7th Canadian Conference on Earthquake Engineering / Montreal / 1995
7ième Conférence canadienne sur le génie paraséismique / Montréal / 1995

Northridge: the Role of Welding Clarified

D.K. Miller[1]

ABSTRACT

On January 17, 1994, a fairly moderate earthquake struck a northwestern suburb of Los Angeles. Suddenly, the city of Northridge became well-known to structural engineers around the world. The damage was widespread, yet steel structures stood. Not one death or a single structural collapse was associated with steel framed buildings. First reports were that steel structures were unscathed. Unfortunately, closer inspection since the earthquake has revealed damage to the beam-to-column connections in up to 100 steel framed buildings in the area. All but one of these structures will eventually be repaired. This paper describes the structural systems employed in steel framed buildings, the specific types of damage the beam-to-column connections sustained during the Northridge earthquake, and methods for avoiding such damage in the future.

SPECIAL MOMENT RESISTING FRAMES

Designers of buildings for seismic zones have several systems available to minimize the impact of the dynamic forces of an earthquake. One approach is through base isolation where structures are put on bearings, springs, or sliding devices that isolate the structure from the movement of the earth. These systems are highly effective, but unfortunately very expensive. No buildings in the Northridge area are known to have utilized this system, so its performance was not evaluated. Concentrically Braced Frames (CBF) utilize diagonal members within rectangular panels of buildings. This significantly strengthens the structure, minimizing lateral displacement and reducing the demands on the connections. The diagonal members are typically fabricated of structural tubing. By carefully selecting the member size, it is possible for these diagonal members to absorb a significant amount of the seismic energy as they endure alternating cycles of compression and tension. Northridge revealed some deficiencies in brace design where the b/t ratio was based on older code requirements, and the diagonals collapsed rather than absorption and transfer of the force. Code provisions, including the 1992 AISC seismic provisions, require thicker tube walls today. In another situation, the brace directly transferred the energy of the earthquake to thick base plates, resulting in fracture of the base plate assemblies. In general, however, the CBF structures performed well in Northridge. To a designer, this is an efficient system for dealing with seismic forces. To the architect, however, it limits space utilization, restricting the placement of windows and walls.

Another system that can be employed is an Eccentrically Braced Frame (EBF). Similar to the CBF, the EBF utilizes diagonal members to strengthen the structure. However, unlike the CBF, this system connects

[1]Welding Design Engineer, the Lincoln Electric Company, Cleveland, Ohio 44117

the diagonals to the horizontal floor beams. Between this intersection point and the column, a "link" is formed. The link is the region where the steel is expected to deform, absorbing seismic energy. Few, if any, of the structures in Northridge utilized the EBF system, so little new data is available on the efficiency of this approach.

The system employed in the majority of buildings in Northridge was the Special Moment Resisting Frame (SMRF). This approach uses no diagonals, but rather relies on the relative strength of the beam as compared to the column to develop a "strong column, weak beam" relationship. As the structure is subjected to lateral forces, the rectangular panels in the structure tend to form parallelograms. However, the connection is assumed to be strong enough that the lateral forces will be absorbed in the floor beam, causing displacement to occur in this region. The SMRF approach was utilized in the majority of the buildings in Northridge. To ensure a column that is stronger than the beam, engineers typically specify A572 Grade 50 (50,000 minimum specified yield strength, 65,000 minimum tensile strength) for the columns, and A36 (36,000 minimum specified yield strength, 50,000 minimum tensile strength) for the floor beams. It is possible to design a building to utilize either a few very massive moment connections per floor, or to replicate a smaller detail throughout the structure. In either approach, the concept is the same, namely, the floor beam will be the member that experiences elastic and in-elastic (plastic) deformations should the seismic forces become great enough. SMRF systems have become very popular in California, and were the predominant method used in the steel buildings in Northridge. And the answer to the preceding questions regarding the basis for all the concern surrounding the Northridge event is simply this: the Special Moment Resisting Frames (SMRF) did not behave as expected.

THE IMPLICATIONS

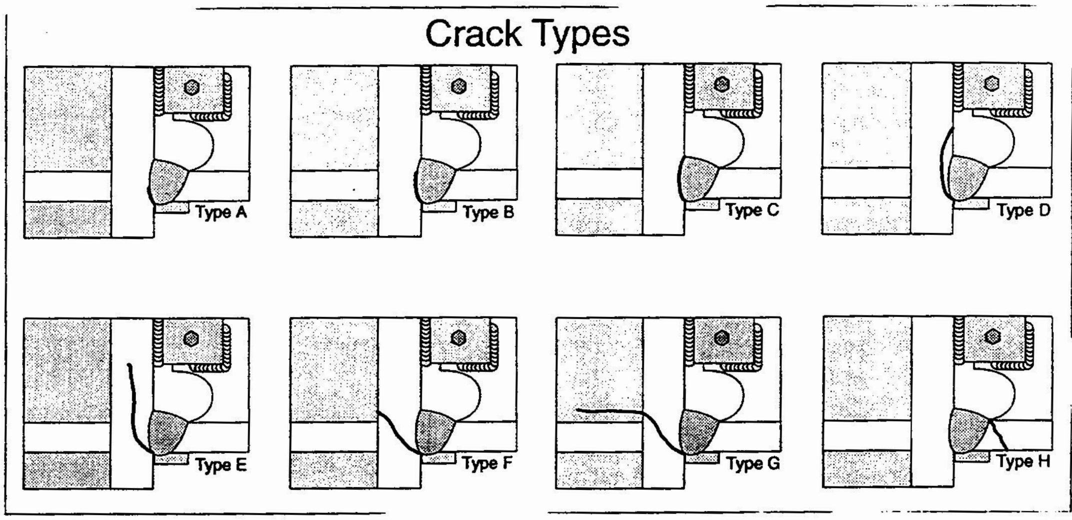
The implications of this statement are many-fold. Overall, the concerns regarding SMRFs are not centered on disappointing performance of the actual systems. Indeed, the highest order objective is to preserve human life; in this regard, the SMRFs performed admirably. The same can be said of the goal of no structural collapse, and even the goal of minimal nonstructural damage. And although the structural systems were damaged, they are being repaired. These systems did not perform as expected, and engineers have had to ask themselves the hard question, "Why?"

THE DAMAGE TO THE SMRF

A pattern quickly emerged among SMRF buildings that were damaged in the earthquake. The damage was typically confined to the lower flange-to-beam portion of the connection, and the top beam flange-to-column flange remained intact. In some cases, the bolted shear tab experienced sheared bolts, tears through the tab between the bolt holes or tears of the fillet weld from the column phase. This type of damage to the shear tab, however, occurred only in the presence of damage to the bottom flange. The top flanges remained intact; this was attributed to the influence of the slab which generated some composite strengthening to the connection.

The nature of the fracture to the bottom flange, however, varied from structure to structure, and even within a specific building. The accompanying drawing illustrates the eight types of cracks that have been observed. Of the eight, seven have their point of initiation at the intersection of the bottom side of the bottom flange to the column flange. This is in the area where the fusible steel backing intercedes the column and beam.

The very specific delineation between fracture types was made in order to classify the damage prior to repair. Depending on the nature of the damage, the approach to repair varies.



Type A cracks initiate at the region of the backing bar where the beam flange and the column flange come together at a 90° angle. The cracks initiate at the root of the weld, following the zone immediately between the weld and the column flange material, that is, the fusion line. A Type A crack is defined as extending upwards less than one-half the beam flange thickness.

Type B cracks are similar to Type A, except by definition, they extend more than half-way through the bottom flange. However, they do not exit to a surface. Again, the only reason to distinguish between these two types of cracks is that the repair procedures employed are slightly different between the two details. From the underside of the beam, Type A and B cracks are evident in the separation of the backing from the column.

Type C cracks can be viewed as an extension of a Type B with a crack exiting at the weld toe. For purposes of definition, the exit point is at the weld face, weld toe, or within 1/4 inch of the weld toe in the column flange. These cracks typically follow immediately along the fusion zone. At first glance, this crack type has the general characteristics associated with either lamellar tearing or underbead cracking.

Type D cracks begin like Type A's, but the crack turns into the column flange material, and exits well above the toe of the groove weld. The fracture is clearly in the base metal. As viewed from the end of the floor beam, looking toward the column face, Type D cracks usually occur on either side of the web, creating two flattened arch-like exit points. In the center of the face of the column flange, the Type D may become a Type C.

Type E cracks are very similar to Type D, but they are buried within the column flange and do not exit to a free surface. Type E cracks can only be positively identified with ultrasonic (UT) inspection. During repair of damaged structures, it has been found that fractures expected to be Type A or B are frequently Type E.

Type F cracks are similar to Type E, but the crack moves into the column flange, and may exit on the interior face of the column. The exit point is typically at the toe of the weld that joins the continuity plates to the inside of the flange. There are some reports of the exit point being below the continuity plate although this is the exception.

Type G cracks are akin to Type F, but the crack continues into the column web. Perhaps the most disturbing of all crack types in some structures, there are Type G cracks that initiate at either side of a column, resulting in complete horizontal fracturing of the column. Of course, the columns are in compression and some lateral stability is offered by the presence of the deck.

The final type of crack is identified as Type H. It initiates at the toe of the weld, or at the intersection point of the weld access hole and the bottom flange of the floor beam. The crack may go though the weld metal, the base metal, or a combination of the two.

Of all the various crack types, only one actually goes through weld metal (Type H), and even there, it does not always go through weld metal, nor is the initiation point in the weld. This distinction is extremely important because, in order to evaluate the feasibility of various possible explanations for the damage that was seen, the fractures must be clearly defined. In the public media, and even in technical journals, these are frequently called "weld failures". This is unfortunate because it incorrectly associates the problem with the weld when, in fact, the problem is associated with the connection. It is equally incorrect, however, to refer to this as a connection failure because, in the parlance of most structural engineers, the connection of a beam-to-column includes both flanges and the web. In all known situations, the connection remained intact, albeit, damaged. Therefore, the term "damaged connection" has been used throughout this paper.

Because the damage was concentrated in the vicinity of the welds, it was reasonable to believe that people would immediately focus on this area for an answer to the question "Why?". It became apparent, however, the answer would not be found by narrowly focusing only on the weld, but that the fundamental design of the connection would have to be reevaluated.

CONNECTION DETAILS

The moment connection of interest utilizes a shear tab that is shop welded to the column. In the field, the beam web is bolted to this shear tab, facilitating alignment and erection of the member. The top and bottom flanges are field welded. The joint detail typically consists of an AWS D1.1 prequalified complete joint penetration groove weld detail, namely a TC-U4a. Most field contractors utilize a root opening of 3/8 inch and an included angle of 30°. The D1 Code requires the application of "weld tabs" to either end of the joint to facilitate quality deposition of weld metal across the entire joint. A backing bar is placed under the weld joint to support the molten weld metal. The D1 Code requires this backing bar to be thoroughly fused by the weld metal.

Depending on the ratio of Z_f/Z (Z is the beam's plastic section modulus, and Z_f is the plastic section modulus of the beam flanges only), supplemental welds of the tab to the beam web may be required. The moment connection, therefore, consists of welded flanges, bolted web, and, in some cases, supplemental fillet welds of shear tab to web. This connection, as indicated before, has shown widely varying performance characteristics under laboratory conditions. Significantly better performance has been seen when the web is directly welded to the column. The improved performance has been attributed to the better transfer of the moment capacity of the web through this welded connection. Most designs have assumed the moment capacity of the beam to be transferred through the flanges, although there may be substantial web moment capacity depending on the Z_f/Z ratio. If this force is not transferred through the web connection, additional forces are passed through the flanges.

DIFFICULT WELDING CONDITIONS

It is possible to weld across the full width of the top flange without interruption. The weld that attaches the bottom flange of the beam to the column is more difficult to make, because the beam web prohibits the deposition of a continuous weld along the flange width. The welder is required to extend the electrode through the weld access hole and travel to the edges of the flange, terminating the weld on the weld tab. Properly sized weld access holes (frequently called "rat holes") facilitate adequate access and visibility for quality welding. The D1.1 Code prescribes certain minimum weld access hole dimensions. However, these minimum dimensions may be too small for some applications. It is imperative that the welder be given ample space to facilitate the deposition of quality weld metal.

After the root pass has been applied to one-half of the beam column weld, the starting point of the weld must be carefully cleaned and inspected to ensure that the subsequent root pass on the opposite side of the web can be sufficiently tied into the other half of the weld. The welder will, once again, extend the electrode through the weld access hole and carry the weld out onto the extension tab. In contrast to the top flange-to-column weld, the bottom flange weld requires welders to use both a left-handed and right-handed technique. Clearly the more difficult of the two welds to make is the bottom flange-to-column joint.

In the investigation of earthquake damaged connections, examples of inadequate fusion and slag entrapment have been identified, generally concentrated in the portion of the weld directly under the beam web. This is the area where one would anticipate the greatest number of difficulties. It is imperative that welders be trained to deposit sound weld metal along the entire length of the joint. Detailers must specify the dimensions of weld access holes to facilitate quality workmanship. In at least one building, generously sized weld access holes were utilized, but the shear tab was so long that it interfered with the access hole.

When these joints are subject to UT inspection, the center of the length of the weld is difficult to inspect because of the interference of the beam web, and the radius between the web and flange. Overhead UT inspection from the underside of the flange can overcome this problem.

In some early analysis of the damaged connections, the greater difficulty associated with fabrication of the bottom flange weld was proposed as an explanation for the greater occurrence of cracking in the bottom vs. top flanges. The contribution of the slab is another explanation, particularly since cracking in the bottom portion of the connection occurred even when there was no evidence of slag inclusions or lack of fusion in this region.

DISCUSSION REGARDING YIELD TO TENSILE RATIO

Mention has already been made of the ratio of Z_f/Z, the relative plastic moment capacity of the beam flange to the total section. If it is assumed that no moment is transferred through the web connection, then it is imperative that the moment capacity of the flanges times the tensile strength of the steel be greater than the total moment of the section, times the yield strength of the steel, or $Z_fF_u>ZF_y$. If this is maintained, the full plastic moment capacity of the beam can be transferred through the connection. The preceding may be rearranged as follows:

$$\frac{Z_f}{Z} > \frac{F_y}{F_u}$$

This suggests that not only is F_y (yield strength) important, but the ratio is important, as well. For rolled W-shapes, Z_f/Z ranged from 0.6 to 0.9. Based on ASTM minimum specified properties, F_y/F_u is as follows:

A36	0.62
A572Gr50	0.77

However, when actual properties of the steel are used, this ratio may increase. In the case of one building, mill test reports indicated this ratio to be 0.83.

Perhaps even more important is the ratio of yield strength of the beam material to the tensile strength of the column material, because it is in the column that many fractures are found. While the yield-to-tensile ratio of a specific piece of steel compensates for a simultaneous increase in both yield and tensile strength, the ratio of beam yield to column tensile strength could easily approach unity.

Careful control of material properties is essential if no moment is transferred through the web. An alternate solution, and a more direct one, is to utilize a more rigid, welded web connection, but even then, material properties must be controlled if the connection is to behave as designed.

In the damaged structures of Northridge, there was only rare evidence that these plastic zones actually were formed. Rather, the seismic energy was passed directly to the connection, overloading it and causing it to fracture. The area under the curve of a stress-strain diagram represents the total energy absorbed. It is essential that yielding take place to have significant plastic energy absorption. When the yield point is higher than expected, yielding will not occur, very little energy is absorbed in these members, and greater energies are transferred through to the connection.

ASTM A36 steel has a *minimum* specified yield strength of 36 Ksi. The delivered steel will obviously have a higher value if it is within specification. Twenty years ago, the average yield strength for A36 was approximately 42 Ksi. In 1994, the average yield strength has increased to approximately 48 Ksi. This average value is 1/3 higher than the value assumed by many designs. Of course, while 48 Ksi is the average value, some steel will have even higher values, with 55 Ksi being fairly routine. For columns, the flange properties will typically be closer to minimum specified values, in part due to the thickness of the flanges. Furthermore, the tensile coupons for the mill test reports on rolled shapes are extracted from the web of the section. The thinner webs routinely exhibit higher values than heavier flanges. According to mill test reports from steel in actual buildings, it is statistically possible to have floor beam material with higher actual yield strength properties than those of the column, in spite of the difference in the steel specifications.

Also misunderstood is the role of the material property of ductility, and the configuration of steel. Most tensile and elongation data is obtained from uniaxial slowly loaded tensile specimens. When stretched, the elongated sample becomes thinner and more narrow. This reduction in the cross-section of the specimen can be measured after fracture and is generally expressed as reduction in area. It is not unusual for steel to exhibit 20 - 30% elongation under these test conditions.

When steel is simultaneously loaded in two or even three directions, however, it will not be able to exhibit its inherent ductility. Rather than behaving in a ductile manner, the steel will fracture without exhibiting any elongation, and the fracture is typically termed "brittle". Yet, a uniaxial test specimen of the same material could exhibit tremendous elongation and ductility. Therefore, the designer must consider both the ductility of the material, and ductility of the configuration of the material. In the case of the most common Northridge connections, the weld itself was highly restrained through the length and in the transverse direction. This made the connection extremely rigid and any fracture that would occur would be expected to be brittle-like in nature, even though the steel and the weld metal were ductile. Regarding yielding of the beam itself, it is reasonable to expect yielding could occur, but probably at a level greater than the uniaxial tensile value.

Another difference between uniaxial testing and the actual forces created in the Northridge earthquake involves the effects of the rate of load application. Uniaxial tensile specimens are slowly loaded as compared to, for example, impact specimens. As the loading rate increases, uniaxial tensile specimens will exhibit an increase in yield strength. The Northridge earthquake applied a dynamic load at a very high rate of speed. Applying this to the Northridge structures, the apparent yield strength that would be delivered by the steel was increased further. A 20% increase in yield strength due to high loading rates is commonplace.

A final design assumption was made regarding the through-thickness properties of the column. Steel exhibits its greatest yield strength, tensile strength, and elongation in the direction of rolling, the so-called X direction. Across the width of the material, the Y direction, these properties are reduced. The greatest reduction occurs in the thickness perpendicular to rolling, the Z axis. This is the plane where nonmetallic inclusions, if present, are flattened. The least amount of ductility is seen in this direction. It is this Z axis property in the column flange that must transfer the energy of the moment connections into the column.

Ductility of a material can only be exhibited in relatively smooth, notch-free configurations. In the presence of a notch, even a uniaxial tensile specimen will exhibit an increase in the apparent yield strength, and a greatly reduced elongation. As configured in the moment connection, geometric notches naturally occur as the horizontal beam flange meets the vertical column flange. The problem is exacerbated by the presence of a properly fused backing bar. It is further compounded by any regions of lack of fusion or slag inclusions in the weld, and by poorly cut weld access holes. Under these conditions, ductile welds and ductile steel will not be able to exhibit ductility.

Notch toughness, or the ability of a material to absorb energy in the presence of a notch or crack, is also affected by geometric variables. Triaxial stresses decrease the apparent notch toughness of a material, and increased loading rates similarly decrease the apparent notch toughness. The rate of load application in Northridge, and the in-phase horizontal and vertical accelerations, significantly reduced the ability of the material to absorb the seismic energy.

CONCLUSION

In summary, the SMRF connections did not behave as expected due to unique loading effects, and a deficient structural design. If the answer is so simple, then why has there been so much confusion surrounding the cause of damaged connections in Northridge, confusion that continues to this day? The answer lies in the above analysis. Northridge serves as yet another reminder that a thorough understanding of materials engineering is essential to effective structural design. To a list of problems that includes the steel with poor notch toughness that was used in Liberty ships, and the infamous O-Rings with low sealing properties at lower temperatures in space shuttles, we can now add the incorrect design assumptions which were a contributing factor in the damage to steel structures caused by the Northridge earthquake.

Perhaps due to the superior performance associated with steel structures in past earthquakes, much of the available research money has been applied to concrete systems. Now, it is clear that steel construction warrants increased research investment. The conditions that led to column fracture must be identified, and the designs changed to eliminate this possible condition. Because there were no deaths and no collapses in Northridge's steel framed buildings, it is imperative that we continue to improve this technology.