RYZUK GEOTECHNICAL

Geotechnical Feasibility Study

1327 Beach Drive – Oak Bay, BC

Prepared for: **The District of Oak Bay**
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1. EXECUTIVE SUMMARY

Ryzuk Geotechnical Ltd. (Ryzuk) was engaged by the District of Oak Bay (The District) to complete a geotechnical study and investigation of the existing parking area at Turkey Head. Based on our discussions and previous work at the site, we understand that The District is exploring development options for the site and, as such, wanted a better understanding of the underlying soil conditions. Accordingly, Ryzuk completed an office-based study of the local conditions followed by a confirmatory drilling investigation, in-situ seismic testing, and laboratory testing program.

Based on our work, we confirmed that the soil conditions at the site generally consist of historic fills placed atop the native marine deposits of sand, clay and/or bedrock. The thickness of the historic fills generally increased when moving northeast from Beach Drive which coincides with our findings of the approximate historical coastline that existed before the marina development.

In terms of development feasibility from a geotechnical perspective, we expect that the site could host a range of development options. However, the existing historical fills should not be considered suitable to provide long-term stable support to any proposed permanent structures because these fills consist of non-structural materials (organics, construction debris, clay, silt, etc.) and were likely not methodically placed and/or compacted. Therefore, we anticipate that a significant site preparation program would be required to remove and replace the existing fills if the use of conventional spread footings is the desired foundation option. Alternatively, the use of deep foundations (piles or caissons) could be considered to support a proposed structure and limit the volume of existing fill to be removed.

Based on our assessment of the site's seismic response, we expect that additional detailed analysis will be required to confirm that the site soils are not susceptible to liquefaction and, if they are, how best to manage such safely. The encountered soil conditions during our investigation suggest that the certain soil layers may be susceptible to liquefaction. Given the subsurface conditions, we would expect that liquefaction would manifest as settlement caused by the historical fills sliding down the underlying sloped bedrock surface and/or localized consolidation of the loose deposits of historical fills. The use of deep foundations (piles or caissons) would greatly alleviate the impact of liquefaction; however, such would have to be specially designed to resist the additional stresses caused by the site soils flowing around the piles/caissons as they slide down the bedrock profile during an earthquake.

In summary, we expect that while development is possible at the site, additional geotechnical and structural engineering input will be required during the preliminary feasibility and/or design phase of a proposed development.

2. INTRODUCTION

As requested, we have completed a geotechnical investigation of the subsurface conditions at the referenced site. The following report summarizes the results of our investigation and the geotechnical recommendations related to the proposed development. Our work in this regard has been carried out in accordance with our proposal, dated December 30, 2024, and previously accepted Terms of Engagement.

3. PROPOSED DEVELOPMENT

Based on our discussions with District of Oak Bay (District) staff, we understand that the District would like to understand the geotechnical conditions of the site to support their feasibility study for future development. However, we explored the feasibility of a range of development options with the most technically complex options having one or more levels of underground space and high building loads.

4. INVESTIGATION PROCEDURE

Our geotechnical investigation included an office-based review of relevant, publicly available, topographical data, and geotechnical information to establish expected subsurface conditions followed by a confirmatory geotechnical drilling investigation and in-situ seismic testing. Due to drill rig availability, in-situ seismic testing was completed before the drilling investigation.

Our desktop study was based on the available aerial imagery, geological mapping, historical maps, historical photos of the marina and parking lot construction, and our previous report regarding the expected foundation subgrade conditions of the existing marina building dated May 31, 2023. Additionally, we reviewed our project database for any previous and/or on-going Ryzuk projects in the nearby area.

The in-situ seismic testing consisted of a Multi-Channel Analysis of Surface Waves (MASW) which is a non-invasive geophysical survey method used to determine the shear wave velocity (Vs) of the subsurface. The in-situ Vs data is a critical component for determining the seismic response of the site and directly supports the structural design process of a proposed building.

We completed our MSAW survey on January 27, 2025, using equipment owned and operated by Ryzuk Geotechnical. The testing featured two consecutive arrays running adjacent and parallel to the northeast-southwest central axis of the parking lot as shown on the Test Hole Location Plan. The MASW data was collected and interpreted in general conformance with Geometrics SeisImager/SW Manual v3.0 software and industry standards. A multi-channel recording seismograph and surface array of geophones was utilized to collect the data. The active data is recorded through repeated strikes of a sledgehammer on a steel plate, offset from the array. The passive data set is created by recording passive surface waves generated at/around the site by natural or manmade sources.

The collected data is converted into a dispersion curve which plots the phase velocity as a function of frequency. From the dispersion curve, the fundamental mode is chosen, and an inversion is run to produce a one-dimensional shear wave profile against depth. Both passive and active data can be combined at each location to image both the shallow and deep properties of the subsurface. The resulting profiles of shear wave velocity with depth are displayed in the attached MASW Shear Wave Velocity Profiles. It should be noted that bedrock shear wave velocities are difficult to accurately determine within the current MASW survey data, however, we can assume such based on our experience in the general area.

Lastly, the subsurface drilling program was completed on March 6 and 7, 2025, and consisted of advancing five test holes (TH25-01 to -05) using a sonic drill rig supplied and operated by Drillwell Enterprises Ltd. Test hole locations are shown on the attached Test Hole Location Plan. Prior to any ground disturbance, we completed a BC One Call ticket, and each test hole location was cleared of utilities by a third-party utility locator. During drilling, recovered disturbed soil samples were logged by Ryzuk Geotechnical personnel based on the Modified Unified Soil Classification System (MUSCS). In-situ testing consisted of Standard Penetration Testing (ASTM D1586M-18). The SPT hammer efficiency of the drill rig used was previously tested/recorded by Ryzuk Geotechnical in 2023 and found to be roughly 75%. Select soil samples were submitted for laboratory testing that consisted of moisture content testing (ASTM D2216-19) and Atterberg Limits testing (ASTM D4318). Test holes were backfilled with drill cuttings and capped with cold-mix asphalt. Any remaining cuttings that were not able to be used as test hole backfill were left on site in steel barrels to be removed by the District of Oak Bay's Public Works staff. Elevations shown on the test hole logs were estimated from Google Earth. A sixth test hole was originally proposed, TH25-06, but had to be cancelled due to time constraints.

Groundwater monitoring wells were not installed as a part of our investigation. However, given the proximity of the site to the ocean, we expect that the groundwater table essentially matches the average ocean elevation and may fluctuate with the tides.

5. INVESTIGATION

5.1 SURFACE CONDITIONS

The site is a peninsula called Turkey Head which is off the east coast of south Oak Bay with an approximate area of 3,278 m^2. Topographically the site is relatively flat, with a site elevation of approximately 4.0 m geodetic. Turkey Head is bound by the Oak Bay shoreline to the north, east, and west, and Beach Drive to the southwest. Currently, the site hosts a private marina building in the northwest portion of the site, an associated paved parking area, a concrete pumphouse in the southwest corner, and a large entry sign between the entry and exit lanes. We observed outcropping bedrock near the driveway entrance/exit and below the cantilevering marina buildings on the northwest perimeter of the site.

Based on the historical information reviewed during our office-based study, we expect that the original surface conditions, prior to the construction of the parking lot, consisted of an undulating bedrock outcrop extending into Oak Bay, similar to the coastal features to the north and south of Turkey Head. Based on the "Insurance Plan of Victoria" Volume 3 published in 1913, we were able to establish an approximate outline of the original coastline, as shown on the attached Test Hole Location Plan.

5.2 SUBSURFACE CONDITIONS

The information reviewed during our office-based study of the site suggested that the native soils within the original coastline consisted of thin soil deposits atop shallow bedrock with areas of outcropping bedrock. Native soils were expected to consist of topsoil, silty clay, and/or glacial till. Beyond the original coastline, we expected the thickness of the historical fills to generally increase when moving northeastward, but due to typically erratic local bedrock profile the fill thickness would vary. Furthermore, we anticipated that the historical fill would have been placed atop either native poorly graded medium grained sand, soft to firm marine clay, and/or bedrock. The composition of the historical fill was expected to vary from boulders and cobbles, typically used as an initial lift below the water line, followed by non-select material (construction waste, clay, sand and gravel, organics, etc.). Archived construction photos indicated that the original marina building was constructed in 1915 and included a small parking lot followed by the construction of the current parking lot in the late 1960's. Therefore, we expect the existing fills were placed with little to no compaction effort aside from the weight of the earth moving machinery (i.e. bulldozer and dump trucks) as was common practice at the time.

The subsurface soils encountered during our investigation were generally consistent with our office-based study. The subsurface stratigraphy generally consisted of 4.0 to 9.1 m of non-select and granular fill, atop native soil and/or bedrock. Notable subsurface encounters consisted of a large log in TH25-03 causing refusal at a depth of 7.2 m below ground surface (mbgs), a shallow deposits of cohesive fills in TH25-01 and TH25-04 above approximately 2.0 mbgs, and a 2.5 m thick layer of native firm, compressible, silty clay at a depth of 9.1 mbgs in TH25-04.

Based on the retrieved samples and recorded SPT blow counts, the encountered granular fill primarily consisted of poorly graded sand and subangular gravel that exhibited relative densities ranging from loose to compact. The termination depths and approximate thicknesses of the existing fills at each test hole location are summarized in Table 1.

Table 1 - Summary of Fill Thickness per Test Hole

Test Hole ID	Approximate Ground Surface Elevation (m geo.)	Approximate thickness of existing fills (m)	Test Hole Termination Depth (mbgs)
TH25-01	4	3.2	3.5
TH25-02	4	4.1	4.1
TH25-03	4	7.2	7.2
TH25-04	4	9.1	14.9
TH25-05	4	6.9	6.9

A sample of the native firm silty clay was submitted for Atterberg limits testing and resulted in a Liquid Limit of 56%, Plastic Limit of 23% and a Plasticity Index of 33%, indicating that the clay is classified as high plastic. Additionally, the results of the moisture content testing indicated that the native clay had a natural moisture content of 39.9%.

Long-term groundwater monitoring was not completed as part of our work. However, we expect that such would be closely linked to the surrounding ocean levels. Additionally, perched groundwater conditions should be expected due to the uncontrolled placement and fill selection during parking lot construction. Such conditions usually lead to infiltrating surface water becoming perched within permeable granular fills that were placed atop relatively impermeable cohesive fills.

6. GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

Based on our investigation, we expect that the development of the site as assumed is feasible from a geotechnical perspective. However, careful consideration regarding the foundation design of a proposed building will be required given the thick deposit of existing fill across much of the site. Therefore, we expect that ground improvements or deep foundations would be required to facilitate building construction. We note that more than one level of underground parking/space would result in building buoyance/drainage/envelope concerns due the local groundwater conditions being linked to the surrounding ocean. Furthermore, if permanent loads are to be placed atop the encountered native silty clay, a detailed analysis should be completed to confirm the magnitude of long-term settlement. Lastly, careful consideration/analysis regarding the potential for liquefaction of the existing fills and loose saturated native sand encountered at depth in TH25-04 during a seismic event. We expect that liquefaction would manifest as localized settlement and/or lateral spreading; the latter being of particular concern for the existing fills atop the sloping local bedrock conditions. Our general and specific recommendations for a proposed development at the site are summarized in the following sections.

6.1 SITE PREPARATION

Site preparation should consist of the removal of all loose and deleterious materials from the proposed building footprints and associated foundation loading splay areas, taken as a 1 horizontal to 1 vertical (H:V) angle from the edge of a footing to native soils or bedrock. However, given the depth of the encountered fills at the site and ocean sourced groundwater conditions, we expect that the total removal of the existing fills may not be feasible/cost-effective for all areas of the site. If the proposed building/development were to be limited to fit within the confines of the original coastline, a remove and replace ground improvement program may be feasible due to the reduced fill thicknesses encountered. Proposed buildings sited beyond the original coastline would likely require deep foundations to be considered feasible.

6.2 EXCAVATION CONSIDERATIONS

We understand that our work is in support of the District's feasibility study for the site and that such is in its preliminary stages. Therefore, the required bulk excavation depths are unknown. However, we anticipate that any excavation work at the site will be limited by the local groundwater conditions. Once excavation plans are available, they should be reviewed by a qualified professional to assess the total excavation depth (if fill removal is selected) and the associated excavation dewatering requirements. If building footprints allow for open cutslopes during bulk excavation, we considered the following cutslope geometries suitable for the encountered conditions:

- 1.5H : 1V for topsoil or fill materials,
- 1H : 1V for native compact to dense sands and gravels, and
- Near-vertical in bedrock.

Adjustments to the above configurations would be required during excavation if variations in the soil/seepage conditions are encountered. Additionally, and in accordance with WorkSafeBC guidelines, excavations deeper than 1.2 m and/or adjacent to existing structures must be inspected and approved by a qualified geotechnical professional before worker entry or approaching within a distance equal to the excavation depth.

6.3 SHORING CONSIDERATIONS

Based on our experience, we expect that if a moderate to major building design is to be constructed such would likely include at least one level of underground parking. Accordingly, there would be desired to limit the bulk excavation footprint to reduce the volume of existing fill to be removed, thereby, minimizing the soil disposal costs to the project. As such, the use of shoring could be considered to avoid relatively flat open cutslopes, thus limiting lateral extents of the bulk excavation.

We anticipate such shoring would consist of a conventional shotcrete and anchor system. This system has been installed successfully numerous times locally and consists of a reinforced shotcrete membrane supported by series of rock anchors. The advantage of this system is that the supporting anchors are tensioned before the bulk excavation continues resulting in little lateral movement of the system as it takes on the lateral soil pressure. The disadvantage of this system at this site particularly is that shorter soil anchors could not be used to support the shotcrete membrane given the presence of non-select fills confining the excavation. Therefore, longer, i.e. more costly, rock anchors would be required.

Alternatively, the use of a cantilevering or internally braced system could be considered instead of a shotcrete and anchor system. A cantilevering system consists of lagging elements, shotcrete and/or timbers, spanning between steel soldier piles which provide stiffness to the system via the piles' embedded depth beneath the proposed excavation depth. This system may encounter challenges during soldier pile installation due to obstructions during piling operations and would require boring/socketing into bedrock in areas where such is shallow. Additionally, longer than typical embedment lengths would be required to compensate for embedment within existing/undocumented non-select fills. An internally braced system typically consists of soldier piles and lagging supported by internal braces rather than the embedded portion of the soldier piles, allowing for shorter piles than the cantilevering system. However, the internal bracing can cause construction inefficiencies due to their placement within the proposed building footprint and, therefore, are typically avoided if possible.

Further guidance for bulk-excavation planning and support can be provided once preliminary building designs become available.

6.4 SEISMIC CONSIDERATIONS

Lower Vancouver Island is situated in a region of high seismicity. Considerable earthquake risk exists, stemming from its proximity to the Cascadia subduction zone and numerous more local faults in southwestern BC and northwestern Washington State.

The current BC Building Code 2024 (BCBC 2024) has significantly changed the seismic hazard in parts of BC. This is due to the adoption of the 6th generation seismic hazard model for Canada, which has incorporated the updated seismic hazard stemming from the proximity to an active tectonic plate boundary (Cascadia Subduction Zone). The new code has increased seismic response depending on the Seismic Site Classification (Site Class), building period, and specific location.

Furthermore, the BCBC 2024 considers the time-averaged shear wave velocity of the site as the main basis for the seismic design process. The definition of the time-averaged shear wave velocity (Vs30) parameter was updated to be measured from the ground surface to a depth of 30 m, rather than from the underside of the Seismic Force Resisting System (SFRS) foundation elements, as it was in the previous BCBC 2018. Additionally, 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 Vs30 value instead of a more conservative design spectrum associated with a Site Class designation.

Our MASW25-01 and MASW25-02 surveys determined a Vs30 of 448 m/s and 414 m/s, respectively. MASW25-01 survey was completed within the southern portion of the site, within an area of shallower bedrock, as confirmed by the drilling investigation and the office-based review. MASW25-02 survey was completed within the northern portion of the site, within an area of deeper bedrock, which was later confirmed with our drilling investigation.

Both values of the Vs30 obtained in the MASW25-01 and MASW 25-02 fall within a conventional Site Classification (Site Class) of 'C' in accordance with the BCBC 2024 seismic provisions, which ranges between 360 m/s – 760 m/s. However, we recommend using the design spectral accelerations associated a Vs30 value of 414 m/s which can be provided by the online 2020 NBCC Seismic Hazard Tool.

6.4.1 Liquefaction

Based on our review of the local bedrock conditions, we expect that the underlying bedrock is generally sloping down towards the northeast. Therefore, once building locations and designs have been established, we recommend that a detailed liquefaction analysis be completed to confirm the existing fills will remain stable during a seismic event and not spread laterally due to the sloping bedrock profile. Additionally, we expect that there is a possibility for densification of the encountered loose, saturated, native sands at depth in TH25-04. However, given the depth of TH25-04 relative to the surrounding test holes, we expect that there may be a localized depression in the bedrock profile, thus confining the loose sand deposit and preventing it from spreading laterally.

The recommended liquefaction analysis would likely require additional geotechnical investigation work consisting of a bedrock probing program to establish the local bedrock profile and Cone Penetration Testing program to collect additional strength parameters of the subsurface stratigraphy. Samples of the native sand deposit should also be submitted for laboratory testing to confirm the soil's fines content.

6.5 FOUNDATIONS

6.5.1 Spread Footings

Based on the encountered soil conditions, we expect that conventional spread footings would be feasible for a building located within the original coastline area, assuming one level of underground parking is included in the building design. In this area, we expect that footings would be placed atop dense glacial till, intact/fractured in placed bedrock, or engineered fill placed atop such. Accordingly, foundation elements could be sized based on the serviceability and ultimate limit state bearing resistances (SLS & ULS) summarized in Table 2.

Table 2 - Summary of SLS and ULS Bearing Resistance for Spread Footing Elements

Subgrade Material	Strip Footings		Pad Footings	
	SLS	ULS	SLS	ULS
Glacial till or engineered fill over intact/fractured in-place bedrock or engineered fill atop such	335 kPa	500 kPa	400 kPa	600 kPa
Clean Intact/fractured in-place bedrock	Note[1]	2,475 kPa	Note[1]	3,000 kPa

Note 1: Settlement for foundations directly atop bedrock are expected to be negligible, therefore ULS bearing resistance governs.

Limit state design (LSD) values use a geotechnical resistance factor of 0.5 as per the current Canadian Foundation Engineering Manual. The above is based on minimum strip footing dimensions of 450 mm and pad footing dimensions of 2 m by 2 m, with a minimum embedment of 0.5 mbgs. SLS bearing resistances are based on a maximum allowable settlement of 25 mm.

If uplift resistance is required for spread footings, we recommend the installation of seismic rock anchors. These anchors typically consist of vertically orientated high-strength steel threadbar that are drilled and fully grouted into bedrock. The sizing and embedment length into bedrock would be calculated once the anchor layout and associated uplift loads are known.

6.5.2 Deep Foundations

As noted previously, we expect the removal and replacement of existing fills placed beyond the original coastline would be too costly to remove and replace with engineered fill. Therefore, for buildings located beyond the original coastline or for building loads in excess of the bearing resistance values in Table 2, we recommend the use of steel piles and/or drilled caissons. The following is a brief summary of both deep foundation options and further details could be provided once build loads and foundation layout have been determined.

6.5.2.1 Driven Piles

Typically, steel piles are suitable for mid-rise buildings and are driven until refusal on dense glacial till (if present) or bedrock. For design considerations, such piles typically comprise concrete filled 324 mm diameter, 9.5 mm wall pipe with a design ultimate limit state load of 800 kN and serviceability loads are generally governed by the structural resistance of the pile. All driven piles should include a hardened 100 mm diameter Oslo Tip or equivalent driving shoe which can be ‘chiselled’ into sloping bedrock during driving operations and thus avoid the pile from sliding. Pile installation will require monitoring and verification for ultimate load capacity. Such verification is typically done using a Pile Driving Analyzer (PDA) which involves attached sensors to the pile has been seated and striking it with the driving hammer to record the pile’s dynamic/elastic response and then calculate the ultimate resistance of the pile.

6.5.2.2 Caissons

Where bedrock is deeper than roughly 10 m and/or building loads are high, drilled caissons may be preferred. Caissons are drilled and socketed into the bedrock to afford high bearing and uplift resistance and locally range in diameter between 0.6 to 1.2 m. The base of all caissons will need to be clean of all soil/muck to expose intact bedrock. Caissons socketed a depth of two pile diameters can be designed to the full structural capacity of the element, while embedment for tensile capacity would be reviewed based on the required uplift resistance criteria provided by the structural engineer.

6.6 ENGINEERED FILL

Engineered fill, if required, should consist of well graded, granular, free draining material placed upon geotechnically approved native subgrade soil. Engineered fill should be compacted to at least 95% Standard Proctor Dry Density (SPMDD), or judged equivalent by the geotechnical engineer. Appropriate lift thickness will depend on fill gradation and type of compaction equipment utilized and must be confirmed by the geotechnical engineer at time of construction. Engineered fill placed beneath foundations should extend beyond the footings as necessary to ensure 1H : 1V lateral splay is present within engineered fill or approved native mineral soil. Placement and compaction of engineered fill shall be monitored by the geotechnical engineer to ensure adequate compaction is achieved. In areas where fill must be placed below the water table, we recommend that dewatering efforts are intensified to completely drain the excavation during fill placement/compaction to avoid mobilizing specialized equipment to site that can effectively compact thicker lifts of fill.

6.7 FOUNDATION WALL BACKFILL & EARTH PRESSURES

In the event, one or more levels of underground space are included in the building design, the foundation walls should be backfilled with clean, free-draining, well-graded granular material, with less than 5% passing the #200 sieve by mass. Backfill should be placed and compacted in a maximum of 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. If adequate drainage cannot be provided, the foundation walls must be designed to withstand the buildup of hydrostatic pressures and tanked to prevent moisture ingress.

Foundation walls backfilled with engineered fill can be designed based on the attached Lateral Earth Pressure Diagrams and Surcharge Loading Diagrams, as well as the following lateral earth pressure coefficients, adjusted for a Vs30 of 414 m/s (NBC 2020 seismic values) for the subject site, which are based on a friction angle of 36˚ assuming well-graded crushed rock is used for backfill:

Table 3 - Summary of Lateral Earth Pressure Coefficients

Lateral Earth Pressure Coefficient			
Wall Type	**Static K**		**ΔK_e**
Flexible (unrestrained)	Active (K_a)	0.24	0.34
Non-flexible (restrained)	At-Rest (K_o)	0.41	*

**Note:* *For non-flexible walls the seismic active earth pressure would have to be calculated using specialized software and be based on the proposed building's embedment depth(s) due to the increased Peak Ground Accelerations (PGA) provided in the BCBC 2024.*

A flexible 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-flexible, at-rest earth pressure coefficient should be used. Seismic earth pressures for flexible and non-flexible walls are based on 50% and 100% of the Site Class adjusted PGA, respectively.

In the case where the design assumptions above and noted in the attached diagrams are not satisfied, a site-specific assessment of the lateral earth pressures would be required.

6.8 FOUNDATION DRAINAGE/PERMANENT DEWATERING

Conventional perimeter foundation drainage tied into the recommended free-draining backfill material would be suitable to limit hydrostatic pressure on the foundation. 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. Interior foundation drains are recommended where space is limited or in areas where seepage is observed at the design foundation elevation, such as in areas of glacial till exposure. Pipe inverts should be kept 300 mm below the underside of the slab. Plumbing and building envelope details will be completed 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.

We note that the above-described system would not be suitable to maintain a lowered groundwater condition for more than one level of underground parking/space due to the local groundwater conditions being linked to the surrounding ocean. In this case, we would recommend the building be tanked as continuous pumping of the ocean is not considered sustainable.

6.9 SLAB ON GRADE

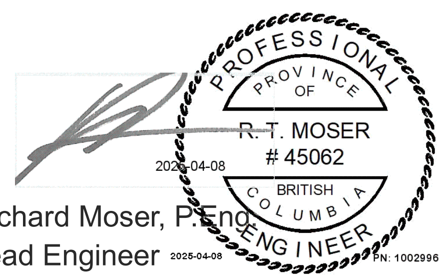
In areas where it is feasible to remove all the existing fill and replaced such with engineered fill, we expect that a grade supported floor slab would be feasible. A minimum 150 mm layer of medium to coarse sand is recommended beneath the slab, as well as a subslab poly barrier, to avoid capillary rise of moisture into the slab. In areas of disturbed bedrock at design subgrade level, care will be necessary to ensure there is an appropriate gradational layer of material to prevent migration of the materials from the recommended subslab course into the voids within the disturbed rock subgrade. All subslab fill should be compacted to at least 95% of SPMDD.

In the remaining areas, we recommend that the lowest floor slab be suspended via grade beams supported by deep foundation elements. However, we still recommend that the subslab fills follow the previously described configuration.

7. CLOSURE

We hope the preceding is suitable for your purposes at present. Please do not hesitate to contact our office if we can be of further assistance.

Sincerely,
Ryzuk Geotechnical
Permit to Practice Number: 1002996



Eugenio Saenz Ramos, EIT
Junior Engineer

Richard Moser, P.Eng.
Lead Engineer

Attachments: Test Hole and MASW Location Plan
Test Hole Logs (TH24-01, TH24-02, TH24-03, TH25-04 and TH25-05)
Modified Unified Soil Classification System Legend
MASW Shear Wave Velocity Profile
Lateral Earth Pressure Diagrams

Distribution:

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RYZUK

GEOTECHNICAL

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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.

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SEAL

PROFESSIONAL PROVINCE OF BRITISH COLUMBIA ENGINEER
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PTPN: 1002996

DRAWN BY: ESR	CLIENT: THE DISTRICT OF OAK BAY		
EOR/LEAD: RTM	PROJECT TITLE: PROPOSED 6-STOREY RESORT		
REVIEW: -	PROJECT ADDRESS: 1327 BEACH DRIVE - OAK BAY, BC		
SCALE: 1:1300	DRAWING NAME: **TEST HOLE AND MASW LOCATION PLAN**	PROJECT No.: 581-1138	
DATE: 2025/03/13		SHEET No.: 1 of 1	REVISION: 1

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TEST HOLE LOG

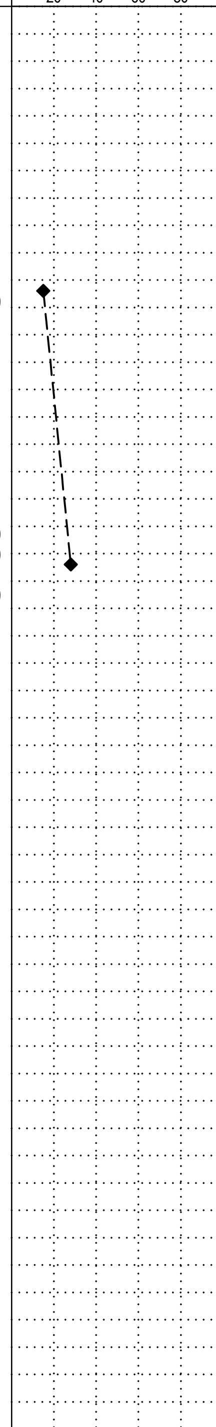
TH25-01

PROJECT: Geotechnical Feasibility Study
CLIENT: District of Oak Bay
LOCATION: Refer to Test Hole Location Plan
COORDINATES (m): UTM N 5363423 E 477646
COMPLETION DATE: 2025-3-6

PROJECT NO.: 0581-1138
METHOD: Sonic
ELEVATION (m): 4
CONTRACTOR: Drillwell Enterprises LTD.
LOGGED/REVIEWED BY: ESR/RTM

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	Recovery (%)	SPT Blow Counts	COMMENTS	ELEVATION (m)
0	ASPHALT - 50 mm						
	SILT - sandy, some clay, trace cobbles, brick, construction debris, well-graded, stiff, cemented, grey, damp (Fill)	GRAB	G1				
1							3
	SAND and GRAVEL - fine to coarse-grained, well-graded, loose, grey, moist (Fill)	GRAB	G2				
	GRAVEL - cobbley, subangular, poorly-graded, compact, grey, wet (Fill)	SPLIT SPOON	S1	5	4 5 10 7		
2		GRAB	G3				2
3		SPLIT SPOON	S2	30	10 20 8 50		1
	SAND and GRAVEL - fine to coarse-grained, compact, grey, moist	GRAB	G4				
	End of test hole at 3.5 m Below Ground Surface (BGS): - Test hole terminated on bedrock. - Test hole backfilled with drill cuttings and bentonite. Remaining 0.2 m capped with asphalt patch.						
4							0
5							-1
6							-2
7							-3
8							

◆ SPT (Standard Pen Test) ◆
(Blows/300mm)
20 40 60 80



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

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www.ryzuk.com

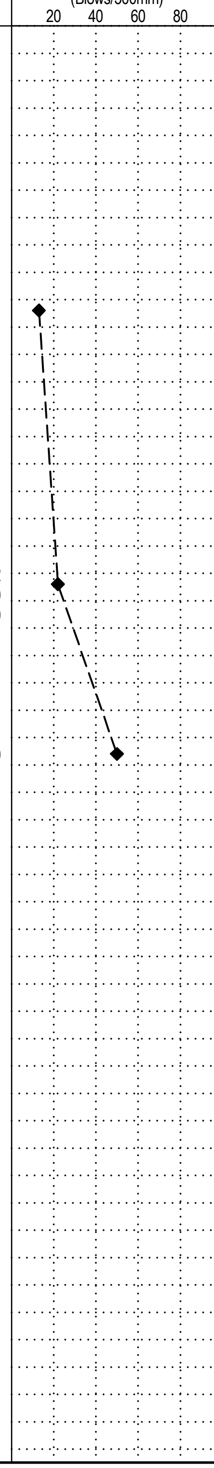
TEST HOLE LOG

TH25-02

PROJECT: Geotechnical Feasibility Study
CLIENT: District of Oak Bay
LOCATION: Refer to Test Hole Location Plan
COORDINATES (m): UTM N 5363409 E 477667
COMPLETION DATE: 2025-3-6
PROJECT NO.: 0581-1138
METHOD: Sonic
ELEVATION (m): 4
CONTRACTOR: Drillwell Enterprises LTD.
LOGGED/REVIEWED BY: ESR/RTM

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	Recovery (%)	SPT Blow Counts	COMMENTS	ELEVATION (m)
0	ASPHALT - 50 mm						
	SAND and GRAVEL - fine to coarse-grained, poorly-graded, loose to compact, grey, moist (Fill)						
	SAND - silty, trace gravel, fine to medium-grained, well-graded, compact, cemented, grey to brown, damp (Fill)	GRAB	G1				
1							3
		GRAB	G2				
	Below 1.5 m - becomes with some cobbles and wet (Fill)	SPLIT SPOON	S1	38	8 5 8 9		
2							2
		GRAB	G3				
3							1
		SPLIT SPOON	S2	7	5 12 10 10		
		GRAB	G4				
4	Below 4.0 m - becomes gravelly	SPLIT SPOON	S3	0	50 0		0
	End of test hole at 4.1 m Below Ground Surface (BGS): - Test hole terminated on bedrock. - Test hole backfilled with drill cuttings and bentonite. Remaining 0.2 m capped with asphalt patch.						
5							-1
6							-2
7							-3
8							

◆ SPT (Standard Pen Test) ◆
(Blows/300mm)



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

TEST HOLE LOG

TH25-03

RYZUK GEOTECHNICAL
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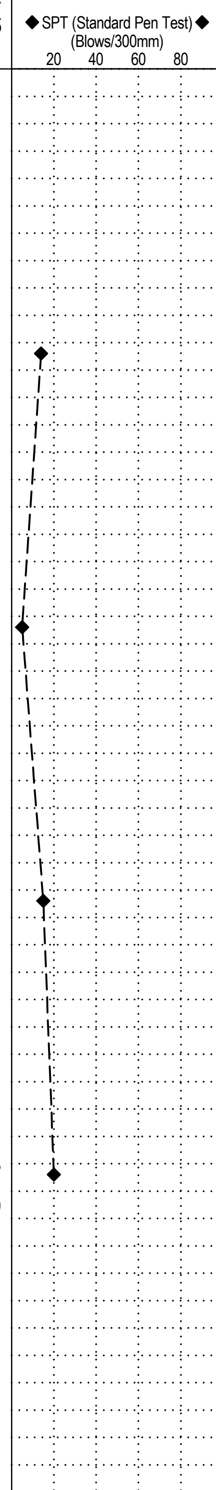
PROJECT: Geotechnical Feasibility Study
CLIENT: District of Oak Bay
LOCATION: Refer to Test Hole Location Plan
COORDINATES (m): UTM N 5363468 E 477687
COMPLETION DATE: 2025-3-6

PROJECT NO.: 0581-1138
METHOD: Sonic
ELEVATION (m): 4
CONTRACTOR: Drillwell Enterprises LTD.
LOGGED/REVIEWED BY: ESR/RTM

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	Recovery (%)	SPT Blow Counts	COMMENTS	ELEVATION (m)
0	ASPHALT - 50 mm						
	SAND and GRAVEL - trace cobbles, medium to coarse-grained, poorly-graded, loose, grey, wet (Fill)	Grab	G1				
	GRAVEL - silty, sandy, some organics, subangular to angular, well-graded, loose, grey, dry (Fill)						
1							3
		Grab	G2				
		Split Spoon	S1	25	8 6 8 5		
2	SAND and GRAVEL - trace silt, fine to coarse-grained, well-graded, compact, grey to brown, wet (Fill)						2
		Grab	G3				
	GRAVEL - cobbley, subangular, poorly-graded, loose, grey, wet (Fill)						
3	Below 3.0 m - trace sand is present	Split Spoon	S2	5	4 2 3 -		1
4						Driller noted they soil changed from compact to dense at 4.1 m BGS	0
		Grab	G4				
		Split Spoon	S3	7	7 7 8 11		
5	SAND - gravelly, fine to coarse-grained, well-graded, compact, grey, moist (Fill)						-1
		Grab	G5				
6		Split Spoon	S4	10	9 13 7 50		-2
	SAND and GRAVEL - silty, trace organics, fine to medium-grained, well-graded, compact, grey, moist (Fill)						
7	Large wood log (Fill)	Grab	G6			Driller noted that a large wood log was encountered at 7.0 m BGS	-3
	End of test hole at 7.2 m Below Ground Surface (BGS): - Test hole terminated because of large wood log encountered. - Test hole backfilled with drill cuttings and bentonite. Remaining 0.2 m capped with asphalt patch.						
8							



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

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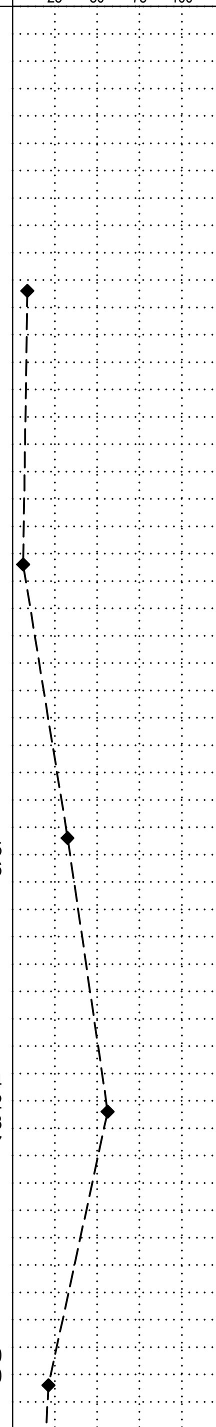
TEST HOLE LOG

TH25-04

PROJECT: Geotechnical Feasibility Study	PROJECT NO.: 0581-1138
CLIENT: District of Oak Bay	METHOD: Sonic
LOCATION: Refer to Test Hole Location Plan	ELEVATION (m): 4
COORDINATES (m): UTM N 5363497 E 477686	CONTRACTOR: Drillwell Enterprises LTD.
COMPLETION DATE: 2025-3-7	LOGGED/REVIEWED BY: ESR/RTM

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	Recovery (%)	SPT Blow Counts	COMMENTS	ELEVATION (m)
0	ASPHALT - 50 mm						4
	SAND and GRAVEL - fine to coarse-grained, well-graded, compact, grey, wet (Fill)						
	CLAY - silty, some organics, stiff, brown, dry (Fill)						
1							3
		GRAB	G1				
		SPLIT SPOON	S1	7	4, 3, 4, 6		
2							2
	SILT - clayey, sandy, firm, brown, damp (Fill)	GRAB	G2				
3		SPLIT SPOON	S2	45	2, 2, 3, 5		1
	Large Boulder Encountered at 3.6 m BGS						
4							0
	SAND and GRAVEL - trace cobbles, medium to coarse-grained, well-graded, compact, grey, damp (Fill)	GRAB	G3				
		SPLIT SPOON	S3	10	9, 11, 15, 26		
5	Below 4.9 m - becomes dense, with some cobbles						-1
		GRAB	G4				
6	Below 6.1 m - becomes wet	SPLIT SPOON	S4	40	14, 22, 23, 17		-2
7		GRAB	G5				-3
		SPLIT SPOON	S5	35	10, 10, 7, 8		
8							

◆ SPT (Standard Pen Test) ◆ (Blows/300mm)



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

TEST HOLE LOG

TH25-04

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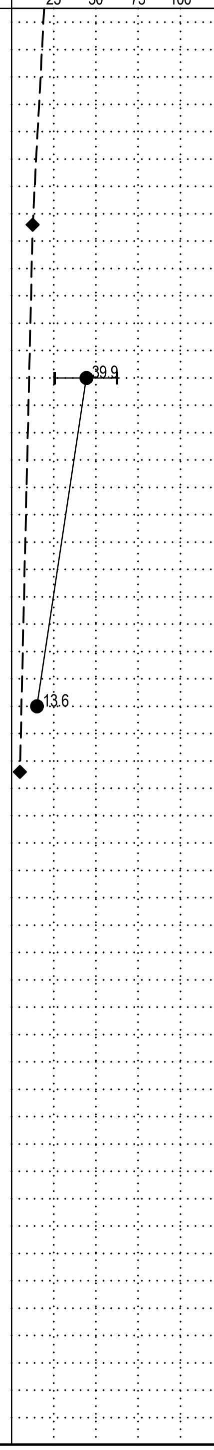
PROJECT: Geotechnical Feasibility Study
PROJECT NO.: 0581-1138
CLIENT: District of Oak Bay
METHOD: Sonic
LOCATION: Refer to Test Hole Location Plan
ELEVATION (m): 4
COORDINATES (m): UTM N 5363497 E 477686
CONTRACTOR: Drillwell Enterprises LTD.
COMPLETION DATE: 2025-3-7
LOGGED/REVIEWED BY: ESR/RTM

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	Recovery (%)	SPT Blow Counts	COMMENTS	ELEVATION (m)
8	Below 7.9 m - becomes compact, brick, construction debris encountered (Fill)						
		GRAB	G6				
9							-5
	CLAY - silty, firm, grey, damp	SPLIT SPOON	S6	25	8 2 8 6		
10	- High plastic	GRAB	G7			Atterberg Test Results at 10.0 mbgs LL: 56 PL: 23 PI: 33	-6
11							-7
	SAND - silty, trace clay, some gravel, fine-grained, well-graded, loose, grey, damp						
		GRAB	G8				
12							-8
		SPLIT SPOON	S7	40	3 3 1 3		
13							-9
		GRAB	G9				
14							-10
		GRAB	G10				
15	End of test hole at 14.9 m Below Ground Surface (BGS): - Test hole terminated on bedrock. - Test hole backfilled with drill cuttings and bentonite. Remaining 0.2 m capped with asphalt patch.						-11
16							

◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 20 40 60 80
PLASTIC M.C. LIQUID 25 50 75 100



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

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TEST HOLE LOG

TH25-05

PROJECT: Geotechnical Feasibility Study
CLIENT: District of Oak Bay
LOCATION: Refer to Test Hole Location Plan
COORDINATES (m): UTM N 5363548 E 477728
COMPLETION DATE: 2025-3-7

PROJECT NO.: 0581-1138
METHOD: Sonic
ELEVATION (m): 4
CONTRACTOR: Drillwell Enterprises LTD.
LOGGED/REVIEWED BY: ESR/RTM

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	Recovery (%)	COMMENTS	ELEVATION (m)
0		ASPHALT - 50 mm					
		SAND and GRAVEL - trace cobbles, fine to coarse-grained, poorly-graded, loose, cemented, grey to brown, damp (Fill)					
1			GRAB	G1			3
2		Large Boulder encountered at 2.3 m BGS					2
3		SAND and GRAVEL - cobbly, medium to coarse-grained, poorly-graded, compact, grey, wet (Fill) Large Boulder encountered at 3.2 m BGS					1
4		SAND and GRAVEL - some cobbles, compact, wet, grey (Fill)	GRAB	G2			0
		GRAVEL - cobbley, some sand, brick, construction debris, subangular, poorly-graded, compact, grey, wet (Fill)					
5							-1
6							-2
7		End of test hole at 6.9 m Below Ground Surface (BGS): - Test hole terminated on bedrock. - Test hole backfilled with drill cuttings and bentonite. Remaining 0.2 m capped with asphalt patch.	GRAB	G3			-3
8							

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

GEOLOGIC LOG SYMBOLS AND ABBREVIATIONS
MODIFIED UNIFIED SOIL CLASSIFICATION SYSTEM

UPDATED SEPT.2024

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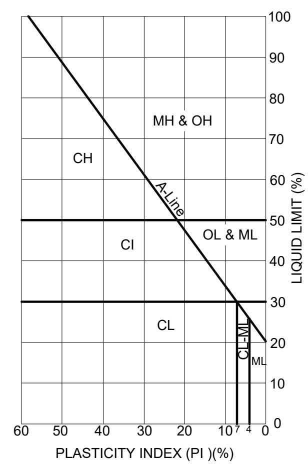
MAJOR DIVISIONS			Symbol	USC	TYPICAL DESCRIPTION	LAB CLASSIFICATION CRITERIA	
COARSE GRAINED SOILS	GRAVELS (MORE THAN HALF COARSE GRAINS LARGER THAN 4.75 mm)	CLEAN GRAVELS (LITTLE TO NO FINES)		GW	WELL GRADED GRAVELS, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 4$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
				GP	POORLY GRADED GRAVELS AND GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	DOES NOT MEET ABOVE REQUIREMENTS	
		GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW 'A' LINE PI LESS THAN 4
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE 'A' LINE PI MORE THAN 7
	SANDS (MORE THAN HALF COARSE GRAINS SMALLER THAN 4.75 mm)	CLEAN SANDS (LITTLE TO NO FINES)		SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 6$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
				SP	POORLY GRADED SANDS, LITTLE OR NO FINES	DOES NOT MEET ABOVE REQUIREMENTS	
		SANDS WITH FINES		SM	SILTY SANDS, SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW 'A' LINE PI LESS THAN 4
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE 'A' LINE PI MORE THAN 7
FINE GRAINED SOILS	SILTS (BELOW 'A' LINE, NEGLIGIBLE ORGANIC CONTENT)	$W_L < 50$		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED ON PLASTICITY CHART 	
		$W_L > 50$		MH	INORGANIC SILTS, MICACEOUS OR DIAMACEOUS FINE SANDY OR SILTY SOILS		
	CLAYS (ABOVE 'A' LINE, NEGLIGIBLE ORGANIC CONTENT)	$W_L < 30$		CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY LEAN CLAYS		
		$30 < W_L < 50$		CI	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS		
		$W_L > 50$		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS		
	ORGANIC SILTS AND CLAYS	$W_L < 50$		OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY		
		$W_L > 50$		OH	ORGANIC CLAYS OF HIGH PLASTICITY		
FILL				FL	SEE REPORT DESCRIPTION		
HIGHLY ORGANIC SOILS				Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOUR OR ODOUR, AND OFTEN FIBROUS TEXTURE	
BEDROCK				BR	SEE REPORT DESCRIPTION		

SPT RESISTANCES (BLOWS/300 mm)		Undrained Shear Strength (S_u) (kPa)	
COHESIONLESS		COHESIVE	
0 - 4	VERY LOOSE	<12	VERY SOFT
4 - 10	LOOSE	12 - 25	SOFT
10 - 30	COMPACT	25 - 50	FIRM
30 - 50	DENSE	50 - 100	STIFF
50 +	VERY DENSE	100 - 200	VERY STIFF
		>200	HARD

DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS	
PERCENT	IDENTIFIER
1 - 10	TRACE
10 - 20	SOME
20 - 35	_____ Y
35 - 50	AND

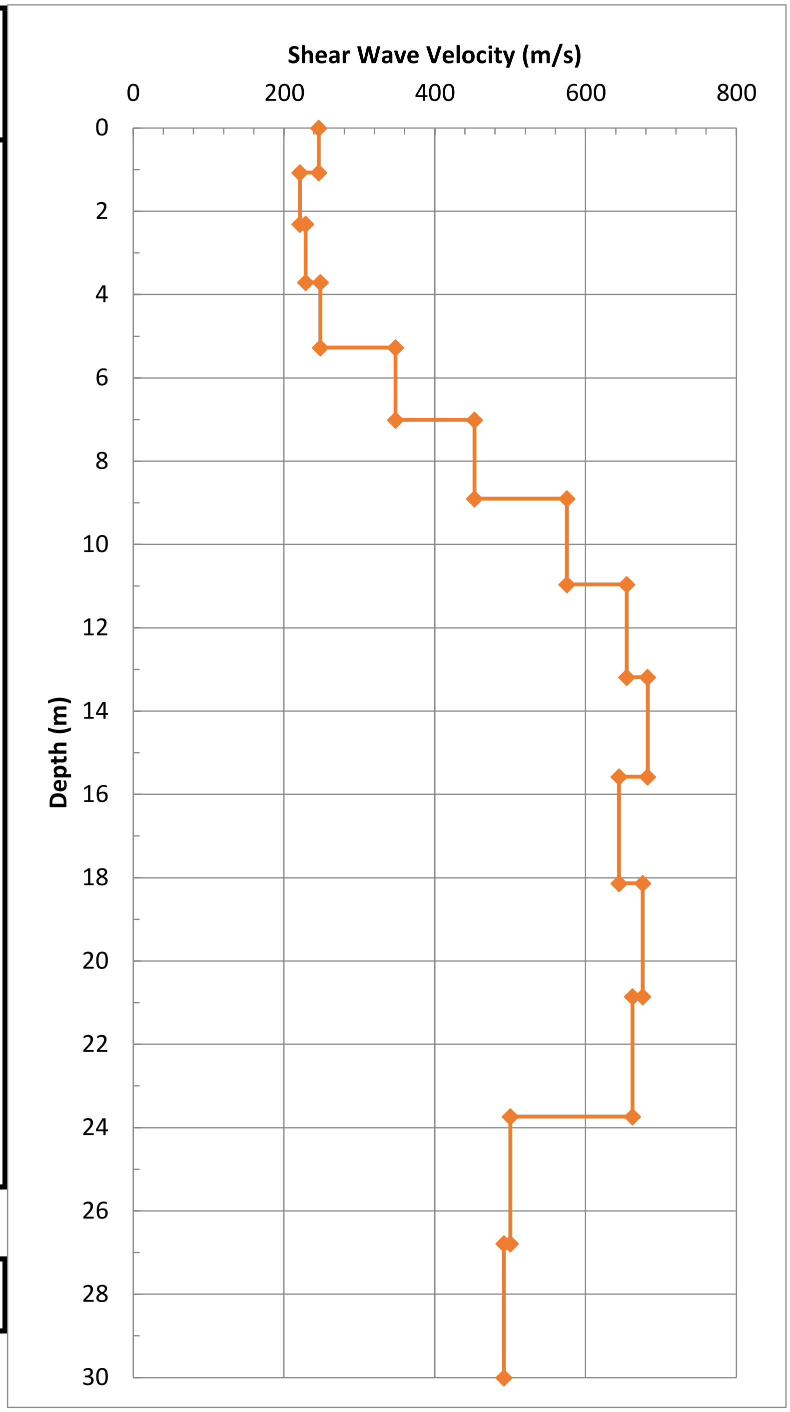
SOIL COMPONENTS (mm)			
BOULDERS		> 200	
COBBLES		75 - 200	
FRACTION		PASSING	RETAINED
GRAVEL	COARSE	75	19
	FINE	19	4.75
SAND	COARSE	4.75	2.00
	MEDIUM	2.00	0.425
	FINE	0.425	0.075
FINE GRAINED SOILS (SILT AND CLAY)		0.075	

Client:	The District of Oak Bay	**Location:**	Refer to Site Plan
Project:	Proposed Geotechnical Study	**Survey ID:**	MASW 25-01
Job No:	0581-1138	**Test Date:**	Jan. 27, 2025

MASW SHEAR WAVE VELOCITY PROFILE

Depth (m)	Shear Wave Velocity, V_s (m/s)
0.0	246
1.1	221
2.3	229
3.7	248
5.3	348
7.0	453
8.9	575
11.0	655
13.2	682
15.6	644
18.1	676
20.9	662
23.7	500
26.8	492
30.0	

Vs30 (m/s)	448



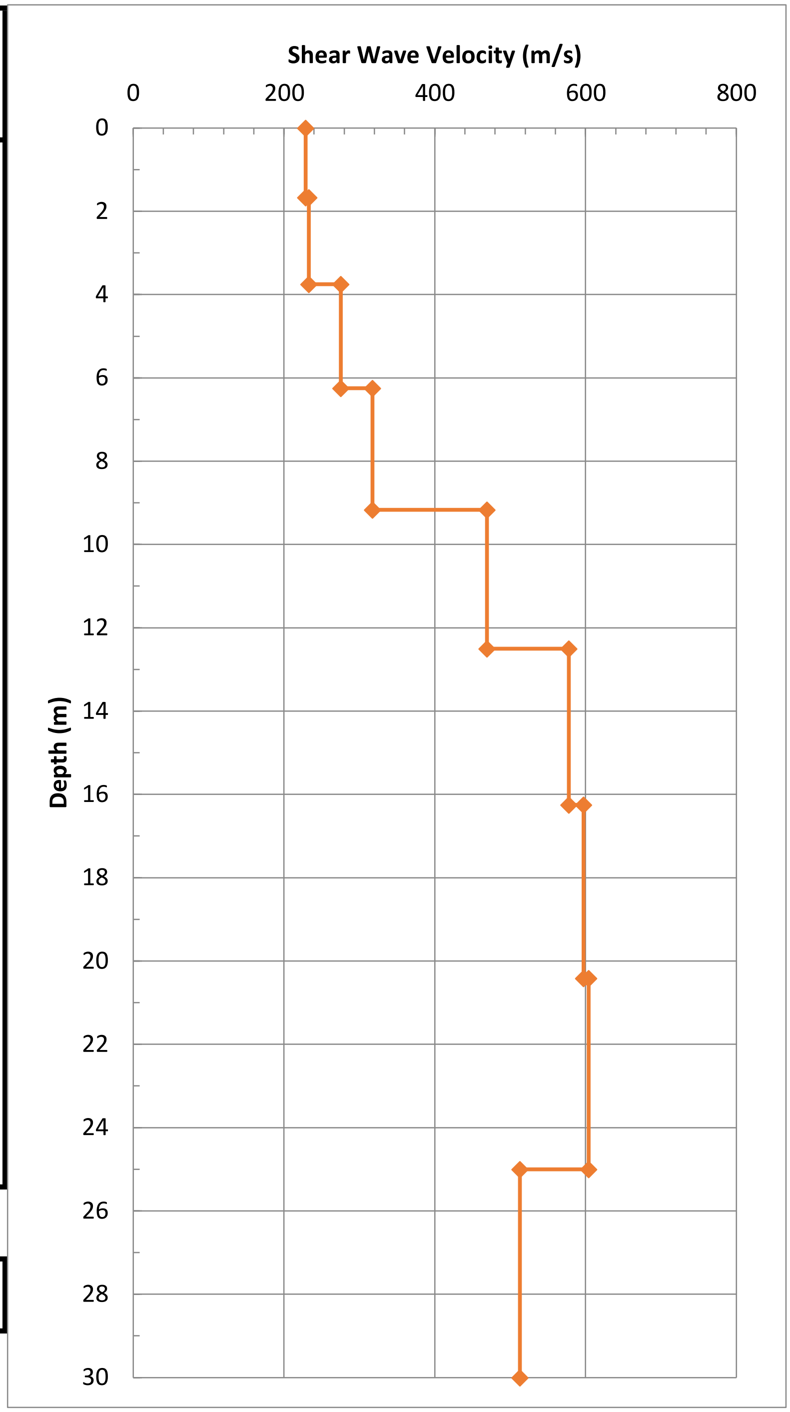
RYZUK
GEOTECHNICAL

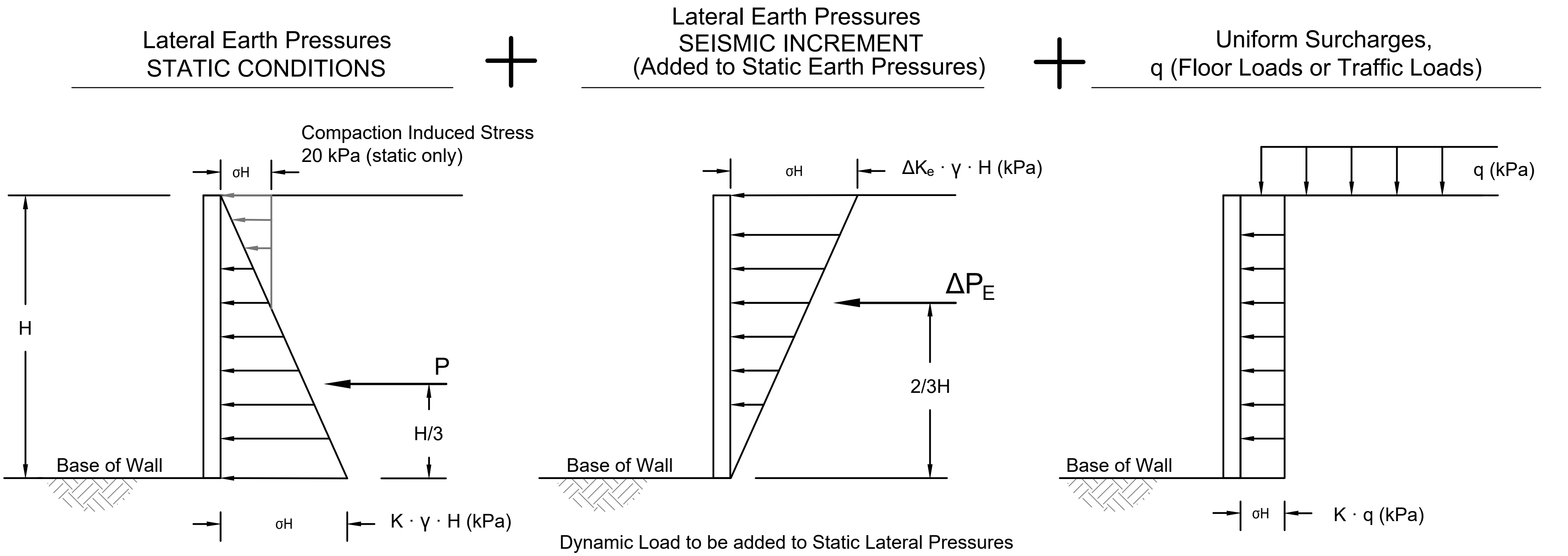
Client:	The District of Oak Bay	**Location:**	Refer to Site Plan
Project:	Proposed Geotechnical Study	**Survey ID:**	MASW 25-02
Job No:	0581-1138	**Test Date:**	Jan. 27, 2025

MASW SHEAR WAVE VELOCITY PROFILE

Depth (m)	Shear Wave Velocity, V_s (m/s)
0.0	229
1.7	233
3.8	275
6.3	317
9.2	469
12.5	578
16.3	597
20.4	604
25.0	513
30.0	604
0.0	0
0.0	0
0.0	0
0.0	0
0.0	

Vs30 (m/s)	414





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)

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

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

UPDATED MAY 2021