GEOQuébec 2015
Challenges from North to South
Des défis du Nord au Sud

# Design and construction of a three-storey concrete building on liquefiable soils in Victoria, BC.

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ABSTRACT
The foreshore in Victoria has undergone a number of changes over the years with large areas of fill having been placed to increase the land surface within much of the foreshore area. One such reclaimed land site was selected for the construction of a new three-storey concrete building. The field investigation consisted of both Becker Penetration tests (BPT) and Standard Penetration Tests (SPT) followed by bedrock probing with an air track percussion rotary drill. The soil conditions generally consist of a variable thickness of gravelly sand to sand and gravel fill material overlying a thick deposit of firm to soft marine clay. The design level earthquake for this structure is the 1:2,475 year event which results in a peak horizontal firm ground acceleration (PGA) of 0.6 g for this site. Estimated liquefaction induced lateral displacements within the fill were up to 6 m and vertical settlement estimates varied from about 0.1 m to 0.3 m. The final design of the building foundations consisted of rock-socketed caissons designed to resist the seismic deformations. Additional challenges included planning and executing a geotechnical investigation within a congested active work site and meeting the environmental requirements for working adjacent to the ocean within contaminated fill.

RÉSUMÉ
L'estran en Victoria a subi un nombre de changements au cours des années. Avec des grandes sections de remblai ayant été placés pour augmenter la surface de la terre dans une grande partie de la zone de l'estran. Un tel site a été choisi pour la construction d'un nouveau bâtiment de béton à trois étages. L'investigation consiste en des essais de pénétration Becker (BPT) et des essais de pénétration standard (SPT), suivie par un sondage du soubassement par forage rotatif. Les conditions du sol sur ce site consistent généralement en une couche de remblai avec des épaisseurs variables qui recouvre un dépôt épais d'argile marine. La structure est conçue pour résister à un tremblement de terre avec une période de retour de 1 :2475 années. Un événement qui se traduit par une accélération horizontale maximale dans la terre ferme (PGA) de 0.6g pour ce site. Les déplacements latéraux estimés induit par la liquéfaction dans le remblai vont jusqu'à 6 mètres, et le tassement vertical varie de 0.1 à 0.3 mètres. La conception finale de la fondation du bâtiment comporte des caissons encastrés en roche pour résister aux déformations sismiques. Les défis additionnels ont inclus la planification et l'exécution d'une étude géotechnique sur un site de travail actif et congestionné, aussi bien que satisfaire les exigences environnementales pour travailler à côté de l'océan dans un remblai contaminés.

## 1 INTRODUCTION

The foreshore in Victoria, BC has undergone a number of changes over the years with large areas of fill having been placed to increase the land surface within much of the foreshore. These traditionally less desirable areas are increasingly becoming the location for new development sites as more desirable building locations become scare. These new development sites provide unique and interesting challenge to engineers, especially in areas with high seismic hazards such as Victoria, BC. One such site is the federally owned and operated Public Works and Government Services Canada (PWGSC) Esquimalt Graving Dock (EGD). The EGD is located in the Township of Esquimalt, BC and is comprised of the north landing wharf, dry dock and south landing wharf (also called the South Jetty). An overall site plan of the EGD is shown in Figure 1.

As part of ongoing upgrades for Victoria Shipyards who leases property at the EGD, a new three-storey concrete building was proposed on a section of reclaimed land on the South Jetty. The new building was to have a plan area of approximately 29 m by 38 m. The main floor of the building will contain at least one 20 tonne crane, in addition to heavy machining equipment. The upper two floors will contain offices and lunch room space.

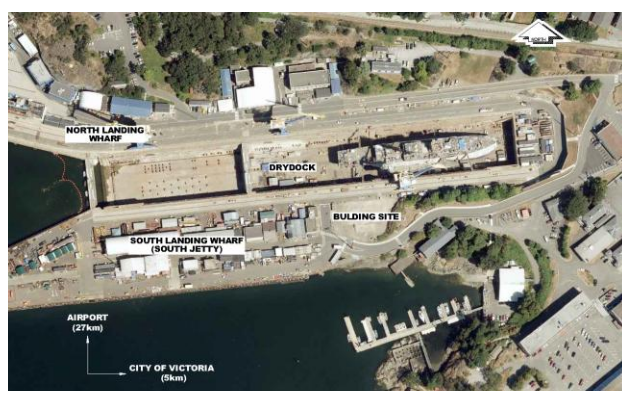


Figure 1. EGD Layout and Location

The site of the new building is located to the west of a large rock outcrop, and is bounded to the south by the South Jetty access road and to the north by the southern dry dock wall. The building site is located in an area that has been substantially filled in the past. The existing foreshore slope is approximately 3 m high and is located approximately 16 m to the south west of the building. The location of the new building is shown in Figure 2.

This paper presents the results of the geotechnical design and construction of the new Victoria Shipyards Operations building. In addition, environmental challenges encountered during construction will be discussed however the results of the environmental testing and monitoring during the project are not discussed herein.

## 2 BACKGROUND

The graving dock was originally constructed between 1921 and 1926. The eastern 110 m of the south wall of the graving dock was founded directly on bedrock and against rock for a large part of the wall height. The western 250 m of the south graving dock wall was also founded on bedrock, with a 2 m wide puddle clay seal placed between the wall and the rubble fill to the south.

The South Jetty was constructed in 1940 to 1941 and is supported on timber piles. This wharf was upgraded in 1985 to 1986. The main upgrade included a long concrete wharf structure parallel to and adjoining the north side of the south timber jetty. The concrete wharf was supported on steel pipe piles driven to practical refusal. A new tied-back, sheet pile wall was also constructed along the north side of the new concrete wharf. The rubble fill placed during development of the South Jetty area generally consists of a mixture of blasted rock fill, sand and gravel, and is mixed with some clay fill.

A number of previous investigation were conducted (by others) at various locations around the South Jetty. These reports were used as background information during the investigation and design of the new building.

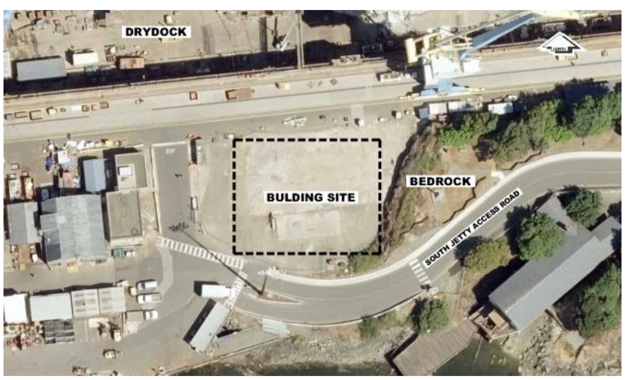


Figure 2. Building Site Location

## 3 SITE INVESTIGATIONS

The site chosen for the new Operations building was occupied by an existing building and maintenance facilities that were to remain intact until the beginning of the new building construction. As a result, a staged site investigation approach was used to characterize the site conditions and provide geotechnical input for design.

### 3.1 Becker Hammer

The initial geotechnical investigation was carried out using a truck-mounted model HAV 180 Becker Hammer drill. This drill rig was selected in order to penetrate the rubble fill anticipated to be found at the site and to obtain blow counts in the fill for use in a liquefaction assessment. This drilling method also allows sampling of the overburden soils through the casing which was an environmental requirement for this work.

The test holes consist of both closed-end penetration tests (BPT) and open-ended casing sampling holes. At selected depths within the open-ended casing the casing advancement was stopped, and an SPT test was conducted.

Four test holes were drilled at accessible locations around the existing building, and a fifth test hole was drilled south of the access road near the crest of the foreshore slope at the south end of the site. A BPT was conducted adjacent to each of the five test hole locations. The BPT was driven to practical refusal on probable bedrock at all 5 locations. Refusal occurred at depths from 7.5 m at the north-east corner of the existing building to 21.5 m at the south-west corner of the existing building. An opened-ended casing was conducted adjacent to the five BPTs to sample the rubble fill and the upper portion of the native soils. The open holes were advanced to depths from 7.6 m to 12 m below the ground surface. Disturbed samples were obtained from the soil cuttings during drilling. SPT's were conducted at selected depths in each borehole in an attempt to sample the soils. Due to the granular nature of the rubble fills, soil recovery from the SPT tests was poor.

At the completion of drilling, both the open-ended test holes and the BPT holes were backfilled with soil cuttings and bentonite seals at selected depth intervals in accordance with the BC Groundwater Regulations. The location of the BPT holes are shown in Figure 3.


Figure 3. Test Hole Locations

### 3.2 Bedrock Probing

Following removal of the existing building and maintenance facilities at the building site and prior to construction, an air track rotary percussion drill was used to probe for bedrock at 24 of the 25 proposed caisson locations (the caisson at the southeast corner is located on bedrock). This method of drilling does not allow for sampling of the soils or bedrock; the comments of the driller and the drill action alone are used to interpret when bedrock is encountered. The holes were advanced at least 1.5 m into inferred rock to confirm its presence. The accuracy of bedrock depths using this method of drilling is typically about + 0.2 m, however the results are occasionally unreliable if very steeply dipping bedrock or large boulders are encountered. The location of the probe holes and depth to bedrock are shown in Figure 6.

Bedrock was not penetrated during the initial investigation; however the BPT was driven to practical refusal on probable bedrock. Inferred bedrock was encountered at depths ranging from 3.4 m to 21.3 m below the ground surface during the bedrock probing investigation. The bedrock at this site has been mapped by the Geological Survey of Canada as Wark Gneiss. The bedrock surface is highly irregular, and is typically hard and fractured. Unconfined compression testing, conducted by others, indicates the compressive strength varies from 50 to 75 MPa.

## 4 SOIL CONDITIONS

The soil conditions at this site generally consist of a variable thickness of fill material overlying a thick deposit of marine clay. The fill materials on site are generally loose to compact and are comprised primarily of gravelly sand to sand and gravel with some rock fill. The fill is generally "clean" with less than 5% passing the 0.075 mm sieve. The fill ranges in thickness from about 7 m to about 11.3 m. There are also zones of silt and clay within the fill, however the exact location and thickness could not be determined during the investigation. Based on previous work conducted on site, thicker zones of blasted rock fill, boulders or other debris are also anticipated at this site. Several of the earlier test pits and boreholes also encountered thicker zones of fine-grained material (silt or clay) within the fill materials. On average the equivalent SPT $N_{60}$ value at each of the test hole locations was found to be less than 10 blows/0.3 m. In some cases, the SPT $N_{60}$ values from the open-ended casing hole were found to be much higher than the equivalent SPT $N_{60}$ values determined from the BPT blow counts. This generally occurred in layers of the fill that contained more gravel than sand. The discrepancy between the two methods was attributed to the poor reliability of conducting SPT's within gravel soils and its associated overestimation of the material density. Therefore, the SPT results were deemed unreliable.

The fill is underlain by marine clay. The upper portion of the marine clay at the test hole locations is a firm to soft silty clay, or silty to clayey sand. The marine clay underlying the South Jetty has been well characterized during previous investigations on site. Based on the existing information, the clay was divided into two layers. The upper zone (approximately 5 m thick) in which the clay has a relatively constant undrained shear strength with depth and a lower zone in which the undrained shear strength increases with depth to the base of the unit. These clay layers were then separated into two zones beneath the project site; the clay beneath the fill (which will have undergone some consolidation and strength gain due to the overlying fill), and the clay outside of the fill prism.

A thin layer of inferred glacial till (based on the BPT blow counts) underlies the marine clay deposit at some of the test hole locations. The inferred till is about 0.1 m thick to about 1.1 m thick. It is possible that BPT refusal occurred on boulder or cobbles within the till and that the actual till thickness could be more than indicated.

## 5 SEISMIC CONSIDERATIONS

The granular fills encountered around the South Jetty area have been identified in previous studies as being liquefiable under a 1:475 year seismic event. The design level earthquake in the 2010 National Building Code of Canada (NBC) is the 1:2,475 year event. Based on the uncorrected BPT blow counts, the fill materials at this site were expected to liquefy. The marine clay deposit is generally considered non-liquefiable but is expected to soften after a significant earthquake. A liquefaction assessment was carried out to confirm that the soils are liquefiable. A seismic assessment was also completed to assess the magnitude of anticipated vertical and lateral displacement associated with a design seismic event.

### 5.1 Seismic slope stability

In order to estimate the potential magnitude of soil displacement within the marine clay, a pseudo-static seismic stability analysis of the existing foreshore slope was carried out using soil parameters shown in Table 1.

Table 1. Soil Parameters

| Soil | Unit Weight ($kN/m^3$) | Friction Angle (degrees) | Undrained Shear Strength (kPa) | Estimated Vs (m/s) |
|---|---|---|---|---|
| Fill | 20 | 34 | - | 180 |
| U. Marine Clay (below fill) | 18 | - | 45 | 160 |
| L. Marine Clay (below fill) | 18 | - | 45 to 80 | 160 |
| U. Marine Clay | 18 | - | 35 | 160 |
| L. Marine Clay | 18 | - | 35 to 50 | 160 |
| Glacial Till | 20 | 38 | - | 1000 |

The lateral displacement of the slope caused by the design earthquake (PGA=0.6g, M=6.8) was estimated using the methodology of Bray and Travasarou (2007). The pseudo-static stability analyses were carried out using the commercial software Slope/W. Estimated lateral displacements within the marine clay range from 200 mm to 500 mm.

An example of one of the stratigraphic models used for the analyses is shown in Figure 4.

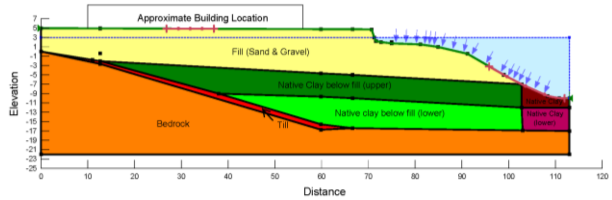


Figure 4. Stratigraphic Cross Section

### 5.2 Liquefaction Assessment

The liquefaction assessment was carried out using the simplified Seed and Idriss (1971) procedure. This procedure requires the determination of SPT equivalent $N_{60}$ values for the granular soils. The BPT blow counts in the granular soil were used to estimate the $N_{60}$ values using the procedure of Harder and Seed (1986).

The results of the liquefaction assessment confirmed that the granular fill materials will liquefy under the 1 in 2,475 year design level earthquake. It should be noted that liquefaction of these soils will be triggered with a seismic event having a much more frequent return period (likely the 1 in 100 year event).

An example of the liquefaction assessment results from one of the test hole locations is shown in Figure 5.

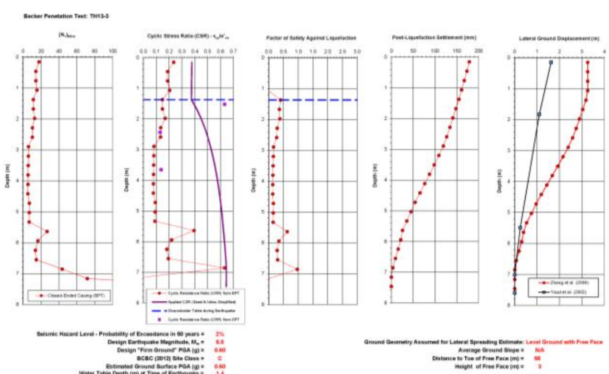

Figure 5. Example of liquefaction assessment results.

#### 5.2.1 Lateral Spreading

The amount of lateral spreading associated with liquefaction of the granular fills was estimated using the empirical equations of Youd et al. (2002) and Zhang et al. (2004).

Although the Youd et al. method is widely recognized within the industry there are a number of disadvantages associated with it. There is no distinction made within the method between the lateral displacements associated with “compact” soils compared to “loose” soils with a lower $N_{1(60)}$ value. In addition, zero displacement is predicted in soils where the $N_{1(60)}$ value is greater than 15. One advantage of this method is that increased liquefaction resistance of gravelly soils is accounted for empirically through the $D_{50}$ variable. However, due to the nature of the existing fill materials and the poor sample recovery encountered during the investigation, the estimation of $D_{50}$ for this particular site contains large uncertainty. Youd et al. (2002) demonstrate that their equations generally predicted displacements within ½ to 2 times the measured displacements from the case history database. However, some measured displacements were as much as 4 times the predictions.

The Zhang et al. method is a semi-empirical prediction method that uses the factor of safety against liquefaction ($FS_{liq}$) to estimate the maximum cyclic shear strain ($\gamma_{max}$) that may develop during undrained cyclic loading in the absence of a static shear bias. One of the disadvantages of the Zhang et al. method is that the laboratory testing results the equations are based on are related to relative density ($D_r$) which cannot be accurately determined from penetration test data of any type. As a result, any errors or conservatism in calculation of $D_r$ or $FS_{liq}$ will result in error in calculating $\gamma_{max}$. Estimation of large $\gamma_{max}$ values with low $D_r$ and/or low $FS_{liq}$ can lead to large estimates of lateral displacement. In addition, the $\gamma_{max}$ relations are based on behaviour of sands and may overestimate the lateral displacement potential of gravelly soils, depending on how the $D_r$ and $FS_{liq}$ of a gravelly soil are estimated. Reported lateral spreading estimates using the Zhang et al method were generally within ½ to 2 times measured displacement magnitudes reported in their case history database, with some predicted displacements being as much as 3.5 times higher than measured displacements.

Both the methods used indicate that the lateral spreading at this site will be large and increases as the proximity to the existing fill slope increases, as expected. In practice, a factor of safety of 2 is commonly applied to the predicted Youd et al. displacements to account for the uncertainty in the $D_{50}$ measurements and the cut off of no liquefaction occurring in materials with $N_{60}$ values greater than 15 blows/0.3m. The Zhang et al method is typically known to provide large displacements in gravelly soils as discussed above, which we have across much of the building site. To account for this overestimation the estimated displacements were reduced by a factor of 2

The results of the analyses indicated that using the Youd et al method and applying a factor of two, the estimated lateral displacements within the fill range from 2 to 5 m. Using the Zhang et al method and reducing the displacements by a factor of two, the estimated lateral displacements within the fill range from 1.5 to 6 m.

#### 5.2.2 Vertical Displacement

Liquefaction induced vertical settlements were estimated using the empirical equations of Tokimatsu and Seed (1987). The estimated vertical settlement at the building site varies from about 0.1 m to 0.3 m.

#### 5.2.3 Flow Slides

The potential for a flow slide following the design level earthquake was also assessed using the software program Slope/W. The analysis was conducted using estimated post-liquefaction residual strength for the granular soil. The marine clay deposit was assumed to have an approximately 20% strength loss following the seismic event. The analyses confirm that a flow slide will likely occur in the granular fills and result in large lateral displacement. A failure through the underlying clay is not considered likely.

## 6 FOUNDATION DESIGN

Due to a number of site constraints, site improvements such as soil densification and soil replacement were not a viable option for this project. Since the predicted seismic displacements were large, a pile supported structure was recommended to support the building on bedrock. To resist the lateral displacements a relatively stiff pile system was required, therefore rock-socketed caissons were used to support the building. Underground utilities and roadways were not designed to resist these deformations. Failure of these systems is therefore likely to occur following a significant earthquake

During the design earthquake, the granular soils at the site will liquefy and result in large ground displacements. The underlying marine clay deposit will also soften and lose strength. At the estimated displacements, the soil will tend to flow around a stiff rigid pile. However just before this occurs, the maximum load applied to the pile is assumed to be equal to the maximum soil resistance. Therefore, the piles were designed to resist the ultimate soil load along the length of the pile within the fill materials and marine clay deposit and assuming no soil resistance in front of the piles.

The building was founded on 24, 914 mm diameter caissons with 865 mm diameter rock sockets as shown in Figure 6. In addition, the 10 piles at the west end of the building contained a 610 mm inner casing. The location for the south east most caisson was sitting directly on bedrock and therefore only a rock-socket and reinforcement were required at this location. The sockets had a minimum length of 2.8 m. Details of the piles including reinforcement are provided in Table 2. The lower floor stab in the building was designed as a structurally suspended slab.

Table 2. Caisson and Rock Socket Details

| Caisson Type | Size (mm) | Reinforcement | Min. Socket Length (m) |
|---|---|---|---|
| CA1 | 914 DIA.x12.7 STEEL PIPE 865 DIA. SOCKET | 24 - 25M VERT + 10M TIES @ 450 | 2.8 |
| CA2 | 914 DIA. x19 STEEL PIPE + 610 DIA. x 16 STEEL PIPE 865 DIA. SOCKET | 18 - 30M VERT + 10M TIES @ 450 | 2.8 |

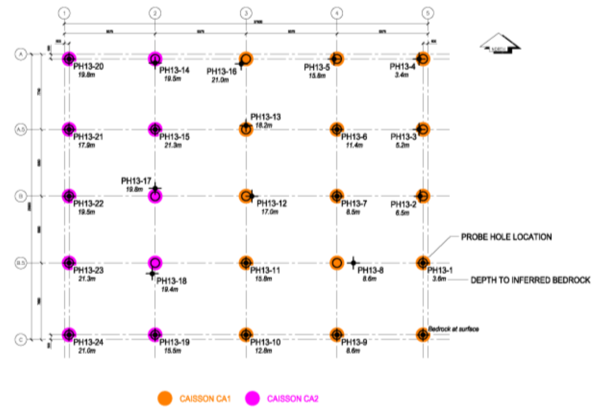

Figure 6. Caisson Layout

## 7 CONSTRUCTION

A variety of geotechnical services were provided during construction of the building including:

- Inspection and compaction testing of trench backfill materials;
- Inspection of rock-sockets;
- Concrete testing for caisson in-fill;
- Inspections of subgrade beneath slab;
- Concrete testing for grade beams and concrete slab;
- Evaluation of the rock cut slope and recommendations for stabilization; and
- Testing of rock bolts installed in the rock slope to support the mesh and stabilize the slope.

### 7.1 Rock-socketed Caissons

Installation of the CA1 caissons was carried out by rotating the 914 mm casing with a Barber DR40 Drill through the overburden and seating the casing in the bedrock. Once seated, the rock sockets were then advanced a minimum of 2.8 m below the casing. For installation of the CA2 caissons, a temporary 1200 mm diameter caisson was oscillated through the upper mixed fill zone into the marine clay deposit to provide an outer casing through the fill for the 914 mm pile. The casing was then cleaned out with a Mait 180 Auger Drill, prior to installation of the 914 mm casing. The annular space created between the outside of the 914 mm pile and the 1200 mm casing was infilled with pea gravel to provide lateral support. Once the 914 mm casing was seated in rock, the socket was advanced a minimum of 2.8 m below the casing.

Following drilling, a video camera was lowered down the center of each pile to visually inspect the pile tip and rock socket. Pile tip depths and rock socket lengths were measured using the video camera and sounder. The pile tips were inspected to check that they are fully embedded within bedrock. Rock conditions along the socket lengths were also inspected. Final cut off steel casing lengths ranged from 2.6 m to 26.1 m and socket lengths ranged from 2.9 to 3.1 m below the casing.

### 7.2 Rock Cuts & Stabilization

As shown in Figure 2, bedrock is exposed along the eastern side of the new building and therefore rock removal was required to allow for construction of the building. The blasted rock slope had a general north/south orientation, was approximately 30 m long, 3 m to 4.5 m high and steeply inclined ranging from overhanging (at the north end), to near vertical within the central and southern portions. The rock contained numerous cross cutting joints with an average spacing of 30 mm to 150 mm with both favourable and unfavourable orientations. Several larger, more pervasive continuous joint sets and shear zones were also observed, including an unfavourable joint within the middle of the rock face oriented at 170° and dipping at 50° towards the west. This joint has resulted in the formation of several potentially unstable blocks including an open-gapped piece within the lower half of the rock face that is approximately 300 mm by 500 mm by 500 mm in size. As a result, it was recommended that the entire blasted rock face be covered by an anchored rock mesh to retain future small-scale rock fall.

The mesh consists of 11 gage, galvanized, triple twist, gabion-type anchored slope mesh and was held tight to the slope with fully grouted #7 Dywidag threadbar dowels set a minimum of 1.5 m into the rock face. The dowels were set into 75 mm diameter drilled holes with Basalite Microcil anchor grout. All the dowels were tested using a 10 minute creep test with readings at 1 minute intervals, under a load of 45 kN.

A pre-existing overhanging rock face adjacent to the recent bedrock blast area at the northeast end of the building site also required stabilization with rock bolts. The rock overhang, consisted of an approximately 3.5 to 4.5 m high, north-facing slope face which overhangs the base of the slope by about 1.8 m horizontal distance (i.e. the rock face dips towards the south at approximately 60°). A series of joints observed on top of the rock face suggested the presence of one or more joint sets paralleling the exposed rock face up to 1.5 m back from the exposed face. The rock face is bisected by an irregular, generally horizontal cross-cutting, open-gapped joint located ~1 m to ~1.8 m below the top. The orientation of the mid-slope fracture plane is irregular but generally sloping upwards into the slope. A large joint demarking a similar fracture plane is located at the base of the slope and oriented with a strike of 325° and a dip of ~35° towards the north. Minor rock scaling and the installation of four rock bolts were recommended to stabilize the slope. The anchors consisted of #8 (25 mm) galvanized Dywidag threadbar anchors. The anchors were a minimum of 4 m long with 1 m of bond length. The anchors were installed inside 75 mm diameter drilled holes inclined at 10° to 15° below the horizontal into the rock face. Initially the anchors were unable to be tested due to the angle between the rock surface and the anchors and due to high point loads of the system causing the surrounding rock to fail. Pads using shotcrete and steel mesh around each anchor head were subsequently constructed. The pads were approximately 600 mm x 600 mm. The anchors were tested to a load of about 180 kN and held for 10 minutes to check for load loss and creep. All four anchors experienced less than 10% load loss over the course of the test. The anchors were locked off following testing at the design load of 140 kN.

### 7.3 Additional Challenges

Construction of the building took place while day to day operations at the EGD continued including operation of the adjacent dry dock and associated cranes. As a result, road access to the south of the site had to be maintained at all times and the crane rails to the north of the site had to remain unobstructed. This attributed to a congested work site with little room outside of the building footprint available for storage of materials, equipment, etc.

Although the details of the environmental investigation, testing and management plan are not discussed herein, the environmental requirements during construction created challenges that are worth discussing. Due to the proximity of the site to the ocean, a comprehensive soil management and environmental monitoring plan was put into place during construction. The fill underlying most of the site was also contaminated.

Management of the caisson drilling fluids and contaminated soil cuttings required the use of a filtration bladder and sediment ponds to remove suspended soils and facilitate the collection of water samples for analytical testing. Water samples were collect from the pond and analyzed to confirm water quality requirement prior to discharge. All excavated soils were stockpiled and sampled prior to disposal off-site which was especially challenging due to the space limitations on site. Soils that were found to be uncontaminated, were used as backfill on site where the geotechnical requirements of the backfill were met.

Sediment control measures, such as silt fences or sand bags were placed in areas where there is potential surface runoff to marine receptors. Filter material was placed over any drains located near the construction area to ensure that no deleterious materials entered the ocean or storm water system. All vehicles and delivery trucks vehicles visiting the site were checked for possible fluid leak and polyethylene sheeting and/or plywood was placed underneath the vehicles visiting to catch oil drips and contain possible spills. Finally, routine environmental inspections were conducted to monitor compliance with the environmental management plan.

## 8 CONCLUSIONS

The site for the new Operations building for Victoria Shipyard came with a number of geotechnical and environmental challenges. Seismic issues governed the design with estimated liquefaction induced lateral displacements of up to 6 m and estimated vertical settlements that varied from about 0.1 m to 0.3 m. The potential for a flow side following the design earthquake was also likely to occur and result in lateral displacements.

The building was founded on rock-socketed caissons that were designed to resist lateral loading from large soil displacements under the design level earthquake. Underground utilities and roadways adjacent to the structure were not designed to resist seismic deformations. A combination of anchored rock mesh and rock bolts were also used to stabilize the adjacent rock slope following blasting.

Additional challenges during this project included working within a congested active work site and meeting the environmental requirements for working adjacent to the ocean.

Design and construction of the building took place over a 2 year period.

## ACKNOWLEDGEMENTS

The authors would like to acknowledge David Latoski of PWSGC and Peter Lutzmann of Victoria Shipyards for approving this work to be published. We would like to thank Stantec Consulting for involving Thurber in this project. Thank you to various sub-contractors for their assistance during the investigation and construction phases of this project. Finally thank you to all the Thurber personnel who assisted in the field investigations, design and construction phases of this project.

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