Foundations for the Cruise Ship Terminal Expansion Project at Canada Place, Vancouver, BC

Dave Smith
Thurber Engineering Ltd., Vancouver, BC.

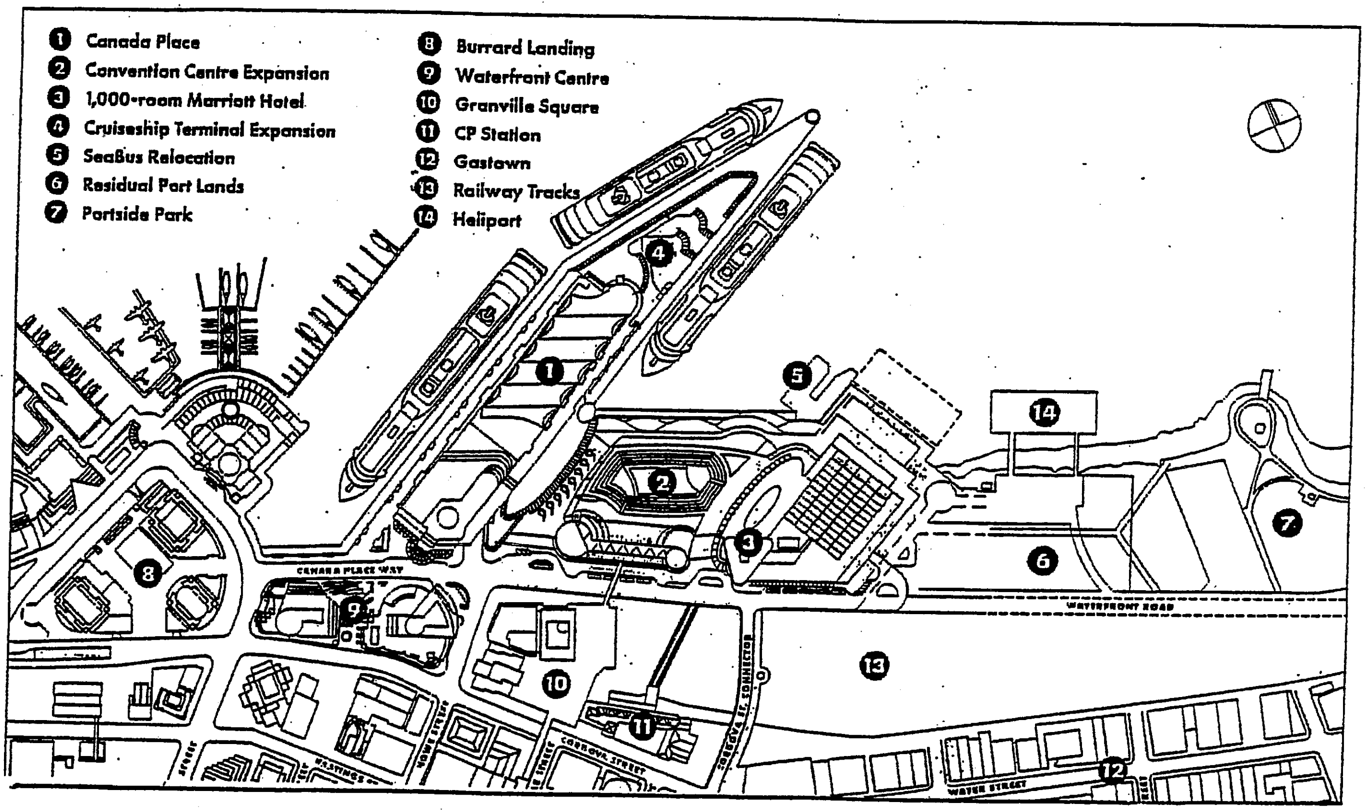
Abstract: Canada Place was built over the original Pier B-C which is underlain by granular fill up to 30 m thick. The fill was dumped underwater into a trench excavated to remove deep, soft silt from above the bedrock or glacial till surface. The existing structures and the apron, which runs around the perimeter, are pile supported. Most of the existing 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 over glacial till. The silt unit reached a maximum thickness of 15 m at the offshore end of the Expansion area where it was not covered by fill. Vertical and batter piles of 610 mm and 914 mm diameters, mainly driven closed-end, were designed for installation in and through the densified fill and silt, generally to be end bearing in the till unit. Soil anchors were designed for the batter piles to resist seismic uplift forces developed by the completed structure.

Introduction

In the mid-1990s, the Portside Project proposal was developed to expand the existing Canada Place to provide additional cruise ship facilities together with a new convention centre and a 1000 room hotel at the east end of the project area, as shown on Fig.1. The project was cancelled in October 1999 due, in part, to a lack of contribution by the Federal Government to the overall $1 billion cost. However, 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 for the Portside Project.

This paper is concerned solely with foundation design for the Terminal Expansion Project. It supplements a paper presented at the 2001 Symposium (Smith, 2001) which described the ground improvement measures required to ensure public health and safety during the 475 year design earthquake.

Fig. 1 Portside Project concept



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 longitudinal cross-section through the northern end of Pier B-C, is shown on Fig. 2 and illustrates the trench excavation, fill and layout of the piles supporting the Pier B-C deck.

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. Prior to densification offshore of the Canada Place deck, the fill was 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 gradation and consistency but is generally soft.

Underlying the fill and silt in the expansion area is glacial till, varying in gradation from silty sand to sandy silt and with a 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

The loose fill and soft silt underlying Canada Place and the Terminal Expansion area would undergo substantial settlement and lateral displacement during a 475 year earthquake with a maximum firm ground acceleration of 0.21g. Therefore, ground improvement was carried out for a distance of 50 m offshore of the north end of Canada Place using vibrocompaction of the fill and construction of stone columns in the silt to minimize lateral displacements of soil surrounding the piles driven in the Terminal Expansion area.

To investigate the expected ground displacements after ground improvement, Dr. P.M. Byrne was retained to complete FLAC analyses of a longitudinal section through

Fig. 2 Longitudinal section through north end of Canada Place

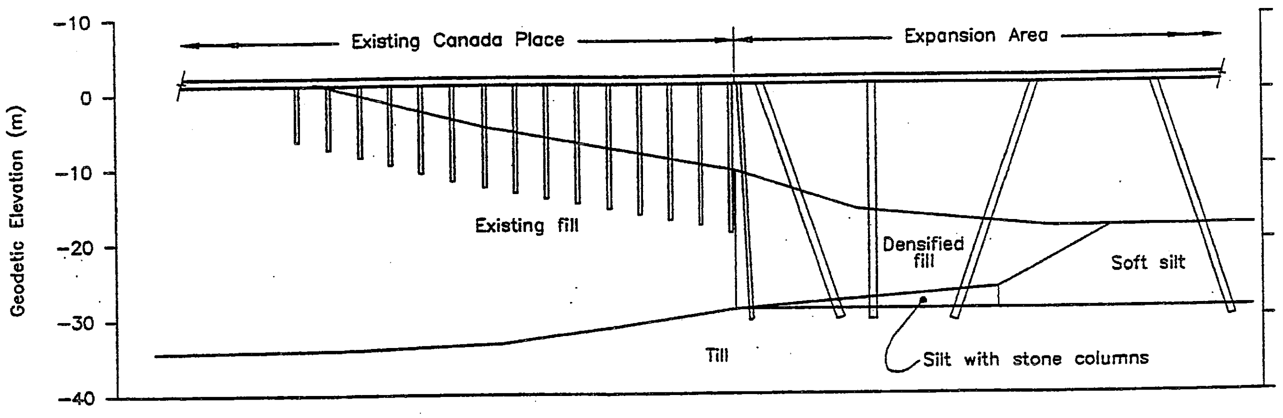


Fig. 3 Test hole locations at north end of Canada Place and estimated till surface contours

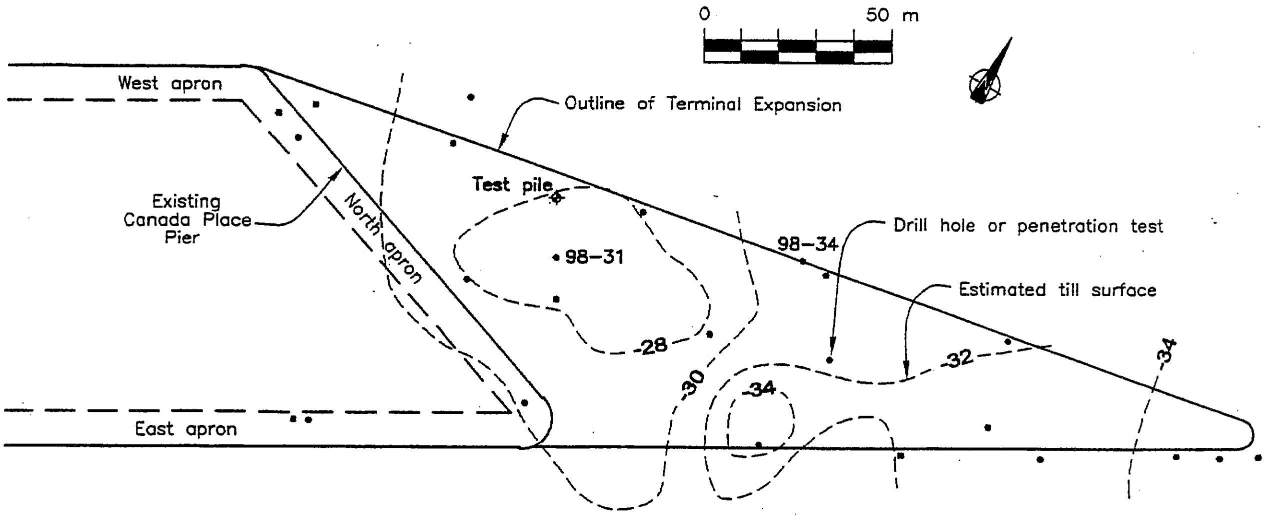


Fig. 4. Log of TH 98-31

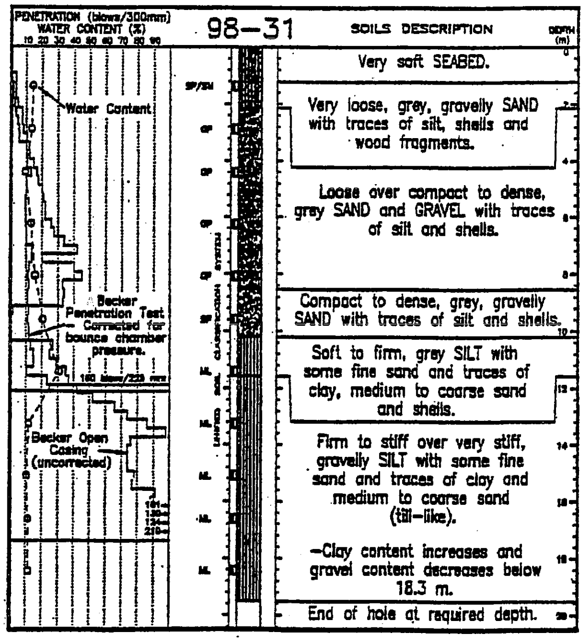
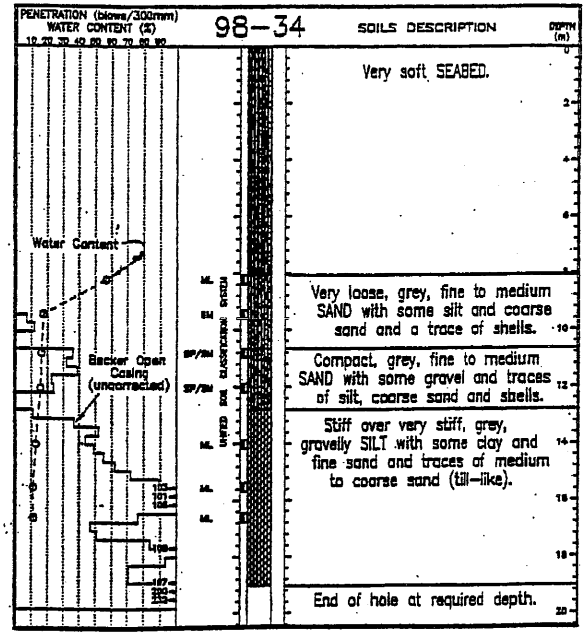


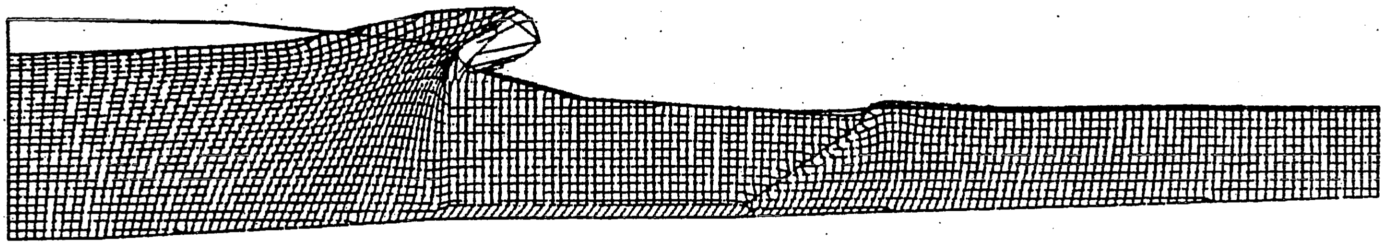
Fig. 5. Log of TH 98-34



the north end of Canada Place for the 475 year earthquake. The result of a FLAC analysis carried out for the maximum upper bound strength considered feasible for the silt unit, achieved by 50% replacement of the silt by stone columns, is illustrated on Fig. 6 . This analysis indicates that the densified fill block would remain intact and slide out about 150 mm, due entirely to strain in the silt unit. Thus, piles driven through the fill and silt into the till undergo the 150 mm displacement entirely within the zero to 3 m thickness of the silt unit, causing concern with regard to stresses developed in the piles. Piles offshore of the densified zone will be affected by horizontal displacement of the soft silt resulting from passive failure under the action of the fill block movement. The over-riding of the densified fill block by the liquefied fill under the existing deck, shown on Fig. 6, will be minimized by the seismic drains installed through the apron at the end of Canada Place as part of the ground improvement work.

The drilling results for test holes completed off shore and near the north end of Canada Place were utilized to prepare till surface contours used to estimate pile lengths for the tender documents and to establish probable tip elevations for pile driving. These contours are shown (in terms of geodetic elevation) on Fig. 3.

Fig. 6 Deformation at end of Canada Place estimated by FLAC (with exaggerated grid distortion)



Pile design

Pile design parameters were established prior to construction by consideration of the geotechnical information provided by the drilling programs, supplemented by a 1980 pile load on the south end of Pier B-C, prior to Canada Place construction, and 1998 pile load tests carried out for the Portside Project. Allowable (service) pile capacities were based on the following:

End bearing in till	5000 kPa
Adhesion in till	125 kPa
End bearing in non-densified granular fill	1000 kPa
Adhesion in non-densified granular fill	20 kPa
Adhesion in densified granular fill	30 kPa

Factors of safety utilized to determine safe working loads from ultimate capacities under various load combinations were as follows:

Dead load	2.5
Dead load + live load	2.0
Dead load + live load + earthquake load	1.25

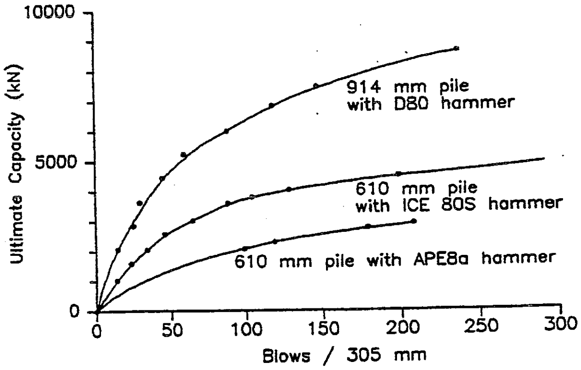
Based on these parameters and the design loads provided by the marine structural engineer (Westmar Consultants Inc.), 610 and 914 mm diameter piles driven closed-end into the till were selected for most of the deck support piles with allowable capacities in compression of, respectively, 1940 kN and 4000 kN. Piles driven through the densified fill offshore of Canada Place were specified to be open-ended, to ensure the piles reached the till surface, with clean-out of the internal soil plug prior to placement of a tremie concrete plug for development of end bearing. Piles driven through the existing deck of Canada Place were specified to have 8 m embedment into the non-densified granular fill under the deck. This is similar to the existing piles supporting the Pier B-C structure and the seismic behaviour of the existing and new piles would, therefore, be similar.

WEAP analyses

To investigate the feasibility of driving 914 mm diameter piles closed end with a Delmag D62 diesel impact hammer, the largest locally available hammer in 1998, WEAP analyses were carried out on several pile lengths and combinations of end bearing and shaft adhesion components. These analyses showed that 8000 kN ultimate capacity could be achieved at a final set of 15 to 25 blows/25 mm, a practicable range for pile driving. WEAP analyses were also completed for 610 mm diameter piles driven closed end with ICE 80S diesel and APE 8a hydraulic hammers. Typical WEAP results are shown on Fig. 7. Based on these analyses, minimum driving resistances (in terms of blows/25 mm pile penetration) were established prior to construction for closed end piles to achieve the design pile capacities as follows:

914 mm diameter into till with Delmag D80	15
610 mm diameter into till with ICE 80S	10
610 mm diameter into till with APE 8a	12

Fig. 7 Typical WEAP results



Pile load tests

Prior to final pile design, 2 pile load tests were carried out, one on land near the SeaBus Terminal and one offshore at the end of Canada Place. The onshore test was carried out in a 914 mm pile driven closed end into bedrock using a Delmag D62 hammer. To remove any shaft adhesion on the test pile resulting from the overlying fill, a 1067 mm diameter casing was first driven to bedrock and cleaned out to form a sleeve for the test pile. Pile Dynamic Analyser (PDA) testing performed during final driving of the test pile indicated a lower bound ultimate pile capacity of 8080 kN at a penetration resistance of 21 blows/25 mm. The pile load test, applied through 4 rock anchors, suggested an ultimate pile capacity of 11000 kN using the Davisson Offset line. A pull out test on an anchor comprising 3 No. 18 Dywidag bars grouted 10 m into bedrock was also carried out but the results were anamolous and not considered to be valid.

The offshore test also utilized a 914 mm pile driven closed end into the till unit through 13.5 m of soft mud. Blow counts reached 23 blows/25 mm at the end of driving. PDA testing was performed during final driving and indicated a lower bond (due to high penetration resistance) ultimate pile capacity of 4230 kN. Reaction was provided by 4 soil anchors formed by Dywidag bundles grouted into 250 mm diameter, 12 m long holes in the till. The holes were drilled uncased and considerable difficulty was encountered in keeping the holes open during drilling and grouting. This experience led to the specified requirement for cased holes to be used for anchors installed in production piles for the Cruise Ship Terminal Expansion. The compression test was aborted at a maximum load of 7000 kN due to premature pull-out of the anchors. At 7000 kN, the recorded displacement was close to the theoretical elastic compression of the pile, indicating that the ultimate capacity was more than 7000 kN. The pull-out test on the anchor indicated an ultimate capacity for short term loading of 1500 kN for the 11.3 m long anchor, suggesting an allowable bond stress of 125 kPa in the till.

Pile installation

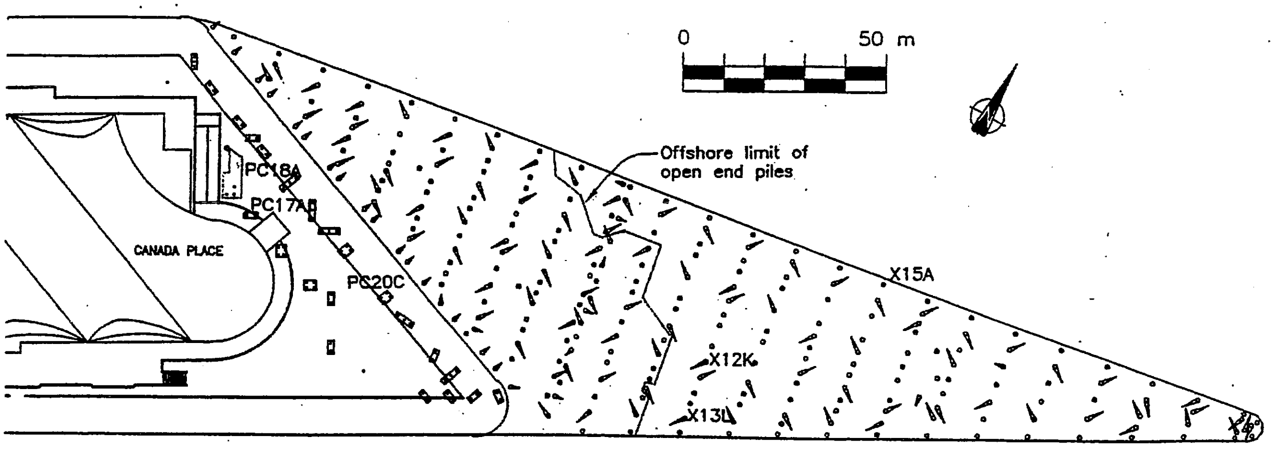
The complete layout of onshore and offshore piles installed for the Terminal Expansion is shown in plan view on Fig. 8. New piles are shown in a longitudinal section on Fig. 3. The direction of the batter only is shown for batter piles since at 1H:3V batter, the pile tips are offset about 10 m from the top of the pile top and, therefore, are close to the next line of piles, 11.5 m away. Vertical and batter pile orientation were considered in 3 dimensions to avoid interference. Separate discussion is given below for offshore compression, offshore anchor and onshore compression piles.

Offshore compression piles

Offshore piles for deck support comprised 262 steel pipes of 914 mm installed vertically or on a 1H:3V batter and 8 610 mm diameter piles installed on a 1H:3V batter. All piles were driven using barge-mounted hammers. About half the larger diameter piles were driven open end through the densified granular fill. The division between open end and closed end piles is shown on Fig. 8. Initial pile driving was carried out using an ICE 66-80 vibratory hammer through the densified granular fill, followed by seating with an ICE 80S diesel impact hammer. Further offshore, this operation was changed to use of a Delmag D80 diesel hammer.

One unexpected problem experienced during offshore pile driving was fish kill produced by shock waves in the water of Burrard Inlet, particularly for the closed end piles. This is believed to be the first documented instance in British Columbia of negative impact of pile driving on the marine environment and has led to increased attention by the Federal Department of Fisheries and Oceans (DFO) to the need for mitigative measures. Hydrophone measurements made close to piles showed pressures of over 150 kPa whereas an overpressure of 100 kPa is considered by DFO to be lethal to fish. The contractor, Fraser River Pile & Dredge Ltd., (FRPD) adapted the air bubble curtain used for underwater blasting to reduce shock wave pressures created by pile driving. The 1H:3V pile batter and currents in

Fig. 8 Pile layout



Burrard Inlet made it difficult for the contractor to ensure that the air bubbles rose to the surface around the pile, but after much experimentation, a design incorporating a multi-level air bubble curtain was developed and proved to be successful in reducing the shock wave to acceptable levels (Longmuir and Lively, 2001).

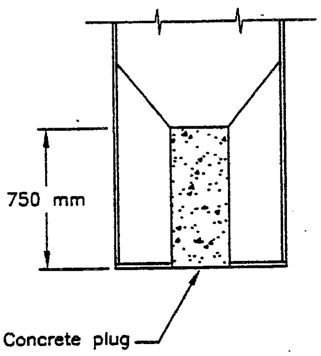
Offshore anchor piles

The tender documents required 51 of the 914 mm diameter piles driven closed end at a 1H:3V batter to be provided with soil anchors capable of withstanding 1200 kN yield load of the anchor bar during the design 475 year earthquake. With this design philosophy, and assuming a 250 mm diameter drilled anchor hole and 125 kPa allowable adhesion in the till, a 14 m long anchor was specified. The piles which were to become anchors were fitted with a 305 mm diameter by 750 mm long concrete plug at the pile tip in a steel pipe guide sleeve, as shown schematically on Fig 9. The plug was intended to allow driving of the pile to refusal in the till, as a conventional closed end pile, but then permit drilling through the plug for anchor installation.

Anchor installation was carried out by Southwest Contracting Ltd., as subcontractor to FRPD. A CSR 1000 drill rig with a lost ring bit system was utilized, installing 220 mm diameter casing with a 250 mm diameter bit using a down hole hammer. The anchor bar comprised a No. 18 Dywidag thread bar with double corrosion protection.

During driving of the first piles, it was found that the plug failed and was pushed up into the pile, allowing several metres of soil to flow into the pile. Various solutions were tried for subsequent piles, including a thick plywood disc attached to the end of the pile and a 100 mm thick fibre-reinforced shotcrete layer sprayed over the end of the pile, but with no certain success. The drill was able to locate the guide sleeve through the soil inside the pile and complete anchor installation. On a subsequent marine project in Vancouver, the guide sleeve was revised to a taper to prevent displacement of the concrete plug, as shown schematically on Fig. 10. Though this design proved successful, it is believed that driving of open end piles to refusal in till, followed by clean-out of the soil plug and anchor installation through the open end with some form of centralizer may be more cost effective and less prone to difficulties if, for instance, sand or silt is encountered at the base of the pile.

Fig. 9 Guide sleeve for tip of anchor pile



During construction, it was concluded that for 19 of the 51 piles, the required uplift resistance (of 1200 kN) could be provided by pile adhesion in the till without the need for anchors. For these piles, end bearing was restored by airlifting the soil from inside the pile and pouring an 8 m long concrete plug inside the pile.

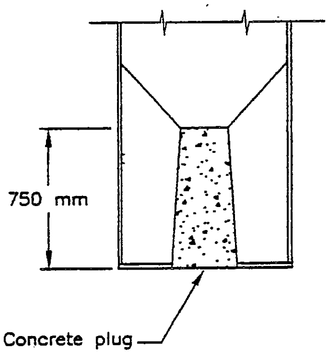
For the 32 anchored piles, each was tested to 1000 kN, 83% of yield load of the Dywidag bar, and accepted.

Onshore piles

Onshore piles comprised 2 914 mm diameter and 95 610 mm diameter piles driven through openings cut into the concrete deck of the existing Canada Place. The piles were originally intended to have the same 10 m embedment as the existing piles and to terminate in the non-densified granular fill. However, pile capacities achieved at that depth were insufficient for the loads to be imposed by the new structure. Therefore, where feasible, additional piles were installed and, where this was not feasible along the edge of the deck, piles were lengthened to achieve refusal in the till underlying the granular fill. A construction joint was provided on the deck to allow differential movement during a major seismic event.

Most of the piles were driven using a ICE 80S diesel impact hammer or an ICE 115 hydraulic hammer. However, piles required under the second floor deck of the existing structure were driven using an APE Model 7.5a low headroom hydraulic impact hammer. This hammer has an operating height of only 2.2 m and generates a related energy of 37 kJ. APE's brochure for the similar Model 8a, which

Fig. 10 Revised tapered guide sleve



was originally proposed for the work, states that it out performs diesel impact hammers with a rated energy of 140 kJ.

Pile Dynamic Analyser (PDA) testing

Two phases of PDA testing were carried out during pile installation. Three offshore 914 mm diameter piles driven closed end to till were tested on restrike (RSTR), 18 days to 6 weeks after initial driving. Three onshore 610 mm diameter piles driven closed end to 8 m embedment in non-densified fill under the deck were tested during initial driving (ID). The results are summarised in Table 1 in terms of mobilized capacity provided by CAPWAP analyses. The 914 mm diameter piles on RSTR include the benefit of set-up, the increase in capacity with time, whereas the 610 mm diameter piles do not. The toe resistance of Pile X12-K may have been affected by displacement of the concrete plug installed for anchor drilling. The result generally confirm assumptions made during the design phase.

Deck settlement

One result of the pile driving and densification work carried out at the end of Canada Place was settlement of the existing deck and the unoccupied 3 storey structure above. Survey monitoring was carried out while the work progressed to establish the magnitude of the settlement and the extent of the remedial work. The maximum settlement recorded was 150 mm in the middle of the north apron. Remedial work primarily comprised jacking of column feet resting on the deck to level the structure. Final concreting of the onshore deck was delayed until all onshore piles were driven.

Acknowledgements

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Table 1. PDA Testing results

Pile	Diameter mm	Inclination	Testing Condition	Mobilized Capacity (kN)		
				Shaft	Toe	Total
X12-K	914	1H:3V	RSTR	2000	3500	5500
X13-L	914	Vertical	RSTR	3500	4500	8000
X15-A	914	Vertical	RSTR	2000	6900	8900
PC17A	610	Vertical	ID	1150	450	1800
PC18A	610	Vertical	ID	550	1000	1550
PC20C	610	Vertical	ID	600	1600	2200