13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 2138

SEISMIC DESIGN OF A NEW PILE AND DECK STRUCTURE ADJACENT TO EXISTING CAISSONS FOUNDED ON POTENTIALLY LIQUEFIABLE GROUND IN VANCOUVER, BC

Dave SMITH[1], Ernest NAESGAARD[2] and Harald KULLMANN[3]

SUMMARY

The Vanterm Terminal was built in Vancouver harbour in the early 1970s using 22 m high by 15 m wide concrete gravity caissons. The caissons were founded on granular fill up to 15 m thick placed through water. Granular fill up to 35 m thick was also dumped through water behind the caissons to provide a level surface for the terminal. The foundation fill and backfill are loose and subject to liquefaction during the design earthquake.

In 2001, the Vancouver Port Authority determined that a 4200 m^2 extension adjoining the caisson structure was required. A pile and deck structure was selected. Design loads included containers, large handling equipment and ship impact. The structure was required to survive the 475 year design earthquake without collapse but substantial damage and displacement were permitted.

Dynamic numerical analyses using the computer program FLAC (Fast Lagrangian Analysis of Continua) were conducted to assess the seismic performance of the existing caissons and new piled deck. Dynamic shaking, liquefaction triggering, consequences of liquefaction and soil-structure interaction were addressed in the analysis. The 3 dimensional aspect of the pile layout was included in the analyses.

The dynamic analyses indicated that:

- Without ground improvement, the caissons were marginally stable during the earthquake and would move laterally up to 4 m, impacting the new piled deck and potentially collapsing some of the piles.

- Densifying or providing drainage in a 15 m wide zone behind the caissons and setting the closest piles 1.5 m away from the caisson reduced the lateral movement of the caisson to less than 0.5 m and significantly reduced the impact of the caisson on the deck.

[1] Principal, Thurber Engineering Ltd., Vancouver, BC, Canada (dsmith@thurber.ca)

[2] Senior Associate, Trow Consulting Engineers Ltd., Burnaby, BC, Canada (ernestn@shaw.ca)

[3] Senior Project Manager, Westmar Consultants Inc., North Vancouver, BC, Canada. (hkullmann@westmar.com)

The final design included a zone with seismic drains in lieu of densification behind the caisson structure. Geotechnical and structural design methodology and observations during construction are presented in the paper.

INTRODUCTION

The Vanterm container terminal is located on the south shore of Burrard Inlet in Vancouver Harbour, about 3 km east of Canada Place, the main cruise ship terminal in the Port. The terminal berth structure utilized 23 concrete gravity caissons to provide a perimeter about 800 m long for ship berthing. In 2001, the Vancouver Port Authority determined that an extension to the berth at its Vanterm container terminal was essential for the long term viability of the terminal's operations. The proposed extension would allow simultaneous berthing of 2 Post-Panamax container ships. The extension was to be located along the west end of the Terminal, with dimensions of 79 m along the wharf face and 52 m perpendicular to the face, as shown by the general arrangement on Figure 1. The rails on which the gantry cranes travelled would also be extended by 61 m to the end of the extended berth. Preliminary design studies showed that a conventional pile and deck structure was the optimal solution. Steel pipe piles were selected for the structure, based on minimizing the impact on the marine environment and cost considerations, since surplus pipe of a suitable diameter was available locally. Site investigation and detailed design studies resulted in the structure described herein. Design criteria for the structure required it to survive the 475 year return period earthquake without collapse but allowed substantial damage and displacement.

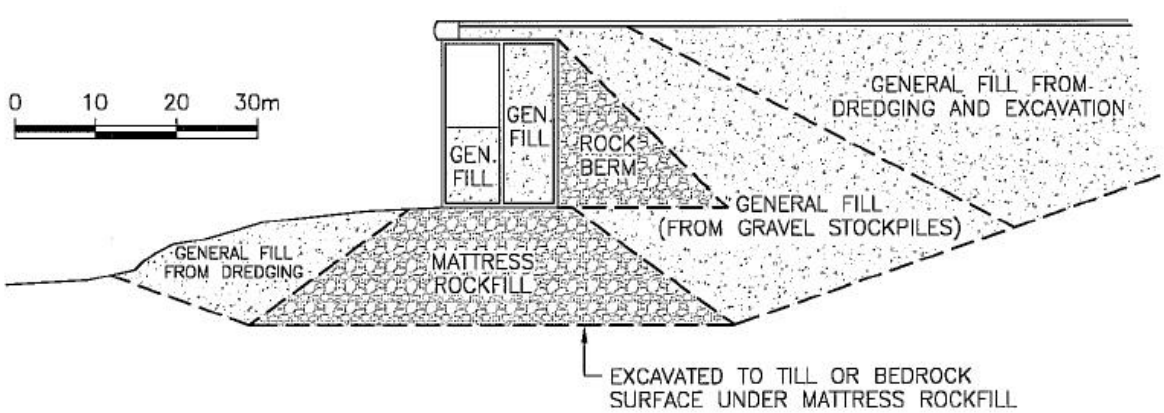


Figure 1
General Arrangement Of Berth Extension

ORIGINAL TERMINAL CONSTRUCTION

The Vanterm Terminal was built in the early 1970s using concrete gravity caissons measuring 22 m high, 15 m wide and 36 m long. Foundations for the caissons were prepared by dredging soft sediments from the sea bed to expose bedrock or glacial till and placing rockfill up to the design elevation at the base of the caisson, to form a mattress. The caissons had internal compartments and were partially filled with gravel after being placed in position. Backfill behind the caissons comprised a rockfill zone and gravel fill, according to historical information provided by the Port. This information, interpreted for a section through Caissons 17 and 18, is shown on Figure 2. Site investigation carried out for the Terminal Expansion, described below, provided results which did not entirely agree with the historical information provided, although the apparent discrepancy may be attributable to the drilling technique.

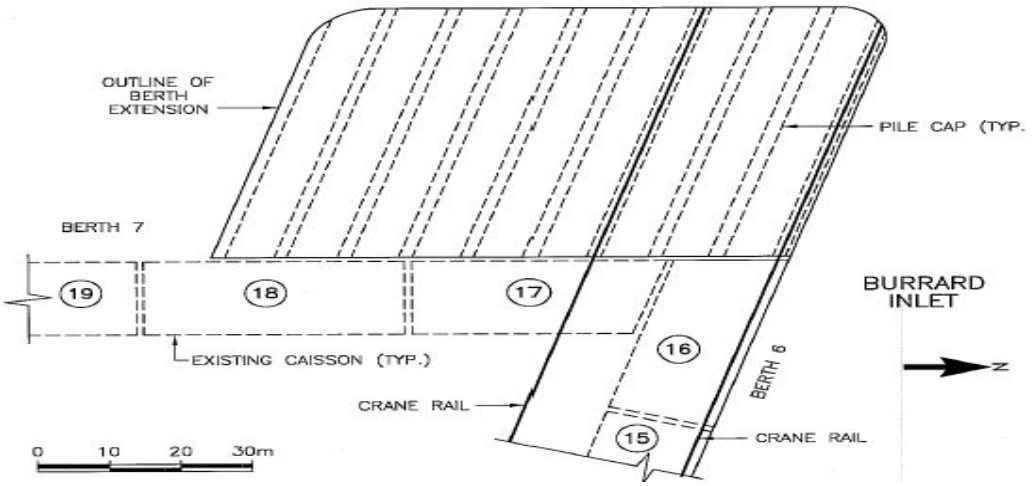


Figure 2
Original Berth Const\ruction

SOIL CONDITIONS

Site Investigation

Three phases of site investigation were carried out, using truck-mounted rigs for on land drilling and raft- or barge-mounted drills for offshore drilling. Initial offshore drilling was carried out using a small, raft-mounted auger drill able to install casing and recover disturbed samples on the auger flights. Dynamic-Cone Penetration Tests were carried out adjacent to each drill hole to provide in-situ soil density information. After completing 4 holes, this drilling method was abandoned due to increasing water depth (of up to 30 m) and wave action.

Offshore and onshore drilling was continued using a Becker hammer drill, mounted on a large spud-equipped barge for the offshore holes. In the Becker method of drilling, an open casing of 80 mm internal diameter is driven by a small diesel pile hammer and soil contained within the casing is ejected at the ground surface by high pressure air injected at the drill bit. Disturbed and somewhat mixed samples are collected as the soil is ejected at the top of the casing. Blow counts were recorded during open hole drilling and identified as BOC blow counts. The casing can also be driven closed end, in the manner of a 140 mm pile, to provide Becker Penetration Test (BPT) blow counts which can be utilized for assessment of the in-situ density and, thus, liquefaction potential of cohesionless soils. BPTs were used for drilling behind the caissons to allow assessment of the potential for liquefaction of the fill during a severe earthquake.

Additional site investigation was carried out by geophysical methods using high frequency sonar probes to investigate to shallow depth in the sea bed and an electrical pulsar system for deeper penetration, in an attempt to locate the remnants of a caisson which inadvertently sank offshore of its intended position during construction. The geophysical investigation was not conclusive, with no positive identification of the remains of the buried concrete structure which had been partially demolished after it sank. The caisson remnants were removed using a clamshell prior to pile installation.

Significance of Soil Conditions

Figure 3 shows a section taken perpendicular to the face of Caissons 17 and 18 illustrating subsurface soil conditions revealed by the drilling program, in comparison to the stratigraphy expected from the historical information provided by the study. The most noticeable discrepancy is the bottom mud encountered in holes drilled 5 m offshore of the wharf face where mattress rockfill was expected. This suggests that the original fill had been eroded, perhaps by prop wash, and replaced by bottom mud. It is also possible that the rockfill had been displaced by the caisson which sank during construction.

The apparent absence of the rockfill zone behind the caissons is also significant, through the Becker drill fragments larger particles and restricts the maximum size of material recovered to about 75 mm. However, test holes showed the fill to have very low penetration resistance down to the bedrock surface, not indicative of rockfill. The fill was judged to be liquefiable during the 475 year earthquake, leading to marginal global stability, tilting and offshore movement of the caissons. In turn, this indicated the need for ground improvement behind the caissons and a seismic isolation gap between the face of the existing wharf and the edge of the new deck.

The drilling results also suggested that bedrock, generally sandstone, would be encountered by end-bearing piles with little or no overlying glacial till within 10 to 15 m of the wharf face. Further offshore, the bedrock surface dipped to the west and the till stratum became thicker.

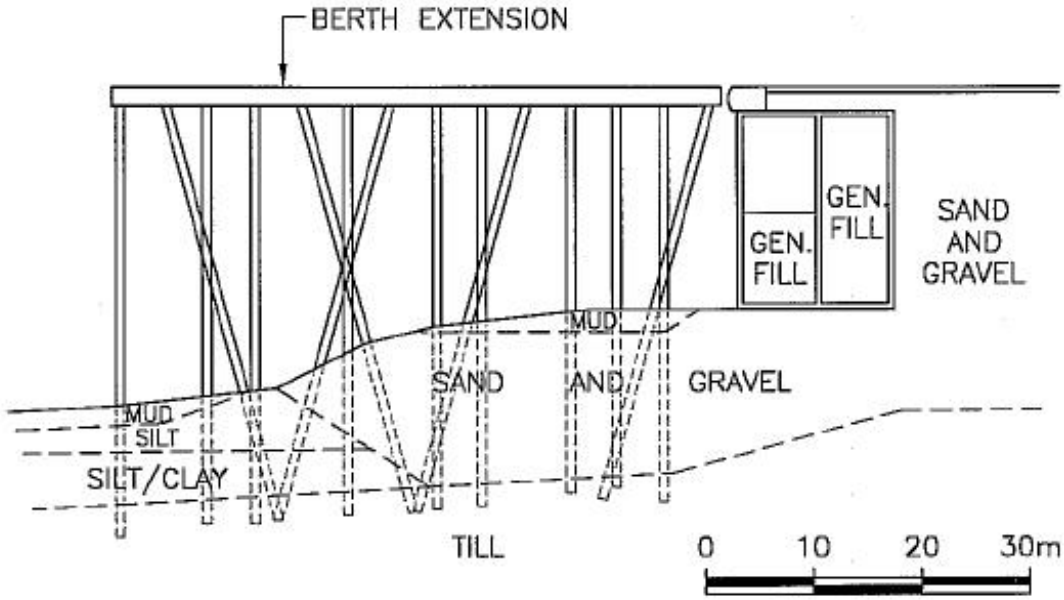


Figure 3
Original Berth and Extension

PILE DESIGN

Capacity

Steel pipe piles of 914 mm diameter were selected for the Berth Extension, primarily because of their availability in Vancouver as a result of the cancellation of a large marine construction project. The piles were to be driven closed end to refusal in till or bedrock. Based on experience gained during installation of similar piles at the Cruise Ship Terminal Expansion at Canada Place, about 3 km to the west, allowable end bearing pressures of 5000 kPa and 7500 kPa were allowed in till and bedrock, respectively. Allowable adhesion values of 125 kPa and 400 kPa were recommended in till and bedrock, respectively. These parameters provided allowable compression capacities of 4000 kN and 5500 kN, respectively, for piles driven about 2 m into dense till and bedrock. For short term loading, as in an earthquake, 50% overstress was allowed.

WEAP Analysis

WEAP analyses were carried out for 37 m long, 914 mm diameter piles driven closed end with Delmag D62 and D80 hammers, with the results shown on Figure 4. The Figure shows that, for the D62 hammer, a refusal criteria for driving of over 200 blows / 300 mm pile penetration is required to achieve a capacity of 8000 kN, twice the allowable capacity with no allowance for pile set-up. The D80 hammer required only 180 blows / 300 mm penetration. The tender documents allowed use of a hammer with 225 kJ rated energy, equivalent to a Delmag D62.

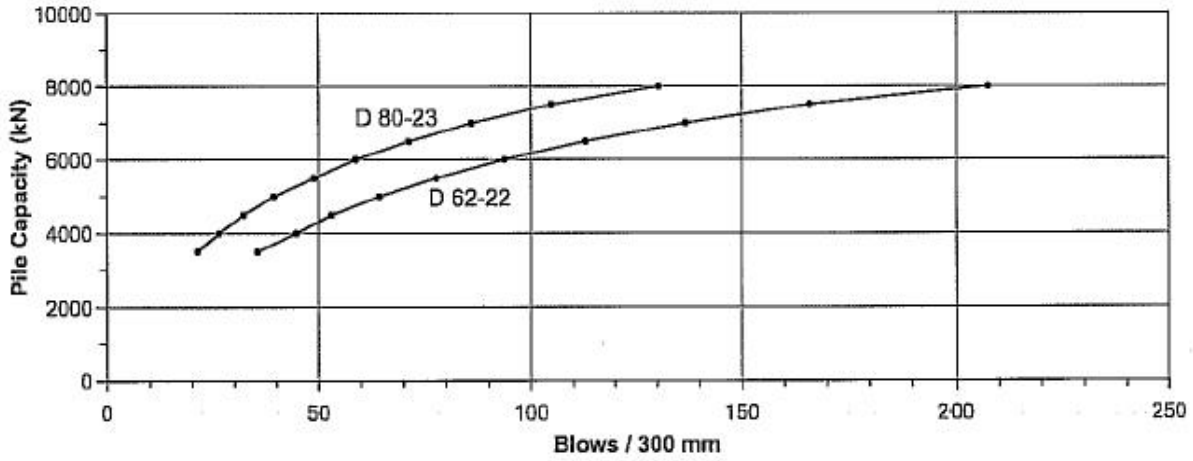


Figure 4
WEAP Analysis Results

Uplift Resistance

Piles driven 2 m into dense till were estimated to develop an allowable uplift capacity of 700 kN, increasing to 1000 kN for seismic loading conditions. Piles driven a similar distance into bedrock were estimated to have allowable uplift capacities of 2200 kN and 3000 kN under working and seismic loading conditions.

The uplift capacities were less than the uplift loads calculated for many of the batter piles during a seismic event. These loads could reach 3000 kN. The additional capacity was provided by anchors installed through the pile end plate.

DESIGN OF PILE AND DECK STRUCTURE

Conventional deck construction was envisaged for the structure. This consisted of piles arranged in baylines spaced 8.1 m apart, parallel to the berth and crane rails, and capped with reinforced cast-in-place pile caps and precast concrete deck panels spanning between the pile caps. Figure 1 shows the deck layout with bayline arrangement. The precast concrete deck panels were solid concrete, designed to be composite with a reinforced concrete topping that was cast over the entire deck, locking the entire deck together and providing a total deck thickness of 600 mm.

While the piles could be driven to achieve a working capacity of 4000 kN, the water depth, substantial thickness of very soft surface mud and secondary bending moments prevented the pipe section of a large proportion of the piles from developing a "column" capacity that could support a load this high . The batter piles were particularly penalized due to the even longer length and higher bending moments. Options available to better utilize the pile section included filling the pile with concrete to develop a composite section, adding piles in the baylines or adding baylines (shortening deck spans) with more piles. Adding more piles under a deck that already appeared congested at the offshore end did not appear to be the practicable solution. Fortunately, the economics alone of concrete cost versus net increased pile capacity demonstrated that concrete filling was the preferred choice.

Battered piles were precluded from being filled with concrete since the bending moments from this additional weight would almost completely consume the structural capacity of the pipe. A trade-off study concluded that unfilled piles battered at 1H:3V was the most practicable and economical for the lateral strength system. The vertical stiffness incompatibility between the concrete filled vertical piles and the unfilled batter piles precluded utilizing the battered piles for significant gravity load capacity. This was particularly desirable, since the already low capacity of the batter piles could be more efficiently utilized for lateral loads on the deck. The decision was made to design the system to take all gravity loads on the vertical piles for static conditions.

It is important to note that the depth of the hard bearing layer for the piles resulted in many of the batter piles passing two adjacent pile bents. This required careful layout of vertical piles and introducing a mild "kick" on the piles to increase the clearance window for batter piles as they passed by during driving, but also to avoid pile tips from encroaching on one another if they either refused shallower or deeper than expected.

Under seismic conditions, ductility in the system was mandatory to allow for manageable design loads. This was achieved by allowing the batter piles, with a pipe of Class 1 (compact plastic design) section and a KL/r for the pile ranging from 90 to 120, to buckle while tension yielding was provided by pile pull-out and yielding of the pile anchors, comprised of bundled, mild steel Dywidag bars. With the vertical piles being designed to accept full gravity loads, this was considered to be a practicable solution since capacity reduction under cyclic buckling would not affect the ability of the deck to support the large storage loads. A ductility factor of 3 was applied to this system.

Batter piles were excluded from the crane rail pile caps to alleviate concern over the very high concentrated loads from the crane wheels over pile caps that would otherwise sustain some level of damage (primarily spalling of the cover and shear cracking) as a result of pile overstrength.

The pile caps with battered piles were designed based on an overstrength capacity of the battered piles. This was considered to be essential to avoid the excessive damage typically associated with pile structures.

DYNAMIC ANALYSIS OF THE STRUCTURE

Procedure

Dynamic analyses of the soil / caisson / piled-deck system were conducted to provide insight into its behavior during the design earthquake. The commercially available two-dimensional computer program FLAC (Fast Lagrangian Analysis of Continua) (ITASCA [1]) with the UBCTOT constitutive model (Beaty [2] and[3]) was used for the analyses. It is a two-dimensional finite difference numerical program developed for analyzing rock, soil and rock/soil/structure systems. Dynamic analyses are conducted in the time domain with very small time steps. Inertial forces are included in the equations of equilibrium solved during each step making the program well suited for analyzing partially stable or unstable systems. The three dimensional aspect of repeating structural members, such as piles, is allowed for by scaling the stiffness and strength parameters proportional to their spacing (in the out of plane direction) and by using 'p-y' curves which allow partial movement of the soil grid past the structural member.

The section analyzed is shown on Figure 3. Soil zones shown on the Figure were assigned the following designations:

1 - Sand and gravel
2 - Bottom mud
3 - Till
4 - Silt
5 - Silt/clay

The dynamic (FLAC) analyses involved the following sequence:

- Establish the grid geometry,
- Calculate elastic soil properties and solve for static equilibrium,
- Calculate and input Mohr Coulomb soil properties, establish water table, pore water pressures and solve for static equilibrium,
- Install the piled deck structure and solve for static equilibrium,
- Change to undrained soil properties and solve for static equilibrium,
- Initiate dynamic analyses by changing to the UBCTOT constitutive model in potentially liquefiable elements, by applying 'free-field' boundaries to ends of model grid and by applying an earthquake time history to the base of the grid,
- During the dynamic analysis the total stress liquefaction triggering model (UBCTOT) evaluates triggering of liquefaction by tracking the dynamic shear stress history on the horizontal plane, $_{xy}$, within each element. The irregular shear stress history caused by the earthquake is interpreted as a succession of half cycles. Each half cycle of cyclic shear stress is transformed into an equivalent number of cycles N_{eq} at $_{15}$, where $_{cyc}$ is the cyclic shear stress required to cause liquefaction in 15 cycles. If the threshold is reached ($N_{eq} \geq 15$) then liquefaction is triggered in the element by changing the soil properties to the post-liquefaction values.
- The analyses are continued to the end of the earthquake time history.

Figure 5 shows a typical FLAC grid. The grid has approximately 7000 soil elements and 615 structural elements. The caisson was modeled using elastic soil elements with reduced density in the half of the caisson which is toward the water in order to allow for that portion only being half filled with soil. Low strain shear moduli (G_{max}) for Zones 1 and 3 were calculated from SPT $(N_1)_{60}$ values assigned to the soil from the drilling results using the following equation from Tokimatsu[4]:

$$G_{max} = 440 \text{ x } P_a \text{ x } ((N_1)_{60})^{1/3} \text{ x } (('_y + 2 \text{ x } '_x) / P_a)^{½}$$

where

P_a = atmospheric pressure and $'_y$ $'_x$ = vertical and horizontal effective stress respectively

For Zones 2, 4 and 5, G_{max} was calculated from the assumed shear wave velocity (V_S) and density values as density x $(V_s)^2$.

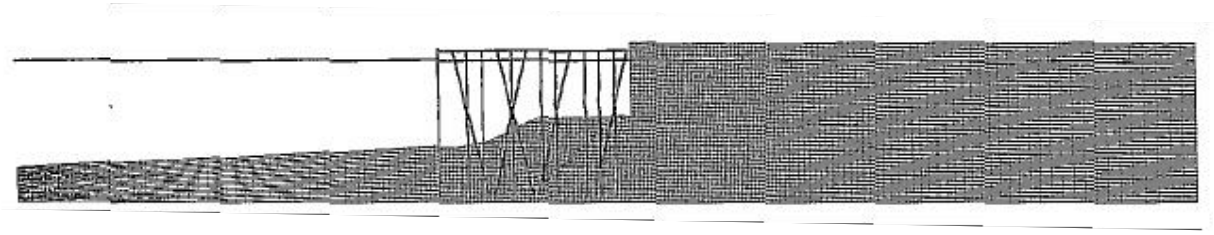


FIGURE 5
Typical FLAC Grid

Shear moduli (G) for the analyses was taken as G_{max} times a strain reduction factor. The strain reduction factor varied from 0.3 to 0.8 and was estimated from a one dimensional dynamic analysis using the program SHAKE91. A random scatter was added to the given $(N_1)_{60}$ with a mean equal to the $(N_1)_{60}$ and a Standard deviation of 0.15 x $(N_1)_{60}$. This was added to include some allowance for the natural variation occurring in the soil. Selected soil and structural properties used in the FLAC analyses are summarized in Tables 1 and 2.

Table 1
SOIL PROPERTIES USED IN FLAC ANALYSES

Zone	Description	(kg/m3)			Su	Friction Angle	Pre-earthquake			During earthquake shaking			
							Max. Shear Mod. Gmax (kPa)	Drained Shear Mod. G (kPa)	Drained Bulk Mod. K (kPa)	Shear wave Velocity Vs m/s	Shear Mod. G (kPa)	Modulus Reduction Factor	Bulk Mod. K (kPa)
1	Sand and Gravel	1800	10 to 50	-	-	33	(1)	Gmax x 0.1	G x 2	-	G max x MRF	0.3 to 0.8	G max
2	Bottom Mud	1600	1	0.15	-	0	1000 Su	G max x 0.1	G x 2	70	G max x MRF	0.6	G max
3	Till	2100	50	-	-	45	(1)	G max x 0.1	G x 2	-	G max x MRF	0.65	G max
4	Silt	1750	2	0.19	-	0	1000 Su	G max x 0.1	G x 2	100	G max x MRF	0.5	G max
5	Clay/silt	1750	10	-	19	0	1000 Su	G max x 0.1	G x 2	120	G max x MRF	0.4	G max

Table 2
STRUCTURAL PROPERTIES USED IN FLAC ANALYSES

Structural Member	Young's Modulus (MPa)	Moment of Inertia (m4)	Area (m2)	Density (kg/m3)	Yield Moment (kN-m)	Perimeter (m)
Pile cap beams and deck	5000	0.64	2.925	2409	elastic	-
Pile above grade	33,000	0.0054	0.053	1310	590	-
Pile in sand and gravel	33,000	0.0054	0.053	1310	590	0.48
Pile in silt	33,000	0.0054	0.053	1310	590	0.48
Pile in till	33,000	0.0054	0.053	1310	590	0.48

Earthquake Time Histories

Four earthquake time histories developed by Dr. Don Anderson of the University of British Columbia for use on bridge projects in the greater Vancouver area were used in the analyses. The records were developed by fitting actual earthquake records to the selected design spectra. The records are deemed to be representative of outcropping firm ground (very dense Pleistocene soil or soft rock) and have a probability of exceedence of 10 percent in 50 years or a return period of 475 years. Following fitting, the records were base-line corrected such that displacements and velocities at the end of the record were zero. The four records used are as follows:

- MIYA-PM = Miyagami subduction earthquake fitted to Prot Mann Bridge (Vancouver) subduction spectrum
- MIYA = Miyagami subduction earthquake fitted to 1999 GSC 1:475 equal hazard spectrum
- 317 = El Centro non-subduction earthquake fitted to 1999 GSC 1:475 equal hazard spectrum
- LPNS = Loma Prieta non-subduction earthquake fitted to 1999 GSC 1:475 equal hazard spectrum

Site specific records were obtained from a one dimensional ground response analysis using the soil profile at the land side of the caisson and the program SHAKE91 (Idriss [5]). The surface response from SHAKE91 was compared to that obtained from a one dimensional FLAC dynamic analysis (with liquefaction triggering inhibited) and the response was almost identical. The LPNS record used for the dynamic analyses is shown on Figure 6.

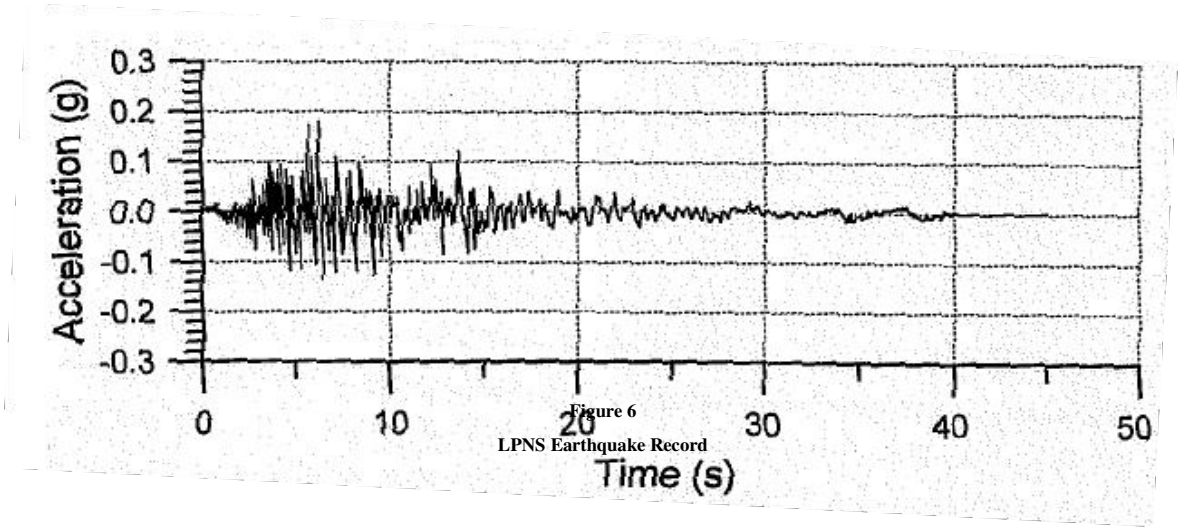


Figure 6
LPNS Earthquake Record

Results

A comparison was made between lateral pile behaviour determined from the computer programs GROUP (ENSOFT[6]) and FLAC. The program GROUP models the soil around the pile using non-linear winkler-type springs. A section in front of the caisson, where the piles pass through predominantly sand and gravel (over till), was used. The top of the pile was assumed to be at the ground surface and pinned in the comparison analyses. In GROUP, the p-y curves were calculated using the method developed by Reese from the Mustang Island pile load tests. Soil and structural properties given in Tables 1 and 2 were used for the FLAC analyses. The pile top deflections from the two methods were similar and maximum pile moments were within 25% of each other.

Twenty one dynamic FLAC analyses, each requiring approximately 24 hours of computer time, were conducted. The conditions assumed for each analysis are identified in Table 3. In summary, the following variables were considered:

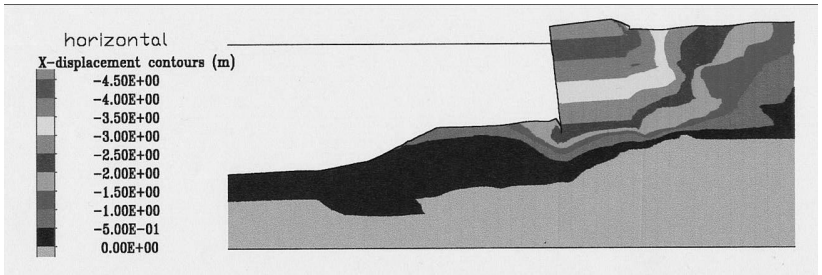
- Full deck mass (69 kPa) or zero deck mass
- Two pile layouts comprising Layout A with a vertical pile within 1 m of the caisson and Layout B, as shown on Figure 3, with a battered pile 1.5 m from the caisson; 5 analyses were completed with no piled deck structure
- 1 m or 1.5 m separation between the caisson and deck

The results of the analyses are presented in Table 3; selected analyses are described below.

Horizontal and vertical displacements calculated for the VTAX3 analysis (no densification; no piled deck; MIYA earthquake) were large (approximately 5 m) and, as in most analyses where no densification was conducted, movement of the caisson continued throughout the duration of strong earthquake shaking. Horizontal displacement are illustrated on Figure 7. A similar analysis with the piled deck in place (analysis VTAX2) limited the movement of the caisson to approximately 1.2m due to the caisson movement being arrested by the piled deck.

Figure 7

Horizontal Displacements for Analysis VTAX3



Densification behind the caisson significantly reduced the caisson movement (to less than 0.3 m) and bending moments in the piles were well below yield. Caisson displacements and residual moments in the piles at the end of the analysis were similar in analyses BX4, CX4, BX5, CX5, BX6 which were either with the 317 or LPNS non-subduction earthquakes, had densification behind the caisson, and had a piled deck with Layout B. Horizontal displacements given by Analysis BX5 are shown on Figure 8. Caisson movements with densification at the toe (analysis BX6) were only marginally less than those without densification at the toe.

The following conclusions were made from the dynamic analyses:

<u>Without densification</u>

- The stability of the existing caisson during the 1:475 year design earthquake is marginal and lateral displacements of several metres may occur.
- The proposed pilings located directly adjacent to the caisson will be pushed laterally near the mud-line and loads on the piles will increase substantially.

<u>With densification</u>

- The stability of the existing caisson during the 1:475 year design earthquake is marginal and lateral displacements of several metres may occur.
- Densification in front of the caisson, in addition to behind, has little added benefit.

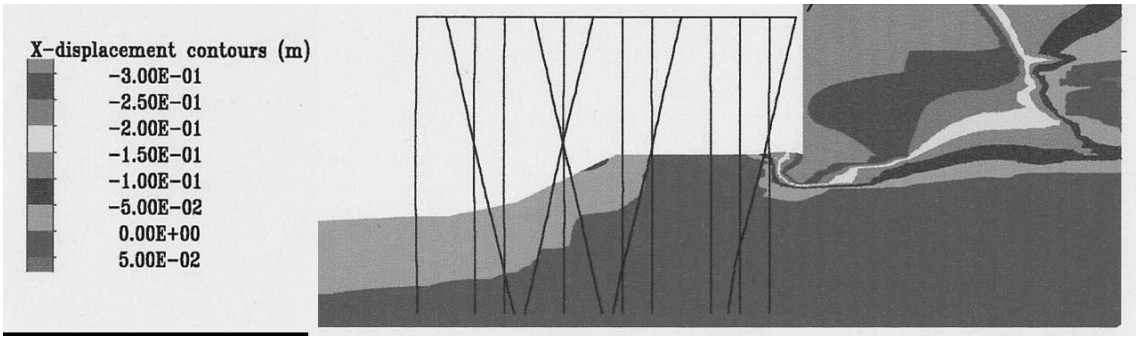


Figure 8
Horizontal Displacements for Analysis BX5

GROUND IMPROVEMENT

Options

Ground improvement was required behind Caissons 17 and 18 to prevent liquefaction of loose saturated backfill during the design earthquake. The improvement measures were required to extend 15 m back from the inside wall of the caisson and to the full depth (up to 35 m) of fill placed above the till or bedrock surface. The options considered for ground improvement were densification by vibroreplacement (stone columns) using top feed or bottom feed techniques or installation of large diameter vertical drains, referred to locally as seismic drains. Both methods allow vertical drainage to promote pore pressure dissipation and prevent liquefaction during the seismic event.

Prior to issue of tender documents, the seismic drain option was selected because of the presence of minus 600 mm diameter rockfill and the greater certainty of successfully installing drains to 35 m depth at the till or bedrock surface as compared to the expected difficulty of completing stone columns to that depth.

Seismic Drain Installation

The drains were specified to be 500 mm diameter and to be installed on a grid of 2.5 m triangular spacing for a total of 146 drains. Installation was required to be completed by installing a closed-end drill casing, thus displacing and densifying the loose soils being penetrated by the casing.

A break-away steel end plate was used to cover the bottom of the casing during driving. The end plate was left in the hole when the casing reached the required depth, being driven by a hydraulic hammer and extracted with a vibrohammer.

The casing was gradually withdrawn while being filled with drain rock of 38 mm to 4 mm size to form a continuous, free-draining zone within the original backfill behind the caissons. The top of the drains was covered by a 1 m thick drain rock layer to further encourage drainage and prevent uplift of the finished asphalt pavement.

Ground and Caisson Displacements

The considerable energy transmitted into the ground by the diesel, vibro and hydraulic hammers during pile driving and seismic drain installation was expected to cause densification of the loose backfill and rock mattress and settlement of the ground surface behind the caisson and displacement of the caissons. Pile installation was not allowed to start until the seismic drains were completed, to avoid loading of the piles due to caisson movement. Ground settlement behind the caissons recorded during drain installation was higher then expected, reaching up to 600 mm. Displacement of Caissons 17 and 18 reached as much as 50 mm laterally and 70 mm vertically. Lateral ground movement behind the caissons extended to about 30 m from the back of the caissons, as indicated by cracking of the existing pavement.

Table 3

SUMMARY OF DYNAMIC FLAC ANALYSIS

File Name	Earthquake Time History	Densification: At Toe of Caisson	Densification: Behind Caisson	Caisson-Deck Separation	Pile Layout	Deck Mass Included	Loglog Slope	Post-liquef. Limiting Shear Strain Factor	Caisson Displacement (m): Lateral: Top	Caisson Displacement (m): Lateral: Bottom	Caisson Displacement (m): Vertical: Front	Caisson Displacement (m): Vertical: Back	Maximum Stress In Piles At End Of Earthquake Or Analysis: Bending Moment (kN-m)	Maximum Stress In Piles At End Of Earthquake Or Analysis: Shear (kN)	Maximum Stress In Piles At End Of Earthquake Or Analysis: Displac. (m)	Remarks
					(2)	*(3)*	*(4)*	*(5)*					*(6)*			
VTAX1	MIYA-PM	NO	NO	1m	A	YES	-2.9	2.0	-1.377	-1.439	-0.135	-0.181	7800	2580	-0.717	No yielding in structure - bad geometry@ 41sec
VTAX2	MIYA	NO	NO	1m	A	YES	-2.9	2.0	-1.142	-1.016	-0.178	-0.079	3540	2460	-0.588	
VTAX3	MIYA	NO	NO	N/A	NONE	N/A	-2.9	2.0	-4.907	-2.336	-1.509	-0.221	N/A	N/A	N/A	Analysis stopped at 40s due to bad geometry
VTAX4	MIYA	YES	YES	N/A	NONE	N/A	-2.9	2.0	-0.748	-0.488	-0.181	-0.001	N/A	N/A	N/A	
VTBX4	317	YES	YES	N/A	NONE	N/A	-2.9	2.0	-0.247	-0.12	-0.076	-0.013	N/A	N/A	N/A	
VTCX4	LPNS	YES	YES	N/A	NONE	N/A	-2.9	2.0	-0.287	-0.138	-0.092	-0.012	N/A	N/A	N/A	
VTC6X4	LPNS	YES	YES	N/A	NONE	N/A	-6.0	2.0	-0.97	-0.822	-0.213	-0.11	N/A	N/A	N/A	
BX2	317	NO	NO	1m	A	YES	-2.9	2.0	-0.978	-1.083	-0.036	-0.11	3540	2460	-0.318	Analysis stopped at 9sec due to bad geometry
CX2	LPNS	NO	NO	1m	A	YES	-2.9	2.0	-0.962	-0.882	-0.093	-0.026	3540	2160	-0.281	Analysis stopped at 15sec due to bad geometry
BX3	317	NO	NO	1.5m	B	YES	-3.6	1.0	-1.564	-2.454	-0.067	-0.635	3360	1236	-0.193	Analysis stopped at 29sec due to bad geometry
CX3	LPNS	NO	NO	1.5m	B	YES	-3.6	1.0	-1.629	-1.911	-0.27	-0.374	2700	1140	-0.19	
BX4	317	NO	YES	1.5m	B	YES	-3.6	1.0	-0.327	-0.318	-0.076	-0.063	1020	540	0.06	
CX4	LPNS	NO	YES	1.5m	B	YES	-3.6	1.0	-0.379	-0.373	-0.086	-0.073	1020	504	-0.046	
BX5	317	NO	YES	1.5m	B	NO	-3.6	1.0	-0.348	-0.33	-0.083	-0.065	900	438	-0.049	
CX5	LPNS	NO	YES	1.5m	B	NO	-3.6	1.0	-0.356	-0.337	-0.077	-0.059	1080	498	-0.058	
BX6	317	YES	YES	1.5m	B	NO	-3.6	1.0	-0.302	-0.255	-0.66	-0.048	1020	450	-0.056	
CX4R	LPNS	NO	YES	1.5m	B	YES	-3.6	1.0	-0.25	-0.21	-0.06	-0.04	1056	-	-0.07	p-y curves softened in piles in liquefied elements
CX5R2	LPNS	NO	YES	1.5m	B	NO	-3.6	1.0	-0.26	-0.22	-0.07	-0.05	684	354	-0.07	p-y curves softened in piles in liquefied elements
CX7R	LPNS	NO	YES	N/A	NONE	N/A	-3.6	1.0	-0.25	-0.18	-0.07	-0.02	N/A	N/A	N/A	
CX7S	LPNS	NO	YES	N/A	NONE	N/A	-3.6	1.0	-0.22	-0.19	-0.06	-0.04	N/A	N/A	N/A	Similar to Cx7R but shear modulus of till increased
CX7T	LPNS	NO	YES	N/A	NONE	N/A	-3.6	1.0	-0.33	-0.35	-0.07	-0.08	N/A	N/A	N/A	Similar to Cx7S but buoyant unit weights used

PILE INSTALLATION

Installed Piles

A total of 131 piles of 914 mm diameter and 19 mm wall thickness were driven closed ended using a barge-mounted Delmag D62 diesel hammer. The piles varied in length from 25 to 60 m and were driven to embedment depths of 10 to 20 m below mud line. Fifty of the piles were vertical, 44 were battered at 1H:20V and 37 were battered at 1H:3V. The 1H:3V piles had two mild steel Dywidag thread bars with Double Corrosion Protection(DCP) installed to between 12 m and 25 m below the pile tips to enhance the tension capacity of the piles.

Pile Driving Analyzer (PDA) testing was carried out on four of the first 20 piles driven to establish pile capacity at the end of initial driving (EOID). Four piles were retested using PDA 2 to 3 weeks after initial driving to determine the ultimate capacity at the beginning of restrike (BOR). CAPWAP analysis was utilized to establish pile capacity. The results are summarized in Table 4.

Environmental Concerns

Environmental requirements imposed by Fisheries and Oceans Canada on the 2001 construction were to achieve zero fish kill during pile driving. Pressure waves developed during driving of large diameter piles were shown by earlier pile driving at Canada Place to be lethal to fish and a bubble curtain system was developed to minimize pressure wave transmission. The Vanterm contractor adopted a similar system to that developed at Canada Place, comprising several perforated plastic rings set around the pile at varying depths in the water. Compressed air pumped down to the rings created a curtain of bubbles surrounding the pile as they rose to the water surface. Piles at 1H:3V batter presented some difficulty in maintaining the curtain over the entire length of the pile above the mud line. Occasional fish kill occurred, primarily when tidal currents caused the air bubbles to drift past the pile, allowing pressures of up to 80 kPa to be measured by hydrophones.

Table 4
SUMMARY OF PDA RESULTS

Pile	PDA Test Condition	Hammer	Ultimate Pile Capacity (kN)			Blows for Last 300mm
			Shaft	Toe	Total	
1G	BOR	Drop	3200	5800	9000	344
1N	EOID	D62*	2300	3700	6000	297
	BOR	Drop	3500	700	11200	
1P	EOID	D62*	2320	3730	6050	270
	BOR	Drop	2310	7190	9500	
2H	BOR	Drop	3200	6200	9400	346
3P	EOID	D62*	1325	3085	4390	855
	EOID	Drop	2300	7200	9500	

* Hammer not able to prove ultimate capacity

Anchor Installation

Some difficulty was encountered in installing the anchors through the end of the battered piles. The specifications required casing to be used "to maintain an open hole where necessary " when drilling the 250 mm diameter hole required for anchor installation up to 25 m below the pile tip. The problem resulted from fine sand below the pile tip entering the bottom of the pile through the small annulus between the drill casing and the sleeve in the pile, despite adoption of several different drilling methods. The loss of sand from below the pile tip also caused some concern regarding the bearing capacity of the piles. Pressure grouting of the anchors was carried out, if necessary, to achieve the 2100 kN anchor capacity that was required to be demonstrated by anchor testing. Grout tubes were also installed to the pile tips to allow pressure grouting of any disturbed soil.

ACKNOWLEDGMENTS

The authors wish to acknowledge the permission of the Vancouver Port Authority to publish this paper and the assistance provided by the contractor, Vancouver Pile Driving Ltd., regarding construction aspects of the project. The authors also wish to thank Dr. Peter Byrne for his review and guidance with the numerical analyses.

REFERENCES

1. ITASCA “FLAC - Fast langrangian analysis of continua, Version 3.4.” ITASCA Consulting Group Inc., Minneapolis, Minnesota, 1996.

2. Beaty, M., and, Byrne, P.M. "A simulation of the Upper San Fernando dam using a synthesized approach." Proceedings of Slope Stability and Landslides, 13th Annual Vancouver Geotechnical Society Symposium, 1999: 63-72.

3. Beaty, M. "A synthesized approach for estimating liquefaction-induced displacements of geotechnical structures." Ph.D. Thesis, University of British Columbia, 2001.

4. Tokimatsu, K.A.M. and Seed, H.B. “Evaluation of Settlement of Sands Due to Earthquake Shaking, “Journal of Geot. Engineering, ASCE, Vol. 113, No. 8, pp. 861-878, 1987.

5. Idriss, I.M. and Sun, J.I.. “SHAKE91, A computer program for conducting equivalent linear seismic response analyses of horizontally layered soil deposits” University of California, Davis, 1992.

6. ENSOFT, “GROUP - A program for the analysis of a group of piles subjected to axial and lateral loading.” Ensoft Inc., P.O. 180348, Austin, Texas, 1996.