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Seismic Testing of an Indigenous Material Troop Constructible Building

James Wilcoski

September 2019

Construction Engineering Research Laboratory

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Final Technical Report (TR)

Prepared for Metna Co., Lansing, MI 48906

Under Small Business Innovative Research (SBIR) Contract W9132T-16-C-0003

Abstract

An indigenous materials construction system was developed by a Small Business Innovative Research project – Small Business Innovative Research (SBIR) project Contract W9132T-15-C-0002. The results of that project included the construction of a full scale 16 foot by 32-foot troop constructible building that was tested on the Engineer Research and Development Center, Construction Engineering Research Laboratory (ERDC-CERL) shake table. This report documents the seismic testing of this building. The building consisted of prefabricated frames with interior and exterior wall panels and roof and ceiling panels. The building was tested with 30-second-long synthetic seismic motions, which began at low levels. The test amplitude was increased so that the final test conducted used motions based on a spectral acceleration tied to the highest seismic hazard in the United States. The base of the building was badly damaged in this final test, but it remained stable, demonstrating relatively good behavior. This report documents the measured response to these motions and the performance of the building.

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Preface

This study was conducted for Metna Co., Lansing, MI under Small Business Innovative Research (SBIR) contract W9132T-16-C-0003, via Proposal Number: A142-088-0167, “Using Indigenous Materials for Construction.” The Metna Project Manager was Dr. Anagi Balachandra.

The work was performed by the Materials and Structures (CFM) Branch of the Facilities Division (CF), Construction Engineering Research Laboratory (CERL). At the time of publication, Vicki Van Blaricum was Chief, CFM and Donald Hicks was Chief, CF. The associated Technical Director was Kurt Kinnevan, CEERD-CZT. The Deputy Director of CERL is Dr. Kirankumar V. Topudurti, and the Director is Dr. Lance D. Hansen.

COL Teresa A. Schlosser was Commander of ERDC, and Dr. David W. Pittman was the Director.

Unit Conversion Factors

Multiply	By	To Obtain
cubic feet (cu ft)	0.02831685	cubic meters (m3)
degrees (angle)	0.01745329	radians (rad)
Feet (ft)	0.3048	meters (m)
inches (in.)	25.4	millimeters (mm)
kilopounds per square inch (ksi)	6.89476	Pascal (Pa)
ounce (oz.)	28.3495	gram (g)
pounds (force) (lb.)	4.448222	Newtons (N)
pounds (force) per cubic foot (lb/cu ft)	16.018463	kilograms/cubic meter (kg.m3)
pounds (force) per inch (lb/in.)	0.113	Newton Meters (N-m)

1 Introduction

1.1 Background

An indigenous materials construction system was developed by a Small Business Innovative Research project – Small Business Innovative Research (SBIR) project Contract W9132T-15-C-0002. The results of that project included the construction of a full scale 16x32-ft troop constructible building (TCB), which was tested on the Engineer Research and Development Center, Construction Engineering Research Laboratory (ERDC-CERL) Triaxial Earthquake and Shock Simulator (TESS) (hereafter referred to as the “shake table”).

TCBs are simple single-story buildings that are constructed by soldiers with minimal training at overseas contingency bases. The buildings are normally constructed with stud wall framing, a wood truss roof, and wood or concrete masonry unit (CMU) piers. TCBs may also be constructed with CMU block walls or prefabricated cement panels. Soldiers may quickly construct hundreds or even thousands of these buildings at a contingency base and use them for barracks or simple offices. The lumber and other construction materials may not be readily available in the host nation, so considerable materials may need to be shipped in from overseas. Transporting the materials is costly and may put soldiers who transport the materials at risk. Therefore, an interest has been growing in determining if alternative TCB can be constructed from materials that can be gathered from the soils of typical host nations.

This SBIR developed a building construction system using a lightweight aerated concrete that is reinforced with only chicken wire mesh. This approach would minimize the amount of materials to be shipped from overseas.

ERDC-CERL constructed and conducted seismic tests on three standard 16x32-ft. TCBs. To support those tests, a large steel test fixture was fabricated to extend the ERDC-CERL Triaxial Earthquake and Shock Simulator (TESS) shake table platform. This test fixture could be configured to support either 16x32-ft. or 20x40-ft. buildings. Test motions were generated for those tests, and instrumentation scheme to document the response of those buildings. Those tests became the baseline for the performance of this type of construction. Therefore, it was decided to use the same test fixture, same test motions, and nearly identical sensor arrangement. This report provides brief documentation on the building construction and details the building performance during the seismic tests.

1.2 Objective

The objective of this study was to observe the construction of the indigenous materials TCB and document the performance of the building when tested with seismic motions.

1.3 Approach

1. The indigenous materials TCB was constructed on a heavy steel test fixture that had earlier been bolted to the top surface of the TESS shake table (Chapter 2).
2. Test motions were developed to test for seismic hazard using a shake table testing acceptance criteria for testing nonstructural components (Chapter 3).
3. The building was instrumented with accelerometers and cable extensometers to measure the response of the TCB (Chapter 4).
4. An Activity Hazard Analysis (AHA) was conducted; video recording devices were set up; and preliminary test steps were defined to prepare for the TCB tests (Chapter 5).
5. Seismic tests were conducted using 10%, 25%, 50% and 100% of the RRS defined motions (Chapter 6).
6. Results were recorded, analyzed, and conclusions were drawn (Chapter 7, Appendixes C to F).

1.4 Scope

The report provides limited documentation on the configuration of the indigenous material TCB, including construction details. Further documentation on the construction method is provided in the SBIR project "Using Indigenous Materials for Construction," ERDC-CERL contract number W9132T-16-C-0003. The report does provide details on the development of the seismic hazard and test motions. The report also defines the instrumentation plan and rational for this plan. The report also describes an AHA that describes numerous hazards applicable to the construction and testing of this building and the actions taken to mitigate them. The report defines detailed test steps and documents the response and performance of the building during seismic tests.

2 Building Configuration

The indigenous materials TCB was constructed on a heavy steel test fixture that had earlier been bolted to the top surface of the TESS shake table. Figure 1 shows the almost completed building shortly before testing. The following sections describe building configuration details and stages of construction.

Figure 1. Overall view of constructed indigenous material TCB shortly before testing.

2.1 Test fixture

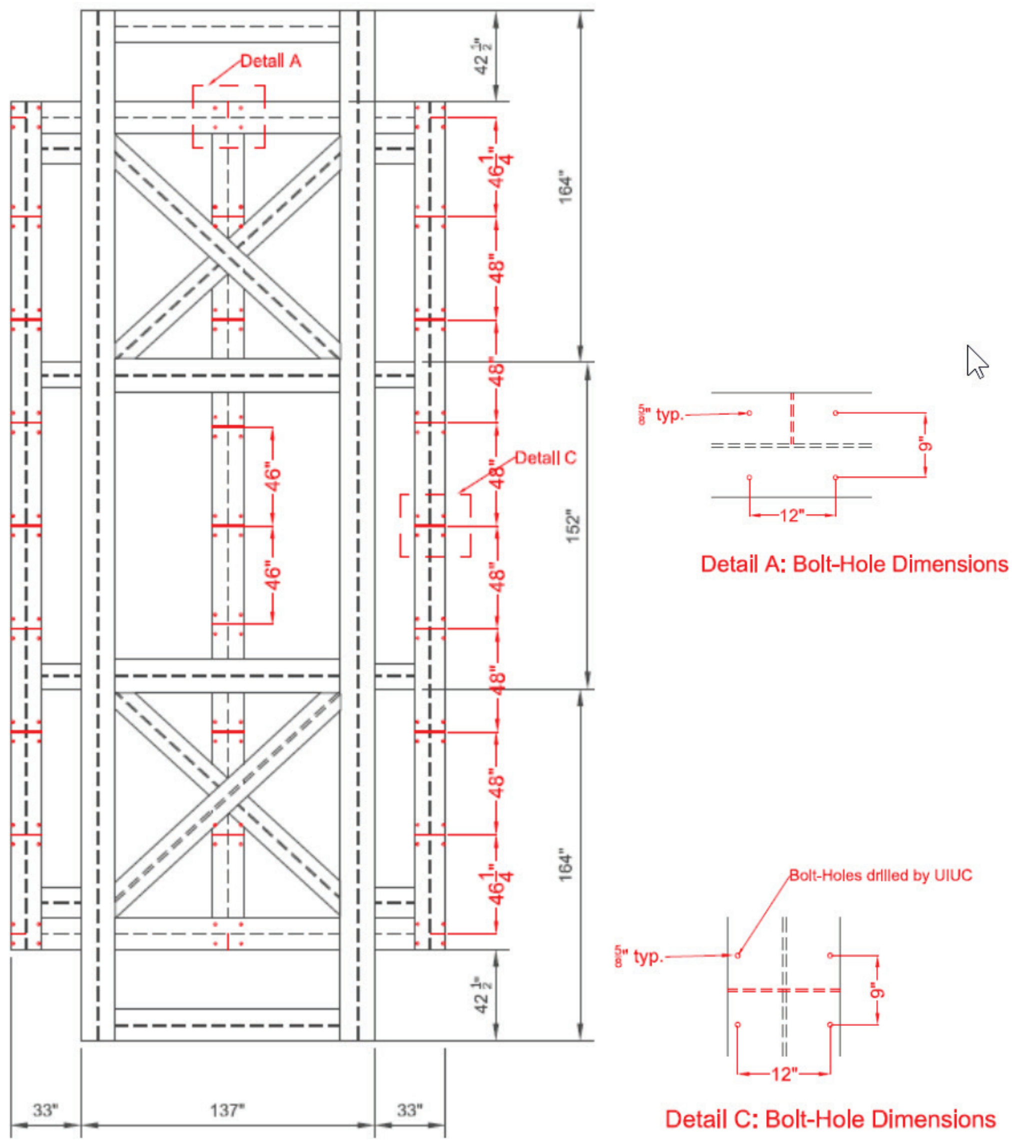
The test fixture was much heavier than would be needed for strength. The shake table surface is 12x12 ft., and the fixture length is 40 ft., so that 14 ft. of the fixture cantilevers off both ends. The fixture design was driven by the requirement that it be stiff enough so that no modes of vibration associated with the deformation of the fixture occur within the frequency range of test motions (maximum of 33 Hz in this case). This should be true when the larger side fixtures are installed and a heavy building is installed on top. The fixture should be a rigid extension of the table, introducing no significant motion associated with fixture deformation. Figure 2 shows this fixture bolted to the TESS surface, and Figure 3 shows the top surface.

Figure 2. Test fixture boltded to the TESS shake table.

The center portion of the fixture was 40-ft. long by 137-in. wide, where the primary members were W44x335 beams. Smaller (32-ft long by 33-in. wide) side fixtures were bolted to the main fixture and their primary members were W14x90 beams. The center portion was 40-ft. long so that larger 20x40-ft. buildings could be tested by removing the small side fixtures shown in Figures 1 and 2 and replacing them with 40-ft. long, wider side fixtures. The assembled fixture had 14.5-in. wide W14x90 beams installed around the entire perimeter to provide a wide surface for installing buildings with a variety of footing or foundation systems.

The top flange of the W14x90 beams had ⅝-in. diameter holes drilled through so that ½-in. diameter bolts could fix the base of TCB piers or footings to the fixture. The earlier tested wood TCBs had wood or CMU block piers spaced at 4 ft. on center below the exterior long walls of the building. Detail C of Figure 3 shows there was a pattern of four bolt holes for each pier spaced 9 in. on center in the long direction of the fixture, and 12 in. on center in the short direction.

Figure 3. Fixture top surface drawing, showing bolt holes for TCB footing anchorage.



The indigenous materials TCB was sized to be compatible with this test fixture, where primary precast framing elements spanned in the short direction of the building and were spaced 48 in. on center. These frames consisted of two columns spaced 16 ft. apart with a roof beam spanning between the columns. The base of each column had a 19-in. long by 17-in. wide by 7.25-in. deep footing. The long axis of the footing was parallel to the frames, which is the short direction of the building. These frames were placed on the steel test fixture so that the footings were centered on the pattern of four ⅝-in. bolt holes in the steel beams (Figure 3). Figure 1 shows that the short walls at the ends of the building had a door opening. Smaller door frames were installed at the center of these walls and they had a single large footing, which was 56-in. long by 17-in. wide by 7.25-in. deep. The footing was centered on the fixture bolt hole pattern shown in Figure 3 and Detail A.

2.2 Protective wood panel deck

A wood sandwich panel deck was constructed on top of the test fixture. This covered the entire fixture surface, except for cutouts for the building footings that rested directly on, and were bolted to, the test fixture. This deck served three purposes: (1) it protected workers and their tools from falling through the fixture, causing injury to themselves or damage to the TESS hydraulics below; (2) it protected the TESS hydraulics and electronics from dust created during construction and testing; and (3) it protected the TESS hydraulics and electronic from potential damage caused by failing portions of the building or even by complete collapse of the building during testing.

This wood deck consisted of a layer of ¾-in. thick plywood spanning in the short direction of the fixture and resting directly on the fixture. Next, 2x4 studs were laid down flat, spaced 12 in. on center and spanning in the short direction of the fixture. The studs were fastened to the plywood below with screws spaced 6 in. on center. Tyvek house wrap was laid on top of the studs and stapled to the studs, covering the entire deck. The house wrap was taped to the fixture along the entire outside edge of the deck. Then a top layer of ¾-in. plywood was placed on top of the studs, spanning in the long direction of the fixture. It was also screwed to the studs at 6 in. on center.

The aerated concrete was a weak material so even when being moved, small portions would flake or rub off, creating considerable dust during construction and testing. This is why the house wrap was installed within the wood deck, to prevent dust from falling down to the TESS hydraulics and electronics. The wrap was placed below the top layer of plywood, so that it would not be damaged by workers and so it would not create a slippery surface that could lead to worker injury. Additional house wrap was also installed around the entire perimeter of the fixture between the wood deck and the TESS reaction mass floor, approximately 4 ft. below the deck top surface. The wrap was stapled to the underside of the deck and then taped to this plywood surface. This location of the wrap would allow any dust from the footings where they anchor directly to the fixture, to fall outside the house wrap. The bottom of the side house wrap was taped to the floor reaction mass, but the wrap was left very slack so that the TESS could move the full stroke of the actuators without the wrap becoming taunt. This wrap created an almost airtight seal between the building and the TESS hydraulics and electronics below. Figure 4 shows the completed wood deck and house wrap along the sides of the test fixture.

2.3 TCB frames and ceiling panels

The lightweight aerated panels were weak by themselves. Figure 4 shows the installation of the first precast frame, consisting of the beam, two columns, and two footings. The beams of these frames were 16-ft. long, the columns below were 93-in. tall, and both the beams and columns were 11-in. deep or wide and 5.5-in. thick. As explained earlier, the footings below the columns were 19-in. long in the axis of the frames, 17-in. wide, and 7.25-in. deep. Both the frames and the footings had 40 layers of chicken wire mesh.

Figure 4. Test fixture with completed wood deck and house wrap, as the first frame is being installed.

The frames were lifted using the ERDC-CERL overhead crane, a spreader frame, and flat nylon straps (Figure 4). The straps were looped around the beams about 2 ft. from the columns. Wood 2x6s were placed between the bottom of the beams and the straps to distribute the lifting load over a larger portion of the beams. These 2x6s were placed against the columns to minimize chances of beam failure in bending while lifting the frames.

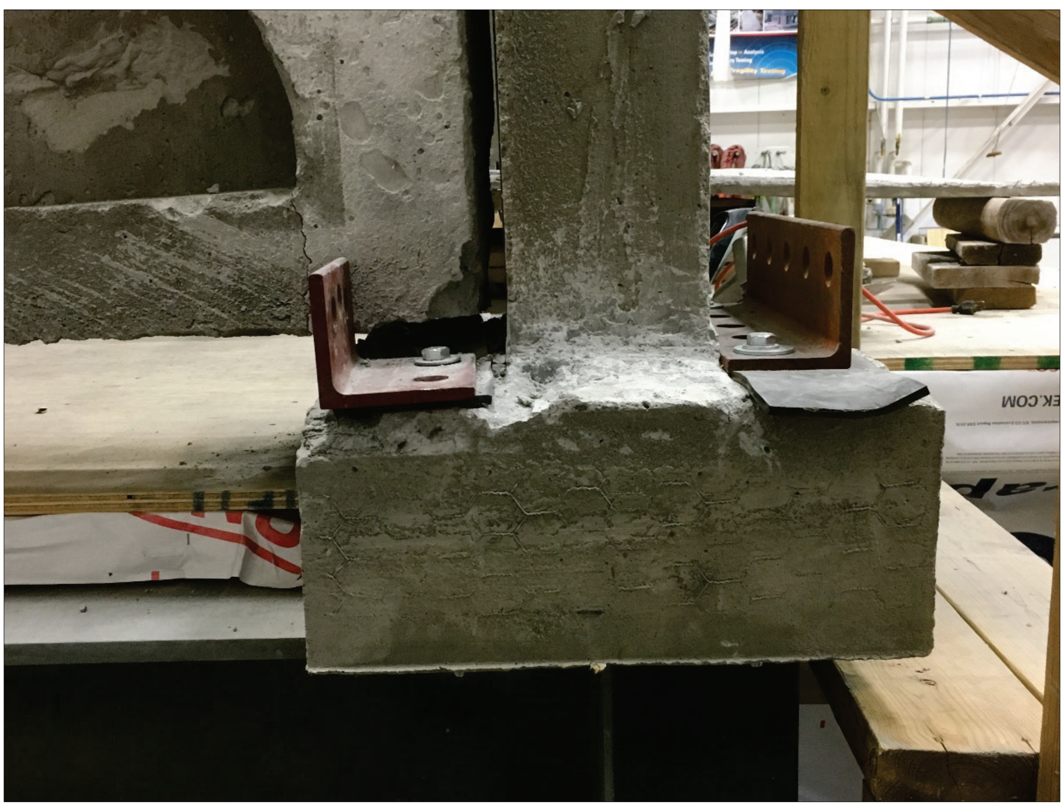
The footings were bolted to the fixture with four ½-in. bolts. In real construction, the footings may be able to slide and rotate. However, it would not be reasonable to permit some degree of sliding or rotation since resistance to this motion is highly variable and should not be included in this testing. It was assumed that full fixity of the column footings should generally be conservative because this would couple or not isolate the building from the motion of the test fixture. In real construction, if the building footings slid on the ground, decoupling the building from the ground motion, the footings could impact large rocks, causing impact loading. The base of piers of the tested wood TCBs were firmly anchored to the fixture; to make the comparison with the wood TCB valid, the footings needed to be well anchored.

A large clamping force was needed to anchor the footings to the top flange of the test fixture. This was done using four ½-in. bolts installed through 1-in. diameter holes drilled in the footings. Because of the weak material of the footings, this clamping force was distributed over a large area of the footing to prevent crushing of the top surface of the footings using 3½-in. x 3½-in. x ¼-in. steel angles that normally were at least 15-in. long and had ⅞-in. diameter bolt holes spaced 12 in. on center. A ¼-in. thick rubber pad was also installed between the steel angle and top of the footings to prevent stress concentrations on the footing surface.

MTS Corporation provided a table for defining the torque that should be applied to Grade 5 bolts to achieve the working stress in the threaded area of the bolts. For ½-in. Grade 5 bolts, the tensile area is 0.142 in^2, and the working stress for Grade 5 bolts is 45 ksi. The working force becomes 6,390 lbs, and the applied torque to achieve this force is 60 ft-lb. This working force is essentially equal to the desired bolt tension of 6,500 lbs, so the bolts were torqued to 60 ft-lb, with a torque wrench. There was a total of nine primary building frames, that each had two columns and four bolts per column, resulting in 72 bolts. Also, four bolts anchored each door frame installed on both ends of the building, resulting in another eight bolts. The total of 80 anchor bolts, applying 6,390 lbs, each provided a total clamping force of 511,000 lbs. If the coefficient of friction between the footings and painted steel fixture was taken as 0.57 (Rabbat and Russell 1985), then the resistance to sliding would be 291,000 lbs, assuming no bearing of the bolts in shear. This resistance to sliding was more than twice the calculated base shear for the 100% test (which is reported in chapter 6).

Figure 5 shows the anchorage of a column, where the angle shown on the right side of the column is the typical anchorage as described above, where the steel angle is 15-in. long, the rubber pad is below the angle, two ½-in. bolts are installed 12 in. apart. Note that the vertical leg of the steel angle is always away from the column. Prior to testing, great care was taken to ensure that the concrete between the horizontal leg of the steel angle and the column was cleared away, creating a gap of at least ¾ in., to ensure that angles would not bear against the columns. The steel angles were only present to provide clamping force; the gap was created so that the load path between the column and footing would only be through the indigenous material alone, and not would not be artificially strengthened by the columns sliding and bearing against the horizontal leg of the steel angles. The left side of the column has an unusual condition in which an indigenous material brace was installed. (This will be explained later.) Two 6-in. long angles (rather than a single steel angle) were installed on the left side of the column so that the angles would not contact the brace.

Figure 5. Anchorage details for a corner column, showing steel angles, rubber pads and bolts.



A second frame was installed and anchored to the fixture, 48 in. to the interior (north) of the frame (Figure 4). Then two ceiling panels were installed between the two frames. These ceiling panels were 4 ft. wide and approximately 7-ft. long, with their long axis in the directions of the frame. These ceiling panels were lifted with the overhead crane and a spreader frame; the panels were placed on and clamped to one end of the spreader frame and lifted up to the space between the frames. Workers then screwed the long edges of the ceiling panels to the frames from below the ceiling panels. They used ¼-in. by 4-in. long self-tapping screws with large washers. Once both ceiling panels were installed, another frame was installed followed by two new ceiling panels. This process was repeated until all nine primary frames and all 16 ceiling panels were installed. The ceiling panels had to be installed along with the frames, so that the overhead crane along with the spreader frame could lift the panels into position. Figure 6 shows the building after all frames, ceiling panels, and interior wall panels had been installed. The ceiling panels were 48-in. wide, a little more than 7-ft. long, and about 1-in. thick, and contained six layers of chicken wire mesh.

Figure 6. Indigenous building with frames, ceiling and interior wall panels installed.

2.4 TCB long wall panels and braces

The interior wall panels were installed next. They were laid on the wood deck with a forklift, then moved by hand and tilted up into position so that their bottom rested on top of the footings and their tops were an inch or two below the ceiling panels (see Figure 6) and there side edges rested against two frames. The two side edges were then screwed into the frames with the ¼-in. x 4-in. long self-tapping screws and large washers.

Indigenous materials X-braces (Figure 7) were installed in the long direction of the building, between the outside frames and the next frame to the interior of the building. Figure 7 shows that these braces included vertical members that were against and bolted to the columns, and horizontal members at the top and bottom of the X. These braces were expected to stiffen the building in the long direction of the building, and also to provide a redundant load path to the wall panels if the wall panel screwed connections loosened or failed.

Figure 7. Long direction X-brace.

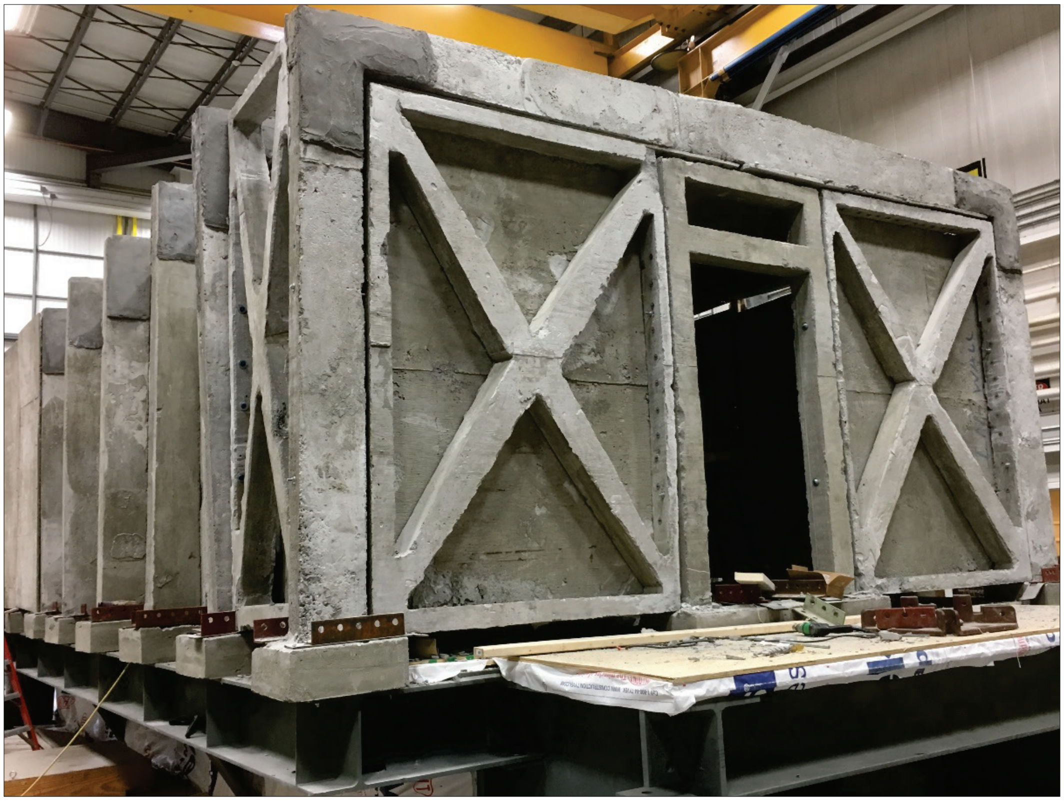
Once all four braces were installed in the long direction of the building, the exterior wall panels were installed. The bottoms of the exterior panels were placed against the column footings and the sides were screwed into the columns along both sides. The tops of these panels were flush with the tops of the building frames (Figure 8).

Figure 8. Exterior wall panels being installed.

2.5 TCB short wall door frame, wall panels and braces

A door frame with a footing, braces, and interior wall panels was installed at the north end of the building before completing the long wall exterior wall panels (Figure 9). An identical door frame (Figure 10) would later be installed at the south end of the building. Figure 10 shows that the door frame columns were supported by a single long footing. Four 1-in. diameter bolt holes were drilled in the footing at the bolt hole pattern shown in Figure 3 (Detail A) for the end walls of this building. Figure 9 shows that two steel angles, two rubber pads, and four ½-in. bolts were used to anchor the footing to the test fixture. The wood seen in the top opening in the frame in Figure 10 was removed once the panel was installed.

Figure 9. End wall door frame, braces and interior wall panels.



The short direction braces shown in Figure 9 were installed next. These braces have similar vertical and horizontal members as the braces in the long direction, but they are shorter and wider. These braces are approximately 93-in. high and 66-in. wide, making them stiffer and stronger than the narrow braces in the long direction, which were about 102-in. tall and 42-in. wide. Because of the relatively short walls in the short direction, the short direction braces were expected to carry a much greater portion of the seismic loads than in the long direction. Figure 9 shows that the brace vertical members that are against the door frames are very narrow, estimated to be 2 in., and these were screwed into the door frames at approximately 6 in. on center. The right side of Figure 9 shows that the brace vertical member against the corner building frame was also screwed into the frame at the same spacing. These braces bore against the building and door fame footings and would have been very well coupled to the short direction wall, most likely carrying a greater portion of the short directional inertial load than did the wall panels.

Figure 10. Door frame precast with a single footing.

The remaining exterior wall panels were installed on the north short wall, and on the long east and west exterior walls. Then the south door fame, the braces, and the interior and exterior wall panels were installed as described earlier for the north wall.

2.6 TCB roof panels

Normally, wood chips would be poured into the cavities between the exterior and interior wall panels and between the ceiling and roof panels for thermal insulation. For the shake table TCB, the wood chips were not installed. The chips would be loosely placed so that their mass would not be well coupled to the building, so that they would not resist load, nor would their mass add horizontal inertial load to the building. They should provide only minor additional damping, which would reduce building response, so that this contribution can be conservatively neglected. Moreover, the test participants also wanted to eliminate that extra debris and dust caused by the wood chips.

Next, the roof panels were installed on the top of the main building frames. These panels were approximately 113-in. long by 48-in. wide by 1-in. thick. The ceiling panels were lifted to the roof using a forklift; workers walking on the plywood sheets laid on the building frames then moved the panels around by hand. Figure 11 shows the installation of these roof panels. The panels were fastened along their long edges to the top of the frames using the ¼-in. x 4-in. screws, spaced at approximately 6 in. on center. Figure 12 shows the eaves of the building where the roof panels extend beyond the exterior long wall panels.

Figure 11. Roof panels being installed.

Figure 12. Indigenous building after roof panel installation showing the roof eaves.

2.7 Application of chicken wire mesh and stucco over joints

Chicken wire mesh was folded over several times to create mats that were screwed over every joint of the completed building. Figure 13 show this mesh installed over the joints between the ceiling panels and interior wall panels. Figure 14 shows the mesh installed over the joints between interior wall panels and over the joints between the interior wall panels and frame footing.

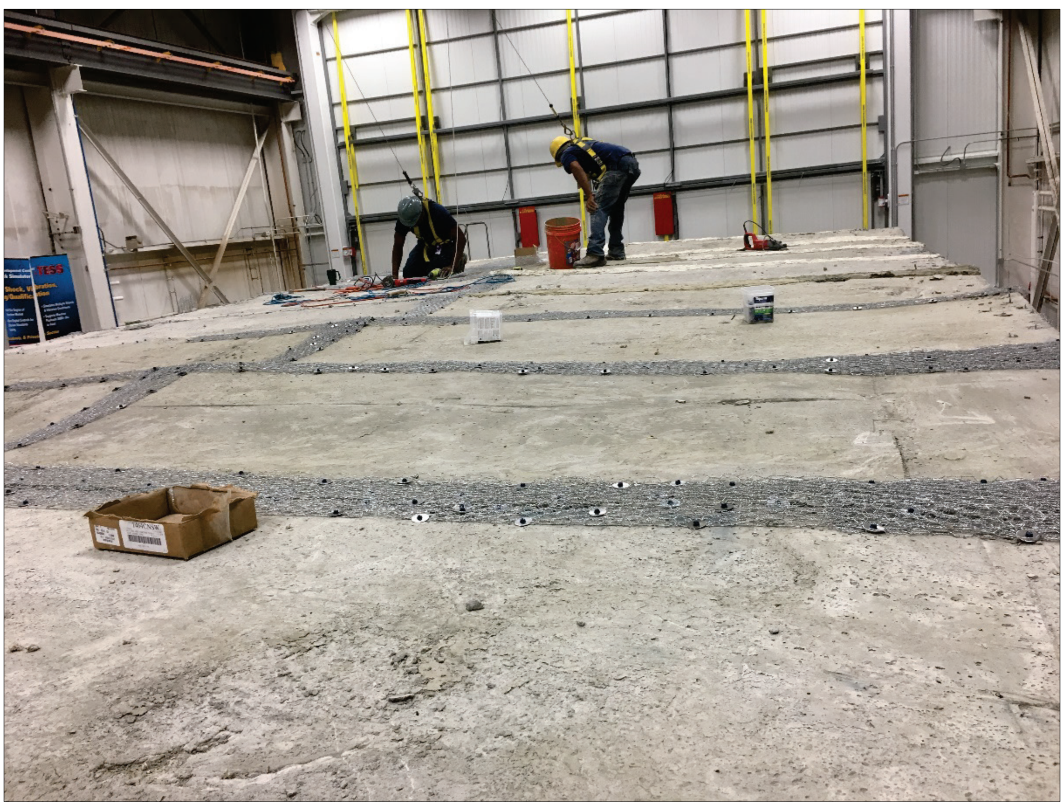
A cementitious stucco was troweled on over these joints, compressing the chicken wire mesh in the process. The stucco material was of similar density and strength as normal concrete (typically 142 lbs/cu ft) (PCA 2018). Figure 15 shows the application of the stucco over the joint between exterior wall panels and between the wall panels and a frame footing. Figure 1 shows an overall view of the completed building after mesh and stucco were applied to all the joints. The most challenging location to apply the stucco was between the ceiling panels because the spring action of the mesh would cause the stucco to be pushed down and drop from the ceiling. With some difficulty the workers were able to get the stucco to adhere and stay in place.

Figure 13. Mat of chicken wire mesh installed over ceiling to interior wall panel joints.

Figure 14. Chicken wire mesh installed over joint between interior wall panels, and between interior wall panels and frame footings.

Figure 15. Applying cementitious stucco over the joints between exterior wall panels, and between the panels and a frame footing.

Figure 16. Applying stucco over roof panel joints. (Note the mesh in the foreground.)



Great care was taken to remove chicken wire mesh and stucco near the steel angles between the wall panels and footing. About a ¾-in. gap was created horizontally between the angles and walls panels and about a ½-in. gap vertically. This was done by chiseling away portions of the wall panels. Shorter angles were also used to reduce the number of panels to be removed. This gap was needed so the angles would not contact the wall panels, because such contact would create an alternate load path and might prevent the development of possible failure where columns frames intersect the footings.

2.8 Building weight calculations and distribution

The weight of all the building components was calculated based on the dimensions of each member and the estimated density of each. After testing, the entire building was lifted as a single unit, and it was weighed with the ERDC-CERL calibrated crane scale. The measured building weight was 39,400 lbs, which is considerably more than the calculated weight. The purpose of these weight calculations is to determine a weight that that can be assumed lumped at a ceiling elevation and lumped at the footing elevation based on the tributary volume of the building. Accelerometers were installed at both the ceiling and footing elevations. The weights calculated for both levels were scaled up based on the measured weight, and these lumped weights were multiplied by the average measured acceleration at both levels to estimate the applied inertial loading of the building. The inertial loads can be plotted with respect to building deformation to define the building performance. Table 1 summarizes these building weight calculations.

Table 1 lists that the calculated weight, assuming dimensions provided by Metna and a lightweight aerated concrete density of 62.42 lbs/cu ft., which is the same as water. Table 1 further lists the assumed total weight of the stucco (5,000 lbs) and the weight of the 10,240 installed screws and washers (512 lbs). It was assumed that the stucco and fastener weights were distributed equally between the ceiling and footing levels. The data listed in Table 1 indicate that the calculated weight lumped at the footing was 10,899 lbs, and at the ceiling was 17,430 lbs, giving a total calculated building weight of 28,329 lbs. The measured weight was 39,400 lbs so the panel thickness may have been greater, the concrete density may have been greater, or more stucco may have been used. Regardless, these increased weights would have been distributed proportionally to the calculated weights. The data listed in Table 1 indicate that the estimated weight lumped at the footing was 15,159 lbs, and the weight lumped at the ceiling,

$W_{ceiling}$, was 24,241 lbs. A weight must be estimated to lump at the bottom of the panels or braces; this was calculated by subtracting the weight of the footing from the weight lumped at the footing, giving a weight at the bottom of the panels, W_{Bot}, equal to 12,348 lbs.

Table 1. Indigenous materials building weight calculations and distribution.

		Lumped at Footing					Lumped at Roof/Ceiling					
	No.	Length (in)	Width (in.)	Thickness (in.)	Volumne (ft^3)	Weight (lb)	Length (in)	Width (in.)	Thickness (in.)	Volumne (ft^3)	Weight (lb)	Total Weight check (lbs)
Frames	9	93	11	5.5	29.3	1829	285	11	5.5	89.8	5606	7435
Side wall panels	32	51.5	48	0.75	34.3	2143	51.5	48	0.75	34.3	2143	4286
Ceiling Panels	16						103	48	0.75	34.3	2143	2143
Roof Pannels	16						113.3	48	0.75	37.8	2356	2356
Front/back wall panels	8	51.4	84	0.75	15.0	935	51.4	84	0.75	15.0	935	1871
Frame Footings	18	19	17	7.25	24.4	1523						1523
Door Frames	2	92	5.5	5	2.9	183	168	5.5	5	5.3	334	517
Door Frame footing	2	56	17	7.25	8.0	499						499
Panel above door	4						32	36	0.75	2.0	125	125
Short dir Braces	4	240	6	3	10.0	623	240	6	3	10.0	623	1246
Long dir Braces	4	157	6	3	6.5	408	157	6	3	6.5	408	817
Stucco						2500					2500	5000
Screws & washers						256					256	512
					Total Calc Weight (lbs.)	10,899				Total Calc Weight (lbs.)	17,430	Total Calc Weight (lbs.) 28,329
					Increased Density and Thickness	4,259				Increased Density and Thickness	6,812	11,071
					Total Weight at Footing Elevation (lbs.) =	**15,159**				Weight at Ceiling, $W_{Ceiling}$ (lbs.) =	**24,241**	39,400
					Corrected Weight of Footings (lbs) =	2811.2						
					Weight at Bottom of TCB, W_{Bot} (lbs.) =	**12,348**						

3 Definition of Seismic Hazard

In 2017, ERDC-CERL tested three wood framed TCBs. The test motions for those TCBs were also used here, so that a direct comparison on performance and capacity can be made. The test motions were developed using a shake table testing acceptance criteria for testing nonstructural components (ICC-ES 2010). This reference was used because of the broad frequency content of the resulting test motions, which represented the direct tie to the International Building Code on which the seismic motions are defined and the broad use of it for the development of shake table motions.

The greatest short period spectral acceleration, S_S in the United States, is 3.730 g for 34.46 degrees N, 119.01 degrees west, which is 6 miles Northwest of Fillmore, CA. The 1-second spectral acceleration, S_1 is 1.283 g for the same site. The building code reference document is the 2015 International Building Code, the Site Soil Classification is D- “Stiff Soil” (ICC 2015) which is also the default soil classification. The Risk Category is I/II/III. The S_{DS} and S_{D1} values for the same site become 2.487 g and 1.283 g, respectfully. This value of S_{DS} was rounded up slightly to 2.5 g, and then the guidance in AC156 (ICC-ES 2010) was used to develop test motions.

AC156 defines a required response spectrum based on the S_{DS} value and the maximum elevation where the equipment to be tested may be installed in the building. If the equipment to be tested were installed on a building’s roof, the horizontal required response spectrum (RSS) would be increased to reflect the amplification of horizontal motions due to building dynamic response to the ground motions. In the case of the TCB, these will always be installed on the ground, so that a z/h value of zero was used in the AC156 relationships that defined the RRS. Table 2 lists the values used to define the RRS for TCBs based on an S_{DS} value of 2.5 g, z/h

of 0, and 5% of critical damping. Table 2 lists the key points for the RRS and Figure 17 plots them for both the horizontal and vertical directions. AC156 requires that independent test motions be generated for longitudinal, lateral and vertical directions, which fit the RRS assuming 5% damping. These motions should have a total duration of at least 30 seconds, including 5 seconds of ramp-up, at least 20 seconds of strong motions and 5 seconds of ramp-down.

Figures 18 through 20 plot longitudinal (x-axis), lateral (y-axis) and vertical (z-axis) proposed test motions that were generated to fit the RRS plotted in Figure 17. Table 2 lists the actual lower frequency values, at 1 Hz through 2.1 Hz. These lower frequency limits are lower in the lateral axis than in the longitudinal and vertical direction because the TCB will have the lowest natural frequencies in the lateral direction (short axis of the TCBs), higher natural frequencies in the longitudinal direction (long axis of the TCBs), and highest natural frequencies in the vertical direction. The displacements will be the greatest for those motions that contain the lower frequency content, and the TCBs are oriented with the short walls in the lateral direction, because the shake table stroke is greatest in the lateral direction. The test motions can be increased in amplitude to over 100% of the amplitudes defined by the RRS and plotted in Figures 18 through 20 before reaching the stroke limits of the table. If motions were needed with amplitudes much greater than 100%, the records could have been high-pass filtered, removing energy below the filter frequency, or new records could have been generated with higher low frequency limits.

Table 2. Required response spectra for TCB (5% of critical damping).

			S_{DS} (g) =	2.50
			A_FLX-H (g) =	2.50
			A_RIG-H (g) =	1.00
Target			A_FLX-V (g) =	1.675
& Lateral	Long		A_RIG-V (g) =	0.675
Actual Freq (Hz)	Actual Freq (Hz)	100% Horiz (g)	Actual Freq (Hz)	100% Vert (g)
1	1.5	1.48	2.0	0.99
1.3	1.6	2.50	2.1	1.68
3.5	3.5	2.50	3.5	1.68
8.3	8.3	2.50	8.3	1.68
33.3	33.3	1.00	33.3	0.68

Figure 17. Plotted required response spectra for TCB.

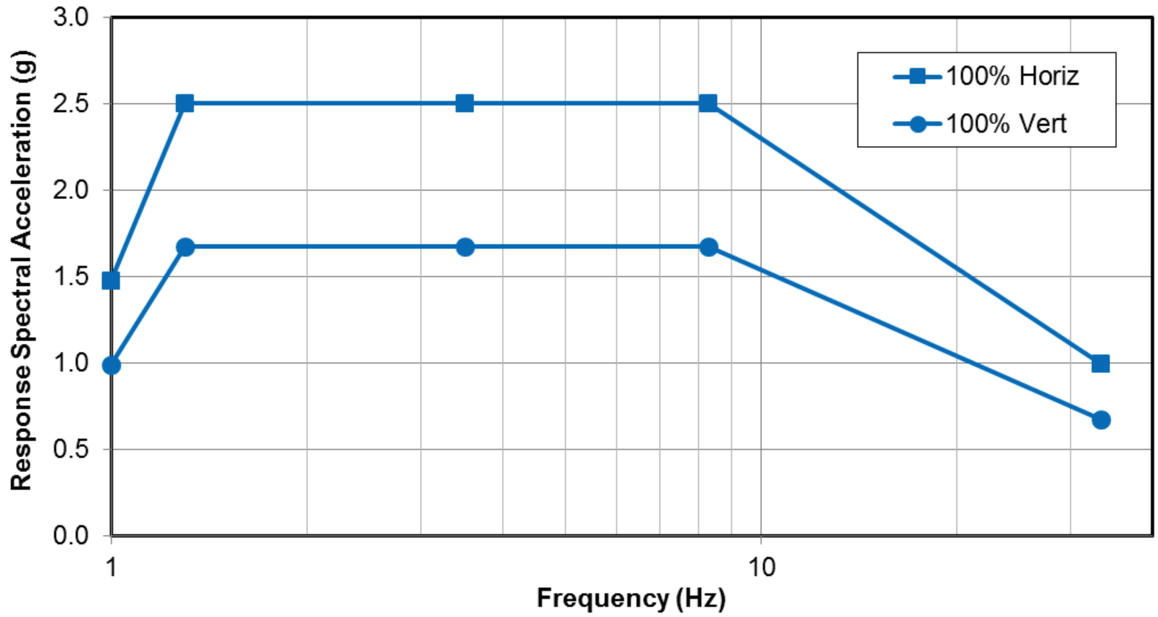


Figure 18. Longitudinal synthetic earthquake test record for testing TCB.

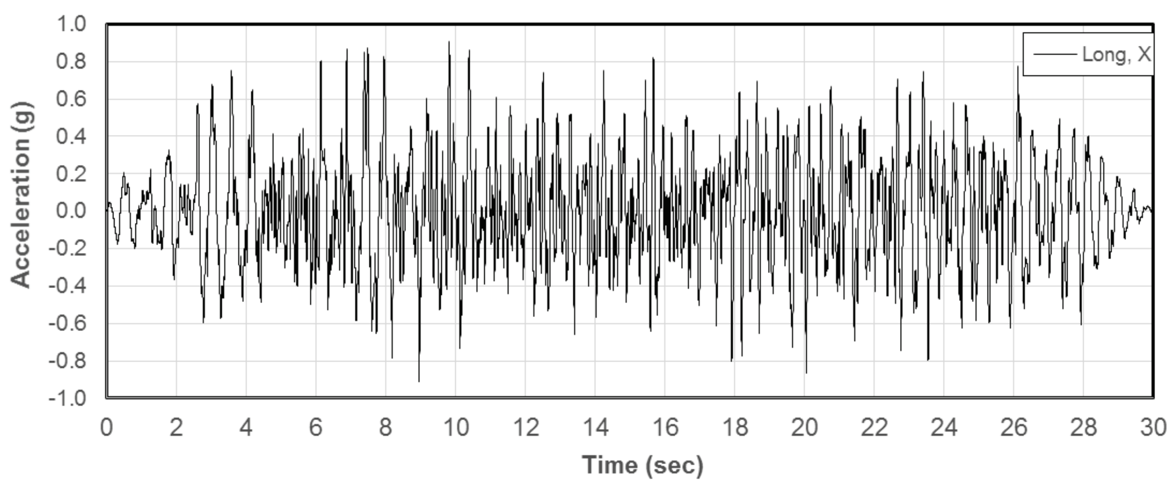


Figure 19. Lateral synthetic earthquake test record for testing TCB.

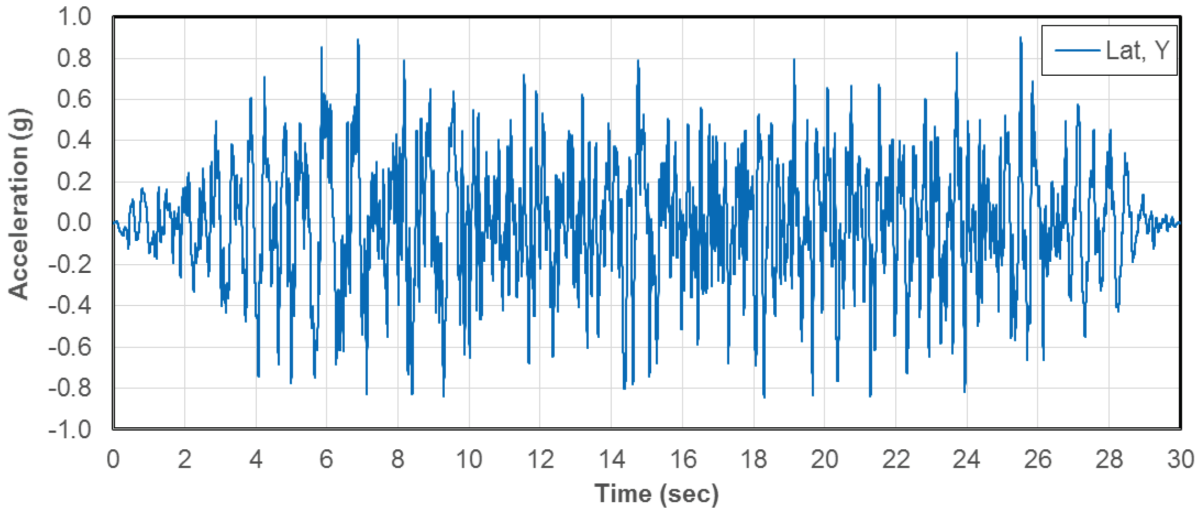
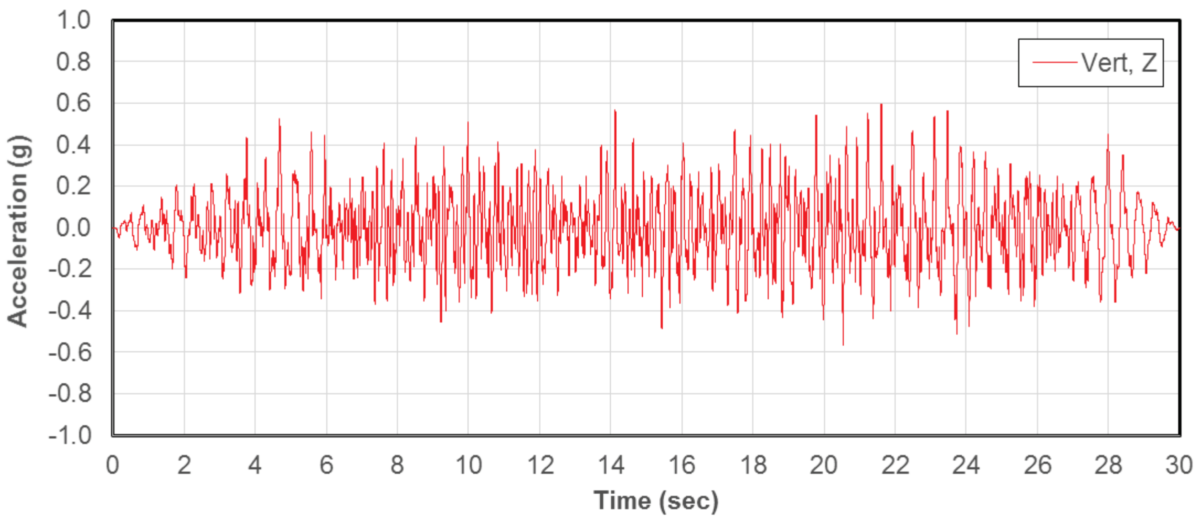


Figure 20. Vertical synthetic earthquake test record for testing TCB.



4 Instrumentation

The building was instrumented with accelerometers and cable extensometers (sometimes called string-pots or yo-yo gages) to measure the response of the TCB at the test footing, the bottom of the wall panels or brace and top of the wall panels or ceiling level. These sensors were installed at the corners of the TCB, because the roof and ceiling diaphragm tied the top of building together well and the fixture tied the footings of the building together. This means that measured response at any point at the ceiling level or foundation level could be accurately estimated by averaging the measurements from the same elevation recorded at the building corners. The accelerometers measured absolute accelerations and were used to define the displacements (when double integrated) of the building and dynamic response and inertial loading of the building. The cable extensometers were used to measure the absolute horizontal displacements of the TCB, where the instruments were mounted off the shake table to a stable reference, and their wires were attached to the building.

4.1 Accelerometers

The building was well tied together at the following elevations: (1) the top and bottom of the footings, where they were bolted to the rigid test fixture; (2) the ceiling; and (3) roof levels if the ceiling and roof diaphragms remain intact. Prior to chicken mesh and stucco installation, it looked doubtful that the ceiling and roof would remain intact, but once these were applied, it looked like these diaphragms would remain intact. Due the limitations on available sensors it was decided that the overall building response could most effectively be measured with accelerometers at the northwest and southeast corners and along the center of the west wall. This response was measured at the elevations of the footings, bottom of the wall panels and top of wall panels at the ceiling elevation. If the ceiling and roof diaphragms did fail the accelerometers at the center of the west wall would provide a basis for estimating the overall building response at the interior. It was expected that one end of the building would begin to fail before the other, and if the roof and ceiling diaphragms failed, interior frames could fail in the lateral (Y) direction before the end frames. The table itself will pitch and roll a small amount and this too should be measured and accounted for. Therefore, triaxial accelerometers were installed on the top of the footing at the northwest corner (A1x, A1y and A1z); lateral and vertical acceleration at the center of the west wall (A2y and A2z); and longitudinal and lateral acceleration at the southeast corner (A3x and A3y). Motions recorded at these three locations can be used to measure the overall motion of the building at the footing elevation in all three translations and rotations.

The accelerometer names all begin with A, followed by the sensor number for a particular location, and followed by an x, y or z indicating the direction of measurement. Building response motions will be measured at other locations; those measurements could have been corrected by the amounts measured at the base of the building to determine building deformations (i.e., if accelerations are double integrated to define displacements). Table 3 lists each accelerometer name, orientation, coordinates, location description, purpose, full measurement range amplitude and resolution for these data. The accelerometer gains were set so that the full amplitude for most accelerometers was set equal to 30 g, and the data were recorded at 16 bits, so that the resolution of this data became 0.0009 g (i.e., 30 g/2^{15}).

Table 3. TCB accelerometer locations, purpose, measurement range and resolution.

Sensor	X (in.)	Y (in.)	Z (in.)	Location	Purpose	Full Range (g)	Resolution (g)
ATLG				Inside the shake table	Average motion	15	0.0005
ATLT						15	0.0005
ATV						15	0.0005
ATROLL	TESS platform average acceleration and Pitch rotational acceleration					6883	0.2100
ATPITCH						6290	0.1919
ATYAW						6290	0.1919
A1x	2.0	2.5	0.0	Northwest corner on top of footing	Base of building motion	30	0.0009
A1y						30	0.0009
A1z						30	0.0009
A2y	209.0	3.5	-4.4	Center West face, South face of footing	Base of building motion	30	0.0009
A2z						30	0.0009
A3x				Southeast corner, North face of footing	Base of building motion	30	0.0009
A3y	380.5	206.5	-3.5			30	0.0009
A4x	22.5	7.7	1.2	NW corner, on West face brace	Failure at base of columns	30	0.0009
A4y	3.5	19.7	5.5	NW corner, on North wall panel		50	0.0015
A4z						50	0.0015
A5x				Center of West face on wall panel	Failure at base of columns	30	0.0009
A5y	212.0	2.0	3.4			30	0.0009
A6x	374.0	202.0	1.5	SE corner, on East face brace	Failure at base of columns	30	0.0009
A6y	390.0	191.0	-1.5	SE corner, below South face brace		50	0.0015
A6z						50	0.0015
A7x				Northwest corner at top of West wall panel	Top of building	50	0.0015
A7y	18.0	2.5	91.5			50	0.0015
A8x				Center of West face at top of wall panels	Top of building	50	0.0015
A8y	208.0	2.5	90.8			50	0.0015
A9x				Southeast corner at top of East wall panels	Top of building	50	0.0015
A9y	378.0	206.0	91.5			50	0.0015
*Origin is the NW corner top of the footing							

The top of Table 3 also lists the shake table motions themselves that were recorded on the data acquisition system. Only the shake table longitudinal or X-axis (ATLG) table lateral or Y-axis (ATLT), table vertical or Z-axis (ATV) and pitch motion (rotation about the Y-axis [ATPITCH]) were recorded. Only the pitch rotations were recorded, because the test fixture and building were cantilevered off the table the furthest in the X-axis, causing the greatest potential fixture deformation and overturning rocking the test fixture in this direction.

Accelerations were recorded on the exterior wall panels or braces just above the tops of the footings. The data listed in Table 3 indicate that these accelerometers were almost directly above the accelerometers on the footings. The measurements at these location (A4x, A4y, A4z, A5x, A5y, A6x, A6y and A6z) can be used to determine if and when failures occur at the base of the columns, between the two sets of accelerometers. The measured acceleration at the bottom of the panels or braces can also be multiplied by the lumped weight at the footing (minus the footing weight themselves) to obtain an estimate of the inertial loads at this elevation.

Accelerations were also recorded top of the wall panels at the northwest corner (A7x and A7y), center of the west face (A8x and A8y) and southeast corner (A9x and A9y). No vertical accelerations were measured at these locations because the vertical motions at this elevation should be essentially the same as at the bottom of the wall panels or wall braces where vertical accelerations were measured at A4z and A6z. The accelerations measured at the top of the walls can be multiplied by the lumped weight at the ceiling elevation to estimate the horizontal inertial loading at the top of the building.

4.2 Cable extensometers

Cable extensometers were used to directly measure the absolute displacements of the TCB. When the deformations of the building are the key parameter being measured, it should be more accurate to directly measure displacements rather than integrating acceleration measurements. Deformations of a given component can be determined by measuring the displacements at two ends of the components and subtracting the one from the other. For example, the deformation of a wall panel can be calculated by measuring the displacement at the top of a panel and subtracting the displacement measured directly below at the bottom of the same panel. The cable extensometers will generally measure lower frequency motions than the accelerometers because the displacements are dominated by the low frequency content. Conversely, the accelerometers are dominated by relatively higher frequency motions, which define the inertial loading of the building.

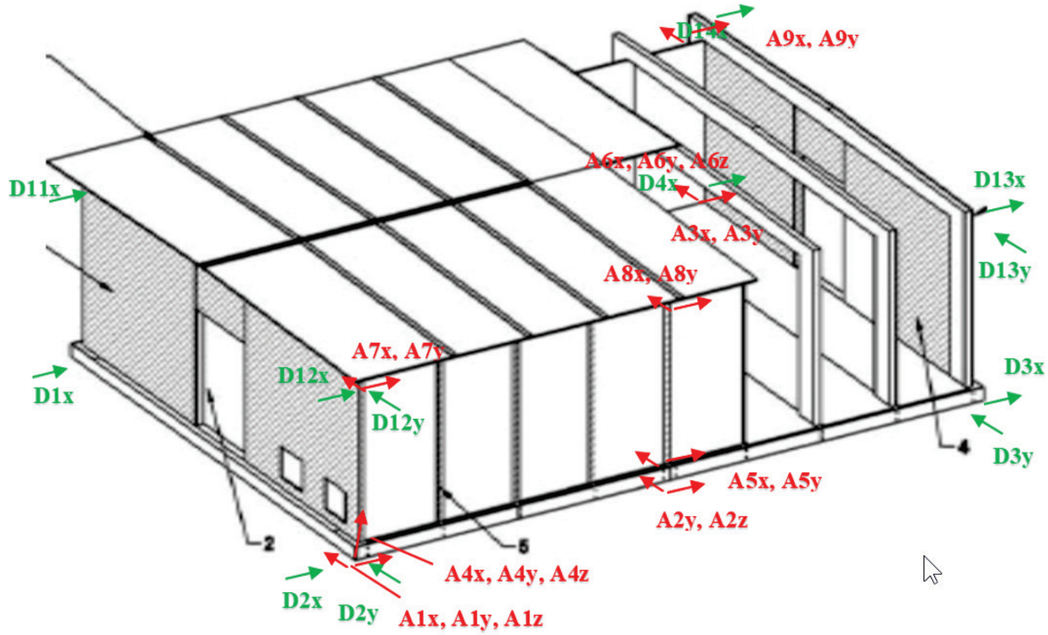
Table 4 lists data pertaining to the cable extensometer sensors (by name), which show that all displacement measurements were recorded in the x and y directions. The displacements were recorded at the top of the footings and top of the building on the wall panels. Displacements were measured in the X-direction at all four corner footings (D1x, D2x, D3x, D4x), but in the Y-direction at only the northwest and southwest corners. Displacements were measured in the X-direction at the top of the wall panels at the four corners (D11x, D12x, D13x and D14x) almost directly above the locations on the footings. Similarly, displacements were measured in the Y-direction at only the northwest and southwest corners at the top of the panels at D12y and D13y. These displacement measurements were used to estimate the overall building deformation in the in both horizontal axes, and at all four corners of the building. For example, the building deformation in the X-direction, measured at the northeast corner is D11x minus D1x. The horizontal deformation in the X-direction could be different at all four corners, depending on where wall panels fail, and separate. In the Y-direction (short direction of the building), the deformation in the northeast and northwest corners should be the same as long as the building frame, including the short wall braces stay intact – in other words D12y minus D2y should reasonably estimate the deformation in the Y-direction along the entire north face of the building. The same applies to the south face of the building, where D13y minus D3y measures this deformation.

Table 4. TCB cable extensometers locations, purpose, measurement range and resolution.

Sensor	X (in.)	Y (in.)	Z (in.)	Location	Purpose	Full Range (in.)	Resolution (in.)
D1x	0.0	198.0	-2.0	NE corner, N face of footing	Top of footing	5	0.00015
D2x	0.0	14.0	-2.5	NW corner, N face of footing		5	0.00015
D2y	14.0	0.0	-2.5	NW corner, W face of		10	0.00031
D3x	399.0	12.0	-2.5	SW corner, S face of footing	Top of footing	5	0.00015
D3y	387.0	0.0	-2.0	SW corner, W face of		10	0.00031
D4x	395.0	202.5	-2.5	SE corner, S face of footing		5	0.00015
D11x	4.0	195.0	97.0	NE corner, top of N wall	Top of building	20	0.00061
D12x	3.0	14.0	91.0	NW corner, top of N wall		20	0.00061
D12y	14.0	2.5	93.5	NW corner, top of W wall		20	0.00061
D13x	394.5	14.5	92.0	SW corner, top of S wall	Top of building	10	0.00031
D13y	382.0	2.5	93.5	SW corner, top of W wall		20	0.00061
D14x	391.5	192.0	96.5	SE corner, top of S wall		10	0.00031
*Origin is the NW corner top of the footing							

Figure 21 graphically shows the location and orientation of the accelerometers and cable extensometers on a schematic drawing of the indigenous materials TCB.

Figure 21. Schematic drawing of TCB showing accelerometer and cable extensometer locations.



5 Other Details to Prepare for Seismic Tests

Several details were developed before seismic testing, including conducting an AHA, defining video recording details, and detailed test steps.

5.1 AHA and critical lift plan

Prior to building construction an AHA was done, defining hazards related to building construction, testing and demolition. Each hazard was described, the severity of risk and likelihood of occurring, and methods for mitigating them were developed. The AHA was reviewed by test participants and the ERDC-CERL safety manager. The project manager, and author of this report, made sure the mitigation steps were followed. The AHA was updated a few times when new hazards were encountered or new mitigation approaches were developed. The final version of the AHA is include in Appendix A of this report.

One mitigation step that was explained earlier was the construction of the wood deck and use of the house wrap – this was explained in detail earlier and will not be repeated here.

Another important mitigation step was installing nylon straps through the four corners of the building, through the roof and ceiling panels, around the end frames and short wall brace top member, and through the interior and exterior wall panels. These straps were supported by a spreader frame and the ERDC-CERL overhead crane as shown in Figure 22. Two additional straps were installed through the roof and ceiling panels and around the two ends of the center frame. These straps were slack during the seismic tests, but if the building began to collapse, complete collapse would be prevented by the straps. The straps and spreader frame had sufficient capacity to lift the entire building.

After testing, it was decided to remove the entire building in one section, because the building was largely still intact with the base of each column failing. This was done because building demolition would create a substantial amount of dust, which could cause significant damage to the TESS hydraulics and electronics. The building was moved with the overhead crane, spreader frame and straps shown in Figure 22 – the spreader frame is above the straps in this picture.

Figure 22. Completed TCB with safety straps. The spreader frame is above the picture view.

Before moving the building, the only change was in adjusting the strap length so that the load would be well shared between the straps, with most of the weight lifted by the four corners, because these straps were tied into the braces and wall panels in addition to the frame, ceiling and roof panels at the center of the building. This method of building removal required a revision of the Activity Hazard Analysis after testing.

The bolts anchoring the footings to the fixture were removed and the entire building, with the footings were moved to the large south door so that one end of the building was outside. Then the two end bays (frames and panels) were cut off with a wet saw. Figure 22 shows that all joints were covered with substantial chicken mesh and stucco. The building components could not be separated from one another and removed without first removing this joint material, with either saws or impact tools. The AHA explains that the wet saw could not have been used over the table because the resulting slurry would leak down to the TESS hydraulics and electronics. A dry saw or impact tools such as a jack hammer, would create an incredible amount of dust, causing damage not just to TESS components, but also other electronics and other items throughout the TESS high-bay.

Therefore, the moving of the building outside as a single intact unit was a key step as described in detail in the AHA.

5.2 Video recording

Each seismic test was recorded using two camcorders, where one was positioned at the northeast corner and the other at the southwest corner, so that all four faces of the building could be seen. The northeast camcorder was installed on a bracket that was attached to a high-bay column, and the southwest one was installed on a tripod.

5.3 Detailed test steps

The following test steps were followed to test this TCB:

Model measurement tests were conducted, which consisted of random motion tests with a frequency range of 0 to 128 Hz and a 3-sigma amplitude value of 0.2 g, for translations and 50 degrees/s^2 for rotational degrees of freedom. The motions were uniaxial and had a duration of about 90 seconds in each direction, including the rotational degrees of freedom (roll, pitch and yaw). These random motion tests were required for creating a compensation model used to control the shake table for the remaining tests. The master gain used in the control of these tests was set at 30% and this value remained the same for all subsequent tests of this TCB.

System identification (SI) tests were conducted, which were done using uniaxial sine-sweep motions in each translational degree of freedom. These tests used an amplitude of 0.05 g, a sweep rate of 2 octaves per minute (i.e., doubling in frequency every 30 seconds) and swept through a frequency range of 1.3 through 33.3 Hz. The tests began in the longitudinal direction (long axis of the building), followed by lateral and vertical.

The 1st seismic test was conducted using 10% of the synthetic earthquake motions that were generated to fit the RRS spectra, plotted in Figure 17. These motions were triaxial as show in Figures 18 through 20. The motions in both horizontal axes were greater than the vertical axis based on the greater RRS plotted in Figure 17. The building was inspected inside and out and minor damage was documented with photographs and notes.

The 2nd seismic test was conducted using 25% of RRS defined motions. The building was again inspected and the slightly greater damage was documented with pictures and notes.

If significant damage had occurred, after any test, the SI and possibly model measurement tests might be repeated but they were not needed.

The 3rd seismic test was conducted using 50% of the RRS defined motions. Much greater damage did occur, and this will be presented with notes and pictures in the next chapter of this report.

The 4th and final seismic test was conducted using 100% of the RRS defined motions. The damage after this test was considerable, and will be documented with motion records, notes and pictures.

6 Seismic Test Results

SI tests were conducted using uniaxial sine-sweep motions as defined in step 2 above. Then seismic tests were conducted at 10%, 25%, 50% and 100% of the RRS defined motions. The following sections describe the results of these tests.

6.1 SI tests

SI tests were conducted using sine-sweep motions to define the natural frequencies of the TCB in all three axes. The natural frequencies have a major impact on the response of the TCB to the seismic motions because the building has an amplified response due to support motions at those frequencies and in the modes of vibration associated with them. For example, the building was expected to have a relatively low frequency mode of vibration in the lateral or short direction of the building and the TCB was expected to oscillate at that frequency at amplitudes higher than at the footings of the building.

The sine-sweep motions in all three directions had an amplitude of 0.05 g, began at 1.3 Hz and swept up to 33.3 Hz, at a sweep rate of 2 octaves per minute (frequency doubling every 30 seconds), result in a test duration of 140 seconds. The 0.05 g motions represent a relatively low amplitude for SI tests, but these were used to minimize damage during the excessive number of building motion cycles that would occur during these tests, which were expected to cause small cracks in the concrete.

Transfer functions were generated between the response acceleration at elevations on the building and the footing of the building, recorded at the northwest and southeast corner and the center of the west wall. Appendix B plots the transfer functions, and Table 5 lists the natural frequencies and modes of vibration determined from the peaks of the transfer functions. For example, for the longitudinal (X-direction) sine-sweep test, transfer functions were generated for bottom of the brace on the west face at the northwest corner over the northwest corner footing (A4x/A1x); and the northwest corner top of the west wall panel over the northwest corner footing (A7x/A1x). A transfer function is the fast Fourier transform of the measured acceleration at one location over another location, providing a plot of the ratio of the two in the frequency domain.

Table 5. TCB natural frequencies and modes of vibration from SI sine-sweep tests.

#	Transfer Function	Measured 1st Mode	Measured 2nd Mode	Mode of Vibration
1	A1x/ATLG	-	-	Fixture response
2	A1y/ATLT	-	-	
3	A1z/ATV	21	-	
4	A4x/A1x	8.3	-	Bottom of brace or wall panel vs. footing in x-direction
5	A5x/A1x	8.3	-	
6	A6x/A3x	6.8	36	
7	A7x/A1x	7.7	-	Top of building or wall panels vs. footing in x-direction
8	A8x/A1x	7.7	-	
8	A9x/A3x	6.7	-	
9	A4y/A1y	5.1	-	Bottom of wall panel or brace vs. footing in y-direction
10	A5y/A2y	20.5	-	
11	A6y/A3y	5.1	35.6	
12	A7y/A1y	5.1	-	Top of building or wall panels vs. footing in y-direction
13	A8y/A2y	5.1	-	
13	A9y/A3y	5.1	-	
14	A4z/A1z	-	-	Bottom of wall panel or brace vs. footing in z-direction
15	A6z/A1z	33	-	

Figure B-2 shows and Table 5 lists that in the short (Y-axis) of the TCB, the building responded with a sharp resonant response at 5.1 Hz, especially when measured at the top of the building at sensors A7y, A8y, and A9y. Even at the northwest corner, north wall panel (A4y) and southeast corner, south face brace (A6y) there was a clear resonant response at 5.1 Hz. In contrast to A4y and A6y, at the west face of the building, on the wall panel at A5y, a resonant response was not seen near 5.1 Hz. This indicates that the dominant building response in the short direction of the building was driven by an almost rigid body response of the short direction wall panels and braces in this direction, where the response at the bottom of the wall panels and braces was almost as great as at the top of the panels.

In the center of the building, where A8y measured the building response at the top of the wall panels, and A5y measured the response at the bottom of the same panels, the resonant response is seen at the top but not the bottom of the wall panels. The response of the top of the wall panels (A8y) is the same as at the top corners of the building (A7y and A9y), with the same frequency and similar amplification. The top of the building at the center moved the same as at the corners, because the roof and ceiling diaphragm tied the building together at this elevation very well. At the bottom center face of the west wall (A5y), directly below A8y, there was very little response near 5.1 Hz, because the interior frame at this location deformed in the short direction of the building, and the wall panel rotated out-of-plane with the frame. In other words, at the interior of the building, the bottom of the wall moved with the footing because the frame and wall panels were able to deform enough to allow the large motion at the roof relative to the footing in the short direction of the building.

At 20.5 Hz, there appears to be a second mode response of the building frame in the short direction of the building. This is a surprisingly sharp resonance and most likely associated with the deformation of interior building frames in the short direction of the building, driven by the mass near the frames and wall panels near the bottom of the frames. Figure B-2 shows that there is no similar response at the top of the building at this frequency, because the top of the building would be braced by the diaphragm. This higher frequency mode would provide minimal additional loading on the interior frames of the building because the relatively small amount of mass and high frequency would not be excited well by the relatively lower frequency seismic test motions.

There may also be a second mode response near the bottom of the building at the corners of the building measured at A4y and A6y at around 36 Hz. However, this high frequency mode would have little participating mass and would not be well coupled with the lower frequency content of the seismic test motions.

Therefore, the overall response of the building is expected to be dominated by the 5.1 Hz mode described above. As the tests progress and the building is damaged, the frequency will gradually decrease because the stiffness will decrease so that the dominant response will be at less than 5.1 Hz.

In the long direction of the building, the dominant resonant response was at 7.7 or 6.7 Hz (Figure B-1 and Table 5). These resonant peaks are not as sharp as in the short direction of the building. Figure B-1 and Table 5 show that the wall panels and braces appear to be more flexible at the east face where the resonant frequency at the top of wall (A9x) is at 6.7 Hz, compared to 7.7 Hz along the top of the west wall (A7x and A8x). Similarly, at the bottom of the walls, the measured frequency at the east face (A6x) is at 6.8 Hz, compared to 8.3 Hz at the west face (A4x and A5x). These transfer functions demonstrate that the building wall panels and braces deform as rigid blocks, where the top and bottoms of the wall move together in the long direction. The frequency in the long direction is greater than in the short direction because the building is much stiffer with the longer wall in the long direction. The building response in the long direction will be dominated by the response in this mode, and the frequency is expected to decrease as damage accumulates.

In the vertical direction, only two high frequency modes were seen. The first is associated with the deformation of the test fixture at 21 Hz (plotted in Figure B-3 as A1z/ATv). This transfer function is the motion of the footing in the northwest corner (A1z) relative to the average of the vertical acceleration measured inside the TESS (ATV). There appears to be another high frequency vertical mode measured at the southeast corner, below the bottom of the south face brace (A6z), with a frequency of 33 Hz. Both of these modes are at such high frequencies that vertical resonant response of the building (or test fixture), should have minimal loading effects on the building.

A much more critical vertical response of the building will be when the building vibrates in one of the horizontal axes, and deformation and even crack opening cause vertical pounding or impacts. These vertical impacts would occur periodically at the horizontal frequencies of the building.

6.2 Seismic test levels

The first seismic test used 10% of the motions (shown in Figures 17 to 20) because it was confidently thought that this level of motions would cause no damage and yet provide sufficient response to determine what test level to increase to that would cause minimal damage. The intent was to increase the test levels in large enough increments to reach the test level that would cause significant damage in a few numbers of tests. The test amplitude should not be increased too quickly because this could risk destroying the building in the first or second test and gather minimal data on the development and progression of failure.

On the other hand, if the increments were too small, the building could be damaged by an excessive number of motion cycles at very low levels, while the same building may have been able to withstand larger motions if the number of motion cycles were much less. This is particularly critical for a cementitious building model, were small cracks begin to form and grow at low test levels. As tests progress, more cracks form, existing cracks grow, cracks open and offset, and some cracks will have permanent offsets. In the case of this building model, the only reinforcing is chicken wire mesh, and for this weak aerated concrete, cracks will form at very low strains, so that the wires are only significantly loaded when cracks have formed and begin to open or offset. In contrast, the normal weight stucco was expected to be much stronger than the precast aerated material. Cracks will form in the weaker aerated concrete and stronger stucco as described above and when those cracks open and try to offset, the wire will be loaded and eventually stretch and debond from the concrete. The chicken mesh was packed into the precast elements and stucco in a dense enough manner so that extensive cracking of the concrete was expected before significant damage to the wires.

Numerous steel screws connect wall panels to the frames and also connect narrow brace elements to the frames. These screws will be loaded in shear as the building racks horizontally and panels try to move relative to the frame elements. The screws also connect the narrower brace elements to the frames. The concrete of the wall panels may crush when loaded in shear by the screws, or the screws could begin to yield and rotate in bending, within the wall panels. Eventually the screws could begin to pull out of the member to which they are anchored, or the head of the screw could pull through the wall panels. All of these would require a crushing of the concrete.

The concrete crack development and growth and tensile pulling apart of the chicken wire along with possible crushing of the concrete around the screws would all result in significant deformation and extensive visible damage to the building before any collapse hazard would be expected. The final test was expected to cause substantial damage to the building with extensive damage to screwed connections, cracking and crushing of concrete, and damage to the chicken mesh wires. Therefore, a relatively aggressive but reasonable sequence of seismic test levels was proposed a few days before testing (Table 6).

Table 6. Proposed seismic test levels.

Seismic Test #	Percent of Figure 18 Motions
1	10%
2	25%
3	50%
4	100%
5	150%
6	200%
7	300%
8	400%

If relatively little damage occurred after the 25% test, the next planned test would be for 50%; however, this level would have been reduced if significant damage had occurred at 25%. Similarly, if significant damaged occurred at 50%, the next test may have been at 75%. Noticeable damaged did occur at 50%, but it was still decided to conduct the next test at 100%, with the recognition that this might be the final test, but collapse would be unlikely. The 100% test was the final test not because collapse was possible, but because all the columns had completely failed in shear just above the footings. The concrete had completely failed in shear across this surface, and there were large horizontal offsets, while the chicken wire was badly damaged but still partly connecting frames to their footings. Further testing would not have produced useful data because the building above would be isolated from the motions below, and further testing would have simply further ground the concrete rubble along the failure plane and pulled apart the chicken mesh wire.

6.3 Method of reporting seismic test results

Seismic test results are documented in terms of measured accelerations and displacements, along with damage observations and photographs. The accelerations were measured at three levels in the building; (1) at the footings; (2) at the bottom of the wall panels or braces; and (3) on the exterior wall panels at or slightly higher than the elevation of the ceiling panels. The ceiling and roof panels created a roof diaphragm that tied the building together well at the top; this diaphragm remained intact throughout all the seismic tests. The building did deform significantly in both the horizontal X- and Y-axes, but it deformed little in torsion such that the motion on the north and south walls were almost equal.

At lower test levels, the building wall panels deformed slightly in shear and the frames deformed in flexure. The braces in the short direction of the building (Y-axis) would have carried a good portion of load in that direction. These braces were wider than the braces in the long direction of the building (X-axis) so they would have been stiffer than the braces in the long direction. There were an equal number of braces in both directions. However, in the short direction, there was approximately 48 linear feet of wall panels, while in the long direction there was about 126 linear feet. The relative stiffness of the wall panels compared to the braces is unknown, but it is reasonable to assume that a good portion of the load was carried by the braces in the short direction, while most of the load in the long direction was carried by the wall panels. The building frames would have carried relatively little load as they would have been fairly flexible in flexure. These frames however tied the wall panels together and anchored the braces. Most significantly, the chicken wire mesh and stucco tied adjoining wall panels together, forming continuous shear walls across their joints. This was particularly important in the long direction, but also in the short direction where the wall panels were tied together above the personnel doors at the ends of the building.

The loading of the building in both horizontal axes could best be approximated by averaging the accelerations measure at the bottom of the panels and top of the panels at the ceiling elevation. These average accelerations were multiplied by the tributary weight or weight that could most reasonably be lumped at the top ($W_{ceiling}$) and bottom (W_{Bot}) of the building (Table 1). Using the average acceleration rather than accelerations measured at particular accelerometers reduces high frequency spikes that may have resulted from localized response but would not be representative of the overall motion of the building. Appendixes C to F include plots of the measured accelerations from each test. The accelerations are plotted for the entire tests, followed by other plots that zoom in on short durations of the tests, and these plots illustrate how the average accelerations reduce high-frequency, localized spikes; and thereby are the best measure of the overall response and loading of the building.

The accelerations at the bottom of the building in the X-direction, Ax_{Bot} is:

$$AxBot = Average(A4x, A5x, A6x) \tag{1}$$

where:

A4x = the acceleration measured at the TCB northwest corner at the bottom of the west face brace (see Table 3 and Figure 21).
A5x = the acceleration measured at the center of the west face, at the bottom of the wall panel (see Table 3 and Figure 21).
A6x = the acceleration measured at the TCB southeast corner at the bottom of the east face brace (see Table 3 and Figure 21).

The accelerations at the top of the building, near the elevation of the ceiling in the X-direction, $Ax_{Ceiling}$ is:

$$AxCeiling = Average(A7x, A8x, A9x) \tag{2}$$

where:

A7x = the acceleration measured at the TCB northwest corner at the top of the west wall panel (see Table 3 and Figure 21).
A8x = the acceleration measured at the center of the west face, at the top of the wall panel (see Table 3 and Figure 21).
A9x the acceleration measured at the TCB southeast corner at the top of the east wall panel (see Table 3 and Figure 21).

The accelerations at the bottom of the building in the Y-direction, Ay_{Bot} is:

$$AyBot = Average(A4y, A5y, A6y) \quad (3)$$

where:

A4y = the acceleration measured at the TCB northwest corner at the bottom of the north wall panel (see Table 3 and Figure 21).
A5y = the acceleration measured at the center of the west face, at the bottom of the wall panel (see Table 3 and Figure 21).
A6y = the acceleration measured at the TCB southeast corner at the bottom of the south face brace (see Table 3 and Figure 21).

The accelerations at the top of the building, near the elevation of the ceiling in the Y-direction, $Ay_{Ceiling}$ is:

$$AyCeiling = Average(A7y, A8y, A9y) \quad (4)$$

where:

A7y = the acceleration measured at the TCB northwest corner at the top of the west wall panel (see Table 3 and Figure 21).
A8y = the acceleration measured at the center of the west face, at the top of the wall panel (see Table 3 and Figure 21).
A9y = the acceleration measured at the TCB southeast corner at the top of the east wall panel (see Table 3 and Figure 21).

These measured and calculated average accelerations are shown in Figures C-4 through C-7 for the 10% seismic test, plotted between 13 and 15 seconds.

The inertial loads applied to the building are estimated to be the forces applied to the bottom of the building and those applied to the top at the ceiling elevation. The TCB eventually fails by shear failure at the base of the building frames, directly above the footings. Therefore, the critical loading is the sum of the inertial forces applied to the bottom of the building and those applied to the top at the ceiling elevation. These forces will be additive if the building response is dominated by a first mode of vibration but cancel each other if the building response is dominated by a second mode response where the ceiling is vibrating out-of-phase with the bottom. The top and bottom loading will never be perfectly in phase, and the most reasonable estimate of the loads applied at the base of the columns, or base shear, Vx and Vy is to directly sum these forces at each instant in time that the accelerations were measured. This total force at the base of the columns in the X-direction, Vx, can be expressed as:

$$Vx = FxBot + Fxceiling = (WBot)(AxBot) + (Wceiling)(Axceiling) \quad (5)$$

where:

Fx_{Bot} = the inertial force applied in the X-direction at the bottom of the wall panel elevation.
$Fx_{Ceiling}$ = the inertial force applied in the X-direction at the top of the building at the ceiling elevation.
W_{Bot} = the weight of the building lumped at the bottom of the wall panel elevation, equal to 12,348 lbs, as defined in Table 1.
Ax_{Bot} = the average acceleration in the X-direction at the bottom of the building as expressed by equation 1.
$W_{Ceiling}$ = the weight of the building lumped at the ceiling elevation, equal to 24,241 lbs, as defined in Table 1.
$Ax_{Ceiling}$ = the average acceleration in the X-direction at the top of the building as expressed by equation 2.

The base shear force at the base of the columns in the Y-direction, Vy, can be expressed as:

$$Vy = FyBot + Fyceiling = (WBot)(AyBot) + (Wceiling)(Ayceiling) \quad (6)$$

where:

Fy_{Bot} = the inertial force applied in the Y-direction at the bottom of the wall panel elevation.
$Fy_{Ceiling}$ = the inertial force applied in the Y-direction at the top of the building at the ceiling elevation.
W_{Bot} = the weight of the building lumped at the bottom of the wall panel elevation, equal to 12,348 lbs, as defined in Table 1.
Ay_{Bot} = the average acceleration in the Y-direction at the bottom of the building as expressed by equation 3.
$W_{Ceiling}$ = the weight of the building lumped at the ceiling elevation, equal to 24,241 lbs, as defined in Table 1.
$Ay_{Ceiling}$ = the average acceleration in the Y-direction at the top of the building as expressed by equation 4.

Horizontal building displacements were measured at the corners indicated in Table 4 and Figure 21 at the footing and top of wall panel elevations. These were measured relative to fixed references off the shake table so that the displacements are absolute measurements. The purpose of these measurements is to determine horizontal building deformations by subtracting the displacements measured at the footings from the ceiling displacements directly above. For example deformation of the building in the X-direction measured at the northeast corner was determined by subtracting the displacement at that location measured at the footing, D1x, from the displacement directly above, near the ceiling, D11x. Displacements were measured at all four corners in the X-direction, and the average deformation in the X-direction, Dx_{Avg}, was calculated as follows:

$$DxAvg = Average[-(D11x - D1x), -(D12x - D2x), (D13x - D3x), (D14x - D4x)] \quad (7)$$

where:

D11x-D1x= the deformation measured in the X-axis at the northeast corner. The negative sign is added to this deformation in equation 7, so that the deformations are in phase with the measurements from the south end of the building. This is because when the building deforms in the positive X-direction, the cables at the south end will retract, while the cables at the north end extend.

D12x-D2x= the deformation measured in the X-axis at the northwest corner. The negative sign is added to the deformations because these measurements were also taken at the north end.
D13x-D3x= the deformation measured in the X-axis at the southwest corner.
D14x-D4x= the deformation measured in the X-axis at the southeast corner.

In the Y-direction, or short direction of the building, displacements were only measured on the west face, at the northwest and southwest corners. Because the building frames spanned in the east-west direction, the Y deformations were expected to remain the same along the entire north or south face of the building. The average deformation in the Y-direction, Dy_{Avg}, was calculated as follows:

$$DyAvg = Average[(D12y - D2y), (D13y - D3y)] \tag{8}$$

The best summary of the performance of the building is to plot the calculated base shear versus the average deformation in the same direction. The plots are essentially hysteretic envelopes that are often plotted showing the results of cyclic tests of shear panels tested using a strong wall and hydraulic actuators. The velocities used in cyclic tests are very slow, and they use displacement-controlled load protocols with gradually increasing displacement amplitude. The cyclic tests do define the non-linear response of wall panels, often up to very large deformations, but they do not account for dynamic loading effects or redistribution of loads to other building components. The hysteretic load versus deformation plots presented here do account for the whole building behavior, but also focus on the deformation behavior of the building, focusing on the deformation of the most critical components. Several observations will be made from these plots in the sections that follow for each seismic test.

Appendixes C to F include plots of the measured accelerations, average accelerations, inertial forces, base shears, measured displacements, deformations, and average deformations, while the summary base shear versus average deformation are plotted in the sections that follow.

6.4 Seismic test at 10%

Appendix C includes measured and average accelerations for the bottom and top of the building, the inertial loads, base shears, displacements, and average building deformations in both horizontal axes for the 10% seismic test. Several of these plots show that the loading and deformation of the building oscillates in the two horizontal axes at the natural frequencies of the building.

Figure 23 plots the base shear versus the average deformation in the X-axis. The data plotted here show that the deformations were relatively small, with the maximum averaged deformation reaching only 0.03 in. Figure 23 also plots an estimate of the stiffness of the building in this direction, with a value of 210,000 lbs/in. The decrease of this stiffness in future tests will be one measure of the damage condition, because the building will soften as damage occurs. Figure 24 plots similar data for the Y-axis, showing the building deforms as much as 0.08 in. in one direction. The stiffness in the Y-axis (or short direction of the building) is much less than the X-axis with a value of 55,000 lbs/in. These plots and visual observations show that there was little damage in the 10% seismic test with only minor cracking in the columns near the footings and the short wall panels. There was considerable cracking of concrete where the corner columns connected to their footings. This damage was to the stucco applied over the chicken wire, with almost no damage to the columns themselves.

Figure 23. Base shear versus deformation and X-axis stiffness, 10% seismic test.

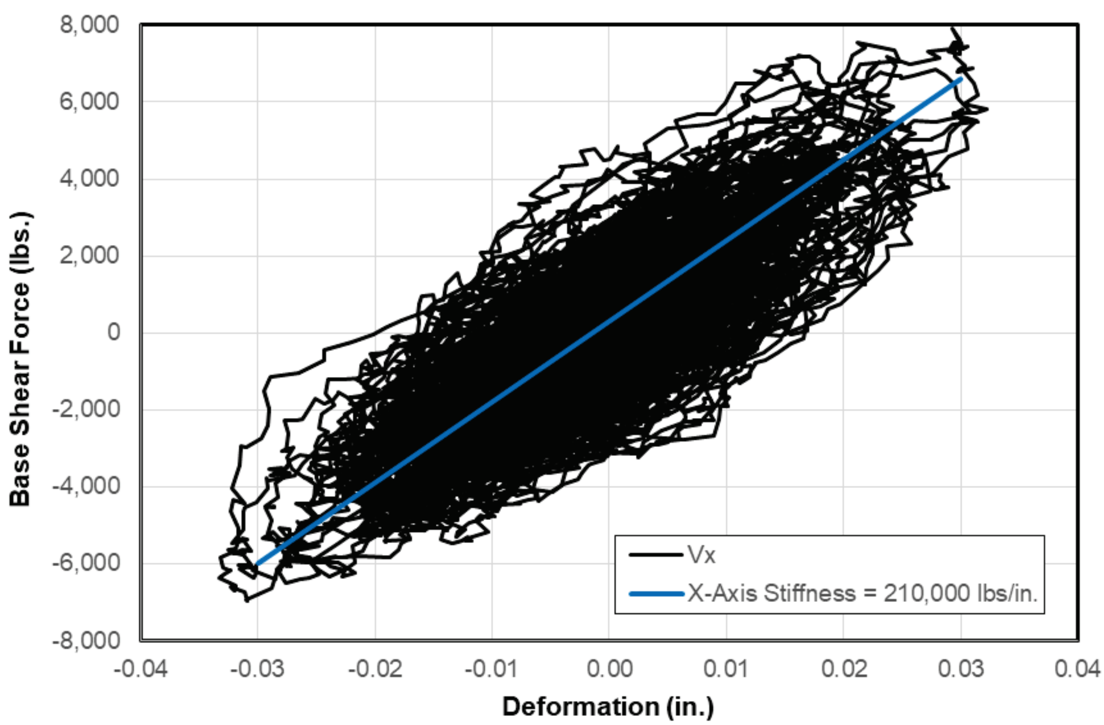
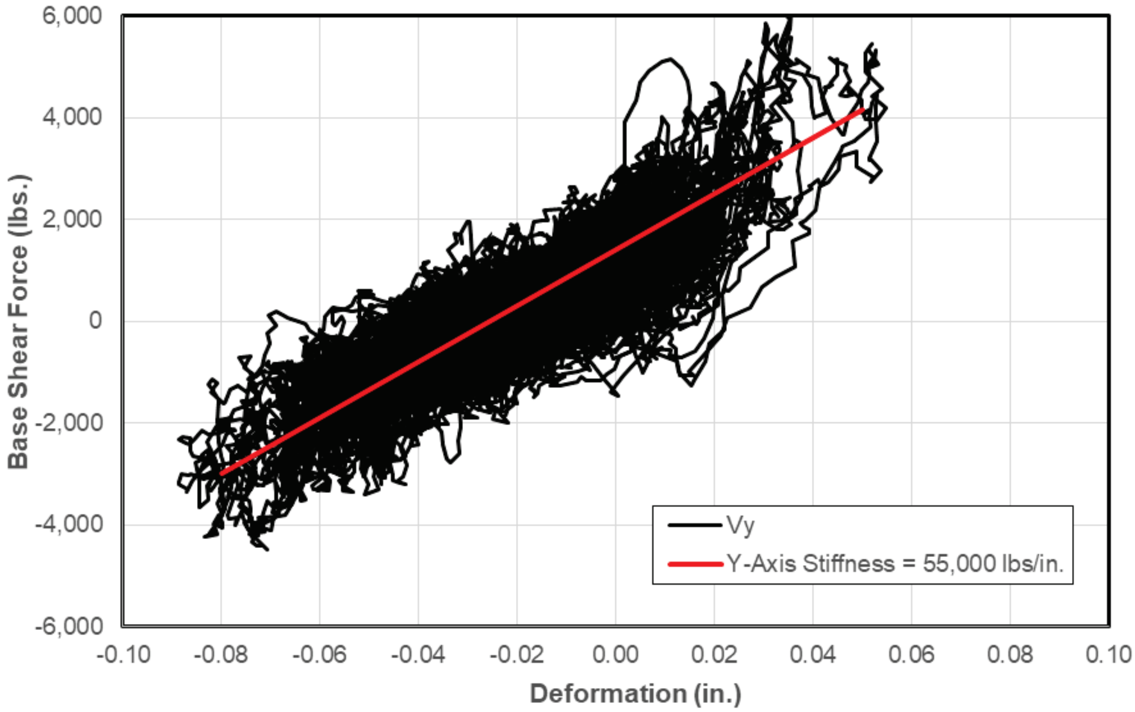


Figure 24. Base shear versus deformation and Y-axis stiffness, 10% seismic test.



6.5 Seismic test at 25%

Appendix D includes measured and average accelerations for the bottom and top of the building, inertial loads, base shears, displacements, and average building deformations in both horizontal axes for the 25% seismic test. The loading and deformation plots show that the building oscillates at the same building natural frequencies as it had in the 10% test.

Figure 25 plots the base shear versus the average deformation in the X-axis. The deformations in the X-axis increased to over 0.10 in. Figure 25 shows that the estimated X-axis building stiffness decreased to 196,000 lbs/in., a 7% decrease, indicating some softening and damage relative to the 10% test. Figure 26 plots the base shear versus deformation for the short direction of the building (Y-axis). This test shows that there was over twice the deformation, and the stiffness decreased to 50,000 lbs/in., a 9% decrease, indicating slightly more damage than in the X-direction.

Figures 27 and 28 show cracking and crushing damage that occurred to the stucco and chicken wire mesh at joints of the southeast and southwest column connections to the footing after the 25% test. There was very little damage to the precast frames themselves where the columns intersected their footings.

Figure 25. Base shear versus deformation and X-axis stiffness, 25% seismic test.

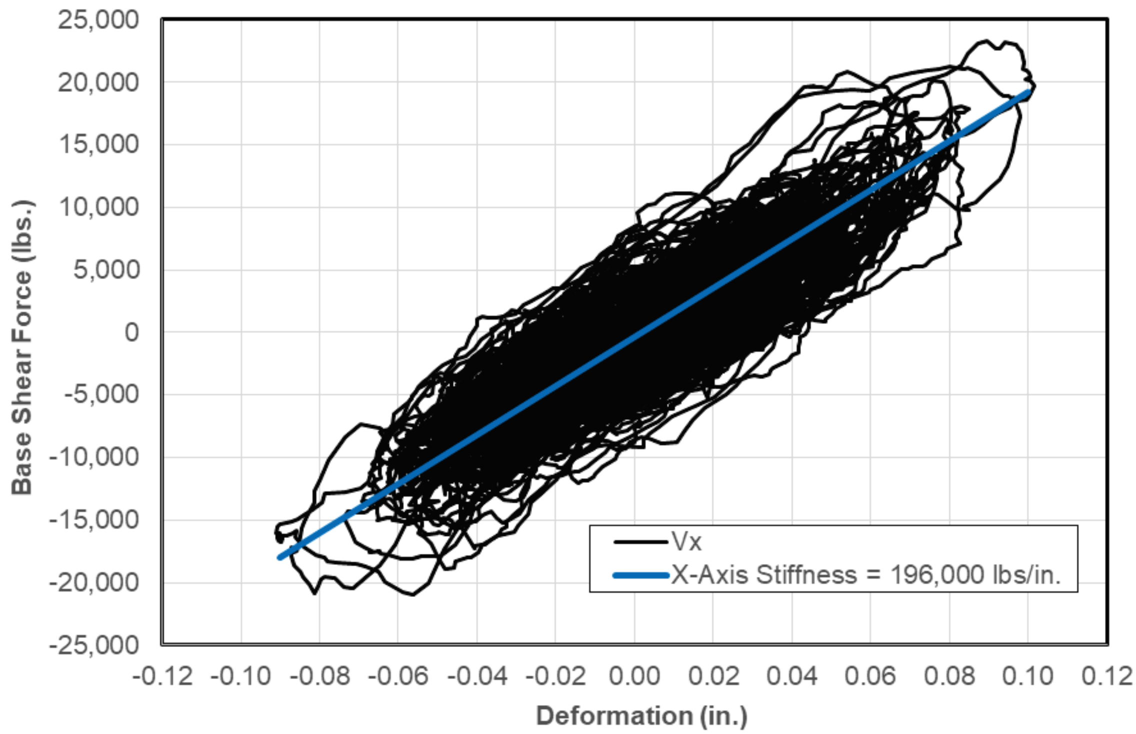


Figure 26. Base shear versus deformation and Y-axis stiffness, 25% seismic test.

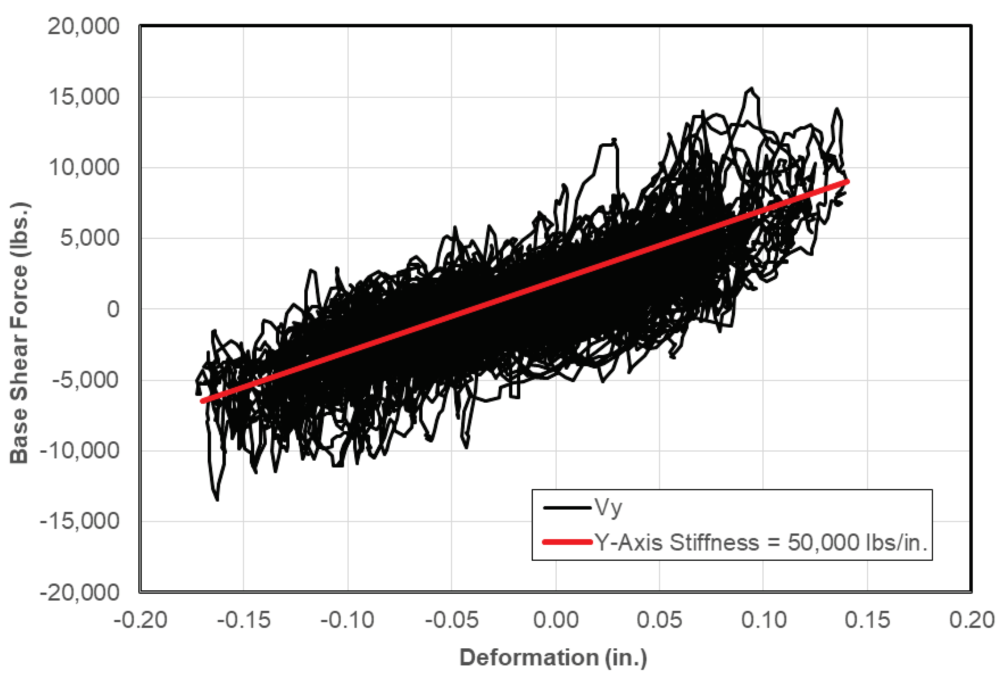


Figure 27. Stucco damage at the southeast column to footing connection, 25% test.

Figure 28. Stucco damage at the southwest column to footing connection, 25% test.

6.6 Seismic test at 50%

Appendix E includes measured and average accelerations for the bottom and top of the building, inertial loads, base shears, displacements, and average building deformations in both horizontal axes for the 50% seismic test. The loading and deformation plots show that the building oscillates at the same building natural frequencies as it had in the 10% test.

Figure 29 plots the base shear versus the average deformation in the X-axis. The deformations in the X-axis increased to 0.25 in. Figure 29 shows that the estimated X-axis building stiffness decreased to 124,000 lbs/in., a 37% decrease, indicating significant softening and damage relative to the 25% test. Figure 29 shows a plot of three segments of base shear versus deformation in time showing that the building becomes more flexible after 5.6 seconds and again after 9.8 seconds. Figure 30 shows a plot of the base shear versus deformation for the short direction of the building (Y-axis). This test shows the stiffness decreased to 34,000 lbs/in., a 32% decrease, indicating significant softening and damage. The segments plotted in time do not indicate a dramatic decrease in stiffness as the test progressed.

Figure 29. Base shear versus deformation and X-axis stiffness, 50% seismic test.

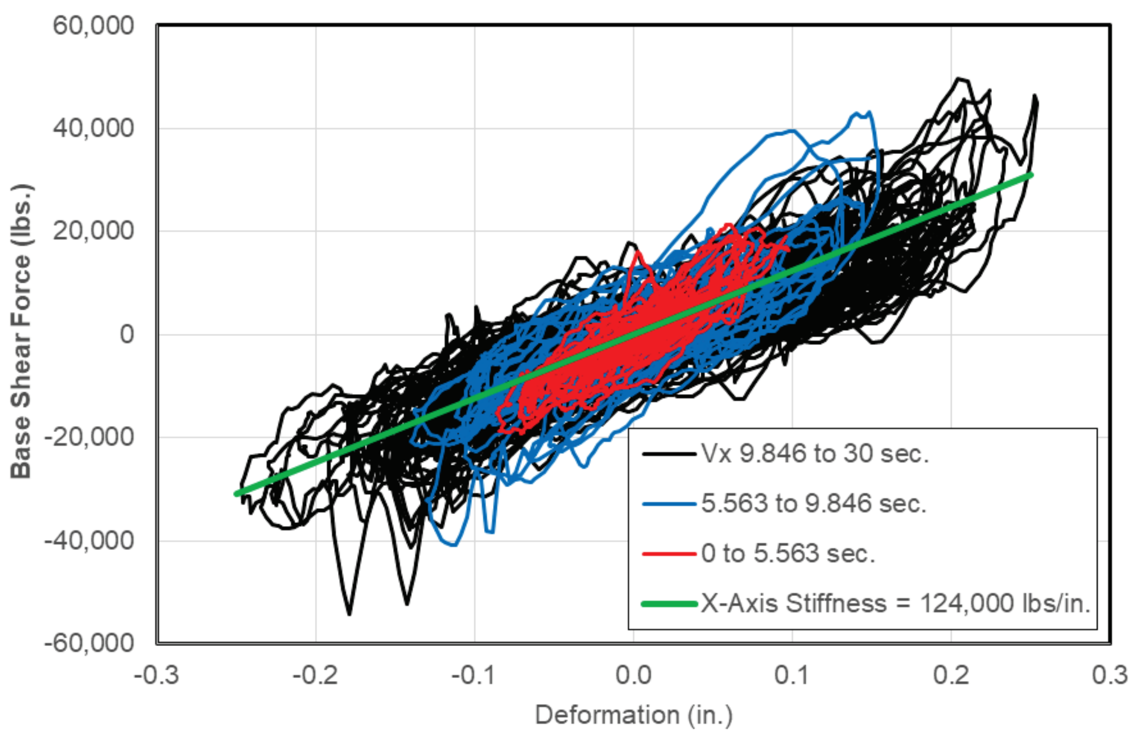
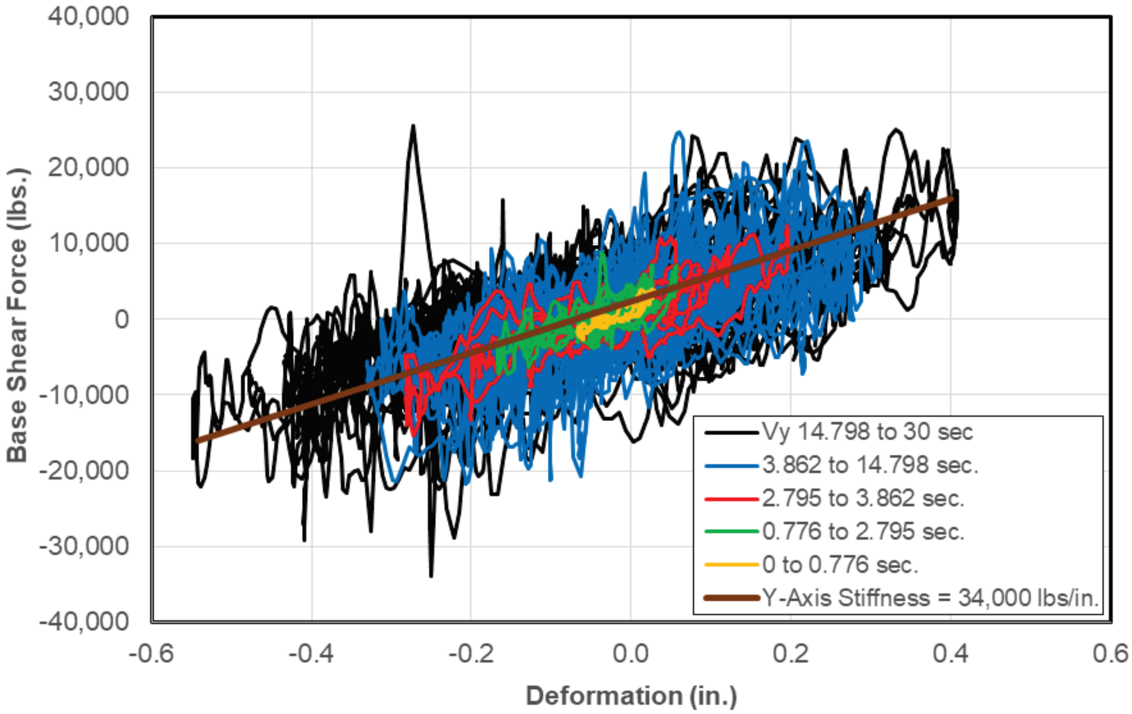


Figure 30. Base shear versus deformation and Y-axis stiffness, 50% seismic test.



The 50% seismic test caused much greater damage to the stucco, plus some cracking to the corner columns where they connect to their footings. Figures 31 and 32 show significant damage to the stucco and Figure 32 appears to show cracking in the column where it connects to the footing. Figure 33 shows limited damage to the stucco where the interior of the north door frame connects to its footing.

Figure 31. Stucco damage at the southeast column to footing connection, 50% test.

Figure 32. Stucco damage at the northeast column to footing connection, 50% test.

Figure 33. Limited damage to stucco at interior of North door frame column to footing connection, 50% test.

6.7 Seismic test at 100%

Appendix F includes measured and average accelerations for the bottom and top of the building, inertial loads, base shears, displacements, and average building deformations in both horizontal axes for the 100% seismic test. The loading and deformation plots show that the building further softens and cycles at lower frequencies than the undamaged building natural frequencies.

Figure 34 plots the base shear versus the average deformation in the X-axis. The deformations in the X-axis increased to 1.8 in. Figure 34 shows that the estimated X-axis building stiffness decreased to 28,000 lbs/in., a 77% decrease, indicating very large softening and damage relative to the 50% test. Figure 34 plots six segments of base shear versus deformation in time showing the building became much more flexible after 9.5 seconds and again after 18.6 seconds. Figure 35 plots the base shear versus deformation for the short direction of the building (Y-axis). This test shows the stiffness decreased to 16,000 lbs/in., a 53% decrease, indicating significant softening and damage. The segments plotted in time do not indicate a dramatic decrease in stiffness as the test progressed.

Figure 34. Base shear versus deformation and X-axis stiffness, 100% seismic test.

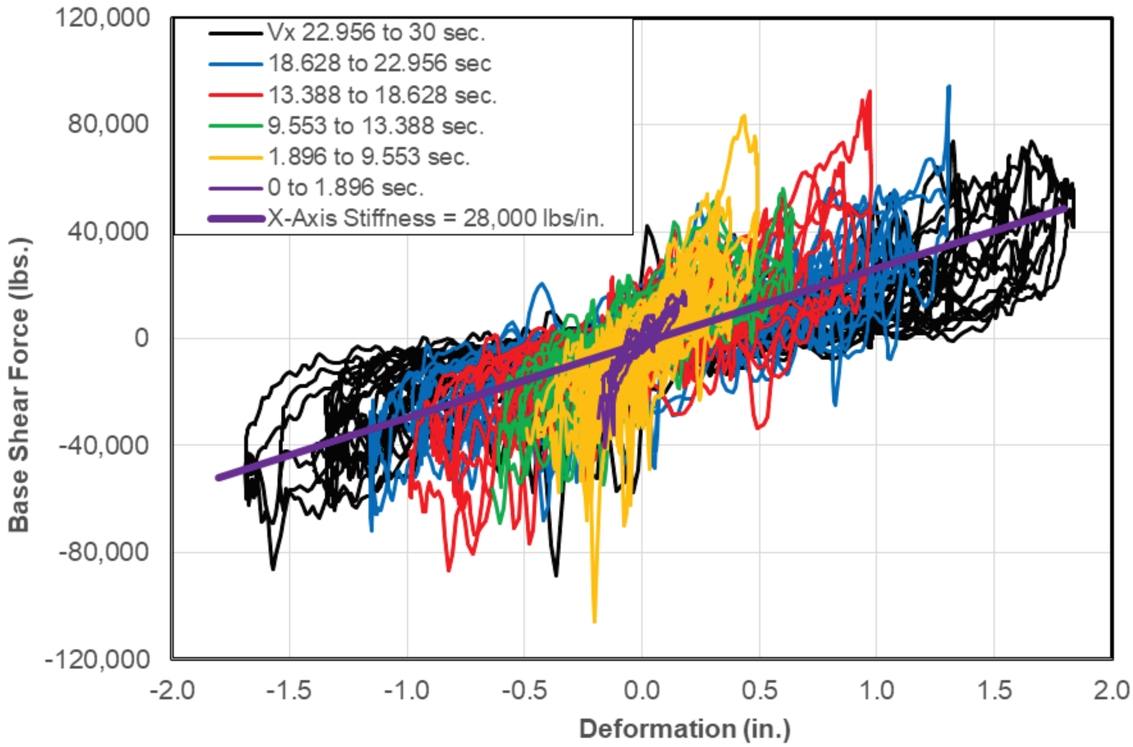
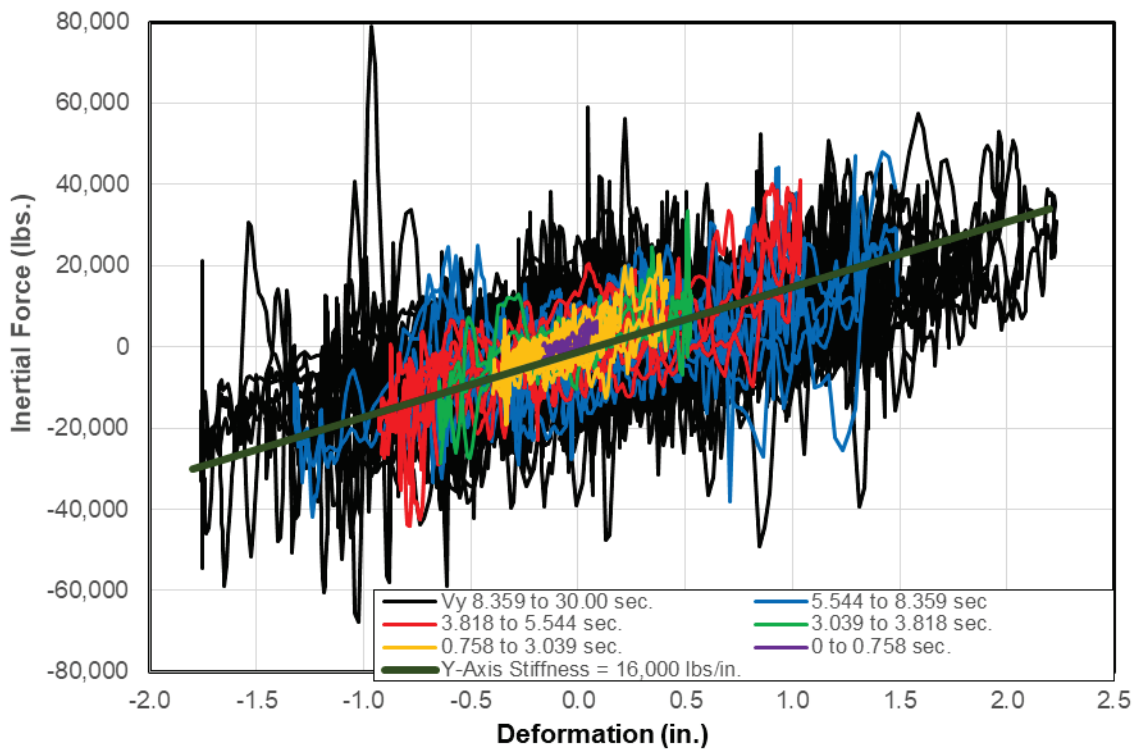


Figure 35. Base shear versus deformation and Y-axis stiffness, 100% seismic test.



In the 100% seismic test, the four columns at the corners of the building completely failed in shear just above their footings. The corners columns failed in shear in the short direction of the building, where the cracks opened with large offsets in the Y-axis of the building early in this test. The columns 4 ft. to the interior, and almost immediately after all the columns along the long east and west walls failed in shear in the long X-axis of the building. Before the corner columns failed, the interior columns would have been more heavily loaded in the Y-axis of the building, which is the strong or stiffer axis of the frames. Once the corner columns failed and offset, the rigid long walls transferred significant load in shear to these interior columns in the X-axis. This progression of failure and offsets was seen in video files of the test. At the end of this test, the concrete along the top of the footings was crushed into golf ball and smaller sized rubble with a three-dimensional chicken wire mesh holding the rubble together. Many of the wires were also fractured, as seen at the southeast corner in Figure 36, southwest corner in Figure 37, and west face in Figure 38.

Figure 36. Bottom of column at Southeast corner showing concrete rubble and partly fracture chicken mesh, 100% test.

Figure 37. Bottom column at southwest corner, 100% test.

Figure 38. Bottom column at the west face, 100% test.

Figures 39, 40, 41, and 42, show severe damage to the stucco and chicken wire mesh at the northeast column and northwest column.

Figure 39. Bottom of northeast column, 100% test.

Figure 40. Bottom of northeast column, after removing the building from the test fixture, 100% test.

Figure 41. Bottom of northwest column, 100% test.

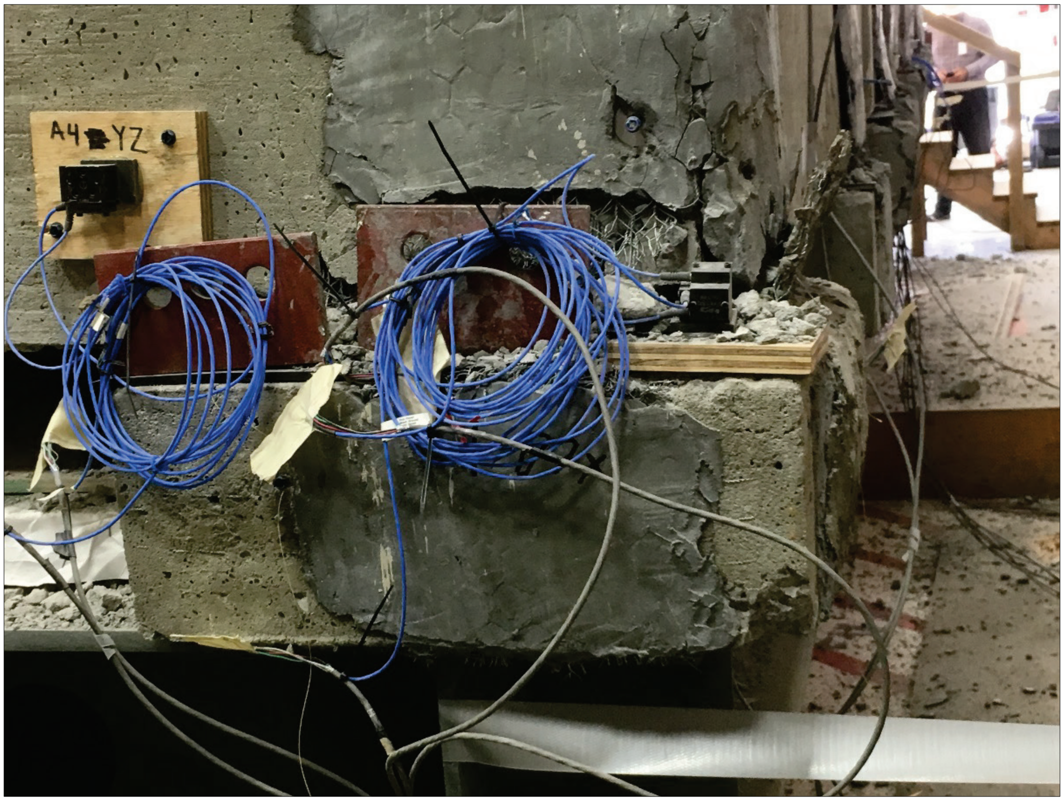


Figure 42. Bottom of northwest column, after removing the building from the test fixture, 100% test.

The small columns of the door frames, at the center of the north and south walls were badly damaged but did not completely fail. The braces especially at the south door frame in the short direction (Y-axis) partly failed at the connection to these columns and the short axis wall panel connections to the door frame columns also partly failed. Figures 43 and 44 show damage to the south wall door frame column and vertical crack in the wall panel near the southeast corner. Figure 45 shows an interior view of the north wall door frame column with a large vertical crack in the stucco and all panel.

Figure 43. East side of south short wall, where brace and wall panel connections partly failed at the bottom left corner of picture – note the vertical crack in the wall panel, 100% test.

Figure 44. Bottom of south wall door frame columns with large cracks, 100% test.

Figure 45. Interior view of North door frame column shows cracks in stucco and wall panel, 100% test.

7 Summary and Conclusions

7.1 Summary

This report presents the seismic testing of a full scale 16 ft. by 32 ft. TCB fabricated from indigenous materials. The report documents the details of the building and large test fixture that supported it and acted as the interface between the TCB and ERDC-CERL TESS. The report provides an AHA, documenting several steps taken to protect workers during construction, and the TESS from hazards associated with the failing and possible collapse of the TCB. The development of test motions is described, as are the sensors used to measure the response of the building.

The rationale for test steps is described, including SI tests used to measure the building's natural frequencies. Tests were conducted with synthetic seismic motions, with amplitudes equivalent to the highest seismic hazard in the United States, where the spectral acceleration, S_{DS} was set equal to 2.5 g. Four seismic tests were conducted with 10%, 25%, 50%, and 100% of these motions.

Appendixes C to F include plots of the measured accelerations at the footing level, bottom of wall panels and braces, and top of wall panels for each test. Measured displacement at the footings and top of wall panels were also plotted. The average accelerations at the bottom and top of wall panels were calculated from measurements, and these were multiplied by the mass of the building lumped at these two elevations. The results were estimates of the inertial loading at the top and bottom of the building. The sum of these inertial loads were estimates of the base shears for both the long and short axes of the building. Appendixes C to F include plots of the time histories of these accelerations and inertial loads and base shears for both horizontal axes of motion.

The displacements at the footings were subtracted from the displacements at the ceiling elevation to calculate building deformations. The deformations from the corners of the building were averaged to calculate time history records of horizontal building deformations in both the long and short axes of the building. Appendixes C to F include plots these accelerations, inertial loads, displacements and deformations for each seismic test level.

The main body of the report provides summary plots of hysteretic envelopes of the base shear versus average deformation for the building in both horizontal axes for each seismic test level. Damage observations are provided for each test level with photographs of damage.

The building experienced only minor cracking of the concrete during the 10% seismic test; greater cracking in the 25% test; and significant cracking in the stucco connecting the wall panels and columns to the footings in the 50% test. In the 100% test, the columns at the corners failed completely through their cross-section in shear in the short axis of the building, just above the footings. The building then moved significantly in the long axis, shearing through all the main columns along the east and west long walls, at the same failure plane just above the footings. At least a portion of the chicken wire mesh stayed intact, connecting the columns to their footings; the columns had relatively small permanent offset along this failure surface. The door frame column connections to the wall panels and braces partly failed, and the stucco and chicken mesh wire between the columns and footings were damaged, but the columns themselves remained largely intact. These columns were narrow in both the long and short axis of the building so they were more flexible and attracted less load from short axis motion than did the main building columns. A real building in this condition could resist the loads of an aftershock through the door frame columns and through the mesh and rubble along the base of each main column.

A critical observation is that the chicken wire mesh and stucco lap joints that connected the wall panels to each other, that connected the roof and ceiling panels to each other and to the walls, and that anchored the wall panels to the footings, provide major strengthening to this building.

The building sustained major damage during the 100% test, including the complete shearing of all the main columns. However, a portion of the chicken wire mesh was still intact, though badly distorted and stretched.

7.2 Conclusions

The building was not about to collapse after the 100% seismic test, and it achieved the life-safety objective of collapse prevention. Once the main columns had completely sheared through their cross-section, the building was essentially base isolated, with the remaining chicken wire mesh, and door frame columns preventing large permanent offsets of the main columns and preventing the columns from falling off their footings.

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Acronyms and Abbreviations

Term	Definition
AHA	Activity Hazard Analysis
ATLG	Table Longitudinal or X-Axis
ATLT	Table Lateral or Y-Axis
ATPITCH	Pitch Motion (Rotation about the Y-Axis)
ATV	Table Vertical or Z-Axis
CERL	Construction Engineering Research Laboratory
CMU	Concrete Masonry Unit
cu ft	cubic feet
ERDC	U.S. Army Engineer Research and Development Center
ERDC-CERL	Engineer Research and Development Center, Construction Engineering Research Laboratory
ft	feet
g	grams
Hz	Hertz
ICC-ES	International Code Council – Evaluation Services
in.	inches
ksi	kilopounds per square inch
lb	pounds
lbs/cu ft	pounds per cubic feet
lbs/in.	pounds per inch
RRS	Required Response Spectrum (RSS)
SBIR	Small Business Innovative Research
SF	Standard Form
SI	System Identification
TCB	Troop Constructible Building
TESS	Triaxial Earthquake and Shock Simulator
TR	Technical Report
z/h	Ratio of the elevation that equipment can be installed (z) over the total height of the building (h) (per AC156 [ICC-ES 2010])

Appendix A: Activity Hazard Analysis

An AHA was conducted to define hazard and methods to mitigate them for each activity used to construct, test and demolish the TCB as shown in the following pages.

Date Prepared:	1 Oct 2018 Revision

Project: Seismic Testing of Indigenous Material Troop Constructible Buildings
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Reviewed By: Darrell Wright , ERDC-CERL Safety Officer
Parviz Soroushian, Metna Principal Investigator
Vicki VanBlaricum, ERDC-CERL, CFM Branch Chief

Recommended Protective Clothing and Equipment:

Steel toe boots, eye protection (e.g., ear plugs & safety glasses), gloves and hard hats for everyone entering the shake table high bay area. Safety harnesses, self-retracting lanyard, for those working above 4 ft. off the floor.

E = Extremely High Risk
H = High Risk
M = Moderate Risk
L = Low Risk

Severity	Probability: Frequent	Likely	Occasional	Seldom	Unlikely
Catastrophic	E	E	H	H	M
Critical	E	H	H	M	L
Marginal	H	M	M	L	L
Negligible	M	L	L	L	L

Job Steps	Hazards	Risk	Actions to Eliminate or Minimize Hazards
1. A large test fixture, consisting of a center section and two narrow side sections will be moved into the TESS high bay, assembled and installed on the shake table, disassembled and moved outside for storage. Six high capacity casters are installed on the center section of the fixture and this is rolled inside using a telehandler and forklift. The side fixtures are brought inside with the telehandler alone. The fixture sections are moved inside the high bay with the overhead crane, telehandler and forklift.	a. Casters are installed at designed locations near the base of the center fixture. The casters weigh 140 lbs., and they easily rotate in three directions, causing potential for crushed fingers or toes. Personnel could be crushed under the fixture when the fixture is lifted with a jack and load is transferred between blocking, jack and casters, on the sloped surface.	**M**	a. Two ERDC-CERL staff move and install the casters - one person can do this alone using the proper technique, but two people reduces the risk of crushing a finger. They work in as way that accounts for sudden rotation of the casters. They bock-up the fixture (with telehandler) in a way that prevents it from rolling when transferring the load from the jack to the casters.
	b. Dropping a fixture on toes of workers or crushing workers between the fixture and another object.	**M**	b. A telehandler was rented along with a senior operator of this machine. Two nylon straps and shackles were used to move the side fixtures outside and carry them inside, where they were moved with an overhead crane. ERDC-CERL personnel worked closely with the telehandler operator to ensure they were never in a position where they could be crushed by a swinging side fixture or crushed if it were to somehow fall. The telehandler outriggers were extended when the boom was extended and they were extended and slightly above the pavement when moving. This ensured the telehandler could not tip if there was a sudden shift in the load. Trained and designated ERDC-CERL crane operators will move the fixtures inside the high bay. Appropriate rigging and hand signals will be used.
	c. A come-along, pry bars and heavy wrenches are used to assemble the fixture. Personnel must be careful not to hit their head on the fixture when crawling under the side fixtures, or crush fingers.	**M**	c. Trained personnel will operate the come-along, pry bars, wrenches and overhead crane. They will wear hard hats and may wear gloves.
	d. Workers will use large socket wrenches, with cheater bar pipes and large crescent wrenches to bolt the fixture to the shake table and bolt the fixture sections to each other. Fingers could be crushed or backs injured when pulling on the wrenches.	**L**	d. Workers will wear a hard hat, steel toe boots and will pull or push on the wrenches using a proper symmetric technique, so they do not twist or injure their back.
	e. Workers could be crushed between the fixture and another object or fall into the pit area around the shake table.	**M**	e. No personnel will be allowed inside the shake table high bay while the fixtures are being moved except those participating in the move. Personnel will be trained on the use of the forklift and overhead crane. They will communicate with each other to safely move the fixture into position above the shake table. They will cover the gaps between the shake table and pit area when bolting the fixture to the shake table.

2. A wood deck will be constructed on top of the assembled test fixture. This will provide a flat surface for workers to walk on during the B-hut construction and will protect the shake table hydraulics sections of the B-hut break loose and fall or the entire building collapse. House wrap will be installed that covers this deck and encloses the sides of the fixture. This is intended to provide almost an air tight seal, which protects the hydraulics from dust created during construction drilling and grinding operations or testing.	a. Workers could fall from the test fixture or deck while it is being constructed or could hit their heads while crawling under the side fixtures.	**M**	a. Staff working on the fixture or deck will wear harnesses and be anchored to a swing arm. They will wear a hard hat when crawling under the fixture sides.
	b, Workers could be injured when sawing lumber, installing screws or cutting house-wrap.	**M**	b. Workers will be trained in the safe proper uses of power tools.
3. The indigenous materials B-hut frames and panels will be unloaded from delivery trailers and the building will be constructed on the fixture.	a. The building frames and panels will be unloaded from delivery trailers using the overhead crane and a forklift. These materials are weak and could fail under their self weight and sections could fall injuring workers. Workers or tools could fall into the pit areas around the shake table surface, or through openings in the fixture.	**M**	a. An I-shaped spreader frame, with flat nylon straps looped under the top member of the frames will be used to lift the frames. 2x6 boards will be placed between the top member and straps to distribute the load over a large surface of the frames. The spreader frame allows the straps to be oriented almost vertically so that the frames are well supported while lifting. Workers will never walk below lifted frames and may use tag lines to guide the frames from a distance a way or they may guide the columns or footing with their hands. The panels are delivered on wood pallets and the side panels that are wider than 5 ft., will be lifted with tine-extensions on the forklift, so that they are well supported. The roof and celling panels will be lifted by hand and placed on one end of the I-shaped spreader frame. This frame will be lifted with the overhead crane at an offset position, so that the panels can be lifted level, where the end supporting the ceiling panels can be guided below the building frames. In this position, they will be screwed to the frames from below. Ropes will be attached to the same ends of the frame, so that when the unloaded frame is lowered (or raised after placing roof panels), the frame can be held level by workers. The wood deck will provide a smooth flat surface for workers to walk on, and these workers will wear harnesses and be anchored to the swing arms so that they do not step backward off the edge of the deck. The workers will also be working off ladders that rest on the deck, and they will also wear harnesses. Other workers will stand on roll able stairs that rest on the high bay floor along the outside of the building and fixture.
	b. Workers bolting the footings of the B-hut to the fixture along the inside of the footing could hit their heads on the fixture or could fall through the house wrap into the pit area along the East and West sides of the shake table.	**L**	b. Workers will wear a hard hat and wood covers will be installed over the gaps along the east and west sides of the shake table.
	d. Workers constructing the B-hut walls and roof will be at elevated locations outside the walls or above the roof, and could fall.	**M**	d. People working at elevated locations will wear a harness and be anchored to one of two swing arms that can be swung over the area of construction. When two people are anchored to the same swing arm, they will ensure the brake is installed.

4. Flat panels of the indigenous concrete panels will be cut with a dry skill saw at ground level. The footings of frames will be drilled for bolt hole anchorage to the fixture and some panels and frame are drilled for pilot holes for installing screws needed to fasten the panels to frames.	a. Significant dust is generated when cutting the sheets. Dust is also created when drilling. The dust is hazardous to workers in the area and to electronics and hydraulics of other machines north of the shake table building.	**M**	a. The panels will be move to the south door and both the north and south door opened, for ventilation. Shop vacuum will be used to suck up as much dust as possible near both sawing and drilling operations and workers will wear face masks.
5. Personnel will install accelerometers and string pot sensor and remove them after testing.	a. Some of the sensors are installed at high elevations and the person installing them could fall.	**M**	a. Personnel will work off rolling stair ladders, and will use the harness and swing arms if needing to use both arms while on a ladder.
6. Install camcorders and operate them during the seismic tests.	a. One camcorder will be installed on a bracket that is clamped to a column of the TESS building, approximately 15 ft. above the floor - the person installing or removing this camcorder could fall from the ladder.	**L**	a. The person installing and removing the camcorder will wear a harness, which is anchored to the east swing arm.
7. Conduct shake table tests on the B-hut, starting with low level model measurement and sine-sweep motions, followed by gradually increasing amplitude seismic tests.	a. The table hydraulics operate at 3,000 psi and if a hose or hydraulic line were to fail, someone could be injured.	**L**	a. All hydraulic lines are below the shake table or in the pump room, and no unauthorized person will be allowed in the pump room , and nobody will be allowed under the table when it is under high pressure.
	b. The shake table, fixture and B-hut could accidentally suddenly move when under high pressure.	**L**	b. Personnel must not touch the shake table, test fixture or B-hut when the table is under high pressure. Because of the weight of the fixture, the table must be under high pressure to even come up out of park. Therefore low level cycling normally done under low pressure to warm up the oil will be done at high pressure.
	c. The indigenous materials B-hut could fail in a brittle manner, most likely locally, where small portions could fall. There remains a small chance where a local failure progresses to a major global collapse of all or most of the building along the base of the columns, or even along the tops of some columns.	**H**	c. A heavy sandwich panel wood deck (two layers of 3/4" plywood with 2x4 studs between, screwed together) has been constructed, consisting two layers of 3/4" plywood with 2x4 studs laid flat between the plywood, and these have been connected with screws spaced 8 in. o.c.. This deck along with the heavy steel test fixture and lateral actuator wood covers will protect the shake table hydraulics from either local or global collapse of the building. Local or global collapse could result in concrete portions falling onto the concrete floor surrounding the shake table. This weak falling concrete would crush and would not damage the much stronger concrete reaction mass. There remains a slight possibility that the entire B-hut could rack over and collapse into the control room. This will be prevented by suspending a larger rectangular spreader frame from the overhead crane and having four flat nylon straps at each corner looped around the corners of two fames of the B-hut. Two additional straps would loop around either one or three frames near the center of the east and west face of the B-hut. These straps will be slack, so as not to interfere with the tests, but Mr. Wilcoski will be ready to raise up the frame to tighten the straps and support the building.
	d. Same as above, with local or global collapse of the B-hut, but resulting in a hazard of hitting or crushing personnel standing too close to the building.	**H**	d. Observers will be briefed on hazards prior to testing, they will wear hard hats during tests, and will stand no closer than 15 ft. from the building during low level model measurement and sine-sweep tests and 25 ft. for seismic tests. Mr. Wilcoski will assure that all observers stay far enough away from the building. Observers will stand in designated areas of the shake table high bay. They should not walk too closely or in front of camcorders. Mr. Wilcoski and Carlson will ensure that visitors that have not been briefed on the hazards or do not have a hard hat stay out of the shake table high bay.

8. Personnel will inspect the building following each seismic test. This inspection includes taking pictures and climbing around to find optimal locations for taking photographs.	a. The building may be in a damaged state when it is being inspected. Early failures should be ductile, such that it is stable, but great caution will be used when entering the building to ensure that loose small sections do not fall. Global collapse during these inspections would not be possible because of the presence of the overhead frame and straps. Mr. Wilcoski will decide where people can walk when inspecting.	**M**	a. Mr. Wilcoski understands ductile versus potential brittle modes of failure, and these can be seen through visual inspection. No one will enter the building until after Mr. Wilcoski inspects it and gives his approval. Mr. Wilcoski will give approval to go near, put a ladder against or go inside the building.
	b. Personnel may climb to elevated locations to take pictures. They may use a ladder or may climb on other surfaces that could create a fall hazard.	**L**	b. Use ladders carefully and use a harness attached to a swing arm if climbing at elevated locations.
9. After testing the building will be unbolted at the footing anchorage to the test fixture, moved to the Southeast corner of the highbay, the spreader frame will be lowered and placed on the building roof, the crane scale installed, the building weighed, the building moved partly outside where it will be demolished by saws and impact tools, and sections loaded into a dumpster. The building will be moved to the floor, weighed and partly moved outside before demolishing. This is being done for worker safely and to minimize dust in the building.	a. The only significant damage to the building is complete shear cracking along the column to footing joints. The footings remained connected to the columns only by chicken wire mesh - the check mesh could fail and the footing fall on workers.	**M**	a. Tether ropes will be attached to two corners of the building frame, so that workers can guide the building, to prevent twisting while it is being lifted with the overhead crane. The workers can stand 15 to 20 ft. away from the building, to ensure building footings or entire portions of the building (see below) do not fall on them if the building beams were to fail in shear or components buckle.
	b. The building frame beams are quite weak, and though undamaged during the seismic tests, they could fail in shear or wall panels and braces could buckle, when lifting the building.	**H**	b. The complete building will be lifted with the overhead crane and spreader frame that was used to restrain the building. The building above the column to footing connections is intact. The building is conservatively estimated to weigh 28,000 lbs., which is well below the capacity of the overhead crane (40,000 lbs.), spreader frame (40,000 lbs.), six flat straps and rigging (62,400 lbs.). Four straps are looped around the end frames and braces at the corners of the building and two straps around the ends of the center frame of the building. The braces and wall panels will effectively distribute the load from the adjoining frames. The center frame is being lifted at the beams only, so this is the weakest portion of the building being lifted. 2x8 boards with rubber pads will be installed between the straps and center beams to distribute loads over a greater portion of beams and connected elements. If the beams begin to fail in shear at these joints, they would only fail partly and loads redistributed to the corner frames through the wall panels. At the corners, the straps are looped through both the frames, top chord of the braces and interior and exterior wall panels. Loads will redistribute to other portions through these elements. The corner diagonal straps will be on a significant angle, which will cause large horizontal forces pulling the exterior frames toward each other. The roof, celling and long axis wall panels, plus the braces in the first bay in the long direction of the building will all resist these loads, and should prevent buckling of the panels or braces. Still it does remain possible for the building frames to fail in shear or the panels and braces to buckle. If this were to happen when first lifted, the building would be set down on the wood deck on the fixture - and these components are designed to absorb impulsive loads. If such failure occur over the floor of the high bay the entire building would safely land on the high bay floor.
	c. The are hazards related to demolishing the building using saws and impact tools such as sledge hammers or jack hammers. Workers cut themselves or components could fall on workers as they are being demolished.	**M**	c. Care will be used and workers will be trained to properly use saws and impact tools. Components may need to be braced or tied off while they are being demolished. A fork lift will be used to load demolished components into a dumpster.

Appendix B: Transfer Function Plots from TCB Sine-Sweep Tests

Three sine-sweep SI tests were conducted to measure the natural frequencies and associated modes of vibration of the TCB in the X-, Y-, and Z-axis of the building. Figure B-1 shows a plot of the transfer functions for the response motions in the X-axis for the sine-sweep test in the same axis. Figures B-2 and B-3 provide similar plots for the Y- and Z-axis.

Figure B-1. Transfer function plots for X-axis sine-sweep test.

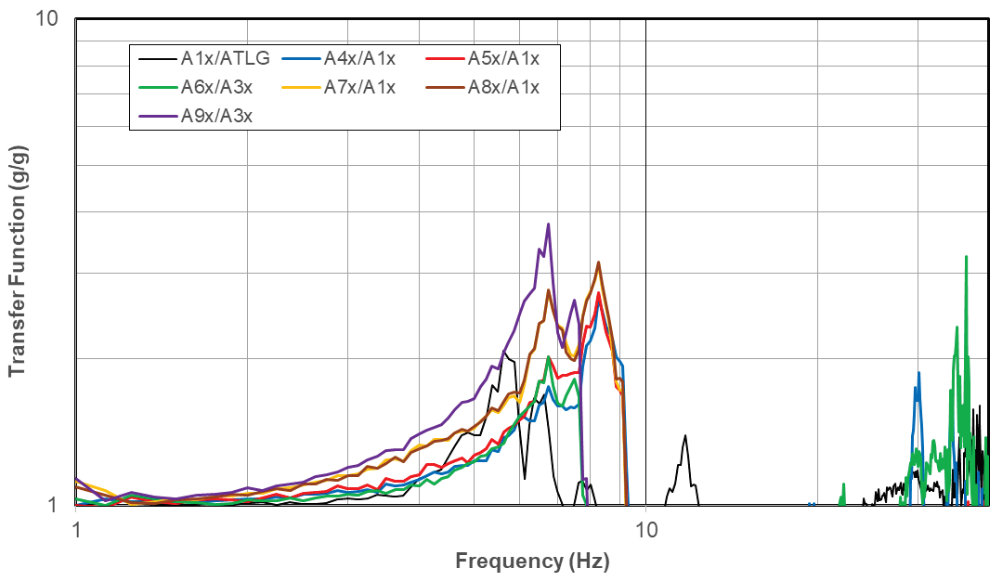


Figure B-2. Transfer function plots for Y-axis sine-sweep test.

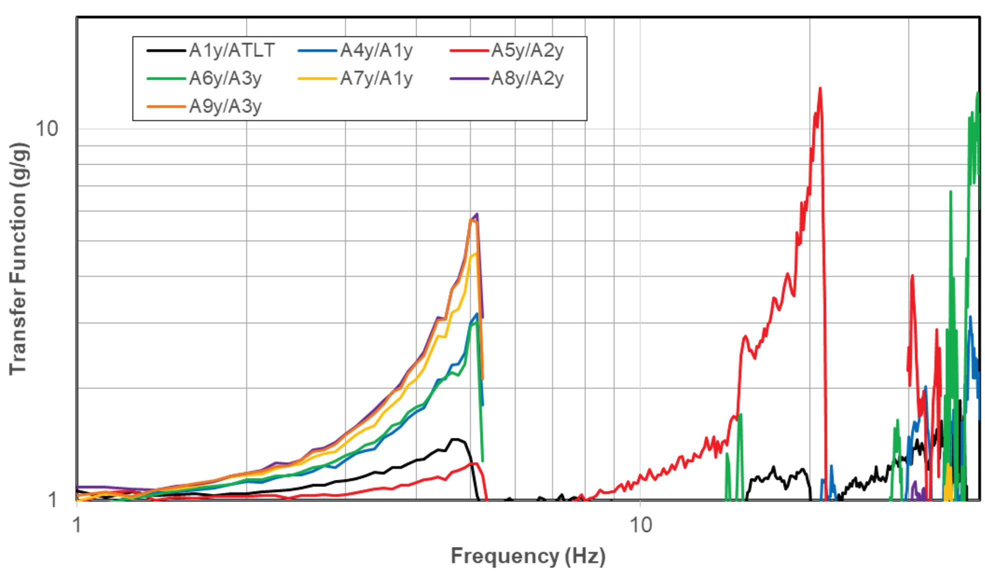
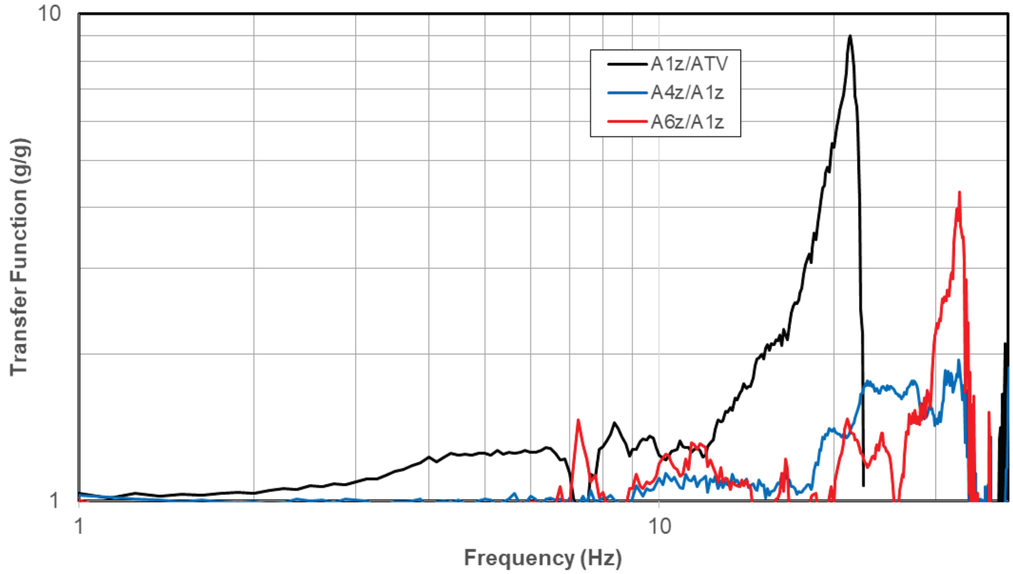


Figure B-3. Transfer function plots for Z-axis sine-sweep test.



Appendix C: Data Plots for 10% Seismic Tests

Figures C-1 through C-3 show plots of all the accelerations measured in the X-, Y-, and Z-axes of the building for the 10% seismic test. Figures C-4 and C-5 show plots of the acceleration measured in the TESS (Long), at the bottom and top of the building, along with the average acceleration at the two levels (Ax_{Bot} and $Ax_{Ceiling}$) in the X-axis. Figures C-6 and C-7 show plots of similar data for the motion in the Y-axis. Figures C-8 and C-9 show plots of the inertial loads (Fx_{Bot} and $Fx_{Ceiling}$) applied to the bottom and top of the building in the X-axis, where these are the average accelerations at those levels multiplied by an approximation of the building mass lumped at those elevations. Figures C-10 and C-11 show plots of the inertial loads (Fy_{Bot} and $Fy_{Ceiling}$) applied to the same elevation of the building in the Y-axis. Figures C-8 through C-11 also show plots of the base shear for both the X-axis (Vx) and Y-axis (Vy). The number of cycles of $Ax_{Ceiling}$ and $Ay_{Ceiling}$ can be counted in Figures C-5 and C-7 to show that the building acceleration and the applied loads (Figures C-9 and C-11) are oscillating at about 7.8 Hz in the X-axis and 5.4 Hz in the Y-axis, showing the building is oscillating at its natural frequencies of 7.7 Hz and 5.1 Hz (Table 5).

The more significant loading and deformation of the building was in the short Y-axis of the building. Researchers “zoomed in on” (expanded) a short time segment of these records between 13 and 15 seconds where short axis deformation was large, to more clearly see cycles of loading and building deformation. Researchers also zoomed in on the same two second segment between 13 and 15 seconds for each seismic test to consistently compare each test level for loading and building deformation.

Figures C-12 and C-13 show plots of the measured displacements in the X-axis and Y-axis. Figures C-14 through C-17 plot the calculated deformations in both the X-axis and Y-axis, plus the average deformation in both directions (Dx_{Avg} and Dy_{Avg}). Figures C-15 and C-17 show that the average deformation oscillated at about 7.7 and 5.8 Hz, which agrees well with the measured natural frequencies of the building. In later tests, as the building is damaged, these oscillations are expected to lengthen in time or decrease in frequency, as the building is damaged and slowly becomes more flexible.

Figure C-1. Accelerations recorded in the X-axis during the 10% seismic test.

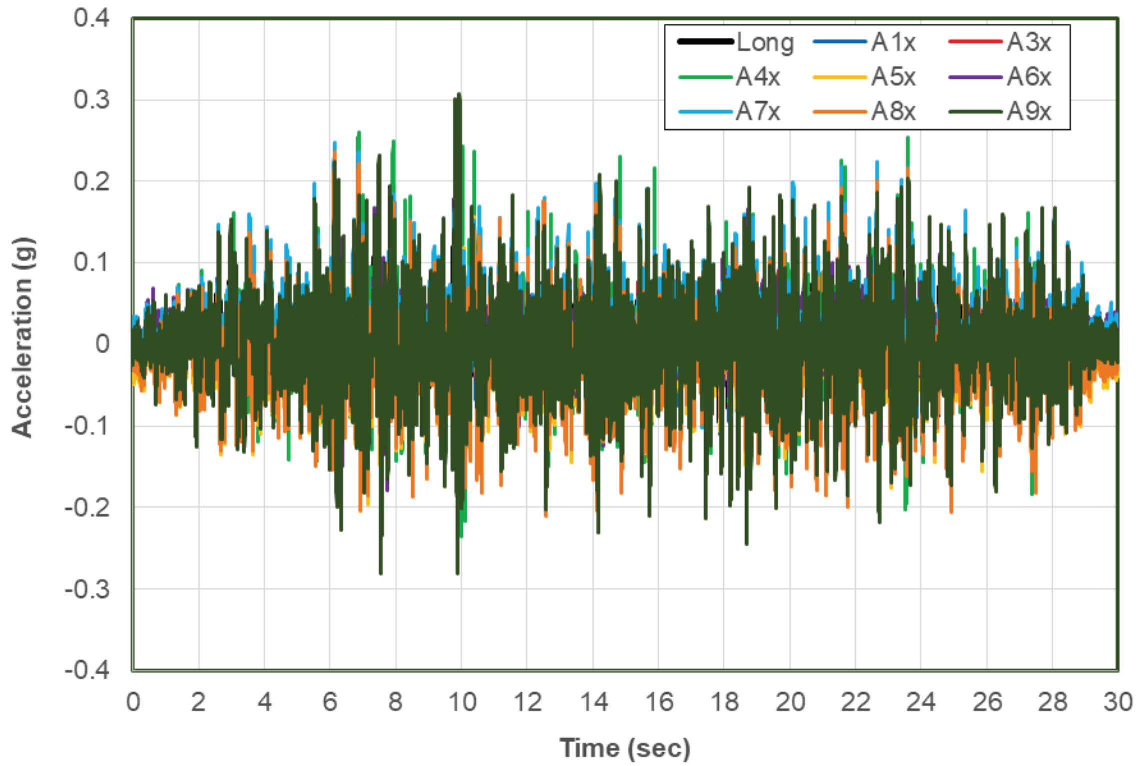


Figure C-2. Accelerations recorded in the Y-axis during the 10% seismic test.

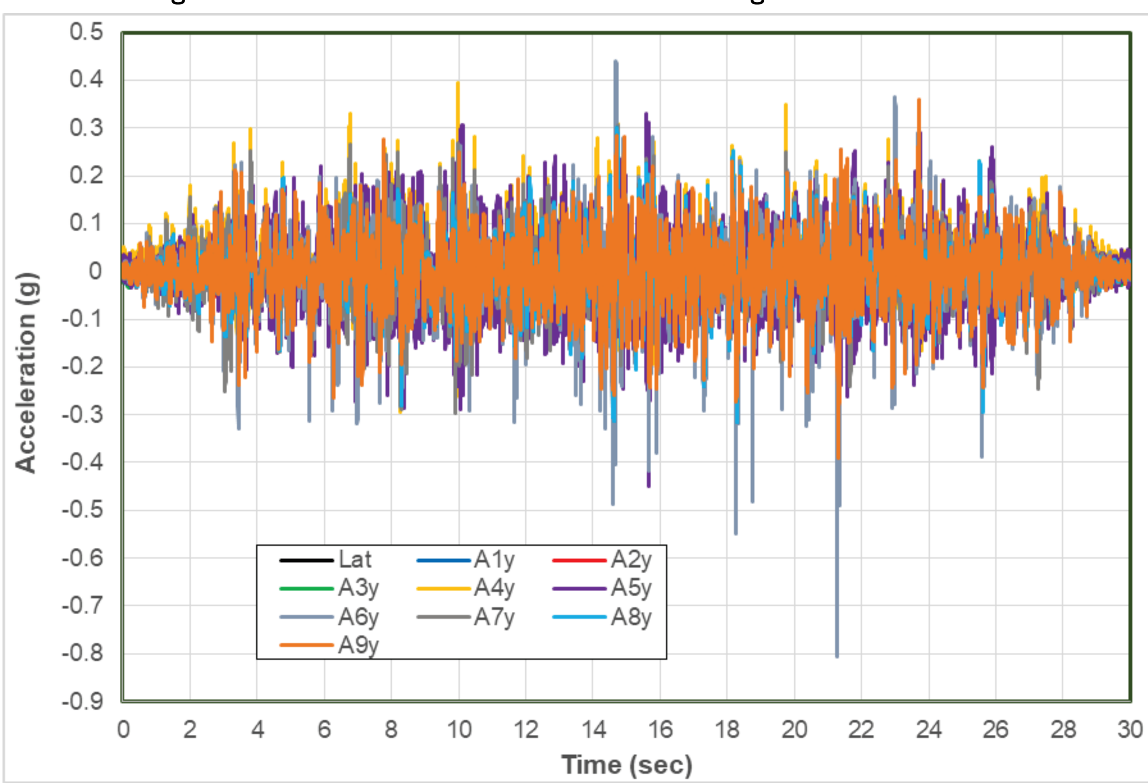


Figure C-3. Accelerations recorded in the Z-axis during the 10% seismic test.

Figure C-4. Accelerations at bottom of walls in X-axis, 10% seismic test, 13 to 15 sec.

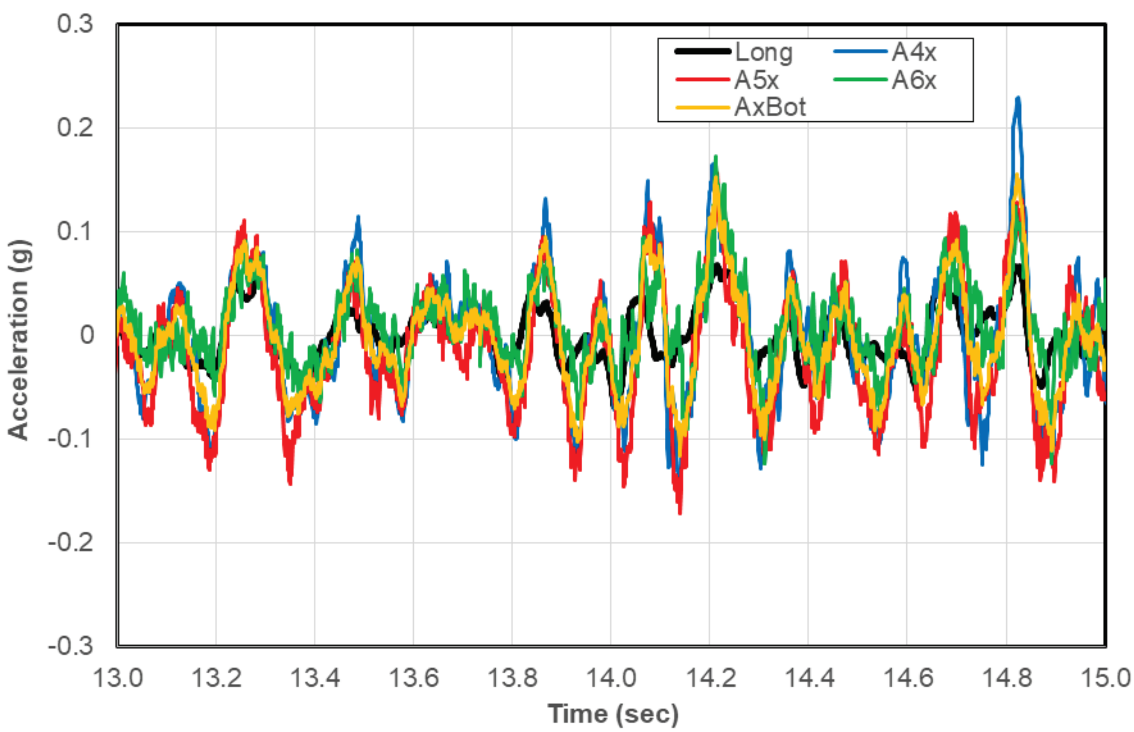


Figure C-5. Accelerations at the ceiling in the X-axis, 10% seismic test.

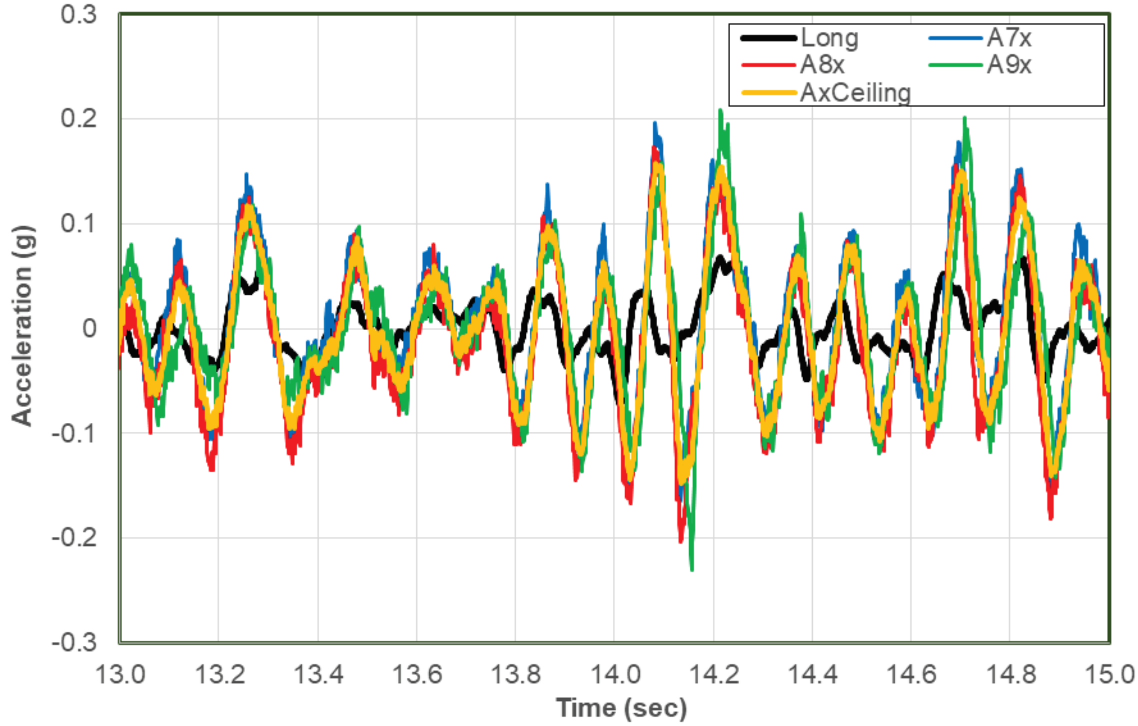


Figure C-6. Accelerations at the bottom of the walls in the Y-axis, 10% seismic test.

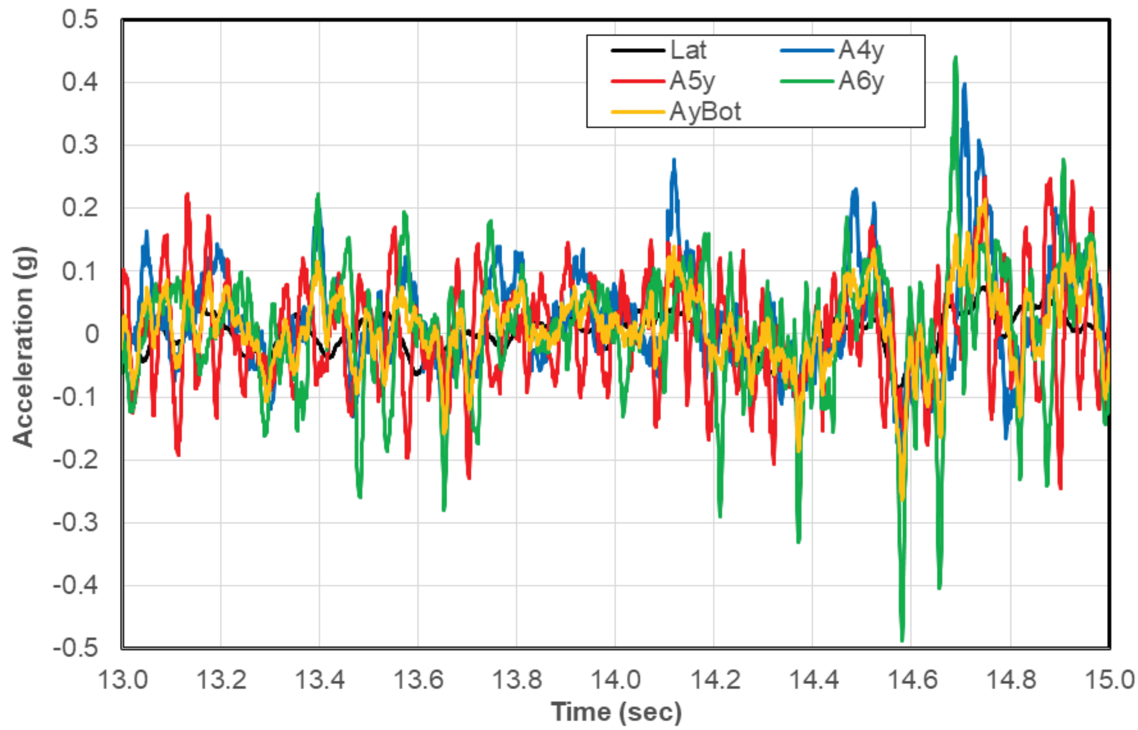


Figure C-7. Accelerations at the ceiling in the Y-axis, 10% seismic test.

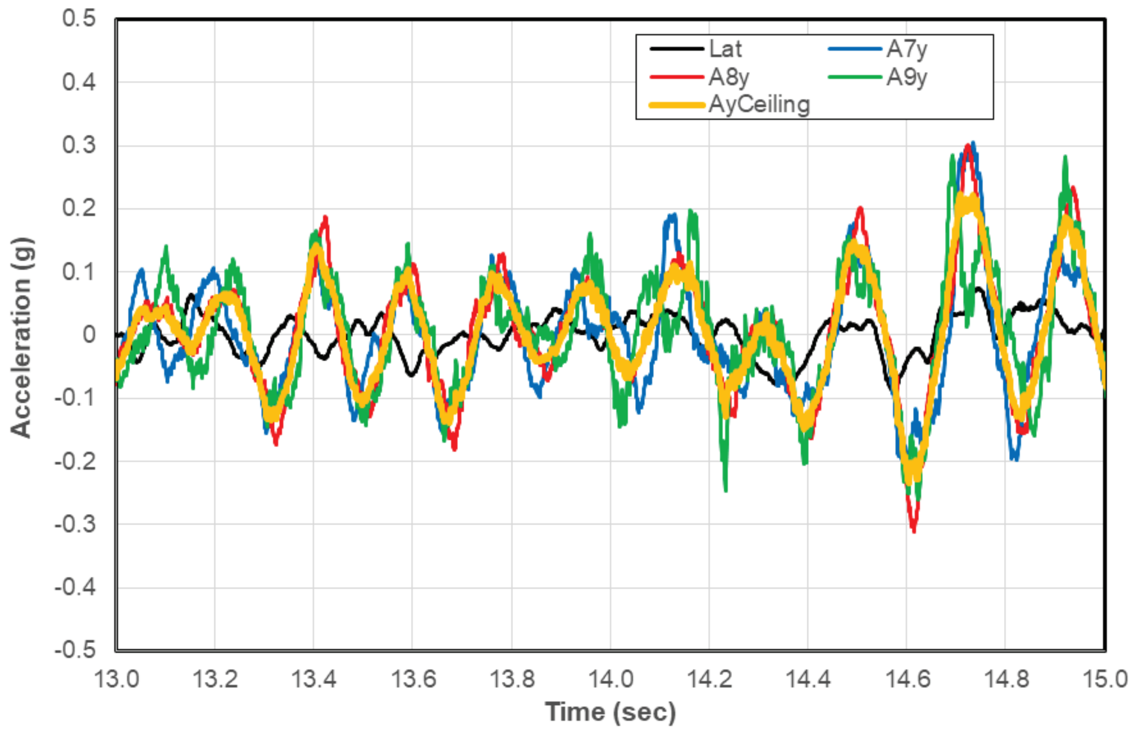


Figure C-8. Inertial forces for bottom, ceiling and base shear, X-axis, 10% seismic test.

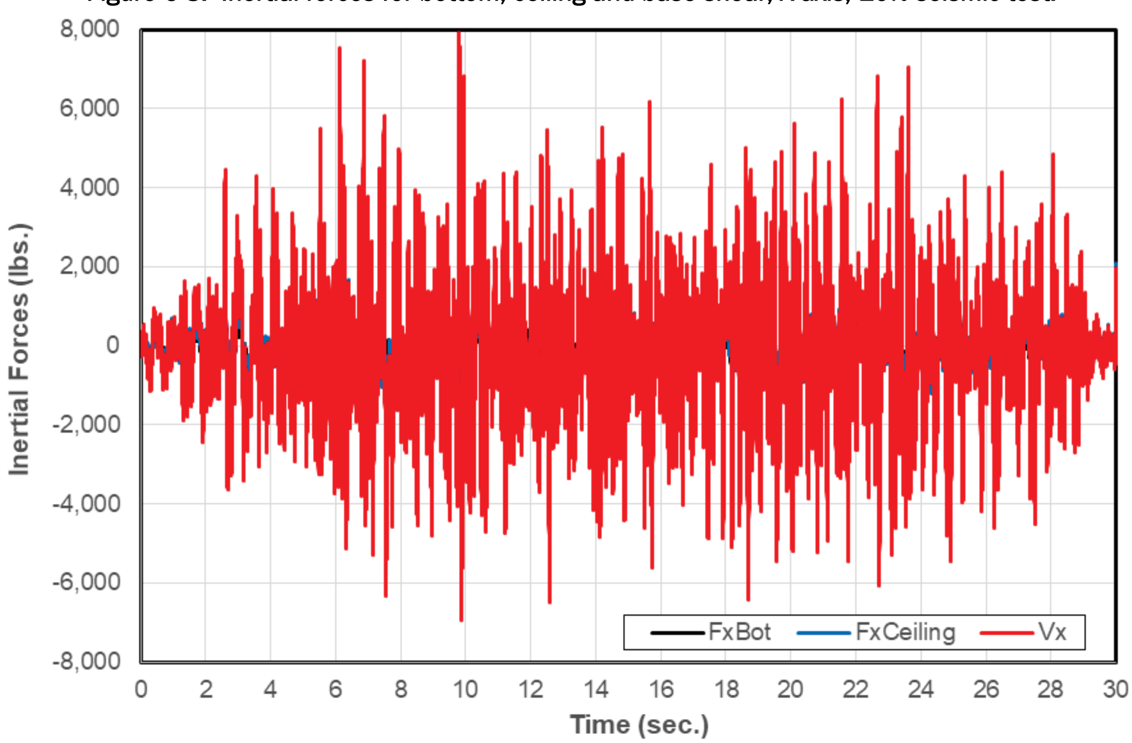


Figure C-9. Bottom & ceiling forces, & base shear, X-axis, 10% seismic test, 13 – 15 sec.

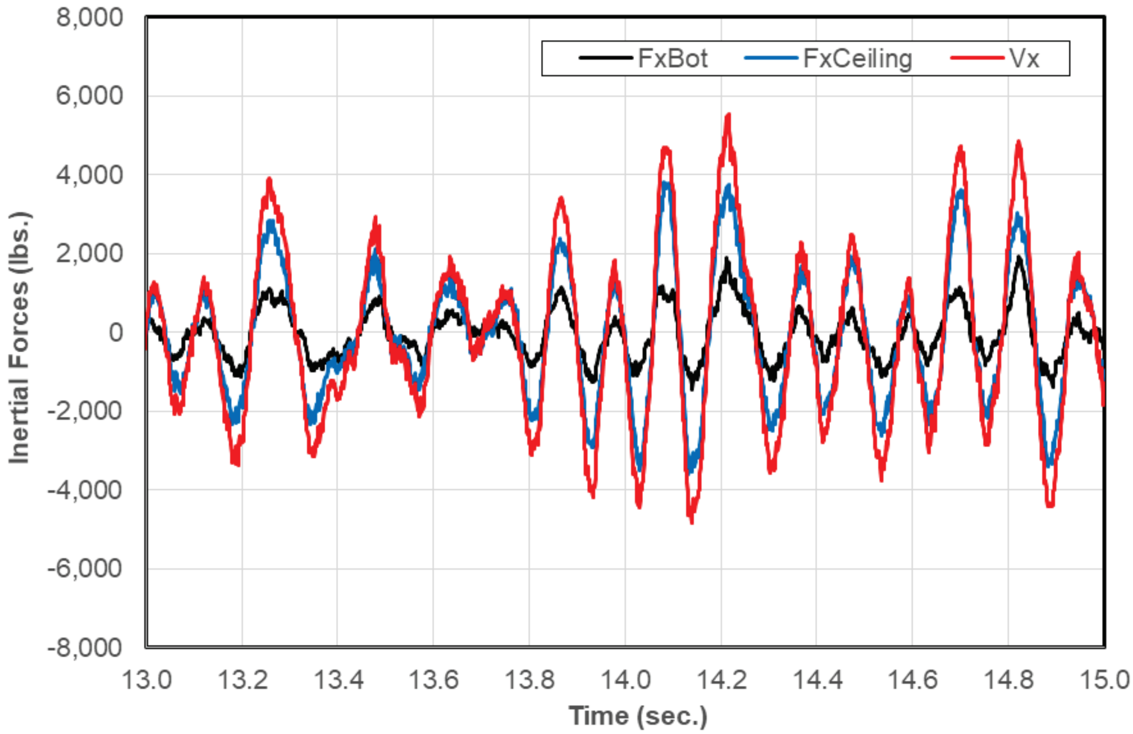


Figure C-10. Inertial forces for bottom, ceiling and base shear, Y-axis, 10% seismic test.

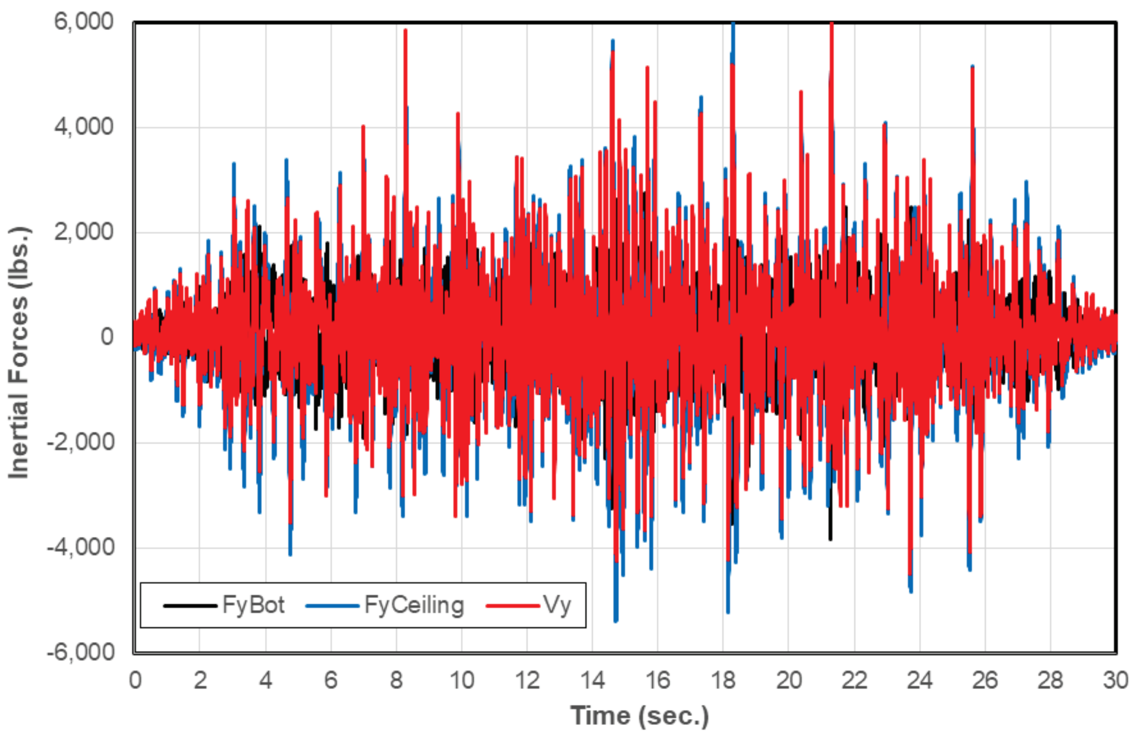


Figure C-11. Bottom & ceiling forces, & base shear, Y-axis, 10% seismic test, 13-15 sec.

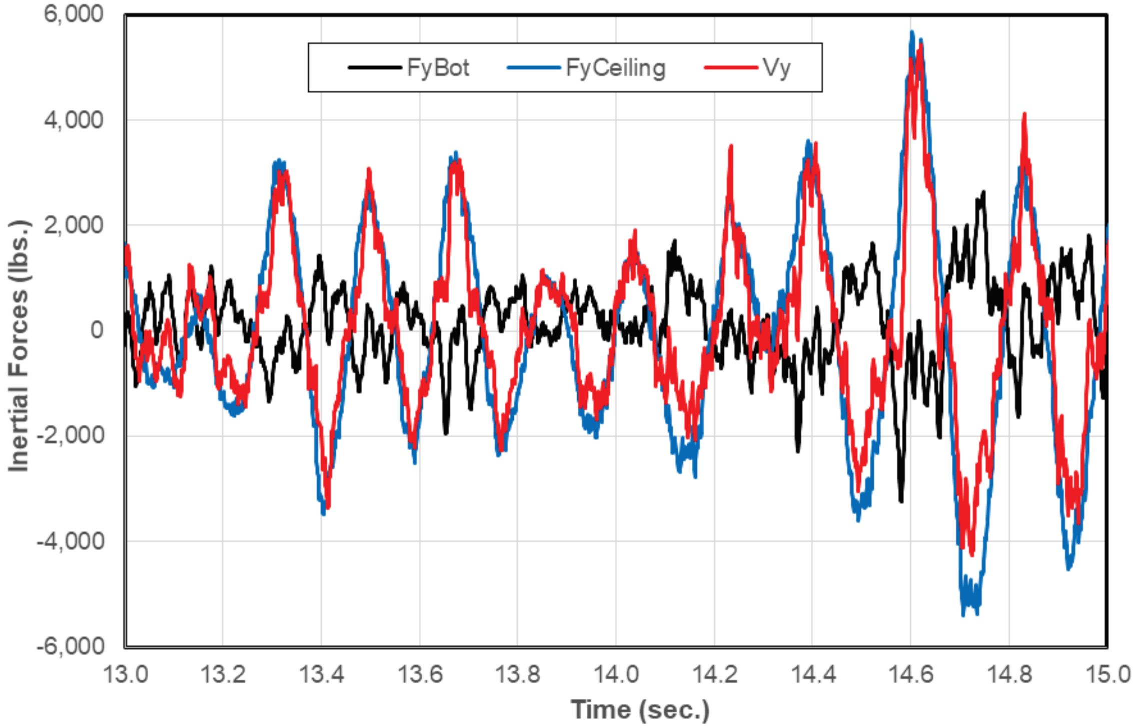


Figure C-12. Displacements recorded in the X-axis during the 10% seismic test.

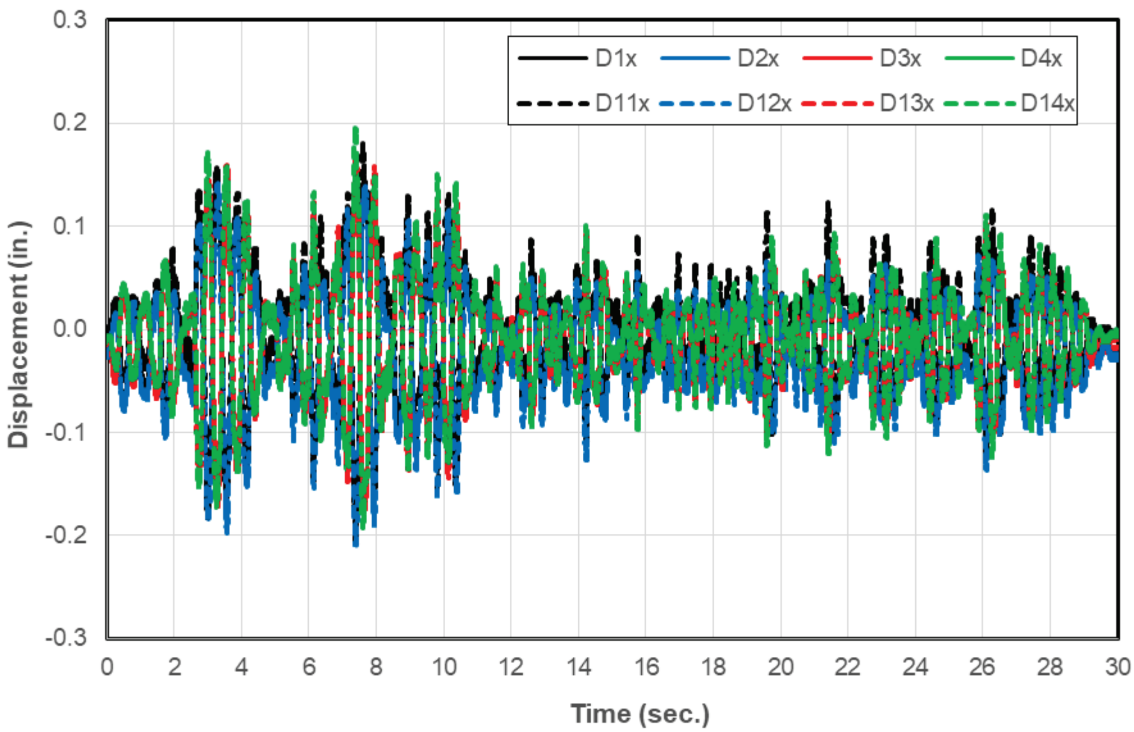


Figure C-13. Displacements recorded in the Y-axis during the 10% seismic test.

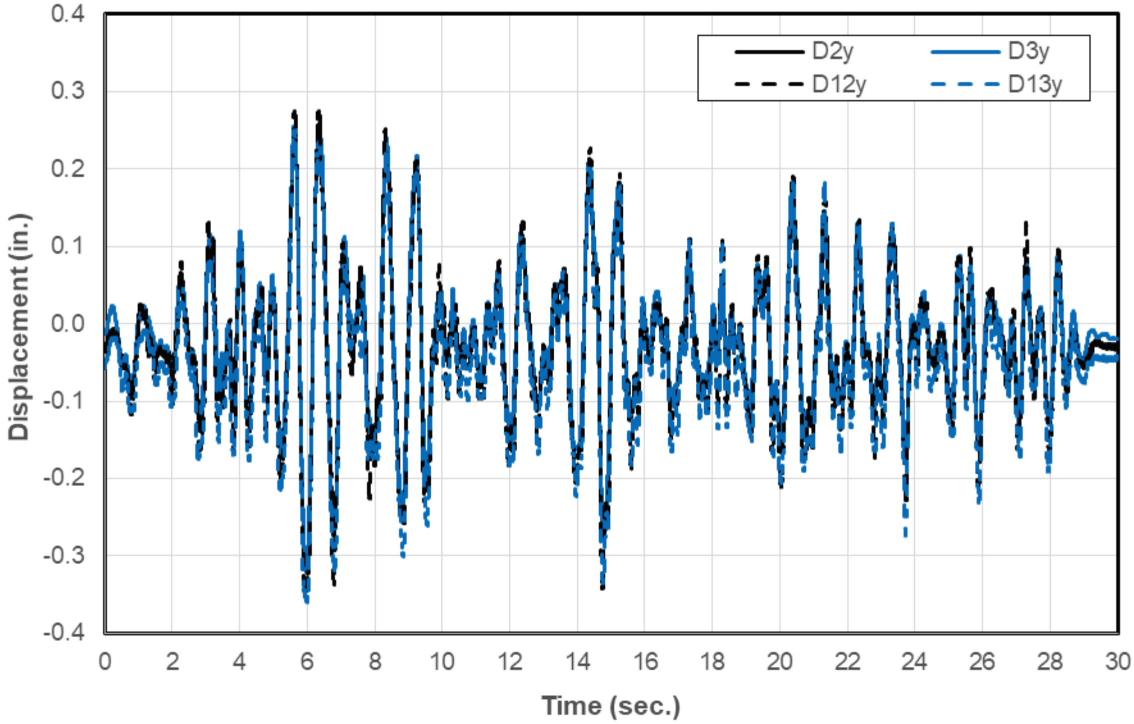


Figure C-14. Deformations in the X-axis during the 10% seismic test.

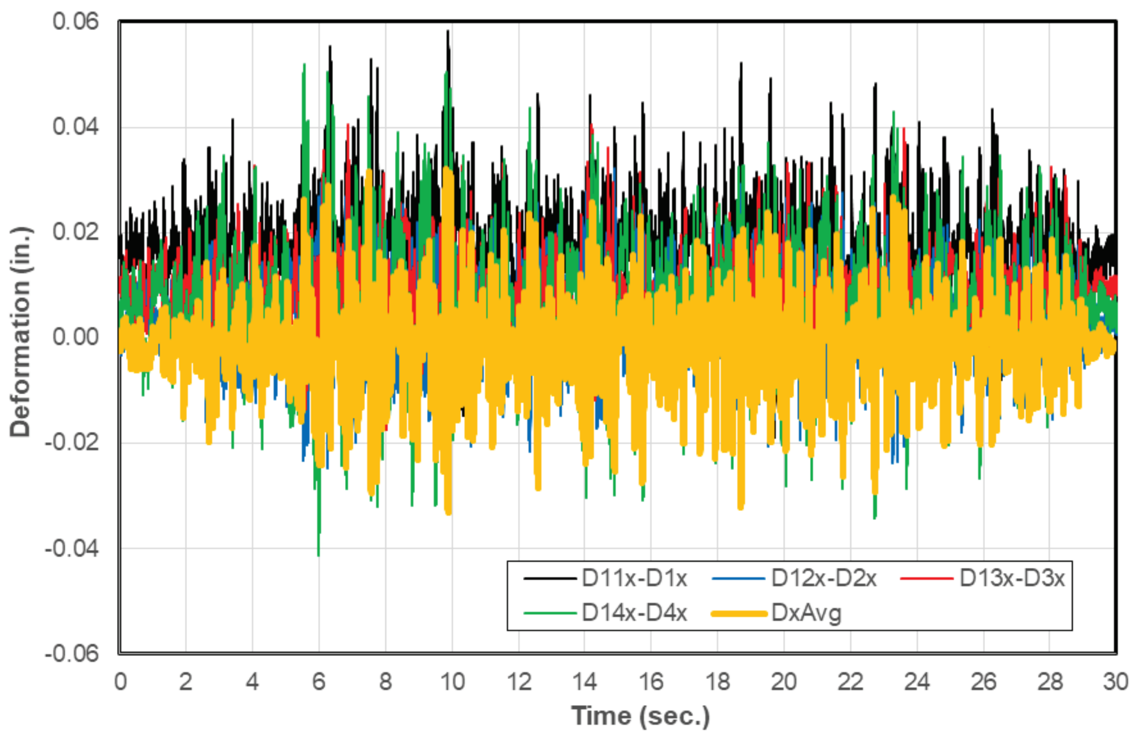


Figure C-15. Deformations in the X-axis during the 10% seismic test, 13 to 15 seconds.

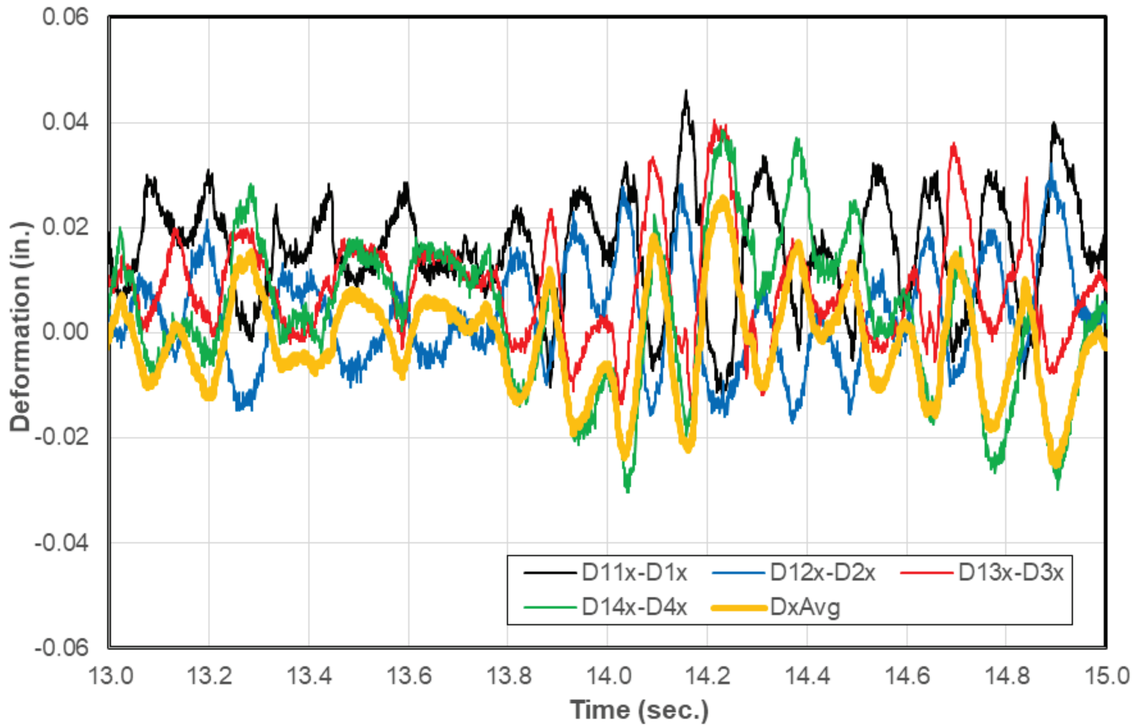


Figure C-16. Deformations in the Y-axis during the 10% seismic test.

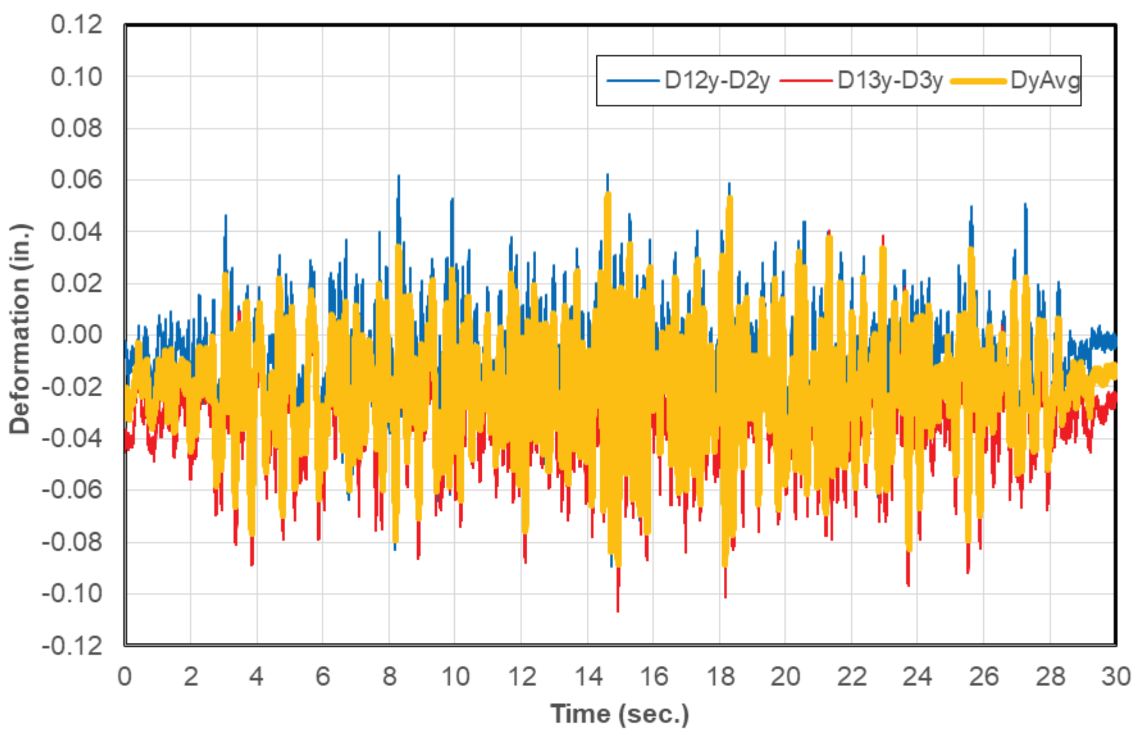
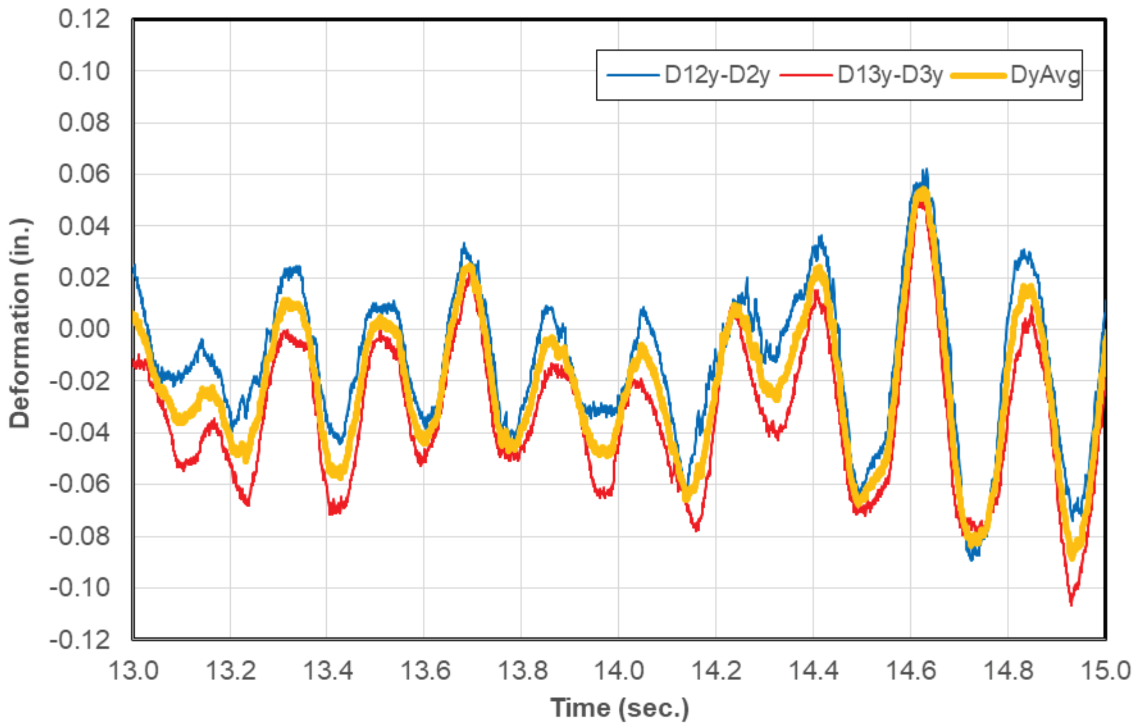


Figure C-17. Deformations in the Y-axis during the 10% seismic test, 13 to 15 seconds.



The base shears (Vx and Vy) are plotted with respect to average deformations (DxAvg and DyAvg) as hysteretic envelopes in the main body of this report (Figure 23 to 24). These plots are the best indicator on the behavior and condition of the building for each test level.

Appendix D: Data Plots for 25% Seismic Tests

Figures D-1 through D-3 show plots of all the accelerations measured in the X-axis, Y-axis and Z-axis of the building for the 25% seismic test. These figures show many high frequency spikes that exceed 1 g even in the vertical direction (Figure D-3). These spikes are more than 2.5 times the values seen in the 10% tests, which suggest that there is some impact loading occurring. These spikes are localized and should not engage large portions of the mass of the building. The average accelerations plotted in Figures D-4 through D-7 eliminate the high frequency spikes, because the spikes do not occur at the same instant in time at all locations. The inertial loads and base shear plotted in Figures D-8 through D-11, appear to scale up well from the loading seen in the 10% test, where the amplitudes are only slightly greater than 2.5 times those of the 10% test. The number of cycles seen in both the average acceleration and applied inertial loads are the same as they were for the 10% test, which agrees well with the building primary natural frequencies.

Figure D-12 and D-13 plot the displacements in the X-axis and Y-axis, and Figures D-14 through D-17 plot the calculated deformations in both the X-axis and Y-axis, plus the average deformation in both directions (Dx_{Avg} and Dy_{Avg}). Figures D-15 and D-17 show that the average deformation oscillated at the same frequencies as during the 10% test, which shows good agreement with the measured natural frequencies of the building.

Figure D-1. Accelerations recorded in the X-axis during the 25% seismic test.

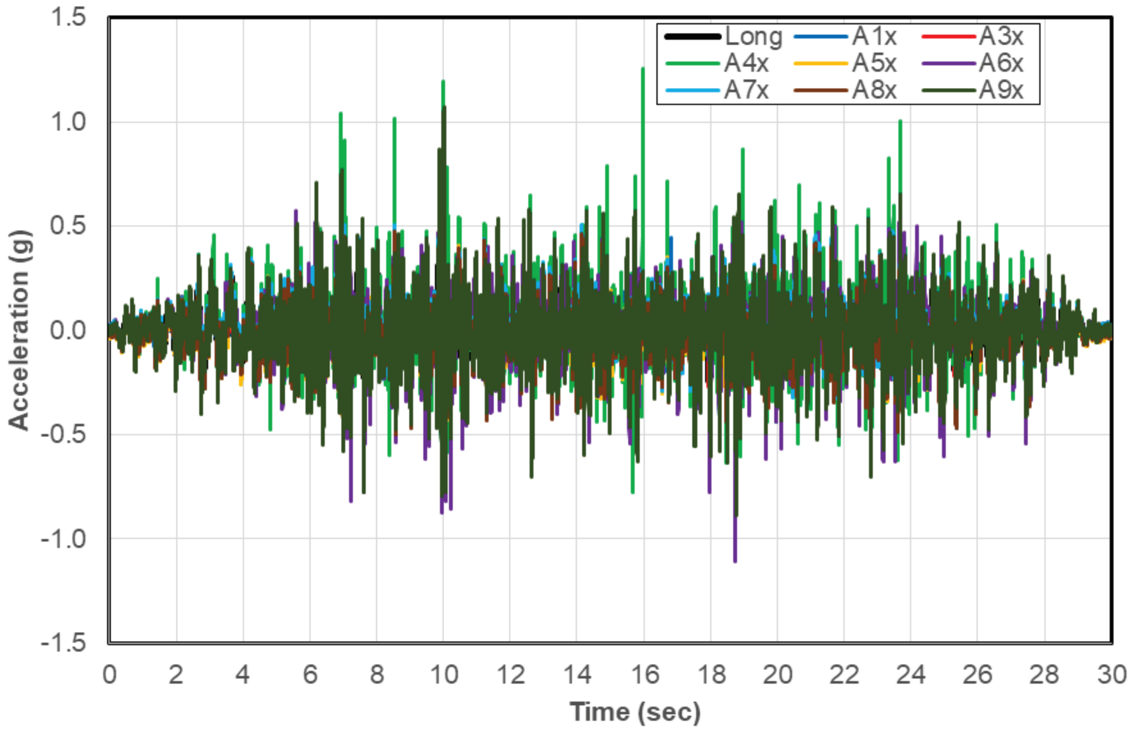


Figure D-2. Accelerations recorded in the Y-axis during the 25% seismic test.

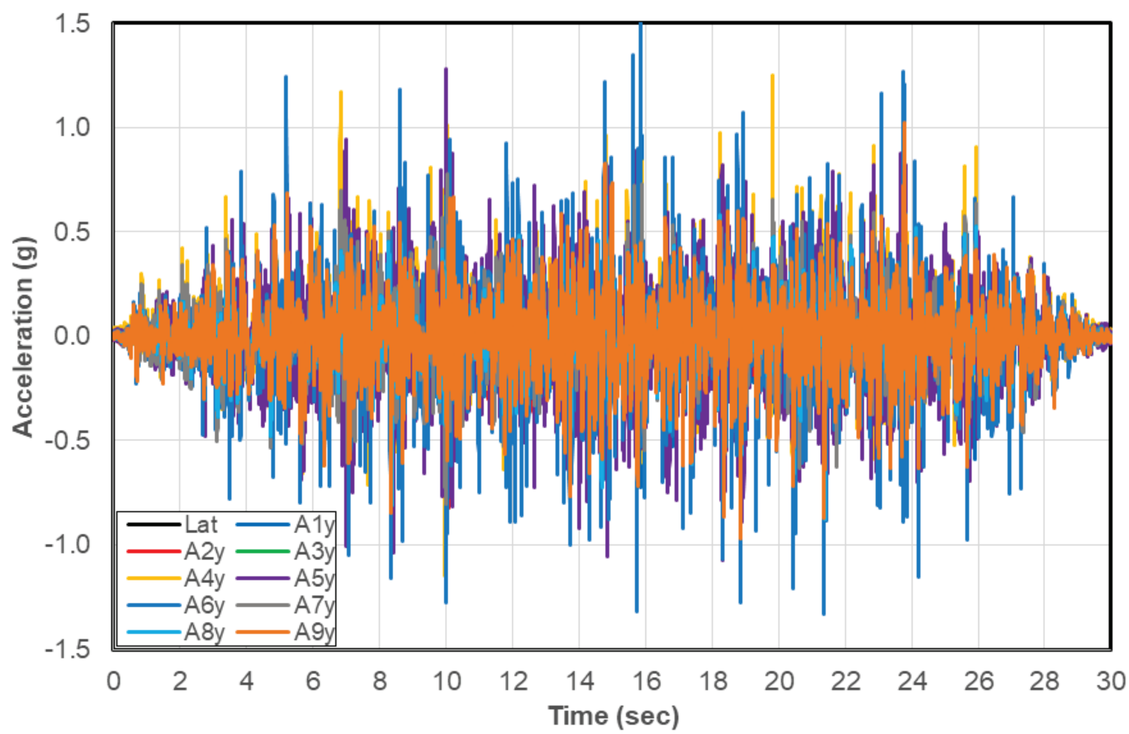


Figure D-3. Accelerations recorded in the Z-axis during the 25% seismic test.

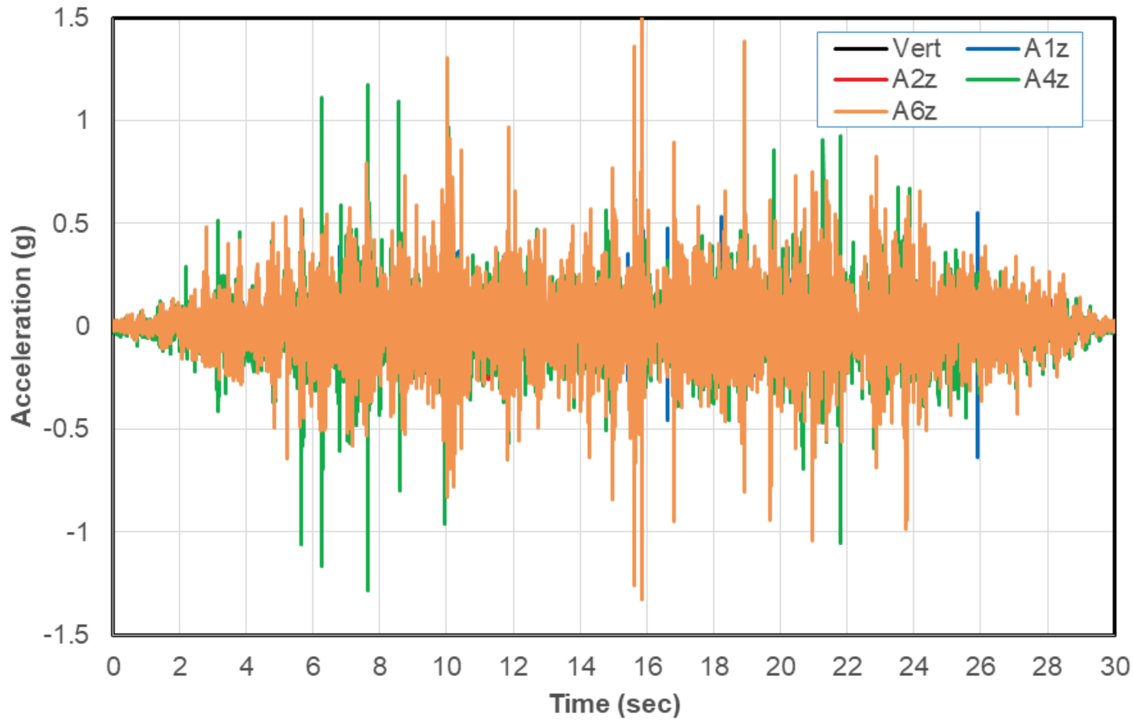


Figure D-4. Accelerations at bottom of walls in X-axis, 25% seismic test, 13 to 15 sec.

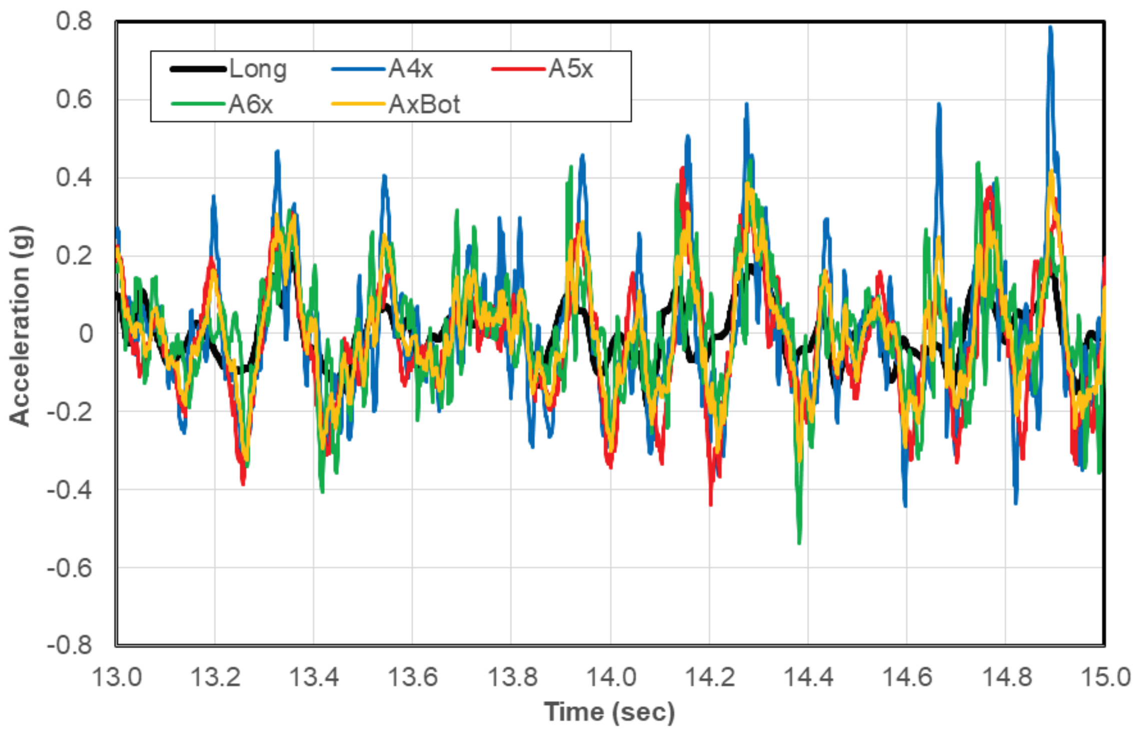


Figure D-5. Accelerations at the ceiling in the X-axis, 25% seismic test.

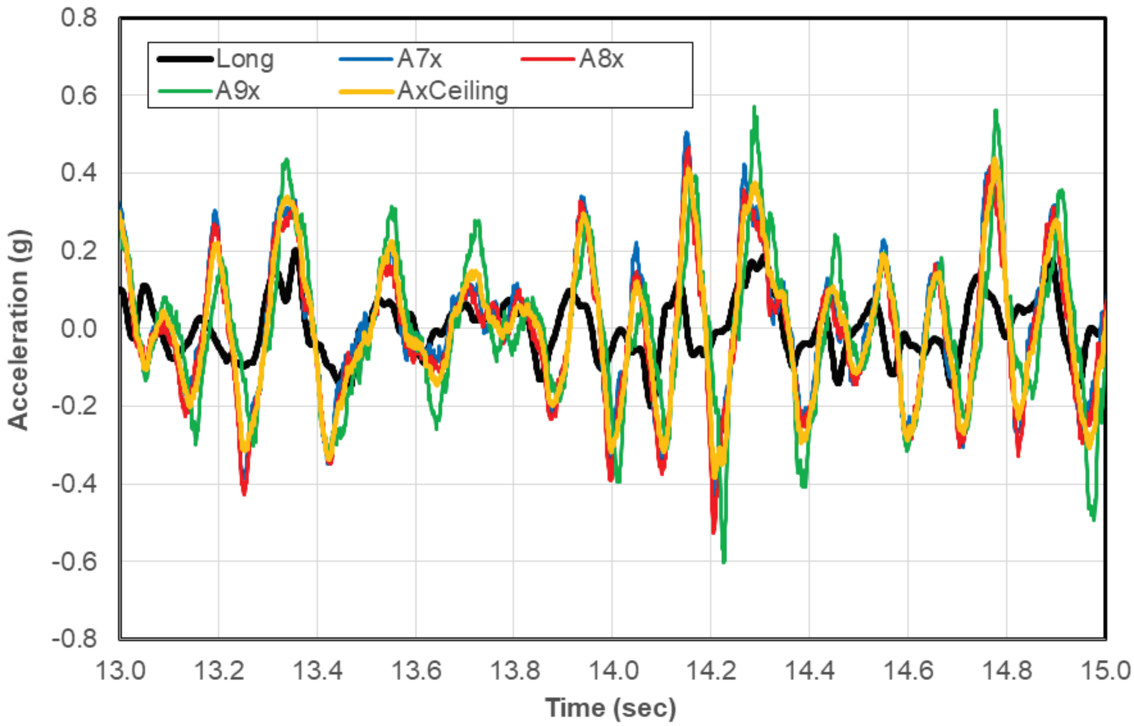


Figure D-6. Accelerations at the bottom of the walls in the Y-axis, 25% seismic test.

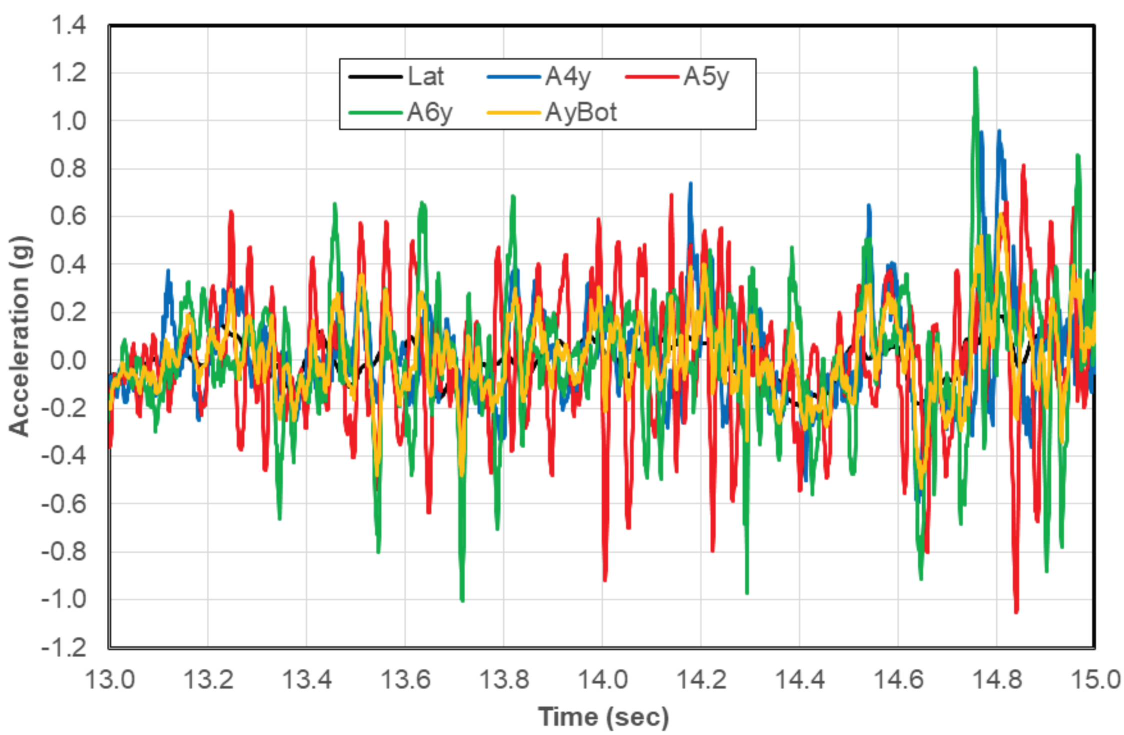


Figure D-7. Accelerations at the ceiling in the Y-axis, 25% seismic test.

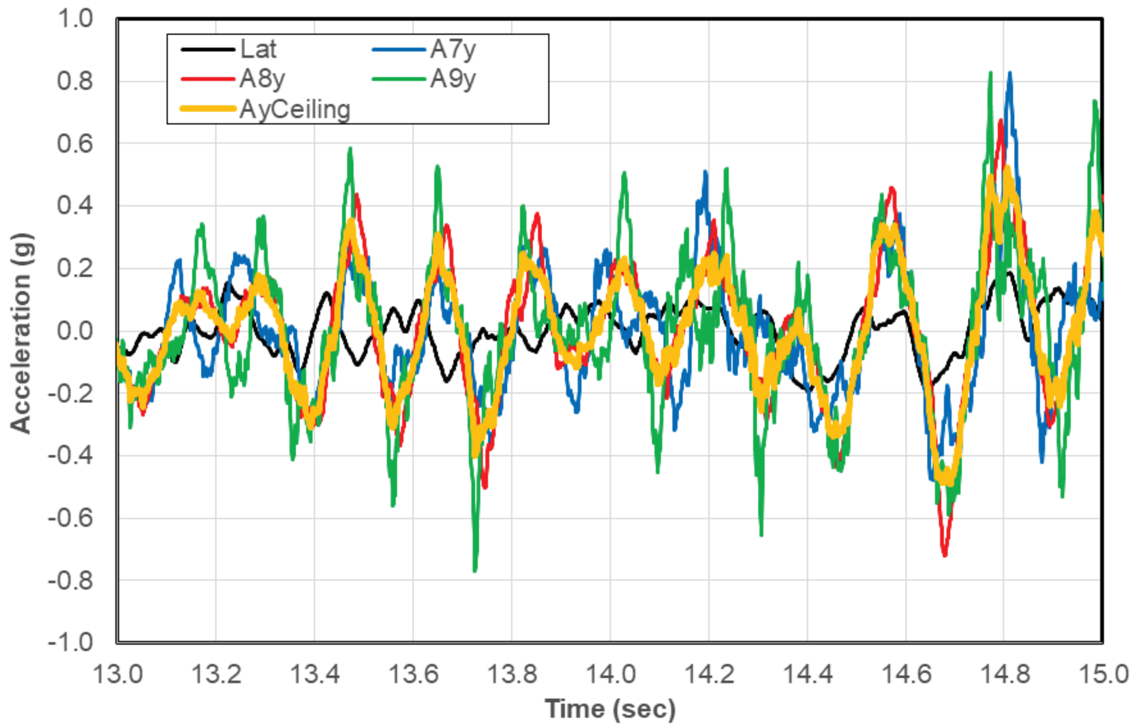


Figure D-8. Inertial forces for bottom, ceiling and base shear, X-axis, 25% seismic test.

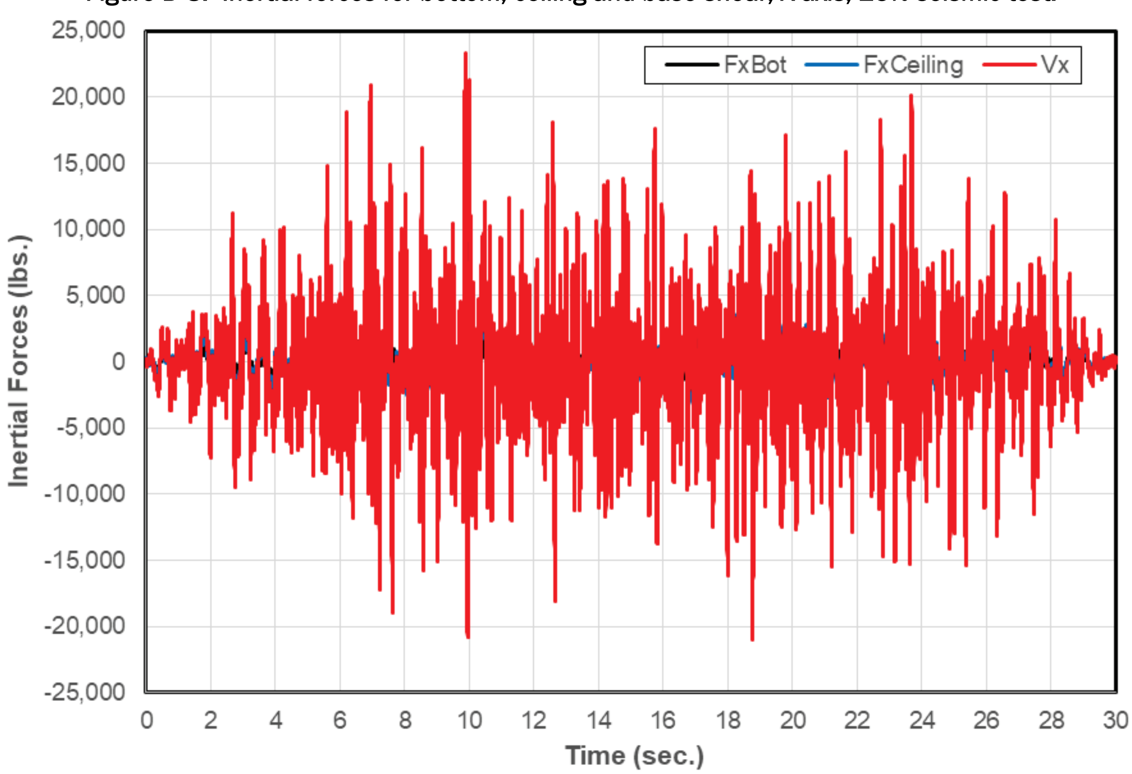


Figure D-9. Bottom & ceiling forces, & base shear, X-axis, 25% seismic test, 13-15 sec.

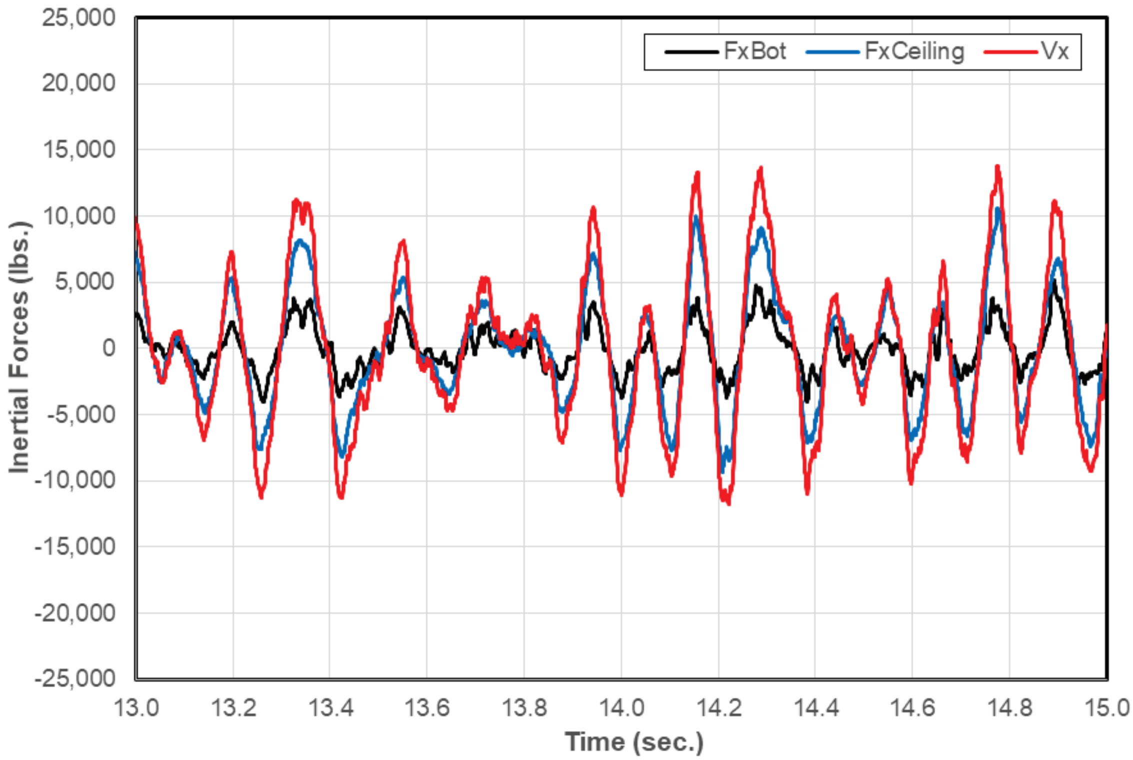


Figure D-10. Inertial forces for bottom, ceiling and base shear, Y-axis, 25% test.

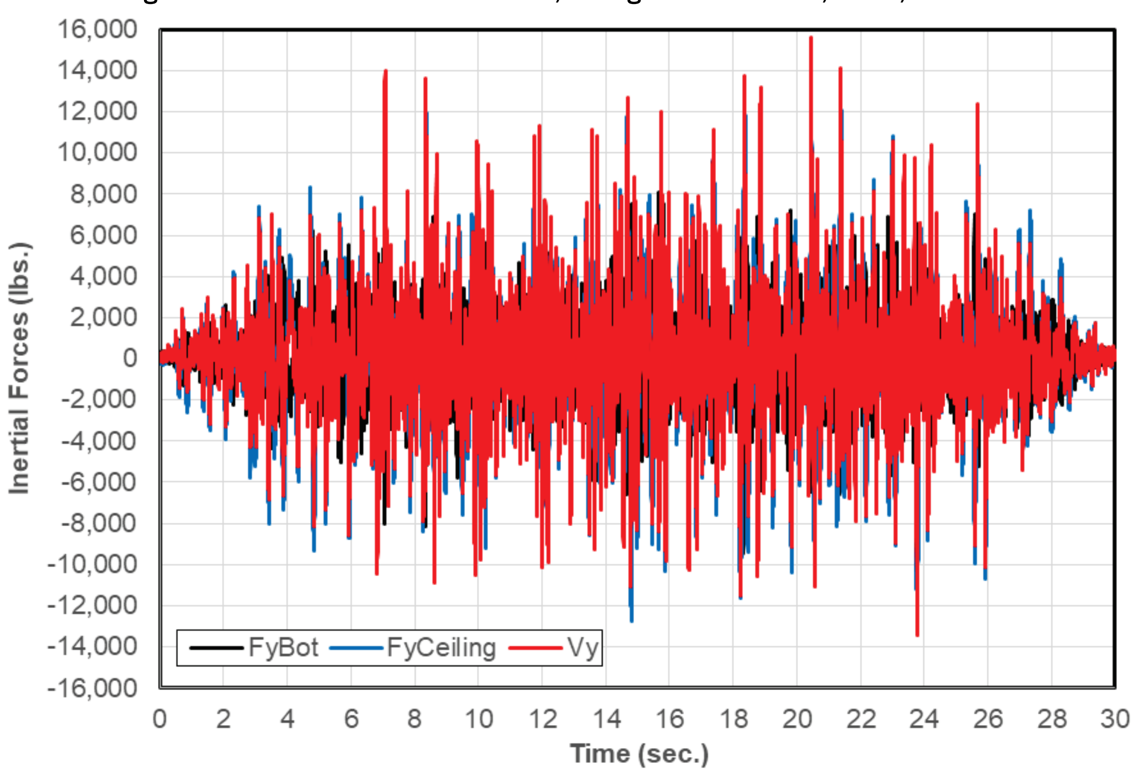


Figure D-11. Bottom & ceiling forces, & base shear, Y-axis, 25% test, 13-15 sec.

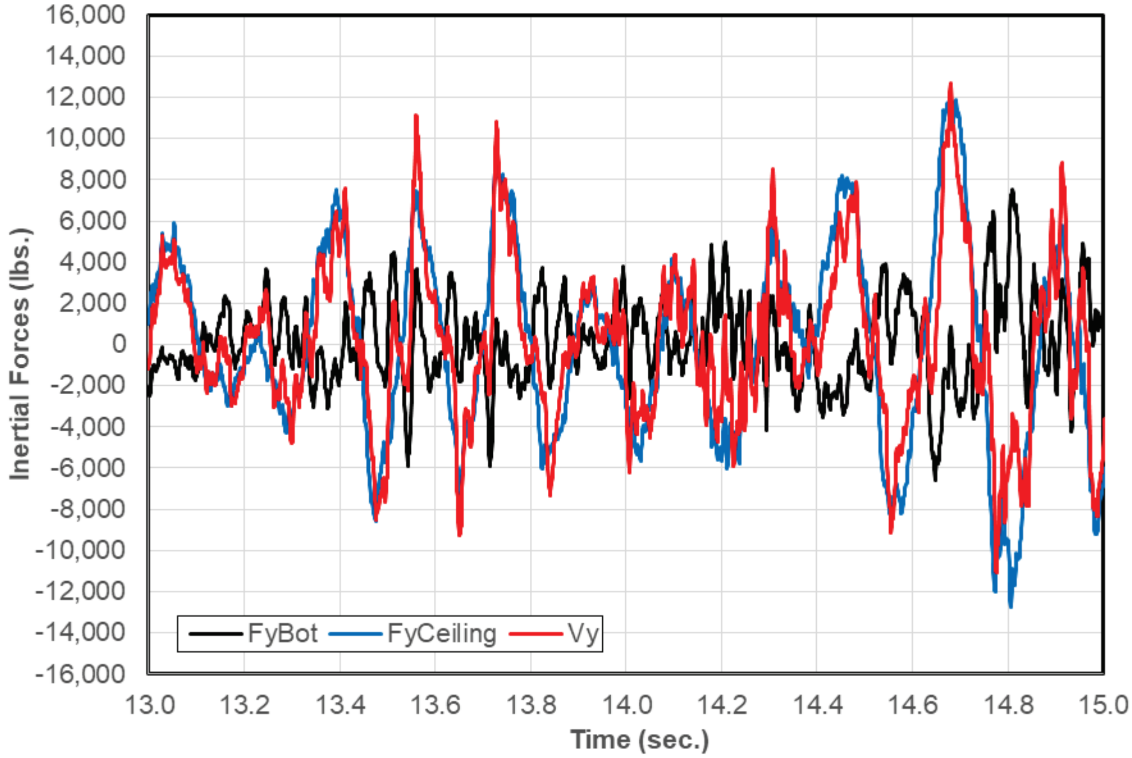


Figure D-12. Displacements recorded in the X-axis during the 25% seismic test.

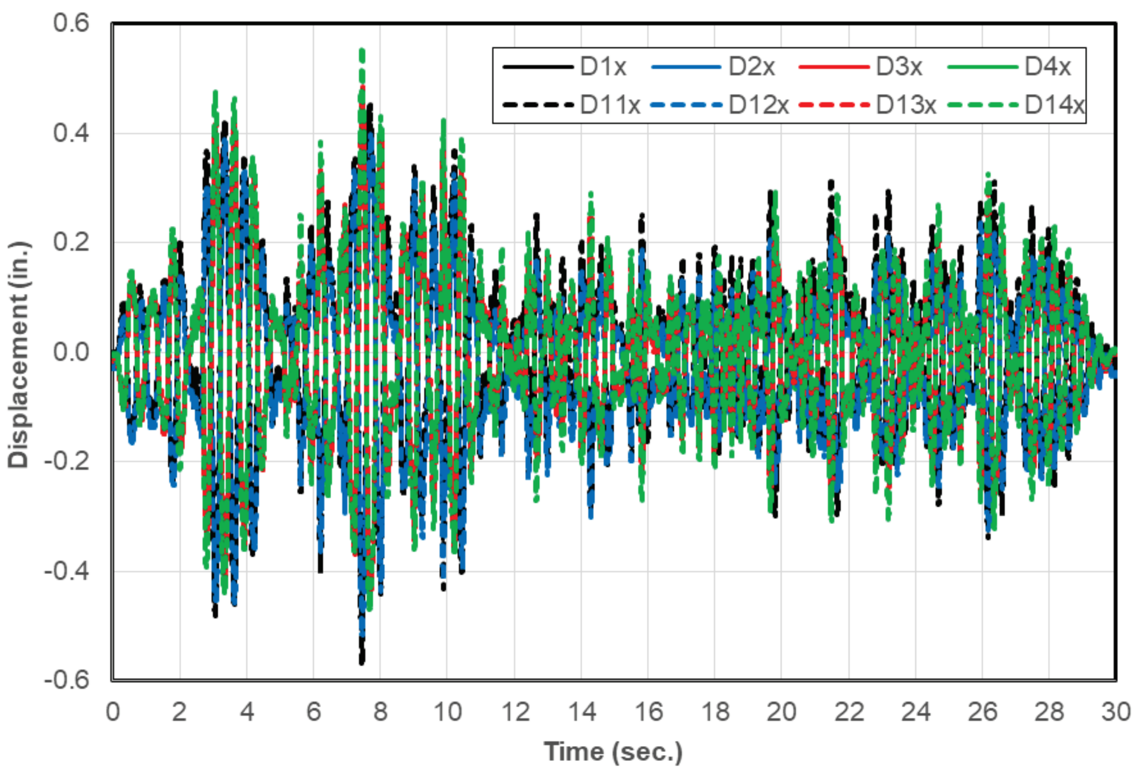


Figure D-13. Displacements recorded in the Y-axis during the 25% seismic test.

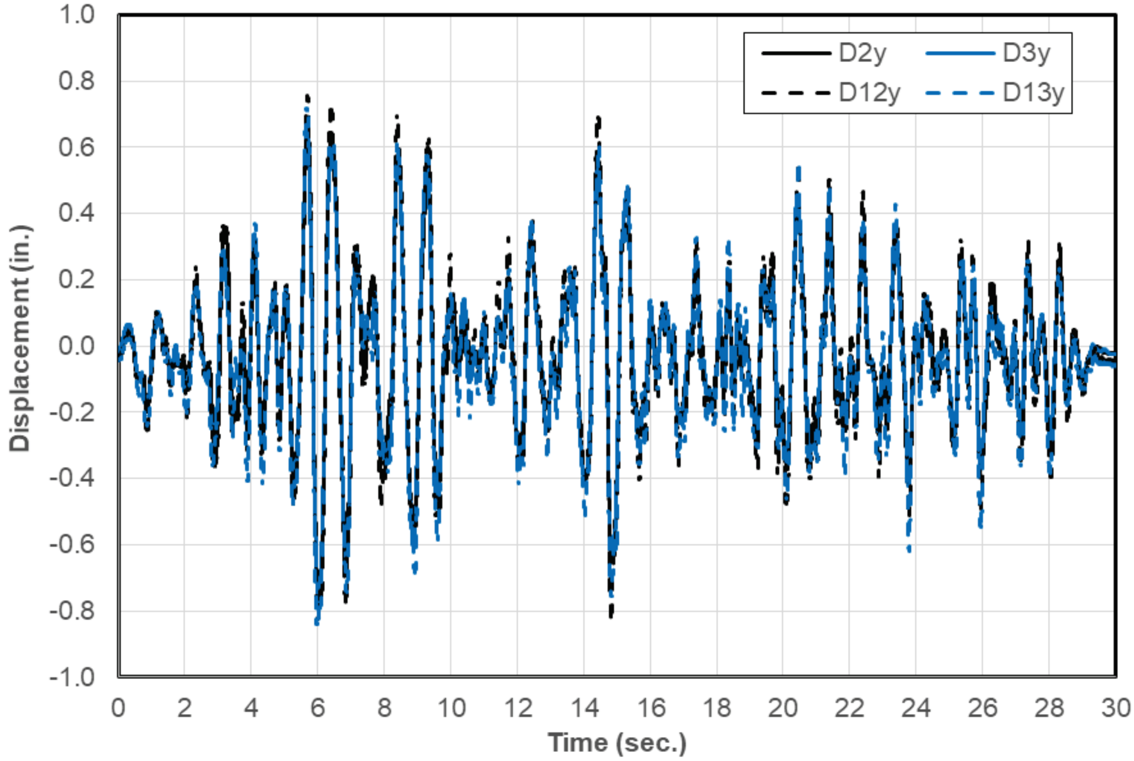


Figure D-14. Deformations in the X-axis during the 25% seismic test.

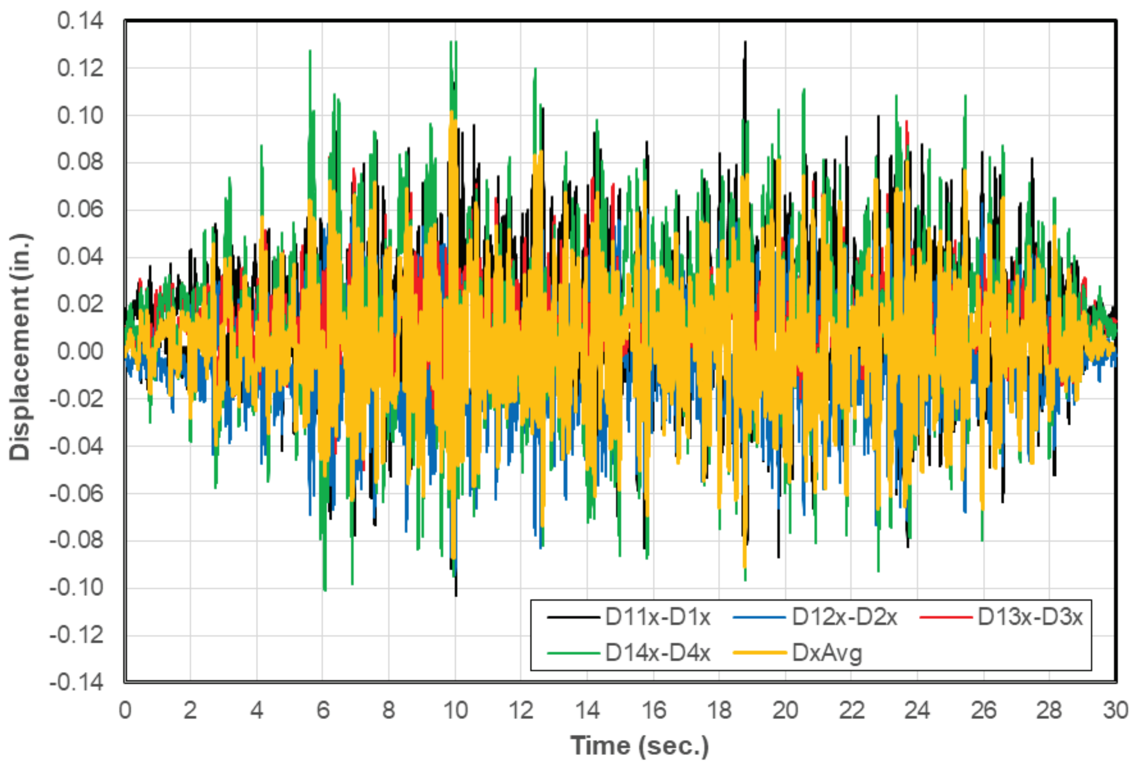


Figure D-15. Deformations in the X-axis during the 25% seismic test, 13 – 15 seconds.

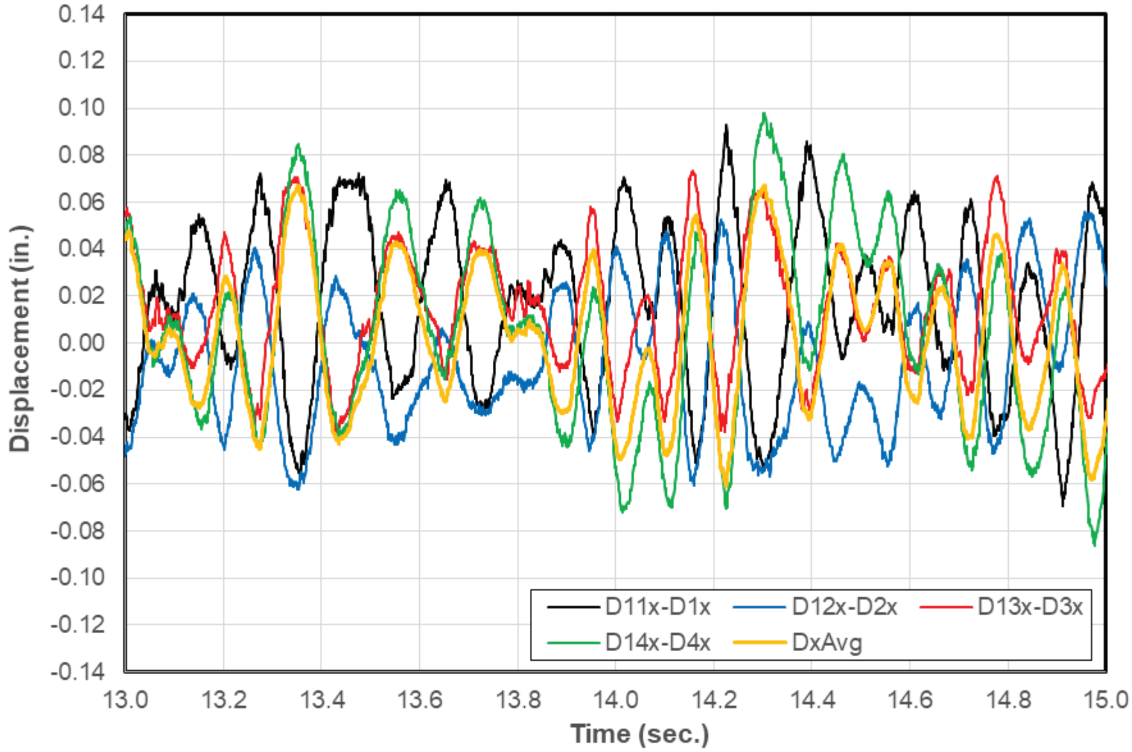


Figure D-16. Deformations in the Y-axis during the 25% seismic test.

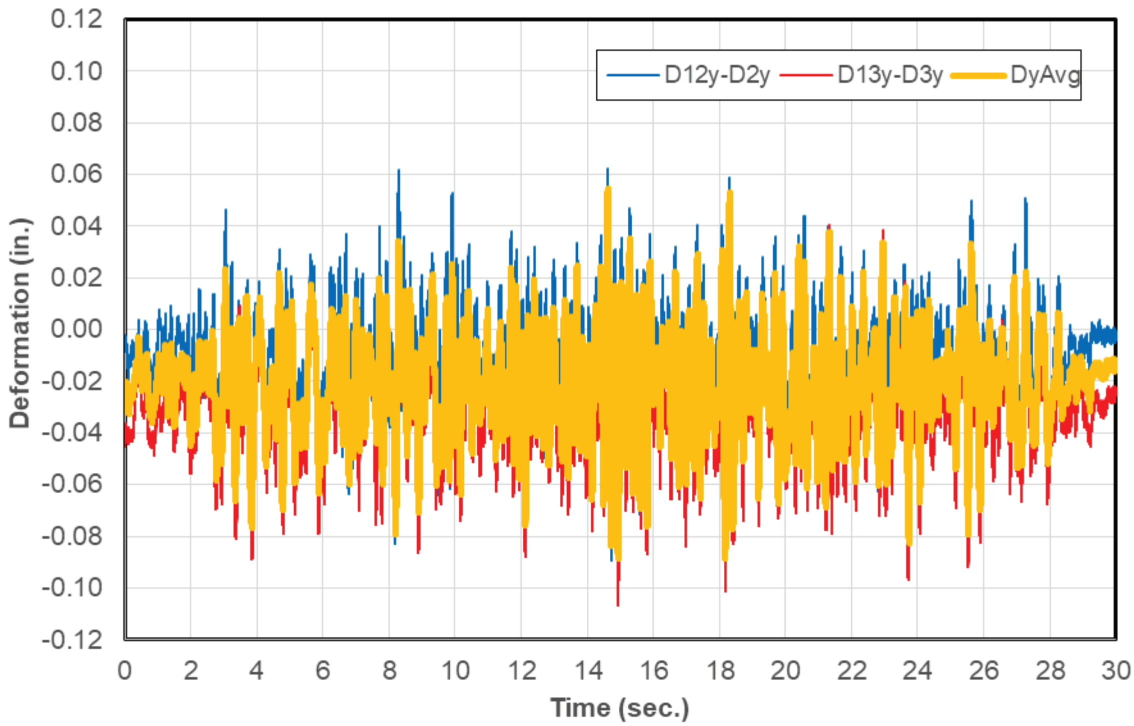
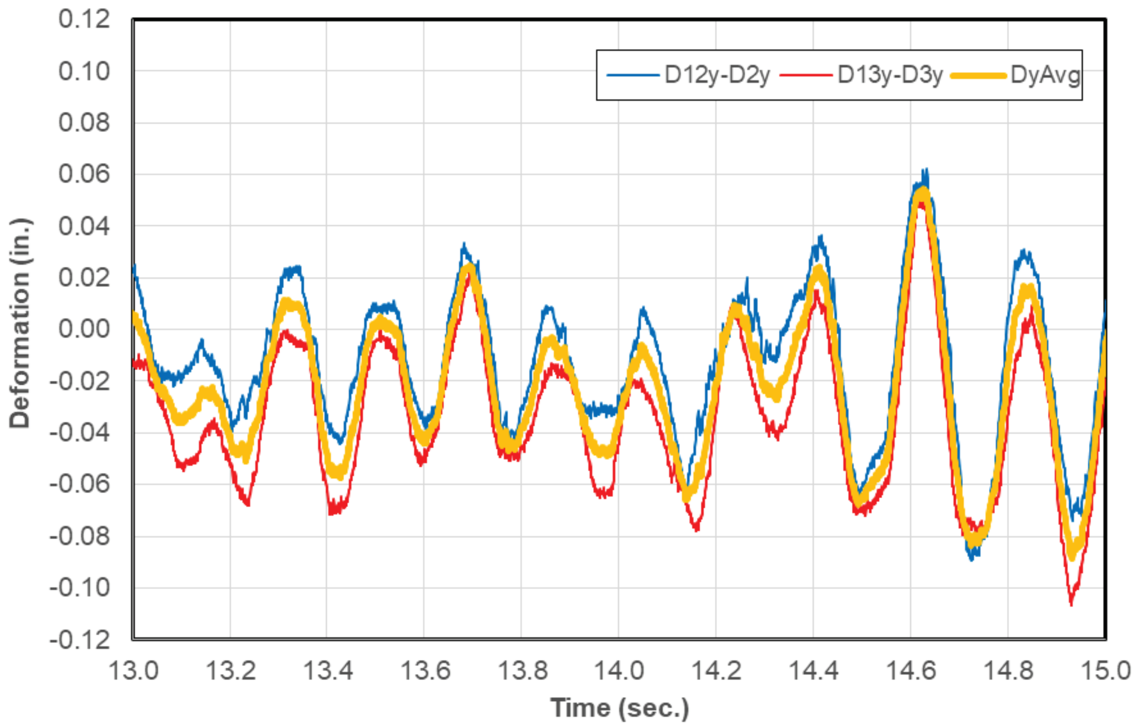


Figure D-17. Deformations in the Y-axis during the 25% seismic test, 13 to 15 seconds.



Appendix E: Data Plots for 50% Seismic Tests

Figures E-1 through E-3 plot all the accelerations measured in the X-axis, Y-axis, and Z-axis of the building for the 50% seismic test. These figures show high frequency spikes that far exceed 1 g even in the vertical direction (Figure E-3). These spikes are more than twice the values seen in the 25% tests, indicating greater impact loading than the 25% test. The spikes are localized and should not engage large portions of the mass of the building. The average accelerations plotted in Figures E-4 through E-7 reduce the high frequency impacts, while still preserving a reasonable portion of this increased loading effect, which does damage the building. The inertial loads and base shear plotted in Figures E-8 through E-11 scale up well from the loading seen in the 25% test, where the amplitudes of the high frequency spike portions of these plots are somewhat greater than twice the values seen in the 25% test. The number of cycles seen in the average acceleration and inertial loading decreased in the X-axis to 7 Hz. The average acceleration and inertial loading at the ceiling both decreased to under 5 Hz. The average acceleration in the Y-axis is greater than at the bottom of the building (Figure E-6) than the ceiling level (Figure E-7). The inertial loads at the ceiling level do oscillate at under 5 Hz (Figure E-11), but the higher frequency, higher amplitude accelerations and inertial loads at the bottom causes the base shear in the Y-axis (Vy) to no longer oscillate at the building natural frequency.

Figure E-12 and E-13 plot the displacements in the X-axis and Y-axis, and Figures E-14 through E-17 plot the calculated deformations in both the X-axis and Y-axis, plus the average deformation in both directions (Dx_{Avg} and Dy_{Avg}). Figure E-15 shows that the X-axis average deformation oscillates at about 6.3 Hz, while the Y-axis deformation oscillates at about 4.3 Hz. These frequencies have decreased from the values of 7.7 Hz and 5.1 Hz (Table 5). These cycle counts are a crude estimate of frequency content, but they do show that the building natural frequencies have decreased, which indicates softening and increased damage.

Figure E-1. Accelerations recorded in the X-axis during the 50% seismic test.

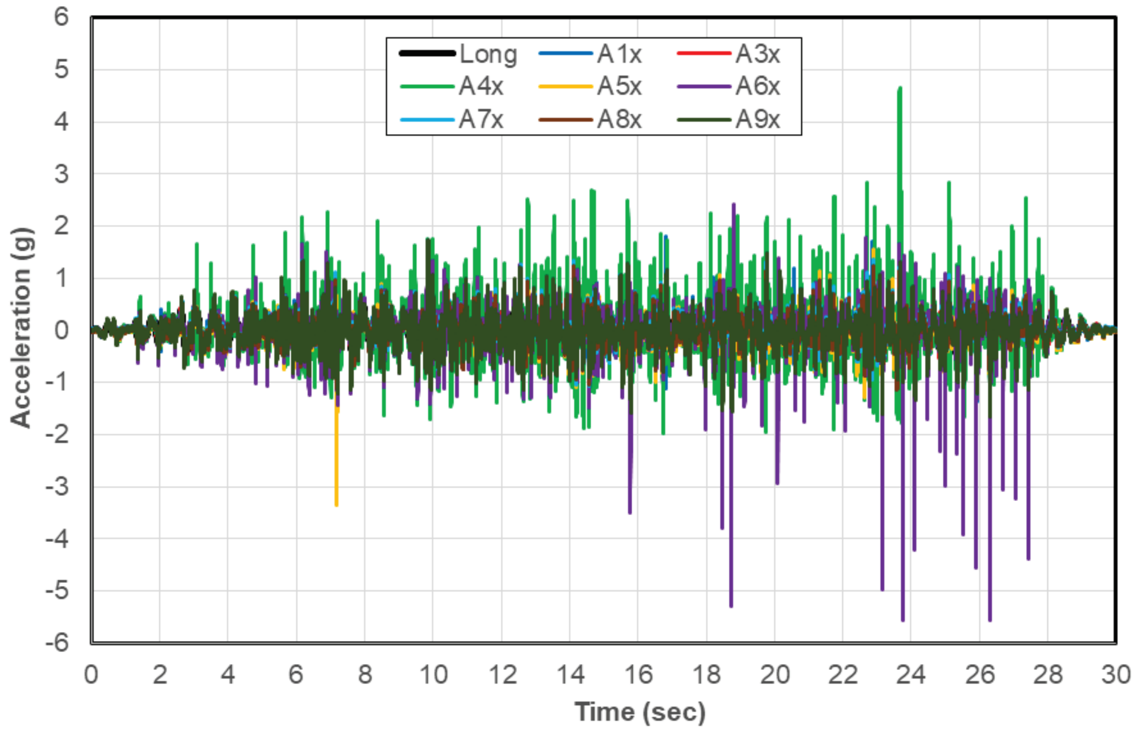


Figure E-2. Accelerations recorded in the Y-axis during the 50% seismic test.

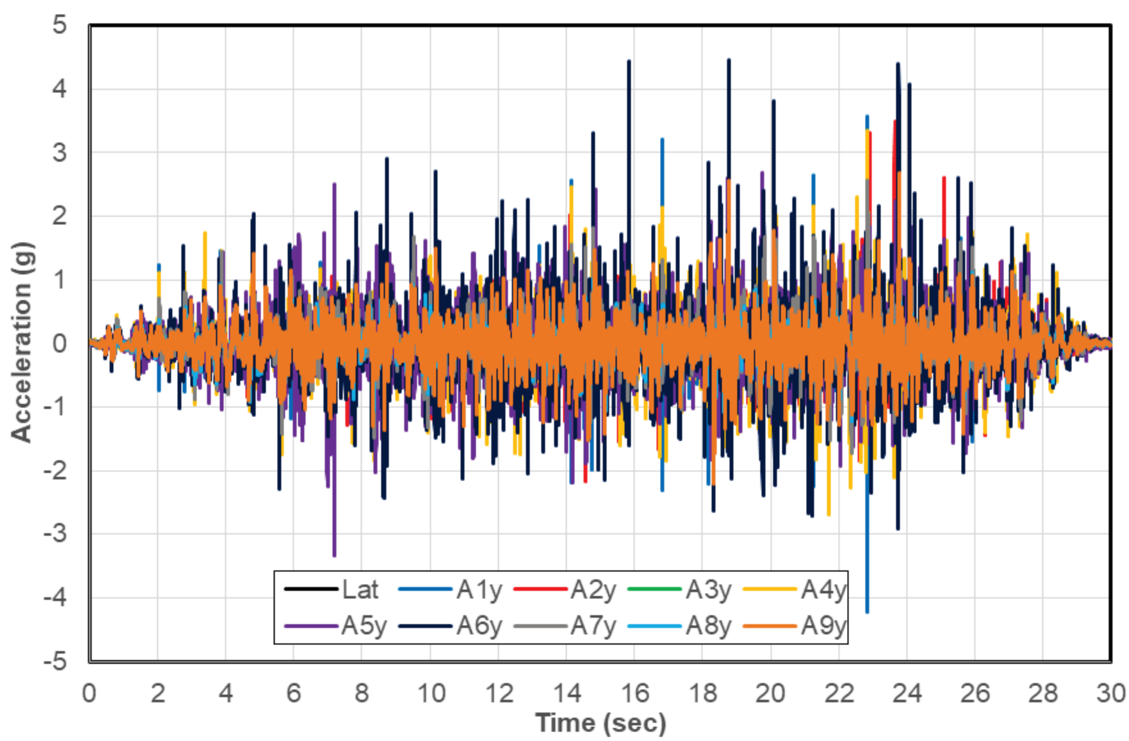


Figure E-3. Accelerations recorded in the Z-axis during the 50% seismic test.

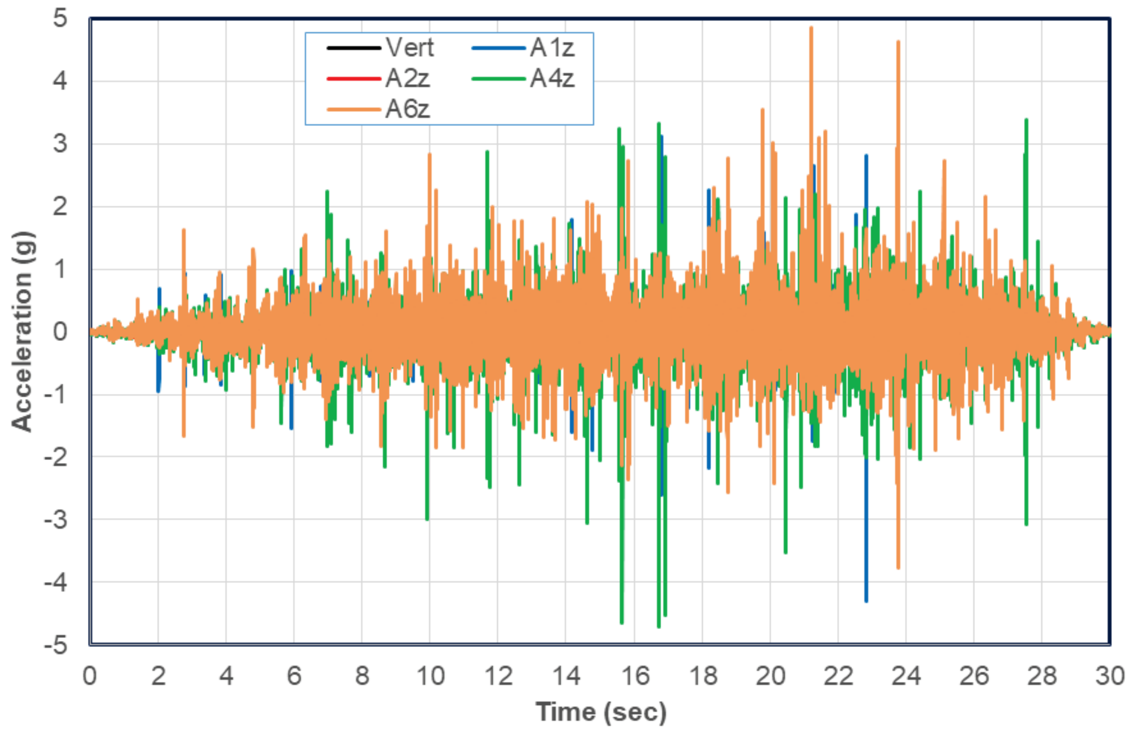


Figure E-4. Accelerations at bottom of walls in X-axis, 50% seismic test, 13 to 15 sec.

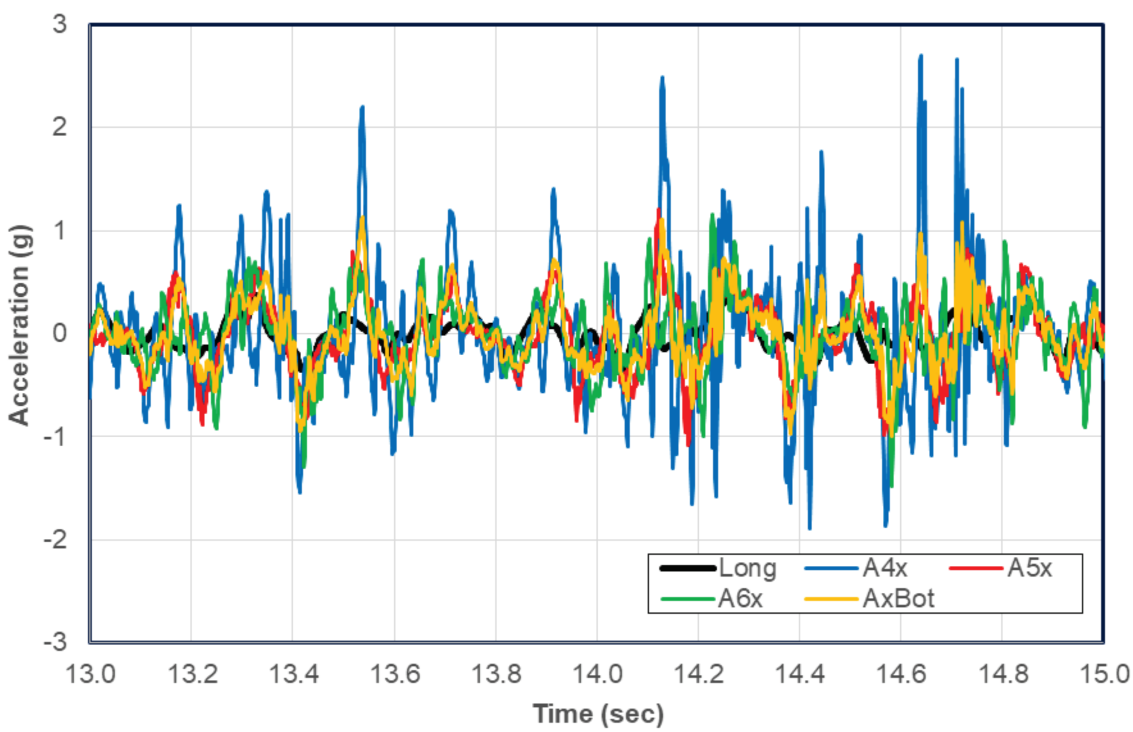


Figure E-5. Accelerations at the ceiling in the X-axis, 50% seismic test.

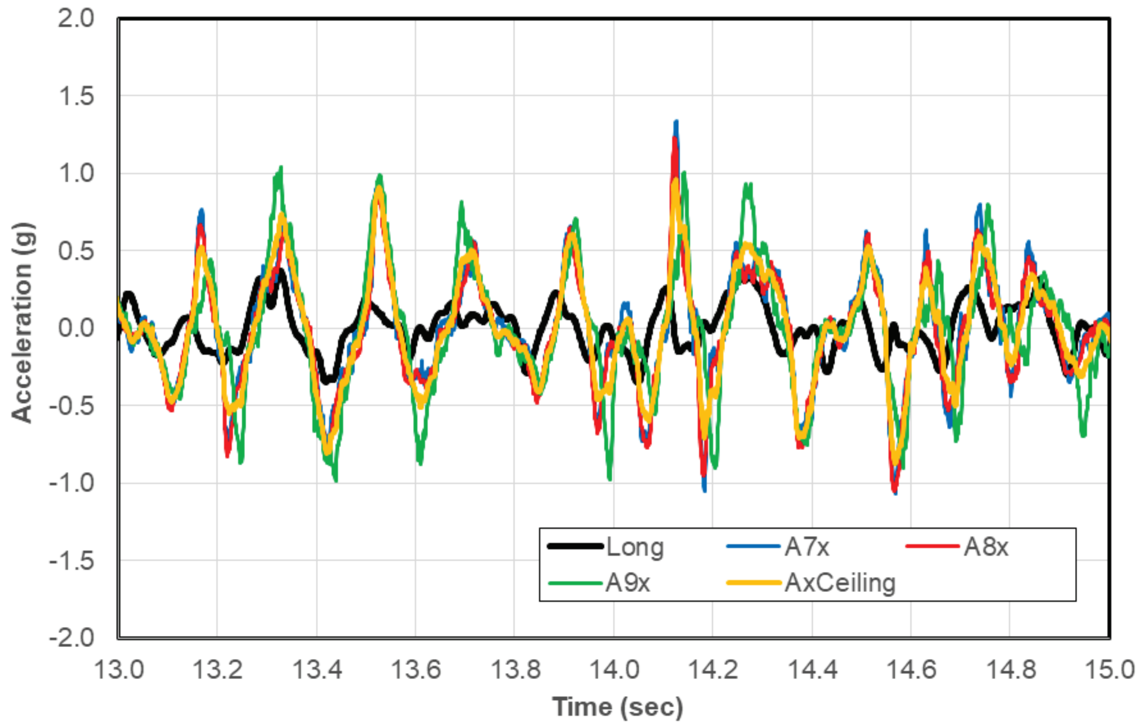


Figure E-6. Accelerations at the bottom of the walls in the Y-axis, 50% seismic test.

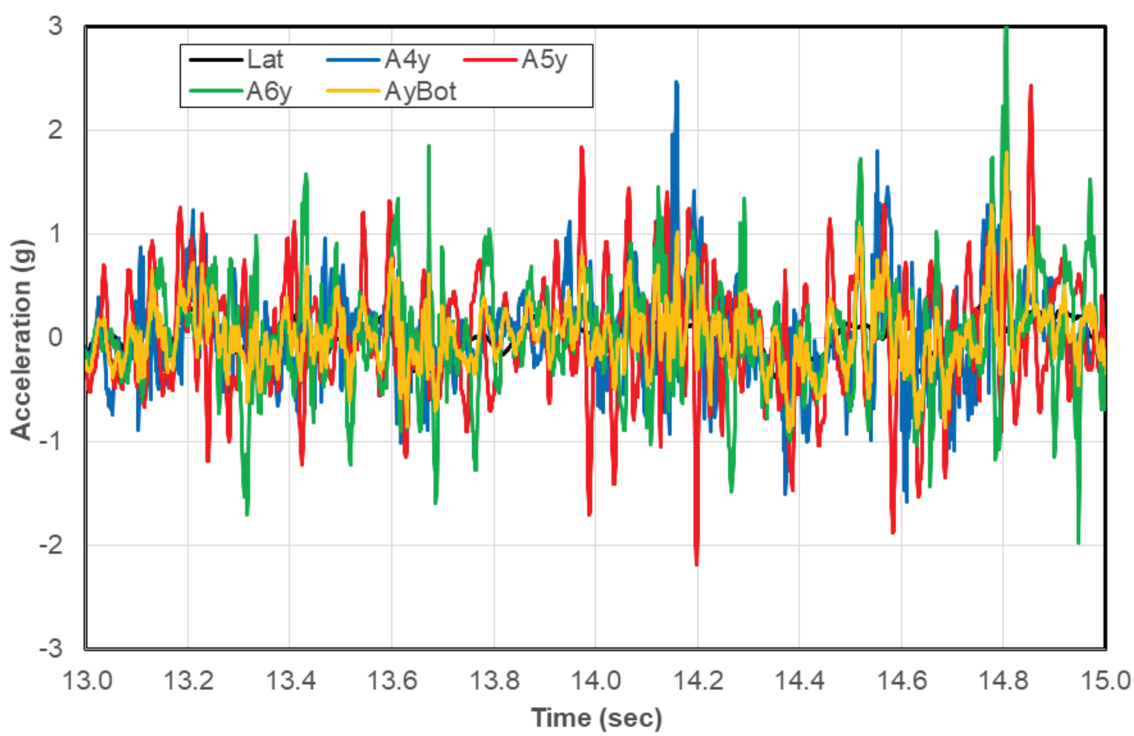


Figure E-7. Accelerations at the ceiling in the Y-axis, 50% seismic test.

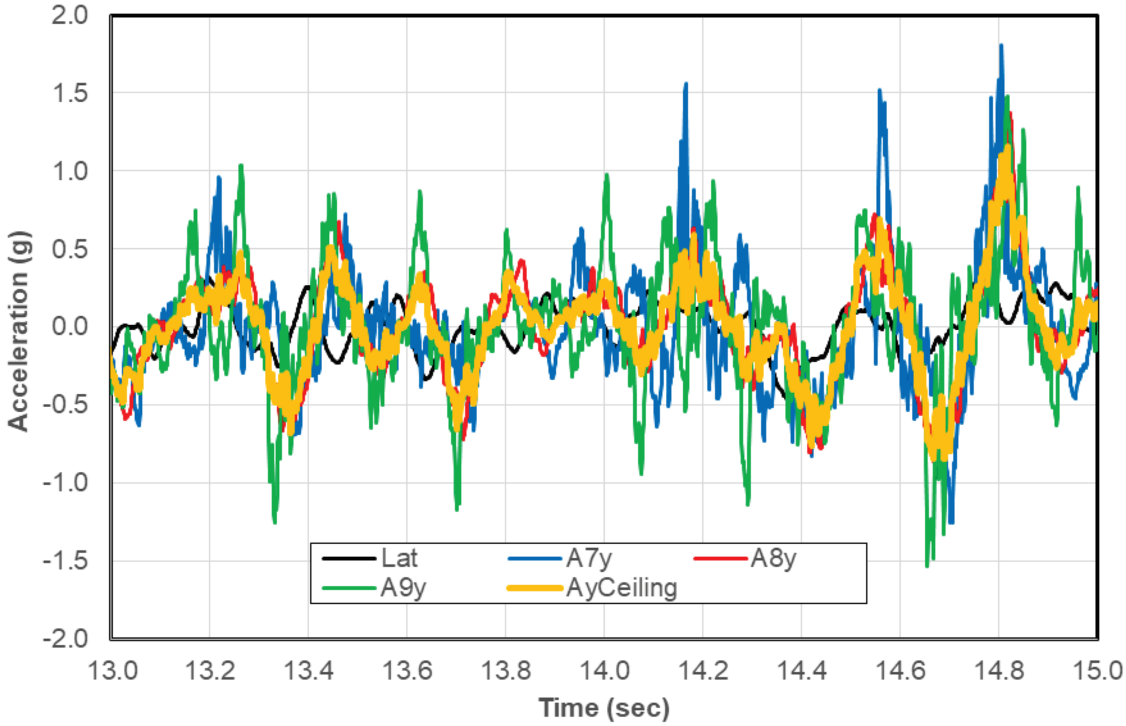


Figure E-8. Inertial forces for bottom, ceiling and base shear, X-axis, 50% seismic test.

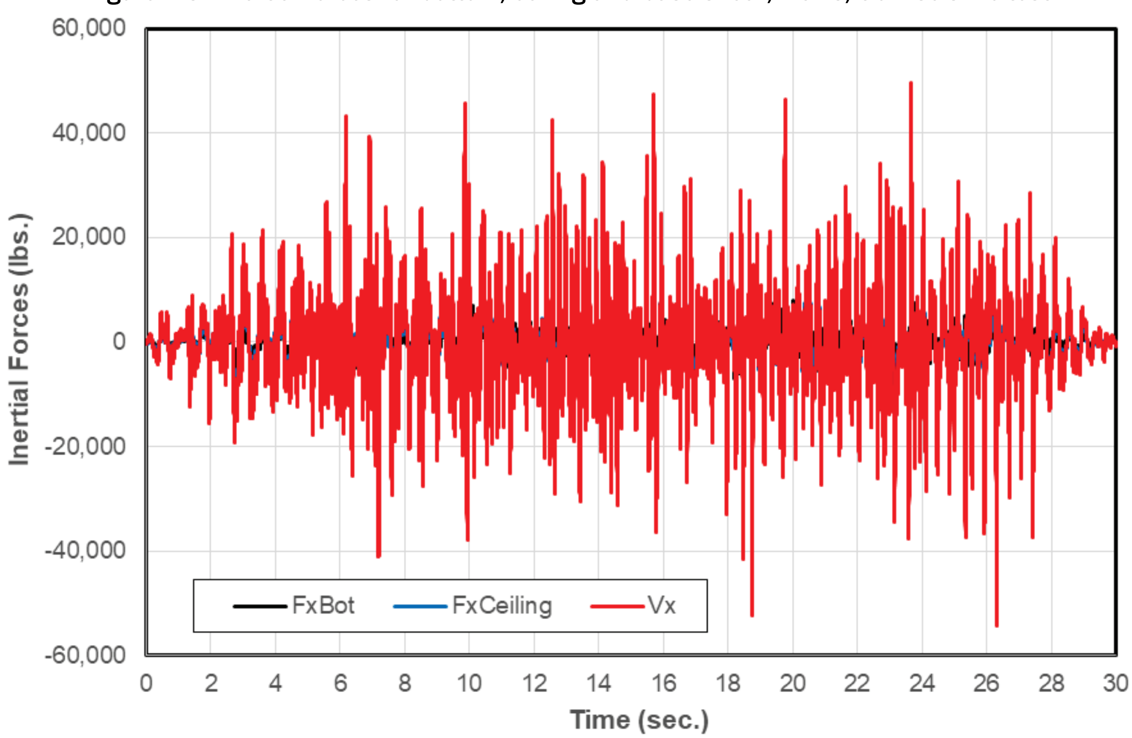


Figure E-9. Bottom & ceiling forces, & base shear, X-axis, 50% seismic test, 13-15 sec.

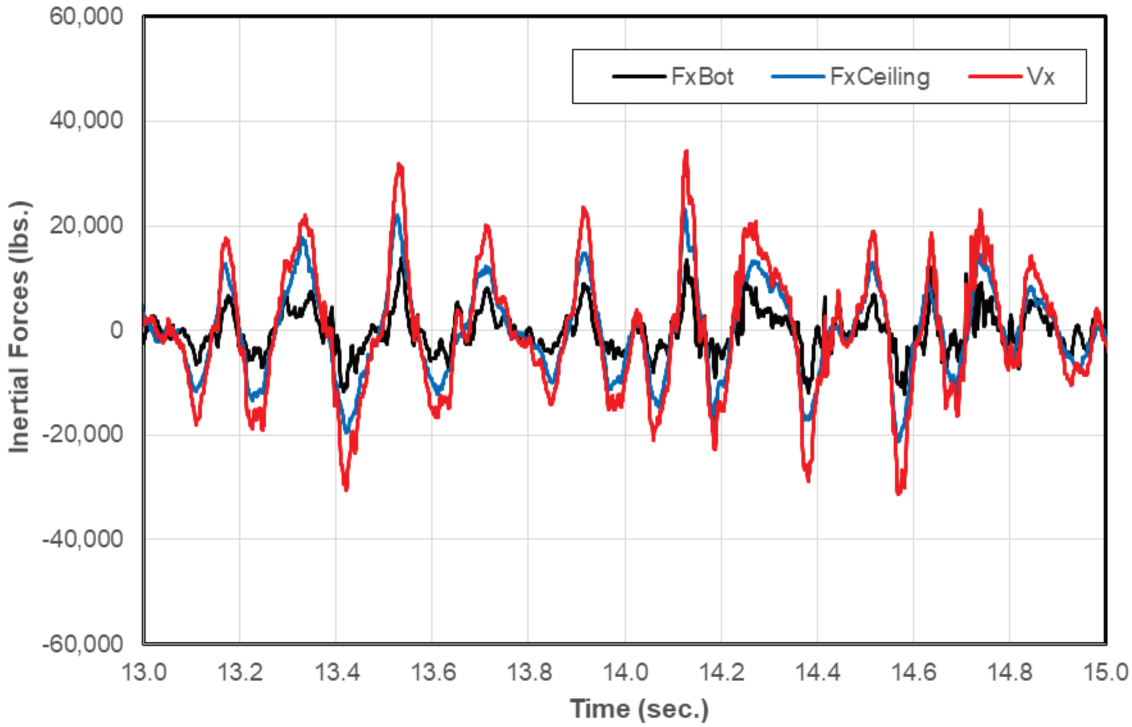


Figure E-10. Inertial forces for bottom, ceiling and base shear, Y-axis, 50% test.

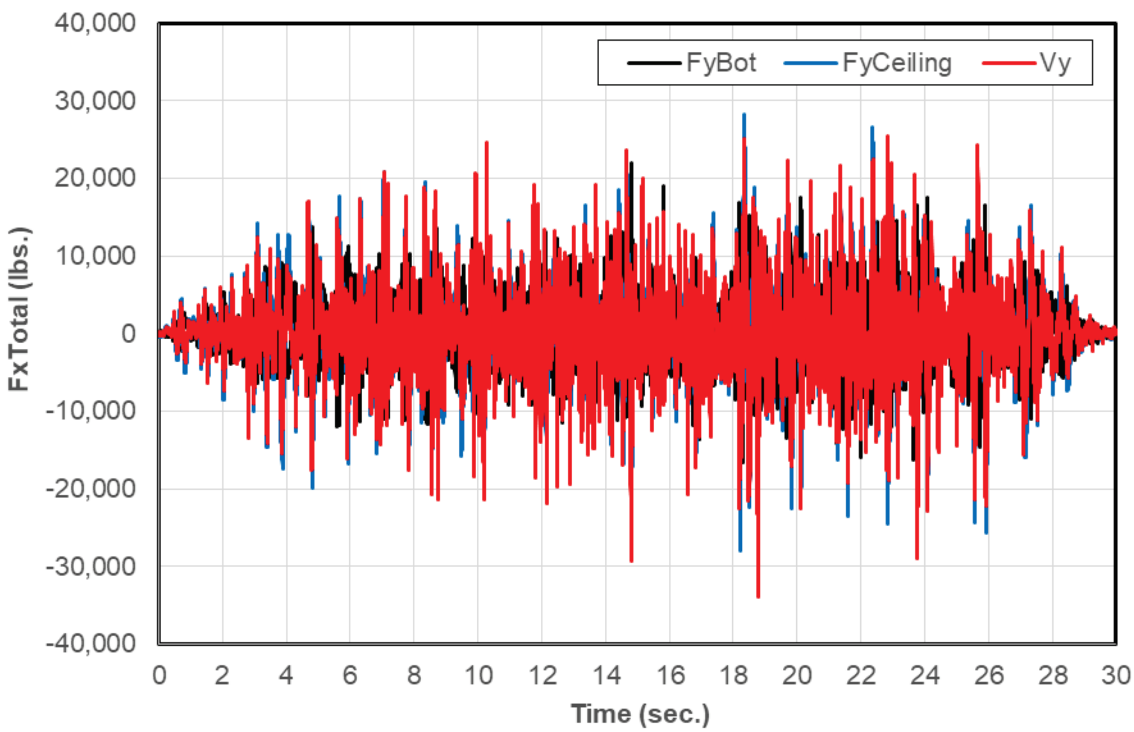


Figure E-11. Bottom & ceiling forces, & base shear, Y-axis, 50% test, 13-15 sec.

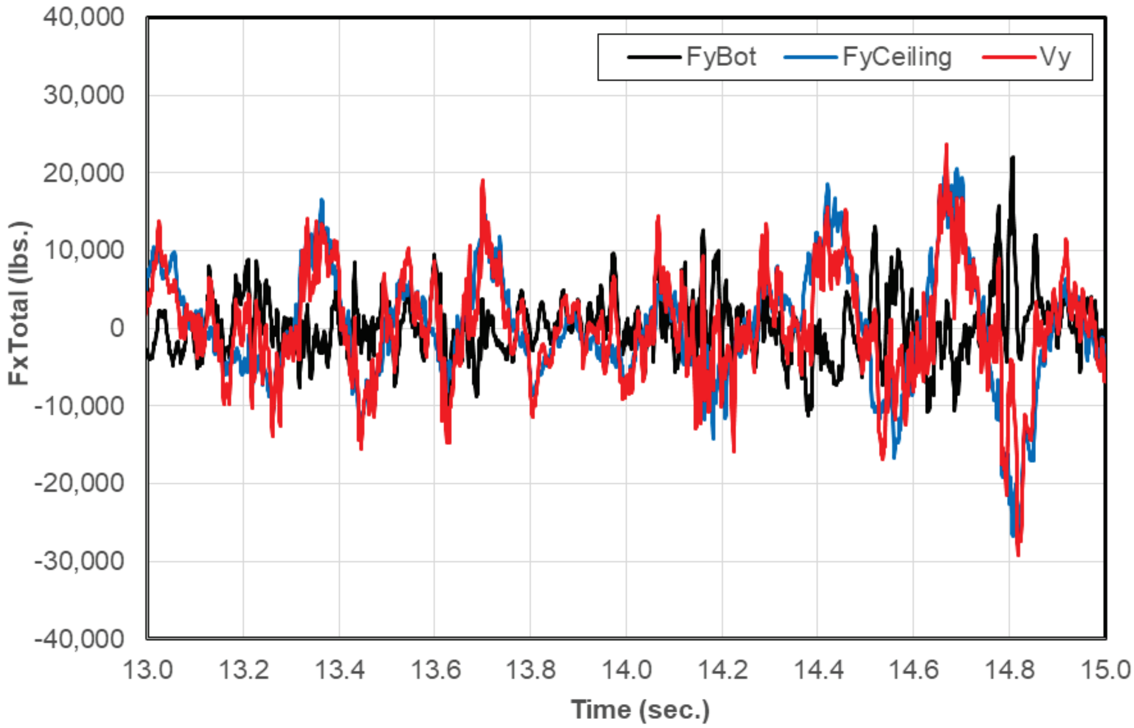


Figure E-12. Displacements recorded in the X-axis during the 50% seismic test.

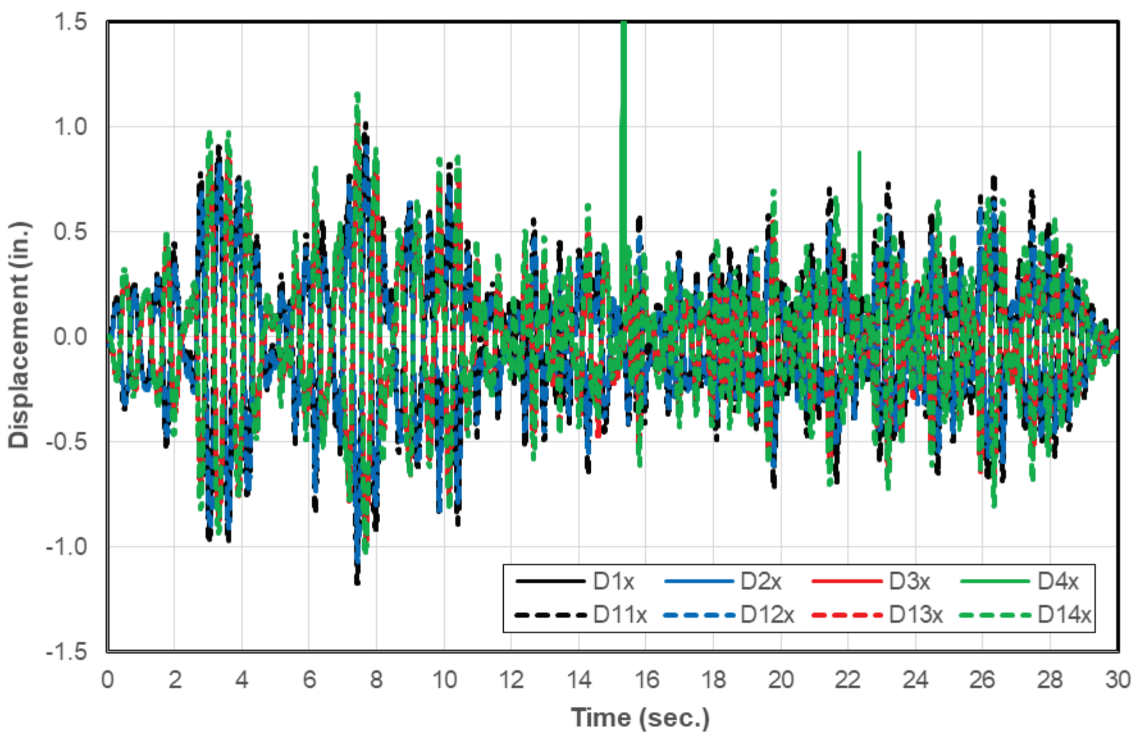


Figure E-13. Displacements recorded in the Y-axis during the 50% seismic test.

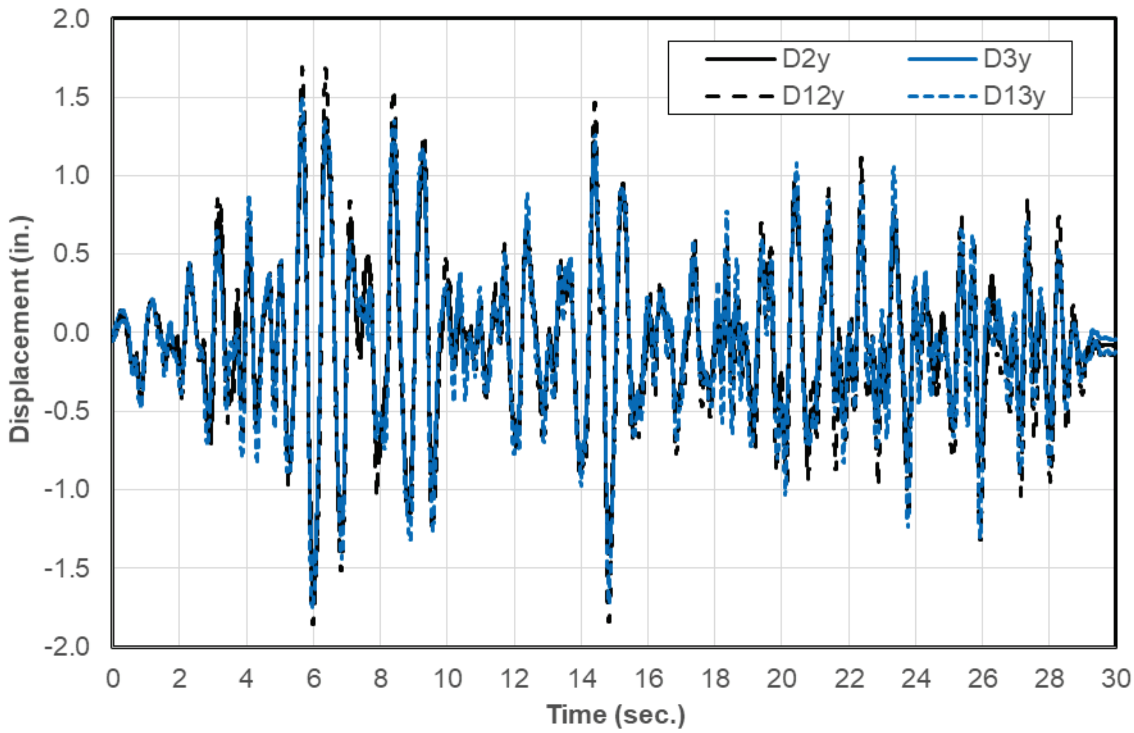


Figure E-14. Deformations in the X-axis during the 50% seismic test.

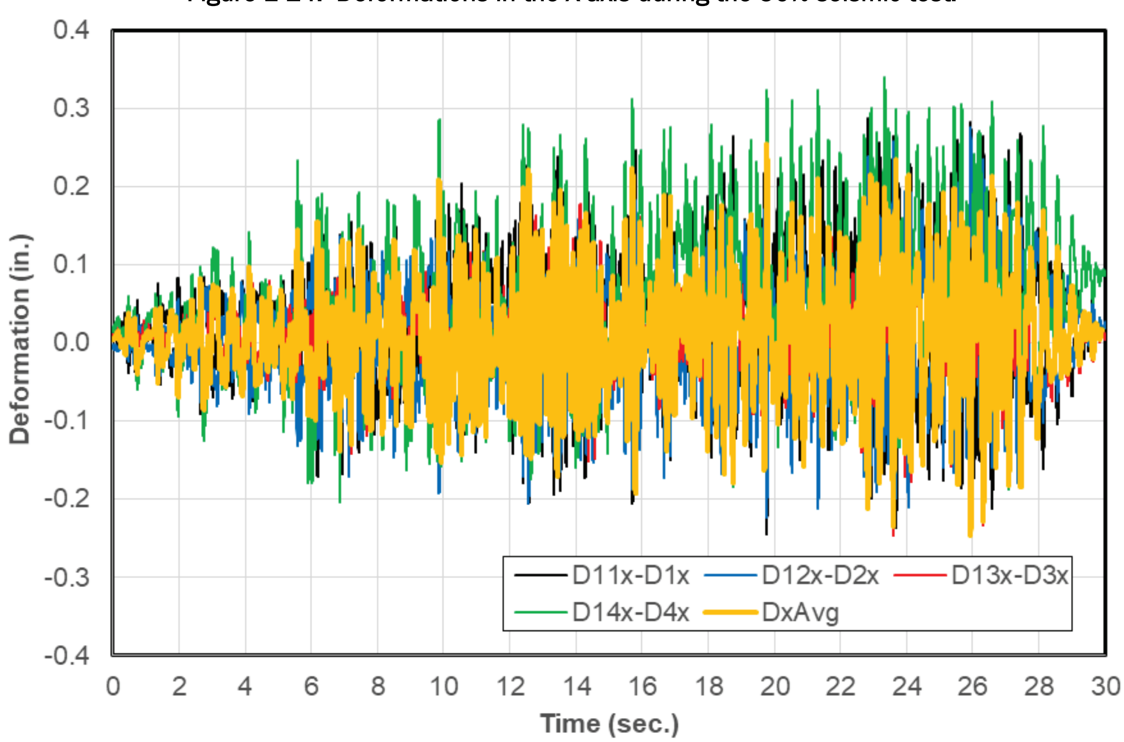


Figure E-15. Deformations in the X-axis during the 50% seismic test, 13 – 15 seconds.

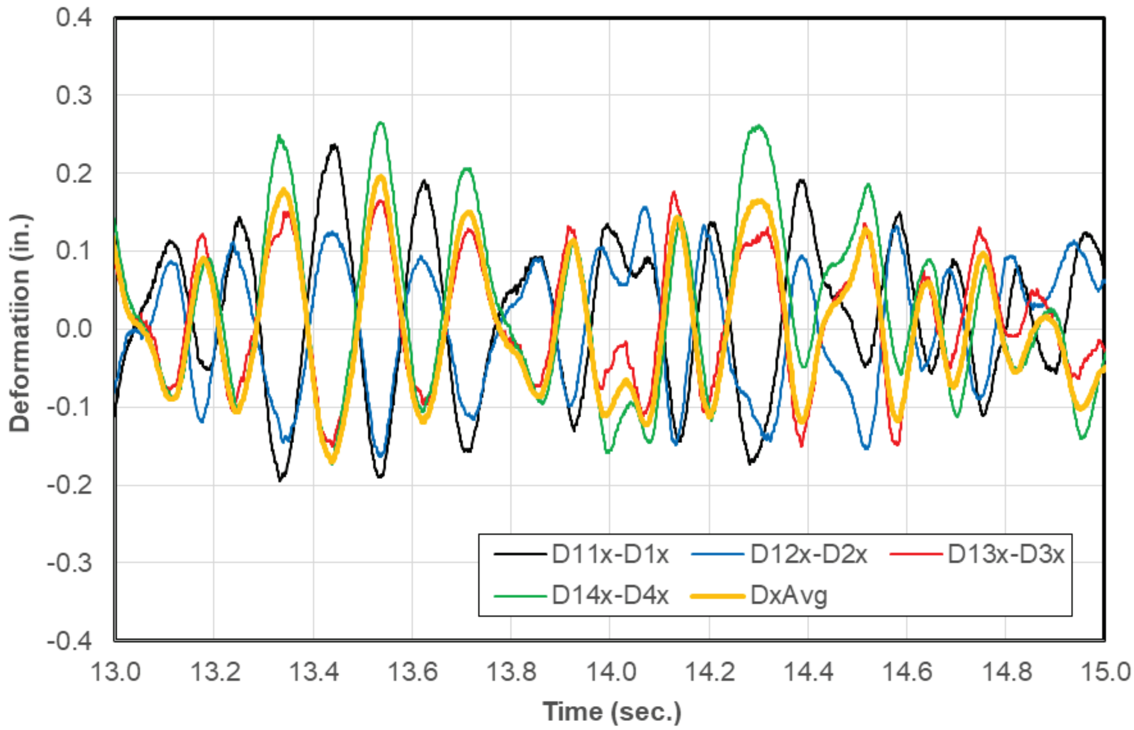


Figure E-16. Deformations in the Y-axis during the 50% seismic test.

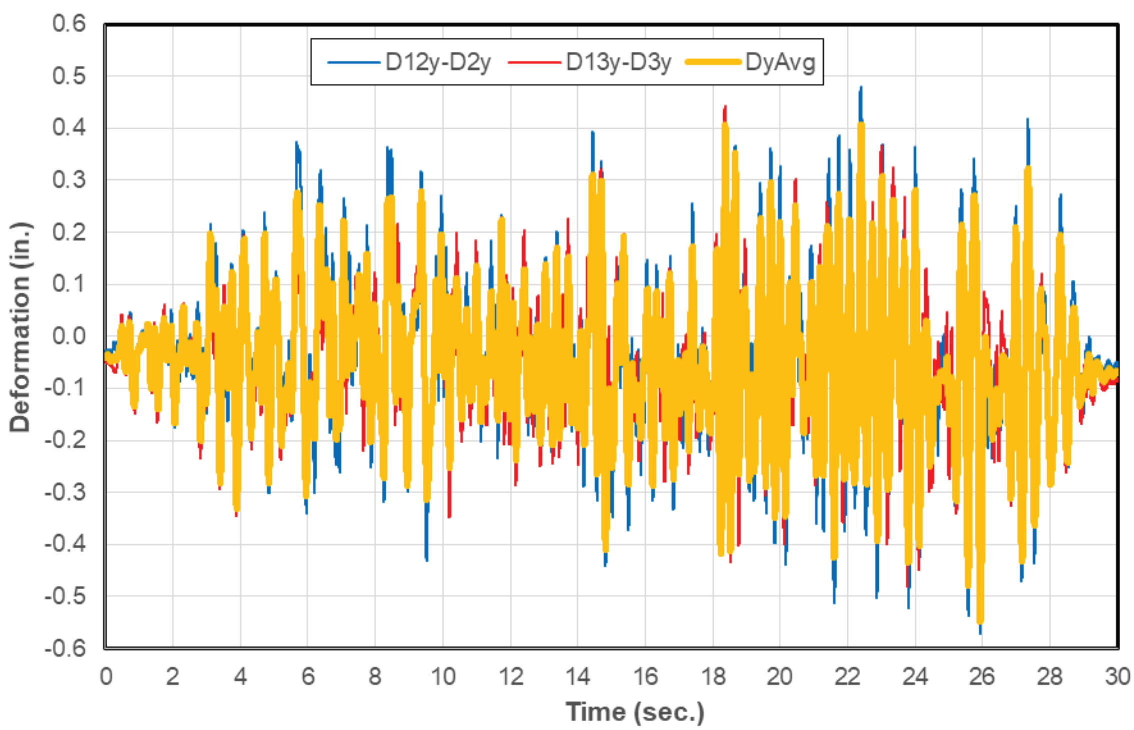
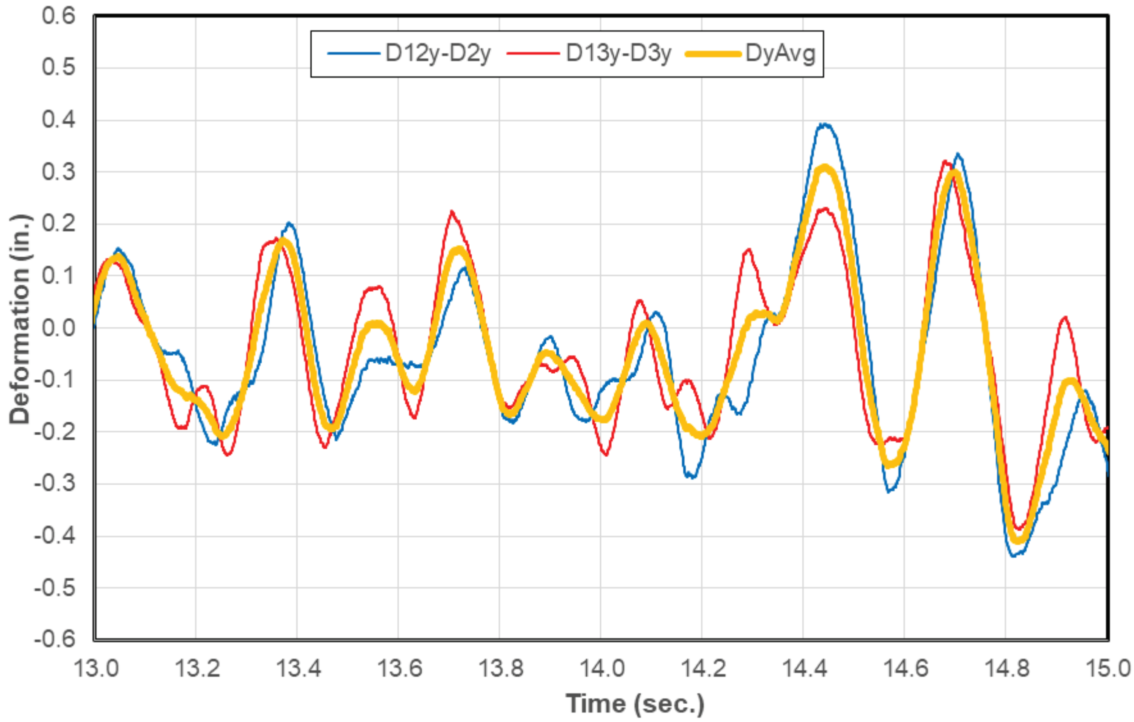


Figure E-17. Deformations in the Y-axis during the 50% seismic test, 13 to 15 seconds.



Appendix F: Data Plots for 100% Seismic Tests

Figures F-1 through F-3 plot all the accelerations measured in the X-axis, Y-axis, and Z-axis of the building for the 100% seismic test. These figures show high frequency spikes. For example, in Figure F-1, the 17 g spike at 23 seconds, measured at accelerometer A6x, has a duration of 0.005 seconds, equivalent to 100 Hz half sine pulse, most likely caused by impact. These spikes reach three times the amplitudes seen in the 50% seismic tests. As presented earlier, these spikes are localized, do not engage large portions of mass; averaging the measured accelerations will preserve some of the real effects of the impact, while reducing the magnitude because the impact take place at different instances in time. The average accelerations plotted in Figures F-4 through F-7 provide a reasonable estimate of the inertial loading when multiplied by the lumped masses at the top and bottom of the building. The inertial loads and base shear plotted in Figures F-8 through F-11, scale up well from the loading seen in the 50% test, where the amplitudes of the high frequency spike portions of these plots are approximately twice the values seen in the 50% test. The number of cycles seen in the average acceleration and inertial loading decreased in the X-axis to 6 Hz. In the short axis of the building (Y-axis), this frequency, which is based on average ceiling cycling (Figure F-7) appears to have decreased to about 3 Hz. However, the acceleration, inertial loading, and building deformation no longer cycles in a clear regular pattern because of significant damage at the bottom of the frame columns.

Figures F-12 and F-13 plot the displacements in the X-axis and Y-axis; Figures F-14 through F-17 plot the calculated deformations in both the X-axis and Y-axis, plus the average deformation in both directions (Dx_{Avg} and Dy_{Avg}). Figures F-12 and F-14 show that the displacements and deformations in the X-axis grow throughout the 100% test. In contrast, in the Y-axis, the displacements start out large, but actually decrease in this test. The building was considerably damaged at the beginning of the 100% test and displaced and deformed considerably in the Y-axis along the bottom of all the corner columns, but the interior columns held together until later in the tests. Once all the interior columns had also failed in the Y-axis, it appears that the entire building, just above the footings, was free to displace and deform just above the footings.

Figure F-1. Accelerations recorded in the X-axis during the 100% seismic test.

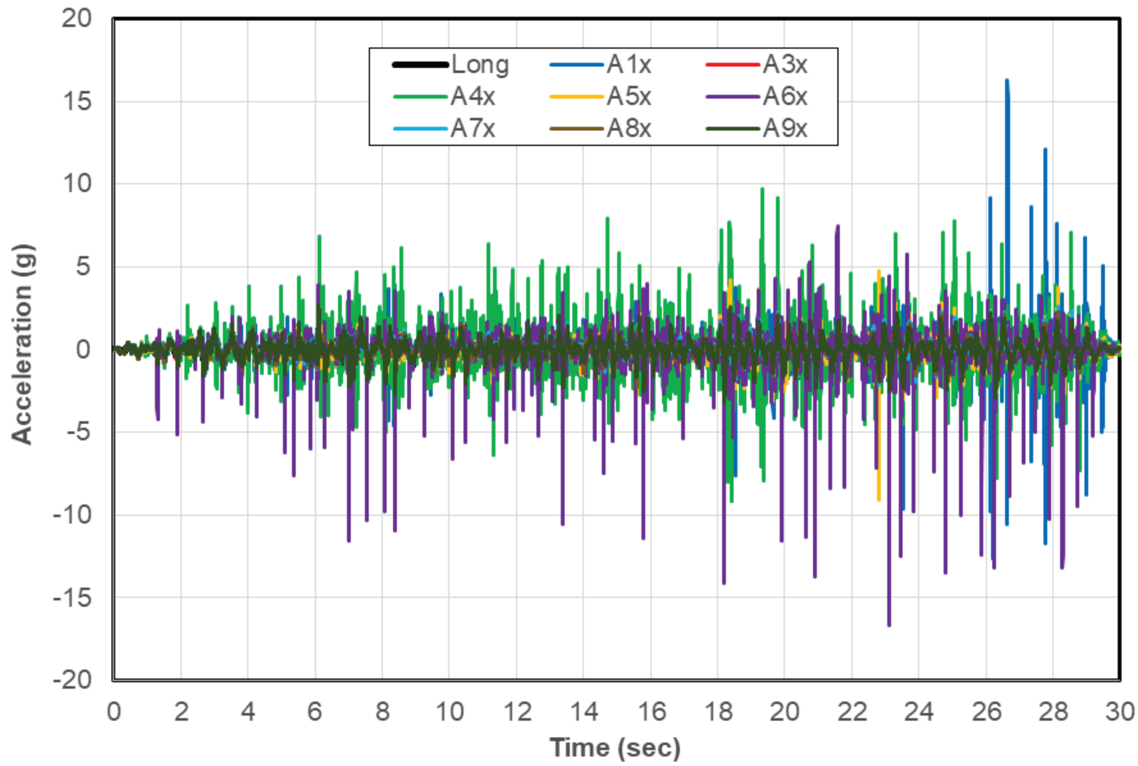


Figure F-2. Accelerations recorded in the Y-axis during the 100% seismic test.

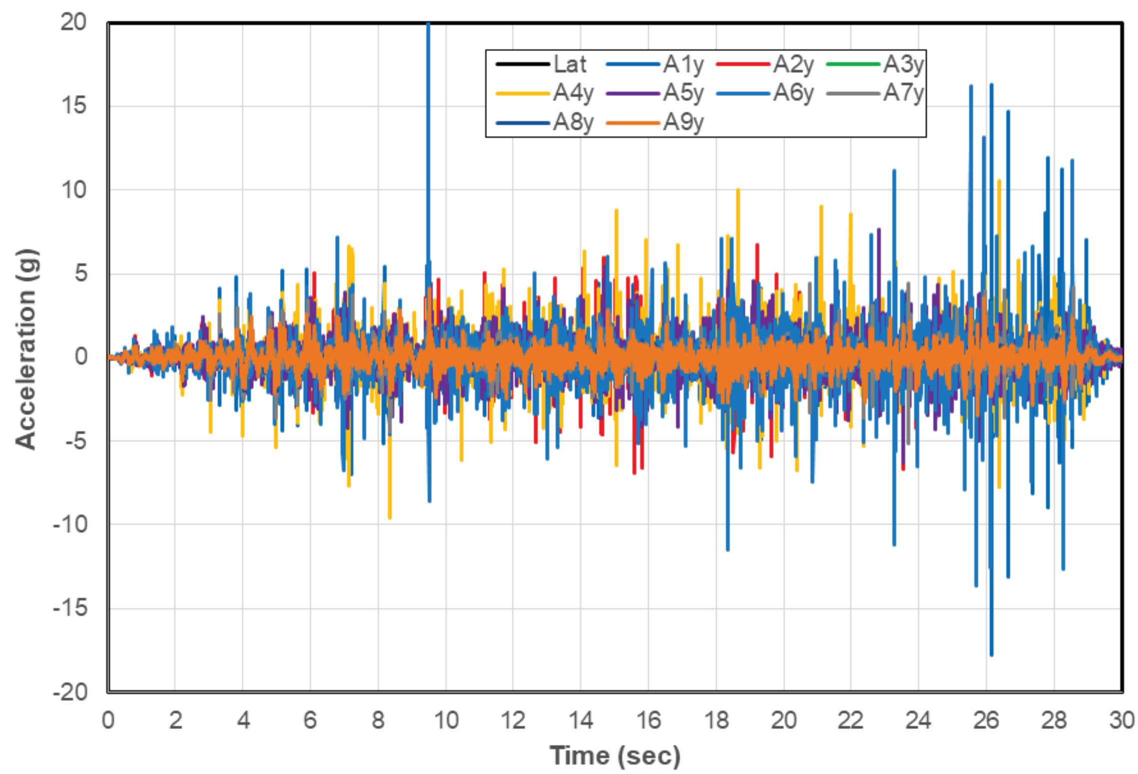


Figure F-3. Accelerations recorded in the Z-axis during the 100% seismic test.

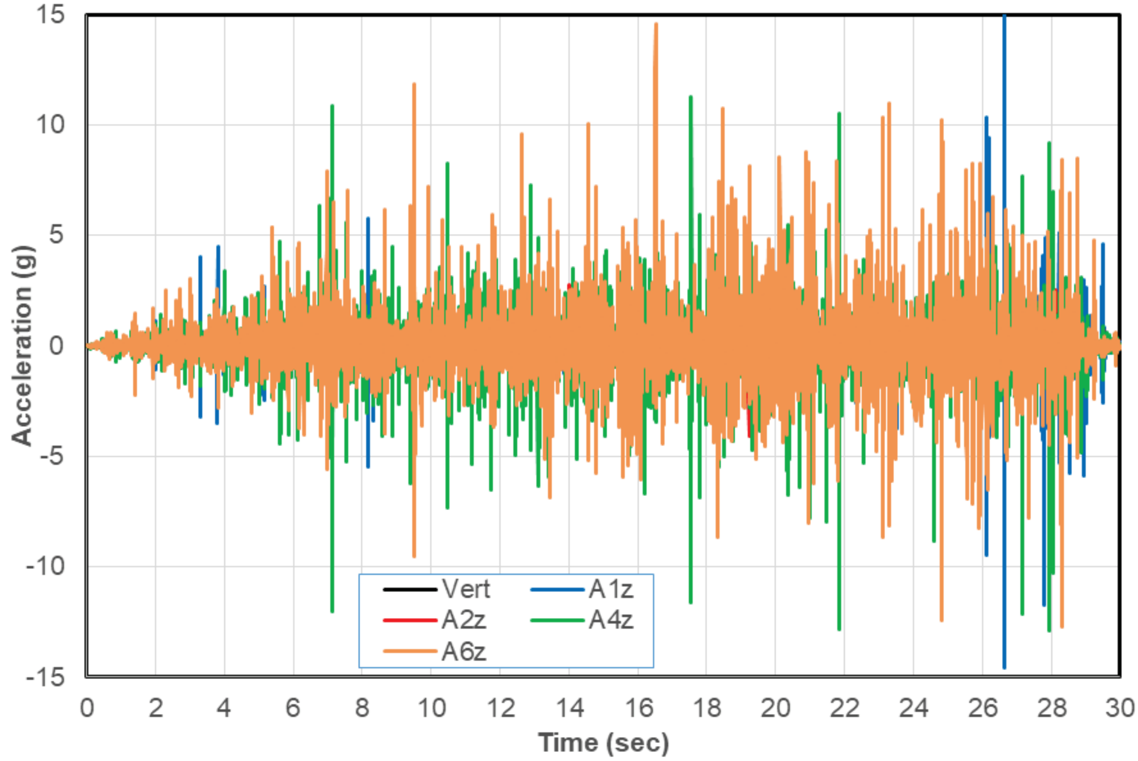


Figure F-4. Accelerations at bottom of walls in X-axis, 100% seismic test, 13 to 15 sec.

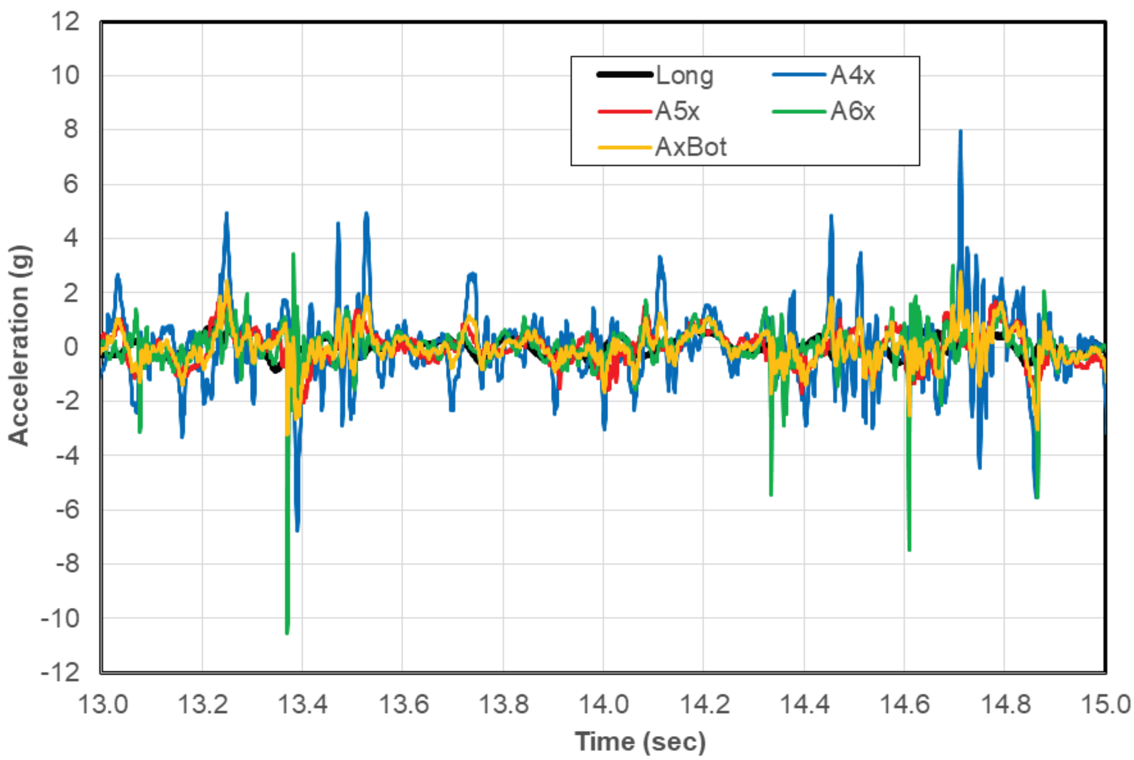


Figure F-5. Accelerations at the ceiling in the X-axis, 100% seismic test.

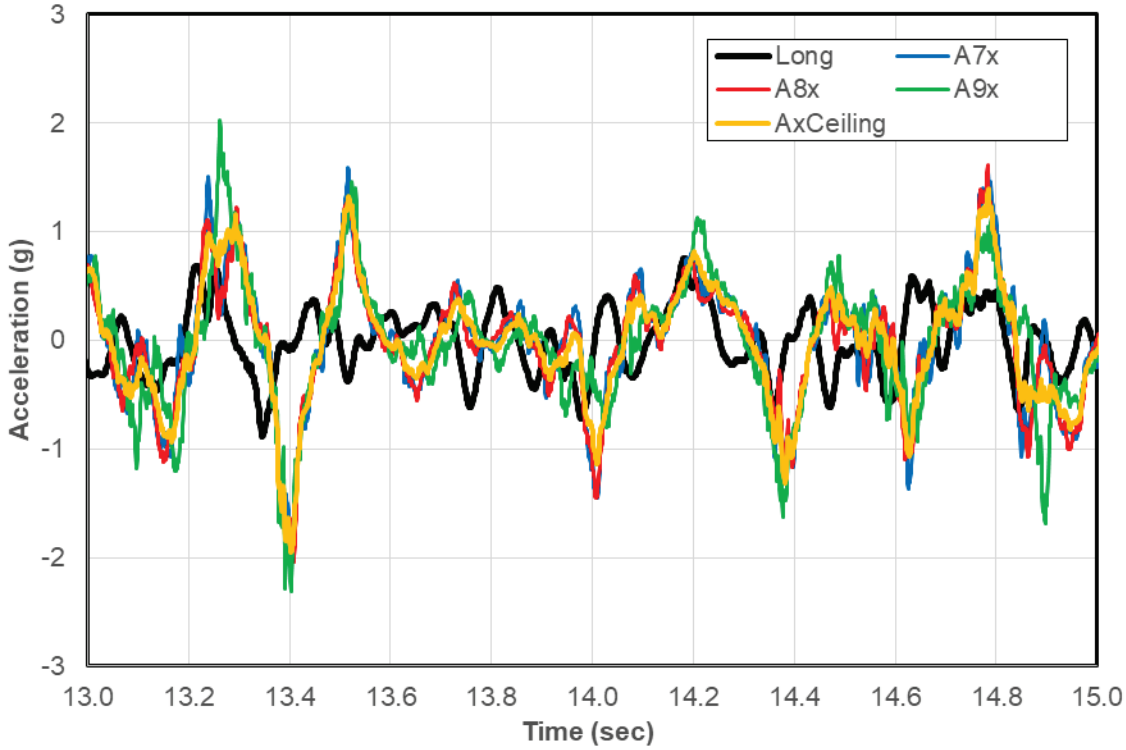


Figure F-6. Accelerations at the bottom of the walls in the Y-axis, 100% seismic test.

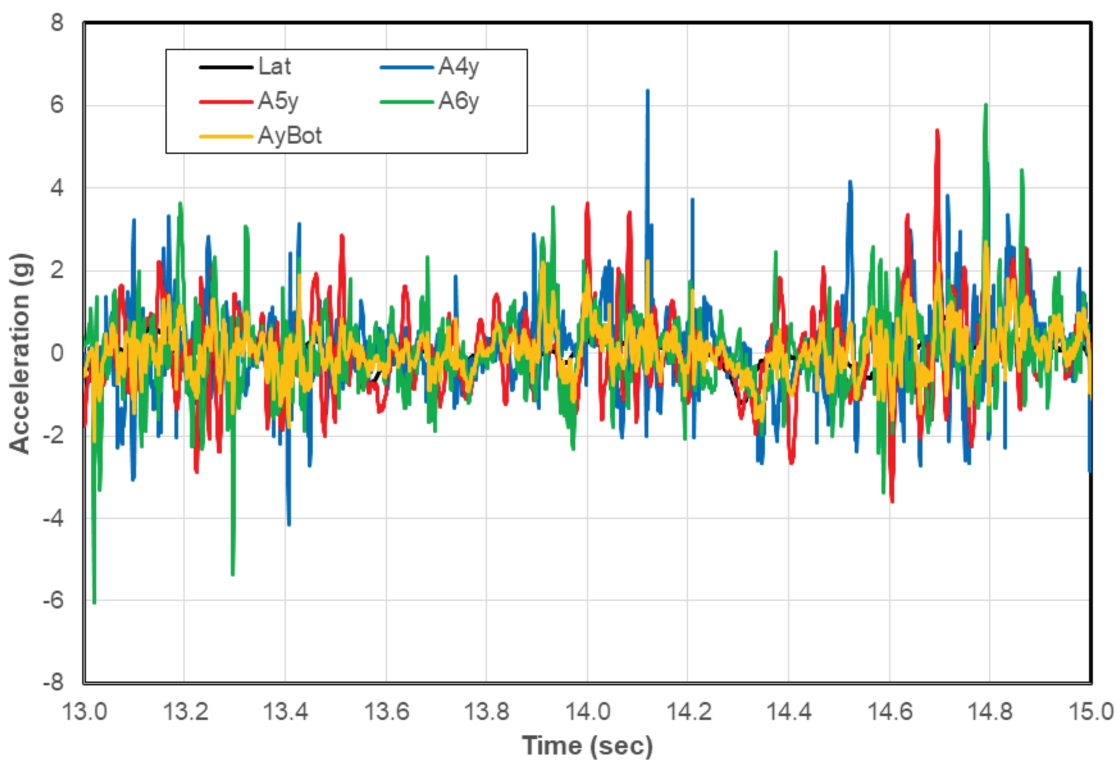


Figure F-7. Accelerations at the ceiling in the Y-axis, 100% seismic test.

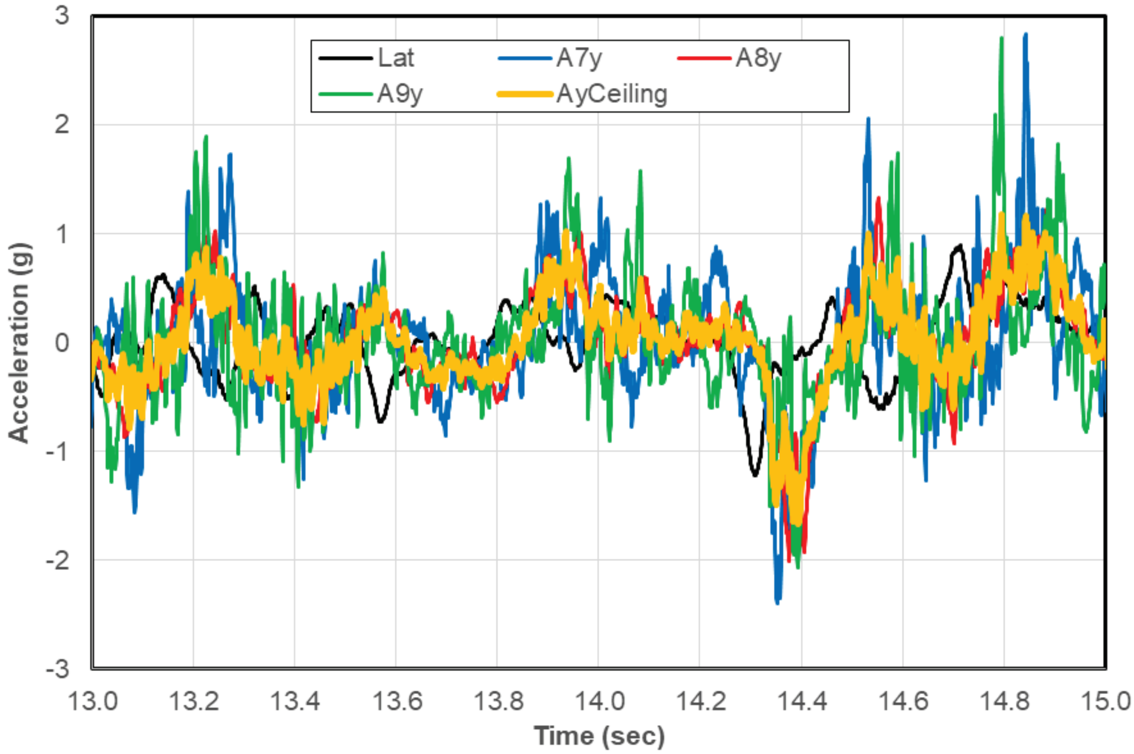


Figure F-8. Inertial forces for bottom, ceiling & base shear, X-axis, 100% seismic test.

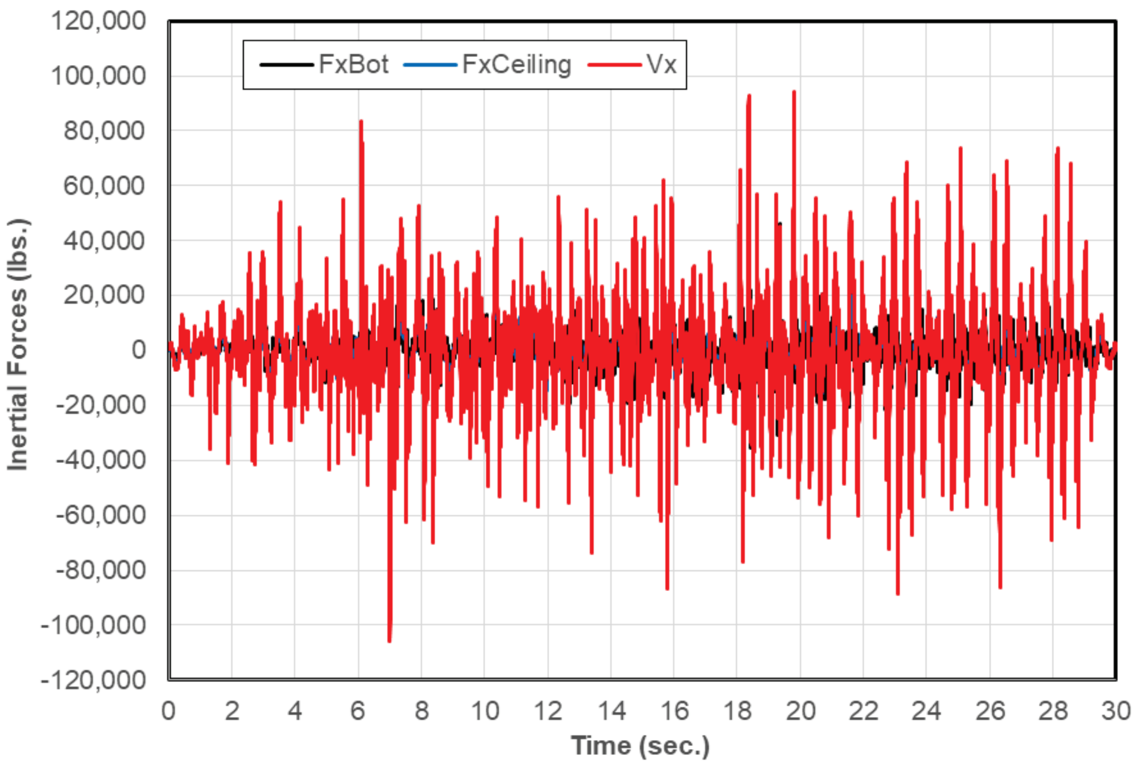


Figure F-9. Bottom & ceiling forces, & base shear, X-axis, 100% test, 13-15 sec.

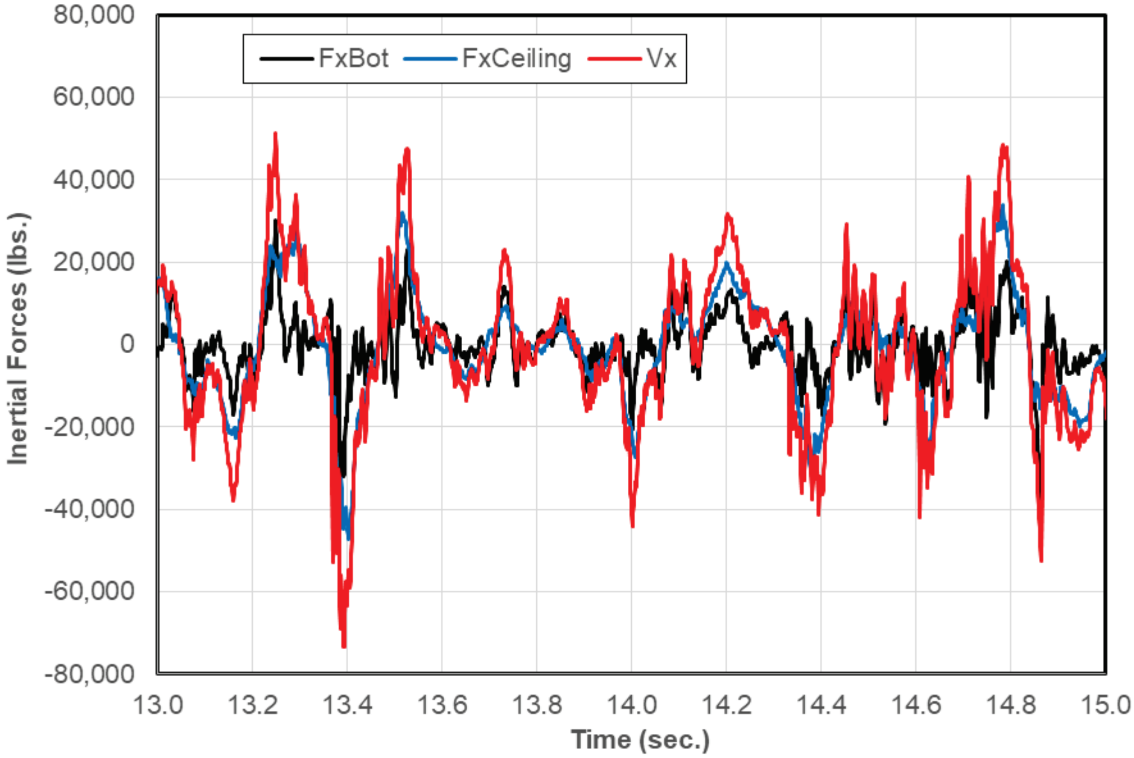


Figure F-10. Inertial forces for bottom, ceiling and base shear, Y-axis, 100% test.

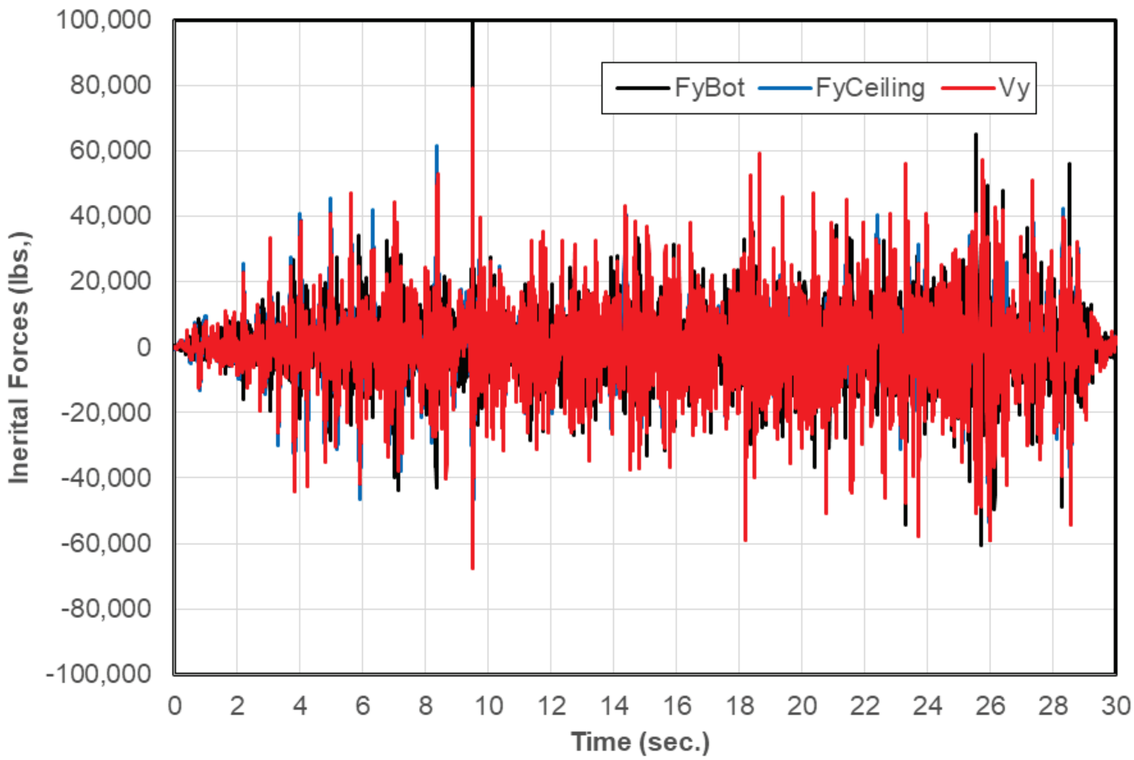


Figure F-11. Bottom & ceiling forces, & base shear, Y-axis, 100% test, 13-15 sec.

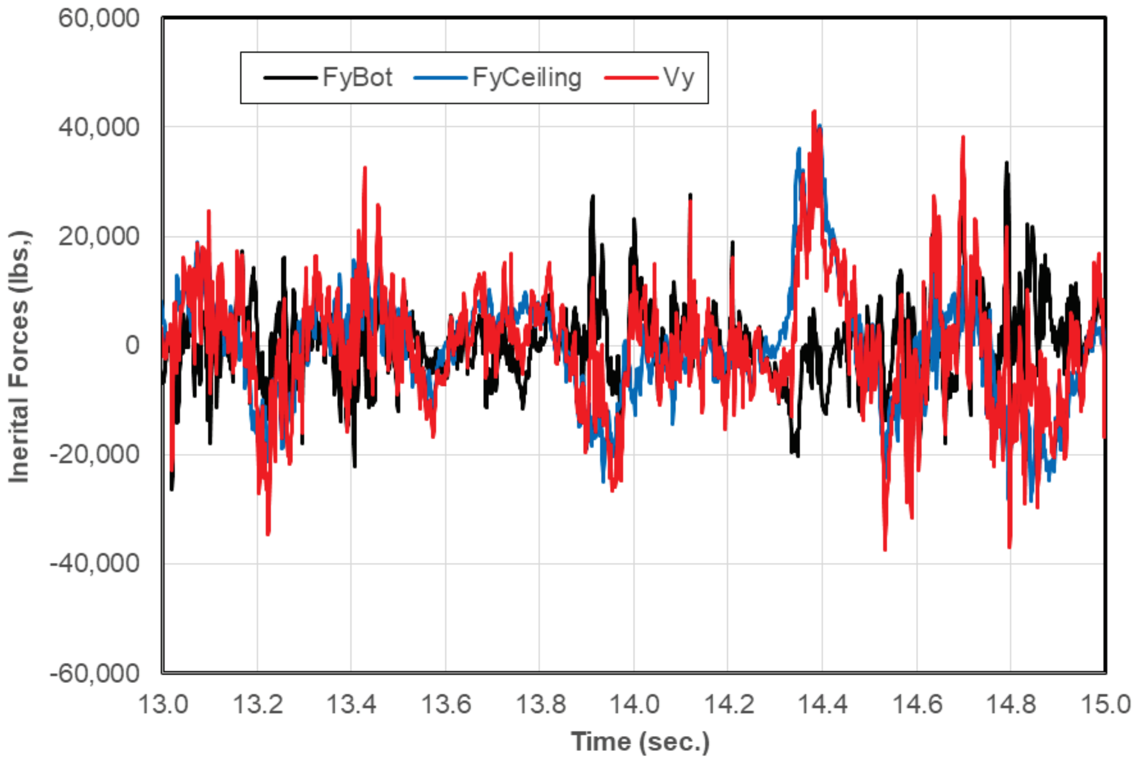


Figure F-12. Displacements recorded in the X-axis during the 100% seismic test.

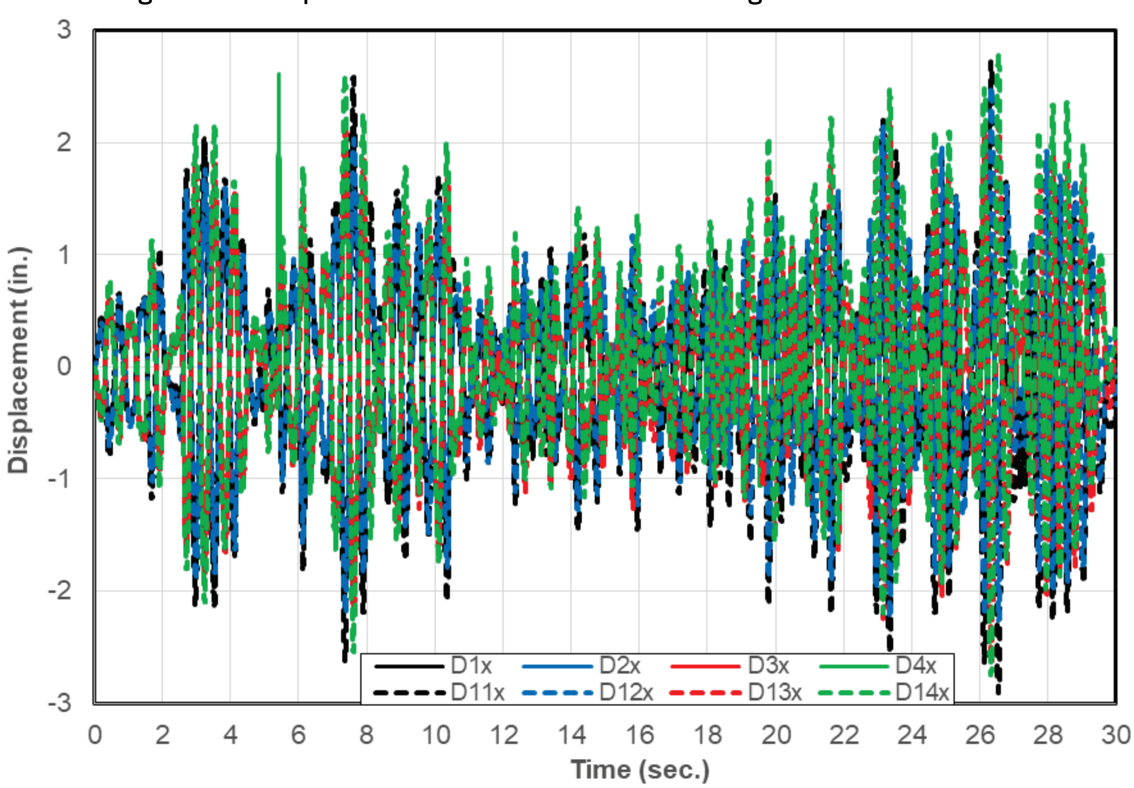


Figure F-13. Displacements recorded in the Y-axis during the 100% seismic test.

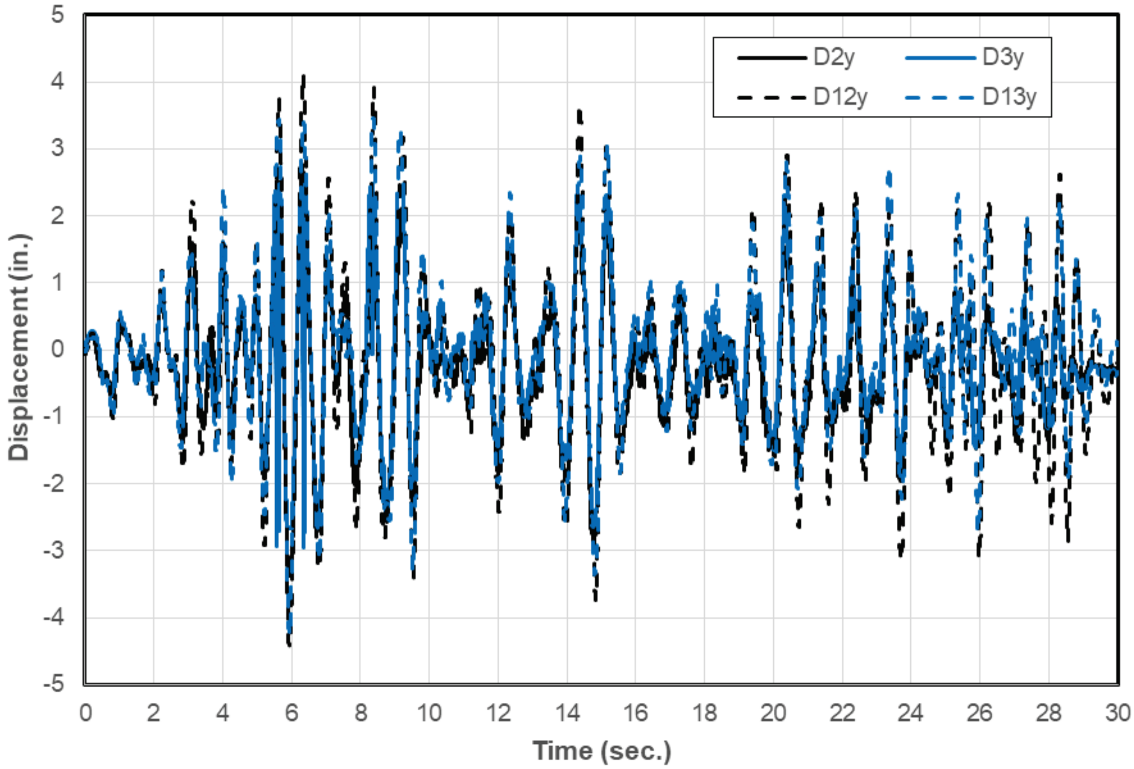


Figure F-14. Deformations in the X-axis during the 100% seismic test.

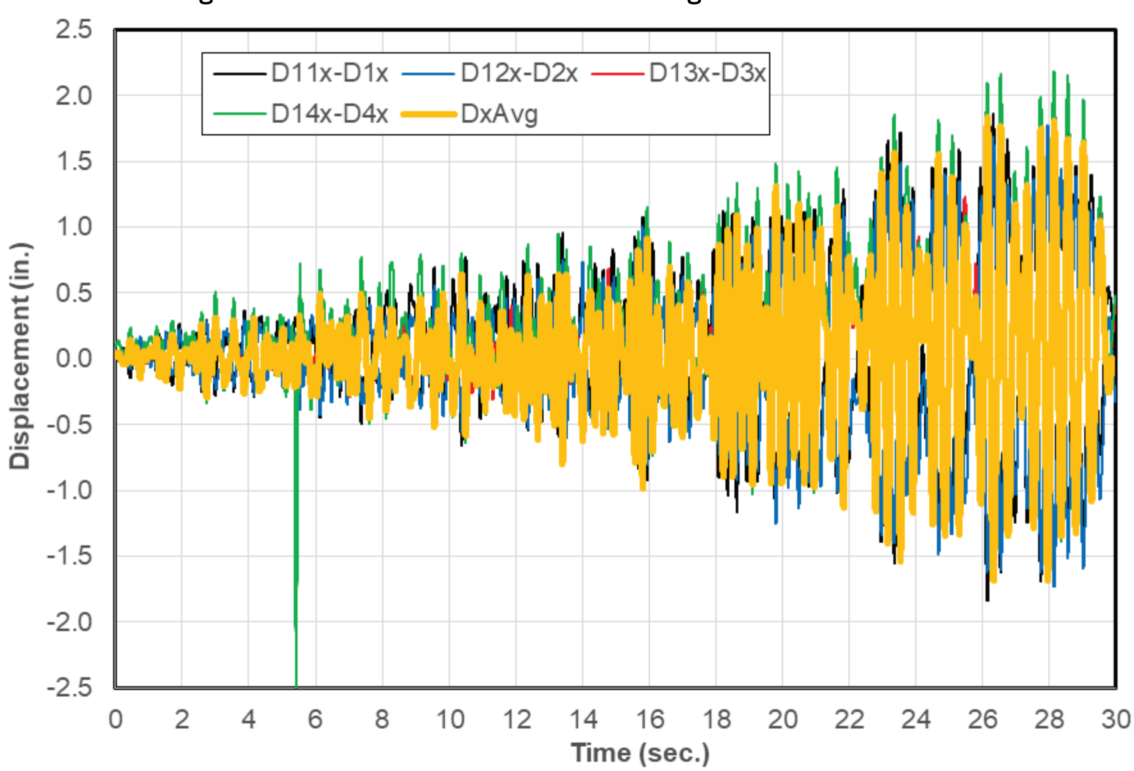


Figure F-15. Deformations in the X-axis during the 100% seismic test, 13 – 15 seconds.

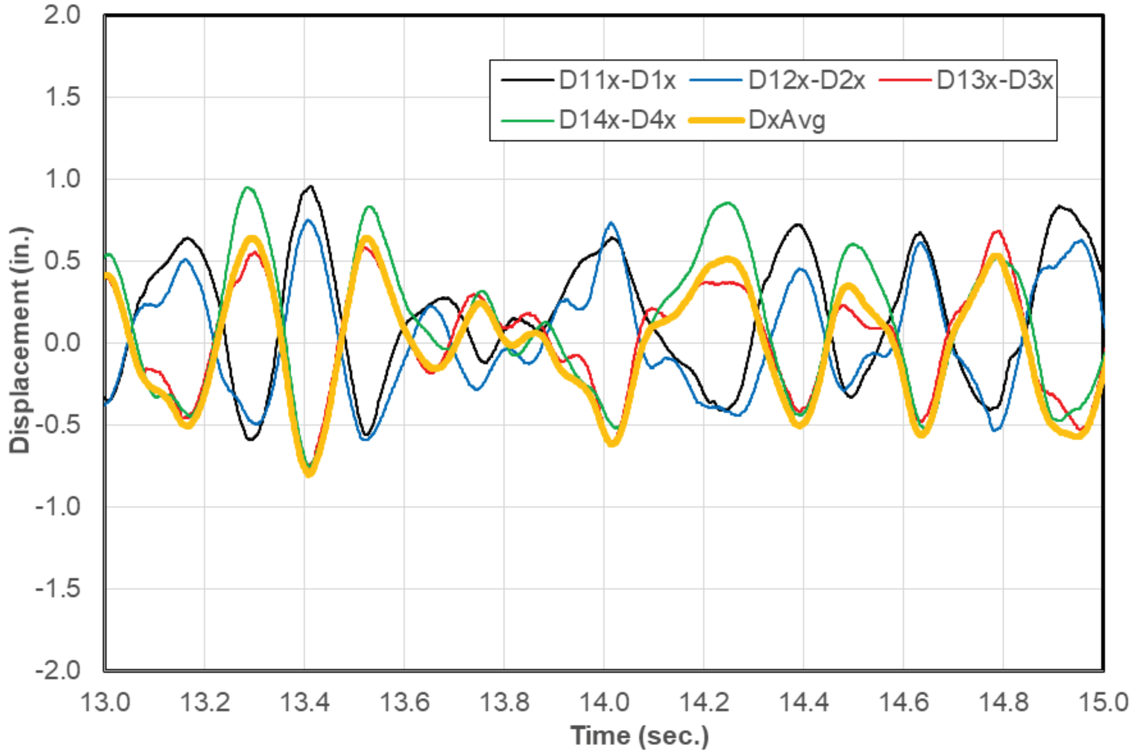


Figure F-16. Deformations in the Y-axis during the 100% seismic test.

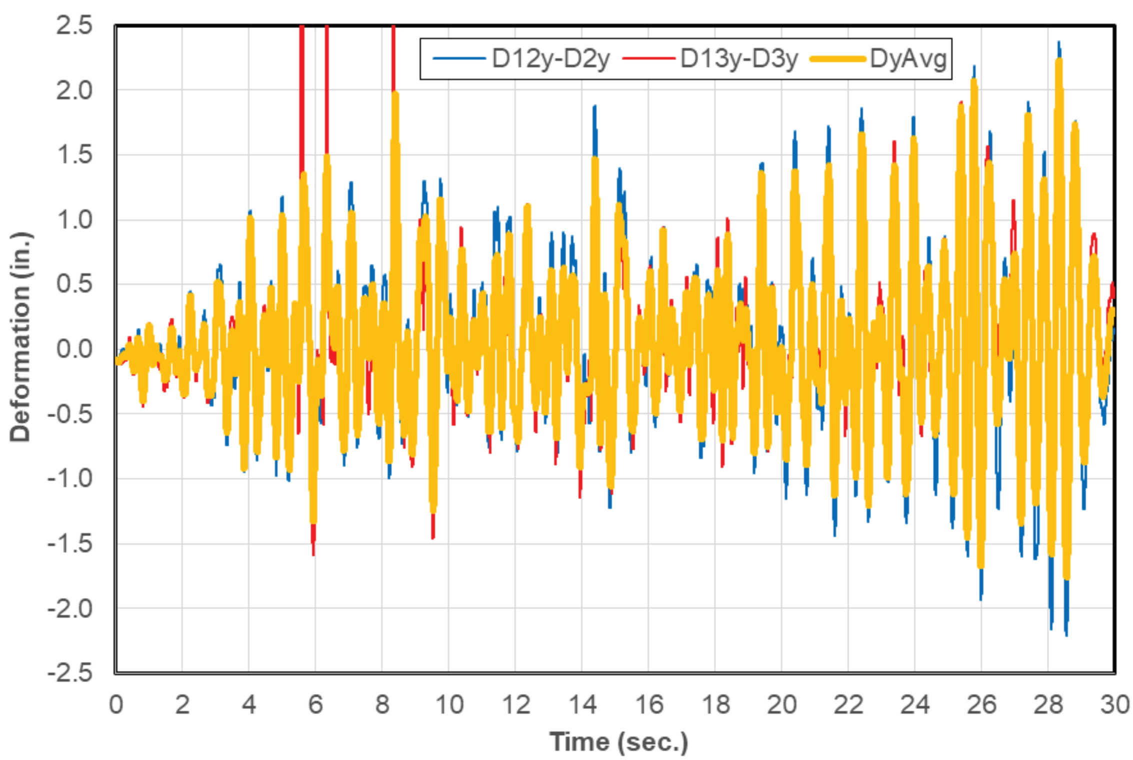
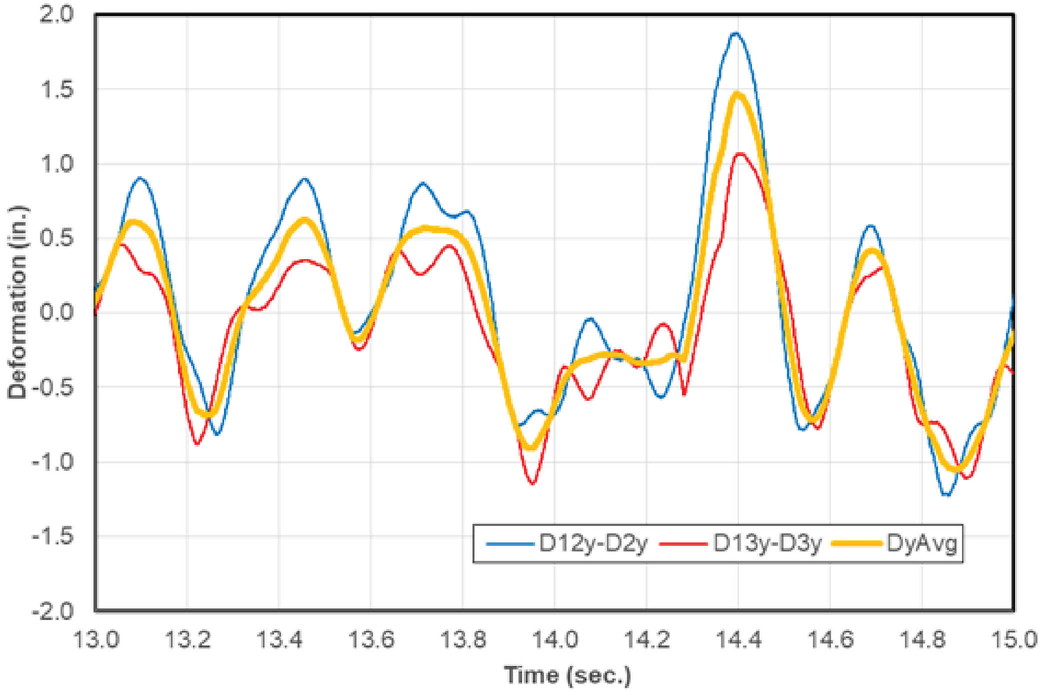


Figure F-17. Deformations in the Y-axis during the 100% seismic test, 13 to 15 seconds.



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14. ABSTRACT
An indigenous materials construction system was developed by a Small Business Innovative Research project – Small Business Innovative Research (SBIR) project Contract W9132T-15-C-0002. The results of that project included the construction of a full scale 16 foot by 32-foot troop constructible building that was tested on the Engineer Research and Development Center, Construction Engineering Research Laboratory (ERDC-CERL) shake table. This report documents the seismic testing of this building. The building consisted of prefabricated frames with interior and exterior wall panels and roof and ceiling panels. The building was tested with 30-second-long synthetic seismic motions, which began at low levels. The test amplitude was increased so that the final test conducted used motions based on a spectral acceleration tied to the highest seismic hazard in the United States. The base of the building was badly damaged in this final test, but it remained stable, demonstrating relatively good behavior. This report documents the measured response to these motions and the performance of the building.

15. SUBJECT TERMS
Building materials, Local building materials, Military bases—Buildings, Earthquake hazard analysis, Troop-constructible buildings, Full scale shake table testing

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