Forensic evaluation of an embankment on soft ground using CPT

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ABSTRACT: Development of a new container terminal in North Charleston, South Carolina required the construction of a roadway embankment. In consideration of very low soil strength, the embankment design included prefabricated vertical drains and within critical areas, two layers of high strength geotextiles as basal reinforcement. The majority of the embankment performed adequately during construction but lateral movements and instabilities occurred within one main segment. A forensic exploration program, that included field vane testing and additional CPT soundings, was performed to develop a remediation plan. Additionally, a closely spaced array of CPT soundings was used to identify geotextile layers and the pre-existing crust. Since the geotextile layers and crust served as "markers", an approximation of the failure surface was developed that greatly improved the back-analysis and subsequent remediation plan.

1 INTRODUCTION

A new container terminal, known as the Charleston Naval Base Container Terminal or CNBCT, is currently under construction in North Charleston, South Carolina, USA. As shown in Figure 1A, the site consisted of salt marsh and tidal creeks as recently as 1939. Over the next 20 to 30 years, much of the site was filled for land reclamation and a significant portion of it was used as a dredge spoil disposal area. The original marsh deposits and the more recently placed dredge spoil consist of predominantly fine-grained, very soft, high plasticity, soils. The low strengths and high compressibility of the soils were considered in the design of the new roadway embankment but approximately 0.2 km of the 1.5 km long embankment experienced large lateral movement during construction. The area of movement or embankment failure is indicated in Figure 1 by the dotted blue line.

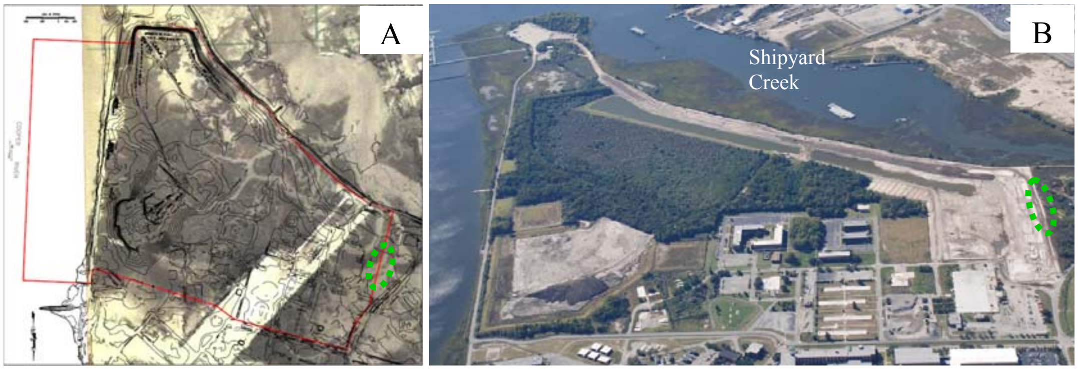

Figure 1. Aerial photos of the project site from 1939 (A) and 2009 (B). The red line in (A) indicates the area of the proposed container terminal and the dotted green line indicates the area of movement that is the focus of this paper. The embankment runs along the western edge of the property (North is down), turns to the east, and then parallels Shipyard Creek.

2 DESIGN-PHASE SITE CHARACTERIZATION

The entire 100 hectare CNBCT site was explored and characterized using cone penetration testing (CPT), Marchetti dilatometer testing (DMT), standard penetration testing (SPT), and laboratory testing on disturbed and "undisturbed" tube samples. In general, stratigraphy was delineated using CPT and then selected strata were targeted for additional in situ testing and sampling. A previous exploration of a similar site located across the river included field vane testing (FVT) and DMT. The DMT data were used to estimate undrained shear strength using the following correlation:

$$s_u = 0.22\sigma'_{vo}\left(0.5K_D\right)^{1.25} \quad \text{(Marchetti 1997)} \qquad (1)$$

The undrained strength estimates using the Marchetti 1997 correlation were in excellent agreement with the FVT. As a result of this experience and considering the difference in speed and efficiency, FVT was not performed at the CNBCT site and DMT was the primary in situ method used to estimate undrained shear strengths.

As noted previously, the site contains various uncontrolled fills that were used to reclaim marsh, tidal flats, and creeks. Additionally, a significant portion of the site was used as a dredge disposal basin, which received pumped spoil material. As a result, the subsurface conditions would be expected to be quite variable. During the design phase, the designers faced two basic choices: 1) attempt to delineate all of the variations in the subsurface and to then design accordingly for the different conditions across the site or 2) generalize the subsurface conditions and to then design for some base case profile. The first approach would have significantly increased the costs and time of the site characterization program but more importantly, it would have substantially complicated the construction plans for the initial site stabilization (i.e., multiple construction stages and surcharge heights for multiple areas). A more detailed delineation of conditions across the site was therefore not warranted and approach 2 was adopted.

For approach 2, the base case profile could be selected as a worse case profile, thereby significantly reducing the chances of a failure during site stabilization or a failure to meet performance objectives. However, use of worse case conditions across the large site would have greatly increased construction costs. Therefore, "average" conditions were used to develop base case profiles for representative portions of the site. The exploration locations were arranged in a grid pattern with a spacing of about 90 m to 120 m between soundings/borings. The data from a typical CPT sounding is presented in Figure 2. For design-phase analyses in the area of the new roadway embankment, we assumed the shear strength at the top of the soft clay stratum was equal to 11 kPa (230 psf) and increased linearly with depth at a rate of 1 kPa per m of depth (7 psf per ft of depth). This strength profile was generally confirmed in the field during the construction and monitoring of two design-phase test embankments (Goldberg et al., 2007).

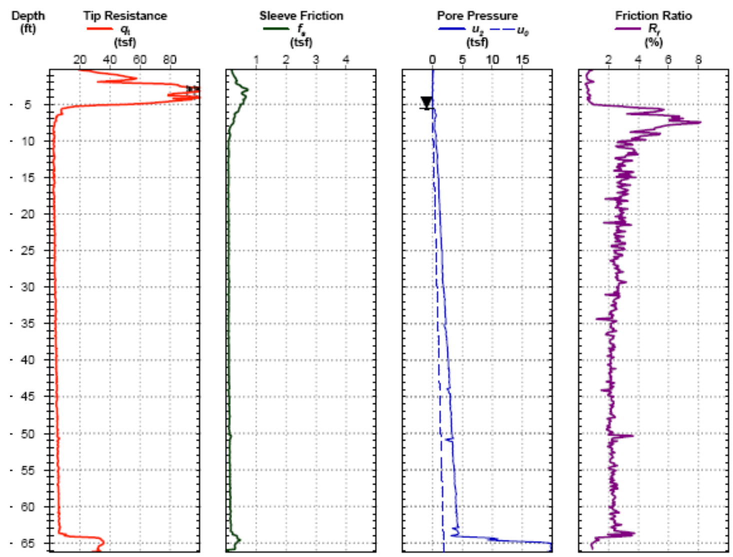

Figure 2. Typical CPT data from the general area of the embankment.

3 ROADWAY EMBANKMENT CONSTRUCTION

The roadway embankment design consisted of the placement of a clean sand drainage blanket (0.5 m thick), the installation of prefabricated vertical drains (PVDs) in a triangular grid with a spacing of 1.5 m, the placement of two high-strength geotextiles (separated by 0.5 m of fill), and the placement of approximately 4.5 m of granular fill. A temporary mechanically-stabilized-earth wall, with a height of 1.4 m, was required along the western edge of the embankment to allow for the required surcharge height.

Approximately 1.5 m of settlement was anticipated following a surcharge period of 9 months, and the remaining 1 to 2 m of surcharge or temporary fill was to be stripped prior to paving. Vibrating wire piezometers (VWP), magnetic extensometers (ME), groundwater observation wells (GOW) and settlement plates (SP) were installed along the embankment alignment to monitor the surcharge. Fill was generally placed at a rate of about 1.5 m per week and most of the embankment construction proceeded without problems. However, between stations 59+00 and 64+50, large lateral (>3m) and vertical movements (>1m up and down) occurred shortly after the fill reached a height of about 3.7 m. Large differential movements were apparent across the crest of the embankment but a scarp was not visible. This may have been due to continued fill placement and grading as the failure was occurring. A large bulge was apparent beyond the western toe of the embankment. The last surveyed cross section prior to the movement and the post-failure survey are shown in Figure 3A and a picture showing the lateral displacement of the MSE wall is shown in Figure 3B. An instrumentation cluster was located near the center of the failed segment at station 62. The data are presented in Figure 4.

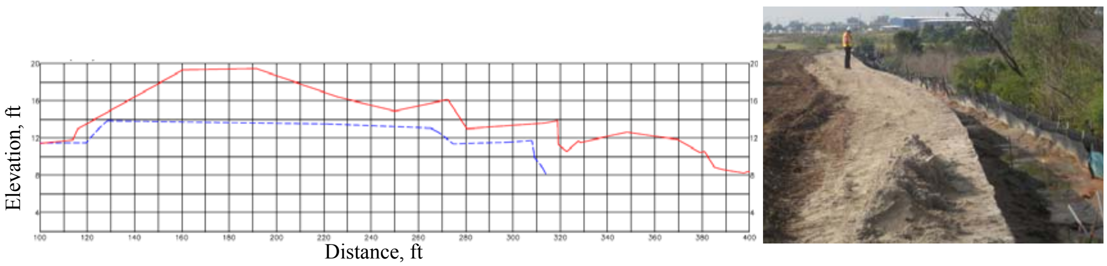

Figure 3. A) Surveys made approximately 1 week before failure (blue line) and 1 week after failure (red line) and B) photo illustrating the lateral displacement of the MSE wall.

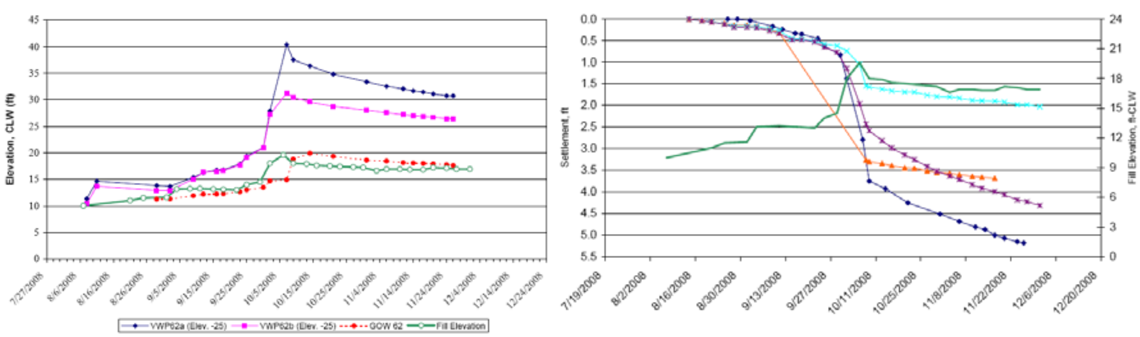

Figure 4. Pore pressure, expressed as piezometric elevation, and settlement data recorded near the midpoint of the embankment failure. The majority of the movement occurred on October 7.

4 FORENSIC EXPLORATION

Following the failure, the height of the embankment was approximately equal to the final design subgrade elevation (i.e., post-surcharge). Since substantial settlement was still anticipated, the performance of the roadway within the area of the failure would not meet the design objectives (i.e., pavement and utilities would be compromised by post-construction settlement) and a do-nothing approach was therefore not appealing. Consequently, a forensic evaluation was performed to provide a better understanding of the cause of the movement and to develop alternatives that would allow construction to continue while meeting the original design objectives.

The forensic exploration consisted of 23 CPT soundings and 5 soil borings with field vane testing (FVT) using an Acker Drill manually-operated geared drive-head vane system. A companion CPT sounding was performed at each vane shear test hole. The FVT results, which were corrected for soil plasticity in accordance with

Chandler 1988, were used to "calibrate" the CPT undrained shear strength correlations. Three CPT to S_u correlations were considered:

$S_u = (q_t - \sigma_{vo})/N_{kT}$ - where $15 \leq N_{kT} \leq 20$ (Lunne et al, 1997) (2)

$S_u = (u_2 - u_o)/N_u$ - where $7 \leq N_u \leq 9$ (Mayne & Holtz, 1988) (3)

$S_u = 0.091(\sigma_{vo}')^{0.2}(q_t - \sigma_{vo})^{0.8}$ (Duncan & Wright, 2006) (4)

where q_t is corrected tip stress; σ_{vo} and σ_{vo}' are total and effective vertical stress, respectively; u_2 is the dynamic porewater pressure measured at the shoulder position; u_o is the hydrostatic pressure, and N_{kT} and N_u are empircal factors. Using the FVT results as the reference undrained shear strength, N_{kT} and N_u factors were back-calculated that would yield reasonably good correlations with the expectation that the "best-fit" factor would be used at the CPT locations without FVT results. However, the agreement between the CPT strength correlations and the FVT results was inconsistent. One method and empirical factor would underpredict at one location and overpredict at another location. Some of these data are presented in Cargill and Camp (in press – this volume). Ultimately, the approach by Mayne & Holtz (1988) with N_u equal to 9 was selected as the most representative correlation. It should be noted that the exploration was performed using a 10 cm^2 cone penetrometer with a tip load cell capacity of 100 kN and a manufacturer's reported accuracy of 0.2%. The recorded tip stresses may have been near the lower-bound limit of the penetrometer while the u_2 pore pressures were within the normal range of operation.

A summary plot of the estimated shear strength at 13 CPT locations is presented in Figure 5. A plot of the "base case" design profile is also included in Figure 5 for comparison. In general, the estimated strengths are considerably less than the design profile. In some cases, the estimated strengths from the supplemental soundings were only half of the strengths assumed in the design profile. The supplemental soundings were concentrated around the perimeter of the failure site and more than tripled the amount of subsurface data as compared to what was obtained during the original subsurface exploration.

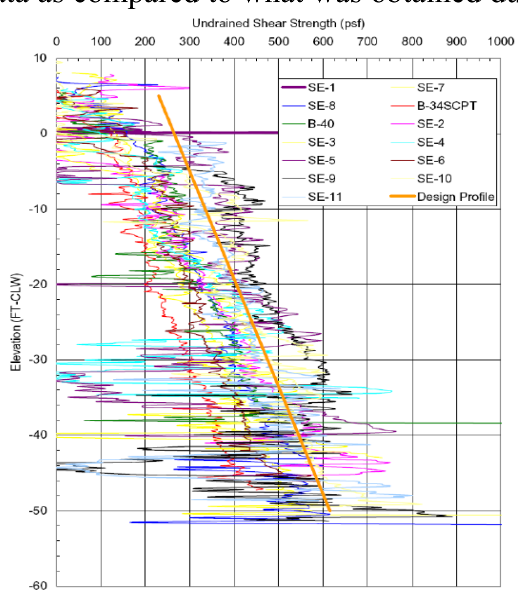

Figure 5. Summary plot of estimated undrained shear strength using Mayne & Holtz 1988 with N_u = 9. Assumed strength profile used for design is shown for comparison.

The supplemental exploration data clearly indicated that the strength within the area of the movement was anomalously low as compared to the assumed design profile. These new estimated strength profiles were obviously useful in evaluating the embankment failure but additional information was still needed. More specifically, it was not clear if the embankment had experienced a conventional rotational/bearing capacity failure or if a lateral squeeze failure had occurred (Bonaparte & Christopher, 1987). Consolidation induced strength gain would gradually improve the bearing capacity thereby making it possible to eventually increase the fill height. However, since lateral squeeze failures are generally controlled by the conditions adjacent to the embankment and outside of its footprint (i.e., a zone that will generally not experience consolidation induced strengthening), such a failure would mean that fill could not be added without additional measures (e.g., structural retention, ground reinforcement along the embankment perimeter, etc.) regardless of the available waiting period. Additionally, the back-analyses were very sensitive to the assumed condition of the two geosynthetic layers. The location of much of the vertical movement coincided roughly with the back edge of the upper geosynthetic (i.e., the failure surface could have passed behind it rather than through it) but a rotational failure surface would have had to pass through the lower geosynthetic layer.

To address these remaining uncertainties, an array of 15 CPT soundings was performed perpendicular to the embankment alignment within the area of greatest observed movement. The first sounding was located on the eastern slope of the embankment and each successive sounding was performed approximately 1 m to the west of the previous sounding. A schematic depiction of the embankment cross-section, the CPT sounding locations and the interpreted subsurface conditions is shown in Figure 6.

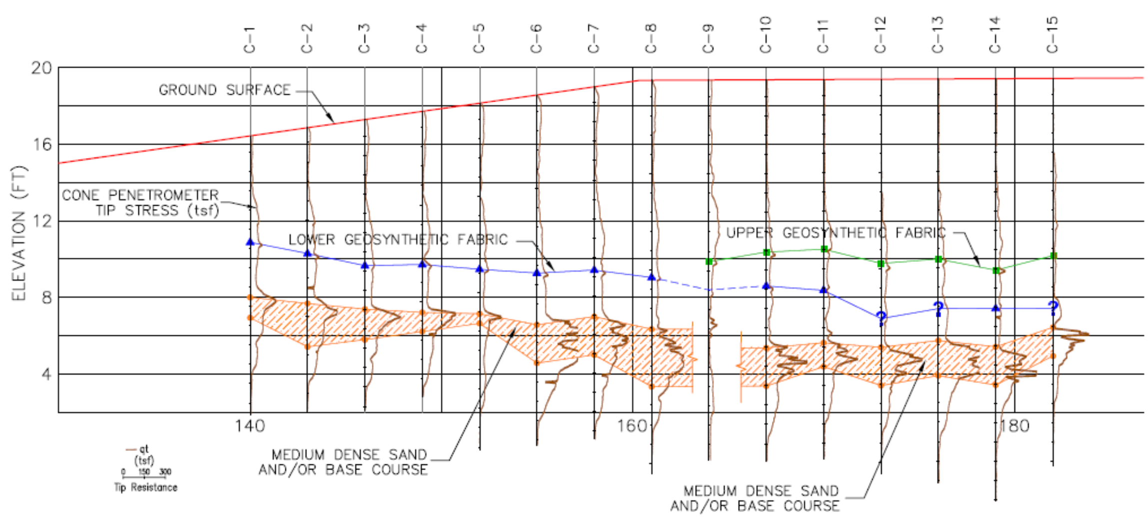

Figure 6. Cross-section of embankment with 15 supplemental CPT sounding locations spaced at 1 m intervals. The interpreted position of geosynthetic layers and pre-construction base course is also shown as well as the tip stresses from each sounding.

The CPT tip stress data were reviewed in an effort to identify the geosynthetic layers. The presence of the fabric was generally apparent as a slight increase followed by an immediate decrease in the tip stress, all occurring over a very short distance. Additionally, the CPT operator reported hearing/sensing a slight "pop" when the apparent geosynthetic "signature" occurred. The fabric was generally identifiable but it was not possible to conclusively determine if it had ruptured. The signature at the likely location of rupture (C-9) was not as obvious but it may still have been present. Additionally, a fabric signature was not observed in two locations where rupture was unlikely (i.e., upper fabric was apparent). However, an unanticipated component of the supplemental data was very beneficial.

The portion of the embankment that failed was located over a previously paved parking area, where the pavement was stripped but the relatively thin base course of sand and coarse aggregate was not removed prior the embankment construction. This "crust" material was readily apparent in all but one of the CPT soundings. The base course was missing in sounding C-9, which was located just beyond the end of the upper geosynthetic layer and where the failure surface would have likely passed through the crust. Back analysis indicated that if a rotational failure (as opposed to the end of a lateral squeeze failure) were to have occurred, the slip surface should have passed beyond the upper geosynthetic layer and through the base course near the location of CPT C-9. It was therefore concluded that a shear displacement had occurred at the location of C-9 thereby indicating that a bearing capacity/rotational displacement was the primary mode of failure.

5 REMEDIATION PLAN AND EPILOGUE

After concluding that the large horizontal and vertical movements were primarily due to bearing capacity/rotational failures, conventional limit-equilibrium stability analyses were used with the additional shear strength estimates from the forensic exploration to back-compute the strength profile at the time of the failure. In general, the back-computed strengths were in agreement with the strengths from the FVT and CPT correlations. Consolidation analyses and the results from CIU triaxial testing and the design-phase test embankments were used to estimate the consolidation-induced strength gain with time. Based on these findings, the embankment construction was modified to include two idle periods followed by two fill stages and then a final idle period before stripping to the design subgrade elevation. The modifications resulted in a total idle or surcharge period that was slightly longer than the originally anticipated period and a slightly lower final surcharge height. The large movements had damaged many of the instrumentation clusters and the revised plans included replacement instruments, additional monitoring (e.g., position surveys of the temporary MSE wall), and more frequent monitoring during fill placement.

The first stage of filling following the post-failure idle period proceeded without incident. Movements were not excessive and pore pressures and settlements generally responded as expected following the fill placement. Following the 2nd idle period, fill was placed to the final revised surcharge height and again, movements and instrumentation data were reasonable. However, 3 days after final filling, a small scarp (approximately 200 mm tall) appeared within the segment of embankment that experienced the previous failure. As compared to the previous movements, the scarp was located closer to the center-line of the embankment or west of what was previously determined to be the likely slip surface. The scarp height or differential between the eastern crest and the embankment mass to the west, gradually increased and reached a maximum value of about 1 m approximately 3 weeks after the completion of filling. It was speculated that the very slow gradual progression of this second failure is due to creep of the geosynthetic or progressive rupture of the fabric. It should be noted that the location of the scarp indicates that the slip surface likely passes through both layers of geosynthetic.

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