DMT Testing for the Estimation of Lateral Earth Pressure in Piedmont Residual Soils

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Keywords: dilatometer, sheet-piling, retaining structures, residual soils, Piedmont, insitu

ABSTRACT: In this study, two instrumented flexible retaining walls were used to measure the earth pressure in Piedmont residual soil (PRS). A research site was established near Statesville in Iredell County, North Carolina within a major PRS region. The site was characterized with Marchetti Dilatometer tests, cone penetration tests, standard penetration tests, borehole shear tests and a single K_0 stepped blade test. Results of the in-situ tests were used to predict the at-rest and active lateral earth pressure. Two 36.9m long cantilevered sheet-pile retaining walls were constructed using 10.7m long PZ22 sheet piles. The walls were instrumented with strain gages and a slope inclinometer to measure bending moments and displacements, respectively. The soil between the walls was excavated in 1.2m lifts to a depth of 6.1m. The bending moments measured in the walls were used to derive the net earth pressure acting on the walls. The earth pressure calculated for the single well driven pile coincides with predictions made using the DMT.

1 INTRODUCTION

The lateral earth pressure on retaining structures due to Piedmont residual soils (PRS) is difficult to quantify by traditional methods and is often over predicted. Thus, large safety factors are used in retaining structure design that increase conservatism but not necessarily the engineer's confidence. Much of this conservatism can be attributed to the divergence between the behavior of PRS and traditional cohesive and cohesionless soils.

Traditional methods for calculation of lateral earth pressures in residual soils over predict the actual insitu stresses. Much of this conservatism is attributable to the additional strength exhibited by Piedmont residual soils due to the fabric-type nature of the material that is overlooked in traditional soil models (i.e. Mohr-Coulomb limiting equilibrium). Unfortunately, it is difficult if not impossible to obtain *undisturbed* samples of Piedmont residuum for laboratory testing; thus, engineers rely on in-situ tests to gather strength parameters used in retaining structure design. Since these tests are calibrated to laboratory tests on **either** cohesionless **or** cohesive soils, they do not provide a true measurement of the strength of Piedmont soil. Thus, engineers often design these structures based on conservative parameters and apply afore-mentioned conservative factors of safety. Yet, there is no direct increase in the engineer's confidence in the design.

Residual soils, which are found throughout the world, have a significant range in the eastern portion of the United States, as shown in Figure 1. Due to the prevalence of PRS in North Carolina, the North Carolina Department of Transportation (NCDOT) must routinely consider PRS for all types of geotechnical design projects – retaining walls, pile and drilled shaft foundations, shallow foundations, embankments, and roadway bases. Beginning FY2005, NCDOT is supporting research to develop a simple earth pressure model for PRS.

This brief paper presents an overview of the concept, some of the in-situ tests, construction of instrumented full-scale field wall, and data reduction carried out on this study.

2 SELECTION AND CHACTERIZATION OF A PRS RESEARCH SITE

Piedmont residual soils cover about one half of the land area of North Carolina. Figure 2 shows three major regions including the Carolina Slate Belt, the Charlotte Belt, and the Inner Piedmont. North Carolina DOT located a project in Statesville, NC that lies directly on the boundary of the Carolina

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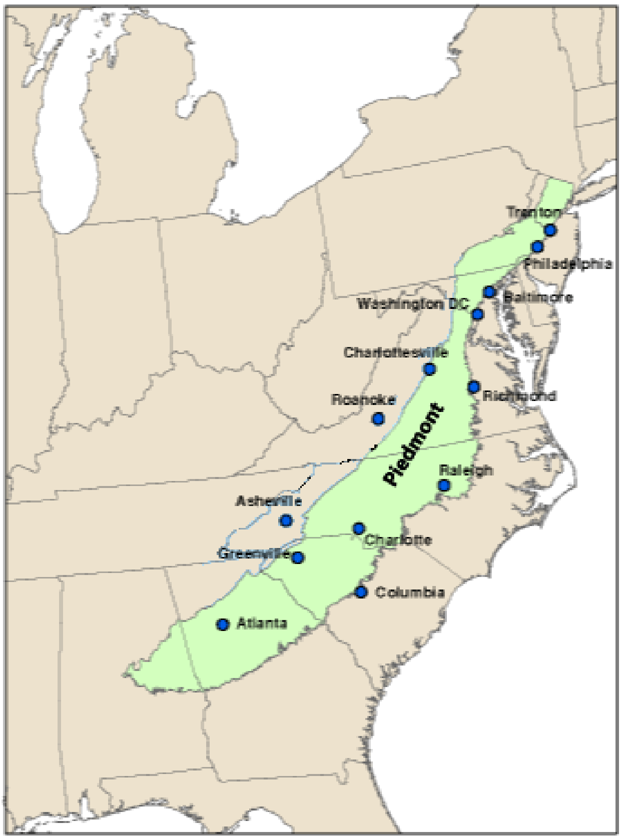

Figure 1. Range of residual soil in the Eastern United States.

Slate Belt and the Charlotte Belt. The site was a borrow pit for the US 70 bypass around Statesville, NC. Initial exploratory investigation revealed thick layers of residual soil with only slight surface disturbance. The site was quickly earmarked for construction of the first set of sheet pile walls.

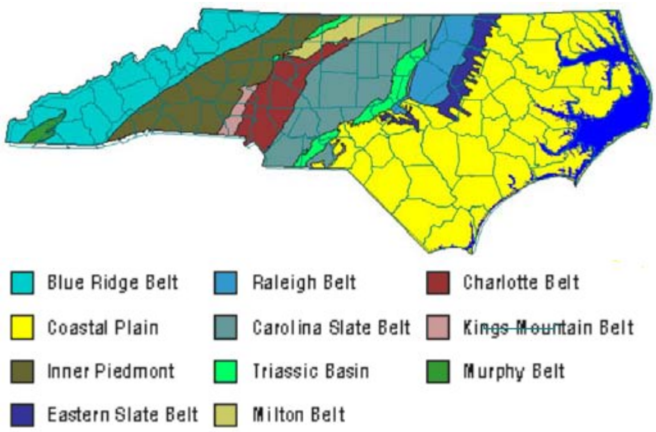

Figure 2 North Carolina Piedmont Residual soils

When the notice-to-proceed work at the site was given, an extensive in-situ testing program was initiated. Tests conducted included standard penetration tests (SPT), cone penetration tests (CPT), dilatometer tests (DMT), borehole shear tests (BST), and K_0 stepped blade tests, all detailed in figure 3

The profiles of SPT-3, CPT-4, and DMT-5 are shown together in figure 4. The SPT boring reported a soil type of residual tan to brown micaceous clayey silt. The CPT classification was OC to NC

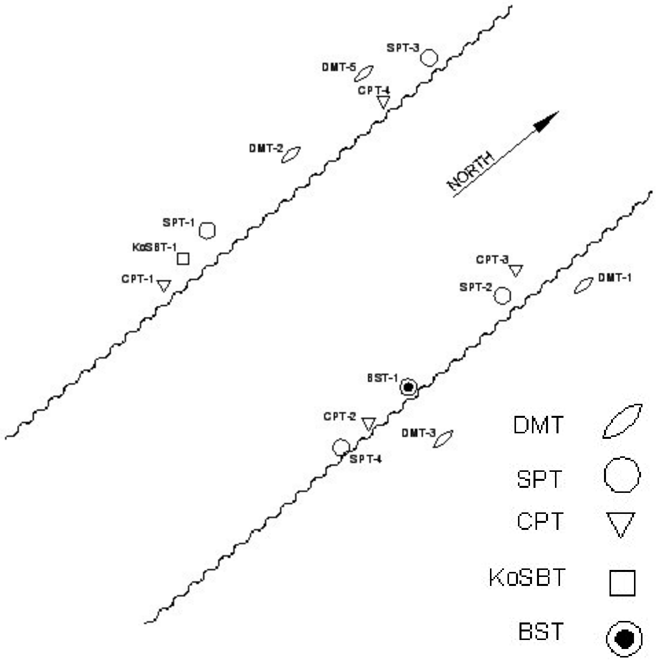

Figure 3 Layout of insitu tests at Statesville site

clay, while the DMT reported silt to clayey silt, much like the SPT. Results of BST hole and K_0 stepped blade are not presented here.

3 PREDICTION OF EARTH PRESSURE BASED ON IN-SITU TESTS

The results of in-situ tests were used to estimate the potential earth pressure on the retaining walls. As the walls would be flexible cantilever, the earth lateral pressure distribution beneath the excavation will be complex consisting of a net active and passive. However, above the base of the excavation should be subject only to at rest or active earth pressure. Therefore, the calculations of at rest and active earth pressures were made. Values of coefficient of lateral earth pressure at-rest, K_0, were estimated from DMT data using correlations developed by Marchetti (1980) and Baladi et al. (1986) presented as equations (1) and (2), respectively.

$$K_0 = \left(\frac{K_D}{1.5}\right)^{0.47} - 0.6 \qquad (1)$$

$$K_0 = 0.376 - 0.095 K_D - 0.005 \frac{q_c}{\sigma'_v} \qquad (2)$$

The DMT sounding was parsed through the equations with the q_c values to develop profiles of K_0 with depth, that were then used to calculate the at rest earth pressure.

Friction angle was correlated from DMT and CPT soundings and used to determine K_a and K_0 for each sounding. For this analysis, the soil was as-

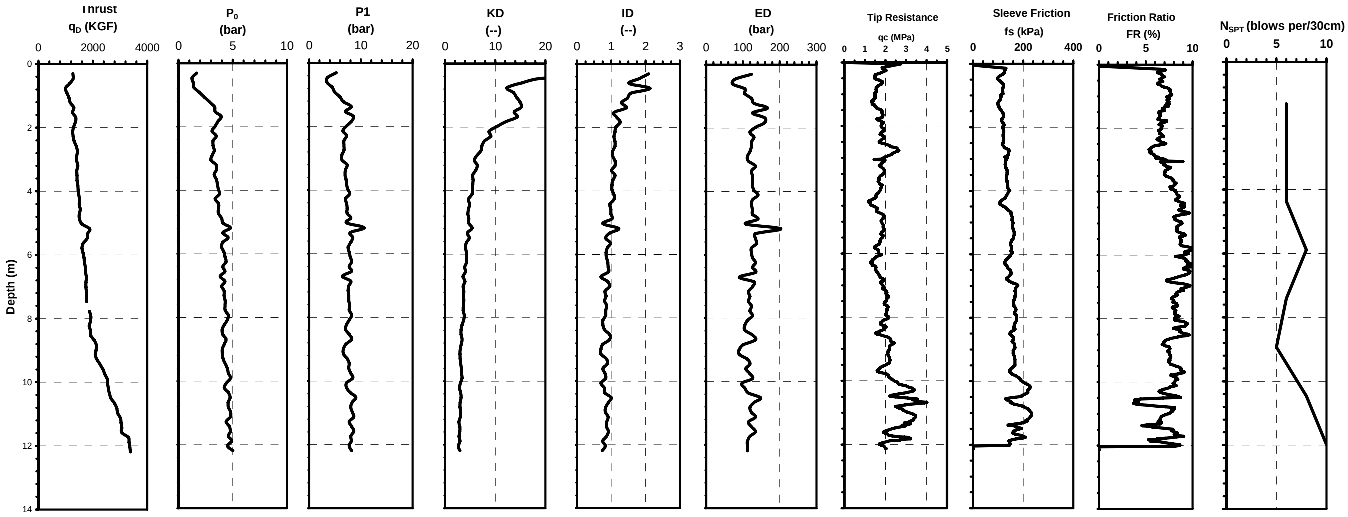

Figure 4 Composite plot of DMT, CPT, and SPT profiles

sumed to be purely frictional. A second set of earth pressures versus depth was developed based upon these coefficients. Figure 5 shows the lateral earth pressures calculated from insitu tests.

4 DEVELOPMENT AND IMPLEMENTATION OF A MECHANISM FOR MEASUREMENT OF EARTH PRESSURE

Measuring earth pressure in-situ is difficult for textbook soils, and even more so for PRS. To measure the lateral stress in place, a device would need to be inserted into the soil profile without the need for excavation, and with a minimum of soil disturbance. These requirements eliminate all but a few possibilities.

With any of the insitu tests, it is likely that any earth pressure measurement would be an estimate at best. Therefore, it was proposed to instrument a full scale retaining structure built in PRS. To meet the criteria of no excavation and minimum soil disturbance, the only choice was sheet piling. Sheet piles could be instrumented, then vibrated or driven into place without excavation. Therefore, it was proposed to construct two sheet-pile retaining walls at each research site in the configuration show in figure 6. After the project was awarded, a plan for the design and instrumentation of the walls was developed. The critical items to be determined were:

1) Section of the sheet pile
2) Total length of sheet piles
3) Minimum separation distance between walls
4) Safe maximum excavation depth
5) Maximum safe deflection of walls
6) Instrumentation type and location

Since the behavior of flexible retaining walls is a soil-structure-interaction problem, the finite element program Plaxis was used to determine the potential earth pressure, shear and bending in the wall, and displacements.

The results of the initial study were that the minimum safe sheet pile section was PZ22. The sheet piles would be 10.7m in length. They would be driven to an embedment of 10.4m. The walls would need to be a minimum of 12.2m apart. The maximum safe excavation depth between the walls would be 6.1m, leaving the sheet piles embedded 4.3m.

Many factors contributed to the instrumentation plan most notably survivability and budget. For survivability concerns, bolt-on vibrating wire strain gages, with weldable mounts, were used. These gages had been widely used in the testing of steel piles in axial and lateral load. Gages were installed in pairs at 1.22m (4 foot) intervals at 8 levels along the sheet piles. The gages were protected from installation damage by a steel angle cover. Additional advantages of the vibrating wire gages were low power consumption and integration with a Campbell Scientific datalogger, tried and true equipment, for long term deployment. Four sheet piles were instrumented with 16 gages each for a total of 64 strain gages.

In case the strain gages did not survive driving, the slope inclinometer was chosen as the backup "low tech" measurement. A box tube steel section with diagonal equal to a slope inclinometer casing was welded to the back side of four sheet piles. Unlike typical slope inclinometer tests, the axes of measurement are skewed at 45^{o} from direction of wall movement. The measurements would be rotated in the data reduction equations to match the offset angle. A schematic layout of the instrumented sheets is presented in figure 7. Finally, the third level of redundant measurements would be made using surveying equipment to monitor movements of the wall at many points.

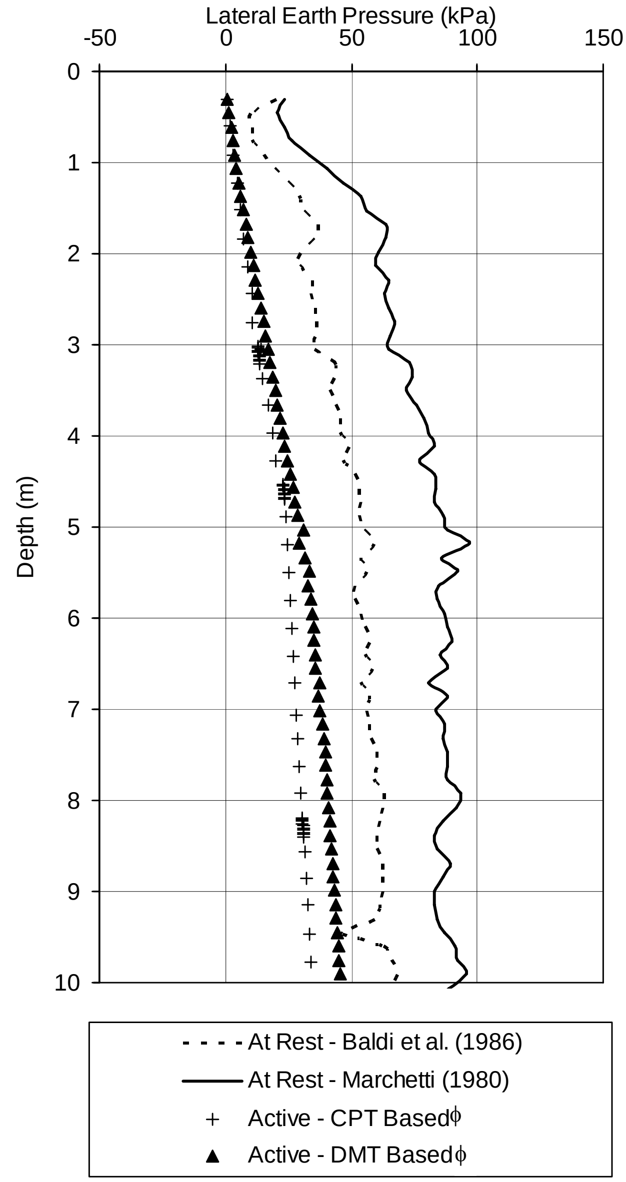

Figure 5 Predicted earth pressure

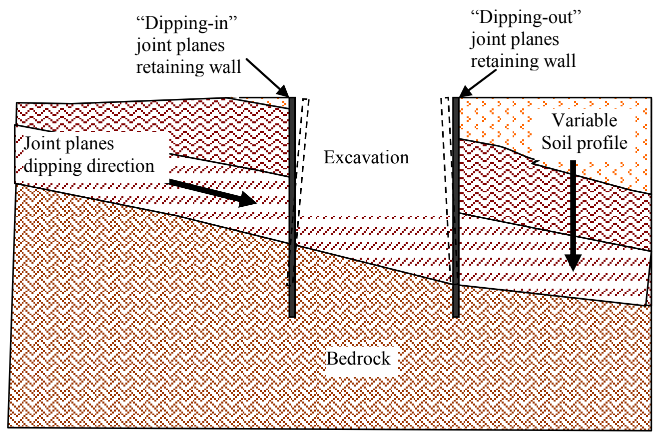

Figure 6. Idealized test wall setup

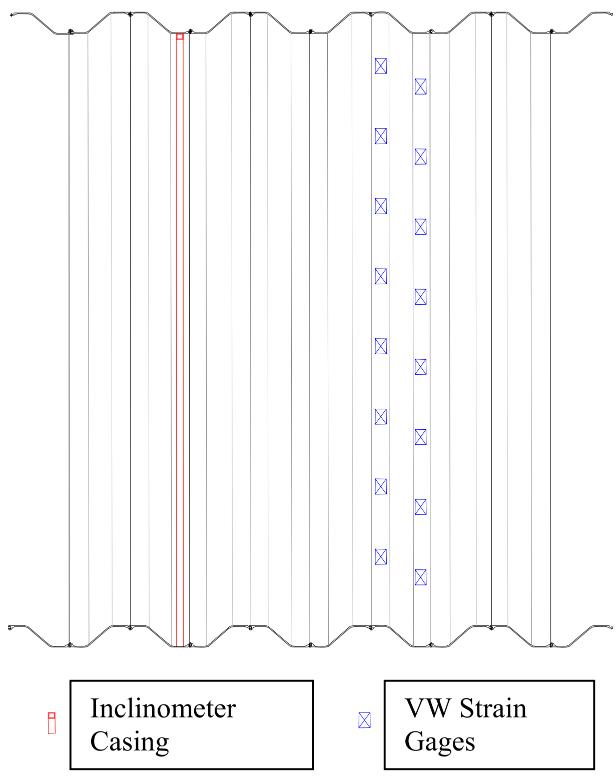

Figure 7. Strain gage and inclinometer layout

The final sheet pile walls at the Statesville site were 36.9m long consisting of 66 sheets per side. The strain gage sheets on the west wall were installed at 17.9m and 22.4m from the north end, and the inclinometer sheets were installed at 2.2m from the strain gage sheets.

The sheet piles were installed beginning September 12, 2005. As mentioned previously, the sheets were to be driven 10.4m leaving 0.3m of exposure. In the northwest corner of the site, this was possible. However, the PRS provided much higher resistance to driving than predicted by the initial tests. As shown in figure 8, the result was that many of the piles were significantly under driven. Additionally, harder driving efforts compromised four gages in the top of the southeast instrumented pile.

The soil between the sheet pile walls was excavated in 5 lifts over a period of ten days between October 17 and October 27 2005. After each excavation step, inclinometer readings were immediately taken. Subsequently, strain gage readings were downloaded from the dataloggers and a survey was conducted on selected points along the sheet pile walls and within the excavation. Figure 9 is a view looking south into the completed excavation.

Due to the driving problems, the only instrumented piles that were installed to the proper depth and completely survived installation were the strain gage-inclinometer pair in the northwest (NW) corner of the site. Subsequent analysis will focus on these piles only.

Figure 8 Installed sheet piles

Figure 9 Excavation complete at 6.1m

Inclinometer readings for the Northwest pile (NWI) are shown in figure 10. The maximum deflection at the ground surface was just less than 24mm. By the final excavation step, a visible gap developed between the sheet pile and the soil. The gap was far more pronounced at other locations along the walls where the sheets had been under driven. Using a tape measure as a crude feeler gage, the depth of soil separation from the wall was at least 3.0m.

Calculation of the bending moment was based on strain measurement. First, the net strains were determined by taking the difference of the strains at the final excavation step from the strains after the piles were driven, before any excavation. The curvature was determined by subtracting the strain measurements from the pair at any given level then dividing by the distance between gages. Knowing the moment of inertia and stiffness of the sheet pile, the curvatures were used to calculate bending moments. Bending moment profiles for strain gage in northwest pile (NWS) are shown in figure 11.

Inspection of the bending moment curves shows expected behavior. As the excavation proceeds, the sheet piles appear to relax as the maximum bending moment increases and propagates down the pile.

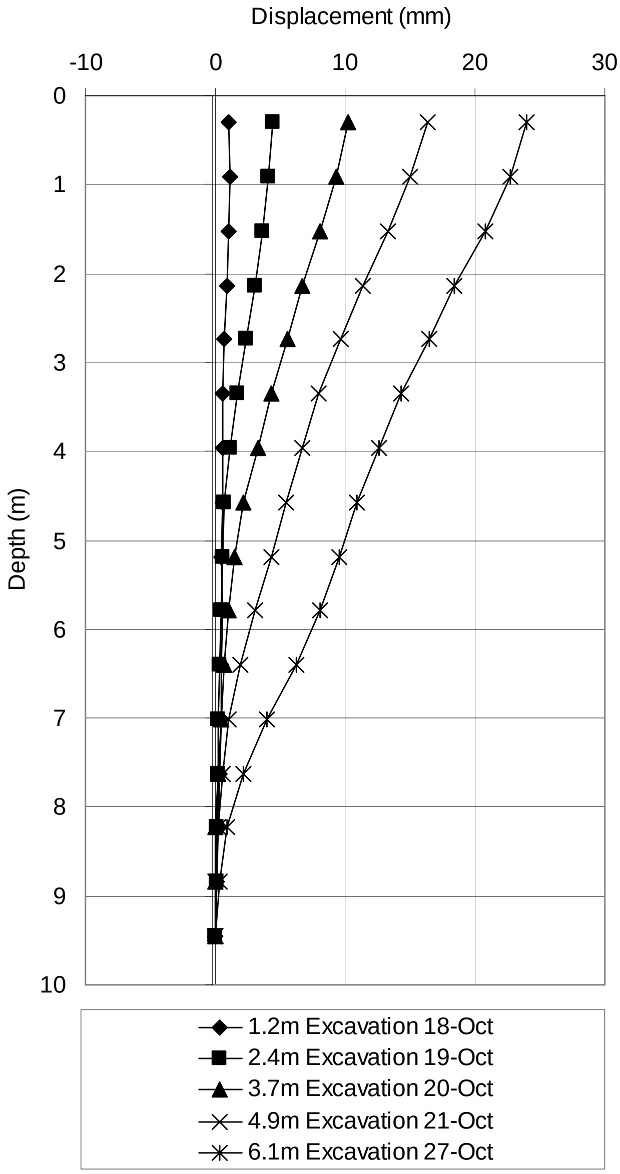

Figure 10 Sheet-pile deflections from inclinometer (NWI)

5 COMPARISON OF PREDICTED AND MEASURED EARTH PRESSURE

Sheet piles were instrumented to measure strain and deflection. Using an analytical model borrowed from laterally loaded piles, the same Winkler model of a beam on an elastic foundation, the functions for bending moment versus depth were generated. Two derivatives of these functions were taken to determine the shear in and soil reaction on the wall, respectively. The resulting earth pressure distribution for the pile NWS is plotted in figure 12 with the earth pressures determined earlier from in-situ tests. The excavation depth was 6.1m and the point of separation was 3.0m or deeper. The calculated distribution of earth pressure fits fairly well into those boundary conditions. Futhermore, the maximum value seems to coincide with active earth pressures estimated based on friction angle measurements from the DMT and CPT.

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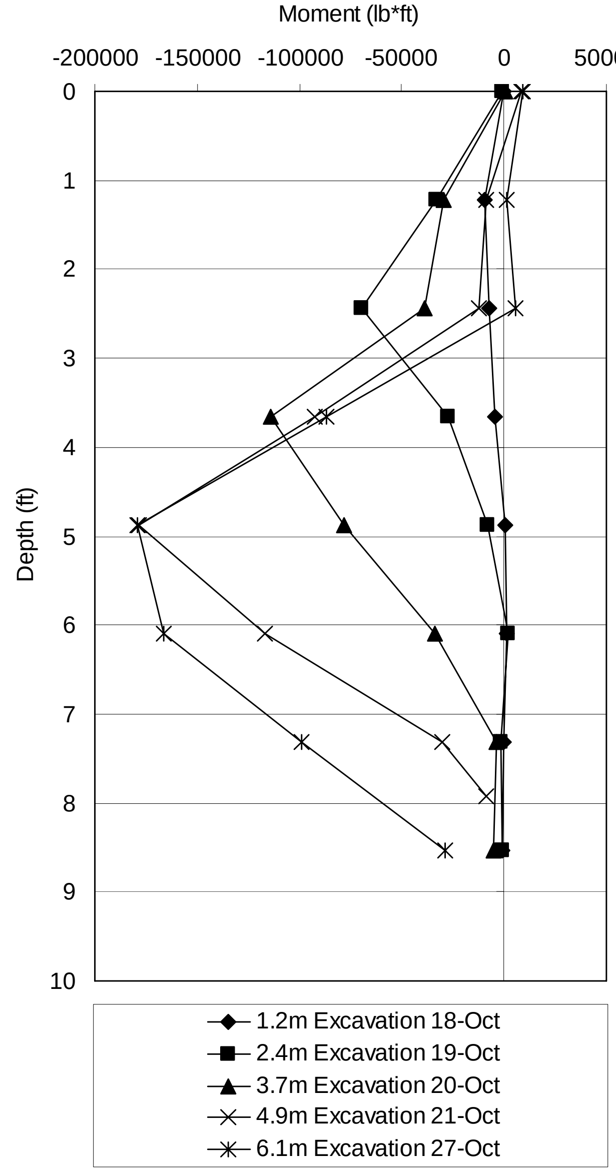

Figure 11 Bending moments from strain gages

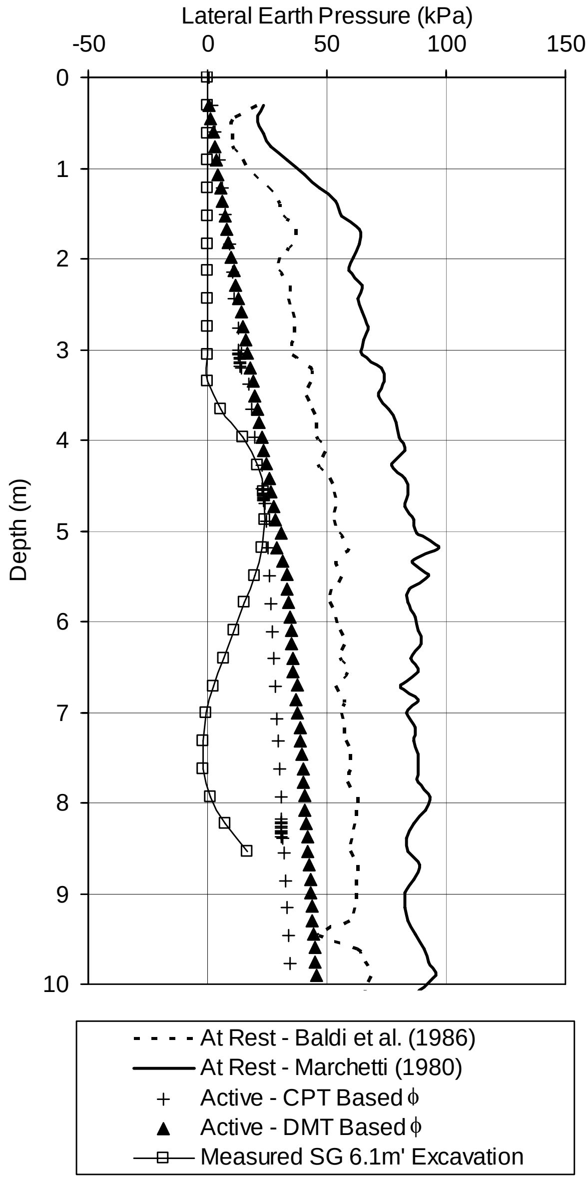

Figure 12 Derived earth pressure versus depth compared to predictions

6 CONCLUSIONS

DMT and CPT are valuable in-situ testing methods for estimating lateral earth pressure in PRS. Back-calculation from bending moment and slope measurements from cantilever sheet pile walls has proved to be viable concept to derive earth pressure distribution in PRS. For the walls excavated to a depth of 6.1m, comparisons of prediction of earth pressure using a non cohesive relationship for PRS based on DMT and CPT leads to a conservative estimate. To predict earth pressure in PRS, the friction angle derived from the DMT should be used with a cohesion value of nearly 9.6 kPa.

ACKNOWLEDGEMENTS

This paper originates from a research project sponsored by NCDOT through research grant 2005-16. The authors would like to acknowledge Tim Cleary and Billy Camp from S&ME Inc. in Charleston, SC for donating four CPT tests at the research site. Mr. Roger Failmezger generously loaned the K_0 stepped blade for use in this study.

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