



ENGINEERING LTD.
RR 22, 1209 Roberts Creek Road
Roberts Creek, B.C.
V0N 2W2

Tel/Fax: (604) 885-8812
Cell: (604) 788-6586
E-mail: eivemark@dccnet.com

April 30, 2006

Project Number CO - 03

Ron Davis,
Concordia Homes Ltd.
11651 227th Street
Maple Ridge, B. C.
V2X 1V2

Dear Mr. Davis:

**Re: Geotechnical Assessment, Phase 1, The Shores Subdivision
Sechelt, B.C.**

Attached are four copies of our report *Geotechnical Assessment, Phase 1, The Shores Subdivision, Sechelt, B.C.* and the final invoice for our professional services related to the geotechnical assessment.

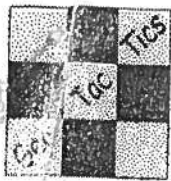
The report is a consolidation of the preliminary geotechnical assessment, the test pit investigation and the results of the construction field reviews for the first year of site preparation. The major recommendations of the preliminary report are confirmed, and, where indicated by field observations during stripping and excavation, the recommendations have been revised or expanded to satisfy actual site conditions.

We trust that this report satisfies your current requirements. If you have any questions, or require any further information, please do not hesitate to contact us.

Yours very truly,

GEOTACTICS ENGINEERING LTD.

M. M. Eivemark, P.Eng.
Principal Engineer



ENGINEERING LTD.
RR 22, 1209 Roberts Creek Road
Roberts Creek, B.C.
V0N 2W2

Tel/Fax: (604) 885-8812
Cell: (604) 788-6586
E-mail: eivemark@dccnet.com

FILE?

April 28, 2006

Gale Ave N.

3341-79

Project Number CO - 03

Ron Davis.
Concordia Homes Ltd.
11651 227th Street
Maple Ridge, B. C.
V2X 1V2

Dear Mr. Davis:

**Re: Geotechnical Assessment, Phase 1, The Shores Subdivision
Sechelt, B.C.**

INTRODUCTION

In accordance with your authorization, GeoTacTics Engineering Ltd. (GeoTacTics) has prepared this report to describe the geotechnical conditions affecting this property and to provide recommendations for site drainage and site preparation, including retention systems for cuts and fills.

The property is located in parts of Development Permit Areas E/H #3 – Shoreline and Foreshore Areas and E/H #4 - Rocky Beach Front and Upland Slopes as defined in the District of Sechelt Official Community Plan (OCP).

Phase 1 includes 28 individual lots including 13 lots along the shoreline. Phase 1 encompasses an area of about 7.5 ha (18 acres) bounded by Sechelt Inlet on the north and the Snake Bay Creek drainage course to the northwest. To the east, the property is bounded by the proposed extension of Crowston Road. The southern limit of Phase 1 is the L-200 Lane presently under construction. Infrastructure for the subdivision would include construction of the extension of Gale Avenue North (L-100 Road) and the L-200 Lane, extension of existing services (power, gas, water, telecommunications and sanitary sewer) to service the individual lots. A 1.8 ha park dedication is located in the northwest corner of the property.

This report is a consolidation of the work done for the preliminary geotechnical assessment in September 2004, the test pit investigation carried out in February 2005 and the field reviews completed during the site preparation activities of the summer and fall of 2005.

This report summarizes the results of the geotechnical assessment and presents geotechnical recommendations for site development. The report has been prepared in accordance with generally accepted geotechnical engineering principles and practices.

The location of the site is shown on the satellite image on Figure 1 and the general layout of the Phase 1 area is shown on Figure 2. Logs of the test pits completed in February 2005 are attached in the Appendix to this report.

SITE DESCRIPTION

The Phase 1 area is located overlooking the west side West Porpoise Bay and extends about 1380 feet westward from the existing end of pavement of Gale Avenue North. The lots vary in depth from about 200 to 260 feet, and vary in area from about 13,550 to 20,200 square feet. The lots rise from sea level to Elev. 42 to 53 m along the southern edge of the L-200 Lane. The September 2004 report described four physiographic zones comprising the entire subdivision; Phase 1 covers three of the physiographic zones, including the Foreshore, the Backshore Marsh and the Lower Slope.

The Foreshore is a shelf about 650 feet wide extending out (northward) from the shoreline to about the 2 fathom contour. Beyond the 2 fathom contour, the bottom of the inlet drops off steeply. The shelf is mantled with detritus washed into the inlet from Snake Bay Creek, from the short watercourses draining the northern slope of the property and from wave erosion of the shoreline. Waves generated in the Inlet shoal on the shelf and cause major erosion along the shoreline. The shoreline is marked by a 5 to 8 feet high vertical bank that has been undercut along several sections of the property with the result that shoreline trees are undermined and topple into the Inlet creating a chaotic jumble of timber along the beach. It is estimated that the average rate of recession of the shoreline under current conditions is between 6 to 12 inches per year.

The Backshore Marsh is a strip bounded by the shoreline and extending southward between 20 and 160 feet to the base of the Lower Slope. The width of the Backshore Marsh decreases toward the eastern side of the property so that, by Lots 1 and 2, the marsh is little more than a narrow terrace at the base of the Lower Slope. The surface of the Backshore Marsh varies between about Elev. 1 and 15 m. Much of the Backshore Marsh will be included within the 15 m shoreline environmental

setback zone. The Backshore Marsh is developed on a terrace where water accumulates from seepage and runoff from the slope above. The surface consists of about 6 to 12 inches of saturated organic soil (peat) overlying firm to stiff silt or dense till-like soil. The marsh supports a moderate to dense growth of salmonberry with some Devil's Club and abundant skunk cabbage. Trees are mainly cedar, but some maple and alder are present.

The Lower Slope rises between Elev. 15 and 50 m. The base of the Lower Slope transitions from the Backshore Marsh with a gently increasing slope rising about 20 feet above the marsh and ending at about a 15° slope and then rising at between 20 and 35° to the southern end of the Phase 1 area. The Lower Slope is relatively uniform through Lots 1 to 4, and begins to develop a distinct bench at about Elev. 20 m on Lots 5 to 8. The bench varies up to 65 feet wide and is inclined at 10 to 15°. On the south side of the bench through Lots 7 and 8, the slope steepens to as much as 50°. Local oversteepening of the slope is common where blow-down trees have been up-rooted and the overturned root mats cause pits in the slope. Most of the Lower Slope has now been cleared and grubbed.

Subsurface Conditions

The general stratigraphy beneath Phase 1 as determined by the test pit program and verified by initial road excavation and grading consists of an upper zone between 50 and 65 feet thick comprised of hard glacial till and marine silt. The upper zone is underlain by 35 and 50 feet of clean fine sand with some lenses of silt. The sand layer overlies 50 to 65 feet of stiff to firm marine silt with lenses of fine sand lenses and some layers of dense glacial till. Many of the sand lenses are water-bearing and springs are common across the entire slope. Recent stripping and excavation for site servicing have exposed multiple layers of water-bearing sand within the marine silt zone.

The general slope has been incised by a number of short drainage courses fed by springs mostly arising in the lower third of the Lower Slope. Some of the drainage courses, such as one east of Lot 1 and the one crossing Lot 10, are fed by a more substantial spring system and also carry a significant discharge of surface runoff during the wet season. The channels of the drainage courses are generally fairly shallow (less than 10 feet deep) and lose definition crossing the Backshore Marsh. As mentioned above, groundwater is generally associated with springs occurring in the Lower Slope and on the Backshore Marsh. Active flow was observed in all of the drainage courses.

Construction activities – mainly excavation for L-100 Road – exposed numerous springs at the base of the fine sand layer between Sta. 2+40 and 4+00. Most of the springs could be handled by

interceptor ditches and culverts beneath the L-100 Road. However, in the vicinity of Sta. 3+00 a major new spring was uncovered at the toe of the slope. The volume of flow has been estimated to be in the order of 200 gpm (1000 l/min.). Further, a large volume of sand was being washed out at this location. Based on a visual estimate of 5% solids per volume of water, the spring area was discharging approximately 0.05 m^3 of sand per minute, or about 70 m^3 of sand per day. Several attempts were made to bring the spring discharge under control. It took 4 weeks to successfully construct a stable filter and buttress over the spring.

For this site, the maximum horizontal firm-ground acceleration for an earthquake with a 10% probability of exceedance in 50 years (475-year recurrence interval) is about 0.22 g. There is a negligible to low probability of liquefaction of the subgrade soil at this site during the design earthquake. Ground shaking due to an earthquake approaching the design earthquake could cause additional movement of the slopes. Damage to non-structural components of buildings due to ground motion could be significant, while damage to structural elements could be moderate.

DISCUSSION AND RECOMMENDATIONS

Slope Stability

General Slope Stability – Natural areas of potential slope instability are confined to the Lower Slope where the inclination is steeper than about 25° . The Lower Slope is experiencing on-going gradual erosion of the surficial soil and localized shallow sloughing on steeper pitches.

Construction activities resulted in soil from excavation being cast over the slope below the work site as a temporary storage measure. When the loose soil cover became saturated and occurred on slopes steeper than about 20° , the soil slid. Several areas of tension cracks developed in the loose soil on the slope.

The major spring at Sta. 3+00 also initiated slope instability by washing out large volumes of soil and leaving cavities within the slope. Local collapses occurred that combined with the slippage of the loose excavated soil described above.

Wave erosion, as previously described, is active along the entire shoreline. It would be technically possible to control the erosion by placing rock armour against the shoreline cutbank, but gaining approval from the Department of Fisheries and Oceans to work on the beach is unlikely.

Stability Analyses - A series of stability analyses were carried out to assess the condition of the natural slopes. For the analyses, a cross-section of the slope was developed, which included assumptions regarding natural planes of weakness (fractures and tension cracks) in the soil. A 2-layer system was analyzed consisting of glacial till overlying marine silt. The soil properties used in the calculations were:

Soil	Glacial Till	Marine Silt
Unit Weight	19 kN/m ³	17.5 kN/m ³
Friction Angle	38°	32°
Cohesion	10 kPa	0 kPa

The stability of the slope for different cases was analyzed using the computer software SLOPE/W, and a Factor of Safety was calculated for each case. The friction angle of the marine silt was decreased to 30° as part of a 'sensitivity analysis' to determine how the calculated Factors of Safety would change as the strength was decreased. The results of the stability calculations are summarized in the table included in the Appendix.

Back analysis of a section of the slope that had experienced instability using the selected soil parameters resulted in a calculated Factor of Safety of 1.01. This formed the basis for assessing the potential for additional slope failures regressing upslope from the initial slide. Adoption of a reduced friction angle for the marine silt was also confirmed to be reasonable, with a Factor of Safety of 0.985 being calculated for the back analysis of a possible slip occurring in the vicinity of the old logging road between L-100 and L-200 (see Figure 3).

From the stability analyses, it is concluded that the natural slope, unaffected by existing planes of weakness or sink holes, has an adequate Factor of Safety. When local factors, such as fractures in the soil or solution cavities are considered, the calculated Factors of Safety decrease and the probability of slope failure increases. This case also indicates that a 'retrogressive' mode of failure is probable.

As part of the proposed site development, the area where the instability has occurred will be graded by cut-and-fill techniques to create building pads. The soil in blocks that have slid will be removed and site grading fill, with properly engineered retaining walls, will be placed to buttress the slope.

Sink Holes – During September and October, 2005, as a result of the major spring at Sta. 3+00, a number of changes to the unstable area were observed. Two sink holes developed along the tension crack at the L-200 level. One occurred at the crest of the slope and was about 10 feet in

diameter and about 8 feet deep. A smaller sink hole had developed on the slope just below the crest. Further to the south, the original tension crack had dropped an additional 0.4 to 0.6 inches and opened horizontally a similar amount.

Not directly related to the sink holes above the spring, two other sink holes were observed on the property. The larger – about 8 feet in diameter – occurred on the west side of the drainage course at the east end of the property, and the smaller was observed in the cut bank of road excavation at about Sta. 2+10 along L-100.

The formation of the sink holes above the spring area is concluded to be the result of the base of pre-existing erosion caverns collapsing due to the loss of soil beneath due to the sediment being washed out of the hillside by the spring. The newly appeared sink holes are aligned with the tension cracks that previously opened, and the alignment can be projected down the slope to where the initial slide along the old logging road occurred.

On further investigation, the occurrence of sink holes in this area was found to be more common than previously believed. Golder Associates¹ described sink holes in similar terrain on the north side of Sechelt Inlet. Large sink holes have also been encountered during residential construction further east on Gale Avenue North and in the Highmoor Subdivision.

Setbacks

Development Permit Area E/H #4 requires a 15 m setback from the Natural Boundary as shown on Figure 2. This setback provides an adequate safe buffer to allow for shoreline erosion.

From a geotechnical point-of-view, areas where building setbacks would otherwise be required will be cut and filled to create stable building pads.

Drainage

Site Drainage - Clearing and stripping required for the development has exposed soil that is potentially susceptible to erosion. Surface runoff and seepage should be controlled to minimize erosion potential.

¹ Golder Associates (1993): Geotechnical and Environmental Reconnaissance Study of the District of Sechelt Official Community Area

Recommendations for drainage swales and interceptor ditches were issued during field inspections of site preparation for L-100 Road and L-200 Lane. The typical details for an interceptor drain at the base of the slope are shown on Figure 4a.

Where a drainage course crosses L-100, particularly between Lots 10 and 20, and possibly elsewhere, a culvert should be provided. The recommended detail consists of a minimum 12-inch diameter CSP with 6 inches of pipe surround and 2 feet of fine blasted rock above the pipe surround, as shown on Figure 4b. The culvert could be ultimately tied into the storm sewer system or left as an independent drainage feature. Care will be required to avoid conflict between the culvert and the future services proposed beneath the road.

Where manholes are installed as part of the interceptor drain systems, the subgrade soils are susceptible to rapid softening and degradation of strength when exposed. It is recommended that, as soon as foundation grade for a manhole is exposed, a sheet of $\frac{3}{4}$ -inch plywood be placed on the subgrade surface followed by 4 to 6 inches of $\frac{3}{4}$ -inch clear crushed gravel as bedding for the concrete barrel of the manhole. Additional $\frac{3}{4}$ -inch clear crushed gravel should be placed inside and outside the manhole barrel. It is also recommended that to minimize damage to the subgrade surface, the surface around the manhole be exposed in small sections with each section being covered with at least 6 inches of gravel before the next section is exposed. Figure 5 shows the recommended detail for manhole installation.

Where water-softened conditions spread faster than the softened material can be removed, particularly along the outflow pipe trench from the manhole, it is recommended that a 2-foot layer of 12-inch minus rock be placed as a stabilizing base for the pipe bedding and trench backfill.

A design was provided on site to control large spring flows where encountered at the toe of the slope. The design comprised the following:

- A layer of geogrid at the horizon between native (sloping) ground and a proposed 'buttress and filter' system.
- 1-foot rock in front of the geogrid. The width of this layer of the 'buttress and filter' system shall be at least 4 feet.
- 6-inch rock on top of the 1-foot rock. The width of this layer of the 'buttress and filter' system shall be at least 3 feet.
- $\frac{3}{4}$ -inch clear crushed gravel in front of the 6-inch rock. The width of this layer of the 'buttress and filter' system shall be at least 2 feet at the top and 6 feet at the base.

- 6-inch rock in front of the $\frac{3}{4}$ -inch clear crushed gravel. This layer is intended to provide protection for the $\frac{3}{4}$ -inch layer, and in effect will serve as 'rip-rap'. The width of this layer of the 'buttress and filter' system shall be about 18 inches.

The final elevation of the top of the above-described 'buttress and filter' system shall correspond to the approximate elevation of the large flow groundwater plus 2 m. The 'buttress and filter' system shall extend a distance of at least 30 feet along the toe of the slope at this location. There shall be a 6-inch perforated pipe placed along the length of the system within the $\frac{3}{4}$ -inch clear crushed gravel layer. The perforated pipe shall be connected to a non-perforated pipe at the approximate centre of the perforated section. The non-perforated section shall extend the full width of the roadway and exit beyond the northern edge of the roadway.

The 'buttress and filter' system will be repeated along the slope toe as necessary to address new areas of seepage as they develop. Further instructions and geotechnical specifications in this regard will follow as construction proceeds.

As the construction of the L-100 recommences, it was recommended that the design grade of the road be raised so that the road fill covers the 'buttress and filter'. Further, a drainage blanket should be constructed across the entire width of the road fill hydraulically connecting the 'buttress and filter' to the downslope side of the road fill where the water can be collected and conveyed to the beach.

As the interceptor ditches are to be integrated with the permanent site drainage system, a 6-inch perforated drain pipe should be laid in the base of the ditches. Big "O" pipe should not be used in the interceptor ditch. The drain pipe should be covered with $\frac{3}{4}$ -inch clear crushed gravel extending at least 18 inches above the base of the trench. Clean sand should be used to complete the backfilling of the interceptor ditch.

On Lots 1 to 14, site specific measures based on the recommendations should be implemented to deal with active spring areas on the Lower Slope. Such measures could include placing clean, clear crushed gravel around the head of the springs to buttress the sidewalls and prevent further headward erosion. At the toe of the gravel buttresses, perforated drain pipe could be installed in the upper end of a drainage course emanating from the spring and extended beyond the toe of the Lower Slope. The perforated pipe in the drainage course channels should be covered with clear crushed gravel. The lower ends of the drainage courses across the Backshore Marsh should be left undisturbed.

Perimeter Drains - Footing drains are required around each house to be constructed and should consist of 4-inch perforated drain pipe surrounded by at least 12 inches of nominal $\frac{3}{4}$ -inch drain rock or clear crushed gravel wrapped in filter fabric. A minimum 2-foot wide zone of free-

draining backfill should be placed against foundation walls above the drain rock and extended to final site grade. Water collected in the footing drains and roof drainage should be discharged to the storm sewer system along the street or to the nearest natural drainage course on the Lower Slope. Roof drainage should be conveyed to discharge in a separate non-perforated pipe and should not be discharged into the footing drains around the house.

Stripping and Excavation

Prior to any other construction activities, road rights-of-way and house sites should be stripped and graded to acceptable foundation subgrade for a minimum distance of at least 5 feet beyond the perimeters of the houses or roads. Roots, sod, topsoil and other deleterious debris from within the footprint of a proposed building should be removed and disposed of away from site. Non-organic soil from excavation could be stockpiled for reuse as backfill; disposed of on site; or disposed of offsite.

Where excavation is required for construction, for depths of excavation greater than 4 feet but less than 8 feet, temporary unsupported slopes should be excavated no steeper than $3/4H : 1V$. For excavated slope heights greater than 8 feet, temporary unsupported slopes should be no steeper than $1 : 1$. If excavation depths exceed 15 feet, intermediate safety benches at least 3.5 feet wide and spaced at 10-foot vertical intervals should be provided on the temporary slopes. If seepage is encountered, the temporary slopes should be flattened to at least $1.5H : 1V$, unless a geotechnical inspection indicates that steeper slopes are safe. All temporary excavated slopes should be covered with plastic sheeting to prevent erosion by precipitation. Slopes steeper than those recommended above may be excavated, but in that case shoring may be required depending on the findings of the Geotechnical Engineer. The toe of stockpiled excavated soil should be placed no closer than 5 feet to the edge of the top of excavation.

Without engineered retaining structures, no permanent excavated or fill slope higher than 1.2 m should be steeper than $2.5H : 1V$.

Site Grading Fill

Where site grading fill is placed for road subgrade or building pads, the following procedures were recommended. After proof-rolling of the subgrade, material for subgrade backfill should consist of imported 3-inch minus pit run sand and gravel, or approved granular soil from on-site excavation.

The backfill should be placed in 12-inch lifts and compacted to 95% MPMDD. As applicable, moisture-conditioning will be required.

Where embankment fill is required to a thickness of more than 4 feet, a drainage layer consisting of 18 inches of clean sand or clean sand and gravel should be placed at the base of the fill. Where the drainage layer day-lights in the face of the embankment slope, a protective cover of coarse gravel or fine blasted rock should be placed over the edge of the drainage blanket.

Figure 6 shows the recommended site grading fill with the drainage blanket.

Footings and Foundations

With respect to actual building construction, the subgrade soils at this site are competent to support the typical structural loads of 2- to 3-storey timber frame buildings without significant post-construction settlement. If loose soil is encountered at footing grade, the subgrade must either be compacted, or the foundation excavated down through the looser soil to competent bearing subgrade. Footings on prepared surface may be sized for a maximum allowable soil bearing pressure of 2,000 psf. The minimum width of wall footings should not be less than 16 inches, and the minimum dimension of pad footings supporting columns should not be less than 3 feet. All perimeter footings should be placed at least 18 inches below final site grade for frost protection.

The Pump Station is a special case. The Pump Station is to be located at the north end of the drainage right-of-way between Lots 12 and 13. At this location, marine silt and fine sand comprise the subgrade. Drainage will be a major issue to maintain a dry working excavation. It is recommended that an interceptor drain be excavated around the top of the south perimeter of the excavation. The dimensions of the excavation should be large enough to allow construction of interceptor ditches around the perimeter of the base of the excavation to capture seepage into the excavation. When the excavation reaches final grade, a 6-inch layer of $\frac{3}{4}$ -inch clear crushed gravel should be immediately spread over the base of the excavation to protect the surface from degradation. Provided the foundation surface does not deteriorate due to exposure to water, a maximum allowable soil bearing pressure of 2,500 psf can be used for the design of footings. The recommended soil bearing pressure may be increased by one-third to account for transient wind loads.

Slabs-on-Grade

For slabs-on-grade, the Geotechnical Engineer should assess the foundation conditions on a lot-by-lot basis to determine the necessity of a granular bedding layer beneath the slab. A vapour barrier consisting of minimum 10 mil polyethylene sheeting should be placed beneath the underside of the slab. A thin layer of sand may be placed on top of the vapour barrier to protect the polyethylene sheeting from tearing during construction of footing forms and pouring concrete.

Retaining Walls and Basement Walls

Basement walls and retaining walls should be designed to resist lateral earth pressure due to the backfill behind the walls. The lateral earth pressure should be calculated based on a triangular distribution for a material with an equivalent fluid density of 45 pcf. The backfill should be compacted, with attention not to damage concrete walls. Drainage should be provided at the base of the walls as described above.

Where excavated cuts are required and slope flattening is not feasible, slope retaining structures will be required. Stacked rock retaining walls are recommended.

For a stacked rock retaining wall up to 8 feet in height, the base layer of rock should be placed on a leveled, prepared surface. A gravel base is recommended. The foundation surface for gravel base beneath the lowest layer of rock should be stripped of all organic material and loose soil. The stacked rock wall should be constructed with angular rock fragments with minimum dimension varying between 2 and 4 feet. Each fragment should be individually placed and adjusted to achieve the optimum interlocking of fragments. As-built, the outer face of the rock wall should be no steeper than 5 (V) : 1 (H).

If wall height is greater than 8 feet, the design detail should be modified to include geogrid reinforcement to stabilize the wall and backfill. The batter on the face of the stacked rock wall can be increased to 6 (V) : 1 (H). The details are shown on the attached Figure 7.

Roads

The details for the road cuts and fills, and service installation, are shown on the site servicing drawings prepared by WEB Engineering Ltd. The drawings include the recommendations made by

GeoTacTics, and have been reviewed by GeoTacTics to verify that the geotechnical issues have been addressed.

Sediment Control Plan

A sediment control plan has been prepared and submitted as a separate document. The initial works for sediment control have been implemented and as the weather improves, sediment ponds as recommended in the sediment control plan will be constructed.

Permanent facilities for collection and conveyance of surface runoff are shown on WEB Engineering's site servicing drawings.

FURTHER INVESTIGATION

The source of the high-flow groundwater encountered during the 2005 site preparation has not been ascertained, but a 'targeted' borehole investigation program as been proposed. However, the timing of such investigation program would best be after the existing slope above the L-100 Road has been excavated to rough lines and grades.

The work for exploratory boreholes will commence once the slope unloading activities are underway and a platform is available for setting up a drill rig at the slope benches. The proposed borehole investigation program will include logging of the subsurface conditions as well as assessing the depth to groundwater. It is anticipated that the latter will be under 'artesian' conditions, and thus standpipe piezometers will be installed to monitor groundwater levels.

CONCLUSIONS

This report provides geotechnical recommendations for site preparation and foundation design based on an overall assessment of the property.

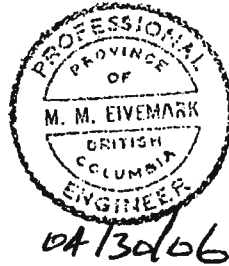
Provided the site preparation, earthworks and foundation construction follow the general guidelines outlined above, the property can be safely used for the intended purposes. Conventional single-family permanent or recreational residences constructed in this subdivision are expected to be safe against reasonably conceivable geotechnical hazards, including slope instability, erosion and flooding. The probability of occurrence of geotechnical hazards is estimated to be less than 10 percent in 50 years.

This geotechnical assessment was completed for the exclusive use of the Concordia Homes Ltd. and their agents for the development of the property as described in this letter. Any use of the materials contained in this letter for other than the intended purpose must first be verified in writing by GeoTacTics.

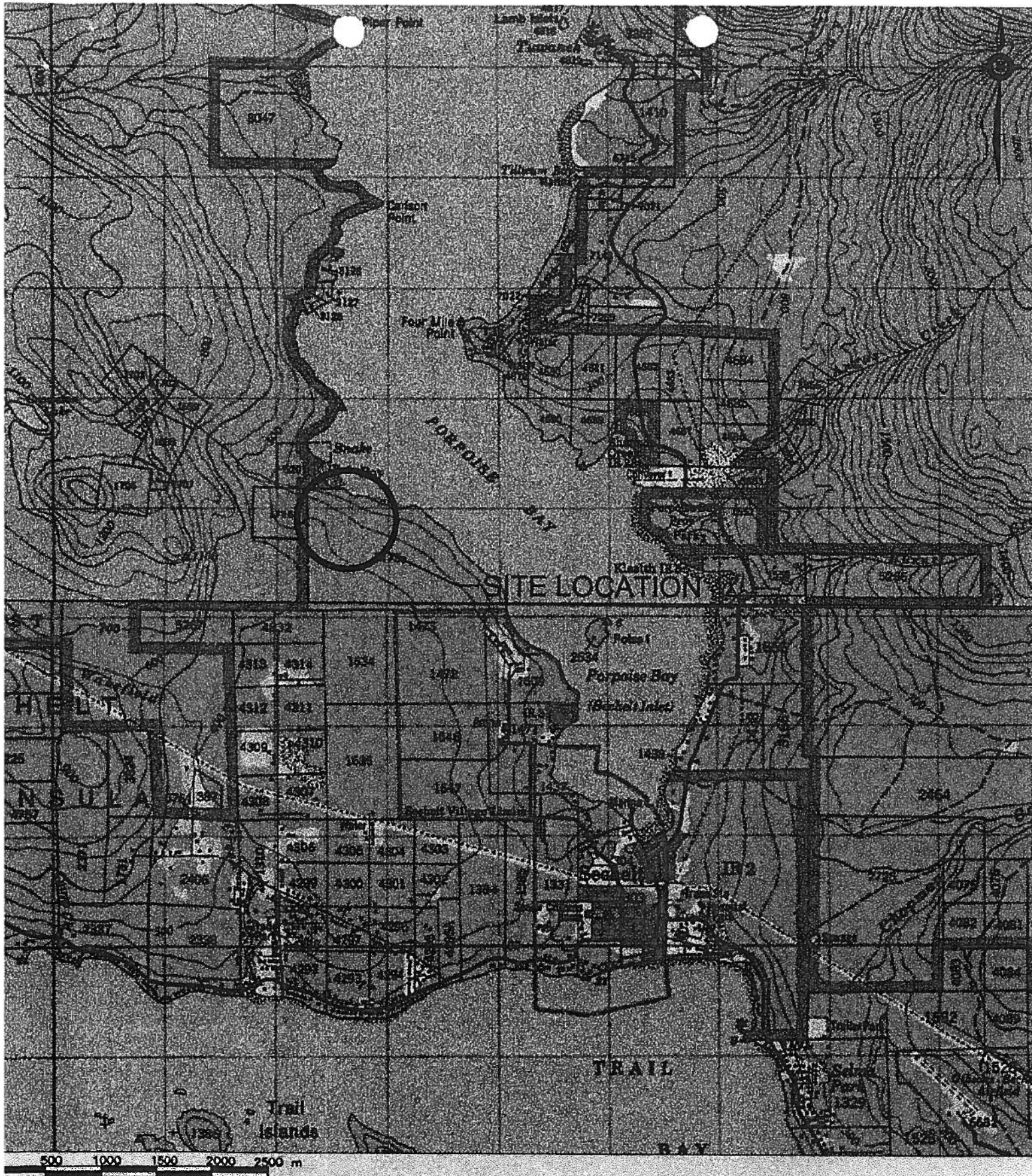
We trust that this letter satisfies your current requirements. If you have any questions, or require any further information, please do not hesitate to contact us.

Yours very truly,

GEOTACTICS ENGINEERING LTD.



M. M. Eivemark, M. Sc., P.Eng.
Principal Engineer



SCALE: 1:50,000



ENGINEERING LTD.

PROJECT

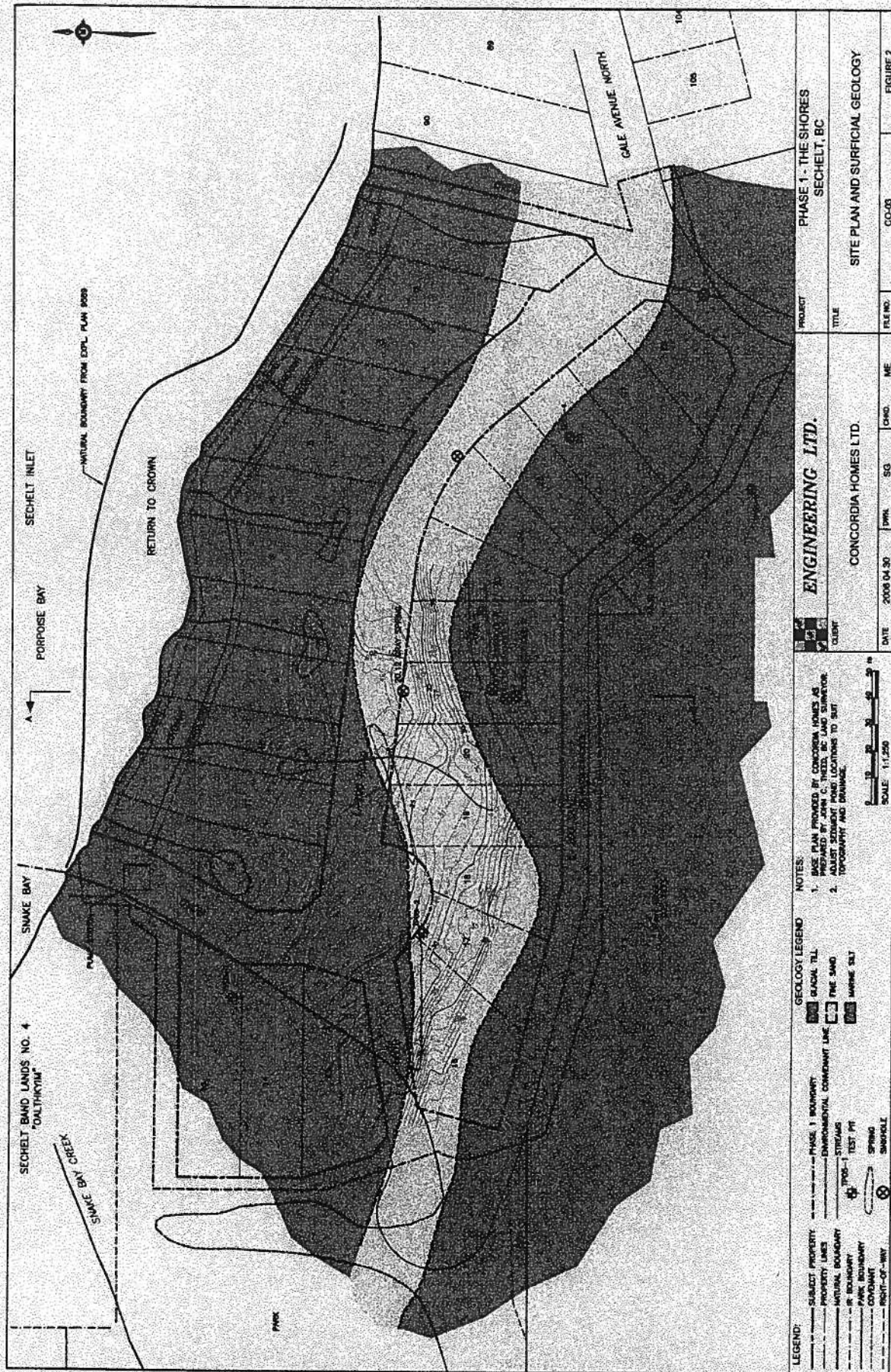
SECHELT, BC

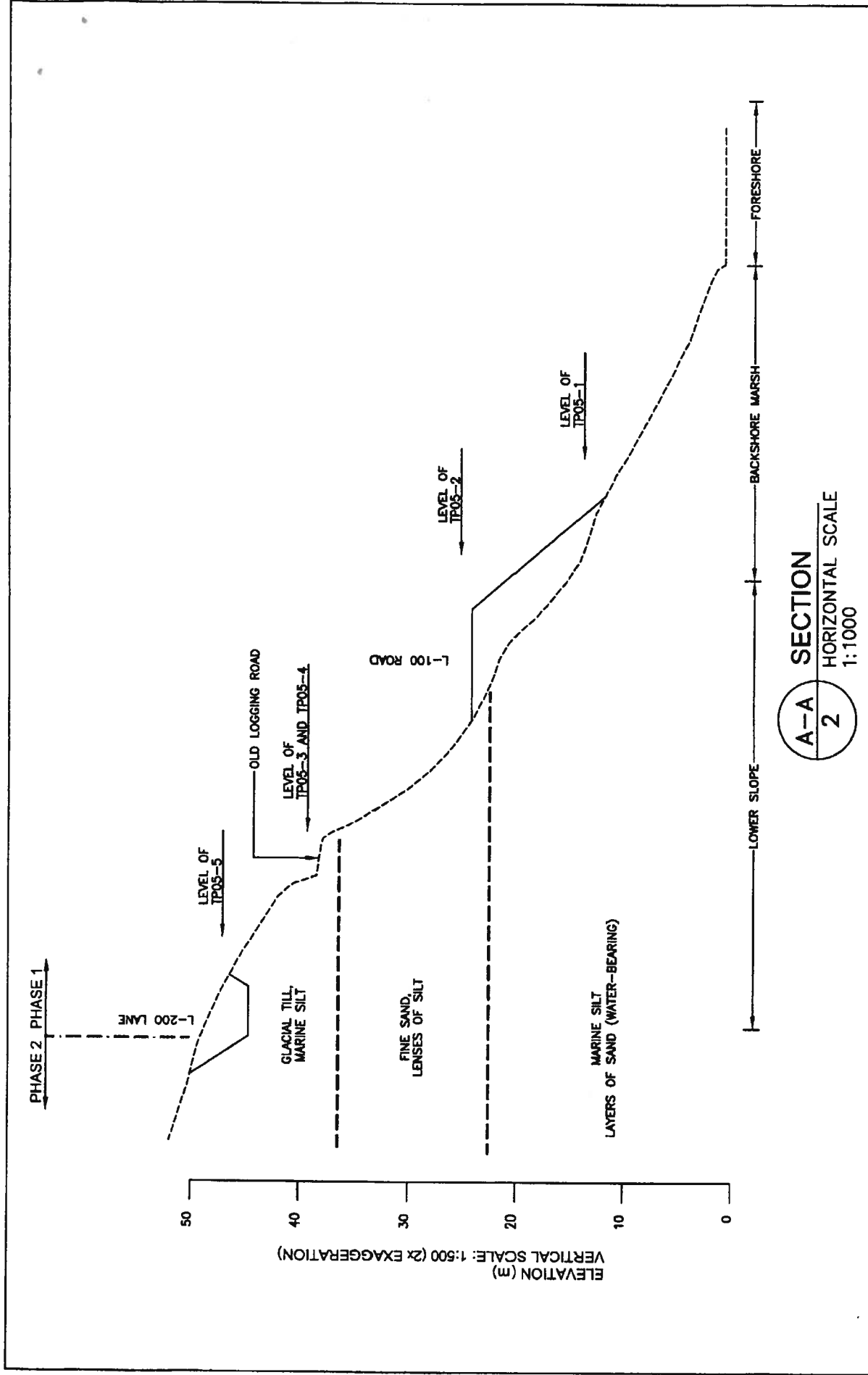
CLIENT

CONCORDIA HOMES - THE SHORES

TITLE

SITE LOCATION PLAN





	PROJECT		PHASE 1 - THE SHORES SECHLT, BC			
	TITLE		SLOPE PROFILE A-A'			
	CLIENT		CONCORDIA HOMES LTD.			
DATE	2006 04 30	DWN.	SG	CHKD.	ME	FILE NO.
					CO-03	FIGURE 3

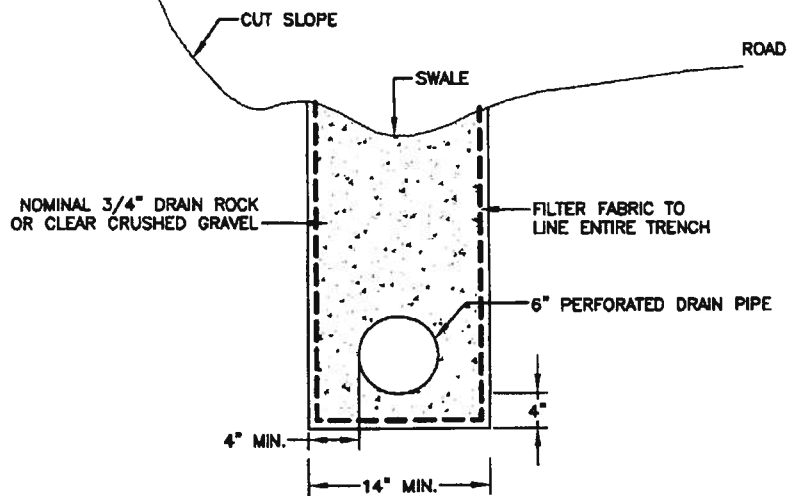


FIG 4a - DETAIL OF INTERCEPTOR DRAIN

NOTE:

TRENCH DEPTH 3' TO 4'

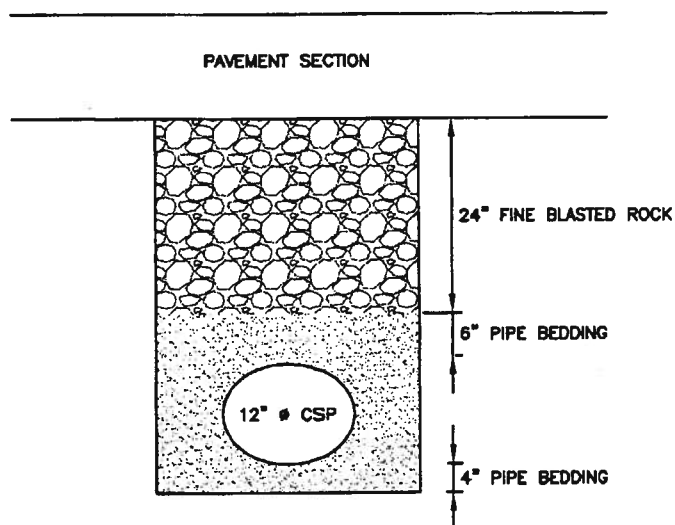


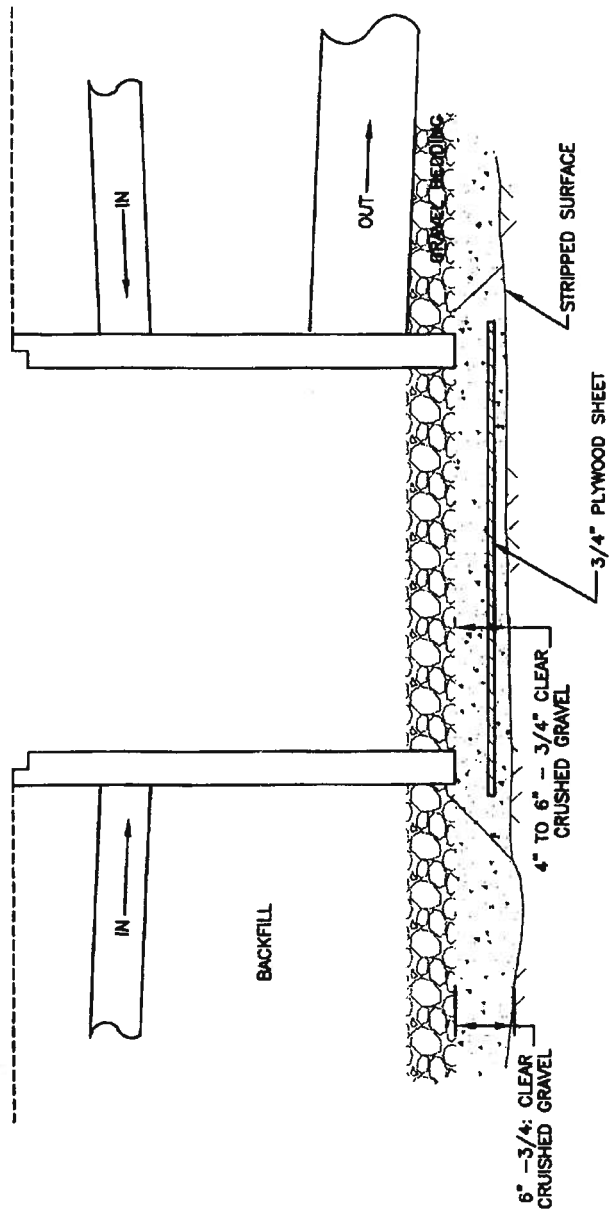
FIG 4b - CULVERT DETAIL

NOTE:

ADJUST DEPTH TO SUIT SERVICE GRADES, BUT
MINIMUM THICKNESS OF BEDDING AND BLASTED ROCK
NOT TO BE LESS THAN SHOWN.

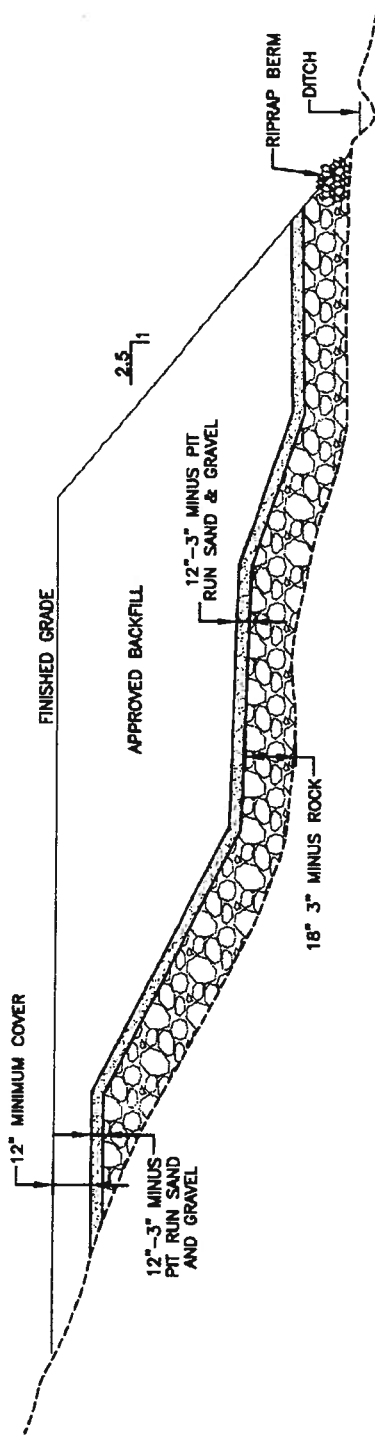
NOT TO SCALE

 ENGINEERING LTD.					PROJECT SECHLT, BC				
CLIENT CONCORDIA HOMES - THE SHORES					TITLE SITE DRAINAGE				
DATE	2006 04 30	DWN.	SG	CHKD.	ME	FILE NO.	CO-03	FIGURE NO	FIGURE 4



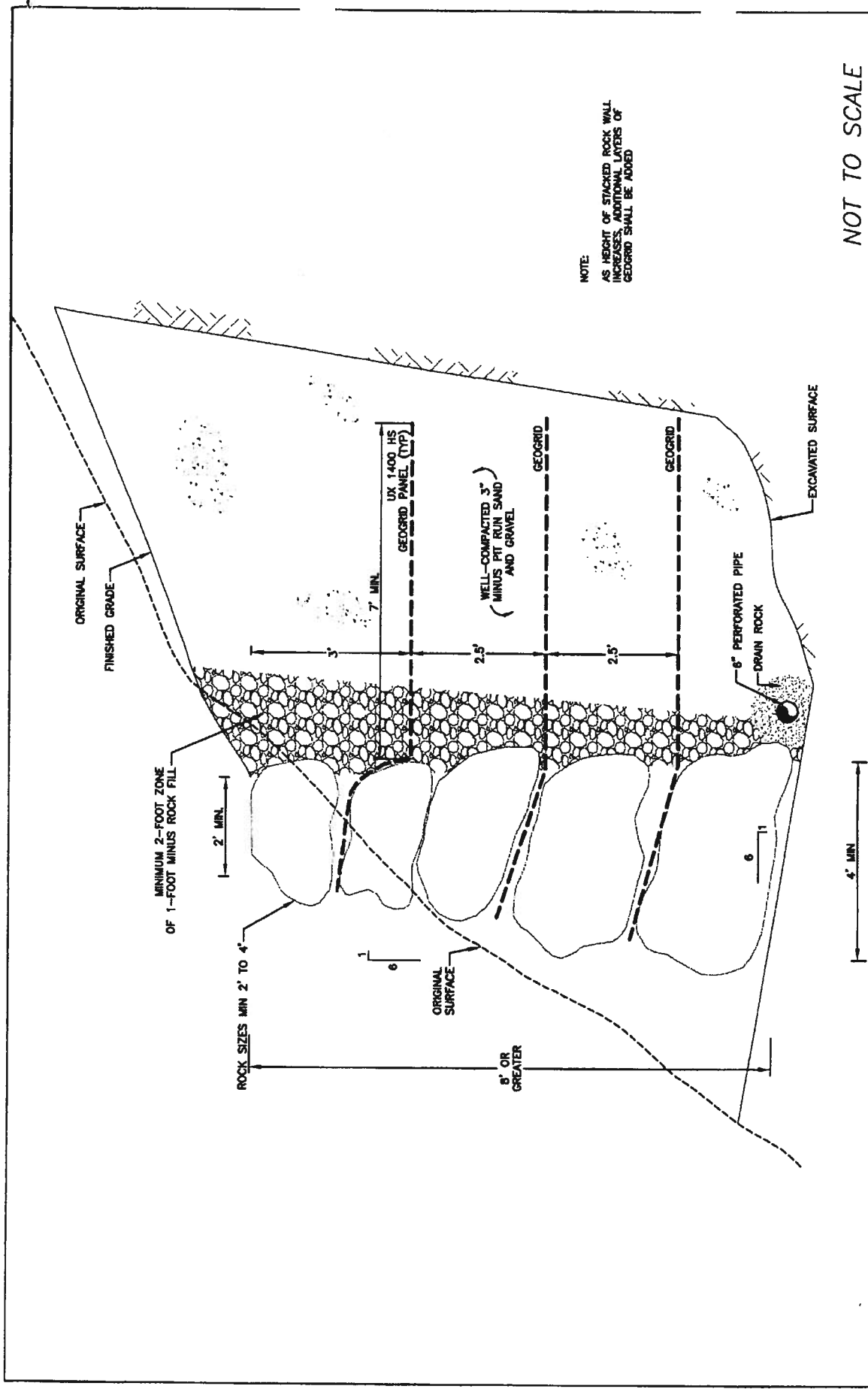
NOT TO SCALE

		ENGINEERING LTD.		PROJECT		SECHULT, BC			
CLIENT		CONCORDIA HOMES - THE SHORES		TITLE		MANHOLE FOUNDATION PREPARATION			
DATE	2006 04 30	DWN.	SG	CHKD.	ME	FILE NO.	CO-03	FIGURE NO	FIGURE 5



NOT TO SCALE

 ENGINEERING LTD.		PROJECT		SECHLT, BC	
CLIENT CONCORDIA HOMES - THE SHORES		TITLE		DETAIL OF DRAINAGE BLANKET BENEATH SITE GRADING FILL	
DATE	2006 04 30	DWN.	SG	CHKD.	ME
FILE NO.		CO-03		FIGURE 6	



NOTE:
AS HEIGHT OF STACKED ROCK WALL
INCREASES, ADDITIONAL LAYERS OF
GEOGRID SHALL BE ADDED

NOT TO SCALE

		ENGINEERING LTD.				PROJECT	PHASE 1 - THE SHORES SECHLT, BC	
	CLIENT	CONCORDIA HOMES LTD.				TITLE	TYPICAL SECTION FOR 8-FOOT OR HIGHER STACKED ROCK RETAINING WALL	
	DATE	2006 04 30	DWN.	SG	CHKD.	ME	FILE NO.	CO-03
								FIGURE 7

APPENDIX A

TEST PIT LOGS

TEST PIT RECORD

TP05-01

CLIENT CONCORDIA HOMES LTD. THE SHORES SUBDIVISION PROJECT No. BCV40527
PROJECT Geotechnical Investigation DATUM _____
LOCATION Sechelt, B.C. ELEVATION _____
EXCAVATION DATE Feb. 17-05 CONTRACTOR McCourt Contracting Ltd. EXCAVATION METHOD Excavator

DEPTH (m)	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLES			TEST RESULTS										DEPTH (ft)	
				TYPE	NUMBER	MOISTURE CONTENT (%)	50kPa				100kPa				150kPa			200kPa
							Moisture Content & Atterberg Limits Standard Penetration Test, blows/0.3m											
							W_p W W_L 10 20 30 40 50 60 70 80 90											
0			Forest Litter (TOPSOIL)														0	
			Yellow brown SILT, SAND, GRAVEL & COBBLES, loose														2	
1			Grey SILT & fine SAND, medium dense														4	
				GS 1-1	27												6	
2			-seepage at 1.8m														8	
			End of Test Pit at 2.1m														10	

Sample Type: GS - Grab Sample SPT - Standard Penetration Test
ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: ☒ Bentonite ☐ Pea Gravel ☒ Drill Cuttings ☐ Sand

Logged by: ME/RJ
Reviewed by: ME
Date: Feb. 217 - 05

 Jacques Whitford
Consulting Engineers
Environmental Scientists
Information Consultants

CLIENT CONCORDIA HOMES LTD. THE SHORES SUBDIVISION

PROJECT No. BCV40527

PROJECT Geotechnical Investigation

DATUM

LOCATION Sechelt, B.C.

ELEVATION

EXCAVATION DATE Feb. 17-05 CONTRACTOR McCourt Contracting Ltd. EXCAVATION METHOD Excavator

DEPTH (m)	USC SOIL SYMBOL	SOIL DESCRIPTION	SAMPLES			☐ Insitu Shear Vane (kPa) ☐ Remoulded Shear Vane (kPa) ☐ Pocket Penetrometer (kPa)				DEPTH (ft)
			TYPE	NUMBER	MOISTURE CONTENT (%)	50kPa	100kPa	150kPa	200kPa	
0	TP	Organics, Roots (TOPSOIL)								0
	GP	Band of yellow brown medium sandy GRAVEL, loose								
		Yellow brown medium SAND with gravel, some cobbles, loose to medium dense								2
1	SP									4
2		-rust stained layer								6
	ML	Yellow brown fine sandy SILT, some clay, trace gravel, stiff								8
			GS 2-1	22						
		End of Test Pit at 2.8m								
3										10

Sample Type: GS - Grab Sample SPT - Standard Penetration Test
 ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: ☒ Bentonite ☒ Pea Gravel ☒ Drill Cuttings ☐ Sand

Logged by: ME/RI
 Reviewed by: ME
 Date: Feb. 217 - 05



Jacques Whitford
 Consulting Engineers
 Environmental Scientists
 Information Consultants

CLIENT CONCORDIA HOMES LTD. THE SHORES SUBDIVISIONPROJECT No. BCV40527PROJECT Geotechnical Investigation

DATUM _____

LOCATION Sechelt, B.C.

ELEVATION _____

EXCAVATION DATE Feb. 17-05CONTRACTOR McCourt Contracting Ltd.EXCAVATION METHOD Excavator

DEPTH (m)	USC SOIL SYMBOL	SOIL DESCRIPTION	SAMPLES			Moisture Content & Atterberg Limits				DEPTH (ft)
			TYPE	NUMBER	MOISTURE CONTENT (%)	W_p	W	W_L	Standard Penetration Test, blows/0.3m	
0	TP	Organics, Roots (TOPSOIL)								0
	SM	Rusty Yellow silty medium SAND, trace gravel, loose to medium dense								2
1	TL	Yellow brown Glacial TILL (sandy SILT & GRAVEL with cobbles, some boulders), dense to very dense								4
2			GS 3-1	12						6
		End of Test Pit at 2.4m								8
3										10

Sample Type: GS - Grab Sample SPT - Standard Penetration Test
ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: ☒ Bentonite ☒ Pca Gravel ☒ Drill Cuttings ☐ Sand

Logged by: ME/RI
Reviewed by: ME
Date: Feb. 217 - 05

Jacques Whitford
Consulting Engineers
Environmental Scientists
Information Consultants

TEST PIT RECORD

TP05-04

CLIENT CONCORDIA HOMES LTD. THE SHORES SUBDIVISION PROJECT No. BCV40527
PROJECT Geotechnical Investigation DATUM _____
LOCATION Sechelt, B.C. ELEVATION _____
EXCAVATION DATE Feb. 17-05 CONTRACTOR McCourt Contracting Ltd. EXCAVATION METHOD Excavator

DEPTH (m)	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLES			☐ Insitu Shear Vane (kPa) ☐ Remoulded Shear Vane (kPa) ☐ Pocket Penetrometer (kPa)				DEPTH (ft)
				TYPE	NUMBER	MOISTURE CONTENT (%)	50kPa	100kPa	150kPa	200kPa	
0	TP		Organics, Roots (TOPSOIL)								0
	GP		Rusty brown silty medium SAND & GRAVEL, some cobbles & roots, loose to medium dense								
	GM										
			Yellow brown Glacial TILL (sandy SILT & GRAVEL with cobbles, some boulders), dense to very dense								2
1	TL										4
2			End of Test Pit at 1.8m								6
3											8
											10

Sample Type: GS - Grab Sample SPT - Standard Penetration Test
ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: ☒ Bentonite ☐ Pea Gravel ☐ Drill Cuttings ☐ Sand

Logged by: ME/RI
Reviewed by: ME
Date: Feb. 217 - 05



Jacques Whitford
Consulting Engineers
Environmental Scientists
Information Consultants

TEST PIT RECORD

TP05-05

CLIENT CONCORDIA HOMES LTD. THE SHORES SUBDIVISION PROJECT No. BCV40527
PROJECT Geotechnical Investigation DATUM _____
LOCATION Sechelt, B.C. ELEVATION _____
EXCAVATION DATE Feb. 17-05 CONTRACTOR McCourt Contracting Ltd. EXCAVATION METHOD Excavator

DEPTH (m)	USC SOIL SYMBOL	SOIL DESCRIPTION	SAMPLES			☐ Insitu Shear Vane (kPa) ☐ Remoulded Shear Vane (kPa) ☐ Pocket Penetrometer (kPa) 50kPa 100kPa 150kPa 200kPa W _p W W _L Moisture Content & Atterberg Limits ● Standard Penetration Test, blows/0.3m										DEPTH (ft)
			TYPE	NUMBER	MOISTURE CONTENT (%)											
0	TP	Organics, Roots (TOPSOIL)														0
	SP	Rusty brown fine to medium SAND, loose to medium dense														
1	TL	Yellow brown Glacial TILL (sandy SILT & GRAVEL with cobbles, some boulders), dense to very dense														2
		End of Test Pit at 1.5m														4
2																6
																8
3																10

Sample Type: GS - Grab Sample SPT - Standard Penetration Test
ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test
Piezometer
Backfill Type: ☒ Bentonite ☒ Pea Gravel ☒ Drill Cuttings ☐ Sand

Logged by: ME/RI
Reviewed by: ME
Date: Feb 217 - 05



Jacques Whitford

Consulting Engineers
Environmental Scientists
Information Consultants

APPENDIX B

SUMMARY OF STABILITY ANALYSES

Table of Calculated Factors of Safety

Slope Stability Run #	Slope Configuration	Case Analysed	Calculated Factor of Safety
3	From WEB dwg	<i>Back-analysis of mid-Sept landslide at L-100 grade</i>	<i>1.010</i>
4	From WEB dwg	Analysis of slope, modelling possible tension crack at L-200 grade	1.178
5	From WEB dwg	Analysis of slope, modelling plane of weakness at L-150 grade	1.028
6	From WEB dwg	Run #4, but with Silt $\phi = 30$	1.120
7	From WEB dwg	Run #5, but with Silt $\phi = 30$	0.985
8	Overall Slope	Analysis of overall slope between the L-100 and L-150 grades	1.541
9	Overall Slope	Run #8, but with Silt $\phi = 30$	1.382
10	Overall Slope	Analysis of overall slope between the L-100 and L-200 grades	1.438
11	Overall Slope	Run #8, but with Silt $\phi = 30$ – "smaller slip surface" than Run #9	1.535
12	Overall Slope	<i>Back-analysis of slope, incorporating the old logging road at the L-150 grade, with Silt $\phi = 30$</i>	<i>0.791</i>
13	Overall Slope	Analysis of slope (incorporating the old logging road) between the L-100 and L-200 grades, with Silt $\phi = 30$	1.168
14	Overall Slope	<i>Back-analysis of slope, incorporating the old logging road at the L-150 grade, with Silt $\phi = 32$</i>	<i>0.875</i>
15	Overall Slope	Analysis of slope (incorporating the old logging road) between the L-100 and L-150 grades, with Silt $\phi = 32$	0.932