3. CLASSIFICATION AND IMPLICATIONS OF DAMAGE

3.1 Summary of Earthquake Damage

There are no modifications to the Guidelines or Commentary of Section 3.1 at this time.

3.2 Damage Types

There are no modifications to the Guidelines or Commentary of Section 3.2 at this time.

3.2.1 Girder Damage

There are no modifications to the Guidelines or Commentary of Section 3.2.1 at this time.

3.2.2 Column Flange Damage

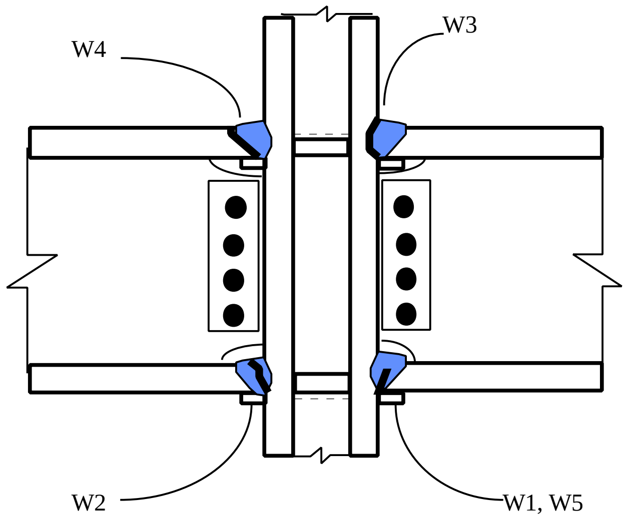
There are no modifications to the Guidelines or Commentary of Section 3.2.2 at this time.

3.2.3 Weld Damage, Defects and Discontinuities

Six types of weld discontinuities, defects and damage are defined in Table 3-3 and illustrated in Figure 3-4. All apply to the complete joint penetration (CJP) welds between the girder flanges and the column flanges. ~~This category of damage was the most commonly reported type f~~Following the Northridge Earthquake, many instances of W1a and W1b conditions were reported as damage. These conditions, which are detectable only by ultrasonic testing or by removal of weld backing, are now thought more likely to be construction defects than damage.

Table 3-3 - Types of Weld Damage, Defects and Discontinuities

Type	Description
W1	Weld root indications
W1a	Incipient indications ~~-~~– depth <3/16" or $t_f/4$; width < $b_f/4$
W1b	Root indications larger than that for W1a
W2	Crack through weld metal thickness
W3	Fracture at column interface
W4	Fracture at girder flange interface
W5	UT detectable indication - non-rejectable



Note: See Figure 3-2 for related column damage and Figure 3-3 for girder damage

Figure 3-4 - Types of Weld Damage

Commentary: Despite significant controversy, type W1 and W5 discovered in buildings following the Northridge earthquake, were commonly reported as damage. These small discontinuities and defects located at the roots of the CJP welds are detectable only by ultrasonic testing (UT) when the weld backing is left in place or by visual testing (VT) or magnetic particle testing (MT) when weld backing is removed. It now seems likely that most such conditions are not damage at all, but rather, are pre-existing construction defects. A number of factors point to this conclusion. First, statistical surveys of damage sustained by buildings in the Northridge earthquake show that if type W1 and W5 conditions are not considered, there was a much greater incidence of damage in frames resisting north-south ground shaking than in frames resisting east-west shaking. This appears to be correlated with the relative strength of the ground shaking experienced along these two directional axes. However, there is no significant difference between the incidence rate of reported W1 and W5 conditions in these two directions, suggesting that these conditions are not correlated with shaking intensity.

The discovery of W1 conditions in welds for which original construction quality assurance documentation is available, indicating that no such defects were present when the building was originally constructed, tends to contradict this argument. However, investigations conducted by SAC under the Phase 2 project have indicated that as a result of the joint geometry, UT techniques are often unable to detect W1 conditions at the weld root, when scanning of the joint is conducted from the top surface of the beam bottom flange. It is important to note that this is the most common method of conducting UT as part of construction quality assurance. When UT scanning of a joint is conducted from the bottom surface of the flange, as is commonly done when inspecting for *earthquake damage, it becomes more likely that such conditions will be detected, since the geometric constraints present for top flange scanning are altered. This leads to the conclusion that it is probable that typical construction quality assurance UT of welded joints would be likely to miss W1 conditions, allowing them to be discovered in later post-earthquake surveys.*

When FEMA-267 was first published, it was recommended that W1 conditions be treated as damage and that UT be used as a routine part of the post-earthquake investigation process, in order to discover these conditions. However, more recent investigations conducted by SAC have revealed that even the careful scanning typically conducted as part of a post-earthquake inspection is not able to reliably detect these conditions. Given that it is both expensive and difficult to locate W1 conditions as part of a post-earthquake investigation, and also, that most of these conditions are unlikely to be damage at all, it is no longer recommended that exhaustive investigations for these conditions be conducted as part of the earthquake damage investigation process.

~~*Type W1 damage, discontinuities and defects and type W5 discontinuities are detectable only by NDT, unless the backing bar is removed, allowing direct detection by visual inspection or magnetic particle testing. Type W5 consists of small discontinuities and may or may not actually be earthquake damage. AWS D1.1 permits small discontinuities in welds. Larger discontinuities are termed defects, and are rejectable per criteria given in the Welding Code. It is likely therefore that some weld indications detected by NDT in a post-earthquake inspection may be discontinuities which pre-existed the earthquake and do not constitute a rejectable condition, per the AWS standards. Repair of these discontinuities, designated as type W5 is not generally recommended. Some type W1 indications are small planar defects, which are rejectable per the AWS D1.1 criteria, but are not large enough to be classified as one of the types W2 through W4. Type W1 is the single most commonly reported non-conforming condition reported in the post-Northridge statistical data survey, and in some structures, represents more than 80 per cent of the total damage reported. The W1 classification is split into two types, W1a and W1b, based on their severity. Type W1a "incipient" root indications are defined as being nominal in extent, less than 3/16" deep or 1/4 of the flange thickness, whichever is less, and having a length less than 1/4 of the flange width. 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 a result of the earthquake than the construction process.*~~

~~*As previously stated, some engineers believe that both type W1a and some type W1b conditions are not earthquake related damage at all, but instead, are*~~

~~*rejectable conditions not detected by the quality control and assurance programs in effect during the original construction. However, in recent large-scale sub-assembly testing of the inelastic rotation capacity of girder-column connections conducted in SAC Phase 1 at the University of Texas at Austin and the Earthquake Engineering Research Center of the University of California at Berkeley, it was reported that significantly more indications were detectable in unfailed CJP welds following the testing than were detectable prior to the test. This tends to indicate that type W1 damage may be related to stresses induced in the structures by their response to the earthquake ground motions. Regardless of whether or not type W1 conditions are directly attributable to earthquake response, it is clear that these conditions result in a reduced capacity for the CJP welds and can act as stress risers, or notches, to initiate fracture in the event of future strong demands.*~~

Type W2 fractures extend completely through the thickness of the weld metal and can be detected by either MT or VI techniques. Type W3 and W4 fractures occur at the zone of fusion between the weld filler metal and base material of the girder and column flanges, respectively. All three types of damage result in a loss of tensile capacity of the girder flange to column flange joint and should be repaired.

As with girder damage, damage to welds has most commonly been reported at the bottom girder to column connection, with fewer instances of reported damage at the top flange. Available data indicates that approximately 25 per cent of the total damage in this category occurs at the top flange, and most often, top flange damage occurs in connections which also have bottom flange damage. For the same reasons previously described for girder damage, less weld damage may be expected at the top flange. However, it is likely that there is a significant amount of damage to welds at the top girder flange which have never been discovered due to the difficulty of accessing this joint. Later sections of these Interim Guidelines provide recommendations for situations when such inspection should be performed.

3.2.4 Shear Tab Damage

There are no modifications to the Guidelines or Commentary of Section 3.2.4 at this time.

3.2.5 Panel Zone Damage

There are no modifications to the Guidelines or Commentary of Section 3.2.5 at this time.

3.2.6 Other Damage

There are no modifications to the Guidelines or Commentary of Section 3.2.6 at this time.

3.3 Safety Implications

The implications of the damage described above with regard to building safety are discussed in this section. As part of the SAC Phase 2 program, extensive nonlinear analyses have been conducted of WSMF buildings to determine the effects of connection fractures on building performance and also to develop an understanding of the risk of earthquake-induced building collapse. These studies indicate that risk of collapse of WSMF buildings designed to modern standards and having connections capable of ductile behavior is quite low. Even in regions of very high seismicity, such as those areas of coastal California adjacent to major active faults, the probability that such a building would experience earthquake-induced collapse appears to be on the order of one occurrence per building, every 20,000 years. For buildings that have brittle connections such as those commonly constructed prior to 1994, the probability of collapse increases somewhat. If only the bottom flange connections of beams to columns is subject to fracture, the risk of global collapse of buildings increases to perhaps one occurrence in 15,000 years, presuming that the fractures do not jeopardize column capacity. However, if both flanges of the connections are subject to fracture, or if substantial column damage occurs, the risk of collapse increases significantly. Also, it is important to note that severe connection fractures can result in significant risk of local collapse and life safety endangerment.

While these studies have been helpful in providing an understanding of the level of risk inherent in WSMF structures with brittle connections, they do not provide sufficient information to ~~There is insufficient knowledge at this time to permit determination of the~~ assess the degree of risk with any real confidence. However, based on the historic performance of modern WSMF buildings, typical of those constructed in the United States, it appears that the risk of collapse in moderate magnitude earthquakes, ranging up to perhaps M7, is very low for buildings which have been properly designed and constructed according to prevailing standards. A possible exception to this may be buildings located in the near field (< 10 km from the surface projection of the fault rupture) of such earthquakes (Heaton, et. al. - 1995), however, this is not uniquely a problem associated with steel buildings. Our current building codes in general, may not be adequate to provide for reliable performance of buildings within the near field of large earthquakes. As is also the case with all other types of construction, buildings with incomplete lateral force resisting systems, severe configuration irregularities, inadequate strength or stiffness, poor construction quality, or deteriorated condition are at higher risk than buildings not possessing these characteristics.

No modern WSMF buildings have been sited within the areas of very strong ground motion from earthquakes larger than M7, or for that matter, within the very near field for events exceeding M6.5. This style of construction has been in wide use only in the past few decades. Consequently, it is not possible to state what level of risk may exist with regard to building response to such events. This same lack of performance data for large magnitude, long duration events exists for virtually all forms of contemporary construction. Consequently, there is considerable uncertainty in assigning levels of risk to any building designed to minimum code requirements for these larger events.

Commentary~~: Research conducted to date has not been conclusive with regard to the risk of collapse of WSMF buildings. Some testing of damaged connections from a building in Santa Clarita, California have been conducted at the University of Southern California (Anderson - 1995). In these tests, connection assemblies which had experienced type P6 damage were subjected to repeated cycles of flexural loading, while the column was maintained under axial compression. Under these conditions, the specimens were capable of resisting as much as 40 per cent of the nominal plastic strength of the girder for several cycles of slowly applied loading, at plastic deformation levels as large as 0.025 radians. However, damage did progress in the specimen, as this testing was performed. It is not known how these assemblies would have performed if the columns were permitted to experience tensile loading. Data from other tests suggests that the residual strength of connections which have experienced types G1, G4, W2, W3, and W4 damage is on the order of 15 per cent of the undamaged strength. Some analytical research (Hall - 1995) in which nonlinear time history analyses simulating the effects of connection degradation due to fractures were included, indicates that typical ground motions resulting in the near field of large earthquakes can cause sufficient drift in these structures to induce instability and collapse. Other researchers (Astaneh - 1995) suggest that damaged structures, even if unrepaired, have the ability to survive additional ground motion similar to that of the Northridge Earthquake.~~

Even though there were no collapses of WSMF buildings in the 1994 Northridge Earthquake, it should not be assumed that no risk of such collapse exists. Indeed, a number of WSMF buildings did experience collapse in the 1995 Kobe Earthquake. The detailing of these collapsed Japanese buildings was somewhat different than that found in typical US practice, however, much of the fracture damage that occurred was similar to that discovered following the Northridge event.

Because of a lack of data and experience with the effects of larger, longer duration earthquakes, there is considerable uncertainty about the performance of all types of buildings in large magnitude seismic events. It is believed that seismic risks in such large events are highly dependent on the individual ground motion at a specific site and the characteristics of the individual buildings. Therefore, generalizations with regard to the probable performance of individual types of construction may not be particularly meaningful.

The risks to occupants of WSMF buildings with brittle connections is regarded as less, in most cases, than to occupants of the types of buildings listed below. However, because of the uncertainties involved, the degree of risk in large events cannot be definitively quantified, nor can it categorically be stated that properly constructed WSMF buildings sited in the near field of large events are either *more or less at risk than many other code designed building systems which do not appear on the following list:*

- *Concentric braced steel frames with bracing connections that are weaker than the braces*
- *Knee braced steel frames*
- *Unreinforced masonry bearing wall buildings*
- *Non-ductile reinforced concrete moment frames (infilled or otherwise)*
- *Reinforced concrete moment frames with gravity load bearing elements that were not designed to participate in the lateral force resisting system and that do not have capacity to withstand earthquake-induced deformations*
- *Tilt-up and reinforced masonry buildings with inadequate anchorage of their heavy walls to their horizontal wood diaphragms*
- *Precast concrete structures without adequate interconnection of their structural elements.*

In addition, WSMF structures <u>with brittle connections</u> would appear to have lower inherent seismic risk than structures of any construction type that:

- *do not having complete, definable load paths*
- *have significant weak and/or soft stories*
- *have major torsional irregularity and insufficient stiffness and strength to resist the resulting seismic demands*
- *minimal redundancy and concentrations of lateral stiffness*

These are general statements that represent a global view of system performance. As with all seismic performance generalizations, there are many steel moment frame buildings that are more vulnerable to damage than some individual buildings of the general categories listed, just as there are many that will perform better.

3.4 Economic Implications

There are no modifications to the Guidelines or Commentary of Section 3.4 at this time.

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