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DELIBERATING THE EFFECT OF ROCKING MOTION AND PLACEMENT OF BRACE FRAMES IN IMPROVING SEISMIC PERFORMANCE OF STEEL BRACED BUILDINGS

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Many buildings need to quickly return to their primary function after an earthquake. The need to implement the functionality recovery process or the resiliency of earthquake-rocking motion buildings after severe ground shaking is now at the forefront of structural engineering challenges and will undoubtedly become parts of codes of practice worldwide in the near future. The structural design of earthquake- ground motion structures demands greater analytic and detailing effort than those for code compliance only. The purpose of this study is to compare performance of mid height steel braced rocking buildings with low rise buildings. For this purpose, the seismic behavior of 3, 10 and 15 story buildings and effect of proper location of brace frames in 3-story buildings are studied under far-field earthquakes records. Nonlinear dynamic analyses are conducted at two maximum PGA levels of 0.35 g and 0.7 g. The three-dimensional models studied are based on the model referred to in the NSCL-Report series 026 [Hall et al., 2010], which is derived from the SAC's administrative design office building. However, this model was an unsymmetric building but in this research the building is modified to a symmetric building also the number of floors has been increased, then the buildings are modeled (SAP2000).

For time history analyses, seven far-field records have been used. Records are selected from appendix A of the FEMA-p695 report. In this research, the earthquake records are applied only in one direction and horizontally. The response spectrum of the records listed in Table 1 is shown in Figure 1. As shown in Figure 2, the buildings in each direction have two rocking braced frames, which the brace arrangement is different in each model. Besides, the height has been increased in model 1. Response parameters including performance level, maximum inter-story drift, vertical and horizontal acceleration, the amount of uplift displacement of the columns base are compared. Horizontal acceleration of the models at two maximum PGA levels of 0.35 g and 0.7 g is shown in Figure 3.

Table 1. Records used in time history analysis (FEMA-P695 report).

Record Number	Earthquake	PGA(g)
EQ1	Northridge, USA	0.52
EQ2	Duzce, Turkey	0.82
EQ3	Imperial Valley, USA	0.38
EQ4	Kobe, Japan	0.51
EQ5	Kocaeli,Turkey	0.36
EQ6	Loma Prieta, USA	0.56
EQ7	Manjil, Iran	0.52

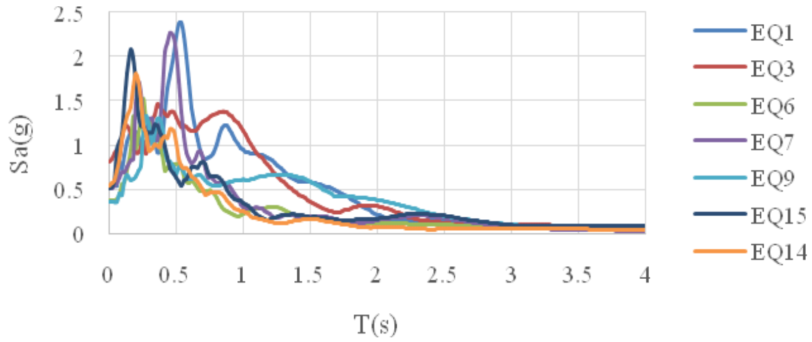


Figure 1. Response spectrum of records.

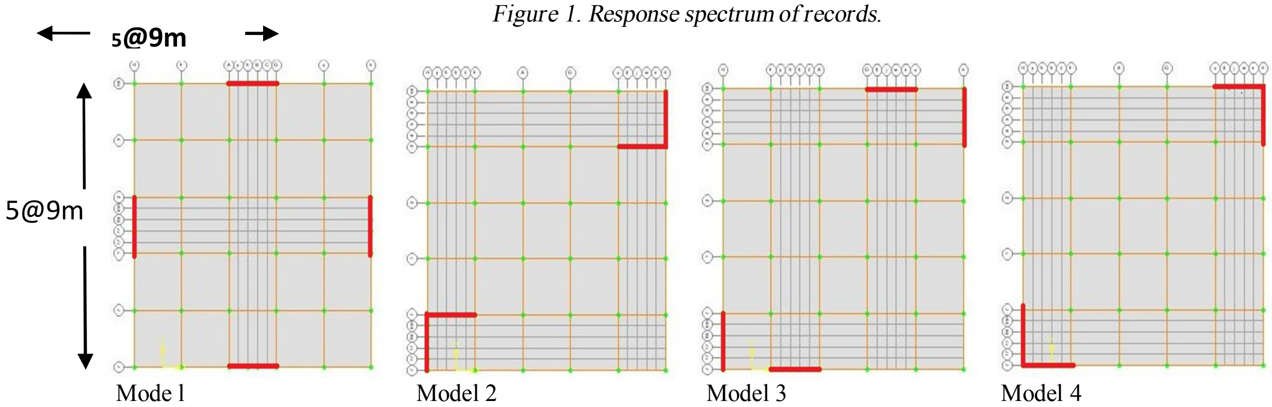
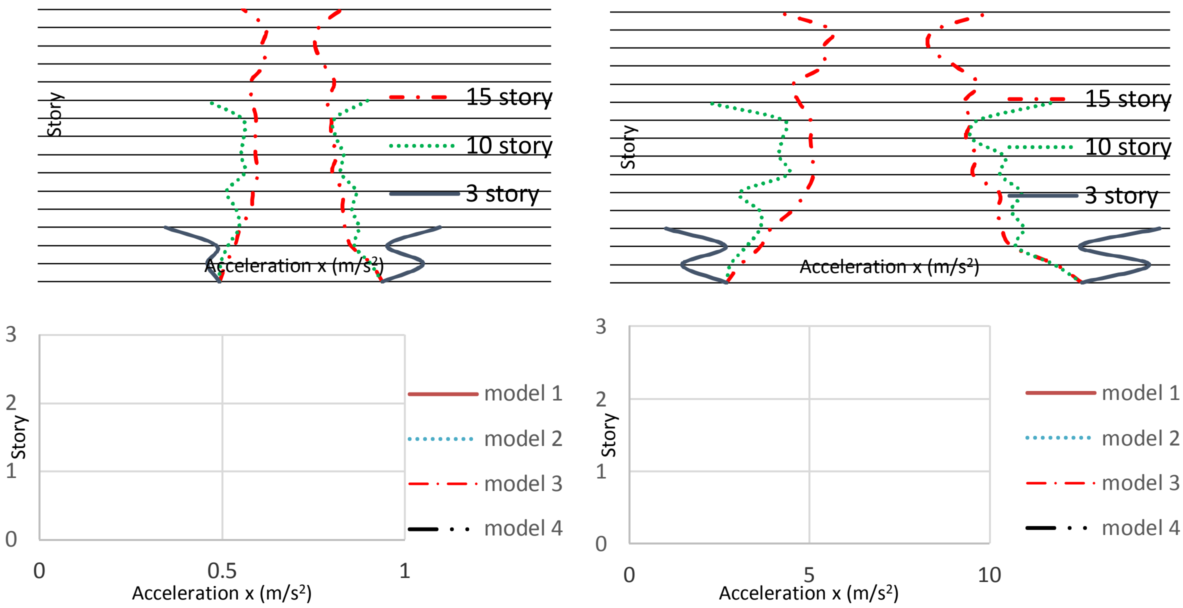


Figure 2. Plan of 3D studied models.



a) PGA equal to 0.35g   b) PGA equal to 0.7g

Figure 3. Horizontal acceleration in models subjected to earthquake No. 7 (Manjil).

The results indicate that all the responses are improved as the building height increases. Besides, by moving the locations of bracing to the corners and inside the building, the responses are more favorable.

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