Experimental Report on

NEES@UCSD Shake Table Test Program: Inertial Force-Limiting Floor Anchorage Systems for Seismic Resistant Building Structures



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ABSTRACT

A new floor anchorage system is being developed that limits the inertial forces that develop in the structure during an earthquake. This "Inertial Force-Limiting Floor Anchorage System" or IFAS is intended for low-damage seismic-resistant building structures. The optimum design properties for the IFAS were developed through extensive analytical research. A prototype anchorage system was developed using state-of-the-art components. The performance of the IFAS was examined in a shake table test of a half scale 4-story concrete building structure. Rocking precast walls and special precast columns were employed for test repeatability. The structure was purposely provided with an eccentric wall layout to examine the performance of the system in two directions. Three test phases were executed: In Phase 1, the IFAS structure was tested; In Phase 2, the floors were "locked" down to the walls to simulate an typical floor anchorage in order to provide a direct comparison with Phase 1. In a Phase 3, the rocking walls were "locked" into the foundation to simulate a reinforced concrete shear wall structure (employing high strength reinforcement). The structure was subjected to 15 strong motions in total. The test results indicated a reduction in response for the IFAS structure, including less shear wall base rotation, building drift, and accelerations. These results indicate the potential for the IFAS to minimize structural and non-structural damage in building structures.

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Table of Contents

Abstract
Project Team
Table of Contents
1. Project Background…….. …….. 1
2. Project Concept……... 2
3. Test Phase ……... 2
3.1 Test Phase Detailed Description ……... 3
3.1.1 Phase 1 ……. 4
3.1.2 Phase 2 ……. 4
3.1.3 Phase 3 ……. 5
4. Instrumentation planning of the Shake table specimen……... 6
5. Construction of Shake table specimen…….. 8
6. Half scale Shake table test of Inertial Force-Limiting Floor Anchorage Systems (IFAS) at NEES @UCSD…….. 8
6.1. Earthquake introduction ……... 9
6.2. Shake Table Testing Schedule…….. 10
6.3. Testing Details ……. 11
7. Test results ……. 12
7.1 Shake table general response…… 12
7.1.1 Lateral Force Resisting System (LFRS) Behavior……. 12
7.1.2 Gravity Load Resisting System (GLRS) Behavior……... 17
7.2 Acceleration response comparison …….. 20
7.3 Comparison between numerical model and test results ……... 21
8. Conclusion ……..... 24

1. Project Background

A shake table testing program was conducted in December 2013 through January 2014 at the NEES@UCSD Large High Performance Outdoor Shake Table at the Englekirk Structural Engineering Research Center. The testing program is part of the National Science Foundation (NSF) George E. Brown Network for Earthquake Engineering Simulation Research (NEESR) project *Inertial Force-Limiting Floor Anchorage Systems for Seismic Resistant Building* being led at the University of Arizona by Principal Investigator Dr. Robert B. Fleischman, with co-investigators Dr. Jose I. Restrepo at University of California-San Diego (UCSD) and Dr. Richard Sause at Lehigh University. The project involves the development of an innovative floor anchorage system that reduces inertial forces during earthquakes.

The inertial force-limiting floor anchorage system has the potential to significantly reduce floor accelerations and diaphragm forces in a building structure, thereby lowering seismic demands on the lateral force resisting system (LFRS) and reducing overall structural/nonstructural damage. The system does so by using a special anchorage for the connection between the floor diaphragm and the main LFRS elements (e.g. shear walls, braced or moment frames). The connection possesses a "slip load", at which the floors begin to "slide" relative to the LFRS elements, with the system dissipating energy. Complementary elements in the system provide stability bracing, elastic restoring and limit excessive floor motion.

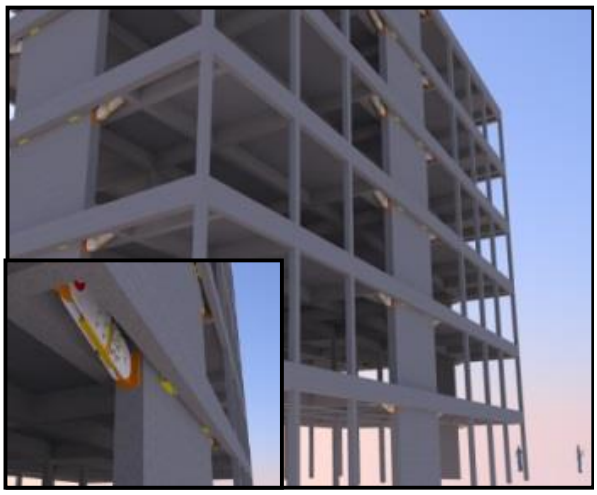
The inertial force-limiting floor anchorage system was developed collaboratively by the research team and the project seismic design consultants, David Mar, principal at Tipping and Mar and Dr. Joseph Maffei, former principal at Rutherford & Chekene and current president of Maffei Structural Engineering. The shake table testing program follows 2 years of analytical research and component testing in developing a prototype design for the anchorage system. The shake table testing program will attempt to demonstrate the floor anchorage system and compare the improved performance of a building with the system relative to a building with regular floor diaphragm connections.

The shake table test structure was designed and detailed by UCSD co-PI Jose Restrepo lead by Ph.D Student Arpit Nema in collaboration from Ph.D. students Gabriele Guerrini and David Duck. The building was constructed by T.B. Penick and Sons; the precast elements were provided by funds from the Precast/Prestressed Concrete Institute (PCI), PCI West and the Charles Pankow Foundation (CPF), and produced by Midstate Precast under the supervision of David Dieter.

The analytical research was supervised by postdoctoral researcher Dr. Dichuan Zhang, now at Nazarbayev University, Kazakhstan, and conducted by UA Ph.D. student Zhi Zhang. The component testing program and prototype anchorage system were developed by co-PI Sause and Lehigh Ph.D. student Georgios Tsampras. The bumper testing and instrumentation deployment was conducted by UA Ph.D. students Ulina Shakya and Anshul Argawal. The instrumentation plan was created by UA Research Experience for Undergraduates (REU) Scott Kuhlman. UA REUs F. Gastelum, D. Lizarraga, A. Houk and M. Lostra assisted in the analytical research and former UCSD student Armita Pebdani assisted in the construction of the structure. International collaborators (1) Giorgio Monti and Giuseppe Quaranta of University of Roma La Sapienza, Giuseppe Marano of Polytechnic of Bari, and Alessandro Scodeggio of Technical University of

Athens, studied optimum device layout, and (2) Juan F. Correal and Nelson Angel, of the Universidad de los Andes in Colombia, studied the constructability of the gravity system. The K12 Outreach was coordinated by Utterback Middle School science teacher Gricelda Meraz.

2. Project Concept



The concept has the potential for producing more economical and safer seismic designs with application to a wide array of construction types and building systems. The benefits of such a system include: (1) reduced seismic forces; (2) reduced structural lateral drift; (3) reduced structural damage; (4) reduced diaphragm acceleration; (5) reduced nonstructural and contents damage; and (6) a higher factor of safety against collapse. In addition, for systems with a potential vulnerability in the floor diaphragms, as has been evidenced in the poor performance of precast concrete diaphragm in previous earthquakes, a major benefit can be achieved by the reduced diaphragm acceleration to eliminate diaphragm demands for reliable seismic performance of precast concrete structures. If successful, this research could transform the manner in which structures are designed to resist earthquakes by modifying the response to strong ground motion and thereby significantly impacting the cost-effectiveness of the U.S. construction industry.

3. Test Phase

In **Phase 1**, the floors are connected using the special anchorage system. The LFRS is unbonded post-tensioned (PT) bar precast hybrid rocking walls. The structure is expected to experience reduced seismic response due to relative motion ("sliding") of the floors. The ground motion sequence will proceed from a service to a design to a maximum considered earthquake at Berkeley CA.

In **Phase 2**, the special floor anchorage system is replaced by a traditional diaphragm connection (JVI PSA). The structure is subjected to same sequence of earthquakes to provide a direct comparison to Phase I. The structure should experience the normal seismic demands anticipated at each seismic hazard level, which are expected to be significantly larger than in Phase I. The three precast hybrid walls are in an eccentric C-shaped layout and thus should undergo bi-directional drift and yaw.

In **Phase 3**, the PT is removed and the walls is grouted into pocket foundations to create Precast Emulative Shear Walls. The shear walls are reinforced with Grade 100 reinforcement to allow the examination of a RC wall structure with high strength reinforcement under strong intensity ground motions up to or exceeding the MCE. The eccentric layout again induced twisting of the structure in plan, causing simultaneous bi-directional motion and yaw of the walls.

Each phase has specific objectives as described in the ***Test Phase Detailed Description*** that follows.

3.1 Test Phase Detailed Description:

The primary objective of the shake table test is to demonstrate the seismic performance of a structure using the proposed floor anchorage system. However for a direct comparison, the structure will be tested afterward using a traditional floor anchorage system. Thus the test program also provides opportunities for investigating the seismic response of traditional structures. For repeatability, the LFRS selected are well-confined precast concrete post-tensioned rocking walls. To maximize the impact of the testing, an eccentric shear wall layout was selected in the direction of the table motion (*See Plan View below left*). This layout will test both the new floor anchorage system and the traditional structure to drift in the direction of the table shaking and twisting causing drift perpendicular to the table shaking direction. The repeatable test structure is needed to carry out the comparison between the new floor anchorage system and the traditional structure. However, a destructive test is possible for the final test(s) of the program. For this reason, the rocking walls were reinforced with Grade 100 reinforcement, and in a third phase, the rocking walls will be grouted in to create a condition that emulates a reinforced concrete shear wall with high strength reinforcement.

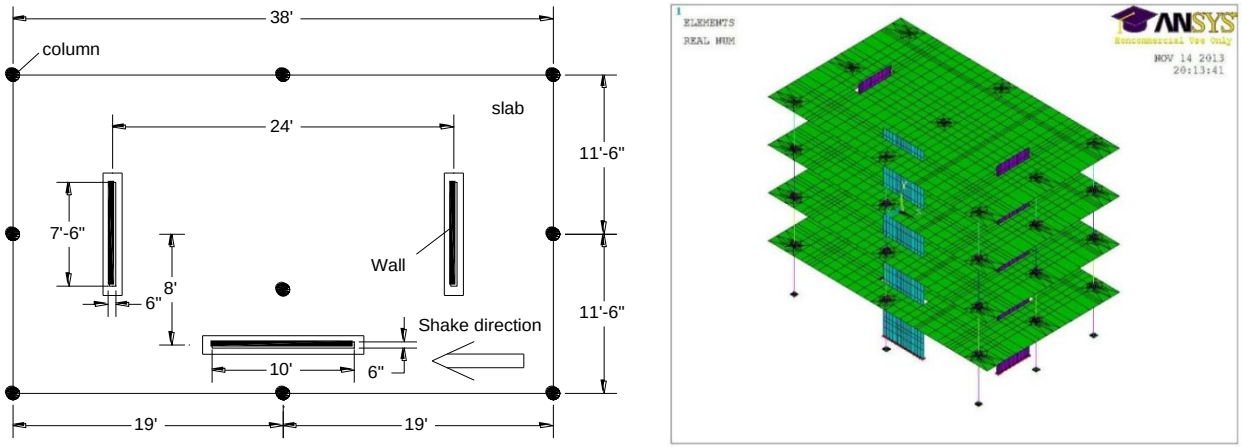
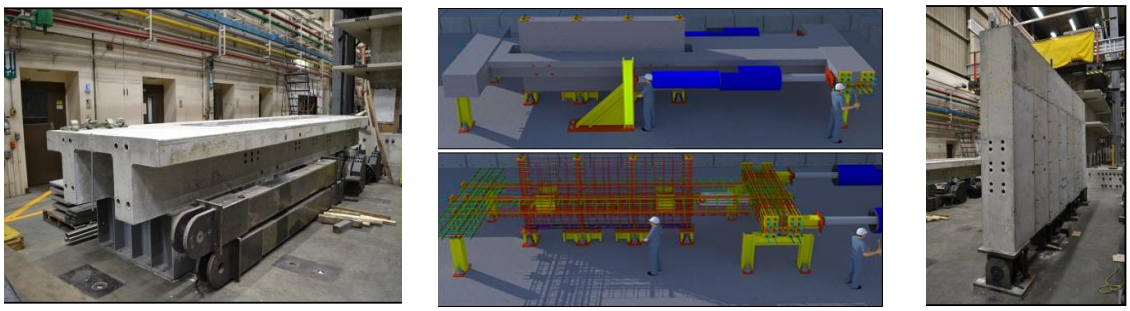


Fig 1. (a) Eccentric Plan; **(b)** ANSYS model of the shake table specimen

The inertial force limiting floor anchorage system prototype was developed at Lehigh University, as part of their structural component and system testing at NEES@Lehigh (See Schematic and Photos of the Structural Testing Specimen below).



3.1.1 Phase 1:

In the first phase of the shaking table program, the floors in the structure is connected using special floor anchorage system.

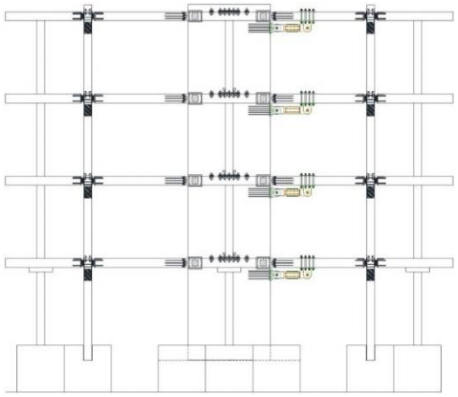


Fig 2. Shake table structure section view at the main wall

A schematic of the shake table structure floor anchorage system is shown in the schematic below: (a) a friction damper on the underside of the floor slab connects to the shear wall in the shake table direction (buckling-restrained braced are used for the walls in the perpendicular direction); (b) rubber bearings placed between the walls faces at each end and the slab to brace the walls out-of-plane; and (c) a bumper between the wall end and the slab, with an initial gap, to prevent the floor from undergoing excessive relative motion with the wall.

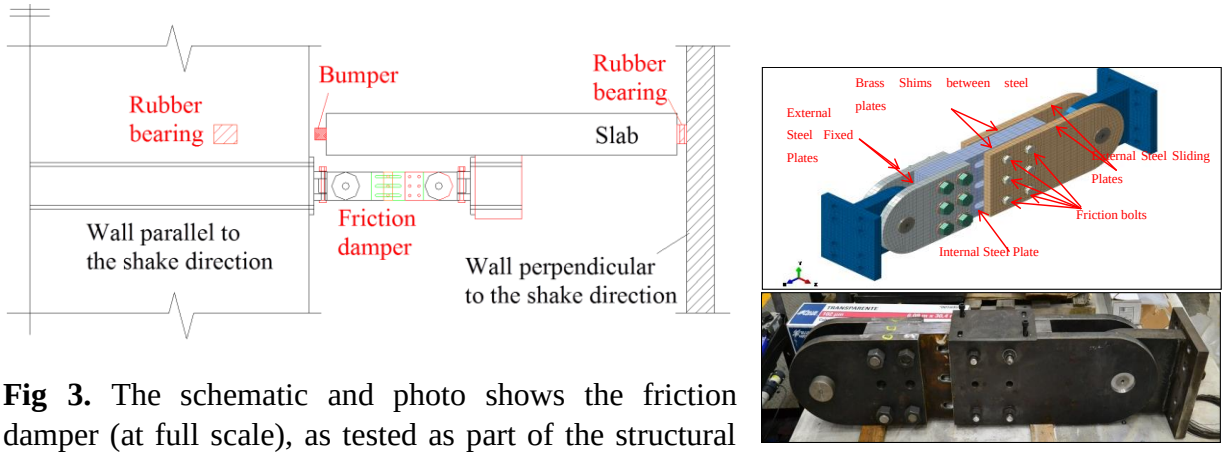


Fig 3. The schematic and photo shows the friction damper (at full scale), as tested as part of the structural component testing at NEES@Lehigh to develop the prototype.

3.1.2 Phase 2:

In the second phase, the special floor anchorage system is replaced by JVI PSA slotted connections, intended to serve as the regular diaphragm connection of traditional construction. The LFRS remain unbonded PT bar precast hybrid rocking walls and the energy dissipators near the base of the walls are replaced. The structure is subjected to same sequence of earthquakes as in Phase 1 to provide a direct comparison to Phase I. The structure experienced the normal seismic demands anticipated at each seismic hazard level, which are significantly larger than Phase I.

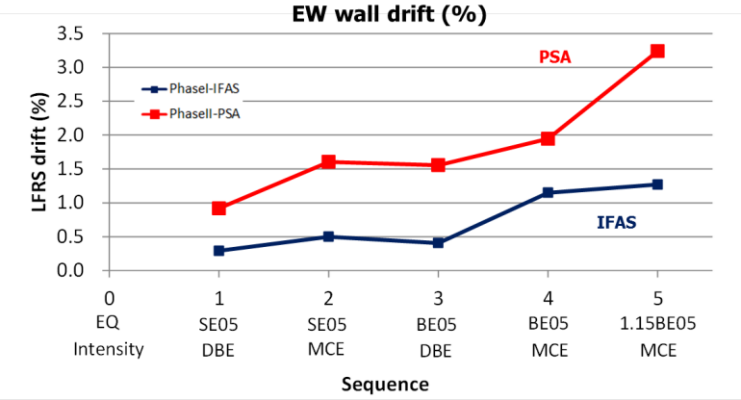


Fig 4. Expected LFRS drifts

This phase, in addition to providing a direct comparison to the special floor anchorage system, permit the examination of unbonded PT bar precast hybrid rocking walls in a shake table test at MCE level, including in an eccentric layout of the primary wall that induce twisting of the structure in plan; and with the other walls oriented transverse to the direction of table motion thereby enabling examination of the Precast Hybrid Rocking Walls for drifts in their out-of-plane direction.

Phase II also test the performance of special single story precast "pinned" gravity columns. These elements were designed to survive large drifts without a story mechanism, required due to the thickened floor slabs provided for sufficient tributary mass, which as a result violate strong column-weak beam principles. The columns are connected at each level (and to the footing) through a single large diameter Gr 100 bar that slides into a corrugated steel duct cast at the base of the precast column above. After erection, the duct and the column base are grouted in a single operation. Similar columns are used in construction that combines precast columns with a cast-in-place slab. The large diameter central bar has a noncontact lap splice with the bars cast in the column. The precast columns have the potential for reducing damage in columns subjected to compatible displacements.

3.1.3 Phase 3:

In the third phase, the PT bars and energy dissipators are removed and the shear walls are grouted to the pocket foundations to create a Precast Emulative Shear Wall. The shear walls are reinforced with Grade 100 reinforcement. Thus, this phase allow the examination of a traditional concrete wall structure reinforced with high strength reinforcement for strong intensity ground motions, including motion representing or even exceeding MCE at Berkeley.

Again, as in Phase II, the eccentric layout of the primary wall induce twisting of the structure in plan resisted by offset orthogonal walls. Thus, this phase examines the walls being subjected to bi-directional actions, including simultaneous in-plane actions and yaw (twisting about the axis of the wall); or simultaneous in-plane motion, out-of-plane motion and yaw out. These actions, which have been rarely included in tests, replicate more closely the actual demands felt in shear walls in buildings during earthquakes than simpler tests subjecting the walls to unidirectional actions.

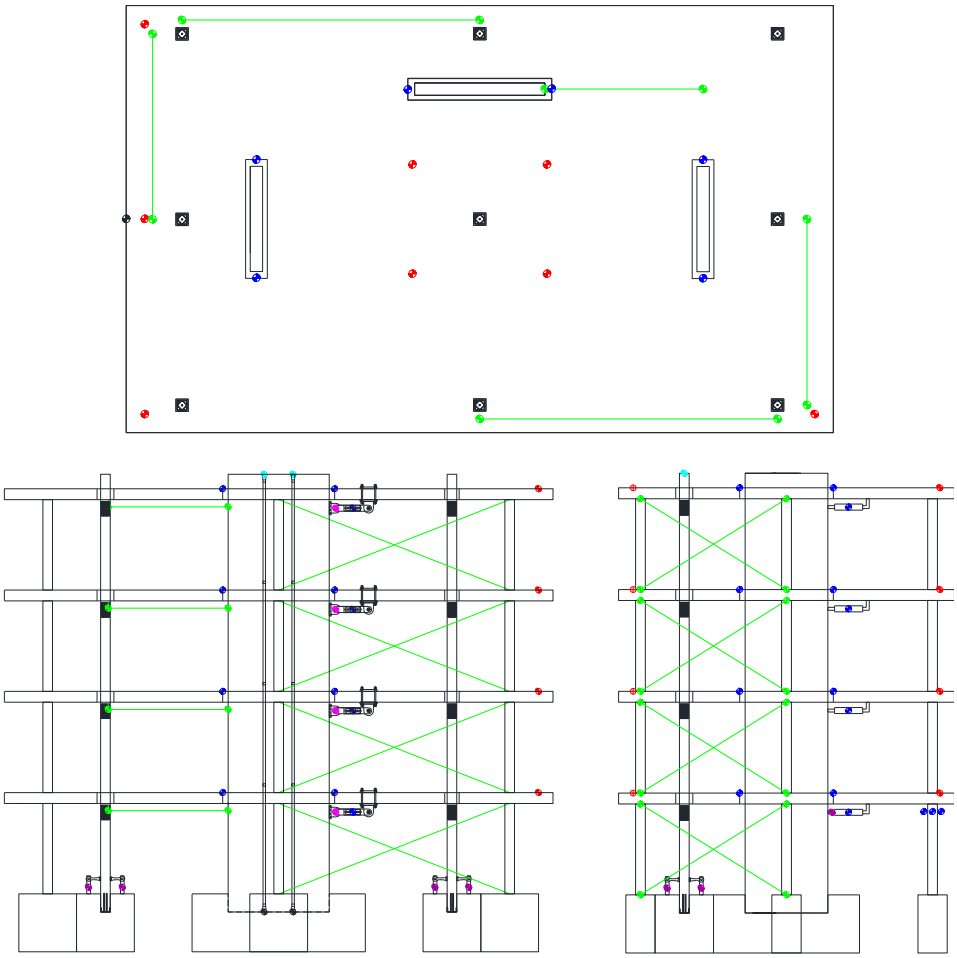
The knowledge gained from this testing phase includes examination of Grade 100 reinforcement broadly in Precast/RC emulative shear walls under bi-directional response to strong ground motions, including specific examinations of: (1) heavily reinforced boundary elements in walls subjected to combined lateral-torsional demands; (2) a short-length grouted wall foundation joint; (3) the local stability of the wall longitudinal reinforcement, as highlighted in a recent ATC-98 report; (4) the overall stability of the wall in the plastic hinge region, important based on recent observed behavior of shear walls in Chile and in New Zealand; (5) Lap splices at the ACI limit; (6) Shear reinforcement anchored outside the confined concrete core; (7) Steel Embeds with ASTM A-496 High-strength Deformed Bar Anchors (DBAs).

4 Instrumentation planning of the Shake Table Specimen

Instrumentation planning was done prior to the construction of the shake table specimen. 268 sensors were used for the testing in total. Some instruments were added for phase 2 and different instrument planning was done for phase three of the project.

The sensors used in the test are accelerometers, linear pots, string pots, spring linear pots, pressure transducers, load cells and strain gages (see in Fig.9 and Fig. 10). Accelerometers (Fig. 10(b)) are used for measuring the structural acceleration response; linear pots (Fig. 10(a)), string pots (Fig. 10(c)) and spring linear pots are used for measuring the structural deformation; pressure transducers are used for measuring the PT bar force; load cells are used for measuring the internal force in FD; strain gages are used for measuring the strain in energy dissipaters (ED), BRB, reinforcing bars and PT bars.

Based on the measurements, we are able to derived hybrid rocking wall base rotation, GLRS inter-story drift, PT force, ED force, BRB force and slab twisting.



Origin

- Accelerometer (AccM)
- String pot (SP)
- Linear pot (LP)
- Load cell (LC)
- Pressure transducer (PreTr)
- Strain gage (SG)

Fig.5. Instrumentation Plan and Elevation view



(a)Linear pot at column top end (b) Accelerometer (c) String Pot

Fig.6. Instrumentation device photos

5 Construction of Shake Table specimen

In Project year 2, a shake table test at NEES@UCSD was planned and designs were initiated. The design of shake table specimen was completed in project year 3. The construction of the shake table was scheduled for fall 2012 and the testing occurred in December 2013 and January 2014. The major objective of the shake table test is to demonstrate the performance of the floor anchorage system proposed in year 1 research (*NSF annual report 2012 - Project Activities*), including a direct comparison of a building with the floor anchorage system to a traditional building. Furthermore the shake table test data is being used for model calibration in the year 3 research.

A half-scale 4-story reinforced concrete structure is selected for the shake table test specimen (See Fig.21). The half scale test structure has a plan of 38' x 23' with a floor-to- floor height of 7'. The slab thickness is increased to 9" in order to add additional mass the structure. With this added mass, the input ground motion acceleration can remain the same as the full scale magnitude, instead of being increased to twice of the full scale magnitude. The LFRS system is selected as unbonded precast post-tensioned (PT) walls: one wall (EW wall) in table shaking direction, offset to create an eccentric loading condition and two walls (NS wall) in other directions to resist the twisting moment generated by the eccentricity.



Fig.7. Construction of the shake table specimen

6 Half scale Shake Table test of Inertial Force-Limiting Floor Anchorage Systems (IFAS) at NEES @UCSD

The Shake table test (SKT) was conducted at the Large High Performance Outdoor Shake Table (LHPOST) at the NEES@UCSD site. The shake table test specimen was a half scale 4-story concrete building structure (See Fig. 1a). The structure was purposely provided with an eccentric wall layout to examine the performance of the system in two directions. Rocking precast walls and special precast columns were employed for test repeatability (See Fig. 1b).

The concept has the potential for producing more economical and safer seismic designs with application to a wide array of construction types and building systems. The benefits of such a system include: (1) reduced seismic forces; (2) reduced structural lateral drift; (3) reduced structural damage; (4) reduced diaphragm acceleration; (5) reduced

nonstructural and contents damage; and (6) a higher factor of safety against collapse. In addition, for systems with a potential vulnerability in the floor diaphragms, as has been evidenced in the poor performance of precast concrete diaphragm in previous earthquakes, a major benefit can be achieved by the reduced diaphragm acceleration to eliminate diaphragm demands for reliable seismic performance of precast concrete structures.

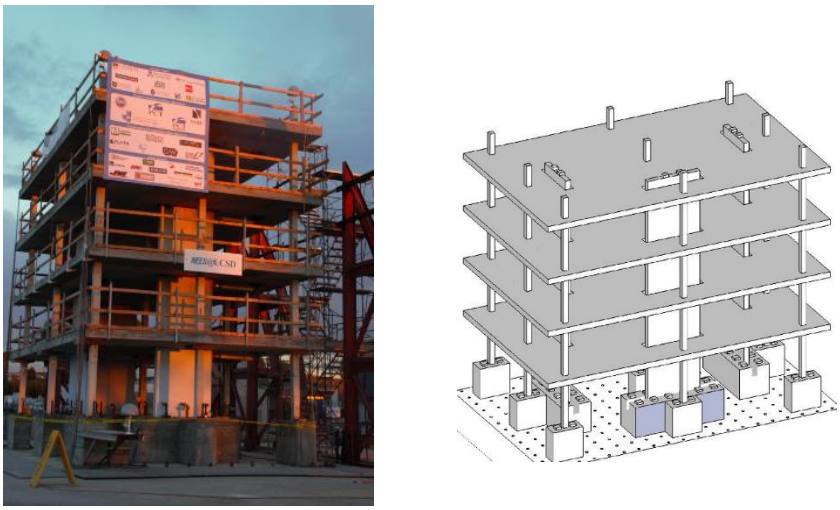


Fig.8. (a) Shake table specimen at NEES@UCSD; (b) 3D drawing of shake table specimen

Three test phases were executed: In Phase 1, the IFAS structure was tested; In Phase 2, the floors were "locked" down to the walls to simulate a typical floor anchorage in order to provide a direct comparison with Phase 1, in this phase, a special steel anchorage device, PSA, is used for allowing the wall free to rocking. In Phase 3, the hybrid rocking walls were "locked" into the foundation to simulate a reinforced concrete shear wall structure (employing high strength reinforcement). Through these phases, the structure was subjected to 22 strong ground motions.

6.1 Earthquake Introduction

The structure is tested for two design sites: Berkeley site and Seattle site with changing PT ratio and FD slip load. Two earthquakes were selected for this shake table test, which are listed in table 1.

Three different earthquakes intensity are selected. The first one is service level earthquake (SER), the second is the design based earthquake level (DBE), the third is the maximum considered earthquake level (MCE).

The time history of the selected earthquakes and the response spectrum of the selected earthquakes are shown in Fig. 21 and Fig. 22.

Table 1. Earthquake used in the test

EQ ID	PGA [g]	Earthquake	Date	Station	Component	Magnitude
SE05	0.59	Imperial Valley	1979/10/15	El Centro Array #5	140	6.5
BE05	0.41	Loma Prieta	1989/10/17	Los Gatos Presentation Center	000	7.0

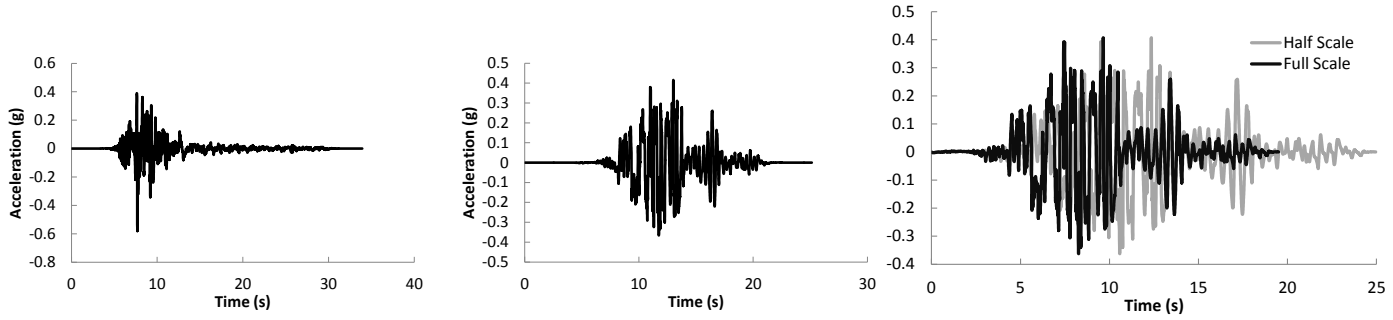


Fig.9. (a) SE05 (b) BE05 (c) Input ground motion

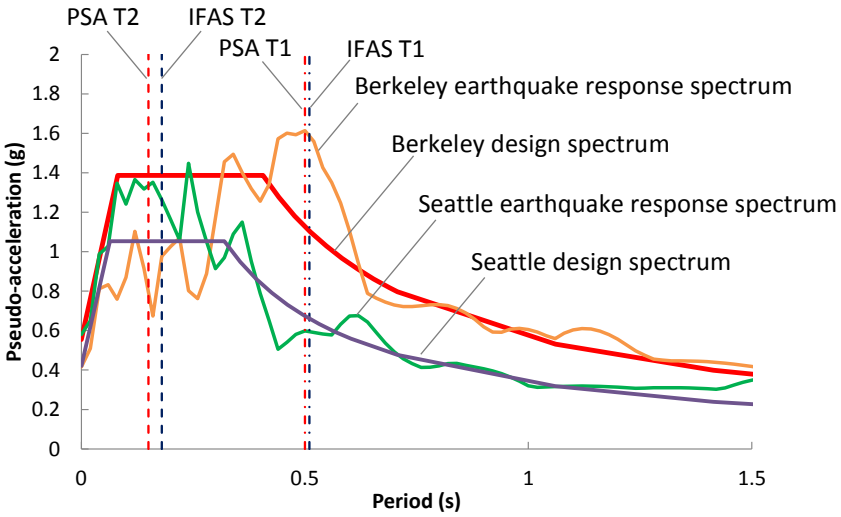


Fig.10. Design spectrum and response spectrum

Three phases of testing was done with the above mentioned ground motions. The schedule of the testing and its details is shown below.

6.2 Shake Testing Schedule

Phase 1	Structure with Inertial Force-Limiting Floor Anchorage System	Fri-Tue Dec 6-10, 2013
Phase 2	Precast Hybrid Rocking Wall System	Mon-Wed Dec 16-18, 2013
Phase 3	Precast Emulative Shear Wall with High Strength Reinforcement	Mon-Wed Jan 13-15, 2014

6.3 Testing details

Phase	TEST ID	Date	EQ name	Intensity	Wall PT Ratio (EW/NS)	FD slip load
Phase 1	1-W1	Dec 10th 2013	White Noise	3% WN	0.60/0.60	12 K / 9K
	1-1		Berkeley Service	0.67 BE05	0.60/0.60	12 K / 9K
	1-W2	Dec 11th 2013	White Noise	3% WN	0.60/0.45	19 K
	1-2		Berkeley Service	0.67 BE05	0.60/0.45	19 K
	1-3		Seattle DBE	SE05	0.60/0.45	19 K
	1-4		Berkeley DBE	BE05	0.60/0.45	19 K
	1-W3		White Noise	3% WN	0.60/0.45	19 K
	1-W4	Dec 12th 2013, am	White Noise	3% WN	0.45/0.45	25 K
	1-5		Berkeley Service	0.67 BE05	0.45/0.45	25 K
	1-6		Seattle DBE	SE05	0.45/0.45	25 K
	1-7		Berkeley DBE	BE05	0.45/0.45	25 K
	1-W5		White Noise	3% WN	0.45/0.45	25 K
	1-W6	Dec 12th 2013, pm	White Noise	3% WN	0.45/0.45	19 K
	1-8		Berkeley DBE	BE05	0.45/0.45	19 K
	1-9		Seattle MCE	1.5SE05	0.45/0.45	19 K
	1-10		Berkeley MCE	1.5BE05	0.45/0.45	19 K
	1-W7		White Noise	3% WN	0.45/0.45	19 K
	1-W8	Dec 19th 2013	White Noise	3% WN	0.60/0.60	25 K
	1-11		Berkeley DBE	BE05	0.60/0.63	25 K
	1-12		Berkeley MCE	1.5BE05	0.60/0.63	25 K
	1-W9		White Noise	3% WN	0.60/0.63	25 K
	1-W10	Dec 20th 2013	White Noise	3% WN	0.60/0.67	19 K
	1-13		Berkeley DBE	BE05	0.60/0.67	19 K
	1-14		Berkeley MCE	1.5BE05	0.60/0.67	19 K
	1-W11		White Noise	3% WN	0.60/0.67	19 K
Phase 2	1-W12	Jan 10th 2014	White Noise	3% WN	0.63/0.70	PSA
	1-15		Seattle DBE	SE05	0.63/0.70	PSA
	1-16		Berkeley SER	0.67BE05	0.63/0.70	PSA
	1-17		Berkeley DBE	BE05	0.63/0.70	PSA
	1-W13		White Noise	3% WN	0.63/0.70	PSA
	1-W14	Jan 17th 2014	White Noise	3% WN	0.63/0.64	roller bearing
	1-18		Berkeley DBE	BE05	0.63/0.64	roller bearing
	1-W15		White Noise	3% WN	0.63/0.64	roller bearing
	1-19		Berkeley MCE	1.5BE05	0.63/0.64	roller bearing
	1-W16		White Noise	3% WN	0.63/0.64	roller bearing
Phase 3	1-W17	Jan 23th 2014	White Noise	3% WN	0.63/0.64	Traditional wall
	1-18		Berkeley DBE	BE05	0.63/0.64	Traditional wall
	1-W18		White Noise	3% WN	0.63/0.64	Traditional wall
	1-19		Berkeley MCE	1.5BE05	0.63/0.64	Traditional wall
	1-20		Berkeley MCE	1.5BE05	0.63/0.64	Traditional wall

	1-W19		White Noise	3% WN	0.63/0.64	Traditional wall

7. Test Result

7.1 Shake Table Test Specimen General Response

The change of the structural period was first checked after the SKT (Figure 1). The period change indicated the damage occurred in the structure.

In Fig.11, the structural period became lower at some point which is not reasonable for a structure suffered sequent damage. This is because some of the damaged structure state has been changed at that point, which is also indicated in Fig.11.

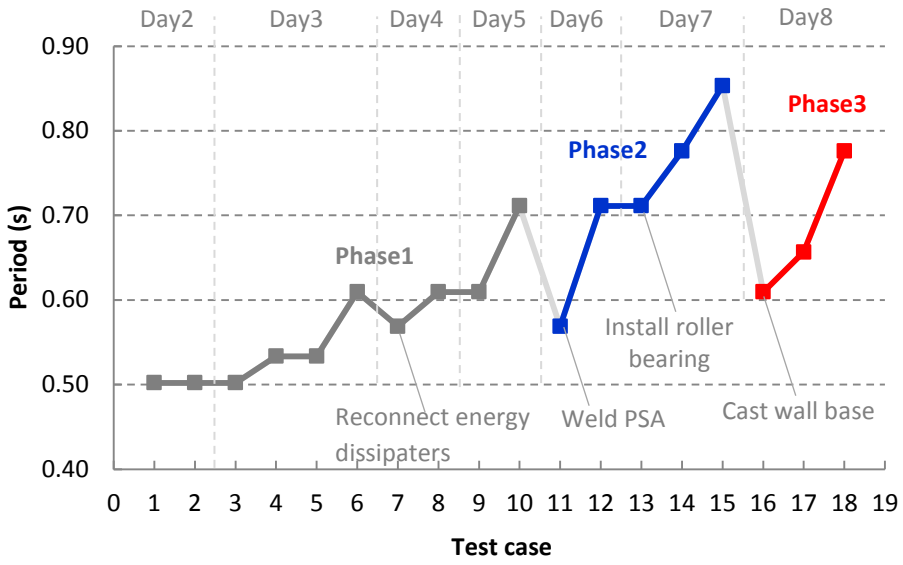


Fig.11. Structural period change

7.1.1 Lateral Force Resisting System (LFRS) Behavior

The LFRS used in this SKT is the hybrid rocking wall (HRW), which is able to suffer small damage for continuous earthquake excitations.

In the test, the uplift of the HRW at both of the two ends was recorded, which is able to be converted to the HRW base rotation. The strain gages were pasted to the posts for the energy dissipaters (ED), which is able to be converted to the force in ED. The pressure transducers were inserted into the Jacks used on the top of the wall, which is able to be converted to the post tension (PT) bar force. By using the above, the HRW base rotation and HRW base moment is able to be obtained.

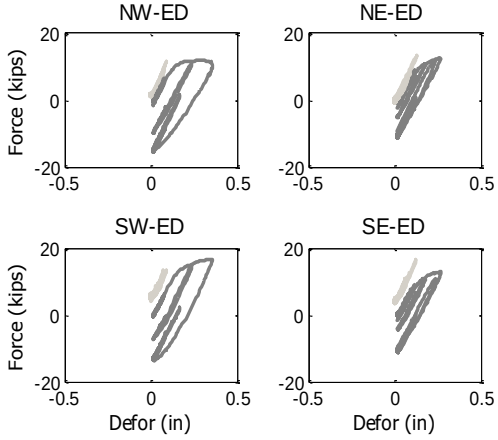
EW direction is the table motion direction, the EW wall is supposed to provide the main lateral force resisting capacity during the test. The comparison for the EW wall hysteresis in DBE during the test is shown in Figure 12. The positive rotation defined is when EW wall rocks eastward.

Test 4 and Test 11 are compared in Fig.12, they were both in DBE level and their PT ratio in the EW wall was both about 0.6. Test 4 was the first EQ intensity got DBE, Test 11 was the test which EQ intensity got DBE after several big ground motions, but ED was replaced at the beginning of Test 11. In Fig.12(c) and Fig.12(e), the wall moment got increased when the rotation near 0 in Test 11, that is caused by the not accurate neutral axis prediction after the wall suffered damage. But this will not generate big influence on the wall general hysteresis response when the wall got higher base rotation.

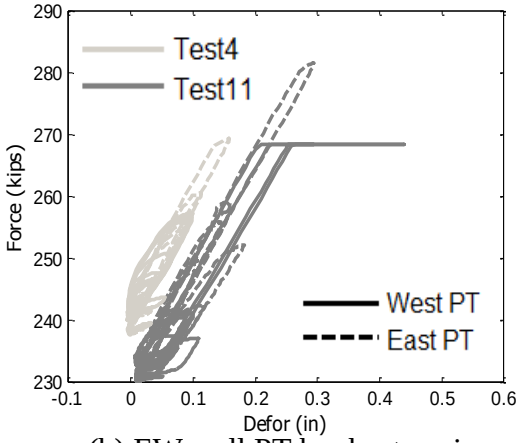
From the comparison, it indicated that the EW wall deformed more in Test 11, there are three reasons for this. The first one is the FDs' slip force in Test 11 was targeted to 25 kips, instead, it was targeted to 19 kips in Test 4, so the force acted on the EW wall was bigger in Test 11 compared to Test 4. The second reason was the initial PT of EW wall in Test 11 was a little smaller than Test 4, which would result to a lower decompression moment. The third reason was the continuous test would introduce inevitable damage to the wall toe area, which would shift the neutral axis inside the wall and reduce the wall moment resistance.

The ED nearly stayed in elastic range in Test 4 but got obvious yielding in Test 11 (Fig.12(a)), this yielding in ED will made negative moment contribution to the wall base moment (Fig.12(d)), which is another reason why the wall total moment resistance was smaller in Test 11. Because of the reading limitation, the pressure transducer reading in the west PT bar got missed when EW wall rocked up eastward (Fig.12(b)), that is the reason why there was a small stiffness reduction in the wall hysteresis when rocking bigger than 0.005 rad eastward (Fig.12(c),(e)).

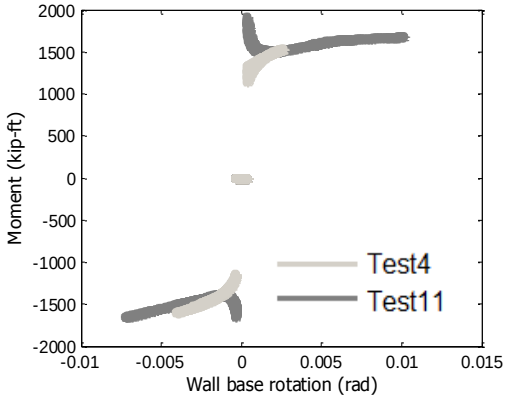
Though the wall moment resistance in Test 3 was bigger than Test 11, the difference between these two is small, which indicates that the EW wall got small damage in the continuous test and good repeatability.

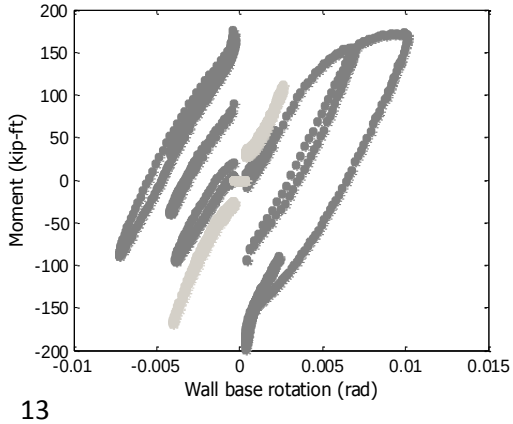


(a) EW wall ED hysteresis

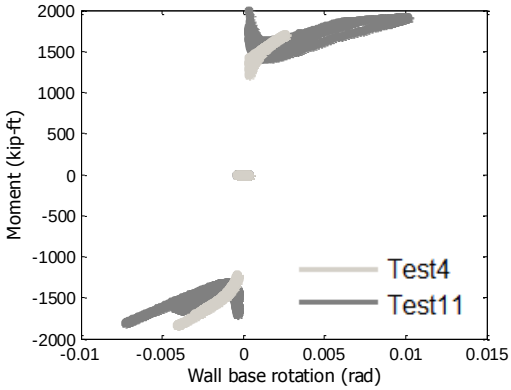


(b) EW wall PT bar hysteresis





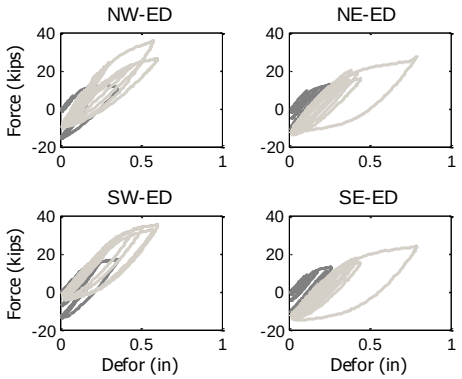
(c) EW wall hysteresis - PT (d) EW wall hysteresis – ED



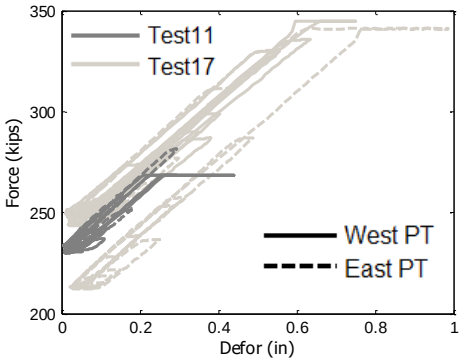
(e) EW wall hysteresis – Total

Fig.12. EW wall hysteresis response comparison in DBE in Phase 1

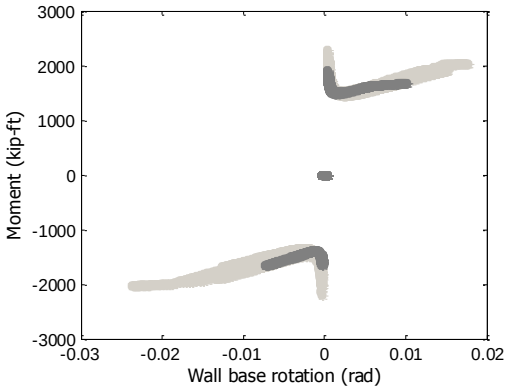
The EW wall response between Test 11 and Test 17 (both DBE level, Test 11 is for phase 1, Test 17 is for phase 2) is compared in Figure 13. Fig.13(a) indicated that ED did suffer strain hardening between Test 11 and Test 17 which generated much higher yielding strength. Fig.13(f) indicates that the wall in Test 17 got bigger rotation than which in Test 11, which is also indicates that the wall in Test 17 will suffer bigger damage. Fig.13 (e) indicates that the wall hysteresis is similar in both Test 11 and Test 17, which also indicates that the little damage occurred in the EW wall during the test.

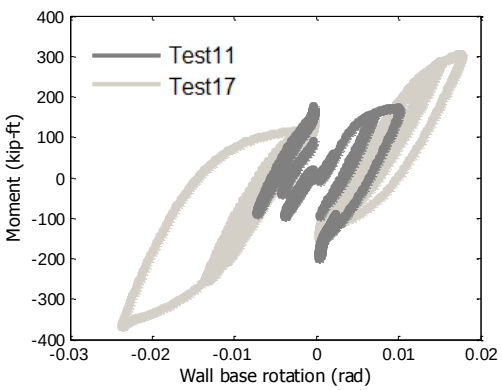


(a) EW wall ED hysteresis

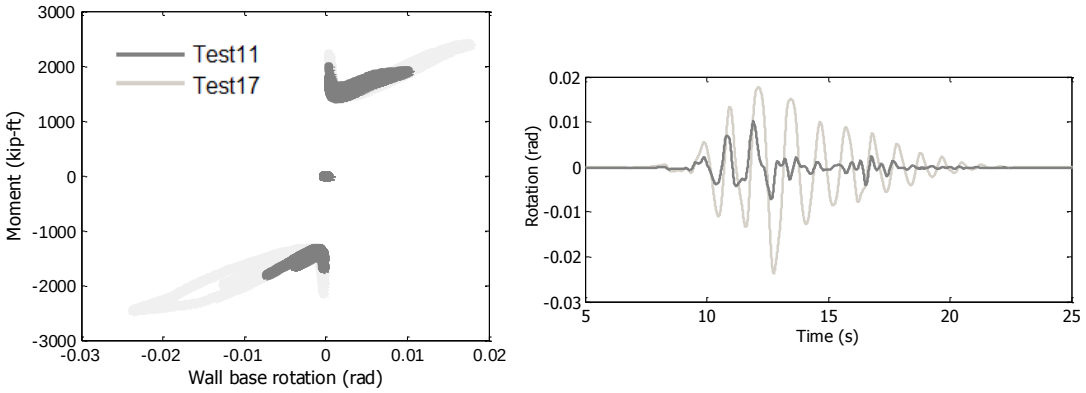


(b) EW wall PT bar hysteresis





(c) EW wall hysteresis - PT (d) EW wall hysteresis – ED



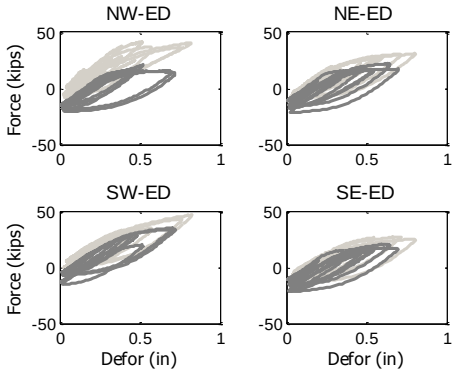
(e) EW wall hysteresis - Total (f) EW wall base rotation time history

Fig.13. EW wall hysteresis response comparison in DBE between phase 1 & 2

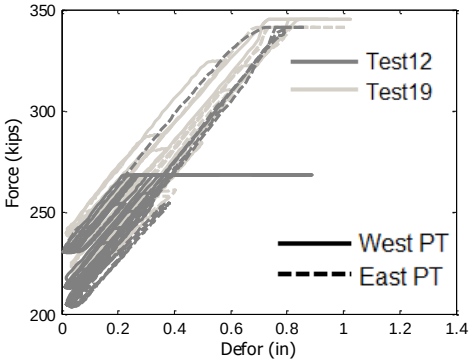
The HRWs' response comparison between phase 1 and phase 2 under MCE is shown in Figure 4. Because the strain gage on NW-ED broke in Test19, that's why NW-ED hysteresis looks strange in Fig.14(a). Fig.14(b) indicates the missing data in getting PT bar force from pressure transducer readings, which is the reason for secondary stiffness change in Fig.14(c). Fig 14(e) shows obvious wall moment difference when the wall rocked eastward, this was mainly caused by the missing recording data for PT bar force of test 12 showing in Fig.14(b).

Fig.14(f) shows EW wall rotation was only a little bigger in Test19, which looked like doesn't show the advantage of IFAS system in reducing LFRS damage. This is because in Test19, roller bearing was added to EW wall purposely to make the slab and EW wall rigidly connected in horizontal direction. Because the gap between roller and steel plate (the one for prevent roller rolling out) is limited by the slab thickness, when the roller rolled to the steel plate, the wall became sliding on the roller surface instead of rolling, which generated higher friction force between EW wall and slab and this friction force tended to drag EW wall downward from rocking. In the other word, the rocking of EW wall needs much more force than before without adding roller bearing. At the same time, there is no roller bearing between slab and HRWs in NS direction, so the structure twisting response was triggered.

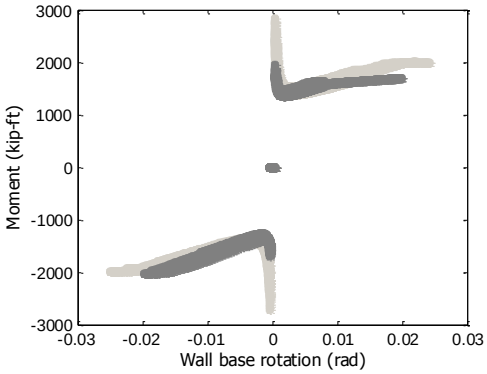
In the real world, the earthquake propagation direction is not be able to predicted, the structure is easy to suffer an earthquake which comes with an angle to its two normal building direction. In order to test this structural behavior under earthquake in this one-direction shake table, the structure twisting response is triggered purposely by adding roller bearing. This could be seen in Fig.14(g) and Fig.14(h) obviously, the NS HRWs rocked more in Test 19 than Test 12. So if the structure response in both two directions is evaluated together, their EW wall rocked similar and NS walls rocked more in Test 19, which also indicates the advantage of IFAS in the structure for reducing the LFRS damage.



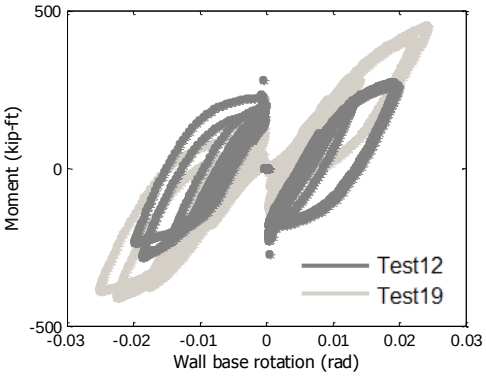
(a) EW wall ED hysteresis



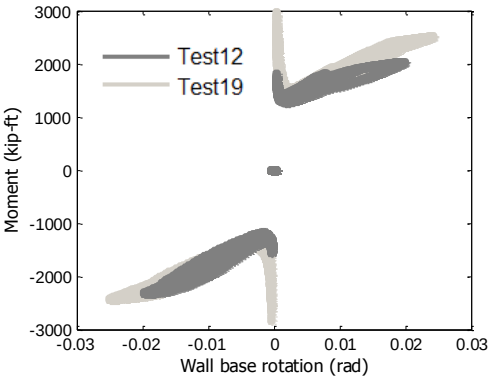
(b) EW wall PT bar hysteresis



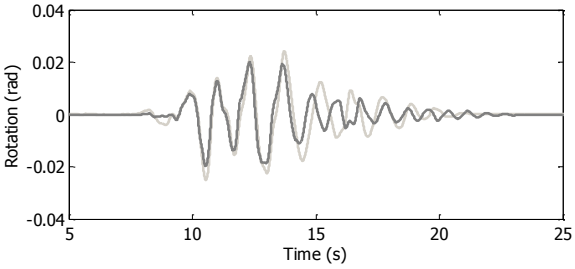
(c) EW wall hysteresis - PT



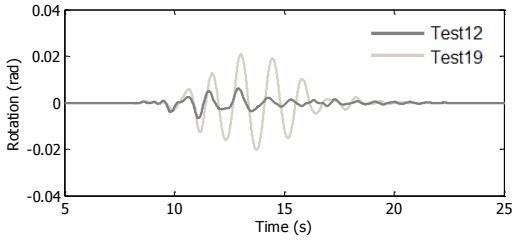
(d) EW wall hysteresis – ED

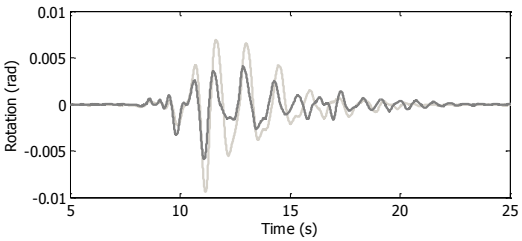


(e) EW wall hysteresis - Total



(f) EW wall base rotation time history



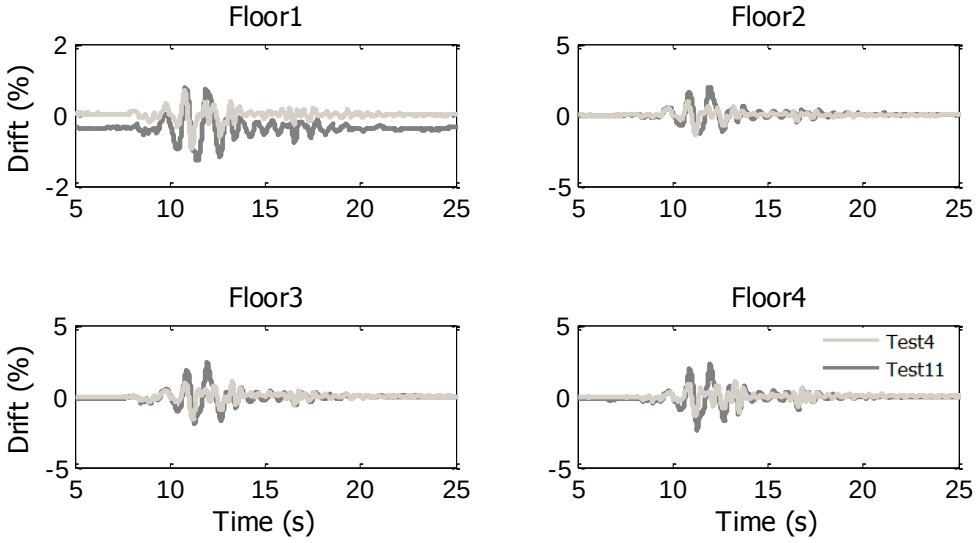


(g) West wall base rotation time history (h) East wall base rotation time history

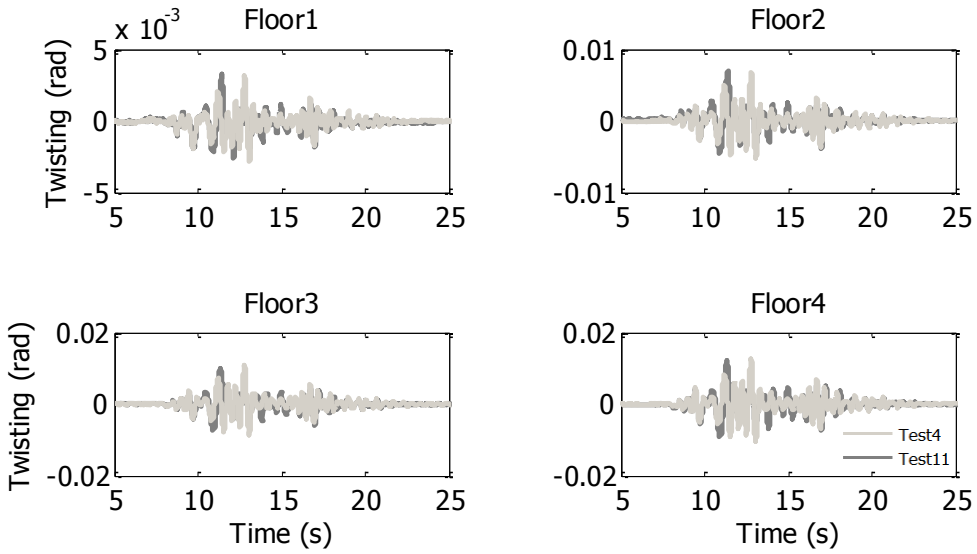
Fig.14. EW wall hysteresis response comparison in MCE between phase 1 & 2

7.1.2 Gravity Load Resisting System (GLRS) Behavior

GLRS were designed for resisting gravity load only in the test. The employed precast concrete columns were used for providing repeatability. The GLRS response IS shown in Figure 5. The GLRS inter-story drift in Test 11 was bigger than Test 4, this probably caused by the damage in GLRS columns, which got lower lateral resistance. The slab twisting in Test 4 and Test 11 were similar to each other.



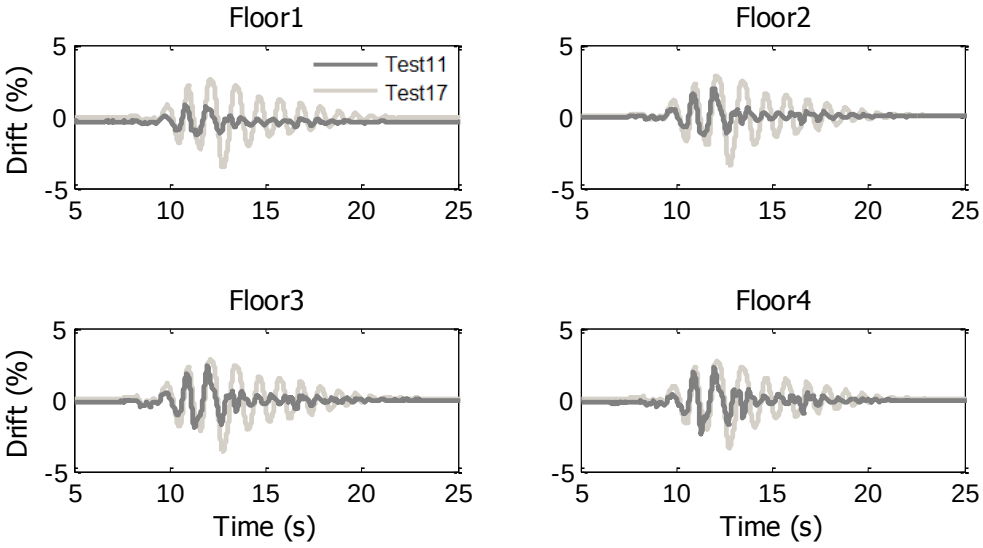
(a) GLRS inter-story drift time history at slab middle



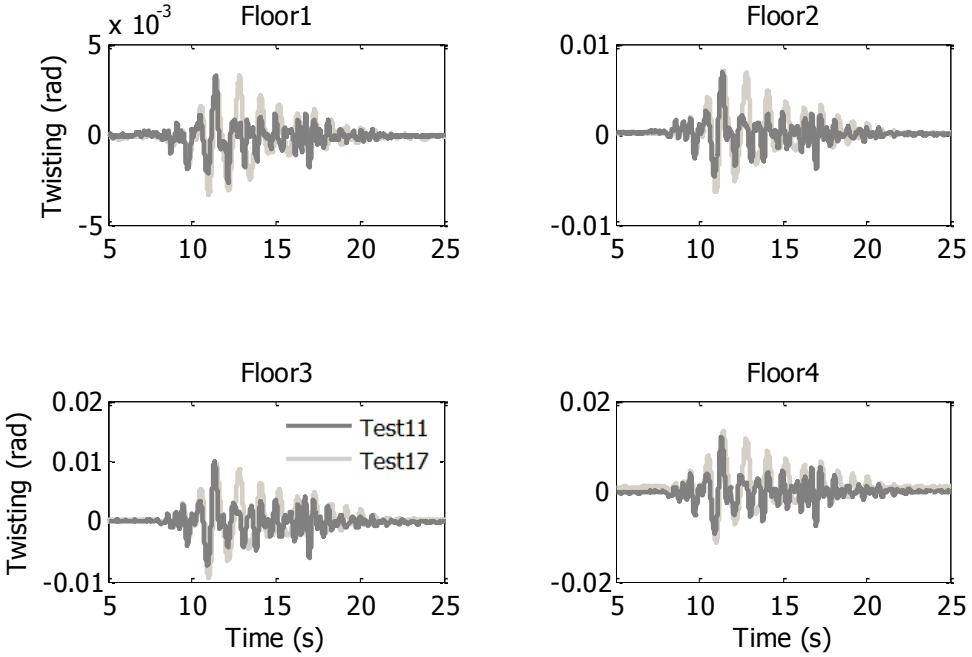
(b) GLRS slab twisting time history

Fig.15. GLRS response comparison in DBE in Phase 1

The GLRS response between Test 11 and Test 17 (both DBE level, Test 11 is for phase 1, Test 17 is for phase 2) is compared in Figure 16. It indicated that the GLRS inter-story drift in phase 2 was bigger than phase 1.



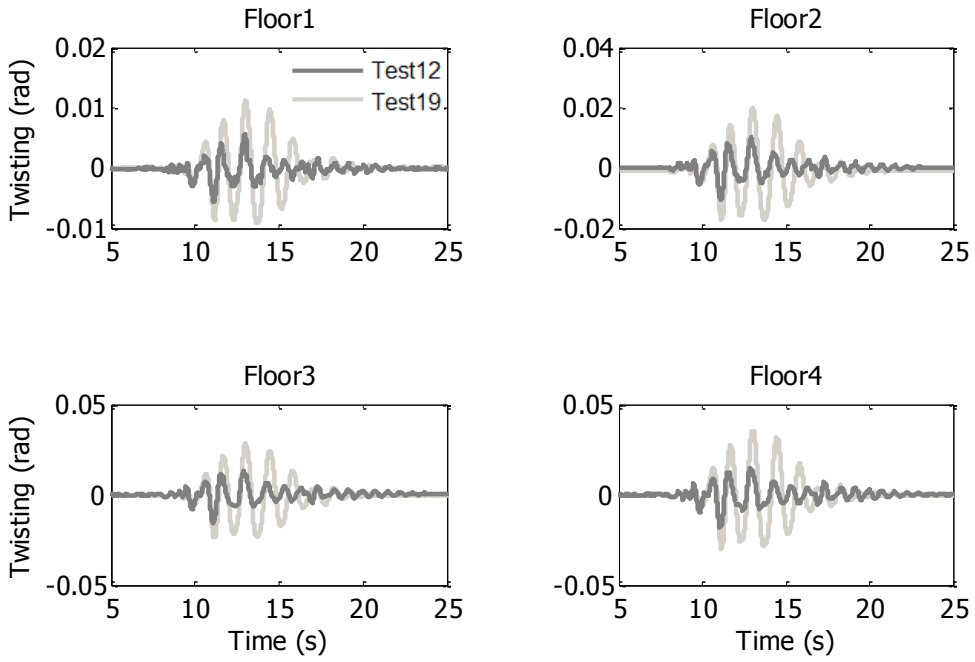
(a) GLRS inter-story drift time history at slab middle



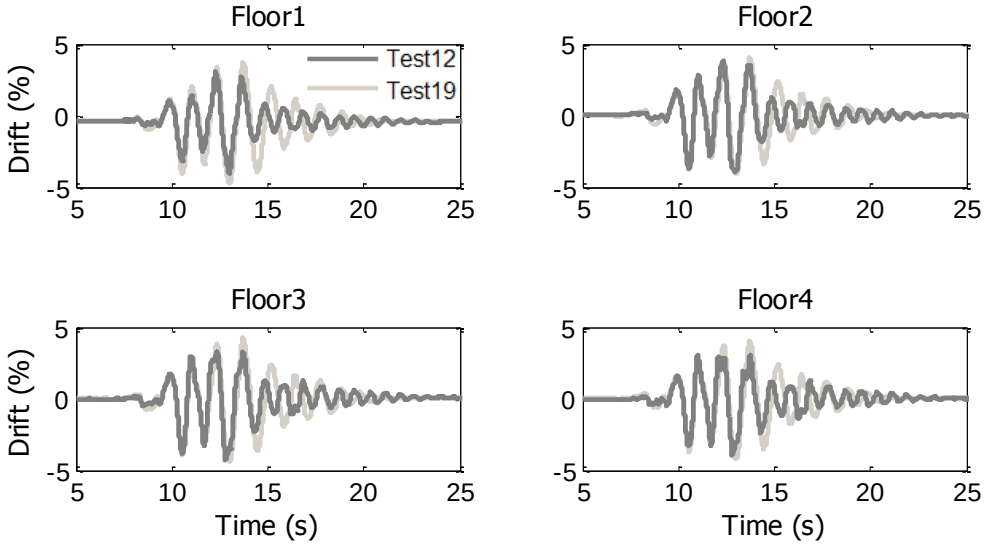
(b) GLRS slab twisting time history

Fig.16. GLRS response comparison in DBE between phase 1 & 2

The GLRS response comparison between phase 1 and phase 2 under MCE is shown in Figure 17. It indicated that the GLRS inter-story drift are similar in both cases, but the twisting in Test 19 was bigger than Test 12, about as twice as Test 12.



(a) GLRS inter-story drift time history at slab middle



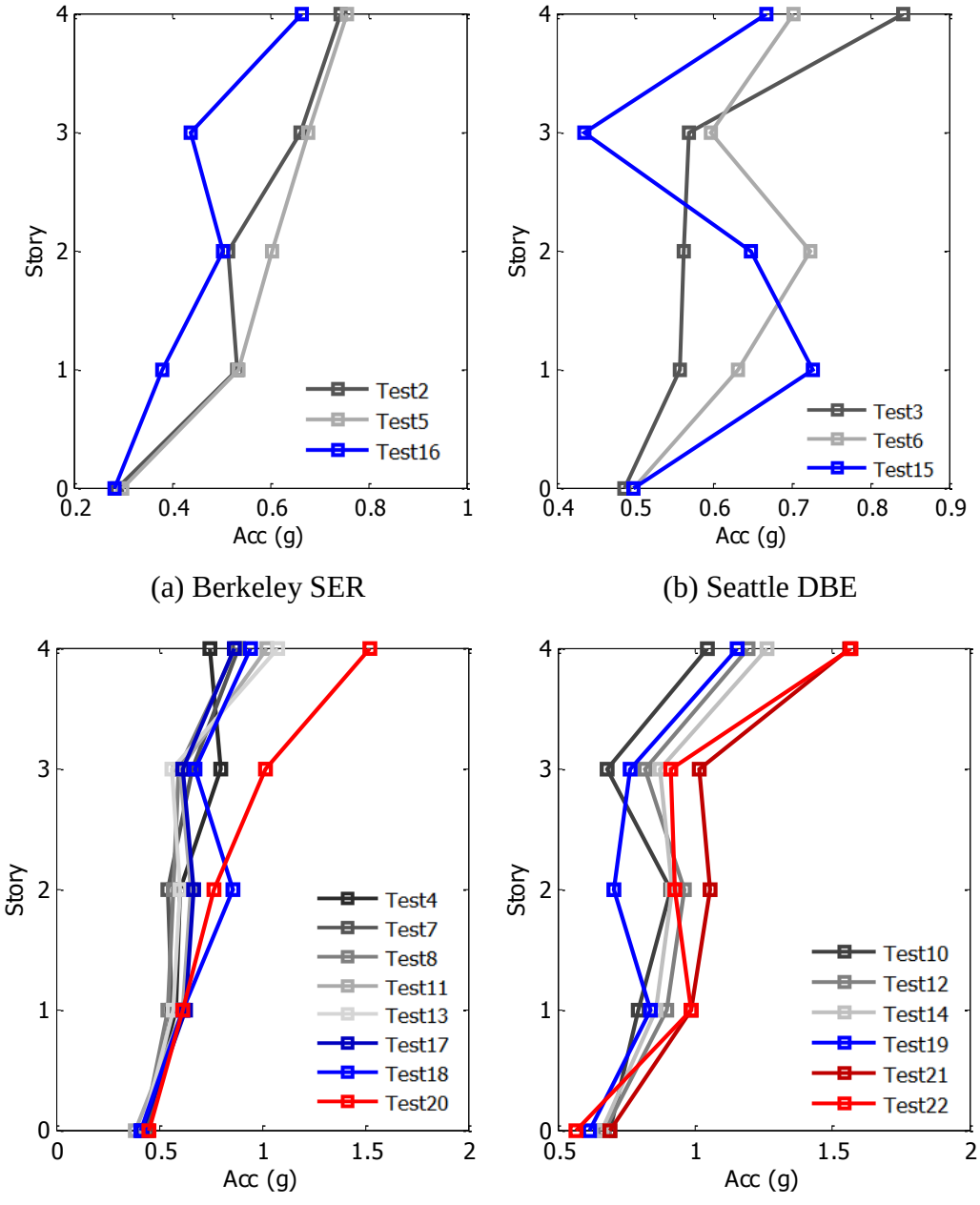
(b) GLRS slab twisting time history

Fig.17. GLRS response comparison in MCE between phase 1 & 2

7.2 Acceleration Response Comparison

The acceleration response is compared in the different test phases. IFAS is proposed to not only reduce the LFRS damage, but also reduce the GLRS acceleration.

Figure 8 shows the GLRS EW direction acceleration response profile comparison. Phase 1 results were shown in gray color, phase 2 results were shown in blue color, phase 3 results were shown in red color. The comparison did show that the acceleration response of the structure with rocking wall using IFAS system was lower than the traditional shear wall reinforced concrete structure. But the acceleration of the structure with rocking wall using IFAS system is not lower than the structure with traditional rocking wall.



(a) Berkeley SER (b) Seattle DBE

(c) Berkeley DBE (b) Berkeley MCE

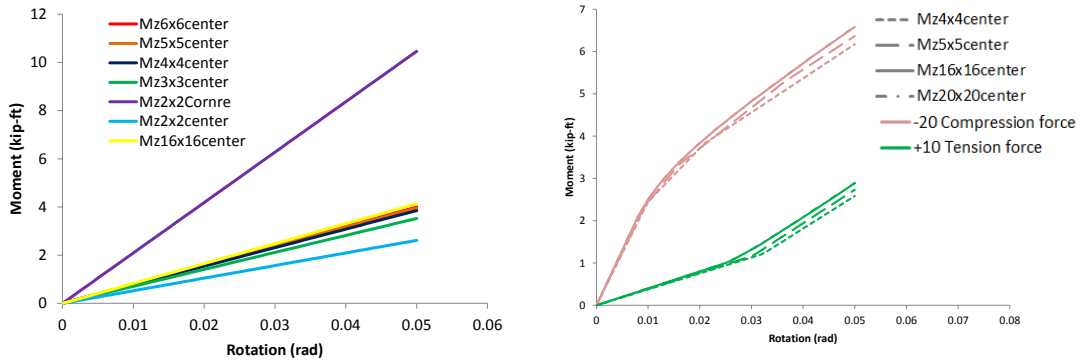
Fig.18. GLRS EW acceleration response in phase 1, 2 & 3

7.3 Comparison between Numerical Model and Test Results

Structural response Comparison – checking the rubber bearing model

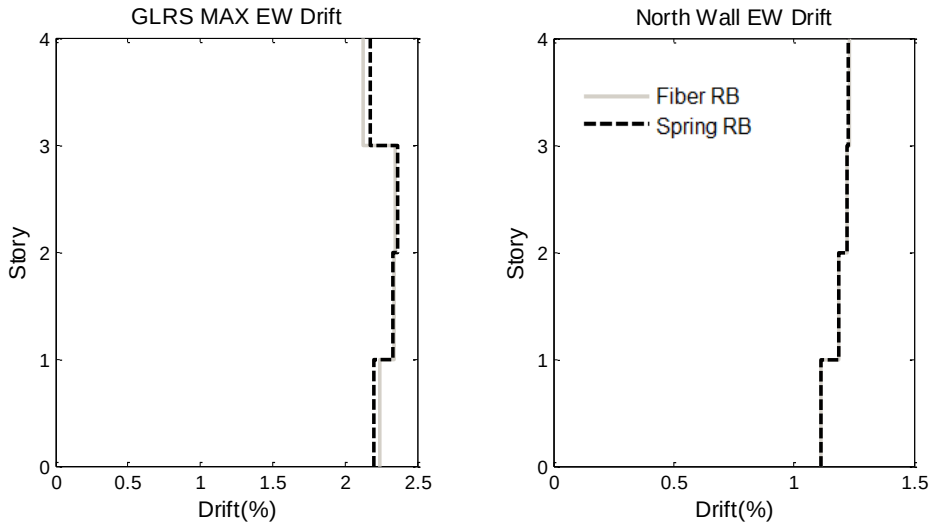
The selected structural model is 7ksi wall strength, 5ksi column and slab. The confined model for shear wall and column is modified Kent-Park model. FD slip load is 25 kips, BRB strength is 9.9 kips.

Fig.19(a) is the pure moment-rotation analysis on RB fiber model, it shows that when the mesh of slab is bigger than 4x4, the analysis results are similar to each other. Fig.19(b) is the moment-rotation analysis under axial load. -20 kips and +10 kips has been chosen for this analysis. It shows that 16x16 fiber mesh is good to model the behavior, but it is too fine. So finally 5x5 RB fiber mesh was chosen for the structural dynamic analysis. The response comparison is shown in Figure 19. It indicated that the use of uncoupled springs for simplifying RB is able to be accepted.

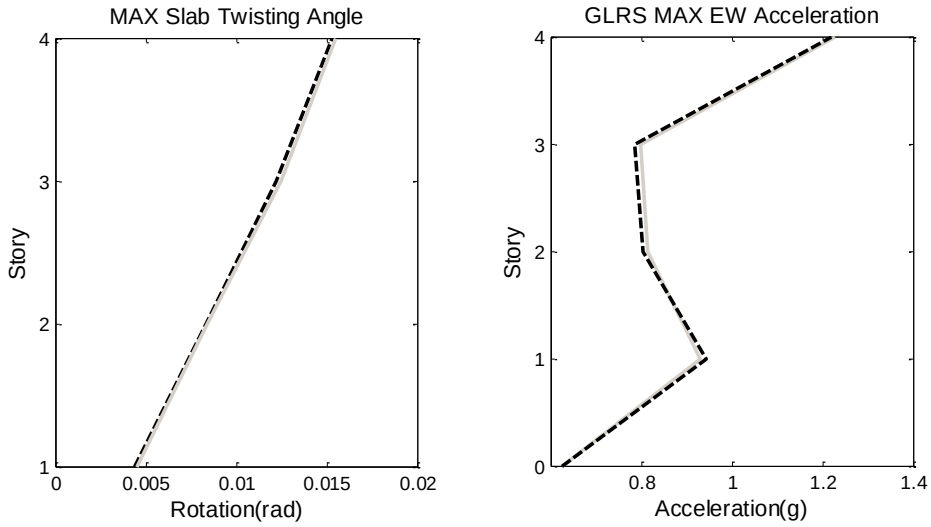


(a) Pure Moment-Rotation (b) Moment-Rotation under axial force

Figure 19. Moment-rotation of multi-fiber rubber bearing



(a) MAX GLRS EW drift (b) MAX North wall drift



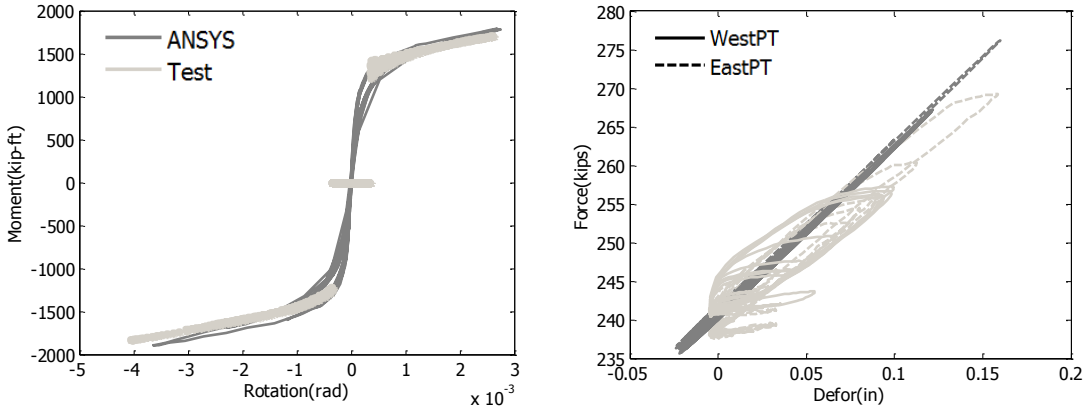
(c) MAX GLRS Twisting (d) MAX GLRS EW Acceleration

Fig.20. Structure response comparison

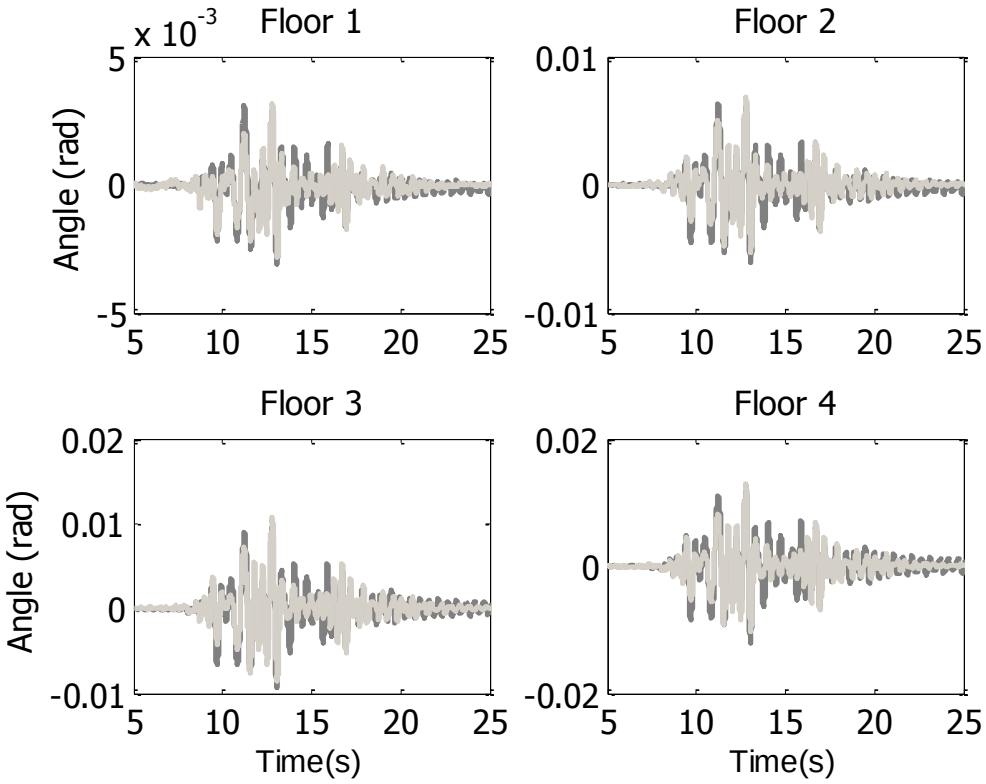
Dec. 11th DBE Test

Three motions were input to the shake table on Dec. 10th. One is a pilot test, one is a white noise test, and one is Berkeley service EQ (Test 1). Three ground motions were input to the shake table on Dec. 11th, two white noise were used at the beginning and end of the whole set, and three EQ motions were used (Test 2 ~ Test 4). Because all the EQ intensity was small, the structure was posed to suffer small damage after the test.

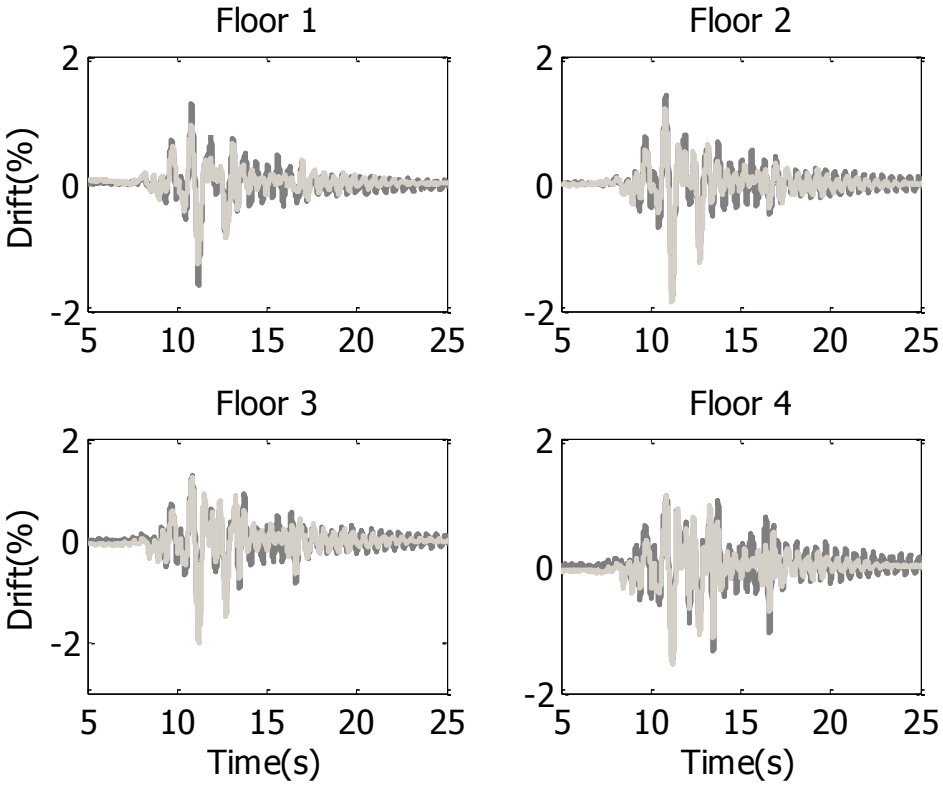
Test 4 is Berkeley DBE intensity level. And the comparison is shown in the following.



(a) North wall hysteresis (b) North wall PT bar hysteresis



(c) GLRS twisting



(d) GLRS south column drift

Fig. 21. Comparison between test and numerical analysis in Test 4.

8. Conclusions:

(1). IFAS is able to reduce the LFRS damage when compared to the traditional HRW structure.

(2). IFAS is able to reduce the structural acceleration response when compared to the tradition shear wall structure. Which is able to reduce the non-structural components damage.

(3) Friction damper and BRB used in the test specimen worked as we expected. The slip load in FD is able to be controlled by changing the pretension force in the bolts.

(3). Changes in slip load in FD will influence the HRW base rotation response. The higher the slip load, the bigger the wall base rotation.

(4). The change in PT level of HRW is able to influence the HRW decompression moment, the higher the PT level, the higher the HRW decompression moment.

(5) The simplified rubber bearing model (three-uncoupled springs) is able to reflect the structural response as a real rubber bearing model (fiber model).

(6) The 3D analysis model in ANSYS is able to predict the test response, but more calibration is needed.