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PERFORMANCE ANALYSIS OF A DAMAGED 11 STORY STEEL MOMENT FRAME BUILDING DURING THE 1994 NORTHRIDGE EARTHQUAKE

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ABSTRACT

This paper presents results of performance analysis studies on an 11 story steel moment frame building which was damaged during the January 17, 1994 Northridge earthquake. Extensive field inspections of damage were performed prior to the start of this investigation. These inspections/tests revealed some degree of damage in more than 28% of the moment frame joints of the building. Extensive two and three dimensional, linear and nonlinear, static and dynamic analyses of the building were conducted to investigate and correlate observed damage with various elastic and inelastic analytical damage predictors. Statistical correlation of observed damage patterns with various analytical damage indicators are presented.

KEYWORDS

Damage; Performance; Inelastic; Analysis; Inspection; Steel; Moment Frame; Northridge; Building

INTRODUCTION

This paper presents an extensive analysis of the seismic performance of a mid-rise commercial building in the January 17, 1994 Northridge earthquake. D uring the field investigations virtually all frame connections in the building were exposed and visually inspected. The full penetration welds were then subjected to ultrasonic tests and fillet welds were tested using magnetic-particle and dye-penetrant techniques to expose damage not visible to the naked eye.

The building consists of six levels of office space over five parking levels. The approximate footprint of the building (plan dimensions of the parking levels) is 190 feet long (N-S direction) and 260 feet long (E-W direction). The office level plan dimensions vary from 145 feet wide by 205 feet long at the roof level to 190 feet by 260 feet at the first office floor. The office plan configurations contain several vertical setbacks and reentrant corners. The elevation of the roof level is approximately 135 feet above the grade.

The building is constructed of composite concrete and steel metal deck slabs which are supported by A36 structural steel beams and columns. The exterior skin is made of precast concrete panels and glass plates. Structural steel columns are supported at the foundation by cast-in-place reinforced concrete friction piles. The seismic load resisting system appears to consist of ordinary moment frames constructed of A36 structural steel girders and columns. We call these moment frames "ordinary" because in many cases they do not satisfy today's strong column - weak girder design provisions and the continuity/doubler plate requirements. There is a 260 feet long two story reinforced masonry shear wall at the south side of the structure which also serves as a lateral load resisting element. This wall has been considered in our three dimensional model of the structure. Seismic loads are delivered to the lateral load resisting elements by the composite concrete and steel deck slabs which act as horizontal diaphragms. Seismic loads in the lateral system are resisted at the foundations by the cast-in-place friction piles, and soil friction below the concrete slab-on-grade.

As a result of the post-earthquake site investigations, the building was considered unsafe for permanent occupancy, yellow-tagged, and evacuated. Out of a total of 920 connections, 913 have been inspected and tested. A total of 258 connections exhibit varying degrees of damage, including some spectacular damage to column flanges

BUILDING DESCRIPTION

A typical floor framing plan for this building is shown in Fig. 1. The lateral load resisting system in each direction consists of four moment-resisting frames (Fig. 2). In addition, in the East-West direction on line 11, there is a two-story, eight inch thick, reinforced masonry block wall which acts as part of the lateral resisting system. The moment resisting frames in the North-South direction are located on frame lines C, E, H, and M. The column spacings in this direction vary from 15 feet to 31 feet. In the East-West direction there are moment frames on lines 2, 6, 9, and 11. The typical column spacing for frames in this direction is 28 feet and 8 inches.

There are several vertical setbacks which create a number of reentrant corners and make a building which is irregular in both the vertical and horizontal planes. Typical girder depths are from the W36 series (A36 material). There is not much variation in the girder sizes either from bay to bay in a given level, nor from the bottom to the top of the structure. This may be an indication that the original design was controlled by the drift limitation criteria rather than stress criteria. The columns are from the W14 series (A36 material) and vary in size from W14x398 sections in the lower levels to W14x257 and W14x233 sections at the uppermost levels. The girders are generally much stronger than adjacent columns. Since the strong column - weak girder design strategy was not in the governing code provisions at the time of design of this structure, it seems that the designers opted for stronger girders as the more efficient means of satisfying the wind design drift limitations. Further details regarding building configuration and connection detailes are contained in Naeim *et al.* (1995).

DAMAGE DESCRIPTION

The 1994 Northridge earthquake caused widespread damage to the moment-resistant beam to column connections of this building. As a result of the site investigations which identified this damage, the building was considered unsafe for permanent occupancy, yellow-tagged, and evacuated. Out of a total of 920 connections, 913 were inspected and tested, and 258 connections exhibited varying degrees of damage.

As seems to be the case for the majority of damaged steel moment frame buildings in the Los Angeles basin, the nonstructural damage to this building was very limited. The building, however, exhibits an overall tilt of about one inch in the North direction. It seems that this tilt was caused by the Northridge earthquake. There is very little, if any, external sign of severe structural member deformation and overstress. For example, in the parking levels where significant connection damage occurred, one can only find hairline cracks on the concrete cover of the encased steel columns. This is true, even for columns whose connection with beams showed significant damage. The two story masonry block wall located at the south side of the building experienced limited damage and some of the concrete blocks were replaced. The concrete cover of the first level parking level ramp was cracked at midspan.

What is particularly interesting about this building is the extent of damage suffered by columns either in the form of torn flange nuggets or complete tearing of the column flanges (Figures 3 and 4). It was the discovery of the extent of column damage which caused inspecting structural engineers to recommend that the building be evacuated. Damage designations as defined by SAC Task 2 were used to identify damage types . The extent of connection damage seriously undermines overall frame integrity at some floors. For at Frame on line "C", all columns oriented in major direction (column lines 3, 4 and 6) have suffered some form of flange damage. Also at levels 10 and 11, two of the three columns show flange damage and one column shows damage propagated to the column web. While damage is spread throughout the height and in both horizontal directions, there is more damage in the upper floors in the North-South direction where the number of bays resisting lateral loads is generally smaller. The density of damage is also high in the connections around the ramp beams.

THE SITE AND GROUND MOTION CHARACTERISTICS

The site of this building is located in west Los Angeles and is underlain by an estimated 300 to 375 feet of Holocene and Pleistocene age alluvium which overlies Tertiary age sedimentary rock. The upper alluvium consists of silts, silty sand, and sands with minor amounts of clay. These soils are estimated to be medium stiff to stiff or medium dense to dense. The water level is anticipated to be at a depth deeper than 50 feet below the ground surface. Postulated ground motions representing the motion at the site during the 1994 Northridge earthquake were developed and utilized in this study.

ANALYSIS RESULTS

A three-dimensional computer model of the building was generated using the ETABS computer program (Habibullah, 1995) and the elastic demand/capacity ratios (DCR) were calculated using the LRFD formulations. Elastic (DCR) values suggest that for the synthetic motion postulated for the site the building should have remained essentially elastic.

Two nonlinear 2-D computer models were constructed (one for the E-W and the other for the N-S direction). In each computer model, all frames in the direction under consideration were included and connected by the rigid floor diaphragm assumption. Bilinear hysteretic behavior was assumed using a 5% strain-hardening ratio. The yield and ultimate curvatures correspond to the cross section's full elastic and full plastic strengths, respectively. 2% damping was assumed for the first mode and the mode nearest to 30 Hz. A version of the IDARC computer program modified for inelastic analysis of steel structures was utilized in this investigation.

To predict structural damage, damage indices were assigned at the element level (beams and columns) as well as story levels and to the overall structure. The damage model was utilized in the following form:

$$D_i = \frac{\Delta\varphi_a}{\Delta\varphi_u} + \beta \frac{E_h}{E_{h0}}$$

where D_i is the damage index, $\Delta\varphi_a$ is the maximum "permanent" curvature experienced, $\Delta\varphi_u$ is the maximum "permanent" monotonic curvature capacity (ultimate). E_h is the accumulative energy dissipation, E_{h0} is the energy dissipated by monotonic failure, and β is a calibrated parameter between 0.1 and 0.3 (a value of 0.25 can be derived from energy balance equations. Both nonlinear dynamic and static (pushover) analyses were utilized. In these figures, the maximum time history response to the synthetic motion at the site is marked by a black circle. Here it can be seen that:

1. The results of the pushover analyses and the time histories show a very good match at all stories. This is an strong indication that for this building, in spite of its complexity and vertical irregularities, the static pushover analyses may be used to obtain a good approximation to nonlinear dynamic analyses results with ground motions of widely differing severity.

2. These results reconfirm the conclusion from elastic analyses, that the response to the synthetic motion developed for the site should have been basically elastic at almost all stories in both directions. The extent of observed damage, hence, points to either premature failure of the joints, or a ground motion much more severe than that indicated by synthetic records. The latter hypothesis seems unlikely however, in view of the observed general patterns of damage in the area and the ground motions recorded elsewhere in west Los Angeles.

3. The pushover instability occurs by formation of a story collapse mechanism at the third floor. This occurs, in both directions, before the roof has a chance of experiencing relatively large story drifts. The maximum pushover roof story drift is 0.006 in the E-W direction and 0.013 in the N-S direction.

4. The influence of vertical ground motion on dynamic response of this building was negligible for all cases considered.

CORRELATION OF OBSERVED DAMAGE AND ANALYTICAL INDICATORS

The following analytical indicators are studied for possible correlation with observed building damage:

(a) beam demand capacity ratios as obtained from the 3-D elastic analysis (RBDCR)

(b) column demand capacity ratios as obtained from the 3-D elastic (RCDCR);

(c) beam damage indices obtained from nonlinear dynamic time history (THBDI);

(d) column damage indices obtained from nonlinear dynamic time history (THCDI);

(e) story damage indices for beams obtained from nonlinear dynamic time history analyses (THBSTDI);

(f) story damage indices for columns obtained from nonlinear dynamic time history analyses (THCSTDI)

(g) story damage indices for beams obtained from nonlinear static pushover analyses (POBSTDI);

(h) story damage indices for columns obtained from nonlinear static pushover analyses (POCSTDI).

Variation of these analytical indicators versus observed incidents of connection damage was studied using whisker plots. The damage occurred at unexpectedly low demand/capacity ratios (mean elastic DCR of 0.49 for beams and 0.72 for columns). Similarly, mean beam and column damage indices for damaged connections are also low. This is true both on a member-to member basis (THBDI and THCDI) and on a story-by-story basis (THBSTDI and THCSTDI). As expected mean damage indices for the pushover analyses are higher. This difference is more significant for columns than for beams. A large range of variation was observed for all analytical indicators for damaged connections.

Detailed statistical correlation studies (see Naeim *et al*., 1995) lead to the following observations (r represents the cross-correlation coefficient):

1. Four of the analytical indicators, two elastic and two inelastic, show correlations of statistical significance with damage severity. These are: elastic beam and column demand/capacity ratios (RBDCR and RCDCR), inelastic column damage index (THCDI) and inelastic column story damage index (THCSTDI).

2. Although statistically significant, none of the correlations with damage are very strong as evidenced by the large scatter of points around the corresponding regression lines..

3. The indicator with strongest correlation ($r = 0.32$) with damage severity is the elastic beam demand/capacity ratio (RBDCR) followed by the inelastic column damage index (THCDI, $r = 0.25$), and the inelastic column story damage index (THCSTDI, $r = 0.18$), which in reality is a weighted average of THCDI across a given floor.

4. Correlation of other analytical indicators with damage severity is not statistically significant.

If a histogram of RBDCR for all moment frame beams in the structure (damaged and undamaged) is compared to the same histogram for damaged connections, it can be noticed that the average beam demand/capacity ratio at damaged connections is about 25% larger than the average for all beams in the structure (mean DCR of 0.5 compared to 0.4). This means that if one is looking for damaged connections, one is better off by looking at beams with higher stress. The unfortunate news, however, is that there is no direct correlation between increased stress levels and a rise in incidents of observed damage (i.e. the beams with the highest DCR may or may not be damaged).

A similar picture is true for column DCR values (RCDCR). The average column DCR at damaged connections is about 33% higher than the average DCR for all columns. The same weakness, of the lack of any direct correlation between increased DCR values and incidents of damage is present for columns too.

The beam damage indices have large values in all the sections that failed or were found deficient during inspections. In addition, no damage occurred in sections with a low beam damage index. However, the beam damage index indicated large potential damage in some sections which were found undamaged during the inspection. The beam damage index missed many cases of observed damage, but it is excellent at pointing out damage whenever it is predicted. As a matter of fact, it is always correct in predicting damage for conventional 2-D, four member, joints (two beam and two column elements coming to the joint).

The column damage index (THCDI) although better than elastic indicators, is not as strong an indicator, in these analyses, as the beam damage index. One reason may be the significant biaxial bending action in many of the damaged columns. A feature that is not, and cannot, be modeled with a two dimensional model. It is expected, however, that the reliability of column damage indices will approach that of the beam indices, if and when three dimensional inelastic computer models are developed.

Distribution of the story damage indices for beams and columns (THBSTDI and THCSTDI) indicated that story levels with a beam story index higher than 0.40, or a column story index higher than 0.80 are good candidates for starting a damage inspection. In many practical cases, the pressing question is where to start looking for possible joint damage. In these cases, the combination of story damage indices and member damage indices may prove to be very useful. For example, one could first choose the floor(s) with the highest story damage index, and then narrow the scope by selecting members which exhibit the highest damage indices within the selected floor(s). The pattern of damage indices obtained from the static push-over analyses is very similar to that obtained from the time history analyses.

CONCLUSIONS

The results were presented of the performance analysis studies on an 11 story steel moment frame building located in west Los Angeles. Extensive two and three dimensional, linear and nonlinear, static and dynamic analyses of the building were conducted to investigate and correlate observed damage with various elastic and inelastic analytical damage predictors. Statistical correlations of observed damage patterns with various analytical damage indicators were presented. The results indicate that:

(a) The damage observed was much more extensive than that predicted by analyses.

(b) The spatial distribution of incipient cracks suggests that incipient cracks probably existed before the Northridge earthquake occurred.

(c) The average beam DCR at damaged connections is 20% higher than average DCR for all connections (damages and undamaged). This ratio for columns is 33%. However, there is no direct correlation between increased DCR and actual damage.

(d) Although there is no one-to-one correspondence between analysis results and damage, certain analytical indicators do provide strong indications of where damage might be present.

(e) Elastic beam stresses correlate better than other analytical indicators with the severity of the observed damage, the inelastic column damage index is the second best.

(f) Inelastic beam damage indices are very useful in predicting damage. These indices have large values in all the sections that failed or were found deficient during inspections. In addition, no damage occurred in sections with a low beam damage index. However, the beam damage index indicated large potential damage in many sections which were found undamaged during the inspection.

(g) Both elastic and inelastic analyses indicate that the columns are relatively weaker than beams and the nature and extent of observed damage corroborates this trait.

(h) The results of the nonlinear pushover analyses correlate relatively well with the results obtained by inelastic dynamic analyses for ground motions of various severity.

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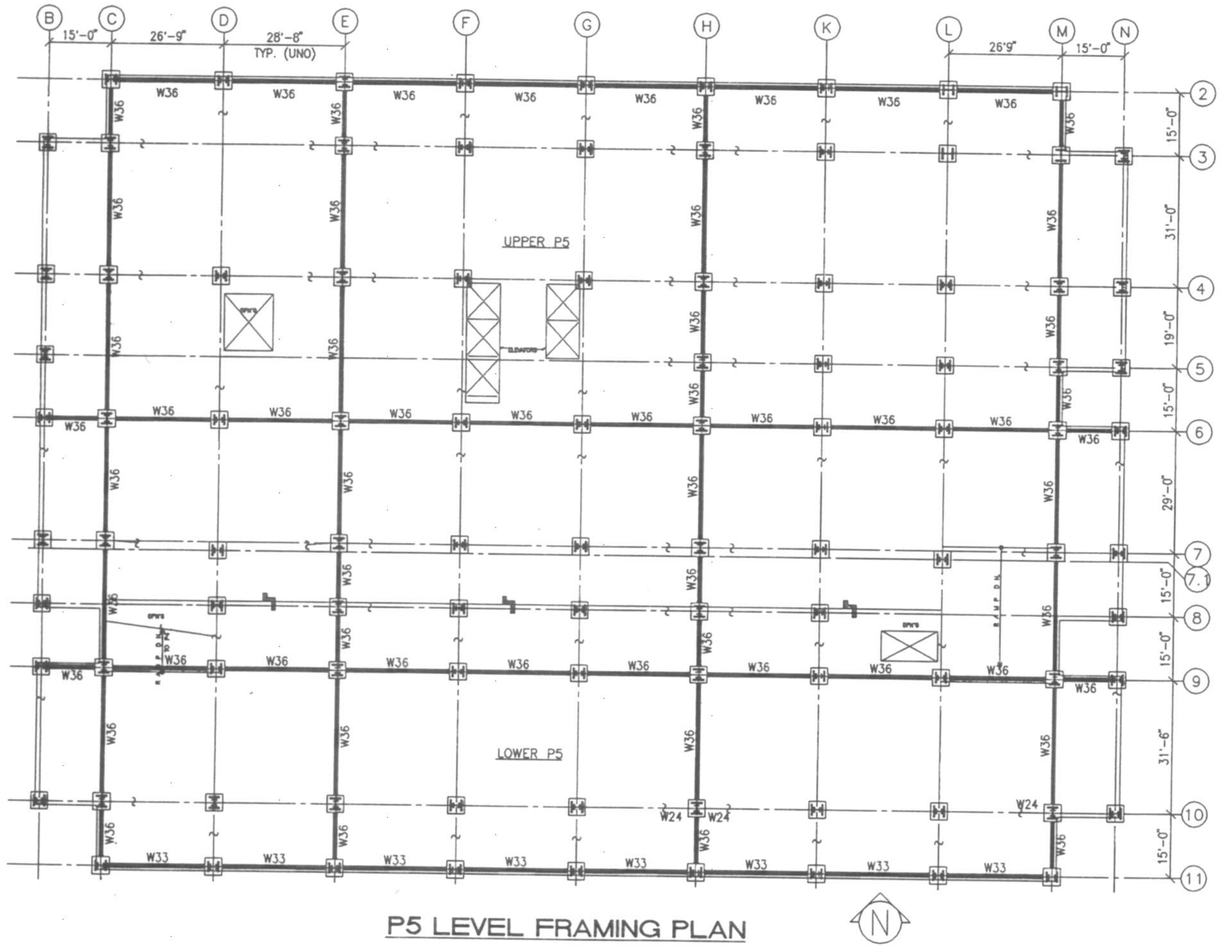

Figure 1. Typical floor framing plan of the Building.

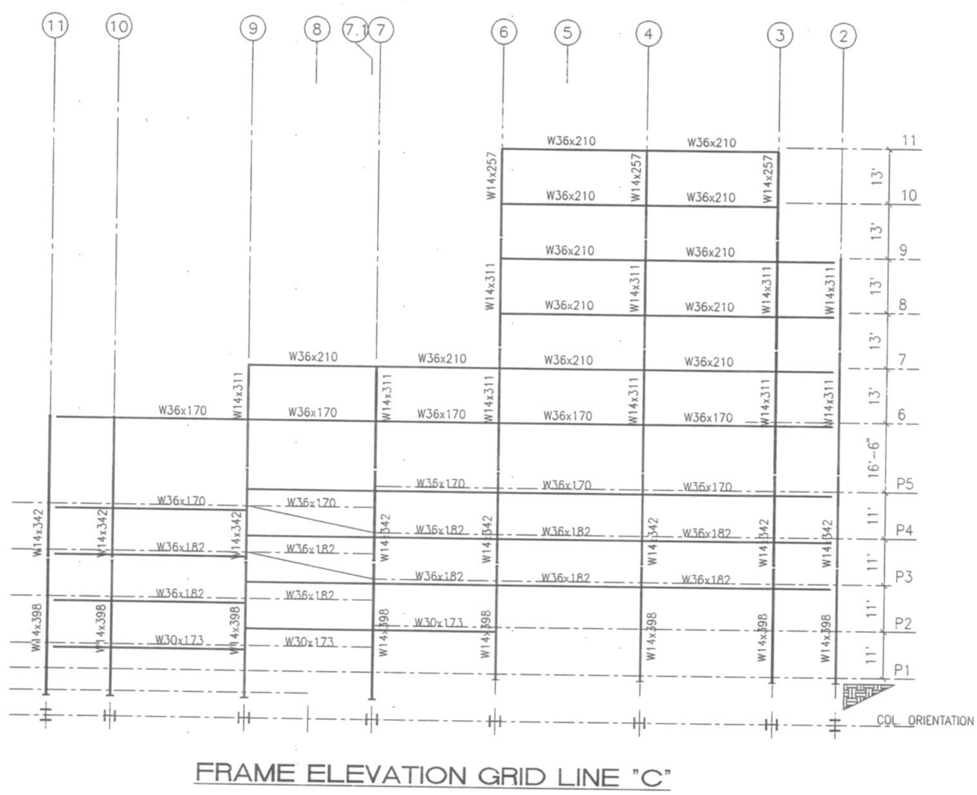

Figure 2. Typical moment frame elevation.

Figure 3. Beam-column connection damage.

Figure 4. Beam-column connection damage.