GEOTECH
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JN 21151

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via email: dkoneru@gmail.com

Subject: **Responses to Geotechnical Third-Party Review Comments**
Proposed Short-Plat and Property Redevelopment
6610 East Mercer Way
Mercer Island, Washington

Greetings:

This letter is intended to respond to the comments in the February 16, 2022 letter from Mercer Island's geotechnical third-party reviewer, which are contained within the March 15, 2022 *Request for Information #1* from the City of Mercer Island.

The conditions encountered on the subject site in our explorations, as well as the geotechnical recommendations for the planned development are presented in our June 8, 2021 *Geotechnical Engineering Report* are typical for waterfront residential developments completed previously by our firm. In fact, we have reviewed geotechnical reports prepared in 2018 and 2019 for sites two lots to the north (6454 East Mercer Way) and six lots to the south (6660 East Mercer Way) that found similar loose, liquefiable soil conditions and which recommended only pipe piles for foundation support. These reports, which are available from Mercer Island's GIS, contained little discussion of liquefaction, and made no reference to lateral spreading.

From the February 16, 2022 Mercer Island letter:
The geotechnical engineer of record, Geotech Consultants, Inc. indicates that the alluvial soils have a moderate to high potential for liquefaction under earthquake loading.

Additional information is required regarding the seismic hazards at this site:
1. To what depths will the liquefaction occur?
Response: From previous experience, as well as liquefaction analyses we have conducted previously in similar soils, we know that it at least partial liquefaction beneath the site and surrounding area is possible during the Maximum Considered Earthquake (MCE) with a 1-in-2,475-year probability. This liquefaction could occur between the groundwater table (5- to 7-foot depth) and the dense soils, which were found at an approximate depth of 30 feet. Considering the variability in the gradation of the alluvial soils, it is most likely that liquefaction would occur within the saturated layers of sand and silty sand, which are interbedded with silt, typically thought to have a low potential for liquefaction.

In order to respond to these review comments, we utilized NovoLIQ to confirm that liquefaction of the soil underlying the water table is likely to occur in the low-probability MCE. The results of our liquefaction analyses are attached.

2. What will be the impact of this liquefaction? What magnitude post-liquefaction settlement is estimated? Provide calculations to support estimated settlement.
Response: The evaluation of the potential for liquefaction under a low probability MCE ground shaking has been required by the ASCE7 since at least 2010.

The potential for liquefaction and resulting ground settlement has been studied for many years, but it is still impossible to accurately determine where, and to what extent, liquefaction could/will occur. However, liquefaction of at least the granular soils beneath the site is likely in the MCE. Using two different methods, NovoLIQ estimates that a total of approximately 12.5 inches of ground settlement is possible following widespread liquefaction extending to a depth of 30 feet. The results of this analysis are attached. The amount of actual ground settlement that could occur as a result of liquefaction will vary with differing soil conditions, and the magnitude, length, and predominant direction of ground shaking associated with an earthquake.

3. How is this settlement taken into account in the design of the deep foundations? Provide a calculation of estimated downdrag loads on the piles.
Response: This is a comment that we have previously responded to numerous times in the City of Seattle. Small-diameter pipe piles are not displacement piles, and their compressive capacity is entirely dependent on end bearing in the dense to very dense glacially-compressed soils they are driven into. Tens of thousands of load tests have been completed throughout Seattle and the remainder of the Puget Sound region by our firm and others using ASTM D-1143, or similar testing methods. These load tests have proven that small-diameter pipe piles driven to refusal rates appropriate for the hammer size have an ultimate capacity of 200-percent, or more, of the typical design allowable capacities, such as those we have recommended in our *Geotechnical Engineering Study*.

The potentially liquefiable soils encountered in the borings below the water table will provide no vertical support to the pipe piles in the event of seismic liquefaction. For a 6-inch-diameter pipe with a 15-ton allowable capacity, an ultimate capacity in excess of 30 tons is achievable in static conditions. Conservatively assuming a skin friction of 300 psf on the pile in the upper approximately 7 feet of non-liquefiable soils, a downdrag load of 3,300 pounds could be applied to the pile. This would allow a residual ultimate compressive capacity of at least 56,700 pounds (28.4 tons). For this short-term loading condition, that would still provide a safety factor in excess of 1.8, which is acceptable for a full-scale seismic event.

As a part of our work for the study on this property, we have reviewed recent geotechnical reports prepared for developments of waterfront lots to the north (#6454) and south (#6660) of the site. These reports, prepared by Earth Solutions and Associated Earth Sciences are available on the Mercer Island GIS. Both reports similarly recommend the use of pipe piles driven into dense soils to support the homes. One report concluded that liquefaction of the loose, saturated soils was unlikely, which we disagree with.

4. Provide stability analyses of potential flow failure or lateral spreading at the site due to seismic loading and/or liquefaction. Show cross section of stability analyses with results, soil stratigraphy, soil properties, etc.
Response: The potential for lateral spreading is even less understood than liquefaction itself. However, some methods have been developed to estimate the potential amount of lateral ground movement that could occur where liquefiable sites lie next to sloping free face conditions, such as the sloped bottom of Lake Washington. NovoLIQ provides estimates for this lateral movement using five different methods. The results, which are attached, indicate that lateral ground movement of 5 to 10 feet could theoretically occur in the MCE. Having completed similar

computations before by hand, we know that large values such as this are common for lakefront projects with more that a few feet of liquefiable soil beneath them.

Unfortunately, there is no accurate method for determining where, and to what extent, lateral spreading could occur. Even more involved methods, such as Finite Element Analyses, are approximate at best, as they rely on a multitude of assumptions about soil properties and potential ground motions from earthquakes.

5. How is this flow failure and/or lateral spreading incorporated into the site development? Provide calculations of estimated deformations. Will the proposed pipe piles have sufficient structural integrity to preclude a slenderness ratio issue or lateral failure under these seismic conditions?
Response: Based on the available information, significant lateral ground movement could occur during the MCE. The risk of this is no higher than on nearby waterfront properties that are underlain by similar loose soils and which have recently been developed with new homes. The theoretical lateral movements are large enough that no pile system, drilled or driven, can prevent them from occurring, or can withstand the potential lateral movements without shearing off.

When the issue of lateral spreading was first brought up in the Code years ago, we met with the geotechnical engineering department of Seattle Department of Construction and Inspections (SDCI) to discuss potential mitigation measures for this hazard. The appropriate mitigation against foundation collapse in the event of lateral spreading was determined to be achieved by the reinforced grade beams or mat slab that interconnects the piles. In the event that the ground moves sideways a sufficient distance to bend or break the piles, the grade beams/mat slab would serve to hold the structure in one piece, even if it tilts a significant amount. This approach is still the underlying mitigation for foundation collapse contained in our *Geotechnical Engineering Study*.

6. What soil improvement techniques are recommended to reduce the potential for liquefaction or to mitigate the impacts of flow failure or lateral spreading at this site? If soil improvement techniques or mitigation measures are not recommended, provide a discussion as to why they are not being considered.
Response: Ground improvement to prevent liquefaction and/or lateral spreading is both infeasible and inappropriate for a waterfront residential site such as this one, for a variety of reasons:

1. Attempting to "improve" the resistance of the granular soils to liquefaction using stone columns or a similar method would involve strong ground vibrations, which would cause ground settlement and likely damage to neighboring properties, structures, and utilities.
2. The high fines content of the alluvial soils, some of which are mostly silt, make the use of ground improvement to reduce the potential for liquefaction infeasible. The density of these fine-grained soils cannot be increased by vibratory or replacement methods. This has been confirmed by our previous discussions with ground improvement designers on other projects underlain by fine-grained soils. The use of other methods, such as deep soil mixing, would provide no reduction of liquefaction and potential lateral spreading in the loose soil below the water table.
3. No localized ground improvement system on an isolated residential lot can resist the significant lateral soil loads that would result from liquefaction and lateral spreading of the upper 30 of soil affecting both the site and adjacent properties. It would be necessary to prevent liquefaction and lateral spreading in the loose soils extending far onto neighboring properties to the north, south, and west to prevent lateral movement within the house footprint on the subject site. This is not practical.

Please contact us if you have any questions regarding this letter, or if we can be of further assistance.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

04/12/2022

Marc R. McGinnis, P.E.
Principal

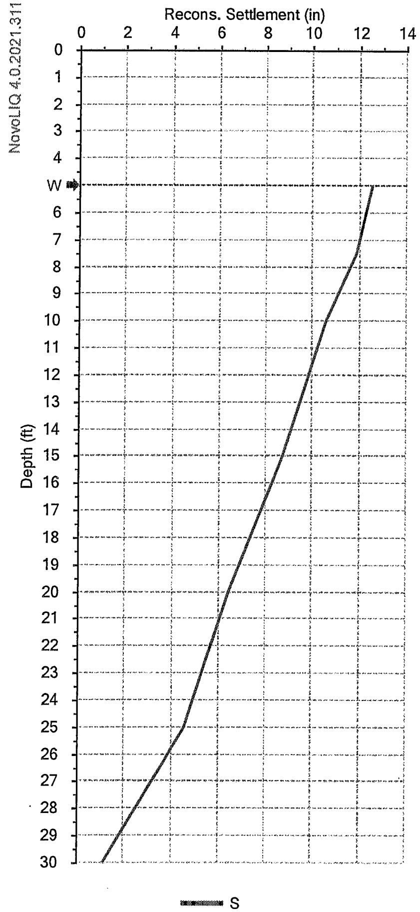
Attachments – NovoLIQ Output

cc: **JMK Homes** – Jed Murphey
via email: jed@jmkhomes.net

MRM:kg

Depth (ft)	Rd	Rd_I&B	Overburden Stress (ksf)		Fines Content (%)	SPT Test				Relative Density Dr (%)	Simplified CSR	CSR_I&B	CRR7.5						
			Total	Effective		N	Co	Cn	N1(60)				NCEER Worksh	Boulanger &	Vancouver Task	Cetin et al.	Chinese Code	Seed et al.	Japanese
2.5	1	1	0.29	0.29	80	2	0.75	1.7	3	41.9	0.443	0.443	-	-	-	-	-	-	-
5	0.993	0.993	0.59	0.59	80	1	0.75	1.68	1	37.6	0.44	0.44	0.05	0.1	0.05	1.92	0.08	0.15	0.52
7.5	0.984	0.984	0.88	0.72	80	3	0.75	1.56	4	44.8	0.53	0.53	0.07	0.12	0.07	1.92	0.11	0.17	0.56
10	0.975	0.975	1.17	0.86	5	2	0.79	1.47	2	22.5	0.589	0.589	0.05	0.08	0.05	1.92	0.16	0.16	0.07
15	0.956	0.956	1.76	1.14	5	8	0.9	1.32	10	45.5	0.656	0.656	0.1	0.12	0.1	1.92	0.12	0.17	0.17
20	0.933	0.933	2.35	1.41	5	3	0.93	1.2	3	27	0.689	0.689	0.06	0.08	0.06	1.92	0.11	0.15	0.08
25	0.91	0.91	2.93	1.68	15	3	0.96	1.11	3	35.6	0.702	0.702	0.06	0.09	0.06	1.92	0.08	0.15	0.11
30	0.884	0.884	3.52	1.96	15	36	0.97	1.02	36	93.1	0.704	0.704	0.8	0.8	0.8	0.53	0.8	0.8	0.35

Tokimatsu and Yoshimi	Shibata (1981)	Kokusho et al. (1983)	CRR7.5 (ave)	Safety Factor										Safety Factor	Probability of Liquefaction PL(%)	
				NCEER Workshop	Boulanger & Idriss	Vancouver Task	Cetin et al. (2004)	Chinese Code	Seed et al. (1983)	Japanese Highway	Tokimatsu and Yoshimi	Shibata (1981)	Kokusho et al. (1983)		Youd & Noble	Cetin et al. 2004
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
0.15	0.21	0.16	0.34	0.13	0.27	0.13	3	0.21	0.4	1.37	0.39	0.55	0.43	0.69	98.9	100
0.17	0.22	0.18	0.36	0.16	0.26	0.16	3	0.24	0.37	1.21	0.36	0.47	0.39	0.66	97.4	100
0.12	0.21	0.14	0.3	0.1	0.15	0.1	3	0.32	0.3	0.13	0.24	0.41	0.28	0.5	98.2	100
0.17	0.22	0.18	0.33	0.18	0.21	0.18	3	0.2	0.3	0.3	0.3	0.38	0.32	0.54	92.2	100
0.13	0.21	0.15	0.29	0.09	0.13	0.09	3	0.19	0.25	0.13	0.21	0.35	0.25	0.47	97.6	100
0.14	0.21	0.16	0.3	0.1	0.15	0.1	3	0.12	0.24	0.18	0.24	0.34	0.26	0.47	97.4	100
0.8	0.8	0.56	0.7	1.31	1.31	1.31	0.86	1.31	1.31	0.57	1.31	1.31	0.91	1.15	1.5	0



NovoLIQ 4.0.2021.311
Recons. Settlement (in)
0
2
4
6
8
10
12
14
Depth (ft)
0
1
2
3
4
W
6
7
8
9
10
11
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13
14
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Type	Method	Movement (inch)
Lateral Spreading	Zhang & Robertson, 2004	130
	Faris, 2006	137
	Youd et al., 2002	17
	Barlett & Youd, 1992	55
	Hamada et al., 1986	66
	Youd & Perkins, 1987	LSI ~41 see details for LSI=50
Vertical Settlement	Ishihara & Yoshimine, 1992	13