GEOTECHNICAL SEISMIC ASSESSMENT OF A PILE SUPPORTED WHARF AND IMPLICATIONS FOR FUTURE UPGRADES

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ABSTRACT: The existing South Jetty wharf structures at the Esquimalt Graving Dock were evaluated with respect to current seismic performance requirements and a possible future upgrade design was developed. These facilities, located in one of the highest seismic hazard regions in Western Canada, were originally constructed in the 1940s. The ground conditions at site consist of sloping, interbedded fills overlying the Victoria Clay, which overlies bedrock. The liquefaction potential of these materials, as well as potential liquefaction induced displacements, are the key issues affecting the seismic performance of the existing jetty, and any future upgrades. Site investigation and advanced cyclic simple shear testing were carried out to assess the liquefaction potential of fills and strain softening potential of Victoria Clay. Liquefaction assessment showed that the sloping interbedded fills would liquefy under the design earthquake, however the Victoria Clay would not trigger liquefaction. Deformation analyses showed that the fills would undergo very large displacements, and the existing structure would not meet the current seismic performance criteria. The foundations for the new structures were designed to withstand the effects of liquefaction induced displacements. This paper presents a summary of the seismic assessment and the preliminary design of the redevelopment of the jetty.

1. Introduction and Historic Design

The Esquimalt Graving Dock (EGD) is located in Skinner Cove, Esquimalt Harbour, BC, as shown in Figure 1, and was constructed between 1921 and 1927. The jetties of the EGD were initially built in the 1940s, and have since been upgraded periodically. The site is located in one of the highest seismic hazard regions in Western Canada, close to the Cascadia subduction zone. The ground conditions at site consist of sloping coarse-grained fill, interbedded with soft fine-grained fill, which overlays the Victoria Clay, and bedrock. All soil units vary in thickness across the site.

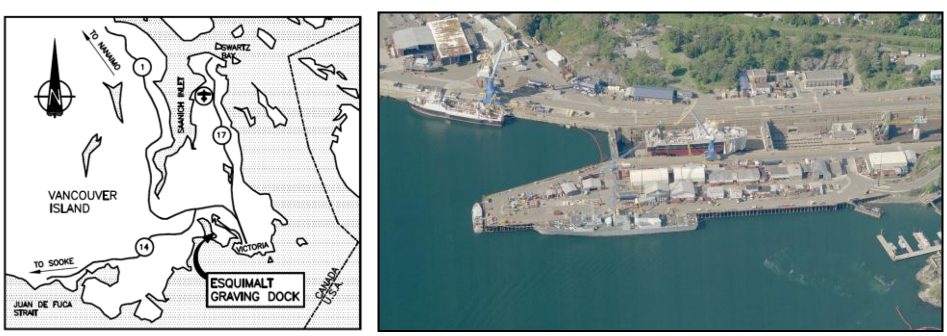

Fig. 1 – Key Plan and Aerial View of Site

The existing wharf infrastructure located on the south side of the graving dock consists of a number of structures, including:

- A concrete-faced timber crib structure, constructed in 1921.
- Two timber jetty structures (South Timber Jetty and West Timber Jetty) supported on timber piles driven into the harbor sediments, constructed in 1940.
- An anchored sheet pile bulkhead wall, constructed in 1985.
- Three segments of concrete jetty supported on steel piles, denoted West, Centre and East, all constructed in 1985.

These features are shown in schematic plan in Figure 2.

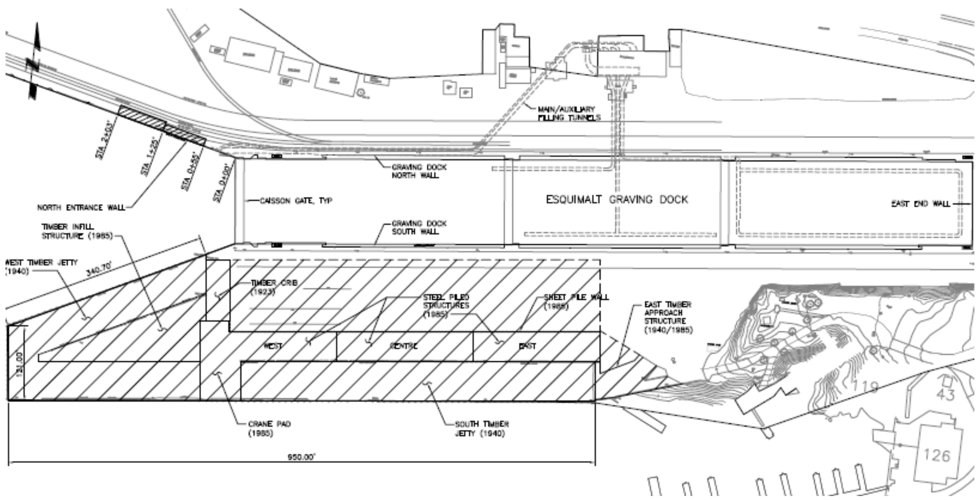

Fig. 2 – Schematic Plan of Esquimalt Graving Dock

A proposed redevelopment of the wharf structures south of the graving dock is currently being assessed, and a fully detailed design for the redevelopment has been developed. The redevelopment study included a review of the potential for dredging of the marine sediments, construction of a new jetty structure to replace the existing South Timber Jetty, and upgrading of other elements of the existing structure to withstand seismic loading.

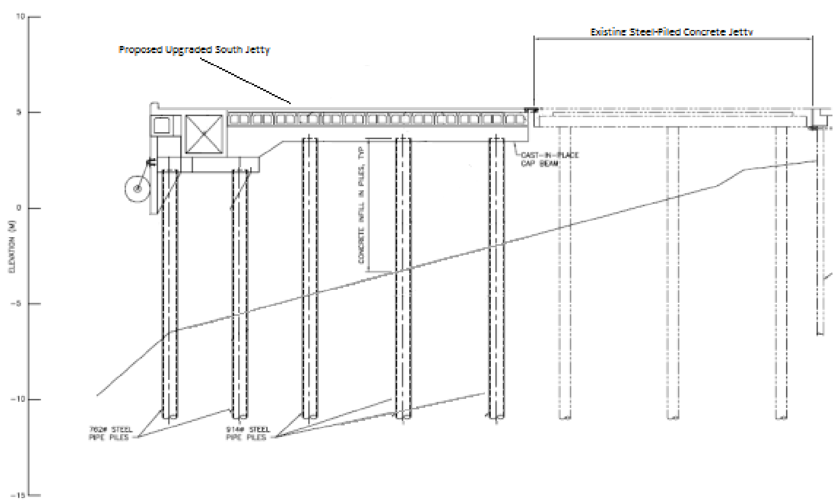

Fig. 3 – Schematic Section of Esquimalt Graving Dock South Jetty Showing Existing Concrete Jetty and Proposed Upgraded South Jetty (Replacing Existing South Timber Jetty)

2. Ground Conditions

Site investigations were carried out at the EGD between 2002 and 2010, using various techniques, including mud rotary drilling, Becker Penetration Tests (BPT) with energy measurements, Nilcon field vane tests, Cone Penetration Tests (CPT), Standard Penetration Tests (SPT), and HQ diamond coring (KC, 2002 / KCB, 2010). Laboratory testing, including Atterberg limit, moisture content, cyclic simple shear, consolidated undrained triaxial, and one-dimensional consolidation testing were carried out on soil samples, and unconfined compressive strength testing was carried out on bedrock samples.

2.1. Soil Profile

The ground conditions at site consist of sloping coarse-grained fill, interbedded with soft fine-grained fill, which overlay the Victoria Clay, which overlays bedrock. A brief summary of each geotechnical unit follows.

2.1.1. Upper Coarse-Grained Sediments

Granular materials consisting of gravel and sand are encountered as the topmost unit beneath the South Jetty. The granular material contains various amounts of white shells and occasional timber pieces. Some portion of the upper coarse-grained sediments may be remnant fill materials from the perimeter dyke built during original excavation and construction of the graving dock in the 1920's.

2.1.2. Upper Fine-Grained Sediments

Silt and clay are encountered below the upper coarse-grained sediments. The silt is generally low plastic, soft, with some fine-grained sand, and a light organic and chemical odour. The clay is generally very soft, with medium to high plasticity. At some locations, the upper coarse-grained sediments were also found underlying the silt and clay, indicating that the upper fine-grained sediments may have been placed as fill.

2.1.3. Victoria Clay

The Victoria clay unit consists of grey, low to medium plastic, soft to increasingly stiff clay, with some silt, and trace sand and gravel. Cone penetration tests (CPT) show an approximately 2 m thick layer of stiff silty clay 'weathered crust' overlying the unweathered Victoria clay, which has a firm to stiff consistency.

2.1.4. Till-Like Deposit

Below the Victoria clay, and overlying the bedrock, is a discontinuous layer of till-like deposit consisting of dense to very dense, well graded sand and gravel, with some silt.

2.1.5. Bedrock

The bedrock at the site consists of strong to very strong greenish-grey meta-diorite of the Wark Gneiss complex. The bedrock surface is highly variable, with rapid changes in elevation over very short distances. This erratic bedrock surface was a key factor to consider when determining assumed pile design lengths.

2.2. Significance of Soil Conditions

The liquefaction potential of the native and fill materials, as well as potential liquefaction induced displacements, are the key issues affecting the seismic performance of the structures, and any possible future upgrades. If liquefaction were to occur, it would likely lead to instability of the underwater slopes, potentially leading to a flow slide of the slope material, and lateral spreading in relatively flat areas. Strain softening of the clays during the cyclic loading is also a concern, as is the soil-pile interaction during a seismic event. The primary soil-structure concerns at the EGD South Jetty, during a seismic event, are considered to be:

- Displacement type sliding failure of the timber crib structure.
- Failure of the anchored sheet pile bulkhead wall.
- Pile-to-deck connection failures, potentially resulting in collapse of some sections, at the existing steel piled concrete jetty segments.

- Collapse of the timber wharf structures as a result of soil slope failure movements.

3. Seismic Setting

The jetty is located in one of the highest seismic hazard regions in Western Canada, close to the Cascadia subduction zone. The design earthquake at EGD is the seismic event that produces ground motions associated with a 475-year return period (A475), or the Cascadia Subduction event (CSE).

3.1. Seismic Ground Motions

In 1999, Pacific Geodynamics (PGI) produced three synthetic earthquake records and eight natural earthquake records for use on the project. These consisted of eight records (1 synthetic, 7 natural) for the 475-year return period event, one record (synthetic) for the 1,000-year return period event, and two records (1 synthetic, 1 natural) for the subduction event. The 1,000-year return period record was not used in this analysis.

The eight 475-year return period earthquake records, and the two subduction earthquake records, were each scaled to a uniform hazard response spectrum (UHRS) for an earthquake with a 10% probability of exceedance in 50 years, which is numerically equivalent to a 475-year return period. The natural 475-year return period earthquake records were uniformly scaled to a PGV of 0.35 m/s, and the natural subduction record was uniformly scaled to a PGA of 0.24 g, The synthetic records were spectrum matched to the UHRS.

Following the development of the records used in this analysis the seismic hazard level at EGD changed slightly due to refinements in the Geological Survey of Canada (GSC) seismic hazard model (Adams & Halchuk, 2003) that were incorporated into the 2005 Edition of the National Building Code of Canada (NRCC, 2005), and subsequently updated in the 2010 Edition of the National Building Code of Canada (NRCC, 2010). The UHRS based on PGI's 1999 work is slightly greater than the NBCC UHRS, as shown in Table 1. Based on this. the records generated by PGI were considered to still be applicable, and were used in the study presented in this report.

Table 1 – Uniform Hazard Response Spectra, 475 Year Return Period

Period (seconds)	NBCC 2010	PGI 1999
PGA	0.33	0.35
0.2	0.66	0.70
0.5	0.44	0.49
1.0	0.20	0.22
2.0	0.09	0.10

3.2. Site Response Analysis

Site response analyses were completed at several locations, including: at the east end of the existing South Timber Jetty, near the middle of the existing South Timber Jetty, and near the dry dock entrance. As well, a separate site response analysis was completed for the existing West Timber Jetty, where the depth of Victoria Clay is significantly deeper.

Site response analyses were carried out for both the 475-year return period and the Cascadia subduction earthquake. A model 1-dimensional soil profile was created using the software package ProShake (EduPro, 1999), and each of the synthetic and natural records were input at the base of the model. Example plots of peak ground acceleration (PGA) and cyclic stress ratio (CSR) with depth for the eight scaled input time histories are shown in Figure 4 for the 475-year return period analysis.

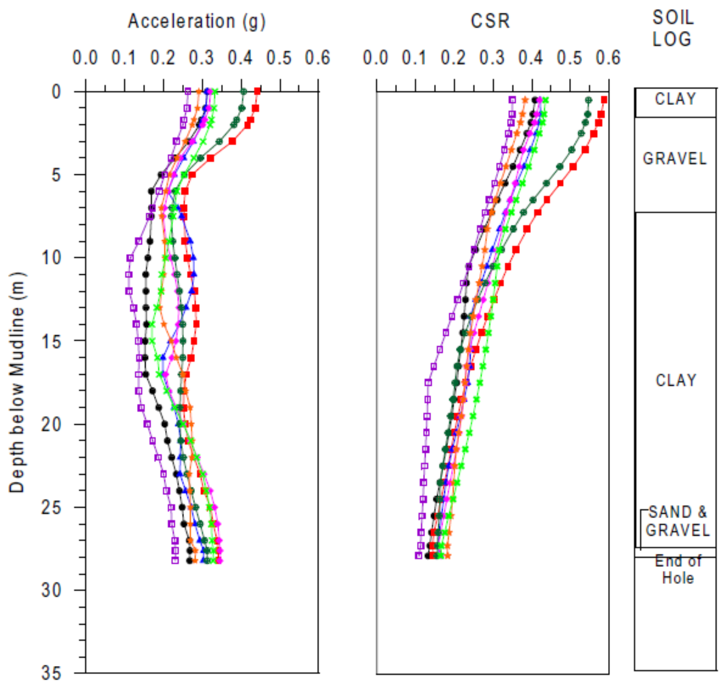

Fig. 4 – Peak Acceleration and Cyclic Stress Ratio with Depth for 475-Year Return Period

These results indicate that de-amplification generally occurs in the thick soft native silt and clay deposit overlying bedrock, and amplification generally occurs in the loose granular fill overlying the silt and clay. In particular, the results show significant de-amplification occurring near the west end of the existing West Timber Jetty, where the thickness of the silt and clay deposits are greatest. It should be noted that a similar analysis was also performed utilizing the two scaled input time histories from the subduction event, however the 475-year design event was found to govern the design.

4. Liquefaction Assessment

4.1. Liquefaction Triggering Analysis

A liquefaction assessment was not conducted as part of the 2010 seismic design work. In 2002, a seismic liquefaction assessment of the saturated granular soils was carried out in general accordance with the Seed approach as recommended in Youd et al. (2001). The liquefaction assessment was considered appropriate for “free field” condition, and neglected soil-structure interaction effects of the wharf. In the Seed liquefaction approach, the cyclic shear stress ratios induced by the design earthquakes are compared to the cyclic resistance ratios derived from field observations of soils with known standard penetration test (SPT) N values from sites which have, or have not, liquefied during earthquakes. The induced cyclic shear stresses were computed directly from the software package ProShake (EduPro, 1999). The cyclic shear resistances of the granular soils were estimated from the BPTs, following the method proposed by Sy (1993) for converting BPT blow counts to equivalent SPT N60 values, and from the measured cone tip resistance (Qt) obtained by CPTs.

The 2002 liquefaction assessment indicated that significant portions of the granular fill would likely liquefy under the design earthquakes. The current PGA and CSR profiles with depth have not changed significantly since 2002. Therefore, it is assumed that significant portions of the granular fill will liquefy.

A typical liquefaction assessment, based on one BPT, is presented in Figure 5.

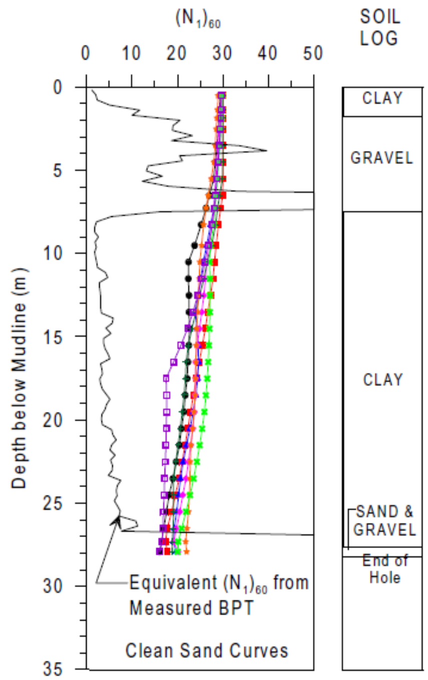

Fig. 5 – Typical Liquefaction Assessment for Granular Fills from BPT

Figure 5 shows, for the design seismic event, the equivalent SPT N60 values from the BPT plotted against the required N60 values for a factor of safety against liquefaction of 1.0. The required N60 values derived from the Seed liquefaction chart (Youd et al. 2001) are for clean sand (i.e. fines content of 5%). Liquefaction is expected to occur during the design earthquakes where the equivalent N60 values estimated from the BPTs are less than the N60 values required to prevent liquefaction. As shown, significant liquefaction of the saturated loose layers in the granular fills will occur for the design seismic event.

4.2. Strain Softening of Victoria Clay

Cyclic simple shear testing was conducted on four samples of Victoria Clay. Samples were initially consolidated to the approximate in-situ vertical effective stress, in several steps, and allowed to consolidate for about 24 hours prior to shearing. Cyclic shear loading was applied at a frequency of 0.5 Hz with a starting CSR of 0.15. Cyclic loading was continued at the starting CSR until a maximum shear strain of 3.75%, or until 15 stress cycles, whichever occurred first. If the maximum shear strain was less than 3.75%, the CSR was increased by 0.05 and testing continued.

Post-cyclic undrained monotonic loading commenced immediately upon completion of the above cyclic loading phase. Monotonic loading was carried out at a strain rate of 1.0% per hour, to a maximum of 20% strain. Monotonic loading was carried out in the opposite direction to that of the residual strain at the conclusion of the cyclic loading.

Based on the results of this testing, it is expected that the Victoria Clay will not undergo liquefaction, however strain softening will likely occur.

4.3. Seismic Induced Displacements

Liquefaction of the granular fills at the EGD South Jetty will induce permanent lateral displacements and ground settlements, which could adversely affect the stability and performance of the existing supporting structures, including the timber crib, the steel and timber pile foundations, the steel sheet pile wall, and the proposed upgrades. It is expected that liquefaction of the granular fills will result in the following ground failures or deformations:

- Flow slides, i.e., lateral ground movements in the range of several metres, are expected to occur in the relatively steep ground slopes underlying the south timber jetty and concrete wharf. The flow slide movements are not dependent on the direction of earthquake impulses.
- In the filled ground north of the anchored sheet pile bulkhead wall and east of the timber crib, and in the east approach area, liquefaction will result in lateral spreading of the order of 0.5 m to 2.0 m, and liquefaction-induced settlements of 0.2 m to 0.5 m. These estimates of ground deformations were partly based on the empirical equations by Youd et al. (2002) and Tomikatsu and Seed (1987).

To determine the magnitude of the seismic induced displacements which would occur during the flow slide, a FLAC analysis (Itasca, 1999) was performed for a section located near the middle of the south jetty. The FLAC analysis indicated that seismic induced displacements in the range of perhaps 3 m to 5 m are expected in the liquefied granular fill. Seismic induced softening, but no significant movements, are expected in the underlying Victoria Clay. An example FLAC section is presented in Figure 6.

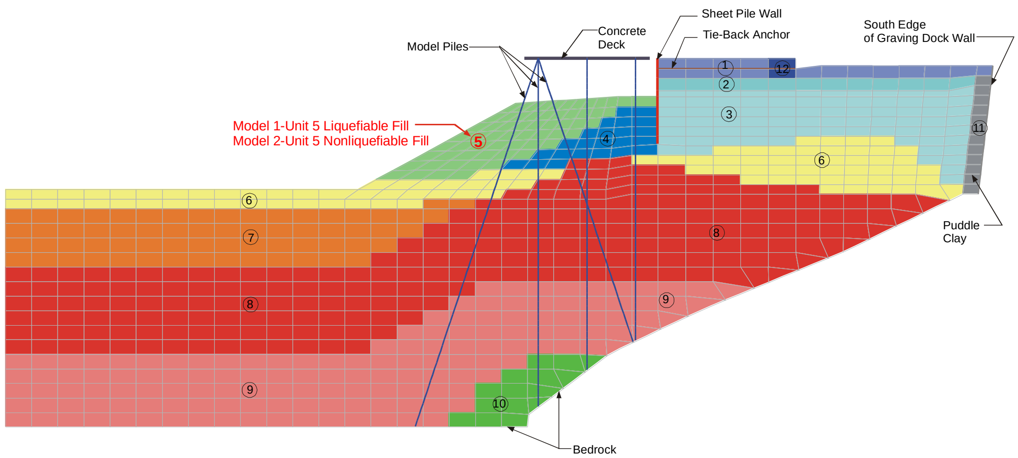

Fig. 6 – Example FLAC Cross Section

A number of limit equilibrium analyses were conducted using the software package Slope-W (Geo-Studio, 2007) to assess the extent of the 'area-of-concern' with respect to the occurrence of flow-slides, post-liquefaction. Limit equilibrium analyses were conducted at 10 cross sections, assuming post-liquefied strengths of the granular fills. The limit equilibrium results for all sections indicated that the post-earthquake factor of safety (FOS) is <1.0. This result suggests that large displacements, or a flow slide failure, will occur in the granular fill across the full width of the South Jetty.

Newmark type seismic displacement analyses were also performed in order to estimate the amount of seismic induced displacement that might occur in the non-liquefied Victoria Clay. The general conclusion of this analysis is that while the displacements in the liquefied granular fill will be large, no significant displacements of the Victoria Clay are expected.

As well, a FLAC analysis was carried out for a section located through the existing timber crib structure, with the soils in front, behind, and below the timber crib expected to undergo liquefaction. The FLAC analyses showed that the timber crib would undergo very large displacements, on the order of 5 m of displacement horizontally and 1 m of displacement vertically.

5. Soil-Structure Interaction

5.1. Liquefaction Induced Pile-Soil Interaction

An analysis was conducted to evaluate the effects of the flow slide movements on the piles at the EGD South Jetty. The objective of the analysis was to determine whether:

- The piles would track with the soil movement; and

- The piles would reach their ultimate moment capacity.

Lateral pile analyses were conducted using the computer program LPile Plus (Ensoft, 2007) for various expected soil conditions. As noted previously, large seismic induced displacements are expected in the fill, which consists of layers of liquefiable granular material and non-liquefiable granular material (Upper Coarse Grained Sediments unit), as well as non-liquefiable silt (Upper Fine Grained Sediments unit). Based on the FLAC results, movement of the fill was assumed to be between 4 m and 5 m.

For the purposes of analysis, the pile was modeled as a linear elastic material and the soil-pile interaction was modeled using a set of soil-pile springs attached to the pile along its length; the springs were modeled using p-y curves. The API (2011) recommended p-y curves were used for the non-liquefiable soils. For liquefiable soils, p-y curve recommendations by API (2011) for soft clay were utilized with the post-liquefaction residual strength of the liquefied soils selected as the ultimate strength.

At each design section along the south jetty, a representative pile was analysed at the north and south edge of the proposed footprint of the jetty. For the northern pile of each section a pinned head condition was assumed, with a limiting moment based on the structural capacity of the connection between the pile and the pile supported deck. For the southern pile of each section, a fixed head condition is more appropriate and was assumed in the analyses.

At each site, analyses were performed for a representative soil profile based on the results of the geotechnical site investigation, before and after completion of a proposed dredging program. The proposed dredging program is expected to remove a significant thickness of the potentially liquefiable fills (typically about 2 m to 3 m), thereby potentially reducing the impact of the flow slide on the piles,

As an example, Figure 7 summarizes the results for a typical Northern (pinned head) pile, neglecting any benefit from the proposed dredging program.

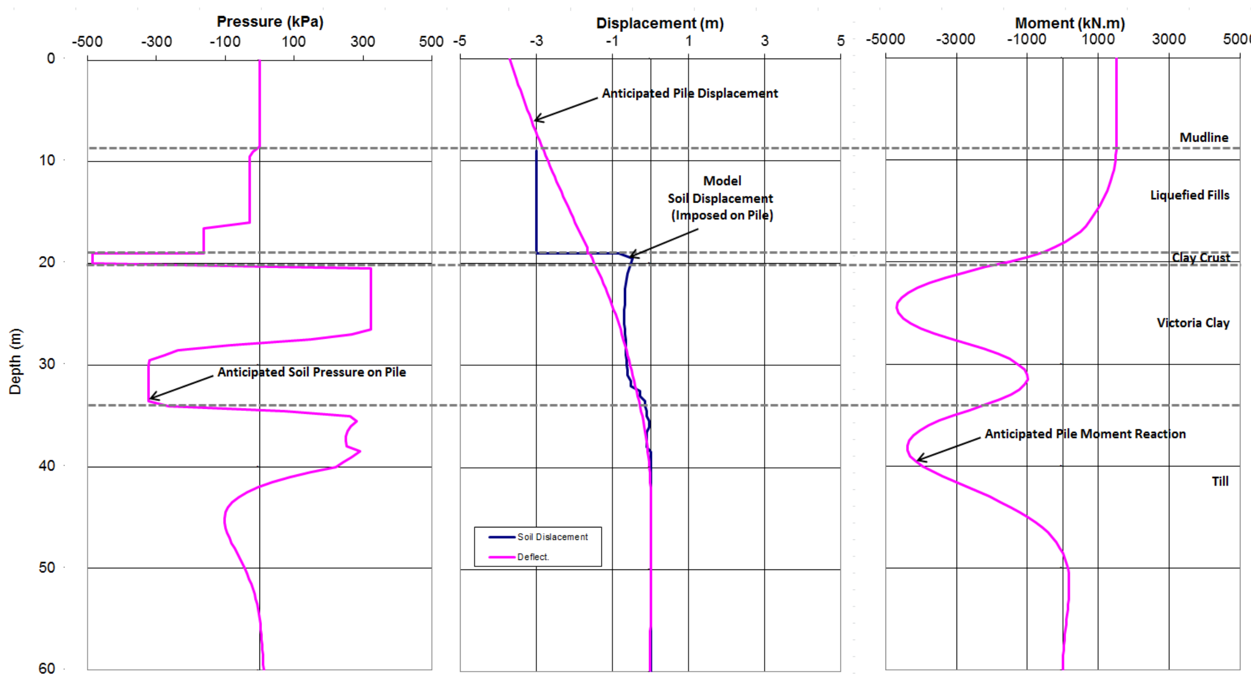

Fig. 7 – Example Displacement Profile – North Pile (Pinned Head) Middle of Proposed South Jetty

6. Implications for Future Possible Upgrades

6.1. Implications on Replacement of Existing Timber Jetty Structures

Based on the above seismic induced flow slide displacements and forces, it is not expected that the existing timber jetty structures would survive the design seismic event. As a result, a future replacement steel piled concrete deck jetty structure was proposed.

The results of the L-Pile analysis at the various design cross sections were used as a guide to estimate the post-seismic forces on proposed piles for input to the structural model for the design of the proposed steel piled concrete deck replacement. As the L-Pile analysis does not model any benefit due to the group effect of adjacent piles, or the increased stiffness from the deck structure, these figures were only used to estimate the forces on the piles for input into the structural model, and not pile displacements.

6.2. Implications on Existing Concrete Jetty

The existing concrete deck and steel pipe pile structure, which includes batter piles, is not capable of resisting the soil movements and associated loads resulting from the liquefaction induced flow slides, and consequently will be forced to move with the soil mass following a design seismic event. It is expected that many of the pile-to-deck connection points will fail resulting in a dramatic loss of lateral stiffness in the structural system. Forced unintended hinges will form in the pile tops, as well as within the supporting soil stratum, and will undergo high rotational demands, which will likely cause failure of some of the connections. Consequently, certain sections of this deck structure are likely to collapse at lower lateral displacements than the expected liquefaction-induced movements.

However, it is also possible that a significant portion of the structure deck mass will remain supported and elevated. Although it would lack lateral stiffness, and have severely limited residual vertical load capacity, it will likely lean towards the adjacent future jetty upgrade structure. The future jetty structure will be much more capable of resisting the seismic induced soil movements. Regardless of how the existing concrete deck and steel pile structure behaves, the adjacent future jetty upgrade structure will be designed to tolerate significant lateral displacement and will remain capable of supporting gravity loads and associated P-delta pile moments from the seismic design event. As a precaution against the future jetty attracting large lateral loads from the existing jetty, the future jetty upgrade plans include a sizable seismic movement joint at the boundary with the existing steel-piled jetty.

6.3. Implications on Timber Crib

To control the displacement of the timber crib and to minimize its impact on the adjacent piled jetty, displacement control piles were proposed. These piles will be driven parallel, and close to, the west side face of the timber crib and socketed into the bedrock. During earthquake shaking, these piles (and their tremie concrete pile cap) are expected to engage the timber crib and reduce its movement so that it does not damage the adjacent West Jetty. Since the displacement control piles only reduce but cannot completely eliminate movement of the timber crib, an articulated deck slab was proposed between the two structures. This articulated deck slab was designed intentionally to fail (as a structural fuse-link) under the design seismic event, in order to prevent structural damage to the steel-piled West Jetty on impact by the timber crib.

Seismic soil-structure interaction analyses were conducted using the computer program FLAC to assess the effectiveness of the displacement control piles, and to aid in the optimization of the pile design and spacing.

6.4. Implications on Anchored Sheet Pile Wall Bulkhead

The soil mass retained by the anchored sheet pile wall bulkhead is highly susceptible to liquefaction. At this time, there is no intention to implement soil densification, and, without mitigation, the liquefaction of the retained fills will impose an additional load to the south jetty structures. It is expected that the toe of the sheet pile wall will be forced outwards to the south as a result of the imposed soil movements during liquefaction. This will directly impact the existing steel pipe pile and concrete deck structure, but is unlikely to cause significant additional load or soil displacement (over and above the general slope failure displacement) against the future steel piled concrete deck structure which is to replace the south timber jetty.

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8. References

Adams, J, Halchuk, S., “Fourth generation seismic hazard maps of Canada: values for over 650 Canadian localities intended for the 2005 National Building Code of Canada”, *Geological Survey of Canada*, Open File 4459, 2003.

American Petroleum Institute (API), *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Load and Resistance Factor Design*, 2011.

EduPro Civil Systems Inc, *ProShake,* Software Version 1.12, 1999.

Ensoft Inc, *LPILE Plus Version 5*, Software Version 5.0.39, 2007.

Geological Survey of Canada (GSC), “Open File xxxx. Fourth Generation Seismic Hazard Maps of Canada: Maps and Grid Values to be Used with the 2010 National Building Code of Canada”, *(in preparation)*.

Geo-Slope International, *Slope/W – Slope Stability Analysis,* 2007.

Itasca Consulting Group, *FLAC, Fast Lagrangian Analysis of Continua,* Software Version 4.0, 1999.

Klohn Crippen (KC), *Esquimalt Graving Dock and South Landing Wharf Site – Investigation and Liquefaction Assessment*, June 2002.

Klohn Crippen Berger (KCB), *Esquimalt Graving Dock – South Jetty Wharf Development Project – Geotechnical Data Report*, March, 2010.

National Research Council of Canada (NRCC). *National Building Code of Canada 2005*, Ottawa, ON., 2005.

National Research Council of Canada (NRCC). *National Building Code of Canada 2010*, Ottawa, ON., 2010.

Pacific Geodynamics Inc. (PGI). *Phase I: Seismic Hazard Analysis of Esquimalt Graving Dock*, November, 1999.

Sy, A.. “Energy Measurements and Correlations of the Standard Penetration Tests (SPT) and the Becker Penetration Tests (BPT)”, *Ph.D. Thesis*, Department of Civil Engineering, University of British Columbia, Vancouver, BC, 1993.

Tomikatsu, K., and Seed, H.B., “Evaluation of Settlements in Sands Due to Earthquake Shaking”, *ASCE J. Geotech. Eng.,* Vol.113, No.8, pp.861-878, 1987.

Youd, T.L., Hansen, C.M., and Bartlett, S.F., “Revised MLR Equations for Predicting Lateral Spread Displacement”, *Proceedings of the ASCE Conference on Recent Advances in Geotechnical Earthquake Engineering*, University of Washington, Seattle, Washington, April, 2002.

Youd, T.L., Idriss, I.M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Harder, L. F., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Marcuson III, W. F., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Seed, R. B., Stoke II, K. H. “Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils”. *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 27, No. 10, pp. 817-833, 2001.