SEISMIC BEHAVIOR AND EVALUATION OF STEEL SMF BUILDINGS WITH VERTICAL IRREGULARITIES

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ABSTRACT:

This paper concentrates on investigating the seismic behaviors of vertically irregular steel special moment frame (SMF) buildings by comparison with the regular counterpart. All buildings of this study were assumed to locate in Los Angeles and subjected to 20 earthquake ground motions with a seismic hazard level of 2% probability of exceedance in 50 years. These 20-story buildings were designed to conform to the requirements for steel SMFs as specified by IBC 2000 provisions, and the beam-column connections of the buildings were modeled to consider the panel zone deformation. Also, a ductile connection model accompanied by strength degradation was incorporated to the analysis program in an effort to obtain more accurate response results. Three types of the irregularities (mass, stiffness and strength irregularity) specified as vertical irregularities in the IBC 2000 provision were imposed to the original building. Nonlinear static and dynamic analyses were performed, and the confidence levels of which the performance objective will be satisfied were calculated as well. The effects of different irregularity types and levels on the seismic behaviors of the buildings were investigated and discussed in terms of the height-wise distribution of story drifts, maximum story drift demands, global collapse story drift capacities and confidence levels.

KEYWORDS: steel special moment frame, vertical irregularity, story drift demand, story drift capacity, seismic hazard level, confidence level.

1.INTRODUCTION

Special moment frame buildings are expected to withstand significant inelastic deformation when subjected to the forces from the motions of design earthquake (AISC, 2002). In reality, many steel SMF buildings are vertically irregular due to requirements of commercial or residential reasons. The definitions for such vertically irregular buildings were prescribed in International Building Code (IBC) 2000. Past experiences from the earthquake show that the seismic behavior of vertically irregular buildings can be significantly different in comparison to the regular counterparts.

In the late 1990's, there were research interests in the area of study for vertically irregular buildings. Duan and Chandler (1995) studied seismic behavior of setback buildings and pointed out that both static and modal spectral analysis were inadequate to predict and prevent damage concentration in members near the setback level. Chatpan Chitanapakdee and Anil K. Chopra (2004) conducted a study to compare the seismic demands for vertically irregular and regular frame buildings as determined by nonlinear response history analysis (RHA) and pointed out that introducing a soft and/or weak story increased the story drift demands on the modified and neighboring stories and decreased the drift demand in other stories. Additionally, they investigated the accuracy of the modal pushover analysis (MPA) as proposed in the study of Chopra and Goel (2002) for the estimation of seismic demands for vertical irregular frames. Michalis, Dimitrios and Manolis (2006) evaluated the influence of vertical irregularities on a 9-story steel frame based on incremental dynamic analysis (IDA). They concluded the proposed methodology enabled a full-range performance evaluation, using the highly accurate analysis method that pinpointed the effect of any source of irregularity for each limit-state.

Recently, research on vertically irregular buildings has grown with the purpose of better understanding of the behavior of such buildings. This research was performed for the same purpose. The seismic behavior of regular steel SMF buildings and of vertical irregularities were investigated and evaluated. The results from analyses of regular and irregular buildings were compared and discussed in terms of story drifts and confidence levels.

2. BUILDING MODELS

Plan and elevation configurations for 20-story buildings are shown in Fig. 1. All the buildings were assumed to locate in a Los Angeles site with stiff soil condition defined as Site Class D in the 2000 IBC provisions. W24 sections were used for the exterior perimeter columns, and box shape steel columns were used for the corner columns. The member sections and the properties of materials used in the buildings can be found in the other study (Lee and Foutch, 2002).

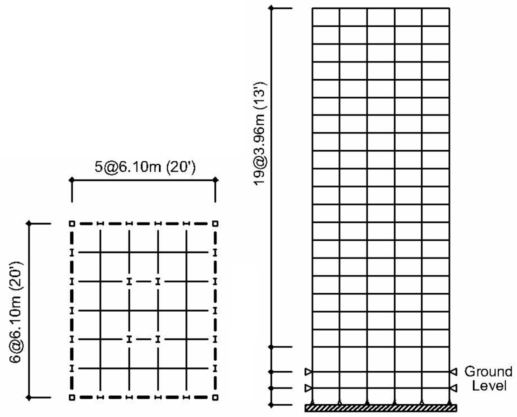


Fig. 1. Plan and elevation views of 20-story buildings

Reduced beam section (RBS) connections were used for the building models. This connection model built by reducing the beam flange area was intended to form plastic hinges away from connections. The connection model was developed using a tri-linear element to capture key attributes of test results such as strength degradation and Bauschinger effect. The parameters for this connection were achieved from matching the hysteresis behaviors from the analysis and from the test by Venti and Engelhardt (1999). Panel zone modeling (Yun and Foutch, 2000) was used to get more accurate results by considering panel zone deformations of the joints.

The vertical irregularity buildings were imposed by changing the irregular quantities (i.e. story mass, stiffness or strength) at different positions from the regular counterparts. In this study, it was assumed that the irregularities present at stories 1, 1-3, 9-11, 18-20 or 20. The levels of 150% and 200%, 70% and 50%, 80% and 50% for mass, stiffness and strength irregularities were used, respectively. The levels of irregularities here are the percentages of irregular quantities at the positions of consideration to that of the upper or lower story.

After changing the irregular quantities, the buildings were uniformly scaled to keep the same fundamental periods and design base shear strengths, which were determined from the regular counterparts for the purpose of the suitable comparison between the regular and the irregular buildings. Vertical irregularities considered in this study are shown in Fig.2. In the figure, the notation MO, KO and SO imply the model for the original (regular) building. The remaining notations of the irregular buildings include three parts referring to the types, levels and positions of the irregularities. For an example, MI-200-1-3 notation refers to the model of the building with mass irregularity at level of 200% existing at stories 1-3 (i.e. masses at stories 1-3 are as twice as that of the above adjacent story).

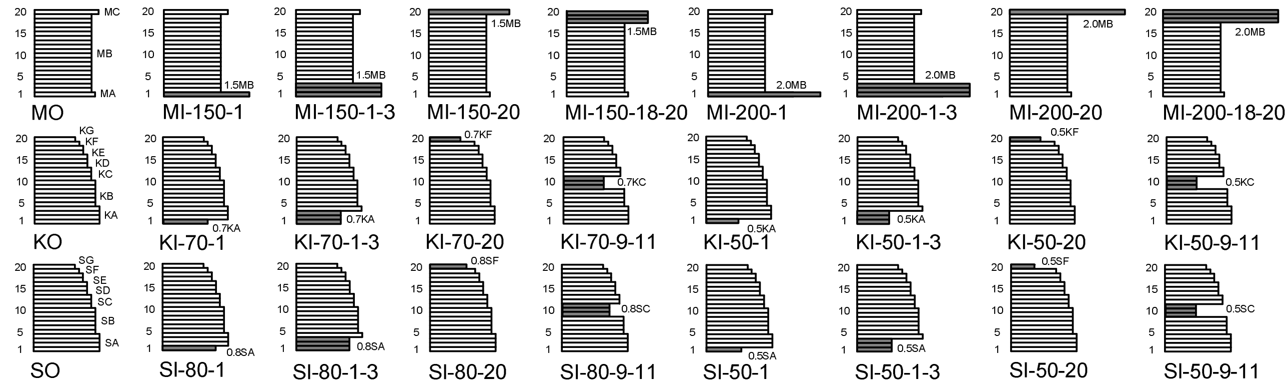


Fig. 2. Models of 20-story buildings

3.NON-LINEAR STATIC PUSHOVER ANALYSIS

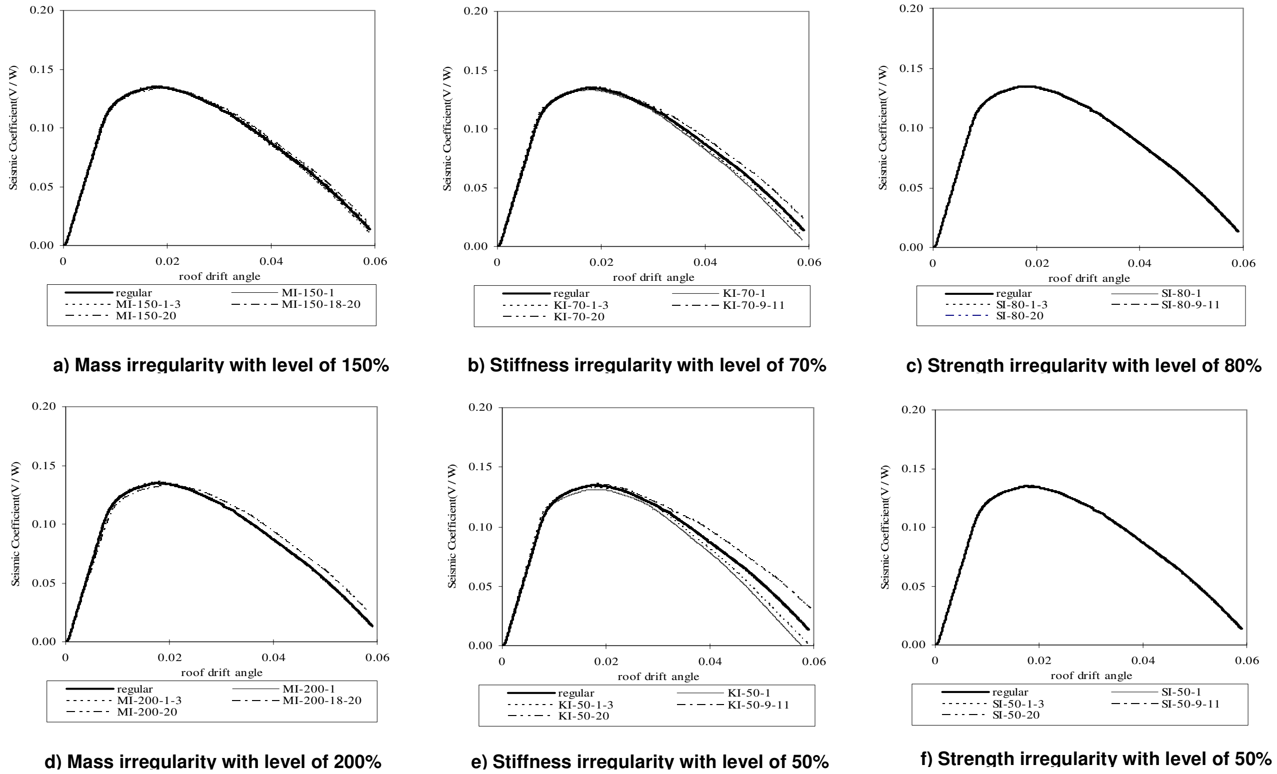
The nonlinear static pushover analyses are useful for the evaluation of the capacity of a structure experiencing substantial inelastic deformation (Lee and Foutch, 2002). The distribution of the lateral force applied to the buildings followed IBC 2000 provisions. The nonlinear analysis was performed until the control displacement (i.e. the roof displacement in this study) reached its target value. The results from the nonlinear pushover analyses as determined by Drain-2DX program for all the buildings with the base shear coefficient (i.e. the base shear divided by the building weight) and the roof drift angle relationship are shown in Fig.3.

Obviously, the initial stiffness and the maximum value of the pushover curves for all the buildings are similar to one another due to the same fundamental periods and design base shear strengths. The post-yield behaviors of the irregular buildings were also similar to that of regular counterparts, except for the post-yield behavior of strength irregular buildings. The slopes of strength degradation curve of the strength irregular buildings decreased when the level of strength irregularities increased.

4.STORY DRIFT DEMANDS

A total of 20 maximum considered earthquake (MCE) ground motions with hazard level of 2% probability of exceedance in 50 years at a Los Angeles site were applied to all the buildings. The basis of these ground motions was provided by Somerville, et. al. (1997). A structural nonlinear analysis program, Drain-2DX, was utilized for the analyses. Fig. 4 shows the comparison of maximum story drifts of irregular buildings and the original counterpart. Base on this figure, the effects of three types of irregularity, i.e. mass, stiffness and strength, on the story drift demands are discussed below.

The higher mass in the top stories caused larger story drift ratio in those stories and some lower adjacent stories in comparison with the regular building. At the same time, it decreased the story drift ratio at remaining lower stories. This impact became more evident as the level of mass irregularity increased. When mass irregularity existed at some top stories (i.e. for the cases of MI-150-18-20 and MI-200-18-20), the story drifts at all stories were smaller than those at corresponding stories of the irregular buildings with the irregularity placed at the bottom story (cases MI-150-1 and MI-200-1). However, for almost all cases above, the maximum story drift demands were not much different from the original one because they usually occurred at bottom stories. The maximum story drift demand significantly increased and changed its position to the top story only for the case of MI-200-20. When higher masses were placed at bottom stories, the story drifts for all stories were found to be larger than those of original one. As the result, the maximum story drift usually increased. The effects were stronger for a higher irregular level and for lower bottom stories when the mass irregularity was applied to them. The largest drift ratio was found for the case MI-200-1-3 with the drift value of 0.03. Obviously, in this case, the higher story mass level of 150%, which is prescribed in the IBC 2000 provisions to distinguish the regular and irregular structures, did not induce a considerable influence on the maximum story drifts of the buildings.



a) Mass irregularity with level of 150% b) Stiffness irregularity with level of 70% c) Strength irregularity with level of 80%

d) Mass irregularity with level of 200% e) Stiffness irregularity with level of 50% f) Strength irregularity with level of 50%

Fig.3. Nonlinear static pushover analysis for all buildings

The lower stiffness at a story caused larger story drifts at that story and some adjacent stories and lower story drifts at remaining stories. Effects of the stiffness irregularity at the top story of the buildings on the maximum story drifts were not significant. The irregular effects were stronger when the irregularities were presented at the first and the middle stories of the buildings. For the case lower stiffness placed in stories 9-11, the drift at these stories considerably increased as much as 130% and 155% for the irregular level of 70% and 50%, respectively, compared to the original one. Simultaneously, the story drifts at top and bottom stories decreased. This is the reason that the maximum drifts were found to be smaller than their counterparts of the original one. The buildings with lower stiffness applied at bottom stories exhibited the most severe behavior among the stiffness irregular cases. In these buildings, the drifts at bottom stories increased, and concurrently the drifts at top stories decreased. As the result, the maximum story drifts became larger. The more severe the level of irregularity was applied, the stronger the effect was. This effect was stronger for the case of irregularities placed at stories 1-3 than that for the case of irregularities placed at first story only. The largest maximum story drift was found to be 0.028 radian (an increase of 20% compared to the original one) for the case of KI-50-1-3.

The presence of strength irregularities at the top story of the buildings was not significantly influential in the story drifts of the buildings for irregularity levels of both 80% and 50%. As for the cases of strength irregular level of 80% at bottom stories, the maximum story drift slightly increased as much as 118% and 120% for the cases SI-80-1-3 and SI-80-1, respectively. Although the drifts at middle stories increased 125% for the case SI-80-9-11, the maximum story drift was still smaller than that of the original one because it occurred at bottom stories. As for the strength irregularity level of 50%, the irregularity effects were the strongest in all cases considered. The drifts at irregular stories increased 155%, 140% and 240% for the cases of SI-50-1, SI-50-1-3 and SI-50-9-11, respectively. The corresponding maximum story drifts were 0.032, 0.031 and 0.032 radian.

5.STORY DRIFT CAPACITIES

This study defines the drift capacity of a structure subjected to an earthquake ground motion as the maximum story drift at the limit state of the structure. The limit state considered is the collapse prevention (CP) performance level as described in FEMA 350, in which the structures is on the verge of experiencing partial or total collapse. This state implies that the structure is substantially damaged and experiences significant degradation in stiffness and strength. The drift capacity associated with global stability of the structure is determined by Incremental Dynamic Analysis (IDA) procedure, which is detailed in FEMA350, in this study.

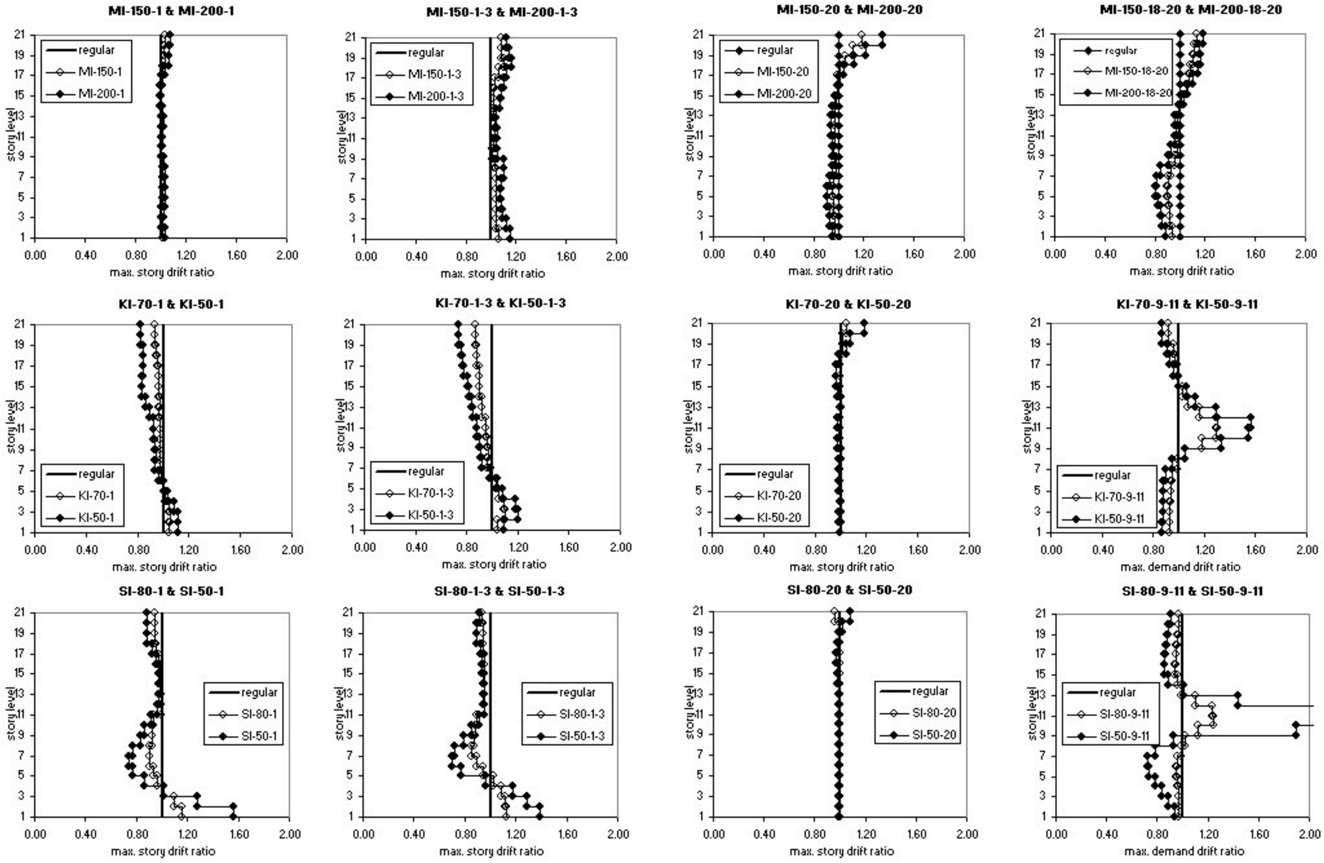


Fig.4. Maximum story drifts of all buildings relatively comparing to that of the original building

The IDA procedure includes steps of nonlinear dynamic analysis of the model structures subjected to a series of scaled ground motions. The IDA curves present the relationship between ground motion intensity and the maximum story drifts obtained from the steps of the analysis This study sets the ground motion intensity to be 5% damping elastic acceleration spectrum at the fundamental period of the building (S_aT_1) under consideration. The analyses were carried out until the slope of the IDA curve became less than 20% of its elastic slope or until the maximum of story drift exceeded the story drift value of 0.10. In the former case, the maximum story drift at the last step was considered the global collapse drift, whereas in the later case the global collapse drift was assumed to be 0.1. Fig.5. shows the median story drift demand as well as the global collapse story drifts for all buildings calculated from the IDA procedure. The story drift capacities of all buildings under the investigation of this study did not change much compared to the original one. Ten out of twenty four buildings manifested drift capacity discrepancies of less than 5%. Eight out of twenty four buildings exhibited discrepancies of 5 to 10%, and the remaining six out of twenty four buildings showed discrepancies of 10 to 15%. This maybe due to the matching the fundamental periods and design base shear strengths of all buildings under the investigation of this study.

6. PERFORMANCE EVALUATION USING CONFIDENCE LEVEL CALCULATION

Prediction of seismic response of a new or existing structure is complex due to not only the large number of factors that need to be considered and the complexity of seismic response but also the large inherent uncertainty associated with making these predictions (Foutch, 2000).

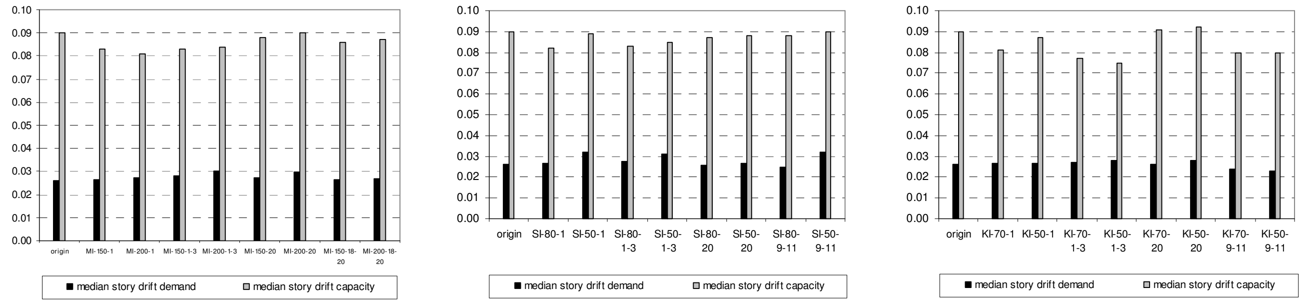


Fig.5. Median story drift demands and median story drift capacities for all buildings

A major issue in the evaluation of design process is the proper treatment and incorporation of the randomness and uncertainty in both seismic loading and building resistance. A reliability-based probabilistic approach was developed by Jalayer and Cornell (1998) to solve this problem by evaluating the degree of confidence in satisfying the performance objective of the building. For a new design, the performance objective is Collapse Prevention (CP) for the seismic hazard which has only 2% probability of being exceeded in 50 years (2/50 hazard).
The confidence level can be obtained from any probability book based on the standard Gaussian variate associated with probability x of not being exceeded, K_x, which is calculated as follows:

$$K_x = \left[\ln(\lambda_{con}) + \frac{1}{2} \cdot k \cdot \beta_{UT}^2 \right] \cdot \frac{1}{\beta_{UT}} \tag{6.1}$$

where λ_{con} = confidence factor, β_{UT} = total uncertainties in the capacity and demand but not randomness; k = slope of the hazard curve. The confidence factor can be evaluated from the equation below:

$$\lambda_{con} = \frac{\phi C}{\gamma \gamma_a D} \tag{6.2}$$

where D = estimate of median drift demand determined from NTHA; C = estimate of median drift capacity determined from IDA procedure; ϕ = resistance factor; γ = demand factor; γ_a = analysis demand factor.

The calculation of confidence level of all the buildings is presented in Table 1. Although the drifts at top stories considerably increased for the buildings with mass irregularities placed at the top stories, the maximum story drifts for the whole buildings were not much larger than those of reference building. Additionally, the drift capacities also did not significantly decrease. Thus, the confidence level did not change markedly.
The confidence levels of the buildings with mass irregularity level of 150% existing in the bottom stories were not smaller than 90%, which is considered as the required value for the design of new buildings under seismic loading. This indicates that these buildings possess adequate confidence level to satisfy the performance objective. For the buildings with irregularity level of 200%, the confidence levels were less than 90%. Based on these results, the authors suggest that the mass irregularity level of 150% need to be considered only for some bottom stories of buildings. For other parts of buildings, the mass irregularity level to distinguish the regular buildings from irregular buildings may be larger.

Both cases of the stiffness irregularities imposed at the top story exhibited confidence levels of 96%, slightly larger than that of the original one. Although the drift capacities of buildings with stiffness irregularities at middle stories considerably decreased to 0.080 (reduction of 11% compared to the original one), the confidence levels were still as high as 97% and 93% for buildings KI-50-9-11 and KI-70-9-11, respectively. It is because the stiffness irregularities increased middle story drifts and simultaneously reduced the drifts at the bottom stories and brought about the reduction of the maximum drift of the whole buildings under investigation. This result implies that the limit of 70% prescribed in IBC 2000 provision for soft story was too conservative for these cases. The stiffness irregularities located at bottom stories caused stronger effects on the confidence levels of the buildings. For the buildings with irregularities at first story only (i.e. KI-70-1 and KI-50-1), the confidence levels decreased slightly to 91% and 93%. The confidence levels for the cases of KI-70-1-3 and KI-

50-1-3 were 88% and 84%, respectively. It is believed that the number of soft stories significantly affected the behavior of the buildings. This point should be carefully considered for the definition of irregular buildings. Again, the limit of 70% was also too conservative for the cases in which the stiffness irregularities were located at one story only.

Table 1 Calculation of confidence levels for all buildings.

	C	β_r	β_u	ϕ	D	β_{acc}	β_{ori}	γ	β_{anal}	γ_a	γ	β_{UT}^2	γ_{con}	K_x	C. L.
origin	0.090	0.138	0.43	0.75	0.0260	0.44	0.26	1.46	0.25	1.09	1.59	0.25	1.62	1.69	95%
MI-150-1	0.083	0.182	0.43	0.73	0.0264	0.44	0.26	1.46	0.25	1.09	1.60	0.25	1.44	1.45	93%
MI-200-1	0.081	0.230	0.43	0.71	0.0273	0.44	0.26	1.45	0.25	1.09	1.58	0.25	1.34	1.29	90%
MI-150-1-3	0.083	0.201	0.43	0.72	0.0280	0.45	0.26	1.47	0.25	1.09	1.60	0.25	1.34	1.30	90%
MI-200-1-3	0.084	0.191	0.43	0.73	0.0300	0.45	0.26	1.47	0.25	1.09	1.61	0.25	1.27	1.19	88%
MI-150-20	0.088	0.170	0.43	0.74	0.0273	0.40	0.26	1.38	0.25	1.09	1.51	0.25	1.57	1.62	95%
MI-200-20	0.090	0.140	0.43	0.75	0.0296	0.35	0.26	1.31	0.25	1.09	1.44	0.25	1.58	1.63	95%
MI-150-18-20	0.086	0.166	0.43	0.74	0.0263	0.39	0.26	1.36	0.25	1.09	1.49	0.25	1.62	1.68	95%
MI-200-18-20	0.087	0.170	0.43	0.74	0.0267	0.36	0.26	1.33	0.25	1.09	1.46	0.25	1.65	1.72	96%
KI-70-1	0.081	0.196	0.43	0.73	0.0265	0.45	0.26	1.47	0.25	1.09	1.61	0.25	1.38	1.36	91%
KI-50-1	0.087	0.152	0.43	0.74	0.0267	0.46	0.26	1.50	0.25	1.09	1.64	0.25	1.48	1.50	93%
KI-70-1-3	0.077	0.217	0.43	0.72	0.0269	0.46	0.26	1.48	0.25	1.09	1.62	0.25	1.27	1.19	88%
KI-50-1-3	0.075	0.258	0.43	0.70	0.0281	0.46	0.26	1.49	0.25	1.09	1.63	0.25	1.15	0.98	84%
KI-70-20	0.091	0.114	0.43	0.75	0.0262	0.44	0.26	1.45	0.25	1.09	1.59	0.25	1.65	1.72	96%
KI-50-20	0.092	0.122	0.43	0.75	0.0278	0.40	0.26	1.39	0.25	1.09	1.52	0.25	1.64	1.71	96%
KI-70-9-11	0.080	0.232	0.43	0.71	0.0239	0.45	0.26	1.47	0.25	1.09	1.61	0.25	1.48	1.50	93%
KI-50-9-11	0.080	0.230	0.43	0.71	0.0230	0.34	0.26	1.30	0.25	1.09	1.42	0.25	1.74	1.83	97%
SI-80-1	0.082	0.215	0.43	0.72	0.0266	0.48	0.26	1.53	0.25	1.09	1.67	0.25	1.32	1.27	90%
SI-50-1	0.089	0.155	0.43	0.74	0.0322	0.47	0.26	1.50	0.25	1.09	1.64	0.25	1.25	1.16	88%
SI-80-1-3	0.083	0.202	0.43	0.72	0.0276	0.49	0.26	1.54	0.25	1.09	1.69	0.25	1.29	1.22	89%
SI-50-1-3	0.085	0.198	0.43	0.73	0.0311	0.55	0.26	1.68	0.25	1.09	1.84	0.25	1.08	0.86	81%
SI-80-20	0.087	0.148	0.43	0.74	0.0256	0.47	0.26	1.51	0.25	1.09	1.65	0.25	1.53	1.57	94%
SI-50-20	0.088	0.150	0.43	0.74	0.0267	0.43	0.26	1.43	0.25	1.09	1.57	0.25	1.56	1.61	95%
SI-80-9-11	0.088	0.145	0.43	0.74	0.0248	0.47	0.26	1.51	0.25	1.09	1.65	0.25	1.60	1.65	95%
SI-50-9-11	0.090	0.165	0.43	0.74	0.0319	0.47	0.26	1.51	0.25	1.09	1.65	0.25	1.26	1.18	88%

The confidence levels of the buildings with lower strength imposed at top story (SI-80-20 and SI-50-20) did not vary markedly in comparison to that of the original building. They were 94% and 95%, respectively. This indicates that the lower strength at top story was not remarkably influential to the confidence levels of the buildings. It means that the strength irregularity should not be considered for the top story. The confidence levels of the buildings with 50% strength irregularity level located at the middle and bottom stories were smaller than 90%. They were 88%, 81% and 88% for the buildings SI-50-1, SI-50-1-3 and SI-50-9-11, respectively. With strength irregular level of 80%, they were larger than 90%, except for the building SI-80-1-3 with the confidence level of 89%. This result demonstrates the rationality for the strength limit of 80% as prescribed in the IBC 2000 provision. Repeatedly, the number of irregular stories considerably affected the confidence levels of the buildings.

7. CONCLUSIONS

A total of 24 irregular 20-story buildings and their original counterpart building were investigated by nonlinear static and dynamic analyses. The pushover curves, the height-wise distribution of story drifts, the maximum story drift demands, the global collapse story drift capacities, the confidence levels were estimated in comparison to those of original buildings and collated with the seismic provisions. Some conclusions are drawn as follows:

1. Although vertical irregularities are placed in the buildings of this investigation, their pushover curves were not much different from that of the original one, except for the cases of strength irregularities, which had the negative slopes considerably decreased as the irregular level increased. This may be due to matching the first mode periods and base shear strengths of the irregular buildings and of the regular one.

2. Through the investigation in this study, it indicates that the weaker and softer top story (i.e in the cases of SI-50-20, SI-80-20, KI-50-20 or KI-70-20) is not necessary for the consideration of vertical irregularities; the presence of higher mass at top stories needs to be considered for the mass irregularity. It causes larger story drift at the top stories and may induce larger maximum story drift of the whole buildings under investigation. The mass irregularities at bottom stories also caused lower confidence levels for the buildings under this

investigation. However, the mass irregularities maybe considered with the higher limit value as prescribed in the seismic provision.

3. For the buildings when stiffness irregularities are placed in only one story, the limit of 70% in the seismic provision may be too conservative, even for the case the irregularity existed in bottom stories. In the most severe case, KI-70-1, the maximum story drift was 0.0265 (as 2.0% larger than that obtained from the original one). The confidence level of this building was also high at the value of 91%.

4. The strength irregularities affected the seismic behavior of irregular buildings the most. Only 20% reduction in the strength of the story may cause a significant decrease in the confidence level of the building. It indicates that the limitation of 80% for strength irregularity as stipulated in IBC 2000 prevision is appropriate.

5. In this study, the behavior of the buildings with the irregularities placed in three stories are often more severe than those of the buildings with the irregularities existed in only one story in terms of maximum story drift demands and confidence levels. Thus, the number of vertically irregular stories is also significantly influential to the confidence levels of the buildings and that needs to be considered carefully.

6. The maximum drift usually occurred at bottom stories and was much larger than the drifts at other stories for the buildings under the investigation of this study. Therefore, it is deemed that the vertical irregularities placed at bottom stories caused more serious effect than those placed at the other locations of the buildings. Because of this reason, the limit values to distinguish regular buildings from irregular buildings may vary in accordance with vertical positions of irregularities.

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REFERENCES

1. American Institute of Steel Construction (AISC, 2002). Seismic Provisions for Structural Steel Buildings. Chicago, Illinois.
2. Chatpan Chintanapakdee and Anil K. Chopra (2004). Seismic response of vertically irregular frames: Response history and modal pushover analyses. *Journal of construction engineering*. ASCE, 130:8.
3. Chopra AK, Goel GK (2002). A modal push over analysis procedure for estimating seismic demands for buildings. *Earthq Eng Struct Dyn.* 31, 561–582.
4. Duan XN, ChandlerAM (1995). Seismic torsional response and design procedures for a class of setback frame buildings. *Earthq Eng Struct Dyn.* 24, 761–777.
5. Federal Emergency Management Agency (2000). Recommended seismic design criteria for new steel moment-frame buildings. *FEMA 350*, Washington, D.C.
6. Foutch, D. A. (2000). State of Art Report on Performance Prediction and Evaluation of Moment-Resisting Steel Frame Structures. *SAC Report No. FEMA 355f.*, Federal Emergency Management Agency.
7. Fragiadakis Michalis, Vamvatsikos Dimitrios and Papadrakakis Manolis (2006). Evaluation of the influence of vertical irregularities on the seismic performance of a nine-story steel frame. *Earthquake engineering and structural dynamics*. John Wiley & Sons, Ltd.
8. International Code Council (2000). International Building Code. Falls Church, Va.
9. Jalayer, F. and Cornell, A. (1998). Development of a Probability-Based Demand and Capacity Factor Design Seismic Format. *Draft 8/11/98, Publication as a SAC Background Document Pending*.
10. Lee K., Foutch DA. (2002). Performance Evaluation of New Steel Frame Buildings for Seismic Loads *Earthquake Engr.eff & Structural Dynamics*, 31:3, 653 - 670.
11. Somerville, P., N. Smith, S. Puntamurthula, and J. Sun (1997). Development of Ground Motion Time Histories for Phase 2 of the FEMA/SAC Steel Project. *Background Document, Report No. SAC/BD-97/04, October 1997.*
12. Venti, M., and Engelhardt, M. D. (1999). Brief Report of Steel Moment Connection Test, Specimen DBBW (Dog Bone – Bolted Web). *Internal SAC phase 2 report.*
13. Yun, S.Y. and Foutch, D.A. (2002). Modeling of steel moment frames for seismic loads. *Journal of Constructional Steel Research* 58, 529–564.