Detection of shear zones in a natural clay slope using the CPT and continuous dynamic sampling

by

Mahmoud Mahmoud[1], David Woeller[2], and Peter K. Robertson[3]

[1]Golder Associates,
10th Floor, 940 - 6th Ave. S.W.
Calgary, Alberta T2P 3T1
Tel: (403)-299-5657
Fax: (403)-299-5606
email: mmahmoud@golder.com.ca
(formerly of Terra Engineering Ltd., Vancouver
and DID Consultants Ltd., Tehran, Iran)
[2]ConeTec Investigations Ltd., Vancouver
[3]University of Alberta, Edmonton

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Abstract

The detection of shear zones along which a mass of soil is moving is essential for understanding the state of stability of natural slopes. Weaker zones in clayey soils can be identified from low values of cone penetration test (CPT) tip resistance measured during penetration. This paper presents a case history illustrating the identification of softened shear zones in clay soils using the CPT and the detection of shear surfaces using continuous dynamic sampling (CDS). The analysis and interpretation of the CPT data are discussed in relation to the detection of shear surfaces using the CDS at a site with a history of slope instability.

Key Words: Slope stability, in-situ testing, case history

Introduction

The analyses and design of remediation of unstable slopes requires a knowledge of the failure mechanism and geometry of the failure surface. Once the mode of failure is established, it is generally possible to arrest further movement and thus achieve stability through the implementation of remedial works. In most soils the detection of the failure surface can be difficult depending on the soil type and ground profile. Conventional drilling and sampling can be expensive due to the large number of samples required. Excavating hand-dug test pits is also expensive due to the labour intensive nature of this method of investigation and because of the safety requirements for shoring works below depths of typically 1.2m.

The cone penetration test (CPT) can be a cost effective technique for investigating ground conditions in soil slopes where penetration testing is possible. Through recent advances the CPT provides continuous profiles of various measurements, including penetration tip resistance (q_t), sleeve friction (fs) and penetration pore pressure (u), in a repeatable manner.

In this paper a case history of an unstable natural clay slope is presented illustrating the application of the CPT combined with an inexpensive continuous dynamic sampling technique to identify shear zones. The history of instability at the site is quantified by means of back-analysis using the available soils data and groundwater information. The stability analyses parameters and results are presented in Appendix 1.

/...3

Site description and history of instability

The case history presented in this paper relates to a 0.8-hectare site located above the Kanaka Creek floodplain in Maple Ridge, British Columbia. The town of Maple Ridge lies north of the Fraser River and is situated approximately 40km east of downtown Vancouver. Two creeks flow past the site, Kanaka Creek at approximately 125 m to the south and Cottonwood Creek at approximately 100 m to the west. A site location plan is given in Figure 1 which shows the site grade varying from approximately 14 m at the south end to approximately 31 m at the north, and sloping at approximately 8° to the horizontal. Beyond the site boundaries, the topography slopes downward in a southerly direction at approximately 12° to 15°.

The surficial geology of the site corresponds to interbedded marine and glaciomarine soils of the Fort Langley formation. This clay formation has a history of slope instability, especially along the Fraser River.

In recent years site vegetation has comprised mixed second growth Douglas firs and Western red cedars, open areas overrun by shrubs, stands of young red alders and black cottonwood, as well as some old pasture. With the exception of the second growth forest, the general state and variety of the vegetation on the site is generally regarded to be indicative of past instability.

The most recent history of slope instability at the site is outlined below and is supplemented /quantified by results of back-analysis which are presented in Figure A1, Appendix 1, together with the parameters used in the stability analyses. The discussion below concentrates on the most recent slope instability associated with three likely slip surfaces, S1, S2 and S3, Figure A1. These slip surfaces are also shown on Figure 3.

In the summer of 1993 a 1.5 to 2 m high cut was made at the toe of the slope to accommodate an access lane. Back-analysis shows that the factor of safety corresponding to a likely slip surface S1 (Figure A1) is near unity, thus indicating that the slope was only marginally stable prior to commencement of excavation works for the access lane. As can be seen from Figure A1, these excavation works resulted in reducing the factor of safety which consequently led to a slide occurring in the general area of this cut slope between October 25 and 26, 1993. This toe slide resulted in the instability of the upslope areas, with the factor of safety corresponding to an upgradient slip surface S2 (Figure A1) - initiated approximately 25 m upslope from the toe - reducing from 1.26 to 0.92. By January 1994 a tension crack was noted at the location of slip surface S2. Subsequent to the development of this tension crack, a slope indicator (SL1, Figure 1) was installed along the south property boundary in February 1994 and indicated a shear zone between 3 to 4 m below ground level.

Analysis shows that slip surface S3 extends beyond the tension cracks observed in January and February 1994 and corresponds to a computed factor of safety of 0.91, thus suggesting that ground movements could have extended further upslope. Although no tension crack was evident in the upslope areas, the presence of shear surfaces in the continuous samples collected in these areas in December 1995, and described later under "Continuous dynamic sampling (CDS)", confirms that ground movements had likely occurred at some time in the past (see, e.g. profiles at CDS5 and CDS6, Figure 3).

Slope remedial works commenced on February 25, 1994. However, following heavy rainfall between February 26 and 27, 1994, additional slope movement occurred and by March 4, 1994, approximately 30 tension cracks had developed upgradient from the slope toe, the aerial extent and depth of which are shown in Figures 1 and 3 respectively. By March 9, 1994 measured lateral movements had reached greater than 1.2 m within the slide area. Slope stabilizing piles were subsequently installed during the summer of 1994, and resulted in arresting further movement of the slide area by the autumn of 1994. The factor of safety following slope stabilization works is computed to have increased to 1.23, Figure A1. This increase in factor of safety is confirmed by the survey data collected between July 1994 and April 1995, which indicate between 5 to 10 mm of downslope movement over a period of nine months following completion of remediation works.

The history of instability at the site is summarized in Table 1.

Site characterization & shear zone detection

Site characterization was achieved through a conventional borehole investigation (BH1 to BH9), continuous dynamic sampling (CDS1 to CDS10) and cone penetration testing

/...6

(CPT3 to CPT6, and CPT8 to CPT10). Dynamic penetration tests (DPT) were carried out adjacent to boreholes BH1 to BH3, with in-situ downhole vane shear tests (VST) made at borehole BH8. The DPT corresponded to the International Reference Test Procedures for Dynamic Probing Super Heavy (DPSH) and comprised driving a 60^{o} solid cone shaped probe (60 mm diameter; 150 mm sleeve) using a 140 lb (63.5 kg) Pilcon trip hammer which utilizes a winch and falls freely a height of 30" (0.76 m), thus simulating closely the Standard Penetration Test (SPT) procedures. The two techniques of continuous dynamic sampling (CDS) and CPT were further utilized for delineating the aerial extent and depth of instability of shear zones within the natural clay soils underlying the site. The locations of these test holes are shown in plan in Figure 1, while Figure 2 presents a typical borehole log together with profiles for Atterberg limits, water content determinations, field and laboratory vane strengths, and DPT blowcounts.

Ground conditions were found to comprise of a weathered light brown to brown, stiff to very stiff over-consolidated clay (to a depth of about 3 m), in turn underlain by an unweathered grey, soft to firm, normally to lightly over-consolidated clay to depths of at least 15 m.

DPT blowcounts (per 300 mm penetration) of between 2 to 5 were measured in the weathered layer, with increasing values of between 6 and 16 measured over increasing depths of 4 to 15 m within the unweathered clay. Peak field vane undrained shear strengths obtained between depths of 4 m and 10 m, corresponding to the unweathered clay stratum, were found to increase with depth, ranging in value between 33 kPa and 45 kPa

/...7

with an average of 40 kPa. Values of sensitivity, measured as the ratio of peak to remoulded vane strength, were approximately 3 in the weathered layer and between 4 to 6 in the unweathered materials.

From monitoring pneumatic piezometers, groundwater conditions were found to correspond to depths of approximately 5 m near the top of the slope (at the location of BH8), and daylighting at one third of the way down the slope (at the location of BH7), see Figures 3 and A1.

Particle size distribution analyses on the weathered materials indicated a clay fraction of about 50%, a silt fraction of about 35% and 15% corresponding to sand sizes, with the unweathered materials having a gradation of approximately 40% clays, 55% silts and 5% sands. Values of natural moisture content between ground surface and depths of 15 m were found to lie in the range 30% to 90%, with plasticity indices of 55% and 28% corresponding to the weathered and unweathered layers, respectively.

Continuous dynamic sampling (CDS)

The CDS technique involves pneumatically driving into the ground a 1143 mm long sampling tube with an inside diameter of 38 mm and a wall thickness of 2 mm (Geoprobe Systems, 1995); these dimensions being similar to the fixed piston thin-walled sampler. The sampler is attached to the end of drill rods which are in turn attached to a pneumatic hammer mounted on a portable rig, and handled by a two-man crew. Continuous samples

/...8

are obtained within clear plastic liners (0.5 mm thick) located in the sampler as it is being advanced into the ground.

CDS test holes were advanced to depths in the range 3.7 m (CDS1) to 8.5 m (CDS5 and CDS10), see Table 2.

Upon retrieval from the ground, each plastic liner was allowed to slide out of the sampler, and its two ends sealed with wax for transportation to a soils laboratory. In the laboratory, each tube was carefully cut along its longitudinal axis, visually examined and classified.

The pre-existing shear surfaces observed from examining continuous samples obtained at CDS1 to CDS10 are summarized in Table 2 and illustrated in Figure 3. As can be seen, shear zones were encountered within the top 1 to 3.5 m of the weathered clay crust, as well as within the top 3 m of the unweathered layer. Water contents of the shear zones within the weathered layer ranged from about 40% to 80%, with values of between 40% to 50% corresponding to the shear zones within the unweathered stratum.

Cone penetration testing (CPT)

Cone penetration testing (CPT) was carried out at seven locations, within 1-2 m of adjacent continuous sampling test holes CDS3 to CDS6, and CDS8 to CDS10, respectively (see Figure 1). The CPT device comprised a standard 10 cm^2 penetrometer with a low capacity 2.5 tonne tip load cell. Except for CPT8 which was advanced to 20 m below the surface, depths of penetration were approximately 10 m. A typical CPT profile from the site is shown in Figure 4, illustrating the variation of penetration tip resistance (q_t) with depth.

The CPT qt profile in Figure 4 indicates some surface fill overlying a clay deposit which appears to be over-consolidated in the upper few metres, becoming less over-consolidated with depth. The CPT profile also indicates that the clay deposit is quite variable with rapid changes in penetration resistance. The measured values of q_t obtained from the seven CPTs vary between about 2 bar and about 25 bar, with the highest values recorded in the desiccated clay near the ground surface, see Figure 5. The low capacity CPT equipment had an accuracy in q_t of $\pm$ 0.25 bar (1 bar = 100 kPa).

To establish the range of q_t for undisturbed clay, CPT5 was carried out (adjacent to CDS5) in an area with no apparent previous surficial (slope) movement and where q_t varied between about 5 bar close to the surface and 9 bar at a depth of 10 m. This undisturbed clay profile is superimposed on Figure 4 and labelled "Undisturbed (O.C.) profile". q_t profiles of this magnitude suggest some degree of over-consolidation and q_t values less than this reference line are indicative of some softening and loss of undrained shear strength, although not necessarily slip surfaces or previous ground movement, see shaded area labelled "Zone of softened clay" on Figure 4.

Normally consolidated clayey deposits have a normalized penetration resistance

($Q_t = (q_t - \sigma_v)/\sigma_v'$) value of approximately 3 to 5 bar based on an undrained shear strength

(S_u) to vertical effective stress (σ_v') ratio of S_u / σ'_v equal to 0.25 to 0.3. A line showing this range of q_t values for undisturbed normally consolidated clay is also shown on Figure 4 and labelled "N.C. profile".

Furthermore, the shear surfaces observed at the adjacent CDS test hole are also incorporated in Figure 4. Of particular note is the agreement between depths at which significantly lower q_t values - labelled as "Softened zones" - were measured with the CPT and approximate depths where shear surfaces were identified by continuous sampling.

Figure 5 presents the CPT q_t profiles along section B-B (Figure 1), together with the softened zones identified. Also shown on Figure 5 are three likely slip surfaces passing through the CPT identified softened zones.

Discussion

It has been illustrated in the preceding sections (Figure 4) that there is a good correlation between CDS detected shear surfaces and softened zones identified from low q_t values in CPT profiles. Table 3 presents a comparison of CPT identified softened zones and CDS detected shear surfaces at the seven test locations where both continuous sampling and cone penetration testing were carried out. As can be seen, there is generally good correlation between the depths over which softened zones were identified and depths at which shear surfaces were detected. It should be remembered that the CDS testholes were located within 1-2 m of each of the adjacent CPT testholes. Furthermore, as outlined in the

/...11

previous section on "Site description and history of instability", the failure mechanisms at the subject slopes are considered as complex, thus implying the presence of a number of shear surfaces at a given location. Nevertheless, the CDS and CPT techniques are considered to be complementary with the CPT yielding continuous measurements of cone tip resistance as penetration proceeds, and the CDS enabling a visual inspection to be made of a continuous column of soil at locations of interest. The recently developed Vision CPT device (Hryciw et al., 1998) combines the two methods of CDS and CPT, and it would be interesting to see the degree of success that would be achieved in identifying pre-existing softened zones with the Vision CPT.

It has also been shown that the softened zones may be identified from CPT measured values of q_t which are lower than those found within the undisturbed silty clay materials at similar depths. These lower q_t values are attributed to the effects of previous shearing and/or ground movement.

There is little published documentation of procedures for identifying pre-existing shear/softened zones based on characteristic geotechnical parameters obtained from the CPT. However, the following publications are worthy of note. Leroueil et al (1995) have shown that the CPT can identify shear zones in eastern Canada sensitive clay soils having a plasticity index of 42% and an over consolidation ratio of about 1.35. In zones of high shearing where the clay was remoulded both the penetration tip resistance and the penetration pore pressure were low relative to the undisturbed clay at the same depth. The initial work by Leroueil et al has recently been expanded in an unpublished paper by

/...12

Demers et al. (1998). Romani et al (1988) also used the CPT together with downhole logging to define the location of an old landslide in the weakly cemented siltstones and sandstones of southern California. The landslide mass was composed of clays and silty clays, and the slip plane was identified from low values of cone penetration resistance and slickensides found within 'suspect zones of possible sliding' in samples.

Conclusions

The marine clays in the Lower Mainland of British Columbia often consist of a weathered, overconsolidated layer in the upper few metres, overlying unweathered materials which become less over consolidated with depth. These marine clays are often sensitive and heterogeneous. The sensitive and heterogeneous nature of these clays can be clearly identified by the CPT.

This paper has illustrated that the CPT can be used to identify pre-existing shear surfaces / softened zones within a clay deposit. Shear zones present within the Maple Ridge marine clays were identified by low values of cone penetration tip resistance compared to values within the undisturbed clay at similar depths. However, due to the influence of such variables as soil stress history, plasticity, sensitivity and structure, as well as variations in soil profile, continuous sampling, such as the CDS technique described here, should be utilized as a calibrator and to aid in the verification of CPT interpretation. In this way, it is possible to establish the presence or otherwise of pre-existing shear zones within clay deposits when performing a geotechnical investigation to evaluate slope stability.

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Appendix 1

Stability analyses parameters and results

Back-analysis of the history of instability at the site was performed using the available soils data and groundwater information. Constant volume/critical state conditions were adopted within the weathered and unweathered soil layers, with the constant volume friction angle (ϕ'_{cv}) established from isotropically consolidated undrained triaxial tests to be 27°. Residual shear strength conditions were assumed within sheared zones. The residual friction angle (ϕ'_{r}) was taken as 10°, based on published data.

Effective stress analyses based on Janbu's routine method were carried out to compute values of factor of safety at different stages of the most recent slope instability (observed since the summer of 1993). The analyses were carried out along Section C-C, Figure 1.

List of Figures

List of Tables

Table 1
History of instability

Events	1993 Jun	Jul	Aug	Sep	Oct	Nov	Dec	1994 Jan	Feb	Mar
1.5 to 2m high cut made at toe of slope.		7/1	8/31							
Small slide (30m x 10m) occurred north of the cut excavation.					10/25 – 10/26					
Tension cracks noted, and slope indicator casing installed in slide area.								1/3 – 1/3		
One tension crack noted at 26m from toe of slope.								1/20 – 1/20		
Slope indicator (SL1) indicated failure plane 3-4m deep with average movements of 35mm/day. Tension cracks were sealed with native clay and topsoil.									2/11 – 2/15	
Remedial works were initiated, consisting of a rock and gravel toe berm with 0.5m ditch excavated above slide toe to divert surface run-off.									2/25 – 2/25	
Surface drainage ditch scoured into fill soils adjacent to the east perimeter road, and resulted in lateral movements and cracks of 10 to 20mm width in the pavement.									2/26 – 2/27	
Washed-out ditch was filled with 75 to 100mm rock. Slope indicator (SL5) and movement stakes were installed across east perimeter road.									2/28 – 2/28	
Construction of remedial works on the slope toe.									2/28	3/9
20 to 30 tension cracks observed within 40m from slope toe, having widths of 150 to 600mm.										3/4 – 3/4
Lateral movements of between 12 and 60mm along east perimeter road, with lateral movements of slide area greater than 1.2m.										3/11 – 3/11
Sanitary sewer in the right-of-way had separated, and was subsequently repaired by municipal crews.										3/14 – 3/14

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Authors: M. Mahmoud, D. Woeller, P. K. Robertson

TABLE 2
Observed shear zones - CDS technique

Testhole	Depth (m)	Depth to weathered clay (m BGL)	Depth to unweathered clay (m BGL)	Depth/Elevation to observed shear zone * (m BGL/m)	Moisture Content (%)	Details of observation
CDS1	3.7	1.5	NE	NE	N/A	N/A
CDS2	6.1	0.2	3.7	NE	N/A	N/A
CDS3	6.1	0.3	3.6	3.05/15.55	80	slip plane at ± 45°
CDS4	4.9	0.3	3.7	1.3 - 1.65/ 20.2-19.8	52 - 66	vertical slip surface
CDS5	8.5	0.3	2.4	1.1/21.8	42	slip plane at 60°‡
CDS6	5.6	0.2	1.8	2.1*/20.0*	45	shear surface
CDS7	6.1	0.95	4.3	NE	N/A	N/A
CDS8	5.3	0.55	4.6	3.05/11.5	78	slickensides at ± 45°
CDS9	4.8	0.5	4.4	2.3/15.1 3.9/13.5	49 47	slickenslides slip surface at 30°‡
CDS10	8.5	0.5	3.7	1.9/18.5 4.7*/15.7* 6.5*/13.9*	73 50 39	slickenslides slickenslides slip plane

NE: Not Encountered
N/A: Not Applicable
*: Shear zones observed in weathered clay stratum, except where indicated otherwise by as asterisk
‡: Relative to the horizontal

TABLE 3

Comparison of CPT identified softened zones

and CDS detected shear surfaces

Test Location	Depth of CPT Sounding (m)	CPT Identified Softened Zones (m BGL)	CDS Detected Sheared Surface (m BGL)	Maximum Depth of CDS Hole (m)
3	10.3	N.E.D.	3.05	6.1
4	10.1	1.7 4.2	1.3 - 1.65	4.9
5	10.0	2.3	1.1	8.5
6	10.3	4.1	2.1	5.6
8	20.0	4.0 5.8	3.05	5.3
9	10.1	1.0	2.3 3.9	4.8
10	10.1	1.6 4.1	1.9 4.7 6.5	8.5

*N.E.D.: Not Easily Discernible

J:\TECH_PAPER\91-1321\TESTHOLE.DWG

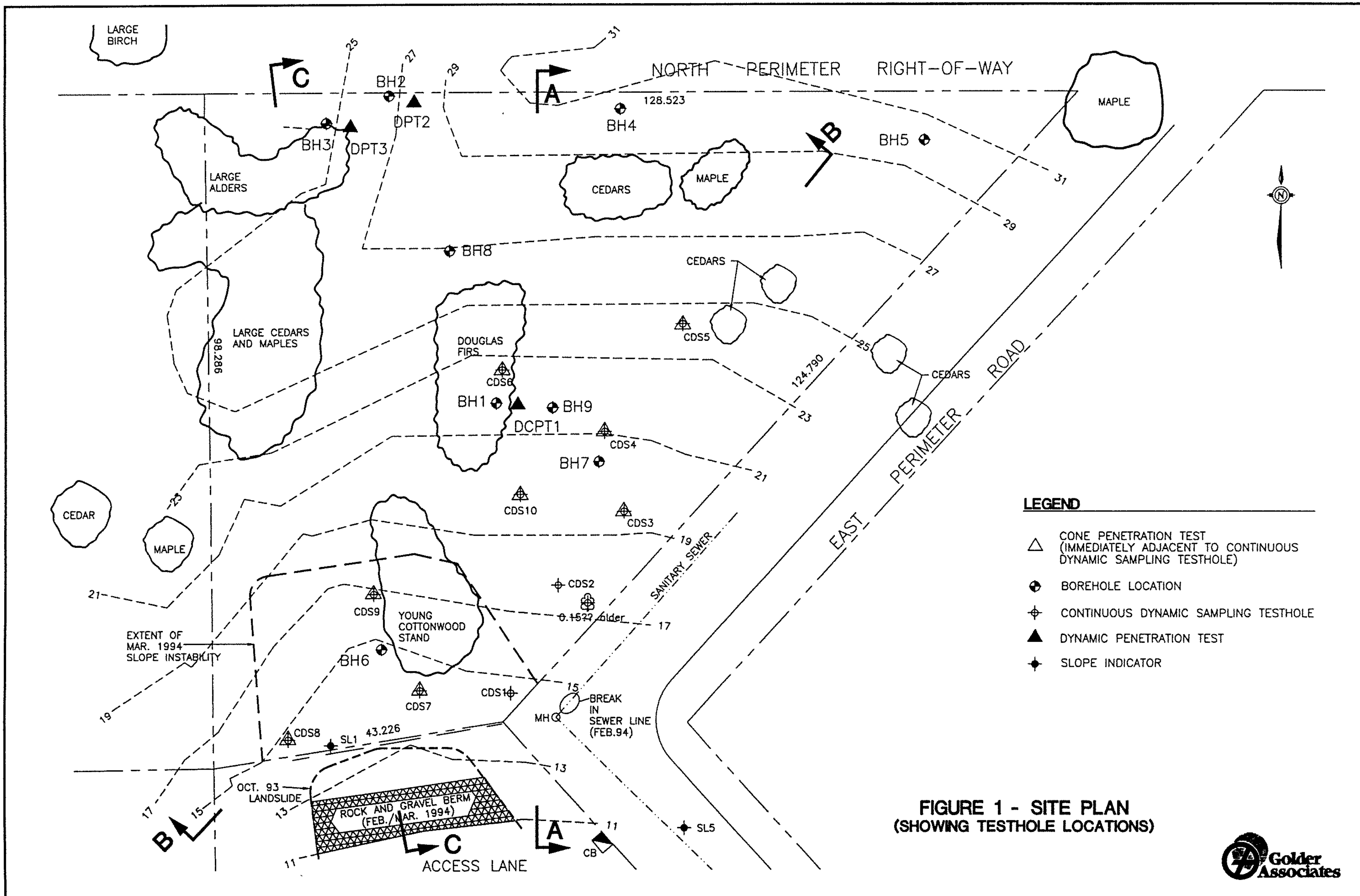

FIGURE 1 - SITE PLAN
(SHOWING TESTHOLE LOCATIONS)

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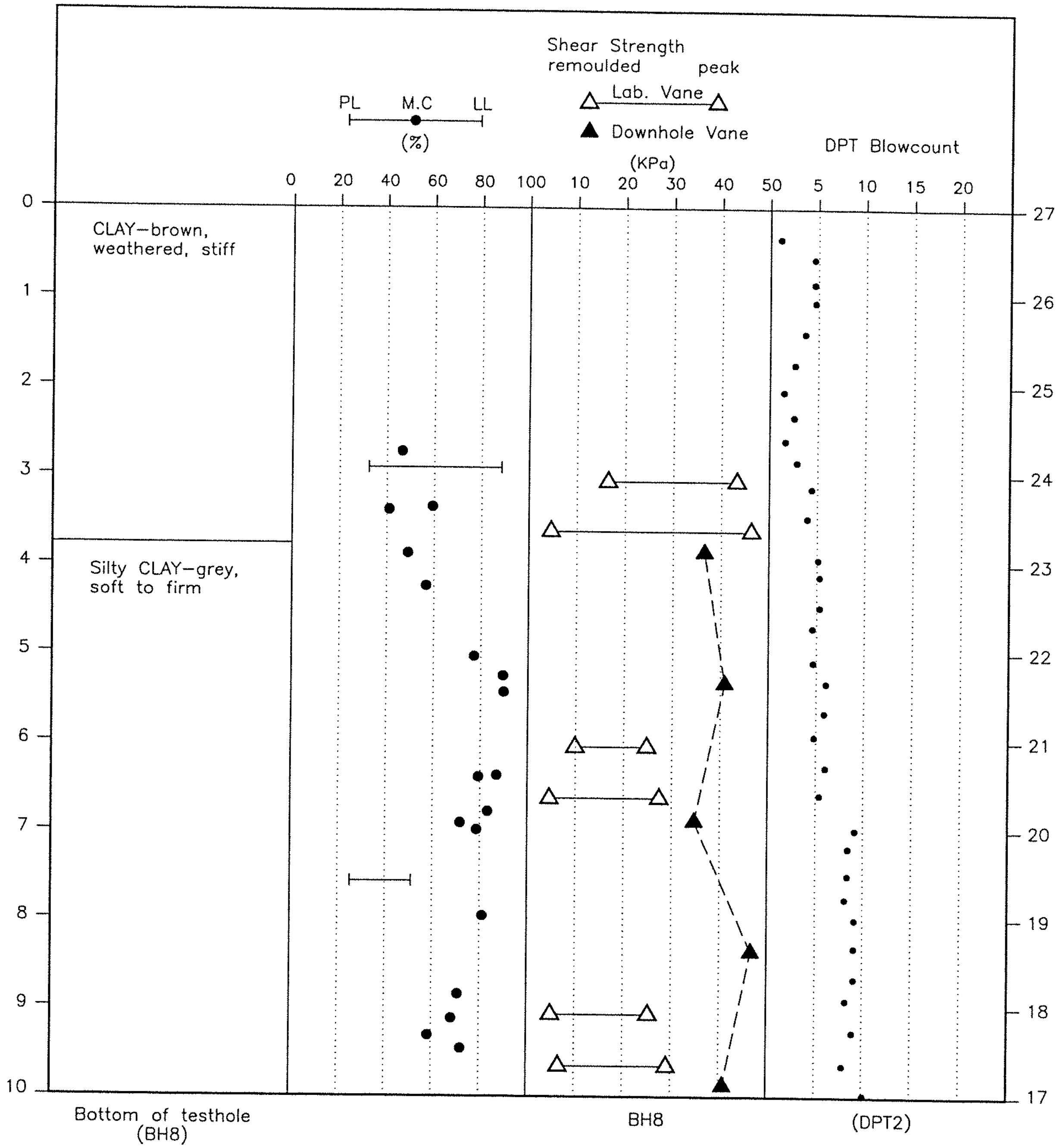

FIGURE 2 - TYPICAL BOREHOLE LOG TOGETHER WITH PROFILES FOR ATTERBERG LIMITS, WATER CONTENT DETERMINATIONS, FIELD AND LABORATORY VANE STRENGTHS, AND DPT BLOWCOUNTS

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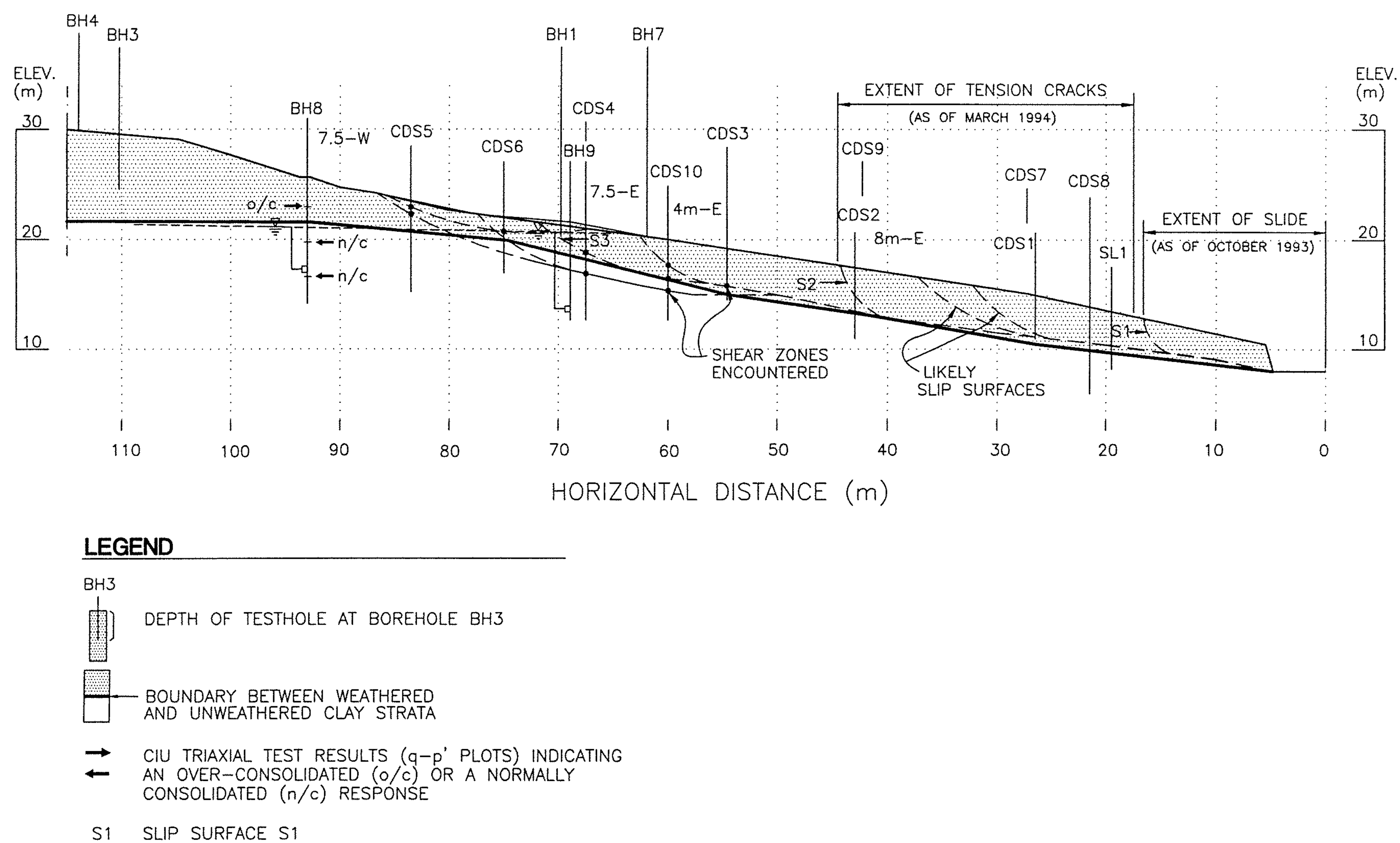

FIGURE 3 - CROSS SECTION A-A
(SHOWING CONTINUOUS SAMPLING PROFILES)

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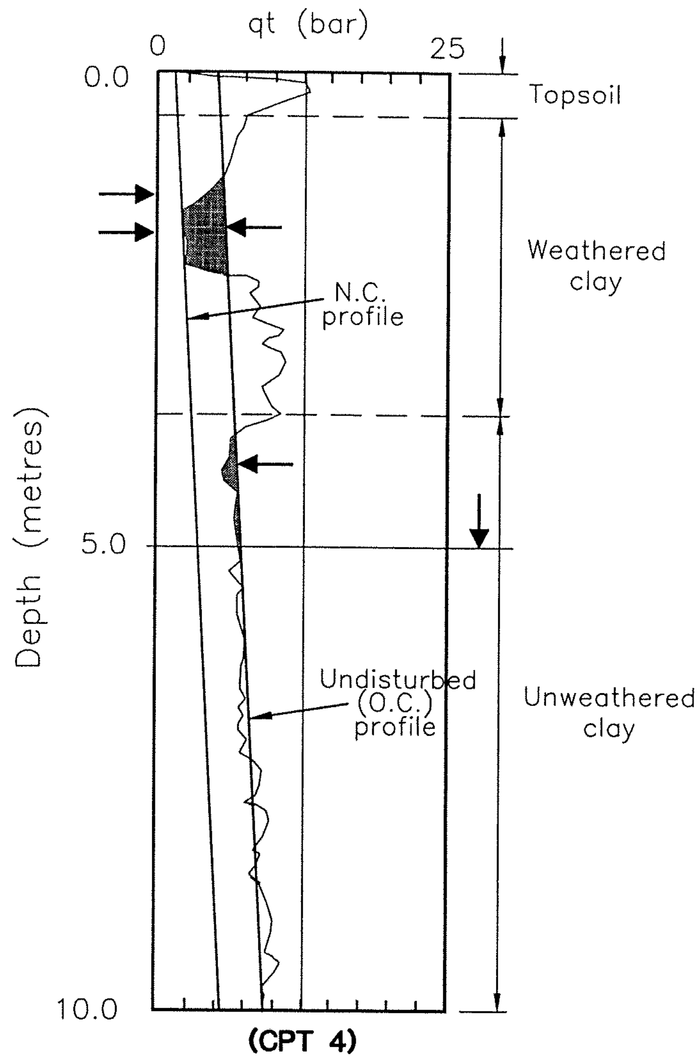

LEGEND

N.C. profile – Estimated qt for normally consolidated clay
Undisturbed (O.C.) profile – Estimated qt for undisturbed clay
▨ – Zone of softened clay
← – Softened zone shown in Figure 5
→ – Observed shear surfaces at adjacent CDS testhole
↓ – Maximum depth of adjacent CDS testhole

FIGURE 4 - IDENTIFICATION OF SOFTENED ZONES FROM CPT PROFILES AND COMPARISION WITH CDS IDENTIFIED SHEAR SURFACES

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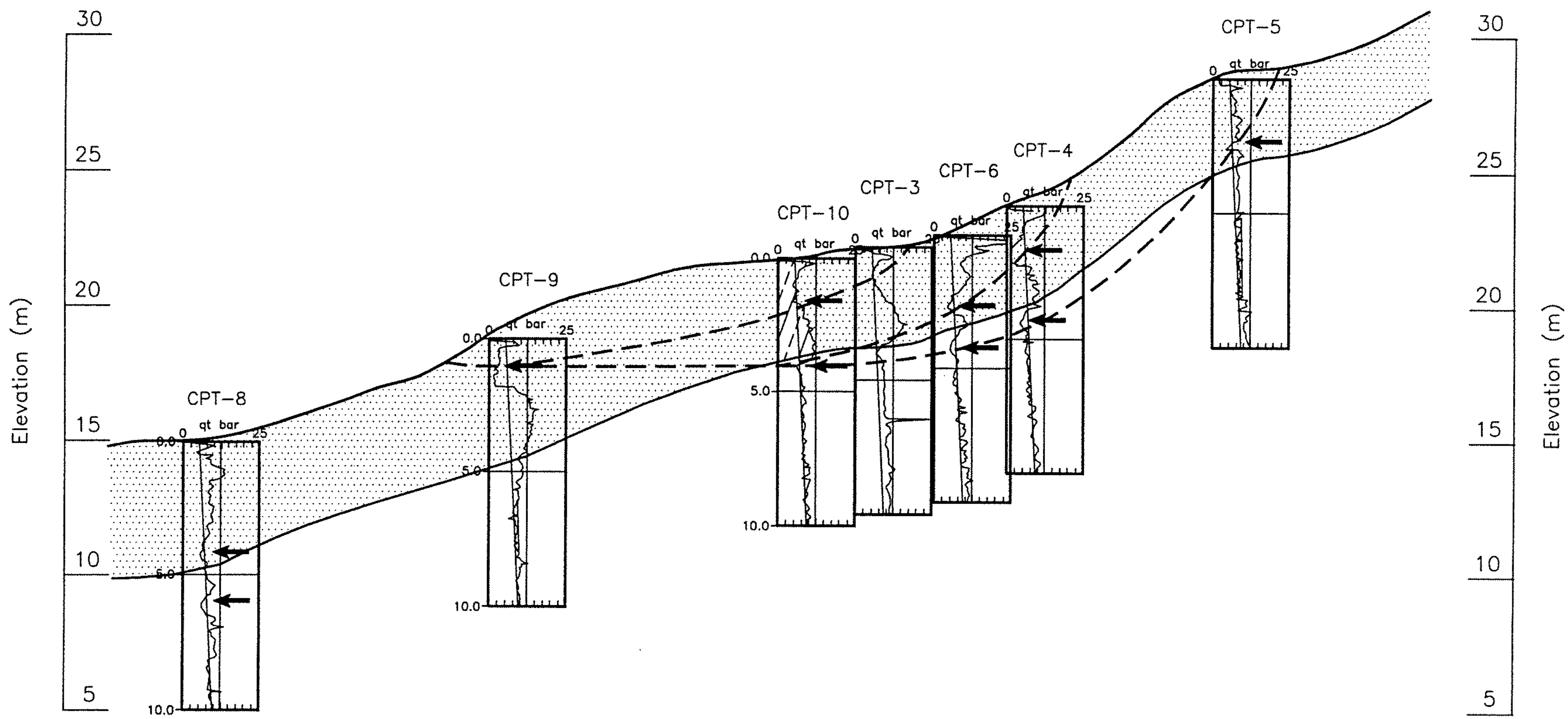

LEGEND

- SOFTENED ZONES IDENTIFIED
- BOUNDARY BETWEEN WEATHERED AND UNWEATHERED CLAY STRATA
- LIKELY SLIP SURFACES

FIGURE 5 - CROSS SECTION B-B
(SHOWING CPT PROFILES)

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J:\TECH_PAPER\91-1321\FIG1A.DWG

Year	Stage	Soil Parameter	F.O.S. along slip surface		
			S1	S2	S3
Summer 1993	Before excavation at slope toe	ϕ'_{cv} throughout slope	0.98	1.26	1.22
Fall 1993	Immediately after excavation at slope toe	ϕ'_r along S1, elsewhere ϕ'_{cv}	0.68	0.92	-
January 1994	After development of tension cracks in upslope areas as far as S2	ϕ'_r along S1 and S2; elsewhere ϕ'_{cv}	-	-	0.91
March 1994	After completion of slope stabilization works (extent of which is shown)	ϕ'_r throughout slope, ϕ' of 40° at toe berm	-	1.23	0.77

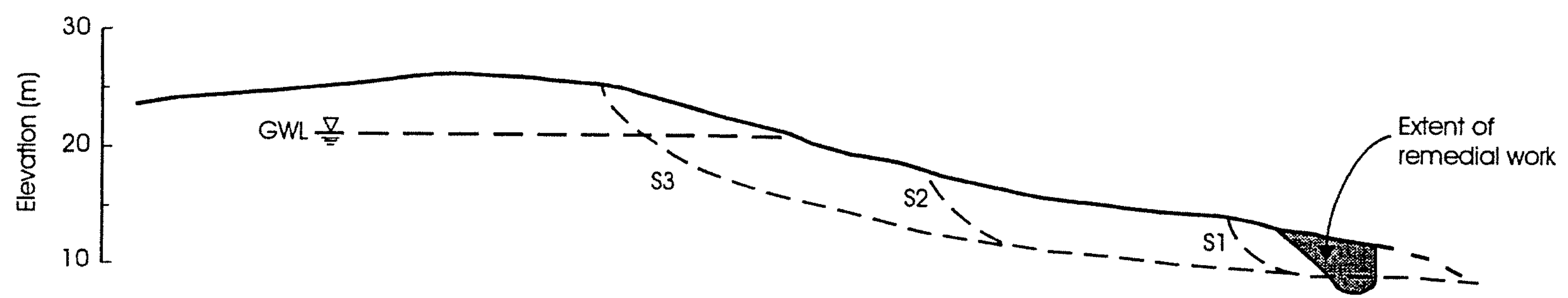

FIGURE A1 - RESULTS OF STABILITY ANALYSES ALONG SECTION C-C

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