J.R. Paine & Associates Ltd.

CONSULTING AND TESTING ENGINEERS
EDMONTON — GRANDE PRAIRIE — PEACE RIVER — WHITEHORSE

17505 106 Avenue
Edmonton, Alberta
T5S 1E7

May 4, 2009
File No. 2461 – 412

LANCE SKINNER
c/o ISL Engineering and Land Services Ltd.
Suite 100, 7909 - 51st Avenue
Edmonton, Alberta
T6E 5L9

ATTENTION: Mr. Darin Hicks, P.Eng.

Dear Sir:

Re: Geotechnical Investigation
Proposed Highland Park Subdivision
Lot 2, Block 1, Plan 0620435
Part of NE 17 – 39 – 1 – W5M
Near Sylvan Lake, Alberta

Please find enclosed our report with respect to the above noted investigation. In brief, this report presents the general soil conditions and construction recommendations concerning the geotechnical aspects of the proposed residential development.

Thank you for the privilege of providing this service to your organization. We will be pleased to meet with you to review the contents of this report at your convenience.

Yours truly,
J.R. PAINE & ASSOCIATES LTD.

John Tsoi, E.I.T.

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EDMONTON 780-489-0700
GRANDE PRAIRIE 780-532-1515
PEACE RIVER 780-624-4966
WHITEHORSE 867-668-4648

REPORT NO: 2461 - 412

GEOTECHNICAL INVESTIGATION
PROPOSED HIGHLAND PARK SUBDIVISION
LOT 2, BLOCK 1, PLAN 0620435
PART OF NE 17 – 39 – 1 – W5M
NEAR SYLVAN LAKE, ALBERTA

MAY 2009

J.R. PAINE & ASSOCIATES LTD.
17505 – 106th Avenue
Edmonton, Alberta
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PHONE: **780-489-0700**
FAX: **780-489-0800**

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PROPOSED HIGHLAND PARK SUBDIVISION
LOT 2, BLOCK 1, PLAN 0620435
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NEAR SYLVAN LAKE, ALBERTA

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GEOTECHNICAL INVESTIGATION

PROJECT: Geotechnical Investigation
Proposed Highland Park Subdivision

LOCATION: Lot 2, Block 1, Plan 0620435
Part of NE 17 – 39 – 1 – W5M
Near Sylvan Lake, Alberta

CLIENT: Lance Skinner
C/O ISL Engineering and Land Services Ltd.
Suite 100, 7909 - 51st Avenue
Edmonton, Alberta
T6E 5L9

ATTENTION: Darin Hicks, P. Eng.

1.0 INTRODUCTION

This report presents the results of the subsurface investigation made within the proposed Highland Park subdivision near Sylvan Lake, Alberta. The project is understood to consist of a fully serviced residential subdivision of mainly single-family houses with basements. The objective of the investigation was to determine the general subsurface soil conditions in order to provide geotechnical recommendations for planning and design aspects of the project.

As part of this study, a slope stability assessment was conducted where applicable within the subject site. Previous land use and environmental issues are beyond the scope of this report. Authorization to proceed was granted by Darin Hicks, P. Eng of ISL Engineering and Land Services Ltd. (ISL) in February 2009.

2.0 SITE DESCRIPTION

The subject site was situated north of Sylvan Lake and roughly 1.6 kilometres west of Highway 20. The site consisted of approximately the eastern most one third of the NE 17 – 39 – 1 – W5M quarter section. The site was fenced on all four sides. The site was located directly west of Range Road 14, and was surrounded by neighbouring farmlands to the south, west and north. A pipeline right of way ran across the northwest corner of the site. Another pipeline was located approximately 60 metres west of and parallel to Range Road 14.

At the time of field investigation, the site was snow covered. Access to the site was gained via two entrances off Range Road 1-4. All wheel drive vehicles were required to travel on site.

Background Information Research

A contour map of the site provided by ISL showed elevations varied from below 970 metres to above 990 metres. The east central tree covered area and the southwest corner of the site were the highest and lowest points of the site respectively. The contour map showed the steepest part of the site was located within the north central portion of the site, between the high ground to the east and a gully to the west.

Several historic air photos of the site between 1962 and 2002 were compared and reviewed. The south half of the site was cleared and farmed before 1962 and has since remained mostly unchanged. However, the entire north half of the site was densely tree covered in 1962. By 1975, the pipeline that runs through the northwest corner of the site was visible. Tree coverage on site was largely reduced and replaced with sparse shrub and grass coverage with odd bare spots. A near rectangular clearing at the northern edge of the site was noted (Area 1 in Figure 1 in Appendix A). In 1982, rows of trees emerged in previously cleared areas. A pond was spotted within the rectangular clearing (Area 1). In 2002, an access road within the treed area was noted.

No coal mining information in the area was found in the Alberta Coal Mine Atlas made available by the Energy Resources Conservation Board.

Site Inspection

The topography of the site was visually examined for slope stability issues on April 1, 2009. Overall, the site terrain was considered hummocky to undulating. As mentioned in the air photos review, the site can be divided into two halves.

The north half of the site was mainly treed covered with two major areas of clearings (labelled Area 1 and Area 2 in Figure 1 in Appendix A). The tree cover consisted of mostly deciduous species and with little to no evergreen. Water accumulation was noted within the clearing of Area 1. Inside the treed area, a relatively narrow ravine runs parallel to the pipeline in the southwest direction toward Sylvan Lake. A shop structure was located on the high ground at the eastern edge of Area 2 clearing. Rows of trees were noted within the clearing of Area 2. Benches were also noted on the north facing slope within Area 2. In areas around Area 2, the south and west facing slopes were mostly vegetated with grass and scrubs while the top half of the north facing slopes were vegetated with medium dense tree cover.

The south half of the site consisted of mostly open crop field with an isolated treed area located adjacent to Range Road 14. The site was generally dipping toward the south and west.

3.0 FIELD INVESTIGATION

The subsurface soil investigation was undertaken in March 2009 utilizing a truck mounted drill rig owned and operated by SPT Drilling Ltd. Eight testholes were drilled to depths ranging from approximately 4.7 to 13.0 metres. Using a contour map provided by ISL, J. R. Paine And Associates Ltd. (JRP) personnel selected the testhole layout prior to drilling. Testhole elevations were surveyed by JRP personnel using a rod and a level. The approximate locations of all testholes are shown in Figure 2 in Appendix A. Due to limited access, no soil information was obtained in the treed area.

All testholes were advanced with 150 millimetre diameter solid stem augers in 1.5 metre increments. Continuous visual descriptions, including the soil types, depths, moisture, transitions, and other pertinent observations, were recorded on site. Soil samples were collected at 750 millimetre intervals for laboratory testing. Standard Penetration Tests (SPT) c/w split spoon sampling were taken at regular 1.5 metre intervals on all testholes. Where suitable soil was encountered, Shelby Tube sampling was taken instead of SPT.

Following the drilling operation, slotted piezometric standpipes were installed in all testholes for watertable level measurement. The testholes were backfilled with cuttings and bentonite was placed near the surface to prevent surface water infiltration.

4.0 LABORATORY TESTING

Soil samples retrieved from augers were bagged and returned to the laboratory for further testing. All samples were tested for moisture content. Representative cohesive samples were also tested to determine the liquid and plastic Atterberg limits, as well as soluble soil sulphate concentrations. Selective cohesionless samples were tested to determine the grain size distribution. Undisturbed Shelby tube samples were tested for dry density and unconfined compressive strength. The results of all laboratory testing and field observations are provided on the attached testhole logs.

5.0 GEOLOGICAL SETTING AND SOIL CONDITIONS

By locating the site on the geological maps made available by the Alberta Geological Survey, the local surficial geology of the site was identified as draped moraine deposit on bedrock. Such draped moraine deposits were described as till of uneven thickness, with minor amounts of water-sorted material, and localized undifferentiated subglacially moulded deposit with streamlined features. Side notes on the geological maps suggested that the undulating surface reflects the topography of the underlying bedrock. The general bedrock geology in the region was identified as the Upper Paskapoo Formation of Paleocene age. This Upper Paskapoo Formation was described as to consist of grey to greenish grey thick bedded calcareous cherty sandstone, grey and green siltstone and mudstone, minor conglomerate, thin limestone, coal and tuff beds.

As noted in the geological maps, bedrock depth was shallow. As a result, drilling was difficult. Half of the testholes experienced auger refusal. Testholes 09-2 and 09-8 encountered cobbles or boulders at shallow depths and were twice moved and redrilled. Soils encountered on site were considered similar to the surficial geology described above. In general, topsoil covered the ground surface, followed by uneven clay and sand deposits, overlain bedrock. Detailed soil description of each testhole in accordance of the Modified Unified Soil Classification System can be found in the attached testhole logs in Appendix B. Soil strata are summarized as follows:

Topsoil

Topsoil between roughly 100 and 150 millimetres in thickness was encountered at the ground surface. The topsoil materials were clayey, frozen, brown, and contained trace gravel. It is emphasized that topsoil depths are only estimated at the testhole locations and may vary significantly away from testhole locations.

Near Surface Deposits (Over Burden)

Layers of clay, clay till, and sand were encountered near the surface below topsoil. The thickness of the surficial deposit appeared to vary with topography, with increasing thickness in lower grounds.

In general, the near surface clay and clay till were brown to greyish brown, medium plastic, damp to moist, frozen to roughly 1.5 metres, and contained trace coal and gravel. SPT "N" values between 14 and 21 blows per 300 millimeters of penetration were recorded, representing stiff to very stiff consistency. Testing of a Shelby Tube sample revealed an unconfined compressive strength of roughly 241 kilopascal for the clay till.

The near surface sand deposits were typically brown, clayey, fine to medium grained, moist, and contained trace gravel. SPT "N" values between 8 and 34 blows per 300 millimeters of penetration were recorded, indicating loose to dense density. In Testholes 09-2 and 09-7, the near surface clayey sands appeared to be highly weathered sandstones. Below 2.0 to 3.0 metre of the stratum surface, the clayey sands became very dense and were described as bedrock like in appearance.

Bedrock

Bedrocks such as clay shale, siltstones, and sandstones, were encountered in all testholes. The top of bedrock was as shallow as 1.8 metres BGS in Testhole 09-4. Drilling was slow and difficult in the bedrock layers. Samples of clay shale, sandstone and siltstones were ground up on the auger and highly disturbed. Typically, the bedrock encountered was low to medium plastic, dry to moist, and very hard. SPT "N" values from 39 to well over 50 blows per 300 millimeters of penetration were recorded, indicating a dense to very dense density.

Upon completion of drilling, no water seepage was observed in any of the testholes, only minor sloughing conditions due to excessive grinding were noted in the bedrock layers.

6.0 GROUNDWATER CONDITIONS

Watertable readings were taken within 4 weeks after the completion of drilling. For all practical purposes, only the highest recorded levels would be considered as the stabilized levels. The watertable readings and the corresponding elevations are summarized below in Table 1. None of the testholes intercepted the watertable.

Table 1: Watertable Measurements

Testholes	Water Table Depth (m)		Ground Elevation (m)	Watertable Elevation (m)
	March 18, 2009	April 1, 2009		
09 - 1	Dry at 8.86		987.42	Below 978.56
09 - 2	Dry at 4.97		981.24	Below 976.27
09 - 3	Dry at 7.49		970.91	Below 963.42
09 - 4	Dry at 12.35	Buried	988.04	Below 975.69
09 - 5	Dry at 6.75		983.89	Below 977.14
09 - 6	Dry at 10.29		967.46	Below 957.17
09 - 7	Dry 4.66		984.26	Below 979.60
09 - 8	Dry 6.47		991.29	Below 984.82

7.0 RECOMMENDATIONS

7.1 Slope Stability

Four cross sectional profiles were extrapolated from the contour map provided by ISL. The profile locations (labelled Profiles A to D in Figure 3 in Appendix A) were chosen to represent the steepest parts of the site. Each cross sectional profile was modelled and analyzed using GSLOPE software. Based on visual examination and laboratory testing, main soil strata were identified and soil properties including unit weights and effective strength parameters were estimated. Only the long term drained soil strength values were considered in the modelling and are considered suitably conservative.

The bedrock depths were kept at 5.5 and 2.5 metres at the bottom and the top the slope profile respectively to reflect the geological characteristics of the site. The strength for the upper 5.0 metres of the bedrock stratum was reduced to model local bedrock weakening effects that include possible fissures, weathering, or rebound.

To account for long term rise in groundwater level due to urban development, the groundwater level was raised to match the bottom of slope surface of each cross section.

The GSLOPE software generates an Interactive Limit Equilibrium Slope Stability Analysis using Bishop's method of slices. With this modelling, a factor of safety (FOS) against failure is given for a specific slip surface. Circular and translational slip surfaces were analyzed. Only the slip surface with the lowest FOS, known as the critical slip surface, would be utilized in geotechnical evaluation of each profile. The area behind the critical slip surface with a minimum FOS of 1.5 would be considered geotechnical satisfactory for residential development. Graphical outputs of the GSLOPE analysis are located in Appendix C.

Based on the GSLOPE analysis conducted, as outlined in this report, and a 7.5 metres building setback from the proposed rear property line was assumed, the slopes evaluated do not present a natural hazard to the development.

No signs of failure or active erosion were observed. Mature medium dense tree or grass coverage was present throughout the site slopes. Overall, the slopes appeared stable with no signs of sliding. It is critical that the design, construction, and ongoing maintenance of the development do not adversely affect the ground conditions. The following development recommendations are directly related to maintenance of continued slope stability.

1. Aggressive lot grading that may produce fill depth in excess of 1.0 metre near any side slope would induce a surcharge load to the slope and should be reviewed by JRP prior implementation.
2. Infiltrating surface water will make slopes less stable. Sources of water should not be allowed on Lots 12 to 15 as shown in Figures 2 and 3 in Appendix A. Sources of water include underground automatic watering, swimming pools, ornamental ponds, or other water storage items. Lot grading should be designed to eliminate accumulation of surface water.
3. Tree clearing within the proposed lots as shown in Figures 2 and 3 in Appendix A should not be a concern. However, surface erosion should be prevented on any side slope where bare grounds would be exposed. To prevent erosion, vegetation should be strategically planted with the use of native species recommended. Surface runoff from roof leaders should be directed to erosion resistant swales or ditches.

7.2 Site Grading

1. Between approximately 100 and 150 millimetres of topsoil and marginal organic soil were encountered. All topsoil and marginal materials were considered unsuitable to support footing foundations, basement slab-on-grade, and roadways. Topsoil should be stripped away within the vicinity of roadways and building footprints. Conventional clearing and stripping should be suitable for most part of the site. Topsoil should not be reused as fill. Topsoil should be stockpiled, separated from the inorganic soil, and reused for landscaping purposes only.

2. Engineered fill may be considered in areas where low elevations necessitate deep fill zones. This option should be reviewed by our firm prior to construction to assess slope and other issues, and evaluate sources of fill material on site and any potential off-site borrow sources. Proper placement of engineered fill will negate the need for pile foundations in deep lot fill areas.

   Engineered fill is soil that is placed in a controlled manner under the full-time monitoring and extensive testing by a qualified soil technician. All organic soil and non-engineered fill must first be removed from the engineered fill area. Any areas of soft native subgrade encountered during fill construction should be evaluated by our firm on site. Engineered fill should be placed in lifts of maximum 150 millimetres of compacted thickness, and compacted to a minimum 98 percent of the corresponding Standard Proctor Density near the optimum moisture content. Each lift should compose of suitable uniform inorganic soil.

3. Due to the uneven nature of the existing ground, lot grading may produce fill areas on or near existing side slopes. Fill depths over 1.0 metre would induce a surcharge load to the slope and should be reviewed by our firm on a case by case basis prior to implementation.

4. Engineered fill depth differential of less than 1.5 metres is required to utilize footing foundation. In some cases, removal of native material may allow for the minimum fill depth or the maximum fill differential conditions to be met.

5. The near surface medium plastic clay, clay till, sand, and bedrock encountered throughout the site would be suitable as fill material. Bedrock should be adequately pulverized.

7.3 Housing Foundations

1. Topsoil is considered unsuitable for footing and basement slab on grade support. Undisturbed near surface inorganic soils encountered throughout this site are considered satisfactory for supporting typical wood framed single-family residential units utilizing standard concrete footing foundations.
2. The medium plastic clay encountered near the surface in all testholes has a low to moderate swelling potential. The sand encountered in Testholes 09-1, 09-2, 09-3 and 09-7 is slightly frost susceptible. To minimize foundation movement due to swelling and shrinkage, basement excavations should be protected from drying, rain, snow, freezing and the ingress of surface or groundwater. The time span between the start of excavation to installation of basement footings, walls, peripheral weeping tile and backfilling operations should be minimized.
3. No loose, disturbed, remolded or slough material should be allowed to remain in the open footing excavations. Hand cleaning is advised if an acceptable surface cannot be prepared by mechanical equipment. In order to reduce the disturbance to the bearing surface, all basement excavations should be advanced by a backhoe operating remotely from the bearing surface.
4. If the design lot grading will produce areas of fill extending to depths below that of footing elevations, it is not recommended that footings be constructed on non-engineered fill. In such cases, one of the following alternatives can be considered:
    - i) Removal of the fill and replace with a compacted coarse clean granular material, or concrete. A normal footing foundation may then be utilized.

      or

    - ii) Utilize a pile foundation.

5. The soils encountered at this site are generally considered suitable for cast-in-place pile foundation although some installation difficulty is anticipated due to shallow bedrock. Houses may be founded on an adequately reinforced grade beam or pile cap supported by bored, cast-in-place, concrete piles. The design capacity can be calculated on the basis of allowable skin friction values. The allowable skin friction values should be determined on a lot by lot basis.

   It is possible that sloughing will occur in the pileholes due to presence of sand if they are left open for extended periods of time. All concrete should be placed immediately after drilling in order to minimize the potential of sloughing condition. Pile casing should be available on site to control or sloughing conditions if necessary.

6. To compensate the possible swelling of the subsoil beneath the pile caps and the effects of frost action, void form or other means to allow soil expansion beneath the grade beams and pile caps are recommended. Also, all piles should be adequately reinforced. Concrete for all piles should be adequately vibrated.

7. Grade supported concrete slabs that are founded on uncontrolled fills may be subjected to excessive vertical movements. In areas of uncontrolled fill, slabs may need to be structurally supported. Structurally supported slabs will require a structural engineer to design.

8. A non-deteriorating vapour barrier should be placed immediately below the floor slab to prevent desiccation of the subgrade material. The subgrade should be a 150 millimetre thick layer of free draining sand or sand-gravel mixture uniformly compacted to 98 percent of the corresponding Standard Proctor Density at optimum moisture content.

9. Typical foundation drainage measures are recommended in all residential lots. At a minimum, peripheral exterior weeping tile lines are recommended for all houses. All lines should be placed at or slightly below footing elevation and connected to ensure positive drainage to an approved system. A minimum 150 millimetres of suitable clean tile rock should be placed around and above the weeping tile lines and separated from the clay backfill by a layer of filer cloth.

10. It is recommended that floor joists be placed prior to backfilling the excavation in order to minimize any detrimental effects on the foundation walls caused by backfilling operations.
11. During cold weather construction, it is essential that all interior fill and load bearing materials remain frost-free. Recommended cold weather construction practices, with respect to hoarding and heating of the forms and the fresh concrete, should be followed. In order to minimize the potential frost heave problems, the interior of the building must be heated as soon as the walls have been poured. The period in which the excavation is left open due to freezing conditions should be as short as possible. If doubts remain as to the suitability of the foundation during construction, the builder should consult a qualified geotechnical engineer.

7.4 Underground Utilities

1. The near surface native inorganic soil encountered throughout the site is generally considered satisfactory for the installation of underground utilities. Topsoil and all other organic materials should be separated from the inorganic soils, and should not be re-used as trench backfill.
2. Very dense bedrock materials were encountered at depths of less than 3.0 metres. Ripping will be required and construction delay can be expected.
3. Open cut trench method of underground utility installation should be feasible to the testhole depths. Standard trenching cutback angles of approximately 30 degrees from the vertical are expected to be adequate for the near surface clay material on this site. Trenching in the sand materials may require increased cutback angles of 45 degrees or more in order to remain stable. The optimum cutback angles for utility trenches should be determined in the field during construction. Exact stable slope values cannot be pinpointed without detailed and extensive analysis. For this reason, this information should be used as a guideline only. Part 32 of the Occupational Health and Safety Regulation should be strictly followed, except where superseded by this report.

4. Temporary surcharge loads, such as spill piles, should not be allowed within 3.0 metres of an unsupported excavation face, while mobile vehicles should be kept back at least 1.0 metre. All excavations should be checked regularly for signs of sloughing or failures, especially after rainfall periods.
5. None of the testholes encountered immediate groundwater seepages. However, if trenches intercept isolated water bearing sand seam, minor dewatering effort may be required. Opening relatively long portions of utility trench should not be a concern for this site.
6. To minimize pipe loading, trench bottom width should be minimized but compatible with safe construction operations. The trench width must be wide enough to accommodate pipe bedding and compaction equipment.
7. Pipe bedding and trench backfill procedures should adhere to the Lacombe County specifications. The backfill material beneath and above the pipe should be an approved bedding sand material where conditions allow. This material should be hand placed and hand tamped, with care taken to fill the underside of the pipe. If ingressing groundwater and saturated, soft material are encountered in trenches, washed rock and geotextile separator are recommended for pipe bedding. The washed rock and geotextile configuration should be determined in the field during construction. The need for this configuration will be low to nil for this site.
8. According to the Lacombe County Policy and Engineering Manual clause C19, trench backfill within any road right of way should be placed in lifts no more than 150 millimetres thick and compacted to minimum 98 percent of the corresponding Standard Proctor Density. However, JRP believes maximum lift thickness of 300 millimetres would be acceptable below 300 millimetres from the top of subgrade.

   JRP recommends trench backfill away from road right of ways to be compacted to minimum 95 percent of the Standard Proctor Density in maximum 300 millimetre lifts. A low to moderate amount of moisture conditioning is expected for the near surface clay soil to achieve the specified degree of compaction. All trench backfill material should be uniform inorganic soil.
9. Bedrock soil excavated from trenches should be pulverized. Failure to ensure adequate pulverization and moisture conditioning may result in settlement or subgrade softening of

the trench backfill. Mixing of clay, sand and bedrock soils will cause a corresponding variability in the utility trench backfill conditions. It may be difficult to work with the sand soils in the testholes because the sands are sensitive to moisture content. Also, mixing sands and clays can make compaction of the mixture difficult.

10. It should be noted that the ultimate performance of the trench backfill is directly related to the consistency and uniformity of the backfill compaction, as well as the contractor's underground construction procedures. In order to achieve this uniformity, the lift thickness and compaction criteria should be strictly enforced. The quality of the trench backfill compaction affects the subgrade and pavement design.

7.5 Surface Utilities

1. The subsurface inorganic soil conditions encountered throughout this site are considered generally satisfactory for the construction of roads, curbs, and sidewalks. Topsoil and all other deleterious materials, should be removed prior to construction of roads, embankments, sidewalks and other surface utilities.
2. Placement of all fill material within road areas should meet the requirements specified in item 7.4.8. The near surface sands encountered were of moderate to high frost susceptibility, with the other testhole soils were of low to moderate frost susceptibility. Given the low watertable condition of the site, the risk of frost heave is low and no action is required.
3. The minimum subgrade preparation recommended is scarification to 150 millimetres depth and recompacted to a minimum 100 percent of Standard Proctor Density at optimum moisture content. The subgrade should be inspected and proof rolled after final compaction and any areas showing visible deflections should be repaired prior to paving. A minimum cross slope of 2.0 percent should be maintained for the subgrade to ensure positive drainage within the gravel base.
4. Although no soft materials were encountered in any of the testholes, improperly compacted fill and poor weather would also produce soft subgrade. If soft subgrade is encountered during construction, our firm should be contacted to assess the soil conditions. Increased subgrade treatment, cement stabilization, or increased pavement

structure may be required. Exact measures in any soft subgrade areas should be decided on site during construction.

5. The following chart shows the recommended pavement designs. An estimated California Bearing Ratio (CBR) of 3.0 percent is used in the design, as well as a design life of 20 years. Due to the lack of traffic loading information, local residential traffic loading of 9.0 x 10^4 ESALs (Equivalent Single Axle Load) was assumed. An additional 2.0 x 10^4 ESALs was estimated to account for routine wastewater hauling traffic. The following JRP recommended pavement structures are comparable to the designs provided in the Lacombe County Planning and Engineering Manual.

Table 2: Roadway Structures

Material	Local Residential		Wastewater Truck Route	
	Lacombe County Specified	JRP Recommended 9.0 x 10^4 ESALs	Lacombe County Specified	JRP Recommended 1.1 x 10^5 ESALs
Asphaltic Concrete M1	100 mm (2 lifts)	75 mm (1 lift)	125 mm (2 lifts)	100 mm (2 lifts)
Crushed Gravel (20 mm)	150 mm	250 mm	250 mm	250 mm
Note: M1 is Alberta Transportation mix design criteria Crush gravel should meet Lacombe County specifications				

6. Roadway embankment slopes of 4H : 1V would be considered geotechnically suitable for this site. Minimum ditch depth of 1.0 metre from top of subgrade to bottom of ditch would be considered adequate.

7.6 Groundwater Issues

1. The watertable level was below the bottom of all the testholes. The watertable level is low and should not be a concern for house, underground utility and road construction.

7.7 Storm Water Management Facilities

1. Testhole 09-3 was drilled within the proposed storm pond location. Excavation of the near surface clay, clay till, and sand encountered within the proposed storm pond area will likely be possible by scrapers. Excavation of bedrock will likely require ripping.

2. Inorganic soil excavated from the storm water pond would be suitable for engineered lot fill. Low to moderate amount of moisture conditioning, including drying and wetting, may be required. Bedrock should be adequately pulverized.
3. The near surface clay and clay till encountered in Testhole 09-3 exhibited low to moderate permeability. Such materials are considered suitable for water retention purposes. No liner should be necessary.
4. The sand encountered between the clay and clay till layers in Testhole 09-3 exhibits high permeability. If the proposed storm pond intercepts such highly permeable material, our firm should be contacted to inspect the material and provide additional pond lining recommendations. Near surface medium plastic clays encountered throughout the site can be utilized as liner material to plug side slopes or pond bottom.
5. To prevent water seepage through the cracks in the bedrock, the bottom of the pond where bedrock is encountered should be scarified to 200 millimetres depth, moisture conditioned to within 2.0 percent of the optimum level, and recompacted to 95 percent of the corresponding Standard Proctor Density.
6. Excavation side slopes of 4 H : 1 V should be geotechnically stable above the normal water level (NWL) for the near surface clays at the proposed storm pond location. Excavation side slopes of 5 H : 1 V below NWL should be adequate. Gentler slopes may be more desirable for maintenance, recreation and safety purposes. Side slopes near NWL should be protected from erosion.

7.8 <u>Cement</u>

1. Concrete used for all underground pipes must be constructed of C.S.A. Type HS, high sulphate resistant hydraulic cement.
2. All concrete for surface improvements such as sidewalks and curbs may be constructed using CSA Type GU, normal hydraulic cement.
3. Tests on selected soil samples indicated negligible concentrations of water soluble soil sulphates within the underlying deposits. Based on C.S.A. Standards A23.1-04, Type GU, normal hydraulic cement may be used for concrete in contact with the soil. However, the

need to use sulphate resistant hydraulic cement should be determine on a lot by lot basis. More testing is recommended.

8.0 CLOSURE

This report has been prepared for the exclusive and confidential use of Mr. Lance Skinner, ISL Engineering and Land Services Ltd., and their authorized agents. Use of this report is limited to the subject proposed residential subdivision only. The recommendations given are based on the subsurface soil conditions encountered during testhole drilling, current construction techniques and generally accepted engineering practices. No other warranty, expressed or implied, is made. Due to geological randomness of many soils formations, no interpolation of soil conditions between or away from the testholes has been made or implied. Soil conditions are known only at the testhole location. Should other soils be encountered during construction or other information pertinent becomes available, the undersigned should be contacted as the recommendations may be altered or modified.

The findings in the slope stability analysis are considered detailed and suitable for final design and construction. The owners of the proposed development should be aware that our slope assessment has endeavoured to describe the risk of developing at this site, and limit this risk with engineering analysis. However, all risk ascribed with the development on a slope cannot be eliminated and must be accepted by the owners.

We trust this information is satisfactory. If you should have any questions, please contact our office.

Respectfully Submitted,
J.R. PAINE & ASSOCIATES LTD.

John Tsoi

John Tsoi, E.I.T.

Reviewed by,

RICHARD A. EVANS

R Evans

May 4/09

Rick Evans, P. Eng.

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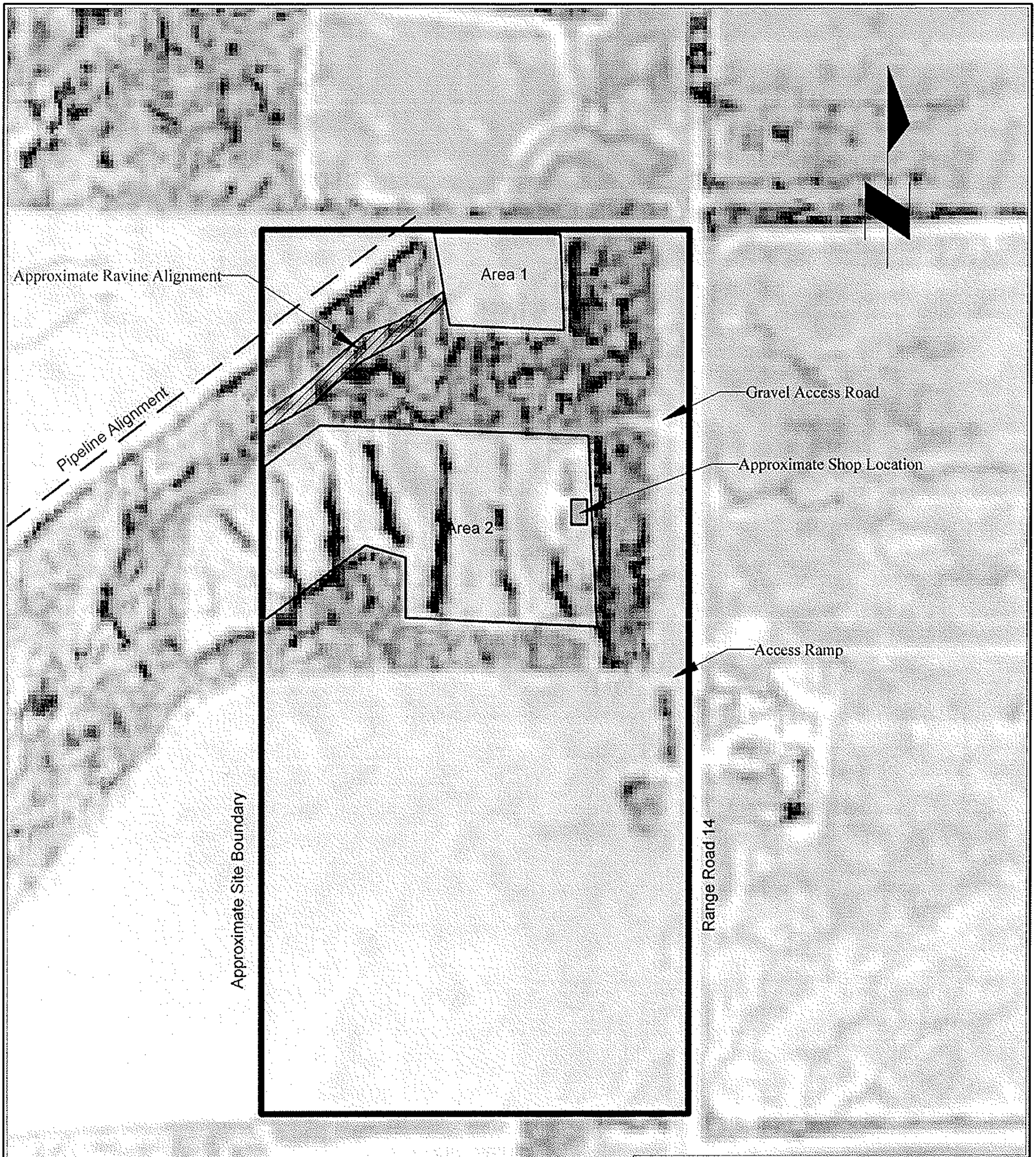
PERMIT TO PRACTICE
JR PAINE & ASSOCIATES LTD.
Signature R Evans
Date May 4/09
PERMIT NUMBER: P 0401
The Association of Professional Engineers, Geologists and Geophysicists of Alberta

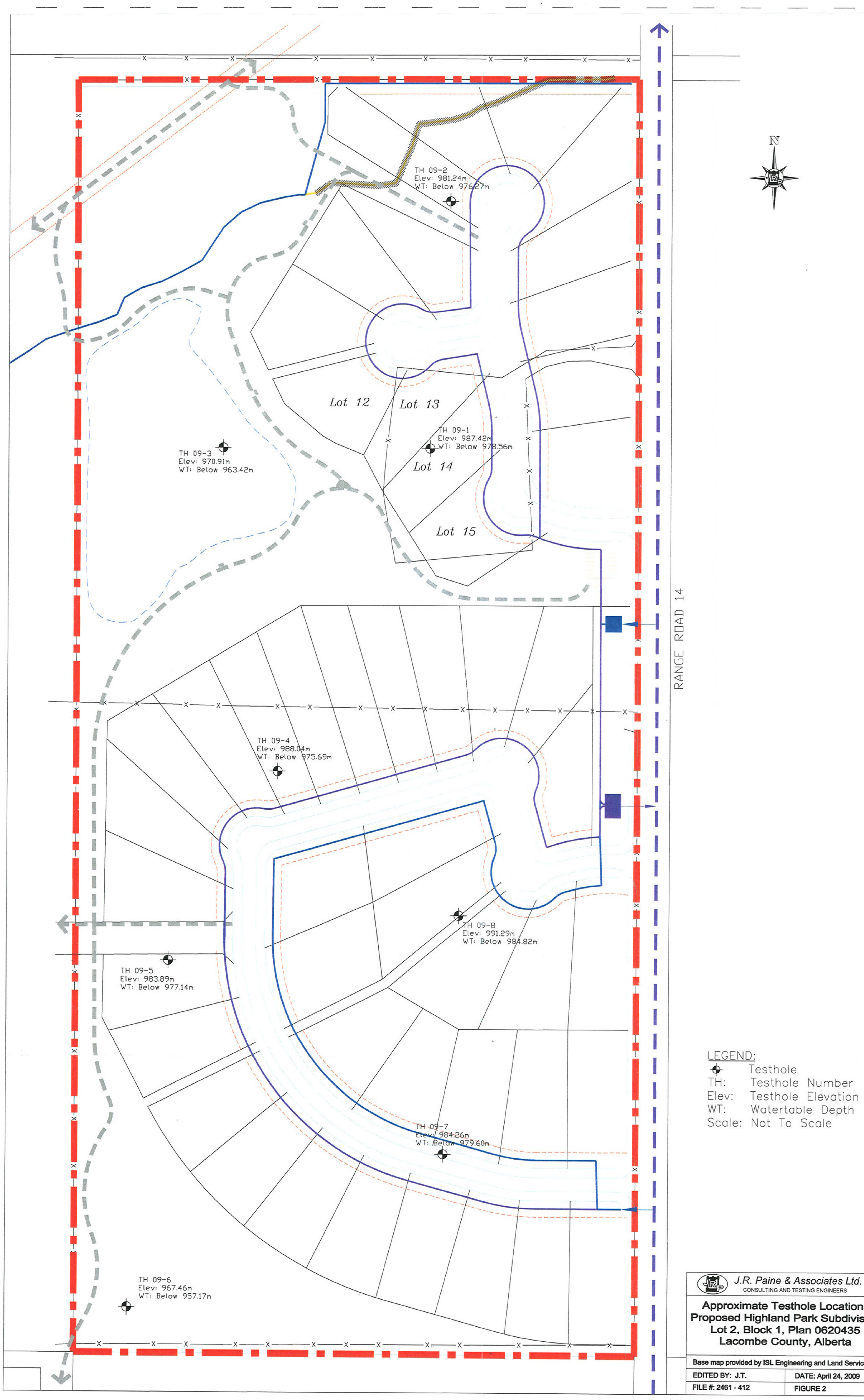
APPENDIX A



J.R. Paine & Associates Ltd.
CONSULTING AND TESTING ENGINEERS

2002 Air Photo
Proposed Highland Park Subdivision
Lot 2, Block 1, Plan 0620435
Near Sylvan Lake, Lacombe County

NOT TO SCALE	April 24, 2009
File: 2461 - 412	Figure 1



LEGEND:

⊕	Testhole
TH:	Testhole Number
Elev:	Testhole Elevation
WT:	Watertable Depth
Scale:	Not To Scale

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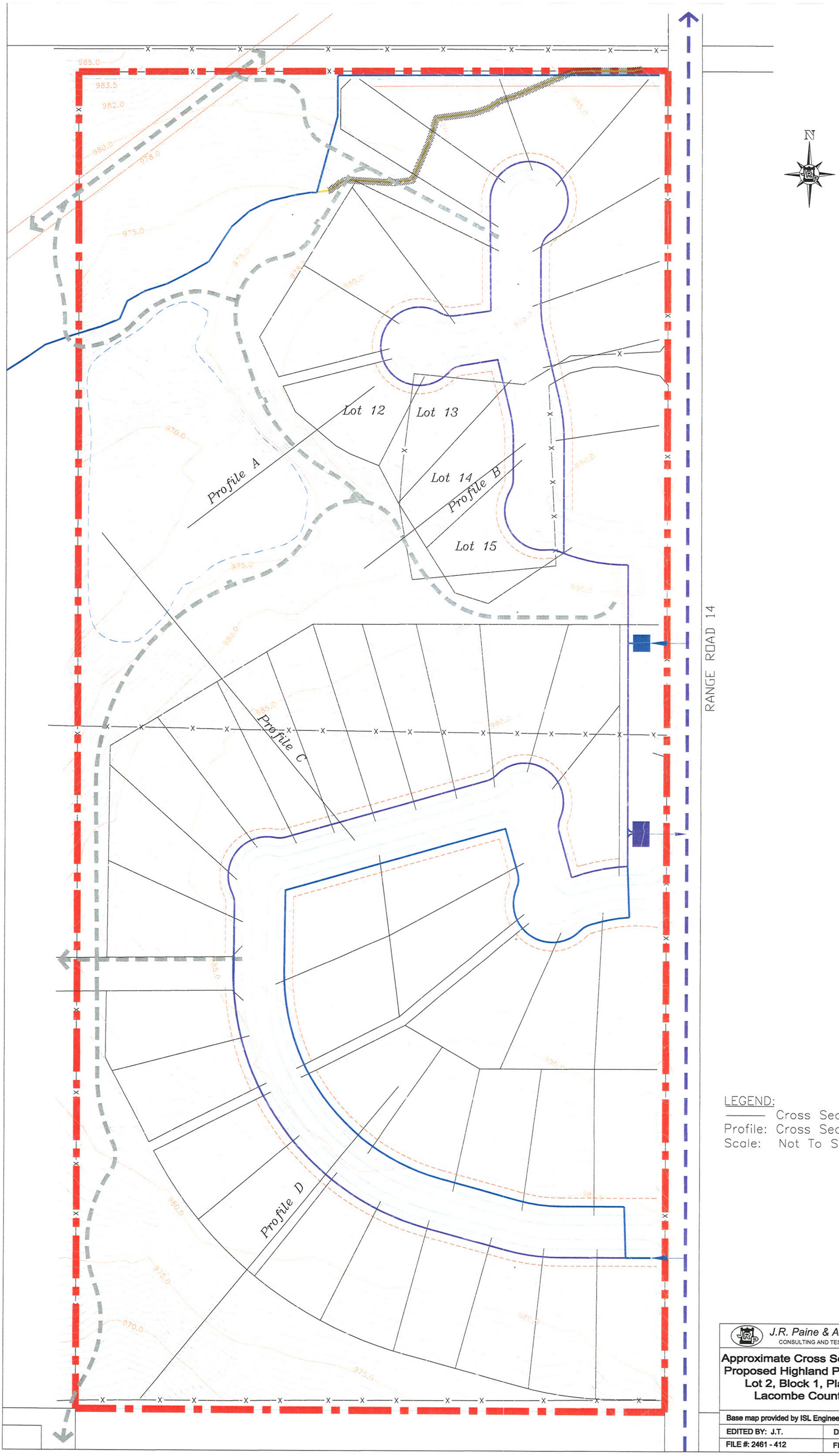
Approximate Testhole Locations
Proposed Highland Park Subdivision
Lot 2, Block 1, Plan 0620435
Lacombe County, Alberta

Base map provided by ISL Engineering and Land Services

EDITED BY: J.T.	DATE: April 24, 2009
FILE #: 2461 - 412	FIGURE 2



LEGEND:
──── Cross Section
Profile: Cross Section Name
Scale: Not To Scale

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Approximate Cross Section Locations
Proposed Highland Park Subdivision
Lot 2, Block 1, Plan 0620435
Lacombe County, Alberta

Base map provided by ISL Engineering and Land Services

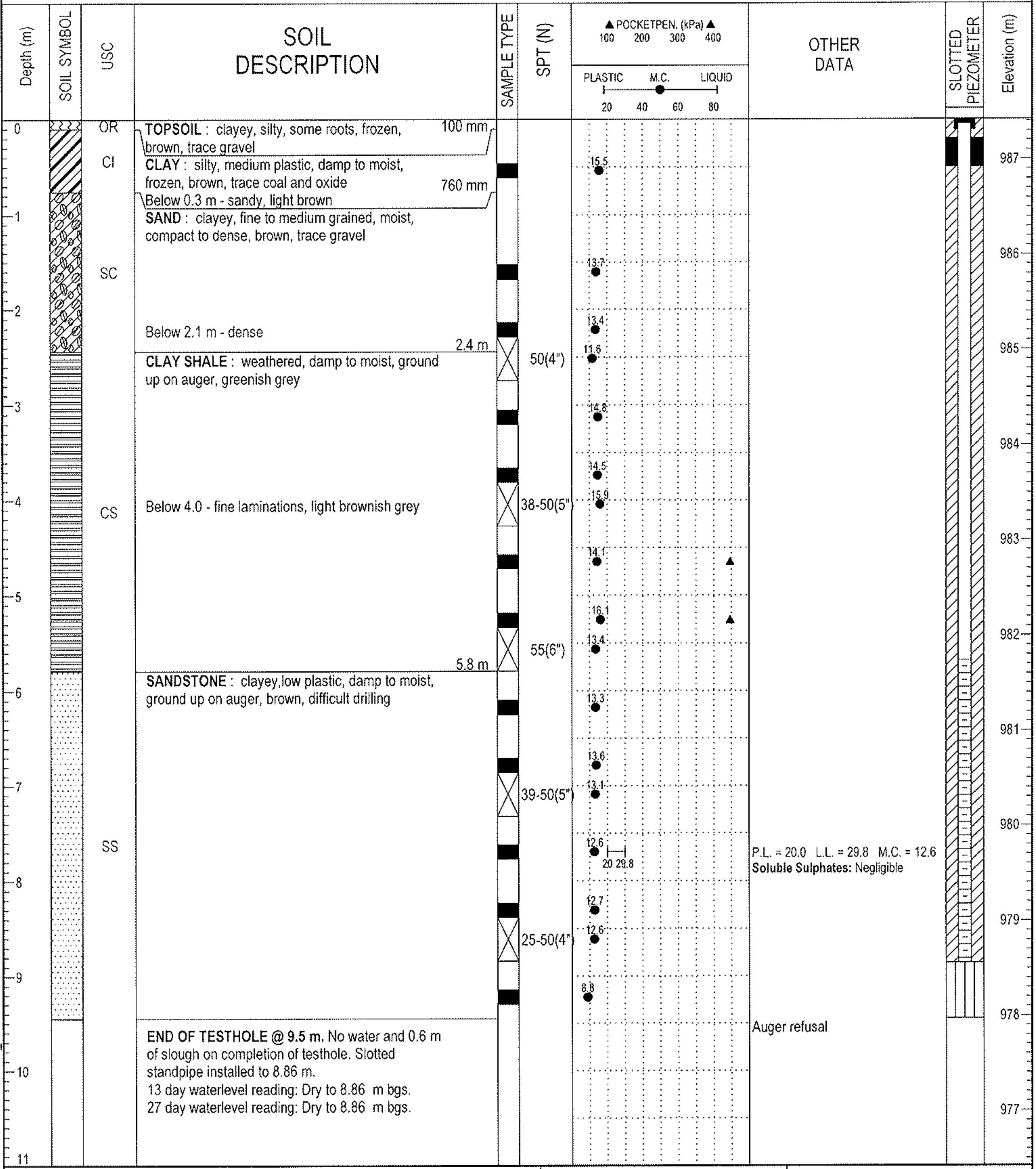
EDITED BY: J.T.	DATE: April 24, 2009
FILE #: 2461 - 412	FIGURE 3

APPENDIX B

PROJECT: Highland Park Subdivision	PROJECT NO: 2461 - 412	BOREHOLE NO: **09 - 1**
CLIENT: ISL Engineering and Land Services Ltd.	DRILL METHOD: Solid Stem Auger	ELEVATION: 987.42 m
OWNER: Lance Skinner	LOCATION: As per site plan	

SAMPLE TYPE ▨ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ■ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE ■ BENTONITE ⛶ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁘ SAND



TOPSOIL : clayey, silty, some roots, frozen, brown, trace gravel

CLAY : silty, medium plastic, damp to moist, frozen, brown, trace coal and oxide

Below 0.3 m - sandy, light brown

SAND : clayey, fine to medium grained, moist, compact to dense, brown, trace gravel

Below 2.1 m - dense

CLAY SHALE : weathered, damp to moist, ground up on auger, greenish grey

Below 4.0 - fine laminations, light brownish grey

SANDSTONE : clayey,low plastic, damp to moist, ground up on auger, brown, difficult drilling

END OF TESTHOLE @ 9.5 m. No water and 0.6 m of slough on completion of testhole. Slotted standpipe installed to 8.86 m.
13 day waterlevel reading: Dry to 8.86 m bgs.
27 day waterlevel reading: Dry to 8.86 m bgs.

P.L. = 20.0 L.L. = 29.8 M.C. = 12.6
Soluble Sulphates: Negligible

Auger refusal

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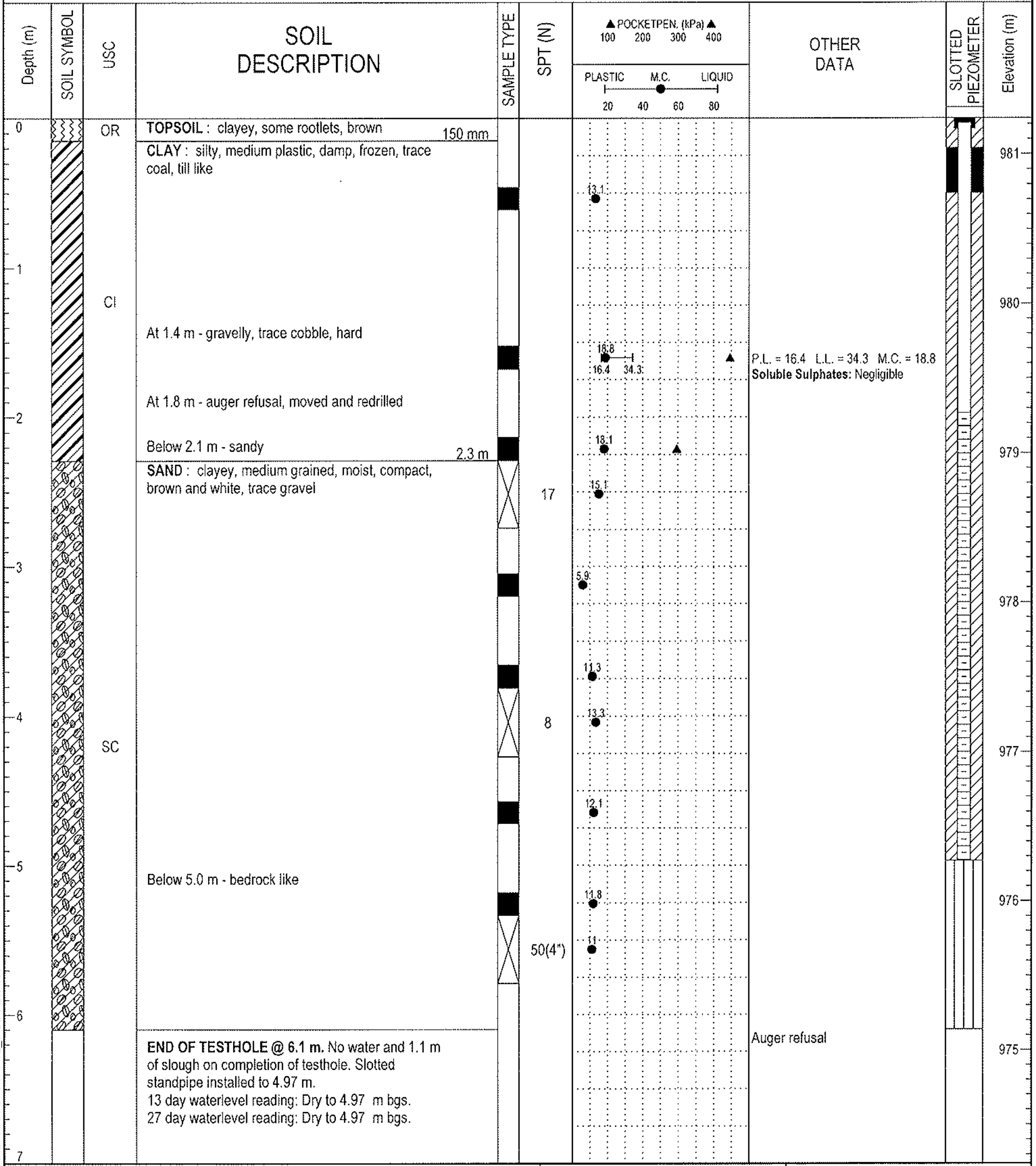
LOGGED BY: J Tsoi	COMPLETION DEPTH: 9.45 m
REVIEWED BY: R Evans	COMPLETION DATE: 5/3/09

PROJECT: Highland Park Subdivision
PROJECT NO: 2461 - 412
BOREHOLE NO: **09 - 2**
CLIENT: ISL Engineering and Land Services Ltd.
DRILL METHOD: Solid Stem Auger
ELEVATION: 981.24 m
OWNER: Lance Skinner
LOCATION: As per site plan

SAMPLE TYPE	▨ SHELBY TUBE	◩ CORE SAMPLE	☒ SPT SAMPLE	■ GRAB SAMPLE	▥ NO RECOVERY	
BACKFILL TYPE	■ BENTONITE	⁚ PEA GRAVEL	▥ SLOUGH	⁘ GROUT	▨ DRILL CUTTINGS	⁘ SAND



TOPSOIL : clayey, some rootlets, brown 150 mm

CLAY : silty, medium plastic, damp, frozen, trace coal, till like

At 1.4 m - gravelly, trace cobble, hard

At 1.8 m - auger refusal, moved and redrilled

Below 2.1 m - sandy 2.3 m

SAND : clayey, medium grained, moist, compact, brown and white, trace gravel

Below 5.0 m - bedrock like

END OF TESTHOLE @ 6.1 m. No water and 1.1 m of slough on completion of testhole. Slotted standpipe installed to 4.97 m.
13 day waterlevel reading: Dry to 4.97 m bgs.
27 day waterlevel reading: Dry to 4.97 m bgs.

P.L. = 16.4 L.L. = 34.3 M.C. = 18.8
Soluble Sulphates: Negligible

Auger refusal

JRP HIGHLANDPARK.GPJ JRPV2_3.GDT 29/4/09

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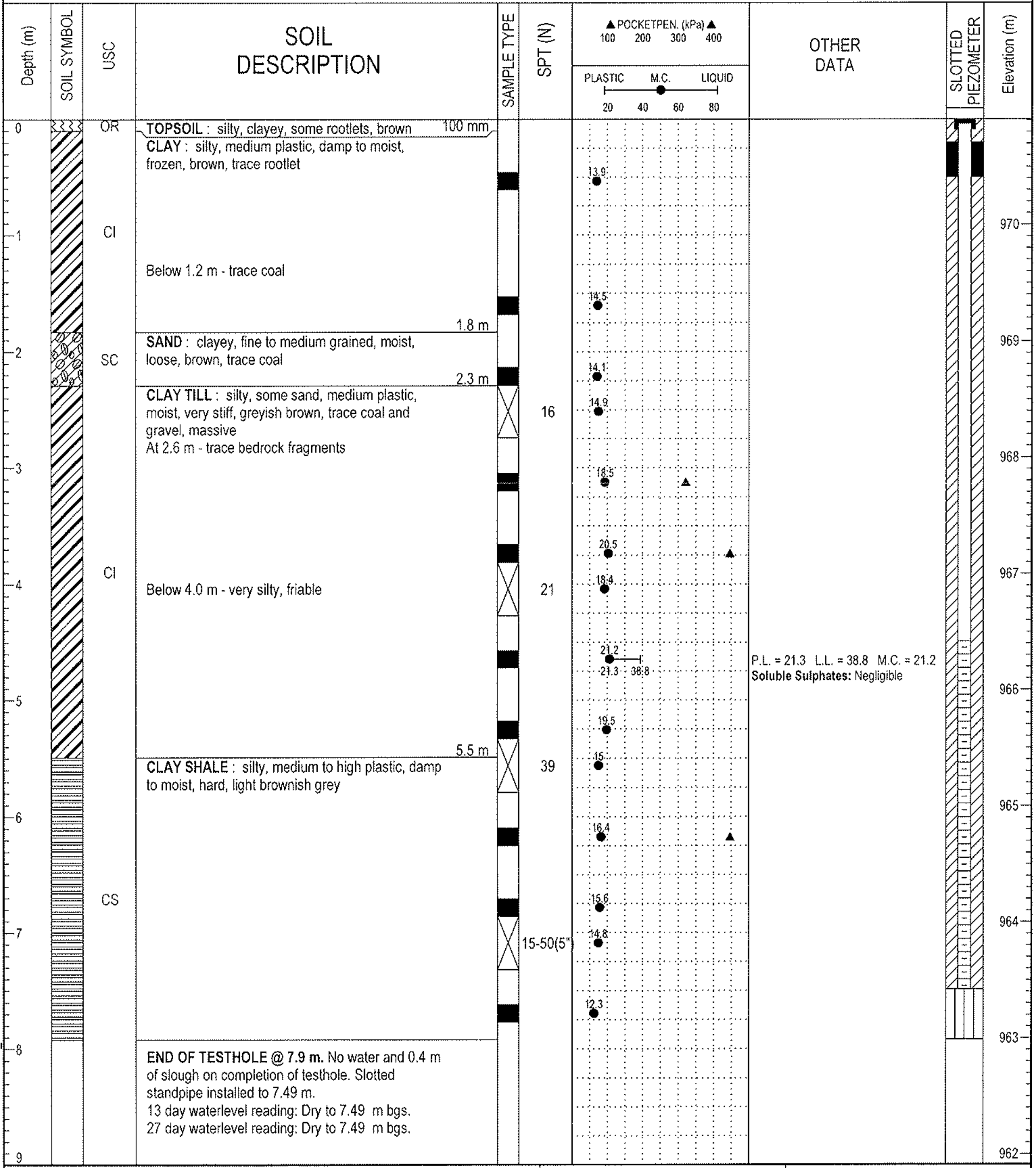
LOGGED BY: J Tsoi
REVIEWED BY: R Evans
COMPLETION DEPTH: 6.10 m
COMPLETION DATE: 5/3/09

PROJECT: Highland Park Subdivision	PROJECT NO: 2461 - 412	BOREHOLE NO: **09 - 3**
CLIENT: ISL Engineering and Land Services Ltd.	DRILL METHOD: Solid Stem Auger	ELEVATION: 970.91 m
OWNER: Lance Skinner	LOCATION: As per site plan	

SAMPLE TYPE: ▨ SHELBY TUBE ◪ CORE SAMPLE ☒ SPT SAMPLE ■ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE: ■ BENTONITE ⁘ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁘ SAND



SOIL DESCRIPTION

TOPSOIL : silty, clayey, some rootlets, brown 100 mm

CLAY : silty, medium plastic, damp to moist, frozen, brown, trace rootlet

Below 1.2 m - trace coal

1.8 m

SAND : clayey, fine to medium grained, moist, loose, brown, trace coal

2.3 m

CLAY TILL : silty, some sand, medium plastic, moist, very stiff, greyish brown, trace coal and gravel, massive

At 2.6 m - trace bedrock fragments

Below 4.0 m - very silty, friable

5.5 m

CLAY SHALE : silty, medium to high plastic, damp to moist, hard, light brownish grey

END OF TESTHOLE @ 7.9 m. No water and 0.4 m of slough on completion of testhole. Slotted standpipe installed to 7.49 m.
13 day waterlevel reading: Dry to 7.49 m bgs.
27 day waterlevel reading: Dry to 7.49 m bgs.

OTHER DATA

P.L. = 21.3 L.L. = 38.8 M.C. = 21.2
Soluble Sulphates: Negligible

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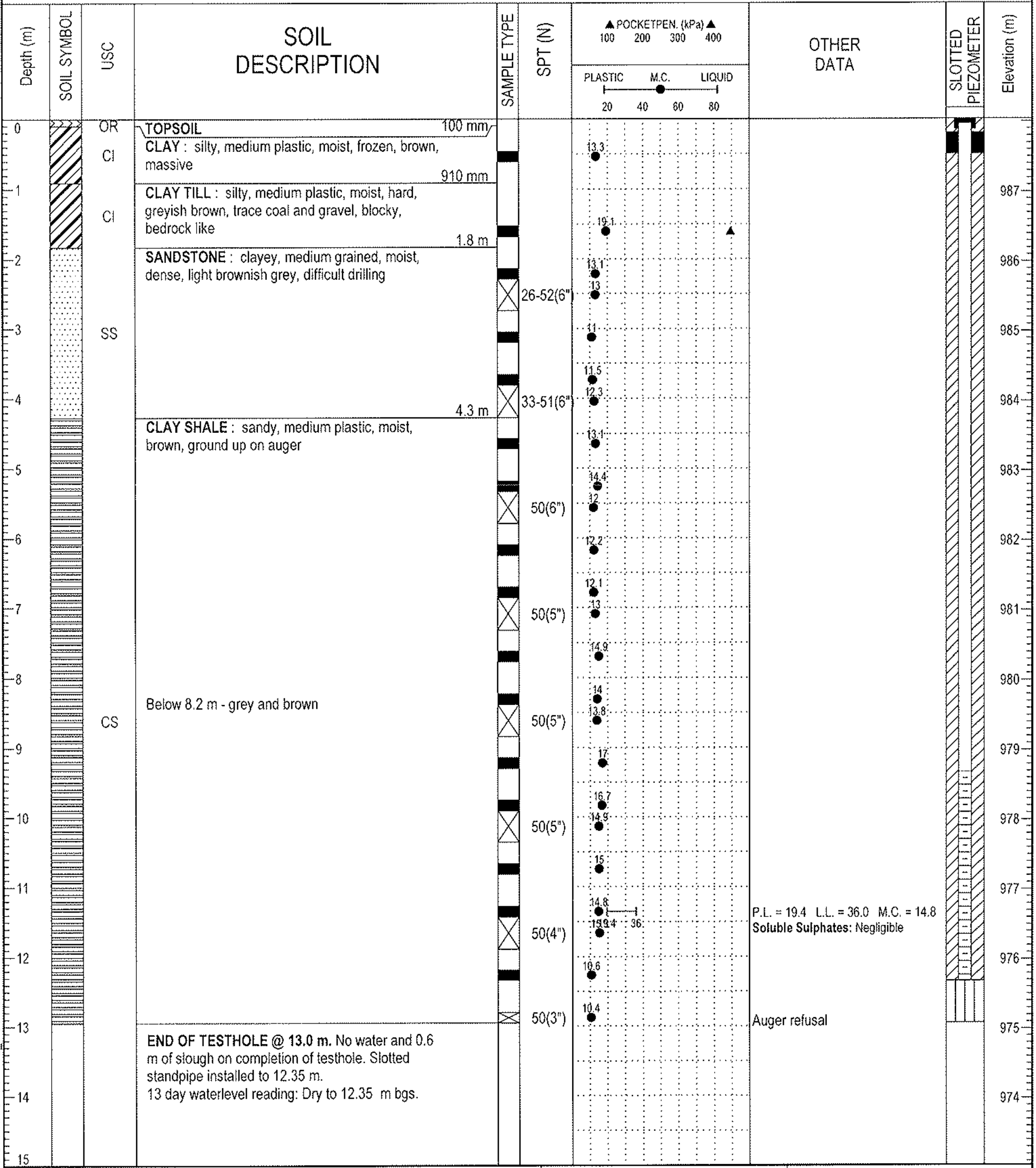
LOGGED BY: J Tsoi	COMPLETION DEPTH: 7.92 m
REVIEWED BY: R Evans	COMPLETION DATE: 5/3/09

PROJECT: Highland Park Subdivision
PROJECT NO: 2461 - 412
BOREHOLE NO: **09 - 4**
CLIENT: ISL Engineering and Land Services Ltd.
DRILL METHOD: Solid Stem Auger
ELEVATION: 988.04 m
OWNER: Lance Skinner
LOCATION: As per site plan

SAMPLE TYPE: ▨ SHELBY TUBE ◪ CORE SAMPLE ☒ SPT SAMPLE ■ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE: ■ BENTONITE ⁘ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁘ SAND



SOIL DESCRIPTION

TOPSOIL 100 mm

CLAY : silty, medium plastic, moist, frozen, brown, massive 910 mm

CLAY TILL : silty, medium plastic, moist, hard, greyish brown, trace coal and gravel, blocky, bedrock like 1.8 m

SANDSTONE : clayey, medium grained, moist, dense, light brownish grey, difficult drilling 4.3 m

CLAY SHALE : sandy, medium plastic, moist, brown, ground up on auger

Below 8.2 m - grey and brown

END OF TESTHOLE @ 13.0 m. No water and 0.6 m of slough on completion of testhole. Slotted standpipe installed to 12.35 m.
13 day waterlevel reading: Dry to 12.35 m bgs.

OTHER DATA

P.L. = 19.4 L.L. = 36.0 M.C. = 14.8
Soluble Sulphates: Negligible

Auger refusal

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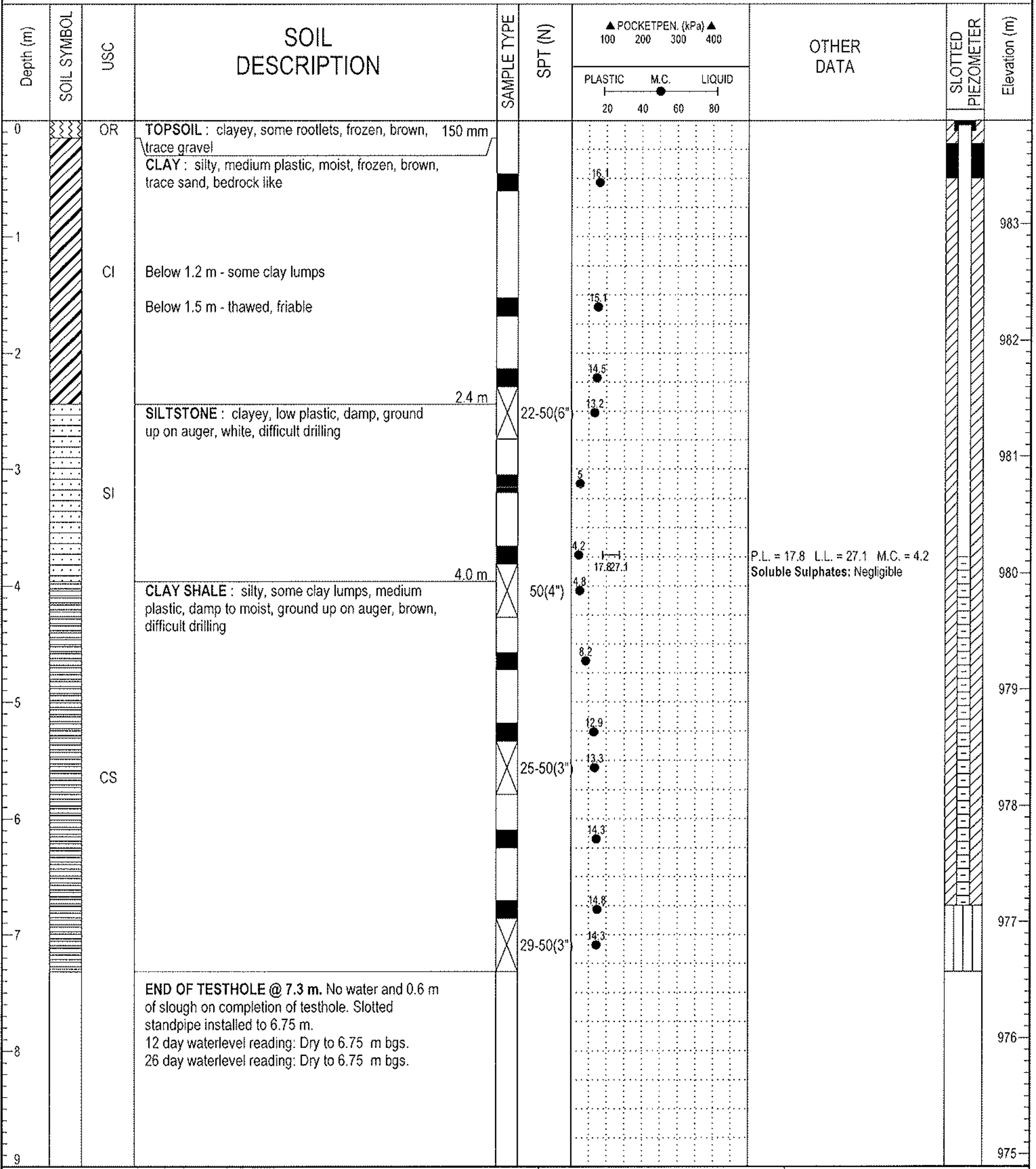
LOGGED BY: J Tsoi
REVIEWED BY: R Evans
COMPLETION DEPTH: 12.95 m
COMPLETION DATE: 5/3/09

PROJECT: Highland Park Subdivision
PROJECT NO: 2461 - 412
BOREHOLE NO: **09 - 5**
CLIENT: ISL Engineering and Land Services Ltd.
DRILL METHOD: Solid Stem Auger
ELEVATION: 983.89 m
OWNER: Lance Skinner
LOCATION: As per site plan

SAMPLE TYPE ▨ SHELBY TUBE ◪ CORE SAMPLE ☒ SPT SAMPLE ■ GRAB SAMPLE ▥ NO RECOVERY
BACKFILL TYPE ■ BENTONITE ⁙ PEA GRAVEL ▥ SLOUGH ⁙ GROUT ▨ DRILL CUTTINGS ⁙ SAND



TOPSOIL : clayey, some rootlets, frozen, brown, trace gravel

CLAY : silty, medium plastic, moist, frozen, brown, trace sand, bedrock like

Below 1.2 m - some clay lumps

Below 1.5 m - thawed, friable

SILTSTONE : clayey, low plastic, damp, ground up on auger, white, difficult drilling

P.L. = 17.8 L.L. = 27.1 M.C. = 4.2
Soluble Sulphates: Negligible

CLAY SHALE : silty, some clay lumps, medium plastic, damp to moist, ground up on auger, brown, difficult drilling

END OF TESTHOLE @ 7.3 m. No water and 0.6 m of slough on completion of testhole. Slotted standpipe installed to 6.75 m.
12 day waterlevel reading: Dry to 6.75 m bgs.
26 day waterlevel reading: Dry to 6.75 m bgs.

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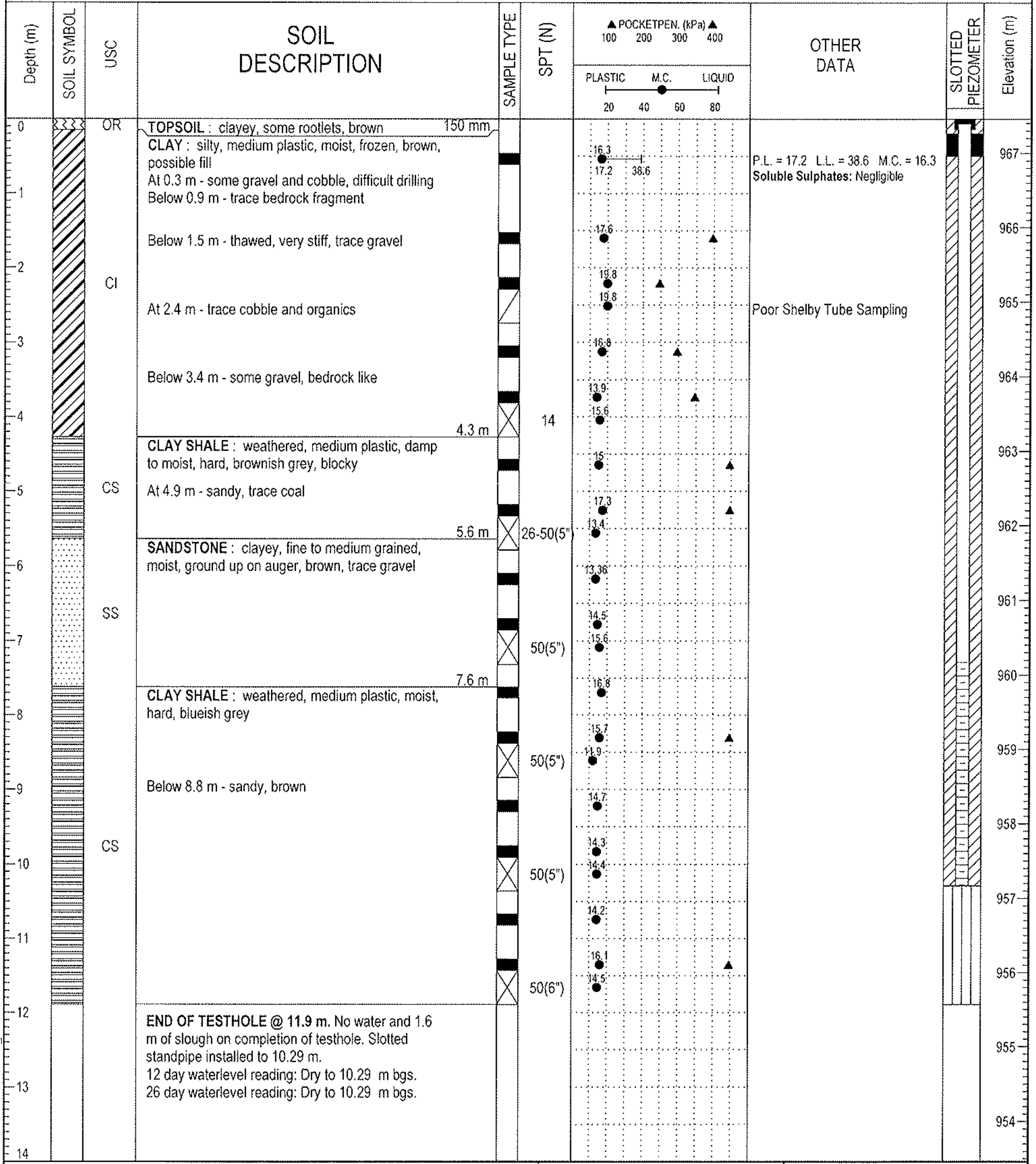
LOGGED BY: J Tsoi
REVIEWED BY: R Evans
COMPLETION DEPTH: 7.32 m
COMPLETION DATE: 6/3/09

PROJECT: Highland Park Subdivision	PROJECT NO: 2461 - 412	BOREHOLE NO: **09 - 6**
CLIENT: ISL Engineering and Land Services Ltd.	DRILL METHOD: Solid Stem Auger	ELEVATION: 967.46 m
OWNER: Lance Skinner	LOCATION: As per site plan	

SAMPLE TYPE ▨ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ■ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE ■ BENTONITE ⁙ PEA GRAVEL ▥ SLOUGH ⁙ GROUT ▨ DRILL CUTTINGS ⁙ SAND



SOIL DESCRIPTION

TOPSOIL : clayey, some rootlets, brown 150 mm

CLAY : silty, medium plastic, moist, frozen, brown, possible fill

At 0.3 m - some gravel and cobble, difficult drilling

Below 0.9 m - trace bedrock fragment

Below 1.5 m - thawed, very stiff, trace gravel

At 2.4 m - trace cobble and organics

Below 3.4 m - some gravel, bedrock like

4.3 m

CLAY SHALE : weathered, medium plastic, damp to moist, hard, brownish grey, blocky

At 4.9 m - sandy, trace coal

5.6 m

SANDSTONE : clayey, fine to medium grained, moist, ground up on auger, brown, trace gravel

7.6 m

CLAY SHALE : weathered, medium plastic, moist, hard, blueish grey

Below 8.8 m - sandy, brown

END OF TESTHOLE @ 11.9 m. No water and 1.6 m of slough on completion of testhole. Slotted standpipe installed to 10.29 m.

12 day waterlevel reading: Dry to 10.29 m bgs.

26 day waterlevel reading: Dry to 10.29 m bgs.

OTHER DATA

P.L. = 17.2 L.L. = 38.6 M.C. = 16.3

Soluble Sulphates: Negligible

Poor Shelby Tube Sampling

JRP HIGHLANDPARK.GPJ JRPV2_3.GDT 29/4/09

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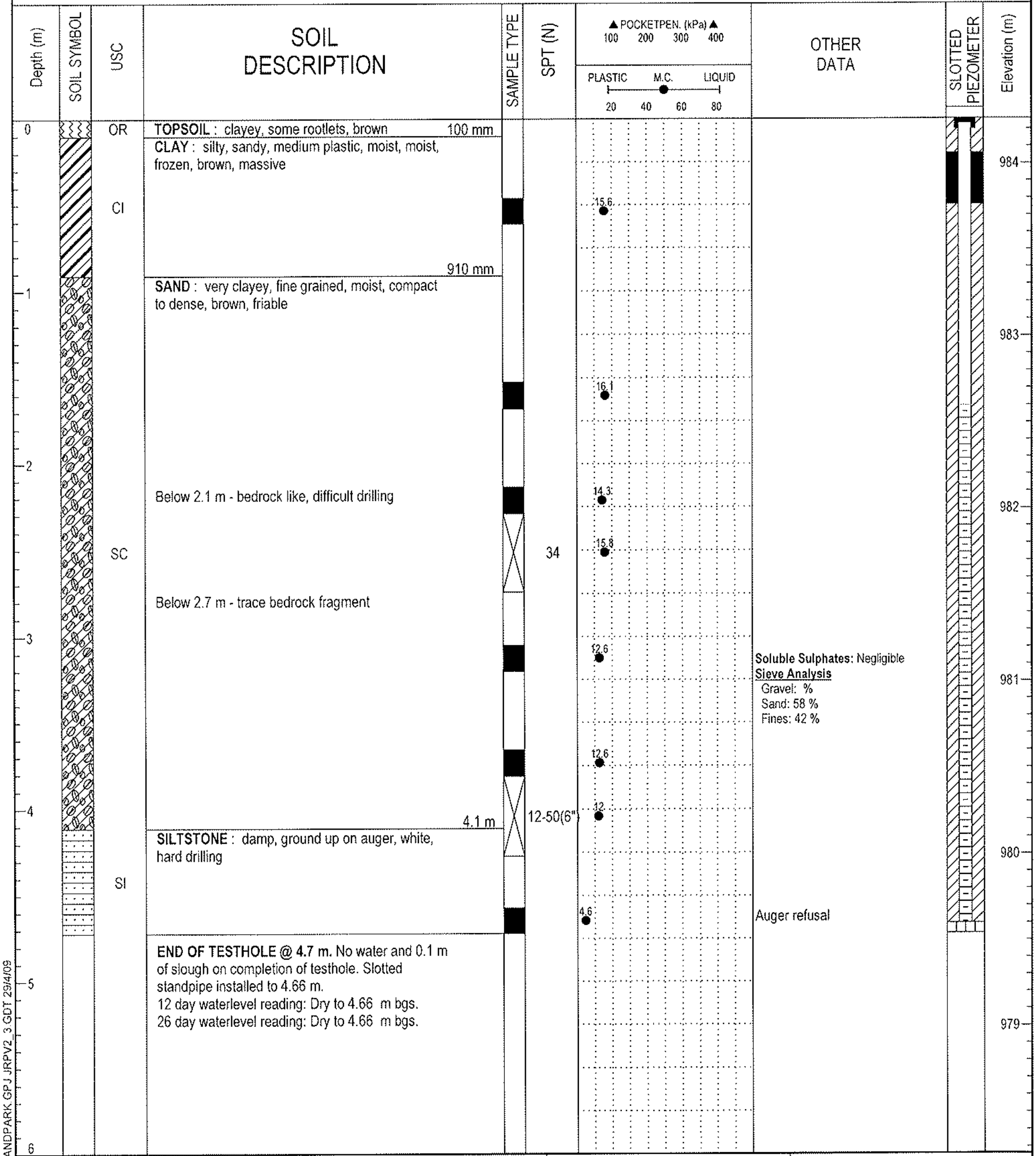
LOGGED BY: J Tsoi	COMPLETION DEPTH: 11.89 m
REVIEWED BY: R Evans	COMPLETION DATE: 6/3/09

PROJECT: Highland Park Subdivision	PROJECT NO: 2461 - 412	BOREHOLE NO: **09 - 7**
CLIENT: ISL Engineering and Land Services Ltd.	DRILL METHOD: Solid Stem Auger	ELEVATION: 984.26 m
OWNER: Lance Skinner	LOCATION: As per site plan	

SAMPLE TYPE ▨ SHELBY TUBE ◪ CORE SAMPLE ☒ SPT SAMPLE ■ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE ■ BENTONITE ⬚ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁘ SAND



Depth (m)	USC	SOIL DESCRIPTION	SPT (N)	OTHER DATA
0	OR	**TOPSOIL** : clayey, some rootlets, brown 100 mm		
	CI	**CLAY** : silty, sandy, medium plastic, moist, moist, frozen, brown, massive 910 mm		
1	SC	**SAND** : very clayey, fine grained, moist, compact to dense, brown, friable		
2		Below 2.1 m - bedrock like, difficult drilling	34	
		Below 2.7 m - trace bedrock fragment		**Soluble Sulphates:** Negligible **Sieve Analysis** Gravel: % Sand: 58 % Fines: 42 %
4		4.1 m	12-50(6")	
	SI	**SILTSTONE** : damp, ground up on auger, white, hard drilling		Auger refusal

END OF TESTHOLE @ 4.7 m. No water and 0.1 m of slough on completion of testhole. Slotted standpipe installed to 4.66 m.
12 day waterlevel reading: Dry to 4.66 m bgs.
26 day waterlevel reading: Dry to 4.66 m bgs.

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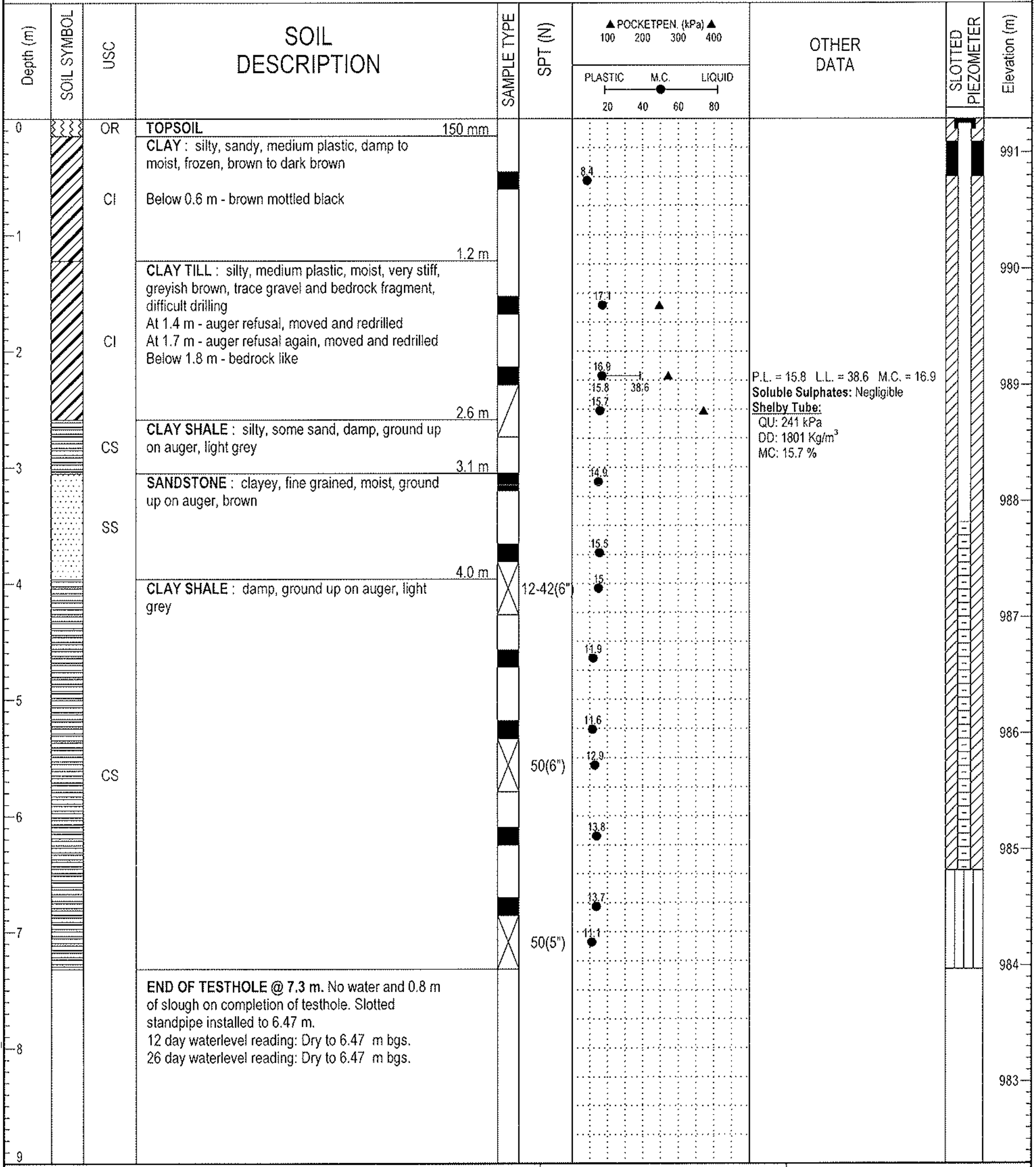
LOGGED BY: J Tsoi	COMPLETION DEPTH: 4.72 m
REVIEWED BY: R Evans	COMPLETION DATE: 6/3/09

JRP HIGHLANDPARK.GPJ JRPV2_3.GDT 29/4/09

PROJECT: Highland Park Subdivision
PROJECT NO: 2461 - 412
BOREHOLE NO: **09 - 8**
CLIENT: ISL Engineering and Land Services Ltd.
DRILL METHOD: Solid Stem Auger
ELEVATION: 991.29 m
OWNER: Lance Skinner
LOCATION: As per site plan

SAMPLE TYPE	▨ SHELBY TUBE	◩ CORE SAMPLE	☒ SPT SAMPLE	■ GRAB SAMPLE	▥ NO RECOVERY	
BACKFILL TYPE	■ BENTONITE	⁘ PEA GRAVEL	▥ SLOUGH	⁘ GROUT	▨ DRILL CUTTINGS	⁘ SAND



SOIL DESCRIPTION

TOPSOIL 150 mm

CLAY : silty, sandy, medium plastic, damp to moist, frozen, brown to dark brown

Below 0.6 m - brown mottled black

1.2 m

CLAY TILL : silty, medium plastic, moist, very stiff, greyish brown, trace gravel and bedrock fragment, difficult drilling
At 1.4 m - auger refusal, moved and redrilled
At 1.7 m - auger refusal again, moved and redrilled
Below 1.8 m - bedrock like

2.6 m

CLAY SHALE : silty, some sand, damp, ground up on auger, light grey

3.1 m

SANDSTONE : clayey, fine grained, moist, ground up on auger, brown

4.0 m

CLAY SHALE : damp, ground up on auger, light grey

END OF TESTHOLE @ 7.3 m. No water and 0.8 m of slough on completion of testhole. Slotted standpipe installed to 6.47 m.
12 day waterlevel reading: Dry to 6.47 m bgs.
26 day waterlevel reading: Dry to 6.47 m bgs.

OTHER DATA

P.L. = 15.8 L.L. = 38.6 M.C. = 16.9
Soluble Sulphates: Negligible
Shelby Tube:
QU: 241 kPa
DD: 1801 Kg/m^3
MC: 15.7 %

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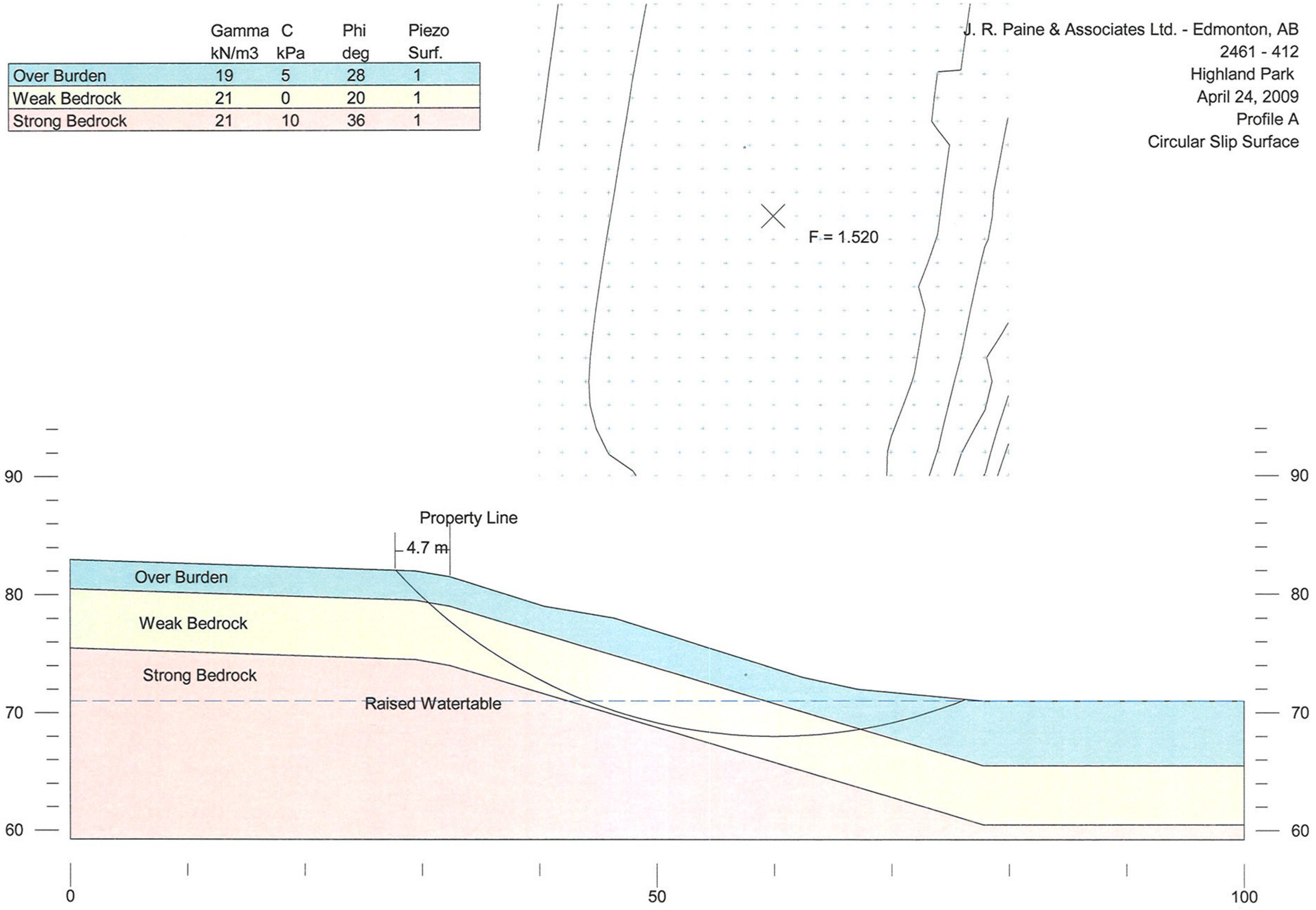
LOGGED BY: J Tsoi
REVIEWED BY: R Evans
COMPLETION DEPTH: 7.32 m
COMPLETION DATE: 6/3/09

APPENDIX C



	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
Over Burden	19	5	28	1
Weak Bedrock	21	0	20	1
Strong Bedrock	21	10	36	1

J. R. Paine & Associates Ltd. - Edmonton, AB
2461 - 412
Highland Park
April 24, 2009
Profile A
Circular Slip Surface

4/29/2009 2:41:13 PM H:\DATA 2009\2461 ISL\2461-412 Geo NE17-39-01-W5M\Slopes\A10.gsl J. R. Paine & Associates Ltd. - Edmonton, AB   F = 1.520

Figure A1

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
Over Burden	19	5	28	1
Weak Bedrock	21	0	20	1
Strong Bedrock	21	10	36	1

J. R. Paine & Associates Ltd. - Edmonton, AB
2461 - 412
Highland Park
April 24, 2009
Profile A
Translational Slip Surface

F = 1.642

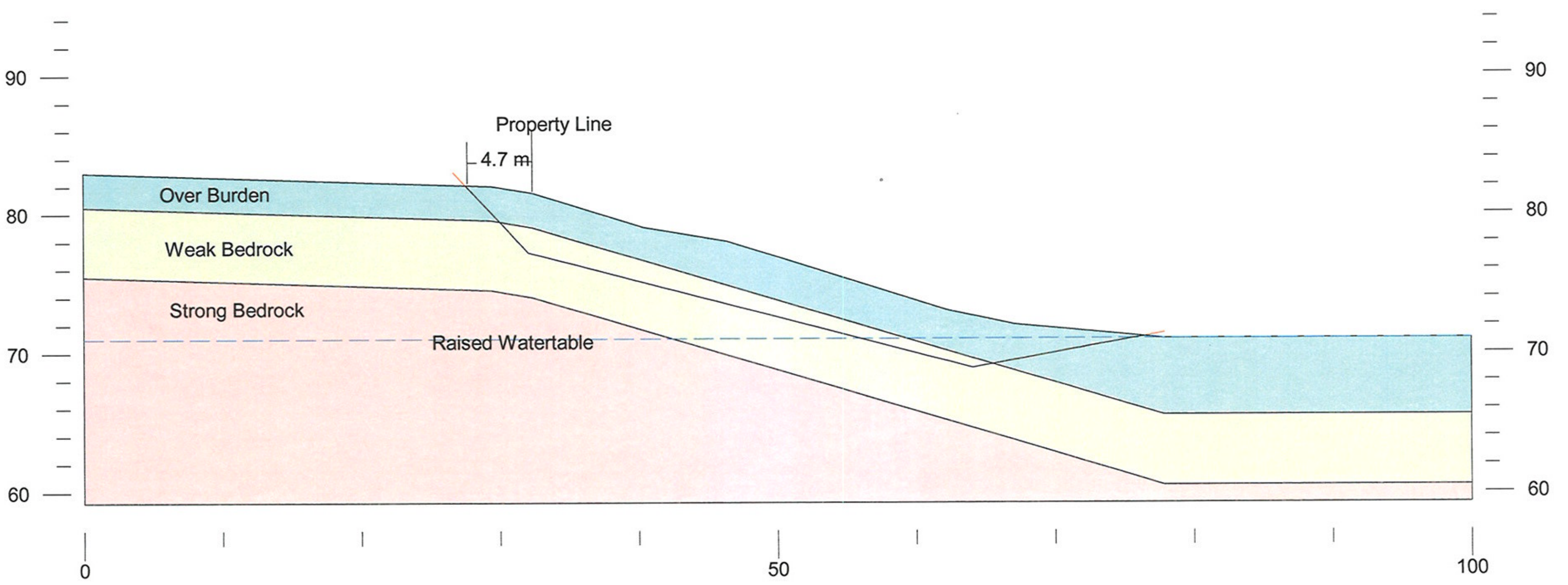


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Figure A2

J. R. Paine & Associates Ltd. - Edmonton, AB
2461 - 412
Highland Park
April 24, 2009
Profile B
Circular Slip Surface

	Gamma	C	Phi deg	Piezo Surf.
Over Burden	19	5	28	1
Weak Bedrock	21	0	20	1
Strong Bedrock	21	10	36	1

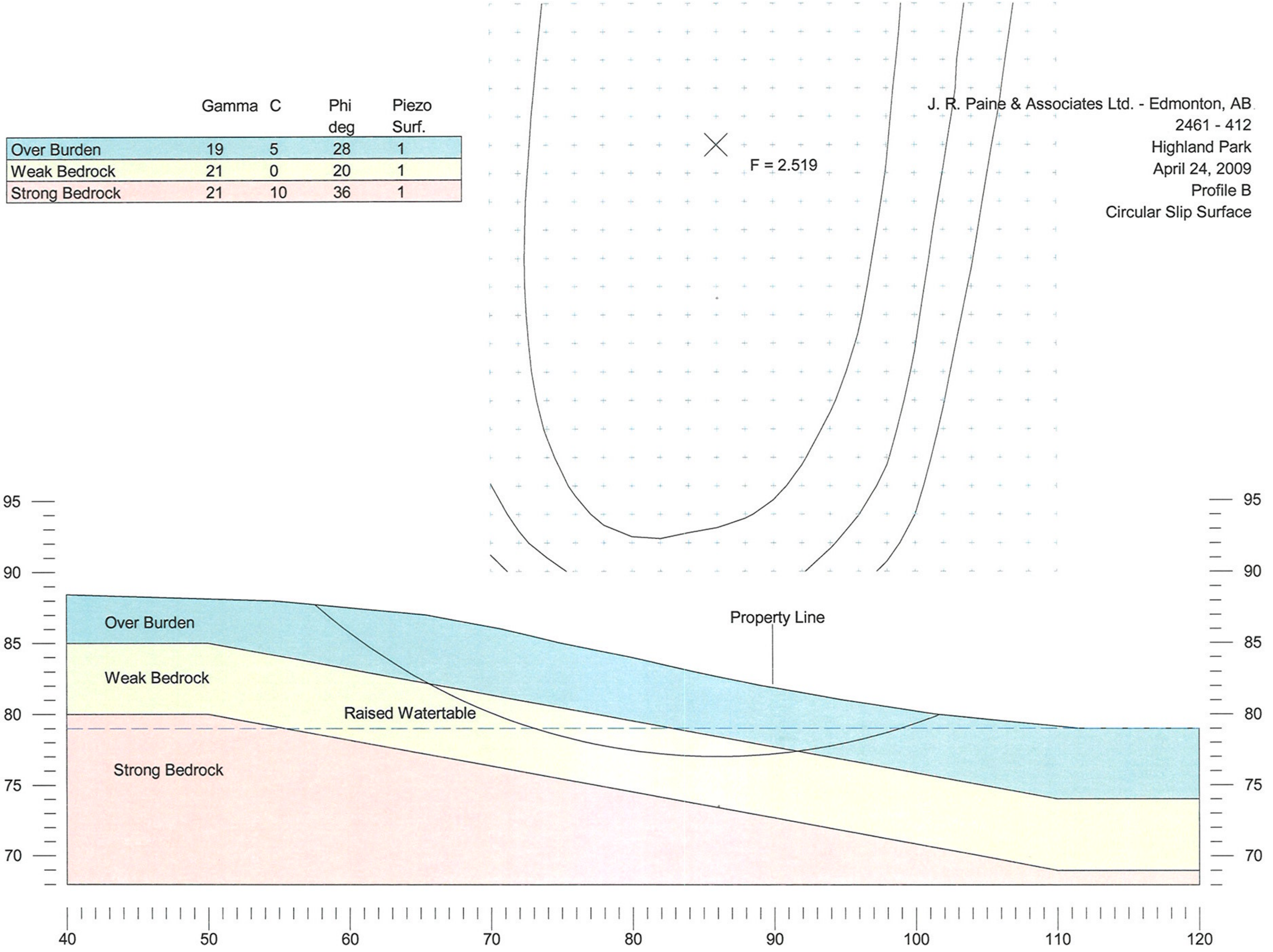

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Figure B1

	Gamma	C	Phi deg	Piezo Surf.
Over Burden	19	5	28	1
Weak Bedrock	21	0	20	1
Strong Bedrock	21	10	36	1

F = 2.639

J. R. Paine & Associates Ltd. - Edmonton, AB
2461 - 412
Highland Park
April 24, 2009
Profile B
Translational Slip Surface

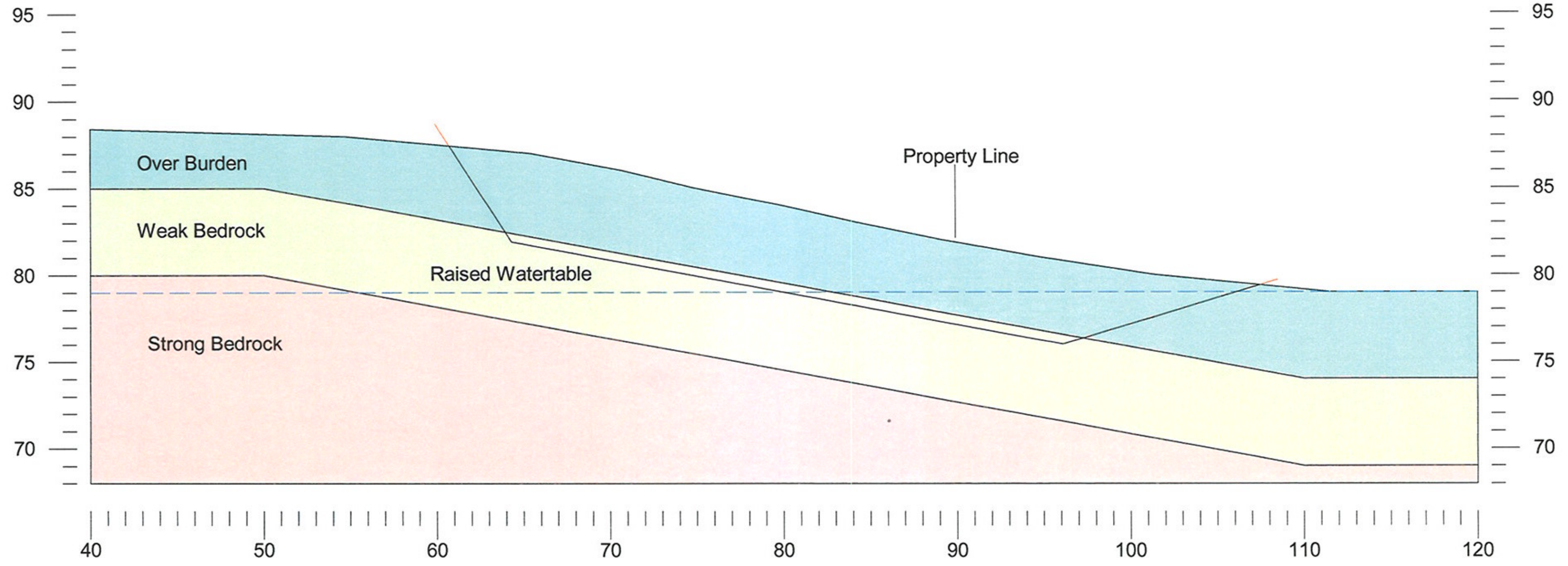

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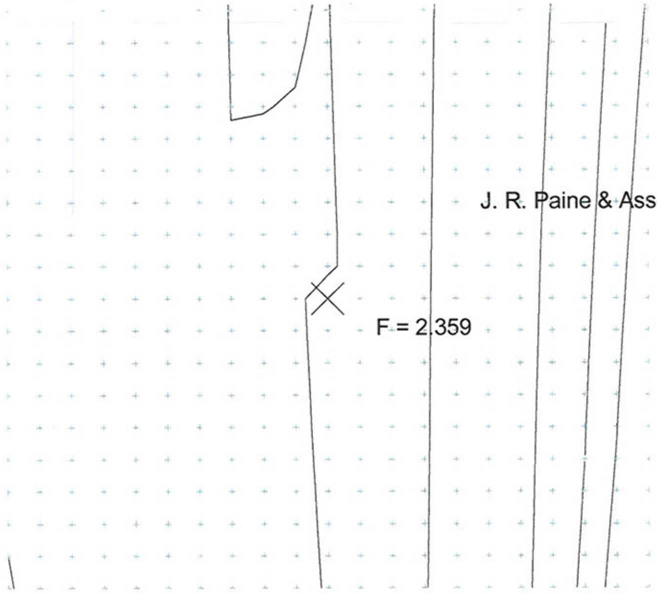
Figure B2

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
Over Burden	19	5	28	1
Weak Bedrock	21	0	20	1
Strong Bedrock	21	10	36	1

J. R. Paine & Associates Ltd. - Edmonton, AB
2461 - 412
Highland Park
April 24, 2009
Profile C
Circular Slip Surface



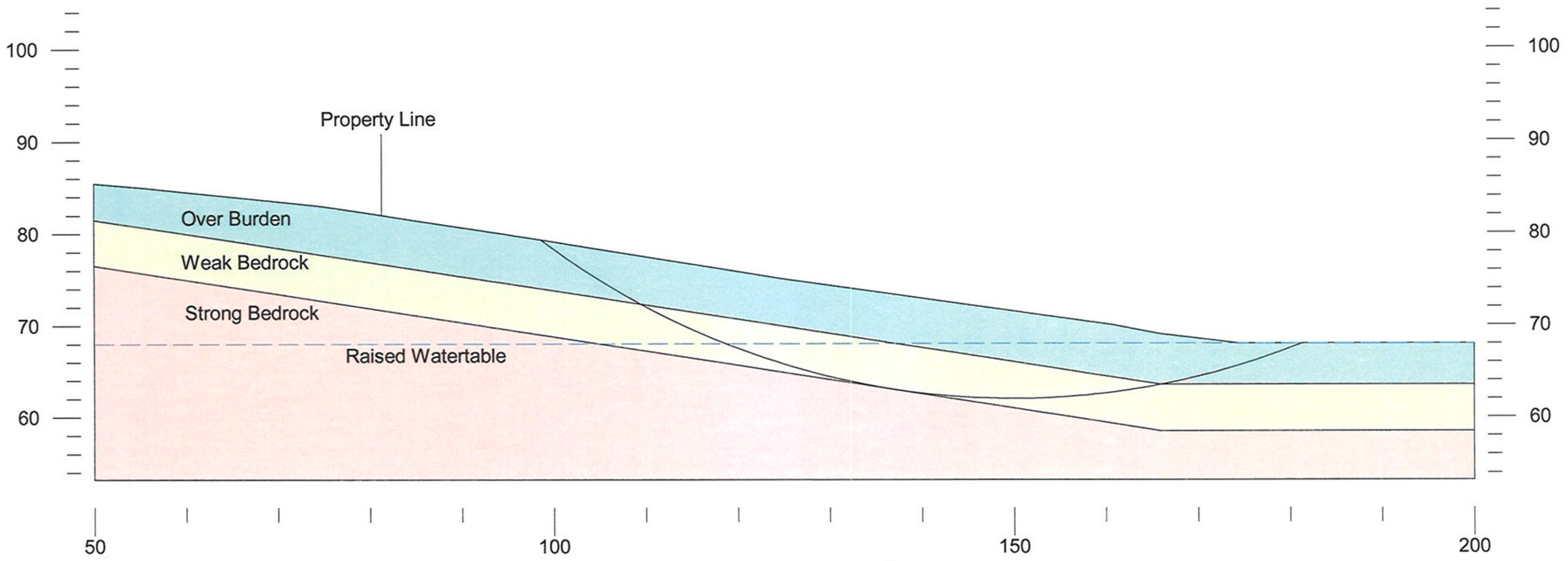


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Figure C1

F = 2.517

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
Over Burden	19	5	28	1
Weak Bedrock	21	0	20	1
Strong Bedrock	21	10	36	1

J. R. Paine & Associates Ltd. - Edmonton, AB
2461 - 412
Highland Park
April 24, 2009
Profile C
Translational Slip Surface

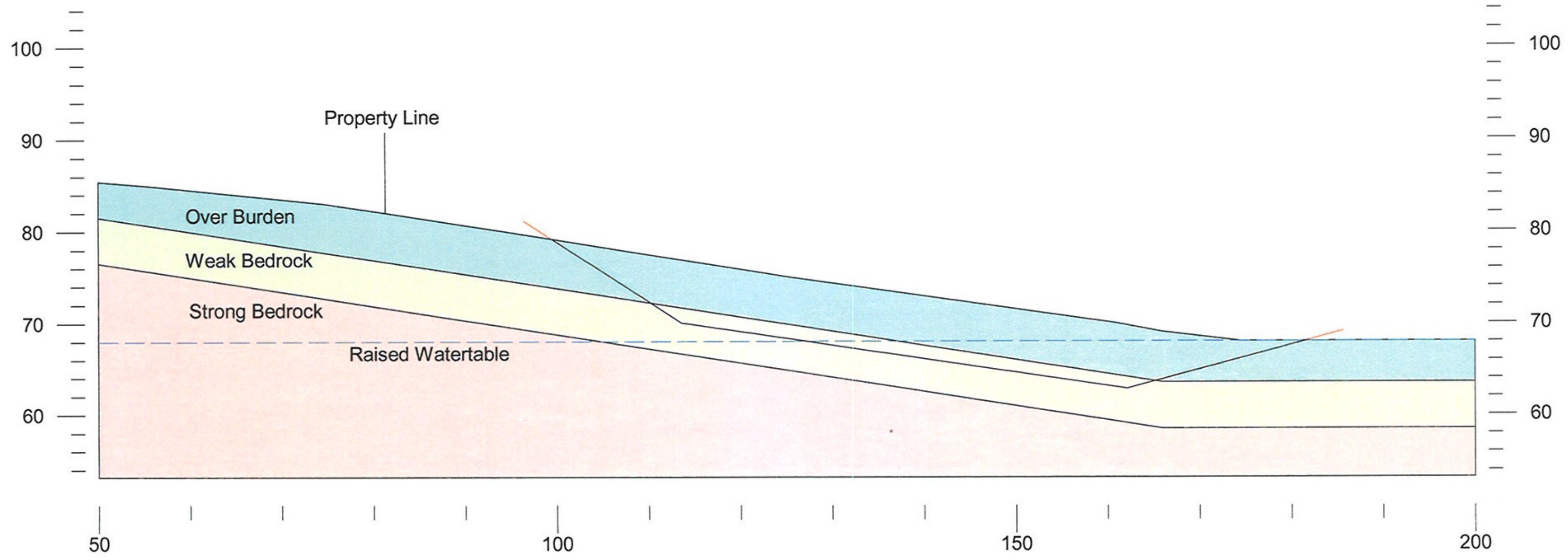

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Figure C2

J. R. Paine & Associates Ltd. - Edmonton, AB
2461 - 412
Highland Park
April 24, 2009
Profile D
Circular Slip Surface

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
Over Burden	19	5	28	1
Weak Bedrock	21	0	20	1
Strong Bedrock	21	10	36	1

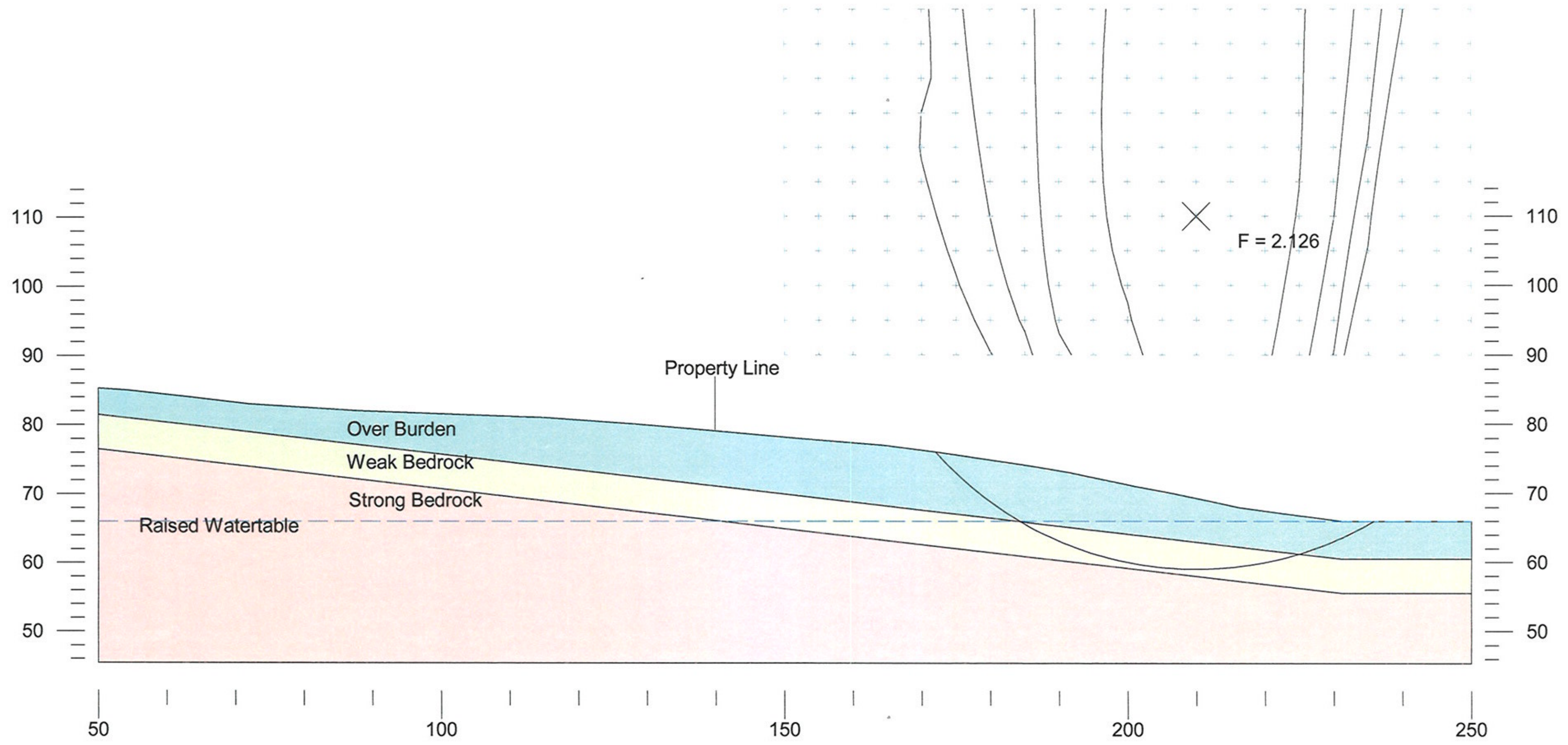

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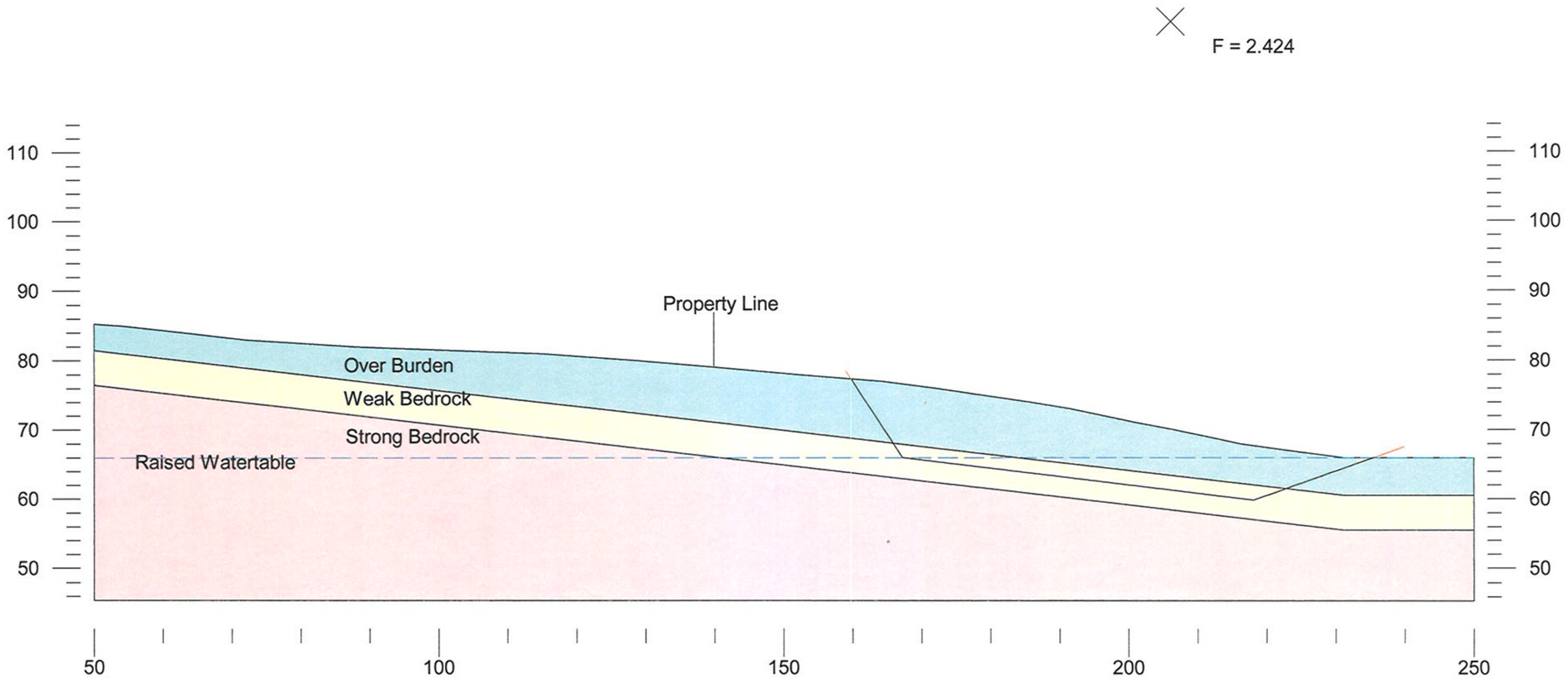
Figure D1

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
Over Burden	19	5	28	1
Weak Bedrock	21	0	20	1
Strong Bedrock	21	10	36	1

J. R. Paine & Associates Ltd. - Edmonton, AB
2461 - 412
Highland Park
April 24, 2009
Profile D
Translational Slip Surface



5/1/2009 2:36:43 PM H:\DATA 2009\2461 ISL\2461-412 Geo NE17-39-01-W5M\Slopes\D4.gsl J. R. Paine & Associates Ltd. - Edmonton, AB F = 2.424

Figure D2