GEOTECHNICAL INVESTIGATION
PROPOSED ALLAN BEACH RESORT UPGRADES
RANGE ROAD 13 SOUTH OF HIGHWAY 16
NE 9-53-1-W5M (PORTION)
PARKLAND COUNTY, ALBERTA

HOGGAN ENGINEERING & TESTING (1980) LTD.

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REPORT NO: 6029 – 2

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MARCH 2012

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GEOTECHNICAL INVESTIGATION

PROJECT: Proposed Allan Beach Resort Upgrades

LOCATION: Range Road 13 South of Highway 16
NE 9-53-1-W5M (portion)
Parkland County, Alberta

CLIENT: TRG Developments Corp.
171 New Brighton Drive SE
Calgary, Alberta
T2Z 0E1

ATTENTION:

1.0 INTRODUCTION

This report presents the results of the subsurface investigation made on the site of the proposed Allan Beach Resort upgrades in Parkland County, Alberta. The project is understood to consist of serviced recreational vehicle (RV) and cabin sites with underground utilities and paved roadways. The roadways may be either urban or rural profile with ditches. The objective of the investigation was to determine the nature and condition of the existing soil and groundwater for use in the geotechnical planning and design aspects of this project.

In addition, given the rolling nature of the site terrain, slope stability assessments were conducted. Previous land use and environmental issues are beyond the scope of this report. Fieldwork for the project was initiated August and September 2008 and completed in July and August 2009. The delay in report submission was due to a change in land ownership.

2.0 SITE DESCRIPTION

The site is located west of Range Road 13 between approximately 0.7 and 1.0 kilometre south of Highway 16 in Parkland County, Alberta. The site encompasses the south portion of the NE 9-53-1-W5M and is approximately 14 hectares (35 acres) in size. The site is bordered to the south and north east by residential acreage developments and on the east by Range Road 13. To the east of Range Road 13 the land is again residential acreage development. To the north west the Allan Beach Resort is bordered by Hubbles Lake.

Original Site Inspection

The site was unevenly graded with two significant water bodies incorporated within or bordering the site. Allan Beach Pond is located within the east portion of the site with site grading towards the pond on all sides. West of Allan Beach Pond is an area of high ground elevation, which generally grades toward the pond on the east side and to the north and west towards Hubbles Lake. The elevation changes over the site were approximately 18 metres. The natural vegetation over the site consists of sparse to medium dense poplar growth with the odd birch and spruce trees. This mature growth is mixed with dense undergrowth of young poplar and low bushes. Open grassed or gravelled portions of the site utilized to position recreation vehicles as well for campers and day uses are positioned around the site. The larger of these sites border Hubbles Lake and Allan Beach Pond as well as area on the east limit of the site. During the time of this investigation, the demolition of the resort facilities was being undertaken in preparation for proposed upgrades. Due to vegetation cover and steep grades, access around the site was limited to existing gravelled trails and clearings. Travel over the site was possible for normal wheeled traffic on these trails and clearings.

Current Site Conditions

The site was revisited on March 26, 2012 to confirm the site conditions. The site was snow covered and fenced along Range Road 13. The gate was locked and no other way to access the site was discovered. Summer time site reconnaissance prior to construction would be required to verify the site conditions.

3.0 FIELD INVESTIGATION

The initial soils investigation for this project was undertaken on August 12, 13 and 14, 2008, utilizing a truck mounted drill rig owned and operated by MARL of Edmonton, Alberta. A total of 15 testholes were drilled at locations shown on the attached site plan. The testholes were advanced to depths of between 5.8 and 21.0 metres below ground surface (BGS). The testhole frequency was selected by Hoggan Engineering & Testing Ltd. (Hoggan) in discussion with the original client. In addition to the use of a drill rig, six hand augered testholes were advanced to depths of between 1.5 and 3.2 metres. The hand augered testholes were located adjacent to Allan Beach Pond to aid in soil profile determination in this area. Access to these locations was not possible in August due to the occupation of the RV sites.

The soils investigation was completed on July 29 and 31, 2009 with the advancement of an additional 7 testholes required to provide further soil profile detail as a result of changes to the original development concepts. Four testholes to depths of between 5.8 and 14.9 metres (BGS) were advanced utilizing a truck mounted drill rig owned and operated by MARL of Edmonton, Alberta. The remaining 3 testholes were advanced to depths of 1.8, 2.2 and 2.3 metres BGS utilizing a hand auger. The hand auger testholes were required due to lack of access to a truck mounted drill rig as a result of overhead power lines and/or uneven ground.

All the testholes were initially located using a Garmin hand held GPS unit with elevations provided by another firm. The elevation datum is unknown at this time. A site plan has been attached showing the approximate testhole locations.

The drilled testholes were advanced with 150 millimetre diameter solid stem augers in 1.5 metre increments. For all testholes a continuous visual description was recorded on site in accordance with the modified Unified Soil Classification System that included the soil types, depths, moisture, transitions, and other pertinent observations. Disturbed samples were removed from the auger cuttings at intervals for laboratory testing. Standard Penetration Tests c/w split spoon sampling were also taken at regular 1.5 metre intervals in the drilled testholes.

Following the drilling operation, slotted piezometric standpipes were inserted into all but one of the drilled testholes for water table level determination. No piezometric standpipes were installed in the hand augered testholes. The testholes were backfilled with cuttings, with a bentonite seal placed at the surface.

4.0 LABORATORY TESTING

All disturbed bag samples returned to the laboratory were tested for moisture content. In addition, the plastic and liquid Atterberg limits, grain size analysis and soluble soil sulphate concentrations were determined on selected samples. Lab results are included on the attached testhole logs located in the Appendix.

5.0 SOIL CONDITIONS

A detailed description of the soils encountered is found on the attached testhole logs in the Appendix. In general the soils encountered in the testholes at this site may be classified as ice contact lacustrine and fluvial deposits. The deposits were comprised of stratified layers of sand, silt and clay. Surficial fill materials and organic materials were noted in most of the testholes.

Fill

At the surface in Testholes 08-3, 08-4, 08-5, 08-5b, 08-7, 08-7b and 09-18 to 09-22, fill soils were encountered, which extended to depths of between approximately 0.2 and 1.1 metres below ground surface (BGS). These fill materials were comprised of topsoil, organic clays, silts, clays and gravel. In 7 of the remaining testholes traces to less than 80 millimetres of fill soils were encountered, which were comprised of mostly thin stratums of road and RV parking surfacing materials. The testholes indicating greater depths of fill materials were located at lower elevations on the site near Allan Beach Pond and Hubbles Lake. It should be noted that the fill depths are known only at the testhole locations, and may vary between testholes.

Topsoil, Peat and Organic Clay

Below the fill materials in Testholes 08-5, 5b, 6, 7 and 09-18 to 09-20, organic soils were encountered, which extended to depths of between 0.9 and 4.3 metres BGS. Traces to 80 millimetres of native organics or organics mixed with gravel surfacing materials were located in various other testhole locations. The testholes indicating greater depths of organic materials were located at lower elevations on the site near Allan Beach Pond and Hubbles Lake. These organic materials were noted to be variable but in general were very moist to wet and soft. It should be noted that the organic soil depths are known only at the testhole locations, and may vary between testholes.

Sand, Silt and Clay

The stratified sand, silt and clay soils encountered on this site were variable with no general soil type trends with depth or between test testholes. In Testholes 08-1, 08- 2, 08-11, 08-12, 08-13, 08-14, 08-15, 09-16 and 09-17 located at higher site elevations, the sands and silts were typically damp to moist and compact while the clays were moist, low to medium plastic and stiff to very stiff. Below elevations of approximately 28 to 30 metres in the testholes located at higher elevations and at relatively shallow depths BGS in the testholes located at lower elevations, adjacent to Hubbles Lake and Allan Beach Pond, the soils were considerably more moist. At these lower elevations, the sand and silts were moist to saturated and loose while the clays were very moist, low to high plastic and soft to firm. Some variation to these generalized soil conditions were noted in the testholes and should be expected as typical for ice contact soil deposits in this area of Parkland County.

At the completion of drilling, accumulations of free water and or slough were noted in 14 of the 20 testholes. The following table summarizes the occurrence of slough and/or free water.

Table 1: Groundwater Seepage And Sloughing Conditions At Completion

Testholes	approximate water accumulation at hole bottom (m)	approximate slough thickness at hole bottom (m)
08 - 1	none	none
08 - 2	none	none
08 - 3	3.1	2.2
08 - 4	1.5	1.5
08 - 5	3.8	3.8
08 - 6	3.8	3.8
08 - 7	4.3	2.7
08 - 8	2.2	1.5
08 - 9	0.9	none
08 - 10	2.7	2.7
08 - 11	none	none
08 - 12	0.9	0.9
08 - 13	none	none
08 - 14	none	none
08 - 15	2.7	2.7
09 - 16	0.3	0.2
09 - 17	none	none
09 - 18	0.8	0.8
09 - 19	1.9	1.4
09 - 20	1.1	1.1

6.0 GROUNDWATER CONDITIONS

The groundwater table within the study area was low to high, with the water table located between 2.3 metres and 17.6 metres BGS and dry in four of 14 testholes. Water table readings were obtained at 2, 7/9 and 14/16 days after completion of the 2008 drilling program. During the 2009 work program, a complete suite of water table readings on existing standpipes were taken approximately 366 and 16 days after completion of drilling. The following tables outlines the water table readings that were taken.

Table 2A: Watertable Measurements

Testholes	Water Table Depth Below Ground Surface (m)				Ground Elevation (m)	Watertable Elevation (m)
	14-Aug-08 (1 to 2 days)	19-Aug-08 (5 to 7 days)	19-Sep-08 (35 to 37 days)	14-Aug-09 (366 days)		
08 - 1	dry				40.50	below 34.69
08 - 2	dry				36.32	below 27.40
08 - 3	4.48	4.5	4.5	4.68	31.19	26.71
08 - 4	3.33	3.34	3.5	destroyed	30.61	27.28
08 - 5	2.06	2.1	2.31	destroyed	29.21	27.15
08 - 6	2.11	2.12	2.35	2.44	29.18	27.07
08 - 7	2.06	2.1	2.26	destroyed	28.87	26.81
08 - 8	No Standpipe				30.38	Unknown
08 - 9	4.85	3.61	3.69	3.81	30.23	26.62
08 - 10	3.05	3.09	3.27	3.37	30.37	27.32
08 - 11	dry				42.24	below 27.42
08 - 12	13.45	13.38	13.52	13.49	39.66	26.28
08 - 13	dry				39.48	below 33.67
08 - 14	13.41	13.37	13.51	13.51	39.99	26.62
08 - 15	17.48	17.46	17.59	17.63	43.94	26.48

Table 2B: Watertable Measurements

Testholes	Water Table Depth Below Ground Surface (m) 14-Aug-09 (16 days)	Ground Elevation (m)	Watertable Elevation (m)
09 - 16	14.08	40.58	26.50
09 - 17	14.6	41.08	26.48
09 - 18	2.73	29.85	27.12
09 - 19	2.53	29.35	26.82
09 - 20	3.19	30.34	27.15

It should be noted that water table levels may fluctuate on a seasonal or yearly basis, with the highest readings obtained in the spring or after periods of heavy rainfall. The above readings would be near to below seasonal average levels. It appeared that the water table elevations were stabilized between approximately 26.28 and 27.32 metres.

7.0 DISCUSSION AND RECOMMENDATIONS

7.1 Site Preparation and Grading

1. Testholes adjacent to Allan Beach Pond and Hubbles Lake encountered soft ground at relatively shallow depths. The soft ground would be associated with fill, organic soil and organics. Other low areas may contain soft ground and some temporary water in the spring or after precipitation. Outside of the low lying areas adjacent to Allan Beach Pond and Hubbles Lake conventional clearing and stripping should be possible. Soft ground is likely to be encountered in low lying areas. The topsoil, fill and organics were recorded as a trace in some locations and extended up to approximately 4.3 metres BGS at some testhole locations.
2. All structural fill should be placed in maximum 150 millimetre thick lifts, and should be compacted to a minimum of 98 percent of Standard Proctor Density. This compaction standard will ensure adequate subgrade support in road areas, as well as support for parking areas and building slabs. Engineered or compacted clay fill is not suitable for building footing foundation support. Engineered fill may be suitable for housing and small building foundations, but will need to be reviewed by a qualified engineer, on a site by site basis.
3. It is expected that the site soils that will be exposed during site grading or may be used for fill over portions of the site will be variable. Selective excavation and soil placement as well as possibly adequate mixing of materials will be required. Field monitoring and testing will be required and is recommended to obtain suitable structural fill.
4. In any areas featuring soft to firm, very moist, underlying soils once stripping has been completed, compacting the first lift of fill material to the above fill standard may be impossible. Where a minimum fill depth condition is met, construction of a clay pad approximately of 300 to 500 millimetres in thickness will be required to obtain an adequate working platform to start from. This pad should be compacted to a minimum of 95 percent of Standard Proctor Density where possible. The normal lift thickness and compaction criteria mentioned above should be applied to successive lifts. To employ this method, a minimum of 1.0 metre of fill must be placed on top of the clay pad. If this minimum fill condition is not met, other measures should be considered, such as

including aggressively drying or replacing the existing very moist soils. Increased parking lot and road structures and upgraded slab construction measures are other options. The low areas encountered onsite would benefit by a grading design of 1-2 metres of fill.

7.2 Underground Utilities

1. The subsurface soil conditions encountered in the testholes are considered generally poor to satisfactory for the installation of underground utilities incorporating the County of Parkland backfilling and compaction requirements. The silt, clay and sand soils would be considered poor to satisfactory for backfill. The poor rating would be due to the high moisture characteristics of some of these soils, generally encountered at lower elevations adjacent to the low lying areas. The silt, clay and sand encountered at higher elevations were below to above their respective optimum moisture contents. Some difficulty can be expected in obtaining compaction when mixtures of cohesive and non-cohesive soils are encountered. Topsoil and other organic materials are not considered suitable for backfill material.
2. The water table was located between 2.3 metres and 17.6 metres BGS and dry in four of 19 testholes, indicating that saturated conditions may be encountered in the trenches below these depths. The free water and sloughing conditions that were encountered during drilling have been outlined at the end of Section 5.0. Water table levels indicate that trenches left open for extended periods of time may experience slow to considerable ingressing water. A moderate to considerable amount of ingressing water may be anticipated in the areas where sand, and sandy deposits of clay were encountered below the water table. Temporary dewatering will likely be required. Opening relatively long portions of utility trench is not recommended.
3. Standard trenching cutback angles of approximately 30 degrees from the vertical are anticipated for most of the higher elevation areas of the site. Increased cutback angles of 45 degrees or more will be required in areas where very moist to saturated sands, sandy clay are encountered. The increased cut back angles should be anticipated in low lying areas adjacent to Allan Beach Pond and Hubbles Lake. Exact values for excavation slopes cannot be pinpointed without detailed and extensive analysis. For this reason, this information

should be used as a guideline only and the optimum cutback angles for utility trenches be determined in the field during construction. It is not recommended that excavations be left open for extensive periods of time. The Occupational Health and Safety Act, Part 32 - Excavations and Tunnelling should be strictly followed except where superseded by this report. All slopes should be monitored regularly for signs of sloughing or movement, especially after any periods of rainfall and remediation performed immediately wherever such signs are observed.

4. To reduce pipe loading. trench widths should be minimized but be compatible with safe construction operations. The trench width must be wide enough to accommodate pipe bedding and compaction equipment.

5. Temporary surcharge loads, such as spill piles, should not be allowed to 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.

6. Pipe bedding and trench backfill procedures should adhere to the County of Parkland Design Standards. 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. The County of Parkland bedding material specifications are considered suitable for this project. To overcome the installation difficulties which may be encountered within sand, sandy clay and moister areas of the site, where ingressing groundwater and/or poor bearing conditions may be a problem, it is recommended that a washed rock and geotextile separator be utilized for the pipe bedding in these areas of poor pipe bedding conditions. The washed rock and geotextile exact dimensions should be determined in the field during construction. The need for this configuration is estimated to be low overall at this site. However, depending on the elevation of the utilities adjacent to Allan Beach Pond and Hubbles Lake, at lower elevations, the need for washed rock and geotextile bedding may be considerable.

7. The moisture content of the materials in the testholes was variable, with silt, clay and sand soils encountered at higher elevations typically ranging from below to above optimum moisture, while the silt, clay and sand soils encountered at lower elevations typically were well above optimum moisture. In general soils were below to well above their respective

optimum moisture contents and will require variable degrees of moisture conditioning to achieve the specified compaction.

8. Trench compaction requirements of County Parkland standards are 98 percent of Standard Proctor Density, in lifts not greater than 300 millimetres. This degree of compaction should be readily achievable with most of the soils encountered in the testholes at higher elevations. For soils in testholes at lower elevations this compaction standard will be difficult to achieve due to the elevated moisture content. Moisture adjustment of trench backfill may be required, likely both the addition of water and the drying of the soils in order to achieve the compaction standard.

   Where the trench intersects wet silt, clay and sands especially below the water table, the backfill conditions would be considered poor. For these conditions the backfill compaction requirement to 98 percent Standard Proctor Density would be difficult and/or not possible to attain. Where wet soils are encountered it would be recommended that backfill be placed in 300 millimetres lifts, after compaction, and compacted to 100 percent of One-Point Proctor Density within 1.5 metres of subgrade elevation. Below 1.5 metres of the subgrade elevation, compaction to 97 percent of a One-Point Proctor Density would be recommended. The need for altering the trench compaction criteria can best be determined in the field during construction. Any altering of the compaction criteria will require discussion with all parties including Parkland County personnel

9. It should be noted that with a higher standard of compaction of the trench backfill materials the more structural capacity these soils would provide for the surface utility support. Utilizing the One-Point Proctor criteria may result in a soft subgrade requiring increased roadway structure as noted in the following section. Further, the ultimate performance of the trench backfill is directly related to the consistency and uniformity of the backfill compaction, as well as the underground contractors construction procedures. In order to achieve this uniformity, the lift thickness and compaction criteria should be strictly enforced.

7.3 Surface Utilities

1. The existing topsoil, clay fill and other deleterious materials should be removed prior to construction of roads, sidewalks and other surface utilities. The surface soil conditions will generally be satisfactory. The condition of the subgrade will depend on the nature of the subsoils encountered in the trenched utility areas. The quality of trench backfill compaction will directly affect the subgrade performance during roadway construction.
2. The road subgrade preparation should conform to the County of Parkland Standards. The minimum involves cement stabilization with 10 kilograms of cement per square metre in the top 150 millimetres of subgrade and a minimum compaction of the subgrade to 100 percent of Standard Proctor Density near optimum moisture content. Some moisture modification, likely the addition of water, may be required. Areas encountering very moist surface conditions and some trenched areas of the site and may require upgraded subgrade treatment. This may involve cement stabilization of 20 to 30 kilograms of cement per square metre of subgrade to a depth of 300 millimetres due to the elevated moisture content soils encountered at depth. All subgrade should be proof rolled after final compaction and any areas showing visible deflections should be inspected and repaired. It should be noted that the degree of cement stabilization is dependant on the time of year, weather conditions, and quality of construction; therefore exact cement contents should be determined in the field during construction.

   Another option, would be the use of a pitrun gravel base. The estimated thickness of subbase to support the roadway is 600 to 900 millimetres. An approved medium duty woven geotextile should initially be placed below the gravel for separation and reinforcement. The subbase should be placed in one lift and static compacted on the surface to the best density achievable. No construction traffic should travel of the exposed subgrade prior to subbase placement. The depth of subbase, as required, should be field determined following a review of trench backfill condition and subgrade load test. The subgrade difficulties will be most prevalent in the lower areas and near the lakeshores.

   Where fill is to be placed, the material should be compacted to 98 percent of Standard Proctor Density. All fill should be placed in maximum thickness lifts of 150 millimetres. It is important that high plastic fill be compacted at minimum 1 percent over optimum (equal to an estimated 3 percent above the plastic limit) to help reduce swelling in the top 1.5 metres of backfill.

3. Sand soils encountered, or sandier clay soils near the surface following site grading pose some concerns as they are highly frost susceptible and are difficult to work with due to anticipated compaction difficulties associated with non-cohesive soils. It should also be noted that our experience shows that cement stabilization of sand soils is relatively ineffective. Therefore, it is recommended to remove the sand soils and replace them with a suitable material. Such measures would be highly dependent upon the final grading of the site and should be determined on site during construction.
4. The near surface sands and clays across the majority of the site are of low to high frost susceptibility. A winter water table within approximately 3.0 metres of the road surface is required for significant frost heaving to occur. The closer the water table is to the surface, the higher is the frost heave potential. Seven of the 19 testholes have stabilized near to above this level and as such, frost heave concerns are foreseen in these areas. The water table in Testholes 08-5 to 08-7 and 09-18 to 09-20 have stabilized near to above this level. Moderate to high frost heave potential is present in the noted high water table areas. For the remaining testholes the potential for frost heave is considered low. The design grade should be set as high as possible and engineered fill placement would be recommended, in the noted areas of high water table. No cuts are recommended. No other frost heave considerations are considered necessary at this time in areas other than noted, provided the road elevations are kept high enough. Hoggan should review the road design when it becomes available to confirm the elevation is adequate.
5. It is recommended that in all cases, qualified personnel inspect the subgrade, during construction, to determine the recommended subgrade treatment. Observations during underground construction would also help determine the subgrade treatment required.
6. Where site uses can accommodate ditches, a minimum 1.0 metre ditch depth is recommended from the bottom of the ditch to the top of the subgrade. A cross slope should be constructed and maintained from subgrade to surface to ensure proper drainage of water away from the road structure. Site grades should be sufficient to prevent water ponding on or adjacent to the roadway.
7. The following 2 year staged pavement design may be applied to the proposed roadways. An

estimated California Bearing Ratio of 3.0 percent is used in the design, as well as a design life of 20 years. The traffic numbers are estimates only, and must be confirmed by others. Collector roads may require increased structures. Road configurations and traffic loading should be analyzed to see where collector structures may be needed.

Table 3: Recommended Roadway Structures

	Traffic Loading	Local Residential	Minor Collector ($1.0x10^5$ ESALs)
Stage 1	Asphaltic Concrete	65 mm ACR	75 mm ACO
	Crushed Gravel (20 mm)	300 mm	400 mm
Stage 2	Asphaltic Concrete	40 mm ACR	40 mm ACO

Note: ACR = City of Edmonton Asphaltic Concrete Residential or equivalent
ACO = City of Edmonton Asphaltic Concrete Overlay or equivalent
All granular base material should be compacted to 100 percent of the Standard Proctor Density in maximum 150 mm lifts.
Cement stabilized clay subgraded is recommended
A minimum of 10 kg of cement per square metre of subgrade mixed to a depth of 150 mm

7.4 House Foundations

1. No major problems are anticipated with construction of residential units or small buildings on the native non organic soils encountered throughout this site. The proposed housing units may be supported by continuous or spread footings bearing on undisturbed native silty clay, silt, or sand soils. However, should soft soils be encountered the bearing capacity may fall below the minimum 75 kilopascals required for applying the Alberta Building Code Section 9. In such cases a wider strip footing will be required. It should be noted that the clay soils are medium to high plastic and do have low to moderate potential for swelling. Organics, organic soils or uncontrolled fills are considered unsuitable for footing or slab on grade support. It is not recommended to place footings below the water table level.
2. Proper lot grading away from the houses must be provided to minimize the ingress of surface water into the subsoil. All houses will require at least 1.5 metres of earthen cover to prevent potential frost heave problems, and to minimize movements associated with seasonal variations in moisture content. The amount of cover should be increased to 2.0 metres for exterior isolated footings or for footings of non-continuously heated structures.

3. Lot grading plans are not known at this time. If general lot grading will produce areas of fill extending in depth below that of the footing elevation, it is strongly recommended that qualified geotechnical personnel inspect the house excavations. Generally, it is not recommended that footings be constructed on non-engineered fill. In such cases, the following alternatives are commonly recommended:

   i) Removal of the fill down to native soil and replacement with a compacted granular material. A normal footing foundation may then be utilized.

   Or

   ii) Utilize a cast-in-place pile foundation.

4. In the case of pile foundations, some installation problems may be encountered. Very moist soil conditions were encountered in the testholes in the low lying areas adjacent to Allan Beach Pond and Hubbles Lake below approximately 3 to 4 metres. Also difficulties may be encountered in the sand layers or silt layers that were noted in several testholes. Casing may be required in some areas, however the need for casing should be limited outside of the site low lying areas. At a minimum, pile concrete should be on-site during the pile drilling to allow for quick concrete placement.

   As an alternate to cast-in-place piles, screw piles would be considered a foundation option for the proposed residences. The design and installation of screw piles is commonly undertaken on a design-build basis. Hoggan can, provide a preliminary design or review a design for screw pile installations prepared by others. If screw piles are considered please contact Hoggan. Design and review services would be beyond the scope of this report.

5. Engineered fill may be considered in areas where low elevations necessitate deep fill zones. This option should be reviewed prior to implementation by a geotechnical consultant to evaluate site conditions and borrow material sources. Basically, engineered fill is fill, which is placed in a controlled manner under the full-time inspection of a qualified soils technician. The fill is placed and compacted to a minimum 98 percent of its Standard Proctor Density near its optimum moisture content, in maximum 150 millimetre lifts. All topsoil and non-engineered fill must first be stripped from the

engineered fill area. It should be noted that engineered fill requires fill depth differentials across the building footprint of less than 1.5 metres.

Engineered fill construction requires full-time monitoring and extensive testing by the geotechnical consultant during construction. However, proper placement of engineered fill will negate the need for pile foundations in deep lot fill areas, and possibly reduce the foundation costs to the builders and developer.

It should be noted that engineered fill construction is not possible in all situations. One of these situations occurs when soft, very moist, underlying soils are exposed once stripping has been completed. Compacting the first lift of fill material over these soft underlying soils to the engineered fill standard may be impossible. Where a minimum fill depth condition is met, construction of a clay pad approximately of 300 to 500 millimetres in thickness will be required to obtain an adequate working platform to start from. This pad should be compacted to a minimum of 95 percent of Standard Proctor Density where possible. The normal engineered fill lift thickness and compaction criteria mentioned above should be applied to successive lifts. To employ this method, a minimum of 1.0 metre of engineered fill must be placed on top of the clay pad. If this condition is not met, the fill would not be considered to have met engineered fill standards. In some cases, removal of native material may allow for the minimum fill depth or the maximum fill differential conditions to be met. However, this may not be the most economical solution.

6. No loose, disturbed, remoulded 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 remote from the bearing surface.
7. Footing excavations should be protected from drying, rain, snow, freezing and the ingress of surface or groundwater. Care should be taken to ensure that all exposed soils are protected from excessive drying or wetting. The soils encountered below the topsoil in the testholes were mostly medium to high plastic, and have a moderate swelling potential.

8. A 150 millimetre layer of free draining sand or gravel should be placed immediately below all floor slabs. This material should be uniformly compacted to 98 percent of the corresponding Standard Proctor Density at optimum moisture content.

9. A non-deteriorating vapour barrier should be placed immediately below the floor slab to prevent desiccation of the subgrade material.

10. Temporary dewatering may be required for basement excavations advanced below the water table. It is recommended to keep the house footing elevations above the water table.

11. Basements should be provided with a suitable peripheral drainage system with an adequate filter placed at footing elevation and connected positively with an approved drainage system. More recommendations on foundation drainage system can be found in Section 7.6.

12. The time span between the start of excavation to installation of basement footings, walls, peripheral weeping tile and backfilling operations should be minimized in order to prevent any problems developing within the excavation due to ingressing of ground or surface waters or desiccation of the subsoil.

13. 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.

14. 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.

7.5 Cement

Tests on selected soil samples indicated negligible to severe concentrations of water soluble soil sulphates throughout the underlying deposits. Based on observed, sulphate levels, and C.S.A. A23.1-04, class of exposure S-2 should be applied to the design requirements for all concrete in contact with the soil and susceptible to sulphate degradation. With respect to the resistance to sulphate attack the following alternatives are advised. Other criteria should be considered when specifying a concrete, see CSA A23.1-04 for further design criteria.

1. Underground Concrete Pipe

   Concrete used for all underground pipes must be constructed of C.S.A. Type HS, sulphate resistant Hydraulic cement.

2. Curbs and Sidewalks

   All concrete for surface improvements such as sidewalks and curbs may be constructed using CSA Type GU, general use Hydraulic cement.

3. Foundation Construction

   All concrete coming into direct contact with the soil must be made with CSA Type HS, sulphate resistant Hydraulic cement. As a minimum concrete should have a minimum 56 day compressive strength of 32 megapascals and be air entrained. In addition, all concrete subject to freeze thaw must be air entrained with 5 to 7 percent air.

7.6 Groundwater Issues

1. At least the top 1.0 metre of backfill around basement walls or underground building structures must be a suitable impermeable clay material. The near surface clay materials found at this site will be suitable for this purpose. This serves to reduce water penetration into the backfill, and subsequently into the weeping tile system.
2. Peripheral weeping tile lines will be required for all basements or underground building structures. All lines should be placed at or slightly below footing elevation and will require a suitable clean tile rock drainage filter, with a minimum of 150 millimetres of rock wrapped in filter cloth around the line. Interior weeping tile and other measures will also be required in structures near or below the groundwater table. A schematic sketch of such foundation drain service is attached in the Appendix.

3. The water table readings at this site were low to high and quite variable, between 2.3 metres and 17.6 metres BGS and dry in four of 19 testholes. Deep cut areas should be avoided during grading design in the moderate to high water table areas. Saturated conditions may be encountered in the trenches below these depths. Free water and sloughing conditions were encountered during drilling and as are outlined at the end of section 5.0. Water table levels indicate that trenches left open for extended periods may experience slow to considerable ingressing water. A moderate to considerable amount of ingressing water is anticipated in the areas where sand, sandy deposits of clay were encountered. Temporary dewatering will likely be required and would consist of in-trench sumps and pumping. Opening relatively long portions of utility trench is not recommended for this site. Deep sewers, below the water table, may require well point dewatering to allow for utility installation.

7.7 Slope Assessment

An assessment was conducted for the slopes located within the site of the proposed Allan Beach Resort. Assessments were conducted at four locations, which represented the steepest grades as noted on the topographical plan of the area. The following section presents the slope assessment completed, and outlines recommendations that are related to maintenance of continued slope stability.

Soil Design Parameters

For slope stability modelling using GSLOPE software the soil properties including unit weights, and effective strength parameters were required. These values, as summarized below, were determined from evaluation of the field and laboratory data outlined in this geotechnical report.

Table 4: Soil Parameters for Slope Modelling

Soil Type	Unit Weight	Effective Cohesion	Effective Friction Angle
	kN/m^3	kPa	degree
Organics	10	0	0
Fill	15 - 18	0	10 - 15
Silt / Clay	19	0	25
Clay	19	0 - 2	23 - 28
Silt / Sand	20	0	23 - 32
Sand	20	0	30

Groundwater

The stabilized ground water level in testholes advance over the entire site was recorded to be at elevations of between 26.5 and 27.0 metres. For the stability analysis a water table rise of 4.0 metres above the maximum current level recorded, elevation 31.0 metres, was assumed at the top of slope. Progressing to Allan Beach Pond or Hubbles Lake from the location near the top of slope, the water level was estimated to slope to an elevation 28.5 metres. An Elevation of 28.5 metres would be 1.5 metres above the Allan Beach Pond and Hubbles Lake elevation at the time of this investigation. These water levels would be considered conservative and were used to represent a potential long term rise in the water table due to lawn watering.

Erosion

During the site review, visual confirmation of slope toe erosion was encountered. The erosion was a result of previous work on the site. Specifically the work involved the removal of soil at the slope toe to provide room for recreation vehicle sites and in some instances to facilitate terraced gardens extending up the slope. The extent of the erosion was variable and was limited to the bottom 3 metres of the slope toe. The modeling undertaken accounted for the toe disturbance by assuming an over steepened slope as a result of erosion.

GSLOPE Slope Analysis

As part of the slope stability assessment, computer modelling using GSLOPE software was undertaken. The GSLOPE software uses an Interactive Limit Equilibrium Slope Stability Analysis methodology. With this modelling a factor of safety (FOS) against failure is given for a specific failure surface. A FOS 1.0 or less represents failure. For slope stability, with respect to building construction adjacent to the slope, a minimum FOS against slope failure of 1.5 is recommended in the GSLOPE modelling.

The topographical plan with elevation contours in 1.0 metre increments was provided by the client and was utilized to obtain the ground surface profiles required in the GSLOPE modelling. The profiles labelled 2-7, 11-5, 14 and 15-6 are shown. The labels denote the nearest testholes to the individual profile ie: Profile 2-7 is through/near Testholes 08-2 and 08-7.

For the GSLOPE modelling, the FOS against failure were determined for both circular and block failure modes. Graphic printouts of the GSLOPE trial failure surfaces modelled are enclosed in the appendix. The results of the GSLOPE analysis for failure surfaces through the top of slope are presented in the following table. This modelling indicates that the FOS against failure, with a failure surface passing through the top of slope, was a minimum of 1.51 as noted for Profile 14.

Table 5: Slope Stability Summary	
Failure Surface Through Top of Slope	**Factor of Safety**
Profile 2 - 7	1.63
Profile 11 - 5	1.86
Profile 14	1.51
Profile 15 - 6	1.74

Assessment Conclusions and Recommendations

Based on the slope assessment conducted, as outlined in this report, the slopes within the subject property as evaluated as part of this assessment do not present a natural hazard to the development.

It is critical that the design, constructing and ongoing maintenance of the development does not adversely affect ground conditions. The following development recommendations are directly related to maintenance of continued slope stability.

1. No development or building should be allowed on the slopes. Disturbance of vegetation cover over the slope should be minimized, and where disturbed re-established as soon as possible to prevent erosion and water permeation into the ground. Revegetation of the slope and top of slope areas should be undertaken with native plant species. The silt and sand soils are highly susceptible to erosion and considerable efforts will be required to prevent erosion on disturbed slopes.
2. Building and recreation vehicle site grades should be maintained at or below existing levels. For lots adjacent to the top of slope, proposed grading with fill depths over 1.0 metre within the building site should not be allowed without a detailed reviewed by a qualified geotechnical engineer. Lot grading should be such that ponding is avoided. No fill or other loads should be placed on the slope.
3. No site or building run off should be allowed to collect or concentrate, and drain over the escarpment or on the slope as the soils are highly susceptible to erosion. Any surface erosion should be immediately corrected and vegetation started to prevent further erosion.

4. It is noted that ingressing surface water may make these slopes unstable. No automatic sprinklers or over watering should be allowed within 30 metres of the top of slope or on the slope. Sources of water should be eliminated or minimized. No soil water retention structures are recommended for this site. Septic field should not be constructed on the slope or near the top of slope.
5. The design subdivision layout is not known by Hoggan at this time. It is assumed that no development is planned on the slopes. Hoggan should review the subdivision design to help ensure the geotechnical issues are adequately addressed.

8.0 CLOSURE

This report has been prepared for the exclusive and confidential use of the TRG Developments Corp. and their representatives. Use of this report is limited to the proposed development at the investigated location only. The recommendations given are based on the subsurface soil conditions encountered during test boring, 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 test boring location. Should other soils be encountered during construction or other information pertinent become available, the undersigned should be contacted as the recommendations may be altered or modified.

The Owner of the subject property 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 Owner.

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

Respectfully Submitted,

HOGGAN ENGINEERING & TESTING (1980) LTD.

John Tsoi, E.I.T.

Reviewed by,

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