MCR ENGINEERS LTD.
GEO-ENVIRONMENTAL CONSULTANTS

111 ZENWAY BLVD., UNIT 4, VAUGHAN, ONTARIO, L4H 3H9
TEL: 416.675.0160 FAX: 905.851.1722
office@mccrak.com

1271 DENISON ST., UNIT 45, MARKHAM, ONTARIO, L3R 4B5
TEL: 905.470.0160 FAX: 905.475.6371
denison@mccrak.com

WWW.MCCRAK.COM

G5480 **JUNE 2024**

GEOTECHNICAL REPORT
PROPOSED MIXED-USE
RESIDENTIAL/COMMERCIAL DEVELOPMENT
145 WELLINGTON STREET WEST
TORONTO, ONTARIO

DISTRIBUTION:

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1 COPY	MCR ENGINEERS LTD.

PREPARED FOR:

H & R REIT – 145 WELLINGTON C/O
3625 DUFFERIN STREET, SUITE 500
TORONTO, ON
M3K 1N4

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1.0 INTRODUCTION

MCR was retained by H & R Reit – 145 Wellington, to prepare a geotechnical report for the proposed residential/commercial development located at 145 Wellington Street West, in the City of Toronto, Ontario.

The objective of the report was to determine design data required for foundations, dewatering, shoring/excavation, backfill, slab on grade, and pavement. The above design and construction issues are addressed in the following report.

2.0 SITE CONDITIONS

The Site is located on the southeast corner of The Wellington Street West and Simcoe Street at the municipal address of 145 Wellington Street West, in a mixed residential and commercial area of the City of Toronto, Ontario.

The Site is currently occupied by a midrise office building with three levels of underground parking.

3.0 PROPOSED DEVELOPMENT

The Site is proposed for mixed-use residential and commercial development consisting of a sixty-five [65] storey building with a mechanical penthouse floor over three [3] levels of below grade parking (Appendix A).

The ground floor finished floor elevation (FFE) is expected to be at 83.10 m, while the P3 FFE is projected to be at 73.70 meters.

4.0 SITE INVESTIGATION

A total of three (BH 1 to BH 3) were drilled by MCR at the subject Site during the period of March 27 to April 3, 2019. Shale samples were taken by rock coring. The borehole records are presented in Appendix B. The borehole locations are shown in Drawing No. 1.

Due to site limitations for drilling, the boreholes were drilled from the existing P3 floor slab. The boreholes were drilled to an approximate depth of 4.60 m below the existing P3 floor slab, using rock coring.

Monitoring wells were installed in all the boreholes BH 1 to BH 3 for long term groundwater monitoring and water sampling.

Borehole elevations, referred to in this report, are geodetic and are referenced to Toronto Benchmark No. 2556 CT, with a published Elevation of 81.256 m.

5.0 SOIL AND GROUNDWATER CONDITIONS

Subsurface conditions encountered at the borehole locations are shown on Borehole Log Sheets, attached in Appendix B and summarized as follows:

Concrete Slab/Granular Base: A concrete slab, approximately 125 mm in thickness, was present at the surface of all the boreholes. A granular base, varying from 275 to 300 mm in thickness and consisting of clear stone was encountered below the concrete slab in Boreholes 1 and 2.

Shale Bedrock: Grey, moist shale bedrock was detected below the clear stone/concrete slab in all the boreholes and extended to the maximum explored depth of the boreholes. The shale bedrock was encountered at approximate Elevations ranging from 73.50 to 73.25 m. The shale bedrock was weathered in the upper 750 to 950 mm.

The depth to shale bedrock should be confirmed during shoring installation and general excavation.

Groundwater: Upon completion of drilling, groundwater was not measured due to rock coring technique.

The results of subsequent groundwater level monitoring are summarized on Table 1.

Table 1 – Groundwater Level Monitoring Results

Monitoring Well Id	Ground Surface Elevation (masl)	Water Level (mbgs)	Groundwater Elevation (masl)	Date of Measurement (mm/dd/yyyy)	Depth of Well (mbgs)	Depth of Bentonite (mbgs)	Length of Screen (m)	Inside Diameter of Pipe (mm)	Top of Monitoring Well
BH 1	73.67	-	-	03/28/2019	4.57	0.90	3.05	50	Flush Mount
		0.73	72.94	04/04/2019					
BH 2	73.67	-	-	03/29/2019	4.57	0.90	3.05	50	Flush Mount
		4.28	69.39	04/04/2019					
BH 3	73.64	-	-	04/03/2019	4.57	0.90	3.05	50	Flush Mount
		1.17	72.47	04/04/2019					
Min	73.64	0.73	69.39	-	4.57	-	-	-	-
Max	73.67	4.28	72.94	-	4.57	-	-	-	-
Average	73.66	2.06	71.60	-	4.57	-	-	-	-

Please note that the groundwater levels are subject to seasonal fluctuations. Consequently, definitive information on the long-term groundwater levels could not be obtained at the present time.

In addition, the sedimentary bedrock contains waterbearing bedding planes. Please note that the presence/thickness of bedding planes is difficult to assess due to rock coring pressure.

When the bedding planes are intercepted in rock excavation, caissons or elevator pistons etc., a substantial amount of water, often under a hydrostatic head may be encountered.

Geohydrology assessment study is currently underway by MCR. The results of the assessment will be presented in a separate report upon completion.

Subject to the owner's approval, groundwater monitoring should continue, and the results should be presented in a separate report addressing Geohydrology/Dewatering induced Settlement issues, if required.

6.0 FOUNDATION

The Site is proposed for mixed-use residential and commercial development consisting of a sixty-five [65] storey building with a mechanical penthouse floor over three [3] levels of below grade parking (Appendix A).

The ground floor finished floor elevation (FFE) is expected to be at 83.10 m, while the P3 FFE is projected to be at 73.70 meters.

Based on the encountered soil/rock conditions and subject to design grades/final loads the proposed development with three [3] level of underground parking, can be supported on conventional spread/strip footings or raft foundation.

The recommendations are based on the current information and design. Should changes are made during the design phase or construction, this office must be retained to modify recommendations accordingly or propose additional field work.

6.1 SPREAD/STRIP FOOTINGS/RAFT

The proposed footings/raft could be proportioned using the following bearing resistance:

Factored Bearing Resistance at ULS = 3500 kPa
Bearing Resistance at SLS = 2500 kPa

When the underside of the footings/raft is founded in weathered shale bedrock, at least 500 mm below the surface of shale, at or below an approximate Elevation of 72.50 m, **subject to the depth of shale bedrock across the site and field inspection during footing/raft installation.**

Coefficient of Subgrade Reaction k (for weathered shale) = 80 MN/m^3 is considered applicable.

Alternatively, the proposed footings/raft could be proportioned using the following bearing resistance:

Factored Bearing Resistance at ULS = 7000 kPa
Bearing Resistance at SLS = 5000 kPa

When founded in sound shale bedrock at or below an Elevation of 71.50 m, and at least 1.50 m below the surface of the shale bedrock, **subject to the depth of shale bedrock across the site and field inspection during footing/raft installation.**

Coefficient of Subgrade Reaction k (for sound shale) = 120 MN/m^3 is considered applicable.

6.2 GENERAL FOUNDATION NOTES

Adjacent footings, founded at different elevations, preferably are to be stepped at 10 horizontal to 7 vertical, subject to rock condition during excavations.

For frost protection requirements, the exterior footings/ exterior of the underside of footings in unheated areas in unheated P3 areas must have a minimum shale bedrock cover of 0.5 m.

Any water or loose materials must be removed from the footing bases prior to placing concrete.

The recommended resistance at SLS allows for up to 25 mm of total settlement. Potential differential settlements are to be evaluated after completion of the foundation drawings.

Furthermore, the recommended bearing resistance and foundation elevations have been calculated from the limited borehole information and are intended for design purposes only. More specific information with respect to rock/foundation conditions will be available when the proposed shoring/foundation construction is underway.

Therefore, the encountered rock/foundation conditions must be verified in the field, and all drilled foundations/footings must be inspected and approved by our office prior to placement of concrete.

7.0 EARTHQUAKE CONSIDERATION

The building must be designed to resist a minimum earthquake force. The National Building Code specifies that the building be designed to withstand a minimum lateral seismic force, V, which is assumed to act non-currently in any direction on the building as per the following expression:

$$V = S(T_a)\, M_v I_E W / R_d R_o$$

It should be noted that V shall not be less than:

$$S(2.0)\, M_v I_E W / R_d R_o$$

In addition, the SFRS (Seismic Force Resisting System (s)) with R_d equal to or greater than 1.5, V should not be greater than:

$$2/3\, S(0.2)\, I_E W / R_d R_o$$

Where $S(T_a)$ shall be calculated by $S_a(T_a)F_a$ or $S_a(T_a)F_v$, depending on fundamental lateral period T_a. The terms, which are relevant to the geotechnical conditions at the site, are acceleration-based site coefficient F_a and velocity-based site coefficient F_v.

For the subject site, classified as Class B, the applicable values of Fa and Fv are 0.8 and 0.6 respectively. A structural consultant should review all factors.

8.0 BASEMENT WALLS

Underground parking walls should be designed to resist a pressure "p", at any depth, "h" below the surface, as given by the expression:

$$p = 0.40[\gamma h + q]$$

Where: K = 0.40 for the earth pressure coefficient considered applicable

K = 0.25 for the shale pressure coefficient considered applicable

γ = 21.7 kN/m^3 is the unit weight of backfill

q = an allowance for surcharge.

The above equation assumes that perimeter drains will be provided and that the backfill against subsurface walls, where applicable, would be a free draining granular material.

However, subject to groundwater conditions and the presence of the wet soils, all subject to further groundwater monitoring results, we suggest that perimeter walls below the groundwater level be designed for hydrostatic pressure to resist a pressure "p", at any depth "h" below the surface, as given by the expression:

$$p = \begin{cases} Kq + K\gamma_m h & h \le D_w \\ Kq + K\gamma_w D_w + K(\gamma_s - \gamma_w)(h - D_w) + \gamma_w(h - D_w) & h > D_w \end{cases}$$

Where: K = 0.50 for the earth pressure coefficient considered applicable

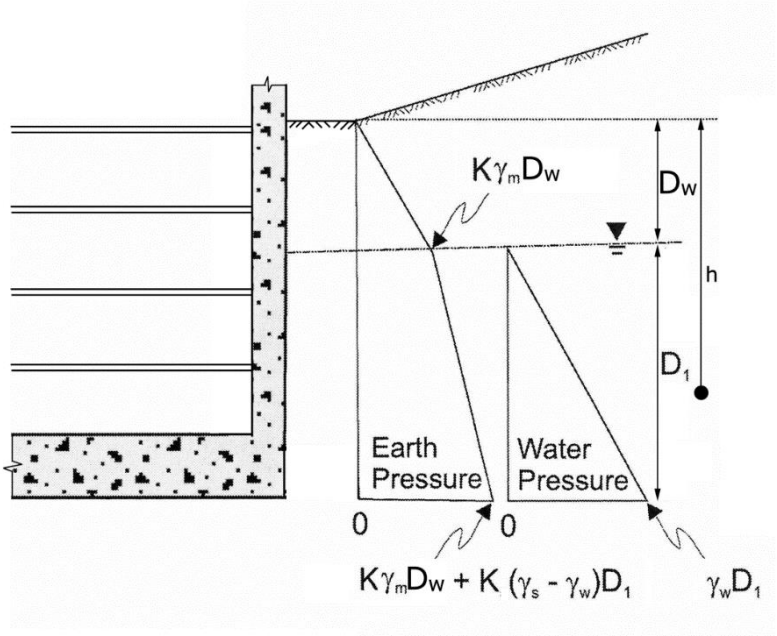
K = 0.25 for the shale pressure coefficient considered applicable

γ_m = 20 kN/m^3 is moist or wet soil unit weight

γ_s = 21.7 kN/m^3 is saturated soil unit weight

γ_w = 9.80 kN/m^3 is the unit weight of water

q = an allowance for surcharge



9.0 DEWATERING

The excavation for the proposed underground parking will extend below the groundwater table, subject to further investigation, groundwater monitoring results and excavation condition.

In order to protect the sides of the excavation from being disturbed by excess groundwater pressure, i.e. to prevent quicksand/dilating silt conditions, the water table must be lowered to at least 1.0 m below the bottom of the caps/raft excavations.

Positive dewatering, above bedrock, should consist of well points/eductors. The selected dewatering system, designed by a speciality contractor, will be most effective if it is installed and activated at the earliest opportunity during general excavation.

To control the potential localized groundwater influx, bedrock could be trenched and temporary sump pumps installed.

Where caisson wall shoring is required, any breaches in caisson wall shoring might result in localized piping. Creation of piping channels might increase the volume of both temporary dewatering and permanent drainage. It is critical that during general excavation **potential formation of localized piping be carefully evaluated and appropriate corrective measures implemented.**

A pre-construction survey of adjacent structures/roads should be carried out prior to the dewatering/shoring construction/foundation installation stage. Potential adverse effects on adjacent structures, due to the dewatering must be assessed/quantified and suitable preventive/remedial measures implemented.

10.0 EXCAVATION AND BACKFILL

Excess soils shall be managed in accordance to O. Reg. 406/19. As of January 1, 2022, the Project Leader may be required to file a notice in the registry as prescribed under Section 8 of the regulation. The notice shall contain the information set out in Schedule 1 of the regulation. Before the notice is filed the Project Leader shall ensure that a Qualified Person (Qualified Person within the meaning of Section 5 or 6 of O. Reg. 153/04) prepares the documents, as required, under Sections 11, 12, 13 of the regulation.

The Project Leader shall, if required to file a notice and before removing excess soil from the project area, develop and apply a tracking system in accordance with the Soil Rules, to track each load of excess soil during its transportation and deposit.

No major problems will be encountered for the anticipated depth of general excavations, carried out within a shoring wall enclosure.

The excavation in weathered shale bedrock can be carried out with a heavy-duty backhoe. However, the shoring/foundation contractor must be aware that the relatively harder and thick limestone slabs or seams, are interbedded in the shale bedrock.

For excavation above the water table, the anticipated water seepage, if any, into the excavations from the more permeable seams/lenses or surface run-off can be handled by conventional pumping methods.

A dewatering system such as well points/eductors will be required for excavation at/below the groundwater level, above bedrock, subject to excavation condition and long-term groundwater monitoring results.

The material to be used for backfilling under floor slab should be size 19 mm clear stone.

In service trenches (outside the building), the fill should be suitable for compaction, i.e. free of limestone fragments of a size greater than 150 mm, and with natural moisture content, which is within 2 percent of the optimum moisture content.

The backfill material should be compacted to at least 98 percent of the Standard Proctor Maximum Dry Density (SPMDD).

The backfill under floor slab against subsurface walls, where applicable, should be free draining granular fill, preferably conforming to the Ontario Provincial Standard Specification for granular base course, Granular B.

11.0 SHORING

Based on the information provided, it is understood that the existing foundation walls are intended to be used as a shoring system. Upon demolition of the existing substructure, the remaining foundation walls should be carefully evaluated by the project's shoring and structural engineers, prior to potential re-use for shoring.

It should be noted that groundwater and cobbles/boulders might be encountered during soldier pile/caisson construction, and the contractor must be prepared to deal with boulders and water seepage into the caisson shafts without undue delays.

Specifically, the shoring contractor may experience difficulties during the drilling the much harder/thick limestone slabs.

Subject to groundwater conditions/monitoring results; it might be difficult to prevent groundwater from penetrating into the excavation through gaps in timber lagging.

The geotechnical parameters, which are considered to be applicable for the design, are as follows:

Active earth pressure coefficient Ka = 0.45 for walls in areas where structures or sensitive services are being supported.

Active earth pressure coefficient Ka = 0.28 for remaining areas.

Natural unit weight of soil = 21.7 kN/m^3

Passive pressure coefficient in shale bedrock Kp = 5

Any surcharge loads must be included in the lateral pressure calculations.

Lateral movements of the shoring wall, designed using Ka = 0.28, are expected to be in order of 15 mm. They are expected to be less if Ka value of 0.45 is used. The expected movements are based on a properly constructed system.

The horizontal and vertical movements should be monitored during construction to

ensure satisfactory performance of the shoring system.

Soil and rock anchors should be designed for 30 and 600 kPa (based on our experience with the area and subject to confirmation by on site load tests). **It is re-iterated that subsurface conditions may vary beyond the site's confines. As a result, the design values must be confirmed by at least two load tests, carried out to twice the design load.**

The encountered sedimentary shale bedrock contains frequent limestone interbeds. In addition, weaker vertical or inclined 'rubble' zones could be intercepted in rock excavations. Typically, as a safety measure, a wire mesh in combination with plywood have been used to cover localized rubble zones.

It is imperative that a stability analysis of the entire support system is undertaken prior to commencement of construction. The final shoring design should be reviewed by our office.

Space and groundwater influx permitting, lowest parking level could be excavated "neat" into the rock face. A sufficient rock bench/rock bolts will be required to secure the integrity of the shoring system.

The exposed rock face could be shotcreted, subject to site condition/field inspection during excavation.

Schematic drawing for the proposed permanent drainage system is enclosed (Drawing No. 2).

In addition, a pre-construction survey of the surrounding structures/roads is recommended prior to commencement of shoring construction.

The shoring system and surrounding structures must be monitored for horizontal and vertical movements, prior to, during and after the excavation.

12.0 SLAB ON GRADE AND PERMANENT DRAINAGE

The City of Toronto – Toronto Water Division requires that any private water to be discharged into the City sewer system must have a permit or agreement in place in order to discharge; this applies to all water not purchased from the City water supply. For temporary dewatering during the construction phase, this includes all groundwater and storm water that is collected or encountered during site excavation.

For Private Water Discharge System (PWDS) this includes all groundwater that is constantly pumped as a result of the PWDS elevation located below the groundwater table elevation or through storm water infiltration.

Alternatively, a fully waterproofed substructure may be required in the event that discharge to sewers is not feasible. The Client must obtain permission and confirm discharge approval from Toronto Water directly (see Appendix C).

A permanent Private Water Drainage System (PWDS), as shown on Drawings No. 2 and 3, where shoring is constructed, should be considered.

Upon completion of foundation work, the SOG should rest on a well compacted bed of size 19 mm clear stone at least 200 mm thick. The stone bed would act as a barrier and prevent capillary rise of moisture from the subgrade to the floor slab.

To minimize siltation, all drainage pipe connections must be solid slotted PVC, with elbows and Ts, no “butt” end connections should be permitted. The pipes should slope to a sump at a minimum 1% slope.

Perimeter drainage pipes, with a positive gravity outlet, should be solid and slotted PVC with a minimum of 0.5% slope. In addition, silt traps must be provided at convenient/accessible locations.

We request that PWDS drawings indicate design elevations for both perimeter and underfloor installation. MCR will provide calculations for sizing of permanent pumps, when required.

Upon completion of general excavation, scope and adequacy of the PWDS is to be re-

evaluated. The installation of PWDS must be inspected by our office, prior to placement of filter stone.

The contractor installing the PWDS should be given a three-year maintenance contract to ensure the continued efficiency of the Permanent Drainage System. This contract would involve semi-annual inspections of ports, sump pits, weepers, sand traps, etc., with the aim of proactively preventing blockage by silt deposits.

Design changes must be approved by the architect and reflected on mandatory as built drawings. *

* A copy of this page “Slab on grade and Permanent Water Drainage System” page should be posted at a site office as a permanent display.

In addition, the elevator pit should be fully waterproofed as shown on Drawing No. 4.

13.0 PAVEMENT

The critical section of pavement will be at the transition from the infinitely rigid substructure onto soil/backfill subgrade.

As a result, we suggest that an approach type slab be constructed at the entrance/exit points, as shown on Drawing No. 5.

The approach slab will alleviate detrimental effects of dynamic loading/settlement/pavement depression in the backfill to the rigid substructure.

All granular materials used in the pavement construction should be compacted to 100% of the Standard Proctor Maximum Dry Density.

Asphaltic concrete layer should be compacted to the range of 92 to 96.5% of maximum relative density.

Pavement structures presented in tables 2 and 3 are typical. Subject to the anticipated road traffic volumes/AADT/axle loads, the pavement structural design matrix as per City of Toronto Standards presented in Appendix D, must be followed.

Table 2 - Typical Flexible Pavement Structure

Pavement Layer	Recommended Thickness for Light Duty Parking	Recommended Thickness for Heavy Duty Parking
Asphaltic Concrete	40 mm OPSS HL 3 40 mm OPSS HL 8	50 mm OPSS HL 3 75 mm OPSS HL 8
OPSS Granular A Base (or 20mm Crushed Limestone)	150 mm	150 mm
OPSS Granular B	200 mm	350 mm

Table 3 - Typical Composite Pavement Structure

Pavement Layer	Compaction Requirements	Heavy Duty Pavement
Asphaltic Concrete	92 to 96.5% of Maximum Relative Density	50 mm OPSS HL 1 or HL 3
Portland Cement Concrete (CAN3-CSA A23.1) - Class C-2	CAN3-CSA A23.1	150 mm
Base Course: Granular A (OPSS 1010) or 19 mm Crusher Run Limestone	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm

Please refer to Drawing No. 6 & 7 for a typical pavement structure above the garage roof slab.

14.0 METHANE GAS

The concentrations of methane gas in the borehole were measured at all samples and no gas was detected. The methane gas concentrations are presented on the attached MCR's borehole log sheets in Appendix B.

15.0 GENERAL COMMENTS

The comments given in this report are intended only as guidance for design engineers and are subject to field verification during construction. As more specific subsurface information, with respect to conditions between boreholes becomes available during excavations on the subject site, this report should be updated.

Contractors bidding on or undertaking the work should decide on their own investigations, as well as their own interpretations of the factual borehole results. This concern specifically applies to the classification of the subsurface soil and the potential reuse of these soils on/off site.

The contractors must draw their own conclusions as to how the near surface and subsurface conditions may affect them.

We trust this report contains information requested at this time. However, if any clarification is required or if we can be of further assistance, please call us.

Respectfully,

MCR ENGINEERS LTD.

Report Prepared by

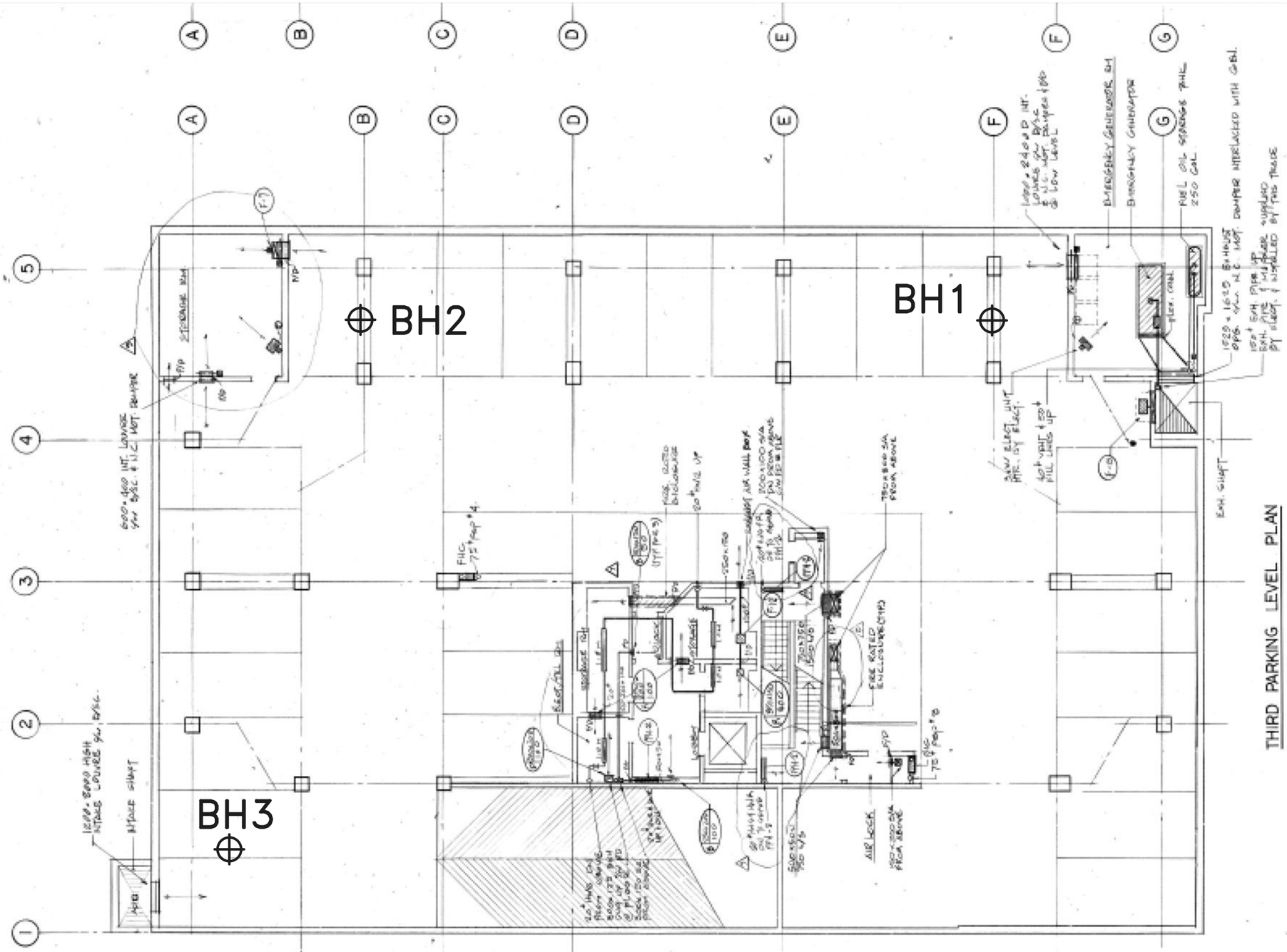
Salman Tavassoli, M.Sc., P.Eng.

Reviewed by

L.J. Rak, M.Eng., P.Eng.

DRAWINGS

145 WELLINGTON STREET WEST
TORONTO, ONTARIO



Legend

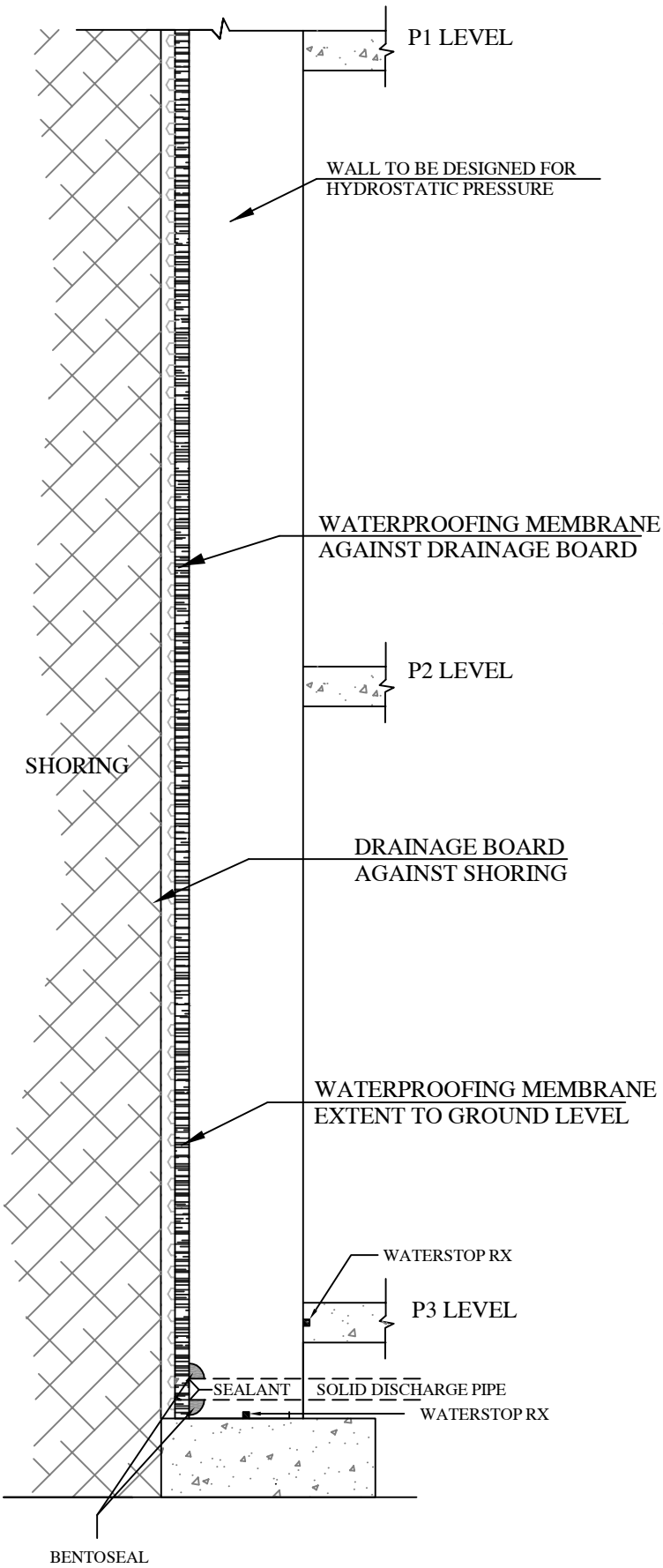
⊕ GROUNDWATER MONITORING WELL BY McCLYMONT & RAK ENGINEERS INC., MARCH/APRIL 2019

McCLYMONT&RAK ENGINEERS, INC.
GEO-ENVIRONMENTAL CONSULTANTS

BOREHOLE LOCATION PLAN

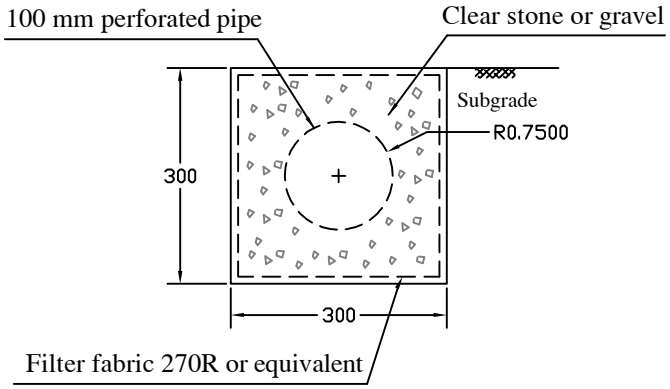
Scale: N/A	Project No.: GE5480
Date: MAY 2019	Drawing No.: 1

SUGGESTED EXTERIOR DRAINAGE AGAINST SHORING



NOTE:

* All permanent drainage pipes must have Geotextile filter sleeve to prevent long term silting. To further minimize siltation of the drainage system, all drainage pipe connections must be solid PVC elbows and Ts, no "butt" end connections should be permitted.
* Perimeter collection pipe to be solid pipe.



DETAILS OF SUB-FLOOR DRAINS
TO BE PLACED IN PARALLEL ROWS 6- 8M (20'- 25')
CENTERLINE TO CENTERLINE.

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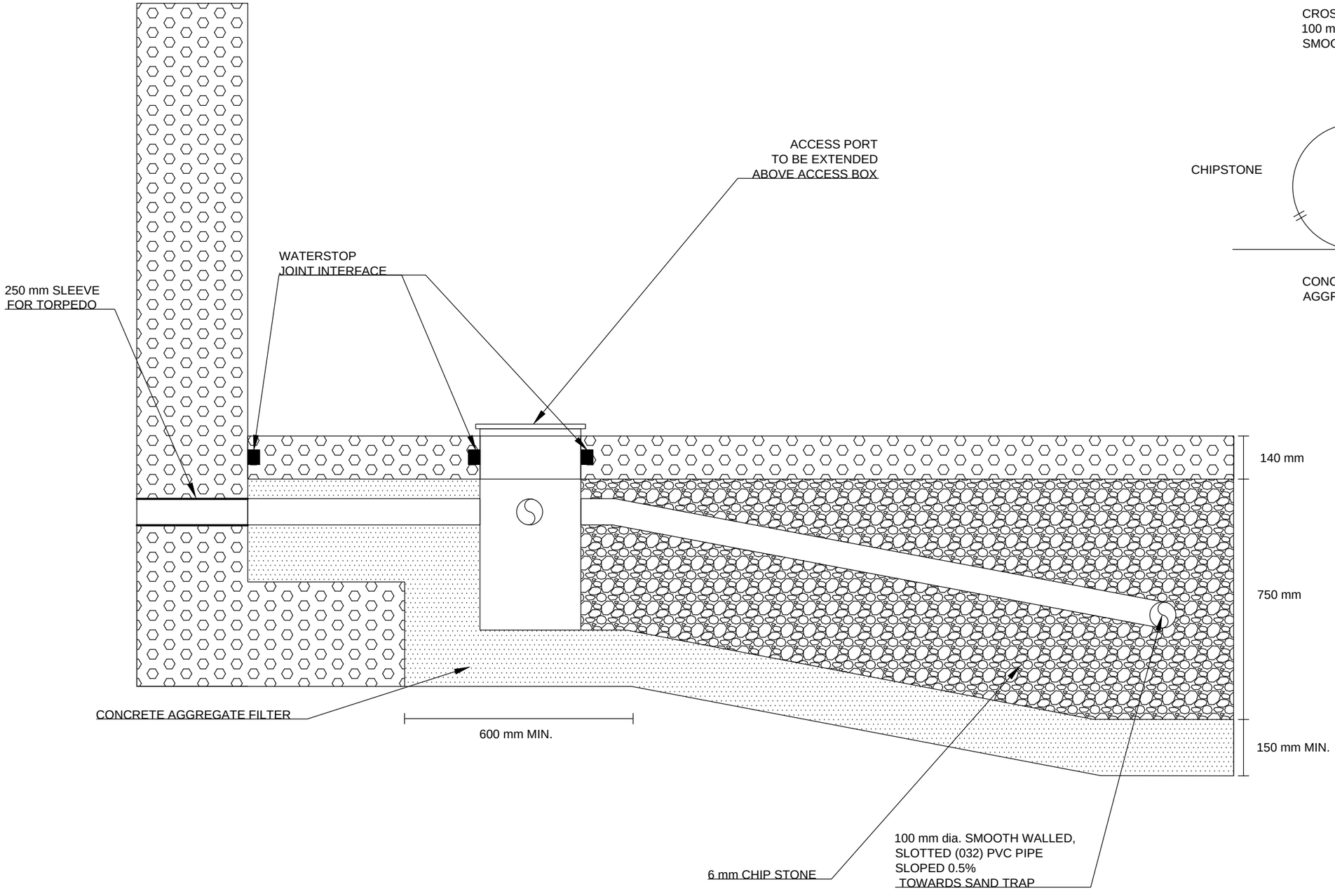
Project :	PROPOSED DEVELOPMENT		
Scale : N.T.S.	Date :	Drawing No. 2	



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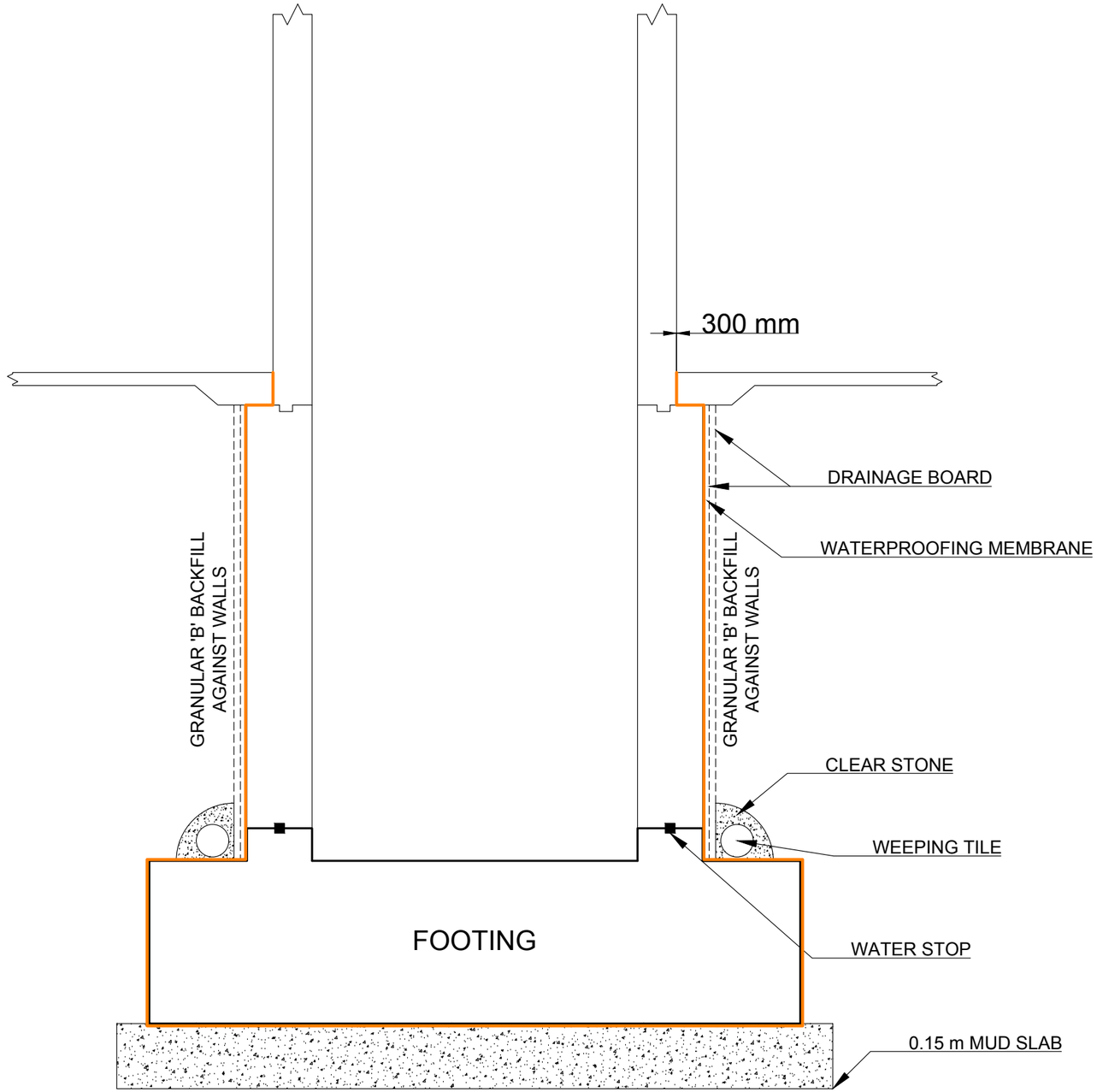
PRIVATE WATER DRAINAGE SYSTEM

Scale:	Drawing No.
NTS	3



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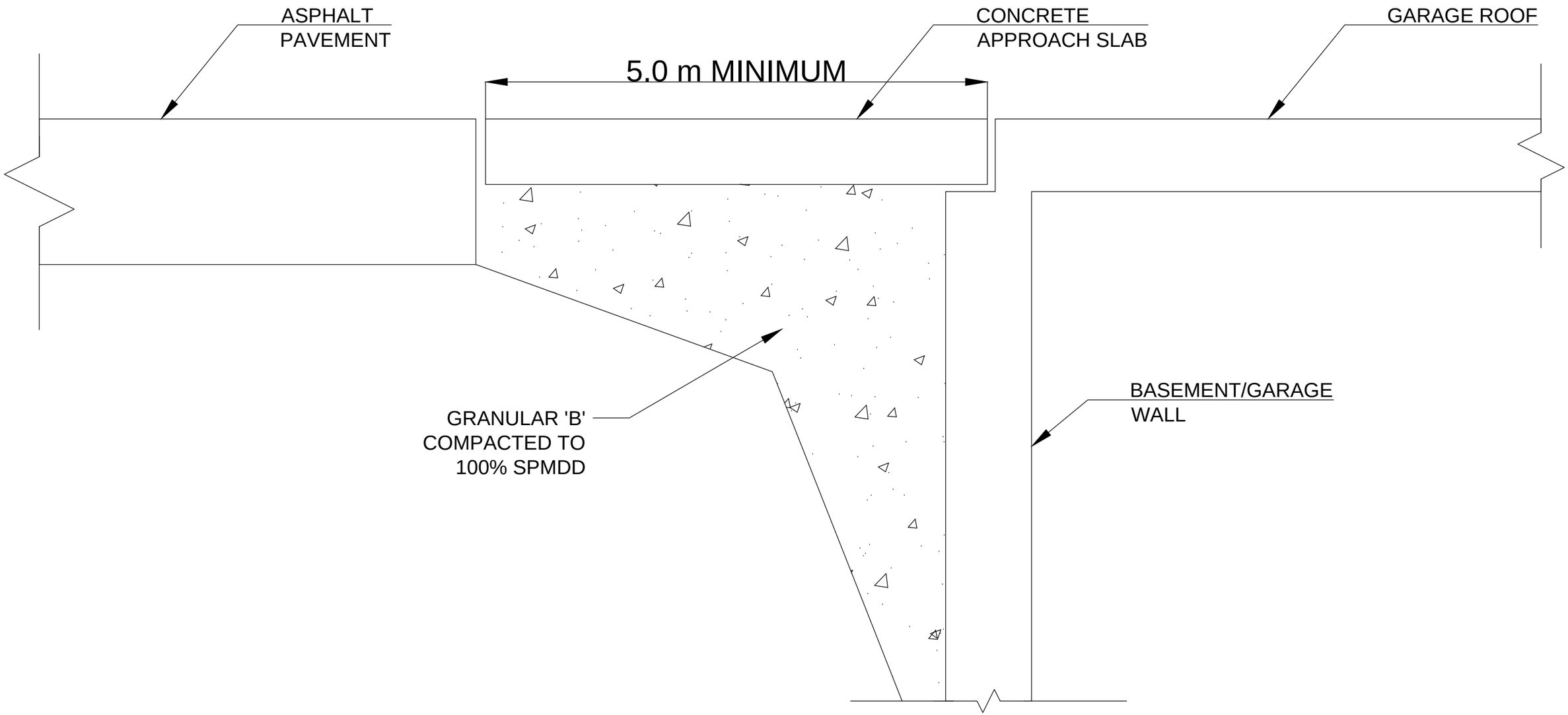
TYPICAL ELEVATOR PIT WATERPROOFING

Scale:	Drawing No.
NTS	4



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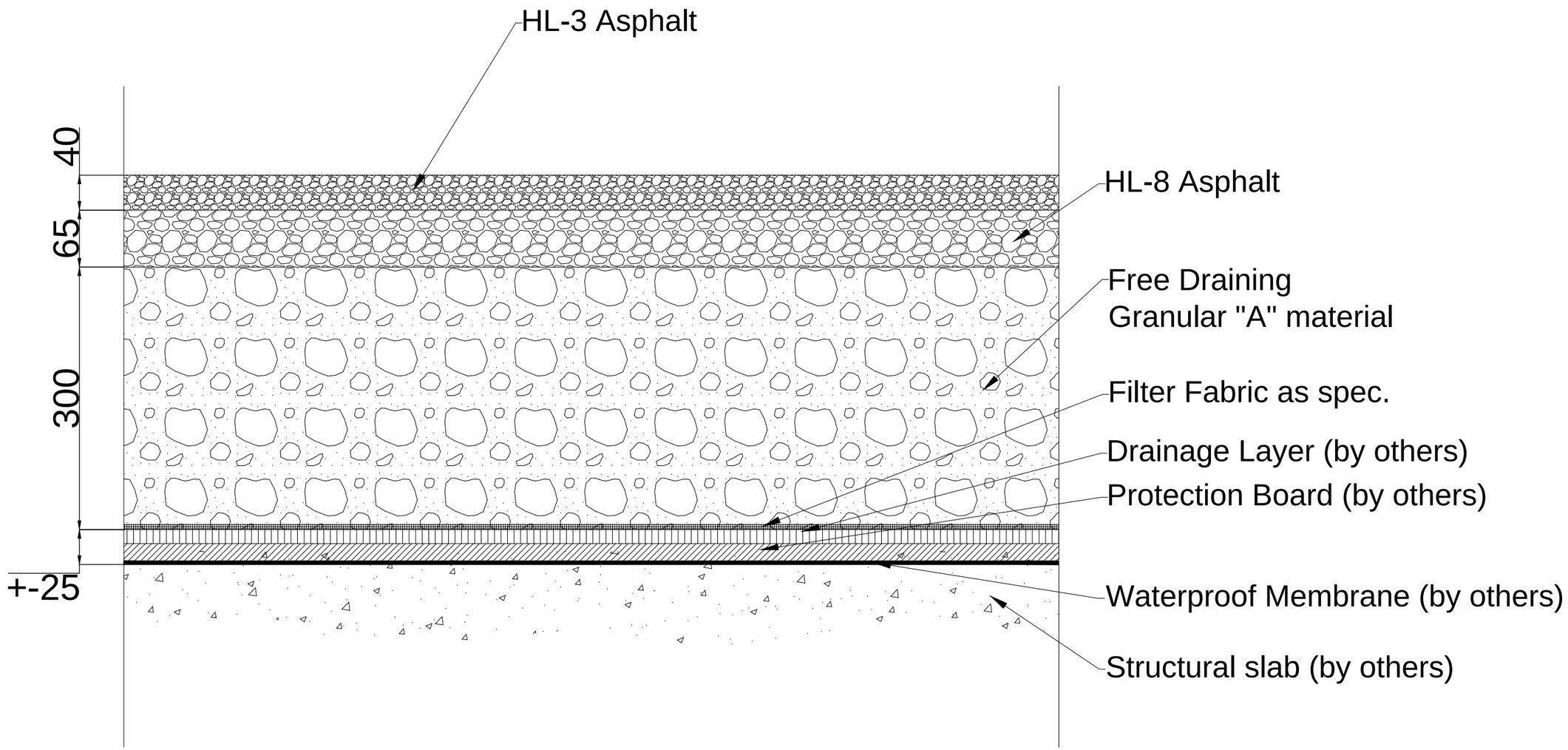
SUGGESTED APPROACH
SLAB DETAIL

Scale:	Drawing No.
NTS	5



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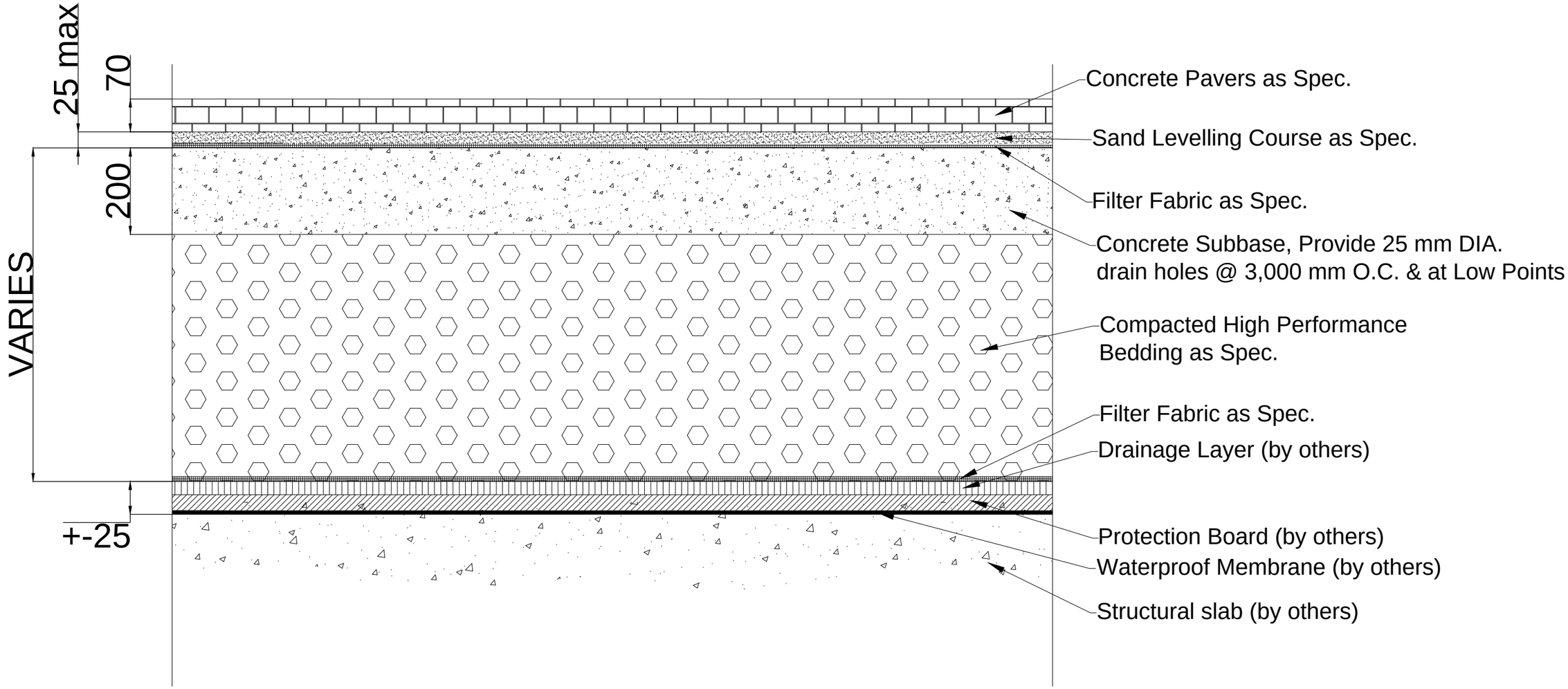
PAVEMENT ABOVE
GARAGE ROOF SLAB

Scale:	Drawing No.
NTS	6



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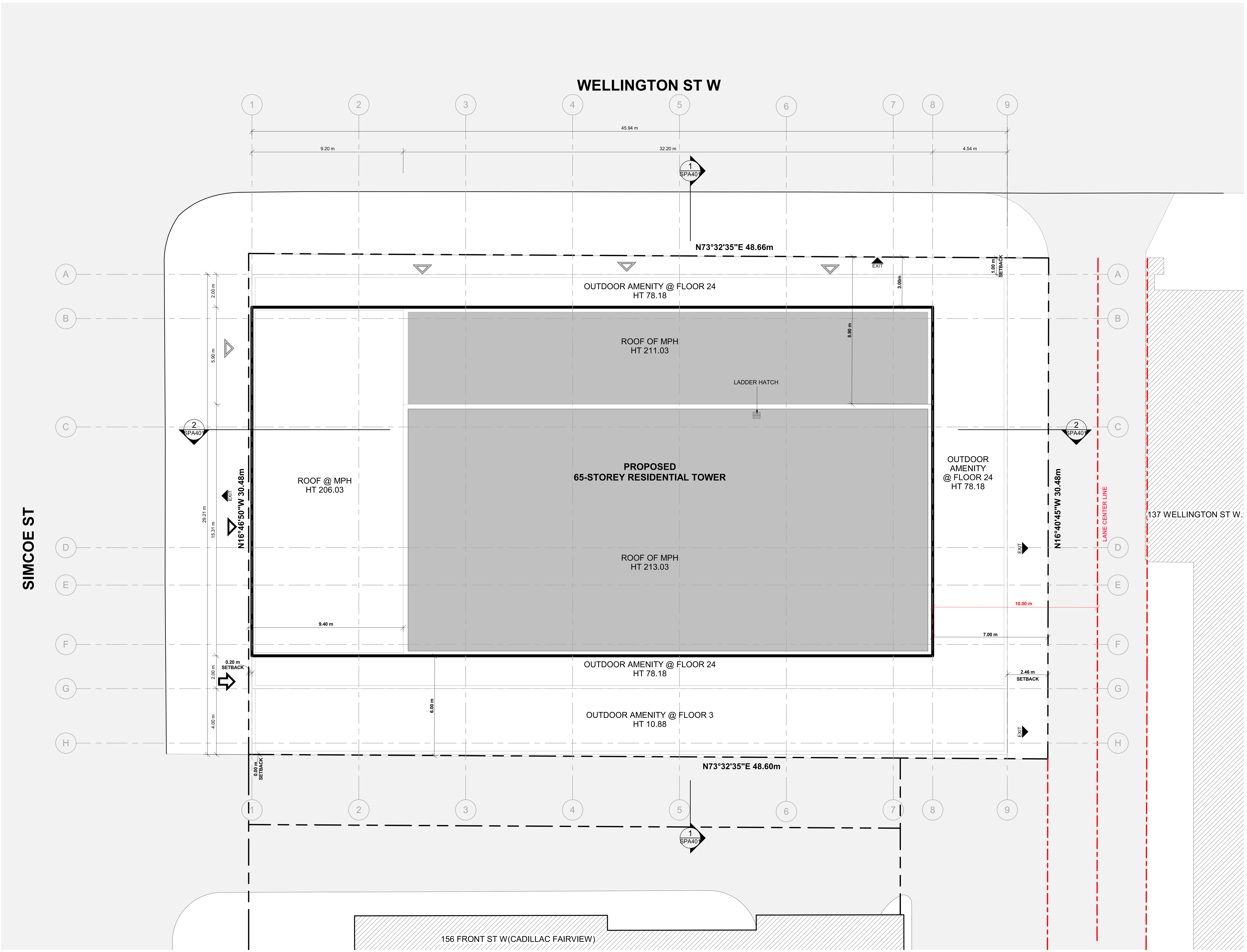
PAVEMENT ABOVE
GARAGE ROOF SLAB

Scale:	Drawing No.
NTS	7

APPENDIX A



TURNER FLEISCHER

Turner Fleischer Architects Inc.

67 Lesmill Road
Toronto, ON, M3B 2T8
T 416 425 2222
turnerfleischer.com

LEGEND

- PRIMARY RESIDENTIAL ENTRANCE
- RETAIL ENTRANCE
- SECONDARY RESIDENTIAL ENTRANCE
- EXIT
- SIAMESE CONNECTION
- CONVEX MIRROR
- FIRE HYDRANT

#	DATE	DESCRIPTION	BY
1	2024-04-10	Issued for PAC	LLE

HR REIT

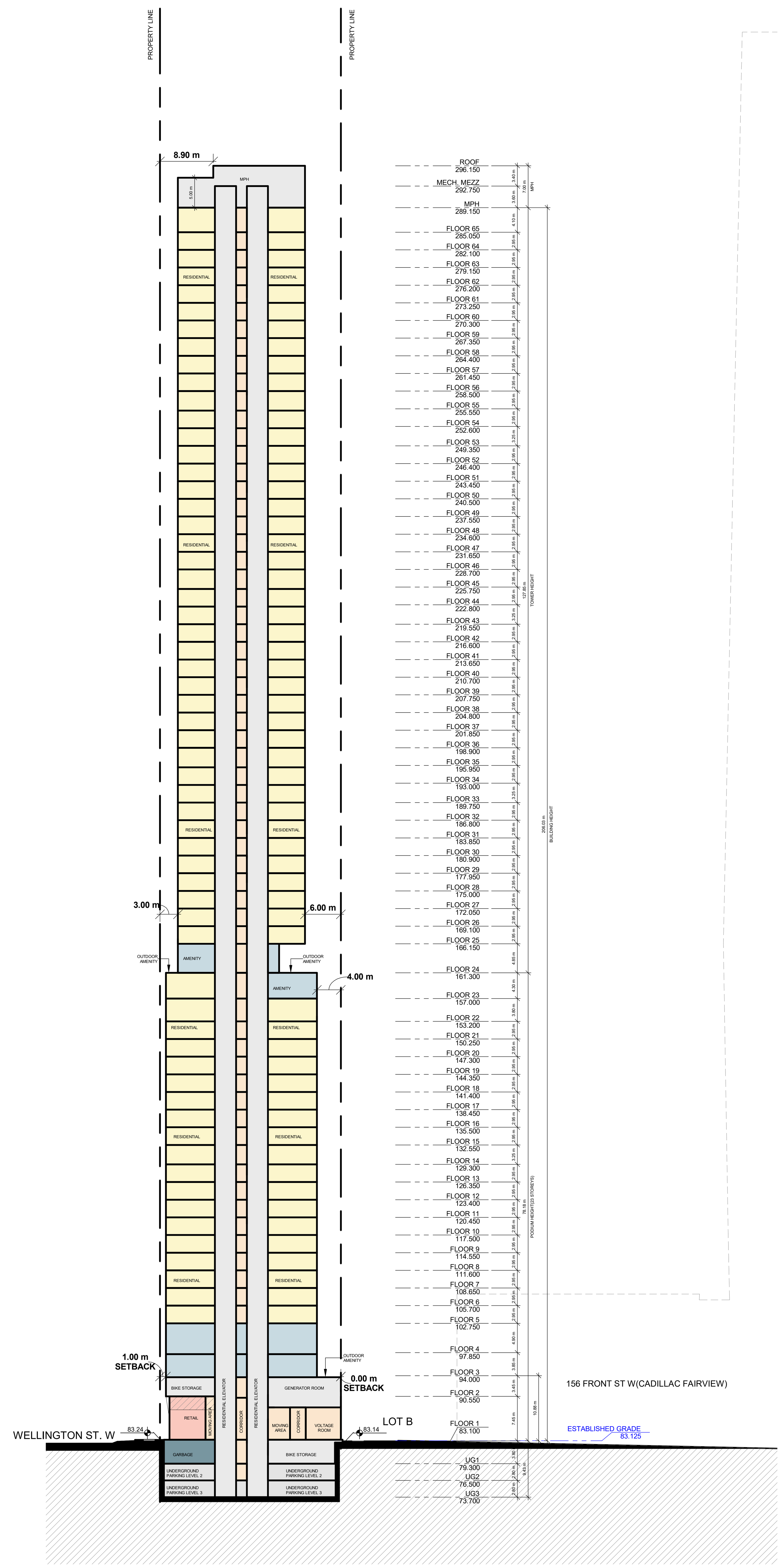
PROJECT
145 Wellington Street West
Toronto, ON

DRAWING
SITE PLAN / ROOF PLAN

PROJECT NO. 18.167P01
PROJECT DATE 2024-04-10
DRAWN BY RYT
CHECKED BY LLE
SCALE 1 : 100

DRAWING NO. SPA005
REV. 1

2024-04-12 1:27:08 PM



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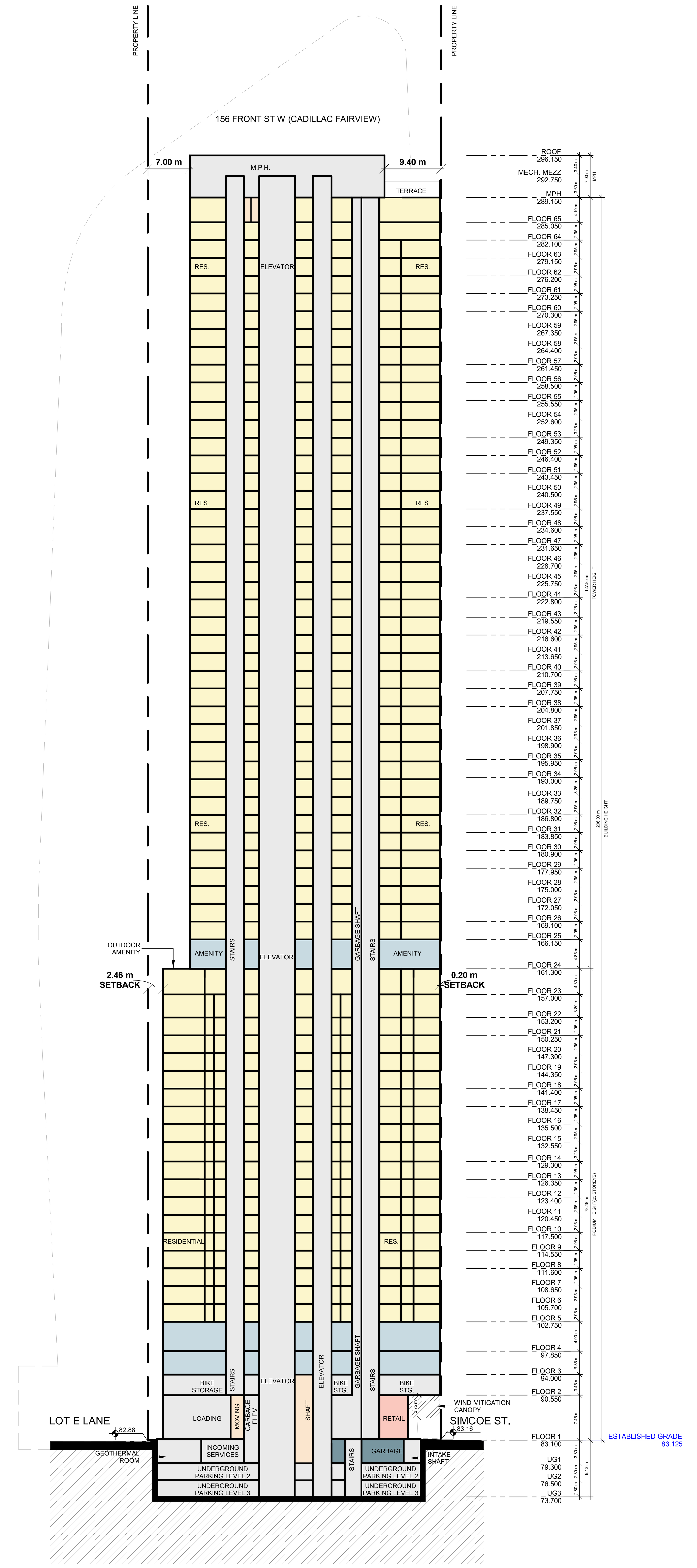
SPA401 1 : 500



2 EAST - WEST SECTION
SPA401 1 : 500

TURNER FLEISCHER

Turner Fleischer Architects Inc.

67 Lesmill Road
Toronto, ON, M3B 2T8
T 416 425 2222
turnerfleischer.com

#	DATE	DESCRIPTION	BY
1	2024-04-10	Issued for PAC	LLE

HR REIT

PROJECT
145 Wellington Street West
Toronto, ON

DRAWING
BUILDING SECTIONS

PROJECT NO. 18.167P01
PROJECT DATE 2024-04-10
DRAWN BY RYT
CHECKED BY LLE
SCALE 1 : 500

DRAWING NO. SPA401
REV. 1

2024-04-12 1:27:35 PM

APPENDIX B

RECORD OF BOREHOLE 1

PROJECT : GE5480
LOCATION : 145 Wellington Street West, Toronto, Ontario
STARTED : March 27, 2019
COMPLETED : March 28, 2019

MC CLYMONT & RAK ENGINEERS, INC.

SHEET 1 OF 1
DATUM Geodetic

DEPTH SCALE (metres)	BORING METHOD	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	ORGANIC VAPOUR READINGS (ppm) ⊗ 100 200 300 400 / % LEL - (hexane) □ 20 40 60 80	SHEAR STRENGTH: Cu, KPa nat V - ✚ rem V - ● Q - ✖ U - ▲ 20 40 60 80 / WATER CONTENT, PERCENT wp w wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		73.67							
		125 mm CONCRETE SLAB		73.54							Flush Mount Cover
		275 mm GRANULAR FILL: crushed limestone ("clear stone"), brown, moist, compact.		0.13	1	SS	>100	0 ⊗			
		SHALE: grey, moist. -weathered in the upper 800 mm.		73.27 0.40							Bentonite ▽ 1.52 m Long 50 mm ID PVC Riser 72.77
		-clay filled fissures at 1.2 m depth.			1	CO		0 ⊗			Silica Sand 72.15
					2	CO		0 ⊗			3.05 m Long 50 mm ID Well Screen
2	POWER BORING				3	CO		0 ⊗			
	ROCK CORING				4	CO		0 ⊗			
4					5	CO		0 ⊗			69.10
		End of Borehole		69.10 4.57							

Note:
1) Water level was not measured on completion of drilling due to use of water.
2) Combustible vapour reading was 0 ppm at 1.8 m depth in open borehole.
3) Soil samples were screened using a RKI Eagle gas meter with methane response mode off.
4) Water level was measured at 0.73 m bgs on April 4, 2019.

GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION — WATER LEVEL: 0.73 m bgs

▼ DEEP/DUAL INSTALLATION — WATER LEVEL:

LOGGED : FR
CHECKED : LM

MCR LOG ENVIRONMENTAL 5480.GPJ 5/9/19

RECORD OF BOREHOLE 2

PROJECT : GE5480
LOCATION : 145 Wellington Street West, Toronto, Ontario
STARTED : March 29, 2019
COMPLETED : March 29, 2019

MC CLYMONT & RAK ENGINEERS, INC.

SHEET 1 OF 1
DATUM Geodetic

DEPTH SCALE (metres)	BORING METHOD	DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	ORGANIC VAPOUR READINGS (ppm) 100 200 300 400 ⊗ / % LEL - (hexane) 20 40 60 80 □	SHEAR STRENGTH: Cu, KPa nat V - ✚ rem V - ● Q - ✖ U - ▲ 20 40 60 80 / WATER CONTENT, PERCENT wp w wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		73.67							
		125 mm CONCRETE SLAB		73.54							Flush Mount Cover
		300 mm GRANULAR FILL: crushed limestone ("clear stone"), brown, moist, compact.		0.13	1	SS	10	0			
				73.24							Bentonite
		SHALE: grey, moist. -weathered in the upper 750 mm.		0.43							1.52 m Long 50 mm ID PVC Riser 72.77
											Silica Sand
					1	CO		0			72.15
					2	CO		0			3.05 m Long 50 mm ID Well Screen
2	POWER BORING										
	ROCK CORING				2	CO		0			
					3	CO		0			
4					4	CO		0			▽
				69.10							69.10
		End of Borehole		4.57							

Note:
1) Water level was not measured on completion of drilling due to use of water.
2) Combustible vapour reading was 0 ppm at 1.8 m depth in open borehole.
3) Soil samples were screened using a RKI Eagle gas meter with methane response mode off.
4) Water level was measured at 4.28 m bgs on April 4, 2019.

GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION
WATER LEVEL: 4.28 m bgs

▼ DEEP/DUAL INSTALLATION
WATER LEVEL:

LOGGED : FR
CHECKED : LM

MCR LOG ENVIRONMENTAL 5480.GPJ 5/9/19

RECORD OF BOREHOLE 3

PROJECT : GE5480
LOCATION : 145 Wellington Street West, Toronto, Ontario
STARTED : March 31, 2019
COMPLETED : April 3, 2019

MC CLYMONT & RAK ENGINEERS, INC.

SHEET 1 OF 1
DATUM Geodetic

DEPTH SCALE (metres)	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	ORGANIC VAPOUR READINGS (ppm) 100 200 300 400 ⊗ / % LEL - (hexane) 20 40 60 80 □	SHEAR STRENGTH: Cu, KPa nat V - ✚ rem V - ● Q - ✖ U - ▲ 20 40 60 80 / WATER CONTENT, PERCENT wp w wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		73.64							
		125 mm CONCRETE SLAB		73.51							Flush Mount Cover
		SHALE: grey, moist. -weathered in the upper 950 mm.		0.13	1	SS	>100	0			Bentonite
	POWER BORING / ROCK CORING				1	CO		0			1.52 m Long 50 mm ID PVC Riser 72.74
2					2	CO		0			Silica Sand 72.12
					3	CO		0			3.05 m Long 50 mm ID Well Screen
4					4	CO		0			69.07
		End of Borehole		69.02 4.62							

Note:
1) Water level was not measured on completion of drilling due to use of water.
2) Combustible vapour reading was 0 ppm at 1.8 m depth in open borehole.
3) Soil samples were screened using a RKI Eagle gas meter with methane response mode off.
4) Water level was measured at 1.17 m bgs on April 4, 2019.

GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION — WATER LEVEL: 1.17 m bgs

▼ DEEP/DUAL INSTALLATION — WATER LEVEL:

LOGGED : FR
CHECKED : LM

MCR LOG ENVIRONMENTAL 5480.GPJ 5/9/19

APPENDIX C

[Company Letterhead]

[Company Name]

[Property Owner Name and Contact Information]

[Date DD/MMM/YYYY]

Attention: Executive Director, Engineering and Construction Services
c/o Manager, Development Engineering
[ADDRESS]

cc: General Manager, Toronto Water
c/o Manager, Environmental Monitoring and Protection Unit
30 Dee Ave, Toronto ON M9N 1S9

Dear Sir or Madam,

I _____, confirm and undertake that I will maintain all building(s) on the subject lands (MUNICIPAL ADDRESS) in a manner which will not discharge, directly or indirectly, any private water collected from subsurface drainage system consisting of but not limited to weeping tile(s), foundation drain(s), private water collection sump(s), private water pump or any combination thereof for the disposal of private water to a private sewer connection directly or indirectly or drainage system for disposal directly or indirectly in a municipal sewer. All the water collected in the subdrainage collection system will be managed onsite all time via infiltration gallery/dry well. There will be no direct or indirect discharge of private water to City's sewer.

I am aware of MOECC and OBC requirements regarding infiltration gallery/dry well.

_____
Name (printed) and Title

_____
Email

_____
Signature

I, [PRINT NAME], have the authority to bind the corporation.

[Company Letterhead]

[Company Name]

[Property Owner Name and Contact Information]

[Date DD/MMM/YYYY]

Attention: Executive Director, Engineering and Construction Services
c/o Manager, Development Engineering
[ADDRESS]

cc: General Manager, Toronto Water
c/o Manager, Environmental Monitoring and Protection Unit
30 Dee Ave, Toronto ON M9N 1S9

Dear Sir or Madam,

I _____, confirm and undertake that I will construct and maintain all building(s) on the subject lands (MUNICIPAL ADDRESS) in a manner which shall be completely water-tight below grade and resistant to hydrostatic pressure without any necessity for Private Water Drainage System (subsurface drainage system) consisting of but not limited to weeping tile(s), foundation drain(s), private water collection sump(s), private water pump or any combination thereof for the disposal of private water on the surface of the ground or to a private sewer connection directly or indirectly or drainage system for disposal directly or indirectly in a municipal sewer.

_____
Name (printed) and Title

_____
Email

_____
Signature

I, [PRINT NAME], have the authority to bind the corporation.

APPENDIX D

Pavement Structural Design Matrix – Minimum Requirements
City of Toronto

		30,000		40, 000		50,000		60,000		75,000	
		30 MPa	50 MPa	30 MPa	50 MPa	30 MPa	50 MPa	30 MPa	50 MPa	30 MPa	50 MPa
Major Arterial	Non-Truck Routes (5% Commercial Vehicles)	40 mm SP12.5 FC2 D 110 mm SP19.0 D 150 mm Granular A 200 mm Granular B* 500 mm Total	40 mm SP12.5 FC2 D 90 mm SP19.0 D 150 mm Granular A 200 mm Granular B* 480 mm Total	40 mm SP12.5 FC2 D 125 mm SP19.0 D 150 mm Granular A 200 mm Granular B* 515 mm Total	40 mm SP12.5 FC2 D 100 mm SP19.0 D 150 mm Granular A 200 mm Granular B* 490 mm Total	40 mm SP12.5 FC2 D 135 mm SP19.0 D 150 mm Granular A 200 mm Granular B* 525 mm Total	40 mm SP12.5 FC2 D 110mm SP19.0 D 150 mm Granular A 200 mm Granular B* 500 mm Total	40 mm SP12.5 FC2 D 145 mm SP19.0 D 150 mm Granular A 200 mm Granular B* 535 mm Total	40 mm SP12.5 FC2 D 120 mm SP19.0 D 150 mm Granular A 200 mm Granular B* 510 mm Total	40 mm SP12.5 FC2 D 155mm SP19.0 D 150 mm Granular A 200 mm Granular B* 545 mm Total	40 mm SP12.5 FC2 D 125 mm SP19.0 D 150 mm Granular A 200 mm Granular B* 515 mm Total
	Truck Routes (7.5% Commercial Vehicles)	40 mm SP12.5 FC2 D 130 mm SP19.0 D 150 mm Granular A 250 mm Granular B* 570 mm Total	40 mm SP12.5 FC2 D 110 mm SP19.0 D 150 mm Granular A 250 mm Granular B* 550 mm Total	40 mm SP12.5 FC2 D 150 mm SP19.0 D 150 mm Granular A 250 mm Granular B 590 mm Total	40 mm SP12.5 FC2 D 130 mm SP19.0 D 150 mm Granular A 250 mm Granular B* 570 mm Total	40 mm SP12.5 FC2 D 155 mm SP19.0 D 150 mm Granular A 250 mm Granular B* 595 mm Total	40 mm SP12.5 FC2 D 135 mm SP19.0 D 150 mm Granular A 250 mm Granular B* 575 mm Total	40 mm SP12.5 FC2 E 160 mm SP19.0 E 150mm Granular A 250 mm Granular B* 600 mm Total	40 mm SP12.5 FC2 E 145 mm SP19.0 E 150 mm Granular A 250 mm Granular B* 585 mm Total	40 mm SP12.5 FC2 E 170 mm SP19.0 E 150 mm Granular A 250 mm Granular B* 610 mm total	40 mm SP12.5 FC2 E 150 mm SP19.0 E 150 mm Granular A 250 mm Granular B* 590 mm Total
	Truck Routes (10% Commercial Vehicles)	40 mm SP12.5 FC2 D 150 mm SP19.0 D 150 mm Granular A 250 mm Granular B* 590 mm Total`	40 mm SP12.5 FC2 D 125 mm SP19.0 D 150 mm Granular A 250 mm Granular B* 565 mm Total	40 mm SP12.5 FC2 D 160 mm SP19.0 D 150 mm Granular A 250 mm Granular B* 600 mm Total	40 mm SP12.5 FC2 D 135 mm SP19.0 D 150 mm Granular A 250 mm Granular B* 575 mm Total	40 mm SP12.5 FC2 E 170 mm SP19.0 E 150 mm Granular A 250 mm Granular B 610 mm Total	40 mm SP12.5 FC2 E 145 mm SP19.0 E 150 mm Granular A 250 mm Granular B* 585 mm Total	40 mm SP12.5 FC2 E 175 mm SP19.0 E 150 mm Granular A 250 mm Granular B* 615 mm Total	40 mm SP12.5 FC2 E 155 mm SP19.0 E 150 mm Granular A 250 mm Granular B* 595 mm Total	40 mm SP12.5 FC2 E 185 mm SP19.0 E 150 mm Granular A 250 mm Granular B* 625 mm Total	40 mm SP12.5 FC2 E 165 mm SP19.0 E 150 mm Granular A 250 mm Granular B* 605 mm Total

		20,000		25,000	
		30 MPa	50 MPa	30 MPa	50 MPa
Minor Arterial	Non-Truck Routes (4% Commercial Vehicles)	40 mm SP12.5 FC1 C 95 mm SP19.0 D 150 mm ranular A 150 mm Granular B* 435 mm Total	40 mm SP12.5 FC1 C 80 mm SP19.0 D 150 mm Granular A 150 mm Granular B* 420 mm Total	40 mm SP12.5 FC1 C 105 mm SP19.0 D 150 mm Granular A 150 mm Granular B* 445 mm Total	40 mm SP12.5 FC1 C 85 mm SP19.0 D 150 mm Granular A 150 mm Granular B* 425 mm Total
	Truck Routes (7.5% Commercial Vehicles)	40 mm SP12.5 FC1 C 135 mm SP19.0 D 150 mm Granular A 150 mm Granular B* 475 mm Total	40 mm SP12.5 FC1 C 110 mm SP19.0 D 150 mm Granular A 150 mm Granular B* 450 mm Total	40 mm SP12.5 FC1 C 140 mm SP19.0 D 150 mm Granular A 150 mm Granular B* 480 mm Total	40 mm SP12.5 FC1 C 120 mm SP19.0 D 150 mm Granular A 150 mm Granular B* 460 mm Total

		5,000		7,500		10,000		15,000	
		30 MPa	50 MPa	30 MPa	50 MPa	30 MPa	50 MPa	30 MPa	50 MPa
Collector	Comm./Ind. (5% Commercial Vehicles)			40 mm SP12.5 B 105 mm SP19.0 B 150 mm Granular A 150 mm Granular B* 445 mm Total	40 mm SP12.5 B 75 mm SP19.0 B 150 mm Granular A 150 mm Granular B* 415 mm Total	40 mm SP12.5 B 115 mm SP19.0 B 150 mm Granular A 150 mm Granular B* 455 mm Total	40 mm SP12.5 B 85 mm SP19.0 B 150 mm Granular A 150 mm Granular B* 425 mm Total	40 mm SP12.5 B 125 mm SP19.0 B 150 mm Granular A 150 mm Granular B* 465 mm Total	40 mm SP12.5 B 95 mm SP19.0 B 150 mm Granular A 150 mm Granular B* 435 mm Total
	Residential (3% Commercial Vehicles)	40 mm SP12.5 B 70 mm SP19.0 B 150 mm Granular A 150 mm Granular B* 410 mm Total	40 mm SP12.5 B 60 mm SP19.0 B 150 mm Granular A 150 mm Granular B* 400 mm Total	40 mm SP12.5 B 85 mm SP19.0 B 150 mm Granular A 150 mm Granular B* 425 mm Total	40 mm SP12.5 B 60 mm SP19.0 B 150 mm Granular A 150 mm Granular B* 400 mm Total	40 mm SP12.5 B 95 mm SP19.0 B 150 mm Granular A 150 mm Granular B* 435 mm Total	40 mm SP12.5 B 60 mm SP19.0 B 150 mm Granular A 150 mm Granular B* 400 mm Total		

	2,500		3,000		4,500	
	Local Residential (3% Commercial Vehicles)		Local Industrial (10% Commercial Vehicles)		Local Throughway (3% Commercial Vehicles)	
	30 MPa	50 MPa	30 MPa	50 MPa	30 MPa	50 MPa
Local	40 mm SP12.5 B 60 mm SP19.0 B 150 mm Granular A 150 mm Granular B* 400 mm Total		40 mm SP12.5 B 80 mm SP19.0 B 150 mm Granular A 150 mm Granular B* 420 mm Total	40 mm SP12.5 B 60 mm SP19.0 B 150 mm Granular A 150 mm Granular B* 400 mm Total	40 mm SP12.5 B 60 mm SP19.0 B 150 mm Granular A 150 mm Granular B* 400 mm Total	

	All Traffic & Subgrade
Composite Pavements	
Major Arterial	40 mm Surface layer** 50 mm Base layer** 250 mm PCC Concrete 150 mm Granular A 490 mm Total
Minor Arterial - Bus/Truck Route	40 mm SP12.5 FC1 C 50 mm SP19.0 D 250 mm PCC Concrete 150 mm Granular A 490 mm Total
Local Collector - Bus/Truck Route	50 mm SP12.5 B 200 mm PCC Concrete 150 mm Granular A 400 mm Total
Local Collector - Non Bus/Truck Route	50 mm SP12.5 B 150 mm PCC Concrete 150 mm Granular A 350 mm Total

Notes:

AADT
Subgrade

* Subbase is Granular B - Type II as specified in TS 1010
** Surface and base layer asphalt mixe types for Major Arterial composite pavements should be selected based on the AADT as prescribed for flexible pavements

Designers shall use the pavement structural design matrix as minimum layer thickness during pavement design and analysis. Layer thickness may need to be increased based on the higher traffic volume, higher truck percentage or in situ resilient modulus.