The Construction and Performance of Post-grouted Micro-piles for Vancouver International Airport Domestic Terminal Building

Ernest Naesgaard[1], Mustapha Zergoun[1] , and William Auyeung[2]
[1] Trow Associates Inc., [2] Kani Foundations Inc.

Introduction

Post-grouted micro-piles were used for the seismic upgrading of the Vancouver International Airport Domestic Terminal Building (DTB). The micro-piles provided vertical resistance to both uplift and compression loads to supplement the existing timber pile foundations ability to resist the earthquake induced forces, and to provide foundation support at locations where the installation of traditional driven piles was difficult. This paper describes the site conditions, design rationale, installation method and quality control testing procedures.

Site and Subsurface Conditions

Vancouver International Airport is located within the central portion of Sea Island, Richmond, British Columbia (Fig. 1). Sea Island is bordered by branches of the Fraser River on the northeast and south sides and the Strait of Georgia along the west side. The natural grade of the island is part of a large delta that has been built up over several thousand years. The general pattern of sediment deposition has followed the direction of the Fraser River flow from east to west, and has built up soft sediments of about 200 m thickness at Sea Island (Luternauer, et al. 1993, Mathews and Shepard 1962). The original grade in 1962 prior to the existing Airport Terminal Building development was at about geodetic elevation 1.1 m with a grass and topsoil cover.

Prior to construction of the airport terminal, and aprons the area was preloaded with river sand (Meyerhof and Sebastian 1970, Bertok, 1987). On completion of preloading the sand was removed to the adjacent apron area, treated timber pile foundations were driven, and the building was constructed. Upon completion, the surrounding apron areas were approximately 3 m higher than the lower building floor thus creating a basement. In the early 1990's a new International Terminal building was constructed and the existing terminal building became the Domestic Terminal Building (DTB). A seismic upgrade design for the DTB was carried out in 1997. Implementation of the design, which occurred in multiple stages, is still on-going. A cross section of the DTB prior to seismic upgrade is shown in Figure 2.

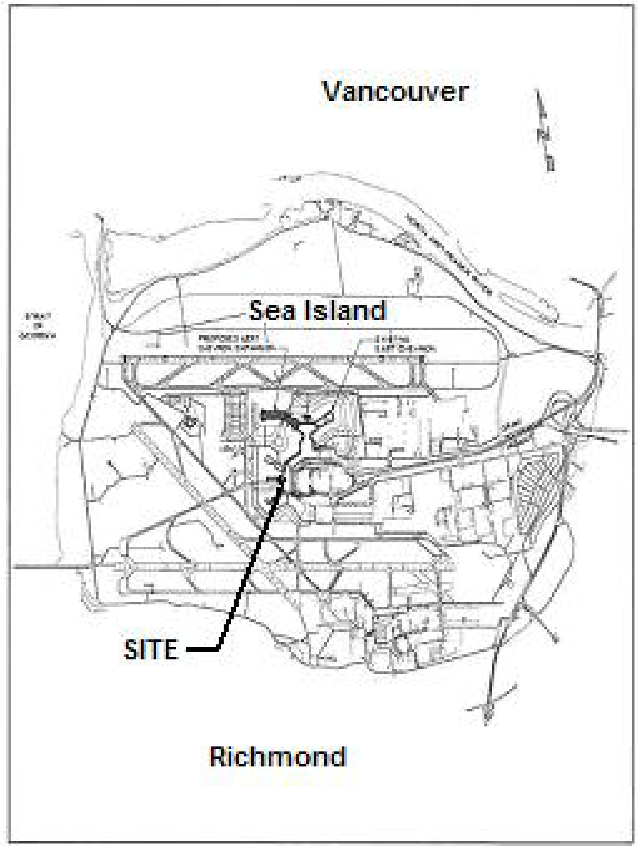

Figure 1 Vancouver International Airport key plan

A generalized subsoil stratigraphy is illustrated by the cone penetration test log in Figure 3. In order of increasing depth the soil profile is as follows:

<u>Zone 1</u>: Granular FILL layer of up to 3m thickness but not present below the DTB basement. Generally the fill consists of relatively clean fine medium coarse sand dredged from the Fraser River.

<u>Zone 2</u>: Firm to stiff clayey SILT with trace of organics, becoming sandy SILT in bottom 1 m (estimated thickness 3 to 4 m). (W_N = 36 to 54% $W_L \approx$ 58%, $W_P \approx$ 33%)

<u>Zone 3</u>: SAND, inter-layered with sandy silt/silty sand in the upper 5 m, loose to dense with depth (estimated thickness 10 to 20 m). The existing timber piles and the new micro-piles are founded within this zone.

<u>Zone 4</u>: Inter-layered fine SAND, SILT and clayey SILT with less fine sand inter-layering with depth

(estimated thickness up to 130 m). Typical silt fractions have W_N = 32 to 40% $W_L \approx$ 33 to 35%, and $W_P \approx$ 26 to 30%.

<u>Zone 5</u>: Pleistocene very dense well graded silts, sands and gravels at a depth of approximately 150 m.

W_N = Natural water content, W_L = liquid limit & W_P = plastic limit.

Groundwater levels vary with the season and range from geodetic elevation -1.2 to 1.7 m.

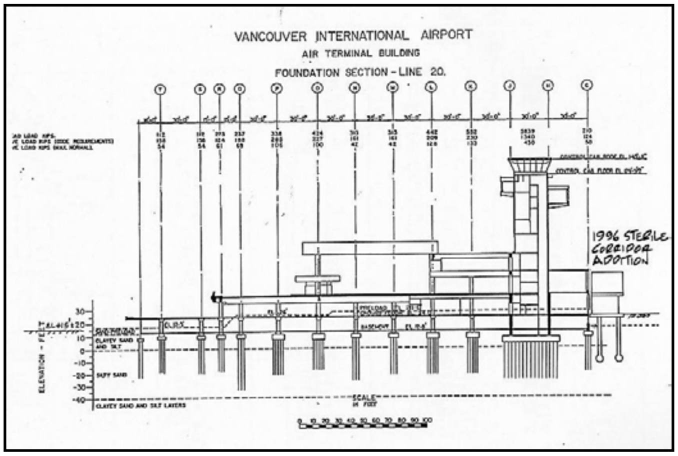

Figure 2 Section through DTB prior to seismic upgrade

<u>Seismic Considerations</u>

The Fraser River Delta and Sea Island are in an area of high seismic activity due to the stresses induced in the underlying bedrock by the collision of the offshore Pacific plate with the American continental plates. Table 1 indicates the peak ground acceleration (PGA) and peak ground velocity (PGV) for outcropping firm ground or bedrock, according to the seismic model of the 1995 National Building Code of Canada (NBCC).

Table 1 Seismic Risk - 1995 NBCC Seismic Model

Probability of Occurrence per year	0.01	0.005	0.0021
Return period (years)	100	200	475
PGA (g)	0.09	0.14	0.22
PGV (m/s)	0.08	0.12	0.22

For the seismic upgrade of the DTB a 0.0021 probability of occurrence per year risk level was chosen. This corresponds to a 10% probability of occurrence every 50 years or return period of 475 years. This relates to earthquakes of magnitude 6.5 to 7.5 with distances to the epicentre ranging from 25 to 100 km. According to the NBCC the Vancouver International Airport is within the seismic zones Z_a=Z_v=4. The thick soft soil deposits overlying the bedrock in the Fraser River delta tended to increase the predominant period and amplify the bedrock ground motion. To allow for this a building code foundation factor (F) equal to 2 was used for structural design.

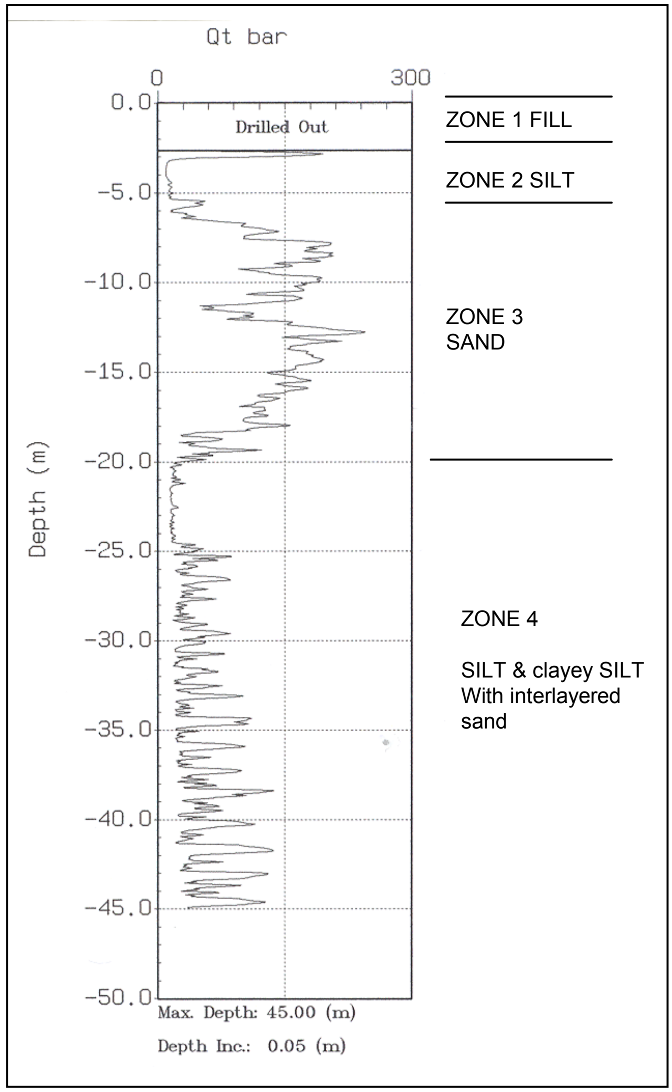

Figure 3 Soil Profile with typical CPT log

<u>Site Specific Ground response analysis</u>

A site specific ground response analysis was conducted for the seismic upgrade of the Domestic Terminal building (DTB). The ground response analysis was done using the program SHAKE with a soil column based on recent deep test holes completed by the Geological Survey of Canada. Earthquake records for dynamic analyses and cyclic stress ratio for liquefaction assessment were obtained from the ground response analyses. Surface peak ground accelerations in the range of 0.21 to 0.31g were obtained.

Liquefaction Analysis

A liquefaction analysis was carried out using the method proposed by Seed and Idriss (1971) and the cyclic stress ratio from the one-dimensional site specific ground response analysis. The results of the liquefaction analysis indicated that for the 475 year return period design earthquake liquefaction within the area of the DTB was sporadic and confined to local lenses of less than one meter thickness. The limited liquefaction at this site compared to other locations within Sea Island and the Fraser River delta is believed to be partially due to a combination of the effects from preloading and driving of a large number of timber piles. Soil within the pile groups and up to 1.5 m beyond was found to be very dense and not subject to liquefaction.

Dynamic Numerical Analysis

A two-dimensional (2-D) dynamic numerical analysis of a section through the DTB was carried out using the program FLAC (Cundall, 1996). The model included the 2-D soil profile, building foundations (including timber piles and micro-piles) and building structure. Building columns, piles, and micro-piles were modelled by scaling their stiffness and strength by the spacing of the members in the out of plane direction. At an appropriate time during the dynamic analysis elements within the soil deemed to be liquefiable were liquefied by changing their properties from those of non-liquefied sand to liquefied sand. Earthquake induced deformations and its effects on the foundation structures were observed from the analyses.

Consequences of earthquake shaking and liquefaction

Earthquake shaking and liquefaction causes relative deformation of the foundation structures. Differential lateral deformations of foundations at pile cap elevation were small because the foundations were tied together by the basement slab. However differential lateral deformations varying with depth in the ground did occur. Based on the numerical analyses, these differential lateral displacements were estimated to be less than 150 mm. Settlements also occurred. These were from two sources: (i) from shear deformations within the soil, particularly within zones that have liquefied that occur during the period of strong shaking; and, (ii) post-liquefaction settlement due to the consolidation that follows the pore water pressure dissipation after the earthquake. Estimates of the shear deformations were obtained from the dynamic numerical analysis while the post-liquefaction consolidation settlement was calculated using the method proposed Tokimatsu and Seed (1987). The combination of the two types of settlements gave potential differential settlements between foundations of less than 100 mm. The earthquake shaking also caused increased lateral and overturning forces on the building foundations that exceeded the original design values as discussed below.

Micro-pile Design & Installation

Seismic upgrade of the DTB involved adding bracing structural elements between select columns to resist earthquake induced shear loads. This resulted in lateral and overturning moments being applied to the foundations that were beyond their original design capacity. The overturning moment from the braced frames exceeded the uplift and compression resistance of the existing timber pile groups and additional micro-piles were added to supplement their load carrying capacity. Figure 4 shows a schematic section through a braced frame illustrating the seismic design concept within the DTB.

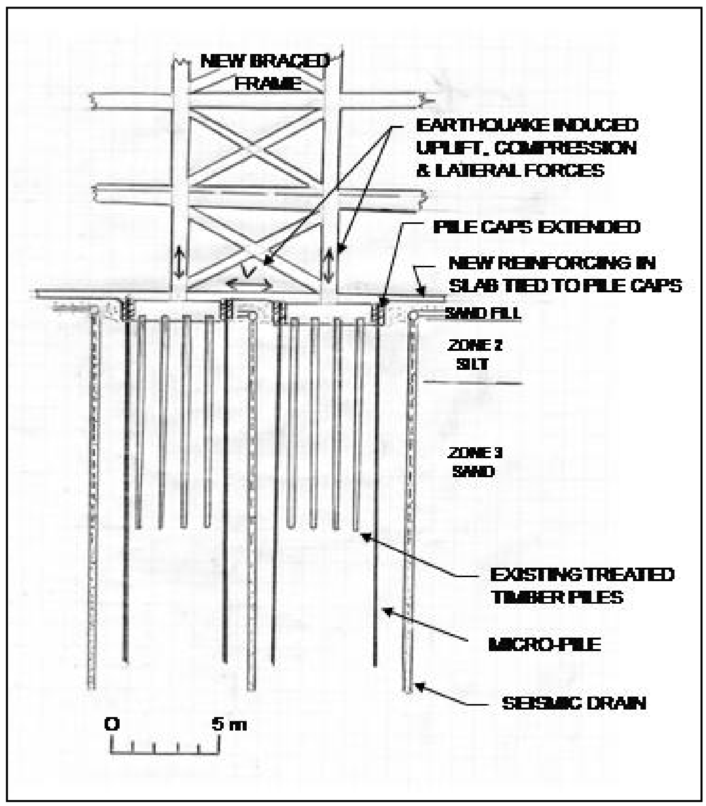

Figure 4 Schematic key seismic upgrade features

17-m long micro-piles were designed for a factored ultimate uplift and compression load of 670 kN (150 kips). The micro-piles consisted of 57-mm (2.25-in) diameter threaded steel bars with 400 MPa (60 ksi) yield strength. Figure 5 shows the details of a typical micro-pile installation. The existing 8 to 10 m long No. 12 treated timber piles were given a factored ultimate capacity in compression of 356 kN (80 kips) and a small tensile resistance due to their limited embedment into the pile cap.

Aspects of the micro-pile installation that were of concern for the designers included:

- Limited lateral load resistance of the micro-piles due to their low bending stiffness and moment capacity.
- Earthquake induced differential shear within the soil would induce bending and shear forces on the micro and timber pile foundations that may cause them to structurally fail or buckle.
- Reduction in strength due to migration of high pore water pressures from adjacent liquefied soil zones.
- Relative stiffness of the micro-piles in compression and tension compared to the existing timber pile foundations.

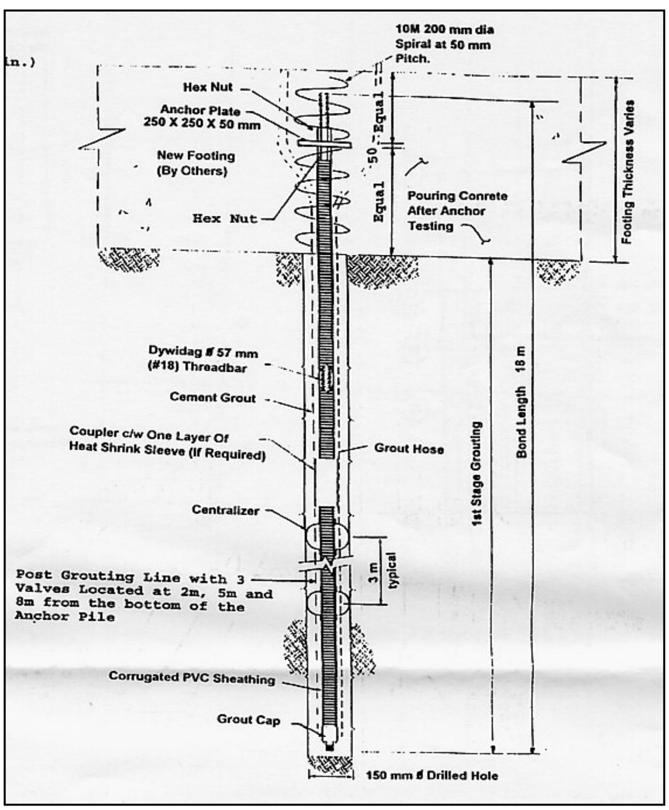

Figure 5 Typical micro-pile detail

Base shear load path

The foundations of the braced frames did not have sufficient lateral resistance against the design structural base shear loads. To alleviate this, the additional lateral loads were distributed through the 150-mm to 300-mm thick concrete floor slab among all the pile foundations within the building and the perimeter walls. The floor slab was strengthened and tied together for this purpose. The micro-piles were assumed not to provide any additional lateral resistance to the foundations.

Differential shear within the soil profile

Even though liquefaction was limited and sporadic in location, the dynamic numerical analysis indicated that some differential shear (up to 150 mm) may occur within the soils underlying the building. It was imperative that the micro and timber piles tolerate this differential shear displacement without buckling or other structural failure mechanism. Lateral pile analyses using the p-y method and the program LATPILE was carried out to confirm that curvature induced in the piles by the earthquake would not result in shear, bending, or buckling failure. Full scale lateral loading tests were conducted by the University of British Columbia on six timber piles to demonstrate that the calculated earthquake induced curvature would not fail them. With the micro-piles it was deemed acceptable if the piles yielded in bending, however they should not buckle under the design factored ultimate seismic load.

Pore water pressure relief by seismic drains

Earthquake shaking causes an increase in the pore water pressure within saturated sandy soils; particularly in the looser zones that liquefy. There is concern that these high pore water pressures will migrate to the pile foundations and weaken them due to the resulting decrease in effective stress. Seismic drains were installed around the timber and micro-pile foundations to mitigate this effect. These seismic drains are shown schematically in Figure 4. The seismic drains were constructed by drilling in a 250-mm (10-in) casing while simultaneously drilling out the interior soil using water flushing. The casing was then backfilled with birds-eye fine clear gravel and a central 37-mm (1.5- in) slotted plastic drain pipe. These drains were placed around the perimeter of the foundations on approximately 1.5-m (5-ft.) centre to centre. The drains were constructed after installation of the micro-piles. Proof load tests on adjacent micro-piles showed that installing the drains closer than 1.5 m from the pile elements would reduce the pile capacity.

Relative stiffness of micro-piles

For a given load, the micro-piles are more deformable than the existing timber piles. To achieve the design load in the micro-piles the timber piles may yield by pushing into the ground slightly. Based on the results of timber pile load tests it was deemed that the timber piles would behave in a ductile manner in axial loading and the additional deformations were accounted for in the design.

The Micro-pile Installation

It was paramount that the installations of the micro-piles not disturb the existing adjacent timber pile foundations. It was specified that the piles should be installed without significant loss or disturbance of the existing soils and use of air flushing for the drilling was not permitted.

The micro-pile bars were supplied grouted into a corrugated plastic sheath (double corrosion protection) and were pre-assembled with spacers every 2 meters, with after-grout bladders at 2, 5 and 8 meters from the bottom of the micro-pile, and with grout tremie and after-grout tubes in place. The thread bars were cut in 3-m (9-ft.) segments and coupled during installation because of the low headroom within the DTB basement. The micro-piles were installed by drilling in a 150-mm casing while flushing with water. The micro-pile threadbar assembly was inserted inside the casing and the casing tremied full of grout. As the casing was then withdrawn pressure grouting was carried out at 2-m increments. Pressures in the range of 0.5 to 0.65 MPa (75 to 95 psi) were achieved. Upon achieving sufficient grout strength, the micro-piles were after-grouted to pressures of approximately 5 MPa (750 psi). Type 30 Portland cement with no additives and a water-cement ratio of less than 0.45 was used for both the primary and after-grouting. Typically 560 litres of grout were used per micro-pile for the initial grouting of the hole plus 230 to 330 litres for each stage of after-grouting. As the work was conducted inside an existing structure all exhaust fumes from the drilling and grout equipment had to be directed outdoor through pipes.

Quality Control and Testing of Micro-piles

Quality control included monitoring of drill cutting volumes, monitoring of grout takes, and monitoring settlement and tilting of existing nearby building foundations. All micro-piles were proof load tested or performance load tested in tension to the design load of 670 kN (150 kips). In addition, a few micro-piles were tested both in tension and compression. Figure 6 shows the results of a tension-compression test and Figure 7 shows the load-displacement response of a micro-pile with excessive elongation on the initial proof load testing. An additional after-grouting phase stiffened up the micro-pile response to the specified tolerances. The design load of 670 kN was achieved on all proof and performance load testing, except those tests where seismic drains were installed within 0.9 m (3 ft.) of the micro-pile prior to testing. When the seismic drains were installed at least 1.5 m from the micro-piles capacity was not affected. Residual elongations from proof load testing varied from 2.5 to 10 mm (0.1” to 0.4”) and was typically 5 to 6 mm (0.2” to 0.3”). The residual elongation is believed to be due to the take-up of slack within the couplers and due to remaining residual stresses within the micro-pile from the load testing.

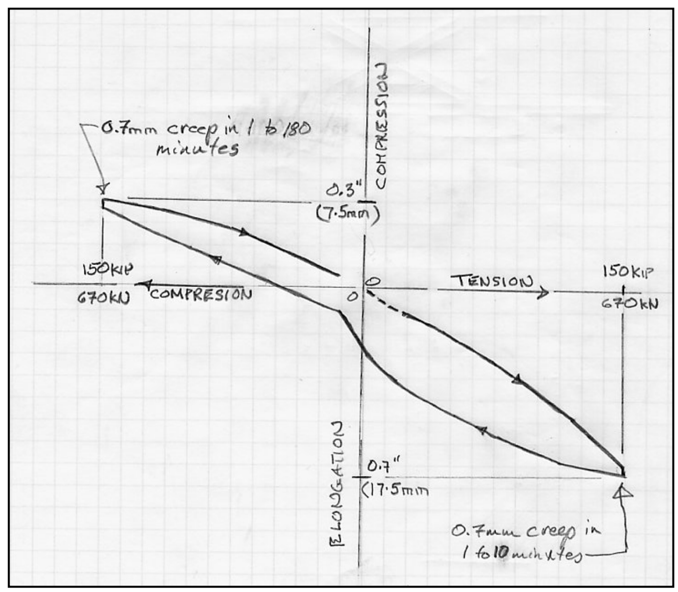

Figure 6 Tension-compression test on micro-pile

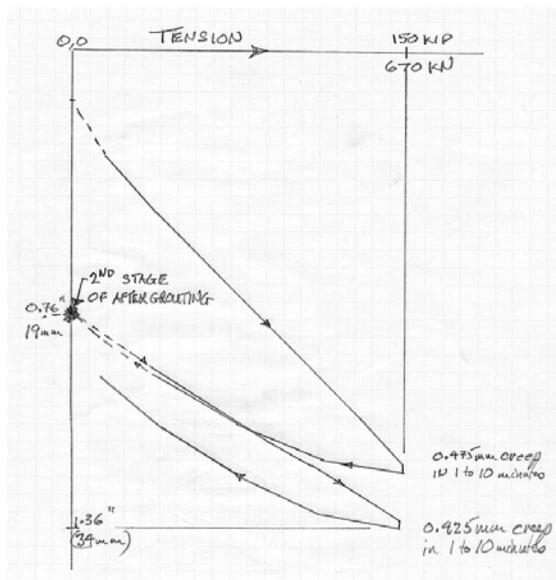

Figure 7 Micro-pile where second stage of after grouting was required due to excessive elongation of initial tension proof test

Conclusions

Grouted in 57-mm diameter steel threadbar micro-piles have been installed at the Vancouver International Airport Domestic Terminal building to supplement existing treated timber pile foundations during earthquake loading. Relative stiffness between the micro-pile and timber pile foundations, differential lateral shear displacements within the soil, and migration of pore water pressure from nearby liquefiable zones were considered in the design.

The anchors were installed in the limited headroom of the existing building basement without detrimental effects on the existing foundations and services. All anchors were proof or performance load tested and achieved the design capacity of 670 kN (150 kips). After-grouting was noted to increase the stiffness of the anchors. Installation of seismic drains within a distance of 1.5 m was demonstrated to reduce the capacity of the micro-piles.

Acknowledgements

The authors would like to thank the Vancouver International Airport Authority for the opportunity to work on this most interesting project and permission to publish this paper. The project work was completed in coordination with Read Jones Christoffersen Ltd., the structural consultants, and Kasian Kennedy, the architects, for the project. We also wish to thank Dr. D.L. Anderson and Dr P.M. Byrne of the University of British Columbia for their consultation during the project.

References

Bertok, J. 1987. Settlement of Embankments and Structures at Vancouver International Airport. Canadian Geotechnical Journal, Vol. 24, pp. 72-80.

Byrne P.M. et al 1991. Earthquake Design in the Fraser Delta, Task Force Report, June 1991.

Cundal, P. (1996), "FLAC Manual Version 3.3, ITASCA Consulting Group Inc., Trasher Square East, 708 South Third St., Suite 310, Minneapolis, Minnesota 55415.

Luternauer, J.L., et al. 1993. The Fraser River Delta, British Columbia: Architecture, Geological Dynamics and Human Impact. Proceedings of the 8th Symposium on Coastal and Ocean Management, American Shore and Beach Preservation Association and ASCE, July 19-23, 1993, New Orleans, Louisiana, USA, pp. 99-113.

Mathews, W.H. and Shepard, F.P. 1962. Sedimentation of Fraser River Delta, British Columbia. The Bulletin of the American Association of Petroleum Geologists, Vol. 46, No. 8, August 1962, pp. 1416-1443.

Meyerhof, G.G. and Sebastian, G.Y. 1970. Settlement Studies on Air Terminal Building and Apron, Vancouver International Airport, British Columbia. Canadian Geotechnical Journal, Vol. 7, pp.433-456.

Seed, H.B. and Idriss, I.M., 1971. "Simplified Procedure for Evaluating Soil Liquefaction Potential." ASCE Journal of the Soil Mechanics and Foundation Division, 97(SM9), 1249-1273.

Tokimatsu, K. and Seed, H.B. 1987. Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8, August, 1987, pp. 861-878.