1. INTRODUCTION

1.1 Purpose

There are no modifications to the Guidelines or Commentary of Section 1.1 at this time.

1.2 Scope

There are no modifications to the Guidelines or Commentary of Section 1.2 at this time.

1.3 Background

Following the January 17, 1994 Northridge, California Earthquake, more than 100 steel buildings with welded moment-resisting frames were found to have experienced beam-to-column connection fractures. The damaged structures cover a wide range of heights ranging from one story to 26 stories; and a wide range of ages spanning from buildings as old as 30 years of age to structures just being erected at the time of the earthquake. The damaged structures ~~are~~ were spread over a large geographical area, including sites that experienced only moderate levels of ground shaking. Although relatively few such buildings were located on sites that experienced the strongest ground shaking, damage to these buildings was quite severe. Discovery of these extensive connection fractures, often with little associated architectural damage to the buildings, was ~~has been~~ alarming. The discovery ~~has~~ also caused some concern that similar, but undiscovered damage may have occurred in other buildings affected by past earthquakes. Indeed, there are now confirmed ~~isolated~~ reports of such damage. In particular, a publicly owned building at Big Bear Lake is ~~known to have been~~ was damaged by the Landers-Big Bear, California sequence of earthquakes, ~~and at least one building, under construction in Oakland, California at the time fo the~~ several buildings were damaged during the 1989 Loma Prieta Earthquake~~, was reported to have experienced such damage~~ in the San Francisco Bay Area.

WSMF construction is used commonly throughout the United States and the world, particularly for mid- and high-rise construction. Prior to the Northridge Earthquake, this type of construction was considered one of the most seismic-resistant structural systems, due to the fact that severe damage to such structures had rarely been reported in past earthquakes and there was no record of earthquake-induced collapse of such buildings, constructed in accordance with contemporary US practice. However, the widespread severe structural damage which occurred to such structures in the Northridge Earthquake called~~s~~ for re-examination of this premise.

The basic intent of the earthquake resistive design provisions contained in the building codes is to protect the public safety, however, there is also an intent to control damage. The developers of the building code provisions have explicitly set forth three specific performance goals for buildings designed and constructed to the code provisions (SEAOC - 1990). These are to provide buildings with the capacity to

- resist minor earthquake ground motion without damage;

- resist moderate earthquake ground motion without structural damage but possibly some nonstructural damage; and

- resist major levels of earthquake ground motion, having an intensity equal to the strongest either experienced or forecast for the building site, without collapse, but possibly with some structural as well as nonstructural damage.

In general, WSMF buildings in the Northridge Earthquake met the basic intent of the building codes, to protect life safety. However, the ground shaking intensity experienced by most of these buildings was significantly less than that anticipated by the building codes. Many buildings that experienced moderate intensity ground shaking experienced significant damage that could be viewed as failing to meet the intended performance goals with respect to damage control. Further, some members of the engineering profession (SEAOC - 1995b) and government agencies (Seismic Safety Commission - 1995) have stated that even these performance goals are inadequate for society's current needs.

WSMF buildings are designed to resist earthquake ground shaking based on the assumption that they are capable of extensive yielding and plastic deformation, without loss of strength. The intended plastic deformation is intended to be developed through a combination of ~~consists of~~ plastic rotations developing within the beams, at their connections to the columns, and plastic shear yielding of the column panel zones,. ~~and is t~~Theoretically these mechanisms should be capable of resulting in benign dissipation of the earthquake energy delivered to the building. Damage is expected to consist of moderate yielding and localized buckling of the steel elements, not brittle fractures. Based on this presumed behavior, building codes require a minimum lateral design strength for WSMF structures that is approximately 1/8 that which would be required for the structure to remain fully elastic. Supplemental provisions within the building code, intended to control the amount of interstory drift sustained by these flexible frame buildings, typically result in structures which are substantially stronger than this minimum requirement and in zones of moderate seismicity, substantial overstrength may be present to accommodate wind and gravity load design conditions. In zones of high seismicity, most such structures designed to minimum code criteria will not start to exhibit plastic behavior until ground motions are experienced that are 1/3 to 1/2 the severity anticipated as a design basis. This design approach has been developed based on historical precedent, the observation of steel building performance in past earthquakes, and limited research that has included laboratory testing of beam-column models, albeit with mixed results, and non-linear analytical studies.

Observation of damage sustained by buildings in the Northridge Earthquake indicates that contrary to the intended behavior, in some ~~many~~ cases brittle fractures initiated within the connections at very low levels of plastic demand, and in some cases, while the structures remained essentially elastic. Typically, but not always, fractures initiated at, or near, the complete joint penetration (CJP) weld between the beam bottom flange and column flange (Figure 1-1). Once initiated, these fractures progressed along a number of different paths, depending on the individual joint and stress conditions. Figure 1-1 indicates just one of these potential fracture growth patterns. ~~Investigators initially identified a number of factors which may have contributed to the initiation of fractures at the weld root including: notch effects created by the backing bar which was commonly left in place following joint completion; sub-standard welding that included excessive porosity and slag inclusions as well as incomplete fusion;~~

~~and potentially, pre-earthquake fractures resulting from initial shrinkage of the highly restrained weld during cool-down. Such problems could be minimized in future construction, with the application of appropriate welding procedures and more careful exercise of quality control during the construction process. However, it is now known that these were not the only causes of the fractures which occurred.~~

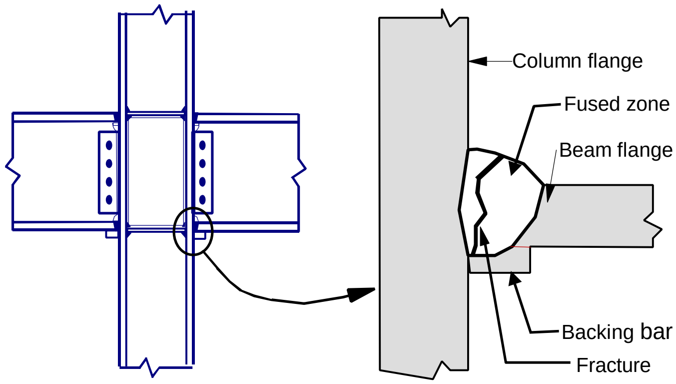


Figure 1-1 - Common Zone of Fracture Initiation in Beam -Column Connection

~~Current production processes for structural steel shapes result in inconsistent strength and deformation capacities for the material in the through-thickness direction. Non-metallic inclusions in the material, together with anisotropic properties introduced by the rolling process can lead to lamellar weakness in the material. Further, the distribution of stress across the girder flange, at the connection to the column is not uniform. Even in connections stiffened by continuity plates across the panel zone, significantly higher stresses tend to occur at the center of the flange, where the column web produces a local stiffness concentration. Large secondary stresses are also induced into the girder flange to column flange joint by kinking of the column flanges resulting from shear deformation of the column panel zone.~~

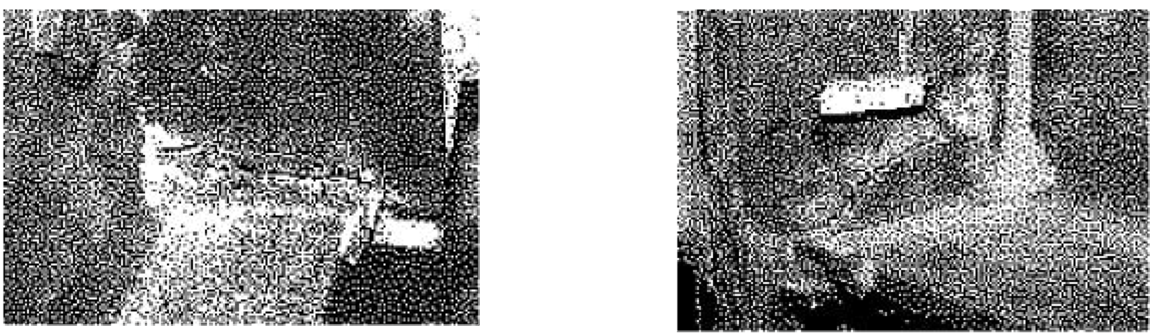
~~The dynamic loading experienced by the moment-resisting connections in earthquakes is characterized by high strain tension-compression cycling. Bridge engineers have long recognized that the dynamic loading associated with bridges necessitates different connection details in order to provide improved fatigue resistance, as compared to traditional building design that is subject to "static" loading due to gravity and wind loads. While the nature of the dynamic loads resulting from earthquakes is somewhat different than the high cycle dynamic loads for which fatigue-prone structures are designed, similar detailing may be desirable for buildings subject to seismic loading.~~

~~In design and construction practice for welded steel bridges, mechanical and metallurgical notches should be avoided because they may be the initiators of fatigue cracking. As fatigue cracks grow under repetitive loading, a critical crack size may be reached whereupon the material toughness (which is a function of temperature) may be unable to resist the onset of brittle (unstable) crack growth. The beam-to-column connections in WSMF buildings are comparable to category C or D bridge details that have a reduced allowable stress range as opposed to category B details for which special metallurgical, inspection and testing requirements are applied. The rapid rate of loading imposed by seismic events, and the complete inelastic range of tension-compression-tension loading applied to these connections is~~

~~much more severe than typical bridge loading applications. The mechanical and metallurgical notches or stress risers created by the beam-column weld joints are a logical point for fracture problems to initiate. This, coupled with the tri-axial restraint provided by the beam web and the column flange, is a recipe for brittle fracture.~~

~~During the Northridge Earthquake, o~~Once fractures initiated in beam-column joints, they progressed in a number of different ways. ~~In some cases, the fractures initiated but did not grow, and could not be detected by visual observation. In other cases,~~ In many cases, the fractures progressed ~~completely~~ directly through the thickness of the weld, and if fireproofing was removed, the fractures were evident as a crack through exposed faces of the weld, or the metal just behind the weld (Figure 1-2a). Other fracture patterns also developed. In some cases, the fracture developed into a surface that resembled a through-thickness failure of the column flange material behind the CJP weld (Figure 1-2b). In these cases, a portion of the column flange remained bonded to the beam flange, but pulled free from the remainder of the column. This fracture pattern has sometimes been termed a “divot” or “nugget” failure.

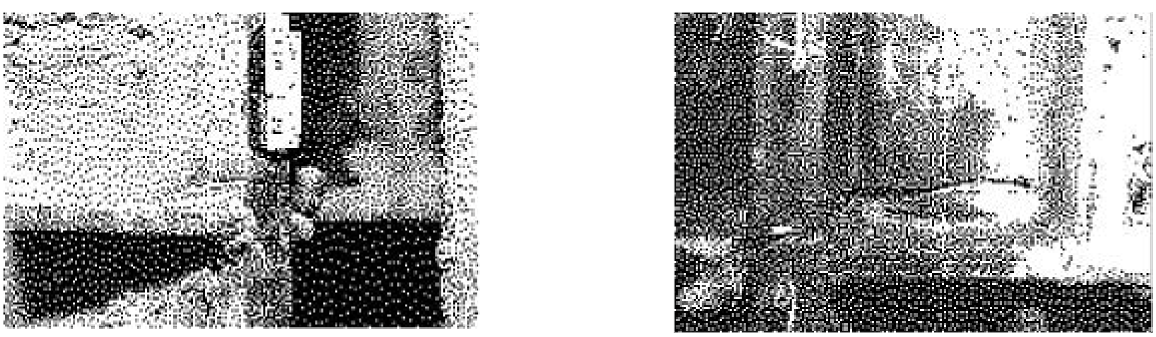
A number of fractures progressed completely through the column flange, along a near horizontal plane that aligns approximately with the beam lower flange (Figure 1-3a). In some cases, these fractures extended into the column web and progressed across the panel zone Figure (1-3b). Investigators have reported some instances where columns fractured entirely across the section.



a. Fracture at Fused Zone

b. Column Flange “Divot” Fracture

Figure 1-2 - Fractures of Beam to Column Joints



a. Fractures through Column Flange

b. Fracture Progresses into Column Web

Figure 1-3 - Column Fractures

Once these fractures have occurred, the beam - column connection has experienced a significant loss of flexural rigidity and capacity. Residual flexural strength and rigidity must be developed through a couple consisting of forces transmitted through the remaining top flange connection and the web bolts. ~~Initial r~~Research suggests that residual stiffness is approximately 20% of that of the undamaged connection and that residual strength varies from 10% to 40% of the undamaged capacity, when loading results in tensile stress normal to the fracture plane. When loading produces compression across the fracture plane, much of the original strength and stiffness remain. However, in providing this residual strength and stiffness, the beam shear connections can themselves be subject to failures, consisting of fracturing of the welds of the shear plate to the column, fracturing of supplemental welds to the beam web or fracturing through the weak section of shear plate aligning with the bolt holes (Figure 1-4).

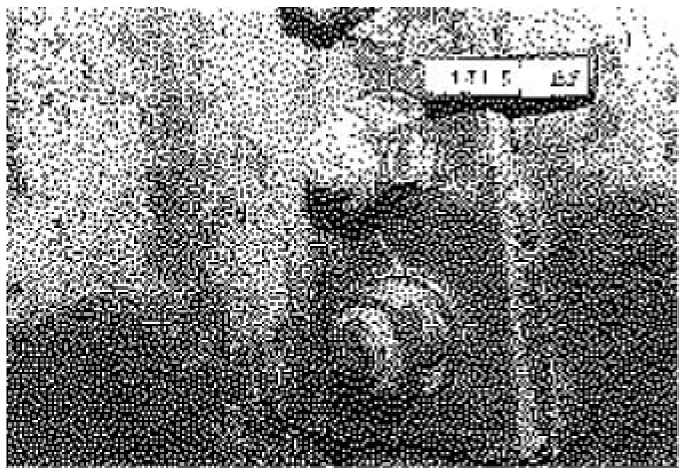


Figure 1-4 - Vertical Fracture through Beam Shear Plate Connection

It is now known that these fractures were the result of a number of complex factors that were not well understood either when these connections were first adopted as a standard design approach, or when the damage was discovered immediately following the Northridge earthquake. Engineers had commonly assumed that when these connections were loaded to yield levels, flexural stresses in the beam would be transferred to the column through a force couple comprised of nearly uniform yield level tensile and compressive stresses in the beam flanges. It was similarly assumed that nearly all of the shear stress in the beam was transferred to the column through the shear tab connection to the beam web. In fact, the actual behavior is quite different from this. As a result of local deformations that occur in the column at the location of the beam connection, a significant portion of the shear stress in the beam is actually transferred to the column through the beam flanges. This causes large localized secondary stresses in the beam flanges, both at the toe of the weld access hole and also in the complete joint penetration weld at the face of the column. The presence of the column web behind the column flange tends to locally stiffen the joint of the beam flange to the column flange, further concentrating the distribution of connection stresses and strains. Finally, the presence of the heavy beam and column flange plates, arranged in a "+" shaped pattern at the beam flange to column flange joint produces a condition of very high restraint, which retards the onset of yielding, by raising the effective yield strength of the material, and allowing the development of very large stresses.

The most severe stresses typically occur at the root of the complete joint penetration weld of the beam bottom flange to the column flange. This is precisely the region of this welded joint that is most difficult for the welder to properly complete, as the access to the weld is restricted by the presence of the beam web and the welder often performs this weld while seated on the top flange, in the so-called "wildcat" position. The welder must therefore work from both sides of the beam web, starting and terminating the weld near the center of the joint, a practice that often results in poor fusion and the presence of slag inclusions at this location. These conditions, which are very difficult to detect when the weld backing is left in place, as was the typical practice, are ready-made crack initiators. When this region of the welded joints is subjected to the large concentrated tensile stresses, the weld defects begin to grow into cracks and these cracks can quickly become unstable and propagate as brittle fractures. Once these brittle fractures initiate, they can grow in a variety of patterns, as described above, under the influence of the stress field and the properties of the base and weld metals present at the zone of the fracture.

Despite the obvious local strength impairment resulting from these fractures, many damaged buildings did not display overt signs of structural damage, such as permanent drifts or extreme damage to architectural elements. Until news of the discovery of connection fractures in some buildings began to spread through the engineering community, it was relatively common for engineers to perform cursory post-earthquake evaluations of WSMF buildings and declare that they were undamaged. In order to reliably determine if a building has sustained connection damage, it is necessary to remove architectural finishes and fireproofing and perform ~~nondestructive examination including visual inspection and ultrasonic testing~~ careful visual inspection of the welded joints supplemented, in some cases, by nondestructive testing. Even if no damage is found, this is a costly process. Repair of damaged connections is even more costly. ~~A few WSMF buildings have sustained so much connection damage that it has been deemed more practical to demolish the structures rather than to repair them.~~ In the case of one WSMF building, damaged by the Northridge earthquake, repair costs were sufficiently large that the owner elected to demolish rather than replace than building.

Immediately following the Northridge Earthquake, a series of tests of beam-column subassemblies were performed at the University of Texas at Austin, under funding provided by the AISC as well as private sources. The test specimens used heavy W14 column sections and deep (W36) beam sections commonly employed in some California construction. Initial specimens were fabricated using the standard prequalified connection specified by the *Uniform Building Code* (*UBC*). Section 2211.7.1.2 of *UBC-94* {*NEHRP-91* Section 10.10.2.3} specified this prequalified connection as follows:

> **"2211.7.1.2 Connection strength**. The girder top column connection may be considered to be adequate to develop the flexural strength of the girder if it conforms to the following:
>
> 1. the flanges have full penetration butt welds to the columns.
>
> 2. the girder web to column connection shall be capable of resisting the girder shear determined for the combination of gravity loads and the seismic shear forces which result from compliance with Section 2211.7.2.1. This connection strength need not exceed that required to develop gravity loads plus 3(R_w/8) times the girder shear resulting from the prescribed seismic forces.

Where the flexural strength of the girder flanges is greater than 70 percent of the flexural strength of the entire section, (i.e. $bt_f/(d-t_f)F_y>0.7Z_xF_y$) the web connection may be made by means of welding or high-strength bolting.

For girders not meeting the criteria in the paragraph above, the girder web-to-column connection shall be made by means of welding the web directly or through shear tabs to the column. That welding shall have a strength capable of developing at least 20 percent of the flexural strength of the girder web. The girder shear shall be resisted by means of additional welds or friction-type slip-critical high strength bolts or both.

and:

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

In order to investigate the effects that backing bars and weld tabs had on connection performance, these were removed from the specimens prior to testing. Despite these precautions, the test specimens failed at very low levels of plastic loading. Following these tests at the University of Texas at Austin, reviews of literature on historic tests of these connection types indicated a significant failure rate in past tests as well, although these had often been ascribed to poor quality in the specimen fabrication. It was concluded that the prequalified connection, specified by the building code, was fundamentally flawed and should not be used for new construction in the future.

In retrospect, this conclusion may have been somewhat premature. More recent testing of connections having configurations similar to those of the prequalified connection, but incorporating tougher weld metals, having backing bars removed from the bottom flange joint, and fabricated with greater care to avoid the defects that can result in crack initiation, have performed better than those initially tested at the University of Texas. However, as a class, when fabricated using currently prevailing construction practice, these connections still do not appear to be capable of consistently developing the levels of ductility presumed by the building codes for service in moment-resisting frames that are subjected to large inelastic demands.~~When the first test specimens for that series were fabricated, the welder failed to follow the intended welding procedures. Further, no special precautions were taken to assure that the materials incorporated in the work had specified toughness. Some engineers, with knowledge of fracture mechanics, have suggested that if materials with adequate toughness are used, and welding procedures are carefully specified and followed, adequate reliability can be obtained from the traditional connection details. Others believe that the conditions of high tri-axial restraint present in the beam flange to column flange joint (Blodgett - 1995) would prevent ductile behavior of these joints regardless of the procedure used to make the welds. Further they point to the important influence of the relative yield and tensile strengths of beam and column materials, and other variables, that can affect connection behavior. To date, there has not been sufficient research conducted to resolve this issue.~~

In reaction to the University of Texas tests as well as the widespread damage discovered following the Northridge Earthquake, and the urging of the California Seismic Safety Commission, in September, 1994 the International Conference of Building Officials (ICBO) adopted an emergency code change to the 1994 edition of the *Uniform Building Code* (*UBC*-94) {1994 *NEHRP Recommended Provisions Section 5.2}*. This code change, jointly developed by the Structural Engineers Association of California, AISI and ICBO staff, deleted the prequalified connection and substituted the following in its place:

> "**2211.7.1.2 Connection Strength**. Connection configurations utilizing welds or high-strength bolts shall demonstrate, by approved cyclic test results or calculation, the ability to sustain inelastic rotation and develop the strength criteria in Section 2211.7.1.1 considering the effect of steel overstrength and strain hardening."
>
> "**2211.7.1.1 Required strength**. The girder-to-column connection 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 from formula 11-1."

Unfortunately, neither the required "inelastic rotation", or calculation and test procedures are well defined by these code provisions. Design Advisory No. 3 (SAC-1995) included an Interim Recommendation (SEAOC-1995) that attempted to clarify the intent of this code change, and the preferred methods of design in the interim period until additional research could be performed and reliable acceptance criteria for designs re-established. The State of California similarly published a joint Interpretation of Regulations (DSA-OSHPD - 1994) indicating the interpretation of the current code requirements which would be enforced by the state for construction under its control. This applied only to the construction of schools and hospitals in the State of California. The intent of these Interim Guidelines is to supplement these previously published documents and to provide updated recommendations based on the results of the limited directed research performed to date.

1.4 The SAC Joint Venture

There are no modifications to the Guidelines or Commentary of Section 1.4 at this time.

1.5 Sponsors

There are no modifications to the Guidelines or Commentary of Section 1.5 at this time.

1.6 Summary of Phase 1 Research

There are no modifications to the Guidelines or Commentary of Section 1.6 at this time.

1.7 Intent

There are no modifications to the Guidelines or Commentary of Section 1.7 at this time.

1.8 Limitations

There are no modifications to the Guidelines or Commentary of Section 1.8 at this time.

1.9 Use of the Guidelines

There are no modifications to the Guidelines or Commentary of Section 1.9 at this time.

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