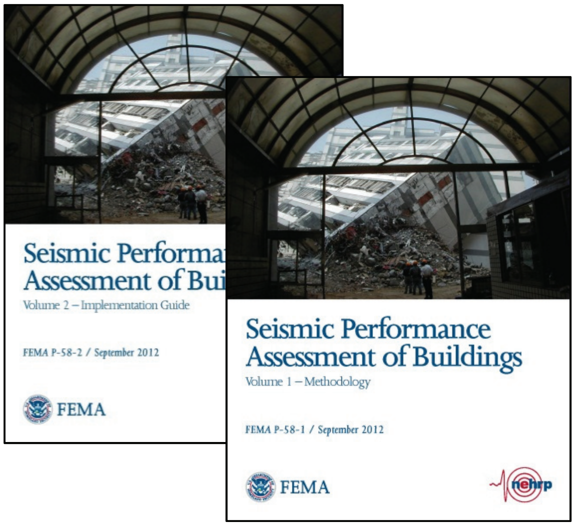

Background Document

FEMA P-58/BD-3.8.3

Fragility Curves for Components of Steel SMF Systems

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Background Documentation

FEMA P-58 Background Documents are a series of reports documenting the technical background and source information for key aspects of the FEMA P-58 methodology and its implementation. These reports were developed over the course of the 10-year ATC-58/ATC-58-1 Projects funded under FEMA Contracts EMW-2001-RP-0056 and HSFEHQ-06-D-1105.

Background Documents were developed by consultants, serving at various levels within the project hierarchy, reporting the results of: (1) decisions on technical development protocols; (2) focused studies on the development of key aspects of the methodology; (3) documentation of recommended procedures; and (4) collection of available data for the development of structural and nonstructural fragilities. They were initially intended to serve as a record of the technical state-of-knowledge at the time they were produced, and as resources for the development of the eventual project reports. As such, they represent a snapshot in time, and may, or may not, match the technical content, recommended procedures, or data incorporated into the final methodology and its implementation.

This Background Document is intended for the purpose of providing supplemental knowledge to users of the FEMA P-58 methodology. Information contained herein has not been independently verified for accuracy as a stand-alone document, and may have been superseded in its final implementation within the methodology. Users of information in this document assume all liability arising from such use.

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ATC 58: Fragility Curves for Components of Steel SMF Systems

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1. Introduction

This document summarizes the development of fragility curves for structural components of ductile steel moment frame systems, which are designed and detailed in conformance with requirements of special moment frame (SMF) systems. For SMF systems it is assumed that the columns, joint panel zones, and beam-column connections are strong enough such that inelastic action will occur primarily in beam hinges and column bases. Damage fragility curves are developed for the following frame components: (1) beam-to-column moment connections, (2) beam-to-column gravity beam shear connections, (3) moment resisting column base plate connections, and (4) welded column splices. Owing to the significant changes in seismic design requirements for welded connections following the 1994 Northridge earthquake, the fragility curves for welded beam-column moment connections are distinguished between “pre-Northridge” and “post-Northridge” details.

The fragility curves are developed using a combination of data from laboratory tests, observations of earthquake damage, and expert judgment. Fairly extensive testing exists of post-Northridge beam column connections, though most of this is limited to isolated subassembly tests – most of which do not included concrete slabs and composite beam action. Fewer tests data are available on the other connection types, and no subassembly test data is available on welded column splices.

For each component, damage states are defined based on the nature of repairs that would be required to restore the component to its pre-earthquake (essentially undamaged) state. The damage fragility curves are typically defined in terms of interstory drift ratios, although where available more fundamental measures of plastic hinge rotations are preserved in the data analysis.

2. Beam-to-Column Connections

2.1 Sources of Information

Considerable data is available on the performance of steel beam-to-column connections through investigations conducted as part of the SAC Joint Venture Steel Program. FEMA 355D, *State of the Art Report on Connection Performance*, provides an excellent overview of the performance of different connection types. This report, together with the underlying SAC topical reports and papers provides a bulk of the data available on steel beam-to-column connections. Where available, the SAC test data are augmented by other published results.

Pre-Northridge Connections: Data from a total of twenty-two laboratory tests from four different investigations (K.H. Lee et al. 2000, Engelhard and Husain 1993, Tsai and Popov 1988, and Leon et al. 1998) were reviewed to assess the performance and damage states of pre-Northridge beam-to-column connections with welded flanges and bolted webs. Data from these tests are generally consistent with damage to steel frames reported in the Northridge earthquake.

Lee et al.(2000) reported data on two tests whose purpose was to determine the cause of the connection damage observed during the Northridge earthquake and to provide insights on the design of improved connection details. Just prior to the Northridge earthquake, Engelhardt and Husain (1993) reported data from nine tests, whose original goal was to investigate the need for supplemental welds on beam web connections where beam web accounts for a substantial portion of the beam's flexural strength. Their initial set of four tests revealed considerable variability in the fracture resistance of the groove welds. A second round of four tests were then conducted on specimens that were fabricated by another more experienced welder and incorporated larger web copes to facilitate welding at the bottom flange. This second set of four specimens performed significantly better and with less variability than the first set, though they are still deemed deficient by post-Northridge design standards. The ninth specimen by Engelhard and Husain was an all welded detail and is excluded from the development of fragilities for welded flange-bolted web connections. In the third investigation, Tsai and Popov (1988) tested eighteen specimens of beam-to-column connections, ten of which were of details with welded flanges and bolted webs and are included in the fragility curve development. The final set are three tests by Leon et al.(1998), which included one bare steel connection and two connections with composite floor slabs.

Post-Northridge Connections: Damage states and fragility curves of Post-Northridge connections were developed based on forty-eight tests results from nine different investigations. All of the test reports provide good descriptions of the damage behavior of the connections at different drift levels.

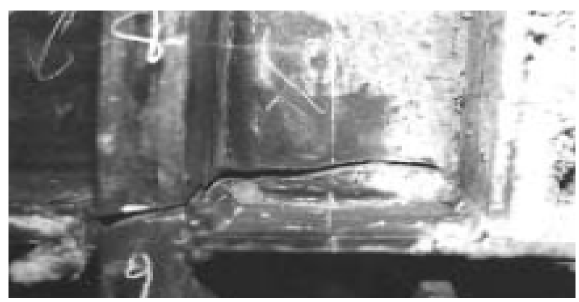
RBS Connections: Data from twenty-one tests of RBS connections were reviewed, eight of which were of tests that included slabs and had reasonably stiff panel zones. The behavior of RBS with and without slab is quite different, where the beams without slabs were much more prone to local flange/web buckling and lateral-torsional instability. Ricles et al (2004) investigated the behavior of steel connections with deep columns, including five RBS connections with slabs and one bare steel RBS connection. Of the three other test programs that investigated RBS details, only one (Engelhardt and Vent, 2000) included beams with composite slabs. Gilton et al. (SAC/BD-00/23, 2000) conducted five tests of bare RBS beams, and Yu et al. (2000) conducted two tests of bare RBS steel beams. Engelhardt and Vent (2000) conducted five tests of bare steel beams and three with composite beams.

Non-RBS Connections: For non-RBS Post-Northridge connections, twenty seven tests from five investigations were considered, only two tests of which included composite beams. In contrast to RBS details, the non-RBS details were less prone to interactive local and lateral-torsional buckling, and thus all of the tests are considered in the fragility curve development. Data on the non-RBS post-Northridge connections are from the following references: Ricles et al (2000), Kim et al. (2000), Venti and Engelhardt (2000), Gilton et al (2000 (SAC/BD-00/19)) and Choi, Stojadinovic and Goal (2000). Another study by Cordova and Deierlein (2005) provides data on the beneficial effects of composite beam action and frame continuity on connection behavior. While this study substantiates the general observations from the connection subassembly tests, it also suggest that the subassembly tests tend to over-estimate the damage.

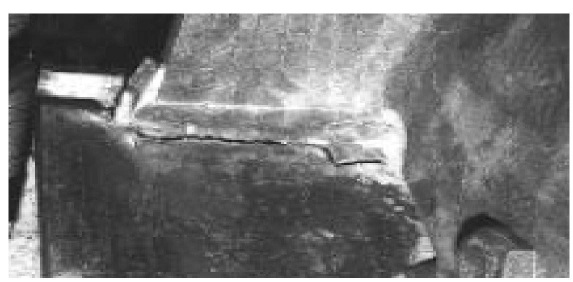
2.2 Definition of Damage States

2.2.1 Pre-Northridge Connections: For Pre-Northridge Connections the damage states were identified from large-scale beam-column connection tests, observations of damage to steel connection after the earthquake and finally based on expert judgment. Damage states *DS-1A* and *DS-1B* were observed in both laboratory tests and buildings damaged by the Northridge earthquake. Damage states DS-2A and DS-2B were observed in damaged buildings but not in laboratory tests. Damage state DS-3 was observed in one set of laboratory tests.

Damage State 1A**:** Fracture of lower beam flange weld and failure of web bolts (shear tab connection), with fractures confined to the weld region. Repair will typically require gouging out and re-welding of the beam flange weld, repair of shear tab, and replacing shear bolts. This damage state was widely observed in buildings damaged in the Northridge earthquake and laboratory tests.



Damage State 1B: Similar to *DS-1A*, except that fracture propagates into column flanges. In addition to measures for *DS-1A*, repairs to column will be necessary that will involve replacing a portion of the column. While not as prevalent as *DS-1A*, this damage state was observed both in damaged buildings and one of the reported tests by Leon et al (1998). The available data suggests that *DS-1A* and *DS-1B* occur at similar deformation levels and that the occurrence of one versus the other is somewhat random.



Damage State 2A and 2B**:** Fracture of upper beam flange weld, either alone or combined with *DS-1* type damage. Fracture may be confined to beam flange region (*DS-2A*) or propagate into column (*DS-2B*). Repairs will be similar to those required for *DS-1*, except that access to weld will likely require removal of a portion of the floor slab above the weld. This damage state was observed in damaged buildings but was not reported in laboratory tests. Its absence in laboratory tests is presumed to be due to the fact that most tests were discontinued after *DS-1*, such that imposed drifts were not large enough to trigger this damage state.

***Damage State 3*:** Ductile fracture initiating at weld access hole and propagating through beam flange. This damage state is mutually exclusive of *DS-1* and *DS-2* and occurs when the welds are able to sustain plastic hinging of the beam. Few if any buildings during the Northridge earthquake experienced induced deformations large enough to cause this damage state. Thus, buildings that did not sustain *DS-1* and *DS-2* in the Northridge earthquake may likely have experienced *DS-3* under more severe ground shaking. DS-3 was observed in two tests of pre-Northridge connection details by Leon et al. (1998), one of which included composite floor slabs.

Damage States DS-1 and DS-2 are ordered states (i.e., DS-2 occurs at larger deformations after DS-1), but where the variants A and B within each damage state are mutually exclusive (i.e., DS-1A and DS-1B do not occur together). DS-3 is mutually exclusive with DS-1 and DS-2.

2.2.2 Post-Northridge: RBS with Slab and Strong Panel Zones

For RBS connections with slabs and strong panel zones, data from eight tests of two investigations were considered in the definition of three damage states. Tests from bare (no slab) RBS connections were excluded, since they tended to buckle earlier and experience more torsional-flexural deformations than the tests with slabs. Tests of specimens with weak panel zones are excluded, since these are not generally permitted by current building code requirements. Since the RBS connections were only introduced in the late 1990's, there is no data from earthquake damage to actual buildings.

Tests show that the web within the RBS is most likely to develop local buckling at relatively low drift levels. The strength of the specimens does not, however, deteriorate with web buckling. Following the local web buckling the bottom flanges buckle within the RBS and that is often closely related to the peak capacity of the specimen. Lateral torsional buckling in the beam flanges starts shortly after the beam flanges local buckling and the strength deteriorates further. Lateral torsional buckling begins first in the bottom flanges and then in the top flanges. Eventually, due to repeated buckling of the flanges a low cycle fatigue fracture develops in either the top or bottom beam flanges within the RBS.

Only one test specimen failed by fast fracture in the beam bottom flange, just outside the groove weld. The fracture occurred at the 6% drift level. That specimen had no doubler plates and the beam web was bolted to the column. One specimen did not fracture at all but the strength deteriorated significantly at the 5% drift level and the test was eventually stopped at 7% drift level due to column twisting.

One set of test results on a RBS specimen with slab and very weak panel zone were omitted (DBBWWPZC, Engelhardt and Venti, 2000). In this specimen nearly all of the inelastic action occurred in the panel zone and the beam flange fractured adjacent to the flange-to-column weld at 6% drift due to kinking associated with panel zone deformation. The specimen did not develop any local web or flange buckling; nor did it experience lateral torsional buckling.

The typical sequence of damage in the RBS specimens with slabs and reasonably strong panel zones was as follows:

DRIFT (%)	**DAMAGE**
0.38% - 0.5%:	Cracking of Concrete Slab
0.5% - 1.5%:	Yielding of bottom flanges in RBS and between RBS and column face.
0.75% - 1.5%:	Initial Yielding of Panel Zone
1% - 1.5%:	Concrete slab crushing against column face
1% - 2%:	Yielding in beam top flange and web.
1.5% - 3%:	Column flange yielding.
2% - 3%:	Beam web local buckling in RBS.
3% - 4%:	Bottom beam flange local buckling within RBS.
3% - 4%:	Maximum capacity of specimen reached.
3% - 4%:	Lateral torsional buckling in beam bottom flanges.
4% - 5%:	Lateral torsional buckling in beam top flanges.
5% - 6%:	Low cycle fatigue cracks initiate in beam flanges within RBS
5% - 7%:	Low cycle fatigue fracture in beam top or bottom flanges in RBS.

Damage State 1**:** Local beam flange and web buckling. The likely repair state for DS-1 is heat straightening of the buckled web and flanges.

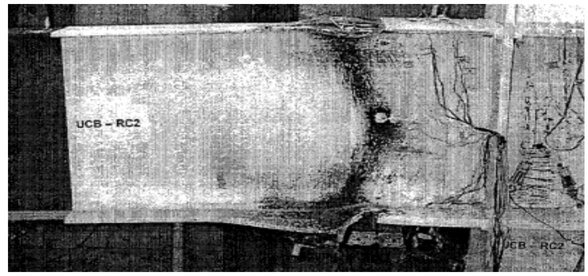
Damage State 2: DS1 plus lateral-torsional distortion of beam in hinge region. Repair by heat straightening may be possible, but it is likely that the distorted portions of the beam may need to be replaced.

Damage State 3**:** Low-cycle fatigue fracture of beam flanges in buckled region of RBS. The fracture is usually precipitated by DS-1 and possibly DS-2. Repair will necessitate removal and replacement of distorted and fractured portion of beam.

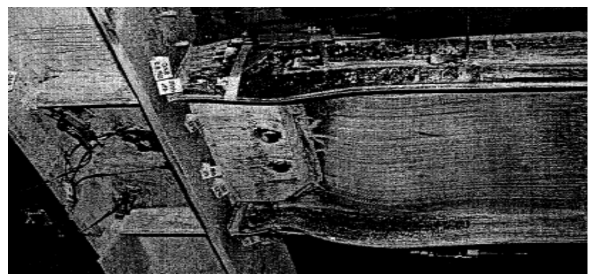
2.2.3 Non-RBS Post-Northridge Connections with Slab

When the beam flanges are not reduced as in the RBS the global stability of the beams is significantly increased and there tend to be less differences between connections with and without slabs. Lateral torsional buckling is less likely to occur, and the tendency for web local buckling is reduced. Moreover, strength deterioration due to buckling is postponed in non-RBS connections, and the ultimate failure mechanism is less certain. While low-cycle fatigue fracture of beam flanges occurs in many specimens, other modes, such as fracture of the base metal close to the HAZ also occur.

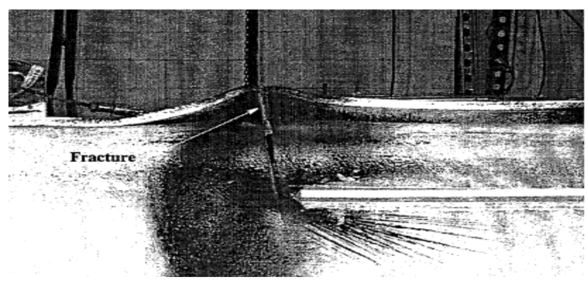
Damage State 1**:** Local beam flange and web buckling. The likely repair state for DS-1 is heat straightening of the buckled web and flanges.



Damage State 2**:** DS1 plus lateral-torsional distortion of beam in hinge region. Repair by heat straightening may be possible, but it is likely that the distorted portions of the beam may need to be replaced. In contrast to RBS details, DS-2 was only observed in about one-quarter of the tests.



Damage State 3**:** Buckling associated with *DS-1* (and sometimes *DS-2*) that leads to low-cycle fatigue fracture of beam flanges in hinge region or adjacent to the flange weld, sometimes accompanied by severe distortion of beam. Repair is likely to involve replacing the distorted and/or fractured portions of the beam.



2.3 Development of Fragility Curves for Beam-Column Connections

As noted previously, the fragility damage states and parameters are developed based on a combination of observations from post-earthquake reconnaissance, laboratory tests and observations, and judgment. To the extent that the laboratory test data represents the full range of conditions and damage expected in steel SMF buildings, these data figure heavily into the fragility parameters. However, there are many limitations with available data that require judgment to develop realistic fragility parameters. For example, some of the limitations and situations faced with respect to limitations of the test data are as follows:

- For the pre-Northridge type connections, the laboratory tests did not replicate the damage states DS2A/B that were observed in buildings damaged in the Northridge earthquake. Therefore, judgments are necessary to fill in gaps in the test data.

- The laboratory tests do not cover the full range of conditions (e.g., beam and column sizes, lateral bracing conditions, variation in weld quality, etc.) that is expected in practice. Therefore, the dispersion based on test statistics underestimates the actual dispersion.

- All of the connection laboratory tests were of subassemblies that do not model the restraint provided in full frames. Assuming that the frame continuity will restrain some of the buckling response, then the fragility data from subassemblies are likely to over-estimate the amount of damage that will actually occur. While difficult to quantify, the large difference between test and actual conditions does warrant some adjustments to the damage demand parameters.

Test data relating demand parameters to damage states are summarized in Tables 1 to 3 for the three categories of steel SMF connections: pre-Northridge, post-Northridge RBS details, and post-Northridge non-RBS details. Demand parameters are generally reported for story drift ratio (Δ/h), and plastic hinge rotation is reported where it is readily available from the test data. Blank fields in the table designed by a dash (-) imply that the referenced damage state was not observed. Bland fields for plastic hinge rotations designated by an asterisk (*) indicate that the damage state did occur but that the hinge rotation demand is not available. More complete reporting of the test data is summarized in the appendices. Appendix A includes a spreadsheet database of the recorded information from each reference. Appendix B includes figures of the typical details, test configurations, loading protocols, etc. Appendix C includes images of the damaged components.

2.3.1 Pre-Northridge Connections: As described in Section 2.2, three damage states (DS-1, 2 and 3) are identified for pre-Northridge connections, with two variants (A and B) for damage states DS-1 and DS-2. As summarized in Table 2.1, most of the tests of pre-Northridge connections only observed damage state DS-1A (fracture to bottom flange weld). Damage states DS-1B and DS-3 were observed in a few tests by Leon et al. (1998). The median demand parameters for DS-1A are $\theta_p = 0.9\%$ (plastic hinge rotation) and $\Delta/h = 1.9\%$ (story drift). Note that most of the references reported plastic hinge rotations and the conversion to drift generally assumes an elastic drift deformation of 1%. The large component of elastic drift, which is assumed equal for all configurations, is the reason by the dispersion of $\beta = 0.28$ for the story drift index is only about half that of the dispersion on plastic hinge rotation. Another factor to consider in drawing conclusions from the data is that the tests by Tsai and Popov (1988) were done on smaller beams (W18 x 35 and W 21x 44) as compared to those tested by others. As it is well established that fracture is more likely in deeper beams with thicker flanges, one could argue that the Tsai and Popov tests tend to bias the median demand in favor of smaller beams. If one excludes the Tsai and Popov tests the median demands for DS-1A are reduced to $\theta_p = 0.7\%$ (plastic hinge rotation) and $\Delta/h = 1.7\%$ (story drift).

Table 2.1: Data used to develop fragility curves for Pre-Northridge Beam-to-Column Connections

Pre-Northridge: Welded Flange-Bolted Web							
Reference	Specimen ID	DS-1A θ	DS-1A Δ/h	DS-1B θ	DS-1B Δ/h	DS-3 θ	DS-3 Δ/h
Lee et al.	SP 1.1	0.007	0.017	-	-	-	-
	SP 1.2	0.007	0.017	-	-	-	-
Engelhardt & Husain	1	0.004	0.014	-	-	-	-
	2	0.003	0.013	-	-	-	-
	3	0.009	0.019	-	-	-	-
	4	0.002	0.012	-	-	-	-
	5	0.013	0.023	-	-	-	-
	6	0.013	0.023	-	-	-	-
	7	0.015	0.025	-	-	-	-
Tsai & Popov	SPEC 2	0.020	0.030	-	-	-	-
	SPEC 3	0.009	0.019	-	-	-	-
	SPEC 4	0.009	0.019	-	-	-	-
	SPEC 5	0.007	0.017	-	-	-	-
	SPEC 6	0.007	0.017	-	-	-	-
	SPEC 7	0.015	0.025	-	-	-	-
	SPEC 13	0.013	0.023	-	-	-	-
	SPEC 14	0.024	0.034	-	-	-	-
	SPEC 17	0.015	0.025	-	-	-	-
	SPEC 18	0.014	0.024	-	-	-	-
Leon et al.	1E	-	-	-	-	*	0.030
	1W	-	-	0.007	0.015	-	-
	2E	0.007	0.015	-	-	-	-
	2W	0.007	0.015	-	-	-	-
	3E	-	-	-	-	-	0.030
	3W	-	-	-	-	-	0.030
Median		0.9%	1.9%	0.007	0.015		3.0%
Dispersion		0.60	0.28	0	0		0
Only DS-1 Passed Lilliefors at 5% Significance Level							

Based on the available data, the fragility function parameters and relationships are for pre-Northridge connections are as summarized in Figure 2.1. Damage states DS-1 and DS-2 are ordered, whereas DS-3 is mutually exclusive from DS-1 and DS-2. Moreover within DS-1 and DS-2 the variants A and B are mutually exclusive within each damage state and uncorrelated between each damage state. Based in part on the tests by Leon et al. (1998) and judgment, the relative probability of brittle weld fractures (DS-1 and DS-2) versus ductile rupture (DS-3) is 0.75 and 0.25, respectively. The median demand parameters for DS-1 are based on reported test data (Table 1), excluding that from the study by Tsai and Popov (1988). The dispersion based on story drift is inflated to 0.4 to account for variations not reflected in the test data. The relative probability of DS-1A (fracture propagation in the weld) versus DS-1B (fracture propagation into the column) is judged to be 0.75 and 0.25. The median capacity for DS-2 is estimated based on the relative values for DS-1 and DS-3, and the relative probabilities between DS-2A and DS-2B are the same as for DS-1A and DS-2A. Finally, the median capacities for DS-3 are based on the data (Table 1) from Leon et al. (1998). Note that for implementation purposes, the distinction between damage states A and B can be handled in the consequence (repair cost) functions. As the variation in damage states for a specified story demand depends on the member sizes, material properties, welding details and welding quality, the damage states are considered to be moderately correlated within a given building.

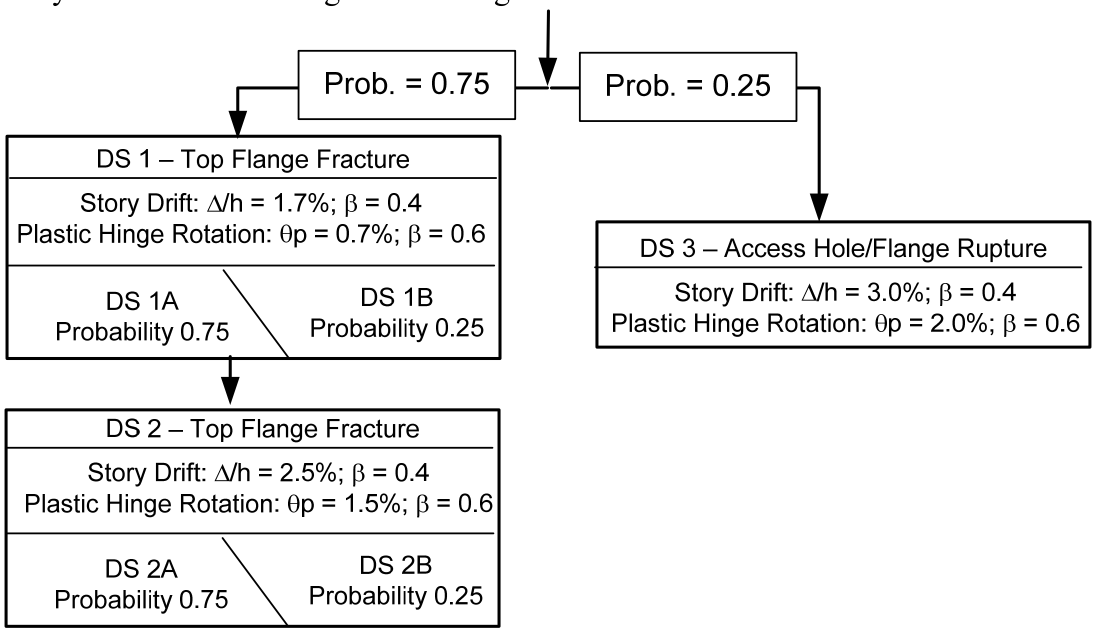


Figure 2.1 – Damage State Fragilities for Pre-Northridge Steel SMF Connections

2.3.2 Post-Northridge RBS Connections: As described in Section 2.2, three ordered damage states (DS-1, 2 and 3) are identified for the post-Northridge RBS connections. Summarize in Table 2.2 are test data for RBS connections with stiff panel zones and slabs. The stiff panel zones ensure that most of the inelastic action occurred in the RBS region, and the slab is significant since the slab plays an important role in stabilizing the RBS beam against torsional-flexural deformations. As indicated in the table footnote, the median story drift demand parameters for specimens without slabs were about 1% (0.01) less than those with slabs. Based on the data in Table 2.2, the median story drift demands for the three damage states are Δ/h = 3%, 4% and 5%. Since the number of tests are limited, relative to the range of member sizes, bracing configurations and other details, the dispersion is inflated to β = 0.3 for all three damage states. Similar to the pre-Northridge damage states, damage to post-Northridge connections are assumed to be moderately correlated within a given building.

2.3.3 Post-Northridge non-RBS Connections: The three ordered damage states (DS-1, 2 and 3) for the post-Northridge non-RBS connections are similar to those for the RBS connections. Summarize in Table 2.3 are test data for non-RBS connections, including tests with and without slabs. Unlike RBS details, where the slab has a significant stiffening effect, for non-RBS details the behavior is similar for connection tests with and without slabs. The reduced tendency for torsional-flexural behavior is evident from the fact that only about one-quarter of the tests experienced DS-2. Based on the data in Table 2.3, the median story drift demands for the three damage states are Δ/h = 3%, 4% and 5%. Given the reduced likelihood of DS-2, the fragility curve for this damage state has an additional conditional probability of 0.25 that it occurs at the predicted demand parameter. Since the number of tests are limited, relative to the range of member sizes, bracing configurations and other details, the dispersion is inflated to β = 0.3 for all three damage states.

2.4 Summary of Fragility Curves for Steel SMF Beam-Column Connections

Tables 2.4 through 2.6 summarize the damage states and fragility parameters for the three types of steel SMF beam-to-column moment connections: (1) pre-Northridge welded flanges and bolted webs, (2) post-Northridge RBS details, and (3) post-Northridge non-RBS details. In addition to the damage states and demand criteria described previously, two additional judgment-based attributes in the summary tables relate to (a) correlation of damage states within a structure, and (b) consequences of the damage state for leading to "red tagging" (closure) of the building. For all connection types and damage states, the correlations are judged to be "moderate" (nominally 50% correlated). This is based on the observation that many of the factors that lead to dispersion in the data will likely be similar within one building design, including member sizes, connection weld details, bracing details, and materials. Damage states are assumed to lead to "red tagging" if a significant fraction (e.g., one-third or more) of the components in the building are in the designated damage state.

Table 2.2: Data used to develop fragility curves for Post-Northridge RBS Connections

RBS: Tests With Slab and Stiff Panel Zone[1]							
Reference	**Specimen ID**	**DS-1**		**DS-2**		**DS-3**	
		θ	**Δ/h**	**θ**	**Δ/h**	**θ**	**Δ/h**
Ricles et al.	Spec 1	*	0.025	*	0.035	0.043	0.050
	Spec 2	*	0.025	*	0.035	0.040	0.050
	Spec 3	*	0.025	*	0.035	0.052	0.060
	Spec 4	*	0.025	*	0.035	0.060	0.060
	Spec 5	*	0.035	*	0.045	0.060	0.060
Engelhard & Vent	DBBWC	*	0.040	*	0.050	0.050	0.050
	DBWWC	*	0.030	*	0.050	0.030	0.040
	DBBWSPZC	*	0.025	*	0.040	0.050	0.060
Median			2.8%		4.0%	4.7%	5.3%
Dispersion			0.19		0.16	0.23	0.15
		Failed Lilliefors		Passed Lilliefors 2.5% Sig. Level		Passed Lilliefors 5% Sig. Level	

Note 1) For RBS connections without slabs, local and torsional-flexural buckling is more likely and fracture is less likely. In such cases the story drift demand parameters are roughly: DS1=2%, DS2=3%, DS3= 4%.

Table 2.3: Data used to develop fragility curves for Post-Northridge non-RBS

Post-Northridge Beam-to-Column (non RBS)							
Reference	**Specimen ID**	**DS-1**		**DS-2[1]**		**DS-3**	
		θ	**Δ/h**	**θ**	**Δ/h**	**θ**	**Δ/h**
Ricles et al.	T1	*	0.030	-	-	0.035	0.050
	T2	*	0.030	-	-	0.025	0.040
	T3	*	0.030	-	-	0.020	0.030
	T4	*	0.030	-	-	0.018	0.040
	T5	*	0.030	-	-	0.054	0.060
	T6	*	0.040	-	-	0.050	0.060
	C1	*	0.030	-	-	0.039	0.050
	C2	*	0.030	-	-	0.050	0.060
	C3	*	0.030	-	-	0.041	0.055
	C4	*	0.030	-	-	0.052	0.060
	C5	*	0.030	-	-	0.026	0.040
Venti & Engel.	FFC1	*	0.030	-	-	0.033	0.05
Gilton et al.	FFNC1	*	0.030	-	-	0.017	0.027
Choi et al.	SP 8.2	*	0.030	-	0.040	0.037	0.047
	SP 9.1	*	0.020	-	-	0.036	0.046
	SP 9.2	*	0.018	-	0.030	0.027	0.037
	SP 10.1	*	0.020	-	-	0.024	0.034
	SP 10.2	*	0.030	-	0.040	0.023	0.033
Kim et al.	RC01	*	0.030	-	-	0.034	0.044
	RC02	*	0.030	-	-	0.035	0.045
	RC03	*	0.030	-	0.040	0.036	0.046
	RC04	*	0.030	-	-	0.041	0.051
	RC05	*	0.030	-	-	0.042	0.053
	RC06	*	0.030	-	0.040	0.035	0.045
	RC08	*	0.030	-	0.040	0.042	0.052
	RC09	*	0.030	-	-	0.047	0.057
	RC10	*	0.030	-	-	0.038	0.048
Median			2.9%		3.8%	3.4%	4.6%
Dispersion			0.16		0.13	0.32	0.22
		Failed Lilliefors		Passes Lilliefors at 2.5%		Passed Lilliefors	

Note 1) DS2 only occurred in about one quarter of the Post-Northridge non-RBS tests.

Table 2.4 - Fragility, damage measures, and consequences for Steel SMF Pre-Northridge Beam-Column Moment Connections

Component category:	Structural				
Basic composition:	Fully restrained beam-column connection with welded flanges and bolted webs. Flange welds are of the pre-Northridge type with non-notch toughness electrodes and backing bars left in place.				
Units:	Number of connections				
Demand parameter:	Story Drift Ratio				
Number of damage states:	5 (DS-1A/B; DS-2A/B, DS-3)				
If multiple damage states:	DS 1 and DS 2 are ordered, and within each of these the A/B variants are mutually exclusive of each other. DS 3 is mutually exclusive of DS 1 and 2.				
Author and date:	Deierlein and Victorsson, August 23, 2008				
Damage states, fragilities, and consequences					
	DS 1 & DS2: Weld Fractures				**DS 3**
	DS 1A	**DS 1B**	**DS 2A**	**DS 2B**	**DS 3**
Description:	Note 1A	Note 1B	Note2A	Note2B	Note 3
Illustration:	Sec. 2.2.1	Sec. 2.2.1	Sec. 2.2.1	-	-
Median demand (θ)[(1)]:	0.017	0.017	0.025	0.025	0.03
Dispersion (β)[(1)]:	0.40	0.40	0.40	0.40	0.40
	0.75				0.25
Probability[(1)]:	0.75	0.25	0.75	0.25	
Correlation:	moderate	moderate	moderate	moderate	moderate
Repairs required:	Note 1A	Note 1B	Note2A	Note2B	Note 3
Possible consequences:					
Repair cost (Y/N/?):	Y	Y	Y	Y	Y
Death or injury (Y/N/?):	-	-	-	-	-
Inoperative facility (Y/N/?):	-	-	-	-	-
Red tagging (Y/N/?)	-	-	Y	Y	Y
Comments:					

Notes:

1A) ***DS-1A*:** Fracture of lower beam flange weld and failure of web bolts (shear tab connection), with fractures confined to the weld region. Repair will typically require gouging out and re-welding of the beam flange weld, repair of shear tab, and replacing shear bolts.

1B) ***DS-1B:*** Similar to *DS-1A*, except that fracture propagates into column flanges. In addition to measures for *DS-1A*, repairs to column will be necessary that will involve replacing a portion of the column.

2A) ***DS-2A*:** Fracture of upper beam flange weld, either alone or combined with *DS-1* type damage. Fracture is confined to beam flange region. Repairs will be similar to those required for *DS-1A*, except that access to weld will likely require removal of a portion of the floor slab above the weld.

2B) ***DS-2B*:** Similar to *DS-2A*, except that fracture propagates into column flanges. In addition to measures for *DS-2A*, repairs to column will be necessary that will involve replacing a portion of the column.

3) ***DS-3*:** Ductile fracture initiating at weld access hole and propagating through beam flange, possibly accompanied by local buckling deformations of web and flange. Repair is similar to that for DS-1A except that a portion of the beam web and flange may need to be heat straightened or replace.

Table 2.5 - Fragility, damage measures, and consequences for Steel SMF Post-Northridge RBS Beam-Column Moment Connections

Component category:	Structural		
Basic composition:	Fully restrained beam-column connection with welded flanges, bolted webs, and reduced beam section (RBS) in the plastic hinge region. Welding details utilize electrodes with high notch-toughness and other modifications to minimize potential for weld root fractures.		
Units:	Number of connections		
Demand parameter:	Story Drift Ratio		
Number of damage states:	3		
If multiple damage states:	DS 1,2 and 3 are ordered.		
Author and date:	Deierlein and Victorsson, August 23, 2008		
Damage states, fragilities, and consequences			
	DS 1	**DS 2**	**DS 3**
Description:	Note 1	Note2	Note 3
Illustration:	Section 2.2.2	Section 2.2.2	Section 2.2.2
Median demand (θ)[(1)]:	0.03	0.04	0.05
Dispersion (β)[(1)]:	0.30	0.30	0.30
Probability[(1)]:			
Correlation:	moderate	moderate	moderate
Repairs required:	Note 1	Note 2	Note 3
Possible consequences:			
Repair cost (Y/N/?):	Y	Y	Y
Death or injury (Y/N/?):	-	-	-
Inoperative facility (Y/N/?):	-	-	-
Red tagging (Y/N/?)	-	Y	Y
Comments:			

Notes:

1) ***DS-1*:** Local beam flange and web buckling. The likely repair state is heat straightening of the buckled web and flanges

2) ***DS-2*:** DS1 plus lateral-torsional distortion of beam in hinge region. Repair by heat straightening may be possible, but it is likely that the distorted portions of the beam may need to be replaced.

3) ***DS-3*:** Low-cycle fatigue fracture of beam flanges in buckled region of RBS. The fracture is usually precipitated by DS-1 and possibly DS-2. Repair will necessitate removal and replacement of distorted and fractured portion of beam.

Table 2.6 - Fragility, damage measures, and consequences for Steel SMF Post-Northridge non-RBS Beam-Column Moment Connections

Component category:	Structural		
Basic composition:	Fully restrained beam-column connection with welded flanges, bolted webs, and other non-RBS post-Northridge connection details. Welding details utilize electrodes with high notch-toughness and other modifications to minimize potential for weld root fractures.		
Units:	Number of connections		
Demand parameter:	Story Drift Ratio		
Number of damage states:	3		
If multiple damage states:	DS 1,2 and 3 are ordered.		
Author and date:	Deierlein and Victorsson, August 23, 2008		
Damage states, fragilities, and consequences			
	DS 1	**DS 2**	**DS 3**
Description:	Note 1	Note2	Note 3
Illustration:	Section 2.2.3	Section 2.2.3	Section 2.2.3
Median demand (θ)(1):	0.03	0.04	0.05
Dispersion (β)(1):	0.30	0.30	0.30
Probability(1):		0.25 (note 4)	
Correlation:	moderate	moderate	moderate
Repairs required:	Note 1	Note 2	Note 3
Possible consequences:			
Repair cost (Y/N/?):	Y	Y	Y
Death or injury (Y/N/?):	-	-	-
Inoperative facility (Y/N/?):	-	-	-
Red tagging (Y/N/?)	-	Y	Y
Comments:			

Notes:

1) ***DS-1*:** Local beam flange and web buckling. The likely repair state is heat straightening of the buckled web and flanges

2) ***DS-2:*** DS1 plus lateral-torsional distortion of beam in hinge region. Repair by heat straightening may be possible, but it is likely that the distorted portions of the beam may need to be replaced.

3) ***DS-3*:** Low-cycle fatigue fracture of beam flanges in buckled region of RBS. The fracture is usually precipitated by DS-1 and possibly DS-2. Repair will necessitate removal and replacement of distorted and fractured portion of beam.

4) Damage state DS-2 was evident in only some connections and is judged to have a 25% chance of occurance when when the demand parameter for DS-2 is reached.

3. Gravity Beam Shear Connections

In modern U.S. construction practice, gravity beam shear connections typically consist of a shear tab (or fin plate) that is welded to the column and bolted to the web of the gravity beam.

3.1 Sources of Information

The fragility curves for Gravity Beam Shear Connections are developed using tests by Liu and Astaneh (2000). These included sixteen tests, thirteen of which had shear tab details, two included additional stiffening seats as possible retrofit schemes, and one was a stiffened seat connection. Only the thirteen shear tab details were considered as being relevant to the fragility curve development. Among the thirteen tests, major parameters included lightweight versus normal weight concrete slabs, strong versus weak-axis column connections, and pre-1980 (versus post-1980) connection details.

In general, the tests show that ductility of the connections decreases with increasing beam and shear tab depth. However, the behavior is otherwise quite similar and different size connections go through similar damage states before ultimate failure. The presence of a slab tends to increase the initial rotational stiffness which tends to initiate deformations and tearing sooner in the bottom half of the shear tab, but otherwise, the slab does not influence the behavior much past 4% story drift. For the pre-1980 connections, the deformation was largely concentrated in the beam web, instead of the shear tabs. The type of concrete, lightweight versus normal weight, does not alter the behavior of the connections significantly.

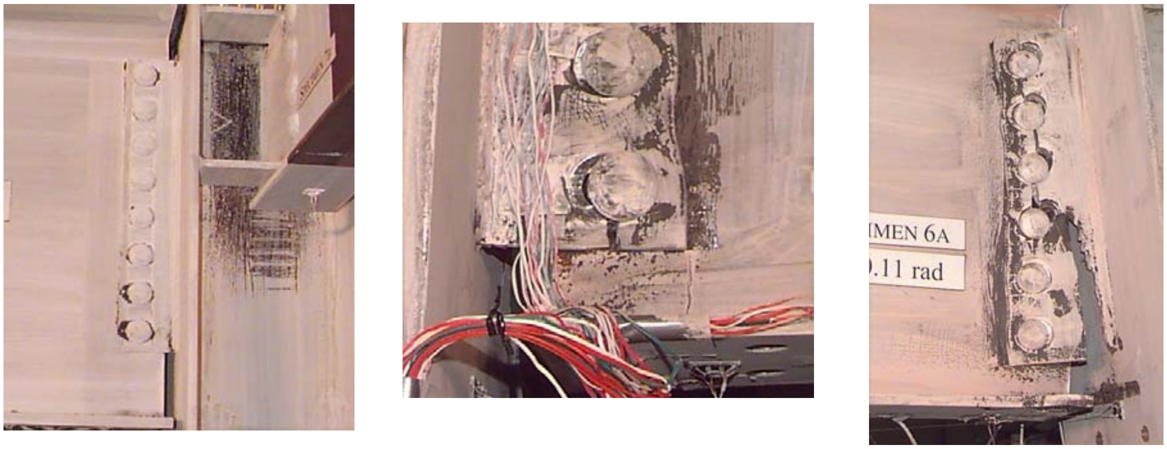
3.2 Definition of Damage States

As described below, three ordered damage states are identified. are based on experimental results from Liu and Astaneh (2000) In the case where the shear tab is thick enough that it does not deform, it could be expected that the bolts fail before the tab. In that case only one Damage State is present, i.e. shear failure of bolts.

***Damage State 1*:** Yielding of shear tab and elongation of bolt holes, possible crack initiation around bolt holes or at shear tab weld. Careful inspection and welded repair to any cracks and possible replacement of shear tab if bolt hole deformations are excessive (possible for deeper 6-bolt or deeper shear tabs).

***Damage State 2*:** Partial tearing of shear tab and possibility of bolt shear failure (6-bolt or deeper connections). Repairs will include either welded repair of shear tab or possible complete replacement of shear tab and installation of new bolts. Repairs may require shoring of beam.

***Damage State 3*:** Complete separation of shear tab, close to complete loss of vertical load resistance. Repair will include complete replacement of shear tab and installation of new bolts. Repairs will require shoring of beam.



(a) (b) (c)

Figure 3.1 Illustrations for damage states of shear tab connections (a) DS-1, (b) DS-2, (c) DS-3.

3.3 Development of Fragility Curves

Data from tests by Liu and Astaneh (2000) on the drift demands associated with the three damage states are summarized in Table 3.1. Based on these data, median drift demands of 0.04, 0.08 and 0.11 are suggested for the three limit states with dispersions of β = 0.40 for DS-1 and DS-2 and β = 0.20 for DS-3. The smaller dispersion for DS-3 will result in overlapping of fragility curves at large drifts, which should be addressed in the loss calculator. The resulting fragility curve data is shown in Table 3.2.

Table 3.1: Data used to develop fragility curves for Gravity Beam Shear Connections

Gravity Beam Shear Connections						
Specimen	**DS1**		**DS2**		**DS3**	
ID	**θ**	**Δ/h**	**θ**	**Δ/h**	**θ**	**Δ/h**
1A	-	0.06	-	0.11	-	0.14
2A	-	0.03	-	0.07	-	0.09
3A	-	0.08	-	0.12	-	0.15
4A	-	0.04	-	0.1	-	0.11
6A	-	0.03	-	0.06	-	0.11
7A	-	0.04	-	0.07	-	0.09
1B	-	0.05	-	0.12	-	0.12
2B	-	0.05	-	0.09	-	0.11
3B	-	0.09	-	0.14	-	0.14
4B	-	0.03	-	0.04	-	0.1
5B	-	0.04	-	NR	-	NR
6B	-	0.03	-	0.05	-	0.09
7B	-	0.03	-	0.05	-	0.08
Median		4.3%		7.9%		11.0%
Dispersion		0.38		0.41		0.2
All DS Passed Lilliefors at 5% Significance Level						

Table 3.2 - Fragility, damage measures, and consequences for Gravity Beam Shear-Tab Connection in Steel Frames

Component category:	Structural		
Basic composition:	Gravity shear connection consisting of vertical shear tab plate that is welded to column and bolted to the supported beam web.		
Units:	Number of connections		
Demand parameter:	Story Drift Ratio		
Number of damage states:	3		
If multiple damage states:	DS 1,2 and 3 are ordered.		
Author and date:	Deierlein and Victorsson, August 23, 2008		
Damage states, fragilities, and consequences			
	DS 1	**DS 2**	**DS 3**
Description:	Note 1	Note2	Note 3
Illustration:	Figure 3.1a	Figure 3.1b	Figure 3.1b
Median demand (θ)[(1)]:	0.04	0.08	0.11
Dispersion (β)[(1)]:	0.40	0.40	0.20
Probability[(1)]:	-	-	-
Correlation:	moderate	moderate	moderate
Repairs required:	Note 1	Note 2	Note 3
Possible consequences:			
Repair cost (Y/N/?):	Y	Y	Y
Death or injury (Y/N/?):	-	-	-
Inoperative facility (Y/N/?):	-	-	-
Red tagging (Y/N/?)	-	-	Y
Comments:			

Notes:

1) ***DS-1*:** Yielding of shear tab and elongation of bolt holes, possible crack initiation around bolt holes or at shear tab weld. Careful inspection and welded repair to any cracks and possible replacement of shear tab if bolt hole deformations are excessive (possible for deeper 6-bolt or deeper shear tabs).
2) ***DS-2*:** Partial tearing of shear tab and possibility of bolt shear failure (6-bolt or deeper connections). Repairs will include either welded repair of shear tab or possible complete replacement of shear tab and installation of new bolts. Repairs may require shoring of beam.
3) ***DS-3*:** Complete separation of shear tab, close to complete loss of vertical load resistance. Repair will include complete replacement of shear tab and installation of new bolts. Repairs will require shoring of beam.

4. Column Base Plate

It is well known that column bases of steel moment frames are often the first location to undergo inelastic deformations under lateral earthquake loading. Compared to beam-column connections, there have been far fewer investigations of column base connections. Therefore, development of the column baseplate fragilities requires considerably more judgment than for the beam-column connections. The column base detail that is addressed is an exposed (non-embedded) moment resisting base plate that is welded to the column and anchored to a concrete footing using vertical anchor rods.

4.1 Sources of Information

Sixteen test results from four different investigations were used to develop the column base fragility curves. These included (a) five tests by Kanvinde et al. (2007) that focused specifically on fracture of the column base weld, two tests by Fahmy et al. (1999) of column bases with strong axis bending, six tests by Burda and Itani (1999) of strong axis bending, and three tests by Lee et al. (2001) to investigate a weak-axis column base plate configuration. In all experiments, the dominant failure mode was fracture of the weld between the column and the base plate. None of the tests experienced failure of the base plate or the anchor bolts. It is not clear whether the extent to which fracture dominated over other possible failure modes is an inherent characteristic of the base plates that is representative of real buildings or whether this was due to peculiarities of the specific tests. The proposed fragility curves are based on the assumption that the anchor rods and base plates are sufficiently strong to develop hinging in the steel column, such that the predominant failure mechanisms are associated with column hinging and weld fracture.

4.2 Damage States

Three ordered damage states are defined for the column base plates.

Damage State 1: Initiation of ductile fracture at the fusion line between the column flange and the baseplate weld. The repair will involve gouging out material surrounding the fracture initiating and re-welding.

Damage State 2: Propagation of brittle crack into column and/or base plate. Depending on the crack trajectory, the repair will range from replacement of a portion of the column or base plate to full replacement of the column base. Replacement will require shoring of column, torch cutting to remove damaged material, and fabrication and field welding to install replacement material.

Damage State 3: Complete fracture of the column (or column weld) and dislocation of column relative to the base plate. Repair may not be feasible depending on the extent of dislocation, which is likely to be accompanied by large residual story drift. If feasible, repair would likely involve replacing the entire base plate assembly and most of the column in the story above the base plate.

Figure 4.1 Illustration of DS-1 Fracture Initiation at Column Based Plate

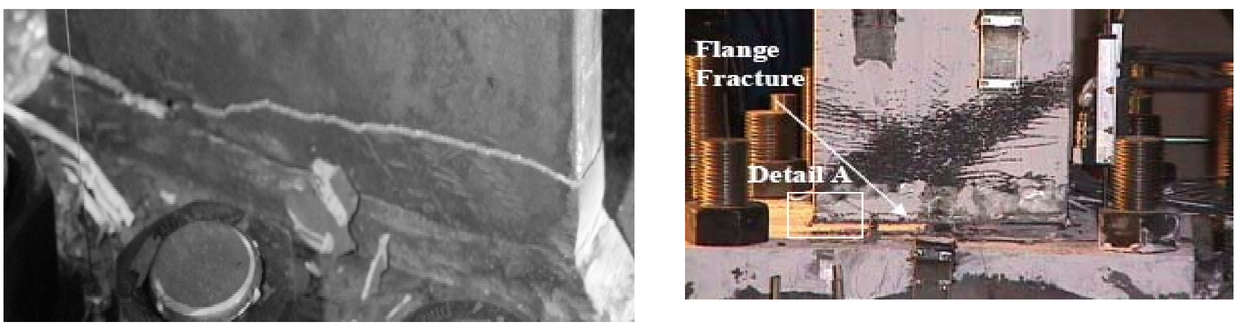


Figure 4.1 Illustration of DS-2 Brittle Fracture Propagation at Column Base Plate

4.3 Summary Fragility Curves

Data on damage state demand parameters are summarized in Tables 4.1 and 4.2 for column bases subjected to major and minor axis bending. As indicated, none of the test specimens were subjected to severe enough loading to cause damage state DS-3. Based on these data, the median story drift demands for the strong axis damage states DS-1 and DS-2 are 0.04 and 0.07 and for minor axis bending are 0.01 and 0.025. Owing to the observed variability and the limited data, the dispersions are set at $\beta = 0.4$. For damage state DS-3, the median demands are assumed to be 0.10 and 0.05 for major- and minor-axis bending, respectively.

Table 4.3 summarizes the complete details for the column base fragilities.

Table 4.1: Data on Column Base Plate Connections – Strong Axis

Steel Column Base Plates							
Reference	Specimen ID	DS1		DS2		DS3	
		θ	Δ/h	θ	Δ/h	θ	Δ/h
Kanvinde et.al.	1	-	0.03	-	0.05	-	-
	3	-	0.04	-	0.05	-	-
	4	-	0.04	-	0.05	-	-
	5	-	0.05	-	0.08	-	-
	6	-	0.06	-	0.07	-	-
Fahmy et al.	2	-	0.02	-	0.05	-	-
	3	-	0.05	-	0.055	-	-
Burda et al.	T1	-	0.031	-	0.094	-	-
	T2	-	0.031	-	0.073	-	-
	T3	-	0.031	-	0.063	-	-
	T4	-	0.052	-	0.135	-	-
	T5	-	0.052	-	0.094	-	-
	T6	-	0.052	-	0.073	-	-
	Median		4.2%		7.1%		-
	Dispersion		0.26		0.31		-
		All DS Passed Lilliefors at 5% Significance Level					

Table 4.2: Data on Column Base Plate Connections – Weak Axis

Post-Northridge Weak-Axis Column Base Plates							
Reference	Specimen ID	DS1		DS2		DS3	
		θ	Δ/h	θ	Δ/h	θ	Δ/h
Lee et al.	SP 4-2	-	0.0075	-	0.015	-	NR
	SP 6-1	-	0.0100	-	0.030	-	NR
	SP 6-2	-	0.0075	-	0.030	-	NR
	Median		0.8%		2.3%		5.0%
	Dispersion		0.17		0.40		0.40
		All Passed Lilliefors at 5% Significance Level					

Table 4.3 - Fragility, damage measures, and consequences for Column Base Plates

Component category:	Structural		
Basic composition:	Column base plates – welded to steel column and anchored to concrete footing to create fixed condition.		
Units:	Number of connections		
Demand parameter:	Story Drift Ratio		
Number of damage states:	3		
If multiple damage states:	DS 1,2 and 3 are ordered.		
Author and date:	Deierlein and Victorsson, August 23, 2008		
Damage states, fragilities, and consequences			
	DS 1	**DS 2**	**DS 3**
Description:	Note 1	Note2	Note 3
Illustration:	Figure 4.1	Figure 4.2	-
Median demand (θ)$^{(1)}$:	0.04 (strong axis) 0.01 (weak axis)	0.07 (strong axis) 0.025 (weak axis)	0.10 (strong axis) 0.05 (weak axis)
Dispersion (β)$^{(1)}$:	0.40	0.40	0.40
Probability$^{(1)}$:	-	-	-
Correlation:	moderate	moderate	moderate
Repairs required:	Note 1	Note 2	Note 3
Possible consequences:			
Repair cost (Y/N/?):	Y	Y	Y
Death or injury (Y/N/?):	-	-	-
Inoperative facility (Y/N/?):	-	-	-
Red tagging (Y/N/?)	-	-	Y
Comments:			

Notes:

1) ***DS-1*:** Initiation of ductile fracture at the fusion line between the column flange and the baseplate weld. The repair will involve gouging out material surrounding the fracture initiating and re-welding.

2) ***DS-2*:** Propagation of brittle crack into column and/or base plate. Depending on the crack trajectory, the repair will range from replacement of a portion of the column or base plate to full replacement of the column base. Replacement will require shoring of column, torch cutting to remove damaged material, and fabrication and field welding to install replacement material.

3) ***DS-3*:** Complete fracture of the column (or column weld) and dislocation of column relative to the base plate. Repair may not be feasible depending on the extent of dislocation, which is likely to be accompanied by large residual story drift. If feasible, repair would likely involve replacing the entire base plate assembly and most of the column in the story above the base plate.

5. Welded Column Splices

Typical column splices in steel moment frames consist of partial penetration groove welds and a splice plate that is attached to the column webs. These splices are usually located 3 to 4 feet above the floor to facilitate construction and to minimize bending induced stresses. While first-order elastic response of steel frames indicate that induced moments are small at the column splice locations, nonlinear analyses have shown that the splices can be subjected to large moments. While splice failures have not been common in post-earthquake damage inspections, concerns exist that splice failures are likely to occur since the partial-strength splices are not designed to develop the full bending strength of the columns.

5.1 Sources of Information

Tests by Bruneau et al. (1987) and fracture analyses by Nuttayasakul (2001) have shown that column splices made with partial penetration groove welds can reliably resist their nominal design strength but that the connections exhibit very limited ductility when loaded beyond their nominal strength. This is due to the limited length over which inelastic deformations can occur in the partial strength welds.

5.2 Damage States

Two ordered damage states are identified for welded column splices.

Damage State 1: Ductile fracture of the groove weld flange splice. Repair would involve gouging out the material adjacent to the fracture and repairing with a new groove weld.

Damage State 2: DS-1 following by complete failure of the web splice plate and dislocation of the two column segments on either side of the splice. Repair may not be practically feasible, but would require either realignment or replacement of adjacent column segments and rewelding of splice.

5.3 Summary of Fragility Curves

Tests and analyses show that fracture associated with *DS-1* occurs when the nominal weld strength capacity is exceeded with sufficient deformation demands to propagate the fracture. Thus, whether or not the weld will fracture depends on a combination of induced stresses in the weld and the imposed drifts. Therefore, for *DS-1* the following two criteria demand parameter is proposed:

Fracture Initiation Trigger: $\sigma_{applied} > 1.5f_u$

Demand Parameter: *Imposed Story Drift > DS-1 Fragility Limit*

where σ_{applied} is the maximum induced stress on the weld due to combined axial load and bending, f_u is the nominal (minimum specified) strength of the weld material. The stress check could either be made based on forces calculated during the nonlinear time history analysis or by simplified calculations to relate the imposed story drifts to the induced stresses, taking into account the structural configuration and member sizes. The DS-1 drift limit is specified by a lognormal fragility curve with a median story demand $\Delta/h = 0.02$ and a dispersion of 0.40. The second damage state is assumed to occur when the story drift is large enough to dislodge the splice. This is estimated as having a median drift demand of $\Delta/h = 0.05$. The resulting fragility information is summarized in Table 5.1.

Table 5.1 - Fragility, damage measures, and consequences for Welded Column Splices

Component category:	Structural	
Basic composition:	Welded column splice consisting of partial penetration groove welds of the column flanges and a web splice plate.	
Units:	Number of connections	
Demand parameter:	Stress Trigger and Story Drift Ratio	
Number of damage states:	3	
If multiple damage states:	DS 1 and 2 are ordered.	
Author and date:	Deierlein and Victorsson, August 23, 2008	
Damage states, fragilities, and consequences		
	DS 1	**DS 2**
Description:	Note 1	Note2
Illustration:	-	-
Median demand (θ)$^{(1)}$:	Stress Trigger (Note 3), 0.02	0.05
Dispersion (β)$^{(1)}$:	0.40	0.40
Probability$^{(1)}$:	-	-
Correlation:	moderate	moderate
Repairs required:	Note 1	Note 2
Possible consequences:		
Repair cost (Y/N/?):	Y	Y
Death or injury (Y/N/?):	-	-
Inoperative facility (Y/N/?):	-	-
Red tagging (Y/N/?)	-	Y
Comments:		

Notes:

1) ***DS-1*:** Ductile fracture of the groove weld flange splice. Repair would involve gouging out the material adjacent to the fracture and repairing with a new groove weld.

2) ***DS-2*:** *DS-1* following by complete failure of the web splice plate and dislocation of the two column segments on either side of the splice. Repair may not be practically feasible, but would require either realignment or replacement of adjacent column segments and rewelding of splice.

3) The stress trigger on DS-1 is to be evaluated as follows: $\sigma_{applied} < 1.5f_u$, where fu is the minimum specified strength of the weld metal and $\sigma_{applied}$ is the maximum *tensile* stress induced by the combination of major- and minor-axis bending and axial load. The stress check could either be made based on forces calculated during the nonlinear time history analysis or by simplified calculations to relate the imposed story drifts to the induced stresses, taking into account the structural configuration and member sizes.

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Appendix A: Spreadsheet with data collected from test reports

Specimen ID	Authors	Test Configuration	$Height_{ex}$	$Length_{ex}$	Column Size	Beam Size	Doubler Plate Thickness	Continuity Plate Thickness	Slab	Beam web attachment detail	Local beam flange or web buckle reported	Lateral Torsional Buckling	Δ/h	θ_p	Building Equivalent Drift	Location of Fracture	Comments
SP 1.1	K.H. Lee, Stojadinovic, Goel, Margarian, Choi, Wongkaew,	2 - Exterior^{T}	144	145	W14x120	W24x68	0	0.375	No	Shear tab with supplemental weld	No	No	0.010	0.007	0.017	Divot type fracture of the bottom flange weld along the backing bar	Pre-Northridge Beam-to-Column Connection. Close to no ductility prior to failure
SP 1.2	-	2- Exterior^{T}	144	145	W14x120	W24x68	0	0.375	No	Shear tab with supplemental weld	No	No	0.010	0.007	0.017	Divot type fracture of the bottom flange weld along the backing bar	Close to no ductility prior to failure
SP 3.1	K.H. Lee, Stojadinovic, Goel, Margarian, Choi, Wongkaew, Reyhe, D.Y. Lee	2 - Exterior^{T}	144	145	W14x120	W24x68	0	0.625	No	Shear tab, no supplemental weld	No	No	0.030	0.016	0.026	Ductile tearing out of the beam top flange	Improved welding
SP 3.2	-	2 - Exterior^{T}	144	145	W14x120	W24x68	0	0.625	No	Shear tab, no supplemental weld	No	No	0.030	0.017	0.027	Ductile tearing out of the beam top and bottom flange	Improved welding
SP 4.1	-	2 - Exterior^{T}	144	145	W14x145	W30x99	0	0.75	No	Shear tab, no supplemental weld	No	No	0.030	0.017	0.027	Fracture in the column k-zone and the continuity plate fillet welds	Improved welding
SP 4.2	-	2 - Exterior^{T}	144	145	W14x145	W30x99	0	0.75	No	Shear tab, no supplemental weld	No	No	0.030	0.019	0.029	Fracture in the beam top flange	Improved welding
SP 5.1	-	2 - Exterior^{T}	144	145	W14x176	W30x99	0	0.75	No	Shear tab, no supplemental weld	No	No	0.040	0.026	0.036	Fracture of both beam flanges	Improved welding
SP 5.2	-	2 - Exterior^{T}	144	145	W14x176	W30x99	0	0.75	No	Shear tab, no supplemental weld	No	No	0.020	0.010	0.020	Fractures in the beam top flanges	Improved welding
SP 6.1	-	2 - Exterior^{T}	144	145	W14x257	W30x99	0	0.75	No	Shear tab, no supplemental weld	No	No	0.020	0.009	0.019	Ductile tearing of the beam top flanges	Improved welding
SP 6.2	-	2 - Exterior^{T}	144	145	W14x257	W30x99	0	0.75	No	Shear tab, no supplemental weld	No	No	0.015	0.006	0.016	Fractures in the beam top flange	Improved welding
SP 7.1	-	2 - Exterior^{T}	144	145	W14x257	W36x150	0	1	No	Shear tab, no supplemental weld	No	No	0.020	0.009	0.019	Fractures of the beam bottom flange	Improved welding
SP 7.2	-	2 - Exterior^{T}	144	145	W14x257	W36x150	0	1	No	Shear tab, no supplemental weld	No	No	0.030	0.018	0.028	Fractures of the beam bottom flange	Improved welding
Δ/h :	Maximum drift during test																
θ_p :	Maximum total plastic deformation during test																

Specimen ID	Authors	Test Configuration	$Height_{ex}$	$Length_{ex}$	Column Size	Beam Size	Doubler Plate Thickness	Continuity Plate Thickness	Slab	Beam web attachment detail	Local beam flange or web buckle reported	Lateral Torsional Buckling	Δ/h	θ_p	Building Equivalent Drift	Location of Fracture	Comments
1	Engelhardt & Hussein	2-Exterior	144	96	W12x136	W24x55	0	0.5	No	Bolted	No	No		0.004	0.014	Sudden fracture at weld-column interface at bottom flange	Pre-Northridge.
2	-	2-Exterior	144	96	W12x136	W24x55	0	0.5	No	Bolted	No	No		0.003	0.013	Sudden fracture at weld-column interface at bottom flange	Pre-Northridge.
3	-	2-Exterior	144	96	W12x136	W24x55	0	0.5	No	Bolted + Web weld	No	No		0.009	0.019	Gradual fracture through bottom beam flange, outside of weld	Pre-Northridge.
4	-	2-Exterior	144	96	W12x136	W18x60	0	0.5	No	Bolted	No	No		0.002	0.012	Sudden fracture at weld-column interface at bottom flange	Pre-Northridge.
5	-	2-Exterior	144	96	W12x136	W18x60	0	0.5	No	Bolted	No	No		0.013	0.023	Gradual fracture at weld-beam interface at bottom flange	Pre-Northridge.
6	-	2-Exterior	144	96	W12x136	W21x57	0	0.5	No	Bolted	No	No		0.013	0.023	Gradual fracture at weld-beam interface at bottom flange	Pre-Northridge.
7	-	2-Exterior	144	96	W12x136	W21x57	0	0.5	No	Bolted + Web weld	No	No		0.015	0.025	Gradual fracture at weld-beam interface at top flange	Pre-Northridge.
8	-	2-Exterior	144	96	W12x136	W21x57	0	0.5	No	All welded	No	No		0.012	0.022	Gradual fracture at weld-beam interface at bottom flange	Pre-Northridge.

Δ/h : Maximum drift during test

θ_p : Maximum total plastic deformation during test

Specimen ID	Authors	Test Configuration	H_{ex}	L_{ex}	Column Size	Beam Size	Doubler Plate Thickness	Continuity Plate Thickness	Slab	Beam web attachment detail	Drift at which local beam flange or web buckle reported - DS1	Drift at which LTB is reported - DS2	θ_{beam}	θ_{PZ}	θ_{column}	Δ/h	θ_p	Building Equivalent Drift - DS3	Location of Fracture	Comments
T1	James M. Ricles, Changshi Mao, Le-Wu Lu, John W. Fisher	1 - Exterior	156	177	W14x311	W36x150	0	1	No	Groove welded, supplemental fillet welds on shear tab	0.03	Not Reported	0.019	0.012	0.004	0.050	0.035	0.050	Beam HAZ, bottom flange	The groove weld on the shear tab started to crack during the 2% drift cycles
T2	-	1 - Exterior	156	177	W14x311	W36x150	0	1	No	Groove welded web, no supplemental fillet welds	0.03	Not Reported	0.01	0.013	0.002	0.040	0.025	0.040	Beam HAZ, top flange	Fracture occured at 0.04 rad drift but test was performed to 0.05 rad drift at which point beam web weld fractured
T3	-	1 - Exterior	156	177	W14x311	W36x150	0	1	No	Fillet welded shear tab	0.03	Not Reported	0.012	0.005	0.003	0.030	0.020	0.030	Shear tab, beam HAZ, bottom flange	The fillet weld of the shear tab was not strong enough to resist the moment which was transferred from the beam web by the shear tab.
T4	-	1 - Exterior	156	177	W14x311	W36x150	0	1	No	Bolted Shear tab	0.03	Not Reported	0.009	0.007	0.002	0.040	0.018	0.040	Beam HAZ, top flange	The beam top flange developed a through thickness crack of 3 inches length across the beam flange at the interface of the weld and base metals, initiating at the edge of the flange during the
T5	-	1 - Exterior	156	177	W14x311	W36x150	0.5	0	No	Groove welded web, supplemental fillet welds	0.03	Not Reported	0.051	0.002	0.001	0.060	0.054	0.060	Beam at local buckle, bottom flange	Improved performance by using doubler plates
T6	-	1 - Exterior	156	177	W14x311	W36x150	0.5	0	No	Groove welded heavy shear tab, fillet welded beam web	0.04	Not Reported	0.049	0.008	0.003	0.060	0.050	0.060	None	Test terminated at 0.06 rad drift without fracture.
C1	-	1 - Interior	156	354	W14x398	W36x150	2x0.75	0	No	Groove welded web, supplemental fillet welds	0.03	Not Reported	0.03	0.007	0.002	0.050	0.039	0.050	Beam HAZ, top flange	No LTB but severe local buckling at 0.04 rad drift
C2	-	1 - Interior	156	354	W14x398	W36x150	2x0.75	1	No	Groove welded web, supplemental fillet welds	0.03	Not Reported	0.05	0	0	0.060	0.050	0.060	Beam flanges at local buckle	No LTB but severe local buckling at 0.04 rad drift
C3	-	1 - Interior	156	354	W27x258	W36x150	2x0.625	0	No	Groove welded web, supplemental fillet welds	0.03	Not Reported	0.041	0	0	0.055	0.041	0.055	Beam HAZ, top flange	No LTB but severe local buckling at 0.04 rad drift
C4	-	1 - Interior	156	354	W27x258	W36x150	2x0.625	1	No	Groove welded web, supplemental fillet welds	0.03	Not Reported	0.051	0.001	0	0.060	0.052	0.060	None	No LTB but severe local buckling at 0.04 rad drift
C5	-	1 - Interior	156	354	W14x398	W36x150	2x0.75	0	Yes	Groove welded web, supplemental fillet welds	0.03	Not Reported	0.022	0.004	0	0.040	0.026	0.040	Shear stud, top flange	No LTB but severe local buckling at 0.04 rad drift

θ_{beam} :	Plastic Deformation in the beam at maximum drift	Δ/h :	Maximum drift during test
θ_{PZ} :	Plastic Deformation of PZ at maximum drift	θ_p :	Maximum total plastic deformation during test

Specimen ID	Authors	Test Configuration	H_{ex}	L_{ex}	Column Size	Beam Size	Doubler Plate Thickness	Continuity Plate Thickness	Slab	Beam web attachment detail	Drift at which local beam flange or web buckle reported - DS1	Drift at which LTB is reported - DS2	θ_{beam}	θ_{PZ}	Dist to where yield in beam occurs from face of column	Δ/h	θ_p	Building Equivalent Drift - DS3	Location of Fracture	Comments
RC01	T. Kim, A.S. Whittaker, A.S.J. Gilani, V.V. Bertero, S.M. Takhirov	2 - Exterior	134	136	W14x176	W30x99	0.4	1.125	No	Beam web was groove welded to the column flange, cover plate connection	0.03	Not Reported	0.041		15	0.048	0.041	0.044	Low-cycle fatigue fracture of the beam flange or k-line region	Cover-Plate. Small Local buckling starts at 0.015 rad
RC02	-	2 - Exterior	134	136	W14x176	W30x99	0.4	1.125	No	Beam web was groove welded to the column flange, cover plate connection	0.03	No	0.042	-	15	0.048	0.042	0.045	Low-cycle fatigue fracture of the beam flange or k-line region	Cover-Plate. Restraint to LTB. Small local buckling starts at 0.015 rad
RC03	-	2 - Exterior	134	136	W14x176	W30x99	0.4	1.125	No	Beam web was groove welded to the column flange, cover plate connection	0.03	0.04	0.043	-	15	0.05	0.043	0.046	Low-cycle fatigue fracture of the beam flange or k-line region	Cover-Plate. Small Local buckling starts at 0.015 rad
RC04	-	2 - Exterior	134	136	W14x176	W30x99	0.4	1.125	No	Beam web was groove welded to the column flange, flange plate connection	0.03	Not Reported	0.049	-	15	0.053	0.049	0.051	Low-cycle fatigue fracture of the beam flange or k-line region	Flange-Plate. Small local buckling starts at 0.020 rad
RC05	-	2 - Exterior	134	136	W14x176	W30x99	0.4	1.125	No	Beam web was groove welded to the column, cover plate connectoin	0.03	Not Reported	0.052	-	15	0.055	0.052	0.053	Low-cycle fatigue fracture of the beam flange or k-line region	Cover-Plate. Small local buckling starts at 0.020 rad
RC06	-	2 - Exterior	134	136	W14x176	W30x99	0.4	1.125	No	Beam web was groove welded to the columns, flange plate connection	0.03	0.04	0.042	-	15	0.048	0.042	0.045	Low-cycle fatigue fracture of the beam flange or k-line region	Flange-Plate. Small local buckling starts at 0.02 and small LTB at 0.03 rad
RC07	-	2 - Exterior	134	136	W14x176	W30x99	0.4	1.125	No	Beam web was groove welded to the column, flange plate connection	-		0.05	-	15	0.054	0.050	0.052	Low-cycle fatigue fracture of the beam flange or k-line region	Flange-Plate. Near-fault loading history
RC08	-	2 - Exterior	134	136	W14x176	W30x99	0.4	1.125	No	Beam web was groove welded to the column, flange plate connection	0.03	0.04	0.05	-	15	0.055	0.050	0.052	Low-cycle fatigue fracture of the beam flange or k-line region	Flange-Plate. Small local buckling starts at 0.02 and small LTB at 0.03 rad
RC09	-	2 - Exterior	134	136	W14x176	W30x99	0	1.125	No	Beam web was groove welded to the column, flange plate connection	0.03	Not Reported	0.056	-	15	0.053	0.056	0.057	Low-cycle fatigue fracture of the beam flange or k-line region	Flange-Plate. Small local buckling observed at 0.02 rad
RC10	-	2 - Exterior	134	136	W14x176	W30x99	0.4	1.125	No	Beam web was groove welded to the column, cover plate connectoin	0.03	Not Reported	0.045	-	15	0.052	0.045	0.048	Low-cycle fatigue fracture of the beam flange or k-line region	Cover-Plate

θ_{beam} :	Plastic Deformation in the beam at maximum drift	Δ/h :	Maximum drift during test
θ_{PZ} :	Plastic Deformation of PZ at maximum drift	θ_p :	Maximum total plastic deformation during test

Specimen ID	Authors	Test Configuration	H_{ex}	L_{ex}	Column Size	Beam Size	Doubler Plate Thickness	Continuity Plate Thickness	Slab	Beam web attachment detail	Drift at which local beam flange or web buckle reported - DS1	Drift at which LTB is reported - DS2	θ_{beam}	θ_{PZ}	θ_{column}	Δ/h	θ_p	Building Equivalent Drift - DS3	Location of Fracture	Comments
FFC1	Venti & Engelhardt	1 - Interior	146	300	W14x398	W36x150	0.75	1	Yes	Free Flange Connection. Heavy Shear Tab.	0.03	No LTB	0.011	0.017	0.005	0.050	0.033	0.050	Toe of weld access hole.	No Degredation in Strength. Mainly panel zone plastic rotation. Cracks in shar tab at 0.04 rad drift. 3/4 inch thick doubler plate
															Dist to plastic hinge from column face					
FFNC1	Chad Gilton, Brandon Chi, Chia-Ming Uang	2 - Exterior	150	141	W14x257	W36x150	0.25	1	No	Free Flange Connection	0.03	No LTB	0.006	0.012	12	0.03	0.018	0.027	Fracture across the column flange near the toe of the shear plate groove weld	No Degredation in strength. Column flange kinking. Mainly panel zone shear deformations.
													Approx distribution of plastic rotation	**Approx distribution of plastic rotation**	**Dist to plastic hinge from column face**					
SP 8.2	Choi, Stojadinovic, Goel	2 - Exterior[T]	144	145	W14x120	W24x68	0.625	0.75	No	Beam Web welded to the shear tab using a fillet weld on both sides	0.03	0.04	85%	15%	12	0.05	0.042	0.047	No Fracture - LTB caused failure	Free Flange Connection
SP 9.1	-	2 - Exterior[T]	144	145	W14x176	W30X99	0	0.75	No	Beam Web welded to the shear tab using a fillet weld on both sides	0.02	Little LTB	50%	50%	0	0.05	0.037	0.046	Ductile fracture in the bottom flange	Free Flange Connection
SP 9.2	-	2 - Exterior[T]	144	145	W14x176	W30X99	0.75	0.75	No	Beam Web welded to the shear tab using a fillet weld on both sides	0.0175	0.03	100%	0%	20	0.04	0.034	0.037	No Fracture - LTB caused failure	Free Flange Connection
SP 10.1	-	2 - Exterior[T]	144	145	W14x257	W30x124	0	0.75	No	Beam Web welded to the shear tab using a fillet weld on both sides	0.02	No LTB	55%	45%	7	0.04	0.025	0.034	No Fracture - Max capacity of actuator reached	Free Flange Connection. Cracking occured at the weld of the shear tab and column face
SP 10.2	-	2 - Exterior[T]	144	145	W14x256	W30x124	0.5	0.75	No	Beam Web welded to the shear tab using a fillet weld on both sides	0.03	0.04	85%	15%	15	0.04	0.027	0.033	No Fracture - Max capacity of actuator reached	Free Flange Connection. Small crack was observed at the beam top flange bottom layer, the junction of the beam flange and web near the access hole at the last cycle of the test

θ_{beam} : Plastic Deformation in the beam at maximum drift — **Δ/h :** Maximum drift during test

θ_{PZ} : Plastic Deformation of PZ at maximum drift — **θ_p :** Maximum total plastic deformation during test

Specimen ID	Authors	Test Configuration	H_{ex}	L_{ex}	Column Size	Beam Size	Distance to center of RBS	RBS Flange Reduction	Beam web attachment detail	Slab	Panel Zone	Drift at which local beam flange or web buckle reported - DS1	Drift at which LTB is reported - DS2	Lateral Torsional Buckling Reported	Δ/h	θ_p	Building Equivalent Drift - DS3	Location of Fracture	Comments
Spec 1	James M. Ricles, Xiaofeng Zhang, Le-Wu Lu, John Fisher	1 - Interior	156	354	W36x230	W36x150	22.5	50%	Welded	Yes	0.25	1	0.025	0.035	0.050	0.043	0.050	Beam bottom flanges, in RBS	RBS. Slab on top. The fracture was due to low cycle fatigue
Spec 2	-	1 - Interior	156	354	W27x194	W36x150	22.5	50%	Welded	Yes	0.5	1	0.025	0.035	0.050	0.040	0.050	Beam top flange, in RBS	Slab on top. The fracture was due to low cycle fatigue
Spec 3	-	1 - Interior	156	354	W27x194	W36x150	22.5	50%	Welded	Yes	0.5	1	0.025	0.035	0.060	0.052	0.060	Beam bottom flanges, in RBS	Slab on top. The fracture was due to low cycle fatigue. Fracture initiated at 0.05 drift
Spec 4	-	1 - Interior	156	354	W36x150	W36x150	22.5	50%	Welded	Yes	0.375	a	0.025	0.035	0.060	0.053	0.060	Beam bottom flanges, in RBS	Slab on top. The fracture was due to low cycle fatigue. Fracture initiated at 0.05 drift
Spec 5	-	1 - Interior	156	354	W27x146	W30x108	22.5	50%	Welded	Yes	0.375	0.75	0.035	0.045	0.060	0.050	0.060	Beam bottom flanges, in RBS	Slab on top. The fracture was due to low cycle fatigue
Spec 6	-	1 - Interior	156	354	W24x131	W30x108	22.5	50%	Welded	No	0.5	0.75	0.03	N/A	0.050	0.040	0.050	Beam botton flanges, HAZ	No beam lateral buckling. Fracture initiated from the crack at the flange fillet weld root
		Δ/h :	Maximum drift during test																
		θ_p :	Maximum total plastic deformation during test																

Specimen ID	Authors	Test Configuration	H_{ex}	L_{ex}	Column Size	Beam Size	Distance to center of RBS	RBS Flange Reduction	Beam web attachment detail	Slab	Panel Zone	Drift at which local beam flange or web buckle reported - DS1	Drift at which LTB is reported - DS2	Lateral Torsional Buckling Reported	Δ/h	θ_p	Building Equivalent Drift - DS3	Location of Fracture	Comments
DBWW	Engelhardt & Venti	1 - Interior	146	300	W14x398	W36x150	22.5	50%	Bolted	No	Balanced	1	0.03	0.04	0.050	0.040	0.050	No fracture reported. Strength below 80% of peak strength	RBS. No Fracture. Test stopped due to lateral movement of column damaging test setup
DBBWC	-	1 - Interior	146	300	W14x398	W36x150	22.5	50%	Bolted	Yes	Balanced	1	0.04	0.05	0.050	0.040	0.050	Fracture initiated in the region of the weld access hole	Failed by fast fracture
DBBWWPZ	-	1 - Interior	146	300	W14X283	W36x150	22.5	40%	Bolted	No	Very Weak	1	N/A	N/A	0.040	0.030	0.040	Fracture initiated close to the interface between the groove weld and the beam flange and then propagated primarily through beam flange base metal	Virtually all yielding was within the panel zone. Panel zone shear distortion and column flange "kinging" was clearly visible
DBBWWPZC	-	1 - Interior	146	300	W14X283	W36x150	22.5	40%	Bolted	Yes	Very Weak	1	N/A	N/A	0.060	0.046	0.060	Facture within the beam flange base metal	Small cracks visible in beam bottom flanges during the 0.05 and 0.06 rad drift angle cycles
DBWW	-	1 - Interior^T	144	300	W14X398	W36x150	22.5	50%	Welded	No	Balanced	1	0.02	0.03	0.040	0.030	0.040	No fracture reported. Strength below 80% of peak strength	Lateral Torsional Buckling and twist of column caused damage to column base plate and clevise assembly. Test terminated to prevent further damage to the test setup.
DBWWC	-	1 - Interior^T	144	300	W14x398	W36x150	22.5	50%	Welded	Yes	Balanced	1	0.03	0.05	0.040	0.030	0.040	Stable ductile tearing through top flanges of beams at local flange buckels within the reduced sections	
DBBWSPZ	-	1 - Interior^T	144	300	W14x398	W36x150	22.5	50%	Bolted	No	2x0.75	1	0.02	0.03	0.030	0.020	0.030	No fracture reported. Strength below 80% of peak strength	Columns provided with 2 3/4 inch doubler plates. Test terminated after 0.03 rad because out-of-plane displacement of column top exceeded in-plane displacement.
DBBWSPZC	-	1 - Interior^T	144	300	W14x398	W36x150	22.5	50%	Bolted	Yes	2x0.75	1	0.025	0.04	0.060	0.050	0.060	No fracture reported. Strength below 80% of peak strength	Loading terminated after 0.07 rad story drift angle because the column top began to displace substantially out-of-plane
		Δ/h :	Maximum drift during test																
		θ_p :	Maximum total plastic deformation during test																

Specimen ID	Authors	Test Configuration	H_{ex}	L_{ex}	Column Size	Beam Size	Distance to center of RBS	RBS Flange Reduction	Beam web attachment detail	Slab	Panel Zone	Drift at which local beam flange or web buckle reported - DS1	Drift at which LTB is reported - DS2	Lateral Torsional Buckling Reported	Δ/h	θ_p	Building Equivalent Drift - DS3	Location of Fracture	Comments
LS1	Qi Song Yu, Chad Gilton, Chia-Ming Uang	2 - Exterior	149	150	W14x176	W30x99	17		Welded	No	0	0.75	0.015	0.03	0.050	0.045	0.048	No fracture reported. Strength below 80% of peak value	Was below the 80% strength at 0.04 rad drift - SEE WHICH VALUE TO USE
LS2	-	2 - Exterior	149	150	W14x176	W30x99	17		Welded	No	0	0.75				0.050	due to different loading history		Near-fault loading history
LS3	-	2 - Exterior	149	150	W14x176	W30x99	17		Welded	No	0	0.75				0.050	due to different loading history		Near-fault loading history
LS4	-	2 - Exterior	149	150	W14x176	W30x99	17		Welded	No	0	0.75	0.015	0.03	0.05	0.04	0.043	No fracture reported. Strength below 80% of peak value	Additional Bracing for LS-4 6 inches outside of RBS region
CW1	Chad Gilton, Brandon Chi, Chia-Ming Uang	2 - Exterior	150	141	W14x398	W36x150	27	50%	Welded	No	0	1.125	0.02	0.03	0.05	0.042	0.041	No fracture reported. Strength below 80% of peak value and test halted.	Weak Axis Moment Connection. 3/4 inch LTB is reported at 0.02 rad drift
CW2	-	2 - Exterior	150	141	W14x176	W24x62	16	50%	Welded	No	0	0.625	0.02	0.03	0.05	0.045	0.047	No fracture reported. Strength below 80% of peak value and test halted.	Weak Axis Moment Connection. 1/2 inch LTB is reported at 0.02 rad drift
DC1	-	2 - Exterior	150	141	W27x146	W36x150	24	50%	Welded	No	0.375	1	0.02	0.03	0.04	0.03	0.032	No fracture reported. Test halted due to beam buckling and out-of-plane deformation of the column.	At 0.02 rad drift minor (7/8in) LTB was reported.
DC2	-	2 - Exterior	150	141	W27x194	W36x150	24	50%	Welded	No	0	1	0.02	0.03	0.05	0.044	0.042	No fracture reported. Strength below 80% of peak value and test halted.	At 0.015 rad drift minor buckling of the web was noticed. LTB of almost 1 in was reported at 0.02 rad. By 4% drift, out-of-plane deformation of the column flanges due to twisting reached almost 3/4 inch.
DC3	-	2 - Exterior	150	141	W27x194	W27x194	22.5	50%	Welded	No	0.625	1	0.02	0.02	0.04	0.028	0.031	Brittle fracture along the k-line of the column at the beam bottom flange.	WLB and LTB became obvious at 0.02 rad drift. Slight out of plane deformation of the column flanges as well as significant yielding of the continuity plates reported at 0.03 rad drift
		Δ/h :	Maximum drift during test																
		θ_p :	Maximum total plastic deformation during test																

Specimen ID	Authors	Journal	Test Configuration	H_{ex}	L_{ex}	Column Size	Column Axis	Beam Size	Elongation of Bolt Holes	Crushing of Concrete. Loss of Composite Action	Initial Fracture of Shear Tab	Compl. Fract. of Shear Tab	Fracture of Shear Bolts	Slab	Beam web attachment detail	Number of Bolts on Web	Comments
1A	Judy iu and Astaneh-Asl	SAC 00-03	1 - Interior	120	300	W14x90	Weak	W18x35	0.06	NA	0.11	0.14	NR	No	Shear Tab	4	Post 1980
2A	-	-	-	120	300	W14x90	Strong	w24x55	0.03	NA	0.07	0.09	NR	No	Shear Tab	6	Post 1980
3A	-	-	-	120	300	W14x90	Weak	W18x35	0.08	0.04	0.12	0.15	NR	Light Weight	Shear Tab	4	Post 1980
4A	-	-	-	120	300	W14x90	Weak	W18x35	0.04	0.03	0.1	0.11	NR	Light Weight	Shear Tab	4	Post 1980
5A	-	-	-	120	300	W14x90	Strong	W18x35	NA	0.03	NA	NA	NA	Light Weight	Stiffened Seat	0	Post 1980
6A	-	-	-	120	300	W14x90	Srong	w24x55	0.03	0.03	0.06	0.11	NR	Light Weight	Shear Tab	6	Post 1980
7A	-	-	-	120	300	W14x90	Strong	w24x55	0.04	0.04	0.07	0.09	NR	Light Weight	Shear Tab	6	Post 1980
8A	-	-	-	120	300	W14x90	Strong	w24x55	0.04	0.04	0.05	0.09	NR	Light Weight	ST + Seat	6	Post 1980
1B	Gravity BSC	-	-	120	300	W14x90	Weak	W18x35	0.05	NA	0.12	0.12	NR	No	Shear Tab	3	Pre 1980
2B	-	-	-	120	300	W14x90	Strong	w24x55	0.05	NA	0.09	0.11	NR	No	Shear Tab	4	Pre 1980
3B	-	-	-	120	300	W14x90	Weak	W18x35	0.09	0.04	0.14	0.14	NR	Normal	Shear Tab	4	Post 1980
4B	-	-	-	120	300	W14x90	Strong	w24x55	0.03	0.04	0.04	0.1	NR	Normal	Shear Tab	6	Post 1980
5B	-	-	-	120	300	W14x90	Strong	w24x55	0.04	0.04	NR	NR	NR	Normal	Shear Tab	4	Pre 1980
6B	-	-	-	120	300	W14x90	Strong	w24x55	0.03	0.03	0.05	0.09	NR	Normal	Shear Tab	6	Post 1980
7B	-	-	-	120	300	W14x90	Strong	W33x118	0.03	0.04	0.05	NR	0.08	Normal	Shear Tab	8	Post 1980
8B	-	-	-	120	300	W14x90	Strong	w24x55	0.04	0.03	NR	NR	0.06	Normal	ST + Seat	0	Post 1980

Specimen ID	Authors	Column Size	Weld Detail	Story Drift at Crack Initiation	Story Drift at Total Flange Fracture	Location of Fracture	Loading Protocol			
Test 1	Kanvinde et al.	W8x67	CJP	0.03	0.05	Corner-HAZ	Far-Field			
Test 2	-	W8x67	CJP	0.06	0.06	Corner-HAZ	Near-Field			
Test 3	-	W8x67	CJP	0.04	0.06	Access Hole	Far-Field			
Test 4	Base Plate	W8x67	CJP	0.04	0.06	Corner-HAZ	Far-Field			
Test 5	-	W8x67	PJP	0.05	0.09	Corner-HAZ	Far-Field			
Test 6	-	W8x67	PJP	0.06	0.08	Corner-HAZ	Far-Field			
Fahmy 2	Mohamed Fahmy	W10x77	PJP	0.02	0.02	Flange Weld	Far-Field	Old Weld Detail		
Fahmy 3	-	W10x77	PJP	0.05	0.06	Flange Weld	Far-Field			
Burda T1	Burda and Itani	W8x48	Fillet Weld	0.031	0.094	Flange Weld	Far-Field	Non-Sac Loading Protocol		1.923076923
Burda T2	-	W8x48	Fillet Weld	0.031	0.073	Flange Weld	Far-Field	Non-Sac Loading Protocol		
Burda T3	-	W8x48	Fillet Weld	0.031	0.063	Flange Weld	Far-Field	Non-Sac Loading Protocol		
Burda T4	Base Plate	W8x48	Groove Weld	0.052	0.135	Flange Weld	Far-Field	Non-Sac Loading Protocol		
Burda T5	-	W8x48	Groove Weld	0.052	0.094	Flange Weld	Far-Field	Non-Sac Loading Protocol		
Burda T6	-	W8x48	Groove Weld	0.052	0.073	Flange Weld	Far-Field	Non-Sac Loading Protocol		
SP 4-1	Daeyong Lee	W12x96	PJP	0.0100	NA	No Fracture	Far-Field			
SP 4-2	-	W12x96	PJP	0.0075	0.015	Flange Weld	Far-Field			
SP 6-1	-	W12x96	PJP	0.0100	0.030	Flange Weld	Far-Field			
SP 6-2	-	W12x96	PJP	0.0075	0.030	Flange Weld	Far-Field			

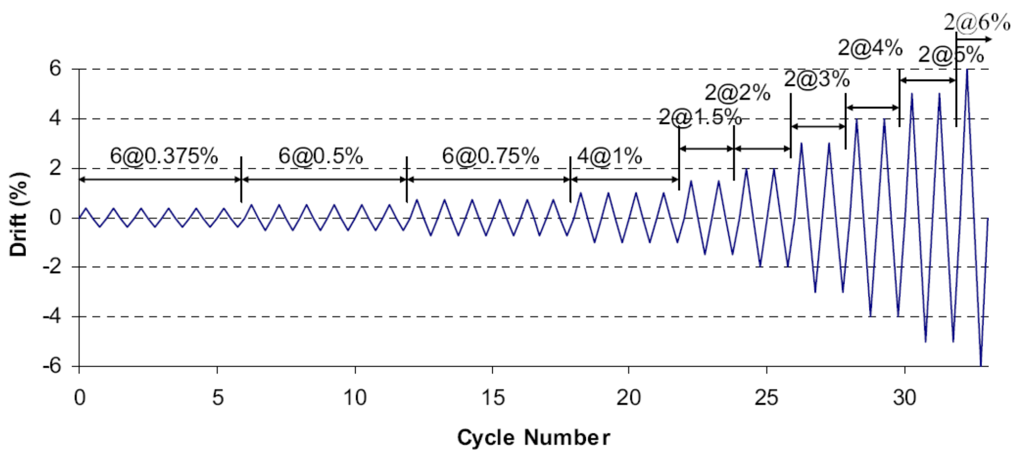
θ_{beam} :	Plastic Deformation in the beam at maximum drift	Δ/h :	Maximum drift during test
θ_{PZ} :	Plastic Deformation of PZ at maximum	θ_p :	Maximum total plastic deformation during test

Appendix B: Supplemental information

This appendix includes supplemental information that was extracted from the papers and reports of laboratory tests of steel beam-column connections and is referenced in the database of tests and observations included in Appendix A.

B.1 Loading Protocol

Generally, all of the connection test used the SAC loading protocol (or a close variant of this), except for the six tests of column base connections by Burda and Itani (1999). The SAC loading protocol is shown in Figure B1.1.



(a)

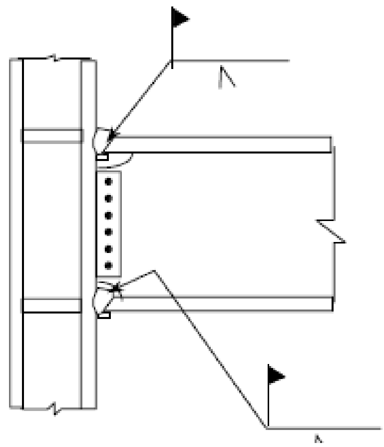
Load Step #	Drift	Number of Cycles, n
1	0.00375	6
2	0.005	6
3	0.0075	6
4	0.01	4
5	0.005	2
6	0.015	2
7	0.005	2
8	0.02	2
9	0.005	2
10	0.03	2
11	0.005	2
12	0.04	2
13	0.005	2
14	0.05	2
15	0.005	2
16	0.05	Until failure

(b)

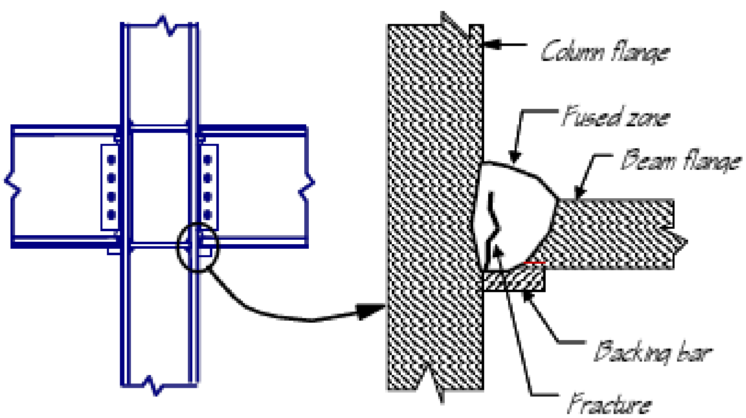
Figure B1.1:Images of the SAC loading protocol a) Loading Protocol b) Loading History

B2. Typical Connection Details

Shown in Figures B2.1 to B2.4 are typical details for the beam-column connections, shear tab connections, and column base connections as reported in the literature for steel SMF systems.



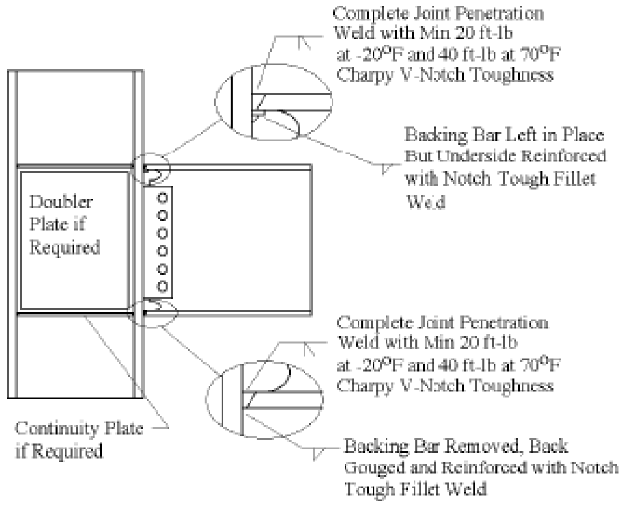
(a)



(b)

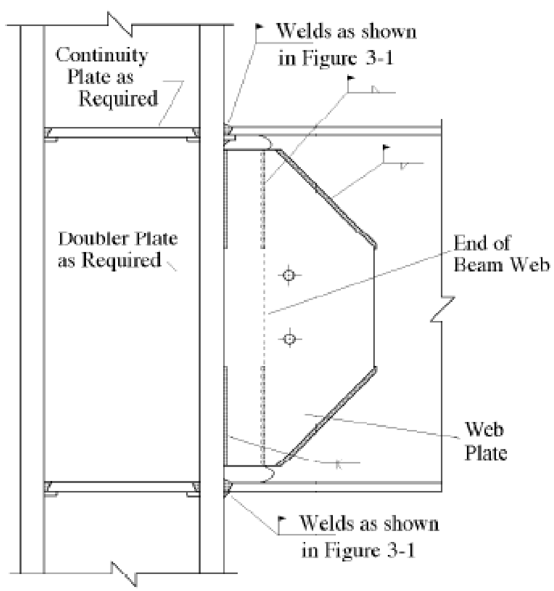
Figure B2.1 Typical Pre-Northridge moment connections a) Exterior connection b) Interior connection

Welded Flange Bolted Web



(a)

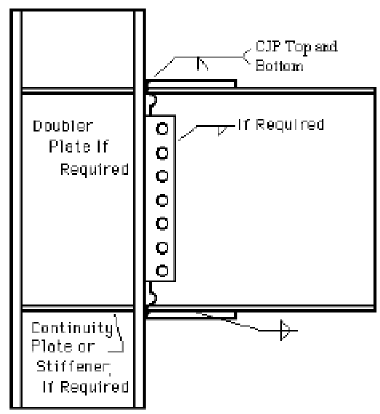
Free Flange Connection



(b)

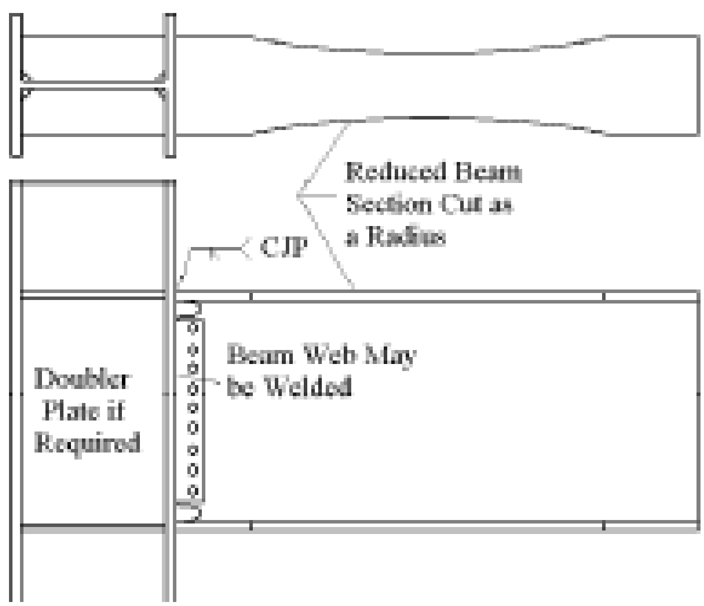
Figure B2.2 Post-Northridge Beam-to-Column connections a)Welded Flange Bolted Web b) Free Flange Connection

Cover Plate Connection



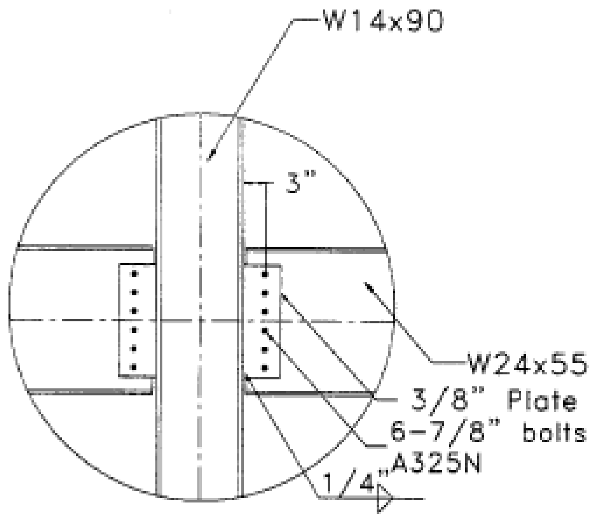
(a)

Reduced Beam Section

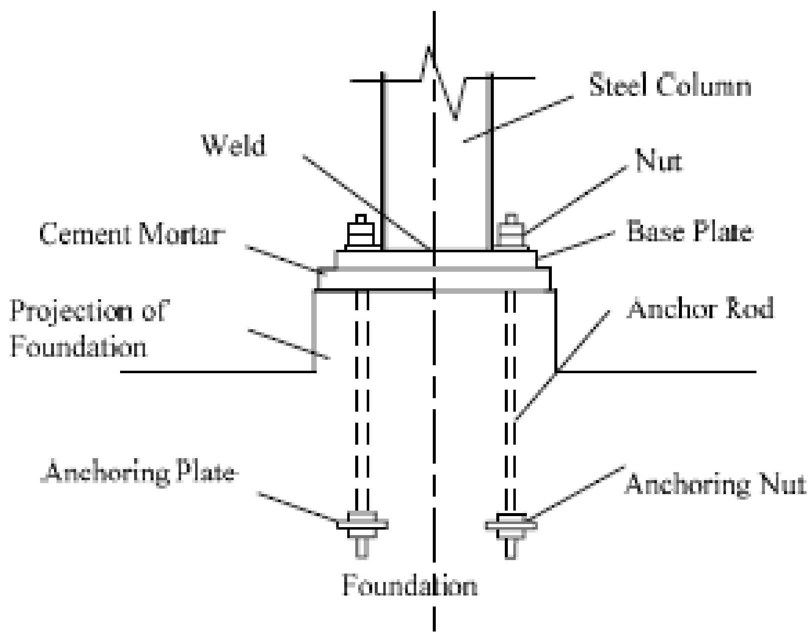


(b)

Figure B2.3: Post-Northridge Beam-to-Column connections a) Cover Plate Connections b) Reduced Beam Section



(a)

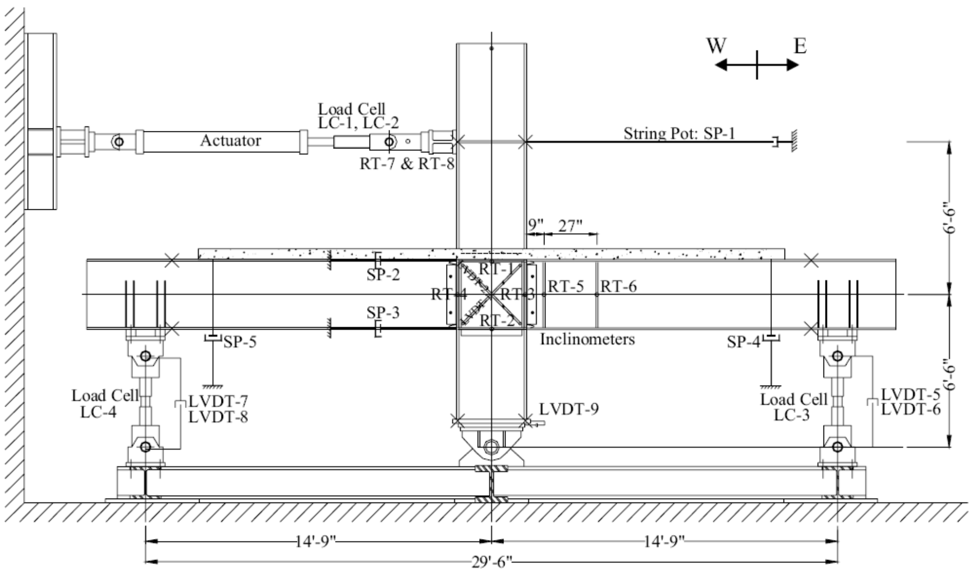


(b)

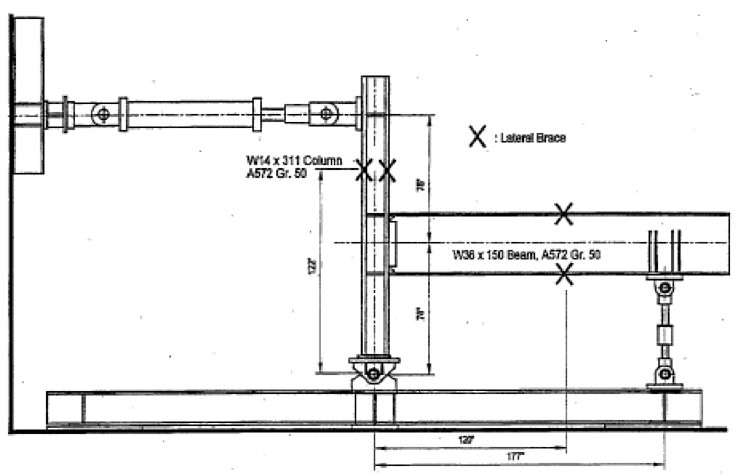
Figure B2.4: a) Typical Gravity Beam Shear connection, b) Column Base Plate connection

B3. TEST CONFIGURATIONS

Figures B3.1 to B3.5 shown the typical test configurations for the steel frame components. The figure titles indicate the test configuration types as references in the spreadsheet database of test information.



(a)



(b)

Figure B3.1: Test configurations of Beam-to-Column connections a) Test Configuration 1 – Interior b) Test Configuration 1 - Exterior

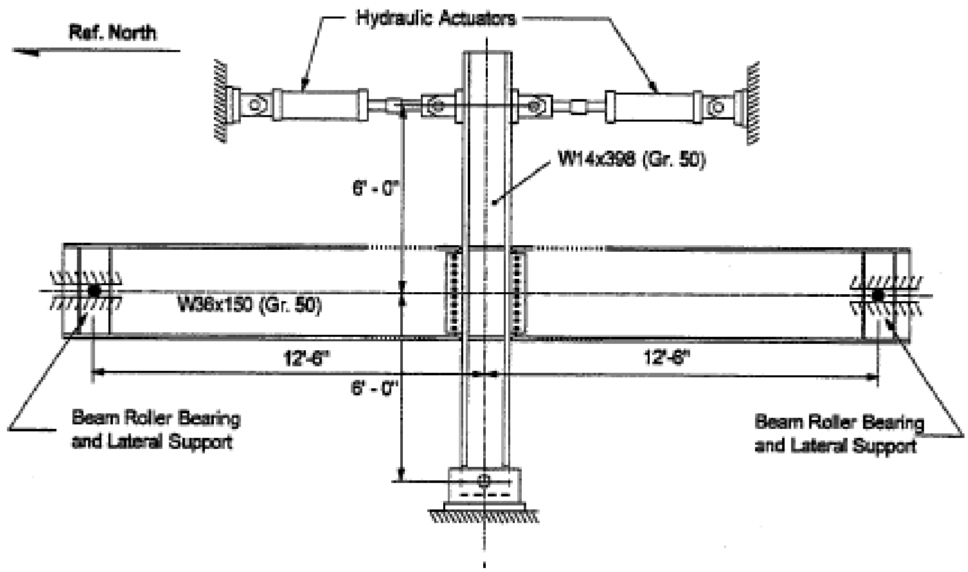
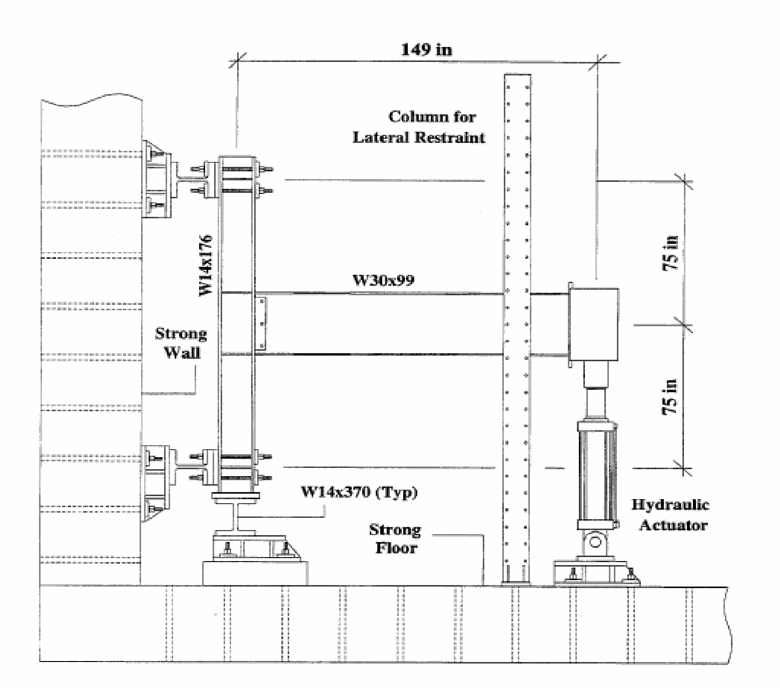
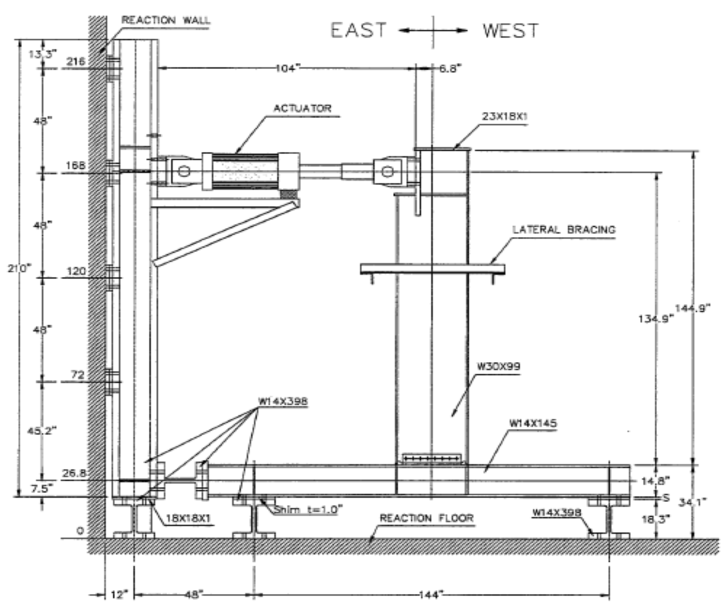


Figure B3.2: Beam-to-Column Test Configuration 1^T – Interior



(a)



(b)

Figure B3.3 Test configurations of Beam-to-Column connections a) Test Configuration 2 – Exterior b) Test Configuration 2^T - Exterior

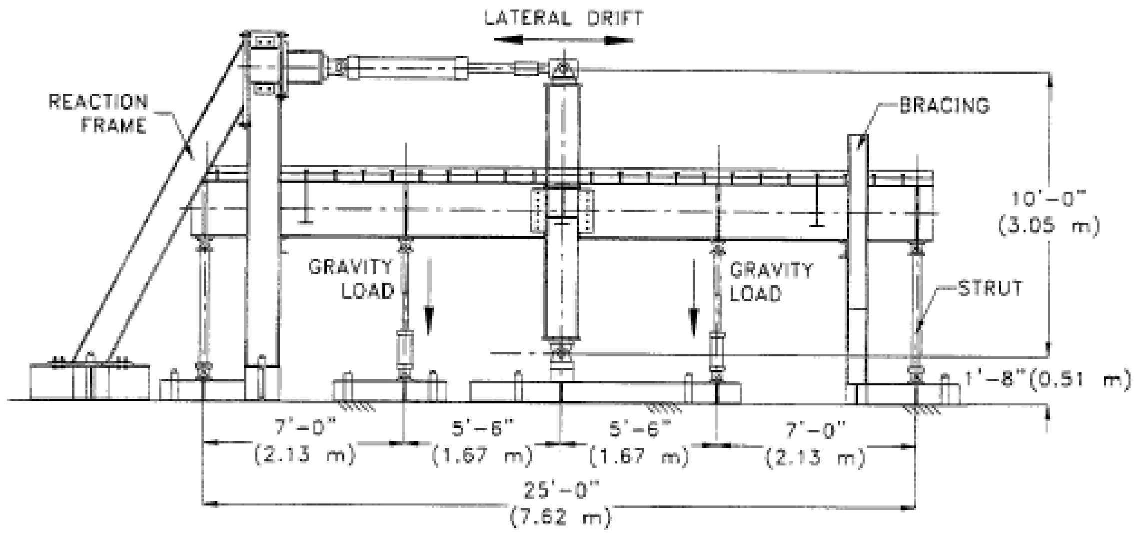
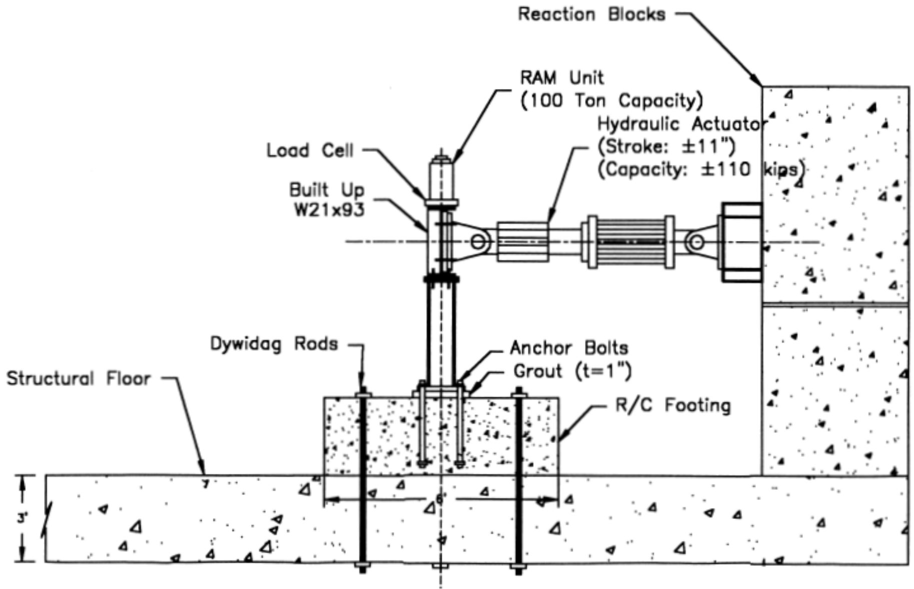
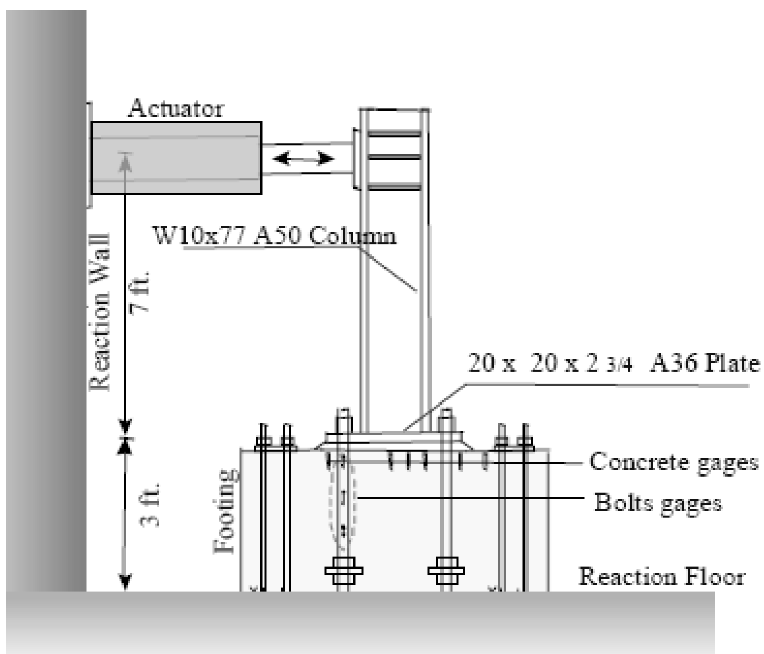


Figure B3.4: Gravity beam shear test configuration



(b)



(a)

Figure B3.5: Column Base Plate test configurations a) Burda and Itani (1999) b) Fahmy (1999)

B.4 Transforming drift from test results to equivalent building drift

For beam-to-column connections the test results were transformed to equivalent building drift based on the idealized definitions shown in Figures B4.1 and B4.2

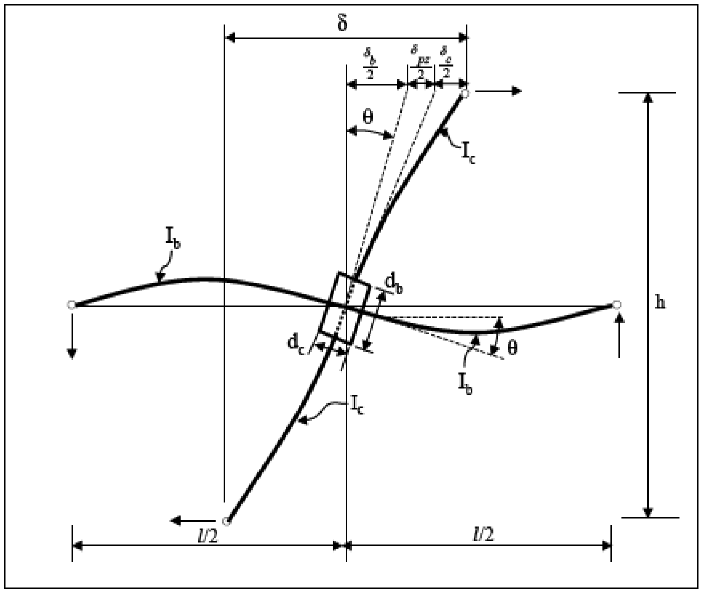


Figure B4.1 Equivalent building drifts (Gupta and Krawinkler (2002))

Components of lateral deflection in beam-column assembly

Test is considered full scale if:
a) Bay width is between 20ft and 30ft and
b) Height is between 12ft and 14ft and
c) Columns are of sections: W14, W24, W27, W36 and
d) Beams are of sections: W24, W27, W30, W33, and W36

1. If test is full scale and of Test Configuration 1:

   Test results are equivalent building drift

2. If test is either small or full scale and of Test Configuration 2:

   a) The plastic deformation due to beam hinging was transformed to column drift.

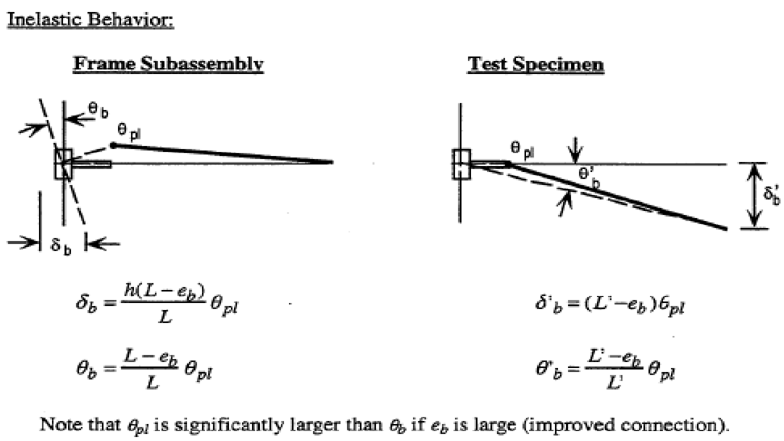


Figure B4.2 Test specimen to frame subassembly (Gupta and Krawinkler (2002))

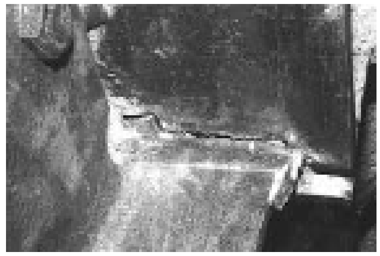
Where e_b is the distance from the center of the joint panel zone to where the plastic hinge was formed and in the case of RBS section to the center of the reduced section.

b) The plastic deformation in the panel zone is not transformed

c) Elastic deformation is assumed to be 0.01 rad and added to the transformed plastic deformations.

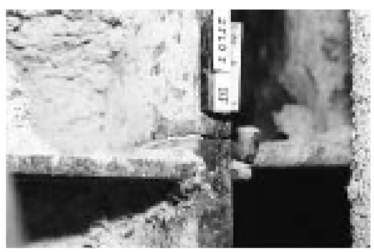
C. Selected Images of Connection Failure Modes

Figures C.1 to C.16 include images of selected failure modes as reported in the referenced publications.

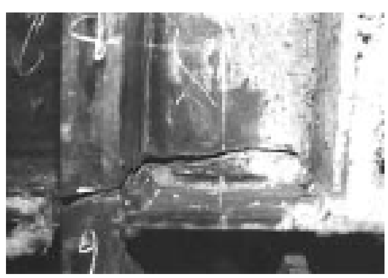


(a)

(b)

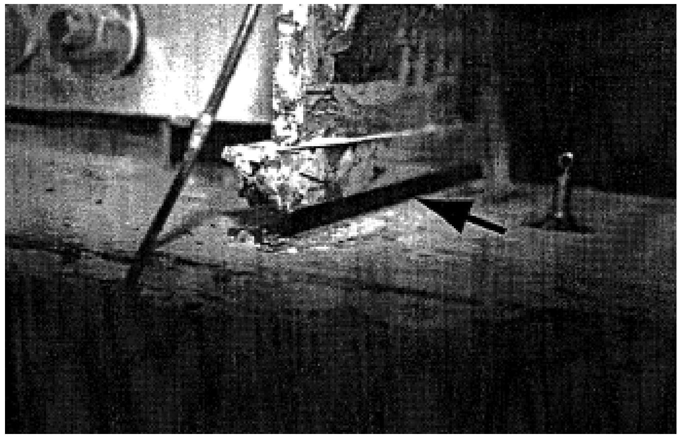


(c)

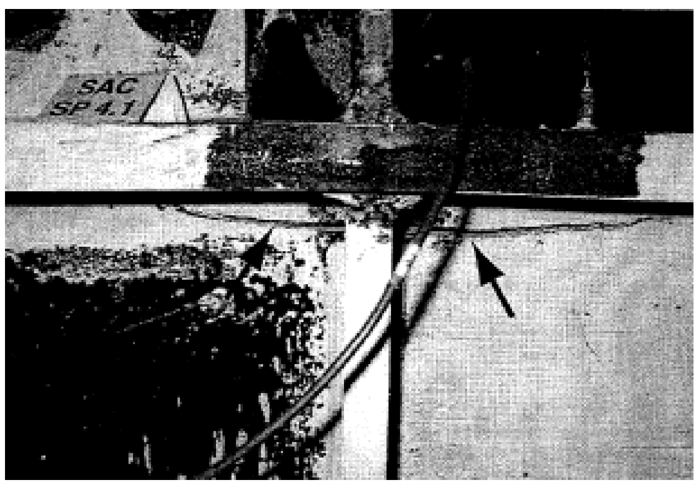


(d)

Figure C.1: Pre-Northridge Beam-Column failure modes a) Fracture at fused zone. b) Column flange “divot” fracture c) Fracture through Column Flange d) Fracture Progresses into Column Web



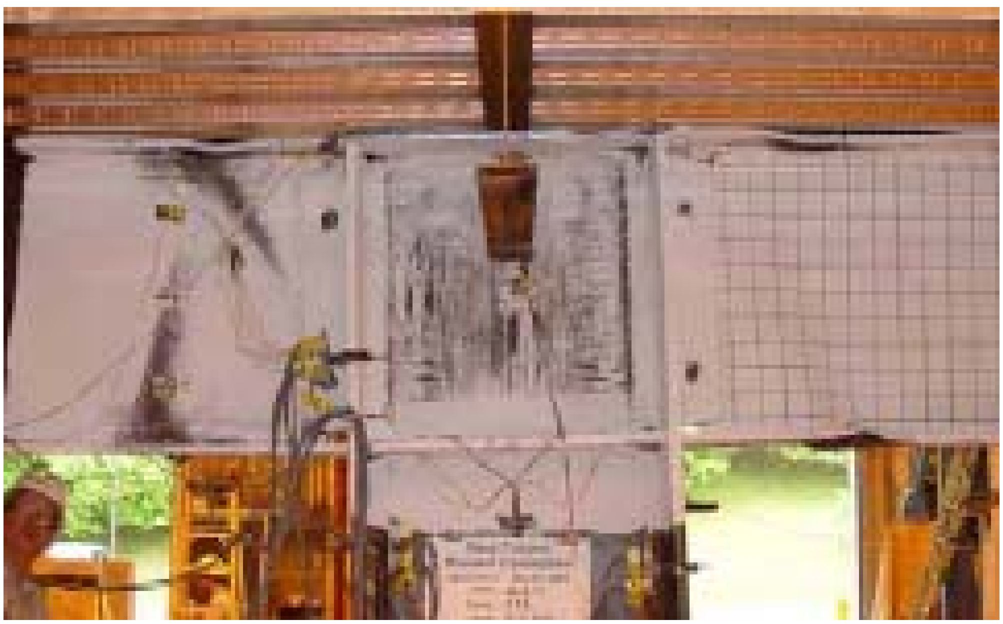
(a)



(b)

Figure C.2: Pre-Northridge Beam-Column failure modes a) Colum web K-zone failure b) Divot pull out fracture of the bottom flange weld along the backing bar.

(a)



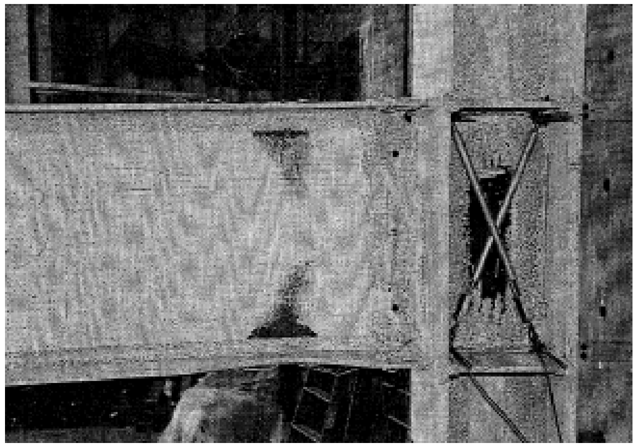
(b)

Figure C.3: RBS failure modes a) Bottom flange out-of-plane movement at 4% drift b) Yielding and buckling in connection region at 4% drift

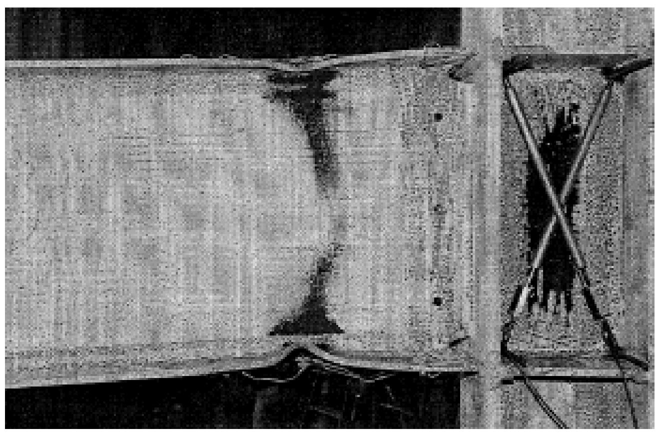
(a)

(b)

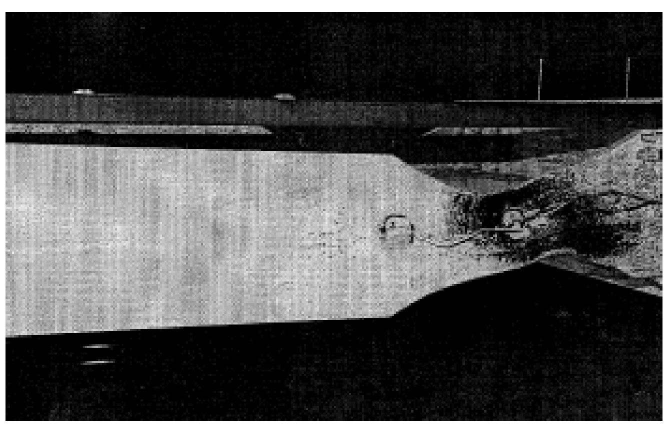
Figure C.4: 4 RBS failure modes a) Fracture in bottom flange at 5% drift. Low cycle fatigue fracture in hinge region b) Pronounced beam yielding and local buckling at RBS, fracture in the east beam bottom flange at center of RBS at 5% drift.



(a)

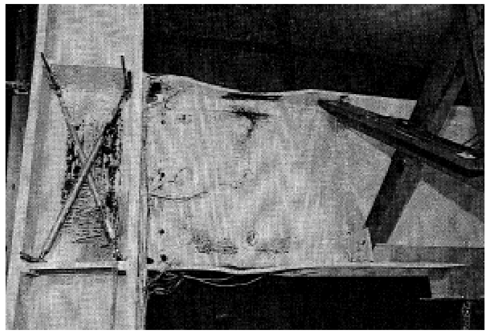


(b)

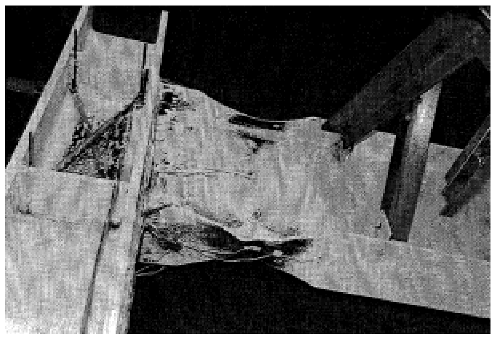


(c)

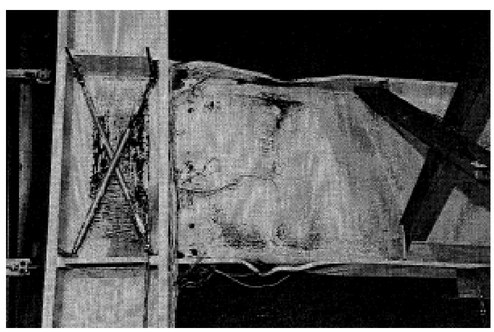
Figure C.5: RBS failure modes a) Yielding pattern at 3% drift b) Yielding and buckling pattern at 4% drift c) Lateral torsional buckling of bottom flange at 4% drift



(a)

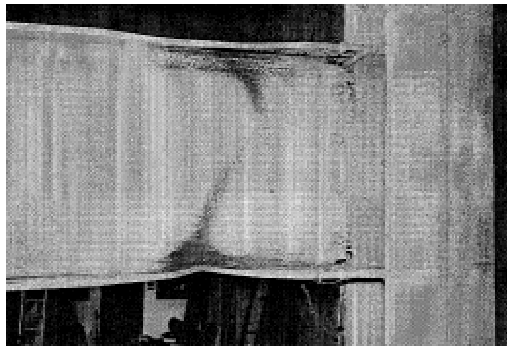


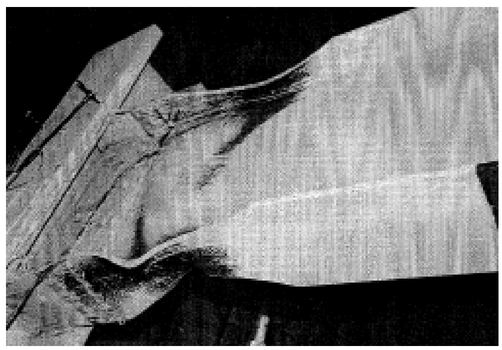
(b)



(c)

Figure C.6: RBS failure modes a) Local buckling at 3% drift b) Lateral torsional buckling at 4% drift c) Increased local buckling and lateral torsion buckling at 5% drift.


(a)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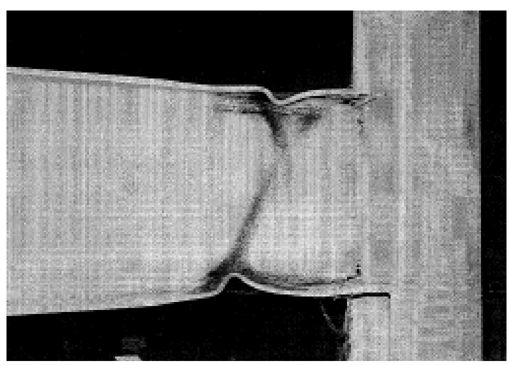
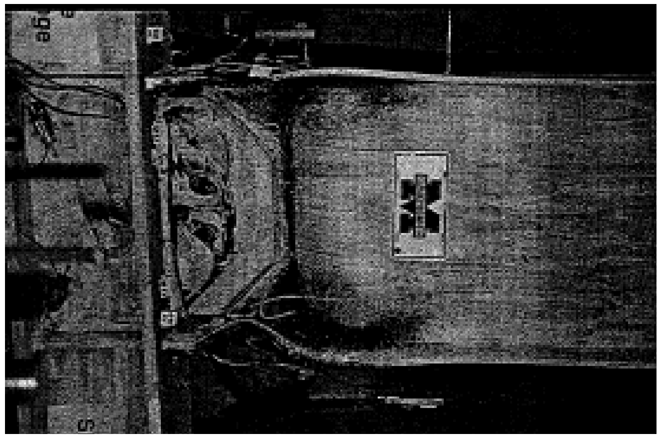
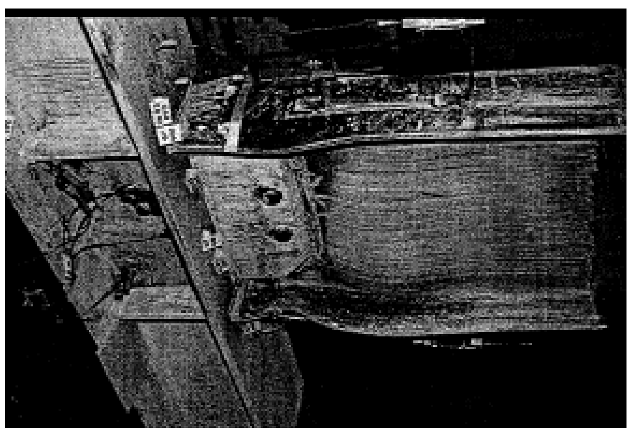

(c)

Figure C.7: RBS failure modes a) Flange local buckling at 3% drift b) Lateral torsional buckling and twisting of beam at 4% drift c) Increased buckling caused strength degradation and test halted at 5% drift

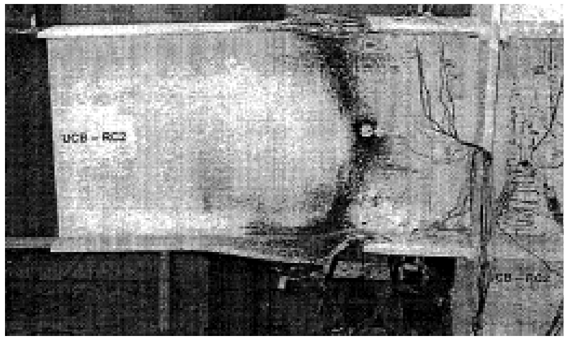


(a)

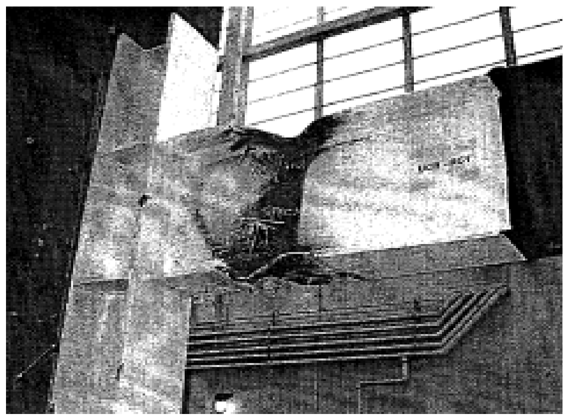


(b)

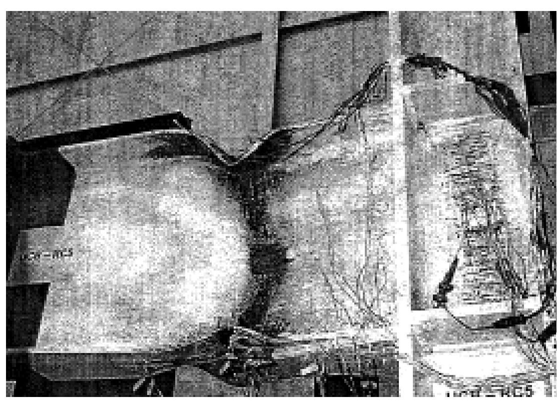
Figure C.8: Free Flange connection failure modes a) Plastic hinge in the beam at 4% drift b) Lateral torsional buckling and twisting of beam at 4% drift.



(a)



(b)



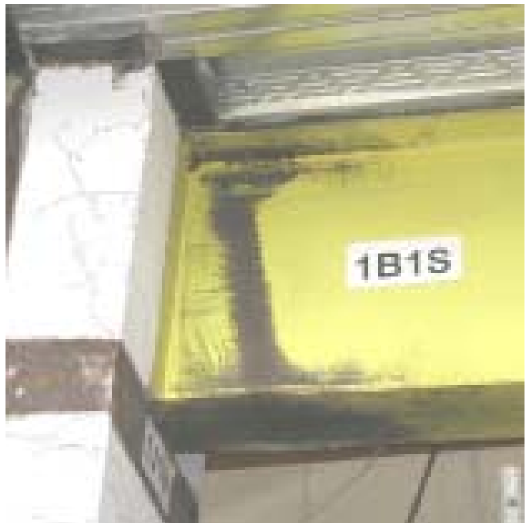
(c)

Figure C.9: Cover Plate connection failure modes a) Beam flange local buckling and web local buckling at 3% drift b) Beam flange local buckling and web local buckling at 4% drift c) Increased buckling and ductile tearing in beam flanges at buckled region at 5% drift

(a)

(b)

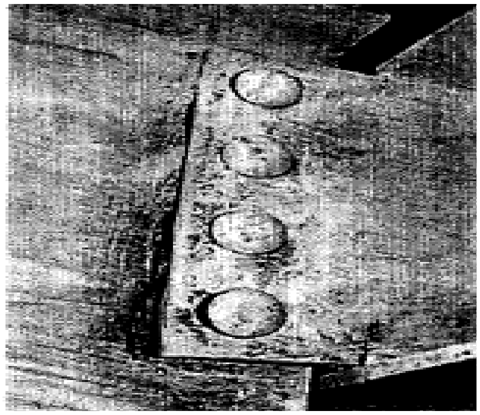
Figure C.10: System effects and effects of slab on top in an RCS frame. The top flange deforms much less when slab is present a) Yielding and local buckling at approximately 3% drift b) Yielding and local buckling at approximately 5.5% drift.



(a)

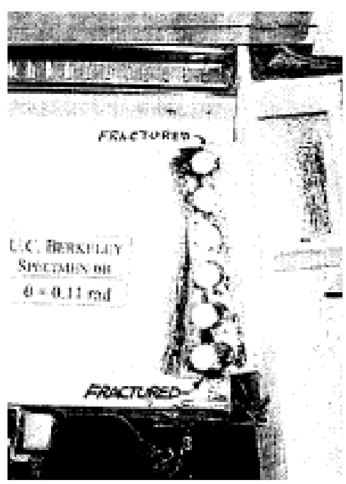
(b)

Figure C.11: System effects of RCS frame. a) Tension side. Yielding and local buckling. Buckles have nearly straightened out on the tension side. b) Compression Side. Yielding and local buckling



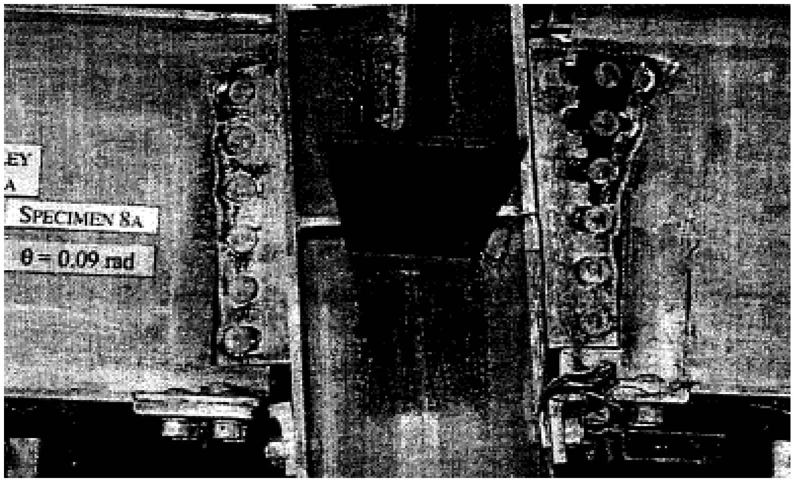
(a)

(b)



(c)

Figure C.12: Failure modes of Gravity Beam Shear connections a) Elongation of bolt holes b) Small crack at the bottom of shear tab c) Shear tab completely fractured



(a)

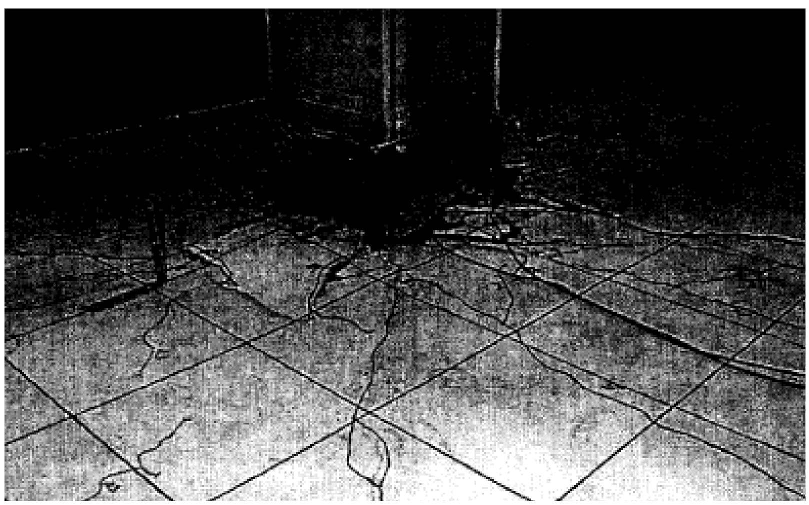
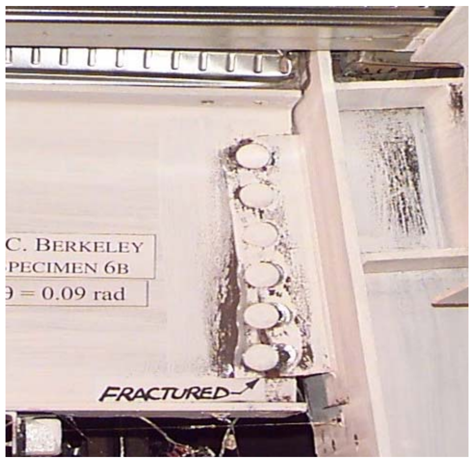
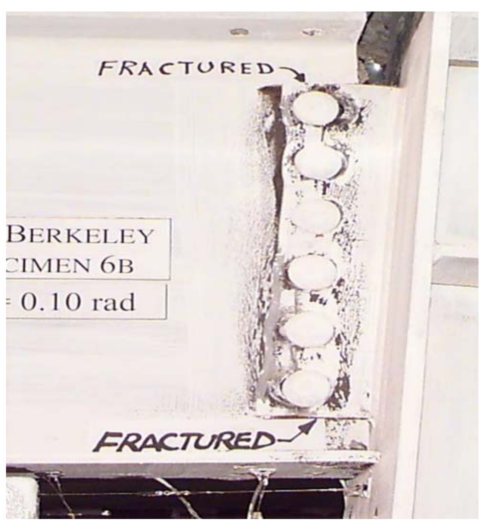

(b)

Figure C.13: Failure modes of Gravity Beam Shear connections a) Interior gravity beam shear connection at the end of test. Shear tab fracture at 9% drift. b) Cracking of concrete around column at 9% drift.

(a)



(b)

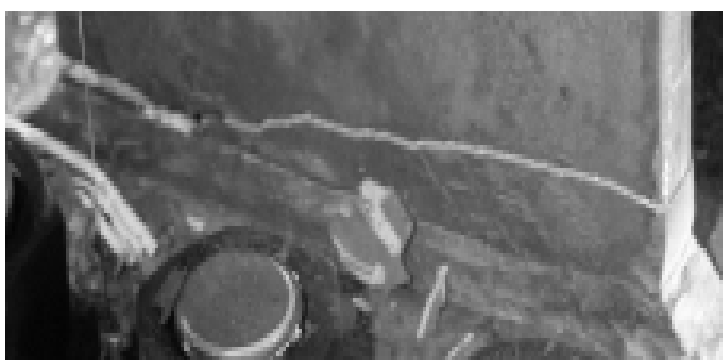


(c))

Figure C.14: Failure modes of Gravity Beam Shear connections a) Partial tearing at bottom of tab and elongation of bolt holes at 4% drift b) Fracture through most of shear tab at 9% drift c) Fracture through shear tab at 10% drift.

(a)

(b)



(c)

Figure C.15 Failure modes of Column Base Plate connections a) Ductile crack initiation b) Maximum extent of ductile crack c) Complete fracture of column flange

(a)

(b)

Figure C.16 Failure modes of Weak-Axis Column Base Plate connections a) Initiation of ductile fracture at column flange tip at 1.5% drift level b) Fracture has propagated into column flange at 3% drift level.