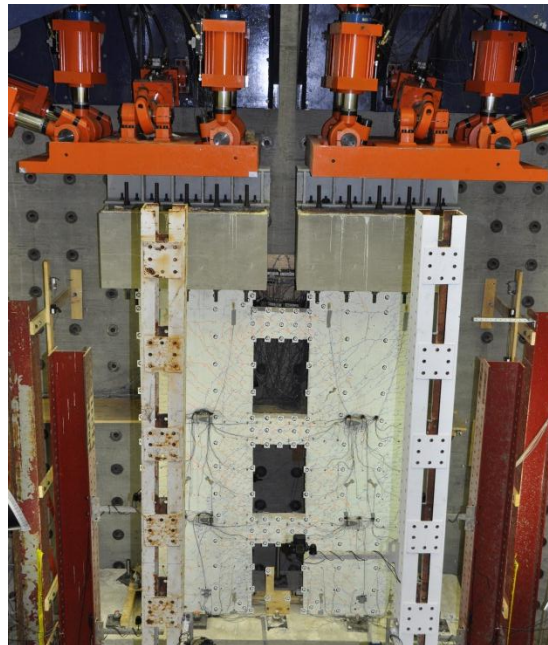


Behavior, Analysis, and Design of Complex Wall Systems

Coupled Wall Test Program Summary Document



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CHAPTER 1: Introduction

This document provides detailed documentation of the design, testing, and preliminary results of the planar wall test program of the project "Behavior, Analysis, and Design of Complex Wall Systems", a joint effort between the University of Washington (PI Laura Lowes and co-PI Dawn Lehman) and the University of Illinois Urbana-Champaign (co-PI Dan Kuchma). Funded by NEES and the Charles Pankow Foundation, the project seeks to establish the seismic performance of modern mid-rise reinforced concrete structural walls and to develop the tools and technologies to advance performance based design of these systems.

In the experimental component of the project, large-scale wall subassemblages were tested to improve the understanding of seismic behavior of walls in modern buildings. Walls were tested at the NEES MUST-SIM Facility at the University of Illinois. Use of this facility allowed collection of high resolution data to assist in characterizing the performance of walls. Equipment at the facility allowed for testing of the subassemblages with force and displacement control in six-degrees of freedom. This allowed for the testing of large-scale (1/3 scale of prototype walls) in that axial load, lateral load, and overturning moment could be applied to the top of the specimens in a manner such that the base reactions were equivalent to that of a much taller wall with a specified lateral load distribution. The actual specimens represented the bottom three stories of a ten story prototype wall. The full experimental test program consisted of 8 specimens: 4 planar (or rectangular) walls, 1 coupled wall, and 3 C-shaped (or U-shaped) walls. This document is limited to the coupled wall test program (CW1); similar documents are available for the planar and C-shaped wall test programs.

The contents of the document are as follows. Chapter 2 provides an overview of the test specimen, including test variables considered, design of the specimen, construction drawings and details on construction of the specimen. Chapter 3 provides material properties data and expected material strengths. Chapter 4 discusses the experimental test set-up, application of loads, and displacement histories. Chapter 5 discusses the data collected including traditional and non-contact instrumentation, high-resolution images, and crack width measurements. Chapter 6 presents the loading and displacement histories applied to the wall specimen. Chapter 7 presents basic experimental results for the tests. Chapter 8 discusses data processing and the availability of the data in the NEES Project Warehouse. The contents of this document are limited to the observed behavior of the test; results of analysis of the experimental data can be found in the dissertations of Turgeon (2011), Birely (2011), Hart (2011), and Marley (2011).

CHAPTER 2: CW1 Specimen

The coupled wall specimen was designed to simulate a “typical” coupled wall in a ten-story building. The objectives were to start the design process by 1) producing a specimen with a nominal shear strength that is determined by the ACI 318-08 minimum horizontal reinforcement ratio, 2) employ a performance-based plastic analysis of the flexural mechanisms in the wall, 3) utilize detailing methods that will reduce the congestion and simplify construction, 4) design a wall that will behave in a predictable and ductile manner.

2.1 Specimen Geometry and Assumptions

The test specimen was designed utilizing results from a building inventory review. The primary objective of the inventory study was to determine a set of general design assumptions that are representative of current construction and uses these assumptions in the design of the coupled wall test specimen. In addition to the building inventory review, consultations with the Advisory Panel and the need for consistency with the planar wall tests specimens, led to the following design assumptions and specifications:

- Concrete compressive strength of 5000 psi
- Reinforcement yield strength of 60 ksi
- Wall pier axial load equal to $0.1A_gf'_c$
- One-third scale wall geometry as shown in Figure 2-1.

The test specimen designed as part of this program was one-third of full scale, in which the full-scale wall had 12 ft. story heights, 30 ft. length, 12 ft. long wall piers and 6 ft. long by 3 ft. deep coupling beams with an aspect ratio (i.e. the span-to-depth ratio) of 2.0. This coupling beam aspect ratio (AR) was chosen based on an average coupling beam aspect ratio computed from the building inventory (i.e. an average equal to 2.1 with a standard deviation of 0.41).

2.2 Specimen Design

Using the prior assumptions and this geometry, the wall was designed following appropriate design documents, including the 2008 ACI Building Code and the SEAOC Seismic Design Manual, Vol. III, which is now the 2006 IBC Structural/Seismic Design Manual, Volume III. A summary of this design process is as follows:

1. The wall flexural strength was designed to correspond to the shear capacity corresponding to the minimum allowable shear reinforcement, V_n per ACI 318-08. As a first step in the process, this shear capacity, V_n , was calculated.
2. The base shear corresponding to this strength was determined to solve the following inequality, $\phi V_n \geq V_u$ where, $\phi = 0.6$.
3. The base moment demand, M_u , was calculated using this shear demand and an equivalent lateral load distribution according to ASCE 7-05.
4. Using these loads, an equivalent elastic analysis was performed to determine the demands on the coupling beams. The coupling beams were then designed according to ACI 318-08, to meet the flexural and shear demands, including $\phi V_n \geq V_u$ where, $\phi = 0.85$.

5. Additional constraints were applied to the wall design including: 1) meeting a target longitudinal reinforcement ratio in the boundary elements of approximately 3.5% and 2) achieve a degree of coupling less than 55% as recommended by Harries (2001).
6. Finally, the longitudinal reinforcement in the wall was designed to satisfy the ACI 318-08 (i.e. including $\phi M_n \geq M_u$ where, $\phi = 0.90$).

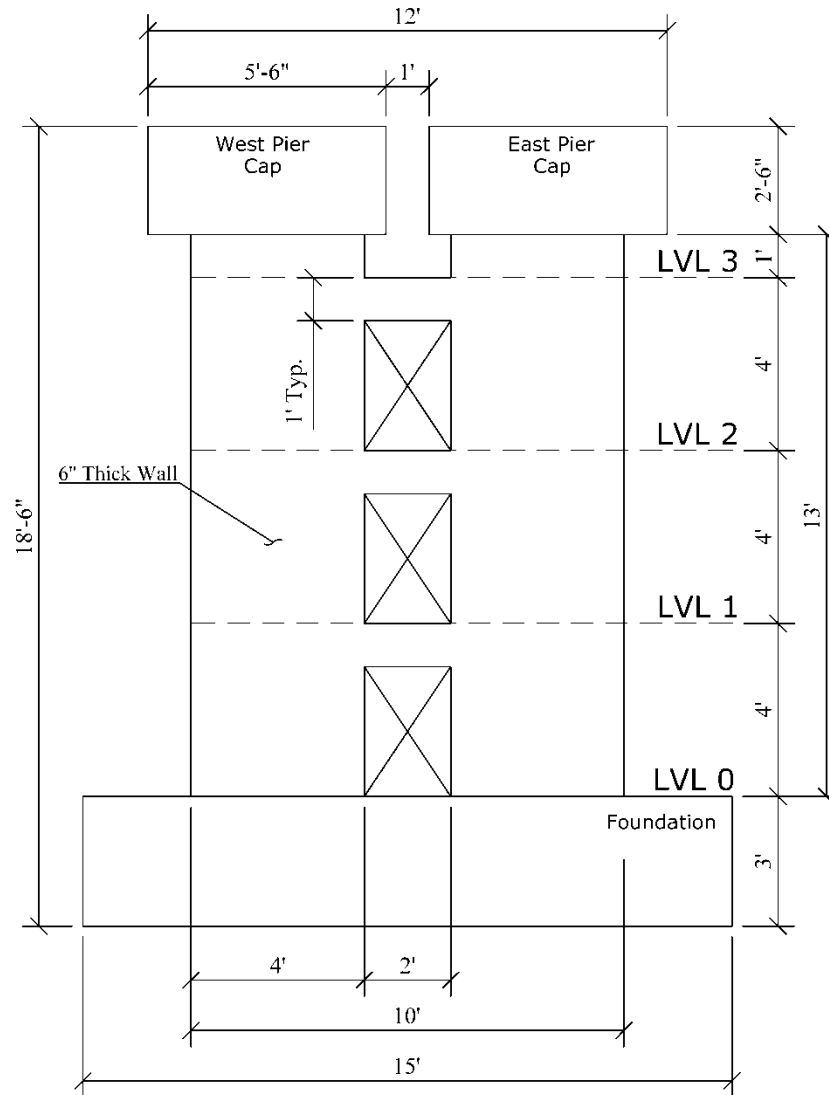


Figure 2-1: Elevation of coupled wall specimen CW1

This design process meets the current ACI and IBC building code requirements. However, it is solely based on elastic response with a prescribed loading pattern. To account for full plasticity in the coupled wall, the design was extended using a capacity-based design approach outlined in 2006 IBC Structural/Seismic Design Manual.

1. Initially, a plastic mechanism was assumed for the wall, which included plastic hinges developing at the base of the wall piers and at the ends of each coupling beam (total of ten coupling beams for ten-story building).
2. For the wall pier and coupling beams, the plastic moment strength, M_{pr} , at each plastic hinge location, as defined by ACI 318-08.
3. Using the principle of virtual work, the internal associated with development of the plastic mechanism was equated with the external work associated with the application of the lateral forces to determine the shear force demand at the base of the wall, V_p .
4. The wall shear reinforcement was designed to sustain this plastic shear demand, using $\phi = 0.75$, as appropriate for a plastic analysis (ACI 318-08). This design process resulting in a spacing of the horizontal reinforcement in the web that was half of the spacing resulting from the ACI procedure outlined above.

2.2.1 Preliminary Design

Mohr (2007) conducted a preliminary design of the coupled wall specimen using similar design assumptions and procedures presented above. One of his research objectives was to evaluate whether the assumed plastic mechanism will actually occur. As a result, nonlinear continuum analyses were conducted to simulate the plastic mechanism in the coupled wall and compare those results to the design assumptions. The author created a finite element model of the one-third scale ten-story coupled wall specimen using the nonlinear continuum software, VecTor2. As a part of this study, the coupled wall specimen was modeled with the following three coupling beam reinforcement variations:

1. The coupling beams were designed to meet ACI 318-05, which required that each group of diagonal reinforcing bars be confined using stirrups.
2. The coupling beams were designed to meet ACI 318-08, which allows for confinement of the entire beam section rather than around each group of diagonal bars. This coupling beam contained the same diagonal reinforcement as the previous design, thus the same design strength.
3. The coupling beams were designed to have a shear strength equal to 25% of the shear strength of the second coupling beam design (i.e. the coupling beam diagonal reinforcement was reduced by 75%).

The last model variation was chosen because results of the first two studies showed the assumed plastic mechanism did not fully develop in the coupled wall. Simulation of the first two models resulted in yielding of the wall piers and concrete crushing in the toe of the compression pier before any of the coupling beams reached yield. Therefore, to reduce the strength of the coupling beams relative to the wall piers, the area of diagonal reinforcement in one group was reduced from 0.80 in^2 to 0.20 in^2 , so the coupling beams would yield when the wall piers reached the design level base shear, V_n . The VecTor2 results of the third design, led to the formation of the desired plastic mechanism with plastic hinging at the ends of all the coupling beams and hinging at the base of the wall piers.

2.2.2 Final Design

The analyses conducted by more Mohr (2007) prompted the research team to iterate the design of the coupled wall specimen such that the coupling beam strengths were proportioned appropriately relative

to the strength of the wall piers and would result in the desired plastic mechanism. The third coupling beam variation achieved this goal using one-quarter the reinforcement for the diagonal bars. However, in order to meet ACI 318 requirements which call for a minimum of four reinforcing bars in each diagonal group, the research team did not want to reduce the diagonal reinforcement in the coupling beams because it was already at the desired minimum bar size (No. 4). As a result the aspect ratio of the coupling beam was increased from 1.5 to 2.0. This resulted in a coupling beam diagonal reinforcement angle of 19 degrees which is approximately 60% less than the preliminary designed coupling beam (i.e. a reduction in shear strength of approximately 62%). Additionally, the change in coupling beam geometry led to a reduction in the coupling beam shear demands calculated using an elastic analysis, resulting in a code-compliant design which was not the case for the third variation of the VecTor2 model. VecTor2 analyses were performed on the new design and the desirable plastic mechanism was verified.

2.3 Design Strengths and Drawings (Files available, see Section 0)

The reinforcement detail for the coupled wall specimen is shown in elevation in Figure 2-2. The details of the coupling beam reinforcement layout are shown in Figure 2-3. Two sets of diagonal reinforcement groups angled at 19 degrees from the horizontal contained 4—#4 reinforcing bars ($\rho_d = 1.25\%$). Using (Eq. 1 as defined by ACI 318-08, the nominal shear strength, V_n , of the coupling beams was equal to 31.3 kips ($6.1\sqrt{f'_c}A_{cw}$ psi).

$$V_n = 2A_{vd}f_y \sin(\alpha) \leq 10\sqrt{f'_c}A_{cw} \quad (\text{Eq. 1})$$

The total area of reinforcing steel for one diagonal group is defined as A_{cv} , and the yield strength of said group is f_y . Additionally, α is the diagonal reinforcement angle of inclination with respect to the horizontal axis of the beam. Finally, A_{cw} is the coupling beam area which is bounded by thickness of the coupling beam web and the height of the beam. The diagonal reinforcement was anchored into the wall piers a distance of 22 inches in order to develop 1.25 times the development length for f_y in tension, as required by ACI.

The coupling beam confinement utilizes the ACI 318-08 code detail for full section confinement. Closed vertical stirrups (No. 2) were spaced along the coupling beam length two inches on center ($\rho_v = 1.22\%$), and No. 2 transverse reinforcing ties were spaced vertically 2.5 inches on center. Finally, horizontal reinforcement (No. 2) was spaced vertically 2.5 inches on center. In order to not develop f_y in these horizontal bars, as required by ACI, the horizontal reinforcing bars were anchored two inches ($8d_b$) in the wall pier on each end of the coupling beam.

While the coupling beam design changed in the second iteration of design, wall piers contained the same reinforcement ratios and geometry that was specified in the preliminary design. The reinforcing detail for the cross-section of the wall piers is shown in Figure 2-4. Each wall pier boundary element contained 16—#4 vertical reinforcing bars spaced horizontally at two inches on center ($\rho_{vert} = 3.8\%$). The No. 2 confinement ties and hoops in the boundary elements were spaced vertically two inches on center, and the horizontal and transverse reinforcement ratio was 0.82% and 0.70%, respectively.

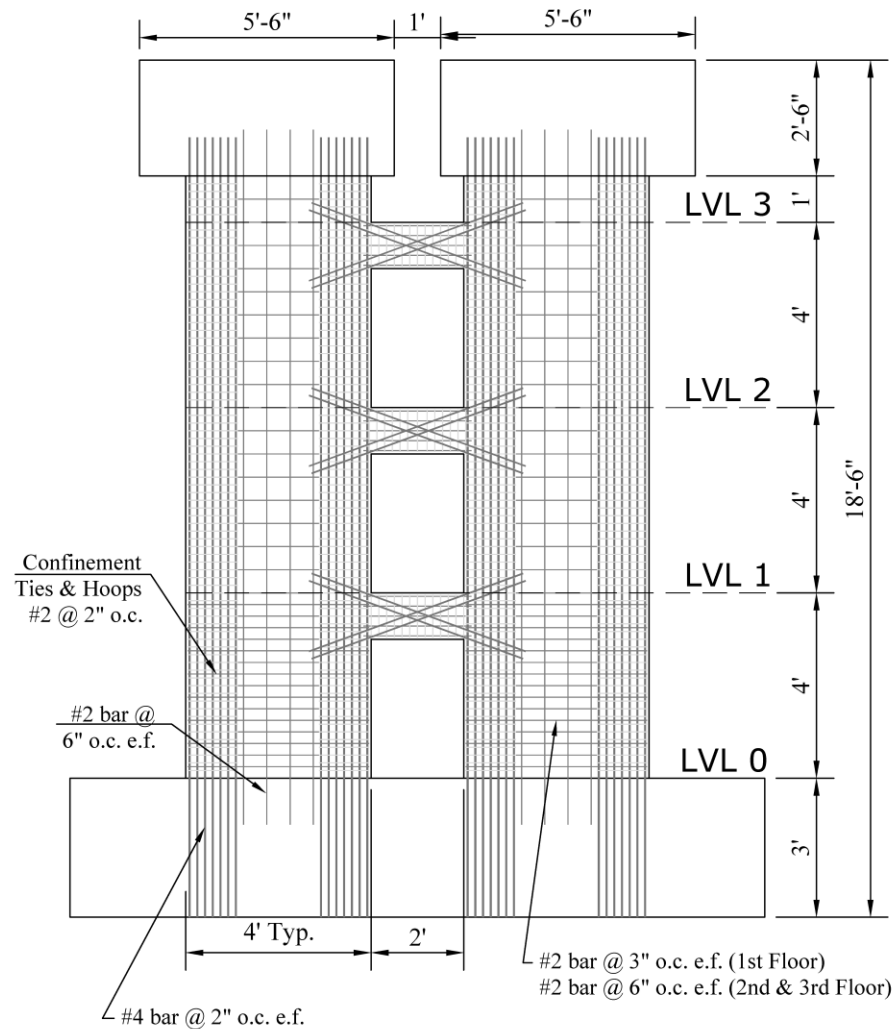
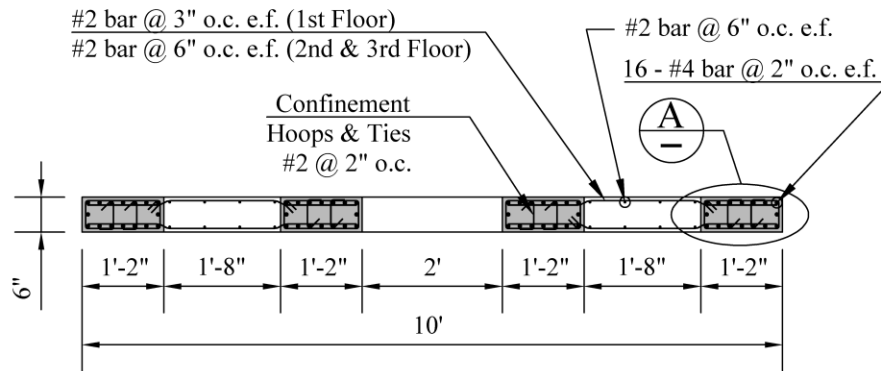


Figure 2-2: Specimen CW1 reinforcement layout (elevation view)

In the web region of the wall piers, vertical reinforcement (No. 2) was spaced at six inches on center corresponding to a reinforcement ratio of 0.27%. Recall, the spacing of the horizontal web reinforcement was reduced from six inches in the first floor to two inches. This was a result of the 2006 IBC Structural/Seismic Design Manual plastic analysis. The horizontal reinforcement ratio, ρ_t , in the first and upper stories equaled 0.54% and 0.27%, respectively.

where, α_c is a coefficient that defines the relative contribution of concrete strength to the nominal shear strength of the wall ($\alpha_c = 2$, for $H/l_w \geq 4.0$).



Detail A

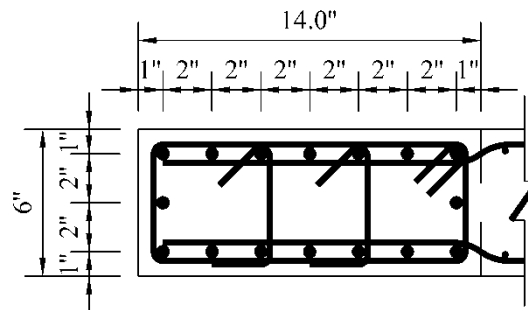


Figure 2-4: Cross-sectional view of wall pier reinforcement detail

2.4 Foundation/Wall Cap Design

Design of the coupled-wall specimen is addressed in the previous sections. To provide a realistic way of testing the coupled wall, the specimen was constructed with a foundation block below the wall and a wall cap above the wall. The foundation and wall pier caps were a means of connecting the specimen to the strong floor and to the LBCBs, respectively. Additionally, these served as means to provide adequate anchorage and development of the longitudinal steel in the wall piers.

2.4.1 Foundation Block

The foundation was a massive concrete block that was used to anchor the entire test specimen to the strong floor and anchor the longitudinal reinforcement of the wall piers. The height of the foundation was 36 inches and was determined based on the necessary development length of the wall pier longitudinal reinforcement. The foundation reinforcement was designed to resist reactions introduced by the wall piers and tension splitting introduced by the post-tensioning system used to anchor the specimens to the reaction floor. Mats of reinforcing steel were provided at the top and bottom of the

foundation to provide steel in the longitudinal and transverse directions. In each mat, #8 bars were spaced at about 6-8 inches. Vertical hooked stirrups (#3 bars) were evenly distributed at 12 inches throughout the foundation in both directions.

PVC tubes were provided to provide ducts for post-tensioning rods used to secure the specimen to the strong floor. Two rows of 5 PVC pipes were provided at 36 inches to align with the holes in the strong floor. The PVC tubes were 3 inch diameter and were supported by #3 spirals (10 inch diameter with 4 inch pitch).

The wall specimen was located off center from the center of foundation block to allow the specimen to be centered beneath the loading units. Figure 2-5 shows a typical elevation view of the rebar cage.

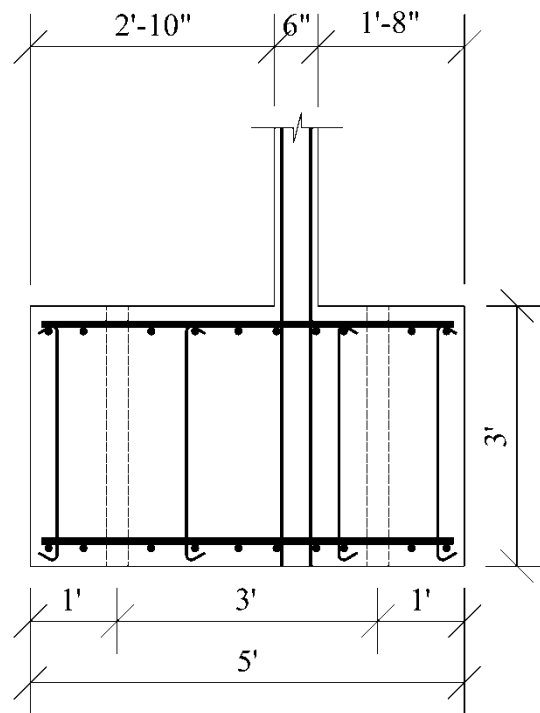


Figure 2-5: Typical side view elevation of foundation block.

2.4.2 Wall Pier Caps

The wall pier caps provided a means of connecting the LBCBs to the specimen while providing a buffer region against the development of stress concentrations.

The cap was detailed similar to the foundation. There were two mats of reinforcing steel with vertical stirrups. Smaller PVC tubes were used as anchoring ducts, approximately 2 inch diameter. The cap was symmetric and centered about the wall.

2.5 Specimen Construction

The coupled wall specimen was constructed at the University of Illinois NEES MUST-SIM facility by the UIUC project team and lab technicians. A reinforced concrete foundation block and two reinforced

concrete pier caps were designed and constructed in order to connect the specimen to the laboratory strong floor and the two LBCBs. These components were designed by the UIUC project team to ensure appropriate dimensions and strength.

The foundation block, wall specimen and pier caps were all casted separately using highly fluid self-consolidating concrete (SCC). Unlike conventional concrete which needs to be manually vibrated around tight reinforcing cages, SCC is a highly fluid, workable concrete. This was ideal for the concrete casting of the one-third scale specimen. A brief summary of the construction sequence is shown below, and more detail regarding the construction of Specimen CW1 is reported in the sections thereafter.

1. Tied reinforcing cage for foundation block and included the vertical reinforcement for the wall piers
2. Assembled foundation block formwork and casted concrete
3. Reinforcement hoops were placed over the boundary element vertical steel to provide horizontal confinement to the boundaries, transverse ties and horizontal reinforcement bars were then tied to this cage
4. Diagonal reinforcement was thread into the wall piers at each floor level and the respective horizontal and vertical coupling beam reinforcement was tied into place
5. Constructed specimen formwork and casted concrete
6. Tied reinforcement for each wall pier cap, applied formwork and casted concrete

2.5.1 Foundation Construction

The reinforcement cage in the foundation block was placed in the wood constructed formwork, along with the PVC pipes that provided ducts for the post-tension connection to the strong floor (vertical tubes) and movement of the wall in the laboratory (horizontal tubes). The longitudinal reinforcing bars that originated in the foundation were placed in the appropriate locations and the tops of the bars were secured to temporary shoring. The wires for the strain gauges on these bars were bundled and routed away from the specimen.

The foundation block was cast using a self-consolidating concrete mix (SCC); the design mix and material properties of this concrete are provided in Section 3.1. Two trucks of concrete were needed to cast the foundation block. The concrete was directly cast into the formwork from a shoot, with an inverted cone diameter ranging from 16-22 inches. After initial set of the concrete, the surface was scoured with 0.5 inch grooves at 2 inch spacing; this provided a mechanical shear key to aid in shear transfer at the wall-foundation interface.

2.5.2 Wall Specimen Construction

The coupled wall was constructed after the foundation had been cast. Engineered steel forms with multiple ties were used to accommodate the large hydrostatic forces expected from casting self-consolidating concrete (SCC) over a height of 12 feet. Steel is a much more robust than a traditional wood formwork system and it would not deteriorate over multiple uses. Most importantly, a steel system could handle the design formwork pressures. The research group decided to purchase a formwork system from EFCO Forms. The system utilized heavy duty plate-girder® formwork. Heavy duty

formwork ties and anchorage bolts were used to stabilize and hold the forms together during casting. The backside of the formwork was erected first to provide a vertical guide for construction.

To begin construction of the wall, the backside of the formwork was erected to help provide a vertical guide for construction (. The cage was erected from the bottom up in layers. Typically, the reinforcing hoops located in the boundary elements of the wall piers were placed over the top of the longitudinal steel and then tied into position. Two transverse reinforcing ties were then added to the third and fifth vertical reinforcing bar (i.e. 5 and 9 inches from the B.E. edge) at each vertical hoop spacing to provide adequate confinement of the boundary element. Wood blocks were used to space the different layers of reinforcement, but the total amount of reinforcement was always accounted for such that the spacing on average always met the design parameters.

Horizontal reinforcing bars that spanned the entire length of the wall piers were then placed vertically at their appropriate spacing (i.e. 3 inches in the first floor and 6 inches elsewhere). At each floor location reinforcement for the coupling beams was placed. For each coupling beam, diagonal reinforcement was thread into the wall piers and the respective horizontal and vertical coupling beam skin reinforcement was tied into place to provide full confinement of the coupling beam cross-section. After the wall specimen reinforcement was completed, strain gauge wires were bundled together and routed through the top of the wall.

The wall specimen portion of CW1 (i.e. wall piers and coupling beams) was cast in one lift. The concrete was brought into the lab in standard concrete trucks and was pumped to the top of the specimen via a concrete pump truck. The wall specimen was cast by pumping the SCC concrete into 3 inch diameter steel tubes located in each wall pier (Figure 2-6). Figure 2-7 shows casting of the specimen on May 13th, 2009.



Figure 2-6: CW1 prior to casting (black steel tube 16 inches in from the left-most reinforcement)



Figure 2-7: Casting of specimen via a concrete pump truck

Manual vibration was never needed but it was utilized infrequently to help consolidation and flow of the concrete into the coupling beams. The coupling beams were by far the most congested rebar cage within the experimental plan. Since the coupling beams were vital to the study of the coupled wall system, it was critical that concrete flowed into the beams and there was not honeycombing or segregation. The research group decided that a little segregation was better than a large honeycomb within the coupling beam, and for this reason it was chosen to use a large rubber mallet on the side of the forms to help force the concrete into the beams, and flow around reinforcement. It was noted during the casting of the coupled wall, the top surface of the third story coupling beam developed a slight air pocket despite the manual vibration. It was determined that the top coupling beam did not have enough hydrostatic pressure to force out the air bubble. However, the air bubble was still small and only affected the cover of the coupling beam. It was patched with a low shrinkage concrete.

2.5.3 Cap Construction

After the wall portion of the specimens was set, the steel forms were removed and shoring was erected to support the pier cap formwork. The formwork was built using EFCO steel formwork similar wall specimen. The pier cap reinforcement was placed into the forms and strain gauge wires were routed

out of the formwork so as not to interfere with the loading unit connections. Casting of the wall cap was achieved by feeding a 1/3-cubic yard concrete bucket and lifting it over the specimen with a crane (Figure 2-8).



Figure 2-8: Casting a typical pier cap

CHAPTER 3: Material Properties

The coupled wall specimen was designed using concrete compressive strength of 5000 psi and a steel yield stress of 60 ksi. The mix design and actual material properties of the constructed specimens are provided in Section 3.1. The measured material properties of the reinforcing steel are provided in Section 0.

3.1 Concrete Material Properties

The concrete used to construct the coupled wall specimen was a highly fluid self-consolidating concrete (SCC). A highly fluid mix was desired to help flow and consolidate around the tight rebar cage. The target design strength was 5000 psi. To accommodate clear cover and clear rebar spacing dimensions as small as 0.5 inches, the maximum top aggregate was limited to 3/8 inch. It was difficult to obtain a low strength of 5000 psi because more cement, flyash, and fines were added to replace typical coarse aggregate. To help keep strength low, the water to cement ratio (w/c) was increased to 0.50. The super plasticizer was a high-range water-reducing admixture, based on polycarboxylate chemistry. Table 3-1 provides the concrete mix design.

Table 3-1: Concrete wall mix design.

Constituent	Amount per Cubic Yard
Coarse Aggregate (3/8" Chips)	1340 lb
Fine Aggregate (Sand)	1383 lb
Cement	450 lb
Fly ash	150 lb
Water	36.7 gal (w/c = 0.49)
Air-entraining agent	0.5 oz

A local ready-mix concrete supplier helped batch and test the concrete mix. After adjusting mix properties to hone in on the target design strength, the flowability of the mix was investigated. Mock rebar cages, shown in Figure 3-1, of the most congested area of steel were built to assess the flowability of the mix. Three separate concrete trucks with two cubic yards of concrete were shipped to the lab and, for each batch, superplasticizer was added until a target inverted cone ring diameter of 20, 24, or 28 inches was reached. Figure 3-2 shows the inverted cone test for a batch test. It was determined that a large ring diameter around 28 inches was needed for the concrete to naturally flow through the tight rebar cage without the help of hydrostatic pressure or manual vibration. A ring diameter larger than 28 inches was susceptible to segregation. This was noted by slight ponding within the middle of the inverted cone test.



Figure 3-1: Boundary element mock-up.



Figure 3-2: Self-consolidating concrete inverted cone test.

For the foundation and wall cap concrete batches, 4 inch by 8 inch, or 4x8, cylinders were cast. For the wall concrete batches, both 4x8 and 6 inch by 12 inch, or 6x12, cylinders, were cast. Additionally,

modulus of rupture (MOR) beams was cast with every wall pour. Cylinders were cast in the same environment as the specimen with wet burlap and covered with a plastic tarp. All compressive cylinder tests were conducted in a Forney testing machine according to ASTM specification C39. For PW-1, which has a similar concrete mix design to CW1, 4x8 cylinders were tested on days 1, 7, 14, 21, 28, and test day. A strength development curve can be seen in Figure 3-3.

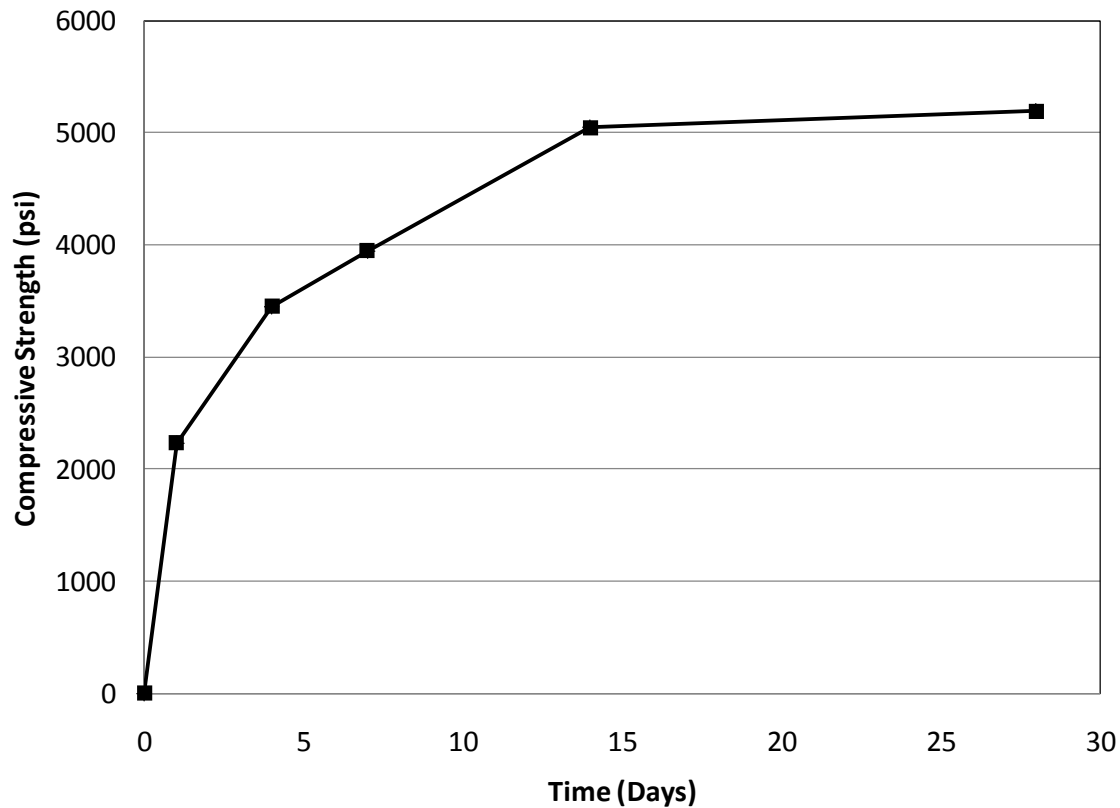


Figure 3-3: Strength development curve for Specimen PW1 (similar concrete design to CW1).

On test day, three 4x8 inch cylinders were tested to obtain an average concrete strength at the time of testing, denoted in Figure 3-3. Table 3-2 as f'_c , not 28-day strength. These were the only material tests performed on the day of testing. Table 3-2 provides material properties, where f'_c is the compressive strength on the day of testing, E_c is the modulus of elasticity ($57,000\sqrt{f'_c}$ psi), and f_t is the tensile strength of the concrete as determined as $7.5\sqrt{f'_c}$ (psi).

Table 3-2: Summary of concrete properties.

f'_c	E_c	f_t
(psi)	(ksi)	(psi)
5690	4300	566

3.2 Reinforcing Steel Properties

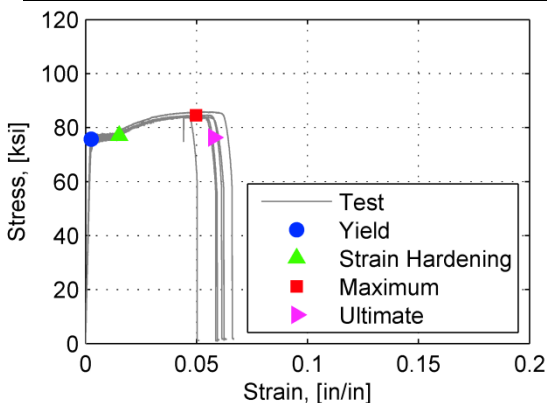
The primary longitudinal reinforcement used to construct each wall was standard No. 4 Grade 60 deformed bars. It was not possible to build all test specimens with the same batch of steel to ensure that all test specimens had the same steel properties. CW1 was built with the same batch of steel and was purchased from a local Illinois steel supplier. This steel was produced under the ASTM specification A615.

Standard tension tests were performed on each batch of steel to determine the as built properties of the steel. Tension tests were conducted in a MTS uniaxial testing frame with hydraulic grips. Calibrated extensometers with 4 inch or 8 inch gauge length were clamped onto the side of each specimen to measure strain in the bars; a 4 inch gauge length was used to measure strain up to the yield point and an 8 inch gauge length was used to measure strain up to failure. Only if the specimen ruptured within the gauge length was the measurement considered valid.

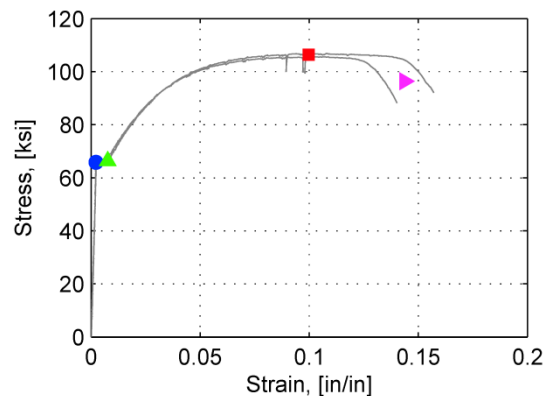
For the No. 4 and No. 2 bars, two and six tension tests were performed, respectively. For each value, the values corresponding to yield, strain hardening, maximum stress, and ultimate strain were determined. The average values for each batch are provided in Table 3-3. Figure 3-4 shows the experimental stress-strain. An elastic modulus of $E_s = 29,000$ ksi (200 MPa) was assumed for all bars. The yield strength was determined by drawing a line with slope E_s offset at 0.2% strain. The stress at which this line crosses the test data is the yield stress, f_y . The corresponding yield strain is $\epsilon_y = f_y/E_s$. To capture the yield plateau behavior, the strain at which strain hardening (ϵ_{sh}) was reported. A stress, f_{sh} , was associated with this point that is larger than the yield stress; this creates a slight slope in the yield plateau, which eliminates issues associated with a zero slope when performing sectional analysis of the specimens. The maximum stress (f_{max}) and the corresponding strain (ϵ_{max}) were also recorded. The ultimate strain, ϵ_u , was reported as that at a stress of $0.9f_{max}$.

Table 3-3: Steel material properties

Bar size	f_y , ksi	ϵ_y , in/in	f_{sh} , ksi	ϵ_{sh} , in/in	f_{max} , ksi	ϵ_m , in/in	f_u , ksi	ϵ_u , in/in
#4	65.8	0.0023	66.4	0.0077	106.4	0.10	96.4	0.14
#2	75.7	0.0026	77.0	0.015	84.6	0.050	76.3	0.058



(a) #2 bars



(b) #4 bars

Figure 3-4: Reinforcement stress-strain response with average critical values.

3.2.1 Development of #2 Bars

Small No. 2 bars are not common construction material and are not produced by any local steel fabricator. However, researchers typically use smooth bar stock when constructing smaller-scale specimens. While this type of bar has the correct net area, it does not have the correct stress-strain characteristics of typical reinforcing bar. Usually, round bar stock have cold rolled response without a defined yield plateau and low ductility common in hot-rolled reinforcing steel.

The research group decided that it would be beneficial to add deformations to standard smooth bar stock and heat treat the steel to give it a hot-rolled response. The steel was purchased in a large round spool that was fed through a special machine that stamped on external deformations in a helical pattern. The deformation pattern that was stamped on the bar was comparable to conventional reinforcing steel such that it had a similar relative rib area defined as:

$$R_r = \frac{\text{projected rib area normal to bar axis}}{\text{nominal bar perimeter} \times \text{center-to-center rib spacing}}$$

Then the bars were cut to 15 foot straight lengths, bundled and shipped to a special heat treating facility. The 15 foot lengths were the maximum length that the oven at the heat treatment place could accommodate. The cold-rolled stress-strain response of these bars is shown in Figure 3-5.

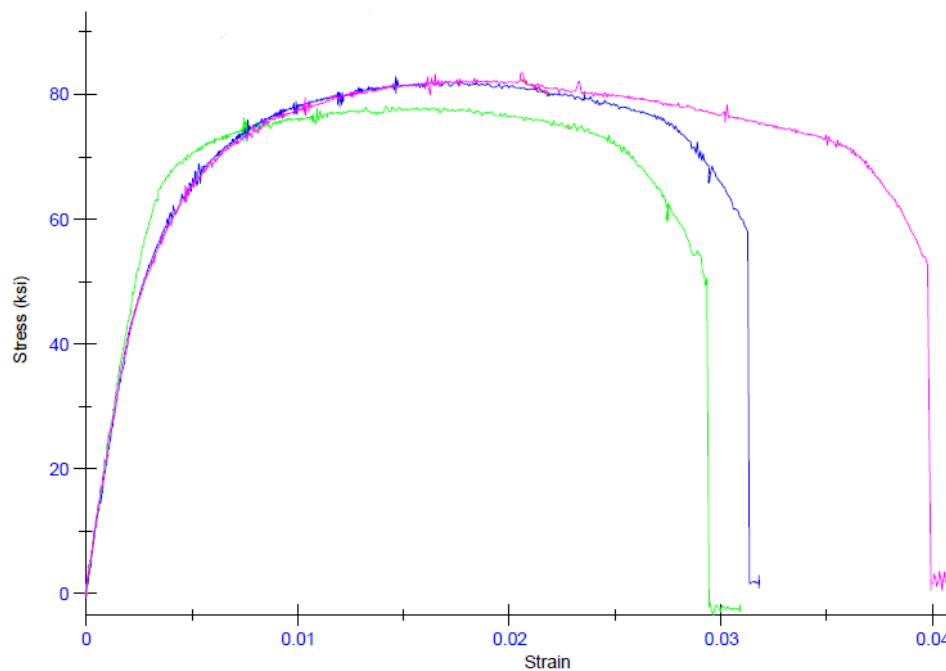


Figure 3-5: Cold-rolled stress-strain response of #2 reinforcing bars prior to heat treatment.

To augment the response of this steel, a series of heat treatments were applied to small samples about 12 inches long, varying both heating temperature and duration. After achieving acceptable results by heating a sample to 1125 degrees for one hour, a test run on a 2,500-pound portion of the total lot was performed by Exotic Metal Treatment, Inc., of Indianapolis, Indiana. Thermocouples were attached at

six locations within the lot, and temperatures were recorded as the forced air heating chamber was activated. The lot remained in the chamber until all thermocouples indicated that temperatures were within ± 25 degrees of the target 1125 degrees for one hour, after which time the lot was allowed to air cool. This lot of steel inside the heating chamber and the attached thermocouples are shown in Figure 3-6.

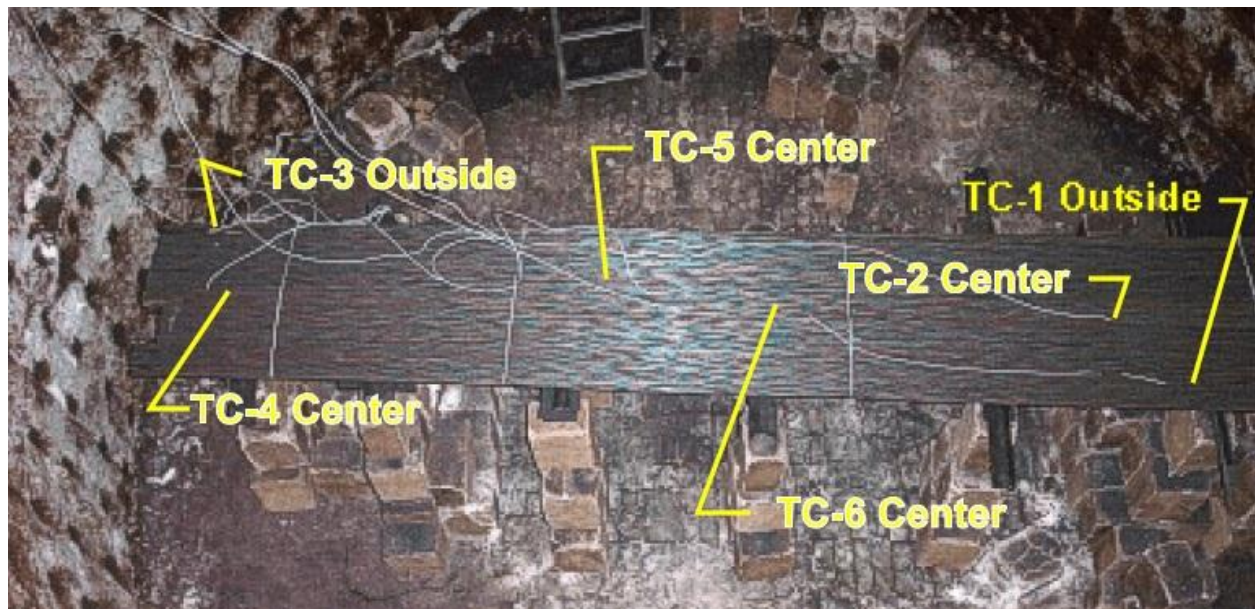


Figure 3-6: No. 2 rebar in heat treatment chamber.

Twelve-inch samples from varying locations were cut and tested for their stress-strain behavior, with results shown in Figure 3-7. While some of the samples exhibited acceptable behavior, the level of variation between all the samples was unacceptable. Steel samples taken from the ends of the bars in particular showed a much lower strength than the interior portions due to the fact that the ends were exposed to elevated temperatures the longest. While the interior reached 1125 degrees for one hour, the end regions experienced this temperature for up to 2.5 hours.

To correct for this imbalance, a new two-stage heating regimen was developed by the heat treaters. First, the entire lot of steel would be brought uniformly to a temperature beneath the stabilization temperature, 1000 degrees in this case. Once achieved, the chamber would resume heating until the thermocouple requirements were satisfied for an 1100 degrees target for one hour. After applying this scheme to another 2,500-pound lot, samples were cut and tested, and the results are presented in Figure 3-8. As shown, the behavior of the #2 reinforcement was now more uniform throughout the entire heat treatment batch. After the second heat treatment regimen was performed on the remaining 5,000-pound lot of steel, all three loads were shipped to the Illinois for use in constructing the test specimens. No reinforcement from the first load was used in construction.

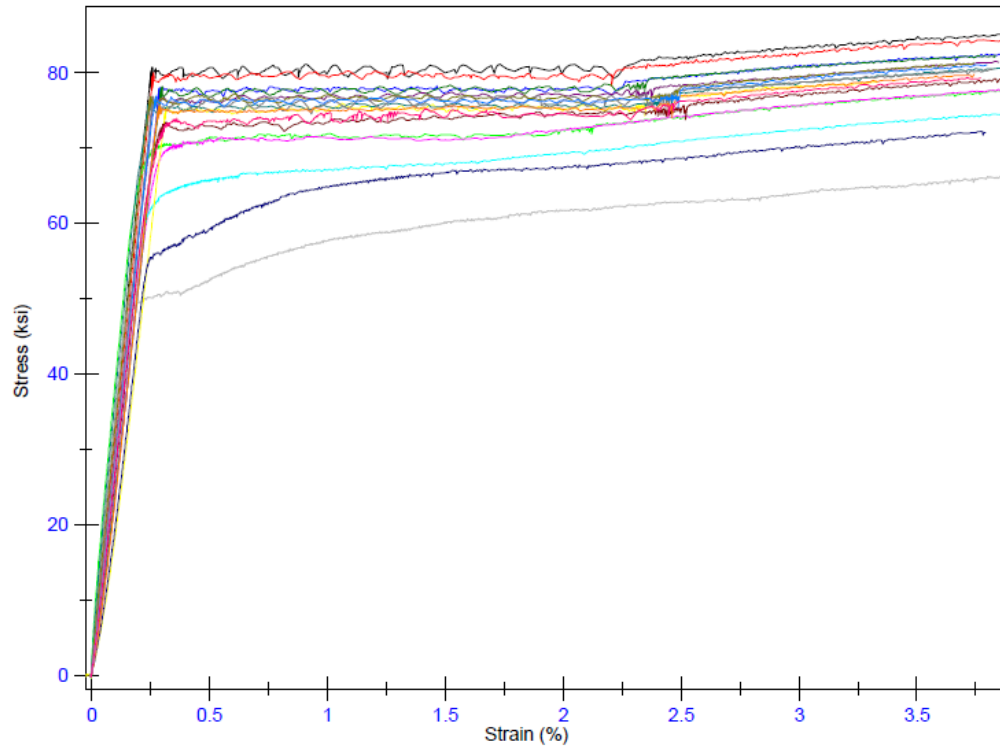


Figure 3-7: #2 rebar behavior from first large heat treatment batch.

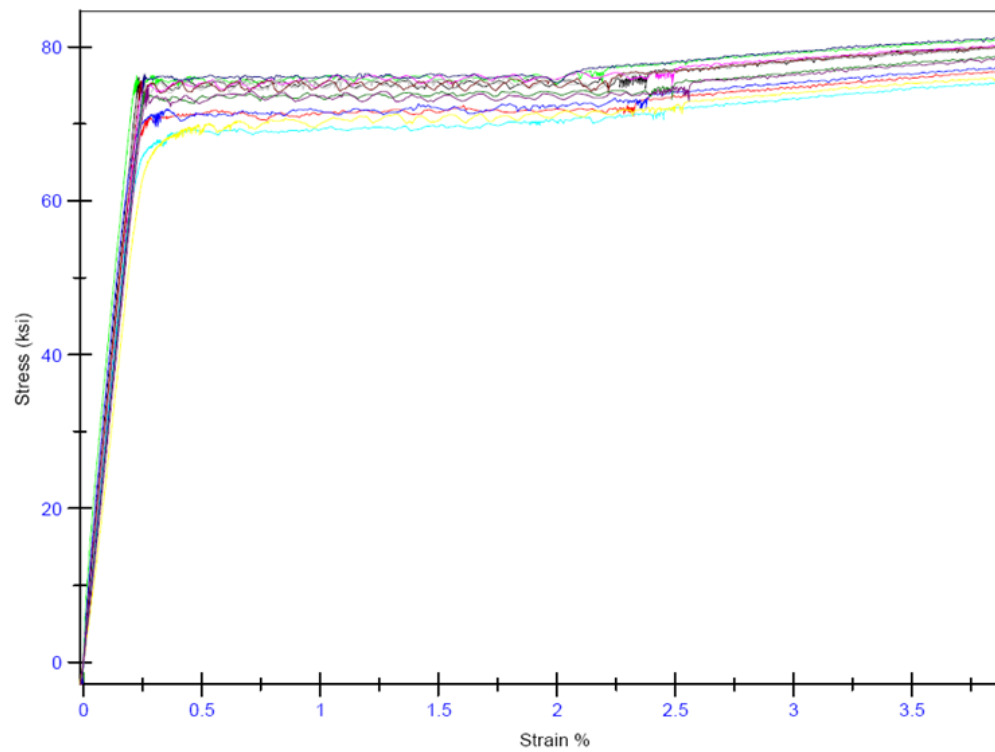


Figure 3-8: #2 rebar behavior from final heat treatment batch.

CHAPTER 4: Test Setup

The coupled wall specimen was tested at the University of Illinois NEES MUST-SIM Facility. This chapter provides details of the how the tests were setup within the facility (Section 4.1), placement of the specimen in the testing location (Section 4.2), and connection of the specimens to the laboratory strong floor and equipment used to load the specimens (Section 4.3).

4.1 Description of Test Setup

The test setup is located in the MUST-SIM facility at the University of Illinois (see Section 5.1). A 5 feet thick, L-shape strong wall that is heavily reinforced and post-tensioned sits atop a strong floor. The test specimens were positioned along the long leg of the L-shape strong wall. The in-plane direction of the wall specimen ran parallel with the long leg of the strong wall which is oriented in the East-West direction of the lab. A figure denoting the orientation of the lab can be seen in Figure 4-1.

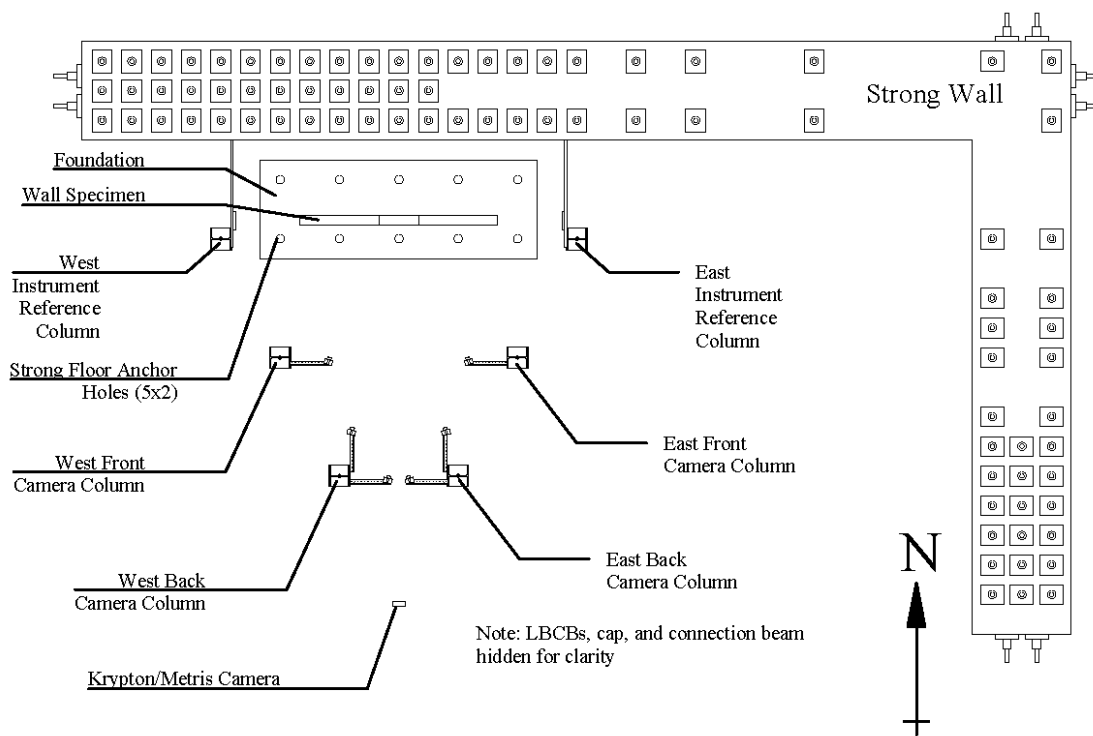


Figure 4-1: Plan view of typical experimental set-up of specimens at the NEES MUST-SIM Facility.

Loads were primarily applied to the specimens using Load and Boundary Condition Boxes (LBCBs) that allow application of forces in six-degrees of freedom. The LBCBs are described in detail in Section 5.2. The wall specimens were strong enough in their in-plane direction that two LBCBs were needed to test the walls to failure(). Each LBCB was mounted to the strong wall with 36 high strength 1.5 inch diameter threaded rods. The threaded rods were post-tensioned with a double acting center hole jack to 100 kips for a total tie down force of 3600 kips. This tie down force guaranteed that the boxes would not slip during loading. To further prevent the boxes from moving upward under extreme downward force,

simple shear keys were installed at the top of the LBCBs. External sensors were used to measure slip, and no significant movements were observed throughout the test.

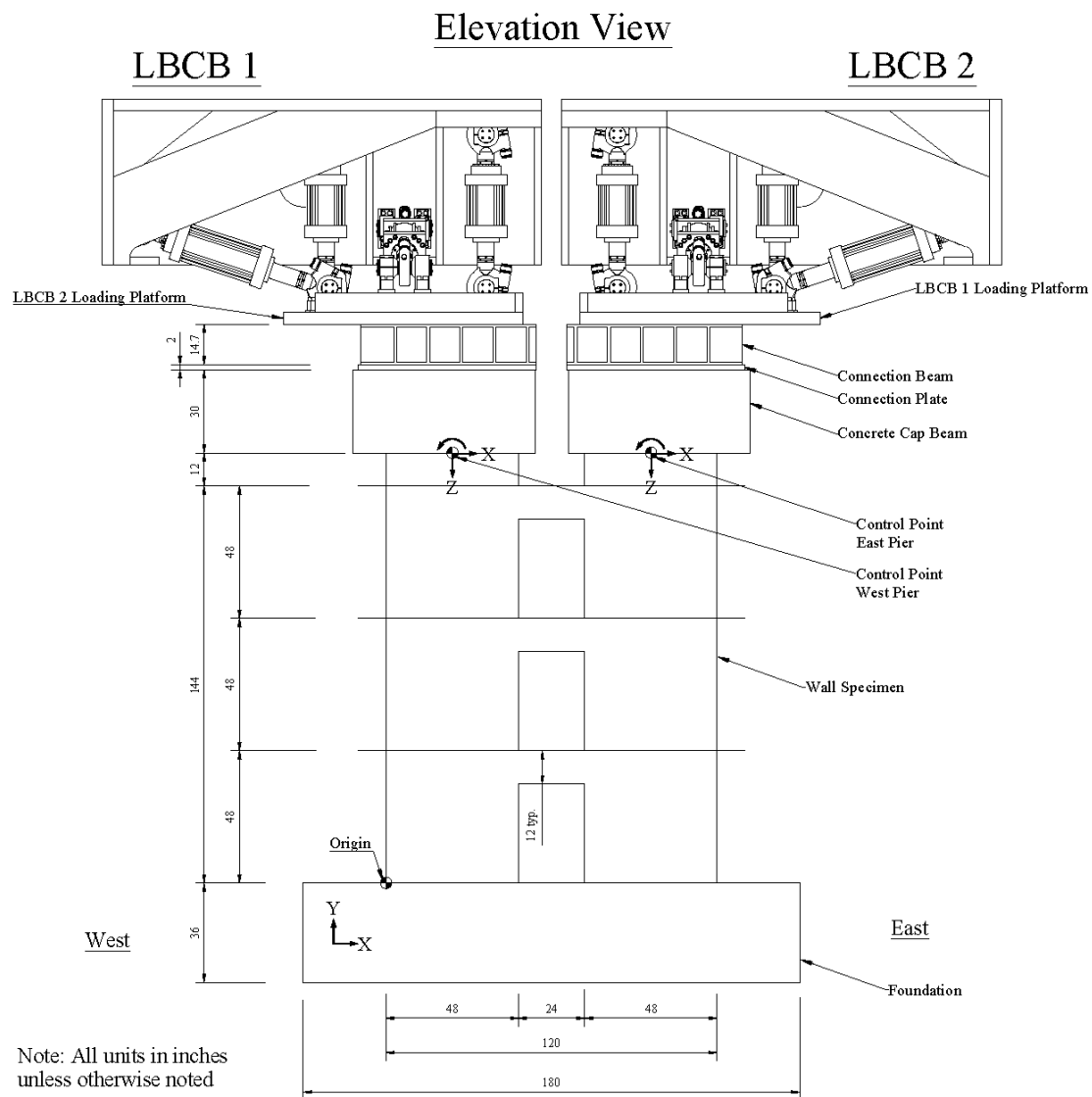


Figure A6-1: CW1 Test Setup

Figure 4-2: Elevation view of LBCB and coupled wall specimen test setup

Instrumentation reference columns were placed around the specimen to mount instruments and high-resolution cameras. Four camera reference columns were placed in front of the wall specimen to mount 11 cameras (Figure 4-1). Also two reference columns were placed on the east and west side of the specimen to affix instruments that measure absolute displacements. A reference stand was needed to mount the Krypton system to maximize the field-of-view of the system. This system is discussed in detail in Section 6.1.1.

4.2 Placement of Wall

The test specimens were constructed away from the testing location and moved into place prior to testing. Shoring was provided between the foundation block and wall cap to provide stability. To move the specimens into place, a special lifting rig was fabricated to help lift the approximately 27-31 ton specimens. The lifting rig is shown in Figure 4-3 for the placement of a planar wall. A spreader beam was used to lift the specimen over its center of gravity. Cables were attached to ducts running through the foundation block.

Because the LBCBs were attached to the strong wall prior to placement of the wall specimen, the specimens could not be fully moved into place using the lifting rig. The specimens were moved as close to the final resting place as possible; then, the final movement was accomplished by pushing on the specimens using standard double acting jacks that reacted against shear keys in the strong floor.



Figure 4-3: Moving a planar test specimen into the testing location (similar for CW1).

4.3 Description of Connection Details

Once the specimen was in the final resting place, connection of the specimen to the strong floor and loading units was accomplished in four stages. Section 4.3.1 details the connection of the specimens to the strong floor. Details of the connection of the LBCBs to the loading beams and of the loading beams to the pier caps are discussed in Sections 4.3.2 and 4.3.3, respectively.

4.3.1 Connection to Strong Floor

The foundation block anchored the specimen to the strong floor with ten heavy duty post tensioning rods. The post tensioning rods were high-strength steel, 100 ksi yield strength, 2 inch diameter. Each

post tensioning rod was stretched to a force of approximately 100 kips for a total tie down force of 1,000 kips. This was done to ensure that the design shear force imposed on the wall would not cause the specimen to slide across the floor. Prior to stretching to the bolts, low shrinkage, high strength hydrocal was placed underneath the wall to provide a smooth uniform area for the wall to sit on.

4.3.2 Loading Beam Connection

The loading platens on the LBCBs can accommodate connections by means of a grid on 1 inch tapped holes, spaced at 6 inch centers across the platen's length and width. Direct connection of the specimen pier caps to the loading platens was not possible due to connection tolerances of the specimens; an intermediate connection was therefore necessary to allow for creating a high-tolerance connection to the LBCB platforms before making a low-tolerance connection to the specimen caps.

This intermediate connection element is referred to as the loading beam, or connection beam. For the coupled wall, the LBCBs were used to control each wall pier separately. The loading beams were necessary to connect the concrete pier cap to each LBCB (Figure 4-4—Figure 4-6). The beam was designed to remain elastic and evenly distribute the load from each LBCB such that any “fighting” between the boxes would be absorbed by the loading beam. The connection beam was a Grade 50, W14x132 steel section with one inch thick web stiffeners placed at 12 inches on center. These stiffeners were added to resist buckling of the beam web. Two additional 62 inch by 26 inch by 2 inch thick spreader plates were attached flush against the wide flange beam with 1 inch bolts running through a counterbored hole. The beam was directly connected to the LBCBs with 1 inch high strength socket head cap screws placed at 6 inches on center on either side of the beam. A total of 40 bolts were used to make this connection (2 rows of 10 for each LBCB).

4.3.3 Cap Connection

After the LBCBs were connected to the connection beams, the LBCB platforms were completely retracted to allow the specimen to be pushed underneath the loading units (see Section 4.2). Once the specimen was in the correct place on the strong floor and anchored to the strong floor with post tension rods (Section 4.3.1), the connection beam was lowered onto the concrete cap. A 0.5-1.0 inch thick layer of wet low shrinkage hydrocal grout was poured between the loading plate to the concrete cap in order to provide a level surface. Once the hydrocal cured, high strength 1.5 inch diameter post tension rods were used to torque the beams to the pier caps. Thick 1 inch plate washers were between the bottom side of the pier cap and the nut.

During the connection process, concrete surface strain gauges were used to monitor the response of the wall. For all specimens there was no significant external load introduced into the specimen during the connection.

CHAPTER 5: Loading and Displacement History

The Illinois MUST-SIM facility's equipment allows for load application in all six degrees of freedom, and successful experimentation requires an extensive attention to detail. This chapter details the components associated with load control for the wall tests. Sections 5.1 and 5.2 provide summaries of the MUST-SIM facility and the loading equipment used, respectively. Section 5.3 provides an overview of the software used to control the tests. Section 5.4 provides details on the control of the coupled wall test, including the applied forces and displacement history.

5.1 Illinois MUST-SIM Overview

The Multi-Axial Full-Scale Sub-Structured Testing and Simulation (MUST-SIM) facility at the University of Illinois is one of the 15 equipment sites that form the George E. Brown Network for Earthquake Engineering Simulation (NEES). Central to the facility are the six-DOF Loading and Boundary Condition Boxes (LBCBs)—three in the large-scale laboratory and three in the small-scale studio.

In the large-scale lab, the LBCBs can be oriented as needed on either the reaction floor or reaction wall. The reaction wall is a five foot thick, 28-foot tall L-shaped post-tensioned concrete wall, with legs measuring 50 and 30 feet long. The large-scale facility with the strong wall and three LBCBs is shown in Figure 5-1, with a closer view of an LBCB shown in Figure 5-2. Examples of structural components tested in the MUST-SIM facility include concrete bridge piers, steel frames, steel beam-column assemblies, and the concrete structural walls discussed herein.

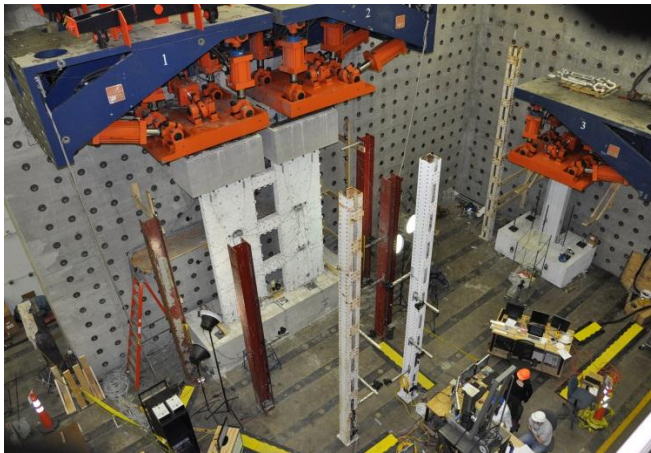


Figure 5-1: Illinois MUST-SIM facility.



Figure 5-2: LBCB resting on lab floor.

The small-scale facility that has a 1/5 scale reaction structure with three 1/5 scale LBCBs. The small-scale facility provides researchers the opportunity to conduct pre-test verifications with various load and displacement scenarios. The small-scale laboratory is essential to the operation of the large-scale facility in that it allows users to understand the capabilities and limitations of the laboratory and the control systems prior to conducting experiments using the large-scale facility.

The MUST-SIM facility is capable of conducting hybrid simulation tests, whereby displacement demands are computed by analytical tools and are updated with feedback from experimental data. Hybrid

simulation coupled with six degree-of-freedom loading units allows the facility to conduct versatile component testing where the structural component of interest is tested experimentally and the remainder of the structure is modeled analytically.

5.2 Load and Boundary Condition Boxes (LBCBs)

The LBCBs are large loading units in which the loading platform (shown in orange in Figure 5-3) is a loading and boundary conditions point where any combination of six actions (three forces and three moments) and six movements (three rotations and three translations) may be controlled. Each LBCB weighs 35 tons and can be attached at the bottom (on the strong floor), or on the side (on the reaction wall). The unit itself is a self-contained system whereby the six actuators react within its own reaction frame, shown in blue in Figure 5-3. Figure 5-3 also shows the naming convention of the six actuators. The six actuators are labeled X1, X2, Y1, Z1, Z2, and Z3 which correspond to the global coordinate space of the LBCB as a whole.

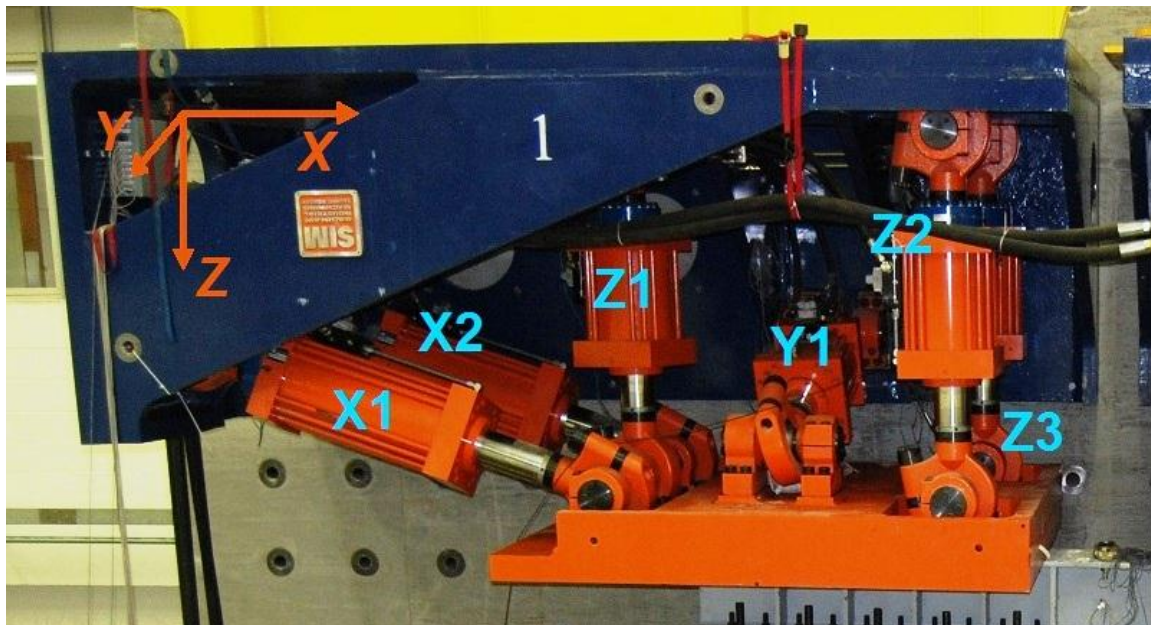


Figure 5-3: LBCB and actuator naming convention.

The overall dimensions of the reaction box are approximately 11.8 feet long, 6 feet wide and 6 feet high. The loading platform is approximately 7.2 feet long and 6.2 feet wide. Each individual actuator has a capacity of 225 kips in tension and 311 kips in compression. While every actuator moves and contributes to each single degree-of-freedom, the majority of each force and displacement limit is controlled by the actuators in the principal direction. The three vertical actuators (Z1, Z2, and Z3) are primarily used to control the z , θ_x (roll) and θ_y (pitch) position of the loading platform. Each of these actuators is anchored to the base of the reaction box and attached to the underside of the loading platform. Two horizontal actuators (X1 and X2) that are attached to the end of the reaction box are used to primarily control the x and θ_z (yaw) position of the loading platform. One additional horizontal actuator (Y1) is used to control the y -direction position of the loading platform. The capacity and

displacement limits of the six degrees-of-freedom can be seen in Table 5-1. Pillow Block Spherical Bearings have been selected so that the motion of the loading platform is not unduly restricted in any of the six degrees of motion.

Table 5-1: Generalized LBCB capacity and displacement limits.

Loading DOF	Force Capacity	Stroke
X-Translation	430/660 kips (T/C)	± 10 in.
Y-Translation	215/330 kips (T/C)	± 5 in.
Z-Translation	645/980 kips (T/C)	± 5 in.
X-Rotation	7600 kip-in	± 16°
Y-Rotation	10000 kip-in	± 11.8°
Z-Rotation	7600 kip-in	± 16°

5.3 Load Control Software

The MUST-SIM facility uses four separate software programs to control an experiment which makes the control architecture complex yet extremely flexible and comprehensive. The architecture can be thought of as a top-down hierarchal chain of steps, substeps, commands, and checks all looped over one another. The four programs are called Simulation Coordinator (SimCor), LBCB Plugin, Operations Manager (OM), and the Shore Western control software. While these are the four software packages directly related to the control of the LBCBs, other software is utilized during the experiment to measure and record data, take pictures, and share this data with offsite researchers and collaborators. These other programs, which will be discussed later in this document, are the data and acquisition software (NEESdaq), camera plugin, Krypton plugin, Data Turbine, Remote Data Viewer (RDV), and NEES Central. The entire software architecture can be seen in Figure 3-2Figure 5-4.

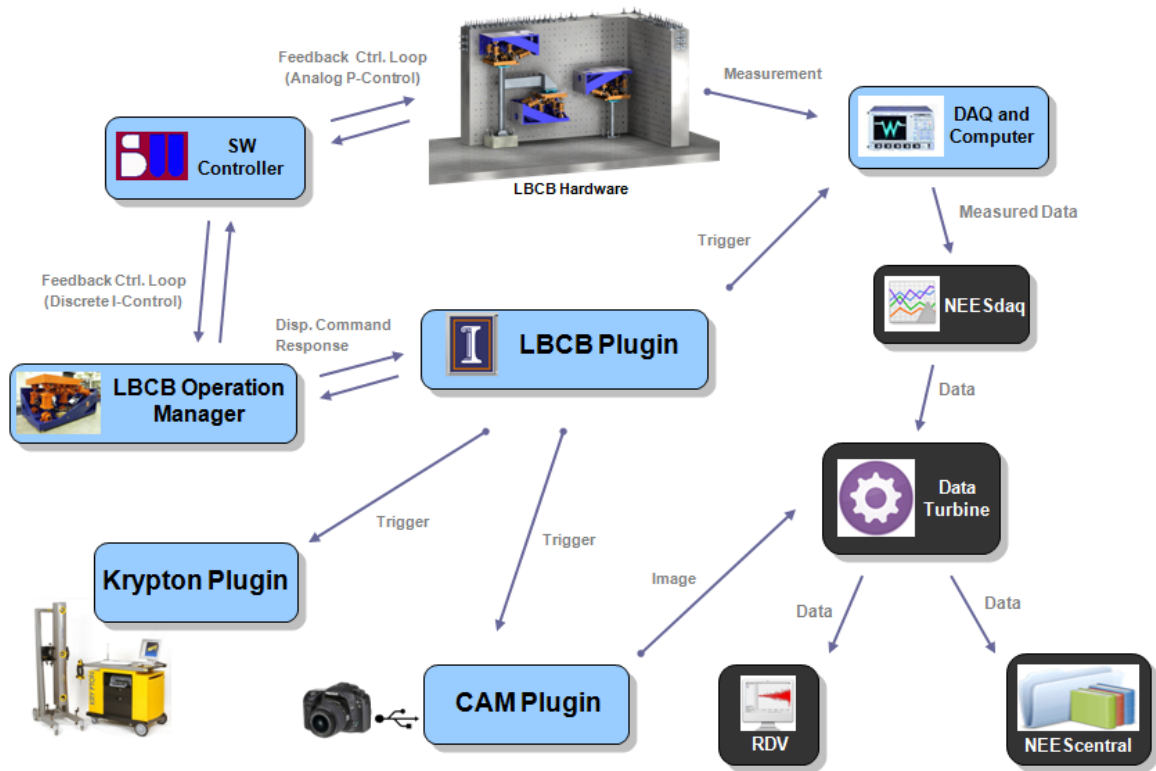


Figure 5-4: MUST-SIM software architecture.

5.3.1 SimCor

SimCor (Simulation Coordinator) is the main program that coordinates the entire experiment. The program was developed at the University of Illinois using Matlab. The user inputs a desired loading protocol into SimCor and it divides it up into loading steps, sometimes referred to as SimCor steps. The main function of this program is to communicate with all the other computers and software packages by sending triggers and messages at the start and/or completion of each loading step. For example, at the completion of a loading step SimCor sends a message to the data acquisition software to take a series of measurements to record step data. A screen shot of this software package can be seen in Figure 5-5.

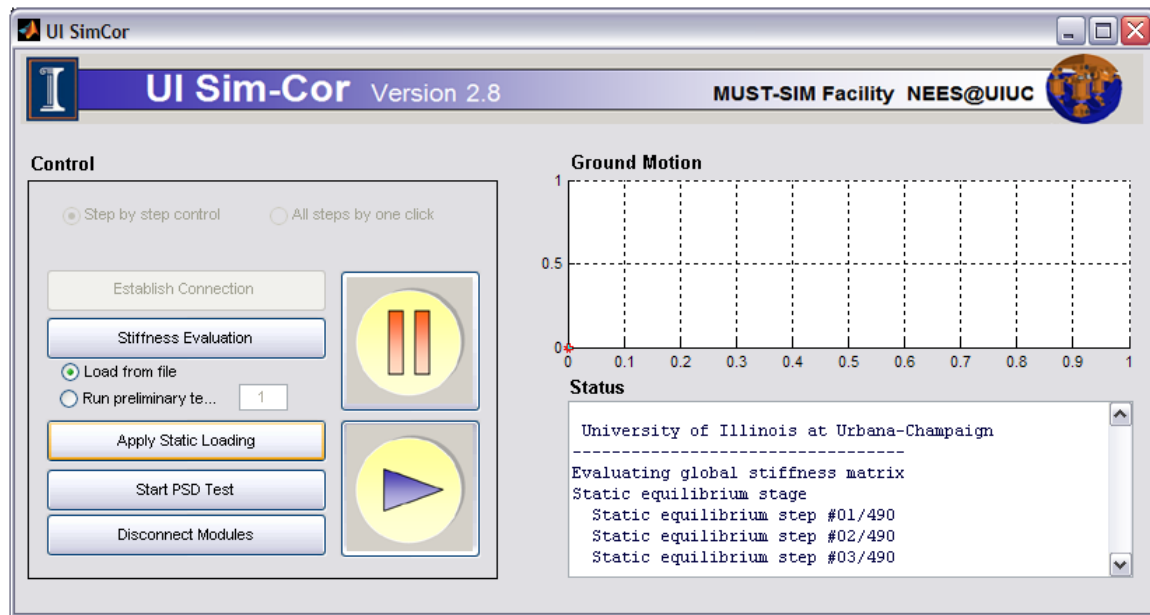


Figure 5-5: SimCor screen shot.

5.3.2 LCB Plugin and Operations Manager

The primary function of SimCor within the control architecture is to pass displacement targets to the LCB Plugin. The displacement targets can be predetermined from a loading protocol, or the displacements can come from hybrid simulation force feedback. The LCB Plugin is another Matlab program that was developed at the University of Illinois. It is a project specific program that has user defined information about the test structure. It breaks a SimCor step into manageable substeps that fit within a desired substep tolerance. For example, a SimCor step may be a tenth of an inch of lateral displacement. It would not be wise to move a concrete specimen abruptly a tenth of an inch, therefore the LCB Plugin will divide the tenth of an inch into manageable substep increments such as 0.01 inches, or whatever is acceptable to the researchers. The LCB Plugin manages user defined displacement and rotation tolerances, as well as force control tolerances and calculations. In the case of the wall project, a desired axial load of $0.1 f_c' A_g$ was targeted and a tolerance of ± 5 kips was specified within the LCB Plugin. If this tolerance is not met, the Plugin will continually update the commands it sends to the Operations Manager until it converges on the tolerance or the user can increase the tolerance to force the program to move onto the next step. Figure 5-6 provides a screen shot of the LCB Plugin.

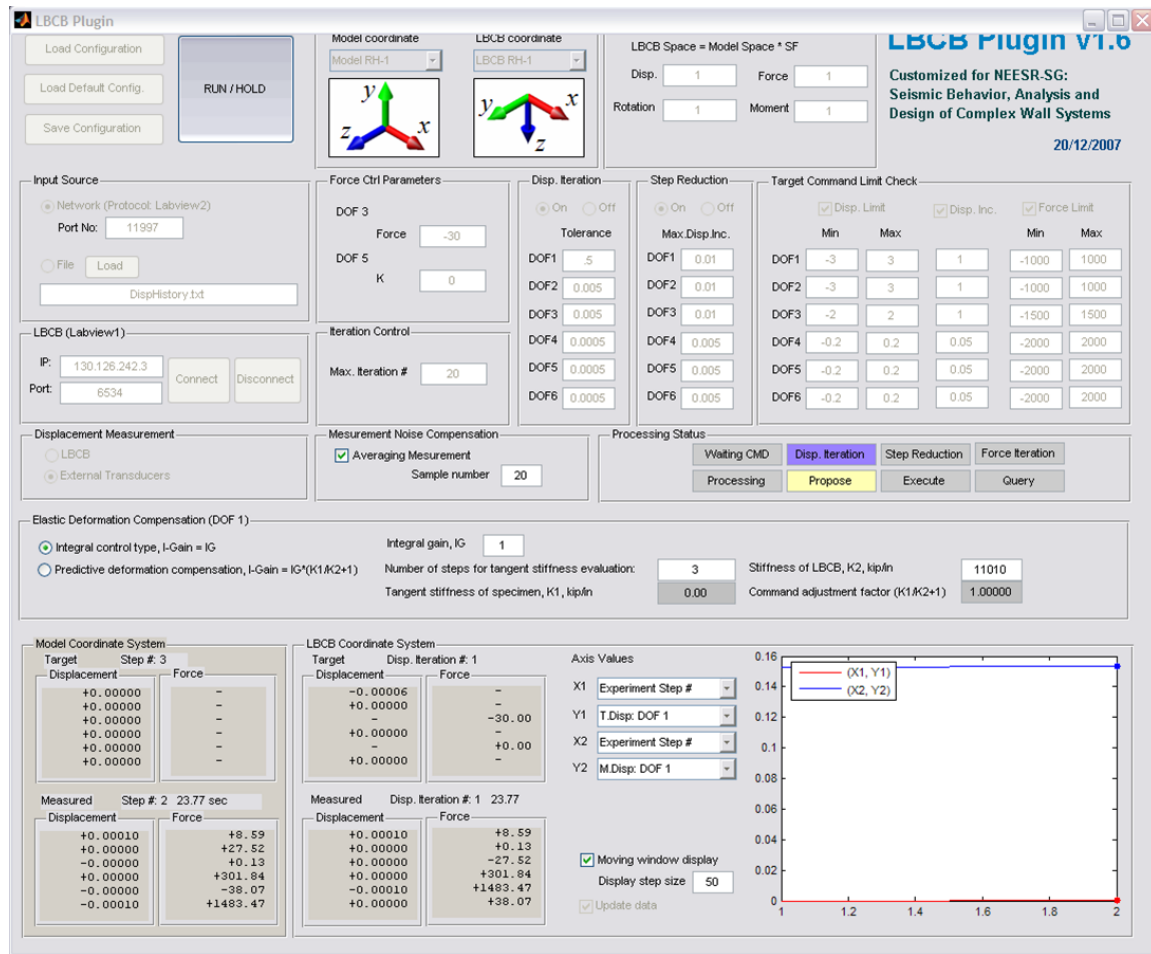


Figure 5-6: LBCB Plugin screen shot.

5.3.3 Operation Manager

The Operation Manager is a LabView based program developed at the University of Illinois. It is continuously receiving displacement, rotation, and force targets from the LBCB Plugin and directly communicates with the Shore Western software, hardware and servo valves to physically move the actuators. The LBCB Plugin is used to send substep commands to the Operations Manager, then the Operation Manager executes the commands and sends back the measured displacement and force readings from the system which the LBCB Plugin then checks against user defined tolerances before proceeding to the next substep. Figure 5-7 shows a screen shot of the Operation Manager.

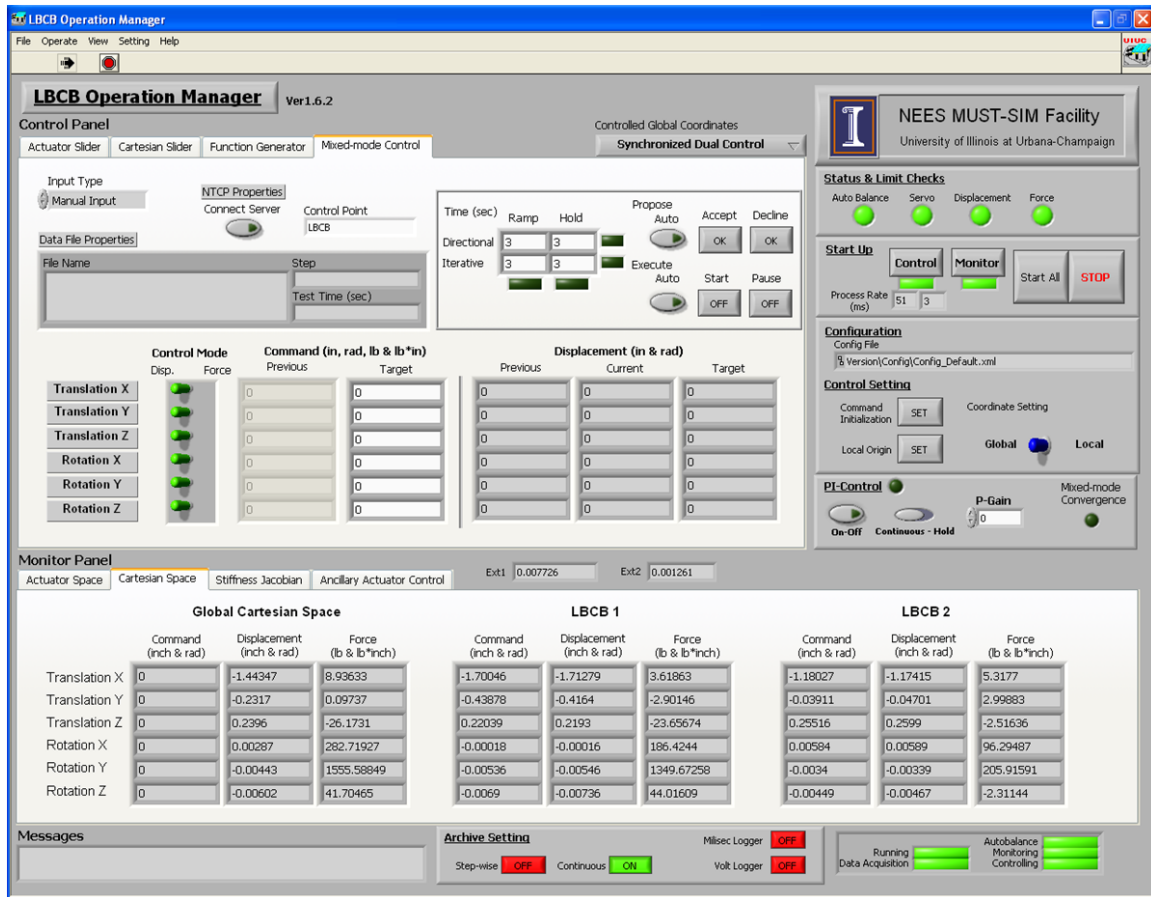


Figure 5-7: Operation Manager (OM) screen shot.

5.3.4 Shore Western Control Software

The Operations Manager is continuously communicating with the Shore Western control software. The Shore Western control software sits on an independent computer that houses the control hardware used to send voltages to the actual servo valves to move the LBCBs. The Shore Western software does nothing more than relay the commands from the Operations Manager. A screen shot of this program can be seen in Figure 5-8.

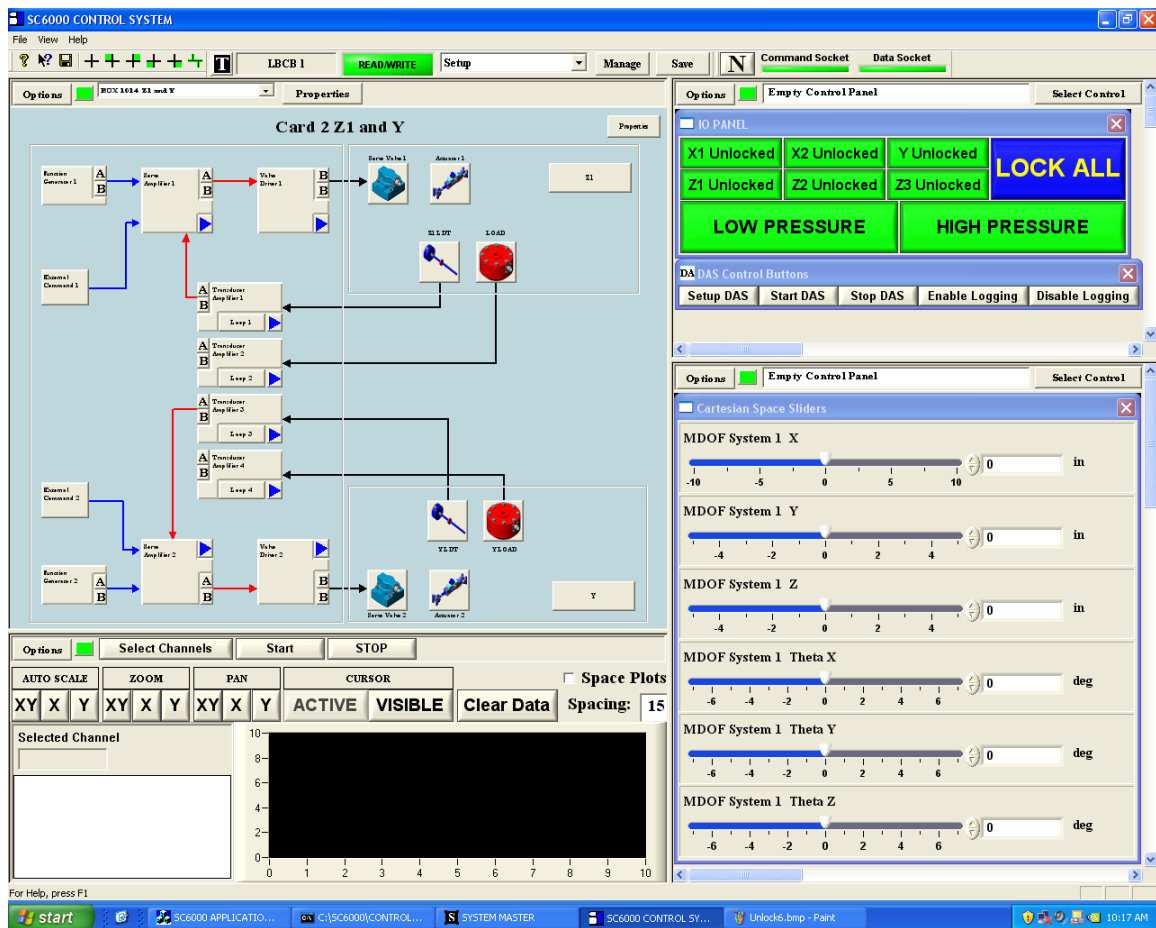


Figure 5-8: Shore Western control software screen shot.

5.3.5 Mixed-mode Control

An important feature within the MUST-SIM control software is the ability to control any degree-of-freedom in either displacement control or force control called mixed-mode control. Displacement control is straight forward in that the actuators are moved precisely according to the command input voltage. However, this is not the case for force control. To impose a desired force on a structure, it is necessary to know the stiffness of the structure. This is an ever complicated value to calculate in all degrees-of-freedom while a structure undergoes nonlinear damage and deterioration. The Operations Manager overcomes this challenge by continuously updating the Jacobian stiffness matrix with every displacement command it receives. This type of force control is referred to as a discrete iterative control loop, rather than traditional proportional integral, and derivative (PID) force control algorithms.

5.3.6 Elastic Deformations Correction

Another unique feature within the control software loop is the ability to correct for elastic deformations (ED). Elastic deformations are defined as the elastic movement of the system outside of the test structure. In other words, the entire system deforms as load is applied to a structure, not just the structure. For example, the LBCBs blue reaction box deforms as load is applied to a structure, as well as the strong floor, and any connection device used to transfer load to a specimen. This unwanted

movement needs to be compensated for. To account for elastic deformations, external instruments are used to monitor the absolute movement of the test structure and this information is fed into the LBCB Plugin and an ED correction step is executed at the completion of each LBCB Plugin step. It is up to the researcher to decide which elastic deformation degrees-of-freedom should be accounted for. In the coupled wall test ED correction steps were not needed because provided tolerances from the external instruments were met.

5.4 Loading Protocol

To maximize the specimen dimensions and focus on regions in which inelastic action occur, only the lower three stories of the ten-story prototype wall were modeled in the laboratory. The LBCBs were used to simulate demands originating from the upper stories of a 10-story structure subjected to an ASCE 7-05 lateral load distribution (Figure 5-9). The loading protocol was more complicated than the planar wall test loading protocol. For the planar wall tests, the relation between the shear and moment at the top of the walls was appropriately set to simulate the demand. In the coupled wall, both the overall loads (shear F_x , moment M_y , and axial F_z) and the local pier loads (pier shear, moment and axial) must simulate load that would be present in a coupled wall. As noted in Figure 5-10, the overall loads are designated with a subscript of *total*; the local demands at the top of the wall are designated with a subscript of 1 or 2, for the west and east piers, respectively.

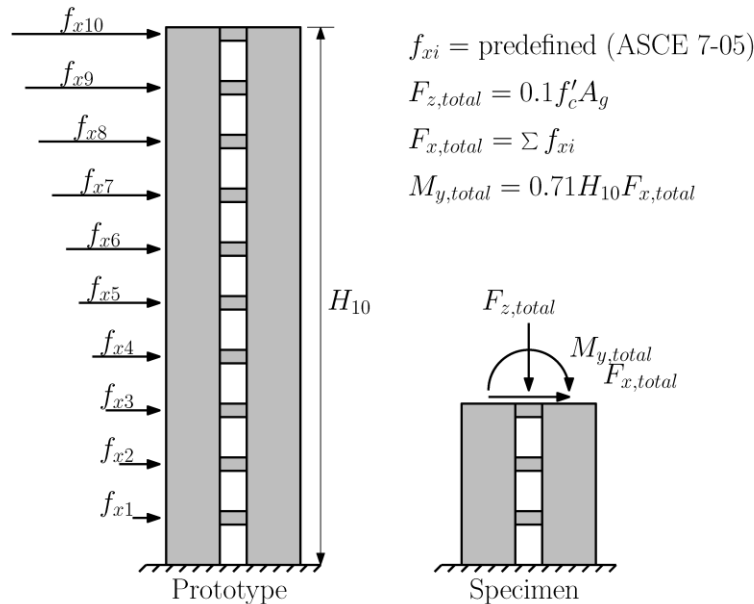


Figure 5-9: Definition of applied loads for 1/3-scale prototype and specimen

Of particular interest was the determination of the relative displacement of the two wall piers, and the relation to proportion the total moment applied to the top of the specimen to meet target individual pier moments and coupled moment ($F_z \times L$). Both of these relations were determined using the results of continuum analyses, which are summarized here. Additional information on the development of the loading protocol may be found in Marley (2011).

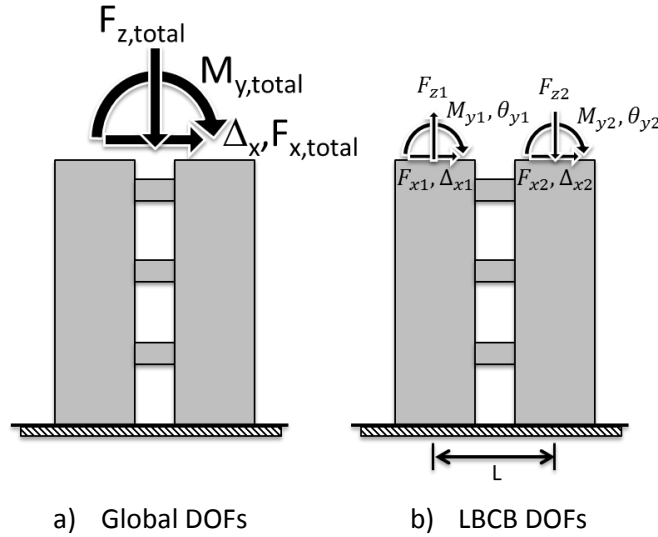


Figure 5-10: Degrees of freedom associated to the loading protocol

The loading protocol was developed to control the combination of displacements and forces in all six degrees of freedom at points of contact between the loading devices and the test structure. A critical aspect of test planning was to develop a test protocol that realistically simulates the demands on a coupled wall in an accurate manner. The constraints were as follows:

1. Because the coupled wall specimen was tested as a planar element, the associated inactive degrees of freedom (DOFs) were maintained to be zero for the entire test.
2. The free-body diagram from the load distribution provides constraints on the total axial load, total shear force, and total moment using the assumed gravity load and lateral load distributions.
3. The remaining loads and deformations must be defined including vertical (axial) displacement, horizontal (shear) displacement, and rotation (moment) for both LBCBs for a total of six active DOFs. Therefore, three additional constraints are required to fully define a viable loading protocol.

The coupled wall loading procedures were based on extensive analytical investigations of the 10-story prototype and the 3-story specimen model using VecTor2, a two-dimensional nonlinear finite element program. The initial results for the analysis were developed to define the relations between the horizontal displacements of the wall piers and horizontal displacements and rotations. This resulted in the following constraints for the loading of the wall piers:

$$M_{y,\text{total}} = 0.71H_{10}F_{x,\text{total}} \quad (\text{Eq. 3})$$

$$F_z = 0.10f'_cA_g \quad (\text{Eq. 4})$$

$$\Delta_{x1} = \text{predefined} \quad (\text{Eq. 5})$$

$$\Delta_{x2} = \Delta_{x1} \quad (\text{Eq. 6})$$

$$\theta_{x1} = \alpha \times \Delta_{x1} \quad (\text{Eq. 7})$$

$$\theta_{x2} = \alpha \times \Delta_{x2} \quad (\text{Eq. 8})$$

where, the variable α was determined from the 10-story cyclic analysis. This loading regime constrains four of the six active degrees of freedom using displacement-control. Furthermore, the global loading objectives call for the total moment and axial load to be force-controlled, this requires iterating on individual pier axial loads using force control after applying their rotations to satisfy both requirements.

The proposed mix-mode control algorithm was implemented in VecTor2 to simulate the LBCBs control logic implemented in the laboratory control software. While the global results showed good correlation between the 10-story and 3-story models. There were important more local differences. In particular, the analyses indicated that the top (third-story) coupling beam was subjected to large normal strains (large compressive force in the top story beam) which resulted from the imposed lateral displacements of the wall piers.

Since using a predefined lateral displacement relation between the two wall piers resulted in erroneous demands in the coupling beam, the loading protocol was modified to include an iterative procedure of controlling the wall piers while satisfying a differential in shear force between each wall pier. This approach resulted in the following loading protocol constraints:

$$M_{y,\text{total}} = 0.71h_{10}F_{x,\text{total}} \quad (\text{Eq. 9})$$

$$F_{z,\text{total}} = 0.10f'_cA_g \quad (\text{Eq. 10})$$

$$\Delta_{x,\text{total}} = 0.5(\Delta_{x1} + \Delta_{x2}) \quad (\text{Eq. 11})$$

$$F_{x2} - F_{x1} = f(F_{x,\text{total}}) \quad (\text{Eq. 12})$$

$$\theta_{x1} = \alpha \times \Delta_{x1} \quad (\text{Eq. 13})$$

$$\theta_{x2} = \alpha \times \Delta_{x2} \quad (\text{Eq. 14})$$

Compared to the first loading protocol, the new loading protocol had two significant differences. First, the average global lateral displacement of the wall system was defined (Eq. 11) Second, a shear force differential, dF , (between the two piers) was defined as a function of the total force applied to the specimen. This revised loading protocol was implemented in VecTor2, and the results indicated an improvement in the stress distributions at the top floor while providing a good correlation between the 10-story and 3-story global results.

The algorithm was then implemented in the laboratory's control software, and the control algorithm was verified in the laboratory at very small cycles. However, implementing this algorithm at small elastic cycles resulted in convergence issues because the imposed rotation-displacement relationship could not be satisfied. The moments induced by a small change in rotation affected the ability for the other DOFs to converge appropriately (i.e. the shear differential and axial load due to coupling).

To troubleshoot the issue seen in the laboratory during verification, a third set of control procedures were developed to control the shear force differential, the relative coupled moment and applied pier moments in the tension and compression piers. The resulting algorithm is provided in Table 5-2. This final control algorithm improved the correlation between the results of both numerical models and provided a robust control logic for use in the laboratory. Three distinct testing phases were identified.

Finally, the prescribed loading history for the experiment is shown in Table 5-3. Note in the table that the loading protocol for Phase 1 was modified for the 0.128% drift level. The only change was the lateral DOF followed Phase 2 loading, but all other loading categories followed Phase 1 protocol.

Table 5-2: Final loading protocol

	Phase 1	Phase 2	Phase 3
Description	Pre-cracking	Cracking/yielding	Concrete Damage
Lateral DOF	Force-controlled	Force-controlled; adjusted as needed to reach target displacements	Displacement- controlled; adjusted as needed to satisfy dF target
Shear Force Differential	dF = 0	dF = 0.18(F _{x1} + F _{x2})	
Rotational DOFs	Force-controlled		
Compression Pier Moment	0.10M _{y,total}	0.15M _{y,total}	
Tension Pier Moment	0.10M _{y,total}	0.05M _{y,total}	
Axial Forces	$F_{zi} = \pm \frac{0.80M_{y,total}}{L} - 0.1f'_cA_g$		

Table 5-3: Loading history

Cycles	Target Force/Drift	Loading Phase	Description
1-2	10 kips	1	Pre-cracking
3-4	20 kips	1	
5-6	0.128%	1*	Cracking
7-8	0.192%	2	Pre-yielding
9-10	0.321%	2	
11-12	0.50%	2	Yielding
13-14	0.75%	2	
15-16	1.00%	2	
17-18	1.50%	3	
19-20	2.25%	3	

*Loading phase modified during test

5.5 Applied Displacement and Force Histories

As discussed in Section 5.4, the testing protocol used to control CW1 employed a force control logic up to the start of the 1.50% drift level and a displacement control for the remainder of the experiment. Figure 5-11 is the measured cyclic load history, and Figure 5-12 is the measured third-story drift history.

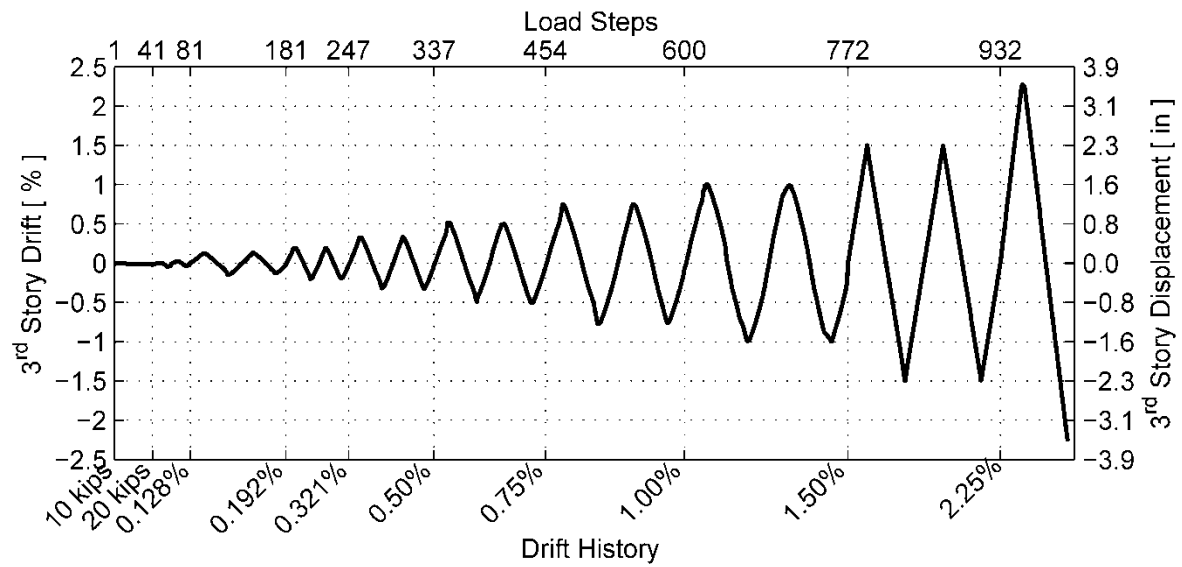


Figure 5-11: CW1 loading program: drift history

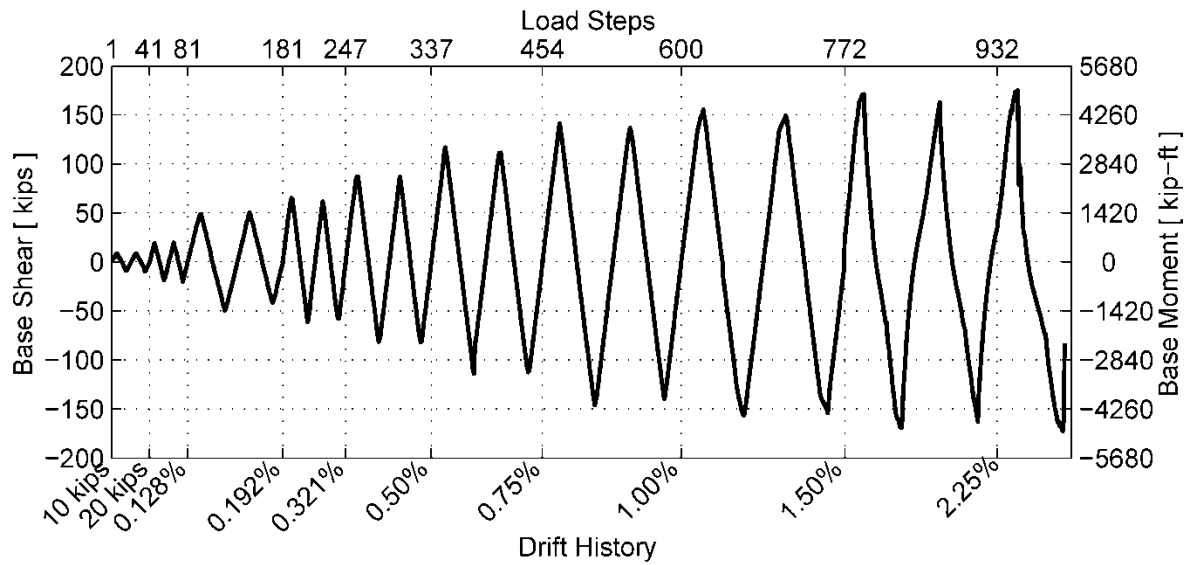


Figure 5-12: CW1 loading program: load history

CHAPTER 6: Instrumentation and Data Collection

One of the unique features of the MUST-SIM facility is a wide range of instrumentation, including conventional and non-contact instrumentation systems (e.g. photogrammetric and Metris) were available. These instruments were used to meet the research objective of capturing local response mechanism to support design and analysis of modern walls. The coupled wall instrumentation plan called for over 450 channels of data and 11 strategically placed high-resolution cameras in order to successfully capture the specimen behavior. Continuous data was recorded at 1.5 Hz and was captured using all the conventional instrumentation as well as the Metris system (recorded at 1.0 Hz). Additionally, data was acquired for all the instrumentation systems, including the photogrammetry system, at the completion of each prescribed load/displacement step. This sophisticated acquisition was achievable due to the sophistication of the LBCB plugin control software developed by UIUC. Files of test specimen instrumentation are located on the NEES Project Warehouse (see Section 8.4.3). These files contain tables and figures which link the instrumentation to the appropriate instrument channel names. Additional instrumentation figures and details are presented in Turgeon (2011) and Hart (2011).

6.1 Non-Contact Instrumentation

Advanced measurements systems were used to aid in the collection of high-resolution data. Section 6.1.1 discusses the Krypton measurement system and Section 6.1.2 discusses cameras used to collect high-resolution photographs of the wall damage.

6.1.1 Krypton System

The Krypton/Metris/Nikon K600 optical coordinate measuring machine (CMM), shown in Figure 6-1, is a system that uses three linear charge-coupled device (CCD) cameras mounted to a fixed unit to measure the position of infrared light-emitting diodes (LEDs) through triangulation. Key features include the ability to define a measurement coordinate system, record rigid body motion of a set of points (dynamic frame), and record the 3D position of up to 256 LEDs. LEDs were attached to the front (south) face of the wall specimens in a grid covering the first two floors (lower two-thirds) of the specimens. By recording the position of these targets throughout the test, the three-dimensional displacement field of the wall surface is known. The limited range of the Krypton system is shown in Figure 6-2.

Targets were distributed in a grid pattern on the south face of each wall pier in the first two stories of the specimen (Figure 6-3). This grid was spaced approximately six inches horizontally and 12 inches vertically. Where rods were embedded in the wall to anchor LDVTs, targets were placed on the exposed ends of the rods; elsewhere targets were glued to the wall surface. An additional 6x8-inch grid of LEDs was attached to embedded rods on the first and second story coupling beams.

K600 data was recorded and stored separate from the traditional instruments. The Krypton system collected continuous data at approximately 1 Hz, depending on the total number of LEDs used. In addition to the continuous data, the LBCB control software triggered recording of data over three seconds at each displacement step. Each three-second interval of data was then averaged to obtain cleaned step data.

In addition to the test measurements, the K600's dynamic frame capabilities were used to measure the motion for both LBCBs relative to the specimen's coordinate system prior to the test. After making these observations, the data can be operated on to produce a transformation matrix that augments the LBCB motion to align with each other and the specimen. In a similar manner, the control points (or centers of rotation) of the LBCBs can be observed and transferred to the appropriate place on the specimens.



Figure 6-1: Krypton non-contact measurement system.

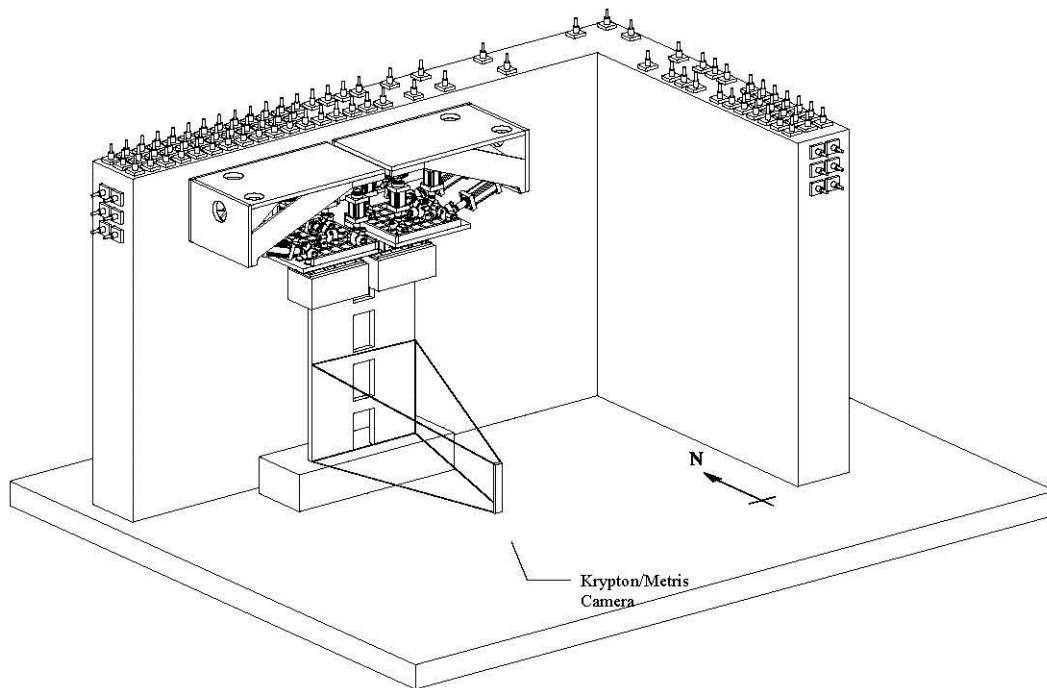


Figure 6-2: Measurement range of Krypton measurement system.

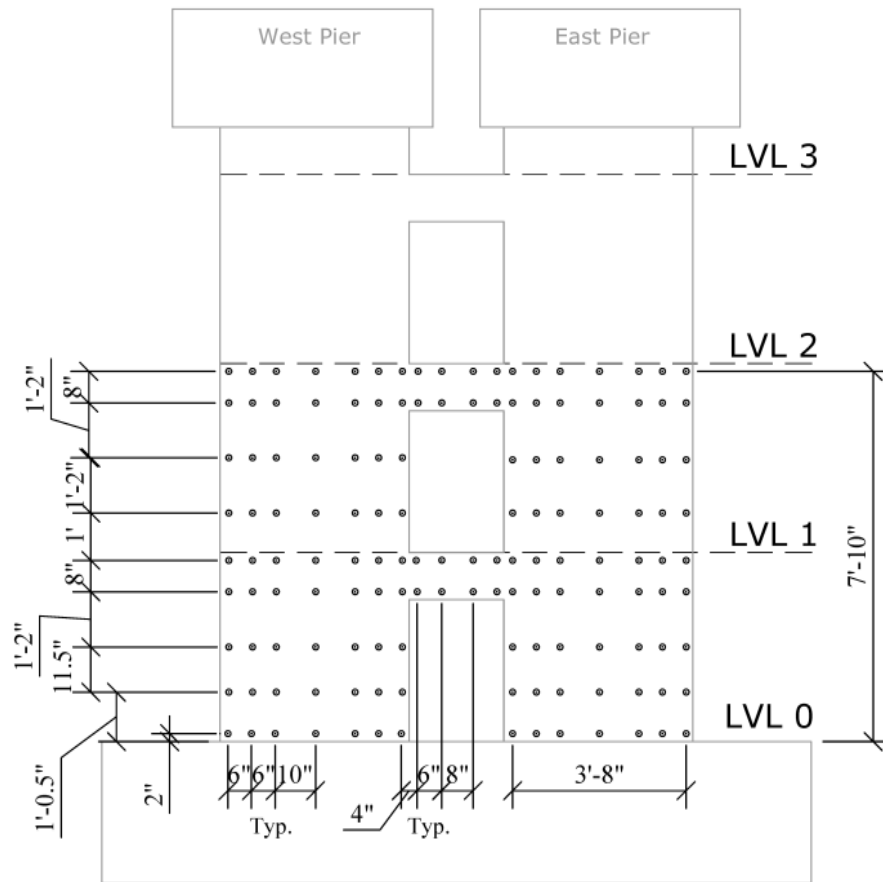


Figure 6-3: Typical layout of LED targets for Krypton measurement system.

6.1.2 High-resolution Cameras

Another advanced instrumentation system that was utilized was the use of high-resolution cameras. Nikon D80 and D90 12.1 megapixels were used to capture high-resolution photographs of the specimen. A program called the Camera Plugin was developed by the MUST-SIM facility staff to communicate with SimCor and take pictures at the completion of each load step. For the coupled wall test, 11 cameras were utilized to capture different regions of the front of the wall. Figure 6-4 shows the location of the cameras in the test setup. Paper targets were glued in 16x12-inch and 6x6-inch grid patterns on the south face of each wall pier and coupling beam, respectively (Figure 6-5).

The photos taken by the high-resolution cameras were used for digital photogrammetry, in which a collection of digital photographs taken by calibrated cameras and stitches them all together using a set of unique targets affixed to a surface. Assembly requires an overlap of targets among the photos. This can be achieved by either taking lots of pictures with one camera, or taking one picture on each of many stationary cameras. It is the second method that is used on the wall tests, with the aid of the commercial package PhotoModeler 6.

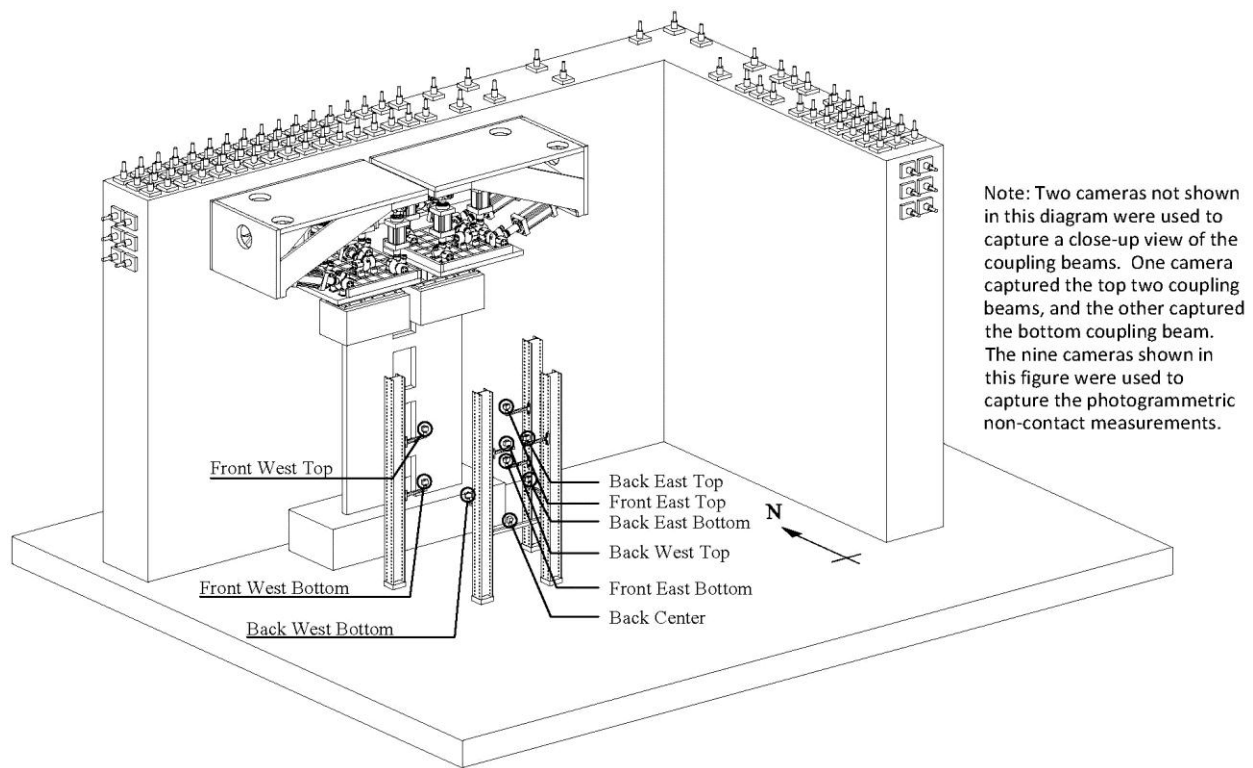


Figure 6-4: Locations of high-resolution cameras.

The finished product is a single image mapped to a defined coordinate system that removes camera lens distortions. This composite picture then provides both a single image of the specimen surface and the coordinates of all photogrammetric targets. In order for PhotoModeler to assemble the images, all targets must be visible in three different pictures from three different points of view. The positions of individual targets are calculated by triangulation, similar to the K600 system. While performing the same basic function of the K600 system, photogrammetric accuracy is lower. Post-processing can also be labor intensive as many targets need to be identified in situations of poor resolution or lighting. In the end, it is the assembled pictures themselves that is the best combination of uniqueness and usefulness to the project. Through the duration of the wall tests, crack formation tracked on the specimens' surfaces with a permanent marker. Photomodeler provides a finished product that shows the damage progression of the entire specimen, which is useful for comparison against analytical predictions or for visualizing specimen damage states. Additional information on the use of photogrammetric data is provided by Hart (2011).

While these 9 cameras were used as a part of the photogrammetry system, they were used to take high-resolution still images of the coupled wall specimen at each load/displacement step. Four cameras were used to capture close up images of the top and bottom halves of the east and west wall piers. Five additional cameras located on the back reference towers provided images of the entire specimen. Additionally, two cameras were used on the back-east reference tower to provide close ups of the coupling beams (bottom-CB1 and top-CB2 and CB3). Finally, a high-resolution "roaming" camera was used to take picture of specific areas of interest throughout the test. All the cameras are labeled with

their respective region of coverage and a four digit long SimCor step number identifier (e.g. BackCenter0001.jpg).

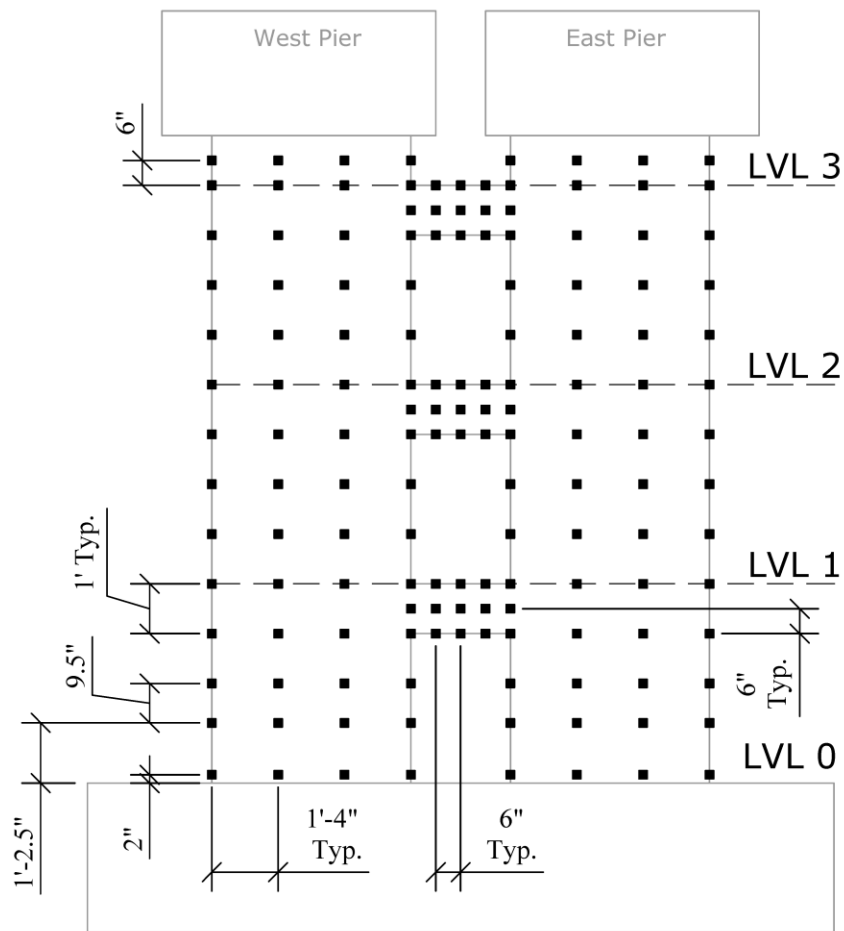


Figure 6-5: Photogrammetry target layout

6.2 Internal Instrumentation

The coupled wall specimen was heavily instrumented with quarter bridge strain gauges on reinforcing bars and the concrete surface.

6.2.1 Steel Reinforcing Gauges

The coupled wall specimen was heavily instrumented with over 200 strain gauges located on the reinforcing bars embedded in the wall piers and coupling beams. These gauges were affixed to the vertical reinforcement in the boundary elements and web region of the specimen (14 inches into foundation (Figure 6-6). The vertical spacing of these gauges tapered up the height of the specimen with a denser spacing of 6-8 inches near the base of the wall piers.

Strain gauges located on the wall pier horizontal reinforcement were located in the center of the wall piers and spaced 12 inches in the first story and 18 inches in the second story (Figure 6-7). Strain gauges located on the confinement stirrups in the inner and outer boundary elements of the west wall pier

were located seven inches in from the edge of the wall. The vertical spacing of these gauges was similar to the spacing used for the strain gauges on the horizontal reinforcing.

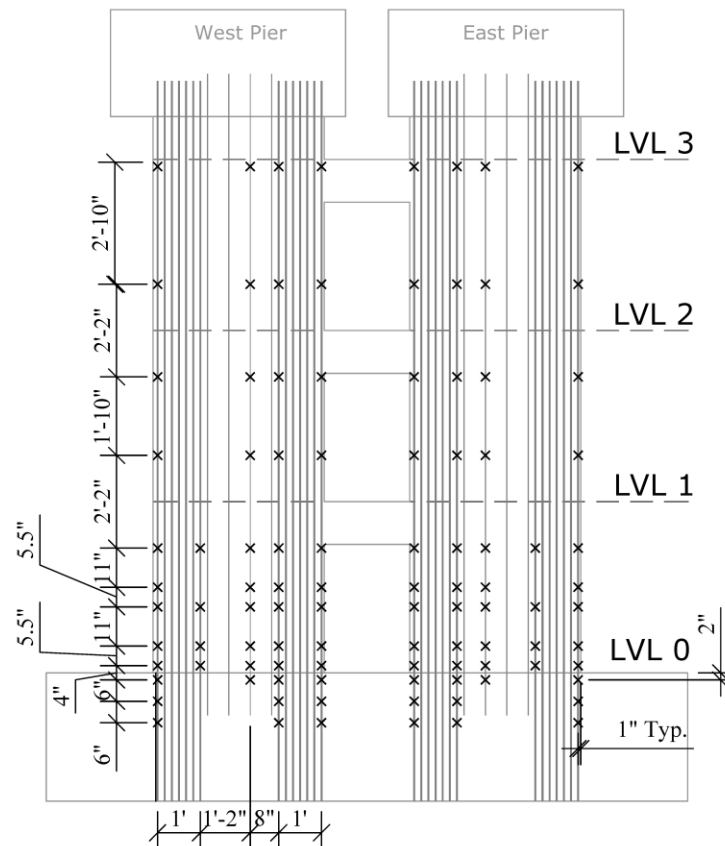


Figure 6-6: Vertical strain gauge layout

Finally, the coupling beam diagonal reinforcement was instrumented with steel strain gauges spaced approximately 4-12 inches along the bar (Figure 6-8). At the corners of the beam, the spacing between the two adjacent gauges (i.e. one in the pier and one in the beam) was approximately four inches. Additionally, four gauges were located near the center of the beam. Unfortunately, a construction error led to all the strain gauges in the first floor coupling beam to be located in the west half of the specimen.

Reinforcing bars were instrumented with Texas Measurements Inc. (TML) high-elongation strain gauges, YEFLA-5-5LT. The gauges were 5 mm in length and are shown in Figure 6-9. Strain gauges were attached to the reinforcing bars by first mechanically grinding and sandpapering the rebar to a smooth, rounded surface. The small gauge length was selected in order to minimize the effects of installing the gauge itself. The gauge location was then cleaned of any debris, and the gauge was affixed with a cyanoacrylate adhesive. The gauge was next coated with polyurethane for moisture protection and covered with a layer of butyl rubber for protection from handling damage. The strain gauge wire was finally strain-relieved by securely zip tying it to the bar.

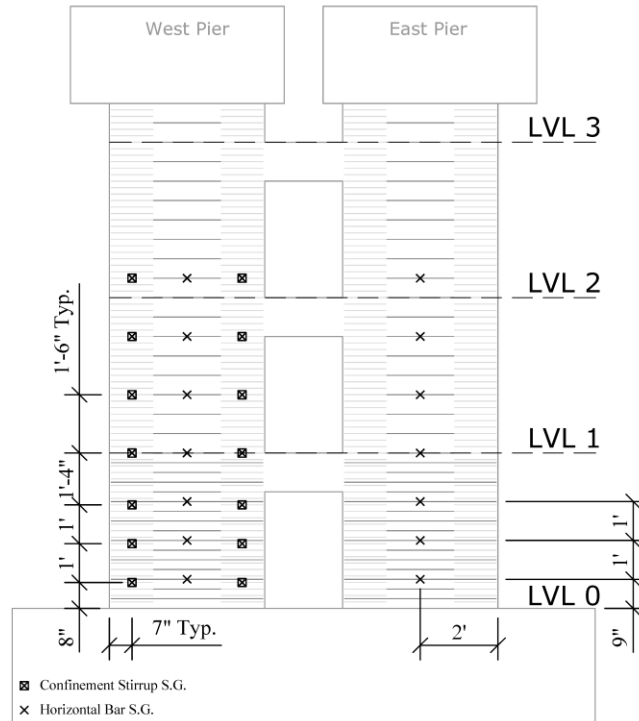


Figure 6-7: Horizontal bar and confinement stirrup strain gauge layout

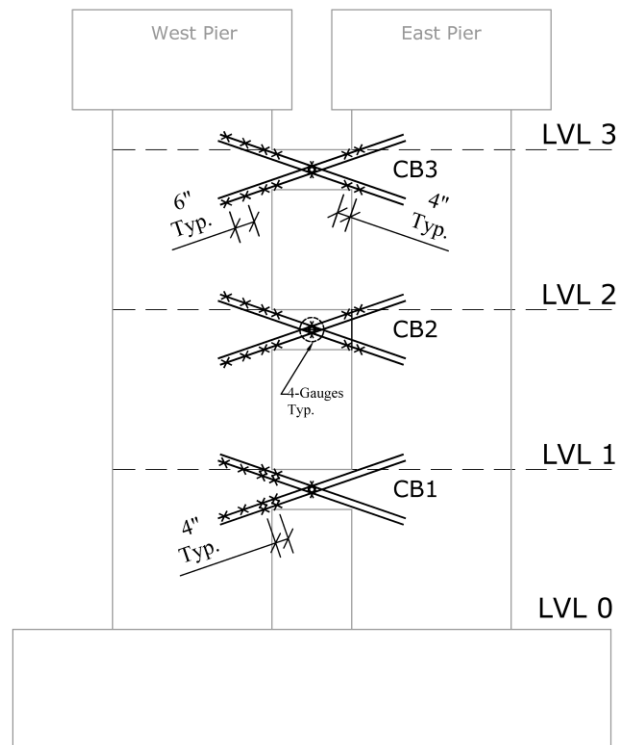


Figure 6-8: Coupling beam diagonal bar strain gauge layout

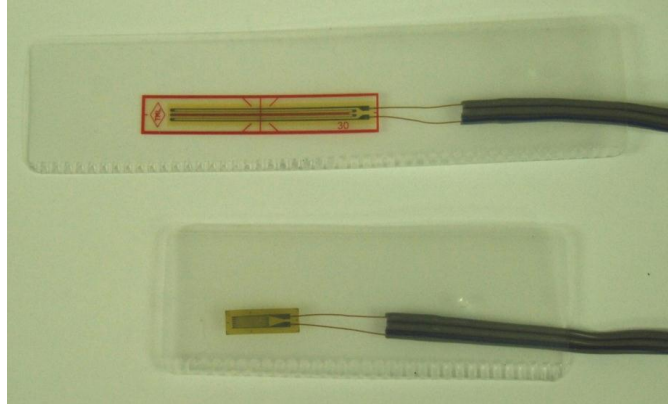


Figure 6-9: Steel gauge (below) and concrete surface gauge (above).

6.2.2 Concrete Surface Gauges

Large gauge length strain gauges were affixed to the concrete surface to measure strain on the concrete surface. The gauges were 30 mm in length and are shown in Figure 6-9. These gauges also came from TML and were PFL-30-11-5LT. A typical layout of the strain gauges affixed to the concrete surface is shown in Figure 6-10. Concrete surface gauges were attached by first cleaning the surface and applying an epoxy base layer. After setting, the base was smoothed with sandpaper and cleaned. The concrete gauge was then affixed with more epoxy onto the base and left to set. No additional protective measures were taken since concrete gauges were only installed in the final stages of test the test specimen loading preparations.

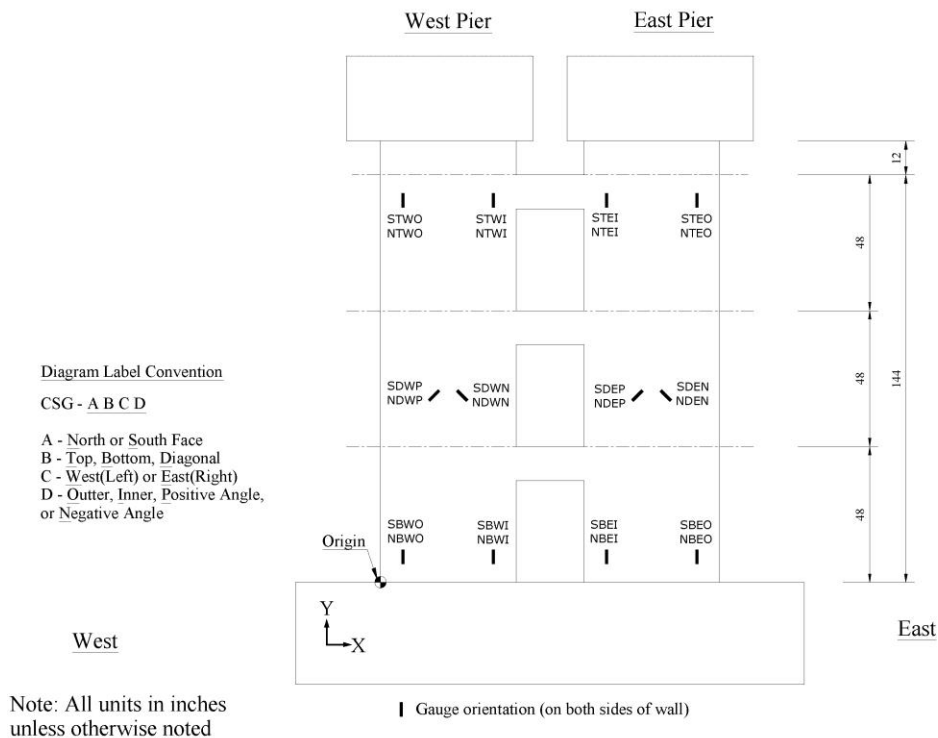


Figure 6-10: Typical instrumentation layout of concrete strain gauges.

6.3 External Instrumentation

The coupled wall specimen was heavily instrumented with external instrumentation in order to capture absolute displacement, relative displacements and component rotations.

6.3.1 Relative Displacement Measurements

Local deformations including vertical, horizontal and diagonal movements of the instrumented segments of the wall piers and coupling beams were measured using linear potentiometers located on the north face of the specimen (Figure 6-11). Because the viewing window of the Metris system did not include the third story, eight additional LVDTs were added to the ends of the top story coupling beam for redundancy.

Gauges with a 1, 2, or 6 inch gauge stroke were used depending upon predicted damage and gauge length. Figure 6-12 shows examples of the gauges used. The gauges were from Celesco and were either the CLP and MLP model. All of these instruments used direct current (DC) voltage. The gauges were mounted to a threaded post that was anchored into the wall with an embedded threaded rod that was cast in the concrete. A ball joint at each end of the instrument accommodate minor rotations, and aluminum tubing was used to limit unwanted out-of-plane buckling of the measurements (Figure 6-13).

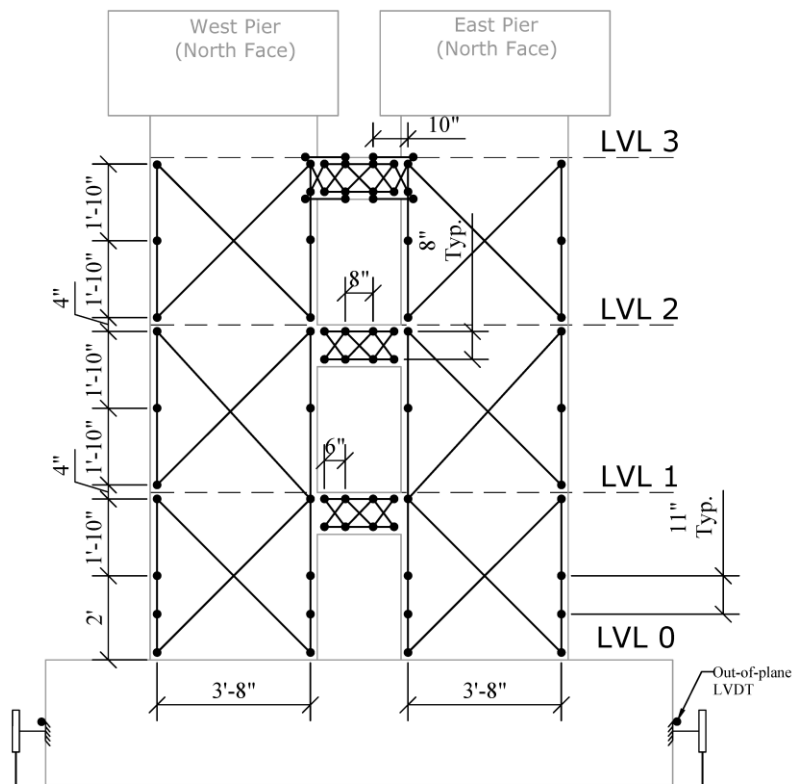


Figure 6-11: Typical instrumentation layout of relative displacement measurements.



Figure 6-12: Linear potentiometers of varying lengths.

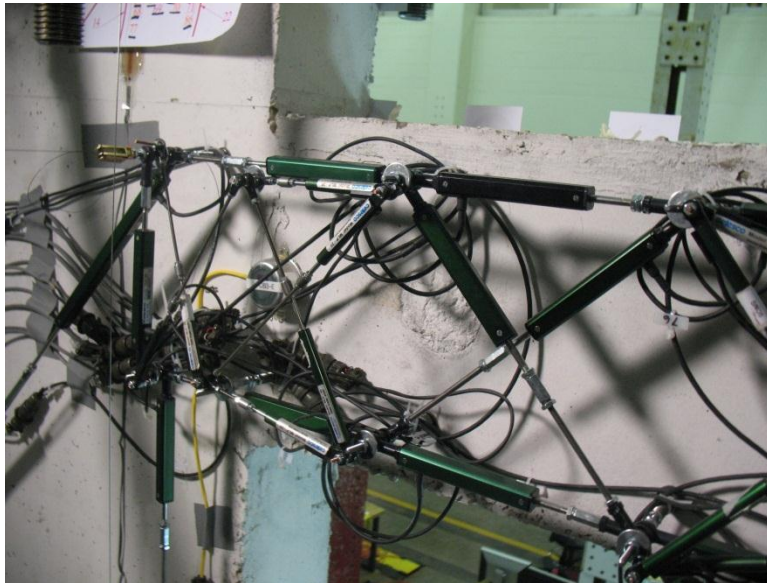


Figure 6-13: Linear potentiometer layout on third-story coupling beam

6.3.2 Absolute Displacement Measurements

A series of absolute measurements were made using either LVDTs or string potentiometers. Two types of string potentiometers were used, which are referred to as standard tension and high tension string potentiometers (shown in Figure 6-15). All the string potentiometers were from Celesco. All standard tension string potentiometers were the model PT1A, and the high tension string potentiometers were either model PT101 or PT8101.

Absolute specimen displacements were measured at several locations along the specimen height using linear string potentiometers attached to a stationary reference frame and connected to rods embedded in ends of the wall piers (Figure 6-11). Two horizontal high-tension string pots were connected to center of each wall pier (154 inches above the foundation) in order to measure horizontal displacements needed for the loading control logic (discussed in the Section 5.4). Both the diagonal and vertical string pots

were also high-tension string pots. A string pot was also connected to the foundation block to measure its horizontal movement relative to the strong floor. Additional string potentiometers (not shown) were included to measure the out-of-plane movement of the wall piers and where anchored to the strong wall and connected to the north face of the specimen.

LVDTs attached to the foundation block of the specimen and the strong floor was used to measure absolute slip and rotation of the foundation block (see Figure 6-12). These instruments were provided by the TransTek group.

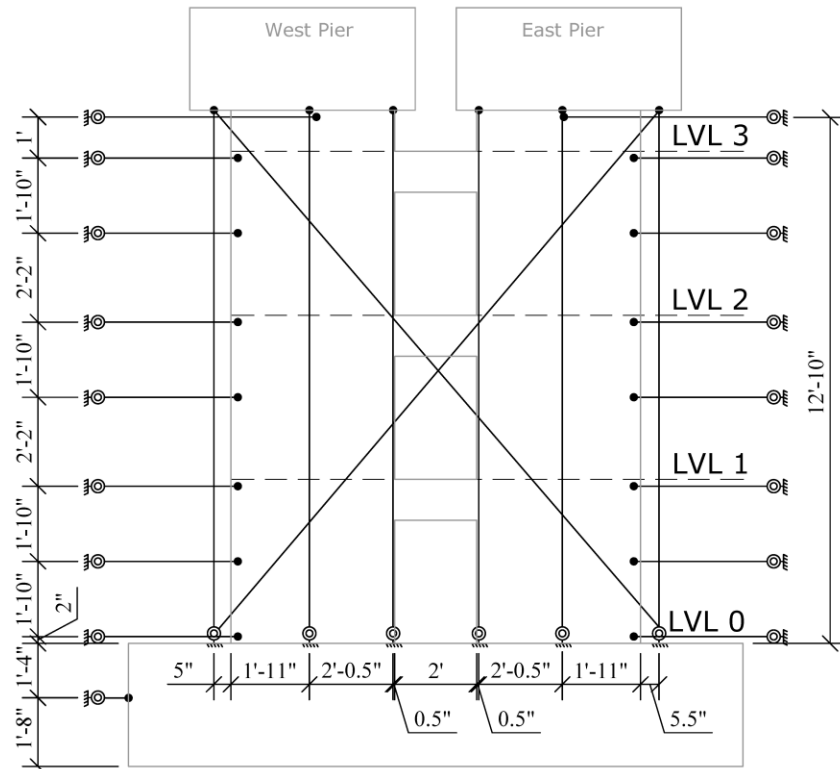


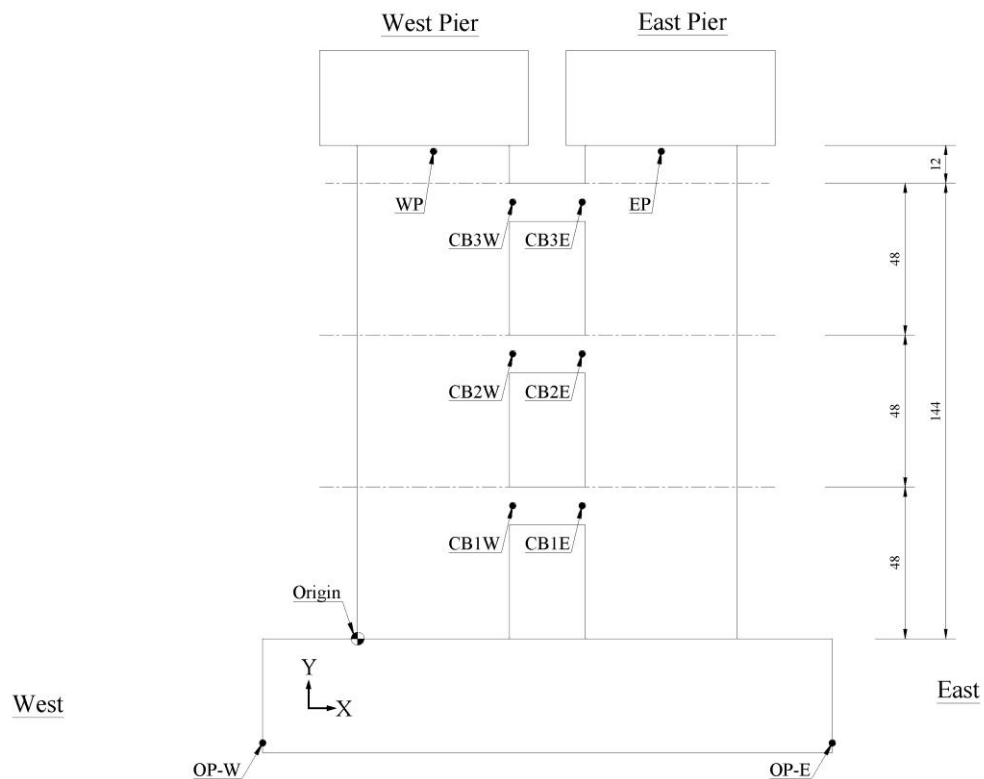
Figure 6-14: Typical instrumentation layout of in-plane absolute measurements. Out-of-plane measurements were also taken and shown in the documentation provided on NEES Project Warehouse.



Figure 6-15: High-tension (left and top) and regular (right) string pots.

6.3.3 Inclinometers

Inclinometers were glued to the coupling beam ends as well as the tops of the wall piers in order to capture absolute rotations of the coupling beam ends and the wall piers. The location of these instruments is shown in Figure 6-16. These instruments were manufactured by Spectron Systems Technology.



Note: All units in inches unless otherwise noted

Figure 6-16: Inclinometer layout for CW1

6.4 Data Acquisition System

While the measurement instruments used in the wall tests are diverse and extensive, collecting their data in a meaningful way is vital to the success of the experiments. Data acquisition at the MUST-SIM facility operates both continuously and discretely. Continuous data collection has been the norm for structural testing, with post-processing work aligning observed behavior with measured results. Such a system is manageable with a low number of instruments, but selecting individual data samples from hundreds of channels becomes a major obstacle.

The NEES DAQ software works around this problem by also recording data samples at discrete times. By linking to the control software over a network connection, the DAQ program is automatically notified when a load step is completed. When such a signal is received, the DAQ program takes a separate reading for each instrument and stores them to a “step log.” In this way, step data is readily available at the end of testing, while continuous data is also logged in case a significant event occurs between load steps.

The data acquisition (DAQ) system is made up of National Instruments (NI) hardware primarily utilizing two SCXI-1001 chassis daisy-chained together. Signal conditioning is provided by a series of modules and terminals that connect into each chassis. The modules used are the SCXI-1540, SCXI-1520, SCXI-1521b, and SCXI-1104c which are connected to the following respective terminal blocks SCXI-1513, SCXI-1314, SCXI-1317, and the BNC-2095. The two chassis are connected directly to a dedicated DAQ computer via a NI PCI-6289 card. Raw data can occupy about 2 gigabytes of disk space, with digital photos taking up about 20 gigabytes. All data is recorded locally within the MUST-SIM data repository, and instrument step data is uploaded to the NEES data repository.

In addition to the robust DAQ software, organizing and setting up a large number of channels is critical and can be quite difficult and tedious to researchers. Because of this, a script was developed to help synthesize metadata and channel parameters to eliminate tedious work that can easily lead to a channel mislabel. The researcher inputs and organizes all of the instrumentation parameters in an Excel spreadsheet that is used to generate input files for the NI hardware and NEESdaq. This information is also necessary to run Realtime Data Viewer (RDV), and organize data within project warehouse on NEEShub and DataTurbine.

6.5 Testing Notes

In order to understand the behavior and performance of CW1, observations of damage were made throughout the test. The following section provides a detailed discussion of observed damage during the test. As mentioned in Section 6.4, the data acquisition system, Metris system and high resolution photographs were triggered at each step of the loading history. Throughout the test, detailed notes were recorded when unique specimen behavior was noticed, observations of damage were made or testing issues occurred. These notes are referenced to a particular step number and are reported on the NEES Project Warehouse (see Section 8.4.4). During cycle A of the loading history, visible cracks were marked with colored pens at any step where new displacements were targeted; however, cracks were only marked at peaks during cycle B. Additionally, crack widths were measured throughout the experiment at the peak drift levels. Residual crack widths were also measured at the end of each drift

levels. These recorded crack measurements are reported in documentation located in the NEES Project Warehouse (see Section 8.4.4). Drift measurements described in the following refer to the average third story lateral drift measured using two string pots located 154 inches above the foundation as shown in Figure 6-15. Finally, steel strain gauge data was used to quantify initial yielding in the specimen. This behavior was observed when the measured strain value for a particular strain gauge exceeded the yield strain determined from the steel material properties discussed in Section 3.2.

6.6 Data Processing

As the data collected by the DAQ (see Section 6.4) collected step data throughout the duration of the tests, the amount of data processing following the test was minimal. The details on the data processing are provided for each wall specimen in documents available on the NEES Project Warehouse (see Section 8.4.5), thus the data processing is not elaborated on here. The data processing tasks necessary included:

- Combination of data files collected on different test days
- Correction of channel names
- Conversion of volt data to engineering units
- Application of offsets (setting initial instrument readings to zero)
- Removal of bad data points
- Reduction of continuous krypton data to step data

6.6.1 Krypton Data

The Krypton/Metris system collected data continuously and thus it was necessary to create “step” data that aligned with the other data collected during testing. Details on how this are discussed in Section 6.1.1

6.7 Instrument/Channel Naming Schemes

Two different naming schemes were used for the instrumentation; one was used by the research team at the University of Washington (Section 6.7.1) and one was used by the research team at the University of Illinois. Both names were combined to create the channel names provided in the sensor metadata information available in the NEES Project Warehouse.

6.7.1 UW Naming Scheme

Names were assigned to the instruments to allow for easy identification of instrument type and location. In general, the instrument names consisted of multiple characters. An example of an instrument name, for the wall pier longitudinal reinforcement strain gauge located at the bottom west corner of the west wall pier is as follows:

$$sgwl01f1a \rightarrow \underbrace{sgwl}_{type} \overset{x \text{ grid}}{\widetilde{01}} \underbrace{f1}_{floor} \overset{z \text{ grid}}{\widetilde{a}}$$

The first characters are letters that identify the type of instrumentation as well as the direction of measurement (see Table 6-1). Following the letter identification are five characters denoting the location of the instrument. This is broken down into i) a two character numerical identification, from 00 to 15, of the location of the instrument along the length of the wall, ii) the floor location, from f0 (foundation) to f3 (third floor), and iii) the location along the height of the wall within the floor, identified as a letter..

Although the intent of the instrument layout was to align the vertical and horizontal labeling systems for each type of instrument with each other, ultimately this did not happen. The naming scheme slightly varies for each instrument type (i.e. Metris grids may fall differently than strain gauge grids).

Table 6-1 lists the instrument identifier and description of the instrumentation used in the CW1 experiment.

Table 6-1: Summary of instrumentation and naming scheme prefixes.

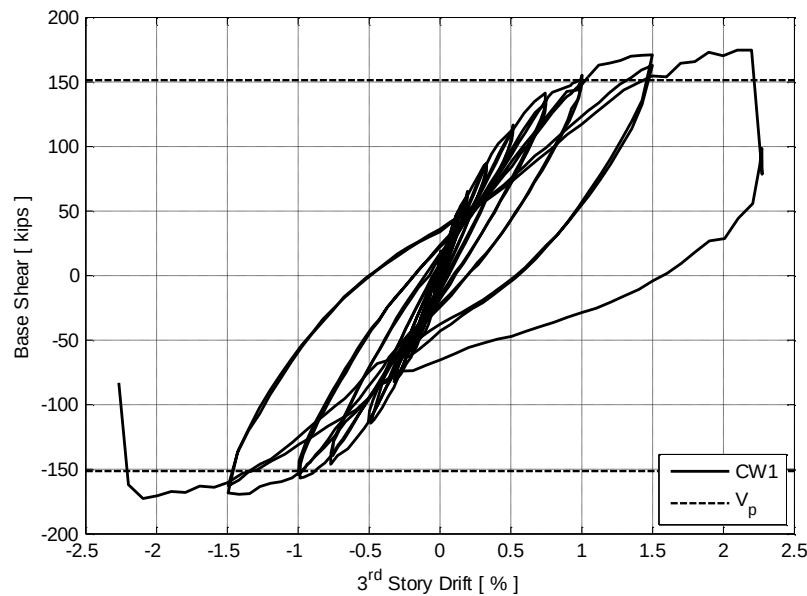
Prefix	Type	Description
SGWL	Steel strain gauge	Long. reinf. bars in wall piers
SGHP	Steel strain gauge	Horizontal web reinf. bars in wall piers
SGST	Steel strain gauge	Confinement stirrups in wall piers
SGC1,2,3	Steel strain gauge	Diagonal reinf. bars in coupling beams
SGCS	Concrete strain gauge	Concrete surface strain gauge
ADH	String potentiometers	Absolute horizontal displacement
ADV	String potentiometers	Absolute vertical displacements
ADO	String potentiometers	Out-of-plane displacements
LVDT	LVDT	Relative disp. (vertical and out-of-plane)
DTV	Linear potentiometers	Relative displacement (vertical)
DTH	Linear potentiometers	Relative displacement (horizontal)
DTP	Linear potentiometers	Relative displacement (positive angle)
DTN	Linear potentiometers	Relative displacement (negative angle)
LED	LED targets	Position targets
PT	Photogrammetric targets	Position targets
INIP	Inclinometers	In-plane inclinometer

CHAPTER 7: Experimental Results

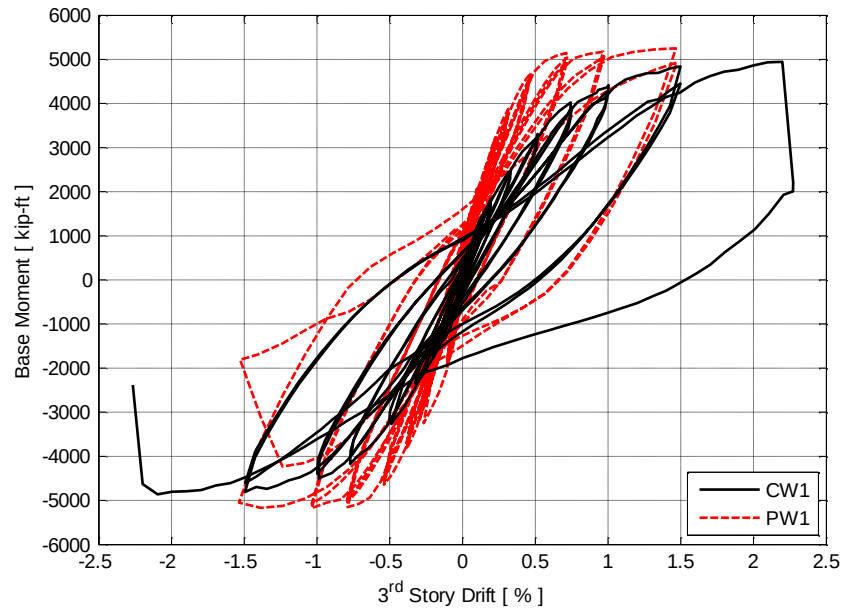
The coupled wall specimen provides first-of-its kind experimental damage and measured data of the seismic performance of modern coupled walls. These results are summarized here. Photographs and accompanying description of the progression of damage are provided. For each wall pier, the contribution of flexural, shear and base deformation to the total drift is presented. Coupling beam deformations were also investigated.

The measured base shear verse third-story drift response of CW1 is shown in Figure 7-1a and the overturning moment verse third-story drift of CW1 and PW1 is shown in Figure 7-2b. Notice PW1 is much stiffer than CW1, but since both specimens were designed with similar base shear strengths and loading using the same effective height ($0.71h_{10}$), CW1 is expected to be more flexible.

Figure 7-1 presents the load-drift response. The coupled wall reached a maximum third-story drift of 2.27%. The maximum base shear was 175 kips, which is a maximum average base shear stress demand of $4.0\sqrt{f'_c}$ psi. At 2.27% drift, the coupled wall base shear capacity dropped 55%, and upon reverse loading to its maximum negative peak, the capacity dropped 52% of the maximum peak base shear (173 kips).



a) CW1 Force-drift



b) PW1 and CW1 Moment-drift comparison

Figure 7-1: Measured base shear and overturning moment verses 3rd story drift

The following presents the progression of damage for CW1.

- Vertical cracks initiated in the top corners of CB1, CB2 and CB3 at 0.111%, 0.033% and 0.033% drift, respectively.
- Horizontal cracks initiated along the edge of the WWP and EWP at drifts of 0.129% and -0.125%, respectively.
- Diagonal cracks initiated in CB1, CB2 and CB3 at 0.324%, -0.148% and 0.103% drift, respectively.
- Diagonal cracking occurred in the web of the WWP and EWP at drifts of 0.264% and -0.237%, respectively.
- Horizontal cracks initiated in the interior face of the WWP and EWP at drifts of -0.227% and -0.224%, respectively.
- CB2 and CB3 diagonal reinforcement initially yielded in tension at 0.364% drift in the 1st cycle of the 0.5% drift level.
- Both wall piers yielded in tension in the outer boundary element at corresponding peaks during the 0.50% drift level.
- CB1 diagonal reinforcement initially yielded in tension at 0.666% drift during the 0.75% drift level
- The horizontal reinforcement in the web region of the EWP and WWP yielded during at drifts of 0.922% and 1.00%, respectively.
- Initial spalling of concrete cover located on the outer face of each wall pier occurred during the peak drifts of the $\pm 1.00\%$ drift cycle, as shown in Figure 7-2.
- Initial spalling of concrete cover located at the top corners of CB1, CB2 and CB3 at drifts of $+1.89\%$, $\pm 1.50\%$ and $\pm 0.988\%$, respectively.

- Spalling of the concrete cover exposed the vertical reinforcement in the outer boundary elements of each wall pier at the peak of the second $\pm 1.50\%$ drift cycle, as shown in Figure 7-3. At $+2.27\%$ drift the outer boundary element in the EWP sustained compression damage including extensive core crushing and buckling of all 16 vertical boundary bars.
- Yielding of the steel confinement in the WWP boundary element occurred at -1.69% (2.25% (A) drift)
- At -2.27% drift the OBE in the WWP failed in compression resulting in extensive core crushing and buckling of all 16 vertical wall boundary bars.

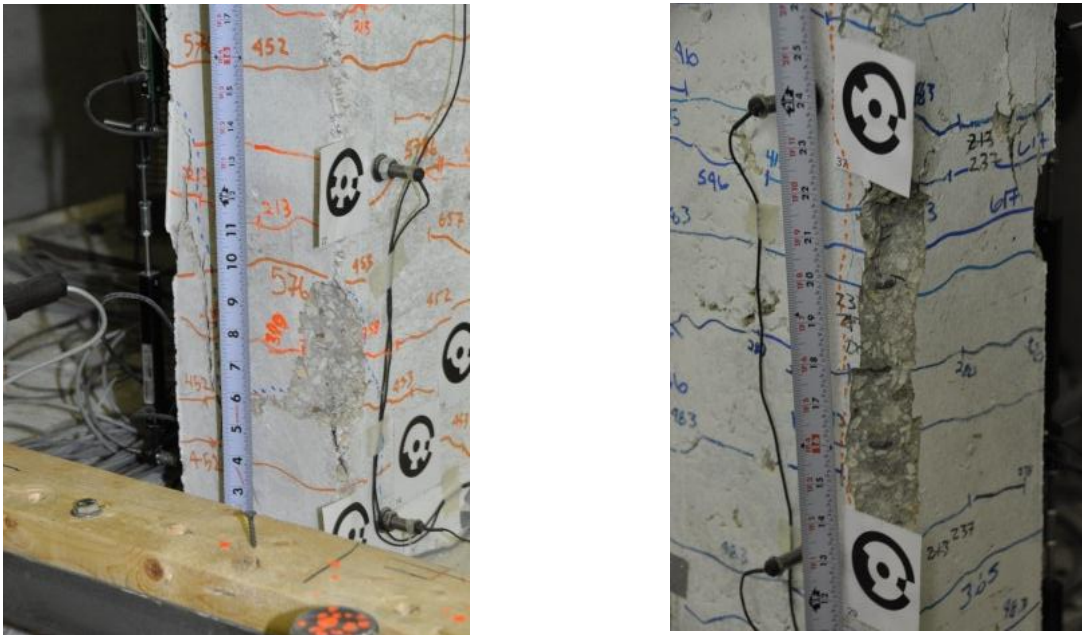


Figure 7-2: Initial spalling at base of wall piers



a) WWP at -1.5% drift



a) EWP at 1.5% drift

Figure 7-3: Exposure of Vertical Reinforcement

Figure 7-4 shows the cracking and spalling pattern of CW1 three steps prior to failure of the east pier (2.1% drift). The different shades of cracking indicate the direction of loading and the blacked out regions indicate concrete spalling. For a given direction, diagonal cracks extending from the tension wall to the compression wall indicates strong coupling between each wall pier.

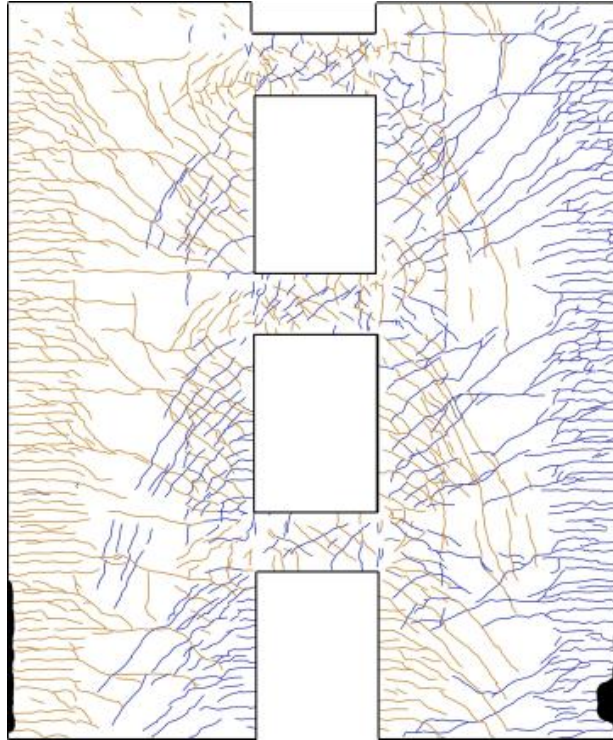


Figure 7-4: Extent of cracking and spalling at 2.10% drift

A sudden compressive failure of the EWP occurred at 2.27% (Step 955) when crushing of the concrete core and buckling of the longitudinal reinforcement cause a sudden drop in forces. All the outer boundary element longitudinal reinforcement buckled as well as two vertical bars in the web. Within the two foot region above the foundation, the toe of the wall shorted by 0.30 inches in one step. Photos capturing the sequence of failure can be seen in Figure 7-7. The CW1 wall piers crushed with no or little out-of-plane buckling.

Figure 7-5 shows the extent of damage after the failure of the WWP during the -2.25% loading half cycle. Figure 7-6 shows the final damage state of the specimen. The progression of damage is illustrated in Figure 7-9, which provides degrees of shading to indicate spalling and hatched regions to indicate yielding.

Figure 7-8 shows the close-up of damage to the coupling beams at the end of the $\pm 1.0\%$, $\pm 1.5\%$ and $\pm 2.25\%$ drift levels. Most of the damage (i.e. vertical cracking) was localized to the beam ends, and maximum measured diagonal cracks in the coupling beams never exceeded $1/16$ of an inch. Additionally, no sliding along the wall-to-beam interface was observed during the test.

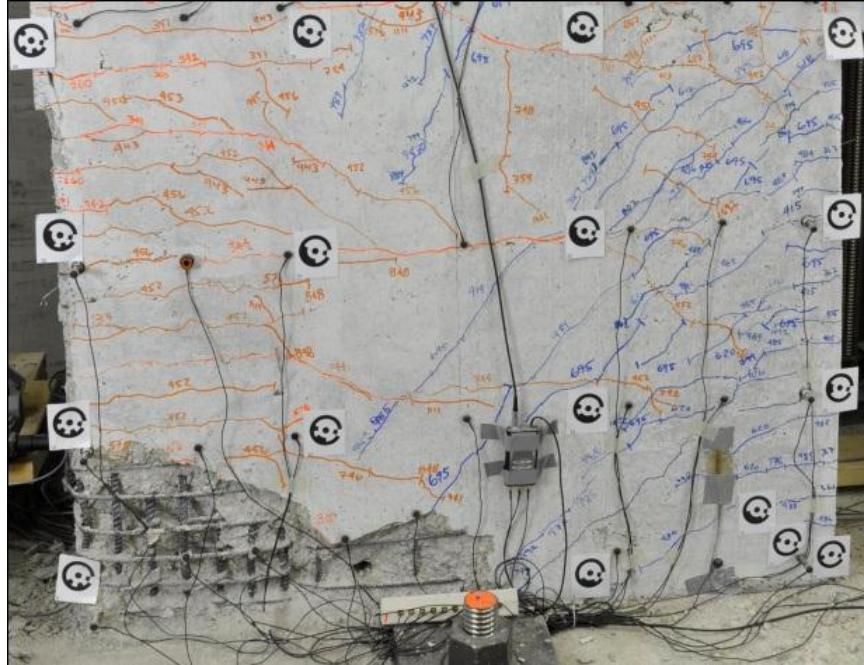


Figure 7-5: West wall pier (-2.27% drift)

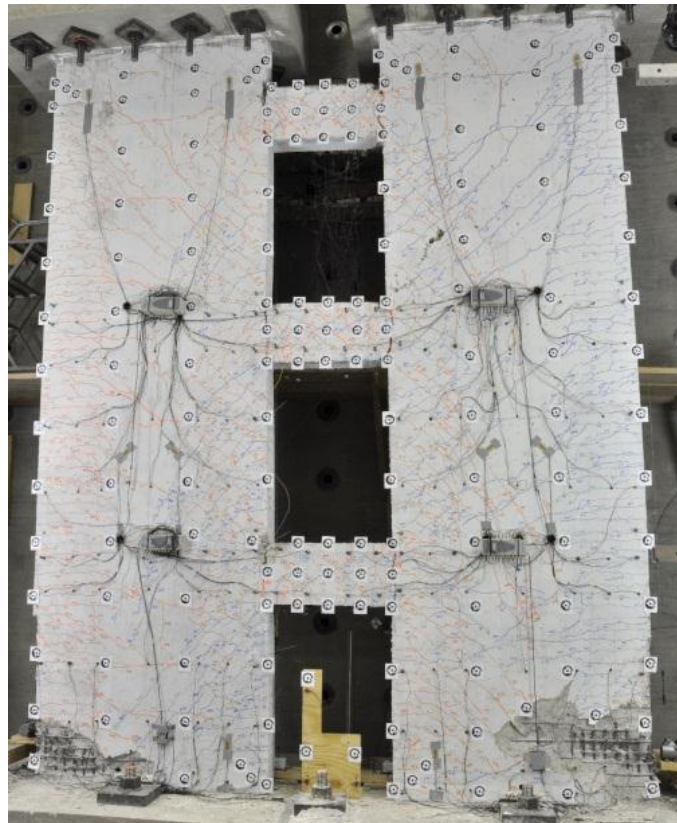
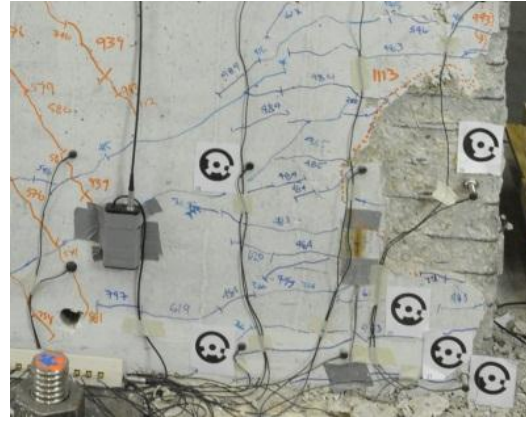


Figure 7-6: Specimen CW1 (-2.27% drift)

Due to extensive damage of the walls in the previous 2.25% cycle, cyclic loading was terminated at this drift level. The final step in the loading regime was to apply increasing equal axial load to each wall pier at the zero lateral load condition to quantify the axial load capacity (post-earthquake). The maximum axial load applied to the specimen was -1400 kips which was approximately $0.48A_gf'_c$.



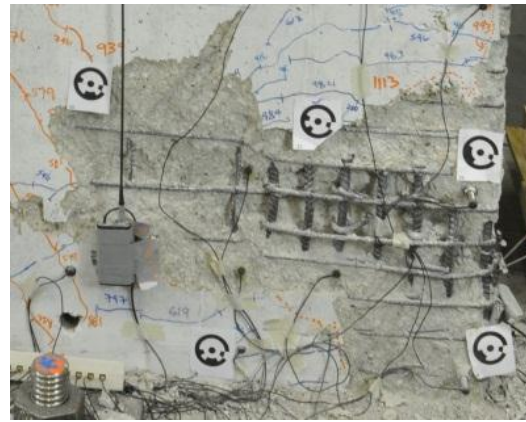
1.80% (Step 950)



2.00% (Step 952)



2.20% (Step 954)



2.27% (Step 955)

Figure 7-7: Damage sequence to +2.27% peak drift

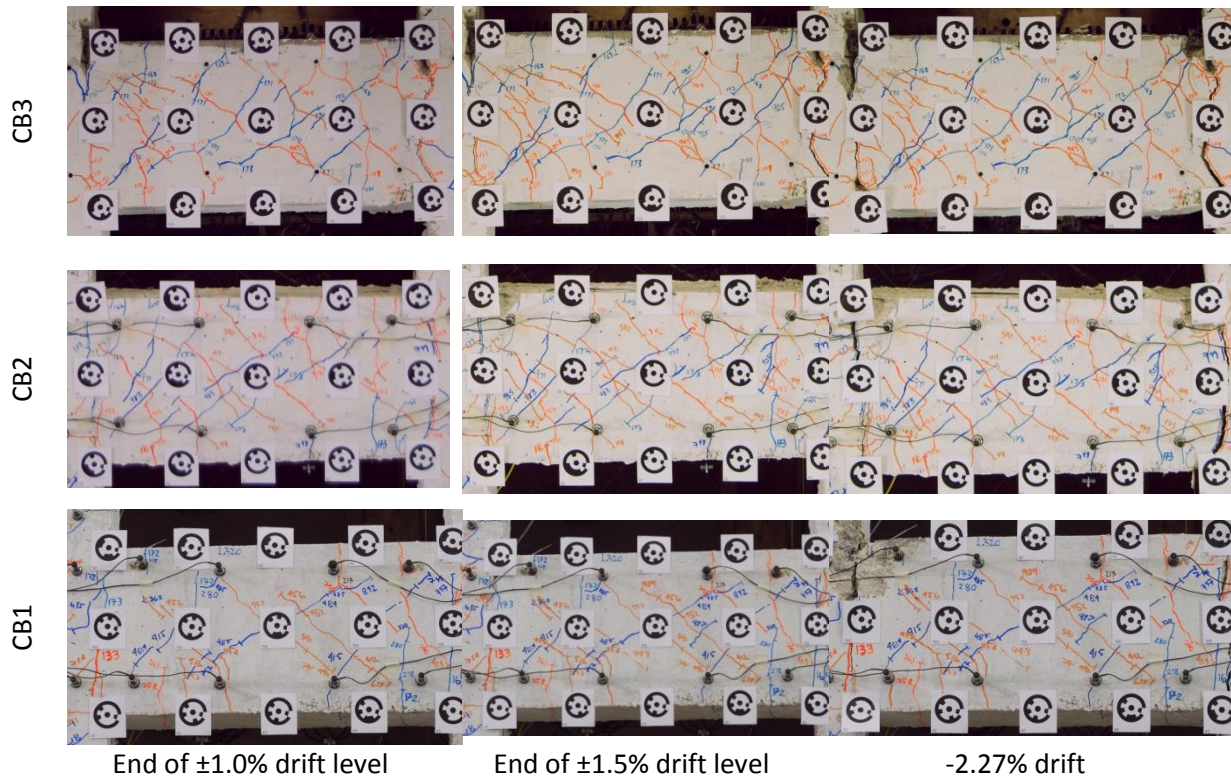


Figure 7-8: Coupling beam damage at key drift levels.

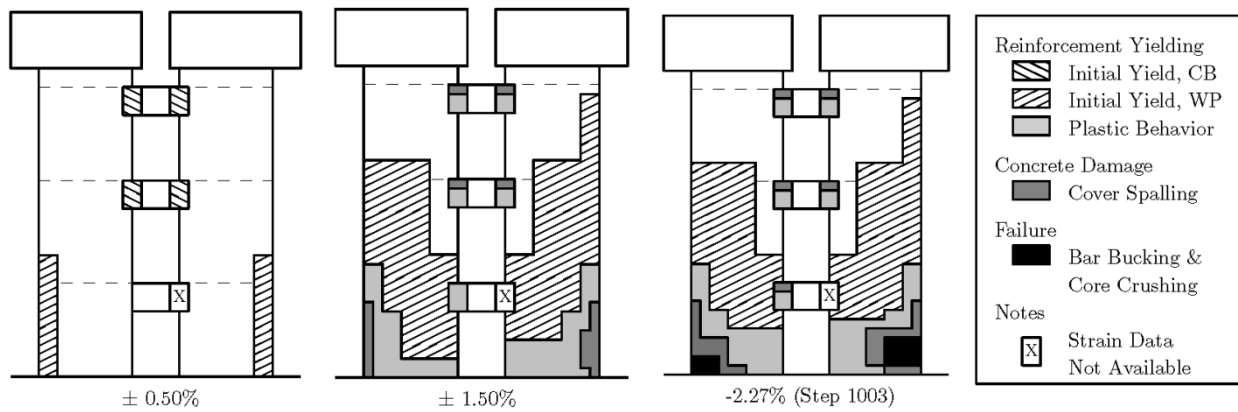


Figure 7-9: Illustration of progression of damage

CHAPTER 8: NEES Project Warehouse

A component of all NEES funded projects is the archival of data and documentation in the NEES Project Warehouse (<https://nees.org/warehouse>), the purpose of which is to provide a central data repository for the earthquake engineering research community. The project "Behavior, Analysis, and Design of Complex Wall Systems" is available in the Project Warehouse as Project #104 and can be found at <https://nees.org/warehouse/project/104/>.

This chapter provides an overview of the data and documentation available in the project warehouse. Section 8.1 discusses the data available in the data directories required by NEES; this includes photographs. Section 8.2 discusses the archival of material data. Section 8.3 discusses the archival of sensor metadata. Section 8.4 discusses the available documentation, including student dissertations. Section 8.5 and 8.6 discuss multimedia records of the experiment, such as photographs of damage and movies experimental data and analysis.

The coupled-wall test is archived in the Project Warehouse as 'Experiment-5' (Specimen CW1).

Project Warehouse file path: `./NEES-2005-0104/Experiment-5/Trial-1/Rep-1`

8.1 Data Directories

The NEES policies for data archival provide four levels of data processing, each with files contained in a separate directory. The following sections discuss each level of data processing and the relevant files for the planar wall test program. Summaries of the cleaning process are located in Documentation Directory (Section 8.4).

8.1.1 Raw (or Unprocessed) Data

This folder contains the data that has no levels of processing. This folder contains both DAQ and Metris/Krypton Data that was recorded as continuous (CONT) and at SimCor steps (STEP). Because the test was multiple days long, the data was recorded and copied into multiple directories. For example data for the first day of testing on the first run would be in director "Day1_Run1." Here each .txt file of data is slip into a header file (_HDR.txt) and a file containing the data. Unprocessed photographs are also achieved in this directory.

Formats: .txt, .jpg

Example:

`./Unprocessed_Data/DAQData/Day1_Run1/STEP/DAQ/DisplacementSensor_Inclinometers_2010_Dec_21_19_17.txt`

`./Unprocessed_Data/Photos/Camera1/Day4_Jan6/Cam0/Front_West_Top_10_0_0.jpg`

8.1.2 Converted Data

This folder contains different levels of processing, which include parsing of data files from specific DayX_RunX directories. Files included in this directory are "STEP" data only. Photographs were also renamed to match the specific SimCor step number which is provided in the testing notes (Section 0), and these photographs are achieved in this directory.

Formats: .txt, .jpg

Example: ./Converted_Data/Data/DisplacementSensor_Inclinometers_StepData.txt

./Converted_Data/Photos/FullResPictures/BackCenter/BackCenter0001.jpg

8.1.3 Corrected Data

This folder contains the data that had one more level of processing beyond that in the converted data folder. This included i) correcting of any errors in channel names, ii) removal of SimCor steps, iii) removal of bad data points, and iv) application of offsets. Each data files (.csv) has two columns of steps numbers. The load steps are what are used to reference the CW1 Test (Turgeon, 2011) and the SimCor step numbers were used during the experiment and data analysis thereafter. Cropped pictures from the Converted_Data directory are achieved here.

Formats: .csv, .jpg

Example: ./Corrected_Data /Data/Inclinometers.csv

./Corrected_Data/Photos/CroppedPictures/CB1/CB10001.jpg

The data in the Corrected_Data directories represent a portion of the final data set used for data analysis by Turgeon (2011).

8.1.4 Derived Data

This folder contains the data that was created from the collected data. This includes i) base reactions calculated from the applied loads, ii) drift values calculated from the measured lateral displacements, iii) steel stress data calculated from the measured strain gauge readings, and iv) strain data calculated from the measured in-plane displacements of Krypton LED targets and Photogrammetric Targets.

Formats: .csv

Example: ./Derived_Data /CalculatedLoadDisplacement.csv

The details of the calculation of this data are available in Turgeon (2011). No pictures are included in this directory.

8.1.4.1 Crack Maps

Under the derived data folder, crack maps of CW1 are located to illustrate the crack patterns marked with pen throughout the experiment.

Formats: .jpg

Example: ./Derived_Data /Photos/CW1_bothDir_step1115Mod2.jpg

8.2 Material Data

The material data for the coupled-wall specimen can be found by selecting by selecting the appropriate experiment on the 'Experiments' tab of the project warehouse.

For the concrete used for the walls, a description of the mix and material properties are provided.

For the reinforcing bars (No. 2 and No. 4 bars), a description of the material and material properties are provided. The measured stress-strain response of the bars is available in an .xls and .txt file format on the project warehouse.

Formats: .xls, .txt

Example: Experiments Tab → Specimen Tab → no_2_bars.xls

Experiments Tab → Specimen Tab → Wall 5 – 4rebar Properties.txt

8.3 Sensor Metadata

Under the sensors tab in the Experiment 5 (CW1) Project Warehouse, information regarding the sensors used to collect experimental data for specimen CW1 is listed. The following information is provided in this field. (Experiments Tab → Sensors Tab)

8.4 Documentation

Under the documentations tab in the Experiment 5 (CW1) Project Warehouse, documents which have information regarding the test specimen are provided. The following information is provided in this field. (Experiments Tab → Documentation Tab)

8.4.1 Design Drawings

Files of test specimen design are located here.

Formats: .dwg, .pdf

Example: CW1CADFileUS.dwg , CW1 Specimen Design.pdf

8.4.2 Test Setup

Files of the test set up are located here

Formats: .pdf

Example: CW1 Test Setup.pdf

8.4.3 Instrumentation

Files of test specimen instrumentation layout are located here. This document contains tables and figures which link the instrumentation to the appropriate instrument channel name. Additional instrumentation figures are presented in Turgeon (2011).

Formats: .dwg, .pdf, .xls

Example: CW1CADFileUS.dwg , CW1 Instrumentation Layouts.pdf, CW1 Sensor Database.xls

8.4.4 Testing Notes

Files of test specimen instrumentation layout are located here. Similar testing notes are located in Appendix B in Turgeon (2011), which is located in this directory.

Formats: .pdf, .xls

Example: TurgeonMSThesis.pdf, CW1 Testing Notes.xls

8.4.5 Data Modifications

Data modifications made for data processing directories are located here.

Formats: .pdf

Example: CW1 Data Modification Summary.pdf, CW1 Derived Data Modification Summary.pdf

8.5 Photos of Damage

Photograms of damage taken throughout the experiment are located in the respective Data Directories (Section 8.1). Additionally, photographs are presented in Turgeon (2011).

8.6 Movies

Movies were created to visually present the observed damage (time-lapse), recording of wall pier failure and other data analysis topics. These movies are located on the "NEESR Behavior, Analysis, and Design of Complex Wall Systems" "YouTube" website. (<http://www.youtube.com/user/NEESRWallProject>).

CHAPTER 9: References

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- Hart, C. R. (2011). Tensile Response of Reinforced Concrete Structural Walls under Reversed Cyclic Loading. Ph.d. dissertation, University of Illinois at Urbana-Champaign, Champaign, IL.
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- Mohr, D. S. (2007). Nonlinear analysis and performance based design methods for reinforced concrete coupled shear walls. Master's thesis, University of Washington, Seattle, WA.
- Turgeon, J. A. (2011). The Seismic Performance of Coupled Reinforced Concrete Walls. Master's thesis, University of Washington, Seattle, WA.