GEOTECHNICAL EARTHQUAKE ENGINEERING ASPECTS OF THE DESIGN OF FOUNDATIONS OF THE VANCOUVER CONVENTION CENTER EXPANSION PROJECT

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ABSTRACT: Key geotechnical earthquake engineering aspects of a displacement-based design that was adopted for a waterfront Convention Center in Vancouver, Canada are presented. The site selected for the facility was underlain by potentially liquefiable heterogeneous fills and marine sediments overlying glaciomarine soils and bedrock. Sequential triggering of liquefaction of soils, the resulting lateral displacements and their complex interaction response with foundations were found to be important in the overall seismic assessment. Modeling soil behaviour both before and after onset of liquefaction was important in assessing lateral spreading-induced drag loads on, and displacements of, the deep foundations penetrating the liquefiable soils into bedrock at depth. Two different foundation schemes comprising drilled shafts and driven piles were analyzed and a piled foundation scheme was selected for final design. The results of a number of numerical simulations were used to arrive at an optimum ground densification scheme for the site and to quantify the anticipated range of seismic lateral movements. The paper presents some of the challenges faced by the designers and the results of detailed coupled soil-foundation-structure analyses undertaken to arrive at a design that was acceptable from structural design considerations.

KEYWORDS: LIQUEFACTION, SOIL-STRUCTURE INTERACTION, DEEP FOUNDATIONS, STONE COLUMNS

1. INTRODUCTION

Vancouver is in an area of high seismic risk associated with a major subduction zone off the Canadian west coast. The dynamic geological setting makes this region one of the most seismically active regions of Canada. Although Vancouver has not yet experienced a major earthquake, a number of large earthquakes have occurred in the region that have been felt in Vancouver. The 1946 Campbell River earthquake is the largest magnitude earthquake that has affected the area, and it is estimated that Downtown Vancouver was subjected to a peak horizontal bedrock acceleration of about 0.06 g. Crustal and sub-crustal earthquakes of magnitudes varying from M6 to 7 have been recorded over the last century within the North American and the subducting Juan de Fuca plates that extend beneath Vancouver. Geological evidence also suggests that there is the potential for a much larger inter-plate earthquake of magnitude M8+ to occur within the off-shore Cascadia subduction zone. A major earthquake associated with either of these sources could have devastating effects in Vancouver.

Figure 1: A Photograph of the New Convention Centre Looking Northwest (May 2008)

In preparation for the 2010 Olympics and Paralympic Winter Games, the City of Vancouver decided to expand its existing Convention Centre in Coal Harbour (Downtown Vancouver). The 106,000 m^2 new Vancouver Convention Centre (VCC) will welcome some 10,000 media members in 2010 who will be attending the winter games. A photograph of the facility that is being constructed is shown on Figure 1. The site is located in Coal Harbour between Canada Place to the east and Harbour Green Park to the west (see Figure 2).

Figure 2: An Aerial View of the Convention Centre Site Looking East (May 2003)

Designing the new Convention Centre to withstand the code-specified 475-year ground motions (A = 0.2 g, M7) was a challenge due to the anticipated liquefaction potential/cyclic mobility of site soils and the potential impact on the deep foundations considered to support the structure. The presence of heterogeneous man-made fills, remnants of previous marine piers, and previous shoreline protection measures made this a unique site for foundation construction and ground improvement.

This paper describes the history of the site development, underlying soil conditions, and some of the seismic ground response and soil-structure interaction analyses undertaken in support of the displacement-based approach adopted for the design of foundations for the structure.

2. SITE DEVELOPMENT

The development of the Coal Harbour area started with the establishment of the western terminus of the Canadian Pacific Railway (CPR) in the 1880's followed by port development and subsequent industrial development along the foreshore. Aerial photographs taken in 1946 through 1999 show the progressive operation of several marine piers projecting into Coal Harbour at the VCC site (see Figure 3). During this period, the marine piers were modified and portions demolished and new smaller piers constructed. These processes resulted in remnants of piers being left in place and reclamation of land by infilling of offshore areas was carried out using miscellaneous fill materials originating from various construction projects in the downtown area.

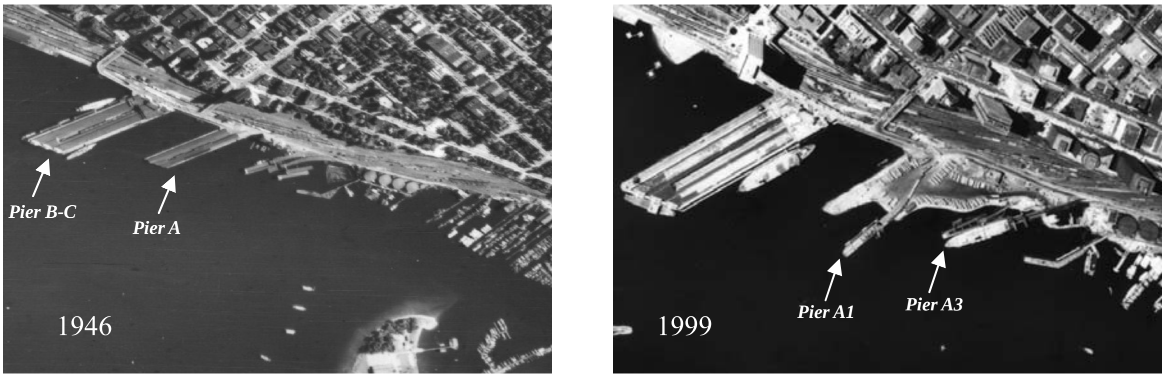

Figure 3: Historical Aerial Photographs of the Site

The VCC site is underlain by fill materials and remnants of foundations of the marine piers that previously occupied the site as well as recently-placed heterogeneous fills during land reclamation. The fill materials, in turn, are underlain by both granular and fine-grained marine sediments and glacial drift and/or bedrock.

An oblique aerial photograph taken on May 2003, at or about the time of project inception, shows the use of the site as a commercial marina and floatplane terminal (see Figure 2). The latest reclamation and shoreline development to the immediate west and in the northwest portion of the proposed VCC site were completed in 2000.

3. GENERAL SUBSURFACE CONDITIONS

A large number of geotechnical boreholes have been drilled and sampled in both the onshore and offshore areas of the VCC site. Typical soil stratigraphy along an approximately north-south axis taken near the middle of the site is shown in Figure 4.

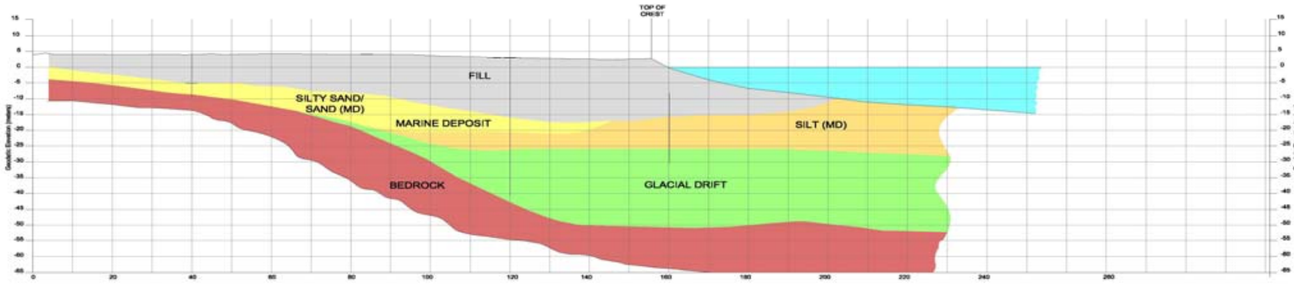

Figure 4: Typical Soil Stratigraphy Along A North-South Section (Looking West)

The site soil conditions can be represented using four main stratigraphic units. In the order of increasing depth, these stratigraphic units consist of (1) man-made fills, (2) marine sediments, (3) glaciomarine and glacial drift deposits, and (4) sedimentary bedrock. Brief descriptions of the materials found in these units are given below.

3.1 Man-made Fills

The thickness of the man-made fill materials is generally least near the southern edge of the site (about 10 m) and increases northward, as the water deepens, to the current shoreline where it is about 20 m. The marine piers at the site were constructed with select fills. However, the fills that were placed subsequent to demolition of the marine piers and elsewhere at the site consist of random fills that are heterogeneous in nature and consist of silts, sands, and gravels with varying amounts of brick fragments, cobbles, boulders, wood waste, concrete rubble and other materials.

3.2 Marine Sediments

Beneath the fills, where they are present, and at the seabed in the areas offshore of the fills, are the original marine seabed deposits covering this area. These range in thickness from about 3 m in the southern part of the site to about 20 m offshore and consist of compressible silts and clayey silts (with some organics) to sand and silty sand (with shells and gravel). The coarse-grained soils are in a state of loose to compact relative density and inferred to be near-surface beach deposits. The fine-grained marine sediments are firm to stiff in consistency in the onshore area, and grade into a very soft to soft consistency some distance offshore of the crest of the shoreline slope.

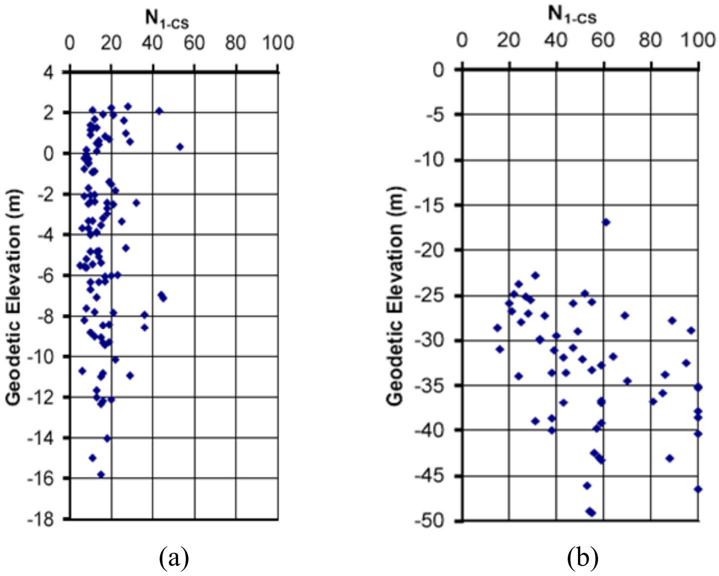

Figure 5: Interpreted Penetration Resistance Profiles

3.3 Glaciomarine and Glacial Drift Deposits

Underlying the marine sediments**,** except in the southern portion of the site, is a stratum of silt, sand, gravel and cobbles inferred to be glaciomarine and glacial drift deposits. The relative density of these soils is compact to very dense in the upper 10 m and dense to very dense at depth. The relative density is variable at any given elevation or depth. The thickness of this deposit, where present, ranges from about 5 to 30 m.

3.4 Sedimentary Bedrock

Bedrock consisting of interbedded sandstone, siltstone and mudstone (i.e. sedimentary rocks) exists across the site at depths ranging from about 10 m below ground surface at the south side of the site to about 45 m below seabed at the north side. The bedrock is in various states of weathering from significantly weathered to slightly weathered across the site**,** and is generally classified as very weak to weak.

Typical profiles of SPT penetration resistance $N_{1\text{-}cs}$ (bls/0.3 m) established for the granular soils comprising the fill materials and glacial drift are shown in Figures 5a and 5b, respectively.

4. FOUNDATIONS

Two major foundation schemes were considered during the concept and preliminary design phases. One of the foundation schemes consisted of drilled shafts varying in diameter from 1.2 to 3.0 m and socketed into the underlying bedrock. The other consisted of 914 mm diameter open-ended steel pipe piles driven to bedrock.

The drilled shaft foundations were considered initially since it would be easier to drill out any obstructions encountered during installation through the heterogeneous fills than smaller diameter driven piles. It was also considered that drilled shafts would be capable of withstanding moderately high liquefaction-induced soil drag loads without excessive lateral movements of the structure. A test drilled shaft was installed at the northern shoreline and near the middle of the site to a depth of about 50 m to assess installation feasibility and to provide baseline data for contractors.

After additional geotechnical field investigations and classification of the site into areas of low to high risk for installation of pile foundations (see Figure 6), the marine structural consultant (Westmar Consultants) concluded that a driven pile foundation scheme would be more economical than drilled shafts even with a contingency for cleaning and re-driving the piles if and when obstructions are encountered. However, the seismic performance requirements dictated that extensive ground improvement measures be implemented at the site to mitigate the effects of soil liquefaction on the proposed 914 mm diameter pile foundations. Extensive ground response and soil-foundation-structure interaction analyses were undertaken to assess the impact of soil liquefaction on the proposed foundations and to design optimum ground improvement measures to meet the seismic performance expectations. Ground improvement using vibro-replacement stone columns was considered to be the most cost-effective method of soil densification.

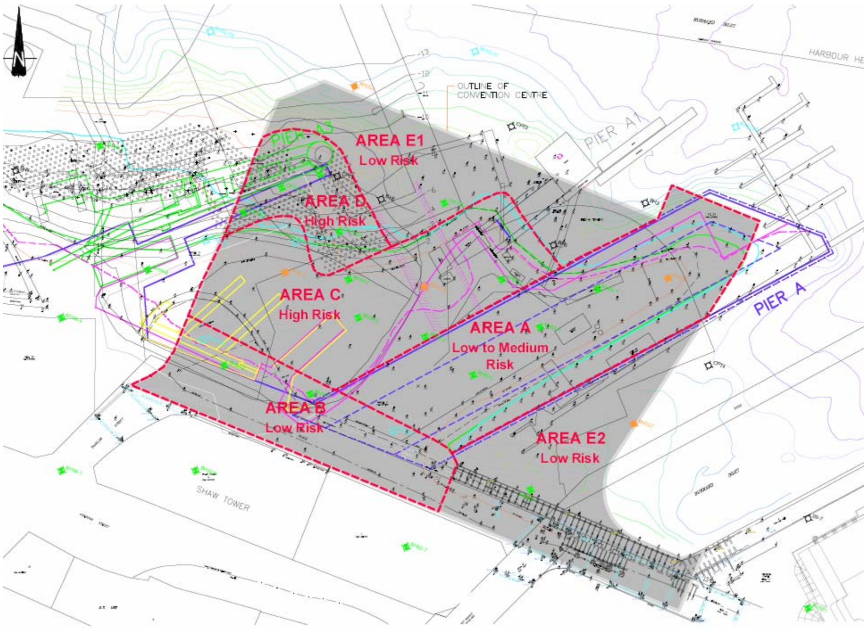

Figure 6: Qualitative Risks - Pile Installation

In order to finalize the foundation scheme, a test program was undertaken in 2003/2004 to verify the ability to improve site soils to the target densification levels, to confirm that the required allowable pile compression capacity could be achieved, to confirm/determine the pile embedment lengths into sedimentary bedrock, and to confirm the drivability of the open-ended piles through the site fills, which contained several potential obstructions for piling. The test program included installation of 5 test piles distributed throughout the site, Stat*namic* testing of the piles to confirm the mobilized ultimate pile capacity, densification of two onshore locations measuring some 10 m by 16 m in plan area, and excavation of test pits at selected locations to assess potential for obstructions.

Based on the results of the test pile program and engineering analyses, it was concluded that the 914 mm diameter piles driven with open ends could achieve allowable geotechnical compression capacities of 4 MN/pile and 5 MN/pile with penetrations of about 20 m and 3 m into glacial drift and bedrock, respectively.

5. RESPONSE UNDER SEISMIC LOADING

Evaluation of the cyclic response of site soils and their interaction with the pile foundations were critical to confirming the seismic performance of the selected foundation scheme. In particular, effects of depth to bedrock on seismic wave propagation, the resulting sequential liquefaction of soils, and the lateral stiffness of pile foundations were considered important aspects that need to be considered in the analysis of the seismic performance of the foundations.

Based on index properties and results of site-specific cyclic simple shear tests, the liquefaction susceptibility of the fine-grained marine sediments was assessed to be low. The risk of liquefaction of glacial drift was ruled out based on the high penetration resistance values obtained in this deposit.

Results of simplified 1D ground response analyses carried out at the onset of the project indicated that the existing heterogeneous fills and the granular portions of the marine deposits have a high risk of liquefaction when subjected to the design seismic loading conditions. The estimated free-field lateral ground displacements induced by liquefaction in areas located closer to the shoreline exceeded 1 m. Analyses were initially undertaken using simplified and uncoupled methods where the response of the individual piles was analyzed using the computer code LPILE using “p-y” curves derived for both liquefied and non-liquefied/competent soils and imposing the lateral free-field displacements estimated using empirical models such as Youd et al (1999). These analyses indicated that foundation displacements could be excessive at deck level. In light of the increasing thickness of fill materials present near the shoreline and depth to bedrock, the likely differences in the time at which liquefaction would be triggered in the different areas of the soil profile and extent of soil liquefaction, and the anticipated complex interaction response between the soil and foundation piles, it was considered necessary to develop coupled soil-foundation interaction models to assess the seismic loading-induced displacements of the substructure to permit displacement-based design. It was also recognized that it was important to incorporate the superstructure inertial loads in these analyses, at least in an approximate manner.

Considering the symmetry of the pile layout prepared by the marine structural consultant and similarities in the soil stratigraphy along the shoreline, plane strain conditions were assumed in the seismic analyses.

5.1 2D FLAC Finite Difference Model(s)

The soil-foundation-structure interaction analyses were carried out using the 2D finite difference code FLAC and incorporating the user-defined constitutive model UBCTOT to simulate the cyclic behaviour of granular soils underlying the site. Details of UBCTOT are described in Beaty & Byrne (1999). The analyses were carried out in the time-domain, allowing for liquefaction to be triggered in individual soil zones during shaking, and taking into consideration the soil-foundation interaction effects. Liquefaction triggering is evaluated by tracking the dynamic shear stress history of each material zone or element. The irregular shear stress history caused by the earthquake is interpreted as a succession of uniform half cycles. Each half cycle is transformed into an equivalent number of cycles that correspond to the penetration resistance of the zone, and liquefaction is triggered when the summation of equivalent uniform cycles is 15. With the onset of liquefaction, the soil is softened and the shear strength reduced to its residual value, as shown schematically in Figures 7a and 7b (after Beaty & Byrne, 1999):

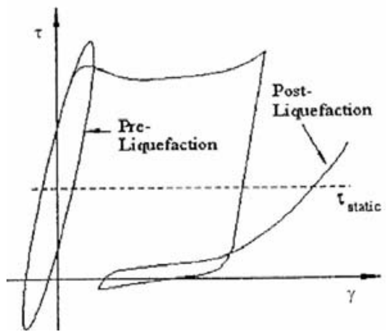

Figure 7a: Idealized Stress-Strain Behaviour

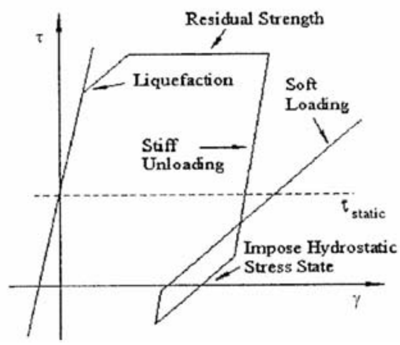

Figure 7b: UBCTOT Model

The residual shear strength was considered to be stress path dependent, varying between about 0.4 and 0.1 times the initial vertical effective overburden stress depending on whether the major principal stress is vertical or at 45 degrees to the vertical, respectively. The cyclic response of the non-liquefiable fine-grained soils and glacial drift was simulated using the Mohr-Coulomb constitutive model available within FLAC.

The effects of the superstructure were incorporated in the interaction response analyses in an approximate manner using simplified structural models developed by the marine structural consultants for the two different foundation schemes. These simplified superstructure models consisted of beam elements and had properties (i.e. mass, stiffness) that produced inertial loads that were equivalent to the prototype superstructure under seismic loading conditions.

Two of the FLAC models developed for the analysis of the drilled shaft and piled foundation systems are shown in Figures 8a and 8b, respectively. Figure 8a shows the drilled shaft foundation scheme that only considered ground improvement in the form of a barrier along the shoreline and no improvement in the fills and marine deposits. Figure 8b shows the piled foundation scheme that considered densification of fills and marine sands plus a 30 m wide zone extending through the marine deposits to the top of glacial drift along the shoreline. Structural design requirements dictated that the upper part of site soils should be densified to a depth of several pile diameters in order to increase the bending moment capacity of the piles. Depending on the configuration and lateral force transfer mechanisms considered for the superstructure, different simplified representations were considered in the model to simulate the superstructure inertia effects. The piled foundation scheme was selected for the final design and all subsequent analyses were carried out for this scheme.

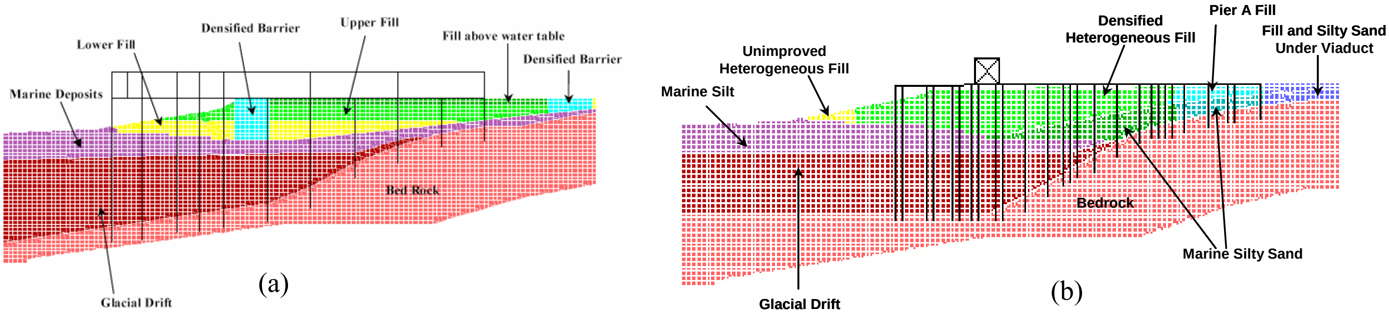

Figure 8: FLAC Finite Difference Models Used to Discretize the Drilled Shaft and Pile Foundation Schemes

5.2 Input Parameters

The input parameters used in the seismic analyses are described in this section.

5.2.1 Ground Motions

The 475-yr Site Class C Uniform Hazard Response Spectrum (UHRS) for Vancouver provided by the Geological Survey of Canada was used to develop the input ground motions. The spectrum was uniformly scaled to the site-specific PFGA of 0.2 g. The resulting 5% damped response spectrum is shown in Figure 9.

Two sets of spectrally-matched ground motions were developed for use as input acceleration time-histories in the ground response analyses. A modified horizontal acceleration time-history used in the analyses is shown in Figure 10.

Details of the earthquake records used for spectral matching are summarized in Table 1.

5.2.2 Structural Properties and Loads of Simplified Structural Models

The structural properties and loads used to characterize the sub-structure and super-structure models are summarized in Table 2. All parameters are given per bay length.

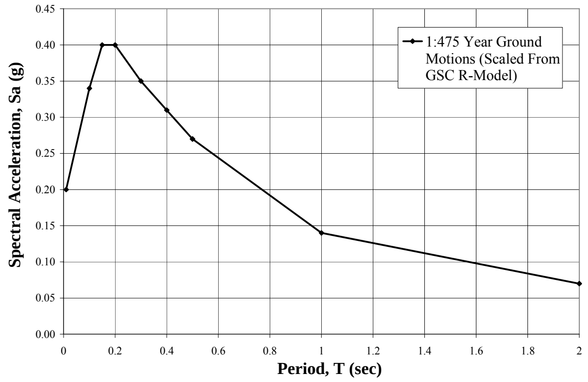

Figure 8: Uniform Hazard Response Spectrum

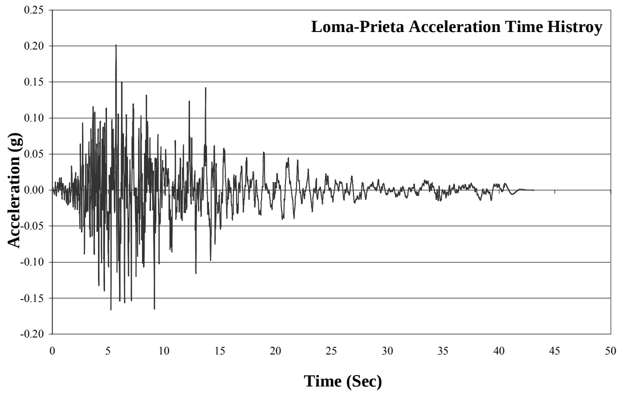

Figure 9: Typical Input Acceleration Time-History

Table 1 – Details of Earthquake Records Used for Spectral Matching

Event	**Date**	**Magnitude and Bracketed Duration**	**Epicentral Distance**	**Peak Horizontal Ground Motions**		**Station**
				a (g)	**v (m/sec)**	
San Fernando (EW)	February 9, 1971	M 6.6 [~7 sec]	36 km	0.20	0.11	Seismological Laboratory
Loma Prieta (NS)	October 18, 1989	M 7.0 [~25 sec]	10 km	0.53	0.35	Capitola

5.2.3 Soil Parameters

The different soil units were characterized based on the results of field investigations and penetration resistance measurements required to prevent triggering soil liquefaction. Parameters established for the east half of the structure are summarized in Table 3.

5.3 Results

The results of the FLAC analyses indicated that the marine sand encountered in the southern half of the site would liquefy and soften under the 475-year ground motions, even if improved using vibro methods to an equivalent clean sand $(N_1)_{60\text{-}cs}$ of 20 blows/0.3 m. One of the interesting observations of the numerical simulations was that liquefaction of soils commenced at the south where bedrock was shallow and progressed towards north where bedrock was deeper. Some localized and shallow zones of the improved heterogeneous fills also liquefied during strong shaking, but extensive liquefaction and/or softening of the fills was not predicted. Due to the lateral confinement provided by the surrounding non-liquefied soils and the structural piles, the impact of liquefied soils on structural performance was seen to be localized.

The shorter piles driven to bedrock at the south end offered more lateral resistance to deformation than the longer piles at the north end. The lateral stiffness of the pile foundations and the tendency of the soil mass to move towards water (to the north) resulted in considerable soil-foundation interaction that resulted in an overall reduction of seismic movements of the foundations.

With ground improvement implemented in the on-shore and off-shore areas of the site, less than 115 mm of transient lateral displacements were computed to occur during strong shaking at the floor slab level of the structure. The computed residual lateral displacements were 15 mm or less. A typical displacement time-history computed at floor slab level is shown in Figure 10.

Table 2 – Structural Input Parameters (East Half)

Per Bay Width = 13.7 m								
Member	E (MPa)	I (m^4)	A (m^2)	P (m)	M_p kN-m	Comp/ Tens Capacity (kN)	Density (T/m^3)	Load kN/m
Sub-Structure								
Piles	$2x10^5$	$5.352x10^{-3}$	$5.342x10^{-2}$	2.87	2,477	-	7.8	-
Foundation Cap: Typical	$3x10^4$	$6.127x10^{-1}$	2.30	-	-	-	*	243
Foundation Cap: Apron	$3x10^4$	$3.517x10^{-1}$	1.97	-	-	-	*	240
Super-Structure								
Columns	$2x10^5$	$2.150x10^{-3}$	$3.080x10^{-2}$	-	-	-	0.001	-
Beams East Model)	$2x10^5$	$2.150x10^{-3}$	$3.080x10^{-2}$	-	-	-	*	3044
Beams West Model	$2x10^5$	$2.150x10^{-3}$	$3.080x10^{-3}$	-	-	-	*	1762
Bracings East Model	$2x10^5$	0.0	$3.900x10^{-3}$	-	-	1365	0.001	-
Bracings West Model	$2x10^5$	0.0	$3.900x10^{-3}$	-	-	790	0.001	-

Notes: * Member self-weight is included in the corresponding load. E is the Young's Modulus; I, the second moment of inertia; A, the area; P, the perimeter; and M_p, the plastic moment of the piles.

Table 3 – Soil Input Parameters (East Half)

Soil Unit	γ_{sat} (kN/m^3)	$(N_1)_{60}$ (blows/0.3m)	ϕ (deg)	Su (kPa)	V_s (m/s)	G_{max} (MPa)	υ
Heterogeneous Fill (Viaduct)	18	10	34	-	-	$94\ (\sigma'_m/Pa)^{1/2}$	0.45/0.33*
Pier A Fill	20	27	36	-	-	$130\ (\sigma'_m/Pa)^{1/2}$	0.45
Densified Fill	20	20	36	-	-	$118\ (\sigma'_m/Pa)^{1/2}$	0.45
Southern Silty Sand	18	17	35	-	-	$112\ (\sigma'_m/Pa)^{1/2}$	0.45
Northern Silty Sand	18	17	35	-	-	$112\ (\sigma'_m/Pa)^{1/2}$	0.45
Marine Silt	17.4	-	-	$0.30\ \sigma'_v \geq 10$ kPa	-	1,300 Su	0.45
Marine Silt (w/ Stone Columns)	17.4	-	-	$0.37\ \sigma'_v \geq 10$ kPa	-	1,300 Su	0.45
Glacial Drift	20	-	38	-	-	$169\ (\sigma'_m/Pa)^{1/2}$	0.33
R0 Bedrock	20	-	45	-	450	$(\gamma/g)V_s^2$	0.125
R1 Bedrock	20	-	45	-	800	$(\gamma/g)V_s^2$	0.125
Bedrock	20	-	45	-	1200	$(\gamma/g)V_s^2$	0.125

6. SUMMARY

Detailed geotechnical earthquake engineering analyses were undertaken to assess the seismic performance of the new Vancouver Convention Centre that will welcome some 10,000 media members in 2010 who will be attending the Winter Olympic Games. The impact of seismic loading-induced soil liquefaction and the resulting lateral displacements of the foundations supporting the structure can be reduced when ground improvement measures are implemented offshore in the form of an in-ground barrier or dike to contain the site soils and onshore within the heterogeneous fills and the underlying marine sands. Using a 2D FLAC finite difference model incorporating soil-foundation-structure interaction response and a user-defined constitutive model that is capable of simulating both the pre- and post-liquefaction stress-strain behaviour of granular soils, it has been demonstrated that the soil-foundation-structure system considered for the new Vancouver Convention Center would experience seismic lateral displacements in the order of 15 mm to 115 mm that are tolerable for structural design. The ability to model sequential liquefaction and soil-pile interaction response with appropriate input seismic ground motions resulting from variable bedrock topography and superstructure effects were important in quantifying the anticipated lateral displacements at floor slab level.

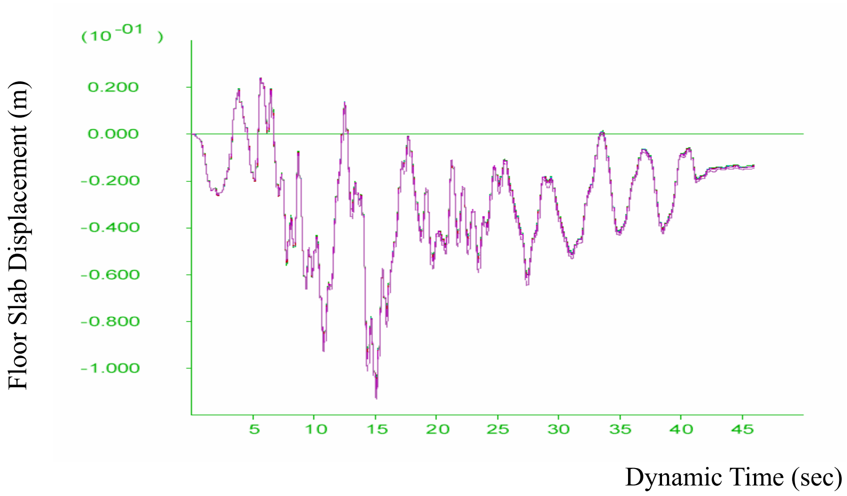

Figure 10: Horizontal Displacement Time-Histories of Floor Slab (475-yr Loading)

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REFERENCES

Beaty, M.H. and Byrne, P.M. (1999). *A Synthesized Approach for Modeling Liquefaction and Displacements*; Proc. FLAC and Numerical Modeling in Geomechanics; Detournay and Hart (Editors); 1999 Balkema, Rotterdam; pp. 339-347.

Youd, T.L., Hansen, C.M. & Bartlett, S.F. (1999). Revised MLR equations for predicting lateral spread displacement. *Proceedings of the Seventh U.S.-Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures Against Soil Liquefaction*, *Technical Report MCEER-99-0019*, Multidisciplinary Center for Earthquake Engineering Research, State University of New York, Buffalo, N.Y.