RABY BAY CANAL BATTER STABILITY UPDATE

Geotechnical Investigation Analysis Report

Prepared for:

REDLAND CITY COUNCIL
PO Box 21
Cleveland QLD 4163

Prepared by:

Kellogg Brown & Root Pty Ltd
ABN 91 007 660 317
GPO Box 633, Brisbane QLD 4001
Telephone (07) 3721 6555, Facsimile (07) 3721 6500

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Acknowledgments

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Revision History

Revision	Date	Comment	Signatures		
			Originated by	Checked by	Approved by
A	12/07/12	Issued to Client for review			
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Summary

In 2010, Kellogg Brown & Root Pty Ltd (KBR) was commissioned by Redland City Council to develop a geotechnical investigation plan to gain an appreciation of the scale of the slope stability problem within the Raby Bay Canal Estate development and to examine rectification options. Based on the factual geotechnical reports 1-13601BR and 1-14061BR provided by Soil Surveys (2012a & b), a summary of KBR's interpretative notes and recommendations follows:

KBR's interpretation of the geotechnical investigation test results:

- the soil strata profile is varied throughout the canal estate, with no obvious spatial pattern
- there appears to be a 'wedge' of uncompacted fill underneath the canal batter rock protection and concrete wall
- under the house platforms and roads there appear to be compacted fill, either imported or sourced from the canal cut
- underlying these two materials there is native very stiff clay overlying clayey sand, hard clays and extremely weathered rock
- the inclinometer measurements indicate movement at every location tested. The movement is more pronounced above –4.0 mAHD. It is not reported whether small indicated movements are due to soil distortion of movement of the inclinometer tube inside it's borehole
- generally, soil shear strength properties increase with depth. In particular, undrained cohesion from the dilatometer tests shows that shear strength increases notably below approximately –4.0 mAHD.

KBR's slope stability analysis confirms the conclusions presented in report 1-14061BR, indicating a relatively shallow failure mainly confined to the uncompacted fill material under the concrete wall and rock protection. Deep slip failures are not indicated.

The recommended rectification plan:

- the existing approach of using screw piles appears to be an overdesign for the shallow failure observed
- grout injection into the uncompacted fill is suggested as an alternative. This option appears to be significantly cheaper than current methods. A preliminary concept for this option involves 600 mm diameter soil/cement piles formed to a length of approximately 3 m at about 1 m centres
- kbr recommends some test rectification sites be built and instrumented, plus a finite element soil model of the tests. The object of the tests and finite element model is to confirm and refine the design. Taking into consideration that there are approximately 20-25kms of canal frontage that might require rectification, optimising the design of the remedial works will generate significant savings for Redland City Council

- it is suggested that periodic laser scanning surveys be carried out to monitor movement of the canal batters and retaining wall. This will inform a strategic plan for managing and implementing rectification works prior to significant damage to the infrastructure.

Revision A of this report was reviewed by Redland City Council (RCC) Project Delivery Group (PDG) 21-09-2012), the Raby Bay Residents Association (01-11-2012) and GHD (01-03-2013). The main outcomes of these reviews, included in this Revision 0, are:

- hypothesised deep seated slip circle slope failures due to the possible presence of a fissured stiff clay stratum are not indicated by the slope stability analysis, nor are they observed in the field. According to the literature, the long term shear strength of the fissured stiff clay does not appear to be much affected by the presence / absence of these fissures (Spangler & Handy 1973, p445; Coduto et al 2011, p582)
- the canal cross sections and soil strata are expected to vary somewhat throughout such a large site and hence rectification works should be adjusted accordingly
- at some properties the current RCC surcharge criterion of 2.0kPa has been exceeded and hence a higher surcharge load should be taken into account at these places
- ‘pre-failure’ and ‘during-failure’ rectification works will necessarily be somewhat different
- whilst past slope movement monitoring methods were appropriate and economic, the recent rapid reduction in the cost of laser scanning methods means that these methods should be considered for future monitoring.

1 Introduction

Raby Bay is a residential canal estate located at Cleveland, Redland City in southern Moreton Bay, Queensland. The site was formerly mainly tidal wetlands. The estate was developed using a cut to fill method, constructed in the dry and subsequently flooded. During and after construction some of the canal batters failed in a classic slip circle fashion.

On a case-by-case basis where the failures occurred, various geotechnical consultants have been engaged to address slope failures and other ground movements over the history of the development. Various remedial responses and restoration methods have been employed generally with technical success, but at high cost. The complexity of mechanisms behind ground movements and slope failures, and the very high projected costs of restoration works have led the Redland City Council (RCC) to investigate more permanent and economical approaches to the problem.

Kellogg Brown & Root Pty Ltd (KBR) was engaged by RCC to summarise the findings from a geotechnical investigation at Raby Bay, Queensland. This extensive geotechnical investigation was performed by Soil Surveys Engineering Pty Limited between January and April 2012.

This report presents a summary of the recent geotechnical investigation and is to be read in conjunction with geotechnical reports 1-13601BR and 1-14061BR provided by Soil Surveys.

Revision A of this report was reviewed by Redland City Council (RCC) Project Delivery Group (PDG) 21-09-2012), the Raby Bay Residents Association (01-11-2012) and GHD (01-03-2013). The outcomes of these reviews are included in this Revision 0.

2 Geotechnical data review

2.1 INTRODUCTION

This geotechnical review is based on information provided in reports 1-13601BR (April 2012) and 1-14061BR (March 2012) prepared by Soil Surveys for RCC. These reports contain the geotechnical data from a total of 20 locations in the Raby Bay canal estate. Of particular interest are boreholes from Piermont Place, where a slope failure was occurring during the testing.

2.2 KEY OBSERVATIONS

The data provided in the reports has been reviewed by KBR to identify reasonable patterns and particular observations that may be relevant to the slip failure that is being observed at sites within the canal estate.

A soil profile summary for each borehole location, along with its relevant soil property data, is presented in Appendix B. Key observations have been made with respect to the soil strata profiles, displacements recorded by the inclinometer and soil strength parameters.

2.2.1 Soil strata profiles

A typical canal cross section is shown in Figure 2.1 based on Cardno & Davies Drawing 956/1-37 in Appendix D. The cross section varies somewhat through the estate.

In report 1-13601BR by Soil Surveys it is noted that significant variation in the borehole logs throughout the canal sections was observed. Additionally, the report also makes comment on the presence of a ‘thin layer of soft to firm clay immediately under the revetment rock’.

Relying mainly on the borehole data and the simplified soil profile of the Piermont Place slope in report 1-14061BR, the soil profile as shown in Figure 2.1 is believed to be typical of Raby Bay. This soil profile layout has been used for the basis of the slope stability calculations in Chapter 3.

The interpretation of this profile is as follows:

- the original ground surface was approximately zero AHD (Department of Harbours and Marine, Peel Island to Russel Island Small Craft Chart, 1979)
- organic marine clay was stripped down to approximately RL -2.0 mAHD where a very stiff clay was encountered. This clay is probably a ‘residual soil’ from when

sea levels were lower in the last Ice Age (from 6,000 years ago when sea levels stabilised)

This stiff clay has been observed to be possibly 'fissured'. This means that at some time in the past the clay was subjected to wetting and drying and hence cracking; the cracks subsequently filled with loose material (Bowles 1988, p81)

- the canal invert was dug in the dry. The resulting clayey sand and very stiff clay was used as compacted fill under houses and road – the 'stiff clay' layer. The 'clayey sand' in the canal invert might be an extremely weathered rock that looks like a clayey sand
- 'general fill' was imported and compacted under house platforms and roads
- in order to build the rock armour and concrete wall, the fill in this area had to be brought up to profile. The usual method is to overfill this area slightly with the 'stiff clay' or the imported 'general fill' and compact with rollers in 300 mm layers. After compaction the profile is cut using an excavator. Instead it appears that 'foundation fill' was pushed into the 'wedge' between the 'stiff clay' batter and the design profile and not compacted (i.e. left loose). This 'wedge' is difficult to compact; a vehicle roller might not have safe access so hand rollers or compaction plates or the addition of cement would have to be used – all of these are expensive; hence it appears many Raby Bay canal frontages have an uncompacted fill wedge under the rock protection and concrete wall
- the author's interpretation of the geotechnical data is the observed slope failures are largely confined to this uncompacted 'foundation fill' wedge, thus the slips appear to be shallow and short in length.

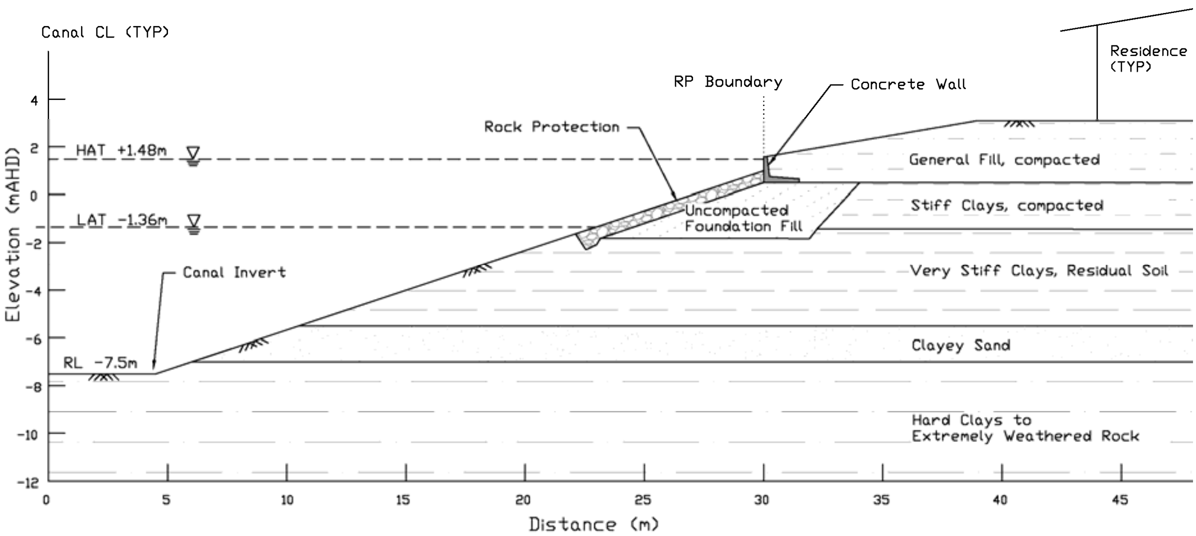

Figure 2.1
SIMPLIFIED SOIL PROFILE LAYOUT

Figure 2.1 is based on construction drawings, Soil Surveys report and KBR interpretation. An A3 copy of this diagram is included in Appendix C.

2.2.2 Displacements

Of the 17 borehole locations reported in 1-13601BR there are six locations that do not have displacement measurements recorded following the initial installation of the inclinometer. Of the locations with displacement data, it is important to note that all borehole locations are showing indication of soil movement. Most of the observed soil movement is above the toe of the rock protection. However movement is indicated down to about RL –5.0 mAHD. Below approximately RL –2.0 m to –3.0 mAHD the reported inclinometer deflections are quite small. It might be possible that the inclinometer tube is moving inside the borehole if it was not tightly backfilled and/or the inclinometers weren't fully 'zeroed'. We have assumed:

- 75 mm diameter chopping tip (i.e. hole diameter)
- 63.5 mm diameter casing
- 58 mm diameter OD inclinometer.

This information suggests that the soil slip is occurring in a shallow zone underneath the rock protection and concrete wall. As the soil begins to move in these higher layers, the movement stresses the lower soil layers which result in the small displacements observed here. Figure 2.2 and 2.3 show the displacement records from the boreholes demonstrating this pattern of soil movement.

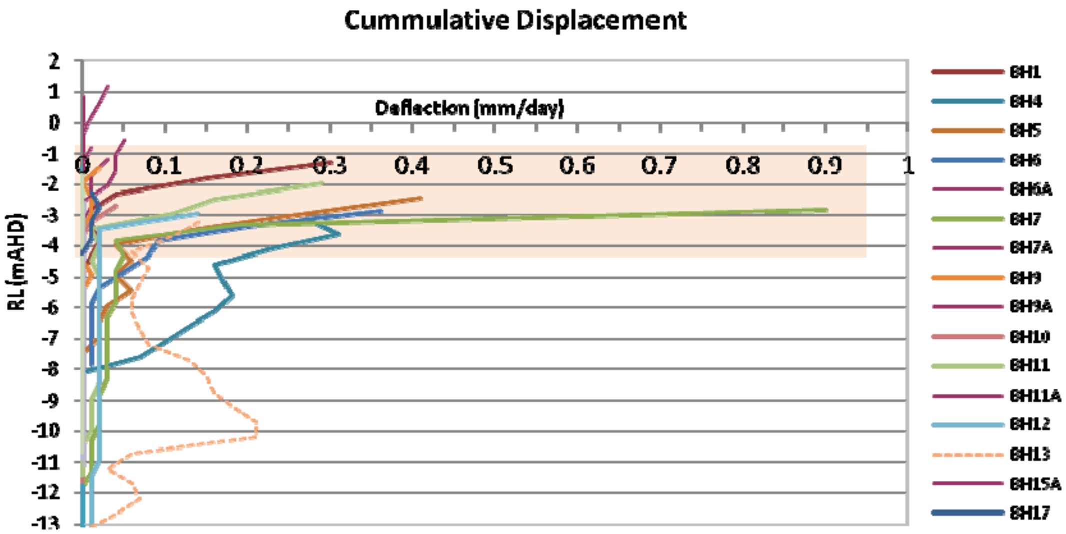

Figure 2.2
BOREHOLE DISPLACEMENT RECORDS

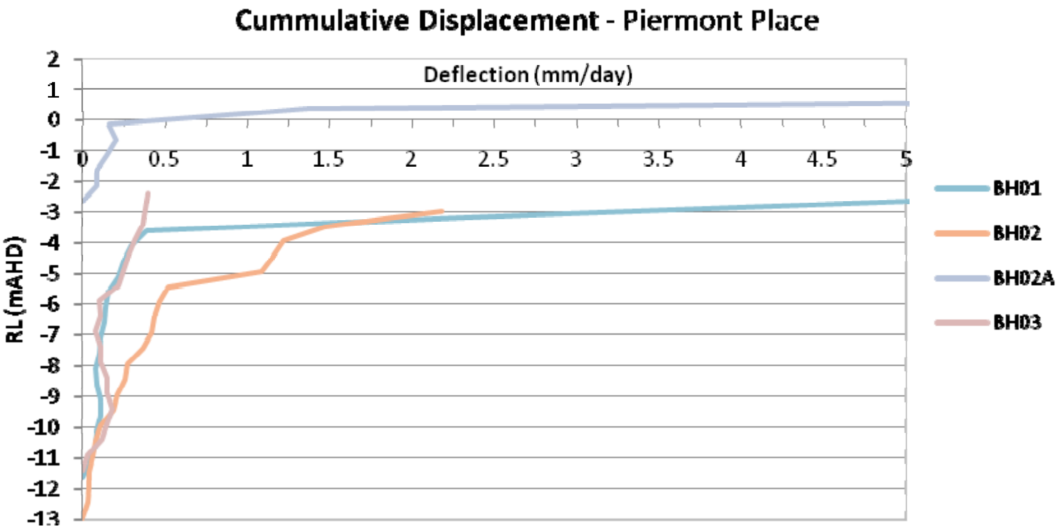

Figure 2.3
BOREHOLE DISPLACEMENT RECORDS – PIERMONT PLACE

2.2.3 Soil properties

Soil density, pore water pressures, cohesion and friction angle values are the main properties that affect slope stability. Figure 2.4 shows two plots of undrained cohesion values c_u with respect to depth. The first plot includes all data from the dilatometer tests, while the second shows these values averaged over 0.5 m bins. It is important to note that there is a significant increase in average recorded shear strength at approximately –4.0 mAHD. This is consistent with the typical level where lateral displacement is first observed in the borehole displacement records in Figure 2.2. It is inferred that this is near the level where very stiff clay was encountered after soft overlying material was removed during construction.

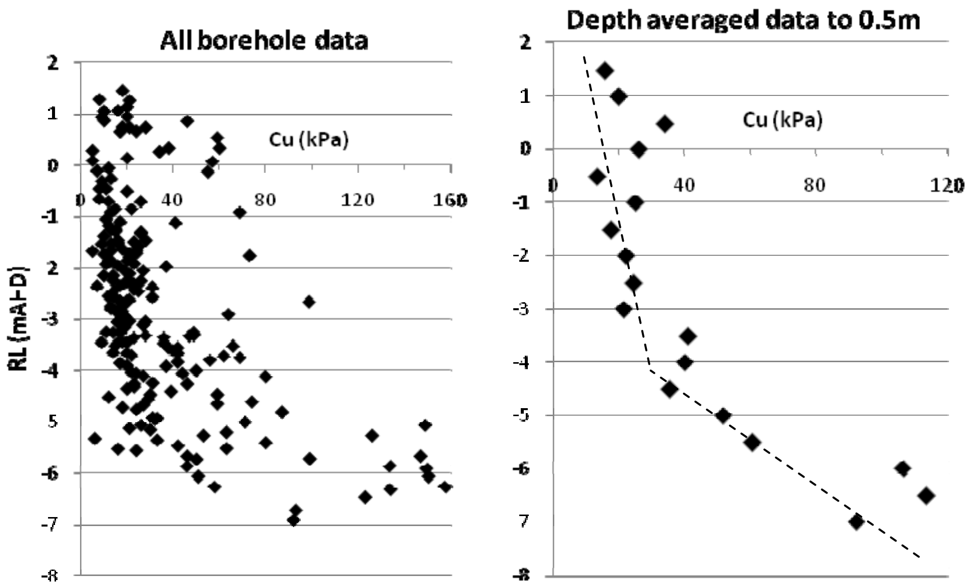

Figure 2.4
UNDRAINED SHEAR STRENGTH MEASUREMENTS FROM DILATOMETER TESTS

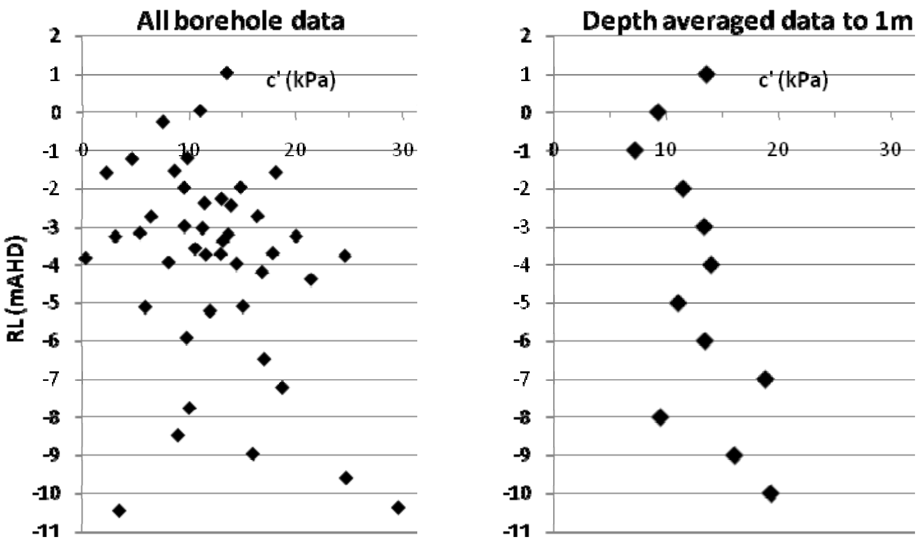

Figure 2.5
DRAINED SHEAR STRENGTH VALUES FROM TRIAXIAL TESTS

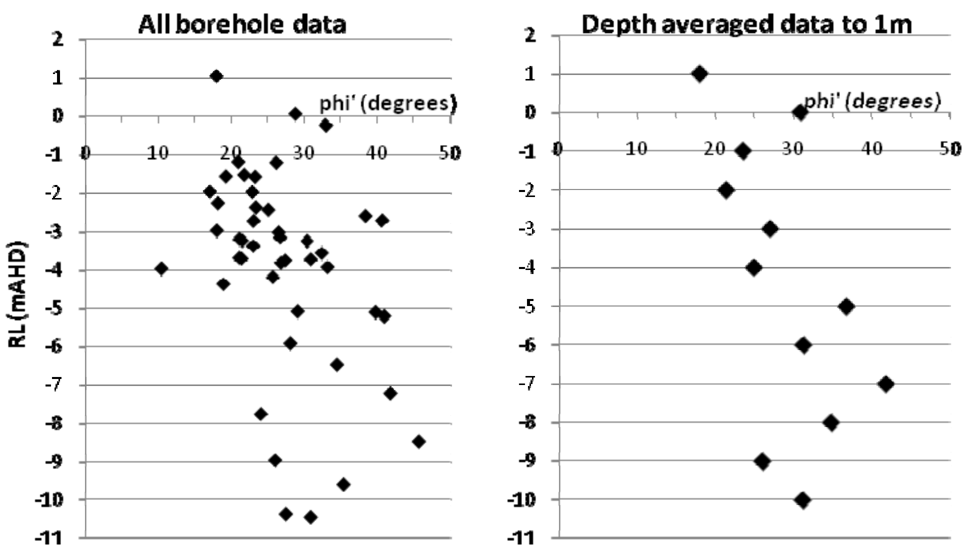

Figure 2.6
DRAINED FRICTION ANGLE VALUES FROM TRIAXIAL TESTS

Note that the tested shear strength of the stiff clay can be affected by the presence / absence of fissures in the test samples (Bowles 1988, p81)

3 Slope stability modelling

3.1 INTRODUCTION

Slope stability analyses were conducted to:

- test the analysis method and assumed soil properties for the Piermont Place. situation where failure was occurring and hence the Factor of Safety (FOS) ≈ 1.0
- test the efficacy of a typical low cost shallow failure repair method: grout injection.

The slope stability analysis was performed using the commercially available software, Geostudio (Slope/W) 2007 version 7.14. Two-dimensional Coulomb (slip circle) method was used with the Morgenstern-Price interslice stress assumptions. The assumed soil parameters are based on consolidated test data in Soil Surveys (2012a&b).

The slope section was analysed using the following assumptions:

- slope geometry as per Drawing No. 956/1-37, located in Appendix D.
- soil profile similar to Soil Surveys (2012a).
- effective strength parameters (i.e. long-term, drained condition) have been assumed, as this is the critical case.
- a slip circle with a factor of safety (FOS) of approximately 1.0 indicates slope failure.
- for the addition of remedial work to the slope, the minimum required stability FOS is 1.5. The key reference for the appropriate FOS is AS 4678 Earth retaining structures code. Clause 4.1 (iii) recommends an FOS =1.5 to be consistent with the loading codes AS 1170 series.

Borehole geotechnical data for each location is summarised in Appendix B.

3.2 ASSUMPTIONS

A number of assumptions and simplifications have been adopted in the slope stability analysis:

- the RCC recommended maximum 2.0 kPa surcharge is applied to the slope above the concrete wall in all cases and represents loads from swimming pools, decks and filling. Some properties appear to have surcharges that exceed this load
- soil is fully saturated behind the revetment wall following a heavy rainfall event (i.e. water table at the surface)

- canal water level at LAT
- an attempt has been made to match modelled mode to resemble the slip observed
- grout-injected piles have been used in the models to demonstrate a plausible low cost slope stabilizing option. The shear strength of the piles are based on a soil/cement compressive strength of 10 MPa
- as there is no recent survey data available, the as built profile has been adopted for models (see Appendix D). As built thickness of rock protection is assumed to be 0.5 m.

3.3 SLOPE/W MODELS

The Slope/W model is shown in Figure 3.1. The assumed soil strength parameters are shown in Table 3.1.

Table 3.1 Soil strength parameters (long term)

Soil Description	Saturated Unit Weight in Air (kN/m^3)	Cohesion (kPa)	Phi (degrees)
Concrete Wall	24	4000	0
Rock Protection	20	0	30
General Fill	18	0	26
Foundation Fill	17.5	2	18
Stiff Clays (refer Note 1)	17.5	5	26
Very Stiff Clays (refer Note 1)	17.5	10	27
Clayey Sand	18	2	30

Note 1. It is possible that that these clays are ‘fissured’. Whilst fissures are expected to reduce the short term shear strength, the literature advice (Spangler & Handy 1973, p445; Coduto et. Al. 2011, p582) is that the long term shear strength is not much affected by the presence / absence of fissures. Hence the long term analyses that follow are reasonably applicable to both fissured and non-fissured stiff clays.

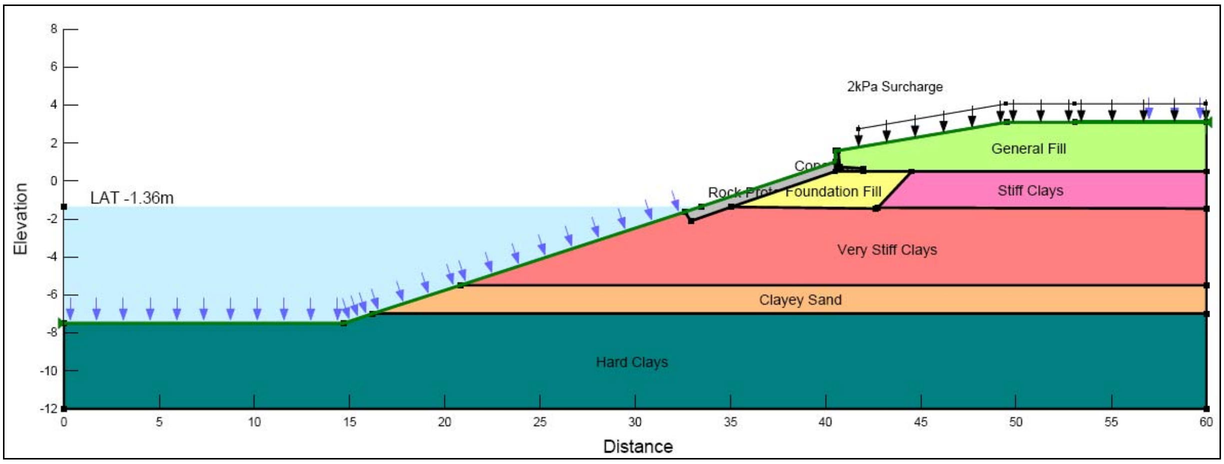

Figure 3.1
SLOPE/W MODEL LAYOUT

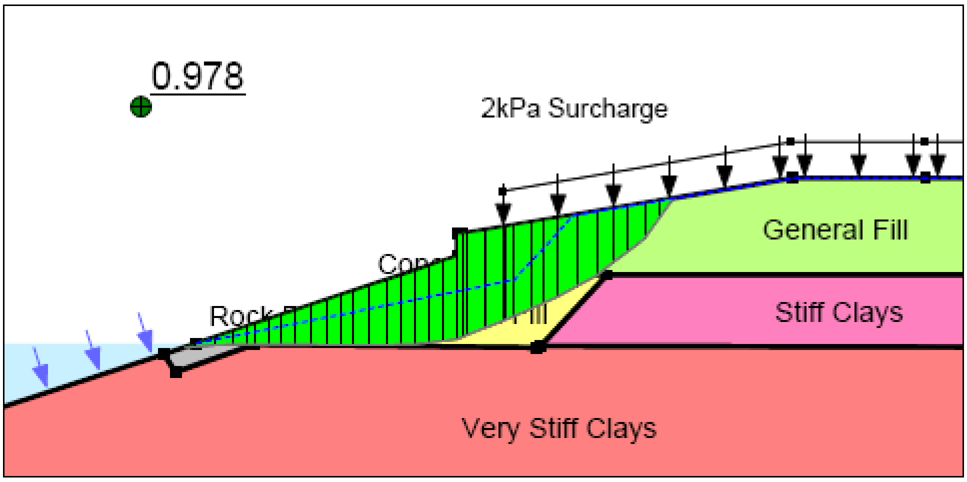

Figure 3.2
CRITICAL SLOPE FAILURE – FOS ≈ 1.0

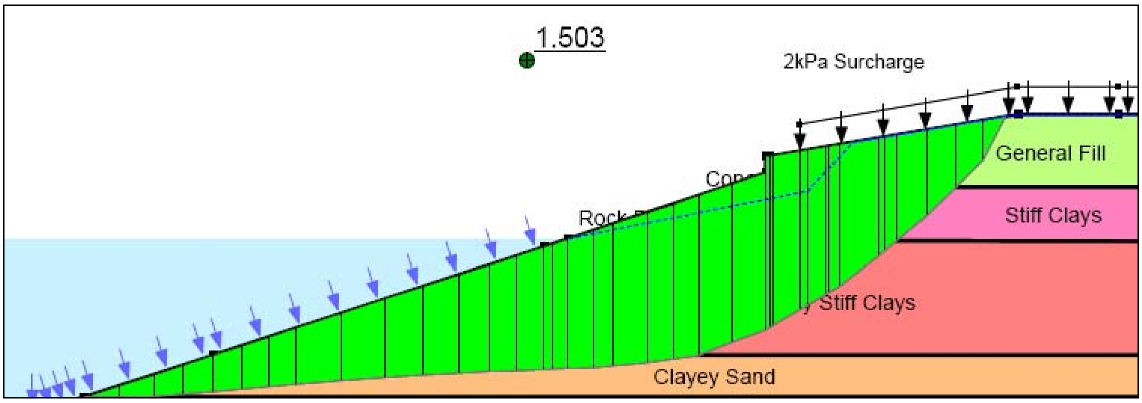

Figure 3.3
CHECK OF DEEP SLIP FAILURE – FOS >1.5

The critical slip surface is shown in Figure 3.2. The authors believe that the model result is consistent with the surface observation of slope failures in Raby Bay; a shallow slip surface through the weak wedge of uncompacted fill behind the rock protection. Borehole data indicates lower shear strength in this region. A hypothetical deep slip failure mode doesn't appear to be critical, nor has it been observed in the field by the writers.

A plausible low cost method of stabilizing a shallow slip is the installation of grout-injected piles into the soil behind the rock protection. Figure 3.4, 3.5 and 3.6 show the Slope/W model with the addition of 0.6 m diameter piles at varying spacings, to a depth of 3 m to test the impact to slope stability.

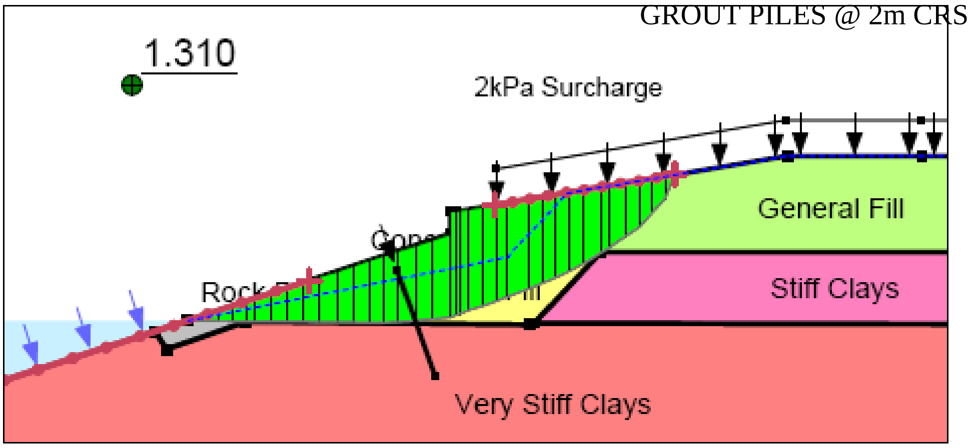

Figure 3.4
GROUTED PILES INCLUDED (A) – FOS < 1.5

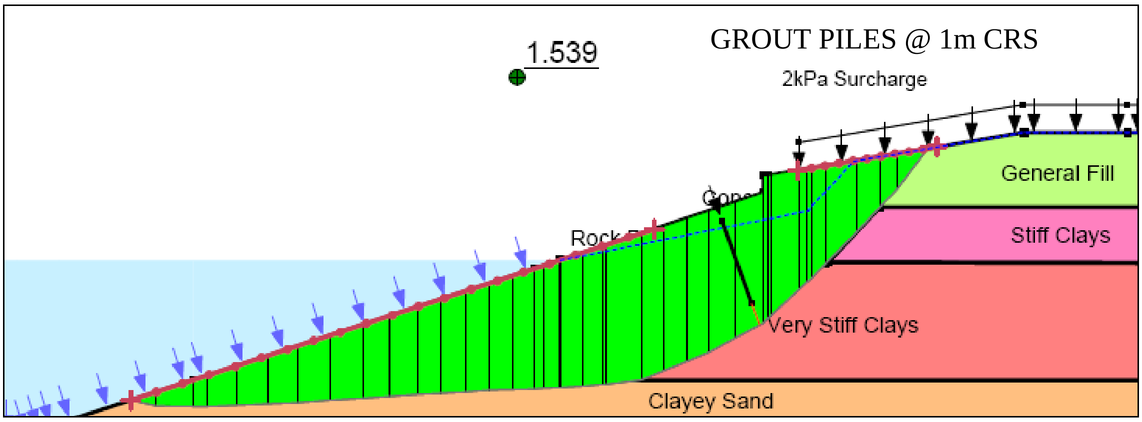

Figure 3.5
GROUTED PILES INCLUDED (B) – FOS > 1.5

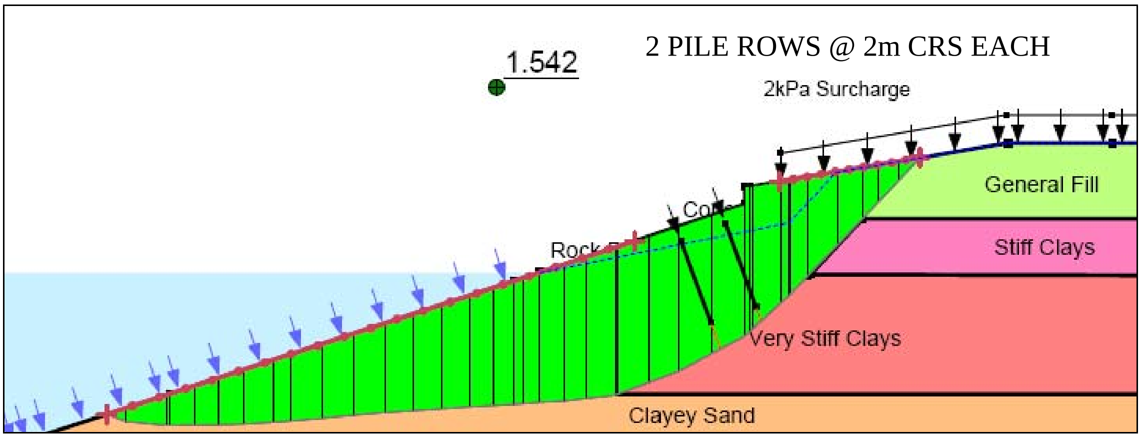

Figure 3.6
GROUTED PILES INCLUDED (VARIED) – FOS > 1.5

3.4 RESULTS

A summary of the Slope/W analysis results are shown in Table 3.1.

Table 3.3 FOS results from Slope/W analysis

Case	FOS	Figure	Type of failure
Critical slip failure for approximated current slope	0.978	3.2	Shallow slip failure through fill material
Check for deep slip failure	1.503	3.3	Deep slip through toe of slope
3 m grout piles at 2 m spacing (0 mAHD)	1.310	3.4	Shallow slip failure through piles in fill material
3 m grout piles at 1 m spacing (0 mAHD)	1.539	3.5	Deep slip below pile depth into sandy clay layer
2x 3m grout piles each at 2 m spacing (0.5 m above and below 0 mAHD)	1.542	3.6	Deep slip below pile depth into sandy clay layer

It appears that grout injection could be an effective method of stabilizing the canal slopes based on the assumed soil profile and properties. Grout piling is further discussed in Section 4.2.1.

The slip surface diagram for each Slope/W model is located in Appendix A.

4 Conclusions and recommendations

4.1 CONCLUSION

From interpretation of the geotechnical data prepared by Soil Surveys, we believe that a shallow slip failure is occurring on the canal slopes. This slip appears to be confined to a wedge of uncompacted fill under the rock armour. This wedge has maximum thickness of approximately 3 m.

Inclinometer readings indicate minor movement below this wedge in a ‘very stiff clay’. The authors believe that movement and/or incomplete ‘zeroing’ of the inclinometer tube inside the borehole might be partially responsible. Distortion of the soil mass below the shear layer is also possible. A finite element soil model might indicate such distortion; a slipe circle analysis concentrates movement into a thin surface shear zone.

As the soil material behind the rock armour goes from an undrained to drained state, the cohesion declines until a critical point is reached where, in combination with factors such as tide level, rainfall and loads behind the concrete wall, the soil begins to fail as a shallow slip.

The current rectification methods used for slope stabilization have used quite long piles which therefore appear to be an overdesign. The assumption behind this overdesign is the existence of a critical deep slip circle failure mode, which we do not observe in the field nor do we find it to be a critical failure mode theoretically. Alternative methods to stabilize the slope such as shallow grout injection may provide a more economic solution.

4.2 RECOMMENDATION

With the current methods of rectification costing approximately $17,000/m, there appears to be alternative rectification methods that would be more economic e.g. grout injection. This method is estimated to be in the order of approximately $1,000/m based on very preliminary advice from one contractor.

KBR recommends that RCC call long rectification. To match RCC’s revenue stream from the special canal levy, the rectification program could be based on a 5 to 10 year construction period. Proposed alternatives to the current methods could be assessed on their suitability through additional modelling.

Once a slope stabilization method is selected, such as grout injection, we recommend that trials be performed over limited length of the canal batter, at vacant lots and parks. These trials could be performed at locations where a slip movement failure is being observed. The trail should be instrumented so that continued movement can be monitored to confirm the effectiveness of the stabilization work. Additionally, a finite element soil model could be built of the tests. The object of the tests and finite element model is to refine the design.

4.2.1 Grout injection

Grout injection could be done from a barge, with grout lines running from a pump on the street next to the properties. Grout injection should be less disruptive than pile driving. It may be possible to grout inject one frontage in one to two days.

An initial slope stability analysis model indicates:

- grout injected pile spacing of 1 m along the shoreline
- grout injected pile diameter of 600 mm
- minimum compressive strength of 10 MPa
- pile length of about 3 m or a specified depth into the stiff clay layer.

For this option, KBR recommends some test grout injection sites be built, possibly in parks owned by RCC. To gain a greater understanding of the effectiveness of the piles, the tests would be instrumented.

4.2.2 Remedial work priority

Soil profile variations means that not all areas of the canal slopes may require remedial work and that some slopes will reach a critical stage before others. It is recommended that a priority system should be set in place to allow rectification work to be performed on-sites that are in the stages of failing or beginning to fail in the short term. Probing should be done in advance of any stabilization work, so if the uncompacted fill is not found, rectification of that area can be omitted. The canal cross sections vary somewhat throughout the estate, plus the details of the soil strata are expected to vary, hence the rectification works will have to be adjusted to suit. Rather than reacting to slip failures, an attempt should be made where possible to provide stabilization work in advance of failures to avoid damage to infrastructure.

KBR recommends that laser scanning of the revetments throughout the entire estate be performed every 6 months. This scanning can be performed from a boat. Special software can then be used to compare these scans to detect movements. Past movement monitoring methods have been appropriate and economic, however the recent rapidly reduction in the cost of laser scanning means that this technology should be considered for future monitoring. It offers speed and completeness advantages.

4.2.3 Final comments

At this stage, we believe that no more geotechnical investigations are needed. We believe that the apparent issue with the canal slopes has been identified and that laser scanning and probing ahead of remedial work is the way forward.

Ideas discussed in previous reports, like maintaining a high water table using a lock, or placing more rock on the toe of the slip circle, are now not considered to be effective based on the apparent slip being quite small and shallow.

5 References

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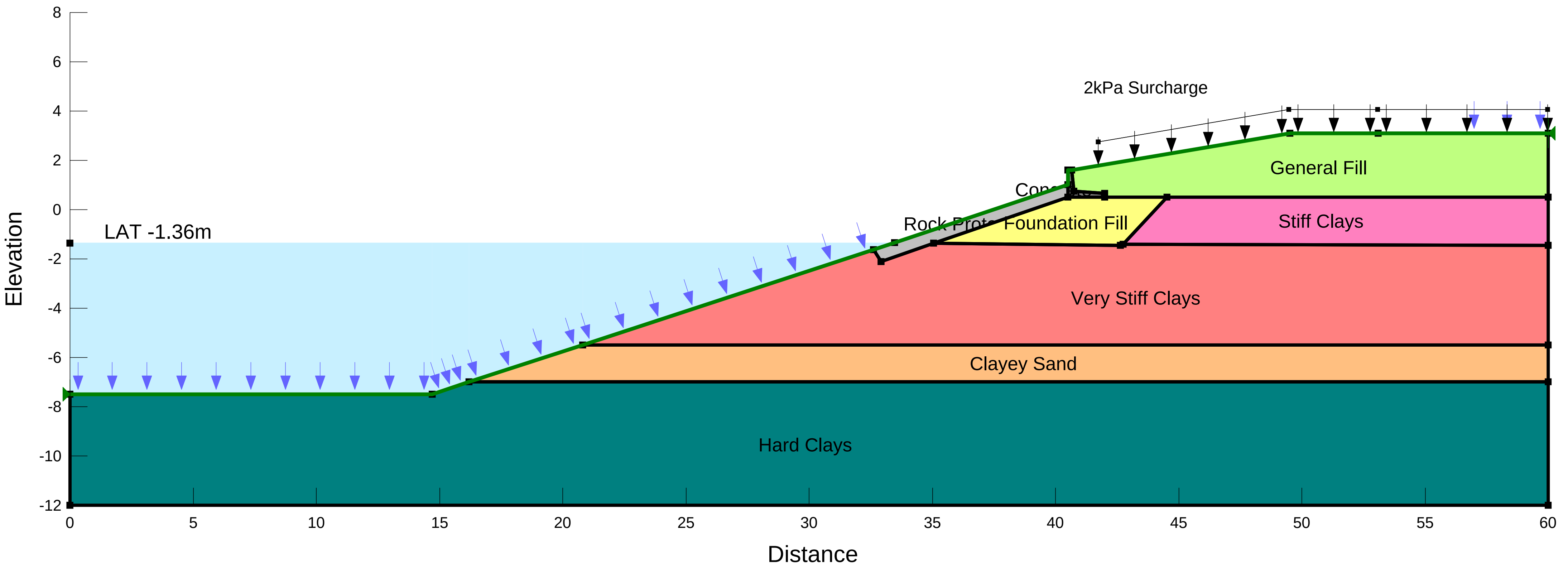
Standards Australia (2002), 'AS 4678 Earth-retaining Structures'.

Appendix A

SLOPE/W MODELS

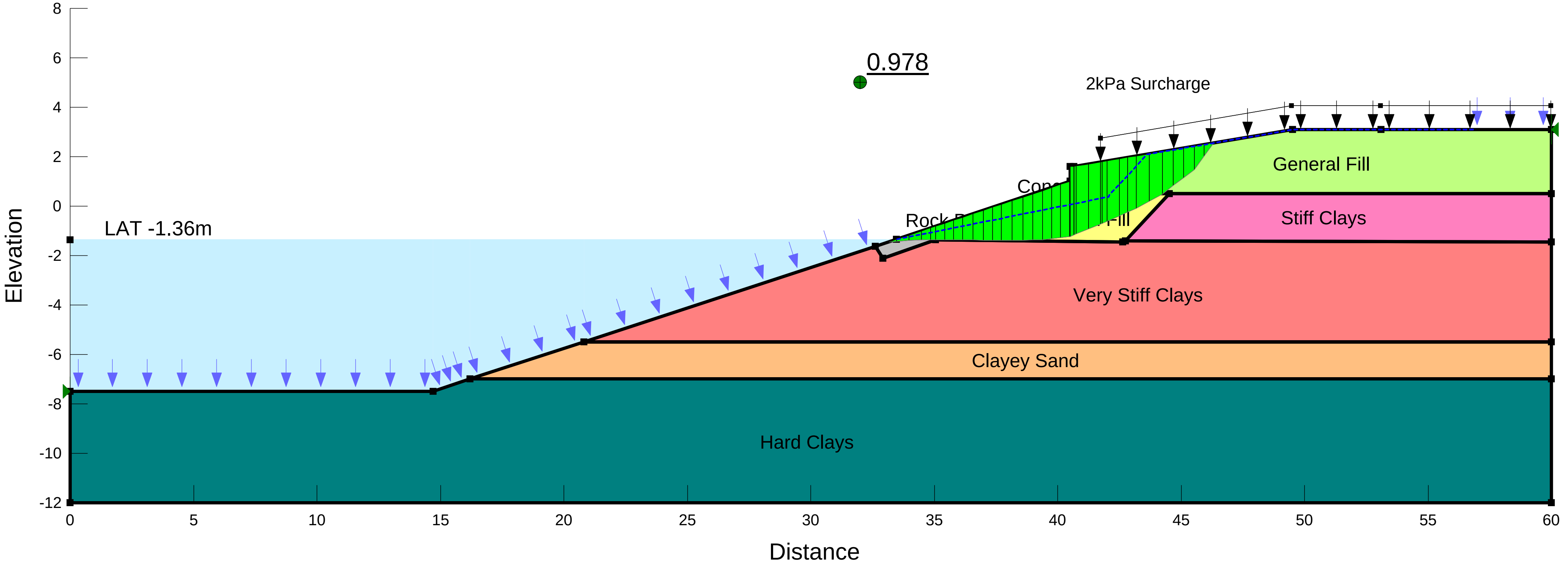
Slope/W Model Layout

Name: Concrete Wall	Model: Mohr-Coulomb	Unit Weight: 24 kN/m³	Cohesion: 4000 kPa	Phi: 0 °
Name: Rock Protection	Model: Mohr-Coulomb	Unit Weight: 20 kN/m³	Cohesion: 0 kPa	Phi: 30 °
Name: General Fill	Model: Mohr-Coulomb	Unit Weight: 18 kN/m³	Cohesion: 0 kPa	Phi: 26 °
Name: Foundation Fill	Model: Mohr-Coulomb	Unit Weight: 17.5 kN/m³	Cohesion: 2 kPa	Phi: 18 °
Name: Stiff Clays	Model: Mohr-Coulomb	Unit Weight: 17.5 kN/m³	Cohesion: 5 kPa	Phi: 26 °
Name: Very Stiff Clays	Model: Mohr-Coulomb	Unit Weight: 17.5 kN/m³	Cohesion: 10 kPa	Phi: 27 °
Name: Clayey Sand	Model: Mohr-Coulomb	Unit Weight: 18 kN/m³	Cohesion: 2 kPa	Phi: 30 °
Name: Hard Clays	Model: Mohr-Coulomb	Unit Weight: 18 kN/m³	Cohesion: 15 kPa	Phi: 28 °



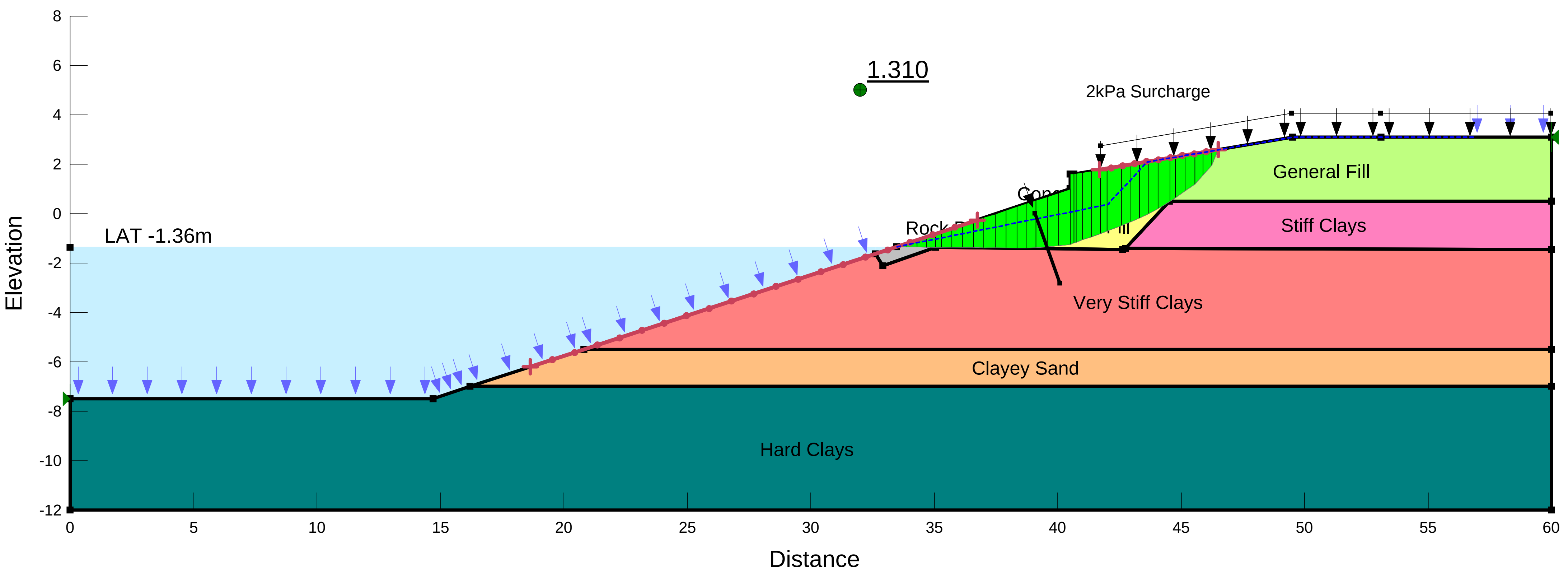
Model: Critical Slip Failure

Name: Concrete Wall	Model: Mohr-Coulomb	Unit Weight: 24 kN/m³	Cohesion: 4000 kPa	Phi: 0 °
Name: Rock Protection	Model: Mohr-Coulomb	Unit Weight: 20 kN/m³	Cohesion: 0 kPa	Phi: 30 °
Name: General Fill	Model: Mohr-Coulomb	Unit Weight: 18 kN/m³	Cohesion: 0 kPa	Phi: 26 °
Name: Foundation Fill	Model: Mohr-Coulomb	Unit Weight: 17.5 kN/m³	Cohesion: 2 kPa	Phi: 18 °
Name: Stiff Clays	Model: Mohr-Coulomb	Unit Weight: 17.5 kN/m³	Cohesion: 5 kPa	Phi: 26 °
Name: Very Stiff Clays	Model: Mohr-Coulomb	Unit Weight: 17.5 kN/m³	Cohesion: 10 kPa	Phi: 27 °
Name: Clayey Sand	Model: Mohr-Coulomb	Unit Weight: 18 kN/m³	Cohesion: 2 kPa	Phi: 30 °
Name: Hard Clays	Model: Mohr-Coulomb	Unit Weight: 18 kN/m³	Cohesion: 15 kPa	Phi: 28 °



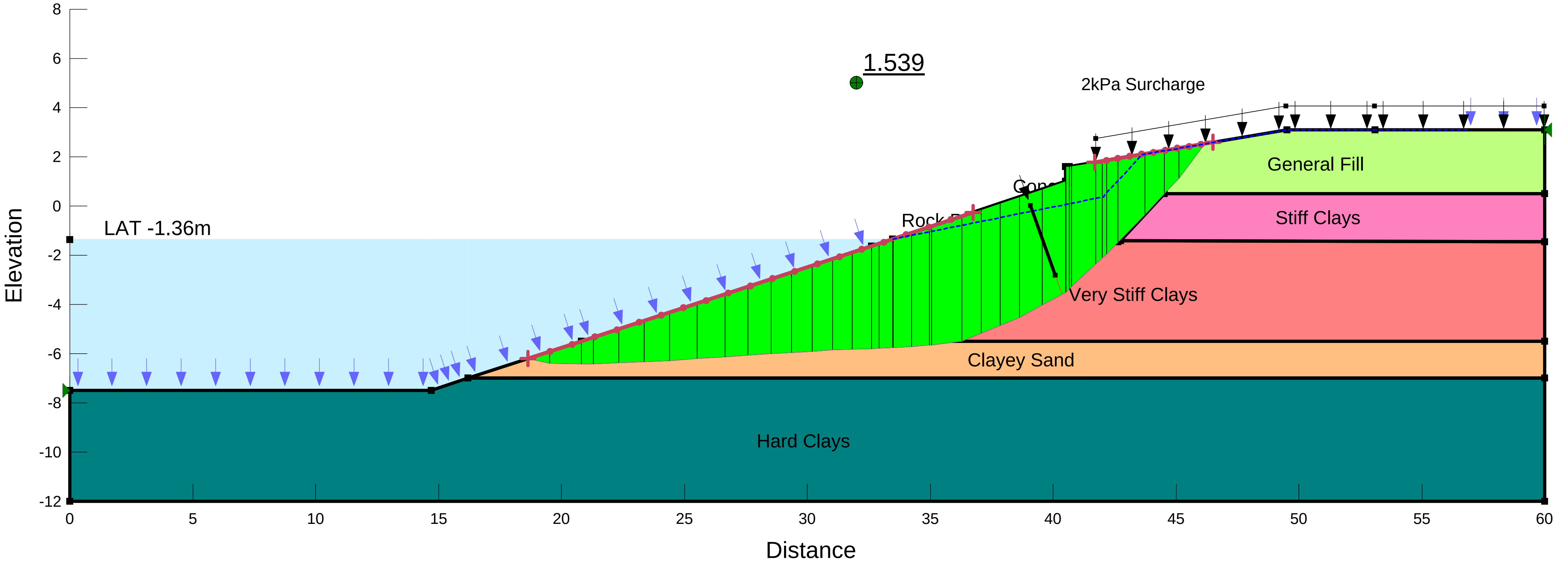
Model: 3m Grout Piles at 2m crs

Name: Concrete Wall	Model: Mohr-Coulomb	Unit Weight: 24 kN/m³	Cohesion: 4000 kPa	Phi: 0 °
Name: Rock Protection	Model: Mohr-Coulomb	Unit Weight: 20 kN/m³	Cohesion: 0 kPa	Phi: 30 °
Name: General Fill	Model: Mohr-Coulomb	Unit Weight: 18 kN/m³	Cohesion: 0 kPa	Phi: 26 °
Name: Foundation Fill	Model: Mohr-Coulomb	Unit Weight: 17.5 kN/m³	Cohesion: 2 kPa	Phi: 18 °
Name: Stiff Clays	Model: Mohr-Coulomb	Unit Weight: 17.5 kN/m³	Cohesion: 5 kPa	Phi: 26 °
Name: Very Stiff Clays	Model: Mohr-Coulomb	Unit Weight: 17.5 kN/m³	Cohesion: 10 kPa	Phi: 27 °
Name: Clayey Sand	Model: Mohr-Coulomb	Unit Weight: 18 kN/m³	Cohesion: 2 kPa	Phi: 30 °
Name: Hard Clays	Model: Mohr-Coulomb	Unit Weight: 18 kN/m³	Cohesion: 15 kPa	Phi: 28 °



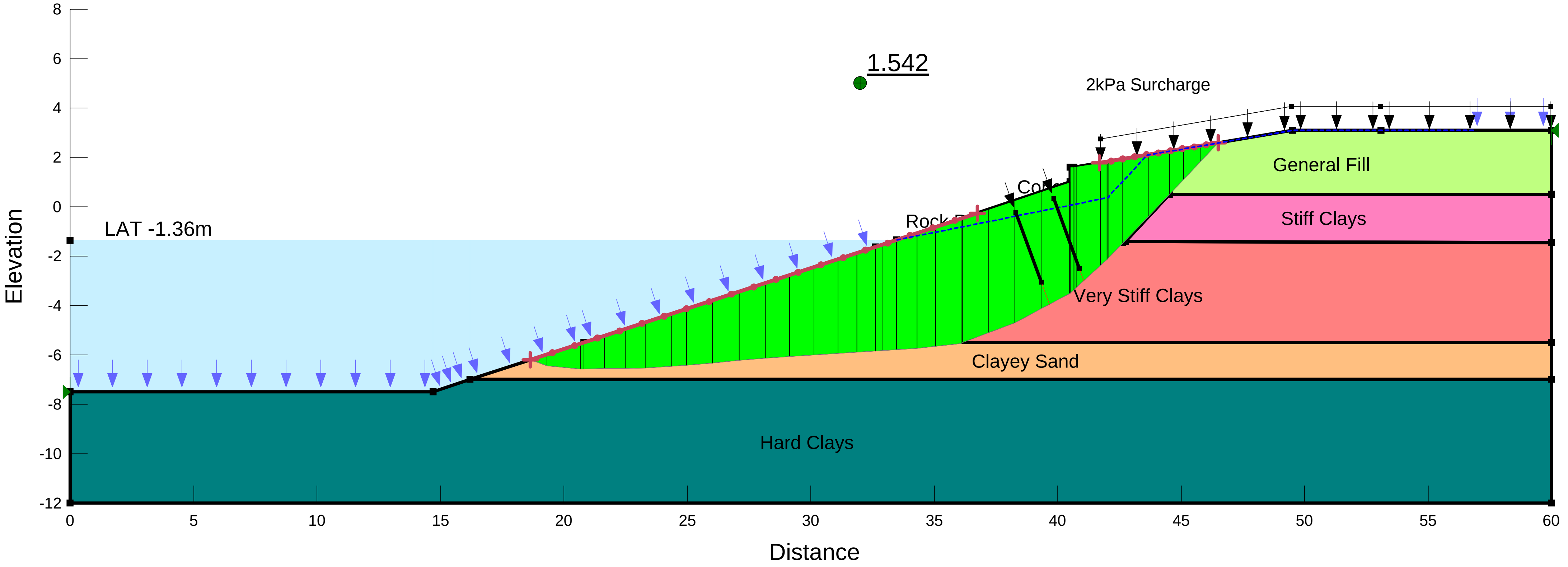
Model: 3m Grout Piles at 1m crs

Name: Concrete Wall Model: Mohr-Coulomb Unit Weight: 24 kN/m³ Cohesion: 4000 kPa Phi: 0 °
Name: Rock Protection Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: General Fill Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 26 °
Name: Foundation Fill Model: Mohr-Coulomb Unit Weight: 17.5 kN/m³ Cohesion: 2 kPa Phi: 18 °
Name: Stiff Clays Model: Mohr-Coulomb Unit Weight: 17.5 kN/m³ Cohesion: 5 kPa Phi: 26 °
Name: Very Stiff Clays Model: Mohr-Coulomb Unit Weight: 17.5 kN/m³ Cohesion: 10 kPa Phi: 27 °
Name: Clayey Sand Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 30 °
Name: Hard Clays Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 15 kPa Phi: 28 °



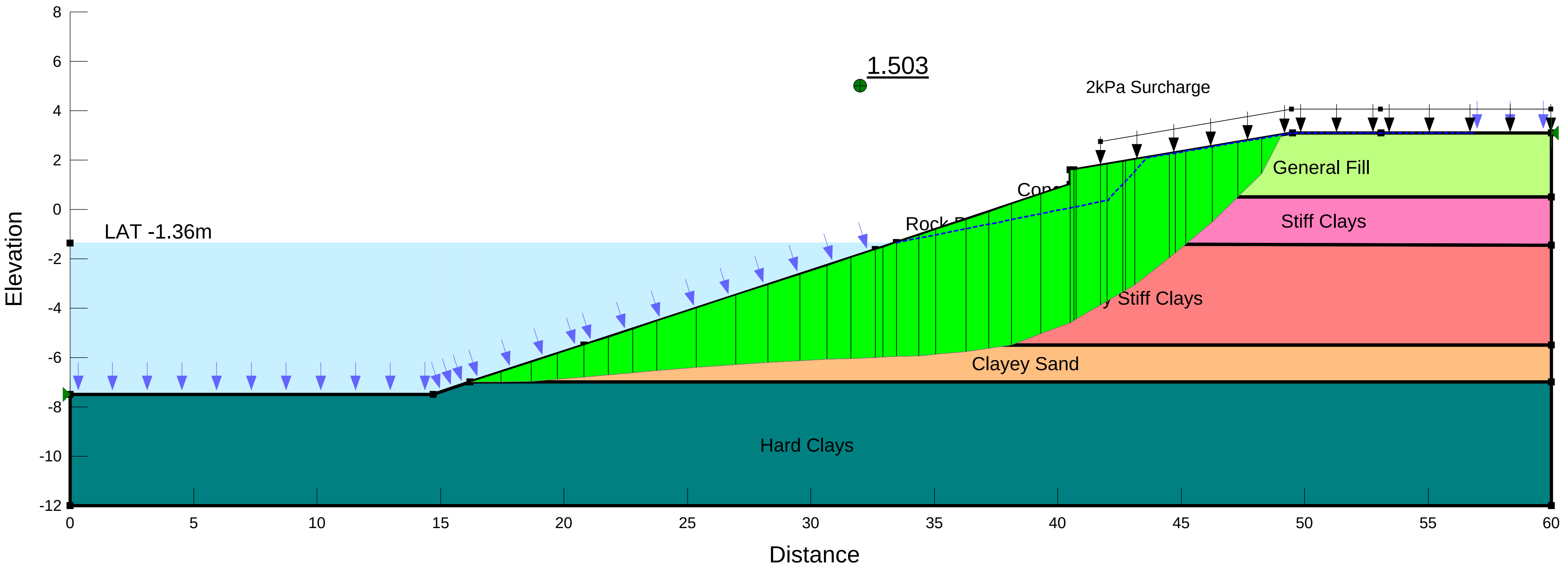
Model: 2x 3m Grout Piles at 2m crs each

Name: Concrete Wall	Model: Mohr-Coulomb	Unit Weight: 24 kN/m³	Cohesion: 4000 kPa	Phi: 0 °
Name: Rock Protection	Model: Mohr-Coulomb	Unit Weight: 20 kN/m³	Cohesion: 0 kPa	Phi: 30 °
Name: General Fill	Model: Mohr-Coulomb	Unit Weight: 18 kN/m³	Cohesion: 0 kPa	Phi: 26 °
Name: Foundation Fill	Model: Mohr-Coulomb	Unit Weight: 17.5 kN/m³	Cohesion: 2 kPa	Phi: 18 °
Name: Stiff Clays	Model: Mohr-Coulomb	Unit Weight: 17.5 kN/m³	Cohesion: 5 kPa	Phi: 26 °
Name: Very Stiff Clays	Model: Mohr-Coulomb	Unit Weight: 17.5 kN/m³	Cohesion: 10 kPa	Phi: 27 °
Name: Clayey Sand	Model: Mohr-Coulomb	Unit Weight: 18 kN/m³	Cohesion: 2 kPa	Phi: 30 °
Name: Hard Clays	Model: Mohr-Coulomb	Unit Weight: 18 kN/m³	Cohesion: 15 kPa	Phi: 28 °



Model: Deep Failure

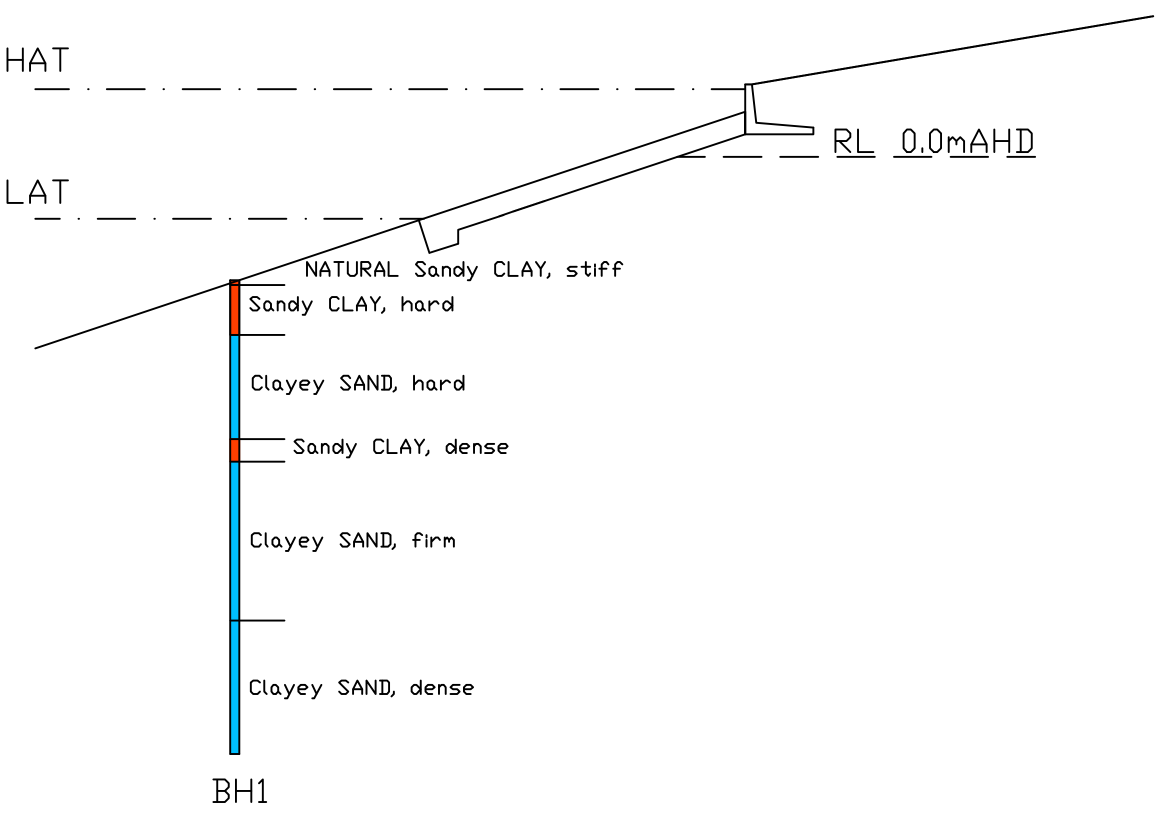
Name: Concrete Wall Model: Mohr-Coulomb Unit Weight: 24 kN/m³ Cohesion: 4000 kPa Phi: 0 °
Name: Rock Protection Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: General Fill Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 26 °
Name: Foundation Fill Model: Mohr-Coulomb Unit Weight: 17.5 kN/m³ Cohesion: 2 kPa Phi: 18 °
Name: Stiff Clays Model: Mohr-Coulomb Unit Weight: 17.5 kN/m³ Cohesion: 5 kPa Phi: 26 °
Name: Very Stiff Clays Model: Mohr-Coulomb Unit Weight: 17.5 kN/m³ Cohesion: 10 kPa Phi: 27 °
Name: Clayey Sand Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 30 °
Name: Hard Clays Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 15 kPa Phi: 28 °

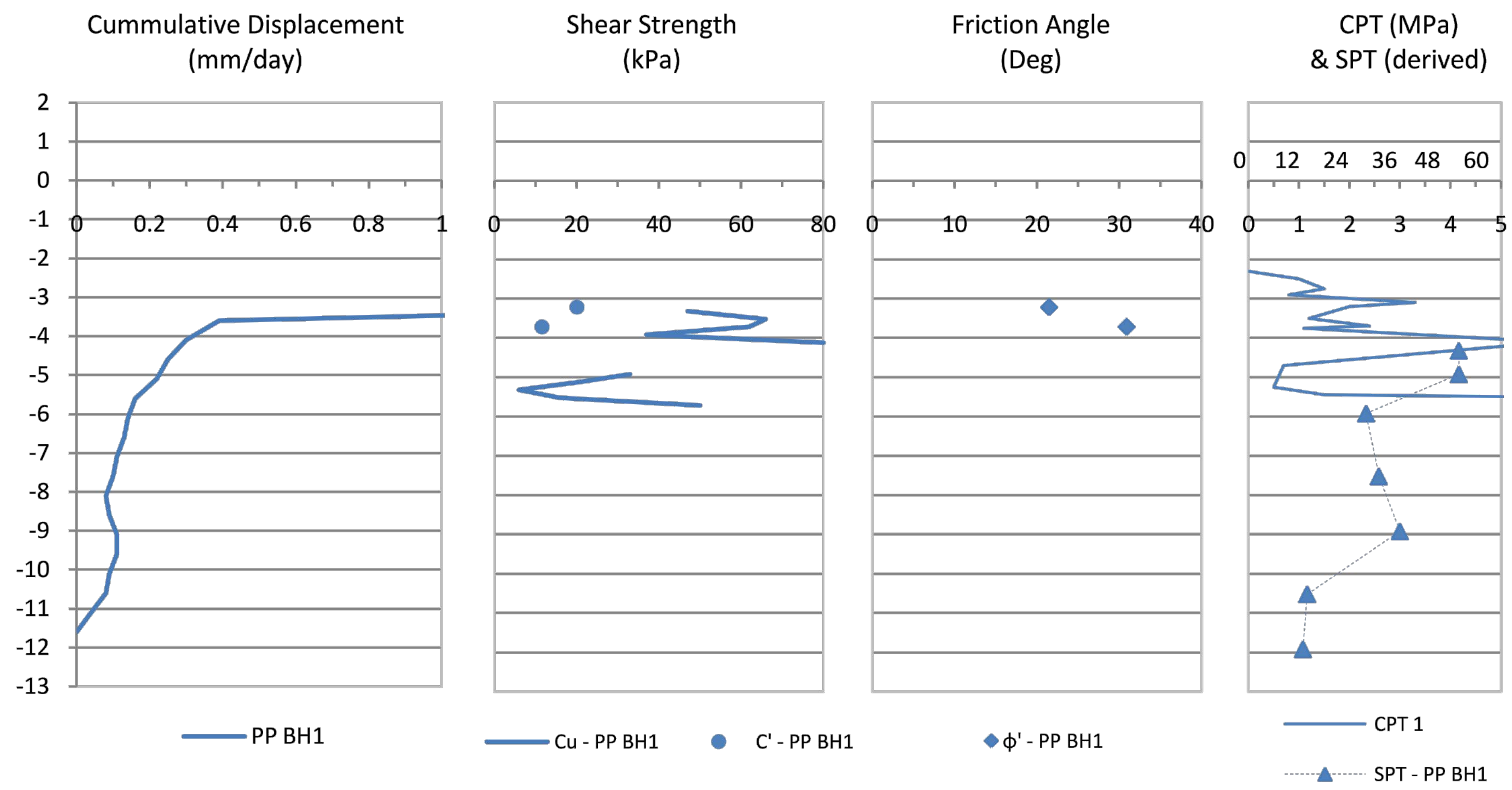


Appendix B

BOREHOLE PROFILES

Piermont Place – Section 1

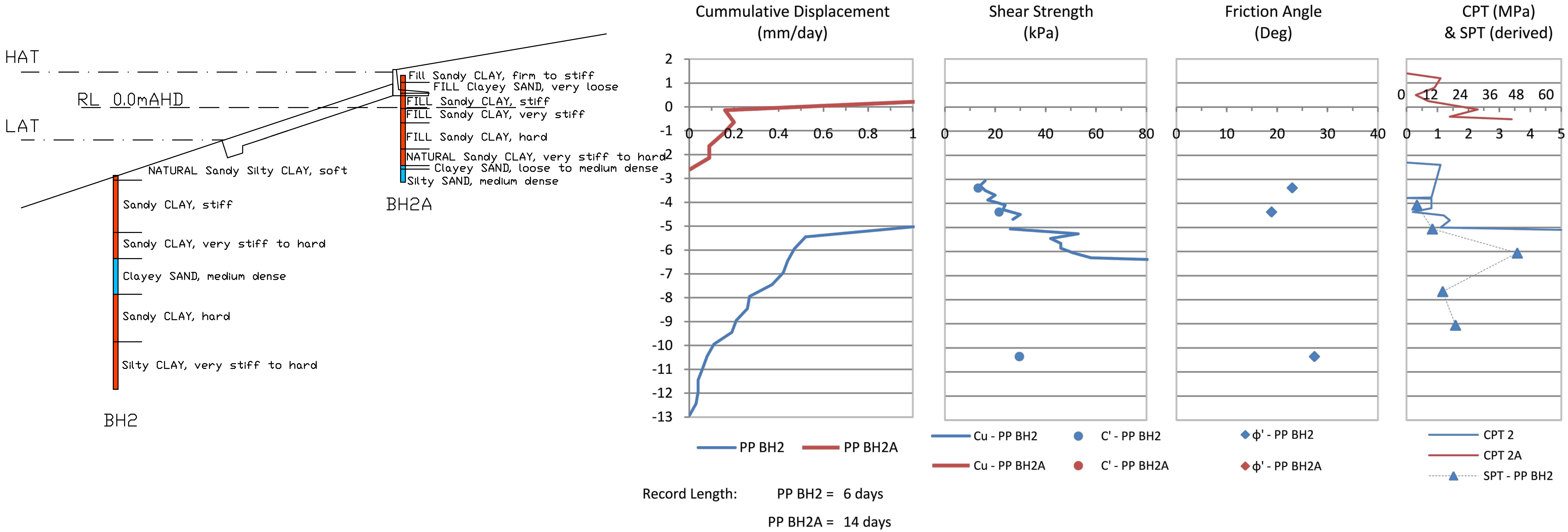




Record Length: PP BH1 = 6 days

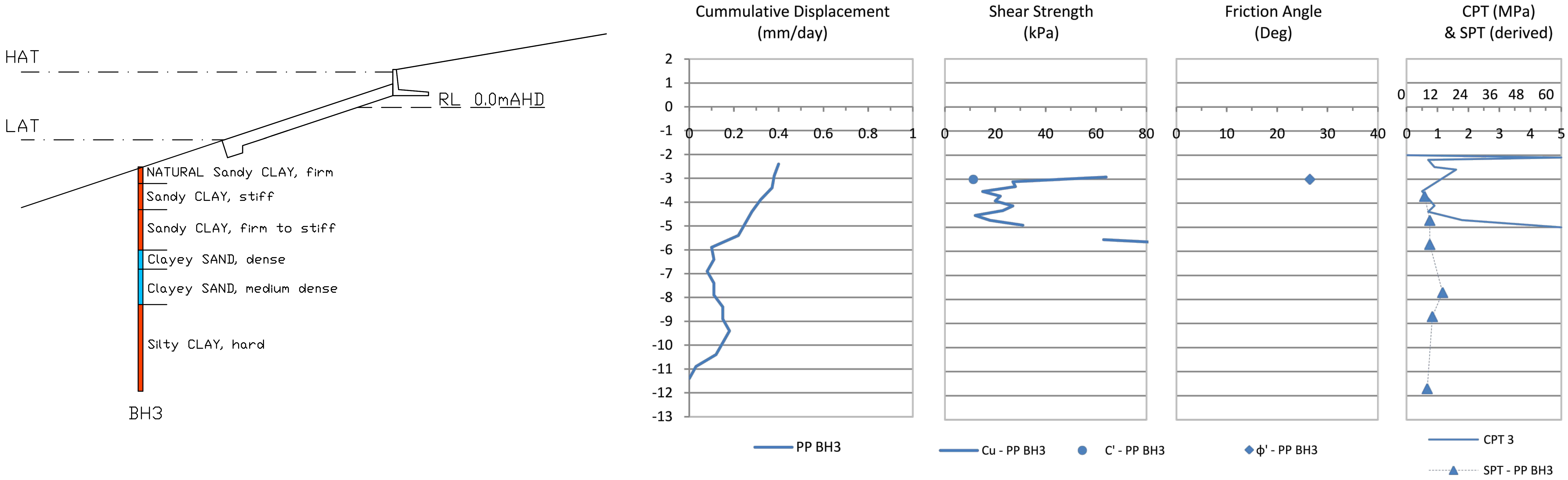
NOTE: Approximate borehole locations shown above based on recorded RL. For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

Piermont Place - Section 2



NOTE: Approximate borehole locations shown above based on recorded RL. For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

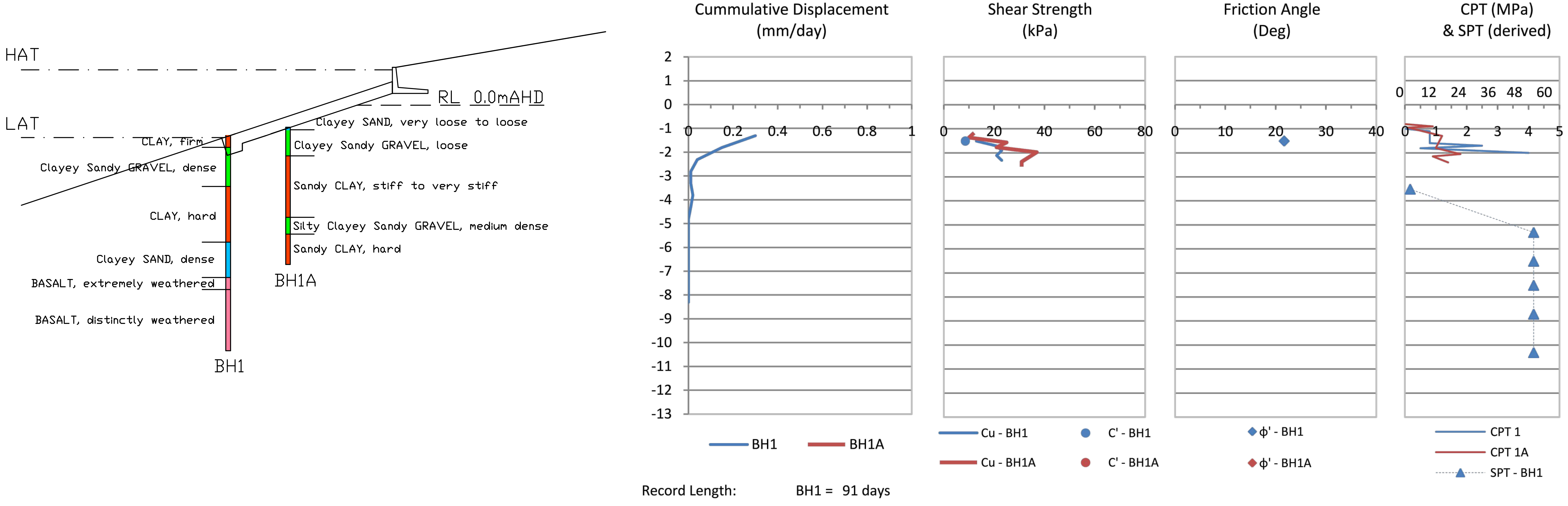
Piermont Place – Section 3



Record Length: PP BH3 = 6 days

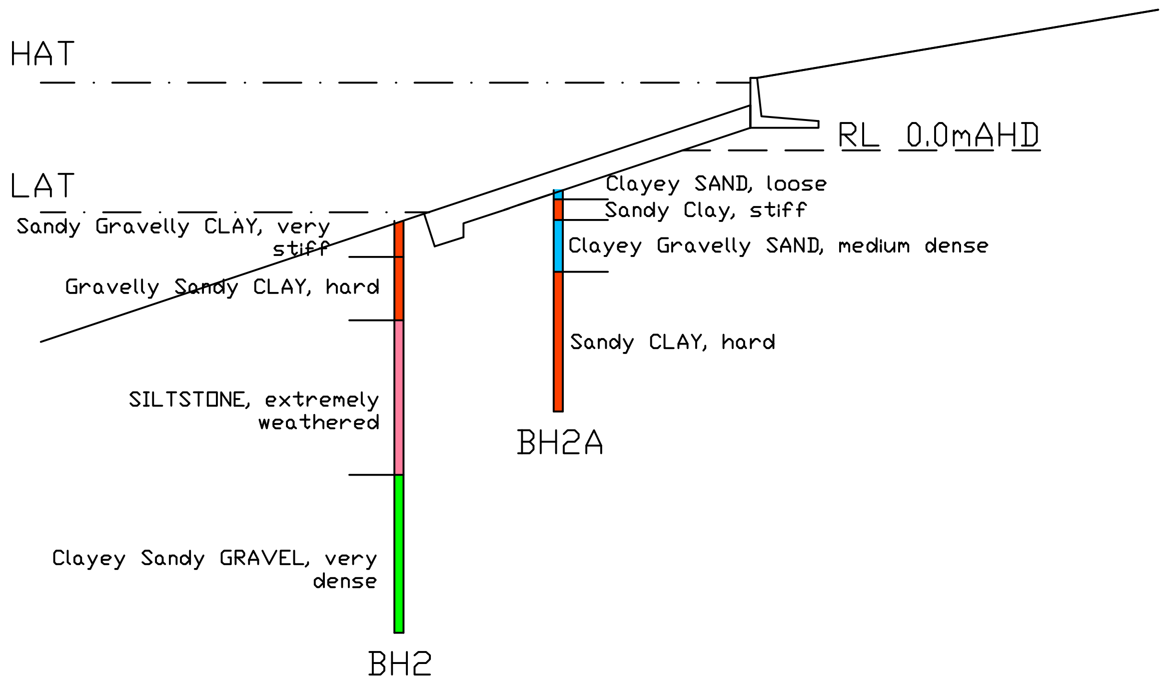
NOTE: Approximate borehole locations shown above based on recorded RL. For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

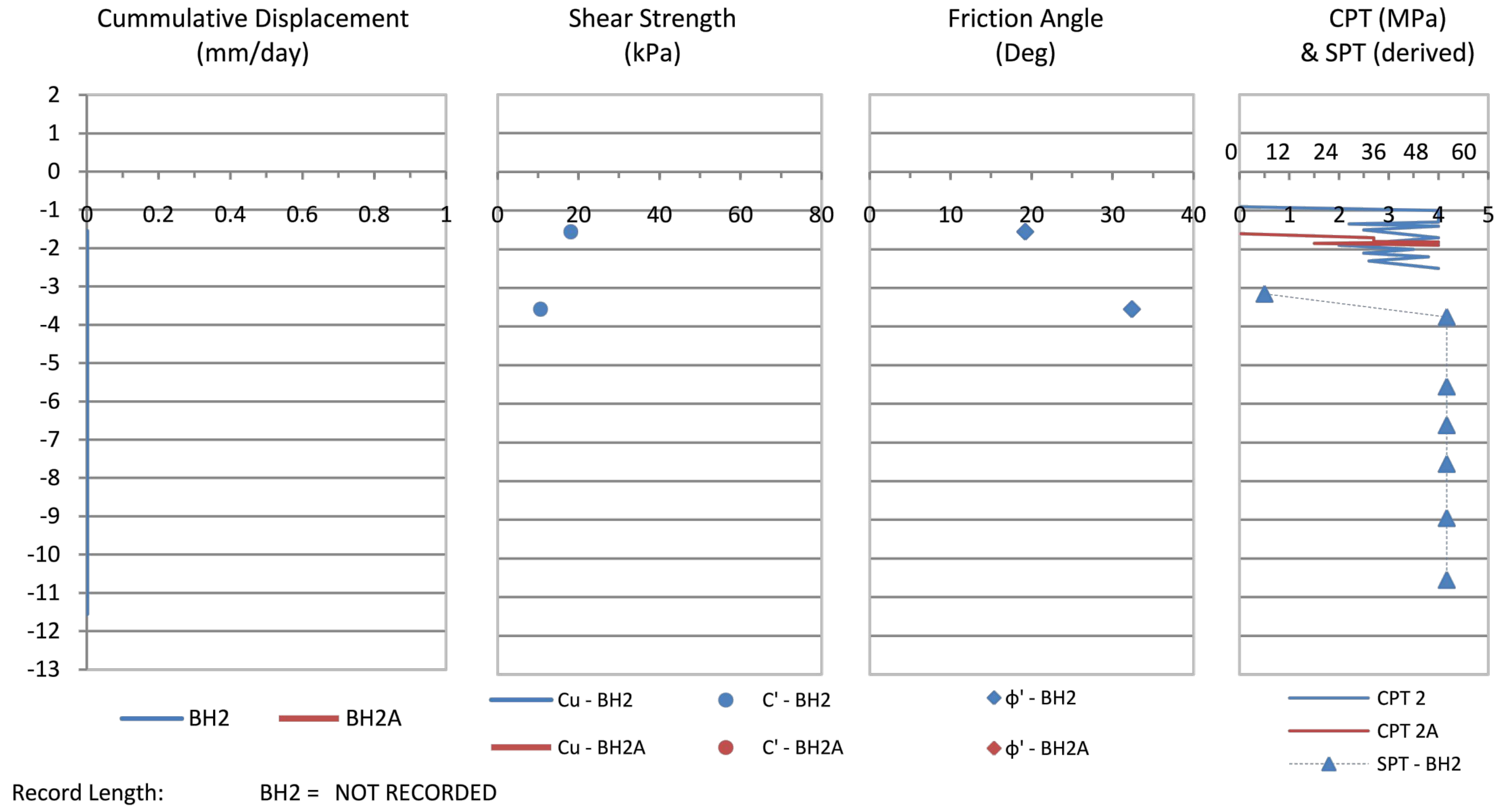
Test Location 1



NOTE: Approximate borehole locations shown above based on recorded RL.
For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

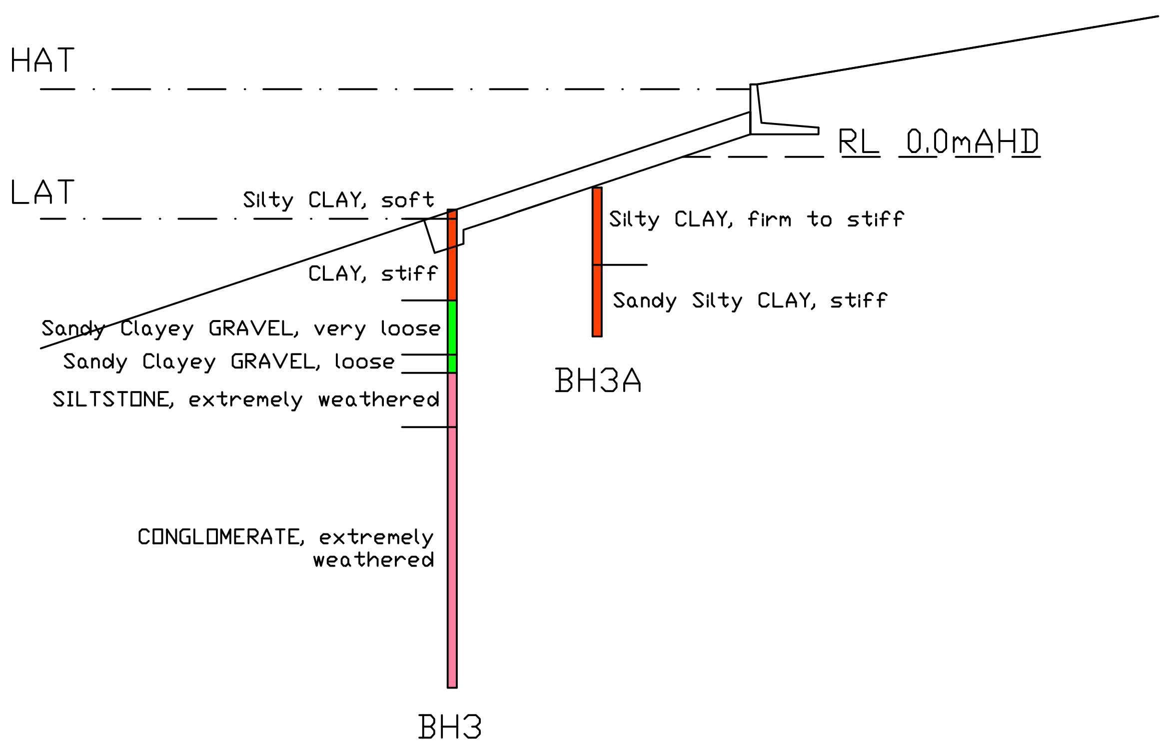
Test Location 2

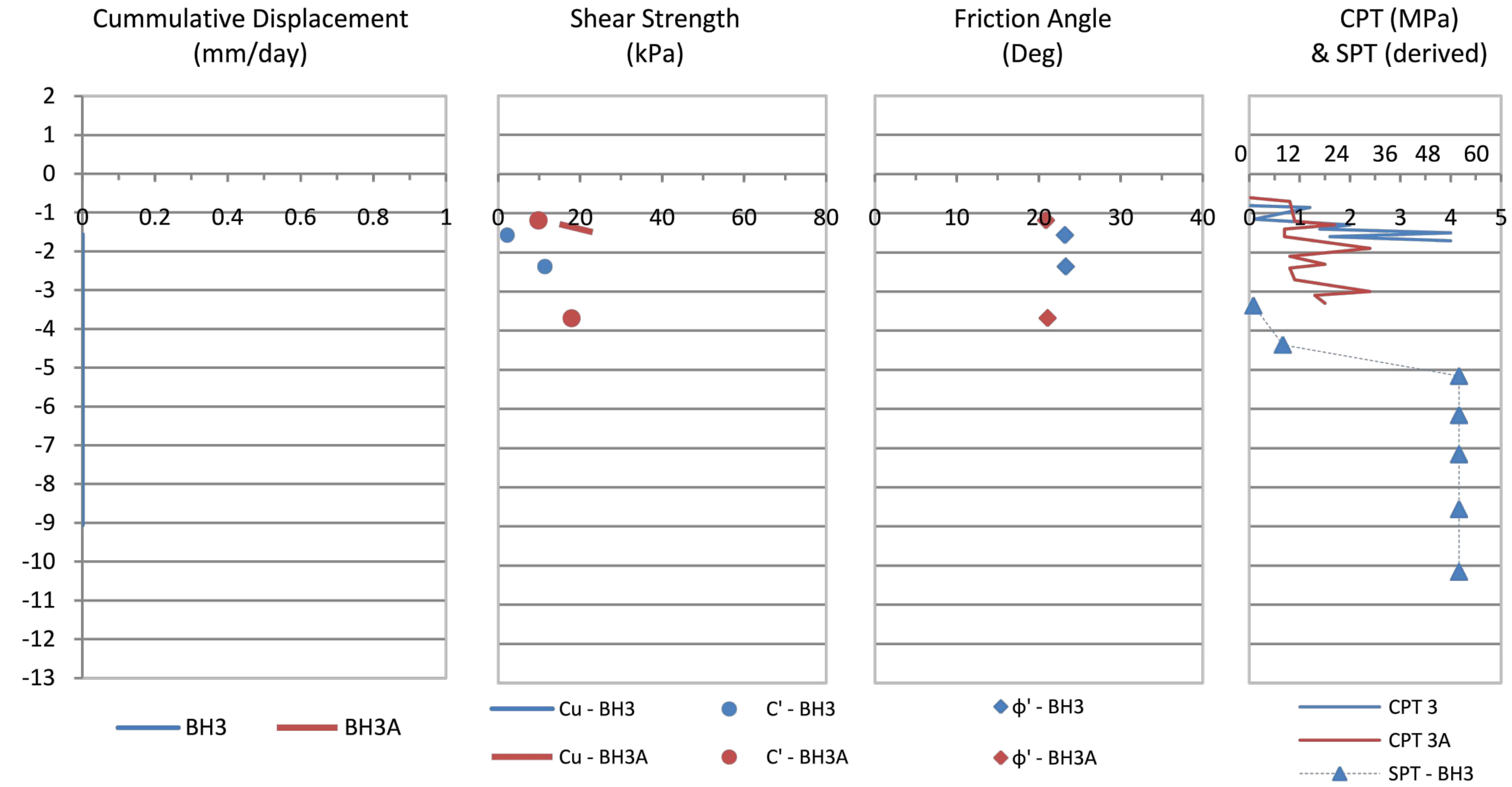




NOTE: Approximate borehole locations shown above based on recorded RL. For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

Test Location 3

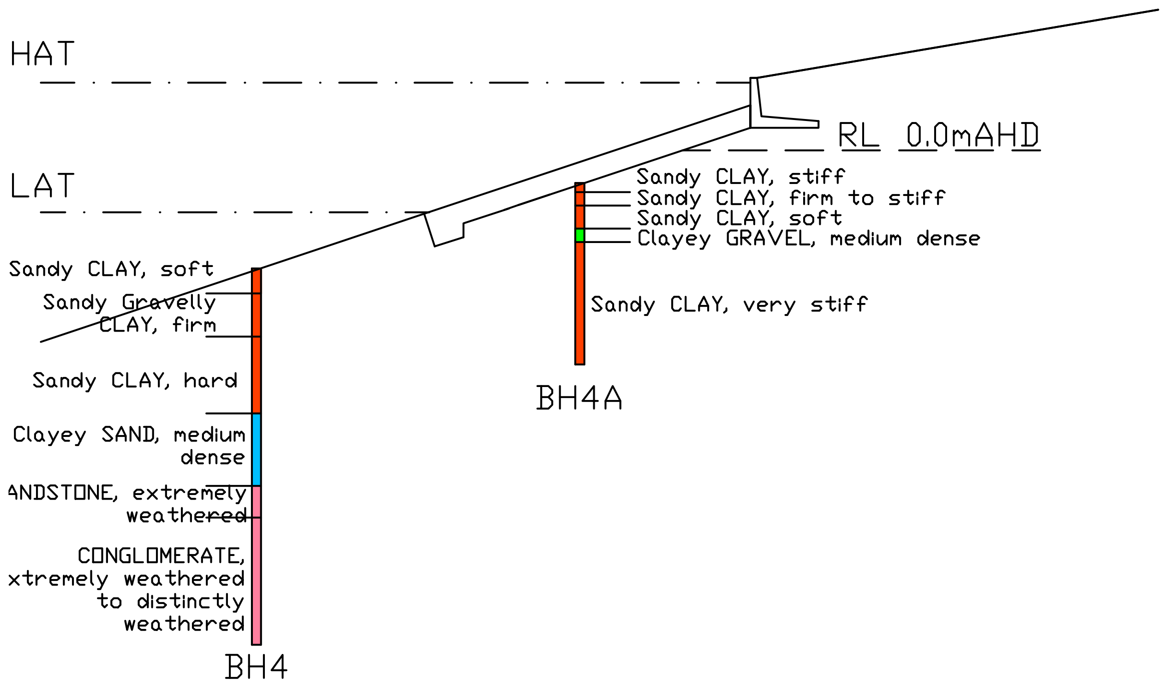


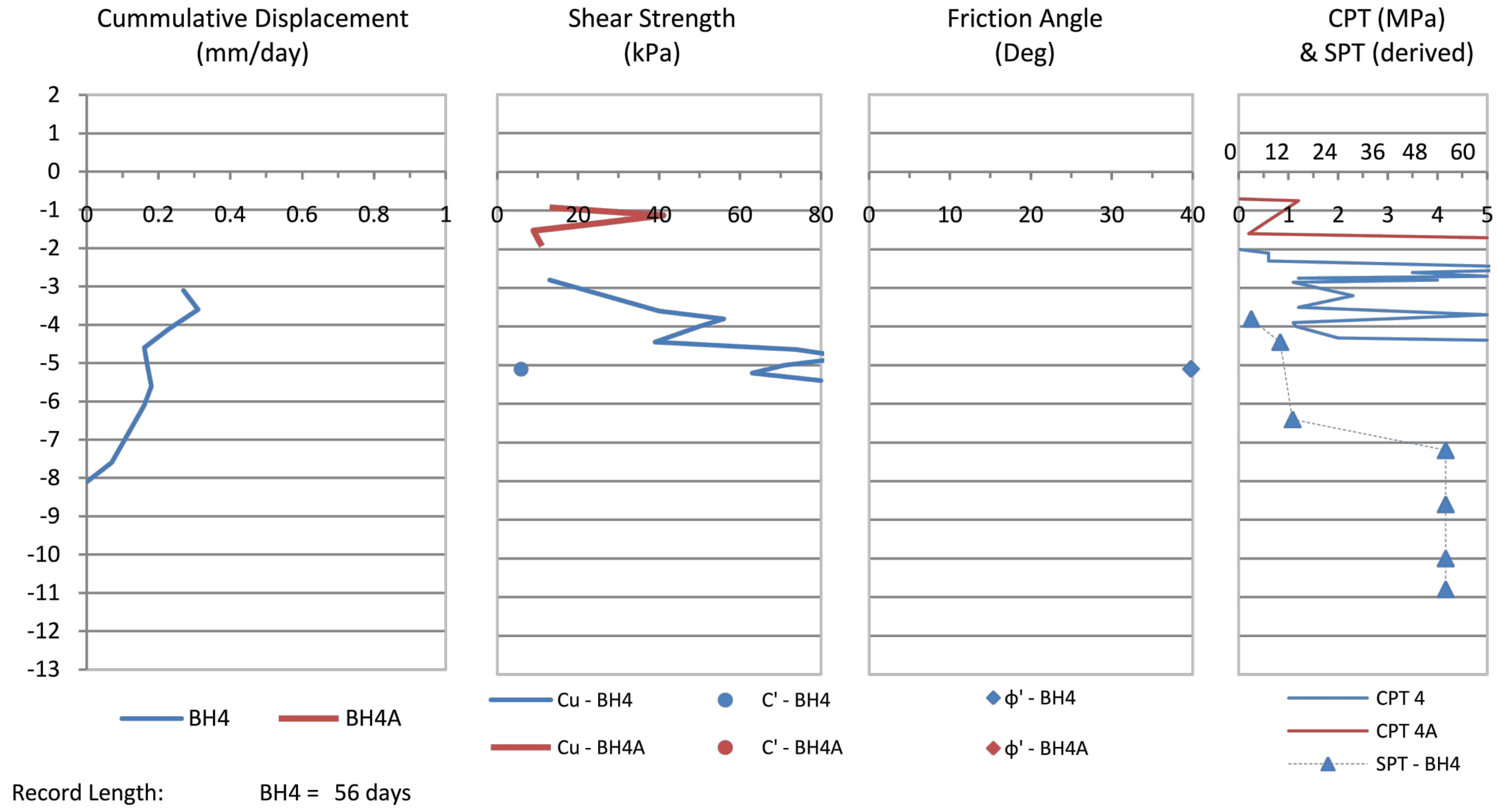


Record Length: BH3 = NOT RECORDED

NOTE: Approximate borehole locations shown above based on recorded RL. For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

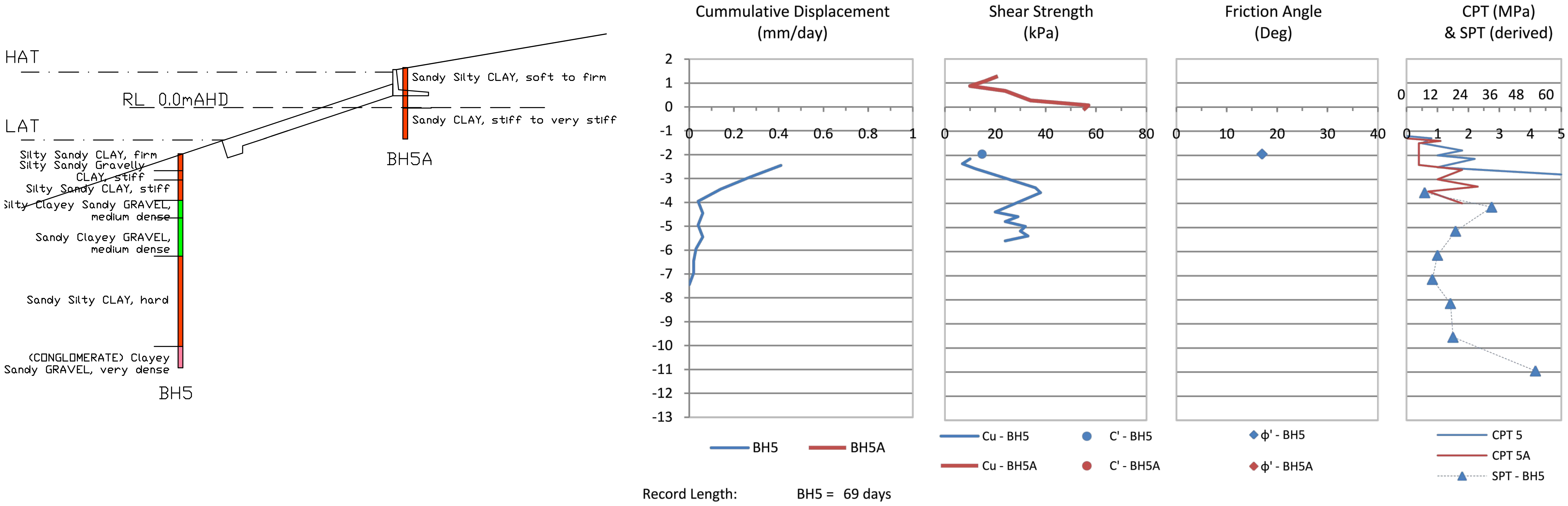
Test Location 4





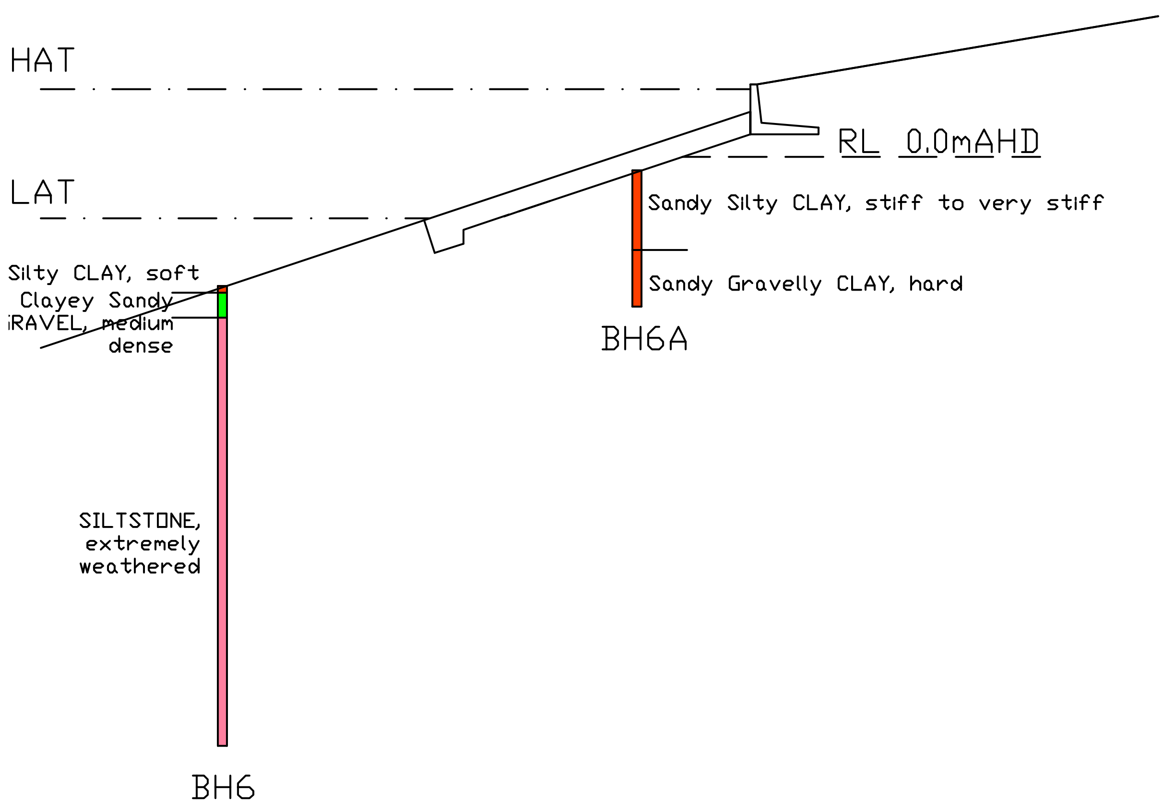
NOTE: Approximate borehole locations shown above based on recorded RL. For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

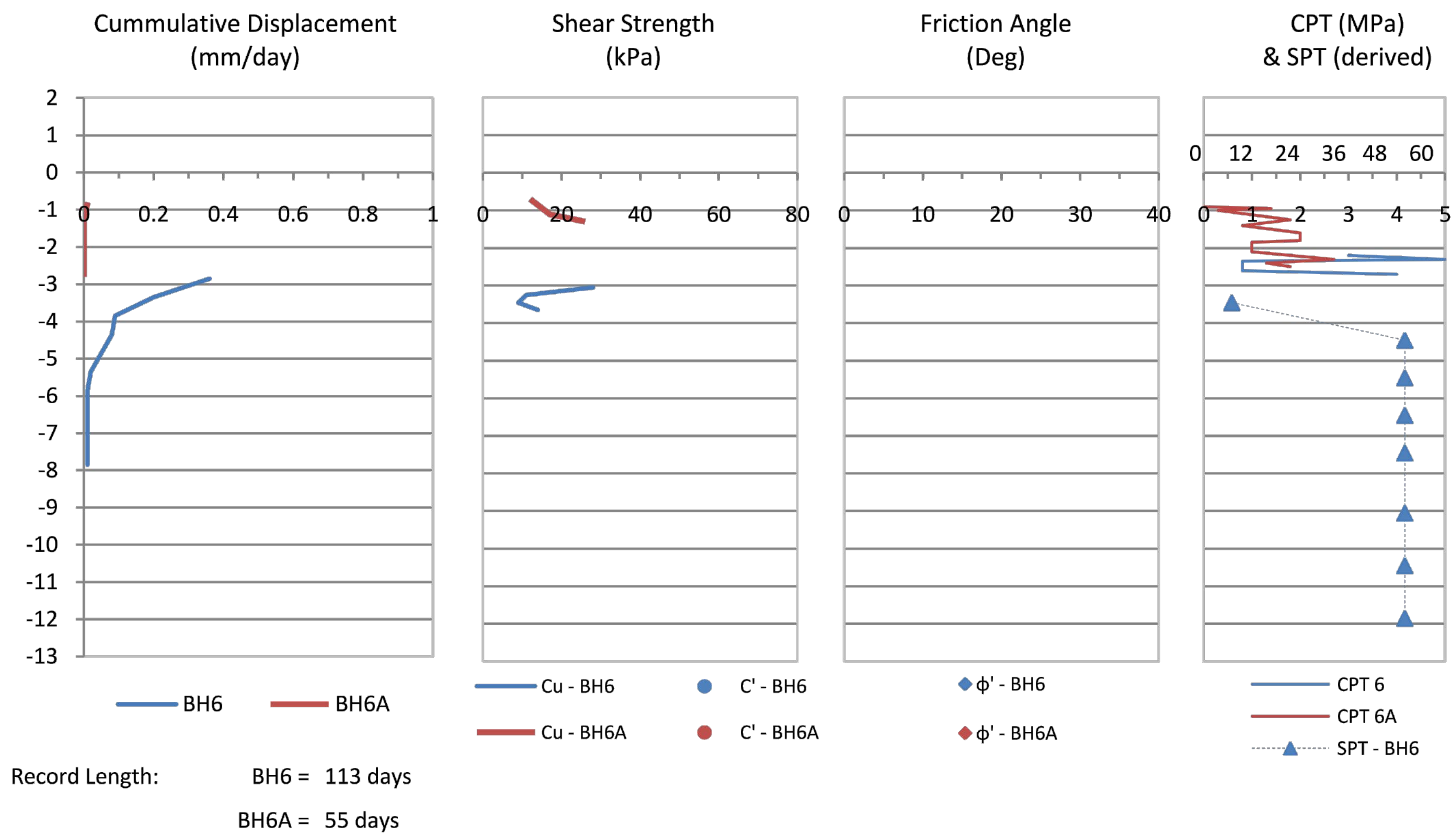
Test Location 5



NOTE: Approximate borehole locations shown above based on recorded RL. For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

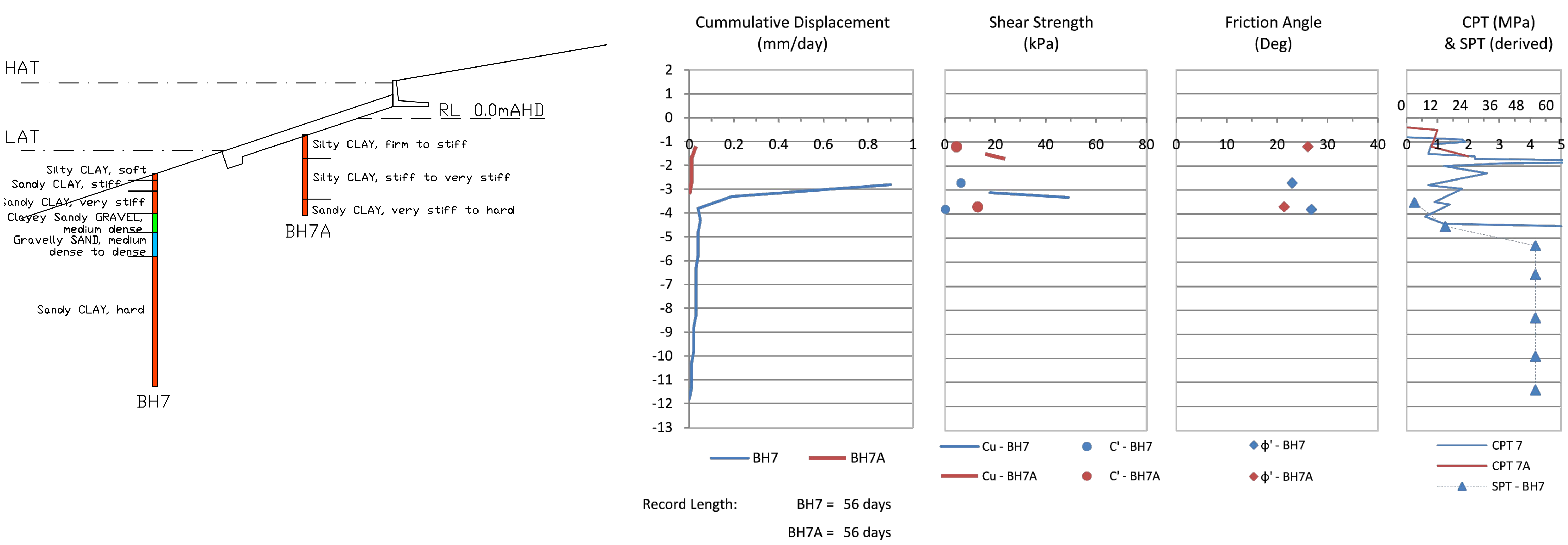
Test Location 6





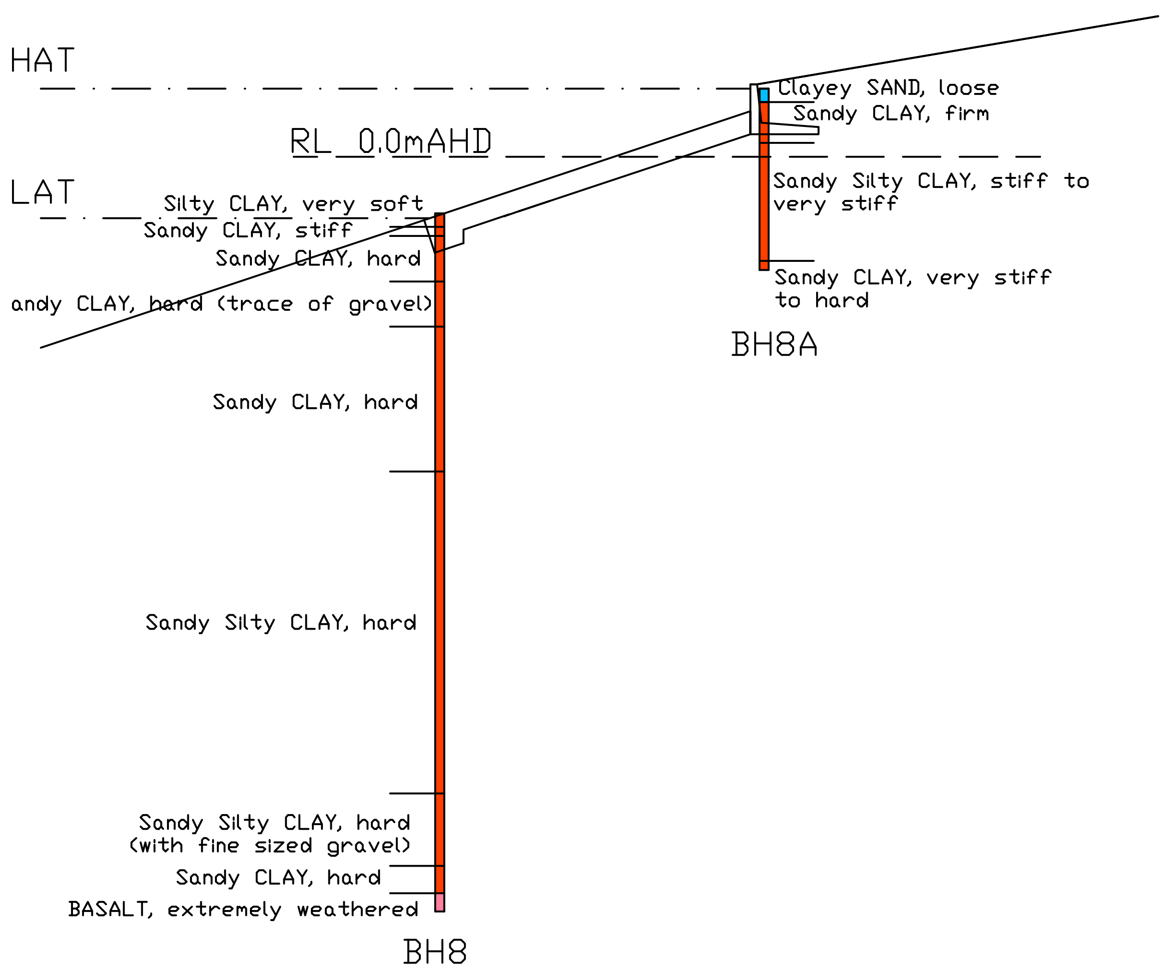
NOTE: Approximate borehole locations shown above based on recorded RL. For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

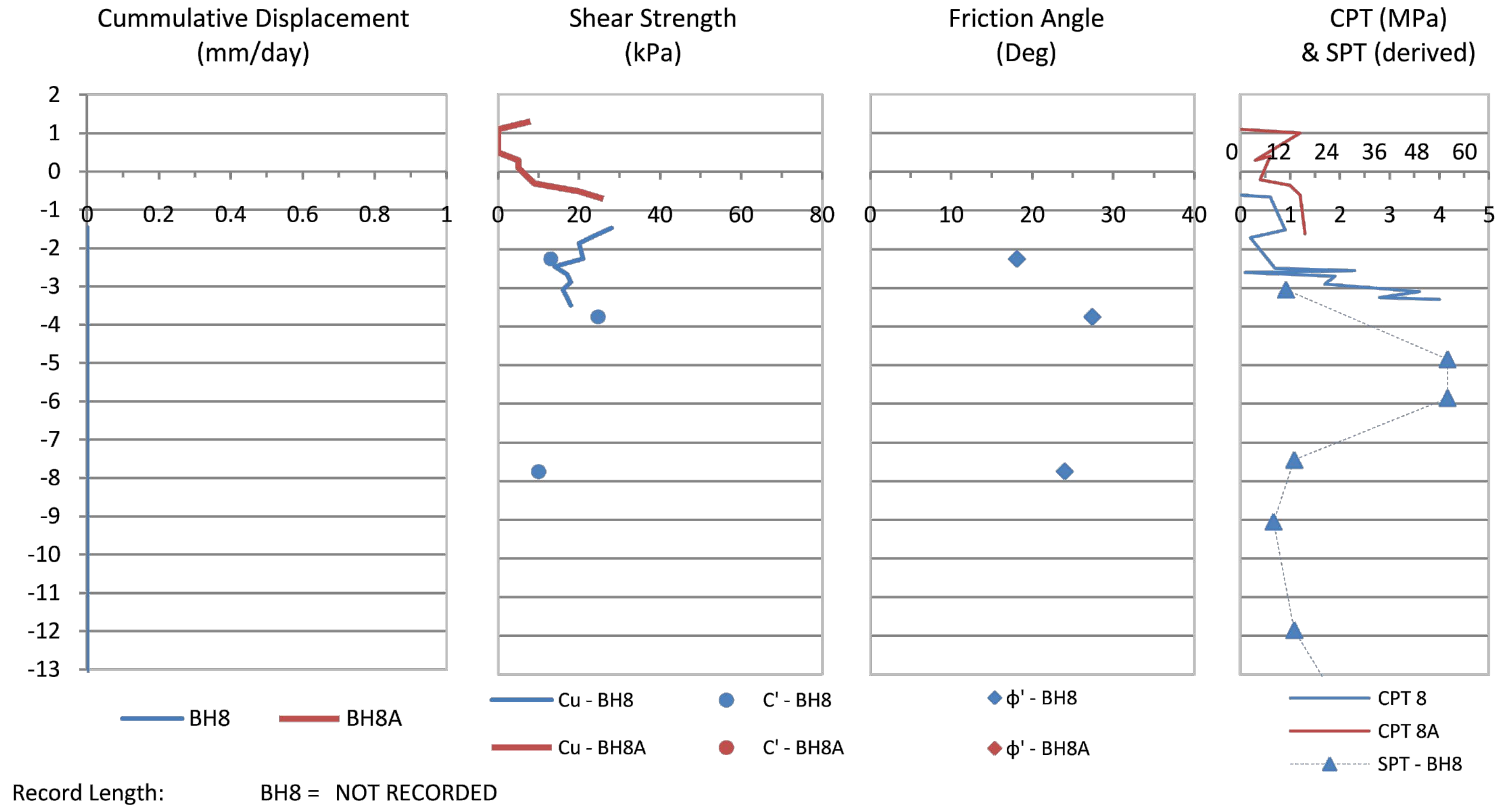
Test Location 7



NOTE: Approximate borehole locations shown above based on recorded RL. For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

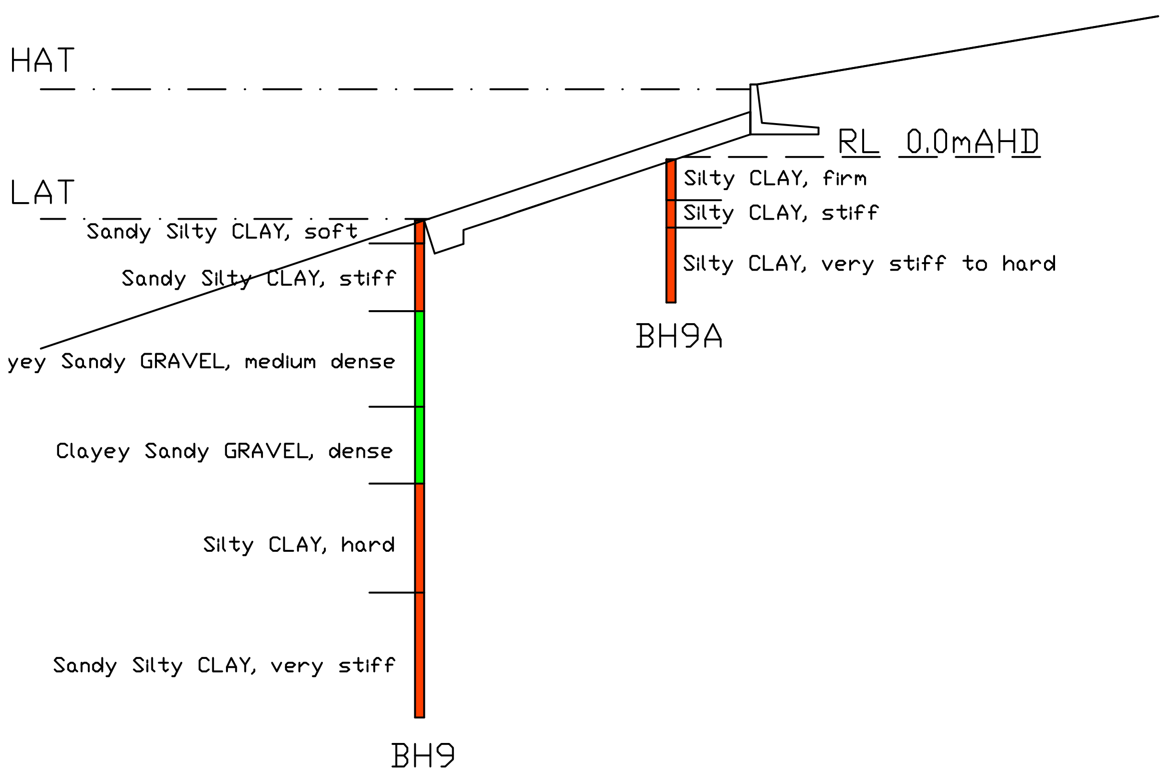
Test Location 8

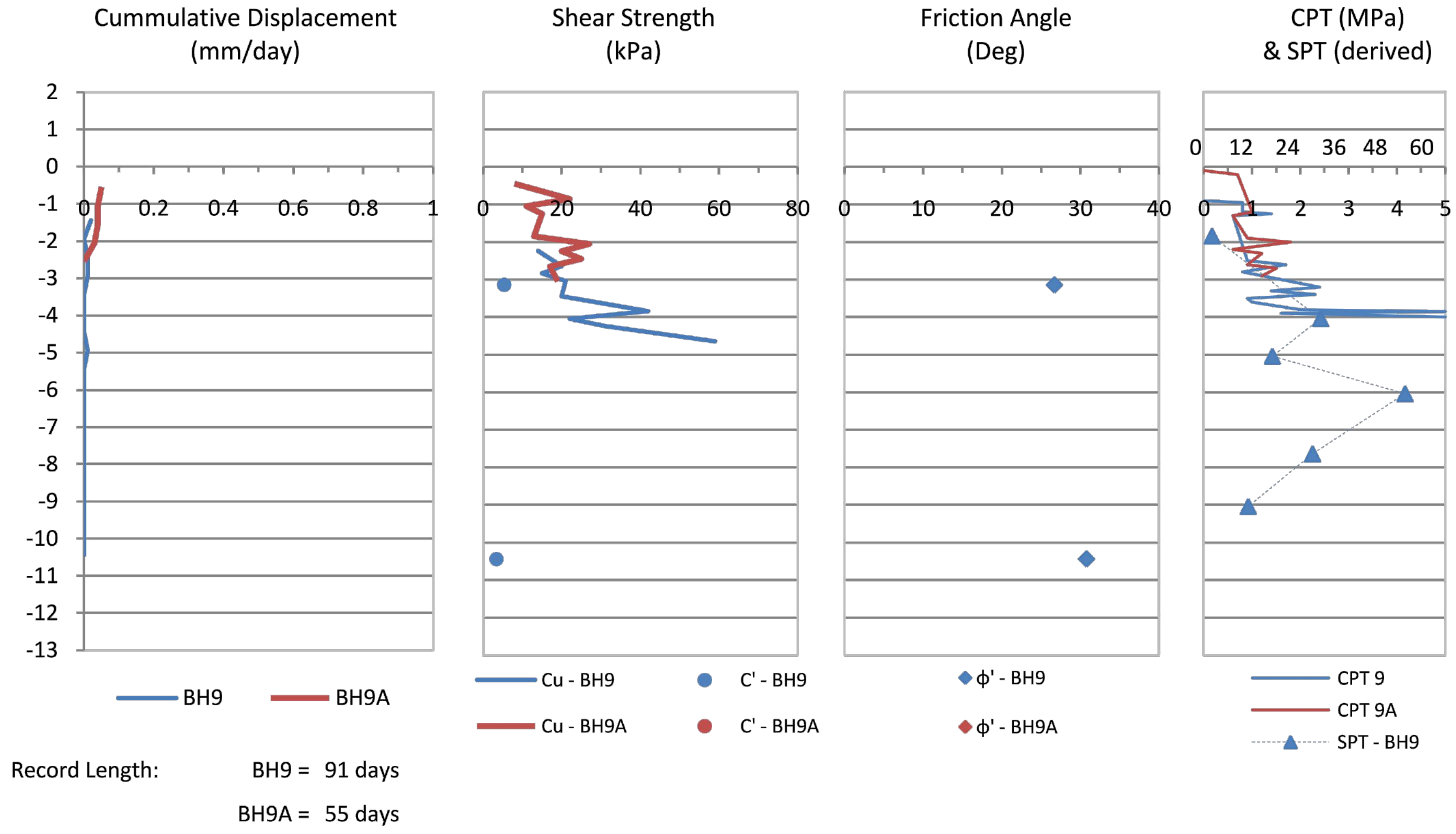




NOTE: Approximate borehole locations shown above based on recorded RL. For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

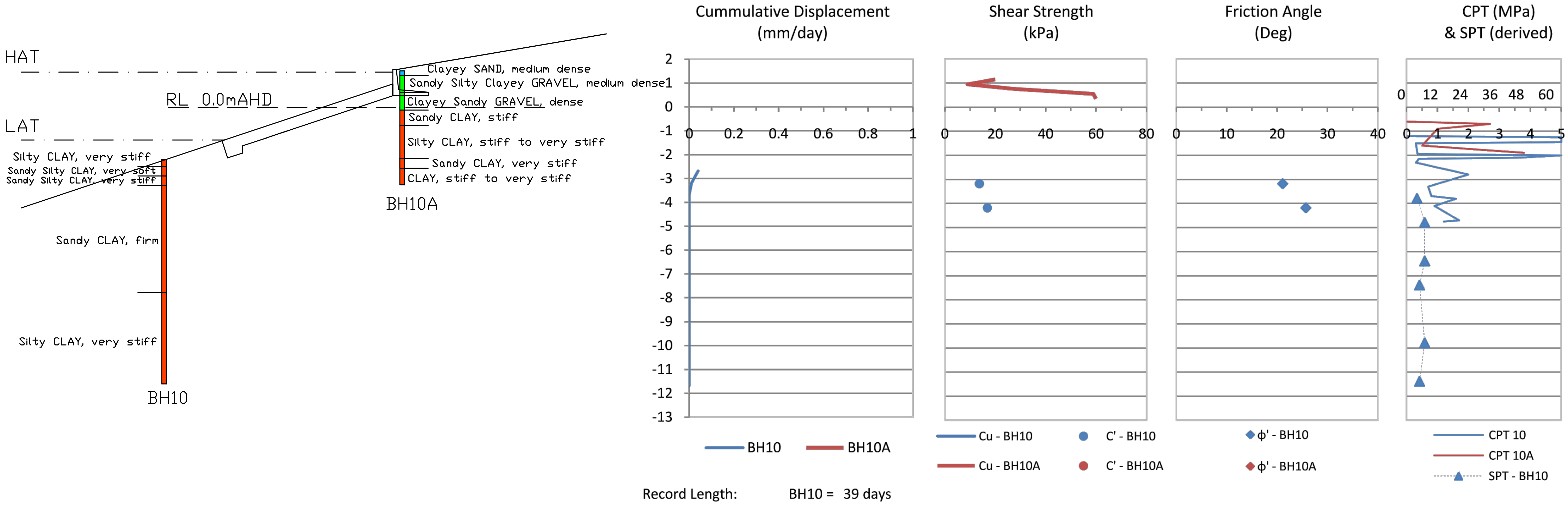
Test Location 9





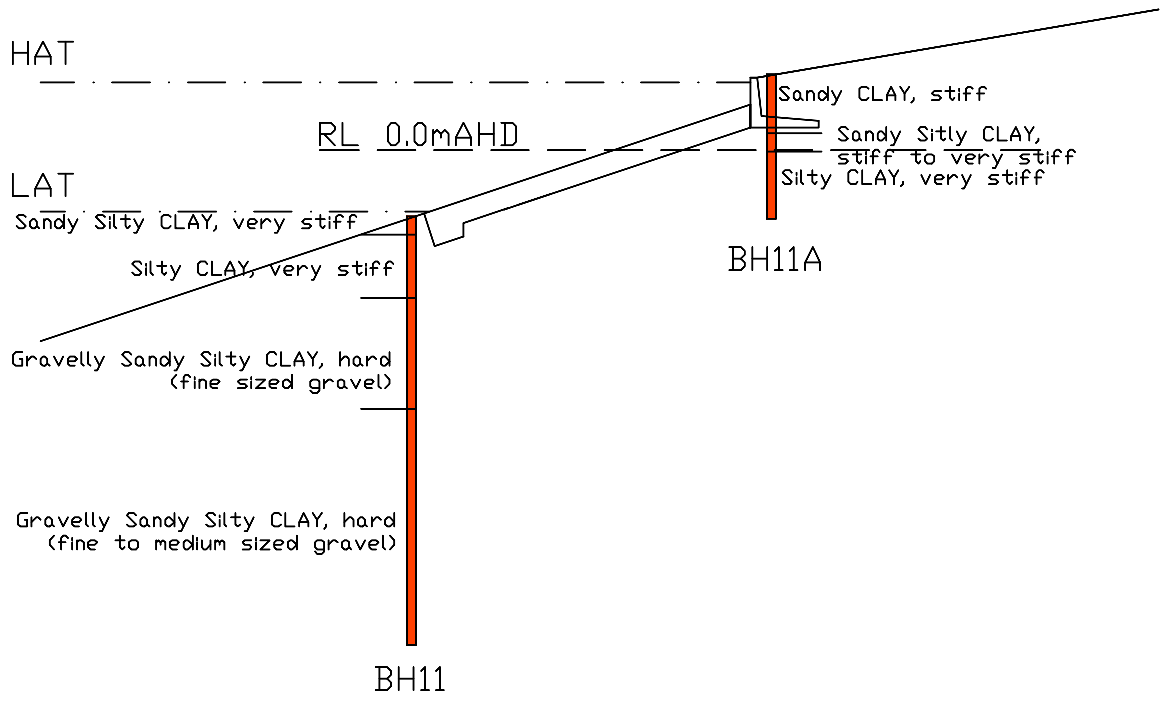
NOTE: Approximate borehole locations shown above based on recorded RL.
For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

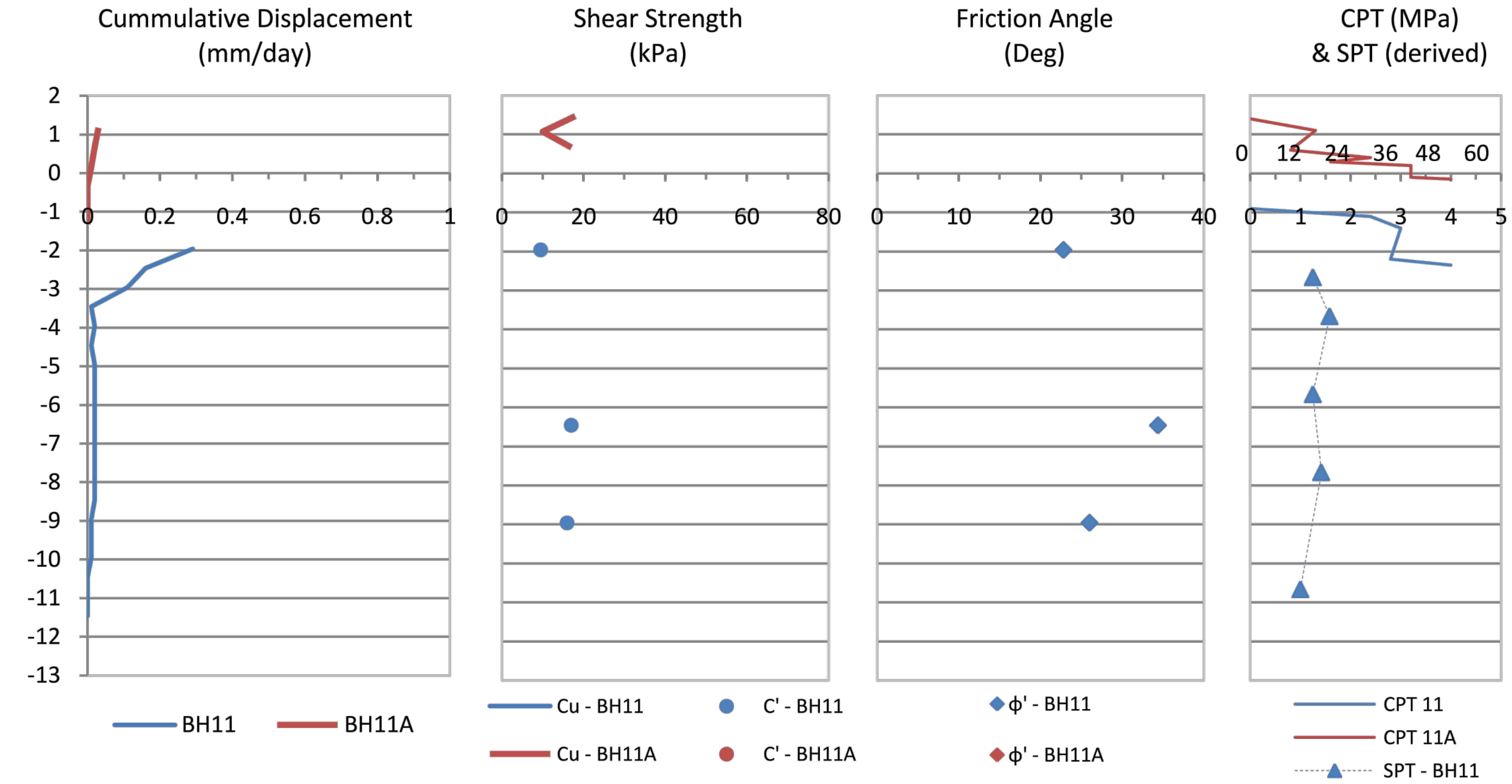
Test Location 10



NOTE: Approximate borehole locations shown above based on recorded RL. For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

Test Location 11



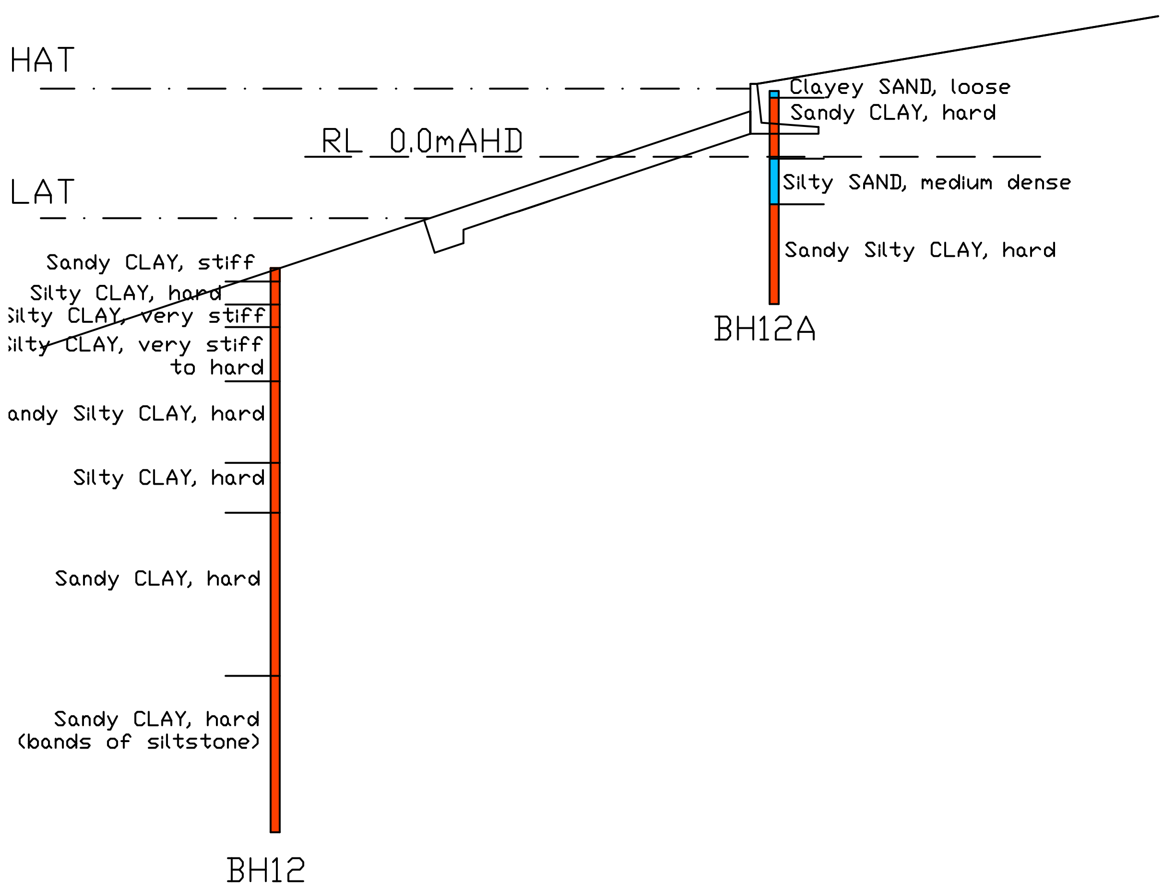


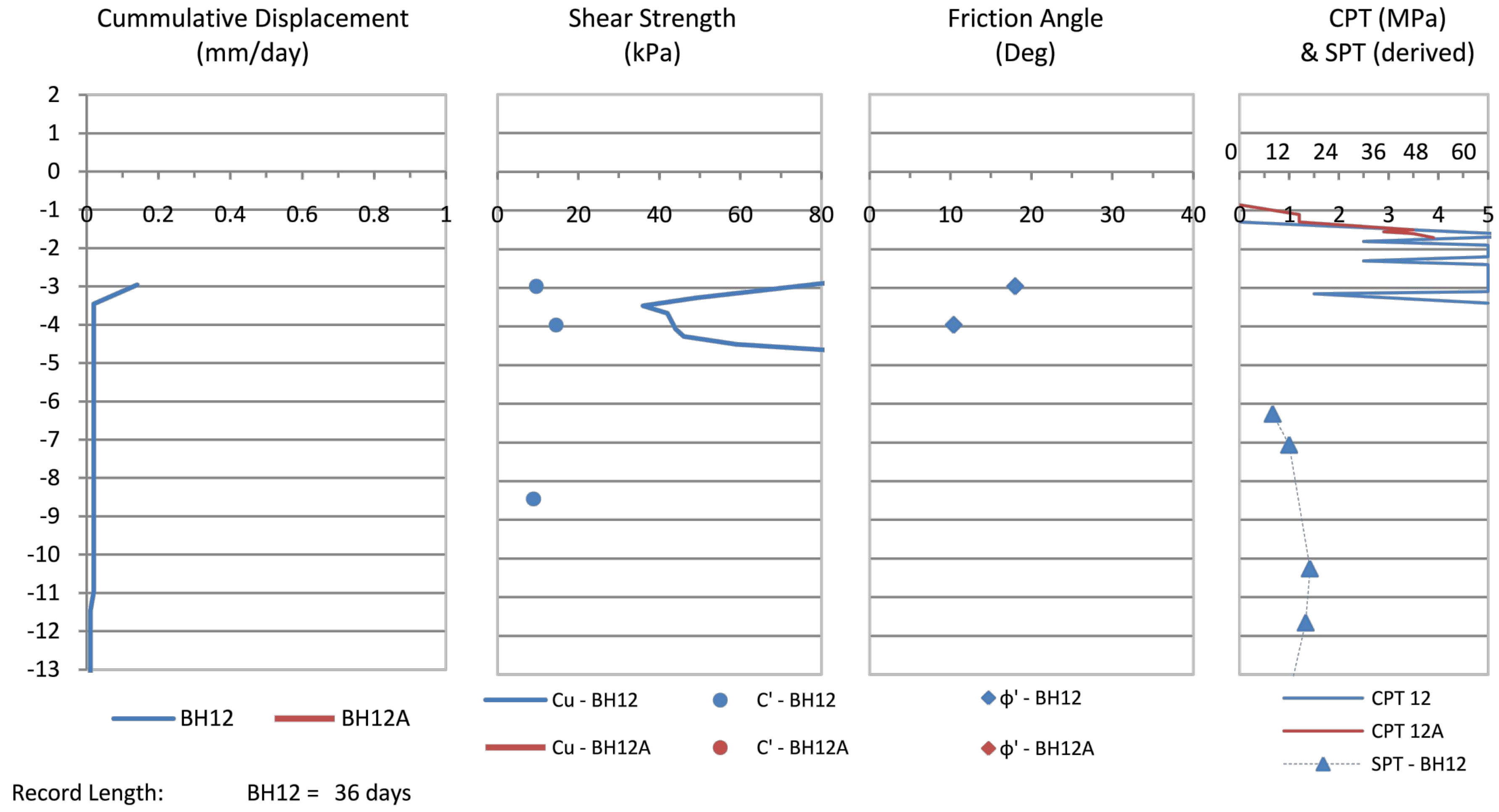
Record Length: BH11 = 37 days

BH11A = 37 days

NOTE: Approximate borehole locations shown above based on recorded RL. For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

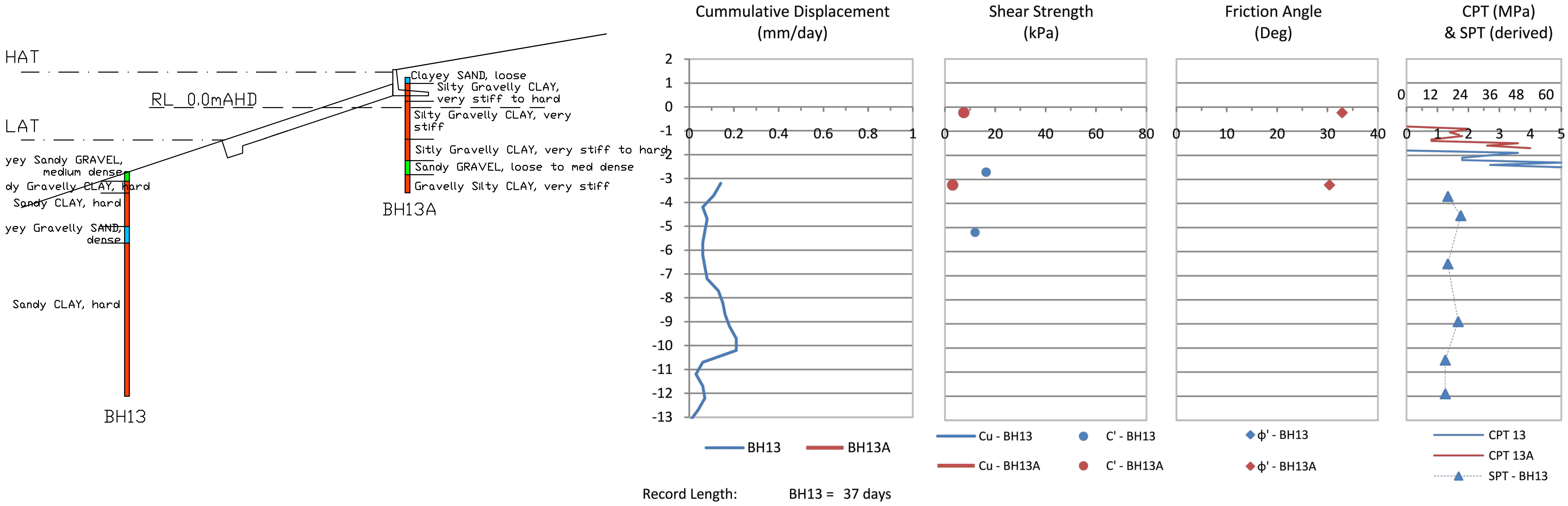
Test Location 12





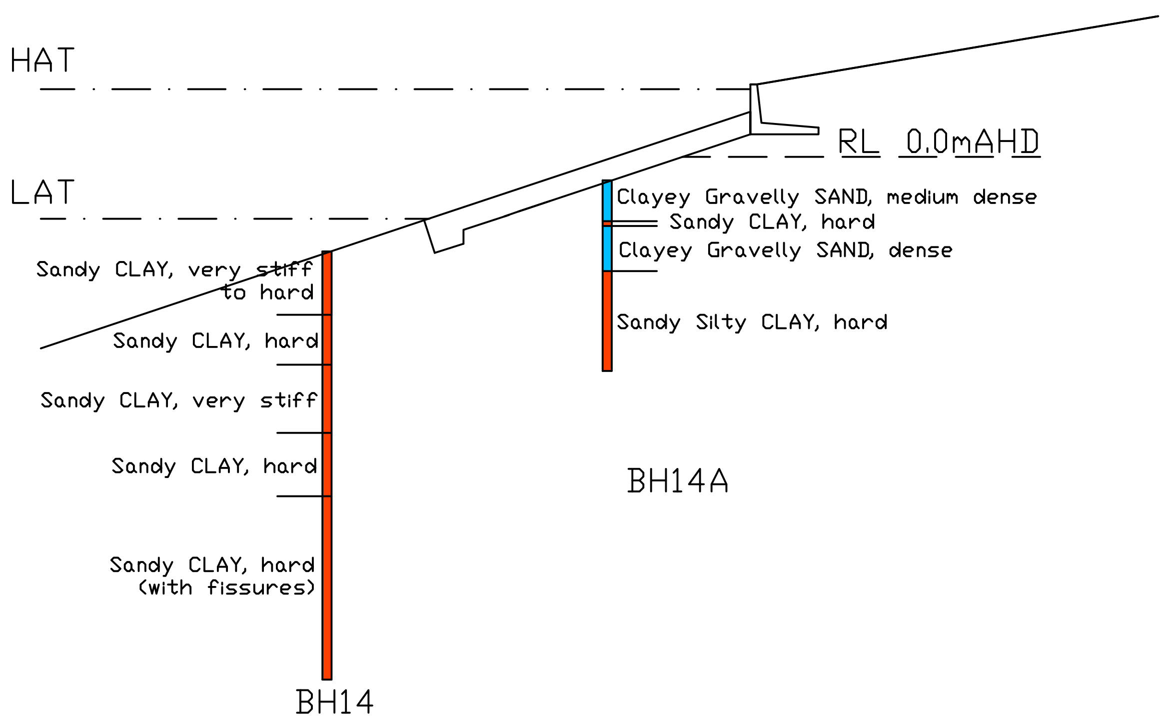
NOTE: Approximate borehole locations shown above based on recorded RL. For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

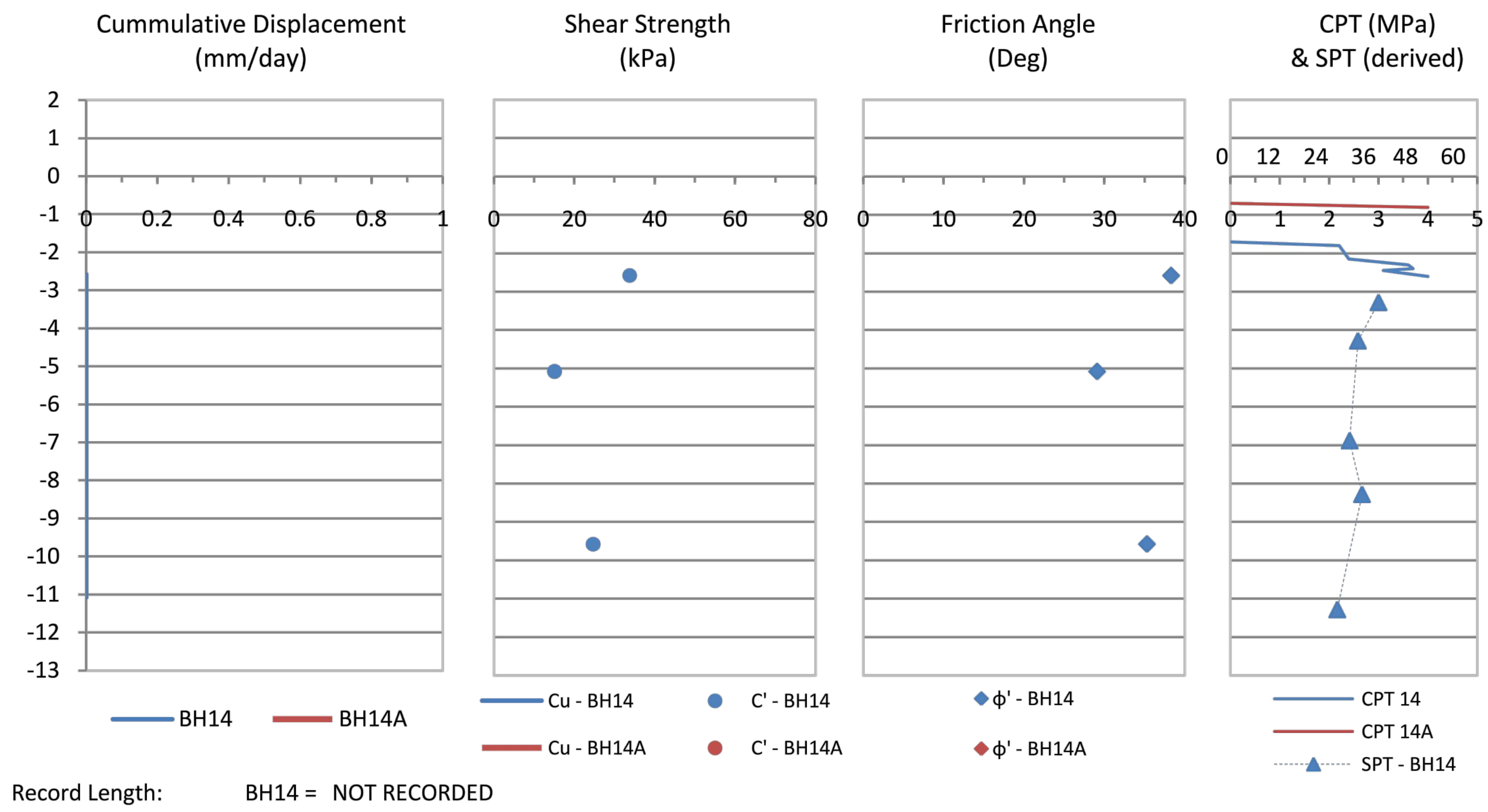
Test Location 13



NOTE: Approximate borehole locations shown above based on recorded RL. For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

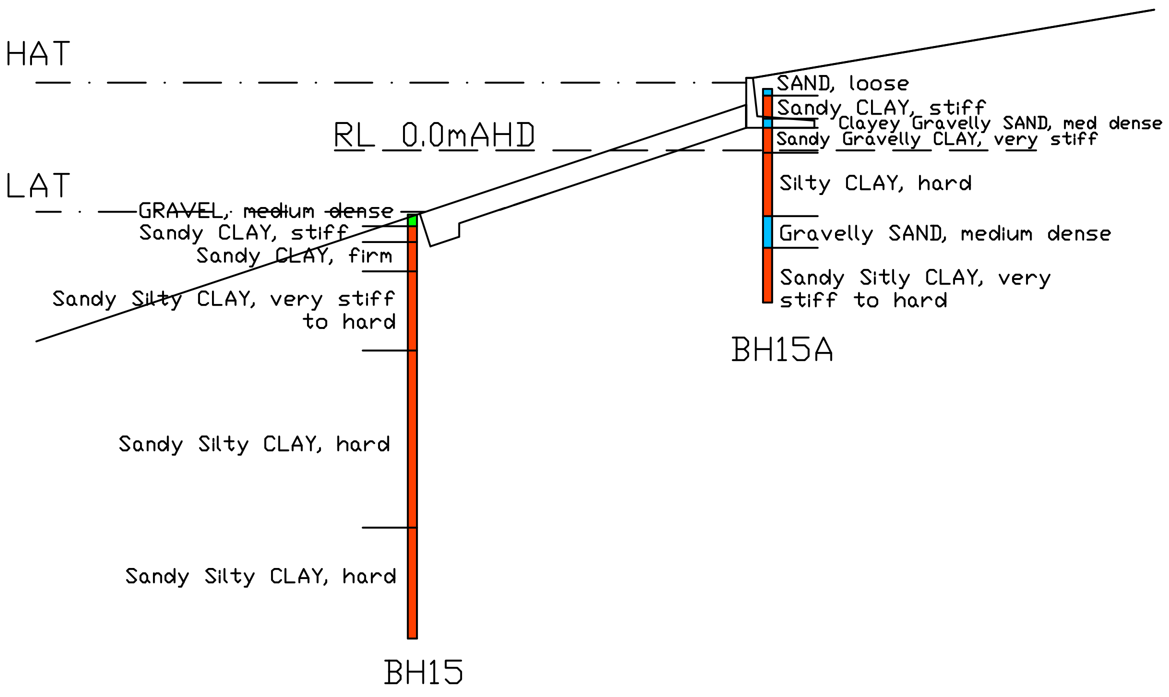
Test Location 14

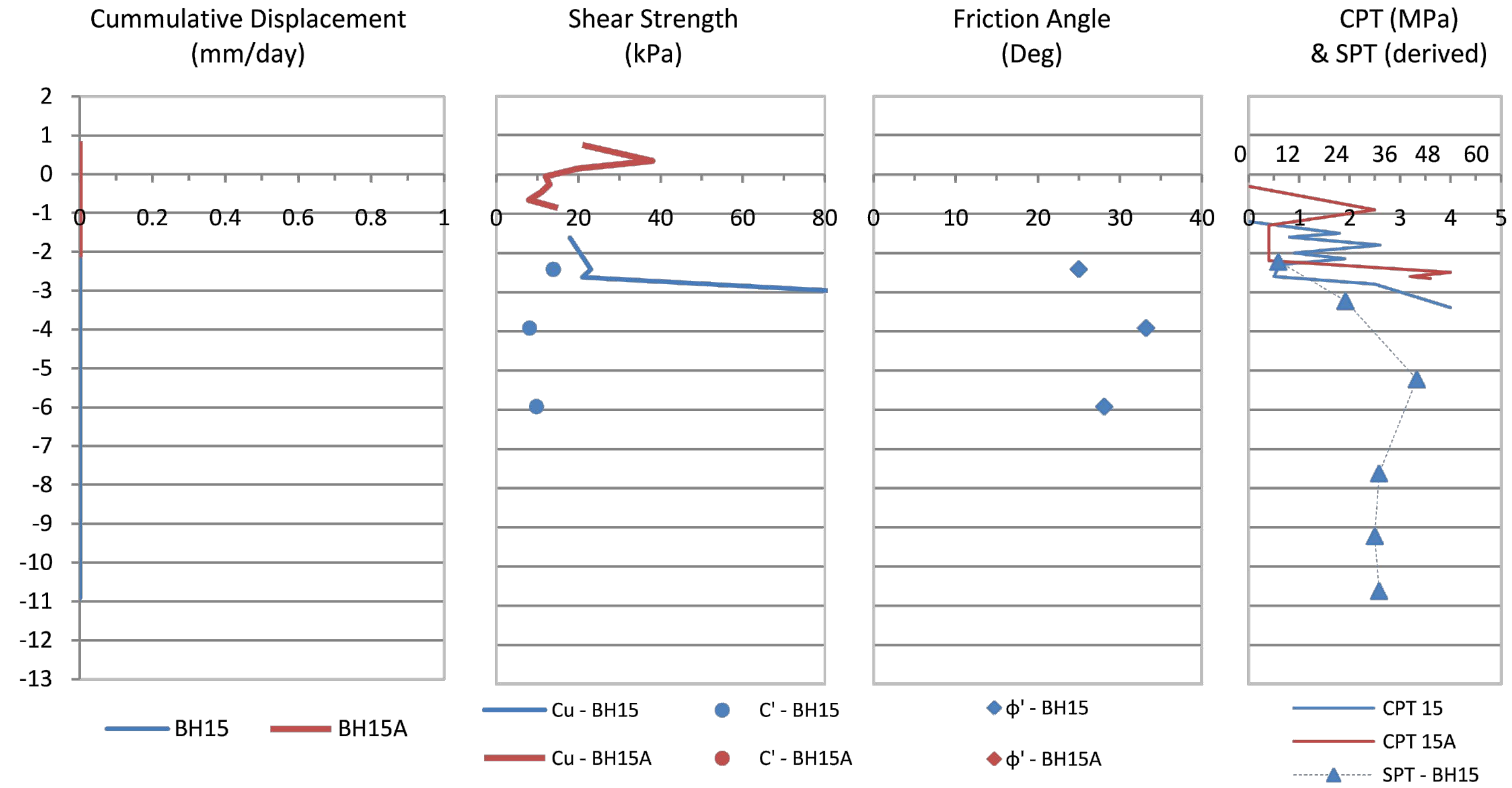




NOTE: Approximate borehole locations shown above based on recorded RL. For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

Test Location 15



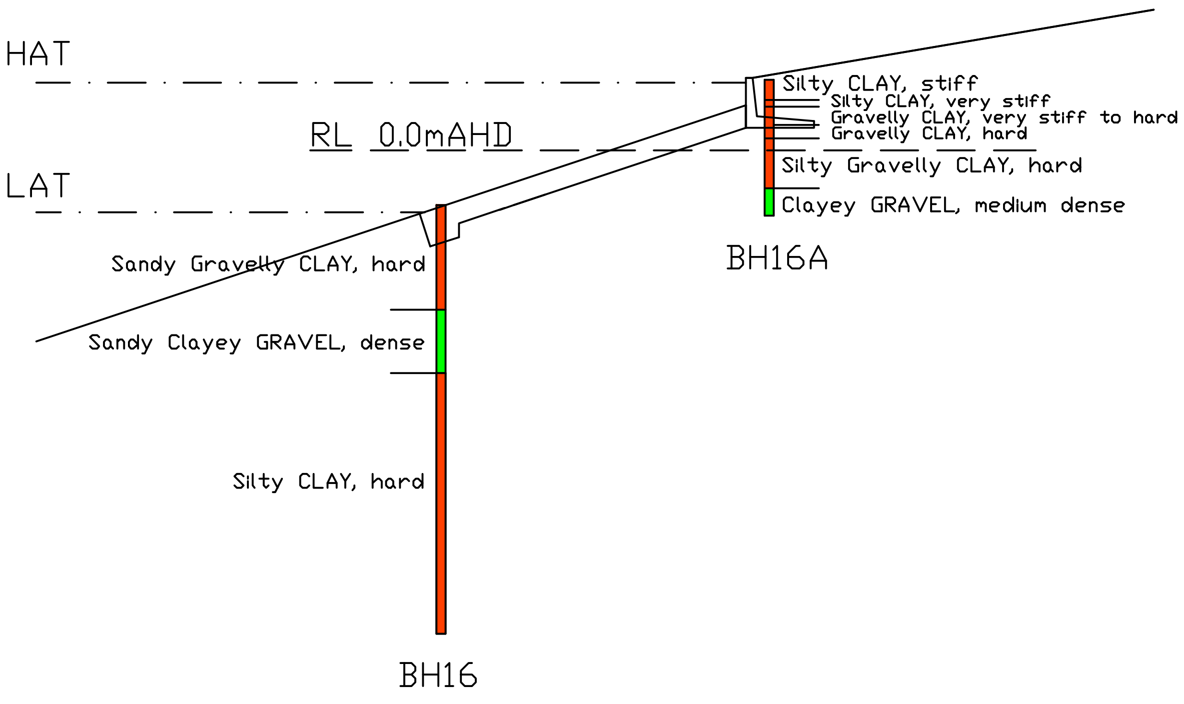


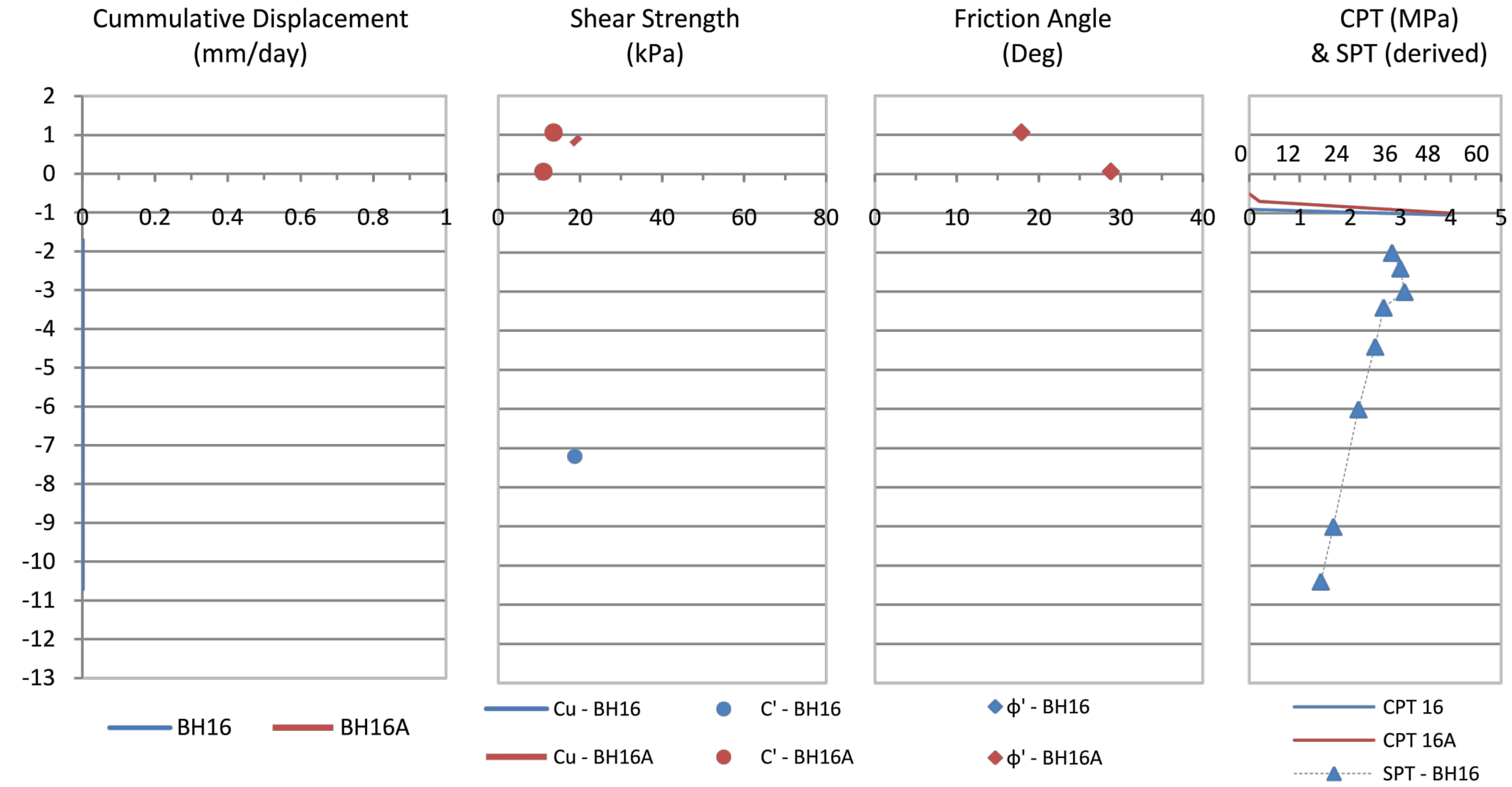
Record Length: BH15 = NOT RECORDED

BH15A = NOT RECORDED

NOTE: Approximate borehole locations shown above based on recorded RL. For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

Test Location 16

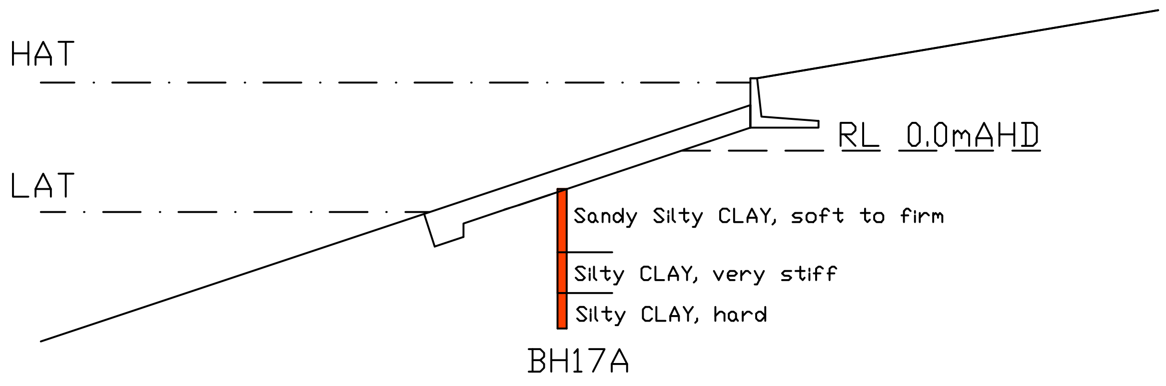


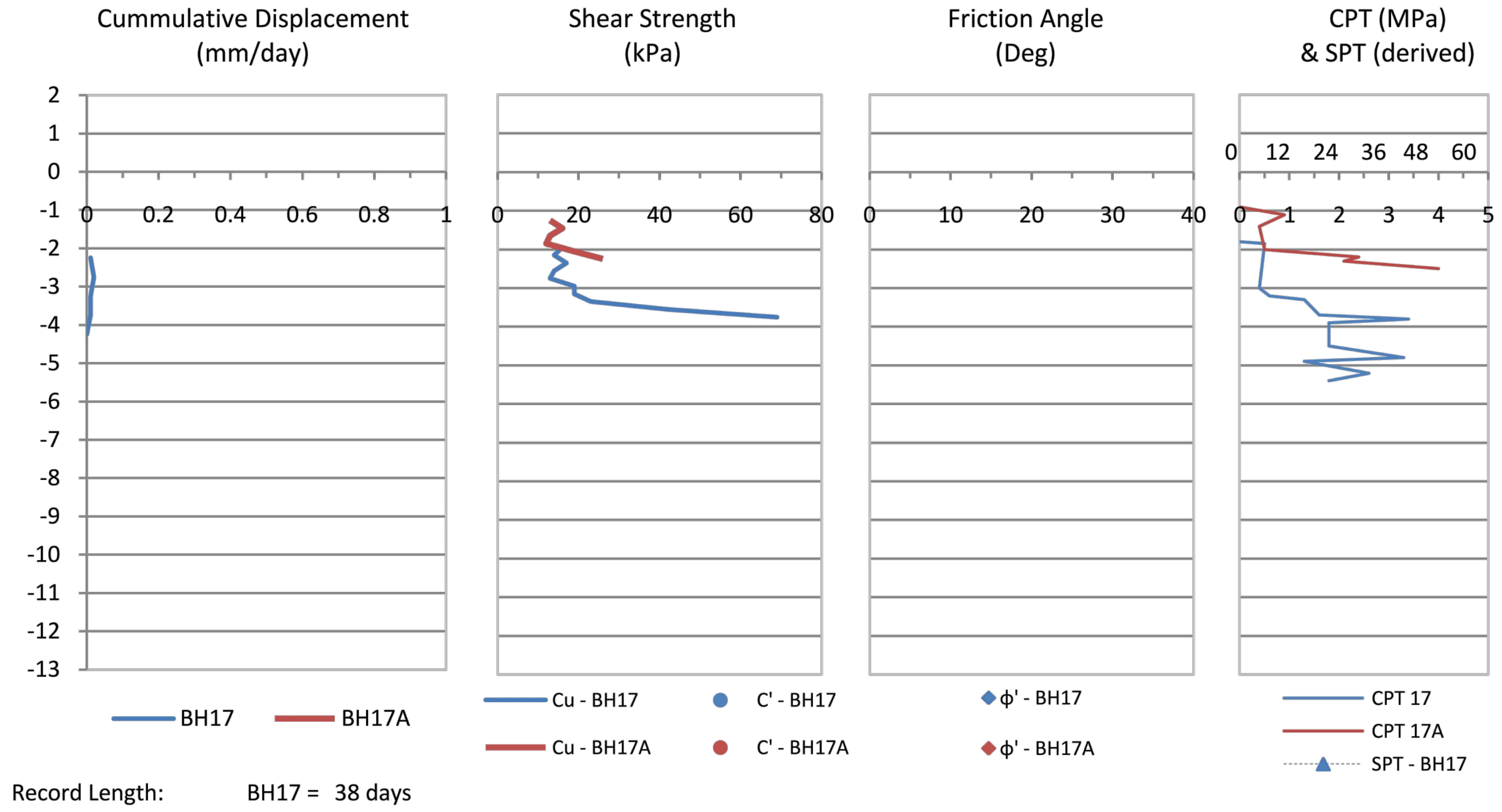


Record Length: BH16 = NOT RECORDED

NOTE: Approximate borehole locations shown above based on recorded RL. For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

Test Location 17





NOTE: Approximate borehole locations shown above based on recorded RL. For location of CPT and dilatometer tests refer borehole locations in Soil Surveys reports.

Appendix C

TYPICAL CANAL BATTER CROSS SECTION

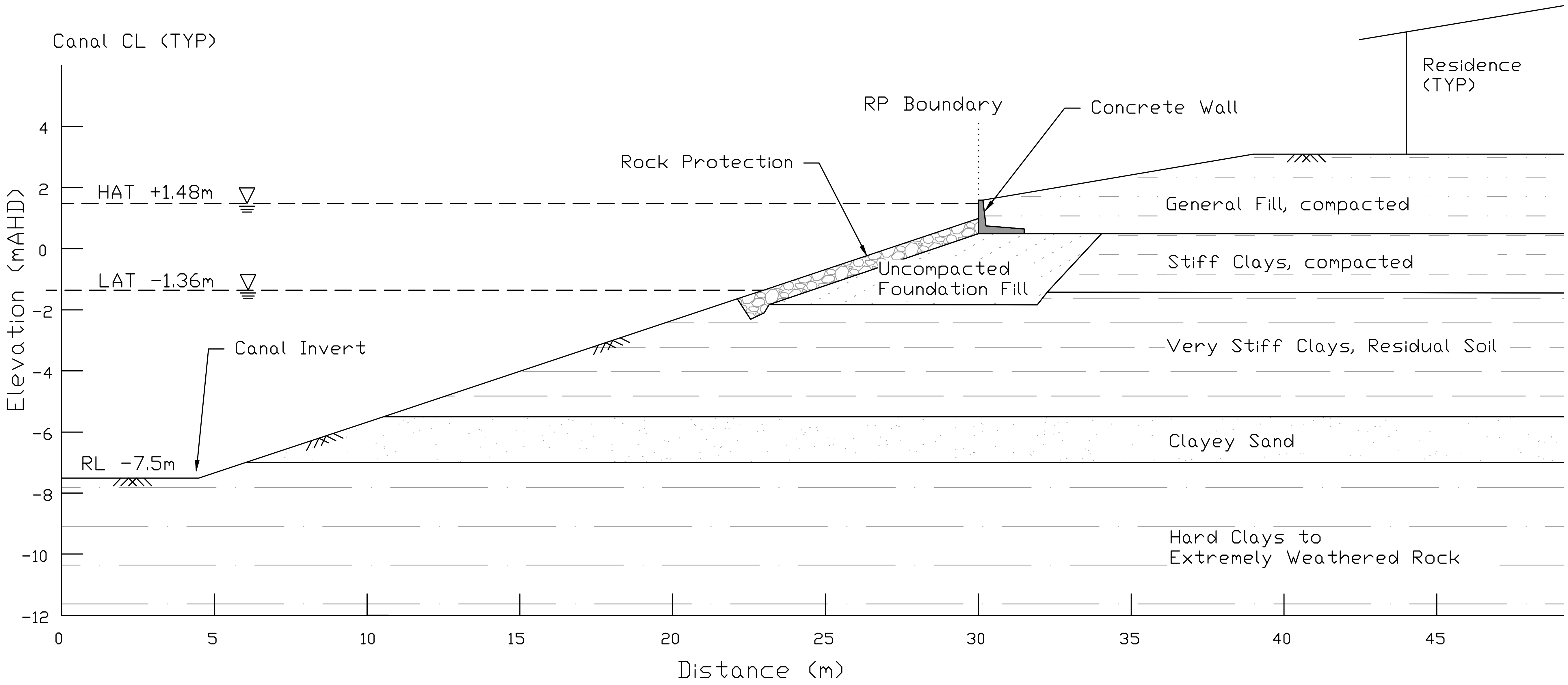
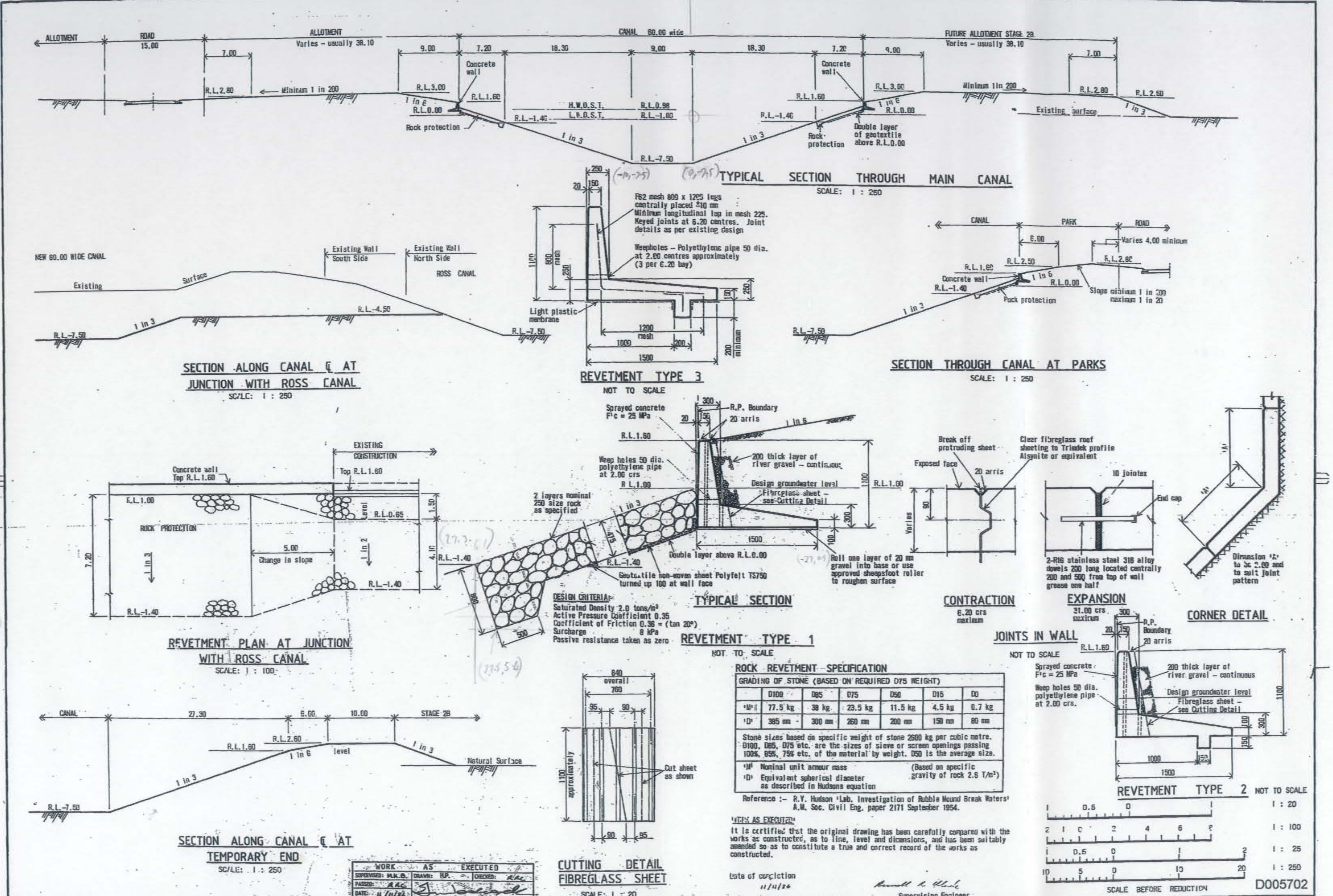


Figure 2.1 SIMPLIFIED SOIL PROFILE LAYOUT

Based on Construction Drawings, Soil Surveys Report and KBR Interpretation

Appendix D

HISTORICAL DRAWINGS



ROCK REVETMENT SPECIFICATION

GRADING OF STONE (BASED ON REQUIRED D75 WEIGHT)						
	D100	D85	D75	D50	D15	D0
'W'	77.5 kg	38 kg	23.5 kg	11.5 kg	4.5 kg	0.7 kg
'D'	385 mm	300 mm	260 mm	200 mm	150 mm	80 mm

Stone sizes based on specific weight of stone 2600 kg per cubic metre. D100, D85, D75 etc. are the sizes of sieve or screen openings passing 100%, 85%, 75% etc. of the material by weight. D50 is the average size.

'W' Nominal unit armour mass
'D' Equivalent spherical diameter as described in Hudsons equation
(Based on specific gravity of rock 2.6 T/m³)

Reference :- R.Y. Hudson 'Lab. Investigation of Rubble Mound Break Waters' A.M. Soc. Civil Eng. paper 2171 September 1954.

'WORKS AS EXECUTED'

It is certified that the original drawing has been carefully compared with the works as constructed, as to line, level and dimensions, and has been suitably amended so as to constitute a true and correct record of the works as constructed.

Date of completion
11/11/86

Supervising Engineer

WORK AS EXECUTED
SUPERVISED: M.K.O. DRAWN: H.P. CHECKED:
PASSED:
DATE: 11/11/86

D005702

DATE	REVISIONS	INIT.	Rev.
16-10-1986	Revetment Types 1 to 3 added	E.W.P.	E
9-4-1986	Note for preparation of base for concrete wall added	S.T.W.	D
7-4-1986	Geotextile position amended	S.T.W.	C
27-3-1986	Rock specification and thickness and wall joints amended - Cutting Detail added	S.T.W.	B
14-3-1986	Typical Section, Contraction and Expansion Joints amended and Corner Detail revised	S.T.W.	A

DATUM A.H.D.

ASSOCIATED CONSULTANTS

ACEA MEMBER

CARDNO & DAVIES Australia Pty. Ltd.
CONSULTING CIVIL STRUCTURAL AND TRAFFIC ENGINEERS
HEAD OFFICE 53 QUAY STREET BRISBANE 4000 PHONE (07) 310131 TELEX 41437
SURFERS PARADISE 4217 PHONE (075) 388333
TOWNSVILLE 4810 PHONE (077) 715092
PORT MORESBY P.N.G. PHONE 212244
MAROOCHYDORE 4558 PHONE (071) 432555

DESIGNED: E.P.M. SCALE: AS SHOWN APPROVED:
DRAWN: D.I.G. REF: A1
CHECKED:

CIVIC PROJECTS (RABY BAY) PTY. LTD.
RABY BAY DEVELOPMENT - STAGES 2-5
TYPICAL CANAL SECTIONS AND REVETMENT DETAILS

DATE: 1-5-1985
DRAWING No:
956/1-37
R1-7/34-24