June 21, 2018
Ref. No.: T18733

Mansfield Ski Club
628213 Sideroad 15
Mulmur, Ontario
L9V 3M6

Attention: Mr. Finley McEwen

Dear Mr. McEwen,

RE: **FEASIBILITY ASSESSMENT**
PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
MANSFIELD SKI CLUB
628213 SIDEROAD 15, MULMUR, ONTARIO

Please find enclosed the Feasibility Assessment - Preliminary Geotechnical Investigation Report prepared for the above-mentioned project. Should you have any questions or require any clarifications, please do not hesitate to contact our office.

We thank you for giving us this opportunity to be of service to you.

Sincerely,
Shad & Associates Inc.

H. Shad

Houshang Shad, Ph.D., P. Eng.
Principal

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FEASIBILITY ASSESSMENT
PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
MANSFIELD SKI CLUB
628213 SIDEROAD 15, MULMUR, ONTARIO

Submitted to:

Mansfield Ski Club
628213 Sideroad 15
Mulmur, Ontario
L9V 3M6

Attention:

Mr. Finley McEwen

Submitted by:

Shad & Associates Inc.
83 Citation Drive, Unit 9
Vaughan, Ontario, L4K 2Z6
Canada
Tel: (905) 760-5566
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1.0 INTRODUCTION

Shad & Associates Inc. was retained by Mansfield Ski Club ('Client') to carry out a preliminary geotechnical investigation for the proposed development being considered at 628213 Sideroad 15 in Mulmur, Ontario. The site location is shown in Figure 1. We understand that the preliminary geotechnical report is required to confirm the general viability of each of the following proposed works:

- Construction of a 6.0 m deep snow making pond on the north end of the property, adjacent to 17 Sideroad;
- Addition of 25 m of fill to be placed on top of the ski hill;
- Construction of six block of chalet type housing units (Blocks 1 to 6), two buildings (Building A and B), a water treatment plant and firefighting tanks to be located on the north end of the main parking lot at Sideroad 15; and
- Construction of a 1.2 to 2.2 m deep Dry Detention Basin to be located at the southeast portion of the site (south of the proposed building structures).

The Client requested the following boreholes to be drilled:

- one borehole in the vicinity of the 6.6 m snow making pond;
- three boreholes for the top of the ski hill where 25 m of fill will be placed;
- three boreholes in the vicinity of proposed buildings, water treatment plant and firefighting tanks; and
- one borehole in the vicinity of the dry detention basin.

Authorization to proceed with this investigation was provided by Mr. Dave Morrison of Mansfield Ski Club on May 18, 2018. The work carried out for this investigation was completed in accordance with Shad Proposal P18666-Revised, dated May 7, 2018.

The purpose of the current preliminary geotechnical feasibility assessment was to obtain some general information about the subsurface conditions at the site by means of a number of boreholes. Based on our interpretation of the data obtained, some preliminary recommendations are provided on the geotechnical aspects of design for the proposed development.

This report contains the findings of our geotechnical investigation, together with our recommendations and comments. These recommendations and comments are based on factual information and are intended only for use by the design engineer.

We recommend on-going liaison with Shad & Associates Inc. during the design and construction phases of the project to ensure that the recommendations provided in this report are applicable and/or correctly interpreted and implemented. Also, any queries concerning the geotechnical aspects of the proposed project should be directed to Shad & Associates Inc. for further elaboration and/or clarification.

2.0 INVESTIGATION PROCEDURES

The fieldwork for the investigation was performed during the period of May 24, 25, 29 and 30, 2018, and it consisted of drilling and sampling altogether eight boreholes down to depths ranging from approximately 4.7 m to 12.3 m below the existing ground surface. The borehole locations were stake-out by the Client and their approximate locations are shown in Figure 2. The Client also provided us with a base plan showing some limited existing topographical information at the development site as well as proposed grading information (un-dated, un-numbered plan 161117-Baseplan-15-319-1 as well as the landscape architectural plan prepared by Stempski Kelly Associates Inc. (Figure 2, dated 2009)). These plans were used to extrapolate the “approximate” existing ground surface elevations at the borehole locations. However, the elevations should only be considered as being approximate and should be confirmed once the actual survey information is received from the Client. We have assumed the elevations to be Geodetic.

The boreholes were advanced using solid and hollow stem continuous flight augers, with a track-mounted drilling rig, under the full-time supervision of geotechnical personnel from our office. Soil samples were taken at 0.76 to 1.5 m intervals for the full depth of the investigation and the Standard Penetration Test (SPT) was performed in accordance with ASTM D1586. This consists of freely dropping a 63.5 kg (140 lbs) hammer a vertical distance of 0.76 m (30 inches) to drive a 51 mm (2 inches) diameter o.d. split-barrel (split spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m (12 inches) is recorded as the SPT ‘N’-value of the soil and this gives an indication of the consistency or the relative density of the soil deposit.

Upon completion of boreholes, the soil samples were transported to our Soils Laboratory for further examination and laboratory testing. Soil laboratory testing, consisting of moisture content determination, gradation analysis (Sieve and Hydrometer tests) and Atterberg Limits (Liquid and Plastic Limits), were performed on selected representative soil samples. The results of the in-situ and laboratory tests are presented on the corresponding Record of Borehole Sheets as well as in Enclosure A.

Samples obtained during this investigation will be stored in our Soils Laboratory for three months and will be disposed thereafter.

3.0 SUB-SURFACE CONDITIONS

The stratigraphic units and groundwater conditions are briefly discussed in the following sections for each of the proposed works. For more detailed information, reference should be made to the Record of Borehole Sheets.

3.1 Proposed Snow Making Pond (Borehole 1)

The snow making pond is proposed to be located on the north end of the property, adjacent to 17

Sideroad. As requested, Borehole 1 was drilled at this location. Based on the subsurface conditions encountered at this borehole, the site is underlain by topsoil and fill extending down to a depth of about 0.8 m below existing grade. It should however be noted that the thickness and quality of topsoil and fill can vary significantly beyond the borehole location. Considering this, the extent of fill at the site and the limited size of an auger hole, we recommend that allowance be made for possible variations when making estimates. Alternatively, the depth and quality of topsoil and fill could be further investigated by test pitting.

The fill was then underlain by gravelly sand and layers of sand deposits with occasional gravel, silty sand interbeddings, trace to some silt and clay, extending to the completion of the borehole at approximately 9.0 m below existing ground surface.

Standard Penetration Tests were performed at the site and the recorded 'N'-values within the gravelly sand and sand layers were found to range widely from 8 to more than 50 blows/0.3m, indicating a loose to very dense, but generally compact relative density. Samples from these deposits were also tested for natural moisture content and the results were found to generally range from 8 to 19%. Considering these results as well as visual and tactile examination of the recovered soil samples, the deposits were generally wet.

Representative samples from the sand deposits were tested for gradation analysis. The results are presented on the Record of Boreholes as well as in Enclosure A and they are summarized below:

	BH 1: S5	BH 1: S8	BH 1: S10
Gravel:	1%	0%	11%
Sand:	92%	74%	87%
Silt and Clay:	7%	26%*	2%

* (Silt: 10%, Clay: 16%)

The groundwater condition at this borehole was monitored during and upon the completion of drilling as well as by installing a monitoring well. The results are summarized in below:

Table 1: Measured Groundwater data

Borehole	"Approx." Geodetic Ground Surface Elevation (m)	Measured Groundwater Depth / Elevation (m)		
		Upon Completion	June 6, 2018	June 13, 2018
BH 1	~264	2.0 / 262	+ 0.9 / 264.9	+ 0.8 / 264.8

It should be mentioned that the groundwater condition at the site would fluctuate seasonally and can be expected to be somewhat higher during the spring months and in response to major weather events.

3.2 Heightening the Top of the Existing Ski Hill (Boreholes 2 to 4)

Boreholes 2, 3 and 4 were drilled for this proposed work and they encountered some surficial topsoil that was underlain by fill, generally consisting of silty clay/clayey silt with occasional organic stains, trace to some rootlets and topsoil, that extended down to depths ranging from approximately 0.7 m in Borehole 2 to 5.2 m in Borehole 3. The fill at Borehole 2 was in turn underlain by a 0.3 m thick topsoil (may be the original topsoil layer). The recorded 'N'-values within the silty clay to clayey silt fill ranged from about 6 to 12 blows/0.3 m. Samples from the fill layer were also tested for moisture content and the results were found to generally range from 12 to 23%. The collected fill samples were generally damp and the higher measured moisture content values could be due to the presence of organic content within the fill deposit. Considering the above results, we are of the opinion that the fill has received some non-systematic compaction and quality control. It should however be noted that the thickness and quality of topsoil and fill can vary significantly between and beyond the boreholes locations. Considering this, the extent of fill at the site and the limited size of an auger hole, we recommend that allowance be made for possible variations when making estimates.

Native silty clay/clayey silt was encountered below the fill layer in Boreholes 2 and 4 and below the lower topsoil layer in Borehole 3 and it extended down to depths ranging from about 0.8 m to 10.0 m below existing grade, where it was in turn underlain by highly weathered to weathered shale.

The measured 'N'-values within the silty clay/clayey silt ranged from about 14 to more than 30 blows/0.3 m, indicating a stiff to hard, but generally very stiff to hard consistency. Representative samples from this layer were also tested for natural moisture content and the results were found to range from 10 to 17%. Based on these results as well as visual and tactile examination of the recovered soil samples, the silty clay/clayey silt was generally damp. A representative sample from this deposit was analyzed for gradation and Atterberg Limits. The results are presented on the Record of Boreholes as well as in Enclosure A and they are summarized below:

	BH 4: S54
Gravel:	0%
Sand:	4%
Silt:	52%
Clay:	44%
Liquid Limit:	36%
Plastic Limit:	21%
Plasticity Index:	15%

Considering the above results, the silty clay/clayey silt has medium plasticity.

Highly weathered to weathered shale was encountered below the silty clay to clayey silt at all three boreholes. This was further confirmed by drilling additional two shallower boreholes at a 2 m radius around Borehole 4 to ensure that a large cobble or boulder was not being encountered. The recorded 'N'-values within the highly weathered to weathered shale deposit were all well in excess of 50 blows/0.3 m. The natural moisture content test results performed on selected samples ranged from 5 to 9%. It should however be noted that the quality and the surface elevation of the highly weathered to weathered shale as described in the borehole logs should be considered as approximate only, as they were inferred from the observations during drilling rather than proven by rock coring.

The groundwater condition at these boreholes were monitored during and upon the completion of drilling as well as by installing a monitoring well in Borehole 3. The results are summarized in Table 2, below:

Table 2: Measured Groundwater data

Borehole	"Approx." Geodetic Ground Surface Elevation (m)	Measured Groundwater Depth / Elevation (m)		
		Upon Completion	June 6, 2018	June 13, 2018
BH 2	~377	Dry	-	-
BH 3	~385	12.2 / 372.8	11.1 / 373.9	11.1 / 373.9
BH 4	~374	Dry	-	-

It should be mentioned that the groundwater at the site would fluctuate seasonally and can be expected to be somewhat higher during the spring months and in response to major weather events. Furthermore, a perched water condition may also exist within the fill deposit.

3.3 Proposed Housing Units, Buildings, Water Treatment Plant & Firefighting Tanks (Boreholes 5 to 7)

As requested, Boreholes 5 to 7 were drilled to obtain some general information below the proposed six block of chalet type housing units (Blocks 1 to 6), two buildings (Building A and B), a water treatment plant and firefighting tanks to be located on the north end of the main parking lot at Sideroad 15. Based on the subsurface conditions encountered at these boreholes, fill soils generally consisting of silty sand to sandy silt, granular fill and clayey silt were contacted at all boreholes that extended down to depths ranging from approximately 0.7 m (at Boreholes 5 and 7) to 2.2 m (at Borehole 6) below existing ground surface. However, at Borehole 6, a topsoil layer was also contacted interbedded within the fill deposit. It should be noted that the thickness and quality of fill and topsoil can vary significantly in between and beyond the borehole locations. Considering this, the extent of fill at the site and the limited size of an auger hole, we recommend that allowance be made for possible variations when making estimates. Alternatively, the depth and quality of topsoil and fill could be further investigated by test pitting.

The fill layer at all boreholes was then underlain by glacial deposits, generally consisting of silty sand till and/or sandy silt till, that extended down to the completion of the boreholes at 4.9 to 5.0

m below existing grade. The recorded ‘N’-values within the glacial deposits ranged from 16 to more than 50 blows/0.3 m penetration, indicating a compact to very dense, but generally dense to very dense relative density. Samples from these deposits were also tested for natural moisture content determination and the results within the sandy silt till were 12 and 13% and within the silty sand till ranged from 7 to 11%. Considering these results as well as visual and tactile examination of the recovered soil samples, the silty sand till and sandy silt till deposits were generally damp and occasionally damp to moist or moist.

Representative samples from the silty sand till and sandy silt till deposits were tested for gradation analysis. The results are presented on the Record of Boreholes as well as in Enclosure A and they are summarized below:

	BH 5: S2	BH 5: S5	BH 7: S2
Gravel:	13%	5%	2%
Sand:	59%	59%	24%
Silt:	28%	35%	69%
Clay:	0%	1%	5%

It should be noted that the occurrence of cobbles and boulders should always be expected when working in glacial till deposits.

The groundwater condition at these boreholes were monitored during and upon the completion of drilling as well as by installing a monitoring well in Borehole 7. The results are summarized in Table 3 below:

Table 3: Measured Groundwater data

Borehole	“Approx.” Geodetic Ground Surface Elevation (m)	Measured Groundwater Depth / Elevation (m)		
		Upon Completion	June 6, 2018	June 13, 2018
BH 5	~307	3.2 / 303.80	-	-
BH 6	~302	4.3 / 297.7	-	-
BH 7	~286	4.5 / 281.5	2.6 / 283.4	2.8 / 283.2

It should be mentioned that the groundwater at the site would fluctuate seasonally and can be expected to be somewhat higher during the spring months and in response to major weather events. Furthermore, a perched water condition may also exist within the fill deposit.

3.4 Proposed Dry Detention Pond Basin (Borehole 8)

The dry basin is proposed to be located south of the proposed building structures, on the southeast portion of the site. Based on the subsurface conditions encountered at Borehole 8 drilled for this structure, the site is underlain by a surficial topsoil layer followed by fill that extended down to approximately 4.4 m below the ground surface. The fill generally consisted of sandy silt with trace clay, organic stains and some topsoil. The fill was found to overlie a layer of deeper topsoil (may be the original topsoil layer) that extended to about 4.8 m below existing grade. It should be noted that the thickness and quality of fill and topsoil can vary significantly beyond the borehole location. Considering this, the extent of fill at the site and the limited size of an auger hole, we recommend that allowance be made for possible variations when making estimates. Alternatively, the depth and quality of topsoil and fill could be further investigated by test pitting.

The fill deposit was then underlain by a relatively thin layer of silty sand to sandy silt that extended to approximately 5.2 m below grade. The recorded 'N'-value with this deposit was 6 blows/0.4 m with a moisture content of more than 40%, indicating a loose and wet condition.

The silty sand to sandy silt was then underlain by glacial till deposits consisting of silty sand till or silty sand to sandy silt till that extended down to the completion of the borehole. The measured 'N'-values within these layers ranged widely from 22 to more than 50 blows/0.3 m penetration, indicating a compact to very dense relative density. Samples from these deposits were also tested for natural moisture content determination and the results were found to range from 9 to 13%. Considering these results as well as visual and tactile examination of the recovered soil samples, the silty sand till and/or sandy silt till deposits were wet at higher elevations and became damp to moist with increased depth.

A representative sample from the silty sand to sandy silt till deposit was tested for gradation analysis. The results are presented on the Record of Boreholes as well as in Enclosure A and they are summarized below:

	BH 8: S11
Gravel:	3%
Sand:	50%
Silt:	42%
Clay:	5%

It should be noted that the occurrence of cobbles and boulders should always be expected when working in glacial till deposits.

The groundwater condition at this borehole was monitored during and upon the completion of drilling as well as by installing a monitoring well. The results are summarized in below:

Table 4: Measured Groundwater data

Borehole	"Approx." Geodetic Ground Surface Elevation (m)	Measured Groundwater Depth / Elevation (m)		
		Upon Completion	June 6, 2018	June 13, 2018
BH 8	~296	4.6 / 291.4	3.7 / 292.3	3.8 / 292.2

It should be mentioned that the groundwater at the site would fluctuate seasonally and can be expected to be somewhat higher during the spring months and in response to major weather events. Furthermore, a perched water condition may also exist within the fill deposit.

4.0 DISCUSSION AND RECOMMENDATIONS

According to the preliminary information provided to us, we understand that the general viability of the following works is being considered at the site:

- Construction of a 6.0 m deep snow making pond on the north end of the property, adjacent to 17 Sideroad;
- Addition of 25 m of fill to be placed on top of the ski hill;
- Construction of six blocks of chalet type housing units (Blocks 1 to 6), two buildings (Building A and B), a water treatment plant and firefighting tanks to be located on the north end of the main parking lot at Sideroad 15. The proposed buildings appear to be slab-on-grade structures; and
- Construction of a 1.2 to 2.2 m deep Dry Detention Basin to be located at the southeast portion of the site (south of the proposed building structures).

Considering the above information and the subsurface conditions encountered at the borehole locations, some discussions and recommendations are provided in this section. However, they should be considered as general in nature and will need to be reviewed and confirmed by supplementary geotechnical investigations once the exact project details are known.

4.1 Construction of a Snow Making Pond

According to the information provided to us, the pond will be located on the north end of the property, adjacent to 17 Sideroad and will be 6.0 m deep with a base elevation of 257.40 m and an approximate side slope of 3H:1V.

Based on the preliminary topographic information provided to us, the existing ground surface elevation within the pond footprint generally varies from about 263 to 264 m and therefore the pond will be generally in cut.

Borehole 1 was drilled at this location. Based on the subsurface conditions encountered at this location, the pond base and walls would generally be within gravelly sand and sand deposits with the exception of the top part of the pond walls, where some engineered fill will be required to replace any existing topsoil and silty sand fill. Furthermore, the short-term groundwater level in the monitoring well installed in this borehole is measured above the existing ground surface, at Elevations 264.8 to 264.9 m and this will fluctuate with time.

Considering the above information, the stability of the pond walls was assessed by assuming a representative Section A, as shown in Figure 3. The section was analysed by assuming conservative soil parameters, as summarized in Table 5 below, based on the borehole information, the field and laboratory tests performed, our experience with similar site conditions as well as published geotechnical data.

Table 5: Assumed Conservative Geotechnical Parameters

Soil Type	Bulk Unit Weight (kN/m^3)	Shear Strength Parameters			
		C' (kPa)	Φ' (degree)	C_u (kPa)	Φ_u (degree)
Loose Fine Sand, Trace to Some Clay and Silt	16.5	0	20	10	10
Compact Sand, Gravelly Sand	19.0	0	30	0	30
Dense to Very Dense Sand	21.0	0	32	0	32
Engineered Fill (Sandy)	19.0	0	30	0	30

For slope stability analysis, computer program Slope/W 2012 and the Bishop's Simplified method for the calculation of the factor of safety for slip surface were used. For a slope to be assessed as being stable, a minimum Factor of Safety of 1.5 is normally required under a static loading condition.

The assumed cross-section was analysed under the following conditions:
-During Construction (Undrained Analysis); and
-Full Pool (Drained Analysis).

Furthermore, the pond wall was analysed under seismic loading conditions. The site specific seismic hazards as per National Building Code of Canada (2015) were obtained from Earthquakes Canada website (www.EarthquakesCanada.ca) and are provided in Appendix A. The peak ground acceleration (PGA) for 2 percent probability in 50 years (0.000404 per annum or return period of 2,475 years) for the site is 0.062g corresponding to Site Class C. For this study, although a geophysical assessment was not completed in assessing the applicable seismic site classification, considering the subsurface conditions encountered at the boreholes drilled at the site, a site Class D (Stiff Soil) is recommended for the property. Therefore, the peak ground acceleration corresponding to Site Class D at the site will be PGA=1.3×0.062g=0.0806g. According to industry standards, the acceleration used in pseudostatic analysis is equal to 0.5 × PGA. Based on these values and in accordance to the Canadian Foundation Manual (4^{th} Edition), the following parameters were used for seismic stability evaluations:

Horizontal Seismic Coefficient	=	0.5 X 0.0806g=0.0403g
Vertical Seismic Coefficient	=	0

For a stable slope under seismic loading using pseudostatic analysis, a minimum FOS of 1.1 is normally recommended.
Considering the above details, the stability of the assumed representative cross-section under construction and ponding conditions was analysed and some of the results are shown in

Enclosures B-1 to B-3. The results indicate that the calculated Factors of Safety for static and seismic loading conditions were all above the recommended minimum values. Based on these results, the pond walls are considered to be stable.

It should be noted that as the pond base and the walls will be constructed in sandy deposits with high groundwater levels, they would need proper protection against erosion and washout. Furthermore, if a permanent water level has to be maintained in the pond, a suitable impermeable liner would be required. The liner would need to be designed against uplift hydrostatic pressures due to high groundwater levels measured at the site. We will review and provide additional recommendations once the pond information is finalized.

The pond excavation would require temporary dewatering to lower the groundwater level to below the pond base. This should be provided by the Project Dewatering Consultant.

4.2 Heightening of the Top of the Ski Hill

According to the information provided to us, we understand that the top of the existing ski hill is proposed to be raised by as much as 25 m on the front side and also a retaining wall system, ranging in height from approximately 4 to more than 18 m, is being considered for the backside.

Boreholes 2, 3 and 4 were drilled within the proposed heightening part of the hill. Based on the subsurface conditions encountered at these boreholes, below some surficial topsoil, the existing top of the hill has about 5 m of silty clay/clayey silt fill at Borehole 3 and this reduces to about 0.2 m at Borehole 2 and to about 1.5 m at Borehole 4. The fill at Borehole 3 was in turn underlain by a 0.3 m thick deeper topsoil layer. The field and laboratory test results appear to indicate that the existing fill has received some relatively non-systematic low compaction and quality control. The topsoil and fill layers were in turn underlain by still to hard silty clay/clayey silt that was found to overlie highly weathered to weathered shale.

Based on the provided topographic survey information, two sections were assumed that pass through the front of the ski hill (i.e. Sections B and C). The sections were analysed by assuming conservative soil parameters, as summarized in Table 6 below, based on the borehole information, the field and laboratory tests performed, our experience with similar site conditions as well as published geotechnical data.

Table 6: Assumed Conservative Geotechnical Parameters

Soil Type	Bulk Unit Weight (kN/m^3)	Shear Strength Parameters			
		C' (kPa)	Φ' (degree)	C_u (kPa)	Φ_u (degree)
Topsoil	16.5	0	15	20	0
Silty Clay/Clayey Silt Fill	17.0	0	22	35	0
Stiff Silty Clay Clayey Silt	17.5	0	25	40	0
Very Stiff to Hard Silty Clay/Clayey Silt	19.5	5	30	100	0
Highly Weathered to Weathered Shale	-	-	-	-	-
Engineered Fill (Clayey)	19.0	0	30	90	0

The assumed cross-sections were analysed under the following conditions:

-During and Immediately After Construction (Undrained Analysis);
-Long-term after Construction (Drained Analysis); and
-Seismic Loading.

Considering the above details, the stability of the assumed representative cross-sections for during and after construction was analysed and some of the results are shown in Enclosure B-4 to B-9. The results indicate that the calculated Factors of Safety under static and seismic loading conditions were all above the recommended minimum values. Based on these results, the proposed slope heightening would have adequate factor of safety against slope failure for the front slope. However, additional boreholes should be drilled within the proposed filling area as well as at lower elevations down the slope face in order to better define the existing subsurface conditions.

For raising the slope, the following 'preliminary' placement procedure is recommended.

(i) The area to receive the engineered fill should be stripped of any topsoil and other compressible, weak and deleterious materials. After stripping, the entire area should be inspected and approved by the geotechnical engineer. Spongy, wet or soft/loose spots should be sub-excavated to stable subgrade and replaced with compactable approved soil, compatible with subgrade conditions, as directed by the geotechnical engineer.

(ii) The fill material should be placed in thin layers not exceeding approximately 200 mm when loose. Oversize particles (cobbles and boulders) larger than 120 mm should be discarded, and each fill layer should be uniformly compacted with heavy compactors, suitable for the type of fill used, to at least 100% of its Standard Proctor Maximum Dry Density.

The on-site inorganic soils are generally acceptable for use as engineered fill, provided they are not contaminated with the overlying organic rich deposits and any organic inclusions are removed. Depending on the construction season, the on-site soils may require some reconditioning, wetting or drying. It should also be noted that the sandy and silty deposits are sensitive to moisture and they will require a more strict control on their moisture content if they are to be used in the engineered fill operation.

(iii) Full-time geotechnical inspection and quality control (by means of frequent field density and laboratory testing) are necessary for the construction of a certifiable engineered fill. Compaction procedures and efficiency should be controlled by a qualified geotechnical technician.

(iv) The engineered fill should not be frozen and should be placed at a moisture content within 2% of the optimum value for compaction. The engineered fill should not be performed during winter months when freezing ambient temperatures occur persistently or intermittently.

The proposed fill will settle under self-weight and it will also cause settlement of the underlying existing fill and native soils. Assuming the above procedure, the total settlement is estimated to range from approximately 40 to 60 cm. However, depending on the actual organic content within the existing fill, the settlement values could be higher. Also, considering the cohesive nature of the existing fill and assuming that a similar soil type is used for raising the hill, the settlements should occur over several years. We would however recommend that the settlements at the site (within the proposed fill areas as well as down the existing hill) to be instrumented and monitored by surveying. Due to the non-homogeneous nature of the existing fill and the presence of topsoil and organic matters within the fill, some excessive differential settlement should also be expected.

With respect to the proposed retaining walls for the backside of the hill, we would recommend that global slope stability analysis to be performed once the wall designs are available. Additional boreholes would be required along the proposed wall to better identify the subsurface conditions. The wall designer should also confirm the internal stability of the walls (sliding, overturning and bearing capacity). We would recommend the walls to be placed on the native and competent silty clay/clayey silt deposit, properly engineered fill or on the highly weathered to weathered shale. Based on these as well as the wall design, additional geotechnical parameters and recommendations will be provided.

4.3 Proposed Housing Units, Buildings, Water Treatment Plant & Fire Fighting Tanks

According to the Client, six blocks of chalet type housing units, two buildings, a water treatment plant and firefighting tanks are being assessed for construction on the north end of the main parking lot at Sideroad 15. The proposed buildings appear to be slab-on-grade structures. Boreholes 5, 6 and 7 were drilled in this area. Based on the subsurface conditions encountered at these boreholes, below some fill and/or topsoil, the site is predominantly underlain by compact to very dense silty sand till and/or sandy silt till. Furthermore, the groundwater level over our short-term monitoring program was measured below a depth of about 2.6 m below existing ground surface.

4.3.1 Site Grading

The development of the site will require clearing and stripping of all topsoil and fill. Where residential lots or structures are being considered, it is recommended that all fill be placed as engineered fill to provide competent subgrade. Prior to placement of engineered fill, all the surficial topsoil and fill should be stripped from planned fill areas to expose the inorganic subgrade. The exposed subgrade should then be proof-rolled with a suitably heavy roller to identify weak areas. Any weak or excessively wet zones identified during proof-rolling should be sub-excavated and replaced with compacted competent material to establish stable and uniform conditions. Prior to placement of engineered fill, the subgrade should be inspected and approved by a geotechnical engineer. Reference is made to Section 4.3.4 for recommendations regarding engineered fill placement.

Provided the above recommendations are followed, and all topsoil and compressible materials are stripped or sub-excavated, the existing deposits are not considered to be highly compressible and long-term settlements should be minimal.

4.3.2 Foundations

Based on the subsurface conditions encountered at Boreholes 5, 6 and 7 drilled at the site, the footings would need to be extended down to the competent undisturbed native deposits or be placed on properly compacted engineered fill. The recommended spread footing depths and allowable soil bearing pressures are given in the following table.

Table 7: Recommended Soil Bearing Capacity Values

Borehole	**Depth Below Existing Grade (m)**	**Recommended Geotechnical Reaction at SLS * (kPa)**	**Factored Geotechnical Resistance at ULS (with a Geotechnical Resistance Factor of 0.5), (kPa)***
BH 5	± 1.1	150	225
BH 6	± 2.3	150	225
BH 7	± 1.1	150	225

* Higher Allowable Soil Bearing Capacity values are available at lower elevations, if required.

The minimum footing sizes, footing thickness, excavations and other footing requirements should be designed in accordance to the latest edition of the Ontario Building Code.

The footing subgrade should be inspected and evaluated by the Geotechnical Engineer prior to concreting to ensure that the footings are founded on competent subgrade capable of supporting the recommended design pressure.

Design frost penetration depth for the general area is 1.6 m. Therefore, a permanent soil cover of 1.6 m or its thermal equivalent is required for frost protection of foundations. All exterior footings and footings beneath unheated areas should have at least 1.6 m of earth cover or equivalent synthetic insulation for frost protection.

Where necessary, the stepping of the footings at different elevations should be carried out at an angle no steeper than 2 horizontal (clear horizontal distance between footings) to 1 vertical (difference in elevation) and no individual footing step should be greater than 0.6 m and may have to be as low as 0.3 m if weaker soils are encountered.

For footings designed and constructed in accordance with the above criteria, total and differential settlements should be less than 25 mm and 15 mm, respectively. These values are usually within tolerable limits for most types of structures.

4.3.3 Earthquake Considerations

In conformance to the Criteria in Table 4.1.8.4.A, Part 4, Division B of the National Building Code (NBC 2005), for footings designed as recommended in Section 4.3.2, the subject site is classified as Site Class “D-Stiff Soil”. The four values of the Spectral Response Acceleration $S_a(T)$ for the different periods and the peak ground acceleration (PGA) can be obtained from Table C-2 in Appendix C, Division B of the NBC (2005). The design values of F_a and F_v for the project site should be calculated in accordance to Table 4.1.8.4.B and C.

4.3.4 Engineered Fill

Depending on the proposed grades for the site, engineered fill may be required to replace the existing fill and topsoil as well as to raise the site grades for the possible support of footings and floor slabs. Engineered fill could be placed after stripping all topsoil, any soils containing excessive organics and otherwise unsuitable soils, within an area extending at least 2.5 m beyond the perimeter of the footprint of the proposed structures. Engineered fill would then be suitable to support the foundations including the slabs provided that the following criteria are strictly followed. Engineered fill may also be carried out to raise the existing grades below the proposed roads and parking lots.

The following placement procedure is recommended.

(i) The areal extent of engineered fill should be controlled by proper surveying techniques to ensure that the top of the engineered fill extends a minimum of 2.5 m beyond the perimeter of the buildings to be supported. Where the depth of engineered fill exceeds 1.5 m, this horizontal distance of 2.5 m beyond the perimeter of the building should be increased by at least 1.0 m for each 1.0 m depth of fill.

(ii) The area to receive the engineered fill should be stripped of any topsoil, fill and other compressible, weak and deleterious materials. After stripping, the entire area should be inspected and approved by the geotechnical engineer. Spongy, wet or soft/loose spots should be sub-excavated to stable subgrade and replaced with compactable approved soil, compatible with subgrade conditions, as directed by the geotechnical engineer.

(iii) The fill material should be placed in thin layers not exceeding approximately 200 mm when loose. Oversize particles (cobbles and boulders) larger than 120 mm should be discarded, and each fill layer should be uniformly compacted with heavy compactors, suitable for the type of fill used, to at least 98% of its Standard Proctor Maximum Dry Density.

The on-site inorganic soils are generally acceptable for use as engineered fill, provided they are not contaminated with the overlying organic rich deposits and any organic inclusions are removed. Depending on the construction season, the on-site soils may require some reconditioning.

(iv) Full-time geotechnical inspection and quality control (by means of frequent field density and laboratory testing) are necessary for the construction of a certifiable engineered fill. Compaction procedures and efficiency should be controlled by a qualified geotechnical technician.

(v) The engineered fill should not be frozen and should be placed at a moisture content within 2% of the optimum value for compaction. The engineered fill should not be performed during winter months when freezing ambient temperatures occur persistently or intermittently.

The allowable soil bearing pressure is 150 kPa for footings supported by at least 1.0 m of engineered fill constructed in accordance with the above recommendations. We also recommend that the footing subgrade be evaluated by the geotechnical engineer prior to placing the formwork. It is recommended to increase the rigidity of foundations of structures erected over engineered fill, and this is generally achieved by making the footings at least 0.5 m wide, and adding reinforcing rebars to the footings and walls. This measure helps to bridge over eventual weak spots in the fill.

All footings should have at least 1.6 m of earth cover or equivalent artificial insulation for frost protection.

For footings designed and constructed in accordance with the above criteria, total and differential settlements should be less than 25 mm and 15 mm, respectively. These values are usually within tolerable limits.

4.3.5 Excavating and Dewatering

All temporary excavations should be carried out in accordance with the Ontario Health and Safety Regulations. The soils to be excavated can be classified as follows:

-Topsoil / Fill	Type 4
-Compact Silty Sand/Sandy Silt Till	Type 3
-Dense to Very Dense Silty Sand/Sandy Silt Till (above groundwater level or when dewatered)	Type 2

Accordingly, for Type 3 Soils, a side slope of 1H:1V is required for excavations in accordance with the Ontario Health and Safety Regulations. Within Type 4 Soils, the side slope of the excavation would need to be flattened to at least 3H:1V. In Type 2 soils, the bottom 1.2 m of the excavation could be carried out close to vertical.

Stockpiles of excavated materials should be kept at least 5 m away from the edge of the excavation to avoid slope instability. This distance should be increased for any stockpiling along the top of the existing slopes (we should be informed to provide additional recommendations if soil stockpiling on top of slopes is being considered). Care should also be taken to avoid overloading of any underground services/structures by stockpiles.

Based on the subsurface conditions encountered at the boreholes, within the recommended depth for footings provided in Table 7, we anticipate all footing excavations to be above the measured groundwater levels, either in engineered fill or within native deposits. Considering this, we do not anticipate major dewatering problems for footing excavations, although some dewatering may have to be carried out for excavations due to surface runoff, from any perched water within the fill layer or groundwater seepage. We are of the opinion that these should be manageable by pumping from temporary sumps protected against erosion. Such sumps should be dug outside the footprint of the structures to minimize disturbance to the footing grade. We recommend that once the structure footing invert information are known and prior to construction, the groundwater conditions at the site to be further assessed by test pitting.

4.3.6 Building Slab Construction & Drainage

Concrete floor slab may be built on properly prepared subgrade or engineered fill. If the existing topsoil/fill is left underneath the slab, long-term settlement and/or cracks may occur. The existing fill and topsoil should be removed and replaced with compacted engineered fill in order to support the floor slab. For engineered fill subgrade, Section 4.3.4 should be followed.

Underneath the building slabs, a 150 mm thick base course consisting of 20 mm size clear stone or OPSS Granular A should be placed to improve the support for the floor slab and function as drainage layer. This base course should be compacted with vibratory equipment to a uniform high density. If the subgrade is wet, the clear stone or OPSS Granular A base should be separated from the subgrade by an approved filter fabric (e.g. non-woven geotextile, with FOS of 75 - 150 μm, Class II).

The site should be graded for drainage away from foundations. A minimum cross fall of three percent (3%) immediately adjacent to foundations is recommended to allow for some settlement and promote good surface drainage.

4.4 Proposed Dry Detention Basin

Based on the information provided to us we understand that a 1.2 to 2.2 m deep dry detention basin is proposed to be constructed on the southeast portion of the site, south of the proposed building structures. The dry pond base will be at Elevation 293.80 m and the top of the pond at Elevation 296.35 m. The pond will have a side slope of 4H:1V.

Based on the preliminary topographic information provided to us, the existing ground surface elevation within the pond footprint generally varies from about 295 to 297 m and therefore the pond will generally in cut.

Borehole 8 was drilled at this location. Based on the subsurface conditions encountered at this location, the pond base and walls would generally be within sandy silt fill with occasional topsoil interbeddings. Furthermore, the short-term highest groundwater level in the monitoring well installed in this borehole was measured at depth of about 3.7 m below existing ground surface and this will fluctuate with time.

Considering the above information, the pond will be placed within a fill deposit which appears to have received little to no compactive effort or quality control and will not provide a long-term stable pond structure. We would therefore recommend the fill to be removed and then the area to be engineered up. For recommendations on the construction of the engineered fill reference should be made to Section 4.3.4. It should be noted that the pond is designed as a dry pond and threfeore by definition it will not be holding a permanent water level. We would however recommend the pond side slopes to be protected against erosion and washout.

Considering the above information, the stability of the pond wall was assessed by assuming a representative section (i.e., Section F), as shown in Figure 5. The section was analysed by assuming conservative soil parameters, as summarized in Table 8 below, based on the borehole information, the field and laboratory tests performed, our experience with similar site conditions as well as published geotechnical data.

Table 8: Assumed Conservative Geotechnical Parameters

Soil Type	**Bulk Unit Weight (kN/m^3)**	**Shear Strength Parameters**			
		C' (kPa)	**Φ' (degree)**	**C_u (kPa)**	**Φ_u (degree)**
Engineered Fill (Sandy)	19.0	0	30	0	30

For slope stability analysis, computer program Slope/W 2012 and the Bishop's Simplified method for the calculation of the factor of safety for slip surface were used.

The assumed cross-section was analysed under the following conditions:

-During Construction (Undrained Analysis);
-Normal Water Level (Drained Analysis); and
-Seismic Loading.

Considering the above details, the stability of the assumed representative cross-section was analysed under construction, normal water level as well as seismic loading conditions. Some of the results are shown in Enclosure B-10 to B-15. The results indicate that the calculated Factors of Safety were all above the recommended minimum values for static and seismic loading conditions. Based on these results, the pond walls are considered to be stable.

The pond excavation would require temporary dewatering to lower the groundwater level down to the native deposit, so that the engineered fill could be constructed. Recommendations on the dewatering methodology should be provided by the Project Dewatering Consultant.

5.0 CLOSURE

As requested, the viability of some proposed works at the ski club was investigated and this preliminary feasibility report was prepared to summarise the subsurface findings at the requested borehole locations together with some preliminary geotechnical comments and recommendations. We would however recommend that once the development details are finalized and a detailed topographic survey map is available, our recommendations should be reviewed for their specific applicability and the recommendations to be finalized. A supplemental geotechnical investigation will be required.

The attached Report Limitations are an integral part of this report.

Sincerely,
Shad & Associates Inc.

LICENSED PROFESSIONAL ENGINEER
H. SHAD
PROVINCE OF ONTARIO

H. Shad

Stephen Chong, P. Eng.
Senior Engineer

Houshang Shad, Ph. D., P. Eng.
Principal

STATEMENT OF LIMITATION

The conclusions and recommendations given in this report are based on information obtained at the testhole locations. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction which could not be detected or foreseen at the time of the site investigation.

The information contained herein in no way reflects on the environmental aspects of the project, unless stated otherwise.

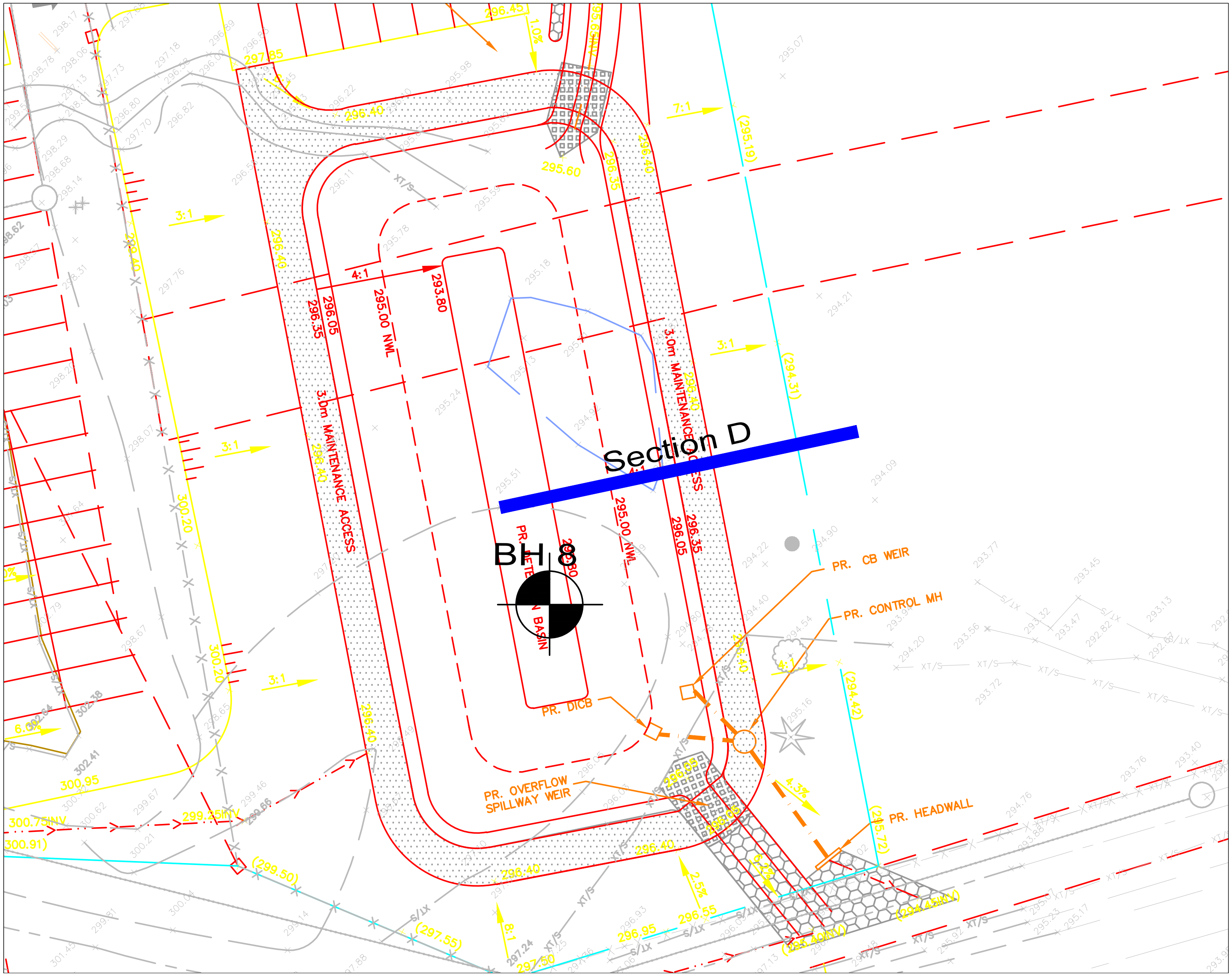
The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as planning, grading, excavating, etc.

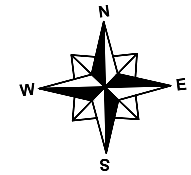
The design recommendations given in this report are project as well as site specific and then only if constructed substantially in accordance with the details stated in this report. We recommend, therefore, that we be retained during the final design stage to review the design drawings and to verify that they are consistent with our recommendations or the assumptions made in our analysis.

The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of the testholes may not be sufficient to determine all the factors that may affect construction methods and costs. The contractors bidding on this project or undertaking construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work.

We recommend that we be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, is the responsibility of such third party. We accept no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.





LEGEND:

BH 1

Borehole Location

Section D

Assumed Sections

NOTES:

1. All Borehole locations are approximate.
2. Drawing not to scale.
3. The base drawing was provided by the Client.
4. The drawing should be read in conjunction with the associated report by Shad & Associates Inc., T18733.

CLIENT: Mansfield Ski Club	**Drawn By:** M.Z.	**TITLE:** Assumed Section for Dry Detention Pond	**Date:** June, 2018
	Checked By: H.S.		**Project No.:** T18733
SHAD & ASSOCIATES INC. GEOTECHNICAL, ENVIRONMENTAL AND MATERIALS CONSULTING ENGINEERS 83 Citation Drive, Unit 9 Vaughan, Ontaruio, L4K 2Z6 Tel: (905) 760-5566 Fax: (905) 760-5567 www.shadinc.ca	**Datum:** - **Projection:** - **Scale:** N.T.S.	**PROJECT:** Feasibility Assessment Preliminary Geotechnical Investigation Proposed Development 628213 Sideroad 15 Mulmur, Ontario	Figure No.: 5

SHAD & ASSOCIATES INC.

EXPLANATION OF BOREHOLE LOG

This form describes some of the information provided on the borehole logs, which is based primarily on examination of the recovered samples, and the results of the field and laboratory tests. It should be noted that materials, boundaries and conditions have been established only at the borehole locations at the time of investigation and are not necessarily representative of subsurface conditions elsewhere across the site. Additional description of the soil/rock encountered is given in the accompanying geotechnical report.

GENERAL INFORMATION

Project details, borehole number, location coordinates and type of drilling equipment used are given at the top of the borehole log.

SOIL LITHOLOGY

Elevation and depth

This column gives the elevation and depth of inferred geologic layers. The elevation is referred to the datum shown in the Description column.

Lithology Plot

This column presents a graphic depiction of the soil and rock stratigraphy encountered within the borehole.

Description

This column gives a description of the soil stratums, based on visual and tactile examination of the samples augmented with field and laboratory test results. Each stratum is described according to the following classification and terminology (Ref. Unified Soil Classification System):

The compactness condition of cohesionless soils (SPT) and the consistency of cohesive soils (undrained shear strength) are defined as follows (Ref. Canadian Foundation Engineering Manual):

Compactness of Cohesionless Soils	SPT N-Value
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

Consistency of Cohesive Soils	SPT N-Value	Undrained Shear Strength kPa	psf
Very soft	0 to 2	0 to 12	0 to 250
Soft	2 to 4	12 to 25	250 to 500
Firm	4 to 8	25 to 50	500 to 1000
Stiff	8 to 15	50 to 100	1000 to 2000
Very stiff	15 to 30	100 to 200	2000 to 4000
Hard	> 30	Over 200	Over 4000

Soil Sampling

Sample types are abbreviated as follows:

SS	Split Spoon	TW	Thin Wall Open (Pushed)	RC	Rock Core
AS	Auger Sample	TP	Thin Wall Piston (Pushed)	WS	Washed Sample

Additional information provided in this section includes sample numbering, sample recovery and numerical testing results.

Field and Laboratory Testing

Results of field testing (e.g., SPT, pocket penetrometer, and vane testing) and laboratory testing (e.g., natural moisture content, and limits) executed on the recovered samples are plotted in this section.

Instrumentation Installation

Instrumentation installations (monitoring wells, piezometers, inclinometers, etc.) are plotted in this section. Water levels, if measured during fieldwork, are also plotted. These water levels may or may not be representative of the static groundwater level depending on the nature of soil stratum where the piezometer tips are located, the time elapsed from installation to reading and other applicable factors.

Comments

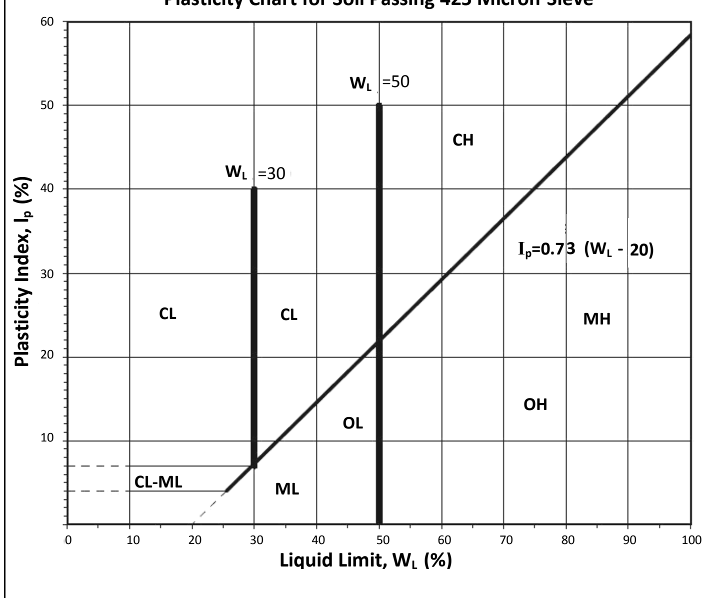
This column is used to describe non-standard situations or notes of interest.

SHAD & ASSOCIATES INC.

MODIFIED * UNIFIED CLASSIFICATION SYSTEM FOR SOILS *The soil of each stratum is described using the Unified Soil Classification System (Technical Memorandum 36-357 prepared by Waterways Experiment Station, Vicksburg, Mississippi, Corps of Engineers, U.S Army. Vol. 1 March 1953.) modified slightly so that an inorganic clay of "medium plasticity" is recognized.					
MAJOR DIVISION			GROUP SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA
COARSE GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 75μm)	GRAVELS MORE THAN HALF THE COARSE FRACTION LARGER THAN 4.75mm	CLEAN GRAVELS (TRACE OR NO FINES)	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 4;\ C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS
		DIRTYGRAVELS (WITH SOME OR MORE FINES)	GM	SILTY GRAVELS, GRAVEL-SAND- SILT MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR P.I. MORE THAN 4
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR P.I. MORE THAN 7
	SANDS MORE THAN HALF THE COARSE FRACTION SMALLER THAN 4.75mm	CLEAN SANDS (TRACE OR NO FINES)	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 6;\ C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$
			SP	POORLY GRADED GRAVELS, GRAVEL- SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS
		DIRTY SANDS (WITH SOME OR MORE FINES)	SM	SILTY SANDS, SAND-SILT MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR P.I MORE THAN 4
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR P.I MORE THAN 7
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT SMALLER THAN 75μm)	SILTS BELOW "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 50\%$	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (SEE BELOW)
		$W_L < 50\%$	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS	
	CLAY ABOVE "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 30\%$	CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY OR SILTY CLAYS, LEAN CLAYS	
		$30\% < W_L < 50\%$	CI	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS	
		$W_L < 50\%$	CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	
	ORGANIC SILTS & CLAYS BELOW "A" LINE	$W_L < 50\%$	OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	WHENEVER THE NATURE OF THE FINES CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER "F", E.G SF IS A MIXTURE OF SAND WITH SILT OR CLAY
		$W_L < 50\%$	OH	ORGANIC CLAYS OF HIGH PLASTICITY	
HIGH ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOUR OR ODOUR, AND OFTEN FIBROUS TEXTURE

SOIL COMPONENTS					
FRACTION	U.S STANDARD SIEVE SIZE			DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS	
		PASSING	RETAINED	PERCENT	DESCRIPTOR
GRAVEL	COARSE	76 mm	19 mm	35-50	AND
				20-35	Y/EY
	FINE	19 mm	4.75 mm	10-20	SOME
SAND	COARSE	4.75 mm	2.00 mm	1-10	TRACE
	MEDIUM	2.00 mm	425 μm		
	FINE	425 μm	75 μm		
FINES (SILT OR CLAY BASED ON PLASTICITY)		75 μm			
OVERSIZED MATERIAL					
ROUNDED OR SUBROUNDED: COBBLES 76 mm TO 200 mm BOULDERS > 200 mm				NOT ROUNDED: ROCK FRAGMENTS > 76 mm ROCKS> 0.76 CUBIC METRE IN VOLUME	

Plasticity Chart for Soil Passing 425 Micron Sieve



Note 1: Soils are classified and described according to their engineering properties and behavior.
Note 2: The modifying adjectives used to define the actual or estimated percentage range by weight of minor components are consistent with the Canadian Foundation Engineering Manual (3rd Edition, Canadian Geotechnical Society, 1992)