McINTOSH • LALANI
ENGINEERING LTD
A Division of Englobe

M•L 8959

Geotechnical Report

Golden Bridge to Bridge Dike Improvements

Golden, British Columbia

Prepared For

Town of Golden
c/o Urban Systems Ltd.

Submitted On

January 30, 2019

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1.0 INTRODUCTION

This report presents the results of a geotechnical evaluation conducted by McIntosh•Lalani Engineering Ltd. (M•L) for a proposed retaining wall in the Town of Golden, British Columbia. This evaluation was undertaken at the request of Ms. Sara Anderson of Urban Systems Ltd. (USL). The objective of this evaluation was to assess the general subsurface soil conditions at the site for the design and construction of the proposed retaining wall.

This report presents the results of the drilling program and provides geotechnical recommendations for construction.

1.1 PROJECT AND SITE DETAILS

The proposed dike upgrades are for a concrete retaining structure to be constructed along the north bank of the Kicking Horse River in Golden, B.C. It is our understanding that the project limits are the Kicking Horse Pedestrian Bridge to the west and the bridge carrying provincial highway 95 to the east. The alignment of the dike follows the alley behind the 9 Avenue North businesses and the pedestrian pathway along 8th Avenue North.

M•L previously completed a geotechnical investigation for the Golden Bridge to Bridge project in June 2009 for the Town of Golden care of USL (M•L f/n: 4373), which included two boreholes advanced by air rotary at locations along the dike. No report was prepared at the time. The primary purpose of the current investigation was to collect soil samples for corrosivity testing for use in design of the proposed retaining wall. Secondarily, the investigation would supplement the soil information from the previous investigation.

2.0 METHODOLOGY

In order to collect the necessary samples, M•L conducted a reconnaissance of the site to determine what challenges existed for site access and determined that the most feasible method to retrieve samples was by use of a mini-excavator. This was due both to the logistical issues of getting an appropriate drilling rig to the site and also to the anticipated difficulties with maneuvering that rig in light of the anticipated utilities and the tight working area. If possible, the depth of fill soils was also to be determined by test-pitting. Testing locations are shown on the attached plan 8959.00.G001. Borehole logs are presented in Appendix A.

2.1 SOILS INVESTIGATION

Upon completion of utility locates, two locations were identified as potential areas where test-pits could be completed. However, upon discussion with the town, one of the locations was found to be too close to an unmarked fibre optic line. M•L proceeded with the other test-pit, which was advanced to a depth of approximately 1.8 metres below the existing grade. This test-pit was advanced using a mini tracked excavator contracted from Golden Installations Ltd. of Golden, B.C. on October 31, 2018. An additional hand shoveled test-pit was advanced near the east project limit to obtain an additional near surface sample for corrosivity testing.

Upon completion of the sampling the test locations were backfilled and bucket tamped.

The subsurface investigation in 2009 consisted of advancing two boreholes within the area to depths of approximately 9.1 metres below the existing grade. The boreholes were advanced on June 23, 2009 using a truck mounted air rotary solid stem auger drill rig contracted from Beck Drilling & Environmental Services of Calgary, Alberta. Classification of the soil was done from the disturbed samples obtained from the air rotary operation and from the Standard Penetration Test (SPT) operation. SPT blow counts were utilized to aid in determining in-situ soil strengths.

2.2 LABORATORY TESTING

Laboratory testing including natural moisture content, soluble sulphate, and Atterberg limit testing was performed on select samples from the boreholes. Samples obtained from the test-pits were submitted to a certified laboratory for a suite of corrosivity testing. Results of this testing are provided as an attachment and discussed within.

3.0 SUBSURFACE CONDITIONS

At the time this report was prepared, information on subsurface stratigraphy was available only at discrete borehole locations. Conditions were extrapolated and interpolated from the borehole locations to develop recommendations. Adequate monitoring should be provided during construction to check that these assumptions are reasonable. The below summarizes the subgrade conditions encountered in the drilling program. More detailed soil description is contained in the borehole logs in Appendix A.

3.1 SURFICIAL GEOLOGY

Soils along the river banks are expected to consist of fluvial deposits, including fine-grained overbank and coarse-grained channel sediments.

3.2 SOILS

The soils encountered at the surface in the boreholes and test pits consist of sandy gravel fill with some cobbles and trace boulders and silt. The fill extended at least to the ends of the test-pit and hand-shovel test hole, and to depths of approximately 2.1 and 4.6 metres in boreholes 1 and 2 respectively. The fill was compact and dry becoming damp to moist with depth.

Following the fill in borehole 1, native silty sand was encountered to a depth of approximately 3.0 metres. The sand was compact and wet with traces of fine gravel.

Following the sand in borehole 1 and fill in borehole 2, native sandy gravel with some boulders and cobbles was encountered and extended at least to the ends of the boreholes. The sandy gravel was compact and wet. Free water was noted within the gravel.

3.3 GROUNDWATER

Groundwater levels were not measured in the standpipes, however it is expected that the groundwater levels will be near the level of the river.

4.0 DISCUSSION AND RECOMMENDATIONS

4.1 GENERAL

The subsurface conditions at the site are considered suitable for the proposed retaining wall. The geotechnical considerations at the site are summarized below:

- A conventional shallow strip and spread footing foundation system may be placed on the native sandy gravel. Excavations for foundations should be visually inspected and any loose or deleterious soils should be removed prior to construction.
- Alternatively, bored cast-in-place or driven steel piles may be designed to resist the anticipated lateral and axial loads. The feasibility of these methods must be verified by prospective contractors prior to construction.
- The soils below slabs-on-grade should be visually inspected prior to construction and any topsoil, vegetation, loose or deleterious materials should be removed.
- Temporary excavations will need to be constructed with minimum backsloping at a gradient of 1 Horizontal to 1 Vertical (1H:1V). Additional backsloping may be required if excessive sloughing or groundwater infiltration is experienced.
- Excavations in the fill and native soils may be difficult due to the presence of cobbles and boulders, and due to groundwater conditions.
- Groundwater was encountered at depths as shallow as 2.1 metres. Seepage into construction excavations may need to be dewatered using a system of ditches, sumps and pumps.
- The site soils are suitable for use as general engineered fill. The depths and condition of existing fill soils are to be verified during construction.

This list should not be considered all-inclusive and should be read in conjunction with the remainder of this report. Geotechnical foundation design parameters, slab-on-grade recommendations, groundwater concerns, pavement design sections, and additional construction recommendations are provided in the sections below.

4.2 FOUNDATION DESIGN RESISTANCE FACTORS

Load and Resistance Factor Design (LRFD) parameters are presented below for shallow and deep foundation design. Ultimate Limit State (ULS) resistances are presented, and should be utilized with the following design formula, as per the Canadian Foundation Manual Fourth Edition 2006:

$$\Phi R_n \geq \Sigma\alpha_i S_{ni}$$

Where:

Φ = Geotechnical resistance factor

R_n = Nominal (ultimate) geotechnical resistance

α_i = Load safety factor determined by structural engineer

S_{ni} = Specified load component

i = Represents various types of loads

The values for load factors (α_i), geotechnical resistance factor (Φ) and load combinations are specified by applicable codes e.g. NBCC. As per the NBCC, we recommend use of the following Φ:

Table 1: Geotechnical Resistance Factors for Foudations

Description		Resistance Factor (Φ)
Shallow Foundations – vertical bearing resistance, semi-empirical analysis		0.5
Shallow Foundation – sliding		0.8
Deep Foundations		
Resistance to compressive axial load	from semi-empirical analysis	0.4
	from static loading test results	0.6
	from dynamic monitoring results (i-e, pile driving analyzer (PDA) testing	0.5
Uplift resistance	from semi-empirical analysis	0.3
	from loading test results	0.4
Horizontal Load resistance		0.5

4.3 STRIP AND SPREAD FOOTINGS

A strip and spread foundation system is feasible for the proposed wall provided the recommendations in this report are followed. The foundation should be placed on the undisturbed native sandy gravel. The anticipated footing depth is 3.5 metres below existing grade. Fill soils and other deleterious materials encountered at the proposed bearing surface will need to be over-excavated. The footings would then be placed at the over-excavated depth. Alternatively, the over-excavation could be backfilled to the design footing elevation using local approved sandy gravel fill or approved imported fill meeting the requirements for structural fill.

All bearing surfaces must be approved by a qualified geotechnical engineer prior to gravel or concrete placement. The bearing surface must be protected against disturbances and ingress of free water and should be protected from meteorological elements including rain, snow, and freezing temperatures.

4.3.1 Design Using Ultimate Limit State

Based on the above assumptions, the following Ultimate Limit States design parameters may be used for design. Should a strip and spread footing foundation system be utilized, a review of the footing sizes and bearing resistances should be undertaken.

Table 2: ULS Bearing Resistance for LRFD

Soil	ULS Unfactored Bearing Resistance (kPa)
Native Sandy Gravel	900

A geotechnical resistance factor of 0.5 may be utilized in conjunction with the above noted ULS value for vertical bearing resistance.

4.3.2 Design Using Serviceable Limit State Method

To undertake the shallow foundation design using the Serviceable Limit State Method, the following Serviceability Limit State (SLS) bearing pressure may be used.

Table 3: Serviceability Limit State Bearing Pressure

Soil	Bearing Pressure (kPa)
Native Sandy Gravel	350

The footing sizes and depths have been estimated to provide the above design values. Should unconventional footing sizes be utilized, a review of the footing sizes and bearing resistances should be undertaken. Generally, serviceable limit state conditions govern for larger sized footings as settlement/consolidation are the limiting condition.

Footings should be placed on homogenous soils to avoid differential settlements that could occur if footings span non-uniform soil types (e.g. fill to native).

4.1 PILES

Deep pile foundations are expected to be a viable alternative to shallow foundations. The potential advantages of a deep pile foundation system include reduction in the required excavation depth, better frost resistance and less in-river work. The piles may consist of either bored cast-in-place concrete piles or driven steel piles. Pile design parameters for axial load resistance are provided below and design considerations for each pile type follow. Some mobilization of the pile is required to activate the full resistance Pile installation monitoring during the installation process of the piles should be undertaken to verify that the encountered soils are in accordance with the soils assumed in the design. The inspections need to be completed by a qualified geotechnical consulting firm.

Table 4: Axial Resistance Parameters

Soil	Approx. Depth Below Existing Grade (m)	Working Stress Method		Unfactored ULS	
		Skin Friction (kPa)	**End Bearing (kPa)**	**Skin Friction (kPa)**	**End Bearing (kPa)**
Surficial Soils, Fill Soils	0 - 3	---	---	---	---
Gravel and Clean Fill	3 - 5	35	---	115	---
Gravel	5 – 9	35	450	115	1500

Note: Geotechnical resistance factors for uplift and compression presented in Section 4.2 of this report should be applied to the ULS design values presented above.

Note: The Serviceability Limit States (SLS) condition should be checked upon design of the foundation loads and pile sizes and depths.

If the piles are designed for depths greater than 9.0 metres, the continuity of strata shall be verified to a depth of no less than three pile diameters below the general basing elevations of the piles. The continuity of the strata may be verified by the piling contractor by over-drilling one or more piles during the course of pile installation, as appropriate, depending upon the design installation depths used.

4.1.1 Lateral Load Soil Parameters

Detailed design of laterally loaded piles should be done using a non-linear Lateral Pile Response Model that also models eccentricity of axial loading due to lateral deflection (P-Δ effects). M•L can provide these services if requested. For preliminary lateral pile design, the coefficient of horizontal subgrade reaction which is a function of pile diameter has been calculated using the Davisson, 1970 method referenced in the Canadian Foundation Manual, 3rd Edition. The recommended values are presented in the table below where D is the pile diameter.

Table 5: Horizontal Moduli of Subgrade Reaction

Soil Type	Horizontal Modulus of Subgrade Reaction (MPa/m)	
	Sustained Loading	**Cyclic Loading**
Fill Soils	4/D	2/D
Gravel	16/D	8/D

4.1.2 Design and Construction of Bored Piles

Bored piles may be designed to resist axial compressive loads on the basis of end bearing if cleaning and visual inspection of the pile base are possible. Skin friction may be used unaccompanied or in combination with end bearing to resist axial compressive loads. Skin friction is also used to resist axial tensile (uplift) loads.

The site soils consist of fluvial gravels, cobbles and boulders. These conditions may present problems for conventional bored pile augering. The contractor should conduct test-piling prior to beginning production piles, and should develop contingency plans for auger refusal or removal of large boulders. Constructing belled piles at the site is not possible due to the soil type. Groundwater that is hydraulically connected to the river level is expected within bored pile holes and sloughing will occur in the non-cohesive soils. The following preparations are recommended and should be considered for inclusion in the tender documents:

- Contractor should have casing onsite to stabilize the hole as sloughing and seepage will occur.
- Contractor should have tremie pipes onsite or access to a concrete boom pump truck to allow placement of concrete through water when seepage occurs.
- Contractor should have appropriate sizes of cleaning buckets onsite to prepare the base properly. If end-bearing is considered in the pile design, the piling contractor must be prepared to properly clean the base.
- It may be necessary to provide a down-the-hole camera for pile base inspection prior to pouring concrete.

Bored piles should have an overall length below finished grade of not less than 5 metres and a shaft diameter of not less than 400 mm. Pile spacing should be considered in the design capacity and for staging of the work. An increase capacity in the piles of 33% may be used for dynamic loading.

4.1.3 Design and Construction of Driven Steel Piles

Given the soil conditions at the site, it is recommended that driven piles be designed to resist axial loads on the basis of skin friction and be driven to sufficient depth to provide appropriate overturning resistance at minimum.

Steel piles should conform to the requirements of the applicable Building Code.

Corrosion of the steel piles is seldom a problem for piles driven into native soils, however, within the fills initially present on the site and above the groundwater table moderate corrosion may occur. As such, precautions such as application of coatings, encasement by way of cast-in-place concrete jackets, cathodic protection or increase in thickness of steel section should be considered to provide an allowance for the mitigation of the effects of corrosion. This coating should be applied at minimum to any part of the pile within the range of fluctuation of the river level.

Splices in steel pile sections should consist of full strength butt welds in order to accommodate the high driving stresses developed with the steel piles during installation. The welded pile should be allowed to cool below a temperature of 300°C before pile driving is resumed.

Piles should be installed to similar tip elevations, particularly when in close proximity. Construction staging must be considered due to the anticipated proximity of the piles. Reductions in pile capacity are not required for skin friction parameters.

Frost heave forces on the steel piles should be considered in the design. A force of 150 kPa in the freezing zone of pile shaft should be used in the design. The upper freezing zone should be considered to be the upper 2.4 metres. Fine-grained soil cover or insulation may be used to reduce the potential for frost heave. Alternatively, a free-moving sleeve could be installed around the pile in the freezing zone if necessary to eliminate the adfreeze induced uplift.

Pile installation monitoring during the installation process of the piles should be undertaken to verify that the installation requirements are achieved. The inspections need to be completed by a qualified geotechnical consulting firm.

The vertical allowable load capacity of a steel pile should be limited to 0.33 Fy, where the force is applied evenly over the cross sectional area of the pipe. Fy is the yield strength of the steel.

Maximum permissible stress during pile driving of the steel piles should be limited to 450 J/cm2 (15000ft-lbs equals approximately 20KJ).

Piles should not be driven beyond practical refusal of the pile which is defined 6 blows per inch over 6 consecutive inches using the maximum driving energy for the respective pipe size. Should the pile reach effective refusal prior to achieving the design depth the design and geotechnical engineers should be notified.

General recommendations for the design and construction of driven steel piles are included in Appendix B.

4.1.4 Group Effects

Upon completion of the pile design, M•L should review the pile layout to ensure there are no pile group effects which will impact the design capacity of the piles.

4.2 SLIDING PARAMETERS

The unfactored ultimate limit state (ULS) coefficient of friction may be taken as 0.40. It is recommended that a geotechnical resistance factor of 0.8 be applied to the unfactored ULS Coefficient of friction as specified by the NBCC 2005.

4.3 SEISMIC DESIGN CONSIDERATIONS

The site classification for seismic response as defined in table 4.1.8.4.A of the National Building Code of Canada (NBCC) 2005 is C.

4.4 SETTLEMENTS

4.4.1 Shallow Foundations

Settlements of strip and spread footings designed to the allowable bearing pressure provided for Working Stress Design (WSD) are expected to be in the range of 50 mm. Differential settlement across foundation elements should be less than 1 mm per 300 mm of separation between measurement points.

These settlements values are typical for the overall Factor of Safety (FOS) applied to obtain the allowable (WSD) parameters, however it should be noted that footing sizes and footing proximities do play a role in the settlements that would occur. It is therefore recommended that the foundation design is reviewed by M•L to ensure that the expected total and differential settlements are acceptable.

Adjacent footings should be placed at a similar elevation. Adjacent footings at different elevations should be situated such that a 2H:1V projection from underside of the higher footing does not intercept the bearing surface of the lower footing.

4.4.2 Pile Foundations

The settlements in the piles will depend on the pile loads, sizes and group effects. Providing the pile tips are installed to near same elevation (within a few meters) and all piles are installed as per installation requirements and the piles are designed as per the recommendations in this report, the piles should undergo relatively uniform movement. The movement required to mobilize the skin friction along the pile shaft will be approximately 1% of the pile diameter. Additional elastic shortening of the pile should be expected. Using the design values in this report the total consolidation at the pile tip should be limited to 25mm.

In addition, both differential settlements and total settlements would also depend on the settlement needed to mobilize the required geotechnical resistance which is dependent on the effective cleaning of the base of the drilled shaft.

4.5 FROST PROTECTION

The soils on site include silt within the soil matrix and are expected to be moderately frost susceptible. The following recommendations can mitigate some of the effects of frost action when correctly implemented.

4.5.1 Structures

For protection against frost action, footings for retaining walls should have a minimum soil cover of 2.1 metres, unless provided with equivalent insulation. If insulation and soil cover are both not feasible, the damaging effects of frost heave can be mitigated by incorporating adequate saw cutting into the design and allowing for some differential movement by increasing the steel reinforcement. For grade beams, a minimum 100 mm void form should be included to prevent interaction with heaving soils.

4.5.2 Surface Concrete

Some precautions should be followed for the design and construction of concrete flatworks at the site.

In all unheated areas, the site soils will likely experience some degree of heave due to frost formation during the winter months. Generally speaking, if proper consideration is given to the recommendations contained in Section 4.6, proper drainage will prevent the subgrade from becoming saturated and will help reduce the severity of frost heave. Nevertheless, concrete flatwork should be designed with anticipation of some frost heave occurring. Concrete sidewalks should be dowelled into footings or grade beams in threshold areas where heave of the concrete panels would obstruct the proper opening of the door and present a tripping hazard. As the outside edge of these panels will still heave, the panel should either be properly jointed to control crack locations, or reinforced by the placement of reinforcing steel 10 mm bars at a 300 mm spacing. The depth of the reinforcement should be controlled so that the reinforcement is properly located within the concrete panels.

Alternatively, rigid insulation can be placed below flatwork to prevent frost formation in the underlying subgrade. M•L can provide recommendations for such insulation if required.

4.6 SITE GRADING AND DRAINAGE

It is recommended that final site grading be provided to direct water to areas remote from the proposed structures. Minimum landscape gradients of 1.5 percent are recommended to reduce the risk of run-off ponding in localized areas. Driveways, parking areas or landscaping within a zone of approximately 2 m of the exterior perimeter of any structure should be graded to drain away from the structures at a minimum gradient of 2 percent. Drainage behind the wall must also be considered and is discussed in section 4.9.

All fill soils placed on site should consist of general engineered fill as per Section 4.16 of this report.

4.7 CONSTRUCTION EXCAVATIONS

The composition and consistencies of the soils encountered at the site are such that conventional hydraulic excavators should generally be able to remove the soils. Boulders and cobbles may present challenges during excavation and these large aggregates should be expected in any excavations on site.

All excavations should be carried out in accordance with British Columbia Occupational Health and Safety (OH&S) Regulations. Excavations in the cohesionless sands, gravels and silts should have backsloping at a minimum gradient of 1H:1V (45°) from the base of excavation. If excessive sloughing is encountered, additional side sloping may be necessary. Should space constraints not allow adequate side sloping for the excavation to ensure a safe temporary excavation, shoring will be necessary. A qualified geotechnical engineer should review the excavation stability to ensure excavation safety prior to workers entering the excavation.

4.8 GROUNDWATER CONSIDERATIONS

Groundwater seepage is expected in all excavations, particularly where they extend below the water table. It is expected that dewatering will be required. It should be possible to dewater flows above the river level using a swale and sumps equipped with submersible pumps.

4.9 WEEPING TILE

Weeping tile should consist of a minimum 100 mm diameter perforated PVC pipe, backfilled with 40 mm drainage gravel. It should be placed above the normal water level behind the wall. The drainage gravel should be wrapped in a 6 oz non-woven geotextile filter fabric to prevent movement of fines into the drainage gravel. The weeping tile should be connected to the storm system if possible. If the weeping tile outlet is through the face of the wall, backflow preventers may be required on the outlets. M•L should review the final drawings to ensure that the weeping tile design is appropriate for the site conditions.

4.10 CORROSIVITY

M•L submitted three samples to WSH Labs (1992) Ltd. for corrosivity suite testing which includes soluble sulphate, chloride, pH and Resistivity. The results of testing are given in the table below.

Table 6: Corrosivity Factors

Sample	Sulphate (%)	Chloride (%)	pH	Resistivity (ohm-metres)
26654-1	0.0025	0.0046	8.46	26.25
26654-2	0.0022	0.0038	8.52	28.33
26654-3	0.0021	0.0029	8.33	26.39

The result indicates a negligible soluble sulphate concentration of 0.0025 percent. Therefore, the use of Type GU (Normal Portland) cement concrete in accordance with CSA A23.1, Table 2 for F-2 exposure is suitable for all concrete in contact with the soil which these samples represent. The F-2 exposure class requires minimum 25 MPa strength at 28 days, a maximum water to cementing materials ratio of 0.55 and 4-7 percent entrained air by volume based on 14-20 mm aggregate.

It is recommended that all imported soils to be utilized on site be tested for soluble sulphate concentrations.

4.11 LATERAL WALL PRESSURES

Permanent and temporary walls should be designed to resist all lateral pressures including those due to soil or backfill, surcharges, water and adjacent footings using the following expressions defined in terms of total and effective stresses:

$$P_{\text{lateral pressure}} = P'_{\text{earth+surcharge}} + P_{\text{net water}} + P'_{\text{adj ft}}$$

where $P_{\text{lateral pressure}}$ = total lateral pressure at a given depth (kN/m^2)

$P'_{\text{earth+surcharge}}$ = lateral earth pressure due to soil or fill and surcharges at a given depth (kN/m^2)

= $K(\gamma h + q)$ above water table or phreatic surface

= $K(\gamma' h + q)$ below water table or phreatic surface

$P_{\text{net water}}$ = net water pressure on wall at a given depth (kN/m^2), calculated by hand drawn flow net or computer solution based on drainage conditions

$P'_{\text{adj ft}}$ = lateral earth pressure due to adjacent footings at given depth (kN/m^2)

K = coefficient of lateral earth pressure, K_a, K_o, K_p or combination of as noted below

K_a = coefficient of active earth pressure

K_o = coefficient of at-rest earth pressure

K_p = coefficient of passive earth pressure

γ' = submerged unit weight of backfill or natural soil (kN/m^3)

γ' = $\gamma - \gamma_w$

γ = bulk unit weight of backfill or natural soil (kN/m^3)

γ_w = unit weight of water 9.81 kN/m^3

h = excavation depth (m)

q = surcharge load (kN/m^2)

Table 7 below presents coefficients of lateral earth pressure and unit weights.

Table 7: Coefficients of Lateral Earth Pressure

	K_a	K_o	K_p	γ (kN/m^3)	Φ (°)
Engineered Fill	0.38	0.58	2.66	22.0	27
Structural Fill	0.31	0.47	3.30	23.0	32
Native Gravel	0.29	0.46	3.40	23.0	33

4.12 PERMANENT LATERAL WALL PRESSURES

The distribution of soil pressure against a permanent wall may be assumed using the general equation given above with a coefficient of lateral earth pressure equal to the at rest coefficient of earth pressure, $k = k_o$. Values of k_o are given above for fill and native silt and clay as permanent walls can be constructed with backfill or poured neat to temporary shoring and native soils.

Permanent walls should be designed to resist the maximum possible water pressure subject to drainage conditions determined by design.

4.13 TEMPORARY LATERAL WALL PRESSURES

The distribution of soil pressure against a temporary wall may be assumed using the general equation given above and values of K according to deformation restrictions as follows:

- If moderate wall movements can be permitted
  $K=K_a$.
- If foundations of buildings or services exist at a shallow depth, at a distance less than H (height of the wall) behind the top of the wall and not closer than 0.5H
  $K= 0.5 (K_a + K_o)$.
- If foundations or services exist at a shallow depth, at a distance less than 0.5H
  $K=K_o$.

4.14 TEMPORARY PASSIVE WALL RESISTANCE

Passive resistance at the base of a temporary wall may be calculated as follows:

$$P'_p = K_p (\gamma' d/1.5)$$

Where P'_p = passive resistance at depth below excavation (kN/m^2)

K_p = coefficient of passive earth pressure

γ' = submerged unit weight (kN/m^3)

d = depth below excavation level (m)

The passive resistance should be taken to act on an area twice the pile diameter below grade.

4.15 SEISMIC LATERAL EARTH PRESSURES

Seismic lateral earth pressure can be estimated using simplified methods. The 2006 Canadian Foundation Engineering Manual suggests the use of the Mononobe-Okabe (M-O) method. Using this method, the seismic lateral earth pressure is defined by the equations

$$P_{AE} = \frac{1}{2} K_{AE}\, \gamma H^2 (1-k_V)$$

and

$$K_{AE} = \frac{\cos^2(\varphi-\theta-\psi)}{\cos\psi \cos^2\theta \cos(\delta+\theta+\psi)\left[1+\frac{\sin(\delta+\varphi)\sin(\varphi-\beta-\psi)}{\cos(\delta+\theta+\psi)\cos(\beta-\theta)}\right]^2}$$

Where P_{AE} = total active thrust (kN/m^2)

K_{AE} = dynamic active earth pressure coefficient

γ = bulk unit weight of backfill or natural soil (kN/m^3)

H = height of backfill (m)

φ = soil angle of internal friction

θ = slope of backfill with horizontal

β = slope of the back face of the retaining wall with vertical

δ = angle of friction of wall-backfill interface

And ψ is defined as

$$\psi = \tan^{-1}\left(\frac{k_h}{1-k_v}\right)$$

Where $k_{h/v} = \frac{a_{h/v}}{g}$

The appropriate value of the peak horizontal acceleration, a_h, is equal to the peak ground acceleration (PGA) for local seismic conditions. For Golden, the PGA is 0.13g. This information was obtained from the 2015 National Building Code of Canada seismic hazard calculator. The peak vertical acceleration, a_v, can be estimated as 2/3 a_h.

4.16 BACKFILL MATERIALS AND COMPACTION

The on-site materials may be suitable for use as general engineered or structural fill subject to material evaluation and removal of deleterious materials. Imported fill should be approved for use as structural or general engineered fill.

Recommended compaction specifications and materials are as follows:

- Structural fill - 100 percent Standard Proctor maximum dry density, maximum compacted lift thickness 250 mm, maximum grain size 200 mm. Structural fill materials should comprise clean well-graded inorganic granular soils.
- General engineered fill - 98 percent Standard Proctor maximum dry density, 0 to +3 percent of optimum moisture content, maximum compacted lift thickness 300 mm. General engineered fill materials should comprise clean well-graded granular soils, or inorganic medium to low plastic cohesive soils.

Where washing of fines is possible, fill material placed should be separated from coarser or finer material by a suitable geotextile.

Backfill comprising cohesive soils should be considered frost susceptible and should not be used in areas where it may become frozen and where frost heaving would be unacceptable.

5.0 REVIEW OF DESIGN AND CONSTRUCTION

M•L should review details of the design and specifications related to geotechnical aspects prior to construction. Adequate monitoring during construction will be required. All construction should be carried out by a qualified contractor experienced in foundation and earthworks construction. Adequate monitoring includes:

- Shallow Foundations - Inspection by a qualified geotechnical engineer prior to placement of footings.
- Earthworks - Full-time monitoring and compaction testing.
- Deep Utility Installation – Full-time monitoring and compaction testing.

All monitoring should be carried out by a qualified person, independent of the contractor. M•L will provide these services if requested. Failure to provide an adequate level of foundation monitoring may be contravention of building code requirements.

5.1 DESIGN AND CONSTRUCTION GUIDELINES

Recommended general design and construction guidelines are provided in Appendix B under the following headings:

- Backfill Materials and Compaction
- Proof-Rolling
- Construction Excavations
- Shallow Foundations

These guidelines are intended to present standards of good practice. Although supplemental to the main text of this report, they should be interpreted as part of the report. Design recommendations presented herein are based on the premise that these guidelines will be followed. The design and construction guidelines are not intended to represent detailed specifications for the work, although they prove useful in the preparation of such specifications. In the event of any discrepancy between the main text of this report and Appendix B, the main text should govern.

6.0 LIMITATIONS

Recommendations presented herein are based on a geotechnical evaluation of the findings in 2 boreholes and 1 test-pit. The conditions encountered during the fieldwork are considered to be reasonably representative of the site. If, however, conditions other than those reported are noted during subsequent phases of the project, M•L should be notified and given the opportunity to review our current recommendations in light of new findings. This report does not include any recommendations related to contaminants in soil or groundwater. Should there be any other documentation indicating any excavation or land disturbance, such as environmental reports, M•L would require these reports prior to site development to confirm the recommendations within this report are suitable in light of new information.

This report has been prepared for the exclusive use of Town of Golden c/o Urban Systems Ltd. and their agents for specific application to the development described in this report. It has been prepared in accordance with generally accepted soil and foundation engineering practices. No warranty is expressed or implied.

7.0 CLOSURE

We trust information presented herein meets with your present requirements. If you have questions or require additional geotechnical services please contact our office.

Respectfully submitted,

McIntosh•Lalani Engineering Ltd.

NEIL KLASSEN

NKlassen

2020-05-22

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Senior Project Engineer

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May 13, 2020

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2020/05/13

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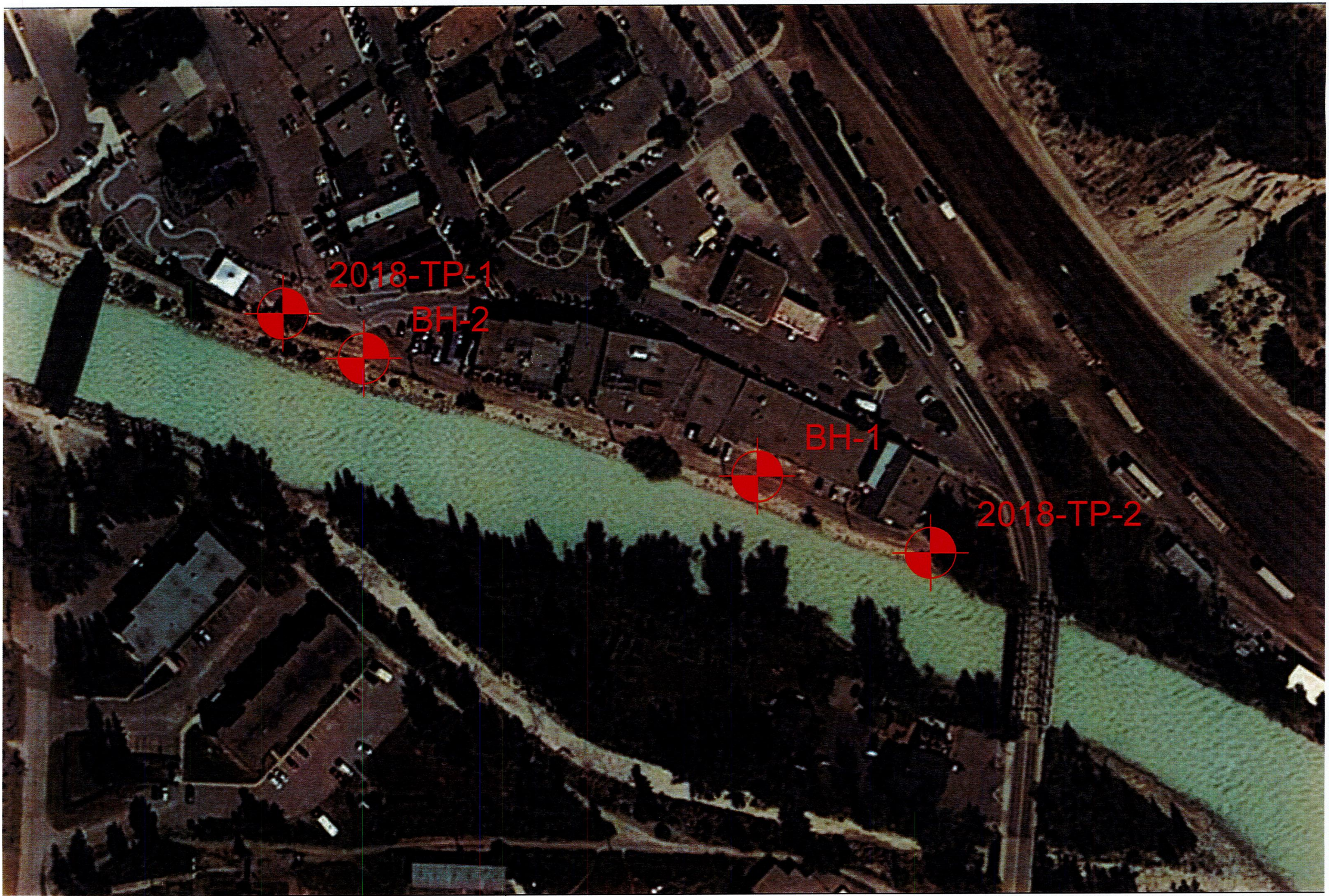
Drawings & Figures



Client: Town of Golden c/o Urban Systems Ltd.
Project: Bridge to Bridge Dike Improvements
Title: Borehole Location Plan

McINTOSH • LALANI ENGINEERING LTD.

Job Number: 8959
Drawing Number: 8959.00.G001
Scale: N.T.S.
Date: Nov. 15, 2018

Tables

Appendix A

Borehole Logs

Project: Golden Bridge to Bridge Dike Upgrades	Drilling Information:	Borehole No.:2018-TP-1
Client: Town of Golden	Golden Installations	Project No.:8959
c/o Urban Systems Ltd.	Backhoe	Elevation:

SAMPLE TYPE ■ SHELBY TUBE ▯ CORE SAMPLE ☒ SPT SAMPLE ✋ GRAB SAMPLE ▮ AUGER SAMPLE ▥ NO RECOVERY

Depth (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	USCS	BLOWS /150 mm	PLASTIC M.C. LIQUID 10 20 30 40	■BLOW COUNT■ 10 20 30 40 ●POCKETPEN (kPa)● 80 160 240 320	OTHER DATA	Elevation (m)
0		Gravelly Sand FILL - compact, damp, medium brown.								
		- Some organic browns, medium to dark brown.								
					SP					
1		- some boulders								
		END OF HOLE at a depth of 1.8 metres. Dry upon completion and backfilled.								
2										

McIntosh Lalani Engineering
Calgary, AB
(403) 291-2345

Logged By: SC	Completion Depth: 1.8 m
Reviewed By: Neil Klassen	Drilled on: 2018-10-31
Groundwater Depth: m	Page 1 of 1

ML STANDARD AUGER 8959. GOLDEN DIKE UPGRADES.GPJ M-L STANDARD.GDT 11-14-18

Project: Golden Bridge to Bridge Dike Upgrades	Drilling Information:	Borehole No.:2018-TP-2
Client: Town of Golden	Golden Installations	Project No.:8959
c/o Urban Systems Ltd.	Backhoe	Elevation:

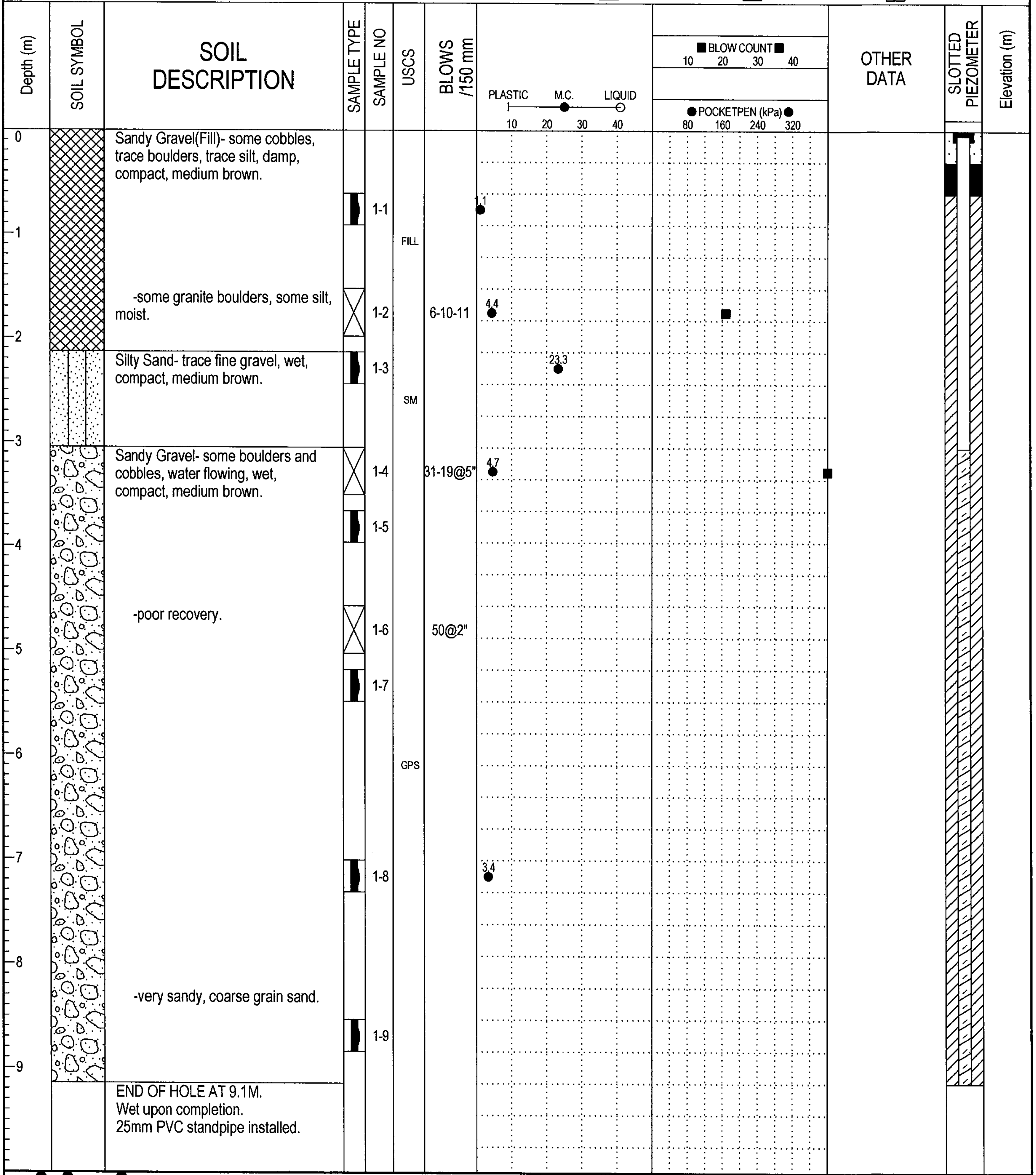
SAMPLE TYPE ■ SHELBY TUBE ▯ CORE SAMPLE ☒ SPT SAMPLE ✋ GRAB SAMPLE ▮ AUGER SAMPLE ▥ NO RECOVERY

Depth (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	USCS	BLOWS /150 mm	PLASTIC M.C. LIQUID 10 20 30 40	■BLOW COUNT■ 10 20 30 40 ●POCKETPEN (kPa)● 80 160 240 320	OTHER DATA	Elevation (m)
0	▨	Gravelly Sand FILL - compact, damp, medium brown.			SP					
		END OF HOLE at a depth of 0.3 metres. Dry upon completion and backfilled.								
1										
2										

McIntosh Lalani Engineering Calgary, AB (403) 291-2345	Logged By: SC	Completion Depth: 0.3 m
	Reviewed By: Neil Klassen	Drilled on: 2018-10-31
	Groundwater Depth: m	Page 1 of 1

Project: Golden Bridge to Bridge Project	Drilling Information:	Borehole No.:1
Client: Town of Golden c/o Urban Systems Ltd.	Beck Drilling & Environmental Services	Project No.:ML-4373
	Air Rotary SS-Auger	Elevation:

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



Depth (m)	SOIL DESCRIPTION	SAMPLE NO	USCS	BLOWS /150 mm	M.C.
0	Sandy Gravel(Fill)- some cobbles, trace boulders, trace silt, damp, compact, medium brown.				
		1-1	FILL		1.1
	-some granite boulders, some silt, moist.	1-2		6-10-11	4.4
	Silty Sand- trace fine gravel, wet, compact, medium brown.	1-3	SM		23.3
3	Sandy Gravel- some boulders and cobbles, water flowing, wet, compact, medium brown.	1-4		31-19@5"	4.7
		1-5			
	-poor recovery.	1-6		50@2"	
		1-7	GPS		
		1-8			3.4
	-very sandy, coarse grain sand.				
		1-9			
	END OF HOLE AT 9.1M. Wet upon completion. 25mm PVC standpipe installed.				

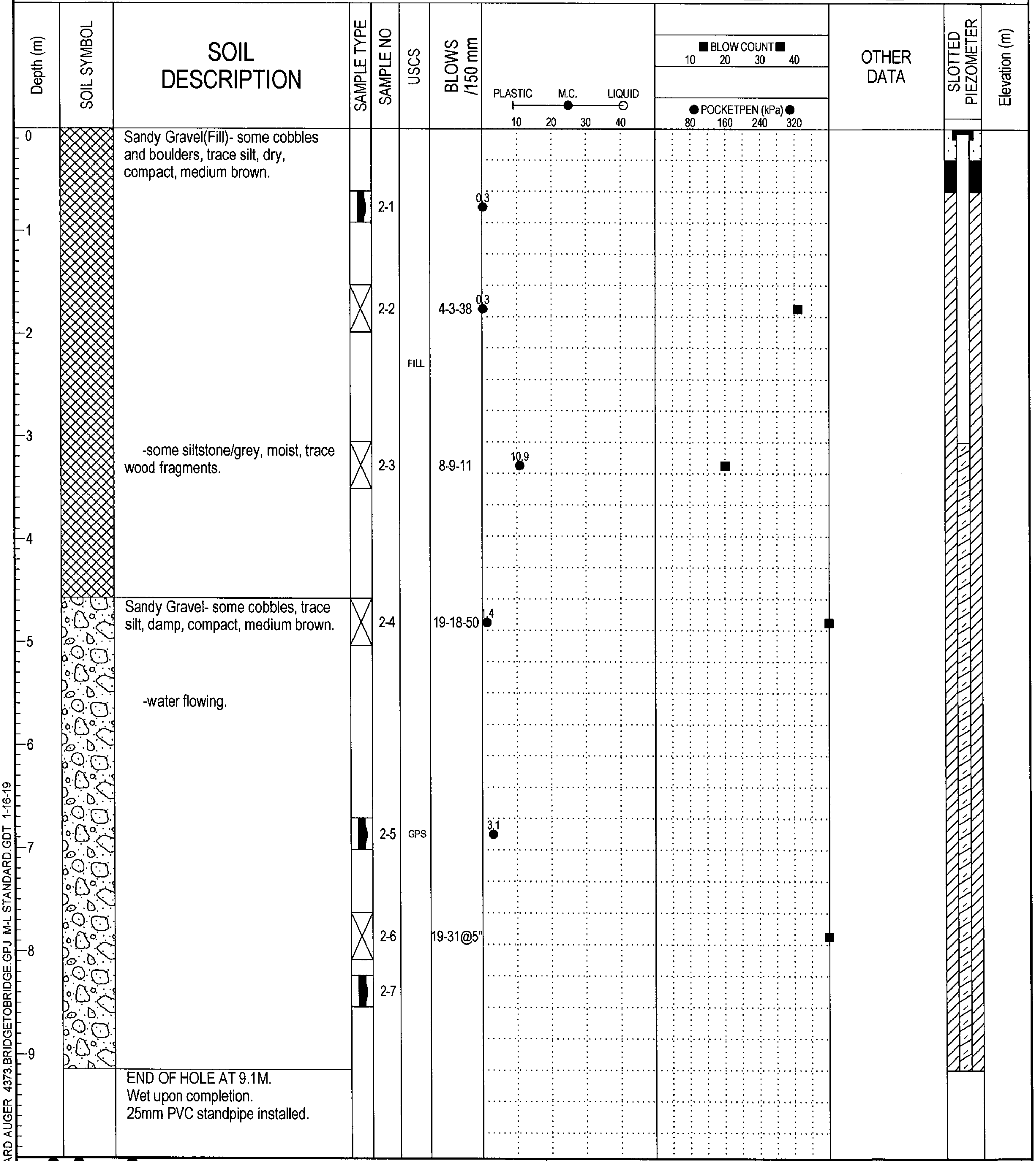
ML STANDARD AUGER 4373.BRIDGETOBRIDGE.GPJ M-L STANDARD.GDT 1-16-19

McIntosh Lalani Engineering
Calgary, AB
(403) 291-2345

Logged By: Ryan Stickel	Completion Depth: 30 ft
Reviewed By: Lee Martin	Drilled on: 2009-06-23
Groundwater Depth: m	Page 1 of 1

Project: Golden Bridge to Bridge Project	Drilling Information:	Borehole No.:2
Client: Town of Golden c/o Urban Systems Ltd.	Beck Drilling & Environmental Services	Project No.:ML-4373
	Air Rotary SS-Auger	Elevation:

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



Sandy Gravel(Fill)- some cobbles and boulders, trace silt, dry, compact, medium brown.

-some siltstone/grey, moist, trace wood fragments.

Sandy Gravel- some cobbles, trace silt, damp, compact, medium brown.

-water flowing.

END OF HOLE AT 9.1M.
Wet upon completion.
25mm PVC standpipe installed.

SAMPLE TYPE	SAMPLE NO	USCS	BLOWS /150 mm	M.C.
▮	2-1			0.3
☒	2-2		4-3-38	0.3
		FILL		
☒	2-3		8-9-11	10.9
☒	2-4		19-18-50	1.4
▮	2-5	GPS		3.1
☒	2-6		19-31@5"	
▮	2-7			

McIntosh Lalani Engineering
Calgary, AB
(403) 291-2345

Logged By: Ryan Stickel	Completion Depth: 30 ft
Reviewed By: Lee Martin	Drilled on: 2009-06-23
Groundwater Depth: m	Page 1 of 1

Appendix B

Design & Construction Guidelines

B.1 BACKFILL MATERIALS AND COMPACTION

Maximum density, as used in this section, means Standard Proctor Maximum Dry Density (ASTM Test D698) unless otherwise noted. Optimum moisture content is as defined in this text.

Backfill adjacent to exterior footings, foundation walls, grade beams and pile caps and within 300 mm of final grade should comprise low-plastic cohesive general engineered fill as defined above. Such backfill should provide a relatively impervious surface layer to reduce seepage in the sub-soil.

Backfill should not be placed against a foundation structure until the structure has sufficient strength to withstand the earth pressures resulting from placement and compaction. During compaction, careful observation of the foundation wall for deflection should be carried out continuously. Where deflection is apparent, the compactive effort should be reduced accordingly. In order to reduce potential compaction induced stresses, only hand held compaction equipment should be used in the compaction of fill within 500 mm of retaining walls or basement walls.

Backfill materials should not be placed in a frozen state or placed on a frozen subgrade. All lumps of materials should be broken down during placement.

Where the maximum-sized particles in any backfill material exceed 50 percent of the lift thickness or minimum dimension of the cross-section to be backfilled, such particles should be removed and placed at other more suitable locations on site or screened-off prior to delivery to site.

Bonding should be provided between backfill lifts, if the previous lift has become desiccated. For fine-grained materials, the previous lift should be scarified to 75 mm in depth followed by proper moisture conditioning and recompaction.

B.1.1 GENERAL ENGINEERED FILL

Backfill adjacent to and above footings, abutment walls, basement walls, grade beams and pile caps or below highway, street or parking lot pavement sections should comprise general engineered fill. "General engineered fill" materials should comprise clean, well-graded granular soils or inorganic, low-plastic cohesive soils. Such material should be placed in lifts not exceeding an uncompacted thickness of 300 mm, and compacted to not less than 98 percent of maximum density, at a moisture content at or slightly above optimum. The uncompacted lift thickness may be adjusted based on the method of fill placement and the size and type of compaction equipment in use.

B.1.2 STRUCTURAL FILL

Backfill supporting structural loads should comprise structural fill materials. "Structural fill" materials should comprise clean, well-graded inorganic granular soils. Such fill should be placed in compacted lifts not exceeding 150 mm and compacted to not less than 98 percent of maximum density, at a moisture content at or slightly (0 to 3 percent) above optimum. The following table provides gradation limits for structural fill of various nominal sizes. The gradation limits have been adapted from the City of Calgary Roads Construction 2015 Standard Specifications, Section 303.00.00 Materials. Other gradations may be approved on a project specific basis by a qualified geotechnical engineer.

Sieve Size (mm)	**Percent Passing By Weight Nominal Gravel Size**		
	80 mm	**50 mm**	**25 mm**
80	100		
75	---		
50	---	100	
40	60 - 90	95 – 100	
25	---	---	100
20	40 - 70	50 – 75	95 - 100
10	25 - 60	25 - 52	55 - 80
5	15 - 45	15 - 40	35 - 65
2.5	10 - 35	10 - 33	28 - 52
0.63	5 - 23	5 - 23	13 - 35
0.315	---	---	9 - 26
0.16	3 - 12	2 - 14	6 - 18
0.08	2 - 10	1 - 10	4 - 10
%Fractures (2 faces)	20	30	60

B.1.3 LEAN MIX CONCRETE

"Lean-mix concrete" should be low strength concrete having a minimum 28 days compressive strength of 3.5 MPa.

B.1.4 LANDSCAPE FILL

"Landscape fill" material may comprise soils without regard to engineering quality. Such soils should be placed in compacted lifts not exceeding 300 mm and compacted to a density of not less than 90 percent of maximum density.

B.1.5 PIPE BEDDING AND DRAINAGE

Bedding for pipes and utilities should generally conform to the manufacturer's specification. The type and depth of bedding material relative to the size of pipe are a function of the rigidity of the utility and the embedment depth. For drainage blankets and weeping tile, an open-graded, clean aggregate is required. The following table represents the gradation limits for bedding gravel. The gradation limits have been adapted from the City of Calgary Standard Specifications: Sewer Construction 2012 Section 402.10.00. Class IA material as defined in the table is also suitable for use in drainage applications. Local municipal specifications or manufacturer's specifications may be substituted at the discretion of a qualified engineer.

Sieve Size (mm)	For Pipe 375 mm and Smaller (20 mm Nominal Size) % passing by mass	Sieve Size (mm)	For Pipe Larger than 375 mm (40 mm Nominal Size) % passing by mass
Class IA*			
20	100	40	100
4.75	0 - 10	4.75	0 – 10
2.5	0 – 5	2.5	0 – 5
0.075	0 - 5	0.075	0 – 5
Class IB			
20	100	40	100
4.75	10 – 50	4.75	10 – 50
2.5	0 – 5	2.5	0 – 5
0.075	0 – 5	0.075	0 – 5
Class II			
20	100	40	100
4.75	0 – 100	4.75	0 – 100
0.075	0 – 12	0.075	0 – 12
Class III			
20	100	40	100
4.75	0 – 100	4.75	0 – 100
0.075	12 – 50	0.075	12 – 50

* Class IA material is suitable for granular material below slabs-on-grade for which a subfloor depressurization system is required for soil gas control, as specified in section 9.16.2.1 of the 2014 Alberta Building Code Volume 2.

B.2 BORED CAST-IN-PLACE CONCRETE PILES

Design and construction of piles should comply with relevant Building Code requirements.

Piles should be installed under full-time inspection of geotechnical personnel. Pile design parameters should be reviewed in light of the findings of the initial bored shafts drilled on a site. Further design review may be necessary if conditions observed during site construction do not conform to design assumptions.

Where fill material, lenses or strata of sand, silt or gravel are present within the designed pile depth, these may be incompetent and/or water bearing and may cause sloughing. Casing should be on hand before drilling starts and be used, if necessary, to seal water and/or prevent sloughing of the hole.

If piles are to be under-reamed (belled), the under-reams should be formed entirely in self-supporting soil and entirely within the competent bearing stratum. Where caving occurs at design elevation, it may be necessary to extend the base of the pile bell to a greater depth. Piles may be constructed with bell having outside diameters up to approximately three times the diameters of their shafts. Piles with shaft diameters of less than 760 mm should not be under-reamed due to difficulties associated with ensuring a clean base.

Prior to pouring concrete, bottoms of pile bells or of straight-shaft end-bearing piles should be cleaned of all disturbed material.

Pile excavation should be visually inspected after completion to ensure that disturbed materials and/or water are not present on the base so that recommended allowable bearing and skin friction parameters may apply.

Visual inspection may be accomplished by the inspector descending into the pile shaft [shaft diameter of 760 mm (30 inches) or greater]. A protective cage and other safety equipment required by government regulations should be provided by the contractor to facilitate down hole inspection.

Other procedures to inspect the pile shafts may be used where shaft diameters of less than 760 mm (30 inch) are constructed, such as inspection with a light.

For safety reasons, where hand cleaning and/or "down shaft" inspection by personnel are required, the pile shaft should be cased full-length prior to personnel entering the shaft.

Reinforcing steel should be on hand and should be placed as soon as the bore has been completed and approved.

Longitudinal reinforcing steel is recommended to counteract the possible tensile stresses induced by frost action and should extend to a minimum depth of 3.5 m. A minimum steel of 0.5 percent of the gross shaft area is recommended.

Where a limited quantity of water is present on the pile base, when permitted or directed by a geotechnical engineer, it should be either removed or absorbed by the addition of dry cement, which should then be thoroughly mixed as an in situ slurry by means of the belling tool, using reverse rotation of the tool. Where significant quantities of water are present and it is impracticable to exclude water from the pile bore, concrete should be placed by tremie techniques or concrete pump.

A "dry" pile should be poured by "free fall" of concrete only where impact of the concrete against the reinforcing cage, which can cause segregation of the concrete, will not occur. A hopper should be used to direct concrete down the centre of the pile base and to prevent impact of concrete against reinforcing steel.

Concrete used for dry piles should be self-compacting and should have a slump of between 50 mm and 130 mm. Concrete for each pile should be poured in one continuous operation and should be placed immediately after excavation and inspection of piles, to reduce the opportunity for the ingress of free water or deterioration of the exposed soil or rock.

If piles cannot be formed in dry conditions, then the concrete should be placed by tremie tube or concrete pump. Concrete placed by tremie should have a slump of not less than 150 mm. A ball or float should be used in the tremie tube to separate the initial charge of concrete from the water in the pile hole.

The outlet of the tremie tube should be maintained at all times 1.0 m to 2.0 m below the surface of the concrete. The diameter of the tremie tube should be at least 200 mm. The tube should be water-tight and not be made of aluminum. Smaller diameter pipes may be used with a concrete pump. The surface of the concrete should be allowed to rise above the cut-off level of the pile, so that when the temporary casing is withdrawn and the surface level of the concrete adjusts to the new volume, the top of the uncontaminated concrete is at or above the cut-off level. The concrete should be placed in one continuous, smooth operation without any halts or delays. Placing the lower portion of the pile by tremie tube and placing the upper portion of the pile by free fall should not be permitted, to ensure that defects in the pile shaft at the top of the tremie concrete do not occur.

As the surface of the concrete rises in the pile bore, the water in the pile bore will be displaced upwards and out of the top of the pile casing. It may be necessary to pump off this water to a container to temporary ditch drain to prevent the formation of ice or flooding conditions and possibly damage to existing structures.

When concreting by tremie techniques, allowance should be made for the removal of contaminated or otherwise defective concrete at the tops of the piles.

The casing should be filled with concrete and then the casing should be withdrawn smoothly and continuously.

Sufficient concrete should be placed to allow for additional volume of the casing and reduction in level of the concrete as the casing is withdrawn. Concrete should not be poured on top of previously poured concrete after the casing is withdrawn.

An accurate record of the volume of concrete placed should be maintained as a check that a continuous pile has been formed.

Concrete should not be placed if its temperature is less than 5°C or exceeds 30°C or if it is more than two hours old.

Where tension, horizontal or bending moment loading on the pile is foreseen, steel reinforcing should be extended and tied into the grade beam or pile cap. The steel should be designed to transfer loads to the required depth in the pile and to resist resultant bending moments and shear forces.

Void formers should be placed beneath all grade beams to reduce the risk of damage due to frost effects or soil moisture changes.

Where the drilling operation might affect the concrete in adjacent pile (ie. where pile spacing is less than about three diameters), drilling should not be carried out before the previously poured pile concrete has set for at least 24 hours.

Where a group of four or more piles are used, the allowable working load on the piles may need to be modified to allow for group effects.

Piles should be spaced no closer than 2.5 times the pile shaft diameter, measured centre-to-centre. Strict control of pile location and vertically should be exercised to provide accurate locations and spacing of piles. In general, piles should be constructed within a tolerance of 75 mm plan distance in any direction and within a vertically of 1 in 75 mm.

A detailed record should be kept of pile construction including information such as pile number, shaft/base diameter, date and time bored, date and time concreted, elevation of piling platform, depths (from piling platform level) to pile base and to concrete cut-off level, length of casing used, detailed of reinforcement, brief description of soils encountered in the bore and details of any unusual occurrences during construction.

If a large number of piles are to be installed, it may be possible to optimize the design on the basis of pile load test.

B.3 CONSTRUCTION EXCAVATIONS

Construction should be in accordance with good practice and comply with the requirements of the responsible agencies.

All excavations greater than 1.5 m deep should be sloped or shored for worker protection.

Shallow excavations up to 3 m depth may use temporary side slopes of 1H:1V. A flatter slope of 2H:1V should be used if groundwater is encountered. Localized sloughing can be expected from these slopes.

Deep excavations or trenches may require temporary support if space limitations or economic considerations preclude the use of sloped excavations.

For excavations greater than 3 m depth, temporary support should be designed by a qualified geotechnical engineer. The design and proposed installation and construction procedures should be submitted to McIntosh•Lalani Engineering Ltd. for review.

The construction of a temporary support system should be monitored. Detailed records should be taken of installation methods, materials, in situ conditions and the movement of the system. If anchors are used, they should be load tested. McIntosh•Lalani Engineering Ltd. can provide further information on monitoring and testing procedures, if required.

Attention should be paid to structures or buried service lines close to the excavation. For structures, a general guideline is that if a line projected down at 45° from a horizontal, from the base of foundations of adjacent structures, intersects the extent of the proposed excavation, then these structures may require underpinning or special shoring techniques to avoid damaging earth movements. The need for any underpinning or special shoring techniques and the scope of monitoring required can be determined when details of the service ducts and vaults, foundation configuration of existing buildings and final design excavation levels are known.

No surface surcharges should be placed closer to the edge of the excavation than a distance equal to the depth of the excavation, unless the excavation support system has been designed to accommodate such surcharge.

B.4 PROOF-ROLLING

Proof-rolling is method of detecting soft areas in an "as-excavated" subgrade for fill, pavement, floor or foundations or detecting non-uniformity of compacted embankment. The intent is to detect soft areas or areas of low shear strength not otherwise revealed by means of test holes, density testing or visual examination of the site surface and to check that any fill placed or subgrade meets the necessary design strength requirements.

Proof-rolling should be observed by qualified geotechnical personnel.

Proof-rolling is generally accomplished by the use of a heavy (15-60 tonne) rubber-tired roller having found wheels abreast on independent axles with high contact wheel pressures [inflation pressures ranging from 550 kPa (80 psi) up to 1,030 kPa (150 psi)].

A heavily-loaded truck may be used in lieu of the equipment described in the paragraph above. The truck should be loaded to approximately 10 tonnes (22,000 lbs) per axle and a minimum tire pressure of 550 kPa (80 psi).

Ground speed to be maximum of 8 km/hr (133 m/min) (5 mph) (400 ft/min). Recommended speed is 4 km/hr (65 m/min) (2.5 mph) (200 ft/min).

The recommended procedures is two complete coverages with the Proof-rolling equipment in one direction and a second series of two coverages made at right angles to the first series; one "coverage" means that every point of the proof-rolled surface has been subjected to the tire pressure of a loaded wheel. Less rigorous procedures may be acceptable under certain conditions subject to the approval of an engineer.

Any soft areas rutted or displaced materials detected should be either recompacted with additional fill or the existing material removed and replaced with general engineered fill or properly moisture conditioned as necessary.

The surface of the grade under the action of the proof-rolled should be observed, noting visible deflection and rebound of the surface or shear failure in the surface of granular soils as ridging between wheel tracks.

If any part of an area indicates significantly more distress than other parts, the cause should be investigated, by, for example, shallow auger holes.

In the case of granular subgrades, distress will generally consist of either compression due to insufficient compaction or shearing under the tires. In the first case, proof-rolling should be continued until no further compression occurs. In the second case, the tire pressure should be reduced to a point where the subgrade can carry the load without significant deflection and subsequently, gradually increased to its specified pressure as the subgrade increases in shear strength under this compaction.

B.5 SHALLOW FOUNDATIONS

Design and construction of shallow foundations should comply with relevant Building Code requirements.

The term "shallow foundations" includes strip and spread footings, mat slab and raft foundations.

Minimum footing dimensions in plan should be 0.45 m for strip footings and 0.9 m for square footings.

No loose, disturbed or sloughed material should be allowed to remain in open foundation excavations. Hand cleaning should be undertaken to prepare an acceptable bearing surface. Recompaction of disturbed or loosened bearing surface may be required.

Foundation excavation and bearing surfaces should be protected from rain, snow, freezing temperatures, drying and the ingress of free water, during and after footing construction.

Footing excavations should be carried down into the designated bearing stratum.

After the bearing surface is approved, a mud slab should be poured to protect the soil and provide a working surface for construction, should immediate foundation construction not be intended.

All constructed foundations should be placed on unfrozen soils, which should be at all times protected from frost penetration.

All foundation excavations and bearing surface should be observed by a qualified geotechnical engineer to confirm that the recommendations contained in this report have been followed and that soil conditions are consistent with those assumed in the design.

Where over-excavation has been carried out through a weak or unsuitable stratum in order to reach a suitable bearing stratum; or where a foundation pad is to be placed above stripped natural ground surface, lean-mix concrete or structural fill may be used to reinstate the grade. These materials are defined under the separate heading "Backfill Materials and Compaction."

B.6 DRIVEN STEEL PILES

Full time observation of pile driving should be carried out by qualified geotechnical personnel.

Piles should initially be designed for minimum section and embedment on the basis of static design loads and shaft resistance.

Final design of driven steel piles could be carried out using a wave equation analysis. Design by this method would enable an optimum match of hammer type and weight to pile type and soil conditions and allow a check to be made on driving stresses.

Nominal Pile Size	Approximate Driving Energy	Final Set Blows Per 25 mm (1") For Last 25 mm (1")
250 mm (10")	37,000 J	20

	(27,500 ft.lbs)	
	55,000 J (40,000 ft.lbs)	15
360 mm (14")	55,000 J (40,000 ft.lbs)	20

Steel piles should conform to the requirements of the applicable Building Code. When steel pipe piles are filled with concrete, it should conform to the requirements of the applicable Building Code, but should be of sufficient slump (150 mm or greater) to prevent voids forming and its consistency should be such as to prevent segregation.

Driving records should be kept for each pile. Information to be recorded should include pile dimensions, hammer type, rated energy, ram weight, cap block weight and type, anvil weight, number of blows for each 0.3 m of penetration and final set.

The elevation of the tops of driven piles should be measured immediately after driving. If uplift occurs in any piles during the driving of adjacent piles, the displaced piles should be re-driven to at least their previous final elevation and final set.

Piles should be spaced no closer than 2.5 times the pile diameter, measured centre-to-centre. Where piles are driven in groups, they should be driven from the centre outwards. In general, all piles in a group should be driven to approximately the same tip elevation.

If a group of four or more piles is required, group effects may reduce the working load of the pile group below calculated from the number of piles multiplied by the working load for an individual pile. If required, McIntosh•Lalani Engineering Ltd. can provide further design parameters for this case at the final design stage.

Strict control of pile location and orientation should be exercised to obtain accurate pile installation. Preboring of the surficial soils may be necessary to ensure proper location of the pile tip.

When piles are to be driven into very hard or frozen strata or boulders, special tips or pre-boring may be required.

For piles which will displace a significant amount of soil during driving, such as closed-end pipe piles, care should be taken that the driving will not cause strains of such magnitude as to cause damage to nearby structures.

Pile driving may result in significant vibrations which may be unacceptable for adjacent structures. In areas where this is a concern, continuous monitoring of vibrations induced in adjacent structures by a seismograph is recommended in order to assess the potential for damage and the need for modification of procedures.

If a large number of piles are to be installed, it may be possible to optimize the design on the basis of pile load tests.