7. NEW CONSTRUCTION

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 IMRF and OMRF structures located in *Uniform Building Code* (*UBC*) Seismic Zones 3 and 4 {*National Earthquake Hazards Reduction Program* (*NEHRP*) Map Areas 6 and 7} or assigned to 1997 NEHRP Seismic Design Categories D, E, or F. 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. In an attempt to address this, both the 1997 edition of the Uniform Building Code and the 1997 edition of the NEHRP Provisions specify design ground motions for structures located close to major active faults that are substantially more severe than those contained in earlier codes. While the more severe ground motion criteria contained in these newer provisions are probably adequate for the design of most structures, analytical studies conducted by SAC confirm that even structures designed to these criteria can experience very large drift demands, and potentially collapse, if the dynamic characteristics of the impulsive loading and those of the structure are matched. 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) and OMRF structures assigned to 1997 NEHRP Seismic Design Categories D, E or F, 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 either the 1997 edition of the *AISC Seismic Provisions for Structural Steel Buildings* (AISC, 1997) or 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 the time the Interim Guidelines were first published, they were based on the 1994 edition of the Uniform Building Code and the 1994 edition of the NEHRP Provisions. In the time since that initial publication, more recent editions of both documents have been published, and codes based on these documents have been adopted by some jurisdictions. In addition, the American Institute of Steel Construction has adopted a major revision to its Seismic Provisions for Structural Steel Buildings (AISC Seismic Provisions), largely incorporating, with some modification, the recommendations contained in the Interim Guidelines. This updated edition of the AISC Seismic Provisions has been incorporated by reference into the 1997 edition of the NEHRP Provisions and has also been adopted by some jurisdictions as an amendment to the model building codes. SMRF and OMRF systems that are designed to comply with the requirements of the 1997 AISC Seismic Provisions may be deemed to comply with the intent of these Interim Guidelines. Where reference is made herein to the *requirements of the 1994 Uniform Building Code or 1994 NERHP Provisions, the parallel provisions of the 1997 editions may be used instead, and should be used in those jurisdictions that have adopted codes based on these updated standards.*

The 1997 edition NEHRP Provisions and AISC Seismic Provisions introduce a new structural system termed an Intermediate Moment Resisting Frame (IMRF). Provisions for IMRF structures include somewhat more restrictive detailing and design requirements than those for OMRF structures, and less than those for SMRF structures. The intent was to provide a system that would be more economical than SMRF structures yet have better inelastic response capability than OMRF structures. The SAC project is currently conducting research to determine if the provisions for the new IMRF system are adequate, but has not developed a position on this at this time.

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 3/8R_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 and Stiffness

7.2.2.1 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.

Many WSMF structures are constructed with concrete floor slabs that are provided with positive shear attachment between the slab and the top flanges of the girders of the moment-resisting frames. Although not generally accounted for in the design of the frames, the resulting composite action can increase the effective strength of the girder significantly, particularly at sections where curvature of the girder places the top flange into compression. Although this effect is directly accounted for in the design of composite systems, it is typically neglected in the design of systems classified as moment resisting steel frames. The increased girder flexural strength caused by this composite action can result in a number of effects including the unintentional creation of weak column - strong beam and weak panel zone conditions. In addition, this composite effect has the potential to reduce the effectiveness of reduced section or "dog-bone" type connection assemblies. Unfortunately, very little laboratory testing of large scale connection assemblies with slabs in place has been performed to date and as a result, these effects are not well quantified. In keeping with typical contemporary design practice, the design formulae provided in these Guidelines neglect the strengthening effects of composite action. Designers should, however, be alert to the fact that these composite effects do exist.

7.2.2.2 Stiffness

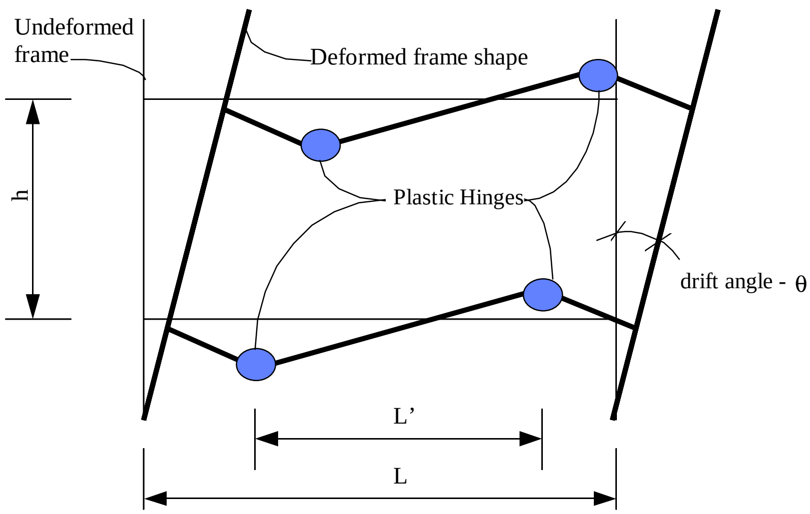
Calculation of frame stiffness for the purpose of determining interstory drift under the influence of the design lateral forces should be based on the properties of the bare steel frame, neglecting the effects of composite action with floor slabs. The stiffening effects of connection reinforcements (e.g.: haunches, side plates, etc.) may be considered in the calculation of overall frame stiffness and drift demands. When reduced beam section connections are utilized, the reduction in overall frame stiffness, due to local reductions in girder cross section, should be considered.

Commentary: For design purposes, frame stiffness is typically calculated considering only the behavior of the bare frame, neglecting the stiffening effects of slabs, gravity framing, and architectural elements. The resulting calculation of building stiffness and period typically underestimates the actual properties, substantially. Although this approach can result in unconservative estimates of design force levels, it typically produces conservative estimates of interstory drift demands. Since the design of most moment-resisting frames are controlled by considerations of drift, this approach is considered preferable to methods that would have the potential to over-estimate building stiffness. Also, many of the *elements that provide additional stiffness may be subject to rapid degradation under severe cyclic lateral deformation, so that the bare frame stiffness may provide a reasonable estimate of the effective stiffness under long duration ground shaking response.*

Notwithstanding the above, designers should be alert to the fact that unintentional stiffness introduced by walls and other non-structural elements can significantly alter the behavior of the structure in response to ground shaking. Of particular concern, if these elements are not uniformly distributed throughout the structure, or isolated from its response, they can cause soft stories and torsional irregularities, conditions known to result in poor behavior.

7.2.3 Configuration

Frames should be proportioned so that the required plastic deformation of the frame can ~~may~~ be accommodated through the development of plastic hinges at pre-determined locations within the girder spans, as indicated in ~~Figure 7-1~~Figure 7.2.3-1. 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~~ Figure 7.2.3-1 - Desired Plastic Frame Behavior

Commentary: Nonlinear deformation of frame structures is typically 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 and compressive fibers and by 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 weak story 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 assumed development of plastic hinge zones within the beams ~~at~~ adjacent to 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. Although ongoing research may reveal conditions of material properties, design and detailing configurations that permit connections with yielding occurring at the column face to perform reliably, for the present it is recommended ~~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, if plastic rotations are large, exhibit significant~~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.

It is important to recognize that in frames with relatively short bays, the flexural hinging indicated in Figure 7.2.3-1 may not be able to form. If the effective flexural length (L' in the figure) of beams in a frame becomes too short, then the beams or girders will yield in shear before zones of flexural plasticity can form, resulting in an inelastic behavior that is more like that of an eccentrically braced frame than that of a moment frame. This behavior may inadvertently occur in frames in which relatively large strengthened connections, such as haunches, cover plates or side plates have been used on beams with relatively short spans. This behavior is illustrated in Figure 7.2.3-2.

The guidelines contained in this section are intended to address the design of flexurally dominated moment resisting frames. When utilizing these guidelines, it is important to confirm that the configuration of the structure is such that the *presumed flexural hinging can actually occur. It is possible that shear yielding of frame beams, such as that schematically illustrated in Figure 7.2.3-2 may be a desirable behavior mode. However, to date, there has not been enough research conducted into the behavior of such frames to develop recommended design guidelines. Designers wishing to utilize such configurations should refer to the code requirements for eccentrically braced frames. Particular care should be taken to brace the shear link of such beams against lateral-torsional buckling and also to adequately stiffen the webs to avoid local buckling following shear plastification.*

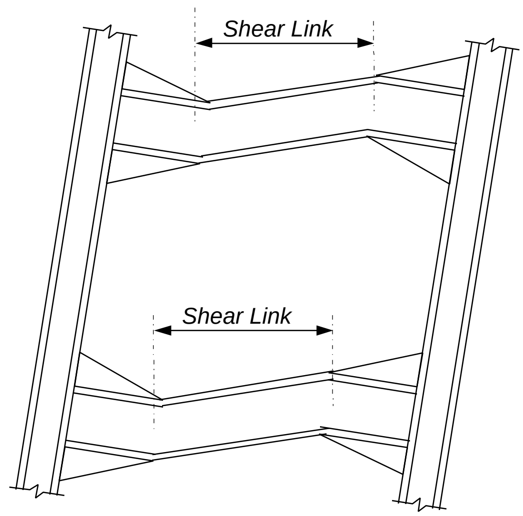


Figure 7.2.3-2 Shear Yielding Dominated Behavior of Short Bay Frames

7.2.4 Plastic Rotation Capacity

The plastic rotation capacity of tested 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. As used in these Guidelines, plastic rotation is defined as the plastic chord rotation angle. The plastic chord rotation angle is calculated using the rotated coordinate system shown in Fig. 7.2.4-1 as the plastic deflection of the beam or girder, at its point of inflection (usually the mid-span,) Δ_{CL}, divided by the distance between this mid-span point and the centerline of the panel zone of the beam column connection, L_{CL}. This convention is illustrated in Figure 7.2.4-1.

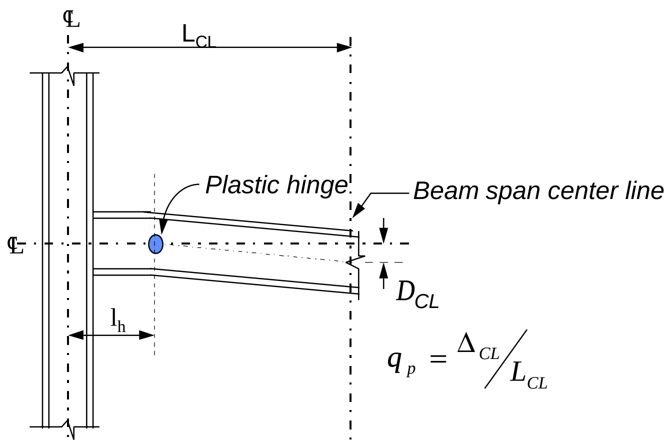


Figure 7.2.4-1 Calculation of Plastic Rotation Angle

It is important to note that this definition of plastic rotation is somewhat different than the plastic rotation that would actually occur within a discrete plastic hinge in a frame model similar to that shown in Figure 7.2.3-1. These two quantities are related to each other, however, and if one of them is known, the other may be calculated from Eq. 7.2.4-1.~~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~~:

$$\theta_p = \theta_{ph} \frac{(L_{CL} - l_h)}{L_{CL}} \qquad (7.2.4\text{-}1)$$

where: θ_p is the plastic chord angle rotation, as used in these Guidelines
θ_{ph} is the plastic rotation, at the location of a discrete hinge
L_{CL} is the distance from the center of the beam-column assembly panel zone to the center of the beam span
l_h is the location of the discrete plastic hinge relative to the center of the beam-column assembly panel zone

$$\cancel{\theta = 0.025(1 + (L - L')/L')} \qquad \cancel{(7\text{-}1)}$$

~~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 measures are taken, nonlinear dynamic analyses should be performed and the connection demands taken as 0.005 radians greater than the plastic rotation demands 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. When using methods of nonlinear analysis to establish the plastic rotation demands on frame connections, the analysis results should not be scaled by the factor R_w (R) or R_{Wi} (R_i), as otherwise permitted by the building code.

Commentary: When the Interim Guidelines were first published, the plastic rotation was defined as that rotation that would occur at a discrete plastic hinge, similar to the definition of q_{ph}, in Eq. 7.2.4-1, above. In subsequent testing of prototype connection assemblies, it was found that it is often very difficult to determine the value of this rotation parameter from test data, since actual plastic hinges do not occur at discrete points in the assembly and because some amount of plasticity also occurs in the panel zone of many assemblies. The plastic chord angle rotation, introduced in this advisory, may more readily be obtained from test data and also more closely relates to the drift experienced by a frame during earthquake response.

This change in the definition of plastic rotation does not result in any significant change in the acceptance criteria for beam-column assembly qualification testing. When the Interim Guidelines were first published, they recommended an acceptance criteria given by Eq. 7.2.4-2, below:

$$q_p = 0.025\left(1 + \frac{L - L'}{L'}\right) \qquad (7.2.4\text{-}2)$$

For typical beam-column assemblies in which the plastic hinge forms relatively close to the face of the column, perhaps within a length of 1/2 the beam depth, this typically resulted in a plastic rotation demand of 0.03 radians, as currently measured.

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 plastic rotation 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 yielded~~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 considered or designed 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 are significantly reduced~~s~~. 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 greater ability to limit ~~contro~~l 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.*

In order to codify the need for more redundant structural systems, the 1997 Uniform Building Code has specifically adopted a reliability coefficient, r_x, tied to the redundancy of framing present in the building. This coefficient, with values varying from 1.0 for highly redundant structures to 1.5 for non-redundant structures, is applied to the design earthquake forces, E, in the load combination equations, and has the effect of requiring more conservative design force levels for structures with nonredundant systems. The ~~Building Seismic Safety Council's Provisions Update Committee has also approved a proposal to include such a coefficient in the~~ 1997 NEHRP Provisions also includes such a coefficient. The formulation of this coefficient and its application are very similar in both the 1997 Uniform Building Code and 1997 NEHRP Provisions.

As ~~proposed~~ contained in the 1997 NEHRP Provisions, the reliability coefficient is given by the equation:

$$r = 2 - \frac{20}{r_{\max}\sqrt{A_x}} \qquad (7.25\text{-}1)$$

where:

$r_{\max x}$ = *the ratio of the design story shear resisted by the single element carrying the most shear force in the story to the total story shear, for a given direction of loading. For moment frames, $r_{\max x}$ is taken as the maximum of the sum of the shears in any two adjacent columns in a moment frame divided by the story shear. For columns common to two bays with moment resisting connections on opposite sides at the level under consideration, 70% of the shear in that column may be used in the column shear summation.*

A_x = *the floor area in square feet of the diaphragm level immediately above the story.*

The 1997 UBC and NEHRP Provisions also require that structures utilizing moment resisting frames as the primary lateral force resisting system be proportioned such that they qualify for a maximum value of r_x of 1.25. Structures located within a few kilometers of major active faults must be configured so as to qualify for a maximum value of r_x of 1.1.

The most redundant moment-resisting frame systems are distributed frames in which all beam-column connections are detailed to be moment resisting. In these types of structures, half of the moment-resisting connections will be to the minor axis of the column which will typically result in weak column/strong beam framing. The ~~1994~~ UBC requirements limit the portion of the building design lateral forces that can be resisted by ~~relative number of~~ weak column/strong beam connections in the moment frame system. This limitation was adopted to avoid the design of frames likely to develop story mechanisms as opposed to concern about the adequacy of moment-resisting connections to the minor axis of columns. However, the limited research data available on such connections suggests that they do not behave well.

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 the redundancy recommendations of 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

There are no modifications to the Guidelines or Commentary of Section 7.2.6 at this time.

7.2.7 Special Systems

There are no modifications to the Guidelines or Commentary of Section 7.2.7 at this time.

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. Section 7.5.2 outlines a design procedure for reinforced connection designs. Section 7.5.3 provides a design procedure for reduced section connections.

7.3.2 Qualification by Testing

There are no modifications to the Guidelines or Commentary of Section 7.3.2 at this time.

7.3.3 Design by Calculation

There are no modifications to the Guidelines or Commentary of Section 7.3.3 at this time.

7.4 Guidelines for Connection Qualification by Testing

7.4.1 Testing Protocol

There are no modifications to the Guidelines or Commentary of Section 7.4.1 at this time.

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, t~~Throughout the loading history required to achieve the required plastic rotation capacity, as indicated in a), above, the connection should develop a minimum moment at the column face as follows:

i) For strengthened connections, the minimum moment at the column face should be equal to the plastic moment of the girder, calculated using the minimum specified yield strength, F_y of the girder. 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.

ii) For reduced section connection designs, the minimum moment at the column face should be equal to the moment corresponding to development of the nominal plastic moment of the reduced section at the reduced section, calculated using the minimum specified yield strength, F_y of the girder, and the plastic section modulus for the reduced section. The moment at the column face should not be less than 80% of the nominal plastic moment capacity of the unreduced girder section.

c) The connection should exhibit ductile behavior throughout the loading history. A specimen that exhibits a brittle limit state (e.g. complete flange fracture, column cracking, through-thickness failures of the column flange, fractures in welds subject to tension, shear tab cracking, etc.) prior to reaching the required plastic rotation should be considered unsuccessful.

d) Throughout the loading history, until the required plastic rotation is achieved, the connection should be judged capable of supporting dead and live loads required by the building code. In those specimens where axial load is applied during the testing, the specimen should be capable of supporting the applied load throughout the loading history.

The evaluation of the test specimen's performance should consistently reflect the relevant limit states. For example, the maximum reported moment and the moment at the maximum plastic rotation are unlikely to be the same. It would be inappropriate to evaluate the connection using the maximum moment and the maximum plastic rotation in a way that implies that they occurred simultaneously. In a similar fashion, the maximum demand on the connection should be evaluated using the maximum moment, not the moment at the maximum plastic rotation unless the behavior of the connection indicated that this limit state produced a more critical condition in the connection.

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. However, there is evidence to suggest that extrapolation of test results to assemblies using members of reduced size is not always conservative. In a recent series of tests of cover plated connections, conducted at the University of California at San Diego, a connection assembly that produced acceptable results for one family of beam sizes, W24, did not behave acceptably when the beam depth was reduced significantly, to W18. In that project, the change in relative flexibilities of the members and connection elements resulted in a shift in the basic behavior of the assembly and initiation of a failure mode that was not observed in the specimens with larger member sizes. In order to minimize the possibility of such occurrences, when extrapolation of test results is performed, it should be done with a basic understanding of the behavior of the assembly, and the likely effects of changes to the assembly configuration on this behavior. Test results should not be extrapolated to assembly configurations that are expected to behave differently than the tested configuration. Extrapolation or interpolation of results with differences in welding procedures, details or material properties is even 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~~ Table 7.5.1-1 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~~Table 7.5.1-1 - 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 5
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 5
A992[2]	Use same values as ASTM A572		
A913-50			
Axial, Flexural	50	58[1]	65 min.
Through-thickness	-	-	Note 5
A913-65			
Axial, Flexural	65	~~75[1]~~(4)	80 min.
Through-thickness			Note 5

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)
4. ASTM A913, Grade 65 material is not recommended for use in the beams of moment resisting frames
5. See Commentary

Commentary: Table7.5.1-1 Note 2 - The ASTM A992 specification was specifically developed by the steel industry in response to expressed concerns of the design community with regard to the permissible variation in chemistry and mechanical properties of structural steel under the A36 and A572 specifications. This new specification, which was adopted in late 1998, is very similar to ASTM A572, except that it includes somewhat more restrictive limits on chemistry and on the permissible variation in yield and ultimate tensile stress, as well as the ratio of yield to tensile strength. At this time, no statistical data base is available to estimate the actual distribution of properties of material produced to this specification. However, the properties are likely to be very similar, albeit with less statistical scatter, to those of material recently produced under ASTM A572, Grade 50.

Table 7.5.1-1 Note 5 - In the period immediately following the Northridge earthquake, the Seismology Committee of the Structural Engineers Association of California and the International Conference of Building Officials issued Interim Recommendation No. 2 (SEAOC-1995) to provide guidance on the design of moment resisting steel frame connections. Interim Recommendation No. 2 included a recommendation that the through-thickness stress demand on column flanges be limited to a value of 40 ksi, applied to the projected area of beam flange attachment. This value was selected somewhat arbitrarily, to ensure that through-thickness yielding did not initiate in the column flanges of moment-resisting connections and 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), rather than being the result of a demonstrated through-thickness capacity of typical column flange material. Despite the somewhat arbitrary nature of the selection of this value, its use often controls the overall design of a connection assembly including the selection of cover plate thickness, haunch depth, and similar parameters.

It would seem to be important to prevent the inelastic behavior of connections from being controlled by through-thickness yielding of the column flanges. This is because it would be necessary to develop very large local ductilities in the column flange material in order to accommodate even modest plastic rotation demands on the assembly. However, extensive investigation of the through-thickness behavior of column flanges in a "T" joint configuration reveals that neither yielding, nor through-thickness failure are likely to occur in these connections. Barsom and Korvink (1997) conducted a statistical survey of available data on the tensile strength of rolled shape material in the through-thickness direction. These tests were generally conducted on small diameter coupons, extracted from flange material of heavy shapes. The data indicates that both the yield stress and ultimate tensile strength of this material in the through-thickness direction is comparable to that of the material in the direction parallel to rolling. However, it does indicate somewhat greater scatter, with a number of reported values where the through-thickness strength was higher, as well as lower than that in the longitudinal direction. Review of this data indicates with high confidence that for small diameter coupons, the yield and ultimate tensile values of the material in a through-thickness direction will exceed 90% and 80% respectively of the comparable values in the longitudinal direction. ~~the actual~~ 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; stress concentrations induced by the presence of backing bars and~~

~~defects at the root of beam flange to column flange welds; 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. Such research is currently being conducted under the SAC phase II program.~~

While this statistical distribution suggests the likelihood that the through-thickness strength of column flanges could be less than the flexural strength of attached beam elements, testing of more than 40 specimens at Lehigh University indicates that this is not the case. In these tests, high strength plates, representing beam flanges and having a yield strength of 100 ksi were welded to the face of A572, Grade 50 and A913, Grade 50 and 65 column shapes, to simulate the portion of a beam-column assembly at the beam flange. These specimens were placed in a universal testing machine and loaded to produce high through-thickness tensile stresses in the column flange material. The tests simulated a wide range of conditions, representing different weld metals as well and also to include eccentrically applied loading. In 40 of 41 specimens tested, the assembly strength was limited by tensile failure of the high strength beam flange plate as opposed to the column flange material. In the one failure that occurred within the column flange material, fracture initiated at the root of a low-toughness weld, at root defects that were intentionally introduced to initiate such a fracture.

The behavior illustrated by this test series is consistent with mechanics of materials theory. At the joint of the beam flange to column flange, the material is very highly restrained. As a result of this, both the yield strength and ultimate tensile strength of the material in this region is significantly elevated. Under these conditions, failure is unlikely to occur unless a large flaw is present that can lead to unstable crack propagation and brittle fracture. In light of this evidence, Interim Guidelines Advisory No. 2 deletes any requirement for evaluation of through-thickness flange stress in columns.

~~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 as well as larger weldments with potential for enlarged heat affected zones, higher residual stresses and conditions of restraint.*~~

~~*Since the initial publication of the Interim Guidelines, a significant number of tests have been performed on reduced beam section connections (See section 7.5.3), most of which employed beam flanges which were welded directly to the column flanges using improved welding techniques, but without reinforcement plates. No through-thickness failures occurred in these tests despite the fact that calculated through-thickness stresses at the root of the beam flange to column flange joint ranged as high as 58 ksi. The successful performance of these welded joints is most probably due to the shifting of the yield area of the assembly away from the column flange and into the beam span. Based on the indications of the above described tests, and noting the undesirability of over reinforcing connections, it is now suggested that a maximum through-thickness stress of*~~ ~~$0.9F_{yc}$~~ ~~*may be appropriate for use with connections that shift the hinging away from the column face. Notwithstanding this recommendation, engineers are still cautioned to carefully consider the through-thickness issue when these other previously listed conditions which are thought to be involved in this type of failure are prevalent. 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.*~~

~~*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 - Strengthened Connections

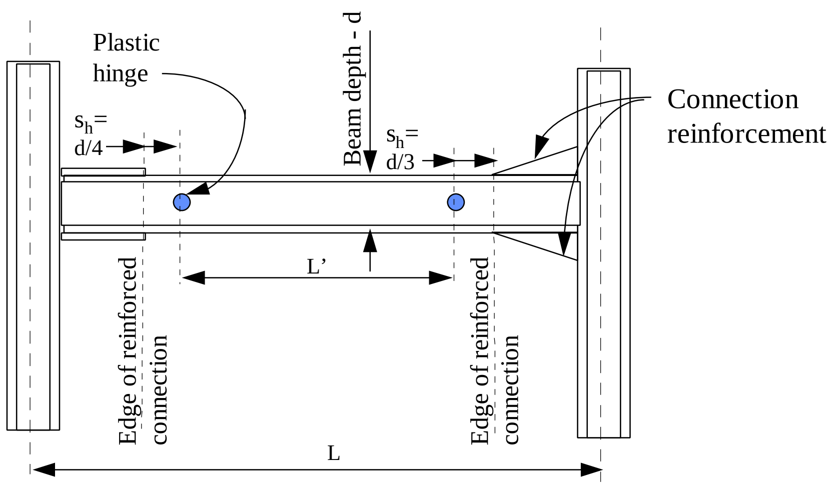
The following procedure may be followed to size the various elements of strengthened connection assemblies that are intended to promote formation of plastic hinges within the beam span by providing a reinforced beam section at the face of the column. Section 7.5.3 provides a modified procedure recommended for use in the design of connection assemblies using reduced beam sections to promote similar inelastic behavior. Begin by selecting ~~Select~~ a connection configuration, such as one of those indicated in Sections 7.9.1, 7.9.2, 7.9.3, 7.9.4, or 7.9.5, 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. Then proceed as described in the following sections. ~~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 location of the plastic hinge may be assumed to occur as indicated in Table 7.5.2.1-1 ~~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 location ~~value~~ is more appropriate. Refer to ~~Figure 7-2~~Figure 7.5.2.1-1.

Table 7.5.2.1-1 Plastic Hinge Location - Strengthened Connections

Connection Type	Reference Section	Hinge Location "s_h"
Cover plates	Sect. 7.9.1	d/4 beyond end of cover plates
Haunches	Sect. 7.9.3, 7.9.4	d/3 beyond toe of haunch
Vertical Ribs	Sect. 7.9.2	d/3 beyond toe of ribs



~~Figure 7-2~~ Figure 7.5.2.1-1 - Location of Plastic Hinge

Commentary: The suggested locations for the plastic hinge~~, at a distance d/3 away from the end of the reinforced section (or beginning of reduced section)~~ indicated in Table 7.5.2.1-1 and Figure 7.5.2.1-1 are ~~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, unless tributary areas for gravity loads are large.*

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:

$$M_{pr} = \beta M_p = \beta Z_b F_y \qquad (7.5.2.2\text{-}1\cancel{2})$$

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. A value of 1.2 should be taken for β for ASTM A572, A992 and A913 steels. ~~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 β, the expected yield strength, strain hardening and an appropriate uncertainty factor need to be determined. The following assumed strengths are recommended:

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

$$F_{ye} = 0.95\, F_{ym} \qquad (7.5.2.2\text{-}2\cancel{\text{-}3})$$

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.

$F_{y\,m}$ for various steels are as shown in ~~Table 7-1~~ Table 7.5.1-1, 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. Refer to the commentary to Section 8.1.3 for further discussion of steel strength issues.

Strain Hardening: 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.

Modeling Uncertainty: ~~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.~~ When the Interim Guidelines were first published, the b coefficient included a 1.2 factor to account for modeling uncertainty when connection designs were based on calculations as opposed to a specific program of qualification testing. The intent of this factor was twofold: to provide additional conservatism in the design when specific test data for a representative connection was not available and also as an inducement to encourage projects to undertake connection qualification testing programs. After the Interim Guidelines had been in use for some time, it became apparent that this approach was not an effective inducement for projects to perform qualification testing, and also that the use of an overly large value for the b coefficient often resulted in excessively large connection reinforcing elements (cover plates, e.g.) and other design features that did not appear conducive to good connection behavior. Consequently, it was decided to remove this modeling uncertainty factor from the calculation of b.

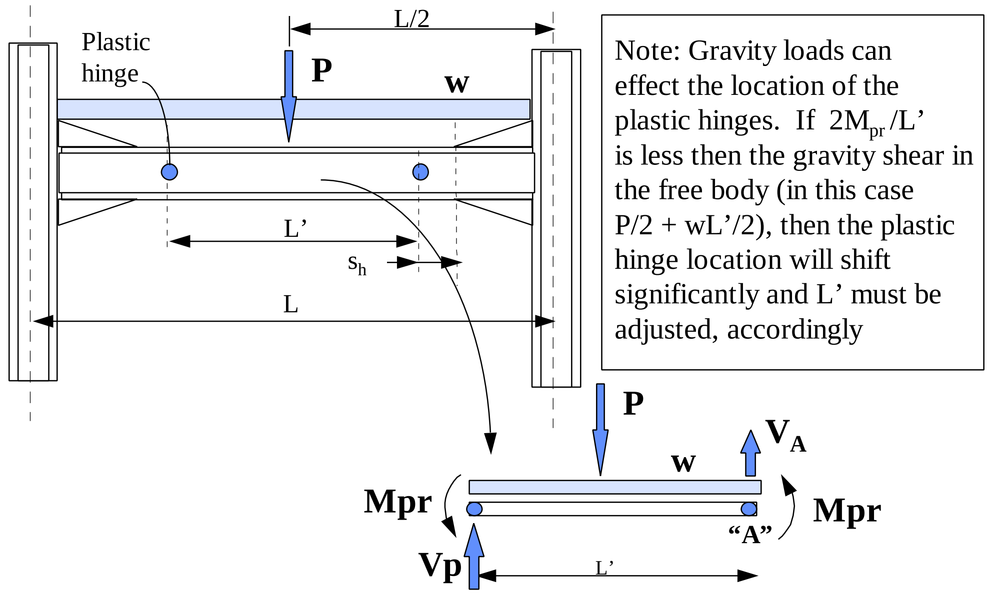
In summary, for Grade 50 steel, we have:

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

b = [0.95 (54 ksi to 59 ksi)/50 ksi] (1.1) = 1.13 to 1.21, say 1.2

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~~ Figure 7.5.2.3-1 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.



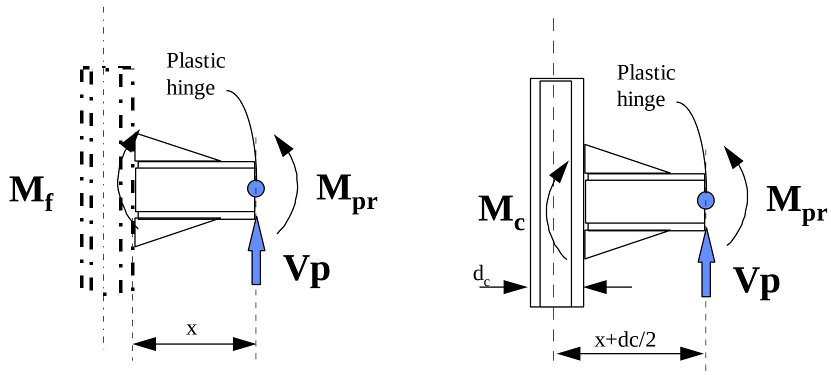
taking the sum of moments about "A" = 0
$V_p = \{M_{pr} + M_{pr} + P\ L'/2 + wL'^2/2\}/L'$

~~Figure 7-3~~ Figure 7.5.2.3-1- 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 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~~ Figure 7.5.2.4-1 demonstrates this procedure for two critical sections, for the beam shown in ~~Figure 7-3~~ Figure 7.5.2.3-1.



$M_f = M_{pr} + V_p x$ $\quad$ $M_c = M_{pr} + V_p(x + d_c/2)$

Critical Section at Column Face $\quad$ Critical Section at Column Centerline

~~Figure 7-4~~ Figure 7.5.2.4-1 - 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

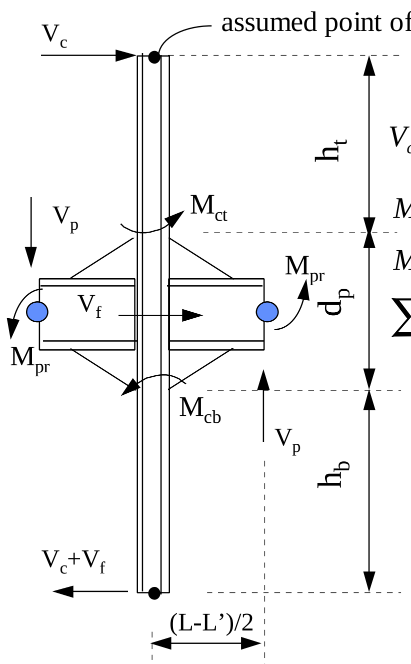
When required by the building code, the connection assembly should be checked to determine if strong column - weak beam conditions are satisfied. In lieu of *UBC-94* equation 11-3.1 {NEHRP-91 equation 10-3}, the following equation should be used:

$$\sum Z_c (F_{yc} - f_a) / \sum M_c > 1.0 \qquad (7.5.2.5\text{-}1\text{-}4)$$

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
ΣM_c is the ~~moment calculated at the center of t~~he column in accordance with ~~Section 7.5.2.4~~ sum of the column moments at the top and bottom of the panel zone, respectively, resulting from the development of the probable beam plastic moments, M_{pr}, within each beam in the connection.

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, that the beams will yield at the face of the column, and that the depth of the columns and beams are small relative to their respective span lengths. This permits the code to use a relatively simple equation to evaluate strong column - weak beam conditions in which the sum of the flexural capacities of columns at a connection are compared against the sums of the flexural capacities in the beams. -The first publication of the Interim Guidelines took this same approach, except that the definition of SM_c was modified to explicitly recognize that because flexural hinging of the beams would occur at a location removed from the face of the column, the moments delivered by the beams to the connection would be larger than the plastic moment strength of the beam. In this equation, SM_c was taken as the sum of the moments at the center of the column, calculated in accordance with the procedures of Sect. 7.5.2.4.



$$V_c = \frac{\sum\left[M_{pr} + V_p(L - L')/2)\right] - V_f\left(h_b + d_p/2\right)}{h_b + d_p + h_t}$$

$$M_{ct} = V_c h_t$$

$$M_{cb} = \left(V_c + V_f\right)h_b$$

$$\sum M_c = M_{ct} + M_{cb}$$

Note:
The quantities M_{pr}, V_p, L, and L' are as previously identified.
V_f is the incremental shear distributed to the column at the floor level.
Other quantities are as shown.

Figure 7.5.2.5-1 Calculation of Column Moment for Strong Column Evaluation

This simplified approach is not always appropriate. If non-symmetrical connection configurations are used, such as a haunch on only the bottom side of the beam, this can result in an uneven distribution of stiffness between the two column segments, and premature yielding of the column, either above, or below, the beam-column connection. Also, it was determined that for connection configurations in which the panel zone depth represents a significant fraction of *the total column height, such as can occur in some haunched and side-plated connections, the definition of SM_c contained in the initial printing of the Guidelines could lead to excessive conservatism in determining whether or not a strong column - weak beam condition exists in a structure. Consequently, Interim Guidelines Advisory No. 1 adopted the current definition of SM_c for use in this evaluation. This definition requires that the moments in the column, at the top and bottom of the panel zone be determined for the condition when a plastic hinge has formed at all beams in the connection. Figure 7.5.2.5-1 illustrates a method for estimating this quantity.*

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 $0.8\Sigma M_s$ 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. In addition, it is recommended that the alternative design criteria indicated in *UBC-94* Section 2211.7.2.1 (*NEHRP-91* Sect. 10.10.3.1), permitting panel zone shear strength to be proportioned for the shear induced by bending moments from gravity loads plus 1.85 times the prescribed seismic forces, not be used. For convenience of reference, UBC-94 Section 2211.7.2.1 is reproduced below, edited, to indicate the recommended application.

2211.7.2.1 Strength (edited)**.** 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 ~~$0.8\Sigma M_s$~~ $0.8\Sigma M_f$ of the girders framing into the column flanges at the joint. The joint panel zone shear strength may be obtained from the following formula:

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

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

Commentary: The effect of panel zone shear yielding on connection behavior is not well understood. In the past, panel zone shear yielding has been viewed as a benign, or even beneficial mechanism that permits overall frame ductility demands to be accommodated while minimizing the extent of inelastic behavior required of the beam and beam flange to column flange joint. The criteria *permitting panel zone shear strength to be proportioned for the shears resulting from moments due to gravity loads plus 1.85 times the design seismic forces was adopted by the code specifically to permit designs with somewhat weak panel zones. However, during recent testing of large scale connection assemblies with weak panel zones, it has been noted that in order to accommodate the large shear deformations that occur in the panel zone, extreme "kinking" deformations were induced into the column flanges at the beam flange to column flange welded joint. While this did not lead to premature joint failure in all cases, it is believed to have contributed to such premature failures in at least some of the specimens. The recommendations of this section are intended to result in stronger panel zones than previously permitted by the code, thereby avoiding potential failures due to this kinking action on the column flanges.*

7.5.3 Design Procedure - Reduced Beam Section Connections

The following procedure may be followed to size the various elements of reduced beam section (RBS) assemblies with circular curved reductions in beam flanges, such as shown in Figure 7.5.3-1.~~, such as those indicated in~~ Section 7.9.6 indicates other configurations for such connections, however, the circular curved configuration shown in Figure 7.5.3-1 is currently preferred. RBS assemblies are intended to promote the formation of plastic hinges within the beam span by developing a segment of the beam with locally reduced section properties and strength. ~~Begin by selecting an RBS configuration, such as one of those indicated in Figure 7.5.3-1, that will permit the formation of a plastic hinge within the reduced section of the beam. Of the configurations shown in the figure, the circular curved configuration is preferred.~~

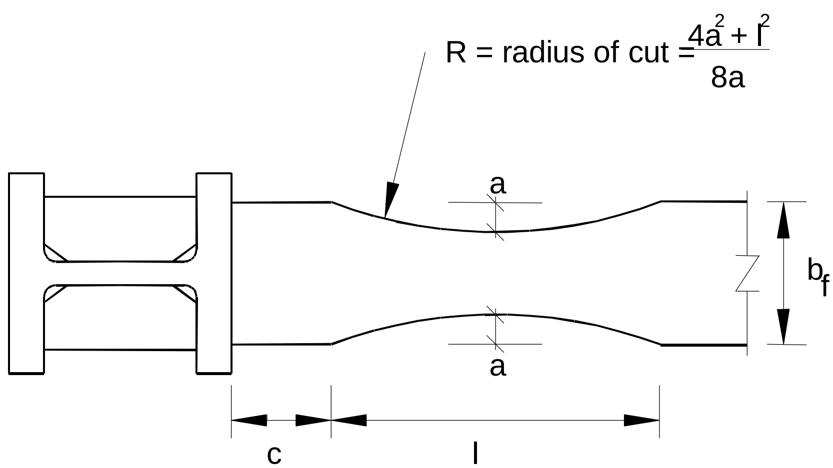


Figure 7.5.3-1 Geometry of Reduced Beam Section

Commentary: Connection assemblies in which inelastic behavior is shifted away from the column face through development of a segment of the beam with intentionally reduced properties, so-called reduced beam section (RBS) or "dogbone" connections, appear to have the potential to provide an economical *solution to the WMSF connection problem. These recommendations are based on ~~limited~~ design configurations that have successfully been tested~~ing that has been conducted of these types of connections~~ to date. ~~While a~~ A large number of RBS tests have been conducted, ~~these tests have not included the effects of floor slabs or loading rates approximating those that would be produced by a building's response to earthquake ground motions~~including some tests of assemblies with floor slabs present. ~~Extensive additional testing of RBS connections, intended to explore these and other factors relevant to connection performance, are currently planned under funding provided by NIST and the SAC phase II program. In the interim, designers specifying RBS connections may wish to consider provision of details to minimize the participation of the slab in the flexural behavior of the beam at the reduced section. The criteria presented in this section are partially based on a draft procedure developed by AISC (Iwankiw, 1996).~~*

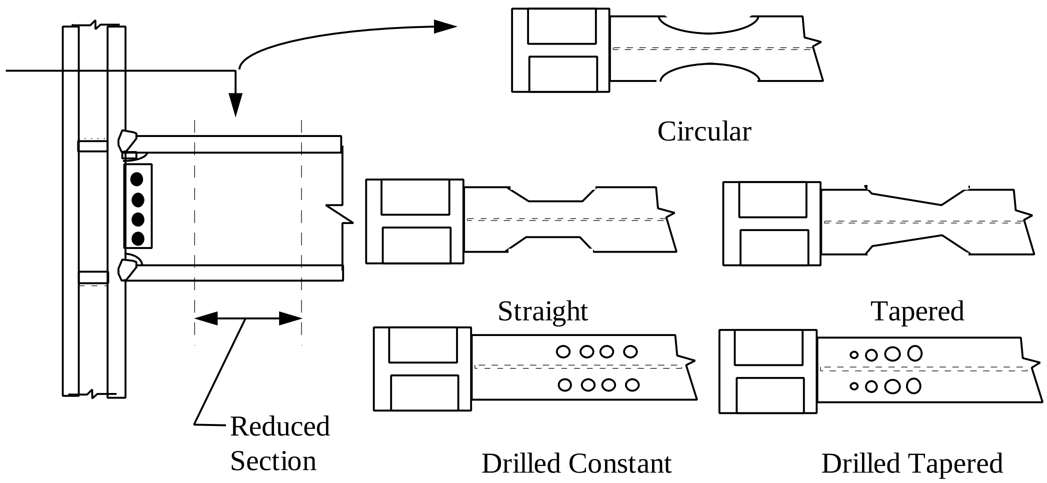


Figure 7.5.3-2 Alternative Reduced-Beam Section Patterns
~~Figure 7.5.3-1 Reduced Beam Section Patterns~~

Several alternative configurations of RBS connections have also been tested to date. As indicated in Figures 7.5.3-2~~1~~ and 7.9.6-1, these include constant section, tapered section, curved section, and drilled hole patterns. It appears that several of these configurations are more desirable than others. In particular, the drilled hole section patterns have been subject to tensile failure across the reduced net section of the flange through the drill holes. A few RBS tests utilizing straight or tapered cuts have failed within the reduced section at plastic rotation demands less than recommended by these Guidelines. In all of these cases, the failure occurred at locations at which there was a change in direction of the cuts in the beam flange, resulting in a geometric stress riser or notch effect. It is also reported that one of these tests failed at the beam flange continuity plate - to column flange joint. There have been no reported failures of RBS connection *assemblies employing the circular curved flange cuts, and therefore, this is the pattern recommended in these Guidelines. ~~This would appear, therefore, to be a more desirable configuration, although some successful tests have been performed using the straight and tapered configurations.~~*

It is important that the pattern of any cuts made in the flange be proportioned so as to avoid sharp cut corners. All corners should be rounded to minimize notch effects and in addition, cut edges should be cut or ground ~~in the direction of the flange length~~ to have a surface roughness meeting the requirements of AWS C4.1-77 class 4, or smoother~~roughness value less than or equal to 1,000, as defined in ANSI/ASME B46.1~~.

Concerns have been raised by some engineers over the strength reduction inherent in the RBS. Clearly, code requirements for strength, considering gravity loads and gravity loads in combination with wind, seismic and other loads must be met. For higher seismic zones, beam sizes are typically governed by elastic stiffness considerations (drift control) and this must be addressed. Also, for seismic loads, the Building Codes typically require that connections for Special Moment Resisting Frames must develop the "strength" or the "plastic bending moment" of the beam. There may be a problem of semantics where these requirements are applied to a system using RBS connections. Is the RBS part of the connection or is it part of the beam, the strength of which must be developed by the connection? Clearly, the latter interpretation should be applied.

Notwithstanding the above, it must be kept in mind that, although unstated, and typically not quantified, there is inherent in design practice an implied relationship between the elastic behavior that we analyze and the inelastic behavior which the building is expected to experience. Elastic drift limitations commonly used are considered to be related to the anticipated inelastic drifts and ultimate lateral stability of the framed structure in at least an intuitively predictable manner. It can be shown that RBS's such as those that have been tested will reduce the elastic stiffness (increase the drift) on the order of 5%. However, because of the reduction in strength, the effect on the inelastic drift may be more significant. Thus, it seems prudent to require that the RBS maintain a reasonably high proportion of the frame inelastic strength. For the connections tested to date, the inelastic strength of the RBS section has been in the range of 70% of that of the full section. However, the moment demand at the face of the column, corresponding to development of this reduced section strength, is likely to be in the range of 85% to 90% of the strength of the full beam. This seems to be quite reasonably high considering the accuracy of other seismic design assumptions.

Although the use of RBS designs tends to reduce the total strength demand on the beam flange - to - column flange connection, relative to strengthened *connections, designs utilizing RBS configurations should continue to follow the recommendations for beam flange continuity plates, weld metal and base metal notch toughness recommended by the Interim Guidelines for strengthened connections.*

7.5.3.1 Determine Reduced Section and Plastic Hinge Locations

The reduced beam section should be located at a sufficient distance from the face of the column flange (dimension "*c*" in Figures 7.5.3-1 and 7.5.3.1-1) to avoid significant inelastic behavior of the material at the beam flange - to - column flange joint. Based on testing performed to date, it appears that a value of "*c*" on the order of ½ to ¾ of the beam width, b_f, is sufficient. ~~*d*/4 (where "d" is the beam depth) is sufficient~~. The total length of the reduced section of beam flange (dimension "*l*" in Figures 7.5.3-1 and 7.5.3.1-1) should be on the order of 0.65*d* to 0.85*d*, where *d* is the beam depth. ~~3*d*/4 to *d*~~. The location of the plastic hinge, s_h,, may be taken as ½ the length of the cut-out, *l*. ~~indicated in Table 7.5.3.1-1, unless test data indicates a more appropriate value should be used. When tapered configurations are utilized, the slope of the tapered cut in the beam flange should be arranged such that the variation of the plastic section modulus, Z_x, within the reduced section approximates the moment gradient in the beam during the condition when plastic hinges have formed within the reduced beam sections at both ends.~~

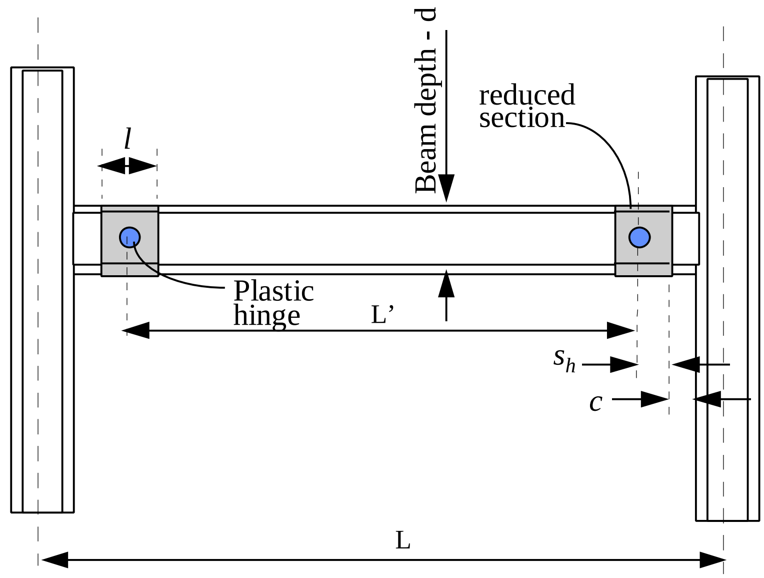


Figure 7.5.3.1-1 Critical Dimensions - RBS Assemblies

7.5.3.2 Determine Strength and Probable Plastic Moment in RBS

The RBS may be proportioned to meet the following criteria:

1. The section at the RBS should be sufficient to satisfy the strength criteria specified by the building code for Dead, Live, Seismic, Snow, Wind, and other applicable design forces.

2. The elastic stiffness of the frame, considering the effects of the RBS, should be sufficient to meet the drift requirements specified by the code, under the design seismic and other forces.

3. The expected stress in the beam flange - to - column flange weld, under the application of gravity forces and that seismic force that results in development of the probable plastic moment of the reduced section at both ends of the beam, should be less than or equal to the strength of the weld, as indicated in Section 7.2.2 of the Interim Guidelines.

4. ~~The expected through-thickness stress on the face of the column flange, calculated as M_f/S_e, under the application of gravity forces and that seismic force that results in development of the probable plastic moment of the reduced section at both ends of the beam, should be less than or equal to the values indicated in Section 7.5.1, where M_f is the moment at the face of the column flange, calculated as indicated in Section 7.5.2.4, and S_e is the elastic section modulus of the beam at the connection considering weld reinforcement, bolt holes, reinforcing plates, etc.~~ The maximum moment at the face of the column should be in the range of 85 percent to 100 percent of the beam's expected plastic moment capacity. The depth of cut-out, *a*, should be selected to be less than or equal to $b_f/4$.

The plastic section modulus of the RBS may be calculated from the equation:

$$Z_{RBS} = Z_x - b_R t_f \left(d - t_f\right) \tag{7.5.3.2-1}$$

where:

- Z_{RBS} is the plastic section modulus of the reduced beam section
- Z_x is the plastic modulus of the unreduced section
- b_R is the total width of material removed from the beam flange
- t_f is the thickness of the beam flange
- d is the depth of the beam

The probable plastic moment, M_{pr}, at the RBS shall be calculated from the equation:

$$M_{pr} = Z_{RBS} b F_y \tag{7.5.3.2-2}$$

where:

- Z_{RBS} is the plastic section modulus of the reduced beam section
- β is as defined in Section 7.5.2.2

The strength demand on the beam flange - to - column flange weld and on the face of the column may be determined by following the procedures of Section .7.5.2.3 and 7.5.2.4 of the Interim Guidelines, using the value of M_{pr} determined in accordance with Eq. 7.5.3.2-2.

> *Commentary: Initial design procedures for RBS connections published by SAC recommended that sufficient reduction of the beam flange be made to maintain flexural stresses in the beam, at the column face, below the anticipated through-thickness yield strength of the column flange material. Since the publication of those recommendations, extensive testing of RBS connections has been conducted, both with and without composite slabs. ~~The testing conducted to date on RBS specimens~~ This testing has typically been for configurations that would result in somewhat larger strength demands at the face of the column flange than suggested by the criteria originally published by SAC. ~~contained in this Advisory.~~ Typically, the tested specimens had reductions in the beam flange area on the order of 35% to 45% and produced moments at the face of the column that resulted in stresses on the weld and column as large as large as 90 to 100% of the expected material strength of the beam, which is often somewhat in excess of the through-thickness yield strength of the column material. The specimens in these tests all developed acceptable levels of inelastic deformation. Recent studies conducted for SAC at Lehigh University confirm that the significant conditions of restraint that exist at the beam flange to column flange joint results in substantially elevated column through-thickness strength, negating a need to reduce flexural stresses below the anticipated column yield strength. In view of this evidence, SAC has elected to adopt design recommendations consistent with configurations that were successfully tested. ~~The criteria contained in this Advisory suggest that these demands be reduced to a level which would maintain weld stresses within their normally specified values and through-thickness column flange stresses at the same levels recommended for strengthened connections. This may require the beam flanges to be reduced by as much as 50% or more for some frame configurations, or that supplemental reinforcement such as cover plates or vertical ribs be provided in addition to the reduced section. This approach was taken to maintain consistency with the criteria recommended for strengthened connections and with the knowledge that the factors affecting the performance of these connections are not yet fully understood.~~*

7.5.3.3 Strong Column - Weak Beam Condition

The adequacy of the design to meet strong column - weak beam conditions should be checked in accordance with the procedures of Section 7.5.2.5

7.5.3.4 Column Panel Zone

The adequacy of the column panel zone should be checked in accordance with the procedures of Section 7.5.2.6.

7.5.3.5 Lateral Bracing

The reduced section of the beam flanges should be provided with adequate lateral support to prevent lateral-torsional buckling of the section. Lateral braces should be located within a distance equal to 1/2 the beam depth from the expected location of plastic hinging, but should not be located within the reduced section of the flanges.

Commentary: Unbraced compression flanges of beams are subject to lateral-torsional buckling, when subjected to large flexural stresses , such as occur in the plastic hinges of beams ~~reduced sections of RBS connections~~ during response to strong ground motion. To prevent ~~such behavior~~ lateral-torsional buckling, it is recommended that both flanges of beams be provided with lateral support. Section 9.8 of the 1997 AISC Seismic Specification requires such bracing in general, and specifically states as follows:

> *"Both flanges of beams shall be laterally supported directly or indirectly. The unbraced length between lateral supports shall not exceed $2500r_y/F_y$. In addition, lateral supports shall be placed near concentrated forces, changes in cross section and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the SMF."*

Adequate lateral support of the top flanges of beams supporting concrete filled metal deck or formed slabs can usually be obtained through the normal welded attachments of the deck to the beam or through shear studs. Lateral support of beam flanges can also be provided through the connections of transverse framing members or by provision of special lateral braces, attached directly to the flanges. Such attachments should not be made within the reduced section of the beam flange as the welding or bolting required to make such attachments can lead to premature fracturing in these regions of high plastic demands.

For beams in moment-resisting frames, it has traditionally been assumed that the direct attachment of the beam flanges to the columns provided sufficient lateral support of both beam flanges to accommodate the plastic hinges anticipated to develop in these frames at the beam-column connection. However, connection configurations like the RBS, developed following the Northridge earthquake, are intended to promote formation of these plastic hinges at some distance from the beam-column interface. This brings to question the adequacy of the beam flange to column flange attachments to provide the necessary lateral *support at the plastic hinge. While this issue is pertinent for any connection configuration that promotes plastic hinge formation remote from the beam-column interface, RBS connections could be more susceptible to lateral-torsion buckling at the plastic hinge because the reductions in the beam flange used to achieve plastic hinge formation also locally reduce the torsional resistance of the section. For that reason, FEMA-267a recommended provision of lateral bracing adjacent to the reduced beam section.*

Provision of lateral bracing does result in some additional cost. Therefore, SAC has engaged in specific investigations to evaluate the effect of lateral bracing both on the hysteretic behavior of individual connections as well as overall frame response to large lateral displacements. Until these investigations have concluded SAC continues to recommend provision of lateral bracing for RBS connections. It should be noted that Section 9.8 of the 1997 AISC Seismic Specification states:

> *"If members with Reduced Beam Sections, tested in accordance with Appendix S are used, the placement of lateral support for the member shall be consistent with that used in the tests."*

Most testing of RBS specimens performed as part of the SAC project have consisted of single beams cantilevered off a column to simulate the exterior connection in a multi-bay moment-resisting frame. The beams have generally been braced at the end of the cantilever length, typically located about 100 inches from the face of the column. For the ASTM A572, Grade 50, W36x150 sections typically tested, this results in a nominal length between lateral supports that is comparable to $2500r_y/F_y$.

The appropriate design strength for lateral bracing of compression elements has long been a matter of debate. Most engineers have applied "rules of thumb" that suggest that the bracing element should be able to resist a small portion, perhaps on the order of 2% to 6% of the compressive force in the element being braced, applied normal to the line of action of the compression. A recent successful test of an RBS specimen conducted at the University of Texas at Austin incorporated lateral bracing with a strength equal to 6% of the nominal compressive yield force in the reduced section.

7.5.3.6 Welded Attachments

Headed studs for composite floor construction should not be placed on the beam flange between the face of the column and the extreme end of the RBS, as indicated in Figure 7.5.3.6-1. Other welded attachments should also be excluded from these regions of the beam.

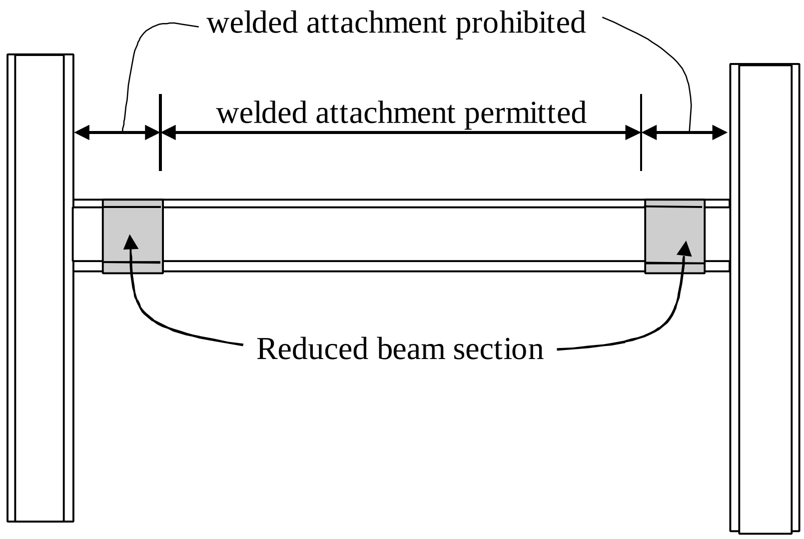


Figure 7.5.3.6-1 Welded Attachments to RBS Beams

Commentary: There are two basic reasons for omitting headed studs in the region between the reduced beam section and the column. The first of these is that composite action of the slab and beam can effectively counteract the reduction in beam section properties achieved by the cutouts in the top beam flange. By omitting shear studs in the end region of the beam, this composite behavior is neutralized, protecting the effectiveness of the section reduction. The second reason is that the portion of the beam at the reduced section is expected to experience large cyclic inelastic strains. If welded attachments are made to the beam in this region, the potential for low-cycle fatigue of the beam, under these large cyclic inelastic strains is greatly increased. For this same reason, other welded attachments should also be excluded from this region.

7.6 Metallurgy and Welding

There are no modifications to the Guidelines or Commentary of Section 7.6 at this time.

7.7 Quality Control/Quality Assurance

There are no modifications to the Guidelines or Commentary of Section 7.7 at this time.

7.8 Guidelines on Other Connection Design Issues

There are no modifications to the Guidelines or Commentary of Section 7.8 at this time.

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. As indicated in Section 7.5.2.6, it is recommended that the formulation for panel zone demand contained in the UBC, based on 1.85 times the prescribed seismic forces, not be utilized. This formulation, which is not contained in either the AISC Seismic Provisions or the NEHRP Provisions, is felt to lead to the design of panel zones that are excessively flexible and weak in shear. 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. For connection designs based on tested configurations, the web connection design should be consistent with the conditions in the tested assemblies.

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.

Some recent finite element studies of typical connections by Goel, Popov and others have suggested that even when the shear tab is welded, shear demands at the connections tend to be resisted by a diagonal tension type behavior in the web that tends to result in much of the shear being resisted by the flanges. Investigation of these effects is continuing.

7.8.3 Design of Continuity Plates

There are no modifications to the Guidelines or Commentary of Section 7.8.3 at this time.

7.8.4 Design of Weak Column and Weak Way Connections

There are no modifications to the Guidelines or Commentary of Section 7.8.4 at this time.

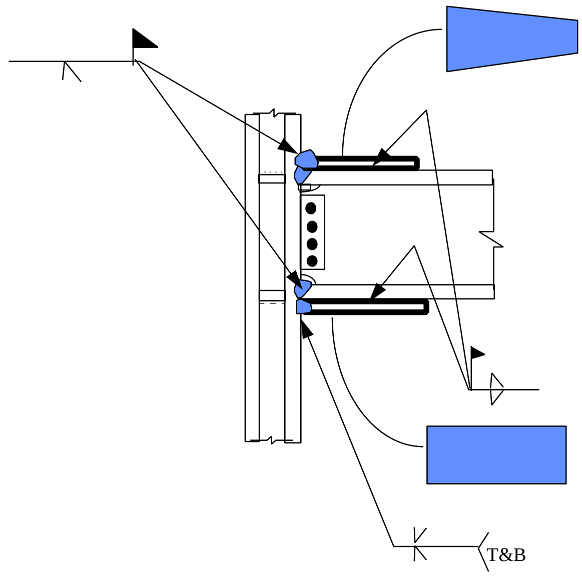
7.9 Moment Frame Connections for Consideration in New Construction

There are no modifications to the Guidelines or Commentary of Section 7.9 at this time.

7.9.1 Cover Plate Connections

~~Figure 7-5~~ Figure 7.9.1-1- 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.~~ More than 30 tests of such connections have been performed, with data on at least 18 of these tests available in the public domain.

A variation of this concept which has been tested successfully ~~very recently~~ (Forell/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~~ Figure 7.9.1-1 - Cover Plate Connection

Design Issues: Following the Northridge earthquake, the University of Texas at Austin conducted a program of research, under private funding, to develop a modified connection configuration for a specific project. Following a series of unsuccessful tests on various types of connections, approximately~~Approximately~~ eight connections similar to that shown in ~~Figure 7-5~~ Figure 7.9.1-1 were ~~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 was~~is~~ correctly executed and through-thickness problems in the column flange were ~~are~~ avoided. This configuration is relatively economical, compared to some other reinforced configurations, and has limited architectural impact. As a result of these factors, and the significant publicity that followed the first successful tests of these connections, cover plated connections quickly became the predominant configuration used in the design of new buildings. As a result, a number of qualification tests have now been performed on different variations of cover plated connections, covering a wide range of member sizes ranging from W16 to W36 beams, as part of the design process for individual building projects. The results of these tests have been somewhat mixed, with a significant number of failures reported. Although this connection type appears to be significantly more reliable than the typical pre-Northridge connection, it should be expected that some connections in buildings incorporating this detail may still be subjected to earthquake initiated fracture damage. Designers should consider using alternative connection types, unless highly redundant framing systems are employed.

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.

Quantitative Results: No. of specimens tested: 18
Girder Size: W21 x 68 to W36 x 150
Column Size: W12 x 106 to W14 x 455
Plastic Rotation achieved-
~~6~~ 13 Specimens : >0.025 radian
~~1~~ 3 Specimens: ~~0.015~~ $0.005 < \theta_p < 0.025$ radian
~~1~~ 2 Specimens: 0.005 radian

Although apparently more reliable than the former prescriptive connection, this configuration is ~~subject to some of the same flaws including dependence~~ dependent on properly executed beam flange to column flange weld~~s, 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.

One of the issues that must be faced by designers utilizing cover plated connections is the sequence of operations used to attach the cover plate and beam flange to the column. In one approach, the bottom cover plate is shop welded to the column, and then used as the backing for the weld of the beam bottom flange to the column flange. This approach has the advantage of providing an erection seat and also results in a somewhat reduced amount of field welding for this joint. A second approach is to attach the cover plate to the beam flange, and then weld it to the column, in the field, as an integral part of the beam flange. There are tradeoffs to both approaches. The latter approach results in a relatively large field weld at the bottom flange with large heat input required into the column and beam. If this operation is not performed with proper preheat and control of the heat input, it can potentially result in an enlarged and brittle heat affected zone in both members. The first approach results in reduced heat input and therefore, somewhat minimized potential for this effect. However, proper control of preheat and heat input remains as important in either case, as improper procedures can still result in brittle conditions in the heat affected zone. Further, the detail in which the cover plate is shop welded to the column can lead to a notch effect for the column flange at the seam between the beam *flange and cover plate. This is effect is illustrated in Figure 7.9.1-2. At least one specimen employing this detail developed a premature fracture across the column flange that has been related to this notch effect. This effect has been confirmed by recent fracture mechanics modeling of this condition conducted by Deierlein.*

When developing cover plated connection details, designers should attempt to minimize the total thickness of beam flange and cover plate, so as to reduce the size of the complete joint penetration weld of these combined elements to the column flange. For some frame configurations and member sizes, this combined thickness and the resulting CJP weld size can approach or even exceed the thickness of the column flange. While there is no specific criteria in the AWS or AISC specifications that would suggest such weldments should not be made, judgementally they would not appear to be desirable from either a constructability or performance perspective. As a rough guideline, it is recommended that for connections in which both the beam flange and cover plate are welded to the column flange, the combined thickness of these elements should not exceed twice the thickness of the beam flange nor 100% of the thickness of the column flange. For cover plated connections in which only the cover plate is welded to the column flange, the same thickness limits should be applied to the cover plate.

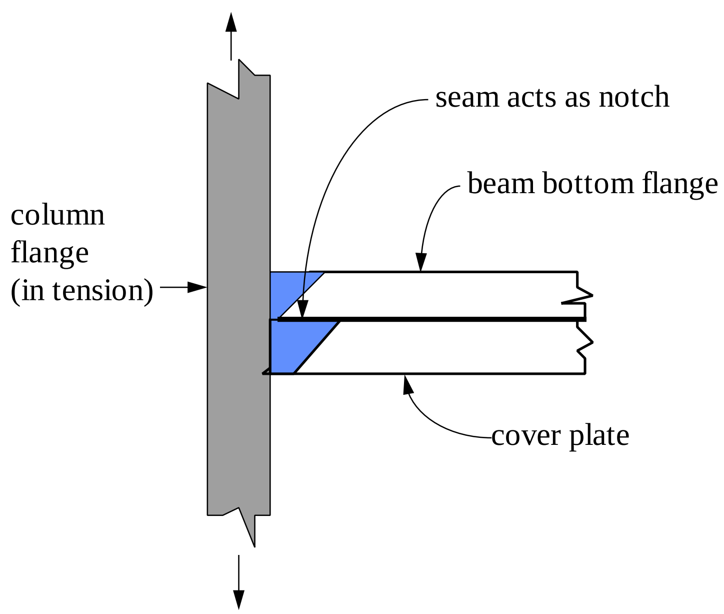


Figure 7.9.1-2 Notch Effect at Cover Plated Connections

7.9.2 Flange Rib Connections

There are no modifications to the Guidelines or Commentary of Section 7.9.2 at this time.

7.9.3 Bottom Haunch Connections

Figure 7.9.3-1 ~~7-7~~ indicates the configuration of a connection with a haunch at the bottom beam flange.~~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.~~ Several tests of this connection type were conducted by Uang under the SAC phase I project (Uang, 1995). Following that work, additional research on the feasibility of improving connection performance with welded haunches was conducted under a project that was jointly sponsored by NIST and AISC (*NIST, 1998*). That project was primarily focused on the problem of upgrading connections in existing buildings. As indicated in the report of that work, the haunched modification improves connection performance by altering the basic behavior of the connection. In essence, the haunch creates a prop type support, beneath the beam bottom flange. This both reduces the effective flexural stresses in the beam at the face of the support, and also greatly reduces the shear that must be transmitted to the column through the beam. A complete procedure for the design of this modification may be found in *NIST, 1998*.

~~**Figure 7-7 - Bottom Haunch Connection Modification**~~

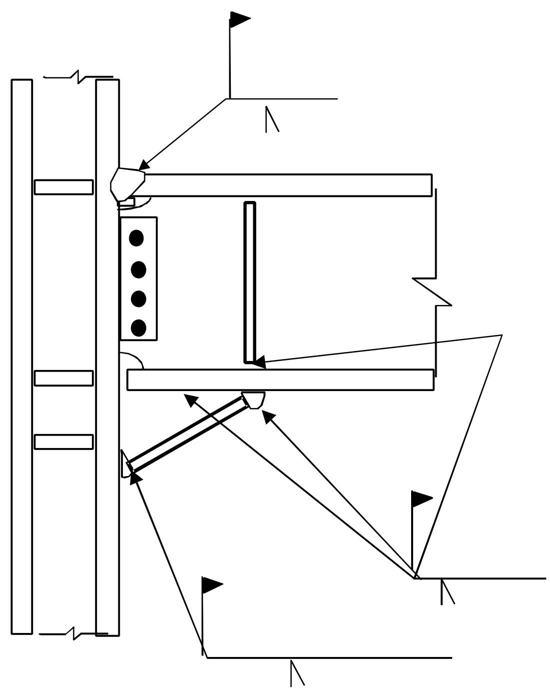


Figure 7.9.3-1 Bottom Haunch Connecction

~~Two~~ Nine tests are known to have been performed to date, ~~both successfully~~ all intended to replicate the condition of an existing connection that has been upgraded. Except for those specimens in which existing vulnerable welded joints were left in place at the top flange, these connections generally achieved large plastic rotations. Several dynamic tests have also been successfully conducted, although only moderate plastic deformation demands could be imposed due to limitations of the laboratory equipment. ~~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: 9~~2~~*

Girder Size: W30 x 99
Column Size: W14 x 176
Plastic Rotation achieved-
Specimen ~~1~~ UCSD-1R:0.04 radian (w/o bottom flange weld and reinforced top flange)
Specimen ~~2~~ UCSD-3R:0.05 radian (with bottom flange weld and reinforced top flange)
Specimen UCSD-4R: 0.014 radian (dynamic- limited by test setup)
Specimen UCSD-5R: 0.015 radian (dynamic- limited by test setup)

Girder Size: W36x150
Column Size: W14x257
Plastic Rotation achieved -
Specimen UCB-RN2: 0.014 radian (no modification of top weld)
Specimen UTA-1R: 0.019 radian (partial modification of top weld)
Specimen UTA-1RB: 0.028 radian (modified top weld)

Girder Size: W36x150
Column Size: W14x455
Plastic Rotation achieved-
Specimen UTA-NSF4: 0.015 radian (no modification of top weld)

Girder Size: W18x86
Column Size: W24x279
Plastic Rotation achieved-
Specimen SFCCC-8: 0.035 radian (cover plated 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

There are no modifications to the Guidelines or Commentary of Section 7.9.4 at this time.

7.9.5 Side-Plate Connections

There are no modifications to the Guidelines or Commentary of Section 7.9.5 at this time.

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~~ Figure 7.9.6-1 illustrates both concepts. The most successful configurations have used reduced sections formed with circular cuts. Configurations which taper the reduced section, through the use of unsymmetrical cut-outs, or variable size holes, to balance the cross section and the flexural demand have also been tested with success.

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.~~ has performed successful testing of 4 linearly tapered RBS connections. In the time since the first publication of the Interim Guidelines, a number of tests have been successfully conducted of RBS connections with circular curved cut-outs, including investigations ~~and~~ at the University of Texas at Austin, ~~has successfully tested 4 circular curved RBS specimens. Others, including Popov at~~ the

University of California at Berkeley, and Texas A&M University.~~, have also tested circular curved RBS connections with success.~~

When this connection type was first proposed, ~~There is~~ a concern was expressed 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 in~~to~~ compression. ~~It may be possible to mitigate this effect with proper detailing of the slab~~. Limited testing of RBS specimens with composite slabs has ~~recently~~ been successfully conducted at Ecole Polytechnic, in Montreal, Canada. In these tests, shear studs were omitted from the portion of the top flange having a reduced section, in order to minimize the influence of the slab on flexural hinging. In addition, a 1 inch wide gap was placed in the slab, around the column, to reduce the influence of the slab on the connection at the column face. More recently, both the University of Texas at Austin and Texas A&M University have conducted successful tests of RBS connections with slabs and without such gaps present between the slab and column. This most recent testing suggests that the presence of the slab actually enhances connection behavior by retarding buckling of the top flange in compression and delaying strength degradation effects commonly observed in specimens tested without slabs.

Design Issues: 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. However, the tendency for lateral-torsional buckling is significantly increased suggesting the need for lateral bracing of the beam flanges, near the reduced section.*~~

Experimental Results: A number of researchers have performed tests on RBS specimens to date. Most tests have utilized the ATC-24 loading protocol, which is similar to the protocol described in Section 7.4.1 of the Interim Guidelines. Testing employed at Ecole Polytechnic, in Montreal, Canada utilized a series of different testing protocols including the ATC-24 procedure and a dynamic excitation simulating the response of a connection in a building to an actual earthquake accelerogram (Tremblay, et. al., 1997). This research included two tests of connections with composite floor slabs. All of the reported tests with circular flange cuts have performed acceptably, however, the dynamic tests at Ecole Polytechnic only imposed 0.025 radians of plastic rotation on the assembly.

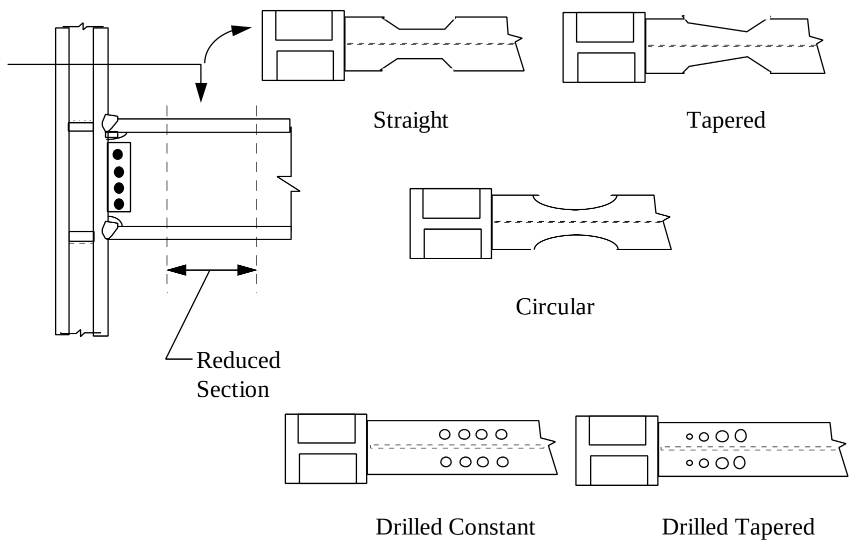


Figure ~~7-11~~ 7.9.6-1 - Reduced Beam Section Connection

Quantitative Results:

No. of specimens tested: 21~~9~~ published (without slabs)~~2~~
Girder Size: W21 x 62~~W30 x 99~~ thru W 36 x 194
Column Size: W14x120~~W14 x 176~~ thru W 14 x 426, W24 x 229
Plastic Rotation achieved:- ~~0.03 radian~~
Straight: - 0.02 radian
Tapered - 0.027 - 0.045 radian
Circular - 0.03 - 0.04 radian
No. of specimens tested: 4~~2published~~ (with slabs)
Girder Size: W21 x 44 to W36 x 150
Column Size: W14 x 90 to W14x257
Plastic Rotation achieved: 0.03-0.05 radians (ATC-24 loading protocol)
0.025 radians (earthquake simulation – limited by laboratory setup, no failure observed)

7.9.7 Slip - Friction Energy Dissipating Connection

There are no modifications to the Guidelines or Commentary of Section 7.9.7 at this time.

7.9.8 Column-Tree Connection

There are no modifications to the Guidelines or Commentary of Section 7.9.8 at this time.

7.9.9 Proprietary 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 and its resulting beam flange prying moment 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 the proprietary 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, and also, to promote local buckling of the beam flanges under compressive loads to limit the amount of demand on the beam flange to column flange weld. Claimed assets for this connection include elimination of the vertical beam shear in the beam flange welds, elimination of the beam lateral torsional buckling mode, and the participation of the beam web in resisting its portion of the beam moment. A number of different configurations for this connection type have been developed and tested. Figure 7.9.9-1~~7-14~~ indicates one such configuration for this connection type that has been successfully tested and which has been used in both new and retrofit steel moment-resisting frames. In this configuration, slots are cut into the beam web, extending from the weld access hole adjacent to the top and bottom flanges, and extending along the beam axis a sufficient length to alleviate the stress concentration effects at the beam flange to column flange weld. The beam web is welded to the column flange. ~~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.

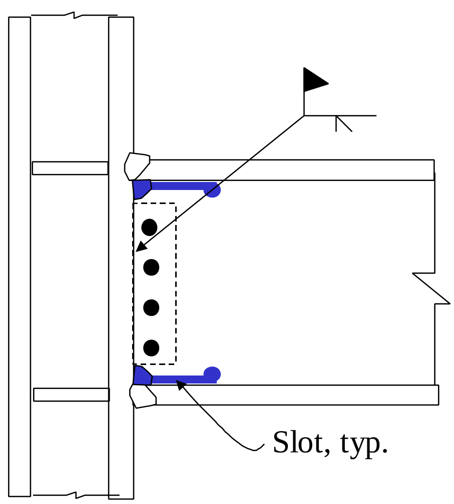


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Figure 7.9.9-1~~7-14~~ - Proprietary Slotted Web Connection

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~~ a number of connections employing details similar to that shown in Figure ~~7-14~~7.9.9-1 have recently been tested successfully ~~(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~~
Quantitative data on connection testing may be obtained from the licensor.

7.9.10 Bolted Bracket Connections

Framing connections employing bolted or riveted brackets have been used in structural steel construction since its inception. Early connections of this type were often quite flexible, and also had limited strength compared to the members they were connecting, resulting in partially restrained type framing. However, it is possible to construct heavy bolted brackets employing high strength bolts to develop fully restrained moment connections. Pretensioing of the bolts or threaded rods attaching the brackets to the column flanges and use of slip-critical connections between the brackets and beam flanges can help to provide the rigidity required to obtain fully restrained behavior. Reinforcement of the column flanges may be required to prevent local yielding and excessive deformation of these elements, as well. Two alternative configurations that have been tested recently are illustrated in Figure 7.9.10-1. The developer of these configurations offers the brackets in the form of proprietary steel castings. Several tests of these alternative connections have been performed on specimens with beams ranging in size from W16 to W36 sections and with large plastic rotations successfully achieved.

Design Issues: The concept of bolted bracket connections is similar to that of the riveted "wind connections" commonly installed in steel frame buildings in the early twentieth century. The primary difference is that the riveted wind connections were typically limited in strength either by flexural yielding of outstanding flanges of the brackets, or shear and tension on the rivets, rather than by flexural hinging of the connected framing. Since the old-style wind connections could not typically develop the flexural strength of the girders and also could be quite flexible, they would be classified either as partial strength or partially restrained connections. Following the Northridge earthquake, the concept of designing such connections with high strength bolts and heavy plates, to behave as fully restrained connections, was developed and tested by a private party who has applied for patents on the concept of using steel castings for this purpose.

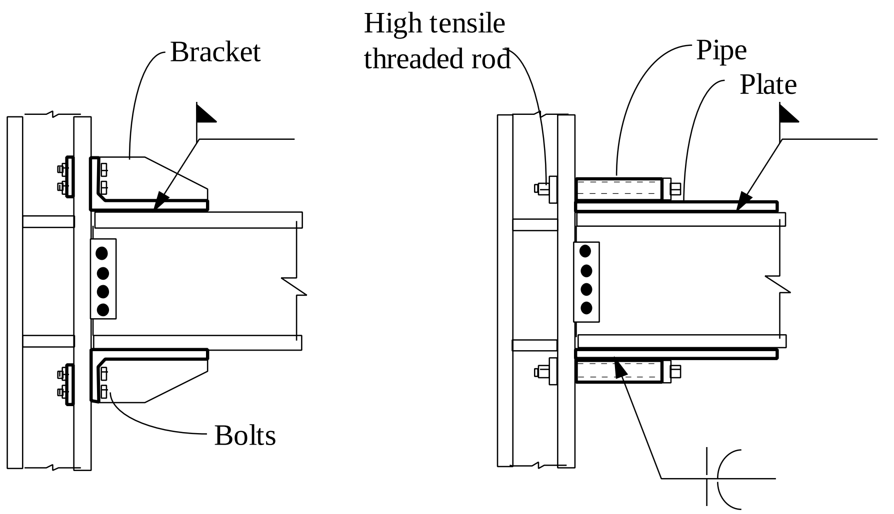

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Figure 7.9.10-1 Bolted Bracket Connections

Bolted connections offer a number of potential advantages over welded connections. Since no field welding is required for these connections, they are inherently less labor intensive during *erection, and also less dependent on the technique of individual welders for successful performance. However, quality assurance should be provided for installation and tensioning of the bolts, as well as correction of any problems with fit-up due to fabrication tolerances.*

Experimental Results: A series of tests on several different configurations of proprietary heavy bolted bracket connections have been performed at Lehigh University (Ksai & Bleiman, 1996) to qualify these connections for use in repair and modification applications. To test repair applications, brackets were placed only on the bottom beam flange to simulate installations on a connection where the bottom flange weld in the original connection had failed. In these specimens, bottom flange welds were not installed, to approximate the condition of a fully fractured weld. The top flange welds of these specimens were made with electrodes rated for notch toughness, to preclude premature failure of the specimens at the top flange. For specimens in which brackets were placed at both the top and bottom beam flanges, both welds were omitted. Acceptable plastic rotations were achieved for each of the specimens tested.

Quantitative Results: No. of specimens tested: 8
Girder Size: W16x40 and W36x150
Column Size: W12x65 and W14x425
Plastic Rotation achieved - 0.05 radians - 0.07 radians

7.10 Other Types of Welded Connection Structures

There are no modifications to the Guidelines or Commentary of Section 7.10 at this time.

7.10.1 Eccentrically Braced Frames (EBF)

There are no modifications to the Guidelines or Commentary of Section 7.10.1 at this time.

7.10.2 Dual Systems

There are no modifications to the Guidelines or Commentary of Section 7.10.2 at this time.

7.10.3 Welded Base Plate Details

There are no modifications to the Guidelines or Commentary of Section 7.10.3 at this time.

7.10.4 Vierendeel Truss Systems

There are no modifications to the Guidelines or Commentary of Section 7.10.4 at this time.

7.10.5 Moment Frame Tubular Systems

There are no modifications to the Guidelines or Commentary of Section 7.10.5 at this time.

7.10.6 Welded Connections of Collectors, Ties and Diaphragm Chords

There are no modifications to the Guidelines or Commentary of Section 7.10.6 at this time.

7.10.7 Welded Column Splices

There are no modifications to the Guidelines or Commentary of Section 7.10.7 at this time.

7.10.8 Built-up Moment Frame Members

There are no modifications to the Guidelines or Commentary of Section 7.10.8 at this time.

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