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SEISMIC SAFETY OF DAMAGED WELDED STEEL MOMENT FRAMES

A. ASTANEH-ASL, D. MODJTAHEDI, K. M. McMULLIN and E. D'AMORE

Department of Civil and Environmental Engineering
University of California, Berkeley, CA, 94720-1710, USA

ABSTRACT

The main goal of the study was to investigate seismic safety and stability of welded steel moment frames damaged during the 1994 Northridge earthquake. A 4-story, a 14-story and a 27-story buildings damaged during the quake were studied. Two inelastic models of the moment frames from the buildings were developed. One model represented the undamaged condition and in the other model all connections were assumed to be damaged. The frame models were subjected to various levels of acceleration time-histories of Northridge and Miyagi-ken-Oki earthquakes. The study indicated that in general, seismic response of the damaged frames was less than the response of the undamaged frames. No tendency to collapse in damaged frames of case-study buildings was detected. Clearly in the damaged frames, where bottom flange welds were cracked, the structure was showing considerable amount of semi-rigidity. Such semi-rigidity and the resulting elongation of the period, increase in damping, and decrease of the stiffness could be the main cause of the reduced response of the damaged frames and their surprising stability.

KEYWORDS

Seismic safety; steel moment frames; Northridge earthquake; weld damage; dynamic response analysis, inelastic behavior; semi-rigidity; shear connections.

INTRODUCTION

The 1994 Northridge earthquake in Los Angeles caused considerable damage in building and bridge structures. More than 100 modern welded steel moment frame structures sustained damage in their girder-to-column joints. Earthquake damage to welded steel moment frames was primarily in the form of cracking of connection welds. In some cases, the weld cracks had propagated into the column, girder or panel zone (Youssef et al. , 1995). The most common type of damage was cracking of welds connecting bottom flange of the girders to the columns.

Even though some structures had cracks in significant number of their connections, no partial or total collapse of any damaged welded moment frame structure occurred. In most cases, there was no visual indication of damage to non-structural elements of the damaged structures. In most cases, only after removing the connection cover and fire-proofing and by ultra-sonic testing the cracks in the connections could be detected.

In the aftermath of the Northridge quake, and as more and more modern structures were being investigated and found damaged, public safety became a major concern. The question was "how safe are the damaged welded steel moment frames?" The question became extremely important as gradually it became clear that there are no rapid and reliable methods to detect the damage and to repair the damaged welded moment connection areas. To provide information that could be used to understand seismic behavior of the damaged welded steel moment frames and to establish level of seismic hazard, the study summarized here was conducted during March through June of 1994. The study focused on most common type of damage which was cracking of weld connecting the bottom flange of the girders to the columns.

SEISMIC SAFETY STUDIES

The main objective of the studies was to investigate the possibility of collapse of damaged welded steel moment frames during an aftershock of the 1994 Northridge earthquake or other independent earthquakes that might occur before the damage is repaired. For the study, three buildings were selected. The buildings were a 4-story, a 14-story and a 27-story modern office buildings and all had various level of damage and cracking in their welded steel moment frame connections.

A typical moment frame from each case-study building shown in Fig. 1, was selected. For each frame two inelastic models were developed. One inelastic model represented each frame without any damage in the connections and the second inelastic model represented the same frame but with bottom flange welds in *all* connections assumed cracked. The undamaged and damaged connections are shown in Fig. 2. The actual damage to connections of case-study frames was considerably less than the assumed level of damage. The conservatism was intentionally introduced to be able to detect any instability in the damaged frames.

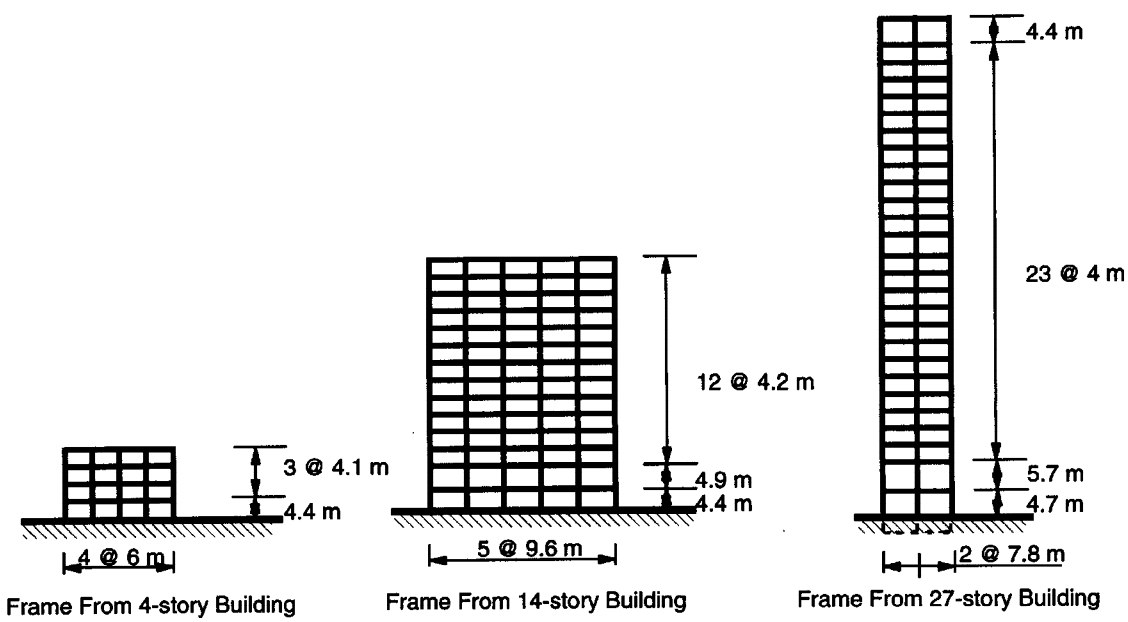

Fig. 1. Moment frames from the case-study buildings

Another conservative assumption in modeling the frames was that the strength or stiffness contribution of floor slabs, shear connections outside the moment frames and the non-structural elements of the building were neglected. It is believed that in actual buildings each of the above-mentioned elements contributes certain amount of stiffness and over-strength to the structure.

The connection hysteresis models were the most important parameter in the study. The connection hysteresis models were represented by a bi-linear model of the moment-rotation behavior. To establish the connection models, data from actual cyclic tests could be used. For undamaged welded connection data on cyclic moment-rotation behavior could be found in the literature. However, at the time of these studies there was no tests data on the cyclic behavior of damaged welded moment connections. In the absence of actual cyclic test data for damaged connections, the phenomenological model shown in Fig. 3 was developed. Later, Anderson et al., 1995 conducted a series of cyclic tests on three damaged moment connection specimens. The specimens were actually taken from a two story damaged building with significant damage which later was demolished. A sample result of these tests is shown in Fig. 4 which confirmed the phenomenological model of moment-rotation shown in Fig. 3.

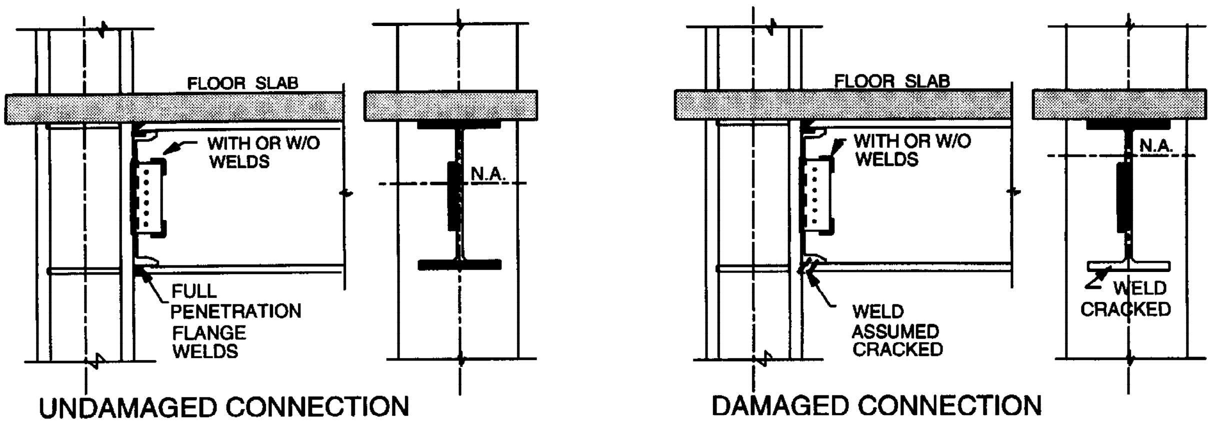

Fig. 2. Undamaged and damaged connections of welded steel moment frames.

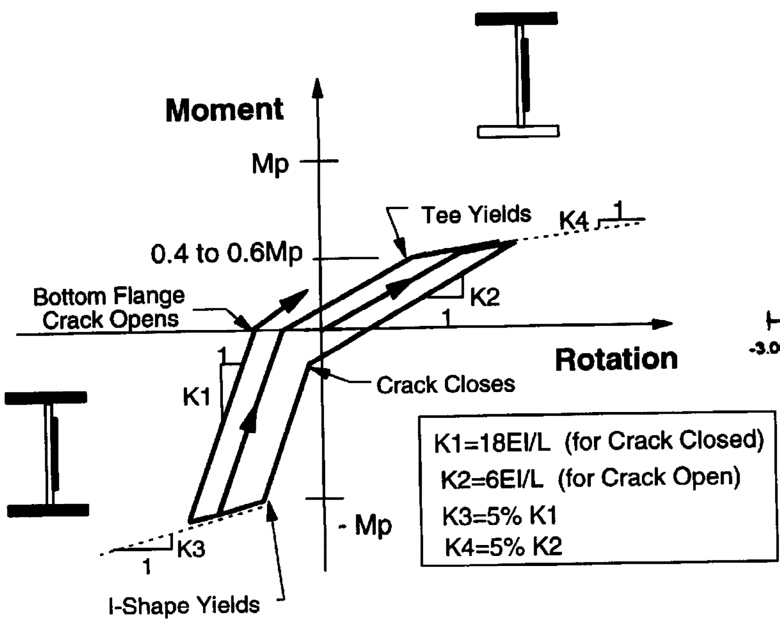

Fig. 3. Connection models.

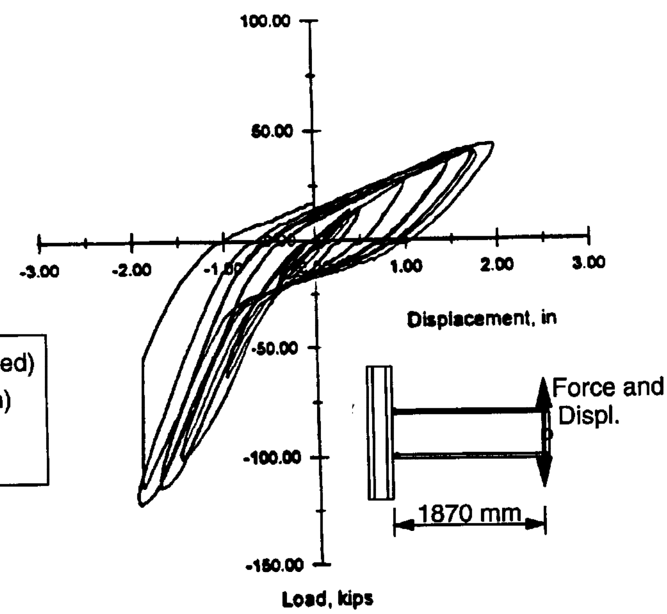

Fig. 4. Results of damaged connections tests by Anderson and Xiao, 1995.

Using the computer program DRAIN-2DX, a number of inelastic time history analyses were conducted. The computer program DRAIN-2DX does not have an inelastic element to represent the unsymmetric hysteresis behavior shown in Fig. 3. Therefore, an equivalent symmetric bi-linear hysteresis model shown in Fig. 5 was developed and used in the analyses.

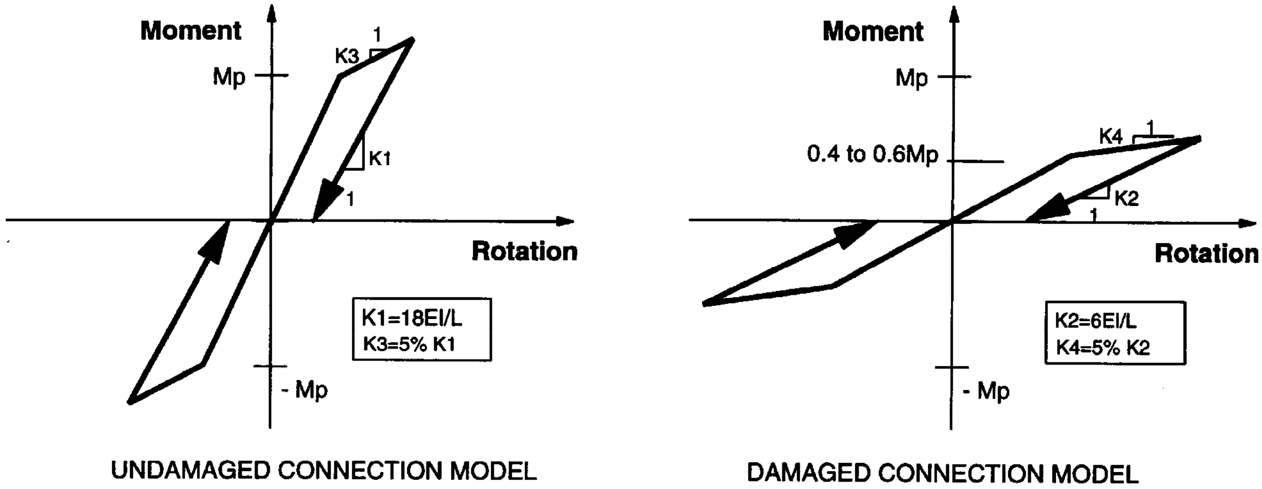

Fig. 5. Hysteresis models of undamaged and damaged connections used in the analyses.

The inelastic models of the frames from three damaged buildings, shown in Fig. 1, were subjected to acceleration time histories of a variety of ground motions recorded in the past including records from the 1994 Northridge and the 1978 Miyagi-ken-Oki earthquakes. The record from the Northridge earthquake represents a high velocity, short duration, near field earthquake while the amplified Miyagi-ken-Oki record represents a relatively long duration earthquake with high energy distribution over a wide range of frequencies. Figure 6 shows acceleration time history of one of the horizontal components of the 1994 Northridge Newhall record and the amplified Miyagi-ken-Oki record used in this study. The Actual record of Northridge-Newhall had a maximum peak acceleration of 0.59g. The Miyagi-ken-Oki record was amplified to have a maximum peak acceleration of 0.6g. In all time-history analyses, the kinematic nonlinearities in the form of P-Δ effects were included.

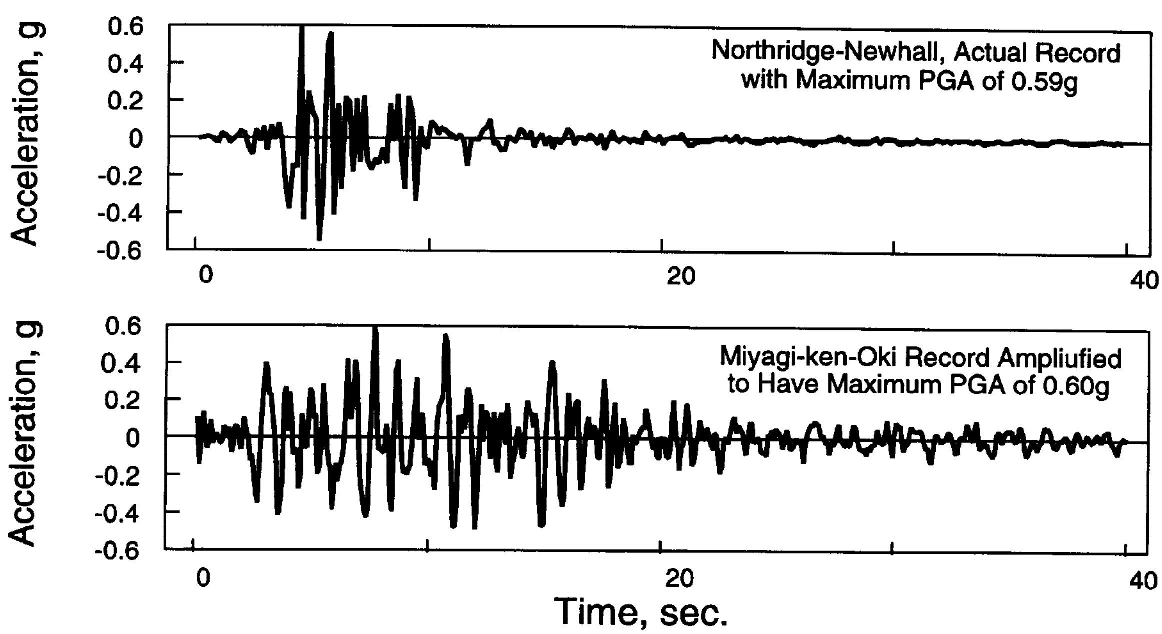

Fig. 6. Time history of the horizontal components of Northridge-Newhall record used in the study.

SUMMARY OF RESULTS

The results of analyses presented herein are in the form of time histories of base shear and drift response. Due to limitations of space only partial results of the response of case-study frames to Northridge and amplified Miyagi-ken-Oki records are presented.

Response of the 4-Story Undamaged and Damaged Frame

Figure 7 shows base shear and roof drift time histories of the 4-story frame to the Northridge-Newhall records. The drift time history plot indicates that during the first cycle roof drift of the damaged frame is greater than the undamaged frame. However, beyond the first cycle, the damaged structure shows much less drift than the undamaged structure. In addition, the damaged structure oscillates about the vertical axis without developing any major permanent out-of-plumbness. However, the undamaged frame, oscillates about a permanent drift of about one percent. Figure 6 indicates that throughout the motion, the base shear developed in the 4-story damaged frame is less than the base shear in the undamaged frame.

It appears that when a rigid undamaged moment frame sustains damage it is converted to a semi-rigid frame. As a result, the damaged semi-rigid frame has smaller response. This phenomenon of smaller seismic response of semi-rigid frames relative to similar but rigid frames has been observed in other analytical studies as well as laboratory tests (Astaneh-Asl, 1994).

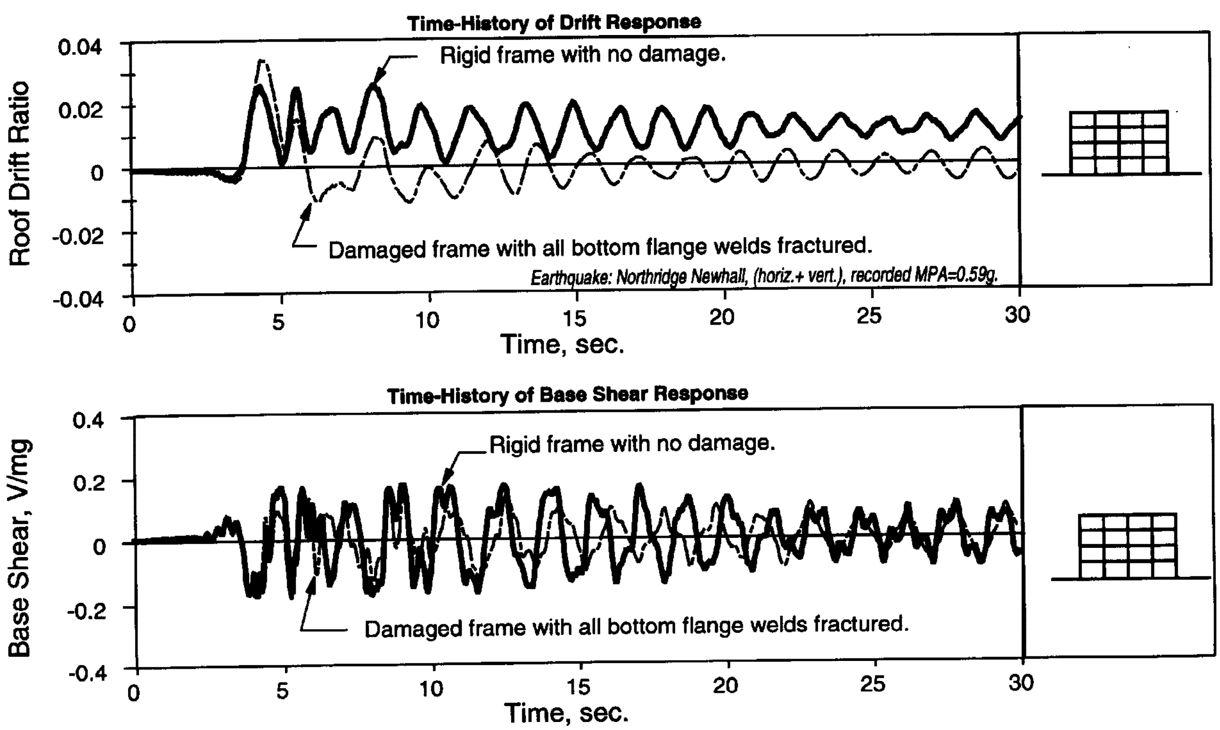

Fig. 7. Response of 4-story undamaged and damaged frames.

Response of the 14-Story Undamaged and Damaged Frame

Figure 8 shows roof drift and base shear time history of the 14-story case-study frame. The ground motion used is the same Northridge-Newhall ground motion which was used for the 4-story frame discussed earlier. For this 14-story frame, the roof drift response of the damaged frame is less than the drift response of the undamaged frame throughout the time history. Particularly, the response of the damaged structure after first 22 seconds is very small while the response of undamaged structure is still considerable, Fig. 8. Once again, it appears that the semi-rigidity of the damaged frame is the main reason in reducing the response of the damaged frame relative to the response of the undamaged frame.

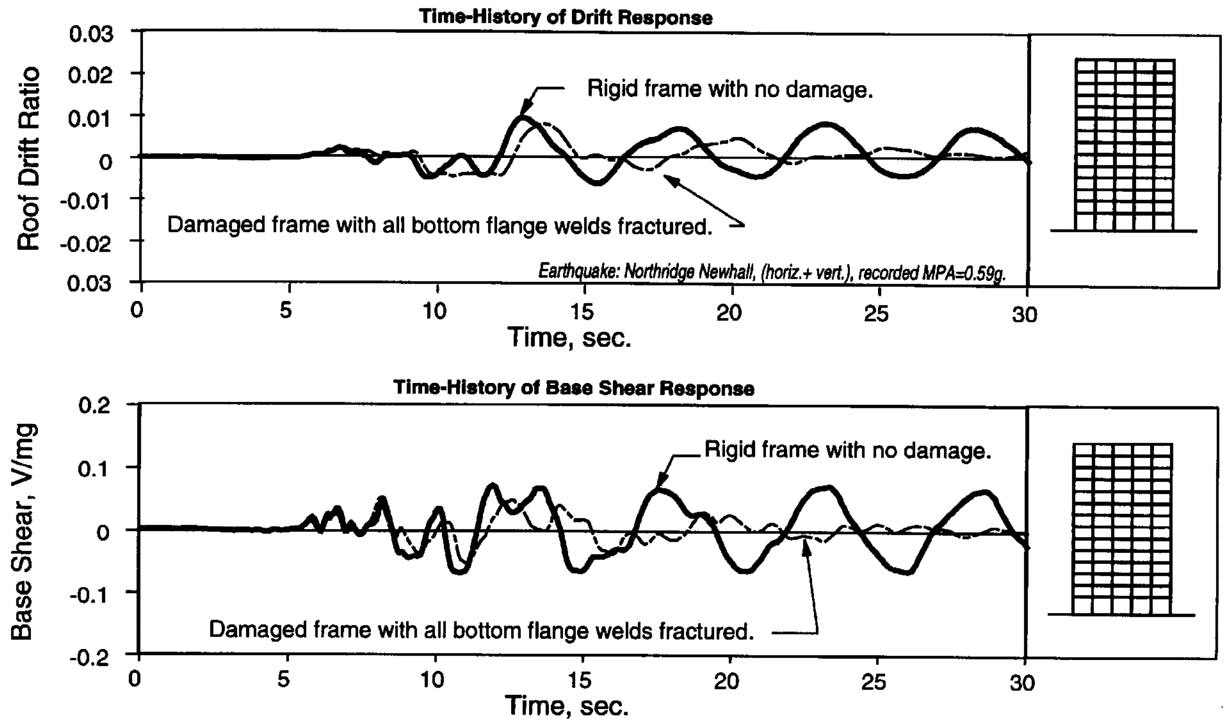

Fig. 8. Response of 14-story undamaged and damaged frames.

Similar to the 4-story frame discussed above, the base shear of the damaged 14-Story frame was also much smaller than the base shear of the undamaged frame. This phenomenon was consistently observed when various levels of intensity of a number of different past earthquake records were used. Another observation of the results of analyses was that for undamaged frames the first mode appeared to be very dominant while for the damaged and semi-rigid frames, higher modes were also equally present. This can be observed in base shear time history shown in Fig. 8 where the full line curve which is for undamaged frame shows dominance of first mode while the dashed curve for damaged frame shows higher modes with shorter periods.

To study the effects of characteristics of ground motions on the response, acceleration records from a number of past earthquakes were used. In some cases, the records were amplified to a certain target maximum peak ground acceleration. Figure 9 shows roof drift and base shear response of 14-story undamaged and damaged frames to amplified Miyagi-ken-Oki records with 0.6g PGA. The response of 14-story structure to amplified Miyagi-ken-Oki was not significantly different than the response to Northridge-Newhall response as discussed earlier.

Response of the 27-Story Undamaged and Damaged Frame

Figure 9 shows roof drift and base shear time history of the 27-story case-study frame. The ground motion used is the Northridge-Newhall ground motion with 0.59 PGA. Similar to the 14-story frame discussed earlier, the roof drift response of the damaged 27-story frame is less than the drift response of the undamaged frame throughout the time history.

To study the possibility of collapse of damaged steel frames, the case-study frames were subjected to ground accelerations with ever-increasing maximum peak accelerations. Figure 10 is a sample of results obtained from these analyses for the 27-story case-study frame. In this case, the frame was subjected to horizontal and vertical components of the Northridge-Newhall records with maximum peak accelerations amplified to 1.5g. As mentioned earlier, maximum PGA for Northridge-Newhall record horizontal and vertical components was 0.59g. As Fig. 10 indicates, the 27-story frame does not show a tendency to collapse. With the exception of first peak, the drift response of damaged frame is less than the drift of the damaged frame. The base shear response of the damaged frame is also less than the undamaged frame.

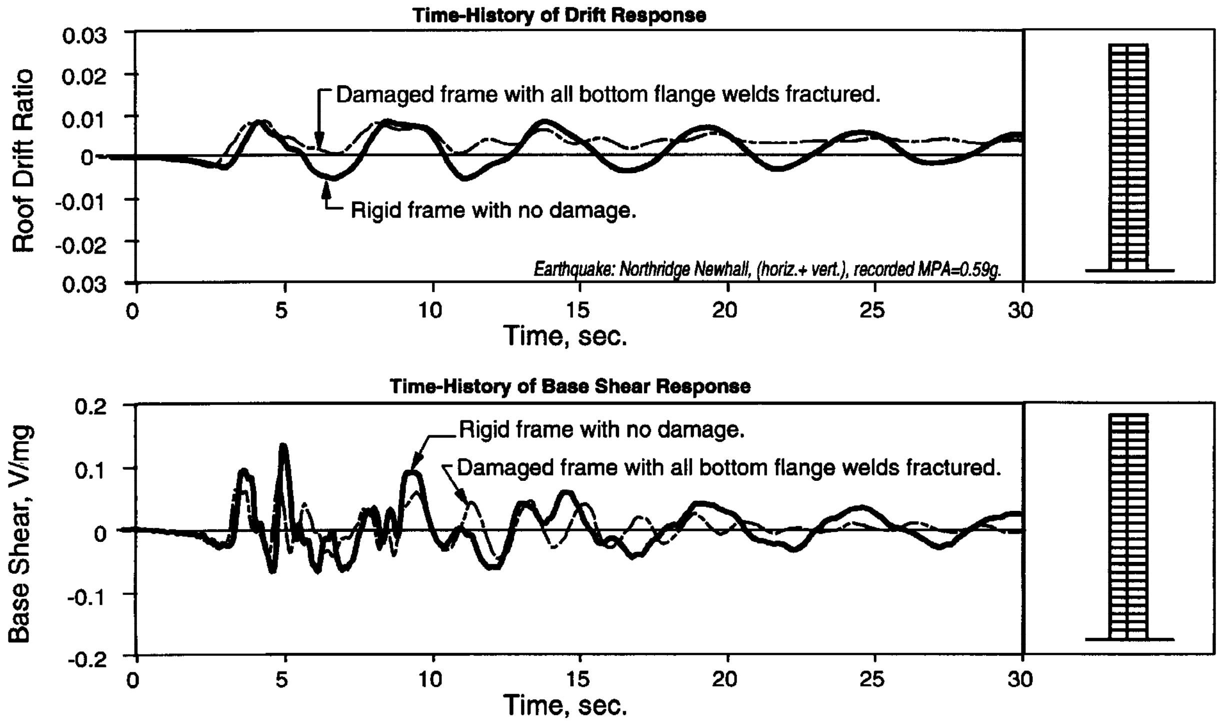

Fig. 9. Response of 27-story undamaged and damaged frames to Northridge-Newhall records.

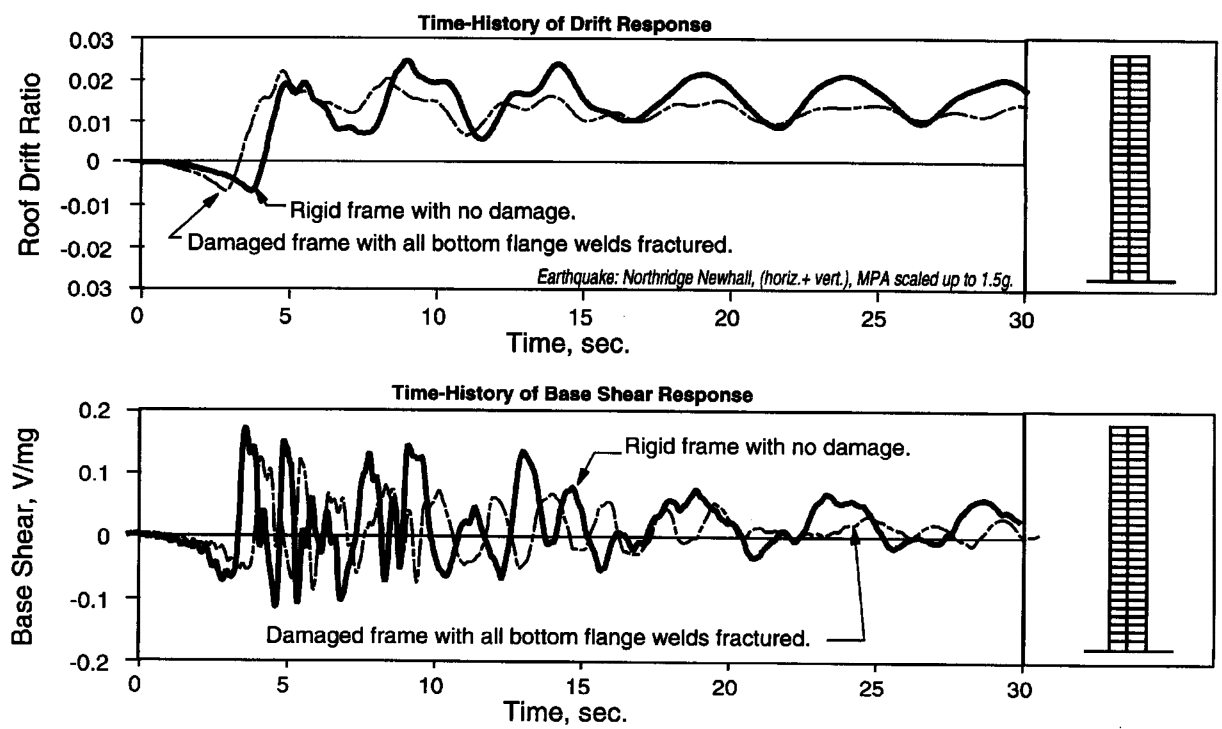

Fig. 10. Response of 27-story undamaged and damaged frames to Northridge Newhall records amplified to have 1.5g maximum PGA.

CONCLUSIONS

Based on the response of three case-study frames, with damage to bottom flange welds subjected to a select number of earthquake records, the following conclusions could be reached;

1. The base shear of damaged frame was almost always less than the undamaged frame

2. The roof drift response of damaged frames was also in general less than the roof drift of undamaged structure.

3. The reduction in seismic response of damaged frames is related to semi-rigidity that occurs in these frames as a result of cracking of relatively stiff and brittle welds.

4. The case study frames did not show a tendency to collapse even when the ground motion records were amplified to maximum PGA of 1.5g.

5. Observing the semi-rigid and beneficial behavior of the damaged steel moment frames one of the potentially useful repair strategies for the damaged welded moment frames can be to convert the welded moment frames to ductile but slightly semi-rigid bolted moment frames (Astaneh-Asl, 1995).

It should be emphasized that in the above case-study frames, damage was only in the bottom flange welds. The conclusions or results should not be applied to other cases of damage such as fracture of girders, columns or the panel zones.

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