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SHAKE-TABLE TESTS OF A THREE-STORY MASONRY-INFILLED RC FRAME

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ABSTRACT

This paper presents the shake table tests of a three-story, two-bay, 2/3-scale, masonry-infilled RC frame. The specimen was representative of the construction practice in California in the 1920s. It had a non-ductile frame design and was infilled with unreinforced masonry panels including a wall with a window opening in one bay and solid wall in the other bay in each story. The structure was subjected to a series of 14 ground motions of increasing intensity before it practically collapsed. The design of the specimen, the testing sequence, and the major findings are discussed here.

Introduction

Reinforced concrete (RC) frames with masonry infill walls can be frequently found in earthquake-prone areas around the world. Even though unreinforced masonry infill walls are often considered as non-structural elements, they can interact with the bounding frame under seismic excitation and influence the load-resisting mechanism and failure pattern of the structural system. The performance of these structures under seismic loads has been studied extensively both analytically (Stafford Smith 1966 and Al-Chaar et al. 2002) and experimentally (Zarnic et al. 2001; Hashemi and Mosalam 2006). Although there has been evidence that masonry infills can be beneficial by increasing the stiffness and strength of RC frames, there is a lack of experimental data from large-scale dynamic tests of multi-story, multi-bay infilled frames to evaluate the exact influence of the infill walls and the post-peak behavior of the structural system. Hence, the seismic behavior of such structures is still an intricate issue as the infills can interact with the bounding frame and influence the load-resisting mechanism and failure pattern of the structural system.

This paper presents the shake-table tests of a three-story, two-bay infilled RC frame conducted at the Large High Performance Outdoor Shake Table (LHPOST) located at the University of California, San Diego (UCSD) in the fall of 2008. The specimen was a 2/3-scale version of the external frame of a prototype structure designed according to the engineering

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practice in California in the 1920s and was the largest structure of this type ever tested on a shake table. The structure was instrumented with a total of 280 sensors and was subjected to a sequence of 44 dynamic tests, including 14 scaled earthquake records of increasing intensity. Issues associated with the design, scaling, construction, and testing of this specimen, as well as an overview of its performance and the earthquake-induced damage are discussed here.

Specimen Design

The three-story specimen corresponds to the exterior frame of a prototype structure designed to represent existing structures built in California in the 1920s. The employed design approach was similar to the allowable stress method and considered only gravity loads. Details on the design of the prototype are provided by Stavridis (2009). The specimen was scaled with a length scaling factor of 2/3 compared to the prototype structure. The dimensions of the frame and the layout of the steel reinforcement are presented in Fig. 1, while the cross sectional details of the RC members are shown in Fig. 2. The design of the columns was modified in view of a second specimen which would be tested to investigate the effectiveness of retrofit schemes consisting of overlays of Engineered Cementitious Composite (ECC) and Fiber Reinforced Polymeric (FRP) materials (Shing et al. 2009). The longitudinal reinforcement of the RC columns was increased from 1% that the exact scaling of the prototype would require to 2% at the first and 1.5% in the second story to provide sufficient capacity to resist the overturning moment, considering the strengthened panels of the retrofitted specimen. Such values of the longitudinal reinforcement ratio are not uncommon for existing structures as they are within the 1% and 4% limits used in the 1920s era. Moreover, lap splices were not included at the base of the bottom-story columns to prevent premature failure in the lap-splice region since this is not an expected failure for the prototype structure. Finally, to facilitate the construction, the roof beam was modified to be identical to the beams at the first two stories. The infill walls had two wythes of brick units built with Type N mortar consisting of 1 part Portland cement, 1 part lime and 5 parts sand. At each story, the specimen had a solid infill in one bay and an infill wall with a window opening in the other bay, as shown in Fig. 3(b).

Scaling Considerations

The specimen tested on the outdoor shake-table at UCSD corresponds to the external frame of the prototype structure. Since the prototype building has infill walls only along the exterior, the internal frames are significantly less stiff and weaker than the external frames and, therefore, their contribution to the lateral resistance of the structure can be ignored. As a result, the seismic mass carried by the external frames is larger than the gravity mass carried by the external frames. To simulate the behavior of the prototype structure, it is desired to maintain the gravity mass which generates the proper stresses on the RC members and the infill panels, but also generate the proper seismic forces which are induced due to the seismic mass. To account for the disparity between the gravity and seismic masses, it was decided to attach to the structure the mass representing the scaled gravity mass and appropriately scale the amplitude of the ground acceleration time-histories and compress the time to satisfy the similitude requirement. The scaling factors for the remaining quantities were determined according to the similitude requirement and are summarized in Table 1. The ground motion levels mentioned in the paper are with respect to the full-scale prototype structure and are appropriately scaled when applied on the specimen with the acceleration and time scaling factors.

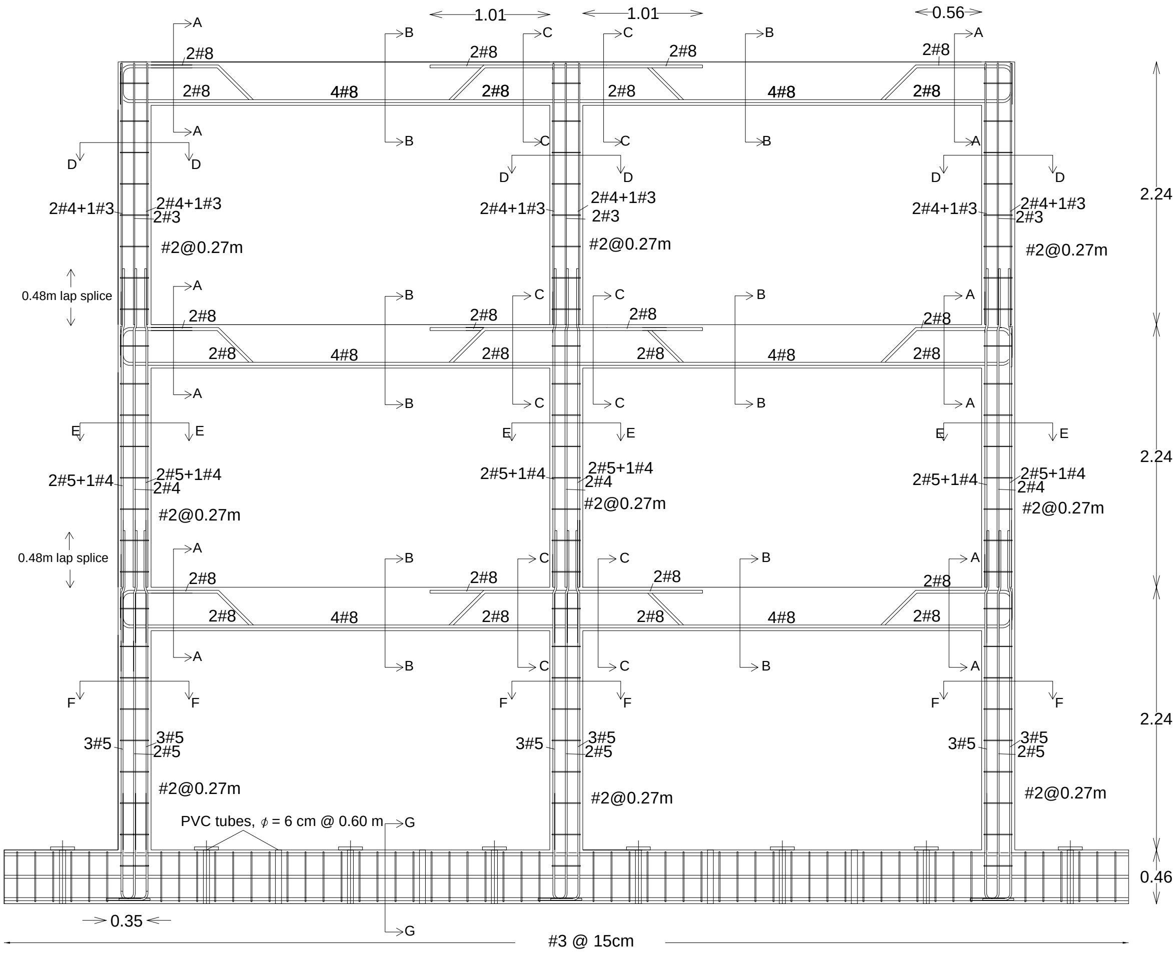


Figure 1. Design of three-story specimen tested at UCSD (dimensions in m).

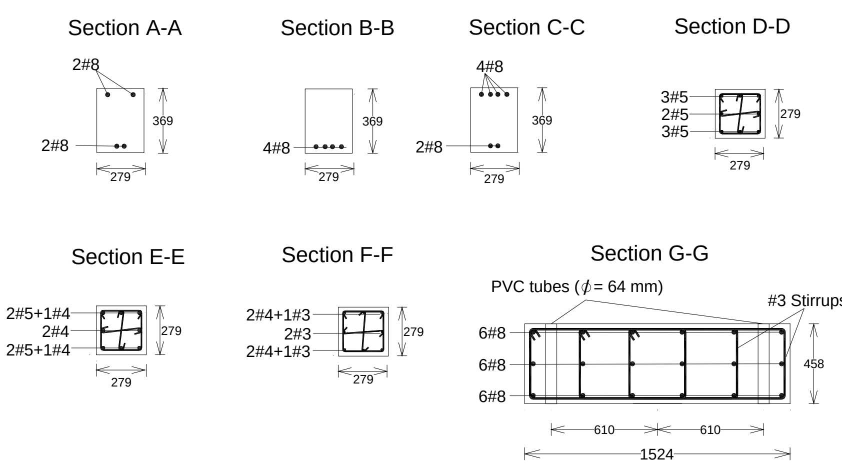


Figure 2. Cross sections of RC members (dimensions in mm).

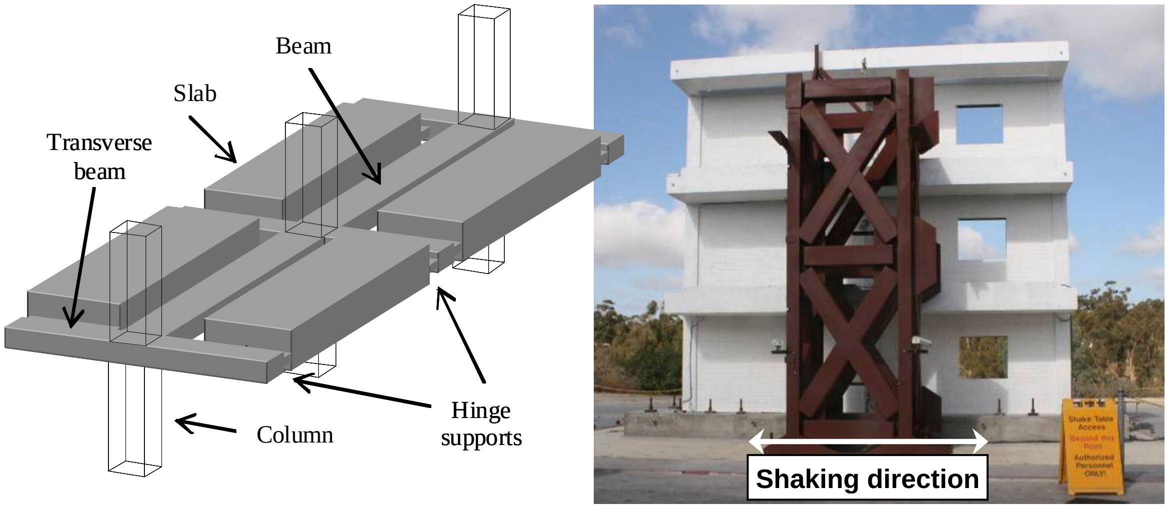
Table 1. Scaling factors.

Quantity	Scale Factor
Length	2/3
Elastic modulus	1.00
Seismic Mass	0.20
Seismic acceleration	2.27
Force	4/9
Stress & Strain	1.00
Time	0.542
Frequency	1.85
Gravity Mass	4/9
Gravitational Acceleration	1.00

Test Setup

The mass required to simulate the gravity loads was attached to the frame, through the construction of thick slabs at levels 1, 2, and 3, as shown in Fig 3(a). Although the specimen simulated an external frame of the prototype, the slabs extended to both sides of the frame to provide symmetry to the test structure and prevent the undesired behavior that an eccentric center of mass would cause under seismic excitation. This adjustment was necessary to represent the behavior of the three-dimensional prototype structure. In the latter, the centers of rigidity and mass would coincide and prevent the out-of-plane motion of the prototype under uniaxial excitation within the plane of the frame.

The slabs simulating the gravity mass were designed to be relatively short but rather thick, having an out-of-plane length of 0.97 m (38 in.) and a height of 0.48 m (19 in.), to prevent issues related to the structural stability introduced by long cantilever slabs. The roof slabs had smaller dimensions due to the reduced amount of gravity load applied at the roof of the prototype. If directly connected to the beams of the frame, the thick slabs would act as a single, unrealistically rigid, beam which would prevent any deformation of the beams and joints of the frame. To avoid this undesired constraint, a gap of 51 mm (2 in.) was introduced between the slabs and the beams and the slabs were only connected to the frame through three transverse beams at each floor. Moreover, slots were introduced at the connection of the slabs with the transverse beams. The slots were 51 mm (2 in.) wide and were arranged in such a way to form a concrete joint of 102 mm (4 in.) height which would connect the slabs to the transverse beam. The reinforcement connecting the slabs to the transverse beams was bended to form an ‘X’ inside the joint. This configuration resulted in the creation of four slabs per floor. The slabs were connected with the frame through pin supports which enabled the transfer of gravity and inertia forces generated by the mass of the slabs. The minimal moment capacity of the joints would minimize the rotational constraints imposed to the longitudinal beams of the frame, as they would crack to allow the relative rotation between the transverse beams and the slabs.



(a) beam-slab configuration. (b) front view

Figure 3. Shake-table specimen.

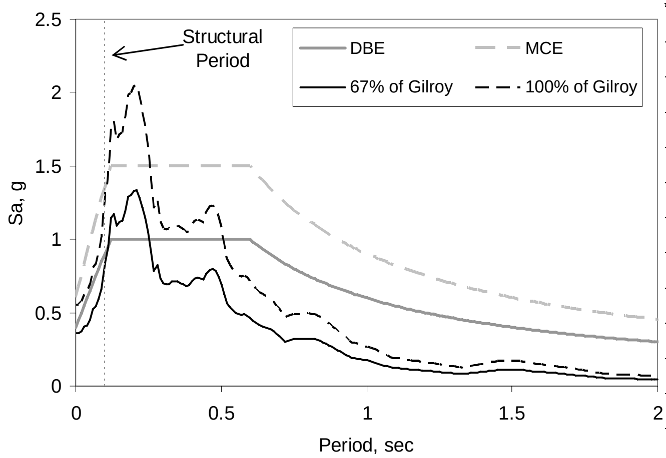
The RC frame and the masonry infills were constructed by professional contractors. The frame was constructed first in four stages that lasted one week each. Then, the masonry walls were built within one week. After the RC frame was constructed, two steel towers were secured on the shake table on the north and south side of the structure to prevent a potential out-of-plane collapse of the specimen during severe shaking. The steel towers did not interact with the structure during the tests as their placement allowed a 13 mm (0.5 in.) gap on each side of the specimen. The final test set-up is illustrated in Fig. 3(b). The instrumentation array deployed on the specimen and steel towers included 144 strain gages, 71 displacement transducers, and 59 uniaxial accelerometers, while 11 cameras were used to monitor and record the structural behavior.

Testing Protocol

The specimen was subjected to a sequence of 44 dynamic tests. The main goal was to gradually damage the structure by subjecting it to seismic motions of increasing intensity. At the beginning and at the end of each testing day, acceleration recordings were obtained from the ambient vibration of the structure while the shake table was resting on six static vertical bearings without any hydraulic support. Moreover, white noise base excitation tests were performed between consecutive earthquake records. These had Root Mean Square (RMS) amplitudes of 0.03 and 0.04g. The ambient vibration recordings and the low-amplitude white-noise tests were conducted so that the modal parameters of the structure could be identified. This process is important, since the change of the modal parameters can be a good indication of the induced damage (Moaveni et al. 2010).

The core of the testing sequence involved 14 scaled earthquake motions which were obtained by scaling the acceleration time histories recorded at the Gilroy 3 station during the 1989 Loma Prieta and at El Centro during the 1940 Imperial Valley earthquake. When applied to the test specimen, the accelerograms were compressed in time and scaled in amplitude based on the scaling factors shown in Table 1. For structures with a natural frequency close to 0.1 sec which is the estimated frequency of the prototype infilled frame considered here, 67% of the Gilroy 3 motion corresponds to a design basis earthquake (DBE) for Seismic Design Category (SDC) D (FEMA P695, ASCE 41-06). Moreover, for this structure, the 100% level of the Gilroy 3 motion corresponds to a maximum considered earthquake (MCE). The acceleration response spectra for these levels of the Gilroy 3 motion are presented in Fig. 4. Based on this correspondence and the goal to gradually inducing damage to the structure, the loading protocol presented in Fig. 4(b) was assembled.

When the seismic excitations were applied to the structure, differences were noted between the intended and the recorded motions at the base of the structure. These differences introduced the need re-evaluate the scaling coefficients for the recorded motions as percentages of the historical records of Gilroy 3 and El Centro. For an elastic structure this could be achieved by considering the elastic acceleration response spectra with respect to its fundamental period. However, the damage induced to the specimen by the test sequence changed the modal periods. Consequently, the estimated from the white-noise tests period of the structure (Moaveni et al. 2010) before and after each seismic test were used to define the range of values of the fundamental period during each test. Then, the scaling coefficient minimizing the difference between the obtained record and the desired time history over that period range was calculated. As shown in Table 5, the re-evaluation of the scaling coefficients indicates that for ground motions up to 91% of Gilroy the table provided in most cases spectral accelerations for the period of interest for each test slightly lower than the desired ones. For gil100 and gil120 though, the input motions were amplified reaching 113% and 133% of the original motion.



(a) Response spectra

Test ID	Intended Level	Test Name
6	10% of Gilroy 3	gil10
8	20% of Gilroy 3	gil20a
12	40% of Gilroy 3	gil40s
16	20% of Gilroy 3	gil20b
18	20% of Gilroy 3	gil20c
19	40% of Gilroy 3	gil40b
21	67% of Gilroy 3	gil67a
23	20% of Gilroy 3	gil20d
26	67% of Gilroy 3	gil67b
28	83% of Gilroy 3	gil83
33	91% of Gilroy 3	gil91
35	100% of Gilroy 3	gil100
40	120% of Gilroy 3	gil120
43	250% of El Centro	e1c250

(b) seismic tests

Figure 4. Acceleration response spectra and conducted seismic tests.

Preliminary Test Results

The testing sequence can be divided in two phases. The initial phase involved low amplitude tests conducted to study the structural behavior in the elastic range. The initial part of the testing sequence included one test at 10% of Gilroy 3, three tests at 20%, and two tests at

40%. The 20% and 40% level tests were repeated to resolve issues related to the supports of the steel towers, the instrumentation, and the control of the shake table. Thorough inspections of the test specimen after each test indicated that the structure did not sustain any visible damage during the first six tests. From the initial tests, only gil40b, the most demanding of these tests is discussed here for conciseness.

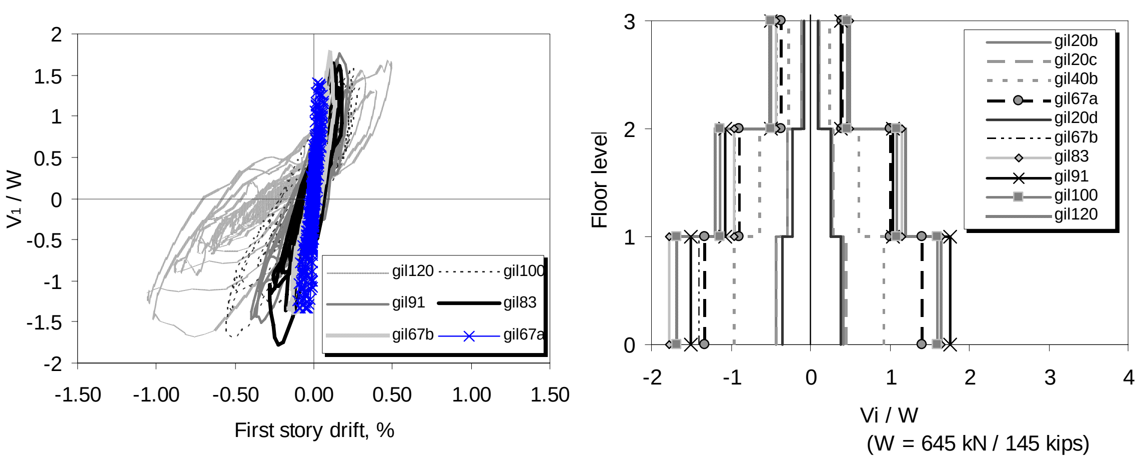
The second phase of testing involved seven tests with acceleration demands beyond the DBE (FEMA P695). Six of these tests were scaled versions of the Gilroy 3 record, while for the last test the El Centro record amplified at 250% was used. The first cracks in the structure developed during the gil67a test; however the cracks were minor and only cosmetic repair would have been needed in a real structure with similar damage. During the second test at 67% the structure reached its peak strength in the positive direction of loading. As Fig. 5 and Table 2 indicate, the structure was able to maintain its strength despite the accumulation of damage and the significant increase of the first story drift which was the result of the soft story mechanism that developed. Moreover, beyond the 67% test, the roof accelerations practically did not increase despite the considerable increase of the PGA and the spectral accelerations of the base motions. Table 2 also summarizes the state of the structure after each test and associates the first-story drift with the damage observed and the repair needed. It can be observed that serious damage was initially observed after the structure was subjected to gil100 during which it reached 0.55% of first-story drift. This value is significantly higher than the 0.3% drift defined in ASCE 41-06, as the drift beyond which drastic loss of shear resistance would be expected for this structure. Furthermore, despite the induced damage, the specimen was able to maintain its integrity when subjected to a base motion exceeding by 55% the MCE, as it maintained the same level of base shear during that test. Hence, it can be concluded that the specimen behaved in satisfactorily as it survived a sequence of six base motions exceeding the DBE level. However, such a claim needs to consider the good quality of construction, the lack of vertical discontinuities and irregularities, and the limitations of the test setup since the specimen was only subjected to a uniaxial base motion while real structures are subjected to three dimensional excitations.

Table 2. Peak values of selected response quantities.

Test	Re-evaluated scale factor [%]	Acceleration			First-story drift [%]	$\frac{V_1}{W}$	Damage level	Repair needed
		Ground	Spectral	Roof				
gil40b	44	0.79	1.20	1.27	0.011	0.97	-	No need
gil67a	65	1.14	1.98	1.90	0.097	1.41	Minor	No need
gil67b	69	1.03	2.34	2.24	0.116	1.75	Minor	No need
gil83	73	1.16	2.7	2.38	0.28	1.77	Mild	Minor
gil91	83	1.29	3.34	2.36	0.40	1.76	Mild	Minor
gil100	113	1.59	3.9	2.38	0.55	1.68	Serious	Extensi
gil120	133	1.90	6.5	2.43	1.06	1.68	Severe	Beyond
ec250	256	1.94	3.7	-	-	-	Collapse	-

* the weight of the structure is 645 kN (145 kips)

During the ec250, a significant portion of the east bay masonry panel collapsed on the shake table and the three first-story columns failed in shear, at different heights, as shown in Fig. 6. Although the specimen failed out-of-plane at the very early stages of the ground motion, the steel towers prevented the collapse by continuously supporting the structure during its lateral motion. At the end of the test, the structure could not carry its own weight and was leaning against the support towers. In a real structure the RC members in planes transverse to the tested frame would have prevented the out-of-plane collapse mechanism. Therefore, this is a possible mechanism for real structures and it is interesting to observe it. The most impressive failure was the shear crack that developed near the bottom of the east column. This failure occurred early in the test and was followed by the gradual collapse of the infill wall in the east bay of the specimen. This was the wall that included a window which played an important role not only in the development of the cracks which initiated at its corners, but also in reducing the out-of-plane stability of the infill panel, when two triangular pieces of the masonry wall on the sides of the window detached from the rest of the wall. As a result, the unsupported lintel beam collapsed in subsequent cycles of motion. It is important, though, to point out that the structure collapsed after being subjected to a series of strong motions that a real structure is not likely to experience.



(a) (b)

Figure 5. (a) First story shear-vs.-drift relation and (b) shear force distribution.

Conclusions

This paper presents the seismic behavior of an infilled RC frame representing existing structures built in California in the 1920s, when no design building code was strictly enforced. The design procedure used a working stress approach and only considered gravity loads. This procedure has been followed to design a prototype three-story building with currently available building materials. An external frame of that structure has been tested on a shake-table under a sequence of 14 scaled earthquake records of increasing intensity. The design and scaling details, as well as the test setup are discussed in the paper. The preliminary test results indicate that the specimen behaved elastically for earthquake excitation below the design level earthquake. The specimen sustained insignificant cracking in the infill under a DBE level base motion. As the intensity of the base motion increased, the cracks propagated causing the development of diagonal shear cracks in the RC columns under a motion corresponding to an MCE event. After this test level, the damage in the structure was considerable; however, still repairable. For higher levels of excitation, major diagonal cracks developed in the columns, and the structure practically collapsed during the initial cycles of the seismic test with the El Centro motion increased by 256%. It should be emphasized, though, that the damage accumulated over a sequence of seven strong ground motions that a real structure is not likely to experience during its life span. Moreover, the specimen was only subjected to uniaxial excitations, while in actual earthquakes structures are subjected to three components of acceleration. Further analysis of the test results is currently in progress (Stavridis 2009).

(a) west column-top

(d) middle column-mid-height

(b) west column-bottom

(c) overview

(e) east column-bottom

Figure 6. Overview of damaged specimen and details of column failure in the first story after ec250.

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