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"Blind Analysis" of an Equivalent Tall Building Subjected to Long Period Ground Motions

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ABSTRACT

A "blind analysis" is conducted to compare the structural response of a full-scale steel structure subjected to ground motions at E-Defense in 2008. E-Defense is the world's largest shake table facility, where numerous structures have been shaken to evaluate their engineering performance. The structure to be evaluated is a 21-story steel building. Since testing a full-scale 21-story building is unfeasible, an equivalent test structure was engineered. The test structure consisted of a four-story, two-span by one-bay steel frame with an equivalent system mounted on top. This equivalent system consisted of concrete slabs, dampers, and rubber isolators, which were designed to mimic the structural response of the upper 17 stories. This structure was subjected to several different ground motions until some of its members finally fractured. The data from the 2008 tests are compared to a SAP2000 computer model that accounted for the steel connections, base isolators, and dampers. The results suggest the computer model was better able to more accurately predict results under complex conditions than under relatively simpler conditions, linear versus nonlinear conditions. This was attributed to the simple fact that actual test data were more affected by external influences under simple conditions than under complex conditions.

1. INTRODUCTION

The 1994 M_w = 6.7 Northridge earthquake caused significant damage to the southern California area and is considered one of the most expensive natural disasters in U.S. history. Fortunately, steel framed buildings were able to withstand the strong ground shaking from the blind thrust fault and no fatalities were attributed to the collapse or performance of steel structures (AISC 1994). However, reconnaissance teams and researchers found notable damage in steel frames, especially in the connections (Bertero et al. 1994 Bonowitz and Youssef 1995, Youssef et al. 1995, O'Sullivan et al. 1998). Their reports suggested about 70% of the floors had serious damage to at least one welded joint, while 25% of the connections did not show

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damage. Additionally, damage tended to occur in the same vicinity, meaning connections within an area of the structure would show some form of damage. Evidence also suggested that tall buildings tended to have greater damage in the upper stories, while low rise buildings tended to have damage even distributed through the stories. The most common form of damage was the brittle fracture in welded steel beam-to-column connections, particularly in or near the welded joint of a girder bottom flange and column flange. Such issues with the welded steel moment resisting frame were surprising because of the supposed plastic deformation capacity, adherence to building codes, and apparent lack of damage in steel moment-resisting framed buildings, perhaps due to architectural cladding. A year later, the 1995 M_w = 6.8 Kobe earthquake shook the Kansai area of Japan with devastating effects. The seismic performance of steel moment resisting connections, of the welded unreinforced flanges and bolted web type, was poor and several steel structures collapsed (AIJ 1995, Tremblay et al. 1996, Nakashima et al. 2000). An example of this connection is shown in Fig. 1.

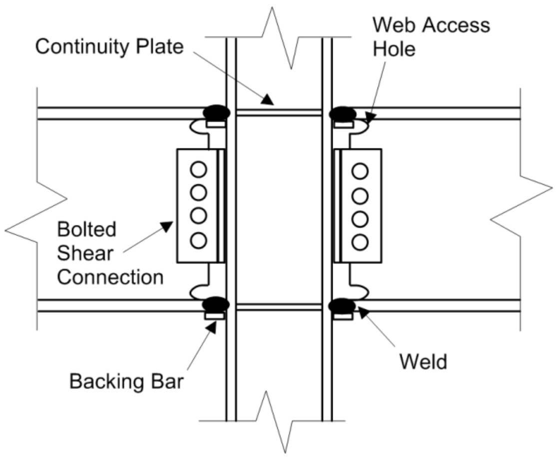

Fig. 1 Example moment resisting connection.

Many of the lessons learned from the devastating 1994 Northridge and 1995 Kobe earthquakes were incorporated into the building codes as well as earthquake engineering practice. However, several tall buildings in Japan were constructed using the older codes. Steel moment resisting connections in these tall buildings also had the welded unreinforced flanges, but other connection details differed. As a result, a project to investigate the seismic behavior of tall buildings with the original older connections was undertaken at E-Defense (Ogawa et al. 2001, Chung et al. 2010, Chung et al. 2011). After the tests of some associated E-Defense projects, a blind analysis is publicly posted before the shake table test results are formally released. This conference paper presents the author’s work on a blind analysis run while at the Disaster Prevention Research Institute in Kyoto, Japan, at that time a close collaborator with E-Defense. A brief summary of the shake table testing performed is presented in the next section, followed by the blind analysis computer simulation and results.

2. SHAKE TABLE TEST

The prototype structure that was to be shaken at E-Defense was a 21-story steel building. However, due to physical restrictions, an equivalent structure was engineered, consisting of a 4-story, 2-span by 1-bay steel frame with an equivalent system representing the upper 17 stories. This physical structure was designed to mimic the same response as the prototype by using concrete slabs, dampers, and base isolators (Chung et al. 2010, Chung et al. 2011) with a schematic shown in Fig. 2. Multiple connections were tested, with Fig. 3 showing an example of one of the welded flange, bolted web connections that was tested in 2008.

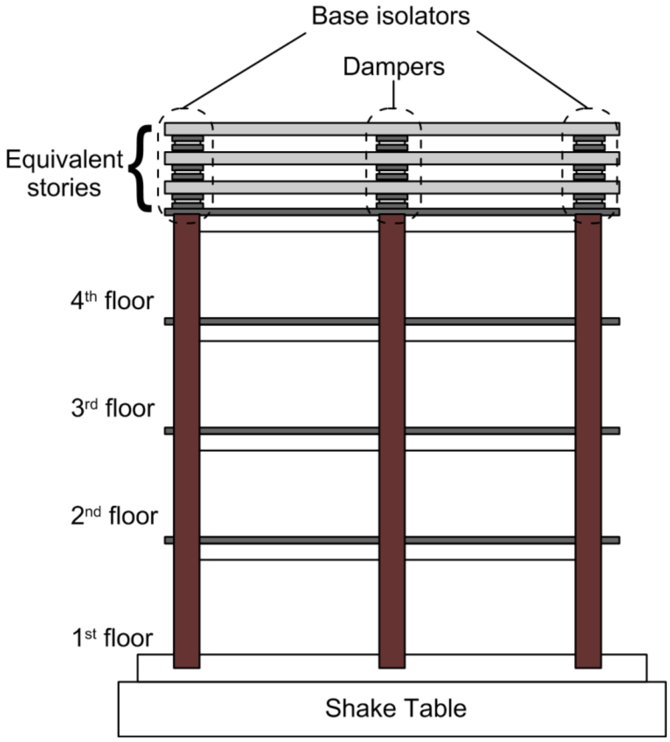

Fig. 2 Equivalent 21-story steel structure schematic.

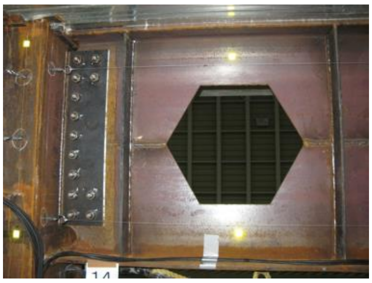

Fig. 3 Sample steel connection.

The test specimen was subjected to random white noise and several long period ground motions. Long period ground motions because the resonant frequency of tall buildings tends to be in the long period range. The ground motions considered were from El Centro scaled to 0.25 m/s in peak ground velocity (PGV), El Centro scaled to 0.5 m/s in PGV, Hog wave with a PGV of 0.40 m/s synthesized from a Kawasaki site, and San wave with a PGV of 0.51 m/s synthesized from a Nagoya site.

Although not released at the time, the connections of the test specimen underwent low cycle fatigue, with fractures typically occurring at the weld boundary next to the weld access hole in the bottom flange (Chung et al. 2010, Chung et al. 2011).

3. COMPUTER SIMULATION

SAP2000 was used to run the two-round blind analysis. To make as few modeling assumptions as possible, major structural elements were modeled as close as possible to the actual test specimen. This was possible because model geometry, member sizes, material properties, and slab properties and dimensions were provided. A rendering of the model used in SAP2000 is shown in Fig. 4. The base isolators and dampers were modeled as 2-joint nonlinear link elements, where stiffness and damping properties were manually inputted. These properties were obtained from component testing provided by a third party. When needed, a damping of 2% was assumed.

Fig. 4 SAP2000 rendering of structural model.

The author understood the purpose of this large-scale 1-g shake table test as well as the lessons learned from the 1994 Northridge and 1995 Kobe earthquakes. Therefore consideration was needed to model the beam-to-column connections. These connections were modeled using the M3 hinge element on each side of each girder. Column hinges were modeled using the P-M2-M3 hinge element applied at different heights. Hinge properties were estimated from member properties and the suggested moment-rotation diagrams and tables provided in FEMA 356 (2000) for structural steel components. As suggested, several of the connections had to be linearly interpolated as they did not satisfy certain slenderness terms.

For the blind analysis, the El Centro ground motion scaled to a PGV = 0.25 m/s and 0.50 m/s was supplied. The ground motion for El Centro scaled to PGV =0.25 m/s is shown in Fig. 5. These ground motions were used to help evaluate elastic and first inelastic response in the shake table test. For comparison, partial results from the shake table test were provided for the 2nd floor and roof level only. Additional data from each specimen floor was provided in the second round. The metric for the blind analysis was maximum acceleration.

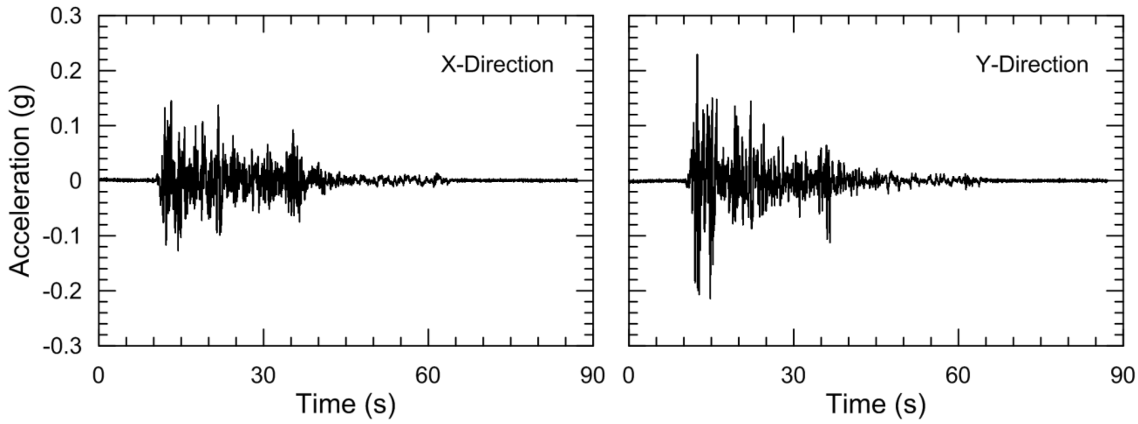

Fig. 5 El Centro input motion scaled to a PGV = 25 kine.

4. RESULTS AND DISCUSSION

The acceleration response of the 2nd floor from the El Centro (PGV = 0.25 m/s) ground motion is shown in Fig. 6, while the response for the roof level is shown in Fig. 7. Fig. 6 shows the model captures general behavior and is better in the X-direction than in the Y-direction. Interestingly, the results show slightly larger accelerations in the steady-state portion of the response. Additionally, model response is significantly better for the 2nd floor than the roof level. The simulated roof level response was drastically different from the measured data. A large difference in magnitude as well as more high frequency content is shown in Fig. 7. Similar observations were made when the model was shaken by the El Centro, PGV = 0.50 m/s time history. This concluded the first round of the blind analysis.

Adjustments were made for the second round of the blind analysis according the aforementioned observations. Since it appears that the roof level response was much less than what was experienced by the test specimen, adjustments were made by initially lowering the isolator stiffness by about 50%. Several simulations showed that lowering the isolator stiffness and yield strengths slightly improved performance at the roof level. Results for the 2nd floor are shown in Fig. 8. Whereas general underestimation was observed in Fig. 6, general overestimation was observed in Fig. 8, but an overall increase in matching results. Fig. 9 shows the results on the roof. Many of the higher frequency components appear to have been filtered out, while an increase in general magnitude in motion is seen. During simulations, it was discovered that the order of link creation would consistently result in different behavior. It is unknown as to why this is the case, but modeling took this into account.

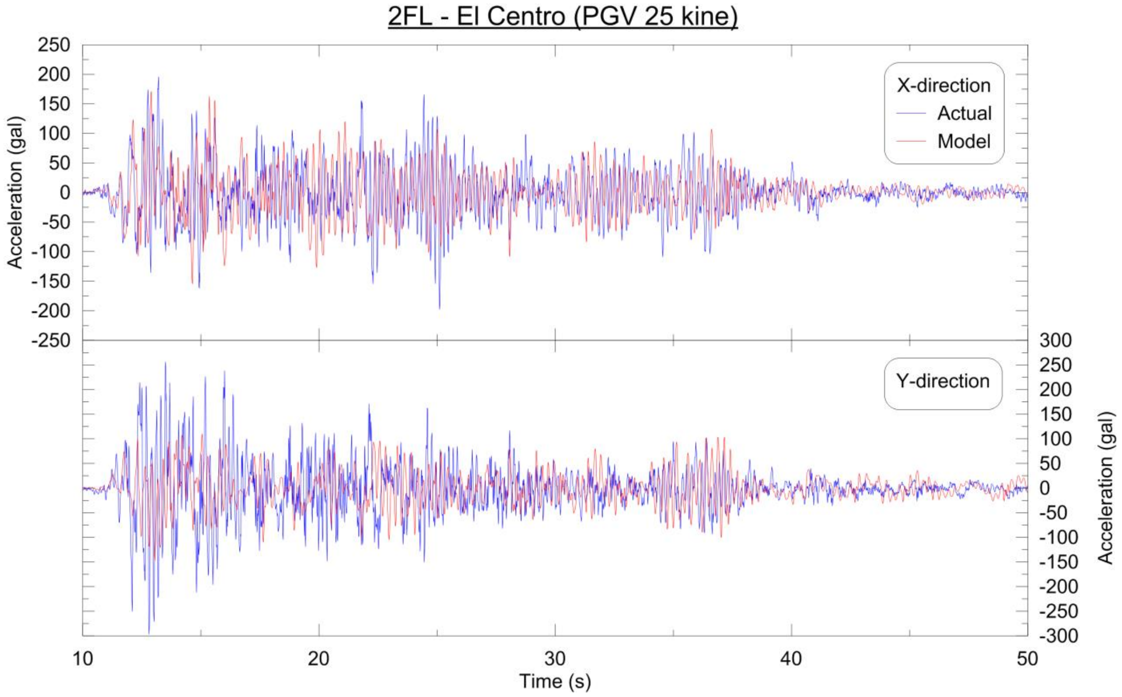

Fig. 6 Directional comparison of 2nd floor acceleration response from the El Centro (PGV = 25 kine) ground motion.

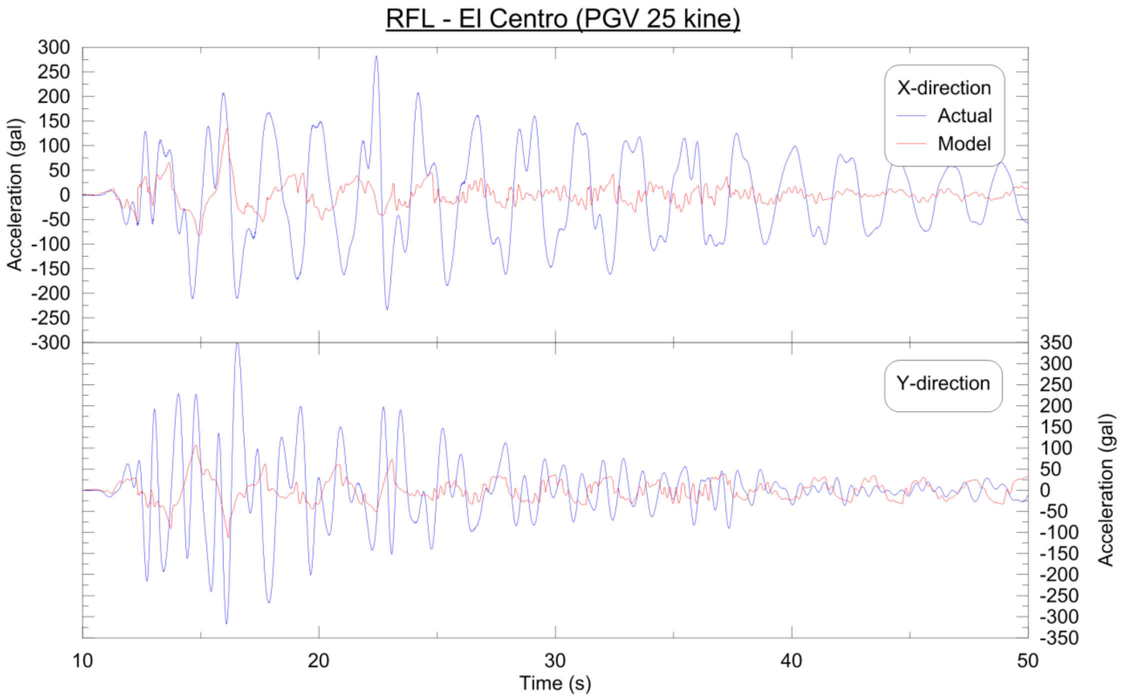

Fig. 7 Directional comparison of roof level acceleration response from the El Centro (PGV = 25 kine) ground motion.

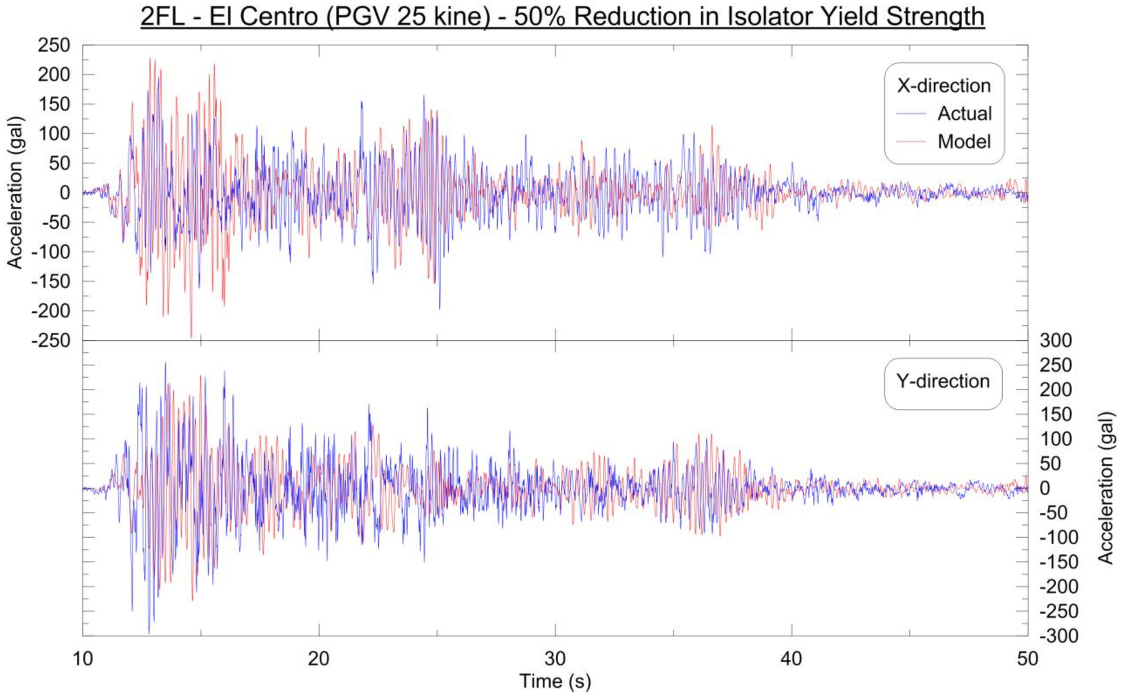

Fig. 8 Directional comparison of 2nd floor acceleration response from the El Centro (PGV = 25 kine) ground motion when there is a reduction in isolator stiffness and yield strength.

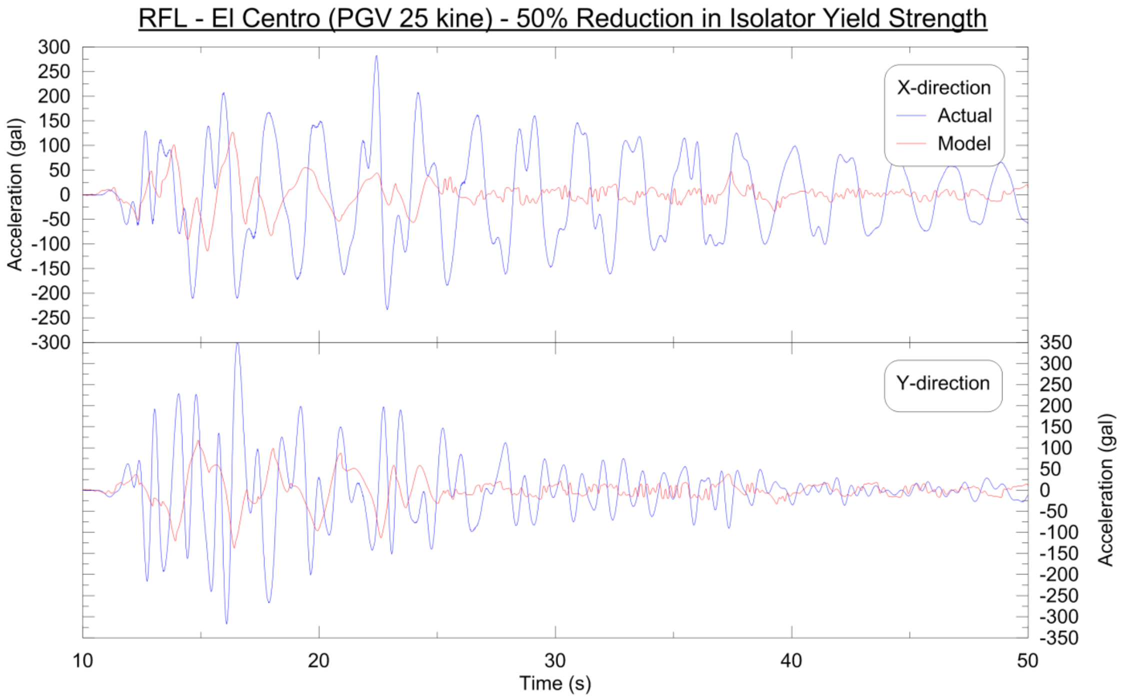

Fig. 9 Directional comparison of roof level acceleration response from the El Centro (PGV = 25 kine) ground motion when there is a reduction in isolator stiffness and yield strength.

The results of the second round are tabulated in Table 1 for the El Centro input motion scaled to PGV = 0.25 m/s and in Table 2 for the El Centro input motion scaled to PGV = 0.50 m/s. Both tables show the results from the first round of the blind analysis, indicated under the column v1, and the second round of the blind analysis, indicated under the column v2. Both tables show the fundamental period of the model to be similar, but less than the physical test specimen. These tables compare the maximum acceleration estimated by SAP2000 for each floor and compare them to the actual test results. Highlighted rows show similar values between measured test results and the second round simulation.

Table 1 shows the computer model to generally underestimate the maximum acceleration in both directions. This is not surprising as it appears the computer model is about 0.5 s stiffer than the physical model. Even so, some floors were able to get close the recorded maximum accelerations in the steel portions of the structure. As expected, the acceleration values are higher for the more intense ground motion as shown in Table 2. Interestingly, Table 2 shows that for the more relatively more intense input motion, the model is able to get close matches on most floors in the X-direction. Five out of the seven floors, covering the first four steel sections and the bottom equivalent section, matched measured responses within 60 gal. The Y-direction did not do as well, but two floors were within 70 gal of the measured response.

Table 1. Comparison of maximum accelerations experienced by each test specimen floor from the El Centro input motion scaled to PGV = 25 kine. Results from the first and second rounds are indicated by columns v1 and v2 respectively. Highlighted rows show similar values.

X-Direction	Test	v1	v2
Period, T_1 (s)	2	1.56	1.56
Maximum Acceleration (gal)			
RFL	282	98	105
7FL	258	94	89
6FL	215	92	91
5FL	238	128	121
4FL	194	144	135
3FL	217	240	197
2FL	195	222	209

Y-Direction	Test	v1	v2
Period, T_1 (s)	2	1.58	1.58
Maximum Acceleration (gal)			
RFL	351	113	204
7FL	369	138	136
6FL	283	141	148
5FL	316	279	198
4FL	330	305	280
3FL	295	341	318
2FL	309	334	448

Table 2. Comparison of maximum accelerations experienced by each test specimen floor from the El Centro input motion scaled to PGV = 50 kine. Results from the first and second rounds are indicated by columns v1 and v2 respectively. Highlighted rows show similar values.

X-Direction	Test	v1	v2
Period, T_1 (s)	2	1.56	1.56
Maximum Acceleration (gal)			
RFL	374	301	291
7FL	376	229	212
6FL	263	296	246
5FL	371	385	346
4FL	310	414	348
3FL	455	594	515
2FL	422	497	450

Y-Direction	Test	v1	v2
Period, T_1 (s)	2	1.58	1.58
Maximum Acceleration (gal)			
RFL	515	301	466
7FL	599	317	328
6FL	534	289	308
5FL	609	653	528
4FL	582	662	482
3FL	681	924	615
2FL	617	814	839

These results imply that the computer model was better able to more accurately predict results under complex conditions than under relatively simpler loading conditions. The results from relatively less intense shaking to mobilize linear elastic behavior were not as good as those under more intense shaking that were expected to mobilize nonlinear inelastic conditions. This was attributed to the fact that actual test data were more affected by external influences under simple conditions than under complex conditions, such as noise.

5. CONCLUSIONS

Due to the 1994 Northridge and 1995 Kobe earthquakes, engineers improved then building codes to account for the observations made on moment resisting connections that prematurely fractured. Japanese engineers became concerned as similar steel connections were used in the construction of tall buildings. A project at E-Defense was made to ascertain the performance of an equivalent 21-story steel building under varying ground shaking intensities. Typical of E-Defense projects, a blind analysis was posted and the author of this paper was given data to predict shake table test results. Modeling was conducted using SAP2000 for two ground motions, El Centro scaled to a peak ground velocity of 0.25 m/s and 0.50 m/s. The metric for the blind analysis was maximum acceleration for each model floor. Results after the second round appeared to suggest a better match with the steel sections in the X-direction, but not so much in the Y-direction. However, both rounds indicate the computer model to be stiffer than the physical model. These imply the computer model was better able to more accurately predict results under complex conditions than under relatively simpler loading conditions, basically linear elastic versus nonlinear inelastic behavior. This could be explained by external factors such as noise and modeling, given that modeling parameters were activated once inelastic behavior was reached.

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