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LARGE-SCALE TESTS FOR PROTECTIVE ENCLOSURE OF A PIPELINE CROSSING THE HAYWARD FAULT

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ABSTRACT

A 2-m-diameter transmission pipeline has been designed to cross the Hayward Fault at approximately 47º where significant compressive deformation will be imposed on the pipeline. The design involves enclosing the pipeline in a segmental, concrete vault with joints that can accommodate the lateral offset and compressive deformation that accompany the design strike-slip displacement of 2m.This paper describes large-scale tests using the facilities of the George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES) at Cornell University to support design by evaluating the soil-structure interaction and large deformation performance of the protective structure. Five tests were performed on 1/10-scale models of the structure to investigate the response of the segmental concrete vault to abrupt fault movement. The test results are summarized, and the effects of key variables are described, including location of fault rupture relative to the protective structure joints, joint mechanical and geometric characteristics, soil density, and excavation secant walls left in place after pipeline construction.

Introduction

Tests were performed at the Large-Scale Lifelines Testing Facility at Cornell University, which is part of the George E. Brown, Jr, Network for Earthquake Engineering Simulation (NEES) supported by the National Science Foundation (NSF). The tests were performed for the San Francisco Public Utilities Commission (SFPUC) to support the design of a new large-diameter water transmission pipeline that crosses the Hayward Fault. The designer of the fault crossing is URS, Inc. Because of right-of-way (ROW) constraints the new pipeline crosses the Hayward Fault at about 47º, such that right lateral strike slip movement on the fault will cause compression in the pipeline. To accommodate the strike slip offset and compression, the pipeline will be constructed in a protective, segmental concrete enclosure at the fault crossing thereby allowing for rotation and compression of the pipeline inside the vault at ball and slip joints, respectively.

The design length of the protective vault is approximately 91.5 m, and the width, height, and length of the reinforced concrete segments are approximately 6.1 m. The joints between the concrete segments are oriented at 45 º with respect to the longitudinal axis of the culvert so that they are approximately parallel to the strike of the fault. A design displacement of 2 m was selected on the basis of recommendations of consultants retained by SFPUC.

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A critical aspect of the design is the relative movement of the protective concrete segments to accommodate lateral offset and compressive deformation during fault rupture. Understanding the relative movement of the segments is key to the design process. The tests were performed to: 1) validate the design approach, 2) provide information about the relative movement of the segments, 3) understand better interaction between the vault and secant pile walls, 4) gather data about the effects of the mechanical characteristics of the joints on vault performance, 5) obtain information about the effects of expanded polystyrene (EPS) used as fill material between the vault and secant pile walls in the zone of fault rupture.

The design concept is illustrated in Figure 1. As shown, a series of concrete segments are aligned such that fault offset is accommodated by relative movement of the segments. A novel design feature is the trapezoidal shape of the segments, in which the joints of the vault are oriented in approximately the same direction as the strike of the fault. This alignment allows for much of the compressive fault movement to be taken up by relative slip along the joints between the segments.

Test Program and Equipment

The test basin, equipment, concrete segments, joints between segments, excavation secant walls, and soil used in the tests are described in detail by O'Rourke et al. (2009) and, due to paper length limitations, only the salient features are provided herein.

Test Basin

A test basin was designed and constructed at Cornell University. It was configured to accommodate 1/10 scale models of the protective concrete enclosure designed for the new pipeline/fault crossing. Tests investigated the performance of box segments with prototype lengths of 6.1m and prototype joint openings of 150 and 300 mm.

Figure 2 shows schematic views and a photo of the test basin and NEES Laboratory at Cornell University. The movable part of the test basin was displaced by four hydraulic actuators. Since the tests were used to impose compression on the segmental culvert, the actuator movements caused the inside dimension of the box to shorten during the testing. The basin was constructed with structural steel framing and plywood sheathing. The design and drawings were prepared at Cornell, and the steel fabricated locally and erected by Cornell staff. The test basin is 3.2 m wide, 13.0 m long (in the fully aligned position) and has a maximum depth of 2.1 m. The basin is split in the center along a 50º sliding plane, which can be seen in the photograph.

Equipment

Displacement of the test basin was through use of four hydraulic actuators. The actuators were fixed to a modular structural reaction wall with segments that are longitudinally post-tensioned together and post-tensioned to threaded bars extending to the underlying bedrock. The actuators were positioned to pull the moveable section of the test basin to the east and had a combined capacity of 1.5 MN in tension. The actuators were linked and operated in displacement control and were programmed to move the test basin in 0.1 m increments (1.0 m in prototype scale). Basin movement was at a rate of 3.05 m per minute in prototype scale.

Concrete Segments

Within the test basin a 1/10 scale model of the protective concrete vault was assembled. The sections were U-shaped with the top remaining open to allow a clear overhead view of the box behavior during testing. The sections were trapezoidal and had outside dimensions of 0.61 m x 0.61 m x 0.61 m and had ends angled at 45°. The sections had 102-mm-wide walls and a 102-mm-thick base. The precast concrete sections were designed by the manufacturer in accordance with ACI-318, PCI 117 and PCI 120 [ACI (2008), PCI (1996), PCI (2004)].

Metal Cover Plates

Metal cover plates were used in four of the model tests. One type of joint cover plate was a bent U section of 16-gauge (1.5 mm nominal thickness) steel plate. This plate was 225 mm wide, 610 mm high, and slightly over 610 mm wide at the base so it could fit around the 610 mm-wide concrete sections. This type of plate encapsulated the sides and bottom of each section, and overlapped the next section by roughly a half-plate width. It was used in Tests 1A and 1B. This continuous joint cover was fabricated with angled ends so that it could be aligned parallel to the angles on the concrete sections. The plate was anchored by screws to one box section. The other cover plates used were rectangular 225-mm-wide x 610-mm-high 16-gauge metal sheets. These plates covered only the sides of the concrete sections. They were used in Tests 1C and 1D. Again, they were attached with screws to one concrete section and overlapped the joint by roughly half the plate width. Both types of cover plates were attached to the southernmost end of each concrete section, overlapping the northern end of the adjacent concrete member.

Secant Pile Walls

Excavation secant pile walls were simulated using 2 x 4 dimensional lumber (actual size 38 mm x 89 mm). Two 1-m-long pieces of lumber were joined together, making a section that was 76 mm x 89 mm x 1 m long, corresponding to a prototype scale of 760 mm x 890 mm x 10 m long. The bottoms of these structural secant wall elements then were nailed to another 2 x 4 base. There was a 64 mm gap between adjacent secant pile sections. The assemblage was nailed to the plywood floor of the test basin, with the inside edge of the 2 x 4 placed 125 mm from where the outside face of each concrete segment would be located. The base sections were split at the location of the fault crossing so they did not interfere with sliding movements at the fault location. The tops of the secant piles were painted yellow, as can be seen in Figure 2.

Soil

Partially saturated sand was used in the test basin. The soil was a glacio-fluvial sand, with a poorly graded grain size distribution. The mean grain size, D_{50}, is 0.67 mm, and the coefficient of uniformity, C_u, is 2.83. The sand was placed with an average dry unit weight = 16.0-16.2 kN/m^3 and water content = 3.5 – 4.5 %. A vibratory plate compactor was used to achieve the desired soil unit weight. The dry unit weight was measured in situ using a nuclear density gage (ASTM, 2001), and moisture content measured by conventional procedures (ASTM, 2003).

Tests were run on partially saturated sand because tests on dry sand do not replicate the great majority of field conditions in which sands are partially saturated. Testing in the dry for the same dry unit weight of sand will produce soil shear strength significantly lower than that for partially saturated sand, and thus will tend to be under conservative for cases where maximum forces between soil and structure need to be modeled. The tests procedures developed at Cornell allow for rigorous control of both dry density and moisture content through frequent nuclear densitometer measurements and techniques for controlling water content in large volumes of soil. Moreover, partially saturated sands are placed at residual water contents at which maximum shear strength associated with suction is generated.

Measurements and Instrumentation

Segment displacement measurements were made using a combination of magneto-strictive devices, string potentiometers, a laser measurement device, and LVDTs. The magneto-strictive devices and string potentiometers were mounted to the inside wall of the test basin and attached to the tops of the concrete segments and were used to measure lateral and longitudinal displacements. LVDTs were placed the measure the closure of the gaps between concrete segments. Vertical movements were measured using a laser device and a reference beam placed across the top of the basin before and after each displacement step.

Tactile pressure sensors, manufactured by Tekscan, Inc., were used in the large-scale tests. The Tekscan I-Scan Pressure Measurement System uses the 5300 Series Mat Sensors with a sensel density of 2016 sensels per mat. Each of these Tekscan mats has a measuring range approximately 207 to 241 kPa and dimensions of 488 mm by 427 mm. The sensors were used to measure axial loads for concrete segment end sections in Test 1A and horizontal loads on the side of select segments in the other tests. Sensors were installed and protected during use in accordance with procedures described in Palmer et al. (2009).

Test Assembly

The two sections of the empty test basin were aligned and the fault crossing was checked for tolerance and integrity. The secant walls were fastened to the plywood test basin floor. Roughly 380 mm of compacted sand was placed in equal lifts. Prior to placing the concrete sections, 13 mm plywood panels, cut to the same geometry as the bases of the concrete section, were positioned on the compacted sand. When used, the metal cover panels were installed on the concrete sections, and the sections set into place with the proper joint spacing and alignment. Some tests had the joint between concrete sections centered on the fault and others with the fault crossing at the middle of the concrete section. One test had the joint located one-quarter length beyond the fault location. The main parameters for each test will be described in a latter section of this report. Once the concrete sections were in place the remaining backfill was compacted in nominal 200 mm lifts until the soil was level with the tops of the secant pile walls and tops of the concrete sections.

Table 1 lists the five tests that were performed and provides information including the date and characteristics of each test. Each test involved five displacement steps of 1.0 m each. As indicated in Table 1, Tests 1C through 1E used expanded polystyrene (EPS) packing pellets as backfill between the concrete sections and the secant pile walls. When the EPS was used, thin polyvinyl plastic sheeting was placed between the concrete and secant walls, the EPS placed to a depth of 610 mm, then manually compressed roughly 125 mm. Sand then was placed over the EPS particles to provide a low level of vertical confinement.

A test program objective was to model the stiffness contrast between the soil at site and the tabular sections of EPS that would be installed in the field. The soils at the fault crossing are stiff to hard clays and relatively dense sands, whereas those used in the lab were medium dense sands. To match the stiffness contrast in the lab with that in the field it was necessary to select and prepare EPS for the tests with lower stiffness than EPS in the field in the same proportion as the relative stiffness of medium dense sand and the soil at site. O'Rourke et al. (2009) describes the EPS and soil compression testing that was used to select and prepare EPS such that the stiffness contrast in the tests was consistent with that in the field.

Summary of Test Results

The presence of the vault affected the ground rupture pattern in a consistent way for all tests. During fault movement, soil moved into contact with the sides of the vault opposing right lateral strike-slip displacement on either side of the fault. Zones of compressive soil movement and partial passive earth pressure were mobilized. On the sides of the vault opposite the compressive movement, soil moved away from the vault, leading to zones of extension and active earth pressure. The zones of soil extension were marked by secondary ground ruptures at an oblique angle to the fault that formed in response to tensile and shear strains in the soil. The zones of soil extension were confined to a distance along the vault of approximately 1 to 1.5 segment lengths either side of the fault. Figure 3 shows this ground rupture.

All tests showed relatively small levels of vertical segment movement and rotation about the lateral and longitudinal axes of the vault. Vertical segment movements for Tests 1B and 1E are shown in Figure 4. The rotations about the vertical axis (in the horizontal plane) were limited to a maximum 4 to 6º and distributed within 3 to 4 segments either side of the fault. These rotations developed gradually without sudden or concentrated deformation in response to increasing fault displacement. Segment rotations for Tests 1B and 1E are shown in Figure 5. As shown, the replacement of soil with EPS between the secant piles and the segments resulted in more segment vertical movement and segment rotation

The deformation of the vault in the horizontal plane was affected by 1) location of the fault as either centered on a segment or centered on a joint, 2) rotational stiffness of the joints, and 3) nature of the backfill between the vault and the secant pile wall. The narrowest distribution of segment rotations, conforming to the most pronounced deformation of the vault, occurred when a joint was centered on the fault. The widest distribution of segment rotations, conforming to the most gradual deformation of the vault, occurred when a segment was centered on the fault. The most flexible joints without steel cover plates resulted in the highest measured rotation and most pronounced deformation of the vault. The use of EPS pellets as backfill between the vault and secant pile wall increased the maximum segment rotation slightly and broadened the distribution of rotation relative to the condition where partially saturated sand was used as backfill between the vault and secant pile wall.

Small to moderate levels of heave (75-200 mm at prototype scale) were observed in all tests, and are related to increased volume, or expansion, of soil near the fault in response to shear during fault rupture. Because the soils at the pipeline crossing of the Hayward Fault are composed mostly of stiff to hard clays and dense sands, they will also expand in volume when sheared during fault rupture.

The highest lateral pressures against the vault segments were measured in tests where partially saturated sand was placed between the vault and secant pile wall. The highest pressures associated with fault displacements ranging for 0.5 to 1.5 x the design offset of 2 m were approximately 10-15 % of the maximum horizontal passive pressure. Even for very large fault offsets approaching 5 m, the highest measured pressures were less than 1/3 of the maximum horizontal passive pressure. Figure 6 shows tactile pressure sensor measurements for Tests 1B and 1D and reveals the reduction in lateral pressure on the segments when soil between the secant wall and the segments is replaced with EPS.

The relatively low measured pressures relative to the maximum passive limits are the result of the secant pile wall, which acts as a barrier between the vault and soil on the outboard side of the wall. During fault rupture, soil moving relative to the vault is resisted by the secant pile wall, resulting in shear transfer between the soil and intact structural wall elements. This interaction between the deforming soil and secant pile wall reduces the pressure that can be transmitted to the vault to a relatively small fraction of the maximum horizontal passive pressure. The lowest horizontal pressures were measured in tests where EPS pellets were used as backfill between the vault and secant pile wall. Pressures in these tests were equal to and less than 1% of the maximum horizontal passive pressure, and less than 10% of the lateral pressure mobilized when saturated sand was placed between the vault and secant pile wall.

During the model tests, compressive shortening of the vault caused progressive closure of the joints that spread further from the fault as the strike-slip displacement increased. For levels of fault displacement ranging from 1 to 3 m (Steps 1 through 3), partial joint closure was observed as far as four to five segments either side of the fault. However, full joint closure to cause contact between adjacent concrete segments was generally confined within three to four segments either side of the fault. At no time during the tests, including fault movement of 5 m, did full closure of the joints progress to the ends of the vault. Tactile pressure sensors positioned on the end surfaces of the end segments showed little to negligible increase in lateral soil pressure during testing.

Concluding Remarks

The test program shows the importance of full-scale testing in the design of critical lifeline projects. The test results provided validation of the design concepts as well as guidance with respect to the effects of key variables, including the location of fault rupture relative to the protective structure joints, joint mechanical and geometric characteristics, soil density, use of EPS to reduce lateral soil pressures, and excavation secant pile walls left in place after pipeline construction. The test program demonstrates the value of industry/academic collaboration through large-scale testing to support and enhance design.

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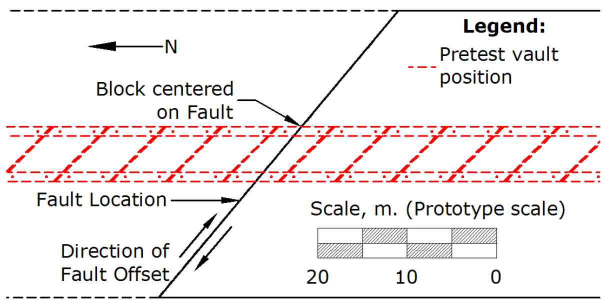

a) Before Fault Offset b) After Fault Offset

Figure 1. Design concept for protective enclosures in test program Note: 1 ft = 0.305 m

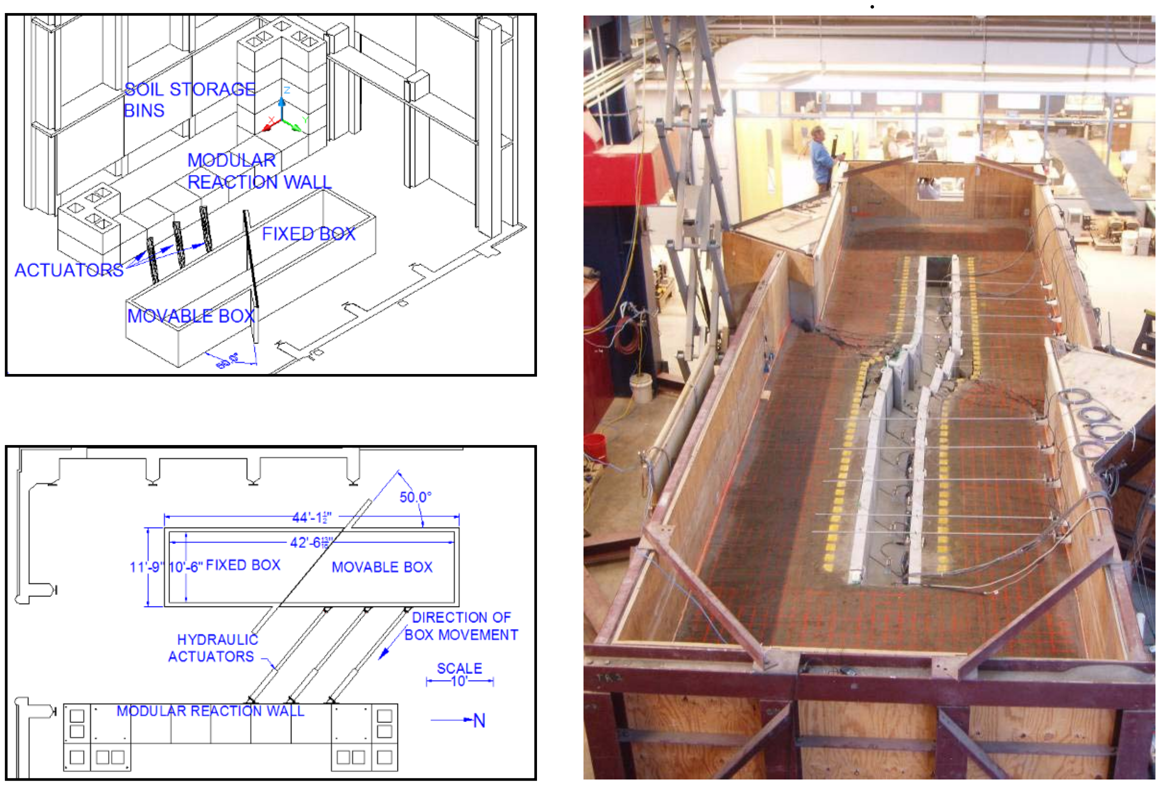

Figure 2. Schematic views of Cornell NEES facility and photo of test basin taken after a displacement step.

Table 1. Summary and Characteristics of Tests Performed

Test No.	Test Date	Fault Location	Number of Segments	Cover Plate Configuration	Segment Gap Prototype Scale	Fill between Secant Wall and Segments
1A	Feb. 12, 2009	Centered on Segment	13	Cover plates at base and two sides of each joint	150 mm	Soil
1B	Mar. 31, 2009	Centered on Joint	12	Cover plates at two sides of each joint.	150 mm	Soil
1C	Apr. 23, 2009	Centered on Segment	13	Cover plates at two sides of each joint	150 mm	EPS
1D	May 13, 2009	Centered on Joint	12	Cover plates at two sides of each joint.	150 mm	EPS
1E	May 28, 2009	5 ft North of Center Segment	13	No cover plates	300 mm	EPS

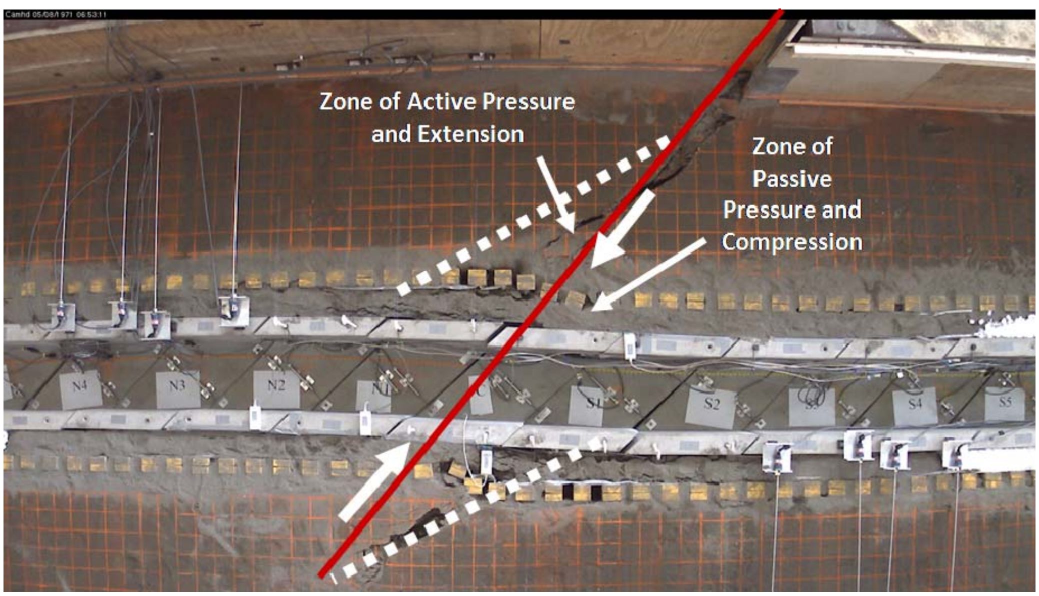

Figure 3. Concrete segment and secant wall effects on ground rupture patterns

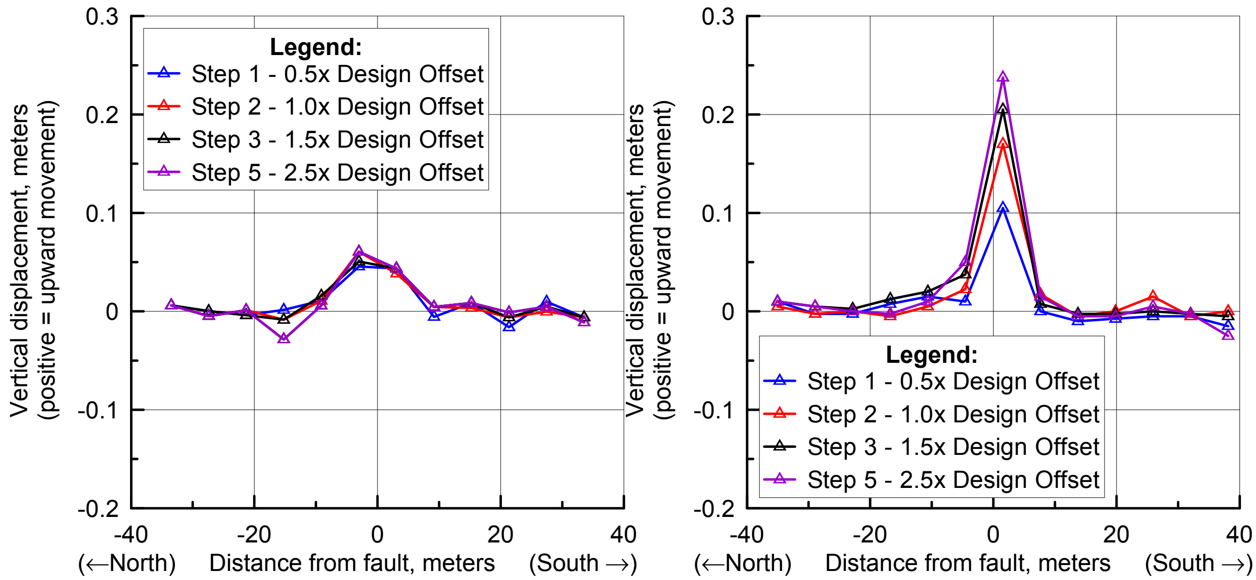

Figure 4. Vertical segment movements for Tests 1B and 1E

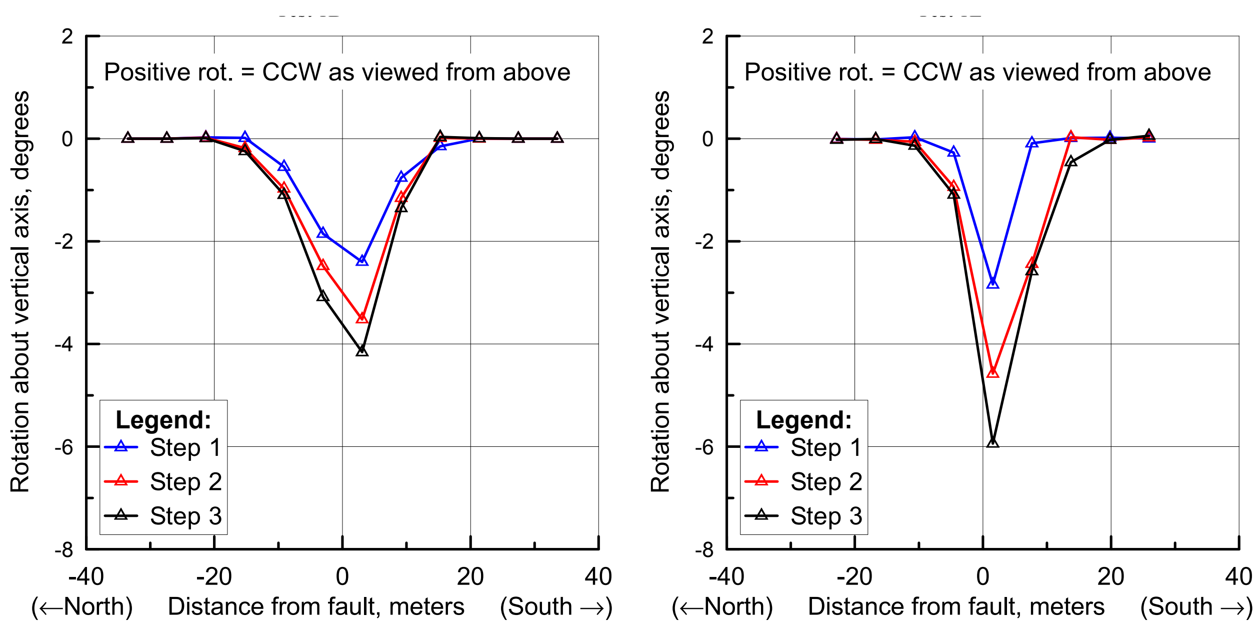

Figure 5. Segment rotations about vertical axis for Tests 1B and 1E

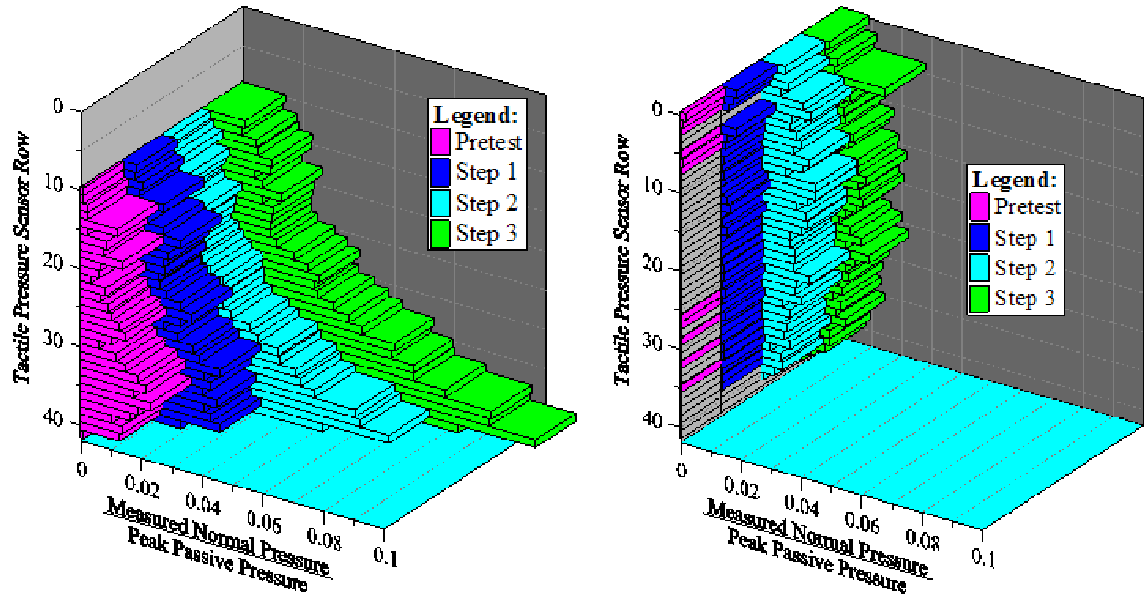

Figure 6. Lateral Pressures for Tests 1B and 1D

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