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BEHAVIOR OF LARGE STEEL BEAM-COLUMN CONNECTIONS

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ABSTRACT

A surprisingly large number of beam-to-column connections in steel frame buildings suffered brittle failure during the 1994 Northridge earthquake. A large scale testing program was initiated by the SAC Joint Venture to evaluate the seismic worthiness and propose repairs for existing connections, and to establish design guidelines for new steel construction. Three full-scale beam-column connections were tested at the University of California at Berkeley within the SAC Joint Venture. The specimens were nominally identical models of typical exterior beam-column connections found in modern steel moment resisting frame buildings. All specimens were detailed according to the industry standards and procedures in use before the Northridge earthquake, and manufactured simulating field conditions.

Testing consisted of subjecting the specimens to cyclic quasi-static loading. All three connections fractured suddenly. Serious fractures of the column entire flange, sometimes observed during Northridge, were reproduced in two specimens. The third specimen failed due to the weld fracture on the beam bottom flange. The three specimens were repaired according to current repair procedures, and tested again using an identical loading protocol. All three repaired connections showed a somewhat improved behavior compared with the original specimens. Their mode of failure, however, was also a non-ductile, sudden fracture.

KEYWORDS

Steel connections, steel moment frames, seismic behavior, welded connections, structural repair.

INTRODUCTION

Before the Northridge earthquake of January 17, 1994, steel moment-resisting frames were considered to be the reliable systems for seismic resistant construction. After observing extensive fractures in beam-to-column connections during this earthquake, their relaibility is now questioned. In responce to this concern, the SAC Joint Venture (Structural Engineers Association of California, Applied Technology Council, and the California Universities for Research in Earthquake Engineering) implemented a comprehensive program to develop improved professional practices and recommend standards for the repair, design and retrofit of the moment resisting frame buildings so that they would provide reliable, cost-effective seismic performance in future earthquakes.

As part of the experimental research effort of the SAC Steel Program, a set of large-scale external beam-to-column connections, constructed following the pre-Northridge practice, was tested at the Department of Civil and Environmental Engineering of the University of California at Berkeley. The main goals of these tests were to study the seismic response of the standard connections, and to verify the adequacy of current repair methods. This paper briefly presents the main findings of this experimental program. Detailed results are given in the final SAC report (Popov et al., 1996).

SPECIMENS, TEST SETUP & INSTRUMENTATION

Three nominally identical full scale models of a typical beam-to-column connection, labeled as PN1, PN2, and PN3 (for Pre-Northridge), were manufactured by a local fabricating company. The details of the connection, the weld specifications, and the erection procedures closely resembled the industry standards in use before the Northridge earthquake. Figure 1 shows specimen geometry and connection details. All columns were fabricated with A572-Gr. 50 steel. Although it was specified that all beams should be fabricated with A36 steel, the beams of PN1 and PN2 were built from A572-Gr. 50 steel; only PN3 had a beam made of A36 steel. This construction error caused the geometrically identical specimens to have different strength characteristics, since the beams of PN1 and PN2 were considerably stronger than the beam of PN3.

Table 1. Material Properties

		Yield Strength F_y (ksi)			Ultimate Strength F_u (ksi)		
Specimen	Shape & Steel	AISC	Mill	Coupon	AISC	Mill	Coupon
All Columns	W14x257, A572-Gr 50	50.0	53.5	48.3	65.0	72.5	67.8
PN1 & PN2 Beam	W36x150, A572-Gr 50	50.0	62.6	60.6	65.0	74.7	68.6
PN3 Beam	W36x150, A36	36.0	56.8	40.3	50.0	40.3	57.4

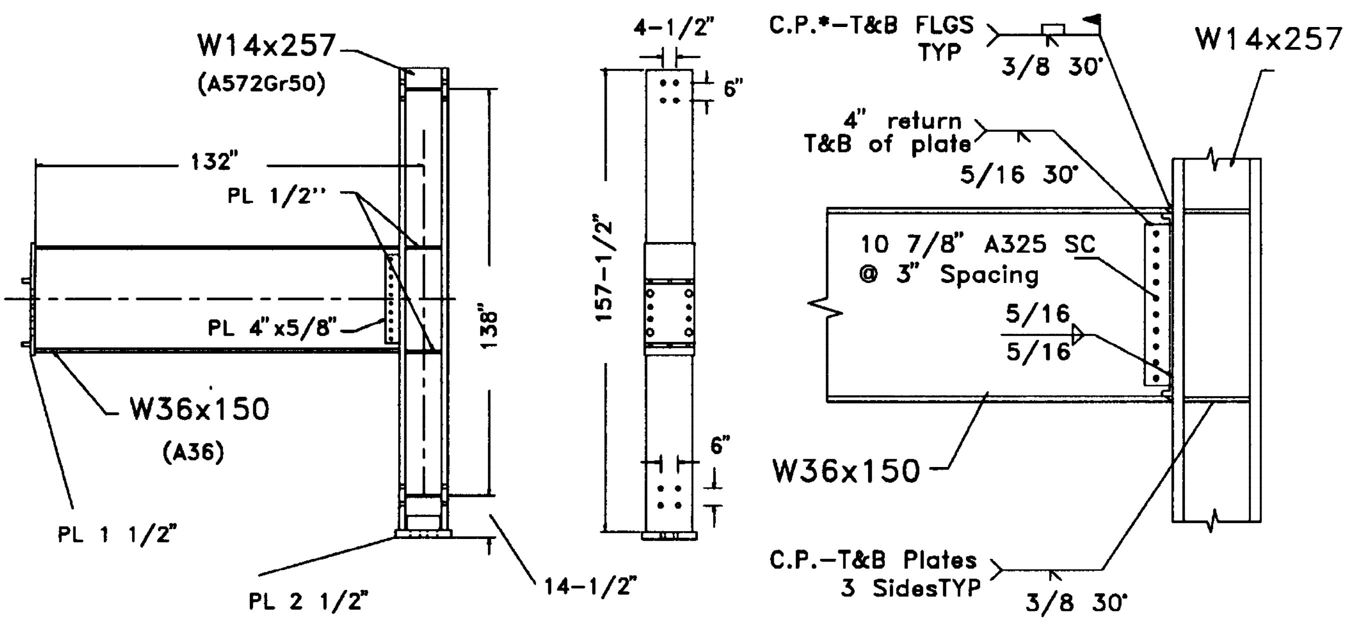


Fig. 1. Specimen dimensions and connection details

The tests were conducted in the Structural Engineering Laboratory of the University of California at Berkeley. The test setup, shown in Fig. 2, was designed to test the specimens in a horizontal position. The support system consisted of three reinforced concrete reaction blocks fixed to the test floor with high strength rods. Both ends of the columns were tightened against the reaction blocks by prestressed steel rods. Load was applied to the cantilever beam end by a servo-hydraulic actuator through a clevis bolted to the beam end plate. To prevent out of plane motion of the beam, a horizontal bracing system was provided near the beam end.

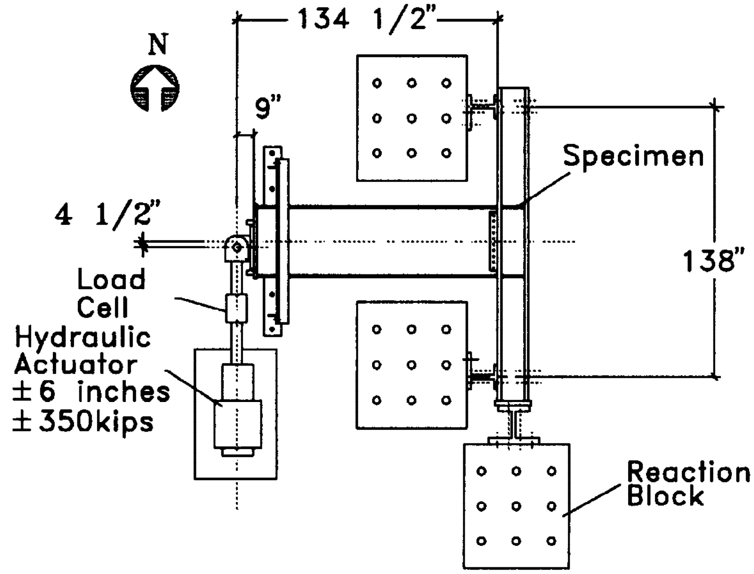
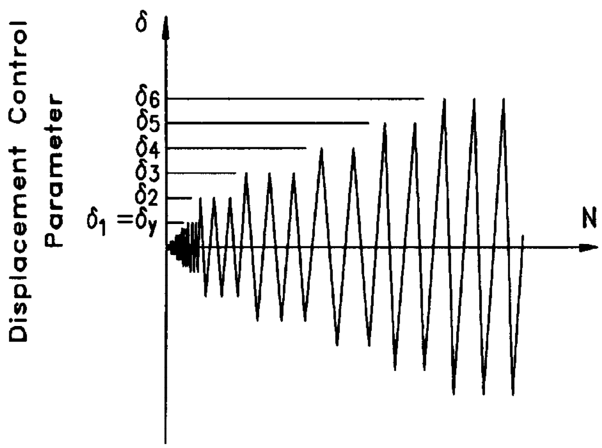


Fig. 2. Test setup

Linear potentiometers were used to measure global behavior: beam end deflection, joint rotation, and panel zone shear deformation. Strain gages and rosettes were glued at critical locations to measure the local response. The applied load was monitored with a load cell attached to the actuator. All instruments were connected to a computer-based data acquisition and control system used to send the displacement command signal to the servohydraulic actuator and to acquire the instrumental readings at programmed times.

All testing was performed under displacement control, following a protocol based on ATC-24 (ATC, 1994). The actuator displacement loading history consisted of a series of cyclic displacements with increasing amplitude, as shown in Fig. 3. The reference displacement of 1 inch corresponds to the estimated beam end yield deflection. Specimens were painted with white-wash prior to testing, to be able to observe yielding in the areas where the mill scale flakes off.



Load Step	Peak Displacement [inch]	Number of Cycles
1	0.10	2
2	0.25	3
3	0.50	3
4	0.75	3
5	1.00	3
6	2.00	3
7	3.00	3
8	4.00	2
9	5.00	2
10	6.00	n

Fig. 3. Loading History

SPECIMEN 1 TEST

PN1 behaved elastically up to and during the 1-inch cycles. During the 2-inch cycles the white-wash spalled due to yielding in both beam flanges, in the column flanges and in the panel zone. PN1 sustained the first 3-inch load cycle, with a maximum force of 223 kips, and a peak displacement of 2.57 inches. This discrepancy with the target displacement of 3 inches, observed consistently throughout the testing of all specimens, was due to slippage of the clevis connection.. The specimen failed suddenly (and very loudly) during the first leg of the second 3-inch load cycle. The failure load and displacement were 206 kips and 1.62 inches, respectively. One of the column flanges fractured completely, and the crack propagated into the column web, as illustrated in Fig. 4a. This mode of failure was unexpected; analysis predicted failure in the A36 steel beam. Some investigation revealed that the beam was built from Grade 50 steel instead of A36. This error was, in a way, fortunate, because this was the first test where this type of column failure, observed during the Northridge earthquake, was reproduced in the laboratory.

The repair procedure consisted of removing the cracked portions of the column flange and web, and welding a new piece made of a Gr. 50, W14 x 257 section. A beam flange splice plate was used to reconnect the beam bottom flange to the repaired column flange and new continuity plates were provided. A beam web doubler plate was welded to the column flange and beam web on the side opposite to the shear tab. Fig. 4b shows a sketch of the repair scheme.

The specimen, now labeled RN1, was retested. Nonlinear response started with the 2-inch cycles. During the 3-inch sequence, significant spalling of the white-wash was observed at the top beam flange and bottom splice plate, at the back column flange, and in the panel zone. RN1 sustained the positive excursion of the first 4-inch cycle with a peak load of 260 kips and 3.54 inch displacement. The specimen failed during the negative excursion of that cycle with a complete fracture of the top flange weld, as shown in Fig. 4c.

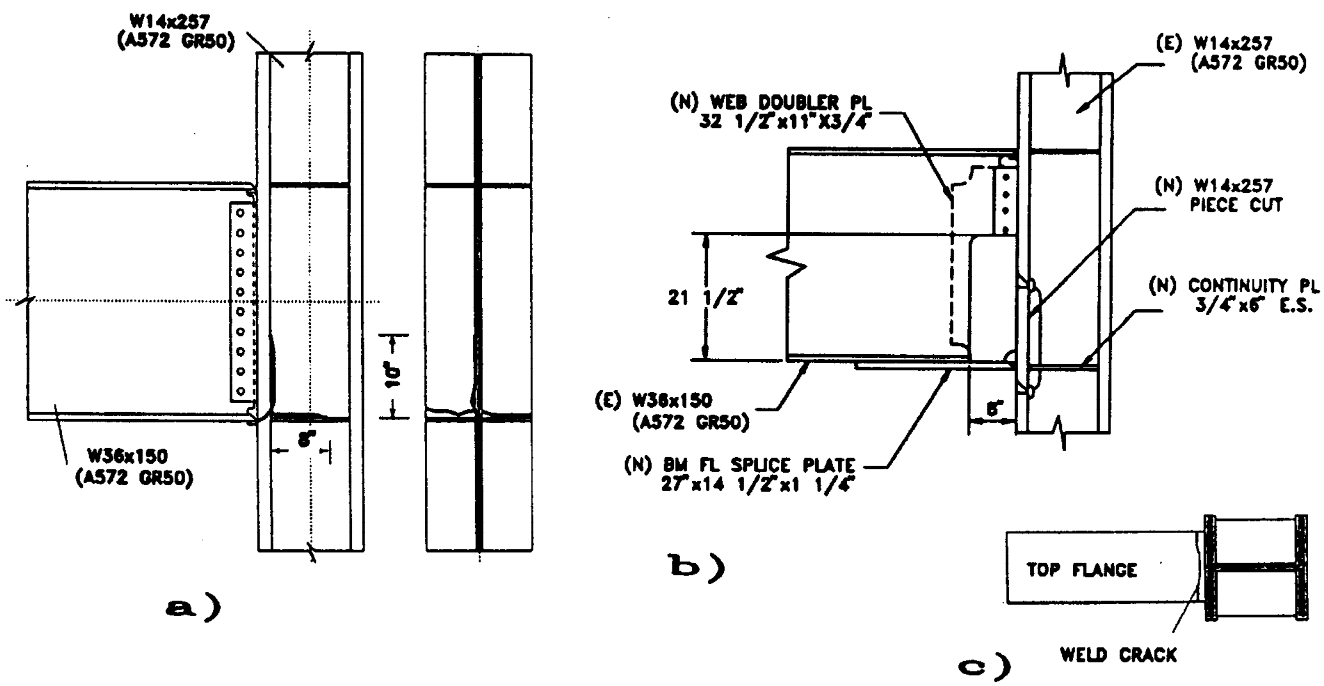


Fig 4. Specimen 1 -- Damage, repair, and damage of repaired connection

SPECIMEN 2 TEST

As with PN1, nonlinear response started during the 2-inch cycles. Spalling of the white-wash indicated significant yielding in the panel zone. PN2 failed suddenly during the first leg of the second 2-inch load cycle at 195 kips load and 1.57 inch displacement. The cracking pattern is sketched in Fig. 5a. Cracking started in the beam bottom flange weld, continued diagonally through the column flange, and entered the panel zone, where it extended upwards along the column flange. This column failure was also due to having the beam mistakenly constructed with Grade 50 steel instead of the specified A36 steel.

Since PN1 and PN2 had similar column failure modes, it was decided to incorporate an improved retrofit scheme by adding a haunch. Damaged portions of the beam and column were removed and replaced with new Grade 50 material. A new A36 plate was used to reconnect the beam bottom flange, and a beam web doubler plate was welded to the column and beam. A tapered haunch made of Grade 50 plate was placed underneath the beam bottom flange. Stiffeners were added to the beam and the column. Figure 5b summarizes the repair procedure. The haunch and stiffeners were initially fabricated and installed with wrong dimensions and it was necessary to remove them and fabricate and install new material.

The repaired and retrofitted specimen, RN2, was able to sustain load up to the 4-inch cycles. First spalling of the white-wash was noted on the bottom beam flange, near the haunch, at the beginning of the 2-inch cycles. Yielding occurred later in the beam top flange, in the column back flange, and in the panel zone. The specimen failed during the first excursion of the first 5-inch cycle. A crack started at the edge of the beam bottom flange between the haunch end and beam stiffener and propagated along the haunch edge up to the beam web, where it branched in two, as shown in Fig. 5c.

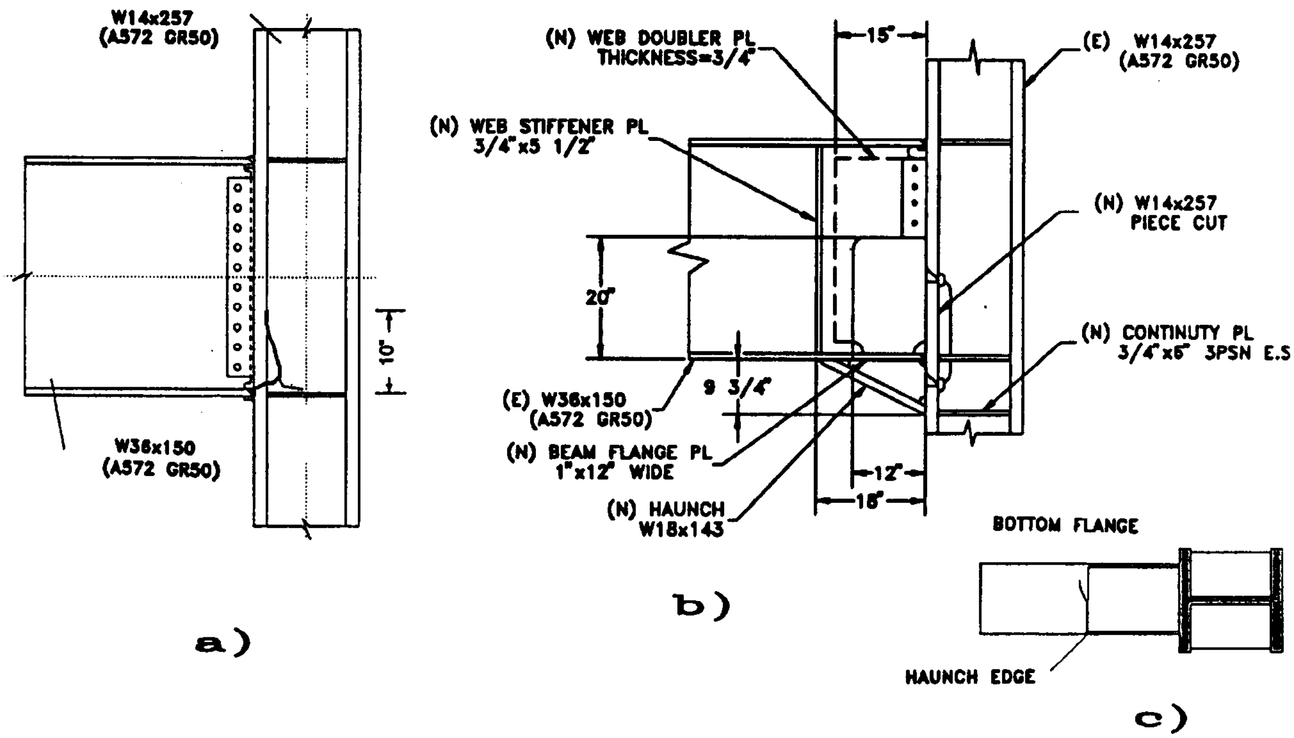


Fig. 5. Specimen 2 -- Damage, repair, and damage of repaired connection

SPECIMEN 3 TEST

PN3 held up to the first 3-inch cycle. Yielding was significant on the beam flanges. The maximum force was equal to 190 kips and the beam end displacement reached during this cycle was 2.67 inches. The specimen failed during the first leg of the second 3-inch load cycle. The bottom beam flange cracked through the weld. Later, some bolts sheared off and the shear tab broke. Loading was continued to finish the cycle. During the negative excursion of the cycle, the beam top flange cracked through approximately 3/4 of its width. This type of beam fracture was expected for all three PN specimens, and occurred only with PN3 which beam was constructed from the correct material, A36 steel. The failure of the specimen is illustrated in Fig. 6a.

The repair of PN3 was achived by gouging out the cracked welds and placing new complete penetration welds at both flanges. All remaining bolts and the cracked shear plate were removed. A new shear plate was welded on the opposite side from the removed old one. Figure 6b shows the repair details.

The repaired specimen, RN3, withstood up to he first 4-inch cycle. Yielding started during the first 2-inch cycle on the beam flanges and on the beam web. After completion of the first 4-inch cycle considerable white-wash spalling could be observed on both beam flanges and beam web and in the panel zone area. The maximum force was equal to 225 kips at the beam end displacement of 3.60 inches.. The specimen failed during the first excursion of the second 4-inch cycle at 196 kips load and 1.48 inch beam end displacement. First crack started at the edge of the beam bottom flange and propagated through the entire flange with a short branch near the flange axis, as shown in Fig. 6c.

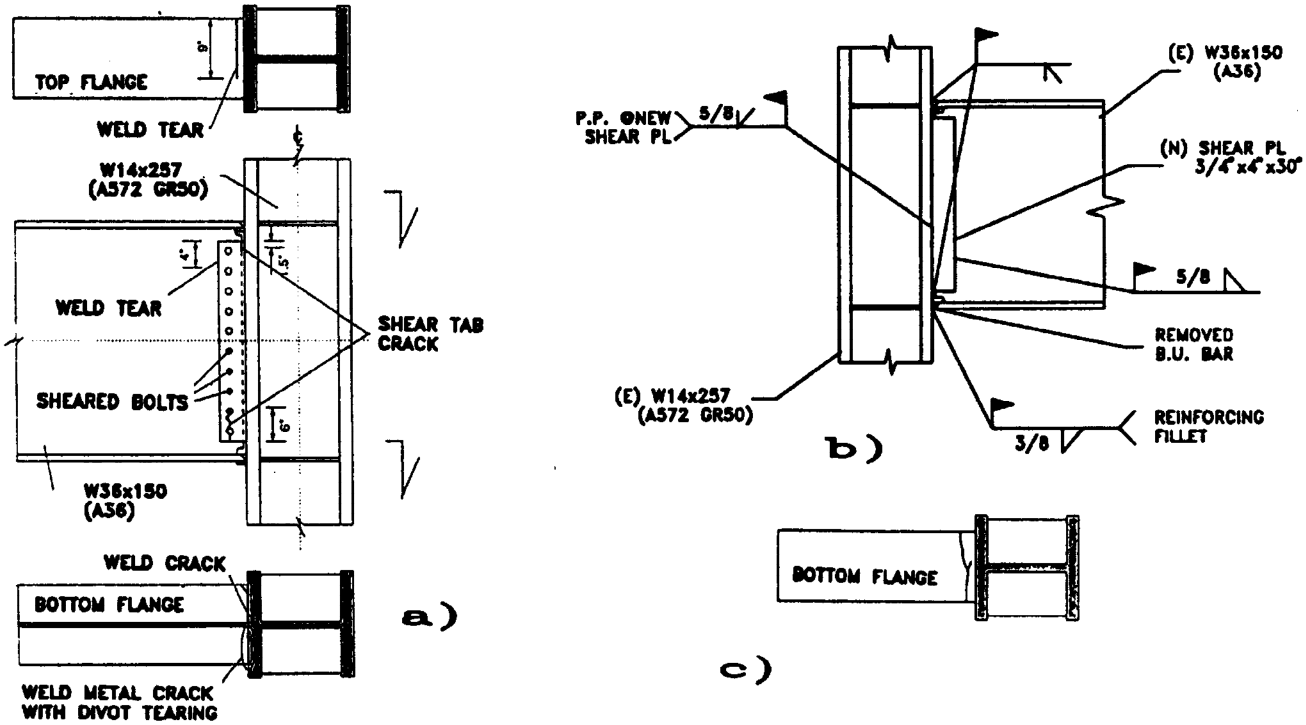


Fig 6. Specimen 3 -- Damage, repair, and damage of repaired connection

EVALUATION OF EXPERIMENTAL RESPONSE

Table 2 shows peak values of various response quantities obtained for Pre-Northridge and repair specimens in this test program. Stiffness values correspond to the applied load versus the beam end displacement. Yield values correspond to the event where there is a sudden increase of energy dissipation through plastic work (Popov et al., 1996). Connection rotation was defined as beam end displacement divided by beam clear length. Energy refers to the total plastic work developed by the specimen up to failure.

Table 2. Summary of Experimental Response

Specimen	PN1	RN1	Δ%	PN2	RN2	Δ%	PN3	RN3	Δ%
Elastic Stiff. (k/in)	151	166	10%	152	183	20%	146	136	-7%
Yield Load (kip)	136	179	32%	130	159	22%	137	136	-1%
Yield Displ. (in)	0.90	1.08	20%	0.86	0.87	1%	0.94	1.00	6%
Max. Load (kip)	223	260	17%	195	319	64%	198	225	14%
Peak. Displ. (in)	2.57	3.54	38%	1.62	4.02	148%	2.67	3.60	35%
Plastic Rot. (%)	0.88	1.55	76%	0.34	1.77	420%	1.07	1.61	50%
Energy (k-ft)	140	370	164%	40	620	1450%	200	420	110%

The hysteretic response of the specimens can be visualized in Figs. 7 and 8, which provide force versus beam end displacement plots and moment at column face versus plastic rotation plots, respectively. The response of the repaired specimens is shown in the second row.

Clearly, the repair techniques evaluated were successful, in the sense that the repaired specimens were stronger and had significantly better energy dissipation characteristics than the Pre-Northridge connections. The most dramatic improvement was obtained for specimen PN2, which was retrofitted with a haunch. RN1 and RN2 were also stiffer than PN1 and PN2 due to the beam/column joint repair. The reweld repair used for PN3 produced a somewhat softer specimen, with improved strength and ductility. It is important to note, however, that the failure mode of all three repaired specimens was abrupt and without previous warning.

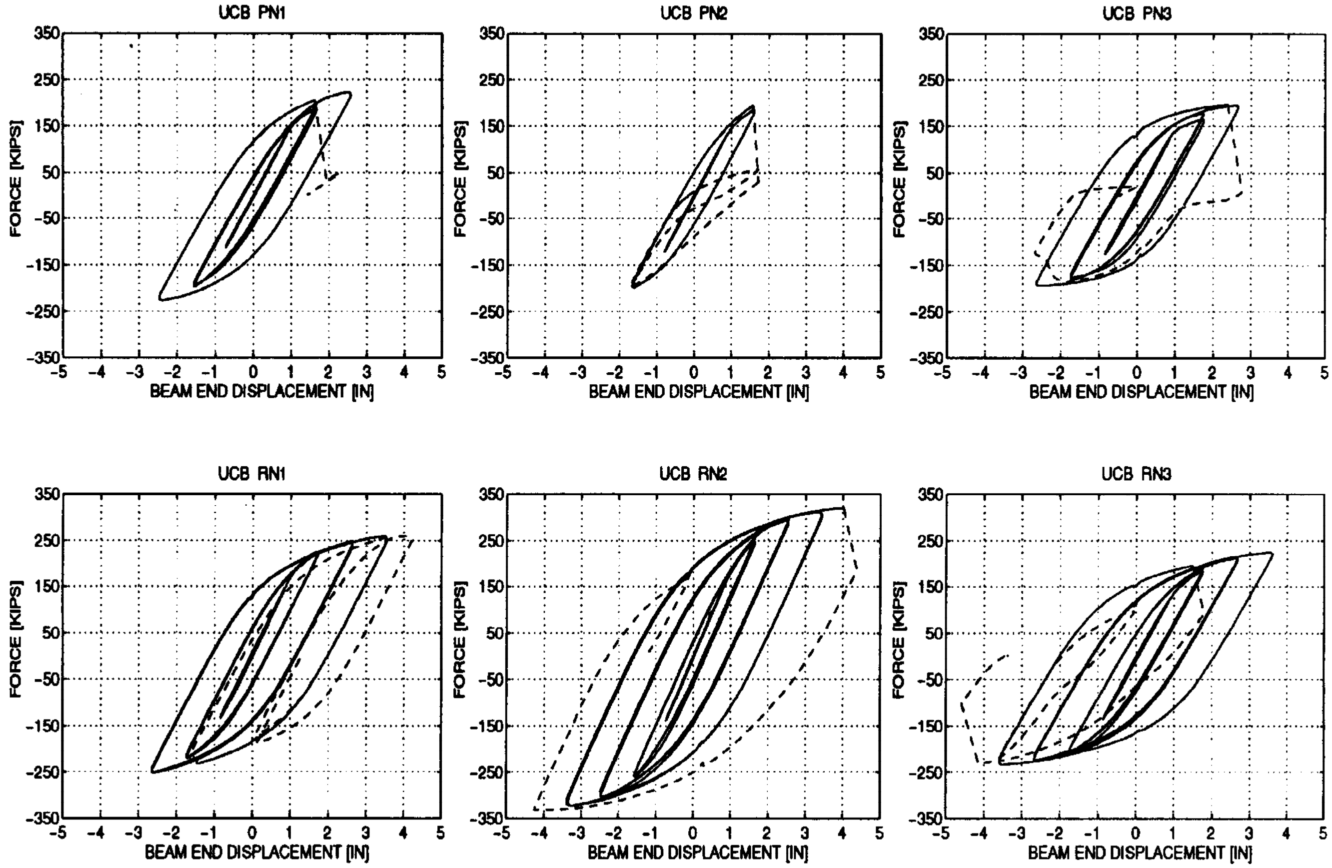


Fig. 7. Applied Load vs. Beam End Displacement

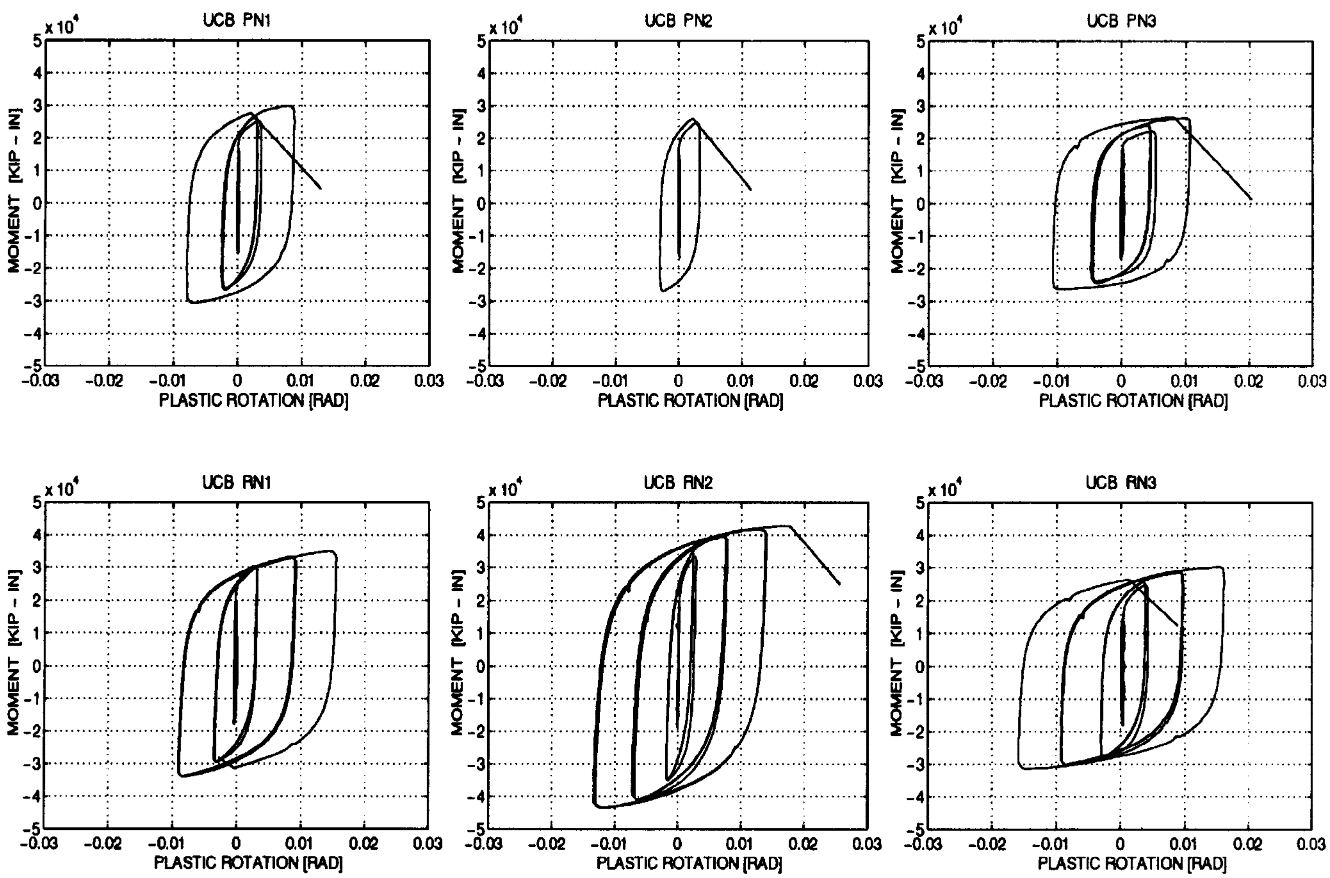


Fig. 8. Moment @Column Face vs. Plastic Rotation

CONCLUSIONS

From the behavior observed during the tests and the results presented above, the following conclusions can be drawn:

1. The pre-Northridge connections showed a very poor response to cyclic loads. They had low capacity for energy dissipation and failed in a brittle, abrupt mode, without warning.
2. The repaired schemes studied here were moderately successful. The repaired specimens were stronger, more ductile, and had better energy dissipation characteristics than the original ones.
3. The failures of the repaired specimens were, however, sudden resulting in brittle fractures. This is a highly objectionable characteristic for structures in seismic zones.

REFERENCES

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