Design and Construction of the New Coal Harbour Shoreline in Downtown Vancouver

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ABSTRACT Construction of the new shoreline at Coal Harbour, extending from the Westin Bayshore Hotel to the west to the Vancouver Convention Centre to the east, required extensive land reclamation of Burrard Inlet. The overall Coal Harbour redevelopment consisted of many major elements including over 1 km of seawall and shoreline slope, Harbour Green Park and Escarpment Wall, the Coal Harbour Marina and Restaurant, a saltwater intake structure associated with City of Vancouver's Dedicated Fire Protection System (DFPS), a floating walking and several roadway extensions.

Imported fill, over 15 m high, was needed for the construction of the new shoreline. Fill placement over the soft marine sediments had to be carried out in several stages to avoid failures. Various ground improvement techniques such as vibro-compaction, vibro-replacement and dynamic compaction were employed to treat both the existing fill and imported fill to meet the design criteria under the static and seismic loading conditions.

This paper describes the geotechnical aspects of the design and analyses of the Phase 2 portion of the Coal Harbour Shoreline Development. This includes the various ground improvement techniques adopted including the rationale for their selection. Results of the ground improvement work along with results of the slope monitoring work during the construction are presented. Challenges encountered during construction and lessons learned are also presented.

Introduction

Starting in 1991, Golder Associates Ltd. provided geotechnical engineering services to Marathon Development Inc.'s Coal Harbour Development which extends from Westin Bayshore to the west and Vancouver Convention Centre to the east. The major components of this development consist of over 1 km of seawall and shoreline slope including fish benches, a floating walkway, the Coal Harbour Marina and Restaurant, Harbour Green Park, and Escarpment Wall and many roadway extensions including several viaducts. Land reclamation up to 50 m wide was carried out to accommodate this development.

The overall development was carried out in several phases. Fig. #1 shows the overall Coal Harbour project site and shoreline in April 1994 with the pile supported Phase 1 seawall substantially complete.

Phase 2 development consisted about 0.6 km of seawall extending east from the eastern end of the Phase 1 seawall. The extent of the Phase 2 Coal Harbour development is also shown in Fig. #1.

Site Development History

The development of the Coal Harbour area started in the 1880s. Examination of aerial photographs indicates that between 1946 and the 1990s, several changes to the shoreline took place within the Phase 2 development area. Aerial photographs showing reclamation of land and developments at Coal Harbour between 1968 and 1976 are shown in Fig. #2. The aerial photographs show that by 1968 the reclamation of the current Phase 2 shoreline had begun and by 1976 the reclamation had been completed.

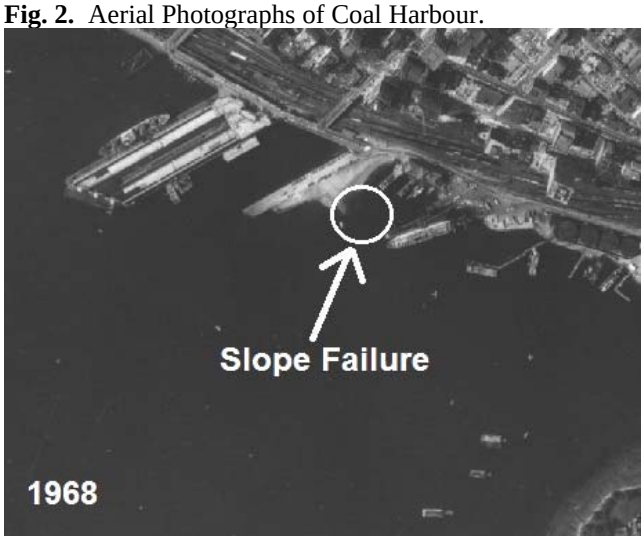
Materials including soil fill and construction debris were end dumped and pushed over the natural shoreline and seabed soils.

As can be seen in the 1968 aerial photograph, the uncontrolled filling had lead to a major slope failure. This slope failure and other observations associated with the performance of the existing major facilities within the general Coal Harbour area emphasized the importance of the care needed in the land reclamation for the new Phase 2 shoreline.

Fig. 1. Extent of Phase 2 Coal Harbour Development.

Photo by: D.E. Waite taken April 23, 1994

Fig. 2. Aerial Photographs of Coal Harbour.



Subsurface Conditions

As noted in the site development history, the project site largely comprises reclaimed land. In general, the depth of fill increased as land reclamation progressed northward towards the current shoreline where the depth of fill ranges from 10 m to over 18 m. The fills used in the reclamation were random in nature comprising silts, sands and gravels with varying amounts and types of bricks, concrete, cobbles, boulders, wood waste and other material(s).

Underlying the layers of fill, where present, and at the seabed in the areas offshore of the fills are the original marine sediments. They consist of compressible silts and clayey silts to sand and silty sand with trace to some organics, shells and gravel. The thickness of the marine sediment layers ranges from a few metres in the southern part of the area to about 17 m at the northern offshore limit of the area. The fine-grained marine sediments are firm to stiff within the onshore (filled) area and are soft to very soft in the offshore area. Underlying the marine sediments are dense to very dense glaciomarine and glacial drift deposits and these are underlain by sedimentary bedrock.

New Shoreline Development

Fig. #3 shows the plan view of the new shoreline within the Phase 2 area. The development plan for the near shore infrastructure in the Phase 2 area consisted of the following:

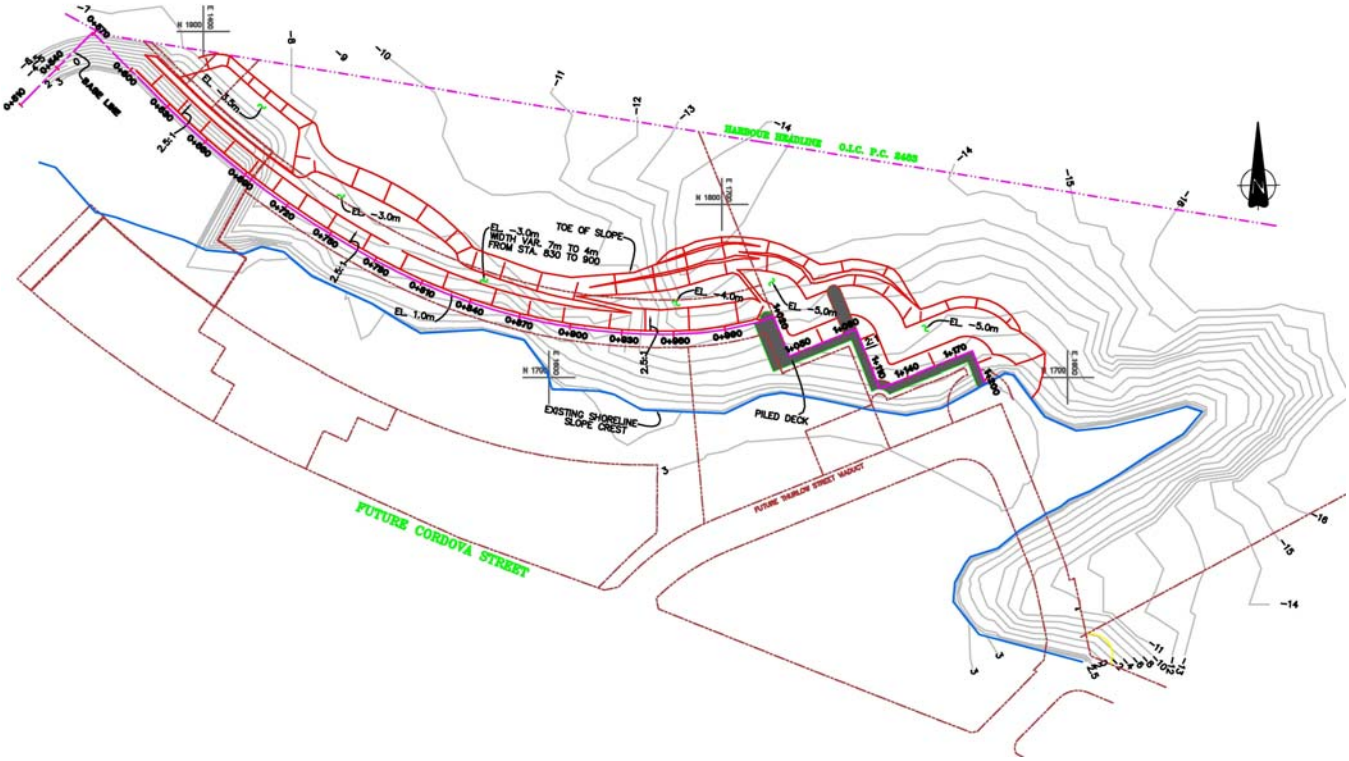
- The new shoreline was to be extended about 15 m to 50 m north of the shoreline that existed in 1994.
- A fill supported walkway at the about elevation 3 m (Geodetic datum) was to be provided along the relocated shoreline.
- The outer edge of the walkway was to be supported by a retaining wall with an exposed height of about 2 m.
- The slope below the retaining wall was to be provided with benches and protected with rock.
- The new shoreline slope would be constructed generally at 2.5 horizontal to 1 vertical.
- Benches were to be provided generally at elevation -3 m or -4 m for fisheries habitat with a width of generally about 4 m.
- A 100 m long floating walkway was to be constructed with the concrete float platform located at a distance of about 30 m from the seawall. Ramps were to be provided between seawall and the float platform for pedestrian access.

Design Criteria

The new shoreline in the Phase 2 development area was to be designed to provide acceptable performance under static loading conditions and to meet the City of Vancouver displacement criteria (300 mm for the fill supported seawall area) under the design seismic loading conditions. The applicable building codes at the time of design and construction of the new shoreline were the 1992 B.C. Building Code and the 1995 National Building Code (NBCC). The design earthquake for this project was selected as an earthquake with a return period of 475 years. This is considered to be an event with acceleration and velocity values having a 10 percent probability of exceedence in a 50 year period.

In addition, the benches in the shoreline slope configuration are to satisfy the habitat requirements of the Department of Fisheries and Ocean (DFO).

Fig. 3. Proposed New Shoreline in the Phase 2 Area.



Geotechnical Design Considerations

The loose to compact heterogeneous fills covering most of the site and underlying soft and loose portions of the marine seabed deposits would be subject to variable settlements upon application of the development loads.

The loose granular portions of the fills would be subject to liquefaction during strong ground motion earthquakes.

Without lateral constraint at the shoreline, the potentially liquefiable soils could undergo large lateral deformation as a result of softening and loss of strength during a strong ground motion earthquake.

Soil improvement techniques were utilized to minimize potential settlements of the fills and to improve the resistance of the granular portions of the fills in the critical areas to liquefaction.

The primary techniques considered practical at the site included:

- preloading to reduce post-construction settlements;
- dynamic compaction or vibro-replacement using stone columns to densify the new granular fills and the granular portions of the existing fills and marine sediments; and
- vibro-replacement using stone columns to enhance the strength of the clayey silt portion of the marine sediments and existing heterogeneous fills.

Construction Considerations

Because of the height of fill required to achieve the proposed site grades and the subsurface conditions, the following construction related challenges and issues had to be carefully considered in establishing a cost effective construction sequence and ground improvement scheme:

- The new shoreline fills should be selected so that they could be readily densified;
- To facilitate ground improvement, rip-rap from the existing shoreline slope and concrete rubble and other obstructions would have to be removed within the proposed ground improvement zone prior to new shoreline fill placement;
- Overbuilding of the shoreline fill, if necessary, should be considered to accommodate the required vibro-replacement treatment and to ensure satisfactory treatment;

- Fill placement to achieve the proposed site grades and any overbuilding would have to be carried out in stages to avoid seabed slope failures such as the 1968 failure; and
- In areas where the fill placement to achieve the proposed grade changes prior to ground improvement treatment would result in an insufficient short-term factor of safety against significant slope failure, either vibro-replacement treatment of the marine deposits and existing random fills prior to placement of the new shoreline fill or vibro-replacement with partial placement of the new shoreline fill would be required.

Ground Improvement Treatment

During the design process, two ground improvement treatment options were studied to determine which of the options would be more economical.

The first option was to improve existing soils using a barge-based ground improvement scheme prior to placing any significant shoreline fill, where possible. In this option, stone columns would extend through the existing random fills and marine deposits and keyed into the underlying dense glacial drift. Once the treatment of the existing soils was complete, the next step in this option would be to place new shoreline fill to the proposed grades. Then, a land-based ground improvement treatment would be carried out only to density the newly placed shoreline fill to the required target density. With this option, it would be possible to place new shoreline fill in fewer stages because of the improved underlying soils.

The second option was to place the new shoreline fill to the proposed final grade in stages to ensure stability and then carry out ground improvement treatment as a single land-based operation. Stone columns would extend through the new shoreline fill, existing random fills, marine deposits and keyed into the dense glacial drift. In this option, depth to the dense glacial drift is a critical factor in selection of an appropriate crane and associated equipment for the ground improvement work.

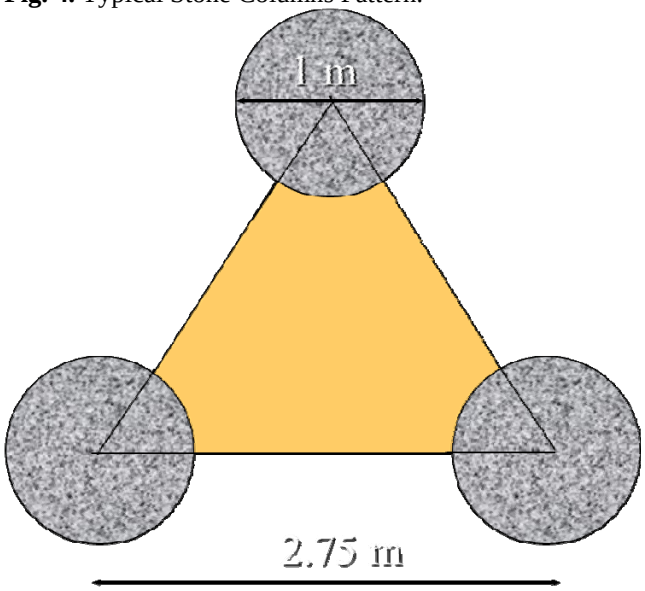
The first option had the advantage of shorter schedules for the new fill placement and densification of the new shoreline fill, but it required a two stage ground improvement process including a barge-based ground improvement scheme.

Based on the preliminary costing, it became clear that the second option, where only a single ground improvement treatment scheme is required, would be much more economical. This way the stone column treatment would be carried out as a single land operation and stone columns would extend through the newly placed fills, existing fills and marine deposits and into the dense glacial drift.

Because of the presence of the extensive soft to firm and loose marine deposits, improving the shear strength of these deposits to an appropriate target strength was critical to the short-term and long-term performance of the new shoreline. Vibro-replacement with approximately 1 m diameter stone columns in a triangular pattern at a maximum center to center spacing of 2.75 m was selected. This represented replacement ratio of about 14 percent. Fig. #4 shows the typical pattern adopted for the design.

Slope stability analyses were carried out for several critical sections to ensure design requirements were met under both static and seismic conditions. To ensure performance requirements under static loading conditions, a target minimum factor of safety of 1.3 under short-term conditions and 1.5 under long-term conditions was established. The short-term condition refers to a condition immediately after construction. Since there would be no time for consolidation of the treated clayey soils under the new shoreline fills to occur, no increase in the strength of the clayey soils was considered in the short-term analyses. The long-term condition refers to a condition well after the construction. The strength increase in the treated clayey soils due to consolidation under the new shoreline fills was taken into account in the stability analyses.

Fig. 4. Typical Stone Columns Pattern.

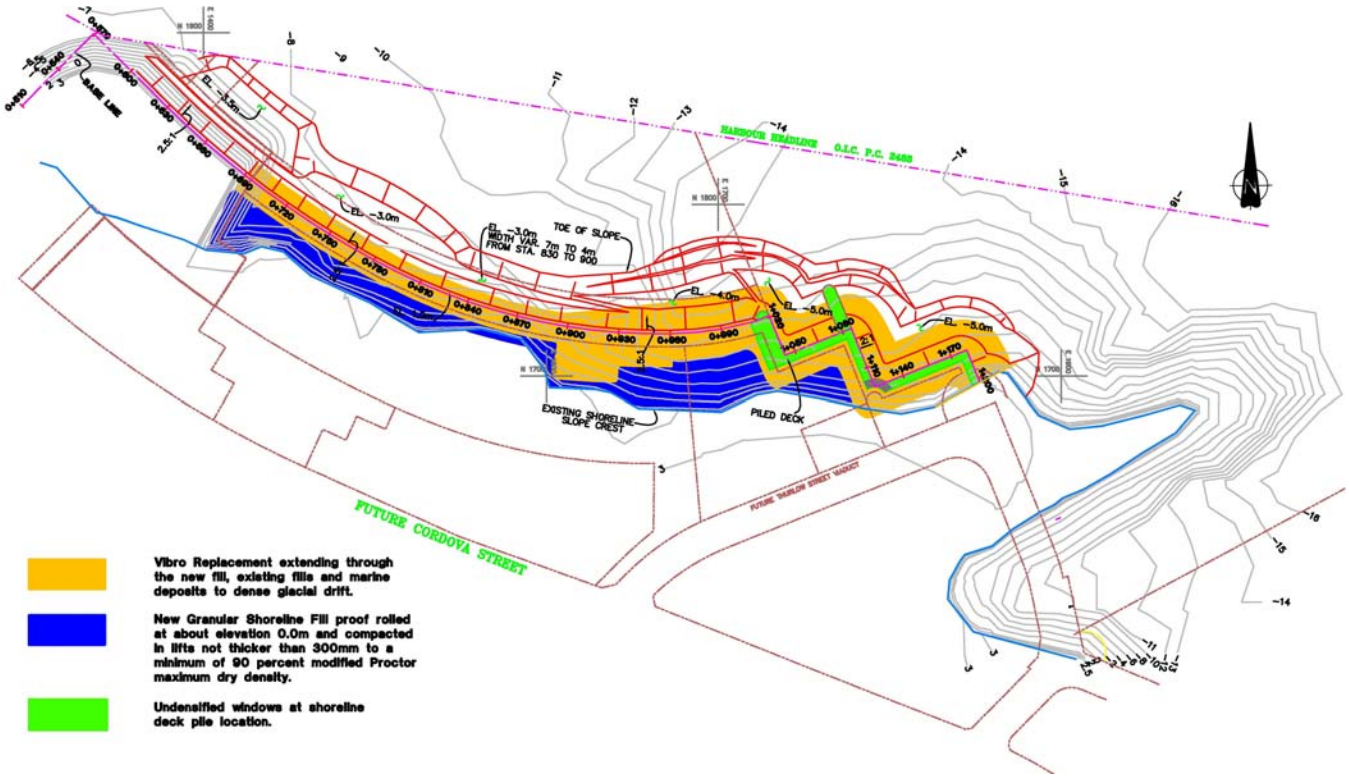


For determining the long-term effective shear strength of the treated clayey soils, a method proposed by Bergoda et. al (1991) which takes into account of the replacement ratio and the shear strength of the untreated clayey soils was used.

For the new shoreline slope configuration, slope stability analyses were carried out to optimize the ground improvement zone. The landward extent of the ground improvement zone was kept generally about 15 m south of the new shoreline crest to ensure adequate long term performance of the settlement sensitive walkway and cycle paths in this area.

The recommended ground improvement zone for the new shoreline in the Phase 2 development area is shown in Fig. #5.

Fig. 5. Recommended Ground Treatment for the Phase 2 Shoreline.



Construction Phase

Removal of rip-rap from the existing shoreline slope and concrete rubble and other obstructions was carried out within the proposed ground improvement zone prior to new shoreline fill placement. The land reclamation started from the western end where water depth was the shallowest. As a first step in the process of the land reclamation, slope stabilization rock toe berms were constructed to retain the new shoreline slope. Then barge loads of new shoreline fill consisting of 150 mm minus well graded sand and gravel from Texada Island were brought to site and bottom dumped onto the seabed. This operation was carried out in stages to ensure that there were no major slope stability issues with the filling. Generally a time gap of about 18 days was imposed initially between stages. Once the new shoreline filling has reached a suitable elevation above the high tide mark, land-based ground improvement work started. The western end of the Phase 2 shoreline was used as a test section to confirm that spacing of the stone columns and other operational variables are adequate to meet the design requirements.

A procedural specification was used for the stone column installation within marine deposits and the existing heterogeneous fills. For the new granular shoreline fill, performance based specification was used. Fig. #6 shows results for the first of many Becker Penetration Tests (BPT) carried out to verify that the new shoreline fills were being densified adequately. Since the test trial produced adequate response, the spacing of stone columns, infill material and other operator related variables such as time spent, amperages achieved, etc. were used as base measure for the subsequent production work.

The ground improvement work was completed in an approximately eight month period with over 1600 stone columns installed. Fig. #7 shows a plot of penetration time versus number of stone columns. This is one of the measures of how well the site was prepared including the adequacy of the removal of potential obstructions for the ground improvement work.

With over 80 percent of the stone columns requiring less than about 10 minutes to penetrate to the target depth, it can be concluded that the removal of obstructions in this challenging site was effective in facilitating ground improvement treatment work.

Following completion of the ground improvement work, the new shoreline area was preloaded to ensure that the walkway and cycle paths met the stringent post construction settlement requirements. During the preloading period, the shoreline slope was monitored for lateral movements using a slope inclinometer installed near the new shoreline crest. A summary of the inclinometer data is shown in Fig. #8. Only minimal lateral movements were observed during preload phase indicating that the shoreline treatment was adequate.

Fig. 6. Results of Verification Test – Test Section.

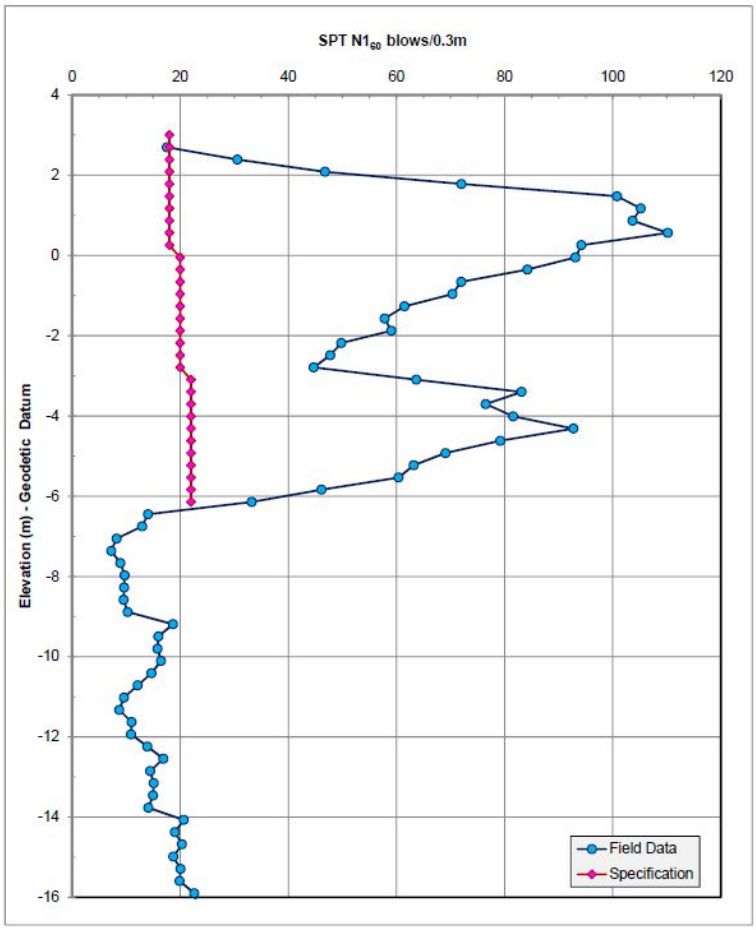
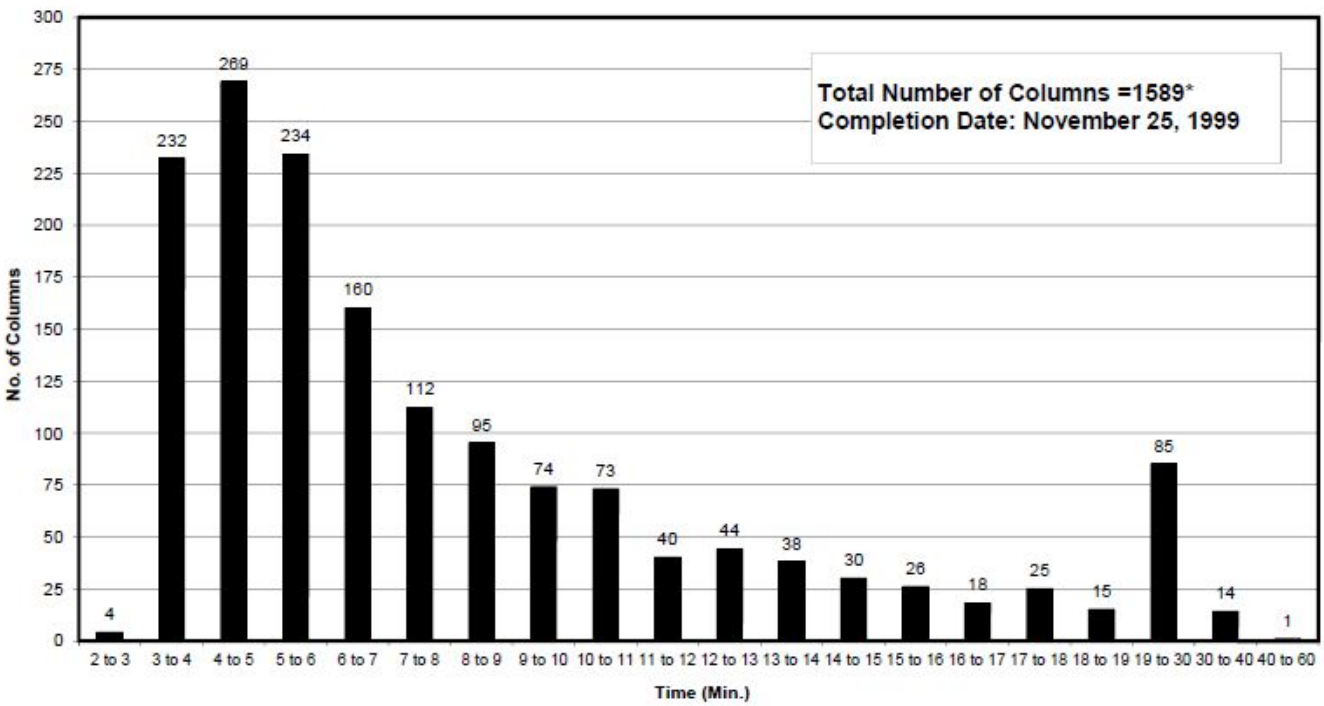
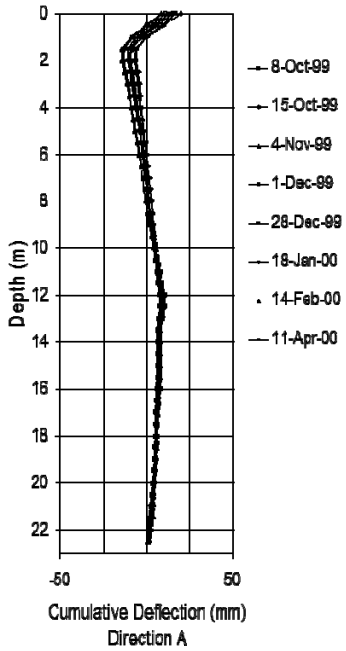


Fig. 7. Penetration Time for Production Stone Columns.



* Note: This total does not include the remedial columns that were constructed.

Fig. 8. Summary of Inclinometer Data.



Lessons Learned

Although for most part the land reclamation went without major problems initially, a minor slope failure of the overbuilt portion occurred in an area where the filling was of significant height. Fig. #9 shows the slope failure in the overbuilt portion during land reclamation. Further examination, including survey of the seabed offshore and as-built of the temporary slope of the overbuilt portion, indicated that the failure was not deep seated but rather a shallow one. Based on this incident, revisions were made to the temporary slope angle and the delay period between filling stages to ensure that there was no further slope instability during filling.

Fig. 9. Minor Slope Failure during Shoreline Fill Placement.

Due to a temporary supply situation, the new shoreline fill material from Texada Island was replaced by a similar granular material from Earl Creek. Routine Becker Penetration Test (BPT) carried out as part of the verification test program indicated that areas where Earl Creek material was placed did not meet the densification specification in certain depth ranges. As a result, additional stone columns were installed in these areas and additional BPTs carried out to ensure that the performance criteria were met within the new shoreline fill area. Whenever the source of the shoreline fill is changed, it is important to check ground improvement methodology to ensure that the required densification is being achieved and to avoid schedule delays and additional effort.

As continuity of stone columns and diameter of stone columns within the marine sediments are critical to the long-term performance of the new shoreline, this was communicated to the contractors. As a result of this consultation, a top feed method with heavier infill material was eventually proposed in lieu of the expensive bottom feed method for construction of the stone columns. Engaging the contractors in a timely manner and communicating critical elements of the project work is important in arriving at a cost effective solution that will benefit all.

Fig. #10 shows the completed Phase 2 shoreline area with the seawall and associated walkway, cycle path and the floating walkway.

Fig. 10. Completed Phase 2 Shoreline.

Acknowledgements

Several individuals and companies were involved in the Coal Harbour redevelopment project and their valuable effort and input are acknowledged. In particular, we appreciate the efforts of late Mr. Norm McCammon of Golder Associates Ltd., Marathon Development Inc. and Ausenco (Formerly known as Sandwell Engineering Inc.)

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