Seismic Evaluation of Steel Moment Resisting Frame Buildings with Different Hysteresis and Stiffness Models

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ABSTRACT :

Current seismic design procedures that apply to an estimation of inelastic deformation capacity of lateral force resisting systems have been questioned since no rationality exists for determining the values of R tabulated in seismic design code. For this study, 3-, 9- and 20-story Steel Moment Resisting Frame (MRF) buildings were designed to satisfy the seismic requirements based on the IBC 2000 including the current value of 8 for the steel special moment resisting frame (MRF) buildings. Then, these analysis building models were redesigned using 6 different hysteresis models, which provide an ability to dissipate seismic input energy, for the beam-column connections. These models were also extended to account for the effects of period of the buildings. A total of 90 different building models were subjected to 20 ground motions representing a hazard level of 2% probability of being exceeded in 50 years to estimate the seismic demands. Pushover and nonlinear time history analysis were performed to calculate story drift and plastic rotation demands. The effects of hysteresis models and various periods of the steel special MRF are investigated and discussed.

KEYWORDS: Steel moment resisting frame, Hysteresis models, Nonlinear time history analysis, Response modification factor, Displacement ductility demand

1. INTORDUCTION

Steel MRF is widely used at the place located in a high seismic hazard area. Many researchers have assumed that steel MRF buildings are ductile structural systems to resist earthquake forces by allowing their connections and members to have inelastic flexural deformation. As a result, MRF structural systems are believed to possess large ductility capacity and thus are designed for smaller loads. Analytical models of such frames are often developed using a line element based on centerline dimensions of beams and columns. A finite dimension of a joint is modeled by including rigid eccentricities at the ends of beam-column element to account for the effect of the geometry of the joint. The joints are usually assumed rigid, in which beam-column elements framing into the joints remain at right angles even after the joints have experienced large inelastic cycles of deformation. However, the use of rigid connections may not properly represent the strength and stiffness of the structural frame as well as the story drift and the overall deflection of the structure. Results from such models may overestimate ductility capacity. For example, instead of the ductile behavior that was expected by structural engineers, a widespread occurrence of brittle fractures was observed in recent earthquakes (1994 Northridge, U.S. and 1995 Kobe, Japan) in welded beam-to-column connections in steel buildings resulting in major strength and stiffness degradation of structural frames. Therefore, in order to evaluate the seismic performance of steel MRF buildings, it is important to consider detailed joint connection models capable of simulating the real joint behavior as close as possible. [1],[2]

This paper provides information on the seismic response of steel MRF buildings applied to various beam-column models to study the effects of various hysteretic behaviors including a bilinear connection model, stiffness degradation as well as strength. Additionally, the analytical models with the different hysteresis models were also extended to the models having five different fundamental periods. The results from the static pushover and nonlinear time history analyses were evaluated and discussed.

2. DESIGN OF BUILDING MODELS

Figure 1 shows plan and elevation for 3-, 9- and 20-story buildings. All buildings were designed for a Los Angeles area, CA, U.S.A. Site Class D defined in the 2000 IBC provisions (IBC 2000) as a stiff soil was assumed to be soil conditions.[4] In order to resist earthquake forces, the perimeter frames for all models were designed to be a main lateral force resisting systems. Due to the symmetry of the structural plan and relatively small amount of gravity loads acting on the perimeter frames, only the N-S direction moment frames were considered in this paper. The response modification factor of 8 was assigned for steel special MRF buildings to consider inelastic deformation. As shown in Table 1, 3- and 9-story buildings were designed using drift limit

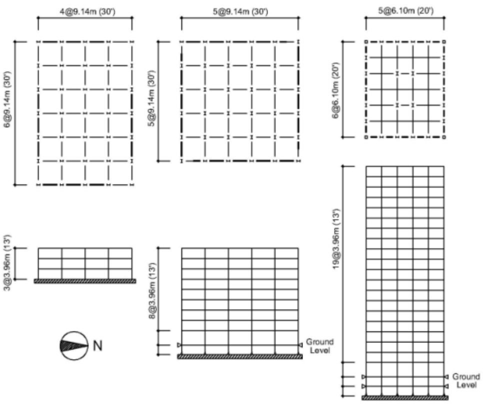


Figure 1 Plan and elevation views of 3-,9- and 20-story buildings

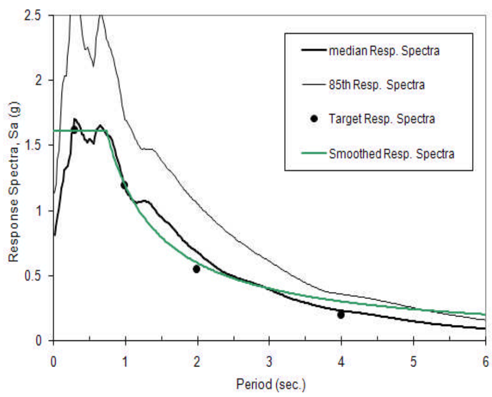
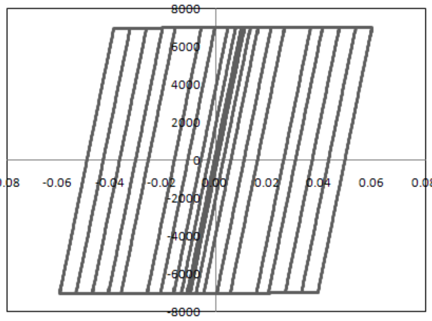
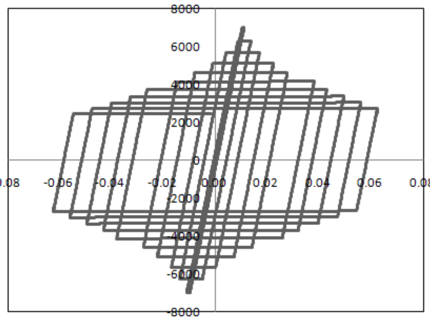


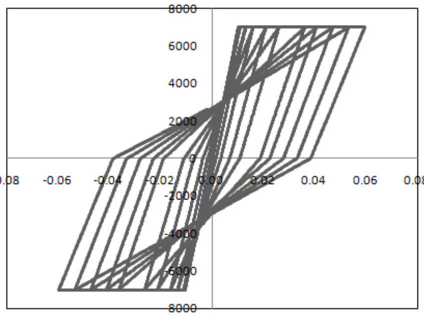
Figure 2. Response Spectra for Los Angeles 2/50 Hazard Level



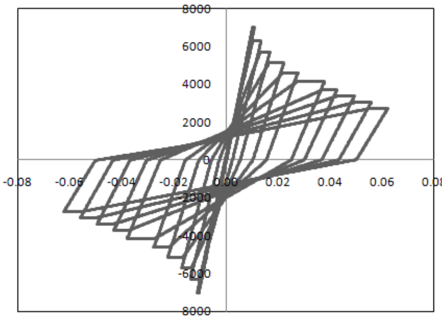
(a) Type A: Bilinear model



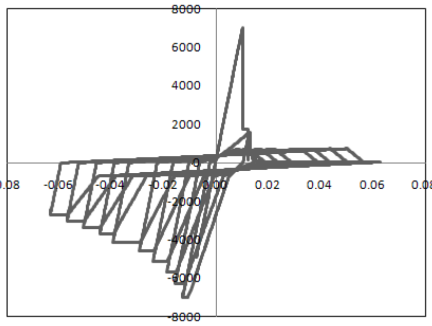
(b) Type B: Bilinear & Strength Degradation model



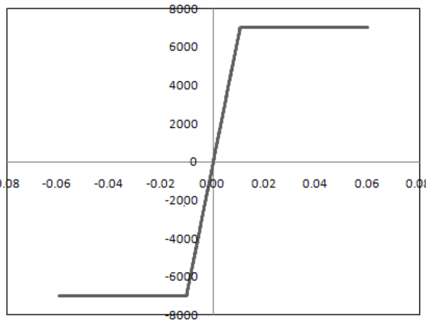
(c) Type C: Modified Takada-Sozen model



(d) Type D: Modified Takada-Sozen model & Strength degradation model



(e) Type E: Brittle connection model



(f) Type F: Hypothetical model

Figure 3 Hysteretic connection models

to comply with requirement for the 2000 IBC provisions. The detailed information for the building models is available in elsewhere. (1)

Twenty ground accelerations, which have 2% probability of exceedance in 50 years, developed by Somervile et al. (1997) were used for this research. Figure 2 shows the median and 84th percentile values of twenty elastic response spectrum of the Los Angeles, CA. Each response spectrum was developed for using 5% damping ratio (5).

3. HYSTERETIC MODELS APPLIED TO THE RESEARCH

Six hysteresis models providing energy dissipation capacity to the beam-column connection models were considered into this study to evaluate the seismic performance of the structures. Each hysteretic connection was developed by using the connection element (Type 10) in Drain-2DX program. (7), (8)

Figure 3 shows six hysteretic models considered in this study. In Figure 3(a), Type A represents a bilinear model defined with two-linear lines representing a range of elastic and plastic after yielding of the connection. This model is not considered for any degradation of strength and stiffness. Although this connection model has been widely used to develop the analysis models, energy dissipation capacity of this model is likely to

Table 1. Periods calculated for the 3-, 9- and 20-story buildings

Building	Ta	T1	T2	**T3**	T4	T5
3-Story	0.52	0.66	0.76	**0.86**	0.96	1.06
9-Story	1.19	1.90	2.10	**2.30**	2.50	2.70
20-Story	2.61	3.01	3.21	**3.41**	3.61	3.81

overestimate the behaviors of real structures. Type B adds a degradation of strength from the Type A connection model. This model is representative of a steel beam with local flange buckling. Type C is to represent modified model suggested by Takada-Sozen (1980) to simulate a hysteresis effect of reinforced concrete (RC) members subjected to cyclic loadings.[9] This model represents a relationship with a degradation of stiffness as the inelastic deformation develops. As shown in the Figure 3 (d), Type D considers the modified Takada-sozen model representing the degradation for strength as well as stiffness. Type E illustrates a brittle behavior observed from past earthquakes for steel connection in Northridge and Kobe earthquakes. Type F is a connection model having no energy dissipation capacity, which is a hypothetical model.
In addition to the hysteresis models of the beam-column connections, the effects of fundamental periods of the model buildings are considered. The fundamental period plays an important role to affect dynamic characteristic for buildings. In this paper, Equation (1), proposed in ASCE7-02, was employed to determine fundamental period (T_a) of the buildings and Drain-2DX computer program was used to calculate the period for buildings.

$$T_a = C_t \times h_n^x \tag{1}$$

The periods of actual structure, calculated by computer analysis program, are named as T_3. In order to consider various dynamic characteristics controlled by the stiffness of the systems, the building models with 4 different periods were designed by adopting different stiffness, except for the periods for original buildings. For instance, the models for the 3-story building with 4 different periods, calculated as the period of 0.86 sec. from the Equation (1), which is calculated from the Drain-2DX program, were additionally developed from 0.66 sec. to 1.06 sec. with regular interval as 0.1 sec. In the Table 2, fundamental periods and the periods considered in this study are shown.

4. ANALYSIS FOR BUILDING MODELS

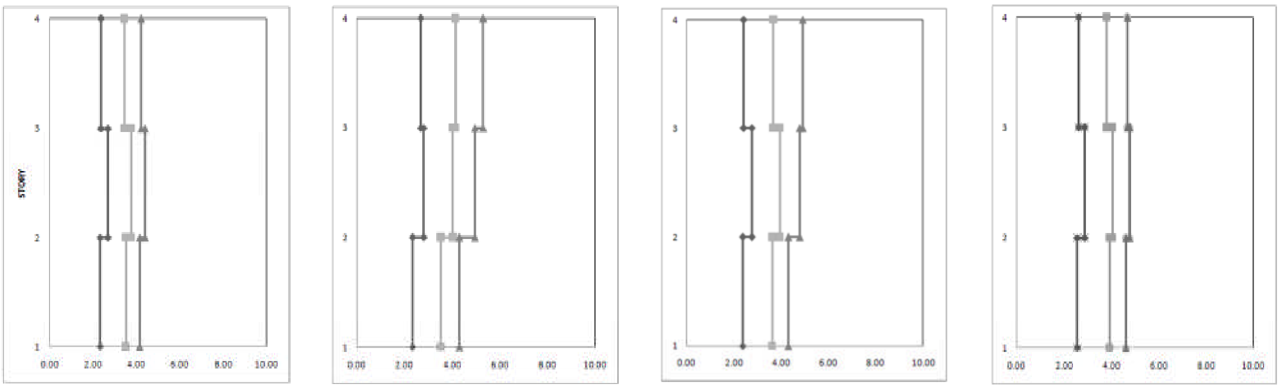
A total of 90 analytical computer models accounting for six different hysteretic models and five different periods for each model were analyzed with push-over analysis and non-linear time history analysis. All computer analysis models were developed using the Drain-2DX program. In order to calculate and evaluate an accurate behavior for steel MRF building, the panel-zone model developed by Krawinkler et al [3]. was considered in this study. Since post-yield stiffness and strength was additionally provided in column flanges after yielding for column web, two rotational springs was added to simulate a tri-linear behavior. In order to simulate as closely as possible with actual behavior for frames, geometric nonlinearity was considered for the static pushover and the nonlinear time history analyses for the P-delta effects. For the P-delta effects acting on the deformed frame configuration, an additional bay consisting of rigid connection links and pinned-end columns was included. [2~4]

5. DYNAMIC ANALYSIS

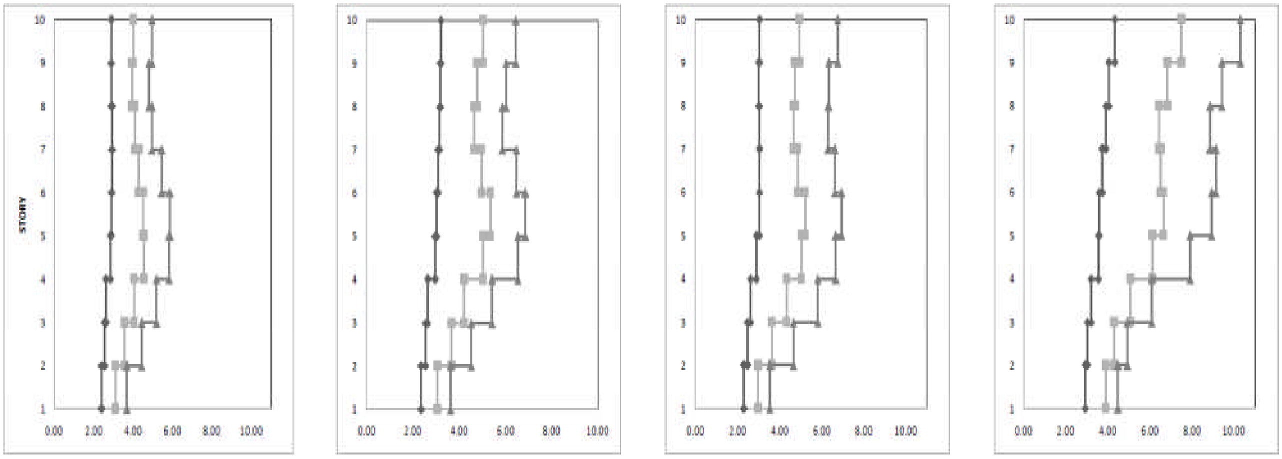
The 90 models were analyzed using twenty ground motions, which represents the earthquakes for a 2/50 hazard level. Figure 4 shows the maximum story drift responses for the median, 84th and 95th percentile values for Type A, Type D, Type E and Type F. Due to the page limit, all results of the 6 Type connections were not shown here. Figure 5 shows displacement ductility demand to evaluate ductility capacity for the structures.

5.1. Drift demand

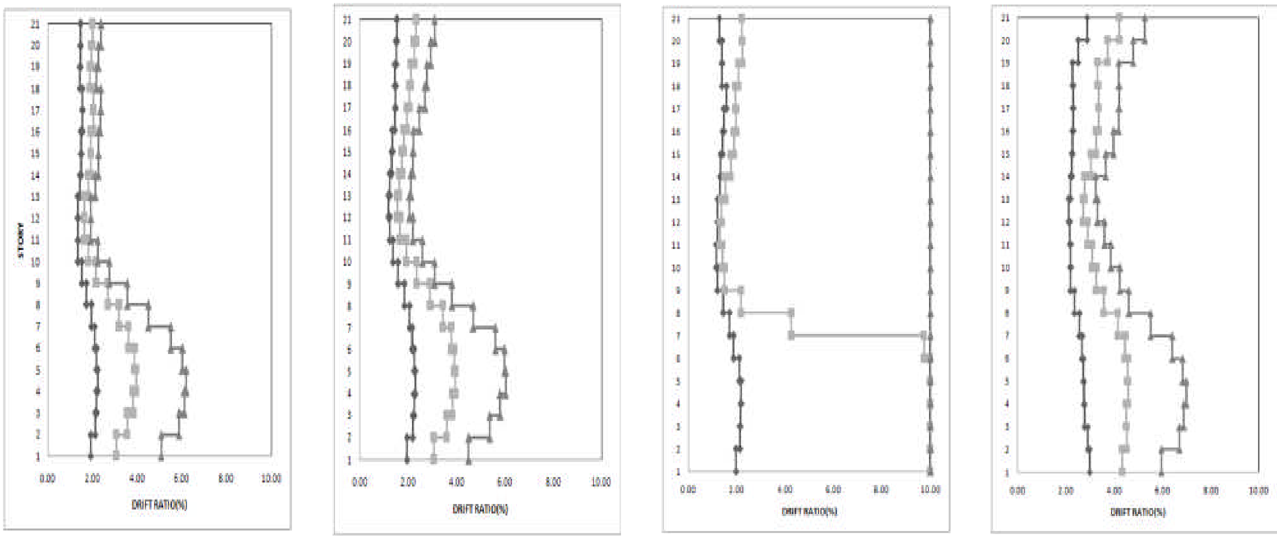
Type A Type D Type E Type F



(a) 3-story Buildings with Type A, Type D, Type E and Type F



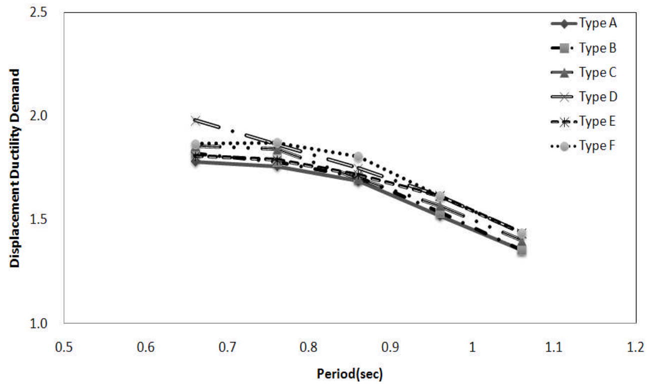
(b) 9-story Buildings with Type A, Type D, Type E and Type F



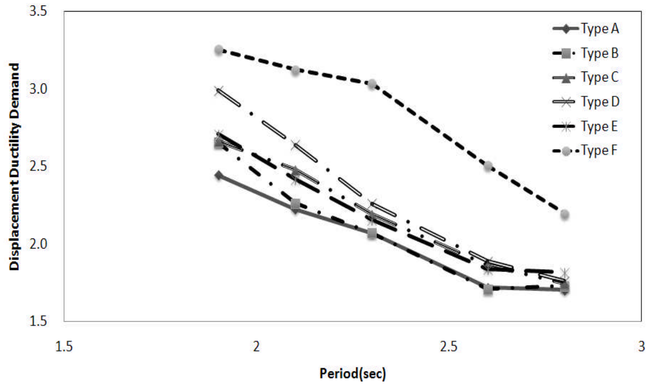
(c) 20-story Buildings with Type A, Type D, Type E and Type F

Figure 4 Drift demands for 3-, 9- and 20- story Buildings for 2/50 hazard level

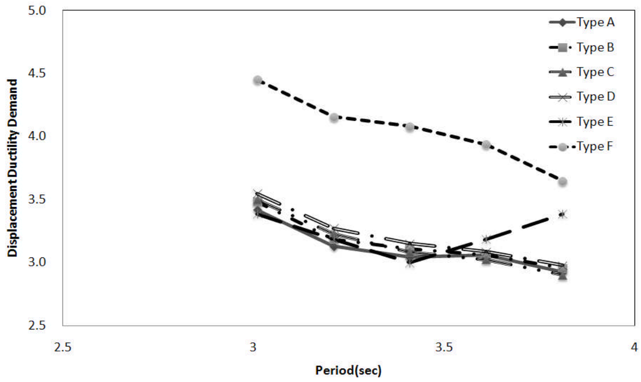
As shown in Figure 4, 3-story buildings subjected to the 20 L.A ground motions with 2/50 hazard level show a stable behavior showing less than a value of 3% for the median drift demand. For the 3-story buildings with a fundamental period of 0.86 sec., each maximum story drift ratio, based on the energy dissipation capacity for Type A, Type D, Type E and Type F, are calculated to 2.70, 2.80, 2.75 and 2.89%, respectively. These models show the difference for each drift demand of 1.81 % between Type A (Bilinear connection) and Type E (Brittle connection) and the difference for drift demand is more than 45% when considering the effect of periods.



(a) The 3-story Building



(b) The 9-story Building



(c) The 20-story Building

Figure 5 Displacement ductility demand for analytical models for 2/50 hazard level

The 9-story buildings' drift ratio for Type A, Type D, Type E and Type F that have a period for 2.30 sec. are calculated to 2.94%, 3.21%, 3.06% and 4.31%, respectively. The difference for drift demand is 3.21% and 3.92% respectively when comparing 9-story building modeled with Type A with Type D and Type E. It is shown that the period effects are more sensitive than effects for energy dissipation capacity and, thus, resulted in larger drift demands for the 9-story building. 20-story buildings designed with Type D and Type E connection models were estimated to be collapsed for 8 models. Especially, all of the 20-story models with Type E were collapsed. For the 20-story buildings, large story drifts were concentrated in the lower middle story levels due to P-delta effects indicating structural instability develops as the inelastic deformation increase. As shown in Figure 4, the largest story drifts occur in lower story between 3rd and 5th story. When the 20-story building with the Type A and Type B connections are considered, the difference in terms of the drift demands is to be 3.5%. In order to compare the effects for increasing period, the Type A, Type D and Type E were considered with respect to each period. For period effects, the difference for all buildings is investigated to be more than 20 %.

5.2 Displacement ductility demand

The ductility capacity for structures is affected to the fundamental period, response modification factor including energy dissipation (R-factor) and structural systems. Displacement ductility demand (D.D.D) is generally employed to investigate ductility capacity with respect to structural systems. Figure 5 shows displacement ductility demand with respect to all structures applied in this study. In order to determine Displacement ductility demand, the equation (2) was employed.

$$\mu = \frac{\Delta(t)_{\max}}{\Delta_y} \tag{2}$$

Where $\Delta(t)_{\max}$ is a result on maximum value of the story drift demand calculated by nonlinear time history analysis and Δ_y is a yielding displacement determined from push-over analysis. The structures with a short period were calculated to be larger displacement ductility demand than the structures with a long period. Generally, displacement ductility demand tends to be small when increasing period for each story building (e.g. stiffness is decreased). That can be well illustrated as “Short Period Effect” that represents the earthquake input energy cannot be dissipated efficiently under the high frequency of structural movement because the structures with a short period are much stiffer. [(12), (13)] Generally, the displacement ductility demand for structure due to effective energy dissipation is declined when periods of the structures are elongated. However, the displacement ductility demand with the Type D, which considered degradation for stiffness and strength, was estimated to be larger than the results of the model with the Type E even though energy dissipation capacity for the Type D connection model was smaller than the Type E connection. The analytical models applied to the Type D connection model were evaluated to be sensitive for results of time history analysis due to effects for strength and stiffness degradation. Since the 20-story building used the Type E representing the brittle connection behavior, the displacement ductility demand from 3.41 sec. is enhanced as period increased. The results for the Type F, which was hypothetical connection model, shows the largest values for displacement ductility demand since this Type doesn’t have an ability to dissipate seismic input energy.

6. CONCLUSION

In this paper, 3-story, 9-story and 20-story steel MRF buildings applied to 6 hysteretic models with 5 periods were analyzed for 20 ground motions having a 2% probability of exceedance in 50 years. A total of 90 models designed based on the 2000 IBC provisions were investigated for push-over and non-linear time history analysis to evaluate dynamic behavior and calculate displacement ductility demand. The results obtained from this study were summarized as follows.

1. Maximum drift ratio of the 20-story building having a 3.41 sec. for the fundamental period with the Type A was calculated as values of 2.22 %, 3.94 % and 6.19 % with median, 84th and 95th percentile, respectively. The building with Type E was collapsed due to the P-delta effect occurred in the lower story. These results show that the drift demands for existing buildings, which were modeled with the Type A (Bilinear) connection, is generally overestimated due to the absence of the simple and large hysteresis area.
2. The results show that the dynamic structural behaviors of the buildings from the period effects were more sensitive than energy dissipation effects to the analytical models when various hysteretic connections’ effects and period effects are considered.
3. Since the structures designed using large stiffness was not effectively dissipated for energy, the structures were relatively evaluated to be large displacement ductility demand when evaluating ductility capacity. The buildings with Type A connections, which overestimate the energy dissipation capacity of beam-column connections, shows the smallest displacement ductility demand while Type F models showed for the largest one.

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ACKNOWLEDGEMENTS

This research was supported by Seoul R&BD Program (grant no. GR070033) funded by City of Seoul and by the Basic Research Program of the Korea Science & Engineering Foundation (Grant No. R01-2007-000-20765-0). The authors would like to express sincere gratitude for this supports.