NETWORKED HYBRID TESTING OF A FULL SCALE RCS COMPOSITE FRAME

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Abstract

This paper focuses on the design, experimental testing of the frame. Based on design experience, composite RCS moment frames, consisting of steel beams and reinforced concrete columns, can provide efficient and economical alternatives to traditional steel or reinforced concrete construction. Apart from the economies achieved by effective use of materials, research shows the viability of RCS beam-column connections to provide strength and ductility exceeding that in conventional steel or reinforced concrete moment frames. The full-scale RCS frame test was undertaken as a capstone project to validate the system performance of composite frames designed according to current seismic code provisions. The test offers additional opportunities to examine innovative pre-cast construction methods and to validate performance simulation models. The frame was loaded pseudo-dynamically to simulate the structural response under ground motions corresponding to earthquake hazards for a high-seismicity site with 50%, 10%, and 2% chance of exceedence in 50 years. Following the pseudo-dynamic tests, the frame was subjected to a monotonic push with inter-story drift ratios up to 0.10. Ground motions scaled to the 10% in 50 year earthquake hazard caused peak interstory drift ratios up to 0.025, accompanied by steel beam yielding and concrete spalling and plastic hinging at the RC column bases. The 2% in 50 year earthquake hazard was more damaging with peak interstory drift ratios of up to 0.055 accompanied by local flange/web buckling in the beam hinge regions and significant spalling and cracking of the concrete columns. Throughout the loading the composite beam-column connections exhibited only minor damage.

INTRODUCTION

The use of the RCS system connecting steel beams to RC columns in Taiwan is only limited to a few low-rise warehouse buildings while the use of this system in the US has been limited to high-rise buildings in low seismic regions. A number of tests have been conducted in the US-Japan Cooperative Research Program on Composite and Hybrid Structures during the past few years. It has concluded that well designed and constructed steel beam-to-RC column joints possess excellent strength and ductility characteristics for seismic applications. However, information such as the system constructability and performance of a RCS frame building are rather limited. Therefore, a large-scale test is launched in the National Center for Research on Earthquake Engineering (NCREE) by the researchers from US, Japan and Taiwan. It is believed that the large-scale test will well provide a proof of concept for this structural system.

The large-scale, 3-story RCS frame shown in Fig. 1 and Photo 1 is employed in this experimental research. The typical bay width of 7000mm and typical story height of 4000mm have been found common in Taiwan and US building configuration. The 2150 mm wide concrete slab is adopted to develop the composite action of the beams. Measuring 12 meters tall and 21 meters long, the frame is among the largest frame tests of its type ever conducted. The three-story prototype structure is designed for a highly seismic location either in Taiwan or California. Prior to the frame test, a series of subassembly tests have been completed in the structural laboratory in NCREE and some recommendations have been concluded (Tsai and Chen 2002, Cheng and Cian 2002, Chen and Lin 2002) for the design of frame specimen. Meanwhile, this experiment also provides the impetus to explore international collaboration and data archiving envisioned for the Networked Earthquake Engineering Simulation (NEES) initiatives and the Internet-based Simulations for Earthquake Engineering (ISEE) (Hsieh et al. 2002, Yang et al. 2002) launched recently in USA and Taiwan, respectively.

DESIGN OF SPECIMEN

Based on a perimeter frame concept, the frame is designed according to several standards, following the Seismic Force Requirements for Building Structures in Taiwan, IBC 2000, the 1997 AISC Seismic Provisions, the ACI 318-99 and the 1994 ASCE Joint Recommendations. Several types of connection details in RCS system have been tested in the subassembly tests conducted prior to the large-scale frame test, and those details adopted in the 3-story, 3-bay specimen are determined based on these test results. All steel beams, H396×199×7×11 for roof level, H500×200×10×16 for second floor and H600×200×11×17mm, are of A572 GR50. All columns are 650 ×650 mm square using 42*Mpa* (6000*psi*) concrete.

Previous researches (Deierlein 1988; Kanno and Deierlein 2000) have suggested the behavior of typical RCS beam-column joints and are used in designing the test frame. Fig.2 depicts the typical joint details adopted in the full-scale specimen, which include face bearing plates and steel band plates. Face bearing plates can help develop the concrete strut within the beam top and bottom flanges. Steel band plates can help confine the concrete above and below the beam and develop the outer concrete struts.

SCWB: An important design provision, which often controls the column sizes in ductile (special) moment frame, is the strong-column weak-beam (SCWB) criterion. The IBC (2000) and ACI-318 (2002) provisions require that the ratio of nominal column to beam strengths should be greater than 1.2, except at the top floor beams where SCWB is not required. As shown in Fig. 3a, the calculated strength ratios based on nominal steel beam and column strengths all equal or exceed the specified ratio of 1.2. However, as shown in Fig. 3b, when calculated based on composite beam strengths, several of the joints (shown in dashed boxes) violate the 1.2 limit. Some ratios indicate that the composite beam strength exceeds the column strengths. The values shown in Fig. 3b are based on the assumption that beams flexed in positive bending will act compositely and those in negative bending will act as steel beams. Fig. 3c summarizes checks based on composite beam behavior using *measured*, as opposed to *nominal*, material strengths of the beams and columns. Being as the main intent of the SCWB criterion is to avoid story mechanisms, the fact that one joint in a story violates the criterion is not necessarily detrimental to the frame behavior.

EXPERIMENTAL PROGRAM

Selection of earthquakes using PISA2D

Extensive nonlinear static and dynamic time-history analyses were performed using OpenSEES (Cordova et al. 2003) and PISA2D (Tsai and Chang 2001) prior to the tests. It was required to select suitable earthquake ground accelerations and investigate the most probable ultimate lateral strength and the inelastic demands imposed on the frame specimen under the simulated earthquake effects. The main focus of these studies was to predict the possible peak responses of the frame during the test while verifying that both the force and stroke limitations of the actuators were not exceeded.

Since this structure was based on a perimeter frame concept, the seismic mass used in the analytical models was based on one-half of the dead load of the 3-D building. Using this same logic, a leaning column must also be included to properly account for the P-Delta effect of the interior gravity columns. The load on this leaning column was based on recommendations from FEMA356 considering the following provision, 1.1DL+0.275LL. The fundamental period of the PISA2D frame model is 1.04 second. The material properties in the analytical models were all based on those measured from material tests of the test frame. Nonlinear static pushover analyses were also performed cyclically using PISA2D by increasing the story drift ratio gradually till 4% rad followed by pushing the structure to a drift ratio of 8% rad (see Fig. 4).

The test specimen were to be subjected to increasingly larger seismic demands that represent incipient damage level earthquakes (50% chance of exceedance in 50 years), design level earthquakes (10% chance of exceedance in 50 years) and the maximum considered level earthquake (2% chance of exceedance in 50 years). TCU082 EW component record of 1999 Chi-Chi Taiwan earthquake record was chosen because the drift ratios in all the three floors (shown in Fig. 5) are more uniform than those obtained from using several other records. Furthermore, the story energy time history as shown in Fig. 6 shows it is increasing evenly across the during of the earthquake effects. The estimated distribution and the extent of the plastic hinges are shown in Fig.7. The maximum lateral frame displacement less than 0.3 m fits quite well within the stroke limitation of the actuators. The results of the analyses demonstrate that the story shear of each floor shown in Fig. 8 is less than 2510 kN. It is noted that the peak story shear in each story usually do not occur simultaneously. Therefore, it does not directly associate with the peak story force. Fig. 9 demonstrates that the maximum story forces of 2621 kN might be developed at the 1st floor, 1852 kN at the 2nd floor and 1629 kN at the 3rd floor. In selecting the earthquake records, the distribution of the peak story forces has been the most important factor to ensure the force capacity of the actuators can satisfy the force demand. The final record chosen from the Northridge and Loma Prieta events is the LP89g04 NS component. The acceleration time history and the response spectrum of the selected ground accelerations (Chi-Chi_TCU082EW and LP89g04NS) are shown in Figs. 10 and 11, respectively.

Testing techniques—ISEE

Internet-based Simulations for Earthquake Engineering (ISEE) has been developed by the researchers in NCREE as a prototype of Internet-based cooperative structural experimental environment (Yang et al. 2002). During the full-scale RCS frame test, ISEE was activated to allow remote participants witness the real time video images of the specimen in the laboratory as well as the digital response data through the Internet. The schematic of the ISEE framework configured for this study is shown in Fig. 12. It consists of three major parts: the Data Center, the Analysis Engine and the Facility Controller. The Data Center is a database server, which processes all the prescribed data sent from the Analysis Engine and the Facility Controller. The Analysis Engine, considering both the analytical and experimental responses of the specimen, deals with the numerical integration of the dynamic responses of the entire system using Newark explicit scheme with a time step size of 0.02 second. The Facility Controller is the software bridging the experimental facilities and the Data Center. When the target displacement was satisfactorily imposed, the load cells measured the force responses of the test specimen and sent all the response data (about 300 channels) to the data logger while sending the actuator force and other prescribed response data (some specific displacement transducer or strain gauge data) back to and the Data Center for real-time Internet distribution. The related information was sent to the Analysis Engine to compute the target displacement for the next time step. Because the Data Center was too important to be interrupted, the web-casting of the digital response data have been done through an independent Web Server. Meanwhile, the Real Video Server broadcasted the video images to allow data viewers witness the experimental responses in the laboratory.

EXPERIMENTAL OBSERVATION

Four earthquake load effects were imposed on the specimen on 11, 14, 15 and 16 of October 2002, respectively. During the tests, the actuators were paused to allow researchers to examine damages, mark the cracks and take photographs. These specific pauses in terms of time step are marked by vertical lines in Figs. 13 to 16. The observations are described following the sequence of the earthquakes applied:

50% chance of exceeding in 50 years (TCU082EW, CC50/50, PGA=0.276g)

At the time step of 5.76 second in the loading time history, an abrupt loud noise was heard from the specimen. It was speculated, and later confirmed, to be the slippage of the beam splice. At the point of 7.14 second, hairline cracks were found in the columns in the first story. At the point of 18.08 second, cracks were found in all the columns, and all the slabs. Meanwhile, the beam-column joints on the first and the third floors began to form hairline cracks. At the point of 23.50 second, the white wash were found flaking in some of the beam flanges on the first and second floors as well as beam flanges and beam webs on the third floor (see Photo 2). All the conditions kept becoming more and more severe, but no drastic damage occurred through out the entire time history.

10% chance of exceeding in 50 years (LP89g04NS, LP10/50x0.8, PGA=0.341g)

At the 5.74 second time step, some of the beam flanges on the first and third floors locally buckled. None of the beams on the second floor were found buckled. In the end of this time history, severe concrete spalling shown in Photo 3 was found in the column bottom ends in the first story.

2% chance of exceeding in 50 years (TCU082EW, CC2/50, PGA=0.622g)

At the step of 12.82 second, concrete spalled under the steel band plate of column on the second floor. Meanwhile, on the third floor, severe local buckling was found in the beams and the concrete slab was crashed near columns. On the first floor, the steel beams were also found severely buckled. At the point of 19.36 second, all the concrete under the steel band plates on the tops of the columns in the first story were crashed due to bearing failure. Columns in the second story were found crashed in the bottom ends along with the crash of the concrete slab around them. Some columns in the second story were found damaged on the top and plastic hinge appeared there. At the step of 27.96 second, the test was stopped because the roof displacement has reached the preset limit (about the stroke capacity) of the actuators. In the end, the column bottom ends in the first story were crashed so drastically that the stirrups of the columns were exposed. At the same time, all the beam-to-column panel joints in the first story were found damaged with shear failure and bearing failure as illustrated in Photo 4. On the second floor, bearing failure was found under the steel band plate, and the beam splice plates yielded slightly.

10% chance of exceeding in 50 years (LP89g04NS, LP10/50x0.8, PGA=0.341g)

Since the test frame had experienced the most severe earthquake in this series of tests, new damage was not as easy to find. In the end of this time history, only the plastic hinges in the bottom end of the columns in the first story were found more pronounced.

Final pushover test

After the final pushover test to reach a roof lateral drift of about 1.0 meter, main bars in column bottom ends in the first story were severely buckled (see Photo 5), and the plastic hinges were evidently formed in the range of about a column-depth high from the top of the footings. Moreover, a beam splice plate at one first floor beam bottom flange also fractured as shown in Photo 6. The damage conditions on the second and the third floors were evidently more severe than those found after all the pseudo-dynamic tests.

PISA3D VERSUS EXPERIMENTAL RESPONSES

Based on the experimental responses of the test frame and those of the subassembly test specimen, PISA3D (Tsai and Lin 2003) has been used to simulate the responses of the full-scale frame specimen. Analytical parameters adopted for the full-scale frame were calibrated using the experimental responses obtained from the subassembly tests noted previously. Material properties are all based on the actual material test results, including concrete cylinder tests conducted on the same day of frame test. The analytical element for RC columns uses the three-parameter degrading beam-column element considering stiffness degrading, strength deterioration and pinching effects of RC elements. The analytical beams are represented by using the two-surface plastic hardening model. To take into account of the composite effect of the steel beams and the RC slabs, the positive bending strength of the beam is assumed higher than the negative bending strength of the beam by 10 percent. Furthermore, a panel joint element, considering 3-parameter degrading effects noted above, was adopted in all the beam-to-column panel zones in the PISA3D frame model. Thus, offset of rigid end zones has been activated in formulating the flexural stiffness of all the beam and column elements. The fundamental period of the PISA3D model is 1.08 second.

Figs. 13 to 16 depict the roof displacement and the base shear versus time relationships of both the experimental and analytical responses. In the CC50/50 test, the roof displacements are accurately captured from the 15th second to the 28th second, while the PISA3D model performs satisfactorily in simulating the base shear effects through the first 28 seconds. In the first LP10/50x0.8 test, the PISA3D analysis is satisfactory in the first 7 seconds in capturing the roof displacement; and the base shear can be captured well through out the first 33 seconds. In the CC2/50 test, although the simulation is not entirely satisfactory, the response trends of the test frame can be represented especially in the magnitude of the base shear. Finally, in the second LP10/50x0.8 test, the analysis can no longer be as good as those in the previous tests but the permanent roof displacement can be well captured as observed in the test frame. Comparing the four analytical simulations, it is found that the analytical model seems less and less capable of simulating the experimental responses with satisfactory precision. This variation might come from the differences between the damage conditions of the real test frame and those considered in the analytical model. After the devastating CC2/50 excitation, more or less, the RC columns, the steel beams and the RC slabs are evidently damaged. The complex damage conditions may contribute to the changes in the strength, the stiffness, the period and the damping ratio of the specimen and therefore increase the difficulties of capturing the behavior of the specimen precisely. Through adjusting the modeling parameters, perhaps, the simulations could be more satisfactory.

CONCLUSION

This experimental program has allowed the international researchers jointly explore the system performance of RCS moment resisting frame structure of realistic size subjected to various levels of earthquake load effects. The experience of constructing the full-scale frame proves that the construction of RCS system is highly efficient. Test results confirm that properly designed and constructed RCS system is a viable alternative to all-steel or all-concrete frames in high-seismic regions. However, designers should note that bolted beam splices using bearing type design might result in slippage and large bangs when the structure is subjected to significant story drift. Analytical studies suggest that numerical model constructed using PISA3D programs are effective in capturing overall seismic responses of the specimen especially when the joint element is employed in the analytical model. It incorporates the contributions of the deformations of the beam-to-column panel zone joints to the global responses of the RCS frame. The software and hardware implemented in this program for the internet-based simulations for earthquake engineering (ISEE) has allowed NCREE effectively disseminate key information before, during and after the actual tests. It clear demonstrates the potential benefits of many networked features promised in the NEES program initiated in the US. Additional information can be found in the web site at: http://rcs.ncree.gov.tw. This program has allowed the mutual exchanges of experiences not just on technical issues, but also on educational perspectives among the participants across the continents. The information of a similar test program focuses on concrete filled steel tube column and buckling restrained braced frame can also be found in http://cft-brbf.ncree.gov.tw.

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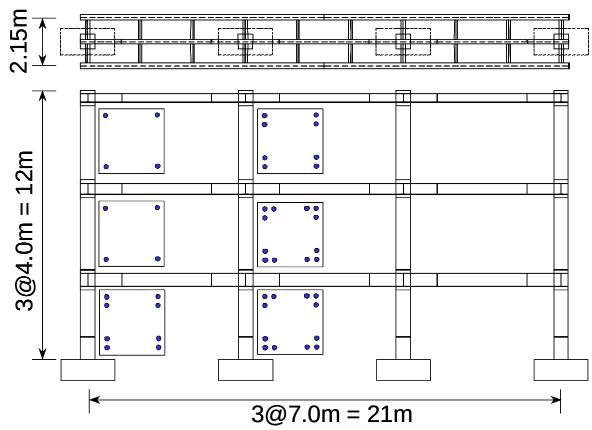

Fig. 1 Plan and elevation of test frame

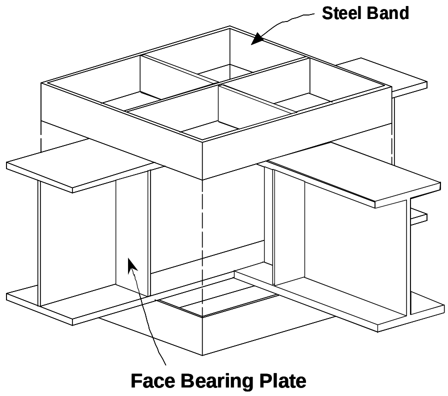

Fig. 2 Beam-column joint of the test frame

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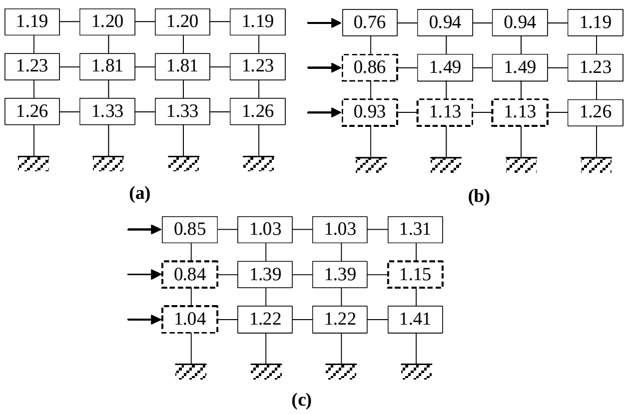

Fig. 3 ΣMc/ΣMg ratios at each joint assuming (a) steel beams with nominal properties, (b) composite beams with nominal properties, and (c) composite beams with measured properties.

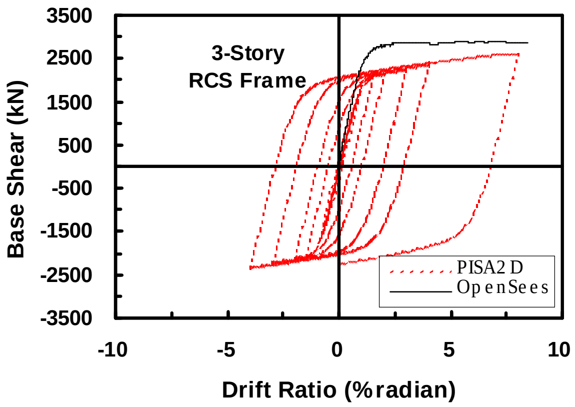

Fig. 4 Simulated nonlinear static response of the test frame

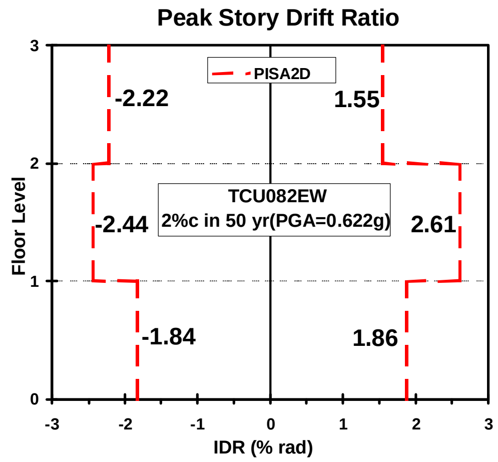

Fig. 5 The predicted peak interstory drift ratio of the test frame

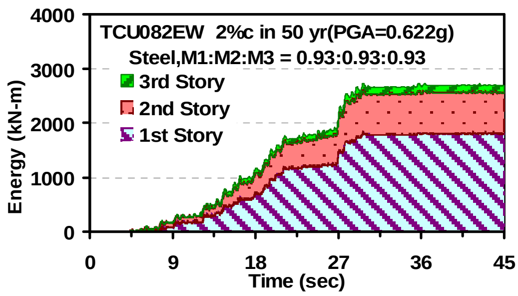

Fig. 6 The predicted energy distribution time history in each level (PISA2D)

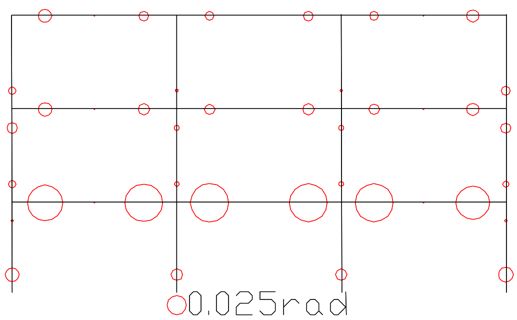

Fig. 7 The distribution of plastic hinges based on dynamic analysis (PISA2D)

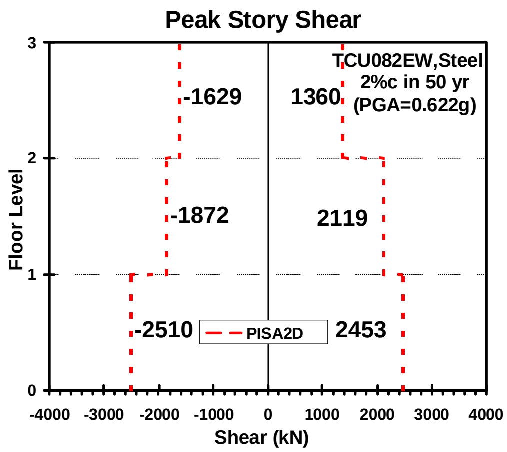

Fig. 8 Predicted peak story shear in each level

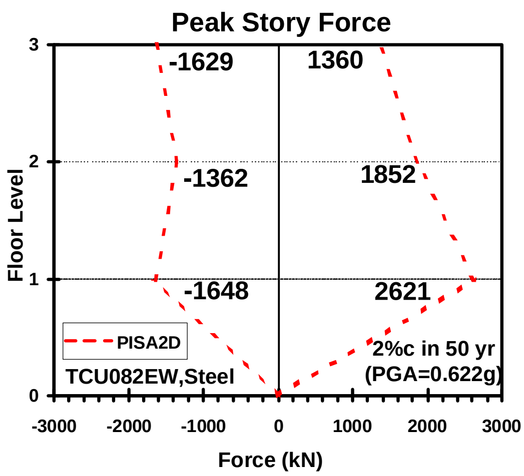

Fig. 9 Predicted peak story force on each floor

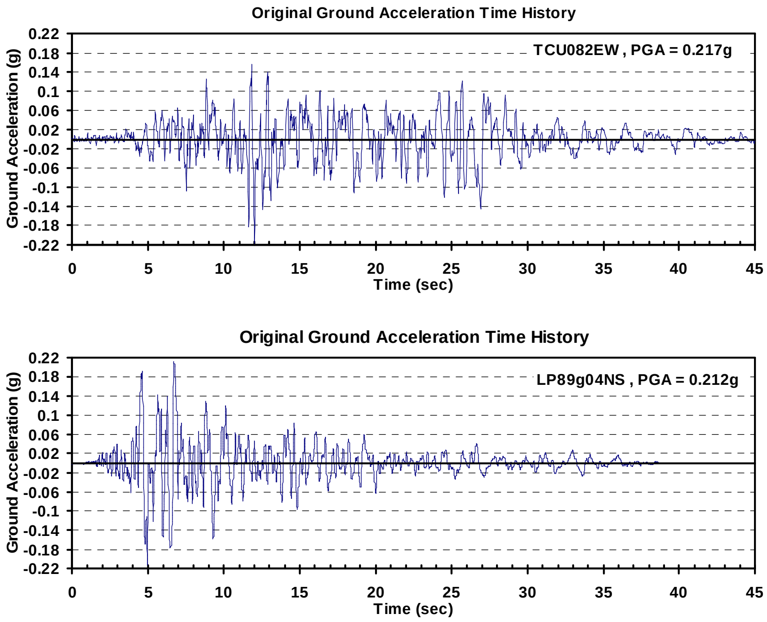

Fig. 10 The original ground accelerations used in test (before scaling)

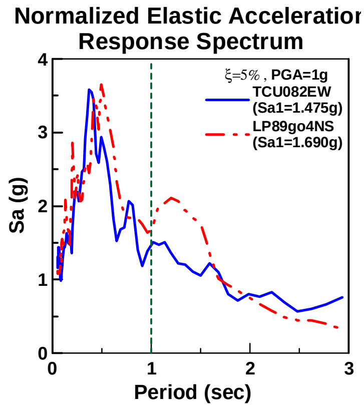

Fig. 11 Normalized response spectrum of the two records

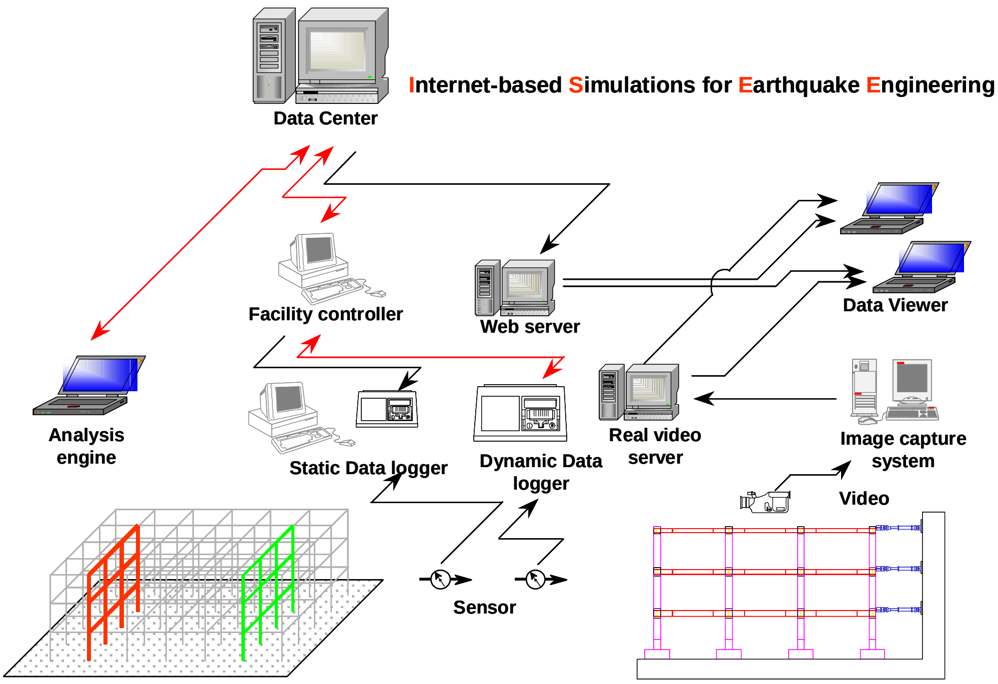

Fig. 12 ISEE framework employed in the pseudo-dynamic test

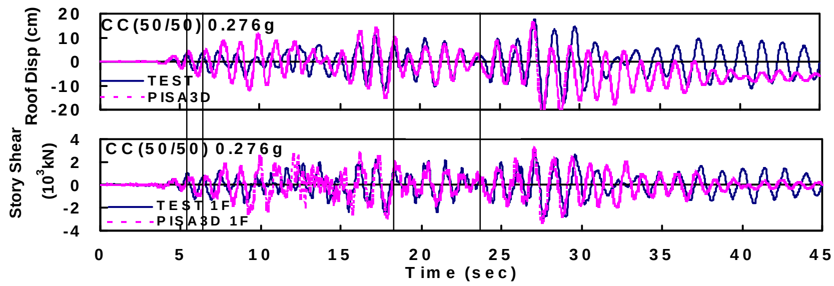

Fig. 13 Comparing the PISA3D analytical responses with experimental results under CC50/50 excitation (damping ratio=1%)

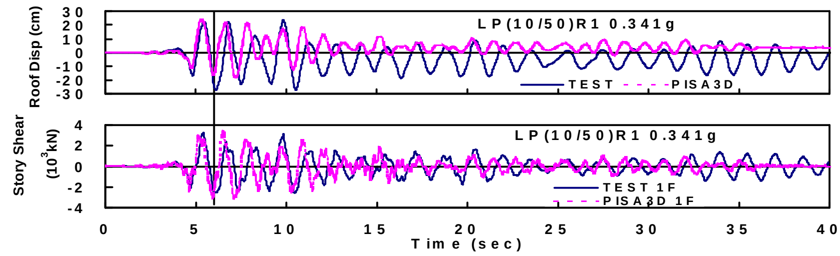

Fig. 14 Comparing the PISA3D analytical responses with experimental results under the first LP10/50x0.8 excitation (damping ratio=1%)

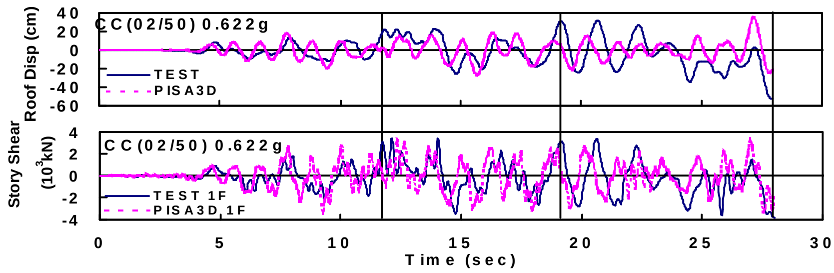

Fig. 15 Comparing the PISA3D analytical responses with experimental results under CC2/50 excitation (damping ratio=1%)

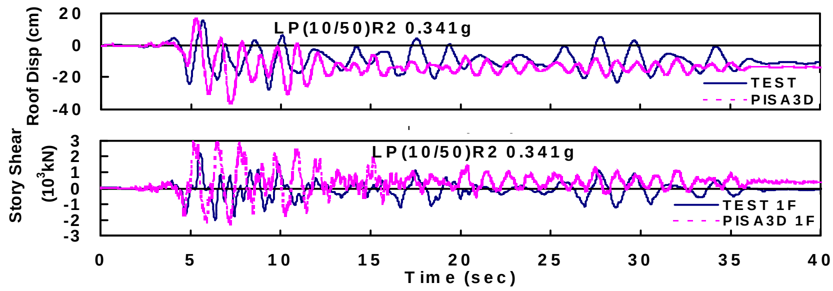

Fig. 16 Comparing PISA3D analytical responses with experimental results under the second LP10/50x0.8 excitation (damping ratio=1%)

Photo 1 Large-scale, three-story, three-bay RCS moment frame

Photo 2 White wash flaking at the beam bottom flange (CC50/50 45 sec)

Photo 3 Concrete spalling in the column bottom ends in the first story (1st LP10/50x0.8 39.96 sec)

Photo 4 Shear failure and bearing failure of beam-column connection in the first story (CC2/50 27.96 sec)

Photo 5 Buckling of main bars in the column bottom end in the first story (Pushover)

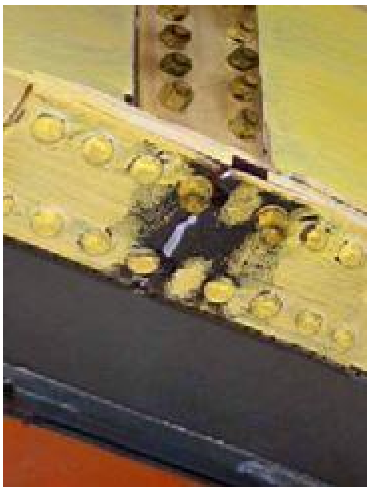

Photo 6 Fracture of beam splice steel plate (Pushover)