PATERSON GROUP

Geotechnical Desktop Review

Proposed High-Rise Development

4105 Kingston Road - Scarborough
Toronto, Ontario

Prepared for Fotenn Consultants Inc.

Report TG0160-1 Revision 2 dated October 31, 2025

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Fotenn Consultants Inc. to prepare a geotechnical desktop study for the proposed high-rise development to be located at 4105 Kingston Road in Scarborough within the City of Toronto, Ontario (reference should be made to Figure 1 - Key Plan in Appendix 2 of this report).

The present report provides a summary of the regional soil information at the subject site along with preliminary geotechnical recommendations pertaining to the future development of the site.

2.0 Proposed Development

Based on the preliminary conceptual plans provided by the client, it is anticipated that the proposed development will consist of 3 pairs of residential towers (Buildings A1/A2, B1/B2, and C1/C2), with each pair sharing a 7-storey mixed-use podium. It is understood that the proposed towers will range from approximately 30 to 60 storeys in height.

Further, the proposed development is anticipated to include up to three levels of shared underground parking which will occupy the majority of the subject site. Associated access lanes, at-grade parking, landscaped areas, and hardscaped zones are also expected. The development is anticipated to be municipally serviced.

3.0 Site Conditions

3.1 Surface Conditions

The subject site is occupied by an existing asphalt paved parking lot associated with the Guildwood GO railway station, which abuts the subject site immediately to the south. However, based on available aerial photographs, commercial buildings were located within the western half of the property as recently as 2002 and were no longer present in 2005.

The site is bordered to the west by Kingston Road, to the north by commercial developments and further by Kingston Road, to the east by residential dwellings and an asphalt paved parking area, and to the south by the Guildwood GO station. The ground surface across the subject site is relatively flat and at grade with the surrounding developments

3.2 Available Subsurface Information

Based on the existing site conditions and available subsurface information in proximity to the subject site, the general subsurface profile is anticipated to consist of asphaltic concrete overlying fill material, which is further underlain by a glacial till deposit.

From our review of available geological mapping, the site is located in an area where the bedrock consists of interbedded shale and limestone of the Georgian Bay Formation, with an overburden drift thickness of 50 to 60 m.

3.3 Groundwater

Considering the developed nature of the site and reviewing available well records in the vicinity of the site, the groundwater table is anticipated to range from approximate depths of 1.5 to 4 m below the existing ground surface. The groundwater table should be further evaluated by carrying out a site-specific groundwater monitoring program.

It should be noted that groundwater levels are subject to seasonal fluctuations and therefore could vary during the time of construction.

4.0 Geotechnical Discussion

4.1 Preliminary Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is recommended that the proposed high-rise buildings be founded on a raft foundation bearing on an undisturbed glacial till bearing surface, or a deep foundation consisting of rock socketed caissons, dependent on the magnitude of the structural loads.

However, it should be noted that a geotechnical investigation has not yet been completed at the subject site, therefore any recommendations provided herein are preliminary and need to be confirmed once the geotechnical investigation has been completed.

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

4.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

Existing foundation walls and other demolition debris should completely be removed from the proposed building perimeter and within the lateral support zones of the foundation. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

Protection of Subgrade

Where a raft foundation is utilized, the subgrade material is expected to consist of a glacial till deposit. Therefore, it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed glacial till subgrade shortly after the completion of the excavation. The main purpose of the mudslab is to reduce the risk of disturbance of the subgrade under the traffic of workers and equipment.

The final excavation to the raft bearing surface level and the placing of the concrete mud slab should be done in smaller sections to avoid exposing large areas of the glacial till to potential disturbance due to drying.

Compacted Granular Fill Working Platform – Deep Foundations

For proposed buildings to be supported on caissons, the use of heavy equipment will be required. It is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the equipment to access the site without getting stuck and causing significant disturbance to the subgrade.

A typical working platform could consist of 0.6 m of OPSS Granular B, Type II crushed stone which is placed and compacted to a minimum of 98% of its SPMDD in lifts not exceeding 300 mm in thickness.

Vibration Considerations

Construction operations can be the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipment could be a source of vibrations: caisson drill rig, compactor, dozer, crane, truck traffic, etc. Vibrations caused by construction operations could be the cause of the source of detrimental vibrations on the nearby buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz).

It should be noted that these guidelines are for today's construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed buildings.

Fill Placement

Fill placed for grading throughout the building footprint should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. Imported fill material should be tested and approved prior to delivery to the site. The fill should be placed in a maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 99% of the standard proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking areas where settlement of the ground surface is of minor concern. These materials should be spread in a maximum of 300 mm thick loose lifts and compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in maximum 300 mm thick loose lifts to at least 98% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

4.3 Preliminary Foundation Design

Conventional Shallow Foundation

Footings placed on an undisturbed, compact to dense glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **300 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **450 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

The bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

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

Raft Foundation

It is expected that a raft foundation may be used to support the proposed high-rise buildings. The maximum SLS contact pressure recommended is **500 kPa** for a raft foundation bearing on the undisturbed glacial till.

It should be noted that the weight of the raft slab and everything above has to be included when designing with this value. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The factored bearing resistance (contact pressure) at ULS can be taken as **750 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **20 MPa/m** for a contact pressure of **500 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A common method of modeling the soil structure interaction is to consider the bearing medium to be elastic and to assign a subgrade modulus. However, glacial till is not elastic and limits have to be placed on the stress ranges of a particular modulus.

The proposed building can be designed using the above parameters with total and differential settlements of 25 and 20 mm, respectively.
The bearing medium under shallow-foundation supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to the undisturbed, compact to dense glacial till above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as that of the bearing medium soil.

Deep Foundations

Rock socketed caissons can be constructed by advancing casing through the overburden soils to the bedrock surface (by vibrator or augering in advance of the casing), seating the casing into the bedrock and then continuing drilling to create a rock socket.

The following design values can be used for the design of rock socketed caissons:

- ❑ A factored socket shear resistance at SLS and ULS of **1,500 kPa** can be used for clean sound limestone bedrock sockets. This value incorporates a geotechnical resistance value of 0.4.

- ❑ End-bearing resistance at SLS and ULS of **3,000 kPa** can be used for clean sound limestone bedrock. This value incorporates a geotechnical resistance value of 0.5.

It is recommended that the specified concrete strength for the caissons be at least 30 MPa, in order that the socket shear values are not limited by the concrete strength. The reinforcement for the caissons should be designed by the structural engineer.

The caissons should be installed using casing which is properly sealed into the bedrock to permit dewatering and visual inspection of the bedrock bearing surface prior to the placement of concrete.

A testing program for caissons should be established prior to the installation of production caissons. Additional foundation design considerations for deep foundations will be provided following completion of the site specific geotechnical investigation.

4.4 Design for Earthquakes

The preliminary site class for seismic site response can be taken as **Site Designation X_D**. The Site Designation should be reviewed upon completion of the geotechnical investigation. If a higher seismic site class is required for the proposed buildings, a site-specific shear wave velocity test should be completed in accordance with the Ontario Building Code (OBC) 2024.

4.5 Basement Floor Slab

With the removal of all topsoil and deleterious materials within the footprint of the proposed building, a soil subgrade or engineered fill pad approved by Paterson personnel at the time of construction, is considered to be an acceptable subgrade surface on which to commence backfilling for the floor slab.

It is anticipated that the underground levels for the proposed buildings will primarily consist of parking areas and the rigid pavement structure recommendation provided in Section 5.8 will be applicable. However, if storage or other uses of the lower level are considered where a concrete floor slab will be constructed, the upper 300 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone.

Any soft areas in the basement slab subgrade should be removed and backfilled with appropriate backfill material prior to placing fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum 98% of the SPMDD.

In consideration of the groundwater conditions anticipated at the site and the depth of excavation, a sub-slab drainage system, consisting of lines of perforated drainage pipe sub-drains connected to a positive outlet, should be provided under the lowest level floor slab.

4.6 Basement Wall

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

Lateral Earth Pressure

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

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

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

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

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 a_c \cdot \gamma \cdot H^2/g$ where:

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

The peak ground acceleration, (a_{max}), for the subject site area is 0.248g according to Ontario Building Code 2024. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5 K_o \gamma H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

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

4.7 Pavement Design

Lowest Underground Parking Level

For design purposes, it is recommended that the rigid pavement structure for the lower underground parking level of the proposed buildings consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The following rigid pavement structure is recommended:

Table 1 – Recommended Rigid Pavement Structure – Lowest Underground Parking Level	
Thickness (mm)	**Material Description**
150	**Exposure Class C2 – 32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** – OPSS Granular A Crushed Stone
SUBGRADE – Existing imported fill, or OPSS Granular B Type **I** or **II** material placed over in bedrock.	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the lower underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example; a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Podium Deck Area

It is anticipated that the podium deck structure will be provided car only parking areas, access lanes, fire truck lanes and loading areas. Based on the concrete slab subgrade, the pavement structure indicated in the Tables 2 and 3 may be considered for design purposes:

Table 2 – Recommended Pavement Structure – Car-Only Parking Areas (Podium Deck)	
Thickness (mm)	**Material Description**
50	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
200**	**Base** – OPSS Granular A Crushed Stone
See Below*	**Thermal Break*** – Rigid insulation (See Paragraph Below)
n/a	**Waterproofing Membrane and Protection Board**
SUBGRADE – Reinforced Concrete Podium Deck *If specified by others, not required from a geotechnical perspective **Thickness is dependent on grade of insulation as noted in proceeding paragraph	

Table 3 – Recommended Pavement Structure – Access Lanes, Fire Truck Lanes, Ramp and Heavy Truck Parking Areas (Podium Deck)	
Thickness (mm)	**Material Description**
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Wear Course** – HL-8 or Superpave 19.0 Asphaltic Concrete
300**	**Base** – OPSS Granular A Crushed Stone
See Below*	**Thermal Break*** – Rigid insulation (See Paragraph Below)
n/a	**Waterproofing Membrane and IKO Protection Board**
SUBGRADE – Reinforced Concrete Podium Deck *If specified by others, not required from a geotechnical perspective **Thickness is dependent on grade of insulation as noted in proceeding paragraph	

The transition between the pavement structure over the podium deck subgrade and soil subgrade beyond the footprint of the podium deck is recommended to be transitioned to match the pavement structures provided in the following section. For this transition, a 5H:1V is recommended between the two subgrade surfaces. Further, the base layer thickness should be increased to a minimum thickness of 500 mm below the top of the podium slab a minimum of 1.5 m from the face of the foundation wall prior to providing the recommended taper.

Should the proposed podium deck be specified to be provided a thermal break by the use of a layer of rigid insulation below the pavement structure, its placement within the pavement structure is recommended to be as per the above-noted tables. The layer of rigid insulation is recommended to consist of a DOW Chemical High-Load 100 (HI-100), High-Load 60 (HI-60) or High Load (HI-40). The pavement structures base layer thickness will be dependent on the grade of insulation considered for this project and should be reassessed by the geotechnical consultant once pertinent design details have been prepared.

The higher grades of insulation have more resistance to deformation under wheel-loading and require less granular cover to avoid being crushing by vehicular loading. It should be noted that SM (Styrofoam) rigid insulation is not considered suitable for this application.

Pavement Structure on Overburden Soils

Beyond the podium deck, the following pavement structures may be considered for car only parking and heavy traffic areas. The proposed pavement structures are shown in Tables 4 and 5 on the following page.

Table 4 – Recommended Pavement Structure – Car Only Parking Areas

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

Table 5 – Recommended Pavement Structure – Access Lanes and Heavy Loading Area

Thickness (mm)	Material Description
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** – HL-8 or Superpave 19 Asphaltic Concrete
150	**BASE** – OPSS Granular A Crushed Stone
400	**SUBBASE** – OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil, bedrock or fill.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment, noting that excessive compaction can result in subgrade softening.

5.0 Design and Construction Precautions

5.1 Waterproofing and Foundation Drainage

Foundation drainage and waterproofing recommendations should be confirmed based on site-specific boreholes and observed groundwater and conditions. It is expected that the shared underground parking levels will extend below the groundwater table. Therefore, it is recommended that a foundation drainage and waterproofing system be designed for proposed buildings with foundation elevations below the groundwater table.

Specific waterproofing recommendations and design can be provided for the proposed buildings once a geotechnical investigation has been completed and after detailed foundation design drawings are available.

As per Foundation Drainage Policy by City of Toronto, Toronto Water, Water Infrastructure Management, Long-term Discharge of Foundation Drainage to the City's sewer system will not be permitted unless exempted by Toronto Water in accordance with Policy Section 5.0.

Backfill against the exterior sides of the foundation walls should consist of free draining non-frost susceptible granular materials or approved other in-situ soils. It is expected that the greater part of the site excavated materials will are considered acceptable for re-use such as backfill as backfill against the foundation walls when used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000.

5.2 Protection of Footings Against Frost Action

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

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

However, the foundations are generally not expected to require protection against frost action due to the founding depth. Unheated structures such as the access ramp may require insulation for protection against the deleterious effects of frost action.

5.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

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

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

Temporary Shoring

Depending on the depth of excavation of the buildings and the proximity of the proposed buildings to the property boundaries, temporary shoring may be required to support the overburden soils of the adjacent properties. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation event will not negatively impact the temporary shoring system or soils supported by the system.

Any changes to the approved temporary shoring system design should be reported immediately to the owner's structural designer prior to implementation.

The temporary shoring system may consist of a soldier pile and lagging system which could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below.

The earth pressure acting on the shoring system may be calculated using the parameters in Table 6 below:

Table 6 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System

Parameter	Value
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	21
Submerged Unit Weight (γ'), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

5.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent material specifications and standard detail drawings from the City of Toronto Standard Specifications and Drawings for Sewers and Watermains.

The pipe bedding for the sewer and water pipes should consist of at least 150 mm of OPSS Granular A. The bedding layer thickness should be increased to a minimum of 300 mm where the subgrade will consist of soft soils or bedrock.

The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 225 mm thick lifts and compacted to a minimum of 99% of its SPMDD.

It should generally be possible to re-use the moist (not wet) site-generated fill above the cover material if the excavation and filling operations are carried out in dry weather conditions.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.2 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

5.5 Groundwater Control

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

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

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

5.6 Winter Construction

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

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

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

6.0 Recommendations

A site-specific geotechnical investigation including boreholes is a requirement for the design recommendations provided herein to be applicable. It is recommended that the following be carried out for the proposed development at the subject site:

- ❑ Undertake a site-specific geotechnical investigation based on the design of the proposed development.
- ❑ Review of the geotechnical aspects of the excavation contractor's shoring design, if not designed by Paterson, prior to construction, if applicable.
- ❑ Review of architectural plans pertaining to groundwater suppression system, underfloor drainage systems and waterproofing details for elevator shafts.

It is a requirement for the foundation design data provided herein to be applicable that a material testing and observation program be performed by the geotechnical consultant. The following aspects of the program should be performed by Paterson:

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

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

7.0 Statement of Limitations

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

Further, as noted above, a site-specific investigation including boreholes should be completed prior to construction to confirm our design recommendations which are based on available soils information.

The client should be aware that any information pertaining to soils are furnished as a matter of general information only and are not to be interpreted as specific conditions at the subject site.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Fotenn Consultants Inc. or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

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Report Distribution:

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- ❑ Paterson Group (1 Copy)

APPENDIX 1

FIGURE 1 - KEY PLAN

DRAWING TG0160-1 - SITE PLAN

DRAWING TG0160-2 - SURFICIAL GEOLOGY PLAN

DRAWING TG0160-3 - BEDROCK GEOLOGY PLAN

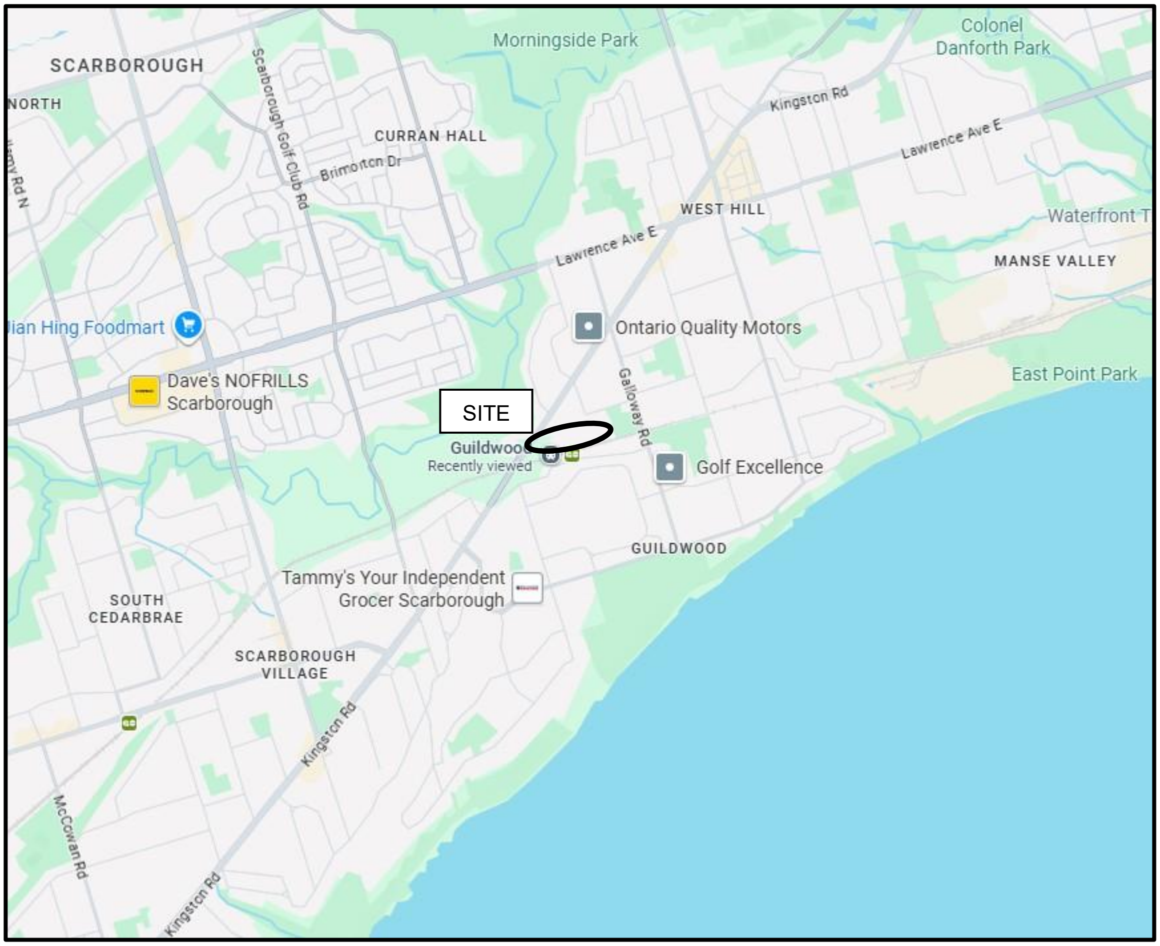

FIGURE 1

KEY PLAN

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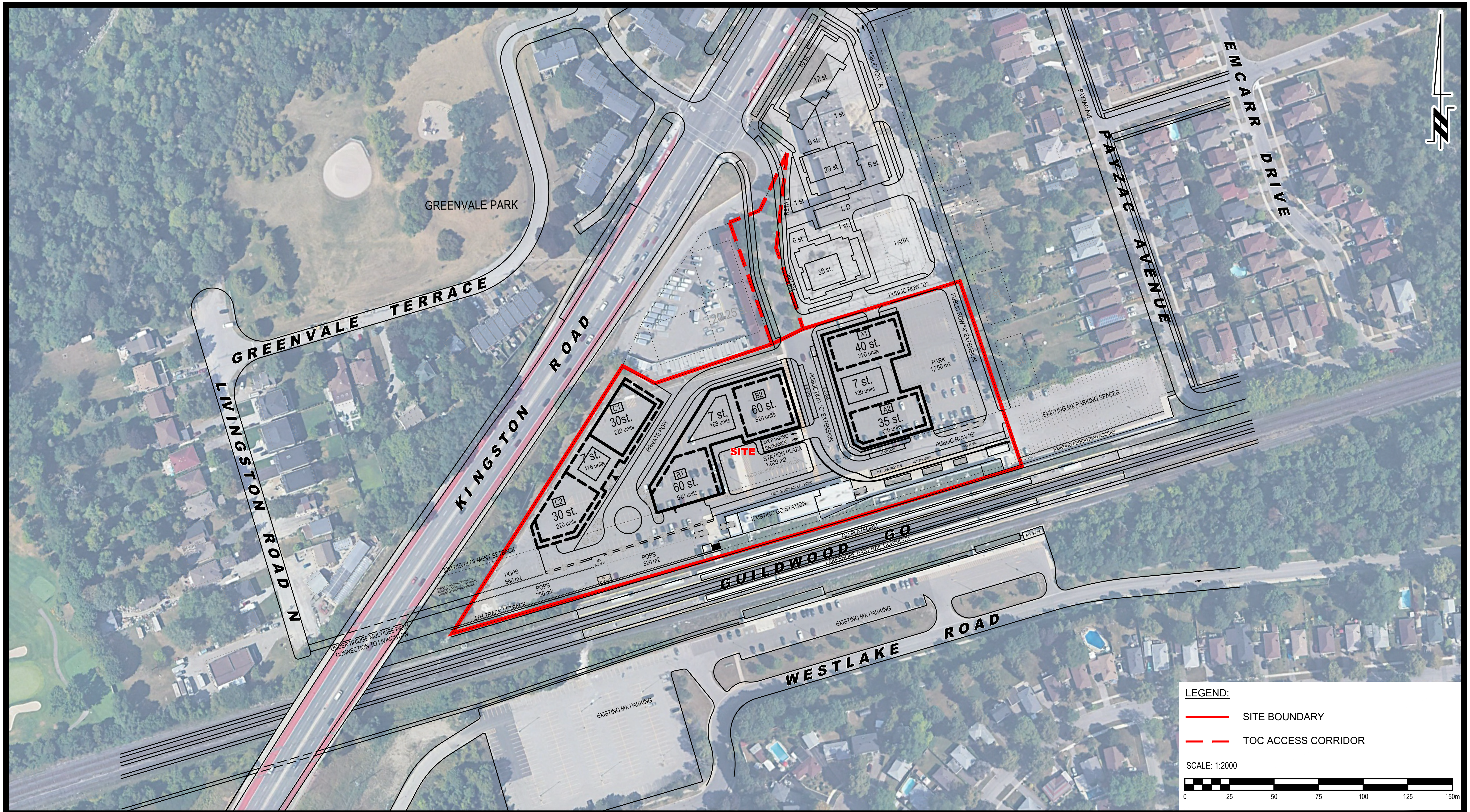

NO.	REVISIONS	DATE	INITIAL
2	ADDED CONCEPTUAL PLAN	31/10/2025	BN
1	SITE BOUNDARY UPDATED	30/09/2025	BN

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

FOTENN

GEOTECHNICAL STUDY
HIGH-RISE DEVELOPMENT - 4105 KINGSTON ROAD

SCARBOROUGH, ONTARIO

Title:

SITE PLAN

Scale:	1:2000	Date:	09/2025
Drawn by:	GK	Report No.:	TH0160-1
Checked by:	BN	Dwg. No.:	TG0160-1
Approved by:	KP	Revision No.:	2

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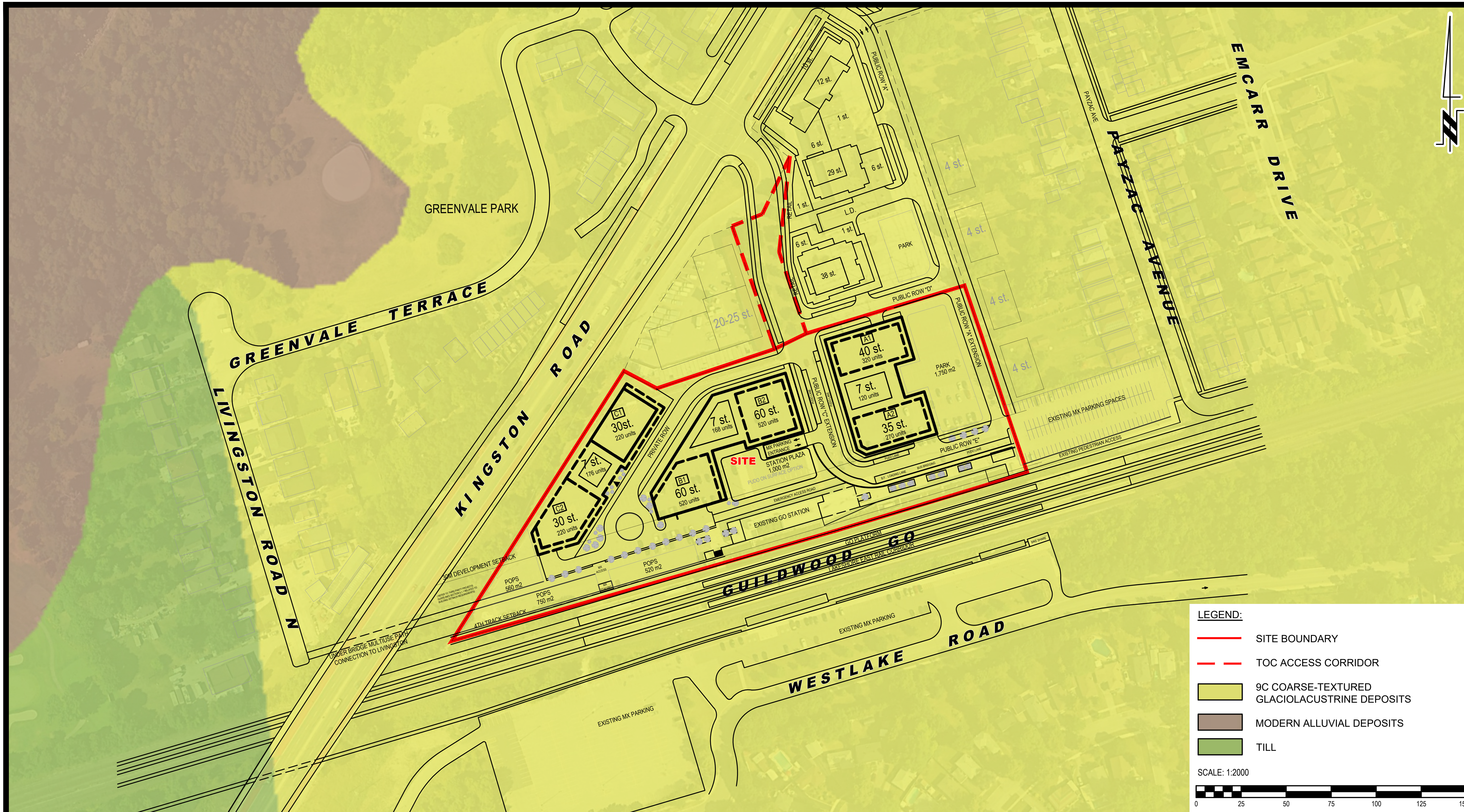

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NO.	REVISIONS	DATE	INITIAL
2	ADDED CONCEPTUAL PLAN	31/10/2025	BN
1	SITE BOUNDARY UPDATED	30/09/2025	BN

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GEOTECHNICAL STUDY
HIGH-RISE DEVELOPMENT - 4105 KINGSTON ROAD

SCARBOROUGH, ONTARIO

Title:

SURFICIAL GEOLOGY PLAN

Scale: 1:2000	Date: 09/2025
Drawn by: GK	Report No.: TG0160-1
Checked by: BN	Dwg. No.: TG0160-2
Approved by: KP	Revision No.: 2

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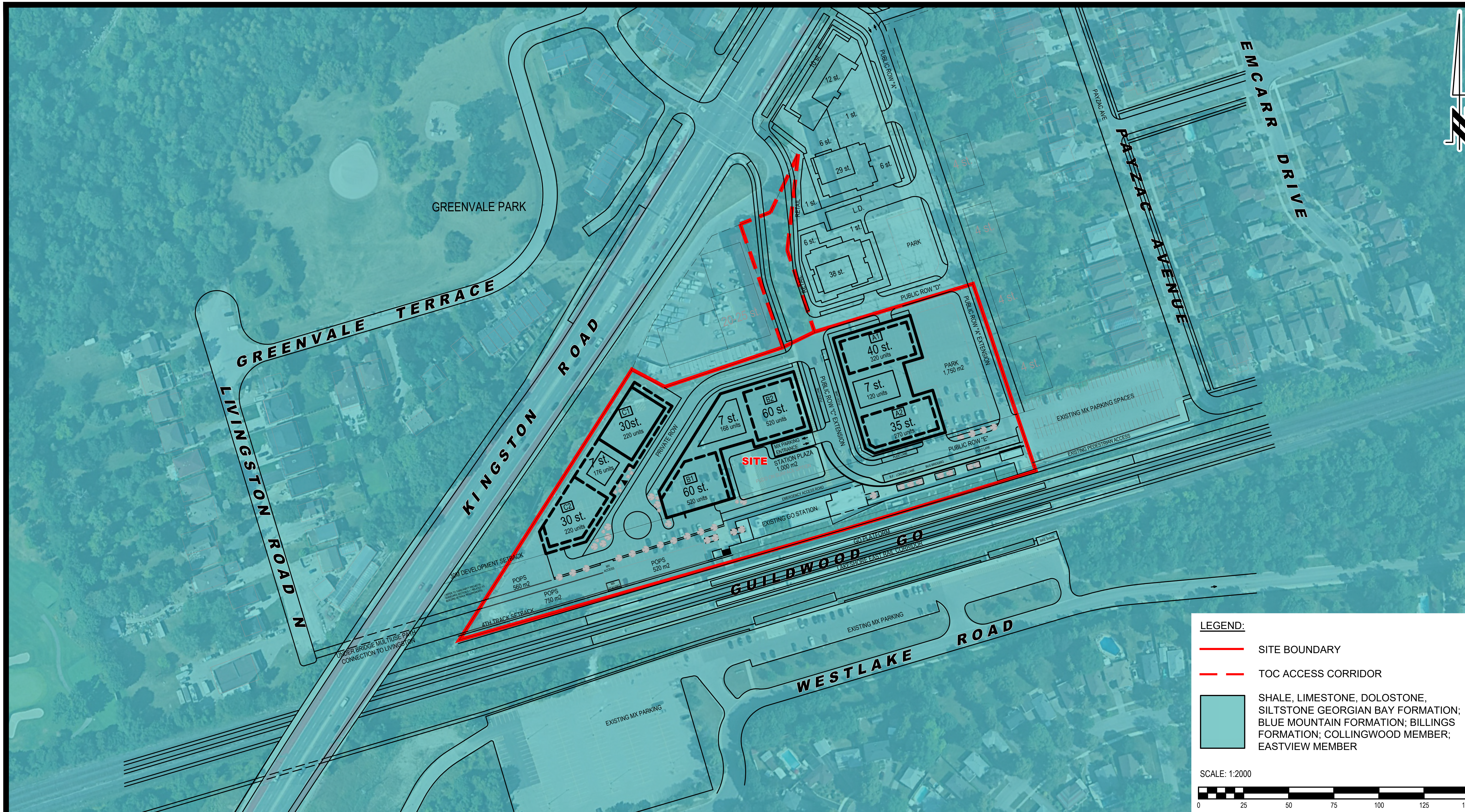

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NO.	REVISIONS	DATE	INITIAL
2	ADDED CONCEPTUAL PLAN	31/10/2025	BN
1	SITE BOUNDARY UPDATED	30/09/2025	BN

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GEOTECHNICAL STUDY
HIGH-RISE DEVELOPMENT - 4105 KINGSTON ROAD

SCARBOROUGH, ONTARIO

Title:

BEDROCK GEOLOGY PLAN

Scale: 1:2000	Date: 09/2025
Drawn by: GK	Report No.: TG0160-1
Checked by: BN	Dwg. No.: TG0160-3
Approved by: KP	Revision No.: 2

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