PRELIMINARY

GEOTECHNICAL INVESTIGATION REPORT

FORA DEVELOPMENTS

PROPOSED RESIDENTIAL DEVELOPMENT

2634, 2636, 2640, 2642, and 2654 Eglinton Avenue West
and 1856 and 1856A Keele Street
Toronto, Ontario

November 3, 2022

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1.0 INTRODUCTION

Terrapex Environmental Ltd. (**Terrapex**) has been retained by Fora Developments to prepare a geotechnical report for the proposed residential development located at 2634, 2636, 2640, 2642, and 2654 Eglinton Avenue West and 1856 and 1856A Keele Street in Toronto, Ontario (the Site). Authorization to proceed with this study was given by Mr. Lyle Levine of Fora Developments.

The Site is located at the northwest corner of the intersection of Eglinton Avenue West and Keele Street. Majority of the Site is occupied by one and two storey buildings with single level basements.

We understand that it is proposed to redevelop the Site with a 33 storey residential building constructed over three underground parking garage levels. Based on the architectural drawings prepared by gh3*, dated October 26, 2022, the floor slab of the lowest parking garage level will be situated 11 to 13 m below grade.

The purpose of this report was to characterize the subsurface conditions, to determine the engineering properties of encountered soils, and based on this data to provide geotechnical engineering recommendations pertaining to the proposed development.

Terrapex used information included in the log of a borehole advanced at the site by Terrapex, as well as information available from logs of boreholes advanced by others directly north of the Site at 1860 and 1868 Keele Street for preparation of this report. The conclusions and recommendations provided in this report are subject to confirmation by Terrapex through additional boreholes advanced at the Site by Terrapex upon demolition of the Site buildings.

This report is intended for the guidance of the client and the design architects or engineers only. It is assumed that the design will be in accordance with the applicable building codes and standards.

2.0 PREVIOUS INVESTIGATIONS

Terrapex advanced one borehole on October 24, 2022, near the northeast corner of the Site as part of a Phase Two Environmental Site Assessment. The borehole was advanced to a depth of 8.2 m.

Six boreholes were advanced at the property to the north during June 2019 and July 2021. Three of the boreholes were extended to bedrock at an approximate depth of 30 m below grade, followed by rock coring to a depth of 34 m below grade. The remaining three boreholes were extended to depths ranging from 12.8 to 15.8 m. All six boreholes were instrumented with monitoring wells.

Standard penetration tests (SPT) were carried out in the course of advancing the boreholes to take representative soil samples and to measure penetration index values (N-values) to characterize the condition of the various soil materials. Pressuremeter testing was carried out in one of the six boreholes to determine the deformation properties of the subsoil. Selected soil samples were subjected for grain-size analyses and Atterberg Limits testing.

3.0 SITE AND SUBSURFACE CONDITIONS

The following paragraphs present a description of the Site and a commentary on the engineering properties of the various soil materials contacted in the boreholes.

It should be noted that the boundaries of soil types indicated on the borehole logs are inferred from non-continuous soil sampling and observations made during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design, and therefore, should not be construed as exact planes of geological change.

3.1 SITE DESCRIPTION

The Site is located at the northwest corner of the intersection of Eglinton Avenue West and Keele Street, and measures approximately 43 m by 25 m. The west section of the Site is occupied by a single storey retail commercial building with the municipal address 2654 Eglinton Avenue West, the east section of the Site is occupied by a two storey retail commercial / residential building with the municipal addresses of 2634, 2636, 2640, 2642, 2654 Eglinton Avenue West and 1856 and 1856A Keele Street. Both buildings contain basement levels.

The Site is bounded by the Keelesdale Subway Station building to the west, Eglinton Avenue West underlain by the Metrolinx Subway tunnel to the south, a laneway to the north and by Keele Street to the east.

3.2 SUBSURFACE CONDITIONS

The borehole advanced near the northeast corner of the site by Terrapex revealed that the asphaltic concrete pavement is underlain by fill soil which extends to a depth of 2.3 m. The fill is underlain by loose to compact moist brown silty sand and silt which extends to a depth of 6.3 m. The sand was observed to be wet below a depth of 3.8 m. The silt is underlain by clayey silt till which extends to the explored depth of the borehole at 8.2 m below grade.

The boreholes advanced at the site to the north reveal that the uppermost stratum of the soil profile consists of fill material which extends to depths ranging from 0.8 to 6.1 m below grade. The fill consists of loose to compact silty sand which includes clay, gravel and foreign matter, including, organics, cinder, wood, and plastics. The native soils underlying the fill consist of a thin layer of sand followed by grey clayey silt and sandy silt glacial tills, which extend to bedrock at an approximate depth of 30 m below grade. An approximately 6 m thick layer of sand was encountered in one of the boreholes between the bedrock and the glacial till.

The native soil is generally grey in colour.

Standard penetration resistance in the till soils provided N-values ranging from 11 to 50 blows for 75 mm penetration, indicating that the compactness condition / consistency of the native soils ranged from compact to very dense or stiff to hard.

The bedrock cores revealed that the uppermost approximately 1.2 m thick zone of the bedrock is weathered.

Groundwater was measured in all boreholes at depths ranging from 2.2 to 5 m below grade.

4.0 DISCUSSION AND RECOMMENDATIONS

The following discussions and recommendations are based on the factual data obtained from the boreholes advanced at the property located north of the subject Site by others. The recommendations are intended for use by the client, and design architects and engineers only and are subject to confirmation by Terrapex through boreholes advanced at the Site by Terrapex upon demolition of the buildings at the Site.

We understand that the Site will be redeveloped with a 33 storey residential building constructed over three underground parking garage levels. The floor slab of the lowest parking garage level will be situated approximately 11 to 13 m below grade.

Contractors bidding on this project or conducting work associated with this project should make their own interpretation of the factual data and/or carry out their own investigations.

4.1 EXCAVATION

Based on the borehole findings, excavation for basement and foundations is not expected to pose any difficulty. Excavation of the soils at this site can be carried out with hydraulic excavators.

All excavations must be carried out in accordance with Occupational Health and Safety Act (OHSA). With respect to OHSA, the near surface fill materials are expected to conform to Type 3 soil classification, as is the sandy soil below the fill situated above the water table. The glacial tills should conform to Type 2 classification. Wet sandy soils which are positioned below the groundwater table are expected to flow in open excavations and are classified as Type 4 soils.

Temporary excavations should not exceed 1.0 horizontal to 1.0 vertical. In the event very loose and/or soft soils are encountered at shallow depths or within zones of persistent seepage, it will be necessary to flatten the side slopes as necessary to achieve stable conditions. Side slopes of excavations extending below the water table in the wet sandy soils (Type 4 soil) should not be any steeper than 3 horizontal to 1 vertical.

For excavations through multiple soil types, the side slope geometry is governed by the soil with the highest number designation. Excavation side-slopes should not be unduly left exposed to inclement weather

Where workers must enter excavations extending deeper than 1.2 m, the excavation side-walls must be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects.

We understand that the proposed underground parking garage levels will span the entire area of the Site and sufficient space will not be available to slope the sidewalls of the basement excavation; as such it will be necessary to shore the basement excavation walls. Shoring recommendations are provided in Section 4.7 of this report.

Where space permits, temporary open cut may be used for basement excavations. The safe side slope angle for open excavations should conform to the Occupational Health and Safety Act requirements.

4.2 REUSE OF ON-SITE EXCAVATED SOIL

On-site excavated native soils which are above the groundwater table are considered suitable for reuse as backfill material, provided their water content is within 2% of their optimum water contents (OWC) as determined by Standard Proctor test, and the materials are effectively compacted with a heavy compaction rollers.

While the quality of the native soils are considered suitable for backfilling; the moisture content of the soils and the lift thickness for compaction must be properly controlled during the backfilling. Alternatively, imported suitable material should be used.

4.3 GROUNDWATER CONTROL

Based on the architectural drawings, the lowest floor slab of the parking garage will be situated approximately 11 to 13 m below grade. The foundations are anticipated to extend to depth of 13 to 14 m below grade. The groundwater table at the Site is situated approximately 4 m below grade.

It is anticipated that adequate control of any groundwater seepage during basement and foundation excavations can be achieved by pumping from filtered sumps in the base of the excavation. It will be necessary to lower the groundwater level at least 0.5 m below the proposed excavation level in advance of excavation.

The contractor should make their own assessment for temporary control of groundwater seepage into the excavation, as well as to maintain basal stability of the subgrade during the foundation construction stage.

Surface water should be directed away from open excavations.

It will be necessary to determine the construction dewatering requirements and to collect the information required for the application for Permit to Take Water (PTTW). The hydrogeological report should be referred to in this regard.

4.4 FOUNDATION DESIGN

The proposed development will consist of a 33-storey building constructed over 3 levels of underground parking garage. The floor slab of the lowest parking garage level will be situated approximately 11 to 13 mbgs. It is anticipated that the foundation will be founded about 2 to 3 m below, approximately 15 to 16 mbgs, about 10 to 12 m below the groundwater table.

As discharge of groundwater from the building to the City sewer system is no longer permitted, it will be necessary to waterproof the substructure of the building and accordingly it will be necessary to use a raft slab/foundation to resist the hydrostatic uplift forces and support the building. It is anticipated that the base of the raft foundation will be situated at about 13 to 14 m below grade. It will be necessary to maintain the water table below the base of the excavation at all times during construction of the foundation, and until such time when the raft slab is sufficiently loaded to prevent its uplift resulting from hydrostatic forces.

Based on the findings of the boreholes, the soil at the proposed base of the raft will consist of very stiff to hard clayey silt till.

The raft foundation can be designed for Serviceability Limit State soil resistances of 200 kPa and 250 kPa for total settlements of 25 mm and 50 mm, respectively. These soil resistances are unlikely to be sufficient to support the 33 storey building. Furthermore, if the raft foundation is used to support the building it will result in settlement of the soils below the subway tunnel, which will not be permitted by Metrolinx. Accordingly, it is anticipated that it will be necessary to utilize caissons extended to bedrock to support the raft slab and the building.

Caissons extended through the weathered portion of the bedrock and founded approximately 2 m into the rock may be designed for a bearing resistance at Ultimate Limit States (ULS) of 8.0

MPa. The following factored shaft resistances for the caissons extended through sandy soils and socketed into shale bedrock can be used.

Material type (elevation)	Factored Shaft Resistance
Clayey and Sandy Silt Till	60kPa
Weathered Shale Bedrock (top 1.5 m of rock)	200 kPa
Intact Shale Bedrock	600 kPa

The caisson installation must be inspected by a qualified geotechnical engineer to ensure that they are constructed in accordance with the design intent. The contractor must take into consideration the excavation method to be used through soft and water bearing soils (continuous lines, mud drilling, etc.) and the concreting technique for installing caissons in accordance with good construction practice.

Caissons should be advanced to the top of sound shale bedrock and confirmed by the Geotechnical Engineer based on field observations. The caisson should then be further advanced a minimum of 1 m into the sound shale. The hole base should be cleaned using the auger and observed and approved by the Geotechnical Engineer.

Concrete should be placed to a minimum thickness of 600 mm in the caisson hole and mixed with the auger. The concrete should then be extracted from the caisson hole and disposed. Concrete placement for the caisson foundation may then proceed.

In the event that more than 150 mm of water is present in the base of the caisson hole, it will be necessary to place concrete using the tremie method to ensure proper placement of the concrete in water.

4.5 CONCRETE SLAB-ON-GRADE

As a raft foundation will be implemented, the floor slab will likely have to be constructed over a 500 to 600 mm thick layer of granular material such as 19 mm clear stone placed directly over the raft foundation to permit placement of sub-floor drainage piping and other utility lines.

4.6 LATERAL EARTH PRESSURE

Parameters used in the determination of earth pressure acting on temporary shoring or basement walls are defined below.

Parameter	Definition	Units
Φ'	angle of internal friction	degrees
γ	bulk unit weight of soil	KN/m^3
Ka	active earth pressure coefficient (Rankine)	dimensionless
Ko	at-rest earth pressure coefficient (Rankine)	dimensionless
Kp	passive earth pressure coefficient (Rankine)	dimensionless

The appropriate un-factored values for use in the design of structures subject to unbalanced earth pressures at this site are tabulated as follows:

Soil	Parameter				
	Φ'	γ	Ka	Kp	Ko
Fill Materials	28°	20.0	0.36	2.77	0.53
Sand	compact - 32°	18.5	0.31	3.25	0.47
Clayey Silt and Sandy Silt Till	Very Stiff - 32° Hard - 34°	20.5 21.5	0.31 0.28	3.25 3.54	0.47 0.44

Subsurface walls that are subject to unbalanced earth and hydrostatic pressures must be designed to resist a pressure that can be calculated based on the following formula:

$$\mathbf{P = K\,[\gamma\,(h - h_w) + \gamma' h_w + q] + \gamma_w h_w}$$

where P = lateral pressure in kPa acting at a depth h (m) below ground surface

K = applicable lateral earth pressure coefficient

H = height at any point along the interface (m)

h_w = depth below the groundwater level at point of interest (m)

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

γ' = the submerged unit weight (kN/m^3) of exterior soil ($\gamma' = \gamma - \gamma_w$)

γ_w = unit weight of water, assume a value of 9.8 kN/m^3

q = the complete surcharge loading (kPa)

The coefficient of earth pressure at rest (Ko) should be used in the calculation of the earth pressure on the basement walls.

Resistance to sliding of earth retaining structures is developed by friction between the base of the footing and the soil. This friction (R) depends on the normal load on the soil contact (N) and the frictional resistance of the soil (tan Φ’) expressed as: R = N tan Φ’. This is an ultimate resistance value and does not contain a factor of safety.

4.7 SHORING DESIGN

Given that the basement walls of the proposed development will extend to the property limits, it will not be possible to slope the banks of the excavation, and it will be necessary to shore the basement excavation walls.

A soldier-pile and wood lagging wall system may be used as the shoring system for the basement excavation walls. A continuous caisson wall is anticipated to be required along the west and south foundation walls due to the presence of the subway tunnel and station structure. Consideration may be given to extending the caisson wall along the entire perimeter of the excavation. The caisson wall will minimize the volume of water seepage into the excavation.

Where space permits, temporary open cut may be used for basement excavations. The safe side slope angle for open excavations should conform to the Occupational Health and Safety Act requirements.

The design of temporary shoring for the support of the excavation walls must account for the presence of structures and buried services on the adjacent properties, and the existing subsurface conditions at the site.

The lateral restraining force for the shoring system may be provided by employing either rakers or tieback anchors. The latter is favorable because they do not protrude into the excavations as is the case with rakers. The use of tieback anchors will depend on whether permission is obtained to extend the anchors to the required distance on to the neighboring properties.

To prevent possible caving of soils, it will be necessary to drill the tie back anchors holes with hollow stem augers, and anchors installed and concrete/cement grout poured through the liner.

Provisions should be made to install temporary liners for the excavation of the soldier pile holes. The shoring contractor must also provide construction method(s) to overcome any groundwater seepage into the pile holes during excavation and subsequent concreting of the piles to comply with good construction practice.

The shoring design should be based on the procedure detailed in the latest edition of the CFEM. The active earth pressure coefficient; **Ka** to be used for the design of the shoring system, should be as follows:

= 0.50 where adjacent building footings or buried services fall within an envelope formed by a 60^0 line drawn from the base of the excavation wall to the ground surface

= 0.3 where adjacent building footings or buried services fall outside an envelope formed by a 60^0 line drawn from the base of the excavation wall to the ground surface

= 0.25 where adjacent building footings or buried services are outside an envelope formed by a 45^0 line drawn from the base of the excavation wall to the ground surface

Excavation for basement is anticipated to terminate in the clayey silt till deposit. The minimum depth of penetration (d) of soldier piles may be estimated from the following expression:

$$\mathbf{R = NB} \; (\tfrac{1}{2}\gamma \mathbf{d^2 K_p})$$

where **R** = required toe resistance

$\mathbf{K_p}$ = passive earth pressure coefficient

N = factor according to three dimensional effect around an isolated pile

B = diameter of concrete filled hole

d = required penetration depth

γ = bulk unit weight of soil

Raker footings should be designed in accordance with the design principals for shallow foundations subject to inclined loading. All raker footings should be located outside the zone of influence of the buried portion of soldier piles, and at a distance of no less than 1.5D from the piles, where D = Depth of penetration of the piles below the base of the excavation. No excavation should be made within two footings widths of the raker footings, on the side opposite the rakers.

Anchors extended into the clayey silt till soil may be designed based on soil/grout bond values of 60. This value depends on the anchor installation method and grouting procedures. Gravity poured concrete can result in low bond values, while pressure grouted anchors will give higher values and produce a more satisfactory anchor.

It will be necessary to perform load tests on the tiebacks to confirm the bond stresses assumed in the design of anchors.

Movement of the shoring system is inevitable. Vertical movements will result from the vertical loads on the soldier piles resulting from the inclined tiebacks and inward horizontal movement will result from the earth and water pressures. The magnitude of this movement can be controlled by sound construction practices. The lateral and vertical movement of the shoring system must be monitored especially at locations in which settlement sensitive structures are present, to ensure that movements are kept within acceptable range.

4.8 PAVEMENT DESIGN

The pavement above the parking garage roof slab may be comprised of a minimum of 75 mm thick layer of granular ‘A’ topped with asphaltic concrete having a minimum thickness of 80 mm (40 mm HL8 and 40 mm HL3). The asphaltic concrete materials should be rolled and compacted in accordance with OPSS 310 requirements.

The gradation and physical properties of HL-3 and HL-8 asphaltic concrete, and Granular ‘A’ shall conform to the OPSS standards. The granular courses of the pavement should be placed in lifts not exceeding 150 mm thick and be compacted to a minimum of 100% SPMDD.

The critical section of pavement will be at the transition between the pavement on grade and the pavement above the garage roof slab. To alleviate the detrimental effects of dynamic loading / settlement / pavement depression in the backfill to the rigid garage roof structure, it is recommended that an approach type slab be constructed at the entrance/exit points, by extending the granular sub-base to greater depths along the exterior garage wall.

4.9 EARTHQUAKE DESIGN PARAMETERS

The 2012 Ontario Building Code (OBC) stipulates the methodology for earthquake design analysis, as set out in Subsection 4.1.8.7. The determination of the type of analysis is predicated on the importance of the structure, the spectral response acceleration and the site classification.

The parameters for determination of the Site Classification for Seismic Site Response are set out in Table 4.1.8.4.A of the 2012 OBC. The classification is based on the determination of the average shear wave velocity in the top 30 metres of the site stratigraphy, where shear wave velocity (vs) measurements have been taken. In the absence of such measurements, the classification is estimated on the basis of empirical analysis of undrained shear strength or penetration resistance. The applicable penetration resistance is that which has been corrected to a rod energy efficiency of 60% of the theoretical maximum or the (N_{60}) value.

Based on the borehole information, the subsurface stratigraphy as revealed in the boreholes generally comprises surficial layer of fill materials, followed by predominantly compact silty sand, underlain by very stiff to hard clayey and sandy silt tills. Based on the above, and provided that the foundations would extend to bedrock, the site can be classified as "**Class C**" for Seismic Site Response.

The site specific 5% damped spectral acceleration coefficients, and the peak ground acceleration factors are provided in the 2012 Ontario Building Code - Supplementary Standards SB-1 (September 14, 2012), Table 1.2, location Toronto, Ontario.

5.0 CLOSURE

The conclusions and recommendations in this report are based partly on information obtained from inspection locations adjacent to the property, and are subject to confirmation by Terrapex through additional, deeper boreholes advanced at the Site by Terrapex upon demolition of the Site buildings. Soil and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction which could not be detected or anticipated at the time of the soil investigation.

The design recommendations given in this report are applicable only to the project described in the text, and then only if constructed substantially in accordance with details of alignment and elevations stated in the report. Since all details of the design may not be known to us, in our analysis certain assumptions had to be made as set out in this report. The actual conditions may, however, vary from those assumed, in which case changes and modifications may be required to our recommendations.

This report was prepared for Fora Developments by Terrapex Environmental Ltd. The material in it reflects Terrapex Environmental Ltd. judgement 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 which the Third Party may make based on it, are the sole responsibility of such Third Parties.

We recommend, therefore, that we be retained during the final design stage to review the design drawings and to verify that they are consistent with our recommendations or the assumptions made in our analysis. We recommend also that we be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the test holes. In cases when these recommendations are not followed, the company's responsibility is limited to accurately interpreting the conditions encountered at the test holes, only.

The comments given in this report on potential construction problems and possible methods are intended for the guidance of the design engineer, only. The number of inspection locations may not be sufficient to determine all the factors that may affect construction methods and costs.

The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work.

Respectfully submitted,

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