Field Observations from Saprolite Test Fills in High Rainfall Locations

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ABSTRACT
Earthworks constructed with fine grained, over-wet residual soils (10% to up to 30% wetter than the optimum water content from the standard proctor test) are unique and site specific. This paper summarizes field observations from two saprolite test fill trials conducted at high rainfall locations (over 3000 mm average annual precipitation) in Latin America. Saprolite wetter than the optimum moisture content (based on standard proctor) was placed and compacted using various equipment and methods. Results of geotechnical site investigations are discussed, including: cone penetration and vane shear tests before and after construction, material characterization, laboratory and in-situ testing, and instrumentation monitoring (pore water pressures and settlement). An overview of the fill performance, challenges for material borrowing, surface water management, equipment utilization logistics, construction techniques, compaction control procedures for quality control and quality assurance (QC/QA) are described and linked to design objectives and criteria to provide guidance for future construction activities at these sites.

RÉSUMÉ
Les mouvements de la terre avec des sols résiduels humides et fins sont uniques et spécifiques au site. Ce document résume les observations sur le terrain de deux essais de saprolite réalisés sur des sites à fortes précipitations (plus de 3000 mm de précipitations annuelles moyennes) en Amérique latine. La saprolite a été placée plus humide que la teneur en humidité optimale (Proctor standard) et compactée en utilisant divers équipements et méthodes. Les résultats des recherches sur les sites géotechniques sont abordés: essais de pénétration de cône et de cisaillement de palettes avant et après la construction, caractérisation de matériaux, essais en laboratoire et in situ, surveillance d'instrumentation (pression d'eau interstitielle et règlement). Il décrit une description générale des performances de remplissage, des défis pour les prêts matériels, la gestion des eaux de surface, la logistique d'utilisation des équipements, les techniques de construction, les procédures de contrôle de compactage pour le contrôle qualité et l'assurance qualité. Se rapportent aux objectifs et aux critères de conception pour fournir des orientations pour les futures activités de construction sur ces sites.

1 INTRODUCTION

Dams and embankments constructed with residual soil, under challenging climate or topographic settings, exist around the world and have well documented literature and case histories.

This paper summarizes field observations from test fill trials conducted at two high rainfall locations in Latin America and summarizes the lessons learned and challenges encountered. The intent is to contribute the collected information to the case histories database.

Details on terminology for describing or classifying residual soils are outside the scope of this paper. Pertinent information on this subject can be found in the following references: Burton (1998), Blight (1997), Deere and Patton (1971), Fookes (1997), ICOLD (2009), and Wesley (2010).

2 BACKGROUND

2.1 Site Characteristics

Characteristics of the two sites, Site 1 and Site 2, are summarized in Table 1. Both sites are located within the fersiallitic soil zone (high rainfall promoting high leaching) described in Figure 1 by Burton (1998). These sites experience average annual precipitation greater than 3000 mm distributed at regular intervals throughout the year (no dry season).

Both sites are unique in their topographic setting. Site 1 has steep mountainous terrain with steep and sharp ridges. Weathering of the granodiorite bedrock combined with surface runoff during heavy rainfall, created a condition of slow mass-wasting in the valley. The progressive regression of the valley slopes undercut the ridges and created matrix supported colluvium lobes creeping down-slope. Site 2 has eroded rolling topography with high, but smoother ridges, that create a larger open drainage catchment with various creeks across the area reporting into two main rivers. Surface water management at both sites is challenging because of high rainfall accumulation.

Table 1. Characteristics of Site 1 and Site 2

Characteristics	Site 1	Site 2
Average Elevation (m.a.s.l)	1500	100
Average Precipitation (mm)	3400	4500
Annual Evaporation (mm)	1100	1100
Main Rock Type	Granodiorite/ Monzonite	Granodiorite/ Andesite
Residual Profile Depth (m)	45	20
Topographic setting	Steep and sharp terrain	Rolling topography

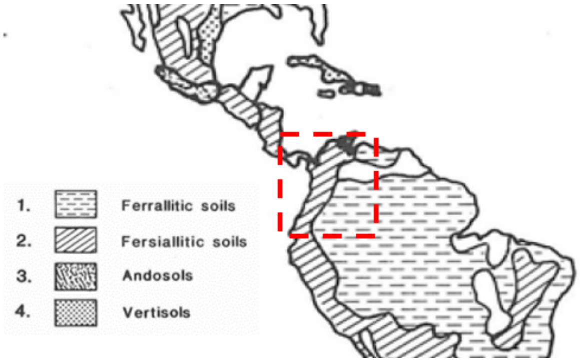

Figure 1. General site location shown on simplified world distribution of tropical soils (from Fookes, 1997; based on F.A.O. World Soil Map, extracted from Burton, 1998)

2.2 Design Objectives

Several site investigations were conducted at both locations from exploration through the beginning of construction. Site investigation programs included: mapping, test pitting, drilling, sampling, and laboratory and in-situ testing. Numerous challenges were encountered, such as: permits, accessibility to the areas, transport, experience of contractors, size of investigation equipment, and climatic effects to name a few. These limited some of the investigation locations and size of samples collected for testing.

Saprolite (soil-like material with fabric of the parent rock present as relic structure) is the main source for construction material to be used as a relatively impervious fill zone for seepage control and/or containment. However, its workability and suitability as dam fill material had to be confirmed due to its high natural moisture content (wetter than the optimum moisture content). Residual soils require more than conventional laboratory compaction methods to confirm workability as dam fill materials; therefore, both sites planned and scheduled test fill trials prior to construction.

Prior to the construction of the test fills, design analyses were conducted using values obtained from triaxial consolidated undrained (TXCU) on compacted samples. The testing used both undrained shear strengths of about 100 kPa and assumed dilative response to shearing with a drained friction angle and apparent cohesion approach.

As noted later in this paper, the test fills at both sites provided new insights on material behavior and performance that modified or added sensitivity analyses to the design parameters and/or approach developed after the site investigations.

2.3 Test Fill Planning

Historical data on existing dams or embankments built with saprolite fill in wet climate show that less compactive effort can provide better results. This is achieved by limiting breakage of bonds (i.e., chemical) between particles releasing structural water which influences material behaviour (Wesley, 2010).

Trafficability was found to be generally poor during placement in wet conditions; thus, most of the existing projects built with these materials allowed for placement of granular materials as part of the design section with restriction on fill placement zones to reduce the risk of tire slippage from trucks or overworking the saprolite. Sealing of the compacted layers with smooth rollers while maintaining a positive drainage grade, was common practice to promote runoff and reduce disturbance. The main objectives of test fill construction were as follows:

- Confirm the feasibility of construction with potentially over-wet saprolite (10% to up to 30% wetter than the optimum from a standard proctor testing) in a wet climate and allow evaluation of fill placement methodology, equipment requirements, trafficability, compaction effectiveness, and in-situ testing control.
- Determine a site-specific relationship for in-situ moisture content and compactive effort on the undrained shear strength and density for design.
- Observe foundation compressibility and strength behaviour under fill loading.

The location of the test fill extents was limited to accessible areas and had to be field-fit to current site conditions (Figure 2). Test fill areas were prepared and graded to a relatively flat platform. The test fill was located within the vicinity of the borrow area to reduce handling and haulage distance.

The proposed test fill dimensions included pre-established lanes of compaction and accesses routes. The staging of lift thickness, placement, compaction methods and testing frequency was planned in detail although left open to modification in the field. Experienced site personnel were key to the success of the test fill due to challenges related to equipment logistics and availability of staff.

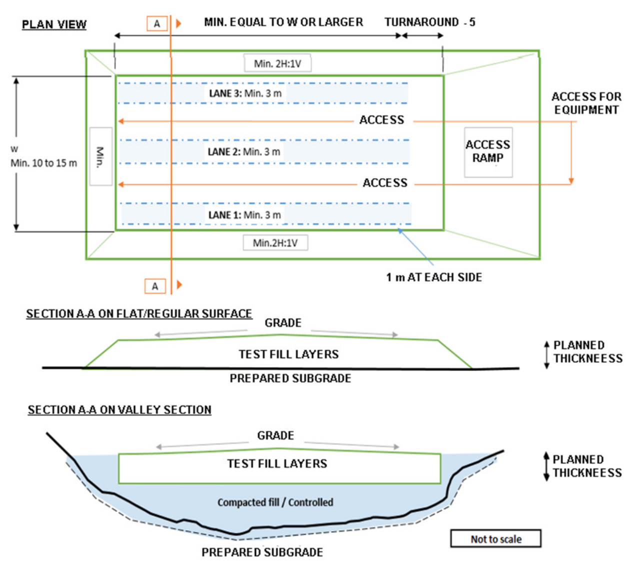

Figure 2. Reference diagram of test fill configuration

3 BORROW AND MATERIAL SPECIFICATIONS

3.1 Borrow Logistics

Borrow sites were established within a 1 km radius of the test fill area. Haulage was reduced to avoid rework or disturbance of material during rainfall. The borrow was developed with an excavator with a typical configuration 5 m high, minimum 5 m wide, including benches cut at a slope of 0.5H:1V to 1H:1V.

Stockpiling was not permitted to avoid saturation and no work could be conducted during rainfall. Construction downtime was estimated to be approximately 20% of the test fill construction duration due to rainfall events.

3.2 Material Characterization

At both sites, the saprolite was determined to be a low plasticity sandy silt (see Figure 3 to 5). At Site 1, the material was noted to have a finer particle size distribution near surface, which meant that the material used for the test fill at Site 1 consisted of a finer saprolite than used at Site 2 with less sand-sized particles and no gravel content.

Moisture content at both sites (see Figure 6) decreased with depth, ranging from 20% to 60% in the upper 5 m (excluding upper residual layer) to about 20% to 40% below 15 m depth. Laboratory testing of samples oven-dried at 50°C (Blight, 1997) and air-dried showed a difference of less than 4% compared to oven-dried at 110°C (ASTM D2216), indicating low influence of structural water in test results.

Standard proctor (ASTM D558) compaction tests were conducted on large samples to obtain the optimum moisture content (OMC) and the maximum dry density (MDD) from a five-point compaction curve involving drying and wetting of the sample. Additionally, one-point compaction tests were conducted to obtain the maximum compacted dry density at a given natural moisture content. Overall, the natural water content at both sites was between 10% and up to 30% over the OMC. Further information on this topic is provided in Section 4.

Continuous testing control for moisture content and compaction should be conducted as maximum dry density varies significantly as material properties change.

Figure 3. Typical photograph of saprolite soil

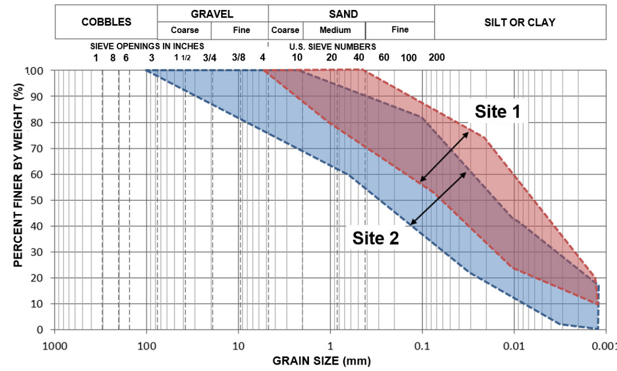

Figure 4. Typical particle size distribution at Sites 1 and 2

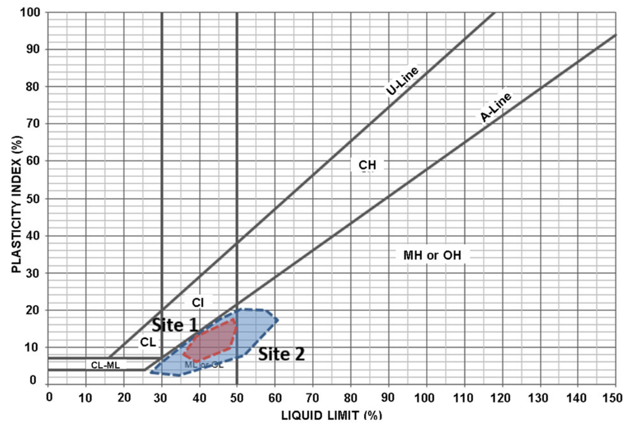

Figure 5. Typical plasticity data from Sites 1 and 2

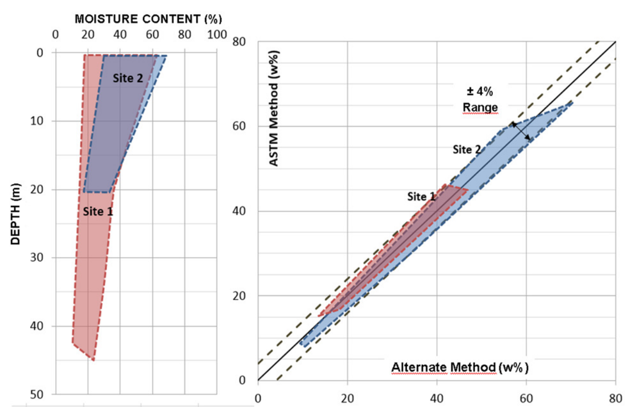

Figure 6. Natural water content data from Sites 1 and 2

4 TEST FILL CONSTRUCTION

4.1 Foundation Preparation and Investigations

Foundation preparation was conducted at both sites to control surface water and prepare the subgrade to acceptable levels for the test fill compaction.

After clearing and grubbing, the foundation was prepared to a targeted undrained shear strength over 50 kPa. Typically, greater strength was encountered after stripping exposing a firm to stiff foundation. When required or as identified during proof-rolling, coarse material was used to gain accessibility. However, these areas were surveyed and limited to areas outside the top surface limits of the test fill to avoid influence in measurements. At both sites, instrumentation for monitoring of the test fill foundation was installed during foundation preparation, including:

- Fully-grouted vibrating wire piezometers (VWPs) typically installed below 1.0 m to 4.0 m (some nested) from the finished subgrade and below the observed water level. Piezometer cables were routed inside trenches towards dataloggers outside the test fill area and initialized for continuous recording.
- Settlement was monitored differently at both sites. At Site 1, settlement plates were installed at subgrade levels and raised with each subsequent lift. At Site 2, pneumatic settlement cells were used.

At Site 1, seismic cone penetration testing (SCPTu) and electric vane shear testing (e-VST) where conducted with lightweight equipment (15 tonne push) prior to and after the test fill construction. Investigations at Site 2 were limited to hand-held VST. Relevant observations included:

- At Site 1, the subgrade was prepared by excavating about 4 m into the saprolite to a relatively flat area. At this depth, the SCPTu showed stiff to very stiff clay-like dilative response with a median estimated-undrained shear strength (S_u) of about 180 kPa (N_{kt} of 10 calibrated with e-VST) increasing with depth, medium to low compressibility with a medium to low (horizontal) hydraulic conductivity, except for the upper 1.5 m (around $4x10^{-7}$ m/s). One refusal below 15 m depth showed a thick zone of sand-like granular coarser-grained saprolite near an excavated ridge. However, the transition to bedrock was inferred at 30 m depth in the area.
- As Site 2, foundation subgrade had to be raised over its natural grade as shown in the valley section in Figure 2. Foundation condition of the native ground was firm to stiff, with S_u ranging from 70 kPa to 140 kPa, with a median value of 93 kPa. Some records showed values as low as 50 kPa.

4.2 Fill Placement and Compaction

Various compaction methods were used at both sites to make a qualitative and quantitative comparison based on workability, trafficability, and compaction results. Equipment included: vibratory plate tamper (0.25 tonne), manual smooth roller double drum compactor (0.6 tonne), sheepsfoot rollers (10 to 12 tonne), smooth drum rollers (10 to 12 tonne), wide track D6 dozers (18 tonne), and excavators (22 tonne).

Material was spread with dozers (large areas with short spreading distances) and/or placed with the excavator (small areas where less handling is required). Lift thickness varied from 250 mm to 500 mm with increasing number of passes from 2 to 8.

VW piezometers were also installed within the fill at least 1.0 m to 1.5 m above the subgrade. When possible, push-in VWPs were installed. In-situ testing and sample collection was conducted at each lift after the required number of passes was achieved.

Observations from both sites were similar in terms of compaction methodology, as follows:

- Saprolite moisture content was found to be 10% to 15% wetter than the OMC at Site 1 and from 10% to up to 30% at Site 2. Only a few tests were near OMC. Typically, saprolite with water content over 38% (Site 1) or over 40% (Site 2) resulted in poor trafficability and performance. See Figure 7.
- Fewer passes by the dozer provided better trafficability and compaction results due to less breakage of bonds between particles. A trial with several dozer passes showed that over-compacting the material reduced strength and increased moisture making it less trafficable for construction equipment.
- Sheepsfoot or dozer track-packing resulted in irregular surfaces that promoted water ponding with increased frequency when thinner lifts were used. A smooth roller was preferred. Although vibration with smooth roller did not show significant decrease in properties, vibration is not recommended when over-wet soil and/or thin lifts are used.
- Sealing and grading (min. 2%) the finished surface with a smooth roller without vibration improved erosion control and drainage significantly. After heavy rainfall, the saprolite surface softened about 0.15 m. A plastic cover was often used to protect the areas overnight; however, small areas required about two hours for installation. This is not practical on a larger scale. Sealing and grading is preferred.
- Varying the number of passes showed that it is questionable to achieve an undrained shear strength of 50 kPa with more than four passes. Even if disturbance is not observed, over-compacting the area will not be beneficial and will only increase equipment time and work duration.
- A lift thickness of 400 mm, loose, with four passes using a minimum 10 tonne smooth roller (static) was established as the accepted procedure; however, performance should be monitored during construction. Water content should be below the critical value established for workability.
- In-situ dry density was obtained using the sand cone method (ASTM 1556). Percentage compaction compared to five-point and one-point standard proctors showed a broad range of results from 85% to over 100% (larger compactive effort than standard test) with no obvious trend of undrained shear strength and moisture content. See Figure 8.
- Manual VST (ASTM D2573) at each lift showed increasing undrained shear strength (S_u) with tested depth (increasing confinement). At Site 1, the S_u ranged from 25 kPa to about 90 kPa with an average of 50 kPa (firm to stiff). At Site 2, the S_u ranged from 28 kPa to 50 kPa, average of 40 kPa.

As anticipated, the compacted strengths were generally lower than native saprolite caused by breakage of the particle structure.

Target values for quality control and quality assurance (QC/QA) included but were not limited to: compliance with the design material specification, moisture content control at the borrow, lift thickness control, number of passes, undrained shear strength, in-situ dry density and percentage compaction, and hydraulic conductivity.

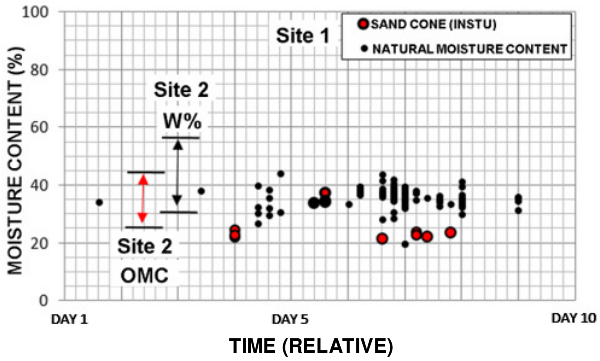

Figure 7. Natural and optimum water contents

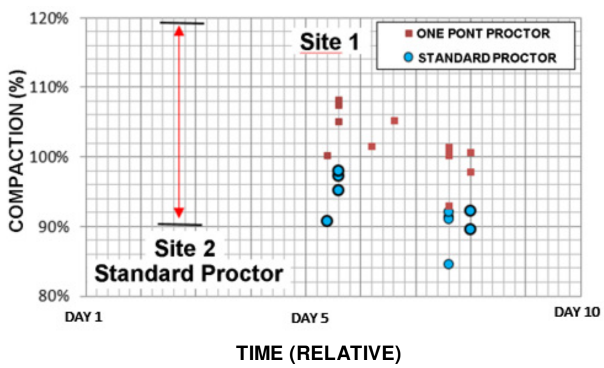

Figure 8. Percentage compaction from five and one point standard proctors

5 INSTRUMENTATION MONITORING AND POST-TEST FILL INVESTIGATIONS

5.1 Instrumentation Monitoring

Instrumentation installed at Site 1 had a monitoring duration that lasted only a few weeks after the test fill completion. Site 2 instrumentation was monitored for about 9 months after test fill completion until the instrumentation was damaged. Figures 9 and 10 show elevation of the fill and piezometer readings (total head) with time.

Piezometers within the foundation at both sites showed response to fill placement followed by rapid pore pressure dissipation midway through fill placement (likely due to drained foundation) with relatively steady pore pressures noted several days after test fill completion. Minor influences due to changing groundwater levels and rainfall are observed at that time. The ratio between change in pore pressure to change in vertical stress (B-bar) ranged from 0.05 to 0.2 for Site 1 and from 0.05 to 0.1 for Site 2.

Piezometers installed within the fill at Site 1 showed fast, elevated responses to fill placement and construction equipment. At Site 1, construction-induced excess pore pressures remained above fill levels for several weeks after test fill completion. At Site 2, readings stabilized 5 months after completion. Full dissipation was not observed at the time of the last recorded readings.

Unfortunately, settlement plates at Site 1 were destroyed or damaged during construction. The collected data, although limited, showed some agreement with anticipated response of a medium to low compressibility, stiff to very stiff foundation under the stress imposed which was less than the excavated depth. At Site 2, settlement data was not readily available and could not be presented herein.

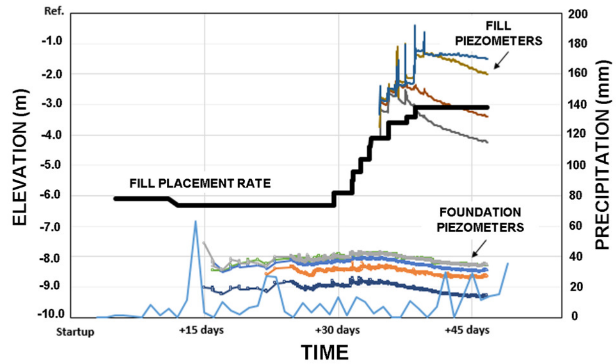

Figure 9. Site 1: Foundation and fill piezometers records

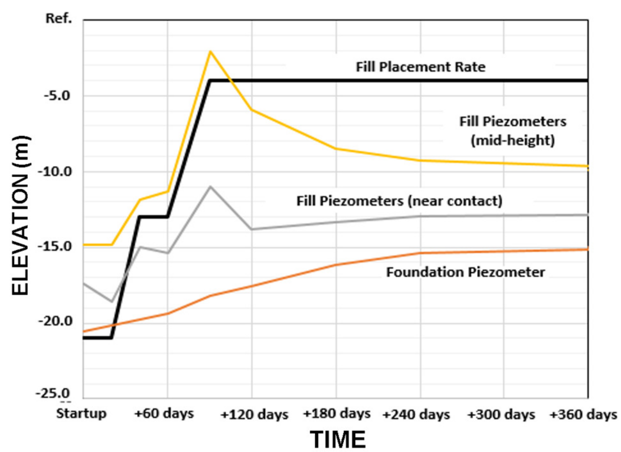

Figure 10. Site 2: Foundation and fill piezometers records

5.2 Post-Test Fill Site Investigations

Site investigations conducted after the test fill completion at Site 1 included: infiltration tests in manually excavated test pits, e-VST, and SCPTu located at same locations of pre-test fill investigations (see Figure 11).

Some of the relevant observations included:

- Infiltration testing was not successful as the manually excavated test pits walls, 0.6 m to 1.0 m depth collapsed upon saturation.
- SCPTu testing showed good agreement with recorded fill procedures and observations. Estimated strength profiles showed firm to stiff compacted fill with dilative response to penetration and a stiff to hard foundation. No obvious trend of increasing strength with depth was observed in the

compacted fill at this thickness. Shear wave velocity was about 100 m/s for the compacted fill.

- Although not conclusive, some influence of the test fill was observed in the first few metres of the foundation in contact with the test fill. Shear wave velocity slightly increased and slower dissipation than pre-test fill values were observed beneath the contact with the subgrade.

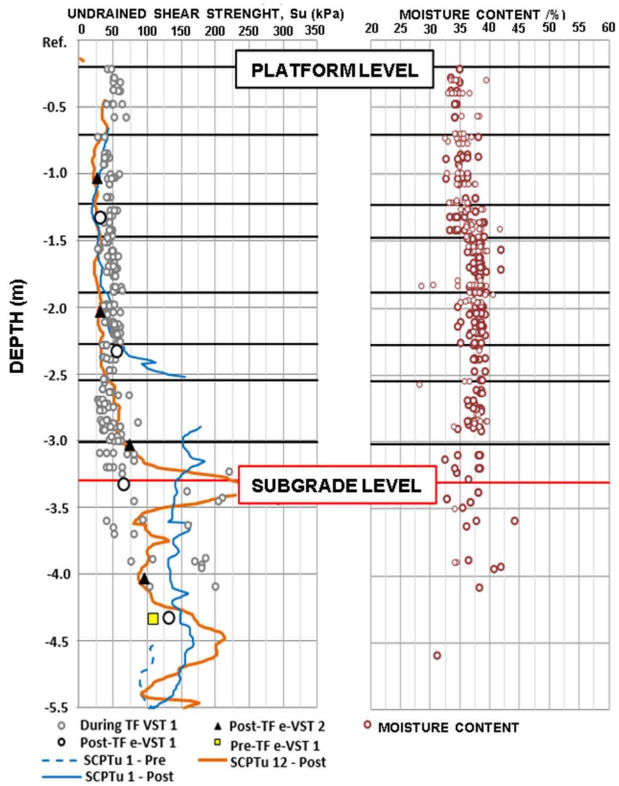

Figure 11. Site 1 - SCPTu and water content profiles

6 CONCLUSIONS

The following conclusions were noted:

- Construction trials were critical to understand performance, site-specific design considerations (maximum strength achievable), and to optimize specifications. Pragmatic experience in the field also showed the contractor key elements of the design and control.
- Modifications to the design included the use of lower undrained shear strengths than those obtained during the site investigations and use of higher construction-induced excess pore pressures within the compacted fill for short-term analysis.
- Although no obvious correlations were made between the S_u and other parameters, a wide range of data was obtained for sensitivity assessments.
- QC/QA procedures were established as critical key performance indicators for performance monitoring. Moisture content control at the borrow, a minimum compacted undrained shear strength based on trials, and a percentage compaction over 95% of the standard proctor or 100% of the one-point compaction at natural moisture content.
- SCPTu testing showed good agreement with site observations during construction and proved to be a non-invasive investigation tool to assess performance and identify potential non-conformance.
- A comprehensive database was critical for the evaluation. Implementing this during the early stages of the design and construction eased data review.

7 REFERENCES

Burton, B. 1998. Earthworks with Wet, Fine Grained Tropical Residual Soils. Master of Engineering Report. Civil Engineering-Geotechnical. University of Alberta. September 8, 1998.

Blight, G.E. 1997. Mechanics of residual soils: a guide to the formation, classification and geotechnical properties of residual soils, with advice for geotechnical design. Balkema/Rotterdam/Brookfield. 2nd edition.

Deere, D.U. and Patton, F. 1971. Slope Stability in Residual Soils. *In* Proceedings, of the 4th Pan American Conference on Soil Mechanics and Foundation Engineering, San Juan. Vol. I: 87-170.

Fookes, P.G. 1997. Tropical Residual Soils – A Geological Society Engineering Group Working Party Revised Report. The Geological Society. London, 184.

ICOLD, Tropical Residual Soils as Dam Foundation and Fill Material – Draft, January 2009.

Wesley, L.D. 2010. Geotechnical Engineering in Residual Soils. John Wiley and Sons Inc. 1st Edition.