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ANALYSIS OF SHAKE-TABLE TESTS OF A FULL-SCALE BUILDING ISOLATED BY LEAD-RUBBER BEARINGS

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ABSTRACT

This paper presents numerical simulation of a full-scale five-story building base-isolated by a system of lead-rubber bearings (LRBs) and cross-liner bearings (CLBs). The structure was subjected to a number of bidirectional and bidirectional-plus-vertical ground motions using the E-Defense shake table. A three-dimensional nonlinear model of the structure was established and subjected to ground motion measured on the shake table. The behavior of LRBs, response of the building, the effect of vertical loads, and the effectiveness of the LRB-CLB isolation system are discussed.

Keywords: base isolation, lead-rubber bearing, shake-table test, numerical simulation, seismic response.

1. INTRODUCTION OF THE TEST SPECIMEN

In August 2011, a full-scale, base-isolated steel moment-frame building was tested using the E-Defense shake table [1]. As shown in Fig. 1(a), the two-by-two bay, five-story building was 10-m and 12-m wide in the X and Y-directions, respectively, and 15.8 m tall. The building had a total weight of 542 tons. The 54-ton steel plates on the roof introduced a mass eccentricity ratio of 3% (the distance between the mass center and geometric center divided by the floor-plan dimension) in the Y-direction of the isolation layer. As shown in Fig. 1(b), the isolation

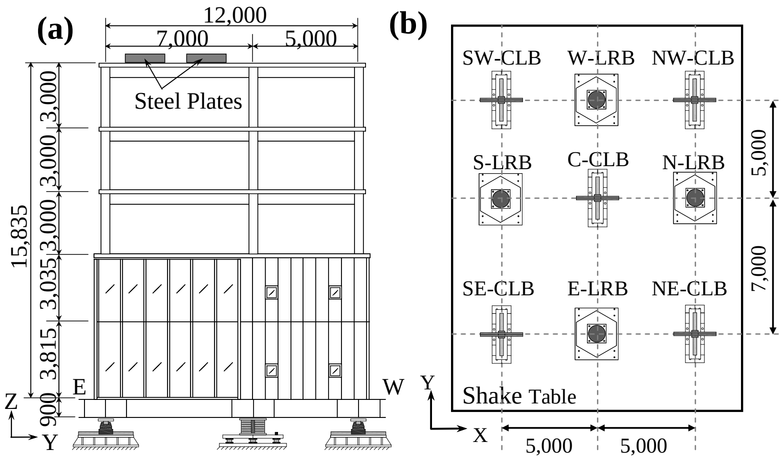

Figure 1. Outline of the building: (a) North elevation; (b) Location of bearings. (Unit: mm)

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system comprised four lead-rubber bearings (LRBs) and five cross-liner bearings (CLBs) between the shake table and the building. The LRBs were placed under the four edge columns, and the CLBs were placed under the four corner columns and the center column.

2. MODEL OF THE STRUCTURE

A 3-D nonlinear model of the structure was produced. The model included all columns and girders of the building and each LRB and CLB. Members of the superstructure were modeled as linear elastic. An end zone was added at every girder-to-column joint to consider the stiffness of the joints. To simulate a rigid floor diaphragm, the relative horizontal distances between the nodes in each floor were fixed. Composite beam effect was accounted for by multiplying the flexural stiffness by 2 for girders with concrete slab at two sides, and multiplying by 1.5 for girders with concrete slab on one side only. Masses were lumped to the girder-to-column joints according to the gravity load distribution of the building and the mass moment of inertia of each floor.

Fig. 2 shows the dimensions of the LRB. A simple bilinear model, as shown in Fig. 3(a), was used for the horizontal response. Based on the parameters provided by the manufacturer, the initial stiffness K_0 was 6.5 kN/mm, the post-yielding stiffness K_F was 0.65 kN/mm, and the yield strength F_U was 73 kN. The vertical response of the LRBs was modeled by a bilinear model, as shown in Fig. 3(b), with a compressive stiffness K_C of 1500 kN/mm and tensile stiffness K_T of 100 kN/mm. The critical compressive stress σ_{cr} of LRB was 35.4 MPa. As shown in Fig. 4, the CLBs comprised two linear bearings oriented orthogonally to each other. The CLBs had negligible horizontal stiffness and very large vertical stiffness compared to the LRBs, and were modeled as bidirectional rollers.

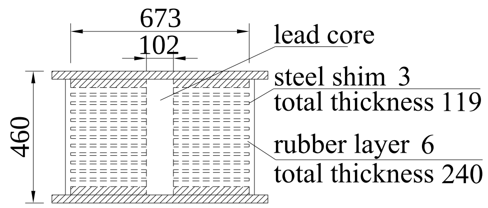


Figure 2. LRB Dimensions. (in mm)

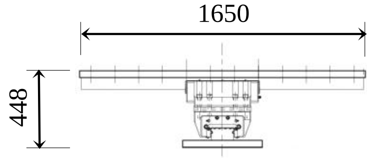


Figure 4. CLB Dimensions. (in mm)

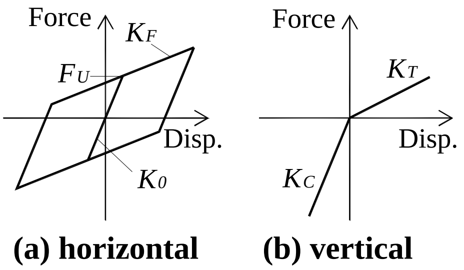


Figure 3. Force-deformation relationship of LRB.

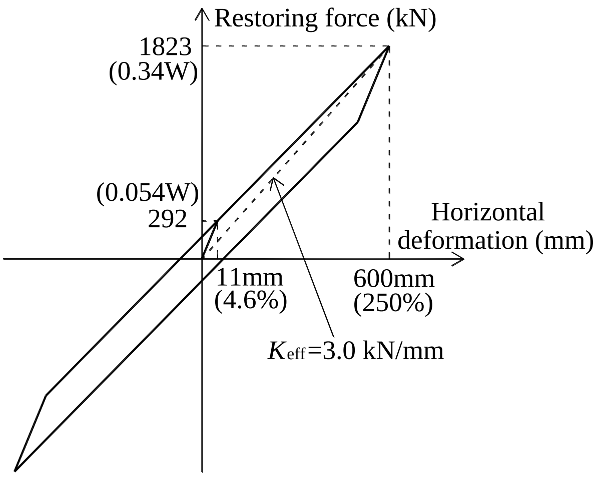


Figure 5. Force vs. deformation relationship of LRB-CLB isolation system.

In the isolation layer, the horizontal restoring force was derived from the four LRBs only.

Fig. 5 shows the uni-directional force versus displacement relationship of the LRB-CLB isolation system. The bracketed values in the ordinate are the restoring force normalized by the total weight of the building, W = 5420 kN. The bracketed values in the abscissa are the horizontal strain assuming that the four LRBs deform equally. The isolation system provided a yield strength of V/W=0.055 and a strength of V/W=0.34 at 250% strain, which is the design limit specified for the specimen. From the formula, $T = 2\pi(M/K_{\mathrm{eff}})^{0.5}$ [2], where K_{eff} =3.0 kN/mm is the effective stiffness at 250% strain, and the effective period is evaluated as T_{eff} = 2.66 s, which is four times the fundamental period of the building when its base was fixed, 0.7 s.

Rayleigh damping was specified by assigning 2% critical damping for the period 0.7 s and 3% for the period of 2.66 s. Analysis was performed with a time increment of 0.01s.

Recorded acceleration of the shake table was used as ground motion. The analysis and test response

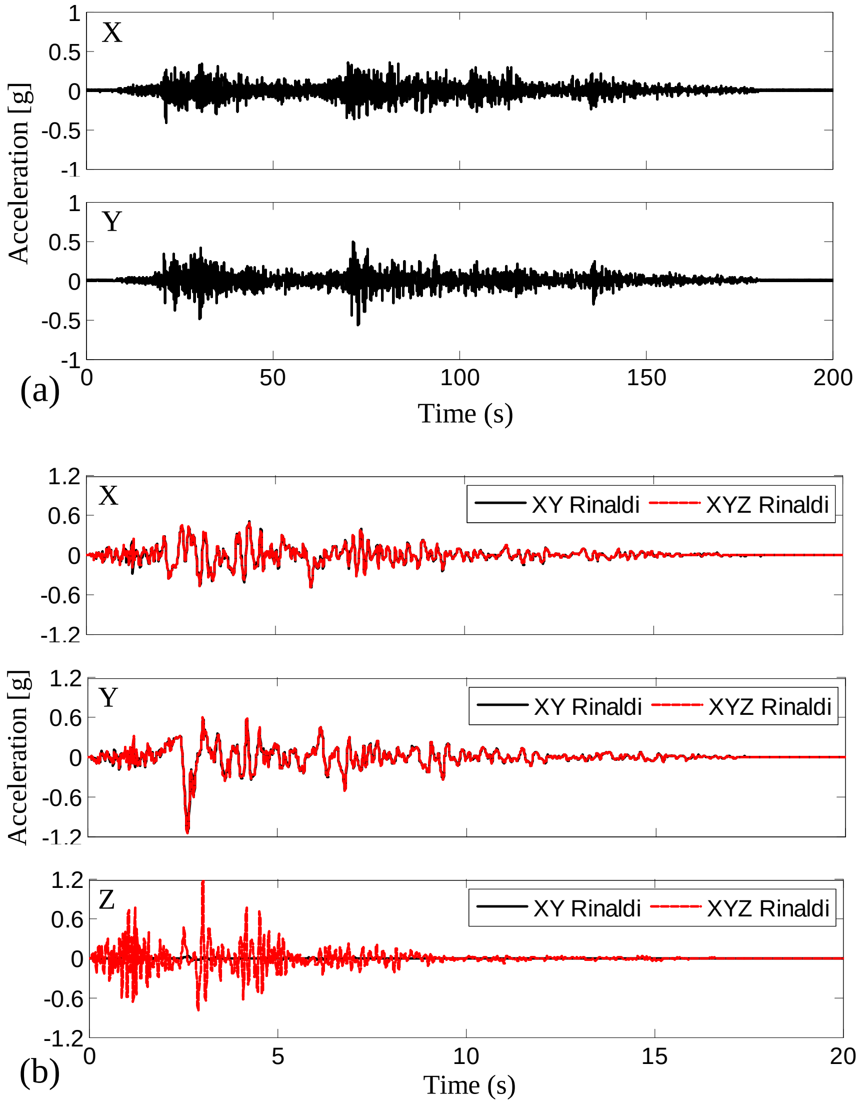


Figure 6. Input motions: (a) 100%-XY Iwanuma; (b) 88%-XY Rinaldi and 88%-XYZ Rinaldi.

of three excitations that were produced from two motions, Iwanuma and Rinaldi, are discussed in this paper. The Iwanuma motion, obtained from the 2011 Tohoku earthquake, was a long duration motion. The Rinaldi motion, obtained from the 1994 Northridge earthquake, featured very large vertical acceleration. Fig. 6 shows the time history of the three excitations, 100%-XY Iwanuma, 88%-XY Rinaldi and 88%-XYZ Rinaldi. The two excitations, 88%-XY Rinaldi and 88%-XYZ Rinaldi, had the same X and Y acceleration, but the former did not include vertical acceleration while the latter did.

3. VALIDATION OF ANALYSIS

The fundamental period of the building with its base fixed was 0.69 s in the analysis, which is very close to the experimental value of 0.7 s identified through white noise excitation.

The computed response is compared against the test results for the 100%-XY Iwanuma and 88%-XY Rinaldi motions in terms of the hysteresis of the S-LRB (Fig. 7), maximum story drift (Fig. 8), maximum floor acceleration (Fig. 9) and translation displacement and twist angle of isolation layer (Fig. 10 and 11).

As shown in Fig. 7, for both the 100%-XY Iwanuma motion and the 88%-XY Rinaldi motion, the computed maximum displacement of the LRBs was similar to that from the test. However, the shape of the computed hysteretic loop showed notable discrepancy from the test. The computed hysteretic loop was odd and more slender than in the test. The authors suspect that the discrepancy is due to the limitation of the LRB model in the software to capture complex bidirectional behavior.

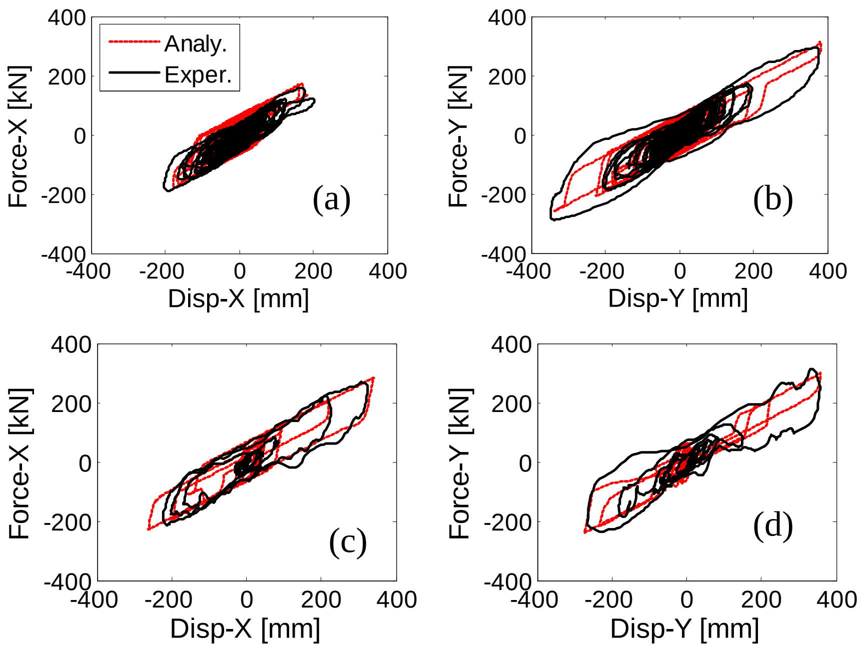


Figure 7. Hysteretic loop of S-LRB: (a), (b) X-dir. and Y-dir. from 100%-XY Iwanuma; (c), (d) X-dir. and Y-dir. from 88%-XY Rinaldi.

Fig. 8 suggests that the analysis provided a good match with the experiment for maximum story drift. A nearly uniform displacement profile along the height of the building was obtained because the base-isolation system was effective. For both motions, the maximum story drift ratio was smaller than 1/200, which is the limit for steel moment frames, in analysis and test. Fig. 9 suggests similar results for maximum floor acceleration. The analysis matched the test in one direction but showed some discrepancy in the other direction. The authors suspect that the discrepancy is due to the deferent performance of the LRBs between the analysis and the test. For these two motions, the maximum floor acceleration was smaller than 0.6g from both analysis and test.

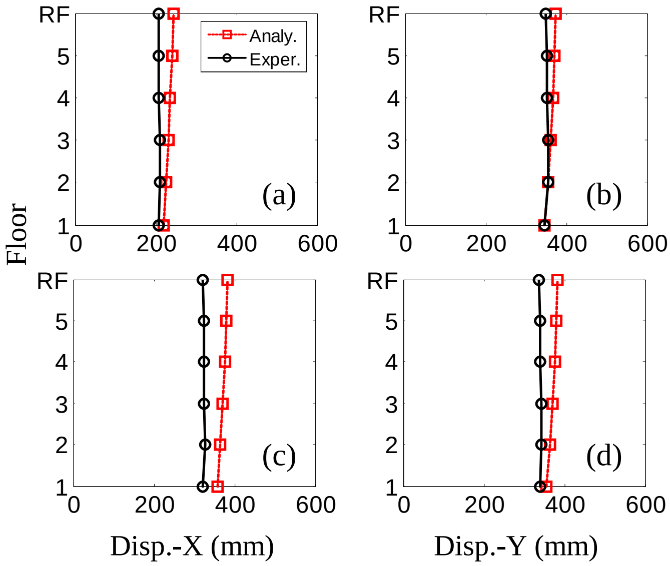


Figure 8. Maximum Story Drift: (a), (b) X-dir. and Y-dir. from 100%-XY Iwanuma; (c), (d) X-dir. and Y-dir. from 88%-XY Rinaldi.

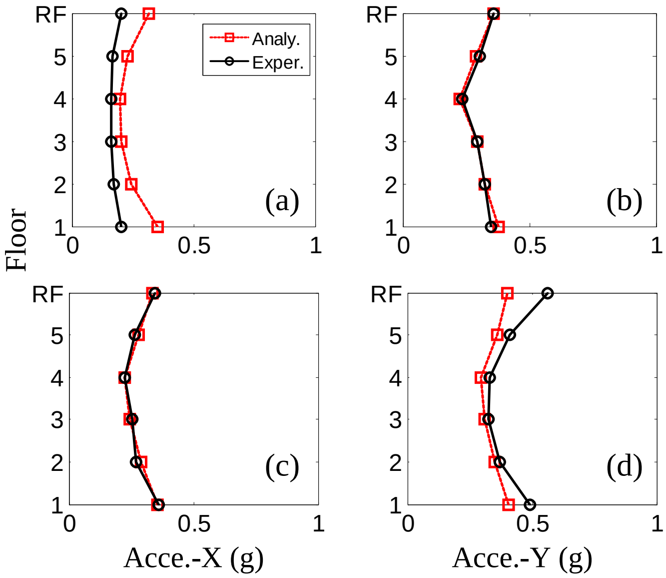


Figure 9. Maximum floor acceleration: (a), (b) X-dir. and Y-dir. from 100%-XY Iwanuma; (c), (d) X-dir. and Y-dir. from 88%-XY Rinaldi.

As previously noted, the isolation layer had a mass eccentricity ratio of 3% in the Y-direction. The mass-stiffness eccentricity ratio was 7% (the distance between the mass center and stiffness center divided by the floor-plan dimension) in the isolation layer.

Fig. 10 and 11 compares the translation and twist angle of the isolation layer represented by the center column location. The computed translation showed good match with the test result. The peak twist angle for the 100%-XY Iwanuma motion was 0.57° in the experiment and 0.75° in the analysis, which represents a 119-mm and 157-mm difference in X-displacement between the W-LRB and E-LRB. The peak twist angle for the 88%-XY Rinaldi motion was 0.96° in the experiment and 1.30° in the analysis, which represents a 201-mm and 272-mm difference in X-displacement between the W-LRB and E-LRB. The computed period and amplitude of the torsional response were a little larger than the test result.

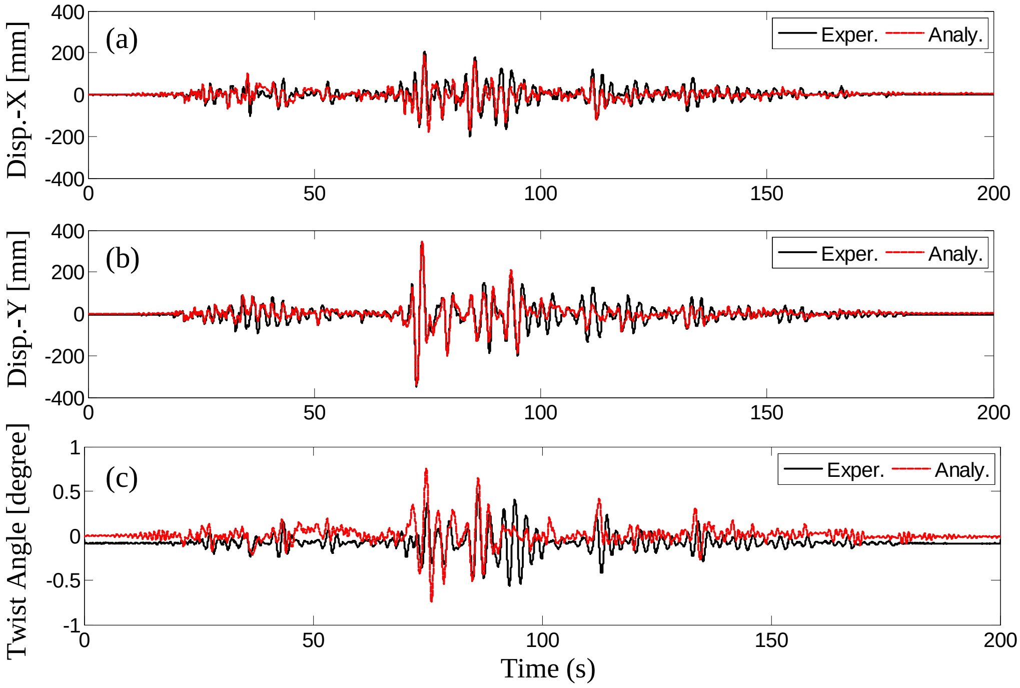


Figure 10. Translation displacement and twist angle of isolation layer for 100%-XY Iwanuma motion: (a) X-direction translation displacement; (b) Y-direction translation displacement; (c) Twist Angle.

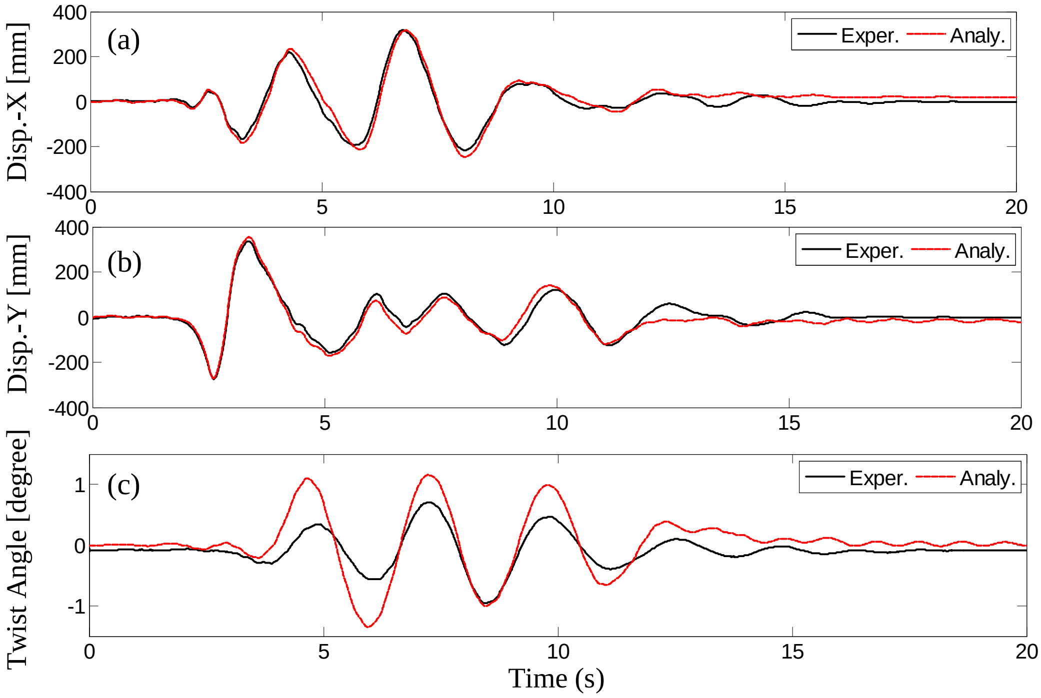
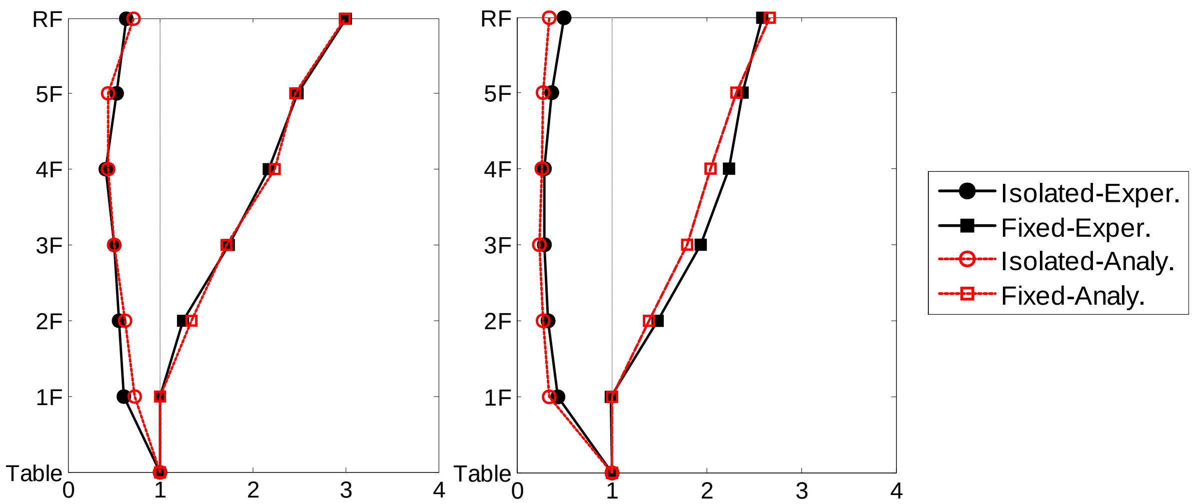


Figure 11. Translation displacement and twist angle of isolation layer for 88%-XY Rinaldi motion: (a) X-direction translation displacement; (b) Y-direction translation displacement; (c) Twist Angle.

4. EFFECT OF LRB-CLB ISOLATION SYSTEM

Fig. 12 compares the maximum floor acceleration of the base-isolated case and base-fixed case. The maximum of the vector-sum acceleration $a_{iF,max}$, normalized by the maximum vector-sum table acceleration $a_{T,max}$, is taken for the abscissa. The maximum measured table acceleration for the Iwanuma and Rinaldi motion were 0.57g and 1.14g, respectively. The factor $a_{iF,max}/a_{T,max}$ increased along the building height in the base-fixed case. The factor $a_{iF,max} / a_{T,max}$ of the base-isolated case was smaller than unity for both motions, reaching at the roof 0.63 and 0.49 in the test, and 0.71 and 0.34 in the analysis. Therefore, the test and analysis agrees that the LRB-CLB system worked effectively to reduce floor acceleration.



(a) Iwanuma motion **(b) Rinaldi motion**

Figure 12. Ratio of horizontal max-vector-sum value between response acceleration and table acceleration

5. VERTICAL LOAD DISTRIBUTION IN ISOLATION LAYER

Fig. 13 and 14 compares the time history of the vertical force (tension as positive) in N-LRB and S-LRB for the 88%-XY Rinaldi and 88%-XYZ Rinaldi motion. The dead load applied to the two LRBs was different between test and analysis, because in the test, the constructed superstructure could not be placed perfectly evenly on the nine bearings.

For the 88%-XY Rinaldi motion, the vertical force on the bearings was caused primarily by overturning moment in the superstructure. As shown in Fig. 13, in instants when the N-LRB was in compression, the opposite S-LRB was in tension. Aside from the difference in initial vertical force, the analysis agreed with the test well when the force was in compression, and the analysis response was smaller than the experiment when the vertical force approached zero. It should be noted that in the LRB-CLB isolation system, the CLBs are much stiffer in the vertical direction, particularly acting in tension, than the LRBs, and this influenced the distribution of the dynamic vertical load between the LRBs and CLBs.

For the 88%-XYZ Rinaldi motion, the vertical force on the bearings was caused by both the overturning moment and vertical table acceleration. As shown in Fig. 14, similar to the XY case in Fig. 13, the force was smaller in the analysis than in the test when the vertical force approached zero or exceeded zero. In the 88%-XYZ Rinaldi motion test, the largest measured stress in E-LRB was -4.62 N/mm^2 in compression and 1.32 N/mm^2 in tension. It is important to note that the compressive stress in LRBs was still quite small compared with the critical compressive stress σ_{cr} = 35.4 MPa. The safety of LRBs was satisfied under combined action of the compressive force and large horizontal deformation.

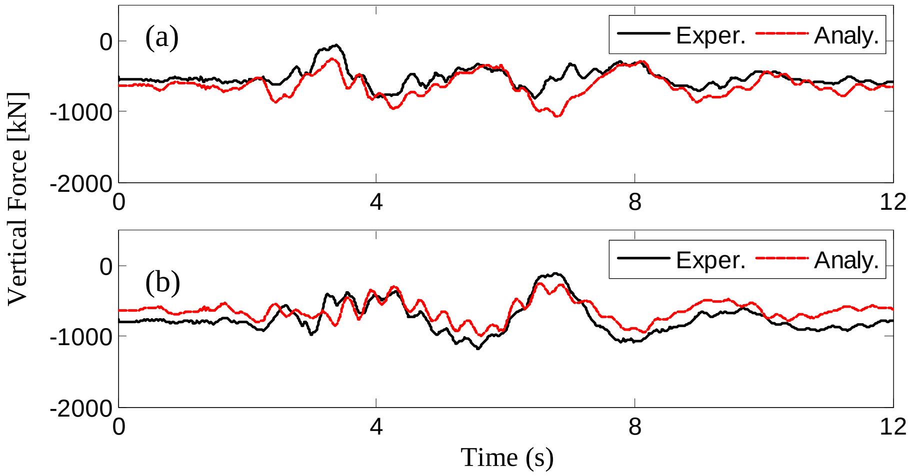


Figure 13. Vertical force in LRBs for 88%-XY Rinaldi: (a) N-LRB; (b) S-LRB.

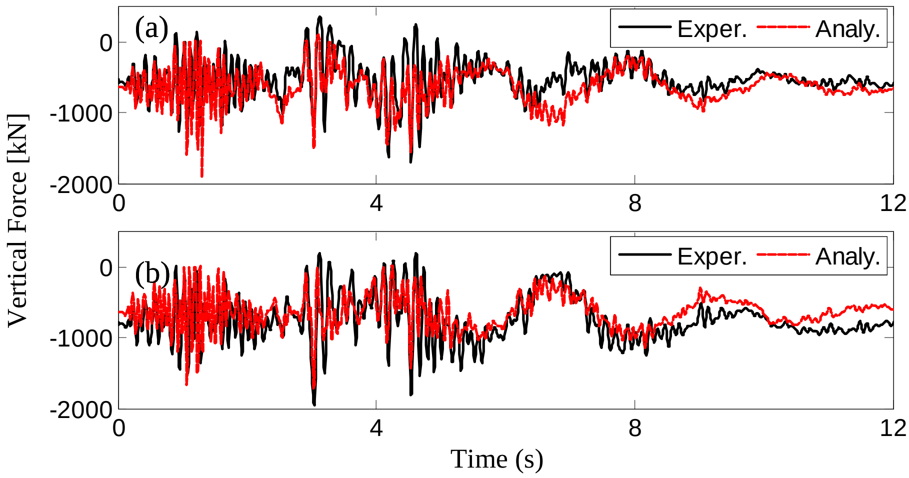


Figure 14. Vertical force in LRBs for 88%-XYZ Rinaldi: (a) N-LRB; (b) S-LRB.

6. CONCLUSIONS

The nonlinear model captured the fundamental behavior of the full-scale base-isolated building specimen. Both test and analysis indicated that floor acceleration and story drift ratio were reduced effectively by the LRB-CLB isolation system. The story drift ratio was kept within 1/200 and the steel moment-frame building remained elastic. Work is ongoing to improve the model to capture the complex bidirectional response of the LRBs more accurately.

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