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EFFECT OF CONNECTION FRACTURE DISPERSION ON SEISMIC DEMANDS OF WELDED STEEL MOMENT FRAMES

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ABSTRACT

Appropriate modeling of connections behavior is needed for seismic evaluation of structures. However, large dispersion is observed in welded steel moment frame connections performance during recent earthquakes and experimental tests. In this study, the effect of dispersion in pre-Northridge connections characteristics is investigated. Two mid-rise steel moment frames are designated. Various types of fractures in beam-column connections with various configurations are modeled. Sensitivity study is carried out on parameters affecting the structural response for two probability levels by implementation of nonlinear dynamic analysis. Results show that variation in connection characteristics moderately affects the displacement demands of structures.

Keywords: Degradation; hysteresis loop; welded connection; IDASS; pre-northridge; connection fracture

1. INTRODUCTION

The 1994 Northridge and a year later the Kobe earthquakes revealed several deficiencies in steel moment connections. One side, widespread losses and the other side existence of numerous older vulnerable steel moment frames, forced engineering society to implement new approaches for design and seismic evaluation purposes. To do this, several questions about modes of failure and parameters affecting the performance of connections, must be answered first. Lee and Foutch [1] investigated performance of steel moment frames with Pre-Northridge connections. They used a reliability framework to evaluate the effect of

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brittle connection, panel zone and interior gravity frames for 2/50 and 50/50 probability levels. Foutch and Yun [2] used two groups of models for seismic evaluation. First, centerline models with nonlinear beam-columns and panel zones and second, Pre-Northridge connections. Results were compared with elastic models which are used for steel frames design. Yun et al. [3] proposed a performance evaluation approach for steel moment frames based on reliability theory using nonlinear dynamic analysis.

Roeder [4] investigated variations in yield mechanisms and failure modes for pre-Northridge connections. Several connection types were evaluated and strategies were proposed to improve the seismic performance of connections. Ibarra et al. [5] proposed hysteresis models for steel and reinforced concrete members. They used bilinear, peakoriented and pinching models to calculate inelastic response of SDOF systems.

Since the quality of connection fracture and affecting parameters are not clearly known, this study attempts to investigate the effect of various possible types of brittle connection behavior on the seismic performance of mid-rise buildings. two model structures were designed in accordance with Iranian design codes. Fracture in both beams and column members were modeled and finally the sensitivity of structural response to hysteresis parameters was investigated.

2. HYSTERETIC BEHAVIOR MODELING

A prevalent failure mode in Pre-Northridge connections is the fracture of the beam flange weld, as shown in Figure 1. Fracture in column is also probable. In this case, fracture is initiated in the beam flange and is propagated toward the column flange or web [6]. When connection fractures, moment strength of the connection, M_{red} , drops to a fraction (20% to 30%) of its plastic moment capacity, M_p . For column fracture, moment strength drop is more expected. Since, the residual moment is about 10% or 20% of the column plastic moment capacity [7]. Connection fracture may occurs before nominal plastic capacity of member called "early fracture" (Figure 3) or at a pre-specified rotation, θ_f , (Figure 4). Reduction in moment capacity is observed when the crack is opened. Initial strength is reversed when crack is closed again [8]. Figure 2 depicts an experimental hysteretic behavior of a pre-Northridge connection.



Figure 1. Brittle fracture in the heat affected zone [9]



Figure 2. Typical hysteresis rule for a pre-Northridge connection [10]

Dispersion of test results due to different types of fracture and also lack of sufficient knowledge about quality of fracture makes attentive modeling of connection behavior difficult. Therefore, analytical models for connection fracture are simplified to some extent. For this reason, analytical models with ability to model post fracture behavior of a connection are scarce [11-14]. Moreover, locations of connection fracture in structure are randomly selected [15].



Figure 3. Early fracture of a connection [16]

Figure 4. Fracture at pre-specified rotation [16]

3. MODELS AND GROUND MOTION RECORDS

Two 5-story and 9-story special steel moment frames designed in accordance with Iranian seismic code [17] and Iranian steel design code [18]. Models have stories with 3.2m height and bays with 4.0m length, located on area with very high seismicity and soil type III [17]. Figure 5 depicts a view of structures in plan and in elevation. Buildings are square in plan. Thus 2D frames are selected for analysis as shown in figure 5. It is supposed that steel's yield stress is 2400 Kg/cm^2 .floor's dead and live load are 700 Kg/m^2 and 200 Kg/m^2 .



Figure 5. Selected frames view in plan and elevation

In addition, a set of twenty ground motion records were selected. All recorded on soil type III and include no directivity effect. The records are selected from the PEER Center Ground Motion Database [19] as listed in Table 1. For 10%/50 probability level, the ground motions were scaled to coincide to the design spectrum in the range of 0.2T to 1.5T. The design spectrum is scaled by a factor of 1.5 to produce the 2%/50 hazard spectrum [20]. Figure 6 shows the scaled ground motions for 10%/50 probability level.

| Record ID | Event | Year | \mathbf{M}^{1} | Station | \mathbf{R}^{2} (km) | Soil ³ | Mechanism | PGA (g) |
|-------------------------|-----------------------------|-----------|------------------|-----------------------------------|-----------------------|-------------------|---------------------|------------|
| LP89agw | Loma Prieta | 1989 | 6.9 | Agnews State Hospital | 28.2 | D | reverse- oblique | 0.172 |
| LP89cap | Loma Prieta | 1989 | 6.9 | Capitola | 14.5 | D | reverse- oblique | 0.443 |
| LP89g03 | Loma Prieta | 1989 | 6.9 | Gilroy Array #3 | 14.4 | D | reverse- oblique | 0.367 |
| LP89g04 | Loma Prieta | 1989 | 6.9 | Gilroy Array #4 | 16.1 | D | reverse- oblique | 0.212 |
| LP89gmr | Loma Prieta | 1989 | 6.9 | Gilroy Array #7 | 24.2 | D | reverse- oblique | 0.226 |
| LP89hch | Loma Prieta | 1989 | 6.9 | Hollister City Hall | 28.2 | D | reverse- oblique | 0.247 |
| LP89hda | Loma Prieta | 1989 | 6.9 | Hollister Differential Array | 25.8 | D | reverse- oblique | 0.279 |
| LP89svl | Loma Prieta | 1989 | 6.9 | Sunnyvale - Colton Ave. | 28.8 | D | reverse- oblique | 0.207 |
| NR94cnp | Northridge | 1994 | 6.7 | Canoga Park - Topanga Can. | 15.8 | D | reverse-slip | 0.420 |
| NR94far | Northridge | 1994 | 6.7 | LA - N Faring Rd. | 23.9 | D | reverse-slip | 0.273 |
| NR94fle | Northridge | 1994 | 6.7 | LA - Fletcher Dr. | 29.5 | D | reverse-slip | 0.240 |
| NR94glp | Northridge | 1994 | 6.7 | Glendale - Las Palmas | 25.4 | D | reverse-slip | 0.206 |
| NR94nya | Northridge | 1994 | 6.7 | La Crescenta-New York | 22.3 | D | reverse-slip | 0.159 |
| NR94stc | Northridge | 1994 | 6.7 | Northridge - 17645 Saticoy St. | 13.3 | D | reverse-slip | 0.368 |
| SF71pel | San Fernando | 1971 | 6.6 | LA - Hollywood Stor Lot | 21.2 | D | reverse-slip | 0.174 |
| SH87icc | Superstition Hills | 1987 | 6.7 | El Centro Imp. Co. Cent | 13.9 | D | strike-slip | 0.258 |
| SH87bra | Superstition Hills | 1987 | 6.7 | Brawley | 18.2 | D | strike-slip | 0.156 |
| SH87icc | Superstition Hills | 1987 | 6.7 | El Centro Imp. Co. Cent | 13.9 | D | strike-slip | 0.358 |
| SH87pls | Superstition Hills | 1987 | 6.7 | Plaster City | 21.0 | D | strike-slip | 0.186 |
| SH87wsm | Superstition Hills | 1987 | 6.7 | Westmorland Fire Station | 13.3 | D | strike-slip | 0.172 |
| ¹ moment mag | gnitude ² closes | st distan | ce to f | ault rupture ³ NEHR | P site class | | | |

Table 1: Selected records for dynamic analysis

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4. MODELING AND SENSITIVITY STUDY

4.1 Modeling assumptions

Nonlinear dynamic analysis was employed to study the connection fracture effects. Brittle and ductile connections are compared using story drifts as response parameter of interest. Due to random nature of connection fracture, sensitivity studies were carried out to make a better understanding of the effect of various parameters on response values. To model the brittle connection, results of Sac Joint Venture experiments on moment connections [21-22] were implemented. For modeling and analysis IDASS [23] program was used. IDASS is a modification of IDARC [24] which is able to model behavior of ductile and brittle connections [25]. Centerline models with no panel zone effect were considered. Every connection is assumed to experience fracture in a pre-defined rotation. For comparison purpose, a ductile bilinear connection behavior was also modeled.

4.2 Sensitivity study

In attempt to investigate the sensitivity of responses to fracture parameters, a base model was defined first. This model is assumed to be the most plausible to experience fracture. To account for sensitivity of drift responses to hysteresis parameters, each parameter is separately changed. To estimate drift demands, the maximum peak story drift angle (θ_{max}) and the average peak story drift angle (θ_{ave}) were used for both 10%/50 and 2%/50 probability levels. In addition, peak story drift angle demands were used to show displacement demand in structure height. The "median" values for drift demands are calculated according to

"median" = exp
$$\left[\frac{1}{n}\sum_{i=1}^{n}\ln(x_i)\right]$$
 (1)

Also 1-sigma level is calculated as

"1-sigma" = "median" exp
$$\left[\sqrt{\frac{1}{n-1}\sum_{i=1}^{n} \left[\ln(x_i) - \ln(\text{"median"})\right]^2}\right]$$
 (2)

5. FRACTURE IN BEAMS

5.1 Base model

It is assumed that 25% of beam connections fracture before reaching plastic moment capacity based on Maison and Bonowitz [26] investigations. For each ground motion, location of connection fractures was selected randomly. The other 75% of connections experience fracture in $\theta_{f+}=0.015$. Hysteresis parameters for the base model are depicted in Table 2. Symbols are shown in Figures 3 and 4.

 Table 2: Base model hysteresis parameters

| Percentage | M_f/M_p | $\theta_f \qquad \theta_{f}$ | M_{red}/M_p |
|------------|-----------|------------------------------|---------------|
| 25% | 0.75 | - ∞ | 0.3 |
| 75% | - | 0.015 ∞ | 0.3 |

The median and 1-sigma levels of θ_{max} and θ_{ave} for the 10%/50 and 2%/50 probability levels are listed in Table 3. In addition, increases in θ_{max} and θ_{ave} from the early fracturing case to the brittle base case are shown in the table. It is evident that connection fracture causes drift demands to considerably increase up to about 35% for 5-story model and 60% for 9-story models.

Table 3: 5-Story Model with ductile and brittle connections

| | | | | 5-story mod | el | 9-story model | | | |
|----------------|-------|---------|----------------|---------------------|-----------------|----------------|---------------------|-----------------|--|
| | | | Ductile (%) | Brittle base (%) | Increase (%) | Ductile (%) | Brittle base (%) | Increase (%) | |
| | 10/50 | Median | 2.11 | 2.58 | 22 | 2.43 | 3.52 | 45 | |
| θ_{max} | 10/30 | 1-sigma | 2.84 | 3.81 | 34 | 3.41 | 4.96 | 45 | |
| | 2/50 | Median | 3.58 | 4.16 | 16 | 3.27 | 5.12 | 57 | |
| | 2/30 | 1-sigma | 4.43 | 5.72 | 29 | 4.48 | 6.97 | 56 | |
| | 10/50 | Median | 1.68 | 1.99 | 18 | 1.76 | 2.38 | 35 | |
| $	heta_{ave}$ | 10/30 | 1-sigma | 2.30 | 2.96 | 29 | 2.47 | 3.32 | 34 | |
| | 2/50 | Median | 2.47 | 2.87 | 16 | 2.37 | 3.63 | 53 | |
| | 2/30 | 1-sigma | 3.58 | 4.25 | 19 | 3.32 | 5.04 | 52 | |





Figure 7 shows that by increase in height the structure is more affected by beam fracture. This increase is more obvious for θ_{max} compared to θ_{ave} , representing the fact that local plastic rotations for brittle connection are increased. Variation of story drifts in brittle cases is more. Moreover, increase in drift demands is more observable for 1-sigma level especially for medial stories.

5.2 Early fracture

To study the effect of early fracture on the structure response, a new model was considered adopted from the base brittle case with the difference that 75% of beam connections are capable to experience early fracture. Location of these connections was selected randomly. Table 4 compares drift demands for these two cases.



Table 4: Comparison of models with different amount of early fracturing connections

It is evident that the number of early fractures has no considerable effect on drift responses. However, for higher probability levels the effect of early fractures is more sensible. It is notable that 9-story model experienced collapse at 2%/50 hazard level which prevented to calculate counted statistics.

5.3 Distribution of early fracture

For the ductile case maximum drift is observed in middle (for 5 and 9-story models) and upper stories (for 5-story model). Thus, to investigate the worst case affecting the drift responses, two other patterns were used including: Early fractures concentrated in upper stories and concentrated in middle stories.

| | | | 5-story model | | | | 9-story model | | | | |
|---------------|-------|---------|-------------------|--------|------------------|--------|-------------------|--------|------------------|--------|--|
| | | | early fracture in | | increase to base | | early fracture in | | increase to base | | |
| | | | upper | middle | upper | middle | upper | middle | upper | middle | |
| $	heta_{max}$ | 10/50 | Median | 2.74 | 2.86 | 6.2 | 10.9 | 3.90 | 3.58 | 10.8 | 1.7 | |
| | 10/30 | 1-sigma | 3.96 | 3.77 | 3.9 | -1.0 | 5.39 | 4.73 | 8.5 | -4.8 | |
| | 2/50 | Median | 4.11 | 3.60 | -1.2 | -13.5 | 5.15 | 4.81 | 0.6 | -6.1 | |
| | | 1-sigma | 5.56 | 5.11 | -2.8 | -10.7 | 6.85 | 6.82 | -1.6 | -2.0 | |
| $	heta_{ave}$ | 10/50 | Median | 1.97 | 2.16 | -1.0 | 8.5 | 2.76 | 2.32 | 16.0 | -2.5 | |
| | 10/30 | 1-sigma | 2.84 | 3.11 | -4.1 | 5.1 | 4.09 | 3.24 | 22.8 | -2.7 | |
| | 2/50 | Median | 3.92 | 2.97 | -14.6 | 3.5 | 3.91 | 3.49 | 7.4 | -4.1 | |
| | 2/50 | 1-sigma | 4.88 | 4.45 | -14.8 | 4.7 | 5.45 | 4.93 | 8.1 | -2.2 | |

Table 5: Comparison of models with different locations of early fracturing connections

By concentrating the early fracturing connections, responses are moderately increased for 5-story building. As expected, for 5-story structure θ_{ave} is increased by locating the early fracture connections to middle stories. This effect is more pronounced for 10% probability level. However, for the case of early fracturing connections located on upper stories, increase in drift responses is not tangible.



For 9-story model structure, increase in drift demands for 10%/50 ground motion is more than 5-story model structure for the case of early Fractures in upper stories. , location of early fracturing connections (in both patterns of early fractures) has a mild effect on demand measures of the 9-story model structure. For the case that early fracturing connections are assigned to the upper stories, increases (relative to the base case) in the median and 1-sigma

5.4 Rotation capacity

level are 10-25% for the 10%/50.

Fracture rotation is changed to study sensitivity of drift demands to fracture rotation values. For this case 0.03 radian rotation is designated as a rational upper limit for fracture rotation.

| | | | 5-story mod | lel | 9-story model | | | |
|---------------|-------|---------|-------------|--------------------|---------------|------|------------------------|----------|
| | | - | base | θ_{f} =0.03 | Increase | base | $\theta_{\rm f}$ =0.03 | Increase |
| $	heta_{ma}$ | 10/50 | Median | 2.58 | 2.36 | -8.5 | 3.52 | 2.62 | -25.6 |
| | | 1-sigma | 3.81 | 3.27 | -14.2 | 4.97 | 3.84 | -22.7 |
| | 2/50 | Median | 4.16 | 3.25 | -21.9 | 5.12 | 3.98 | -22.3 |
| | | 1-sigma | 5.72 | 4.64 | -18.9 | 6.96 | 5.61 | -19.4 |
| $	heta_{ave}$ | 10/50 | Median | 1.99 | 1.92 | -3.5 | 2.38 | 1.98 | -16.8 |
| | | 1-sigma | 2.96 | 2.81 | -5.1 | 3.33 | 2.86 | -14.1 |
| | 2/50 | Median | 2.87 | 2.49 | -16.2 | 3.64 | 2.72 | -25.3 |
| | 2/50 | 1-sigma | 4.25 | 3.35 | -21.2 | 5.04 | 3.98 | -21.0 |

Table 6: Comparison of models with different fracture rotations



Figure 10. peak inter-story drift angles for θ_f =0.03

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Table 6 lists the statistics for these two patterns of fracture rotation. By increase in the fracture rotation, maximum values of drift responses are decreased. It has relatively significant effect on drift statistics as far as drift statistics decreased up to 25%. For the case of 2%/50, the effect of plastic rotation capacity is more visible. Since, more plastic rotations lead to more connection fractures.

5.4 Residual moment capacity

The base case was substituted with a model with residual moment, M_{red} , of 20% of plastic moment, M_p to investigate the effect of residual moment on drift responses. When M_{red}/M_p reduces to 20%, the median and 1-sigma level θ_{max} and θ_{ave} for the 10/50 and 2/50 ground motions in both Model Structure are increased less than 10%. These results indicate that although decrease in *Mred* /*Mp* value increases drift demands as expected, but this effect is not considerable.

| | Table 7: | Compar | ison of m | odels with | n different | residual | moment | capacities |
|--|----------|--------|-----------|------------|-------------|----------|--------|------------|
|--|----------|--------|-----------|------------|-------------|----------|--------|------------|

| | | | _ | 5-story model | | 9-story model | | | |
|----------------|-------|---------|------|--|----------|---------------|---------------------|----------|--|
| | | | base | $\mathbf{M}_{\mathrm{red}} = 0.2\mathbf{M}_{\mathrm{p}}$ | Increase | base | $M_{red} = 0.2 M_p$ | Increase | |
| | 10/50 | Median | 2.58 | 2.78 | 7.8 | 3.52 | 3.76 | 6.8 | |
| θ_{max} | 10/30 | 1-sigma | 3.81 | 3.99 | 4.7 | 4.97 | 5.17 | 4.0 | |
| | 2/50 | Median | 4.16 | 4.10 | -1.4 | 5.12 | 5.56 | 8.6 | |
| | | 1-sigma | 5.72 | 5.85 | 2.3 | 6.96 | 7.56 | 8.6 | |
| $	heta_{ave}$ | 10/50 | Median | 1.99 | 2.05 | 3.0 | 2.38 | 2.52 | 5.9 | |
| | | 1-sigma | 2.96 | 2.84 | -4.1 | 3.33 | 3.61 | 8.4 | |
| | 2/50 | Median | 2.87 | 2.98 | 3.8 | 3.64 | 3.90 | 7.1 | |
| | 2/30 | 1-sigma | 4.25 | 4.18 | -1.6 | 5.04 | 5.43 | 7.7 | |





6. FRACTURE IN COLUMNS

To model column fracture, fracture was allowed to propagate from beam flange to column underneath. Consequently the adjacent beam does not fracture itself. In each structure, it was assumed that columns have 25% probability to experience fracture. Residual moment capacity of columns was taken as 20% of plastic moment capacity.

| | | 5-story mod | | | | | del 9-story model | | | | | |
|---------------|-------|-------------|------|--------------------|-----|------------------------------------|-------------------|----------------------|-----|------------------------------------|--|--|
| | | | Mr | M _{red} = | | Increase for M _{red} = | | $\mathbf{M}_{red} =$ | | Increase for M _{red} = | | |
| | | | 0.2 | 0.1 | 0.2 | 0.1 | 0.2 | 0.1 | 0.2 | 0.1 | | |
| $	heta_{max}$ | 10/50 | Median | 2.22 | 2.27 | 5 | 2 | 2.33 | 2.23 | -4 | -4 | | |
| | | 1-sigma | 3.11 | 3.24 | 10 | 4 | 3.12 | 2.94 | -9 | -6 | | |
| | 2/50 | Median | 3.19 | 3.42 | -11 | 7 | 3.43 | 3.30 | 5 | -4 | | |
| | | 1-sigma | 4.67 | 5.26 | 5 | 13 | 5.07 | 4.98 | 13 | -2 | | |
| $	heta_{ave}$ | 10/50 | Median | 1.69 | 1.75 | 1 | 4 | 1.69 | 1.61 | -4 | -5 | | |
| | 10/30 | 1-sigma | 2.33 | 2.42 | 1 | 4 | 2.28 | 2.12 | -8 | -7 | | |
| | 2/50 | Median | 2.45 | 2.54 | -1 | 4 | 2.44 | 2.33 | 3 | -5 | | |
| | | 1-sigma | 3.68 | 3.84 | 3 | 4 | 3.62 | 3.43 | 9 | -5 | | |

Table 8: 5-Story Model with different residual moment capacities



As can be seen from table 8, column fracture is not important for mid-rise structures. For 5-story structure, increase in drift demands is more observable for the case with lower residual moment. But, this is limited to 13%. For 9-story structure the similar conclusion can be drawn with a difference that the case of $M_{red}=0.1$ has less effect compared to the case of $M_{red}=0.2$.

7. CONCLUSIONS

Base on calculations the following conclusions can be drawn:

- A considerable increase in drift demands is observed for the base case compared to the ductile connection case which is more pronounced for the 9-story structure.
- Increase in possibility of early fracture in connections to 75% resulted in drift demands

to increase less than 15%.

- When early fracture connections positioned in upper stories, increase in drift demands is more. The maximum increment in drift statistics is 23% beyond the base model.
- Change in fracture rotation to 0.03 radian led to decrease in drift demands in all of model structures. However this is limited to less than about 20%.
- The largest change in drift statistics was 13% for column fracture which confirms that the effect of column fracture in seismic drift response is not significant.
- In general, hysteresis parameters of fracture are not clearly known. However, these parameters do not have an important role on drift responses.

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