CAST-IN-PLACE TIED-BACK PILE WALL INSTALLED TO RESIST COASTAL EROSION AND EARTHQUAKE FORCES, CORDOVA BAY ROAD, VICTORIA, B.C.

Stephen M. Bean [1] and Geoffrey F. Buck [2]

ABSTRACT

Slope regression has been an intermittent but continuing problem along the length of the Cordova Bay shoreline. Previous studies of the shoreline led to the placement of shore protection in some areas and the installation of a sewer system to reduce recharge of groundwater that was contributing to slope instability. This study consists of a geotechnical investigation, slope stability assessment and design and construction of a drainage system and a cast-in-place, tied-back pile wall for a short section of slope adjacent to Cordova Bay Road located in the Municipality of Saanich, B.C. The purpose of the wall construction is to protect the existing roadway against encroachment by landsliding. This paper will discuss the geological conditions encountered during the investigation, the results of the stability analysis, and the design and construction of the pile retaining wall.

INTRODUCTION

The site is located along Cordova Bay Road (see Figure 1) in the Municipality of Saanich (MoS). The existing Cordova Bay Road through Mt. Douglas Park was constructed in the 1930s to replace a former road which followed an alignment on the seaward side. The slope is underlain by the Quadra sediments (sometimes referred to as the Cordova Bay sediments in this area). These sediments consist of dense silt, sand and gravel which were deposited as glaciofluvial outwash. The slope is actively, though intermittently, regressing.

A length of the shoreline in the adjacent park has been protected from wave action at the toe of the slope by the placement of rip rap. No sea protection has been placed in the study area although this has been recommended and funds have been appropriated for the work. Thurber Consultants Ltd. (TCL) previously made an evaluation of the slope and estimated the future regression of the slope crest (at road level) for the condition that the toe was protected from sea erosion but without any changes to the slope. The long term projection was that the road would eventually be eroded (as it had been in the previous seaward location) and that it would have to be moved again. A preliminary cost projection to relocate the road was 2.6 million dollars with environmental impacts on the adjacent

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[1,2] Thurber Engineering Ltd. Suite 210, 4475 Viewmont Avenue, Victoria, B.C. V8Z 6L8

park that were of concern to the Municipality at large. The slope crest had regressed to within 3.5 m of the road pavement in a slide alcove where the most recent slump had occurred in 1990.

In December 1991, MoS conducted an auger drilling program along Cordova Bay Road. The investigation encountered groundwater at shallow depth and the potential affect on slope stability was recognized. Given the groundwater condition and the proximity of the slide crest to the road, the study reported in this paper was commissioned to assess the feasibility of protecting the road from slope regression.

SITE DESCRIPTION

The site is located along the shoreline of Cordova Bay about 8 km northeast of downtown Victoria. The shoreline consists of steep (30 to 40°) sandy slopes, about 40 to 50 m high, which are actively regressing due to wave action at the toe and landsliding above. Some small V-shaped gullies have formed on the slopes, many of which have rounded oversteepened alcoves (head scarps) located near the crest of the slope. Three alcoves (A, B and C) have been identified in the study area (see Figure 2). At the crest of the slope lies Cordova Bay Road which is a 2 lane arterial collector. Residential structures are located on the landward side of Cordova Bay Road.

INVESTIGATION

Surficial Geological Mapping

Surficial geological mapping was carried out to allow inspection of the geological conditions of the slopes, note areas of groundwater seepage, observe areas of recent slope instability and assess erosion characteristics of the slopes. The observed geological conditions were used to assist in choosing geotechnical parameters for the stability analysis. The surficial mapping was referenced to survey points installed by the MoS surveyors and the results are shown on Figure 2.

Subsurface Investigation

A total of 6 test holes were drilled at the site using hollow stem auger and a Canterra 250 drill rig. Three slope indicators and three standpipe piezometers were installed in the test holes. Four test pits were excavated on the upslope side of Cordova Bay Road to inspect the surficial soils and groundwater conditions. The locations of the test holes and test pits are shown on Figure 2.

Piezometers installed in the lower sand deposits were found to be dry or recorded low groundwater levels. Slope Indicator 92-1 and the three standpipe piezometers were destroyed during construction of the wall and the drainage measures. Slope Indicator 92-2 has been read several times since installation and has shown no significant movement.

GEOLOGY

Geologic History

The Cowichan Head, Gordon Head and Cordova Bay Areas are underlain by the Quadra sediments (sometimes referred to as the Cordova Bay sediments). These sediments consist of dense silts, sands and gravels deposited as glaciofluvial outwash in front of an ice tongue which advanced down the Cowichan Valley between 27,000 and 37,000 years ago. Following deposition, the Quadra sediments were overridden by several thousand feet of ice during the Vashon Stage of the Fraser Glaciation. During this time the sediments were selectively eroded, moved and compacted by the weight and force of the moving ice. A cap of Vashon till was also deposited on top of the Quadra sediments.

By about 9,000 years ago, land in the Saanich Peninsula area had rebounded so that sea level was about 8 to 12 m lower than present. Subsequently, the sea level rose to its current level which has remained unchanged for the last several thousand years. This rather constant sea level has resulted in extensive erosion of shoreline areas. Toe erosion, which occurs during periods of high energy storm wave action, coupled with high tides, controls the long term slope geometry.

Site Geology

Generally, the site stratigraphy encountered during the investigation consists of the following (from road level downwards):

- loose organic silty sands and gravelly sands (0.5 to 2.5 m thick)
- very dense upper glacial till (Vashon Till) (2 to 8 m thick)
- very dense sand and gravelly sand (20 to 25 m thick)
- very dense silt and silty sand (8 to 12 m thick)
- lower glacial till (lower 1 to 3 m of slope)

Of these layers only the upper three were penetrated during the drilling investigation. The lower two units were observed during the surficial mapping. Figures 3 and 4 are cross sections D-D and A-A through the slope showing the stratigraphy. Figure 2 shows the locations of the sections.

The lower glacial till is exposed along the toe of the slope as a near vertical face about 1 to 3 metres high. The till is very dense, relatively erosion resistant, has low permeability and contains some large boulders. As these boulders are eroded out from the till they help to protect the toe of the slope from wave action. Seepage was observed coming from the top of the till and at the base of the overlying silts and silty sands.

The very dense silt and silty sands overlie the lower till. They are more easily eroded than the till, however cohesion within the silts enables the unit to stand at steep angles of about 60° in some locations. This unit varies in permeability characteristics from low in the pure silt layers to quite high in the silty sand layers. The silt is more likely to fail as slabs or slump blocks rather than as saturated flows. Some areas of seepage and active sloughing and slumping within the silt were observed and are shown on Figure 2.

The very dense sands and gravelly sands comprise the middle section of the soil slope. These sands were encountered during the drilling investigation and samples of the sand collected during drilling yielded natural moisture contents ranging from about 9 to 13%. Standard Penetration Test blow count values taken in the sand were above 70 blows for 300 mm indicating that the sand is in a very dense state. The sand, which is fine to medium grained, was found to vary from clean to silty, and was often gravelly. It is also quite erodible since it has little or no cohesion. The permeability of the sand is considered to vary with gradation and silt content. The silty, less permeable layers within the deposit would tend to support perched aquifers. However, the slope surface of the sand deposit shows little visible seepage except at the lower contact with the underlying silts.

The upper till (Vashon Till) forms a cap on top of the slope. The till is variable in composition ranging from a sandy clay to a gravelly sand. Natural moisture contents taken from samples of the till ranged from about 9 to 18%. Standard Penetration Test blow count values were also above 70 indicating that the till is in a very dense state. The upper till stands at a relatively steep slope angle due to some cohesion and its very dense nature. Although the till will vary somewhat in permeability characteristics it is likely that it acts as a groundwater barrier and prevents much of the rainfall from percolating directly into the underlying permeable sands. The till varies in thickness from about 2 m to about 8.0 m in the study area. Test pits excavated into the till encountered seepage coming from the upper portion of the till and the overlying silts and sands. Although no visible seepage from the till slope face was observed, photographs taken during the winter of 1990 after a small slope failure within the till had occurred, clearly showed ice and seepage occurring about 2 m below the crest of the slope.

Overlying the upper till is a thin layer of organic silty sands and gravelly sands (upper sands). This unit is distinctively reddish brown in colour and contains organic debris and topsoil. These loose to compact silty sands and gravelly sands are relatively permeable and were essentially saturated during the investigation which was conducted during heavy rainfall. The four test pits excavated through this layer and into the underlying till all encountered seepage coming from the upper sands and the uppermost portion of the till layer.

Bedrock is exposed above the houses on the south side of Cordova Bay Road and just above the ditch below the motel. The bedrock surface is irregular and drops off steeply. Except for an outcrop near the stream at the south perimeter of the site, no bedrock exposures were observed on the slope and the exact profile of the bedrock surface is generally not known.

Hydrogeology

The regional groundwater table below which all material is permanently saturated lies within the silt layer near the base of the slope. This is based on seepage conditions observed during the field mapping and the standpipe piezometer tip located in the silt layer. However, perched water conditions do exist within the sand, upper till and surficial sand layers. Within the uplands zone of the study area the thin permeable organic sands overlie a relatively low permeability glacial till layer. Surface water and groundwater percolating through the upper sands is captured in the shallow ditch and is carried away from the slope to the creek situated just east of the study area. However, it is believed that during heavy rainfall events the ditch and culvert system became overloaded leading to saturation and overland flow across the road and onto the slope. This causes substantial surface erosion and results in undermining of tree root systems and small earth flows of saturated soil and debris.

SLOPE REGRESSION

The shoreline (toe of steep bank) and the upper slope scarp (top of bank) were surveyed in February 1992 and are shown on Figure 2. Figure 3 is a section (D-D) through the steepest part of the slope in the study area that has been fairly stable over the longer term as indicated by the mature fir found there. The slope crest is located about 25 m from the road pavement at Section D-D. More active regression has occurred at the alcoves found at sections A, B and C as shown on Figure 2. At these locations the crest of the slope is within 3.5 m, 5 m and 8 m of the road for these three alcoves respectively. Information suggests that Alcove A has regressed 2.4 m in the last 20 years and about 6.5 m in the last 38 years. During the last major event in November, 1990 the crest broke back a further 0.6 m to 0.9 m. Alcoves B and C appear to be less active than Alcove A but do show signs of soil creep. It was decided to protect the road by construction of the wall reported in this paper at Alcove A, with the option of extending the wall to the other alcove(s) in the future if required.

A review of airphotos suggests that there was a major activation of sliding north-east of Alcove A between 1946 and 1954. It is of interest to note that increased slumping of the slopes in Gordon Head was observed in the period from 1967 to 1972. The 19 year cycle of tides reached a high in 1972 with the subsequent high in 1991. The high tide varies by 0.14 m through the cycle and may be a contributing factor to the slope activation at that time.

The slope regression is the result of a combination of toe erosion due to wave action and upper slope erosion due to groundwater seepage and surface runoff as shown schematically on Figure 5.

SLOPE STABILITY ASSESSMENT

Geological Model

The slope geometry was provided by the MoS surveying, and the geological unit boundaries were selected based on the surficial field mapping and drilling results. A total of 5 different geological units were selected for the analysis. They included the lower till, silt or silty sand, sand and gravelly sand, upper till and surficial sand. The strength parameters were selected by first estimating them from the drilling results and past experience and then modifying them based on a back analysis. A back analysis was conducted on Section D-D shown on Figure 3 (the steepest slope within the study area). It was assumed that this slope was at limit equilibrium and a stability analysis was repeated with the input of modified strength parameters until a factor of safety of one was obtained. The rather complex groundwater conditions at the site were modelled using r_u values (ratio of pore water pressure to total overburden pressure) for each geological layer to simulate perched water conditions during periods of wet weather.

Stability Analysis

The slope stability analyses were conducted using the microcomputer software program CLARA. A 2 dimensional limit equilibrium analysis assuming an infinite slope was adopted. For each of the four chosen sections the slope geometry, geotechnical strength parameters and groundwater conditions shown on Figures 3 and 4 were input into the program for analysis.

Slope stability analyses were conducted on Sections A-A, B-B and C-C through the slope at Alcoves A, B and C respectively. The soil strength and pore water parameters were obtained from the back analysis at Section D-D described above. The geological model used for section A-A is shown on Figure 4.

Results of Analyses

Using the estimated parameters obtained from the back analysis of Section D-D the calculated factors of safety (FS) at Sections A-A, B-B and C-C were 1.36, 1.13 and 1.18 respectively, using relatively dry slope conditions. For Section A-A the FS is considered adequate for the minimum stability surface that extends from the toe to the crest of the slope within the roadway. For Section B-B the lowest factor of safety was found for a shallow surface through the lower half of the slope. At Section C-C the minimum stability surface was found to extend full height to intersect the ground surface seaward of the road.

By field observation, Alcove A is considered the most active and the most critical with respect to road safety. Because this gully is the most deeply incised into the slope, it is expected to be a drainage point for groundwater from the adjacent slope and have higher pore water pressures closer to the surface. On Section A-A, this greater presence of groundwater would apply in the upper slope where the gully is located. An analysis was done for the condition of saturation of the upper till unit, an r_u of 0.5. This assumption had very little impact on overall deep seated stability as the factor of safety dropped only to 1.34. However, a factor of safety of below 1.0 was calculated for the upper slope that would result in a shallow rotational failure in the upper till unit near the crest of the slope (see Figure 4). By further analysis it was found that the upper till would be at the point of shallow rotational failure for an r_u value of 0.33.

The results of the slope stability analyses for the geotechnical parameters shown on Figures 3 and 4 are provided in Table 1.

TABLE 1. Results of Slope Stability Analyses

Section	Failure Mechanism	r_u of Upper Till	Factor of Safety
A-A	Deep Seated - Full Height	0.1	1.36
B-B	Shallow - Lower Slope	N/A	1.13
C-C	Deep Seated - Full Height	0.1	1.18
D-D	Deep Seated - Full Height	0.1	1.0 (Back Analysis)
A-A	Deep Seated - Full Height	0.1	1.34
A-A	Shallow - Upper Slope	0.33	0.99

The slope stability would be further reduced during a seismic event, however the Willmar Bluffs at Comox, B.C. (which are of similar configuration and composition) did not suffer any significant failures during the major 1946 earthquake. As is later described, the wall was designed to resist collapse under the design earthquake.

The results of the analysis show that the lowest factors of safety with respect to slope failure are obtained for shallow potential slide surfaces that could extend to the existing roadway under wet conditions. The calculated factor of safety with respect to a large full height slope failure is considered adequate for a roadway at the existing location. This finding is in agreement with field observations and reports that the past sliding has been shallow and that the reported extensive regression of the crest of the slope has resulted from a series of shallow regressive failures over time. These failures are on-going, and in particular shallow slope failures are predicted within the gullies for the condition of higher perched groundwater in the upper soils.

DESIGN OF REMEDIAL MEASURES

Pre-Construction Conditions

The slope failure mechanism is retrogressive starting with sea erosion at the toe and progressing up slope due to oversteepening and seepage at various levels as shown schematically on Figure 5. In the past the average rate of overall slope regression has been slow as indicated by the legal survey records referred to previously and the mature fir found on the slope. This average rate can be expected to continue with periods of more rapid regression following major storms together with high tides and heavy precipitation.

Three gullies have developed on the slope in the study area. These indentations in the slope profile tend to cause a concentration of surface runoff and seepage which, together with the local deposits of stratified sand and silty sand, cause a greater rate of regression. If ignored it was expected that the scarps of the gullies would eventually cut into the roadway. Alcove A was considered to present the greatest hazard to the roadway in the study area and it was recommended that drainage works together with remedial stabilizing measures be constructed or the road be relocated.

Discussion of Stabilization Options

After reviewing the road relocation option, MoS informed us that they desired to maintain the road at its present location and proceed with the design of stabilizing measures at Alcove A. It was recognized that for a moderate to long term solution to this problem the following objectives needed to be met:

- erosion at the toe of the slope by wave action would have to be controlled by constructing some form of revetment
- drainage above and on the slope would have to be improved
- a retaining structure should be built at the crest of the slope to retain the road as the crest continues to regress

Some drainage work consisting of drainage ditches and perforated subdrains was designed and installed at the same time as the wall. The toe protection has not been installed yet and continues to be the subject of debate. The remainder of this paper describes the retaining wall design and construction.

The conceptual design consisted of the following possible retaining structures:

- conventional cast-in-place concrete wall
- geogrid reinforced modular block wall
- sheet pile tie-back wall
- shotcrete and soil nailing
- tied-back tangent pile wall

The actual slope is owned by private property owners and MoS desired a wall solution which could be constructed within the road right-of-way and without trespassing on private property. This requirement basically eliminated construction of standard retaining walls since these would require significant excavation at the crest of the slope and an access road to the base of the wall through private property. It was desirable to construct a wall which could be installed from the road surface such as a sheet pile wall or a pile wall. The sheet pile wall option was eliminated due to the anticipated difficult driving conditions in the dense upper till. It was also considered that a sheet pile wall may act as a seepage barrier. A tied-back pile wall was therefore selected for the design.

Design of Tied-Back Pile Wall

The following conditions were assumed for the design of the retaining wall:

- wall would be located on road right-of-way near property line
- toe of slope will be protected to prevent further toe regression
- upper portion of the slope will continue to regress back to a long term slope angle of 27.5° resulting in a required wall height of 8.0 m
- drainage work constructed concurrently with retaining wall will limit groundwater forces acting on wall to perched saturation condition with head of 4.5 m
- under seismic conditions (0.3 g) the wall would be permitted to yield but not fail catastrophically
- there is a low probability that the design seismic event would occur during saturated perched water conditions

Free-Earth Support Method

The wall was designed using conventional design assumptions and procedures for anchored sheet pile walls. The "free-earth" support method was used due to the ease of calculation and the relatively rigid nature of the pile. The free-earth method assumes that the piling is rigid and may rotate about the anchor point with failure occurring by rotation about the fixed anchor rod. It is known that this method yields significantly greater bending moments than the "fixed-earth" method and is therefore conservative. A latpile analysis was also completed and the results are discussed.

Figure 6 shows the soil parameters used in the analysis and the earth pressure distribution on both sides of the wall. The projected break line (27.5°) is a conservative estimate of the long term stable slope angle based on observations of adjacent slopes. The perched water conditions chosen for the static analysis were based on the following assumptions:

- installed subdrains will drain the upper 3 m
- the upper till layer may become saturated during very wet conditions
- the pile wall may be shotcreted in the future and act as a seepage cut-off
- the dense sand will have perched water layers with a maximum head of 4.5 m (based on nearby piezometer results)

Seismic earth pressures were calculated using the Mononobe-Okabe solution with a seismic acceleration of 0.3 g.

Figures 7 and 8 show the net earth pressure diagram (active pressure - passive pressure) and the bending moment diagram for the free-earth method. The maximum calculated bending moments were about 860 kN-m for the seismic case and about 590 kN-m for the static case. The bending moments were not reduced by Rowe's moment reduction method due to the relatively high friction angle values used in the analysis (42°), the very rigid nature of the pile wall and the sloping ground in the passive zone. The bending moment and net earth pressure diagrams were provided to the structural engineer (KPA Engineering Ltd.) who designed the structural components of the wall. A summary of the results of the free-earth method of analysis is shown in Table 2.

TABLE 2. Results of Free-Earth Analysis

Case	Maximum Bending Moment (kN-m)	Required Deadman Anchor Force (kN/m)	Required Pile Length (m)
Static	590	160	14.5
Seismic	860	215	16.0

Figure 9 shows the pile wall layout. The wall consists of 24 - 760 mm diameter cast-in-place concrete piles spaced at 1.25 m c/c. It is considered that the dense soil will likely bridge between the piles, however if material sloughs out from between the piles the face of the wall can be shotcreted. Figure 10 shows a cross section through the slope showing the piles, tie-backs and deadman anchor. Figure 11 shows the details of the structural design of the piles.

The deadman anchor was designed to develop a resistance of 160 kN/m for the static case and 215 kN/m for the seismic case. The deadman consisted of a cast-in-place concrete block 30 m long, 1.75 m high and 0.6 m thick with the base of the anchor lying at least 2.5 m below the final ground surface. A total of 9 - 57 M threadbar, double corrosion protected, tie-back anchors stressed to 130 kN were used for the deadman anchor.

Lateral Pile Analysis

The wall was also modelled using a lateral pile analysis. This was accomplished with the software program COM624/G which is a finite difference program designed to analyze the stress and deflection of laterally loaded piles. A number of assumptions were used in the analysis:

- pile has a modulus of 27,400 MPa and a Moment of Inertia of 0.007 m^4
- distributed active load of static earth pressure and perched water conditions shown in Figure 7 was applied to the pile
- sloping ground in front of pile was modelled with an effective friction angle to match k_p for sloping ground (effective $\varphi=18^{\circ}$)
- P-Y curves were calculated using Reese et.al.'s (1974) criteria for sand
- calculated P-Y curves were modified by P-Y curve multipliers as derived by Brown and Shie 1991 for group effects (P-mult = 0.8, Y-mult = 1.4)
- k for the dense sand is 61,000 kN/m^3
- tie-back anchor load was modelled with a negative lateral load at top of pile

The program was run for a free headed pile with varying anchor loads. An anchor load of about 100 kN/m corresponded to the anticipated pile deflection at the head due to stretching of the tie-back anchors of about 5 to 10 mm. Figures 12 and 13 show the results of the pile deflection and bending moments for an anchor load of 100 kN/m. The maximum bending moment obtained from the latpile analysis was 460 kN-m.

Discussion of Results

The results of the lateral pile analysis indicate a required anchor force and design bending moment about 20 to 30% less than those obtained from the free-earth method of analysis (see Figure 13). This appears reasonable since it is known that the "free-earth" method overestimates the bending moments and assumes that the pile is rigid and develops only a positive moment. The results of the latpile method are similar to the "fixed-earth" method in that a point of counterflexure is developed below the dredge line leading to both positive and negative bending moments. The difference in bending moments between the two methods can be partially attributed to the different anchor forces required. In the free-earth method the anchor force is derived by balancing the horizontal forces once the pile embedment depth has been determined by moment summation about the anchor point. In the latpile analysis the anchor force was determined by simulating the anticipated deflection due to stretching of the tie-back anchors at the top of the pile.

If Rowe's moment reduction parameters are applied to the results of the free-earth analysis the maximum bending moment is reduced from 590 kN-m to about 415 kN-m. This value corresponds well with the 460 kN-m obtained from the latpile analysis considering the completely different method of analysis and the inherent assumptions.

CONSTRUCTION

The project was tendered for construction in September 1992 and awarded to Chew Construction Ltd. of Victoria, B.C. The piles were installed by Pacific Piledriving Ltd. using an auger piling rig with uncased holes. Some boulders were encountered in the till and had to be removed by blasting. The long rebar cages were picked up with a crane and carefully lowered into the hole prior to pouring the concrete. Only minor seepage was encountered during piling. The deadman anchor was poured neat against the native soil and the tie-back anchors were stressed to 130 kN. The construction took about 7 weeks to complete for a cost of about $380,000.

CONCLUSIONS

The "free-earth" method of retaining wall design yielded slightly higher values of required anchor force and bending moment compared to the latpile analysis. It was shown that a latpile analysis can be used to design pile retaining walls with similar deflection and bending moment diagrams to those obtained from the "fixed-earth" method and with results comparable to those obtained from the "free-earth" method.

The design concept of toe protection, upslope drainage and piled retaining wall construction will provide a medium to long term solution for stabilizing a section of Cordova Bay Road. The construction of the wall and the drainage measures has been successfully completed and removed the need for a costly road relocation. The remedial measures have performed well since construction with an observed reduced rate of sloughing thought to be due to the upslope drainage measures. The toe protection must be completed for the long term stability of the wall. Continued observations of the slope are required to assess the performance of the wall as the height increases over time.

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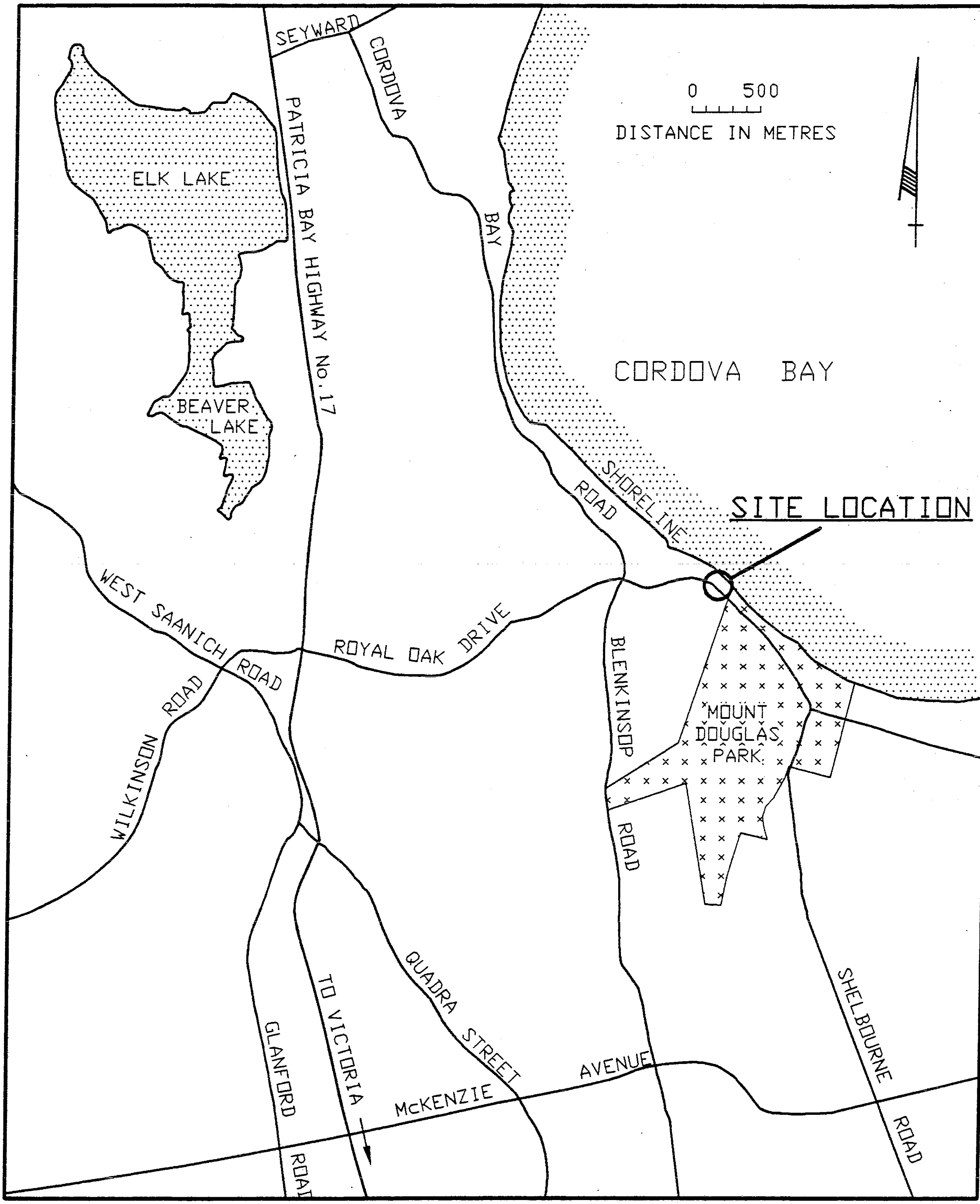
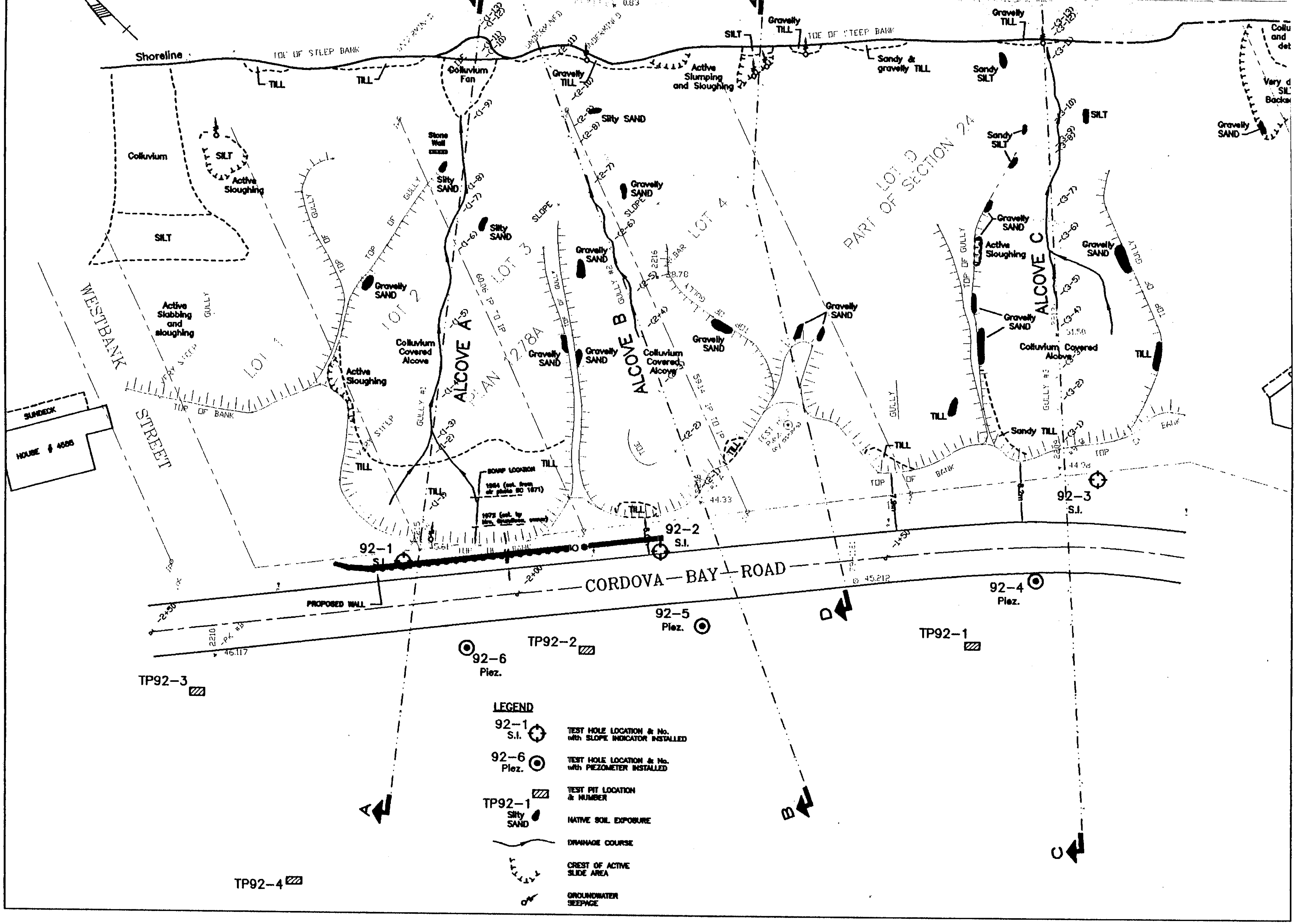


Figure 1. Location Plan



Shoreline
TOE OF STEEP BANK
TILL
Colluvium Fan
Gravelly TILL
Active Slumping and Sloughing
SILT
Sandy & gravelly TILL
Sandy SILT
Colluvium
SILT
Active Sloughing
Stone Wall
Silty SAND
Gravelly SAND
Active Slabbing and sloughing
WESTBANK STREET
LOT 1
LOT 2
LOT 3
LOT 4
LOT D PART OF SECTION 24
PLAN 1278A
ALCOVE A
ALCOVE B
ALCOVE C
Colluvium Covered Alcove
TOP OF BANK
TOP OF GULLY
GULLY #1
GULLY #2
GULLY #3
SLOPE
SUNDECK
HOUSE # 4655
SCARP LOCATION
Sandy TILL
CORDOVA—BAY—ROAD
PROPOSED WALL
92-1 S.I.
92-2 S.I.
92-3 S.I.
92-4 Piez.
92-5 Piez.
92-6 Piez.
TP92-1
TP92-2
TP92-3
TP92-4
LEGEND
92-1 S.I. TEST HOLE LOCATION & No. with SLOPE INDICATOR INSTALLED
92-6 Piez. TEST HOLE LOCATION & No. with PIEZOMETER INSTALLED
TP92-1 TEST PIT LOCATION & NUMBER
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DRAINAGE COURSE
CREST OF ACTIVE SLIDE AREA
GROUNDWATER SEEPAGE
A
B
C
D

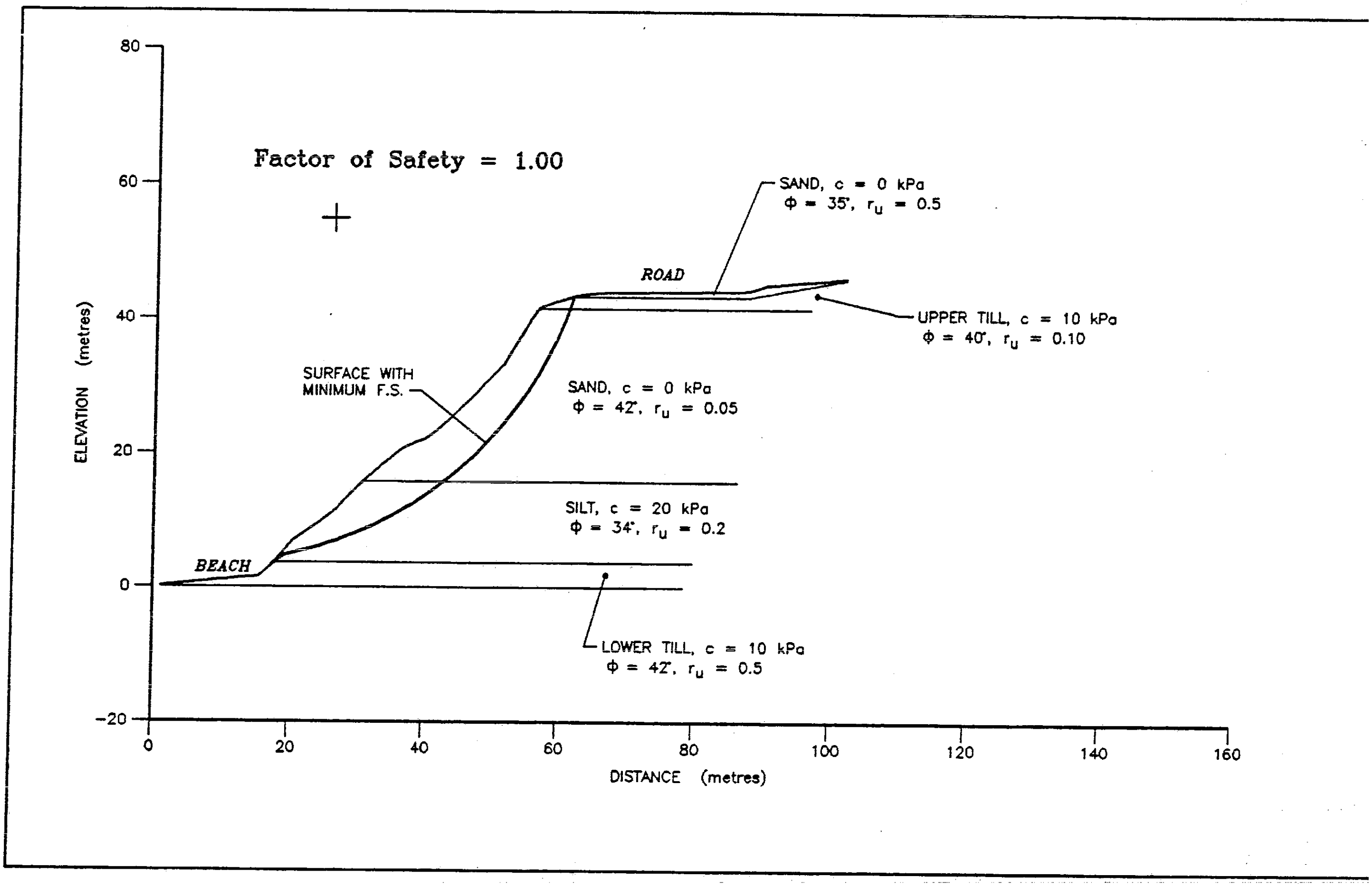


Figure 3. Section D-D Geotechnical Parameters

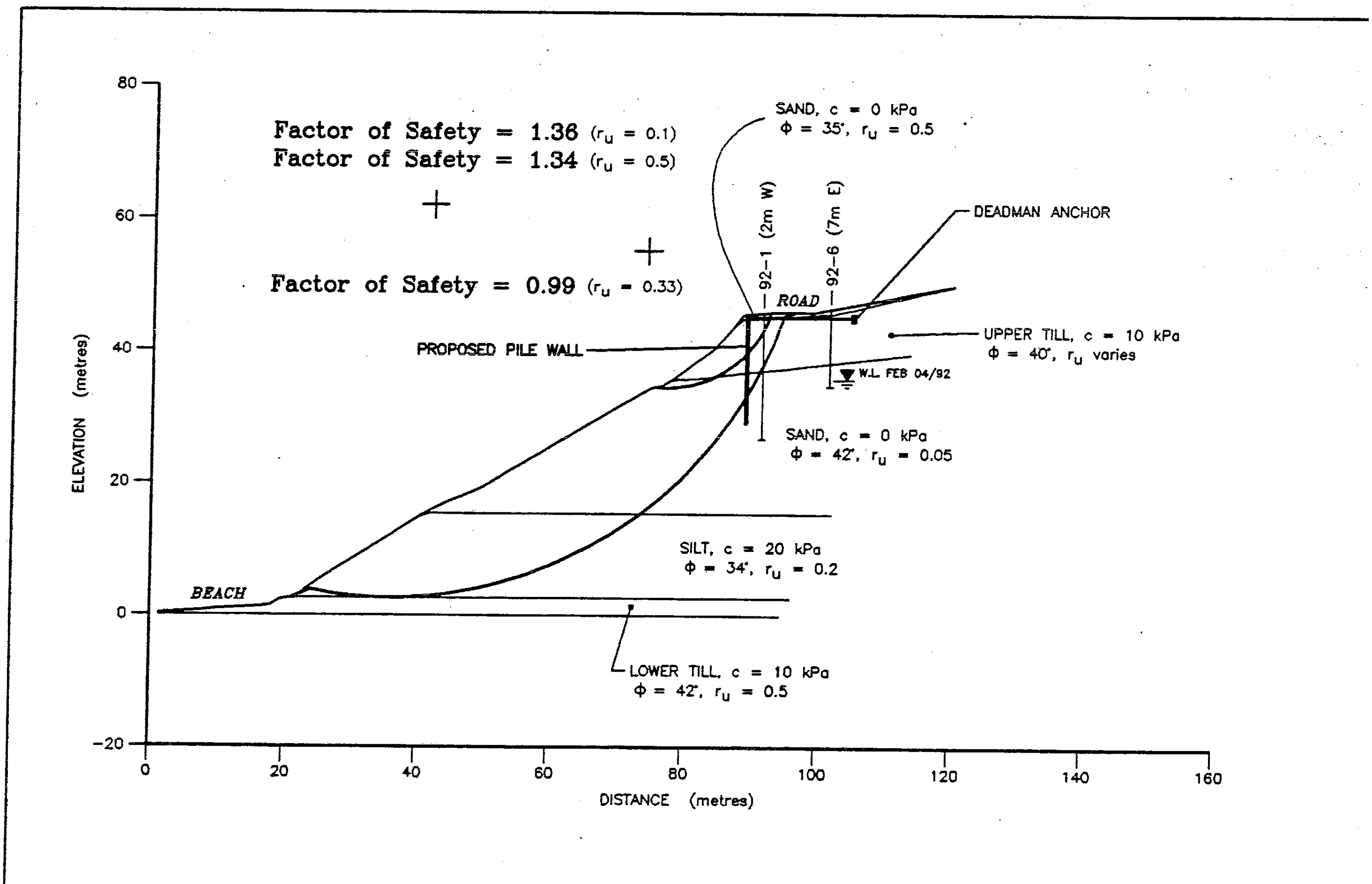


Figure 4. Section A-A Geotechnical Parameters

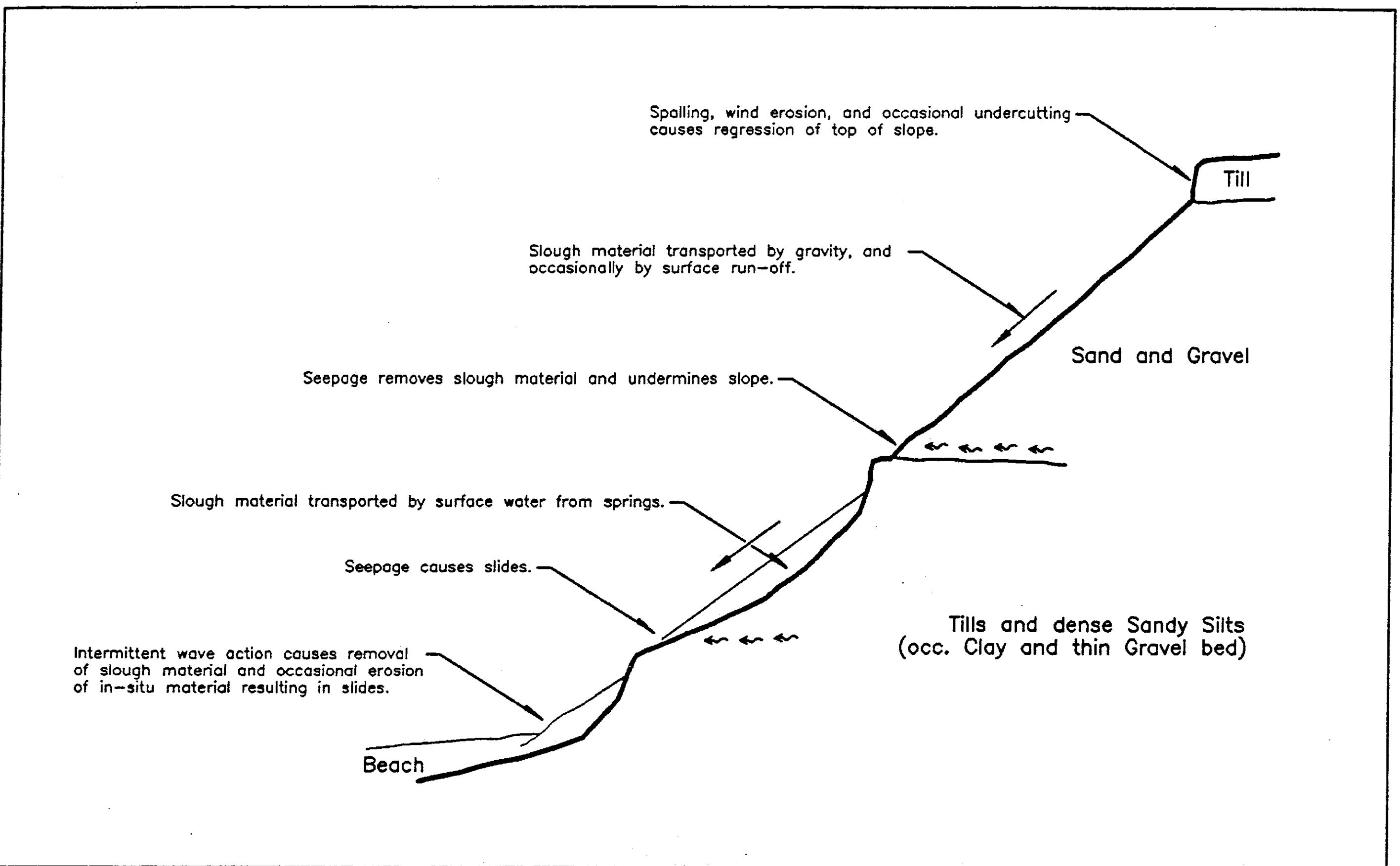


Figure 5. Schematic Section of Slope Regression Mechanisms

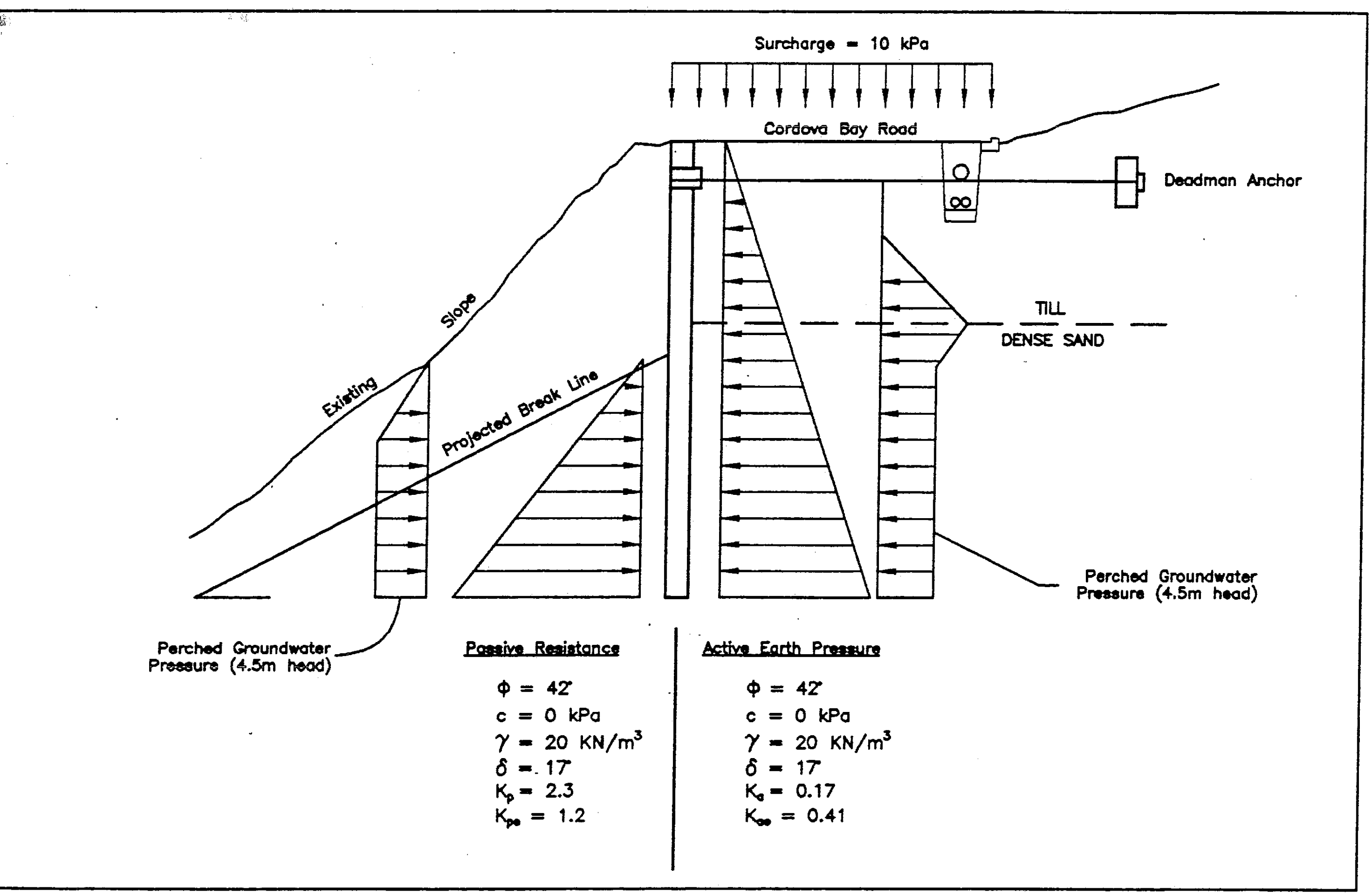


Figure 6. Design Earth Pressure Distribution

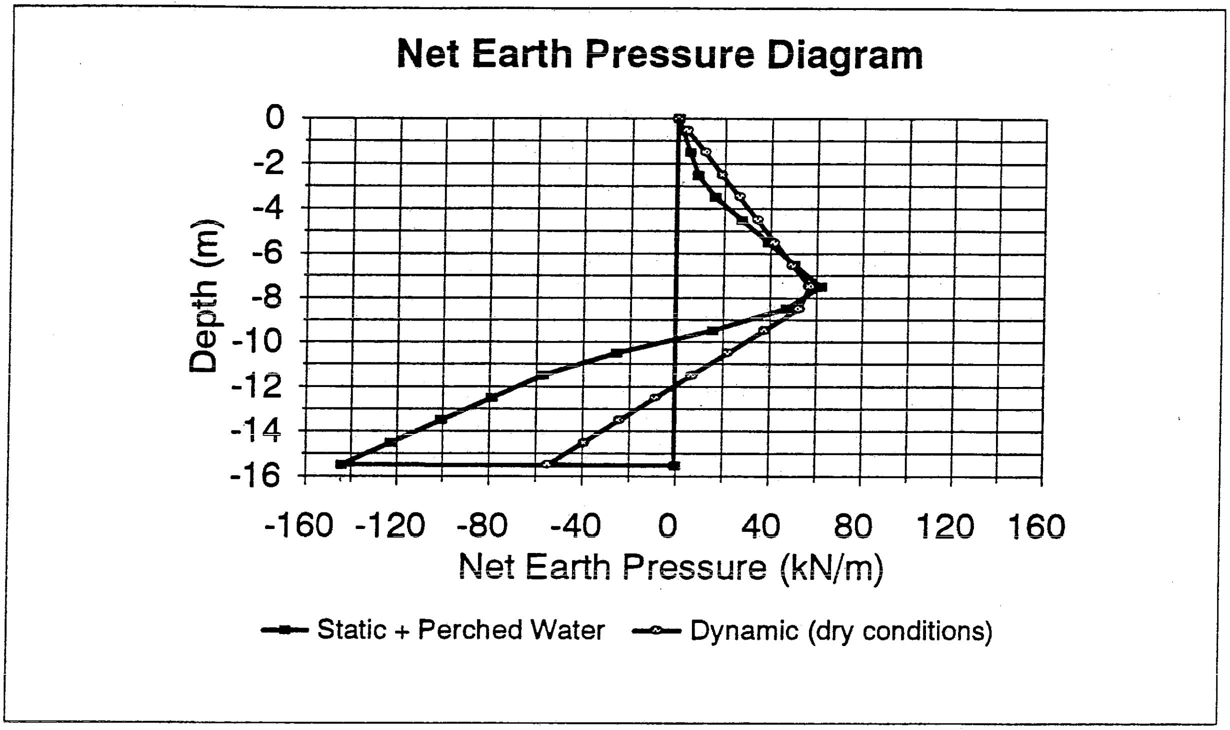


Figure 7. Net Earth Pressure Diagram

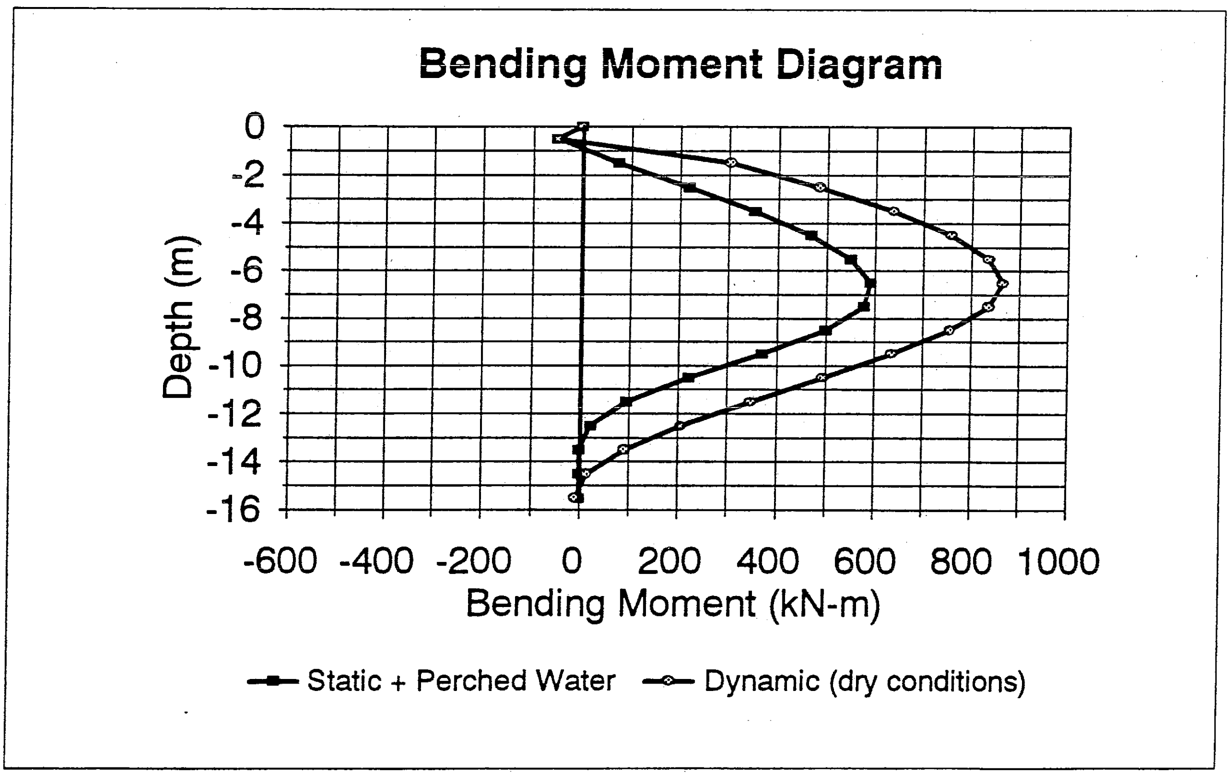
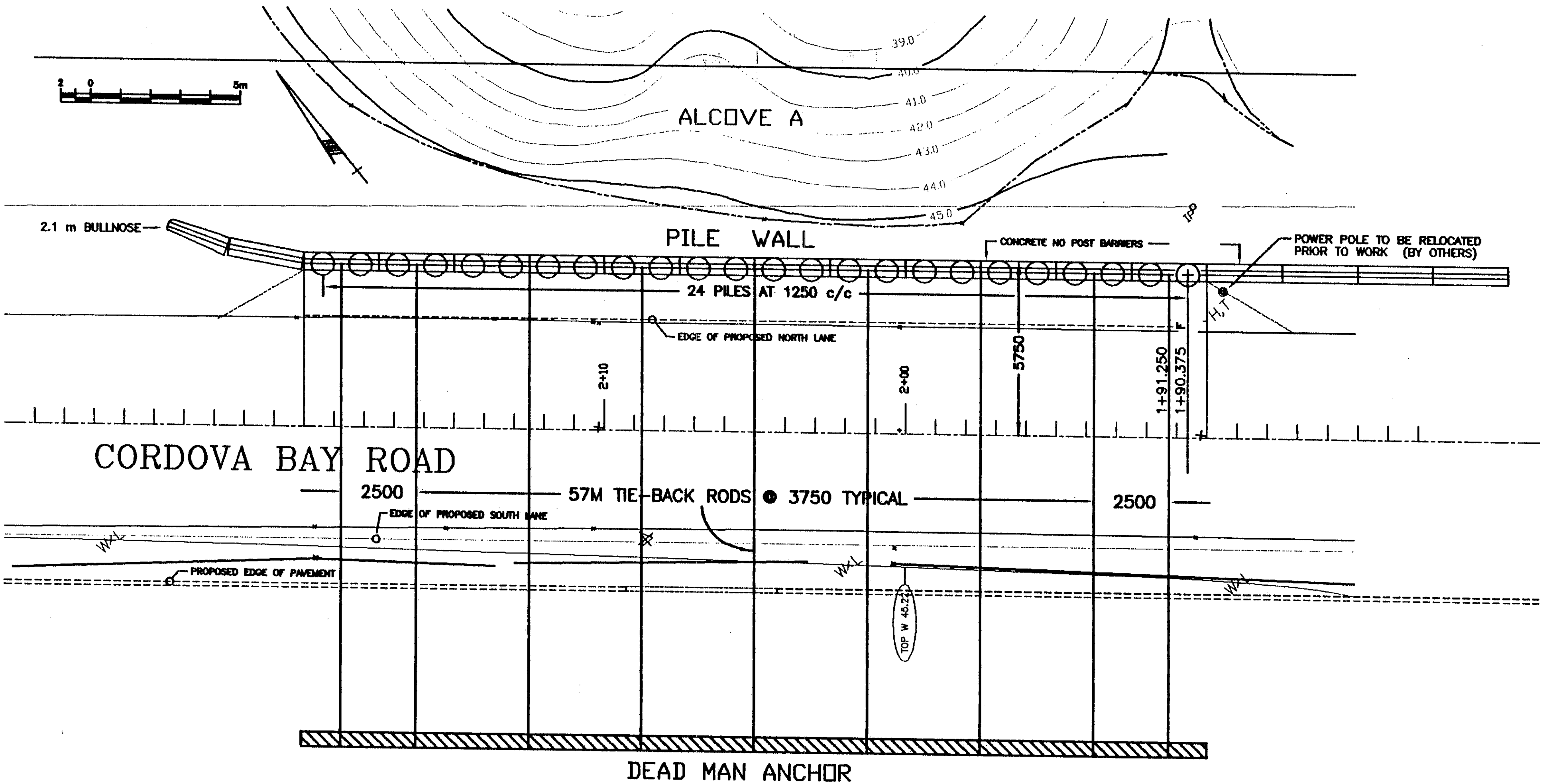


Figure 8. Bending Moment Diagram


ALCOVE A
39.0
41.0
42.0
43.0
44.0
45.0
2 0 5m
2.1 m BULLNOSE
PILE WALL
CONCRETE NO POST BARRIERS
POWER POLE TO BE RELOCATED PRIOR TO WORK (BY OTHERS)
24 PILES AT 1250 c/c
EDGE OF PROPOSED NORTH LANE
2+10
2+00
5750
1+91.250
1+90.375
CORDOVA BAY ROAD
2500
57M TIE-BACK RODS @ 3750 TYPICAL
2500
EDGE OF PROPOSED SOUTH LANE
PROPOSED EDGE OF PAVEMENT
TOP W 45.22
DEAD MAN ANCHOR

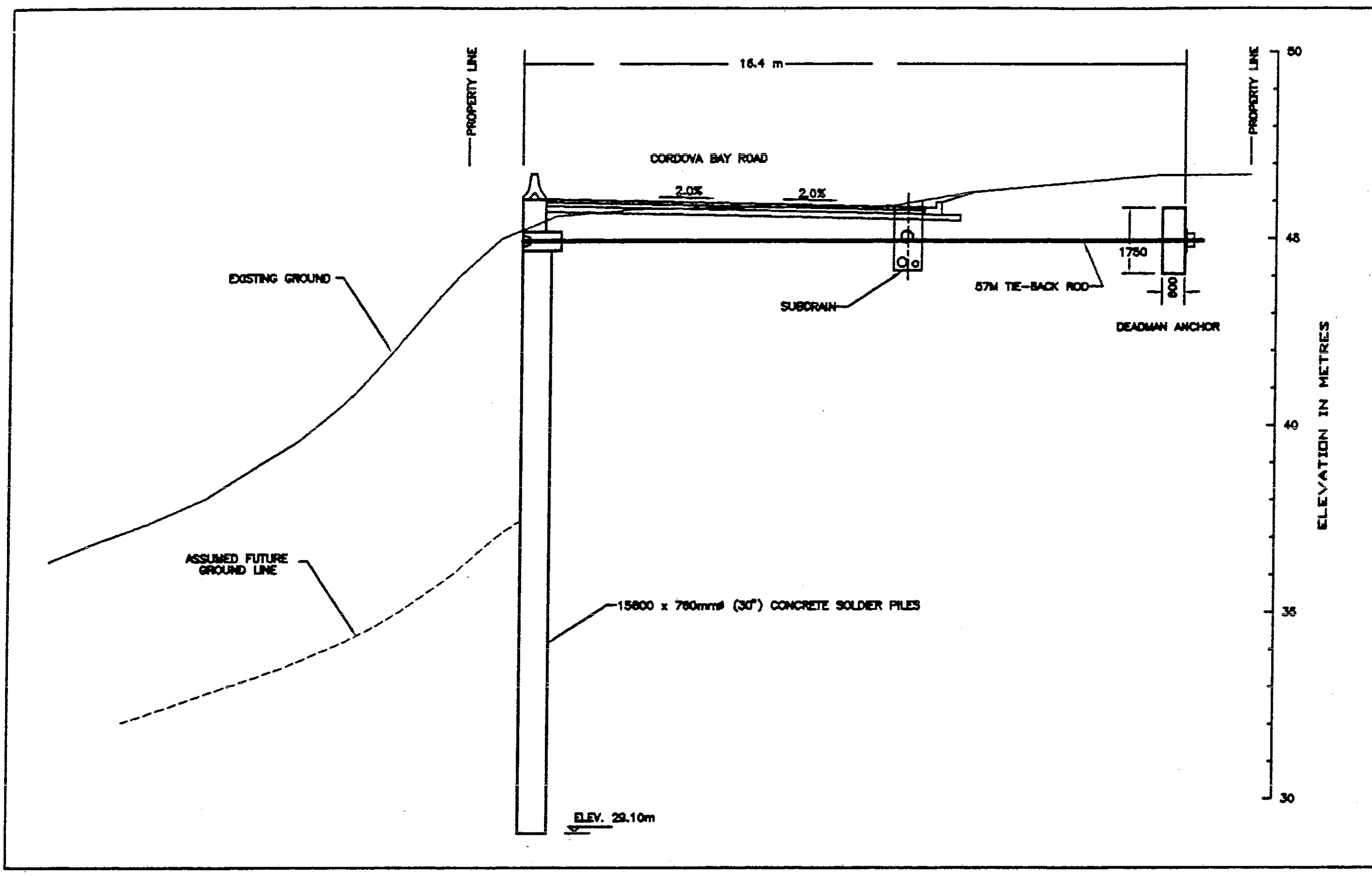


Figure 10. Cross Section of Pile Wall

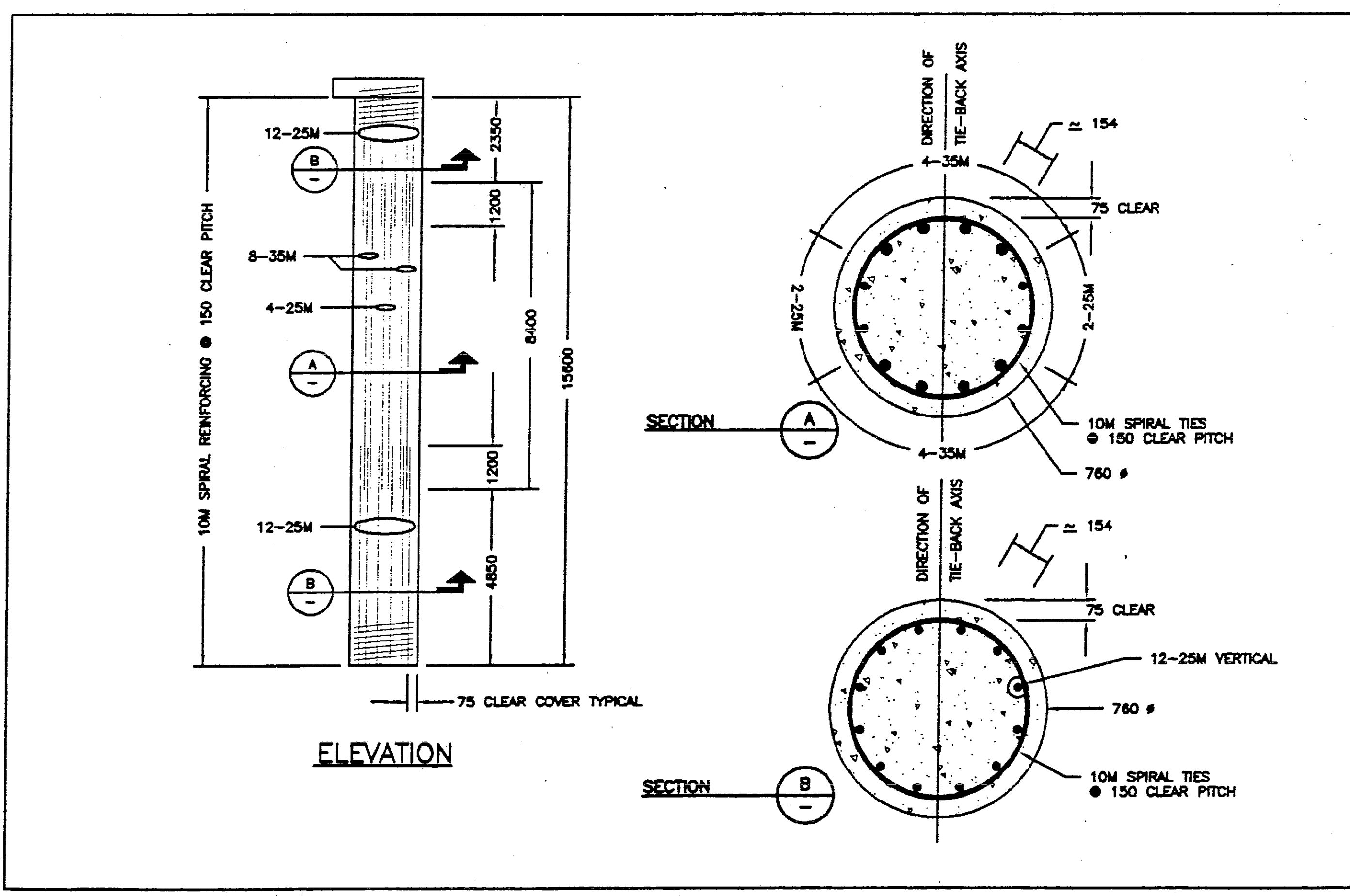


Figure 11. Structural Details of Piles

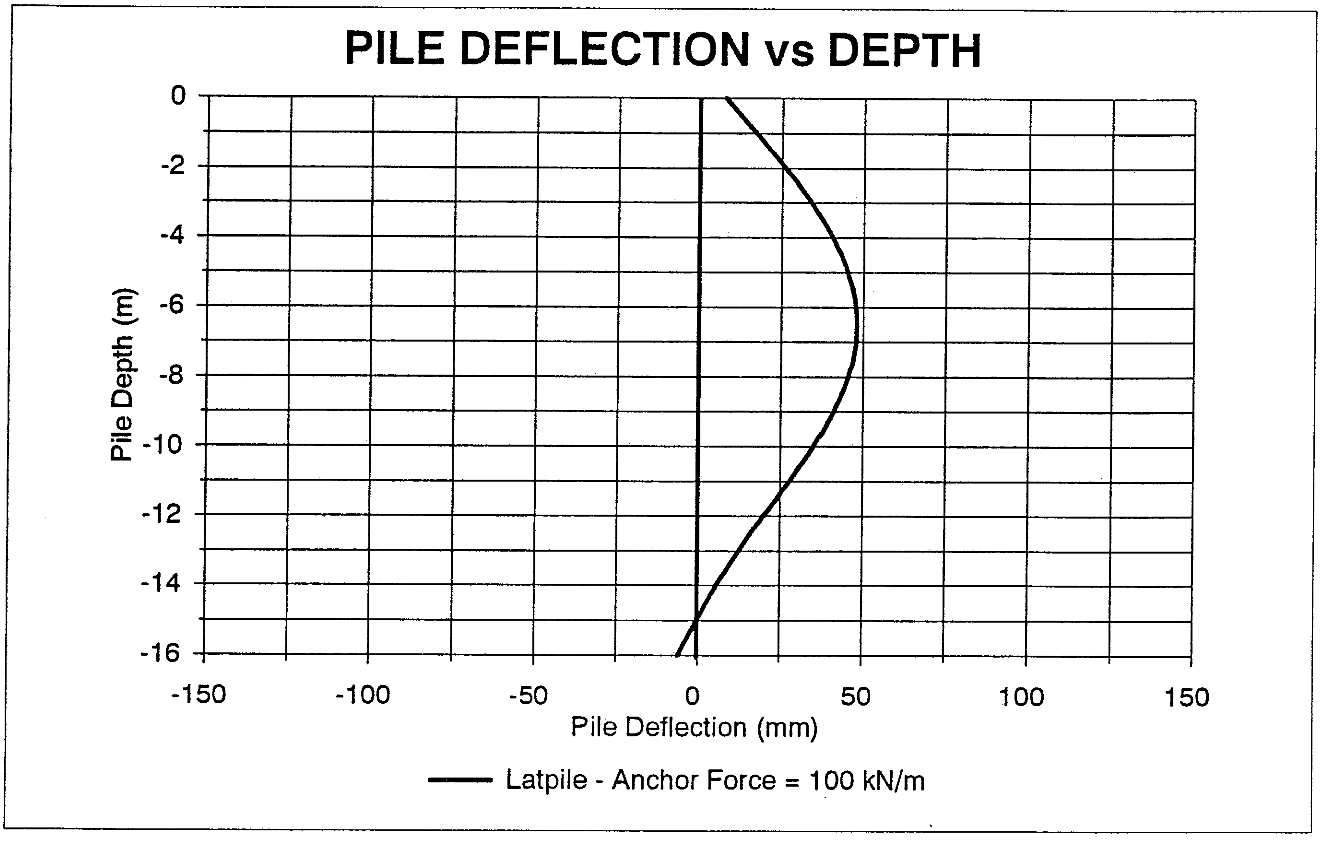


Figure 12. Latpile Analysis Pile Deflection Diagram

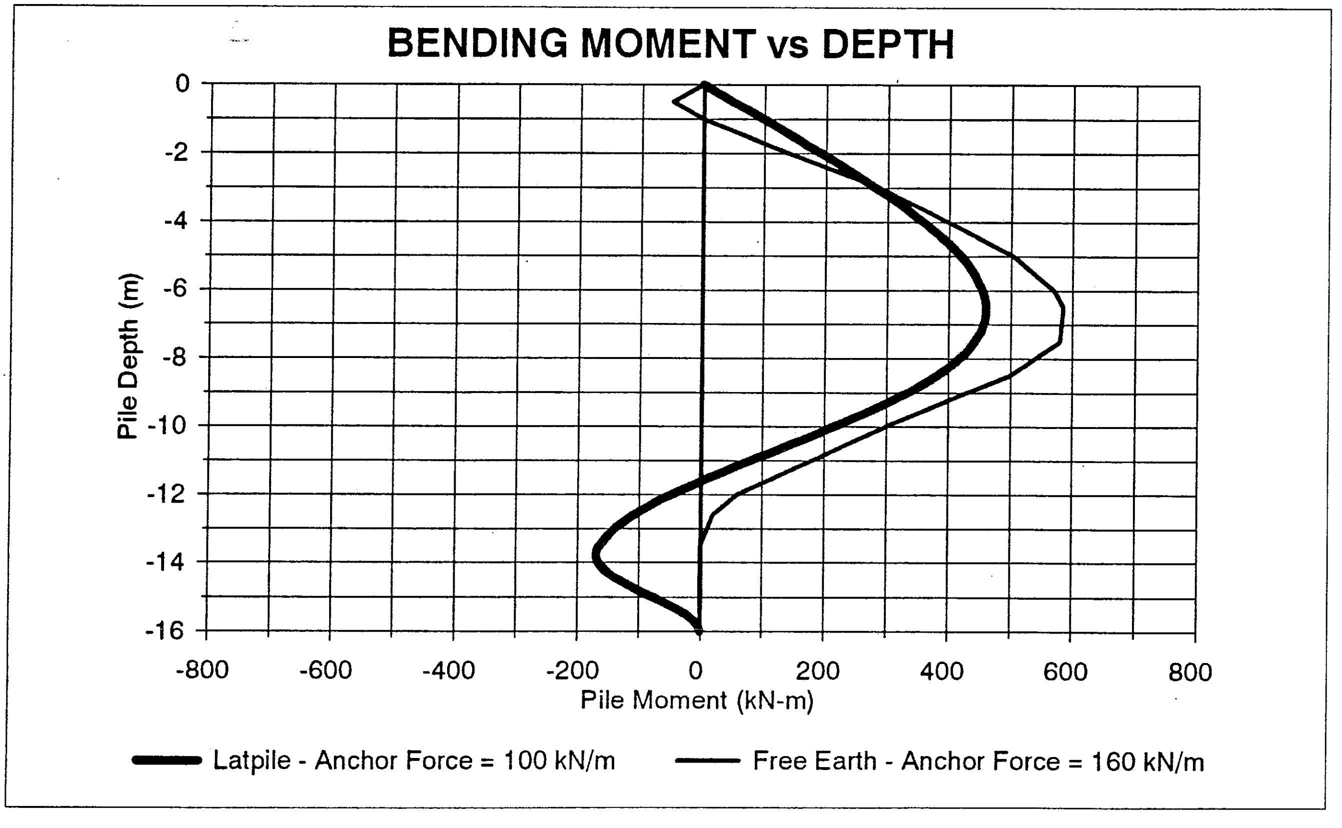


Figure 13. Latpile Analysis Bending Moment Diagram