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EARTHQUAKE SIMULATOR TESTING ON BEHAVIOR OF SEISMIC COLLECTORS IN STEEL BUILDINGS

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Abstract

Seismic collectors are structural components that transmit inertia forces to the primary vertical-plane elements of the Seismic Force-Resisting System (SFRS) in a building structure. Collector failure can be catastrophic as evidenced in several building collapses in recent earthquakes. The collector load path involves complicated indeterminate assemblages of different materials and geometries, acting at different elevations, and connected by elements which may not be intended for collector action. Complex load paths develop in the floor system as a result not only of collector action, but also interaction with lateral load and the gravity load-resisting system. A series of shake table tests have been conducted on a half-scale, two-story multi-bay steel test frame by using the NHERI@UCSD large high performance outdoor shake table. The main objectives of this test program are to (1) investigate the load path of the inertia force transferring from the floor diaphragm through collectors and their connections to the SFRS, and (2) examine the behavior and performance of steel collectors and several commonly used collector-to-column connection details. The test frame included a composite floor diaphragm at the second floor and a bare metal deck diaphragm at the roof level. Three commonly used collector-to-column connection details included in the specimen design were (1) all-flange weld (AFW) detail, (2) top-flange weld (TFW) detail, and 3) bolted-web connection detail. This testing program was conducted in two main phases. The Phase 1 tests were performed on the "single-story phase" of the test frame, in which only the first story of the test frame with a composite slab had been constructed. An innovative test technique was developed such that the floor absolute acceleration time histories of any floor in a multi-story building could be simulated by this single-story frame through a transfer function approach. Phase 2 tests were conducted after the second story with a bare steel roof deck was added to the test frame. In this phase, the historical ground motions were used as the input motion to study the load path and the cyclic response of collectors and their connections. Key test results and design implications are presented.

Keywords: Collector, Connections, Diaphragm, Shake Table, Seismic Design

1. Introduction

Collectors are the critical elements that bring inertial forces that develop in the floor or roof diaphragm to the primary vertical-plane elements of the Seismic Force-Resisting System (SFRS) when a building is subjected to an earthquake. In steel building structures, the collector is provided by beams in the floor or roof system. Due to the reversing nature of earthquake loads, collectors must alternately carry tension and compression, while under the presence of effects from gravity load and frame lateral drift. Therefore, the collectors must be designed both as tension members and compression members. Thus, both collector connection strength and collector element stability are key aspects of collector design. Tension design focuses on the collector connections. The collector element itself is designed as a beam-column, since the member is under combined flexure (due to gravity load) and axial load (due to collector action). The controlling compression limit state for a steel collector member depends on the bracing condition of the floor or roof system, including strong-axis or weak axis flexural buckling, torsional or constrained-axis flexural torsional buckling [1]. The latter mode, with center of rotation about the top flange braced by the deck or slab, is particular to collectors.

The connections used to transmit the collector forces across the gravity load resisting columns or to the primary vertical plane SFRS members vary depending on the magnitude of the collector force. For lower level collector forces, the conventional shear tab connection used for gravity load can be designed to carry combined shear and tension, sometimes supplemented by collector reinforcing bars in the slab properly anchored in the slab [2]. As collector forces increase, a modified version of the shear tab connection employing multiple bolt rows is often employed. As the collector forces increase further, a typical design practice involves connecting the top flange. The typical detail in the US involves welding the top flange. Finally, collector axial force levels can become sufficiently large that the top flange connection is not adequate, and thus the connection involves welding of both flanges.

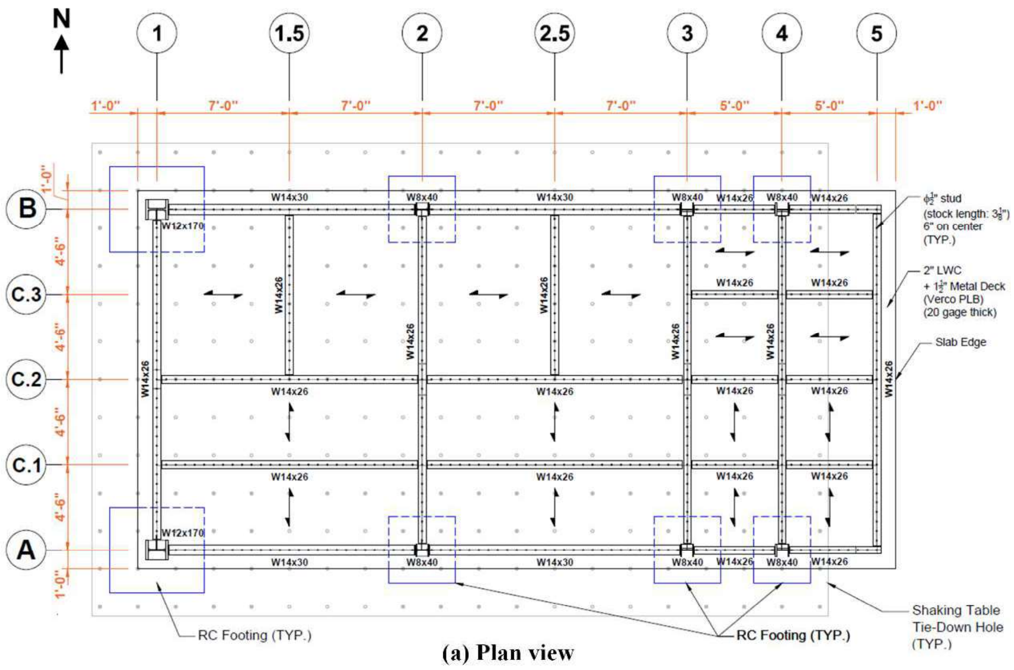
Loss of collector elements is potentially catastrophic, as has been shown by failures of collectors in concrete structures, including collapses in the 2011 Christchurch earthquake [3], and the 1994 Northridge earthquake [4] in which shear or core walls were undamaged, while the floor system detached, resulting in collapse of the Gravity Load Resisting System. Yet little research has focused on collectors, and both the seismic behavior and demands on these elements are not well understood. Instead, current design code provisions rely on amplified collector design forces and simplifying design approximations. A joint research among University of Arizona (UA), Lehigh University, and University of California, San Diego (UCSD) has being conducted from 2017 to investigate the seismic behavior of the collectors in the steel strucutral buildings. This integrated experimental and analytical research program makes use of the U.S. National Science Foundation (NSF) Natural Hazard Engineering Research Infrastructure (NHERI) Facilities. This paper decribes the shake table testing of a two-story structure, possessing collectors in a steel composite floor system and an unfilled roof deck, at the NHERI@UCSD large high performance outdoor shake table. The two main objectives of this test program (1) investigate the load path of the inertia force transferring from the floor diaphragm through collectors and their connections to the SFRS, and (2) examine the behavior and performance of steel collectors and several commonly used collector-to-column connection details.

2. Test Frame Design

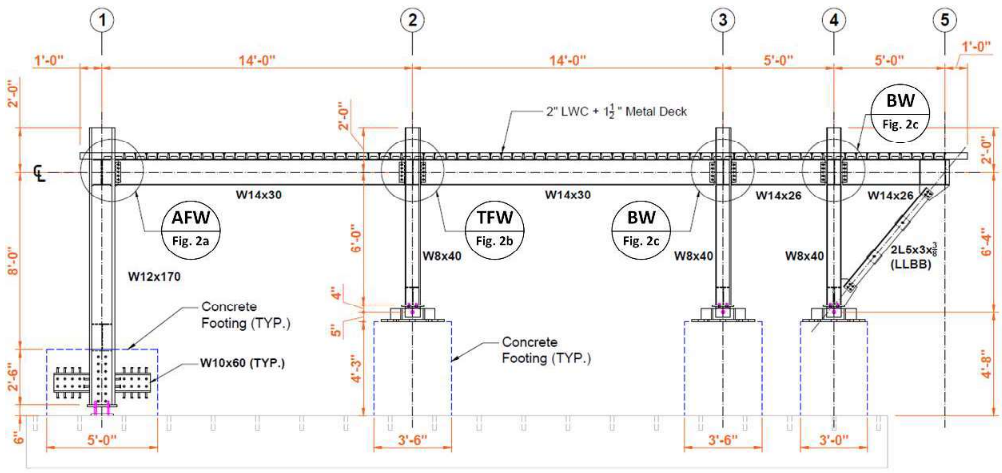
2.1 Phase 1: Single-story test frame

In Phase 1 of the test program, a 1/2-scale one-story specimen with a composite floor slab (Figure 1) was designed to simulate the earthquake-induced absolute acceleration time-history response of any floor of a prototype multi-story building such that a realistic inertia force mechanism could be reproduced in the floor diaphragm system of the test frame. As shown in Fig. 1a, the floor plan of the test frame was 38 feet long and 18 feet wide. It had two 14-ft long bays, one 5-ft long bay, and one 5-ft long cantilever span in the longitudinal direction, which was the direction of shaking. In the transverse direction, four single-bay braced frames were used to prevent torsion of the structure. Two W12×170 cantilever columns at the west end of the structure served as the lateral force-resisting system (LFRS). The column height from the top of the concrete footing to the beam center was 8-ft high. Collectors were placed along Column Lines A and B. Thus, the inertial force of the slab would be transmitted to the two cantilever columns through these collectors and their connections.

Figure 1b shows the elevation of the exterior frame. Each cantilever column along Column Line 1 was embedded into a 3-ft tall concrete footing, which was post-tensioned to the shake table and provided a fixed-base condition for the column. The remaining columns along Column Lines 2, 3, amd 4 were pin-supported W8×40 gravity columns. A 4’-3” tall concrete block was placed underneath each gravity column’s pin-support and anchored to the shake table. The gravity column height from the center of pin-support to the beam center was 6’-4’ high. These gravity columns were designed to be shorter than the cantilever columns so that higher collector-to-column joint rotations could be achieved during the tests.



(a) Plan view



(b) Elevation view

Fig. 1 – Phase 1 test frame: (a) plan view; and (b) elevation view

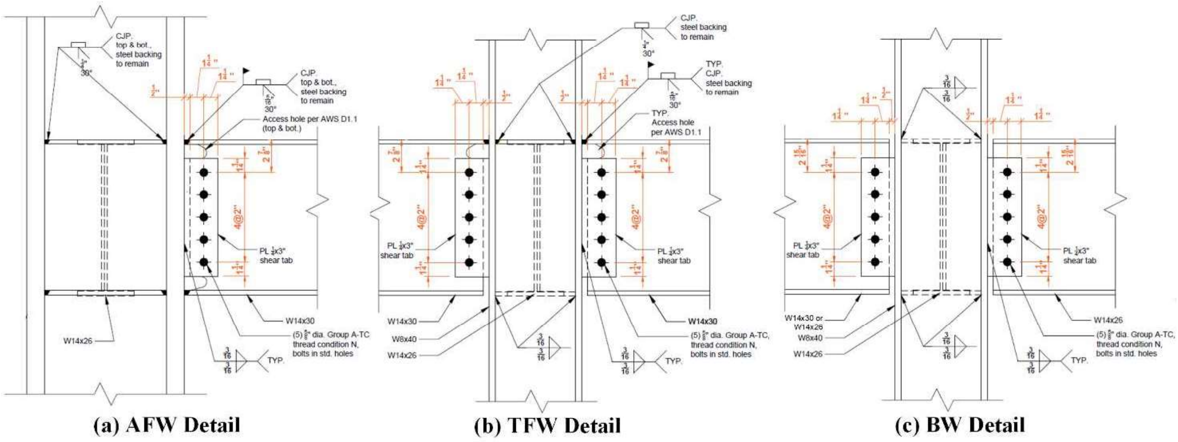
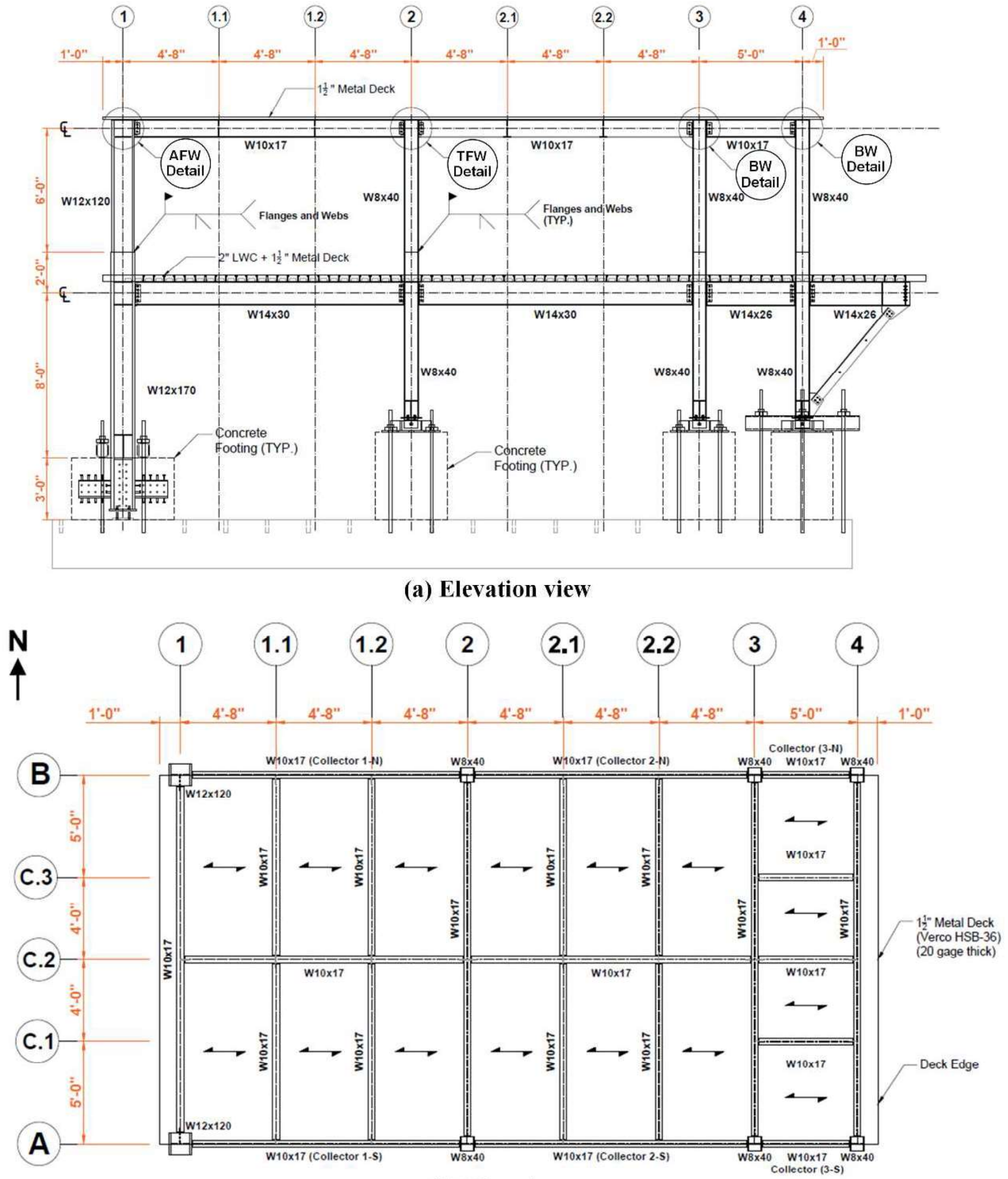


Fig. 2 – Collector-to-column connection details: (a) AFW; (b) TFW; and (c) BW details

Fig. 3 –Phase 1 test frame

From the cantilver column to the east end of each frame elevation (Fig. 1b), sequentially there were two 14-ft long W14×30 collectors followed by two 5-ft long W14×26 collectors; the last of which was a cantilever beam supported by a brace. The four 14-ft long collectors were the main subjects in the test specimen because their beam length is approximately ½-scale of the prototype. Due to the limitation of the shaking table size, only two bays of 14-ft collectors could be constructed. The two remaining 5-ft long spans, including the cantilever span, were used to accommodate required mass of the test frame to ensure that the whole structure could generate sufficient inertial forces for the subject collectors. Since the inertia force had to be resisted by the cantilever columns, the forces in the collectors and their connections increased progressively from east to west of the test frame. Therefore, three collector connection details, which are commonly used in the U.S. design practice, were employed. For the collector-to-column connection adjacent to the cantilever column, as shown in Fig. 2a, the all-flange weld (AFW) detail was used. The complete-joint-penetration (CJP) groove welds were used for the top and bottom flanges; steel backing was not removed for both flanges. For the collector-to-column connections for the first gravity columns (Column Line 2), the top flange weld (TFW) detail (see Fig. 2b) was employed. Only the top flange of the collector was connected to the column using CJP weld with backing bar not removed. Lastly, the remaining collector-to-column connections used the bolted-web (BW) connection detail (see Fig. 2c). W14×26 was specified for all the floor beams.

The test structure was designed for an inertial foce of 2g absolute acceleration of the floor. All the W-shape beam and columns were made of A992 steel and all plates, including shear tabs, stiffeners, and gussets, were made of A572 Gr. 50 steel. Braces in the trasverse direction of the test frmae were made by A36 double-angle sections. A composite floor system with a 2-in. thick concrete slab over a 1.5-in. deep steel decking was used for the test frame. The fill was lightweight concrete with a spcified compressive strength of f'_c = 4 ksi. In order to investigate the effects of the deck orientation on the inertia force transfer mechanism, the metal deck was oriented to be perpendicular to the collectors for the south half of the test frame (see Fig. 2a), while the north-half of the metal deck was parallel to the collectors. Figure 3 shows the photo of the Phase 1 test frame. A total of 75 kips concrete blocks, serving as the added mass, were attached onto the top of the composite slab through post-tension rods.



(a) Elevation view

(b) Plan view

Fig. 4 – Phase 2 test frame: (a) elevation view; and (b) plan view

Fig. 5 – Photo of Phase 2 test frame

2.1 Phase 2: Two-story test frame

After the Phase 1 tests were completed, the second story of the test frame was added to make the specimen a two-story test frame (see Fig. 4a) for the Phase 2 tetsing. The 1½"-deep unfilled metal deck was installed at the top level to represent a roof floor in a steel building. The second-story columns were spliced on the top of first-story columns through CJP splices. (Columns in the Phase 1 test frame were designed to extend above the concrete slab by 1-ft to facilitate column splicing.) For the second story, the cantilever columns and the gravity columns were made by W12×120 and W8×40 columns, respectively. Figure 4b shows the plan view of the roof level. The entire roof deck was oriented to be parallel to the collectors. W10×17 was specified for all collectors and roof beams. Similar to the first story, three types of collector-to-column connection details were employed. AFW connection was used for the connections to the cantilever columns, while TFW was employed for the connections to gravity columns along Column Line 2. The remaining connections used the bolted-web (BW) connection detail. Double-angle braces were provided in the transverse frames. Figure 5 shows the Phase 2 test frame. A total of 44 kips steel plates, used as the added mass, were placed on the top of the roof deck and were secured to the floor beams.

3. Test Program

3.1 Phase 1: floor-acceleration-simulating tests of the single-story test frame

In the Phase 1 test program, the main goal of the shake table test control was to excite the 1/2-scale sigle-story test frame to reproduce the absolute acceleration responses of any specific floor in a multi-story prototype building under various intensities of earthquakes. As shown in Fig. 6, a twelve-story (12F) example building proposed by the Steel Diaphragm Innovation Initiative (SDII) joint research team [6] was chosen as the prototype structure. The 12F building was designed by using buckling-restrained braced frames (BRBFs) as LFRS. An analytical model was constructed to represent the prototype building by using the nonlinear structural analysis software PISA3D [7]. A series of nonlinear time history analyses were conducted on the prototype model to obtain the floor acceleration responses. The input acceleration for the prototye analyses was the ground motion recorded at Beverly Hills-Mulhol station from the 1994 Northridge earthquake. The input motion was scaled to three intensity levels and then used for performing three separate time history analyses. The three intensity levels were minor, moderate, design earthquake (DE) levels, which were 0.2, 0.5, 1.0 times the DE earthquake level, respectively. The fifth floor absolute acceleration responses from the three analyses were employed as the target floor accelerations for the test frame. Table 1 lists the text matrix. Phase 1 test program performed three floor-acceleration-simulating tests for these three earthquake intensity levels.

Figure 6 illustrates the procedure for generating the table input motion for the test frame. Since the test frame is a 1/2-scale specimen, based on the similitude law, the time of the floor acceleration time history, $\ddot{u}^t(t)$, was scaled by a scale of $\sqrt{0.5}$. The scaled acceleration time series, $\ddot{u}_{sq}^t(t)$, then served as the target floor acceleration for the test frame. Before conducting each shake table test, white-noise and impuse tests were coducted to identify the fundamental period and damping ratio of the test frame. Consdering the single-story test frame as a single-degree-of-freedom (SDOF) system, the transfer function in the frequncy domain, $H(\omega)$, between the output absolute acceleration, $\ddot{u}^{t*}(\omega)$, and the input ground acceleration, $\ddot{u}_g^*(\omega)$ was obtained:

$$H(\omega) = \frac{\ddot{u}^{t*}(\omega)}{\ddot{u}_g^*(\omega)} = \frac{i2\xi\omega_n\omega + \omega_n^2}{-\omega^2 + i2\xi\omega_n\omega + \omega_n^2}$$

where ω_n and ξ are the experimetally-determined angular frequency and damping ratio of the test frame, respectively. Then the required input table motion in the frequency-domain was determined from $\ddot{u}_g^*(\omega) = \ddot{u}_{sq}^t(\omega)/H(\omega)$. Finally, by performing an inverse fast Fourier transform (IFFT) to $\ddot{u}_g^*(\omega)$, the input table motion time history for the test frame, $\ddot{u}_g^*(t)$, was obtained.

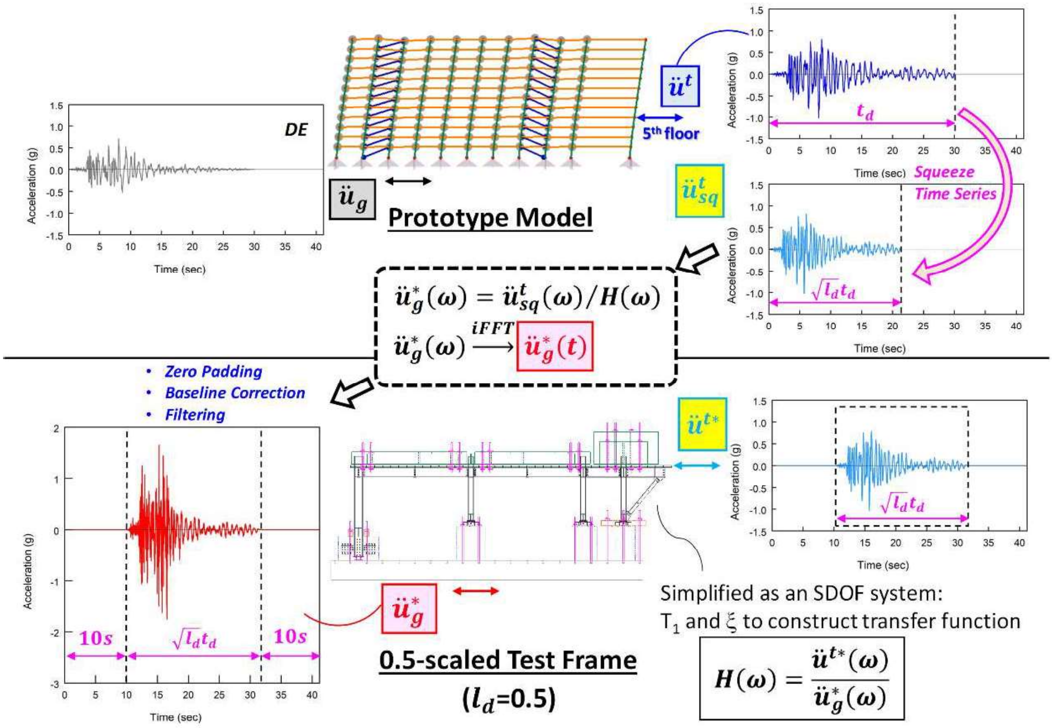


Fig. 6 – Phase 1 Test Frame

3.2 Phase 2: earthquake tests of the two-story test frame

In the Phase 2 tests, the two-story test frame was treated as a building and subjected to the earquake tests. The input table motions were generated by scaling the ground motion record from 1994 Northridge earthquake (Beverly Hills-Mulhol station) to different intensities. Since the test frame is a 1/2-scale specimen, the time of the original ground motion was scaled by a factor of $\sqrt{0.5}$. Then, the acceleration response of the time-scaled ground motion was scaled to the target intensities by using the $S_a(T_1)$ method [8] such that 5% damped spectral acceleration of the time-scaled ground motion matched the target design response spectrum at the experimetally-determined foundemental period of the test frame. White-noise and impulse tests were also conducted before and after each eathquake test to monitor the variations of the stuctural period and damping.

Five earthquake tests were performed for three intensity levels in Phase 2 testing: 50% DE, 100% DE and 125% DE (see Table 1). For some tests, the test denoted with "inverted" in the motion direciton means that the sign of the input motion was reversed. Reversing the sign of input motion was intended to achieve both maximum tension and compression axial forces in the collectors in this test program.

Table 1 – Test matrix

Phase	Test ID	Test Type	Intensity Level	Motion Direction	Motion Source
Phase 1	1-A	Floor Acceleration Simulating Test	20% DE	Forward	Simulated Motion
	1-B		50% DE	Forward	
	1-C		100% DE	Forward	
Phase 2	2-A	Earthquake Test	50% DE	Forward	1994 Northridge ("Beverly Hills-Mulhol")
	2-B		100% DE	Forward	
	2-C		100% DE	Inverted	
	2-D		100% DE	Forward	
	2-E		125% DE	Inverted	

4. Test Results and Design Implications

Figure 7 shows the story drift angle and floor absolute acceleration time histories of Test 1-C (100% DE level). The maximun story drift was 0.6% rad and the peak floor absolute acceleration reached about 1.6*g*. Figure 8 shows the strain profiles of the composite beam sections near the collector-to-column connections when the floor absolute acceleration reached the peak values in the both direcrtions. Figures 8a, 8b, and 8c show the strain profiles for those sections at the east side of the AFW (Column 1), TFW (Column 2) and BW (Column 3) connections, respectively. The maxiamum strain in the steel collector sections slightly exceeded 0.1%, indicating that all the steel collector sections remained elastic in this test.

For the AFW connection (see Fig. 8a), the strain profile of composite beam exhibited a linear distribution from the steel bottom flange up to top of the concrete slab at the peak negative floor acceleration. On the other hand, at the peak positive floor acceleration, the strain in the concrete slab was very small while the vertical strain distribution within the steel collector section appeared linear. The strain profiles near the AFW connection exhibited the charactristics of a typical rigid composite beam-to-column connection. The concrete slab contributed in the positive moment, while its contribution to the negative moment was negligible. Note that an inertial force of positive absolute floor accelertion would produce compression in the collectors, while a negative floor acceleration would trigger tensile axial forces in the collectors. Test results show that the concrete slab was in compreesion when the collector was in tension, whereas the concrete slab was in tension when the collocter was in compression.

As shown in Fig. 8b, the strain profiles imply the TFW connections pivoted about the top flange regardless the sign of the floor accleration. In additon, the strain at the bottom flange ramained very small for both acceleration directions, indicating a stress-free status of the bottom flange. However, the linearly distributed strain profile from the quarter height to the top of the steel beam section suggested that this connection had certain ability to transmit a bending moment. For the BW connection, as shown in Fig. 8c, the top and bottom flanges developed negligible strain. The strain profile around the mid-height of the steel collector section appears linearly distributed, indicating that this connectin conveyed moments and axial forces through the bolted connection at the beam web. In addition, the concrete slab would contribute to moment resistance when the collector was in tension.

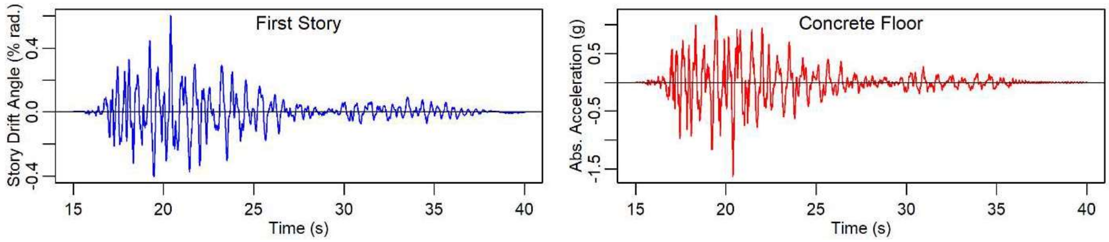


Fig. 7 – Test 1-C results: story drift angle and floor absolute acceleration time histories

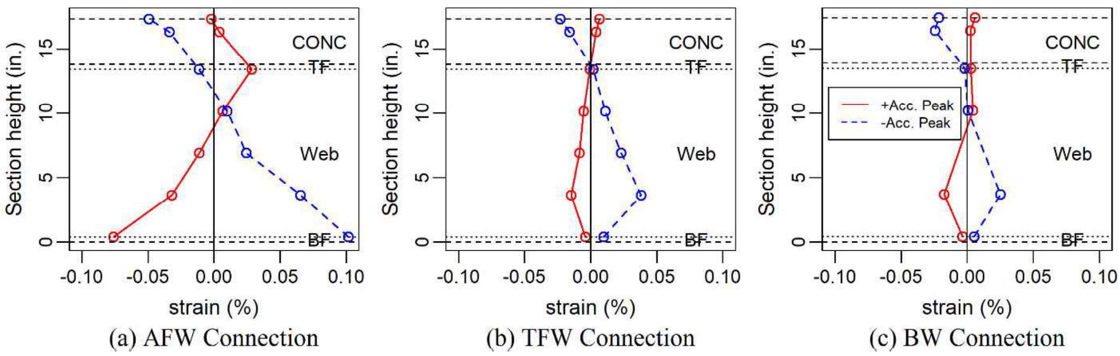


Fig. 8 – Test 1-C results: strain profiles of sections near collector-to-column connections

Figure 9 shows the story drift angle and floor absolute acceleration time histories of Test 2-E (125% DE level). The maximun story drifts of the first and second stories were 1.7% rad and 1.8% rad, respectively. In addition, the peak floor absolute accelerations of the concrete slab and metal deck roof reached 1.7*g* and 2.9*g*, respectively. Figure 10 shows the longitudinal axial strain profiles of the collector sections at peak positive and negative accelerations for Test 2-E. As shown in Fig. 10a, the bottom flange of the AFW connection developed a strain exceeding +0.3% at the peak negative acceleration, indcating that the bottom flange had yielded in tension. This is also supported by the observed flaking of the whitewash on the bottom flange (see Fig. 11) observed at the end of Test 2-E. Furthermore, as shown in Fig. 11, local bucking of the bottom flange was observed at AFW connectios after Test 2-E. Note that both top and bottom flanges of the collectors were welded to the columns at AFW connections, which would make the AFW connection behave like a moment connection such that the flanges resisted significant flexure stress during shaking. Test results showed that the flange local buckling would become the limit state of the collector using AFW connection detail. Considering that the collectors should remain fuctional to transmit the inertia force when the LFRS is deformed into the inelastic range, the collectors using the AFW detail should possess adequate flexural deformation capacity. The section of collectors should be sufficiently compact to prevent premature local bucking.

Figure 10b shows that the top flange of the TFW connection developed a significant tension strain at the negative acceleration peak when the collector was subjected to a tension force. This indicates that top flange participated in trasmitting the collector tnesile axial force. By contrast, at the postive acceleration peak, the top flange did not develop a signficant strain. This suggests that the collector memebr forces were tramsmitted mainly by the steel beam web when the TFW conneciton was in compression due to the inertia force mechanism. For the BW connections, as shown in Fig. 10c, the steel beam web developed significant strains while the both flanges remained about stress free regardless of the direction of the floor acceleration. This suggests that the collector memebr forces were transmitted mainly by the single-plate bolted connection in the beam web.

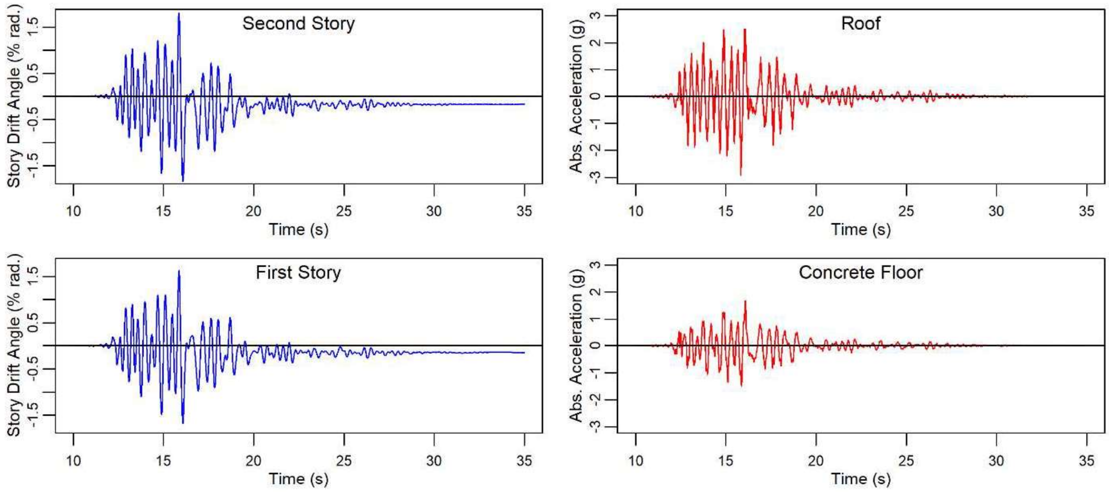
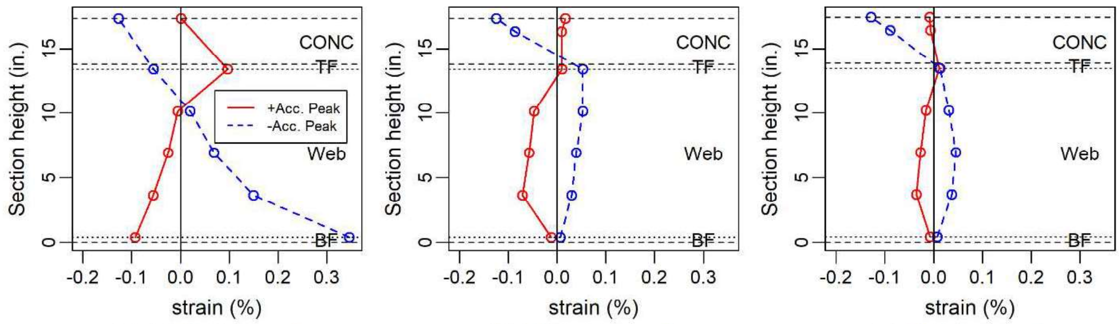


Fig. 9 – Test 2-E results: story drift angle and floor absolute acceler ation time histories



(a) AFW Connection (b) TFW Connection (c) BW Connection

Fig. 10 – Test 2-E results: strain profiles of sections near collector-to-column connections

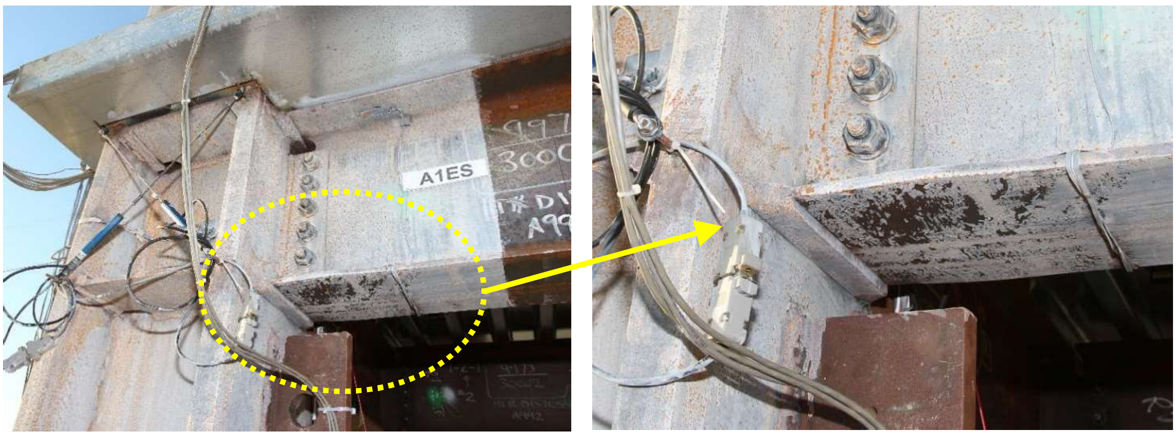


Fig. 11 – Local buckling of AFW conneciton bottom flange after Test 2-E

5. Conclusions

A series of shaking table tests have been conducted at the NHERI@UCSD large high performance outdoor shake table for the investigation on seismic behavior of collectors in steel strucutral buildings. Test results showed that different types of collector-to-column connection details would result in different patterns of strain profiles of the composite collector beam sections. The test results also showed that the AFW collector-to-column detail would behave like a moment connections. The AFW connections would resist a significant bending moment during earthquake shaking, and the member section of the collector should possess adequte compactness.

6. Acknowledgements

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