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WestUrban Developments Ltd.
111-2036 Island Hwy South
Campbell River, BC
V9W 0E8

Attn: Navi Sandhu BSc, PMP (By E-mail: nsandhu@westurban.ca)

Re: Proposed Residential-Commercial Development
89 & 107 Evans St – Duncan, BC

As requested, we have completed a geotechnical investigation of the referenced site as such relates to the development of the proposed multi-unit residential building. This report summarizes the results of our investigation to aid in the finalization of your purchase of the property and for application for Development Permit. Our work has been undertaken in accordance with our proposal dated February 3, 2021, as well as in accordance with, and is subject to, our attached Terms of Engagement.

This report has been updated to reflect our review of the Issued for Development Permit plans by THUJA Architecture & Design, dated January 24, 2022, topographic survey by McElhanney, March 16, 2012, as well as revisions/clarifications requested in the Application Review from City of Duncan ("the City"), dated February 15, 2022.

PROPOSED DEVELOPMENT

The roughly 0.4 hectare site is located at the northwest corner of the intersection of Evans Street and Canada Avenue. The site is comprised of two lots: 89 Evans Street, which formerly hosted a bank building (now removed), and 107 Evans Street, which is currently occupied by a single-story commercial building. In general, the majority of the site is paved, except for the commercial building along the south property line, and the area east of the building (former location of the bank), which is grassed and hosts mature trees around the south and east perimeters. Topographically, the site is flat with an approximate geodetic elevation of 12-13 m.

Based on our review of the architectural plans, we understand the proposed development is expected to consist of removal of the existing building, and construction of a six-storey building

with a single level of underground parking. The building will consist primarily of commercial and parking space on the ground floor, with residential use above. We anticipate the upper levels would use conventional wood framed construction, while the underground parkade would use conventional reinforced concrete construction with a podium slab at grade.

INVESTIGATION METHODOLOGY

Our investigation consisted of a desktop study and subsurface drilling investigation. The desktop study consisted of a review of our previous projects in the immediate vicinity and on the site, available quaternary and geological mapping, satellite imagery, and topographical mapping.

Our geotechnical report of March 2017 (Ryzuk project number 8065-1), which was provided to you by the previous owner, summarizes the results of our geotechnical investigation within the grassed area of the former bank. This investigation included advancement of two test holes (TH17-01 and TH17-02) using a tracked auger drill rig. The location of the test holes are included in the attached Test Hole Location Plan. Upon completion of drilling, a groundwater monitoring well was installed in TH17-02.

Our recent subsurface investigation was carried out on March 15 and 16, 2021. During this time we advanced two test holes (TH21-01 and TH21-02) using a track mounted sonic drill rig supplied and operated by Drillwell Enterprises Ltd. Test holes were advanced to 19.8 m and 22.9 m, respectively, using a combination of sonic drilling and washing down the casing using a mud slurry where loose soils and heave were encountered. Split Spoon Testing (SPT) was carried out at 1.5 m intervals within the soil stratum. 50 mm downhole casings were installed and grouted into each test hole upon completion for later testing. Bedrock confirmation was carried out using the sonic coring method. The location of the test holes are provided in the Test Hole Location Plan.

Split spoon and grab samples were collected at regular intervals for laboratory testing to confirm field identification. Atterberg limit testing was completed on representative fine-grained samples. Washed sieve testing and determination of fines contents were carried out on representative coarse-grained samples.

While not completed during our time on this site, we have recently measured hammer efficiency of the drill employed on this project using an SPT analyzer. Hammer efficiency was found to vary from 70% to 75 %.

SURFACE AND SUBSURFACE CONDITIONS

In general, soils encountered during our drilling investigation consisted of surficial fills overlying variable loose to compact interbedded sands and gravels in the upper approximately 9-10 m, overlying medium to fine grained loose to compact sand and silt. In TH21-01, the sand/silt was underlain by an approximately 1.5 m thick layer of inferred dense silt, atop inferred

bedrock (possibly very dense glacial till; recovered sample consisted of pulverized material with mudstone gravel inclusions) present below 18.7 m depth. In TH21-02, a 1.4 m thick layer of inferred dense silt was present below approximately 10 m, at the transition between the interbedded sand/gravel and the loose sand/silt layer, which became sandy and gravelly in the bottom 0.5 m of the layer. Bedrock was encountered at 20.6 m depth in TH21-02, and confirmed by recovery of rock core, consisting of mudstone. Based on the core recovered, the upper 1-1.5 m portion of the rock appeared to be heavily fractured.

The groundwater monitoring well installed in TH17-02 indicated a groundwater level to be approximately 4.0 m below the ground surface, which generally coincides with groundwater conditions observed at other nearby sites. This is anticipated to vary seasonally.

GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

On the basis of our investigation and our experience in the area, while we consider the proposed development to be feasible, significant challenges will be encountered with regards to the mitigation of liquefaction related risks.

Seismic Site Classification and Liquefaction Assessment

Given that this report is provided to aid in preliminary design costing and for aid of the design/build team's analysis, liquefaction assessments have been limited in scope. The following sections includes seismic response spectra based on the current BC Building Code as well as a preliminary empirical liquefaction assessment to assess likely problem areas and aid in design as well as proposed mitigation measures based on the associated results. We expect that more realistic results could be obtained through a 1D non-linear site specific assessment or using a 2D dynamic fully coupled finite difference modeling package such as FLAC and employing constitutive models such as UBC Sand or Fynn-Byrne, however, such is beyond this scope of work. To this end, we expect that the result from such an analysis will result in lower PGA values and less liquefaction than anticipated through the simplified approach.

Site Classification

The current BC Building Code classifies sites with liquefiable soils with a Classification for Seismic Site Response (Site Class) of 'F', and requires completion of a site-specific response evaluation. However, the code also includes a clause allowing for this analysis to be omitted provided the structures have a fundamental period of <0.5 seconds. For a four-storey building, we anticipate the fundamental period would be less than 0.5 seconds. We anticipate a six-storey building would have a fundamental period greater than 0.5 seconds. The following site classes have been estimated based on assumed shear wave velocity values while ignoring liquefaction. All classifications provided are in accordance with the current BC Building code.

For preliminary purposes, we recommend that a site classification of 'E' be assumed based on our past experience in the area. We do, however, recommend carrying out a downhole Vs profile in the installed downhole wells and anticipate that this will result in an average shear wave velocity in the 180 and 360 m/s range corresponding to site class 'D'. In any case, if the project goes ahead, a Site Specific Response Assessment would need to be carried out for building periods of >0.5 seconds. This would be recommended regardless as it usually results in significant reductions in ground accelerations for lower building periods.

Information from Natural Resources Canada indicates response spectral accelerations for site class as noted, for a 2% in 50 year probability of exceedance, is summarized for Site Class 'C' in Table 1.

Table 1. Summary of PGA and Spectral Acceleration Values (BCBC 2018)

Period (sec)	0.2	0.5	PGA (g)	PGV (m/s)
Response (g) Site Class 'C'	1.170	1.087	0.512	0.790
Response (g) Site Class 'D'	1.20	1.20	0.45	0.87
Response (g) Site Class 'E'	1.27	1.27	0.38	0.92

Liquefaction Potential

To assess the potential for seismically induced liquefaction of the observed soils, an assessment was carried out using the conventional CSR/CRR approach (cyclical stress ratio to cyclical resistance ratio). These values were estimated through correlations based on the obtained SPT data. To assess liquefaction potential the NCEER (1997) and Idriss & Boulanger (2004) methods were used, while settlements were estimated based on Ishihara & Yoshimine (1992) and Tokimatsu & Seed (1984). SPT depth correction (Cn) was based on Idriss & Boulanger (2004). Industry standard within the southern portion of BC is to base results on Idriss & Boulanger and Tokimatsu & Seed for liquefaction potential and anticipated settlements, respectively. Other methods were used for comparative purposes and to aid in providing an error range on the anticipated conditions. Additionally, for simplification, the mean magnitude of the 2% in 50 year seismic event (7.52 based on the attached deaggregation data) as well as its peak ground accelerations for Site Class 'D' (0.45g) have been used for this analysis. A 5% fines content was assumed where sieve testing was not carried out but will need to be adjusted if/when a more detailed analysis is carried out.

Based on our analyses, the majority of observed soils are anticipated to be liquefiable in the event of an earthquake. Based on NCEER (1997) and Idriss & Boulanger (2004), an N1(60) of approximately 30 is required to prevent liquefaction. This corresponds to an uncorrected field N values of the order of 21, respectively, assuming a hammer energy ratio of 75%. This energy

efficiency is consistent with those recorded on site but higher than those typically assumed when measurements are not made.

The empirical analyses for the soils on site predicted total liquefaction induced settlements to range from 0.25 m to 0.5 m across the site. Given the flat and bounded nature of the area, we do not expect lateral spread to be of concern on this site. Results of our liquefaction results are attached.

Liquefaction Bearing Reduction

To assess the potential for bearing failure and significant seismically induced damage, the Ishihara boundary curves were used based on the Greater Vancouver Taskforce report. Based on the anticipated PGA of between 0.4 g and 0.5 g and for a liquefiable thickness of the order of 8 m to 10 m thick, a dense crust (non-liquefiable zone) immediately below the building foundations of approximately 9 m is required.

Foundations Considerations and Soil Remediation Options

Given the anticipated liquefication induced settlements on this site, mitigative measures will be required to ensure the building can handle the associated differential/total settlements. We consider the following options to be worth considering for this project:

1. In-situ ground densification;
2. Deep foundations.

In situ ground densification consists of the use of external means to densify the loose soils. Common techniques for this process include vibro-compaction/vibro-replacement (vibro-stone columns), dynamic compaction, compaction piles, deep densification by blasting, and compaction grouting. Of these, deep densification blasting is considered impractical and unreliable, while compaction grouting and compaction piles are anticipated to be cost prohibitive. Dynamic compaction may be feasible and should be discussed but should be noted to be extremely disruptive and has difficulty with silty deposits. Vibro-stone columns are also anticipated to be feasible, but may have difficulty for soils with >15% fines (silt) content. These methods are discussed further in the following section.

Deep foundations would also be feasible and could consist of either end bearing driven piles or drilled concrete piers. These transmit the building loads from the surface to a denser stratum below the liquefaction (bedrock in this case). While various pile types are considered feasible, it should be noted that the liquefaction will result in a significant reduction in pile restraint resulting in reduced capacities. Additionally, the induced settlement (down-drag) from liquefaction will induce significant loads on these piles following a seismic event. This means that the piles will have lower design capacities than would typically be expected. We expect

either displacement (closed ended) driven piles ranging from 450 mm to 600 mm diameter would be feasible as would larger diameter (600 mm to 1200 mm) drilled shaft concrete piles.

Foundations Risk/Benefit Assessment

Given the anticipated liquefication induced settlements on this site, mitigative measures will be required. Of the noted options above, we expect these to fall into 2 general categories: densification of the upper 9 m of soil below the proposed foundations or use of deep foundations.

Deep Foundations

We expect that either 450 mm diameter closed ended or 600 mm diameter open ended pipe piles advanced to bedrock would be feasible for this project. Alternatively larger cast-in place concrete piles of bored-shaft concrete infilled caissons could also be considered. These options provide the most reliable option for building design. The design of a pile foundation would also require a network of pile caps with possible grade beams to tie them together as well as either a slab on grade. Alternatively, a raft slab tying the piles together could be considered. Buckling and down-drag considerations are anticipated to reduce pile capacities, while base shear transfer may also be difficult to manage. As such, larger diameter concrete piles may be more cost effective as they can transfer much larger loads and debonding can be used to eliminate seismic down-drag on the piles. If loads can be spaced significantly by the structural engineer, this may prove more cost effective. The use of deep foundations would be the most resilient in that they can be designed to provide a repairable structure following the design earthquake and would behave similarly in smaller seismic events.

For preliminary purposes, we expect that driven pipe piles of 450 mm and 600 mm diameters will be able to provide 800 kN and 1200 kN ULS respectively while concrete piles can be assumed to have bearing capacities based on 5 MPa ULS capacity on sound bedrock which capacity being able to be increased through provision of significant rock sockets.

Soil Densification – Dynamic Compaction

Dynamic compaction involves a crane dropping a large mass from significant elevations to induce densification of the soils below. If dynamic compaction is determined to be feasible, this would occur from the approximate planned foundation elevation to densify soils below. Subsequently the soils would need to be tested to confirm adequate densification (drilling or CPT) and if an adequate crust has been formed, a raft slab could likely be employed.

Soil Densification - Vibro-Stone Columns

Vibro-stone columns have two mechanisms at preventing liquefaction: increasing soil resistance through densification or decreasing the induced cyclical stress through replacement with higher

strength material. The first approach works in soils containing less than 15% silt and less than 2% clay. In cleaner granular soils, the installation of vibro-stone columns will densify the surrounding soils, thereby increasing the resistance to liquefaction. A typical area replacement ratio of 5-15% is generally adequate for the required densification. Where higher fines contents are present, the soils are generally unable to dissipate generated porewater pressure quickly enough to provide adequate densification. In areas of higher silt content, stone columns can help improve the soils by decreasing the anticipated seismic stress through the second approach. Given the observed soils, a combination of the two mechanisms will need to be assumed to prevent liquefaction in different layers of soils on this site.

We anticipate that the stone column length will need to be of the order of 9 m below proposed foundation elements. Additionally, based on the observed fines contents in the upper 10 m of the soils, and based on Baez (1995) an area replacement ratio of 10% is anticipated to be adequate to produce sufficient densification of soils. As with dynamic compaction, subsequently the soils would need to be tested to confirm adequate densification (drilling or CPT) and if an adequate crust has been formed, a raft slab could likely be employed.

Building Foundations

We expect that a raft slab will be the best option for this site where ground improvement is the selected approach. Geotechnical input with respect to design of a raft slab generally involves determination of a design value for the modulus of subgrade reaction (k_s). The subgrade reaction modulus is not a fundamental soil property and is a difficult parameter to estimate accurately. From a geotechnical standpoint, the modulus of subgrade reaction is in large part a function of the in-situ soil density, stiffness of the raft and size of the raft/load influence area. For preliminary purposes, we consider a subgrade reaction modulus of 10 MPa/m to be reasonable.

With the GRS pad, non-seismic Serviceability Limit State (SLS) and Ultimate Limit State (ULS) bearing resistances of 150 kPa/225 kPa respectively can be considered.

Excavation Considerations

Based on available information and the potential FCL considerations, we expect that excavation will be required to depths in the order of 3.0 m below existing site grades for construction of the underground parkade and greater if a raft slab is used and for the required elevator pit. Given the soil conditions, we expect cutslopes above the groundwater table would only be stable in relatively flat cutslope configurations in the order of 0.75 H : 1V (Horizontal : Vertical). If sufficient setbacks from property line are not feasible, shoring (likely consisting of cantilevered soldier piles with shotcrete or timber lagging) would be required.

As previously noted, groundwater was encountered at 4 m below the existing ground surface. Groundwater levels do fluctuate seasonally. Excavation below the groundwater table would

prove to be very challenging and would likely require locally lowering the groundwater table using well points, or the installation of cutoff walls around the perimeter. Further information can be provided once excavation depths are known and subsequent to additional groundwater level monitoring. In any case, it would be recommended to carry out the below grade work during the late summer when groundwater is expected to be at its lowest.

Flood Construction Level & Dewatering

Based on the Cowichan Valley Regional District's Floodplain Mapping (Northwest Hydraulic Consultants, "NHC", dated October 20, 2021), the site is located within the 200 year Cowichan River flood plain. As outlined in the associated NHC memo re: Estimating FCL Values from Flood Maps Rev. 0, the designated Flood Construction Level (FCL) for the site may be calculated based on linear interpolation between flood elevation isolines. Using the calculation methodology outlined by NHC, we estimate the FCL for the site is 13.3 m geodetic (m geo, CGVD2013).

The main floor elevation (MFE) for the proposed development has not been provided to us at this time, however based on recent discussions, we understand such would be roughly located at or near the adjacent sidewalk elevation. We expect this will be approximately 12.3 to 12.5 m geo. As such, both the below grade parkade and the MFE fall below the FCL, as well as the roads and adjacent properties in the surrounding areas. We understand that the main floor would be designated for commercial use, lobby/elevator access, and parking; all residential units would be located above the FCL.

While a site-specific comprehensive assessment of the flood risk has not be undertaken as part of our work, it is worth noting that a series of flood dikes are present along the Cowichan River channel. The suitability/adequacy of these dikes cannot be confirmed, and therefore cannot be relied upon per the Professional Practice Guidelines for Legislated Flood Assessments in BC. However, we understand that the City has adopted / is in the process of adopting a suitable bylaw / land use regulation which will allow new construction within the known flood plain area.

In determining the suitability of the proposed development to ensure the safety of the occupants during the design river flooding event, our assessment considers both the provision and absence of an advanced warning system. River flooding events are generally known to be rapid, with advance notice for evacuation being on the order of hours to minutes. Similarly, flood waters are typically quick to recede. Where advance notice is available, we consider the ability of residences to evacuate the building and the downtown area achievable prior to inundation, noting that higher ground is available within roughly 500 m of the site. However, where advance notice is unavailable or evacuation within the noted timeline is unfeasible, we consider occupants will be imminently safe to remain in the residential building levels located above the FCL. Emergency evacuation could then be completed at a later time via vehicle, boat, or helicopter, depending on actual flood water elevation, if required in the event that the flood waters are slow

to recede. We understand that the City is accepting of the lack of access and egress off the property under this condition, with consideration that the surrounding public roadways and access will also be inundated.

While the City has confirmed that the underground parkade and main floor as proposed to be allowable below the FCL, appropriate mitigation measures should be incorporated into the building design (outlined below). It should be understood that any portion of the structure below the FCL will be below water during a significant flooding event.

We understand that the City will require the following for all non-residential space (as sampled from District of Squamish Bylaw No. 2751, 2021, Section 7.0 – Local Area Exemptions, [*] modified by Ryzuk for the purpose of the proposed development):

- major fixed equipment including major electrical switchgear, furnaces, ventilation systems and hot water tanks that are integral to and necessary for the functioning of a building according to the BC Building Code are located above the FCL (*or suitably tanked);
- the MFE is at least 150 mm above the crown of the road, and 300 mm above the gutter of any adjacent road;
- all elevators to have an automatic shut-off to prevent occupants from inadvertently descending into an inundated area; and
- only 1 level of underground parking is below the finished grade, with clearly marked and accessible exits to the surface*.

As noted, the parkade is expected to flood during the design flooding event, wherein flood waters would enter the parkade from overland flow/over the ramp. However, for seasonally high water table conditions, we recommend the parkade be tanked to mitigate ingress of water. We expect the building is heavy enough to handle hydrostatic uplift pressures, however tanking will likely require significant tie-down anchors for the raft slab. Relief plugs could be considered which will allow the parkade to flood when substantial hydrostatic pressures are experienced; alternatively, foundation walls will require to withstand excess hydrostatic pressure associated with such conditions.

Foundation Wall Backfill

Foundation walls should be backfilled with clean, well graded granular material, compacted in maximum 300 mm lifts to at least 95% of SPMDD value. Where the grade elevation differs significantly between the two sides of a perimeter wall, and the wall is free to rotate in order to develop the active earth pressure state (rotation of 0.1% of the wall height, non-rigid wall), the wall should be designed to resist a lateral earth pressure (due to granular backfill) similar in magnitude and distribution to that of a fluid having a unit weight of 6.3 kN/m^3.

Where the wall cannot rotate (rigid wall), it should be designed to resist an at rest lateral earth pressure loading, similar in magnitude and distribution to that of a fluid having a unit weight of 8.6 kN/m^3. Equipment larger than a skid-steer should not be allowed within 1.5 m of the foundation walls during backfilling.

Lateral earth pressures resulting from seismic activity can be calculated according to the following equations:

Non Rigid Wall : $P_E = 0.375\ k_h\ \gamma\ H^2$

Rigid Wall : $P_E = 0.5\ k_h\ \gamma\ H^2$

where:

- P_E is the resultant force per unit length of wall;
- the coefficients of 0.375 and 0.5 are dimensionless;
- k_h is the design peak horizontal ground acceleration coefficient (see seismic data provided above);
- γ is the moist unit weight of the backfill material, which is approximately 20.4 kN/m^3 for most granular backfill;
- H is the height of the wall

In the case of the non-rigid wall, the backfill pressure distribution resulting from the earthquake loading can be assumed to be triangular, increasing from zero at the base of the wall to a maximum of $0.75\ k_h\ \gamma\ H$ at the top of the wall, with the resultant force acting at 0.67H above the base of the wall.

In the case of the rigid wall, the backfill pressure distribution resulting from the earthquake loading can be assumed to be parabolic, with the resultant force acting at 0.5H above the base of the wall. Additionally, should partial tanking of the parkade be considered, hydrostatic pressures should be considered as well. Associated loading can be provided if required.

For design purposes, the pressure distribution resulting from earthquake loading on the backfill should be added to either the active or at rest pressure distribution depending on whether the noted wall rotation can occur.

Radon Gas Considerations

Following the recent update to the building code, Duncan was added to the list of municipalities with a risk of radon gas. An assessment should be carried out to determine whether radon gas is present in this location. Mitigation of such typically consists of the inclusion of thicker polyethylene vapor barrier (typically a minimum of 10 mils) below the slab and an active venting

system consisting of drain rock. The vapor barrier would, at a minimum, be taped at the seams and the active venting system vented to roof level into the atmosphere.

Pavement Considerations

In preparation of paved areas, all surficial organics and any deleterious fill material must be stripped to expose undisturbed native subgrade upon which the pavement structure can be placed. In areas of light traffic, 50 mm of asphalt over 100 mm of 20 mm minus crushed rock over 150 mm of 75 mm minus crushed rock are recommended.

For heavier traffic areas (garbage truck access, etc.), we suggest 75 mm of asphalt over a minimum of 150 mm of 20 mm minus crushed rock above a further 150 mm of 75 mm minus crushed rock. Alternatively, 300 mm of 20 mm minus could be used provided it is low in fines (<5%) for good water drainage, or a concrete support pad could be considered.

CLOSURE

Based on the above, and provided our recommendations are followed, we consider the development to be feasible from a geotechnical perspective. However, while we have identified the hazard associated with flood water inundation at the site and surrounding areas, we understand the risk associated with residential and commercial development within a known flood plain is acceptable to the City of Duncan. It is our opinion that immediate egress from the building would not be required to ensure the safety of the occupants against the risk of geohazard associated with the design flood event, although we expect such would be feasible where an advance warning system is in place.

In accordance with Section 219 of the Land Title Act, we recommend a covenant be registered on the title such that future owners are aware of the risk for any part of the structure situated below the 13.3 m geo FCL, with the insurers and the City held harmless with respect to flooding that may occur below the FCL.

Based on the preceding, it is our opinion that the land may be used safely for the use intended in accordance with Section 56 of the Community Charter, and Section 219 and 221 of the Land Title Act, and as such relates to the sudden and impactful geohazards referenced.

We trust that the preceding is suitable for your purposes at this time. If we can provide further information or clarification in this regard, please contact us.

Yours very truly,
Ryzuk Geotechnical
PN 1002996

PROFESSIONAL PROVINCE OF 22.04.21 T. L. R. GIERC # 54022 BRITISH COLUMBIA ENGINEER

Teressa Gierc, P.Eng.
Intermediate Engineer

PROFESSIONAL PROVINCE OF APR 20, 2022 C. J. FLANAGAN # 45682 BRITISH COLUMBIA ENGINEER

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PROFESSIONAL PROVINCE OF S. W. MOORE #30954 BRITISH COLUMBIA GEOSCIENTIST

Shane Moore, P.Geo.
Senior Geoscientist

Attachments:
- Test Hole Location Plan
- Test Hole Logs and Laboratory Testing Sheets
- Liquefaction Assessment Results
- NBCC 2015 Seismic Hazard Calculator Printout
- Terms of Engagement