Ground improvement for Cruise Ship Terminal Expansion Project at Canada Place, Vancouver, BC

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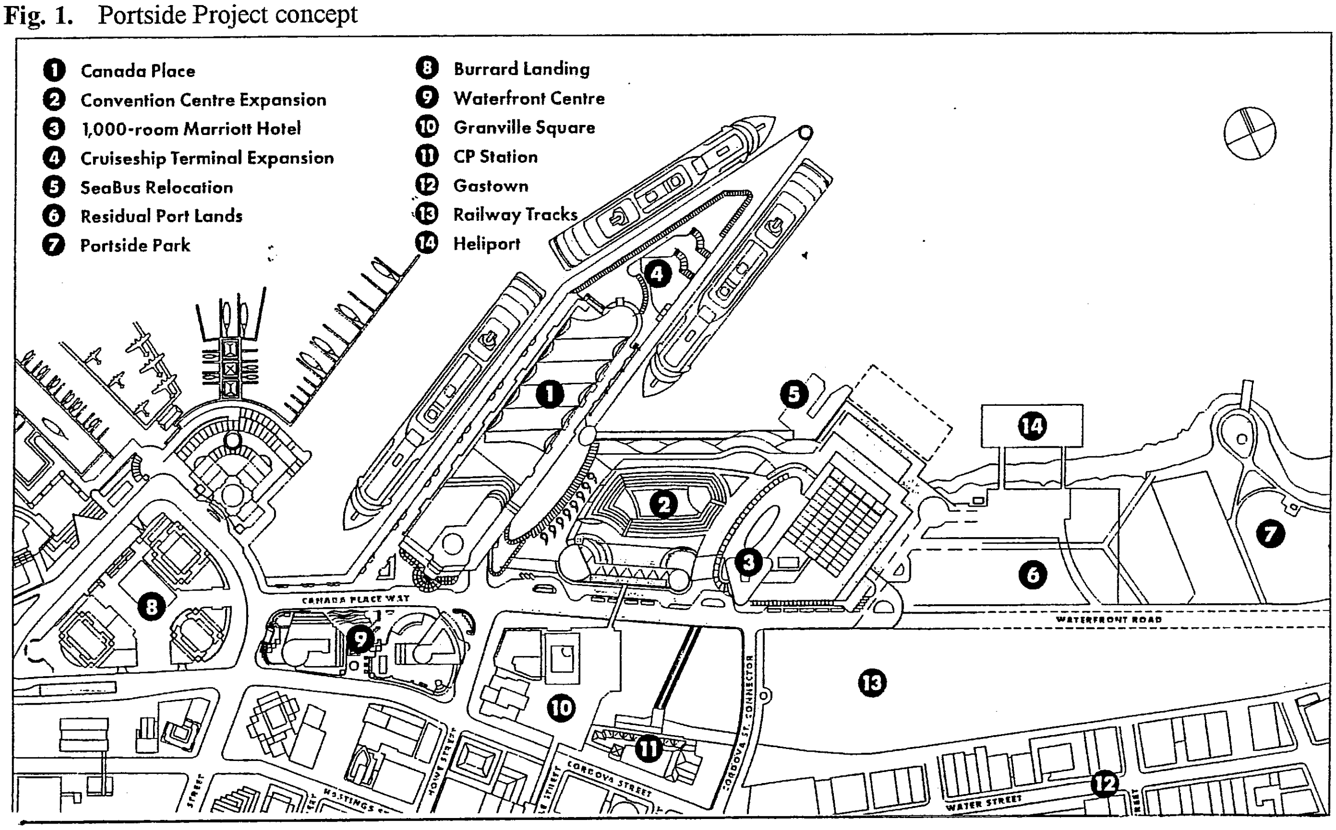
Abstract: Canada Place was built over the original Pier B-C which is underlain by granular fill up to 30 m thick. The fill was dumped underwater into a trench excavated to remove deep, soft silt from above the bedrock or glacial till surface. The existing structures and the apron, which runs around the perimeter, are pile supported. Most of the piles are between 8 and 10 m long and terminate in the fill. The Terminal Expansion at the offshore end of Canada Place will provide additional cruise ship berthing and visitor facilities on a pile-supported deck structure. The expansion area was shown by Becker hammer, auger and sonic drilling to be underlain by loose, liquefiable granular fill and a variable thickness of soft silt between the fill and the till surface. The silt unit reached a maximum thickness of 15 m at the offshore end where it was not covered by fill. Ground improvement measures were required to provide acceptable ground behaviour for the Terminal Expansion foundations during the design 475 year earthquake. Offshore of the existing apron, vibro-densification of the granular fill and self-feeding of gravel into the silt unit were utilized. Under the apron surrounding the existing structure, 145 seismic drains were installed to 13 m depth in the granular fill to prevent liquefaction.

Introduction

In the mid 1990s, the Portside Project proposal was developed to expand the Vancouver waterfront in the vicinity of Canada Place, as shown on Figure 1. The proposal incorporated extension of Canada Place to provide additional cruise ship facilities, combined with a new convention centre on the east side of Canada Place and a 1000 room hotel at the east end of the project area. As part of this comprehensive development, the existing SeaBus terminal was to be temporarily located to the Heliport site and, subsequently, returned to the convention centre to be incorporated into the facility. The planning, architectural, and engineering studies for the proposal were managed by Greystone Properties Ltd. (renamed Concert Properties Ltd. in 1999) and funded by the BC Provincial Government. The Province cancelled the project in October 5, 1999 due, in part, to a lack of commitment of Federal Government funds to the overall $1 billion cost. However, a portion of the

Fig. 1. Portside Project concept



project survived due to the need for improved cruise ship facilities in Vancouver. The Vancouver Port Authority assumed responsibility for the Cruise Ship Terminal Expansion Project utilizing much of the conceptual and detailed design work that had been completed under Concert Properties' mantle.

This paper is concerned solely with the Terminal Expansion Project. The soil conditions which influenced foundation design for the 10,000 m^2 pile-supported deck and the associated ground improvement measures required to assure public health and safety during the 475 year design earthquake are described.

History

Canada Place was built in time for Expo 86 over the original Pier B-C. The Pier was built in the 1920s by dredging soft silt from a trench along the Pier alignment. The trench reached a maximum depth of about 15 m at the offshore end and was filled by granular fill dumped from scows. The fill was dredged from the First Narrows at the mouth of the Capilano River. Below tide level, the fill was end dumped from the scows. Above tide level, the fill was side cast. The gravel fill was built to about 20 m above the adjacent sea bed, with a crest width of 30 m and side slopes of between 2H:1V and 3H:1V.

The Pier B-C structure was supported on 6000 precast, solid concrete piles driven to about 8 m embedment into the granular fill. The piles varied in size from 380 to 585 mm square and were driven by hammers varying in energy from 20 to 75 kJ. It was reported that considerable difficulty was encountered during pile driving in keeping the piles on line and avoiding damage. The reason for these difficulties is not clear but is not evidently related to the coarseness of the fill or to its density, though it is presumed that pile driving would have densified the top 10 m or so of the fill. A cross-section through Pier B-C, taken near the northern end, is shown on Fig. 2 and illustrates the trench excavation, fill and layout of the piles supporting the deck.

When Canada Place was built in the 1980s on top of the Pier, the structure was supported by additional piles, generally expanded base type, driven to a similar 8 m embedment depth. Heavily loaded columns, such as those under the masts of the sails, were taken down to the underlying glacial till. Considerable distress was caused to the Pier B-C deck by the 1980s pile driving due, apparently, to densification of the fill and settlement of the original piles. The distress was repaired by a levelling overlay to the concrete slab and epoxy and shotcrete repairs to the substructure.

The Canada Place structure left a 10 m wide pile supported apron around the east, west and north sides of the structure for gangways and service trucks required for handling cruise ships.

Soil conditions

Soil conditions at Canada Place have been investigated by Vancouver-based geotechnical engineering firms and contractors since 1967. The investigations have included:

- Drilling and penetration testing using mud-rotary, Becker hammer and sonic drills from 1980 through 1999
- Test pile driving in 1981, 1996 and 1998
- Pile load testing in 1981 and 1998

Test hole locations near the end and offshore of Canada Place Pier are shown on Fig. 3. Logs of THs 98-31 and 98-34, typical for the expansion area, are shown on Figs. 4 and 5.

The fill is a clean sand and gravel with some cobbles to about 150 mm maximum diameter. A gradation envelope for 22 of 24 Standard Penetration Test samples from 4 boreholes completed by Golder Associates (1980) and 4 samples from Becker holes reported by Thurber (1999) is shown on Fig. 6. The fill is generally in loose to very loose condition, with a corrected Becker Penetration Test (BPT) blow count of about 10.

The native silt which underlies the fill along the east, west and north sides of the existing pier, is a remnant of the 1920s dredging outside of the Pier B-C footprint. The silt is variable in consistency and gradation.

Fig 2. Typical cross-section through Pier B-C

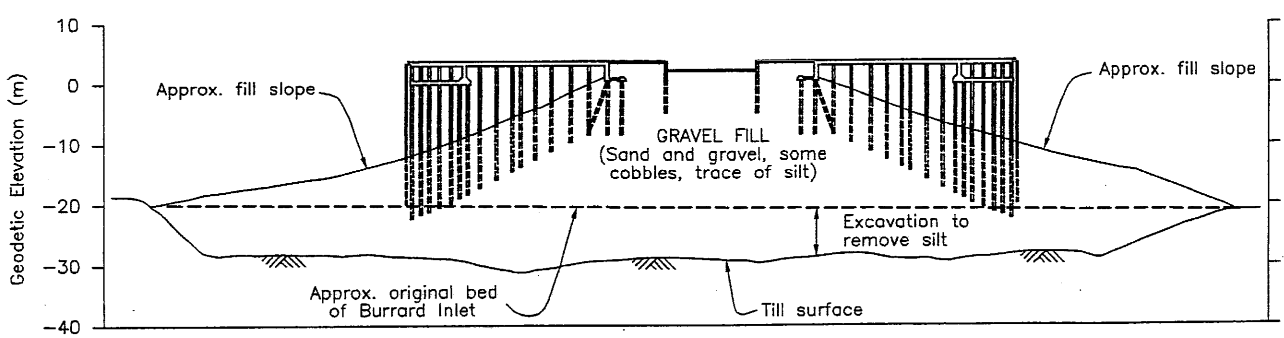


Fig. 3. Test hole locations at north end of Canada Place

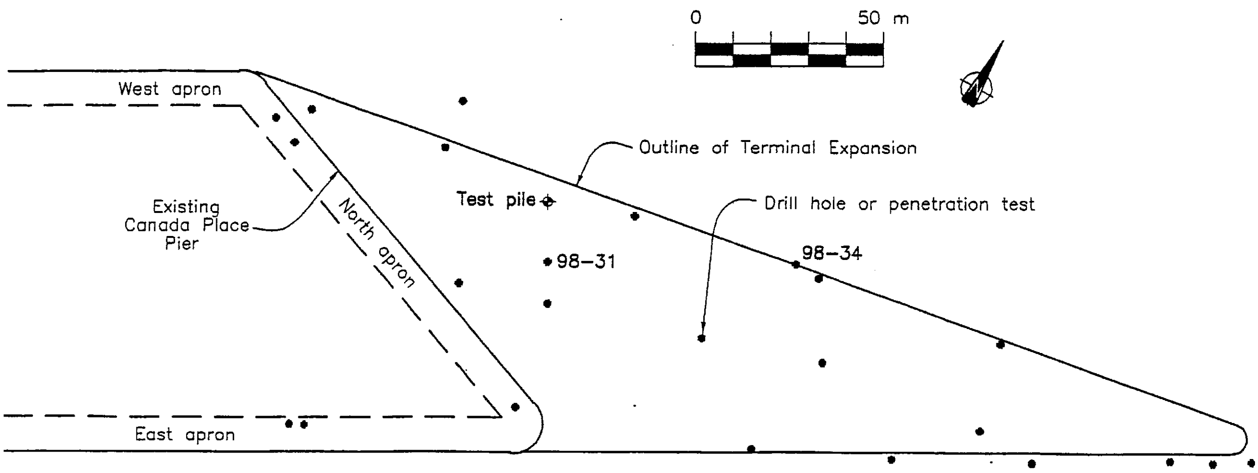


Fig. 4. Log of TH 98-31

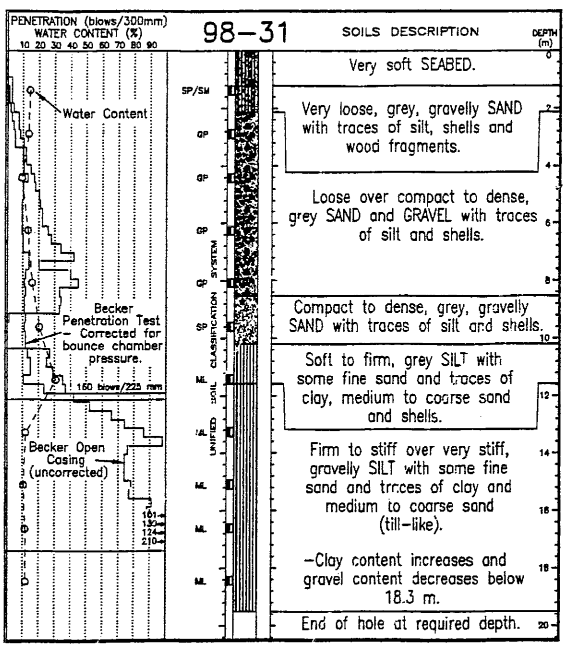
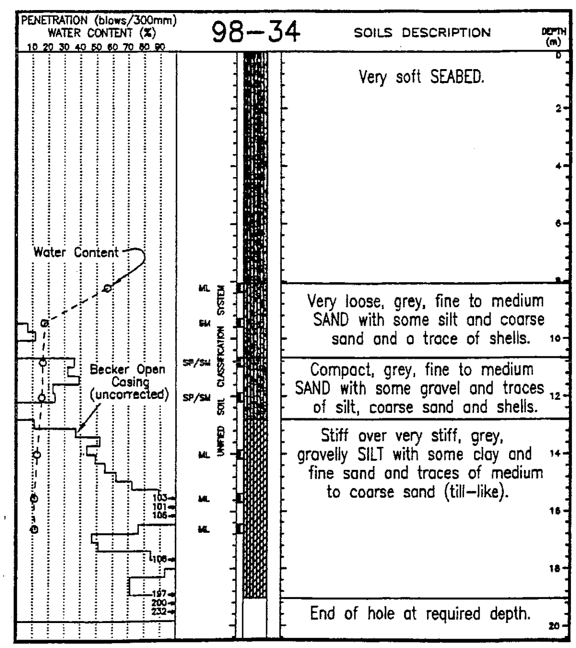


Fig. 5. Log of TH 98-34

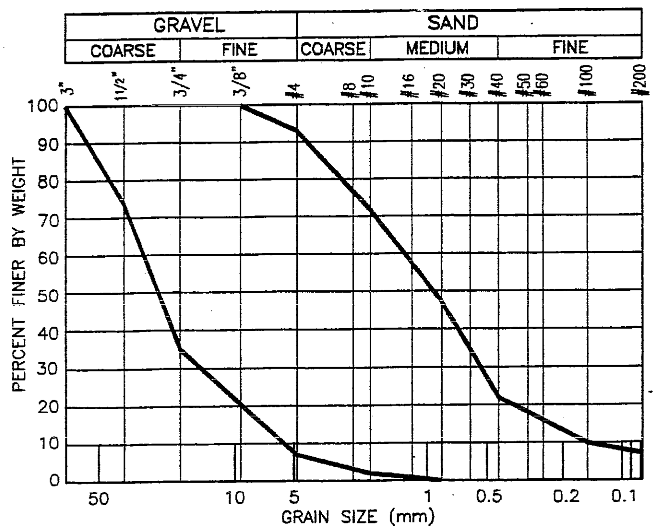


Underlying the fill and silt in the expansion area is glacial till, varying in gradation from silty sand to sandy silt and with variable content of gravel, cobbles and boulders, up to about 500 mm maximum diameter. In general, the upper 1 to 2 m of till is weathered with lower blow counts than in the underlying, denser material, which often has a blow count well in excess of 100/305 mm. Blow counts in the till, however, can dip to 50 or less, indicating some variability.

Significance of existing soil conditions

Without ground improvement, the loose fill and soft silt which underlie Canada Place and the Terminal Expansion area would undergo substantial settlement and lateral ground displacement during a 475 year earthquake with a maximum firm ground acceleration of 0.21g. These movements are expected to be sufficient to cause failure of any structure built at grade or on pile foundations installed through the unimproved fill. Therefore, it was considered that ground improvement was required to increase the insitu density of the fill and the strength of the native silt which underlies the fill. Vibrocompaction of the fill and construction of stone columns in the silt were, conceptually, the selected ground improvement measures. This work was to be confined to the expansion area offshore of the north apron to avoid opening up holes at close spacing in the deck and prevent, to the maximum extent practicable, settlement and damage to the existing piles and deck resulting from compaction of the fill.

Fig. 6. Gradation envelope for 24 fill samples from drill holes



To investigate the efficacy of ground improvement, Dr. P.M. Byrne was retained to complete FLAC analyses of transverse and longitudinal sections through Canada Place for the 475 year earthquake. The analyses were based on the drilling results, combined with consideration of the procedures used in the 1920s dredging and fill placement operations. A critical component of the FLAC analyses was the assumed strength of the silt unit. Therefore, a sensitivity analysis was carried out using a range of silt strengths to investigate the effect on horizontal displacements.

Results of FLAC analyses

The schematic longitudinal cross-section in the expansion area used for the FLAC analyses is shown on Fig. 7. Soil properties used in the analyses are listed in Table 1. Silt strengths in zone 4, defined by $^{Su}/_p$ equal to 0.125, 0.3 and 0.4 were utilized. The results of the FLAC analysis for $^{Su}/_p$=0.125 are illustrated on Fig. 8, which shows large displacements (of 3.3 m) in the liquefied (Zone 1) fill, causing this material to over-ride the densified (Zone 3) fill. Horizontal displacement of the densified fill leads to passive failure of the soft silt (Zone 5).

Table 1
Soil properties used for FLAC analysis

Zone	Material	ϕ (o)	c (kPa)	γ (kN/m³)	$(N_1)_{60}$
1	Loose fill	33	0	18	10
3	Densified fill	36	0	19	18
4	Densified silt	25	0	17	10
5	Soft silt	0	5	16	1
6	Till	45	0	21	

If $^{Su}/_p$ is increased to 0.3 or 0.4, large (2 m) displacements still occur in the liquefied (Zone 1) fill as it over-rides the densified (Zone 3) fill but the displacements elsewhere, and particularly in Zone 5, are much less. A maximum feasible upper bound strength of $^{Su}/_p$=0.4 in Zone 4, achieved by 50% replacement of the silt by stone columns, was targeted to limit the horizontal displacement of the densified fill block to 150 mm. This displacement occurs entirely within the silt unit, so that the densified fill block remains virtually intact as it slides out. Piles driven through the fill and silt into the till will move with the fill block and undergo the 150 mm displacement primarily within the silt layer. This behaviour was of considerable concern to pile design with regard to stresses developed in the pile.

A subsequent FLAC analysis was carried out to investigate the behaviour of the fill under Canada place if it were prevented from liquefying by allowing drainage (by provision of seismic drains). This analysis showed that the maximum displacement of the Zone 1 surface was 300 mm, compared to 2.2 m if it were allowed to liquefy. This result was used to establish the efficacy of seismic drains in limiting damage to the northern end of the existing Canada Place structure.

Fig. 7. Schematic longitudinal cross-section

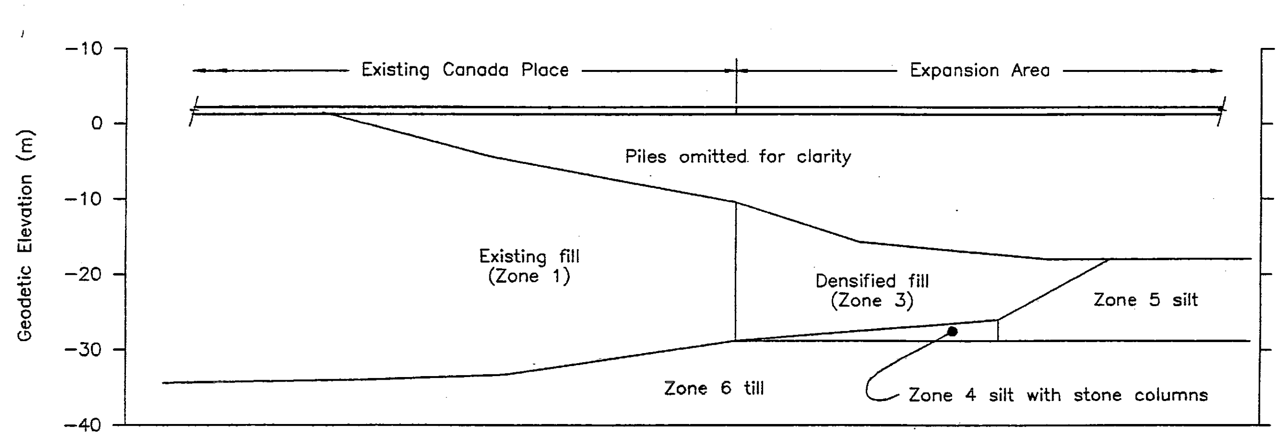
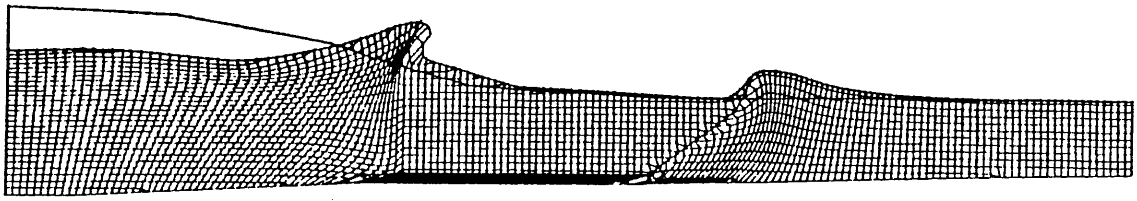


Fig. 8. Deformations at end of Canada Place estimated by FLAC (with exaggerated grid distortion).



Ground improvement offshore of the Canada Place north apron

Objectives and general procedures

The principal objectives of ground improvement offshore of the north apron were to densify the loose granular fill to make it non-liquefiable and to strengthen or stiffen the underlying silt unit to minimize earthquake induced displacements. To achieve these objectives, specifications were prepared requiring vibro-densification using bottom feed procedures on a 2 m triangular grid to replace 50% of the silt by gravel in the form of a stone column. This procedure required the silt to be flushed through the overlying granular fill into Burrard Inlet (to be constrained by a silt curtain) whilst introducing stone into the resulting void through the bottom feed. While this procedure was judged to be feasible, the close spacing of the probe holes required to achieve 50% replacement and the larger diameter of the bottom feed vibroflot increased the difficulty of ensuring penetration and retrieval of the vibroflot during densification. Consequently, prior to award of the marine work contract to Fraser River Pile & Dredge Ltd. with Geopac West Ltd., as densification subcontractor, discussions were held to establish if alternative, acceptable procedures were available. As a result of these discussions, it was agreed that self feeding of the granular fill into the silt using multiple penetrations of the vibroflot in the silt unit would be adopted. The maximum thickness of silt able to be treated in this manner was judged to be 3 m. The limits of densification off the end of Canada Place are shown on Fig. 9. A schematic cross-section, showing the densification zones is shown on Fig. 10.

Results

Densification of approximately 50,000 m ^{3} of fill and 10,000m^3 of silt was achieved by means of a total of 1250 probes using a VFAG V23 vibroflot. A 2 m triangular probe spacing was utilized for densification of the fill and silt in the area with an expected silt thickness or 3 m of less while the spacing was increased to 3 m further offshore where the silt was expected to be too thick to improve by self-feeding with the overlying gravel fill.

A test program was carried out to establish an acceptable procedure. The program was carried out over a 100 m^2 area, involving about 30 probes. BPTs were carried out before and after densification to identify the top and bottom of the silt unit, using previous test hole results as a guide. The silt unit was also identified during initial vibroflot penetration at each probe hole by a sudden reduction in amps operating the vibroflot motor from as high as 300 in gravel to about 85 in the silt. At the till surface, the ammeter reading typically rose to above 200. Once the silt unit was identified, the vibroflot was raised and lowered

Fig. 9. Densification plan

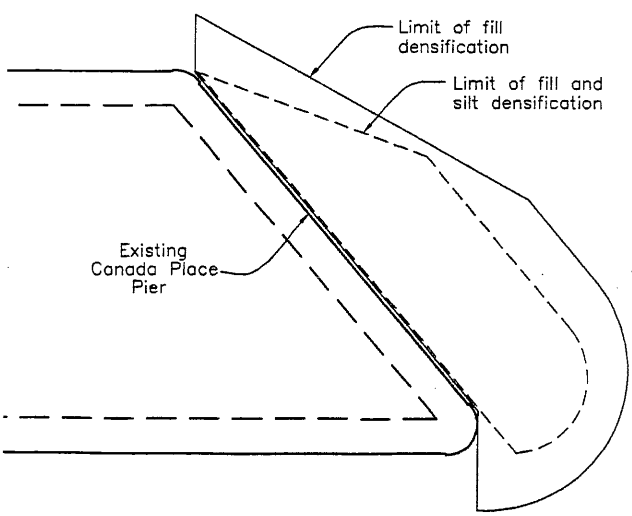
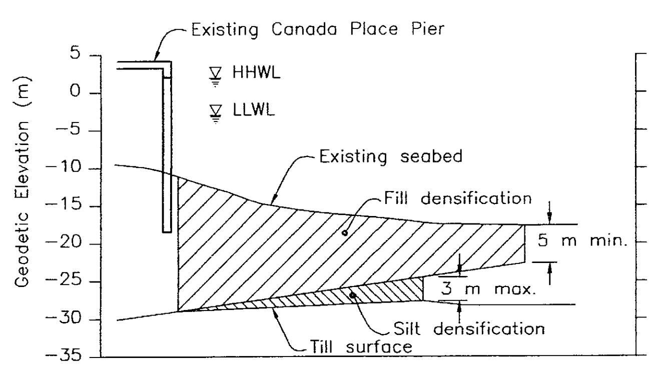
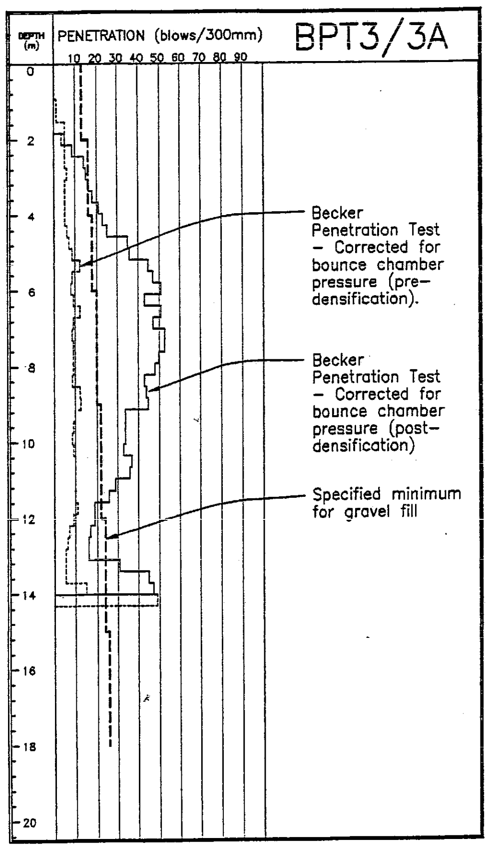


Fig. 10. Schematic cross-section of densification



several times in this unit, attempting to increase the amp reading to150 each time. It was our belief that gravel was being drawn into the silt unit to replace the silt that was being washed up the probe hole. Typical BPT results before and after vibro-densification in the test area are shown on Fig. 11. It is evident that the 2 m spacing achieved much higher density than that required to prevent liquefaction of the fill, despite the vibroflot being raised rapidly through the fill after densification of the silt was completed. This also led to refusal of the vibroflot above the silt surface on first penetration in a small proportion of generally widely spaced probes. Densification of the fill zone only at 3 m spacing was also successful.

Fig. 11. Typical BPT results



Seismic drain installation

The FLAC analyses indicate that the zone of liquefaction of the existing fill under the Canada Place Pier during the 475 year earthquake is below the 8 m embedment depth of the piles which support the existing building, except at the extreme north end of the building. Thus, plunging failure of the piles under the building, leading to collapse of the structure, is not likely to occur, though fill and pile displacement and building damage are likely. However, the liquefiable zone extends for the entire fill depth under the 10 m wide apron around the east, north and west sides and would result in plunging failure of the piles under the deck.

Thus, it was concluded that ground improvement was required under the entire north apron and along the east and west aprons for about 50 m from the ends to prevent collapse of the new structure to be built over the apron. Densification from deck level was ruled out because of the close spacing of the openings required in the concrete deck, interference with probe distribution caused by the extensive network of grade beams under the deck and the likelihood of settlement of the existing piles and resultant structural damage caused by densification. Therefore, it was determined that drainage was a better solution for preventing liquefaction.

Installation of gravel drains to prevent liquefaction of loose granular soils was pioneered by the Japanese, with the first reported installation to be in 1978 at an ore unloading wharf in Tokyo Bay (Sonu et al., 1993). By 1992, 200,000 seismic drains had been installed in Japan with no verification of their efficiency. However, in January 1993, a Magnitude 7.8 earthquake struck the Port of Kushiro on the south eastern shore of Hokkaido. Seismic drains, which has been installed at 6 locations in the port during a retrofit some years before, were effective in preventing liquefaction of the fill at these facilities, despite a peak ground acceleration of 0.47 g. Damage to unprotected Port facilities was extensive.

More recently, drains were installed at the Vancouver Airport terminal building to prevent liquefaction of loose Fraser River sand and loss of support to the pile foundations. However, no marine structure in the Vancouver area is known to have been seismically protected by installation of drains similar to those used at the Cruise Ship Terminal.

To establish the design flow for the drains, a FLAC analysis was carried out by Dr. P.M. Byrne assuming a depth of 13 m in the fill, a diameter of 305 mm and a hydraulic conductivity (permeability) of the granular fill of 0.1 cm/s. The 13 m depth was selected to ensure that liquefaction was prevented to several metres below the existing and new piles supporting the apron. This permeability was selected from the results of gradation tests carried out on samples of the fill obtained from drill holes and constant head permeability tests carried out by Golder Associates (1980). The FLAC analysis indicated that drains at 4 m spacing would be adequate to prevent the development of significant excess pore pressure in the fill in the event of a 475 year earthquake, providing the vertical capacity of the drains was adequate to prevent the expected water to flow unchecked to the surface. For the 13 m depth of the drains, the FLAC analysis indicated a flow of 0.02 m^3/s from each drain. Furthermore, the drain was required to have a permeability of 10 cm/s. To achieve these objectives, a 300 mm diameter drain was specified, comprised of a 100 mm diameter stainless steel well screen surrounded by a 100 mm annulus of minus 38 mm drain rock.

Installation from the aprons was accomplished by driving a 305 mm pipe pile fitted with a knock off end plate, using a drop hammer. The well screen was inserted in the pile with the top projecting 1 m above the sea bed and the pile withdrawn by a vibrohammer, as drain rock was placed in the annulus. Drain spacing was idealized to be 4 m, but actual locations were required to avoid the considerable array of beams supporting the deck, as shown on Fig. 12. A total of 145 drains was installed.

Acknowledgements

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References

Golder Associates, 1980. Phase I Geotechnical Investigation - Proposed Pacific Rim Trade/Convention Centre, Vancouver, BC. Report to Pier B-C Development Board Ltd.

Sonu, C.J.,K. Ito and H. Ooshi, 1993. Harry Seed, Liquefaction and the Gravel Drain. Civil Engineering, December 1993.

Thurber Engineering Ltd., 1999. Portside Project, Vancouver, BC - Geotechnical Investigation. Report to Concert Properties Ltd.

Fig. 12. Seismic drain layout and details