6. POST-EARTHQUAKE REPAIR AND MODIFICATION

6.1 Scope

There are no modifications to the Guidelines or Commentary of Section 6.1 at this time.

6.2 Shoring

There are no modifications to the Guidelines or Commentary of Section 6.2 at this time.

6.3 Repair Details

There are no modifications to the Guidelines or Commentary of Section 6.3 at this time.

6.4 Preparation

There are no modifications to the Guidelines or Commentary of Section 6.4 at this time.

6.5 Execution

There are no modifications to the Guidelines or Commentary of Section 6.5 at this time.

6.6 STRUCTURAL MODIFICATION

6.6.1 Definition of Modification

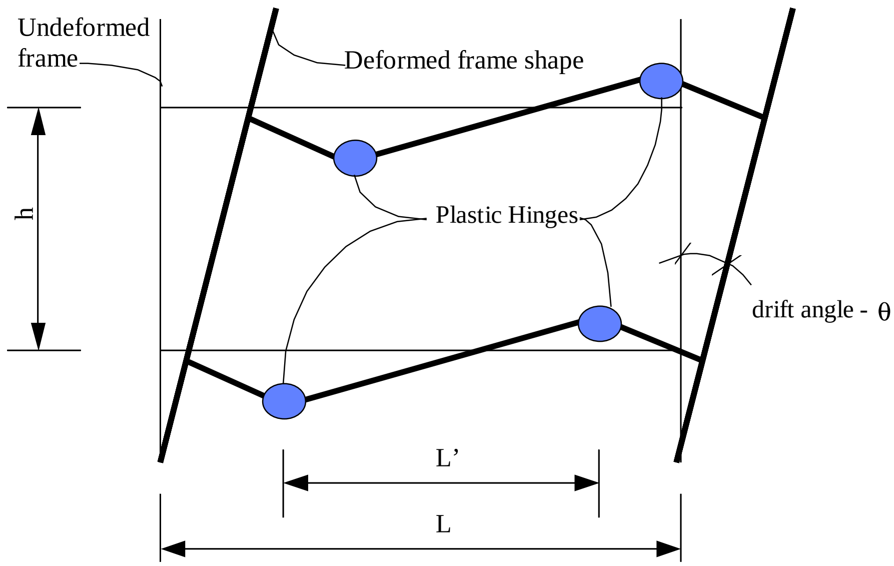
There are no modifications to the Guidelines or Commentary of Section 6.6.1 at this time.

6.6.2 Damaged vs. Undamaged Connections

There are no modifications to the Guidelines or Commentary of Section 6.6.2 at this time.

6.6.3 Criteria

Connection modification intended to permit inelastic frame behavior should be proportioned so that the required plastic deformation of the frame may be accommodated through the development of plastic hinges at pre-determined locations within the girder spans, as indicated in ~~Figure 6-12~~ Figure 6.6.3-1. 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. 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.



~~Figure 6-12~~ Figure 6.6.3-1 - 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 fibers and buckling at compressive fibers. If a sufficient number of plastic hinges develop in a frame, a mechanism is formed and the frame can deform laterally in a plastic manner. This behavior is accompanied by significant energy dissipation, particularly if a number of members are involved in the plastic behavior, as well as substantial local damage to the highly strained elements. The formation of hinges in columns, as opposed to beams, is undesirable, as this results in the formation of weak story mechanisms with relatively few elements participating, 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~~ stress and strain demands on the welded beam flange to column flange joint. 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 modified to be sufficiently strong to force the inelastic action (plastic hinge) away from the column face. 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, the flexural demands on the columns 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 connection modifications 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 modified as described in these Interim Guidelines should experience many fewer such brittle fractures than unmodified connections. However, the formation of a plastic hinge within the span of a beam is not a completely benign event. Beams which have formed such hinges may exhibit large buckling and yielding deformation, damage which typically must be repaired. The cost of such repairs could be comparable to the costs incurred in repairing fracture damage experienced in the Northridge Earthquake. The primary difference is that life safety protection will be significantly enhanced and most structures that have experienced such plastic deformation damage should continue to be safe for occupancy while repairs are made.

If the types of damage described above are unacceptable for a given building, then alternative methods of structural modification 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, and similar systematic modifications of the building's basic lateral force resisting system.

It is important to recognize that in frames with relatively short bays, the flexural hinging indicated in Figure 6.6.3-1 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 6.6.3-2.*

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 6.6.3-2 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. If modifications to an existing frame result in such a configuration designers should consider referring 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.

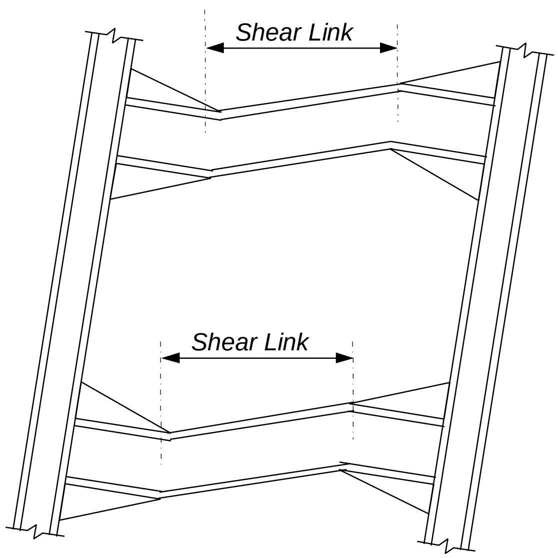


Figure 6.6.3-2 Shear Yielding Dominated Behavior of Short Bay Frames

6.6.4 Strength and Stiffness

6.6.4.1 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

Alternatively, the criteria contained in the 1997 edition of the *AISC Seismic Provisions for Structural Steel Buildings* (AISC, 1997) may be used.

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. Structural upgrades 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.

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. Similar, and perhaps more severe, effects may also exist where steel beams support masonry or concrete walls.*

6.6.4.2 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 such as walls. 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.*

6.6.5 Plastic Rotation Capacity

The plastic rotation capacity of modified connections should reflect realistic estimates of the required level of plastic rotation demand. In the absence of detailed calculations of rotation demand, connections should be shown to be capable of developing a minimum plastic rotation capacity on the order of 0.025 to 0.030 radian. The demand may be lower when braced frames, supplemental damping, base isolation, or other elements are introduced into the moment frame system, to control its lateral deformation; when the design ground motion is relatively low in the range of predominant periods for the structure; and when the frame is sufficiently strong and stiff.

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. 6.6.5-1 as the plastic deflection of the beam or girder, at the point of inflection (usually at the center of its span,) Δ_{CL}, divided by the distance between the center of the beam span and the centerline of the panel zone of the beam column connection, L_{CL}. This convention is illustrated in Figure 6.6.5-1.

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 6.6.3-1. These two quantities are related to each other, however, and if one of them is known, the other may be calculated from Eq. 6.6.5-1.

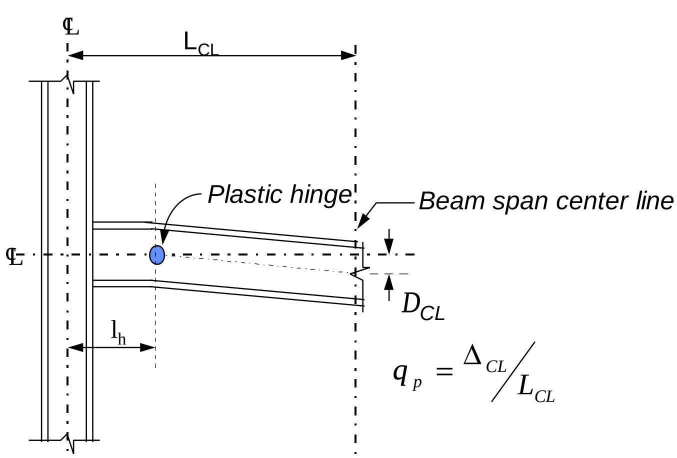


Figure 6.6.5-1 Calculation of Plastic Rotation Angle

$$\theta_p = \theta_{ph} \frac{(L_{CL} - l_h)}{L_{CL}} \qquad (6.6.5\text{-}1)$$

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 span to the center of the beam-column assembly panel zone
l_h is the assumed location of the discrete plastic hinge relative to the center of the beam-column assembly panel zone

If calculations are performed to determine the required connection plastic rotation capacity, the capacity should be taken somewhat greater than the calculated deformation demand, due to the high variability and uncertainty inherent in predictions of inelastic seismic response. Until better guidelines become available, a required plastic rotation capacity on the order of 0.005 radians greater than the demand calculated for the design basis earthquake (or if greater conservatism is desired - the maximum ~~capable~~ considered earthquake) is recommended. Rotation demand calculations should consider the effect of plastic hinge location within the beam span, as indicated in ~~Figure 6-12~~ Figure 6.6.3-1, on plastic rotation demand. Calculations should be performed to the same level of detail specified for nonlinear dynamic analysis for base isolated structures in *UBC-94* Section 1655 {*NEHRP-94* Section 2.6.4.4}. 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 instead of the base isolated period, T_I, the structure period, T, calculated in accordance with *UBC-94* Section 1628 {*NEHRP-94* Section 2.3.3.1} should be used.

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 θ_{ph}, in Eq. 6.6.5-1, 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 Interim Guidelines Advisory No. 1, may more readily be obtained from test data and also more closely relates to the drift experienced by a frame during earthquake response.

Traditionally, structural 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. It was assumed that the prescribed connections would be strong enough so that the 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. It is now known that the prescriptive connection is often incapable of behaving in this manner.*

In the 1994 Northridge earthquake, many moment-frame connections fractured with little evidence of plastic hinging of the girders or yielding of the column panel zones. Testing of moment frame connections both prior to and subsequent to the earthquake suggests that the standard welded flange-bolted web connection is unable to reliably provide plastic rotations beyond about 0.005 radian for all ranges of girder depths and often fails below that level. Thus, for frames designed for code forces and for the code drift limits, new connection configurations must be developed to reliably accommodate such rotation without brittle fracture.

In order to develop reasonable estimates of the plastic rotation demands on a frame's connections, it is necessary to perform inelastic time history analyses. For regular structures, approximations of the plastic rotation demands can be obtained from linear elastic analyses. Analytical research (Newmark and Hall - 1982) suggests that for structures having the dynamic characteristics of most WSMF buildings, and for the ground motions typical of western US earthquakes, the total frame deflections obtained from an unreduced (no R or R_w factor) dynamic analysis provide an approximate estimate of those which would be experienced by the inelastic structure. For the typical spectra contained in the building code, this would indicate expected drift ratios on the order of 1%. The drift demands in a real structure, responding inelastically, tend to concentrate in a few stories, rather than being uniformly distributed throughout the structure's height. Therefore, it is reasonable to expect typical drift demands in individual stories on the order of 1.5% to 2% of the story height. As a rough approximation, the drift demand may be equated to the joint rotation demand, yielding expected rotation demands on the order of perhaps 2%. Since there is considerable variation in ground motion intensity and spectra, as well as the inelastic response of buildings to these ground motions, conservatism in selection of an appropriate connection rotation demand is warranted.

In recent testing of large scale subassemblies incorporating modified connection details, conducted by SAC and others, when the connection design was able to achieve a plastic rotation demand of 0.025 radians or more for several cycles, the ultimate failure of the subassembly generally did not occur in the connection, but rather in the members themselves. Therefore, the stated connection capacity criteria would appear to result in connections capable of providing reliable performance.

It should be noted that the connection assembly capacity criteria for the modification of existing buildings, recommended by these Interim Guidelines, is *somewhat reduced compared to that recommended for new buildings (Chapter 7). This is typical of approaches normally taken for existing structures. For new buildings, these Interim Guidelines discourage building-specific calculation of required plastic rotation capacity for connections and instead, encourage the development of highly ductile connection designs. For existing buildings, such an approach may lead to modification designs that are excessively costly, as well as the modification of structures which do not require such modification. Consequently, an approach which permits the development of semi-ductile connection designs, with sufficient plastic rotation capacity to withstand the expected demands from a design earthquake is adopted. It should be understood that buildings modified to this reduced criteria will not have the same reliability as new buildings, designed in accordance with the recommendations of Chapter 7. The criteria of Chapter 7 could be applied to existing buildings, if superior reliability is desired.*

When performing inelastic frame analysis, in order to determine the required connection plastic rotation capacity, it is important to accurately account for the locations at which the plastic hinges will occur. Simplified models, which represent the hinge as occurring at the face of the column, may~~will~~ underestimate the plastic rotation demand. This problem becomes more severe as the column spacing, L, becomes shorter and the distance between plastic hinges, L', a greater portion of the total beam span. Eq. 6.6.5-1 may be used to convert calculated values of plastic rotation at a hinge remotely located from the column, to the chord angle rotation, used for the definition of acceptance criteria contained in these Guidelines. In extreme cases, the girder will not form plastic hinges at all, but instead, will develop a shear yield, similar to an eccentric braced frame.

6.6.6 Connection Qualification and Design

Modified girder-column connections may be qualified by testing or designed using calculations. Qualification by testing is the preferred approach. Preliminary designs of connections to be qualified by test may be obtained using the calculation procedures of Section 6.6.6.3. The procedures of that section may also be used to calibrate previous tests of similar connection configurations to slightly different applications, by extrapolation. Extrapolation of test results should be limited to connections of elements having similar geometries and material specifications as the tested connections. Designs based on calculation alone should be subject to qualified independent third party review.

Commentary: 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.*

6.6.6.1 Qualification Test Protocol

There are no modifications to the Guidelines or Commentary of Section 6.6.6.1 at this time.

6.6.6.2 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 6.6.5, for at least one complete cycle.

b) The connection should develop a minimum strength equal to 80% of the plastic strength of the girder, calculated using minimum specified yield strength F_y, throughout the loading history required to achieve the required plastic rotation capacity, as indicated in a), above.

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 shall 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: Many connection configurations will be able to withstand plastic rotations on the order of 0.025 radians or more, but will have sustained significant damage and degradation of stiffness and strength in achieving this deformation. The intent of the acceptance criteria presented in this Section is to assure that when connections experience the required plastic rotation demand, they will still have significant remaining ability to participate in the structure's lateral load resisting system.

In evaluating the performance of specimens during testing, it is important to distinguish between brittle behavior and ductile behavior. It is not uncommon for small cracks to develop in specimens after relatively few cycles of inelastic deformation. In some cases these initial cracks will rapidly lead to ultimate failure of the specimen and in other cases they will remain stable, perhaps growing slowly with repeated cycles, and may or may not participate in the ultimate failure mode. The development of minor cracks in a specimen, prior to achievement of the target plastic rotation demand should not be cause for rejection of the design if the cracks remain stable during repeated cycling. Similarly, the occurrence of brittle fracture at inelastic rotations significantly in excess of the target plastic rotation should not be cause for rejection of the design.

6.6.6.3 Calculations

There are no modifications to the Guidelines or Commentary of Section 6.6.6.3 at this time.

6.6.6.3.1 Material Strength Properties

In the absence of project specific material property information (for example, mill test reports), the values listed in ~~Table 6-3~~ Table 6.6.6.3.1-1 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.

~~Table 6-3~~Table 6.6.6.3-1 - Properties for Use in Connection Modification Design

Material	F_y (ksi)	$F_{y\,m}$ (ksi)	F_u (ksi)
A36 Beam	36	[1]	[1]
Dual Certified Beam			
Axial, Flexural	50		65 min.
Shape Group 1		55[2]	
Shape Group 2		58[2]	
Shape Group 3		57[2]	
Shape Group 4		54[2]	
Through-Thickness	-	-	Note 3
A572 Column/Beam			
Axial, Flexural	50		65 min.
Shape Group 1		58[2]	
Shape Group 2		58[2]	
Shape Group 3		57[2]	
Shape Group 4		57[2]	
Shape Group 5		55[2]	
Through-Thickness	-	-	Note 3
A992 Structural Shape[1]	Use same values as for A572, Gr. 50		

Notes:

1. See Commentary
2. Based on coupons from web. For thick flanges, the $F_{y\,flange}$ is approximately 0.95 $F_{y\,web}$.
3. See Commentary

Commentary: Table 6-3, Note 1 - The material properties for steel nominally designated on the construction documents as ASTM A36 can be highly variable and in recent years, steel meeting the specified requirements for both ASTM A36 and A572 has routinely been incorporated in projects calling for A36 steel. Consequently, unless project specific data is available to indicate the actual strength of material incorporated into the project, the properties for ASTM A572 steel should be assumed when ASTM A36 is indicated on the drawings, and the assumption of a higher yield stress results in a more severe design condition.

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 6-3~~Table 6.6.6.3-1, Note 3 - 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~~~~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 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 cover plated assemblies 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.*~~

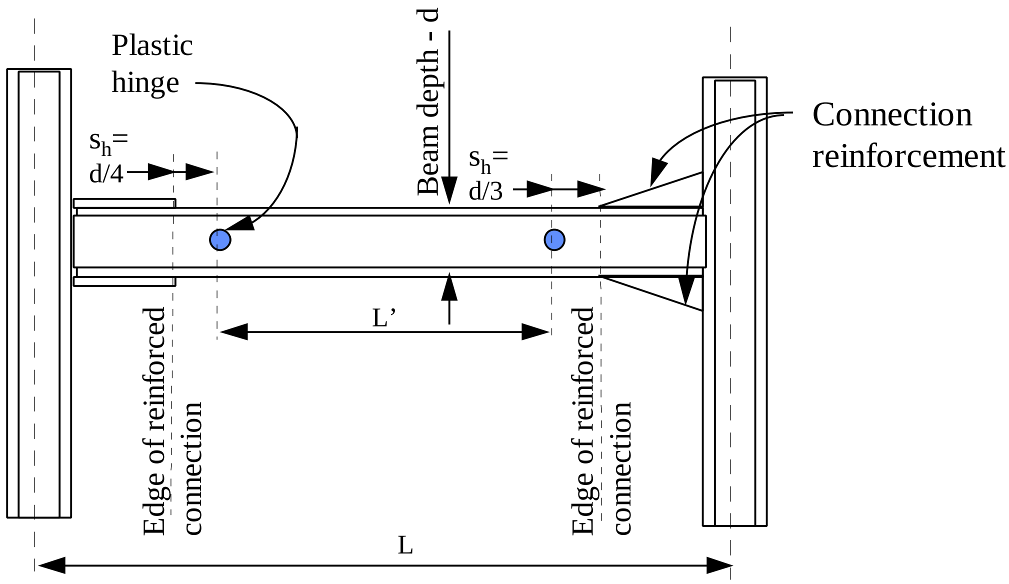
~~*Notwithstanding all of the above, successful tests using cover plates and other measures of moving hinges (and coincidentally reducing through-thickness stress) continue to be performed. In the interim, structural engineers choosing to utilize connections relying on through-thickness strength should 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.*~~

6.6.6.3.2 Determine Plastic Hinge Location

The desired location for the formation of plastic hinges should be determined as a basic parameter for the calculations. 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 6.6.6.3.2-1 and illustrated in Figure 6.6.6.3.2-1, ~~at a distance equal to 1/3 of the beam depth from the edge of the reinforced connection (or start of the weakened beam section),~~ unless specific test data for the connection indicates that a different value is appropriate.~~ Refer to Figure 6-13.~~

Table 6.6.6.3.2-1 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 6-13~~ Figure 6.6.6.3.2-1 - 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~~ indicated in Table 6.6.6.3.2-1 and Figure 6.6.6.3.2-1 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. Note that 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.*

6.6.6.3.3 Determine Probable Plastic Moment at Hinges

The probable value of the plastic moment, M_{pr}, at the location of the plastic hinges should be determined from the equation:

$$\cancel{M_{pr} = 0.95 a Z_b F_{ya}} \qquad \text{~~(6-1)~~}$$

$$M_{pr} = 1.1 Z_b F_{ya} \qquad (6.6.6.3.3\text{-}1)$$

where: ~~α is a coefficient that accounts for the effects of strain hardening and modeling uncertainty, taken as:~~

~~1.1 when qualification testing is performed or calculations are correlated with previous qualification testing~~

~~1.3 when design is based on calculations, alone.~~

F_{ya} is the actual yield stress of the material, as identified from mill test reports. Where mill test data for the project is not traceable to specific framing elements, the average of mill test data for the project for the given shape may be used. When mill test data for the project is not available, the value of F_{ym}, from ~~table 6-3~~Table 6.6.6.3-1 may be used.

Z_b is the plastic modulus of the section

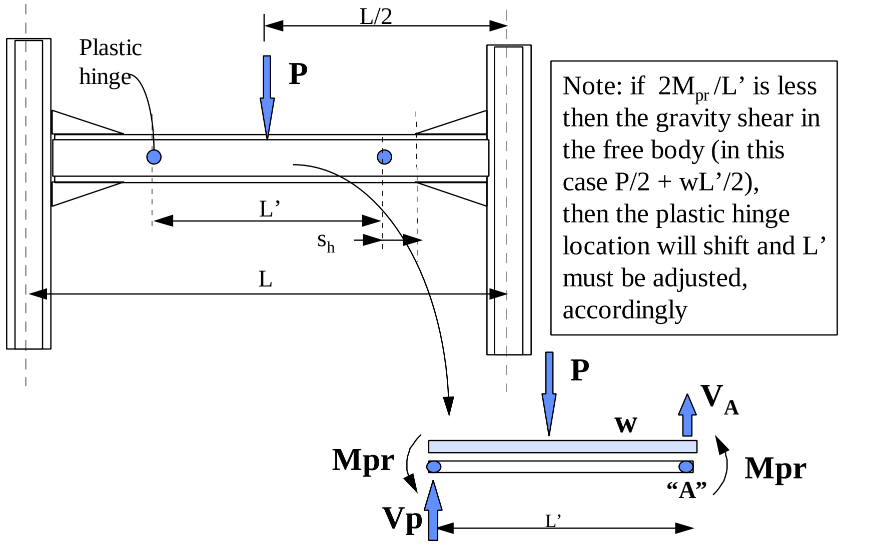
Commentary: The 1.1~~0.95~~ factor, in equation 6.6.6.3.3-1, is used to ~~adjust~~ account for two effects. First, it is intended to account for the typical difference between the yield stress in the beam web, where coupons for mill certification tests are normally extracted, and~~to~~ the value in the beam flange. Beam flanges, being comprised of thicker material, typically have somewhat lower yield strengths than do beam web material. Second, it is intended to~~The factor of 1.1 recommended to~~ account for strain hardening, or other sources of strength above yield, and agrees fairly well with available test results. It should be noted that the 1.1 factor could underestimate the over-strength where significant flange buckling does not act as the gradual limit on the connection. Nevertheless, the 1.1 factor seems a reasonable expectation of over-strength considering the complexities involved.

Connection designs that result in excessive strength in the girder connection relative to the column or excessive demands on the column panel zone are not expected to produce superior performance. There is a careful balance that must be maintained between developing connections that provide for an appropriate allowance for girder overstrength and those that arbitrarily increase connection demand in the quest for a "conservative" connection design. The factors suggested above were chosen in an attempt to achieve this balance, and arbitrary increases in these values are not recommended.

When the Interim Guidelines were first published, Eq. 6.6.6.3.3-1 included a coefficient, a, intended to account both for the effects of strain hardening and also for modeling uncertainty when connection designs were based on calculations as opposed to a specific program of qualification testing. The intent of this modeling uncertainty 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 a 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 the probable strength of an assembly.

6.6.6.3.4 Determine Beam Shear

The shear in the beam at the location of the plastic hinge should be determined. A free body diagram of that portion of the beam located between plastic hinges is a useful tool for obtaining the shear at each plastic hinge. ~~Figure 6-14~~Figure 6.6.3.4-1 provides an example of such a calculation.

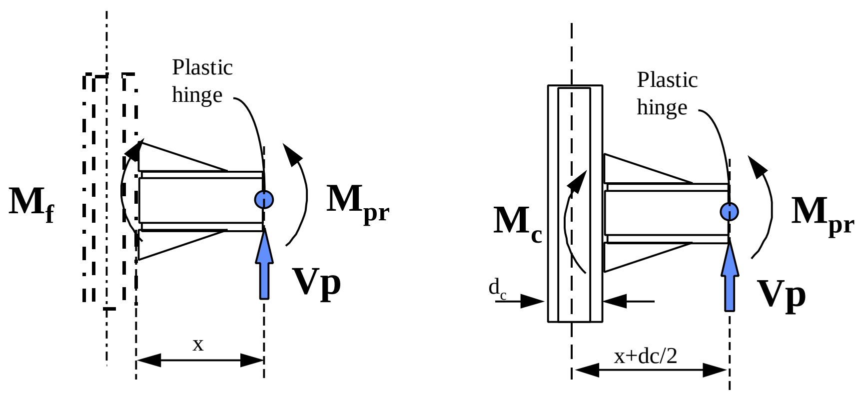


taking the sum of moments about "A" = 0
Vp ={Mpr + Mpr + P L'/2 + wL'²/2}/L'

~~Figure 6-14~~ Figure 6.6.3.4-1 - Sample Calculation of Shear at Plastic Hinge

6.6.6.3.5 Determine Strength Demands on Connection

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 6-15~~ Figure 6.6.3.5-1 demonstrates this procedure for two critical sections, for the beam shown in ~~Figure 6-14~~Figure 6.6.3.4-1.



$M_f = M_{pr} + V_p x$

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

Critical Section at Column Face

Critical Section at Column Centerline

~~Figure 6-15~~ Figure 6.6.3.5-1 - 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 weak beam - strong column conditions. Other critical sections should be selected as appropriate to the connection configuration.

6.6.6.3.6 Check for Strong Column - Weak Beam Condition

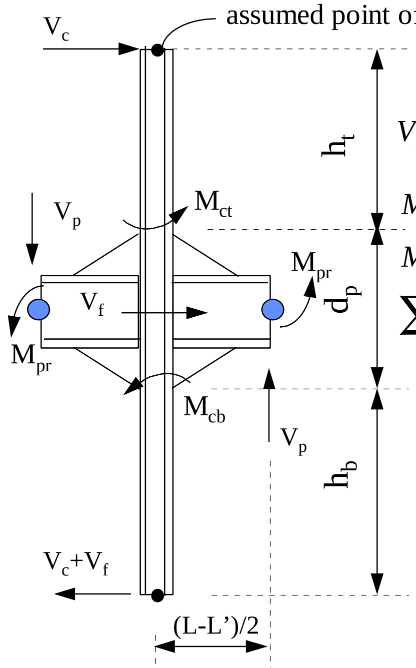
Buildings which form sidesway mechanisms through the formation of plastic hinges in the beams can dissipate more energy than buildings that develop mechanisms consisting primarily of plastic hinges in the columns. Therefore, if an existing building's original design was such that hinging would occur in the beams rather than the columns, care should be taken not to alter this behavior with the addition of connection reinforcement. To determine if the desired strong column - weak beam condition exists, the connection assembly should be checked to determine if the following equation is satisfied:

$$\sum Z_c (F_{yc} - f_a) / \sum M_c > 1.0 \qquad (6.6.6.3.6\text{-}1\cancel{2})$$

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 the column in accordance with Section 6.6.6.3.5~~ 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: Equation 6.6.6.3.6-1~~2~~ is based on the building code provisions for strong column - weak beam design. 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 to 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. 6.6.3.5.*

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 6.6.6.3.6-1 illustrates a method for determining this quantity. ~~In such cases,~~ When evaluation indicates that a strong column - weak beam condition does not exist, a plastic analysis should be considered to determine if an undesirable story mechanism is likely to form in the building.



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

$$M_{ct} = V_c h_t$$

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

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

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 6.6.6.3.6-1 Calculation of Column Moment for Strong Column Evaluation

6.6.6.3.7 Check Column Panel Zone

The adequacy of the shear strength of the column panel zone should be checked. For this purpose, the term $0.8\Sigma M_f$ should be substituted for the term $0.8\Sigma Ms$ in *UBC-94* Section 2211.7.2.1 {$0.9\Sigma\phi_b M_p$ in *NEHRP-91* Section 10.10.3.1} ~~repeated below for convenience of reference~~. M_f is the calculated moment at the face of the column, when the beam mechanism forms, calculated as indicated in Section 6.6.6.3.5, above. In addition, it is recommended not to use 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. 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:

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

where: b_c = width of column flange
d_b = the depth of the beam (including 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 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 encourage designs with 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.*

6.6.7 Modification Details

There are no modifications to the Guidelines or Commentary of Section 6.6.7 at this time.

6.6.7.1 Haunch at Bottom Flange

Figure ~~6-16~~6.6.7.1-1 illustrates the basic configuration for a connection modification consisting of the addition of a welded haunch at the bottom beam flange. Several tests of such a modification 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*). 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. Laboratory tests indicate this modification can be effective when the existing low-toughness welds between the beam bottom flange and column are left in place, however, more reliable performance is obtained when the top welds are modified. A complete procedure for the design of this modification may be found in *NIST, 1998*.~~, two alternative configurations of this detail that have been tested (Uang - 1995). The basic concept is to reinforce the connection with the provision of a triangular haunch at the bottom flange. The intended behavior of both configurations is to shift the plastic hinge from the face of the column and to reduce the demand on the CJP weld by increasing the effective depth of the section. In one test, shown on the left of Figure 6-16, the joint between the girder bottom flange and column was cut free, to simulate a condition which might occur if the bottom joint had been damaged, but not repaired. In a second tested configuration, the bottom flange joint was repaired and the top flange was replaced with a locally thickened plate, similar to the detail shown in Figure 6-9.~~

Design Issues: This approach developed acceptable levels of plastic rotation. Acceptable levels of connection strength were also maintained during large inelastic deformations of the plastic hinge. This approach does not require that the top flange be modified, or slab disturbed, unless other conditions require repair of the top flange~~, as in the detail on the left of Figure 6-16~~. The bottom flange is generally far more accessible than the top flange because a slab does not have to be removed. In addition, the haunch can be installed at perimeter frames without removal of the exterior building cladding. There did not appear to be any appreciable degradation in performance when the bottom beam flange was not re-welded to the face of the column. Eliminating this additional welding should help reduce the cost of the repair.

Performance is dependent on properly executed complete joint penetration welds at the column face and at the attachment of the haunch to the girder bottom flange. 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. The skewed groove welds of the haunch flanges to the girder and column flanges may be difficult to execute.*

Experimental Results: ~~*This approach developed excellent levels of plastic rotation. In Specimen 1, the bottom flange CJP weld was damaged in a prior test but was not repaired: only the bottom haunch was added. During the test of specimen 1, a slowly growing crack developed at the underside of the top flange-web intersection, perhaps exacerbated by significant local buckling of the top flange. Some of the buckling may be attributed to lateral torsional buckling that occurred because the bottom flange was not restrained by a CJP weld. A significant portion of the flexural strength was lost during the cycles of large plastic rotation. In the second specimen, the bottom girder flange weld was intact during the haunch testing, and its performance was significantly improved compared with the first specimen. The test was stopped when significant local buckling led to a slowly growing crack at the beam flange and web intersection. At this time, it appears that repairing damaged bottom flange welds in this configuration can produce better performance. Acceptable levels of flexural strength were maintained during large inelastic deformations of the plastic hinge for both specimens.*~~ *As reported in NIST, 1998, a total of 9 beam-column connection tests incorporating bottom haunch modifications of pre-Northridge connections have been tested in the laboratory, including two dynamic tests. Most of the connection assemblies tested resisted in excess of 0.02 radians of imposed plastic rotation. However, for those specimens in which the existing low-toughness weld was left in place at the beam top flange, without modification, connection behavior was generally limited by fractures generating at these welds at relatively low plastic rotations. It may be expected that enhanced performance can be obtained by replacing or reinforcing these welds as part of the modification.*

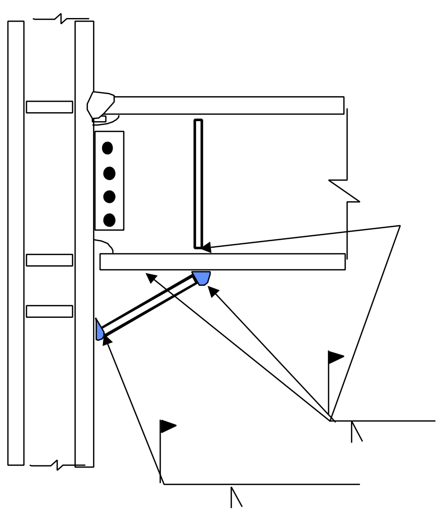


Figure ~~6-16~~6.6.7.1-1 - Bottom Haunch Connection Modification

Quantitative Results: *No. of specimens tested: ~~2~~9*

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)

Specimen ~~2~~ UCSD-3R:0.05 radian (with bottom flange weld)

Specimen UCSD-4R: 0.014 radian (dynamic- limited by test setup)

Speciemn 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-

Specment UTA-NSF4: 0.015 radian (no modification of top weld)

Girder Size: W18x86

Column Size: W24x279

Plastic Rotation achieved-

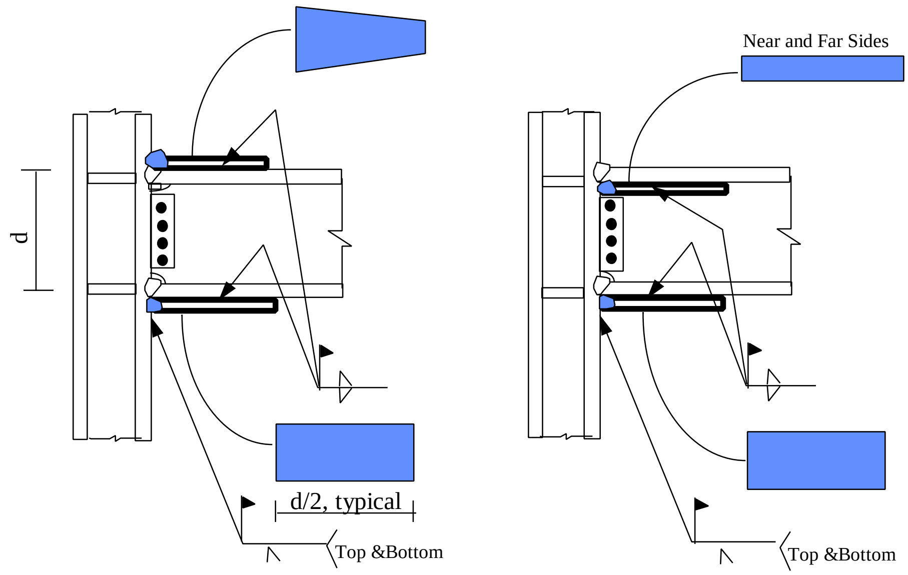
Specimen SFCCC-8: 0.035 radian (cover plated top flange)

6.6.7.2 Top and Bottom Haunch

There are no modifications to the Guidelines or Commentary of Section 6.6.7.2 at this time.

6.6.7.3 Cover Plate Sections

Figure 6.6.7.3-1 ~~Figure 6-18~~ illustrates the basic configurations of cover plate connections. The assumption behind the cover plate is that it reduces the applied stress demand on the weld at the column flange and shifts the plastic hinge away from the column face. Only the connection with cover plates on the top of the top flange has been tested. There are no quantitative results for cover plates on the bottom side of the top flange, such as might be used in repair. It is likely that thicker plates would be required where the plates are installed on the underside of the top flange. The implications of this deviation from the tested configuration should be considered.



~~Figure 6-18~~ Figure 6.6.7.3-1 - Cover Plate Connection Modification

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~~ eight connections similar to that shown in ~~Figure 6-18~~Figure 6.7.3-1 ~~have been~~ were tested (Engelhardt & Sabol - 1994), and ~~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. Due to the significant publicity that followed these successful tests, as well as the economy of these connections relative to some other alternatives, 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.

The option with the top flange cover plate located on top of the flange can be used on perimeter frames where access to the outer side of the beam is restricted by existing building cladding. The option with the cover plate for the top flange located beneath the flange can be installed without requiring modification of the slab. In the figures shown, the bottom cover plate is rectangular, and sized slightly wider than the beam flange to allow downhand fillet welding of the joint between the two plates. Some configurations using triangular plates at the bottom flange, similar to the top flange have also been tested.

Designers using this detail are cautioned to be mindful of not making cover plates so thick that excessively large welds of the beam flange combination to column flange result. As the cover plates increase in size, the weld size must also increase. Larger welds invariably result in greater shrinkage stresses and increased potential for cracking prior to actual loading. In addition, larger welds will lead to larger heat affected zones in the column flange, a potentially brittle area.

Performance is dependent on properly executed girder flange welds. The joint can be subject to through-thickness failures in the column flange. Access to the top of the top flange requires demolition of the existing slab. Access to the bottom of the top flange requires overhead welding and may be problematic for perimeter frames. Costs are greater than those associated with approaches that concentrate modifications on the bottom flange

Experimental Results: 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. These tests were performed using heavy column sections which forced nearly all of the plastic deformation into the beam plastic hinge; very little column panel zone deformation occurred. 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 pulled out (type C2 failure). The successful tests were terminated either when twisting of the specimen threatened to damage the test setup or the maximum stroke of the loading ram was achieved. Since the completion of that testing, a number of additional tests have been performed. Data for 18 tests, including those performed by Engelhardt and referenced above, are in the public domain. At least 12 other tests have been performed on behalf of private parties, however, the data from these tests are not available. Some of those tests exhibited premature fractures.

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~~, and 426~~
Plastic Rotation achieved-
~~6~~ 13 Specimens : >.025 radian to 0.05 radian
~~1~~3 Specimens: 0.005 < θ_p < 0.025~~0.015 radian (W2 failure)~~
~~1~~2 Specimens: 0.005 radian ~~(C2 failure)~~

6.6.7.4 Upstanding Ribs

There are no modifications to the Guidelines or Commentary of Section 6.6.7.4 at this time.

6.6.7.5 Side-Plate Connections

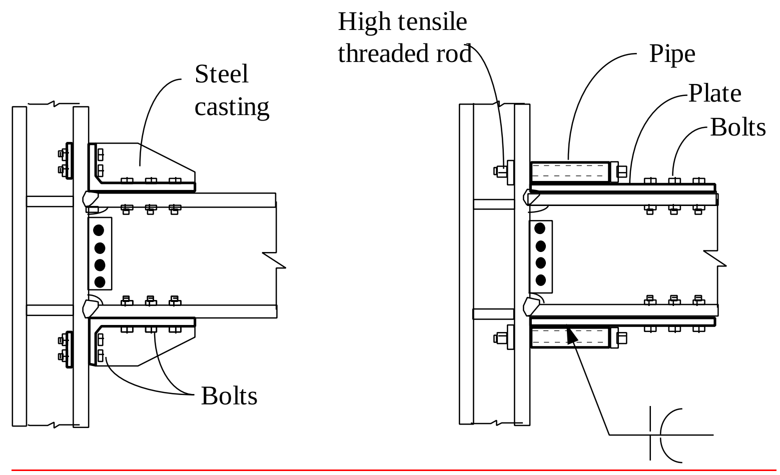
There are no modifications to the Guidelines or Commentary of Section 6.6.7.5 at this time.

6.6.7.6 Bolted Brackets

Heavy bolted brackets, incorporating high strength bolts, may be added to existing welded connections to provide an alternative load path for transfer of stress between the beams and columns. To be compatible with existing welded connections, the brackets must have sufficient strength and rigidity to transfer beam stresses with negligible deformation. Pre-tensioning of the bolts or threaded rods attaching the brackets to the column flanges and use of welds or slip-critical connections between the brackets and beam flanges can help to minimize deformation under load. Reinforcement of the column flanges may be required to prevent local yielding and excessive deformation of these elements. Two alternative configurations, which may be used either to repair an existing damaged, welded connection or to reinforce an existing undamaged connection are illustrated in Figure 6.6.7.6-1. The developer of these connections 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. Under a project jointly funded by NIST and AISC, the use of a single bracket at the bottom flange of the beam was investigated. It was determined that significant improvement in connection behavior could be obtained by placing a bracket at the bottom beam flange and by replacing existing low-toughness welds at the top flange with tougher material. *NIST, 1998* provides a recommended design procedure for such connection modifications.

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.

Bolted bracket connections can be installed in an existing building without field welding. Since this reduces the risk of construction-induced fire, brackets may be installed with somewhat less demolition of existing architectural features than with welded connections. In addition, the quality assurance issues related to field welding are eliminated. However, the fabrication of the brackets themselves does require quality assurance. Quality assurance is also required for operations related to the drilling of bolt holes for installation of bolts, surface preparation of faying surfaces and for installation and tensioning of the bolts themselves.



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

Figure 6.6.7.6-1 Bolted Bracket Modification

Bolted brackets can be used to repair damaged connections. If damage is limited to the beam flange to column flange welds, the damaged welds should be dressed out by grinding. Any existing fractures in base metal should be repaired as indicated in Section 6.3, in order to restore the strength of the damaged members and also to prevent growth of the fractures under applied stress. Since repairs to base metal typically require cutting and welding, this reduces somewhat the advantages cited above, with regard to avoidance of field welding.

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. No testing has yet been performed to determine the effectiveness of bolted brackets when applied to an existing undamaged connection with full penetration beam flange to column flange welds with low toughness or significant defects or discontinuities.

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

This page intentionally left blank