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A REPORT TO
1407 LAKESHORE DEVELOPMENTS INC.

A GEOTECHNICAL INVESTIGATION FOR
PROPOSED BUILDING

1041 LAKESHORE ROAD EAST

CITY OF MISSISSAUGA

REFERENCE NO. 2108-S066

FEBRUARY 2022

DISTRIBUTION

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ENCLOSURES

1.0 INTRODUCTION

In accordance with written authorization from Dr. Vikas Soota, President of 1407 Lakeshore Developments Inc., dated December 7, 2021, a geotechnical investigation was carried out at 1041 Lakeshore Road East, in the City of Mississauga.

The purpose of the investigation was to reveal the subsurface conditions and determine the engineering properties of the disclosed soils for the design and construction of the proposed Building. The geotechnical findings and resulting recommendations are presented in this Report.

2.0 SITE AND PROJECT DESCRIPTION

The City of Mississauga is situated on Halton-Peel till plain where drift extends onto a shale bedrock of Georgian Bay formation at shallow to moderate depths. In places, the drift has been eroded by the glacial lake (Peel Ponding) and filled with lacustrine sand, silt and clay.

The subject site, approximately 0.32 hectare in size, is located on the north side of Lakeshore Road East, between Strathy Avenue and Ogden Avenue in the City of Mississauga. It is fenced with hoarding board or chain-linked fence. The site was excavated to a depth of approximately 2.8 m from the ground level and was shored with soldier piles and lagging boards, with raker supports. An access ramp was built at the east of the excavation.

The architectural plan prepared by Raw Design Inc. indicates that the proposed development will consist of a 10-storey mixed-use building with ground floor retail units. The building will be provided with 2-underground parking levels extending to the limits of the property.

3.0 FIELD WORK

The original field work, consisting of two (2) sampled boreholes, extending to refusal depths of 0.5 m and 0.6 m, were completed on September 7, 2021. Rock coring was performed at the boreholes, extending to depths of 10.1 m and 10.4 m, between February 2 and 4, 2022. In addition, monitoring wells (MW1, MW2 and MW3), 50 mm in diameter, were installed within the property to facilitate a hydrogeological assessment. The depth and details of the monitoring wells are shown on the corresponding Borehole Logs. The locations of the boreholes and monitoring wells are illustrated on Drawing No. 1.

The boreholes were advanced at intervals to the sampling depths by track-mounted, continuous-flight power-auger machines equipped for soil sampling. Standard Penetration Tests, using the procedures described on the enclosed "List of Abbreviations and Terms", were performed at the sampling depths. The test results are recorded as the Standard Penetration Resistance (or 'N' values) of the subsoil. The relative density of the non-cohesive strata and the consistency of the cohesive strata are inferred from the 'N' values. Split-spoon samples were recovered for soil classification and laboratory testing.

'HQ' size (63.5 mm core diameter) rock coring was carried out in the vicinity of Boreholes 1 and 2 to assess the quality and soundness of the encountered shale bedrock. The quality of the rock has been assessed by applying the 'Rock Quality Designation' (RQD) classification, considering the total length of the recovered pieces 10 cm or longer against the length of the core run. The results are expressed as a percentage and are recorded on the Borehole Logs.

The field work was supervised and the findings were recorded by Geotechnical Technicians. The ground elevation at each borehole location was determined using a handheld Global Navigation Satellite System (Trimble Geoexplorer 6000 series) equipment.

4.0 SUBSURFACE CONDITIONS

The boreholes were completed in the excavated area within the property, on shale bedrock.

Detailed descriptions of the encountered subsurface conditions at the boreholes are presented on the Borehole Logs, comprising Figures 1 and 2. The engineering properties of the disclosed soils are discussed herein.

4.1 Shale Bedrock (All Boreholes)

Shale bedrock was encountered at the ground surface within the excavated area, which is approximately 2.8 m below the street level.

The shale is grey in colour, indicating that it is of Georgian Bay Formation; it is a laminated, sedimentary, moderately soft rock composed predominantly of clay material, and it is interbedded with about 20% sandstone and limy shale bands. The shale is susceptible to disintegration and swelling upon exposure to air and water, with subsequent reversion to a clay soil, but the laminated limy and sandy layers would remain as rock slabs.

Rock coring was carried out in the shale bedrock starting at depths ranging from 1.1 m and 0.9 m, and terminating at depths of 10.1 m and 10.4 m, at Boreholes 1 and 2, respectively. The recovery of 'HQ' size rock cores ranges from 52% to 100% while the RQD values range from 31% to 95%, indicating the quality of the shale varies from poor to good.

Since the property has already been excavated, it is anticipated that the excavation of bedrock for the lower parking level will likely require the aid of pneumatic hammering.

In sound shale excavation, slight lateral displacement of the excavation walls is often experienced. This is due to the release of residual stress stored in the bedrock mantle and the swelling characteristics of the rock.

5.0 GROUNDWATER CONDITION

Borehole 1 and 2 remained dry upon completion in September 2021. Based on field observations of the excavation at the time, no sign of groundwater seepage was evident from the walls of shoring. Dewatering equipment is not evident within the property and no continuous dewatering operation was in place in the excavation.

The monitoring wells were checked for the presence of groundwater during installation on February 1, 2022, water seepage from melting snow and ice was detected at MW1. The water level was recorded at a depth of 2.4 m, or El. 80.1 m at MW2, it was likely due to infiltrated melting snow and ice. MW3 remained dry upon completion of the well installation.

The groundwater levels in the monitoring wells were also recorded on February 24, 2022. The results are summarized in Table 1:

Table 1 - Groundwater Record in Monitoring Wells

Borehole/ Monitoring Well No.	Ground Elevation (m)	February 24, 2022	
		Depth (m)	Elevation (m)
MW1	82.6	1.5	84.1
MW2	82.6	Well Frozen	
MW3	82.6	3.7	78.9

Groundwater was recorded in the monitoring wells at a depth of 1.5 m and 3.7 m, or El. 84.1 m and 78.9 m at MW1 and MW3, respectively. MW2 was frozen and not accessible at the time of the site visit. The recorded water level represents perched groundwater in the area. Continuous groundwater is not anticipated within the site.

Additional groundwater records in the monitoring wells will be presented in the hydrogeological report under a separate cover.

6.0 DISCUSSION AND RECOMMENDATIONS

The investigation has disclosed that within the excavated area, approximately 2.8 m from the street level, the ground surface of the excavated area generally consists of shale bedrock.

Boreholes 1 and 2 remained dry upon completion of the field work in September 2021. Groundwater was recorded in the monitoring wells at a depth of 1.5 m to 3.7 m, or between El. 84.1 m and 78.9 m. The recorded water level represents perched groundwater in the area. Continuous groundwater is not anticipated within the site.

The architectural plan indicates that the proposed development will consist of a 10-storey building with ground floor retail units and two underground parking levels, having the finished floor elevation of the lower parking level at approximately 6.3 m below the ground surface. The geotechnical findings which warrant special consideration are presented below:

1. The foundation details of the adjacent structures must be investigated and incorporated into the excavation, design and construction of the underground structure. A pre-construction survey and a monitoring program should be carried out for all adjacent structures in order to verify any potential future claims.
2. Bulk excavation for the underground structure is anticipated to extend to 7 to 8 m below the street level, or 4 to 5 m into below the excavated level into the shale bedrock, which is suitable to support the proposed development.
3. In conventional design and construction, the underground structure should be provided with a drainage system connecting into the municipal sewer. If the municipality does not allow the removal of groundwater into the sewer, the subsurface water has to be discharged into a cistern in the building.
4. Excavation should be carried out in accordance with Ontario Regulation 213/91. The design of the existing shoring system is not known. The existing piles must be extended beyond the base of the founding elevation prior to the excavation for the lower parking level, with lagging board extended to cover the overburden and the weathered shale.

The recommendations appropriate for the project described in Section 2.0 are presented herein. One must be aware that the subsurface conditions may vary between boreholes. Should subsurface variances become apparent during construction, a geotechnical engineer must be consulted to determine whether the following recommendations require revision.

6.1 Site Preparation

The foundation details of the adjacent structures must be investigated and incorporated into the excavation, design and construction of the underground structure. A pre-construction survey and a monitoring program should be carried out for the adjacent structures in order to verify any potential future claims.

6.2 Foundation

The site will be developed with a 10-storey building with 2 underground parking levels extending near the limit of the property. Bulk excavation will extend an additional 4 to 5 m into the surface of the shale bedrock from its current level.

In conventional design and construction, with an effective drainage system in the underground structure, the proposed development can be constructed on conventional spread and strip footings. The design bearing pressures for the design of footings are presented below:

- Maximum Bearing Pressure at Serviceability Limit State (SLS) = 1000 kPa
- Factored Ultimate Bearing Pressure at Ultimate Limit State (ULS) = 1500 kPa

The total and differential settlements of foundations in shale bedrock will be negligible.

The building foundation subgrade should be inspected by a geotechnical engineer or a senior geotechnical technician to ensure that the revealed conditions are compatible with the foundation design requirements.

The shale bedrock will slake if left exposed for any length of time. It is, therefore, important that the footings are poured with concrete immediately on excavation and inspection. Alternatively, the footings should be skim coated with lean mix concrete, 8 to 10 cm in thickness to minimize deterioration of rock at the bearing surface.

The shale bedrock is non frost susceptible; there is no minimum depth requirement for foundation founded on sound shale. However, the foundation exposed to freezing temperature must be covered with 1.2 m of earth fill for frost protection.

The building foundations should meet the requirements specified in the latest Ontario Building Code. The structure should be designed to resist an earthquake force using Site Classification ‘C’ (very dense soil and soft rock).

6.3 Underground Parking

The underground structure should be designed to sustain a lateral earth pressure calculated using the soil parameters stated in Section 6.8. Any applicable surcharge loads adjacent to the proposed building and hydrostatic pressure, if any, must also be considered in the design of the underground structure.

A compressible material, such as sprayed foam, 80 to 100 mm in thickness, should be placed between the concrete wall and the bedrock. This is to allow lateral expansion or movement of the rock face without causing damage to the foundation walls.

In conventional design, the perimeter walls of the underground garage should be dampproofed and provided with a perimeter subdrain encased in a fabric filter at the wall base. Prefabricated drainage board, such as Miradrain 6000 or equivalent, must be provided between the shoring wall or rock face and the cast-in-place foundation wall, as shown on Drawing No. 3.

The lower parking level slab should be constructed on a 200 mm thick granular bedding, consisting of 19-mm Crusher-Run Limestone (CRL), or equivalent, compacted to at least 98% Standard Proctor Dry Density (SPDD).

If the Municipality does not allow any discharge of subsurface water into the sewer system, a separate storage cistern should be provided and can be used for irrigation purposes or discharge overtime.

The elevator pit, which normally extends a few metres below the floor level, should be designed as a submerged 'tank' structure with waterproofed pit walls and pit floor.

The ground around the buildings must be graded to direct water away from the structures.

6.4 Underground Services

The subgrade for the underground services should consist of properly compacted inorganic earth fill, sound natural soils or bedrock. A Class 'B' bedding, consisting of compacted 19-mm CRL, or equivalent, compacted to at least 98% SPDD.

In order to prevent pipe floatation when the sewer trench is deluged with water, a soil cover of at least equal two times the diameter of the pipe should be in place at all times after completion of the pipe installation.

The pipe joints into manholes or catch basins should be leak-proof or wrapped with a waterproof membrane. Openings to subdrains and catch basins should be shielded by a fabric filter to prevent blockage by silting.

6.5 Trench Backfilling

The backfill in service trenches or beside foundation walls or below transformer pads should be compacted to at least 95% SPDD. In the zone within 1.0 m below the pavement, floor subgrade or concrete slabs, the material should be compacted with the water content 2% to 3% drier than the optimum, compacted to 98% of the respective maximum SPDD.

In normal construction practice, the problem areas of settlement largely occur adjacent to manholes, catch basins, services crossings, foundation walls and columns. A granular backfill should be used for compaction in confined spaces with a smaller vibratory compactor.

6.6 Interlocking Stone Pavement and Landscaping

Interlocking stone pavement and landscaping structures in areas which are sensitive to frost-induced ground movement, such as in front of building entrances, must be constructed on a free-draining, non-frost-susceptible granular material such as Granular 'B'. This material must extend to at least 0.3 to 1.2 m below the slab or pavement surface, depending on the degree of tolerance of ground movement, and be provided with positive drainage, such as weeper subdrains connected to manholes or catch basins. Alternatively, the subgrade should be properly insulated with 50-mm Styrofoam, or equivalent.

The exterior grading around structures must be such that it directs runoff away from the structures.

6.7 Pavement Design

Where the pavement is to be built on structural slabs, such as the rooftop of the underground garage, a sufficient granular base and adequate drainage must be provided to prevent frost damage to the pavement. A waterproof membrane must be placed above the structural slab exposed to weathering to prevent water leakage, as well as to protect the steel reinforcing bars against brine corrosion.

The recommended pavement structure to be placed on top of the underground garage or on grade pavement is presented in Table 1.

Table 2 - Pavement Design

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL3
Asphalt Binder	65	HL8
Granular Base	200	Granular 'A'
Granular Sub-base	250	Granular 'B'

Where the grade is to be raised prior to the placement the pavement structure, the material must be non-frost susceptible and uniformly compacted to at least 98% SPDD. The granular bases should be compacted to 100% SPDD.

Along the perimeter where surface runoff may drain onto the pavement, an intercept subdrain system should be installed to prevent infiltrating precipitation from seeping into the granular bases (since this may inflict frost damage on the pavement). The subdrains should consist of filter wrapped weepers, and connected to the catch basins and storm manholes. The subdrains should be backfilled with free-draining granular material.

6.8 Soil Parameters

The recommended soil parameters for the project design are given in Table 2.

Table 3 - Soil Parameters

Unit Weight and Bulk Factor			
	Bulk Unit Weight (kN/m^3)	**Estimated Bulk Factor**	
		Loose	**Compacted**
Weathered/Broken Shale	24.0	1.40	1.10
Lateral Earth Pressure Coefficients			
	Active K_a	**At Rest K_0**	**Passive K_p**
Shale Bedrock	0.20	0.30	5.00
Coefficients of Friction			
Between Concrete and Granular Base			0.50
Between Concrete and Shale Bedrock			0.35

6.9 Excavation

Where excavation is to be carried out close to any existing underground structure or services, one must be aware that the previous backfill is amorphous in structure and is susceptible to sloughing and sudden side collapse. Extreme caution must be exercised and test pits should be used to evaluate the safety of such excavation. The existing services must be properly secured, where necessary.

Excavation should be carried out in accordance with Ontario Regulation 213/91. The types of soils are classified in Table 3.

Table 4 - Classification of Soils for Excavation

Material	Type
Shale Bedrock	1
Overburden above bedrock, if any	3

Continuous groundwater is not anticipated within the depth of excavation. Any water seepage can be drained into a sump and removed by conventional pumping.

For excavation in shale bedrock, a cut slope steeper than 1V (Vertical):1H (Horizontal) can be allowed, provided that the bedding plane of the rock is relatively horizontal and any loose rocks protruding from the excavation are removed for safety.

Excavation into the shale will require extra effort using mechanical means with a rock-ripper to facilitate the excavation. This method can generally be employed to excavate the shale near the ground surface. Efficient removal of the sound shale will require the aid of pneumatic hammering.

The shale is susceptible to disintegration and swelling upon exposure to air and water, with subsequent reversion to a clay soil. When excavating the sound shale, slight lateral displacement of the excavation walls is often experienced. This is due to the release of residual stress stored in the bedrock mantle and the swelling characteristic of the rock. A compressible material, such as sprayed foam, 80 to 100 mm in thickness, should be placed on the shale bedrock in order to slow down the disintegration if it will be exposed for more than a few weeks.

The deeper excavation has to be supported by shoring. The overburden load, the surcharge from adjacent structures and the hydrostatic pressure, if any, should be included in the design of the shoring. The design parameters and our recommendations are provided in the Appendix.

6.10 Monitoring of Performance

It is recommended that close monitoring of vertical and lateral movement of the shoring wall should be carried out and frequent site inspections be conducted to ensure that the excavation does not adversely affect the structural stability of the adjacent buildings and the existing underground utilities. Extra bracing or support may be required if any movement is found excessive. The contractor should maintain the shoring to ensure any movement is within the design limit.

Due to the presence of nearby buildings, the foundation details of the adjacent structures must be investigated and incorporated into the design and construction of the proposed project. It is recommended that a pre-construction survey and a monitoring program be carried out for all adjacent structures in order to verify any potential future liability claims.

Vibration control and pre-construction survey is strongly recommended for the adjacent properties and structures prior to any excavation activities at the site. Further advice or undertaking of the vibration control and pre-construction survey can be provided as necessary.

7.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the accounts of 1407 Lakeshore Developments Inc., and for review by the designated consultants and government agencies. Use of the report is subject to the conditions and limitations of the contractual agreement.

The material in the report reflects the judgement of Kelvin Hung, P.Eng., and Kin Fung Li, P.Eng., in light of the information available to it at the time of preparation. Any use which a Third Party makes of this report, or any reliance on decisions to be made are the responsibility of such Third Parties. Soil Engineers Ltd. accepts no responsibility for damages, if any, suffered by any Third Party as a result of decisions made or actions based on this report.

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LIST OF ABBREVIATIONS AND DESCRIPTION OF TERMS

The abbreviations and terms commonly employed on the borehole logs and figures, and in the text of the report, are as follows:

SAMPLE TYPES

AS Auger sample
CS Chunk sample
DO Drive open (split spoon)
DS Denison type sample
FS Foil sample
RC Rock core (with size and percentage recovery)
ST Slotted tube
TO Thin-walled, open
TP Thin-walled, piston
WS Wash sample

PENETRATION RESISTANCE

Dynamic Cone Penetration Resistance:

A continuous profile showing the number of blows for each foot of penetration of a 2-inch diameter, 90° point cone driven by a 140-pound hammer falling 30 inches.

Plotted as '—•—'

Standard Penetration Resistance or 'N' Value:

The number of blows of a 140-pound hammer falling 30 inches required to advance a 2-inch O.D. drive open sampler one foot into undisturbed soil.

Plotted as '○'

WH Sampler advanced by static weight
PH Sampler advanced by hydraulic pressure
PM Sampler advanced by manual pressure
NP No penetration

SOIL DESCRIPTION

Cohesionless Soils:

'N' (blows/ft)	Relative Density
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
over 50	very dense

Cohesive Soils:

Undrained Shear Strength (ksf)	'N' (blows/ft)	Consistency
less than 0.25	0 to 2	very soft
0.25 to 0.50	2 to 4	soft
0.50 to 1.0	4 to 8	firm
1.0 to 2.0	8 to 16	stiff
2.0 to 4.0	16 to 32	very stiff
over 4.0	over 32	hard

Method of Determination of Undrained Shear Strength of Cohesive Soils:

x 0.0 Field vane test in borehole; the number denotes the sensitivity to remoulding

△ Laboratory vane test

□ Compression test in laboratory

For a saturated cohesive soil, the undrained shear strength is taken as one half of the undrained compressive strength

METRIC CONVERSION FACTORS

1 ft = 0.3048 metres
1lb = 0.454 kg
1 inch = 25.4 mm
1ksf = 47.88 kPa

JOB NO.: 2108-S066

LOG OF BOREHOLE NO.: 1

FIGURE NO.: 1

PROJECT DESCRIPTION: Proposed New Building

METHOD OF BORING: Flight-Auger/ HQ Core

PROJECT LOCATION: 1041 Lakeshore Road East, City of Mississauga

DRILLING DATE: September 7, 2021
February 3/4, 2022 (Rock Core

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90 / Shear Strength (kN/m²) 50 100 150 200 / Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL / Moisture Content (%) 10 20 30 40	WATER LEVEL
82.5	Ground Surface							
0.0	Grey	1	DO	50/13	0	○ 4	●	Dry on completion
		~~2~~	~~AS~~	~~-~~				
		3	RC RQD	52% 31%	1 2			
	SHALE BEDROCK	4	RC RQD	98% 90%	3 4			
		5	RC RQD	97% 95%	5			
	occ. limestone layers	6	RC RQD	97% 84%	6 7			
		7	RC RQD	100% 60%	8			
		8	RC RQD	100% 75%	9			
72.4 10.1	**END OF BOREHOLE**				10 11 12 13 14 15			

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JOB NO.: 2108-S066

LOG OF BOREHOLE NO.: 2

FIGURE NO.: 2

PROJECT DESCRIPTION: Proposed New Building

METHOD OF BORING: Flight-Auger/ HQ Core

PROJECT LOCATION: 1041 Lakeshore Road East, City of Mississauga

DRILLING DATE: September 7, 2021
February 2, 2022 (Rock Core)

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90 / Shear Strength (kN/m²) 50 100 150 200 / Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL / Moisture Content (%) 10 20 30 40	WATER LEVEL
82.5	Ground Surface							
0.0	Grey	1	DO	83	0		5	Dry on completion
		2	AS	-				
		3	RC RQD	92% 52%	1			
		4	RC RQD	99% 66%	2			
	SHALE BEDROCK				3			
		5	RC RQD	100% 87%	4			
		6	RC RQD	100% 87%	5			
	occ. limestone layers				6			
		7	RC RQD	100% 42%	7			
		8	RC RQD	100% 75%	8			
		9	RC RQD	100% 58%	9			
72.1					10			
10.4	**END OF BOREHOLE**				11			
					12			
					13			
					14			
					15			

Soil Engineers Ltd.

JOB NO.: 2108-S066

LOG OF BOREHOLE NO.: MW1

FIGURE NO.: 3

PROJECT DESCRIPTION: Proposed New Building

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 1041 Lakeshore Road East, City of Mississauga

DRILLING DATE: February 1, 2022

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90 / Shear Strength (kN/m²) 50 100 150 200 / Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL / Moisture Content (%) 10 20 30 40	WATER LEVEL
82.6	Ground Surface							
0.0	**Augered from ground surface to 6.1 m** (Shale Bedrock)				0 1 2 3 4 5 6			Seepage from ground surface W.L. @ El. 81.1 m on February 24, 2022
76.5 / 6.1	**END OF AUGER HOLE** Installed 50 mm Ø monitoring well to 5.9 m completed with 3.0 m screen Sand backfill from 2.3 to 5.9 m Bentonite seal from 0.0 m to 2.3 m Provided with a monument steel casing				7 8 9 10 11 12 13 14 15			

Soil Engineers Ltd.

JOB NO.: 2108-S066

LOG OF BOREHOLE NO.: MW2

FIGURE NO.: 4

PROJECT DESCRIPTION: Proposed New Building

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 1041 Lakeshore Road East, City of Mississauga

DRILLING DATE: February 1, 2022

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90 / Shear Strength (kN/m²) 50 100 150 200 / Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL / Moisture Content (%) 10 20 30 40	WATER LEVEL
82.6	Ground Surface							
0.0	**Augered from ground surface to 6.1 m** (Shale Bedrock)				0 1 2 3 4 5 6			W.L. @ El. 80.1 m on completion Monitoring well frozen on February 24, 2022
76.5 / 6.1	**END OF AUGER HOLE** Installed 50 mm Ø monitoring well to 5.9 m completed with 3.0 m screen Sand backfill from 2.3 to 5.9 m Bentonite seal from 0.0 m to 2.3 m Provided with a monument steel casing				7 8 9 10 11 12 13 14 15			

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JOB NO.: 2108-S066

LOG OF BOREHOLE NO.: MW3

FIGURE NO.: 5

PROJECT DESCRIPTION: Proposed New Building

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 1041 Lakeshore Road East, City of Mississauga

DRILLING DATE: February 1, 2022

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90 / Shear Strength (kN/m²) 50 100 150 200 / Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL / Moisture Content (%) 10 20 30 40	WATER LEVEL
82.6	Ground Surface							
0.0	**Augered from ground surface to 6.1 m** (Shale Bedrock)				0 1 2 3 4 5 6			Dry on completion W.L. @ El. 78.9 m on February 24, 2022
76.5 / 6.1	**END OF AUGER HOLE** Installed 50 mm Ø monitoring well to 6.1 m completed with 3.0 m screen Sand backfill from 2.4 to 6.1 m Bentonite seal from 0.0 m to 2.4 m Provided with a monument steel casing				7 8 9 10 11 12 13 14 15			

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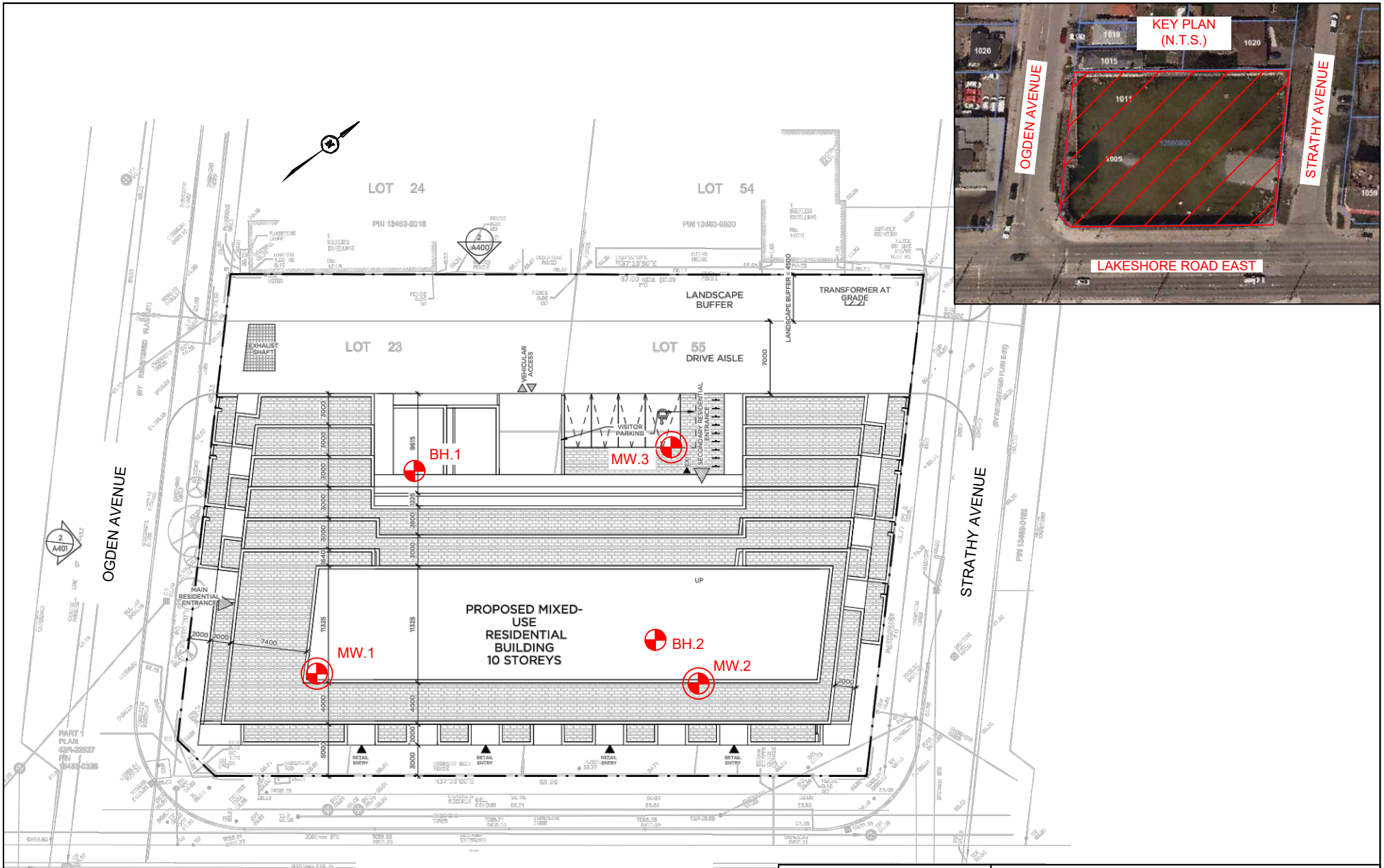

LEGEND

- Monitoring well

- Borehole

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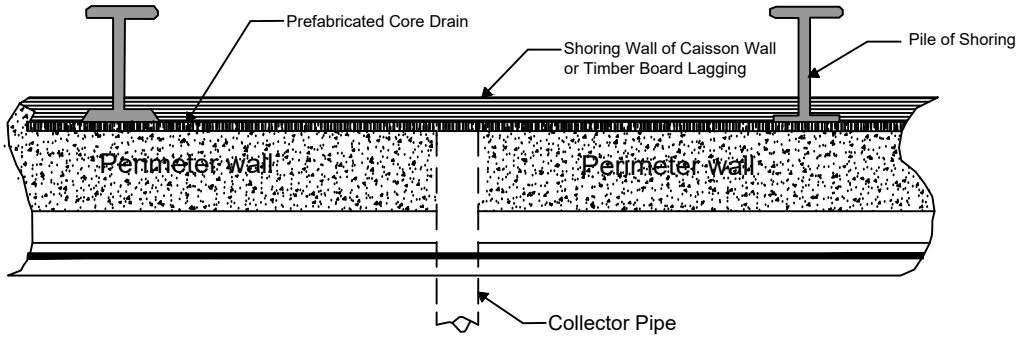
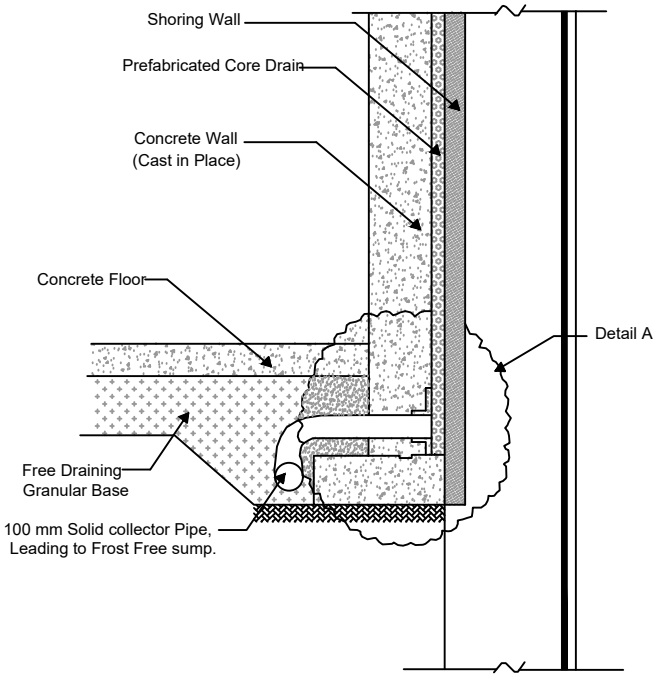
BOREHOLE & MONITORING WELL LOCATION PLAN

SITE: 1041 Lakeshore Road East, City of Mississauga

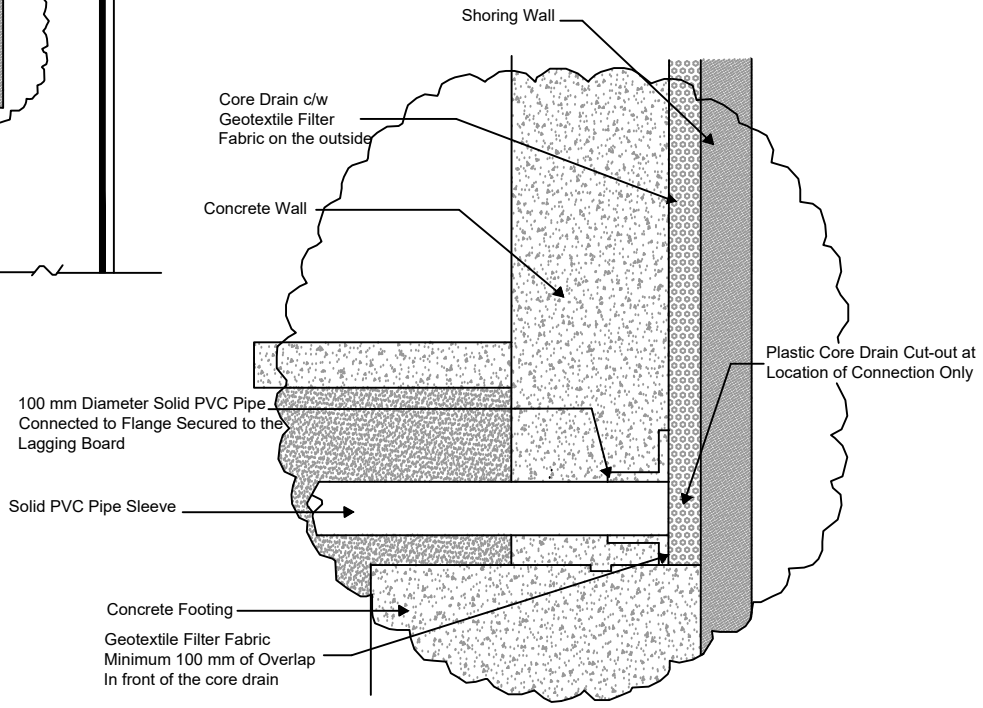
DESIGNED BY: -	**CHECKED BY:** -	**DWG NO.:** 1	
SCALE: 1:400	**REF. NO.:** 2108-S066	**DATE:** February 2022	**REV** 1



PLAN



TYPICAL SECTION



DETAIL A

NOTES:

1. A continuous blanket of prefabricated drainage system, Miradrain 6000 or equivalent, should extend continuously from the top of footings to the ground surface.

2. All joints of the Miradrain should be taped. All openings above the concrete footing must be covered with filter fabric to prevent intrusion of fresh concrete into the core of the drain.

3. Backfill behind the lagging board must be free draining. Filter fabric or straw should be used to prevent loss of fines behind the lagging.

4. The perimeter drainage and any subfloor drainage systems must be kept separate.

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PERMANENT PERIMETER DRAINAGE SYSTEM
(WITH SHORING)

SITE: 1041 Lakeshore Road East, City of Mississauga			
DESIGNED BY: K.L.	**CHECKED BY:** B.S.	**DWG NO.:** 2	
SCALE: N.T.S.	**REF. NO.:** 2108-S066	**DATE:** February 2022	**REV** -

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APPENDIX

SHORING DESIGN

REFERENCE NO. 2108-S066

SHORING SYSTEM

Shoring will be required in an excavation to limit the horizontal and vertical movements of adjacent properties.

A shoring system consisting of soldier piles and lagging boards can be used in an excavation where slight movement in the adjacent properties is tolerable. In areas in close proximity to adjacent structures and where the excavation will be extending below the foundation level so that any movement in the adjacent properties is a concern, or in an excavation embedding into saturated sand or silt deposits, an interlocking caisson wall is more appropriate.

The design and construction of the shoring system should be carried out by a specialist designer and contractor experienced in this type of construction. All specifications for the design of the shoring system should be in accordance with the latest edition of the Canadian Foundation Engineering Manual (CFEM).

LATERAL EARTH PRESSURE

For single and multiple level supporting systems, the lateral earth pressure distributions on the shoring walls are shown on Drawing A1. The design soil parameters are provided in the geotechnical report.

The lateral earth pressure expressions do not include hydrostatic pressure build up behind the shoring. If the wall is designed to be water tight or undrained, such as a caisson wall, the anticipated hydrostatic pressure must be included behind the structure.

PILE PENETRATION

The depth of pile support into shale bedrock should be at least 1.0 m below the bottom of excavation.

The shoring system should be designed for a factor of safety of F = 2.

For anchor supported shoring system, the global factor of safety against sliding and overturning of the anchored block of soil must also be considered.

The steel soldier piles in the shoring system must be installed in pre-augured holes. The lower portion will have to be filled with 20 MPa (3000 psi) concrete to the excavation level. The upper portion of the pile within the excavation depth should be filled with lean mix concrete or non-shrinkable cementitious filler (U-fill).

LAGGING

The following thicknesses of lagging boards have been recommended in CFEM:

Thickness of Lagging	Maximum Spacing of Soldier Piles
50 mm (2 in)	1.5 m (5 ft)
75 mm (3 in)	2.5 m (8 ft)
100 mm (4 in)	3.0 m (10 ft)

Local experience has indicated that the lagging board thickness of 75 mm has been adequate for soldier pile spacing of 3 m for soil conditions similar to those encountered at the subject site. However, it is important to consider all local conditions, such as the duration of excavation, the weather likely to be encountered through the construction period, seasonal variations in the ground water and ice lensing causing frost heave and softening of soils in determining the lagging thickness. During winter months, the shoring should be covered with thermal blankets to prevent frost penetration behind the shoring system which may result in unacceptable movements.

During construction of shoring, all the spaces behind the lagging board must be filled with free draining granular fill. If wet conditions are encountered, the space between the boards should be packed with a geotextile filter fabric or straw to prevent the loss of fine particles.

TIEBACK ANCHORS

The minimum spacing and the depths of the soil anchors should be as recommended in the CFEM.

All drilled holes for tieback anchors should be temporarily cased or lined to minimize the risk of caving. Systems involving high grout pressures should be avoided if working near other basements or buried services.

The tieback anchor lengths can be estimated using an adhesion value of 600 kPa where the tieback anchor is grouted in shale. Full scale load tests should be carried out on the tieback anchors in each type of soils and at each level of anchor support at the site to confirm the design parameters and the adhesion values. The test anchors should be loaded in a pattern as described in CFEM, to 200% of the design load or until there is a significant increase in the pullout rate. In the latter case, the design load must be limited to 50% of the maximum load at which the pullout increases. Based on the results of the pullout test, it may be necessary to modify the anchor design of the production anchors.

Each tieback anchor must be proof-loaded to 133% of the design load, and the anchor must be capable of sustaining this load for a minimum of 10 minutes without creep. The load may then be relaxed to 100% of the design and locked in. The higher the lock-in loads, the less will be the outward movement on the shoring wall after excavation.

RAKERS

An alternative to tieback anchor support of the shoring is to use raker footings. Rakers inclining at an angle of 45º, founded in the shale bedrock below the bottom of excavation should be designed for the allowable bearing pressure of 500 kPa. Raker footings extending into the sound shale can be designed for the allowable bearing pressure of 800 kPa.

The raker footings should be located outside the zone of influence of the buried portion of the soldier piles at a distance of not less than 1.5 of the length of embedment of the soldier pile.

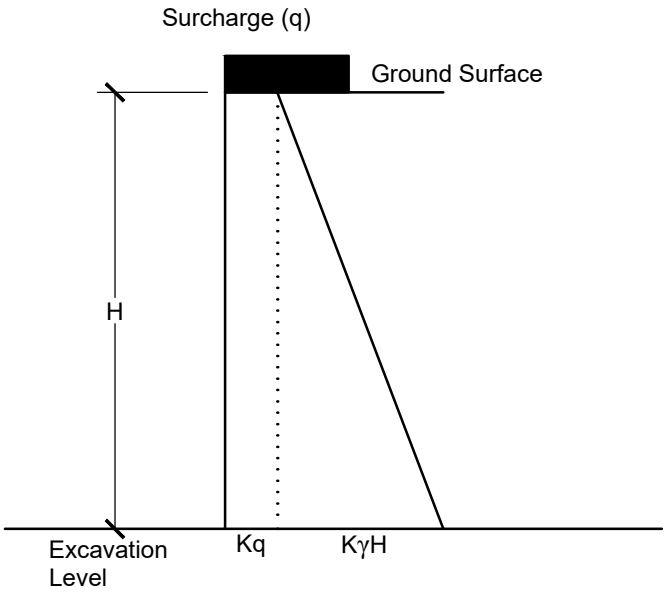
To prevent undermining of the raker footing, no excavation should be made within two times the width of raker footing on the opposite side of the raker.

When sloping berm excavation procedures are used, the rakers should be installed in trenches in the berm to minimize movement of the shoring wall being supported. In addition, the rakers can be pre-loaded and secured in place before removal of the earth berm.

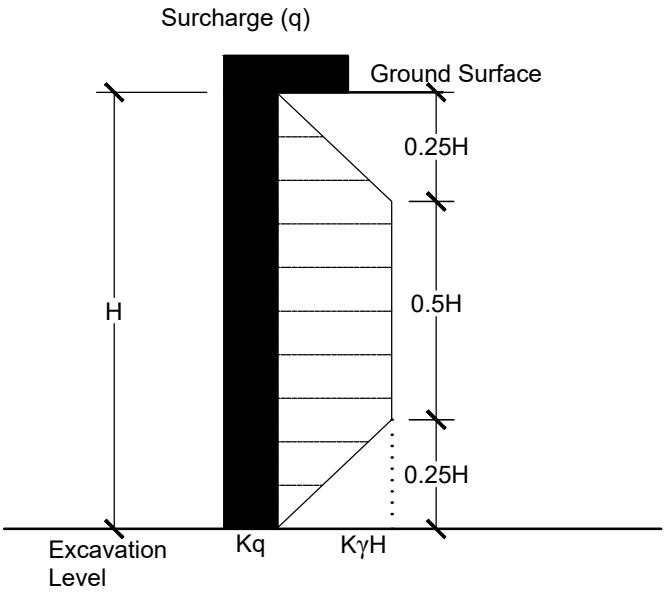
MONITORING OF PERFORMANCE

Close monitoring of the vertical and lateral movement of the shoring system, by inclinometers or by survey on targets, should be carried out at the site. Extra bracing or support may be required if any movement is found excessive. The contractor should maintain the shoring to ensure any movement is within the design limit.

TEMPORARY SHORING
Lateral Earth Pressures



Single Support System



Multiple Support System

Lateral Pressure $P = K(\gamma H + q)$

Where
H = Height of Shoring (m)
γ = Unit Weight of Retained Soil (kN/m^3)
q = Surcharge (kPa)
K = Earth Pressure Coefficient

- If moderate ground and shoring movements are permissible then:
  $K = K_a$ = Active Earth Pressure Coefficient

- if there are building foundations within a distance of 0.5 H behind the shoring then:
  $K = K_o$ = Earth Pressure at rest

- If there are building foundations within a distance of between 0.5 H and H behind the shoring then:
  $K = 0.5\,(K_a + K_o)$

Note:
1. The lateral pressure expression assumes effective drainage from behind the temporary shoring.
2. The earth pressure coefficients are specified in the geotechnical report.

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TEMPORARY SHORING
LATERAL EARTH PRESSURES

SITE: 1041 Lakeshore Road East, City of Mississauga			
DESIGNED BY: K.L.	**CHECKED BY:** B.S.	**DWG NO.:** A1	
SCALE: N.T.S.	**REF. NO.:** 2108-S066	**DATE:** February 2022	**REV** -