conference preview

REVISITING W1 INDICATIONS

BY DUANE MILLER, P.E.

Looking back can reveal how different our final conclusions are from our initial thoughts.

TWENTY YEARS AGO, the Northridge earthquake imposed its damage on buildings, bridges and pipelines in southern California.

The damage associated with the welded connections in steel moment frames was extensively publicized. Early reports indicated that "a 60% to 80% connection failure rate occurred in some moment frames." The number of damaged steel-framed buildings started with early reports of 12 to 20, then more than 40 and eventually reached over 100. The "Northridge problem" appeared immense, affecting a substantial number of buildings in a significant way. Responsible engineers and organizations properly began to look for solutions to the problem.

The early investigations quickly showed widespread anomalies in the weld root, conditions that would eventually be called W1s. These commonly occurring non-conforming conditions were not well understood in the early post-Northridge days and prompted many well-intended, albeit misdirected, attempts to mitigate the formation of W1 indications in new construction. Unfortunately, 20 years later, some of these misunderstandings persist, resulting in more well-intended but expensive measures that add little or no value.

As news of damage to special moment resisting frames (SMRFs) began to emerge, different crack patterns and characteristics became evident. Very quickly, investigators developed a system to classify joint damage that permitted consistent reporting of results between various inspection organizations, engineering firms and repair contractors. Importantly, the common classification system allowed observations made by different individuals to be uniformly introduced into a data base for future analysis.

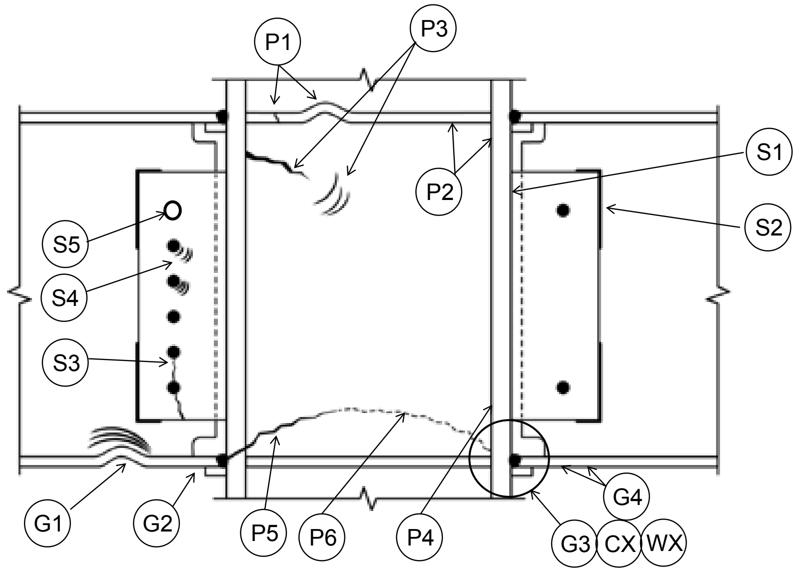
Inspection forms and sketches were developed using this common terminology (although it took some time before common formats and descriptions were fully established). Damage associated with columns received a prefix of "C" followed by a number. Similarly, panel zone damage was designated with a prefix of "P" and weld damage with a "W." W2 denoted a "full or partial crack through weld metal," W3 described "fracture at girder interface" and W4 was used for "fracture at column interface." In the earliest post-earthquake investigations, it was observed that the analysis of "a few damaged welds reveals that only half of the bottom flange has cracked." The term "only half" refers to half the thickness of the weld, not half the width of the weld. The classification W1 was used to identify this condition of an "incipient weld crack."

Duane Miller (duane_miller@lincolnelectric.com) is manager of engineering services with The Lincoln Electric Company.

Northridge Damage: Early Views

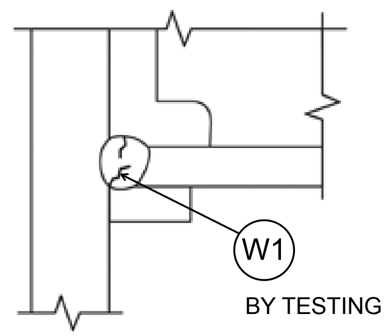
From the earliest post-Northridge days, a fundamental question arose: Was the Northridge damage a result of poor workmanship or some other issue?" Without question, the stress concentrations caused by W1s would diminish the fracture resistance of the connections. It was unknown, however, whether the pre-Northridge connection could be made to work if W1s were eliminated. Even though the answer to this question was unknown, there was great interest in eliminating W1s.

When the FEMA 267 report ("Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures") was issued in August 1995, sufficient investigation had been done to answer this question. According to that report, "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…substandard welding…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 cause of the fractures which occurred."



◀ A typical Northridge survey form illustration, denoting potential forms of connection damage. The general designation "WX" in the lower right corner was used to identify various types of weld damage, including W1s. (Adapted from SAC 94-01.)

▼ The details of W1s as shown on inspections forms. W1s could be detected only "by testing." (Adapted from SAC 94-01.)



While it had been established that W1s were not required to initiate the fractures, it was still unknown whether the pre-Northridge connection could be made to work. Two camps existed, as described in FEMA 267: "Some engineers...have suggested 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… would further prevent ductile behavior of these joints regardless of the procedure used to make the welds." The report properly concluded the following: "To date, there has not been sufficient research conducted to resolve this issue."

A subtle but important distinction was made in FEMA 267: The description of W1s was changed from "incipient weld cracks" to "weld root indications." This was a significant change, reflecting the reality that ultrasonic testing (UT) could not identify "cracks" but only "indications." An indication, of course, could be due to a crack (in the weld, or in the fusion zone, or in the heat-affected zone), but could also be the result of incomplete fusion or lamellar tearing. The more generic term "indication" was more accurate.

FEMA 267 also separated W1s into two subdivisions: W1a and W1b. A W1a was defined as incipient indication with a depth of up to 3⁄16 in. or $t_f/4$, and a width of less than $b_f/4$ in., while a W1b was a root indication larger than a W1a. This subdivision was used (in part) to separate pre-existing conditions from earthquake damage, as can be seen in this statement: "Some engineers believe that type W1a indications are not earthquake damage at all, but rather, previously undetected defects from the original construction process. A W1b indication is one that exceeds these limits but is not clearly characterized by one of the other types. It is more likely that W1b indications are the result of the earthquake than the construction process."

While UT was capable of detecting W1s, it was impossible for UT to distinguish between an earthquake-induced crack and a crack that occurred during erection. Further, it is difficult to determine if a weld root crack is in the weld, or in the fusion zone, or in the heat affected zone (HAZ) or whether it is lamellar tearing in the base metal. Finally, incomplete fusion in the root of the weld may appear very much like a crack to the UT technician.

On one level, the distinction was immaterial: UT was detecting a planar defect in the root of the weld, one that created a stress raiser from which cracking could be expected to extend if enough tensile force was applied during a major earthquake. On another level, however, the distinction between the types of root conditions that were being observed by UT was critical since the causation for each of the named conditions was different. Most importantly, the cures for the various conditions are different as well. Unfortunately, UT inspection of in-situ connections that contained W1s made this determination impossible.

A review of documents produced in the first year after the earthquake reveal a pattern of recommendations that were based on the assumption that W1s were due to hydrogen-related HAZ cracking. Accordingly, recommendations were made to preclude such cracking, such as increased preheat, slow cooling after welding, and post weld heat treatments—all based on the assumption that W1s were due to cracking during the original construction process.

SAC Topical Investigations

The SAC topical investigations sought to identify the causation of the Northridge fractures, as well as to determine what must be done to preclude such fractures in the future. W1s were the primary focus of one investigation, performed by Terrence F. Paret. The goal of his report "Clarifying the Extent of Northridge-Induced Weld Fracturing; Examining the Related Issue of UT Reliability" was to provide "more meaningful data about W1s." This was done by examining welds from damaged and undamaged buildings. Paret came to four conclusions, three of which are directly related to W1s (emphasis added):

1. *W1s are a result of poor welding* and inspection practices during construction, not a result of earthquake ground motions.

2. Ultrasonic inspection, as normally employed by testing laboratory personnel, *is not a reliable inspection technique* for identifying defects in the roots of welded full penetration "T" joints with backing.
3. The extent of earthquake damage to welded steel moment frame (WSMF) buildings is *substantially less* than has previously been reported.

The data supporting these conclusions included the observations from W1bs that were found to contain "only areas of non-fusion and slag, without any crack extension or other potentially earthquake-related conditions." It is noteworthy that this observation was made from W1bs, previously thought to be more likely due to actual earthquake damage. It is also interesting to note the similarity of this observation to the March 1994 report of damaged connections that "some welds appear to have been cracked prior to the earthquake. These cracks have been identified through the presence of rust in the weld crack." The issue of whether W1s were earthquake-generated cracks or original construction cracks was resolved: W1s were not cracks at all. Thus, Paret provides two distinct observations: W1s were pre-existing defects (not earthquake damage) and W1s were due to incomplete fusion and slag inclusions (not cracks).

Although W1s were not earthquake damage, they were nevertheless widespread and were not ignored. Further, while W1 indications were not necessary to cause connection fracture, it was also true that connections with W1 indications did not always fracture. The data from 209 steel buildings inspected after the earthquake was evaluated. As contained in Figure 6 of Paret's report, in general terms, approximately one-third of all inspected buildings had no damage, one-third contained only W1 indications and one-third had non-W1 damage. When the W1 indications were classified as earthquake damage, two-thirds of the 209 inspected buildings had damage, justifying the claim of "over a hundred" damaged buildings. On the other hand, with the conclusion that W1s were not earthquake related damage, the number of earthquake damaged buildings dropped to less than 100.

While some were speculating that W1s were cracks, others were predisposed to suspect the problem was incomplete fusion and slag inclusions, as Paret would eventually conclude. AWS D1.1 *Structural Welding Code*—Steel was reviewed in light of the Northridge findings, searching for potential areas for code improvement.

In AWS D1.1-94 (as well in previous versions), the maximum layer thickness for welds made with flux-cored arc welding (FCAW) using prequalified welding procedure specifications (WPS) was governed by clause 4.14.1.5 which states: "The thickness of the weld layers in groove welds, except root and surface layers, shall not exceed ¼ in. (6 mm)." The purpose of this clause is to restrict the overall weld bead thickness for prequalified WPSs. Other clauses provide similar restrictions on weld bead widths. The applicability of clause 4.14.1.5, however, did not extend to weld root passes. Accordingly, work done in accordance with D1.1-94 at least theoretically allowed for root pass thicknesses of any dimension.

In AWS D1.1-96 (the first edition after the Northridge earthquake), a new limit of ⅜ in. (10 mm) was imposed on the root pass thickness for prequalified FCAW WPSs made in the flat position. This limit was one step taken to assist in overcoming the conditions that lead to W1 indications.

Other clauses in D1.1, in effect prior to the 1994 earthquake, required uniform fusion between passes and to the base metal, including a specific requirement for fusion to left-in-place steel backing. The change to D1.1 in 1996 to control root pass thickness was not required to cause rejection of the welds that contained W1s; rather, the changes were made to promote WPSs that would be more likely to result in welds that were free of W1s.

Conclusions

FEMA 353 ("Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications") contained the definitive conclusion of the SAC investigations: "The typical moment-resisting connection detail employed in steel moment-frame construction prior to the 1994 Northridge earthquake… had a number of features that rendered it inherently susceptible to brittle fracture." As a result, a variety of alternate moment connection details are used today, including as an example the reduced beam section (RBS) detail. Eliminating W1 indications was not enough to the fix the "Northridge problem."

In some circles, the misunderstanding of W1s appears to persist to this day. While AISC specifications and D1 codes (including AWS D1.8 *Seismic Welding Supplement*) found no justification for changes in preheat levels, interpass temperatures, regulation of cooling rates or post-weld heat treatments, job-specific specifications still invoke recommendations apparently based on the some of the earliest recommendations that assumed W1s represented weld root cracks. Today, 20 years after Northridge, citation of the appropriate AISC prequalified connection detail and the AISC *Seismic Specifications* alone, without any special job-specific specifications, is typically sufficient to govern most moment frame construction.

A final comment on the Northridge record: There continues to be a widespread belief that the damage to steel-framed structures was also widespread, involving hundreds of structures. As recently as 2004, the steel damage in Northridge was still being cited as having affected "about 200 buildings." The facts suggest something much different: approximately two-thirds of the reported "damage" was really W1 and the actual earthquake damage was concentrated in a relatively small number of buildings. ■

*This article serves as a preview of Session C3, "Revisiting W1 Indications" at NASCC: The Steel Conference, taking place March 26-28 in Toronto. Learn more about the conference at **www.aisc.org/nascc**.*