Draft – final version available in proceedings of IFCEE 2021 -
https://doi.org/10.1061/9780784483411.032

Cone Penetration and Dilatometer Tests to Evaluate Stresses in a Soil-Bentonite Slurry Trench Cutoff Wall

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ABSTRACT

Soil bentonite (SB) slurry trench cutoff walls have been widely used for over 40 years to control ground water flow, seepage through dams and levees, and contaminant transport. The hydraulic conductivity of SB backfill is stress dependent and the concept that the state-of-stress in SB slurry trench cutoff walls is less than geostatic was first published over 30 years ago. A full-scale, instrumented SB slurry trench cutoff wall was constructed with embedded instrumentation that directly measures the stress and pore pressure. In addition, cone penetration (CPT) and Marchetti dilatometer (DMT) tests were performed in the wall after the wall was allowed to sit for one year. The combination of direct stress measurements and in situ tests presents a unique opportunity to compare results from CPT and DMT tests to the direct measurements of stress in the wall and to predictions from available models. Finally, the authors provide summary opinions regarding cone and dilatometer testing for predicting stress in low strength materials along with practical recommendations for the design and construction of SB walls.

INTRODUCTION

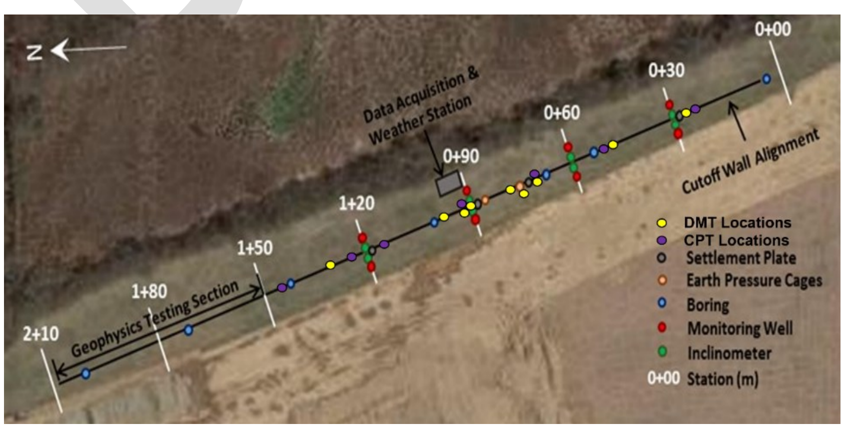
Soil-bentonite (SB) slurry trench cutoff walls are constructed using bentonite water slurry to maintain trench stability during excavation. The slurry is then displaced with backfill composed of a mixture of in situ excavated or borrow soil and slurry made from sodium bentonite and water with the occasional addition of 1% to 3% dry bentonite. SB cutoff walls are often used to control the migration of groundwater and numerous publications have been devoted to the wider discussion of installation methods and properties (e.g. D'Appolonia 1980; Evans 1993; Sharma and Reddy 2004). Generally, the main performance criterion is a maximum hydraulic conductivity (k) of the wall with a common value being 1×10^{-7} cm/s.

SB slurry trench cutoff walls are widely used for critical applications like seepage control, levee repair, and contaminant containment. Although use in these critical applications is common, field data from in-service walls are scarce. In order to ensure an understanding of short and long term behavior, a better understanding of the as-built conditions and long-term behavior are needed. Importantly, the hydraulic conductivity (*k*) of an SB cutoff wall is heavily influenced by the state-of-stress in the backfill (Evans 1994; Shackelford 1994; Filz et al. 2001; Ruffing and Evans 2010). The extent of the stress dependency varies and is dependent upon the grain size distribution and bentonite content (Evans and Huang 2016, Barlow 2018). Further, the stress-state in SB walls is now understood to be considerably less than geostatic as first postulated in 1985 (Evans et al. 1985). Finally, due to schedule and cost constraints, typical construction quality control (QC) programs for backfill *k* almost exclusively rely on permeability tests conducted on grab samples of backfill with the tests performed in the laboratory at assumed confining stresses. In order to ensure that the results from these laboratory tests are representative of the installed wall condition, the confining stresses must be properly selected. Hence the need for more and deeper evaluation of this topic.

Several laboratory studies, field studies, and theoretical (desktop) examinations have been conducted to assess the in situ stress state of SB walls (e.g. Evans et al. 1995, Ruffing et al., 2010; Ruffing and Evans 2010; Ruffing et al. 2011; Ruffing et al., 2015). The test wall for this study was designed and constructed to reinvestigate lessons from previous studies and to fill gaps in understanding. In short, the test wall is an instrumented SB cutoff wall that is ~200 m long, ~7 m deep and 0.9 m wide. The wall was installed adjacent to a commercial sand/gravel quarry pit near the Bucknell University (BU) campus. Subsequent to installation, numerous in situ tests, including the cone penetration tests (CPTs) and Marchetti dilatometer tests (DMTs) described in this paper, have been conducted to examine the results and usefulness of various methods for determining the stress, and/or hydraulic conductivity, of the backfill. This paper presents the data collected with the CPT and DMT and compares that data to previously published stress data in this wall and further to stresses predicted from existing models.

SB CUTOFF WALL INSTALLATION, PROPERTIES, AND INSTRUMENTATION

A summary of the installation methods, construction QC, and instrumentation is provided here for ease of reference. For additional details about the information presented in this section, please refer to Evans and Ruffing (2017) and Malusis et al. (2017). The full-scale test SB cutoff wall was located within a permitted mining area of a commercial sand/gravel quarry in Montandon, PA (Fig. 1).



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https://doi.org/10.1061/9780784483411.032

Fig. 1. Plan view of the cutoff wall with instrumentation and CPT/DMT testing locations.

The subsurface consists of primarily alluvial materials derived from historic flooding and river channels of the Susquehanna River. The wall was constructed in July 2016 by Geo-Solutions, Inc., New Kensington, PA, USA. After completion of construction, the wall was covered by geotextile followed by 0.3 m of topsoil over the geotextile.

Because the excavated soils contained a high percentage of cobbles and coarse gravels, the soil used for the backfill was brought from another part of the site and composed of a well graded silty clayey sand with approximately 10% gravel. Grab samples of the backfill were tested in the laboratory and the hydraulic conductivity was found to range from 1×10^{-7} to 7×10^{-8} to cm/s. The laboratory tests were flexible-wall triaxial hydraulic conductivity tests performed using effective consolidating stresses of 21 kPa to 35 kPa. Results from grain size distribution tests for the base soil and mixed backfill is shown on Fig. 2. The backfill water content and void ratio averaged 30.1% and 0.84 respectively. Groundwater depth fluctuated from 3 to 6 m below ground surface.

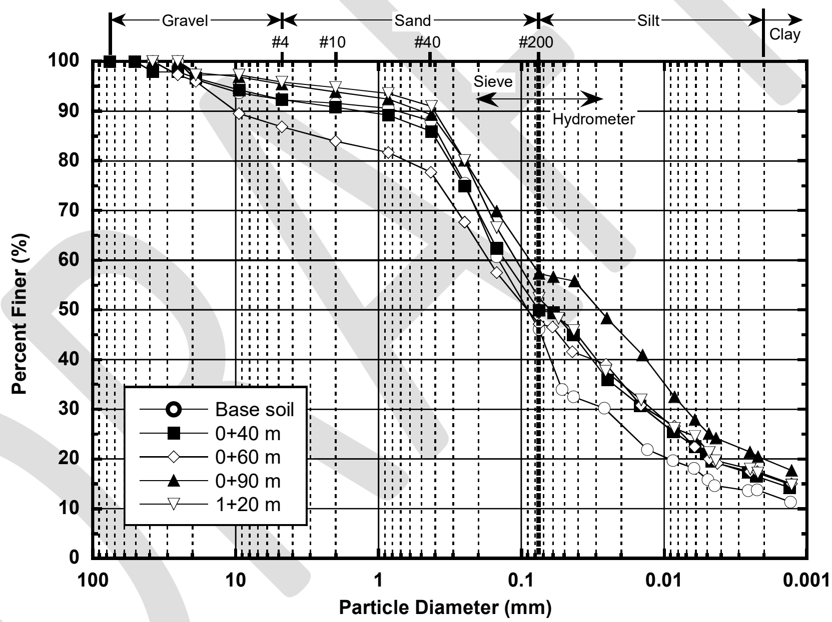

Fig. 2. Grain size distribution of base soil and backfill (legend indicts station of backfill sample).

Instrumentation in and around the wall includes monitoring wells, slope inclinometers, settlement plates, and earth pressure cells. Instrumentation cages were installed with three earth pressure cells each measuring stresses in the vertical, transverse and longitudinal directions and sensors for pore water pressure, temperature, bi-axial tilt (measures tilt and roll) and a magnetic compass to determine orientation of the cage. The cages were installed at three different depths at one location and one depth at a separate location. Measurements are automatically taken every hour and transmitted to an online portal for real-time viewing on a devoted website. Slope inclinometers, settlement plates and monitoring wells are manually read periodically.

IN SITU TESTING

Approximately one year after the SB wall construction was complete, a program of in situ testing was implemented including vane shear (VS), CPT and DMT. The purpose of the in situ testing was performed to examine the correlation between the direct measurements of stress from the earth pressure cages and estimates of stress from correlations to stress measurement from common in situ testing tools. The findings from the vane shear testing are published in Evans et al. (2018). This paper presents the findings from the CPT and DMT investigations. Note that primary consolidation was complete by the time of testing as determined from piezometer, inclinometer, and settlement plate data.

CPT ANALYSES AND RESULTS

Cone penetration tests were performed for the full depth of the wall at eight different locations. The shear strength of the backfill was calculated individually for each data point according to the procedure below which follows the work and conclusions of previous studies, Ruffing et al (2010) and Ruffing et al (2015). Accordingly, the CPT data was analyzed using the "Effective Cone Resistance" method to determine the shear strength of the cutoff wall. As in previous studies, this method was chosen because it requires no previous knowledge of any stresses acting on the wall. The shear strength of the wall was calculated using equation 1.

$$S_u = \frac{((q_c + (1-a) \times u_2) - u_2)}{N_{ke}} \qquad \text{Eq. 1}$$

where S_u is the shear strength, q_c is the raw tip resistance, a is the area ratio, u_2 is the pore pressure at the shoulder of the cone, and N_{ke} is a cone factor, which typically falls between 1 and 13.

Using data from vane shear tests (see Evans et al 2018) to back-calculate the cone factor resulted in a selection of 11. Interestingly, this cone factor matches well with the cone factor determined for a different SB cutoff wall in another study where a cone factor of 11.5 was used (Ruffing et al., 2015).

Shear strength values over 300 kPa were not included in the averages because these unusually high values are not representative of the true shear strength of the backfill and are rather likely due to encountering oversized material. That is, these unusually high values are likely due to the cone hitting gravel, cobbles, or large clay clods in the cutoff wall during the test.

Presented on Fig. 3. are the shear strength results from the eight individual CPT profiles. Notice the relative consistency between all eight CPT soundings including 1) relatively high values of shear strength in the shallow zone affected by the cover and cycles of the fluctuating water table, 2) low values of shear strength (less than 20 kPa) in the depth range of 1 to about 5.5 m, 3) occasional high shear strength values throughout where the CPT encountered gravel or other oversized material, and 4) increased values of shear strength as the CPT encounters in situ materials at the base of the wall.

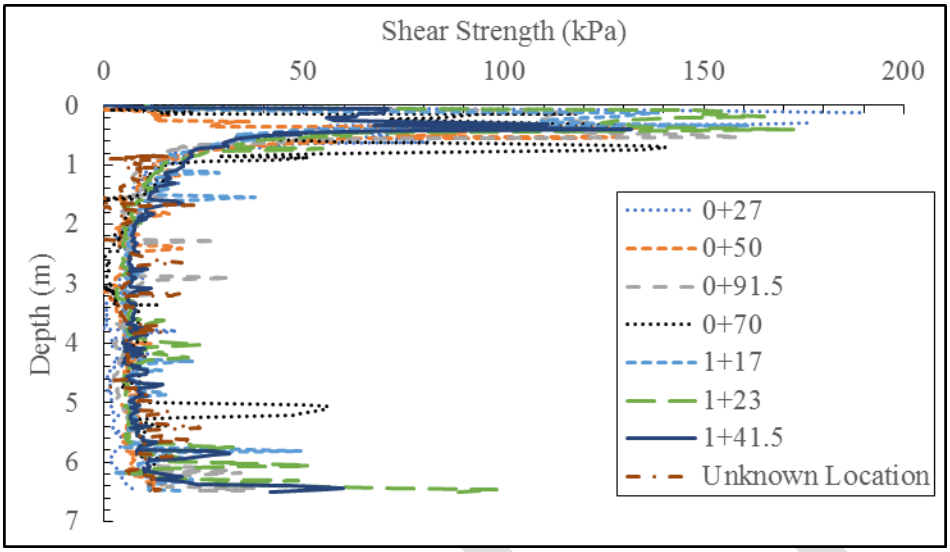

Fig. 3. Shear Strength vs Depth of Cutoff Wall

Once the individual data points were calculated for each of the CPT profiles, the data from all eight profiles was averaged. Fig. 4 is the resulting plot of shear strength as calculated from the CPT versus depth, including deletion of high values (see above). The vane shear measurements from Evans et al. (2018) are also included for reference. The left plot in Fig. 4 shows the results plotted over the full range of values where on the right side of Fig. 4 the scale of the x-axis has been reduced to better illustrate the bulk of the values in the 0 to 20 kPa range.

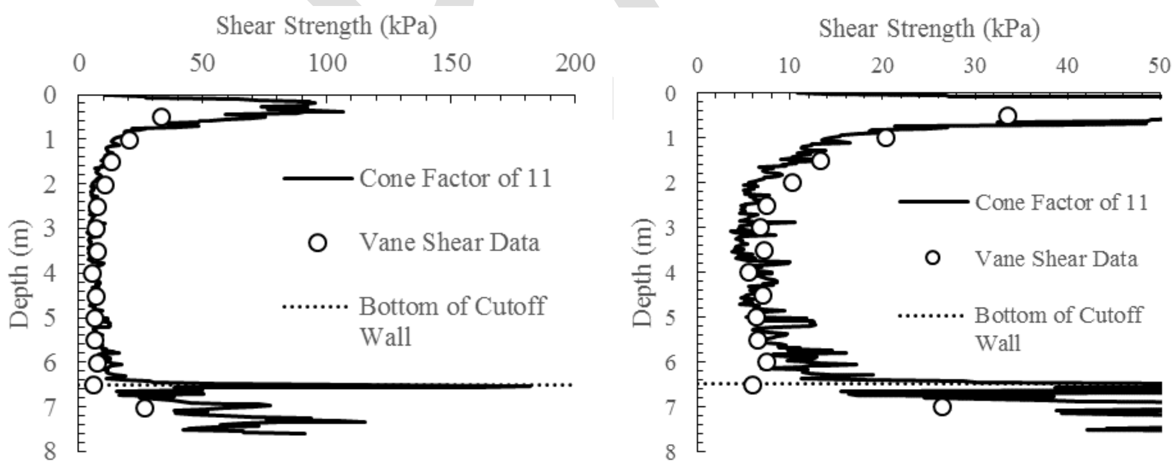

Fig. 4. Shear Strength vs Depth of Cutoff Wall

A shear strength value of 50 kPa or less characterizes the wall as a very soft clayey material which is an accurate description of a general SB backfill and specifically for the backfill from this site. At a depth of about 6.5 meters there is large jump in the shear strength values which the authors attribute to the CPT encountering formation materials at the bottom of the wall.

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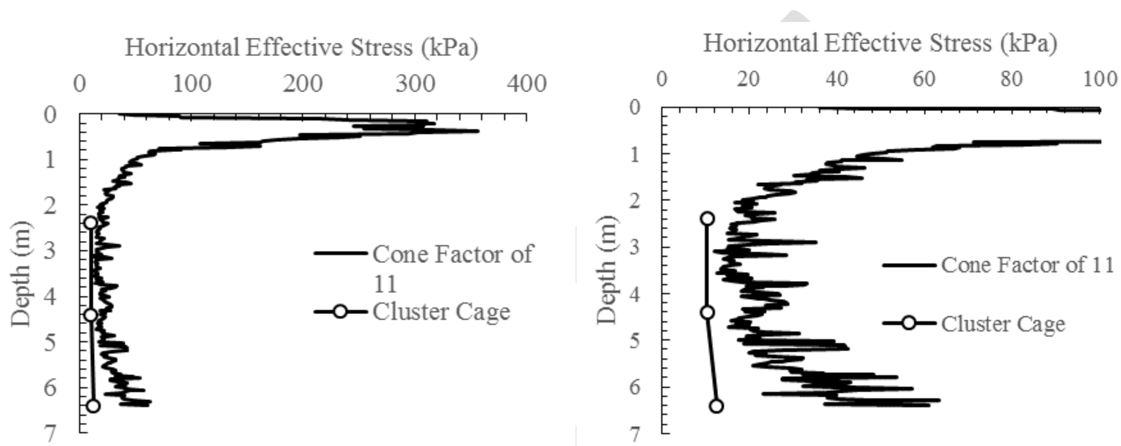
Once the shear strength values were determined, the data was also used to calculate the horizontal effective stresses in the cutoff wall using the logic presented in Ruffing et al. (2015) which relies on the conventional relationship between shear strength and stress, but also accounts for the principal stress rotation in SB walls (Evans and Ruffing 2019). The resulting horizontal effective stress, using a value of 0.3 for the strength/stress ratio is plotted in Fig. 5 along with stresses from the earth pressure cages (from Evans and Ruffing 2019). The left plot in Fig. 5 shows the full range of the data whereas the right side shows the data against a reduced x-axis scale to better illustrate the results of the bulk of the data.



Fig, 5. Horizontal Effective Stress (S_u/σ_h' = 0.3)

Note that this method to compute the value of horizontal stress would typically represent the mean horizontal stress and, for many geologic deposits, the horizontal stress would be the same in all horizontal directions. For SB slurry trench cutoff walls, the transverse horizontal stress would be the major principal stress, the vertical stress would be the minor principal stress and the longitudinal horizontal stress would be an intermediate stress.

Two observations are clear from a review of Fig. 5. First, the horizontal effective stress calculated from the CPT data is slightly larger than the stress values from the cluster cages. Second, the higher shear strength and thus stress values at the top of the wall are a result of suction from the upper portion of the wall undergoing cyclic wetting and drying. The horizontal effective stress values from the cluster cages are direct measurements and might be considered more accurate than the horizontal effective stress values calculated from the CPT data. Further, it is also known that the very act of making a measurement may alter the value of the parameter being measured, so a corrected stress from the CPT data with disturbance removed would be expected to be closer to the stress measured with the cages anyway.

DMT ANALYSES AND RESULTS

As with the CPT, the DMTs were performed at eight locations along the wall. At each location, measurements were taken every half meter until a depth of seven meters was reached. The methods of testing and analysis followed those of Ruffing et al 2010 and Ruffing et al 2011. Following those previous studies, the DMTs were performed at two orientations, transverse and longitudinal, relative to the cutoff wall centerline (see Fig. 6).

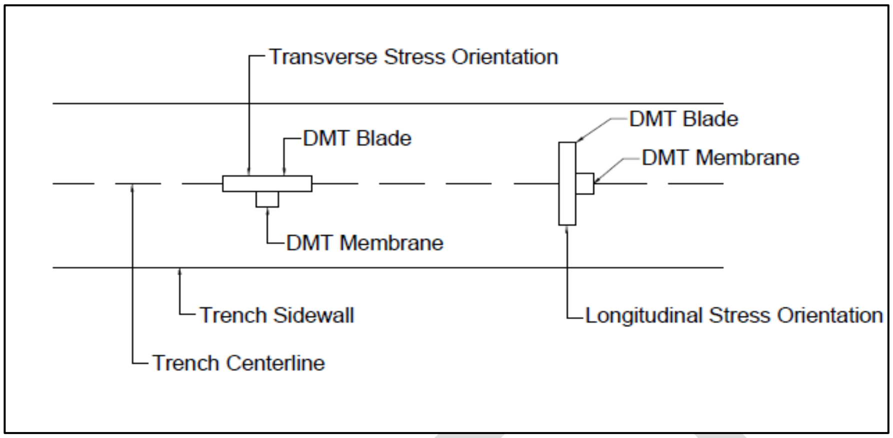


Fig. 6. Transverse and Longitudinal Dilatometer Orientations (after Ruffing et al. 2011)

Using the field readings, the horizontal stress and pore water pressure in the slurry wall were calculated using dilatometer data and the following equations (Marchetti, 1980):

$$P_0 = 1.05(A + \Delta A) - 0.05(B - \Delta B) \qquad \text{Eq. 4}$$

$$U_0 = C + \Delta A \qquad \text{Eq. 5}$$

where P_0 is the total lateral (horizontal) stress, ΔA and ΔB are calibration factors determined before insertion of the dilatometer blade, and U_0 is the pore water pressure.

The horizontal effective stress was calculated by subtracting these values.

$$\sigma_h' = P_0 - U_0 \qquad \text{Eq. 6}$$

where σ_h' is the horizontal effective stress in the slurry wall.

Using these results, an effective stress was calculated for each depth at each location and then the effective stresses were averaged resulting in one value for each depth, similar to the CPT analysis described above. The average horizontal effective stress values calculated from the dilatometer data in both the transverse and longitudinal directions are shown in Fig. 7 along with the lateral effective stresses calculated from the earth pressure cages (from Evans and Ruffing 2019).

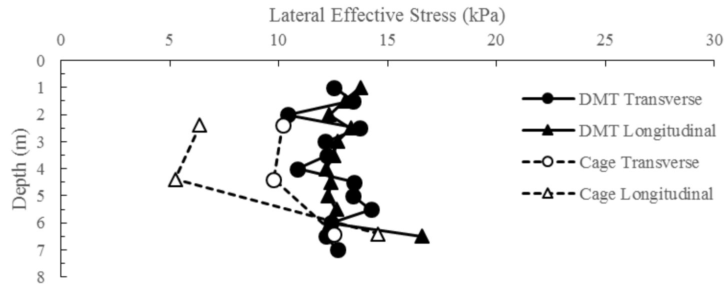


Fig. 7. Lateral Effective Stress Averages from DMT and Instrumentation Cages

The dilatometer measurements taken at a depth of 0.5 meters are omitted from Fig. 7 as the authors do not believe the data is representative of the SB backfill due to the proximity of the cover soil. A review of Fig. 7 shows the effective stresses measured with the DMT in the transverse direction were larger than the stresses in the longitudinal direction as similarly measured with the earth pressure cells. However, the DMT stresses in both directions were very similar to one another whereas the in situ measurements show a greater difference in stress with direction, specifically above 5 or 6 m depth. The longitudinal DMT effective stresses show greater divergence from the in situ instrumentation results than the transverse DMT results.

Some variability in the data is possible due to the in situ instrumentation obtaining more measurements over time than the dilatometer test and the possible disturbance effects of the DMT blade insertion. That said, the in situ instrumentation are also limited to three depths. Another difference could be variations along the trench alignment in that the in situ measurement were taken at Station 0+84 whereas the dilatometer measurements were obtained over a range from Stations 0+28 to 1+41.

Figure 8 shows the lateral effective stress for both the transverse and longitudinal directions for dilatometer measurements taken at Stations 0+81 and 0+81.5, respectively. These two positions were selected because they were the closest dilatometer measurements to the in situ measurements. In addition to being at approximately the same location the measurements were taken at the same time for both the dilatometer and cluster cages to ensure the conditions were as similar as possible.

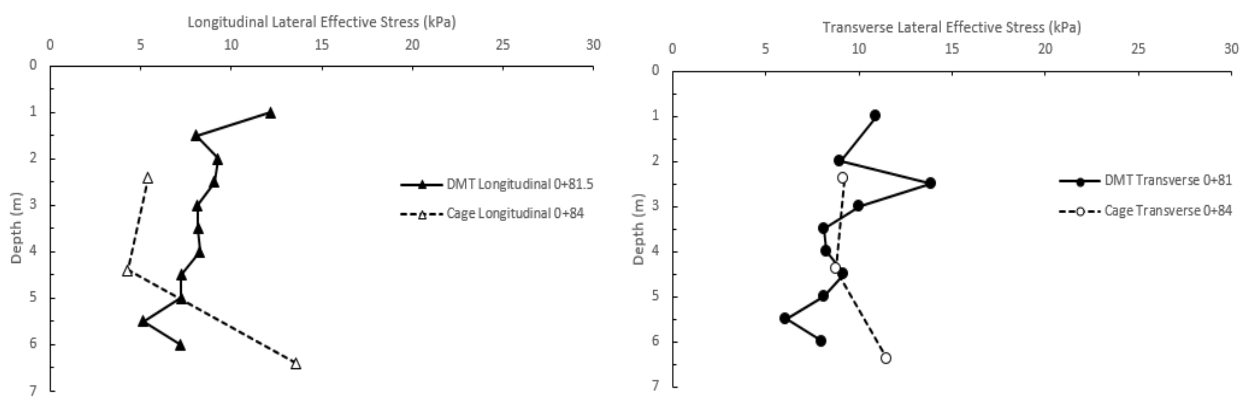


Figure 8. Lateral Effective Stress at Specific Locations near Instrumentation Cages

As can be seen in Fig. 8, the dilatometer correlated better with the in situ results when using only the nearest DMT results rather than the averaged DMT results. In general, occasional outliers in the dilatometer data from the dilatometer hitting a stone in the backfill can be expected and the impact of those measurements can be expected to be large considering the relatively small diameter of the dilatometer membrane (Ruffing et al. 2011). Comparing the results presented in Figs. 7 and 8 show reasonable agreement in effective stresses between the averages for the entire length of the wall (Fig. 7) and the stresses at one location (Fig. 8). While there is reasonable agreement when comparing measured stress results with averaged in situ stress results, the agreement is improved when using the nearby in situ results rather than the average values.

SUMMARY AND CONCLUSIONS

The construction of an instrumented SB slurry trench cutoff wall allowed for a unique opportunity to compare in situ measured stresses with those back-calculated from CPT and DMT test data. Direct measurements of effective stress found the transverse lateral stress to be the highest followed by longitudinal lateral stress with the vertical stress being the lowest (first presented in Evans and Ruffing 2019). These results are consistent with the mechanics of a compressible material consolidating in a narrow trench where the principal stresses are rotated. Importantly, all stresses were low compared to geostatic and, in general, were less than 15 kPa. The calculated values of effective stress from the CPT and DMT were generally greater than those measured with the in situ instrumentation although better agreement was found when comparing the nearest DMT to the transverse cage measurement taken at the same time. As is well known, in situ measurements are also influenced by the disturbance of the device itself which may partially or fully explain the higher measurements from the CPT and DMT relative to the in situ cages. Finally, and perhaps most importantly, the results show that SB backfill is very soft and the stresses are very low compared to geostatic stresses. These results should be considered in the selection of confining pressures for geotechnical engineering laboratory permeability testing.

Given both the speed of testing and the reliability of the prediction, the CPT is recommended as a means to estimate the in situ stresses for SB slurry trench cutoff walls. As expected, variability was evident in all of the stress measurements, but this variability does not void the general conclusions and observations. If the CPT is used in this application for evaluation of full scale commercial installations, the results must be evaluated with inherent and expected variability kept in mind. It is recommended that where CPT and DMT testing are used on commercial projects that these results be published to add to the data base of projects and, hopefully, further validate the approach.

ACKNOWLEDGMENTS

The authors would like to acknowledge the contributions of CETCO which donated the bentonite for this project, Central Builders Supply which provided on-site support during construction and the study location, and Geo-Solutions, Inc. which performed the construction. The project would not have been possible without the support of these companies. In addition, the authors acknowledge the Jeffrey C Evans Geotechnical Engineering Laboratory endowment, established through the generosity of Michael and Laureen Costa, which was used to provide financial support for the students and faculty. The authors appreciate the contributions of Benjamin Summers who performed the CPT and DMT field work. Special thanks are also given to James Gutelius, Director of Bucknell University's Civil Engineering Laboratories, for his numerous contributions to this project.

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