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EARTHQUAKE RESPONSE OF STRENGTHENED STEEL SPECIAL MOMENT RESISTING FRAMES

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ABSTRACT

The damage to steel beam column welded moment connections has left engineers scrambling for solutions to a problem that hitherto was not known to exist. A number of solutions to these damaged connections have been proposed, categorized under three global courses of action. a) Reweld only, b) Strengthen Connection and c) Introduce active or passive energy dissipating devices. This paper presents the response of steel special moment resisting frames for the first two repair options. Two different nonlinear dynamic analysis programs are used to study the effect of repairing and strengthening these weld failures, or predicting the expected response assuming the connection is not repaired. For the first program the effect of repair and strengthening are investigated. For the second computer program the effect of dropping the moment capacity of all beams to 50%, 20% and 5% of the original moment capacity after their connection reaches its first yield is studied.

The frames selected for the study correspond to a real building damaged during the Northridge earthquake. For comparative studies, they are subjected to an ensemble of artificial earthquake records that have an equal probability of occurring at the site. Conclusions are drawn based on the observations made as a result of the selective repair and/or strengthening schemes adopted for the analyses of steel special moment resisting frames herein.

KEYWORDS

ISTAR-ST, DRAIN-2DX, IDARC, Moment Capacity, Weld Failure, Strengthened Connection, Moment Capacity Reduction, Steel Special Moment Resisting Frames, Inelastic Analysis.

INTRODUCTION

Prior to the Northridge earthquake, the expected behavior of steel special moment resisting frames was that, for large earthquake ground motions, inelastic deformations were expected in the beams and sufficient curvature ductility capacity existed to accommodate this curvature demand. The Northridge earthquake proved this to be an invalid design assumption. The curvature ductility demand that weld failure can be expected to occur is only slightly greater than one for the most optimistic perspective. It is clear that in steel moment resisting frames with weld failures some action must be taken to account for the existence of the weld failure. There are at least three global courses of action that the structural engineer can take and they are: a) Reweld Only: In this case, all of the damaged welds, base material of beams and columns are repaired. This is referred to in this study as the original structure, even though it is probably better but clearly not the same. b) Strengthen Connection: In this case, the connections where the welds have failed have been strengthened with the region of plastic yielding moving away from the joint. c) High Tech: It is possible to use base isolation, dampers or other new "high tech" solutions to reduce the demands on the structure to ensure an acceptable response. With the current emphasis on the first two courses of action, they will be the basic subject of this paper.

Available computer programs such as the DRAIN family of computer programs have been used for many years to analyze the two-dimensional nonlinear response of steel frames. The program used in this study is DRAIN-2DX (Prakash et al. 1993). The moment curvature relationship of the cross section is bilinear. The initial elastic EI is used till the yield moment after which a 5 or 10% post yield stiffness is assumed. This computer program, hereafter referred to as DRAIN-2DX, does not account for the loss in moment capacity associated with the failure of the bottom weld in the moment connection.

Recognizing the inability of most structural engineers to evaluate the response of the building due to a reduction in the moment capacity of a moment connection two actions have typically taken place. One action has been to replace all weld failures with new welds, with probably better quality control and weld toughness, and hope for the best in the event of a future earthquake. The other action has been to provide new structural elements at the location of all weld failures, and to design these new elements to move the yielding in the beam away from the connection. This approach has the apparent desirable effect of strengthening all weld failure locations, and since this structure can be analyzed using DRAIN-2DX, for example, the expected future response can be calculated. Unfortunately, this latter analysis is seldom, if ever, done. This study evaluates the impact of such an action using DRAIN-2DX for a real building located in the San Fernando Valley, California. This study presents the results of the earthquake response of three repair options. The response assuming strengthened connections are compared with the response of the original structure.

It is recognized by many engineers that the failure of the bottom weld does not mean the moment connection is without residual moment capacity. In a review of the test data funded by SAC, NSF and the steel industry, two items are clear. First, the residual capacity is building dependent and can best be estimated for a specific building by the structural engineer for the building analysis/repair. This should always be based on experience and analysis. Second, the residual moment is non-zero and values in the range of 10 to 25% are not unreasonable. Therefore, the authors have started with the computer program IDARC (Kunnath et al. 1992) developed for reinforced concrete, and performed major revisions to enable the modeling of this reduction in moment capacity in steel frames due to weld failure. The moment curvature relationship modeled in this new computer program, hereafter referred to as ISTAR-ST is shown in Fig. 1. The results of the analysis of the structural systems modeled using DRAIN-2DX for the case where there is no moment capacity reduction during the earthquake motion, and the results using ISTAR-ST where there is a moment capacity reduction during the earthquake due to weld failures are presented.

DESCRIPTION OF STRUCTURE

The steel frames studied in this project and described herein are from a damaged building located in the Woodland Hills area of the San Fernando Valley. This building was studied in a previous SAC steel research program and a description of that work is available in published literature (Hart et al. 1995a, Hart et al. 1995b). In those previous SAC studies a SAP90 linear elastic analysis and DRAIN-2DX inelastic analysis were performed on the building for different ground motions. Sensitivity studies were performed to see the impact of the different modeling assumptions on the response of the undamaged building. Two of the steel frames of the building that experienced weld damage are considered in this study. Each frame consists of six

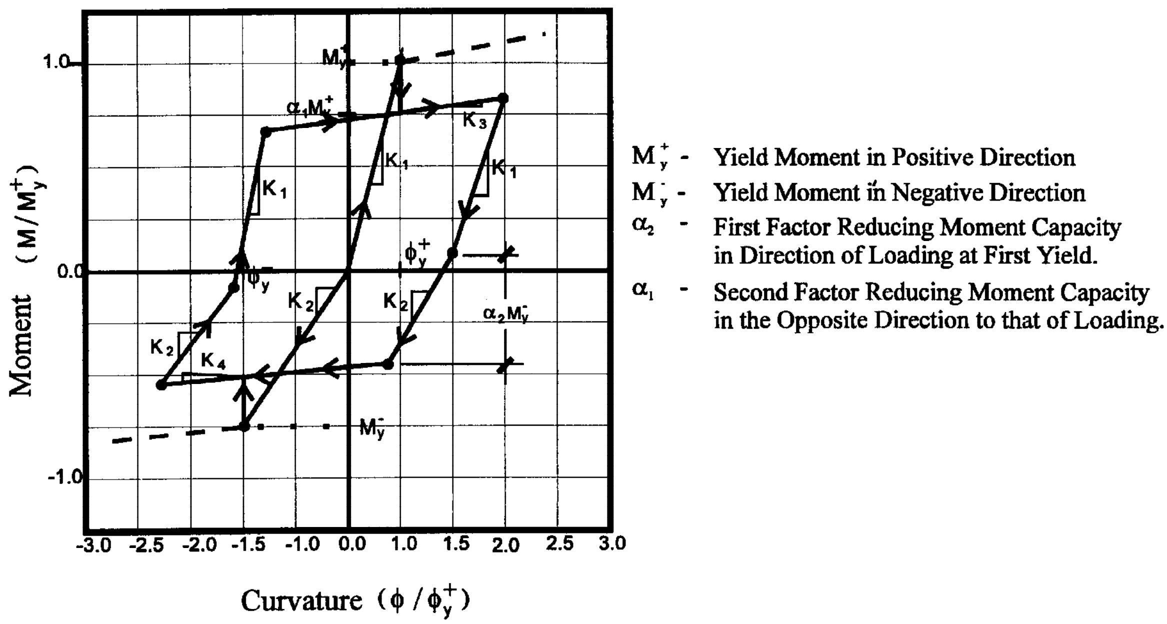


Fig. 1. Moment-Curvatute Loop for Fractured Connection

stories and three bays, and have identical geometry and member sizes. The first floor is supported laterally to simulate shear wall below grade. The frame elevation showing the location of the weld damage in the respective frames is shown in Fig. 2.

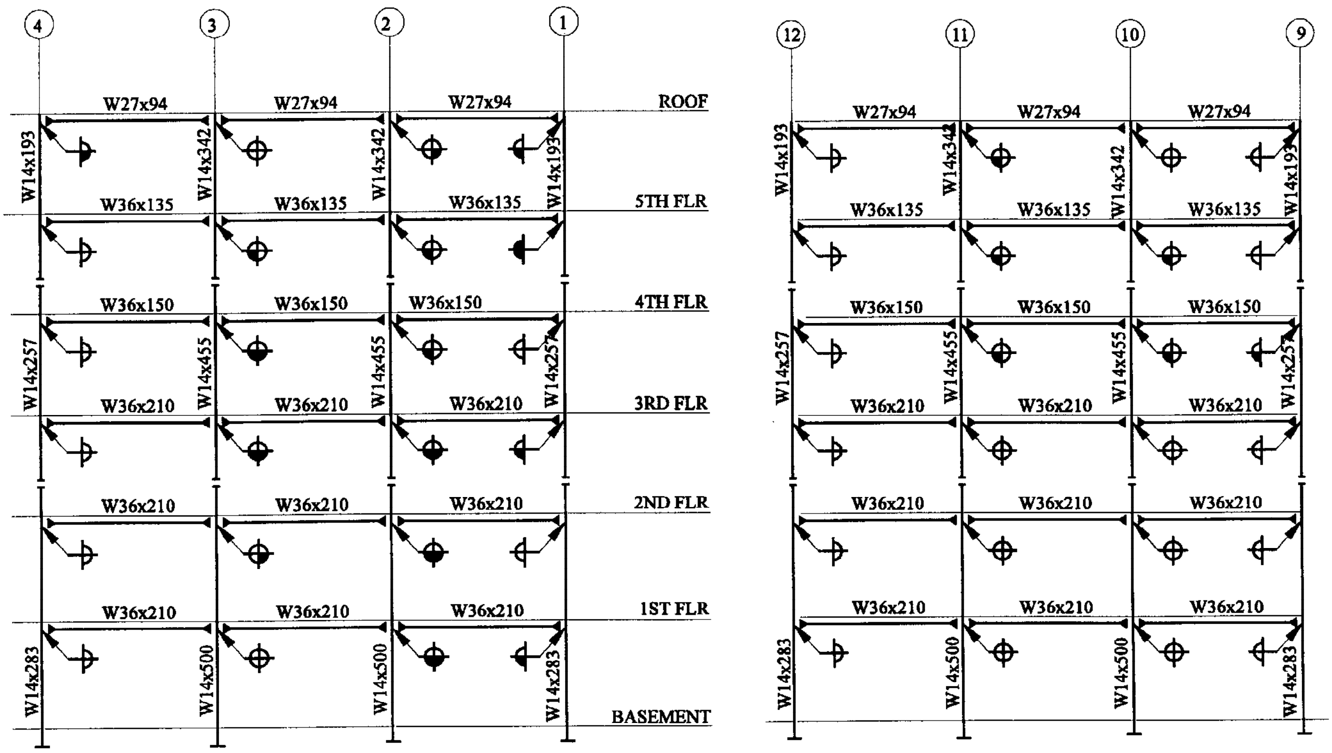


Fig. 2. Elevation Showing Location of Weld Damage.

Model Description

The models for this frame are made using one quarter of the associated building mass. The modeling assumptions are rigid floor diaphragms, 5% damping in the 1st Mode and at 30 Hz, stiff support at the ground level, joints with rigid end zones, and joint shear deformations achieved through shear panel zones, and 2% strain hardening is assumed for the beams and columns. Additional moment due to P-Delta effects were also included. A more detailed description of the model description and modeling assumptions can be found in Hart et al. 1995a.

DRAIN-2DX ANALYSIS OF ORIGINAL AND STRENGTHENED STRUCTURE

The frame spanning lines 9 through 12 (Fig. 2) experienced damage to *6 connections* during the Northridge earthquake. The original design of this frame, modified with each damaged connection repaired using haunches at the top and bottom of the beam, is defined as *Case 1.* Therefore, at each of these strengthened connection locations the yielding will move away from the joint. The frame spanning lines 1 through 4 (Fig. 2), experienced severe damage to *20 connections.* The model of this frame with each damaged connection repaired with haunches and thus is similar to Case 1 but with different strengthened connection locations is defined as *Case 2.* The model defined to be *Case 3* consists of the original frame but with all connections strengthened. The model called the *Original Model* is the model of the frame as it was intended during design. It can be visualized as the case with only rewelding and no connection repairs with new haunches or plates.

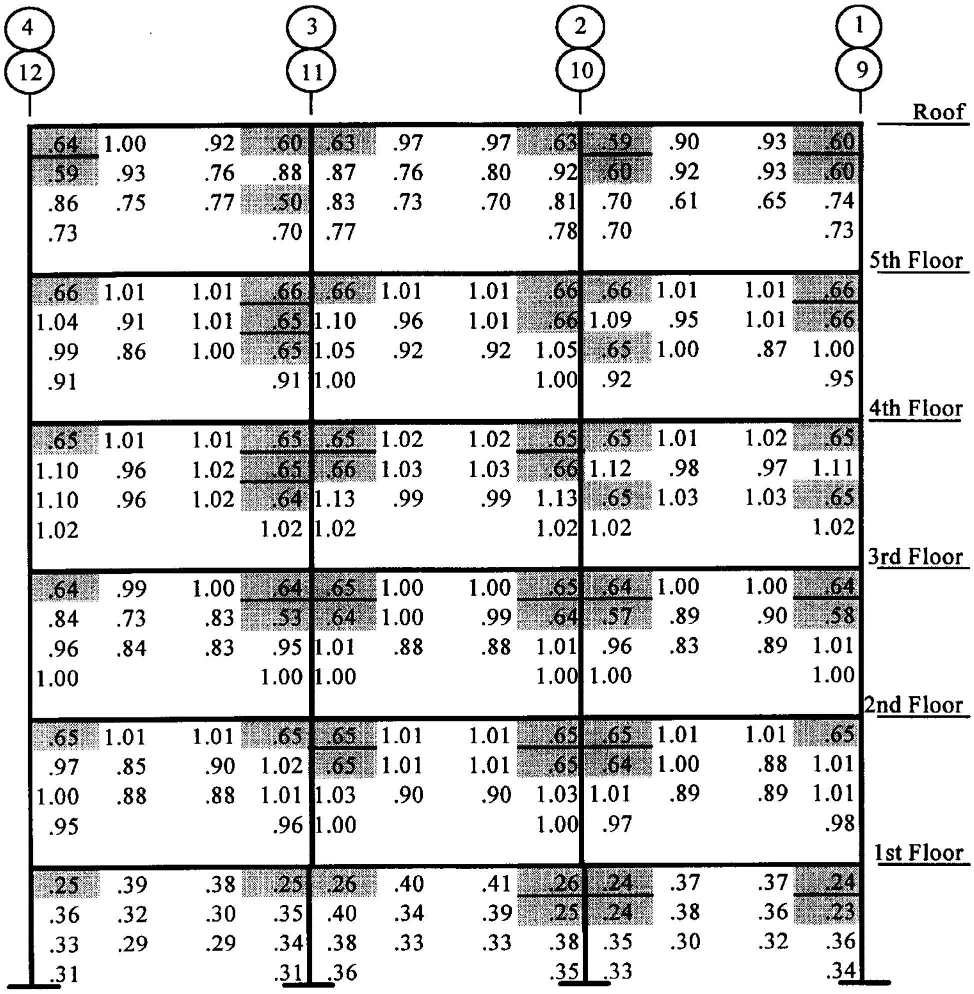
DRAIN-2DX analyses were performed for the above mentioned cases using 9 synthetic time histories for site 1 of the SAC Joint venture (FEMA Report No. SAC-95-02). Each of the 9 synthetic time histories represented a ground motion that was equally probable to have occurred at the building site. Table 1 provides a basic summary of the earthquake records. The definition of effective peak acceleration is the maximum value of the 5% damped spectral acceleration divided by 2.5. The NS motion was used from each of the 9 SAC pairs of time histories.

Table 1. Earthquake Characteristics and Responses for Nine Synthetic Earthqukes for the Northridge Site.

		Earthquake Characteristics (g's)									
		N	S	E	W	NE	NW	SE	SW	C	
PGA		0.42	0.52	0.55	0.54	0.51	0.43	0.48	0.61	0.63	
EPA		0.70	0.64	0.84	0.71	0.85	0.52	0.78	0.88	0.70	
	Period	Maximum Base Shears in Frame (kips)									MAX/ Orig.
Original	0.84	1791	1909	1668	1801	1618	1245	1419	1554	1477	1.00
Case1	0.82	1845	1714	1826	1688	957	859	733	802	1652	0.97
Case2	0.77	2047	1934	1999	1756	1024	1039	870	930	1819	1.07
Case3	0.74	2113	1373	2142	1568	1104	1040	920	862	1510	1.12
		Maximum Roof Displacements (in)									
Original		8.32	7.12	7.92	6.24	10.17	5.43	6.66	6.41	5.87	
Case1		10.03	8.01	9.20	6.79	10.25	5.73	6.95	6.85	6.08	
Case2		11.12	7.06	9.58	6.11	9.22	7.40	7.24	5.99	6.94	
Case3		11.19	5.56	9.09	6.16	8.38	7.81	7.03	5.77	7.15	
		Maximum Roof Displacements / Original Model Roof Displacements									
Case1		1.21	1.12	1.16	1.09	1.01	1.06	1.04	1.07	1.04	
Case2		1.34	0.99	1.21	0.98	0.91	1.36	1.09	0.93	1.18	
Case3		1.34	0.78	1.15	0.99	0.82	1.44	1.06	0.90	1.22	

The fundamental period of vibration for the various cases, also shown in Table 1, decreases as more connections are strengthened, which is expected. For all connections strengthened Case 3, the period reduced from 0.84 sec to 0.74 sec (a 12% reduction). The sensitivity of the maximum base shear force to the earthquake record for each case is shown in Table 1. The greatest base shear force in the original model was for the South record and was 1909 kips. For this same record the Case 2 model increases the base shear and the Case 3 repair significantly decreases the base shear. From this observation it is clear that one cannot conclude that strengthening of the connections will reduce the base shear of the building.

The maximum roof and floor displacements for each of the cases and each of the earthquake ground motions are shown in Table 1. The roof displacement for the three cases are divided by the original model roof drift and these ratios are also shown in Table 1. A value greater than 1 in this column corresponds to an increase in roof response resulting from the connection strengthening. Note that in many situations the roof displacement increases substantially with the greatest increase being 44% (see Northwest earthquake, Case 3). If Case 3 is eliminated from consideration, then the greatest increase is 34% (see Northwest earthquake, Case 2).

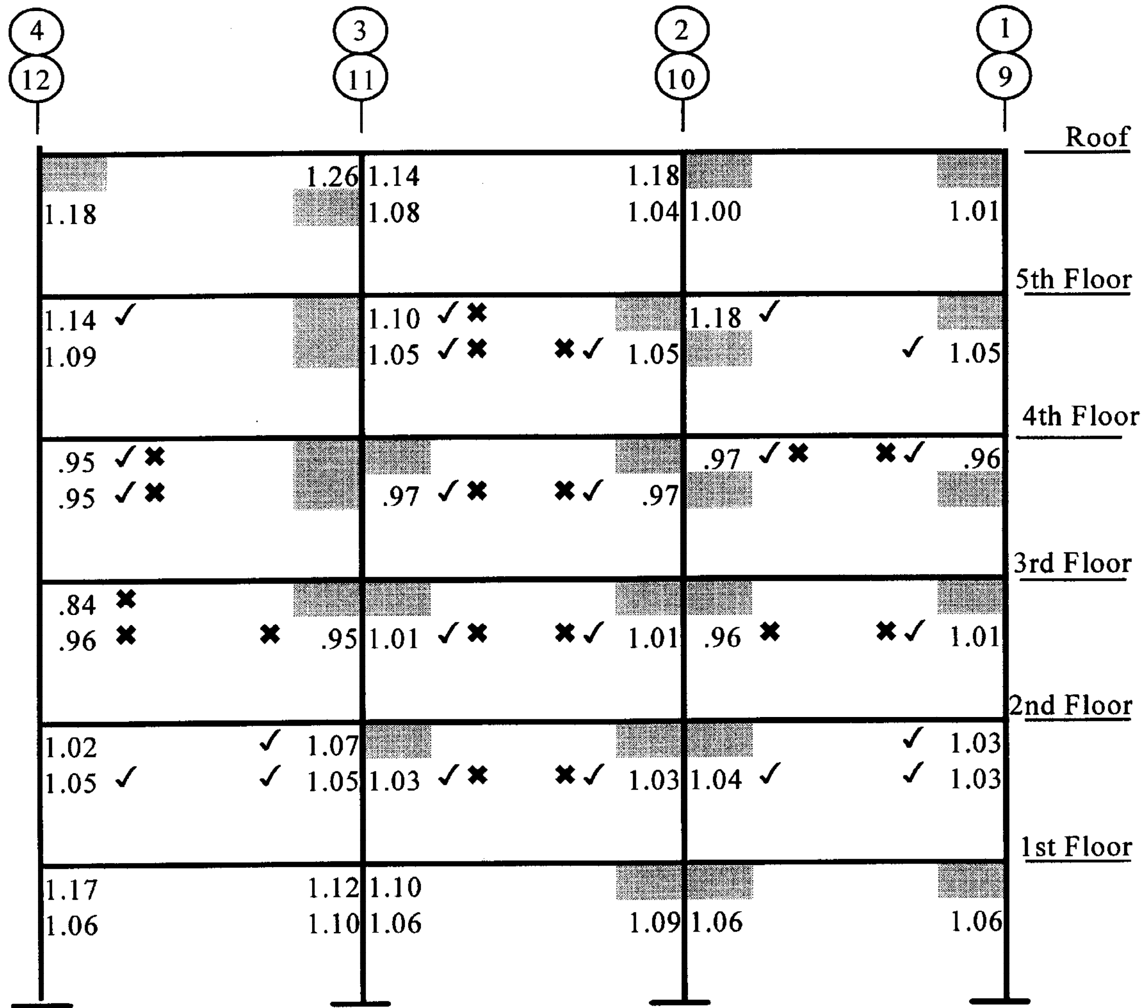


First row numbers represent the case with all joints repaired. (Case 3).
Second row represents the case with only damaged joints in frame 4,3,2,1 repaired. (Case 2).
Third row represents the case with only damaged joints in frame 12,11,10,9 repaired. (Case 1).
Shaded numbers represent joints that have been repaired.
Numbers next to columns are values at column face.
Other numbers are values at cross section 18" away from column face.

Fig. 3. Moment Demand Capacity Ratios For The North-East Earthquake.

Figure 3 shows the beam moment demand/capacity ratios for the North-East earthquake. The ratios are listed next to their corresponding joint. There are two numbers at each column joint. The number closer to the column joint represents the moment ratio at the beam-column connection. The number furthest away from the column is the moment ratio at the end of the beam, i.e. before the beam cross section changes due to the haunch repair. In the original model, there are no haunch repairs so there is only one number listed: the moment ratio at the column face. Shaded numbers mean the joint was repaired with a haunch. A key at the bottom of each figure summarizes the above paragraph.

The results shown in Fig. 4 clearly show that the strengthening of connections, either in Case 1, 2, or 3, can result in a significant increase in the demand on and response of the structure. In addition to this negative effect, the strengthening of the connections induces moment demands on connections that did not fail that are greater than the yield moment.



First row numbers represent the frame long grid lines 1 through 4.
Second row numbers represent the frame along grid line 9 through 12.
A ✓ represents a yielded connection in the repaired frame.
A ✖ represents a yielded connection in the original frame.
Shaded numbers represent joints that have been repaired.
Values are for cross sections at column face.

Fig. 4. Ratio of Moment Demand in Repaired Frame to Moment Demand in Unrepaired Frame (NE Record).

ISTAR-ST RESPONSE OF ORIGINAL STRUCTURE

The analysis results presented in this section were done using the Hart Consultant Group in-house computer program called ISTAR-ST. The hysteretic model (Fig. 1) describes the input parameters for the new hysteretic model. Specifically, it enables the reduction of the positive and/or negative moment capacity of any connection after the first time the moment reaches its yield moment. The modeling assumptions used in ISTAR-ST are similar to the assumptions used in DRAIN-2DX. However, the two programs being different a few modification were made to these assumptions. No effect of panel zones were considered in the analysis, and the effects of P-Delta were ignored. The model of the original frame, assuming no moment reduction post yield, of the building using the above modeling assumptions was analyzed using the nine synthetic time histories described earlier. The results from the original frame without capacity reductions were used as the base for normalization of the results to show the relative effect of the different capacity reductions.

It is important to evaluate the impact of the inclusion of this moment reduction for a larger earthquake because the design earthquake exceeds the earthquake motion experienced at the site during the Northridge earthquake. Therefore to provide insight into this, each of the nine synthetic time histories were scaled by a factor of 1.5, for analyses performed on the original and reduction to 50%, 20% and 5% of the initial capacity after failure. The results of the base shears, roof drifts and roof displacements are given in Table 2. The table shows the ratio of the response normalized by the response of the original model.

Table 2: Response Ratios For Roof Displacement, Roof Interstory Drfits and Base Shears At 1.5 A(T)

	Ratios of Roof Disp to Original			Ratios of Roof Drifts to Original			Ratios of Base Shears to Original		
EQ	Drop to 50%	Drop to 20%	Drop to 5%	Drop to 50%	Drop to 20%	Drop to 5%	Drop to 50%	Drop to 20%	Drop to 5%
N	1.05	0.96	1.59	0.66	0.55	1.46	0.81	0.81	0.81
S	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
E	0.85	0.83	1.09	0.76	0.76	0.76	0.87	0.87	0.87
W	1.03	1.32	2.01	0.82	0.75	1.57	0.88	0.88	0.88
NE	1.38	1.30	1.01	2.99	2.84	0.84	0.80	0.78	0.76
NW	0.70	0.88	1.03	0.72	0.80	0.76	0.88	0.88	1.03
SE	0.99	0.94	1.77	0.97	1.70	4.54	0.84	0.84	0.84
SW	0.96	1.01	1.49	0.74	1.33	3.31	0.87	0.92	0.79
C	1.05	1.00	2.98	1.64	1.01	6.37	0.93	0.98	0.93
Mean	1.00	1.03	1.55	1.14	1.19	2.29	0.88	0.88	0.88
COV	0.18	0.17	0.42	0.66	0.59	0.88	0.07	0.08	0.11

The results show a considerable variation with the earthquake record used. The roof drift ratio for a capacity reduction to 5% is 6.37, for the C record, and a corresponding displacement ratio of 2.98. Clearly the damage would be extensive, probable collapse of the building is anticipated. However a capacity reduction to 50% or 20% for the same record does not prove to be as damaging. The degree of uncertainty in the response to varying capacity reductions to different input ground motions is significant. Clearly more research needs to be done in this area for different repair schemes for various earthquakes and capacity reductions.

CONCLUSIONS

The entire subject of steel weld and connection failure demands extensive experiments and research prior to reaching the final conclusions. However, based on the research described in this paper, it is important to put forth the following conclusions for review and comment:

1. It is not acceptable to strengthen a connection with weld or material failure without a detailed nonlinear dynamic analysis to evaluate the impact of the strength increase on the load path and increased demand on other connections.

2. Strengthening connections with weld or material failure can be expected to increase the load and ductility demand on connections that did not suffer damage.

3. It is very important to quantify through testing and analysis the expected moment/rotational relationship for a building's connections. The determination of the expected moment drop is a critical parameter for quantifying the life safety of the frame.

4. The results for analyses where the drop in moment capacity is to 20% of the original indicates that drift control and life safety may be able to be provided with selected strengthening of connections or minimal expense for high tech options.

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