RYZUK GEOTECHNICAL
Engineering & Materials Testing
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May 12, 2023
File No: 11602-1

Gill Developments
4774 Westwinds Drive NE
Calgary, AB
T3J 0L7

Attn: Joey Gill (By E-mail: joey@gilldevelopments.ca)

Re: Proposed Apartment Block
3447, 3449, 3451, 3461 Cook Street – Saanich, BC

As requested, and in accordance with our accepted proposal of April 19, 2023, we have completed a geotechnical investigation of the subsurface conditions at the referenced site. This report summarizes the results of our investigation and our associated recommendations regarding the proposed development.

SITE AND PROPOSED DEVELOPMENT

The subject site is comprised of three single family residential lots (3447, 3449, and 3451 Cook Street) and a commercial lot (3461 Cook Street). The site is bounded by Cobb Lane to the east and south, Cook Street to the west, and a commercial lo to the north. The single family lots currently accommodate existing residences in roughly the center of the lot, with associated yard areas and driveways. The commercial lot currently accommodates an existing at grade medical building on the southwest portion of the site with a grass surfaced yard area and asphalt surfaced parking on the northeast and northwest portions of the lot, respectively.

We have not yet been provided with any design drawings of the proposed development. Based on our discussions, we understand that the proposed development will consist of an amalgamation of the four lots, removal of the existing buildings and associated infrastructure, and construction of a new residential building. The proposed building is anticipated to consist of six storeys of wood framed construction atop one to two levels of underground parking of reinforced concrete construction.

INVESTIGATION PROCEDURE

Our investigation consisted of both an office-based review of available relevant information, as well as a subsurface investigation. Our review involved perusal of available imagery, available aerial photography, geology mapping, as well as review of our historical file database of previous projects completed in the vicinity. Our field investigation consisted of a one-day drilling investigation with a solid stem tracked auger owned and operated by Drillwell Enterprises Ltd. of Duncan, BC. Four test holes (TH23-01 to TH23-04) were drilled by solid stem auger and advanced to refusal upon inferred dense glacial till or bedrock. Soils were logged in the field by Ryzuk personnel as drilling progressed.

Our investigation also included a micotremor survey using a geophysical device (Tromino), which measures and records ambient vibrations at the ground surface. This can be assessed to infer a predominant soil resonance (natural) frequency within the general area of the acquisition point. Local Victoria-based studies have further correlated the natural frequency with the thickness of common Victoria soils and depth to bedrock, as well as the seismic shear wave velocity of the soils. The Tromino data is interpreted based on the observed soils which were generally consistent with the resonance frequency for the Greater Victoria Area, however, the frequencies are averaged over the observed depth to hard stratum. As such, calculated bedrock depths are approximate and should only be considered accurate to +/- 3 m. A total of eight readings were taken across the site (TR23-01 to TR23-08).

Test hole and Tromino locations, refusal and estimated depth to hard stratums, and detailed summaries of soil conditions are provided on the attached Test Hole Location Plan and Test Hole Logs TH23-01 to TH23-04. Test hole collar elevations have been estimated based on information from the available CRD Regional Online Map.

SURFACE AND SUBSURFACE CONDITIONS

Based on our desktop study of the site, the native soils are expected to consist of the typical Victoria Clay sequence atop dense glacial till over bedrock. The Victoria Clay sequence consists of a glaciomarine deposit of silty clay, which is typically brown, stiff to very stiff and over-consolidated in the upper 4 to 6 m due to dessication. Where the clay is thicker than about 5 to 6 m, such typically transitions to grey, stiff to firm and near normally consolidated. Our review of projects in the area indicates that the thickness of the grey silty clay varies greatly in this area and is commonly greater than 8 m and locally exceeds 15 m. The transition from brown to grey clay is typically interpreted to be at the historical ground water table. The bedrock in this region is expected to consist of Colquitz gneiss, common throughout much of Greater Victoria, and known to have a highly erratic surface profile that may vary in depth considerably over short distances.

The results of our investigation conformed closely with the anticipated soil conditions. Soils within the northeastern portion of the site (TH22-01) consisted of 1.5 m of fill associated with the current land use overlying a 3.4 m thick layer of stiff to hard brown silty clay overlying 3.9 m of firm grey silty clay atop a 4.4 m thick layer of dense silty sand with some gravel (glacial till). Similar soil conditions were observed in TH23-02; however, the firm grey clay was not observed to be present, instead transitioning to glacial till at a depth of 5.7 m. The soil conditions in TH23-03 and TH23-04 consisted of up to 0.9 m of fill overlying up to 3.0 m of hard brown clay with very stiff seams, atop a layer of glacial till up to 0.8 m thick.

Results of the Tromino investigation suggest that the depth to hard stratum (glacial till/bedrock) is consistent with that observed in nearby test holes, suggesting that till/bedrock is shallower within the southern portion of the site and possibly sloping down to the northeast. We note that the bedrock surface is very erratic throughout Greater Victoria and the Tromino results are considered estimates with a +/- 3 m error margin due to ambient noise such as wind, proximity to buildings, busy roads, etc.

Long term groundwater observations were not undertaken as part of our work at the site. The relatively impermeable nature of the clays generally precludes free water conditions, however seepage may be encountered within surficial fills overlying the relatively impermeable native clays, and/or from any sandy layers within the clay. Seepage is also commonly encountered within the till, atop bedrock. We also note that during periods of excessive rain, perched water may accumulate at the surface of the native clay soils.

GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

On the basis of our investigation, we consider the proposed development to be feasible from a geotechnical perspective. The subsurface conditions were observed to be variable with auger refusal on inferred glacial till/bedrock at 2.6 m and 13.2 m in the southern and northern extents of the site, respectively. The near normally consolidated firm grey silty clay observed in TH23-01 can be prone to consolidation in response to increased building loads, fill placement, and/or dewatering and careful consideration with regard to settlement mitigation will be required. Excavation considerations will depend on the finalized building design and desired levels of underground parking and shoring will be required if near lot line construction is utilized.

Excavation Considerations

Excavations will generally consist of removal of existing fills and native mineral soils to achieve design grades. We expect the depth of excavations will depend on the final building design and may range between 3.5 to 6.5 m for 1 and 2 levels of underground parking, respectively. If near lot line construction is desired, we expect that excavation shoring will be required; however, for planning purposes, temporary open excavation cutslopes will be stable at the following configurations:

- 1H:1V (Horizontal: Vertical) in topsoil/fill;
- 0.5H:1V in stiff to very stiff brown silty clay;
- 1H:1V in firm grey clay;
- 0.75H:1V in glacial till, and;
- Near vertical for bedrock.

The excavation cutslope configurations above are subject to review at the time of construction. Depending on when the work is carried out, cutslopes in the native soils may need to be covered with polyethylene sheeting immediately after excavation to reduce erosion damage to cutslopes as a result of drying out and/or excessive rainfall. According to WorkSafe BC guidelines, excavations deeper than 1.2 m with slopes graded steeper than 0.75H:1V must be inspected and approved by a qualified geotechnical professional prior to workers entering the excavation.

Finalized building designs have not yet been provided; however, it has been our experience that near lot line construction is desired to maximize building footprints. Accordingly, we anticipate that shoring will be required given the depth of excavation required for 1 to 2 levels of underground parking. Open cutslopes may be feasible where bedrock is observed to be shallow; however, encroachment agreements with neighboring owners would need to be sought to allow for excavation beyond property line. In most cases this would impact public sidewalks and vehicle lanes. Where such is not feasible shoring would be required. Shoring would likely consist of conventional reinforced shotcrete and soil/rock anchor system. If 2 levels of underground parking are considered, we expect that vertical supports (i.e. soldier piles) will be required within the shotcrete and soil/rock anchor system where the height of soil above bedrock exceeds 5 m. Subsurface encroachment agreements for the underground installation of anchors will be required where such cross property lines. If surface or subsurface encroachment is required for construction, the District of Saanich may request detailed shoring/cutslope drawings in support of excavation/building permit application. If sidewalks are removed (i.e., for open cutslopes), some compensation/rental agreement should be expected. Further details on temporary excavation support methods can be provided once building designs are better defined and encroachment options are known. It should be noted that the site is bordered by several utility poles, and as such, BC Hydro will likely need to review and approve the proposed excavation geometry.

We expect that as the excavation progresses groundwater ingress will increase and will need to be addressed. Periodic dewatering will be required in low areas as the excavation deepens and discharge volumes will need to be treated in accordance with the erosion and sedimentation plan as specified by the civil consultant. The bulk excavation and shoring installations should be undertaken in a manner that avoids surface water ponding against the soils around the perimeter by maintaining a lower bulk out level within the center of the site to allow drainage away from cutslopes/shoring.

Rock excavation may be required to achieve design grades for portions of the site for the proposed underground parkade. Blasting will be required to remove intact bedrock, and such will need to be undertaken using controlled blasting techniques so as not to impact the adjacent properties or infrastructure. Vibration monitoring during rock blasting should be completed to ensure that peak particle velocities (PPV) are kept below threshold values relating to adjacent structures/infrastructure. There is potential that neighboring residents will raise concerns regarding vibrations, even if some are not structurally harmful, and a pre-blast analysis/survey is strongly recommended to later verify or alleviate such concerns. All rock cutslopes should be reviewed by Ryzuk Geotechnical immediately following blasting to confirm rock quality and the need for additional rock stabilization measures.

Settlement Considerations

The near normally consolidated firm grey silty clay present beneath the northern portion of the site is prone to consolidation in response to increased loading, such as building loads, fill placement and/or dewatering. Additionally, given that it is anticipated that the southern portions of the building may be founded directly atop bedrock, there is an increased risk of differential settlement. Where settlement (total and differential) are anticipated to exceed tolerable limits, such can be mitigated by adequately unloading the site to offset building loads or the use of deep foundations. Given the unloading of the site associated with 1 and 2 levels of underground parking, we anticipate that such will be sufficient; however, a detailed settlement analysis is recommended upon availability of a conceptual structural design.

Seismic Considerations

Greater Victoria is situated in a region of very high seismicity. Considerable earthquake risk exists, stemming from our proximity to the Cascadia Subduction Zone and numerous local faults in southwestern BC and northwestern Washington State.

Based on the soil conditions encountered it is reasonable to expect the shear wave velocity in the upper 30 m (V_s^{30}) to be between 360 and 760 m/s. This corresponds to a Site Classification for Seismic Site Response (Site Class) of 'C', in accordance with the current BC Building Code (2018). However, if Seismic Force Resisting System (SFRS) foundation elements are placed directly atop bedrock, which is likely on the southern portions of the site, a Site Class of 'A' may be used, subject to review at the time of excavation. Alternatively, if less than 3.0 m of soil is present between bedrock and the base of foundation, a Seismic Site Response of 'B' may be used. The seismic hazard values from the 2015 National Building Code Seismic Hazard calculator for Site Class 'C' are provided as an attachment to this report.

The National Research Council of Canada (NRCC) has recently released an updated National Building Code (NBCC 2020) which is anticipated to be adopted by the Province of British Columbia for the BC Building Code in December of 2023 (BCBC 2023), however exact date of release is uncertain. We expect that the BCBC 2023 will fully adopt the NBCC 2020 Division B – 4.1.8. Earthquake Loads and Effects Section. The Structural Commentaries, which accompanies the new code have not been released; however, we can comment on the preliminary information available to date.

The new building code has significantly changed the seismic hazard within Southern Vancouver Island. This is due to an updated 6th generation seismic hazard model for Canada, which has incorporated the Leech River Valley/Devils Mountain Fault, in addition to the updated seismic hazard stemming from the proximity to an active tectonic plate boundary (Cascadia Subduction Zone). The new code has increased seismic response by 15-50% depending on the Site Class, building period and specific location.

The definition of V_s^{30} was also updated to be defined as the time-averaged shear wave velocity **measured from the ground surface** to a depth of 30 m, rather than the underside of the SFRS foundation elements, which has a significant impact on buildings with underground structures. Possible embedment factors may be applicable for projects constructed below grade, but details of such have not be released to date.

For sites where the shear wave velocity is directly measured in-situ, the design spectrum of the site may be calculated using a site specific V_s^{30} value. When in-situ shear wave velocities are not determined, the site may be designated through Site Classification, similar to NBCC 2015, however, the design spectrum is calculated based on the lowest end of the shear wave velocity range of each Site Class, resulting in a more conservative (higher) response.

In-situ shear wave velocity measurements were not completed as part of our investigation. However, based on soil conditions observed it is reasonable to expect the V_s^{30} measured from surface to be between 180 and 360 m/s. This corresponds to a Site Classification for Seismic Site Response (Site Class) of 'D', in accordance with the 2020 NBCC, subject to review upon release of the new code.

Based on the soil conditions encountered during drilling and depending on the locations of seismically relevant foundations, it may be possible to provide an improved Seismic Site Classification of 'C', following additional drilling of a borehole at least 30 m deep to directly measure the shear wave velocity profile. This should only be considered if the structural engineer deems the potential cost savings of an improved Site Class to be significant. If the building spans varying depths to bedrock a seismic break must be incorporated or the more conservative Site Class must be used.

In the event that the building permit for this project is expected to be issued under the new code, likely on or after November/December of 2023 (the actual date is unknown), then the Site Class and seismic information will need to be reevaluated.

Foundation Considerations

We anticipate that conventional shallow foundations will be the desired choice, and will be feasible provided settlement concerns have been appropriately mitigated. Topsoil/organics, non-select fills, and disturbed native soils should not be relied upon for support of foundations. The native stiff to very stiff brown silty clay, firm grey silty clay, glacial till, bedrock, or approved engineered fill atop such are considered capable of providing stable, long term support to the proposed foundations. For design, foundations can be sized using the factored bearing resistance provided below.

Table 1. Summary of bearing resistances for various subgrade conditions

Subgrade Material	**Limit State Design (LSD)**	
	Strip Footing	**Pad Footing**
Native stiff to very stiff brown silty clay or approved engineered fill atop such	145 kPa (SLS) 220 kPa (ULS)	175 kPa (SLS) 265 kPa (ULS)
Native firm grey clay or approved engineered fill atop such	75 kPa (SLS) 115 kPa (ULS)	90 kPa (SLS) 135 kPa (ULS)

Native dense silty sand and gravel (glacial till)	200 kPa (SLS) 300 kPa (ULS)	240 kPa (SLS) 360 kPa (ULS)
Intact/fractured-in-place bedrock*	3000 kPa (ULS)*	3600 kPa (ULS)*

*Settlement if foundations on bedrock are anticipated to be negligible and bearing resistance will be governed by Ultimate Limit State (ULS)

Limit state design (LSD) values use a geotechnical resistance factor of 0.5 as per the current Canadian Foundation Engineering Manual. We recommend minimum footing widths of 400 mm and 600 mm for strip and pad footings, respectively. All foundations should be placed at least 450 mm below grade to provide adequate frost protection.

The above values are provided for design, and should be confirmed at the time of construction. All foundation subgrade must be inspected by the project geotechnical personnel to confirm the noted bearing resistances prior to placement of engineered fill or concrete for foundations. The native silty clay is prone to disturbance from worker traffic and/or water due to runoff or precipitation. Any disturbance of subgrade soils during construction activity will need to be rectified by removal of loose/disturbed soil prior to placement of engineered fill or concrete. If construction will be carried out during the wet season, it is recommended that clay soils be blinded with a layer of select granular fill or concrete soon after exposure to prevent degradation of the subgrade.

Engineered Fill

Engineered fill, if/where required, should consist of well-graded, select granular material and be placed upon approved subgrade. Crushed rock such as 75 mm or 19 mm minus is often used for these purposes. Fill should be placed in maximum 300 mm lifts and compacted to at least 95% Standard Proctor Dry Density (SPMDD) or judged equivalent. Fill placed beneath foundations must extend beyond the footings a distance equal to the fill thickness to ensure 1H:1V lateral splay is present within engineered fill or approved native mineral soil. Placement and compaction of engineered fill should be monitored by a geotechnical professional to ensure proper compaction is achieved.

Grade Supported Lower Floor Slab

Use of a grade supported floor slab is considered feasible provided such bear atop undisturbed native mineral soils or bedrock. We recommend that a poly vapour barrier be placed beneath the slab, atop a minimum of 150 mm of well-graded crushed rock such as 19 mm minus, in order to provide uniform support and minimize capillary rise of moisture into the slab. All sub slab fills should be compacted to at least 95% of Standard Proctor Maximum Dry Density (SPMDD) value.

Foundation Wall Backfill

Foundation walls should be backfilled with clean, well graded granular material, with less than 5% passing the #200 sieve. Backfill should be placed and compacted in maximum 300 mm lifts to at least 95% of the SPMDD value. Additionally, adequate drainage should be provided for the backfill to prevent the buildup of hydrostatic pressure against the foundation walls.

Foundation walls can be designed on the attached Lateral Earth Pressure Diagrams and the following lateral earth pressure coefficients:

Table 2: Lateral Earth Pressure Coefficients

Lateral Earth Pressure Coefficient			
Wall Type	**Static K**		**ΔKae**
Yielding (unrestrained)	Active (K_a)	0.25	0.22
Non-yielding (restrained)	At-Rest (K_o)	0.43	0.54

A yielding wall is able to move a minimum of 0.2% of the height of the wall (rotation or translation) to allow active pressures to develop. Where such movement cannot occur, the non-yielding, at-rest earth pressure coefficient should be used. The above earth pressure coefficients are based on a friction angle of 35 degrees (compact gravelly sand). Lower earth pressure coefficients could be provided if better quality backfill material, such as imported crushed rock. Seismic earth pressures for yielding and non-yielding walls are based on 50% and 100% of the site class PGA, adjusted for Site Class 'C', respectively.

In the case where the design assumptions above and noted on the attached diagrams are not satisfied, a site-specific assessment of the lateral earth pressures would be required. Once the Seismic Site Response of the buildings have been defined there may be adjustments to the above. Additionally, the lateral earth pressure coefficients will need to be adjusted following the release of the new Building Code and adjusted Site Class.

Permanent Dewatering

The nature of the native mineral soils on site precludes any significant free groundwater flow. It is envisioned that conventional perimeter foundation drainage tied into the recommended free draining backfill material would be suitable to limit hydrostatic pressure on the foundation walls. This, however, does not preclude the possibility of dampness and/or minor seepage, which would be considered building envelope concerns.

The foundation drain arrangement (perforated pipe and uniform gravel/drain rock) should be covered with non-woven geotextile filter fabric (not landscape fabric), or a suitably graded granular medium, to prevent migration of finer materials from the backfill into voids within the drain arrangement. Where perimeter drains will be located on the inside of the building, weep holes should be provided in the foundation wall with clear drain rock providing hydraulic connectivity between the free draining exterior backfill and/or drainage mat, and the perforated drain. Where interior perimeter drains are required, minimum 100 mm diameter weep holes should be installed every 2 m. Plumbing and building envelope details will be by others. Any foundation elements, slab on grades, pits or elevator shafts that are not effectively drained to the perimeter drains will require their own drainage arrangement or will need to be designed to resist hydrostatic pressures.

For perimeter drain design, ground water seepage flow rates will need to be assessed as the excavation progresses. Overall, anticipated seepage rages are expected to be relatively low, and well within typical sump and pump capacities.

Any structure that extends below the invert level of the perimeter drains, such as mechanical pits, etc. could be subject to hydrostatic pressure unless provided with some means of drainage. Alternatively, these structures could be designed to accommodate hydrostatic pressure and tanked to avoid moisture ingress.

The final grade at the site should be shaped to direct surface water away from the building and foundation areas. Wherever possible, the surfacing of the backfill material should be such that infiltration of surface water into the perimeter wall backfill and foundation drains is inhibited.

CLOSURE

We trust the preceding is suitable for your purposes at present. Please don't hesitate to contact our office if we can be of further assistance.

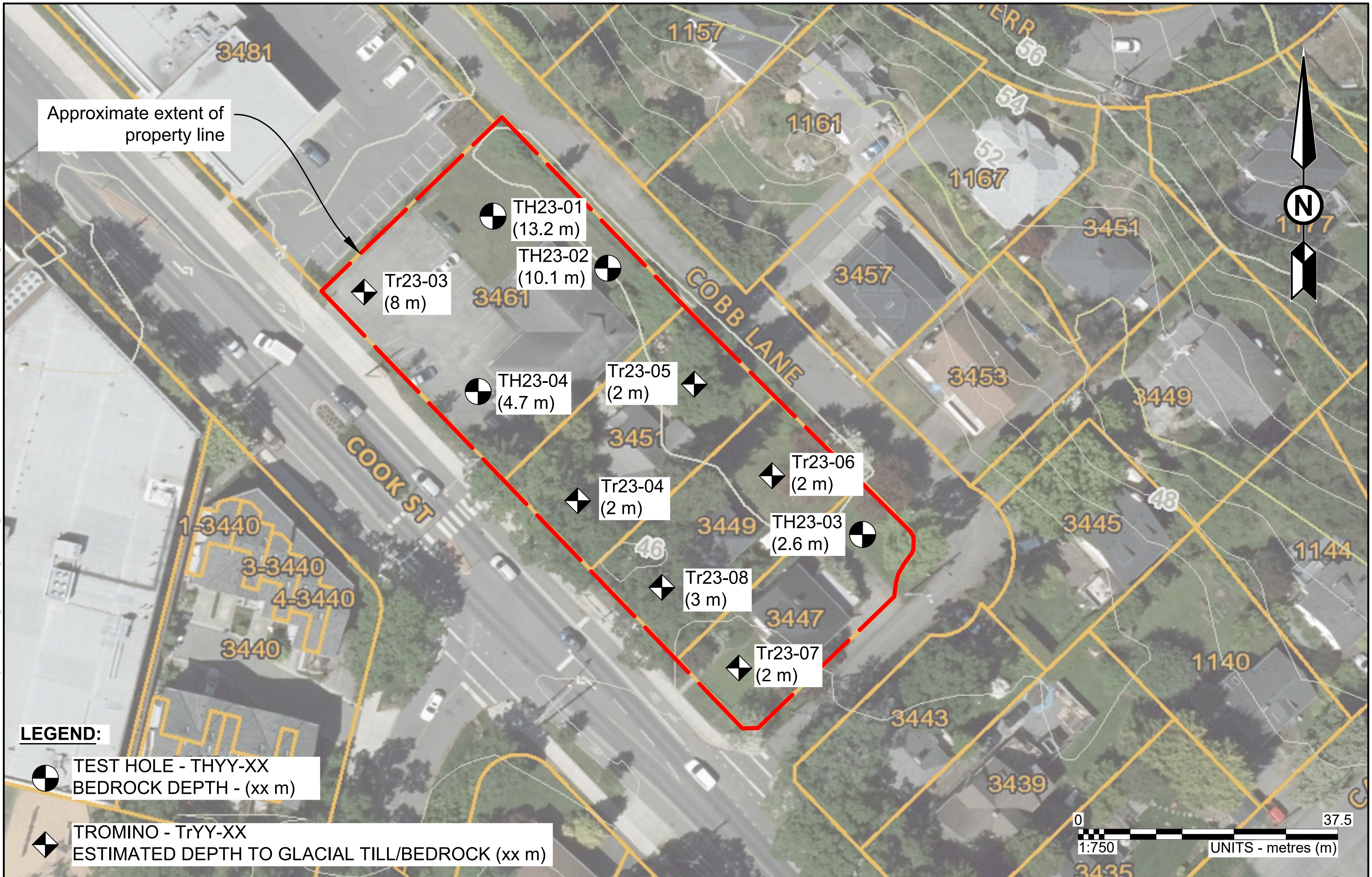
Sincerely,
Ryzuk Geotechnical
PTPN:1002996

Matt Holbrook, EIT
Advanced Junior Engineer

PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
N. KLASSEN
51083
12-May-23

Neil Klassen, P.Eng.
Lead Geotechnical Engineer

Attachments – Test Hole Location Plan
– Test Hole Logs (TH23-01 to TH23-04)
– 2015 NBC Seismic Hazard Calculator
– Lateral Earth Pressure Diagram



NOTES

1. This drawing is scaled for 8.5x11 sheet and does not require further scaling to fit. Scales will differ if printed on different sheet size.
2. Test hole and Tromino locations are based relative to site features. Accurate to +/- 3 m.
3. Tromino depths to glacial till/bedrock are estimates based on the measured site frequency. Accurate to +/- 3 m.

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SEAL

PTPN: 1002996

DRAWN BY	MDH
EOR/LEAD	NK
REVIEW	-
SCALE	1:750
DATE	2023/05/12
CLIENT	GILL DEVELOPMENTS LTD.
PROJECT TITLE	PROPOSED APARTMENT BLOCK
PROJECT ADDRESS	3447, 3449, 3451, 3461 COOK STREET - SAANICH,BC
DRAWING NAME	**TEST PIT LOCATION PLAN**
PROJECT No.	11602-1
SHEET No.	01 of 01
REVISION	0

R:\Ryzuk Data\8-11000 to 8-11999\11602-1 3447, 3449, 3451, 3461 Cook St\5 Ryzuk Drawings\Working Drawings\11602-1.2023.05.05.Test Hole Location Plan.mdh.dwg

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TEST HOLE LOG

TH23-01

PROJECT: Proposed Apartment Block	PROJECT NO.: 11602-1
CLIENT: Gill Developments Ltd.	METHOD: SOLID STEM AUGER
LOCATION: SEE TEST HOLE LOCATION PLAN	ELEVATION (m): 45 m (est.)
COORDINATES (m): UTM N 5366943 E 473553	CONTRACTOR: DRILLWELL
COMPLETION DATE: 2023-4-27	LOGGED/REVIEWED BY: MDH/NK

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	Recovery (%)	Field Vane (kPa) 50 / 100 / 150 / 200; PLASTIC / M.C. / LIQUID 12.5 / 25.0 / 37.5 / 50.0	COMMENTS
0	FILL - grass over topsoil, silt, some gravel, some organics, trace sand, brown, damp					
0.15	Below 0.15 m - interbedded topsoil seams with clay, trace gravel, brown, damp					
1.5	CLAY - silty, trace gravel, hard, brown, damp					
2.4	Below 2.4 m - very stiff					
4.0	Below 4.0 m - stiff					
4.9	Below 4.9 m - firm, grey					
5.0		GRAB	1		M.C. 47.0	
6.0		GRAB	2		M.C. 42.6	
6.7					Field Vane ~49.4	FIELD VANE = 49.4 kPa FIELD VANE(r) = 19.6 kPa
7.0		GRAB	3		M.C. 42.2	
8.0		GRAB	4		M.C. 44.1	
8.7	SAND - silty, some gravel, dense, grey, moist (glacial till)					
13.2	End of Test Hole at 13.2 m - Refusal on inferred dense till or bedrock					

SAMPLE TYPE: SPLIT SPOON, GRAB, SHELBY TUBE, BULK, CORE, NO RECOVERY

TEST HOLE LOG

TH23-02

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Tel: 250-475-3131 E-mail: mail@ryzuk.com
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PROJECT: Proposed Apartment Block
CLIENT: Gill Developments Ltd.
LOCATION: SEE TEST HOLE LOCATION PLAN
COORDINATES (m): UTM N 5366938 E 473575
COMPLETION DATE: 2023-4-27

PROJECT NO.: 11602-1
METHOD: SOLID STEM AUGER
ELEVATION (m): 45 m (est.)
CONTRACTOR: DRILLWELL
LOGGED/REVIEWED BY: MDH/NK

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	Recovery (%)	PLASTIC — M.C. — LIQUID (12.5, 25.0, 37.5, 50.0)	COMMENTS	DEPTH (m)
0		FILL - grass over topsoil, silt, some organics, brown, damp						
		FILL - clay, silty, some organics, brown, damp						
1		CLAY - silty, hard, brown, damp						1
2								2
3		Below 2.7 m - very stiff, grey						3
4		Below 4.0 m - stiff						4
5			GRAB	1		43.0		5
6		SAND - silty, some gravel, dense, grey, damp (glacial till)						6
7								7
8								8
9								9
10		End of Test Hole at 10.1 m - Refusal on inferred dense till or bedrock						10
11								

SAMPLE TYPE: SPLIT SPOON | GRAB | SHELBY TUBE | BULK | CORE | NO RECOVERY

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TEST HOLE LOG

TH23-03

PROJECT: Proposed Apartment Block	PROJECT NO.: 11602-1
CLIENT: Gill Developments Ltd.	METHOD: SOLID STEM AUGER
LOCATION: SEE TEST HOLE LOCATION PLAN	ELEVATION (m): 45 m (est.)
COORDINATES (m): UTM N 5366906 E 473607	CONTRACTOR: DRILLWELL
COMPLETION DATE: 2023-4-27	LOGGED/REVIEWED BY: MDH/NK

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	Recovery (%)	COMMENTS	DEPTH (m)
0		FILL - grass over topsoil, silt, some organics, brown, damp					
		CLAY - silty, hard with very stiff seams, brown, damp					
1							1
		SAND - silty, some gravel, dense, brown, damp (glacial till)					
2							2
		End of Test Hole at 2.6 m - Refusal on inferred dense till or bedrock					
3							3
4							

SAMPLE TYPE: SPLIT SPOON, GRAB, SHELBY TUBE, BULK, CORE, NO RECOVERY

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TEST HOLE LOG

TH23-04

PROJECT: Proposed Apartment Block
CLIENT: Gill Developments Ltd.
LOCATION: SEE TEST HOLE LOCATION PLAN
COORDINATES (m): UTM N 5366927 E 473552
COMPLETION DATE: 2023-4-27

PROJECT NO.: 11602-1
METHOD: SOLID STEM AUGER
ELEVATION (m): 45 m (est.)
CONTRACTOR: DRILLWELL
LOGGED/REVIEWED BY: MDH/NK

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	Recovery (%)	COMMENTS	DEPTH (m)
0		FILL - asphalt					
		FILL - clay, silty, some sand, trace gravel, brown, damp					
1		CLAY - silty, trace gravel, hard, brown, damp					1
		Below 1.5 m - very stiff seams					
2							2
3							3
4		SAND - silty, some gravel, dense, brown, wet					4
		End of Test Hole at 4.7 m - Refusal on inferred dense till or bedrock					
5							5
6							

SAMPLE TYPE: SPLIT SPOON, GRAB, SHELBY TUBE, BULK, CORE, NO RECOVERY

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 48.455N 123.358W **User File Reference:** 3447 - 3461 Cook Street - Saanich, BC 2023-05-10 16:08 U

Requested by: Matt Holbrook, EIT, Ryzuk Geotechnical

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.707	0.504	0.370	0.165
Sa (0.1)	1.078	0.779	0.569	0.254
Sa (0.2)	1.295	0.936	0.690	0.309
Sa (0.3)	1.295	0.936	0.688	0.304
Sa (0.5)	1.147	0.820	0.593	0.249
Sa (1.0)	0.668	0.452	0.311	0.118
Sa (2.0)	0.392	0.257	0.170	0.061
Sa (5.0)	0.122	0.071	0.038	0.012
Sa (10.0)	0.043	0.024	0.013	0.004
PGA (g)	0.576	0.417	0.305	0.134
PGV (m/s)	0.825	0.566	0.393	0.149

Notes: Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

National Building Code of Canada 2015 NRCC no. 56190; Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

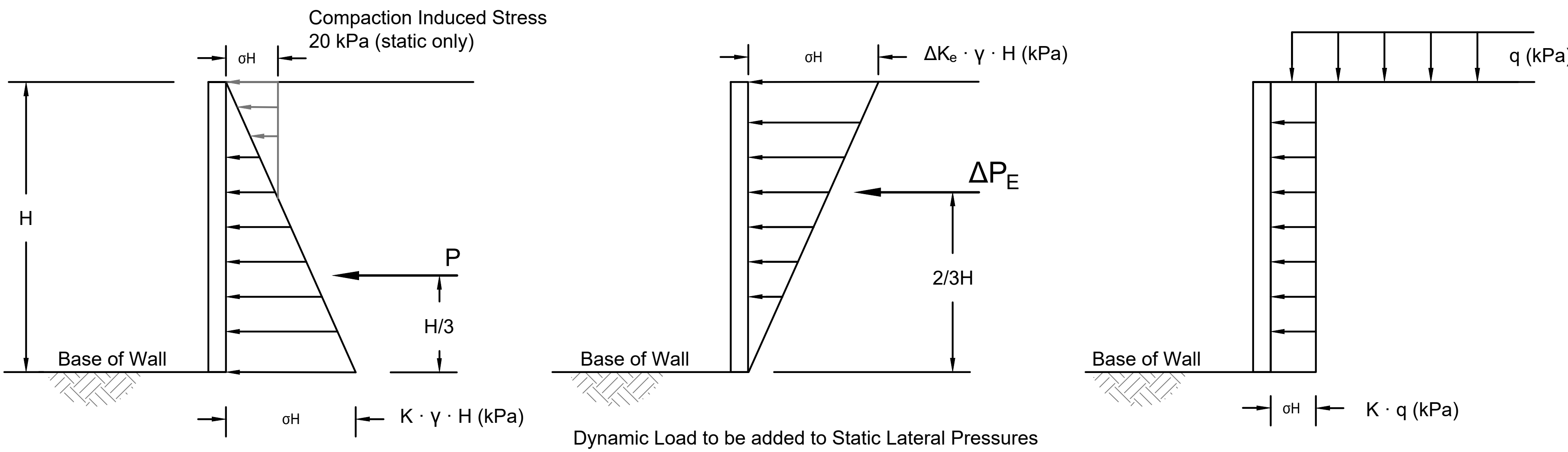
Lateral Earth Pressures
STATIC CONDITIONS

+

Lateral Earth Pressures
SEISMIC INCREMENT
(Added to Static Earth Pressures)

+

Uniform Surcharges,
q (Floor Loads or Traffic Loads)



Dynamic Load to be added to Static Lateral Pressures

σH* = Lateral Pressure from Uniform Surchage
*Only applicable where surchage load is less than 30% of total lateral load on wall

Where:
γ = Dry Backfill unit weight 20.4 kN/m^3
H = Wall height (m)
σH = lateral earth pressure (kPa)
P = Resultant load (kN)
K = dimensionless coefficient, K_a or K_o (see Report)

Analysis Assumptions:
- Wall friction is half the soil
- Drainage is provided, such that hydrostatic pressures do not develop against wall
- Dynamic loading based on 50% of the Peak Ground Acceleration (PGA) for yielding wall and 100% PGA for a non-yielding wall
- Yielding wall assumes that wall movement of 0.2%H (rotation or translation) is possible
- The grade is flat and level adjacent to the wall
- No surcharge loads from adjacent structures or stockpiles within a horizontal distance equal to the wall height
- No equipment larger than a skid steer permitted within 1.5 m of the wall during backfill
- Compaction induced stresses will be relieved during a seismic event and are not included in Seismic load

NOTES
1. Above Diagrams are not to scale
2. All loads are unfactored.

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LATERAL EARTH PRESSURE DIAGRAMS

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