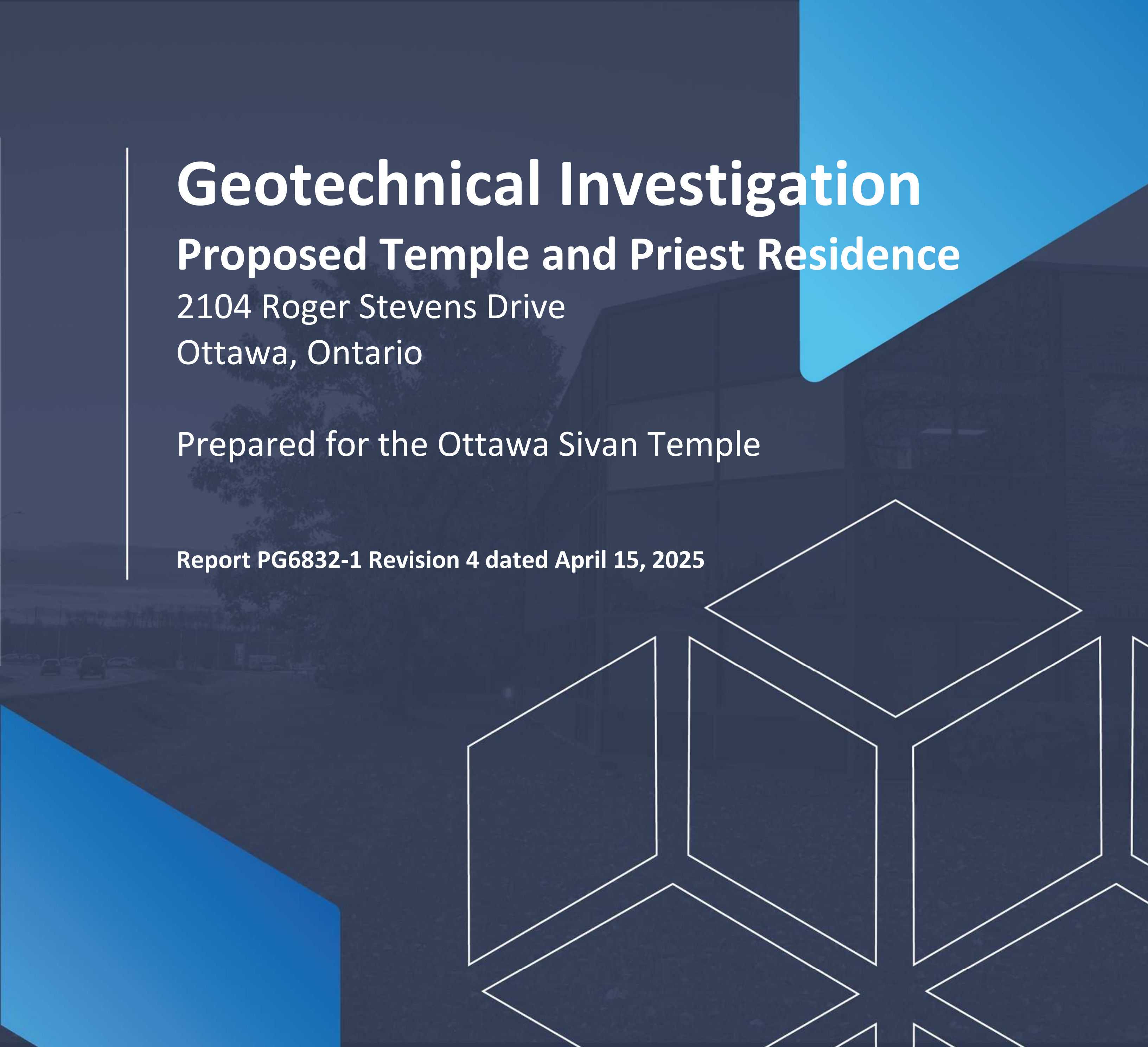

Geotechnical Investigation

Proposed Temple and Priest Residence

2104 Roger Stevens Drive
Ottawa, Ontario

Prepared for the Ottawa Sivan Temple

Report PG6832-1 Revision 4 dated April 15, 2025

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by the Ottawa Sivan Temple to conduct a geotechnical investigation for the Proposed Temple and Priest Residence to be located at 2104 Roger Stevens Drive in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

❏ Determine the subsoil and groundwater conditions at this site by means of boreholes.

❏ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed Temple will consist of a single-storey building with a basement level and an approximate footprint of 1,000 m^2.

The proposed development will further include a two-story Priest Residence structure with an approximate footprint of 240 m^2, located behind the proposed Temple structure. It is understood that this residential building will have a septic tank zone located to the south of the residential building, with an approximate footprint of 480 m^2.

The proposed buildings will be immediately surrounded by heavy duty pavement, with asphalt-paved access lanes and permeable paver parking areas at the southern and western boundaries of the development. A retaining wall is also anticipated to be located to the east and south of the proposed buildings.

The development will involve demolishing the existing building located on-site.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the investigation was carried out on September 19, 2023, and consisted of a total of 3 boreholes sampled to a maximum depth of 6.7 m below ground surface. The borehole locations were distributed in a manner to provide general coverage of the proposed development, taking into consideration underground utilities and site features. The locations of the boreholes are shown on Drawing PG6832-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a track-mounted drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer.

Sampling and In Situ Testing

The borehole samples were recovered from the auger flights and using a 50 mm diameter split-spoon sampler. The samples were initially classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the auger and split-spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

The thickness of the overburden was evaluated during the course of the investigation by a dynamic cone penetration test (DCPT) at borehole BH 2-23. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

Groundwater

Each borehole was fitted with a flexible polyethylene standpipe to allow for groundwater level monitoring. The groundwater observations are discussed in Section 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The borehole locations, and the ground surface elevation at each borehole location, were surveyed by Paterson using a GPS unit with respect to a geodetic datum. The locations of the boreholes, and ground surface elevation at each borehole location, are presented in Drawing PG6832-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. All samples from the current investigation will be stored in the laboratory for one month after this report is completed. They will then be discarded unless we are otherwise directed.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by an existing Temple building with an asphalt paved driveway. The site is bordered by Roger Stevens Drive to the north, agricultural land to the south and east, and residential properties to the west.

Generally, the ground surface at the subject site slopes downward gradually from north to south, from an approximate geodetic elevation of 92 m near Roger Stevens Drive to about geodetic elevation of 90 m at the southern end of the site.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consists of an approximate 0.1 m thickness of topsoil underlain by fill and/or glacial till. Fill was encountered within boreholes BH 2-23 and BH 3-23, consisting of an approximate 1.4 to 2.1 m thickness of loose to compact, brown silty sand with gravel. Fill was not encountered in borehole BH 1-23.

Underlying the topsoil and/or fill, the glacial till deposit was observed, consisting of compact to very dense, brown to grey silty sand to sandy silt with varying amounts of gravel, cobbles, and boulders.

Practical refusal to augering was encountered within borehole BH 1-23 at an approximate depth of 5 m below the existing ground surface. However, practical refusal to the DCPT was encountered at a depth of about 10.8 m below the existing ground surface at borehole BH 3-23.

Reference should be made to the Soil Profile and Test Data Sheets in Appendix 1 for details of the soil profile encountered at each borehole location.

Bedrock

Based on available geological mapping, the bedrock in the area of the subject site consists of dolomite of Oxford formation with an overburden thickness ranging between 15 to 25 m.

4.3 Groundwater

Groundwater levels were recorded at each piezometer location on September 21, 2023 and March 25, 2025. The measured groundwater levels are presented in Table 1 below, and on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Level (m)	Groundwater Elevation (m)	Recording Date
BH 1-23	91.00	2.20	88.80	Sept. 21, 2023
		Inaccessible	-	March 25, 2025
BH 2-23	90.49	2.47	88.02	Sept. 21, 2023
		Inaccessible	-	March 25, 2025
BH 3-23	90.23	2.57	87.66	Sept. 21, 2023
		1.99	88.24	March 25, 2025

Note: Ground surface elevations at borehole locations were surveyed by Paterson and are referenced to a geodetic datum.

It should be noted, however, that groundwater levels may vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is recommended that foundation support for the proposed temple structure and residential building consist of conventional spread footings bearing on the undisturbed, compact to very dense glacial till.

A silty clay deposit was not encountered within the boreholes, therefore a permissible grade raise restriction is not applicable for the subject site.

Due to the anticipated retaining wall along the southern boundary of the development which is to be more than 1 m in height, a global stability analysis has been conducted, which is provided in Section 6.8.

During the proposed building excavation, the contractor should expect to encounter boulders within the glacial till deposit.

The above and other considerations are discussed in the following paragraphs.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures. It is anticipated that the existing fill within the future building footprints, free of deleterious material and significant amounts of organics, can be left in place below the proposed building footprints outside of lateral support zones for the footings. However, it is recommended that the existing fill layer be proof-rolled several times under dry conditions and above freezing temperatures and approved by Paterson personnel at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill.

Fill Placement

Engineered fill placed for grading beneath the proposed building, where required, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where the settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on an undisturbed, compact to very dense glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**, incorporating a geotechnical factor of 0.5.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

Footings placed designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a soil bearing medium when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through the soil of the same or higher capacity as that of the bearing medium.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C**. Reference should be made to the latest revision of the 2024 Ontario Building Code for a full discussion of the earthquake design requirements.

The soils below the underside of footing (USF) elevation, consisting of the glacial till deposit, have been evaluated for liquefaction potential in accordance with the "Liquefaction Resistance of Soils" document prepared by Youd et al. (2001), and were determined to have suitable factors of safety exceeding 1.1 against liquefaction. Accordingly, soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab / Slab-on-Grade Construction

With the removal of all topsoil and deleterious fill from within the footprints of the proposed buildings, the existing fill or undisturbed, glacial till will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

Where the slab subgrade consists of the existing fill, a vibratory drum roller should complete several passes over the subgrade surface as a proof-rolling program. Any poor performing areas should be removed and reinstated with an engineered fill, such as OPSS Granular B Type II.

For structures with slab-on-grade construction, the upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone. All backfill material within the footprints of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

For structures with basement slabs, it is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone. Further, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided underlying any basement slabs. This is discussed further in Subsection 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

Two distinct conditions, static and seismic, should be reviewed for design calculations. The corresponding parameters are presented in the following page.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45-a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.338g according to the OBC 2024. Note that the vertical seismic coefficient is assumed to be zero. The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$$h = \{P_o \cdot (H/3)+\Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2024.

5.7 Pavement Design

For the proposed surface parking areas, the pavement structures provided in Tables 2, 3 and 4 on the next page are recommended.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ, soil or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 3 - Recommended Pavement Structure
Access Lanes and Heavy Truck Parking Areas

Thickness (mm)	Material Description
40	**Wear Course** - Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ, soil or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 4 - Recommended Permeable Pavement Structure – Car Only Parking

Thickness (mm)	Material Description
-	**Permeable Paver Structure** (as per manufacturer specifications)
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soils, or OPSS Granular B Type I or II material placed over in situ soil or bedrock	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in a maximum of 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Backfill

Foundation Drainage

A perimeter foundation drainage system is recommended to be provided for structures with below-grade space. The system, where required, should consist of a 100 mm or 150 mm diameter perforated and corrugated plastic pipe, surrounded on all-sides by 150 mm of 19 mm clear crushed stone, which is placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer or sump pit provided below the basement level of the structure.

Underslab Drainage

For any buildings with below-grade space, underslab drainage will be required to control water infiltration below the lowest level floor slab. For preliminary design purposes, we recommend that 100 mm or 150 mm diameter perforated pipes be placed at approximate 6 m centres. The spacing of the underslab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite board, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type II granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent combination of soil cover and foundation insulation, should be provided in this regard.

Exterior unheated footings, such as isolated piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure, and require additional protection, such as soil cover of 2.1 m, or an equivalent combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is expected that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e., unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. A flatter slope is required for excavation below the groundwater level. Excavations below the groundwater level should be cut back at a maximum slope of 1.5H:1V.

The subsoil at this site is considered to be mainly a Type 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in a maximum of 225 mm thick lifts and compacted to 98% of the SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in a maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be moderate and controllable using open sumps. The contractor should be prepared to direct water away from all subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16.

Impacts to Neighboring Properties

As the subsurface conditions consist of a compact to very dense glacial till dewatering operations would not cause settlement or other impacts to nearby structures.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures using straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at the founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost into the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided if required.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a moderately aggressive corrosive environment.

6.8 Global Stability Analysis

Due to the proposed retaining wall along the eastern and southern boundary of the development, which is greater than 1 m in height, a global stability analysis is required in accordance with the City of Ottawa’s “Slope Stability Guidelines for Development Applications”.

Accordingly, a global stability analysis of the proposed site conditions was conducted using SLIDE, a computer program which permits a two-dimensional stability analysis using several methods including the Bishop’s method and Morgenstern-Price method, which are widely used and accepted analysis methods. A horizontal acceleration of 0.16 g was utilized for the seismic analyses.

The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favouring failure. Theoretically, a factor of safety (F.o.S.) of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, an F.o.S. greater than one is usually required to ascertain that the risks of failure are acceptable. A minimum F.o.S. of 1.5 is generally recommended for static analysis conditions and a minimum F.o.S. of 1.1 is generally recommended for seismic analysis conditions, where the failure of the slope would endanger permanent structures.

Three analysis cross-sections (Sections A-A, B-B, and C-C) were analyzed based on the proposed site conditions and a review of the available topographic mapping. The locations of these cross-sections are indicated in Drawing PG6832-1 in Appendix 2.

The effective and total strength soil parameters used for the static and seismic analyses, respectively, were chosen based on the subsoil information recovered during the geotechnical investigation and in general accordance with the typical ranges of values provided in the City of Ottawa's "Slope Stability Guidelines for Development Applications", referenced above.

The total and effective strength soil parameters used for static analysis are presented in Table 5 below.

Table 5 - Effective Strength Soil and Material Parameters (Static Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Topsoil	16	33	5
Crushed Stone Fill	18	31	0
Silty Sand Fill	20	35	0
Glacial Till	20	33	0

The total and effective strength soil parameters used for seismic analysis are presented in Table 6 below.

Table 6 - Effective Strength Soil and Material Parameters (Seismic Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Topsoil	16	33	5
Crushed Stone Fill	18	31	0
Silty Sand Fill	20	35	0
Glacial Till	20	33	0

Analysis Results

The results for the global stability analyses under static and seismic conditions at cross-sections A-A, B-B, and C-C are shown in Figures 2A, 2B, 2C, 2D, 2E and 2F in Appendix 2. The results of the global stability analyses indicate that the factor of safety exceeds 1.5 and 1.1 under static and seismic analysis conditions, respectively.

Therefore, the global stability of the proposed retaining wall is considered acceptable, from a geotechnical perspective.

7.0 Recommendations

It is a requirement for the foundation data provided herein to be applicable that the following material testing and observation program be performed by the geotechnical consultant.

- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon request, following the completion of a satisfactory material testing and observation program by Paterson.

All excess soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

8.0 Statement of Limitations

The recommendations provided herein are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than the Ottawa Sivan Temple, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
April 15, 2025
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Scott Dennis

Deepak k Rajendran, E.I.T.

Scott S. Dennis, P.Eng.

Report Distribution:

☐ Ottawa Sivan Temple (e-mail copy)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2104 Roger Stevens Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 367934.45 **NORTHING:** 5000149.98 **ELEVATION:** 91.00

PROJECT: Proposed Hindu Temple

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 19, 2023

FILE NO. : PG6832

HOLE NO. : BH 1-23

SAMPLE DESCRIPTION

GROUND SURFACE

TOPSOIL 0.05m [90.95m]

GLACIAL TILL: Dense to very dense, brown silty sand to sandy silt, with gravel, cobbles and boulders, trace clay

4.95m [86.05m]

End of Borehole

Practical refusal to augering at 4.95 m depth

(GWL at 2.20 m depth - September 21, 2023)

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			
SS 2	75	7-19-27-33 46	
SS 3	60	20-50-/-/ 50/0.1	
SS 4	50	20-42-48-42 90	
SS 5	58	31-40-38-27 78	
SS 6	67	5-11-13-12 24	
SS 7	25	6-9-50-/ 59/0.23	

■ PEN. RESIST. (BLOWS/0.3m)
DCPT (50mm DIA. CONE)
20 40 60 80

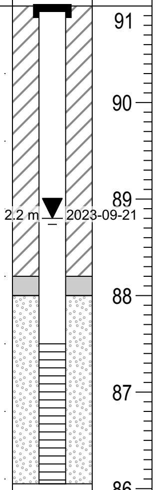
△ REMOULDED SHEAR STRENGTH, Cur (kPa)
▲ PEAK SHEAR STRENGTH, Cu (kPa)
20 40 60 80

PL (%) WATER CONTENT (%) LL (%)
20 40 60 80

DEPTH (m): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14

ELEVATION (m): 91 90 89 88 87 86 85 84 83 82 81 80 79 78 77

PIEZOMETER CONSTRUCTION



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PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2104 Roger Stevens Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 367950.64 **NORTHING:** 5000130.91 **ELEVATION:** 90.48

PROJECT: Proposed Hindu Temple

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 19, 2023

FILE NO. : PG6832

HOLE NO. : BH 2-23

SAMPLE DESCRIPTION

GROUND SURFACE

TOPSOIL 0.10m [90.38m]

FILL: Brown silty sand, trace gravel 2.13m [88.35m]

GLACIAL TILL: Dense, brown silty sand to sandy silt, with gravel, cobbles and boulders 6.71m [83.77m]

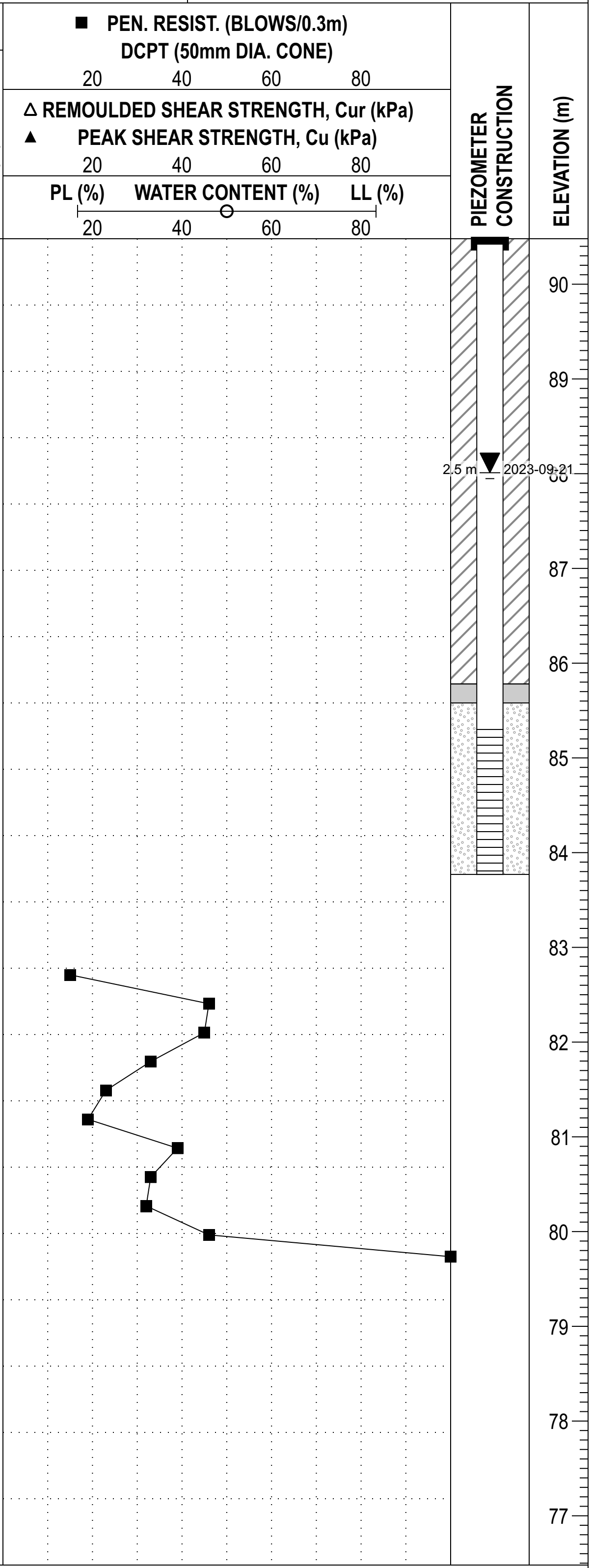
Dynamic Cone Penetration Test commenced at 6.71 m depth 10.82m [79.66m]

End of Borehole

Practical refusal to DCPT at 10.82 m depth

(GWL at 2.47 m depth - September 21, 2023)

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			
SS 2	67	3-3-6-5 9	
SS 3	50	8-12-12-12 24	
SS 4	50	11-20-26-16 46	
SS 5	46	10-13-11-12 24	
SS 6	50	4-12-13-14 25	
SS 7	79	10-12-21-17 33	
SS 8	9	13-17-23-50 40	
SS 9	88	13-19-21-19 40	



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2104 Roger Stevens Drive, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 367971.19 **NORTHING:** 5000144.83 **ELEVATION:** 90.23

PROJECT: Proposed Hindu Temple

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 19, 2023

FILE NO. : PG6832

HOLE NO. : BH 3-23

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
GROUND SURFACE	0				
TOPSOIL 0.10m [90.13m]		AU 1			
FILL: Brown silty sand, trace gravel 1.35m [88.88m]	1	SS 2	50	3-3-3-3 6	
GLACIAL TILL: Compact to very dense, brown silty sand to sandy silt, with gravel, cobbles and boulders	2	SS 3	0	5-5-6-10 11	
	3	SS 4	67	3-8-7-8 15	
		SS 5	33	4-11-22-25 33	
	4	SS 6	79	5-7-9-7 16	
	5	SS 7	67	13-13-17-15 30	
	6	SS 8	67	5-9-9-18 18	
6.71m [83.52m]		SS 9	71	16-20-49-30 69	
End of Borehole (GWL at 2.57 m depth - September 21, 2023)	7				

■ PEN. RESIST. (BLOWS/0.3m)
DCPT (50mm DIA. CONE)
20 40 60 80

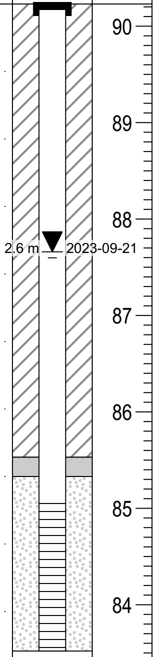
△ REMOULDED SHEAR STRENGTH, Cur (kPa)
▲ PEAK SHEAR STRENGTH, Cu (kPa)
20 40 60 80

PL (%) WATER CONTENT (%) LL (%)
20 40 60 80

DEPTH (m): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14

ELEVATION (m): 90 89 88 87 86 85 84 83 82 81 80 79 78 77

PIEZOMETER CONSTRUCTION



DISCLAIMER: THE DATA PRESENTED IN THIS LOG IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHO IT WAS PRODUCED. THIS LOG SHOULD BE READ IN CONJUNCTION WITH ITS COORESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	‘N’ Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

Consistency	Undrained Shear Strength (kPa)	‘N’ Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity: $S_t < 2$
Medium Sensitivity: $2 < S_t < 4$
Sensitive: $4 < S_t < 8$
Extra Sensitive: $8 < S_t < 16$
Quick Clay: $S_t > 16$

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW - Thin wall tube or Shelby tube, generally recovered using a piston sampler
G - "Grab" sample from test pit or surface materials
AU - Auger sample or bulk sample
WS - Wash sample
RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic Limit, % (water content above which soil behaves plastically)
PI - Plasticity Index, % (difference between LL and PL)

Dxx - Grain size at which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = $D60 / D10$

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

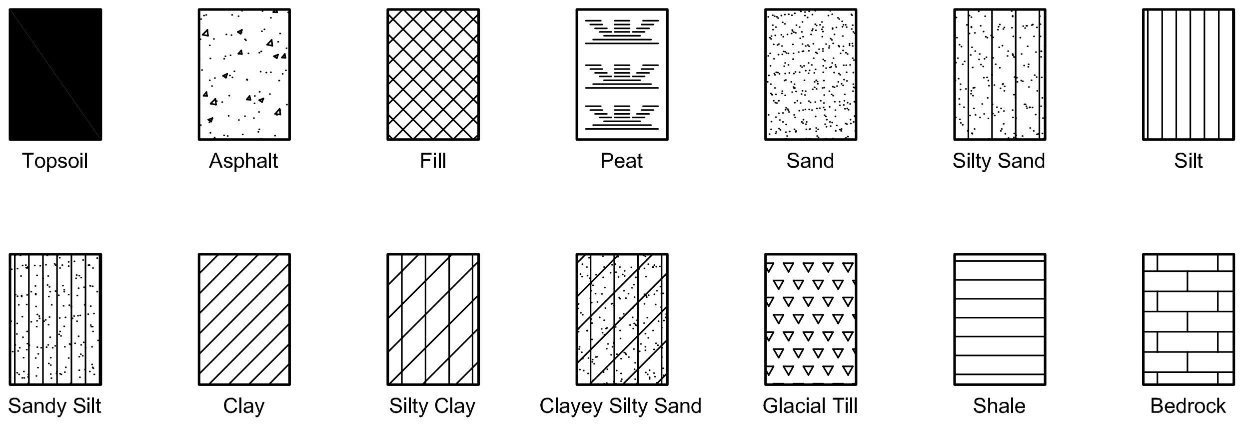
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

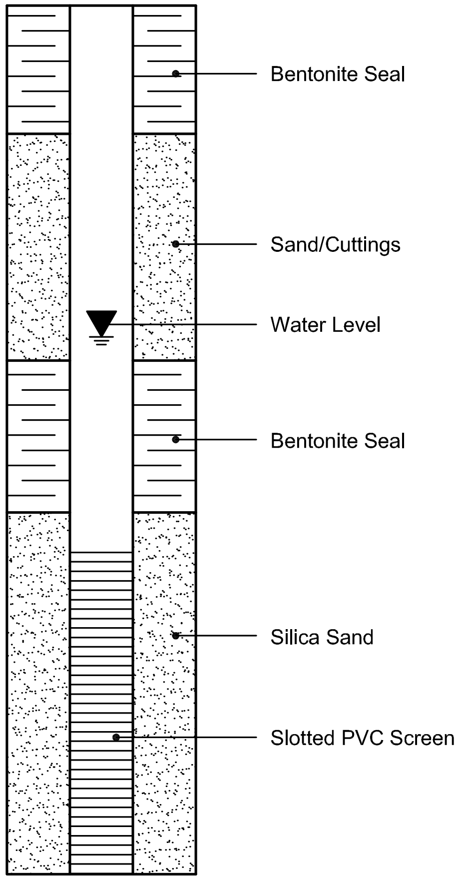
SYMBOLS AND TERMS (continued)

STRATA PLOT

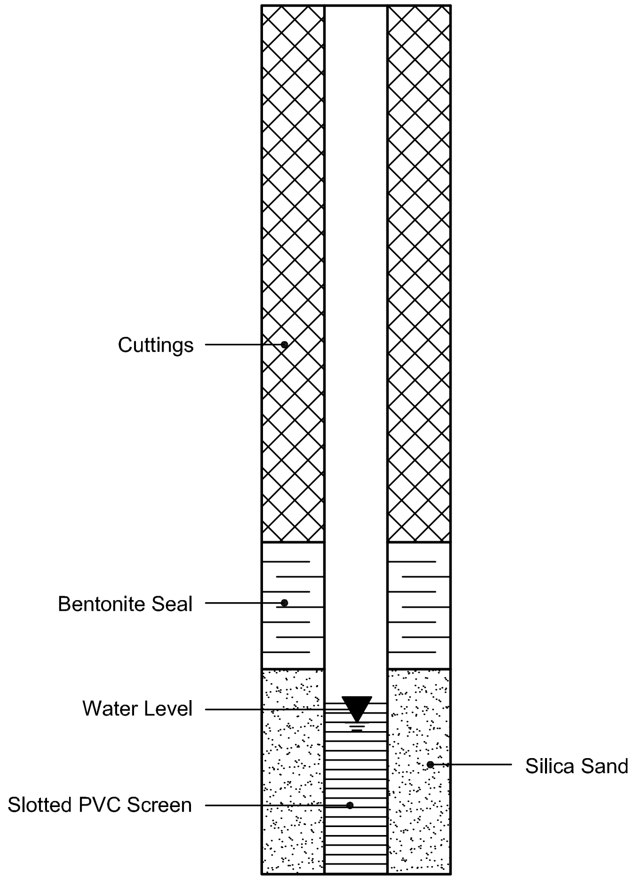


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 22-Sep-2023

Client: **Paterson Group Consulting Engineers (Ottawa)**

Order Date: 20-Sep-2023

Client PO: 58409

Project Description: PG6832

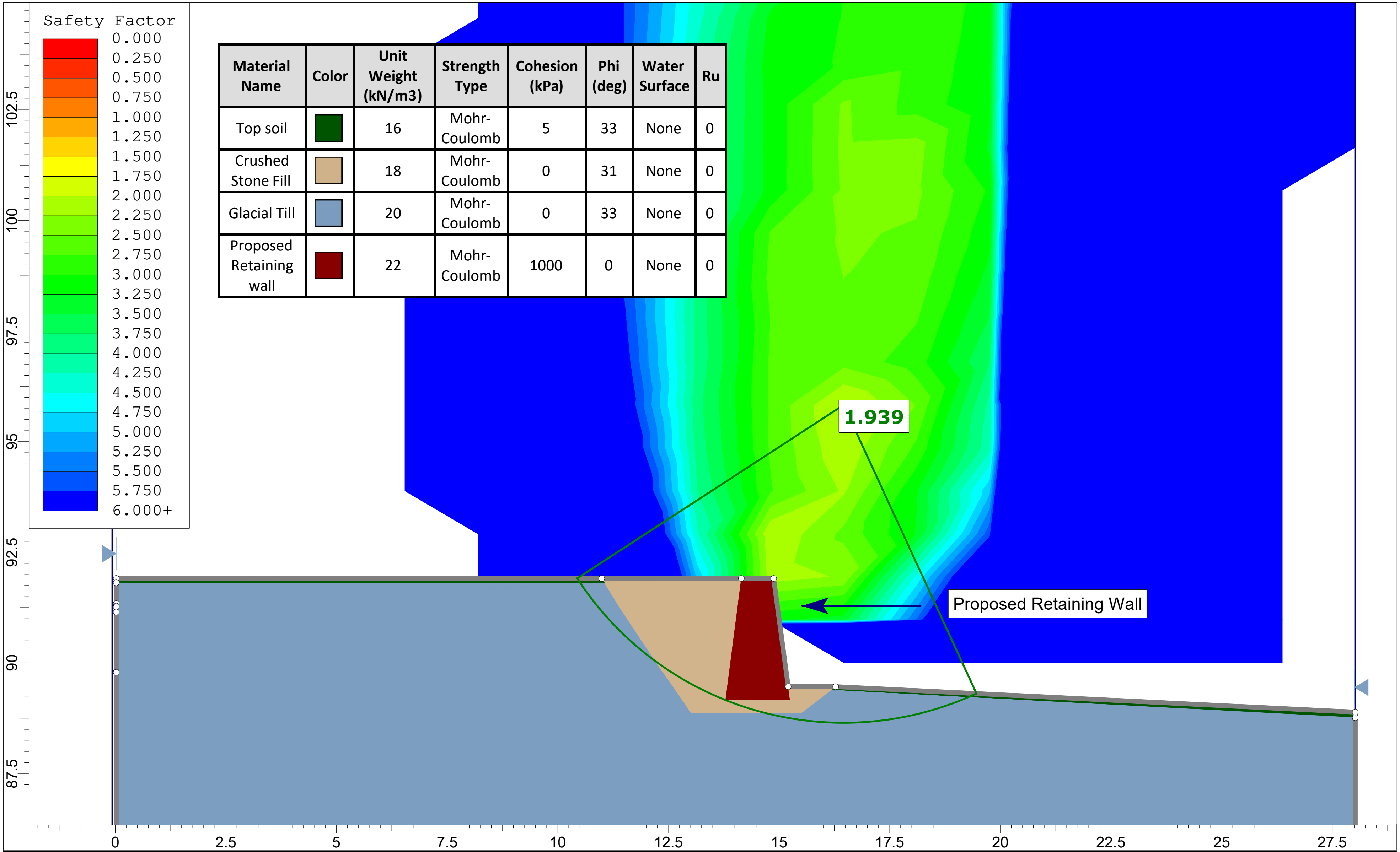
	Client ID:	BH1-23 SS3	-	-	-		
	Sample Date:	19-Sep-23 09:00	-	-	-	-	-
	Sample ID:	2338291-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	92.9	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.90	-	-	-	-	-
Resistivity	0.1 Ohm.m	58.6	-	-	-	-	-
Anions							
Chloride	10 ug/g	31	-	-	-	-	-
Sulphate	10 ug/g	17	-	-	-	-	-

APPENDIX 2

FIGURE 1

KEY PLAN

PATERSON
GROUP

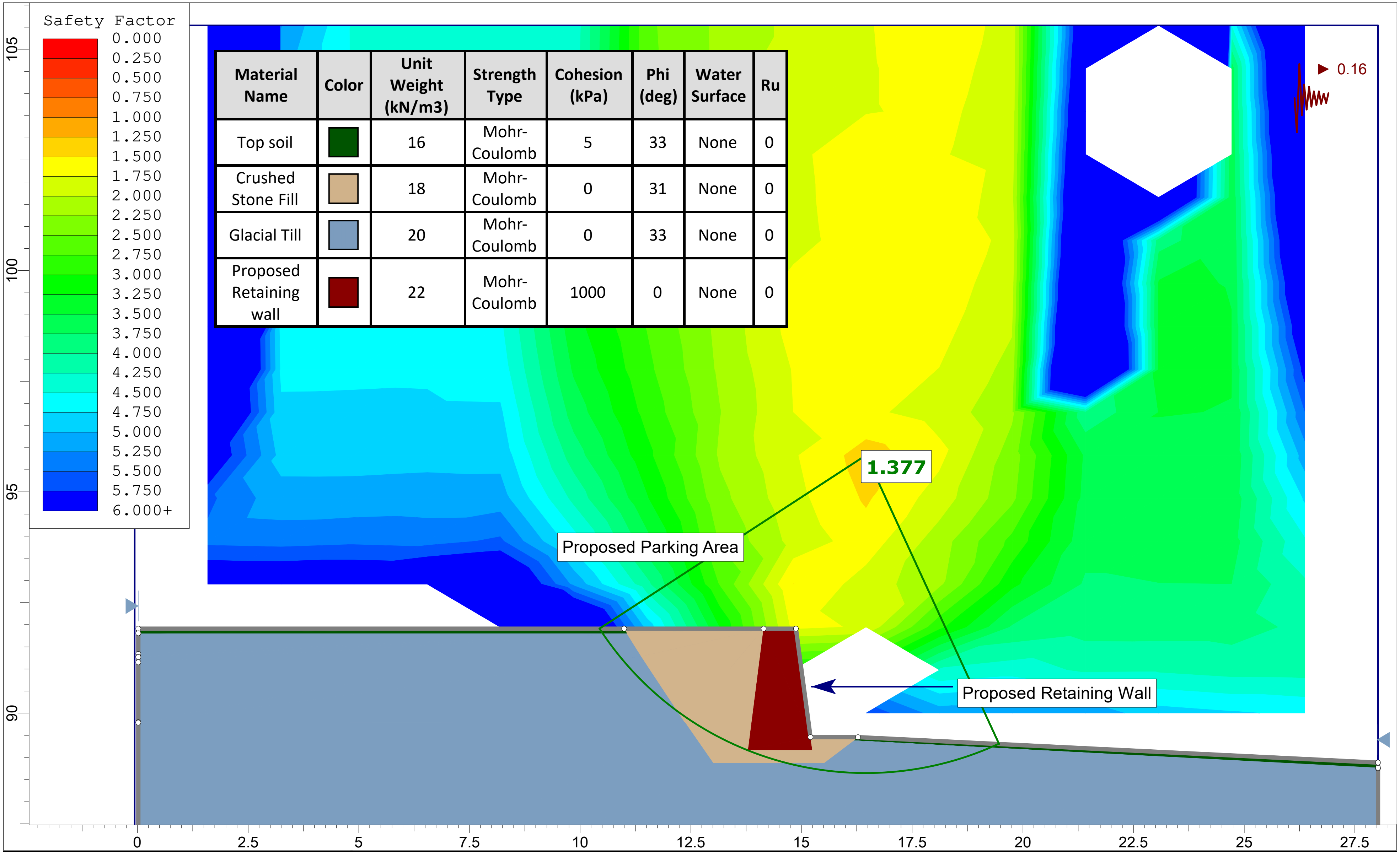


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Ru
Top soil	🟩	16	Mohr-Coulomb	5	33	None	0
Crushed Stone Fill	🟫	18	Mohr-Coulomb	0	31	None	0
Glacial Till	🟦	20	Mohr-Coulomb	0	33	None	0
Proposed Retaining wall	🟥	22	Mohr-Coulomb	1000	0	None	0

PATERSON GROUP

SLIDEINTERPRET 9.025

Project	PG6832 - 2104 Roger Stevens Drive		
Slope Section	A	*Scenario*	Proposed Static Loading
Drawn By	DR	*Company*	Ottawa Sivan Temple
Date	2025-01-27	*File Name*	Figure 2A

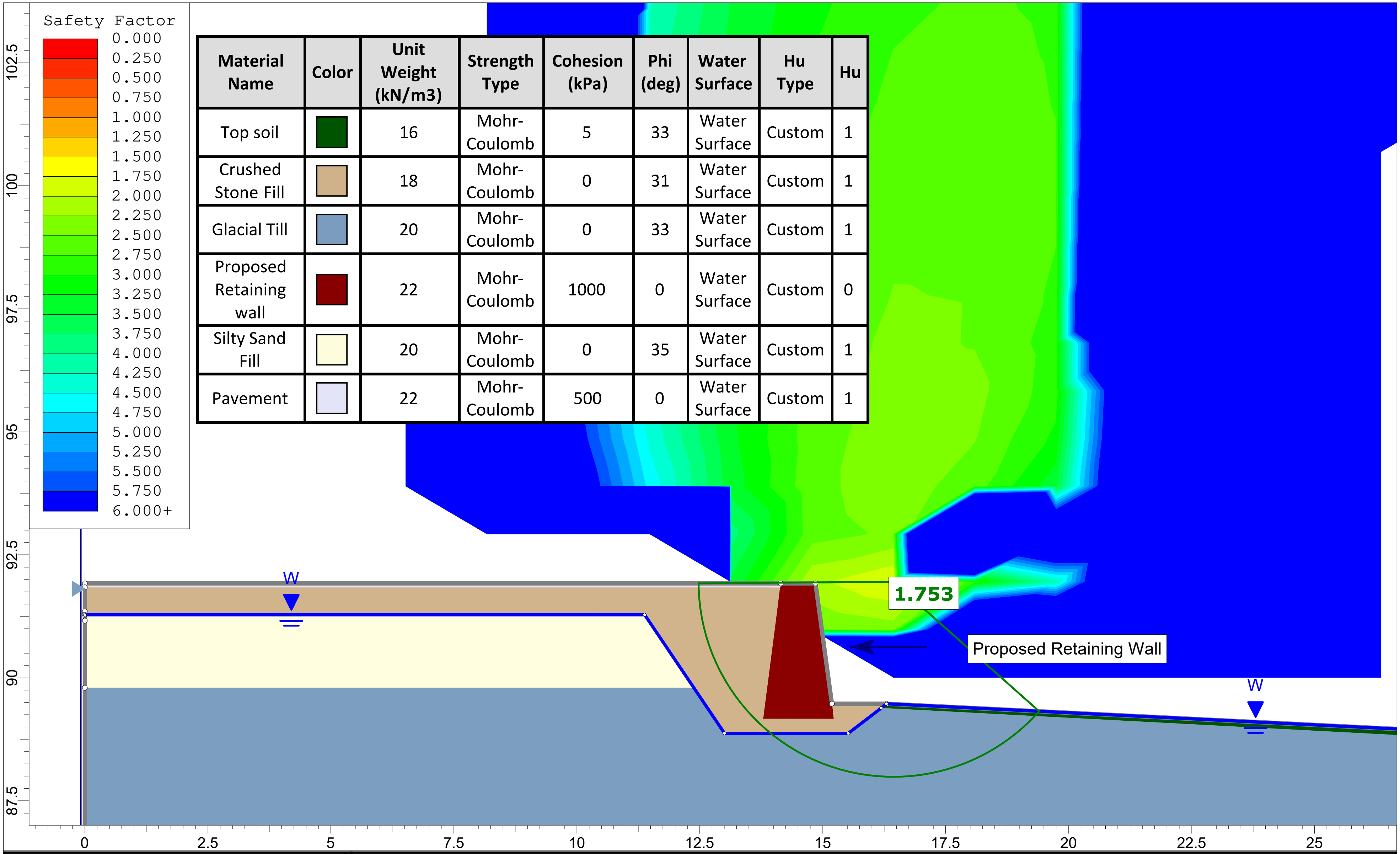


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Ru
Top soil	🟩	16	Mohr-Coulomb	5	33	None	0
Crushed Stone Fill	🟫	18	Mohr-Coulomb	0	31	None	0
Glacial Till	🟦	20	Mohr-Coulomb	0	33	None	0
Proposed Retaining wall	🟥	22	Mohr-Coulomb	1000	0	None	0

PATERSON GROUP

SLIDEINTERPRET 9.025

Project	PG6832 - 2104 Roger Stevens Drive		
Slope Section	A	*Scenario*	Proposed Seismic Loading
Drawn By	DR	*Company*	Ottawa Sivan Temple
Date	2025-01-27	*File Name*	Figure 2B



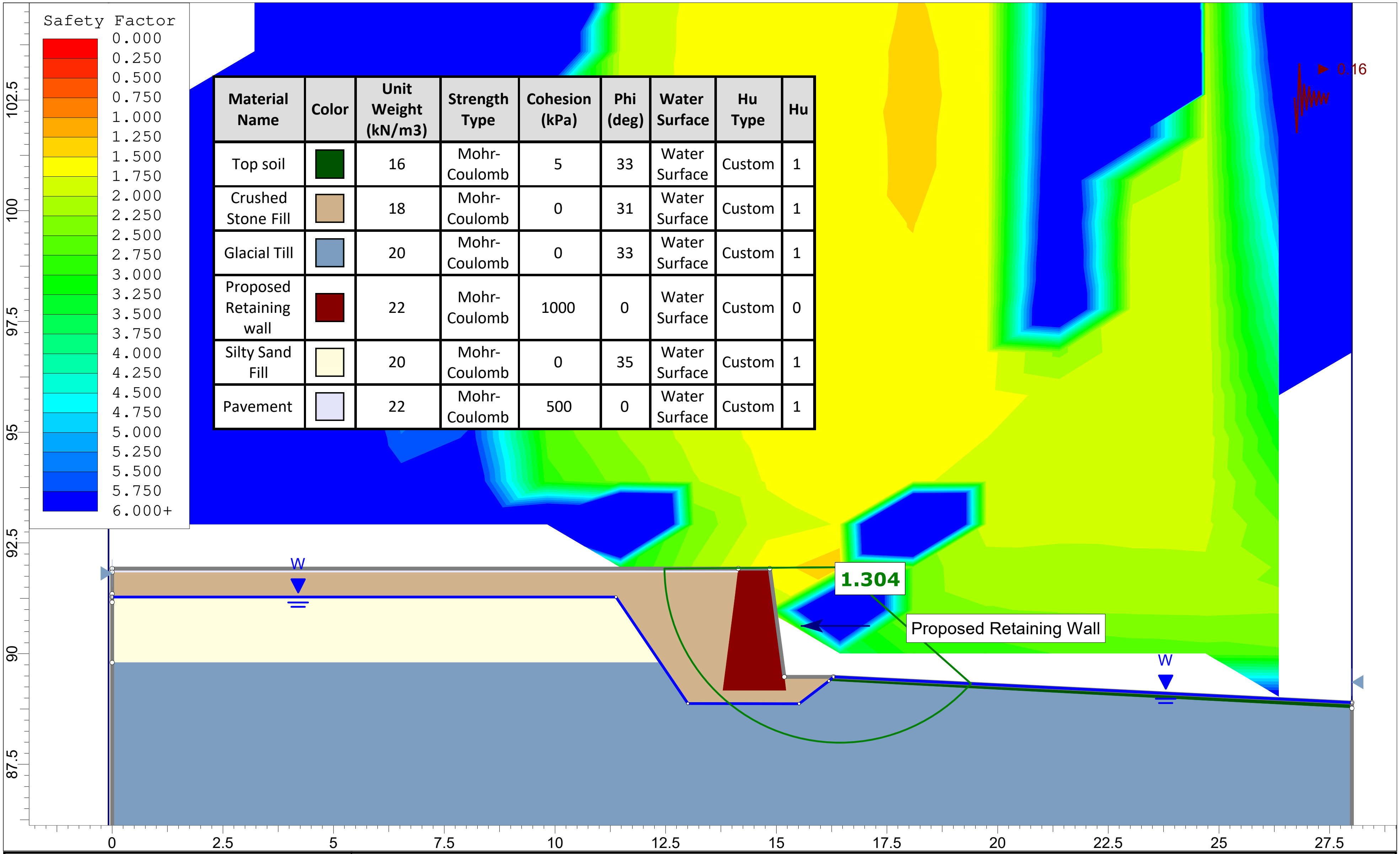
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Hu Type	Hu
Top soil	🟩	16	Mohr-Coulomb	5	33	Water Surface	Custom	1
Crushed Stone Fill	🟫	18	Mohr-Coulomb	0	31	Water Surface	Custom	1
Glacial Till	🟦	20	Mohr-Coulomb	0	33	Water Surface	Custom	1
Proposed Retaining wall	🟥	22	Mohr-Coulomb	1000	0	Water Surface	Custom	0
Silty Sand Fill	⬜	20	Mohr-Coulomb	0	35	Water Surface	Custom	1
Pavement	⬜	22	Mohr-Coulomb	500	0	Water Surface	Custom	1

PATERSON GROUP

SLIDEINTERPRET 9.025

Project	PG6832 - 2104 Roger Stevens Drive

Slope Section	C	*Scenario*	Proposed Static Loading
Drawn By	DR	*Company*	Ottawa Sivan Temple
Date	2025-01-27	*File Name*	Figure 2E

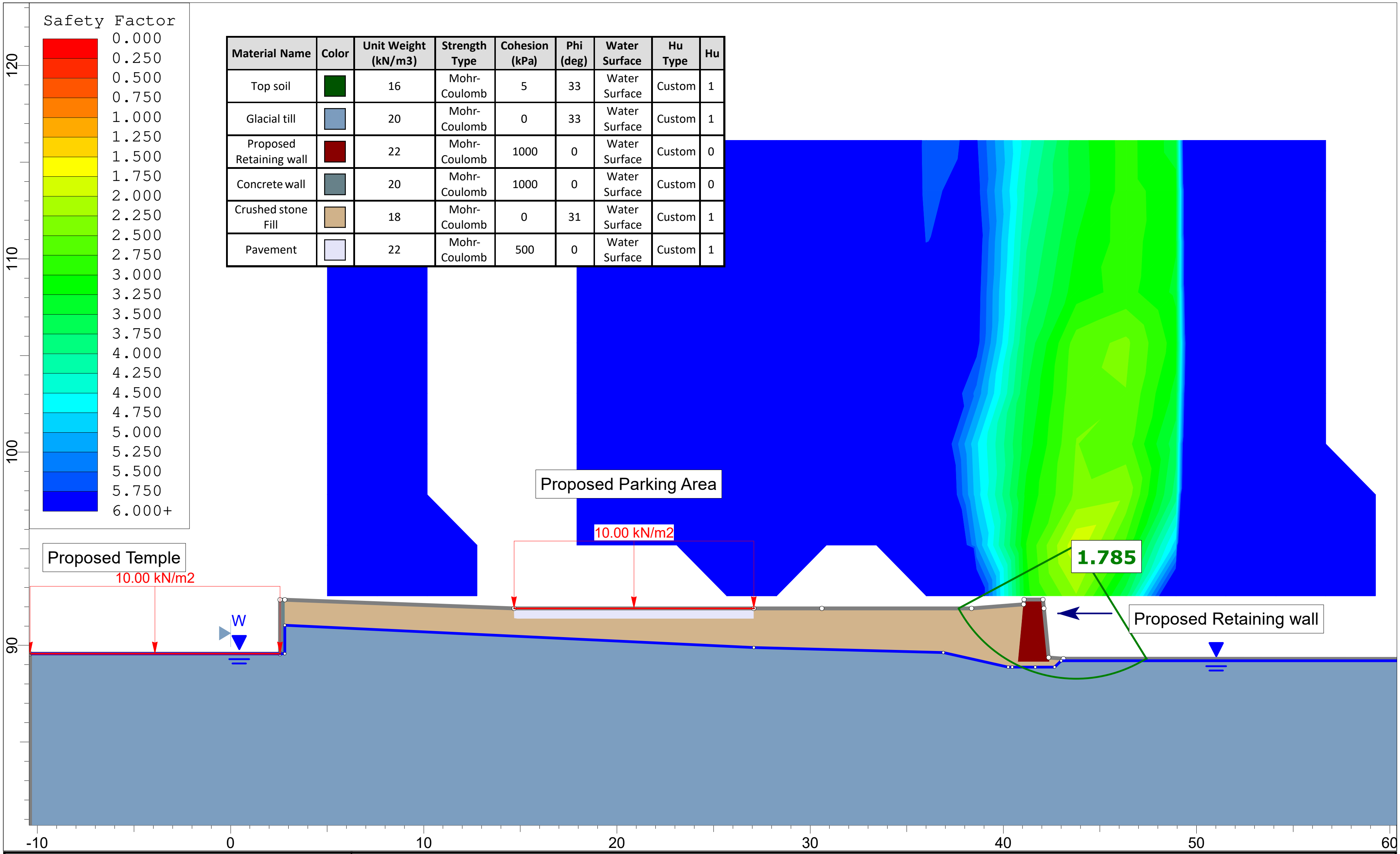


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Hu Type	Hu
Top soil	🟩	16	Mohr-Coulomb	5	33	Water Surface	Custom	1
Crushed Stone Fill	🟫	18	Mohr-Coulomb	0	31	Water Surface	Custom	1
Glacial Till	🟦	20	Mohr-Coulomb	0	33	Water Surface	Custom	1
Proposed Retaining wall	🟥	22	Mohr-Coulomb	1000	0	Water Surface	Custom	0
Silty Sand Fill	⬜	20	Mohr-Coulomb	0	35	Water Surface	Custom	1
Pavement	⬜	22	Mohr-Coulomb	500	0	Water Surface	Custom	1

PATERSON GROUP

SLIDEINTERPRET 9.025

Project	PG6832 - 2104 Roger Stevens Drive		
Slope Section	C	*Scenario*	Proposed Seismic Loading
Drawn By	DR	*Company*	Ottawa Sivan Temple
Date	2025-01-27	*File Name*	Figure 2F

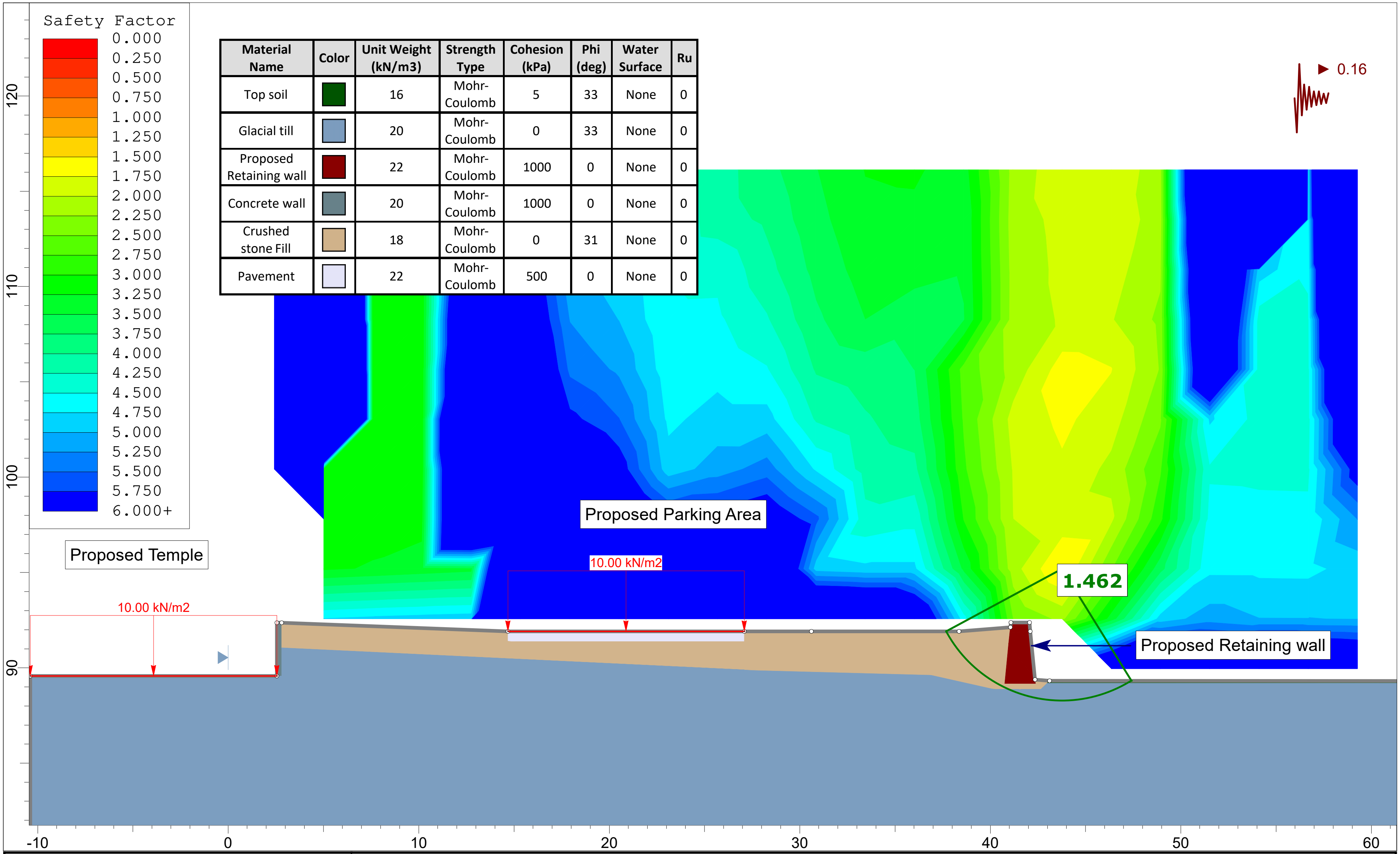


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Hu Type	Hu
Top soil	🟩	16	Mohr-Coulomb	5	33	Water Surface	Custom	1
Glacial till	🟦	20	Mohr-Coulomb	0	33	Water Surface	Custom	1
Proposed Retaining wall	🟥	22	Mohr-Coulomb	1000	0	Water Surface	Custom	0
Concrete wall	⬛	20	Mohr-Coulomb	1000	0	Water Surface	Custom	0
Crushed stone Fill	🟫	18	Mohr-Coulomb	0	31	Water Surface	Custom	1
Pavement	⬜	22	Mohr-Coulomb	500	0	Water Surface	Custom	1

PATERSON GROUP

SLIDEINTERPRET 9.025

Project	PG6832 - 2104 Roger Stevens Drive		
Slope Section	B	*Scenario*	Proposed Static Loading
Drawn By	DR	*Company*	Ottawa Sivan Temple
Date	2025-04-07	*File Name*	Figure 2C

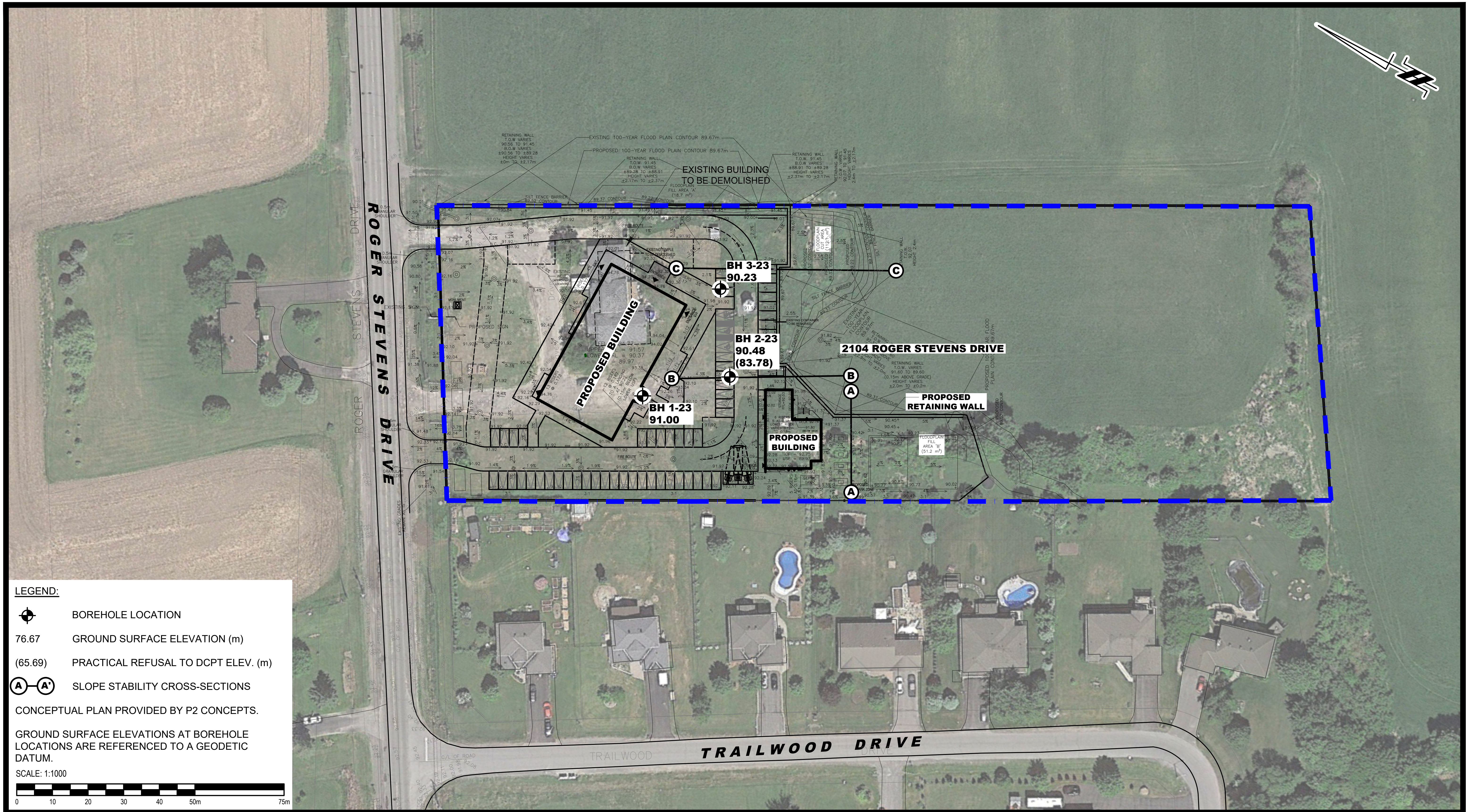


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Ru
Top soil	🟩	16	Mohr-Coulomb	5	33	None	0
Glacial till	🟦	20	Mohr-Coulomb	0	33	None	0
Proposed Retaining wall	🟥	22	Mohr-Coulomb	1000	0	None	0
Concrete wall	⬛	20	Mohr-Coulomb	1000	0	None	0
Crushed stone Fill	🟫	18	Mohr-Coulomb	0	31	None	0
Pavement	⬜	22	Mohr-Coulomb	500	0	None	0

PATERSON GROUP

SLIDEINTERPRET 9.025

Project	PG6832 - 2104 Roger Stevens Drive		
Slope Section	B	*Scenario*	Proposed Seismic Loading
Drawn By	DR	*Company*	Ottawa Sivan Temple
Date	2025-04-07	*File Name*	Figure 2D



LEGEND:

⌖	BOREHOLE LOCATION
76.67	GROUND SURFACE ELEVATION (m)
(65.69)	PRACTICAL REFUSAL TO DCPT ELEV. (m)
Ⓐ—Ⓐ'	SLOPE STABILITY CROSS-SECTIONS

CONCEPTUAL PLAN PROVIDED BY P2 CONCEPTS.

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:1000

0 10 20 30 40 50m 75m

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
3	AS PER REVISED CONCEPTUAL PLAN	07/04/2025	DR
2	ADDED SLOPE STABILITY CROSS SECTION C-C TO PLAN	27/01/2025	DR
1	AS PER REVISED CONCEPTUAL PLAN	24/06/2024	OM

OTTAWA SIVAN TEMPLE
GEOTECHNICAL INVESTIGATION
PROPOSED HINDU TEMPLE
2104 ROGER STEVENS DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1000

Date: 09/2023

Drawn by: NFRV

Report No.: PG6832-1

Checked by: OM

Dwg. No.: PG6832-1

Approved by: SD

Revision No.: 3

11x17

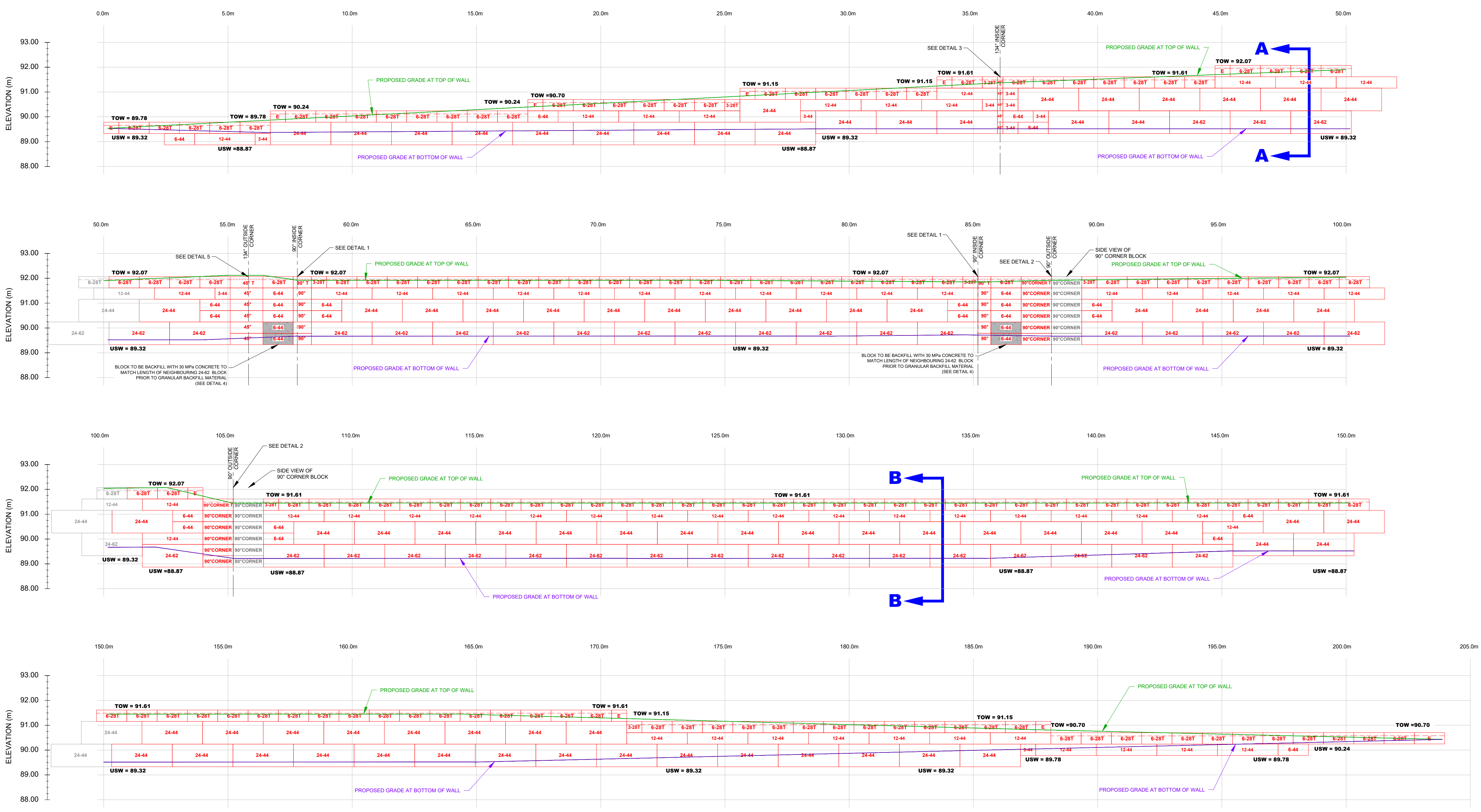
p:\autocad drawings\geotechnical\pg68xx\pg6832\pg6832-1 test hole location plan (rev.3).dwg

APPENDIX 3

STONE STRONG RETAINING WALL DESIGN SS1

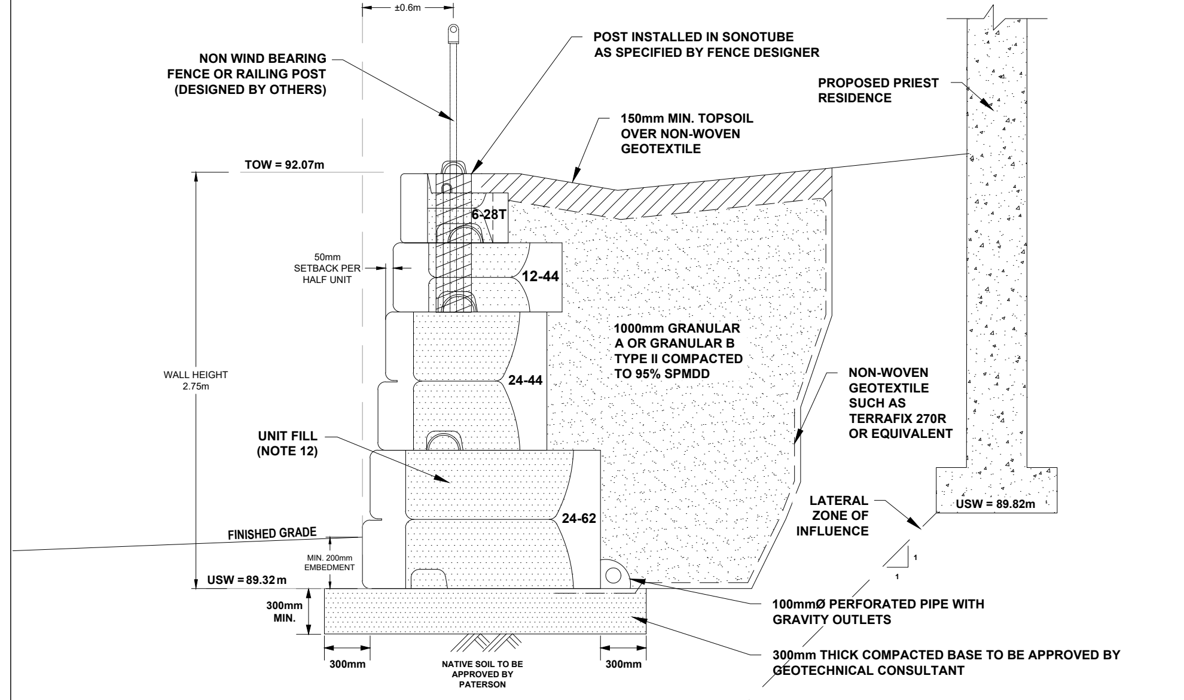
PROFILE VIEW (SS1):

SCALE 1:150



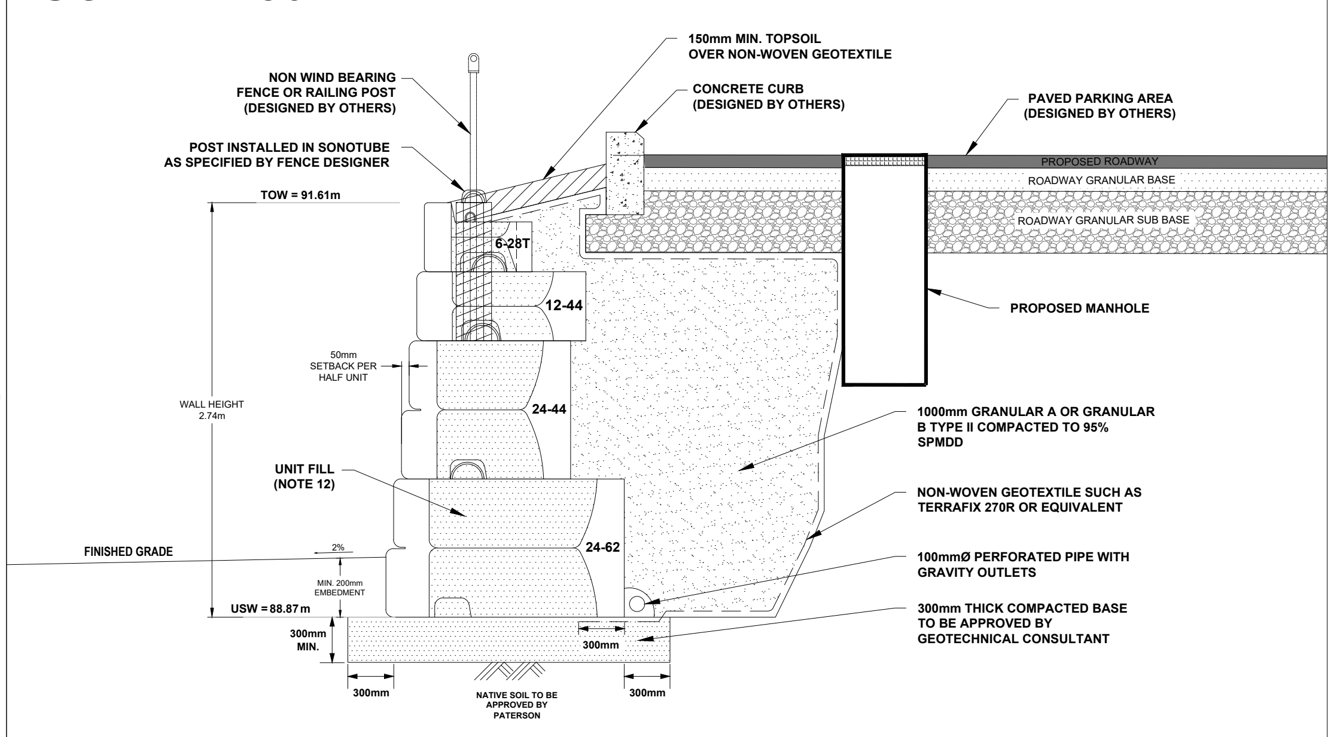
CROSS SECTION A-A:

SCALE 1:50



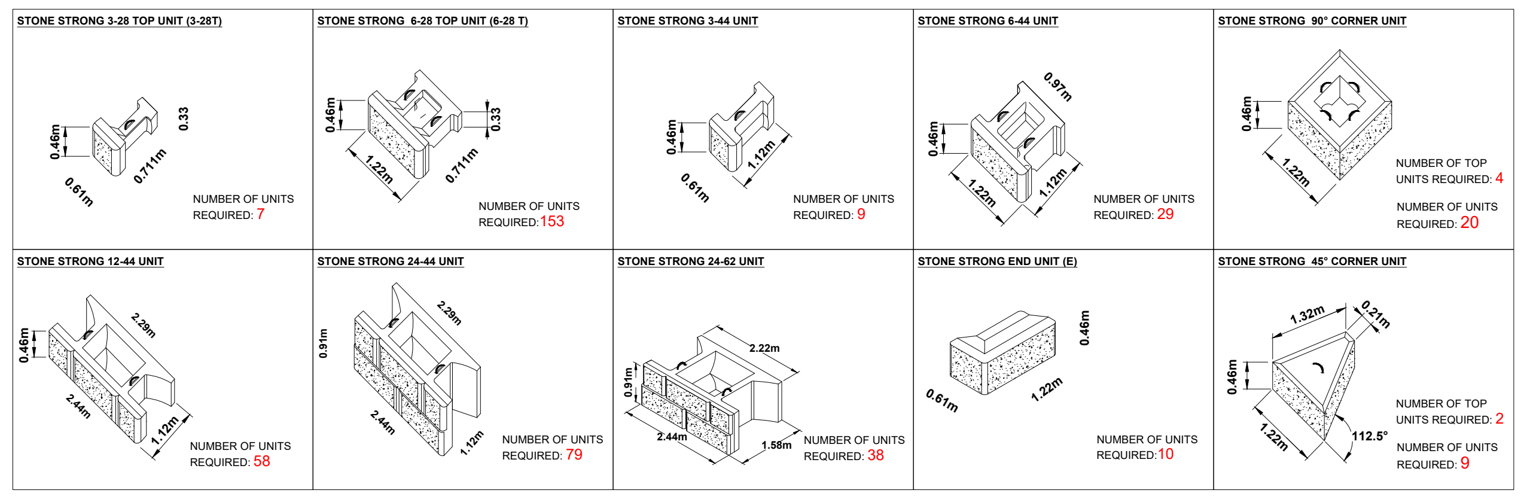
CROSS SECTION B-B:

SCALE 1:50



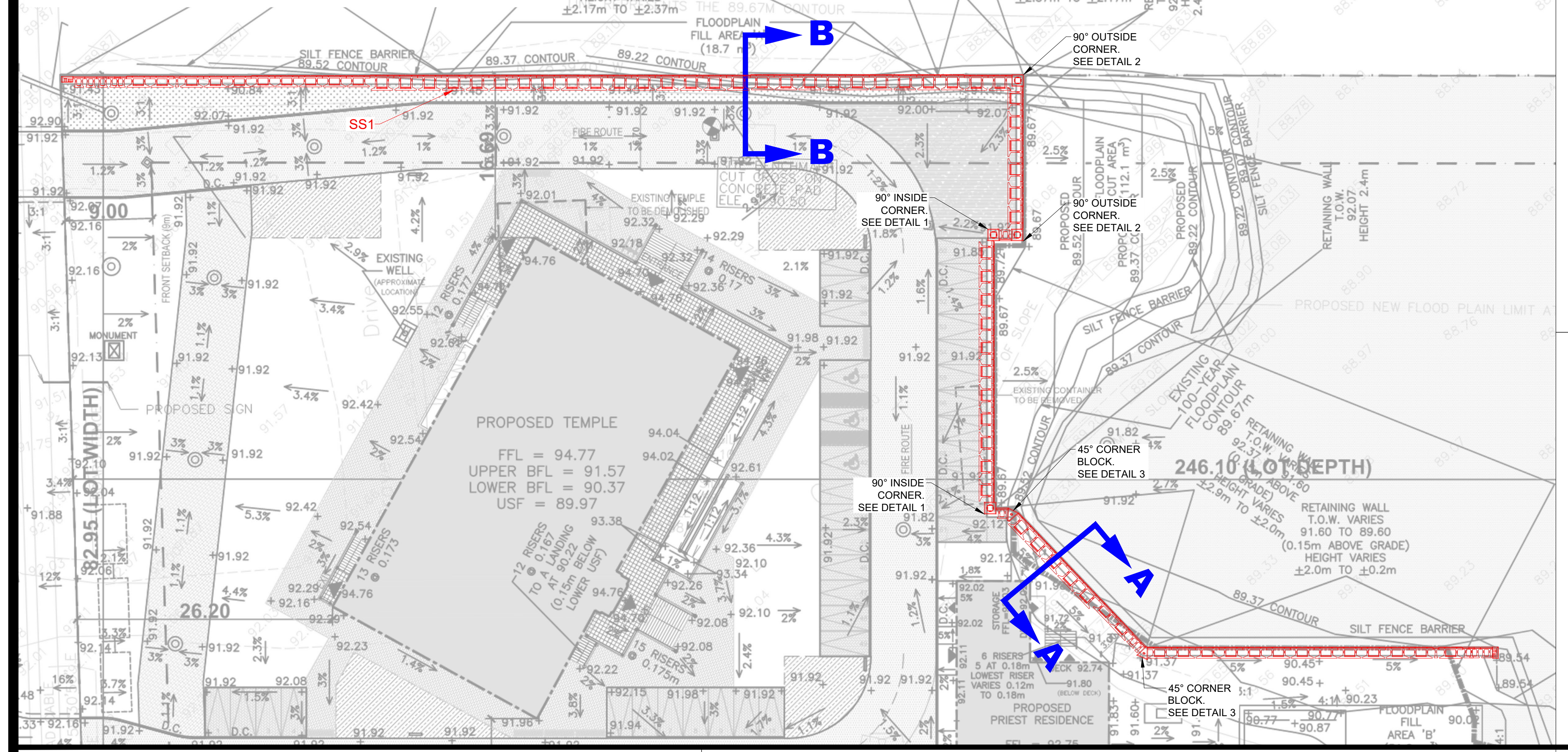
BLOCK COUNT:

N.T.S.



GRADING PLAN:

SCALE 1:500



DETAIL 1:

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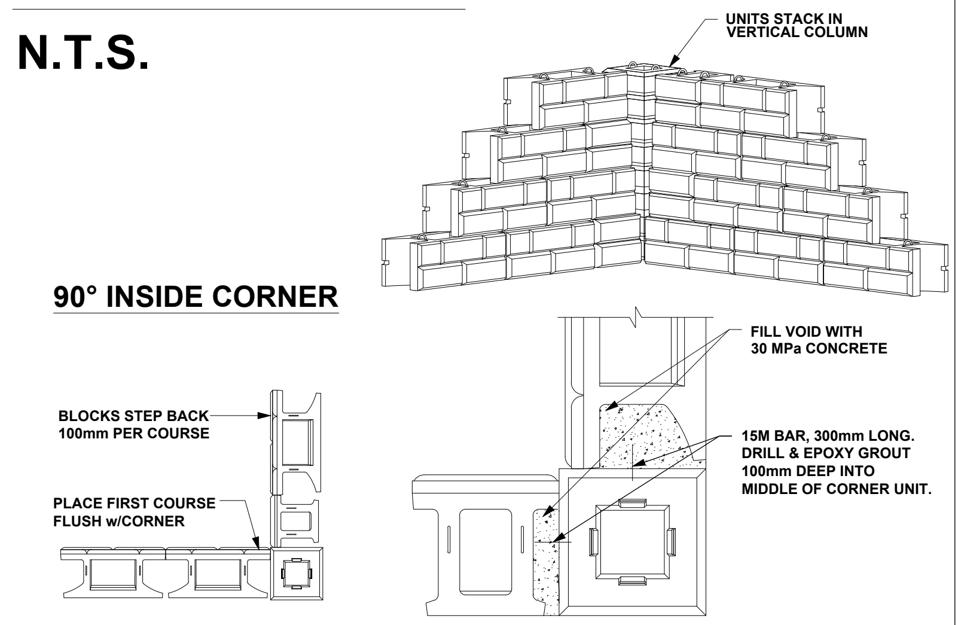

DETAIL 2:

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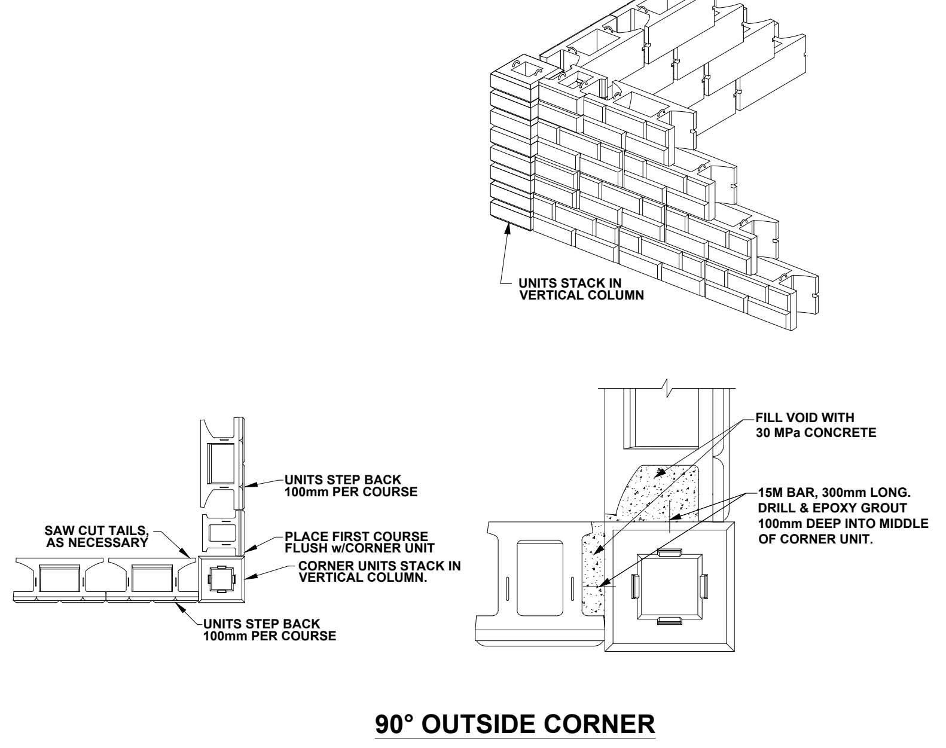
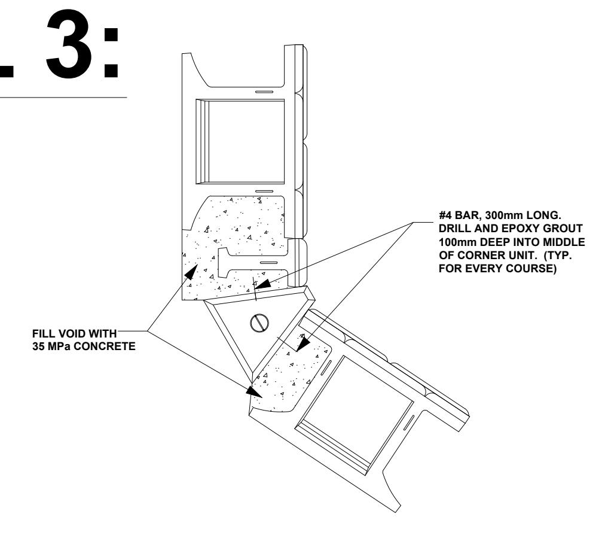


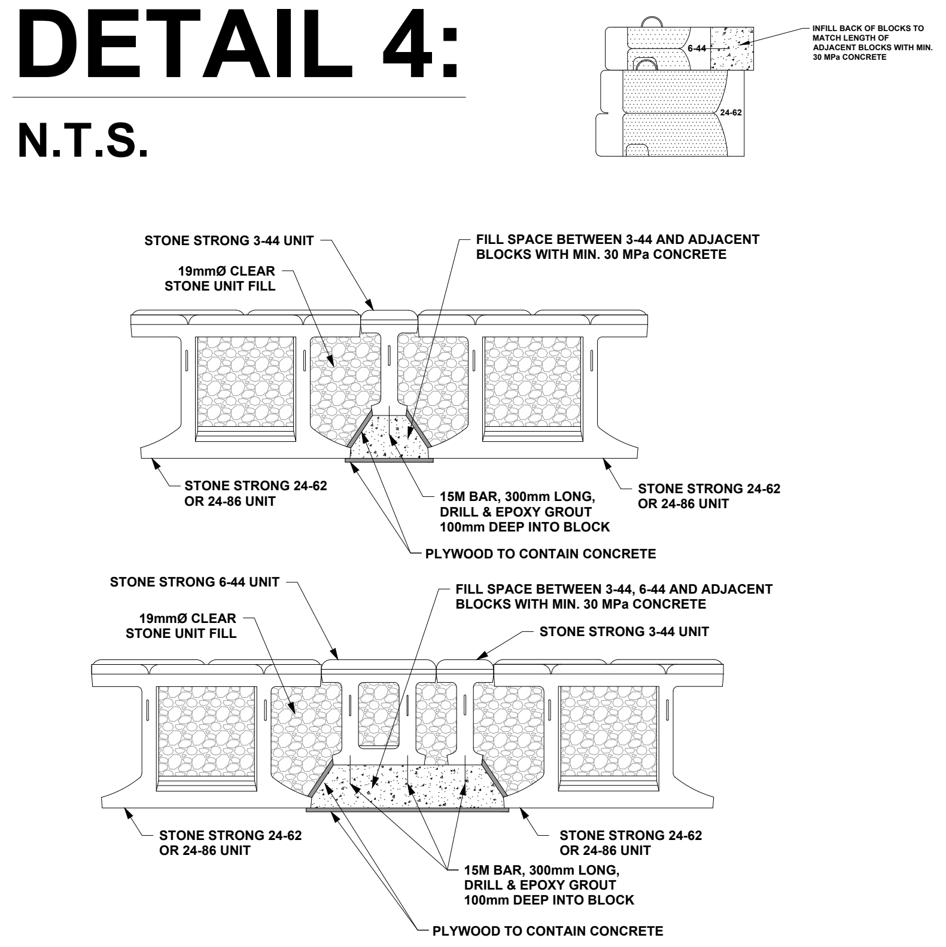
DETAIL 3:

N.T.S.



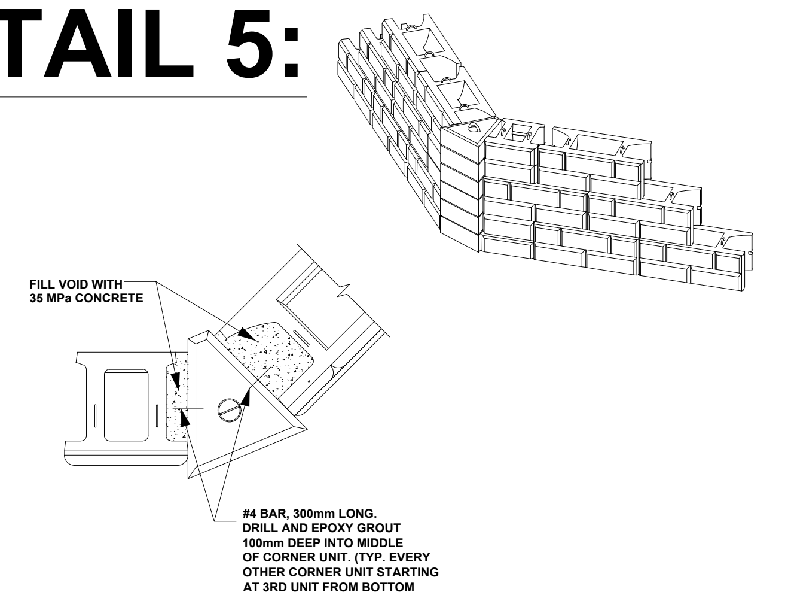
DETAIL 4:

N.T.S.



DETAIL 5:

N.T.S.



NOTES:

1. THE CONTRACTOR IS SOLELY RESPONSIBLE FOR UTILITY CLEARANCE AND CONSTRUCTION SITE SAFETY. PATERSON GROUP SHALL NOT BE RESPONSIBLE FOR MEANS OR METHODS OF CONSTRUCTION OR FOR SAFETY OF WORKERS OR OF THE PUBLIC. THE LOCATION OF EXISTING OR PROPOSED UTILITIES MUST BE VERIFIED PRIOR TO CONSTRUCTION. IT IS RECOMMENDED THAT UTILITIES BE OFFSET FROM THE WALL TO PREVENT ADDITIONAL LOADING ON ANY CONDUIT UNLESS ACCOUNTED FOR IN DESIGN OF THE UTILITY, AS WELL AS TO ENSURE FUTURE ACCESS TO THE UTILITY WITHOUT UNDERMINING THE WALL.

2. THIS DESIGN IS BASED ON THE FOLLOWING SOIL PROPERTIES:

PROPERTY	RETAINED FILL	FOUNDATION MEDIUM
SOIL TYPE	GRANULAR B TYPE II	SILTY SAND
FRICTION ANGLE - ϕ	36°	30°
UNIT WEIGHT - γ	22 kN/m³	19 kN/m³
COHESION - C	0 kPa	0 kPa

MATERIAL PROPERTIES ARE BASED ON SITE EVALUATION BY PATERSON GROUP, SEISMIC LOADING WAS EVALUATED ACCORDING TO THE CHBDC 2015 CSA-S6:19, WITH A PEAK GROUND ACCELERATION VALUE OF 0.274. A SURCHARGE OF 17 kPa WAS CONSIDERED FOR THE DESIGN.

3. THE DESIGN ELEVATIONS USED WERE BASED ON A GRADING PLAN DRAWN BY P2 CONCEPTS., JOB No. 0399, DRAWING No. SP01, REV.8 (DATE: APR 09-2025). THE WALL BASE DESIGN ASSUMES A BEARING RESISTANCE AT SLS OF 100 kPa ON SILTY SAND. PATERSON GROUP ENGINEER SHOULD OBSERVE THE BEARING CONDITIONS AND ADJUST THE THICKNESS OF THE GRANULAR BASE TO ACCOMMODATE THE SITE CONDITIONS, IF NECESSARY.

4. THE DESIGN HAS BEEN REVIEWED FOR THE STABILITY OF THE PRECAST MODULAR RETAINING WALL SYSTEM AND GLOBAL STABILITY WITH A FACTOR OF SAFETY OF 1.5 FOR STATIC CONDITIONS AND 1.1 UNDER SEISMIC CONDITIONS. WALL GEOMETRY AND GRADE ELEVATIONS ABOVE AND BELOW THE WALL SHOULD CONFORM WITH THE GRADING PLAN PROVIDED HEREIN. IF ACTUAL SITE GRADES VARY SIGNIFICANTLY FROM THOSE SHOWN OR IF THE BACK SLOPE DOES NOT CONFORM, INSTALLATION SHALL NOT PROCEED UNTIL THE DESIGN IS VERIFIED OR MODIFIED IN THE APPLICABLE AREA.

5. HORIZONTAL LAYOUT DIMENSIONS ARE MEASURED ALONG THE FACE OF THE WALL.

6. PRECAST UNITS SHALL BE STONE STRONG RETAINING WALL UNITS MANUFACTURED UNDER LICENSE FROM STONE STRONG SYSTEMS.

7. THE WALL BASE SHALL CONSIST OF A MINIMUM OF 300mm OF OPSS GRANULAR A OR GRANULAR B TYPE II ON NATIVE SOIL. A MINIMUM OF 200mm OF GRANULAR MATERIAL CAN BE USED WHERE BEDROCK IS ENCOUNTERED ALONG THE BASE OF THE WALL. THE BASE SHALL BE COMPACTED AS TO PROVIDE A LEVEL AND HARD SURFACE ON WHICH TO PLACE THE FIRST COURSE OF UNITS. GRANULAR BASE MATERIAL SHALL BE COMPACTED TO A MINIMUM 98% OF STANDARD PROCTOR MAXIMUM DRY DENSITY (SPMDD). THE BASE SHALL BE SMOOTHED TO ENSURE COMPLETE CONTACT OF RETAINING WALL UNITS WITH BASE. THE SURFACE OF GRANULAR BASE MAY BE DRESSED WITH FINER AGGREGATE TO AID LEVELING. ENSURE GRADATION OF DRESSING MATERIAL IS SUCH AS TO PRECLUDE LOSS OF FINES INTO BASE. THE THICKNESS OF DRESSING LAYER SHOULD NOT EXCEED 3 TIMES THE MAXIMUM PARTICLE SIZE USED. THE CONTRACTOR MAY SUBSTITUTE CONCRETE WITH A MINIMUM 28-DAY COMPRESSIVE STRENGTH OF 20MPa AND AIR ENTRAINMENT FOR THE GRANULAR BASE MATERIAL.

8. INSTALL 100mm DIAMETER PERFORATED PIPE DRAIN WRAPPED IN GEOTEXTILE BEHIND HEEL OF WALL (OR ALTERNATIVELY UNDER LOWER COURSE OF WALL). PROVIDE CLEAR STONE SURROUNDING THE DRAIN TO PROTECT PIPE FROM CLOGGING AND DAMAGE. PROVIDE OUTLETS THROUGH WALL BASE LAYER AT LOW AREAS AND CORNERS. IF OUTLET NOT AVAILABLE, RAISE DRAINAGE PIPE TO FINISHED GRADE AND DRAIN AT THE ENDS OF THE WALL AND OUTLET THROUGH THE FACE OF THE WALL (WITH RODENT GUARD) NO FURTHER APART THAN 30m CENTRES.

9. PATERSON SHOULD REVIEW THE BEARING SURFACE DURING THE CONSTRUCTION. IF FILL MATERIAL IS ENCOUNTERED, A REVIEW OF THE BEARING CONDITIONS SHOULD BE CONDUCTED BY PATERSON PERSONNEL PRIOR TO THE PLACEMENT OF THE GRANULAR BASE. PROOF ROLLING OF THE BEARING SURFACE WILL ALSO BE REQUIRED UNDER THE SUPERVISION OF PATERSON PERSONNEL TO REHABILITATE THE BEARING MEDIUM AND TO ACHIEVE THE DESIGN BEARING CAPACITIES. A BIAXIAL GEOGRID SUCH AS TBX 2500 MAY BE REQUIRED TO BE PLACED ON THE BEARING SURFACE AND WRAP AROUND THE EDGES OF THE GRANULAR BASE. ALTERNATIVELY, FILL MATERIAL CAN BE REMOVED AND REPLACED WITH ENGINEERED FILL SUCH AS GRANULAR B TYPE II PLACED IN MIN. 300mm THICK LIFTS COMPACTED TO A MINIMUM 98% OF THE MATERIAL'S SPMDD EXTENDING TO THE UNDERLYING NATIVE SOIL. A REVIEW OF THE BEARING SURFACE SHOULD BE CONDUCTED ON SITE AT THE TIME OF EXCAVATION.

10. WALL IS DESIGNED FOR A MINIMUM OF 300mm TOE EMBEDMENT WITH A MINIMUM HORIZONTAL LEDGE OF 300mm BEYOND THE FACE AND REAR OF BASE BLOCK. WHERE GRANULAR BEDDING WILL NOT BE SUFFICIENT, THE USE OF CONCRETE BEDDING MAY BE REQUIRED. EXTRA PRECAUTIONS MUST BE TAKEN TO PROVIDE TOE EMBEDMENT IN AREAS WHERE BASE OF WALL STEPS.

11. THE RETAINING WALL IS A BATTERED WALL. ALIGNMENT OF THE BOTTOM WALL UNIT COURSE SHOULD BE PLANNED TO CONSIDER THAT A NOMINAL 50 mm AUTOMATIC SETBACK WILL OCCUR WITH EACH 0.45 m HIGH UNIT.

12. UNIT FILL SHALL BE A CLEAN, COURSE GRANULAR MATERIAL. UNIT FILL SHALL BE 19mmØ CLEAR STONE MEETING THE SATISFACTION OF THE GEOTECHNICAL ENGINEER. UNIT FILL SHALL FILL CAVITIES WITHIN AND BETWEEN THE UNITS AND MAY EXTEND BEHIND THE FACING UNITS FOR THE CONTRACTOR'S CONVENIENCE.

13. BACKFILL MATERIAL SHALL BE APPROVED BY THE SITE GEOTECHNICAL ENGINEER PRIOR TO USE AND SHOULD CONSIST OF OPSS GRANULAR B TYPE II BUFFER OF 1000mm (AS SHOWN) WIDTH. ALL FILL WITHIN A 1H:1V ZONE UP AND BACK FROM THE HEEL SHOULD ALSO BE COMPACTED. BACKFILL SHALL BE PLACED IN MAXIMUM 300 mm LOOSE LIFTS AND COMPACTED TO A MINIMUM OF 95% OF THE MATERIAL'S SPMDD. MOISTURE CONTENT SHOULD BE CONTROLLED AND MAINTAINED WITHIN -3 TO +4 PERCENT OF OPTIMUM. ONLY WHERE WALL PASSES DIRECTLY AGAINST A FENCE POST SHOULD CLEAR STONE BE USED.

14. ENSURE EACH COARSE IS COMPLETELY FILLED AND BACKFILL IS PLACED TO THE SAME LEVEL PRIOR TO PROCEEDING TO THE NEXT COURSE. ENSURE ADJACENT UNITS ARE IN CONTACT SO THAT UNIT FILL MAY NOT ESCAPE THROUGH THE JOINT BETWEEN UNITS. STRIPS OF GEOTEXTILE CAN BE PLACED ON THE INSIDE OF THE BLOCK AT THE JOINTS TO RETAIN FILL. GAPS GREATER THAN 6 mm BETWEEN UNITS (AT THE FACE) SHALL NOT BE ALLOWED. AT THE INTERSECTIONS WITH STRUCTURES, CUT UNITS TO OBTAIN A NEAT FIT. PULL BLOCK UNITS FORWARD TO ENGAGE THE ALIGNMENT LOOPS ON THE UNIT BELOW BEFORE INFILLING IN ALL CASES.

15. MAINTAIN TEMPORARY GRADES TO DIVERT SURFACE WATER AWAY FROM THE RETAINING WALL EXCAVATION. SLOPE FINAL BACKFILL TO PROVIDE POSITIVE DRAINAGE AND TO ELIMINATE PONDING.

16. IF WINTER CONSTRUCTION IS CONSIDERED, HEAT MUST BE MAINTAINED WHEN THE BASE IS EXPOSED. THE WALL BASE MUST BE COVERED WITH INSULATION TARPS TO MAINTAIN HEAT AND PROTECT THE BASE FROM POTENTIAL FROSTHEAVE. ONCE THE BASE IS BACKFILLED, THE TOP OF WALL MUST BE COVERED WITH INSULATION TARPS OVERNIGHT UNTIL THE WALL CONSTRUCTION IS COMPLETED.

17. THE GEOTECHNICAL CONSULTANT SHOULD BE NOTIFIED AT THE BEGINNING OF THE WALL CONSTRUCTION TO COMPLETE PERIODIC INSPECTIONS AND PROVIDE GEOTECHNICAL RECOMMENDATIONS AS THE WALL CONSTRUCTION PROGRESSES.

18. DURING THE CONSTRUCTION OF THE RETAINING WALL, THE CONTRACTOR MUST ENSURE THAT A SAFE SLOPE IS PROVIDED BEHIND THE RETAINING WALL. THE GEOTECHNICAL CONSULTANT SHOULD COMPLETE PERIODIC INSPECTIONS TO ENSURE A PROPER SLOPE IS PROVIDED AS PER THE SITE GEOTECHNICAL RECOMMENDATIONS.

19. ANY INADEQUATE PERFORMING SUBGRADE SHOULD BE SUB-EXCAVATED AND REPLACED WITH OPSS GRANULAR B TYPE II, COMPACTED TO 98% OF THE MATERIALS SPMDD.

20. ANY CUTTING OF BLOCKS TO SUIT SITE CONDITIONS OR WALL DESIGN WILL BE THE RESPONSIBILITY OF THE CONTRACTOR. REMOVAL/CUTTING OF LIFTING LOOPS ON THE FINAL ROW OF BLOCKS WILL BE THE RESPONSIBILITY OF THE CONTRACTOR.

21. LEVELING OF THE BASE COURSE BLOCKS IS CRITICAL TO PROPER CONSTRUCTION OF THE WALL. THE USE OF SHIMS TO LEVEL THE BLOCKS IS NOT PERMITTED UNLESS REVIEWED ON SITE PRIOR TO THEIR USE. SHOULD SHIMS BE APPROVED FOR USE BY PATERSON, THE SPECIFICATIONS AND DETAILS OF THE SHIMS USED TO SUPPORT THE BLOCKS SHOULD BE PROVIDED TO PATERSON'S DESIGNER TO CONFIRM THAT NO LONG-TERM ISSUES MAY OCCUR AS A RESULT OF THE USE OF NON-SUITABLE SHIMS IN RELATION TO THE LOAD EXPECTED FROM THE BLOCKS ABOVE.

22. THE DESIGN ASSUMES THE FOLLOWING: THE MAXIMUM GROUNDWATER ELEVATION IS BELOW THE BASE OF THE WALL, THERE WILL BE NO HYDROSTATIC PRESSURE WITHIN OR BEHIND THE WALL, THE SURROUNDING STRUCTURES WILL NOT EXERT ANY ADDITIONAL LOADING ON THE WALL, THERE ARE NO STRUCTURES (UTILITIES SUCH AS GAS/WATER MAINS, STORM SEWERS, ELECTRICAL/COMMUNICATIONS CABLES, ETC) TO BE PLACED WITHIN OR BELOW THE REINFORCED FILL DURING OR AFTER CONSTRUCTION. ALTERNATIVELY, SEE DETAILS.

23. THE CONTRACTOR SHOULD REFER TO THE INSTALLATION MANUAL PROVIDED FOR THE RETAINING WALL BLOCK TYPE PROVIDED HEREIN FOR ADDITIONAL DETAILS ON ACCEPTABLE INSTALLATION PRACTICES.

24. RETAINING WALL CONSTRUCTION SHOULD BEGIN AT LOW POINTS, CORNERS OF THE WALL, OR KNOWN PROVIDED WORKING POINTS TO ENSURE WALL DIMENSIONS ARE FOLLOWED. DIMENSIONS PROVIDED MIGHT REQUIRE FIELD CUTTING TO ADJUST FOR FIELD CONDITIONS BASED ON BLOCK TOLERANCES.

25. SETBACK FROM PROPERTY LINE SHOULD CONSIDER COURSE SETBACK (WALL BATTER) BASED ON THE POSITION OF THE LOWER COURSE. EACH SUBSEQUENT COURSE OF 0.9m WILL HAVE A SUPPLEMENTAL SETBACK OF 100mm.

24. THE FENCING DETAIL PROVIDED IN THE CURRENT DESIGN ASSUMES NON-WIND RESISTING FENCES. IF DIFFERENT FENCING SUCH AS PRIVACY, NOISE BARRIER AND/OR VEHICLE GUIDERAILS ARE PROPOSED, PATERSON MUST BE CONTACTED IMMEDIATELY PRIOR TO CONSTRUCTION AS ADDITIONAL RECOMMENDATIONS AND POSSIBLE CHANGES MAY APPLY TO THE SELECTED RETAINING WALL BLOCKS AT THESE SPECIFIC AREAS.

ISSUED FOR REVIEW

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NO.	REVISIONS	DATE	INITIAL
1	UPDATED GRADING PLAN	11/04/2025	SD

P-SQUARED CONCEPTS INC.

PROPOSED HINDU TEMPLE
2014 ROGER STEVENS DRIVE

NORTH GOWER, ONTARIO

Title:
STONE STRONG RETAINING WALL DESIGN SS1

Stamp:
11/04/2025
S. S. DENNIS
100519516
Scott Dennis

Stamp:

Scale: AS SHOWN
Drawn by: MLB
Checked by: CT
Approved by: SD
Date: 01/2025

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