3. CLASSIFICATION AND IMPLICATIONS OF DAMAGE

A broad range of damage and defects were found in steel moment resisting connections, following the Northridge Earthquake. Communication between engineers and technicians was often confused, due to the use of different terminology for reporting these conditions. To avoid such confusion, a uniform system for damage classification is presented in this Chapter, and referenced throughout these Interim Guidelines. The implication of each damage type is also discussed.

Some reported damage in WSMF buildings; i.e. local buckling and yielding, was consistent with the expected behavior of these structures. However, the widespread brittle fractures which occurred were inconsistent with previous expectations. This calls into question the ability of existing WSMF structures to provide adequate protection of life safety in major earthquakes.

There has never been an earthquake-induced collapse of a WSMF building in the United States. However, the severe damage experienced by some WSMF buildings in the Northridge Earthquake suggests that collapse is credible given the right combination of building characteristics and ground motion. Based on historic evidence, it seems unlikely that earthquakes with magnitudes less than approximately 7 would produce such ground motion, except in the very near field. However, larger events could produce such ground motions over large regions. Therefore, new moment frame buildings should not continue to be designed and constructed using traditional methods.

The risk associated with existing WSMF structures should be assessed in comparison with other construction types. Many other types of buildings have occasionally experienced collapse in past moderate earthquakes. Therefore, it seems unwarranted to mandate upgrades of existing WSMF structures prior to addressing these other building types. However, some building owners may wish to perform such upgrades in order to reduce the risks associated with individual buildings.

The repair costs associated with some WSMF structures, following the Northridge Earthquake, were substantially higher than would previously have been projected. Statistical analysis of data collected on damaged buildings has been used to project, with varying levels of confidence, the likely repair costs for such structures, when subjected to ground motion of different severities. A summary of these statistics is presented to permit estimation of the probable repair costs for WSMF buildings that have experienced different levels of ground motion.

3.1 Summary of Earthquake Damage

Following the Northridge Earthquake, structural damage observed in Los Angeles area WSMF buildings included yielding, buckling and excessive fracturing of the steel framing elements (beams and columns) and their connections, as well as permanent lateral drift in some structures. Damaged elements included girders, columns, column panel zones (including girder flange continuity plates and column web doubler plates), the welds of the beam to column flanges and the shear tabs which connect the girder webs to column flanges. There has been speculation that column splices and base plates would also be subject to fracture damage, however no instance of such damage has been reported in WSMF buildings damaged by the Northridge Earthquake. There have been reports of such damage in buildings affected by the 1995 Kobe (Great Hanshin), Japan earthquake. Figure 3-1 illustrates the location of these elements.

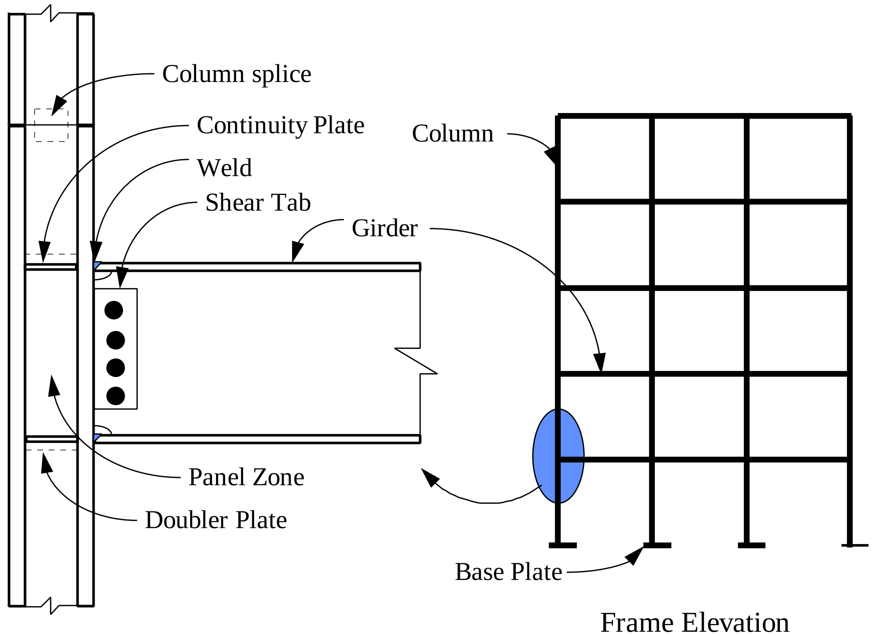


Figure 3-1 - Elements of Welded Steel Moment Frame

3.2 Damage Types

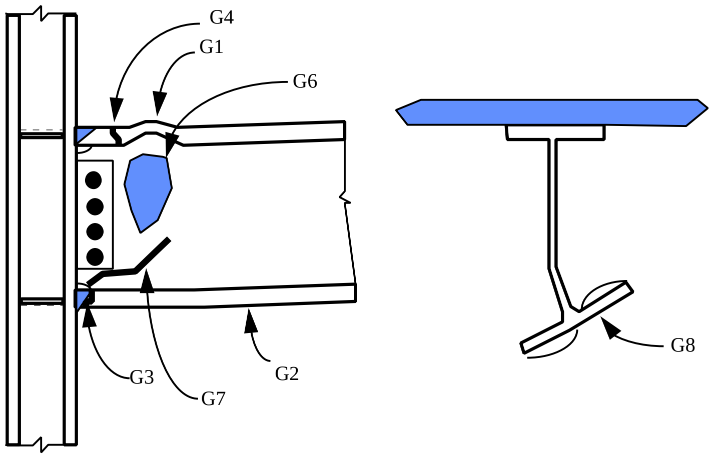
Damage to framing elements of WSMFs may be categorized as belonging to the weld (W), girder (G), column (C), panel zone (P) or shear tab (S) categories. This section defines a uniform system for classification and reporting of damage to elements of WSMF structures, that is utilized throughout these Interim Guidelines. The damage types indicated below are not mutually exclusive. A given girder-column connection may experience several types of damage simultaneously. In addition to the individual element damage types, a damaged WSMF may also exhibit global effects, such as permanent interstory drifts.

Following a post-earthquake inspection, classification of the damage found, as to its type and degree of severity is the first step in performing an assessment of the condition and safety of a damaged WSMF structure. In Section 4 these classifications are used for the assignment of damage indices. These damage indices are statistically combined and extrapolated to provide an indication of the severity of damage to a structure’s lateral force resisting system and are used as a basis for selecting building repair strategies. Section 6 addresses specific techniques and design criteria recommended for the repair and modification of the different types of damage.

Commentary: The damage types contained in this Chapter are based on a system first defined in a statistical study of damage reported in NISTR-5625 (Youssef et. al.- 1994). The original classes contained in that study have been expanded somewhat to include some conditions not previously identified. Damage classes have not been standardized within the profession, and many individual engineers and inspection agencies engaged in the inspection and repair of structures damaged by the Northridge Earthquake have used other terminologies. It is recommended that the definitions given below be adopted as the uniform standard for reporting and classifying damage in the future. This will provide a common basis for communication as well as enhance the ability to develop an understanding of the performance of WSMF structures in earthquakes.

3.2.1 Girder Damage

Girder damage may consist of yielding, buckling or fracturing of the flanges of girders at or near the girder-column connection. Eight separate types are defined in Table 3-1. Figure 3-2 illustrates these various types of damage. See section 3.2.3 and 3.2.4 for damage to adjacent welds and shear tabs, respectively.



Note: condition G5 consists of types G3 and/or G4 damage occurring at both the top and bottom flanges.

Figure 3-2 - Types of Girder Damage

Table 3-1 - Types of Girder Damage

Type	Description
G1	Buckled flange (top or bottom)
G2	Yielded flange (top or bottom)
G3	Flange fracture in HAZ (top or bottom)
G4	Flange fracture outside HAZ (top or bottom)
G5	Flange fracture top and bottom
G6	Yielding or buckling of web
G7	Fracture of web
G8	Lateral torsional buckling of section

Commentary: Minor yielding of girder flanges (type G2) is the least significant type of girder damage. It is often difficult to detect and may be exhibited only by local flaking of mill scale and the formation of characteristic visible lines in the material, running across the flange. Removal of finishes, by scraping, may often obscure the detection of this type of damage. Girder flange yielding, without local buckling or fracture, results in negligible degradation of structural strength and typically need not be repaired.

Girder flange buckling (type G1) can result in a significant loss of girder plastic strength. For compact sections, this strength loss occurs gradually, and increases with the number of inelastic cycles and the extent of the inelastic excursion. Following the initial onset of buckling, additional buckling will often occur at lower load levels and result in further reductions in strength, compared to previous cycles. The localized secondary stresses which occur in the girder flanges due to the buckling can result in initiation of flange fracture damage (G4). Once this type of damage occurs, the girder flange may rapidly loose all tensile capacity under continued or reversed loading, however, it may retain some capacity in compression. Visually evident girder flange buckling should be repaired.

With the conventional structural steels used in WSMF buildings, girder flange cracking within the HAZ (type G3) is most likely to occur at connections in which improper welding procedures were followed, resulting in local embrittlement of the HAZ. Like the visually similar type G4 damage, it results in a complete loss of flange tensile capacity, and consequently, significant reduction in the contribution to frame lateral strength and stiffness from the connection. Little G4 or G5 damage was actually seen in buildings following the Northridge Earthquake. In some cases, this damage was found to extend from the weld access hole in the web of the girder, a metallurgically complex area, into the flange. As shown in Figure 3-2, this damage occurs at a location of local flange buckling, which is where it has been observed in some testing of large-scale assemblies, after many cycles of load.

In the Northridge Earthquake girder damage has most commonly been detected at the bottom flanges, although some instances of top flange failure have also been reported. There are several apparent reasons for this. First the composite action induced by the presence of a floor slab at the girder top flange, tends to shift the neutral axis of the beam towards the top flange. This results in larger tensile deformation demands on the bottom flange than on the top. In addition, the presence of the slab tends to greatly reduce the chance of local buckling of the top flange. The bottom flange, however, being less restrained can experience buckling relatively easily. Preliminary large-scale testing conducted by SAC included specimens without a slab present. Flange fractures in these specimens tended to occur randomly, sometimes at the top and sometimes at the *bottom flange, somewhat confirming that the slab may have significant influence on connection behavior.*

There are a number of other factors that could lead to the greater incidence of bottom flange fractures observed in the field. The location of the weld backing is one of the most important of these. At the bottom flange joint, the backing is located at the extreme tension fiber, while at the top flange, it is located at a point of lesser stress and strain demand, both due to the fact that it is located on the inside face of the flange and because the floor slab tends to alter the section properties. Therefore, any notch effects created by the backing are more severe at the bottom flange. Another important factor is that welders can typically make the CJP weld at the girder top flange without obstruction, while the bottom flange weld must be made with the restriction induced by the girder web. Also the welder typically has better and more comfortable access to the top flange joint. Thus, top flange welds tend to be of higher quality, and have fewer stress risers, which can initiate fracture. Finally, studies have shown that UT inspection of the top flange weld is more easily achieved than at the bottom flange, contributing to the better quality likely to occur in top flange welds.

3.2.2 Column Flange Damage

Seven types of column flange damage are defined in Table 3-2 and illustrated in Figure 3-3. Column damage typically results in degradation of a structure's gravity load carrying strength as well as lateral load resistance.. For related damage to column panel zones, refer to Section 3.2.5.

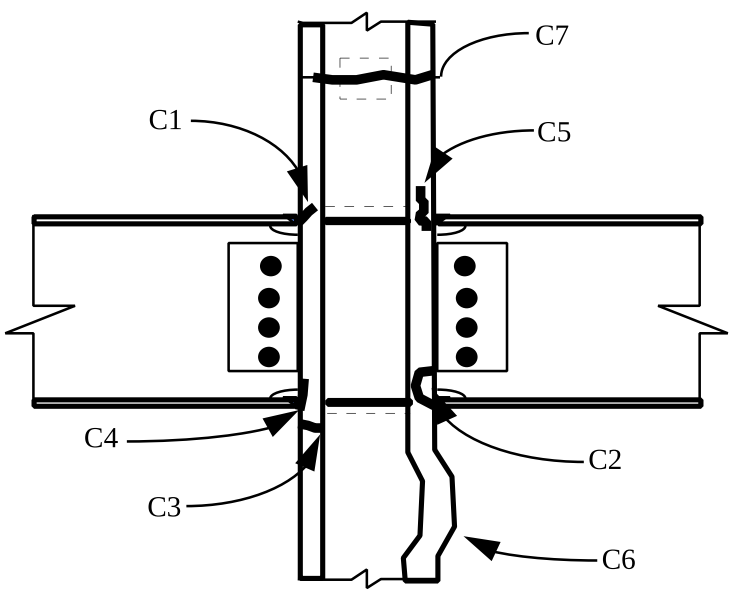


Figure 3-3 - Types of Column Damage

Table 3-2 - Types of Column Damage

Type	Description
C1	Incipient flange crack
C2	Flange tear-out or divot
C3	Full or partial flange crack outside HAZ
C4	Full or partial flange crack in HAZ
C5	Lamellar flange tearing
C6	Buckled flange
C7	Column Splice Failure

Commentary: Column flange damage includes types C1 through C7. Type C1 damage consists of a small crack within the column flange thickness, typically at the location of adjoining girder flange. C1 damage does not go through the thickness of the column flange and can be detected only by NDT, such as UT. Type C2 damage is an extension of type C1, in which a curved failure surface extends from an initiation point, usually at the root of the girder to column flange weld, and extends longitudinally into the column flange. In some cases this failure surface may emerge on the same face of the column flange where it initiated. When this occurs, a characteristic "nugget" or "divot" can be withdrawn from the flange. Types C3 and C4 fractures extend through the thickness of the column flange and may extend into the panel zone. Type C5 damage is characterized by a stepped shape failure surface within the thickness of the column flange and aligned parallel to it. This damage is often detectable only with the use of NDT.

Type C1 damage does not result in an immediate large strength loss to the column; however, such small fractures can easily progress into more serious types of damage if subjected to additional large tensile loading by aftershocks or future earthquakes. Type C2 damage results in both a loss of effective attachment of the girder flange to the column for tensile demands and a significant reduction in available column flange area for resistance of axial and flexural demands. Type C3 and C4 damage result in a loss of column flange tensile capacity and under additional loading can progress into other types of damage.

Type C5 damage may occur as a result of non-metallic inclusions within the column flange. The potential for this type of fracture under conditions of high restraint and large through-thickness tensile demands has been known for a number of years and has sometimes been identified as a contributing mechanism for type C2 column flange through-thickness failures. Many engineers have adopted a practice of specifying mandatory NDT investigation of column sections in the vicinity of girder-column connections, in accordance with ASTM A898, both before and after welding to detect type C5 discontinuities.

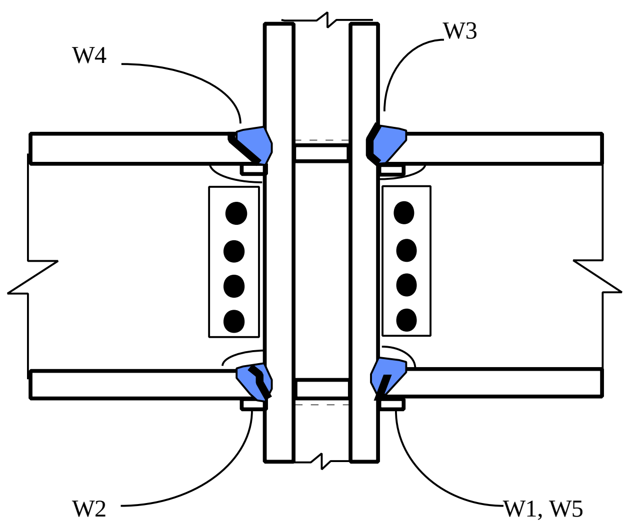
As a result of the potential safety consequences of complete column failure, all column damage should be considered as significant, and repaired accordingly.

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 CJP welds between the girder flanges and the column flanges. This category of damage was the most commonly reported type following the Northridge Earthquake.

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: 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

Eight types of damage to girder web to column flange shear tabs are defined in Table 3-4 and illustrated in Figure 3-5. Severe damage to shear tabs is often an indication that other damage has occurred to the connection including column, girder, panel zone, or weld damage.

Table 3-4 - Types of Shear Tab Damage

Type	Description
S1	Partial crack at weld to column
S1a	girder flanges sound
S1b	girder flange cracked
S2	Fracture of supplemental weld
S2a	girder flanges sound
S2b	girder flange cracked
S3	Fracture through tab at bolts or severe distortion
S4	Yielding or buckling of tab
S5	Loose, damaged or missing bolts
S6	Full length fracture of weld to column

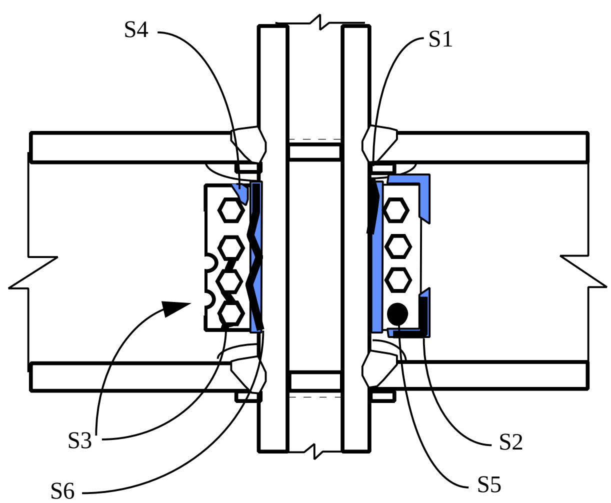


Figure 3-5 - Types of Shear Tab Damage

Commentary: Shear tab damage should always be considered significant, as failure of a shear tab connection can lead to loss of gravity load carrying capacity for the girder, and potentially partial collapse of the supported floor. Severe shear tab damage typically does not occur unless other significant damage has occurred at the connection. If the girder flange joints and adjacent base metal are sound, than they prevent significant differential rotations from occurring between the column and girder. This protects the shear tab from damage, unless excessively large shear demands are experienced. If excessive shear demands do occur, than failure of the shear tab is likely to trigger distress in the welded joints of the girder flanges.

3.2.5 Panel Zone Damage

Nine types of damage to the column web panel zone and adjacent elements are defined in Table 3-5 and illustrated in Figure 3-6. This class of damage can be among the most difficult to detect since elements of the panel zone may be obscured by beams framing into the weak axis of the column. In addition, the difficult access to the column panel zone and the difficulty of removing sections of the column for repair, without jeopardizing gravity load support, make this damage among the most costly to repair.

Table 3-5 - Types of Panel Zone Damage

Type	Description
P1	Fracture, buckle or yield of continuity plate
P2	Fracture in continuity plate welds
P3	Yielding or ductile deformation of web
P4	Fracture of doubler plate welds
P5	Partial depth fracture in doubler plate
P6	Partial depth fracture in web
P7	Full or near full depth fracture in web or doubler
P8	Web buckling
P9	Severed column

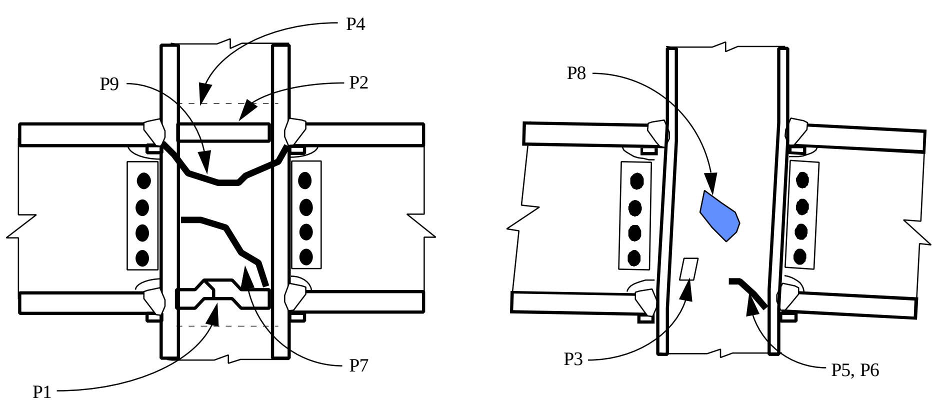


Figure 3-6 - Types of Panel Zone Damage

Commentary: Fractures in the welds of continuity plates to columns (type P2), or damage consisting of fracturing, yielding, or buckling of the continuity plates themselves (type P1) may be of relatively little consequence to the structure, so long as the fracture does not extend into the column material itself. Fracture of doubler plate welds (type P4) is more significant in that this results in a loss of effectiveness of the doubler plate and the fractures may propagate into the column material.

Although shear yielding of the panel zone (type P3) is not by itself undesirable, under large deformations such shear yielding can result in kinking of the column flanges and can induce large secondary stresses into the girder flange to column flange connection. In recent SAC Phase 1 testing at the University of California at Berkeley, excessive deformation of the column panel zone was identified as a contributing cause to the initiation of type W2 fractures at the top girder flange. It is reasonable to expect that such damage could also be initiated in real buildings, under certain circumstances.

Fractures extending into the column web panel zone (types P5, P6 and P7) have the potential under additional loading to grow and become type P9 resulting in a complete disconnection of the upper half of a column from the lower half, and are therefore potentially as severe as column splice failures. When such damage has occurred, the column has lost all tensile capacity and its ability to transfer shear is severely limited. Such damage results in a total loss of reliable seismic capacity. It appears that such damage is most likely to occur in connections that are subject to column tensile loads, and/or in which beam yield strength exceeds the yield strength of the column material.

Panel zone web buckling (type P8) may result in rapid loss of shear stiffness of the panel zone with potential negative effects as described above. Such buckling is unlikely to occur in connections which are stiffened by the presence of a vertical shear tab for support of a beam framing into the column's minor axis.

3.2.6 Other Damage

In addition to the types of damage discussed in the previous sections, other types of structural damage may also be found in WSMF buildings. Other framing elements which may experience damage include column base plates, beams, columns, and their connections that were not intended in the original design to participate in lateral force resistance, and floor and roof diaphragms. In addition, large permanent interstory drifts may develop in the structures. Based on observations of structures affected by the Northridge Earthquake, such damage is unlikely unless extensive damage has also occurred to the lateral force resisting system. When such damage is discovered in a building, it should be reported and repaired, as suggested by later sections of these Interim Guidelines.

3.3 Safety Implications

The implications of the damage described above with regard to building safety are discussed in this section. There is insufficient knowledge at this time to permit determination of 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 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 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 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

This section provides data which may be used to estimate probable repair costs for WSMF buildings conforming to typical pre-Northridge Earthquake design and construction practices, in the event that they are affected by future strong earthquake ground motion. This information may be considered, together with other data, when making investment decisions relative to such buildings, or when conducting cost-benefit studies to determine if structural upgrade of existing buildings is economically justified.

Economic losses resulting from earthquake induced building damage include directs costs resulting from inspection to determine the extent of damage, engineering design fees, actual costs related to the structural repairs, demolition and replacement costs for architectural finishes and utilities (that must be removed to allow access for inspection and repair), and repair of damaged non-structural components, as well as indirect costs resulting from loss of use, lost income from rents that are not collected on spaces vacated during the repair period, and project financing costs. The loss estimation data provided in this section only includes consideration of the direct damage repair costs. It does not include consideration of indirect costs related to lost rents, interruption of business and similar issues. These indirect costs often result in a greater economic impact than do the actual costs of repair, but are difficult to estimate on a general basis. Allowance for such indirect costs should be made in any economic analysis conducted for individual buildings.

The loss estimation data presented in this section is compatible with that presented in *ATC-13* (Applied Technology Council -1985), a document frequently used as the basis for loss estimation studies. In that document, vulnerability functions are presented for broad classes of buildings, based on the expert opinion of groups of individuals familiar with the performance of those structures. The vulnerability functions relate the expected repair costs, expressed as a percentage of building replacement value, to a ground motion parameter (Modified Mercalli Intensity), and a level of confidence.

Table 3-6 presents a proposed vulnerability function for WSMF buildings typical of California construction prior to the Northridge Earthquake. Each column of the table provides an estimate of the percentage of the total population of these buildings within a region affected by ground motion of defined intensity, expected to have repair costs "d", expressed as a percentage of building replacement value, within the indicated ranges. Figure 3-7 provides a plot of this data in a format which may be more useful for application to loss estimation estimates. The statistics contained in the table were calculated using a loss estimation model developed by Thiel and Zsutty (Thiel and Zsutty - 1987), and data obtained on the performance of 89 buildings affected by the Northridge Earthquake (Bonowitz and Youssef - 1995).

**Table 3-6 - Estimated Distribution of WSMF Buildings[1]
by Severity of Damage in Regions of Varying Ground Motion Intensity**

Damage d[2]	Modified Mercalli Intensity			
	VII	VIII	IX	X
d≤5%	71%	57%	40%	30%
5%<d≤25%	21%	29%	34%	35%
25%<d≤50%	7%	12%	20%	26%
50%<d≤75%	1%	2%	5%	8%
75%<d≤100%	0%	0%	1%	1%

1. WSMF buildings conforming to pre-Northridge Earthquake design and construction practice for regions of high seismicity (*UBC* seismic zones 3 and 4) {*NEHRP* Map Areas 6 and 7}.
2. "d' is the direct damage repair cost, expressed as a percentage of building replacement cost

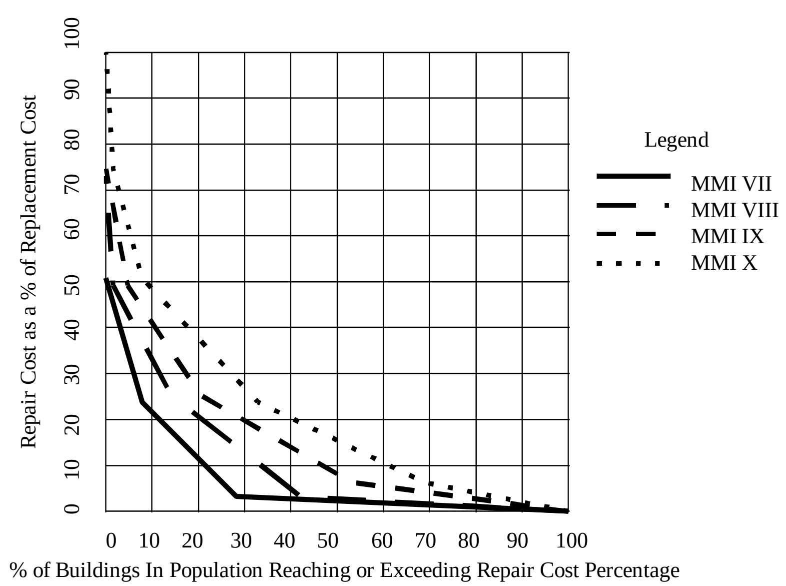


**Figure 3-7 - Vulnerability Estimates for WSMF Buildings
Conforming to Typical California Practice Prior to the Northridge Earthquake**

These loss estimation statistics should be used with caution, when applied to individual buildings. The unique characteristics of any individual building, including the strength and stiffness of its lateral force resisting system, its inherent redundancy, its condition, and the quality of its construction, will affect the relative vulnerability of the building. The statistics presented may be considered as representative of average buildings, in general conformance with the applicable building code provisions. Buildings that have substantial deficiencies relative to those provisions would be expected to be significantly more vulnerable. Similarly, buildings that have superior earthquake resisting characteristics, relative to the requirements of the building code, would be expected to be less vulnerable.

The statistics contained in Table 3-6 were established based on case studies conducted by SAC of the damage experienced by selected buildings affected by the Northridge Earthquake. It appears that typical repair costs for structural damage to connections can range from about $7,000 per connection to approximately $20,000. These costs are dominated not by the structural work, but rather by costs related to mobilizing into discrete areas of the building, performing local demolition of finishes and utilities as required to gain access and to create a safe working environment, and reconstruction of these finishes and utilities upon completion of the structural work. The cost of the structural work itself tends to vary from about $2,000 for the simplest repairs of damage (type W1 and W2) to perhaps $5,000 or more for repairs of the most complex types. These cost estimates do not include allowances for hazardous materials abatement, which will be required if either asbestos containing materials or lead based paint are present in the original construction. Such materials are likely to be present in buildings constructed prior to about 1980. The above costs relate only to the restoration of connections. They do not include costs related to re-establishing vertical plumbness of the building, which may be impractical to accomplish, or costs related to repair of architectural, mechanical, and electrical components which are directly damaged by the building's response to the ground motion. These statistics assume that the building is repaired, rather than demolished and reconstructed. It should be noted that at least one building, in Santa Clarita, was demolished and reconstructed rather than repaired. A number of factors may have contributed to the owner's decision to take such action, however, it is clear that the cost associated with this decision was much greater than would be indicated by the statistics presented in this Section.

Commentary: The damageabilities indicated in Table 3-6 and Figure 3-7 were estimated based on statistics available on a data set of 89 buildings (Bonowitz and Youssef - 1995). From this data set, it was possible to establish the probability of a building incurring damage to a given percentage of its total connections. This data set also allowed estimation of the number of connections per square foot of floor space provided by a building. From these statistics, an estimated average repair cost per connection of $12,500 was applied against the probable number of damaged connections per square foot of floor space. Building value was taken as $125/square foot of floor space. This computation permitted calculation of the expected loss percentage to a typical building. This data was then entered into a loss estimation model developed by Thiel and Zsutty (Thiel and Zsutty - 1987). The model was developed to replicate damage statistics *observed in historic earthquakes and extended to current construction types using, in part, the expert opinion results of ATC-13.*

Ground motion is characterized in Table 3-6 and Figure 3-7 using Modified Mercalli Intensity (MMI). Although MMI has been the most common ground motion parameter used for loss estimation studies in the past, it is subjective and interpretation can be varied. MMI can only be assigned after an earthquake has occurred and is based on observation of damage and other effects that have actually occurred. It is dependent, to a very great extent, on the types of construction which are present in the affected region. The distributions of damage indicated in Table 3-6 and Figure 3-7 are considered appropriate for California, and other regions with similar seismic design and construction practices. However, these data may not be appropriate for other regions.

It should be noted that when the repair cost for a building approaches 60 per cent or more of its replacement value (d≥60%) many owners will determine, based on a number of factors, that complete building replacement, rather than repair is warranted. Therefore, it is probable that the actual costs for repair of some buildings will be 100per cent of the replacement value. This possibility has not been reflected in the development of the damage repair cost distributions presented in Table 3-6.

It should also be noted that the statistics used to develop the above vulnerability estimates were taken from an incomplete data set of buildings. The data set may or may not have been representative of the distribution of damage in the total set of buildings affected by the Northridge Earthquake. If the data set is biased, this is likely to be a bias towards buildings that are more heavily damaged, since the data was collected soon after the earthquake, when only those buildings most likely to have been damaged had been inspected. A review of the applicability of the statistics used for generating the vulnerability estimates should be conducted, when more complete data on the distribution of damage becomes available.

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