by WSMF connections during the Northridge earthquake.

Recognition of the probable strength of steel in the link beam could be critical to the performance of these structures. The inelastic behavior of EBF structures is intended to be controlled through yielding of these links. If link beams are fabricated from excessively strong material, they may not yield before other parts of the frame become damaged.

7.10.2 Dual Systems

The provisions for Dual Systems in the code require that the system include a Special Moment-Resisting Frame (SMRF) designed according to the same provisions as if it were the primary system but capable of resisting at least 25% of the required lateral forces. In addition, it is required to have a primary system consisting of either concrete shear walls, Special Concentrically Braced Frames (SCBF), Concentrically Braced Frames (CBF), or Eccentrically Braced Frames (EBFs). Connection design for moment frames used in Dual Systems should conform to the recommendations of these Interim Guidelines for SMRF systems.

Commentary: Prior to the 1967 UBC, Dual Systems design required a primary system (shear walls or CBFs) capable of resisting 100% of the required lateral forces in conjunction with a "back-up" SMRF capable of resisting at least 25% of the total forces. The assumption for this type of system was that the SMRF would take over and prevent collapse of the structure in the event of failure of the stiffer, *but ostensibly less ductile, primary system. In this concept of design, the SMRF was there solely for redundancy. In the 1967 UBC, an additional provision was added in which the primary system and the SMRF were required to be designed to share the total required lateral force according to their elastic stiffness. In the 1988 UBC, the requirement that the primary system (shear walls or braced frames) be designed to resist 100% of the required total force was eliminated, but the other two requirements remain. This potentially makes it even more important that the SMRF portion of a dual system have adequate ductility to survive a major event.*

In general, dual systems have been a somewhat controversial system. Some engineers believe that the added redundancy provided by the backup system is quite beneficial while others do not believe that the relatively weak and often flexible back-up system improves building performance significantly. Little analytical research of these systems has been performed. Such research would be beneficial, however, in providing guidance as to the amount of ductility required of the backup frame system.

7.10.3 Welded Base Plate Details

The detail of concern is in any system of steel framing where a column, which is subject to high axial tension or flexure, or both, is directly welded to its baseplate in a manner similar to that used for beam-to-column moment connections. Additional concerns occur when anchor bolts are fastened to the base plate in close proximity to the bottom of the column.

Commentary: When a column is welded directly to the base plate and has the potential for being loaded with significant tension or tension in combination with flexure, CJP welds and the through-thickness strength of the base plate are required to resist the tensile forces. The combination of uncertainty of the through-thickness strength and the uncertainty of the axial loading suggests that another type of connection detail should be chosen. Frequently, the anchor bolts are placed close to the face of the column flanges. If the anchor bolts are strong enough so that the mechanism of failure is flexure in the base plate, the short flexural span makes it impossible for flexural yielding to occur and may result in a brittle fracture of the plate or of the CJP welds.

7.10.4 Vierendeel Truss Systems

A Vierendeel Truss (VT) is a type of truss without diagonals in which shear forces are resisted by the vertical members and chords, acting together as moment-resisting frames. VT's may have diagonals in some bays in some designs, but may also be designed to rely totally on the verticals. Where both chords and verticals of VT's are wide flange shapes, the connections of the verticals to the chords and the chords to the columns are often detailed in the same manner as the beam-to-column flange connection of WSMFs. A variation on the conventional horizontal Vierendeel Truss which deserves similar attention is a system where vertical loads in a discontinuous column are supported by moment connected beams at several floors, rather than by a single transfer girder above the location of the column discontinuity.

Commentary: Considering the brittle nature of the damage to steel structures due to the Northridge earthquake, Engineers should have some concerns about VT systems as described above even when they are designed to carry vertical loads only, particularly if the loadings are variable and could significantly exceed design loads in extreme cases. Where VT's are a part of the lateral system, either serving simply as a moment frame girder or as a transfer girder, seismic deformations potentially could lead to yielding at the connections of the truss verticals to the chords. Such connections should be designed in the same manner as the beam-to-column connections of WSMFs. If such yielding is possible, the effect of such yielding on the vertical load capacity and deformation should be investigated.

7.10.5 Moment Frame Tubular Systems

This type of lateral-force-resisting system is common in very tall buildings. The moment frame is arranged with relatively short spacing of columns around the perimeter of the structure. The system is actually a special type of WSMF which has very stiff beams so that the chord forces at the end of a moment frame can be distributed to adjacent columns perpendicular to the plane or around the corner of the moment frame. The system is defined as a three-dimensional space frame structure composed of three or more frames connected at the corners (or intersections) to form a vertical tube-like structure (or a structure composed of several adjacent tubes). Of particular concern is the short beam span which renders some of the solutions for local strengthening of beams difficult to achieve. On the other hand, plastic rotational demands due to high seismic forces may be shown to be very low in some designs.

Commentary: Moment frame tube structures are normally very redundant systems with many moment-resisting members and connections. A thorough analysis of the structural system should be made to determine what potential plastic rotation demand would be required on the connections. With very tall buildings, seismic response becomes more heavily influenced by the higher modes of vibration, and design of members and connections might be controlled by wind forces.

7.10.6 Welded Connections of Collectors, Ties and Diaphragm Chords

These members are part of a building’s lateral-force-resisting system. They are usually horizontal members which, in addition to supporting adjacent gravity load, are also required to transmit large axial tensile and compressive forces. If development of the tensile and flexural forces at the connections to the columns requires welding of the member flanges to the column, all of the recommendations for WSMF connections should be followed.

Commentary: Where chord, collector and time members are nearly pure axial members, such as would occur in a building with shear walls or braced frames that is laterally very stiff, the former prescriptive connection may be found to be sufficient, depending on the size of the member..

7.10.7 Welded Column Splices

Even though no column splice damage has been reported from the Northridge earthquake, column splices incorporating partial penetration flange welds should be used cautiously, particularly if the potential for large tensile and/or flexural forces are present. Partial penetration welds result in a crack-like feature, which can initiate fracture under conditions of high stress. A number of structures experienced failure of column splices in the 1995 Kobe Earthquake, with some such failures leading to structural collapse.

Commentary: Of particular concern would be the use of partial penetration butt welds on the column flanges. The configuration of partial penetration welds provides a notch on the inner edge of the weld. Thus other methods of effecting a column splice should be used if significant or unpredictable tensile or flexural forces are possible. When considering bending in column splices of moment frames, it has been shown by inelastic time-history analyses that reliance should not be placed on the inflection point occurring at the mid height of the column. The studies show that the location of the hinge can change significantly as the structure deforms, both due to higher mode effects and due to the inelastic response of the members.

7.10.8 Built-up Moment Frame Members

Built-up beams and columns used in moment frame systems have the same concerns in the design of connections as the rolled shape systems previously discussed. The welds connecting the various component parts of the built-up members should be designed to be capable of resisting the effects of potential plastic behavior and connections of built-up members should be designed to preclude reliance on yielding of steel in areas of confined or restrained joints.

Commentary: In beams, the effect of the shape of the components, including the relative thickness of flanges and web can be significant in determining the forces required to be developed in the various joints in the connection of the beam to the column. Also the joint between the web and flanges of the built-up beams, particularly in the areas of potential plastic hinges, should be designed to be capable of permitting flange buckling without weld failure.

In H shaped built-up columns, the welded joints between the web and flanges should be designed to develop the panel zone shears based on the probable location of the plastic hinges. In tubular or box shaped columns, the placing of the plates and the selection of the type of weld connecting webs to flanges is *important in providing adequate joints to resist the forces in the beam-to-column connection zone. Some testing has been performed Taiwan (Tsai - 1995).*