



Geotechnical Investigation

[Final Report]

8995 Boston Church Rd, Milton,
ON L9T 9C7

CLIENT



KING CONSULTANTS INC.



Pavement • Materials • Geo-Environmental • Engineering



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1.0 INTRODUCTION

PNJ Engineering Inc. (PNJ) was retained by King Consultants Inc. (the Client) to undertake a geotechnical investigation for the proposed industrial development at 8995 Boston Church Rd., Milton, ON L9T 0C7 (the Site). The Site location map is presented in **Appendix B** of this report.

The purpose of the geotechnical investigation was to assess the subsurface soil and groundwater conditions within the site through the drilling of ten (10) sampled boreholes included converting four (4) boreholes to Monitoring Wells (MWs) and to provide geotechnical comments and recommendations to support the construction of the proposed development. Details regarding the geotechnical scope of work for this project are provided in **Section 2.0** below.

1.1 Site Description and Project Understanding

The site is located on the east side of the intersection of Boston Church Rd. and 5 Side Rd., Milton, ON. The field investigation was carried out on an undulating to gently sloping terrain with partial vegetation cover and exposed soil surface. The site conditions at the time of drilling are depicted in the figures below.



Figure 1-1: Site Conditions near BH-2 - Facing North



Figure 1-2: Site Conditions near BH-10 - Facing North-East



Figure 1-3: Site Conditions near MW-1 - Facing North-East

The proposed development concept is shown as an overlay in **Figure 1-4** below.

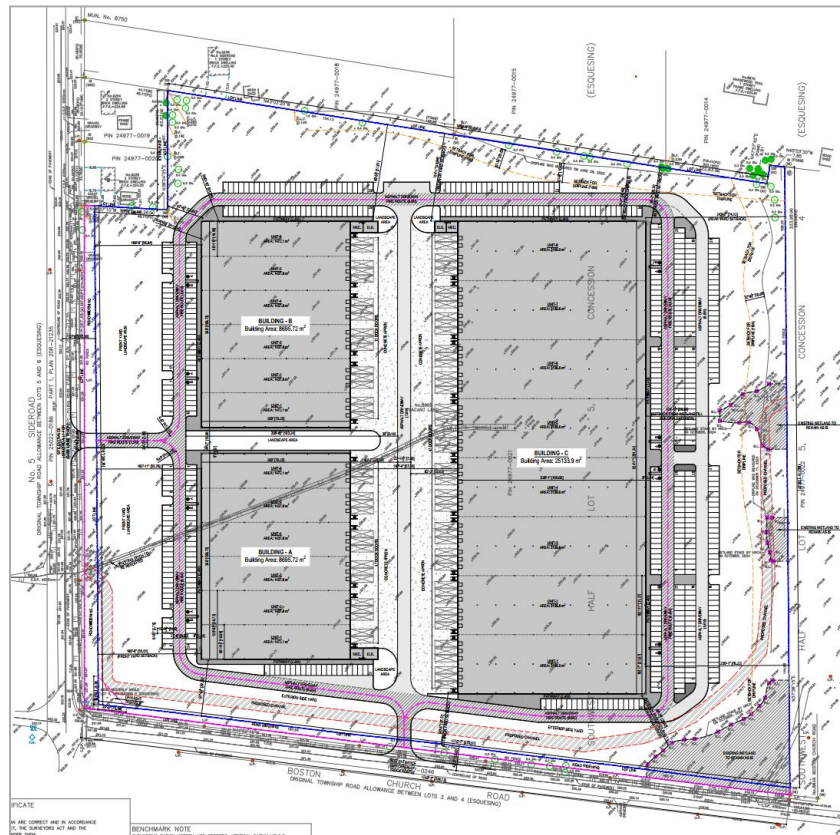


Figure 1-4: Proposed Industrial development

2.0 SCOPE OF WORK

PNJ's scope of work involved the following tasks:

- **Pre-Planning activities:**

- Preparation of a Site-specific health and safety plan (HASP).
- Predrilling site access assessment and borehole Stakeout.
- Completion of underground utilities locate clearance (public and private).

- **Field Activities**

- Visual reconnaissance of the existing site conditions.
- Advancement of Ten (10) boreholes drilled as below with in the site. The layout and/or distribution of these boreholes are presented in **Appendix B**.
 - Three (3) boreholes to approximately 7.6 meters below ground surface (mbgs) (25 ft.).
 - Four (4) boreholes to approximately 6.1 mbgs (20 ft.). All four (4) boreholes converted into monitoring wells (MW).
 - Three (3) boreholes to approximately 4.5 mbgs (15 ft.).
- Standard Penetration Test (SPT) and associated split spoon soil sampling in accordance with ASTM D1586.
- Collection of groundwater levels upon completion of borehole drilling.
 - This includes water level reading immediately after drilling and readings after development of the wells.
- Collection of soil samples for geotechnical and analytical laboratory testing.
- Collection of two (2) soil samples for corrosivity analysis.

- **Laboratory testing:**

- Geotechnical laboratory testing on selected soil samples, as described in **Section 5.0**.
- Limited analytical testing as described in **Section 6.0**.

- **Reports:**

- Preparation of a geotechnical investigation report (factual data, analysis, and recommendation), as outlined in the table of contents of this document.

3.0 PHYSIOGRAPHY

Based on the Physiography of Southern Ontario¹ and the surficial geology map², the Site is located within Bevelled Till Plains and comprises silt and clay, minor sand and gravel from Glaciolacustrine deposits. This site is expected to be underlain by shale, limestone, dolostone, siltstone bedrock, associated with Queenston formation.

4.0 METHOD OF INVESTIGATION

The field investigation methodologies for this investigation are presented in the following sections.

4.1 Safety Planning

Upon project initiation, a Site-specific Health and Safety Plan (HASP) was prepared for implementation during the field investigation program. The HASP presents the visually observed Site conditions to identify potential physical hazards to field personnel. Accordingly, the required personal protective equipment was also listed in the HASP. It is mandatory for all **PNJ** personnel involved in the field program to read the HASP and have a hard copy of the HASP available at the Site during the investigative work.

4.2 Borehole Location Clearance

PNJ completed a pre-drilling Site visit to review the Site conditions and access restrictions. Based on the limits of approach, the boreholes were marked at locations to avoid above ground and underground utilities or structures. All the boreholes were marked in the field based on the proposed borehole location plan.

Prior to initiating the subsurface investigation activities, all applicable utility companies (gas, hydro, network cables, water, wastewater, etc.) were contacted, to demarcate the location of their respective underground utilities and ensure that the service lines will not be damaged during the investigative works.

PNJ retained a private services locator (True North Locates Inc.) to locate any underground private utilities that could potentially be present at the Site.

¹ L.J. Chapman and D.F. Putnam, (1984): Physiography of Southern Ontario, 2nd Edition, Ontario Geological Survey.

² Sharpe, D. R. 1980: Quaternary Geology of Toronto and Surrounding Area; Ontario Geological Survey Preliminary Map P. 2204, Geological Series. Scale 1:100 000. Compiled 1980.

4.3 Field Investigation

The fieldwork for the geotechnical investigation was conducted over a two-day period, from July 02 to July 03, 2025. On July 02, 2025, three (3) boreholes (BHs) were drilled, and two (2) Monitoring Wells (MWs) were advanced on the North-East side of the site. This was followed by the installation of two (2) MWs and the drilling of three (3) boreholes (BHs) on July 03, 2025, to complete the investigation. The locations and corresponding depths are listed in **Table 4-1** below. The borehole location plan is provided in **Appendix B**.

The boreholes were advanced using solid-stem drilling and soil samples were collected every 0.75 meters interval to 3 meters below ground surface (mbgs), and at 1.5 m interval thereafter to the termination depth of the borehole. All soil sampling was conducted using a 50 mm outer diameter, split spoon sampler in accordance with the specifications of the Standard Penetration Test Method (ASTM D1586).

At the drilled borehole locations, the compactness condition or consistency of the subsurface soil layers were assessed using the Standard Penetration Test (SPT) method, by recording the number of blows ('N' values) required to drive a conventional split barrel soil sampler, 0.3 m into the material and these are presented on the borehole logs in **Appendix C** as penetration index values.

PNJ technical representative logged the soil samples encountered in the borehole and examined the samples as they were obtained. The recovered soil samples were transferred to **PNJ's** geotechnical laboratory in Toronto, ON, where they were reviewed by a senior geotechnical engineer. The detailed description of the individual soil deposits, groundwater condition and ground stratigraphy as encountered at the borehole location are shown on the accompanying borehole logs presented in **Appendix C**.

Groundwater and cave-in observations were made in the borehole immediately upon completion of drilling. Details on the recorded groundwater are presented in **Section 8.0** below.

4.4 Borehole Location and Ground Surface Elevation

The investigated locations and associated ground surface elevations were collected by **PNJ's** field supervisor using handheld GPS upon completion of the fieldwork. The coordinates, ground surface elevations, proposed and actual depths of the drilled boreholes are presented in the following table.

Table 4-1: Coordinates of Boreholes

MW / Test pit ID	Approximate Easting (m)	Approximate Northing (m)	Approximate Ground Surface Elevation (m AMSL) *	Proposed Depth (mbgs)	Actual Drilled Depth (mbgs)
MW-1	588240.5	4822320.8	222	6.1	6.7
BH-2	588315.9	4822309.1	222	4.5	5.1
BH-3	588122.4	4822407.8	222	4.5	5.1
BH-5	588396.4	4822414.7	222	4.5	5.1
MW-6	588384.6	4822477.0	222	6.1	6.7
BH-7	588291.3	4822486.3	222	7.6	8.2
MW-9	588184.1	4822613.9	222	6.1	6.7
BH-10	588196.1	4822545.6	222	7.6	8.2
MW-11	588128.2	4822546.4	222	6.1	6.7
BH-13	588070.7	4822474.0	222	7.6	8.2

* Above Mean Sea Level

It should be noted that the provided coordinates and elevations are approximate and are provided for geotechnical study purposes only and should not be used for construction purposes.

5.0 GEOTECHNICAL LABORATORY TESTS

The soil samples recovered from the split spoon sampler were properly sealed, labeled, and transported to **PNJ's** laboratory in Toronto, Ontario. Visual soil classifications made in the field were verified by a Senior Geotechnical Engineer. Geotechnical laboratory testing was conducted on the collected samples, including:

- Hydrometer on five (5) selected samples;
- Atterberg limits on two (2) selected samples; and
- Moisture content on all recovered soil samples.

The laboratory test results are reported in the borehole stratigraphy logs (**Appendix C**), with detailed results provided in **Appendix D**.

The geotechnical laboratory testing was conducted in accordance with American Society for Testing and Materials (ASTM) applicable standards, while MTO LS-702³ was used for hydrometer

³ ASTM D422-63(2007)- "Standard Test Method for Particle-Size Analysis of Soils" has been withdrawn without replacement from January 2016. As such, the hydrometer testing method MTO LS-702 'Particle-Size Analysis of Soils' was used for soil samples gradation analysis.

analysis of soils. The soil testing program and soil classification conformed to the latest edition of the following standards:

MTO LS-702 Standard Test Method for Particle Size Analysis of Soils (Hydrometer Analysis)

ASTM D4318 Standard Test Method for Liquid Limit, Plastic Limit and Plasticity Index of Soils

ASTM D2216 Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

The Unified Soil Classification System (USCS) was used for soil description and classification.

6.0 LIMITED ANALYTICAL TESTING

Limited analytical including chemical and electrochemical analytical tests were conducted on two (2) selected soil samples including pH, chloride, sulfate, oxidation-reduction potential, sulfides, resistivity, and conductivity. The aim of these tests was to evaluate the soil's corrosivity towards grey ductile cast iron elements as well as the potential for Sulfate attack on construction concrete. The results of these laboratory tests are presented in **Appendix E** and discussed in **Section 9.0**.

7.0 SUBSURFACE CONDITIONS

The subsurface soil conditions encountered in the boreholes are shown in detail on the Record of Borehole sheets in **Appendix C**. Method of Soil Classification, Abbreviations and Terms Used on Records of Boreholes and List of Symbols are also provided in **Appendix C** to assist in the interpretation of the Record of Borehole sheets.

The summaries of the subsurface conditions at each borehole location are described in **Section 7.1** to **Section 7.4**. The following summaries are to assist the designers of the project with an understanding of the anticipated soil conditions across the site. However, 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 considered as exact planes of geological change.

The **Figure 7-1** below provides a visual representation of the soil layers with thickness encountered in all boreholes.

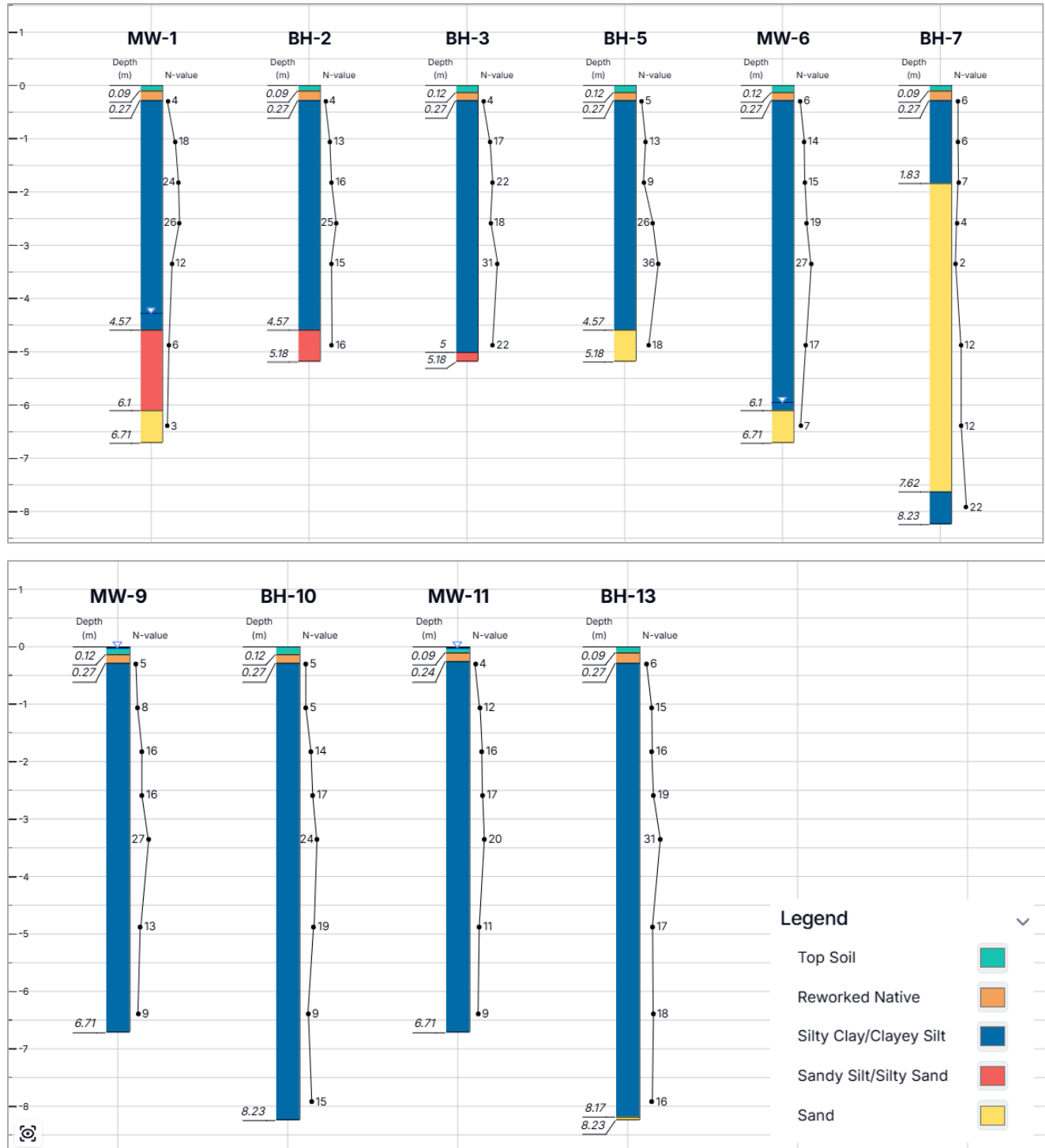


Figure 7-1: Encountered soils material in all Boreholes.

7.1 Topsoil

Topsoil of approximate thickness ranging from 90 mm to 120 mm was encountered in all boreholes. It should be noted that the thickness of the topsoil may vary significantly across the site and may not closely align with the conditions observed during drilling.

7.2 Fill Soils/Reworked native

Fill or reworked native soils were encountered in all boreholes and compose of mostly sandy silt material with some clay, gravels and organics content with brown to dark brown in colour and moist condition. The fill/reworked native layer extended to depths ranging from approximately 0.24 to 0.27 mbgs.

7.3 Native Soils

7.3.1 Silty Clay/Clayey Silt

This layer, primarily composed of Silty Clay/Clayey Silt was observed in all boreholes immediately beneath the Fill layer, extending to depths ranging from approximately 4.5 mbgs to 8.2 mbgs. The moisture content within this layer ranged from approximately 9.9% to 27.8%.

The SPT performed within this soil stratum, yielded N-values ranging from 5 to 36 blows per 300 mm of penetration, indicating firm to hard cohesive soil.

Five (5) samples were selected from this layer for hydrometer analysis and two (2) sample for Atterberg limit analysis. The test results are enclosed in **Appendix D** and summarized in **Table 7-1** and **Table 7-12** below.

Table 7-1: Grain size and Hydrometer Analysis

Sample No.	Approx. Depth (mbgs)	Grain Sizes (%)				Soil Description	Estimated coefficient of permeability (k) (cm/sec.) *
		Gravel	Sand	Silt	Clay		
MW-1-SS3	1.8	13.8	18.8	48.5	18.9	Silt, some clay, some sand, some gravel	3.05E-06
BH-5-SS2	1.2	7.4	22.6	56.5	13.5	Sandy Silt, some clay, trace gravel	1.23E-05
BH-7-SS3	1.8	n/a	11.9	74.4	13.7	Silt, some clay, some sand	3.33E-05
BH-10-SS7	6.4	n/a	7.8	87	5.2	Silt, trace sand, trace clay	1.84E-05
BH-13-SS3	1.8	3.2	21	53.7	22.1	Clayey Sandy Silt, trace gravel	9.66E-07

* Estimated based on grain size analysis

Table 7-2: Atterberg Limit Analysis

Sample No.	Approx. Depth (mbgs)	Atterberg Limits		Soil Classification *
		LL (%)	PL (%)	
MW-1-SS3	1.8	21.5	15.0	CL-ML
BH-10-SS3	1.8	25.6	15.1	CL

* Unified Soil Classification System (USCS)

7.3.2 Sandy Silt/Silty Sand

A sandy silt/silty sand layer was encountered in borehole MW-1, BH-2, and BH-3. The depth of this layer extending approximately of 1.6 meters in MW-1, 0.6 meters in BH-2, and 0.18 meter in BH-3. The moisture content within this layer ranging from 24.2% to 24.6%

7.3.3 Sand

A sand layer was encountered in borehole MW-1, BH-5, MW-6 and BH-7 at different depths, with thickness ranging from approximately 0.6 m to 5.7 m, as illustrated in **Figure 7-1** above. The layer was encountered at depth from 6.1 mbgs to termination depth of 6.71 mbgs in MW-1 and MW-6. In BH-5 it was encountered from 4.5 mbgs to 5.1 mbgs. For BH-7, sand layer was identified from 1.8 mbgs to 7.6 mbgs.

The moisture content within this layer ranges from approximately from 14.8% to 32.6%

The SPT performed within this soil stratum, yielded N-values ranging from 3 to 18 blows per 300 mm of penetration, indicating very loose to compact non-cohesive soil.

8.0 GROUNDWATER

Upon the completion of each borehole, the groundwater level and sidewall cave-ins were measured. Groundwater was observed in all boreholes except BH-3 and BH-13, with recorded water levels as shallow as 1.7 mbgs. No cave-in was observed in all boreholes.

The table below presents the groundwater level readings taken immediately and at later dates after drilling. Groundwater readings are also presented in the borehole logs in **Appendix C**.

Table 8-1: Groundwater Readings

Borehole ID	Ground Elevation (m AMSL) ¹	Measurement Date (Immediately after drilling)	Water Depth (mbgs) ²	Water Level Elevation (m)	Measurement Date (2 nd Reading)	Water Depth (mbgs) ²	Water Level Elevation (m)
MW-1	222	July 03, 2025	4.2	217.8	July 10, 2025	2.6	219.4
BH-2	222	July 03, 2025	3.9	218.1	-	-	-
BH-3	222	July 03, 2025	Dry	Dry	-	-	-
BH-5	222	July 02, 2025	2.3	219.7	-	-	-
MW-6	222	July 02, 2025	5.9	216.1	July 10, 2025	5.2	216.8
BH-7	222	July 02, 2025	5.3	216.7	-	-	-
MW-9	222	July 02, 2025	Dry	Dry	July 10, 2025	4.2	217.8
BH-10	222	July 02, 2025	6.1	215.9	-	-	-
MW-11	222	July 03, 2025	Dry	Dry	July 10, 2025	1.7	220.3
BH-13	222	July 03, 2025	Dry	Dry	-	-	-

1- Approximate ground elevation, meters Above Mean Sea Level (m AMSL)

2- Approximate water depth, meters below ground surface (mbgs)

It's worth noting that the water level can fluctuate throughout the year, and water seepage into the drilled hole may not be immediate.

9.0 SOIL AGGRESSIVENESS

9.1 Corrosivity Potential on Ductile Iron Pipes

Two (2) soil samples were selected for analysis of pH, Chloride, sulfate, Oxidation-reduction potential, sulfides, resistivity, and conductivity, to evaluate soil corrosivity on grey ductile cast iron elements.

The results of the chemical tests were used to assign points as described in Table A.1 of the American Water Works Association (AWWA) publication ANSI/AWWA C105/A21.5-10. As per the AWWA publication points system, a soil that has a total score of 10 or greater is potentially corrosive to ductile iron pipes. In addition, soils with resistivity value of less than 3000 ohm-cm are also considered potentially corrosive.

The ANSI/AWWA rating System employs soil resistivity, pH value, redox potential, sulfide content and drainage conditions as the main markers of soil aggressiveness. In this method, a point

system is used to evaluate the corrosivity of the tested soil. Points are assigned to each marker in accordance with its expected contribution to the total corrosion potential of the soil as established by laboratory testing and visual examination of the soil. For this project the chemical analysis was carried out by Eurofins laboratories, a specialist chemical testing laboratory. The test results are provided in **Appendix E**.

The sulfide and chloride concentrations of the samples tested were evaluated using a chromatograph, the pH using a pH meter, resistivity using an electrical conductivity test and redox using a redox potential electrode. Accordingly, some interpretation of the data was required to assess corrosion potential. It is noted that sulfide analysis presented in AWWA is a qualitative test where a positive, trace, or negative determination is based on the presence of bubbles because of a chemical reaction. The laboratory results, however, present a concentration that is unrelated to the scale provided by AWWA. It was assumed that samples with a laboratory result less than the reported detection limit (RDL) for sulfide would be considered to have a 'trace' condition (score of 2) and results greater than the RDL would be positive and a maximum score of 3.5 was selected.

Table 9-1: Corrosivity Test Results

Sample ID (approx. depth)	Corrosivity Potential Parameters					Total Points	Corrosive (Yes/No) (1)
	Resistivity (ohm.cm)	pH	Redox Potential (mV)	Moisture (%)	Sulfides ⁽²⁾ (mg/kg)		
BH-7-SS3 (1.5– 2.1 mbgs)	8333	7.4	259	15.8	<0.01	-	-
Points as per AWWA C105 / A21.5.10	0	0	0	1	2	3	No
BH-13-SS3 (1.5 – 2.1 mbgs)	5556	8.4	304	12.7	<0.01	-	-
Points as per AWWA C105 / A21.5.10	0	0	0	1	2	3	No

1- Soil test evaluation was carried out using AWWA C105/A21.5-05. A score of ten (10) points or more indicates the soil is corrosive to ductile iron pipes; protection needed.

2- Sulfides values less than the reported detection limit (RDL) are considered as "trace" (score of 2) and sulfides contents in excess of the RDL are considered as "Positive" (score of 3.5)

The chloride content of each sample mentioned above was also tested. However, since the AWWA system does not use chloride as an indicator of corrosivity, an assessment of the potential contribution of the quantified chloride concentration to corrosion was based on our experience on similar soil condition. The table above summarizes the ANSI/AWWA rating of the tested soil samples and their potential for corrosion towards buried grey or ductile cast iron pipes.

Based on the test results and following the guidelines provided in the AWWA publication ANSI/AWWA C105/A21.5-10, both samples tested received a score less than 10 points. Therefore, *according to the ANSI/AWWA rating system, the collected soil samples indicate that the soils are not corrosive to grey or ductile iron pipes at their respective depths.* It's important to note that these results are only representative of the collected samples and different results may be encountered at different locations within the site.

Note that there may be other overriding factors in the assessment of soil corrosion potential, such as application of de-icing salts on the roadway and subsequent leaching into the subsurface soils, stray currents, the nature of effluent conveyed by underground pipes, etc.

9.2 Buried Concrete

The potential for Sulfate attack on construction concrete is estimated using Table 3 provided in CSA A23.1-14 and reproduced as the following Table.

Table 9-2: Exposure Degree based on Sulfate Concentration

Degree (Class) of Exposure	Water Soluble SO ₄ in Soil Sample (%)
Very Severe (S-1)	> 2.0
Severe (S-2)	0.20 - 2.0
Moderate (S-3)	0.10 - 0.20

Reference: Table 3, Page 136 CSA A23.1-14/A23.2-14

Table 3 of the Canadian Standards Association (CSA) document A23.1-14/A23.2-14 'Concrete Materials and Methods of Concrete Construction/Methods of Test and Standard Practices for Concrete' classifies the degree of exposure into the above three classes.

The aggressiveness of the soil on concrete in contact with the soils was evaluated by performing water-soluble sulfate content tests on select samples. Summary of the sulfate results and the correlated Degree of Exposure are presented in **Table 9-3** below.

Table 9-3: Sulfate results on select samples.

Sample ID (depth)	Sulfate (%)	Degree (Class) of Exposure
BH-7-SS3 (1.5– 2.1 mbgs)	<0.01	Below Moderate (S-3)
BH-13-SS3 (1.5– 2.1 mbgs)	<0.01	Below Moderate (S-3)

The sulfate ion content analysis reveals that the soil sample falls below the S-3 exposure levels outlined in CSA A23.1-14, indicating a below-moderate sulfate exposure for sub-surface concrete within the immediate vicinity of the tested samples. It's important to note that these results are only representative of the collected samples and different results may be encountered at different locations within the site. Nonetheless, according to this limited testing, ordinary Type 10 Portland Cement may be suitable for use in buried concrete structures. However, additional testing is recommended to determine if a special type of cement is required throughout the extent of the project. The certificate of analysis is presented in **Appendix E**.

10.0 ENGINEERING DISCUSSION AND RECOMMENDATION

10.1 Proposed Development

This section provides geotechnical recommendations to support the construction of the proposed industrial development, which consists of an industrial complex comprising three (3) single-storey warehouse buildings (Buildings A, B, and C) with integrated office components and optional internal mezzanine space, having a total gross floor area of approximately 33,829.62 m². The 11.71 ha Site is proposed to be developed at a lot coverage of 36.5% and will be re-designated from its current Agricultural zoning to M2 – General Industrial Zone to permit warehouse, storage, and associated office uses.

Buildings A and B each provide approximately 8,695.72 m² of floor area across six (6) units, while Building C provides approximately 25,133.9 m² across eight (8) units. All three buildings are planned to feature contemporary precast and metal panel façades, with clear internal heights of approximately 37 ft.

The Site will operate as a fully serviced employment parcel and will incorporate 81 loading docks, two (2) vehicular access points with dedicated truck and passenger-vehicle circulation routes, internal fire routes, and 595 surface parking spaces, including sixteen (16) barrier-free spaces, thereby exceeding applicable parking requirements.

The following discussions and recommendations are based on the factual subsurface data obtained from the field investigation program carried out at the Site, as described in previous sections. The proposed development is discussed in **section 1.1**.

The following discussions and recommendations are based on the factual data obtained from advancing ten (10) boreholes at the above-mentioned site, as described in previous sections.

10.2 Site Preparation and Material Reuse

10.2.1 Grading

Deleterious materials such as topsoil, organic rich soil, or loose / soft materials should be removed prior to site grading activities. Such material should not be used as backfill in settlement-sensitive areas. The subgrade soils exposed after the removal of the unsuitable material should consist of approved earth fill or native soil. The subgrade soils should be visually inspected by an experienced geotechnical engineer from **PNJ**; compacted and proof rolled where required, using appropriate equipment, compatible with the type of soil.

After approval from the geotechnical engineer, any required fill to raise grades in *settlement-sensitive* areas should be placed in thin layers (200 mm thick or less) and compacted to a minimum of 100% of the Standard Proctor Maximum Dry Density (SPMDD).

Drying of excavated soils may be required depending on the weather conditions at the time of construction or the depth of excavation. If any materials are discovered to have high water content values, they can be left to dry or mixed with drier material that will be used as backfill in settlement-sensitive areas.

It is recommended that final grading around buildings provide positive drainage away from structures, with a minimum slope of 2% within the first 2 metres of the building footprint, unless otherwise specified by the professional engineer.

10.2.1 Materials reuse

Existing Fill

The encountered fill material on-site was found to be variable in composition, with overlying layers consisting of Sand and Gravel and Silty Clay / Clayey Silt. The fill layers also contained organics and brick pieces, which suggests the possible presence of other deleterious materials within the fill layers. Therefore, based on the available data and without further assessment and testing by a geotechnical engineer/specialist during the construction phase, the reusability of these materials as backfill is not recommended. However, these materials can be used for non-structural areas, such as landscaping.

If the client wishes to further explore the potential reuse of the existing fill materials, the on-site geotechnical specialist could visually classify the excavated material, oversee the removal of deleterious materials, and direct the separation of excavated materials into distinct piles based on the general soil composition. Representative samples will then be collected from each stockpile for further testing, including Proctor testing and sieve/hydrometer analysis, which will be

compared against OPSS 1010 guidelines, as well as for the requisite environmental testing. A letter will then be issued to present and discuss the findings, including the potential for reuse and any recommendations.

Native Soils

Native soils (Clayey Silt/Silty Clay, Sandy Silt/Silty Sand, and Sand) are expected to be encountered within the excavation depths. Hydrometer test results and visual classification indicate that these native soils have a moderate to high susceptibility to frost heave; therefore, they are not recommended for reuse as backfill material within the frost depth specified in **Section 10.7**. However, these materials can be reused to raise the grade of compatible soils below the frost depth, provided they are prepared and compacted as discussed in **Section 10.2.1**.

10.3 Excavation and Temporary Shoring

Based on the field results, excavations for site servicing, subgrade preparation, and foundations may pose some difficulties due to the presence of a high-water table, as further discussed in **Section 10.4**. The hydrogeological report (prepared under separate cover) should be consulted for the anticipated groundwater level during construction. Nevertheless, excavation of the soils at this site can be carried out with hydraulic excavators.

All excavation work must comply with the Occupational Health and Safety Act (OHSA). In accordance with OHSA, the encountered native soils are classified as **Type 3** above the groundwater table and **Type 4** soils below the groundwater table (see **Section 8.0**.) If soils and/or groundwater conditions, observed at the time of construction, differ from those described previously, **PNJ** should be notified to reassess the provided soil classification.

In the event very loose and/or soft soils are encountered at shallow depths or within zones of seepage, it will be necessary to flatten the side slopes to achieve stable conditions.

OHSA specifies maximum slope of the excavations for four broad soil types as summarized in the following table:

Table 10-1: Excavation Slopes for Each Soil Type as per OHSA

Soil Type	Base of Slope	Slope inclination
1	Within 1.2 meters of bottom of excavation	1 H:1V
2	Within 1.2 meters of bottom of excavation	1 H:1V
3	From Bottom of excavation	1 H:1V
4	From Bottom of excavation	3 H:1V

Excavation side-slopes should not be unduly left exposed to inclement weather. If the above recommended excavation side slopes cannot be maintained due to lack of space or any other reason, the excavation side walls must be supported by an engineered shoring system. The shoring system should be designed in accordance with Canadian Foundation Engineering Manual (4th Edition) and the OHSA Regulations for Construction Projects.

Where workers must enter excavations extending deeper than 1.2 m below grade, the excavation sidewalls must be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. It is anticipated that foundation and utility excavations can be made with conventional equipment. However, construction debris (e.g., concrete slabs or fragments) should be expected within the fill soil, and the contract should allow for the removal of such material.

10.4 Backfilling

For trench backfill operations, the following criteria must be satisfied:

- Trench backfill should consist of uniform, homogeneous material. The fill material should also be free of organics, deleterious materials (i.e. building debris such as bricks, metal etc.). Materials meeting OPSS 1010 guidelines for Granular B Type I or II specifications would be considered a suitable Engineered Fill material.
- Prior to the placement of trench backfill, the fill to be used must be evaluated for suitability by a geotechnical engineer and through geotechnical laboratory testing, such as the Standard Proctor test and grain size analysis.

Fill must be compactable, and of a suitable moisture content such that it is within +/- 2.0% of its optimum moisture content, as determined through laboratory testing.

Fill must be placed under the continuous supervision of a geotechnical engineer or their designate.

Each layer of material should be placed in maximum 200 mm lifts and uniformly compacted with heavy compaction equipment suitable for the type of fill used, to 100% of its SPMDD.

Field density tests must be taken by the Geotechnical Engineer on each lift of Engineered Fill. Any Engineered Fill which is tested and found to be out of specification shall be either removed, reworked or retested.

As a general requirement, where trenches are located within the present or future paved areas, and elsewhere where surface settlements cannot be tolerated, the degree of compaction of the

backfill material should be a minimum of 95% of the SPMDD, increasing to 98% within the top 1.5m of the backfill.

10.5 Site Servicing

This section discusses site servicing by open-cut trenching, including bedding and backfill recommendations for underground utilities including water mains, sewer pipes, manholes, catch basins and other related structures.

Pipe bedding for flexible pipes should follow the requirements in Ontario Provincial Standard Drawing 802.010 or 802.013 or applicable municipal standards. Pipe bedding for rigid pipes should follow the requirements in Ontario Provincial Standard Drawings 802.030 to 802.033 or applicable municipal standards.

The native soils present at the site will provide adequate support for pipes with the bedding requirements as identified in the above referenced OPS drawings. Where disturbance of the trench base has occurred from groundwater seepage, construction traffic, etc., the disturbed soils should be sub-excavated and replaced with suitably compacted granular fill. If weak zones are encountered, additional bedding materials and differing construction practices may be required and should be determined during construction.

Regardless of flexible or rigid pipes, granular bedding and cover material should consist of a well graded, free draining material, such as Granular "A" (OPSS.MUNI 1010). All granular bedding must be compacted to a minimum of 98% SPMDD. Clear stone or high-performance bedding is permitted at this site provided it is fully wrapped in a non-woven filter fabric to prevent the migration of fines and loss of pipe support.

It is recommended that prior to commencing the construction of the site servicing, consideration be given to the excavation of a series of trial excavations along the alignment of the proposed sewers/watermains to determine more accurately the soil behavior and if any dewatering works are required.

10.6 Groundwater Control

Based on the results of grain size analyses and the estimated k values, the native silty clay / clayey silt, which extend below the anticipated excavation depths, are expected to have low permeability coefficients. If groundwater is encountered during excavations, the water yield from these soils are expected to be small. Therefore, it is anticipated that adequate control of groundwater seepage during subgrade preparation and foundation excavations can be achieved by conventional construction dewatering techniques such as filtered pumping from sumps and/or ditches to manage groundwater and surface runoff accumulation. It is further recommended to

excavate a few test pits to estimate the expected total amount of water into the excavated trenches and plan the dewatering requirements for the larger excavations.

Water bearing sand and silt layers under artesian pressure are known to be embedded in the lacustrine deposits. The quantities of dewatering from these seams/layers may be small, however the flow intensity at the time of release may be high.

In areas where excavations may extend below groundwater table, a water taking Permit may be required during construction. Consequently, the hydrological study prepared by others should be utilized in this regard.

It should be noted that the estimate of permeability coefficients for the native soils was based on our understanding of the soil type at the drilled locations, supplemented by grain size analyses at selected depths, as shown in the **Table 7-1**.

10.7 Depth of Frost Penetration

The design depth of frost penetration in the area is approximately 1.2 m as per the OPSD 3090.101 and as such, a permanent soil cover of at least 1.2 m or its thermal equivalent synthetic insulation is required for frost protection of foundations (foundations in unheated areas). All foundations exposed to freezing temperatures should have at least such earth cover or equivalent synthetic insulation for frost protection. During winter construction, exposed surfaces to support foundations must be protected against freezing by means of loose straw and tarpaulins, heating, etc.

10.8 Foundation Design

10.8.1 Strip / Spread Foundation

This section provides bearing capacities for a shallow foundation to be considered as an option to support the construction of the proposed industrial development.

It is recommended that all foundations be founded on native soil, below any fill or reworked native soils, where the soil is stiff to hard or compact to very dense. According to the borehole findings and understanding of the project, the recommended minimum foundation depths are provided in **Table 10-2** below.

The bearing capacities were calculated for a conventional footing constructed in native soil based on the geotechnical information obtained from the drilled boreholes. The proposed soil resistance values below can be used when designing for Serviceability Limit State (SLS to limit the total and differential settlements to 25 mm) and factored ($\Phi = 0.5$) Ultimate Limit State (ULS) design. It is noted that foundation settlement is a function of the footing dimensions. Footings with larger width

could experience proportionally larger settlements and lower SLS values. The factored ULS values are used in the Load and resistance factor design (LRFD) approach, in which factored loads must be specified to take into account the uncertainties in loads and their probability of occurrence. Footings with lower embedment depth will result in lower ULS values.

The table below presents bearing capacities at the minimum recommended founding depth for a conventional spread footing foundation up to 2.5 m in width. The following bearing capacities assume vertical loading. Once load inclinations are available, the capacities should be re-evaluated to account for their effects.

Table 10-2: Bearing Capacity at SLS and ULS – Strip / Spread Foundations

Borehole ID	Depth (m)	Bearing Capacity at	
		SLS (kPa)	ULS (kPa)
Industrial Building A (BH-10, MW-9, MW-11)	1.3	68	123
	1.5	98	177
	2.0	133	235
Industrial Building B (BH-13)	1.3	130	230
	1.5	130	230
	2.0	140	245
Proposed Industrial Building E Wearhouse/Distribution (MW-1, MW-6, BH-7)	1.3	108	192
	1.5	117	208
	2.0	133	233

It should be noted that the recommended bearing capacities have been calculated by PNJ from the borehole information for the design stage only. The investigation and comments are necessarily on-going as new information on the underground conditions becomes available. For example, more specific information is available with respect to conditions between boreholes when foundation construction is underway. The interpretation between boreholes and the recommendations of this report must therefore be checked through field inspections provided by PNJ to validate the information for use during the construction stage.

Due to variations in the consistency or compactness of the founding soils and/or loosening caused by excavating disturbance and/or seasonal frost effects, all footing subgrades must be evaluated by the Geotechnical Engineer prior to placing formwork and foundation concrete to ensure that the soil exposed at the excavation base is consistent with the design geotechnical bearing resistance.

Where footings are required to be constructed at different elevations, the stepping between adjacent footings should be performed at a slope not steeper than 2 horizontals (clear horizontal distance between footings) to 1 vertical (difference in elevation) (2H:1V). Additionally, the vertical height of each individual footing step should not exceed 0.6 m, unless otherwise justified by a geotechnical or structural assessment. This stepped arrangement ensures adequate load transfer and reduces the potential for differential settlement or soil disturbance between the footings.

The native soils tend to weather and deteriorate rapidly on exposure to atmosphere or surface water; therefore, **PNJ** recommends that footings placed on the exposed soil should be poured on the same day as they are excavated, after removal of all unsuitable founding materials and approval of the bearing surface. Alternatively, a concrete mud slab may be used to protect the bearing surface where footing construction is to be delayed. Rainwater or groundwater seepage entering the foundation excavations must be pumped away (not allowed to pond). The foundation subgrade soils should always be protected from freezing, inundation, and equipment traffic. If unstable subgrade conditions develop, **PNJ** should be contacted in order to assess the conditions and make appropriate recommendations.

If construction proceeds during freezing weather conditions, adequate temporary frost protection for the footing bases and concrete must be provided. All exterior footings and footings in unheated areas should be provided by a soil cover to the depth discussed in **Section 10.7**, or equivalent artificial thermal insulation for frost protection purposes.

10.8.2 Deep Foundation Considerations

Both driven piles, such as HP piles, and drilled cast-in-place concrete piles, such as caissons, are considered suitable to support the proposed industrial development, provided appropriate groundwater control measures are implemented during construction.

Based on the borehole data, the native soils, which consist predominantly of firm to stiff cohesive silty clay/clayey silt, extend to a ranging depth from 5.1 mbgs to 8.2 mbgs within the proposed industrial development area. Design methods for piles in fine-grained soils are considered of doubtful reliability in some cases, especially when estimating the ultimate geotechnical shaft resistance of friction piles in clays with medium to high shear strength. For this reason, it is recommended that pile load testing be carried out. In accordance with the Canadian Highway Bridge Design Code (CHBDC), SPT- and CPT-based methods to determine the geotechnical axial resistance of piles shall only be used when confirmed with field load testing or local experience. Since no piles have been installed at the site to validate the SPT-based estimates presented below, pile load testing shall be completed to confirm the provided geotechnical resistances in the following subsections.

Driven Piles

Driven piles in cohesive soils with undrained shear strength greater than 100 kPa can mobilize geotechnical resistance through a combination of shaft friction and end bearing. However, as previously noted, shaft resistance in such soils cannot be predicted reliably, primarily due to the limited understanding of how pile driving affects both the mobilized resistance and the effective contact area between the pile and the surrounding clay. Therefore, **Table 10-3** below provides preliminary design parameters; for final design purposes, it is recommended that the pile capacity be determined by either a static loading test or by dynamic methods. The parameters below are based on HP 310x132.

Table 10-3: Preliminary Design Parameters – Driven Piles

Borehole ID	Soil Type	Depth (m)	Shaft Resistance (kPa)		End Bearing (kPa)	
			SLS	Factored ¹ ULS	SLS	Factored ¹ ULS
Industrial Building A (BH-10, MW-9, MW-11)	Firm to Stiff Silty Clay / Clayey Silt	2 to 8.2	10	16	181	302
Industrial Building B (BH-13)	Firm to Stiff Silty Clay / Clayey Silt	2 to 8.2	10	17	215	359
Proposed Industrial Building E Warehouse/Distribution (MW-1, MW-6, BH-7)	Firm to Stiff Silty Clay / Clayey Silt (MW-1, MW-6)	2 to 6.7	10	17	193	321
	Loose to Compact Sand (BH-7)	2 to 8.2	6	11	504	840

1- ULS Resistance Factor of 0.4

Drilled Piles

Large diameter drilled piles, such as caissons, have been successfully used in cohesive soils where the undrained shear strength exceeds 100 kPa. Due to their larger shaft and base areas. These piles can develop higher total geotechnical capacities. The following table provides preliminary design parameters for such drilled piles. The parameters below are based on a pile diameter of 0.5 to 1 meter.

Table 10-4: Preliminary Design Parameters – Drilled Piles

Borehole ID	Soil Type	Depth (m)	Shaft Resistance (kPa)		End Bearing (kPa)	
			SLS	Factored ¹ ULS	SLS	Factored ¹ ULS
Industrial Building A (BH-10, MW-9, MW-11)	Firm to Stiff Silty Clay / Clayey Silt	2 to 8.2	11	18	161	269
Industrial Building B (BH-13)	Firm to Stiff Silty Clay / Clayey Silt	2 to 8.2	12	20	192	319
Proposed Industrial Building E Warehouse/Distribution (MW-1, MW-6, BH-7)	Firm to Stiff Silty Clay / Clayey Silt (MW-1, MW-6)	2 to 6.7	11	19	171	286
	Loose to Compact Sand (BH-7)	2 to 8.2	4	6	384	640

1- ULS Resistance Factor of 0.4

2- Based on a 1m diameter pile

It is recommended that the minimum embedment depth should be at least four times pile diameters into the hard soils.

If the above provided capacities in **Table 10-3 and Table 10-4** are found insufficient to support the proposed industrial development, options such as increasing the number of piles or diameter may be considered. Alternatively additional deep boreholes can be advanced to explore deeper strata and possibly reach the bedrock in this area.

Uplift Resistance

The ultimate uplift resistance of a pile can be considered equal to the ultimate shaft resistance. A geotechnical resistance factor of 0.3 can be used to obtain the factored geotechnical uplift resistance. Additionally, the dead weight of the pile may be included in the uplift resistance calculation, using the appropriate structural resistance factor for dead weight.

Lateral Resistance

Vertical piles resist laterally applied loads by deflecting until the surrounding soils mobilize sufficient resistance. The lateral load capacity depends on several factors, including pile stiffness, the fixity condition at the top of the pile, the strength of the soil, pile embedment, and tolerable displacements. Failure of such piles may occur in two principal modes: (i) geotechnical failure, when the lateral capacity of the soil is exceeded, leading to significant pile deflections; or (ii)

structural failure, when induced bending moments or shear forces surpass the structural capacity of the pile.

At ULS, the lateral resistance of a pile shall be taken as the lesser of its ultimate structural resistance and the ultimate resistance provided by the surrounding soils. At SLS, lateral displacements must remain within acceptable limits for the structure and/or geotechnical system.

Static, dynamic, or rapid pile load tests may be considered in accordance with ASTM D3966, as outlined in CSA S6:19 CHBDC Section 6.11.2.2.1.3, where such testing is used to supplement or validate static analysis.

Ultimate Lateral Resistance of a Pile

Broms' method provides a static analysis approach for estimating the ultimate lateral resistance of piles. For long pile failure, where the pile's bending capacity is fully mobilized, Broms presented solutions in graphical form based on simplifying assumptions. These solutions consider various parameters, including:

d = Pile diameter (m)

L = Embedded length (m)

H_u = Lateral load capacity (kN)

M_{plast} = Yield moment of pile (kN·m)

S_u = Undrained shear strength (kPa)

K_p = Coefficient of passive sand resistance (dimensionless)

e = Height of lateral load above groundline (m)

γ = Soil unit weight (kN/m³).

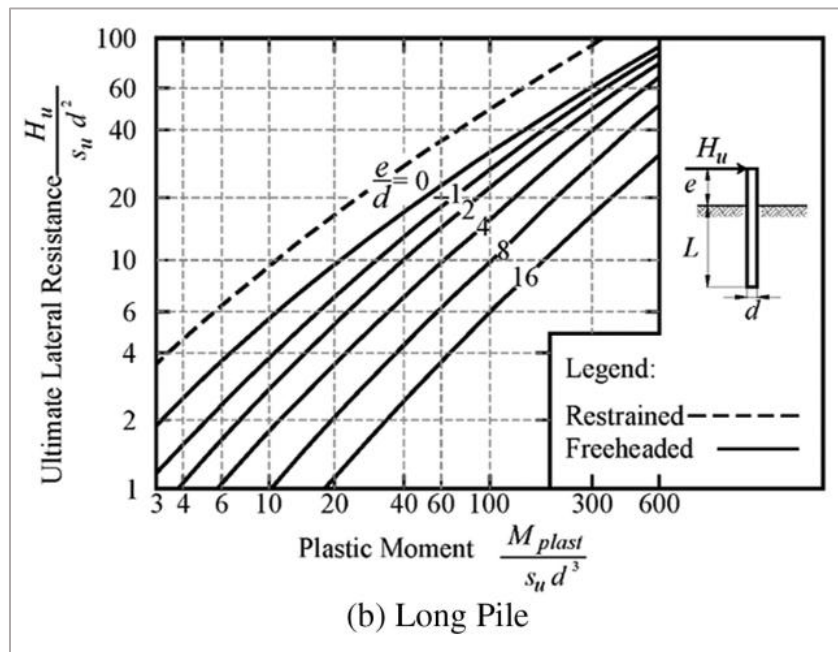
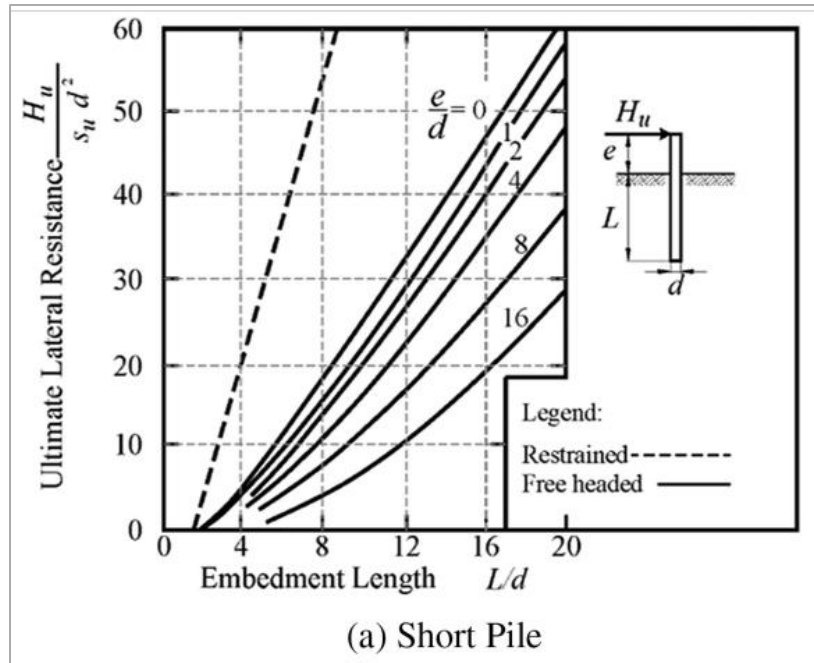


Figure 10-1: Ultimate lateral resistance of piles in cohesive soils (after Broms, CFEM).

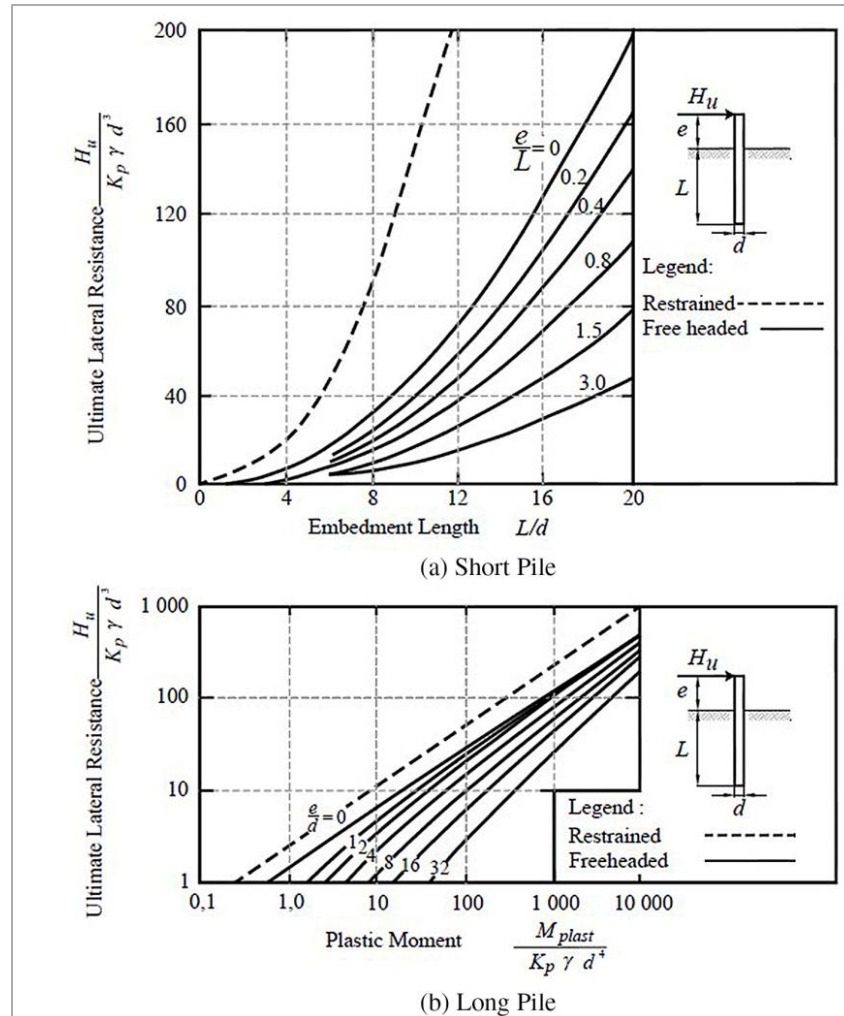


Figure 10-2: Ultimate lateral resistance of piles in **cohesionless soils** (after Broms, CFEM).

The passive lateral resistance of soils within the frost depth should be ignored.

Lateral pile deflections of single piles at SLS

The lateral response of piles under service limit state (SLS) conditions is highly nonlinear. According to the Canadian Foundation Engineering Manual (CFEM 2023), methods that assume linear behavior, such as the theory of subgrade reaction or theory of elasticity, are only suitable when all of the following conditions are met:

- Maximum deflection is less than 1% of the pile diameter
- Loading is purely static (non-cyclic)
- The pile material exhibits linear-elastic behavior (e.g., steel).

In most practical applications, one or more of these conditions are typically not satisfied. As such, nonlinear analysis methods such as p-y curve method are required. The p-y curve method, is widely accepted for evaluating lateral pile deflections under static, cyclic, or transient loading.

Should detailed assessment of lateral pile deflections be required, PNJ can carry out a p–y analysis using LPILE software one more detail are available.

Negative Skin Friction (Downdrag)

When the soil around a pile settles, it exerts a downward force along the pile shaft. This force, called negative skin friction or downdrag, adds to the axial load on the pile and should be accounted for in design. It typically occurs in compressible soils undergoing consolidation.

The unit negative skin friction can be estimated as:

$$q_n = \alpha \cdot s_u$$

where:

q_n = unit negative skin friction (kPa)

α = reduction factor (typically 0.5–1.0)

s_u = undrained shear strength of the consolidated soil (kPa)

The total drag load is computed as:

$$Q_n = q_n \cdot C \cdot D_n$$

where:

Q_n = total drag load (kN)

C = pile perimeter (m)

D_n = embedded length in settling soil (m)

Downdrag does not reduce the geotechnical axial capacity of the pile but must be included in structural and settlement evaluations. Design checks should consider two load cases (CFEM 2023):

- Permanent load plus drag load, but no transient live load
- Permanent load and transient live load, but no drag load.

Pile Groups

The behavior of a pile group is not necessarily equivalent to the sum of the individual pile capacities, due to interaction effects among closely spaced piles. These effects can result in either a reduction or, in some cases, an increase in overall group efficiency depending on the soil conditions, pile type, installation method, and pile spacing.

Group Effect on Axial Resistance

In cohesive soils with an undrained shear strength greater than 100 kPa, and where the pile spacing is equal to or greater than 2.5 times the pile diameter, a group efficiency factor of one (1.0) may be applied, regardless of whether the pile cap is resting on or above the ground. Under these conditions, the capacity of the pile group can be estimated as the sum of the capacities of the individual piles.

Group Effect for Lateral Resistance

Lateral loading of pile groups causes interaction between piles, leading to increased deflections compared to single piles. The effect depends on the direction of the applied load relative to the group, especially for rectangular configurations where one dimension is much longer than the other. Since soil response near the surface is nonlinear, analysis must account for that behavior. The p-y curve method is commonly used, where each pile is modeled individually but with reduced soil resistance using a reduction factor p-multiplier (pm). This approach is typically implemented using analysis software.

Group Effect for Downdrag

Downdrag can be approximated by considering the downdrag stresses on the outer perimeter of the group, unless the piles are very widely spaced.

10.9 Slab on Grade Construction

Subgrade preparation for slab-on-grade should include the removal of any weak or disturbed soils. After removal of all unsuitable materials, the subgrade should be proof rolled. The proof-rolling operation should be witnessed by the Geotechnical Engineer. Any soft or unsuitable subgrade areas which deflect significantly should be sub-excavated and replaced with suitable approved earth fill material compacted to 98% of Standard Proctor Maximum Dry Density (SPMDD).

A moisture barrier having a minimum thickness of 200 mm and comprised of free draining material such as 19 mm clear stone (OPSS 1004) compacted by vibration to a dense state should be used as a base for the slab-on-grade.

A modulus of Subgrade Reaction (K_s) of 15 MPa/m may be used for the design of the slab in accordance with the recommendations provided above.

10.10 Lateral Earth Pressure

The lateral pressure acting on retaining walls or bracing systems is influenced by several factors, including the type of backfill material, the method of compaction, groundwater conditions, as well as the amount and direction of movement permitted by the retaining structure.

The lateral earth pressure can be calculated using the following equation:

$$P = K (\gamma h + q)$$

Where,

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

K = applicable lateral earth pressure coefficient

γ = bulk unit weight of backfill (kN/m³)

q = the complete surcharge loading (kPa)

This equation assumes that free draining backfill and positive drainage are provided to ensure that there is no hydrostatic pressure acting in conjunction with the earth pressure. The coefficient of earth pressure at rest (K_0) should be used in the calculation of the earth pressure on the underground walls.

The design earth pressures in compacted backfill should be augmented with the dynamic effects of the compaction efforts, which typically are taken as a uniform 12 kPa pressure over the entire depth below grade where the calculated earth pressure based on the above earth pressure factors is less than 12 kPa.

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 ($\tan \Phi'$) expressed as: $R = N \tan \Phi'$. This is an ultimate resistance value and does not contain a factor of safety.

The lateral earth pressure can be calculated using the lateral earth pressure coefficients as presented in the following table for static condition. For yielding walls, the active earth pressure coefficient K_a is recommended to be used. For non yielding wall, the at rest K_0 should be used. It is to be noted that large deformation will be required prior to the full mobilization of passive earth pressure. Therefore, a factor of safety is required to be applied with passive earth pressure for design purposes. Values given for horizontal earth pressures are for horizontal backfill. For sloping backfill, the design requirements outlined in the CFEM 2006 should be used. It is noted that for the temporary shoring system that will support the overburden soils, Chapter 26 of CFEM 2006 should be used by designers regarding the distribution of the forces. Earthquakes will induce additional pressures on retaining structures. The lateral earth pressure under seismic condition

can be estimated based on horizontal and vertical seismic coefficient. Site specific seismic hazards as per National Building Code of Canada (2020) were obtained from Earthquakes Canada website (www.EarthQuakesCanada.ca) and are provided in **Appendix F** of this report.

Table 10-5: Coefficient of Lateral Earth Pressure

Soil	γ (kN/m ³)	Angle of Internal Friction ϕ	Coefficient of Lateral Earth Pressure		
			Active K_a	At Rest K_o	Passive K_p
Compact Granular B (OPSS.MUNI 1010)	21	32	0.31	0.47	3.25
Fill (Sandy Silt)	17	24	0.42	0.59	2.37
Silty Clay / Clayey Silt / Silt	20	29	0.35	0.52	2.88
Sandy Silt / Silty Sand	19.5	30	0.33	0.50	3.00
Sand	20	30	0.33	0.50	3.00

Notes:

1. Compacted to a minimum of 95% Standard Proctor Maximum Dry Density.
2. Passive and sliding resistance within the zone subject to frost action should be disregarded in the lateral resistance computations.
3. In the case of a structure below the groundwater table, the use of submerged soil weight should be considered along with the appropriate hydrostatic pressures.
4. Temporary and/or permanent surcharges at the ground surface should be considered in accordance with the applicable soil mechanics methods.

11.0 PAVEMENT DESIGN

11.1 Subgrade Preparation

Based on the borehole findings, and anticipated site grading, the pavement subgrade for the proposed parking area will likely consist of native material (Silty Clay/Clayey Silt), provided a subgrade is proof rolled and approved by the geotechnical engineer prior to the construction of the pavement structure.

Where there is existing fill on the site, caution needs to be exercised in the preparation of the subgrade on this material. It is recommended that any subgrades comprising of existing fill be inspected for obvious soft/loose areas and presence of deleterious materials. Should such areas be found, PNJ can provide appropriate advice for the replacement of the material and addressing local weak areas at that time.

The most severe loading conditions on pavement areas and the subgrade may occur during construction. Consequently, special provisions such as end dumping and forward spreading of sub-base fills, restricted construction lanes, and half-loads during paving may be required, especially if construction is carried out during wet weather conditions.

11.2 Drainage

Control of surface water is an important factor in achieving a good pavement life. The subgrade must be free of depression and sloped (preferably at a minimum grade of 2 percent) to provide effective drainage toward edges and catch basins. Grading adjacent to the proposed driveway area should be designed to ensure that water is not allowed to pond adjacent to the outside edges.

Continuous subdrains comprised of 100 mm diameter perforated pipes, wrapped in filter cloth, and surrounded by 150 mm 19-mm clear stone should be provided and connected to catch basins to facilitate drainage of granular materials. The subdrain invert should be maintained at least 0.1 meters below subgrade level. To minimize the problems of differential movement between the pavement and catch basins / manhole due to frost action, the backfill around the structures should consist of free-draining granular material.

11.3 Recommended Flexible Pavement Structure

The following flexible pavement thickness may be used for the design of potential driveways and parking areas. The pavement designs include a Heavy Duty for access routes and a Standard Duty for car parking areas and are based on providing a maximum design life of 20 years.

Table 11-1: Recommended Pavement – Component Thicknesses

Pavement Layer	Compaction Requirements	Heavy Duty Pavement Design (Driveways)	Light Duty Pavement Design (Parking Areas)
Surface Course Asphaltic Concrete HL3 (OPSS 1150)	92% Maximum Relative Density (OPSS 310)	50 mm	40 mm
Base Course Asphaltic Concrete HL8 (OPSS 1150)	92% Maximum Relative Density (OPSS 310)	70 mm	50 mm
Base Course: Granular 'A' or 19mm Crusher Run	98% to 100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm	150 mm
Subbase Course: Granular B or 50mm Crusher Run		350 mm	250 mm

12.0 EARTHQUAKE DESIGN PARAMETERS

The Ontario Building Code (2024) 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.B of the Ontario Building Code (2024). The classification is based on the determination of the average shear wave velocity in the top 30 meters of the site stratigraphy, where shear wave velocity (v_s) measurements have been taken. In the absence of such measurements, the classification can be 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 (N60) value.

Based on the borehole information and provided that the proposed structures are founded on the native soils, the site designation for seismic analysis is **Class "D"**. A better seismic class can be achieved subject to carrying out Multi Channel Analysis of Surface Waves (MASW) testing within the footprint area of the proposed development. This will allow seismic site class determination based on measured shear wave velocities.

The design spectral acceleration shall be determined in accordance with 2024 Ontario Building Code - Table 4.1.8.4.C, using log-log or linear interpolation for intermediate values of T.

13.0 CONSTRUCTION MONITORING

The foundation installations must be closely monitored and inspected by qualified personnel to ensure that the founding achieved is consistent with the design bearing intended by the design engineer. The on-site review of the condition of the foundation soil as the foundations are constructed is an integral part of the geotechnical design function and is required by Section 4.2.2.3 of the Ontario Building Code 2024.

Qualified Geotechnical personnel should inspect and test all stages of the proposed development. Specifically, they should ensure that the materials and conditions comply with this geotechnical assessment report. In addition, qualified geotechnical personnel should provide material testing services prior to and during foundation preparation and construction. Should soil conditions be encountered that vary from those described in this report, our office should be informed immediately that the proper measures are undertaken.

14.0 LIMITATIONS OF REPORT

The Limitations of Report, as quoted in **Appendix A**, are an integral part of this report.

Yours respectfully,

PNJ Engineering Inc.

Junaid Ashraf., M. Eng., E.I.T

Geotechnical EIT

Geotechnical & Geoenvironmental Services

Abid Sahi., M.A.Sc., P.Geo., P.Eng., QP_{ESA}

Principal

Manager Geotechnical & Geoenvironmental
Services



APPENDICES

APPENDIX A

LIMITATIONS OF THE REPORT

LIMITATIONS OF REPORT

This report is intended solely for King Consultant Inc. (the Client) and their designers and is prohibited for use by others without PNJ's prior written consent. This report is considered PNJ's professional work product and shall remain the sole property of PNJ. Any unauthorized reuse, redistribution of or reliance on the report shall be at the Client and recipient's sole risk, without liability to PNJ. No portion of this report may be used as a separate entity; it is to be read in its entirety and shall include all supporting drawings and appendices.

The description provided in this report are based on our present understanding of the project, the current site use, ground surface elevation and conditions, and are based on the work scope approved by the Client and described in the report. The services were performed in a manner consistent with the level of care and skill ordinarily exercised by members of geotechnical engineering professions currently practicing under similar conditions. No other representations, and no warranties or representations of any kind, either expressed or implied, are made. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties.

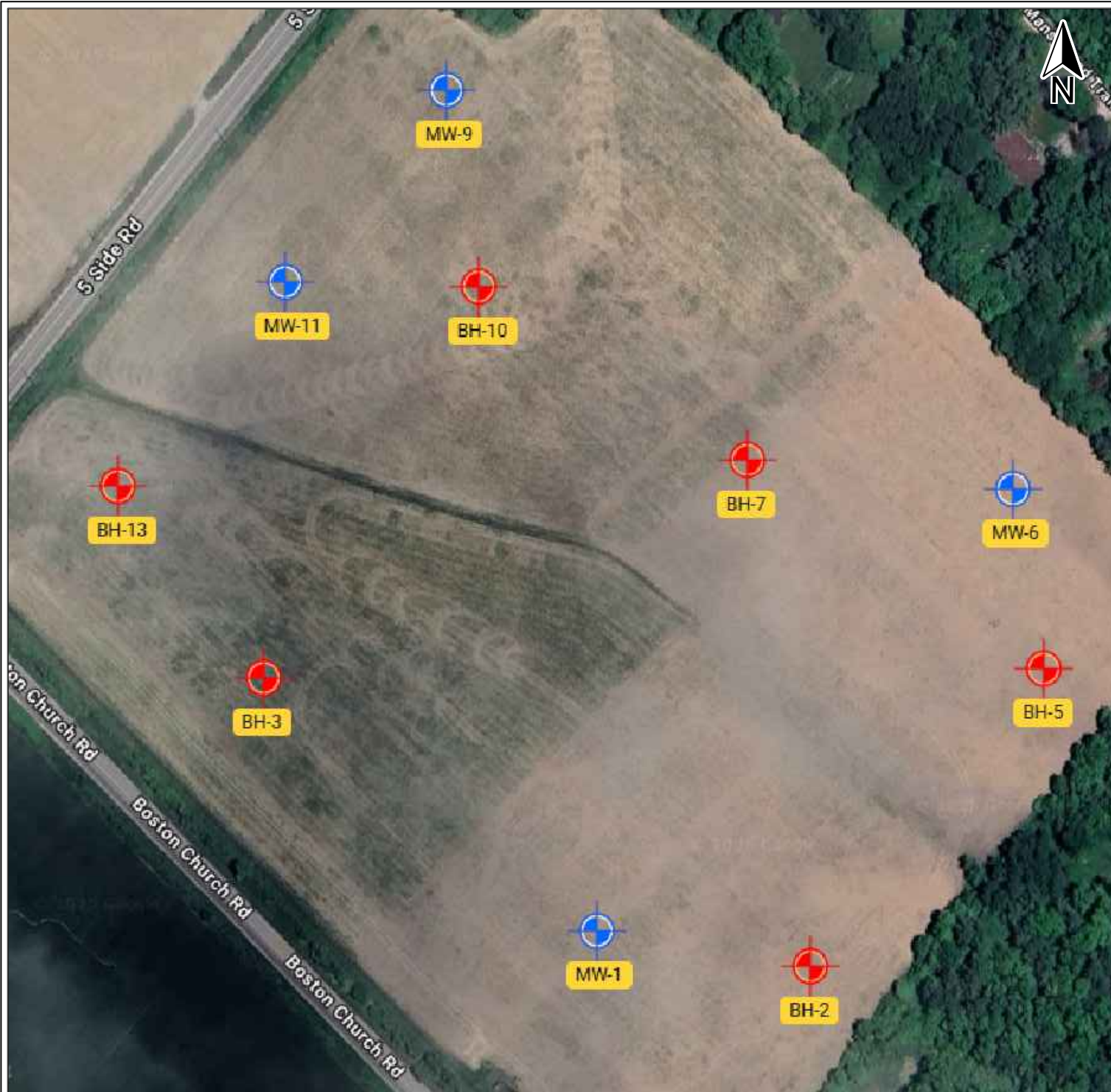
All details of design and construction are rarely known at the time of completion of a geotechnical study. The recommendations and comments made in the study report are based on our completed subsurface investigation and resulting understanding of the project, as defined at the time of the study. We should be retained to review our recommendations when the drawings and specifications are complete. Without this review, PNJ will not be liable for any misunderstanding of our recommendations or their application and adaptation into the final design.

By issuing this report, PNJ is the geotechnical engineer of record. It is recommended that PNJ be retained during construction of all foundations and during earthwork operations to confirm the conditions of the subsoil are confirm if the encountered soils are like those observed during our study. The intent of this requirement is to verify that conditions encountered during construction are consistent with the findings in the report and that inherent knowledge developed as part of our study is correctly carried forward to at the construction phases.

It is important to emphasize that a soil investigation is, in fact, a random sampling of a site and the comments included in this report are based on the results obtained at the test locations only. The subsurface conditions confirmed at the test locations may vary at other locations. The subsurface conditions can also be significantly modified by the construction activities on site (e.g., excavation, dewatering and drainage, blasting, pile driving, etc.). These conditions can also be modified by exposure of soils or bedrock to humidity, dry periods, or frost. Soil and groundwater conditions between and beyond the test locations may differ both horizontally and vertically from those encountered at the test locations and conditions may become apparent during construction which could not be detected or anticipated at the time of our investigation. Should any conditions at the site be encountered which differ from those found at the test locations, we request that we be notified immediately for a reassessment of our recommendations. If changed conditions are identified during construction, no matter how minor, the recommendations in this report shall be considered invalid until sufficient review and written assessment of these conditions is completed by PNJ.

APPENDIX B

BOREHOLE LOCATION PLAN



LEGEND  MONITORING WELL MW-1, MW-6, MW-9, MW-11 (20 ft. / 6.09 m)  BOREHOLE BH-2, BH-3, BH-5 (15 ft. / 4.5 m) BH-7, BH-10, BH-13 (25 ft. / 7.6 m)	
NOTES	
REFERENCES - Google Earth	
TITLE Borehole Location Plan	
PROJECT Geotechnical Investigation 8995 Boston Church Rd., Milton, ON	
CLIENT BC5L Inc.	
DRAWING NO.	1
DATE	July 28, 2025
PREPARED BY	JA
REVIEWED BY	AS
 Pavement - Materials - Geo-Environmental - Engineering 70 Galaxy Blvd Suite 100, Toronto, ON M9W 4Y6 Tel: 905.597.8383 Fax: 905.597.0825 www.pnjeng.com	
PROPOSAL NO. 25-1242-01	
REV. 0	

APPENDIX C

BOREHOLE LOG SHEETS

Notes on Borehole and Test Pit Reports

Soil description :

Each subsurface stratum is described using the following terminology. The relative density of granular soils is determined by the Standard Penetration Index ("N" value), while the consistency of clayey soils is measured by the value of undrained shear strength (Cu).

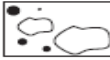
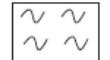
Classification (Unified system)			
Clay	< 0.002 mm		
Silt	0.002 to 0.075 mm		
Sand	0.075 to 4.75 mm	fine	0.075 to 4.25 mm
		medium	0.425 to 2.0 mm
		coarse	2.0 to 4.75 mm
Gravel	4.75 to 75 mm	fine	4.75 to 19 mm
Cobbles	75 to 300 mm	coarse	19 to 75 mm
Boulders	>300 mm		

Terminology	
"trace"	1-10%
"some"	10-20%
adjective (silty, sandy)	20-35%
"and"	35-50%

Relative density of granular soils	Standard penetration index "N" value (BLOWS/ft – 300 mm)
Very loose	0-4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

Consistency of cohesive soils	Undrained shear strength (Cu)	
	(P.S.F)	(kPa)
Very soft	<250	<12
Soft	250-500	12-25
Firm	500-1000	25-50
Stiff	1000-2000	50-100
Very stiff	2000-4000	100-200
Hard	>4000	>200

Rock quality designation	
"RQD" (%) Value	Quality
<25	Very poor
25-50	Poor
50-75	Fair
75-90	Good
>90	Excellent

STRATIGRAPHIC LEGEND			
			
Sand	Gravel	Cobbles & boulders	Bedrock
			
Silt	Clay	Organic soil	Fill

Samples:

Type and Number

The type of sample recovered is shown on the log by the abbreviation listed hereafter. The numbering of samples is sequential for each type of sample.

SS: Split spoon

ST: Shelby tube

AG: Auger

SSE, GSE, AGE: Environmental sampling

PS: Piston sample (Osterberg)

RC: Rock core

GS: Grab sample

Recovery

The recovery, shown as a percentage, is the ratio of length of the sample obtained to the distance the sampler was driven/pushed into the soil

RQD

The "Rock Quality Designation" or "RQD" value, expressed as percentage, is the ratio of the total length of all core fragments of 4 inches (10 cm) or more to the total length of the run.

IN-SITU TESTS:

N: Standard penetration index

Nc: Dynamic cone penetration index

k: Permeability

R: Refusal to penetration

Cu: Undrained shear strength

ABS: Absorption (Packer test)

Pr: Pressure meter

LABORATORY TESTS:

I_p: Plasticity index

H: Hydrometer analysis

A: Atterberg limits

C: Consolidation

O.V.: Organic vapor

W_i: Liquid limit

GSA: Grain size analysis

w: Water content

CS: Swedish fall cone

W_p: Plastic limit

γ: Unit weight

CHEM: Chemical analysis

DRAFT

Monitoring Well: MW-1

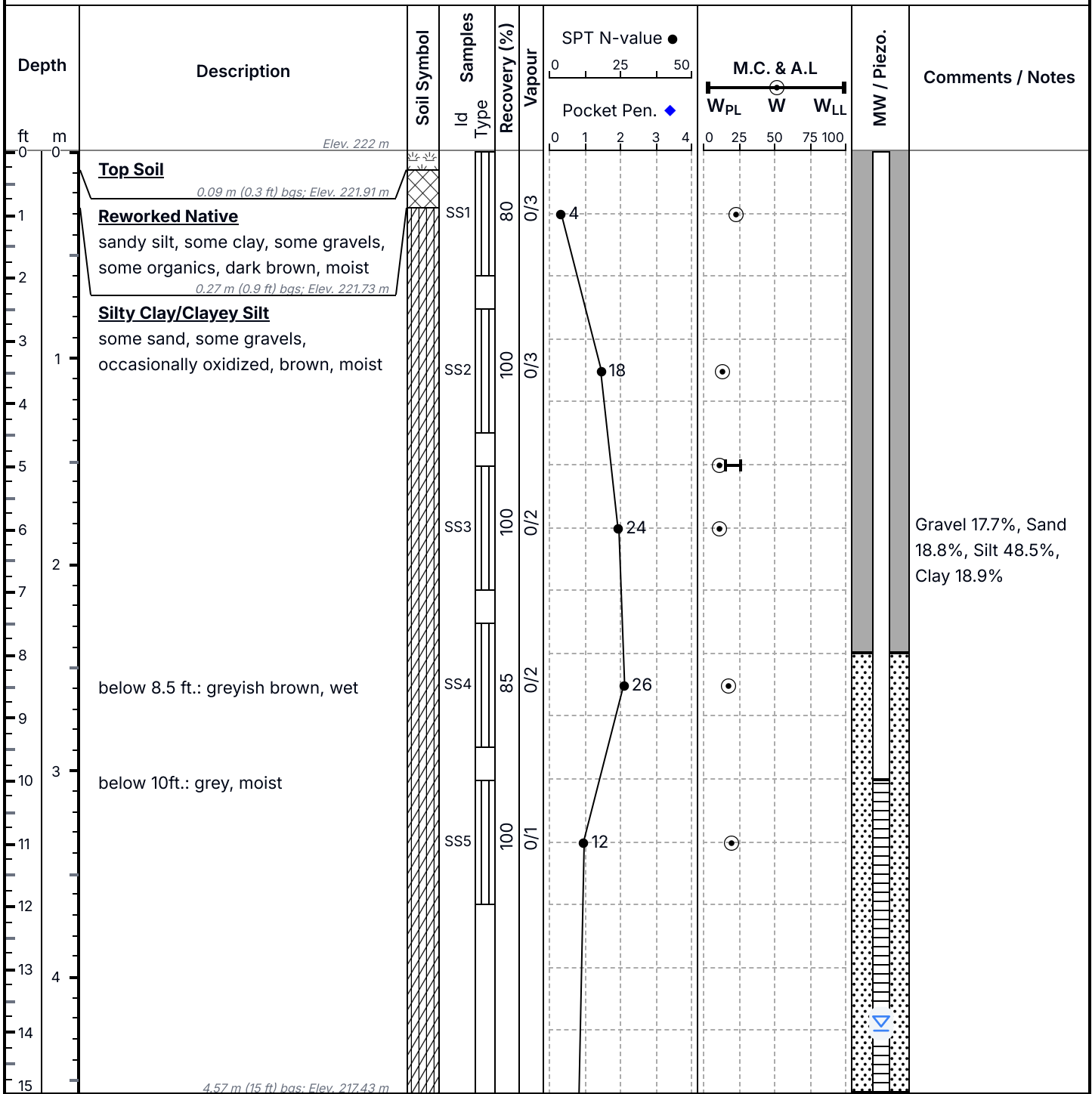
8995 Boston Church Road, Milton

Start Date: 2025-JUL-03

End Date: 2025-JUL-03

Elevation: 222 m AMSL

588240.5 E, 4822320.8 N (NAD83 - 17)



Notes: - Groundwater was measured at 4.2 mbgs immediately after the drilling.

- Borehole was open immediately after the drilling.



Logged By: MUH	Contractor: DrillTech Drilling Ltd.	Client: BC5L Inc.
Approved By: AS	Drilling Details: Solid Stem	Project Title: 8995 Boston Church Road, Milton
Project Eng.: Abid Sahi., P.Eng.		Project No.: 25-1242-01

DRAFT

Monitoring Well: MW-1

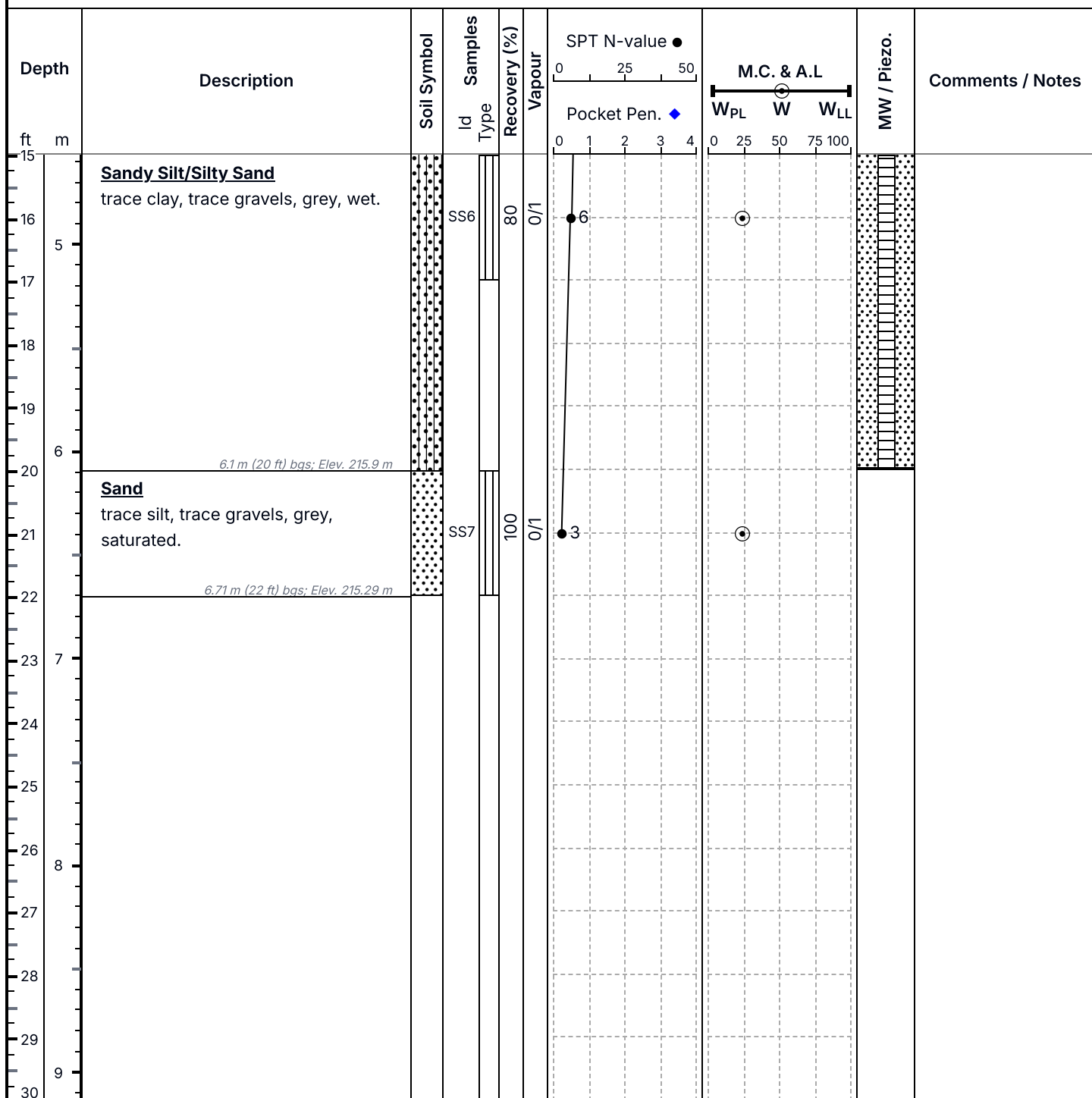
8995 Boston Church Road, Milton

Start Date: 2025-JUL-03

End Date: 2025-JUL-03

Elevation: 222 m AMSL

588240.5 E, 4822320.8 N (NAD83 - 17)



Notes: - Groundwater was measured at 4.2 mbgs immediately after the drilling.

- Borehole was open immediately after the drilling.



Logged By: MUH

Contractor: DrillTech Drilling Ltd.

Client: BC5L Inc.

Approved By: AS

Drilling Details: Solid Stem

Project Title: 8995 Boston Church Road, Milton

Project Eng.: Abid Sahi., P.Eng.

Project No.: 25-1242-01

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DRAFT

Borehole: BH-2

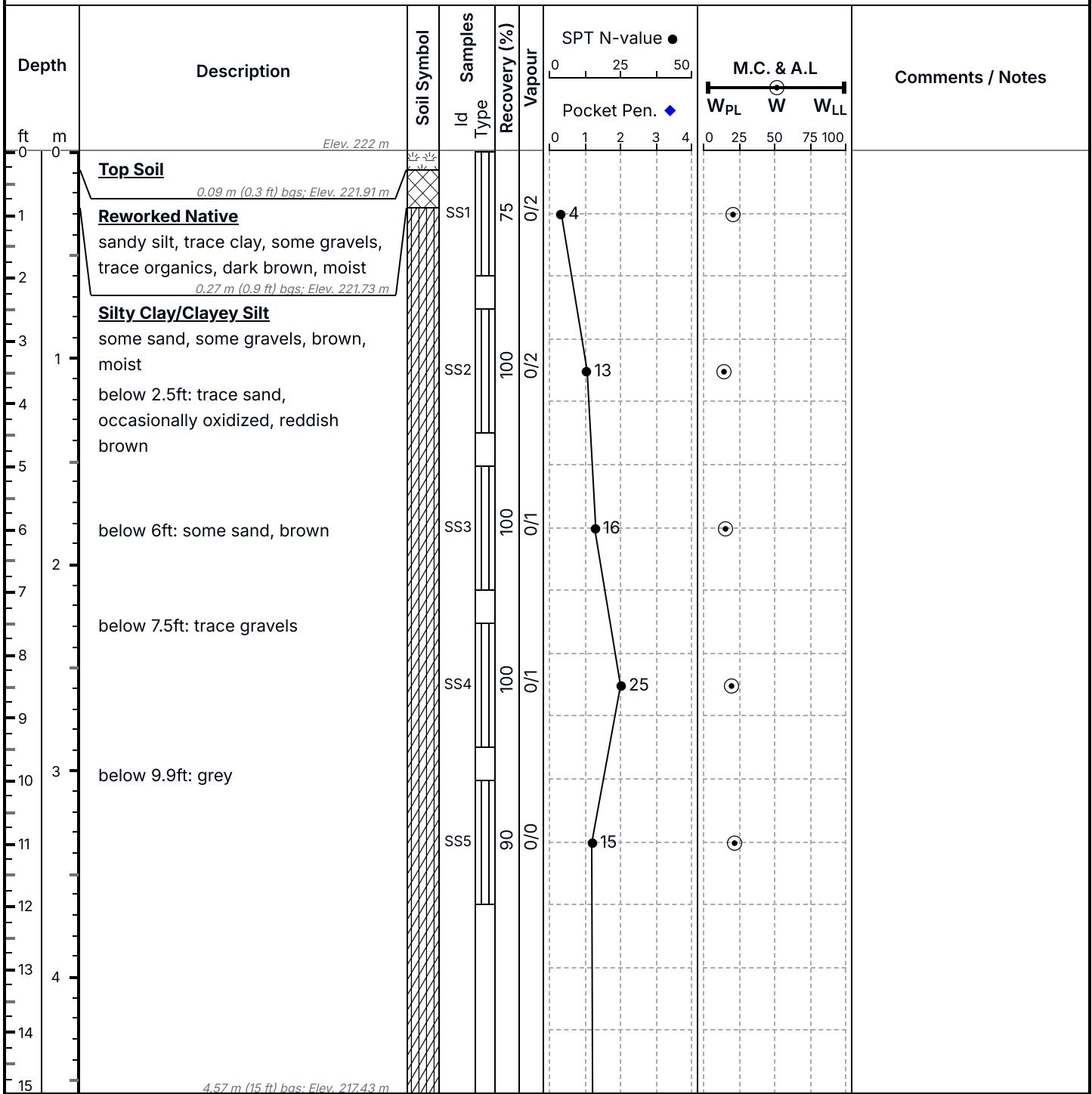
8995 Boston Church Road, Milton

Start Date: 2025-JUL-03

End Date: 2025-JUL-03

Elevation: 222 m AMSL

588315.9 E, 4822309.1 N (NAD83 - 17)



Notes: - Groundwater was measured at 3.9 mbgs immediately after drilling.

- Borehole was open immediately after the drilling.



Logged By: MUH	Contractor: DrillTech Drilling Ltd.	Client: BC5L Inc.
Approved By: AS	Drilling Details: Solid Stem	Project Title: 8995 Boston Church Road, Milton
Project Eng.: Abid Sahi., P.Eng.		Project No.: 25-1242-01
		Page 1/2

DRAFT

Borehole: BH-2

8995 Boston Church Road, Milton

Start Date: 2025-JUL-03

End Date: 2025-JUL-03

Elevation: 222 m AMSL

588315.9 E, 4822309.1 N (NAD83 - 17)

Depth	Description	Soil Symbol	Samples Id Type	Recovery (%)	Vapour	SPT N-value ● 0 25 50 Pocket Pen. ◆ 0 1 2 3 4	M.C. & A.L. W _{PL} W W _{LL} 0 25 50 75 100	Comments / Notes
ft m								
15	Sandy Silt/Silty Sand some clay, trace gravels, grey, wet		SS6	95	0/0	16		
16								
5								
17	5.18 m (17 ft) bgs; Elev. 216.82 m							
18								
19								
20								
21								
6								
22								
23								
24								
25								
26								
8								
27								
28								
29								
9								
30								

Notes: - Groundwater was measured at 3.9 mbgs immediately after drilling.

- Borehole was open immediately after the drilling.



Logged By: MUH	Contractor: DrillTech Drilling Ltd.	Client: BC5L Inc.
Approved By: AS	Drilling Details: Solid Stem	Project Title: 8995 Boston Church Road, Milton
Project Eng.: Abid Sahi., P.Eng.		Project No.: 25-1242-01
		Page 2/2

Borehole: BH-3
8995 Boston Church Road, Milton

Elevation: 222 m AMSL
588122.4 E, 4822407.8 N (NAD83 - 17)



Logged By: MUH	Contractor: DrillTech Drilling Ltd.	Client: BC5L Inc.	
Approved By: AS	Drilling Details: Solid Stem	Project Title: 8995 Boston Church Road, Milton	
Project Eng.: Abid Sahi., P.Eng.		Project No.: 25-1242-01	Page 1/2

DRAFT

Borehole: BH-3

8995 Boston Church Road, Milton

Start Date: 2025-JUL-03

End Date: 2025-JUL-03

Elevation: 222 m AMSL

588122.4 E, 4822407.8 N (NAD83 - 17)

Depth ft m	Description	Soil Symbol	Samples Id Type	Recovery (%)	Vapour	SPT N-value ● 0 25 50	M.C. & A.L. W _{PL} W W _{LL}	Comments / Notes
						Pocket Pen. ◆ 0 1 2 3 4		
15	...							
16	...continued: Silty Clay/Clayey Silt below 15ft: some sand 5 m (16.4 ft) bgs; Elev. 217 m		SS6	100	0/1	22		
17	Sandy Silt/Silty Sand trace clay, trace gravels, gray, moist to wet. 5.18 m (17 ft) bgs; Elev. 216.82 m							
18								
19								
20								
21								
22								
23								
24								
25								
26								
27								
28								
29								
30								

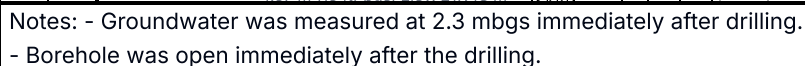
Notes: - Borehole was open and dry immediately after the drilling.



Logged By: MUH	Contractor: DrillTech Drilling Ltd.	Client: BC5L Inc.
Approved By: AS	Drilling Details: Solid Stem	Project Title: 8995 Boston Church Road, Milton
Project Eng.: Abid Sahi., P.Eng.		Project No.: 25-1242-01
		Page 2/2

Borehole: BH-5
8995 Boston Church Road, Milton

Elevation: 222 m AMSL
588396.4 E, 4822414.7 N (NAD83 - 17)



Logged By: MUH	Contractor: DrillTech Drilling Ltd.	Client: BC5L Inc.	
Approved By: AS	Drilling Details: Solid Stem	Project Title: 8995 Boston Church Road, Milton	
Project Eng.: Abid Sahi., P.Eng.		Project No.: 25-1242-01	Page 1/2

DRAFT

Borehole: BH-5

8995 Boston Church Road, Milton

Start Date: 2025-JUL-02

End Date: 2025-JUL-02

Elevation: 222 m AMSL

588396.4 E, 4822414.7 N (NAD83 - 17)

Depth		Description	Soil Symbol	Samples		Recovery (%)	Vapour	SPT N-value ●		M.C. & A.L			Comments / Notes			
ft	m			Id	Type			0	25	50	W _{PL}	W		W _{LL}		
15		Sand trace silt, trace gravels, gray, wet <i>5.18 m (17 ft) bgs; Elev. 216.82 m</i>		SS6		80	0/1	0	25	50						
16	5															
17																
18																
19																
20	6															
21																
22																
23	7															
24																
25																
26	8															
27																
28																
29																
30	9															

Notes: - Groundwater was measured at 2.3 mbgs immediately after drilling.

- Borehole was open immediately after the drilling.



Logged By: MUH	Contractor: DrillTech Drilling Ltd.	Client: BC5L Inc.
Approved By: AS	Drilling Details: Solid Stem	Project Title: 8995 Boston Church Road, Milton
Project Eng.: Abid Sahi, P.Eng.		Project No.: 25-1242-01
		Page 2/2

DRAFT

Monitoring Well: MW-6

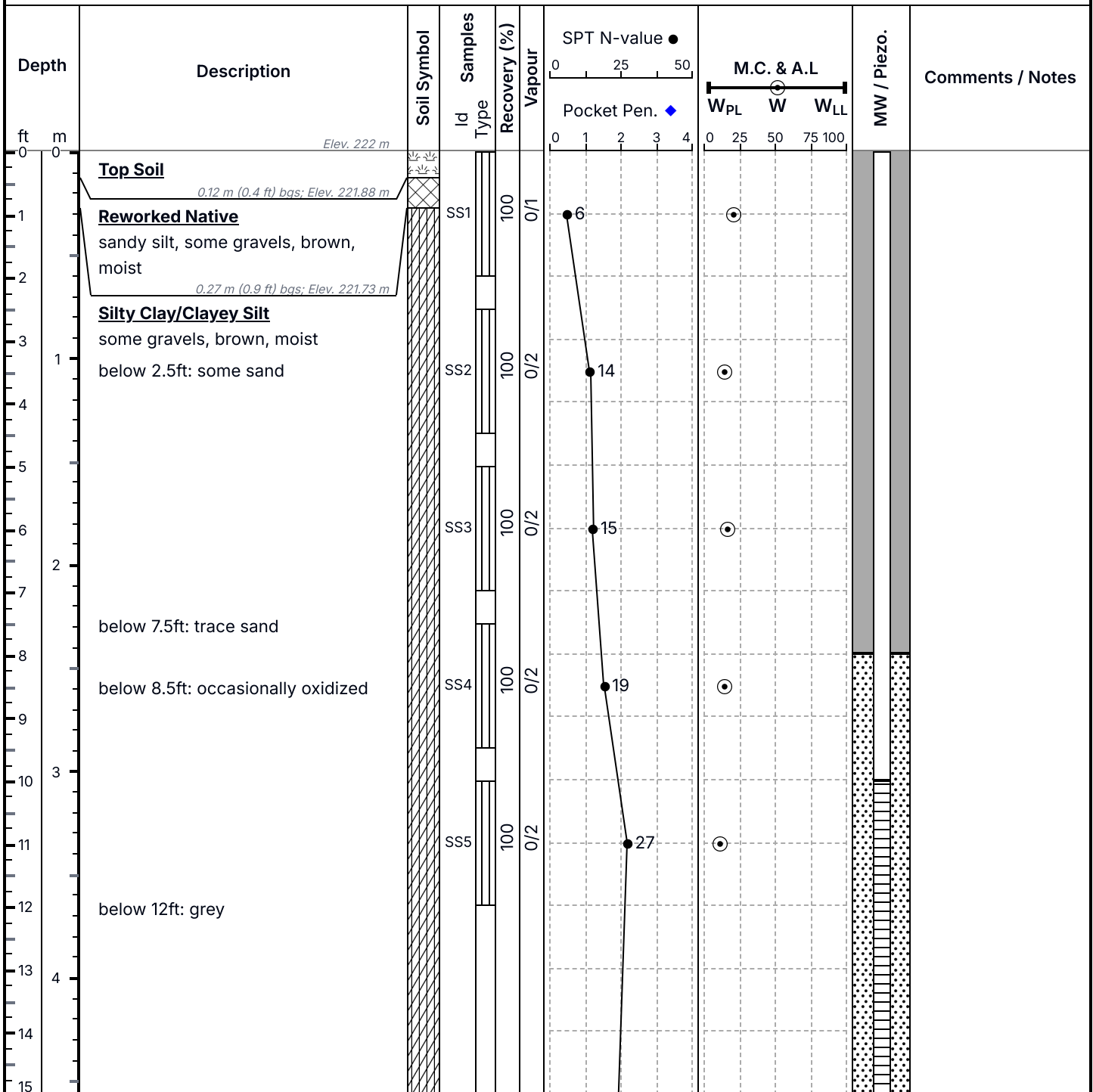
8995 Boston Church Road, Milton

Start Date: 2025-JUL-02

End Date: 2025-JUL-02

Elevation: 222 m AMSL

588384.6 E, 4822477 N (NAD83 - 17)



Notes: - Groundwater was measured at 5.9 mbgs immediately after drilling.

- Borehole was open immediately after the drilling.



Logged By: MUH	Contractor: DrillTech Drilling Ltd.	Client: BC5L Inc.
Approved By: AS	Drilling Details: Solid Stem	Project Title: 8995 Boston Church Road, Milton
Project Eng.: Abid Sahi., P.Eng.		Project No.: 25-1242-01
		Page 1/2

DRAFT

Monitoring Well: MW-6

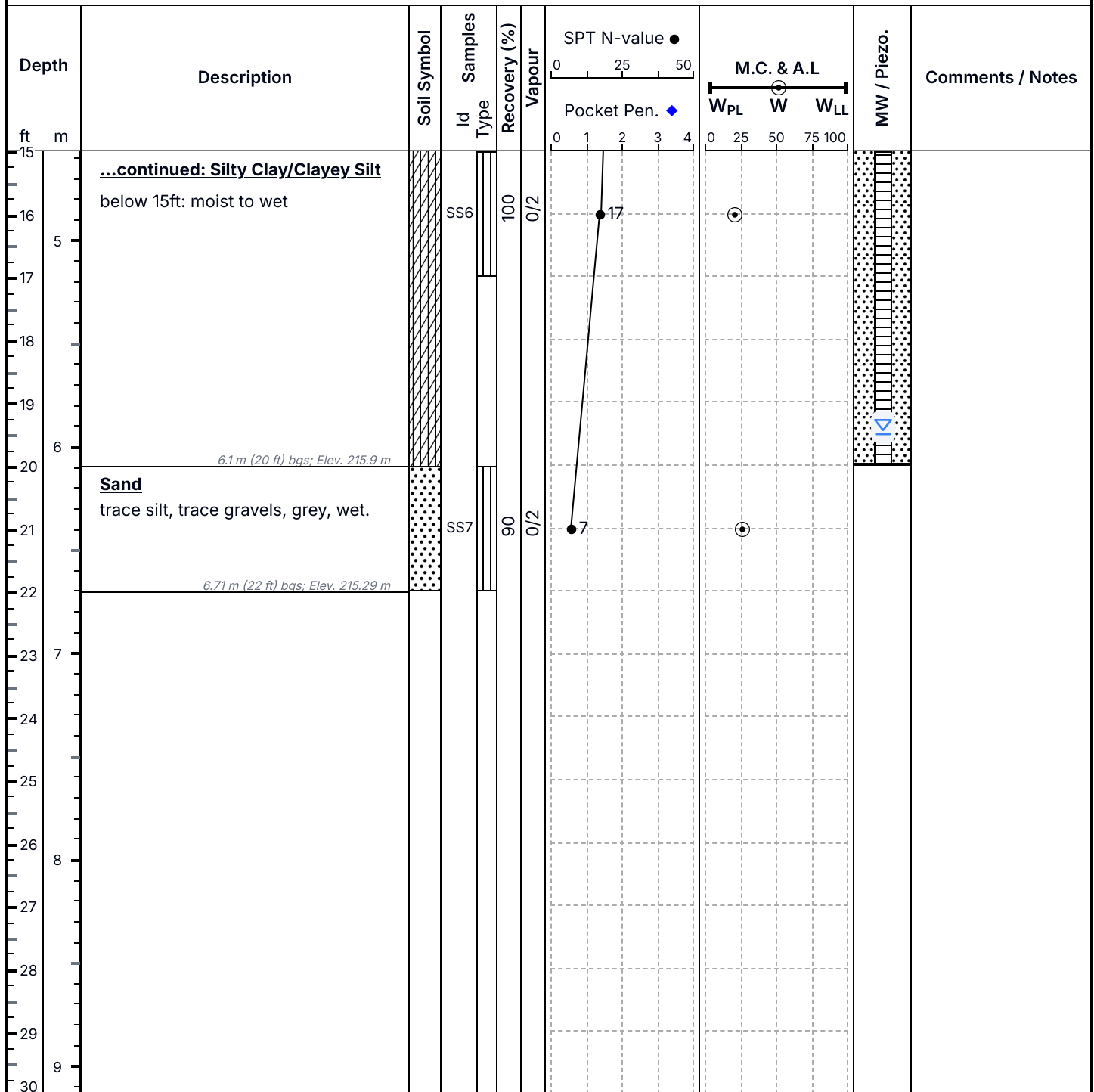
8995 Boston Church Road, Milton

Start Date: 2025-JUL-02

End Date: 2025-JUL-02

Elevation: 222 m AMSL

588384.6 E, 4822477 N (NAD83 - 17)



Notes: - Groundwater was measured at 5.9 mbgs immediately after drilling.

- Borehole was open immediately after the drilling.



Logged By: MUH	Contractor: DrillTech Drilling Ltd.	Client: BC5L Inc.
Approved By: AS	Drilling Details: Solid Stem	Project Title: 8995 Boston Church Road, Milton
Project Eng.: Abid Sahi, P.Eng.		Project No.: 25-1242-01
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DRAFT

Borehole: BH-7

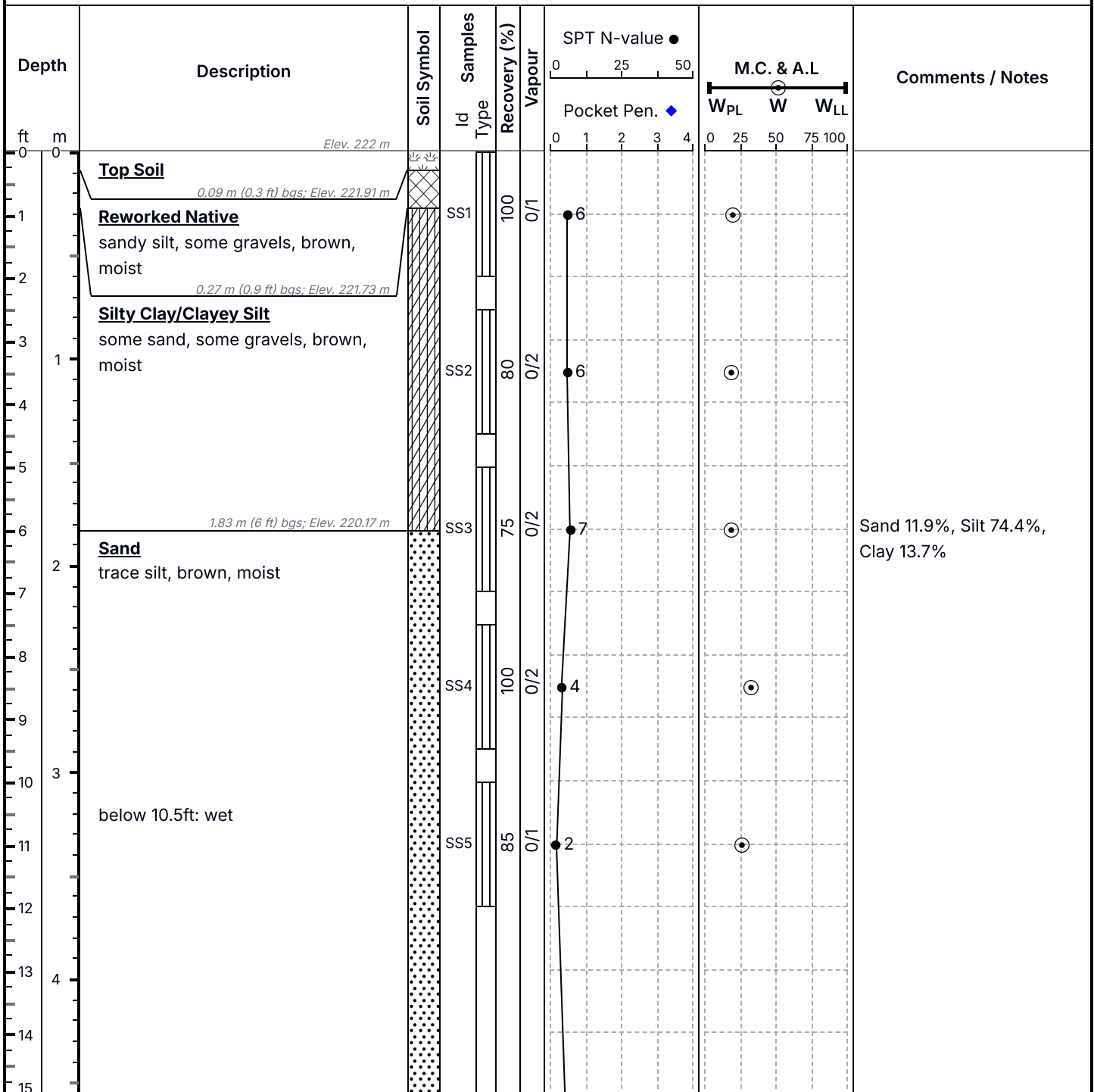
8995 Boston Church Road, Milton

Start Date: 2025-JUL-02

End Date: 2025-JUL-02

Elevation: 222 m AMSL

588291.3 E, 4822486.3 N (NAD83 - 17)



Notes: - Groundwater was measured at 5.3 mbgs immediately after drilling.

- Borehole was open immediately after the drilling.



Logged By: MUH	Contractor: DrillTech Drilling Ltd.	Client: BC5L Inc.
Approved By: AS	Drilling Details: Solid Stem	Project Title: 8995 Boston Church Road, Milton
Project Eng.: Abid Sahi., P.Eng.		Project No.: 25-1242-01
		Page 1/2

DRAFT

Borehole: BH-7

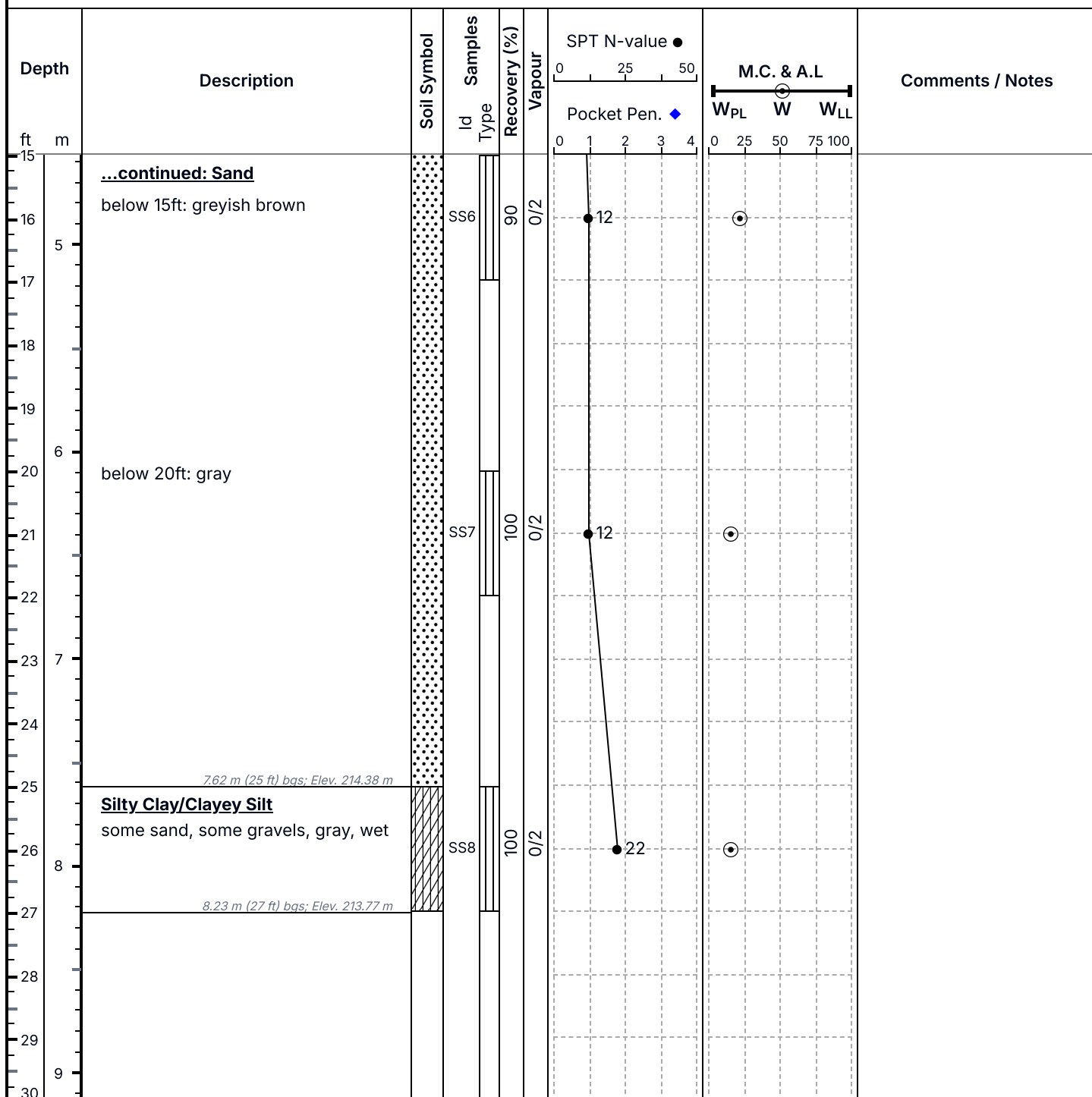
8995 Boston Church Road, Milton

Start Date: 2025-JUL-02

End Date: 2025-JUL-02

Elevation: 222 m AMSL

588291.3 E, 4822486.3 N (NAD83 - 17)



Notes: - Groundwater was measured at 5.3 mbgs immediately after drilling.

- Borehole was open immediately after the drilling.



Logged By: MUH

Contractor: DrillTech Drilling Ltd.

Client: BC5L Inc.

Approved By: AS

Drilling Details: Solid Stem

Project Title: 8995 Boston Church Road, Milton

Project Eng.: Abid Sahi., P.Eng.

Project No.: 25-1242-01

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DRAFT

Monitoring Well: MW-9

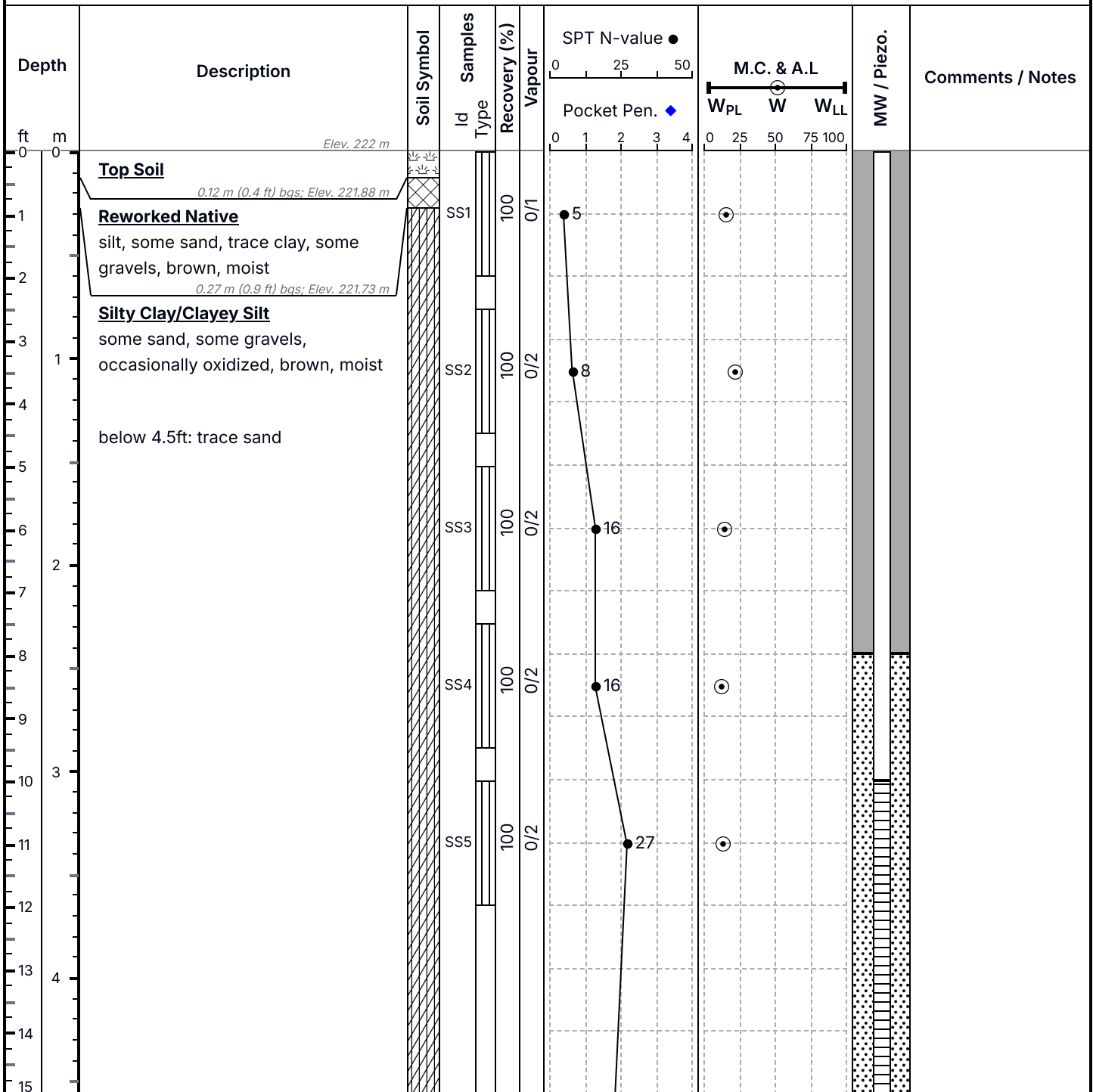
8995 Boston Church Road, Milton

Start Date: 2025-JUL-02

End Date: 2025-JUL-02

Elevation: 222 m AMSL

588184.1 E, 4822613.9 N (NAD83 - 17)



Notes: - Borehole was open and dry immediately after drilling.



Logged By: MUH	Contractor: DrillTech Drilling Ltd.	Client: BC5L Inc.
Approved By: AS	Drilling Details: Solid Stem	Project Title: 8995 Boston Church Road, Milton
Project Eng.: Abid Sahi., P.Eng.		Project No.: 25-1242-01
		Page 1/2

DRAFT

Monitoring Well: MW-9

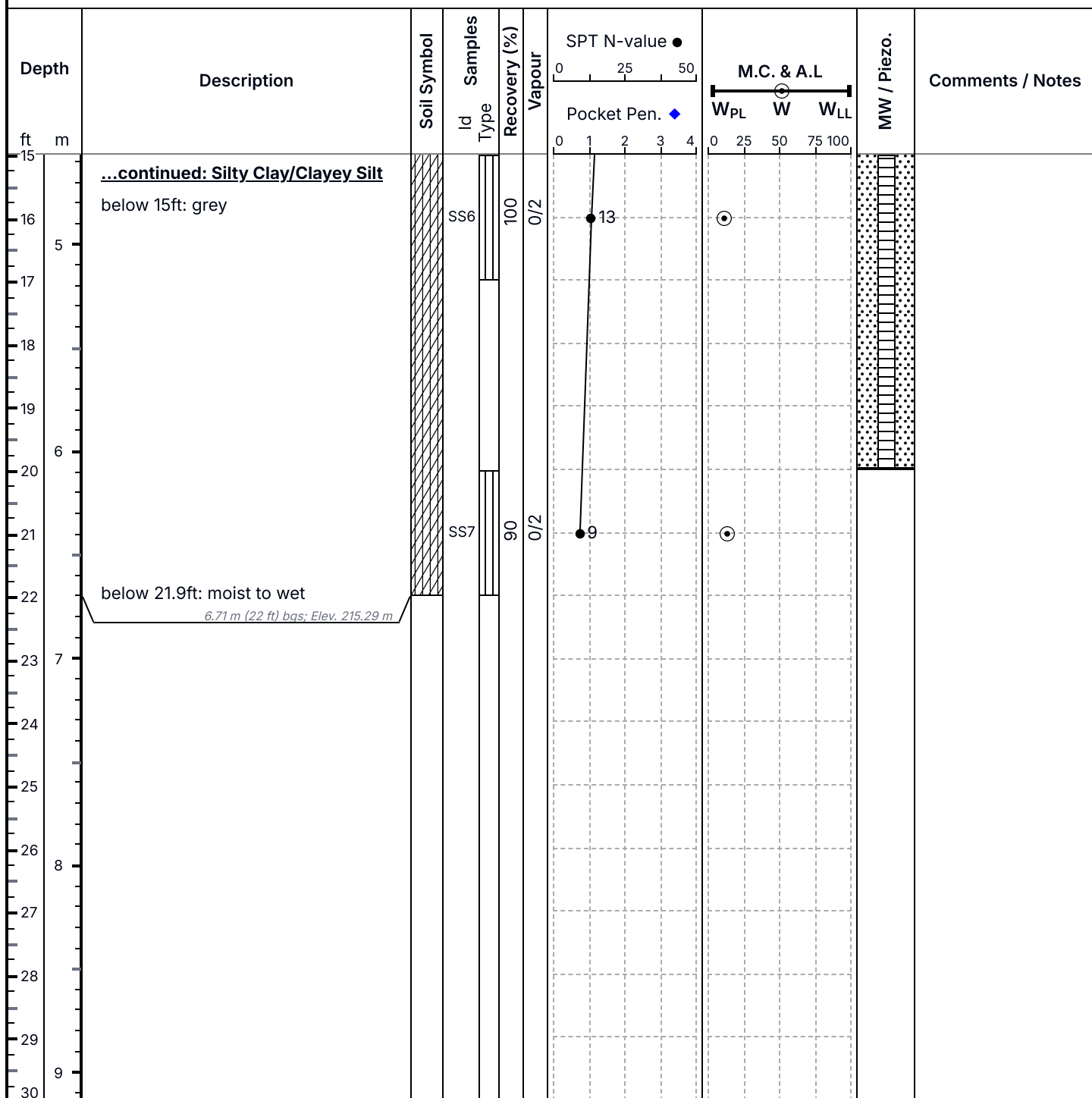
8995 Boston Church Road, Milton

Start Date: 2025-JUL-02

End Date: 2025-JUL-02

Elevation: 222 m AMSL

588184.1 E, 4822613.9 N (NAD83 - 17)



Notes: - Borehole was open and dry immediately after drilling.



Logged By: MUH	Contractor: DrillTech Drilling Ltd.	Client: BC5L Inc.
Approved By: AS	Drilling Details: Solid Stem	Project Title: 8995 Boston Church Road, Milton
Project Eng.: Abid Sahi., P.Eng.		Project No.: 25-1242-01
		Page 2/2

DRAFT

Borehole: BH-10

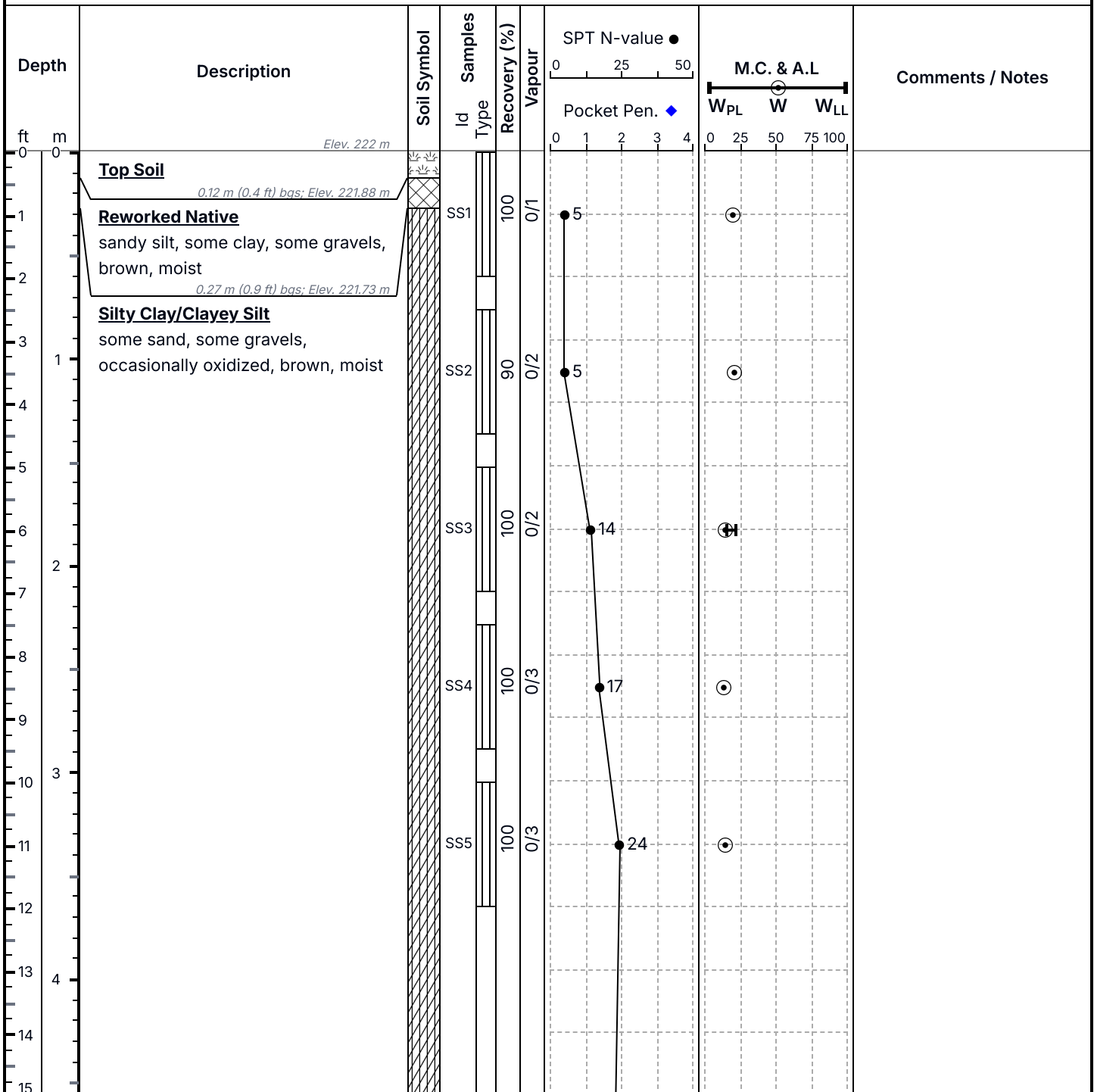
8995 Boston Church Road, Milton

Start Date: 2025-JUL-02

End Date: 2025-JUL-02

Elevation: 222 m AMSL

588196.1 E, 4822545.6 N (NAD83 - 17)



Notes: - Groundwater was measured at 6.1 mbgs immediately after drilling.

- Borehole was open immediately after the drilling.



Logged By: MUH	Contractor: DrillTech Drilling Ltd.	Client: BC5L Inc.
Approved By: AS	Drilling Details: Solid Stem	Project Title: 8995 Boston Church Road, Milton
Project Eng.: Abid Sahi., P.Eng.		Project No.: 25-1242-01
		Page 1/2

DRAFT

Borehole: BH-10

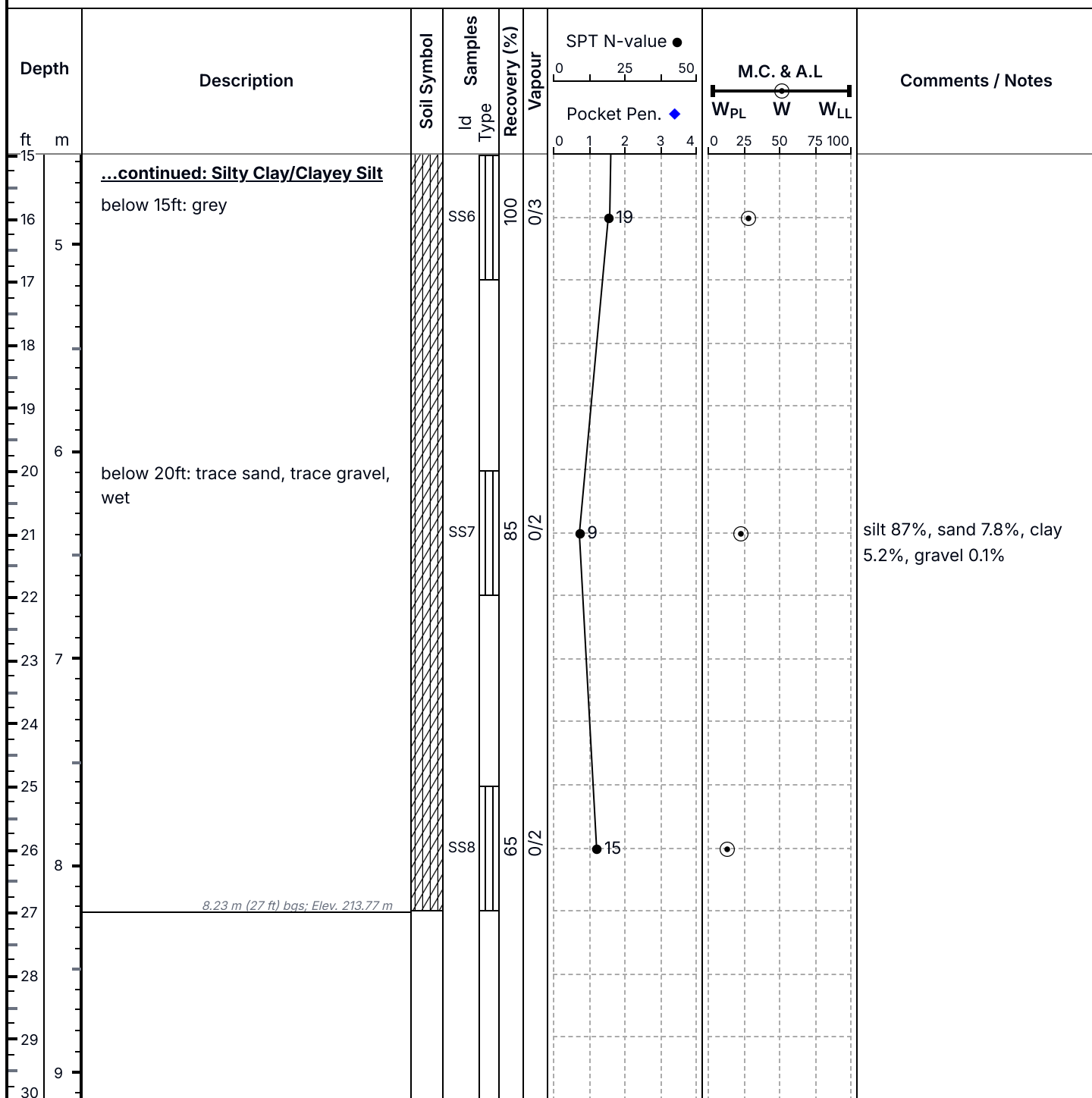
8995 Boston Church Road, Milton

Start Date: 2025-JUL-02

End Date: 2025-JUL-02

Elevation: 222 m AMSL

588196.1 E, 4822545.6 N (NAD83 - 17)



Notes: - Groundwater was measured at 6.1 mbgs immediately after drilling.

- Borehole was open immediately after the drilling.



Logged By: MUH

Contractor: DrillTech Drilling Ltd.

Client: BC5L Inc.

Approved By: AS

Drilling Details: Solid Stem

Project Title: 8995 Boston Church Road, Milton

Project Eng.: Abid Sahi, P.Eng.

Project No.: 25-1242-01

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DRAFT

Monitoring Well: MW-11

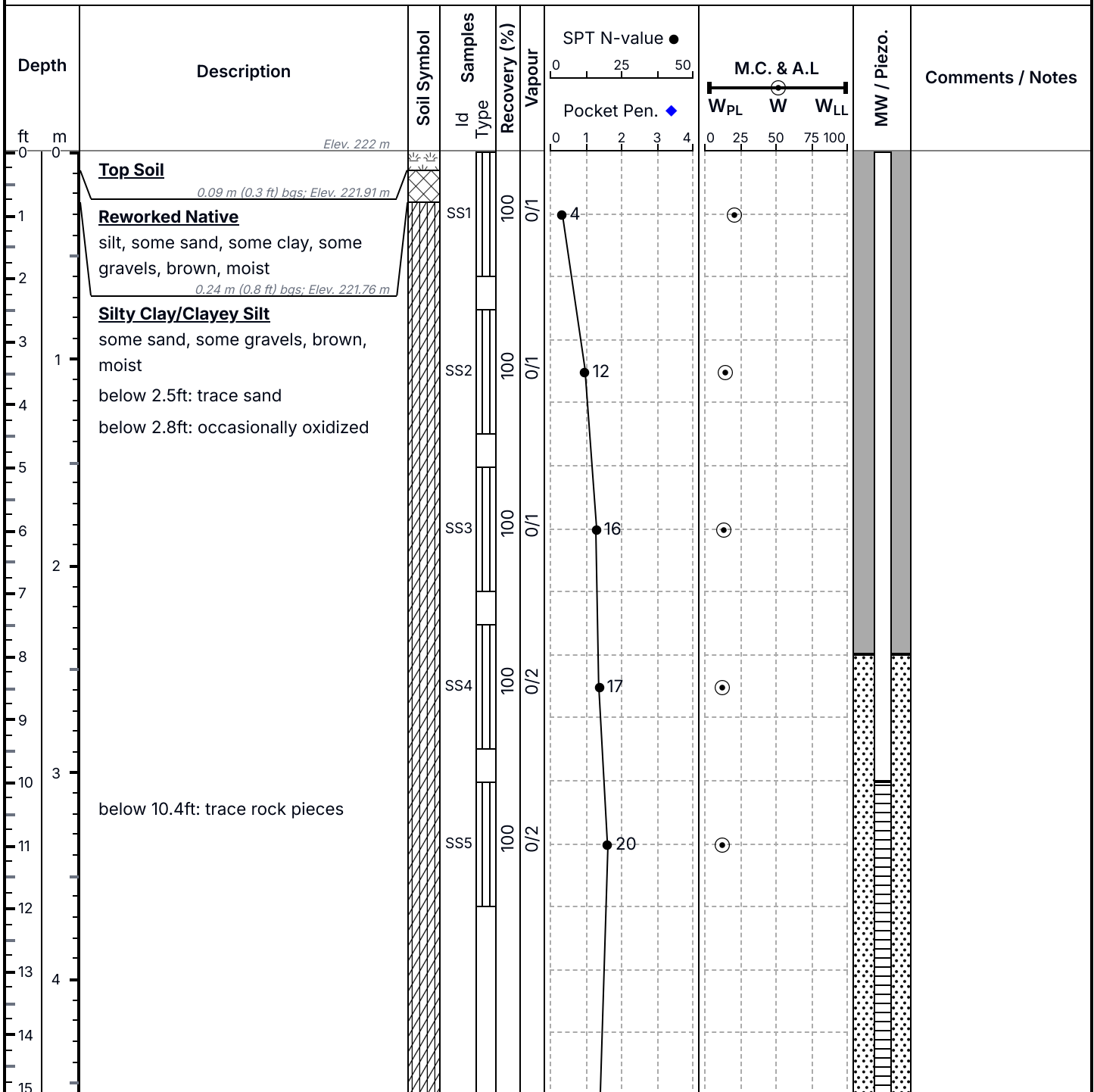
8995 Boston Church Road, Milton

Start Date: 2025-JUL-03

End Date: 2025-JUL-03

Elevation: 222 m AMSL

588128.2 E, 4822546.4 N (NAD83 - 17)



Notes: - Borehole was open and dry immediately after the drilling.



Logged By: MUH	Contractor: DrillTech Drilling Ltd.	Client: BC5L Inc.
Approved By: AS	Drilling Details: Solid Stem	Project Title: 8995 Boston Church Road, Milton
Project Eng.: Abid Sahi., P.Eng.		Project No.: 25-1242-01
		Page 1/2

DRAFT

Monitoring Well: MW-11

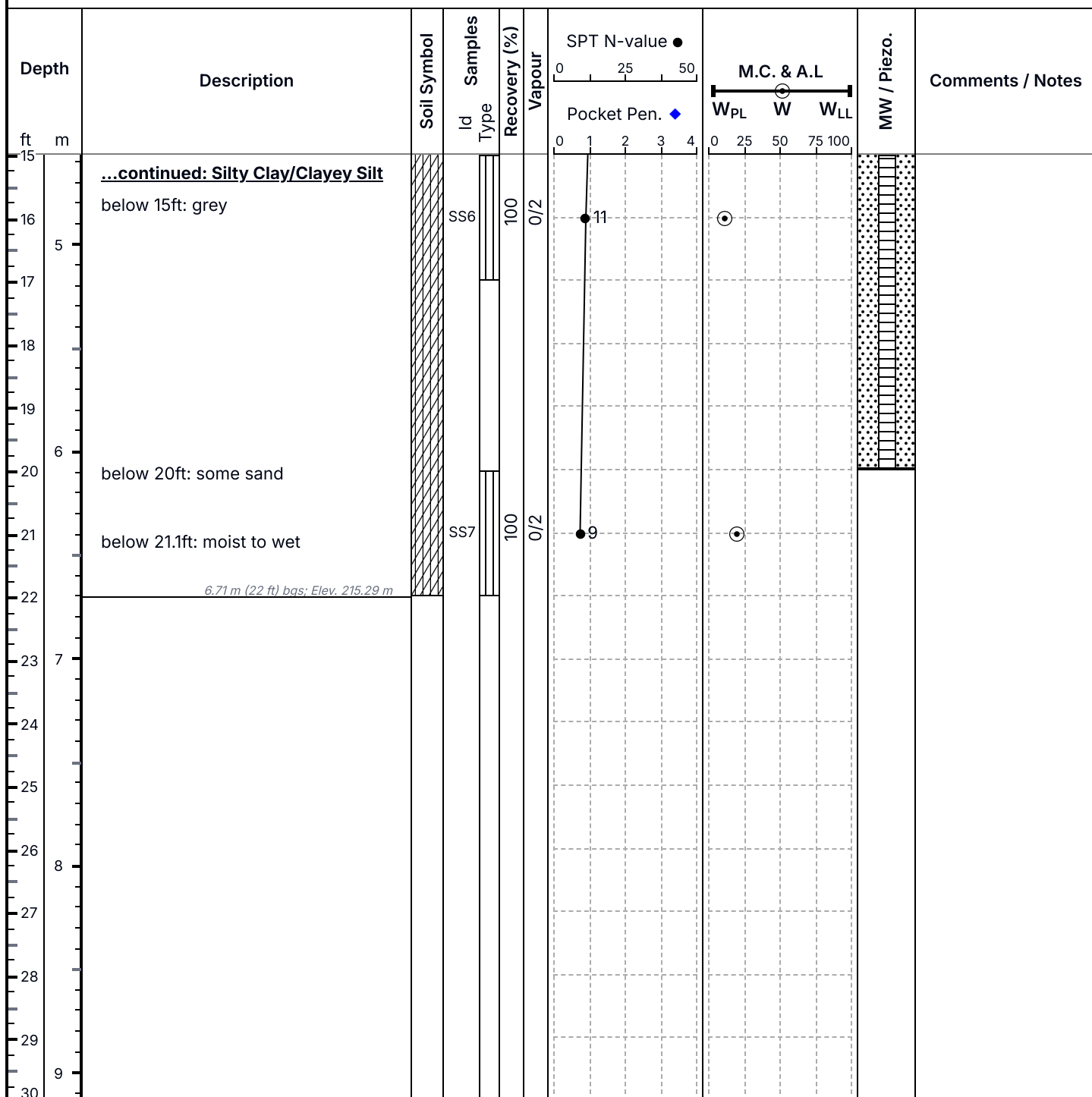
8995 Boston Church Road, Milton

Start Date: 2025-JUL-03

End Date: 2025-JUL-03

Elevation: 222 m AMSL

588128.2 E, 4822546.4 N (NAD83 - 17)



Notes: - Borehole was open and dry immediately after the drilling.



Logged By: MUH	Contractor: DrillTech Drilling Ltd.	Client: BC5L Inc.
Approved By: AS	Drilling Details: Solid Stem	Project Title: 8995 Boston Church Road, Milton
Project Eng.: Abid Sahi., P.Eng.		Project No.: 25-1242-01
		Page 2/2

DRAFT

Borehole: BH-13

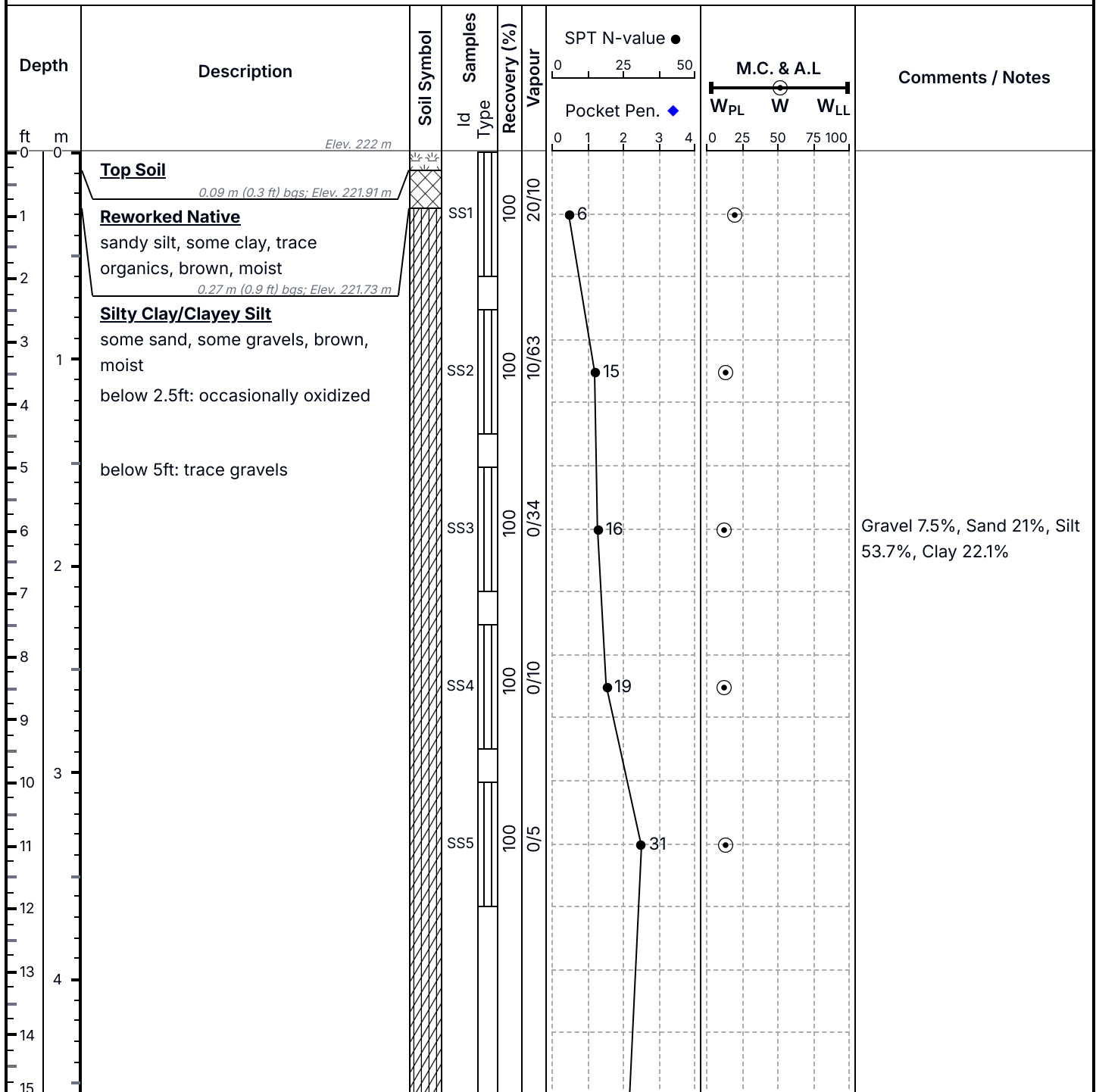
8995 Boston Church Road, Milton

Start Date: 2025-JUL-03

End Date: 2025-JUL-03

Elevation: 222 m AMSL

588070.7 E, 4822474 N (NAD83 - 17)



Notes: Borehole was open and dry immediately after the drilling.



Logged By: MUH	Contractor: DrillTech Drilling Ltd.	Client: BC5L Inc.
Approved By: AS	Drilling Details: Solid Stem	Project Title: 8995 Boston Church Road, Milton
Project Eng.: Abid Sahi., P.Eng.		Project No.: 25-1242-01
		Page 1/2

DRAFT

Borehole: BH-13

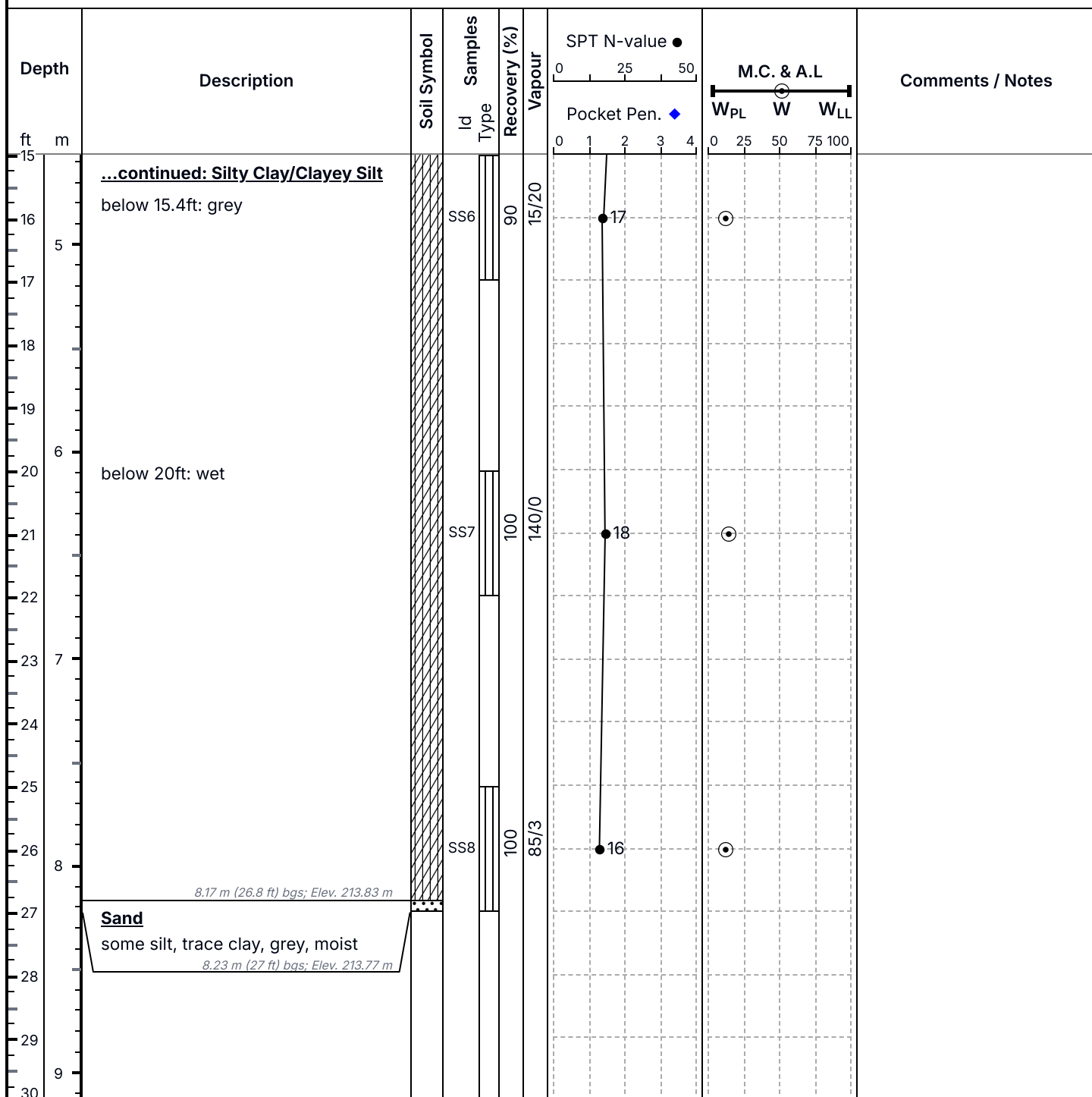
8995 Boston Church Road, Milton

Start Date: 2025-JUL-03

End Date: 2025-JUL-03

Elevation: 222 m AMSL

588070.7 E, 4822474 N (NAD83 - 17)



Notes: Borehole was open and dry immediately after the drilling.



Logged By: MUH	Contractor: DrillTech Drilling Ltd.	Client: BC5L Inc.
Approved By: AS	Drilling Details: Solid Stem	Project Title: 8995 Boston Church Road, Milton
Project Eng.: Abid Sahi., P.Eng.		Project No.: 25-1242-01
		Page 2/2

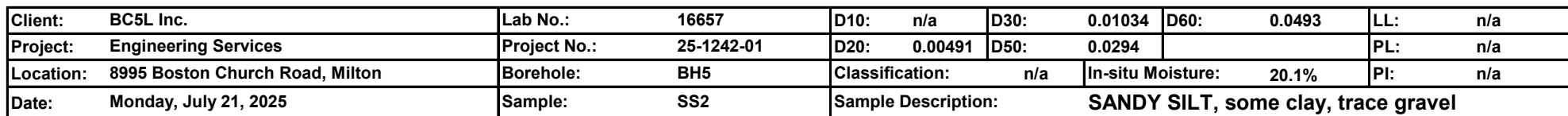
APPENDIX D

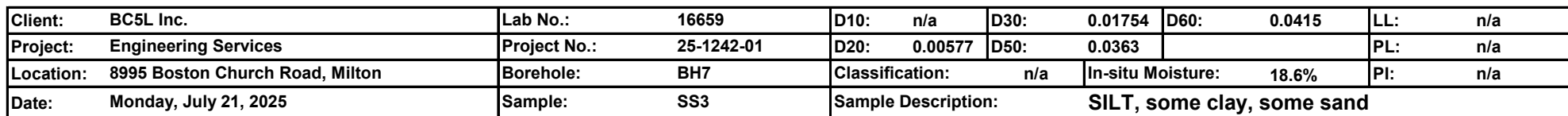
GEOTECHNICAL LABORATORY TEST RESULTS

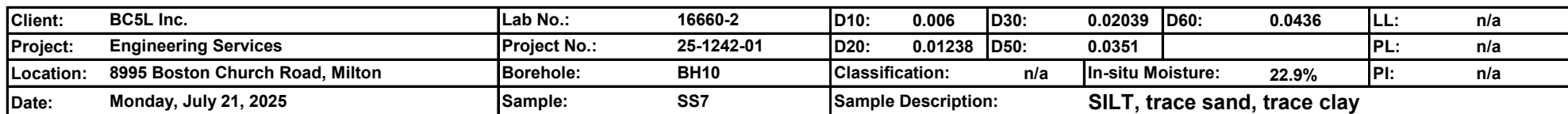
Borehole: MW11

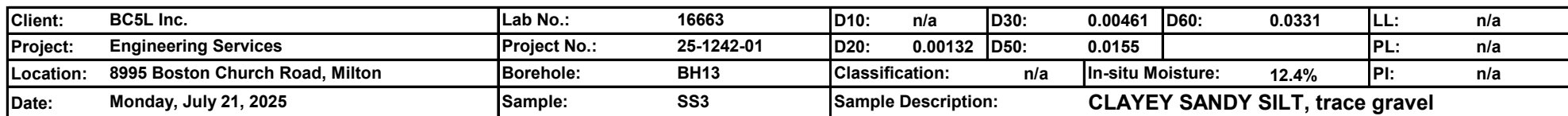
Lab No.: 16662

[illegible]





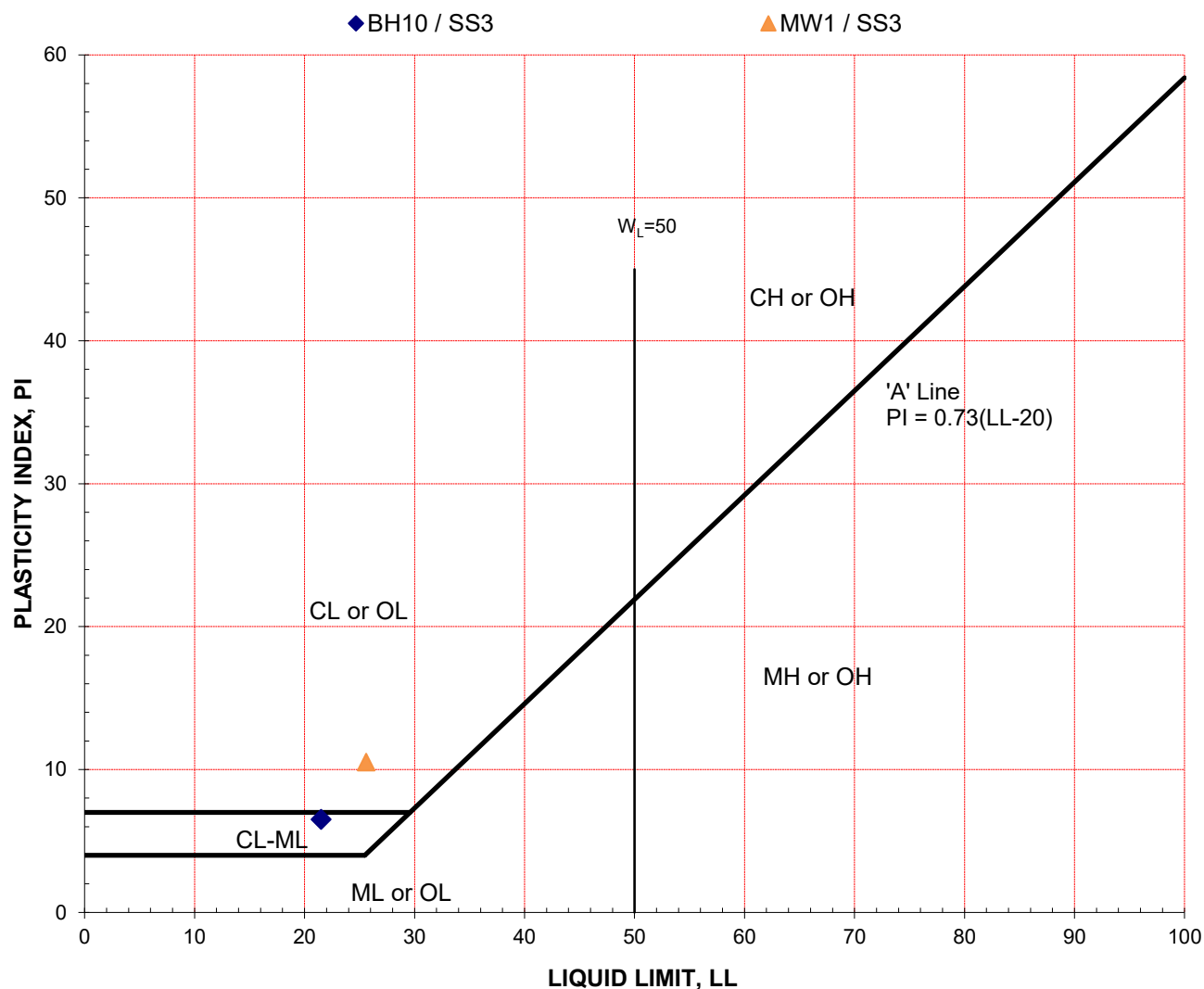




Liquid Limit, Plastic Limit and Plasticity Index

Project No. :	25-1242-01	Technician :	K.S.
Project Client :	BC5L Inc.	Supervisor :	K.J.
Project :	Engineering Services	Date :	21/Jul/25
Location :	8995 Boston Church Road, Milton		

TEST RESULTS									
Specimen #	Sample #	Depth	LL%	PL%	PI	Fines	W%	Classification	Remarks
BH10 / SS3	16660-1	-	21.5	15.0	6.5	-	14.6	CL-ML	-
MW1 / SS3	16665	-	25.6	15.1	10.5	-	11.6	CL	-



APPENDIX E

ANALYTICAL LABORATORY TEST RESULTS



Certificate of Analysis

Client: PNJ Engineering Inc.
70 Galaxy Blvd Suite 100,
Toronto, Ontario
M9W 4Y6
Attention: Muhammad Minhas
PO#:
Invoice to: PNJ Engineering Inc.

Report Number: 3018009
Date Submitted: 2025-07-16
Date Reported: 2025-07-23
Project: 25-1242-01
COC #: 921008

Page 1 of 3

Dear Muhammad Minhas:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

APPROVAL:

Patrick Jacques, Chemist

All analysis is completed at Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) unless otherwise indicated.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on the scope of accreditation. The scope is available at: <https://directory.cala.ca/>.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is licensed by the Ontario Ministry of the Environment, Conservation, and Parks (MECP) for specific tests in drinking water (license #2318). A copy of the license is available upon request.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by the Ontario Ministry of Agriculture, Food, and Rural Affairs for specific tests in agricultural soils.

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official provincial or federal guideline as required. Unless otherwise stated, measurement uncertainty is not taken into account when determining guideline or regulatory exceedances.

Eurofins_multisample(L)45.rpt

Certificate of Analysis

Client: PNJ Engineering Inc.
70 Galaxy Blvd Suite 100,
Toronto, Ontario
M9W 4Y6
Attention: Muhammad Minhas
PO#:
Invoice to: PNJ Engineering Inc.

Report Number: 3018009
Date Submitted: 2025-07-16
Date Reported: 2025-07-23
Project: 25-1242-01
COC #: 921008

					Lab I.D. Sample Matrix Sample Type Sampling Date Sample I.D.	
Group	Analyte	MRL	Units	Guideline	1774551 Soil 2025-07-16 BH13 SS3	1774552 Soil 2025-07-16 BH7 SS3
Anions	Cl	0.002	%		0.003	<0.002
	SO4	0.01	%		<0.01	<0.01
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.18	0.12
	pH	2.00			8.48	7.47
	Resistivity	1	ohm-cm		5556	8333
Moisture	Moisture-Humidite	0.1	%		12.7	15.8
Redox Potential	REDOX Potential		mV		304	259
Subcontract	S2-	0.01	%		<0.01	<0.01

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Certificate of Analysis

Client: PNJ Engineering Inc.
70 Galaxy Blvd Suite 100,
Toronto, Ontario
M9W 4Y6
Attention: Muhammad Minhas
PO#:
Invoice to: PNJ Engineering Inc.

Report Number: 3018009
Date Submitted: 2025-07-16
Date Reported: 2025-07-23
Project: 25-1242-01
COC #: 921008

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 478646 Analysis/Extraction Date 2025-07-21 Analyst AsN Method AG SOIL			
SO4	<0.01 %	101	70-130
Run No 478700 Analysis/Extraction Date 2025-07-22 Analyst AsA Method C CSA A23.2-4B			
Chloride	<0.002 %	104	75-125
Run No 478754 Analysis/Extraction Date 2025-07-22 Analyst NK Method ASTM 2216			
Moisture-Humidite			80-120
REDOX Potential	269 mV	100	97-103
Run No 478758 Analysis/Extraction Date 2025-07-23 Analyst IP Method Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	108	90-110
pH	4.76	99	90-110
Resistivity			
Run No 478762 Analysis/Extraction Date 2025-07-22 Analyst AET Method SUBCONTRACT-SGS			
S2-			

Guideline =

* = Guideline Exceedence

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

APPENDIX F

SEISMIC HAZARD CALCULATION



2020 National Building Code of Canada Seismic Hazard Tool



This application provides seismic values for the design of buildings in Canada under Part 4 of the National Building Code of Canada (NBC) 2020 as prescribed in Article 1.1.3.1. of Division B of the NBC 2020.

Seismic Hazard Values

User requested values

Code edition	NBC 2020
Site designation X_S	X_D
Latitude (°)	43.55
Longitude (°)	-79.911

Please select one of the tabs below.

NBC 2020

Additional Values

Plots

API

Background Information

The 2%-in-50-year seismic hazard values are provided in accordance with Article 4.1.8.4. of the NBC 2020. The 5%- and 10%-in-50-year values are provided for additional performance checks in accordance with Article 4.1.8.23. of the NBC 2020.

NBC 2020 - 2%/50 years (0.000404 per annum) probability							
S _a (0.2, X _D)	S _a (0.5, X _D)	S _a (1.0, X _D)	S _a (2.0, X _D)	S _a (5.0, X _D)	S _a (10.0, X _D)	PGA(X _D)	PGV(X _D)
0.326	0.289	0.168	0.0789	0.0206	0.00641	0.202	0.185

▼ Tables for 5% and 10% in 50 year values

NBC 2020 - 5%/50 years (0.001 per annum) probability							
$S_a(0.2, X_D)$	$S_a(0.5, X_D)$	$S_a(1.0, X_D)$	$S_a(2.0, X_D)$	$S_a(5.0, X_D)$	$S_a(10.0, X_D)$	$PGA(X_D)$	$PGV(X_D)$
0.197	0.177	0.0996	0.0454	0.011	0.00347	0.121	0.106

The log-log interpolated 5%/50 year $S_a(4.0, X_D)$ value is : **0.0155**

NBC 2020 - 10%/50 years (0.0021 per annum) probability

$S_a(0.2, X_D)$	$S_a(0.5, X_D)$	$S_a(1.0, X_D)$	$S_a(2.0, X_D)$	$S_a(5.0, X_D)$	$S_a(10.0, X_D)$	PGA(X_D)	PGV(X_D)
0.121	0.112	0.0617	0.0272	0.00617	0.00195	0.0726	0.0632

The log-log interpolated 10%/50 year $S_a(4.0, X_D)$ value is : **0.0089**

Download CSV

← Go back to the [seismic hazard calculator form](#)

Date modified: 2021-04-06



2020 National Building Code of Canada Seismic Hazard Tool



This application provides seismic values for the design of buildings in Canada under Part 4 of the National Building Code of Canada (NBC) 2020 as prescribed in Article 1.1.3.1. of Division B of the NBC 2020.

Seismic Hazard Values

User requested values

Code edition	NBC 2020
Site designation X_S	X_D
Latitude (°)	43.55
Longitude (°)	-79.911

Please select one of the tabs below.

NBC 2020

Additional Values

Plots

API

Background Information

The NBC 5% damped spectral acceleration values can be viewed in the NBC tab. Additional hazard values for your site can be found below.

The 5%-damped spectral acceleration ($S_a(T)$, where T is the period, in s) and peak ground acceleration (PGA) values are given in units of acceleration due to gravity (g , 9.81 m/s^2). Peak ground velocity (PGV) is given in m/s. Probability is expressed in terms of percent (%) exceedance in 50 years.

By default, all probabilities for the user-specified site designation are shown. Other site designations can be selected from the respective drop-down menu in the table. In low hazard regions, a minimum value of $0.001g$ for $T \leq 2.0s$ and of $0.0001g$ for $T > 2.0s$ is assigned. Further information on the calculation of seismic hazard is provided in the *Background Information* tab.

Site Designation	Probability	$S_a(0.05)$	$S_a(0.1)$	$S_a(0.2)$	$S_a(0.3)$	$S_a(0.5)$	$S_a(1.0)$	$S_a(2.0)$	$S_a(5.0)$	$S_a(10.0)$	PGA	PGV
XD ▼	All ▼											
X_D	2	0.305	0.342	0.326	0.317	0.289	0.168	0.0789	0.0206	0.00641	0.202	0.185
X_D	2.5	0.271	0.305	0.292	0.284	0.26	0.149	0.0698	0.0179	0.00559	0.181	0.164
X_D	3.5	0.222	0.253	0.243	0.238	0.218	0.124	0.0571	0.0143	0.00448	0.151	0.134
X_D	5	0.175	0.203	0.197	0.194	0.177	0.0996	0.0454	0.011	0.00347	0.121	0.106
X_D	7	0.135	0.159	0.157	0.156	0.143	0.0798	0.0358	0.00844	0.00267	0.0959	0.0834
X_D	10	0.0991	0.12	0.121	0.121	0.112	0.0617	0.0272	0.00617	0.00195	0.0726	0.0632
X_D	14	0.0716	0.0893	0.0922	0.0933	0.0867	0.0473	0.0203	0.00444	0.0014	0.0542	0.0473
X_D	20	0.0487	0.063	0.0672	0.0689	0.064	0.0344	0.0143	0.00296	0.00093	0.0385	0.0335
X_D	30	0.0296	0.0402	0.0448	0.0465	0.0432	0.0226	0.00896	0.00173	0.000534	0.0247	0.0214
X_D	40	0.0196	0.0278	0.0321	0.0337	0.031	0.0158	0.00598	0.00109	0.00033	0.0172	0.0147



2020 National Building Code of Canada Seismic Hazard Tool



This application provides seismic values for the design of buildings in Canada under Part 4 of the National Building Code of Canada (NBC) 2020 as prescribed in Article 1.1.3.1. of Division B of the NBC 2020.

Seismic Hazard Values

User requested values

Code edition	NBC 2020
Site designation X_S	X_D
Latitude (°)	43.55
Longitude (°)	-79.911

Please select one of the tabs below.

NBC 2020

Additional Values

Plots

API

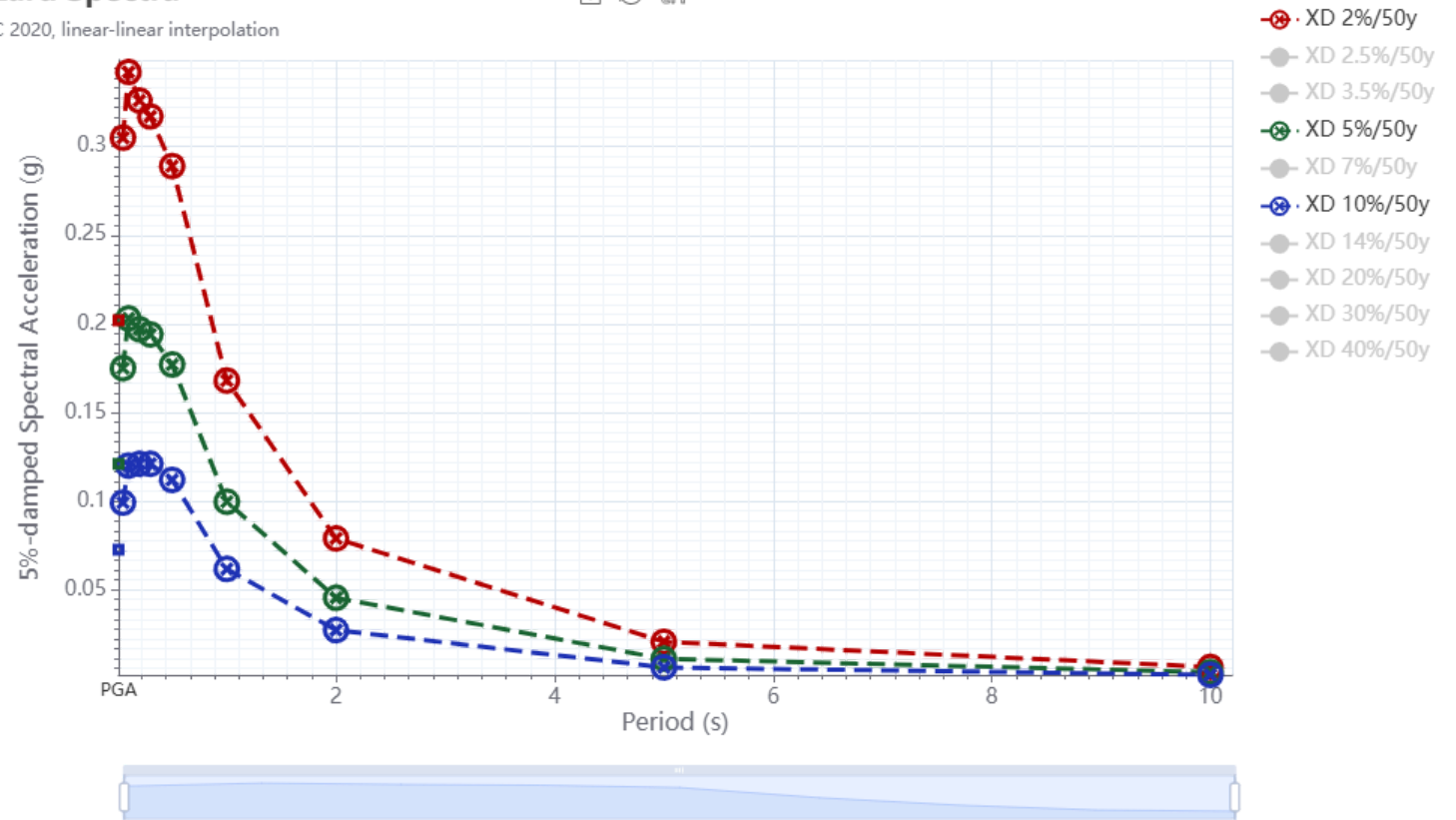
Background Information

Uniform hazard spectra, hazard curves, and site-amplification plots are provided below for the NBC and additional probabilities. By default, only the probabilities referred to in the NBC are displayed, others can be enabled/disabled by selecting the respective item in the legend. Curves representing intermediate points are determined through either linear-linear or log-log interpolation dependent on the scale selected. For convenience, PGA is plotted on the same scale. For low hazard regions, values below 0.001g for $T \leq 2.0s$ and 0.0001g for $T > 2.0s$ are not shown.

► Instructions

Uniform Hazard Spectra

43.55N 79.911W, NBC 2020, linear-linear interpolation



← Go back to the [seismic hazard calculator form](#)

Date modified: 2021-04-06



2020 National Building Code of Canada Seismic Hazard Tool



This application provides seismic values for the design of buildings in Canada under Part 4 of the National Building Code of Canada (NBC) 2020 as prescribed in Article 1.1.3.1. of Division B of the NBC 2020.

Seismic Hazard Values

User requested values

Code edition	NBC 2020
Site designation X_S	X_D
Latitude (°)	43.55
Longitude (°)	-79.911

Please select one of the tabs below.

NBC 2020

Additional Values

Plots

API

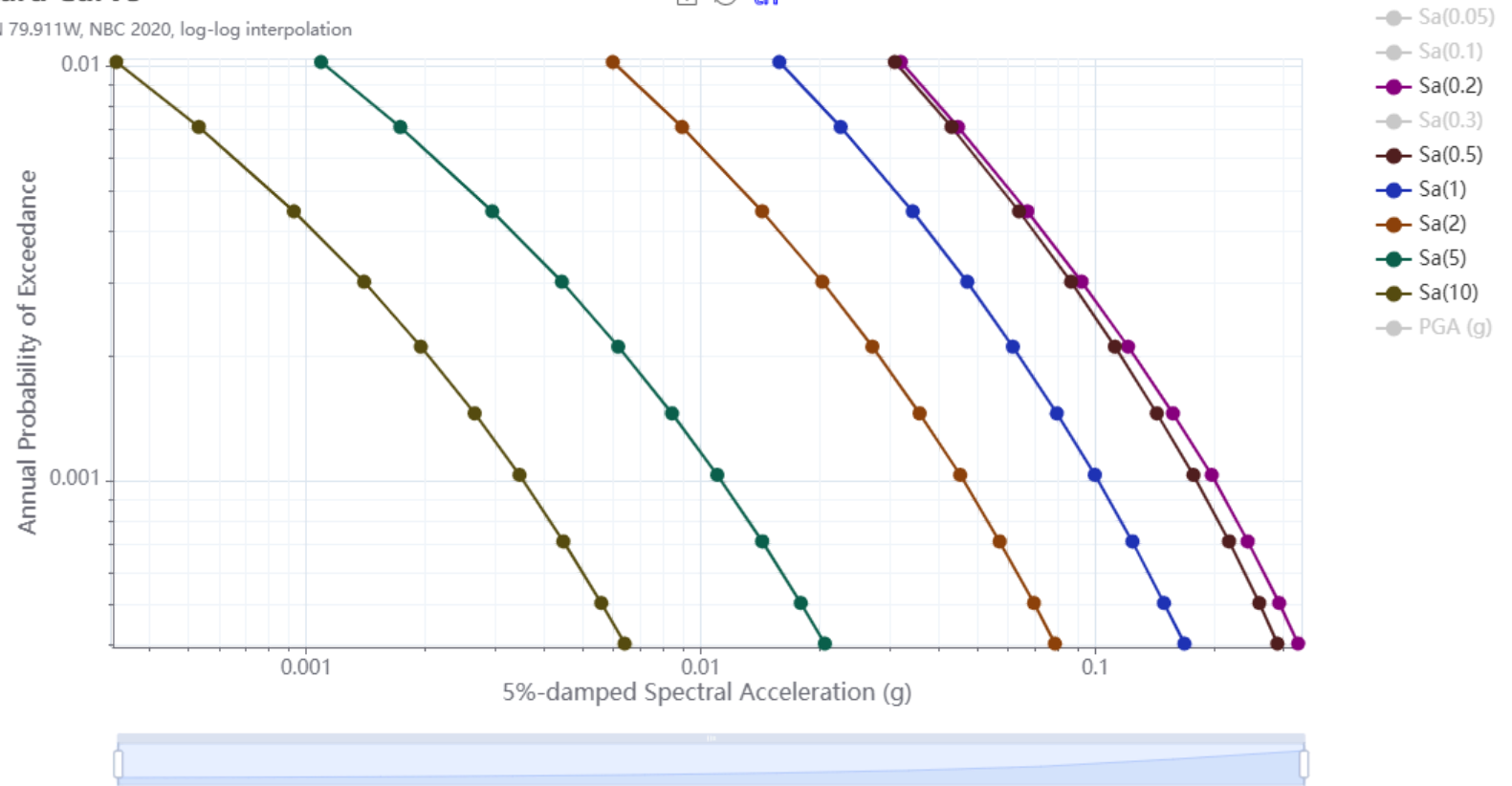
Background Information

Uniform hazard spectra, hazard curves, and site-amplification plots are provided below for the NBC and additional probabilities. By default, only the probabilities referred to in the NBC are displayed, others can be enabled/disabled by selecting the respective item in the legend. Curves representing intermediate points are determined through either linear-linear or log-log interpolation dependent on the scale selected. For convenience, PGA is plotted on the same scale. For low hazard regions, values below 0.001g for $T \leq 2.0s$ and 0.0001g for $T > 2.0s$ are not shown.

► Instructions

Hazard Curve

43.55N 79.911W, NBC 2020, log-log interpolation



← Go back to the [seismic hazard calculator form](#)

Date modified: 2021-04-06



2020 National Building Code of Canada Seismic Hazard Tool



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Seismic Hazard Values

User requested values

Code edition	NBC 2020
Site designation X_S	X_D
Latitude (°)	43.55
Longitude (°)	-79.911

Please select one of the tabs below.

NBC 2020

Additional Values

Plots

API

Background Information

Uniform hazard spectra, hazard curves, and site-amplification plots are provided below for the NBC and additional probabilities. By default, only the probabilities referred to in the NBC are displayed, others can be enabled/disabled by selecting the respective item in the legend. Curves representing intermediate points are determined through either linear-linear or log-log interpolation dependent on the scale selected. For convenience, PGA is plotted on the same scale. For low hazard regions, values below 0.001g for $T \leq 2.0s$ and 0.0001g for $T > 2.0s$ are not shown.

► Instructions

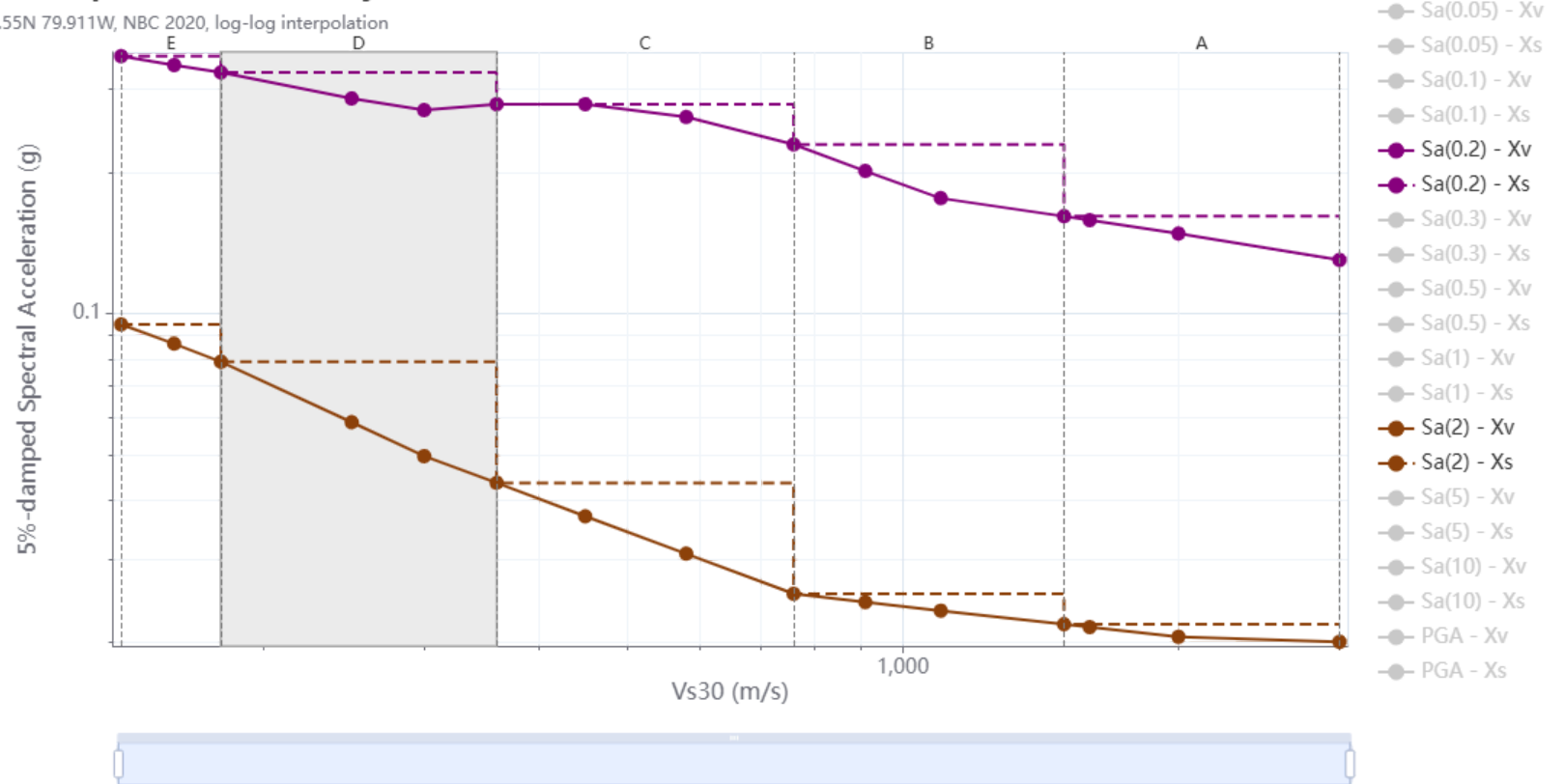
Probability

Site Amplification - 2%/50y



Site Amplification - 2%/50y

43.55N 79.911W, NBC 2020, log-log interpolation

[← Go back to the seismic hazard calculator form](#)

Date modified: 2021-04-06



2020 National Building Code of Canada Seismic Hazard Tool



This application provides seismic values for the design of buildings in Canada under Part 4 of the National Building Code of Canada (NBC) 2020 as prescribed in Article 1.1.3.1. of Division B of the NBC 2020.

Seismic Hazard Values

User requested values

Code edition	NBC 2020
Site designation X_S	X_D
Latitude (°)	43.55
Longitude (°)	-79.911

Please select one of the tabs below.

NBC 2020

Additional Values

Plots

API

Background Information

Uniform hazard spectra, hazard curves, and site-amplification plots are provided below for the NBC and additional probabilities. By default, only the probabilities referred to in the NBC are displayed, others can be enabled/disabled by selecting the respective item in the legend. Curves representing intermediate points are determined through either linear-linear or log-log interpolation dependent on the scale selected. For convenience, PGA is plotted on the same scale. For low hazard regions, values below 0.001g for $T \leq 2.0s$ and 0.0001g for $T > 2.0s$ are not shown.

► Instructions

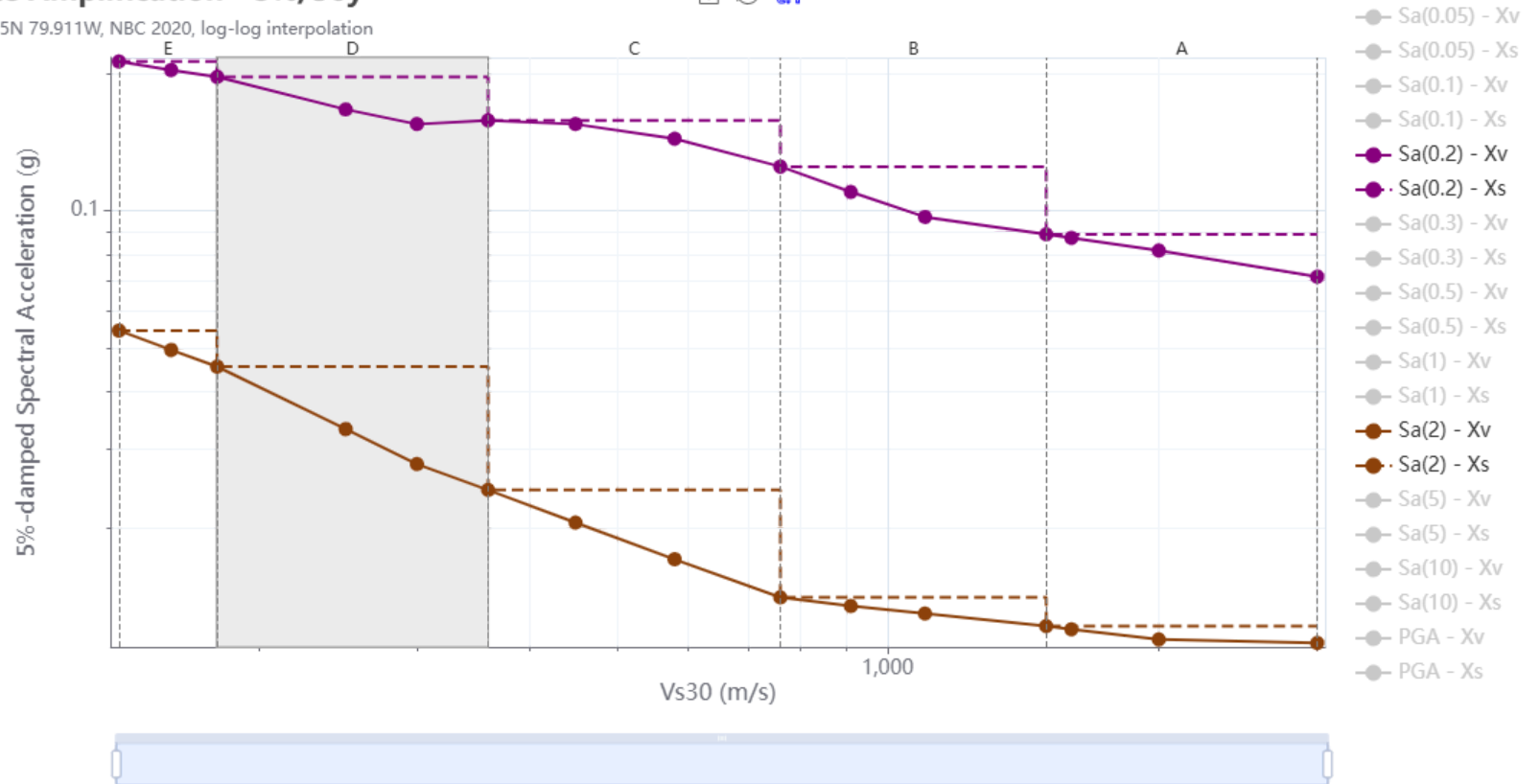
Probability

Site Amplification - 5%/50y



Site Amplification - 5%/50y

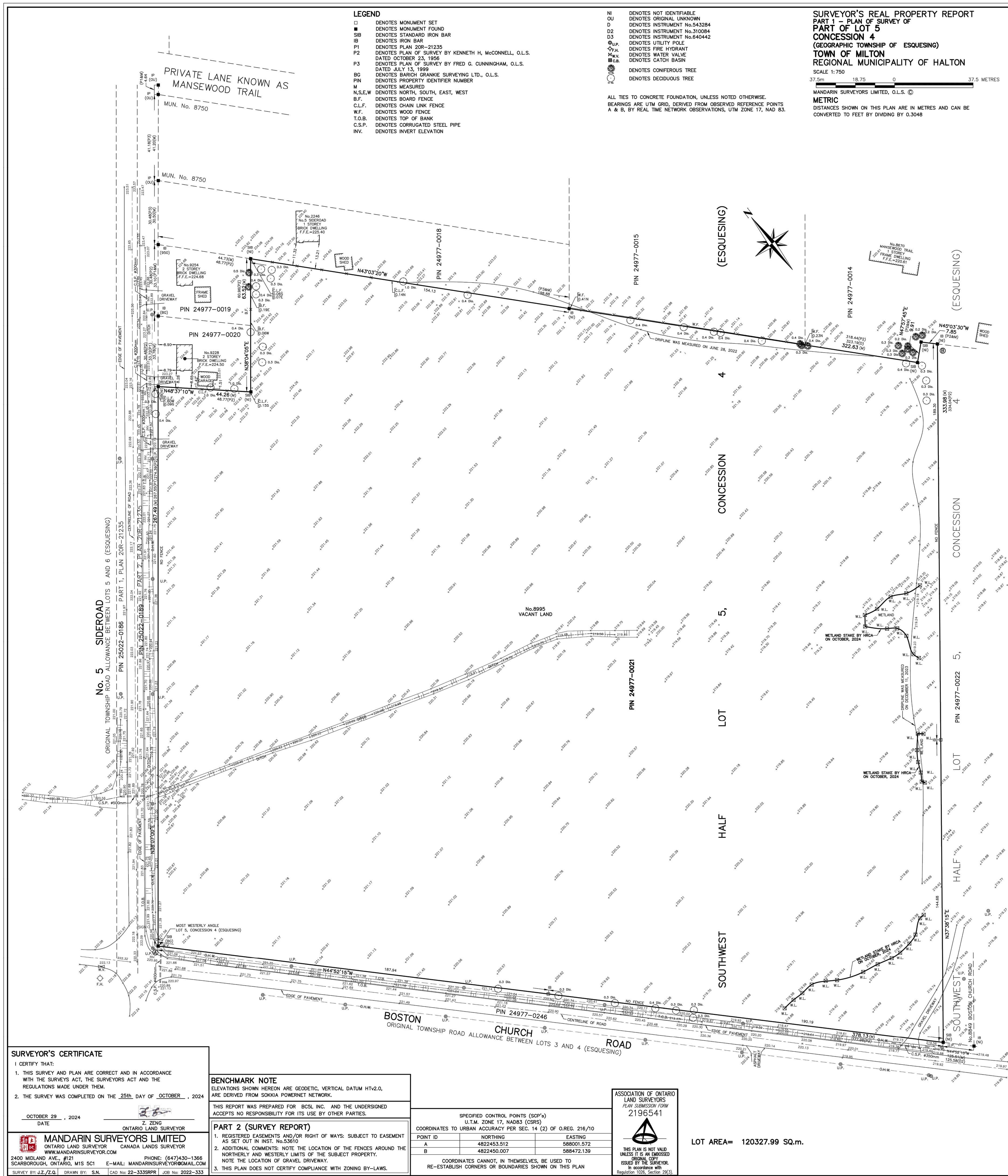
43.55N 79.911W, NBC 2020, log-log interpolation

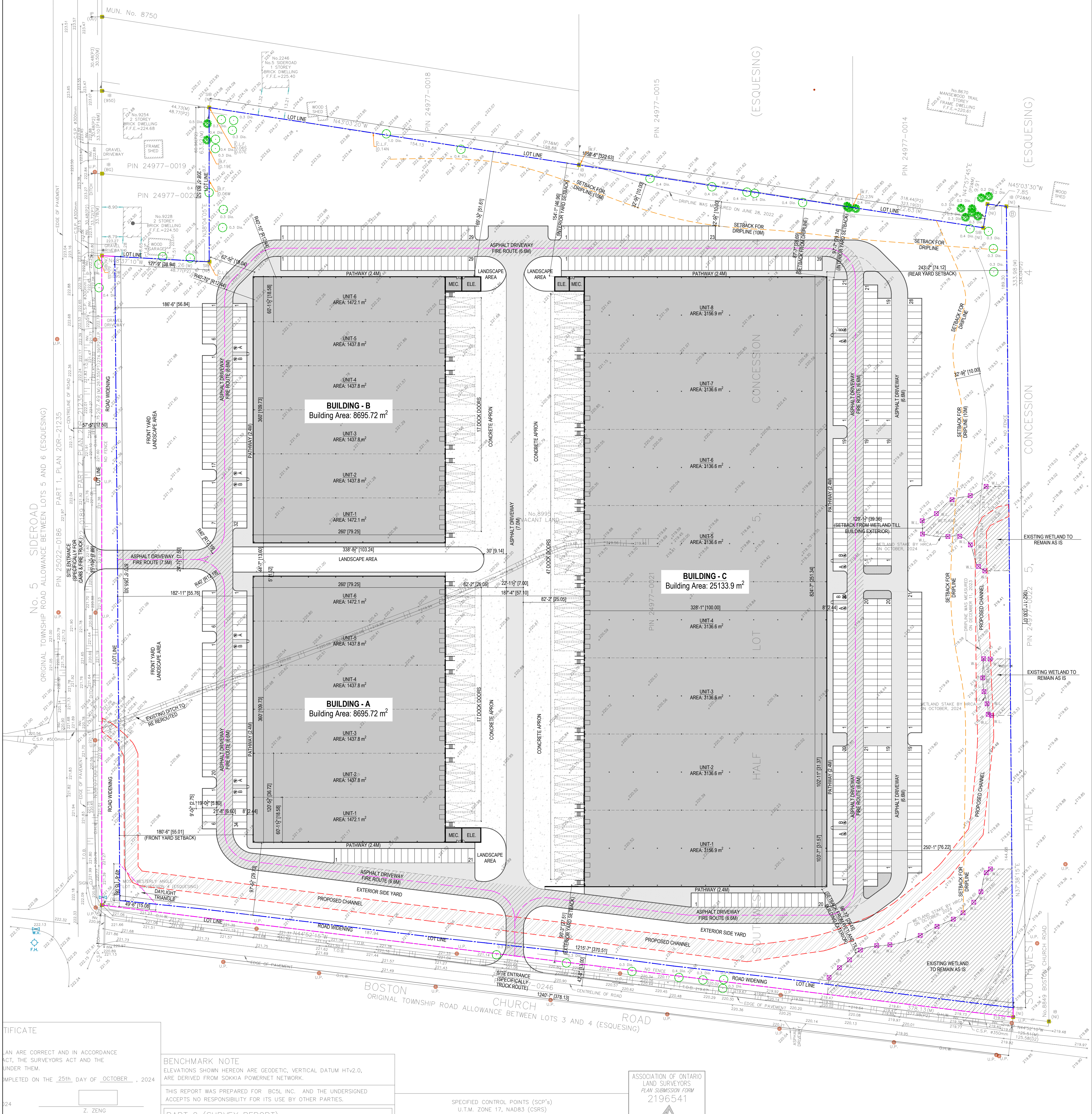
[← Go back to the seismic hazard calculator form](#)

Date modified: 2021-04-06

APPENDIX G

Client Documents





CERTIFICATE

THE PLANS ARE CORRECT AND IN ACCORDANCE WITH THE ACT, THE SURVEYORS ACT AND THE REGULATION.

COMPLETED ON THE 25th DAY OF OCTOBER, 2024

Z. ZENG

BENCHMARK NOTE

ELEVATIONS SHOWN HEREON ARE GEODETIC, VERTICAL DATUM HTV2.0, ARE DERIVED FROM SOKKIA POWERNET NETWORK.

THIS REPORT WAS PREPARED FOR BCSL INC. AND THE UNDERSIGNED ACCEPTS NO RESPONSIBILITY FOR ITS USE BY OTHER PARTIES.

PART 2 (SURVEY REPORT)

ASSOCIATION OF ONTARIO LAND SURVEYORS

PLAN SUBMISSION FORM

2196541

SPECIFIED CONTROL POINTS (SCP's)

U.T.M. ZONE 17, NAD83 (CSRS)

SUMMARY			
ADDRESS	8995 BOSTON CHURCH RD, MILTON		
ZONE	AGRICULTURE		
SITE STATISTICS			
ZONE VARIATION	PERMITTED	EXISTING	PROPOSED
LOT AREA (MINIMUM)		11.71 Hec. (117192.46 sq.m)	
LOT FRONTAGE (MINIMUM)		267.49m	265.30m (After Road Widening)
LOT COVERAGE (MAXIMUM)			36.5% (42778.23 m ²)
FRONT YARD SETBACK (M) (MINIMUM)			55.01m
INTERIOR SIDE YARD DEPTH (M) (MINIMUM)			29.74m
EXTERIOR SIDE YARD DEPTH (M) (MINIMUM)			27.51m
REAR YARD DEPTH (M) (MINIMUM)			74.12m
HEIGHT (M) MAXIMUM			10m
VEHICULAR PARKING			
Parking is calculated for each individual Cannabis Production and Processing Facility, industrial and warehouse/distribution premises, including accessory office space, as follows:			
For the first 1000 m2, 1 parking space per 30m2 of gross floor area shall be provided.			
Any additional gross floor area shall be calculated in addition to the above provision and provided in accordance with the following rates:			
☐ For gross floor areas between 1001m2 to 5000m2 shall provide 1 parking space per 100m2 of gross floor area.			
☐ Gross floor areas greater than 5000m2 shall provide 1 parking space per 200m2 of gross floor area.			
Notwithstanding any provisions of the By-law to the contrary, accessory retail uses associated with the industrial or warehouse/distribution use shall be included within the above noted calculations provided that the retail area does not exceed the lesser of 5% or 232.2 m2 of the total gross floor			
PARKING CALCULATION			
	REQUIRED		PROVIDED
TOTAL GROSS FLOOR AREA (BUILDING A+B+C) = 33829.62 m ²	1/30 m ² for GFA 1000m ² = 33.3		595
Parking Ratio:	1/100m ² for GFA 5000m ² = 50		(INCLUDING 16
1/30 m ² for GFA 1000m ²	1/200m ² for GFA 27829.62m ² = 139.14		BARRIER-FREE PARKING)
1/100m ² for GFA 1001m ² to 5000m ²	TOTAL PARKING REQUIRED = 223		
1/200m ² for GFA greater than 5000m ²			



INSPIRE
ARCHITECTURAL GROUP

251520 Ontario Ltd
218-50 Sunny Meadows Blvd, Brampton, ON L6R 0Y7
+1 905-861-2032
harpnest@inspireag.ca

This drawing, as an instrument of service, is provided by and is the property of Inspire Architectural Group. The contractor must verify and accept responsibility for all dimensions and conditions on site and must notify Inspire Architectural Group of any variations from the supplied information. This drawing is not to be scaled. The architect is not responsible for the accuracy of survey, structural, mechanical, electrical, etc. information shown on this drawing. Refer to the appropriate consultant's drawings before proceeding with the work. Construction must conform to all applicable codes and requirements of authorities having jurisdiction. The contractor working from drawings not specifically marked 'For Construction' must assume full responsibility and bear costs for any corrections or damages resulting from his work.

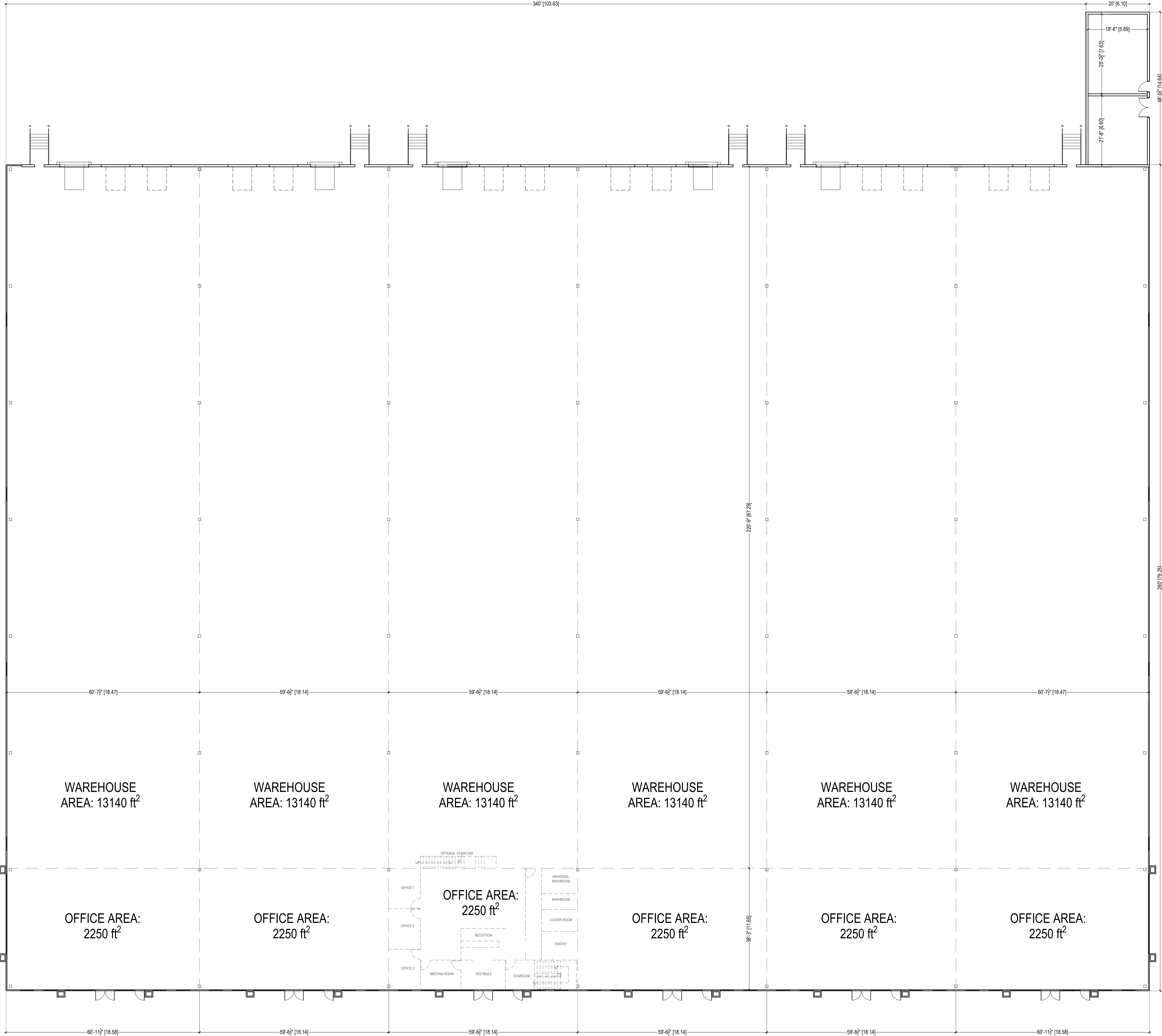
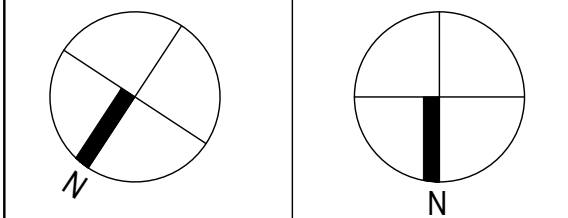
RO	2025-06-27	CLIENT'S REVIEW
#	DATE	ISSUED FOR
PROJECT NAME		
PROPOSED WAREHOUSE/STORAGE DEVELOPMENT		
AT 8995 BOSTON CHURCH RD, MILTON		
DRAWING NAME		
PROPOSED SITE PLAN		
DRAWN BY	PROJECT NO.	
RB	25.020	
CHECKED BY	PROJECT DATE	
HK	2025-06-27	
Scale	Drawing no.	
1:200	A1.0	
SHEET SIZE	24"X36"	
TRUE NORTH	PROJECT NORTH	

RO	2025-06-27	CLIENT'S REVIEW
#	DATE	ISSUED FOR

PROJECT NAME
PROPOSED WAREHOUSE/STORAGE DEVELOPMENT
AT 8995 BOSTON CHURCH RD, MILTON

DRAWING NAME
FIRST FLOOR PLAN
BUILDING A

DRAWN BY RB	PROJECT NO. 25.020
CHECKED BY HK	PROJECT DATE 2025-06-27
Scale 1/16"=1'-0"	Drawing no. A2.0
SHEET SIZE TRUE NORTH	24"X36" PROJECT NORTH



1

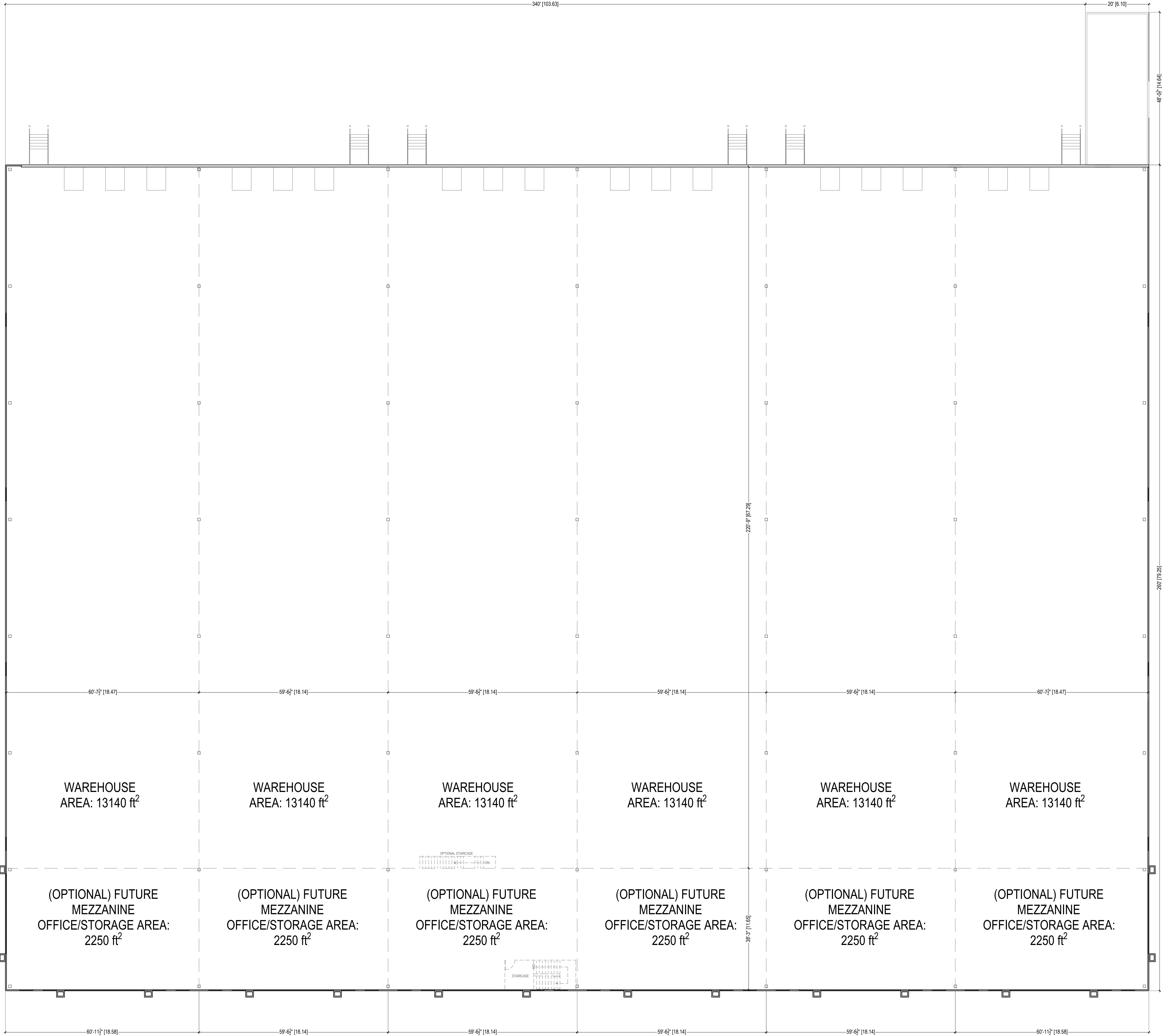
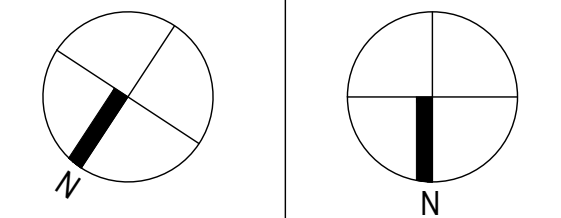
BUILDING A - FIRST FLOOR PLAN
1/16"=1'-0"

RO	2025-06-27	CLIENT'S REVIEW
#	DATE	ISSUED FOR

PROJECT NAME
PROPOSED WAREHOUSE/STORAGE DEVELOPMENT
AT 8995 BOSTON CHURCH RD, MILTON

DRAWING NAME
MEZZANINE FLOOR PLAN
BUILDING A

DRAWN BY RB	PROJECT NO. 25.020
CHECKED BY HK	PROJECT DATE 2025-06-27
Scale 1/16"=1'-0"	Drawing no. A2.1
SHEET SIZE TRUE NORTH	24"X36" PROJECT NORTH



1

BUILDING A - MEZZANINE FLOOR PLAN

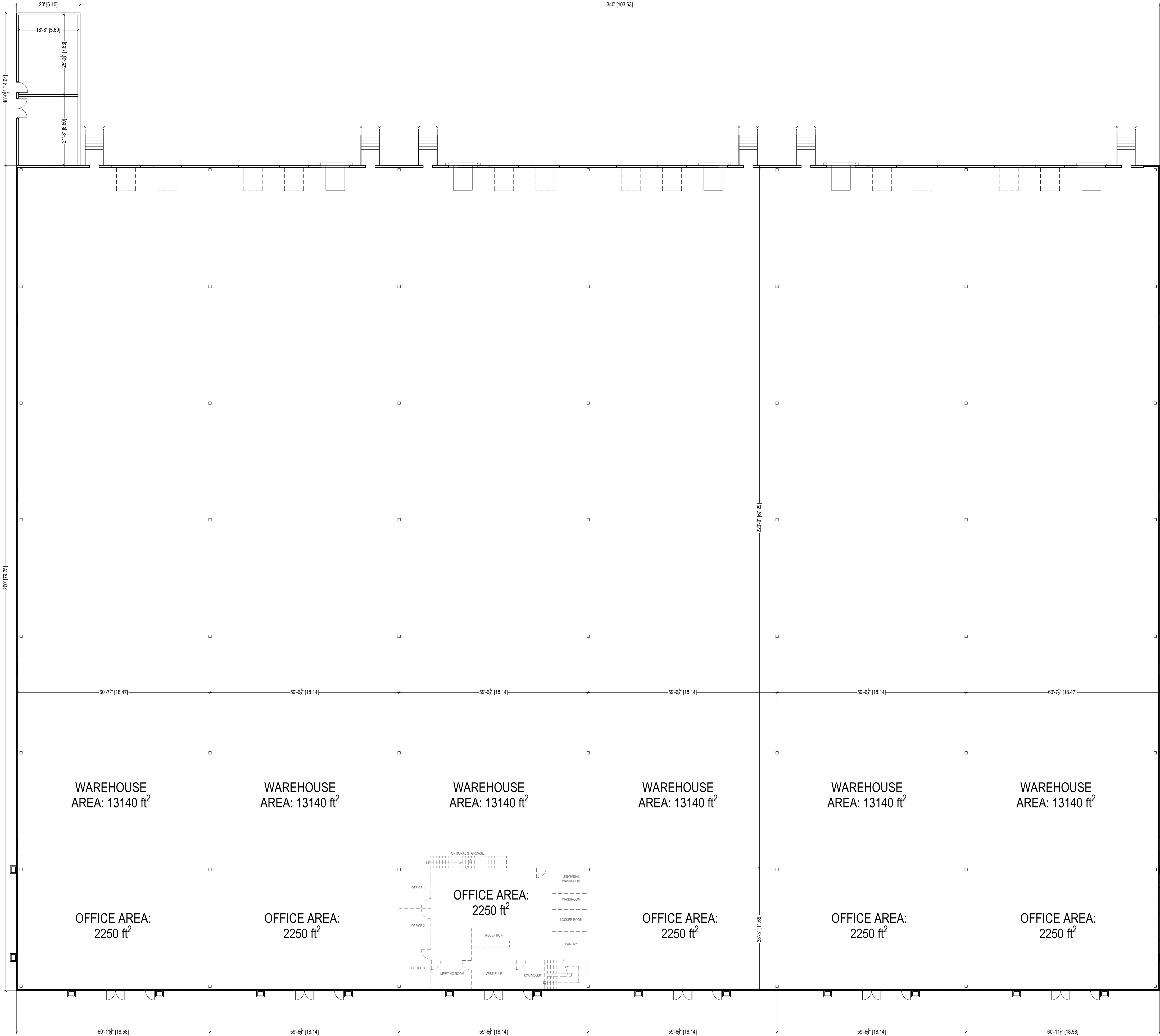
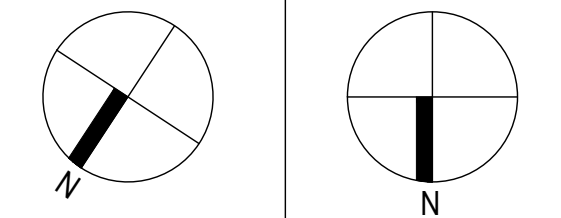
1/16"=1'-0"

RO	2025-06-27	CLIENT'S REVIEW
#	DATE	ISSUED FOR

PROJECT NAME
PROPOSED WAREHOUSE/STORAGE DEVELOPMENT
AT 8995 BOSTON CHURCH RD, MILTON

DRAWING NAME
FIRST FLOOR PLAN
BUILDING B

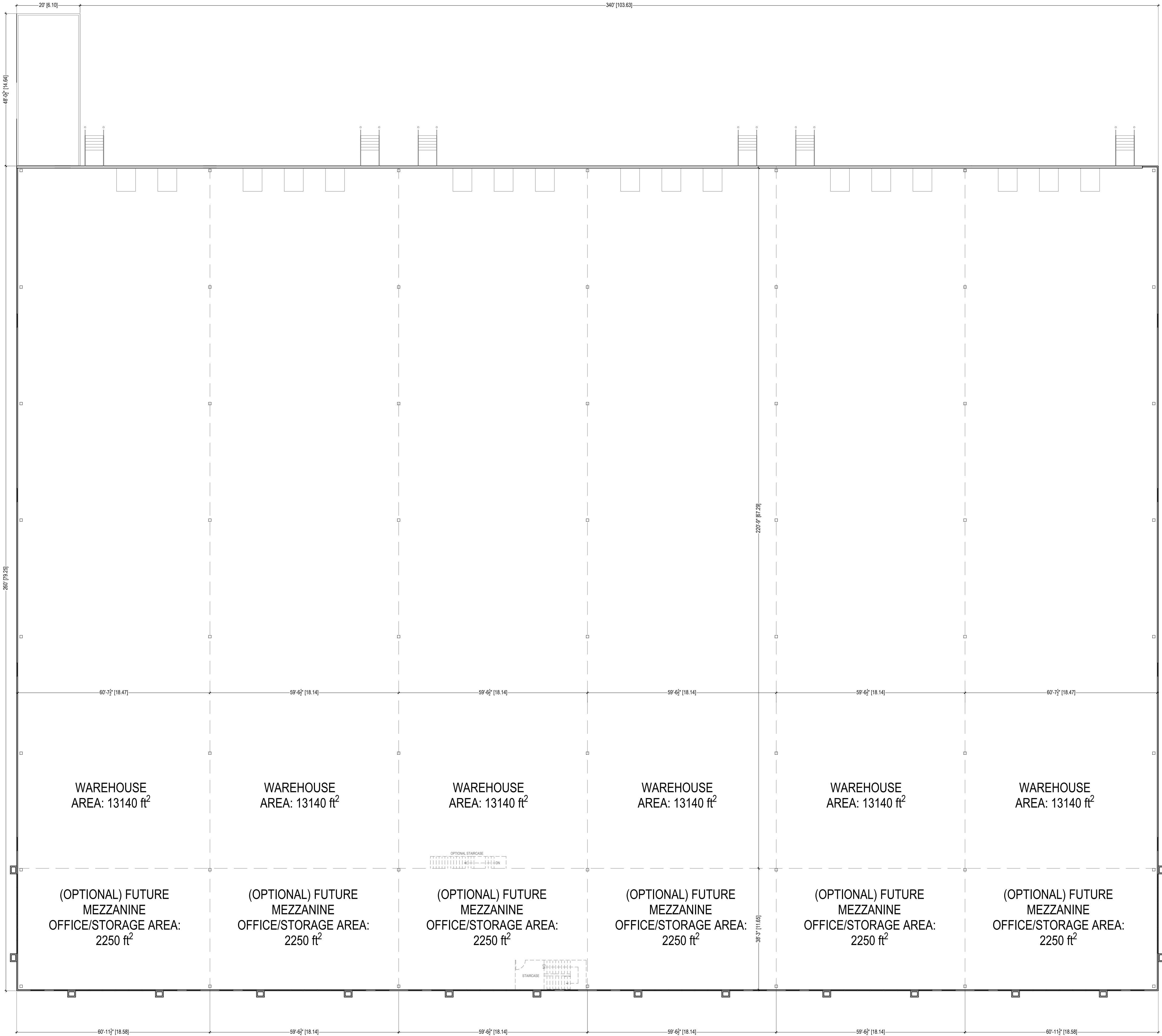
DRAWN BY RB	PROJECT NO. 25.020
CHECKED BY HK	PROJECT DATE 2025-06-27
Scale 1/16"=1'-0"	Drawing no. A2.2
SHEET SIZE TRUE NORTH	24"X36" PROJECT NORTH



1

BUILDING B - FIRST FLOOR PLAN
1/16"=1'-0"

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1

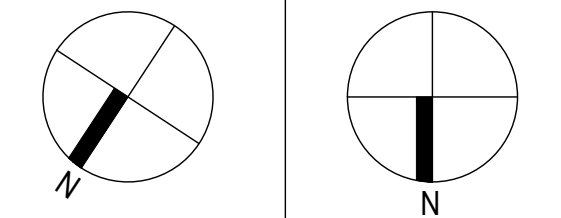
BUILDING B - MEZZANINE FLOOR PLAN
1/16"=1'-0"

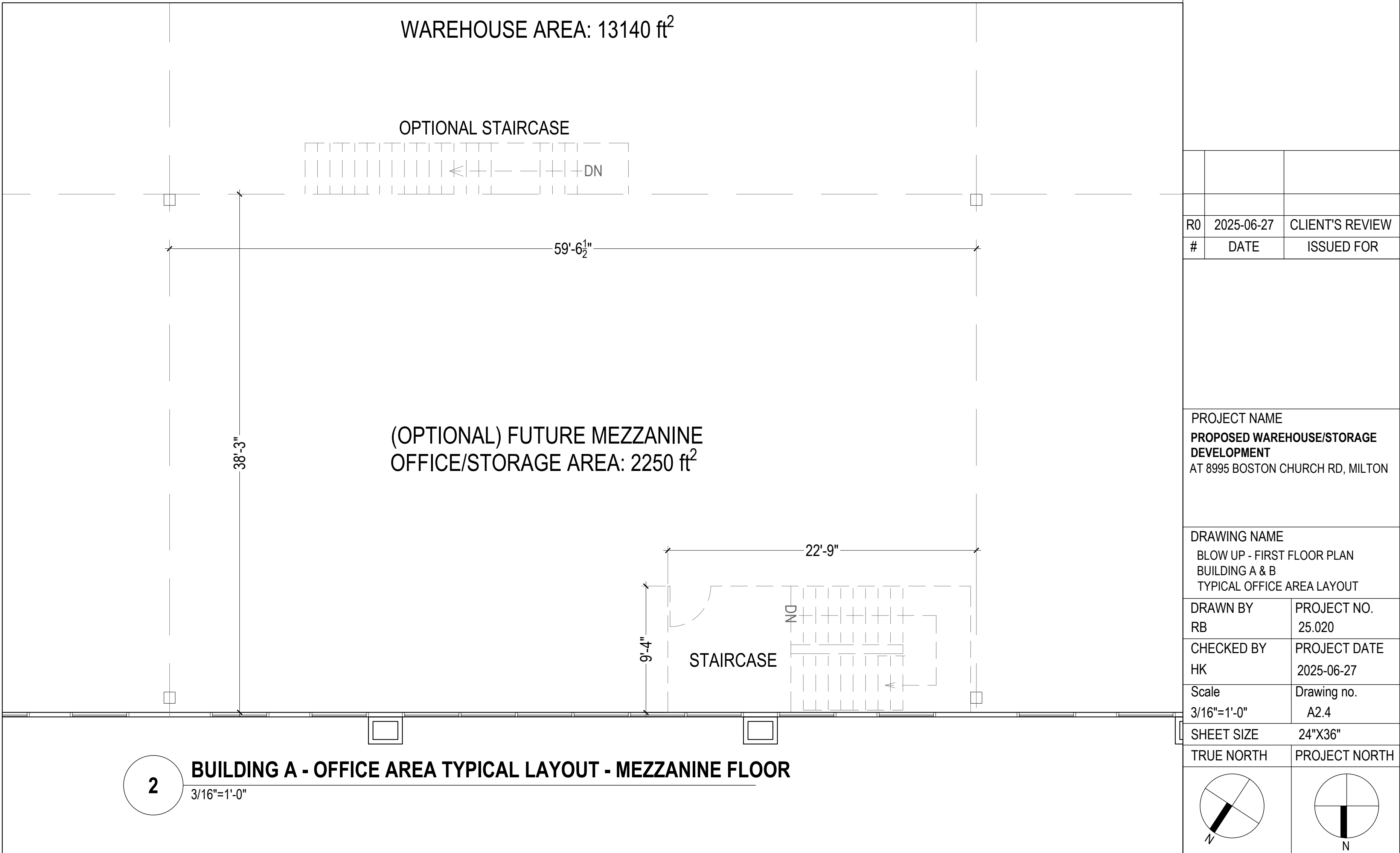
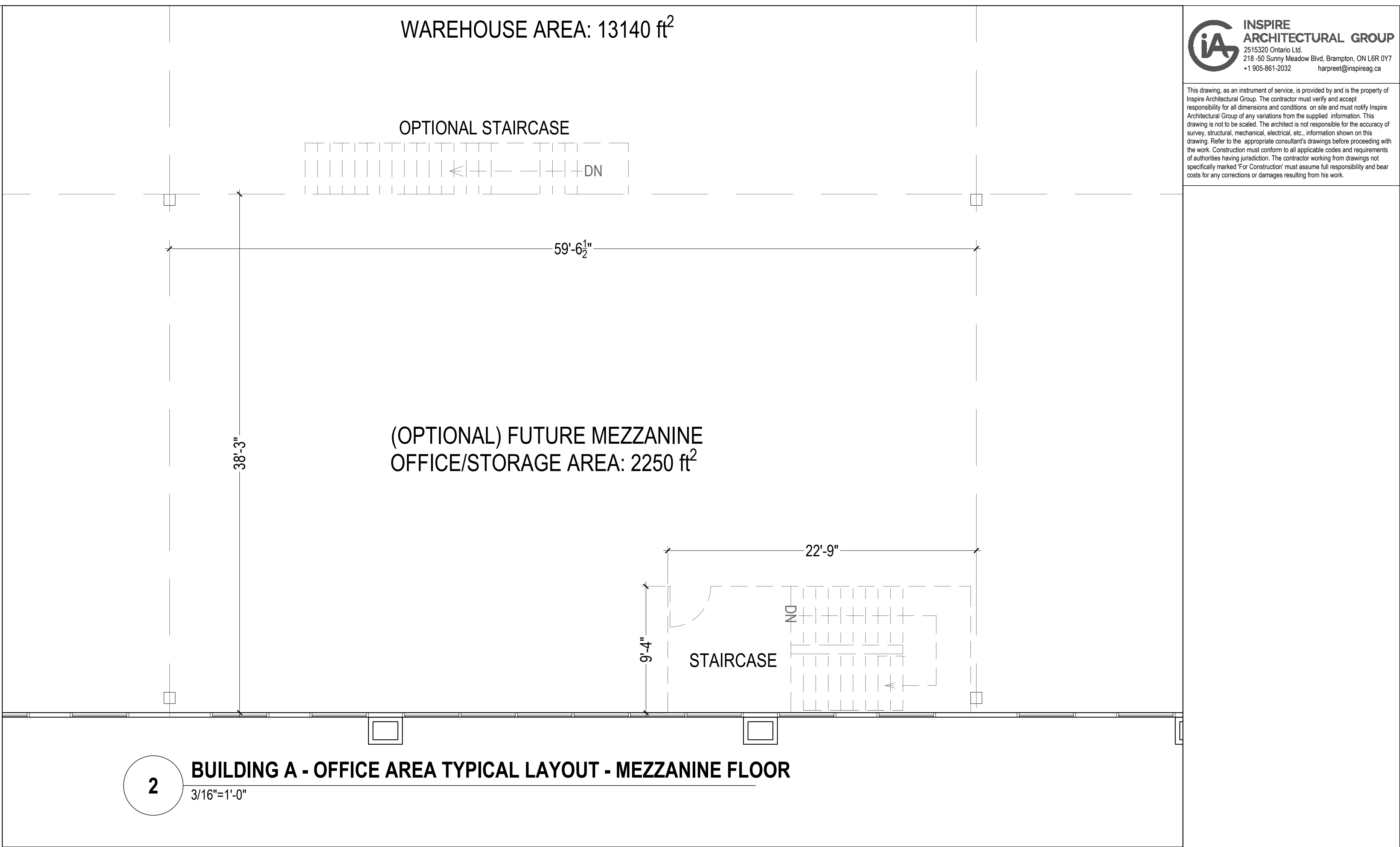
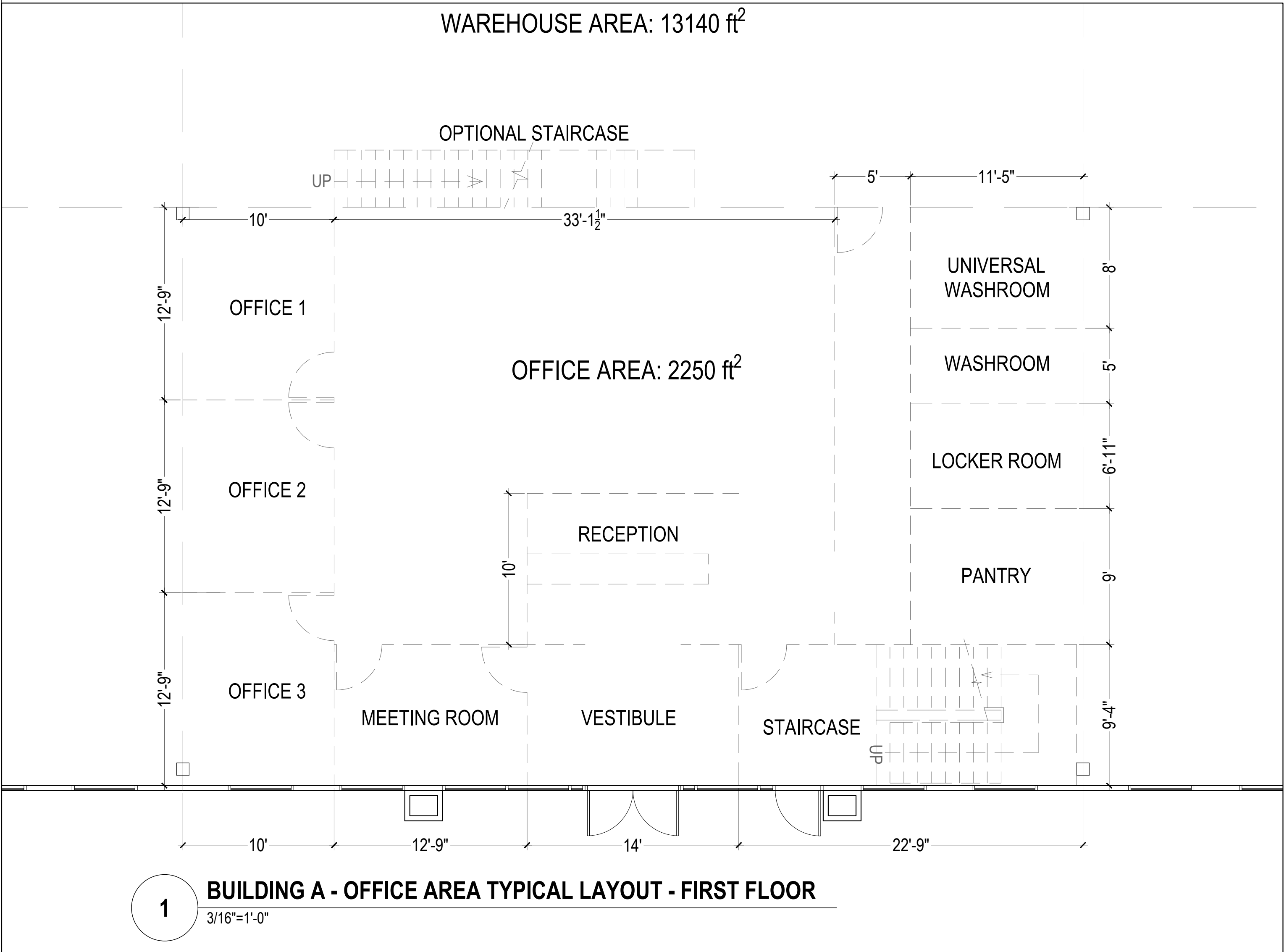
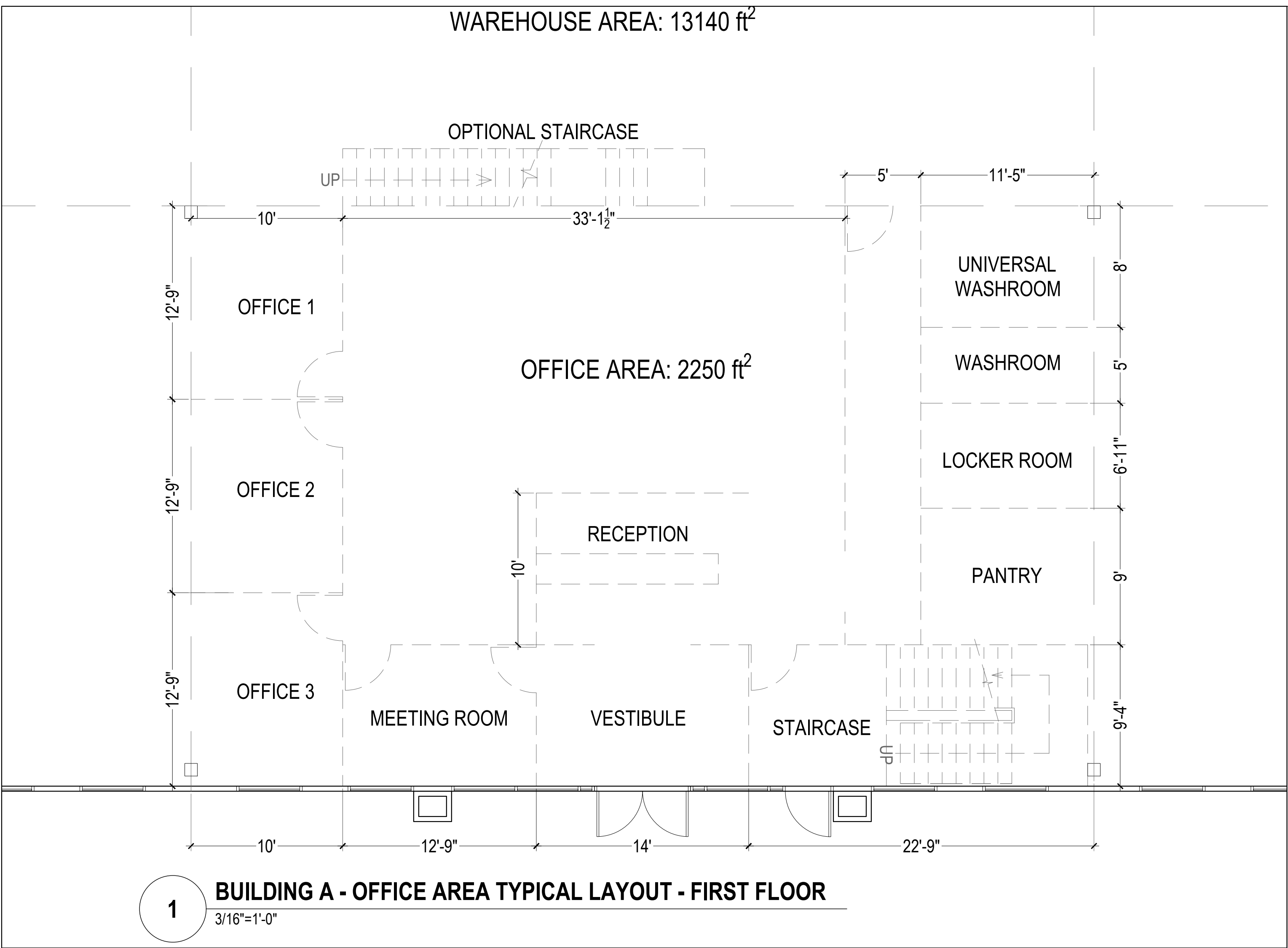
RO	2025-06-27	CLIENT'S REVIEW
#	DATE	ISSUED FOR

PROJECT NAME
PROPOSED WAREHOUSE/STORAGE DEVELOPMENT
AT 8995 BOSTON CHURCH RD, MILTON

DRAWING NAME
MEZZANINE FLOOR PLAN
BUILDING B

DRAWN BY RB	PROJECT NO. 25.020
CHECKED BY HK	PROJECT DATE 2025-06-27
Scale 1/16"= 1'-0"	Drawing no. A2.3
SHEET SIZE TRUE NORTH	24"X36" PROJECT NORTH



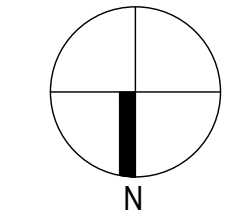
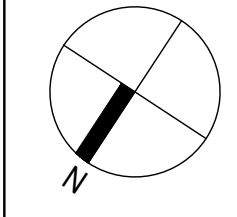




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RO	2025-06-27	CLIENT'S REVIEW
#	DATE	ISSUED FOR
PROJECT NAME PROPOSED WAREHOUSE/STORAGE DEVELOPMENT AT 8995 BOSTON CHURCH RD, MILTON		
DRAWING NAME BLOW UP - FIRST FLOOR PLAN BUILDING A & B TYPICAL OFFICE AREA LAYOUT		
DRAWN BY RB	PROJECT NO. 25.020	
CHECKED BY HK	PROJECT DATE 2025-06-27	
Scale 3/16"=1'-0"	Drawing no. A2.4	
SHEET SIZE TRUE NORTH	24"X36" PROJECT NORTH	

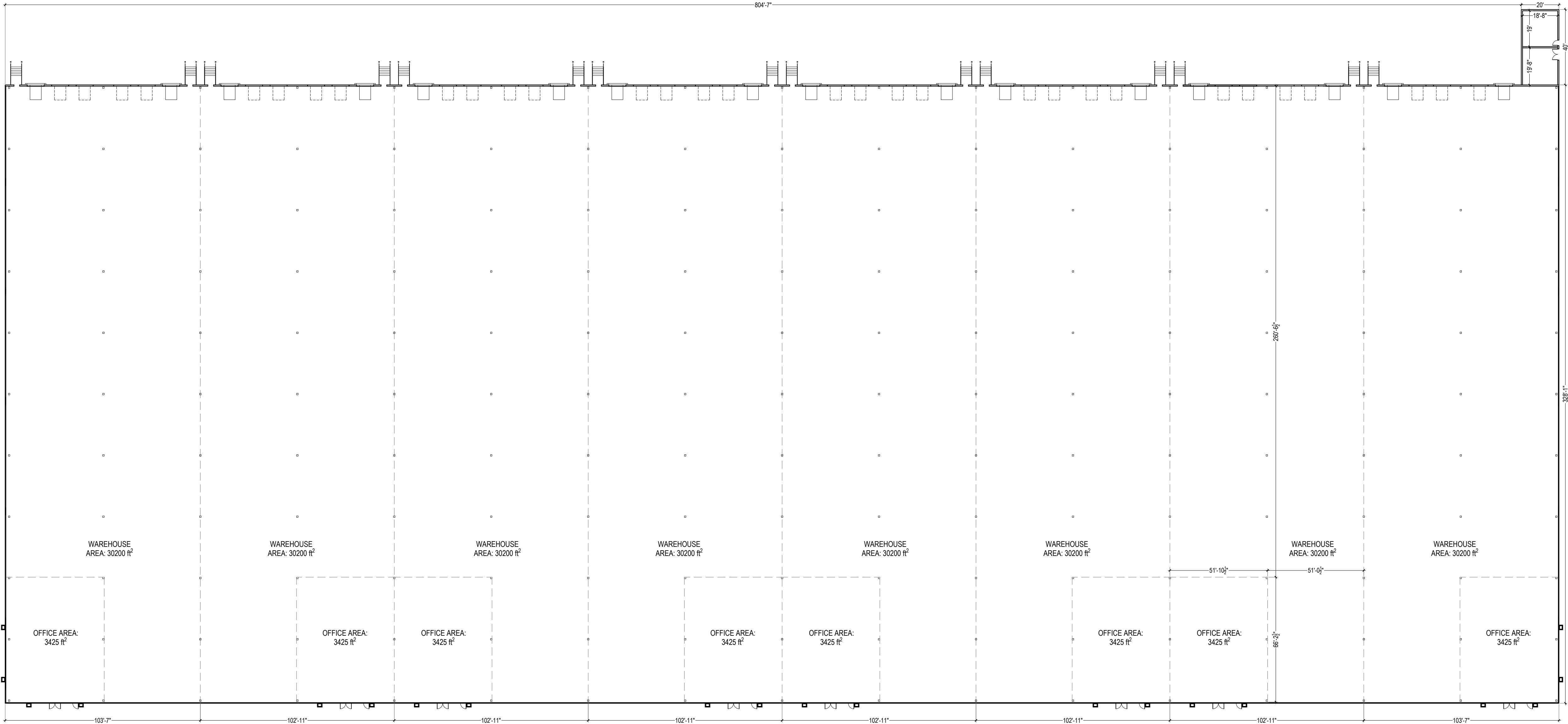
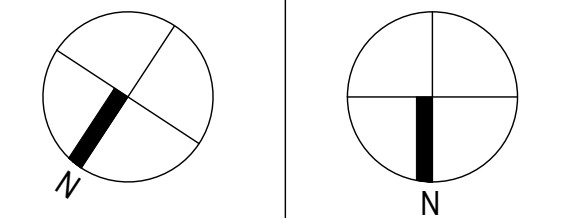


R0	2025-06-27	CLIENT'S REVIEW
#	DATE	ISSUED FOR

PROJECT NAME
PROPOSED WAREHOUSE/STORAGE DEVELOPMENT
AT 8995 BOSTON CHURCH RD, MILTON

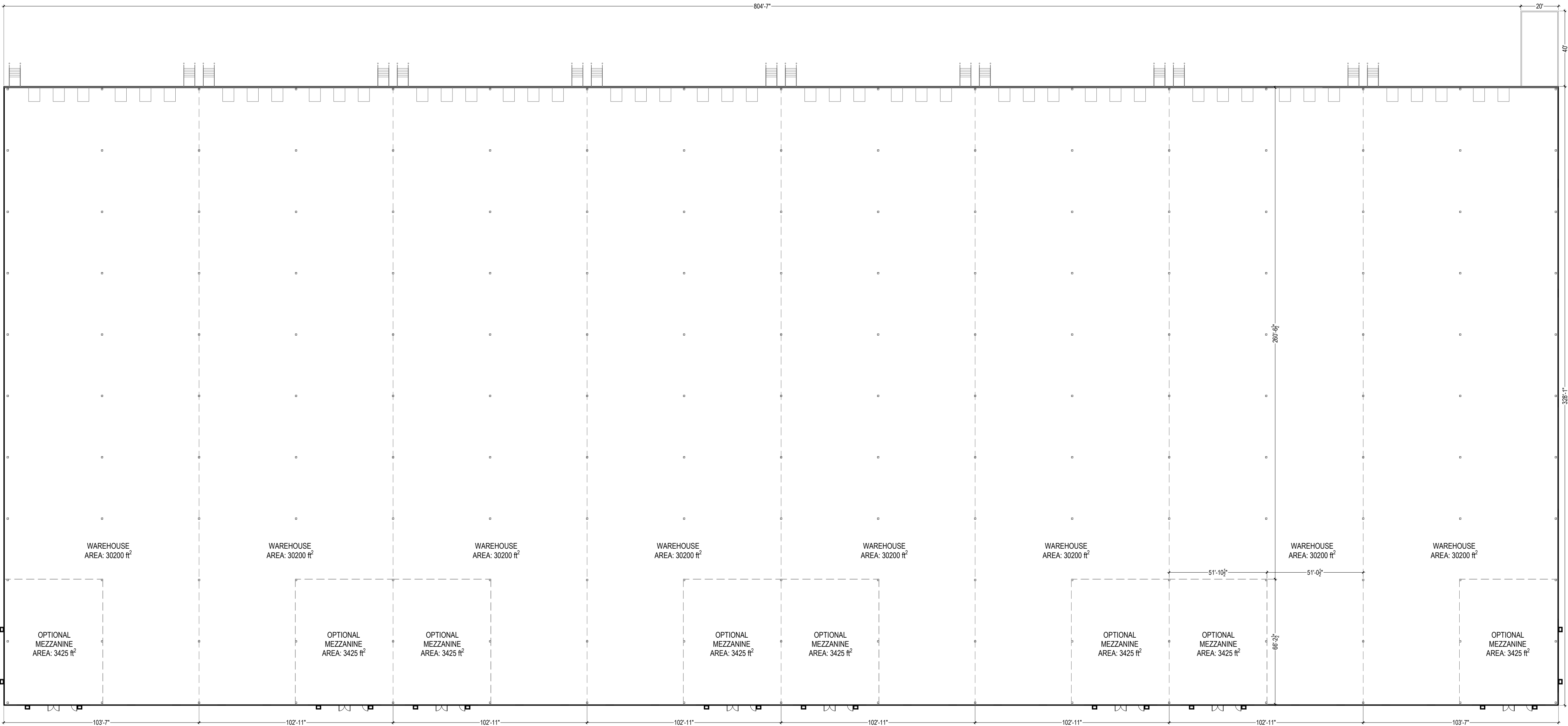
DRAWING NAME
FIRST FLOOR PLAN
BUILDING C

DRAWN BY RB	PROJECT NO. 25.020
CHECKED BY HK	PROJECT DATE 2025-06-27
Scale 1/32"= 1'-0"	Drawing no. A2.5
SHEET SIZE TRUE NORTH	24"X36" PROJECT NORTH



1

BUILDING C - FIRST FLOOR PLAN
1/32"=1'-0"



1

BUILDING C - MEZZANINE FLOOR PLAN

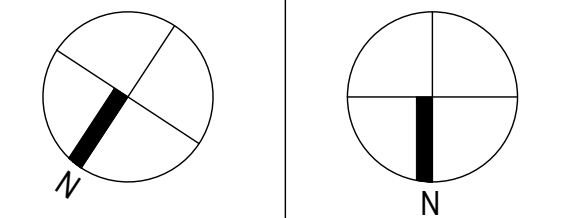
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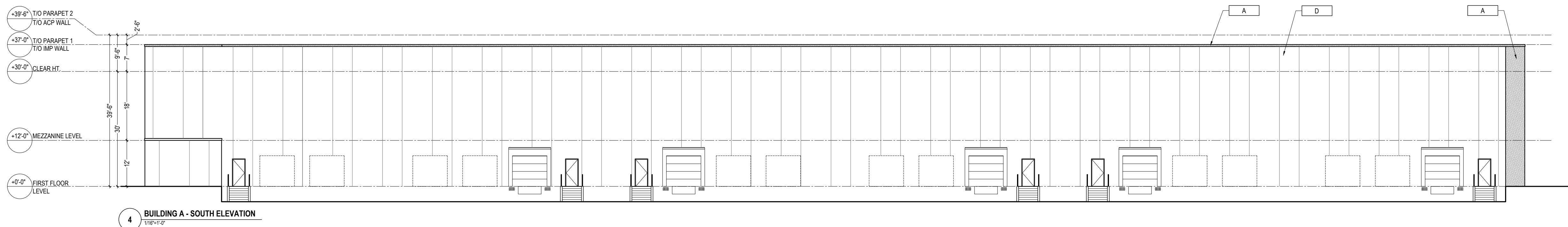
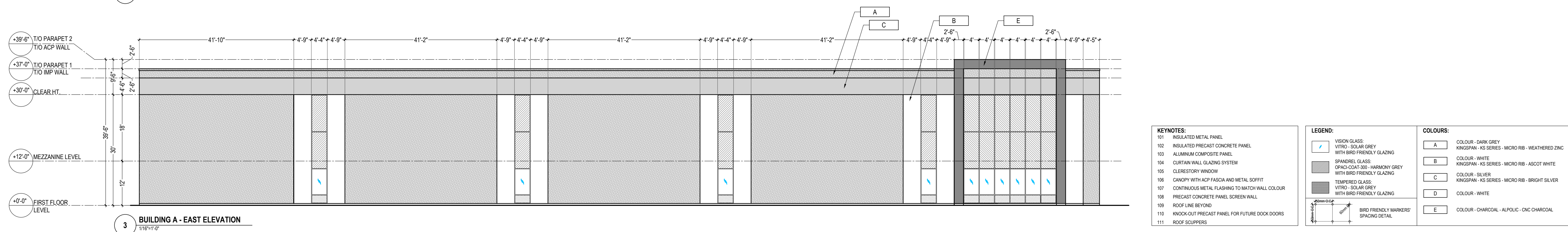
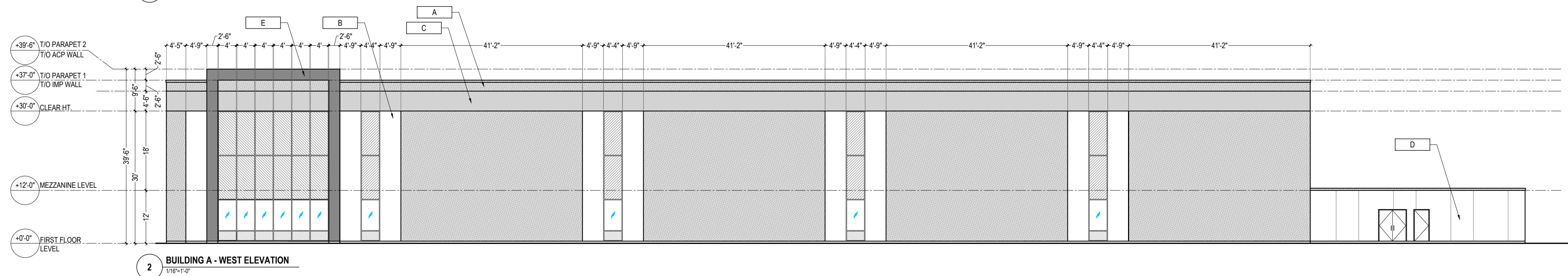
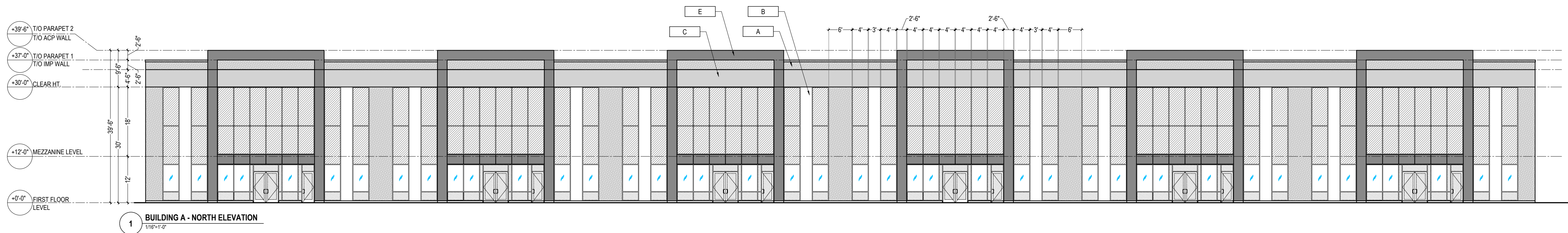
RO	2025-06-27	CLIENT'S REVIEW
#	DATE	ISSUED FOR

PROJECT NAME
PROPOSED WAREHOUSE/STORAGE
DEVELOPMENT
AT 8995 BOSTON CHURCH RD, MILTON

DRAWING NAME
MEZZANINE FLOOR PLAN
BUILDING C

DRAWN BY RB	PROJECT NO. 25.020
CHECKED BY HK	PROJECT DATE 2025-06-27
Scale 1/32"=1'-0"	Drawing no. A2.6
SHEET SIZE	24"X36"
TRUE NORTH	PROJECT NORTH



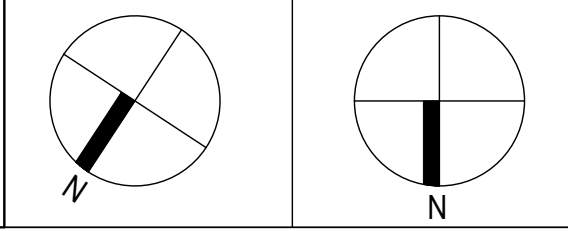


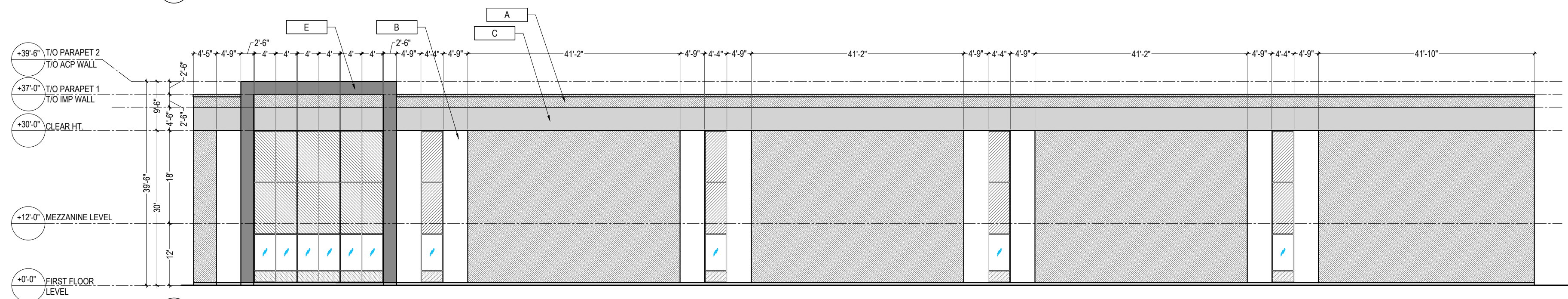
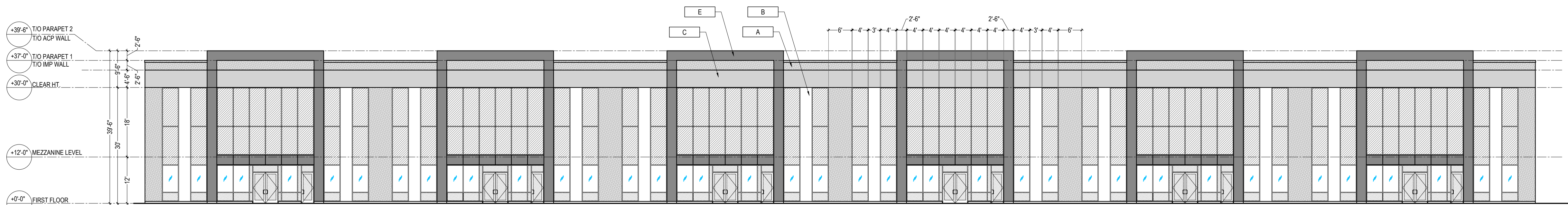
R0	2025-06-27	CLIENT'S REVIEW
#	DATE	ISSUED FOR

PROJECT NAME
PROPOSED WAREHOUSE/STORAGE DEVELOPMENT
AT 8995 BOSTON CHURCH RD, MILTON

DRAWING NAME
ELEVATIONS
BUILDING A

DRAWN BY RB	PROJECT NO. 25.020
CHECKED BY HK	PROJECT DATE 2025-06-27
Scale 1/16"=1'-0"	Drawing no. A3.0
SHEET SIZE TRUE NORTH	24"X36" PROJECT NORTH

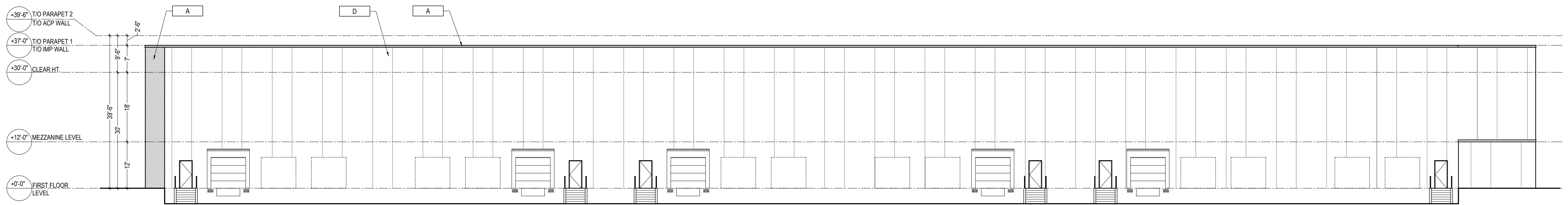
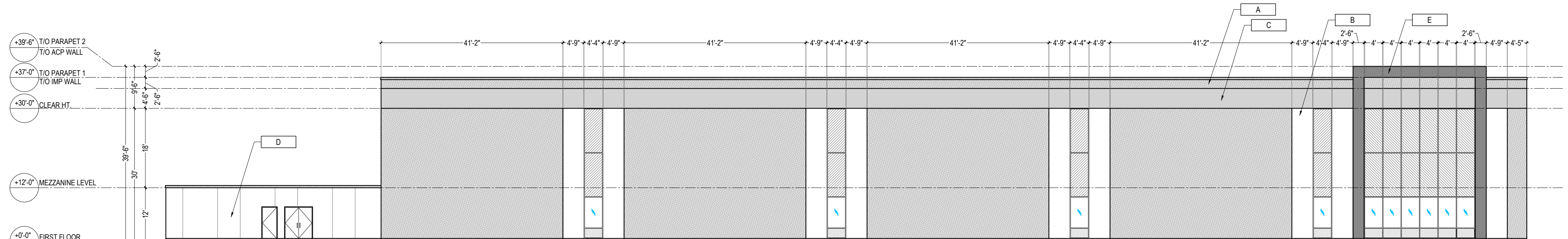




- KEYNOTES:**
- 101 INSULATED METAL PANEL
 - 102 INSULATED PRECAST CONCRETE PANEL
 - 103 ALUMINUM COMPOSITE PANEL
 - 104 CURTAIN WALL GLAZING SYSTEM
 - 105 CLERESTORY WINDOW
 - 106 CANOPY WITH ACP FASCIA AND METAL SOFFIT
 - 107 CONTINUOUS METAL FLASHING TO MATCH WALL COLOUR
 - 108 PRECAST CONCRETE PANEL SCREEN WALL
 - 109 ROOF LINE BEYOND
 - 110 KNOCK-OUT PRECAST PANEL FOR FUTURE DOCK DOORS
 - 111 ROOF SLOPPERS

- LEGEND:**
- VISION GLASS: VTRO - SOLAR GREY WITH BIRD-FRIENDLY GLAZING
 - SPANDREL GLASS: SPANDREL GLASS - HARBORWAY GREY WITH BIRD-FRIENDLY GLAZING
 - TEMPERED GLASS: VTRO - SOLAR GREY WITH BIRD-FRIENDLY GLAZING
 - BIRD-FRIENDLY MARKERS/SPACING DETAIL

- COLOURS:**
- A COLOUR - DARK GREY KINGSPAN - KS SERIES - MICRO RIB - WEATHERED ZINC
 - B COLOUR - WHITE KINGSPAN - KS SERIES - MICRO RIB - ASCOT WHITE
 - C COLOUR - SILVER KINGSPAN - KS SERIES - MICRO RIB - BRIGHT SILVER
 - D COLOUR - WHITE
 - E COLOUR - CHARCOAL - ALPOLIC - CNC CHARCOAL

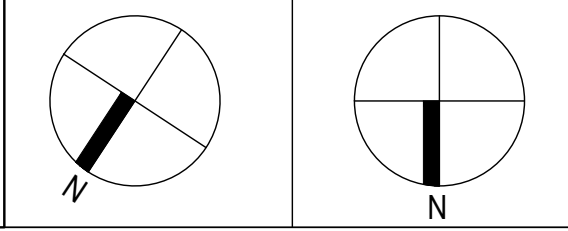


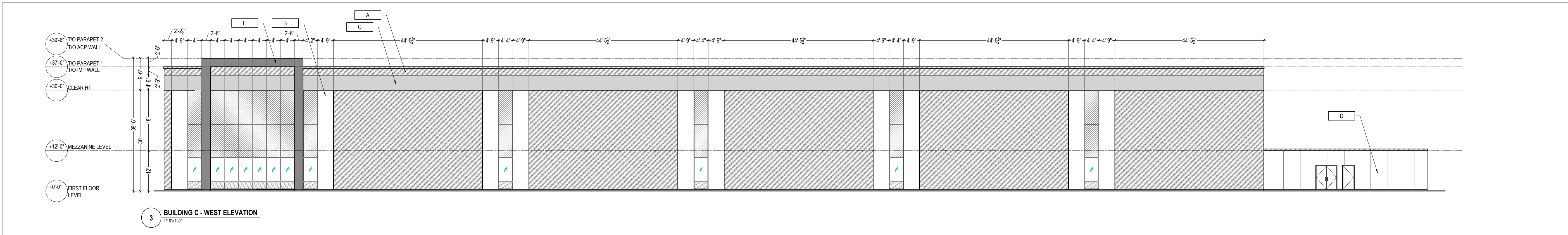
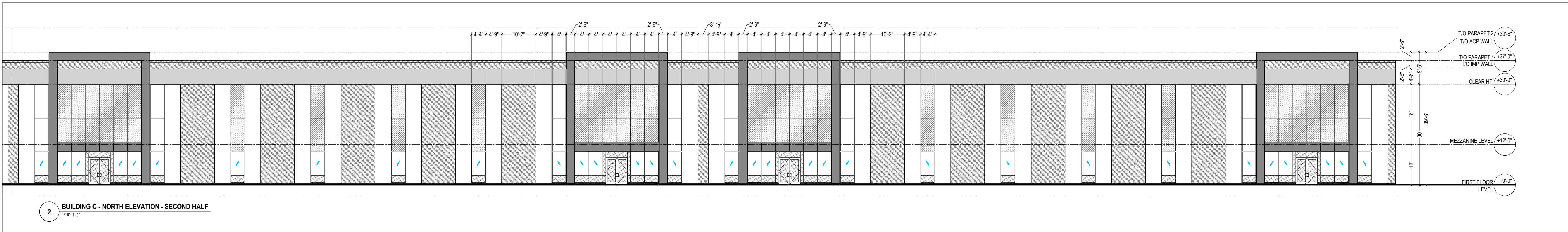
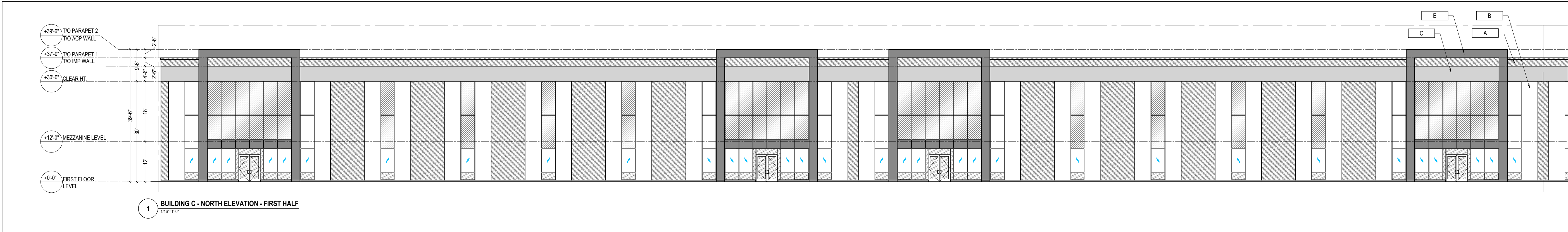
R0	2025-06-27	CLIENT'S REVIEW
#	DATE	ISSUED FOR

PROJECT NAME
PROPOSED WAREHOUSE/STORAGE DEVELOPMENT
AT 8995 BOSTON CHURCH RD, MILTON

DRAWING NAME
ELEVATIONS
BUILDING B

DRAWN BY RB	PROJECT NO. 25.020
CHECKED BY HK	PROJECT DATE 2025-06-27
Scale 1/16"=1'-0"	Drawing no. A3.1
SHEET SIZE TRUE NORTH	24"X36" PROJECT NORTH





KEYNOTES:

- 101 INSULATED METAL PANEL
- 102 INSULATED PRECAST CONCRETE PANEL
- 103 ALUMINUM COMPOSITE PANEL
- 104 CURTAIN WALL GLAZING SYSTEM
- 105 CLERESTORY WINDOW
- 106 CANOPY WITH ACP FASCIA AND METAL SOFFIT
- 107 CONTINUOUS METAL FLASHING TO MATCH WALL COLOUR
- 108 PRECAST CONCRETE PANEL SCREEN WALL
- 109 ROOF LINE BEYOND
- 110 KNOCK-OUT PRECAST PANEL FOR FUTURE DOCK DOORS
- 111 ROOF SCUPPERS

LEGEND:

- VISION GLASS:
VITRO - SOLAR GREY
WITH BIRD FRIENDLY GLAZING
- SPANDREL GLASS:
OPACH-COAT-300 - HARMONY GREY
WITH BIRD FRIENDLY GLAZING
- TEMPERED GLASS:
VITRO - SOLAR GREY
WITH BIRD FRIENDLY GLAZING
- BIRD FRIENDLY MARKERS'
SPACING DETAIL

COLOURS:

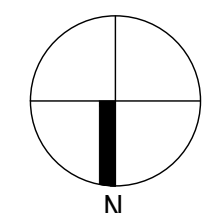
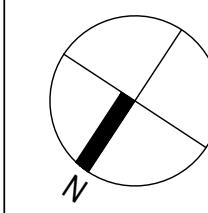
- A COLOUR - DARK GREY
KINGSPAN - KS SERIES - MICRO RIB - WEATHERED ZINC
- B COLOUR - WHITE
KINGSPAN - KS SERIES - MICRO RIB - ASCOT WHITE
- C COLOUR - SILVER
KINGSPAN - KS SERIES - MICRO RIB - BRIGHT SILVER
- D COLOUR - WHITE
- E COLOUR - CHARCOAL - ALPOLIC - CNC CHARCOAL

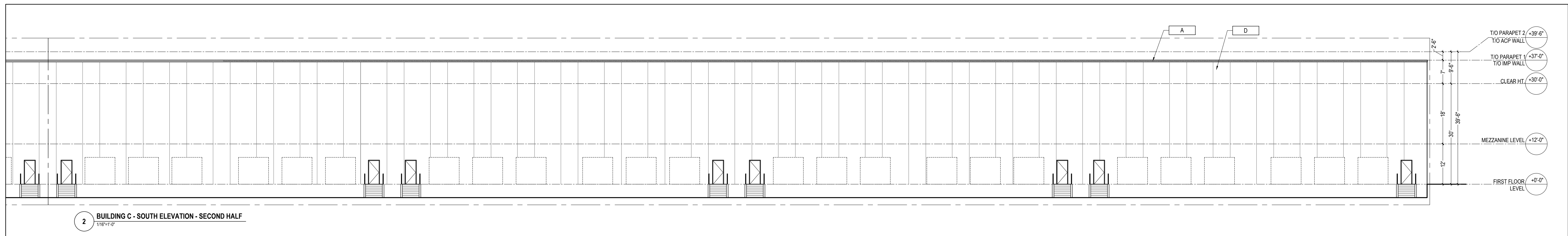
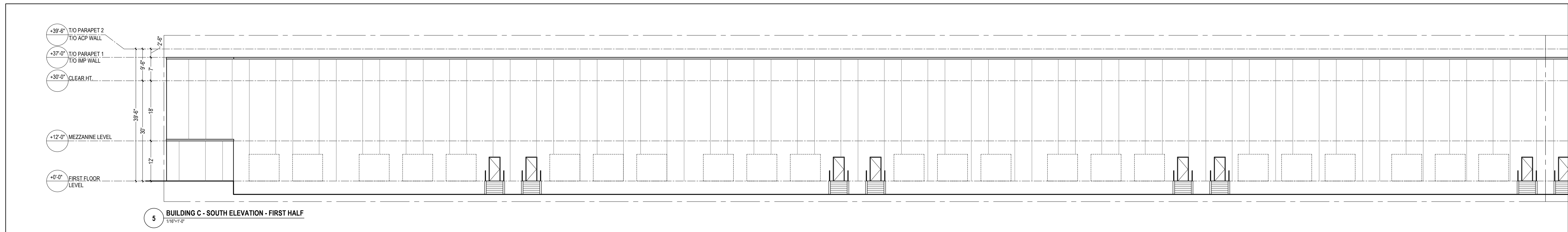
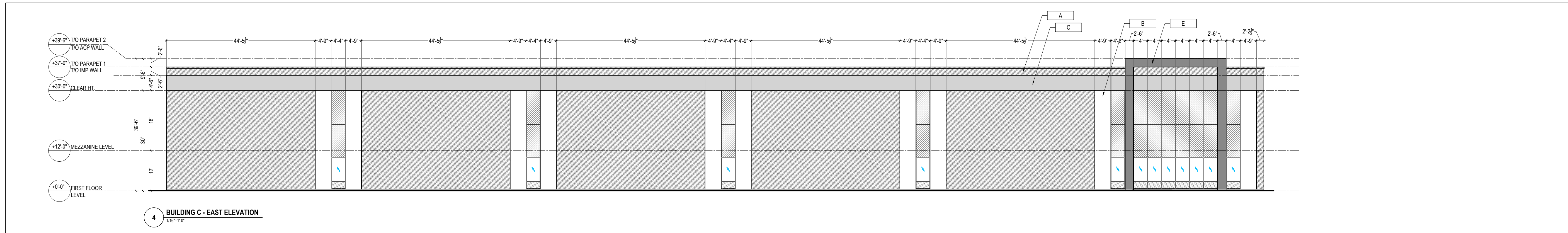
RO	2025-06-27	CLIENT'S REVIEW
#	DATE	ISSUED FOR

PROJECT NAME
**PROPOSED WAREHOUSE/STORAGE
DEVELOPMENT**
AT 8995 BOSTON CHURCH RD, MILTON

DRAWING NAME
NORTH & WEST ELEVATIONS
BUILDING C

DRAWN BY RB	PROJECT NO. 25.020
CHECKED BY HK	PROJECT DATE 2025-06-27
Scale 1/16"=1'-0"	Drawing no. A3.2
SHEET SIZE TRUE NORTH	24"X36" PROJECT NORTH





KEYNOTES:		LEGEND:		COLOURS:			
101	INSULATED METAL PANEL		VISION GLASS: VITRO - SOLAR GREY WITH BIRD FRIENDLY GLAZING		COLOUR - DARK GREY KINGSPAN - KS SERIES - MICRO RIB - WEATHERED ZINC		
102	INSULATED PRECAST CONCRETE PANEL				SPANDREL GLASS: OPAC-COAT-300 - HARMONY GREY WITH BIRD FRIENDLY GLAZING		COLOUR - WHITE KINGSPAN - KS SERIES - MICRO RIB - ASCOT WHITE
103	ALUMINUM COMPOSITE PANEL			TEMPERED GLASS: VITRO - SOLAR GREY WITH BIRD FRIENDLY GLAZING			COLOUR - SILVER KINGSPAN - KS SERIES - MICRO RIB - BRIGHT SILVER
104	CURTAIN WALL GLAZING SYSTEM			BIRD FRIENDLY MARKERS' SPACING DETAIL			COLOUR - WHITE
105	CLERESTORY WINDOW				COLOUR - CHARCOAL - ALPOLIC - CNC CHARCOAL		
106	CANOPY WITH APC FASCIA AND METAL SOFFIT						
107	CONTINUOUS METAL FLASHING TO MATCH WALL COLOUR						
108	PRECAST CONCRETE PANEL SCREEN WALL						
109	ROOF LINE BEYOND						
110	KNOCK-OUT PRECAST PANEL FOR FUTURE DOCK DOORS						
111	ROOF SCUPPERS						

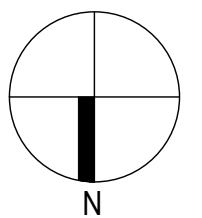
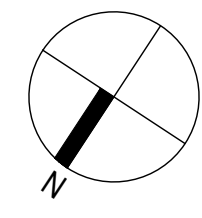
RO	2025-06-27	CLIENT'S REVIEW
#	DATE	ISSUED FOR

PROJECT NAME
PROPOSED WAREHOUSE/STORAGE
DEVELOPMENT
AT 8995 BOSTON CHURCH RD, MILTON

DRAWING NAME

EAST & SOUTH ELEVATIONS
BUILDING C

DRAWN BY RB	PROJECT NO. 25.020
CHECKED BY HK	PROJECT DATE 2025-06-27
Scale 1/16"=1'-0"	Drawing no. A3.2
SHEET SIZE	24"x36"
TRUE NORTH	PROJECT NORTH





1 AREIAL VIEW



2 VIEW 1



3 VIEW 2



4 VIEW 3



4 VIEW 5

R0	2025-06-27	CLIENT'S REVIEW
#	DATE	ISSUED FOR
PROJECT NAME PROPOSED WAREHOUSE/STORAGE DEVELOPMENT AT 8995 BOSTON CHURCH RD, MILTON		
DRAWING NAME 3D ELEVATIONS		
DRAWN BY RB	PROJECT NO. 25.020	
CHECKED BY HK	PROJECT DATE 2025-06-27	
Scale NTS	Drawing no. A3.2	
SHEET SIZE	24"X36"	
TRUE NORTH	PROJECT NORTH	

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