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Monitoring a Drilled Shaft Retaining Wall in Expansive Clay: Long-Term Performance in Response to Moisture Fluctuations

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ABSTRACT

A full-scale drilled shaft retaining wall was constructed in Manor, Texas on a site underlain by approximately 15 meters of the highly overconsolidated, expansive Taylor Clay. This wall was instrumented with inclinometers and fiber optic strain gauges. During a four-year monitoring period, performance data were recorded during construction, excavation, seasonal moisture fluctuations, and controlled inundation testing which provided the retained soil with access to water at the ground surface.

This paper discusses performance of the test wall over a range of soil moisture conditions, including an extremely dry state caused by the most extreme drought on record in this area and the wettest possible state when wall deflections ultimately reached equilibrium after eight months of controlled inundation. The wettest possible state corresponded to the development of drained conditions in both the retained soil and the foundation soil. The maximum earth pressures applied by the retained soil were comparable to active conditions mobilizing the drained, fully-softened shear strength; there was no evidence of greater pressures being applied as the soil swelled to its ultimate equilibrium condition. The measured p-y curves for the foundation soil were consistent with passive conditions mobilizing a drained shear strength between the peak and fully-softened strengths.

INTRODUCTION

Cantilever drilled shaft retaining walls are well suited for use in urban environments where noise, space, and damage to adjacent structures are major considerations. The design of drilled shaft retaining walls has evolved over time. While initial design methods were based on limit equilibrium calculations, more refined p-y analyses based on soil-structure interaction were subsequently developed and are currently in use by the Texas Department of Transportation (TxDOT) (Wang and Reese 1986; TxDOT 2009).

There is uncertainty in how to account for lateral earth pressures acting on drilled shaft walls installed through expansive clay. In Texas, some of the most problematic expansive clay deposits are also highly overconsolidated. Therefore, an examination of retaining wall design procedures for overconsolidated, highly plastic clay can provide a reference point for the design of walls in expansive clay deposits.

Commonly, the earth pressure on walls in stiff, overconsolidated clay is estimated using Coulomb active earth pressures with drained properties. The TxDOT design procedure for cantilever drilled shaft walls employs this method with a recommended friction angle of 30° for "medium to stiff clays" (TxDOT 2009). For clays commonly encountered in Texas, this approach results in earth pressures that correspond to an equivalent fluid unit weight of approximately 5.5 to 6.3 kilonewtons per cubic meter (kN/m^3). The p-y curves for the foundation soil are modeled as "stiff clay without free water" (TxDOT 2009) using in situ undrained soil shear strength measured before excavation and reduced to account for stress relief of the excavation.

SITE CONDITIONS

The project site is located in Manor, Texas, on a site underlain by approximately 15 meters of the Taylor Formation, an overconsolidated, stiff to hard, highly plastic clay. The upper 1.8 meters of clay is weathered and dark brown, while underlying unweathered clay is yellow. The groundwater table is approximately 2.4 meters below the ground surface, based on data from an on-site piezometer.

Three 15-meter deep soil borings were drilled in January 2010, a relatively wet season. Laboratory testing included conventional one-dimensional consolidation tests, cyclic shrink-swell tests on horizontally oriented specimens, Unconsolidated-Undrained (UU) triaxial compression tests, and direct shear tests. Details for these tests and their results are provided in Ellis (2011).

Results of the site investigation testing are shown in Figure 1. The drained strength results of the direct shear testing are shown in Figure 2. Based on these data, the peak, drained shear strength can be represented by a friction angle of approximately 37º and the fully-softened, drained shear strength can be represented by a friction angle of approximately 24º (Figure 2).

TEST WALL DESIGN AND CONSTRUCTION

The test wall was designed using standard of practice methods, with the goal of providing a structure consistent with typical TxDOT walls, while producing enough deformations to infer the earth pressures acting on the wall. The test wall consists of 25 drilled shafts (Figure 3), each with a diameter of 0.61 meters and a center to center spacing of 0.76 meters. The reinforcing bar cage consists of 12 #7 bars. At the centerline, the cantilevered height is 4.57 meters, the penetration depth is 6.10 meters below the cantilever, and the shafts end four feet above ground surface (Figure 4).

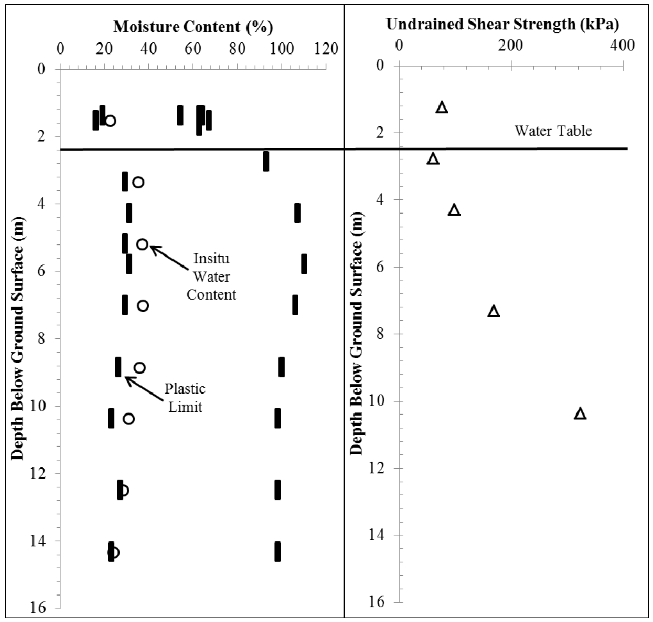

Figure 1: Results from site investigation testing from January 2010 (3 months before shaft construction; 7 months before excavation).

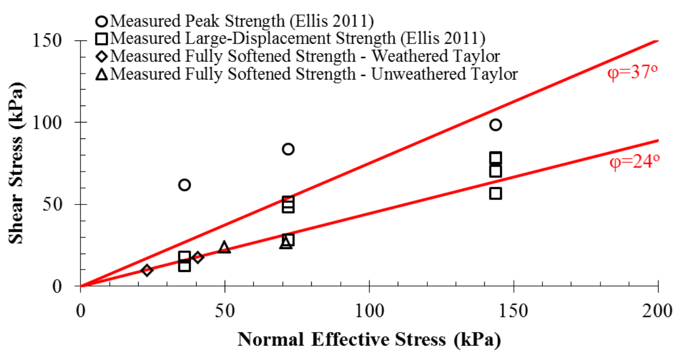

Figure 2: Test results for drained shear strength of Taylor Clay.

Three shafts in the center of the test wall are instrumented (Figure 3). In each of these shafts, there are 30 fiber optic strain gauges and one inclinometer casing (Figure 5). On the tension and compression sides of an instrumented shaft's neutral axes, 15 fiber optic strain gauges are placed on two foot intervals at depths of 0.30 to 8.84 meters. Additionally, three thermocouples were installed in the center shaft at depths of 0.91, 4.57, and 8.84 meters below ground surface; one inclinometer casing was installed 1.70 meters behind the wall; and a linear displacement potentiometer was attached at the top of the wall. The drilled shafts and instrumentation were installed in early April 2010, and the excavation was made in August 2010.

An inundation berm was constructed in April 2012. The inundation berm encloses an area approximately 12.19 meters wide and 6.10 meters behind the test wall (Figure 6). The berm is keyed into the native soil with a 0.61-meter deep trench and is lined on the perimeter with a geomembrane to minimize lateral loss of water.

Five stand pipe piezometers were installed on the project site. In January 2010, a piezometer with a screened interval 1.50 to 4.60 meters below ground surface was installed in a boring from the geotechnical site investigation. This piezometer was used to monitor the local groundwater level. In February 2012, four more piezometers were installed in boring holes in order to monitor the water levels in the inundation test area (Figure 6).

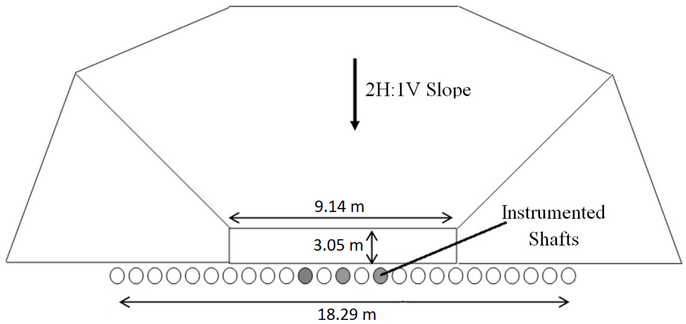

Figure 3: Plan view of wall and excavation design.

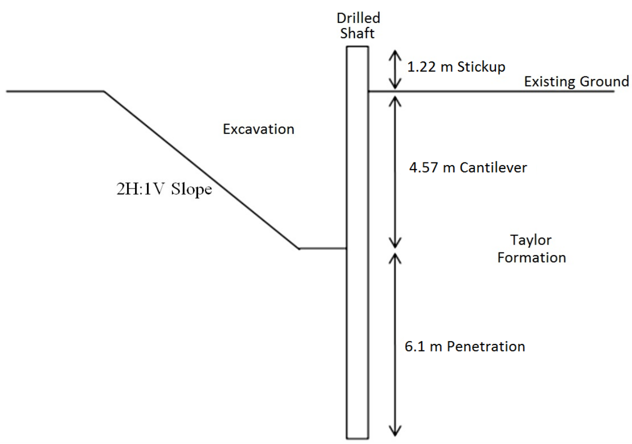

Figure 4: Cross-section of wall and excavation design.

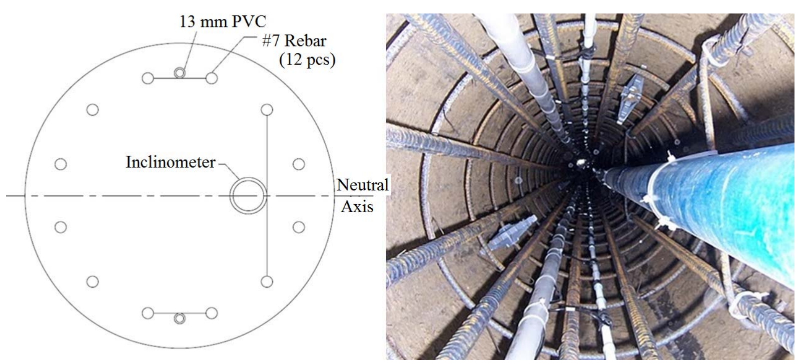

Figure 5: Instrumented cage before concrete placement.

Figure 6: Inundation berm with four stand pipe piezometers behind the wall.

TEST WALL PERFORMANCE

The completion of the excavation and installation of the shotcrete facing material was finished in October 2010. Between October 2010 and April 2012, the wall experienced a range of climatic conditions, which were reflected in the observed wall movements (Figure 7). Because the application of the facing represents a practical "zero" value for wall performance, subsequent test wall measurements are referenced here to the first survey after the installation of facing material, October 8, 2010. Information about stresses and displacements in the wall during construction and excavation are presented and discussed in Brown et al. (2011 and 2014).

After the facing installation was completed in October 2010, the test wall experienced approximately three months of below average rainfall, followed by a series of storms in January 2011 which briefly inundated the excavation area. During the spring and summer of 2011, the test wall experienced an extended dry period, widely reported to be the most severe one-year drought on record in central Texas. During this record-breaking drought, top-of-wall deflections decreased (moved inwards) and reached a minimum average top-of-wall deflection of -5.1 millimeters relative to the installation of facing material (Figure 7 and Figure 8).

In 2012 and 2013, controlled inundation was performed behind the wall in order to create an upper-bound loading condition. Inundation was performed by keeping a constant water level in the berm behind the wall. By increasing soil moisture content behind the wall to an upper-bound condition, the influence of soil wetting and expansion on the earth pressures can be more readily estimated. Beginning in May 2012, the retained soil was provided unlimited access to water for 2 months, followed by a 7-month drying cycle. In February 2013, the retained soil was subjected to a second inundation cycle until the top-of-wall deflections reached equilibrium, a period of approximately 4 additional months. The wall deflections reached equilibrium at approximately the same time as the ground water level behind the wall reached equilibrium near ground surface. Top-of-wall deflections during inundation testing, and deflection profiles at key dates, are shown in Figure 7 and Figure 8. Data from measured displacements at the top of the wall and the deep inclinometer behind the wall were used to confirm that the base of the shafts did not translate laterally after excavation for the purposes of interpreting inclinometer data.

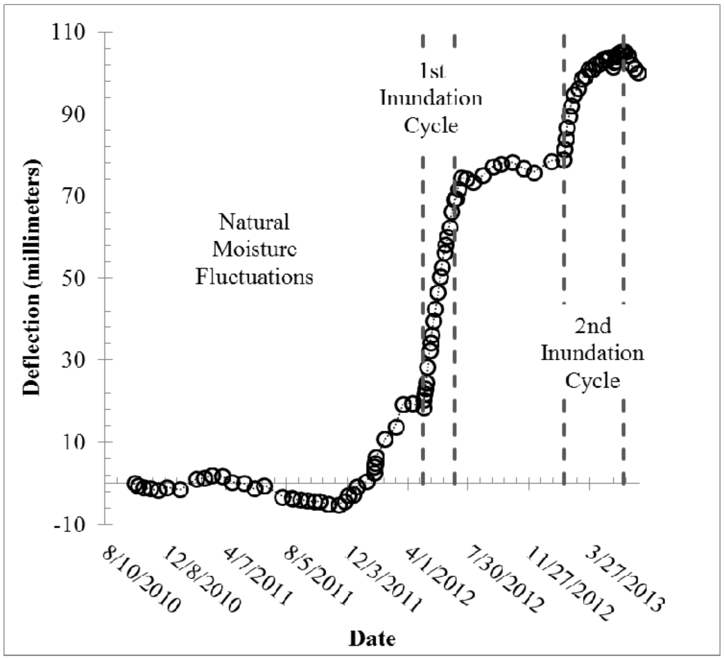

Figure 7: Top-of-wall deflections during natural moisture fluctuations and inundation testing. Reference survey is after facing installation in October, 2010.

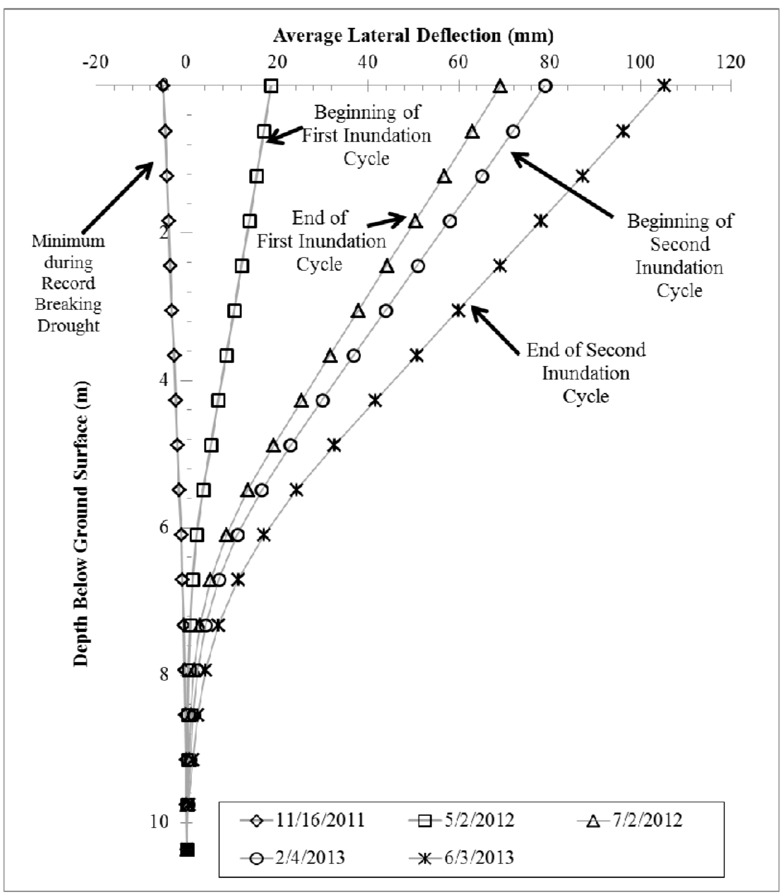

Figure 8: Average deflected shapes at key dates. Data are referenced to installation of facing material in October, 2010.

ANALYSIS

The active earth pressures applied to the wall by the retained soil were estimated for the long-term equilibrium condition assuming (1) hydrostatic conditions with the water table at the ground surface in the retained soil (Brown et al. 2014 show with field measurements and a finite element method analysis that this assumption simplistically but reasonably captures that actual seepage conditions), (2) a friction angle for the retained soil equal to the fully softened, drained shear strength (Figure 2), and (3) an interface friction angle between the wall and the retained soil equal to 2/3 the friction angle for the retained soil. The fully softened, drained shear strength is recommended for the design of slopes and retaining walls in highly plastic clays under long-term conditions by many practitioners (e.g., Skempton 1970; Stark and Eid 1997; Wright 2005; Gregory and Bumpas 2013). An excavation depth of 4.11 meters was used to model the long-term equilibrium due to erosion of the slopes moving soil to the base of the excavation.

The long-term active earth pressures were calculated using measured data from the inclinometers and strain gauges, and they represent the changes in earth pressures with time from the end of excavation and facing installation (see Brown et al. 2014 for details). A comparison of predicted and measured active earth pressures is shown in Figure 9. The assumption of a fully softened, drained shear strength envelope and hydrostatic conditions led to predictions that match the measurements reasonably well. The soil reactions based on the inclinometer and strain gauge measurements in the upper 1.80 meters are difficult to interpret because there is relatively little bending in the shaft due to earth pressure loading, and shaft curvature is additionally influenced by daily thermal effects. The calculated earth pressures in this case are not very sensitive to the friction angle of the retained soil due to the high water table in the retained soil when it reached long-term equilibrium with the inundation pond. For reference, a hypothesized envelope of earth pressures based on swell pressures is also shown in Figure 9 (Hong 2008). Based on the measured test wall data, there is no evidence that the earth pressures in the retained soil exceeded the active earth pressures for a fully drained condition at any point in time.

The passive soil resistance against the shafts was estimated for long-term loading conditions assuming drained conditions in the clay. Lateral soil springs were modeled using "sand" or "cohesionless" p-y curves with an initial stiffness, k_{py}, correlated to the undrained shear strength. A group reduction factor of 0.62 was applied to account for the shaft spacing (Wang and Reese 1986 and TxDOT 2012).

Iterative methods were used to estimate a drained soil strength envelope that can predict p-y curves similar to the measured lateral response of the wall. A drained friction angle of 30° produces a reasonable match to the measured relationships between lateral soil pressure and lateral displacement at different depths, particularly at the shallowest depths which have the most influence on wall deflections (Figure 10). For this soil, a drained friction angle of 30° is between the peak, drained friction angle of 37° and the fully softened, drained friction angle of 24° (Figure 2).

LPILE® analyses were performed using fully softened, drained shear strength envelope and hydrostatic conditions for the active earth pressure and varying the p-y curves for the foundation soil. The measured displacement profile above the excavation is reduced by the thermal effects causing a reduced curvature, which is shown in Figure 11 as apparent negative bending moments (Brown et al. 2014). In order to represent these thermal effects in the analysis, a negative bending moment of 34 kN-m was imposed at the top of the wall. Figure 11 compares the LPILE analyses using drained p-y curves and the conventional p-y curves used in TxDOT design practice, which are based on the profile of undrained shear strength versus depth (undrained strengths are reduced by 50 percent in the upper 3 meters to account for stress relief from the excavation); the measured response is softer than these undrained p-y curves predict.

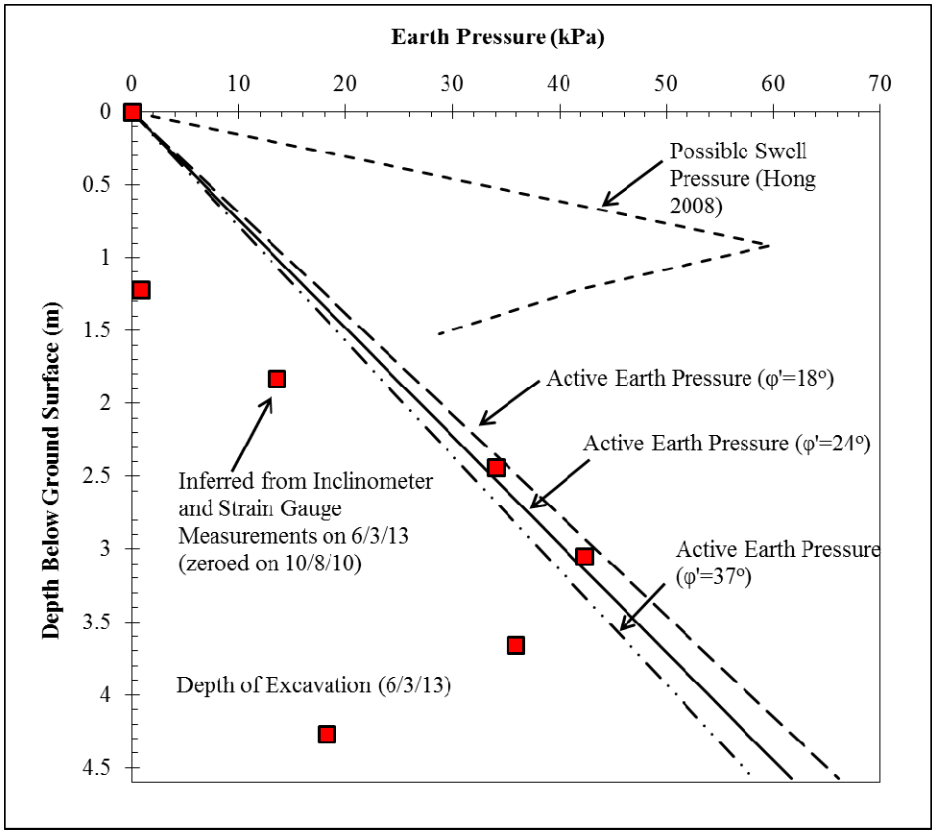

Figure 9: Predicted and measured active earth pressures in retained soil at equilibrium under full inundation.

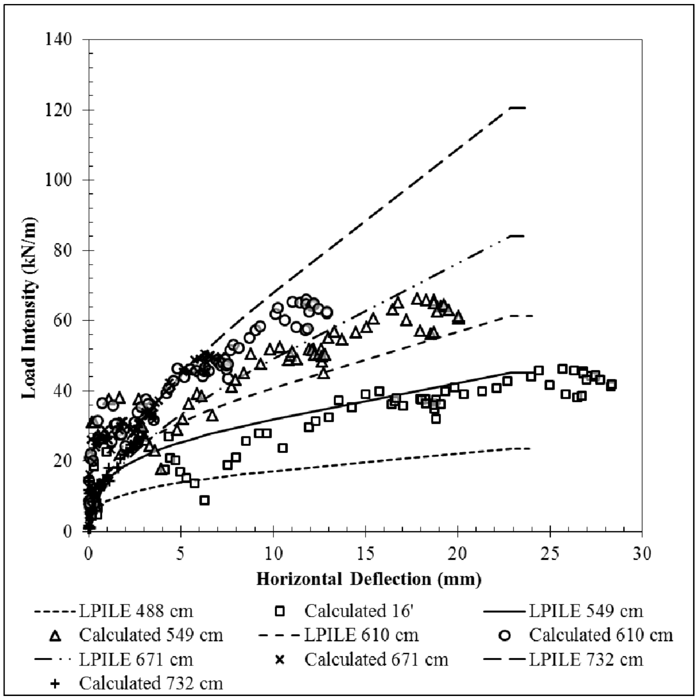

Figure 10: Predicted and measured passive earth pressures in retained soil.

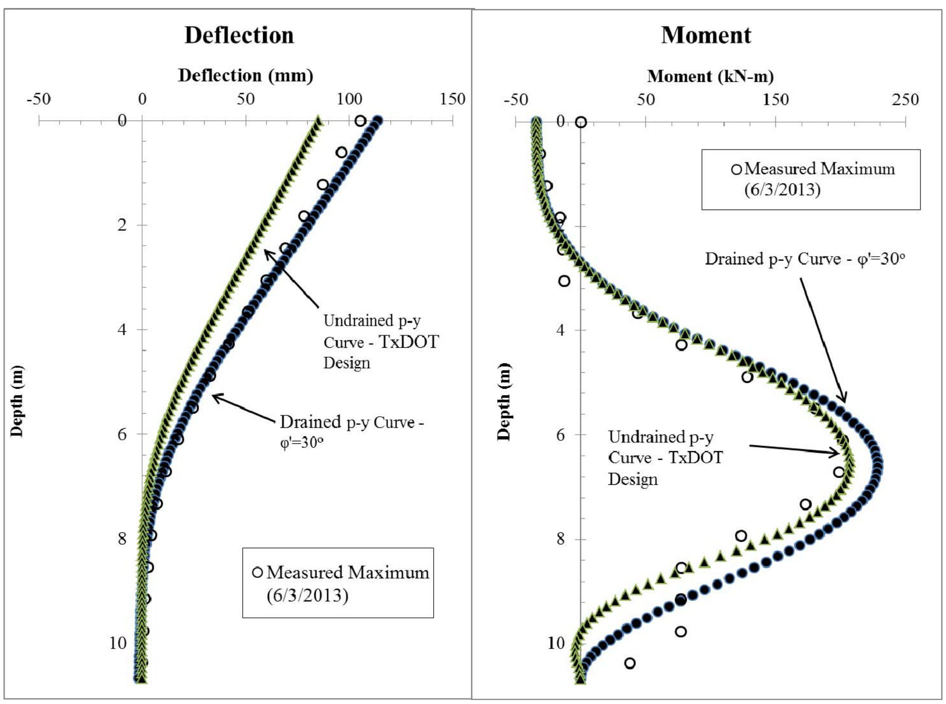

Figure 11: Comparison of LPILE predictions using different drained strengths with field measurements for long-term loading condition.

CONCLUSIONS

A full-scaled drilled shaft retaining wall was constructed in overconsolidated, highly plastic clay and monitored over a four-year period. Controlled inundation testing was performed in order to subject the wall to an upper-bound loading condition. The deflection reaching equilibrium due to inundation corresponded to predictions involving the development of drained conditions in both the retained soil and the foundation soil. The maximum earth pressures applied by the retained soil were comparable to active conditions mobilizing the drained, fully-softened shear strength; there was no evidence of greater pressures being applied as the soil swelled to its ultimate equilibrium condition. The p-y curves for the foundation soil were consistent with passive conditions mobilizing a drained shear strength between the peak and fully-softened strengths, which are softer than those typically used in practice.

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